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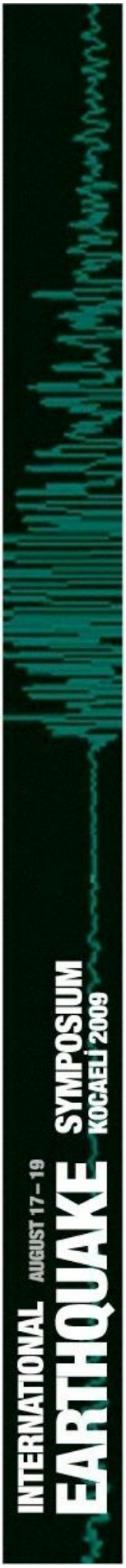

EARTH SCIENCES

YER BİLİMLERİ

INTERNATIONAL EARTHQUAKE SYMPOSIUM AUGUST 17 – 19 KOCAELİ 2009

DEDUCING THE GEOMETRY OF STRIKE-SLIP FAULTING BY RESISTIVITY METHOD

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Key Words: Resistivity, Wenner, Fault, Iznik.

ABSTRACT Tectonic based formation of river basins resulted from normal or strike-slip faults, and formation of river basins can only be understood by revealing the mechanisms or geometry of these faults. Consequently, the structure of North Anatolian Fault Zone in Geyve and Iznik river basins is tried to be revealed by resistivity method in order to explain the formation of the mentioned river basins. To understand Geyve and İznik river basins, data were collected perpendicular to the fault line. Wenner configurations were applied. Then we performed an inversion algorithm developed based on the least square method for evaluating the data. In this way we showed that how the fault continues the downwards. Three directions were selected on the Geyve and İznik faults which are considered as the western central branch of the North Anatolian Fault Zone. Separate measurements perpendicular to the fault were performed for each direction. The measurements were collectively evaluated and the geometrical status of the fault in that region was presented. According to the results obtained, the surface and underground traces of the fault were jointly mapped. The underground traces were found to be tending towards the north when compared to the surface traces. The level of this tendency has provided us valuable tectonic information on the basins' formation and their evaluation to date.

INTRODUCTION

This study area is in the south-eastern part of the Marmara region. It is cut by many active faults forming some distinct tectonic features; such as Lake İznik.

Bifurcation of the North Anatolian Fault (NAF) master strand east of the Almacık block, the southern strand of the NAF zone follows the Mudurnu valley and further west bifurcates again around Pamukova village (Şengor 1979; Barka & Kadinsky-Cade 1988; Kocyiğit 1988; Barka 1992; Bozkurt 2001; Yaltırak 2002). One of these faults extends westward from Pamukova into Lake İznik. (Kocyiğit 1988), while another fault extends from Mekece to the northern slopes of Uludağ Mountain along the Yenişehir plain (Barka & Kadinsky-Cade 1988; Yaltırak 2002). Lake İznik is situated on the fault segments of the northern block of the North Anatolian Fault Middle Strand (NAFMS), which extend from Pamukova to the Gulf of Gemlik, along the southern coast of Lake İznik and through Gemlik town. Within this framework, the lake itself and almost all of its coastal plains are located on the NAFMS, which can be traced along a 97-km-long corridor from the town of Geyve to Gemlik. This region is made up of a series of structurally active basins, namely the Geyve-Pamukova Basin, Lake İznik and the Gulf of Gemlik (Bargu 1982; Kocyiğit, 1998; Barka & Kuşcu 1996). In this region, Tsukuda et al. (1988) defined two separate fault segments: the Geyve segment between the towns of Geyve and İznik, and the İznik segment (i.e., the Soloz Fault). Both structures exhibit morphological features characteristic of strike-slip faults.

METHOD

The used apparent resistivity equations have been proposed by Telford (1976). In this study Marquardt, 1963 technique was used and this technique is given below;

$$\Delta p = \left(A^T A + \lambda I\right)^{-1} A^T \Delta g \qquad (1)$$

Here,

Δp Parameter correcting vector, Δg Observed and calculated anomaly distinction A jacobian matrix, λ Marquardt factor, I Identity matrix.

Marquardt factor initially were chosen 2. In case of convergence was split into two and In case of divergence was multiplied by two.

MODELING STUDIES

Fault Modeling study and underground structure have been shown Figure1.

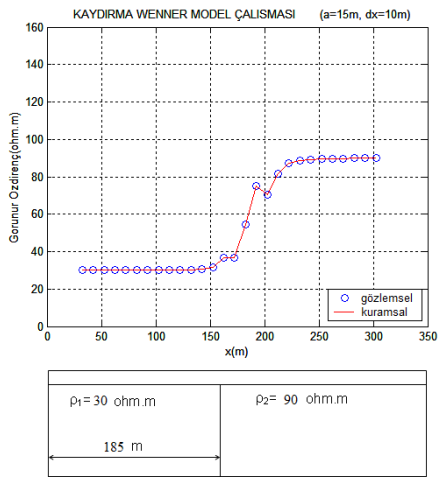


Fig. 1. Fault model.

GEOLOGY OF THE STUDY AREA

The geology of the region surrounding Lake İznik includes Late Palaeozoic, Mesozoic and Palaeogene successions forming the pre-Miocene basement. The Miocene sequence unconformably overlies the basement rocks and is covered by Pliocene–Quaternary units Pliocene–Quaternary units consist of sand and gravel layers with laterally variable thicknesses. Pliocene–Quaternary deposits around Lake İznik include alluvial plains, fans, marsh areas, lake terraces and beach facies, especially in the eastern and western sectors (Öztürk at al.2008).

FIELD INVESTIGATIONS

In this field study three direction were selected. Direction1 composed of three profiling measurement. Solutions and approximate cross-section respectively have been shown Figure2 and Figure 3.

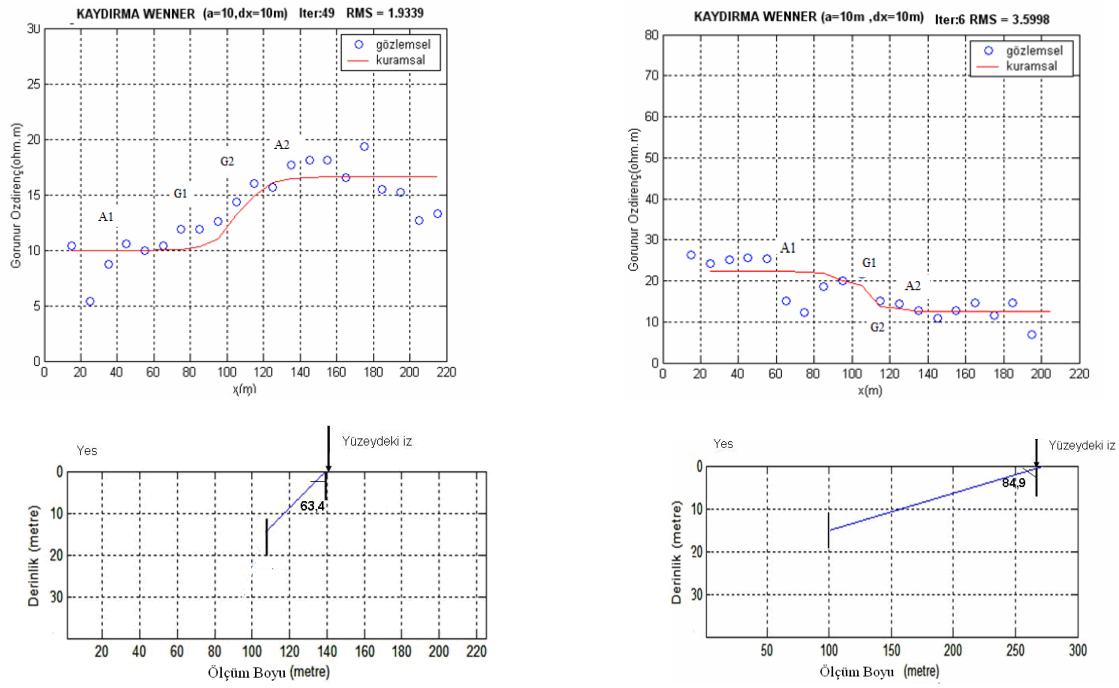


Fig. 2. Solution of G1 ve G2 profiling and geometry.

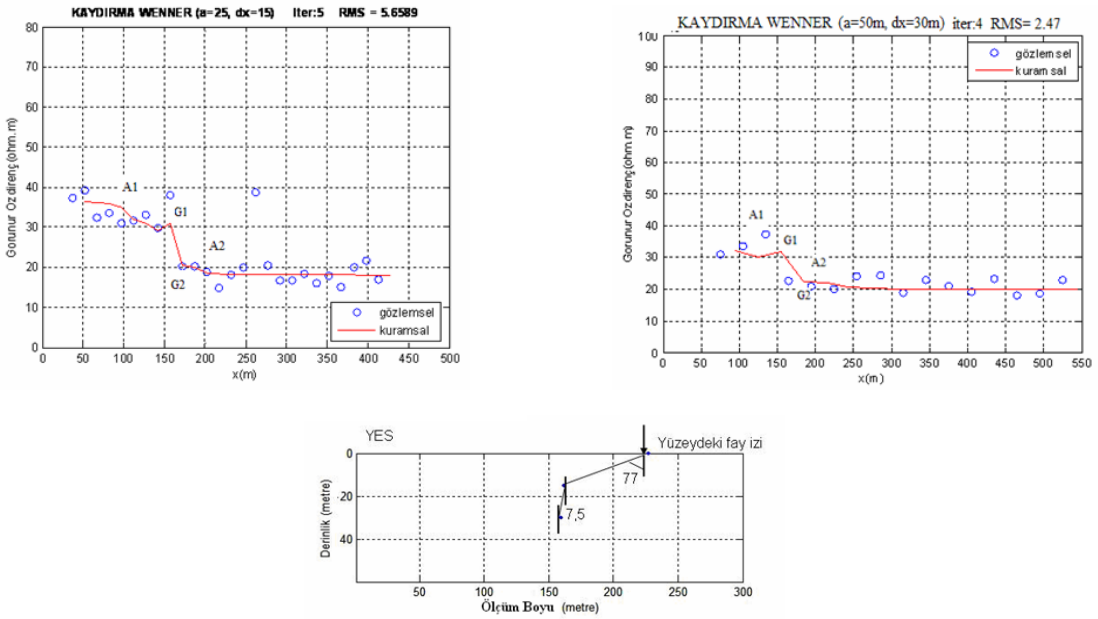


Fig. 3. Solution of G3 profiling and geometry.

According to the results obtained, the surface and underground traces of the fault map has been shown Figure 4.

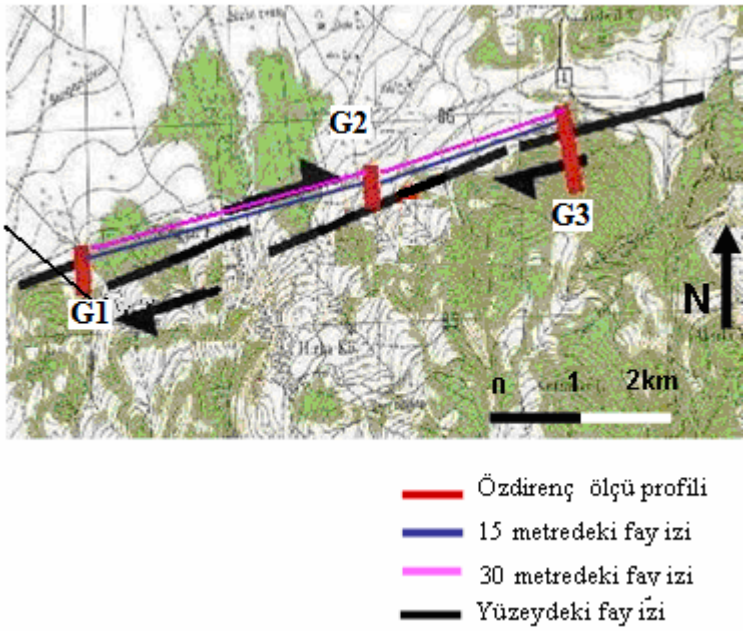
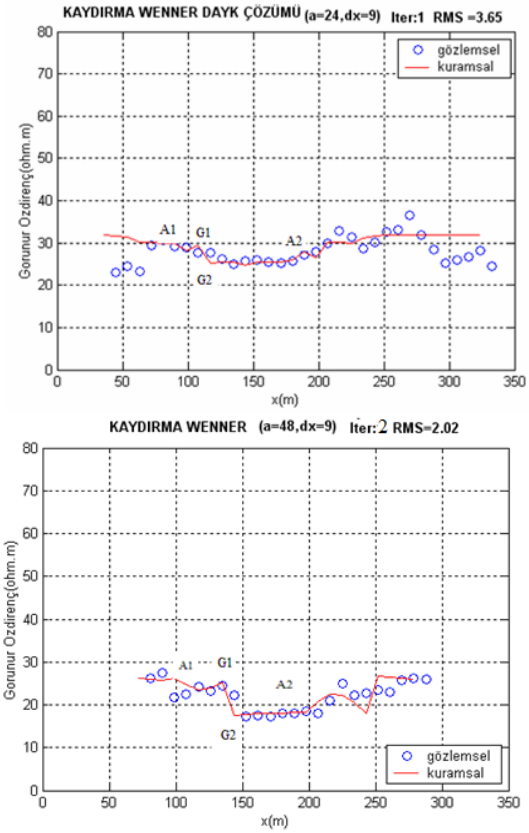


Fig. 4. Obtained from solutions fault map.

Direction 2 composed of two profiling measurement. Solutions and approximate cross-section respectively have been shown Figure 5 and Figure 6.



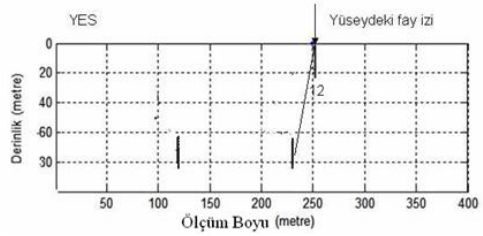


Fig. 5. Solution of IZN1 profiling and geometry.

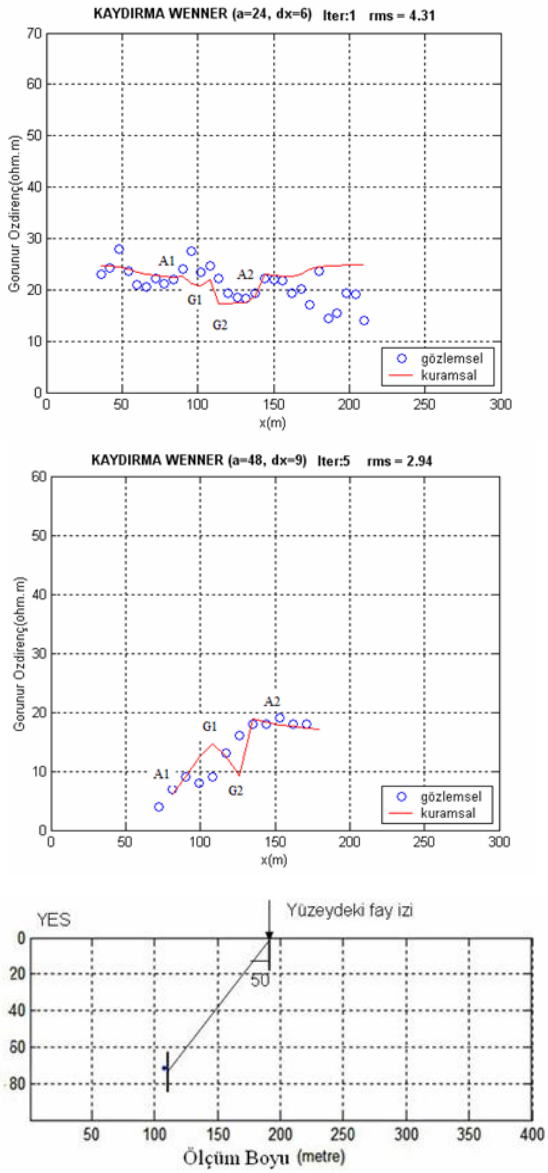


Fig. 6. Solution of IZN2 profiling and geometry.

According to the results obtained, the surface and underground traces of the fault map has been shown Figure 7.

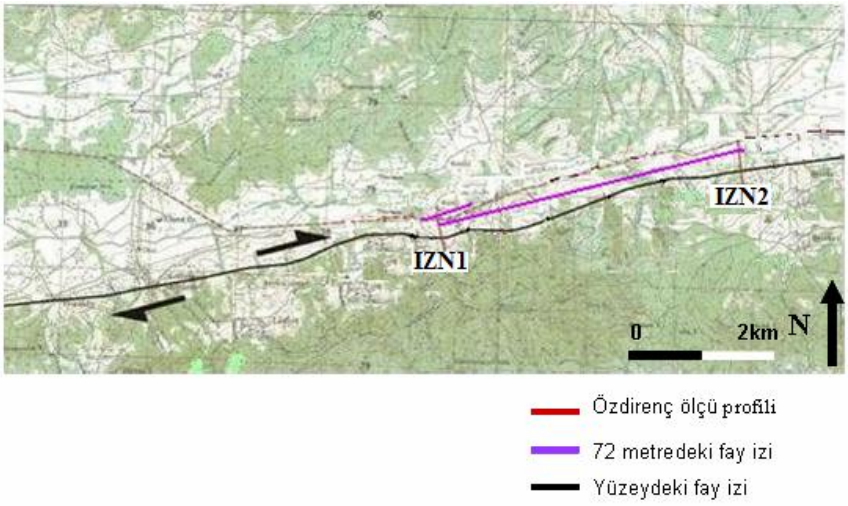


Fig. 7. Obtained from solutions fault map.

Two profile were selected on this direction. Solution of IZN3 profile and approximate cross-section have been shown Figure 8. Solution of IZN4 profile has been shown Figure9.

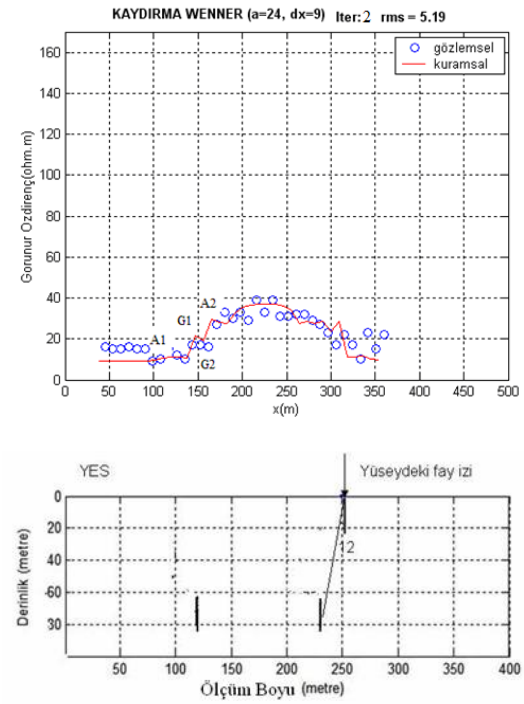


Fig. 8. Solution of IZN3 profiling and geometry.

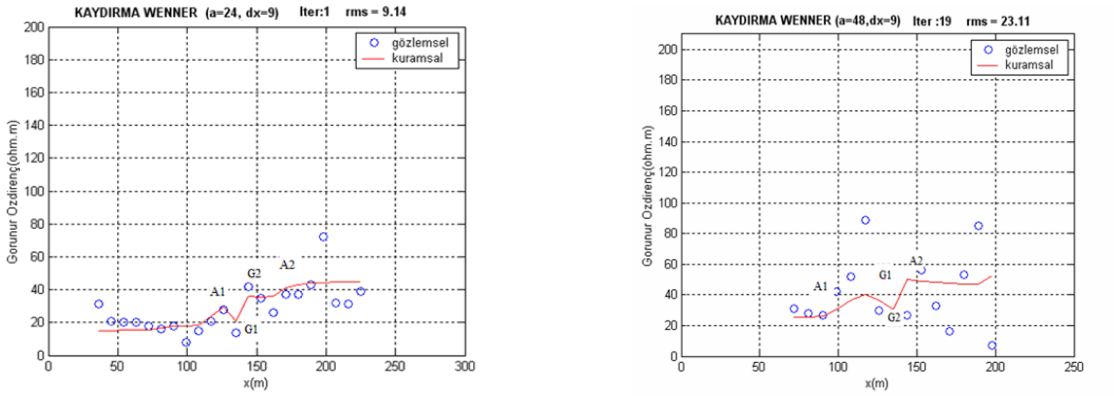


Fig. 9. Solution of IZN4 profiling.

According to the results obtained, the surface and underground traces of the fault map has been shown Figure 10.

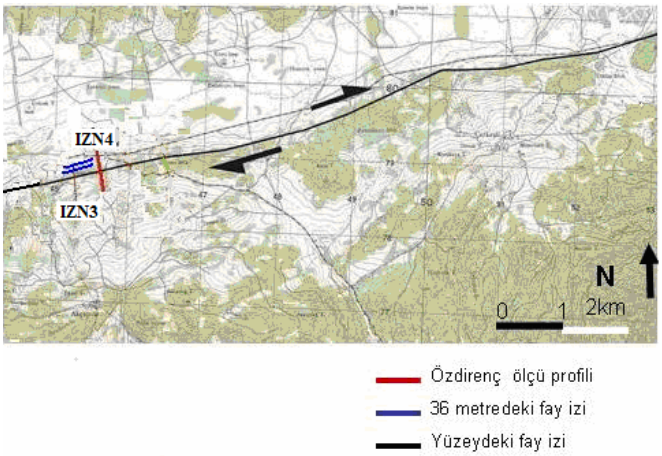


Fig. 10. Obtained from solutions fault map.

Results of inversion has been shown Table1.

ρ_1, ρ_2 : Apparent resistivity

a : The distance between electrodes

d : Valid for distance definitions frompoint of measurement to fault

dg : The distance between the two faults

dx : The amount of shift

Table 1. Results of inversion.

	Results of inversion											
	Initial Parameter				Calculated Parameter							
	ρ_1	ρ_2	d	Dg	ρ_1	ρ_2	d	Dg	a	dx	İter	rms
G1	10	18	96	----	9	16	100	----	10	10	49	1,9
G2	25	12	118	----	22	12	100	----	10	10	6	3,5
G3 Gidiş	36	15	131	----	37	18	160	----	25	15	5	5,6
G3 Dönüş	36	18	131	----	32	20	162	----	50	30	4	2,4
IZN1 Gidiş	32	24	100	90	28	23	99	113	24	9	2	3,6
IZN1Dönüş	30	16	120	100	29	15	119	116	48	9	2	2,0
IZN2 Gidiş	25	16	100	30	20	16	100	46	24	9	2	4,3

IZN2Dönüş	5	20	110	----	5	25	108	----	48	9	5	2,9
IZN3	9	38	150	150	14	36	148	187	24	9	2	5,1
IZN4 Gidiş	12	45	120	----	15	46	125	----	24	9	1	9,1
IZN4Dönüş	30	60	110	----	22	59	112	----	48	9	19	23,1

CONCLUSION

The present inversion scheme has been tested on synthetic and field data. The results of the tests show that the procedure works well and the convergence process is stable even in the fault model. Three directions were selected on the Geyve and İznik faults which are considered as the western central branch of the North Anatolian Fault Zone. The profile data was interpreted by the application of Damped Least-Squares (Marquardt) inversion techniques. According to resuts obtained from inversion the underground traces were found to be tending towards the north when compared to the surface traces. The results obtained were supported by the magnetic studies conducted by the Bogazici University Kandilli Observatory and Earthquake Research Instutute in the same region.

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ÜÇ BOYUTLU KARMAŞIK YER MODELLERİNİN YAPAY SİSMOGRAMLARLA TANIMLANMASI

DETERMINATION OF RESPONSE OF 3D COMPLEX VELOCITY MODELS BY USING SYNTHETIC SEISMOGRAMS

Arslan[1]

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Anahtar Kelimeler: Sismik, Akustik Dalga Yayılımı, Sonlu Farklar Yöntemi, Yapay Sismogram, 3B Düz Çözüm,

ÖZ Bu çalışmada, üç boyutlu (3B) yer yapısı ve nokta kaynak için akustik dalga denklemleri zaman ortamında çözümlenmiştir. Akustik dalga denkleminin sayısal benzetimi, sonlu farklar(SF) yöntemi kullanılarak yapılmıştır. Sismik kaynak olarak Ricker dalgacığı kullanılmıştır. Farklı hız modelleri için akustik dalga yayılımı hesaplanmış ve bu hız modellerine ait yapay sismogramlar üretilmiştir.

Sonlu farklar yöntemi ile dalga yayılımının modellemesi yapılırken, karşılaşılan en önemli sorunlardan biri, hesaplama alanının sınırlarında meydana gelen yansımalardır. Bu yansımaları bastırmak için, hem sınır bölgesi hesaplama alanından uzak tutulmuş, hem de soğuran sınır koşulları kullanılmıştır.

Çeşitli hız modelleri için üretilen yapay sismogramlarda gözlemlenen değişimler 2B ve 3B yaklaşımların farkını ortaya koymuştur. 1B ve 2B hız modelleri üzerinde elde edilen sismogramlardaki ilk varışlarda farklılık yoktur. Fakat uygulama açısından bakıldığında gerçek ölçümlerde ölçüm hattına koşut süreksiz yapıların etkisinin yorumu önem kazanmaktadır. Bu tür modelleme çalışmaları beklenen belirtileri tanımlama açısından önemlidir ve yorumcuya yol gösterici olabilir.

GİRİŞ

Sismik yöntemlerde kaynak dalga etkisi ile titreşen malzemeler, üzerlerindeki dinamik yükleri çevresindeki birimlere iletirler. Kaynak dalgacık yer içinde ilerlerken süreksizlikler ile karşılaştığında, jeolojiye bağlı olarak yansıma veya kırılmalara uğrayacaktır. Gerçek koşullarda üç boyutta yayılan kaynak dalgacık yer içinde seyahatini sürdürürken kayıtçılara ulaşıncaya dek izlediği bütün yolların etkisini taşıyacaktır. Bu açıdan bakıldığında karmaşık şekilli jeolojik yapıların neden olacağı belirtilerin hesaplanmasında kullanılacak yöntemler önem taşımaktadır.

Karmaşık yer modellerinin yanı sıra zamana ve konuma göre değişim gösteren ortam değişkenlerinin (parametre) göze alınması gerektiğinden yapılacak olan analitik çözümlerde güçlükler söz konusu olabilmektedir. Buna karşın sismik dalgaların yayılım modellemeleri bilgisayar ortamında yapılan sayısal hesaplama teknikleri ile birçok koşulu bir araya getirerek etkin çözümler üretebilmektedir. Sismik yöntemlerde akustik özelliğe bağlı olan dalga yayılımı, akustik dalga denklemi ile kontrol edilmektedir. Üç boyutlu akustik dalga yayılım modellemeleri kullanılarak yapılan nokta bazında hesaplamalar, üç boyutta süreksiz, karmaşık şekilli yapıların etkisini yapay sismogram üzerinde ortaya koyabilmektedir. Bu şekilde tanımlanan koşullar doğrultusunda; yer içi hız modelinin kaynak sinyale vereceği tepki, gerçeğe daha yakın bir şekilde gözlenebilecek ve ortamda tanımlanan keyfi sayıda kayıtçılar, istenilen noktalarda yapay sismogram üretebilmeyi mümkün kılacaktır.

Çalışma kapsamında akustik dalga denklemi, zaman uzayında sonlu farklar yöntemi ile sayısallaştırılmış ve gerekli hesaplamaların yapılabilmesi için bir bilgisayar algoritması geliştirilmiştir. Matlab programlama dili ile işlevsellik kazandırılan algoritma sayesinde, hesaplanacak akustik dalga yayılımı; tanımlanacak olan, karmaşık şekilli hız modellerine ait yapay sismogramların, kolayca oluşturulabilmesini sağlamaktadır. Gerçekleştirilen hesaplamalardan elde edilen sonuçlara ilişkin olarak sunulan örneklerde, 2 ve 3 boyutlu yapıların, tek hat üzerinde elde edilen yapay sismogram üzerindeki etkisi incelenmiştir.

YÖNTEM

AKUSTİK DALGA DENKLEMİ

Üç boyutlu, tekdüze, yön bağımsız (homojen, izotrop) bir ortamda, 3B akustik dalga hareketi Kartezyen koordinatlarda nokta kaynak için aşağıda ifade edilen akustik dalga denklemi tarafından kontrol edilmektedir (örn. Mufti ve ark., 1996).

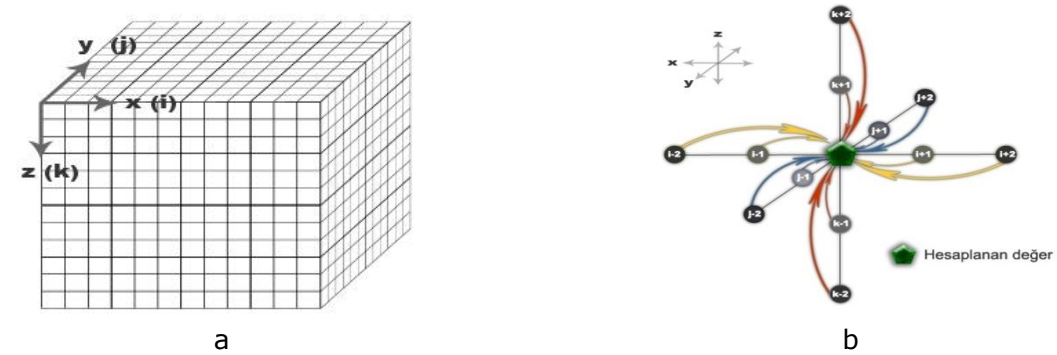
$$\frac{1}{v^2(x,y,z)}\frac{\partial^2 U}{\partial t^2} = \frac{\partial^2 U}{\partial x^2} + \frac{\partial^2 U}{\partial y^2} + \frac{\partial^2 U}{\partial z^2} + f(x,y,z,t) \qquad (1)$$

Burada *(x,y,z)* uzaysal konumları, *t*: zamansal konumu, *U:* yer değiştirme bileşenlerine bağlı oluşan dalga alanını, $f(x,y,z,t)$: 3b ortam için noktasal kaynak fonksiyonu ve v(x,y,z): P dalga hızını ifade etmektedir.

SONLU FARKLAR YÖNTEMİ

Akustik dalga denklemlerinde bulunan konuma ve zamana bağlı türev işleçlerinin Taylor seri açılımı kullanılarak ayrık noktalarda tanımlanıp hesaplanması sonucunda zaman uzayında sonlu farklar yöntemi (yinelemeli SFY) elde edilmektedir. Yinelemeli sonlu farklar yöntemi kullanılarak yapılacak modelleme çalışmalarında, dalga alanının tanımlanabileceği yarısonsuz bir ortam seçilerek, kendi boyutlarından ve kaynak sinyalin dalga boyundan çok daha küçük ebatlarda hücrelere bölünmektedir. Her bir hücre yapısı diğer hücrelerden bağımsız olarak tanımlanırken, hesaplama maliyeti ve kolaylığı göz önünde tutularak, kendi içerisinde tekdüze olduğu kabul edilmektedir. Ayrık noktalarda tanımlanan hücrelerin birbiri ile konumsal ilişkilerini ifade eden örnekleme aralıkları Δx, Δy, Δz, ile tanımlanırken zaman uzayındaki ilişkilerini temsil eden örnekleme aralığı ise Δt ile belirtilmektedir.

Çalışma kapsamında işlemlerin gerçekleştirilebilmesi için Kartezyen koordinatlarda (i,j,k) 3B bir hesaplama alanı seçilerek, ortam küp şekilli (Δx=Δy=Δz) hücrelere bölünmüştür (Şekil-1). Akustik dalga denkleminde yoğunluğun hız ile doğru orantılı olarak arttığı varsayılarak sabit alınıp, hücre içlerine P (düşey) dalga hızları tanımlanmaktadır. Akustik dalga denklemlerinin sonlu farklar yöntemi ile ayrıklaştırılması yapılırken, zaman uzayında 2. konum uzayında ise 4. dereceden fark yaklaşımları kullanılmıştır. Bu fark yaklaşımları ile yapılacak hesaplamalarda bir hücre değerinin hesaplanmasında kullanılan Kartezyen koordinatlardaki hücre konumları Şekil-1 ile verilmiştir. Bu şekilde ayrık noktalarda tanımlanmış olan hız bileşenleri kullanılarak, yer değiştirme bileşenleri, yinelemeli sonlu farklar yöntemi ile her bir zaman adımında hesaplanıp, dalga alanı elde edilmektedir. Yöntemin hesaplamasına dair detaylara Mufti (1990) ye bakılabilir. Bu şekilde yer içinde tanımlanabilecek olan keyfi çeşitlilikte ve karmaşık şekilli yapıları temsil eden hız modellerinde yapılan hesaplamalar doğrultusunda başka koşula gerek duymadan dalga yayılımı gerçekleştirilebilecektir. Dalga yayılımı sırasında ortamda keyfi sayıda ve konumda tanımlanabilecek olan kayıtçılar, her bir yineleme adımında genlik değerlerinin kaydını tutarak, yapay sismogram üretebilmeyi mümkün kılmaktadır.

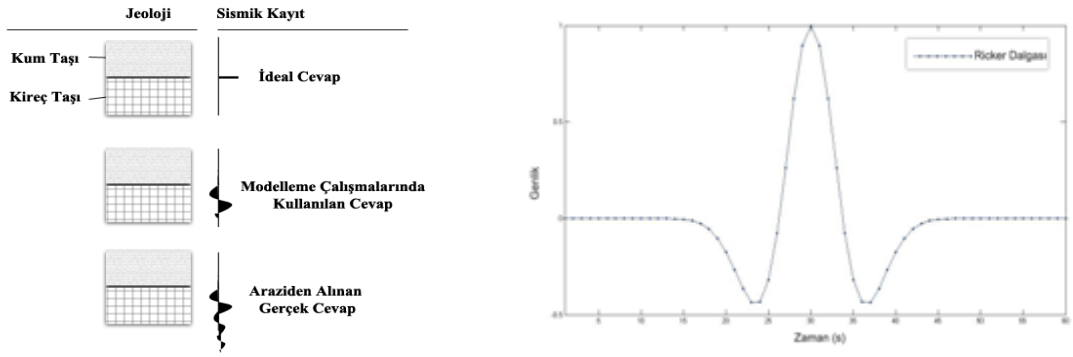


a b

Şekil 1. a) Kartezyen koordinatlarda örnek bir hesaplama alanı, b) Konum Uzayında 4. derece fark yaklaşımları ile hesaplanan SF yönteminde kullanılan hücre konumları.

KAYNAK UYGULAMASI

Kaynağın yer içi ile evrişiminden elde edilmek istenen ideal tepki; bir atım (pik) şeklinde olacaktır. Bu şekilde tanımlanabilecek bir dalga formu mümkün olmamakla beraber modelleme çalışmalarında dalga formları sıfır fazlı dalgacık kullanılarak bir atım şekline yaklaştırılmaya çalışılmaktadır (Şekil-2).



a b

Şekil 2. a) Sismik kayıtta, süreksizliklerin tepkisini ifade eden cevap türleri b) Ricker dalgacığı.

Yinelemeli sonlu farklar yöntemi doğrultusunda hesaplama işlemlerinde başlangıç (t=0) anında hesaplama alanı durağan ($U^0_{i,j,k}$=0) olarak kabul edilmektedir. Hesaplama işleminin başlaması ile yer değiştirme bileşenlerine eklenen kaynak sinyal, dalga yayılımını tetikleyecektir. Çalışma kapsamında dalgacığın akustik özellikleri ve sismik ayrımlılığı artırması göz önünde bulundurularak, sıfır fazlı bir dalgacık olan Ricker dalgacığı kullanılmıştır (şekil-2). Hesaplama sırasında Ricker dalgacığı, her bir zaman adımında tek bir noktada yer değiştirme bileşenlerine eklenerek, nokta kaynak etkisi oluşturmaktadır. Ricker dalgacığı aşağıdaki bağıntı ile ifade edilebilir.

$$F(t) = (1 - 2(\pi f_{maks}(t - 1/f_{maks}))^2)e^{-(\pi f_{maks}(t-1/f_{maks}))^2} \quad (2)$$

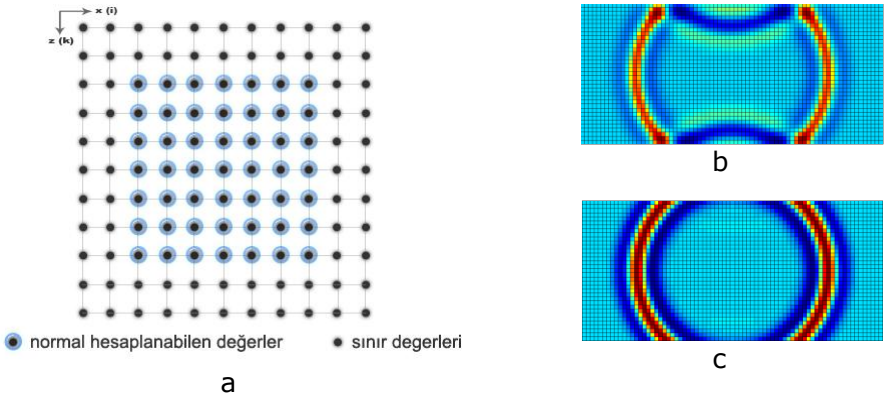
Burada $F(t)$: kaynak fonksiyonu, t: zaman, f_{maks}: baskın frekansı temsil etmektedir. Ortama uygulanan sismik kaynağın frekansı, en büyük/baskın frekans olarak kabul edilerek, dalgacığın yer içindeki soğurulma miktarını, inebileceği derinliğini, kırılma ve yansıma genliklerindeki belirleyiciliği yönünden önem taşımaktadır.

SINIR KOŞULLARI

Sonlu farklar yöntemi kullanılarak yapılan dalga yayılım modellemelerinde karşılaşılan en büyük sorunlardan biri sınır koşullarıdır. Seçilen hesaplama alanının kenarlarında bulunan birkaç hücrede tanımlı ayrık dalga alan bileşenlerinin hesaplanamamasından kaynaklanan bu sorun, üzerine gelen dalga cephesini tam yansımaya uğratmaktadır. Bu koşullar altında yapılacak olan hesaplamalarda dalga yayılımı ve oluşturulacak olan yapay sismogram üzerinde yanıltıcı bir etki gözlenecektir. Çalışma kapsamında seçilen yönteme bağlı olarak sınırlarda bulunan iki düğüm noktasının değeri hesaplanamamaktadır (Şekil-3). Bu durumun engellenebilmesi amacıyla deneye dayalı

$$U_{1,j,k}^{n+1} = +2U_{1,j,k}^{n} + U_{2,j,k}^{n} - U_{2,j,k}^{n-1} + \sqrt{\frac{(v^2{}_{1,j,k}\Delta t^2)^2 + (v^2{}_{maks}\Delta t^2)^2}{2\Delta x^4}}\left(2U_{2,j,k}^{n-1} - U_{1,j,k}^{n-1} - U_{3,j,k}^{n-1}\right) \quad (3)$$

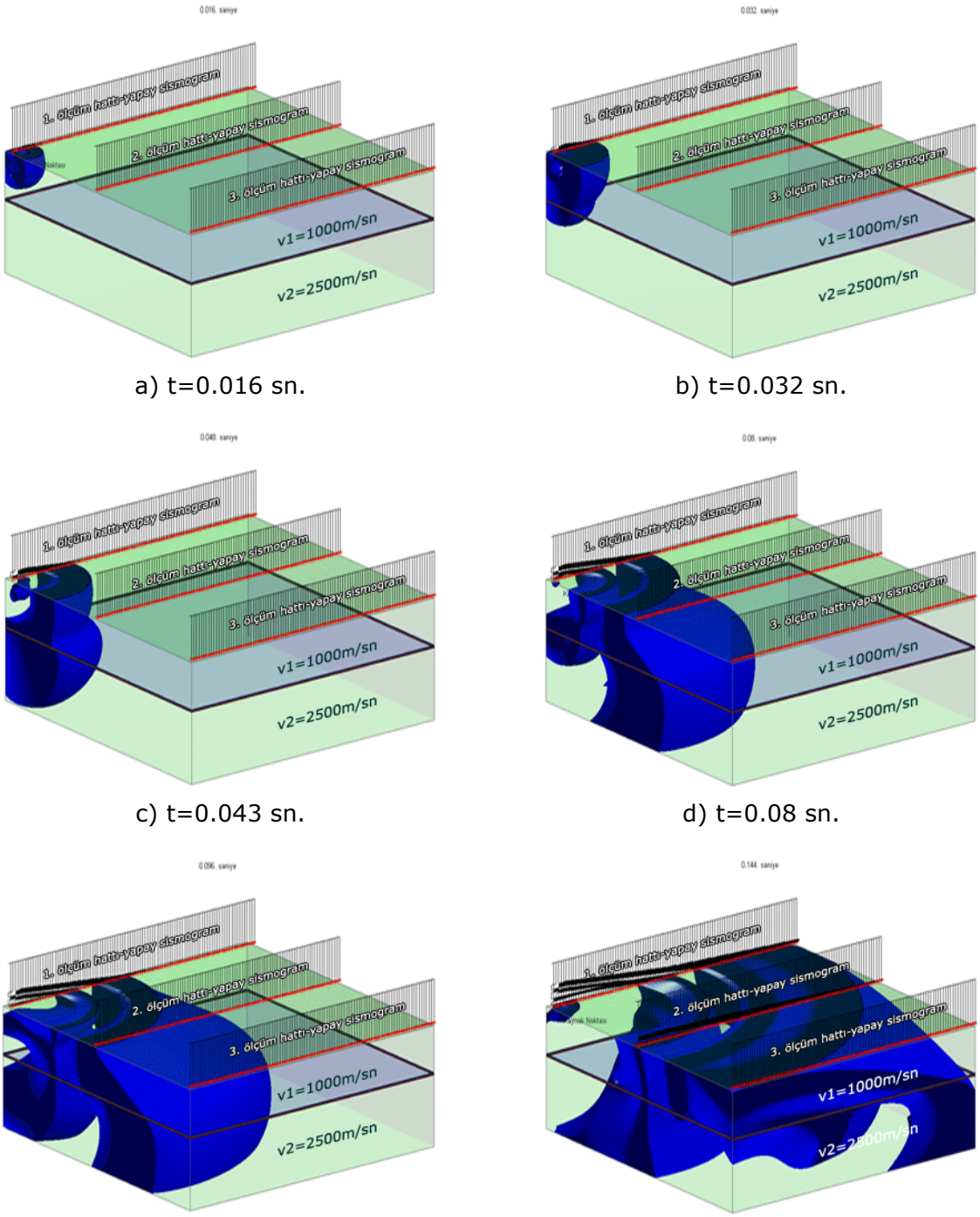
şeklinde bir bağıntı kullanılıp, yansıyan dalga genlikleri bastırılmaya çalışılmıştır. Burada üst imge n; zaman adımını, U; yerdeğiştirme bileşenlerine bağlı dalga alanının düğüm noktalarındaki ayrık değerlerini temsil etmektedir. i=1 için gösterilen bu bağıntı diğer sınır değerlerine uyarlanarak kullanılmıştır. Bağıntının yeterli görülmediği durumlarda hesaplama alanı sınır hücrelerinden uzaklaştırılmıştır. Sınır değerlerinin ihmali sonucunda tekdüze bir ortamda hesaplama alanı kenarlarında oluşacak olan yansımlar ve kullanılan yöntem doğrultusunda, yansımaların bastırılmış halleri Şekil-3 ile 2B kesit üzerinde gösterilmiştir.



Şekil 3. a) Örnek bir hesaplama alanında sınır değerlerine ait konumların 2B kesit yüzeyinde gösterimi, b) Sınır değlerinin etkisi ile hesaplama alanında oluşan yansımalar, c) Soğuran sınır koşulları etkisiyle sınır değerlerindeki yansımaların bastırılmış halleri.

UYGULAMALAR

Çalışma sonucunda geliştirilmiş olan akustik dalga yayılımının gerçekleştirilmesine örnek olarak iki tabakalı hız modeli Şekil-4 ile sunulmuştur. Basit bir gösterim olması amacıyla, üç boyutlu ortamda bir boyutlu olarak tanımlanan, iki tabakalı hız modelinde kaynak sinyalin oluşturduğu dalga cephesinin ortamda ilerleyişi gözlenebilmektedir. Örnekteki dalga yayılımı hesaplanırken, 3 hat boyunca tanımlanan kayıtçılar ile her bir zaman adımında kayıt tutularak yapay sismogram oluşturulması gözlenebilmektedir



a) t=0.016 sn.

b) t=0.032 sn.

c) t=0.043 sn.

d) t=0.08 sn.

e) t=0.096 sn.

f) t=0.144 sn

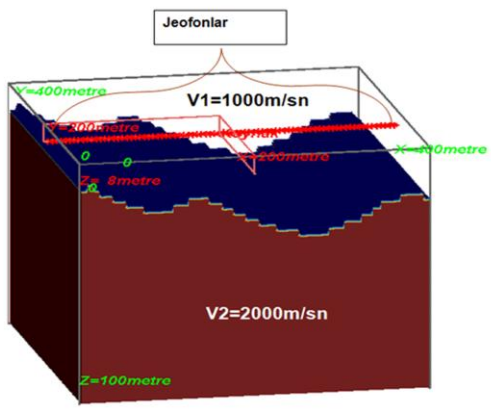
Şekil 4. İki tabakalı örnek bir hız modelinde, zaman uzayında akustik dalga yayılımı ve yapay sismogramın oluşturulmasının farklı zaman adımlarındaki gösterimi.

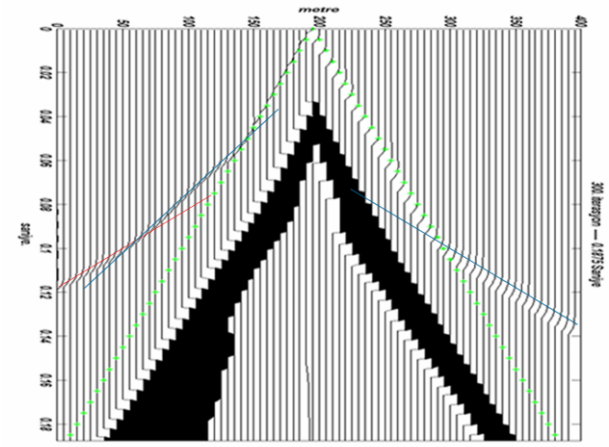
Burada mavi ile gösterilen alan, dalga cephesini temsilen ricker dalgacağının sıfırdan küçük olan değerlerinin çizdirilmesi ile elde edilmiştir.

ÖRNEK-1 ENGEBELİ ARA YÜZEY HIZ MODELİ

Engebeli ara yüzey bulunduran (ondülasyonlu) bir hız modelinin incelenmesi amacıyla, 1000m/sn ve 2000m/sn hızlı, iki tabakalı 2B bir model ele alınmıştır. Orta atış tekniği kullanılarak ortamda dalga yayılımı gerçekleştirilmiş ve yapay sismogram

oluşturulmuştur. Yapay sismogram incelendiğinde, ortamda yayılan sinyalin, ara yüzeyin şekline göre davranış göstereceği gözlemlenebilmektedir. Derinlik arttıkça kırılarak gelen dalgalarda gecikmeler söz konusu olacaktır. Tümsek ve çukur şeklindeki yapılar, sismogram üzerinde birden fazla kırılma gözlenmesine sebep olabilmektedir. Bir boyutlu yapılan çalışmalarda yanıltıcı etkiye sebep olan bu durum göz önünde bulundurulmalıdır.





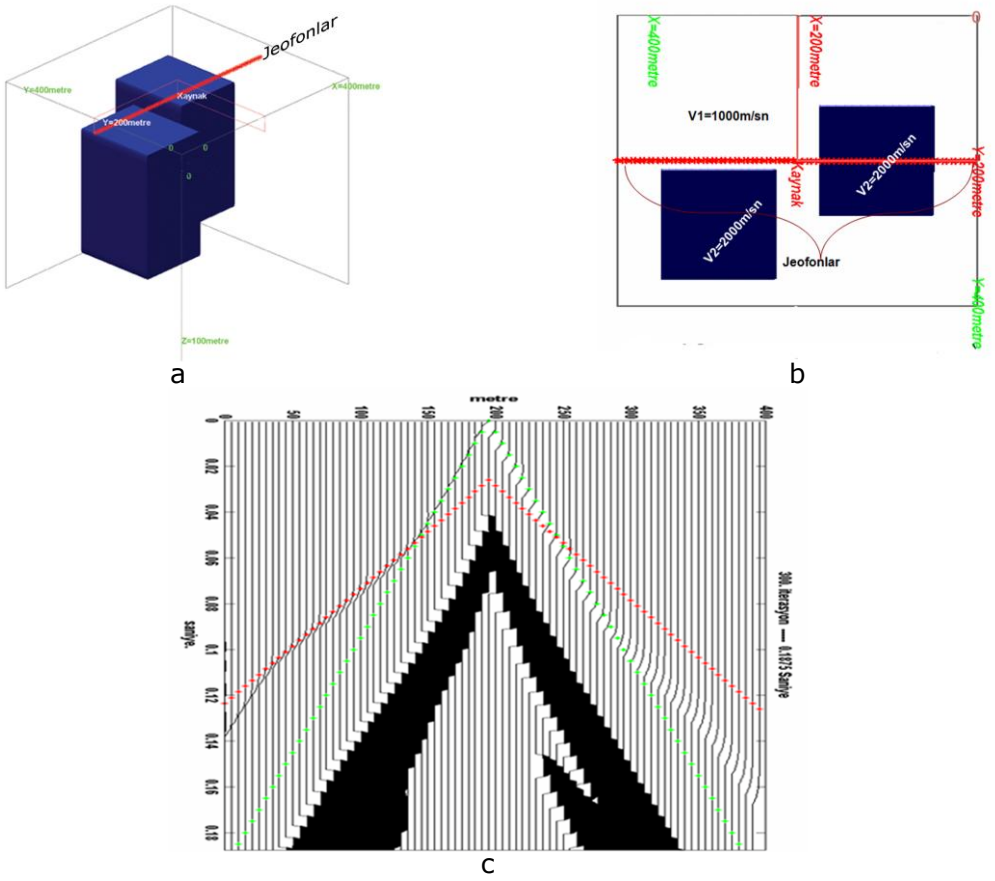
a b

Şekil 5. a) 2B engebeli arayüzey hız modeli b) Engebeli arayüzey hız modeli kullanılarak oluşturulmuş olan yapay seismogram.

Yapay sismogram üzerindeki yeşil ile işaretlenen noktalar, doğrudan gelenler için hız=yol/zaman bağıntısı ile kuramsal olarak hesaplanarak çizdirilmiştir. Sismogram üzerinde görülebileceği gibi, kuramsal veri ile sismogram birbirini desteklemektedir. İkinci tabakanın etkisiyle oluşan kırılmalar, kuramsal olarak hesaplanmamış, kırmızı ve mavi doğru parçaları, sismogram üzerinde ilk varışların gösterilebilmesi amacıyla çizilmiştir.

ÖRNEK-2 FARKLI KONUMLARDA TANIMLI PRİZMATİK YAPILARA AİT HIZ MODELİ

Yer içinde gömülü olan; birbiri ile özdeş, iki prizmatik yapının, ölçüm hattının düşeyinde veya koşut bir konumda bulunması durumunda, sismograma etkisinin incelenmesi amacıyla bir hız modeli oluşturulmuştur. Yapılar yüzeyden 20 m. derinlikte tanımlanmıştır. Birinci yapı ölçüm hattının düşeyinde, ikinci yapı ölçüm hattına 15 metre yatay uzaklıkta tanımlanmıştır. Yapılar profil hattı boyunca 100 m., düşeyde ise 50 m. uzunluğundadır. Ortam hızı 1000m/sn, yapılar ise 2000 m/sn hızlarla tanımlanmıştır. Oluşturulan yapay sismogram incelendiğinde ölçüm hattı düşeyindeki yapının etkisi 50-140 m. aralığında düzgün olarak gözlenebilmektedir. Ölçüm hattına koşut olarak konumlandırılan, diğer prizmatik yapı, üç boyutta yayılan dalganın etkisiyle 280-400m aralığındaki alıcılarda etkisini göstermektedir. İki boyutlu çalışmalarda göz ardı edilebilecek bu durum yorumcuyu yanlış yönlendirebilecektir. Yapay sismogram üzerindeki yeşil noktalar doğrudan gelenleri, kırmızı ile işaretlenmiş noktalar ise ölçüm hattının düşeyindeki yapının etkisiyle oluşan kırılmaları temsil etmektedir. Bu noktalar düzgün yapı özelliği gösterdiği için düşeydeki yapının oluşturacağı kırılmaları ve doğrudan gelenleri temsilen, sistemin doğruluğunun denetlenmesi amacıyla kuramsal olarak hesaplanarak çizdirilmiştir.



a b c

Şekil 6. a) Prizmatik yapıların 3B gösterimi, b) Prizmatik yapıların yüzeyden 2B gösterimi, c) Prizmatik yapılar kullanılarak elde edilen yapay seismogram.

SONUÇ

Çalışmada gerçekleştirilen akustik dalga yayılımı, muhtemel olarak tanımlanabilecek jeolojik yapıya uygun sismik hız modellerinin yapay sismogram üzerindeki etkisini ortaya koymaktadır. Karmaşık bir yapı özelliği gösteren yeraltını bir ve iki boyutlu modelleme çalışmaları veya düzgün geometrik şekilli varsayımlar ile modellemeye çalışmak muhtemel etkileri belirlemekte yetersiz kalabilmektedir.

Çalışma sonucunda çeşitli hız modelleri için üretilen yapay sismogramlarda gözlemlenen değişimler 2B ve 3B yaklaşımın farkını ortaya koymuştur. Üç boyutlu ortamda oluşturulan 1B ve 2B hız modelleri ile elde edilen sismogramlardaki ilk varışlarda farklılık yoktur, fakat uygulama açısından bakıldığında gerçek ölçümlerde ölçüm hattına koşut süreksiz yapıların etkisinin yorumu önem kazanmaktadır. Bu tür modelleme çalışmaları, beklenen belirtileri tanımlama açısından önemlidir ve yorumcuya yol gösterici olabilmektedir. Beklenen etkilerin modellenmesinde ve yorumlanmasında, gerçekçi yaklaşımlar yapabilmek için, 3B yer yapısında, 3. Boyutta süreksiz yapıların sismik iz üzerindeki oluşturabileceği etkilerin göz ardı edilmemesi kesinlikle tavsiye edilir.

TEŞEKKÜR

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KAYNAKLAR

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DETERMINING OF SUBSURFACE STRUCTURES BY RESISTIVITY METHOD

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Key Words: Wenner, Schlumberger, Damped Least Squares (Marquardt) Techniqe, Inversion.

ABSTRACT Modelling process means determining of geometrical and physical parameters which are representing geological structure. Resistivity method is often prefered in modelling studies because of its easy applicability, speed and cheapness In this study, to examine the usefulness of the electrical resistivity inversion method in the horizontal and vertical discontinuities, modelling studies were carried out using synthetic and field 1-D resistivity data. The models used in the study were composed of fault, dayk and layer earth model. The Schlumberger and Wenner electrode configurations widely used in the resistivity method were employed to generate apparent resistivity data and the solutions were obtained by the Damped Least-Squares (Marquardt) algorithm. Iteration progress and the modification of Marquardt factor are controlled by rms value. Accomplishment of these methods were tested by modeling studies on different problems. Then, these models were applied to the field data. Five different models applied to the apparent resistivity data obtained in the field. The reliability of the solutions was studied by data and model misfit approaches. Study resulted with satisfactory solutions, field data and theoretical data are congruent for all model surveys.

INTRODUCTION

By using the lateral or vertical electrical conductivity, determination of geological structure is one of the main goals of the application of electrical methods. In this study, the applicability of the damped least squares algorithm to the determination of model parameters related to different model is investigated. Inversion of resistivity anomalies by damped least squares (Marquardt) technique consist of some mathematical and statistical operations on the the data to generate the most suitable geophysical model approximating the subsurface geological structure. The initial values of the model parameters are assumed approximately in the inversion procedure. The initial values of model parameters are improved iteratively based on the differences between the observed anomalies and theoretical anomalies caused by the model structure.

METHOD

Reflexive resistivity function is based while evaluating vertical electricity drilling datas. In the lateral orientated studies, the principles of image theory is used.

Lateral electricity drilling ;

The used apparent resistivity equations have been proposed by Telford (1976).

Vertical electricity drilling;

n-layer earth model is shown in Figure 1.

A, B : Current electrode

M, N : Potential electrode

When the number of soil exceeds two, the determination of the resistivity and depth for layer from resistivity sounding becames more difficult. For the Schlumberger dc resistivty method, the layered-earth forward response is given by Koefed, 1970.

$$\rho_{as}(s) = s^2 \int_0^\infty T(\lambda) J_1(\lambda_s) \lambda \, d\lambda \qquad (1)$$

s : Half the current electrode spacing

T(λ) : Resistivity transfer function

λ : Variable having the dimension of inverse distance

J : Bessel function

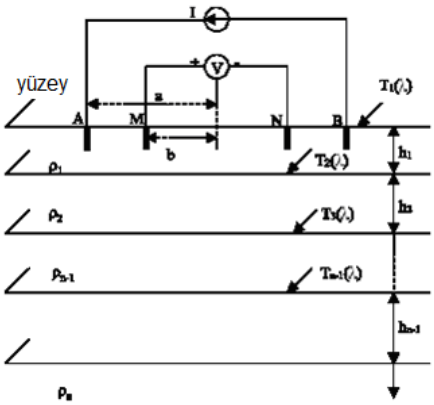


Fig. 1. n-layer earth model with Schlumberger configuration earth and definition of the symbols used in formulas.

DAMPED LEAST-SQUARES (MARQUARDT) TECHNIQUE

$$\Delta p = (A^T A + \lambda I)^{-1} A^T \Delta g \qquad (2)$$

Δp : Parameter correcting vector

Δg : Observed and calculated anomaly distinction

A : jacobian matrix

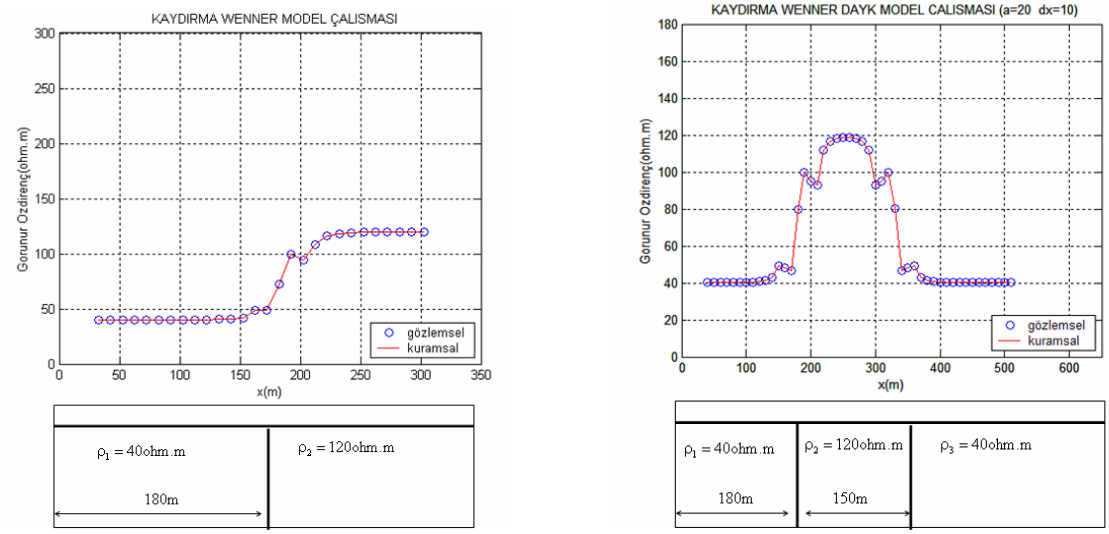
λ : Marquardt factor

I : Identity matrix

Marquardt factor initially were chosen 2. In case of convergence was split into two and In case of divergence was multiplied by two.

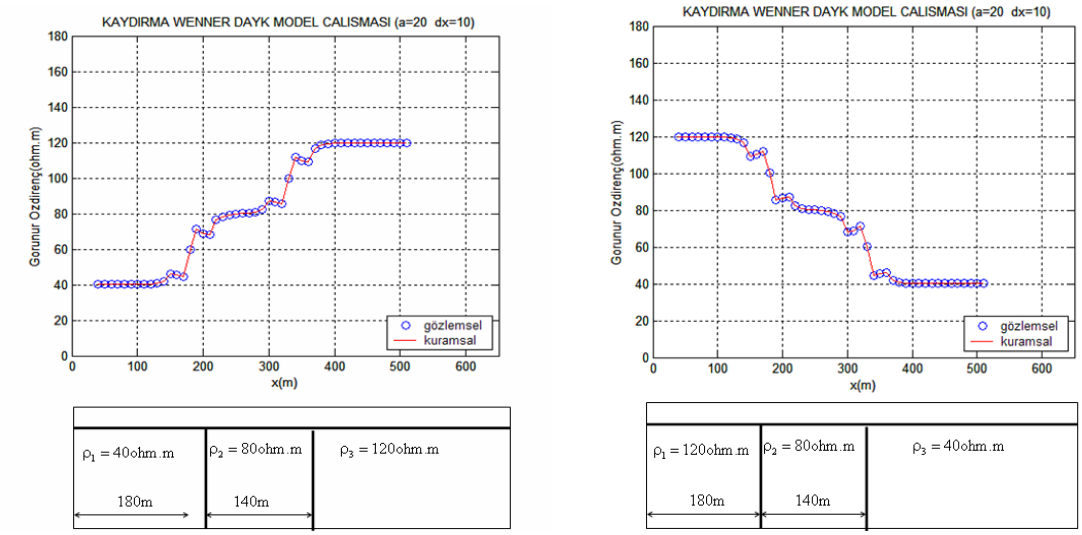
MODELING STUDY

Modelling studies were carried out using synthetic and field 1-D resistivity data. The synthetic data were obtained using four different models. Figure 2,3,4,5 shows model survey results of various model.



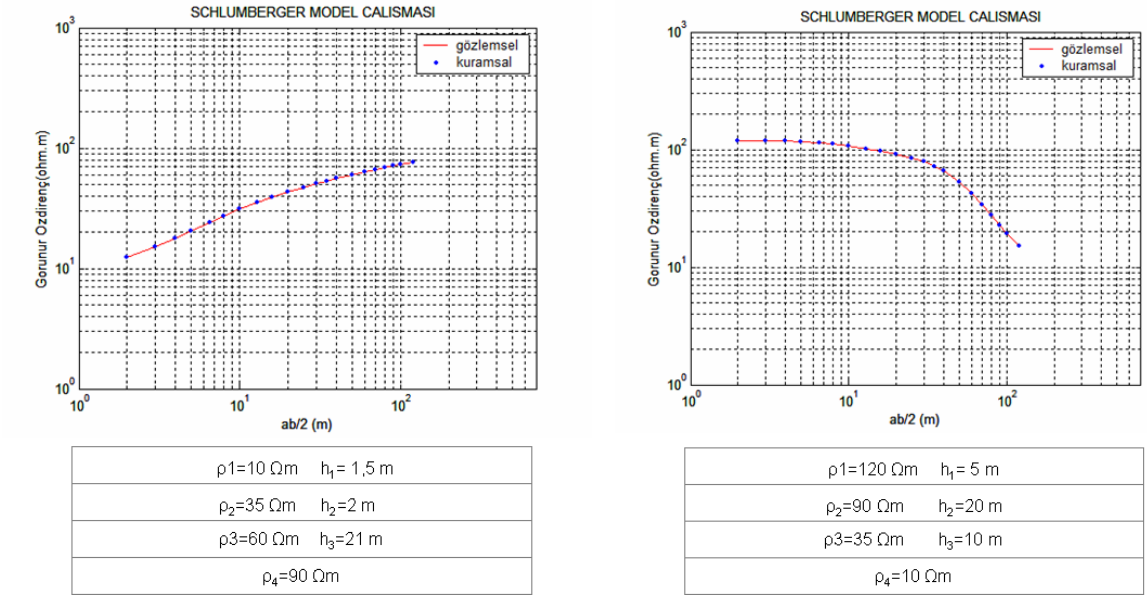
(a) (b)

Fig. 2.a) Fault model. **b)** Dike model.



(a) (b)

Fig. 3.Dike model a)$\rho_1 < \rho_2 < \rho_3$ b) $\rho_1 > \rho_2 > \rho_3$.



(a) (b)

Fig. 4. Four-layer earth model a)$\rho_1 < \rho_2 < \rho_3 < \rho_4$ b) $\rho_1 > \rho_2 > \rho_3 > \rho_4$.

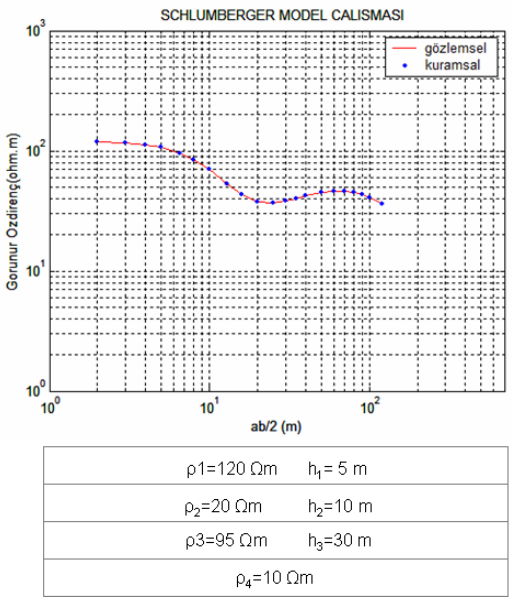


Fig. 5. Four-layer earth model a)$\rho_1 < \rho_2 < \rho_3 < \rho_4$ b) $\rho_1 > \rho_2 > \rho_3 > \rho_4 \rho_1 > \rho_2 < \rho_3 > \rho_4$. .

GEOLOGY OF THE STUDY AREA

The North Anatolian fault zone splays into three strands in the eastern Marmara region. The northern strand crosses the Sapanca lake and extends through north of Armutlu peninsula, northern Marmara Sea and Saros Bay. The middle strand splays from Mudurnu valley in SW direction and extends through Geyve, Iznik, Gemlik, Bandirma and Bayramic. The study area is located on the southeast of Marmara region. Pamukova Metamorphics is composed of amphibolites with marble and quartzite interlayers, which is intruded by granites. This metamorphic basement is uncorformably overlain by metaclastics and recrystallized limestone.

FIELD STUDY

Investigated field is shown in figure 6. Wenner configurations were applied. Then we performed an inversion algorithm developed based on the least square method for evaluating the data (Figure 7,8).

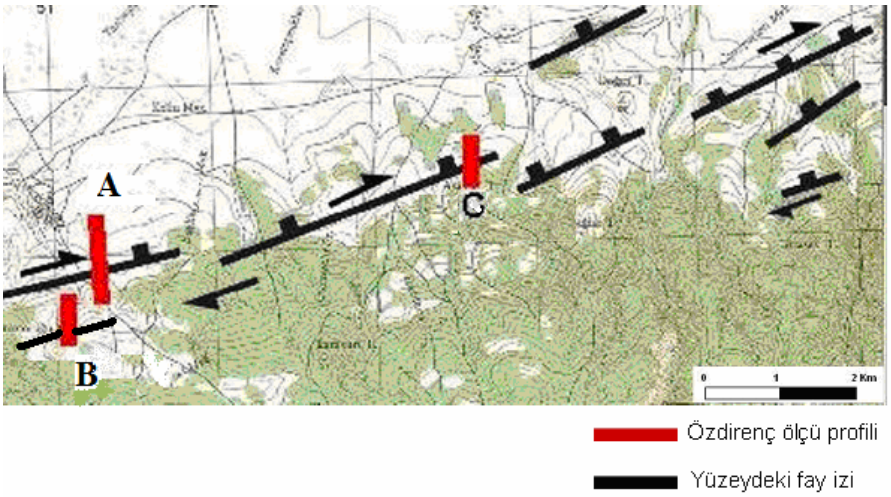


Fig. 6. Location map of study area.

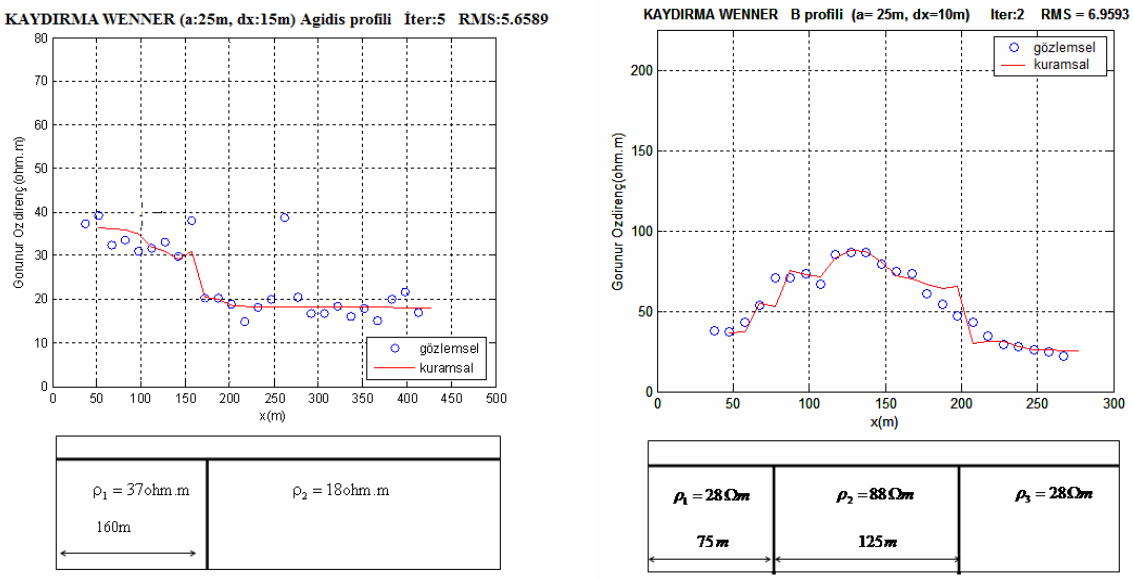


Fig. 7. Fault and dike solutions for A and B profile anomaly.

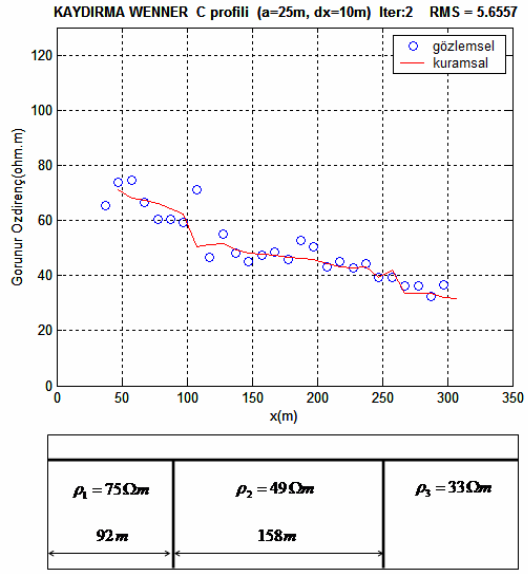


Fig. 8. Solution for C profile anomaly.

Results of inversion for Wenner configuration is shown table1.

ρ_1, ρ_2 : Apparent resistivity

a : Distance between electrode

d : Valid for distance definitions frompoint of measurement to fault

b : Width of dike

Table 1. Results of inversion (Wenner configuration).

Results of inversion (Wenner configuration)

	Initial Parameter					Calculated Parameter								
	ρ_1	ρ_2	ρ_3	d	b	ρ_1	ρ_2	ρ_3	d	b	a	dx	İter	rms
A profile	36	18	----	131	----	37	18	----	160	----	25	15	5	5,6
B Profile	30	92	25	72	120	28	88	28	75	125	25	10	2	6,9
C Profile	74	47	31	94	150	75	49	33	92	158	25	10	2	5,6

Schlumberger measurement and subsurface structure is shown Figure 9.

Cagniard (1953) charts have been used to determine initial parameters. Then the initial parameters have been used as input parameters for inversion process.

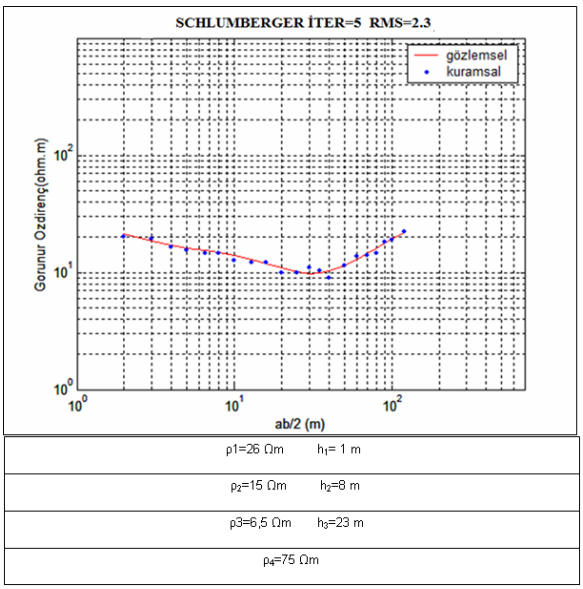


Fig. 9. Solution for schlumberger configuration.

Results of inversion for Schlumberger configuration in the same region is shown Table 2.

Table 2. Results of inversion (Schlumberger configuration).

	Results of inversion (Schlumberger configuration)								
	ρ_1	h_1	ρ_2	h_2	ρ_3	h_3	ρ_4	İter	RMS
Initial Parameter	25	0,8	12	12	5	26	60	5	2,3
Calculated Parameter	26	1	15	8	6,5	23	75	-----	-----

CONCLUSION

Resistivity method has adevantage of being fast and economical. It can be applied to superficial or deep problems. The presented method for estimation of soil parameters from Schlumberger and Wenner measurements. The procedure has been applied using field test data and has been shown to provide o satisfactory earth model.

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EVALUATION OF SOURCE PARAMETERS OF SOME MODERATELY EARTHQUAKES IN SOUTH -EAST OF IRAN

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Key Words: source parameters, source time function, waveform modeling, Gowk fault.

ABSTRACT The occurrence of the South east of Iran earthquake of four large earthquakes provides an important opportunity to study source properties of earthquakes in this region. We constrain source parameters of the March 14 1998 (M_w 6.6) earthquake by analyzing body wave form seismogram and compiled source parameters of 11 June 1981 (Ms 6.7), 28 July 1981 (Ms7.1), 20 November 1989 (mb5.6), events that is obtained from body wave form modeling. The results from waveform modeling (March 14 1998) indicate that source depth was 4 km and that right-lateral strike slip motion mechanism. We have calculated source time function (STF) of March 14 1998 earthquake and compiled this function for other events of Gowk fault system earthquakes that is obtained from teleseismic body waveform modeling. The duration of each event was determined from source time function. Corner frequency and source radius have been calculated for each event by using pulse duration from source time function. Corner frequencies and source radius range are changed from 0.013 Hz to 0.16 Hz and 8.14 to 100 km respectively. Minimum and maximum displacement is observed 8.14×10^{-4} km and 0.01 km respectively.

INTRODUCTION

On March 14, 1998 at 19: 40: 28.3 GMT a large earthquake (M_W= 6.0) occurred on the Gowk fault in south east of Iran, (Fig. 1) (Vernant et al., 2007). Study of instrumentally earthquake records show that some of the large earthquakes had occurred in this suture zone during the past 26 years (Table 1). Much of the mechanical deformation resulting from Arabia-Eurasia collision is accommodated in Iran plateau. Active faulting in Iran is related to the convergence between the Eurasia and Arabia plates, which occurs at about 40 mm yr^{-1} at longitude 60^0 E and is mostly accommodated by distributed shortening within the political borders of Iran (Berberian et al., 2001). While much of this shortening is taken up in the main earthquake and mountains belts of the Zagros, Alborz and Kope Dagh (Quttmeyer, R. C, and K. H. Jacob, 1979), some is also accommodated in central Iran, of which Kerman province is part. Recent and active deformation in Kerman province is dominated by NW- striking thrusts and N-to NNE striking right-lateral strike slip motion, related to the indentation of Iran by the Arabian shield (Berberian et al., 2000). We focus on the Gowk Fault system in south east of Iran ranges.

The essential purpose of this study is to determine fault rupture or source complexity, and prepare information about time history of displacement on the Gowk fault system. In addition we want to evaluate the seismic moment duration (Table 1). Figure 1 shows the geographical location of study area.

Table 1. List of seismic events had occurred in Kerman province during 26 years ago.

Date	Time	Lat.	Lon.	M_w
1981.06.11	07:24:24	29.86	57.68	6.58
1981.07.28	17:22:24	29.99	57.79	6.98
1989.11.20	04:19:07	29.90	57.72	5.83
1998.03.14	19:40:28	30.138	57.588	**6.6**

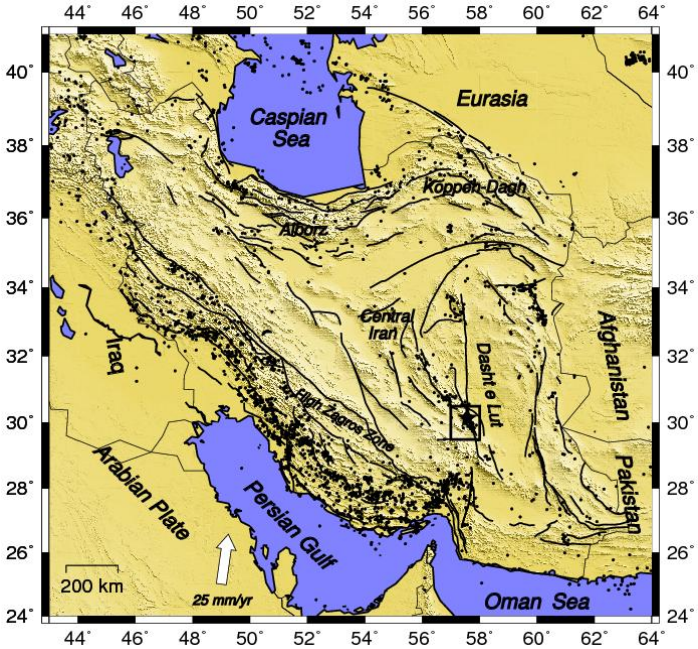

Fig. 1. Iran map which shows the major fault zones and geographical regions. White arrow with 25mm/yr rate represents Arabia- Eurasia plate convergence (Vernant et al., 2007). Arabia- Eurasia convergence occurs in the Zagros, the Alborz, Kopeh Dagh, and possibly in central Iran by the rotation of strike-slip faults. Right lateral shear between central Iran and Afghanistan is taken up on N-S right –lateral faults of the Gowk-Nayband and Sistan suture zone systems, which surround the Dasht-e-Lut. The study area that marked by black box.

HISTORICAL AND INSTRUMENTAL EARTHQUAKES

Study of historical earthquake records (Berberian, M., 1994) shows that some of damaging earthquakes had occurred in Kerman province (the capital of Kerman province is Kerman city) (figure 2). The most significant events are:

The 27 May 1897 earthquake with magnitude Ms=5.7 which affected a larger area, caused damage in the Kerman. In 17 January 1864, the Chatrood earthquake (Ms=6.0) occurred in the region. In April 1854, the Horjand earthquake (Ms =5.8, I_o=VIII) occurred in northeast of Kerman. This catalogue shows that the earthquake had a same trend as Lakarkuh fault. In 1877 Sirch-Hasan-abad earthquake (Ms=5.6) destroyed some of villages (Ab-e-garm, Sirch, Hasan-Abad, Deh-Gholi and Hashtadan villages). Figure 3 shows the location of the five major instrumentally earthquakes with magnitudes of greater than 5 that have struck the cities and villages in Kerman province during the period of 1981 to 1998. These events are listed bellow:

1- The Golbaf earthquake of 11 June 1981, Ms6.7,

2- The Sirch earthquake of 28 July 1981, Ms7.1,

3- The South Golbaf earthquake of 20 November 1989, mb5.6,

4- The North Golbaf (Fandogha) earthquake of 14 March 1998, Mw6.6.

Sirch earthquake is the largest event recorded instrumentally in the Kerman province. The large earthquakes in 1981 were associated with a total 64 km of fresh movements along northern end of Golbaf (Gowk) fault and 10 km at the southern segment of Lakarkuh fault. Maximum vertical displacement of 10 cm were observed east of Golbaf, whereas after the second shock displacement of 14 cm vertical and 20 cm horizontal

(Dextral) were measured near Chahar-Farsang and Poshteh along the Lakarkuh fault system (Zohoorian et al., 1984).

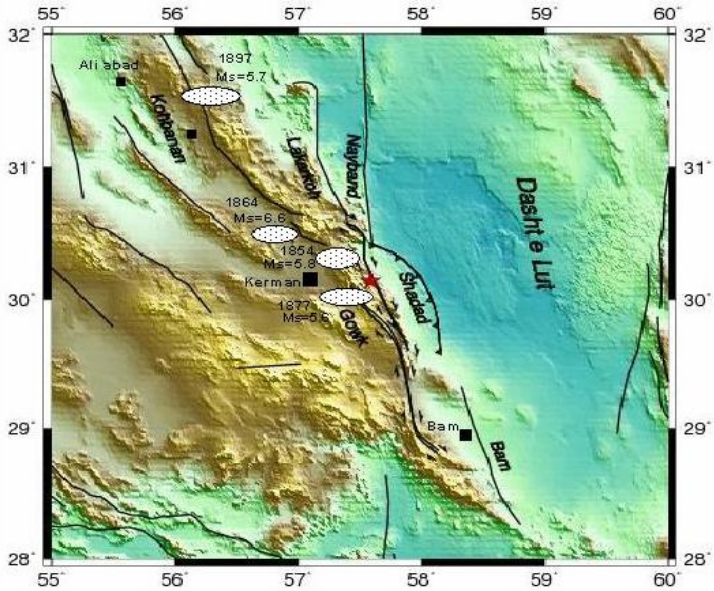

Fig. 2. Map showing the location of some historical records in Kerman province, ( ).

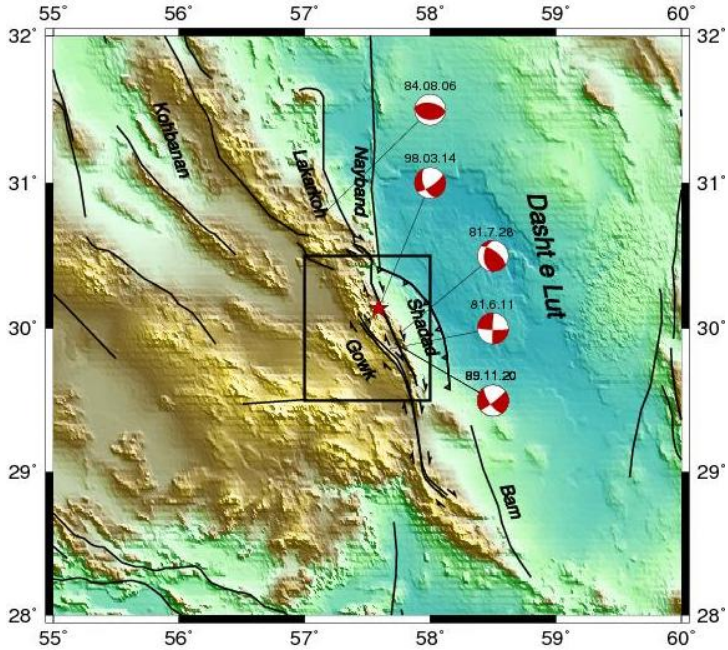

Fig. 3. Tectonic map of the Gowk area and adjacent regions, we reported the main faults from IIEES reports. Beach balls show distribution and earthquake focal mechanisms of instrumentally recorded events with Mw ≥ 5.5 which occurred in study area.

LONG-PERIOD WAVEFORM MODELLING

Body waveform modeling has become one of the most important tools available to seismologist for refining earth structure models and understanding fault-rupturing process. Three component waveform data from the far-field GDSN stations were obtained for the selected earthquakes. IASPEI SYN4 algorithm (McCaffery et al., 1994), which is a recent version of (Nabelek, J. L., 1984) inversion procedure based on a weighted least squares method, was used for waveform inversion. The source time function (described

by a series of overlapping isosceles triangles) (McCaffery et al., 1994), centroid depth, and the fault orientation parameters (strike, dip, and the rake) are used in order to compute synthetic seismograms and the seismic moment.

The inversion procedure adjusts the relative amplitudes of the source time function element, the centroid depth, the seismic moment and source orientation. We refer to this solution as the minimum misfit solution. The Green's function for P and SH waves can be express in the form:

$$g(t)= C^{R}(t)*M(t)*g^{S}(t) \quad (1)$$

Where $g^{S}(t)$ is the displacement of the P or SH waves emerging at the base of the crust in the source region in response to a impulse, M(t) and $C^{R}(t)$ are the responses to these waves by the mantle and crust at the receiver respectively (Taymaz, T. 1990).

Amplitudes are corrected for geometrical spreading and attenuation is introduced with a t^{*}=1s for P wave and t^{*}= 4 s for SH wave (Nabelek, J. L., 1984). As explained by (Fredrich et al., 1988) uncertainties in t^{*} effect mainly the source duration and seismic moment, rather than the source orientation or centroid depth .

The seismic moment clearly depends on the duration of the source time function, and to some extends on centroid depth and velocity structure (Taymaz, T., 1990). We estimated the lengths of time functions by increasing the number of isosceles triangles until the amplitudes of the later ones became insignificant.

INVERSION RESUALTS

The focal mechanism of the 1998 March 14 earthquake was computed by inverting 26 P- and 15 SH- Long-Period waves with good azimuthally coverage (Fig. 4). Having found a set of acceptable source parameters, we followed the procedure described by MaCaffrey (McCaffery , R . & Nabelek, J., 1987), (Nelson et al., 1987) **,** (Fredrich et al., 1988), in which the inversion routine is used to carry out experiments to test how well individual source parameters are resolved. We investigated one parameter at a time by fixing it at a series of values either side of its value yielded by the minimum misfit solution, and allowing the other parameters to be found by the inversion routine. We then visually examined the quality of fit between observed and synthetic seismograms to see whether it had deteriorated from the minimum misfit solution. In this way we were able to estimate the uncertainty in strike, dip, rake and depth for each event (figure 5).

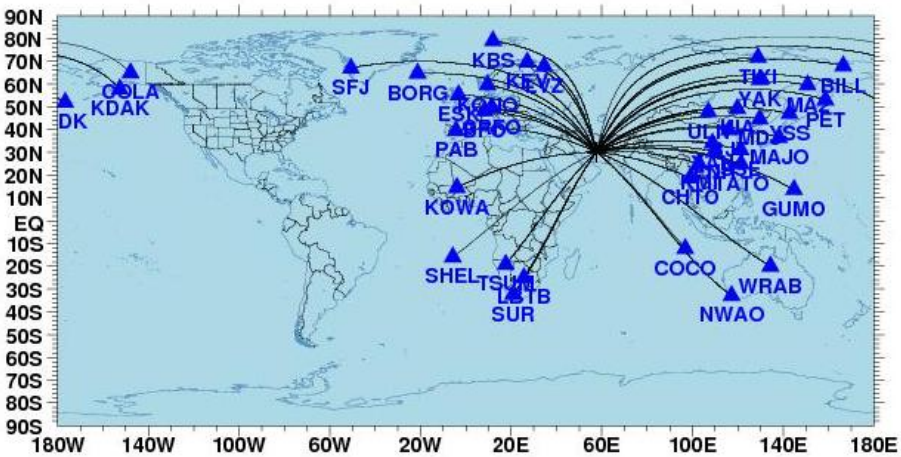

Fig. 4. Map of Incorporated Research Institutions for Seismology (IRIS) stations (Δ =30°-90°) which their seismograms are used in this study, show the appropriated azimuthally distribution. The triangles show the station location, and alphabetic marks show the station code.

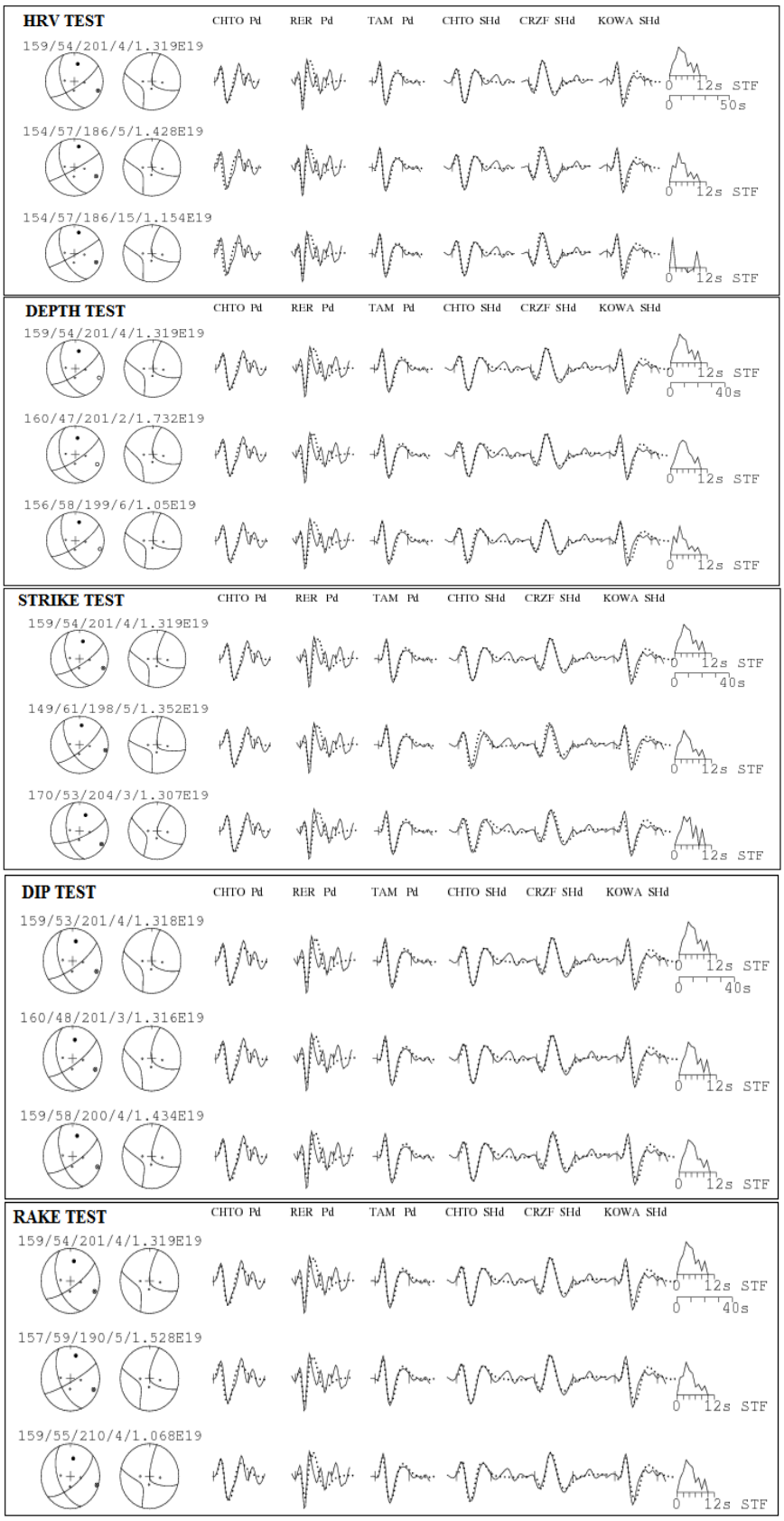

Fig. 5. Test to check of the inversion of 1998 Mach 14 event (sensitivity of source parameters). Synthetic seismograms are doted, observed are solid lines. The first line in each shows the minimum misfit solution. P and SH focal spheres are shown, with the time function and numbers showing the strike, dip, rake, centroid depth and seismic

moment. First box shows the HRV CMT tests, first line in this box indicate the best fit in this study, second line is the HRV solutions with all parameters fixed, and the last line shows the CMT solutions Depth free. We carried out the velocity structure in this study a half space of *Vp* = 6.5 kms^{-1}, *Vs* =3.7 kms^{-1}, and ρ =2.85 gcm^{-3}, respectively. The boxes show the depth, strike, dip, and rake tests.

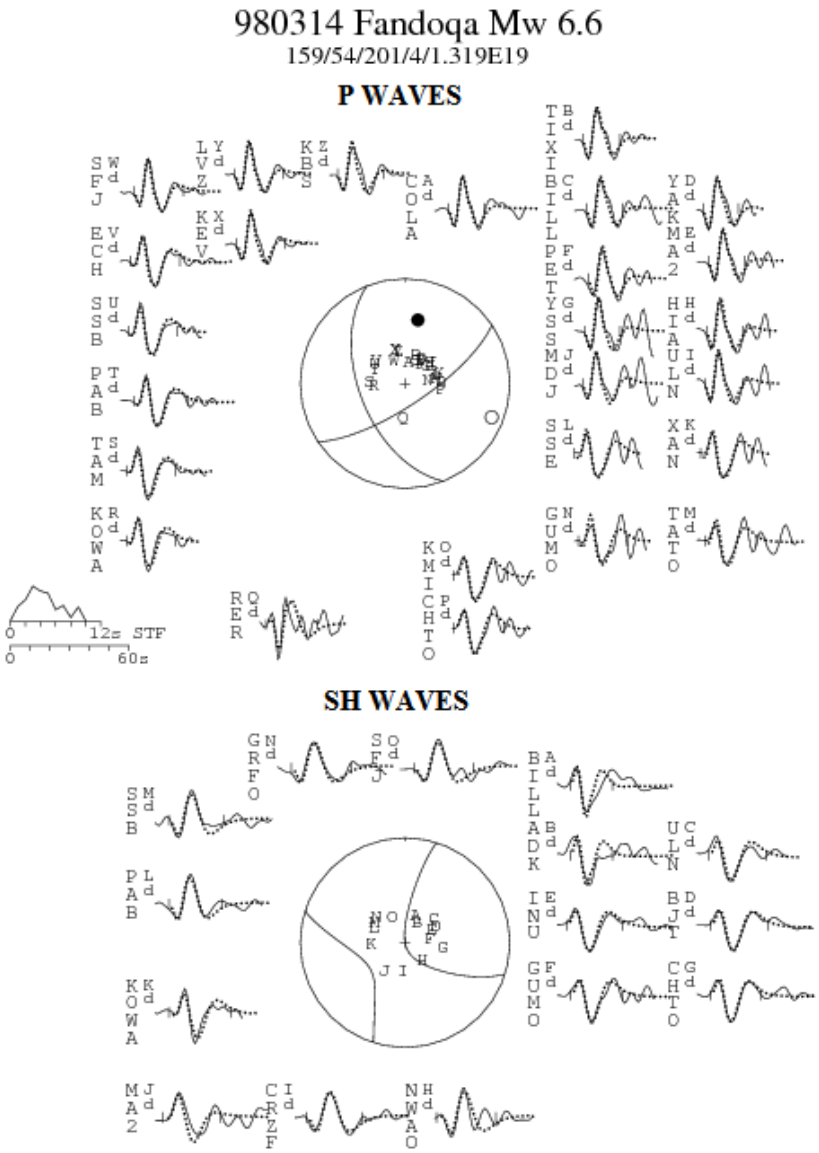

Fig. 6. Minimum misfit solution for the 14 March 1998 earthquake in Gowk fault (Table 2) showing P (top) and SH (bottom) observed (solid) and synthetic (doted) waveforms. Waveforms for each station are arranged azimuthally around the focal spheres. Station positions on focal spheres are identified by capital letters and arranged clockwise starting from north. STF is the source time function. Vertical ticks on the seismograms indicate the inversion window. Numbers beneath the header line are strike, dip, rake, centroid depth (km) and scalar moment (N.M). Stations were weighted according to azimuthally density and then S seismograms weights were halved, to compensate for their larger amplitudes. P and T axes are represented by a black and white dot.

The synthetic seismogram solution indicates right lateral strike-slip on a fault dipping 54° (± 5°) NW, with a strike of 158° (-10°/+11°), a rake of 200° (-11°/+9°) a hypocentral depth of 4 km (± 2), a source time function with a duration of 10 seconds while the 95%

of the energy was abruptly released within first 9 sec., and a scalar moment of 1.319 $\times 10^{19}$ N.m (figure 6), (table 2). We carried out all inversions on supposed a half space with V_p = 6.5 km.s^{-1}, V_s = 3.7 km.s^{-1}, and ρ = 2.85 gr.cm^{-3}. Our solution is good agreement with the published in the Harvard CMT catalogue and (Beberian et al., 2001), except in seismic moment that our result indicates a larger value than those obtained in previous studies (see Table 2.) and we think it is more consistence's with observed average displacement.

Table 2. Table 2. Source parameters of recent earthquakes of Gowk region. Seismic moment (M_0) is in units of 10^{18} N.M. The last column gives the earthquake source parameters on each line: from body wave modeling in this paper (T, this study) and Berberian (Berberian, M., et al., 2001). (B, with B1and B2 signifying the first and second subevents in the 1981 June 11 earthquake) or from the CMT solutions by Harvard (H) or USGS (U).

Date	Time	Lat	Long	Depth	mb	Ms	Mw	Mo	strike	dip	rake	R
1981.06.11	07:24:24	29.86	57.68	20	6.1	6.7	6.58	41.8	169	52	156	B1
				12				5.30	182	88	198	B2
				20			6.59	9.82	172	37	171	H
				8			6.59	9.73	169	22	142	U
1981.07.28	17:22:24	29.99	57.79	18	5.7	7.1	6.98	36.69	177	69	184	B
				15			7.24	90.10	150	13	119	H
				22			7.02	43.20	293	67	115	U
1989.11.20	04:19:07	29.90	57.72	10	5.6	5.5	5.83	0.70	145	69	188	B
							5.88	0.82	148	81	165	H
1998.03.14	19:40:28	30.138	57.588	4	5.9	6.9	6.6	13.19	158	54	200	T
				5			6.57	9.09	156	54	195	B
				15			6.58	9.43	154	57	186	H
				8			6.52	7.70	146	58	181	U

SOURCE TIME FUNCTION

The teleseismic source time function gives information about fault ruptures or source complexity. Studies of source complexity are also important to evaluate the validity of asperity models of faulting, where the fault is characterized by localized regions of higher strength. The physical features of teleseismic source time functions appraise the source complexity of the earthquakes. These features include the overall duration, multiple or single event character, individual source pulse widths, and roughness of the time function. The measures of source size and complexity can then be compared with the age of subducted lithosphere, plate convergence rate, and other physical parameters of subduction zone (Hartzell, S. H., and Heaton, T. H., 1985). The earthquakes larger than about M_s 6.9 can rarely be represented by a single point source, even at the wavelengths recorded by the WWSSN 15-100 long period instruments (with a peak response at a bout 15 s period). These earthquakes usually consist of several discrete ruptures, separated by several seconds in time and several km in space (Kanamori, H. & Stewart, G. S., 1987), often occurring on faults with different orientations (Butler et al., 1979).

PREVIOUS WORKS

Three measures of source duration of an earthquake have been reported in the literature (Fig.7 taken from Singh et al.[16]): the total rupture duration (τ_t), the pulse duration of each subevent of a complex earthquake (τ_p), and rise time (τ_r) (Singh et al., 2000).

It is well known that a constant stress drop source model implies that M_o / τ^3 is a constant. For shallow earthquakes the reported values of M_o/τ_t^3 lie between 0.25×10^{23} and 1.0×10^{23} dyne cm/s^3 (Kanamori, H., and J.W.Giwen., 1981), (Furumoto, M., and I. Nakanishi., 1983), (Ekstom , G., R.Engahl., 1989), (Ekstom et al., 1992). These results were obtained from the analysis of teleseismic data. Kikuchi and Ishida (Kikuchi, M., and M. Ishida., 1993) studied source time function (STF) and subevent pulse duration, τ_p, of earthquakes in the Kanto region of Japan, which were recorded by broadband seismographic network near the epicentral region.

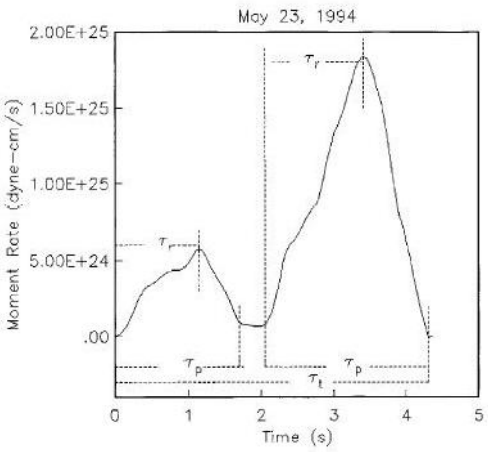

Fig. 7. An example of three measures of source duration of an earthquake (taken from Singh et. al., 2000).

INTERPRETATION AND CONCLUSION

The 11 June 1981 Golbaf earthquake produced surface ruptures on two subparallel N-S strands of the Gowk fault system. Displacement were small, typically with 3 cm right-lateral strike slip and 5 cm vertical offset on the longer eastern fault and hairline cracks on the shorter western one (Berberian et al., 1984), (Priestly et al., 1994), (Jackson ET AL., 2000). This event shows a complex rupture process, involving slip in at least two

sub events of different orientation (Berberian et al., 1984). We can see clearly same pattern in STF (Fig. 8). The complexity of the source time function as evidenced by the complexity of the body phases, low fault rupture velocity, is explained in terms of a multiple source.

The STF of 28 July 1981 Sirch earthquake have larger moment release in the second part of the process and consisted several impulses that can be interpreted as a more complex rupture process (Fig.8). These earthquakes have larger moment release in the second part of the process, while ratios of subevent for foreshock occurs in the first part and initiate with small release followed after 4s.

The STF of 20 November 1989 and 14 March 1998 earthquakes started with a high release of energy in first part of subevent one. Varying the seismic moment along total duration of STF is direct related to varying the source velocity structure did have an effect on centroid depth and seismic moment. In addition uncertainties in attenuation factor, t^*, mainly affect estimates of source duration and seismic moment. The STF of these events show that the rupture characteristics contain a different size of subevents with same shape and same rupture history. The rise time of subevent 1 is larger than another subevents and most part of seismic moment release in first rupturing process. The nature of this function shows that the faulting consists of several fractures separated by strong barriers, which remain unbroken after the event. If the barriers are completely broken, there may be no aftershocks within the main-shock fault plane.

We have used that the corner frequency is very approximately given by $f_0 = 1/\pi\tau_p$ where τ_p is the largest pulse duration (Table 3) (Helmberger, D. V. and S. D. Malone., 1975), (Husebye, E. S. Mykkeltveit, S., 1980). The essential purpose of calculates the corner frequency is evaluating the source dimension. Then we have calculated source radius using Brune (Brune, J. N., 1970) relation for a circular fault (Table 3). We used 3.5 km/ sec for rupture velocity.

$R = 2.34\ v / 2\pi f_0$

The seismic moment, M_o, is given by $M_o = \mu A u$ where μ is the rigidity ($\sim 3\times 10^{10}$ Nm^{-2}), A is the fault area, and u is the average displacement. Let us assume that the fault is roughly circular in area with diameter L, and that the ratio u /L are $\sim 5\times 10^{-5}$ (see e.g. Scholz, Aviles & Wessnousky., 1986). The displacement value for each event calculated by using above assuming (Table 3),.

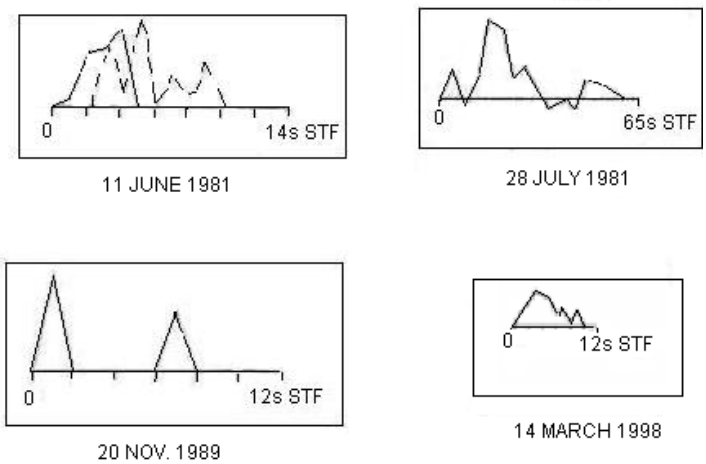

Fig. 8. Far-field source time functions of the 3 January 1969, 30 July 1970, 22 February 1984, 6 March 1986, 14 February 1971, 17 September 1989, 28 November 1991 earthquakes showing emergent seismic moment release (see Priestley et. al., 1994, Jackson et. al., 2002).

Table 3. Source parameters of largest sub- events that is observed from source time functions.

No	Date	f_o (hz)	r_0 (km)	τ_t (sec)	Displacement (km)
1	1981.06.11	0.063	20.69	5	2×10^{-3}
2	1981.07.28	0.013	100	25	0.01
3	1989.11.20	0.16	8.14	2	8.14×10^{-4}
4	1998.03.14	0.053	24.59	6	2.4×10^{-3}

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EVIDENCE FOR THE SEISMIC QUIESCENCE PRIOR TO THE JANUARY 25, 2005 HAKKARI (M_W=5.9) EARTHQUAKE, TURKEY

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Key Words: Hakkari, Precursory, Seismic quiescence, Decluster, Earthquake prediction.

ABSTRACT In this study, we focused on the detection of the precursory seismic quiescence before the January 25, 2005 Hakkari earthquake occurred in Turkey. The analysis was carried out on the instrumental catalogue of the Bogazici University, Kandilli Observatory and Research Institute (KOERI), covering the period from February 1970 to December 2005 for the region 37.2º-39.6ºN and 42.4º-44.6ºE. The catalogue is homogeneous for duration magnitude, M_D, including 664 crustal earthquakes with magnitude $M_D \geq 2.0$ and depths less than 70 km. The magnitude completeness for Hakkari region is 3.2 and the number of events exceeding this magnitude level is 372. In the preliminary phase, the catalogue was declustered using the Reasenberg algorithm. Declustering procedure took away the earthquakes of 12% and 45 events were removed from the catalogue. Thus, the number of events for quiescence analysis was reduced to 327. In order to rank the significance of seismic quiescence, we used standard deviate *Z*-value. Making use of the *ZMAP* software package, we measured the significance of seismicity rate changes at the nodes of a 0.02º grid space in longitude and 0.02º in latitude. The number of events associated with each grid point was selected as 50 after a few tests. The data are subdivided in bins of 28 days for each sample and we used a mowing time window T_W=3 years for the imaging of space and time rate changes. Finally, using the declustered catalogue regional variability of *Z*-value is mapped for every six month between 1990 and 2002. After 1997, it is observed a clear quiescence in and around the main shock epicenter of January 25, 2005 Hakkari earthquake. This quiescence period is best revealed after 1998. To identify the starting time and duration of precursory quiescence we plotted cumulative number of events for a circle of 25.26 km radius centered on Hakkari main shock. The results show that there is an area having a quiescence with Z_{max}=1.6 level at 1999.0 in and around the main shock region. Thus, we found that the M_W=5.9 Hakkari earthquake on January 25, 2005 followed an outstanding seismic quiescence starting 6.07 years before main shock.

INTRODUCTION

Investigation of anomalous seismicity has been an important aspect of earthquake forecast research. Some cases have shown that anomalous seismicity patterns exist, and even time-variations of anomalous seismicity rates, that are medium-short term precursors that emerge before many major earthquakes (e.g. Wyss and Habermann, 1988; Wiemer and Wyss, 1994). Many authors have reported that precursory seismic quiescence occurred in and around focal areas before many earthquakes: Aleutians (Habermann, 1981), Hokkaido (Taylor et al., 1991), Kurile (Katsumata and Kasahara, 1999), Kefalonia (Chouliaras and Stavrakakis, 2001), Bingöl (Öztürk and Bayrak, 2008; 2009). The rate decrease, which can last between one and several years, must precede and lead up to the time of the main shock, or may be separated from it by a relatively short period of increased seismicity rate. The dependence of the duration of the precursory quiescence on the expected main shock magnitude and the quiescence dimensions with respect to those of the source volume of the likely main shock are not yet well known. They may also be a function of the tectonic structure. Thus it is difficult to know what characteristics of quiescence to expect, and it is uncertain whether quiescence occurs in all tectonic settings (Murru et al. 1999). The seismic quiescence

hypothesis assumes that some main shocks are preceded by seismic quiescence, which is a significant decrease of the mean seismicity rate. In some cases, it has been observed that quiescence lasts until the time of the main shock, but in others the quiescence is separated from the main shock by a period of increased seismicity.

Precursor seismic quiescence is defined (Wyss and Habermann, 1988) as a significant decrease (according to some clearly established standard criteria) in mean seismicity rate, as compared with the preceding declustered background rate within all or major part of the source volume. Seismicity and seismic quiescence analyses can be recognized by the methodology introduced by Wiemer and Wyss (1994) and implemented in the *ZMAP* software package. The *ZMAP* technique allows the user to analyze and to obtain graphic displays of seismicty rate changes in both space and time, in selected magnitude ranges. It allows also the quantitative evaluation of the statistical significance of any rate change (quiescence). Thus seismic quiescence has shown promising results in identifying precursory anomalies related to crustal main shocks.

DATA and DESCRIPTION OF THE METHOD

The aim of this study is to define the precursory seismic quiescence prior to the Hakkari earthquake of February 25, 2005. For this purpose, the catalogue data from the website of the Bogazici University, Kandilli Observatory and Research Institute (KOERI) was used. The main shock of 2005 Hakkari earthquake (M_W=5.9, M_D=5.4, where M_W and M_D are moment magnitude and duration magnitude, respectively) occurred at 18:44:06 UTC on January 25 and the epicenter coordinates were determined as 37.68°N-43.87°E (KOERI).The catalog used in this study is complete and homogeneous for duration magnitude, M_D, and covers the events between February 1970 and December 2005. Our analysis is performed in the area limited by latitudes 37.2º to 39.6ºN and longitudes 42.4º and 44.6ºE. The catalog contains 664 events (M_D≥2.0) for epicentral depths less than 70 km. Epicentral map of the shallow seismicity both the original and declustered earthquakes, the tectonics from Şaroğlu et al. (1992) and principal main shocks (M_D≥5.0) in study region are shown in Figure 1.

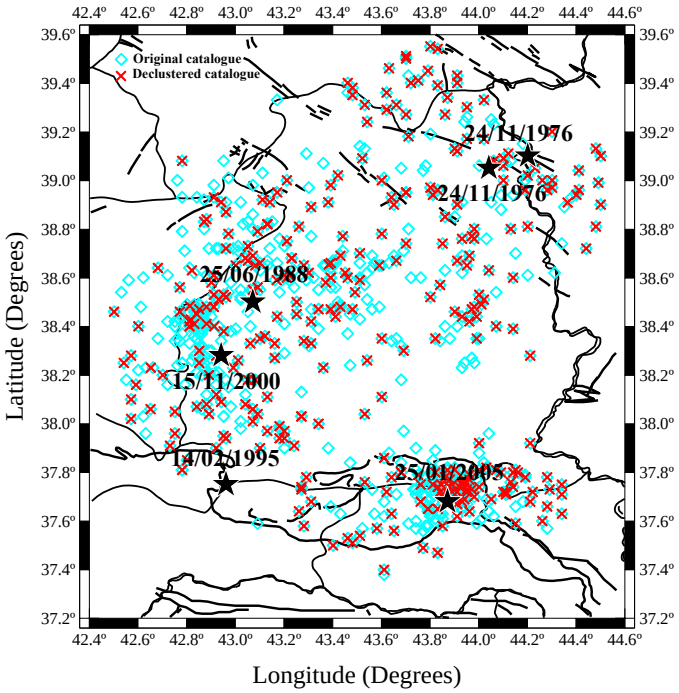


Fig. 1. Epicenter map of the shallow seismicity (M_D≥3.2) in Hakkari region (February 1970-December 2005). The principal main shocks with M_D≥5.0 are indicated with stars. Also, dates of principal earthquakes are given.

Aftershock activity generally masks temporal variations of the earthquake numbers and the related statistics. For the quantitative analysis of seismicity rate changes, it is necessary to eliminate the dependent events (aftershocks, foreshocks, swarms and doublets) from the catalog. To separate the dependent events from the independent ones we declustered the earthquake catalog using the Reasenberg (1985) algorithm. The cluster analysis algorithm of Reasenberg (1985) "declusters" or decomposes a regional earthquake catalogue into main and secondary events (Arabasz and Hill, 1996). It removes all the dependent events from each cluster, and substitutes them with a unique event, equivalent in energy to that of whole series.

The magnitude completeness for Hakkari region is 3.2 and the number of earthquakes exceeding this magnitude range is 372. Declustering procedure took away 45 events and 12% of the events were removed from the data set. Thus, the number of events for *Z*-value analysis was reduced to 327. Thus, after completing declustering processes, a more reliable, homogeneous and robust seismicity data has been obtained. The cumulative number of earthquakes versus time for events with $M_D \geq 3.2$ (original catalog) and for declustered events (excluding dependent events from original catalog) is shown in Figure 2. It is shown no significant changes of reporting as a function of time between 1970 and 1977. As shown in the cumulative number of earthquakes as a function of time, there is a little change in the seismic activity after 1977 until 1995 but further on, it is seen a great seismic changes in the studied area, especially after 2000. Since this evaluation deals with the seismicity rate change and not of energy or moment release, the magnitude of completeness of the catalog used to detect the quiescence is an important parameter because this changes regionally depending on the seismic activity of the area under investigation and the detectability of the network.

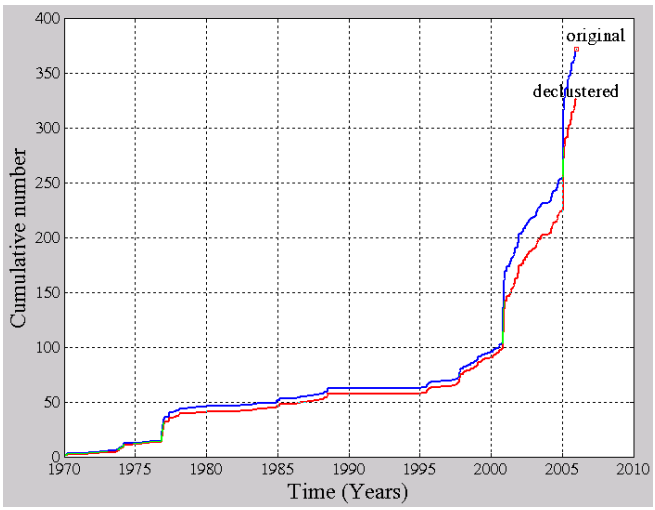


Fig. 2. Cumulative number of events versus time for the original earthquake catalog containing 372 events and for the declustered catalogue containing 327 events, with $M_D \geq 3.2$ and depths less than 70 kilometers from 1970 until 2005.

Many techniques have been used to identify and describe the seismic activity and most of them focus on the phenomenon of seismic quiescence. Visual inspection of the epicenter distribution, the time-distance plots and some statistic methods as well are widely used by researchers (e.g. Ishida and Kanamori, 1977, 1978; Kanamori, 1981). The standard normal deviate *Z*-test is one of the statistic methods frequently used for analyzing the seismic quiescence (Wyss, 1986; Habermann, 1988; Wiemer and Wyss, 1994; Wu and Chiao, 2006). A continuous image of space and time rate changes in seismicity is produced by *ZMAP*, creating a grid of geographical co-ordinates, and associating to each grid node a selected number of nearest events. The subset of events belonging to each grid node is sampled in short time windows (usually a few weeks), so that the average number of events occurred in a time period of several consecutive samples (foreground)

can be compared with that of all the remaining samples (background) (Console et al., 2000). We applied the *ZMAP* method (Wiemer and Zuniga, 1994) for imaging the areas exhibiting a seismic quiescence. Wiemer and Wyss (1994) described details of the method. In order to rank the significance of quiescence, we used the standard deviate *Z*-test, generating the *LTA*(*t*) (Log Term Average) function for the statistical evaluation of the confidence level in units of standard deviations:

$$Z(t) = \frac{R_1 - R_2}{\sqrt{\frac{S_1^2}{N_1} + \frac{S_2^2}{N_2}}} \tag{1}$$

Where R_2 is the mean sesimicity rate computed in the foreground window, R_1 is the average number of events in the whole background period, *S* and *N* are the standard deviations and number of samples, within and outside the window. The *Z*-value computed as function of the time, letting the foreground window slide along the time duration of the catalogue, is called *LTA* (*t*). The duration of quiescence is the important parameter to be determined and its significance is maximized when the length of the time window, T_W (*iwl*), is equal to that value and for meaningful results we demand that they do not depend on the choice of T_W. Since it is not known how long quiescence may last it is varied the window length from 1.5 to 5.5 years, because this is in the range of reported seismic quiescence prior to crustal main shocks.

RESULT of *ZMAP* ANALYSIS

Our investigation of the spatial and temporal variations of seismic quiescence precursor to the Hakkari earthquake of January 25, 2005 is based on the seismic tool *ZMAP*. *ZMAP* is a software package tool for investigation of seismic quiescence and artificial seismic rate changes. The theoretical basis of the software is presented in Wiemer and Wyss (1994:1997) while the coding is given in Wiemer (1996). *ZMAP* allows a custom selection of the size of the grid: latitudinal and longitudinal distance between grid points in degrees as well as of the number of events per grid point.

The spatial variations of *Z*-value are computed with a grid of points spacing 0.02º in longitude by 0.02º in latitude using *ZMAP* software. After a few preliminary tests, we decided to take the nearest earthquakes *N*=50 at each node and we analyzed the rate changes by a moving time window T_W. The sampling interval is selected as a time step of 28 days in order to have a continuous and dense coverage in time. *N* and T_W values are usually selected accordingly in order to enhance the quiescence signal and this choice does not influence the results in anyway. The total duration of the catalogue is 35.87 years. The geographical distributions of *Z*-value in and around Hakkari are shown in Figure 3. Spatial variability of *Z*-values is presented for every six month between 1990 and 2002 based on the declustered earthquake catalogue. T_W value is selected as 3 years. As shown in *Z*-value maps, there is a clear area of quiescence after 1997, nearly overlapping the epicenter of the Hakkari, 2005 main shock with *Z* level between 1 and 2. This quiescence is best revealed at the epicenteral area of the main shock of Hakkari earthquakes between 1998 and 2002.

Our second aim in this study is to find the beginning and duration of quiescence for study region. For this purpose, we plotted the cumulative number of earthquakes in a circular area including the epicenter of the 2005 Hakkari earthquake. Figure 4 shows the cumulative number of events versus time with *LTA*(*t*) function for a circle of 25.26 km radius centered on Hakkari main shock. The *Z*-value peaked with Z_{max}=1.6 at 1999.0. Thus, we observed that the duration of quiescence is 6.07 years before the 2005.07 (February 25, 2005) Hakkari earthquake.

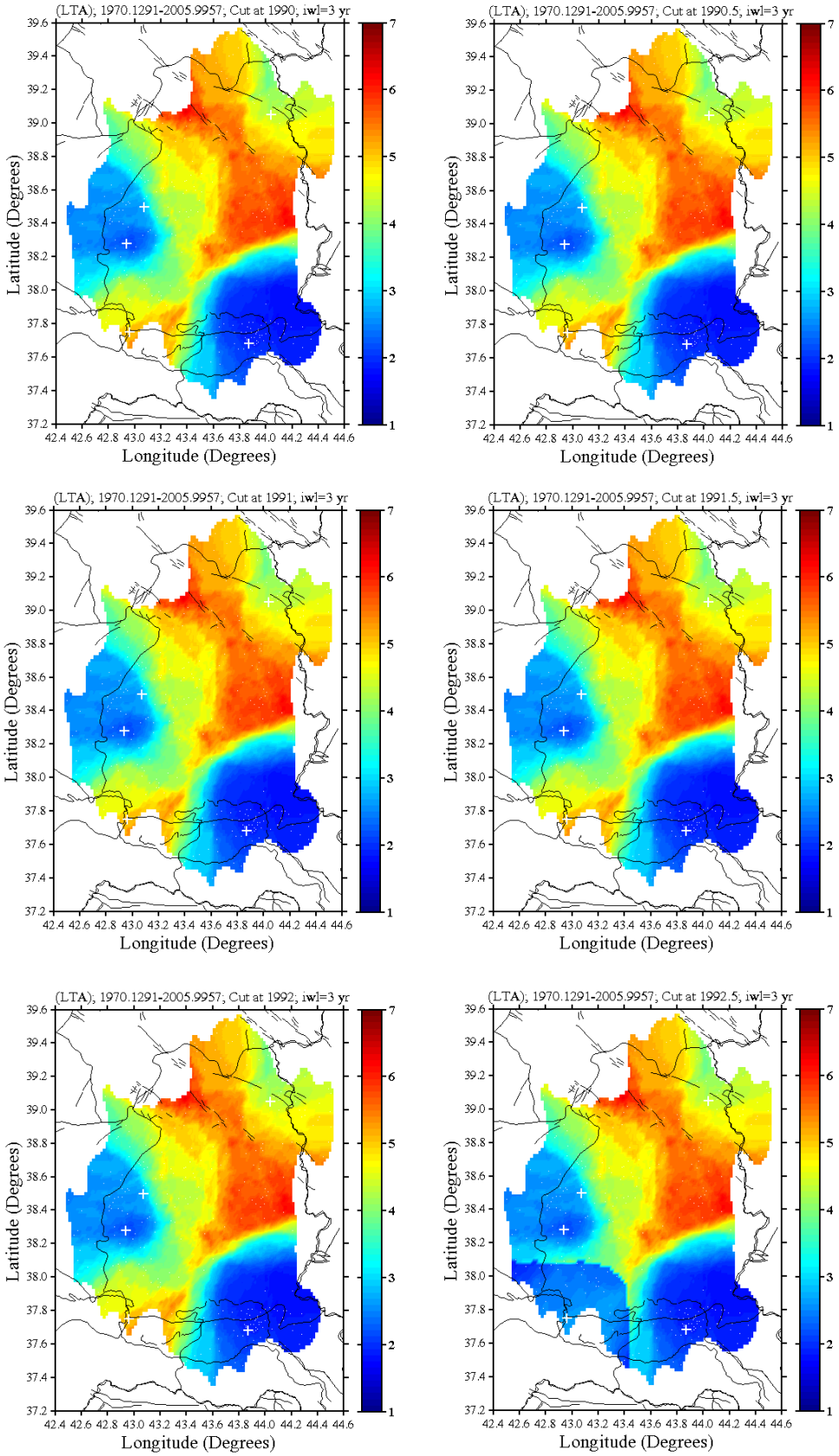


Fig. 3. Seismicity rate changes with *Z*-value for every six month between 1990 and 2002. The length of time window T_W (*iwl*) is selected as 3 years. The white "+" symbol marks the main shocks with $M_D \geq 5.0$. Red color (positive *Z*-value) represents a decrease in seismicity rate.

Figure 3 (continued)

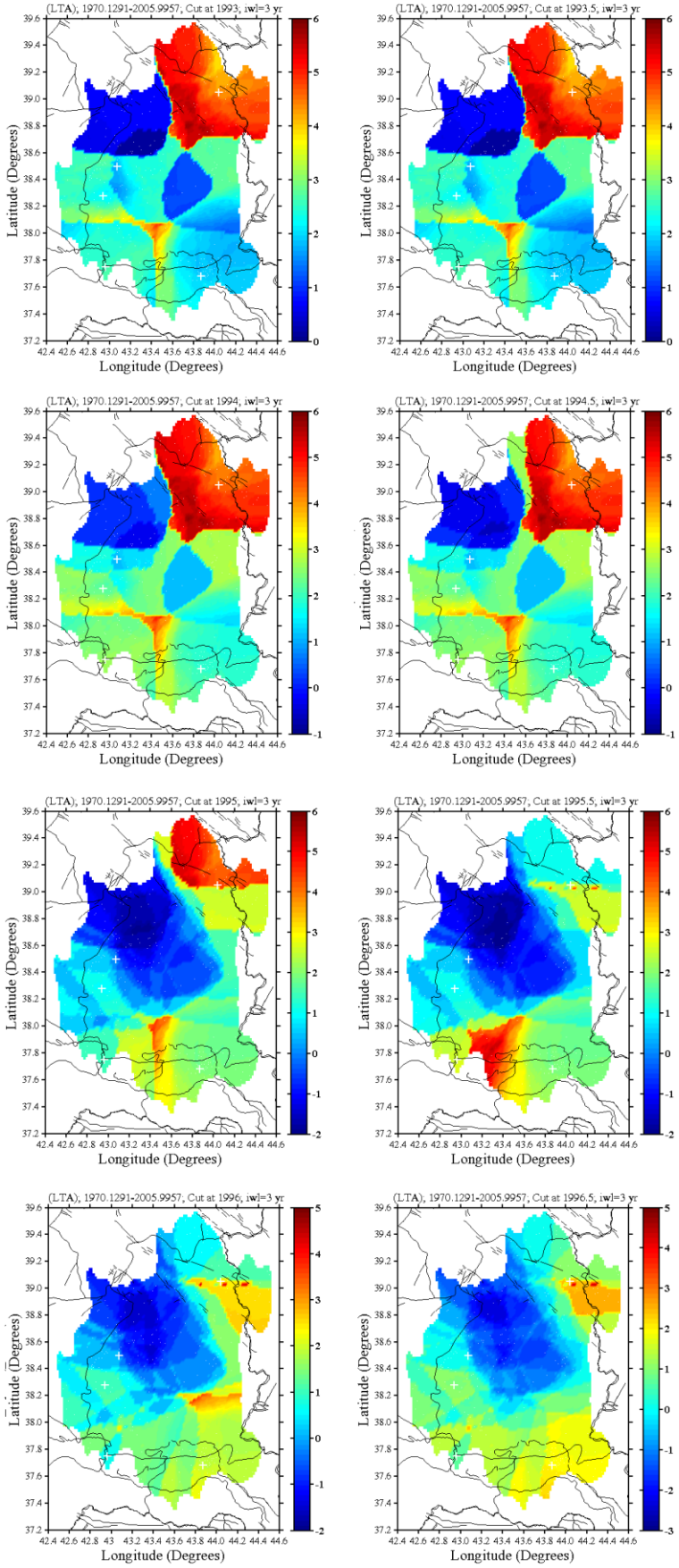
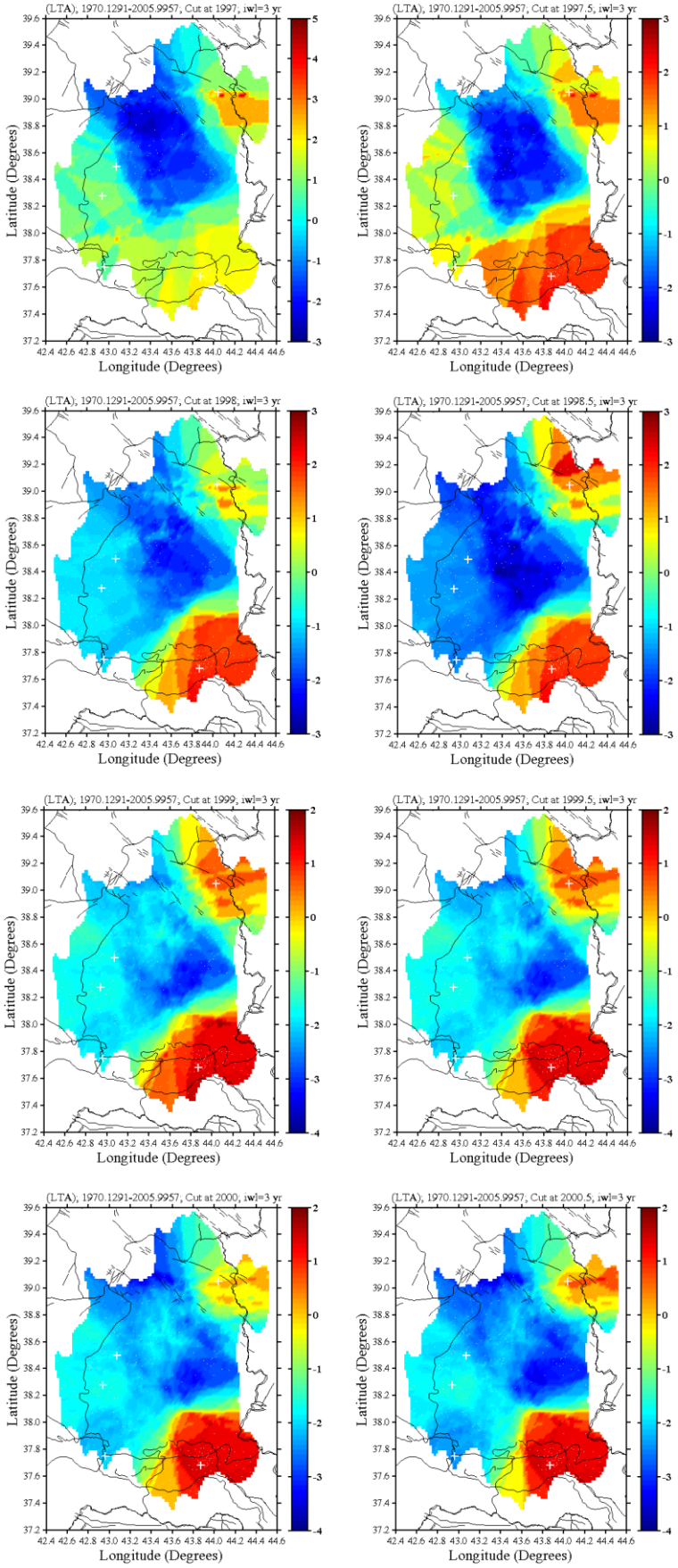


Figure 3 (continued)



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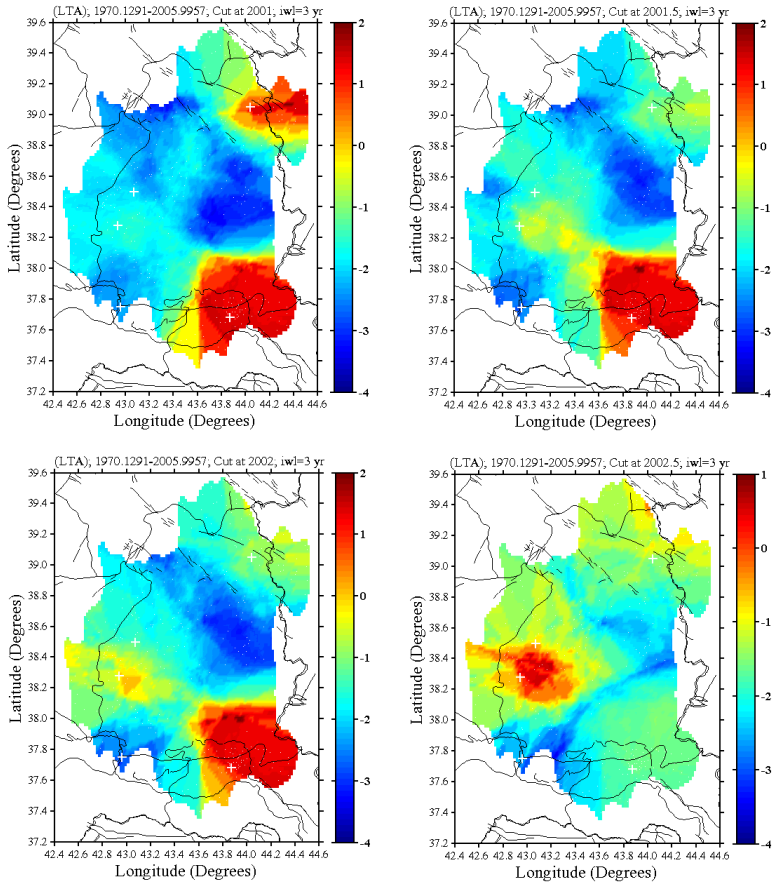

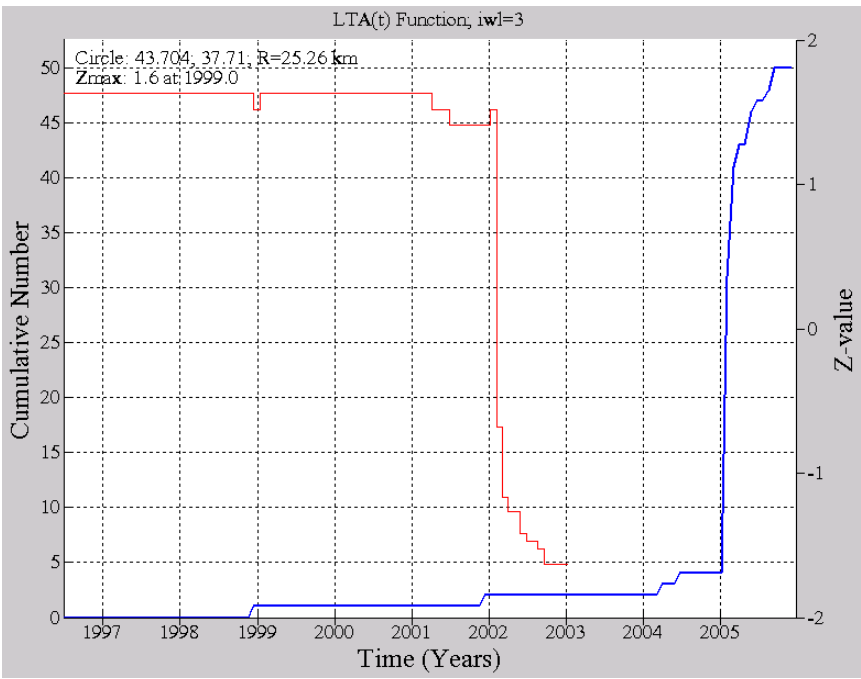

Fig. 4. Cumulative number plot for the anomaly area for 2005 Hakkari earthquake detected in Figure 3. Blue lines plots show the cumulative number and red lines *Z*-value as a function of time. Also, the center and radius of circle, Z_{max} value and the beginning time of the quiescence are given.

CONCLUSIONS

Our investigation of the spatial and temporal variations of seismicity rate changes in and around Hakkari is based on the seismic tool *ZMAP*. Based on the phenomenon of precursory seismic sequence before crustal main shocks we focused this study on the detection of the seismic quiescence situation in and around Hakkari (Turkey). For this purpose, we mapped the seismicity rate changes before the Hakkari earthquake of February 25, 2005. The comparison is made by means of the standard deviate *Z*-test generating the LTA(*t*) function for the statistical evaluation of the confidence level in units of standard deviations. We used the instrumental earthquake catalog of the Bogazici University, Kandilli Observatory and Earthquake Research Institute from February 1970 until December 2005, for 664 crustal earthquakes of magnitude equal and greater than 2.0, with depths less than 70 km. The catalogue is homogeneous and complete for duration magnitude, M_D, and the completeness magnitude is 3.2 for studied area. The number of earthquakes exceeding this magnitude range is 372. We used the Reasenberg algorithm in order to separate the dependent events from independent ones and we declustered the earthquake catalogue for the calculations. The number of events is reduced to 327 with the declustering process and 12% of the events are removed. We mapped the spatial variations of *Z*-values in the rectangular area limited by the co-ordinates 37.2ºN and 39.6ºN in latitude and by the co-ordinates 42.4ºE and 44.6ºE in longitude. We measured the significance of seismicity rate changes at the nodes of a 0.02º grid space in longitude and 0.02º in latitude. After some preliminary tests, we took the nearest earthquakes N=50 at each node and searched for rate changes by a moving time window T_W (*iwl*), stepping forward through the time series by a sampling interval. The sampling interval is selected as a time step of 28 days in order to have a continuous and dense coverage in time and N and T_W values are usually selected accordingly in order to enhance the quiescence signal and this choice does not influence the results in anyway. The total duration of the catalogue is 35.87 years. For the rate changes, it is used a moving time window T_W=3 years after a few different test values (indicated as *iwl* in the respective figures; 1.5, 2, 2.5, 3, 3.5, 4, 4.5 and 5 years) and different starting times for the foreground windows. In the final phase, geographical disrtibution of *Z*-values is mapped for every six month between 1990 and 2002 using the declustered catalogue and cumulative number of events is plotted in a circular area including the epicenter of the Hakkari earthquake. Corresponding LTA(*t*) function for a circle of 25.26 km radius centered on 2005 Hakkari main shock, a decrease of the seismicity rate is respectively found at 1999.0 with Z_{max}=1.6 level for 2005 Hakkari earthquake. Thus, 6.07 years of seismic quiescence was clearly observed before February 25, 2005 Hakkari main shock. In conclusion, the detection of seismic quiescence is the key to the success of intermediate term earthquake prediction. If such features could be recognized as a constant and reliable character of the seismicity, they could eventually contribute to the forecast of impending main shocks in future circumstances.

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EXTRACTION OF SPATIAL CHARACTERISTIC OF EARTHQUAKE USING ARTIFICIAL NEURAL NETWORK

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Key Words: velocity model of crust, inverted model, primary model, neural network, hypocenter.

ABSTRACT With due attention to human and economical damages measurement problem and locating the earthquake is of great importance. Nowadays there are different methods locating earthquake around the world. Most of the poly station ways use simple velocity model of the earth determining source parameters such as position and focal depth of the earthquakes occurred. As it is well known velocity structure of the earth is usually estimated with use of inversion of travel time data, recorded in seismological stations. In this study primary parameters of earthquakes and velocity model of crust determined in north-western of Iran, 36-40 northern degree, 44-50 westerner degree, has been used for purpose of locating earthquakes. In this area velocity structure are estimated using a 3D inversion of travel time data, recorded in the stations of the area. Then, estimated velocities are used to locate earthquakes. For locating earthquakes a neural network inversion is employed which in it number of observations are more than the number of model parameters. This advantage, called over-determined problem, put more constraint on the inversion for estimating model parameters. In this way inverse problem will be stable. The usable networks in this case are supervised Learning type and algorithm learning is back propagation error. These networks use first arrival travel times for estimating parameters. Travel time data used in the inversion are recorded in eight stations in north western part of Iran. Results of locating earthquakes by choosing a neural network including 5 and 3 stations in optimization procedure shows good results. Obtained results demonstrate that the neural network method used is an efficient method for locating earthquakes from recorded first arrival travel time data.

INTRODUCTION

One of the most important responsibilities in seismology is locating seismic source. Generally locating source needs identification of seismic phases and knowing velocity structure of earth crust between hypocenter and seismic station. Locating the exact position of occurred earthquake in each region is important for identifying potential seismology and hidden faults and accepting danger in that region can help very much. One of the suitable regions in Iran for crust modeling by considering the importance of seismology of that region is northwestern of Iran, which the existence of large fault of north Tabriz (figure 1) by historical records of destroying earthquake is of more importance. In this way the time of arriving with velocity structure (Byramnejad,2008) of crust (crust model) are considered as known parameters and earthquake source as unknown parameters of model that is optimized. With attention to non linear relation between model parameters and response model and the capability of artificial neural network in solving such problems, it seems that these networks could make available suitable choice for modeling and data processing.

THEORY and METHOD

Generally locating the source needs identifying seismic phases and measuring their arrival time and also identifying the velocity structure between hypocenter and seismic station. By having location of seismic source we can calculate the travel time for each special phase to a seismic station. By having location of seismic source we can calculate the travel time for each special phase to a seismic station at each location of an arbitrary complex velocity model. This kind of problem is called pioneer model and the arrival times is calculated based on compilation model. On the other hand finding earthquake location usually inverse the problem in which data is identified (arrival time phase), But the problem should be solved for the source location and the time of event that is in harmony with data (Thurber1981). The total inverse theory that is the most important new means for interpreting seismology and also pointing to other geographical problems is used. When there are several stations and exact locating can be done by use of S&P arrival time by its own. The Cartesian ´s characteristics a real hypocenter and the seismic station i in order are (x,y,z) and (x_i, y_i, z_i). t And t_i are in order of time of earthquake event and the first arrival time to i station. If we have n station in which the arrival time is really measured, we can consider the t_i observation as i compilation of data vector d that have n compilation and can be written $d = (t_1, t_2, t_3, \dots, t_n)$. We can consider the vector model as m that has k compilation generally k=4 are for time and local coordinates of earthquake then we have:

$$F(m) = d \qquad (1)$$

It is obvious that t_i is an element of data vector d and x, y, z is the elements of m vector model that we want to specify. Ideally only one and unique combination for compilation of hypocenter can be true in observation time. The formula is for nonlinear d that prevents us from solving formula by minimum linear square. The standard way of linearing problem and repeating m guesses is in relation to improving problem. The data vector relation by n dimensions (number of observations) to vector model by k.

Dimension relates to the number of compilation model. Generally most of the earthquake problems are over determine so that the number of observation is more than unknowns (n>k).

THE MULTILAYER PERCEPTRON NETWORK WITH INSTRUCTIONAL ALGORITHM BACK PROPAGATION (MLP)

Neural networks, are very simple model of human neural system, their learning system is like human learning system. The artificial neural system by receiving input pattern is capable to extract their Specification and relate them to a specific group. In this order between training processes they memorize patterns specification and the ideal group by matching some weight coefficient (Poulton, 2001).

If the training is done in correct way they can classify patterns. At least all the neural net works are Common in three main complaints.

1. Neuron or node or processing unit PE

2. Relational weights

3. Separated layers that contain neurons and related to each other by weights in each neuron the product dot of input vector P and the weight matrix W compose the weight sum, sum_j.

The sum weight of inputs in arrival time from an activation function f, is compared with its threshold T. If this weight sum does not go up from threshold, neuron becomes active, otherwise it remains inactive. Weight and threshold amount with attention to

problem type is calculated and changed. In such a way that the aim of problem is achieved. Usually networks have several layers, these layers are: the input layer with number of neurons equal with number of element inputs. The output layer that its output is the ultimate output of network. The hidden layers are between input and output layers. It is necessary to say that different layers can have different stimulation functions. In this research a feed forward network of supervised learning type and instructional algorithm back propagation error has been used that in every layer, each neuron is attached to the next layers neurons. In feed forward network the neurons , only receive their data from the previous layer and send their output to the next layers. In a network with supervised learning for training network the ideal output d_{pk} should be in accordance with each input pattern.

In this way learning the ideal output patter n is compared with calculated out put by network as $\mathbf{O_{pk}}$ real output. The difference between them $\mathbf{e_p}$ should be minimized by changing network weights by below relation.

It is necessary to say that the primary weights in network are by chance (Poulton et al., 1992 Poultun, 2001).

$$e_p = \frac{1}{2}\sum_{k=1}^{l}(d_{pk} - o_{pk})^2 \qquad (2)$$

The e_p Error is calculated for the p pattern of input data. In a network with back propagation error training algorithm, first of all the network input for calculating the network output is used directly. If the net work output values be different with ideal values at output layer by use of defined error, The values of weight changes $\mathbf{\Delta w_{ji}}$, for each coefficient weights calculated and the weights by returning these changes at the network direction are corrected. The quality of correction for coefficient of weight

Is as follow (Turhan, 1995):

$$\mathbf{\Delta w_{ji}} = {-\alpha \partial e_p}/{\partial w_{ji}} \qquad (3)$$

In which α is a positive and fixed quantity that is called learning rate that really the value of steps for achieving minimum error is regulated and the Suitable choice for correct action is vital learning way. The learning algorithm back propagation error is away for decreasing slope. The main weakness of this algorithm is the low speed of convergence and the stop of learning network because of trapping network in minimum local point in this way because of fixing rate learning at training time, the action of learning algorithm is sensitive to suitable choice of learning rate. since before training choice suitable learning rate is not practical, if we let the rate of learning to change between training process. The operation of gradient decrease of algorithm can spread. in faster training ways such as Levenberg& Marquart(LM) that has been resulted by changing back propagation algorithm ways, the learning rate is not fixed and between training network changes. For this reason in this research the Levenberg &Marquart algorithm has been used as a fast training way for data modeling. in designing a network its input and output parameters are not among free parameters and determined by problem(Poultun,1992).

GEOLOGY OF STUDY AREA FOR DESIGNING AND TRAINING NEURAL NETWORK

region by 44--50 geographical longitude eastern degree and 36-40 geographical latitude northern degree for this research has been chosen. A training case of two input and output vector is formed. The input vector involves produced data at stations and the output vectors constitute model parameters. The final optimized velocity model which is used for north western of IRAN is in the figure 1. The name code and geographical coordinate of stations has been brought in the table 1.

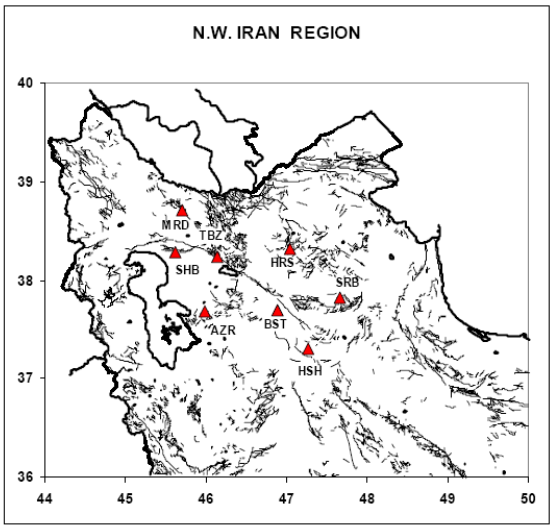

Fig.1. The under study region and the position of seismic network of eastern Azarbaijan (red triangles) by identified fault of region.

Table1. Code, name and geographical coordinate station of seismic network of Azarbaijan.

Number	St. code	St.Name	Long	Lat.	Elev.
			E	N	(m)
1	TAB	Tabriz	46.146	38.233	1650
2	HRS	Heris	47.041	38.318	2100
3	SRB	Sarab	47.666	37.825	1950
4	HSH	Hashrud	47.263	37.306	2100
5	BST	Bostanabad	46.891	37.700	2100
6	AZR	Azarshahr	45.980	37.678	2300
7	SHB	Shabestar	45.616	38.283	2150
8	MRD	Marand	45.703	38.713	2150

In this research the required training samples has been produced by Zelt(1994)velocity model.

That has three dimensional modeling capability of earth velocity. The travel time at all point of network from ray tracing eikonal equation 3D Finite-differential modeling calculations of travel time has been resulted.

$$\left(\frac{\partial t}{\partial x}\right)^2 + \left(\frac{\partial t}{\partial y}\right)^2 + \left(\frac{\partial t}{\partial z}\right)^2 = s^2(x, y, z) \qquad (4)$$

t: travel time and S(slowness) : inverse velocity .

Software calculations by soft ware package FAST that is used for tomography of first input travel time 2D and 3D. The final optimized velocity model which is given to this software for north western of country. (figure 3).

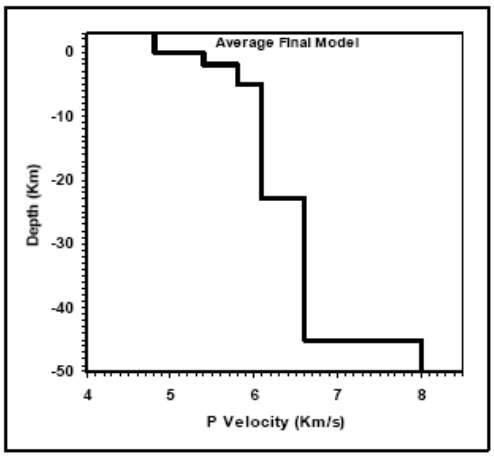

Fig. 3. Final optimized velocity model.

A big network needs a big training series for producing equation between the number of relative weights and training samples. By producing and using a lot of training samples with attention to available limited computer possibilities and lack of information from the number of internal weights before training is a difficult job. For this reason the other method is called early stopping for generalizing and acceleration training network was used.

In this way training series divide to three group: training, validating, testing (Spickak&Popova,2000). After learning the relation the classes of valid data is used. These data are used for assurance of network generality. In reality when the number of neurons in hidden layer is high, the network instead of learning input and output relation and a suitable relation does the act of protecting data in that case we will not have a suitable relation .

This condition that is known by over fitting can disappear by decrease neuron in hidden layer. In reality the number of neurons in hidden layer by trying and error and by use of valid data is suitable .

Then after assurance that there is not over fitting network put under test data.

These data are the third class of data and are used for testing the network. If the network under all the test data that are produced accidentally have acceptable error it should be possible to have certainty about correct operation of network. Then the tested network under three data classes for estimating the earthquake location in lieu of new data is prepare. For study of the network operation the RMS error was used as follow. This error on test series and in lieu of all parameters has been defined:

$$RMS = [\frac{1}{N_{test}N_{par}}\sum_{n,j} err^2\,(n,j)]^{1/2} \tag{5}$$

$$err(n,j) = [target(n,j) - neural(n,j)]/[\max(j) - \min\,(j)] \tag{6}$$

$$j = 1, \dots N_{par} \qquad n = 1, \dots, N_{test}$$

In addition to it for studying the modeling quality of data by neural network, for each parameter the jth unknown, the relative mean of error on all test sample is calculated as follow:

$$Err_j = \tag{7}$$

$$\frac{1}{N_{test}}\sum_n \frac{|target(n,j) - neural(n,j)|}{target(n,j)} \times 100\%$$

$$1, \dots, N_{par} \qquad n = 1, \dots, N_{test}$$

That in above relation:

$N_{test} = 500$: The number of testing sample. $N_{par=}\mathbf{3}$: The number of output neurons. ,max(j),min(j):The minimum and maximum values of jth parameter in training series $neural(n,j)$ the predict result of network for jth parameter in nth test sample $target(n,j)$ the goal of jth value in nth test sample(Spickak&Popova,2000).

Because the locating by data as an estimate function is depended practically the activation function nonlinear for hidden layer neuron and linear function for output layer is used in this research various function was tested and in practical form Tangant sigmoid for presenting better result was used. Since the domain of under study changes data are very large on the other hand the nature of activation functions that is used in most networks needs to put their input in a limited domain. Usually this function in small and large value is flated and in their continuations is not sensible to variation of input values. So often instead of using the data, their normalized values are used. The use of methods for placing input data in a limited domain is called rescaling. The other advantage of putting inputs in a limited domain is preventing high growth of weight. This limitation decreases the convergent time of network and gaining minimum error. The output data must be also normalized for training the network and before result analysis are returned to main units.

For normalizing the under study data many method has been tested, that using domain [-1,1] presented better result (Poulton,2001). For studying the application and selection some network parameters a smaller network by random choice of training series was produced, as response to each model, like a column vector was place in a matrix column so a matrix was resulted that is number of columns was equated by the number of its lines were equated with the number of measured element in lieu of each model. Determining the number of layers and the hidden neuron as test and error was done. To this order a three layer network for locating earthquake was designed. The most suitable designed network structure for data training after necessary surveys as a three layer network by 110-40-2 structure of figure 4 was chosen. The network back propagation error has 3 layer structures. That its output layers two neuron and its first internal layer 110 neuron and second internal layer have 40 layer. And activation function in two internal layers Tangant sigmoid and in output layer is linear. The inputs of neural network are the coordinates of five station with their arrival time the network output is

the coordinate of earthquake occurrence . For training network the LM learning law has been used.

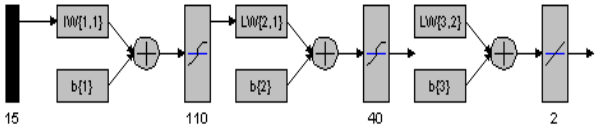

Fig. 4. The neural network used in this research for calculating epicenter earthquake by structure 110-40-2.

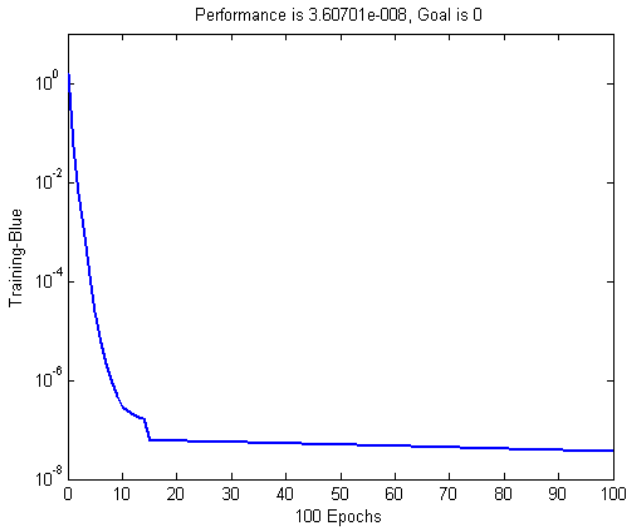

Fig. 5. The manner of curve changes, the sum of error squares for calculating epicenter earthquake by structure 110-40-2 at training time.

But the most important factor that is of due attention in the training neural network in Figure 5 is the manner of curve changes, the sum of error squares training time. In LM learning error never increases but if the curve be fixed for a long time, it means betraying network in a local minimum. In this way the performance of learning is done slowly. In this case the learning from first, by weight vectors and new biases the act of learning should be done. But if the curve during learning has a considerable decrease the learning is doing well and we can be satisfied of network action. Figure 5 the sum of error squares for training data in zelt model is shown. Just like that it is obvious in this graph. The decrease manner in the sum square of curve shows that learning in network is done correctly .In this picture the horizontal axis show the epochs number for gaining the ideal amount of error.

The curves of figures 8 and 9 show the relative percent of error about mentioned network for first and second elements epicenter of earthquake (x,y). The another network back propagation error for calculating the hypocenter coordinate(x,y,z) that have three layer structure 120-15-3, Figure 6 that its outer layer has three neuron and internal first layer 120 neuron and the second internal layer 15 neurons. Activation function in two internal layers is Tangant sigmoid and in output layer is linear.

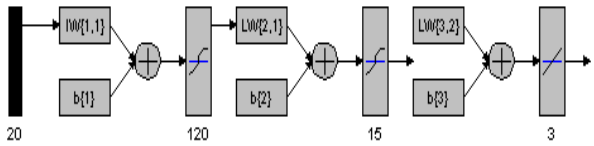

Fig. 6. The neural network used in this research for calculating hypocenter earthquake by structure 120-15-3

The neural network input the coordinates with heights of 5 station with arrival time and the outputs network are the coordinates of earthquake occurrence (epicenter and depth). For training network the LM learning has been used. The above designed network at testing stage achieved the rms=0.0041 error. The curve manner is sum of squares error curve Figure 7.

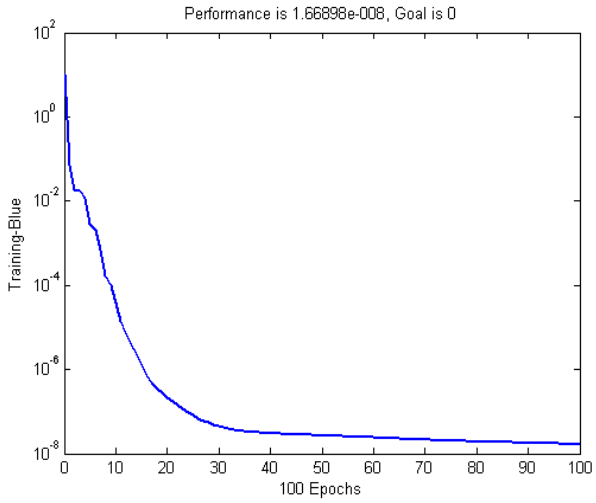

Fig. 7. The manner of curve changes, the sum of error squares for calculating hypocenter earthquake by structure 120-15-3 at training time.

The relative percent of mean value is calculated in lieu of each parameter for testing series of data has been shown in graphs, figure (10 to 12). the minimum obtained error related to first parameter and the maximum error is related to third parameter. On contrary of monotonous covering of the space related to all parameters is observed that the amount of error about various parameters is different. The reason of this matter should be searched about the manner of learning in network. the network by extracting the prominent specifications of input pattern try to use the determined training algorithm the relation between these specifications are obtained by output parameters. Each of this output parameters are a function of these specifications. Change of a function for all parameters that by means of it can at a certain time estimate 3 parameters cause the approximated function by network for estimating one parameter be more suitable from the other one or by other work the amount of network sensibility to a parameter be more than another parameter .The figure 13 shows the epicenter of testing data on the basis of the output network and ideal values , that in software surfur8 is drawing.

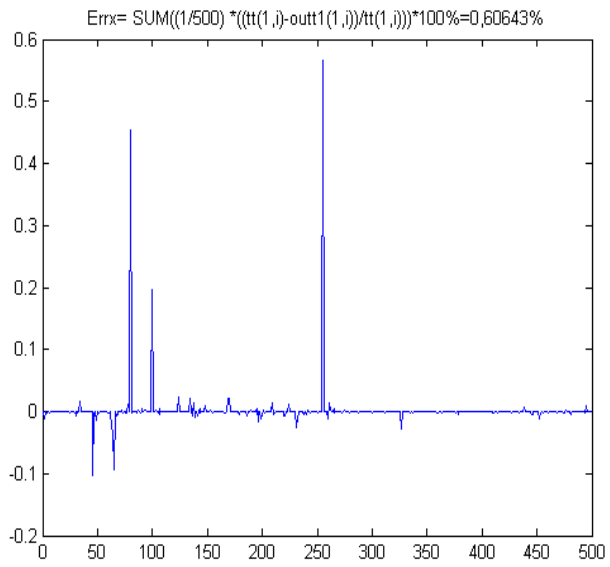

Fig. 8. The percent relative error for testing neural network used in this research, for first element model epicenter earthquake by structure 110-40-2.

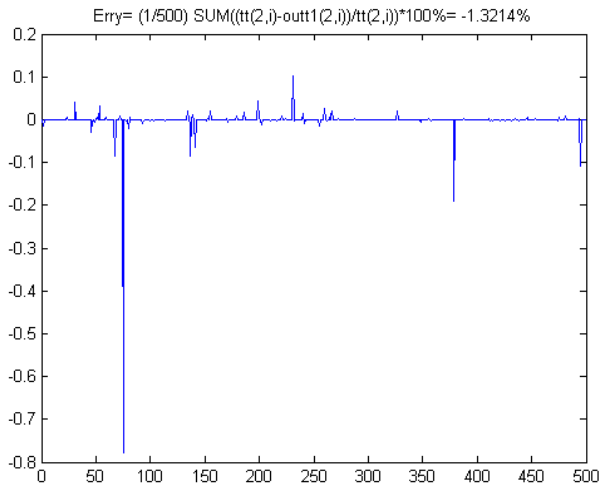

Fig. 9. The percent relative error for testing neural network used in this research, for second element model epicenter earthquake by structure 110-40-2.

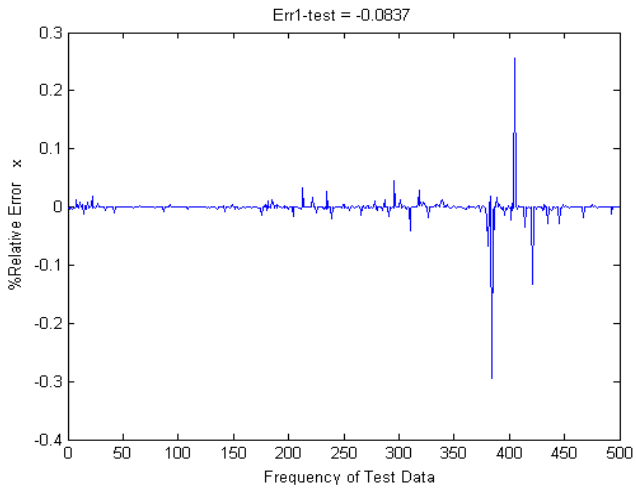

Fig. 10. The percent relative error for testing neural network used in this research, for first element model hypocenter earthquake by structure 120-15-3.

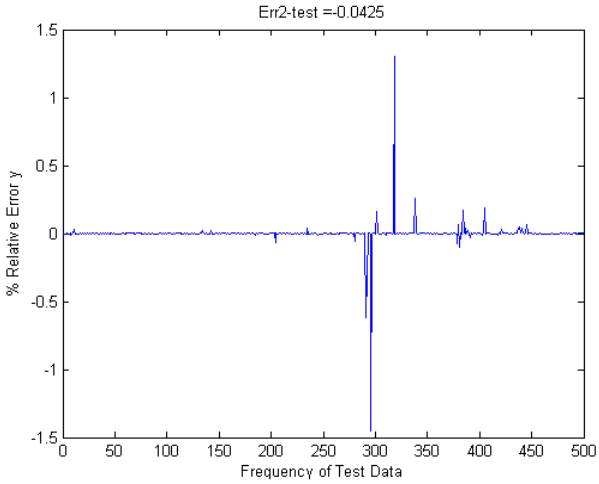

Fig. 11. The percent relative error for testing neural network used in this research, for second element model hypocenter earthquake by structure 120-15-3.

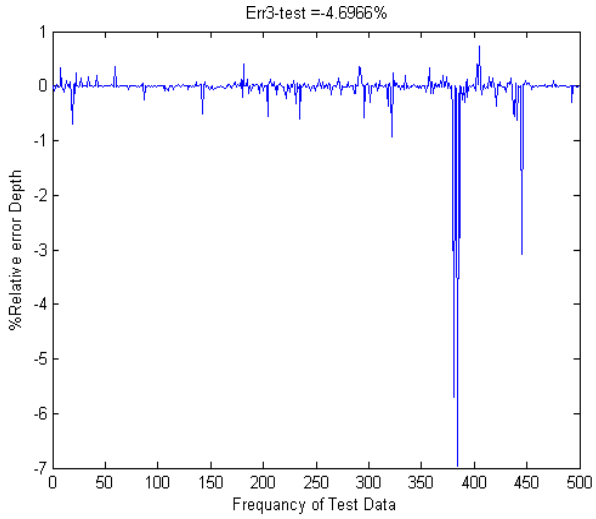

Fig. 12. The percent relative error for testing neural network used in this research, for third element model hypocenter earthquake by structure 120-15-3.

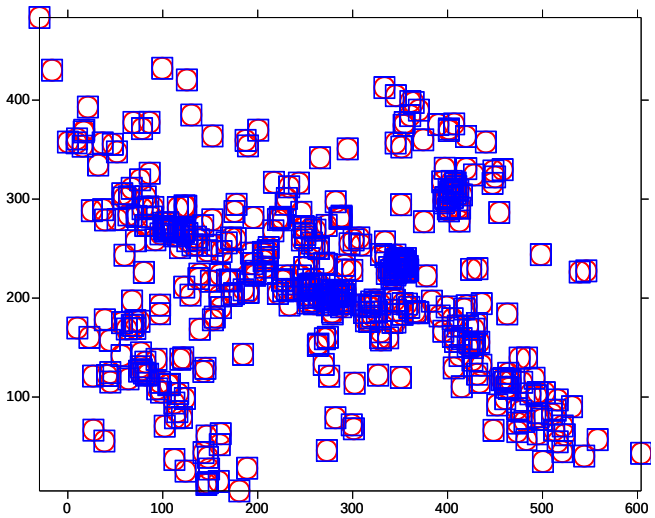

Fig. 13. The output network for epicenter of testing data (blue circle) and desired values(red circle) , that in software surfur8 is drawing.

CONCLUSION

the main advantages of using neural network in the locating that once the network in a specific form of problem is trained when training and generality of network is done well then the network could be able to estimate the output for specific input in a very short time. So the combination of more various training samples for capability of network is recommended. In this matter the aim is the possibility of locating earthquake by use of neural network. In this method with attention to the arrival time for each earthquake station is computable. The inverse method of locating, the velocity model of ZELT and the intelligent model idea for estimating the location and depth of earthquake has been used

ACKNOWLEDGMENTS

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FREQUENCY ANALYSIS OF CLAMPED LAMINATED COMPOSITE PLATES USING ARTIFICAL NEURAL NETWORK

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Key Words: Laminated plates, finite element solution, artificial neural network, fundamental frequency

ABSTRACT The general goal of this paper is the determination of the fundamental frequency of clamped laminated plates using artificial neural network (ANN). Based on the first-order shear deformation theory for laminated plates, finite element solution (FEM), capable of evaluating the first natural frequency is prepared of which the outputs are used for training and testing the developed artifical neural neural networks. The numerical analysis is carried out for 2- and 4-layered angle-ply plates. To automate the response predictions, the results are trained using a back propagation neural network. Trained network is tested with unknown set of problems and it is found that network results are in close agreements with the finite element results. Such applications of neural networks reduce considerable time.

INTRODUCTION

Laminated plates made of advanced fiber-reinforced composite materials have many excellent advantages, and have been widely used in many engineering fields such as aerospace, marine vehicles, automobile, civil, mechanical, chemical and nuclear engineering. As one of the excellent properties of the composite laminates, through proper arrangement of stacking sequence, fiber orientation, thickness and material properties of each layer, the strength and stiffness of the laminated fiber-reinforced composite lamination can be tailored to satisfying the given requirements, result in materials with almost unlimited potential of fiber-reinforced composite laminates.

Computer simulation of dynamics behaviour of composite laminates is related with advanced FEM software and quality expertise. The response of laminate plates depends on fibre orientation, laminate thicknesses and type of loading. It takes quite sometime to carry out modelling and simulation and there is a need of quick prediction of responses.

Great progress has been made over past decades towards better understanding of the vibration characteristics of laminated composite plates. Due to limited availability of analytic solutions for practical applications, numerical approximate methods have become the most effective tools. There is a vast amount of literature on the free vibration analysis of laminated plates. Kuo and Shiau (2009), studied the vibration of composite laminated plates with variable fiber spacing using finite element method. Ganapathi *et al.* (2009), investigated the free vibrations characteristics of simply supported anisotropic composite laminates are using analytical approach. The formulation was based on the first-order shear deformation theory and the shear correction factors were employed based on energy consideration that depends on the lay-up as well as material properties. Singh *et al.* (2009), studied the effects of dispersion in material properties on free vibration response of composite plates with geometric nonlinearity in von-Karman sense. The higher order shear deformation theory was employed for the study. Ferreira *et al.* (2009), presented free vibrations of laminated composite plates with a first-order shear deformation theory. Numerical results for symmetric laminated composite plates were presented and discussed for various thickness-to-length ratios. Ashour (2009), investigated the analysis of the free vibration

of thin laminated skew plates with fully clamped edges. The numerical results were obtained for different values of skew angles, fiber orientation angles and for different composites laminates. Cetkovic and Vuksanovic (2009), studied the free vibrations of laminated plates using layerwise displacement theory. The parametric effects of plate aspect ratio, side-to-thickness ratio, lamination angle schemes and degree of orthotropy on fundamental frequencies were investigated. More results can be found in the literature.

ARTIFICIAL NEURAL NETWORK

The term 'artificial neural network' comes from the intended analogy with the functioning of the human brain adopting simplified models of 'biological neural network'. The human brain consists of nearly 1011 neurons of different types. In a typical neuron, one can find nucleus with which the connections with other neurons are made through a network of fibres called dendrites. Extending out from the nucleus is the axon which transmits, by means of complex chemical process, electric potentials to the neurons, with which the axon is connected to. When signals, received by neuron, become equal or surpass their threshold values, it 'triggers' sending an electric signal of constant level and duration through axon. In this way, the message is transferred from one neuron to the other. In an artificial neural network (ANN), the neurons or the processing units may have several input paths corresponding to the dendrites. The units combine usually by a simple summation, that is, the weighted values of these paths. The weighted value is passed to the neuron, where it is modified by activation function such as sigmoid function. The modified value is directly presented to the next neuron. The connections between various neurons are strenghened or weakened according to the experiences obtained during the training. The algorithm for training the back propagation neural network can be explained in the following steps.

Step 1

Select the number of hidden layers, number of iterations, error tolerance and initialize the weights and bias functions.

Step 2

Present the normalized input–output pattern sets to the network. At each node of the network except the nodes on input layer, calculate the weighted sum of the inputs, apply sigmoid function.

Step 3

Calculate total mean error. If error is less than the permissible limit, the training process is stopped, otherwise, next step is followed.

Step 4

Change the weights and repeat the Step 2.

In Figure 1, schematic representation of an artificial neural network is given.

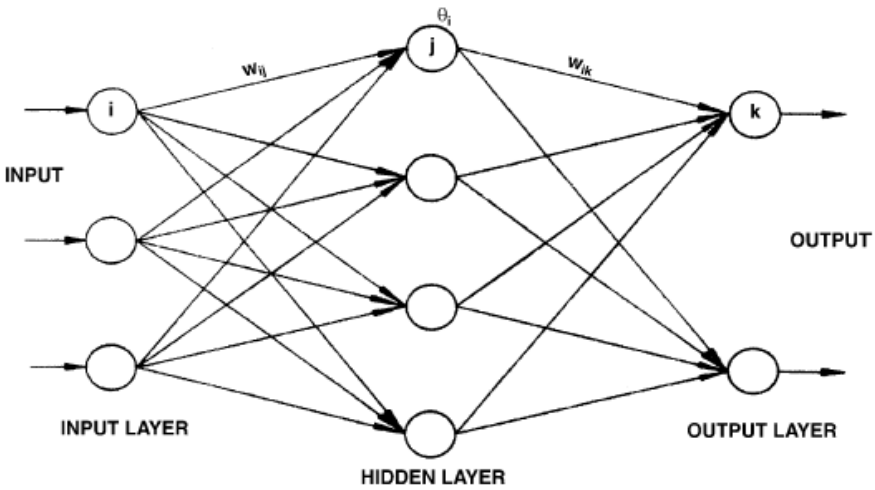

Fig. 1. Schematic representation of an artificial neural network

FINITE ELEMENT FORMULATION

Consider a fiber-reinforced laminated plate of dimension *axb*, and total thickness *h*, as shown in Figure 2. The laminated plate composed of an arbitrary number of N of orthotropic layers and the fibre orientation θ_k (k=1, 2,…., N) where is measured counterclockwise about the z-axis from the element local x-axis to the material 1-axis.

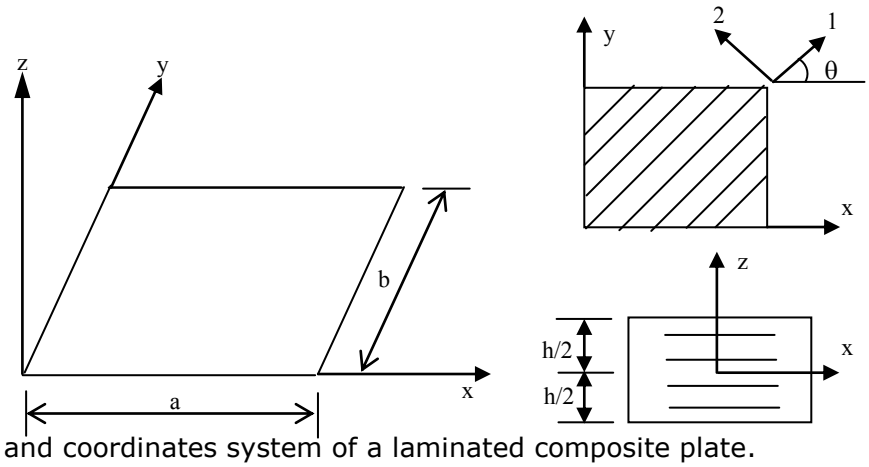

Fig. 2. Geometry and coordinates system of a laminated composite plate.

The finite element model has fives degrees of freedom per node, which are the displacements u_o, v_o, w_o and rotations ψ_x , ψ_y . Using the same shape functions associated with node i (i=1, 2,….., n) for interpolating the variables in each element, we can write

$$u_o^e = \sum_{i=1}^{N_n} u_i^e N_i^e, v_o^e = \sum_{i=1}^{N_n} v_i^e N_i^e, w_o^e = \sum_{i=1}^{N_n} w_i^e N_i^e \ , \ \psi_x^e = \sum_{i=1}^{N_n} \psi_{xi}^e N_i^e, \psi_y^e = \sum_{i=1}^{N_n} \psi_{yi}^e N_i^e \tag{1}$$

where N_i represents the element interpolation functions and N_n is the number of nodes per element. In this study, the four-node Lagrangian finite element approach is used for free vibration analysis of laminated plates.

As in the standard finite element procedure, one obtains the global eigenvalue equation is

$$([K]-\lambda[M])\{\bar{D}\} = 0 \tag{2}$$

where $[K]$ and $[M]$ are the global stiffness and mass matrices, respectively, and $\{\bar{D}\}$ is a global displacement vector. Eq. (2) is a set of homogeneous linear equations in the unknown displacements $\{\bar{D}\}$. For non-trivial solution, the determinant is equal to zero and the eigenvalues correspond to natural frequencies of the laminated plates.

NUMERICAL RESULTS AND DISCUSSION

In this study, numerical results are carried out for clamped 2- and 4- layered angle-ply square plates (b/h=20). The dimensions of the plate is a=b=1 m. The material properties are given below.

$E_1 = 25E_2$, $G_{12} = G_{13} = 0,5E_2$, $G_{23} = 0,2E_2$, $\nu_{12} = 0,25$

In this study, fibre orientations and fundamental frequencies are treated as input and output, respectively for ANN. The neural networks have one input layer, two hidden layers (sixteen neurons per hidden layer) and one output layer. All of the training and test sets are obtained from the finite element solution of the laminates. Their learning is performed using back propagation method and sigmoid function is used as activation function. The network is trained for error tolerance of 0,005. The learning rate is taken as 1. ANN is coded using Visual Basic program with a personal computer having Intel Centrino 2 dual processors and 4 GB RAM. Firstly, 90 training examples are used for 2-layered plates. In Table 1, the results of the analyzes calculated from the finite element solution are given for 2-layered plates.

Table-1. Training examples of artificial neural network for 2-layered plates.

set	$\theta_1\,(^\circ)$	$\theta_2\,(^\circ)$	ω (rad/sec)	set	$\theta_1\,(^\circ)$	$\theta_2\,(^\circ)$	ω (rad/sec)	set	$\theta_1\,(^\circ)$	$\theta_2\,(^\circ)$	ω (rad/sec)
1	50	-45	916,921	**31**	50	-5	956,171	**61**	5	-25	1071,690
2	-40	65	928,890	**32**	-50	5	956,171	**62**	-10	20	1077,800
3	-30	40	929,750	**33**	-30	25	958,864	**63**	75	-75	1079,870
4	-45	25	932,017	**34**	55	5	959,104	**64**	20	40	1082,560
5	-55	20	933,473	**35**	-75	-30	959,787	**65**	75	55	1105,440
6	40	-70	935,299	**36**	90	40	963,263	**66**	-90	65	1107,590
7	-70	25	935,809	**37**	55	10	964,701	**67**	-60	-80	1128,130
8	-60	15	938,373	**38**	55	10	964,701	**68**	50	45	1129,530
9	15	-55	939,021	**39**	30	70	966,744	**69**	35	20	1135,220
10	25	-75	939,543	**40**	40	-10	970,284	**70**	5	25	1148,000
11	10	-60	942,627	**41**	40	85	970,924	**71**	35	25	1159,640
12	10	-65	942,951	**42**	-55	-15	972,093	**72**	20	0	1163,240
13	-75	15	944,439	**43**	30	65	977,702	**73**	60	75	1163,450
14	15	-80	946,839	**44**	-80	-40	979,939	**74**	-35	-35	1166,360
15	-55	65	946,841	**45**	45	85	988,968	**75**	55	60	1171,690
16	-90	-25	947,660	**46**	50	15	991,323	**76**	30	35	1171,690
17	20	90	947,953	**47**	-70	65	992,610	**77**	-75	85	1172,740
18	90	20	947,953	**48**	60	30	994,229	**78**	10	-10	1175,960
19	-5	-70	948,463	**49**	50	90	997,840	**79**	80	65	1189,830
20	45	-75	948,466	**50**	-10	-45	1002,580	**80**	75	65	1225,820
21	80	-10	948,953	**51**	-30	10	1011,210	**81**	-15	0	1226,990
22	25	85	949,076	**52**	20	-20	1015,890	**82**	-25	-25	1238,740
23	-75	-5	949,317	**53**	30	55	1017,780	**83**	-70	-70	1278,280
24	0	60	949,430	**54**	-45	-15	1019,130	**84**	70	75	1281,240
25	20	75	949,478	**55**	-55	-90	1025,590	**85**	90	80	1292,080
26	10	-85	950,441	**56**	40	10	1034,090	**86**	-75	-75	1313,610
27	-10	90	951,024	**57**	-70	-45	1039,230	**87**	10	5	1332,980
28	60	5	952,232	**58**	-20	15	1044,060	**88**	10	10	1341,200
29	0	85	952,310	**59**	35	5	1048,900	**89**	5	5	1358,650
30	0	55	954,281	**60**	75	50	1056,940	**90**	0	0	1364,610

The computation time is about 35 min for training sets for 2-layered plates. The maximum error in the neural networks output for fundamental frequency is 0,49999973328501 %. In Figure 3, iteration history is given of ANN for 2-layered plates.

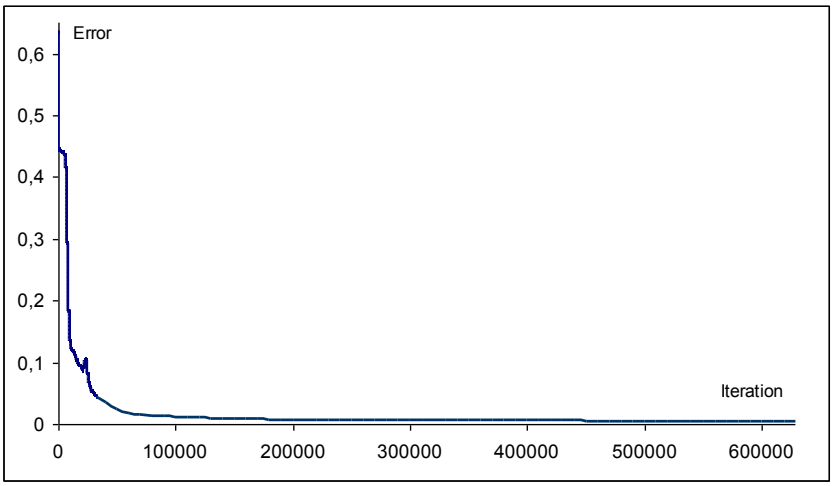

Fig. 3. Iteration history of ANN for 2-layered plates.

In order to verify the ability of the trained neural networks to reproduce the fundamental frequency of the structure accurately, 25 test points randomly are selected far from the training sets (Table 2).

Table 2. Testing examples of artificial neural network for 2-layered plates.

set	$\theta_1\ (^\circ)$	$\theta_2\ (^\circ)$	ω	set	$\theta_1\ (^\circ)$	$\theta_2\ (^\circ)$	ω
1	35	-60	923,766	**14**	35	65	997,353
2	-25	65	932,269	**15**	40	5	1014,530
3	-20	65	935,809	**16**	25	50	1025,380
4	-75	20	941,801	**17**	35	10	1075,750
5	-85	20	946,716	**18**	-5	-30	1093,230
6	-15	85	948,550	**19**	35	45	1112,010
7	15	90	949,353	**20**	40	40	1143,940
8	80	25	950,485	**21**	-70	90	1163,240
9	75	25	952,760	**22**	30	25	1206,980
10	-45	10	957,578	**23**	10	20	1255,140
11	-45	5	967,152	**24**	15	10	1311,440
12	45	0	977,429	**25**	5	0	1344,150
13	-30	15	991,060				

In Table 3, the comparisons between FEM and ANN results are given. As seen from the results, the absolute mean error and maximum absolute error are 0,364% and 1,062%, respectively. It shows, neural networks can be used in quick, approximate response predictions.

Table 3. Comparisons between FEM and ANN results for 2-layered plates.

No	FEM	ANN	%	No	FEM	ANN	%
1	923,766	922,443	0,143	**14**	997,353	993,924	0,344
2	932,269	929,712	0,274	**15**	1014,53	1017,39	-0,282
3	935,809	932,050	0,402	**16**	1025,38	1027,48	-0,205
4	941,801	942,193	-0,042	**17**	1075,75	1073,29	0,229
5	946,716	949,583	-0,303	**18**	1093,23	1097,83	-0,421
6	948,550	947,117	0,151	**19**	1112,01	1100,20	1,062
7	949,353	951,956	-0,274	**20**	1143,94	1138,84	0,446
8	950,485	940,859	1,013	**21**	1163,24	1165,17	-0,166

9	952,760	943,256	0,998	**22**	1206,98	1211,61	-0,384
10	957,578	955,378	0,230	**23**	1255,14	1264,98	-0,784
11	967,152	964,825	0,241	**24**	1311,44	1314,47	-0,232
12	977,429	980,261	-0,290	**25**	1344,15	1344,78	-0,047
13	991,060	989,587	0,149				
Absolute Mean Error (%)			**0,364**				
Max. Absolute Error (%)			**1,062**				

For 4-layered plates, 250 examples are used for training test. In Table 4, the results of the analyzes calculated from the finite element solution are given for 4-layered plates.

Table 4. Training examples of artificial neural network for 4-layered plates.

set	θ_1 (°)	θ_2	θ_3	θ_4	ω	set	θ_1	θ_2	θ_3	θ_4	ω
1	10	-20	65	80	951,542	**36**	20	75	0	-35	1075,150
2	-70	-30	45	90	1205,250	**37**	65	-70	-55	40	1164,68
3	35	55	10	-60	1004,090	**38**	90	80	70	60	1070,50
4	-85	5	-35	-40	1054,360	**39**	-25	-35	-45	-55	1009,650
5	5	85	20	35	1183,830	**40**	-85	-75	-65	-55	1058,840
6	-65	-25	0	15	909,940	**41**	35	-80	25	65	1226,110
7	90	70	40	-25	992,690	**42**	0	-65	35	15	1242,430
8	-30	65	-70	10	1287,700	**43**	-15	60	-20	-25	1235,950
9	0	35	-90	75	942,696	**44**	70	-30	80	75	1267,260
10	20	-45	45	-20	1321,290	**45**	30	35	90	70	996,584
11	-60	90	30	0	966,294	**46**	-35	55	-10	45	1183,590
12	30	-30	15	45	1182,010	**47**	60	5	5	60	1234,430
13	-15	-45	90	30	1070,930	**48**	15	35	-55	75	991,486
14	0	-90	-90	90	1114,030	**49**	-75	-55	-35	-15	897,064
15	-30	-90	0	60	1106,270	**50**	0	90	35	-65	1275,010
16	75	-90	15	60	1169,960	**51**	10	-70	-20	-5	1277,440
17	15	30	75	45	1052,980	**52**	25	45	65	85	896,959
18	-90	-75	-60	60	1248,330	**53**	90	40	15	65	1242,330
19	-15	-75	75	90	1060,790	**54**	35	55	90	0	1112,58
20	-70	0	50	-15	1226,260	**55**	-10	45	-65	35	1230,030
21	25	15	15	0	1163,950	**56**	55	55	55	55	1166,360
22	45	-80	-60	5	1093,380	**57**	40	-20	-60	15	1271,240
23	50	75	65	25	1128,780	**58**	50	25	40	-25	1057,560
24	-55	-60	-25	-85	1175,610	**59**	80	35	0	20	1017,110
25	60	90	35	5	978,516	**60**	85	-20	60	-75	1328,840
26	-35	35	-35	25	1275,810	**61**	-70	80	80	70	1089,790
27	0	-5	85	90	951,195	**62**	0	35	0	-35	1185,690
28	-85	35	20	70	1265,830	**63**	35	-50	25	0	1213,710
29	40	35	10	-65	986,802	**64**	30	75	55	5	1217,070
30	75	30	-55	5	1085,410	**65**	10	55	-5	-85	1155,060
31	-55	55	-55	55	1265,250	**66**	-80	25	15	90	1332,870
32	-10	70	15	80	1265,260	**67**	60	0	-45	10	1198,630
33	15	55	70	35	1183,080	**68**	15	90	55	5	1307,300
34	90	5	30	10	1119,370	**69**	70	70	70	70	1278,280
35	-40	-40	15	80	986,977	**70**	55	-55	15	-15	1040,420
71	-45	10	-35	5	1255,880	**131**	-45	-45	10	80	1008,000
72	35	50	5	-80	1039,040	**132**	-55	30	-85	5	1292,680
73	20	15	25	10	1295,520	**133**	-30	-65	65	-30	1199,670
74	-25	0	40	-65	1043,870	**134**	85	5	-55	20	1263,030
075	10	50	55	5	1315,570	**135**	40	15	-60	55	1102,360
76	40	-40	40	-40	1261,190	**136**	75	-80	90	-40	1081,530
77	15	-70	-35	90	1204,250	**137**	-65	0	60	60	1012,940
78	80	25	30	85	1318,970	**138**	-5	80	40	-10	1322,310

79	85	45	65	30	1112,480	**139**	-15	45	45	20	1207,040
80	75	-35	-35	-15	1040,010	**140**	35	35	-50	15	1157,620
81	-55	60	25	0	1015,720	**141**	25	-75	5	5	1216,050
82	90	-25	40	10	1041,690	**142**	55	90	10	40	1083,610
83	-10	15	-70	40	1082,240	**143**	5	-50	0	65	1192,520
84	5	5	90	-35	1073,320	**144**	0	-75	25	20	1217,810
85	35	-40	70	85	1067,870	**145**	40	10	85	55	1081,470
86	-25	40	-25	40	1270,300	**146**	-25	0	-20	-75	1099,760
87	50	-15	65	90	1107,020	**147**	60	15	90	-15	1117,070
88	45	40	55	50	1120,840	**148**	35	-5	80	70	1028,640
89	-60	50	-55	10	1181,200	**149**	-40	85	15	0	1069,370
90	0	-25	35	-85	1016,140	**150**	45	-15	80	10	1290,510
91	30	-85	10	-40	1092,750	**151**	-35	5	85	-70	1032,4
92	40	-80	-20	25	1135,720	**152**	20	20	70	80	930,341
93	90	35	20	85	1330,810	**153**	-10	35	-35	60	1159,38
94	80	-30	-15	70	1287,350	**154**	-15	0	0	15	1139,57
95	65	0	20	-40	1136,160	**155**	25	85	-5	65	1273,9
96	15	-55	80	70	1005,710	**156**	0	-55	-15	-85	1219,35
97	-20	45	60	15	1242,610	**157**	25	25	0	-35	951,83
98	90	-70	50	15	963,690	**158**	35	35	90	-55	899,147
99	75	20	-35	0	1062,800	**159**	45	-45	70	10	1258,51
100	35	-15	0	50	1208,640	**160**	-25	10	5	0	1261,13
101	25	10	70	55	1026,200	**161**	40	5	65	20	1240,72
102	-70	-25	10	65	1076,630	**162**	-75	-50	30	40	965,828
103	5	5	40	-50	1103,030	**163**	70	60	-35	-45	963,240
104	15	90	-25	85	1250,160	**164**	-85	-65	40	50	1036,500
105	0	0	60	-15	1290,370	**165**	90	-70	-45	60	1218,660
106	-50	40	-20	-20	1119,660	**166**	-10	0	-85	-10	1299,940
107	45	85	85	90	1114,320	**167**	0	25	35	85	958,536
108	90	10	-40	20	1210,360	**168**	15	-80	5	5	1280,850
109	80	80	35	10	920,795	**169**	30	30	-50	-50	923,411
110	-60	-30	-45	85	1168,680	**170**	40	15	-65	20	1201,700
111	50	-15	40	-5	1257,290	**171**	-50	-20	0	40	894,775
112	-40	20	-30	90	1164,430	**172**	-55	10	35	-85	1154,240
113	30	25	25	-40	1046,880	**173**	10	90	-80	10	1326,810
114	-10	-10	50	40	988,321	**174**	-30	-35	90	45	928,730
115	65	65	65	65	1238,740	**175**	20	50	-10	80	1121,530
116	-5	5	-5	5	1348,550	**176**	90	15	20	65	1229,960
117	70	-35	0	10	1024,750	**177**	0	-85	70	70	1050,490
118	-30	-30	40	90	975,837	**178**	-15	15	-75	-75	967,417
119	0	40	80	25	1231,100	**179**	75	-75	20	0	942,868
120	75	25	75	60	1248,560	**180**	35	-40	45	-30	1278,770
121	-35	90	90	-40	1212,30	**181**	-80	-45	15	60	1090,660
122	40	-10	25	80	1152,210	**182**	70	5	-60	90	1210,500
123	25	-25	65	65	1018,320	**183**	25	-80	0	-40	1096,430
124	45	80	-5	-5	1003,910	**184**	-30	30	75	55	972,710
125	50	-5	15	85	1206,560	**185**	5	-25	-5	70	1150,850
126	20	90	0	0	1219,960	**186**	90	-35	-70	-60	1215,370
127	-80	0	15	-35	1169,360	**187**	-10	-30	85	15	1195,240
128	15	-70	-70	25	1289,670	**188**	-5	90	90	25	1235,590
129	10	40	-55	30	1249,980	**189**	-70	30	55	55	1075,020
130	-30	65	25	-20	1218,150	**190**	85	-80	-50	-40	904,973
191	-80	35	45	15	1060,840	**221**	40	0	-65	-65	930,074
192	0	-70	70	30	1143,130	**222**	5	-65	35	30	1160,530
193	40	10	-20	65	1117,290	**223**	-55	90	70	-10	1058,720
194	-55	0	-85	-75	1139,350	**224**	0	-70	-70	85	1053,320

195	75	-5	35	90	1314,490	**225**	10	60	15	-55	1180,510
196	85	10	0	-10	1055,330	**226**	20	20	-30	15	1260,620
197	-65	-40	-80	-65	1219,580	**227**	-40	30	40	-25	1251,580
198	5	65	-60	35	1269,140	**228**	-35	-10	40	70	920,855
199	50	50	85	-35	943,939	**229**	-70	5	90	65	1126,760
200	-45	-55	15	90	1062,150	**230**	60	-20	0	25	1118,330
201	35	90	15	5	1115,320	**231**	-85	-85	60	-60	1261,770
202	-85	40	75	-10	1204,820	**232**	-10	70	-5	5	1299,760
203	55	0	-20	65	1214,350	**233**	0	20	65	40	1027,410
204	10	-25	-70	35	1155,770	**234**	65	-5	-65	55	1241,700
205	-40	65	60	-5	1178,170	**235**	70	0	-35	80	1258,920
206	0	-80	70	-25	1304,220	**236**	-75	-75	-80	20	1087,180
207	75	-45	-80	10	1213,870	**237**	15	-10	70	-55	977,920
208	-20	-20	5	40	959,638	**238**	-15	35	-60	85	991,849
209	35	75	0	0	1078,890	**239**	80	0	5	60	1235,810
210	-10	10	45	10	1249,760	**240**	-70	55	-80	-65	1250,110
211	90	-5	65	-85	1321,200	**241**	0	-30	-15	25	1266,260
212	5	35	-5	-5	1286,050	**242**	90	90	-85	30	1139,390
213	-65	-10	15	10	1021,390	**243**	-5	70	-65	-25	1196,950
214	70	-70	25	60	1178,100	**244**	-10	-40	80	-15	1288,110
215	0	20	20	0	1356,160	**245**	20	-20	-55	80	957,972
216	60	-50	0	-85	1304,770	**246**	35	90	40	15	1169,080
217	80	15	-40	10	1148,040	**247**	85	55	0	-80	1256,430
218	-60	-15	-55	-55	1175,430	**248**	-25	-25	70	-35	1170,710
219	-25	-20	-80	70	911,225	**249**	0	-80	0	25	1249,980
220	90	25	90	55	1268,740	**250**	75	0	-10	10	1131,330

The computation time is about 90 min for training sets for 4-layered plates. The maximum error in the neural networks output for fundamental frequency is 0,499999728578199 %. In Figure 4, iteration history is given of ANN for 2-layered plates.

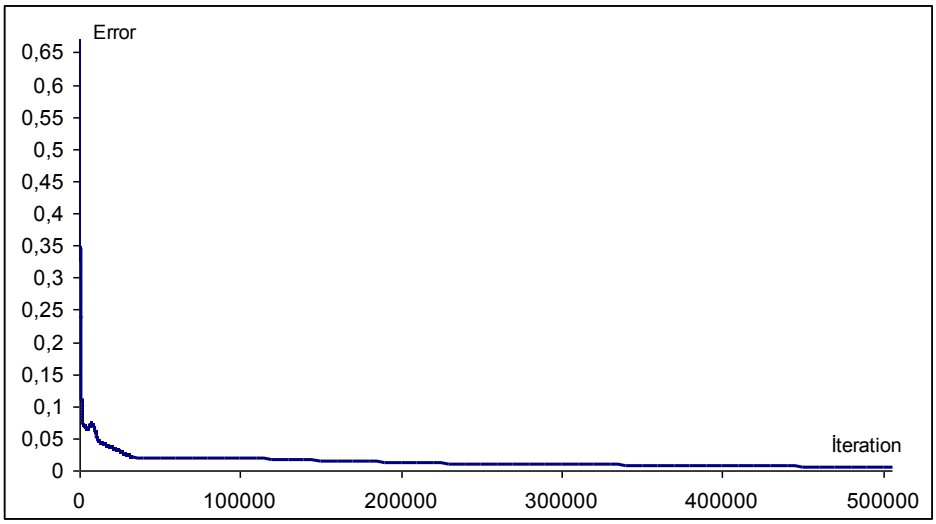

Fig. 4. Iteration history of ANN for 4-layered plates.

In order to verify the ability of the trained neural networks to reproduce the fundamental frequency of the structure accurately, 25 test points randomly are selected far from the training sets for 4-layered plates (Table 5).

Table 5. Testing examples of artificial neural network for 4-layered plates.

set	θ_1	θ_2	θ_3	θ_4	ω	set	θ_1	θ_2	θ_3	θ_4	ω
1	80	40	-5	20	1001,270	**14**	-10	70	-45	60	1247,900
2	40	-10	75	55	1090,090	**15**	-55	5	-10	-20	1162,480
3	30	-90	-60	45	1229,570	**16**	20	35	65	30	1189,460
4	-35	10	80	-65	1046,470	**17**	-20	-20	65	-60	1025,510
5	30	-5	80	55	1051,870	**18**	0	-25	0	85	1130,940
6	10	20	30	40	1045,170	**19**	90	90	35	80	1281,380
7	45	90	10	0	1020,520	**20**	20	90	-40	5	1233,760
8	-55	-65	30	10	967,040	**21**	90	-20	20	75	1251,200
9	-45	-85	85	-70	1194,560	**22**	60	-65	-65	-50	999,798
10	-55	70	-10	40	1019,300	**23**	-20	65	55	70	1080,180
11	65	-85	-10	45	1097,440	**24**	-55	65	35	-40	1153,880
12	0	0	-80	30	1148,660	**25**	-35	20	75	0	1277,170
13	20	90	-35	0	1214,400						

In Table 6, the comparisons between FEM and ANN results are given for 4-layered plates. As seen from the results, the absolute mean error and maximum absolute error are 3,046% and 9,812%, respectively. Normally, there are approximately 180^4 different combinations for 4-layered plates. But we used only 250 different combinations for training set in this study. It can be obviously said that, as the examples for training set increases, the errors between FEM and ANN results decrease.

Table 6. Comparisons between FEM and ANN results for 4-layered plates.

No	FEM	ANN	% Error	No	FEM	ANN	% Error
1	1001,270	1025,234	-2,393	**14**	1247,900	1261,6	-1,098
2	1090,090	1134,042	-4,032	**15**	1162,480	1188,483	-2,237
3	1229,570	1313,587	1,728	**16**	1189,460	1164,869	2,067
4	1046,470	1075,035	-2,730	**17**	1025,510	1041,905	-1,599
5	1051,870	1063,225	-1,080	**18**	1130,940	1111,687	1,702
6	1045,170	1107,602	-5,973	**19**	1281,380	1334,608	-4,154
7	1020,520	1019,889	0,062	**20**	1233,760	1330,169	-7,814
8	967,040	913,9994	5,485	**21**	1251,200	1265,097	-1,111
9	1194,560	1166,822	2,322	**22**	999,798	901,6984	9,812
10	1019,300	980,4959	3,807	**23**	1080,180	1080,155	0,002
11	1097,440	1117,537	-1,831	**24**	1153,880	1164,082	-0,884
12	1148,660	1163,05	-1,253	**25**	1277,170	1320,815	-3,417
13	1214,400	1306,086	-7,550				
Absolute Mean Error (%)			**3,046**				
Max. Absolute Error (%)			**9,812**				

CONCLUSIONS

The general goal of this paper is the determination of the fundamental frequency of clamped laminated plates using artificial neural network. Followings are the salient inferences of this study.

- Neural network offers an attractive route to predict structural responses.

• It helps reduce the modelling and computational time that goes into computer simulation.

• This method can be used even when FEM software is not available.

• As the examples of the training set increase, the errors between FEM and ANN results decrease.

• This paper can be investigated for other a/b and b/h ratios, number of layers, stacking sequences and boundary conditions.

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IMAGE PROCESSING OF SHARP BOUNDARIES IN 2D ELECTRICAL TOMOGRAPHY OF FAULTS

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Key Words: Image processing; Sharp boundary; Fault; Electrical tomography

ABSTRACT Automatic detection of boundaries between different geological bodies in 2D electrical tomography, using Image processing techniques is proposed here. This method is especially efficient for detection of faulting regions, which variation of resistivity values is sharp in one direction. Direction and intensity of faulting are found out by the way of an image processing method. The method is based on calculation of gradient of model resistivity data. Gradient of model resistivity values are calculated in four directions. Direction of maximum gradient vector is proposed as normal to the fault plane. Intensity of faulting is proportional to the magnitude of gradient vector. This method is applied for two synthetic models and one real data set of a faulted region in Iran. The synthetic data are representative of vertical and oblique faults. The result of applying this technique to the real data set is in well agreement with geological evidences. This method is applicable on various geophysical inversion data such as electromagnetic and seismic tomography data.

INTRODUCTION

In last two decade, electrical tomography methods increasingly have been applied in near surface geophysics. In order to acquire a proper image of subsurface, apparent resistivity data must be inverted through inverse modeling. This procedure has many problems in practice, because the inverted model is highly influenced by inversion options and initial model. Detection of sharp boundaries such as fault interfaces is even more difficult. Most of inversion algorithms have been evolved for smooth modeling. However, some inversion algorithms such as blocky inversion have been developed for tomography of sharp boundaries, but there is still a serious problem for exact detection of sharp boundaries. In traditional interpretation of sharp boundary tomography sections, delineating the exact position of boundaries is often a difficult task, depending on the colour scale and on the eye sensitivity of the interpreter. Indeed, when the final image presents smooth resistivity variations, boundaries appear as continuous transitions between zones of different resistivities. Therefore, each interpreters, has his own interpretation, greatly differs from the others.

Image processing techniques have been used in broad range of science and engineering. Recently, these techniques have been used in geophysics and help the interpreter to have objective interpretation. For example Nguyen et al. (2005) has utilized image processing technique to detect faults in 2D resistivity data. Morozov and Smithson (1996) have used an image processing algorithm called histogram equalization technique on seismic signals to compare and plot coherency measures.

In this study maximum normalized directional gradient (MNDG) algorithm was used to detect faults as sharp boundaries in electrical tomography images. This method was applied on two synthetic data and one real data.

METHODOLOGY

MNDG algorithm must apply on electrical tomography images. In this technique we consider a 3×3 block with central cell and its eight nearest neighborhood cells (Fig. 1). Then normalized gradient of resistivity is computed in four directions (Fig. 1). Intensity and direction of the maximum of four computed normalized gradient ($G_{ij}.\alpha_{ij}$), are put in the central cell. The maximum normalized gradient intensity, G_{ij} is computed as follows,

$$G_{ij} = \max\left\{ \frac{\left|\rho_{i-1,j-1} - \rho_{i+1,j+1}\right|}{\rho_{i,j} \times d[(i-1,j-1),(i+1,j+1)]}, \frac{\left|\rho_{i-1,j} - \rho_{i+1,j}\right|}{\rho_{i,j} \times d[(i-1,j),(i+1,j)]}, \right.$$

$$\left. \frac{\left|\rho_{i+1,j-1} - \rho_{i-1,j+1}\right|}{\rho_{i,j} \times d[(i+1,j-1),(i-1,j+1)]}, \frac{\left|\rho_{i,j-1} - \rho_{i,j+1}\right|}{\rho_{i,j} \times d[(i,j-1),(i,j+1)]} \right\} \quad (1)$$

where ρ is the resistivity value in Ohm.m, *ij* the indices of central cell and *d* distance between two cells. Which component that has maximum value, identify the α_{ij} that is perpendicular to the direction of maximum normalized gradient. This procedure is repeated for all cells of image, except the cells placed in the edge of image. Finally the technique is leading to two different images, first maximum gradient intensity image that is regarded as a contour map (further referred as a gradient intensity image) and second one consists of segments. The length of each segment is proportional to the maximum gradient intensity also its direction (α_{ij}) is perpendicular to the direction of maximum gradient (further referred as a gradient direction image).

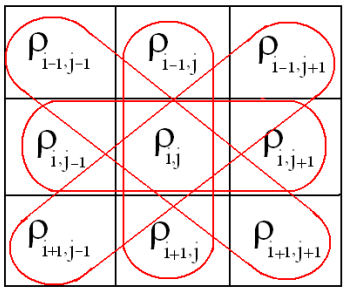

Fig. 1. MNDG algorithm is applied on each 3×3 block of cells in resistivity tomography image.

TEST ON SYNTHETIC MODELS

In this study two synthetic models were considered. These models were prepared with Res2mod electrical tomography software. The first model simulate vertical fault, and the second one simulate dipping fault. For both modeling, 51 electrodes with 20 m electrode spacing were considered on 1000 m total electrode spread. Resistivity value of one side of faulted zone is 20 Ω.m, and the other side has resistivity value of 300 Ω.m. These values were considered to make adequate resistivity contrast in faulted zone while the models conform to real earth. In both models, faulted zone is buried by thin 100 Ω.m superficial layer. To resemble simulation results to real data, also 20 percent noise was added to synthetic data. Model response was calculated for both models with pole-dipole electrode array. In the next step these model responses were inverted by Res2inv inversion electrical tomography software.

Finally the gradient intensity and gradient direction images were earned from inverted sections. For the vertical fault model, the gradient intensity and gradient direction images (Figs 2c and d) show the vertical boundary of fault precisely. Several segments in the left down of gradient direction image show weak layers boundary that isn't completely

horizontal. This is because inversion algorithm recognize horizontal layers boundary very smoothly (Fig. 2b). Horizontal boundary between superficial layer and lower layer is specified especially for the left side of images that has higher resistivity contrast. Several minor segments appear especially in gradient direction image. They can be discarded as they are irregular and often exhibit weaker gradient values.

Fig. 3 shows results of dipping fault modeling. As you can see in gradient intensity and gradient direction images (Figs 3c and d) dipping fault also is recognized precisely with adequate intensity contrast in the middle of images.

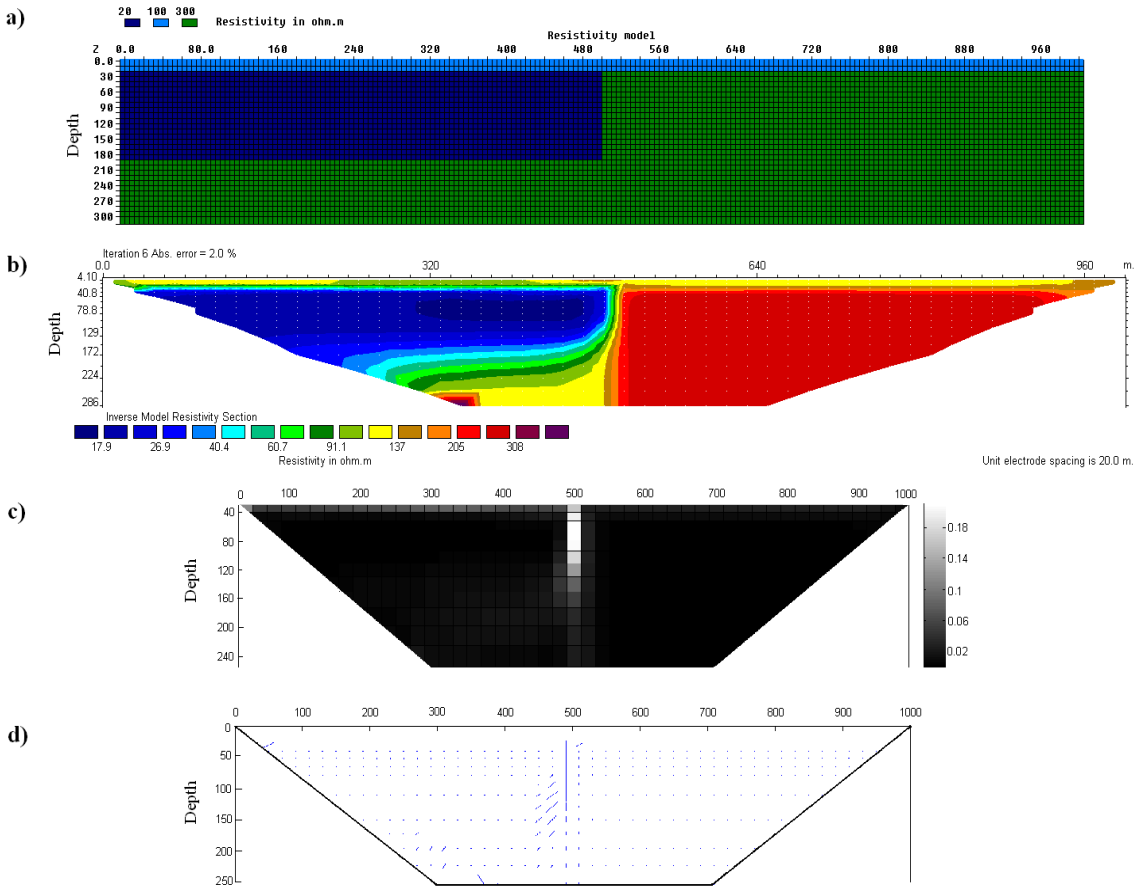

Fig. 2. Applying MNDG algorithm on vertical fault synthetic model. a) Synthetic model, b) Inverted model, c) Gradient intensity image, d) Gradient direction image.

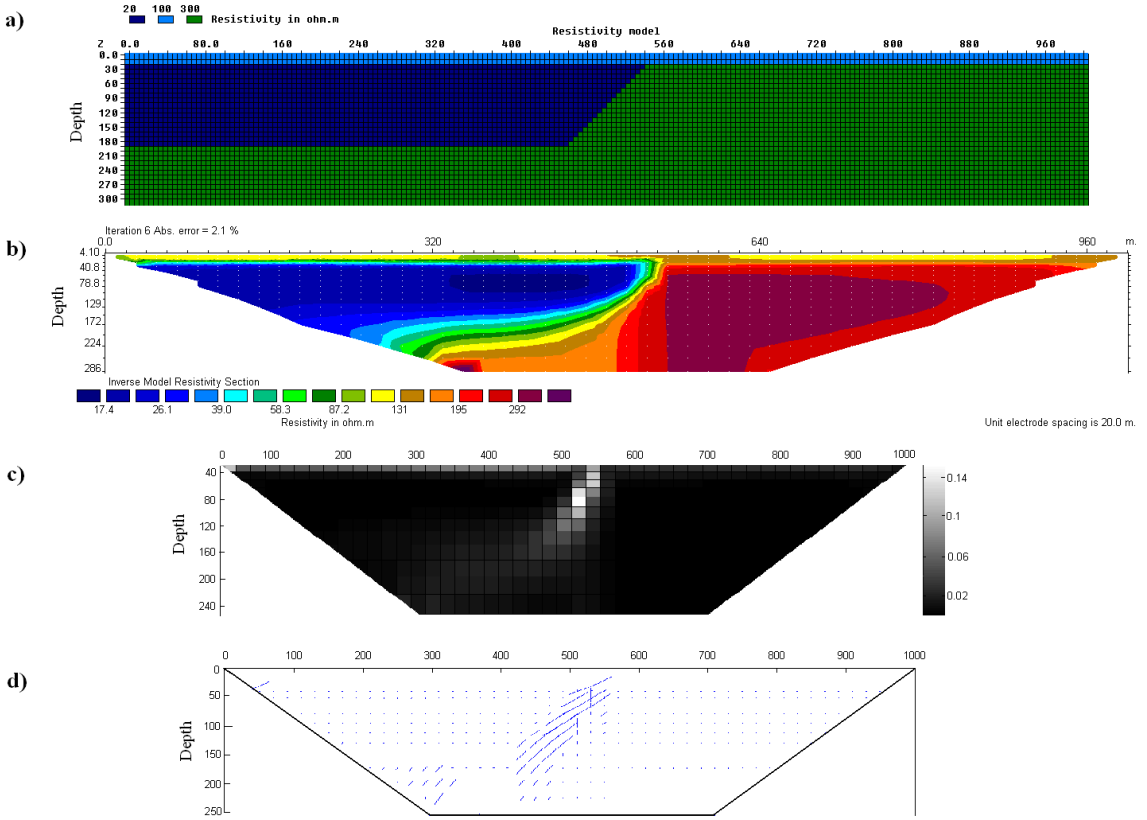

Fig. 3. Applying MNDG algorithm on dipping fault synthetic model, a) Synthetic model, b) Inverted model, c) Gradient intensity image, d) Gradient direction image.

TEST ON REAL DATA

We applied the algorithm on real data set from a copper deposit in Iran. The investigated area is located in a tectonically active zone with several fracture systems and mineral veins. Electrical tomography survey was accomplished by pole-dipole array for detecting copper vines in fractured zones in this area. For testing the validity of algorithm in detecting faulted zones, an image of 2D electrical tomography was used. Fig. 4a shows the result of inversion after 4 iterations. The result of tomography shows a sharp contrast resistivity in the left side of section and a resistivity boundary with low contrast in the right side of section. The aim of this method is precise detecting the probable fracture zones in all parts of section especially the zones that is not recognized easily from the tomography image. Fig. 4b is gradient intensity image of tomography section. Two fractured zone is detectable in this image:

1. A high gradient zone in the left of image that conforms with the high resistivity contrast in the left side of resistivity section.

2. A vertical lower gradient anomaly in the right.

The other gradient anomalies are due to surface fluctuations and presence of noise in data. Gradient direction image (Fig. 4c) shows direction of fractures clearly in two high gradient zones of Fig. 4b.

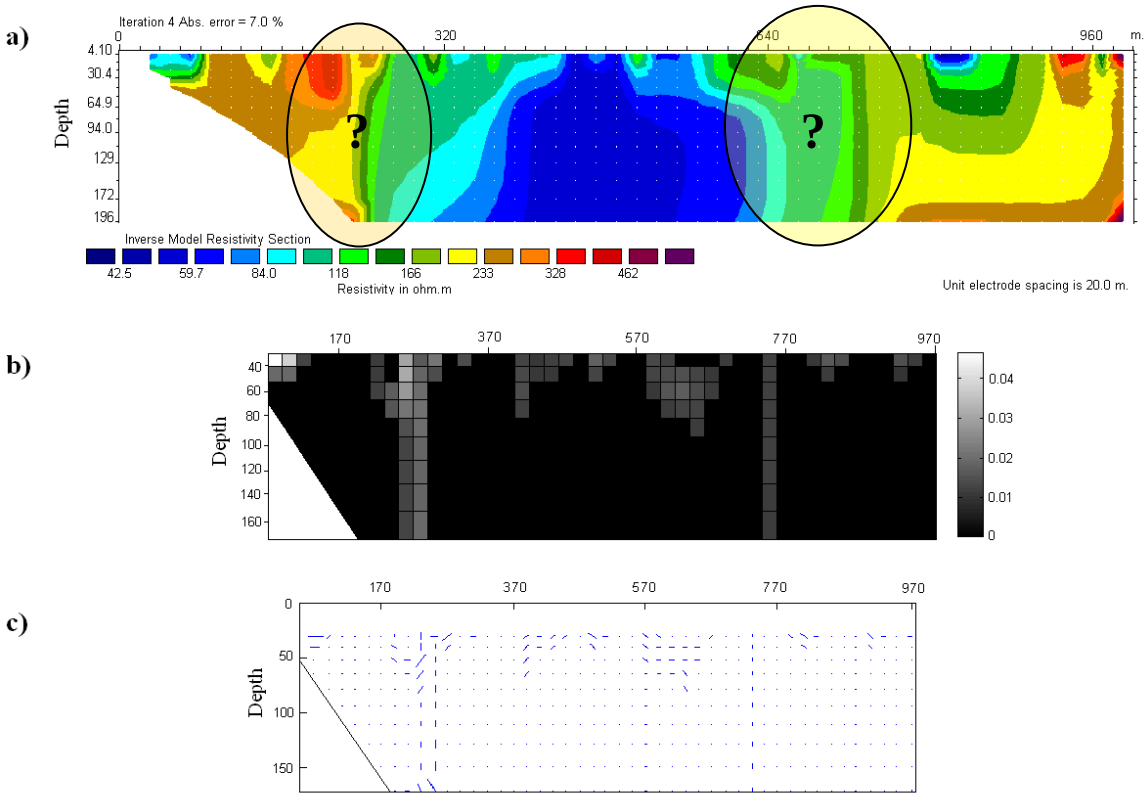

Fig. 4. The results of testing MNDG algorithm on real data set, a) Inverted section, b) Gradient intensity image, c) Gradient direction image.

CONCLUSION

The results of this study prove MNDG algorithm can recognize sharp boundaries that are not identifiable in electrical tomography sections precisely. Major advantage of this algorithm is capability of specifying the direction of faulting. Because of this advantage, this algorithm is strongly proposed for detecting faulted zones.

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MICROZONATION STUDIES BY USING SHEAR WAVE VELOCITY: GEBZE (KOCAELİ) REGION

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Key Words: Microzonation, Soil Amplification, Shear Wave.

ABSTRACT Local soil condition is the most important factor on earthquake damage and effects. Thick but smooth sediment layers comparing to the hard bedrock can be the main cause of the increasing of earthquake damage which resulted in some lands affected significantly. A simple proof might be that Avcılar is 100 km far away from the zone of the earthquake center which occurred in 1999, as its affection is felt mostly there. The knowledge of site effect is important to build earthquake- resistant structures. This study focused on investigations of soil amplification effects in Gebze during a possible earthquake. In the field, soil amplification related to the shear wave velocity, which estimated by 45 seismic refraction experiments, has been assesed by empirical approach which was introduced by Midorikawa. Microzonation, soil amplification which defined to the shear wave velocity and dominant period maps of study area conducted by using GIS (Geographic Information Systems) and were interpreted.

INTRODUCTION

Microzonation maps gives quite greater support to the planning in making decisions in use of the area and becomes the first and the most important step in reducing the earthquake damages. This step aims at identifying the sites which will be able to be affected in the different rate from the earthquake effects. In this study, soil amplifications analysis will be evaluated in microzonation studies.

THEORY(AND METHODS)

Seismic Microzonation studies are very useful for the evaluatıon of shear wave velocities on soils (ISSMFE, 1993). Shima. (1978) has proposed to be proportional the amplication factor and bedrock seismic velocities. Moreover Midorikawa (1987) emphasized the significant impact of the efficiency of shear wave velocities on the soil amplications. Midorikawa (1987) has also proposed a corelation between avarege shear wave velocities and amplification factor.

$$V < 1100 \text{ m/sn} \longrightarrow A = 68V^{-0.6}$$

$$V > 1100 \text{ m/sn} \longrightarrow A=1.0 \qquad (1)$$

A: Soil amplifications

V: Shear wave velocities (m/sn)

Location map of study area have been shown Figure1.

Fig. 1. Location map of study area.

GEOLOGY OF STUDY AREA

Dolayoba formation: Sandy limestones grade into coralline limestones and bituminous alternating limestones and marls of silurian age.

Kurtköy formation: The lowermost unit lying above the metamorphic basement association is a red arkosic conglomerate, sandstone, and mudstone unit Early Ordovician age.

Tuzla formation: This formation consist of bluish gray limestone, brown shale, pinkish variegated shales and nodular limestones.

Trakya formation: Formation in general consists of thin-layer and parallel laminated shale

Belgrade formation: Belgrad Formation of sand, gravel and silt, and Quaternary aged sediments.

Shear wave velocity, map dominant period map and soil amplification map of study area respectively have been shown Figure 2, Figure 3, Figure 4.

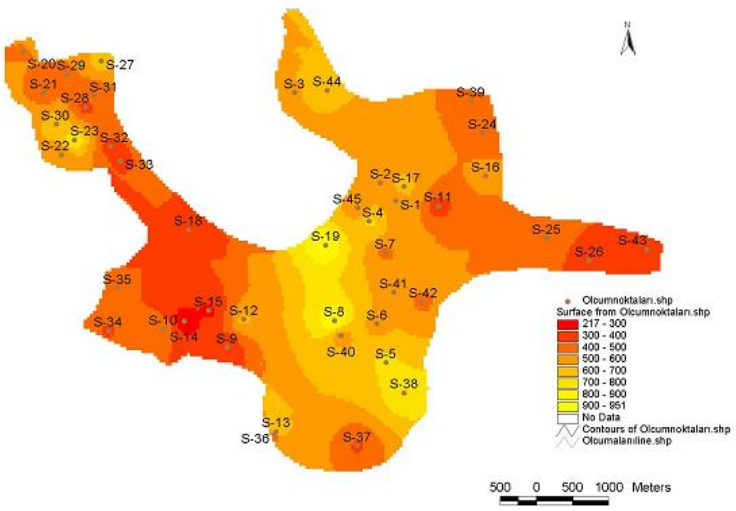

Fig. 2. Shear wave velocity map.

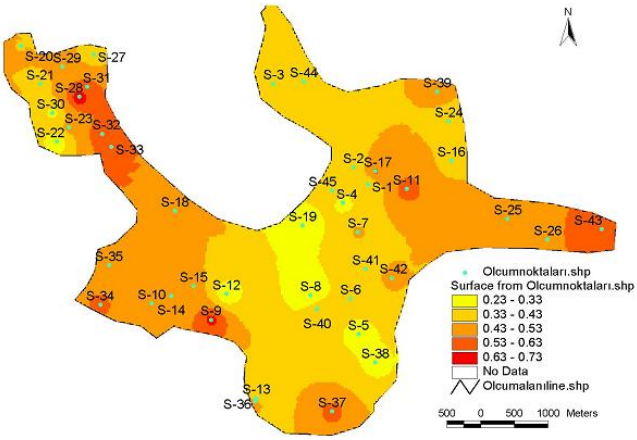

Fig. 3. Dominant period map.

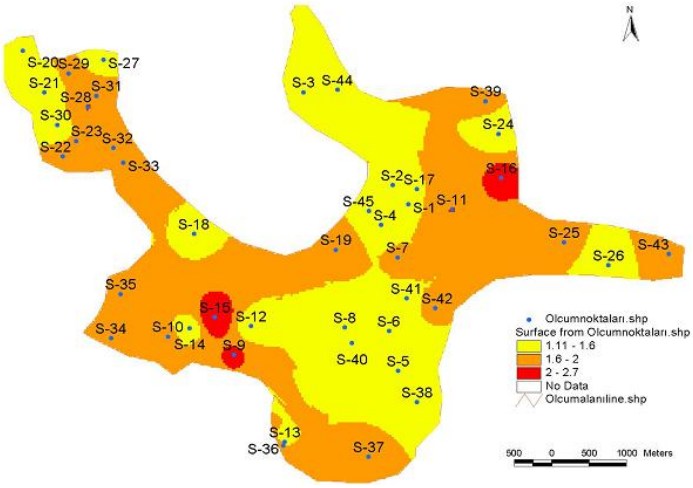

Fig. 4. Soil amplification map.

CONCLUSION

Turkey has met much more deaths and very important economical loss at outcome of earthquake because of geographical location. Determination of the behavior of soil and the site effect during strong ground motion has an important role in engineer studies. In this study,two important parameters for the microzonation studies were determined, they are dominant period and soil amplification. Soil amplification values estimated by empirical relationships by shear wave velocities are in range between 1,1 and 2,7 values. Shear wave velocity values are 217 and 951 (m/s) values. Site Characteristic period range are between 0,23 and 0,73 s.

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ON MONITORING OF NEAR SPACE PLASMA PARAMETERS FOR PURPOSE OF EARTHQUAKES PREDICTION

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ABSTRACT Analysis of data on earthquakes with representative magnitude 4.5 from the global NEIC catalogue for 1973-2009 shows: 1) seismic areas along the southern part of the Mid - Atlantic Ridge are magnetically conjugate to seismic areas along its northern part; seismic areas along the southern boundary of the Pacific plate are in rather cloze magnetic conjugancy with seismic areas along its northern boundary; and seismic areas along the southern boundary of the Nazca plate are in very close magnetic conjugancy with seismic areas along the northern boundaries of the Caribbean and Cocos plates. 2) Both, the seismic regime at the globe, and the regime of seismotectonic deformation of earth's crust with increased conductivity, respond to 11 year variation of solar activity and/or intensity of cosmic rays flux. Obtained results support a modern idea that earthquake might be an element of global electric circuit. In a such case, for purpose of earthquake prediction, one needs the data of regular monitoring of electromagnetic parameters along the whole magnetospheric tube loaned onto the area of earthquake preparation.

INTRODUCTION

In last decades, a lot of evidences on the electromagnetic effects associated with earthquakes have been observed. The ground-based ionosounder data (e.g. Ondoh and Hayakawa, 1999; Trigunait et al., 2004; Pulinets and Boyarchuk, 2004; Pulinets et al., 2007), topside soundings by satellites (e.g. Sarkar et al., 2007, Horie et al., 2007a; Ondoh, 2008) and GPS total electron content measurements (e.g. Calais and Minster, 1995) demonstrated the presence of ionospheric perturbations prior to earthquakes. Electromagnetic emissions (e.g. Gokhberg et al., 1982; Parrot, 1994; Uyeda et al., 2000; Horie et al., 2007b; Ouzounov and Freund, 2004) and high-energy particle precipitation from radiation belt (Galper et al., 1989, 1995; Aleksandrin et al., 2003), and lightning (Finkelstein and Powel, 1970) prior the earthquakes have been observed. Electromagnetic effects associated with earthquakes have been registered not only over the area of earthquake preparation, but in magnetically conjugate area as well. The conjugate effect was observed simultaneously in different parameters: the extra-low frequency (ELF), very-low frequency (VLF) electromagnetic emissions and particles precipitation (e.g. Ruzhin et. al., 1998 and references in herein), in electron concentration variations and formation of the large scale ionospheric irregularity [Pulinets and Bouarchuk, 2004]. To explain the occurrence of electromagnetic effects associated with earthquakes, the model of lithosphere-atmosphere-ionosphere-magnetosphere coupling has been developed (Hayakawa and Molchanov, 2002; Pulinets and Boyarchuk, 2004; Trigunait et al., 2004). The model suggests that the generation mechanism of these effects is a modification of the electric field in the global electric circuit (GEC) due to earthquake preparation (Pulinets, 2009; and references in herein). The classical concept of GEC was firstly suggested by Wilson (1920). It presents a system of stationary electric currents between the ground and ionosphere driven by global thunderstorm activity (Rycroft et al., 2000]. Some years ago, it was recognized that the study of GEC provides a platform for understanding the mechanism of solar-terrestrial relationship (e.g. Harrison, 2004; Rycroft et al., 2000; Makarova et al., 1998). As a result, a modified concept of GEC is now under consideration (Makarova and Shirochkov, 1998). In modified configuration of GEC [1], its external boundary is placed on the magnetopause and includes the electro-motive force generator driven by the solar wind,

while its internal boundary is placed into the solid earth and includes the generator driven by the geodynamical processes. The electric processes coupling different parts of GEC operate at (almost) the speed of light, and electrostatic and/or electromagnetic field changes in one part of GEC will produce simultaneous changing in all another parts. In modified configuration of circuit, the conductive geomagnetic field lines (magnetospheric tubes) may play a role of current wires. If this works, then one may expect: 1) electromagnetic processes occurring in the area of earthquake preparation will modify earth's environment along the whole magnetospheric tube including the magnetically conjugate area in the lithosphere of the opposite hemisphere, as a result, areas of high seismic activity may be magnetically conjugate; 2) since the magnetopause that is an external part of GEC responds strongly to variations of solar activity, the seismic regime of the Earth also may respond to solar variations. In our report, the results obtained on these subjects are presented.

RESULTS AND DISCUSSION

MAGNETICALLY CONJUGATE BOUNDARIES OF LITHOSPHERIC PLATES

In figure 1, a schematic picture of a magnetospheric tube is copied from [2]. Here, 1- is the geomagnetic force line, 2 – the trajectory of charged particle trapped into the radiation belt, and 3 – the lower boundary of the radiation belt for quiet conditions. As was discussed in above, in agreement with a concept of global electric circuit (GEC) one may expect that electrostatic and/or electromagnetic field changes in the area of earthquake preparation in one hemisphere have to modify the electromagnetic situation in magnetically conjugate area of lithosphere in the opposite hemisphere. Additionally, electrostatic and/or electromagnetic field changes in the upper part of the GEC (magnetopause, radiation belt, ionosphere) have tol produce simultaneous electrostatic and/or electromagnetic field changes in both magnetically conjugate areas of lithosphere onto which this magnetospheric tube is loaned. As a result, one may suggest that the areas of increased seismic activity could be magnetically conjugate. Khachikjan and Petelina (2008) checked up this suggestion with using the online programme [3]. Figure 2 presents the global map of epicenters from the NEIC catalogue. Solid lines in this figure join the magnetically conjugate points in corrected geomagnetic coordinates estimated with help of [3] for epoch 2001. Digits in boxes indicate the magnetic L - shell (McIlwain) parameter.

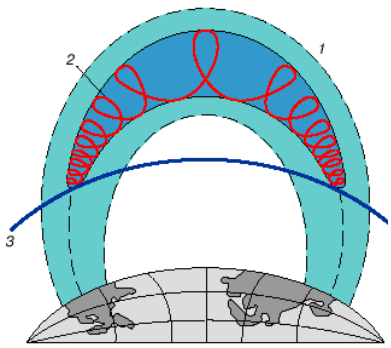

Fig. 1. Schematic picture of a magnetospheric tube copied from [2], which is loaned onto magnetically conjugate areas of southern and northern hemispheres. Here, 1-is geomagnetic force line, 2- trajectory of trapped energetic particle, 3- lower boundary of radiation belt for non disturbed conditions

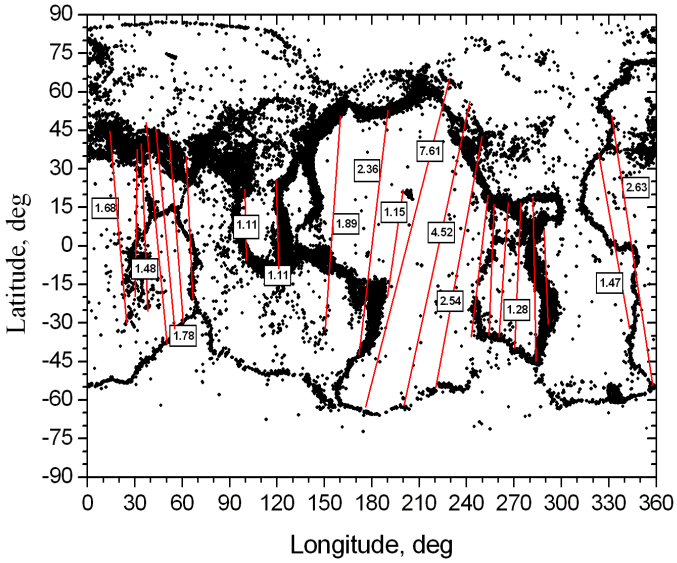

Fig. 2. Global map of earthquake epicentres from the NEIC catalogue, where red lines connect magnetically conjugate points at earth's surface in corrected geomagnetic coordinates, and digits indicate the corresponding McIlwain (L-shell) parameter (Khachikjan and Petelina, 2008).

At present, approach [3] allows estimate magnetically conjugate points only for areas located between geographical latitudes 20 - 60 degrees in both hemispheres, more over, there are some uncertainties in the results of such estimations. For example, [3] uses geomagnetic dipole which poles are located at 74.17°S, 126.26°E and 81.767°N, 277.34°E (epoch 2001), while according to the IGRF model for epoch 2001, geomagnetic poles at the earth's surface are located at 64.7°S, 138.6°E and 79.0°N, 254.9°E. Nevertheless, in spite on expected uncertainties, one may notice in figure 2 that boundaries of some lithospheric plates traced with earthquake epicentres are indeed magnetically conjugate. In particular, the southern part of the Mid - Atlantic Ridge shows magnetic conjugancy to its northern part; the southern boundary of the Pacific plate shows rather close magnetic conjugancy to its northern boundary; the southern boundary of the Nazca plate is very cloze magnetically conjugate to the northern boundaries of the Caribbean and Cocos plates. It may be mentioned that the area of Kocaeli – 1999 epicenter (40.75N, 29.86E) is magnetically conjugate to an area of increased seismic activity in Africa (25.03S, 34.67E).

SEISMIC REGIME AND SEISMOTECTONIC DEFORMATION OF EARTH'S CRUST RESPOND TO 11 YEAR VARIATION OF SOLAR ACTIVITY AND/OR FLUX OF COSMIC RAYS

One of the problems with operating of the global electric circuit (GEC) is non conductive troposphere and stratosphere. It is believed at present that a source of ions in the near ground troposphere is air ionization produced due to release of radioactive gases from the soil, while the principal source of atmospheric ions away from the boundary layer is galactic cosmic rays GCR (Rycroft et al., 2000). The flux of GCR has 11 year variation, which is opposite to the 11 year variation of solar activity. If indeed an earthquake may be considered as an element of GEC, then one may suggest that the number of earthquake will be increased in solar minimum, when the intensity of GCR flux is being largest. We checked up this suggestion with using the data on representative earthquakes (M≥4.5) from the global NEIC catalogue for 1973-2009 years (166528 events). The results are presented in figure 3.

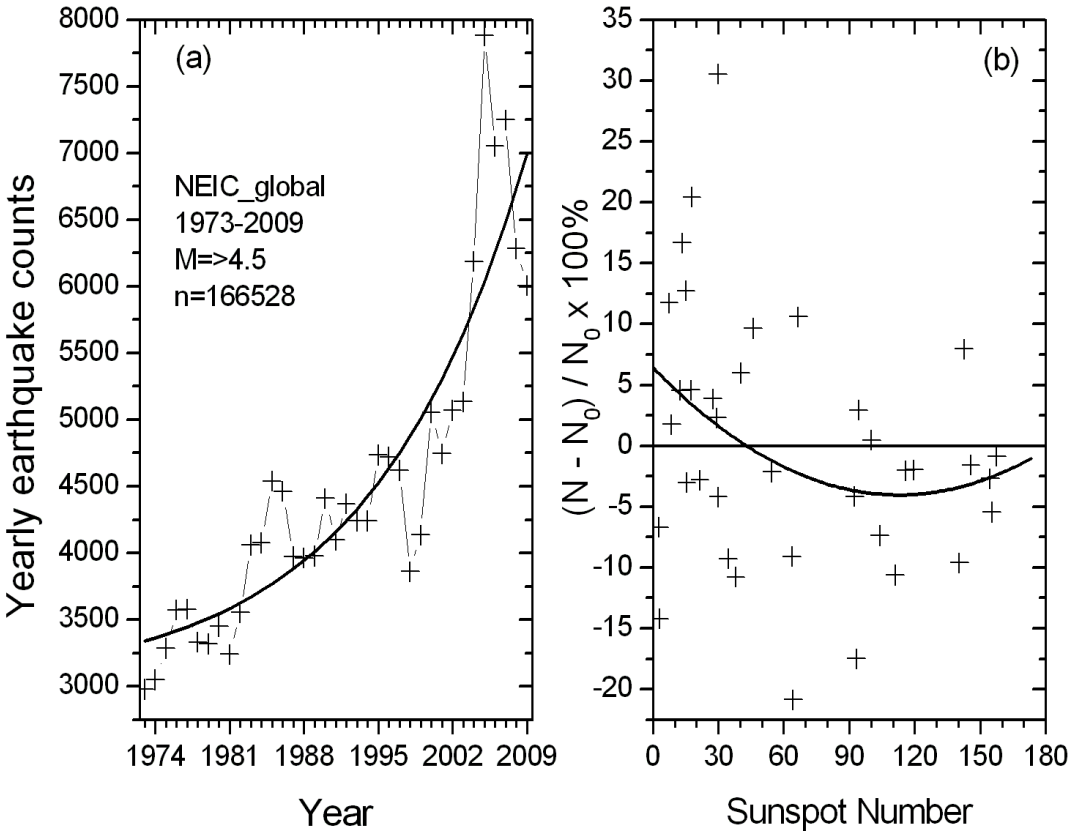

Fig. 3. In (a)- variation of yearly number of earthquakes with M≥4.5 from 1973 to 2009 (crosses) and approximation of this variation with the exponential growth fitting (thick curve). In (b) – scatter plot of deviations in percents of observed yearly number of earthquakes (N) from fitted one (N_0) versus yearly mean sunspot numbers and approximation of their dependence with polynomial of order n=2 (thick line).

It is seen from figure 3a that the number of registered at the globe earthquakes with M≥4.5 increases from 1973 to 2009. Evidently, this long term variation results mainly from increasing sensitivity of the global seismological network from 1973 to 2009. The functional form of observed long term variation was obtained with using exponential fit to data, as follows: $y = y_0 + A\ exp^{\wedge}(x/t)$, where y_0=3004±446, A=313±260, t=14.5±4.0 with a correlation coefficient R=0.89. The difference (ΔN) between registered yearly number of earthquakes (N) and its fitted value (N_0) was obtained as follows: $\Delta N\% = (N - N_0)/N_0 \times 100\%$. Panel 3b is a scatter plot of ΔN versus yearly mean sunspot numbers (W). The fit of scattered data to a polynomial of order n=2 is shown by the thick line. Its equation is as follows: $\Delta N\% = 6.5 - 0.19W + 0.008W^2$ with a correlation coefficient R=0.34. It is seen from figure 3b: at the globe, the smallest number of earthquakes with M≥4.5 occurs for moderate solar activity, when W varies between ~ 60 and 90. In solar minimum (W<40), the yearly number of earthquakes is increased according to that in moderate solar activity at about 10-15% on the average. This is time when the flux of galactic cosmic rays is increased. For periods of solar maximum (W>140), the yearly number of earthquakes also has a tendency to be increased at about 3-5 % according to the period of moderate solar activity. This is the time when the flux of GCR is decreased, but instead the solar cosmic rays flux may be increased due to intensive coronal mass ejections. Recently, Abakanov et al. (2007) revealed that at the local area of the North Tien-Shan in vicinity of area 42.5–43.25^{0}N, 76.5-77.25^{0}E, where the earth's crust is underlying with a thick (about 100 km) layer of active mantle (electric conductivity in this area is increased in comparison with neighbourhood areas), the parameters of seismo-tectonic deformation, calculated on the basis of focal mechanisms for small and moderate earthquakes, vary with 11 year variation of solar activity. In particular, this is evident

from figure 4, where the yearly mean values of the Lode-Nadai coefficient (one of the main parameters of seismotectonic deformation) are compared with yearly mean sunspot numbers and intensity of cosmic rays flux measured at Alma-Ata observatory (43.25^{0}N, 76.9^{0}E).

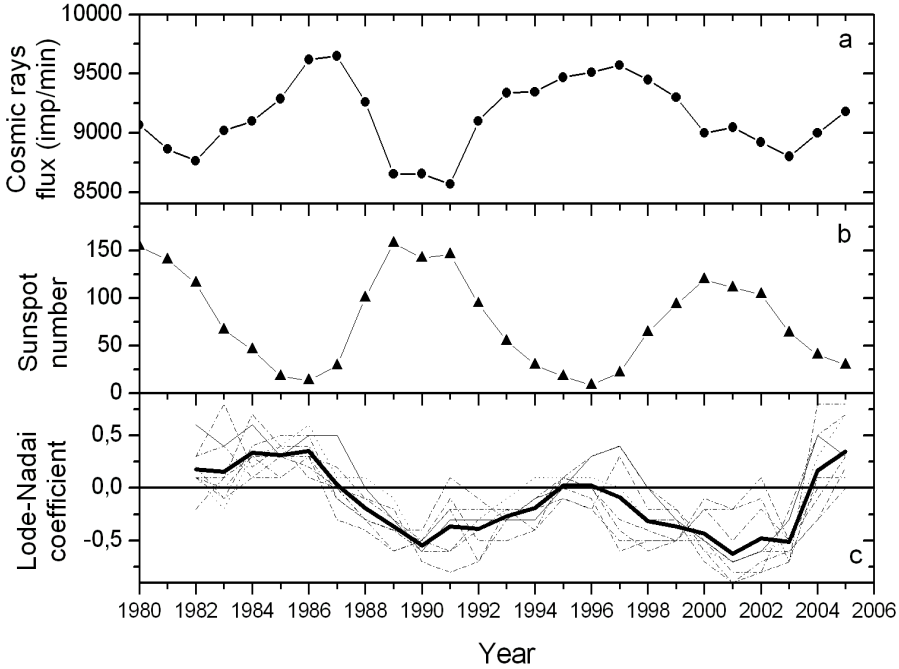

Fig. 4. In (a, b) – variations of yearly mean intensity of cosmic rays flux and sunspot numbers, respectively. In (c) – variations of yearly mean values of Lode-Nadai coefficient for eight elementary cells with dimentions 0.5^0 x 0.5^0 located in vicinity of central area 42.5^{0}N-43.75^{0}N, 75.00^{0}E-79.00^{0}E (thin curves) and their averages (thick) from (Abakanov et al., 2007).

CONCLUSION

In this report, we present the results of testing a new idea that earthquake is an element of global electric circuit GEC (Pulinets, 2009; and references in herein). In modified configuration of GEC (Makarova and Shirochkov, 1998), its external and internal boundaries are placed on the magnetopause and deeply into the solid earth, respectively, thus, the magnetospheric tube (schematic picture in figure 1) plays a role of connecting electric wire. Electrostatic and/or electromagnetic field changes in one part of GEC have to produce simultaneous changing in all another parts, as a result one may expect i) a magnetic conjugancy of the areas of increased seismic activity, and ii) a response of the seismotectonic regime on variations of intensity of cosmic rays flux and/or solar activity variations. Our statistical result (figure 2) indeed shows that seismic areas along southern part of the Mid - Atlantic Ridge are magnetically conjugate to seismic areas along its northern part; seismic areas along southern boundary of the Pacific plate are in rather cloze magnetic conjugancy with seismic areas along its northern boundary; and seismic areas along southern boundary of the Nazca plate are in very close magnetic conjugancy with seismic areas along northern boundaries of the Caribbean and Cocos plates. Figures 3, 4 prove that both the seismic regime at the globe, and the regime of seismotectonic deformation of earth's crust with increased electrical conductivity respond to 11 year variation of solar activity and/or intensity of cosmic rays flux. We consider that obtained results may support to some extent a new idea that earthquake is an element of global electric circuit, while much work remains to further verify this idea. If ths works, then for purpose of earthquake prediction, one needs the data of regular

monitoring of electromagnetic parameters along the whole magnetospheric tube loaned onto the area of earthquake preparation.

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PREDICTION OF POROSITY FROM SEISMIC ATTRIBUTE TRACES USING LOCAL LINEAR NEURO-FUZZY MODEL IN GORGAN BASIN

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Key Words: Local linear neuro-fuzzy model; LOLIMOT; Porosity; Seismic attribute; Gorgan.

ABSTRACT In this study we use local linear neuro-fuzzy model to predict log porosity from seismic attribute traces in Gorgan basin located in northern part of Iran. The study area data consist of both 3-D seismic and also well logging data. From total of 20 attributes, 4 attributes are selected for training the neuro-fuzzy model. The multi-linear stepwise regression method applied to select attributes one by one. The learning algorithm used for optimization of neuro-fuzzy model is LOLIMOT algorithm. Neuro-fuzzy model optimized with 7 LLMs (neurons), this model has a minimum validation RMS error. Validation of the optimized neuro-fuzzy model is confirmed by comparing the model output with porosity log of the well which not used in the model training. Porosity sections are presented as the output of the method, showing two low porosity sandy and shaly-sand channels in Brown Beds formation. Lateral variations of these channels can clearly be recognized in these sections.

INTRODUCTION

Prediction of porosity in hydrocarbon reservoirs is essential for estimating reserves and planning production operations. In most of the cases, however, lateral variations of porosity cannot be delineated from measurements made at sparsely located wells. The integration of 3D seismic data with petrophysical measurements can significantly improve the spatial description of porosity. In contrast with the sparse well observations, 3D seismic data provide a dense and regular areal sampling of the acoustic properties of the producing reservoirs. Acoustic impedance is indirectly related to porosity. A number of inversion methods are available in the industry to convert seismic amplitude into acoustic impedance (Lindseth, 1979). The major problems often generated by inversion are the band-limited nature of seismic data, presence of the noise, amplitude-scaling issues, and their non-uniqueness as these schemes are highly dependent on the initial guessed model. Further, these computed impedances are affected by different geological variables such as lithology, fluid saturation, pore pressure and temperature. Therefore, prediction of porosity using only acoustic impedance generated from the post-stack 3D data volume may not be reliable.

An alternate approach for the integration of well and seismic data is geo-statistical methods such as kriging and co-kriging. In the late 1980s, horizon-based single seismic attributes were used after averaging between the two boundaries to predict log properties with geo-statistical methods (Doyen, 1988). In the early 1990s, the concept of using more than one attribute (i.e., multi-attribute analysis) came into practice so that predictive power of estimating log properties could be increased. The idea of using multiple seismic attributes to predict log properties was first proposed by Schultz et al., (1994a, b). In this approach, the well-to-seismic ties are considered to be the training points, and the statistical methods are used to derive the relationship between the attributes and well-log data. In latest methods, intelligent techniques such as artificial

neural network and neuro-fuzzy techniques are used to predict log properties from multiple seismic attributes.

In this paper, we predicted porosity of a reservoir in Gorgan basin from multi-attribute cubes by a kind of neuro-fuzzy techniques, called Local Linear Neuro-Fuzzy technique. The algorithm used to train neuro-fuzzy model is LOLIMOT (Local Linear Model Tree) proposed by Nelles (1999).

LOCAL LINEAR NEURO-FUZZY MODELS

Local linear neuro-fuzzy modeling also referred to as Takagi-Suggeno modeling is based on a divide-and-conquer strategy. A complex modeling problem is divided into a number of smaller and thus simpler sub-problems, solved (almost) independently by identifying simple i.e., linear, models. The most important factor for the success of such an approach is the division strategy for the original complex problem. Therefore, the properties of local linear neuro-fuzzy models crucially depend on the applied construction algorithm that implements a certain division strategy.

The network structure of a local linear neuro-fuzzy model is depicted in figure 1. Each neuron realizes a local linear model (LLM) and an associated validity function that determines the region of validity of LLM. The output of the LLMs is (Nelles, 1999)

$$\hat{y}_i = \omega_{i0} + \omega_{i1}u_1 + \omega_{i2}u_2 + \ldots + \omega_{ip}u_p \tag{1}$$

where ω_{ij} denote the LLM parameters for neuron i. Validity functions obtained from individual membership functions

$$\phi_i(\underline{u}) = \frac{\mu_i(\underline{u})}{\sum_{j=1}^{M} \mu_j(\underline{u})} \tag{2}$$

The validity functions form a partition of unity, i.e., they are normalized such that

$$\sum_{i=1}^{M} \phi_i(\underline{u}) = 1 \tag{3}$$

This property is necessary for a proper interpretation of ϕ_i as validity functions because it ensures that the contributions of all local linear models sum up to 100%. The output of a local linear neuro-fuzzy model becomes

$$\hat{y} = \sum_{i=1}^{M} \hat{y}_i \phi_i(\underline{u}) \tag{4}$$

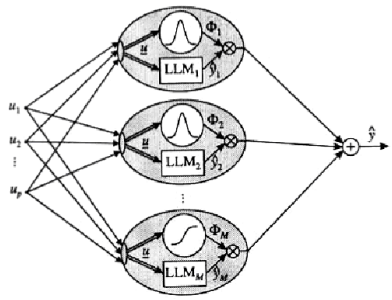

Fig. 1. Network structure of a local linear neuro-fuzzy model with M neurons for p inputs (Nelles, 2001).

PARAMETER OPTIMIZATION

Estimating the local linear model parameters (ω_{ij}) is a linear optimization problem under the assumption that the validity functions are known. The validity functions estimated during each run of LOLIMOT algorithm. LOLIMOT is an incremental tree-construction algorithm that partitions the input space by axis-orthogonal splits (Fig. 2). This algorithm consists of five steps:

1. Start with initial model: set the number of LLMs equal to one.
2. Find worst LLM: The local loss function is computed for each of LLMs. The local loss functions can be computed by weighting the squared model error with the degree of validity of the corresponding local model. Find the worst performing LLM that has maximum loss function value.
3. Consider all divisions: The worst LLM is selected for future refinement.
4. Find best division: The best of p alternatives considered in step 3 is selected. This division decreases the total model error. The number of LLMs is incremented.
5. Test for convergence: If the algorithm criterion is met then stop, else go to step 2.

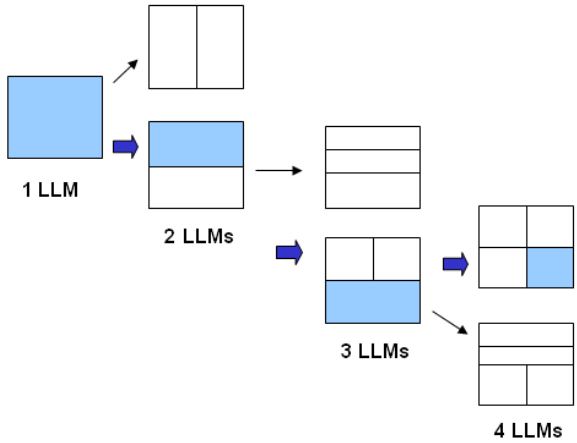

Fig. 2. Four iterations of LOLIMOT algorithm for a problem with 2D input space.

PREDICTION OF POROSITY IN GORGAN BASIN

Gorgan basin is located in northern part of Iran, in south east of Caspian Sea. The discovery of gas from the Brown Bed Formation in GO3 well has given hopeful point to future exploration activities in the Gorgan Basin. The Brown Bed Formation has shale and sand units. Due to the stratigraphy nature of the hydrocarbon entrapment, the systematic delineation and development of this reservoir is a challenging task for drilling. The objectives of the survey were delineation of channels and distinguishing between sand and shale bodies. The data consist of both 3D seismic data which ties two wells. In this study, we tried to predict the porosity from 3D seismic data.

In the first step, 20 synthetic seismic attributes were extracted from raw seismic data. Then stepwise regression method (Draper and Smith, 1966) was used for selection of 4 attributes from total of 20 attributes. These attributes are:

- instantaneous frequency
- amplitude weighted frequency
- apparent polarity
- second derivative instantaneous amplitude

These selected synthetic seismic attributes and porosity log for well GO3 were used for training neuro-fuzzy model. Also porosity log for well GO5 was used for validation of neuro-fuzzy model. With adding each LLM during the training procedure, validation of neuro-fuzzy model was performed. Training and validation error of final model with 7 LLMs are 0.047 and 0.064 respectively (Fig. 3).

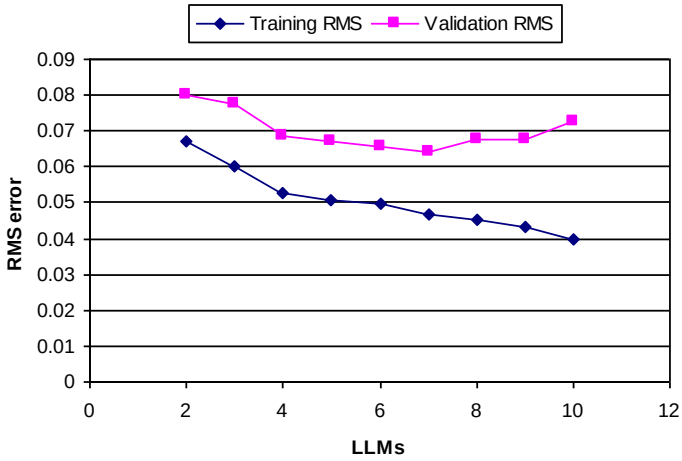

Fig. 3. Error diagrams of neuro-fuzzy model optimization.

Figures 4-a and 4-b show the porosity prediction from the seismic attributes for the inline 156 and xline 292, respectively. The color scale is in porosity units, low porosities are presented with blue color and high porosities with yellow and red. Two low porosity sandy and shaly-sand Channels can be recognized in Brown Beds formation from porosity sections. In these sections, we can see lateral variations of these cannels clearly. These cannels intersect by well GO3 at 750-775 ms and 1060-1105 ms. Samples obtained at depths 1640, 1740 and 2035 m of well GO3 and contain oil and gas, confirm existence of two channels.

CONCLUSIONS

The results obtained in this research confirm the prediction power of local linear neuro-fuzzy model as robust tool in reservoir characterization.

Based on the porosity distribution sections, two sandy and shaly-sand Channels are identifiable in the Brown Beds formation. Prediction of porosity distribution provides better understanding about the quality of the reservoir, and it also allows the quantitative evaluation of the reserves. It also is helpful in formulating the strategy for exploration and development of hydrocarbons.

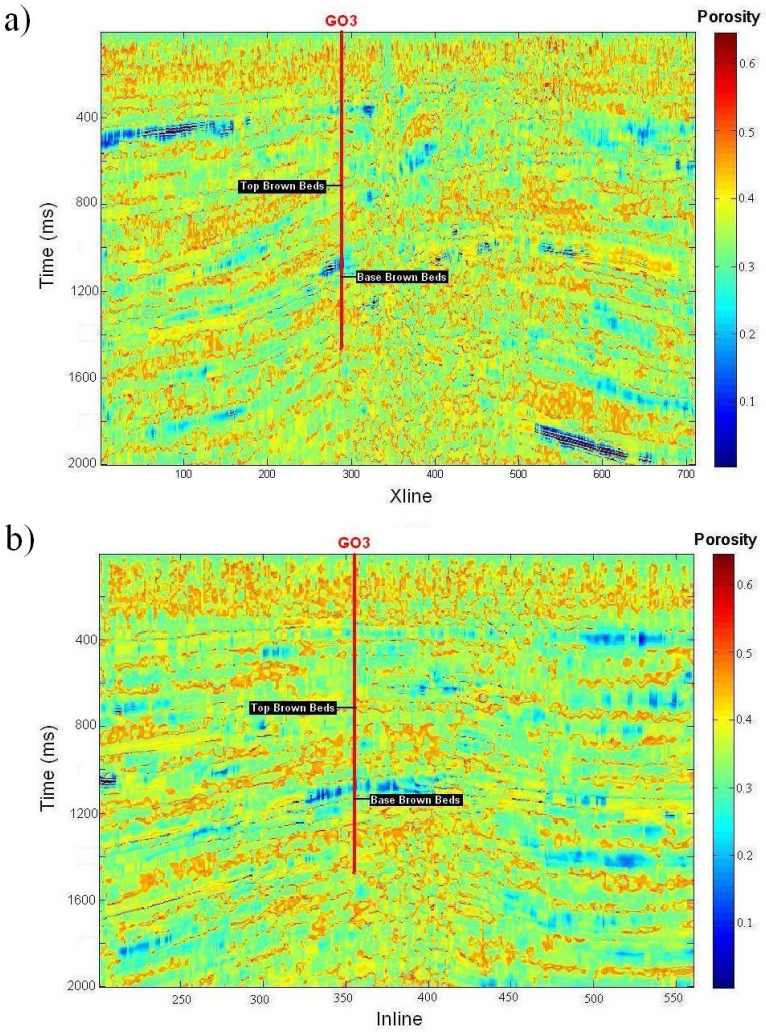

Fig. 4. Porosity sections obtained from local linear neuro-fuzzy model, (a) Inline 156, (b) Xline 292.

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SEISMIC HAZARD ASSESSMENT FOR KOPEH DAGH REGION

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Keywords: Seismicity, Kopeh Dagh, hazard assessment, Shirin Darreh dam

ABSTRACT The region studied in this research is situated in Kopeh Dagh region enclosed between 36° to 39° north latitudes and 55.5^o to 60° east longitudes and covers Ashghabad , Quchan and Bojnurd cities. This region lies within a broad zone of deformation that forms part of Alpine-Himalayan orogenic belt. The studied region forms the Northeastern boundary of Iran plateau and is located on Southwestern margin of Turan (Turkmenistan) platform. Kopeh Dagh main fault separates the region from Turan plate in north. In this study we use the data from ISC catalogue (www.isc.ac.uk) from 1964 to 2008 early instrumental and historical earthquakes from Ambraseys & Melville, (1982). The earthquake hazard is assessed using empirical-statistical and deterministic methods. Seismicity parameters are evaluated and the earthquake hazard curve is plotted with hazard software. Finally earthquake probability for magnitude 6.5 in this region in the next 25,50 and 75 years are 40%,65% and 79% respectively, furthermore earthquake probability for magnitude 7 in this region in the next 25, 50 and 75 years are 18%,34% and 45% respectively. On the basis of deterministic method, the Shirin Darreh dam is proposed as a high potential seismic hazard site and PGA parameter assessed 0.9g for this site that evaluated from Quchan, Baghan and Espharayen faults activity.

GEOLOGY, SEISMICITY AND SEISMOTECONIC:

The Iranian plateau is a relatively wide zone of compressional deformation along the Alpine-Himalayan active mountain belt, which is entrapped between two stable platforms, the Arabian plate in the southwest and the Turan platform (Eurasia) in the northeast. Its deformation is related to the continuing convergent movement between the Arabian plate to the southwest and the Turan platform to the northeast, by north-northeastward drift Arabia against Eurasia. lithospheric movement has led Iran to be one of the seismically active areas of the world and frequently affected by destructive earthquakes, imposing heavy losses in human lives and widespread damage. One of the Five major seismotectonic provinces in Iran is Kope Dagh that we use in this article is delineated based on all available geophysical, geological, tectonic and earthquake data. Continental collision zone of Kopeh Dagh in northeast is a northern segment of Alpine-Himalayan seismic belt which faces the Turan platform in the north.

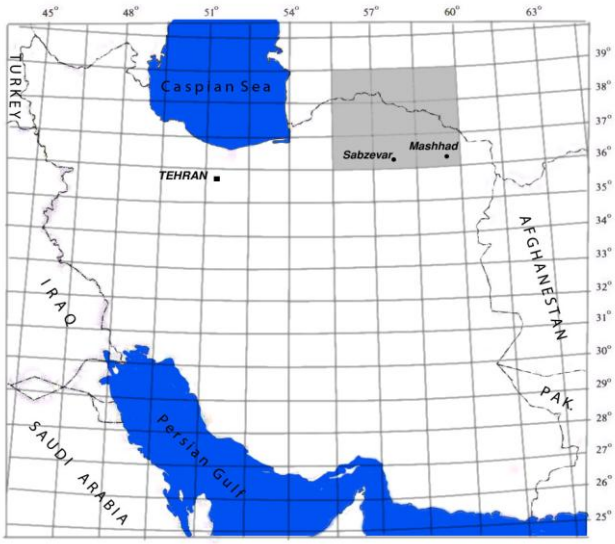

Fig. 1. Studied region (shadowed square).

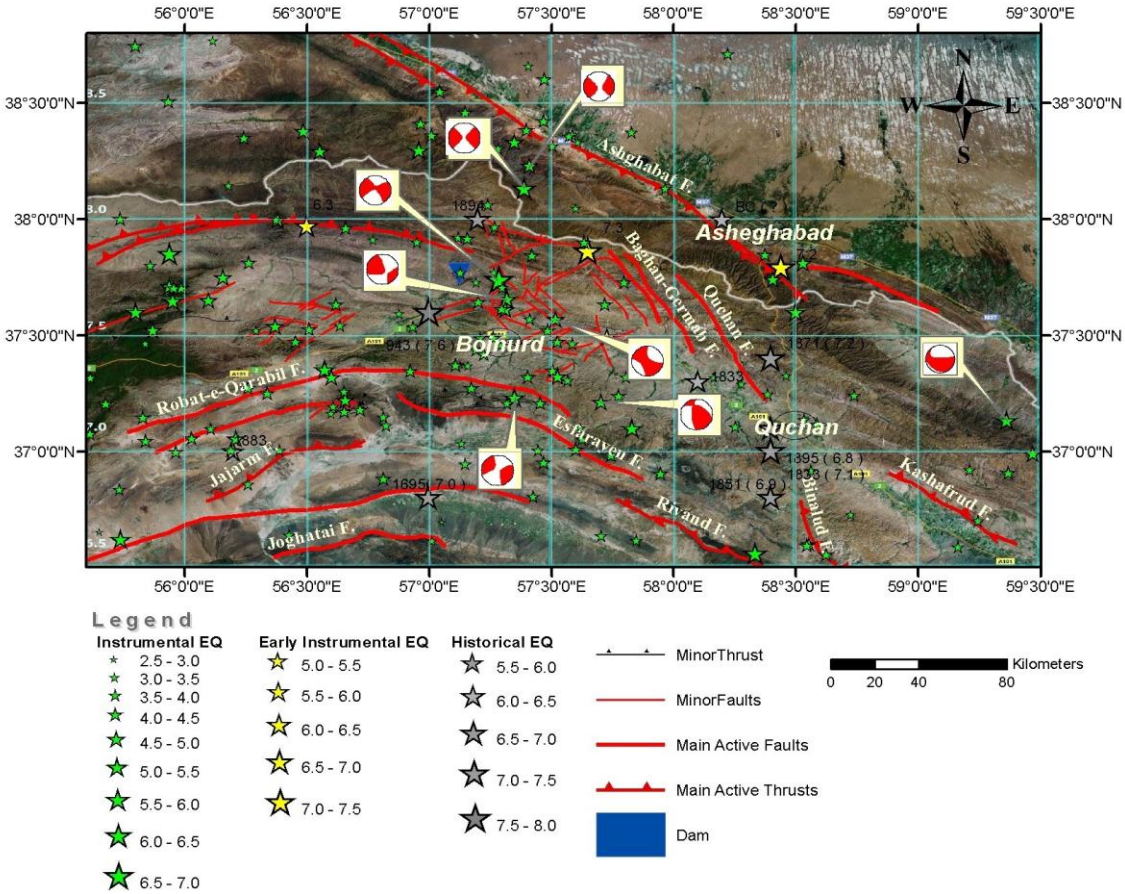

Fig. 2. Seismotectonic map of studied region.

Our study area is situated in Kopeh Dagh region enclosed between 36° to 39° north latitudes and 55.5^o to 60° east longitudes (Fig1), especially in the Northern Khorasan province (adjacent to the central Afghanistan) to the East, the Turkmenian portion of the Kope Dagh in Turkmenistan to the North, Central Iran and Central and East Alborz to the West and Southwest, and the Lut block to the South ,The Khorasan province is mountainous in nature with NW–SE ranges and valleys. The northern and northeastern Khorasan province corresponds to the large highland system of the Kope Dagh. This system includes three NW–SE trending ranges, the Hezar Masjed, Binalud and Jogatai. West of study area, the Ala Dagh mountains is an extension of the Binalud Range and connects the mountains between Bojnurd and Astarabad: this Ala Dagh mountain belongs to the Aborz system. As the southern limit of our study area, the northern part of the Lut block has an average elevation of scarcely 500 ft. In this region, the Doruneh Fault with SW–NE trend in the western part, E–W in central part and NW–SE trend in eastern part, separates the Lut block from the Central-East Iran microplate and NE Iran.The central Iranian block is believed to be rigid and part of the deformation is transmitted to the north in the Alborz, Kope Dagh and Caucasus mountains in the north. Many faults located in this region that responsible for a lot of earthquakes. For destructive earthquakes occurred near Quchan (1851, 1871–1872,1893 and 1895) near Shirvan (1929), Ashkabad (1948) and Bojnurd(1997) which, if repeated, would have very serious consequences in the now more-densely populated Atrak valley. These faults that located in this region most of them in the western part of the NNW–SSE right-lateral strike-slip zone between Bojnurd and Quchan (Fig2). These faults name are Espharayen, Kope Dagh, Baghan, Marave Tappe, Quchan, Kashafrood, Robate Qarbil, Joghatai, Rivand and Jajarm.

EARTHQUAKE HAZARD ASSESSMENT USING EMPIRICAL-STATISTICAL METHOD

This method is the simplest method for assessing hazard that according to earthquake statistics. In this method we use from last earthquake parameter for determine seismicity directly. Earthquake occurrence probability in future, equal to average frequency of last earthquakes. At last we acquire 3 parameters for determine earthquake probability. First we use the data from ISC catalogue (www.isc.ac.uk) from 1964 to 2008 (instrumental earthquakes), and early instrumental and historical earthquakes from Ambraseys & Melville, (1982). For the harmony of data we transform instrumental earthquakes from mb to Ms (Fig3) and RMS error assessed 0.79.

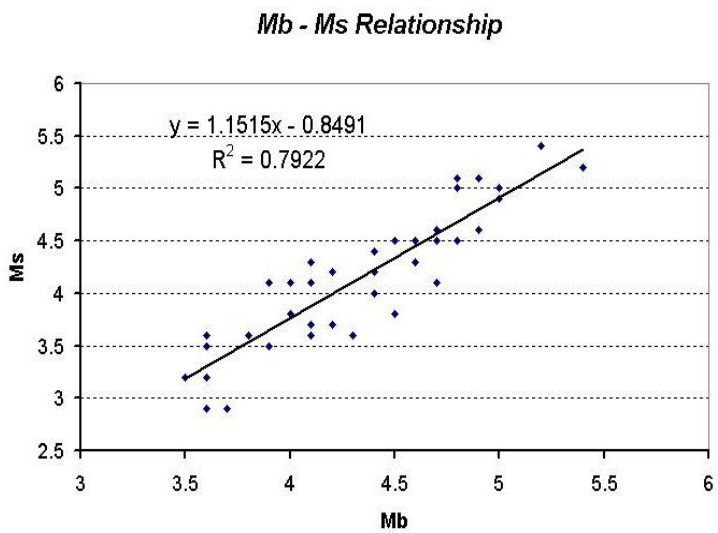

Fig. 3. Ms-mb Relationship.

Then Isc data divided to two different period, Complete and incomplete period (Fig4). Then we acquire threshold magnitudes (MC=4) for 1964-1997 period and (MC=3.2) from Zmap software (Fig5&6)

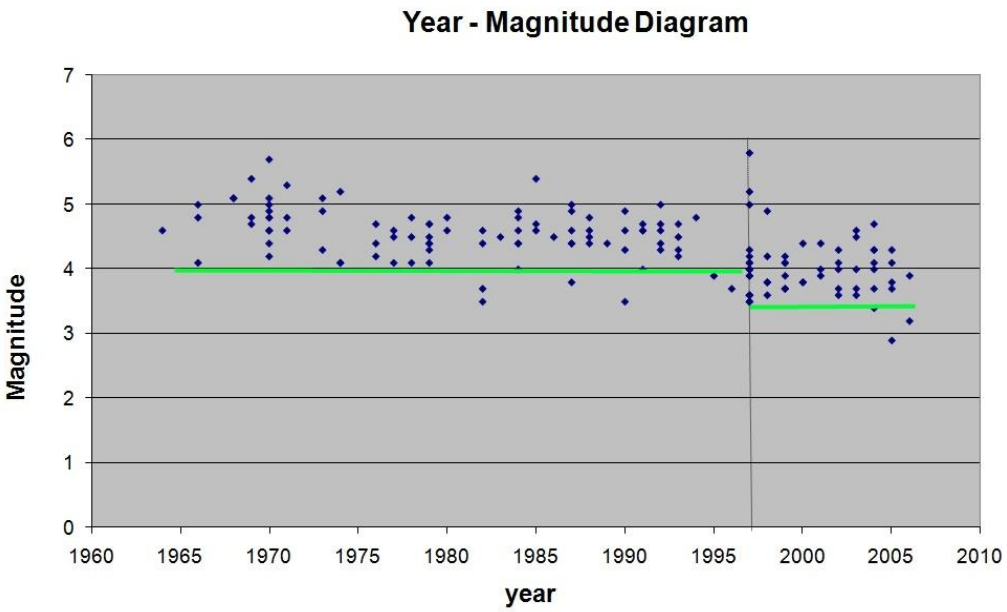

Fig. 4. Year magnitude diagram for Isc data.

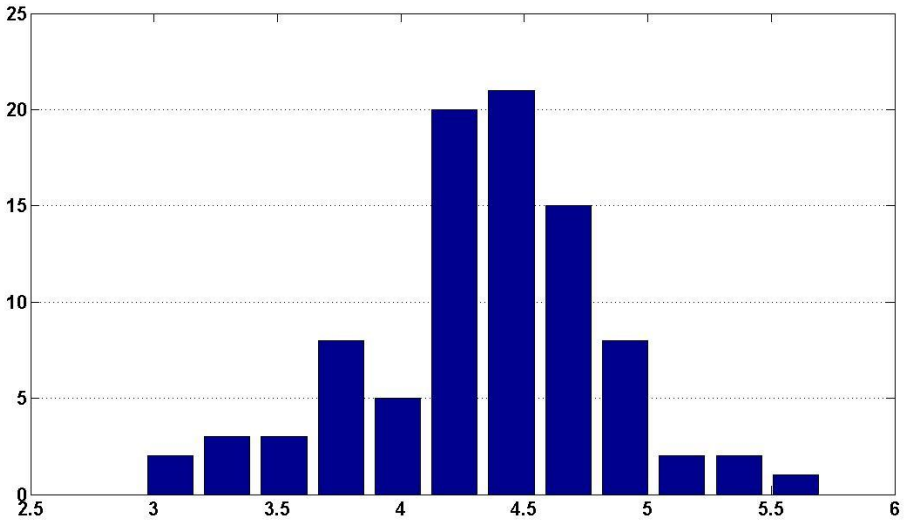

Fig. 5. Magnitude frequency diagram from 1964 to 1997.

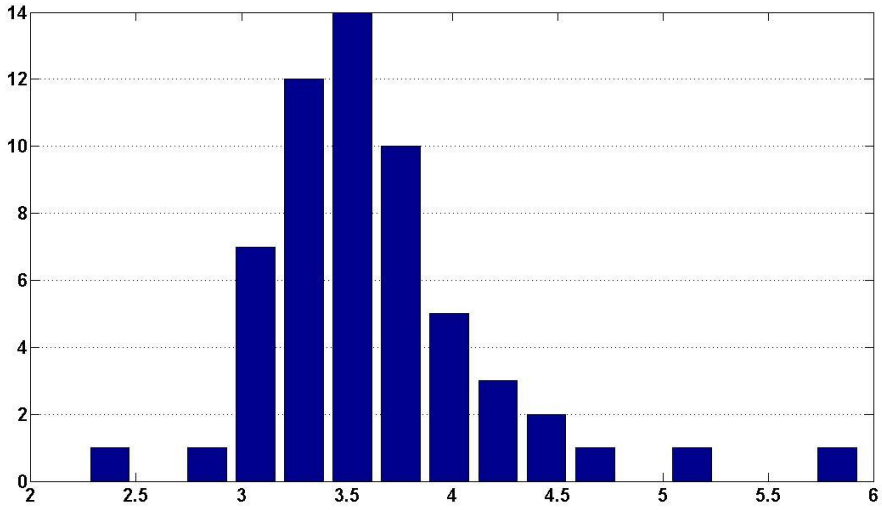

Fig. 6. Magnitude frequency diagram from 1997 to 2007.

After collecting earthquake parameters, determining complete and incomplete period and devote mc to them, we acquire seismicity parameter (β , $\lambda, M_{\max}$) using Hazard software.

(1) Beta = 1.61 ± .07 (b = .68 ± .03)

λ :seismicity rate

(λ)Lambda = 23.99 ± 2.92

Mmax:maximum earthquake in oresent geodynamic regime Mmax = 7.90 ± .78

After running Hazard program earthquake occurrence probability assessed and we showed in the below chart (Fig7).

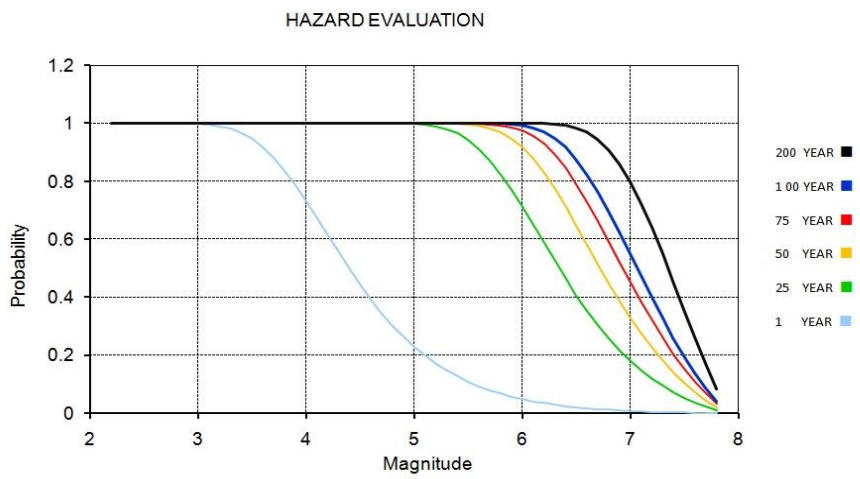

Fig. 7. Earthquake hazard curve.

For example earthquake probability for magnitude 6.5 in this region in the next 25,50 and 75 years are 40%,65% and 79% respectively.

EARTHQUAKE HAZARD ASSESSMENT FOR SHIRIN DARREH DAM USING DETERMINISTIC METHOD

The principle of analysis involved in the deterministic approach is to evaluate the critical seismogenic sources, like capable faults and the selection of a maximum potential magnitude assigned to each of these faults. then with the help of suitable attenuation equations ,peak horizontal accelerations associated with maximum earthquake along these faults are determined. Therefore the deterministic method includes the following steps:

1-identify all critical tectonic features in the vicinity of the site likely to generate significant ground motions.

2-assign to each of these a maximum potential magnitude on the basis of key fault parameters like fault ruptures length, displacement or slip rate.

3-compute the ground motion parameters at the site associated with each feature as a function of magnitude and distance using the appropriate attenuation relationships.

On the basis of deterministic method the Shirin Darreh dam is proposed as a high potential seismic hazard site. location of this dam is showed in (fig8).

This dam is located in 37.42 north latitudes and 57.06 east longitudes and its situated in northwestern of Bojnurd.

For deterministic method we assume circular region with radius 150 kilometers around this dam (fig9).

Fig. 8. Shirin Darreh location in satellite map.

Fig. 9. Studied region 150 km around dam.

Then seismotectonic map of region plotted and all of faults as sources were segmented (fig10).then Mmax was assessed for each segment.

(2) I = 11.926 + 0.831Ms − 2.7Ln(R + 22)

(3) $a = 1.1e^{0.5M}(R+25)^{-1.32}$

I: Intensity, R: distance between dam and earthquake epicenter, a: acceleration ground motion

Finally with attenuation equation (2,3) ground acceleration parameters and earthquake probable intensity assessed and showed in table 1.

Table 1. Ground acceleration parameters and earthquake probable intensity.

label	Mmax	Distance(km)	I	a(g)
10	7.5	92.5	10	0.7
17	8	4.6	10	0.9
18	8	65.5	10	0.9
32	8	140.5	10	0.9
33	7.5	78.5	10	0.7

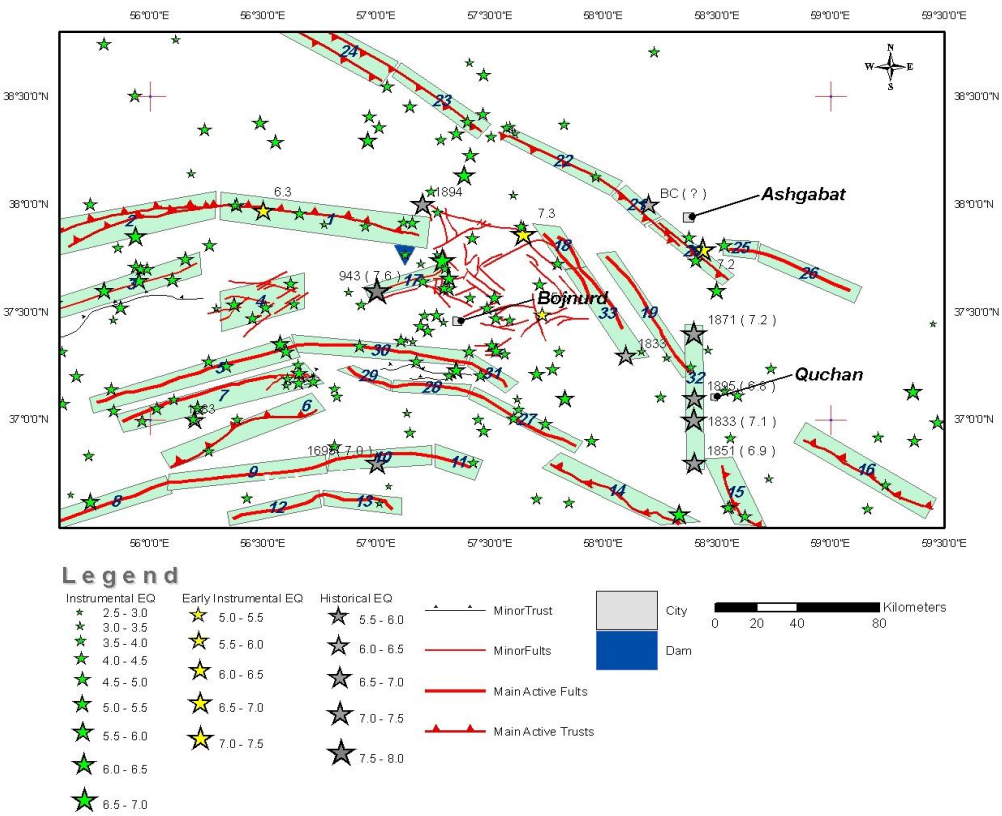

Fig. 10. Segmented seismotectonic map.

CONCLUSION

In this article we use two hazard assessment methods for Kope Dagh region and Shirin Darreh dam and we assessed earthquake probability for magnitude 6.5 in this region in the next 25,50 and 75 years are 40%,65% and 79% respectively, furthermore earthquake probability for magnitude 7 in this region in the next 25, 50 and 75 years are 18%,34% and 45% respectively Also PGA parameter assessed 0.9g for Shirin Darreh dam that evaluated from Quchan, Baghan and Espharayen faults activity. Also we understand that segments No.32 , No.17 and No.18 (fig10-table1) are the most effective segment for this site, but segment No.32 doesn't have any fault so we must investigate on seismotectonic, tectonic, hidden fault and etc in future study.

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SEISMICITY OF TEHRAN AND SURROUNDING REGIONS

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ABSTRACT Historical documents, instrumentally recorded earthquakes during the past hundred years and the existence of many Geological faults all indicate that Tehran region has a high potential of seismic activity. Although in the past centuries many destructive earthquakes occurred around Tehran region, but no major instrumentally recorded earthquake has been reported during the past hundred years. From 1975, ILPA seismic network in southwest of Tehran has recorded many micro-earthquakes in Tehran region. In this study, all the recorded local events were extracted and analyzed. The results are in good agreements with the distribution of local faults. In some areas a kind of seismic gap can be understood. Taking into account the density of population in this region and of the economic and political conditions, it is necessary to carry on more extensive studies.

INTRODUCTION

A new GPS observation indicates that N-S reduction over Alborz happens at 5(+ or -) 2mm/yr and also across the overall belt, the left-lateral shear has a 4(+ or -)2 mm/yr rate (Vernant et al., 2004). The Recorded seismological data in Alborz and other regions around the southern Caspian Basin helped Jackson et al.(2002) to reach this conclusion: The southern Caspian Basin is on the move towards NW with respect to Eurasia; According to the GPS data set, the highest northern component of southern Caspian plate has a 5-6 mm/yr movement. The seismic activity across the Apsheron ridge, The overthrusting of Talesh to the east, and the left lateral movement across the WNW-ESE Rudbar fault, to the north, west, and southern Caspian basin's southwest are accounted in this model respectively. The southwestward motion of the southern Caspian Basinwith respect to central Iran, coupled with the NS convergence of central Iran, leads to a NNE-SSW transpressional regime in Alborz which would have started between 3 and 7 m.y. ago. Right- lateral movements observed across range- parallel strikeslip faults in western Alborz before this date, suggest that the range was eforming against the solid and stable southern Caspian domain under a N-S compressional regime (Axen et al., 2001; Jackson et al., 2002; Allen et al., 2003). From 1975, ILPA seismic network in southwest of Tehran has recorded many micro-earthquakes in Tehran region. In this study, all the recorded local events were extracted and analyzed.

DISCUSSION

The region that has been studied in this article, is on 49-52.5 longitude degree and 35-37 latitude degree. The Iranian Plateau, in the middle part of alpine – himalayan active mountain belt, is one of the mose seismically active zone in the world. Convergence of the Arabia surface toward Orasia causes the Alborz & Zagros wrinkles and too many earthquakes. most of the Iran's earthquakes have the contractional componant that make the mot destructive focal mechanism.

According to the information from historical and instrumental earthquakes, the cause of the most earthquakes in this region are the active Fault like: Mosha – North of Tehran – Ipak – Khazar – Garmsar and … that you can see in map 1

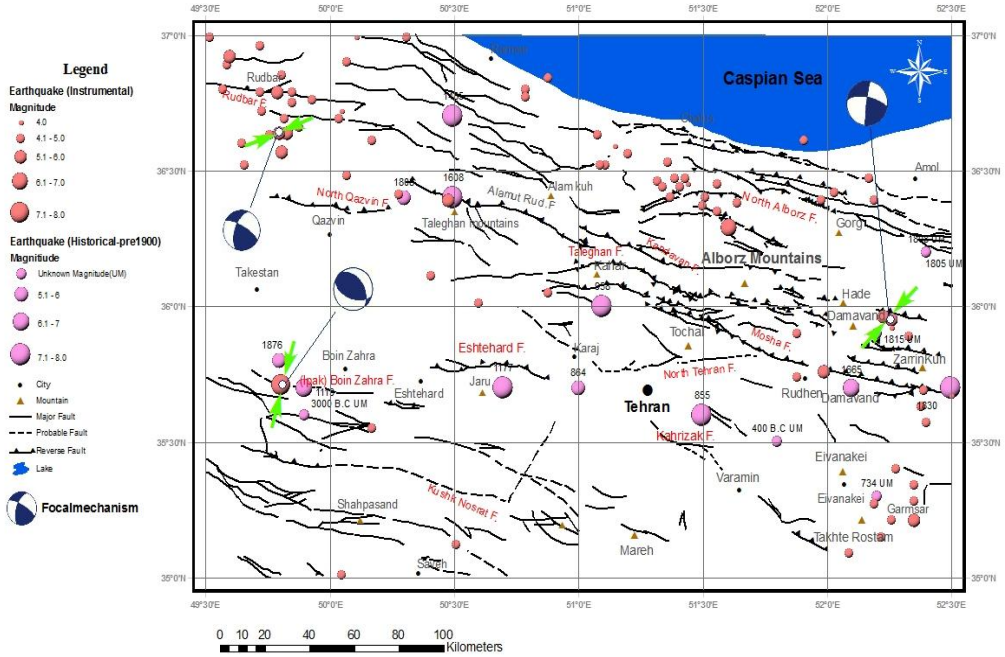

Fig. 1. Seismotectonic map of Tehran and surroundings

As you see in figure 1, opposite to the historical earthquakes, these earthquakes have the power of 4-5 magnitude, except for the Boin Zahra earthquake, and we see the almost calmness in the southern area of Tehran.

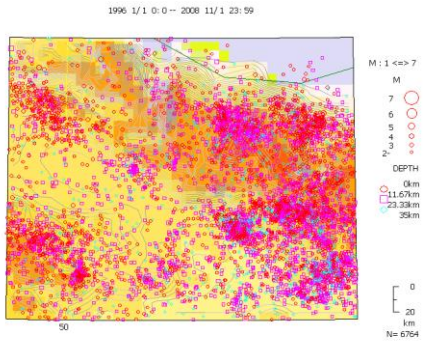

Fig. 2. Distribution of Earthquake epicenter (1996 – 2009).

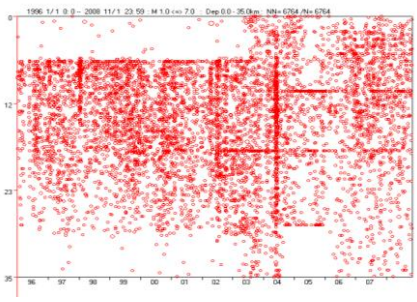

Fig. 3. Distribution of Earthquake source according to the depth (1996 – 2009).

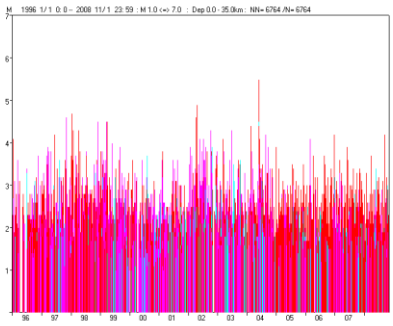

Fig. 4. The number of event according Magnitude (1996-2009).

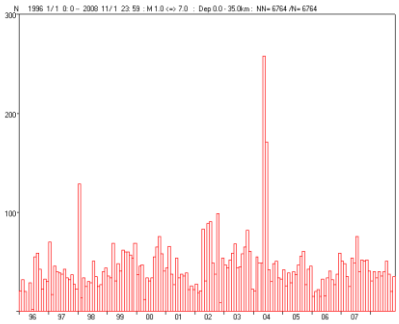

Fig. 5. Daily number of event (1996-2009).

6764 earthquakes and microearthquakes from 1996 to 2009 in this region has been surveid .

Figure 2 shows the number of the earthquakes and micro-earthquakes with magnitude from 1 to 7.

Blue signs , show the earthquakes with the depth of more than 25 km , and the purple signs shows the earthquakes with the depth range of 10 to 25 km and the red signs shows the earthquakes with the depth of lower than 10 km. Because this region is a continental -continental collision zone and the depth of the earthquakes , as you see in figure 3, is shallow, but in comparing with the other continental zone, is deeper that has the average of 10-25 kilometer .

In figure 4, chart is based on the magnitude of the earthquakes in 10 year period. Blue signs show the depth more than 25.

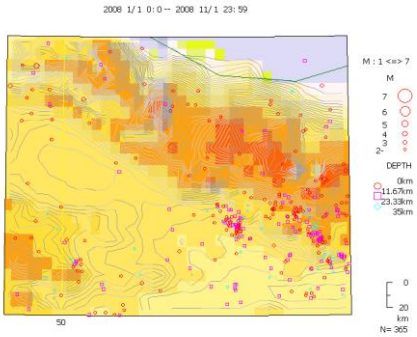

Fig. 6. Distribution of Earthquake epicenter during last year.

Fig. 7. Daily number of Earthquake during last year.

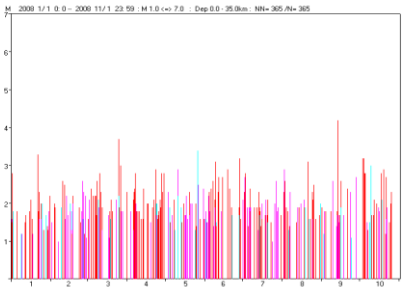

Fig. 8. The number of event according Magnitude during last year.

According to the figures 6,7,8 that Magnitude of the earthquakes has been studied, we also see the shallow depth and we have 365 micro-earthquakes, averagely, 1 earthquake, each day.

Figure 8, shows the Magnitude of the earthquakes in recent year.

CONCLUSIONS

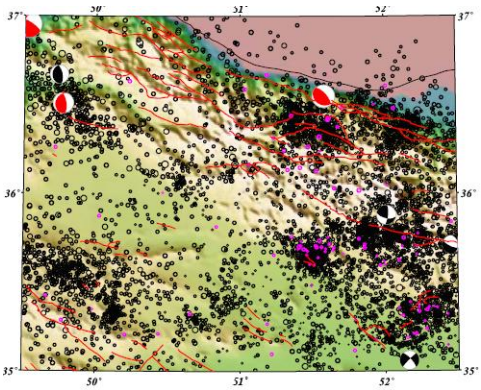

Fig. 9. Distribution of Earthquake epicenter.

According to the distribution of Earthquake epicenters and the Tehran earthquakes Catalog, Tehran and the surroundings is in the state of consanguineous calm and as we see in the seismic history of this region and the tectonical parameters, we can predict the long return peiod for this region but with the destructive earthquakes.

according to the figure 9, in the continuation of the Caspian sea, we see the more earthquakes that had been cause but the Khazar and Alborz fault, and also surrounding the Ipak fault that had been the cause of the destructive earthquake in 1 august 1963 in Boin Zahra, and the surroundings of Damavand that the contractional fault of the Mosha is crossing there.

depth of the earthquakes, as we see in the figures, is in the upper crust and in the range of 29 kilometers.

with the development seismic instruments , as you sees in the figures 2, 3, 4, 5 , almost 6764 earthquakes below 4 magnitude occurred in the past 10 years and also 365 earthquakes below 2 magnitude has been registered (figure 8)

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STATISTICAL PROPERTIES OF THE AFTERSHOCK SEQUENCE OF M_D=5.3 ANKARA (BALA) EARTHQAUKE OF DECEMBER 27,2007

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Key Words: Bala, Aftershock, Gutenberg-Richter relation, modified Omori law.

ABSTRACT In this study, the statistical properties of the aftershock sequence of Ankara (Bala) earthquake of December 27, 2007 is analyzed. For this purpose, the *b*-value of the frequency-magnitude relationship and temporal decay rate of aftershocks as described by the *p*-value are calculated in order to assessment the some characteristics of the aftershock sequence in the five months after the $M_{D=}$5.3 Bala (Ankara) earthquake of 27 December 2007. The catalog taken from the KOERI contains 539 aftershocks with magnitude M_D larger than and equal to 2.6. Aftershock sequence is concentrated around the main shock epicenter and shows an elliptic area in the NW-SE direction. All of the computations are programmed in *ZMAP* software. Using the maximum likelihood procedure, *b*-value of Gutenberg-Richter relation is computed as 1.48±0.06, rather large, with magnitude completeness *Mc*=2.8. The catalogue does not consist any aftershocks with magnitude M_D>5.0 and there are only 6 aftershocks greater than *M*4.0. This large *b*-value may be resulted by the paucity of aftershocks whose magnitudes are larger than 5.0. By using $M_D \geq Mc$ (=2.8) and T_{start}=0.002 days, *p*-value of modified Omori law is calculated as 0.75±0.03 to be relatively small. Relatively small *p*-value may be a result of the slow decay rate of the aftershock activity or small number of aftershocks. Thus, the limitation of the data set strongly affects the results and more detailed future studies are needed for underlying physical mechanism of the *b* and *p*-values variations.

INTRODUCTION

Several authors have noted the importance of systematic investigation of aftershock sequences to earthquake prediction and a number of statistical models have been proposed to describe seismicity characters in time, space, and magnitude (e.g., Utsu, 1971; Gou and Ogata, 1997). Recent years have shown a significant increase in attention paid to aftershock sequences, because they can provide an understanding of the mechanism of earthquakes and they are potential sources of information about earthquakes nucleation and the physical properties of the materials in fault zone within which slip occurs during an earthquake. The characteristics of sequences that may yield useful information are the spatial distribution, total number of aftershocks, and the decay rate of sequence with time. Also, aftershock sequences offer a source of information on the Earth's crust and source properties of large earthquakes, because a very large number of events occur over a short period of time in a small area. Two basic relations describe the aftershock activity, (a) the Gutenberg-Richter's (G-R) law which describes the power-law of size distribution of earthquakes (b) the modified Omori law which describes the decay rate of the aftershock activity.

The frequency-magnitude distribution (Gutenberg and Richter, 1944) describes the relation between the frequency of occurrence and magnitude of earthquakes:

$$\log_{10} N(M) = a - bM \tag{1}$$

where *N(M)* is the cumulative number of earthquakes with magnitudes equal to or larger than *M*, *b* describes the slope of the size distribution of events, and *a* is proportional to the productivity of a volume, or the seismicity rate. Although *a* and *b*-values are

determined empirically from seismic catalogues, it is imperative to understand their physical meaning.

On the other hand, the temporal distribution of aftershocks is empirically well described by the modified Omori law (e.g., Utsu et al., 1995) through the following equation:

$$n(t) = \frac{K}{(t+c)^p} \tag{2}$$

where *n(t)* is the number of events per unit time at time *t* after the mainshock. *K*, *c*, and *p* are constants. *K* depends on the total number of events in the sequence, *c* on the rate of activity in the earliest part of the sequences. The constant *c* is strongly influenced by incomplete detection of small aftershocks in the early stage of sequence. Of these three parameters, *p* is a decay parameter and the most important one.

DATA and RESULT of ANALYSIS

In this study, the statistical characteristics of aftershock sequence of 2007 Bala earthquake is analyzed. The data are taken from the website of the Bogazici University, Kandilli Observatory and Earthquake Research Institute (KOERI). The main shock (M_D=5.3) occurred at 01:47:08 UTC on December 27 and the epicenter coordinates were determined as 39.42°N-33.09°E (KOERI). The catalogue with magnitude M_D larger than or equal to 2.6 contains 539 aftershocks. This catalogue contains five month's time period, that is, from the time of the main shock (December 27, 2007) until May 27, 2008. The aftershock sequence of Bala is completed between 2.6 and 5.0 magnitude bands. It is a remarkable fact that there are no aftershocks with magnitude M_D>5.0. There are 6 shocks with magnitude 4.0≤M_D<5.0 and M_D5.0 shock is the largest of all. Also, the depths of aftershocks are between 1.5 and 29.1 km. Figure 1 shows the epicenters of the aftershock sequence and main tectonics of surrounding area. Aftershocks of 2007 Bala earthquake are generally located between the Bala-Afşar-Mandıra-Bahçekaradalak regions (NW-SE direction). There are numerous small faults in the aftershock region (Fig. 1). The highest density of large events (M_D≥4.0) is observed in the northwest part of the main shock while the all size of aftershocks is occurred in the southeast of the main shock. Considering the reports of Bala earthquake which are issued in KOERI and the other institutes, we used the longitudes between 39.0°E and 39.8°E, and latitudes between 32.8°N and 33.6°N as the aftershock area.

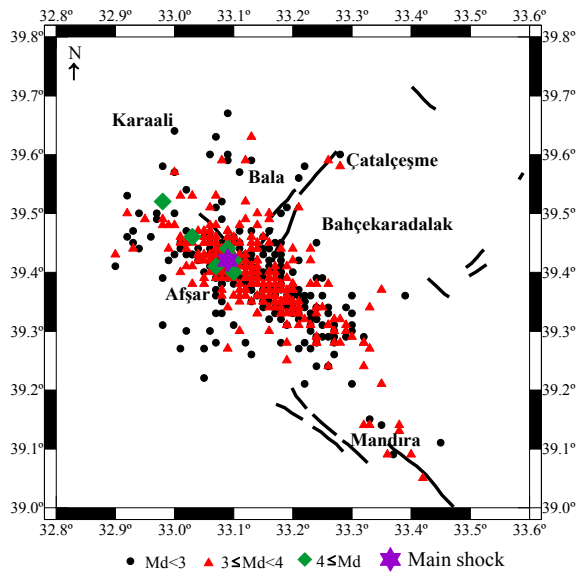

Fig. 1. Epicenter distributions of aftershocks. Magnitude sizes are marked by different symbols.

The frequency-magnitude relationship of Bala sequence is shown in Figure 2. The magnitude of completeness was taken as 2.8, based on the goodness of fit of the data. The *b*-value is calculated as 1.48±0.06 using the maximum likelihood procedure and this value is rather large. Bender (1983) presented a detailed analysis of the dependence of the *b*-value on the interval size, maximum magnitude, sample size, and the data fitting techniques. As stated in the data section, there are no aftershocks with magnitude M_D>5.0 and there are only 6 aftershocks greater than M_D4.0. Thus, this larger *b*-value may be resulted by the paucity of aftershocks whose magnitudes are larger than 5.0.

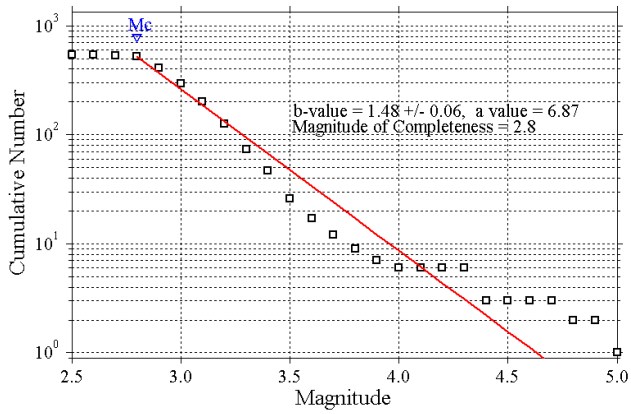

Fig. 2. Frequency-magnitude distribution of Bala aftershocks. The *b*-value and its standard deviation, as well as the *a*-value in the Gutenberg-Richter relation for the sequence are given.

Temporal decay rate of aftershocks sequence of Bala earthquake is given in Figure 3. As in the *b*-value, the maximum likelihood procedure was used to obtain the *p*, *c* and *K* parameters and the temporal decay rate was modeled by the modified Omori formula. *p*=0.75±0.03, relatively small, is calculated for the sequence assuming to be M_{min}=2.8, T_{start}=0.002. *c*-value is 0.042±0.026. The small *p*-values especially for the case of small number of aftershocks suggest that the effect of background seismicity has not completely been removed and small *p*-values have often been reported for superposed sequences. Also, for simple aftershock sequences following relatively small mainshocks, estimated *c*-values are usually small (*c*≤0.01 days). According to the *c*-value calculated above there is not incompleteness at the beginning of the sequence. So, the calculated relatively small *p*-value may be a result of the slow decay rate of the aftershock activity and small number of aftershocks.

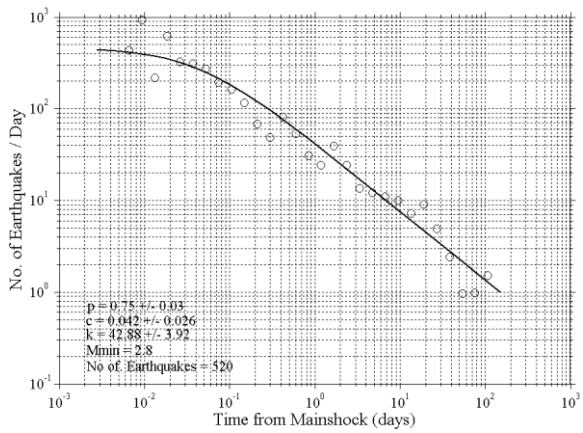

Fig. 3. Decay rate of aftershock activity versus time after the main shock of Bala (for the cases: M_D≥2.8). Some relevant parameters such *p, c* and *K*-values in the modified Omori formula, the minimum magnitude for the data and the number of earthquakes as are given.

DISCUSSIONS AND CONCLUSIONS

The statistical properties of the occurrence of aftershocks have been one of the main objects of seismological studies in connection with the processes of earthquake generation. In past studies, the frequency-magnitude distribution and decay rate of aftershock sequences have commonly been simply assigned one overall value. Utsu (1971) summarized that *b*-values change roughly in the range 0.3 to 2.0, depending on the different region. The deduced maximum *b*-values, 1.5 (or with error limits $1.30 \leq b_{max} \leq 1.64$) agrees very well with the maximum *b*-values computed from earthquake catalogues or published elsewhere in the literature. Thus, b_{max} is about 1.5 for intermediate and large shocks when considering the error limits (Olsson, 1999). The small *p*-values especially for the case of small number of aftershocks suggest that the effect of background seismicity has not completely been removed and small *p*-values have often been reported for superposed sequences. The superposed sequences consist of mostly small sized aftershocks and a portion of these may not be real aftershocks; they may only represent background seismicity (Utsu et al., 1995). Olsson, (1999) stated that *p*-value changes from 0.5 to 1.8. There are two opinions concerning with the *c*-value: one is that the *c*-value is essentially 0 and all the reported positive *c*-values result from incompleteness in the early stage of an aftershock sequence and another is that positive *c*-value do exist. Thus, for simple aftershock sequences following relatively small main shocks, estimated *c*-values are usually small ($c \leq 0.01$ days). In this study, the statistical properties of aftershocks sequence of Ankara (Bala) earthquake which occurred in Turkey in 27 December 2007 are analyzed. For this purpose, the *b*-value of Gutenberg-Richter relation, the *p*-value of Omori law are calculated using the data containing 539 aftershocks in five months' time interval, which is taken from the website of KOERI. The *b*-value, computed by using the events with completeness magnitude $Mc=2.8$, equals to 1.48±0.06. This value is very close to the maximum *b*-value explained in the literature. This larger value may be connected with the error limits and the catalogue not including the aftershocks with magnitude $M_D \geq 5.0$. Fitting the data for events with $M_{min}=2.8$ and $T_{start}=0.002$, *p*-value was obtained as 0.75±0.03, relatively small, and *c*-value 0.042±0.026. Since the aftershock activity shows a relatively slow decay rate, it is computed a relatively lower *p*-value. Also, this small *p*-value may be a result of the small number of aftershocks. According to the *c*-value calculated above there is not incompleteness at the beginning of the sequence. Thus, the simple modified Omori law is suitable for this analysis. In order to understand the physical mechanism of the *b* and *p*-values variations, more detailed future studies are needed.

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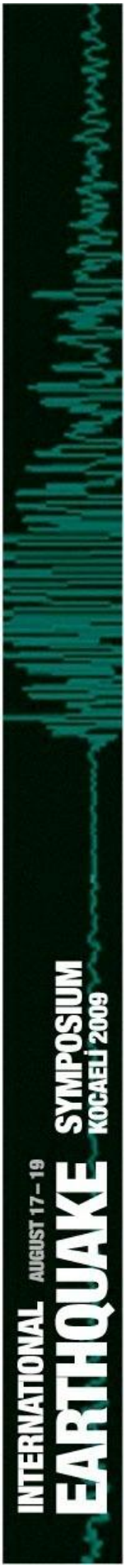

ENGINEERING

MÜHENDİSLİK

A CASE STUDY: EFFECTS OF SELECTED MODELING PARAMETERS ON PERFORMANCE EXPECTATIONS OF BUILDINGS

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Key Words: Pushover Analysis, Nonlinear Static Procedures, Seismic Response Assessment, 1999 Marmara Earthquake.

ABSTRACT During the last two decades, many competing nonlinear analysis procedures have been proposed for earthquake performance determination of engineering structures. The nonlinear response history analysis (NRHA) is accepted as the most accurate source of information for nonlinear seismic response; there are several shortcomings (i.e. amount of run time, post processing efforts, stability and convergence problems, and difficulties in idealization) in the application. Nonlinear static procedures (NSP) have been developed to determine the nonlinear seismic response in terms of demand and performance evaluation. NSPs may also provide reasonable estimates of seismic demand and inelastic behavior. However, adequacy of all proposed NSPs is limited due to the simplifications and approaches as invariant load pattern and consideration of single mode. Due to other inherent approximate assumptions, determination of local or global demands may be difficult or erroneous. Considering the effect of higher modes to the seismic response, many adaptive modal pushover analysis procedures have been proposed, recently.

In this study, a case study has been formed. Two damaged buildings (one moderately and one severely) have been selected from a building model pool that we have crafted and a number of analysis procedures have been applied to them. The buildings for the case study have been selected from among the damaged buildings in Adapazarı city center during 1999 Marmara Earthquake. Since the damage states for buildings are known, comparing them with predictions from NSP analyses, effects of different modeling parameters have been evaluated.

In the sample building models the following parameters have been changed; i. load pattern for each NSP, ii. hinge properties of frame elements, iii. infill wall modeling (considered or neglected), iv. material properties, v. P-Δ effects (included or neglected).

For each parameter combination, the "performance point" values are obtained. The results are compared. The difference between these results has also been compared with the case of considering higher mode effects (in adaptive modal pushover analysis procedures).

Comparing with the results of analyses, approximations of NSP's have been evaluated. This way, reliability of NSP Methods is evaluated and checked whether they are suitable "route signs" for engineers. Since the performance expectations for the models have been different from the "real" damage state, the study has been concluded that, there is as yet no fool-proof guarantee for agreement of pushover procedures with field observations.

INTRODUCTION

The recent research and development in Earthquake Engineering have been significantly concentrated on the idea of "performance based earthquake engineering". In order to determine the seismic response of the structures, many competing nonlinear analysis procedures have been proposed for earthquake performance determination of

engineering structures. Among these procedures, the nonlinear response history analysis (NRHA) is accepted as the most accurate source of information for nonlinear seismic response, but there are several shortcomings (i.e. amount of run time, post processing efforts, and stability and convergence problems) in the application. Therefore, nonlinear static procedures (NSP) have been developed to determine the nonlinear seismic response in terms of demand and performance evaluation. Beside its easy application NSPs may also provide reasonable estimates of seismic demand and inelastic behavior. However, adequacy of all proposed NSPs is limited due to the simplifications and approaches as invariant load pattern and consideration of single mode. Due to other inherent approximate assumptions, determination of local or global demands may be difficult or erroneous. Considering the effect of higher modes to the seismic response, different adaptive modal pushover analysis procedures have been proposed, recently.

In general, the reliability of NSP's has been evaluated comparing their expectations with the results of Nonlinear Response History Analyses (NRHA), which have been accepted as the "exact analyses". For a better comparison, experience from earthquakes should be considered because it is there that the ultimate test of correctness lies.

In this study, the results of the NSPs have not been compared with the NRHA. For the simplicity of the case study, the NSP results have been compared each other and the real response in earthquake.

Turkey experienced a destructive earthquake on August 17, 1999 that created a large number of damaged buildings. This study has been initiated from the question of "had we known one day in advance that this earthquake would occur and its ground motion, could we have estimated their damage states using the widely used NSPs?" The principal modality is to compare observed performance of buildings on the basis of field observations with estimates using nonlinear static procedures. This way a calibration of these procedures may be possible. Moderately and heavily damaged buildings have been sampled from Adapazari / Sakarya and their analytical models built for this purpose. One moderately damaged and one severely damaged building, with different geotechnical and structural attributes, have been selected from the model pool and a number of analysis procedures have been applied to them. Since the damage states for buildings are known, comparing them with predictions from NSP analyses, effects of different modeling parameters have been evaluated.

In the sample building models the following parameters have been changed: i. load pattern for each NSP (first mode shape and inverse triangular shapes for lateral loading), ii. hinge properties of frame elements (flexural behavior and increased shear effects), iii. infill wall modeling (considered or neglected), iv. material properties (cracked and uncracked sections), v. P-Δ effects (included or neglected).

For each parameter combination, the "performance point" values are obtained. The results are compared. The difference between these results has also been compared with the case of considering higher mode effects (in adaptive modal pushover analysis procedures).

For the performance assessment of buildings, the following analysis procedures have been used: NSPs of Nonlinear Dynamic Analysis of Equivalent SDOF System (Eq. SDOF), Capacity Spectrum Method (ATC-40 and FEMA-440) (ATC 1996, ATC 2005), Displacement Coefficients Method (FEMA-356 and FEMA-440) (ASCE 2000, ATC 2005), and Modified Modal Pushover Analysis (MMPA) (Chopra et al 2004).

GENERAL DESCRIPTION OF THE BUILDINGS STUDIED

In this case study, two buildings that are located in the Adapazari city center and that have experienced damage have been selected for the assessment. One moderately and one severely damaged sample buildings' blueprints have been obtained from the archives

of the Adapazari Metropolitan Municipality for modeling. For the models and analyses commercial software has been used [1].

The structural system for each of the building is reinforced concrete frame system. Basement has not been built in either building. The amount of infill walls has been decreased if the ground floor is used commercially. Thus, the sample models reflect the general situation of building stock in Adapazari. The general features of the buildings are given in Table 1.

The geotechnical information given in Table 1 has been obtained from the results of the microzonation studies held by DRM and GDDA (DRM 2003, GDDA 2004).

Table 1. General features of the buildings.

							Coordinates				
	No.	District - Cadastral Location	Built	No. of Story	Concrete f_{cd} (kg/cm^2)	Reinf. Steel f_{yd} (kg/cm^2)	Lat (deg.)	Long (deg.)	Distance to Fault (km)	Footprint Dimensions (m)	T_1 (s)
Moderate Damage	5	Tekeler 107/783/442	12.08.1994	5	95	1910	40.79496	30.39966	13.7	11,5 * 16,5	0.63
	No.	District - Cadastral Location	Built	No. of Story	Concrete f_{cd} (kg/cm^2)	Reinf. Steel f_{yd} (kg/cm^2)	Lat (deg.)	Long (deg.)	Distance to Fault (km)	Footprint Dimensions (m)	T_1 (s)
Severe Damage	7	Semerciler 54/388/32	31.12.1993	6	95	1910	40.78106	30.39536	12	11 * 11,5	0.96

	No.	Bedrock Depth (m)	NEHRP Site Class	Turkish Site Class	Vs (m/s)	T (s) Pred. Per.	Groundwater Depth (m)	Liquefaction Severity I.	Note
Moderate Damage	5	150	E	Z4	< 263	< 0.82	> 15	0	Technical Report Available
	No.	Bedrock Depth (m)	NEHRP Site Class	Turkish Site Class	Vs (m/s)	T (s) Pred. Per.	Groundwater Depth (m)	Liquefaction Severity I.	Note
Severe Damage	7	175	E	Z3	263< Vs< 371	> 1.23	0 - 5	2	Designed using Ahmet Babalıoğlu software

BUILDING 1 (MODERATE DAMAGE)

The building was located in the Tekeler district of Adapazarı. The total height of the 5-story building is 13.6 m. Moderately damaged building during the 1999 earthquake and situated approximately 13.7 km away from the North Anatolian Fault (NAF) the structure has been strengthened after the earthquake. The general view of the 3D model is shown in Fig.1.

BUILDING 2 (SEVERE DAMAGE)

The building was in Semerciler district of Adapazarı. The ground floor of the 6-story building is commercially used. The total height is 17.5 m. This building which is located approximately 12 km away from the NAF experienced severe damage during 1999 earthquake. The general view of the 3D model is shown in Fig.2.

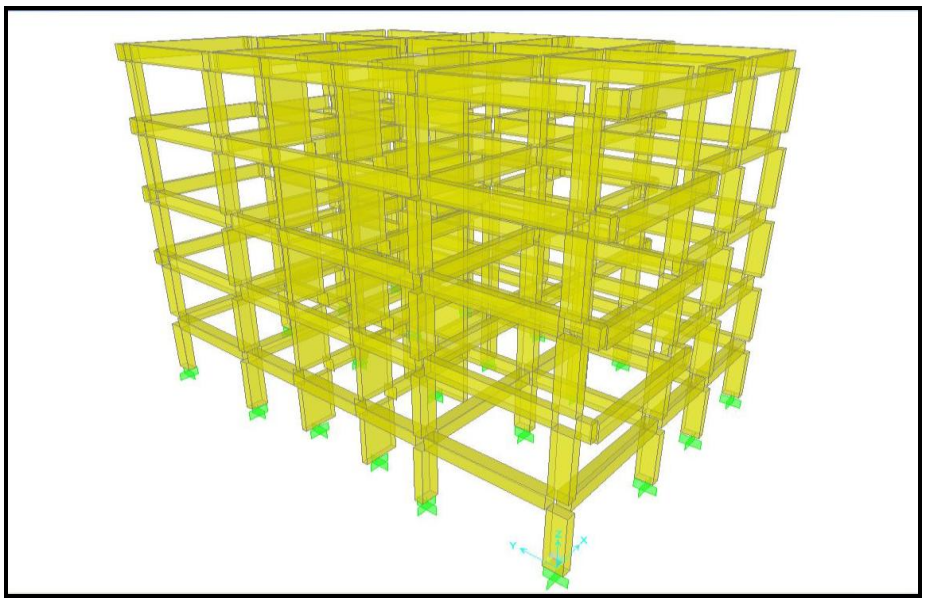

Fig. 1. 3D model of moderately damaged building.

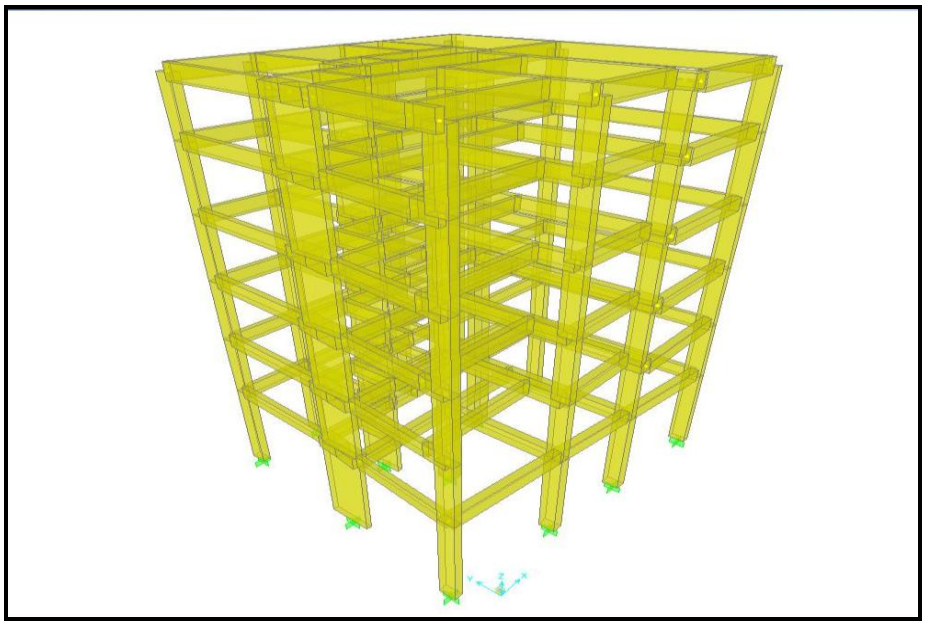

Fig. 2. 3D model of severely damaged building.

SITE SPECIFIC STRONG GROUND MOTION

Adapazarı is a city that sits entirely on deep alluvium deposits (Bakır et al 2002). Therefore, soil effects must be considered in any study related with the seismic performance of the structures.

In the study, the foundations have been assumed to be "fixed" to the soil. However, the rules of assessment procedures for "soft soils" have been considered and the "site specific strong ground motion record" has been used for entire assessments.

The site specific ground motion string which has been obtained from the original strong ground motion – recorded at Sakarya Station (SKR-EW) – is used for the analyses (Bakır et al 2002). At the Sakarya Station only one component (SKR-EW) of the motion could be recorded during the earthquake due to instrument malfunction.

The bedrock depth for the buildings studied has been assumed to be 150 m (DRM 2003, GDDA 2004, Bakır et al 2002, Bakır et al 2005) and "site specific strong ground motion record" has been taken for this depth. The site specific ground motion record is shown in Fig.3, and the corresponding response spectrum is shown in Fig.4.

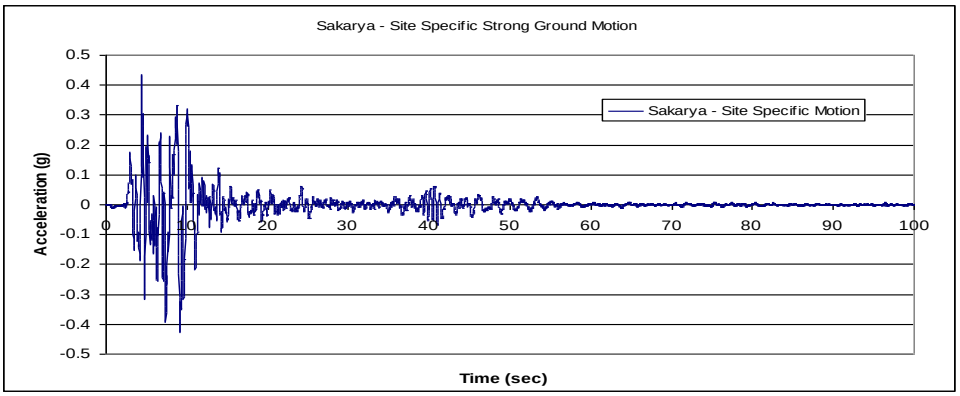

Fig. 3. Site specific strong ground motion record (soft soil – bedrock depth: 150 m).

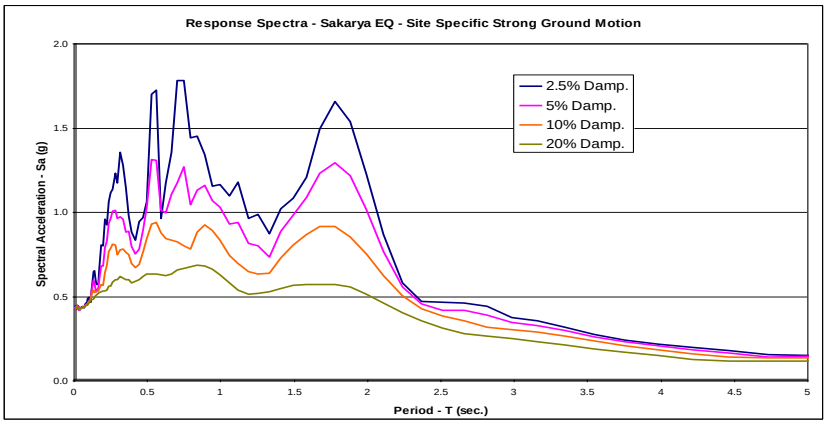

Fig. 4. Response spectra (corresponding to the site specific strong ground motion).

SEISMIC PERFORMANCE ASSESSMENT OF THE BUILDINGS

The 3D models of the buildings have been evaluated using the NSPs given in Section 1. Following customary practice the roof displacement demand parameter has been used as the seismic response parameter of the buildings. The deformation demand of each structural element differs by changing level of the roof displacement. Thus, the roof displacement is used as a global parameter for estimation of the probable damage of the building. If the demand is within narrow limits, the performance estimations can be consistent.

In order to apply the assessment procedures and for the comparison of the parameter changes, a "base model" has been determined. The base model has been formed with the base parameters; i. modal shape has been used as loading pattern, ii. cracked section have been used, iii. infill walls have not been modeled, iv. ductile hinge properties have been used, and v. P-Δ effects have been neglected. These modeling features have been changed one at a time for each comparison, assuming the others are same to the base model.

In this case study, the effects of five different modeling parameters on seismic response expectations have been investigated. These parameters are as follows;

i- Load Pattern: Triangular loading has been used instead of modal shape,

ii- Material Properties: Uncracked sections have been used instead of cracked sections,

iii- Infill Walls: Model with infill walls has been used instead of model without infills,

iv- Hinge Properties: Model with increased shear effect has been used instead of the default hinge model of the software which assumes the failure controlled by flexure,

v- P-Δ Effects: P-Δ effects are included in the model.

The comparison for the material properties has been applied, considering the changes of stiffness in reinforced concrete elements. In order to calculate the reduction of the stiffness of member according to the cracking, the provisions given in Turkish Earthquake Code (Sect. 7.4.13) have been used (MPWS 2007).

The effect of infill wall modeling has also been studied, using the infill wall model given in literature. The stress-strain relation used for the model is shown in Fig.5. (Binici et al 2007).

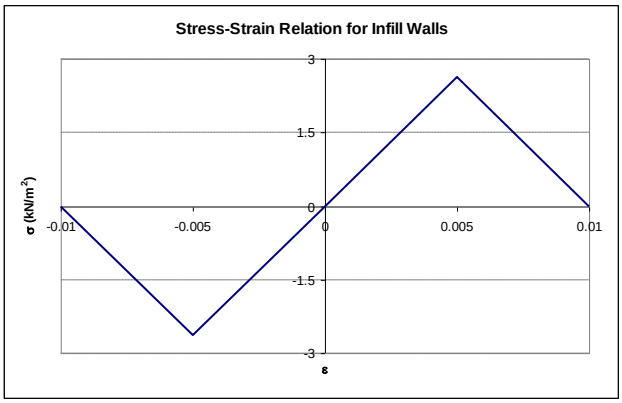

Fig. 5. Stress-strain relation for the infill walls.

In FEMA356 (ASCE 2000), a generalized force-deformation relation for concrete elements shown in Fig.6 is given. In the same document, nonlinear modeling parameters and numerical acceptance criteria for the structural elements controlled by flexure are also available. The acceptance criteria have been given according to the amount of axial loading, transverse reinforcement and amount of shear force for columns, and according to the amount of longitudinal reinforcement, transverse reinforcement and amount of shear force for beams. In this study, transverse reinforcement, amount of axial loading and longitudinal reinforcement have been assumed to be constant. Considering the shear effects, new acceptance criteria have been selected from FEMA356. In this case, the shear force ratio [$V / (b_w\, d\, \sqrt{f'_c})$] assumed to be two times is increased according to the default case.

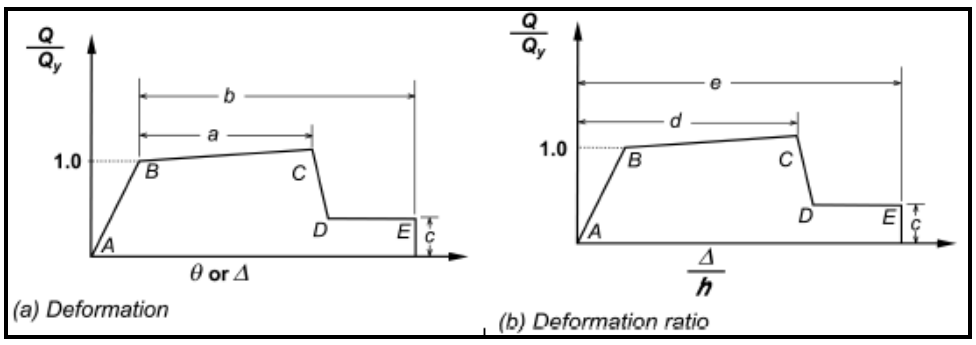

Fig. 6. Generalized force-deformation relations for concrete elements or components (ASCE 2000).

The buildings whose general structural and geotechnical features are given in Table-1 and in Section-2 of this study have been assessed using the approximate nonlinear procedures. The pushover results and the performance estimates of NSPs for each of the building are given in the following sections.

BUILDING 1

The building during the 1999 earthquake has been strengthened after the earthquake. Pushover curves with performance estimations of the building in X and Y directions are shown in Fig.7 and Fig.8, respectively.

The estimations of Eq. SDOF, DCM (ASCE 2000, ATC 2005) and MMPA in X direction fit the moderate damage state description of the building model with uncracked sections only. In the Y direction, however, only the estimations of Eq. SDOF and MMPA agree with moderate damage of the two building models (base model and model with infill walls). For the other models and assessment procedures, estimations imply that the building would have been severely damaged during the earthquake.

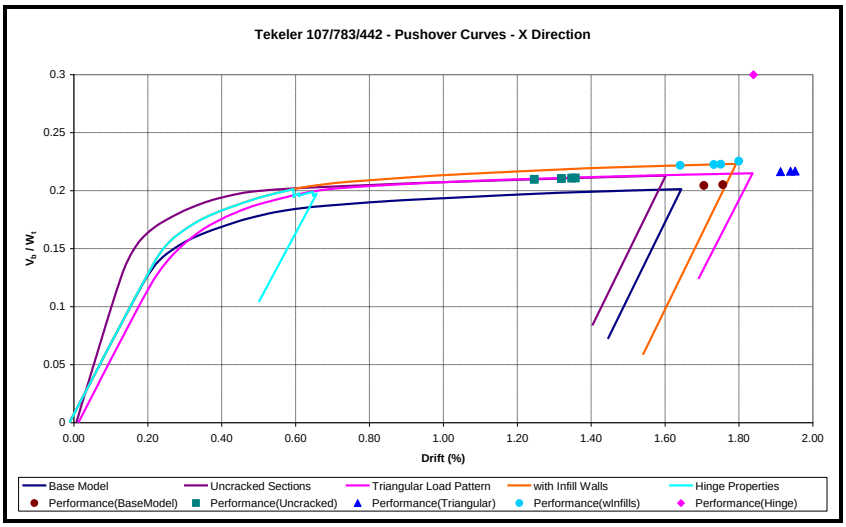

Fig. 7. Pushover curves and the performance estimations for Building 1 in X direction.

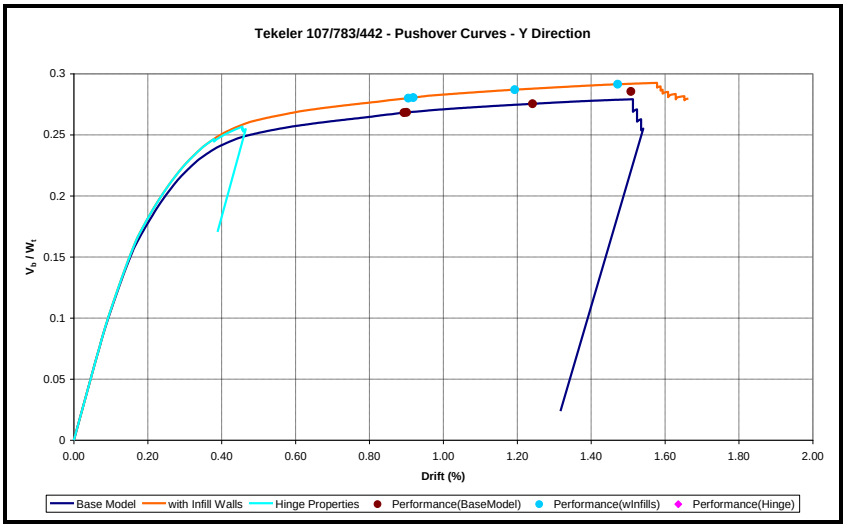

Fig. 8. Pushover curves and the performance estimations for Building 1 in Y direction.

BUILDING 2

Pushover curves with performance estimations of the building in X and Y directions are shown in Fig.9 and Fig.10, respectively.

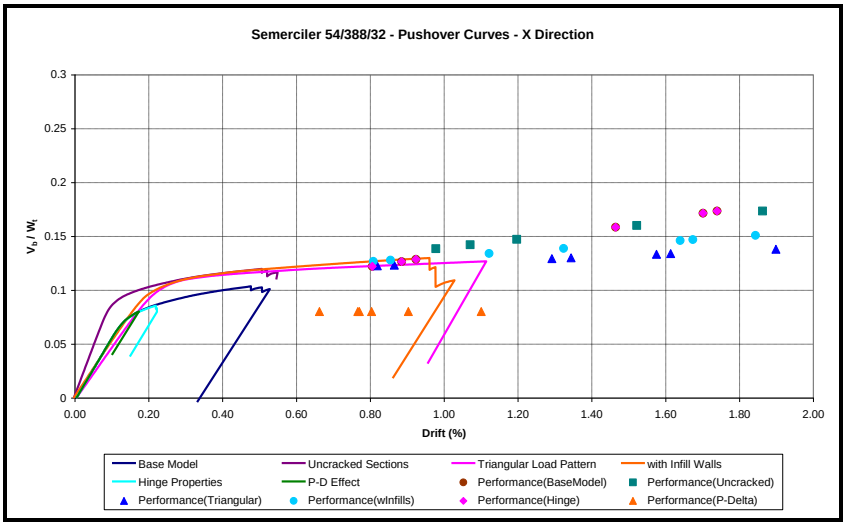

Fig. 9. Pushover curves and the performance estimations for Building 2 in X direction.

The estimations of DCM (ASCE 2000, ATC 2005) in X direction agree with the severe damage of the building model with triangular loading pattern and model with infill walls only. On the other hand, in Y direction estimations of DCM (ASCE 2000, ATC 2005) for the building model with uncracked sections and the estimations of Eq. SDOF for the model with triangular loading pattern reflect the severe damage of building. The other performance point estimations of assessment procedures are beyond the building capacity, which imply that the building would collapse.

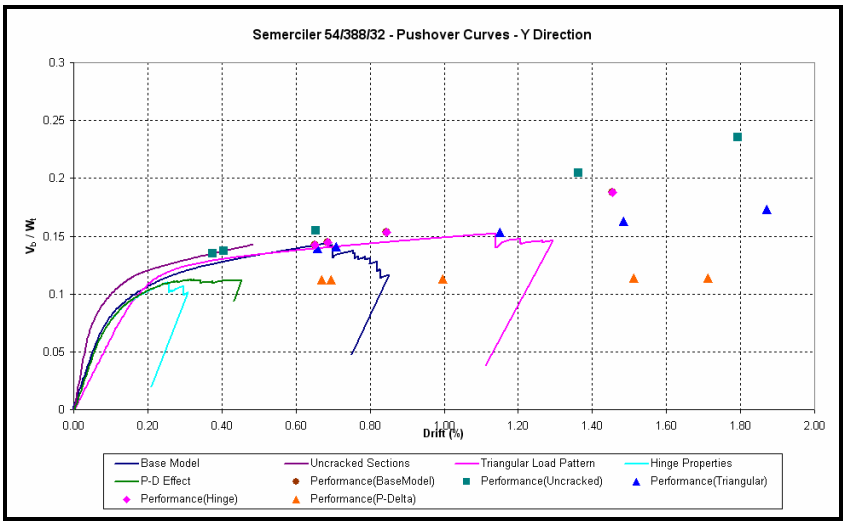

Fig. 10. Pushover curves and the performance estimations for Building 2 in Y direction.

CONCLUSIONS

The study has concentrated on the modeling parameters of the buildings for the application of NSPs to two samples from the large building stock in Turkey. The results of approximate assessment procedures with different modeling parameters are compared with the real building performance records of selected buildings. Damage states for buildings are known, and comparison with predictions from analyses using NSPs is done.

This way, reliability of NSP Methods and the effect of some modeling techniques on the expectation results have been evaluated and checked whether they are suitable "route signs" for future applications.

Assessment procedures for idealized building models are compared with more sophisticated procedures. These may be found satisfactory. However, the results of same procedures for real buildings can be very misleading. The building assessment examples given in this study clearly show such misleading results. The results of the analyses are seriously affected by inadequate information about the soil effects and the approximations for the structural modeling. On the other hand, the workmanship effects and shear failure or bonding effects can not be modeled exactly. This shortcoming is more acute if the building collapses and its ruin are removed.

The pushover curve results which have been given in the preceding section show that the effect of parameters used for modeling the building is deciding. While we have considered only one accepted ground motion input, and carried out no response history analyses the indications show that many "standard" methods may produce erroneous results for future applications such as loss estimates. We can not qualify whether these would be correct in the mean either. The differences in the results of pushover curves have been summarized in Table.2. The modeling parameters produce grave effect on both base shear and nonlinear displacement capacity of buildings.

Table 2. Summary of the results for pushover curves.

		Base Shear Capacity	Displacement Capacity
Base Model vs...	Uncracked Sections	+ 15 %	~
	Triangular Load Pattern	~ in one case + 5 %	+ 50 %
	with Infill Walls	+ 3~5 %	+ 5~10 % in one case + 80 %
	Hinge Properties	~	decreased seriously
	P-D Effect *	- 10 % especially in nonlinear region of the pushover curve.	- 80 %

* : Was no observed for one of the two buildings

The first building in this study (2.1) experienced moderate damage and the other building (2.2) was severely damaged during the earthquake. However, most of the analyses results did not predict the level of damage accurately. Using these results it is not possible to determine the seismic response and the damage of the buildings before the occurrence of earthquake.

The study concludes that there is as yet no assurance for the compatibility of pushover procedures with field observations. It is necessary to investigate the proposed assessment procedures in a detailed manner and to check the results for "real buildings". The approximate nonlinear static assessment procedures should be improved for reliable damage estimation. Unless field experience is brought to bear on refinement of the methods that have been developed they run the risk of providing disservice to the engineering community. It is only through back-calculation and empirical field comparison that nonlinear static procedures may become reliable engineering tools.

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APPLICATION OF CAPACITY SPECTRUM METHOD FOR PERFORMANCE ASSESSMENT OF AN INDUSTRIAL BUILDING STRUCTURE LOCATED IN AN EARTHQUAKE-PRONE REGION IN TURKEY

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Key Words: Industrial building, Seismic performance, Earthquake-prone region, Capacity spectrum method.

ABSTRACT Industrial buildings in Turkey are mainly constructed as precast concrete building structures. Marmara Earthquake (August 17th, 1999) and Duzce Earthquake (November 12th, 1999) occurred subsequently negatively affecting the most industrialized part of Turkey. Excessive levels of damage were observed at precast concrete structures after these earthquakes. The damage was mainly observed as destruction at column-girder connections or column failures. Investigation of seismic behavior of precast concrete structures located at this highly earthquake-prone region in Turkey is needed. In this study, seismic performance of a precast concrete industrial building structure, which is built in correspondence with the current design code and construction practice in Turkey, is investigated. The building is subjected to several ground motions from recent destructive earthquakes. The capacity spectrum method is applied in order to evaluate the seismic performance of investigated precast concrete building. In addition, the results of capacity spectrum method application are discussed for the examined industrial building structure.

INTRODUCTION

Major earthquakes repetitively occur in Turkey. In late 90's, two earthquakes which are namely Marmara Earthquake (August 17th, 1999) and Duzce Earthquake (November 12th, 1999) occurred at a highly industrialized part of the country. After these two events, excessive levels of damage were observed at precast concrete structures. Regarding this observation, there is a need for investigation of seismic behaviour of precast concrete structures since the observed damage was mainly in the form of destruction at column-girder connections or column failures. In this study, the results of application of capacity spectrum method on a precast concrete industrial building, which represents the current construction practice for industrial buildings in Turkey, is discussed. The building is designed considering the current seismic code in Turkey.

There has been research (Ozturk and Demiralan, 2007, Ozturk et al., 2008, Demiralan, 2009, Sadak, 2009) aiming at determination of seismic behavior of precast concrete building structures. In this study, the seismic behaviour of a precast concrete industrial structure will be investigated considering the capacity spectrum method (CSM) which was provided in ATC-40 (1996) for estimating the peak inelastic responses. It is a performance-based seismic analysis technique which compares the capacity of the structure with the demands on the structure (Chopra and Goel, 1999, Fajfar, 1999, Fajfar, 2000, Kim et al., 2005). It provides a graphical representation of the expected seismic performance of existing or retrofitted structure by the intersection of structure's capacity spectrum with a response spectrum representing the displacement demand on the structure due to earthquake. The intersection point refers to the performance point which is the intersection of both the spectral response curve and the nonlinear demand curve. The CSM is currently being used to solve various earthquake engineering problems

(Guyader and Iwan, 2006, Casarotti and Pinho, 2007, Zhao and Zhang, 2007, Gencturk and Elnashai, 2008, Barbat et al., 2008).

The capacity spectrum method is applied in order to assess the seismic performance of a precast concrete industrial building. The building is located at a region of high seismicity (seismic zone 1). It is constructed considering the current earthquake design practice. In design of the building and evaluation of its seismic performance, the regulations provided in TS 498 (1987), TS 9967 (1992), TS 500 (2000) and Regulation for Buildings to be Constructed at Earthquake-Prone Regions (2007) are applied. The building is subjected to different ground motions which were obtained during recent earthquakes in Turkey (i.e. Ceyhan 1998, Marmara 1999, Duzce 1999). The capacity spectrum diagram is obtained and discussed for the investigated building upon application of the capacity spectrum method.

INFORMATION ABOUT THE INVESTIGATED BUILDING

In this study, an industrial building which is designed to be built at seismic zone 1 is investigated considering the application of capacity spectrum method (Figure 1). In its design current seismic codes in Turkey (TS 498, 1987, TS 9967, 1992, TS 500, 2000) and Regulation for Buildings to be Constructed at Earthquake-Prone Regions (2007) are used.

The building has a length of 31 m in X direction and 62 m in Y direction. The columns are 4,5 m in height. In X direction there are two spans with span lengths of 21 m and 10 m, respectively. In Y direction there are seven spans with a span length of 8,85 m each. Total weight of the structure is around 5000 kN.

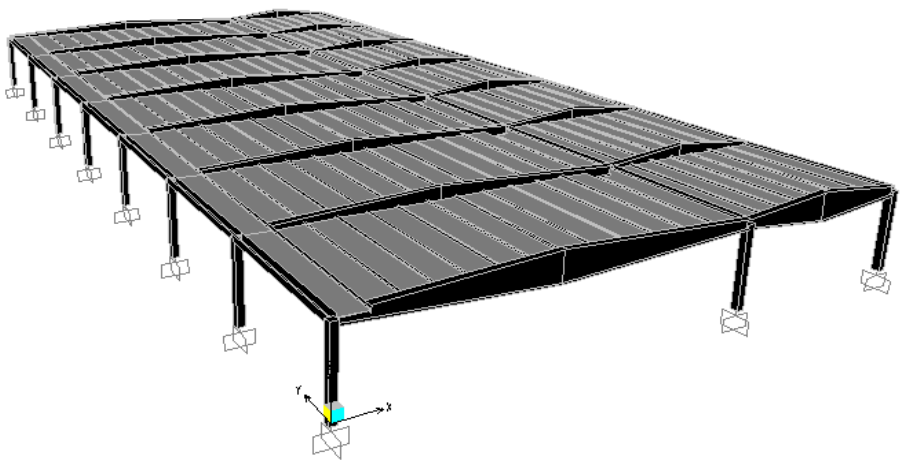

Fig. 1. Structural System of the Building (Demiralan, 2009).

MODAL AND RESPONSE SPECTRUM ANALYSIS OF THE STRUCTURE

For the analyses of the structure, SAP2000 Program (Computers and Structures Inc., 2000) is used. First four structural periods obtained as a result of modal analyses are provided in Table 1. In addition, a response spectrum analysis is applied. In the analyses, a damping ratio of 5 % is considered. The periods of first two modes of the Building are 0.53 sec both in the short direction (Mode 1) and in the long direction (Mode 2).

Table 1. Structural Periods of the Building (Demiralan, 2009).

Mode	Period (sec)	Mode	Period (sec)
1	0.53	2	0.53
3	0.47	4	0.33

The soil type on which the Building is constructed is defined as Z2 with T_A= 0.15 sec and T_B= 0.4 sec, respectively. The corresponding spectrum function is given in Figure 2 and spectrum constant, *K* is evaluated using Equation 1. In the analysis, effective ground acceleration constant, A_O value is 0.4, building importance constant, *I* value is 1.0 and building ductility constant, *R* value is taken as 3. *K* value is evaluated as 1.31 m/sec^2, accordingly.

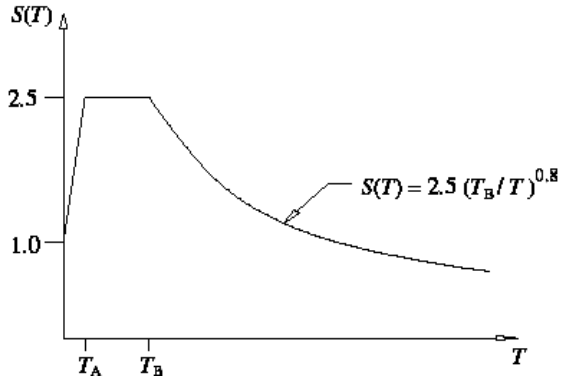

Fig. 2. Spectrum Function.

$$K = \frac{A_O \cdot I \cdot g}{R} \tag{1}$$

Spectrum constant, *S(T)* is calculated to be 1.88 for the Building.

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} \tag{2}$$

DETERMINATION OF PERFORMANCE POINT USING CAPACITY SPECTRUM METHOD

Nonlinear deformations, which increase the structural damping and decrease the displacement demand, occur under increasing earthquake loads. In capacity spectrum method, due to the occurrence of nonlinear structural deformations elastic demand spectrum is reduced. The equilibrium point of capacity and demand, which is named the performance point, is defined afterwards. The realization of performance target is controlled at this point. In capacity spectrum method, determination of three essential parameters, which are capacity (Figures 3-4), displacement and acceleration demands (Figures 5-6), and performance point (Figure 7) are required.

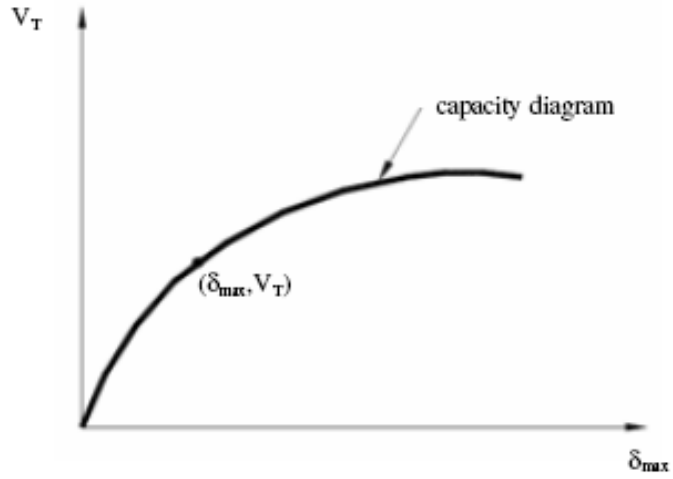

Fig. 3. Capacity diagram.

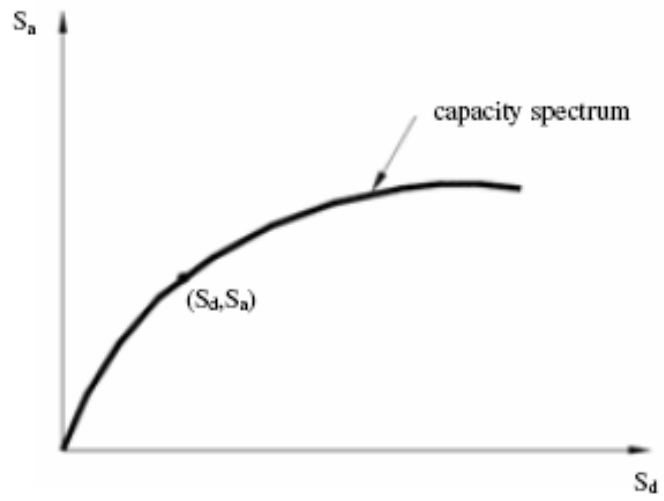

Fig. 4. Capacity spectrum.

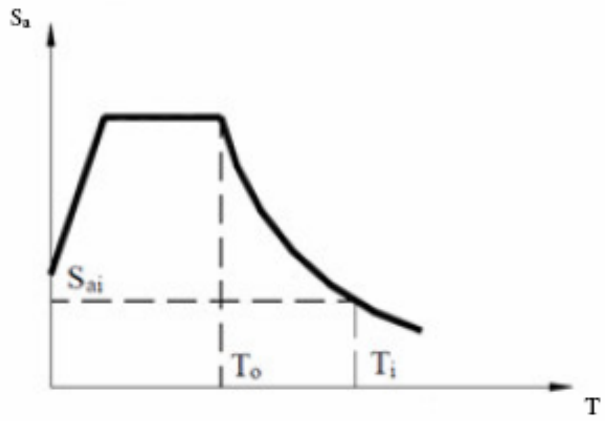

Fig. 5. Elastic demand spectrum.

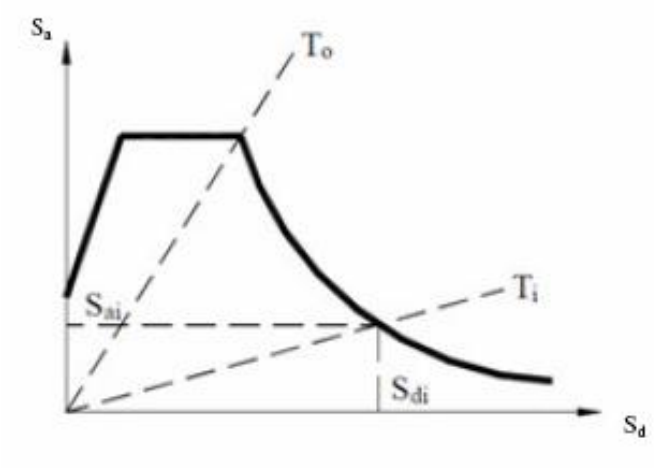

Fig. 6. Spectral acceleration – spectral displacement demand spectrum.

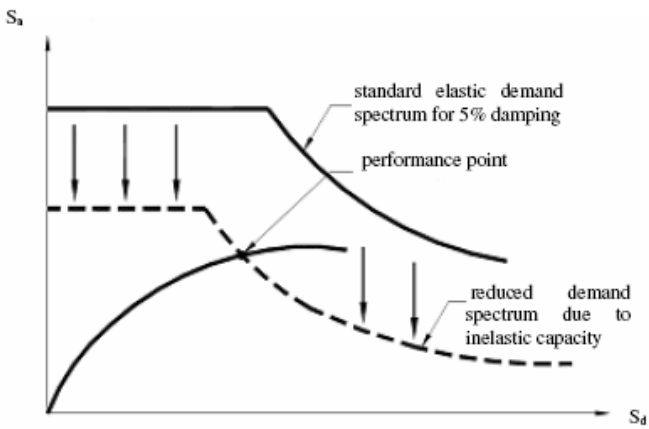

Fig. 7. Determination of performance point using capacity spectrum method.

In order to compare the capacity curve with the demand spectrum, there is a need for its conversion to spectral format which is composed of spectral acceleration, S_a and spectral displacement, S_d. However, since the demand spectrum is for a single-degree-of-freedom system there is a need for transformation of capacity curve for multi-degree-of-freedom system into an equivalent single-degree-of-freedom system.

The coordinate values of the performance point are tabulated in Table 2. The performance point of the Building has a spectral displacement value of 0,09 m and a spectral acceleration value of 0,15g for a base shear force, V of 1040 kN and displacement of 0,09 m (Table 2). The capacity spectrum curve and the corresponding performance point for the Building are shown in Figure 8.

Table 2. Performance point coordinate values for the Building (Sadak, 2009).

Parameter	**V, kN**	**D, m**	**S_a, g**	**S_d, m**
Value	1040	0.09	0.15	0.09

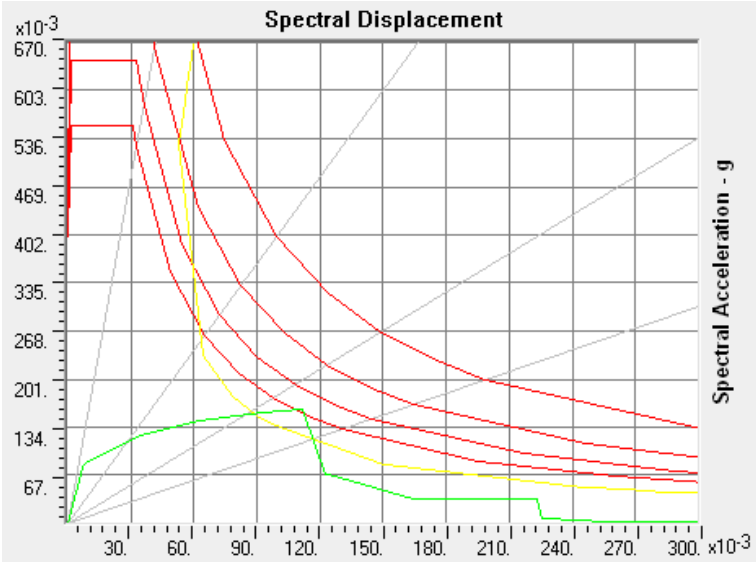

Fig. 8. Determination of performance point using capacity spectrum method for the Building (Sadak, 2009).

CONCLUSIONS

In this study, seismic performance of a precast concrete industrial building is evaluated and discussed using the capacity spectrum method (CSM). The building is located at seismic zone 1 and is designed according to the current design codes in Turkey. The application of capacity spectrum method provides the performance point which has a spectral displacement value of 0,09 m and a spectral acceleration value of 0,15g for a base shear force, V of 1040 kN and displacement of 0,09 m for the Building. It is provided that application of the capacity spectrum method provides an effective tool in determination of the performance point for the investigated precast concrete industrial building and its seismic performance assessment.

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APPLYING SPAC METHOD TO DETERMINE SHEAR WAVE VELOCITY, A CASE STUDY IN MILAD TOWER, TEHHRAN

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Key Words: Spatial autocorrelation coefficient method (SPAC), Microtremor, site effect, S-wave structure

ABSTRACT The dynamic soil properties such as shear wave velocity and geometry of the soil layers could produce large amplification and are essential in site effect studies. Therefore, it is essential to identify the soil profile up to depth where it is difficult or even impossible to be explored by the conventional seismic exploration methods in engineering practice. Spatial autocorrelation coefficient method (SPAC) originally introduced by Aki (1957) is a useful method for determining S-wave profile in sedimentary basin because of its simple data acquisition, analysis, low cost and large penetration. In the present study, SPAC method is applied to determine the structure of sedimentary materials nearby Milad Tower at Pardisan Park, Tehran (Iran).

INTRODUCTION

The relevance of the variations in elastic properties of the shallow structure on the dynamical response of the ground to a seismic excitation (site effects) has been extensively shown as cause of local variations in the seismic intensity even in cases of moderate earthquakes. Therefore a detailed knowledge of the physical properties of shallow structures is a valuable information for the simulation of seismic scenarios. Scenario results have a direct applicability to seismic risk management particularly when combined with data of building behavior characteristics and their geographical distribution. Array observation of microtremors is a passive useful method for determining s-wave velocity structure in sedimentary basin with large penetration because of simple data acquisition, analysis and low cost. The spatial autocorrelation (SPAC) method, first presented by Aki (1957), is a technique of geophysical exploration that is popularly used to infer phase velocities of surface (Rayleigh and Love) waves using records of microtremors (ambient vibrations) from a circular array of seismic sensors deployed on the ground surface. (Henstridge, 1979; Okada et al 1997; Apostolidis et al, 2004; Asten, 2003, Kudo et al., 2002,…) extended SPAC method to an exploration method which provides reliable profiles that are useful for engineering studies and strong motion simulations.

THEORETICAL BASIS OF SPAC METHOD

The SPAC method is a "Microtremor Survey Method" (MSM), which makes use of microtremors found in abundance anywhere on the surface of the earth (Okada, 1998, 2003; Okada et al., 1990). This method is based on theory developed by Aki (1957) to comprehend the relationship between the temporal and spatial spectra of seismic waves. This theory has also formed the foundation for the SPAC method, which has become the key to successful extraction of dispersion characteristics of Rayleigh waves from microtremors. Developing by Henstridge, the relationship between the spatial autocorrelation coefficient and the phase velocity of fundamental mode Rayleigh waves

was determined. Okada et al. extended it to an exploration method that is currently called the SPAC method.

Suppose two wave forms (vertical component of microtremor records) of u(x,y,t) at the centre of array and u(x′,y′,t) at the station (x′,y′) on the circumference (figure-1). a space-correlation function ($S(r,\theta)$) is defined as:

$$S(r,\theta)=E[u^*(x,y,t)u(x+r\cos(\theta),y+r\sin(\theta),t)] \tag{1}$$

where E[ ] denotes the ensemble average in which the asterisk (*) stands for the complex conjugate. If the space correlation function in Eq. (1) is averaged over all the azimuths, the azimuthally averaged spatial correlation ($\bar{S}(r)$) is given as:

$$\bar{S}(r)=\frac{1}{2\pi}\int_0^{2\pi} S(r,\theta)d\theta \tag{2}$$

If the power spectrum $h(\omega)$, is estimated for the vertical component of microtremors within the array space, the averaged spatial correlation function can be related to the power spectrum as:

$$\bar{S}(r)=\frac{1}{\pi}\int_0^{\infty} h(\omega)J_0\left(\frac{\omega}{c(\omega)}r\right)d\omega \tag{3}$$

where $\omega=2\pi f$ is the angular frequency, f the frequency, $c(\omega)$ the phase velocity at angular frequency; and J_0 the Bessel function of the 1st kind of order zero. When microtremor records from the array are band-pass filtered over a narrow bandwidth centered at ω_0 ; the spatial autocorrelation function (or the spatial autocorrelation coefficient) normalized with the power spectrum $h(\omega)$, $\bar{S}(r,\omega_0)$ is given as:

$$\bar{S}(r,\omega_0)=J_0\left(\frac{\omega_0 r}{c(\omega_0)}\right) \tag{4}$$

Therefore, the SPAC coefficient at frequency ω_0 is related to the phase velocity $c(\omega_0)$. Then, phase velocity is obtained for every frequency that results dispersion curve for Rayleigh waves which is finally inverted in order to estimate the subsurface Vs velocity profile.

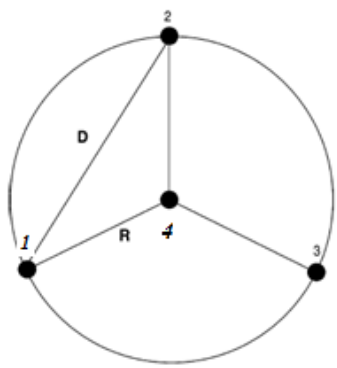

Fig. 1. Geometry of the 4-station SPAC array. Distances R and D were used for better resolution. R (internal distance) indicates the distance between the central and outer stations while D (external distance) is the distance between two outer stations.

Furthermore, To obtain more resolution results at all depths it is needed to apply this array at different sizes.

GEOLOGICAL SETTING

City of Tehran is located at Tehran Plain which consists mainly of Quaternary formations. These formations are often the result of erosion and redeposition of former sediments. It extends to the south as a young fan and generally consists of unsorted fluvial and river deposits. Both, the effects of climatic processes and young tectonic activity, caused an alluvium of various types, thicknesses, and grain sizes to be formed. Rieben (1955), and then Pedrami (1981), divided the Tehran Plain into five units as shown in Figure-2. These units include units A and Bn in the north, unit Bs in the south, unit C in the north, west, and center, and unit D in the center and south of the Tehran Plain. The borders of these units in the north of Tehran have been further defined by Abbasi and Shabanian (1999). The general characteristics of the different units in the Tehran Plain are shown in Table-1.

Table 1. Characteristics of different units in the Tehran Plain (Shafiee, 2007).

Unit	Period	Formation	Constituting materials
A	Plio-Pleistocene	Hezardareh	Conglomerate with silt–sand–gravel and silt–clay mixtures
Bn	Quaternary	Hezardareh	Cobble, boulder, gravel, and sand
Bs	Quaternary	Kahrizak	Silty sand
C	Quaternary	Kahrizak	Gravel, sand, silt, and clay
D	Quaternary	Kahrizak	Silt and clay

FIELD MEASUREMENTS BY APPLYING SPAC ARRAYS

The studied site is shown in the geological map of Tehran city (figure-2) which is Located at the border of unit A and unit B at the western part of the Pardisan Park, about 4km away from Milad Tower, Tehran. Array measurements were taken at the relatively quiet residential part of the city Tehran, besides Farahzad Valley. Microtremors data were recorded in three concentric circular arrays with radius of 11, 30 and 60 meters for 60-140 minutes. The recording instruments were four Guralp CMG-6T with GPS timing (Only vertical data were considered for SPAC estimation).

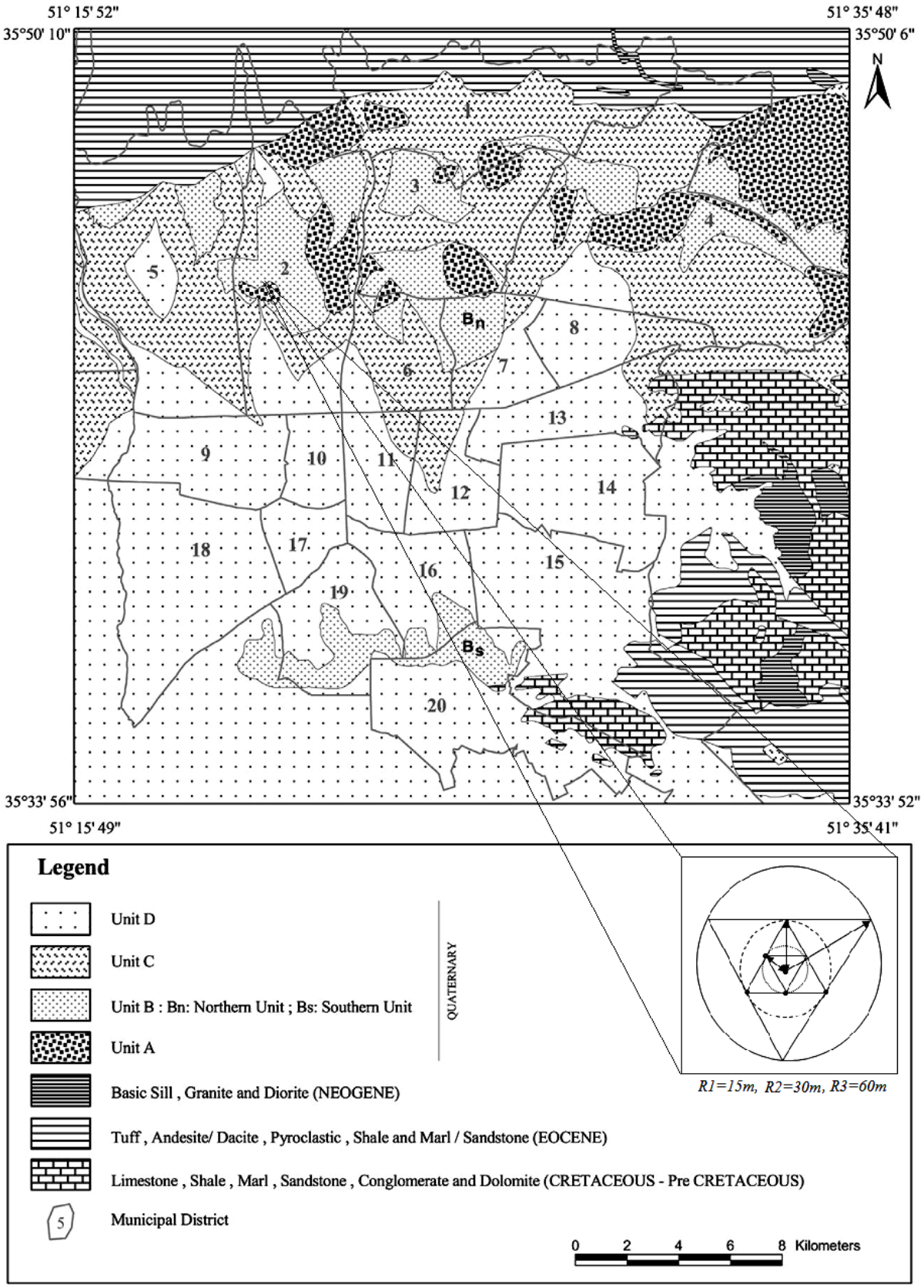

Fig. 2. Location of the studied array in geological map of Tehran city.

ANALYSIS OF MICROTREMORS RECORDS

First of all, the obtained waveforms from all stations were band-pass from 0.3 to 20Hz and corrected for baseline and instrument responses. Then Several 4096 points (40.96sec) windows were selected for stationary and their power spectral densities (PSD) were calculated. (Figure-3-a) shows power spectral densities of intermediate array (R=30m) for one statation and different time windows. Comparisons of different stations' PSDs revealed the frequency range in which the microtremor signal appears to be stationary and therefore can be used to estimate the phase velocities (Figure-3-b). In addition to, the usable frequency ranges of interest were cross-checked through a comparison of the frequency coherence function estimated for different stations. The similarly shaped power spectra calculated for one time window (or mean of all windows) of the four stations and their coherency functions are shown in Figure-3-c (for internal distances) and Figure-3-d (for external distances). Then, the SPAC coefficients were calculated in the selected frequency ranges corresponding to arrays size. Afterward, averaging over all pairs of stations, we obtained SPAC coefficient curve. (Figure-3-e) and (Figure-3-f) shows SPAC coefficients for internal and external pairs of intermediate array.

Similarly, we obtained PSDs, coherencies and SPAC coefficient for small and large arrays. PCDs and coherency functions of small, intermediate and large arrays imply that the recorded ambient noise is stable in the frequency range from 1.2 to 8Hz. Then, Based on the assumption that microtremors mainly consist of the surface waves, dispersion curves of Rayleigh wave were extracted using SPAC coefficients. In order to, the best fit of the Bessel function or the mean phase velocity was obtained using the mean SPAC coefficients by the method of least square. (Figure-4) represents the experimental phase velocity dispersion curve of each array. We also set these three curves to obtain one combined phase velocity dispersion curve (green thick line in Figure-4).

INVERSION OF S-WAVE PROFILE

Surface-wave dispersion analysis is one method used to extract a subsurface structural model. In inversions of dispersion curves for observed surface waves, a structural model is determined by fitting the observed dispersion curves with a theoretical curve. A dispersion curve for a surface wave propagating in a horizontally layered structure can be calculated by a matrix method (e.g., Haskell, 1953). Generally, a dispersion curve is a nonlinear function of shear- and/or compressional-wave velocities, densities, and thicknesses for each layer. To invert a dispersion curve into these parameters, we used genetic algorithm or GA obtained by S. Sugi that is a robust algorithm using an analogy to population genetics. As we discussed before, we considered a Rayliegh wave propagating in a horizontally layered half-space. In the inversion, we fixed the density of the layers and chose the shear-wave velocities and thicknesses of the layers as the parameters to be determined, because it is known that they significantly affect the dispersion curve. In the GA, it is better to encode these physical parameters gene type. In this case, search areas for the S-wave velocity and thickness for each layer should be defined. Also, each parameter in the search area could be digitized with an n-bit binary string. Then, the binary strings for all parameters in a subsurface structural model were concatenated to generate a "chromosome" or structural model to be used in the genetic operations.

This algorithm can simultaneously search both globally and locally for an optimal solution by using several models. In analogy to genetics, GA consists of three main operations named: selection, crossover, and mutation. These operations were applied to initial population which is generated by using of random numbers to produce a new population with the same population size. By iteration of these three genetic operations, the initial population approached a global optimal solution. These calculations were iterated until the number of renewed generations predetermined number, which was 70 in our case.

Using the GA, Total dispersion curve resulting from combination of the all arrays was inverted. Figure-4 shows final S-wave velocity structure with the minimum misfit.

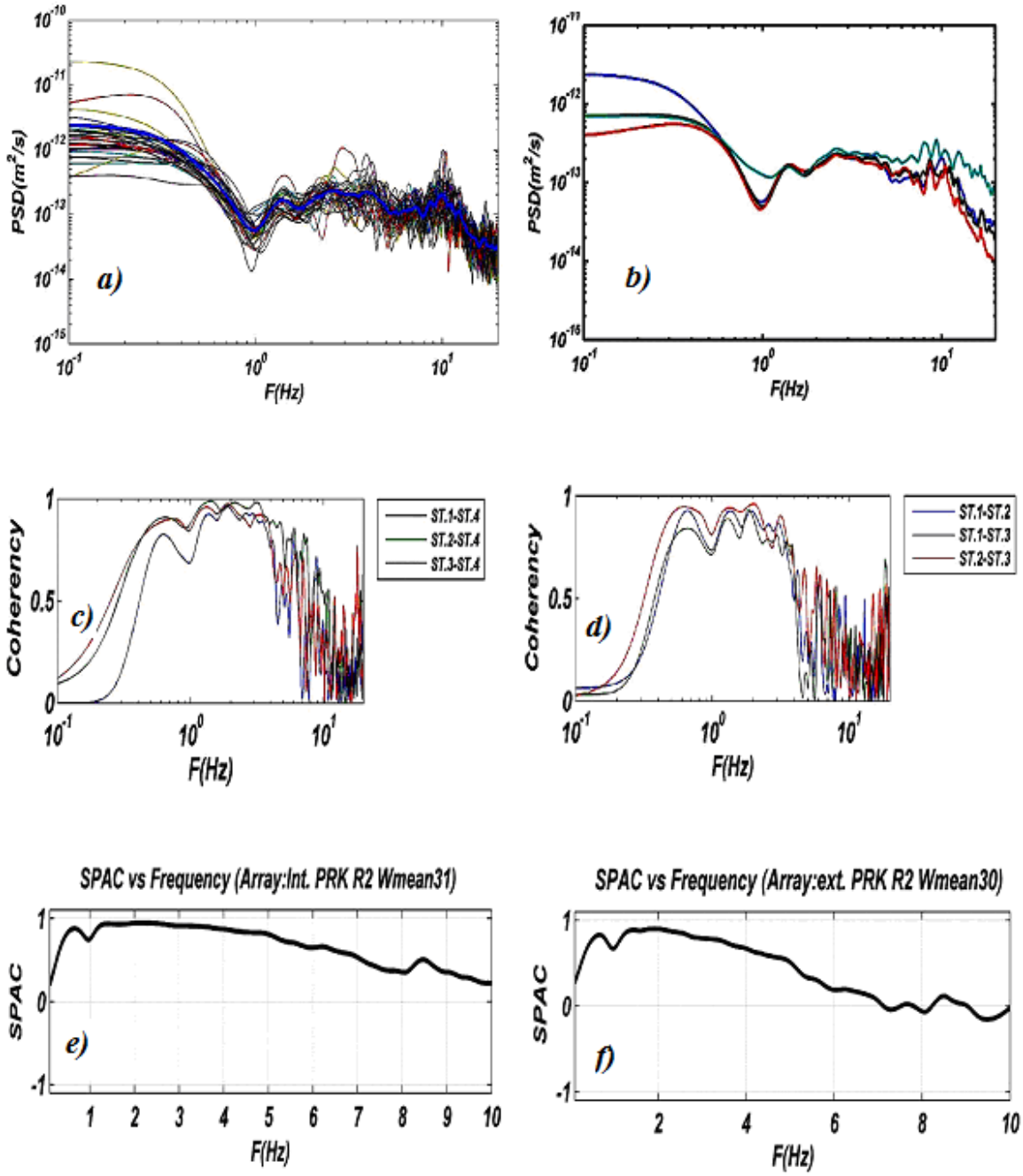

Fig. 3-a) PSDs of one station for intermediate array (R2=15m). **b)** Mean PSDs of 4 stations **c)** Mean coherency functions of 4 stations for internal pairs. **d)** Mean coherency functions of 4 stations for external pairs. **e)** Mean SPAC coefficients (Internal pairs). **f)** Mean SPAC coefficients (external pairs).

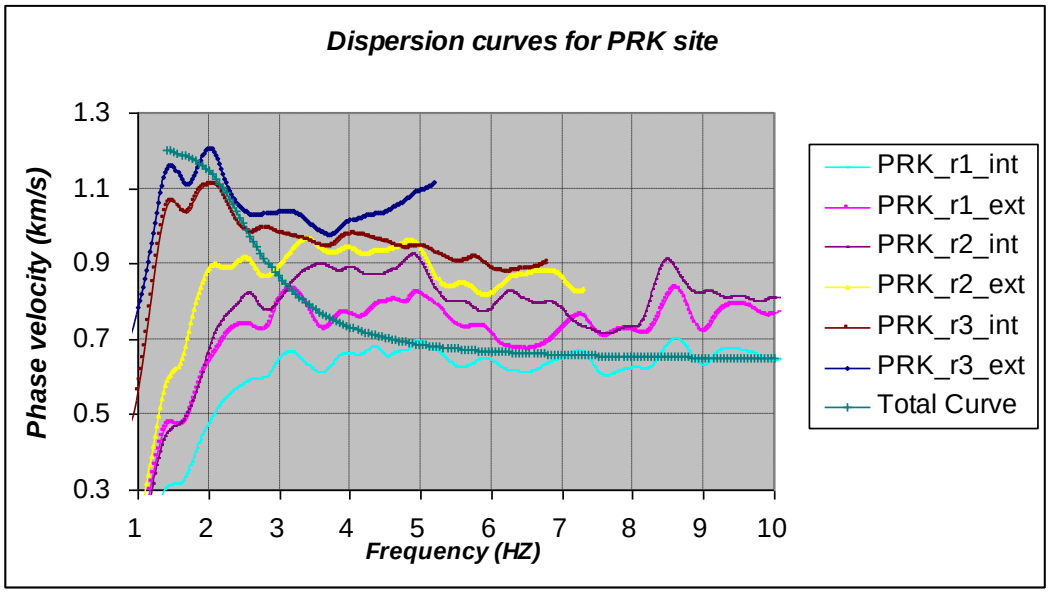

Fig. 4. Dispersion curves for small, intermediate and large arrays and Total Dispersion curve (thick green line).

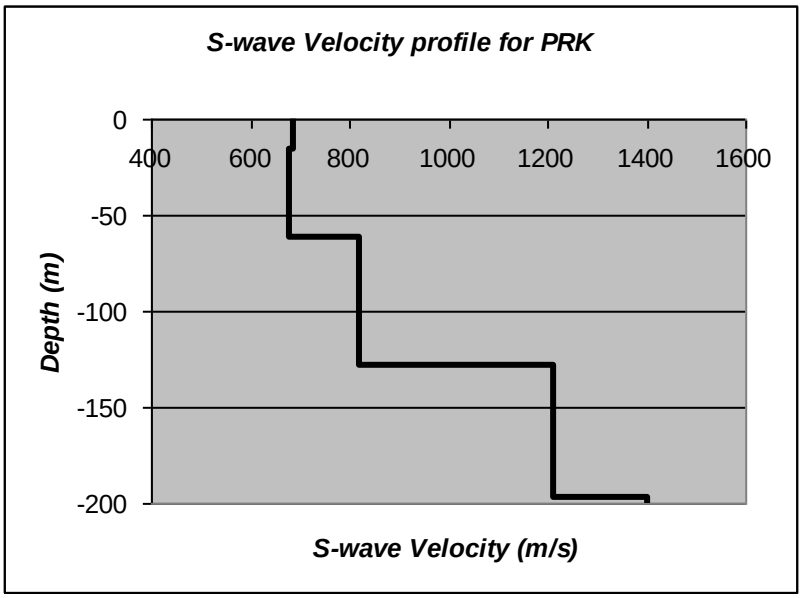

Fig. 5. S-wave velocity structure.

CONCLUSIONS

In this study, SPAC method was used to estimate the S-wave structure of sedimentary materials in north-western part of Tehran city up to the depth of 200 meters which could be worthy information for further earthquake geotechnical studies. Furthermore, we provided additional resolution in the near surface by using small array radius (R1=15) and larger arrays (R2=30m and R3=60m) for better sampling of lower frequencies and hence better resolution at depth. We also Compared the result with available geotechnical data at the site that revealed validity of the S-wave velocity structure from derived microtremor array analysis.

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ASSESSMENT OF SYNTHETIC ACCELEROGRAMS FOR DYNAMIC ANALYSES OF STRUCTURES

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Key Words: Ground motion simulation, Stochastic Point-Source Modeling, Stochastic finite- Source modeling, Response spectra.

ABSTRACT Seismic design codes generally define ground shaking in the form of a response spectrum of acceleration and allow using response spectrum compatible time history records in linear and nonlinear time history analyses. These records can be obtained from synthetically, and artificially generated accelerograms or natural earthquake accelerograms and have to be selected and scaled according to the rules defined in codes.

Synthetic accelerograms can be generated from seismological source models and accounting for path and site effects. These models can be used to produce more realistic time-histories than those produced by methods that do not contain seismological controls. In order to generate synthetic accelerograms there is a need for a definition of a specific earthquake scenario in terms of magnitude, rupture mechanism in addition to geological conditions and location of the site.

Stochastic point source modeling (SMSIM) is applied to simulate one component synthetic accelerograms by generating a random white noise for expected source duration to obtain time-history with a band-limited Fourier amplitude spectrum specified by the w-squared source model and incorporate wave propagation effects of a homogeneous crust. Stochastic finite-source simulation (FINSIM) takes into account the effects of faulting geometry, distributed rupture, and rupture inhomogeneity by subdividing the fault surface of an earthquake into a grid of sub-sources, each of which could be treated as a point source. The contributions to ground motion can be summed at the observation site, over all of the sub-sources comprising the fault, considering proper delays of sub-sources due to rupture propagation.

In this study, synthetic accelerograms records are generated using SMSIM and FINSIM for a set of different locations with different site classes. The average peak ground acceleration and spectral acceleration for the simulated records are compared with the results of attenuation relationships. The nonlinear response of Single degree of freedom system (SDOF) to the generated synthetic time history input motions are assessed and discussed in terms of engineering design parameters.

INTRODUCTION

With the advances in computational tools, time history analyses are commonly used for seismic response analysis of buildings and bridges. Selection and scaling the input ground motion time histories are critical steps to perform linear and nonlinear response history analyses. In literature, there are three sources of acceleration time histories: design response spectrum compatible artificial records, synthetic records obtained from seismological models and accelerograms recorded in real earthquakes. Utilizing the available strong ground motion records is the simplest way to get acceleration time

histories that can be used in structural time history analyses. The selection criteria of proper time history records to fit the design code spectrum are also taking into account the geological and seismological conditions at specific site. The seismological and geological parameters can be classified in terms of magnitude, faulting type, distance to fault, rupture directivity, site condition and spectral content. Owing to the lack of recorded data and the randomness of earthquake ground motion that may be experienced by the structure in the future, usually it is difficult to obtain recorded data which satisfy all the seismological, geological requirements especially for severe earthquake with magnitude greater than 6.5.

An alternative or supplement to using recorded time-histories is to develop synthetic time-histories based on the theoretical models of earthquake source processes and seismic wave propagation. These models can be used to produce more realistic time-histories than those produced by methods that do not contain seismological controls. The computational model for synthetic seismograms consists of three processes: seismic source, seismic wave propagation from source to site, and site response to generate synthetic accelerograms there is a need for a definition of a specific earthquake scenario in terms of magnitude, rupture mechanism in addition to geological conditions and location of the site.

In general, two methodologies are used for the stochastic simulations of the synthetic accelerograms: Point-source and finite-source models. The stochastic point source model assumes that the source is concentrated at a point, and that the acceleration time histories generated at a site carry both deterministic and random aspects of ground motion shaking. The point-source assumption is reasonable when source to site distance is much larger than source dimensions (Boore, 2003; Atkinson and Boore, 1997; Atkinson and Silva, 2000). The finite-source model considers the finite-fault effects in ground motion modeling. Therefore it takes into account important factors that influence the ground motions from large earthquakes that are not included in the stochastic point source model, such as the effects of faulting geometry, distributed rupture, and rupture inhomogeneity.

In this study, the acceleration time histories of the horizontal components at bedrock of selected sites in the vicinity of Marmara sea for possible strong earthquakes are simulated using both point-source (SMSIM) and finite-source (FINSIM) stochastic methods. Two earthquake scenarios are considered and the averages of fifty simulations at each site are considered. The maximum peak ground accelerations and spectral acceleration of SMSIM and FINSIM results are compared with those computed for the empirical attenuation relationships of Boore et al (1997) and Sadigh et al (1997). The resulting time histories are investigated in terms of suitability as input to time history analyses of civil engineering structures.

STOCHASTIC POINT-SOURCE MODELING:

The stochastic point source model assumes that the source is concentrated at a point, and that the acceleration time series generated at a site carry both deterministic and random aspects of ground motion shaking. The deterministic aspects are specified by the average Fourier spectrum, typically as a function of magnitude and distance. The stochastic aspects are treated by modeling the motions as noise with the specified underlying spectrum (Boore, 2003). Fourier amplitude spectrum of ground motion (Figure 1) for the point-source model at a site Y (M, R, f) is a sum of contributions from earthquake source spectrum, $E(M_0,f)$, path P(R,F), site G(f) and instrument or type of motion I(f) as:

$$Y(M_0, R, f) = E(M_0, f)P(R, f)G(f)I(f)$$

STOCHASTIC FINITE SOURCE MODELING.

To consider the finite-fault effects in ground motion modeling, Hartzell (1978) proposed subdividing the fault surface of an earthquake into a grid of sub-sources, each of which could be treated as a point source. The contributions to ground motion can be summed at the observation site, over all of the sub-sources comprising the fault, considering proper delays of sub-sources due to rupture propagation. The sub-fault acceleration time history is propagated to the observation point using empirical distance-dependent duration, geometric attenuation, and attenuation (Q) models (Boore and Atkinson, 1987; Beresnev and Atkinson, 1997). The point-source method of Boore (1983) is used to generate stochastic acceleration time histories from each of the sub-faults. The stochastic finite source code of Beresnev and Atkinson (1998) was named FINSIM. Motazedian and Atkinson (2005) made a major modification introduce the concept of "dynamic corner frequency" in order to overcome the main difficulties associated with the heavy dependence of the simulation on sub-source size and produce a new code named (EXSIM).

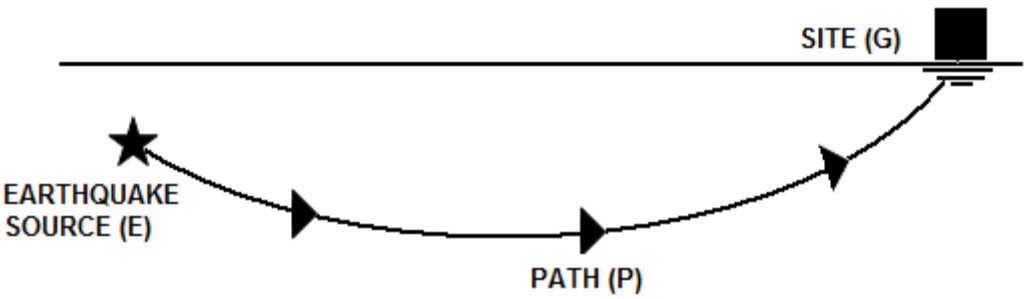


Fig. 1. Components of the point-source Fourier amplitude spectrum of ground motion.

SEISMICITY AND SEISMOTECTONIC OF THE MARMARA REGION

The Sea of Marmara is a major trans-tensional step-over of the North Anatolian Fault (NAF) (Armijo et al. 1999, 2002). It forms an active system of strike-slip basins developed along the submerged portion of the NAF system. Bathymetric mapping reveals that the main fault strand in the Sea of Marmara is formed by discrete segments some tens of kilometers long, separated by geometric fault complexities (Armijo et al. 2002). Coseismic slip measured along these segments is about 1–5 m (Armijo et al. 2005; Pondard 2006). According to strike variations, each segment accommodates different proportions of strike- and normal-slip.

The Marmara Sea appears as a transitional area between two different tectonic regimes. Both shear and extensional or compressional motions are reported in the area of the Marmara Sea (Barka and Kadinsky-Cade, 1988), and normal faults with an approximately EW strike are known in the region (Taymaz et al., 1991).

The historical seismicity in the Marmara Sea is extremely high, especially for the northern branch of the emerged NAF (Ambraseys and Finkel, 1991; Ambraseys and Jackson, 2000; Ambraseys, 2002a). Strong historical earthquakes (M>7) and the presence of known seismic gaps imply a high level of seismic hazard. The long-term seismicity of the Marmara region is illustrated in Fig. 2.

Over the last three centuries, 18 destructive earthquakes have occurred in the Marmara Sea and its surroundings. At least five of these earthquakes affected the Istanbul area and caused significant damage.

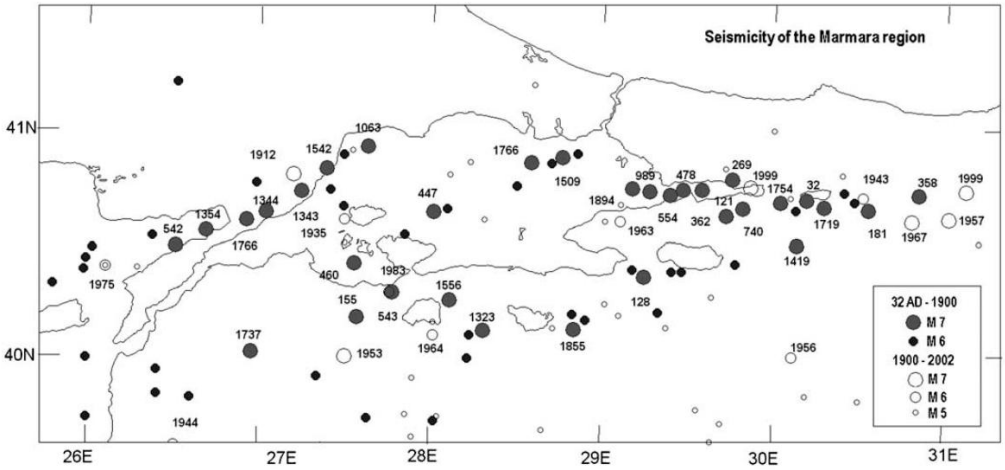


Fig. 2. The long-term seismicity of the Marmara region (Seismicity between 32 AD -1983 taken from Ambraseys et al., 1991).

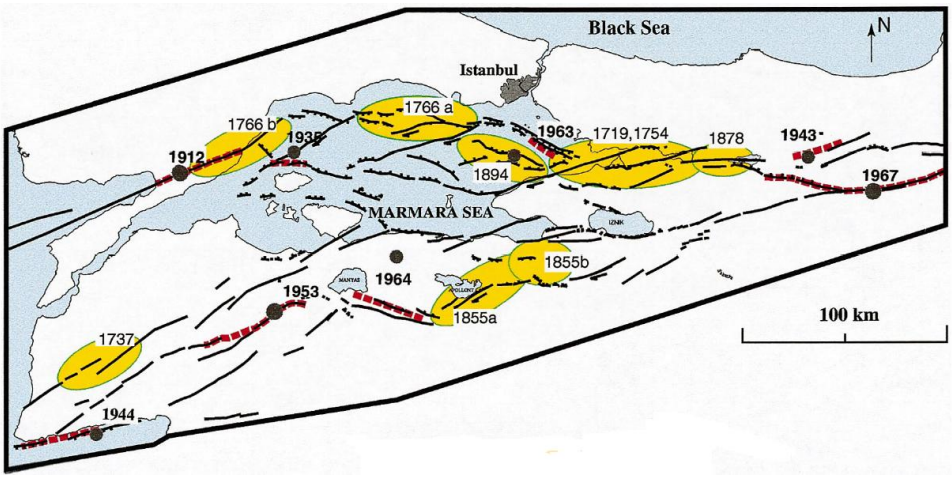


Fig. 3. Active faulting and historical earthquakes in the Marmara region (modified after Barka, 1997). Black thick lines: active faults recognised by geology and geophysics. Red lines: surface ruptures of earthquakes of this century. Yellow ellipses: estimated rupture areas of historical earthquakes within the period 1700–1900 AD (Gurbuz et al., 1999).

Frequent occurrence of historic destructive earthquakes clearly demonstrates the high seismic activity and the potential seismic hazard in the area. Earthquake records spanning two millennia indicate that, on average, at least one medium intensity (VII–VIII) earthquake has affected Istanbul in every 50 years. The average return period for high intensity (VIII–IX) events has been 300 years.

The westward propagation of the seismic ruptures along the NAF during the 20th century together with the 1999 recent ruptures have increased the probability that a next rupture could occur offshore, in the Marmara Sea. Since the recent 7.4 Kocaeli earthquake, the scientific community has reached a consensus concerning the large probability of rupture within the next few tens of years of the identified seismic gap between the 1999 Kocaeli rupture and the 1912 Ganos rupture [Hubert-Ferrari et al., 2000; Parsons et al., 2000]. However, there is no consensus on the nature of the expected earthquakes, their magnitude, and the rupture locations.

The recently determined GPS velocities rates and the fact that more than two centuries have passed since the previous major moment release clearly indicate that the seismic hazard potential has already drastically increased and encourages researchers to asses

the seismic hazard of the region (Smith et al. 1995; Wong et al. 1995; Okay et al. 1999, 2000; Parke et al. 1999; Imren et al. 2001).

SYNTHETIC ACCELEROGRAMS SIMULATION FOR MARMARA REGION

In this study, two rupture scenarios in Marmara Sea are considered. The first scenario is associated with prince island fault with moment magnitude, Mw, of 7.2, while the second scenario is considered the central Marmara fault with moment magnitude to be equal 7.5. The simulations are performed for six stations (site) in the vicinity of Marmara Sea as it is shown in Figure 4. The coordinates of the stations and the epicentral distances of the stations to the first and second scenarios faults are provided in Table1. The simulation parameters for both SMSIM and FINSIM are given in Table 2 and the geometrical spreading relationship are shown in Figure 5.

A total of 50 simulations for each station are calculated for each scenario separately. The acceleration time histories are processed to find the average peak ground acceleration (PGA) and the average spectral accelerations (Ss,S1) at 0.2 sec and 1.0 sec. Comparisons of the average results of SMSIM and FINSIM with the average of attenuation relationships of Boore el al (1997) and Sadigh et al (1997) are given in table 3 and 4 for the first and second scenario respectively. The PGA and spectral acceleration at (0.2,1.0 sec) of the simulated records provide underestimated values compared with the attenuation relationships of Boore el al (1997) and Sadigh et al (1997) especially for the stations with large epicentral distances. A randomness of hypocenter location along the fault may be considered in future study. The average response spectra for the time history records generated for the first scenario by SMSIM and FINSIM are plotted and compared in Figures 6 and 7. The results for the second scenario are plotted in Figures 8 and 9.

The nonlinear response of SDOF for the generated synthetic time histories are examined by computing the ductility factor (μ) for different structural behavior factors, R=1,2,4,8. In real earthquake records, average of the ductility factor is expected to be equal to structural behavior factor at longer periods (equal displacement rule) especially for velocity and displacement sensitive spectral regions (Chopra, 2000). Equal displacement rule may not be satisfied by time histories recorded near the source.

The averages of nonlinear response of the synthetic time histories generated by SMSIM and FINSIM for the first and second scenarios are plotted in Figures 10-13. The synthetic accelerograms generated by SMSIM have incompatible behavior with the equal displacement rule for both short and long periods. The averages of nonlinear response of the synthetic time histories have a better compatibility with the equal displacement rule

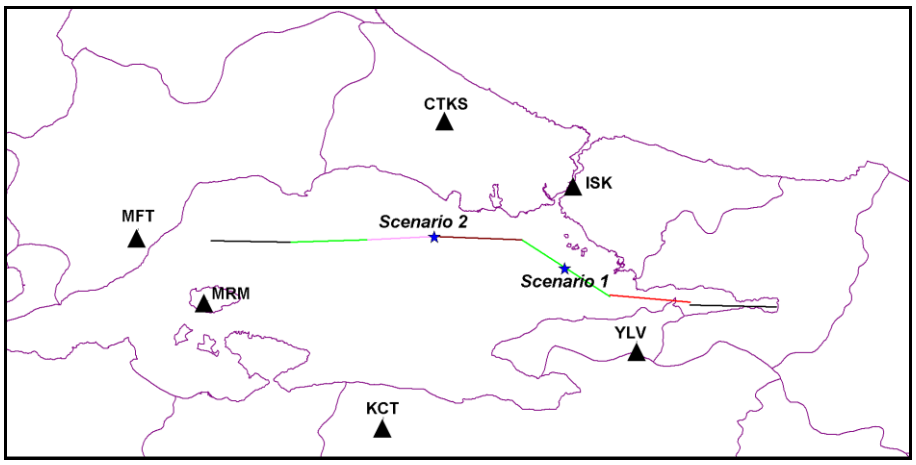


Fig. 4. Map of Scenario Earthquakes.

Table 1. List of Station Coordinates.

Station Name	Coordinates	Epicentral Distance for Scenario 1	Epicentral Distance for Scenario 2
ISK	41.07N 29.06E	28.95 km	51.56 km
YLV	40.57N 29.37E	37.99 km	80.69 km
CTKS	41.237N 28.51E	65.70 km	40.19 km
KCT	40.265N 28.36E	84.19 km	69.09 km
MRM	40.605N 27.58E	126.13 km	83.49 km
MFT	40.786N 27.28E	149.19 km	103.51 km

Table 2. Earthquake Source Parameters for Marmara Sea.

	Scenario 1	Scenario 2
Earthquake Coordinate	40.81N 29.05E	40.876N 28.03E
Mw	7.2	7.5
Moment	$0.9 * 10^{20}$	$2.0 * 10^{20}$
Kapa	0.055	0.055
Stress drop	100 Bar	100 Bar
Q(s-1)	$180.f^{0.45}$	$180.f^{0.45}$
Rho	2.75 – 3.0 gr/cm3	2.75 – 3.0 gr/cm3
Vs	3.85 km/sec	3.85 km/sec

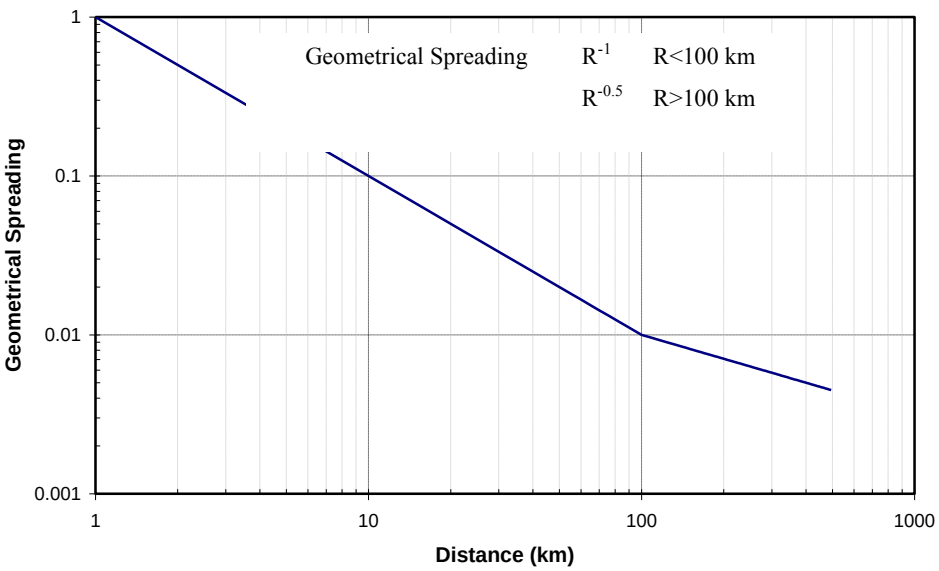


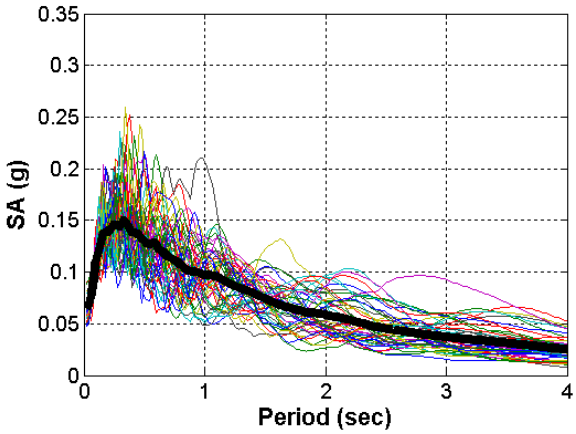
Fig. 5. Geometrical Spreading Relationship.

Table 3. Comparison for Average Spectral Acceleration for Scenario 1.

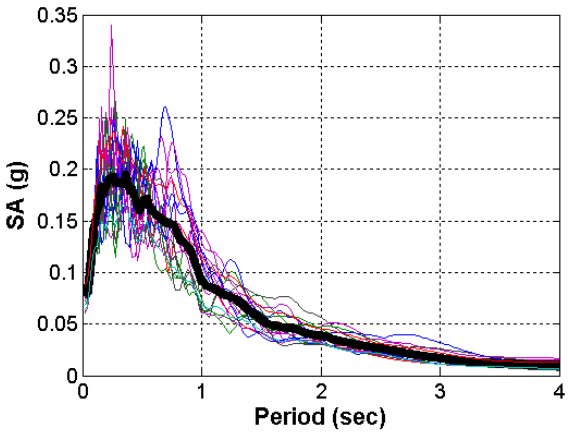
Station	PGA (g)			Spectral Acc. At 0.2 Sec (Ss)			Spectral Acc. At 1.0 Sec (S1)		
	SMSIM*	FINSIM*	Atten**	SMSIM*	FINSIM*	Atten	SMSIM*	FINSIM*	Atten
ISK	0.063	0.073	0.145	0.139	0.184	0.332	0.097	0.092	0.140
YLV	0.047	0.081	0.111	0.105	0.193	0.251	0.072	0.121	0.109
CTKS	0.021	0.033	0.062	0.044	0.074	0.134	0.037	0.052	0.063
KCT	0.014	0.021	0.047	0.027	0.051	0.099	0.025	0.033	0.048
MRM	0.008	0.013	0.030	0.014	0.027	0.060	0.016	0.022	0.031
MFT	0.005	0.010	0.025	0.010	0.020	0.049	0.012	0.021	0.026

Table 4. Comparison for Average Spectral Acceleration for Scenario 2.

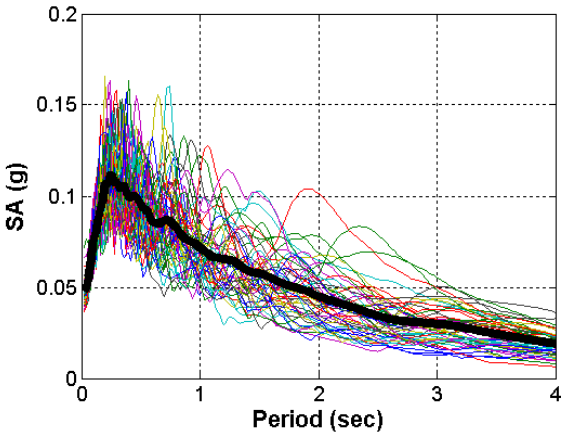
Station	PGA (g)			Spectral Acc. At 0.2 Sec (Ss)			Spectral Acc. At 1.0 Sec (S1)		
	SMSIM*	FINSIM*	Atten**	SMSIM*		Atten	SMSIM*	FINSIM*	Atten
ISK	0.037	0.067	0.097	0.082	0.171	0.205	0.066	0.092	0.107
YLV	0.019	0.035	0.060	0.039	0.080	0.121	0.036	0.055	0.068
CTKS	0.053	0.052	0.125	0.118	0.124	0.269	0.093	0.092	0.136
KCT	0.026	0.033	0.071	0.051	0.076	0.146	0.045	0.059	0.080
MRM	0.019	0.088	0.057	0.036	0.197	0.116	0.035	0.121	0.066
MFT	0.013	0.062	0.045	0.024	0.152	0.089	0.025	0.099	0.052



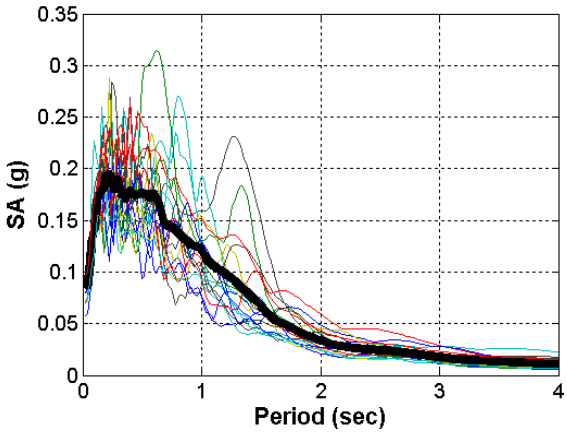
SMSIM-ISK Station



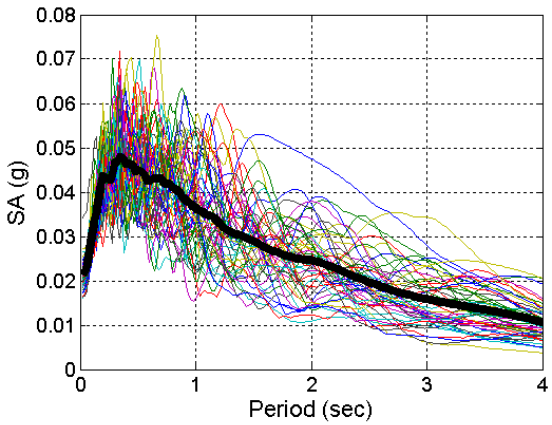
FINSIM-ISK Station



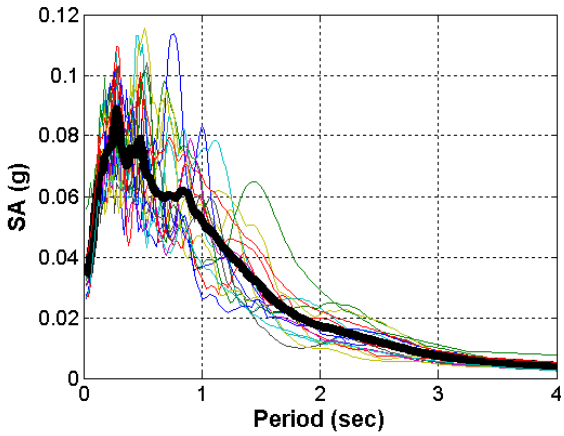
FINSIM- YLV Station



FINSIM- YLV Station

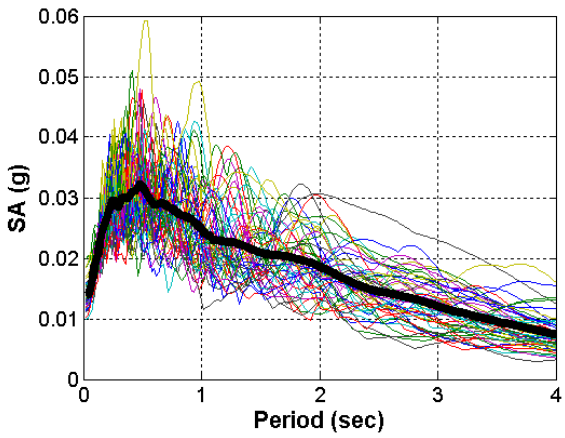


FINSIM- CTKS Station

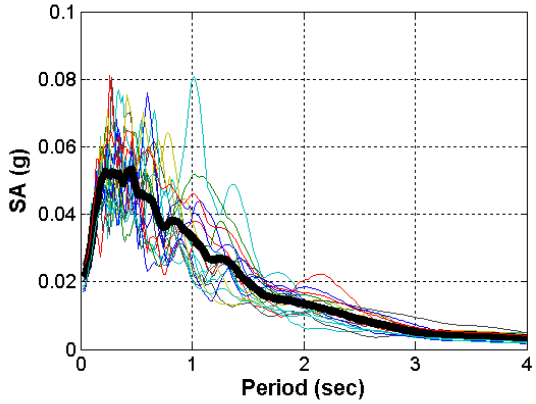


FINSIM- CTKS Station

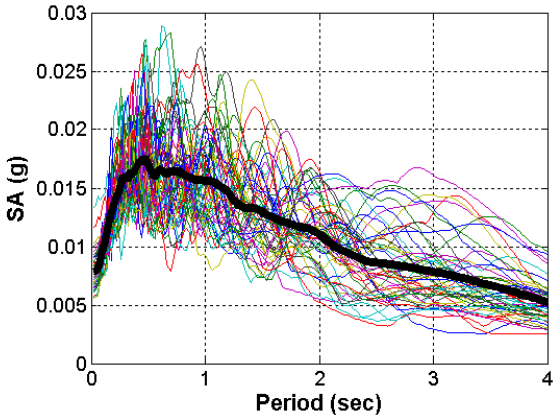
Fig. 6. Average Response Spectra of Records Generated for Scenario 1 by SMSIM and FINSIM.



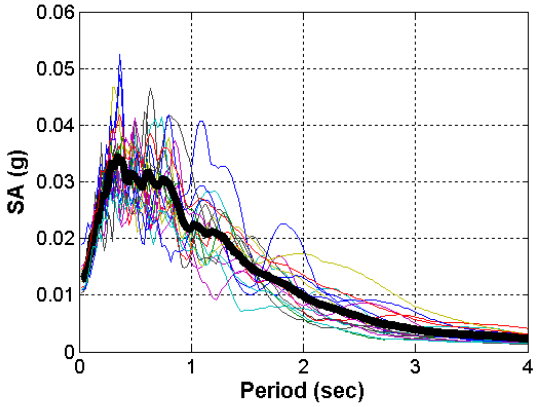
SMSIM- KCT Station



FINSIM- KCT Station



FINSIM- MRM Station



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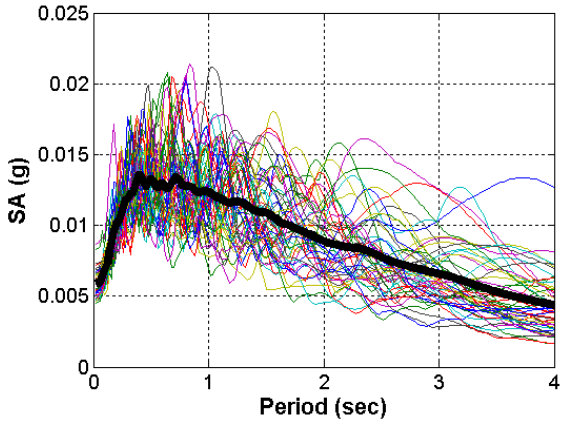
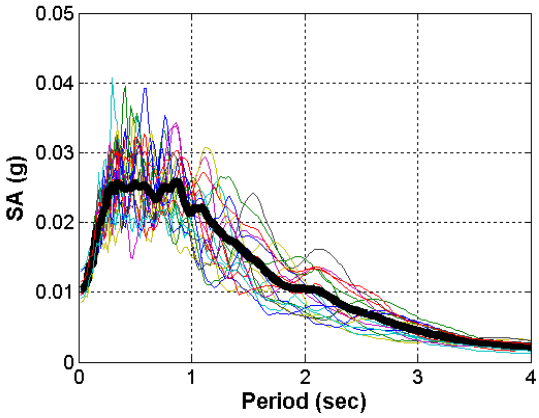
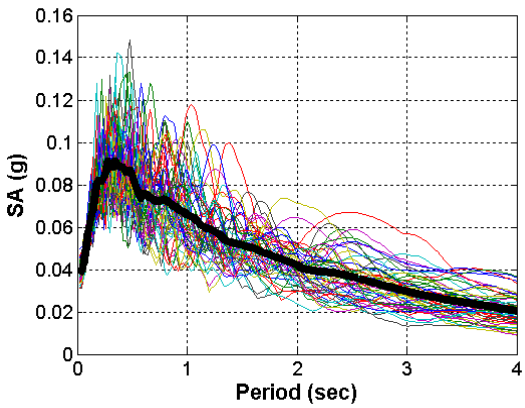


FINSIM- MFT Station

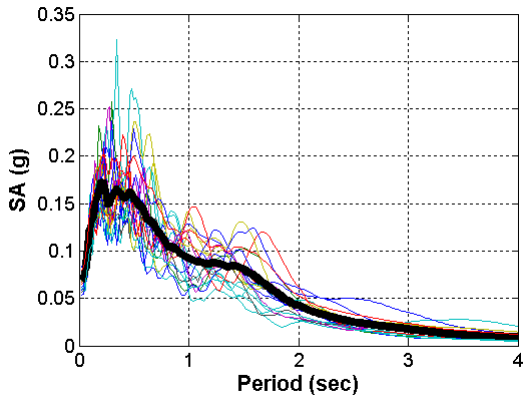


FINSIM- MFT Station

Fig. 7. Average Response Spectra of Records Generated for Scenario 1 by SMSIM and FINSIM.



SMSIM-ISK Station



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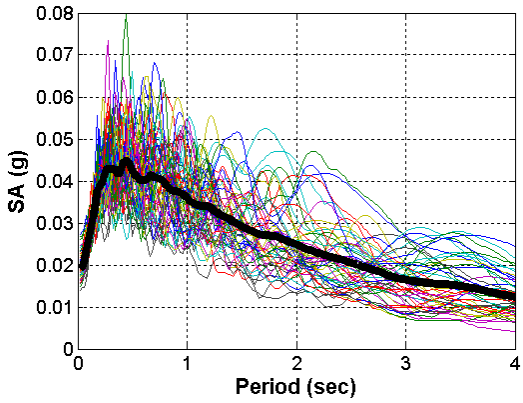
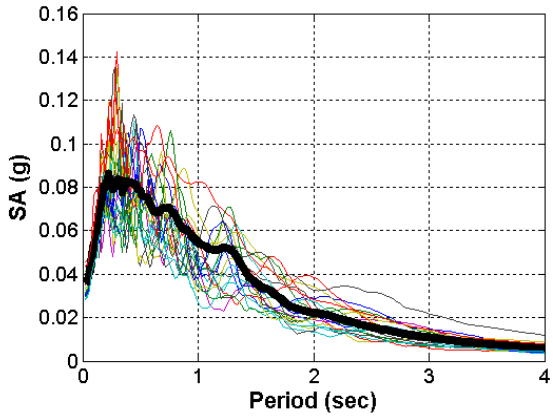


FINSIM- YLV Station



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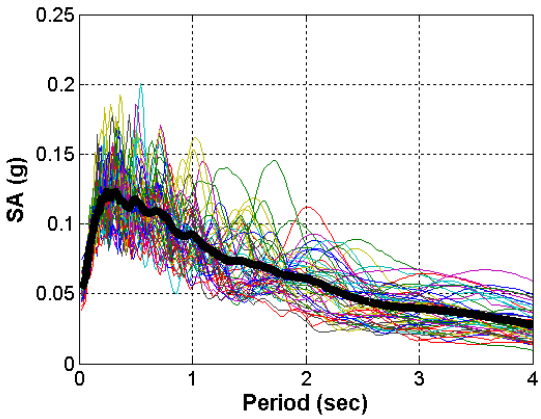
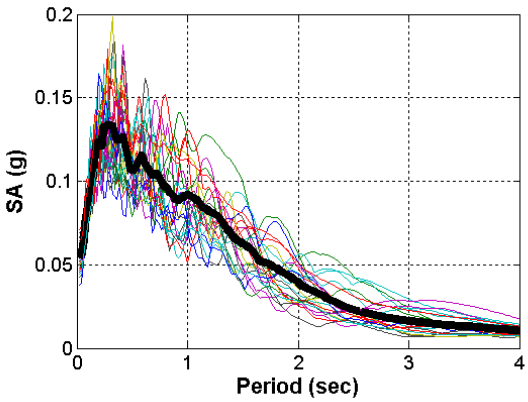
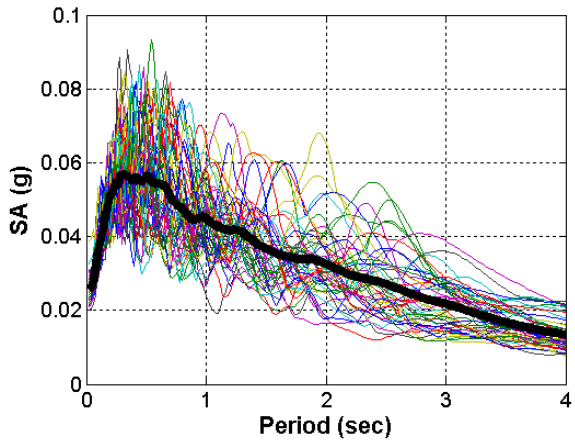


FINSIM- CTKS Station

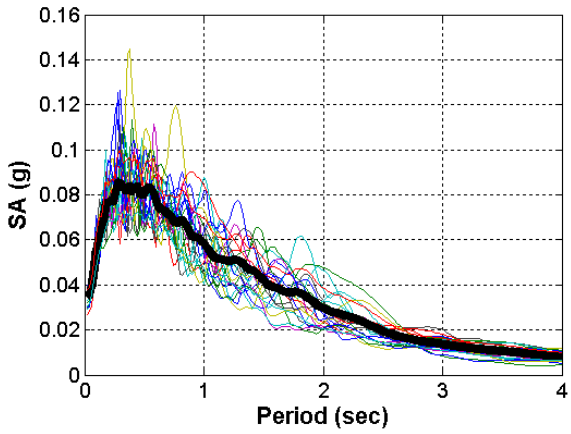


FINSIM- CTKS Station

Fig. 8. Average Response Spectra of Records Generated for Scenario 2 by SMSIM and FINSIM.



SMSIM- KCT Station



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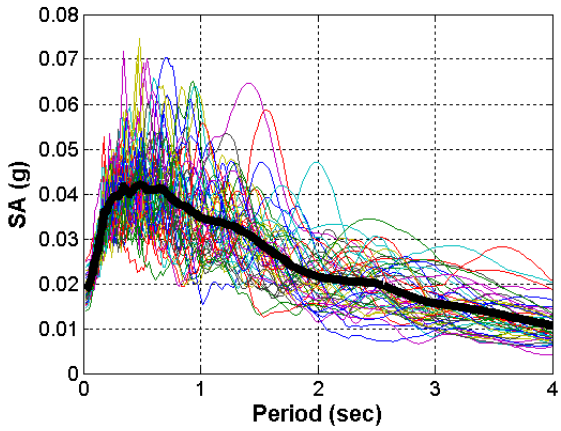
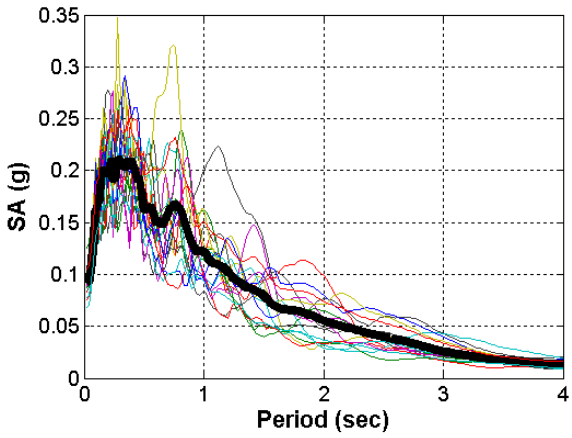


FINSIM- MRM Station



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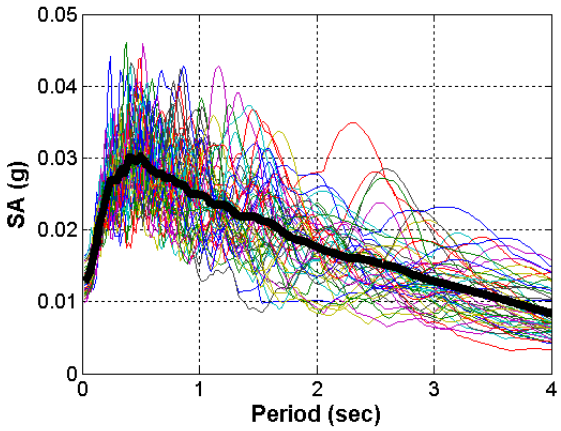
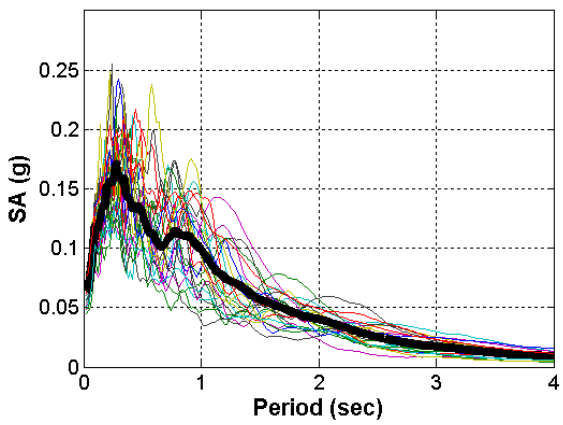


FINSIM- MFT Station



FINSIM- MFT Station

Fig. 9. Average Response Spectra of Records Generated for Scenario 2 by SMSIM and FINSIM.

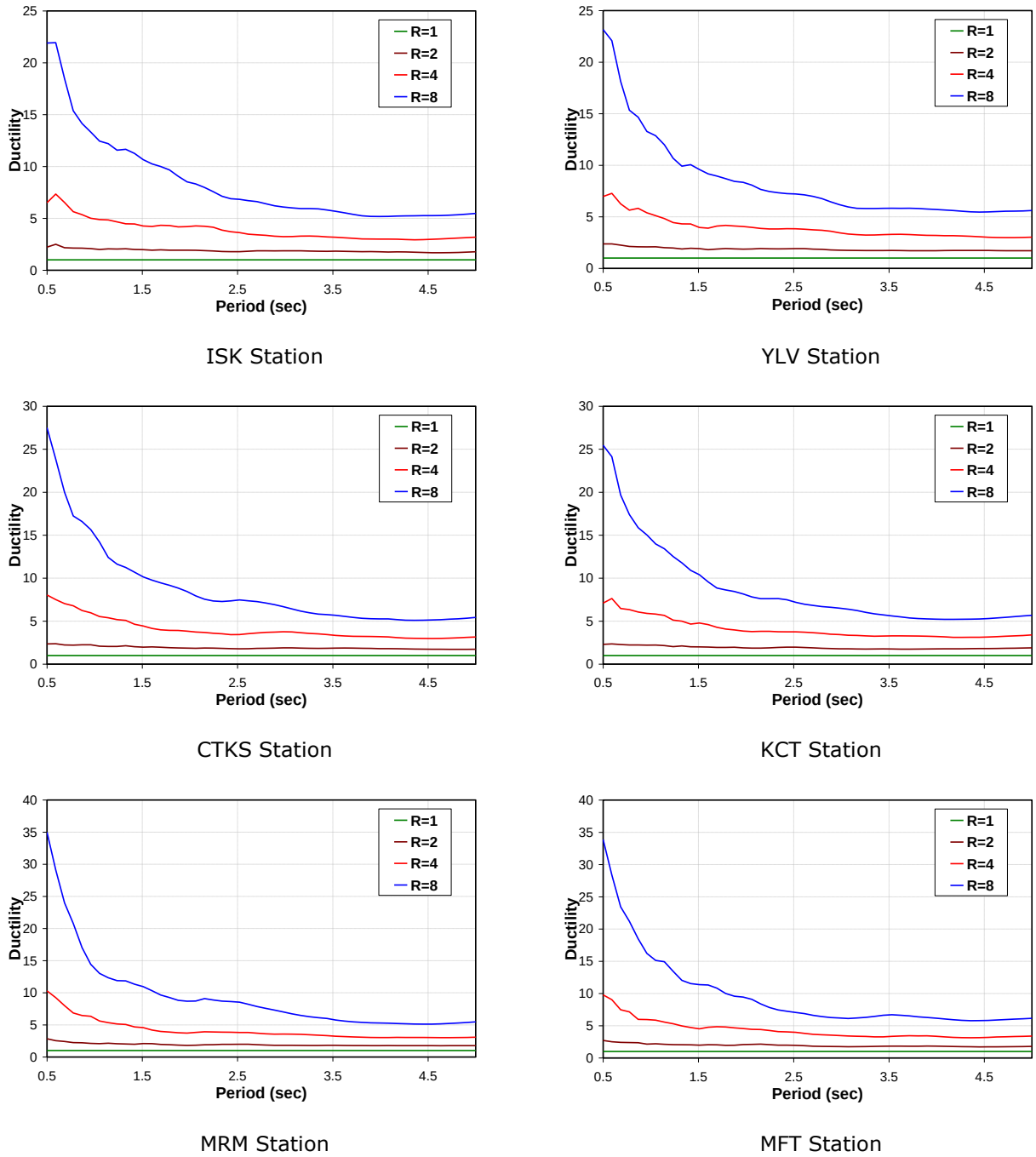


Fig. 10. Average Ductility Demand of Records Generated for Scenario 1 by SMSIM.

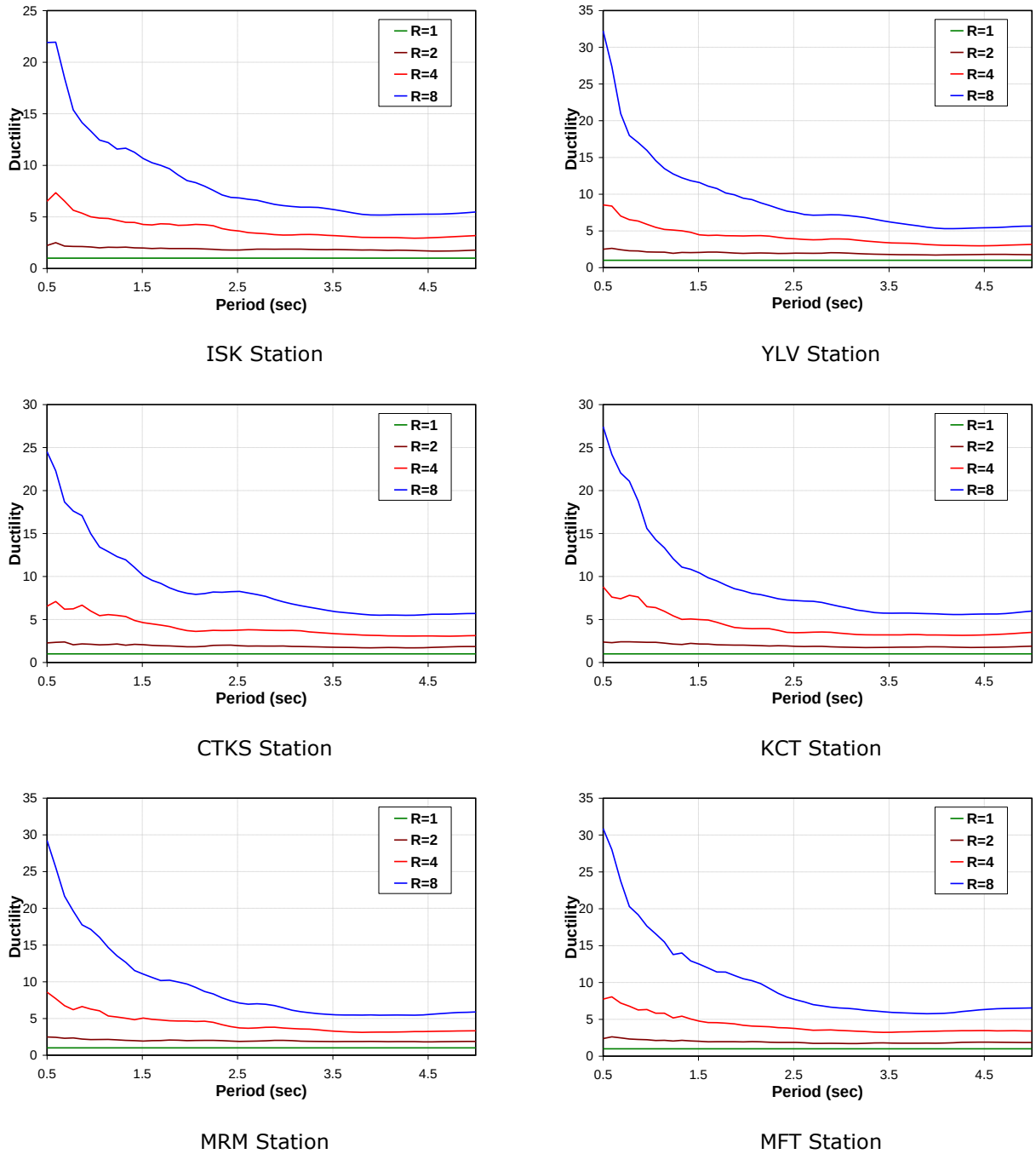


Fig. 11. Average Ductility Demand of Records Generated for Scenario 2 by SMSIM.

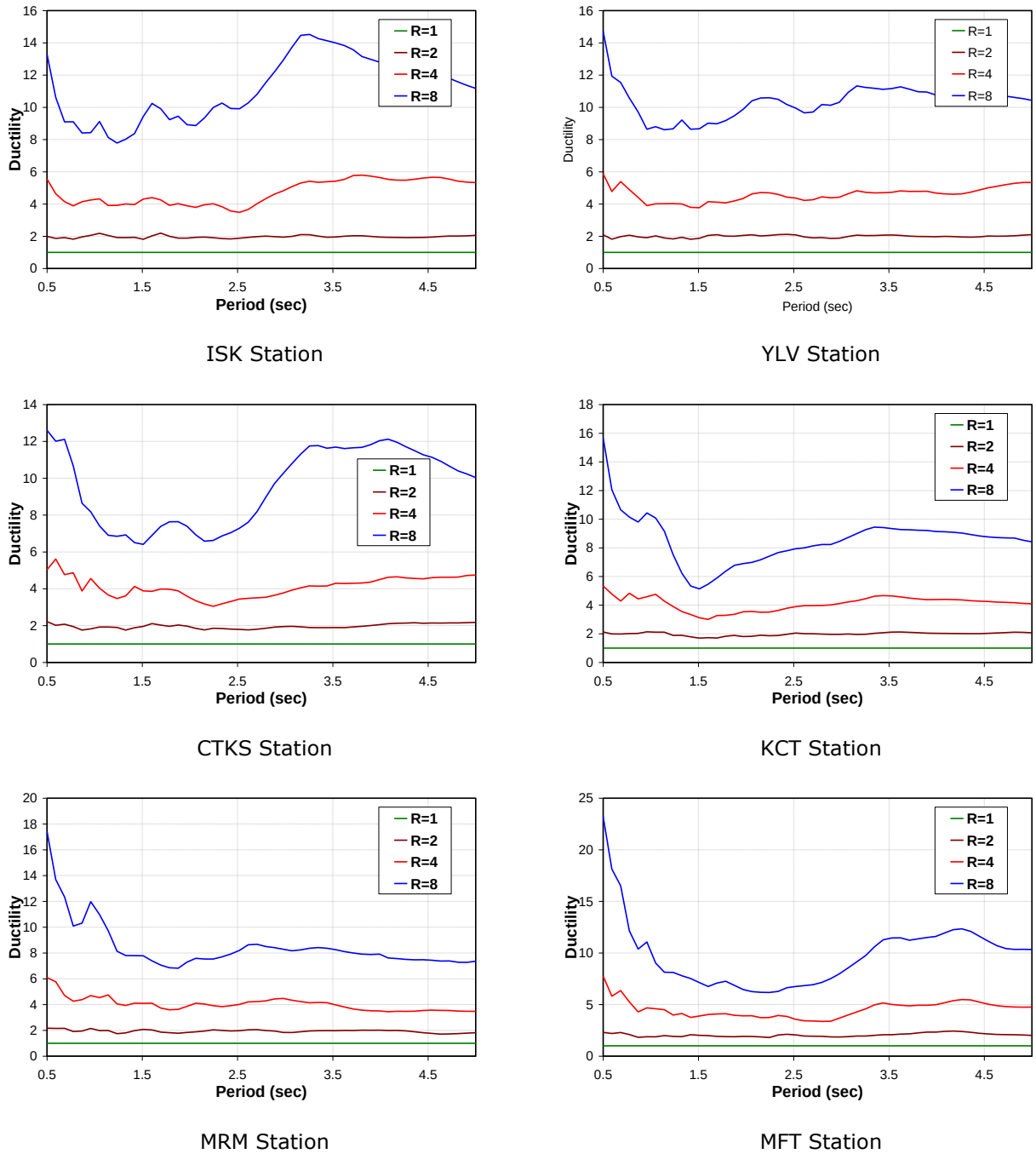


Fig. 12. Average Ductility Demand of Records Generated for Scenario 1 by FINSIM.

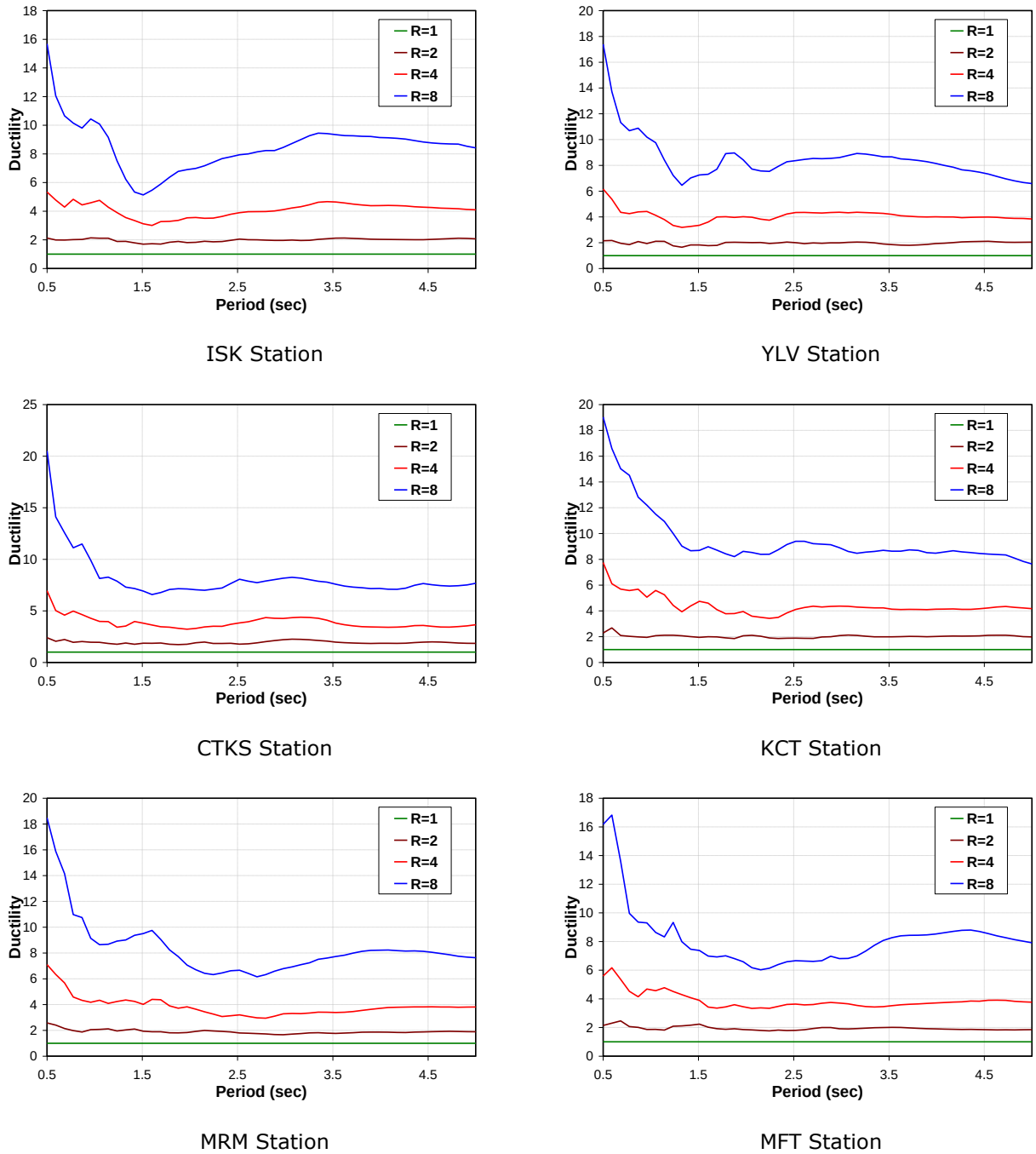


Fig. 13. Average Ductility Demand of Records Generated for Scenario 2 by FINSIM.

CONCLUSIONS

The stochastic method is a simple, yet powerful, means for simulating ground motions. It is particularly useful for obtaining ground motions at frequencies of interest to earthquake engineers. On the other hand the method can only be able to simulate one component that does not include any phase effects due to the propagating rupture and to the wave propagation to the site. Stochastic point-source and stochastic finite-source methods are utilized to generate 50 simulations for six stations in the vicinity of Marmara Sea. Two scenarios are considered with moment magnitude of 7.2 and 7.5 for Prince Island and Central Marmara faults respectively. The averages of PGA and spectral displacement at (0.2,1.0 sec) of the simulated records results provide underestimated values compared with the Attenuation Relationships of Boore el al (1997) and Sadigh et

al (1997). For more comparable results, a randomness of hypocenter location along the fault may be considered in future study. Using SMSIM generated synthetic records, the spectral displacements responses for the period longer than 1 sec have up to 100% deviations from the average. The results of FINSIM provide realistic deviation for the same period range. The nonlinear response of SDOF for the generated time histories are examined by computing the ductility factor (μ) for different structural behavior factors, R=1,2,4,8. In real earthquake records, average of the ductility factor is expected to be equal to structural behavior factor at longer periods (equal displacement rule) especially for velocity and displacement sensitive spectral regions (Chopra, 2000). The synthetic accelerograms generated by SMSIM have incompatible behavior with the equal displacement rule for both short and long periods. The averages of nonlinear response of the synthetic time histories have a better compatibility with the equal displacement rule

ACKNOWLEDGMENTS

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COMPARISON OF METHODS WITH REGARD TO ENERGY PRODUCTION IN A MULTIPLE RESERVOIRS SYSTEM

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Key Words: Multiple Reservoirs System, Multi-objective Optimization, Energy Production, Dynamic Programming with Successive Approximations.

ABSTRACT Water resource planning integrated with national economics and desired living standards is a multi-dimensional and complicated process. Water resource planning and management must incorporate all activities to provide the optimal uses of water resources, while meeting requirements and taking criticisms into account. In this study, optimization model with energy production objective for long term planning to a multi-object water resource system with multiple reservoirs presented on the Ceyhan River in the Ceyhan Basin of Turkey is developed. Dynamic programming with successive approximations as optimization technique is used in the model. In this optimization model, firstly, maximization of the firm power value of the system by using the monthly inflow data of the drought period is realized and monthly minimum operational levels are obtained. Then, in the same optimization model, the maximization of the total energy of the system is achieved by locating the firm power value as a constant and using the monthly average inflows and normal operational levels are determined. When operational levels obtained from optimization process are illustrated, it has controlled and managed by reservoirs with large storage. This system by an approach used to the basin planning and management of DSİ is analyzed. The results obtained are compared with results of optimization model. The firm power value obtained from optimization model is presented 13 % bigger than its value obtained from approach of DSİ and also total energy production of the system is 153%

INTRODUCTION

Water resource planning integrated with national economics and desired living standards is a multi-dimensional and complicated process. Water resource planning and management must incorporate all activities to provide the optimal uses of water resources, while meeting requirements and taking criticisms into account (Opan, 2007). Energy production in the scope of the long-term planning is generally taken into consideration, determining optimal operational politics in a multiple reservoirs system.

In the research interested with energy optimization, the simulation and the optimization technique of the dynamic programming with regard to the non-linear of the energy production function are used in a multiple reservoirs system (Güvel, 1997, Sert, Kızıltan, Dalgıç, 1982, Sert, Öcal, Oktay, 1983, Yakowitz, 1982, Yeh, 1985, Yurtal, 1993). The Any research presented for a short-term optimization with the objective of the hourly drought control and management is not reached. However, studies presented to maximize the firm energy by using monthly data of the drought period in long-term planning have been produced (Sert, Kızıltan, Dalgıç, 1982, Sert, Öcal, Oktay, 1983 and Yurtal, 1993).

In this study, optimization model with energy production objective for long term planning to a multi-object water resource system with multiple reservoirs presented on the Ceyhan River in the Ceyhan Basin of Turkey is developed. Dynamic programming with successive approximations as optimization technique is used in the model. In this

optimization model, firstly, the maximization of the firm power value of the system by using the monthly inflow data of the drought period is realized and monthly minimum operational levels are obtained. Then, in the same optimization model, the maximization of the total energy of the system is achieved by locating the firm power value as a constant and using the monthly average inflows and normal operational levels are determined. Furthermore, the system by an approach used to the basin planning and management of DSİ is analyzed. The results obtained are compared with results of optimization model.

MATHEMATICAL MODEL OF SYSTEM

A mathematical model of a multiple reservoir system for long term planning is usually based on a monthly time scale thus; the monthly flows are used as the system input. The system output to be optimized is represented in the objective function, while the general description of the system operation must be given in the constituent equations and the system constraints expressed as inequalities.

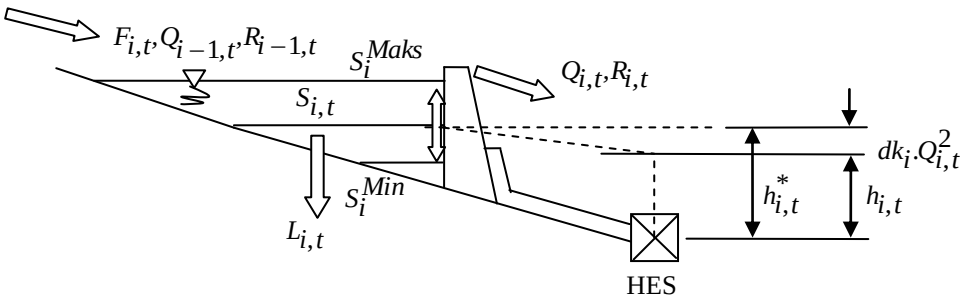

Fig. 1. Operational variables in a system with multiple reservoirs i at time period t.

The system may be presented as a series of reservoirs, each with a storage capacity, a power production unit, inflows from upstream and releases to downstream (Figure 1). Taking $i = 1,2,..N$, where N is the number of reservoirs, and $t = 1,2,..M$, where M is the number of time intervals (months), the basic constituent equations of the system are the water balance relations of each reservoir for each time interval (for all i and t)

$$S_{i,t+1} = S_{i,t} + F_{i,t} + Q_{i-1,t} + R_{i-1,t} - Q_{i,t} - R_{i,t} - L_{i,t} \tag{1}$$

where $S_{i,t}$ is the water stored in reservoir; $F_{i,t}$ is the inflow into reservoir from its sub-drainage area; $Q_{i,t}$ is the water released for energy production from reservoir; $R_{i,t}$ is the spilled water from reservoir; and $L_{i,t}$ is the water loss through evaporation and seepage from reservoir. The other constituent equations are for power generation

$$P_{i,t} = k_i.Q_{i,t}.h_{i,t} \tag{2}$$

$$h_{i,t} = h_{i,t}^* - h_f^i \tag{3}$$

$$h_{i,t} = h_{i,t}^* - \alpha^i.Q_{i,t}^2 \tag{4}$$

where $P_{i,t}$ is the power generation from reservoir; $h_{i,t}$ is the net turbine head for reservoir; k_i is the average power generation coefficient for reservoir; $h^*_{i,t}$ is the water height in reservoir (measured from turbine level) and α^i is the average head loss coefficient for reservoir; and $h^*_{i,t}$ will be obtained as a function of the average storage volume (at the beginning and end of the time interval),

$$h^*_{i,t}=h(\frac{S_{i,t}+S_{i,t+1}}{2}) \tag{5}$$

The total power generation of the system at time t is given by

$$\sum_{i=1}^{N} P_{i,t} = \sum_{i=1}^{N} k_i.Q_{i,t}.h_{i,t} \tag{6}$$

The system constraints are related to the storage capacity, power generation capacity and water usage (for energy production, irrigation and other purposes).

1. Constraints on the storage capacities

$$S_i^{Min} \leq S_{i,t} \leq S_i^{Max} \tag{7}$$

where S_i^{Min} and S_i^{Max} are the minimum and maximum storage capacities of the reservoir.

2. Constraints on the power generation,

$$0 \leq P_{i,t} \leq P_{ki} \tag{8}$$

where P_{ki} is the installed power generation capacity for reservoir.

3. Constraints on the releases for energy production

$$Q_i^{Min} \leq Q_{i,t} \leq Q_i^{Max} \tag{9}$$

where Q_i^{Min} and Q_i^{Max} are the minimum and maximum releases for reservoir. Obviously, Q_i^{Max} is related to the installed power generation capacity P_{ki}, while Q_i^{Max} is related to the minimum required release downstream D_i^{Min} such that

$$Q_i^{Min} = D_i^{Min}, \text{ when } R_{i,t} = 0 \tag{10}$$

4. Constraints on the spill water

$$0 \leq R_{i,t} \leq R_i^{Max} \tag{11}$$

where R_i^{Max} is the maximum spillway capacity for reservoir.

6. Constraints on the total water release

$$D_i^{Min} \leq (Q_{i,t} + \mathrm{R}_{\mathrm{i,t}}) \leq D_i^{Max} \tag{12}$$

where D_i^{Min} is the minimum release for pollution control or navigation. D_i^{Max} is the maximum safe discharge for downstream of the reservoir

OPTIMIZATION MODEL

A dynamic programming with successive approximations (DPSA) optimization technique is used in the optimization model. In the DPSA formulation, the above equations represent the "stage transformations equations" where the time periods (t) are the "stages" and the storage levels at each reservoir ($S_{i,t}$) are the "states". Thus, the releases from the reservoirs ($Q_{i,t}$, $R_{i,t}$) appear as the basic decision variables. However, it must be noted that the spilled water release $R_{i,t}$ will only take place when the storage and turbine release capacity constraints of the system are about to be violated, otherwise it will be zero. Thus, $R_{i,t}$ is a dependent variable and the real decision variable is $Q_{i,t}$.

In the objective function, the primary objective will be maximizing the energy production, which may be divided into two separate objectives: Firm energy and secondary energy (or the total energy production) maximization.

1. For maximization of the firm power, the critical dry period within the observed monthly flow series must be selected and using these critical period flow series (Sert et al. 1982), the objective function may be stated as,

$$\max P_F = \max \left[\min \sum_{i=1}^{N} P_{i,t} \right] \tag{13}$$

where N is the total number of months in the critical period (is observed as 12 months). The solution may be evaluated by DPSA.

1. For maximization of the total energy, average monthly flows may be used, and the already maximized firm power $P_{F\max}$ will be imposed as a parametric constraint,

$$\sum_{i=1}^{N} P_{i,t} \geq P_{F\max} \tag{14}$$

$$\max \left[\sum_{t=1}^{M} \sum_{i=1}^{N} P_{i,t} \right] \tag{15}$$

which is equivalent to maximizing the secondary energy $\sum_t \sum_i (P_{i,t} - P_{F\max})$, since $P_{F\max}$ is a parametric constant.

In general, the objective function will describe benefit functions that depend upon the water stored in each reservoir and the releases from these storages. These usually represent non-linear relations and the solution for the optimization model becomes very complicated when more than one expected benefit of storage or release is taken into account at a given time. In optimization model, optimization technique of the dynamic programming with successive approximations (DPSA) is preferred. The DPSA's purpose is to divide the problem of dynamic programming with multi-decision variables to the sub-problems of dynamic programming with one-decision variable in each one and, thus, to solve the problem while taking decision variables one-by-one. The DPSA optimization technique has more advantage than other versions of dynamic programming in terms of reducing both the calculated time and the memory requirements of the computer. There

are three variables in the DPSA: state, decision and stage variables. The team of these values related to some constrains of these variables is called the system politic. The criterion determined the effect of this system politic is also described as the objective function. The schematic view of the state-decision-stage variables in the DPSA of a reservoir is shown in Figure 2

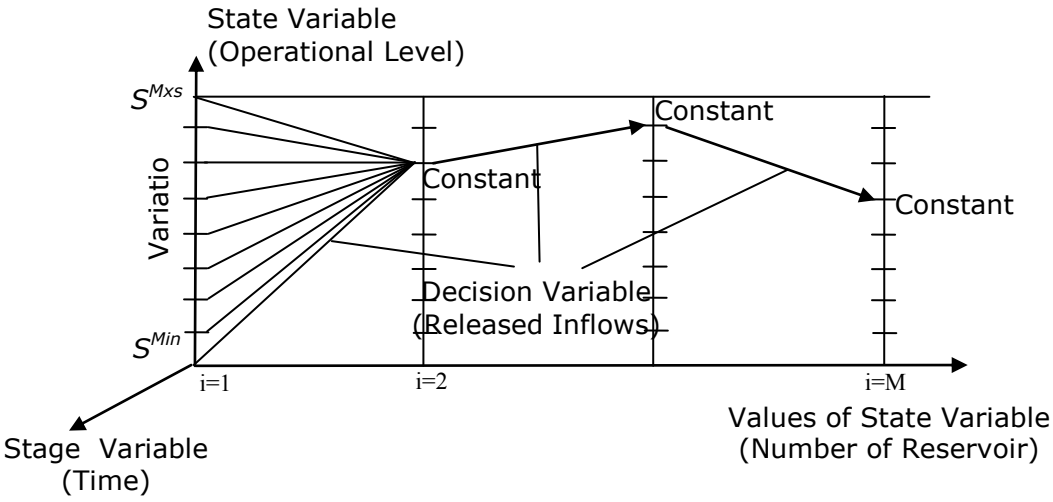

Fig. 2. Schematic view of the state-decision-stage variables in the DPSA of a reservoir.

DSI APPROACH

Equation 16 is proposed to calculate the firm power of the system with multiple reservoirs. According to this, the firm power is,

$$P_G = 3.08.10^{-3}(\frac{V_a}{L} + q_{\min})(h_{\min} + \frac{2}{3}h_a) \tag{16}$$

where V_a is the reservoir storage, L is the month number of the drought period, q_{Min} is the monthly minimum inflows h_{Min} is the height on turbine and h_a is the effective height with the reservoir storage [3].

Equation 17 is proposed to calculate the average power of the system with multiple reservoirs. According to this, the average power is,

$$P_{ort} = 3.08.10^{-3} q_{ort} . \sum_{i=1}^{5} h_{i_{maks}} \tag{17}$$

The average inflow q_{ort} is,

$$q_{ort} = \frac{\sum_{i=1}^{5} q_{i_{ort}} . h_{i_{maks}}}{\sum_{i=1}^{5} h_{i_{maks}}} \tag{18}$$

where $h_{i_{maks}}$ is the maximum height on the turbine and $q_{i_{ort}}$ is the monthly mean inflows in each reservoir [3].

APPLICATION

A system with multiple reservoirs in the Ceyhan Basin of Turkey shown in Figure 3 and 4 are selected for application. There has been a water resource system of five sequential reservoirs situated on the Ceyhan River (at the upper reaches of the Mediterranean) with energy and irrigation objectives in the Ceyhan Basin. The system was originally planned for energy purposes only, but an irrigation demand will be defined for the sake of application in the coming section. In this section, a general definition of the system related to energy production will be given. The basic characteristics of five reservoirs in the system are shown in Table 1. The variation of reservoir storage volume with height over turbines is given in Table 2. Inflows into the reservoirs have been obtained from the flow gauging stations data with correlation of drainage areas of the reservoirs and the gauging stations. Monthly mean inflows into reservoirs from sub-drainage areas are shown in Figure 5. Monthly inflows of the drought period into reservoirs from sub-drainage areas (May 1989-April 1993) are shown in Figure 6.

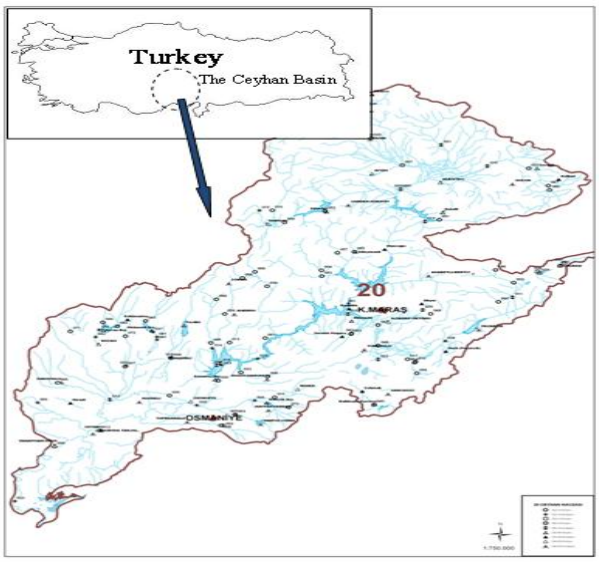

Fig. 3. Location map of the Ceyhan Basin of Turkey.

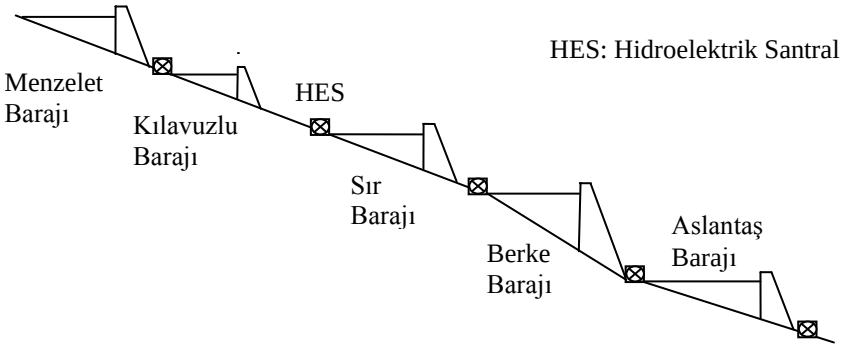

Fig. 4. A system with multiple reservoirs in the Ceyhan Basin.

Table 1. Basic characteristics of reservoirs in the system.

Reservoirs	Menzelet	Kılavuzlu	Sır	Berke	Aslanta
Drainage Area (km^2)	8340	8486	12950	13222	14874
Elevations (m)	609.4 - 485.5	485.5 - 440	440 - 345	345 - 159	159 - 84
Installed Power (MW)	124	54	273	510	138
Dam Height (m)	150,5	56	120	201	95
Maximum operational level (m)	609,4	485,5	440	345	159
Minimum operational level (m)	560,2	483,5	418,8	288,8	130
Maximum Volume (10^6 m^3)	1950	74	1117,94	427	1990
Minimum Volume (10^6 m^3)	533,6	69	451,3	119	530
HPP Elevation (m)	485,5	440	345	159	84
Maximum Spillway Capacity (m^3/s)	6112	6112	7460	2000	10700
Maximum Release for Energy Production (m^3/s)	130,8	148,4	114,25	300	270

HPP: Hydroelectric Power Plant

Table 2. Volume-height relationships for reservoirs.

Reservoirs	Menzelet	Kılavuzlu	Sır	Berke	Aslantaş
a	30,898	24,182	45,328	76,157	9,5129
b	0,2998	0,4092	0,2067	0,2581	0,3947

($h=a.S^b$, h: Height (m), S:Volume or storage capacity (10^7 m^3))

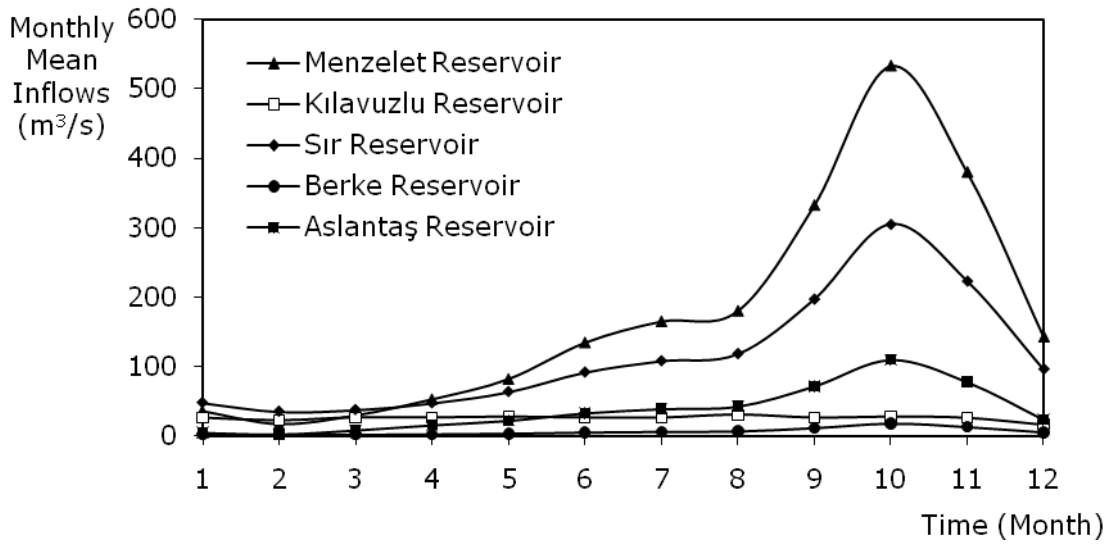

Fig. 5. Monthly mean inflows into reservoirs from sub-drainage areas.

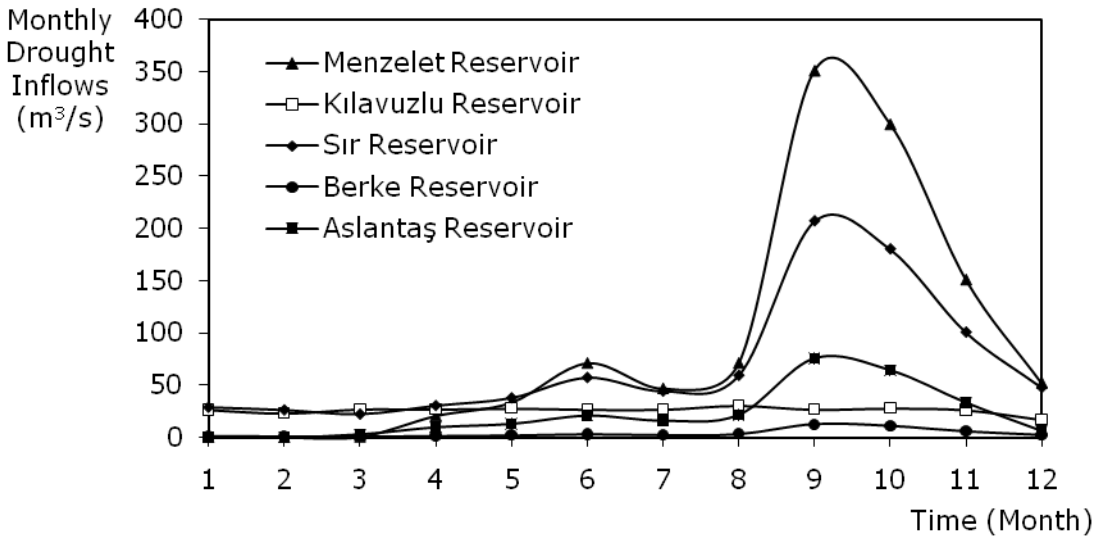

Fig. 6. Monthly drought inflows into reservoirs from sub-drainage areas.

For the DPSA solution technique $10x10^6$ m^3 discrete volumes both for storages and releases have been used, with operations starting at the beginning of the drought season (May) when the reservoirs were full.

RESULTS

At the end of the optimization process, firm and average powers are obtained as 163.970 MW and 438.340 MW, respectively. Table 3 is prepared by using Equation 18 proposed of the DSİ approach. According to this approach, firm power is obtained as 144.980 MW. As a result, The firm power value obtained from optimization model is presented 13 % bigger than its value obtained from DSI approach

Table 3. Firm power values of the reservoirs.

Reservoirs	V_a (10^6 m^3)	q_{Min} (10^6 m^3)	h_{Min} (m)	h_a (m)	P_G (MW)
Menzelet	1416,4	0	74,7	49,2	39,081
Kılavuzlu	5	23,37984	43,5	2	3,286
Sır	666,64	49,3776	73,8	21,2	28,419
Berke	308	50,9328	129,8	56,2	39,463
Aslantaş	1460	50,9328	46	29	34,732
				Total	144,980

Table 4 is presented by using Equations 19 and 20 proposed of DSI approach. According to this approach, the average power is obtained as 173,151 MW. The average power value obtained from optimization model is presented 153 % bigger than its value obtained from DSI approach

Table 4. Average power values of the reservoirs.

Reservoirs	$q_{i_{ort}}$	$h_{i_{maks}}$	$\sum q_{i_{ort}} . h_{i_{ort}}$	q_{ort}	$\sum h_{i_{maks}}$	P_{ort} (MW)
Menzelet	66	123,9	8170			
Kılavuzlu	76	45,5	3463			
Sır	120	95	11367	107	525,4	173,151
Berke	122	186	22741			
Aslantaş	137	75	10252			

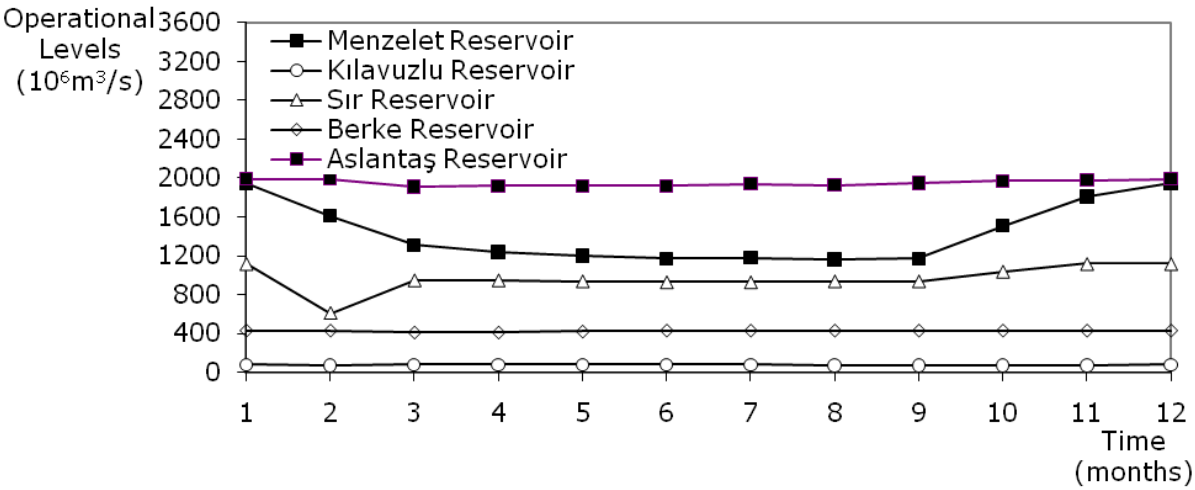

Fig. 6. Minimum operational levels in reservoirs.

In addition, when operational levels obtained from optimization process are illustrated, as shown in Figures 6 and 7, it has controlled and managed by reservoirs with large storage.

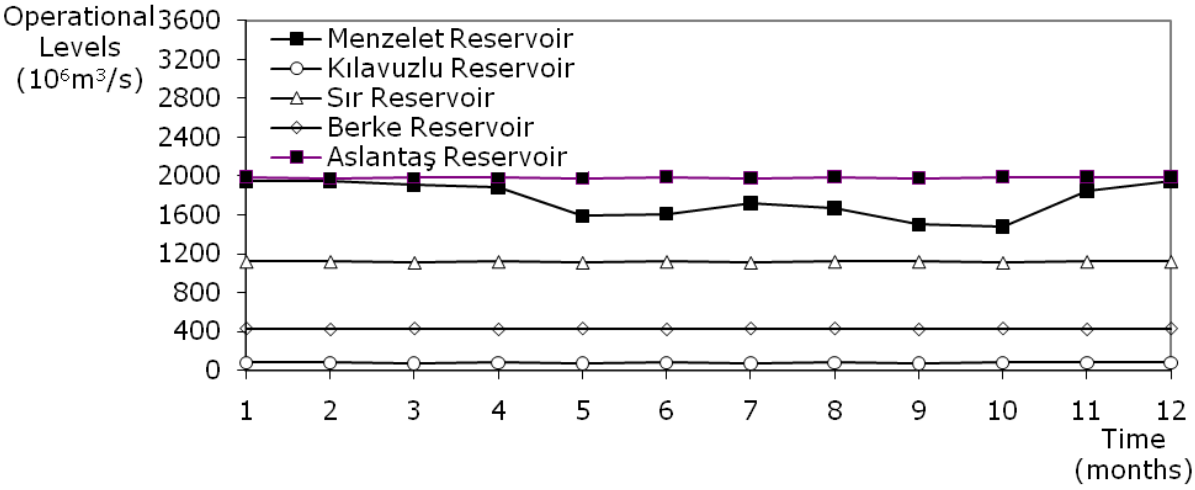

Fig. 7. Normal operational levels in the reservoirs.

CONCLUSIONS

An optimization model with energy production objective for long term planning to a multi-object water resource system with multiple reservoirs presented on the Ceyhan River in the Ceyhan Basin of Turkey is developed. Firstly, the firm power is maximized by using this model. Then, the total energy of the system is maximized. The firm power and the total energy of the system by using equations proposed of DSI approach are determined. The firm power value obtained from optimization model is presented 13 % bigger than its value obtained from approach of DSİ and also total energy production of the system is 153%. When operational levels obtained from optimization process are illustrated, it has controlled and managed by reservoirs with large storage. In the next studies, the developed model can be compared with HEC-5 simulation.

ACKNOWLEDGEMENTS

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COMPRESSIVE BEHAVIOR OF RUBBER-BASED SEISMIC ISOLATION BEARINGS

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Key Words: Seismic Isolation, Earthquake Resistant Design, Rubber Bearing, Compression Modulus, Rubber, Shape Factor, Poisson's Ratio, Fiber Reinforcement.

ABSTRACT In seismic isolation technique, which has recently been developed as an alternative to the traditional earthquake resistant design techniques, flexible and dissipative elements, called bearings or isolators, are inserted at the base of the structure to reduce the seismic force transmitted from the soil to the structure. This technique has an important advantage over most of the others in that, if properly used, it reduces two basic seismic response quantities, namely, floor accelerations and interstory drifts, simultaneously. A properly designed seismic isolated structure has sufficient flexibility due to the presence of an isolation system at its base to reduce the floor accelerations, and at the same time has sufficient rigidity with its rigidly moving superstructure on flexible isolators to reduce the interstory drifts. Composed of thin rubber layers bonded to interleaving reinforcing sheets, rubber bearings are now widely used in many seismic isolation approaches. In the design of a multilayered rubber bearing, it is generally assumed that the presence of the reinforcing sheets does not influence the horizontal behavior of the bearing. Thus, it is usually not so difficult to compute the horizontal stiffness of the bearing. On the other hand, the determination of the vertical or bending stiffness of such a bearing may not always be so easy since this requires well understanding of the behavior of "bonded" rubber layers. Studies on bonded elastic layers have clearly shown that there are three basic parameters that control the compressive behavior of a rubber bearing: shape factor of the bearing (i.e., the ratio of one loaded area of a single bonded rubber layer to its bulge free areas), compressibility of the rubber material and extensibility of the interior reinforcing sheets. This paper provides a review on the behavior of circular multi-layered elastomeric bearings under uniform compression with particular emphasis on the effects of the above-mentioned three basic parameters on their "compression modulus".

INTRODUCTION

According to the traditional seismic design philosophies, the resistance of a structure to high seismic effects is ensured by designing the structure in such a way that it will have either sufficiently high strength or sufficiently high ductility when it is subjected to strong ground excitations. One of the conflicts a structural engineer is faced with while designing such a structure is the requirement that neither the interstory drifts nor floor accelerations exceed the code-limited values. The drifts can easily be reduced by stiffening the structure, however, increasing the stiffness of the structure usually leads to the amplified accelerations. Thus, by using conventional structural systems, it is not usually possible to limit the drifts and accelerations simultaneously. On the other hand, it *is* possible to do this by using a recently-developed seismic design technique, named "seismic isolation". In this technique, flexible and dissipative elements, called bearings or isolators, are inserted at the base of the structure to reduce the seismic force transmitted from the soil to the structure. Accordingly, a properly designed seismic isolated structure has sufficient flexibility due to the presence of an isolation system at its base to reduce the floor accelerations, and at the same time has sufficient rigidity with its rigidly moving superstructure on flexible isolators to reduce the interstory drifts.

Although seismic isolation technique is known as a relatively new approach for earthquake resistant design, the idea of isolating a structure from movements is not new. Composed of thin rubber layers bonded to interleaving reinforcing sheets, rubber bearings were originally developed to isolate bridges from movements produced by creep and shrinkage of concrete, thermal expansion, and traffic-induced forces, and to isolate machines and equipment from mechanical vibrations. During the development of seismic isolation technique, it has been recognized that this type of bearings can also be effectively used in mitigating the devastating effects of earthquakes since a multi-layered rubber bearing incorporates two basic abilities that any seismic isolator has to have into a single unit; namely, flexibility under shear and rigidity under compression and bending. Free to deform under shearing forces, soft rubber layers provide lateral flexibility required to isolate the superstructure from lateral seismic loads. On the other hand, acting as confinement, reinforcing sheets limit the bulging of rubber layers to which they bonded, which increases the compression and bending stiffnesses of soft rubber layers, which, in turn, provides large vertical and flexural rigidities required to support the heavy weight of the superstructure safely. Nowadays, besides sliding bearings, rubber bearings are accepted as the most widely used seismic isolator types.

In the design of a multilayered rubber bearing, it is generally assumed that the presence of the reinforcing sheets does not influence the horizontal stiffness of the bearing. Thus, it is usually not so difficult to compute the horizontal stiffness of the bearing. On the other hand, the determination of the vertical or bending stiffness of such a bearing may not always be so easy since this requires well understanding of the behavior of "bonded" rubber layers. Studies on bonded elastic layers have clearly shown that there are three basic parameters that control the compressive behavior of a rubber bearing: shape factor of the bearing (i.e., the ratio of one loaded area of a single bonded rubber layer to its bulge free areas), compressibility of the rubber material and extensibility of the interior reinforcing sheets. This paper provides a review on the behavior of circular multi-layered elastomeric bearings under uniform compression with particular emphasis on the effects of the above-mentioned three basic parameters on their "compression modulus".

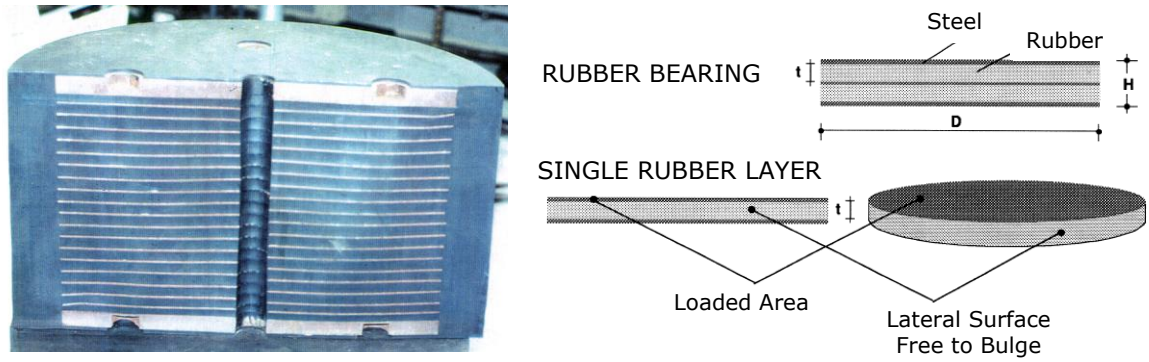
THEORY

Two basic parameters governing the seismic behavior of a multilayered rubber bearing, namely horizontal stiffness (K_H) and vertical stiffness (K_v), can be computed by using the following well-known expressions (Naeim and Kelly, 1999):

$$K_H = \frac{GA}{t_r} \quad \text{and} \quad K_V = \frac{E_c A}{t_r} \tag{1}$$

where A is the cross sectional area of the reinforcing sheets, t_r is the total rubber thickness, G is the shear modulus of the rubber and E_c is called the "effective compression modulus", or simply the "compression modulus", of the bearing. From Eq.(1), it can be seen that among the three parameters in K_V expression, two of them (A and t_r) also appear in K_H expression. Thus, it can easily be said that it is the compression modulus which controls the vertical stiffness of a multilayered elastomeric bearing. From Eq.(1), it can also be seen that although the shear modulus (G) of the rubber can directly be used as the shear modulus of the bearing, elasticity modulus (E) of the rubber cannot be used as the compression modulus of the bearing. This behavior difference comes from the fact that while the reinforcing sheets bonded to top and bottom faces of interior rubber layers do not have significant influence on deformation pattern of rubber layers under shearing forces, they can restrain free bulging of rubber layers under compression considerably and as stated by Lindley (1968), "for materials such as rubber which have a low shear modulus but a relatively high bulk modulus, any restrictions on their freedom to change shape can have a very marked effect on their stiffness in compression".

Analytical studies performed on elastomeric bearings show that the behavior of a multi-layered elastomeric bearing (Figure 1a) under compression is controlled by the behavior of its typical interior "bonded rubber layer", which is composed of a rubber layer with reinforcing sheets at its top and bottom faces (Figure 1b).

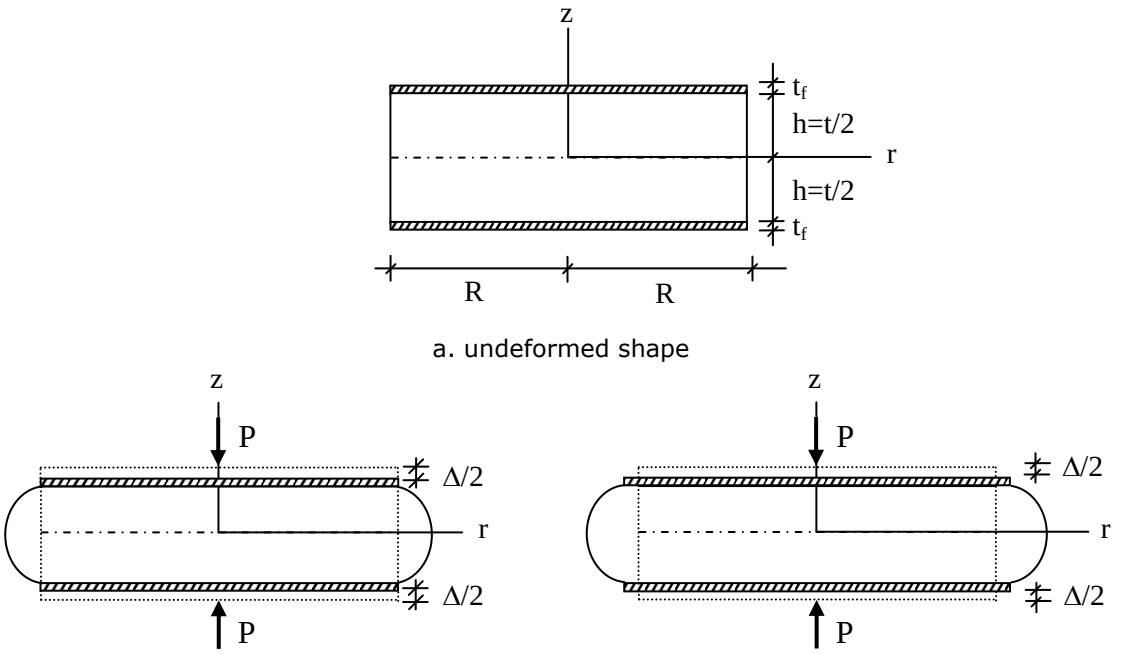


a. Circular steel reinforced bearing b. Typical interior "bonded" rubber layer

Fig. 1. A typical circular steel-reinforced rubber bearing (taken from Kelly (2007)) with its typical bonded rubber layer (Imbimbo and DeLuca (1998)).

It is now very well known that the compression stiffness of a bonded elastic layer may be several orders of magnitude greater than its stiffness when it is *not* bonded, especially if the compressibility of the layer material is considerably low, as in the case of natural rubber, whose Poisson's ratio is reported by Lindley (1968) to be 0.49989, which is extremely close to the incompressible value, 0.5. This behavior change is directly related to the well-known Poisson's effect: any restraint on bulging tendency of the layer material changes stress distributions in the layer considerably, which in turn, cause significant increase in layer stiffness. As mentioned previously, while the reinforcing sheets at the faces of a rubber layer may influence the compressive behavior of the layer so greatly, they have almost no effect on its shear behavior since they do not provide any restraint on shear deformation of the layer. This characteristic of bonded rubber layers makes, in fact, multilayered rubber bearings ideal components for seismic isolation since they have large deformation capability under horizontal seismic excitations due to their low shear stiffness and at the same time have large stiffness under vertical loads and bending moments which are necessary to support the weight of the superstructure.

In the last century, many researchers (e.g., Gent and Lindley, 1959; Gent and Meinecke, 1970; Lindley, 1979) have studied the behavior of bonded elastic layers, specifically bonded rubber layers (Figure 2a). Most of these studies have been conducted to determine the compression stiffness of the rigidly-bonded rubber layers (simulating steel-reinforced rubber layers) (Figure 2b). These earliest studies put forward three basic assumptions for small deformation and linear analysis of bonded elastic layers: (i) horizontal plane sections remain plane after deformation, (ii) initially vertical lateral surfaces take a parabolic shape in the deformed configuration (parabolic bulging assumption), (iii) state of stress at any point in the material is dominated by the hydrostatic pressure ("pressure" assumption). These three assumptions are accepted as the fundamental assumptions of the linear theory developed for the analysis of bonded elastic layers and, due to the third assumption, the formulations derived based on these assumptions are frequently named as the "pressure method". In pressure method, first the pressure distribution in the rubber layer, which is assumed to be constant through the layer thickness, is determined. The axial load (P) on the bearing is, then, obtained by integrating the pressure function over the bonded face of the rubber layer. By dividing the nominal compressive stress, $\sigma_c=P/A$, to the nominal compressive strain, $\varepsilon_c=\Delta/t$ (Figure 2b), the compression modulus (E_c) of the bearing is determined.



b. deformed shape under uniform compression, rigid reinforcement case

c. deformed shape under uniform compression, flexible reinforcement case

Fig. 2. A circular rubber layer bonded to reinforcing sheets under uniform compression.

Assuming strict incompressibility (ν=0.5) and using the pressure method, Gent et al. (Gent and Lindley, 1959; Gent and Meinecke, 1970) derived simple expressions for compression modulus of rigidly-bonded elastic layers of various shape. In their analysis, they have considered that the total displacement of a bonded rubber layer subject to uniform compression is composed of the superposition of two simple displacements: pure homogeneous compression of the corresponding unbonded rubber layer and additional shear displacements to keep the bonded surfaces in their original positions. For a circular steel-reinforced bearing of radius R with interior rubber thickness of t (Figure 2b), their formulation leads to the following simple expression for incompressible, rigid-reinforcement compression modulus $E_c^{incomp,rigid}$:

$$E_c^{incomp,rigid} = E\left(1+2S^2\right) \quad \text{where} \quad S = \frac{R}{2t} \tag{2}$$

In Eq. (2), S is named as the "shape factor" of the bearing and can be computed from the ratio of one loaded area of a single rubber layer to its bulge-free lateral surface area (Figure 1b).

It is to be noted that due to the limited length of the paper, only the expressions for "compression modulus" of *circular* bearings, which have widespread applications in seismic isolation technique, will be presented in this paper. For closed form expressions derived in these studies for "stress distributions" and for the expressions obtained for compression modulus of *other shapes* with a more detailed review, Pinarbasi et al. (2006, 2008) and Pinarbasi and Mengi (2008) can be referred.

Realizing the significant effect of material compressibility in bearings with high shape factors (HSF bearings), Gent and Lindley (1959) also proposed the use of the either of the following modifications to find compressible, rigid-reinforcement compression modulus $E_c^{comp,rigid}$:

$$\frac{1}{E_c^{comp,rigid}} = \frac{1}{E_c^{incomp,rigid}} + \frac{1}{K} \qquad \text{or} \qquad E_c^{comp,rigid} = E\left(1 + 2kS^2\right) \tag{3}$$

where K is the bulk modulus of the rubber and k is an empirically determined factor ranging, depending on the hardness of the rubber, from 0.5 to 1.0, whose exact values can be obtained from Table 3 in Lindley (1974).

Incorporating the material compressibility directly into the pressure method, Kelly (1997) obtained the following expression for $E_c^{comp,rigid}$ of steel-reinforced circular bearings:

$$E_c^{comp,rigid} = K\left[1 - \frac{2I_1(\lambda R)}{(\lambda R)\, I_0(\lambda R)}\right] \qquad \text{where} \qquad \lambda^2 = \frac{12G}{Kt^2} \tag{4}$$

where I_0 and I_1 are modified Bessel functions of the first kind of zero and first orders, respectively. It can be recognized that Eq. (4) includes only the second stage solutions. It is known that as the shape factor of a bearing increases, the constant term appearing in compression modulus expression (see Eq. (2)) coming from the first stage solution, i.e., E term, becomes negligibly small compared to the shape-factor-dependent terms coming from the second stage. Thus, it is a common practice to ignore the first stage solutions in the design of HSF bearings. However, it should not be forgotten that it may be essential to include the first stage solutions corresponding to homogenous compression of the layer especially if the shape factor of the bearing is low (LSF bearings) and/or the compressibility of the elastomer is high.

As already stated, the pressure method assumes "fluid-like stress state". Removing this assumption, using a more direct solution in terms of *mean* pressure and keeping the remaining two basic assumptions of the pressure method in their formulation, Tsai and Lee (1998) derived the following closed-form expression for $E_c^{comp,rigid}$ of circular bearings:

$$E_c^{comp,rigid} = 2G + \lambda\left[1 - \left(\frac{\lambda}{\lambda + 2G}\right)\frac{1}{\dfrac{(\lambda^* R)\, I_0(\lambda^* R)}{2I_1(\lambda^* R)} - G}\right] \qquad \text{where} \qquad \left(\lambda^*\right)^2 = \frac{12G}{(\lambda + 2G)t^2} \tag{5}$$

where λ is the Lame's constant for the rubber. Comparing their solutions with the pressure solution, Tsai and Lee (1998) concluded that the pressure assumption is valid only for HSF bearings with nearly or strictly incompressible materials.

Recently, using a more advanced formulation and removing further the "plane sections remain plane" assumption, Pinarbasi et al. (2008) derived advanced closed-form solutions for displacement and stress distributions in circular and hollow circular bearings, which enabled them to investigate, in detail, the displacement and stress distributions and locations and magnitudes of critical stresses in the bearing. They also show that since in computation of compression modulus, nominal values of compressive stress and strain are used, E_c expression is *not* affected from the removal of the "plane sections remain plane" assumption. Their expression is exactly the same as that is given in Eq. (5).

In an attempt to develop light-weight and low-cost isolators, recent studies (Kelly, 1999, 2002; Moon et al., 2002, Kelly and Takhirov, 2001, 2002) have proposed the use of fiber reinforcement in place of steel reinforcement. Although the "rigid" reinforcement

assumption is a fairly reasonable assumption in the analysis of a steel-reinforced bearing, it is necessary to include the effect of reinforcement extensibility while studying the compressive behavior of a fiber-reinforced bearing (Figure 2c).

Using the pressure method and assuming incompressible rubber behavior, Tsai and Kelly (2001) derived the following expression for incompressible, flexible-reinforcement compression modulus $E_c^{incomp,flex}$ for a circular bearing of radius R with interior rubber thickness of t reinforced with flexible reinforcing sheets of equivalent thickness t_f (Figure 2c):

$$E_c^{incomp,flex} = \frac{k_f}{t}\frac{(1+\nu_f)}{2}\left[\frac{1-\dfrac{2I_1(\alpha R)}{(\alpha R)I_0(\alpha R)}}{1-(1-\nu_f)\dfrac{I_1(\alpha R)}{(\alpha R)I_0(\alpha R)}}\right] \quad \text{where} \quad \alpha^2 = \frac{12G}{k_f t} \quad \text{and} \quad k_f = \frac{E_f t_f}{(1-\nu_f^2)} \qquad (6)$$

In Eq. (6), E_f and ν_f are, respectively, the elasticity modulus and Poisson's ratio of the flexible reinforcing sheets.

In a recent study (not published yet), using the pressure method and including the rubber compressibility, the author derived the following expression for the compressible case:

$$E_c^{comp,flex} = \frac{k_f}{t}\frac{(1+\nu_f)}{2}\left[\frac{1-\dfrac{2I_1(\beta R)}{(\beta R)I_0(\beta R)}}{\left[1+\dfrac{k_f}{2Kt}(1+\nu_f)\right]-(1-\nu_f)\dfrac{I_1(\beta R)}{(\beta R)I_0(\beta R)}}\right] \quad \text{where} \quad \beta^2 = \lambda^2 + \alpha^2 \qquad (7)$$

Finally, removing both the pressure and the "plane sections remain plane" assumptions, the author derived (not published yet) the following advanced formula for $E_c^{comp,flex}$, which can be shown to yield the same results with the expression given in Tsai (2006).

$$E_c^{comp,flex} = (\lambda+2G) + \frac{2\lambda^2}{(\lambda+2G)}\left[\frac{\dfrac{I_1(\beta^* R)}{(\beta^* R)I_0(\beta^* R)}\left[1-\dfrac{\alpha^2}{(\lambda^*)^2}\dfrac{(1-\nu_f)}{(1+\nu_f)}\right]+\dfrac{\alpha^2}{(\lambda^*)^2}\dfrac{1}{(1+\nu_f)}}{\left\{\begin{array}{l}\dfrac{I_1(\beta^* R)}{(\beta^* R)I_0(\beta^* R)}\left[\dfrac{2G}{\lambda+2G}+\dfrac{\alpha^2}{(\lambda^*)^2}\dfrac{(1-\nu_f)}{(1+\nu_f)}\left(1+\dfrac{\lambda}{\lambda+2G}\right)\right] \\ -\left[1+\dfrac{\alpha^2}{(\lambda^*)^2}\dfrac{1}{(1+\nu_f)}\left(1+\dfrac{\lambda}{\lambda+2G}\right)\right]\end{array}\right\}}\right] \qquad (8)$$

where $(\beta^*)^2 = (\lambda^*)^2 + \alpha^2$.

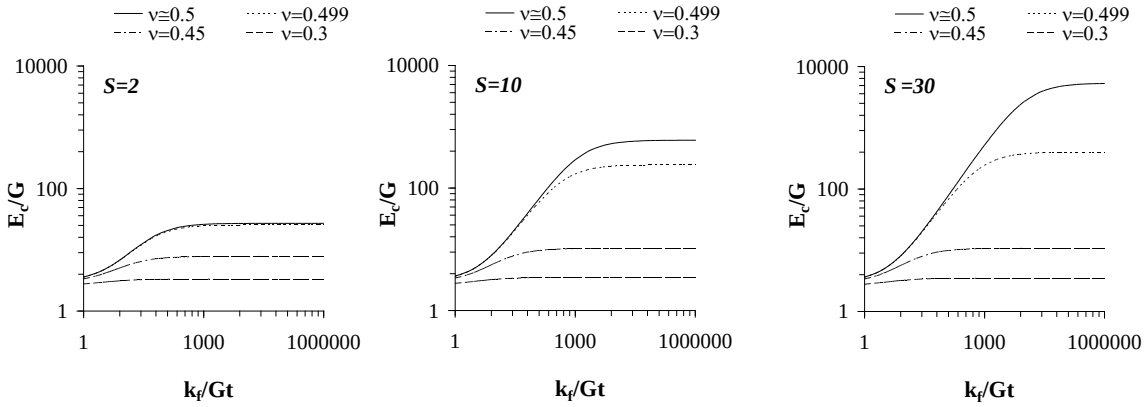
COMPRESSION MODULUS

The graphs in Figure 3 show the variation of compression modulus with reinforcement flexibility for bearings of three different shape factors; S=2, representing low-shape-factor (LSF) bearings, S=10, representing moderate-shape-factor (MSF) bearings and S=30, representing high-shape-factor (HSF) bearings. Horizontal axis plots $k^*_f = k_f / Gt$ ratio, which equals to the ratio of the in-plane stiffness of the reinforcement to the shear stiffness of rubber and can simply be called "stiffness ratio". It is to be noted that, unless otherwise stated, E_c values plotted in this paper are computed using Eq.(8), which considers both rubber compressibility and reinforcement extensibility and which excludes the pressure assumption. It is also to be noted that the studies indicate that the effect of ν_f on bearing behavior is not as much as the effects of S, ν or k_f. For this reason, the graphs presented in this paper are plotted for a fixed typical value of ν_f =0.3, which reduces the number of basic parameters governing the behavior of a fiber-reinforced bearing to three; namely, shape factor, rubber compressibility and reinforcement flexibility. To study the effect of rubber compressibility, the graphs in Figure 3 are plotted for four different values of Poisson's ratio; ν=0.5, corresponding to the incompressible case, ν=0.499, corresponding approximately to the natural rubber, ν=0.45, corresponding approximately to foam rubber, and ν=0.3, as a limiting case.

From Figure 3, it is seen that E_c of a fiber-reinforced rubber bearing approaches E_c of a steel-reinforced bearing as k^*_f approaches infinity. The value of stiffness ratio at which this asymptotic value is attained (approximately 100000 for S=30, 10000 for S=10 and 500 for S=2 for incompressible case) decreases as the shape factor of the bearing decreases. In other words, while for an HSF fiber-reinforced bearing to behave as if it were a steel-reinforced bearing, it is have to be reinforced with reinforcing sheets of considerably high stiffness, for an LSF bearing, there is no need to have such high stiffness values. From Figure 3, it can also be said that the value of k^*_f at which the rigid-reinforced behavior is attained also decreases as rubber compressibility increases.

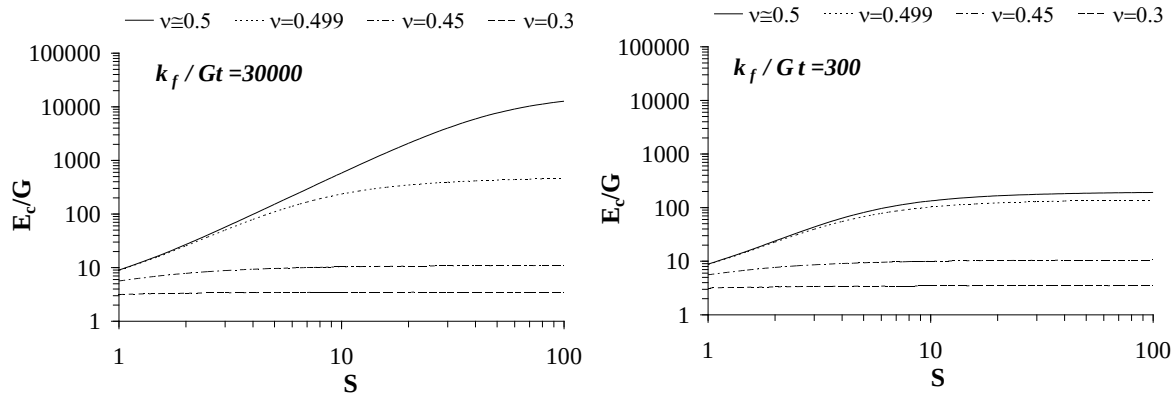
The effects of shape factor on compression modulus are more apparent in the graphs presented in Figure 4, which are plotted for two specific values of stiffness ratio, 30000 and 300, corresponding, respectively, to a considerably high and a relatively low stiffness ratio. It is worth noting that the value of 30000 for k^*_f, which can be considered as a typical value for a fiber-reinforced rubber bearing, is calculated using the typical values (E_f=210 GPa, ν_f=0.3, t_f=0.27 mm, t=3 mm, G=0.7 MPa) quoted in literature (e.g., Kelly 2002). Figure 4 clearly shows the significance of the inclusion of rubber compressibility in E_c computations especially if S and k^*_f are large, when the curves plotted for ν=0.499 deviate from ν=0.5 curves considerably. The deviation from incompressible behavior increases with increasing S and/or k^*_f. On the contrary, if S or k^*_f of a bearing is small, the designer can use the simpler incompressible formulas unless compressibility of rubber is considerably high.

The effect of rubber compressibility on compression modulus of a rubber bearing can also be seen from Figure 5, where E_c is plotted with respect to ν, for two reinforcement types; steel-reinforcement and flexible reinforcement with k^*_f=300. In the same figure, the predictions of the pressure solution are also plotted in discrete points so that the assessment of the pressure solution can also be made. It is to be noted that in order to show slightly, nearly and strictly incompressible values to the same plot, a kind of logarithmic scale is used in the horizontal axis. It can be seen that as ν increases, E_c increases until the asymptotic incompressible value of modulus is reached. An elastomeric bearing attains its "incompressible" modulus at a smaller value of ν if it has a smaller S and/or k^*_f. The graphs also show that pressure method can predict the compression modulus of HSF and MSF bearings satisfactorily unless rubber compressibility is considerably large. The deviation of discrete points from the continuous curves ν approaches zero is mostly due to disregarding the homogeneous compression of the bearing, which has to be considered if S and/or k^*_f and/or is ν low.



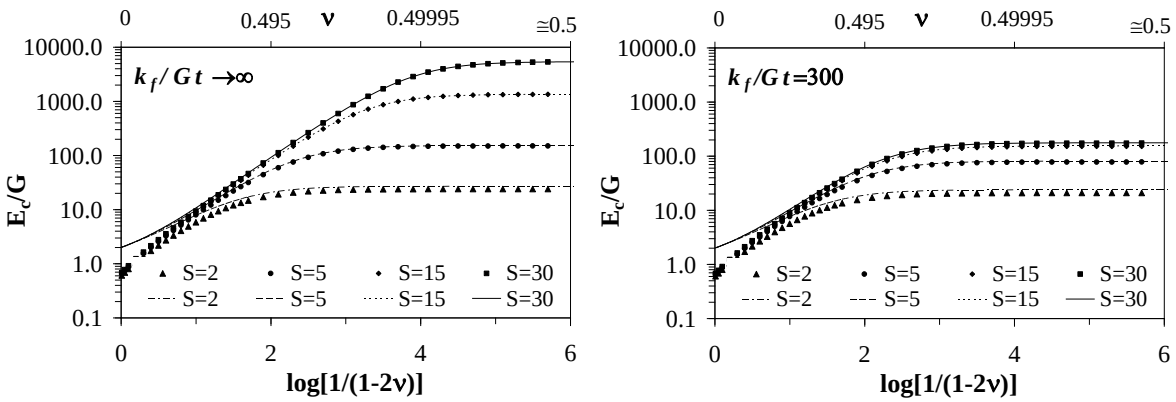
a. an LSF bearing (S=2) b. an MSF bearing (S=10) c. an HSF bearing (S=30)

Effect of reinforcement flexibility on compression modulus.



a. k_f/Gt=30000 b. k_f/Gt=300

Fig. 4. Effect of shape factor on compression modulus.



a. k_f/Gt→∞ b. k_f/Gt=300

Fig. 5. Effect of rubber compressibility on compression modulus of a circular bearing (discrete points shows the predictions of pressure solution).

STRESS DISTRIBUTIONS AND ASSESSMENT OF PRESSURE METHOD

While most of the studies on bonded rubber layers are conducted to derive closed-form expressions only for compression modulus of the layer, it is indeed also necessary to have comprehensive knowledge on detailed displacement and stress distributions and locations and magnitudes of critical stresses that will develop in a multi-layered rubber bearing under uniform compression for its rational design. In this section of the paper, stress distributions in circular bearings under compression will be discussed very shortly with some discussion on the effects of shape factor, material compressibility and reinforcement flexibility.

Figures 6 and 7 show the stress distributions in, respectively, radial and axial directions in a steel-reinforced elastomeric bearing of two different shape factors; S=2 and S=30. The graphs are plotted by using the advanced expressions derived by Pinarbasi et al. (2008). In the graphs, where radial stress is designated as τ_{rr}, axial stress as τ_{zz} and shear stress as τ_{rz}, stress values are normalized with respect to the uniform pressure $E_c\varepsilon_c$.

As it can also be seen from these graphs, in a multilayered rubber bearing, independent of the geometric and material properties, maximum radial stress is observed at the center of the reinforcement-rubber interface (i.e., r=0, z=±t/2); maximum axial stress at the center of the rubber layer (i.e., r=0, z=0) and maximum shear stress at the edges of the reinforcement-rubber interface (i.e., r=R, z=±t/2).

The effect of compressibility on stress distributions is striking. Although the presence of slight compressibility (ν=0.499) does not change the behavior of an LSF bearing, where the incompressible values can practically be used, ignoring the compressibility of rubber causes significant overestimation of stresses in an HSF bearing. As compressibility of rubber decreases, normal stress distributions changes from a a uniform shape (unbonded) to a parabolic shape (perfect bond) in radial direction. As well as the shear stresses are concerned, the LSF bearing appears to be subjected to more shear stress under the same amount of compression. However, it should be recognized that the plots in Figure 6 and 7 are normalized with respect to the compression modulus of the bearing. As it has already been discussed in the previous section, E_c of an HSF bearing can be much larger than that of an LSF bearing. Thus; the shear stress developed in an HSF bearing can be much larger than that in an LSF bearing. In fact, this can be seen from Figure 8, where the effect of reinforcement flexibility on maximum shear stress developing in a rubber bearing, which is one of the basic parameters in the design of seismic isolation bearings, is shown for S=2 and 30. Stress values in Figure 8a are normalized with respect to $E_c\varepsilon_c$ while those in Figure 8b to $G\varepsilon_c$. An order of difference between the stress values for the studied two shape factors can be recognized easily. These graphs also clearly indicate how the behavior of the bearing changes from rigid-reinforced behavior to the unbonded behavior as k^*_f decreases. The graphs also show the different effect of reinforcement flexibility to the bearings of different shape factors. While for an HSF bearing to behave rigid-reinforced, stiffness ratio has to be considerably high, there is no need to have such large values of stiffness ratio for an LSF bearing.

In the previous sections, it has already been discussed that the pressure solution is based on the pressure assumption (i.e., $-p=\tau_{rr}=\tau_{zz}$). The validity of this assumption can be assessed by comparing the normal stress plots in Figures 6 and 7. As shown in the figures, pressure assumption is valid only if ν is very close to 0.5 and shape factor of the bearing is large. It is worth noting that, although not presented here, it can easily be shown that this assumption looses its validity also when the reinforcement flexibility decreases.

In the previous sections, the formulations based on "mean pressure", which assumes "plane sections remain plane" are also be discussed. As shown in Figure 9, this assumption is valid in a wide range of shape factors in steel reinforced bearings. Actual stress distributions deviate from mean pressure only when S is considerably low (S<1).

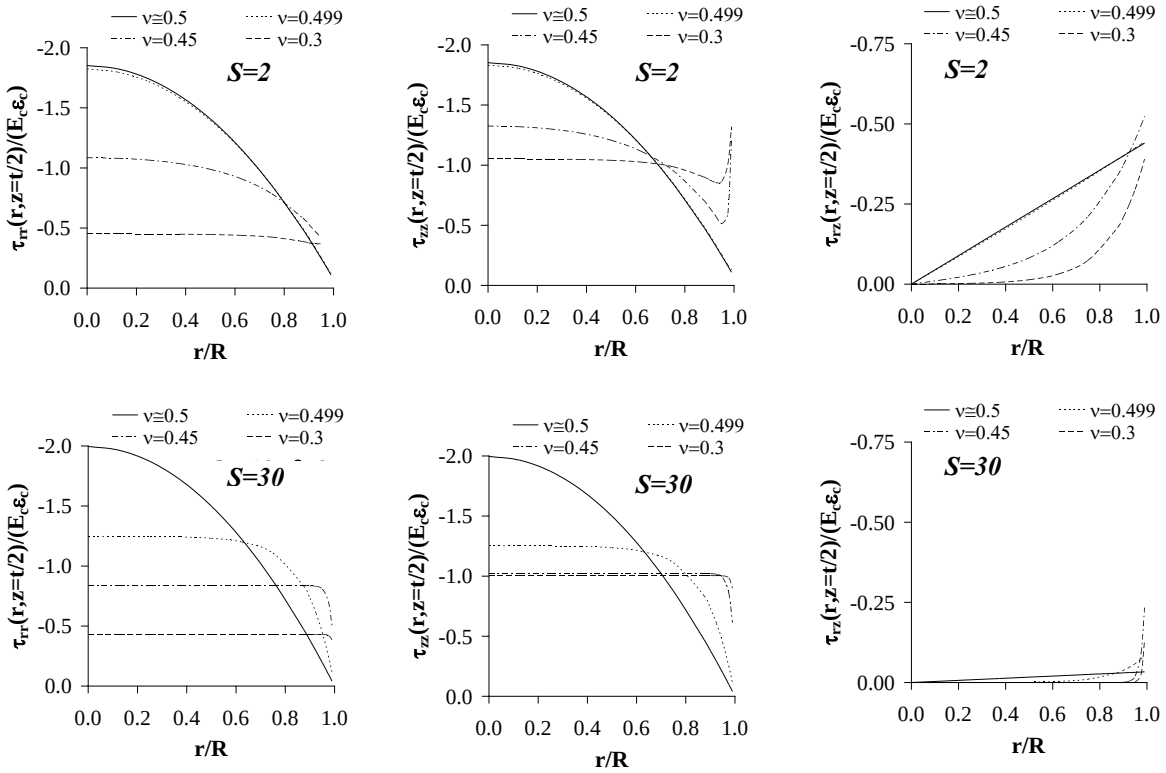


Fig. 6. Stress distributions over the steel-rubber interface in an LSF (S=2, top) and HSF (S=30, bottom) steel-reinforced ($k_f \to \infty$) circular bearing.

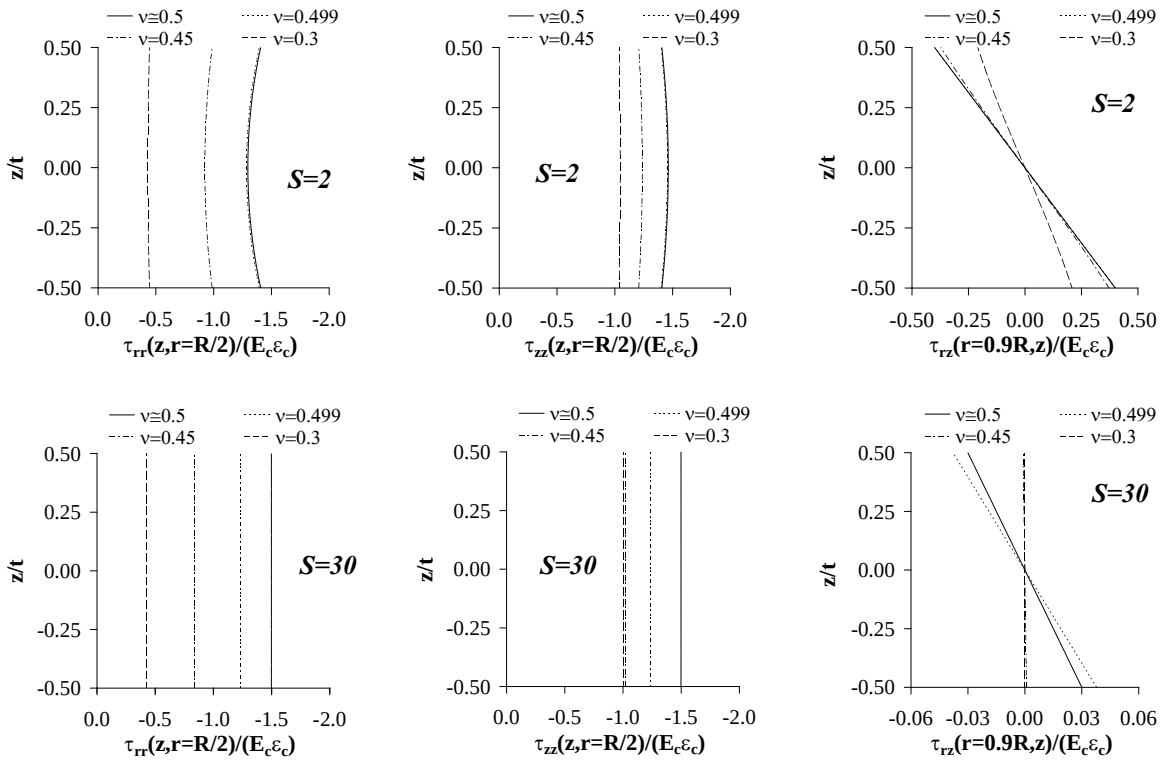
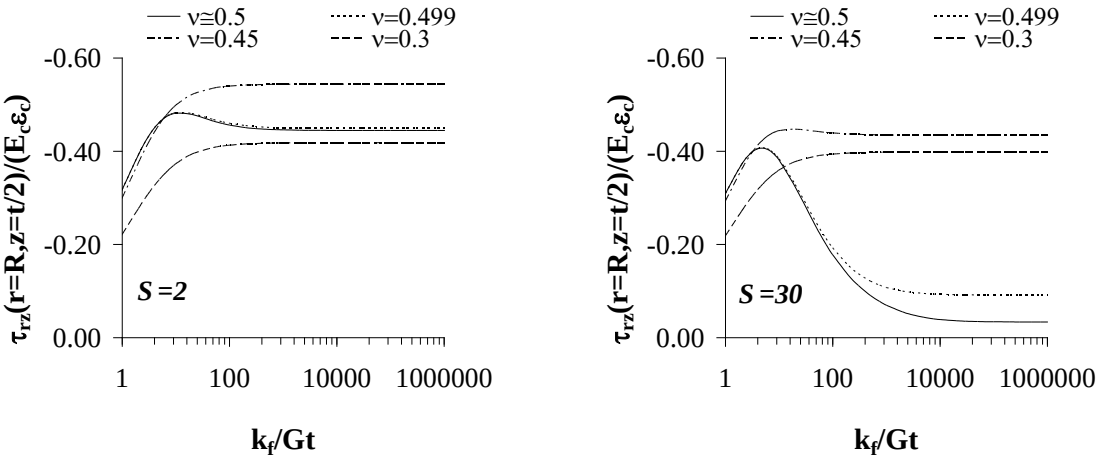
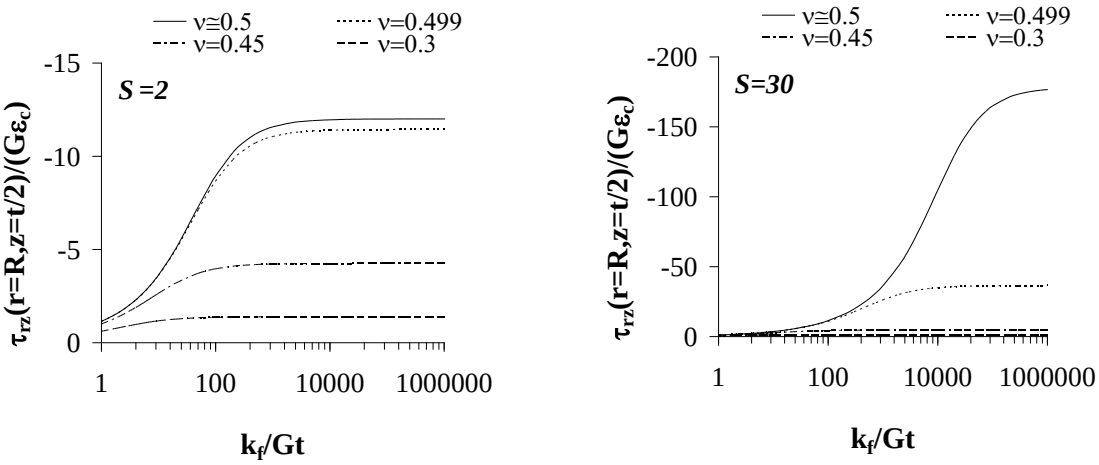


Fig. 7. Stress distributions through rubber thickness in an HSF (S=30) and LSF (S=2) steel-reinforced ($k_f \to \infty$) circular bearing.



a. stress values are normalized with respect to $E_c\varepsilon_c$



b. stress values are normalized with respect to $G\varepsilon_c$

Fig. 8. Effect of reinforcement flexibility on maximum shear stress in a circular bearing (Note that the graphs are plotted for ν_f=0.3.).

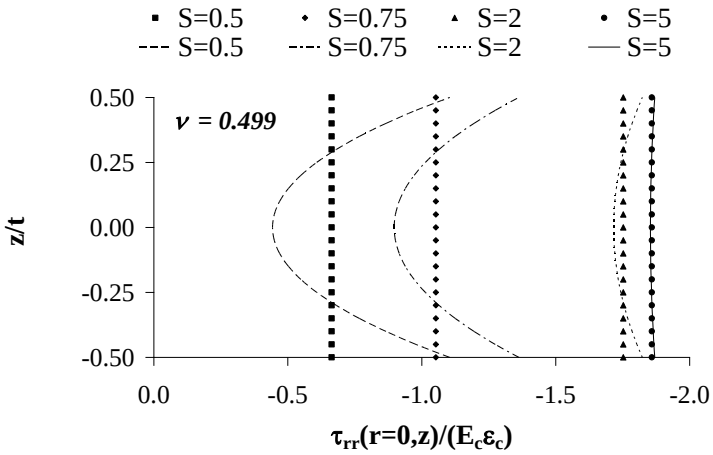


Fig. 9. Assessment of the "plane sections remain plane" assumption by comparing the radial stress distributions in steel-reinforced rubber bearings for low shape factors.

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CONTROL OF SEISMIC RESPONSE OF THREE-DIMENSIONAL MODEL STRUCTURE BY MAGNETORHEOLOGICAL DAMPERS

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Key Words: MR damper, semi-active control device, vibration control, seismic response reduction.

ABSTRACT Several lives and millions of dollars have been lost in recent earthquakes all over the world, including Kobe, Japan (1995), Kocaeli, Turkey (1999), Düzce, Turkey (1999), Chi-Chi, Taiwan (1999), Bhuj, India (2001), West Coast North Sumatra, Indonesia (2004), Kashmir, Pakistan (2005), Eastern Sichuan, China (2008). These detrimental effects due to earthquakes can be reduced by using control devices in civil structures for vibration reduction. The technology has received attention in recent years due to the afflicting losses in earthquakes. Three main categories exist for structural control: active, semi-active and passive control. Among these, semi-active control devices are more energy-efficient than active devices and are more effective in reducing structural vibrations than passive devices [Wilson]. Therefore semi-active control devices became popular in recent times. Furthermore instability problems in the sense of bounded-input-bounded-output stability do not occur since semi-active control devices do not add energy into the system being controlled. The scope of this study is on magnetorheological (MR) dampers, which are semi-active control devices that utilize magnetorheological fluids to generate controllable damping forces. In the present research a three dimensional model structure is excited individually by two horizontal components of the 1999 Düzce earthquake. The modified Bouc-Wen model proposed by Spencer et.al. is utilized for phenomenological model. A modified clipped-optimal acceleration feedback control strategy is applied to control the magnetorheological damper. The effectiveness of magnetorheological dampers to control vibrations of three dimensional structures are demonstrated through numerical examples.

INTRODUCTION

Semi-active control devices encompass the versatility and adaptability of active devices without requiring large power sources and also offer the reliability of passive devices. They have received significant attention in recent years. One of the popular semi-active control devices are magnetorheological (MR) dampers. They are semi-active control devices that utilize MR fluids to produce controllable damping forces. MR fluids are suspensions of micron-sized, magnetizable particles randomly dispersed in a carrier medium such as mineral or silicon oil. In the absence of a magnetic field, MR fluids are free-flowing, linear viscous fluid (similar to motor oil). But the iron particles form linear chains parallel to the field when exposed to a magnetic field and become a semi-solid behaving as a viscoplastic material.

Some advantages of MR dampers are their low power requirements, high yield strength that allows large force capacity, low viscosity, and stable hysteretic behavior over a wide temperature range. The most attractive property of the controllable fluids is that their ability to reversibly change from a free-flowing, linear viscous fluid to a semi-solid with a controllable yield strength in milliseconds when exposed to an electric or magnetic field [Dyke].

In the present research MR damper's mechanical behavior is modeled by the modified Bouc-Wen model. A semi-active optimal control algorithm which consists of two stages is utilized. the desired control forces are calculated by the linear quadratic regulator (LQR) method.

MECHANICAL MODEL

The phenomenological model which is proposed by Spencer et.al. (Spencer, 1997) is utilized for numerical calculations. The MR damper is governed by seven simultaneous equations in the modified Bouc-Wen model, shown in Figure 1.

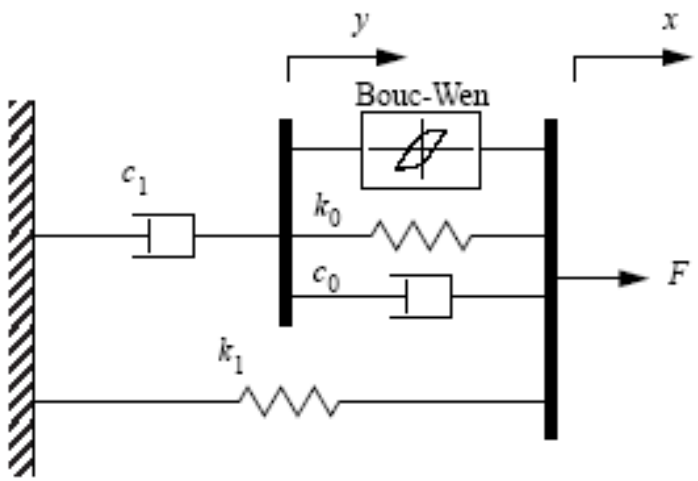

Fig. 1. The modified Bouc-Wen model.

The hysteresis damping force can be expressed as,

$$F = c_1\dot{y} + k_1(x - x_0) \tag{1}$$

$$\dot{y} = \frac{1}{c_0 + c_1}[\alpha s + c_0\dot{x} + k_0(x - y)] \tag{2}$$

$$\dot{s} = -\gamma|\dot{x} - \dot{y}|s|s|^{n-1} - \beta(\dot{x} - \dot{y})|s|^n + A(\dot{x} - \dot{y}) \tag{3}$$

where s is the evolutionary variable of the Bouc-Wen model. The viscous damping observed at high velocities and the viscous damping for the force roll-of at low speeds are represented by c_0 and c_1 respectively. k_1 is the accumulator stiffness and x_0 is the initial displacement of spring k_1 associated with the nominal damper force due to the accumulator. k_0 is the stiffness at high velocities. x and y are the external excitation and the internal variable of the Bouc-Wen model, respectively. γ, β, and A are characteristic parameters. n is the shaping parameter of the hysteresis.

$$c_0(u) = c_{0a} + c_{0b}u, \qquad c_1(u) = c_{1a} + c_{1b}u, \tag{4}$$

$$\alpha(u) = \alpha_a + \alpha_b u, \quad \dot{u} = -\eta(u - v) \tag{5}$$

v is the input voltage sent to the current driver governing the MR damper, and u is the armature voltage. The parameters of the magnetorheological damper are given in Table 1.

Tablo 1. Parameters of the MR damper model.

Parameter	Value	Parameter	Value
c_{0a}	2100 N s/m	α_a	14000 N/m
c_{0b}	350 N s/m V	α_b	69500 N V/m
c_{1a}	28300 N s/m	γ	3630000 m^{-2}
c_{1b}	295 N s/m V	β	3630000 m^{-2}
k_0	4690 N/M	A	301
k_1	500 N/M	n	2
x_0	0.143 m	η	190 sec^{-1}

SEMI-ACTIVE OPTIMAL CONTROL

In the current research the desired control forces are determined by a modified clipped optimal control algorithm. The controller consists of two stages: a linear optimal control part and a modified clipped algorithm. In the optimal control algorithm, control signals that will cause the system to satisfy some physical constraints and at the same time maximize or minimize a chosen performance criteria (cost function) are determined. The block diagram of the semi-active control system is presented in Figure 2.

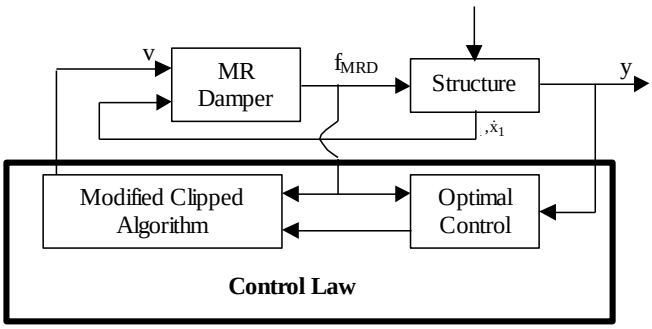

Fig. 2. Block diagram of the semi-active control system.

By choosing the optimal control algorithm, control signals that will cause the system to satisfy some physical constraints and at the same time maximize or minimize a chosen performance criteria (cost function) are determined. A linear optimal controller $\mathbf{K}_c$ is designed in order to provide the optimal control force u_c based on the states $\mathbf{x}$ which are the story displacements and velocities. The linear quadratic regulator (LQR) computes an optimal control $u_c = -\mathbf{K}_c\mathbf{x}$ to stabilize the plant and to minimize a cost function given by

$$J = \int_0^\infty (\mathbf{x}^T \mathbf{Q}\, \mathbf{x} + u_c{}^T R\, u_c)dt \tag{8}$$

where $\mathbf{Q}$ and R are weights of states and control. The optimal control is given by

$$u_c = -R^{-1}\mathbf{B}^T P\, \mathbf{x} \tag{9}$$

where P is the solution of the algebraic Riccati equation given in (10). $\mathbf{A}$ and $\mathbf{B}$ matrices are the system matrices in state space representation.

$$\mathbf{A}^T P + P\mathbf{A} + \mathbf{Q} + P\mathbf{B}R^{-1}\mathbf{B}^T P = \mathbf{0} \tag{10}$$

The **Q** values are 0.001 for displacements and are 0.01 for velocities. The states are treated as uncoupled. Thus the off-diagonal terms of the **Q** matrix are zero. The R value for each controller is 1e-8. R should be strictly positive definite. Actually the important issue in LQR is not the individual values of **Q** and R but the ratio between them.

The magnetic field in the damper is set by a modified clipped controller to develop damping forces that are equal to those obtained by the optimal control. The MR damper is driven by the magnetic field around it, hence driven by the voltage applied to the electromagnet. The applied voltage is set by a clipped algorithm to obtain the desired forces by the MR damper. When the MR damper generate the desired control force u_C, the voltage should be kept at the present level ($u_C = f_{MRD}$). If damper force's magnitude is smaller than the desired control force's magnitude and both forces have the same sign, then maximum voltage should be applied in order to increase the MR damper force and to catch the desired control force level. Otherwise, zero voltage should be sent. The algorithm explained above is graphically represented in Figure 3 and is stated as

$$v = v_{ref}\ H\{(u_c - f_{MRD}) f_{MRD}\} \tag{11}$$

$$v_{ref} = \begin{cases} \dfrac{u_c}{f_{MRDmax}} v_{max}, & \text{for } u_c \le f_{MRDmax} \\ v_{max}, & \text{for } u_c > f_{MRDmax} \end{cases} \tag{12}$$

where v is the applied voltage, v_{ref} is an intermediate voltage value, v_{max} is the maximum voltage, u_C is the desired control force, f_{MRD} is the force generated by MR damper, f_{MRDmax} is the maximum force capacity of MR damper, and H{.} is the Heaviside step function.

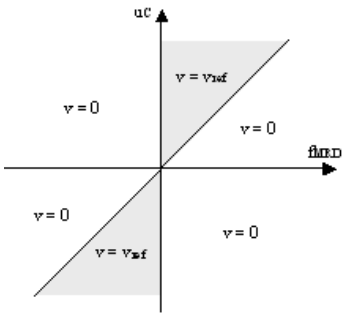

Fig. 3. Graphical representation of clipped control algorithm.

In the clipped control strategy, the command voltage is either zero or maximum. In order to avoid the damper to work at higher force levels, the voltage value is proportioned according to the calculated control force in the modified control algorithm. The modified clipped algorithm is given in Figure 4.

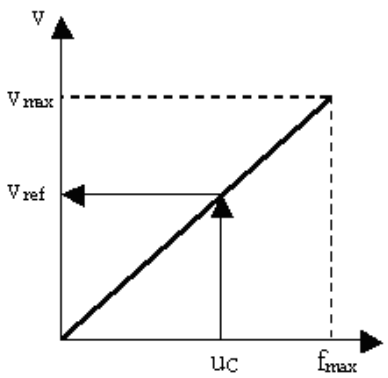

Fig. 4. Graphical representation of modified clipped control algorithm.

NUMERICAL SIMULATIONS

The seismic response of a three-dimensional model structure is simulated. The equations of motion for the model structure is written by assuming that the controlled building response remains in the linear region.

$$\mathbf{M}_s\ddot{\mathbf{x}}_d + \mathbf{C}_s\dot{\mathbf{x}}_d + \mathbf{K}_s\mathbf{x}_d = -\mathbf{\Gamma} f_{MR} - \mathbf{M}_s\mathbf{\Lambda}\ddot{x}_g \tag{13}$$

where $\mathbf{M}_s$, $\mathbf{C}_s$, $\mathbf{K}_s$ are the mass, damping, and stiffness matrices, respectively. $\mathbf{x}_d$ are the floor displacements relative to the ground. $\mathbf{\Gamma}$ and $\mathbf{\Lambda}$ are the location matrices of the control force and the external excitation. They specify how the control force $\mathbf{f}_{MR}$ and the ground excitation $\ddot{x}_g$ enter into the system. $\mathbf{f}_{MR}$ is the force generated by the MR damper.

The model structure is given in Figure 5. The whole system is reduced to a smaller system by some assumptions in order to decrease the computation time. But the system reduction is not mentioned here in detail. The first floor's response values are bounded to the responses of the fourth node. Similarly, the third floor's response values are bounded to the eighth nodes'.

The displacement values of the nodes which the dampers are attached are required during calculations. Furthermore, the floor response values are plotted to see the improvements in the controlled case. Therefore fourth, sixth, and eighth nodes' displacements in x- and y-directions and their rotations about z-axis are calculated. Eleventh and twelfth nodes' displacements in x- and y-directions are obtained.

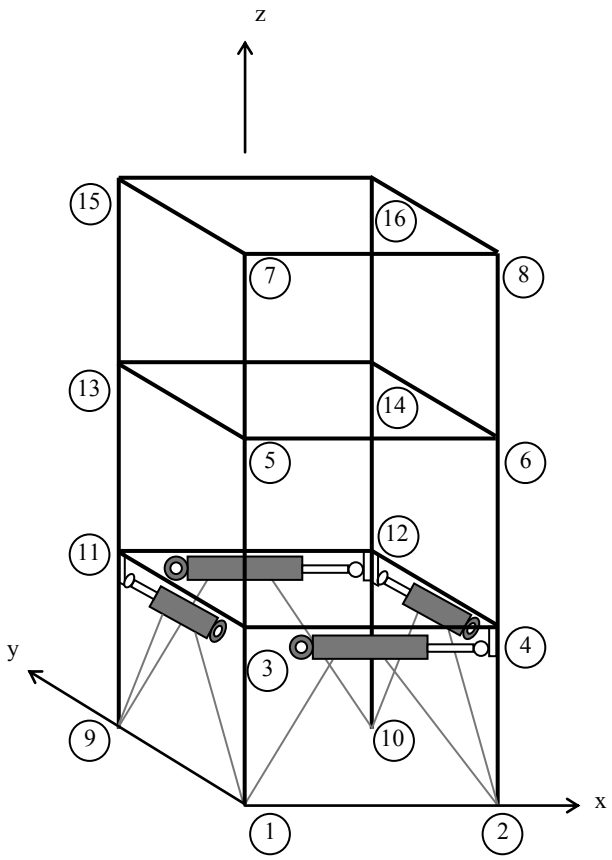

Fig. 5. Three-dimensional model structure.

Four MR dampers are attached between the ground and the first floor. The displacement of each damper is independent of each other. Therefore their responses are treated

individually in the code. The dampers in x-direction are attached to fourth and twelfth nodes. The dampers in y-direction are attached to eleventh and twelfth nodes.

The ground excitation is applied in x- and y-directions. The North-South and East-West components of the Bolu station's acceleration record of the 1999 Düzce earthquake are utilized (Figure 6).

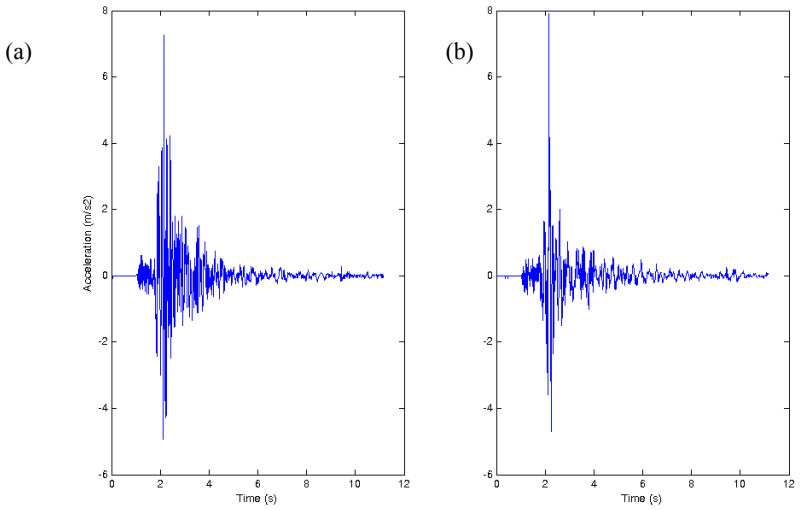

Fig. 6. The (a) N-S and (b) E-W components of the Bolu station's acceleration record of the 1999 Düzce earthquake.

The displacement response of node4 on the first floor are given in Figure 7(a) and (b) in x- and y-directions, respectively.

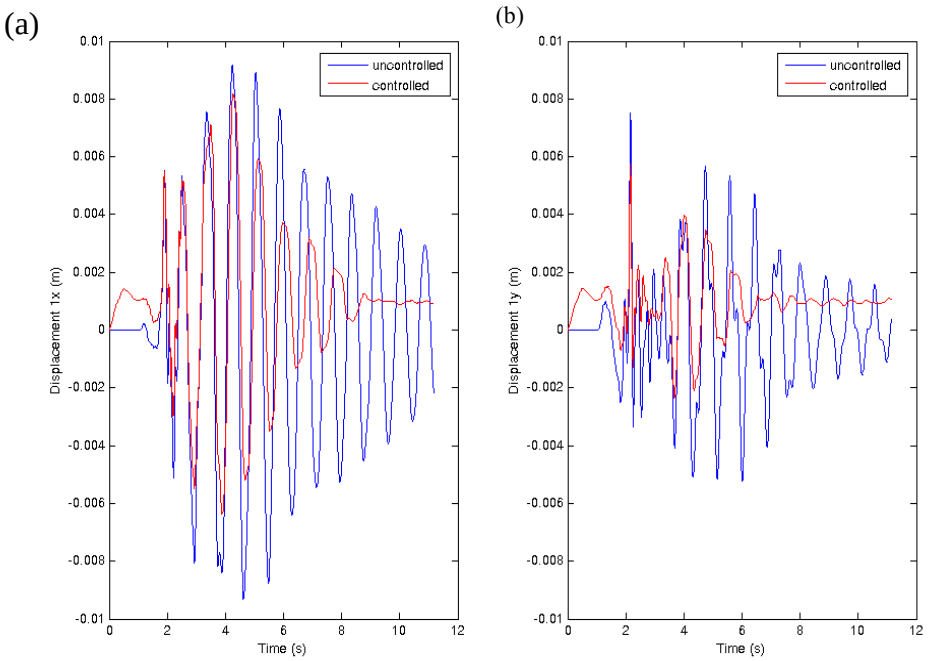

Fig. 7. The displacement responses of node4 on the first floor in (a) x- and (b) y-directions.

The displacement response of node8 on the third floor are given in Figure 8(a) and (b) in x- and y-directions, respectively. The reductions in displacements are satisfactory comparing to the uncontrolled case.

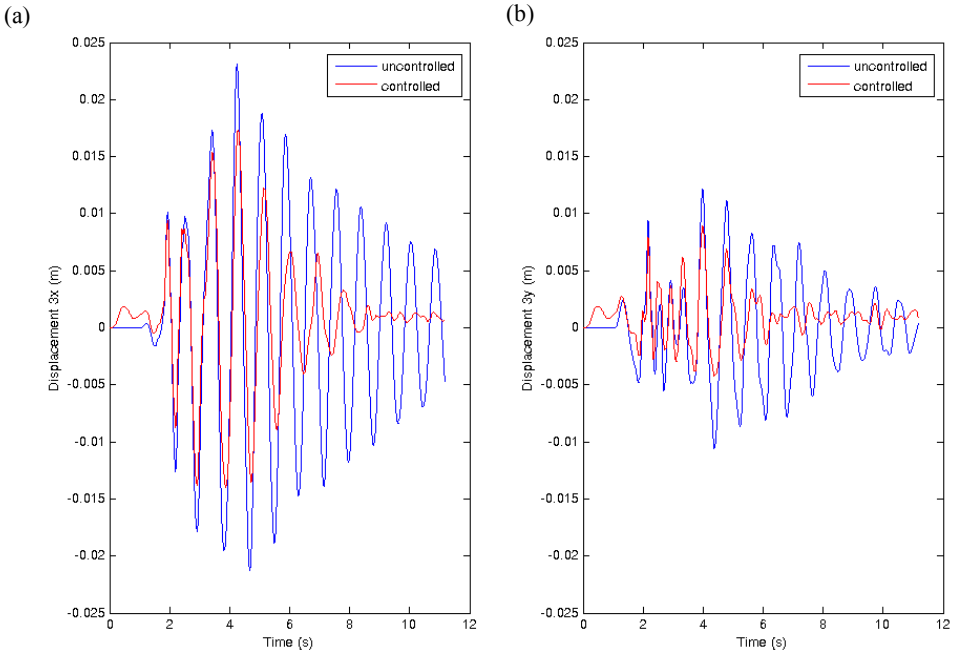

Fig. 8. The displacement responses of node8 on the third floor in (a) x- and (b) y-directions.

The acceleration response of node4 on the first floor are given in Figure 9(a) and (b) in x- and y-directions, respectively. The first floor's acceleration values are not reduced significantly by the controller. This fact may be due to the evidence of the dampers which supply external force at the first floor level. The acceleration time response of node8 on the third floor are given in Figure 10(a) and (b) in x- and y-directions, respectively.

Actually second and third floors' absolute accelerations give more correct information about the behavior of the structure. Because first floor absolute accelerations contain not only the structure's accelerations, but also accelerations coming from the MR damper forces.

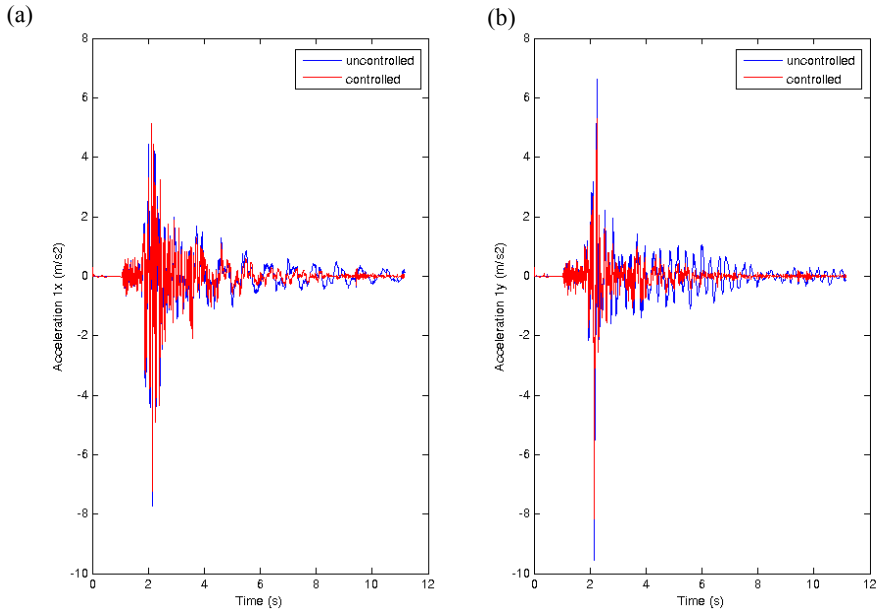

Fig. 9. The acceleration responses of node4 on the first floor in (a) x- and (b) y-directions.

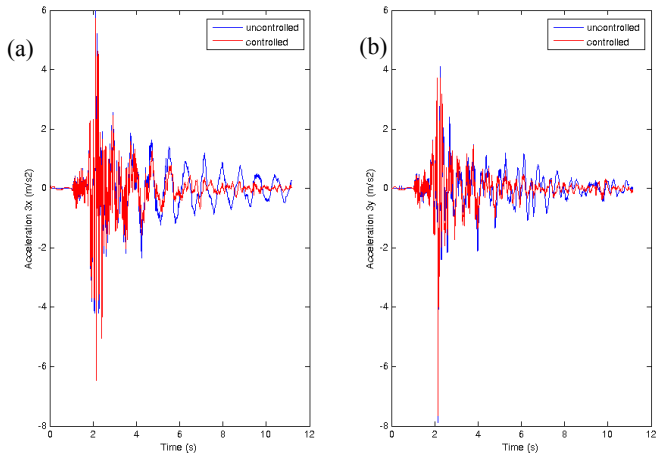

Fig. 10. The acceleration responses of node8 on the third floor in (a) x- and (b) y-directions.

CONCLUSIONS

The response of magnetorheological dampers which are attached to a three dimensional model structure is investigated in the content of the current research. The model structure is excited by two horizontal earthquake components.

Acceptable reductions in maximum values are observed. After 8 seconds the controlled response becomes very small comparing to the uncontrolled one. On the other hand, the improvements in the maximum values of accelerations are not as good as the displacements'.

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CREATING THE LITHOLOGY MAPS OF ERENLER REGION IN SAKARYA BY USING GEOGRAPHIC INFORMATION SYSTEMS (GIS)

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Key Words: Lithology, gis, alluvial, erenler

ABSTRACT Erenler Region is located on southern part of Sakarya province. In this study, we aimed to point out the lithologic distribution in the study area on GIS based maps. For this purpose, 488 drilling data collected and transferred to numerical domain. The study area and also Sakarya province is located on thick alluvial deposits. Becose of the historical earthquakes and the active tectonism, lithologics have quite complex conditions in the alluvial deposits. We carried out a lithology classification before the lithology maps were prepared. For the lithology classification, considered only gravel, sand, silt and clay soils and their mixtures, ignored the placticity, firmness, softness of the soils. According to lithology classification, lithology number came down to 37 from 531. Then, 0,5-1 m, 1,5 m and 3 m depth lithology maps are formed by GIS. 0,5-1 m depth lithology map show that, the study area covered by botanical soils at related depths. It is clear that on the other maps, 1,5 m depths are controlled by silty soils, conversely 3 m depths are controlled by clayey soils.

INTRODUCTION

Turkey is located on an active tectonic structure and has a suitable topography for natural disasters. It is necessary to determine the better soil sides in settlement areas and constitute the microzonation maps. Microzonation maps can created by collected, saved and updated the soil parameters data of settlement areas.

Erenler Region is located on southern part of Sakarya that has a very active tectonism features according to the North Anatolian Fault System. The principal reason of disasters stem from earthquakes is high storey buildings on this alluvial soil that built without any soil analysis. In settlement areas which have earthquake risk, soil conditions can clearly determine with microzonation maps that created by using GIS and reliable interpretations can be presented by this way.

In this study, investigated the lithologic distribution in Erenler Region by GIS based maps. 488 drilling data collected and transferred to numerical domain. The authors carried out a lithology classification for the study area.

GEOLOGICAL SETTING

Sakarya province formed by Quaternary aged alluvium that spread out large area in this region. This alluvium unit constituted Adapazari, Akyazi, Hendek plains and consist of sand-silt-clay-gravel which are not firm [Akartuna, 1968 and Önalp, Bol, 1998]. Adapazari is the central district of Sakarya province and situated in the west part of the Sakarya plain surrounded by mountain chains on four sides. The altitude of Adapazari is about 31 m above sea level. Quaternary sediments carried primarily by the Sakarya River and also by Çark Suyu (River) and Mudurnu River from NAFZ and the rocks at the South of the fault were deposited in the Sakarya Plain. The accumulation of thick alluvial deposits reaches a thickness up to 300 m at the central part of the plain (Beyen, Erdik, 2004). Komazawa et al., 2001, claimed that the thickness of this alluvium is approximately 1000 m.

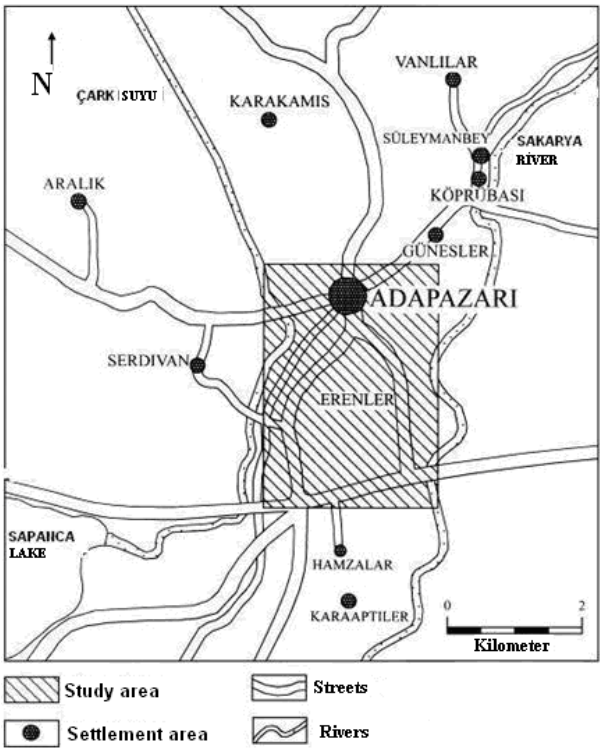

Fig. 1. Location of the Erenler region in Sakarya province.

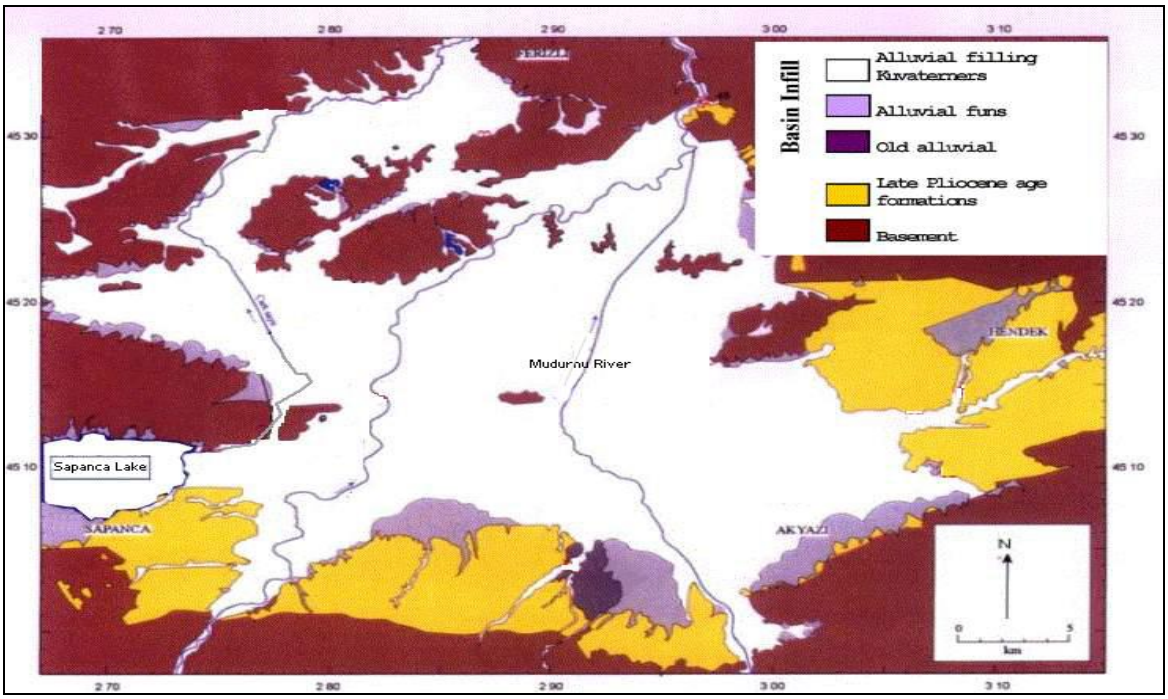

Fig. 2. Simplified Adapazari plain surface geology map (Beyen, Erdik, 2004).

The surface geology of the Sakarya plain as shown in the simplified map in Fig. 2 can be divided into mainly five different formations (Beyen, Erdik, 2004). They are, namely, alluvial filling kuvaterners, alluvial funs, old alluvial, late Pliocene age formations and basement. In the southern parts of the city, there seems to be firm ground on one side and hilly surface formations up to 160 m as an outcrop of the basement rock on the other side of the passage at the entrance to the city from intercity highways. Towards the north in the cross-section, recent alluvial deposit, consisting of loose silty, soil cover

a surface 7 km in length. The west part of the city is located on hill-like formations and outliers of mountains next to Lake Sapanca, the eastern and central parts as well as many other districts, are located in the sedimentary basin which extends towards southeast Akyazi 25 km in length.

In Erenler Region , there are two main geological formation. These are Eocene aged flysch and Quaternary aged alluvium. The high topographic areas covered by flysch and all plain areas covered bye alluvium. The flysch consist of claystone, marl, siltstone and sandstone. The alluvium consist of clay, silt and sand.

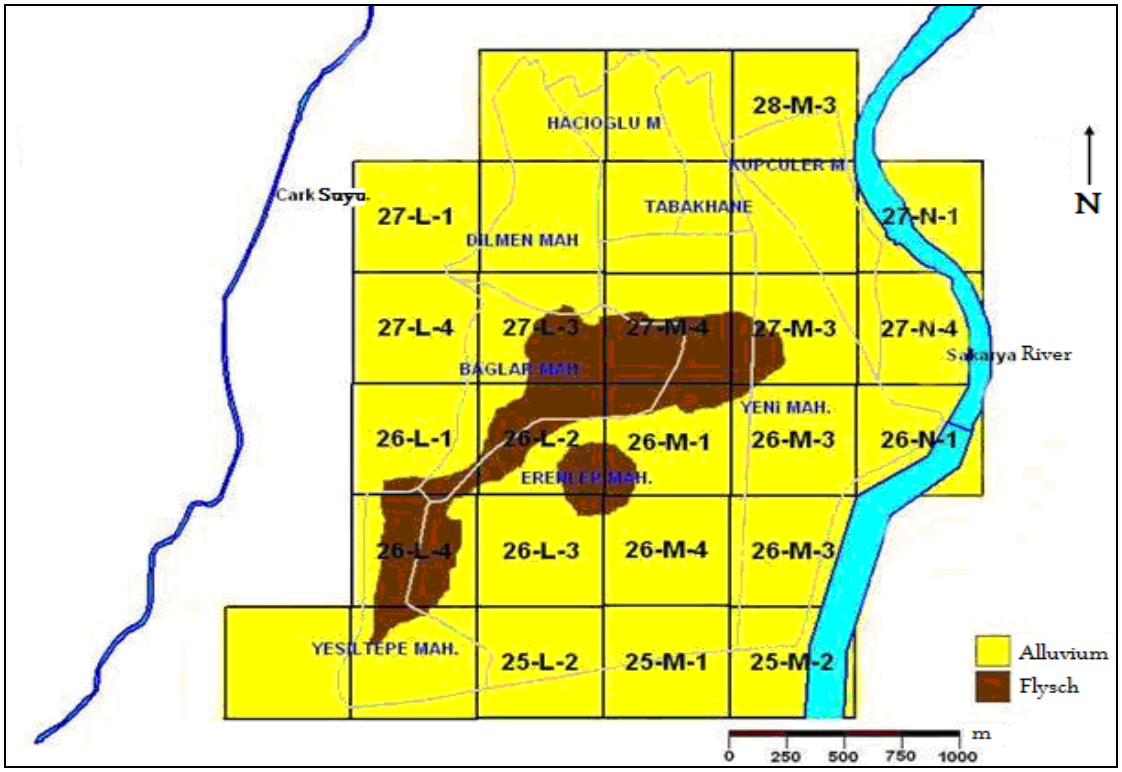

Fig. 3. Geology map of Erenler Region.

PREPARING THE LITHOLOGY MAPS OF ERENLER REGION

OBTAINED DATA AND ANALYSIS

Data, were obtained from 488 drilling application results. The results were taken from Erenler municipality after 1999 Marmara Earthquake, applied by Municipality and some companies. 1/40000 scale section of a map involves border of streets, public works areas, cadastral section maps. These are digitalized and make projection. Obtained data were loaded into the maps with their coordinates and looked at section of data distribution. To understand data reliability from 27 piece section of maps, applied SPSS data election set for each sections.

LITHOLOGY MAPS

We carried out a lithology classification before the lithology maps were prepared. For the lithology classification, considered only gravel, sand, silt and clay soils and their mixtures, ignored the placticity, firmness, softness of the soils. According to lithology classification, lithology number came down to 37 from 531. Eventually, 0,5-1 m, 1,5 m and 3 m depth lithology maps are formed by GIS.

Table 1. Determined lithologic units (according to litholgy classification).

Gravel	Sand	Silt	Clay	Botanical soil
Clayey gravel	Clayey sand	Clayey silt	Silty clay	Flysch
Clayey sandy gravel	Clayey silty sand	Clayey sandy silt	Silty sandy clay	Limestone
Silty gravel	Clayey silty gravelly sand	Clayey sandy gravelly silt	Silty sandy gravelly clay	Agglomerate
Silty sandy gravel	Clayey gravelly sand	Clayey gravelly silt	Silty gravelly clay	
Sandy gravel	Silty sand	Sandy silt	Sandy clay	
	Silty gravelly sand	Sandy gravelly silt	Sandy gravelly clay	
	Gravelly sand	Gravelly silt	Claystone	
	Sandstone	Siltstone		
	Marl			

Figure 4 is show that the lithology distribution at 0,5 -1 m depth in the study area. It is clear that the study area covered by botanical soils at related depths.

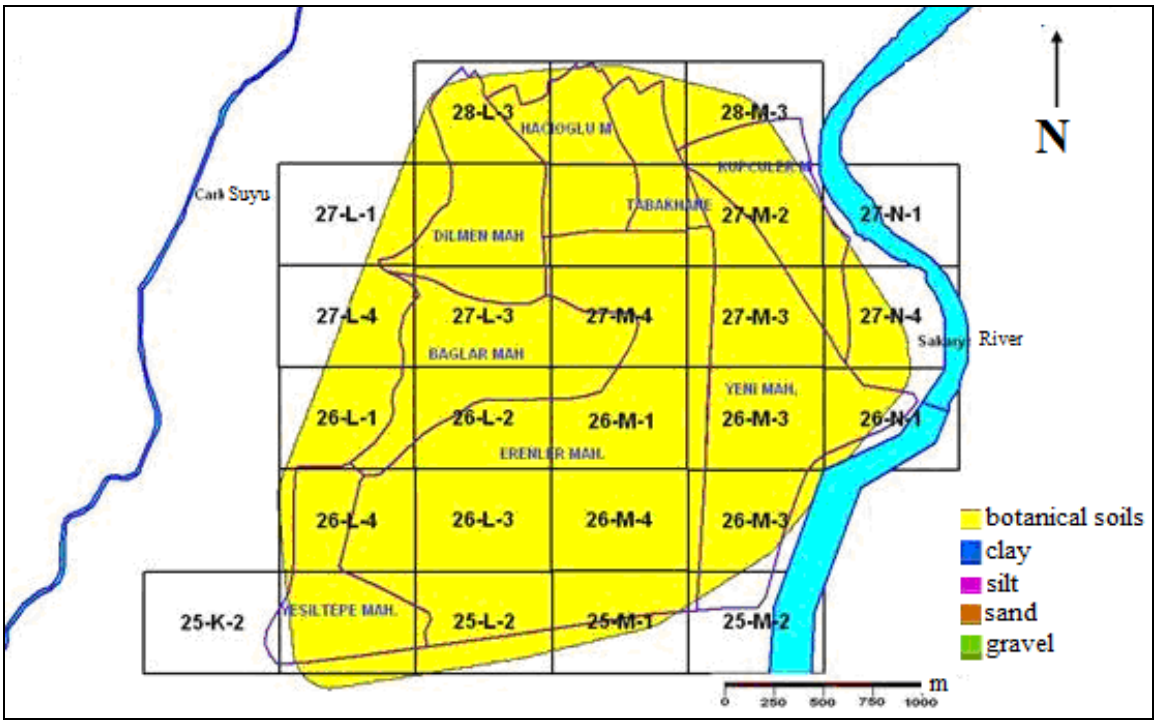

Fig. 4. 0,5-1 m depth lithology map of Erenler Region.

Figure 5 is show that the lithology distribution at 1,5 m depth in the study area. It is shown that 1,5 m depths are controlled by silty soils. Some local areas are controlled by clayey soils and southern part of the study area is still covered by botanical soils at related depth.

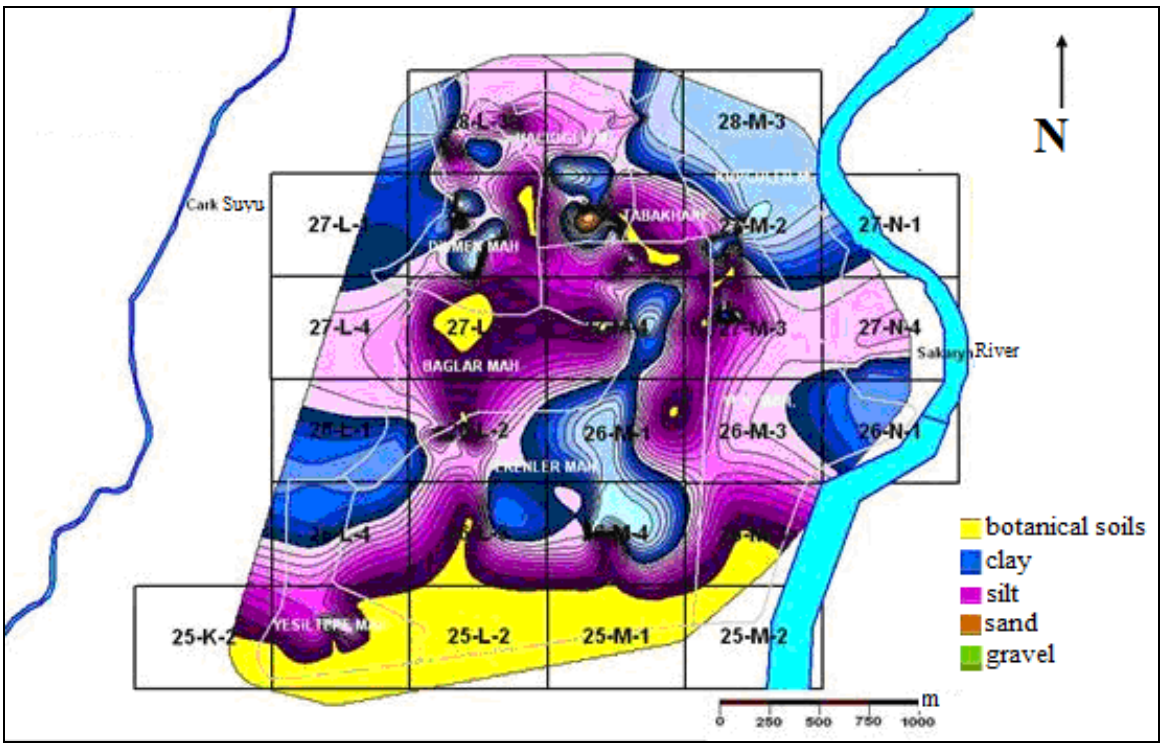

Fig. 5. 1,5 m depth lithology map of Erenler Region.

Figure 6 is show that the lithology distribution at 3 m depth in the study area. It is shown that at the western part of the study area, there are some sand-gravel intercalations just like a river sediments form. Besides the high topographic areas and some local areas are controlled by sandy soils, study area generally controlled by clayey soils at related depth.

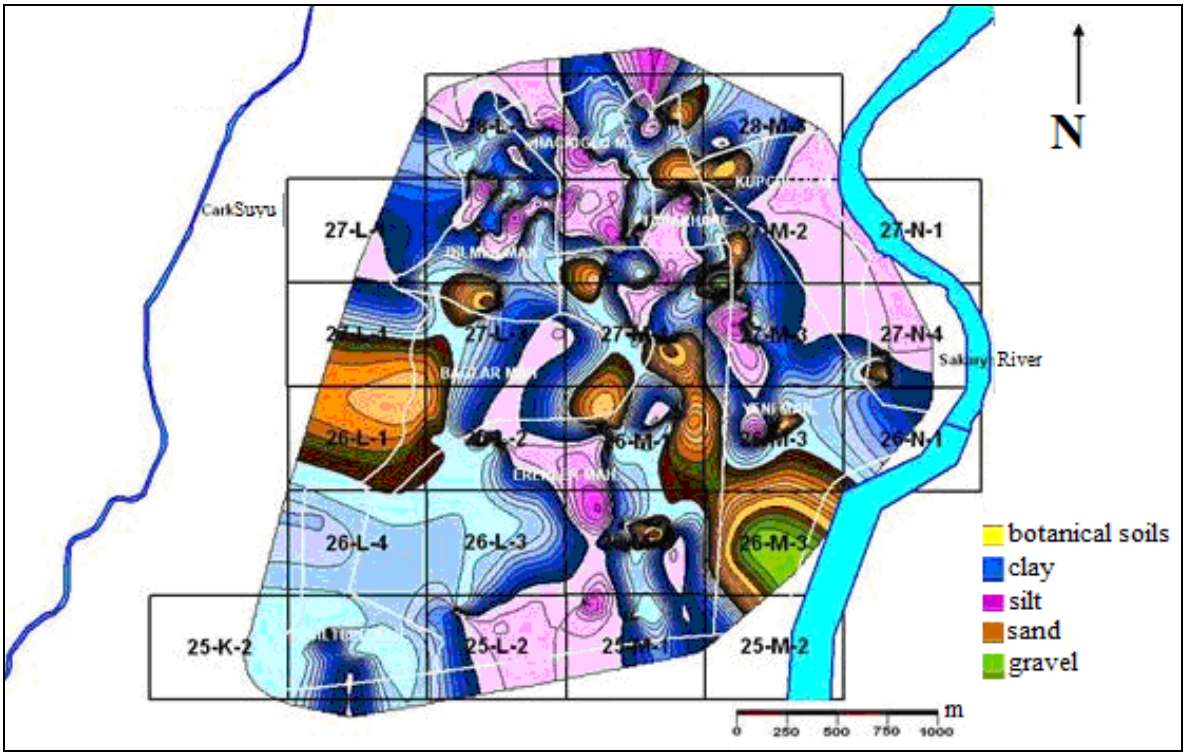

Fig. 6. 3 m depth lithology map of Erenler Region.

CONCLUSIONS

In this study, we aimed to point out the lithologic distribution in the study area on GIS based maps. For this purpose, 488 drilling data collected and transferred to numerical domain. The authors carried out a lithology classification for the study area before the lithology maps were prepared. Then, 0,5-1 m, 1,5 m and 3 m depth lithology maps are formed by GIS. 0,5-1 m depths (Figure 4) are covered by botanical soils in the study area. It is clear that on the other maps, 1,5 m depths (Figure 5) are controlled by silty soils, conversely 3 m depths (Figure 6) are controlled by clayey soils. It is obviously understood that the lithology variation in Erenler Region will bring about different ground behaviour in the study area during the earthquakes.

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CREATING THE MICROZONATION MAPS OF SAPANCA TOWN BY USING GEOGRAPHIC INFORMATION SYSTEMS (GIS)

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Key Words: Gis, microzonation, Sapanca.

ABSTRACT In this study we aimed to form the GIS based microzonation maps of Sapanca town. Fort his purpose 63 drilling data and 61 seismic refraction data collected and transferred to numerical domain. We carried out a data analysis for the database and eliminated the faulty data. The SPT values belongs to 1,5 m, 3 m, 4,5 m and 6 m, shown on GIS based maps and observed the variation on SPT values along with the depth rises. The seismic velocity maps are also formed by GIS and we investigated the harmony with the SPT maps. All the maps are show that the southern parts of the study area have better soil parameters. The Mahmudiye region have also firm soil parameter located on southwest part of the study area but towards the Kirkpinar, there are some rapid transition on the soil characteristics. We can see that the SPT values and seismic velocities are point out some variations in short ranges in the study area.

INTRODUCTION

Turkey is located on an active tectonic structure and has a suitable topography for natural disasters. It is necessary to determine the better soil sides in settlement areas and constitute the microzonation maps. Microzonation maps can created by collected, saved and updated the soil parameters data of settlement areas.

Sapanca town is located on southeast part of Sakarya province that has a very active tectonism features according to the North Anatolian Fault System. The principal reason of disasters stem from earthquakes is high storey buildings on this alluvial soil that built without any soil analysis. In settlement areas which have earthquake risk, soil conditions can clearly determine with microzonation maps that created by using GIS and reliable interpretations can be presented by this way.

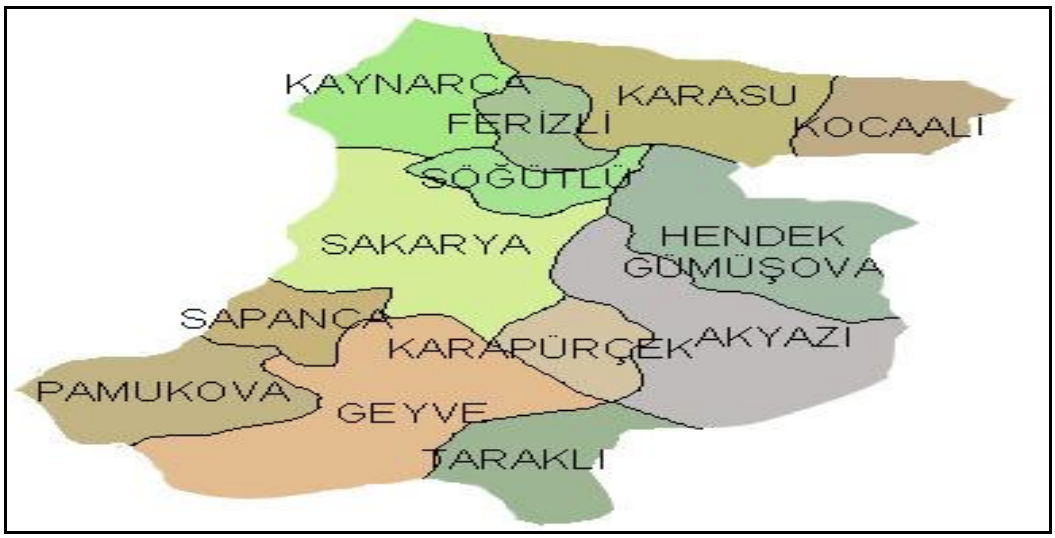

Fig. 1. Location of the Sapanca town in Sakarya province.

In this study, investigated the soil stiffness characteristics of Sapanca town by 63 drilling and 61 seismic refraction data. The SPT values and seismic velocities are shown on gis based microzonation maps.

GEOLOGICAL SETTING

The survey area is located in a tectonic pull-apart basin, which extends between Sakarya River and Gulf of Izmit. Elevation of the survey area ranges between 30 and 45 m. Inclination of natural slopes is very low and ranges between 0.1% and 0.3%, and 2% and 4% at the western and southern parts of Lake Sapanca, respectively. Depending on the activity of the North Anatolian Fault Zone (NAFZ), recent morphological conditions of Lake Sapanca have occurred (Yilmaz et al., 1990). Sapanca and its vicinity are comprised of Paleozoic aged metamorphic rocks called Iznik metamorphites, Late Pliocene Karapurcek formation and Quaternary deposits (Fig. 2). The Iznik metamorphites, which are mainly represented by marbles, schists, phyllites and meta-detritics, are observed along the natural slopes at the south of Sapanca town. The Karapurcek formation is represented by the fluvial and lacustrine sedimentary sequences and crops out at limited areas forming fault terraces at the south of the town. It is composed of siltstone, mudstone, canal conglomerates and local limnic limestones and claystones with high amounts of organic matter (ODTU-MTA, 1999). Many alluvial fans and recent alluvium have deposited along the southern shore of the lake. They are composed of gravel, sand, silt and clay sized material, and their thickness reaches up to 100 m. These deposits are carried by several creeks (Fig. 2) flowing from the mountains at 1250 m elevation. Their deltas cover areas from 3 to 6 km^2. The study site is situated at a tectonically active region of Turkey that is structurally controlled by the segments of the North Anatolian Fault Zone (NAFZ). Lake Sapanca is one of the products of pull-apart mechanism associated with the strike-slip motion of the NAFZ (Barka, 1997). The main northern segment of the NAFZ, called Sapanca–Golcuk segment, trends E–W and extends between Lake Sapanca and Izmit Gulf with an echelon geometry (Fig. 2), and then it changes its direction to south westward at the south of Golcuk town.

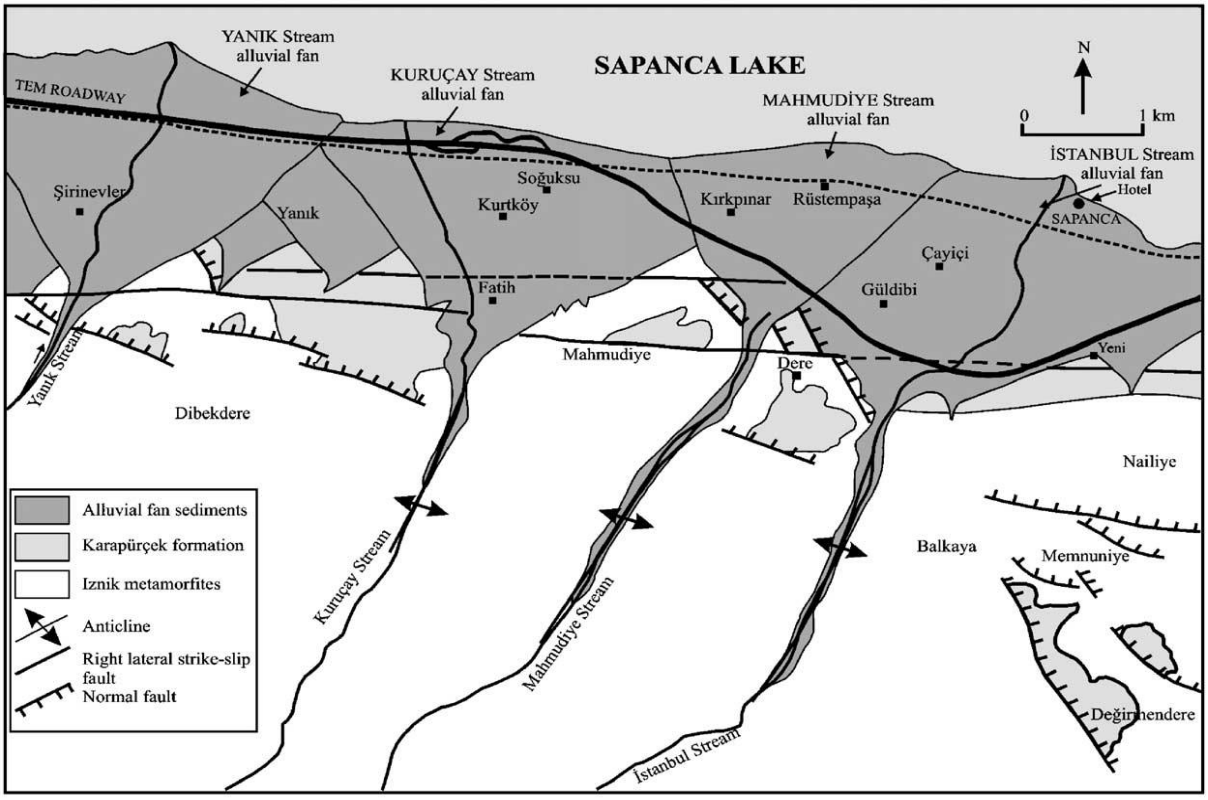

Fig. 2.Geological map of Sapanca and its close vicinity (after ODTU -MTA, 1999).

PREPARING THE MICROZONATION MAPS OF SAPANCA TOWN

OBTAINED DATA AND ANALYSIS

Data, were obtained from 63 drilling and 61 seismic refraction application results. The results were taken from Sapanca municipality after 1999 Marmara Earthquake, applied by Municipality and some companies. 1/17000 scale section of a map involves border of streets, public works areas, cadastral section maps. These are digitalized and make projection. Obtained data were loaded into the maps with their coordinates and looked at section of data distribution. To understand data reliability from 32 piece section of maps, applied SPSS data election set for each sections.

SPT (STANDART PENETRATION TEST) MAPS

The test uses a thick-walled sample tube, with an outside diameter of 50 mm and an inside diameter of 35 mm, and a length of around 650 mm. This is driven into the ground at the bottom of a borehole by blows from a slide hammer with a weight of 63.5 kg (140 lb) falling through a distance of 760 mm (30 in). The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm (6 in) up to a depth of 450 mm (18 in) is recorded. The sum of the number of blows required for the second and third 6 in. of penetration is termed the "standard penetration resistance" or the "N-value". In cases where 50 blows are insufficient to advance it through a 150 mm (6 in) interval the penetration after 50 blows is recorded. The blow count provides an indication of the density of the ground, and it is used in many empirical geotechnical engineering formulae.

Figure 3 and Figure 4 are show the SPT values at 1.5 m and 3 m depth respectively. We can understood from SPT 1 and SPT 2 maps that the study area generally have low SPT values at related depths. Especially center and west part of the study areas are have quite low SPT values.

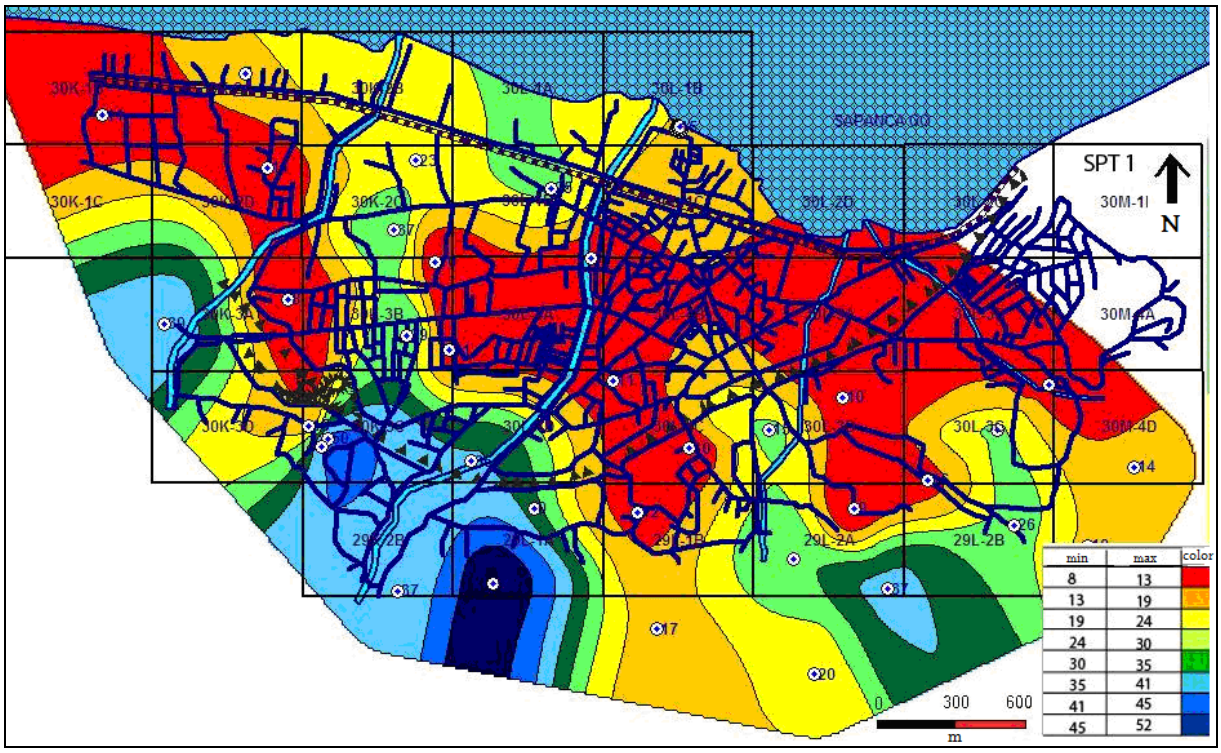

Fig. 3. SPT 1 map (1.5 m depth) of study area.

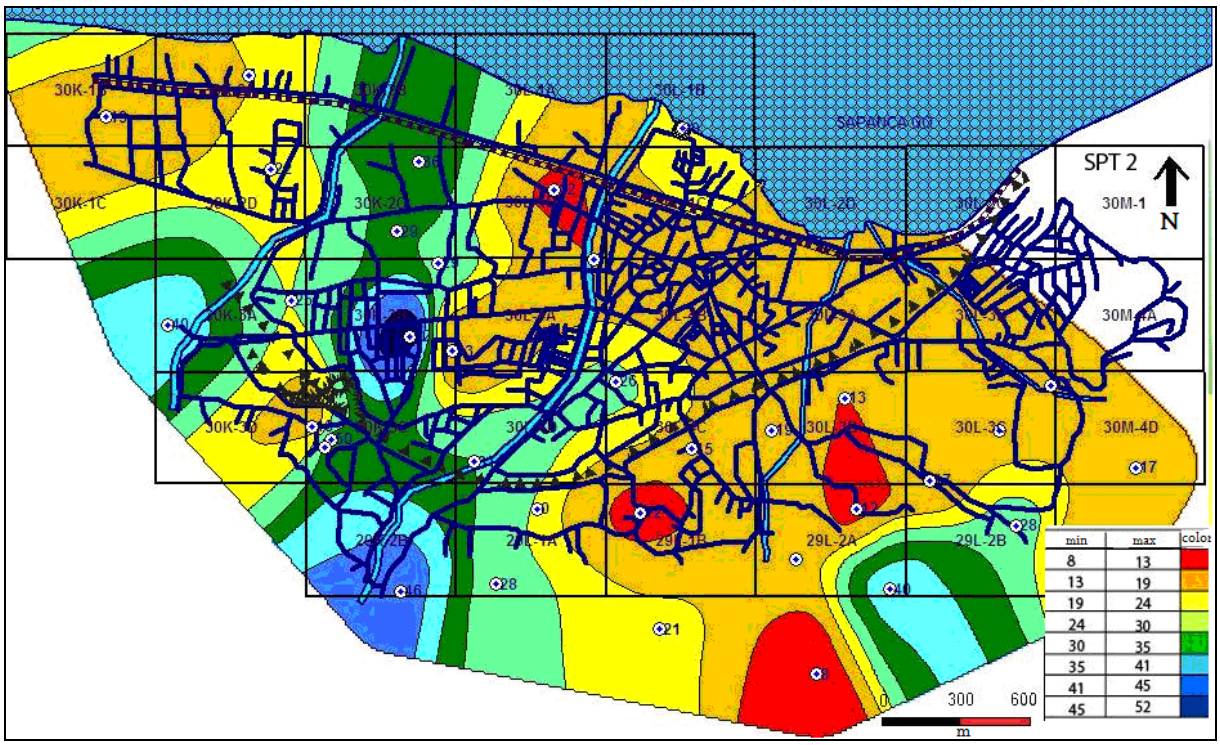

Fig. 4. SPT 2 map (3 m depth) of study area.

Figure 5 and Figure 6 are show the SPT values at 4.5 m and 6 m depth respectively. SPT 3 and SPT 4 maps are show that, there are some local increases on the SPT values in the study area. It is clear that, south and southeast parts of the study area have better SPT values. The SPT maps show that some local areas on the lakeside have high SPT values. The authors thought that it must be result from the backfill application at these sides.

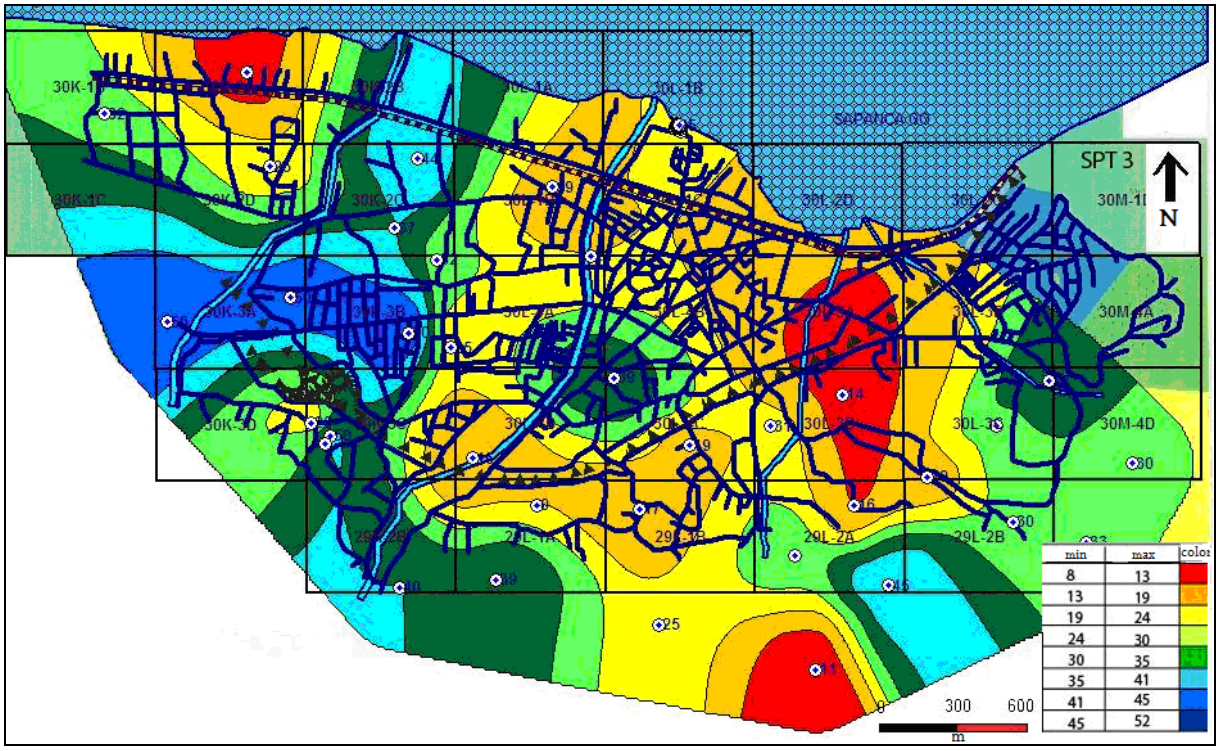

Fig. 5. SPT 3 map (4.5 m depth) of study area.

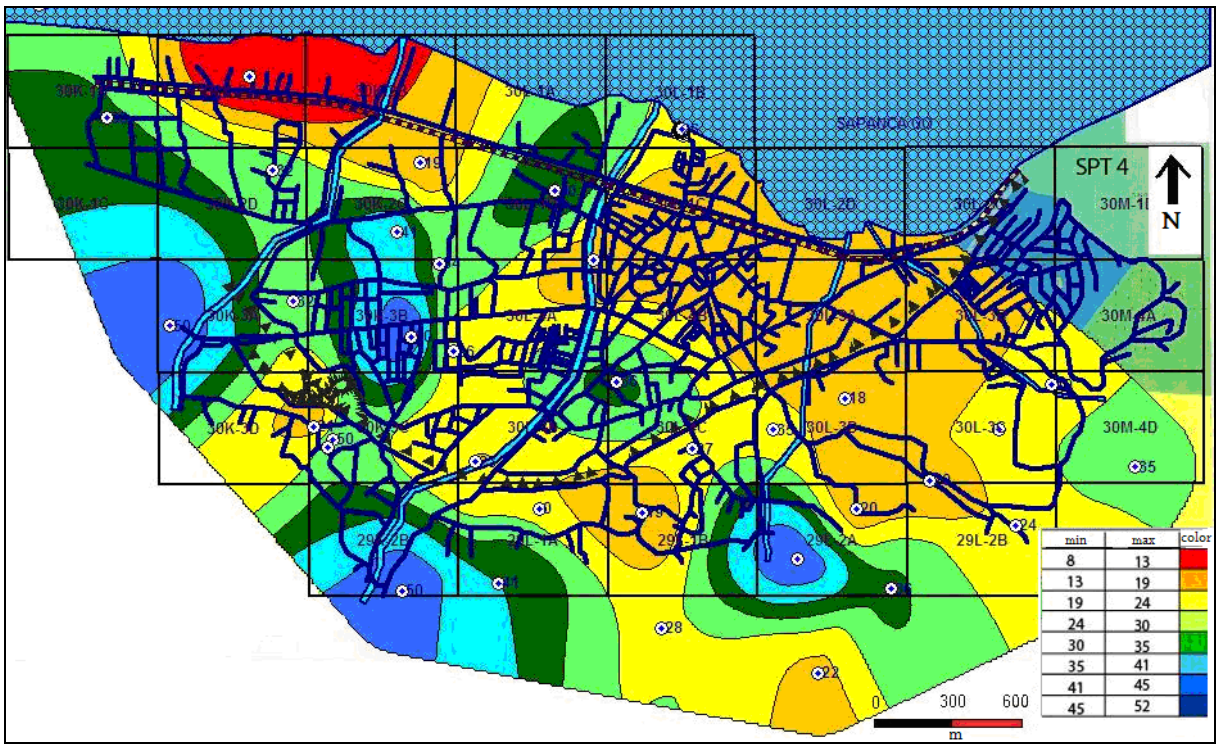

Fig. 6. SPT 4 map (6 m depth) of study area.

SEISMIC VELOCITY MAPS

Seismic velocity maps are also formed by GIS and we investigated the harmony with the SPT maps. Figure 7 and Figure 8 are show the first and second layer shear wave velocity maps respectively.

Figure 7 show the first layer shear wave velocities and it is clear that, the study area generally have low shear wave velocities at related depths. The shear wave velocities point out some increases along with the depth rises on second layer map (Fig. 8) such as SPT 3 and SPT maps (Fig. 5 and Fig. 6).

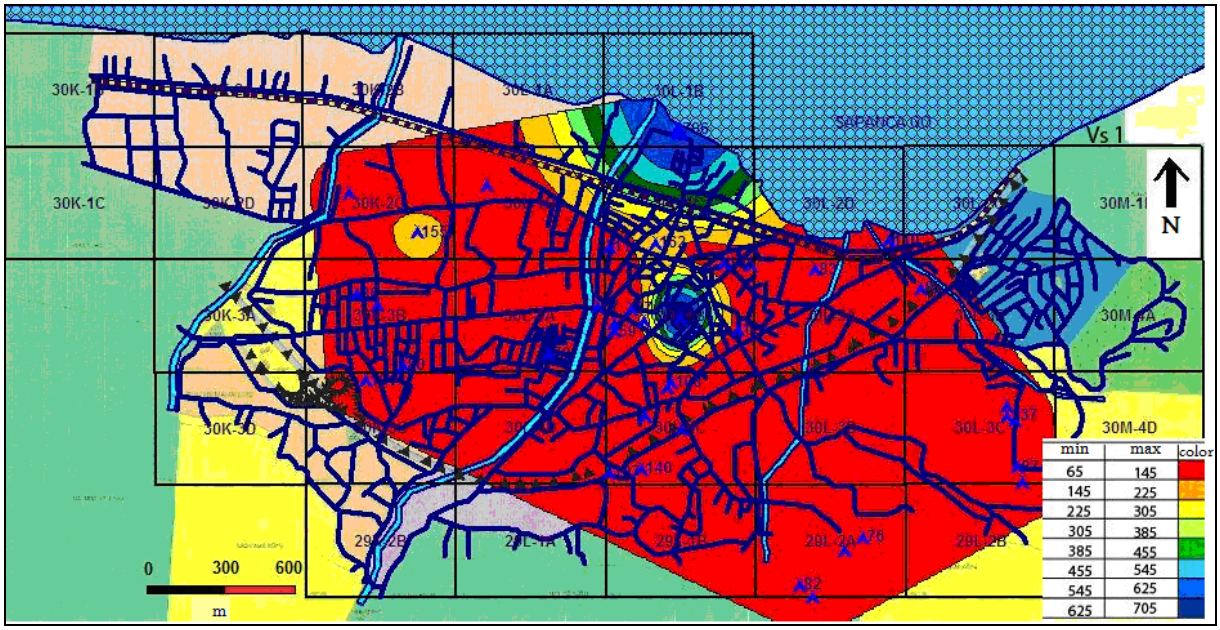

Fig. 7. First layer shear wave velocity map of the study area.

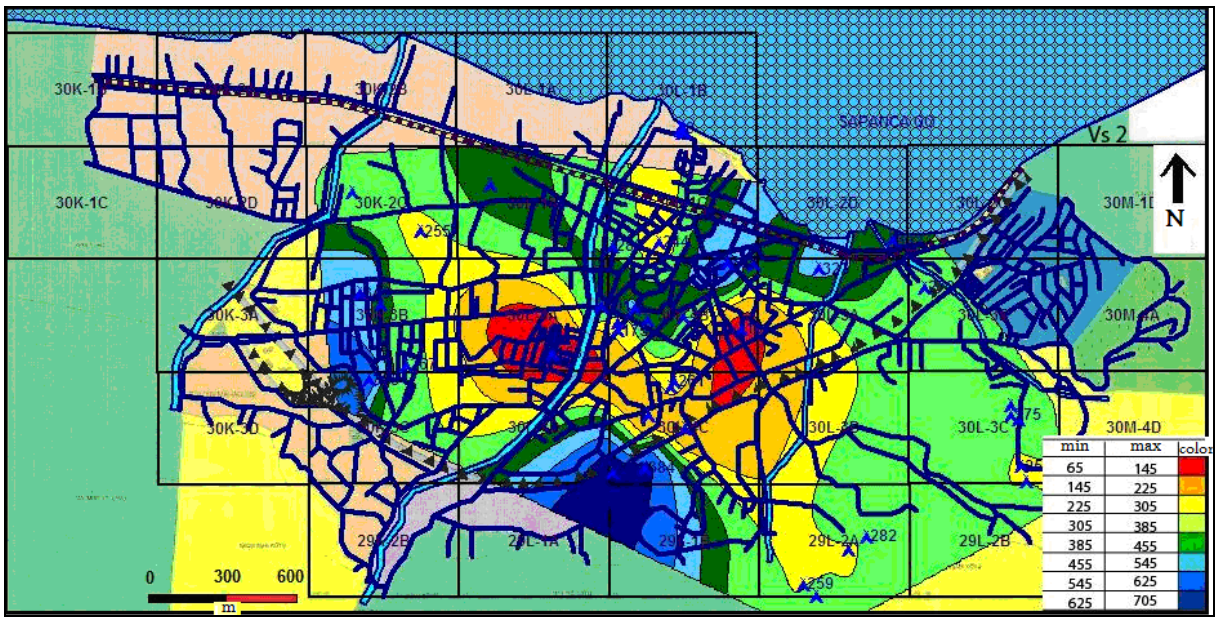

Fig. 8. Second layer shear wave velocity map of the study area.

CONCLUSIONS

In this study we aimed to form the GIS based microzonation maps of Sapanca town. The SPT values belongs to 1,5 m, 3 m, 4,5 m and 6 m, shown on GIS based maps and observed the variation on SPT values along with the depth rises. The seismic velocity maps are also formed by GIS and we investigated the harmony with the SPT maps. All the maps are show that the southern parts of the study area have better soil parameters. The Mahmudiye region have also firm soil parameter located on southwest part of the study area but towards the Kirkpinar, there are some rapid transition on the soil characteristics.

Sapanca town is located on southwest part of Sakarya province. The study area has high level seismic hazard according to the North Anatolian Fault System which is located on southern part of Sakarya province. Considering the thickness of aluvium and high ground water level, it will be useful to make precaution such as drainage, excavation and ground improvement before construct a building in the study area.

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CREATING THE SPT (STANDART PENETRATION TEST) MAPS OF DERİNCE TOWN (KOCAELI) BY USING GEOGRAPHIC INFORMATION SYSTEMS (GIS)

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fkurnaz@sakarya.edu.tr, akiyak@sakarya.edu.tr, karavul@sakarya.edu.tr

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Key Words: Spt, gis, drilling, Derince

ABSTRACT In this study, we aimed to point out the SPT values of Derince town on GIS based maps. For this purpose, 41 drilling data collected and transferred to numerical domain. We carried out a data analysis for the database and eliminated the faulty data. The SPT values belongs to 1,5 m, 3 m and 4,5 m shown on GIS based maps and observed the variation on SPT values along with the depth rises. SPT value is the most important parameter to determine the stiffness of soils. The maps are show that the SPT values are increasing from south to northern part of study area. The low SPT values at southern part of study area result from the sedimets of Marmara Sea. Along with the depth rises the SPT values are increasing all over the study area.

INTRODUCTION

Turkey is located on an active tectonic structure and has a suitable topography for natural disasters. It is necessary to determine the better soil sides in settlement areas and constitute the microzonation maps. Microzonation maps can created by collected, saved and updated the soil parameters data of settlement areas.

Derince town is located on eastern part of Kocaeli province that has a very active tectonism features according to the North Anatolian Fault System. In settlement areas which have earthquake risk, soil conditions can clearly determine with microzonation maps that created by using GIS and reliable interpretations can be presented by this way.

Fig. 1. Location of the Derince town in Kocaeli province.

In this study, investigated the soil stiffness characteristics of Derince town by 41 drilling data. The SPT values are shown on gis based microzonation maps.

GEOLOGICAL SETTING

The Bay of Izmit is located in an east–west trending active graben system which is dynamically affected by the interaction of the North Anatolian Fault Zone and the Marmara Graben System. It is bounded by two horsts: (i) Kocaeli Peninsula to the north and Armutlu Peninsula to the south, exhibiting completely different geomorphological features and by well defined fault scarps. The graben varying between 6 and 10 km in width is a comparatively large, long and narrow basin filled with young sediments of marine and continental facies [12]. As shown in Fig. 2, the southern shores of Izmit Bay are covered by Holocene deposits except a relatively small area, which was classified geologically as Bakacak formation of Companian–Maastrihtien age consisting of marn, mudstone, conglomerate and sandstone. From sedimentalogical point of view, the southern shores of Izmit Bay is covered by sandy deposits which gets finer (siltier and more clayey) as one moves towards north in to the depths of Izmit Bay.

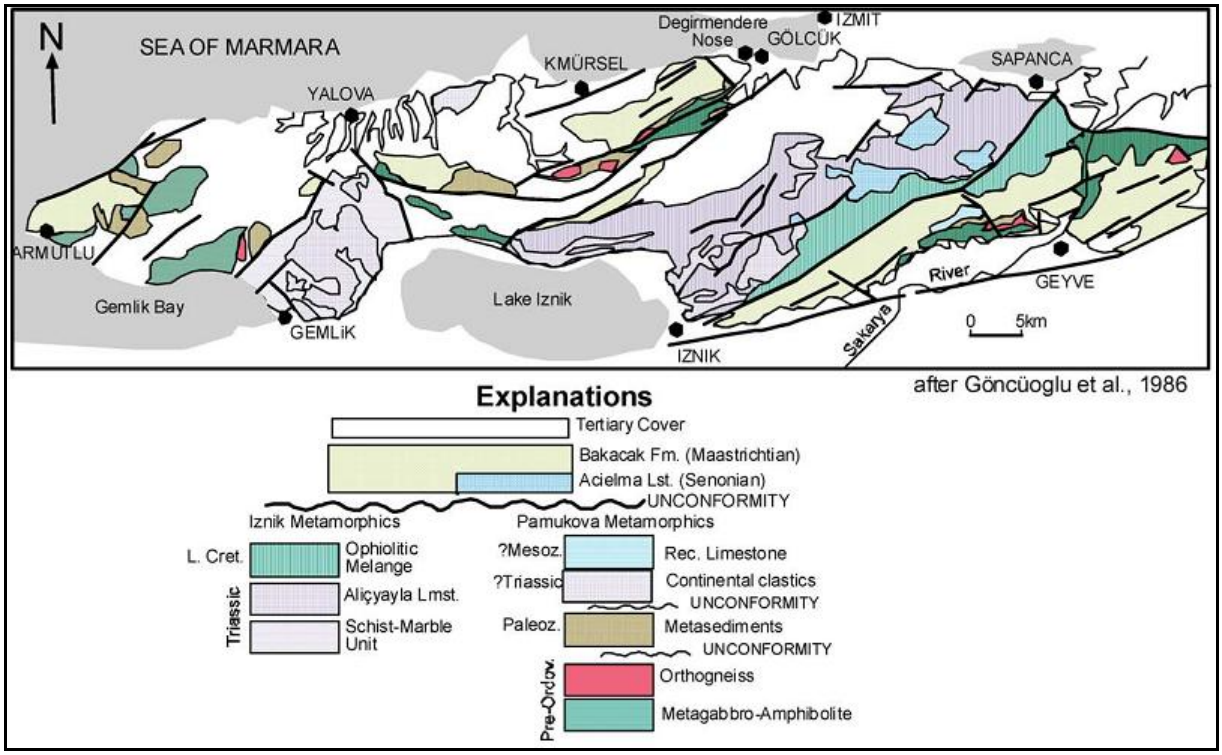

Fig. 2. Simplified geological map of Armutlu Peninsula (after Goncuoglu et al.) .

Figure 4 is show the groundwater map of study area. Groundwater level is generally at high grades in the southern part and generally at low grades in the northern part of study area.

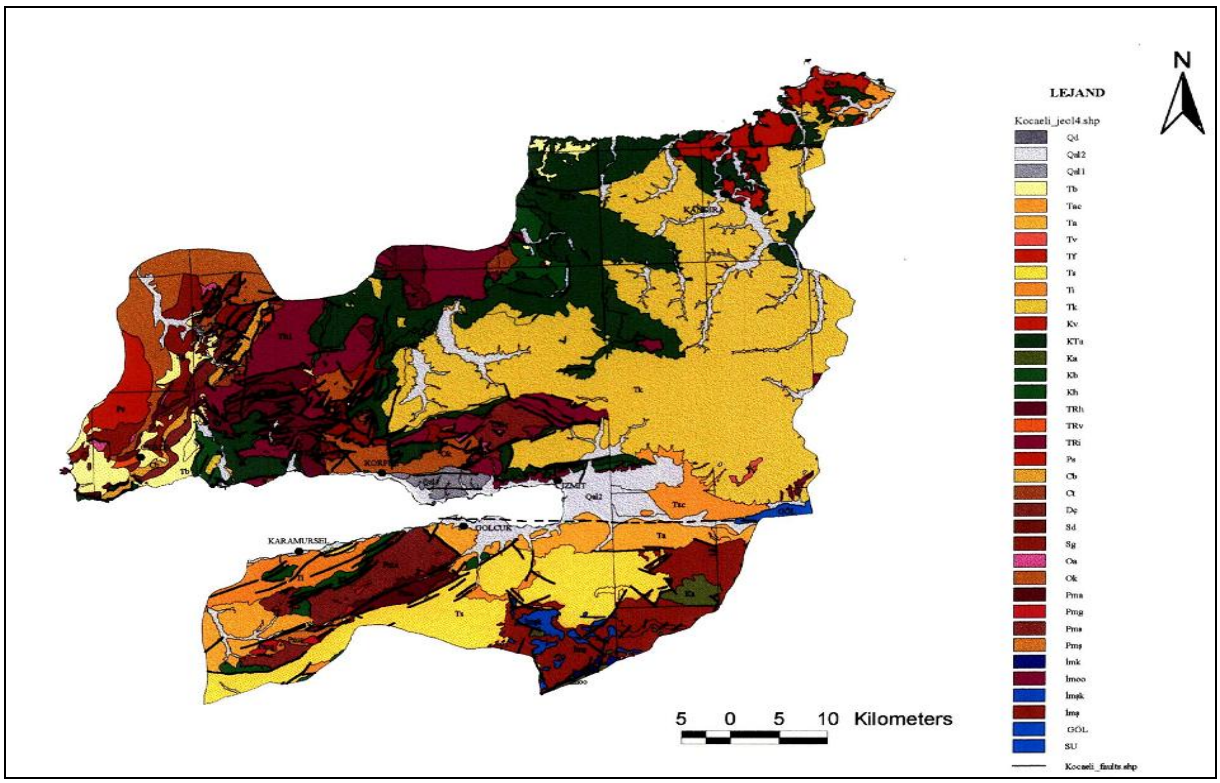

Fig. 3. Geology map of Kocaeli province (Ozer et al., 2005).

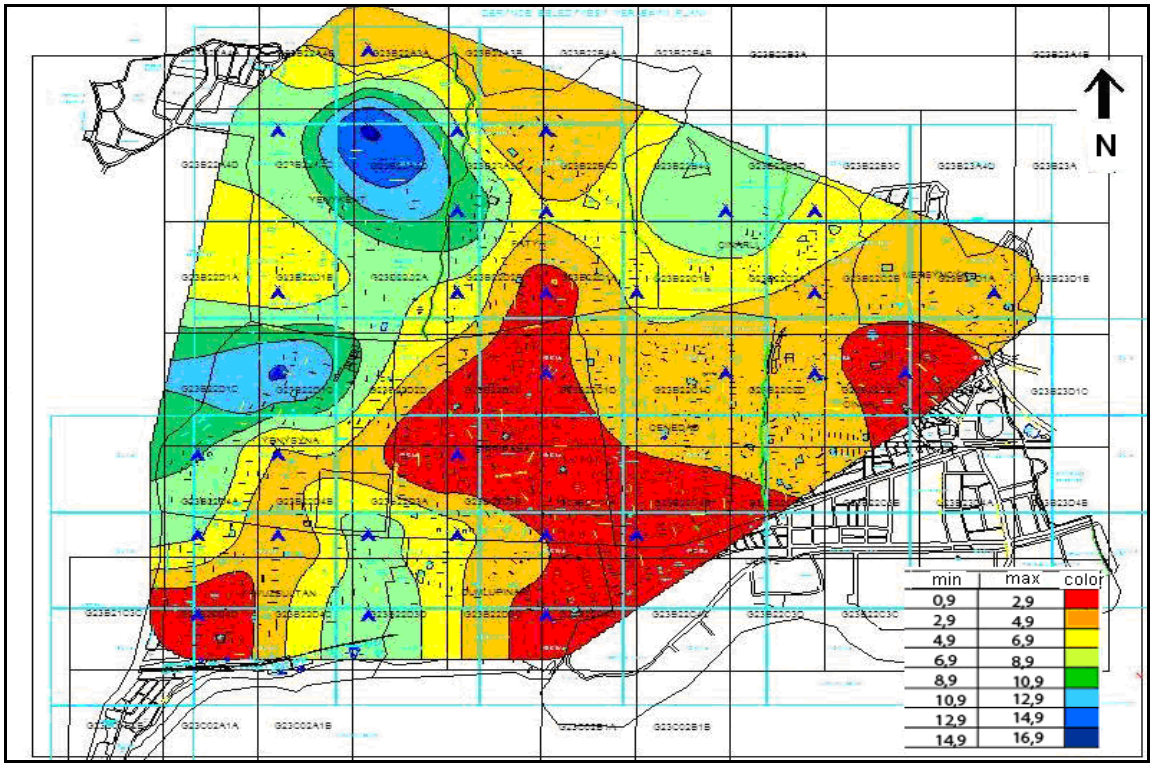

Fig. 4. Groundwater map of study area.

PREPARING THE SPT MAPS OF DERİNCE TOWN

OBTAINED DATA AND ANALYSIS

Data, were obtained from 41 drilling application results. The results were taken from Derince municipality, applied by Municipality and some companies. 1/18000 scale section of a map involves border of streets, public works areas, cadastral section maps. These are digitalized and make projection. Obtained data were loaded into the maps with their coordinates and looked at section of data distribution. To understand data reliability, applied SPSS data election set for each sections.

SPT (STANDART PENETRATION TEST) MAPS

The test uses a thick-walled sample tube, with an outside diameter of 50 mm and an inside diameter of 35 mm, and a length of around 650 mm. This is driven into the ground at the bottom of a borehole by blows from a slide hammer with a weight of 63.5 kg (140 lb) falling through a distance of 760 mm (30 in). The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm (6 in) up to a depth of 450 mm (18 in) is recorded. The sum of the number of blows required for the second and third 6 in. of penetration is termed the "standard penetration resistance" or the "N-value". In cases where 50 blows are insufficient to advance it through a 150 mm (6 in) interval the penetration after 50 blows is recorded. The blow count provides an indication of the density of the ground, and it is used in many empirical geotechnical engineering formulae.

Figure 5 shows the SPT values at 1.5 m depth. The southern parts of the study area have low SPT values at related depth.

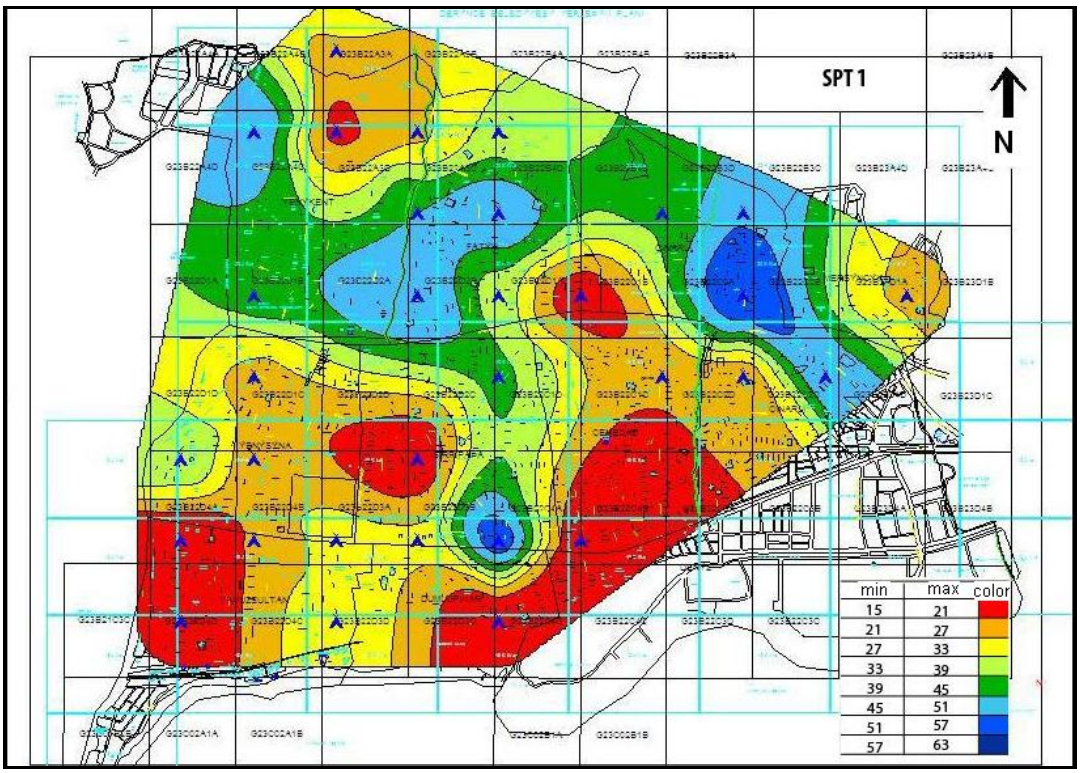

Fig. 5. SPT 1 map (1.5 m depth) of study area.

Figure 6 shows the SPT values at 3 m depth. It is shown that the SPT values generally point out rising at related depth. But, there are some local areas at the southern parts of the study area still have low SPT values.

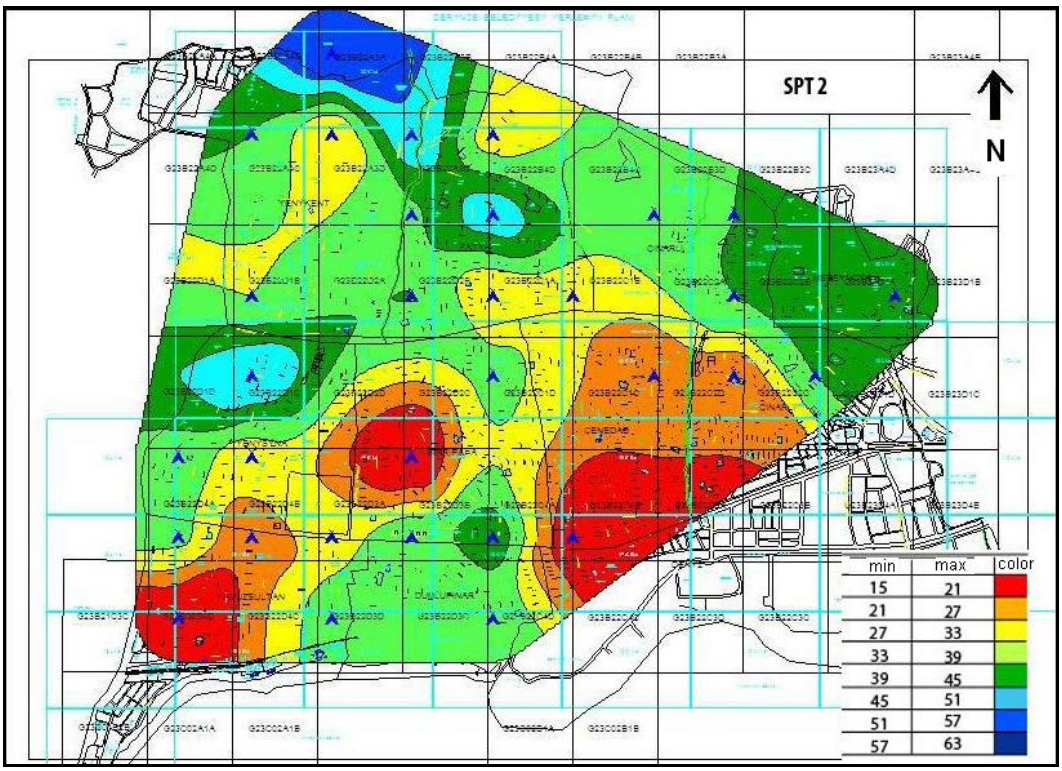

Fig. 6. SPT 2 map (3 m depth) of study area.

Figure 7 shows the SPT values at 4,5 m depth. It is clear that the study area generally have high SPT values at related depth. There are also some local areas at the southern part of the study area still have low SPT values than the other sides.

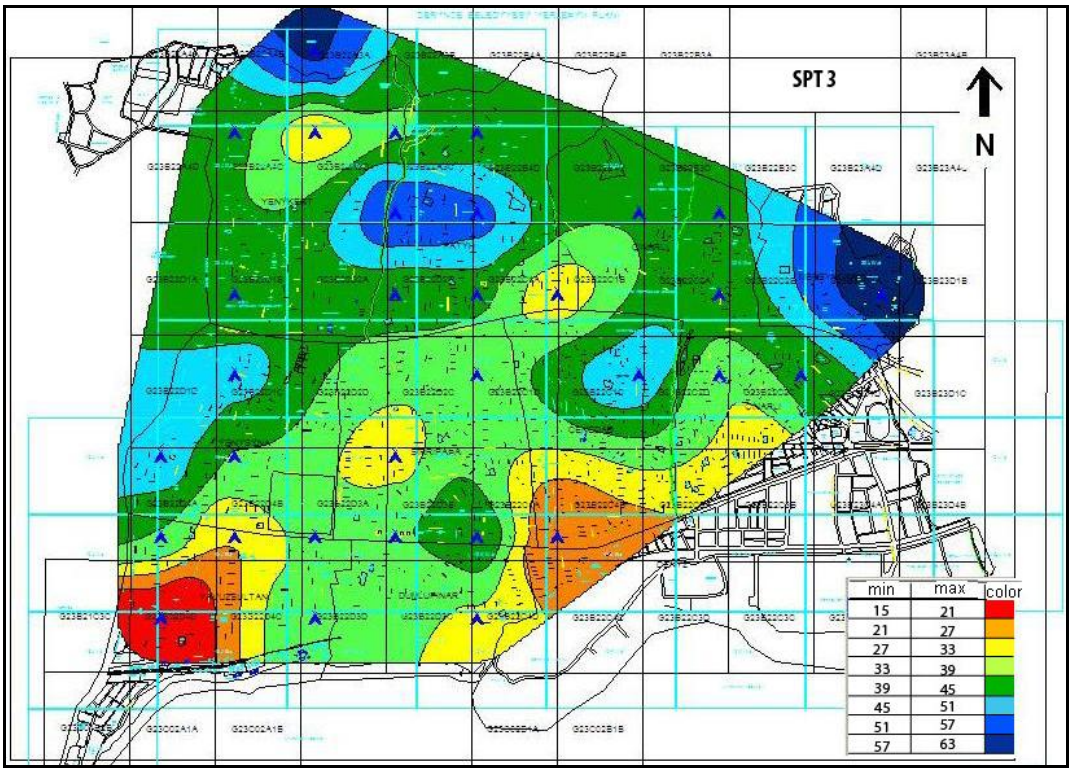

Fig. 7. SPT 3 map (4.5 m depth) of study area.

CONCLUSIONS

In this study, we aimed to point out the SPT values of Derince town on GIS based maps. The SPT values belongs to 1,5 m, 3 m and 4,5 m depths shown on GIS based maps and observed the variation on SPT values along with the depth rises.

We can clearly understood from all SPT maps that the northern parts of the Derince town, have better soil conditions. Along with the depth rises northern parts of the study area keep their strong soil characteristics but southern parts of study area are still have low SPT values.

Derince town is located on eastern part of Kocaeli province. The study area has high level seismic hazard according to the North Anatolian Fault System. The Marmara Sea is located on the southern part of Derince town. The authors thought that the low SPT values at the southern part of the study area stem from the sediments which were carried by Marmara Sea.

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CREATING THE SPT (STANDART PENETRATION TEST) MAPS OF ERENLER REGION IN SAKARYA BY USING GEOGRAPHIC INFORMATION SYSTEMS (GIS)

Kurnaz[1], Kiyak[1], Karavul[1]

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Key Words: spt, gis, drilling, erenler

ABSTRACT Erenler Region is located on southern part of Sakarya that has a very active tectonism features according to the North Anatolian Fault System. In this study, we aimed to point out the SPT values of Erenler region on GIS based maps. For this purpose, 488 drilling data collected and transferred to numerical domain. We carried out a data analysis for the database and eliminated the faulty data. The SPT values belongs to 1,5 m, 3 m, 4,5 m and 6 m, shown on GIS based maps and observed the variation on SPT values along with the depth rises. SPT value is the most important parameter to determine the stiffness of soils. SPT 1 and SPT 2 maps are show that, study area generally have low SPT values at related depths. Yeşiltepe, Baglar and Yeni districts have low groundwater depths and also they have high SPT values. SPT 3 and SPT 4 maps are show that, there are some local increases on the SPT values at the study area. Along with the depth rises, Yeşiltepe, Baglar and Yeni districts keep their strong soil characteristics but Tabakhane and Hacıoglu districts and southern parts of study area are still have low SPT values.

INTRODUCTION

Turkey is located on an active tectonic structure and has a suitable topography for natural disasters. It is necessary to determine the better soil sides in settlement areas and constitute the microzonation maps. Microzonation maps can created by collected, saved and updated the soil parameters data of settlement areas.

Erenler Region is located on southern part of Sakarya that has a very active tectonism features according to the North Anatolian Fault System. The principal reason of disasters stem from earthquakes is high storey buildings on this alluvial soil that built without any soil analysis. In settlement areas which have earthquake risk, soil conditions can clearly determine with microzonation maps that created by using GIS and reliable interpretations can be presented by this way.

In this study, investigated the soil stiffness characteristics of Erenler Region by 488 drilling data. The SPT values are shown on gis based microzonation maps.

GEOLOGICAL SETTING

Sakarya province formed by Quaternary aged alluvium that spread out large area in this region. This alluvium unit constituted Adapazari, Akyazi, Hendek plains and consist of sand-silt-clay-gravel which are not firm [Akartuna, 1968 and Önalp, Bol, 1998]. Adapazari is the central district of Sakarya province and situated in the west part of the Sakarya plain surrounded by mountain chains on four sides. The altitude of Adapazari is about 31 m above sea level. Quaternary sediments carried primarily by the Sakarya River and also by Çark Suyu (River) and Mudurnu River from NAFZ and the rocks at the South of the fault were deposited in the Sakarya Plain. The accumulation of thick alluvial deposits reaches a thickness up to 300 m at the central part of the plain (Beyen, Erdik,

2004). Komazawa et al., 2001, claimed that the thickness of this alluvium is approximately 1000 m.

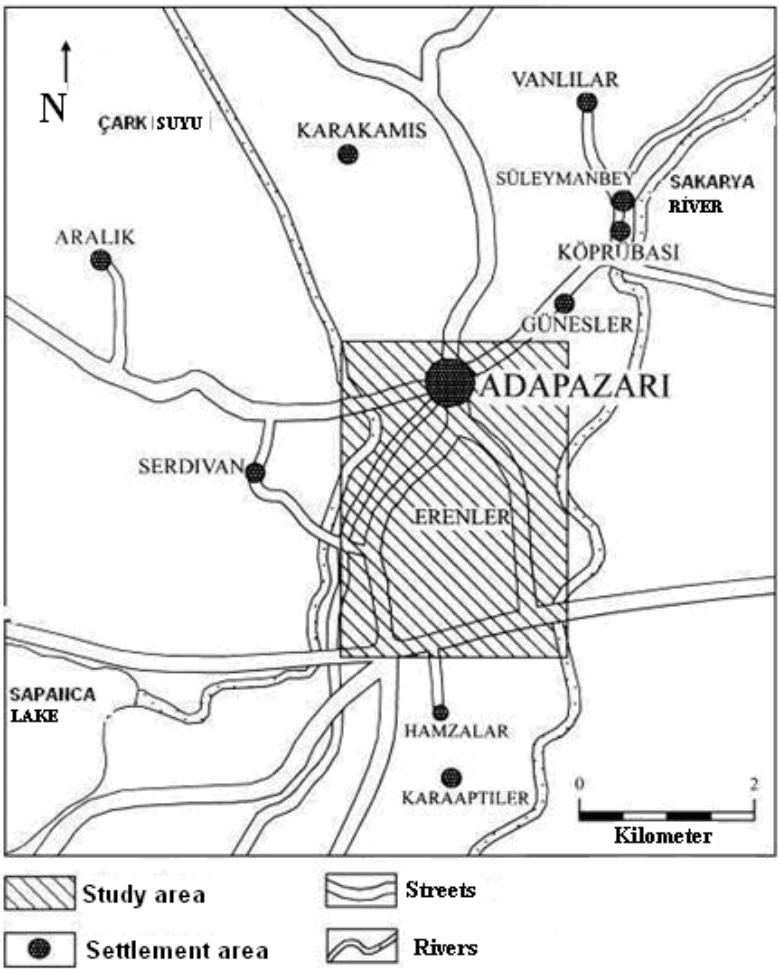

Fig. 1. Location of the Erenler region in Sakarya province.

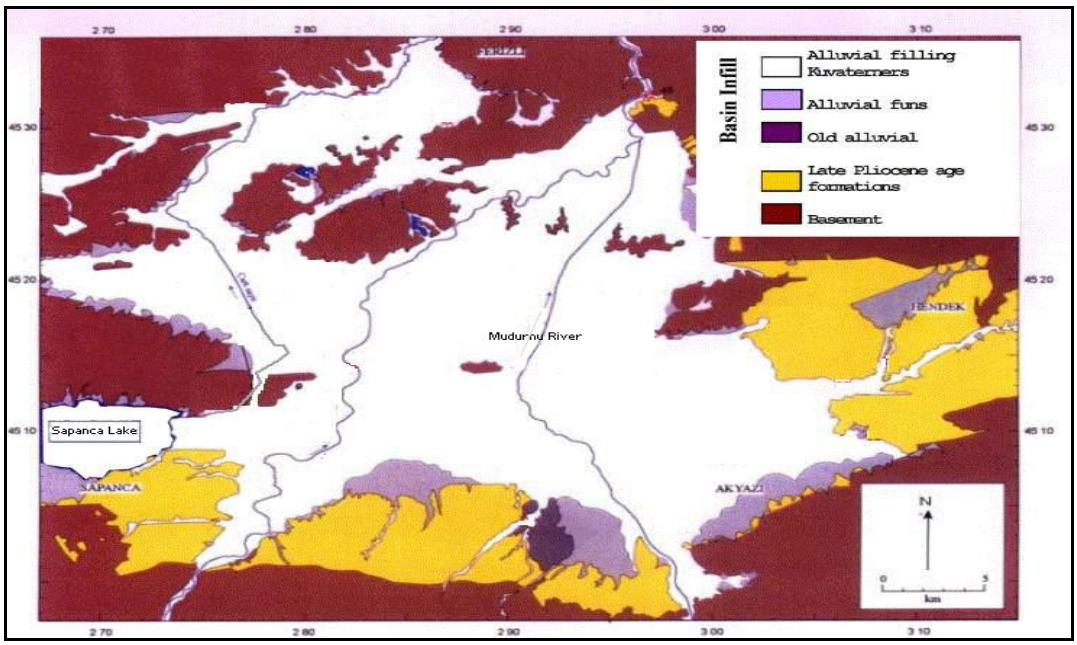

Fig. 2. Simplified Adapazari plain surface geology map (Beyen, Erdik, 2004).

The surface geology of the Sakarya plain as shown in the simplified map in Fig. 2 can be divided into mainly five different formations (Beyen, Erdik, 2004). They are, namely, alluvial filling kuvaterners, alluvial funs, old alluvial, late Pliocene age formations and basement. In the southern parts of the city, there seems to be firm ground on one side and hilly surface formations up to 160 m as an outcrop of the basement rock on the other side of the passage at the entrance to the city from intercity highways. Towards the north in the cross-section, recent alluvial deposit, consisting of loose silty, soil cover a surface 7 km in length. The west part of the city is located on hill-like formations and outliers of mountains next to Lake Sapanca, the eastern and central parts as well as many other districts, are located in the sedimentary basin which extends towards southeast Akyazi 25 km in length.

In Erenler Region , there are two main geological formation. These are Eocene aged flysch and Quaternary aged alluvium. The high topographic areas covered by flysch and all plain areas covered bye alluvium. The flysch consist of claystone, marl, siltstone and sandstone. The alluvium consist of clay, silt and sand.

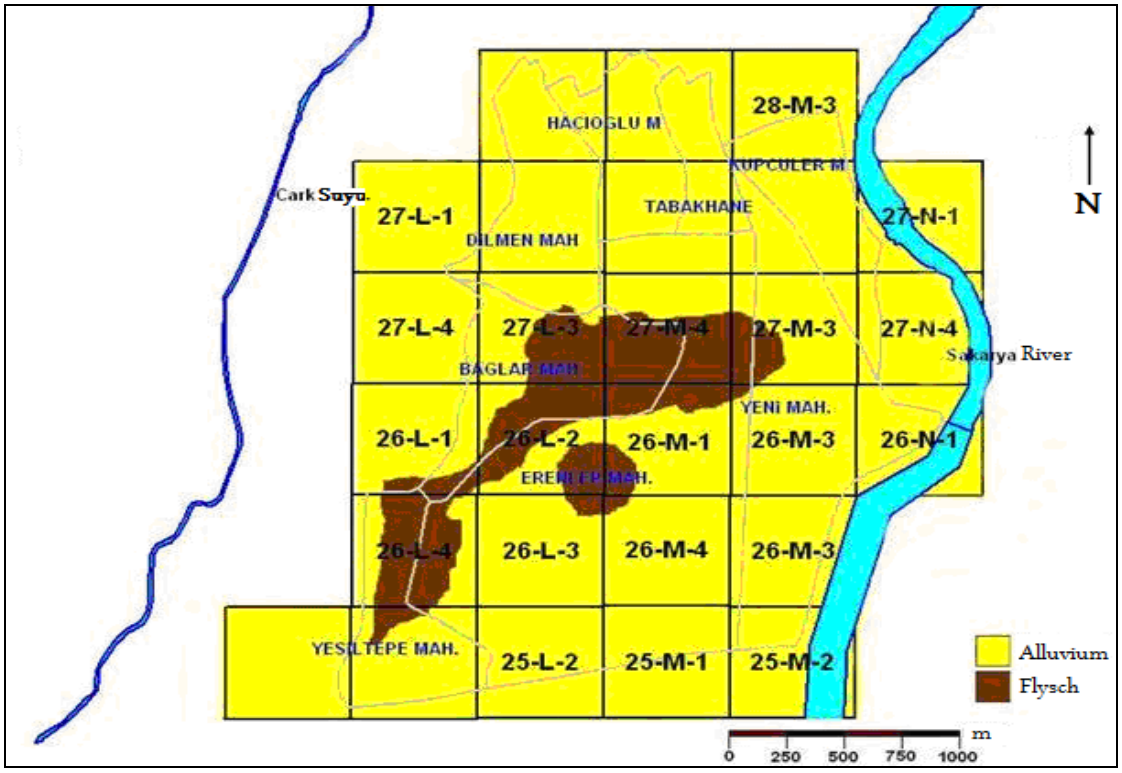

Fig. 3. Geology map of Erenler Region.

Figure 4 is show the groundwater map of study area. Groundwater level is generally at high grades in the study area. Yeşiltepe, Baglar and Yeni districts relatively have low groundwater levels.

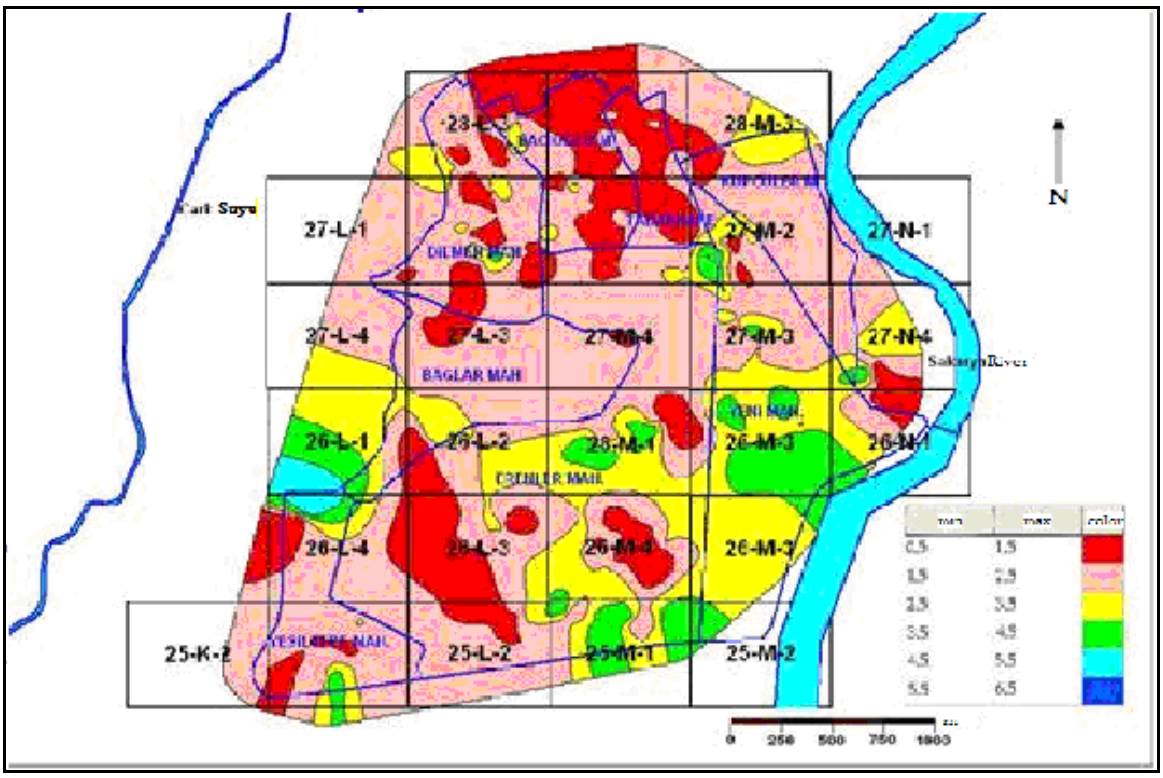

Fig. 4. Groundwater map of study area.

PREPARING THE SPT MAPS OF ERENLER REGION

OBTAINED DATA AND ANALYSIS

Data, were obtained from 488 drilling application results. The results were taken from Erenler municipality after 1999 Marmara Earthquake, applied by Municipality and some companies. 1/40000 scale section of a map involves border of streets, public works areas, cadastral section maps. These are digitalized and make projection. Obtained data were loaded into the maps with their coordinates and looked at section of data distribution. To understand data reliability from 27 piece section of maps, applied SPSS data election set for each sections.

SPT (STANDART PENETRATION TEST) MAPS

The test uses a thick-walled sample tube, with an outside diameter of 50 mm and an inside diameter of 35 mm, and a length of around 650 mm. This is driven into the ground at the bottom of a borehole by blows from a slide hammer with a weight of 63.5 kg (140 lb) falling through a distance of 760 mm (30 in). The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm (6 in) up to a depth of 450 mm (18 in) is recorded. The sum of the number of blows required for the second and third 6 in. of penetration is termed the "standard penetration resistance" or the "N-value". In cases where 50 blows are insufficient to advance it through a 150 mm (6 in) interval the penetration after 50 blows is recorded. The blow count provides an indication of the density of the ground, and it is used in many empirical geotechnical engineering formulae.

Figure 5 and Figure 6 are show the SPT values at 1.5 m and 3 m depth respectively. We can understood from SPT 1 and SPT 2 maps that the study area generally have low SPT values at related depths. East-southeast and southwest part of the study area have more better SPT values.

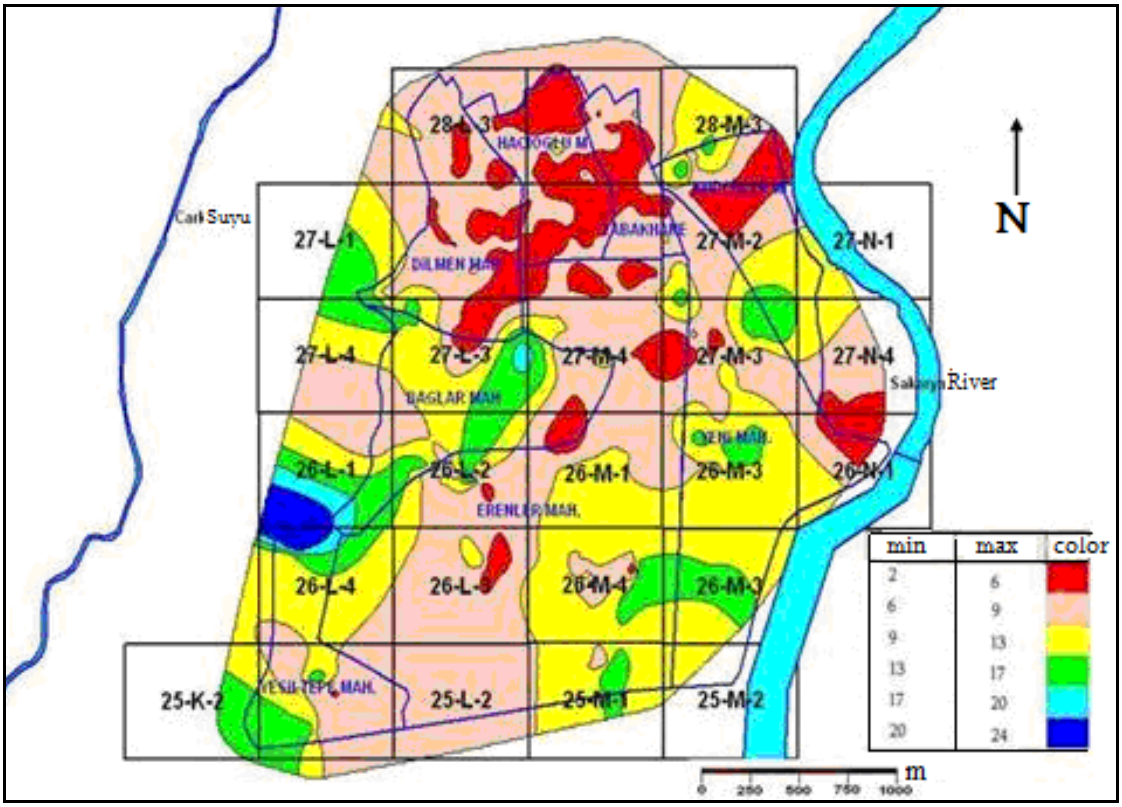

Fig. 5. SPT 1 map (1.5 m depth) of study area.

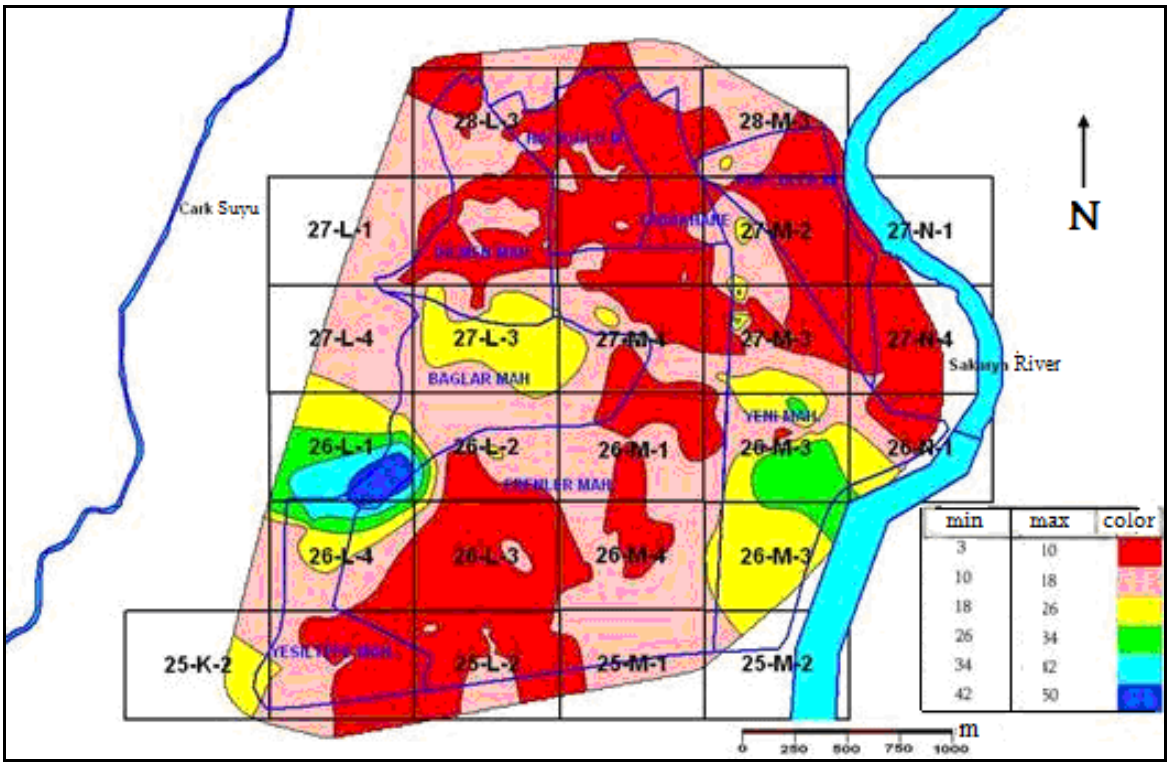

Fig. 6. SPT 2 map (3 m depth) of study area.

Figure 7 and Figure 8 are show the SPT values at 4.5 m and 6 m depth respectively. SPT 3 and SPT 4 maps are show that, there are some local increases on the SPT values in the

study area. Yeşiltepe, Baglar and Yeni districts have also high SPT values at these depths.

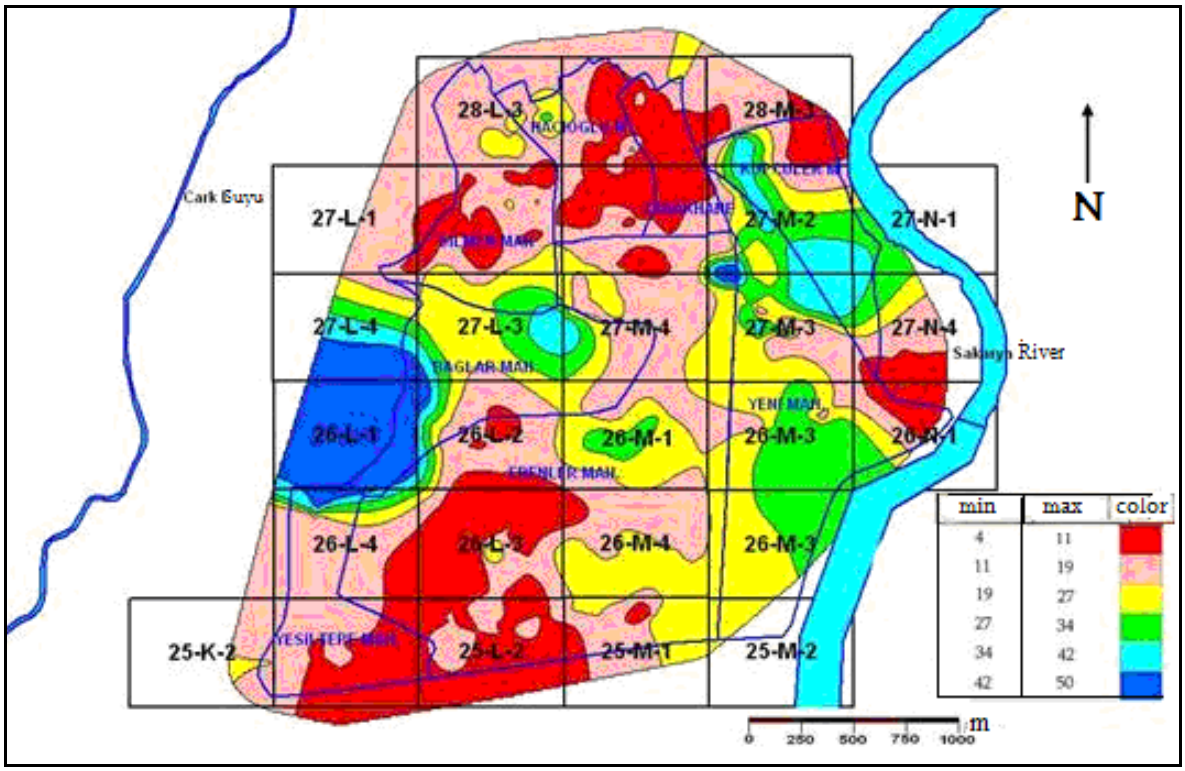

Fig. 7. SPT 3 map (4.5 m depth) of study area.

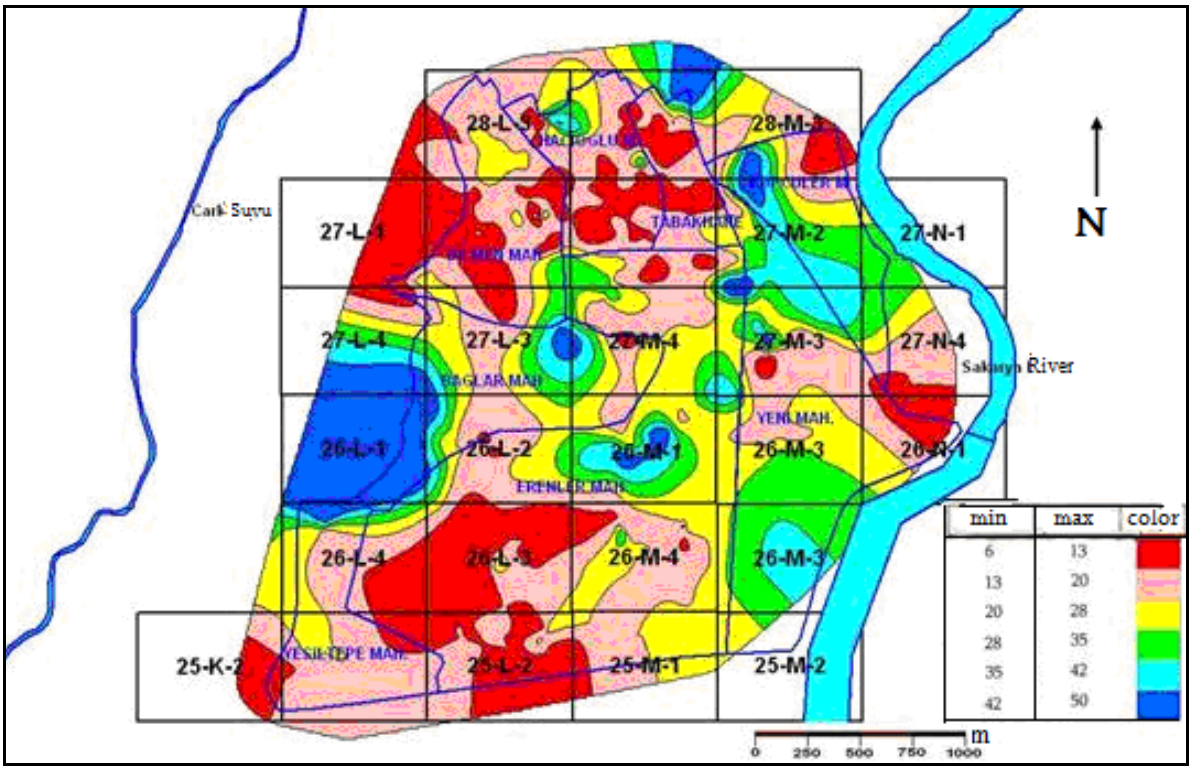

Fig. 8. SPT 4 map (6 m depth) of study area.

CONCLUSIONS

In this study, we aimed to point out the SPT values of Erenler region on GIS based maps. The SPT values belongs to 1,5 m, 3 m, 4,5 m and 6 m depths shown on GIS based maps and observed the variation on SPT values along with the depth rises.

We can clearly understood from all SPT maps that, the high topographic sides of study area (Yeşiltepe and Baglar districts) which are covered by flysch deposits have better soil conditions in the study area. At the same time, high SPT values at southwest part of the study area are stem from gravelly soils.

Along with the depth rises, Yeşiltepe, Baglar and Yeni districts keep their strong soil characteristics but Tabakhane and Hacıoglu districts and southern parts of study area are still have low SPT values.

Erenler region is located on southern part of Sakarya province. The study area has high level seismic hazard according to the North Anatolian Fault System which is located on southern part of Sakarya province and also Erenler Region. Considering the thickness of aluvium and high ground water level, it will be useful to make precaution such as drainage, excavation and ground improvement before construct a building in the study area.

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DETERMINATION OF RELATIONSHIPS BETWEEN DEFLECTION AND SAFETY FACTORS FOR A CONCRETE GRAVITY DAM IN EARTHQUAKE

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Key Words: Concrete Gravity Dam, Earthquake, Pseudo Analysis, Structural Analysis, Safety Factors.

ABSTRACT The aim of this study is to determine the relationship between deflection and safety factors for a concrete gravity dam (CGD) in earthquake. Tipping and sliding safety factors for a selected concrete gravity dam with base width "b" and height "H" were specified using the pseudo analysis in case of the b/H ratios and earthquake acceleration value between 0.1g-0.4g. Deflection values of CGD for the specified parameters were obtained from the structural analysis program "SAP2000". Deflection-safety factors curves depends on the b/H ratios and the values of earthquake acceleration were obtained. As a result of this analysis, safety factors reduced while the strain values were increasing.

INTRODUCTION

Rapid developments in computer technology enabled the application of advanced structural analysis techniques in the earthquake safety of concrete dams in the last few decades. However, many uncertainties remain in predicting the behavior of dams during heavy earthquakes. Amongst these are the crack formation and propagation processes in the concrete body of the dams, the nonlinear behavior of the foundations and the interaction of the water body in the reservoir with the dam structure, [Ghrib et.al. (1997), Chopra (1998), Kreuzer (2000)]. Static and dynamic approaches and linear or nonlinear finite element models have been used together with statistical methods in the studies of stability analysis for the safety evaluation of dams, [Hall (1998), Tinawi et. al. (2000), Leclerc et.al. (2007)]. Another issue is the most appropriate definition of the seismic input which may have many uncertainties, but its affect is very significant in the seismic design and seismic safety evaluation of the dams. Progress has been made in defining maximum credible earthquake (MCE), maximum design earthquake (MDE) and safety evaluation earthquake (SEE). With more seismic records available, understanding the effects of the very short duration peak ground acceleration (PGA) which may be 0.5g-1.0g or even higher in heavy earthquakes and the sustained effective seismic acceleration (which about 0.5-0.67 PGA)have improved. A very important indirect result of the seismic activity is the affect on the uplift forces due to increasing pressure acting through the cracks in the concrete body or under the foundations resulting from increased seepage. This must be taken into account in the stability analysis for the seismic design as it may have very significant effect for the safety of the dam.

It was reported that safety factors were decreased with increasing in earthquake acceleration and pressure reducer factor [Opan ve Temiz (2007)].

The aim of this study, determining the relation between deflection and safety factor at topping and toe for concrete gravity dam under earthquake. Selected concrete gravity dam with base width "b" and height "H" were specified using the pseudo analysis in case of the b/H ratios and earthquake acceleration value between 0.1g-0.4g. Deflection values of CGD for the specified parameters were obtained from the structural analysis program "SAP2000". Graphs are going to be obtained from the results of b/H ratio and

earthquake acceleration for the selected dam and deflection and safety factor are going to be evaluated.

FORCES ON A CONCRETE GRAVITY DAM UNDER EARTHQUAKE

Forces on the concrete gravity dam under earthquake were shown in Figure 1. In figure 1, F_E is the horizontal hydrodynamic load, F_x is the horizontal hydrostatic load, F_u is the vertical pore water pressure load, W is the weight of concrete dam and We is the concrete dam weight and earthquake load on the horizontal direction. Also, b, c, h, and H were shown parameters of the concrete dam.

Horizontal hydrostatic load is

$$F_X = \frac{1}{2}\gamma_W h^2 \tag{1}$$

Horizontal hydrodynamic load is

$$F_E = 0.555\alpha\gamma_W h^2 \tag{2}$$

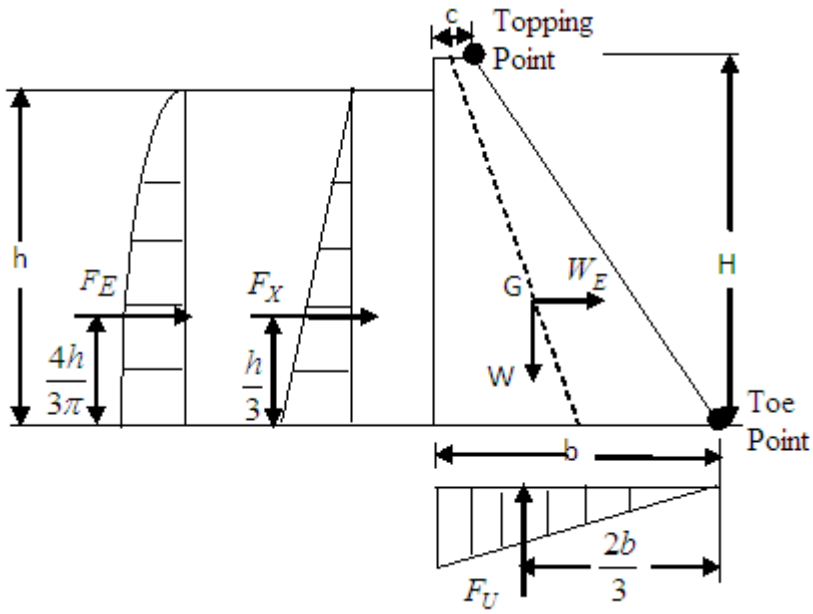

Fig. 1. Load on the dam mass under earthquake.

Vertical pore water pressure force is

$$F_U = \frac{1}{2}m\gamma_W hb(1+\alpha) \tag{3}$$

Weight of concrete gravity dam is

$$W = \frac{1}{2}\gamma_C bh \tag{4}$$

Earthquake force due to concrete gravity dam weight is

$$W_E = \frac{1}{2}\alpha\gamma_C bh \tag{5}$$

where γ_w and γ_c are the specific weight of concrete and water relatively, m is the pressure reducer factor for the water pressure force ($0 \le m \le 1$), α is the earthquake acceleration, and is the cofactor connected with earthquake acceleration.

PSEUDO ANALYSIS

Pseudo analysis for dams is based on the dam stability. Stability of the concrete gravity dam is provided by the moments. If moment that try to tipping dam at the toe points are lower than moments that resist to the moments, stability is supplied. This can be explained with following equation.

$$W.\frac{2b}{3} \ge F_X.\frac{h}{3} + F_E.\frac{4h}{3\pi} + F_U.\frac{2b}{3} + W_E.\frac{h}{3} \tag{6}$$

with arrangement of Equation 1, this is

$$\left[(2+\alpha)\frac{\gamma_C}{\gamma_W} - 2m(1+\alpha)\right]\frac{b^2}{h^2} \ge (1+1.41\alpha) \tag{7}$$

and then, this is

$$\frac{b}{h} \ge \frac{1}{\sqrt{2}}\left[\frac{(1+1.41\alpha)}{(1+\frac{\alpha}{2})\frac{\gamma_C}{\gamma_W} - (1+\alpha)m}\right]^{\frac{1}{2}} \tag{8}$$

Foundation width/height ratio of the dam was obtained as a result of stability. Converting from inequality to equality, this ratio is shown below,

$$\frac{b}{h} = \left[\frac{(1+1.41\alpha)}{(1+\frac{\alpha}{2})\frac{\gamma_C}{\gamma_W} - (1+\alpha)m}\right]^{\frac{1}{2}} \tag{9}$$

From this equation, relationship between b/h ratio and m and α is obtained. Figure 2 is shown this relationship. In Figure 2, $\frac{\gamma_C}{\gamma_W}$ is 2,5.

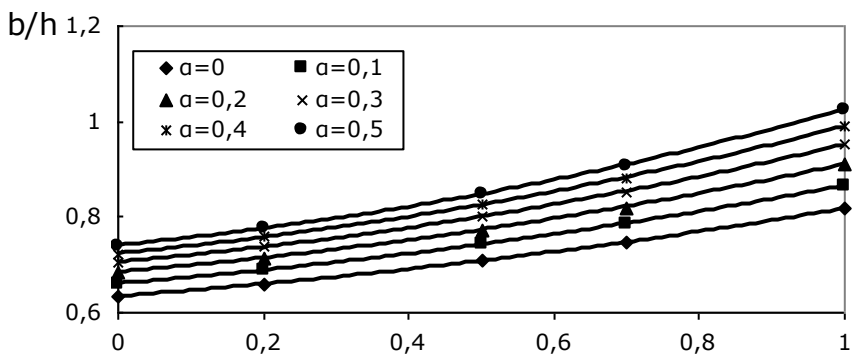

Fig. 2. Relationship between b/h ratio and m and α (Opan, Temiz, 2007).

Various safety factors are obtained from the pseudo analysis. These safety factors are tipping and sliding safety factors.

Tipping safety factor is attained by a ratio of resist moment to subvert moment. According to this definition, tipping safety factor is obtained below.

$$\varepsilon_d = \frac{W.\frac{2b}{3}}{F_{X.}\frac{h}{3} + F_{U.}\frac{2b}{3} + F_E.\frac{4h}{3\pi} + W_E.\frac{h}{3}} \tag{10}$$

When Equation 10 is arranged, Equation 11 is

$$\varepsilon_d = \frac{2.\frac{\gamma_C}{\gamma_W}(\frac{b}{h})^2}{(1+1.41\alpha) + \left[2m(1+\alpha) + \alpha.\frac{\gamma_C}{\gamma_W}\right]\left(\frac{b}{h}\right)^2} \tag{11}$$

where $\frac{\gamma_C}{\gamma_W}$ is accepted as 2.5 and b/h is found versus m and α. Changes of ε_d connected with m and α are obtained. ε_d is decreased with increasing in m and α.

Tipping safety factor of the dam is attained from a ratio of resist force to sliding force. According to this definition, sliding safety factor is obtained below.

$$\varepsilon_k = \frac{(W - F_U)k}{F_X + F_E + W_E} \tag{12}$$

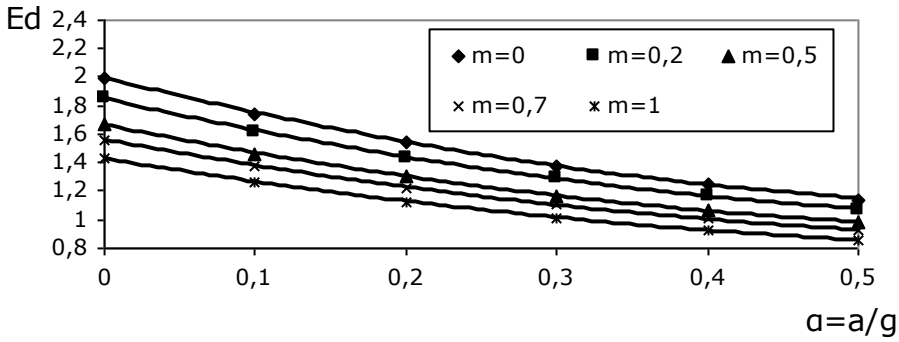

Fig. 3. Changes of ε_d connected with m and α (Opan, Temiz, 2007).

with the arrangement of this equation, Eq.(13) is formed.

$$\varepsilon_k = \frac{\left[\frac{\gamma_C}{\gamma_W} - m(1+\alpha)\right]k.\frac{b}{h}}{(1+1.11\alpha) + \frac{\gamma_C}{\gamma_W}.\alpha.\frac{b}{h}} \tag{13}$$

Where $\frac{\gamma_C}{\gamma_W} = 2.5$ and $k = 1.0$ are accepted and b/h is obtained versus m and α. Changes of as ε_k connected with m ve α are calculated as shown in Figure 4. It is seen that ε_k decreased with increasing in m and α.

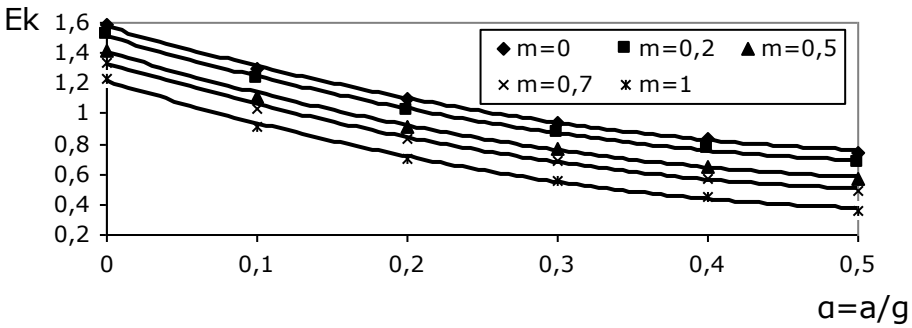

Fig. 4. Changes of ε_k connected with m and α (Opan, Temiz, 2007).

STRUCTURAL ANALYSIS VIA SAP2000

Concrete gravity dam is modeled by using the shells of the finite element in SAP2000 program. Principal axis of the dam in this program is shown in Figure 5. Shells are separated into 0.50X0.50 meshes as given in Figure 6. These meshes are chosen so little dimension because of obtaining results too close the real. Deflection values of topping and points are attained by establishing different model in different mesh dimension. Mesh dimension is defined by considering solving time and the situation that there are no changes in deflection. Foundation soil of dam was modeled via spring element. Soil is accepted as sand-loose and sand-medium dense and modulus of sub-grade reaction is chosen as k_S= 2500 t/m^3. Stiffness of spring element was determined by using modulus of sub-grade reaction and foundation effective area. Analysis are performed at h=10 m, H=10, m=0.2, 0.6 ve 1.0, b/h ratio 0.6, 0.8 ve 1.0. Deflection values at every direction are read from topping and toe points as shown in Figure 1. Deflection vectors of these points are obtained below.

$$\delta = \sqrt{\delta_x^2 + \delta_y^2 + \delta_z^2} \tag{14}$$

where δ is the deflection vector.

As a results of performed analysis, changes of deflection of topping point connected with m and α at b/H=0.6, 0.8 and 1.0 are shown in Figures 7, 8 and 9, respectively. Changes of deflection of the point connected with m and α at b/H=0.6, 0.8 and 1.0 are shown in Figures 10, 11 and 12, respectively. Deflections increased with increasing in m and α, and deflection decreased with increasing in b/H ratio.

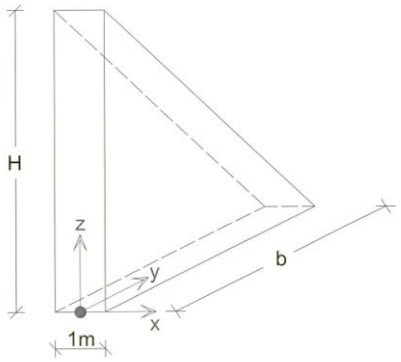

Fig. 5. Principal axial of dam.

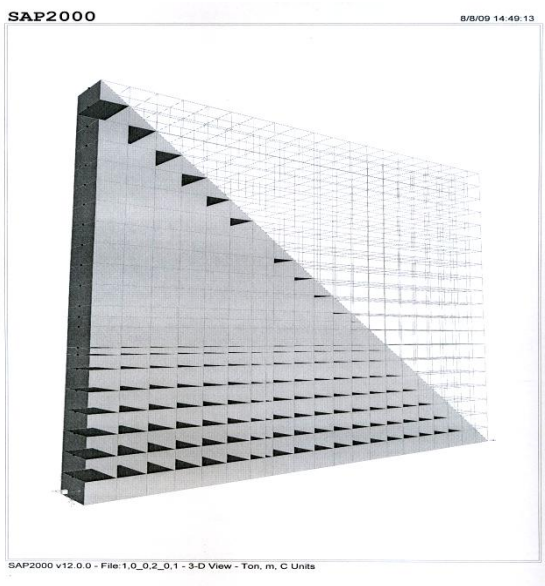

Fig. 6. View of the meshes on the shells.

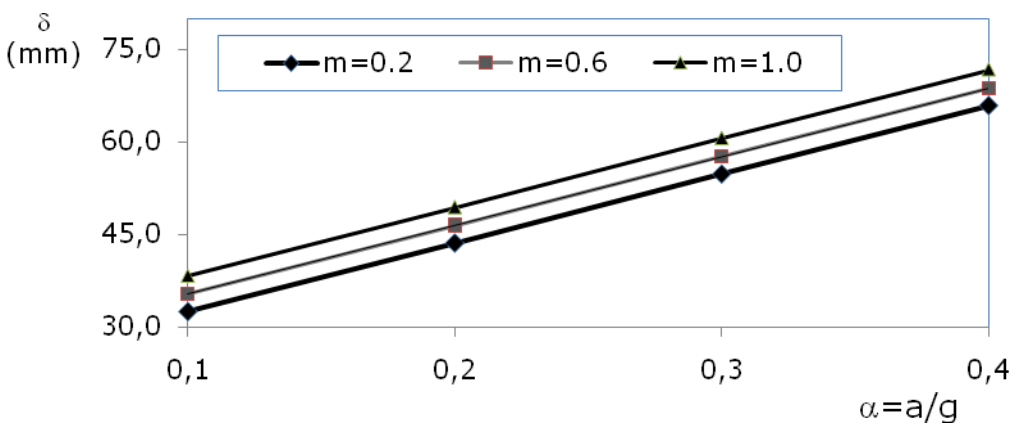

Fig. 7. Changes of deflection connected with m and α on (y)-direction as weight at topping point for b/H=0.6.

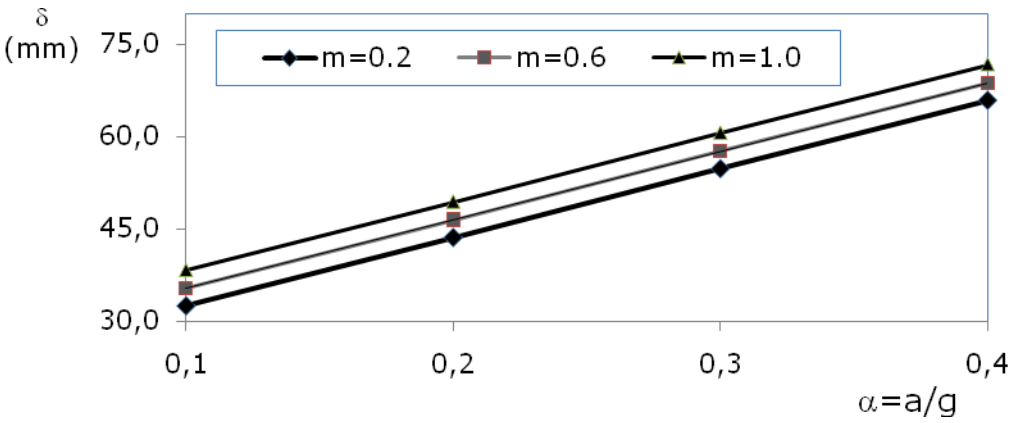

Fig. 8. Changes of deflection connected with m and α on (y)-direction as weight at topping point for b/H=0.8.

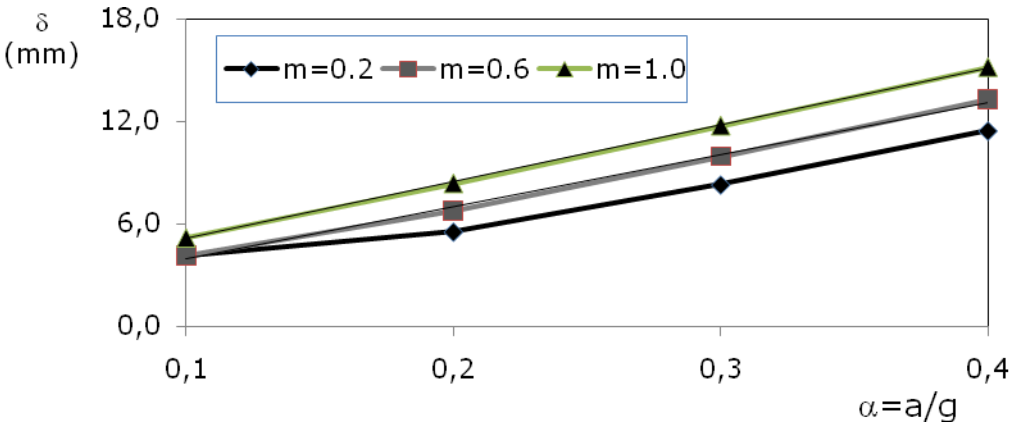

Fig. 9. Changes of deflection connected with m and α on (y)-direction as weight at topping point for b/H=1.0.

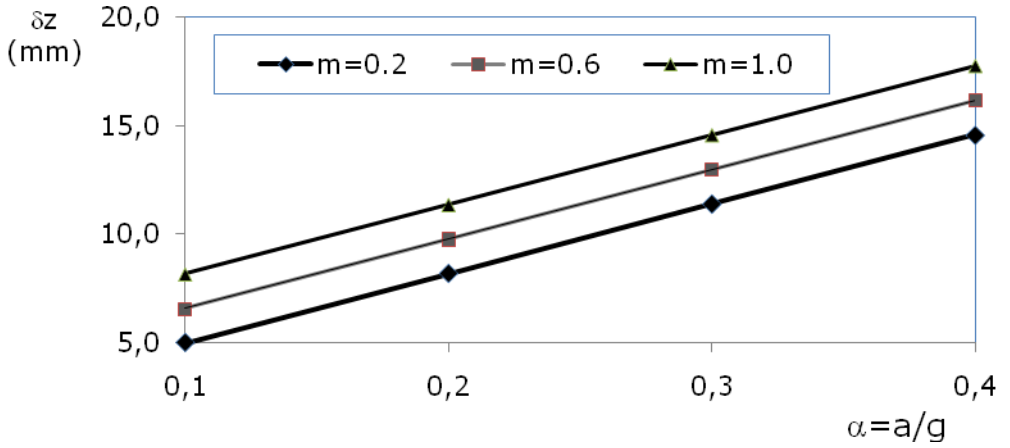

Fig. 10. Changes of deflection connected with m and α on (y)-direction as weight at toe point for b/H=0.6.

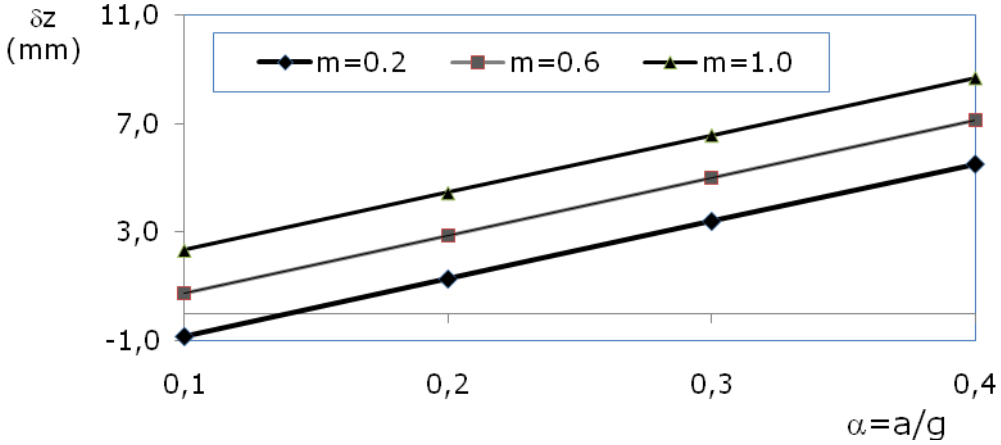

Fig. 11. Changes of deflection connected with m and α on (y)-direction as weight at toe point for b/H=0.8.

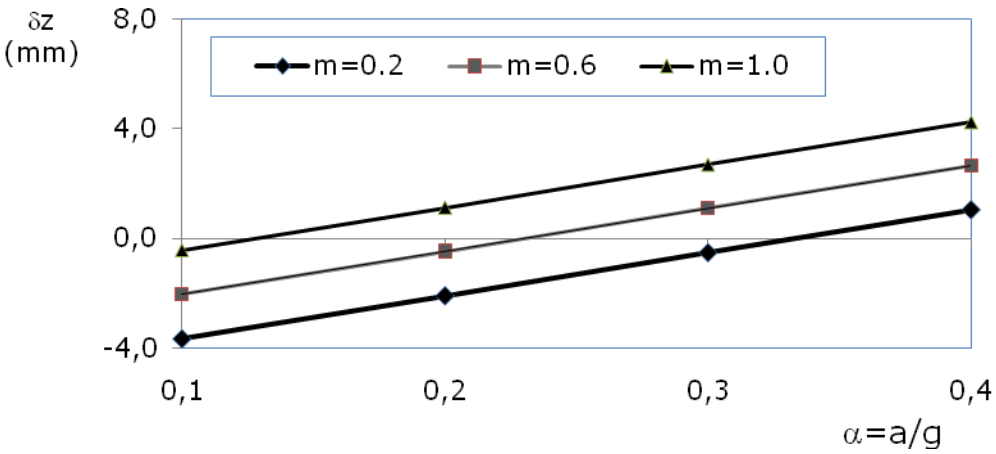

Fig. 12. Changes of deflection connected with m and α on (y)-direction as weight at toe point for b/H=1.0.

CONCLUSIONS

In this study, tipping and sliding safety factor of dam under earthquake and deflection at toe and topping point are determined. Pseudo analysis is performed in order to define safety factor. Changes of these safety factors versus m and α are investigated and it is seen that safety factors decreased with increasing in m and α. SAP2000 is used for determining of deflection at toe and topping point. According to changes of these deflection versus m and α it can be said that deflection increased with increasing in m and α. In addition, safety factors increased with increasing in b/H but deflection decreased with increasing in b/H.

In further studies, analysis and studies about cracks formation and dispersion on concrete gravity dam under earthquake can be performed by extending of this study.

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SULAMA AMAÇLI BARAJLARDA OLASI TAŞKIN VE KURAKLIK DURUMU IÇIN BARAJ KAPASITELERININ BELIRLENMESI

DETERMINATION OF RESERVOIR CAPACITIES FOR PROBABLE FLOOD AND DROUGHT CASES IN THE RESERVOIRS WITH IRRIGATION OBJECTIVE

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Anahtar Kelimeler: Çok Barajlı Sistem, Çok Amaçlı Optimizasyon, Taşkın/Kuraklık Kontrolü, Depolama Kapasitesi, Ardışık Yaklaştırmalı Dinamik Programlama.

ÖZ Bu çalışmada, Ceyhan Havzası'nda bulunan sulama amaçlı ve çok barajlı bir su kaynakları sistemi üzerine uzun ve kısa sureli planlama için enerji üretimi ve taşkın/kuraklık kontrolü amaçlı olarak optimizasyon modelleri kurulmuştur. Modellerde optimizasyon tekniği olarak ardışık yaklaştırmalı dinamik programlama kullanılmıştır. Uzun süreli optimal işletme modelinde, amaç fonksiyonu, enerji üretiminin maksimize edilmesi şeklindedir. Kısa süreli optimal işletme modellerinde, amaç fonksiyonu, ilave taşkın/kuraklık kontrol hacmine ve mansaptaki debi kontrol kısıdına bağlı olarak üç aşamalıdır. Barajlardan bırakılan akımlar minimize edilmeye çalışılmakta ve diğer barajın aksına akarsu yatağı boyunca kinematik dalga yaklaşımı kullanılarak taşınmaktadır. Optimizasyon sonucu elde edilen işletme seviyelerine bakıldığında optimizasyon sürecini büyük hacimli barajların kontrol ettiği ve yönettiği görülmüştür. Elde edilen işletme seviyeleri kullanılarak barajların depolama kapasiteleri belirlenmiştir.

GİRİŞ

Su kaynakları planlaması, ülke ekonomisi ve öngörülen hayat standartları ile bütünleşmiş, çok boyutlu ve karmaşık bir süreçtir. Su kaynakları planlaması ve yönetimi, istenilen amaçlar doğrultusunda ve öngörülen kriterler çerçevesinde su kaynaklarının en verimli kullanımını sağlayacak faaliyetlerin tümünü kapsamalıdır (Opan, 2007). Çok barajlı sistemlerde kuraklık/taşkın kontrolü ve yönetimi kısa süreli planlama kapsamında göz önüne alınmaktadır. Barajların depolama kapasiteleri, taşkın ve kuraklık gibi kısa süreli olaylar karşısında gelen akımları kontrol edilecek ve yönetebilecek durumda olmalıdır. Bunun için depolama kapasiteleri, taşkın ve kuraklık durumu için gerekli olabilecek hacimler dikkate alınarak belirlenmelidir.

Çok barajlı sistemlerde enerji optimizasyonu, taşkın kontrolü ve sulama gibi amaçları gerçekleştirmek için çeşitli yöntemlerin kullanıldığı birçok çalışma bulunmaktadır, (Labadie, 2004). Enerji optimizasyonu ile ilgili çalışmalarda, enerji üretim fonksiyonunun doğrusal olmaması bakımından, genellikle, dinamik programlama optimizasyon tekniği ve simülasyon kullanılmıştır, (Güvel, 1997, Sert, Kızıltan, Dalgıç, 1982, Sert, Öcal, Oktay, Ertuğrul, 1983, Yakowitz, 1982, Yeh, 1985, Yurtal, 1993). Taşkın kontrolü ve sulama gibi amaçlar için ise, su miktarı ve finansal açıdan simülasyon, doğrusal ve doğrusal olmayan programlama optimizasyon teknikleri ile yapılmış çalışmalar vardır, (Needham, Watkins and Lund, 2000).

Kısa süreli işletme optimizasyonunda zaman dilimi olarak saat (veya özel durumlarda gün) kullanıldığı için akımların akarsu yatağında ötelenerek baraj aksına taşınmasında kullanılacak bir yaklaşıma gerek vardır. Akarsu yatağında akım öteleme üzerine kinematik dalga, dinamik dalga ve Muskingum gibi yaklaşımlar bulunmaktadır. Bu

çalışmada, akımların baraj aksına taşınmasında kinematik dalga yaklaşımı kullanılmaktadır. Kinematik dalga yaklaşımında, St. Venant denklemlerindeki ivme ve basınç terimleri ihmal edildiğinden, dalga hareketi sadece süreklilik denklemi ile tanımlanmaktadır. Kinematik dalga yaklaşımı için çeşitli çalışmalar yapılmıştır. Kinematik ve difüzyon yaklaşımlarının uygulanabilirliğini gösteren çalışmalar bulunmaktadır, [6]. Yüzeysel sulama için kinematik dalga yaklaşımı ile ilgili öneri niteliğinde çeşitli araştırmalar yapılmıştır, (Sherman and Singh, 1978, Sherman and Singh, 1982). HEC–1 benzetim programı içerisinde kinematik dalga öteleme tekniğini tanımlanmış ve uygulanmıştır, (DeVrices and MacArthur, 1979).

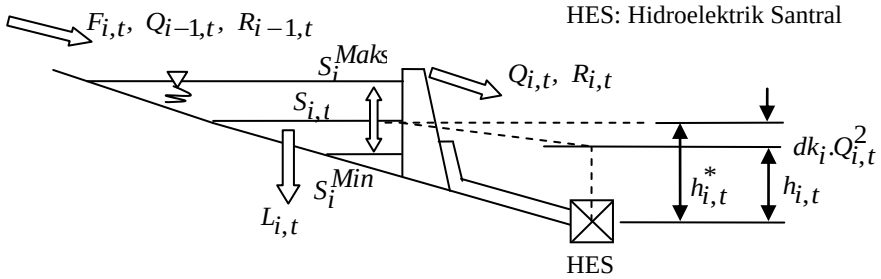
Bu çalışmada, Ceyhan Havzası'nda bulunan sulama amaçlı ve çok barajlı bir su kaynakları sistemi üzerine uzun ve kısa sureli planlama için enerji üretimi ve taşkın/kuraklık kontrolü amaçlı olarak optimizasyon modelleri kurulmuştur. Modellerde optimizasyon tekniği olarak ardışık yaklaştırmalı dinamik programlama kullanılmıştır. Uzun süreli optimal işletme modelinde, amaç fonksiyonu, enerji üretiminin maksimize edilmesi şeklindedir. Kısa süreli optimal işletme modellerinde, amaç fonksiyonu, ilave taşkın/kuraklık kontrol hacmine ve mansaptaki debi kontrol kısıdına bağlı olarak üç aşamalıdır. Barajlardan bırakılan akımlar minimize edilmeye çalışılmakta ve diğer barajın aksına akarsu yatağı boyunca kinematik dalga yaklaşımı kullanılarak taşınmaktadır.

SİSTEMİN MATEMATİKSEL MODELİ

Bir akarsu üzerinde bulunan birçok sayıda baraj ve hidroelektrik santralden (HES) oluşan bir su kaynakları sistemi çok barajlı ve çok amaçlı (taşkın kontrolü ve enerji üretimi gibi) olarak tanımlanabilir. Şekil 1' de çok barajlı sisteme ait i -barajının t -ayı f -saati ile ilgili işletme değişkenleri gösterilmiştir. Burada, $i = 1,2,3,..M$: Baraj sayısını, $t = 1,2,3,..KM$: Ay olarak zamanı ve $f = 1,2,3,..TM$: Saat olarak zamanı göstermektedir. Bu sisteme ait herhangi bir i -barajı için t -Ayı f -saati için su dengesi ilişkisi;

$$(S_{i,f+1} - S_{i,f} = F_{i,f} + Q_{i-1,f} + R_{i-1,f} - Q_{i,f} - R_{i,f} - L_{i,f})_t \qquad (1)$$

şeklinde ifade edilebilir.

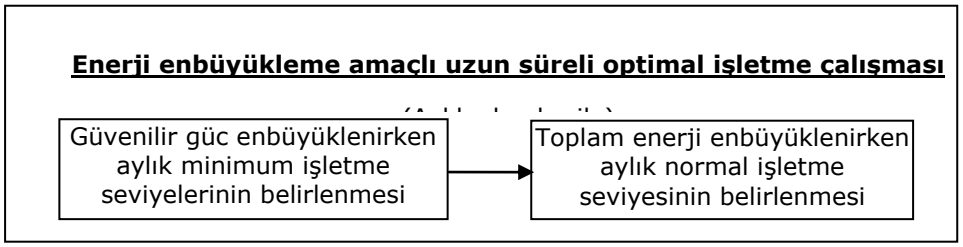


Şekil 1. Sisteme ait i -barajının t -ayı f -saati ile ilgili işletme değişkenleri.

Burada, $S_{i,f}$: Barajda depolanan su miktarını, $F_{i,f}$: Barajın havzasından gelen akım miktarını, $Q_{i,f}$: Barajdan enerji üretimi için bırakılan akım miktarını, $R_{i,f}$: Barajdaki dolu savaktan bırakılan akım miktarını, $L_{i,f}$: Baraja ait buharlaşma ve diğer kayıpları göstermektedir.

ENERJİ ENBÜYÜKLEME AMAÇLI UZUN SÜRELİ OPTİMAL İŞLETME

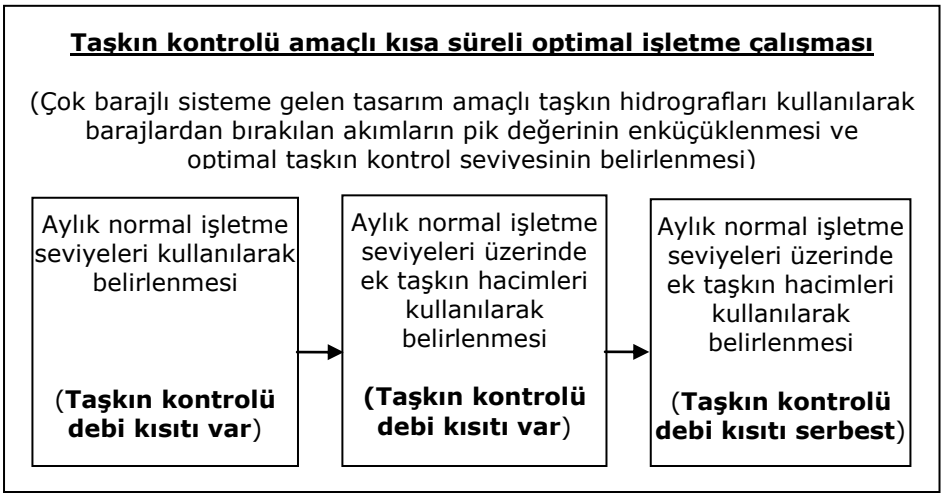
Uzun süreli planlama için kurulan optimizasyon modelinde, kritik dönemin (kurak dönem) aylık akımları kullanılarak güvenilir güç enbüyüklenmekte ve aylık minimum işletme seviyeleri elde edilmektedir. Buradan enbüyüklenen güvenilir güç, aynı modelde kısıt olarak kullanılıp, aylık ortalama akımlar ile toplam enerji enbüyüklenmekte ve aylık normal işletme seviyeleri belirlenmektedir (Opan, 2007). Şekil 2' de, enerji enbüyükleme amaçlı uzun süreli optimal işletme çalışması süreci özetlenmiştir.



Şekil 2. Enerji enbüyükleme amaçlı uzun süreli optimal işletme çalışması süreci.

TAŞKIN KONTROLÜ AMAÇLI KISA SÜRELİ PLANLAMA İÇİN OPTİMİZASYON MODELİ

Taşkın kontrolü amaçlı kısa süreli planlama için kurulan optimizasyon modelinde, barajlara havzasından gelen tasarım amaçlı taşkın hidrografı akım verileri ile, taşkın kontrolü debi kısıtı ve ek taşkın kontrol hacmi kullanılması durumuna göre barajdan bırakılan akımlar enküçüklenmekte ve aynı zamanda sistemden beklenen optimal faydaya (enerji üretimi v.s.) ulaşılmaya çalışılmaktadır. Burada, kullanılan ek taşkın kontrol hacimleri, işletme sonucu elde edilen işletme seviyelerinden düşülerek taşkın kontrol seviyeleri belirlenmektedir. Herhangi bir baraj için elde edilen bu taşkın kontrol seviyelerinden en küçük olanı optimal taşkın kontrol seviyesi olarak seçilmektedir [5]. Şekil 3' de, taşkın kontrolü amaçlı kısa süreli optimal işletme süreci gösterilmektedir.

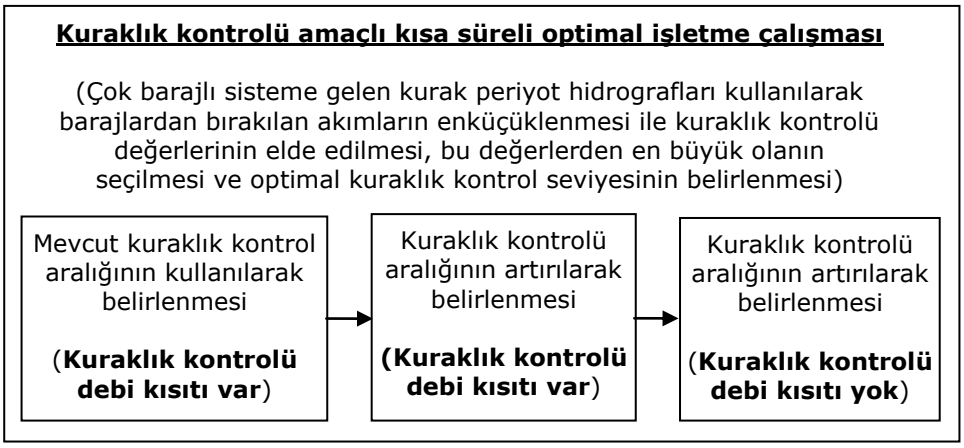


Şekil 3. Taşkın kontrolü amaçlı kısa süreli optimal işletme süreci.

KURAKLIK KONTROLÜ AMAÇLI KISA SÜRELİ PLANLAMA İÇİN OPTIMIZASYON MODELİ

Kuraklık kontrolü amaçlı kısa süreli planlama için kurulan optimizasyon modelinde, belirli bir ay için girilen kuraklık hidrografının sistem tarafından depolanıp barajdan bırakılan enbüyük akımın enküçüklenmesi amaç fonksiyonu olarak tanımlanırken, barajdan toplam debi çıkışının (enerji üretimi için ve savaklanan) mansap için kuraklık ve kirlilik kontrolü

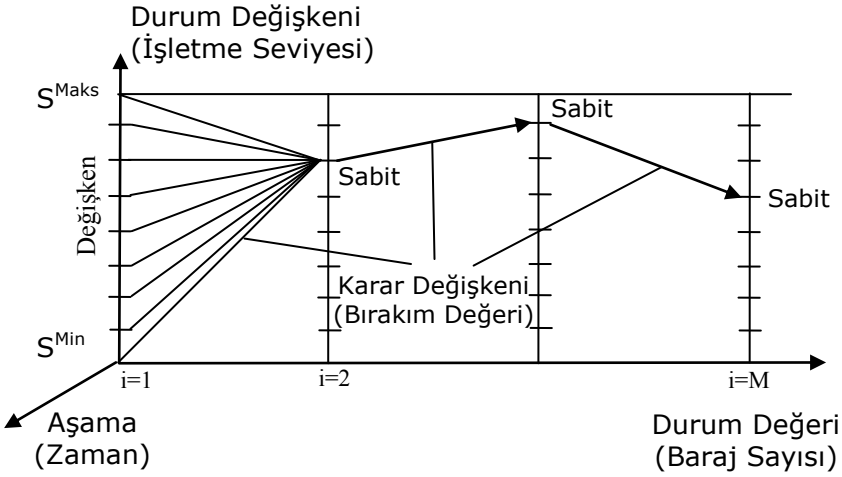
debisinin altında olması kısıtı konulmaktadır. Burada, kullanılan ek kuraklık kontrol hacmi, elde edilen işletme seviyelerinden düşülerek kuraklık kontrol seviyeleri belirlenmektedir. Bu seviyelerdeki en büyük işletme değeri optimal kuraklık kontrol seviyesini göstermektedir (Opan, 2007). Şekil 4' de, kuraklık kontrolü amaçlı kısa süreli optimal işletme süreci verilmektedir.



Şekil 4. Kuraklık kontrolü amaçlı kısa süreli optimal işletme süreci.

KULLANILAN OPTİMİZASYON TEKNİĞİ: DPSA

Ardışık yaklaştırmalı dinamik programlama yönteminin amacı, çok karar değişkenli bir dinamik programlama problemini, her birinde tek bir karar değişkeni olan dinamik programlama ait problemlere ayrıştırmak ve böylece karar değişkenlerini teker teker ele alarak ana problemi çözmektir.



Şekil 5. Dinamik programlamada herhangi bir aşamasındaki durum değeri ve durum-karar değişkeninin şematik gösterimi (Opan, 2007).

Bu metodun avantajı, (n) boyutlu bir dinamik programlama problemini bir dizi tek boyutlu probleme indirgeyerek, hem hesaplama zamanını hem de bilgisayar bellek gereksinimlerini azaltmaktır. Çok barajlı ve çok amaçlı baraj işletmesinde uygulanan dinamik programlama yöntemlerinin kullanıldığı bilgisayar belleği ve zamanı (n)'ye bağlı olarak üssel (y) doğrusal bir artış göstermektedir. En iyi çözümün bulunabilmesi için başlangıç çözümünün dikkatli seçilmesi ve birden fazla başlangıç çözümü denenerek sonuçların karşılaştırılması gerekebilir. Dinamik programlamada üç değişken tipi vardır. Bunlar:

1. Sistemin davranışını olduğu gibi tanımlayan ve sistemin herhangi bir andaki durumunu gösteren durum değişkenleri, (x, sistemde barajın işletme seviyesidir.)

2. Seçilen bir amaca göre sistemin kontrol edilebilir girdileri hakkında karar vermeyi sağlayan karar değişkenleri, (d, sistemde barajdan enerji üretimi için bırakılan ve dolu savaktan savaklanan su miktarlarıdır.)

3. Bu kararların verildikleri aralıkları belirleyen aşama değişkenleri, (t, sistemde işletme zamanını göstermektedir.)

şeklindedir. Şekil 5'de, dinamik programlamada herhangi bir aşamasındaki durum değeri ve durum-karar değişkeninin şematik gösterimi gösterilmiştir. Bu girdilerin bazı kısıtlara bağlı olarak aldıkları değer takımına politika denmektedir. Bu politikanın sistemin çıktıları üzerine etkisini belirleyen ölçüt ise, amaç fonksiyonu olarak tanımlanmaktadır.

KISA SÜRELİ İŞLETME OPTİMİZASYONDA KİNEMATİK DALGA YAKLAŞIMI

Kinematik dalga yaklaşımında, St. Venant denklemlerinde ivme ve basınç terimleri ihmal edildiğinden, barajlardan bırakılan akımların oluşturduğu dalga hareketi, süreklilik denklemi ile tanımlanmaktadır. Bu yaklaşım kullanılarak barajlardan bırakılan akımları diğer baraja ulaşma ve zamanını gösteren eğriler oluşturulmuştur.

Bu yaklaşımla yapılan modellemede, barajlardan bırakılan akımlar, ilk olarak dolu savak ve enerji üretimi kapasitelerinin oluşturduğu toplam kapasite ile karşılaştırılmaktadır. Burada bırakılan akım, bu kapasiteden küçük ise, bırakılma zamanı, akımın diğer baraja ulaşma zamanını belirleyen denklemden elde edilen zamanın üzerine ilave edilmekte ve diğer baraja ulaşma değerini belirleyen denklemler ile akımın ötelenmesi gerçekleştirilmektedir. Bırakılan akım bu kapasiteden büyük ise, kapasiteye aşmayacak şekilde öngörülen bırakılma hidrografı ile, akımın diğer baraja ulaşma zamanı ve değeri belirlenmektedir. Bu elde edilen değerler diğer baraj aksında, barajın havzasından gelen akımlar üzerine ilgili zaman durumuna göre ilave edilmektedir.

KAPASİTENİN BELİRLENMESİ

Barajların aktif depolama hacmi, barajın kapasitesi olarak ele alınabilir. Barajın kapasitesi iki ayrı kapasiteden oluşmaktadır. Birincisi, ham kapasite, diğeri ise, taşkın/kuraklık gibi riskli durumlara bağlı kapasite şeklindedir. Baraja aylık olarak gelen ortalama akımlar kullanılarak ham kapasite belirlenirken, riskli kapasite ise, tekerrür aralığı olan saatlik veya günlük akımlar söz konusudur. Kısa süreli optimal işletme ile taşkın/kuraklık kontrol kapasiteleri belirlenebilir. Taşkın için kontrol kapasitesi yılın sulak döneminde, kuraklık için kontrol kapasitesi ise yılın kurak döneminde söz konusu olduğundan toplam kapasite, bunların ham kapasite üzerine doğrudan toplamından değil uygun bir birleşiminden elde edilmelidir. Buna göre, kapasitenin en büyük durumuna karşılık gelen işletme seviyesi barajın maksimum işletme seviyesidir. Kapasiteyi belirlenmesinde aşağıdaki adımlar izlenebilir:

1. Barajın bilinen maksimum işletme seviyesi kullanılarak kısa süreli optimal işletme ile optimal taşkın/kuraklık kontrol seviyeleri belirlenir.

2. Uzun süreli optimal işletme ile aylık normal işletme seviyesi elde edilir.

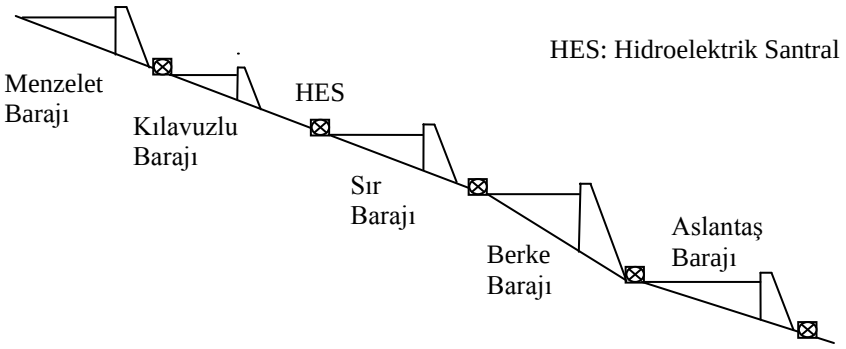
3. Normal işletme seviyesinden taşkın/kuraklık kontrol seviyesinin mutlak olarak farkı bulunur.

4. Bu fark normal işletme seviyesi üzerine ilave edilerek maksimum işletme seviyesi yeniden belirlenmiş olur.

5. Yenilenen maksimum işletme seviyesine karşılık gelen hacim barajın kapasitesi olmaktadır.

OPTİMİZASYON MODELLERİ İÇİN UYGULAMA

Ceyhan havzasında Ceyhan nehri ana kolu üzerinde ardışık şekilde bulunan çok barajlı bir su kaynakları sistemi uygulama amaçlı olarak seçilmiştir. Şekil 6' da, seçilen çok barajlı bir su kaynakları sisteminin şematik görünüşü verilmektedir. Bu sisteme ait barajlardan Menzelet ve Kılavuzlu Barajları, enerji ve sulama amaçlı; Sır ve Berke Barajları, enerji amaçlı ve Aslantaş Barajı ise, enerji, sulama, içmesuyu ve taşkından koruma amaçlı olarak planlanmışlardır. Tablo 1' de Ceyhan Havzası' nda Ceyhan Nehri ana kolu üzerinde ardışık şekilde bulunan barajlara ait veriler verilmiştir. Tablo 2' de, barajlardaki hacim yükseklik ilişkileri gösterilmiştir. Bu veriler uzun/kısa süreli optimal işletme modelinde kullanılmıştır.



Şekil 6. Ceyhan Havzası'nda Ceyhan Nehri ana kolu üzerinde ardışık olarak bulunan çok amaçlı ve çok barajlı bir su kaynakları sisteminin şematik görünüşü.

Tablo 1. Ceyhan Havzası' nda Ceyhan Nehri ana kolu üzerinde ardışık şekilde bulunan barajlara ait veriler.

Barajlar	Menzelet	Kılavuzlu	Sır	Berke	Aslantaş
Yağış Alanı (km^2)	8340	8486	12950	13222	14874
Kurulu Güç (MW)	124	54	273	510	138
Baraj Yüksekliği (m)	150.5	56	120	201	95
Maksimum İşletme Seviyesi (m)	609.4	485.5	440	345	159
Minimum işletme seviyesi (m)	560.2	483.5	418.8	288.8	130
Maksimum Hacim (10^6 m^3)	1950	74	1117.94	427	1990
Minimum Hacim (10^6 m^3)	533.6	69	451.3	119	530
Kuyruksuyu kotu (m)	485.5	440	345	159	84
Dolu Savak Kapasitesi (m^3/s)	6112	6112	7460	2000	10700
Enerji Üretimi Kapasitesi (m^3/s)	130.8	148.4	114.25	300	270

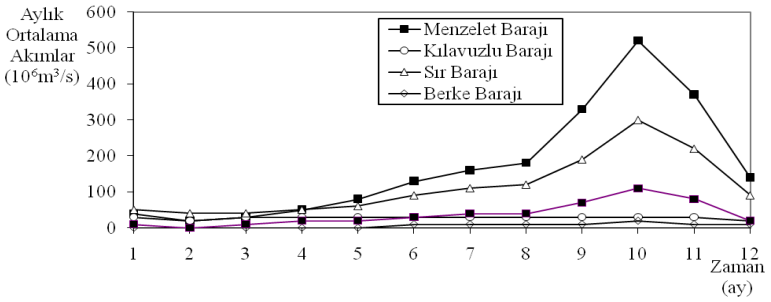
Tablo 2. Barajlarda hacim yükseklik ilişkisi ($h=a.S^b$, h (m), S (10^6 m^3)).

Barajlar	Menzelet	Kılavuzlu	Sır	Berke	Aslantaş
a	15.494	9.4261	28.264	42.036	3.8336
b	0.2998	0.4092	0.2067	0.2581	0.3947

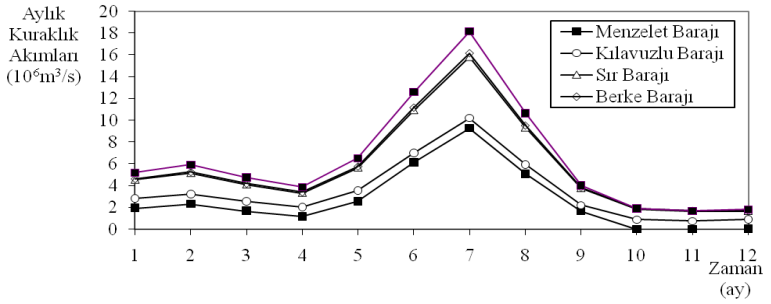
Tablo 3'de, barajlar arası akımların taşınması için kinematik dalga yaklaşımında kullanılmak üzere barajlar arası akarsu kesitinin ve yatağının özellikleri verilmiştir. Burada belirtilen akarsuya ait kesit özellikleri ve bazı parametreler, akarsuya ait ortalama akım verileri gözetilerek üretilmiştir. Tablo 3'de, L: Barajlar arası akarsuyun uzunluğunu, H: Barajlar arası kot farkını, S: Akarsudaki hidrolik eğimi, n: Tabandaki Manning katsayısını, b,h: Kesit büyüklüğünü, A: Kesit alanını, P: Islak çevreyi, R: Hidrolik yarıçapı, Q:ortalama debiyi göstermektedir.

Tablo 3. Barajlar arası akarsu kesitinin ve yatağının özellikleri.

Barajlar arası	**L (m)**	**H (m)**	**S**	**n**	**h (m)**	**b (m)**	**A (m^2)**	**P (m)**	**R (m)**	**Q (m^3/s)**
Menz.-Kılavuzlu	6820	123.9	0.0182	0.025	1.5	30	45	33	1.36	298
Kılavuzlu-Sır	35610	45.5	0.0013	0.025	2.5	48	120	53	2.26	295
Sır-Berke	4340	95	0.0219	0.025	1.4	30	42	32.8	1.28	293
Berke-Aslantaş	45610	186	0.0041	0.025	2	40	80	44	1.82	304



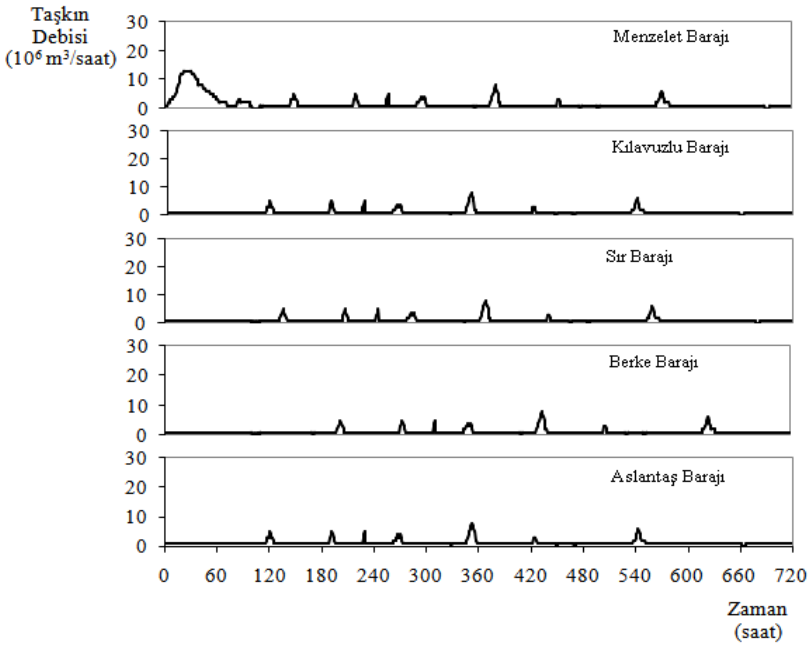
Şekil 7. Barajlara havzasından gelen aylık ortalama akımlar.



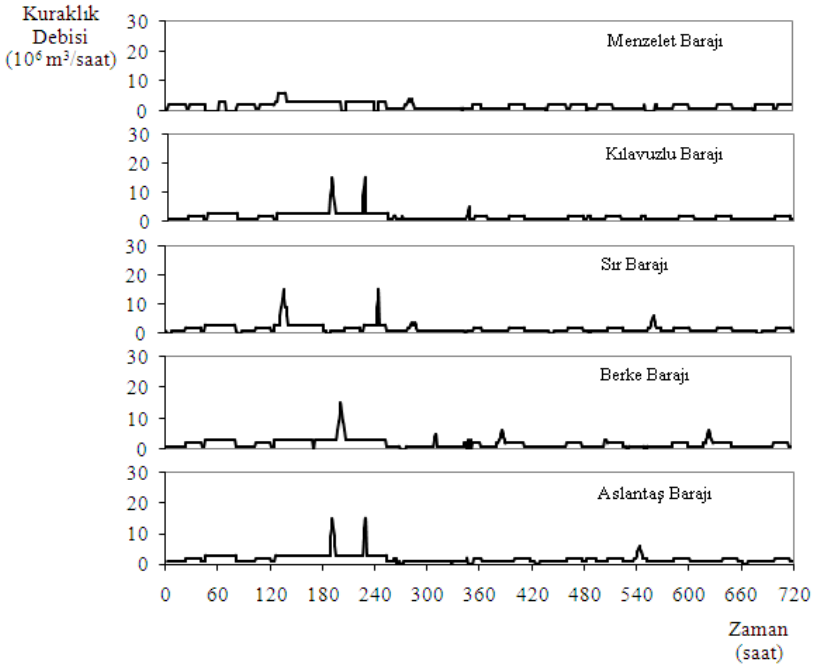
Şekil 8. Barajlara kurak dönemde havzasından gelen aylık akımlar.

Şekil 7'de kritik döneme ait aylık kurak akımları verilmektedir. Bu akım verileri uzun süreli optimal işletme modelinde kullanılarak güvenilir güç enbüyüklenirken aylık minimum işletme seviyeleri elde edilmektedir. Şekil 8' de aylık ortalama akımlar verilmiştir. Bu aylık ortalama akım verileri 27 yıllık akım verisinden elde edilmiştir. Bu veriler uzun süreli optimal işletme modelinde kullanılarak toplam enerji enbüyüklenirken

aylık normal işletme seviyeleri elde edilmiştir. Şekil 9 ve 10' da, barajlara taşkın/kuraklık olması beklenen Mayıs ayında havzasından gelen akım verileri verilmektedir. Bu veriler kullanılarak taşkın/kuraklık kontrolü amaçlı kısa süreli optimal işletme yapılmıştır.



Şekil 9. Barajlara taşkın olması beklenen Mayıs ayında havzasından gelen akımlar.



Şekil 10. Barajlara kuraklık olması beklenen Mayıs ayında havzasından gelen akımlar.

ELDE EDİLEN SONUÇLAR

Uzun ve kısa süreli optimal işletme modellerinden, Tablo 4'de verilen optimal işletme seviyeleri elde edilmektedir. Burada verilen optimal işletme seviyeleri kullanılarak barajların maksimum işletme seviyesi yenilenmekte ve Mayıs ayı için baraj kapasiteleri Tablo 5' de verildiği gibi elde edilmektedir. Bu işlem her bir ay için yapılarak elde edilen

maksimum işletme seviyelerinin en büyüğü baraj için maksimum işletme seviyesi olarak atanmalıdır.

Tablo 4. Barajlarda Mayıs ayı için elde edilen optimal işletme seviyeleri.

Barajlar	Menzelet	Kılavuzlu	Sır	Berke	Aslantaş
$S_{i,t}^{Min}$ (10^6 m^3)	1810	70	1120	430	1980
$S_{i,t}^{Nor}$ (10^6 m^3)	1850	80	1120	420	1990
$S_{i,t}^{Op,Taş}$ (10^6 m^3)	752	70	705	120	530
$S_{i,t}^{Op,Kur}$ (10^6 m^3)	1932	80	1110	430	1990
S_{i}^{Min}(10^6 m^3)	530	70	450	120	530
S_{i}^{Maks} (10^6 m^3)	1950	80	1120	430	1990

Tablo 5. Barajların Mayıs ayı için kapasitesinin belirlenmesi.

Barajlar		Menzelet	Kılavuzlu	Sır	Berke	Aslantaş
$S_{i,t}^{Op,Taş}$	(10^6 m^3)	752	70	705	120	530
$S_{i,t}^{Op,Kur}$	(10^6 m^3)	1932	80	1110	430	1990
$S_{i,t}^{Nor}$	(10^6 m^3)	1850	80	1120	420	1990
S_{i}^{Maks}	(10^6 m^3)	1950	80	1120	430	1990
S_{i}^{Min}	(10^6 m^3)	530	70	450	120	530
S_{Yeni}^{Maks}	(10^6 m^3)	2948	90	1535	720	3450
V_{Eski}	(10^6 m^3)	1420	10	670	310	1460
V_{Yeni}	(10^6 m^3)	2418	20	1085	600	2920

V: Kapasite

SONUÇLAR

Enerji üretiminin maksimize edilmesi amaçlı uzun süreli optimal işletme modeli ve taşkın/kuraklık kontrolü amaçlı olarak kısa süreli optimal işletme modelleri Ceyhan Havzasında bulunan çok barajlı bir su kaynakları sistemine başarılı bir şekilde uygulanmıştır. Yapılan işletme neticesinde elde edilen kapasiteler incelendiğinde, kapasite miktarlarındaki artışın taşkın kontrolünden dolayı kaynaklandığı görülmüştür. Bunun sebebinin Ceyhan Havzası'nın sulak bir havza olmasından dolayı olduğu söylenebilir.

Kuraklığın öne çıktığı havzalarda kuraklık kontrol hacminin kapasiteyi belirleyecek etken olacağı düşünülebilir.

TEŞEKKÜR

Mahmut Sert 1 Mayıs 2007 tarihinde vefat etmiştir. Bu tarihe kadar bu çalışmanın ortaya çıkartılması ve geliştirilmesi üzerine çok büyük katkılarda bulunmuştur. O olmadan bu çalışma asla yapılmazdı. Katkılarından dolayı kendisine çok teşekkür ederiz.

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DETERMINATION OF THE EFFECTS OF 1855 EARTHQUAKES ON MASONRY STRUCTURES IN BURSA: CASE STUDY BURSA ÖRDEKLI PUBLIC BATH

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Key Words: Historical monuments, structural analyses, 1855 Bursa Earthquakes

ABSTRACT Between the 14th and 15th centuries, the city of Bursa had major commercial activities. There were commercial routes to central Anatolia such as Silk Road, Spice Road. Because of the important location of the city many commercial, public, religious and educational buildings were built during this period due to the commercial activities between Anatolia and Europe. Bursa Ördekli Bath was also built at this period. An intense earthquake in 1855 damaged the city and many historical buildings were destroyed. Bursa Ördekli Bath is an example of these destroyed buildings. The authors of this paper want to investigate the characteristics of the materials used in the historical bath and find out the relationship between the 1855 earthquakes and the collapsing part of the men's frigidarium by structural analyses.

INTRODUCTION

The Ordekli Bath is dated to the end of 14th century in the city of Bursa, the first capital of the Ottoman Empire. Ördekli Bath began to be built in the period of Sultan Yıldırım Bâyezid I (1389–1402) and was completed in the 15th century by Çelebi Sultan Mehmet. The bath consisted of two separated parts which were for women and men. Both parts have same spaces which called soğukluk (frigidarium), ılıklık (tepidarium), sıcalık (caldarium), water reservoir and furnace.

The first earthquake in 31st January and the second in 18th April in 1855 were damaged the Bursa City and many historical buildings were destroyed. Many of the historical monuments fully or partly collapsed in this earthquakes. Bursa Ördekli Bath is known as an example of these partly destroyed buildings. 1600 people were died in this earthquakes in Bursa City. There were cracks and big cuts on the ground, water blow out under the ground and every where smells heavy sulphur gases.

In this study authors want to find out the characteristics of the materials used in the historical bath, prepare the finite element model of the Ördekli Bath and perform the structural analyses under dead load and earthquake loads. After the analyses the conclusions are discussed and evaluate the collapsing reasons of the men's frigidarium part.

THEORY and METHOD

The Ördekli Bath still preserves its authentic characteristics such as plan layout, mass order, spatial features of construction techniques and most of the original material. The building has a lengthwise rectangular plan -except men's frigidarium- extending along an axis in the east-west direction. Its exterior dimensions are 27m on the west, 42m on the North, 37 m on the South, 26 and 18 m oblique dimension on the east direction. At present, the central part of the men's frigidarium dome is collapsed (Fig.1).

According to the function and the different temperatures a bath, called''Hamam'' in Turkish, is composed of three main spaces- soyunmalık, ılıklık, sıcaklık and a water reservoir (Fig. 2).

The building was built with rubble stone, limestone horosan mortar, brick and timber in the masonry system. The exterior walls were 100–110 cm. in thickness, and interior walls were 100cm in thickness. Stone, in the form of rubble stone, was the major structural material in the walls. Limestone was found to be used on the sides of the door and window openings and at the corner of the walls. Brick, on the other hand, was generally used on the upper parts of the masonry such as arches, vaults and domes. The walls weren't plastered on the exterior, but plastered with Horasan on the interior. Horasan plaster was used as the covering material for the surfaces. The traces of horizontal timber beams in the walls point out the use of timber beams in the construction technique to distribute loads vertically. The vaults and domes, consisting of the superstructure, were built with brick and horasan mortar as binding material and their surfaces were plastered. On the horizontal terracotta pipes belonging to the water system were placed along the walls of the sıcaklık space.

Fig. 1. Perspective view of Ördekli Bath and the collapsed part of the Men's frigidarium part (Osmangazi Municipality, 2005).

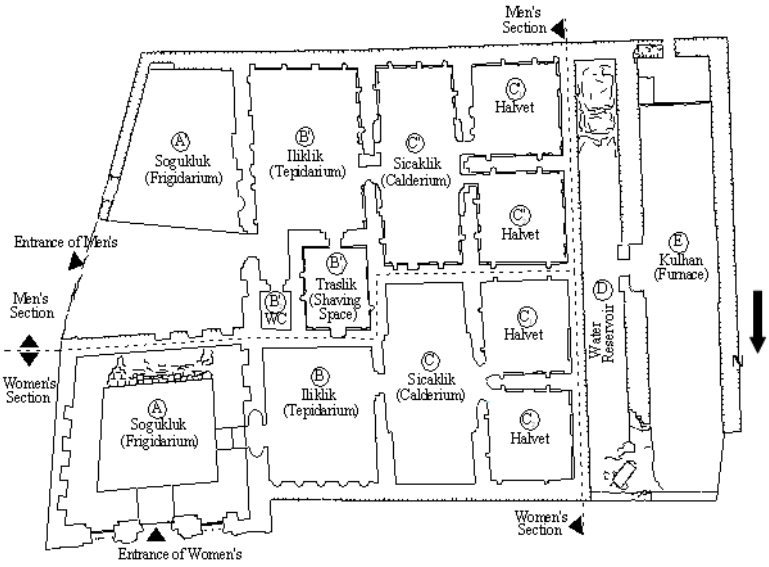

Fig. 2. Plan of the bath.

The men's frigidarium part was partly collapsed. Figure 3 and 4 shows us the collapsed part of the men's frigidarium. Except east side and half of the south side of the wall the other side of the walls are stable. Transition elements are partly stable but have big cracks and gaps. The main dome, half dome and the arch between them were fully collapsed.

Fig. 3. North side of the collapsed part of men's frigidarium.

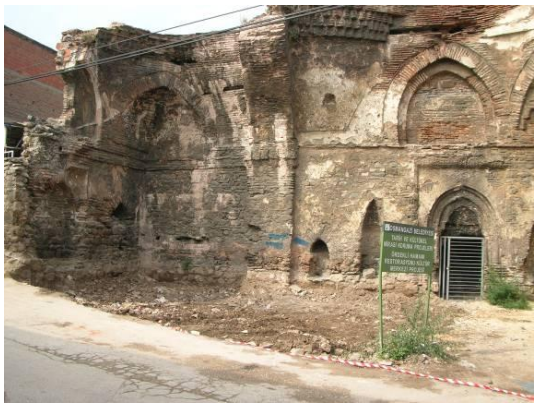

Fig. 4. South & West side walls, transition elements and the arch between the main and half dome.

EXPERIMENTAL STUDY

The experimental work for characterization of the materials used in the bath were physical, mechanical and chemical tests. Experimental works also helped us to understand the characterization differences between the stable and collapsed parts of the bath to set light to the reconstruction works. The order of the tests which were applied presented in Fig.1. Tests were done with the accordance of Turkish Standard Institute (TS 699) and the related American Standards (ASTM).

Physical, chemical and mechanical tests are carried out in İstanbul Technical University of Architectural Faculty in Building Science Laboratory. SEM and XRD tests are in the laboratory of TUBITAK Marmara Research Center Laboratories. Petrographical tests are in İstanbul University laboratories.

The tests were applied as follows:

Physical Tests:

Samples capillary coefficient (C, gr/m²√min.) Fig.-5, volume density (β, g/cm³), specific density (γ, g/cm³), porosity (p,%), mass of water absorption (Ks,%) and volumetric water absorption (Hs,%) were conducted.

Mechanical Tests:

After physical tests the same materials used for determination of mechanical properties of Ördekli Bath. The Module of Elasticity (E, N/mm²) Fig.-6, The Bending Strength (fb, N/mm²) Fig.-7 and the Compressive Strength (fc, N/mm²) Fig.-8 is determined.

Chemical Tests:

Samples for the ignition loss test (500mg) were placed in a porcelain crucible and weighed. The samples were heated in an oven at 105±5ºC, 550±5ºC and 1050±5ºC for 2.1 and 0.5 hours. After heating the samples were cooled in a desicator and weighed. The weight differences gave the percentage of moisture absorption, ignition loss at 550ºC and CaCO3 contents were calculated.

Acid loss and sieve analysis, samples were reacted with hydrochloric acid HCL (%10) and the siliceous aggregates was evaluated by sieve analysis.

In XRD and SEM-EDX Analysis micro structures, minerals, phases and physical aspects of the matrix of the samples were indicated and XRD tests were conducted to find the basic minerological composition of matrix.

Fig. 5. Capillary test samples of Ördekli Bath.

Fig. 6. The Module of Elasticity test.

Fig. 7. The Bending Strength test.

Fig. 8. The Compressive Strength test.

The calculated mechanical and physical properties for the Stone-Mortar Composite and the Brick-Mortar Composite shown in Table 1.

Table 1. Mechanical and physical properties of composite materials.

	Stone-Mortar Composite	Brick-Mortar Composite
Compressive Strength	16,692 N/mm²	4,69 N/mm²
Tensile Strength	1,67N/mm²	0,47N/mm²
Modulus of Elasticity	13,913 GPa	4,14 GPa
Shear Modulus	5,565 GPa	1,66 GPa
Poisson's ratio	0,17	0,17
Unit Weight	25 kN/m^3	15,75 kN/m^3

SEISMOLOGICAL PROPERTIES OF BURSA:

Bursa city is in the first degree Eathquake zone. Many earthquakes accured by the time but the most important earthquakes accured in 1855 as shown in figure 9.

The day of 31st of January in 1855 at 12:00 o'clock a forceful rain with thunder was occured in Bursa City. After this rain at 15:15 first eathquake accured and continued for 50 seconds. This earhquake also noticed in İnegöl, Kütahya, Afyonkarahisar, Akşehir, Gelibolu, Trakya, İstanbul cities.

The second earthquake occured in 1855 at April 18th. This earthquake damaged the city but the fire after the earthquake multiplied the hazards of the city. This day 01:10 o'clock at night an earthquake awake the people in their beds. After 2 hours from this earthquake at 03:00 o'clock the second earthquake which was 5-10 times bigger than the first earthquake was occured. It is said that the city was shaked like a ship under the storm and many people thought that the city sanked under the ground. Big panic occured during and after the earthquake and fire. 1600 people died in and around the city. In 15 hours 150 aftershock was occured. Many historical buildings Baths, Khans, mosques, churches, City Walls, Bazaars fully or partly collapsed. There were cracks and big cuts on the ground, water blow out under the ground and every where smells heavy sulphur gases and big rocks falling down Uludag Mountain. Wooden houses were not collapsed from the earthquake but the fire occured after the earthquake collapsed them. This earthquake noticed in Atina, Balıkesir, Yozgat and Rodos. Bursa Ördekli Bath is known as an example of these partly destroyed buildings.

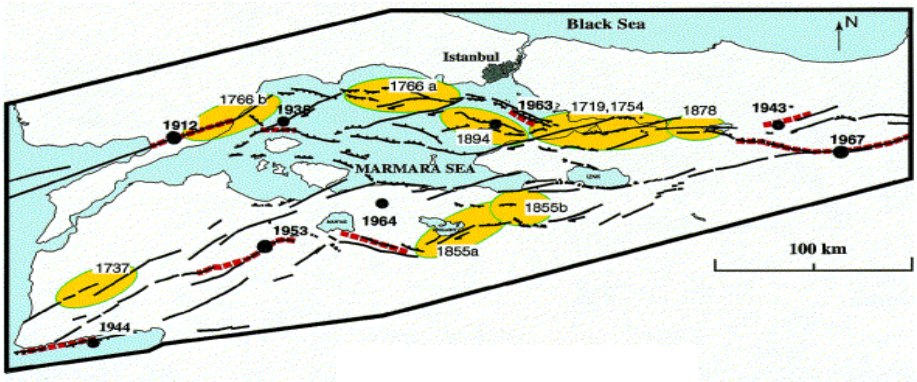

Fig. 9. Historical earthquakes in Marmara Region.

There were few photos came nowadays after the 1855 earthquakes, one of them is shown in Figure 10. On the left side of the photo we can see Pirinc Khan which was partly collapsed after the 1855 earthquakes.

Fig. 10. Bursa Pirinç Khan partly collapsed after 1855 Earthquakes.

STRUCTURAL ANALYSES:

In the analyses, the bath was modeled by using Algor programme. The solid element, in three dimensional and 8 noded-brick type is used for the analyses. The analytical model contains all elements such as squinches, pandantives, arches, vaults and light holes on the domes. Figure 11 shows the finite element model of the Bath.

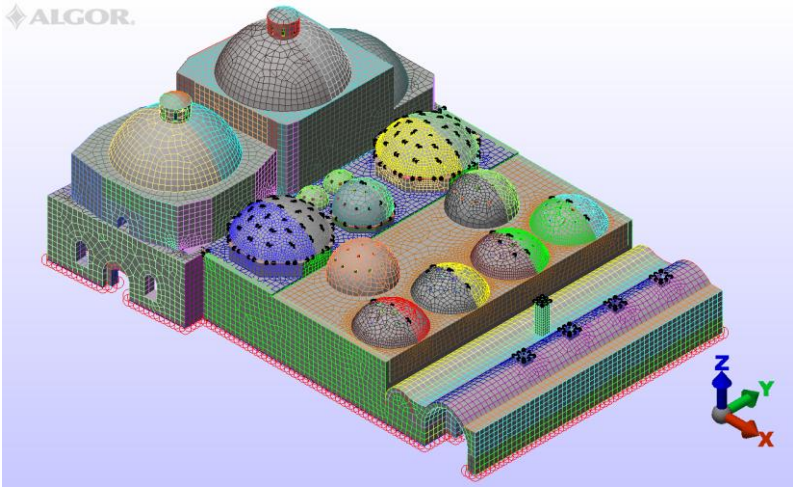

Fig. 11. The finite element modeling of the Bath.

In the finite element mesh 276845 solid element and the 134597 nodes are used. The details of three dimensional modeling are given in Fig. 12-15. The detail mesh is important for the accurate analyses results.

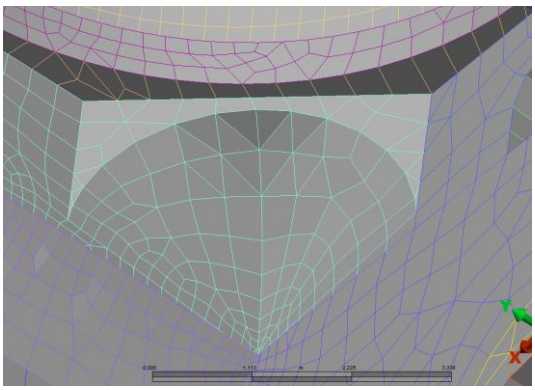

Fig. 12. The detail mesh of the squinch.

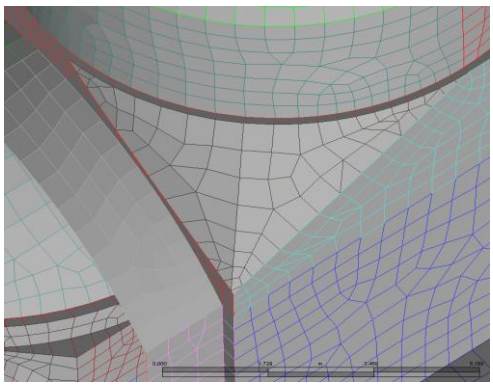

Fig. 13. The detail mesh of the pandantive.

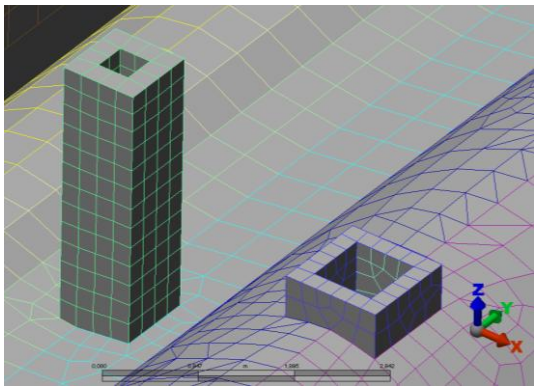

Fig. 14. The detail mesh of the chimneys.

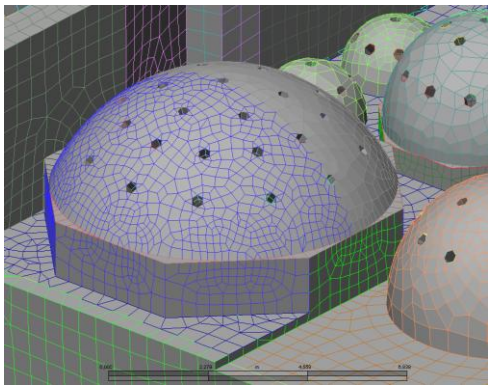

Fig. 15. The detail mesh of the light holes on domes.

-Self Weight Analyses:

In the self weight analyses the highest vertical displacement is observed at the men's frigidarium dome. The vertical displacement is occurred 0.8mm on the squinch which was between the main and the half dome (Fig. 16). The tensile stresses also occurred at the same location but this stress is under the limit stress value (Fig. 17). Compressive stress values are about 0.2 MPa which was low values for the Structure.

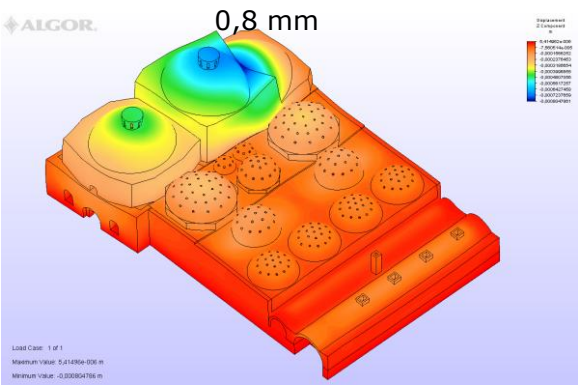

Fig. 16. Maximum vertical displacement at squinch.

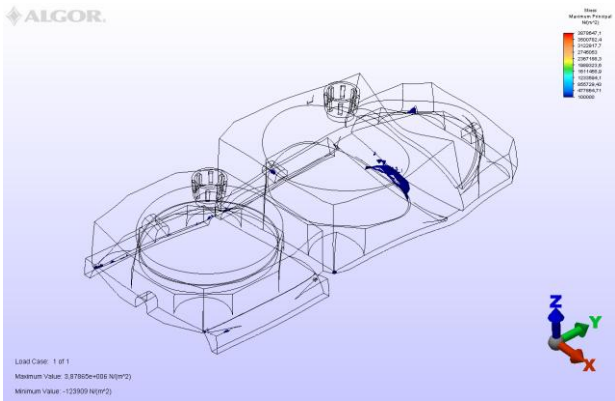

Fig. 17. Tensile stress is ~0,3 MPa –limit tensile stress 0,47MPa for brick composite-

-Earthquake Analyses:

The earthquake analysis was conducted using response spectrum approach. The Bath is located on earthquake Zone 1, the most seismically risky regions in Turkey. In determining the earthquake loads, "Turkish Earthquake Code,2007, Section 5" is used. Ground spectral accelaration coefficient was taken 0,4. The soil conditions were determined siltly clay material which equal to ground type Z2 and the characteristic spectrum periods were Ta=0.15sec. and Tb=0.40sec.

The first four mode shapes obtained from the modal analyses two bending modes followed by torsional modes are presented here in Figure 18-21. For the earthquake analyses the first 40 modes were used.

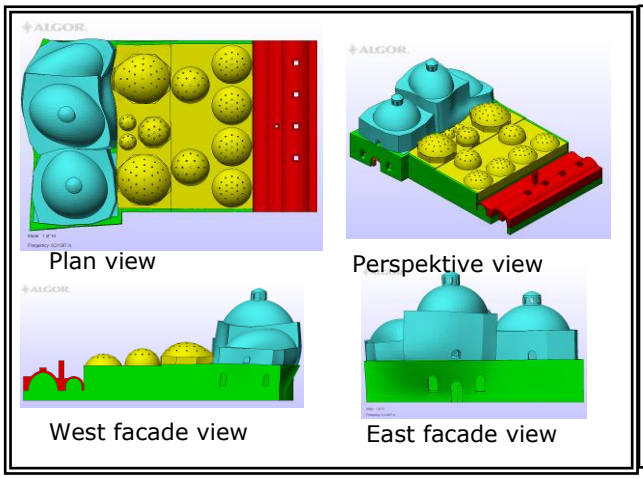

Fig. 18. The 1st mode of Bath -9,21 Hz.-

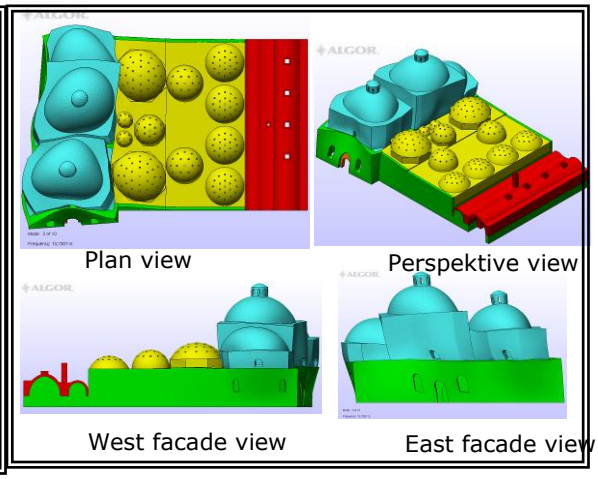

Fig. 19. The 2nd mode of Bath -13,73 Hz.-

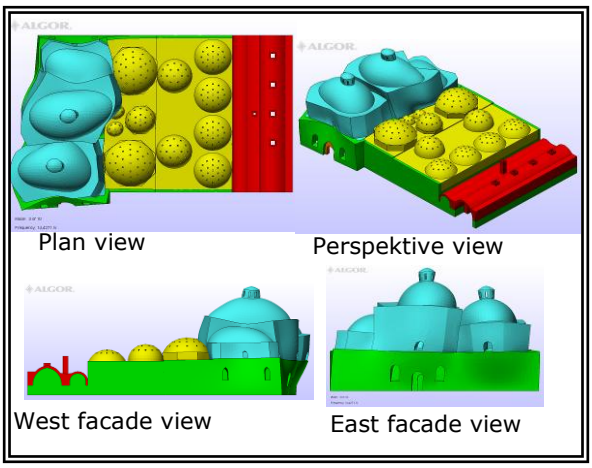

Fig. 20. The 3rd mode of Bath -14,43 Hz.-

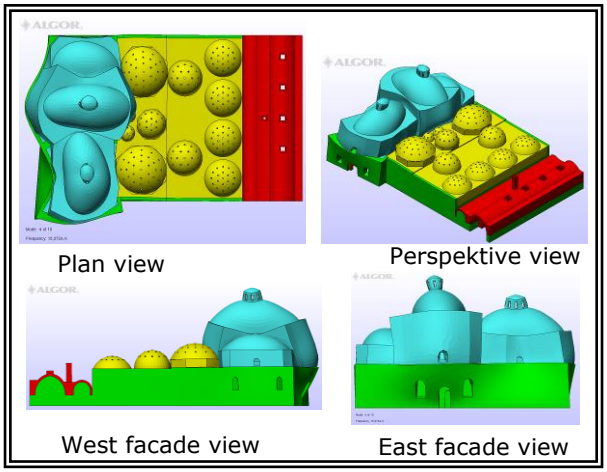

Fig. 21. The 4th mode of Bath -15,37 Hz.-

The earthquake response spectrum analyses show that the tensile stresses occurred at walls are in the limit values for the stone composite. The highest stresses are formed in x direction earthquake analyses at east walls of men's frigidarium (Fig. 22). For the brick composite, the tensile stresses are higher than the limit values only at men's frigidarium part. Especially earthquake analyses performed in x direction, maximum tensile stresses are formed at the big area of the half dome, pandantives and the beginnings of the arch which carried the half and the main dome (Fig 23).

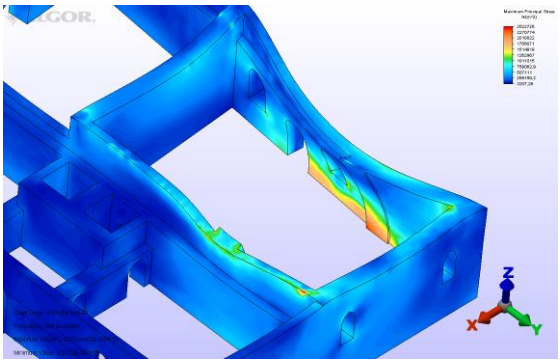

Fig. 22. Earthquake analyses in x direction **-**The tensile stresses lover than the limit tensile stress for stone composite (Max. value occurred at south wall of the men's frigidarium wall)-

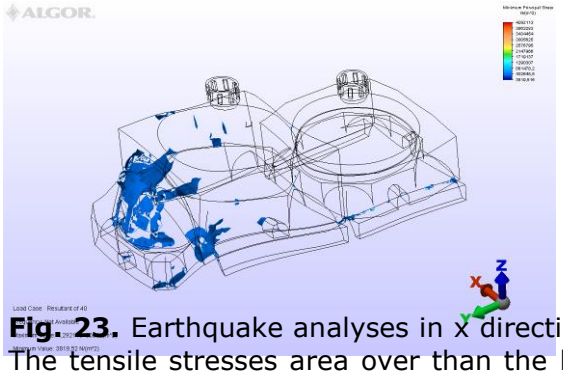

Fig. 23. Earthquake analyses in x direction **-** The tensile stresses area over than the limit tensile stress for brick composite (blue color areas)-

Earthquake analyses performed in y direction, tensile stresses at walls occured at men's frigidarium walls. Maximum tensile stresses are at door, window corners and beginning af the arches under the limit values (Fig. 24). Tensile stresses are occured over limit values at men's and women's frigidarium part only little local places of pandantives (Fig. 25).

In the z direction Earthquake analyses, the maximum tensile stresses at walls are occured at the east direction of the men's frigidarium wall (Fig. 26) and the tensile stresses are over than the limit values at men's frigidarium pandantive beginings (Fig. 27).

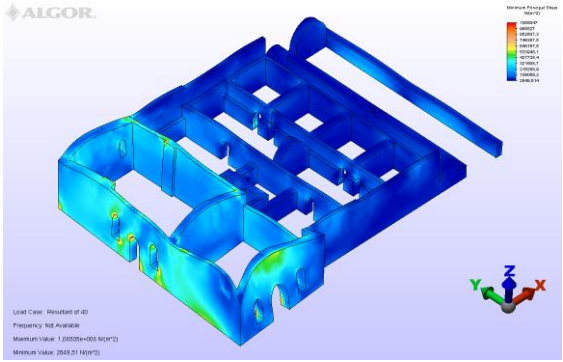

Fig. 24. Earthquake analyses in y direction **-**The tensile stresses lover than the limit tensile stress for stone composite (Max. value occurred at east walls of the frigidarium parts)-

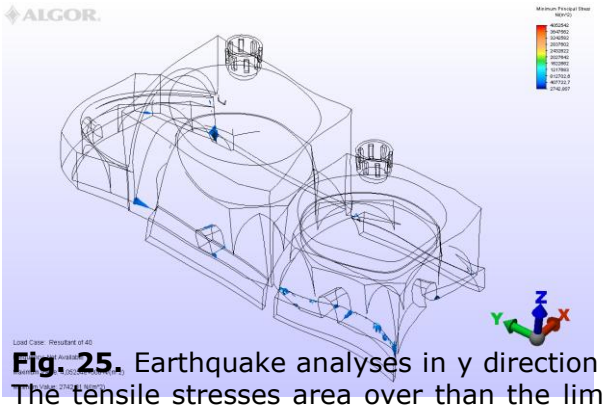

Fig. 25. Earthquake analyses in y direction **-** The tensile stresses area over than the limit tensile stress for brick composite (blue color areas)-

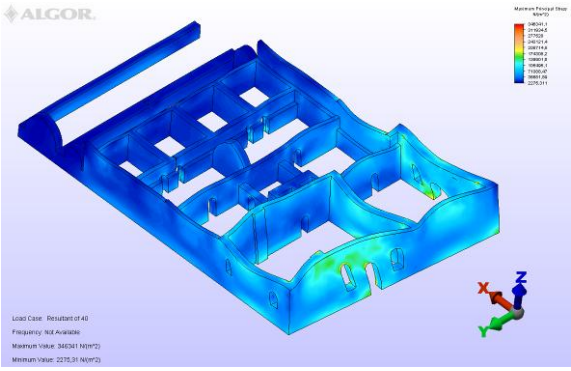

Fig. 26. Earthquake analyses in z direction **-**The tensile stresses lover than the limit tensile stress for stone composite (Max. value occurred at east and north walls of the frigidarium parts)-

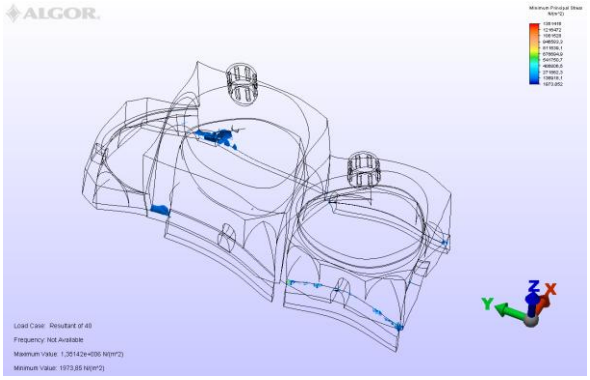

Fig. 27. Earthquake analyses in z direction **-** The tensile stresses area over than the limit tensile stress for brick composite (blue color areas)-

CONCLUSIONS

The intensive earthquakes are the most important dangers for the historical monuments. Because of that reason we have to analyse the historical monuments before restoration works. In this case study analyses results shows that:

1- The collapsed part of the Ördekli Bath and the principal stresses occured in earthquake analyses at X-X (East-West) direction results are closed to each other.

2- The stresses which was over than the limit stresses (brick composite 0,47MPa) occured in men's frigidarium arches, pandantives and the half dome parts. In this locations there were many cracks seen in-situ.

3- Two big earthquakes in 1855; one was occured in the East side of Bursa, the other was in West side of Bursa and also several aftershocks in this directions gave big hazardous to the men's frigidarium part. Authors of this paper estimated that the cracks and metarial decays began with the first earthquake especially gave big hazards to the half dome and the arch between the half dome and the main dome. With the other big earthquake and aftershocks in this direction made the cracks and material decays are getting bigger, limit stresses are being exceeded and finally collapse the men's frigidarium part.

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ÇOK BARAJLI SİSTEMDE OPTİMAL İŞLETME SEVİYELERİNİN BELİRLENMESİ: CEYHAN HAVZASI, TÜRKİYE

DETERMINATION OF THE OPTIMAL OPERATIONAL LEVELS IN MULTIPLE RESERVOIRS SYSTEM: THE CEYHAN BASIN, TURKEY

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Anahtar Kelimeler: Çok Barajlı Sistem, Enerji Üretimi, Taşkın/Kuraklık Kontrolü, Optimal Seviye, Ardışık Yaklaştırmalı Dinamik Programlama.

ÖZ Bu çalışmada, çok amaçlı ve çok barajlı bir su kaynakları sistemi tanımlanmıştır. Bu sistem kullanılarak enerji üretimi amaçlı uzun süreli ve taşkın/kuraklık kontrolü amaçlı kısa süreli optimal işletme modelleri geliştirilmiştir. Bu modellerde ardışık yaklaştırmalı dinamik programlama optimizasyon tekniği kullanılmıştır. Kısa süreli optimal işletmede, amaç fonksiyonu, ilave taşkın/kuraklık kontrol hacmine ve mansaptaki debi kontrol kısıdına bağlı olarak üç aşamalı olup barajdan bırakılan akımların diğer barajın aksına taşınması için kinematik dalga yaklaşımı kullanılmıştır. Aylık akımların kullanıldığı uzun süreli optimal işletmede amaç fonksiyonu güvenilir ve toplam enerjinin enbüyüklenmesi şeklindedir. Geliştirilen optimal işletme modelleri, Ceyhan Havzası'nda Ceyhan Nehri ana kolu üzerinde bulunan çok barajlı bir su kaynakları sistemine uygulanmıştır. Buradan elde edilen optimal işletme seviyeleri, taşkın/kuraklık kontrolü ile enerji üretimi bakımından değerlendirilmiştir.

GİRİŞ

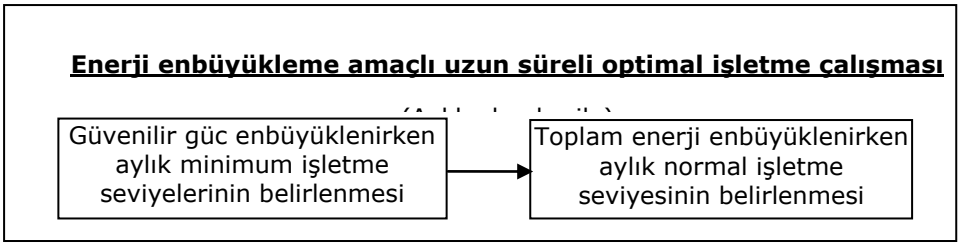
Su kaynakları planlaması, ülke ekonomisi ve öngörülen hayat standartları ile bütünleşmiş, çok boyutlu ve karmaşık bir süreçtir. Su kaynakları planlaması ve yönetimi, istenilen amaçlar doğrultusunda ve öngörülen kriterler çerçevesinde su kaynaklarının en verimli kullanımını sağlayacak faaliyetlerin tümünü kapsamalıdır. Çok barajlı sistemlerde optimal işletme politikalarının belirlenmesi için genelde uzun süreli planlama kapsamında enerji üretimi ve kısa süreli planlama kapsamında ise kuraklık ve taşkın kontrolü göz önüne alınmaktadır.

Çok barajlı sistemlerde enerji optimizasyonu, taşkın kontrolü ve sulama gibi amaçları gerçekleştirmek için çeşitli yöntemlerin kullanıldığı birçok çalışma bulunmaktadır, (Labadie, 2004).

Enerji optimizasyonu ile ilgili çalışmalarda, enerji üretim fonksiyonunun doğrusal olmaması bakımından, genellikle, dinamik programlama optimizasyon tekniği ve simülasyon kullanılmıştır, (Güvel, 1997, Sert, 1982, Sert, Öcal, Oktay, Ertuğrul, 1983, Yakowitz, 1982, Yeh, 1985, Yurtal, 1993). Taşkın kontrolü ve sulama gibi amaçlar için ise, su miktarı ve finansal açıdan simülasyon, doğrusal ve doğrusal olmayan programlama optimizasyon teknikleri ile yapılmış çalışmalar vardır, [4]. Uzun süreli planlamada aylık kuraklık verileri kullanılarak güvenilir enerjiyi enbüyüklemeye yönelik çalışmalar yapılmış olmasına rağmen (Sert, 1982, Sert, Öcal, Oktay, Ertuğrul, 1983, Yurtal, 1993), saatlik olarak kuraklık kontrolü ve yönetimi amaçlı olarak yapılmış kısa süreli optimizasyon çalışmaları ilgili araştırmalara ulaşılamamıştır.

Çok barajlı sistemler için aylık ortalama akımlar kullanılarak yapılan uzun süreli işletme neticesinde normal işletme seviyeleri elde edilmektedir. Taşkın olması beklenen aylarda

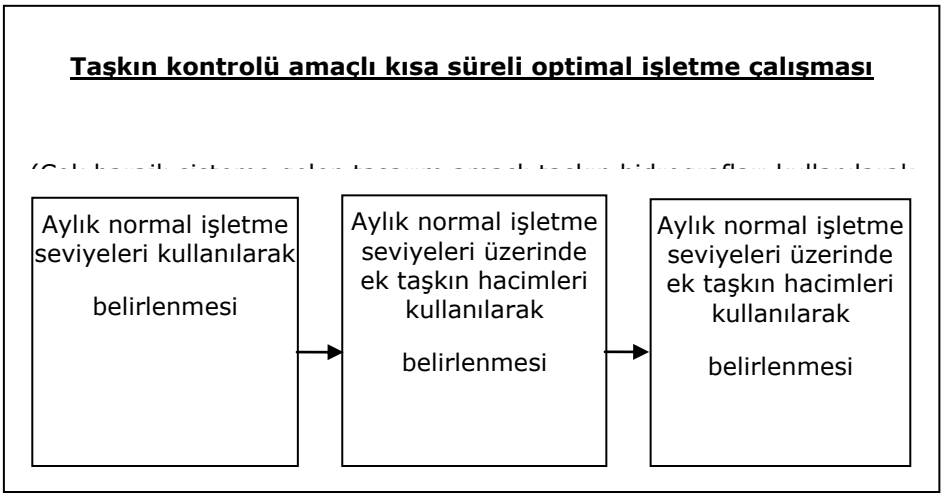
sistemdeki barajlarda normal işletme seviyelerinden bir miktar (taşkın kontrolü için gerekli hacim) inilerek taşkın akımları karşılanmaktadır. Bunun için taşkın olacak aylarda optimal taşkın kontrol seviyesi belirlenerek, bu aylarda normal işletme seviyesinden optimal taşkın kontrol seviyesine inilerek işletme yapılması gerekmektedir. Böylece taşkın olduğu zaman yapılan kısa süreli işletme neticesinde bir sonraki ayda normal işletme seviyesine çıkılabilsin. Benzer şekilde kısa süreli kuraklık olması beklenen aylarda sistemdeki barajlarda normal işletme seviyelerinin bir miktar (kısa süreli kuraklık kontrolü için gerekli olan hacim) üzerine çıkılarak, gelen akımların karşılanması gerekmektedir. Bunun için optimal kuraklık kontrol miktarı belirlenerek, buna göre kurak ay için yapılacak işletme neticesinde, bir sonraki ayda normal işletme seviyesine ulaşılabilmektedir. Yani, çok barajlı sisteme ait herhangi bir barajda göz önüne alınan ay için taşkın veya kısa süreli kuraklık söz konusu değilse, işletme seviyesi normal işletme seviyesinde olmalı; taşkın söz konusu ise, normal işletme seviyesinde optimal taşkın kontrol seviyesine inilmeli; kısa süreli kuraklık söz konusu ise, normal işletme seviyesinde optimal kuraklık kontrol seviyesine çıkılmalıdır ki, bir sonraki ayın normal işletme seviyesine ulaşılabilsin. Bu çalışmada yukarıda belirtilen işlemleri gerçekleştirebilmek için, kısa süreli kuraklık durumu, taşkın gibi kısa süreli işletmede ele alınması ve bunun neticesinde optimal kuraklık kontrol seviyelerinin elde edilmesi gerekmektedir.



Şekil 1. Enerji enbüyükleme amaçlı uzun süreli optimal işletme çalışması süreci.

Bu çalışmada, uzun süreli planlama için kurulan optimizasyon modelinde, kritik dönemin (kurak dönem) aylık akımları kullanılarak güvenilir güç enbüyüklenmekte ve aylık minimum işletme seviyeleri elde edilmektedir. Buradan enbüyüklenen güvenilir güç, aynı modelde kısıt olarak kullanılıp, aylık ortalama akımlar ile toplam enerji enbüyüklenmekte ve aylık normal işletme seviyeleri belirlenmektedir. Şekil 1' de, enerji enbüyükleme amaçlı uzun süreli optimal işletme çalışması süreci özetlenmiştir.

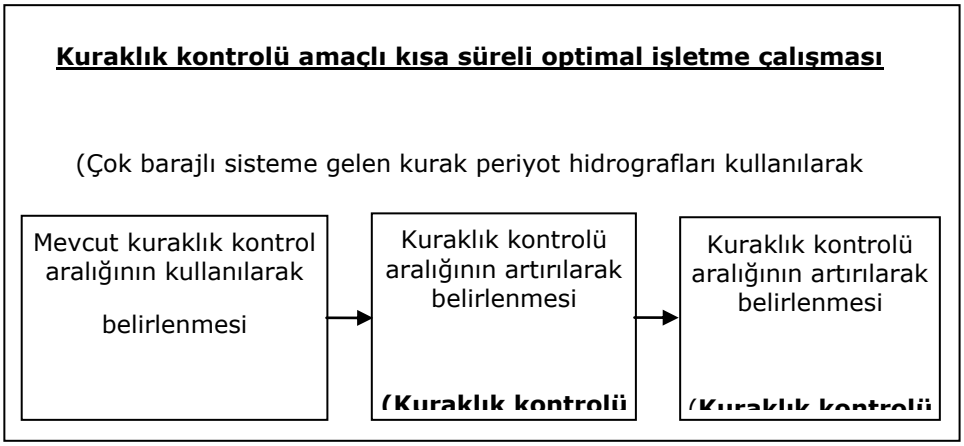
Uzun süreli planlamada enerji üretimi için aylık akımlar bakımından en kritik dönem (yıl veya yıllar arası olarak) belirlenip, bu döneme ait aylık akımlar kullanılarak, optimal enerji üretimi bakımından yapılan işletme sonucu barajlarda minimum işletme seviyeleri ve sisteme ait güvenilir güç belirlenmektedir. Yılın herhangi bir ayında beklenen akımdan fazla bir akımın gelmesi, o ay için taşkın kontrolü yapılmasını gerektirebilir. Benzer şekilde, beklenen akımdan az bir akımın gelmesi de, o ay için kısa süreli kuraklık kontrolü yapılması anlamına gelecektir. Kuraklık (düşük akımların takviyesi) veya taşkın kontrolü gibi amaçlar, tekerrür aralığı bakımından uzun süreli planlamanın parçası olarak görülse de, bu olayların sistem tarafından kontrol edilmesi ve yönetilmesi, kısa süreli (çoğunlukla saatlik veya günlük) bir işletme optimizasyonunda ele alınmak zorundadır (Opan, 2007).



Şekil 2. Taşkın kontrolü amaçlı kısa süreli optimal işletme süreci.

Taşkın kontrolü amaçlı kısa süreli planlama için kurulan optimizasyon modelinde, barajlara havzasından gelen tasarım amaçlı taşkın hidrografı akım verileri ile, taşkın kontrolü debi kısıtı ve ek taşkın kontrol hacmi kullanılması durumuna göre barajdan bırakılan akımlar enküçüklenmekte ve aynı zamanda sistemden beklenen optimal faydaya (enerji üretimi v.s.) ulaşılmaya çalışılmaktadır. Burada, kullanılan ek taşkın kontrol hacimleri, işletme sonucu elde edilen işletme seviyelerinden düşülerek taşkın kontrol seviyeleri belirlenmektedir. Herhangi bir baraj için elde edilen bu taşkın kontrol seviyelerinden en küçük olanı optimal taşkın kontrol seviyesi olarak seçilmektedir. Şekil 2'de, taşkın kontrolü amaçlı kısa süreli optimal işletme süreci gösterilmektedir.

Kuraklık kontrolü amaçlı kısa süreli planlama için kurulan optimizasyon modelinde, belirli bir ay için girilen kuraklık hidrografının sistem tarafından depolanıp barajdan bırakılan enbüyük akımın enküçüklenmesi amaç fonksiyonu olarak tanımlanırken, barajdan toplam debi çıkışının (enerji üretimi için ve savaklanan) mansap için kuraklık ve kirlilik kontrolü debisinin altında olması kısıtı konulmaktadır. Burada, kullanılan ek kuraklık kontrol hacmi, elde edilen işletme seviyelerinden düşülerek kuraklık kontrol seviyeleri belirlenmektedir. Bu seviyelerdeki en büyük işletme değeri optimal kuraklık kontrol seviyesini göstermektedir. Şekil 3'de, kuraklık kontrolü amaçlı kısa süreli optimal işletme süreci verilmektedir.



Şekil 3. Kuraklık kontrolü amaçlı kısa süreli optimal işletme süreci.

Kısa süreli işletme optimizasyonunda zaman dilimi olarak saat (veya özel durumlarda gün) kullanıldığı için akımların akarsu yatağında ötelenerek baraj aksına taşınmasında

kullanılacak bir yaklaşıma gerek vardır. Akarsu yatağında akım öteleme üzerine kinematik dalga, dinamik dalga ve Muskingam gibi yaklaşımlar bulunmaktadır. Bu çalışmada, akımların baraj aksına taşınmasında kinematik dalga yaklaşımı kullanılmaktadır. Kinematik dalga yaklaşımında, St. Venant denklemlerindeki ivme ve basınç terimleri ihmal edildiğinden, dalga hareketi sadece süreklilik denklemi ile tanımlanmaktadır. Kinematik dalga yaklaşımı için çeşitli çalışmalar yapılmıştır. Kinematik ve difüzyon yaklaşımlarının uygulanabilirliğini gösteren çalışmalar bulunmaktadır, Ponce, Li, Simons, 1978).

Yüzeysel sulama için kinematik dalga yaklaşımı ile ilgili öneri niteliğinde çeşitli araştırmalar yapılmıştır, (Sherman, Singh, 1978, Sherman, Singh, 1982). HEC–1 benzetim programı içerisinde kinematik dalga öteleme tekniğini tanımlanmış ve uygulanmıştır, (DeVrices, MacArthur, 1979).

SİSTEMİN MATEMATİKSEL MODELİ

Bir akarsu üzerinde bulunan birçok sayıda baraj ve hidroelektrik santralden (HES) oluşan bir su kaynakları sistemi çok amaçlı ve çok barajlı olarak tanımlanabilir. Şekil 4' de, çok barajlı bir su kaynakları sistemi ve bu sistem içinde bulunan herhangi bir i-barajının t-zamanına ait işletme ile ilgili değişkenleri gösterilmiştir. Burada, i=l,2…M; Baraj sayısını, t=1,2…KM; Ay gün veya saat olarak zamanı göstermektedir. Bu sisteme ait herhangi bir i-barajı için t-zamandaki su dengesi ilişkisi;

$$\frac{dS}{dt} = X_{i,t} - Y_{i,t} \tag{1}$$

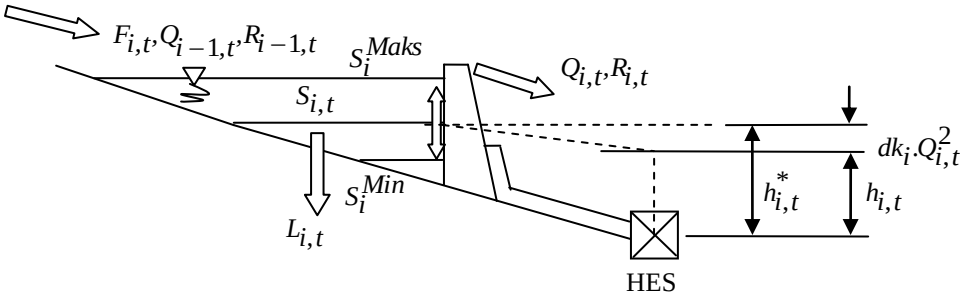
şeklinde yazılabilir. Burada,

$X_{i,t}$:i-barajına t-zamanda giren akımları,

$Y_{i,t}$:i-barajından t-zamanda çıkan akımları,

$\frac{dS}{dt}$:i-barajında depolanan su miktarının zamanla değişimi

olarak tanımlanmıştır.



Şekil 4. Çok barajlı bir su kaynakları sisteminde herhangi bir i-barajına ait t-zamandaki işletme ile ilgili değişkenler.

Su dengesi ilişkisini ayrık zaman dilimi için aşağıdaki gibi,

$$\frac{dS}{dt} = \frac{\Delta S}{\Delta t} \tag{2}$$

gösterildiğinde, buradan,

$$\frac{\Delta S_i}{\Delta t} = X_{i,t} - Y_{i,t} \tag{3}$$

olarak yazılabilir. Buradan,

$$X_{i,t} = F_{i,t} + Q_{i-1,t} + R_{i-1,t} \tag{4}$$

$$Y_{i,t} = Q_{i,t} + R_{i,t} + L_{i,t} \tag{5}$$

olarak gösterilirse, burada,

$S_{i,t}$: i- barajında t-zamanda depolanan su miktarı

$F_{i,t}$: i-barajının havzasından t-zamanda gelen akım miktarı

$Q_{i,t}$: i-barajından t-zamanda enerji üretimi için bırakılan akım miktarı

$R_{i,t}$: i-barajında t-zamanda dolu savaktan bırakılan akım miktarı

$L_{i,t}$: i-barajında t-zamana ait buharlaşma ve diğer kayıplar

olarak tanımlanabilir. Eğer $\Delta t = 1$ birim saat olarak alındığında, i-barajında t-zamandaki su dengesi ilişkisi,

$$\Delta S_i = S_{i,t+1} - S_{i,t} \tag{6}$$

ile buradan,

$$S_{i,t+1} - S_{i,t} = F_{i,t} + Q_{i-1,t} + R_{i-1,t} - Q_{i,t} - R_{i,t} - L_{i,t} \tag{7}$$

şeklinde ifade edilebilir.

Çok amaçlı çok barajlı bir su kaynakları sisteminde her bir baraj için depolanan su miktarları, baraj maksimum ve minimum hacimleri ile sınırlanmış olmaktadır. Buna göre her bir barajda depolanmış su miktarı

$$S_i^{Min} \le S_{i,t} \le S_i^{Maks} \tag{8}$$

arasında olmakta ve,

S_i^{Min} : i-barajında depolanan minimum su miktarını

S_i^{Maks} : i-barajında depolanan maksimum su miktarını

göstermektedir. Barajdan bırakılacak akımlar enerji üretim kapasitesine ve dolu savak kapasitesine bağlı olarak sınırlanabilir. Buna göre, barajdan bırakılan akımlar

$$0 \le Q_{i,t} \le Q_i^{Maks} \tag{9}$$

$$0 \le R_{i,t} \le R_i^{Maks} \tag{10}$$

olmaktadır. Burada,

Q_i^{Maks} : i-barajında enerji üretimi için bırakılabilecek maksimum su miktarını

R_i^{Maks} : i-barajında dolu savaktan bırakılabilecek maksimum su miktarını

göstermektedir.

Diğer taraftan, barajlardan bırakılan akımların toplamına alt ve üst sınırlar tanımlanabilir. O zaman,

$$W_{i,t} \le (Q_{i,t} + R_{i,t}) \le W_i^{Maks} \tag{11}$$

arasında olup, burada,

$W_{i,t}$:i-barajında t-ayında akarsu yatağına bırakılması gereken kuraklık ve kirlilik kontrolü için minimum su miktarı

W_i^{Maks} :i-barajında akarsu yatağına bırakılabilecek taşkın kontrolü için maksimum su miktarı

olarak tanımlanmaktadır.

Her bir barajda enerji üretimi için bırakılan akımdan elde edilen güç hidroelektrik santral kurulu gücünü aşmaması gerekmektedir. Yani, t-zamanda i-barajında elde edilen ortalama güç

$$P_{i,t} = k_i.Q_{i,t}.h_{i,t} \tag{12}$$

olup, $P_{i,t} \le P_{k_i}$, olmalıdır. Burada, P_{k_i} , i-barajı için kurulu güç, k_i enerji üretim katsayısı ve $h_{i,t}$ i-barajında t-zamandaki ortalama net düşüdür.

Barajlardaki ortalama su yüksekliği,

$$h_{i,t}^* = h\left(\frac{S_{i,t} + S_{i,t+1}}{2}\right) \tag{13}$$

şeklinde (ortalama depolanmış su miktarının bir fonksiyonu olarak) elde edilmektedir. Cebri boru veya taşıma tünelindeki sürtünme kayıpları dikkate alındığında,

$$h_{i,t} = h_{i,t}^* - (h_f)_{i,t} \tag{14}$$

olmakta ve buradan

$$h_{i,t} = h_{i,t}^* - dk_i.Q_{i,t}^2.l_i \tag{15}$$

yazılmaktadır. Burada,

$h_{i,t}^*$: Tüneldeki sürtünme kaybı düşünülmeden önce i-barajında t-zamandaki ortalama su yüksekliği

$(h_f)_{i,t}$: i-barajında t-zamandaki sürtünmeden dolayı yük kaybı

dk_i : i-barajında tünel çapına ve sürtünme özelliklerine bağlı bir katsayı

l_i : i-barajında taşıma tüneli uzunluğu

olarak tanımlanmaktadır. Burada, barajdaki $h_{i,t}$ yüksekliğine karşılık gelen depolama yüzey alanı belirlenip, bu değer t-zamandaki buharlaşma yüksekliği ile çarpılarak buharlaşma miktarı belirlenmektedir.

ENERJİ ÜRETİMİ AMAÇLI UZUN SÜRELİ PLANLAMA İÇİN OPTİMİZASYON MODELİ

Çok amaçlı çok barajlı su kaynakları sisteminde uzun süreli planlama yaklaşımında, zaman adımları olarak aylar kullanılmaktadır. Sistemin verilen boyutları için, ardışık yaklaştırmalı dinamik programlama (DPSA) modeli ile aylık kurak dönem akımları kullanılarak güvenilir gücü enbüyükleyecek şekilde aylık minimum işletme seviyeleri($S_{i,t}^{Min}$) belirlenmektedir. Buradan elde edilen güvenilir güç, modelde kısıt olarak kullanılıp, aylık ortalama akımlar ile toplam enerji enbüyüklenmekte ve aylık normal işletme seviyeleri ($S_{i,t}^{Nor}$) bulunmaktadır. Bunun için, modelde, amaç fonksiyonu, iki kriterli olarak ele alınmaktadır

1. Kurak dönemin aylık akımları ile güvenilir gücün enbüyüklenmesi,

$$Maks(Min \sum_{i=1}^{M} P_{i,t}) \tag{16}$$

Burada, t=1,2,3…KM, KM: Kurak dönem ay sayısını ifade etmektedir.

2. Aylık ortalama akımlarla toplam enerjinin enbüyüklennıesi, (P_G =sabit),

$$Maks \sum_{t=1}^{KM} \left[(\sum_{i=1}^{M} P_{i,t} - P_G).p_S + P_G.p_G \right] \tag{17}$$

($Max \sum_{t=1}^{KM} \sum_{i=1}^{M} P_{i,t}$ ile eşdeğer)

$$\sum_{i=1}^{M} P_{i,t} \geq P_G \tag{18}$$

şeklindedir. Burada,

P_G :Eniyilenmiş güvenilir gücü,

p_G :Güvenilir enerji birim fiyatını,

p_S :Sekonder enerji birim fiyatını,

göstermektedir. Güvenilir güç (P_G), kurak dönemin aylık akımları kullanılarak belirlenmekte ve aylık ortalama akımlara göre sekonder enerjinin enbüyüklenmesi ile toplam enerjinin enbüyüklenmesi sağlanmış olmaktadır. Burada kullanılan enerji birim fiyatlarının sonuç üzerinde bir etkisi olmamakla birlikte, sadece toplam enerji faydasının değerini belirlemek için kullanılmaktadır.

Yapısal denklemler olarak her bir baraj ve dönem için su dengesi ilişkisi, Denklem 5' de ifade edilmektedir.

Sistemdeki kısıtlar ise, sisteme ait her bir barajdaki işletme seviyesinin değişimi, Denklem 6 ile gösterilmektedir. Barajdan bırakılacak akım miktarları, enerji üretimi ve

dolu savaktan bırakılan akımlar, sırasıyla, Denklem 7 ve 8' de gösterildiği gibi tanımlanmaktadır.

TAŞKIN KONTROLÜ AMAÇLI KISA SÜRELİ PLANLAMA İÇİN OPTİMİZASYON MODELİ

Kısa süreli planlama için işletme optimizasyonunda, ardışık yaklaştırmalı dinamik programlama (DPSA) tekniği kullanılmaktadır. Barajlara havzasından gelen tasarım amaçlı taşkın hidrografı akım verileri ile, taşkın emniyeti debi kısıtı ve ek taşkın kontrol hacmi kullanılması durumuna göre barajdan bırakılan akımların pik değeri enküçüklenmekte ve aynı zamanda sistemden beklenen optimal faydaya (enerji üretimi v.s.) ulaşılmaya çalışılmaktadır. Burada, kullanılan ek taşkın kontrol hacimleri, işletme sonucu elde edilen işletme seviyelerinden düşülerek taşkın kontrol seviyeleri belirlenmektedir. Elde edilen taşkın kontrol seviyelerinde her bir baraj için en küçük olanı, söz konusu ay için optimal taşkın kontrol seviyesi olarak atanmaktadır.

Amaç fonksiyonu, taşkın durumunda barajdan bırakılan toplam su miktarının kabul edilen işletme süresi içinde en büyük değerinin enküçüklenmesi şeklinde seçilmektedir. Yani, amaç fonksiyonu,

$$Min[Maks((Q_{i,f} + R_{i,f})_t)] \tag{19}$$

şeklinde yazılabilir. Burada, f=1,2,3…TM: Taşkın hidrografı için kısa süreli işletme süresi içindeki zaman dilimidir.

Taşkın kontrolü amaçlı kısa süreli planlama için işletme optimizasyonu modelinde izlenen adımlar üç aşamalı olarak aşağıdaki gibidir:

1. Aşama: Çok barajlı sistemde uzun süreli optimizasyon sonucu belirlenen aylık normal işletme seviyeleri, minimum ve maksimum işletme seviyeleri (S_i^{Min} , S_i^{Maks}) arasında değişmektedir. Taşkın durumunda, çok barajlı sisteme gelen tasarım amaçlı taşkın hidrografları, aylık normal işletme seviyelerinde ($S_{i,t}^{Nor}$) karşılanmakta ve taşkın kontrolü için mansapta debi kısıtı ile barajdan bırakılan akımın pik değerinin enküçüklenmesine çalışılmaktadır. Başlangıçta, (f=1 için) taşkın karşılama seviyesi $(S_{i,1}^{Taş})_t = S_{i,t}^{Nor}$ olarak alınmakta ve tasarım amaçlı taşkın hidrografı akım verileri kullanılarak elde edilen optimal çözümde $(S_{i,f}^{Taş})_t$ değerlerinden en küçük olanı, söz konusu ay için optimal taşkın kontrol seviyesi ($S_{i,t}^{Op,Taş}$) olarak alınmaktadır. Bu şekilde her bir ay için, olabilecek tasarım amaçlı taşkın hidrografları ile optimal taşkın kontrol seviyeleri ($S_{i,t}^{Op,Taş}$) belirlenmektedir.

2. Aşama: Bu çözüm gerçekleşmiyorsa, taşkın kontrolü için ek taşkın hacmi gerektiği anlaşılmaktadır. Bu amaçla, maksimum işletme seviyesi (S_i^{Maks}) üzerine ek taşkın kontrol hacmi ilave edilmelidir. Bu ek hacim, aylık normal işletme seviyesi ($S_{i,t}^{Nor}$) ile minimum işletme seviyesi (S_i^{Min}) arasındaki fark kadar olmalıdır. Buna göre, ek hacimlerle birlikte her baraj için, artırılmış maksimum işletme seviyeleri

$$S_i^{Maks*} = S_i^{Maks} + (S_{i,t}^{Nor} - S_i^{Min}) \tag{21}$$

şeklindedir. Burada, taşkın kontrol seviyeleri,

$$S_i^{Min} \le (S_{i,f}^{Taş})_t \le S_i^{Maks*} \quad (22)$$

arasında oluşacaktır. Buradan, barajlara gelen tasarım amaçlı taşkın hidrografları, başlangıçta (f=1 için $(S_{i,1}^{Taş})_t = S_{i,t}^{Nor}$) aylık normal işletme seviyesi ile karşılanmakta ve aynı zamanda mansapta taşkın kontrolü için konulan kısıt altında barajdan bırakılan akımın enküçüklenmesine çalışılmaktadır. Bu şekilde elde edilen işletme seviyelerinden ($(S_{i,f}^{G})_t$) başlangıçta eklenen hacim $(S_{i,t}^{Nor} - S_i^{Min})$ düşülerek taşkın kontrol seviyeleri ($(S_{i,f}^{Taş})_t$) bulunmaktadır. Buna göre, taşkın kontrol seviyeleri

$$(S_{i,f}^{Taş})_t = (S_{i,f}^{G})_t - (S_{i,t}^{Nor} - S_i^{Min}) \quad (23)$$

şeklinde elde edilmektedir. $(S_{i,f}^{Taş})_t$ değerlerinden en küçük olanı, söz konusu ay için optimal taşkın kontrol seviyesi ($S_{i,t}^{Op,Taş}$) olarak alınmaktadır.

3. Aşama: Bu şekilde de çözüme ulaşılamıyorsa, taşkın kontrolü için debi kısıtı serbest bırakılarak, taşkın pik debisini, dolayısıyla, taşkından oluşacak zararı enküçükleyecek akımın bırakılmasına çalışılmaktadır. Burada, taşkın kontrolü için mansapta serbest bırakılan debi,

$$W_i^{Maks*} = Min[Maks((Q_{i,f} + R_{i,f})_t)] \quad (24)$$

şeklinde belirlenmelidir. Buna göre, taşkın kontrolü için bırakılacak debi kısıtı,

$$(Q_{i,f} + R_{i,f})_t \le W_i^{Maks*} \quad (25)$$

ile tanımlanmalıdır. Bu duruma göre elde edilen işletme seviyelerinden ek taşkın kontrol hacimleri düşülerek taşkın kontrol seviyeleri ($(S_{i,f}^{Taş})_t$) elde edilmektedir. $(S_{i,f}^{Taş})_t$ değerlerinden en küçük olanı, söz konusu ay için optimal taşkın kontrol seviyesi ($S_{i,t}^{Op,Taş}$) olarak alınmaktadır.

Kısıtlar: Kapasite kısıtları olarak karşımıza çıkmaktadır. Bunlar:

1.Depolama kısıdı iki türlüdür. Birincisi, saatlik olarak elde edilen işletme seviyesi, mevcut minimum ve maksimum işletme seviyesi arasında olması (mevcut taşkın/kuraklık kontrol aralığı) şeklinde olup aşağıda verildiği gibidir.

$$S_i^{Min} \le (S_{i,f}^{Taş})_t \le S_i^{Maks} \quad (26)$$

İkincisi, ek taşkın kontrol hacminin kullanılması durumunda depolama kısıtları Denklem 22' de gösterilmiştir.

2.Enerji üretimi için bırakılacak su miktarı kısıtları,

$$0 \le (Q_{i,f})_t \le Q_i^{Maks} \quad (27)$$

şeklindedir.

3.Dolu savaktan bırakılacak su miktarı kısıtları,

$$0 \le (R_{i,f})_t \le R_i^{Maks} \tag{28}$$

4. Barajlardan bırakılan akımların mansapta taşkın emniyeti için öngörülen debi miktarından küçük olması şartı, kısıt olarak kullanılmaktadır. Yani, taşkın emniyeti için mansapta debi kısıdı,

$$(Q_{i,f} + R_{i,f})_t \le W_i^{Maks} \tag{29}$$

şeklindedir. Burada, W_i^{Maks}, i-barajı için taşkın emniyeti bakımından mansaptaki maksimum debiyi göstermektedir.

Yapısal denklemler: Her bir baraj için yazılmaktadır. Buna göre, her bir i-barajında t-ayı ve f-saati için yazılan su dengesi ilişkisi,

$$(S_{i,f+1} - S_{i,f})_t = (F_{i,f} + Q_{i-1,f} + R_{i-1,f} - Q_{i,f} - R_{i,f} - L_{i,f})_t \tag{30}$$

şeklinde verilmiştir.

KURAKLIK KONTROLÜ AMAÇLI KISA SÜRELİ PLANLAMA İÇİN OPTİMİZASYON MODELİ

Çok barajlı sistemde kısa süreli planlama için kuraklık kontrolü amaçlı optimizasyon modelinde, sisteme gelen kurak periyot hidrografları kullanılarak barajlardan bırakılan akımların enküçüklenmesi ile kuraklık kontrolü değerlerinin elde edilmesi, her bir barajda elde edilen bu kontrol değerlerinden en büyük olanının seçilmesi ve optimal kuraklık kontrol seviyesinin belirlenmesi amaçlanmıştır.

Amaç Fonksiyonu: Burada, belirli bir ay için girilen kuraklık hidrografının sistem tarafından depolanıp barajdan bırakılan enbüyük akımın enküçüklenmesi amaç fonksiyonu olarak tanımlanırken aynı zamanda, barajdan toplam debi çıkışının (enerji üretimi için ve savaklanan) mansap için kuraklık ve kirlilik kontrolü debisinin altında olması kısıtı konulmuştur. Amaç fonksiyonu üç aşamalıdır. Bu aşamalar aşağıda verilmiştir.:

1.Aşama: Mevcut kuraklık kontrolü aralığı kullanılarak, mansapta kuraklık ve kirlilik kontrolü debi kısıtı olma durumuna göre amaç fonksiyonu, Denklem 19' da verildiği gibidir. Eğer bu çözüm gerçekleşmezse, ikinci aşamaya geçilmektedir.

2.Aşama: Artırılmış kuraklık kontrolü aralığı ile mansapta kuraklık ve kirlilik kontrolü debi kısıtı olma durumuna göre amaç fonksiyonu Denklem 19'da verildiği gibidir. Eğer bu çözüm gerçekleşmezse, üçüncü aşamaya geçilmektedir.

3.Aşama: Artırılmış kuraklık kontrolü aralığı ile mansapta kuraklık ve kirlilik kontrolü debi kısıtı serbest bırakılması durumuna göre amaç fonksiyonu Denklem 19'da verildiği gibidir.

Kısıtlar: Kapasite kısıtları olarak karşımıza çıkmaktadır. Bunlar:

1.Depolama kısıtı iki türlüdür. Birincisi, mevcut kuraklık/taşkın kontrol aralığının kullanılması durumunda depolama kısıtları Denklem 26 verildiği gibidir. İkincisi, artırılmış kuraklık aralığının kullanılması durumunda depolama kısıtları,

$$S_i^{Min*} \le (S_{i,f})_t \le S_i^{Maks} \tag{31}$$

olup, burada, S_i^{Min*}: i-barajında kuraklık kontrol aralığını artırmak için azaltılmış minimum depolama miktarını göstermektedir. Bu değer,

$$S_i^{Min*} = S_i^{Min} - (S_i^{Maks} - S_{i,t}^{Nor}) \tag{32}$$

şeklindedir.

2. Enerji üretimi için bırakılacak su miktarı kısıtları, Denklem 27' de verildiği gibidir.

3. Dolu savaktan bırakılacak su miktarı kısıtları, Denklem 28' de verildiği gibidir.

4. Kuraklık ve kirlilik kontrolü için kısıt,

$$W_{i,t} \leq (Q_{i,f} + R_{i,f})_t \tag{33}$$

şeklindedir.

Yapısal denklemler: Her bir baraj için yazılmaktadır. Buna göre, her bir i-barajında t-ayı ve f-saati için yazılan su dengesi ilişkisi, Denklem 30' da verilmiştir.

Optimal kuraklık kontrol seviyesinin belirlenmesi: Mevcut kuraklık kontrol aralığı kullanılarak yapılan optimal işletme neticesinde her bir baraj için elde edilen işletme değerlerinden ($(S_{i,f}^{Kur})_t$) en büyük olanı maksimum kuraklık kontrol değeri ($(S_{i,f}^{Maks,Kur})_t$) olarak seçilmektedir. Artırılmış kuraklık kontrol aralığı kullanılarak yapılan optimal işletme neticesinde her bir baraj için elde edilen işletme değerlerine kuraklık kontrol aralığını artırmak için eklenen miktar eklenerek kuraklık kontrol işletme değerleri elde edilmektedir. Buna göre kuraklık kontrol işletme değerleri,

$$(S_{i,f}^{Kur})_t = (S_{i,f})_t + (S_i^{Maks} - S_{i,t}^{Nor}) \tag{34}$$

şeklinde belirlenmektedir. Bu değerlerinin en büyük olanı, maksimum kuraklık kontrol değeri ($(S_{i,f}^{Maks,Kur})_t$) olarak seçilmektedir. Seçilen maksimum kuraklık kontrol değeri, normal işletme seviyesi ve maksimum işletme seviyesi ile karşılaştırılmaktadır. Bu değer normal işletme seviyesi ile maksimum işletme seviyesi arasında ise, $S_{i,t}^{Nor} > (S_{i,f}^{Maks,Kur})_t \geq S_i^{Maks}$, optimal kuraklık kontrol seviyesi olarak atanmaktadır. Yani, bu duruma göre, optimal kuraklık kontrol işletme seviyesi,

$$S_{i,t}^{Op,Kur} = (S_{i,f}^{Maks,Kur})_t \tag{35}$$

şeklinde ifade edilmektedir. Ancak maksimum kuraklık kontrol değeri normal işletme seviyesinden küçük veya eşit ise, $(S_{i,f}^{Maks,Kur})_t \leq S_{i,t}^{Nor}$, normal işletme seviyesi ile bu değer arasındaki fark normal işletme seviyesinin üstüne ilave edilmekte

$$(S_{i,f}^{Kur*})_t = S_{i,t}^{Nor} + (S_{i,t}^{Nor} - (S_{i,f}^{Maks,Kur})_t) \tag{36}$$

ve elde edilen değer ($(S_{i,f}^{Kur*})_t$) maksimum işletme seviyesinden küçükse, $(S_{i,f}^{Kur*})_t < S_i^{Maks}$, bu değer optimal kuraklık kontrol seviyesi,

$$(S_{i,f}^{Op,Kur})_t = (S_{i,f}^{Kur*})_t \tag{37}$$

olarak atanmaktadır. Normal işletme seviyesi ile maksimum kuraklık kontrol değeri arasındaki fark değerinin normal işletme seviyesinin üstüne ilave edildikten sonra elde edilen değeri ($(S_{i,f}^{Kur*})_t$) maksimum işletme seviyesinden büyükse, $(S_{i,f}^{Kur*})_t > S_i^{Maks}$, optimal kuraklık kontrol seviyesi,

$$S_{i,t}^{Op,Kur} = S_i^{Maks} \tag{38}$$

kadar olmalıdır.

KULLANILAN OPTİMİZASYON TEKNİĞİ: DPSA

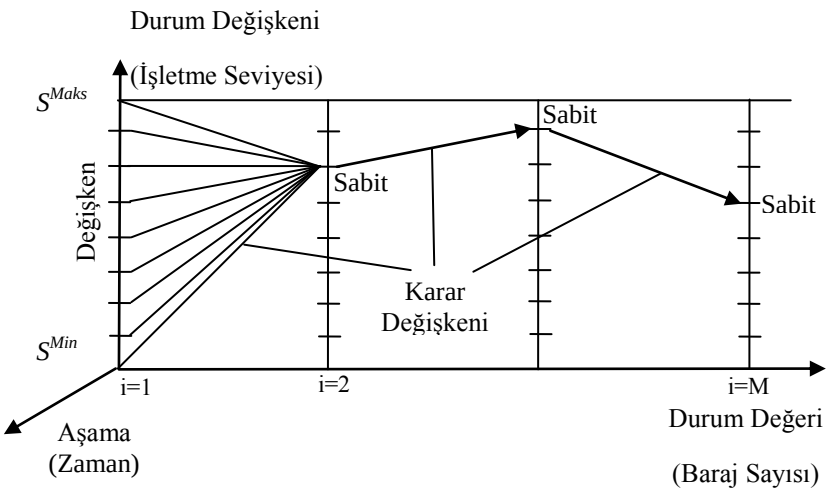
Ardışık yaklaştırmalı dinamik programlama yönteminin amacı, çok karar değişkenli bir dinamik programlama problemini, her birinde tek bir karar değişkeni olan dinamik programlama ait problemlere ayrıştırmak ve böylece karar değişkenlerini teker teker ele alarak ana problemi çözmektir.

Bu metodun avantajı, (n) boyutlu bir dinamik programlama problemini bir dizi tek boyutlu probleme indirgeyerek, hem hesaplama zamanını hem de bilgisayar bellek gereksinimlerini azaltmaktır. Çok barajlı ve çok amaçlı baraj işletmesinde uygulanan dinamik programlama yöntemlerinin kullanıldığı bilgisayar belleği ve zamanı (n)'ye bağlı olarak üssel (y) doğrusal bir artış göstermektedir. En iyi çözümün bulunabilmesi için başlangıç çözümünün dikkatli seçilmesi ve birden fazla başlangıç çözümü denenerek sonuçların karşılaştırılması gerekebilir. Dinamik programlamada üç değişken tipi vardır. Bunlar:

1. Sistemin davranışını olduğu gibi tanımlayan ve sistemin herhangi bir andaki durumunu gösteren durum değişkenleri, (x, sistemde barajın işletme seviyesidir.)

2. Seçilen bir amaca göre sistemin kontrol edilebilir girdileri hakkında karar vermeyi sağlayan karar değişkenleri, (d, sistemde barajdan enerji üretimi için bırakılan ve dolu savaktan savaklanan su miktarlarıdır.)

3. Bu kararların verildikleri aralıkları belirleyen aşama değişkenleri, (t, sistemde işletme zamanını göstermektedir.)



Şekil 5. Dinamik programlamada herhangi bir aşamasındaki durum değeri ve durum-karar değişkeninin şematik gösterimi.

şeklindedir. Şekil 5'de, dinamik programlamada herhangi bir aşamasındaki durum değeri ve durum-karar değişkeninin şematik gösterimi gösterilmiştir. Bu girdilerin bazı kısıtlara bağlı olarak aldıkları değer takımına politika denmektedir. Bu politikanın sistemin çıktıları üzerine etkisini belirleyen ölçüt ise, amaç fonksiyonu olarak tanımlanmaktadır.

KISA SÜRELİ İŞLETME OPTİMİZASYONDA KİNEMATİK DALGA YAKLAŞIMI

Kinematik dalga yaklaşımında, St. Venant denklemlerinde ivme ve basınç terimleri ihmal edildiğinden, barajlardan bırakılan akımların oluşturduğu dalga hareketi, süreklilik denklemi ile tanımlanmaktadır. Bu yaklaşım kullanılarak barajlardan bırakılan akımları diğer baraja ulaşma ve zamanını gösteren eğriler oluşturulmuştur.

Bu yaklaşımla yapılan modellemede, barajlardan bırakılan akımlar, ilk olarak dolu savak ve enerji üretimi kapasitelerinin oluşturduğu toplam kapasite ile karşılaştırılmaktadır. Burada bırakılan akım, bu kapasiteden küçük ise, bırakılma zamanı, akımın diğer baraja ulaşma zamanını belirleyen denklemden elde edilen zamanın üzerine ilave edilmekte ve diğer baraja ulaşma değerini belirleyen denklemler ile akımın ötelenmesi gerçekleştirilmektedir. Bırakılan akım bu kapasiteden büyük ise, kapasiteye aşmayacak şekilde öngörülen bırakılma hidrografı ile, akımın diğer baraja ulaşma zamanı ve değeri belirlenmektedir. Bu elde edilen değerler diğer baraj aksında, barajın havzasından gelen akımlar üzerine ilgili zaman durumuna göre ilave edilmektedir.

OPTİMİZASYON MODELLERİ İÇİN UYGULAMA

Ceyhan havzasında Ceyhan nehri ana kolu üzerinde ardışık şekilde bulunan çok barajlı bir su kaynakları sistemi uygulama amaçlı olarak seçilmiştir. Şekil 6'da, seçilen çok barajlı bir su kaynakları sisteminin şematik görünüşü verilmektedir. Bu sisteme ait barajlardan Menzelet ve Kılavuzlu Barajları, enerji ve sulama amaçlı; Sır ve Berke Barajları, enerji amaçlı ve Aslantaş Barajı ise, enerji, sulama, içmesuyu ve taşkından koruma amaçlı olarak planlanmışlardır. Tablo 1' de Ceyhan Havzası' nda Ceyhan Nehri ana kolu üzerinde ardışık şekilde bulunan barajlara ait veriler verilmiştir. Tablo 2' de, barajlardaki hacim yükseklik ilişkileri gösterilmiştir. Bu veriler uzun/kısa süreli optimal işletme modelinde kullanılmıştır.

Tablo 3'de, barajlar arası akımların taşınması için kinematik dalga yaklaşımında kullanılmak üzere barajlar arası akarsu kesitinin ve yatağının özellikleri verilmiştir. Burada belirtilen akarsuya ait kesit özellikleri ve bazı parametreler, akarsuya ait ortalama akım verileri gözetilerek üretilmiştir. Tablo 3'de, L: Barajlar arası akarsuyun uzunluğunu, H: Barajlar arası kot farkını, S: Akarsudaki hidrolik eğimi, n: Tabandaki Manning katsayısını, b,h: Kesit büyüklüğünü, A: Kesit alanını, P: Islak çevreyi, R: Hidrolik yarıçapı, Q:ortalama debiyi göstermektedir.

Şekil 7'de kritik döneme ait aylık kurak akımları verilmektedir. Bu akım verileri uzun süreli optimal işletme modelinde kullanılarak güvenilir güç enbüyüklenirken aylık minimum işletme seviyeleri elde edilmektedir. Şekil 8' de aylık ortalama akımlar verilmiştir. Bu aylık ortalama akım verileri 27 yıllık akım verisinden elde edilmiştir. Bu veriler uzun süreli optimal işletme modelinde kullanılarak toplam enerji enbüyüklenirken aylık normal işletme seviyeleri elde edilmiştir. Şekil 9 ve 10' da, barajlara taşkın/kuraklık olması beklenen Mayıs ayında havzasından gelen akım verileri verilmektedir. Bu veriler kullanılarak taşkın/kuraklık kontrolü amaçlı kısa süreli optimal işletme yapılmıştır.

Tablo 1. Ceyhan Havzası' nda Ceyhan Nehri ana kolu üzerinde ardışık şekilde bulunan barajlara ait veriler.

Barajlar	Menzelet	Kılavuzlu	Sır	Berke	Aslantaş
Yağış Alanı (km^2)	8340	8486	12950	13222	14874

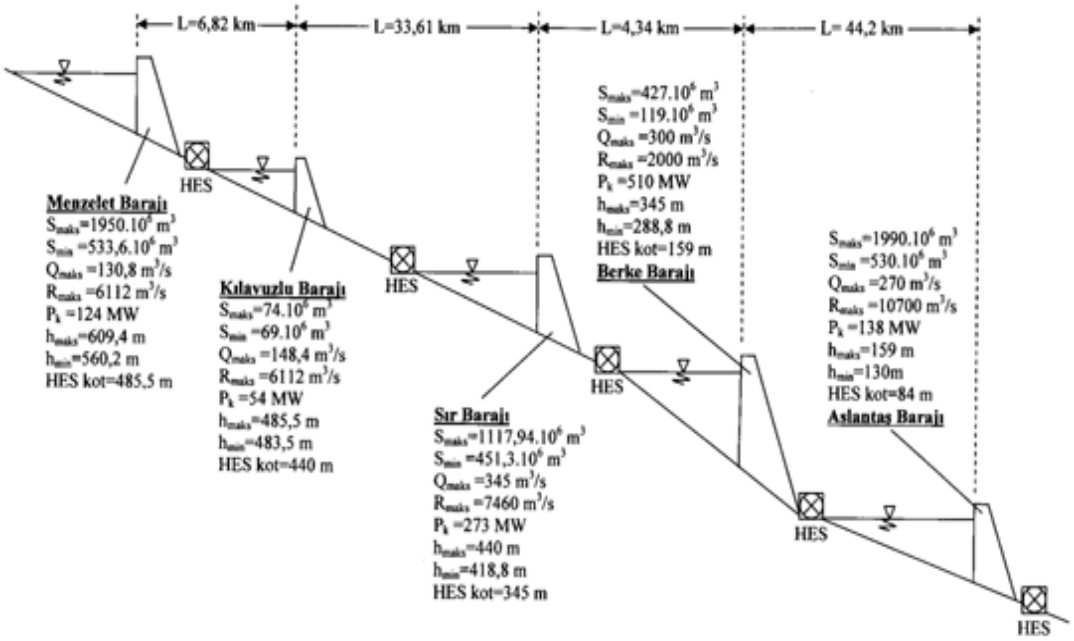
Kurulu Güç (MW)	124	54	273	510	138
Baraj Yüksekliği (m)	150.5	56	120	201	95
Maksimum İşletme Seviyesi (m)	609.4	485.5	440	345	159
Minimum işletme seviyesi (m)	560.2	483.5	418.8	288.8	130
Maksimum Hacim (10^6 m^3)	1950	74	1117.94	427	1990
Minimum Hacim (10^6 m^3)	533.6	69	451.3	119	530
Kuyruksuyu kotu (m)	485.5	440	345	159	84
Dolu Savak Kapasitesi (m^3/s)	6112	6112	7460	2000	10700
Enerji Üretimi Kapasitesi (m^3/s)	130.8	148.4	114.25	300	270

Tablo 2. Barajlarda hacim yükseklik ilişkisi ($h=a.S^b$, h (m), S (10^6 m^3)).

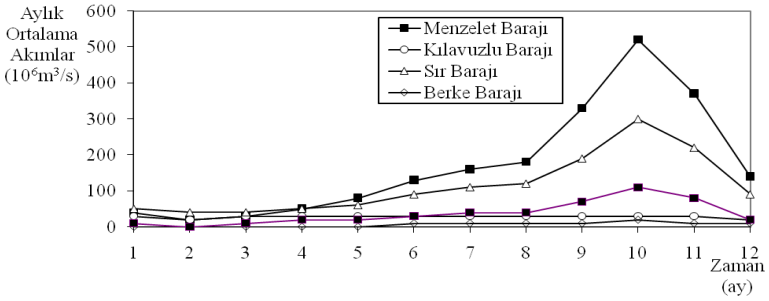
Barajlar	Menzelet	Kılavuzlu	Sır	Berke	Aslantaş
a	15.494	9.4261	28.264	42.036	3.8336
b	0.2998	0.4092	0.2067	0.2581	0.3947

Tablo 3. Barajlar arası akarsu kesitinin ve yatağının özellikleri.

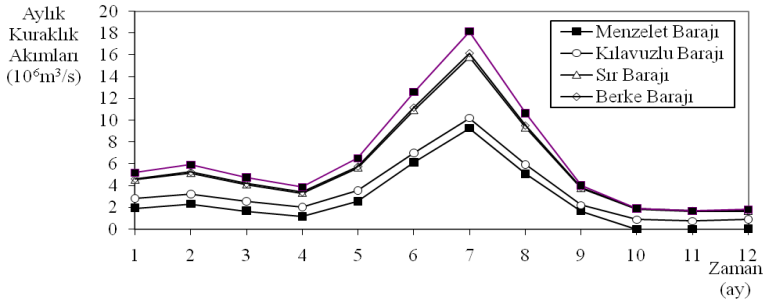
Barajlar arası	L (m)	H (m)	S	n	h (m)	b (m)	A (m^2)	P (m)	R (m)	Q (m^3/s)
Menz.-Kılavuzlu	6820	123.9	0.0182	0.025	1.5	30	45	33	1.36	298
Kılavuzlu-Sır	35610	45.5	0.0013	0.025	2.5	48	120	53	2.26	295
Sır-Berke	4340	95	0.0219	0.025	1.4	30	42	32.8	1.28	293
Berke-Aslantaş	45610	186	0.0041	0.025	2	40	80	44	1.82	304



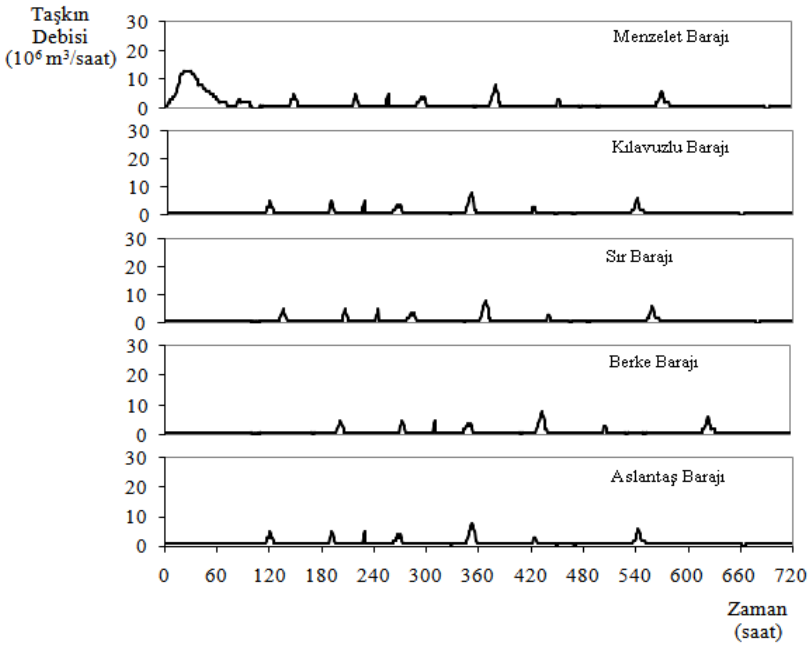
Şekil 6. Ceyhan Havzası'nda Ceyhan Nehri ana kolu üzerinde ardışık olarak bulunan çok amaçlı ve çok barajlı bir su kaynakları sisteminin şematik görünüşü.



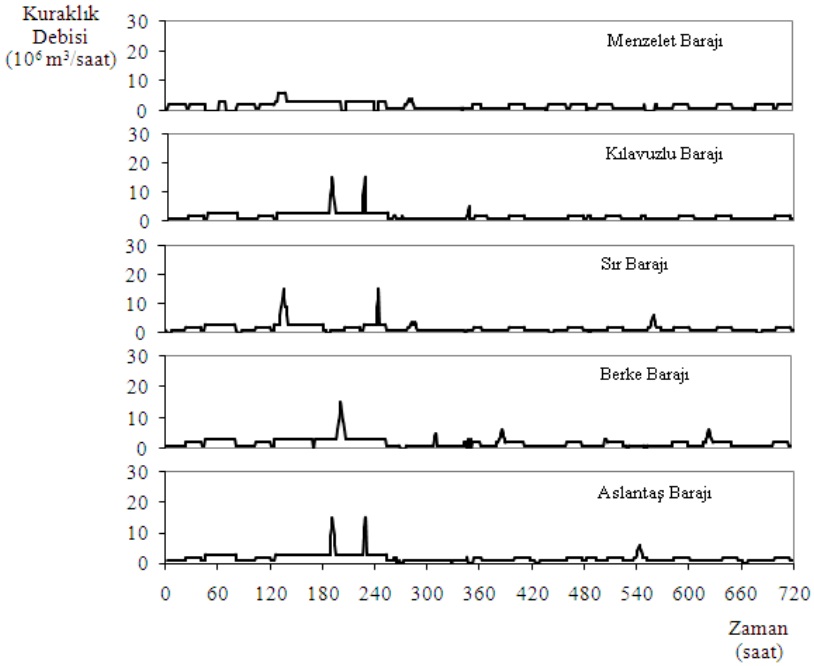
Şekil 7. Barajlara havzasından gelen aylık ortalama akımlar.



Şekil 8. Barajlara kurak dönemde havzasından gelen aylık akımlar.



Şekil 9. Barajlara taşkın olması beklenen Mayıs ayında havzasından gelen akımlar.



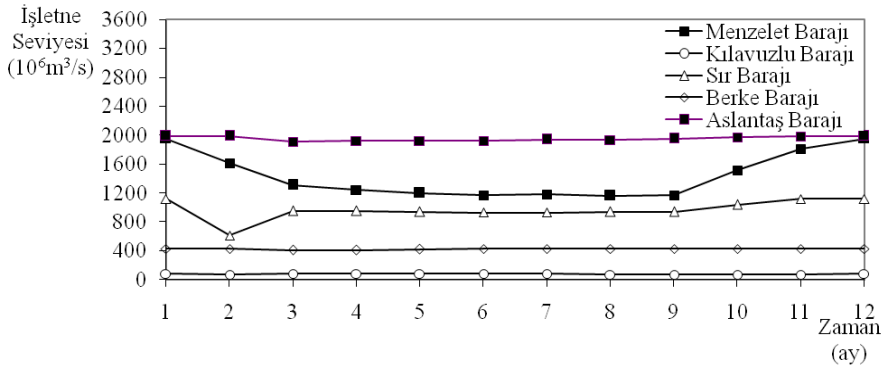
Şekil 10. Barajlara kuraklık olması beklenen Mayıs ayında havzasından gelen akımlar.

ELDE EDİLEN SONUÇLAR

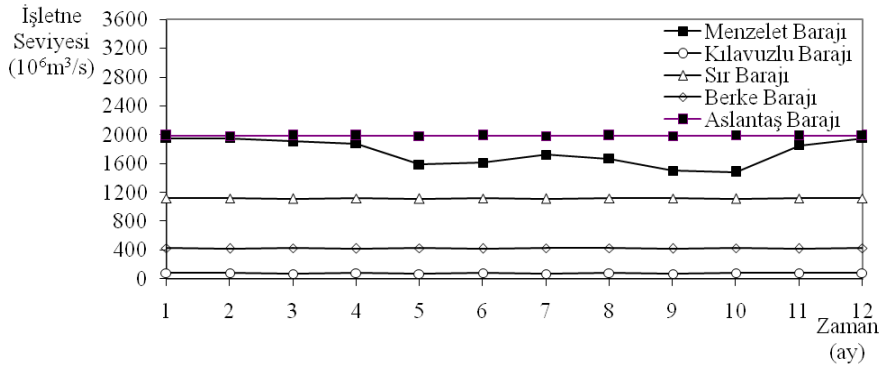
Uzun süreli optimal işletme neticesinde güvenilir güç 163,970 MW, ortalama güç ise 438,340 MW olarak elde edilmiştir. Bu optimal işletme sonucunda elde edilen Şekil 11' deki aylık minimum işletme seviyelerine ve Şekil 12' deki aylık normal işletme seviyelerine bakıldığı zaman, en büyük işletme hacmine sahip barajların optimizasyon sürecini kontrol ettiği ve yönettiği görülmektedir.

Taşkın/kuraklık kontrolü amaçlı kısa süreli optimal işletme neticesinde elde edilen Şekil 12' deki taşkın kontrol seviyelerine ve Şekil 13' deki kuraklık kontrol seviyelerine bakıldığı zaman, çok barajlı sisteme gelen akımların sistem tarafından depolandığı, sistemden

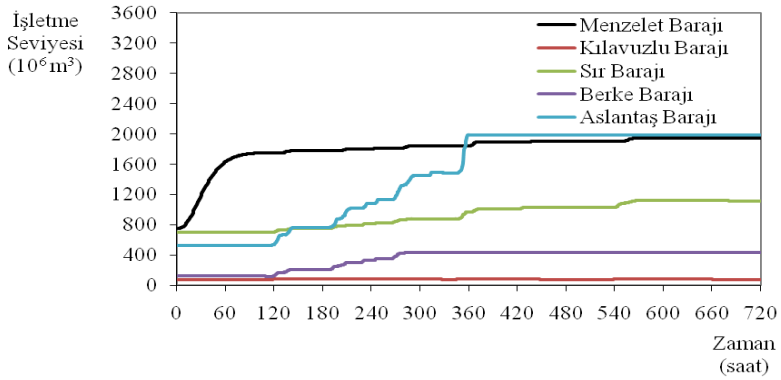
bırakılan akımların kuraklık kontrolü için gerekli şartları sağladığı ve en büyük işletme hacmine sahip barajların optimizasyon sürecini kontrol ettiği ve yönettiği çıkarımları yapılabilir. Bu işletme seviyelerinden Tablo 4'de verilen optimal işletme seviyeleri elde edilmektedir. Elde edilen optimal işletme seviyeleri, çok barajlı sistemi yöneten ve yönlendiren için kılavuz seviyeler şeklinde kullanılabilir.



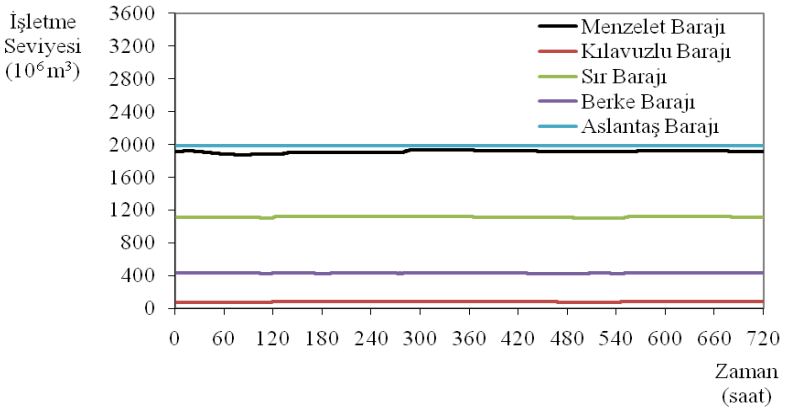
Şekil 11. Barajlarda kurak dönemde oluşan minimum işletme seviyeleri.



Şekil 12. Barajlardaki aylık normal işletme seviyeleri.



Şekil 13. Barajlarda Mayıs ayı için taşkın kontrol seviyeleri.



Şekil 14. Barajlarda Mayıs ayı için kuraklık kontrol seviyeleri.

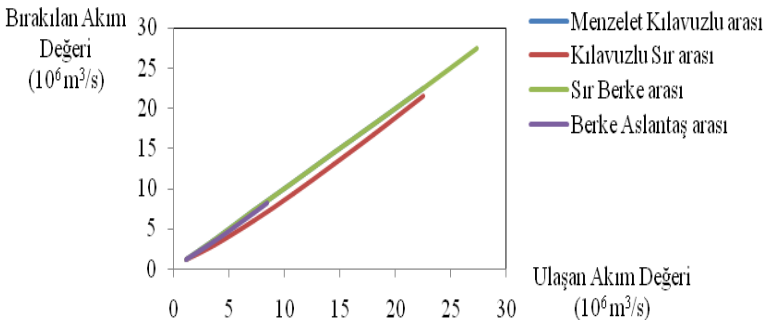
Tablo 4. Barajlarda Mayıs ayı için elde edilen optimal işletme seviyeleri.

Barajlar	Menzelet	Kılavuzlu	Sır	Berke	Aslantaş
$S_{i,t}^{Min}$ (10^6 m^3)	1810	70	1120	430	1980
$S_{i,t}^{Nor}$ (10^6 m^3)	1850	80	1120	420	1990
$S_{i,t}^{Op,Taş}$ (10^6 m^3)	752	70	705	120	530
$S_{i,t}^{Op,Kur}$ (10^6 m^3)	1932	80	1110	430	1990
S_{i}^{Min}(10^6 m^3)	530	70	450	120	530
S_{i}^{Maks} (10^6 m^3)	1950	80	1120	430	1990

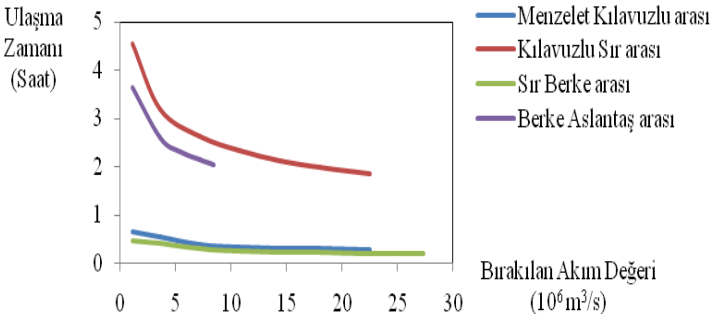
Kinematik dalga yaklaşımı kullanılarak barajdan bırakılan akım değerinin diğer baraja ulaşma değeri ve zamanını gösteren grafikler Şekil 15 ve 16'da gösterildiği gibidir. Bu grafikler incelendiğinde;

- Barajlar arası mesafe ne kadar artarsa barajdan bırakılan akım değeri, diğer baraja o kadar azalarak ulaştığı,
- Barajlar arası mesafe çok küçük ise, barajdan bırakılan akım, diğer baraja değişmeden ulaştığı,
- Barajdan bırakılan akım değeri büyüdüğü zaman ulaşma zamanının küçüldüğü,
- Barajlar arası mesafe artıkça ulaşma zamanı artmakta, azaldıkça azalmakta olduğu

görülmektedir.



Şekil 15. Barajdan bırakılan akımın diğer baraja ulaşma değerinin eğrisi.



Şekil 16. Barajdan bırakılan akımın diğer baraja ulaşma zamanının eğrisi.

SONUÇLAR

Enerji enbüyüklenmesi amaçlı uzun süreli optimal işletme modeli ve taşkın/kuraklık kontrolü amaçlı olarak kısa süreli optimal işletme modeli Ceyhan Havzasında Ceyhan Nehri ana kolu üzerinde bulunan çok barajlı bir su kaynakları sistemine başarılı bir şekilde uygulanmıştır. Yapılan işletme neticesinde, aşağıdaki sonuçlar üretilmiştir. Buna göre;

- Çok barajlı sisteme gelen akımların sistem tarafından depolandığı,
- Sistemden bırakılan akımların kuraklık kontrolü için gerekli şartları sağladığı,
- En büyük işletme hacmine sahip barajların optimizasyon sürecini kontrol ettiği ve yönettiği

görülmüştür. Kısa süreli optimizasyon yapıldığı için akımların akarsu yatağında ötelenerek bir barajdan diğerine taşınması esnasında kinematik dalga yaklaşımı, optimizasyon modeli içerisinde başarılı bir şekilde kullanılmıştır. Ayrıca barajlardan bırakılan akımların barajların havzasından gelen akım değerlerini etkilemediği görülmüştür.

Çok barajlı sisteme ait herhangi bir barajda göz önüne alınan ay için kısa süreli taşkın/kuraklık durumu söz konusu olduğunda yapılan kısa süreli işletme neticesinde normal işletme seviyesinin bir miktar altında/üzerinde optimal taşkın/kuraklık kontrol seviyesi elde edilmiştir. Bu şekilde yapılacak işletme sonucunda barajlardaki işletme seviyesinin bir sonraki ayda normal işletme seviyesine ulaştığı görülmüştür. Böylece diğer aylarda uzun süreli işletme için öngörülen normal işletme seviyelerinden uzaklaşılmamaktadır. Yani, kısa süreli taşkın/kuraklık durumu için yapılan optimal işletme neticesinde uzun süreli planlama için öngörülen optimal politikaların dışına çıkılmasına gerek kalmamaktadır.

Sonuç olarak, kısa süreli taşkın/kuraklık olması beklenen (kuraklık veya havzadan gelecek akımın yeterli olmadığı öngörülen) aylarda, hem barajlardaki seviyelerin normal

işletme seviyesinin ne kadar altında/üzerinde olacağının belirlenmesi için hem de uzun süreli işletme esnasında normal işletme seviyelerinden uzaklaşmamak için taşkın/kuraklık amaçlı kısa süreli bir optimal işletme yapılmıştır. Elde edilen optimal işletme seviyeleri, çok barajlı sistemi yöneten ve yönlendiren için kılavuz seviyeler şeklinde kullanılabilir. Ayrıca, bu kılavuz seviyeleri her ay için elde edilerek gerçek zamanlı optimal işletmede kullanılabilir.

TEŞEKKÜR

Mahmut Sert 1 Mayıs 2007 tarihinde vefat etmiştir. Bu tarihe kadar bu çalışmanın ortaya çıkartılması ve geliştirilmesi üzerine çok büyük katkılarda bulunmuştur. O olmadan bu çalışma asla yapılmazdı. Katkılarından dolayı kendisine çok teşekkür ederim.

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EFFECT OF DISTRIBUTED PATCH MASS ON FREQUENCY OPTIMIZATION OF LAMINATED PLATES

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Key Words: Laminated plate with patch mass, fundamental frequency, optimal designs

ABSTRACT This paper deals with effect of distributed patch mass on the frequency optimization of simply supported symmetric angle-ply laminated plates. The objective function is the maximization of the fundamental frequency of the laminates and the design variable is the fibre orientation. The first-order shear deformation theory (FSDT) is used for the finite element solutions. Firstly, the numerical results are compared with the results of some investigators for without mass. The modified feasible direction (MFD) is used for the optimization method. Finally, the effect of mass ratio, location of the distributed patch mass and plate aspect ratios on the optimum results is investigated and the results are compared.

INTRODUCTION

Vibration problems of the plates with and without mass loading are very common in engineering applications. Rectangular plates have wide applications in civil and mechanical engineering. Plates form an essential part of many aerospace, marine and automobile structures. These structural components, in many instances, are subjected to vibration. The problem of plates carrying distributed masses initiated from the design of electronic systems. The printed circuit boards and plate-like chassis can be approximated as flat rectangular plates carrying distributed masses and subjected to vibration. Laminated composite plates are widely used in industry and new fields of technology. Due to high degrees of anisotropy and low rigidity in transverse shear, Kirchhoff hypothesis as a classical theory is no longer adequate. The hypothesis states that transverse normal to the mid-plane of a plate remains straight and normal after deformation because of the negligible transverse shear effects. Refined theories based on removing those restrictions of transverse normal have been recently used. As a result, the free vibration frequencies calculated by using the classical thin plate theory are higher than those obtained by Mindlin plate theory, in which transverse shear and rotary inertia effects are included.

While there are several reports on plate vibrations with added point masses, very few reports on the plate vibrations with distributed mass loading can be found in the literature (Kompaz and Telli, (2002); Wong, (2002), Pandit *et al.*, (2007)).

BASIC EQUATIONS

Consider a rectangular laminated plate of length *a*, width *b* and thickness *h* (Figure 1).

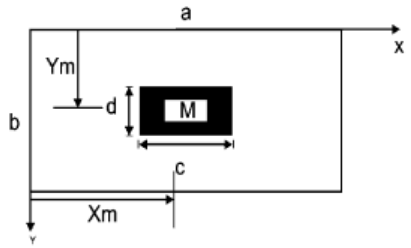


Fig. 1. Plate with distributed patch mass.

The first order shear deformation theory is based on the following assumed displacement field:

$$u = u_o(x,y) + z\psi_x(x,y)\,,\ \ v = v_o(x,y) + z\psi_y(x,y),\ \ w = w(x,y) \tag{1}$$

The in-plane strain components $\left(\varepsilon_x, \varepsilon_y, \gamma_{xy}\right)$ and the transverse shear strains $\left(\gamma_{xz}, \gamma_{yz}\right)$ can be written as follows

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = \begin{vmatrix} \dfrac{\partial u_o}{\partial x} \\ \dfrac{\partial v_o}{\partial y} \\ \dfrac{\partial u_o}{\partial y} + \dfrac{\partial v_o}{\partial x} \end{vmatrix} + z \begin{vmatrix} \dfrac{\partial \psi_x}{\partial x} \\ \dfrac{\partial \psi_y}{\partial y} \\ \dfrac{\partial \psi_x}{\partial y} + \dfrac{\partial \psi_y}{\partial x} \end{vmatrix}, \quad \begin{vmatrix} \gamma_{yz} \\ \gamma_{xz} \end{vmatrix} = \begin{vmatrix} \dfrac{\partial w}{\partial y} - \psi_y \\ \dfrac{\partial w}{\partial x} + \psi_x \end{vmatrix} \tag{2}$$

The equations for in-plane stresses $\left(\sigma_x, \sigma_y, \sigma_{xy}\right)$ and the transverse shear stresses $\left(\tau_{yz}, \tau_{xz}\right)$ of the *kth* layer can be written as

$$\begin{pmatrix} \sigma_x \\ \sigma_y \\ \sigma_{xy} \end{pmatrix}_{(k)} = \begin{pmatrix} \bar{Q}_{11} & \bar{Q}_{12} & \bar{Q}_{16} \\ \bar{Q}_{12} & \bar{Q}_{22} & \bar{Q}_{26} \\ \bar{Q}_{16} & \bar{Q}_{26} & \bar{Q}_{66} \end{pmatrix}_{(k)} \begin{pmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_{xy} \end{pmatrix}, \quad \begin{pmatrix} \tau_{yz} \\ \tau_{xz} \end{pmatrix}_{(k)} = \begin{pmatrix} \bar{Q}_{44} & \bar{Q}_{45} \\ \bar{Q}_{45} & \bar{Q}_{55} \end{pmatrix}_{(k)} \begin{pmatrix} \gamma_{yz} \\ \gamma_{xz} \end{pmatrix} \tag{3}$$

The resultant moments and shearing forces for symmetrically laminated plates are given as

$$\begin{Bmatrix} M_x \\ M_y \\ M_{xy} \end{Bmatrix} = \int_{-h/2}^{h/2} \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} z dz\,, \quad \begin{Bmatrix} Q_x \\ Q_y \end{Bmatrix} = \int_{-h/2}^{h/2} \begin{Bmatrix} \tau_{xz} \\ \tau_{yz} \end{Bmatrix} dz \tag{4}$$

FINITE ELEMENT FORMULATION

The finite element model has fives degrees of freedom per node, which are the displacements u_o, v_o, w_o and rotations ψ_x , ψ_y . Using the same shape functions associated with node i (i=1, 2,….., n) for interpolating the variables in each element, we can write

$$u_o^e = \sum_{i=1}^{N_n} u_i^e N_i^e, v_o^e = \sum_{i=1}^{N_n} v_i^e N_i^e, w_o^e = \sum_{i=1}^{N_n} w_i^e N_i^e\ ,\ \ \psi_x^e = \sum_{i=1}^{N_n} \psi_{xi}^e N_i^e, \psi_y^e = \sum_{i=1}^{N_n} \psi_{yi}^e N_i^e \tag{5}$$

where N_i represents the element interpolation functions and N_n is the number of nodes per element. In this study, the four-node Lagrangian finite element approach is used for free vibration analysis of laminated plates. As in the standard finite element procedure, one obtains the global eigenvalue equation is

$$\left([K] - \lambda[M]\right)\left\{\bar{D}\right\} = 0 \tag{6}$$

where $[K]$ and $[M]$ are the global stiffness and mass matrices, respectively, and $\left\{\bar{D}\right\}$ is a global displacement vector. Eq. (6) is a set of homogeneous linear equations in the unknown displacements $\left\{\bar{D}\right\}$. For non-trivial solution, the determinant is equal to zero

and the eigenvalues correspond to natural frequencies of the laminated plate. The subspace iteration method is used for frequency analysis.

OPTIMIZATION PROBLEM

The objective of the design problem is to maximize the fundamental frequency of the one- and two-folded laminated composite plates. The fibre orientation is taken as design variable. No behavioral constraints are specified. The optimization of the problem is formulated as

Maximise: $\omega_1 = \omega_1(\theta)$

Subjected to: $0^o \le \theta_k \le 90^o$ (7)

The fundamental frequency ω_1 for a given fibre orientation is determined from the finite element solution of the eigenvalue problems given by Eq. (7). The optimization procedure involves the stages of evaluating the fundamental frequency and improving the fiber orientation θ to maximize ω_1. Thus, the computational solution consists of successive stages of analysis and optimization until a convergence is obtained and the optimal angle θ_{opt} is determined within a specified accuracy. The modified feasible direction method (MFD) is used in the optimization stage. The flow chart of this method is given in Figure 2.

MFD

1. *q=0,* $X^q = X^m$
2. *q=q+1*
3. Evaluate objective function $F(X_i)$
4. Calculate gradient of the objective function $F(X_i)$
5. Find the usable-feasible direction S_q
6. Perform a one-dimensional search
   $X^q = X^{q-1} + \alpha\, S_q$
7. Check convergence. If satisfied, go to 8. Otherwise go to 2
8. $X^{m+1} = X^q$

Fig. 2. The flow chart of modified feasible direction method.

NUMERICAL RESULTS AND DISCUSSION

Firstly, the accuracy of the present finite element formulation is evaluated for laminated plates without mass with the available literature results. The dimensionless frequency is defined as

$$\varpi = (\omega b^2 / h)(\rho / E_2)^{1/2} \quad (8)$$

The dimensionless material properties of the lamina are as follows:

$E_1 / E_2 = 40$, $G_{12} / E_2 = 0,6$, $G_{13} / E_2 = G_{23} / E_2 = 0,5$, $\upsilon_{12} = 0,25$

In Table 1, the dimensionless natural frequencies of a four-layered simply supported square composite plate are presented and compared with the literature results. As seen,

the results obtained from present study are good agreement with those of literature results. The discrepancy is due to the different procedures used by the authors.

Table 1. Dimensionless first natural frequency for $[45/-45]_2$ with various b/h ratios.

b/h		ϖ
20	Reddy, (1979)	22,58
	Bert and Chen, (1978)	21,87
	Shankara and Iyengar, (1996)	21,68
	Present study	22,81
30	Reddy, (1979)	23,68
	Bert and Chen, (1978)	22,74
	Shankara and Iyengar, (1996)	22,69
	Present study	23,72
40	Reddy, (1979)	24,12
	Bert and Chen, (1978)	23,08
	Shankara and Iyengar, (1996)	23,09
	Present study	24,02

OPTIMIZATION RESULTS

Consider a multi-layered laminate made of T300/5208 graphite/epoxy layers. The material properties are given as $E_1 = 181\text{GPa}$, $E_2 = 10,3\text{GPa}$, $G_{12} = 7,17\text{GPa}$, $\nu_{12} = 0,28$, ρ =1600 kg/m^3. The four layered symmetric angle ply $[\theta/-\theta/-\theta/\theta]$ laminated plate is constructed of equal thickness.

Consider a laminated plate with distributed patch mass acted on it in a square area (c=d) at the center of the plate. In Figure 3, the effect of mass ratio M/M_p (M= distributed patch mass's mass, $M_p =$ plate's mass) on the maximum fundamental frequency is given (b/h=20). According to this figure, by increasing the amout of the local mass, the stiffness of the plate decreases and causes to decrease the maximum fundamental frequencies. But, this decrease in the maximum fundamental frequency decreases as the mass ratio increases. For example, the changes in the dimensionless

frequency are 13,37%, 7,67% and 5,91% for M/M_p =0,1, M/M_p =0,4 and M/M_p =0,6, respectively. On the other hand, the optimum fibre orientation ,θ_{opt}, is obtained 45° for all mass ratios. As seen, the mass ratio has no any effect on the optimum fibre orientations.

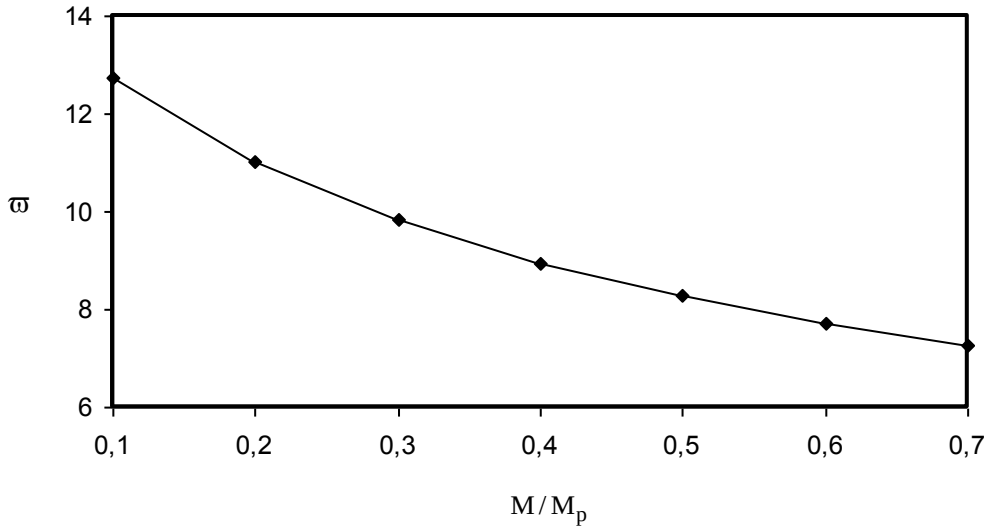
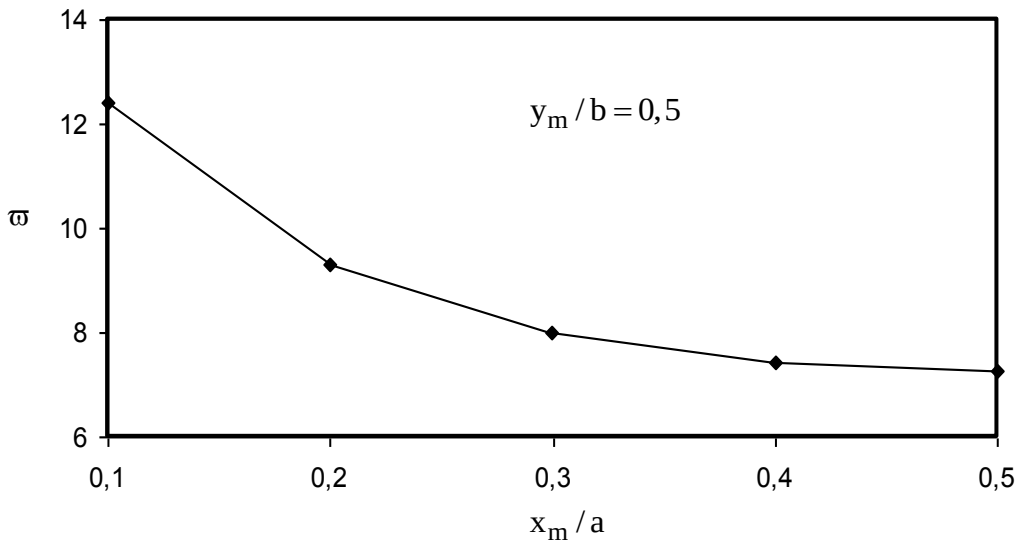


Fig. 3. Effect of mass ratio on the maximum fundamental frequency.

In Figure 4a and 4b, the influences of the x- and y- positions of the patch mass on the maximum fundamental frequency are given for square laminated plates, respectively (M/M_p =0,7, b/h=20). As seen from Figure 4a, increasing the distance between the local mass and x=0 edge of the plate causes to decrease the stiffness of plate, and consequently causes to decrease the maximum fundamental frequencies. Figure 4b presents the same results for different y-positions of the mass. Also as expected, the same results are obtained for x- and y- positions because of the symmetry. In Table 2 and Table 3, the optimum fibre orientations are given for x- and y- positions of the patch mass.



(a)

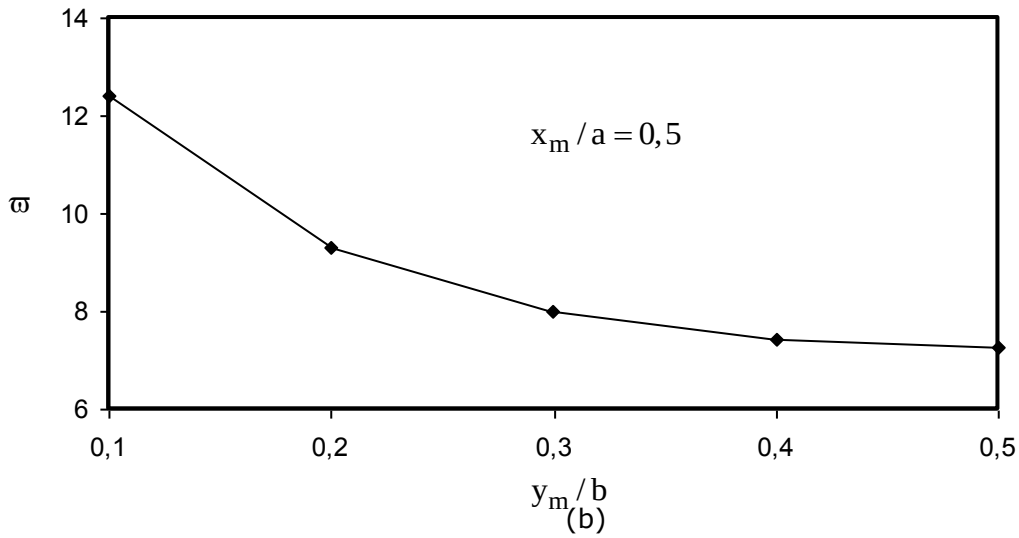


Fig. 4. Effect of the x- (a) and y- (b) positions of the patch mass on the maximum fundamental frequency.

Table 2. Effect of x- position of the patch mass on the optimum fibre orientations ($y_m / b = 0,5$).

x_m / a	$\theta_{opt}(^\circ)$
0,1	41,1
0,2	40,5
0,3	43,0
0,4	44,5
0,5	45,0

Table 3. Effect of y- position of the patch mass on the optimum fibre orientations ($x_m / a = 0,5$).

y_m / b	$\theta_{opt}(^\circ)$
0,1	48,9
0,2	49,5
0,3	47,0
0,4	45,5
0,5	45,0

In Figure 5, effect of plate aspect ratio (a/b) on the maximum fundamental frequency is illustrated for laminated plates (M/M_p =0,7, b/h=20, x_m / a =0.5, y_m / b =0.5). As seen from the figure, as a/b ratio increases, the maximum fundamental frequency decreases. But, the influence of the aspect ratio on the maximum fundamental frequency for the plate with patch mass is less than for the plate without patch mass (Topal and Uzman, (2006)). This means that the stiffness of the laminated plates with patch mass is not changed substantially by the larger plate aspect ratios. The optimum fibre orientations

are given in Table 4 for a/b ratios. As seen, the optimum fibre orientation increases with increase in the a/b ratio.

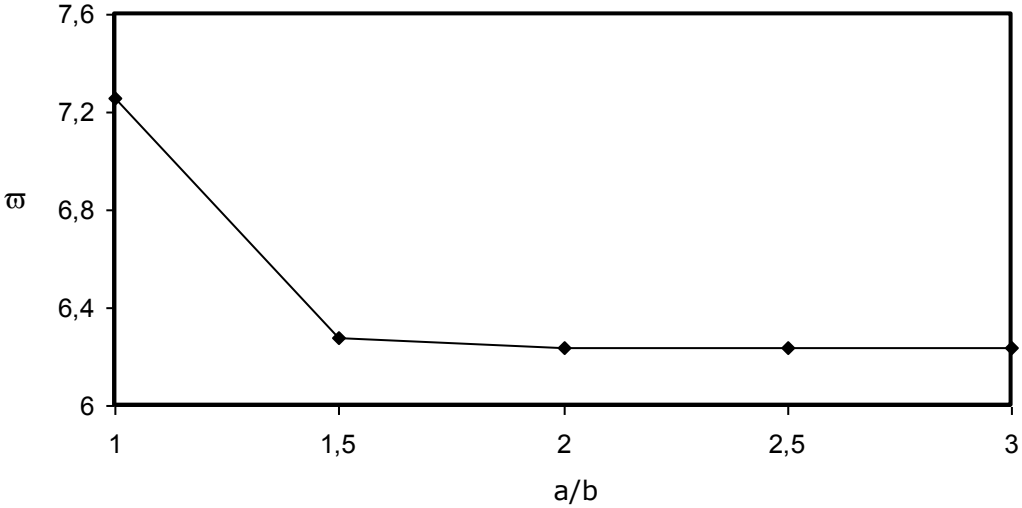


Fig. 5. Effect of plate aspect ratio on the maximum fundamental frequency.

Table 4. Effect of plate aspect ratio on the optimum fibre orientations.

a / b	$\theta_{opt}(^{\circ})$
1	45,0
1,5	66,2
2	73,7
2,5	74,2
3	74,3

CONCLUSIONS

This paper investigates effect of distributed patch mass on the frequency optimization of simply supported symmetric angle-ply laminated plates. The objective function is the maximization of the fundamental frequency of the laminates and the design variable is the fibre orientation. The effects of mass ratio, location of distributed patch mass and aspect ratio on the optimum results are investigated. As the results of this research, the following conclusions are obtained:

- As the mass ratio increases the fundamental frequency decreases. But, the mass ratio has no any effect on the optimum fibre orientations.
- As the distance between the local mass and x=0 or y=0 edges of the plate increases the maximum fundamental frequency decreases.
- The influence of the aspect ratio on the maximum fundamental frequency for the plate with patch mass is less than for the plate without patch mass. The optimum fibre orientation increases with increase in the a/b ratio.
- This paper can be investigated for different c/d ratios.

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EXPERIMENTAL AND ANALYTICAL MODAL ANALYSES OF A PROTOTYPE ARCH DAM-RESERVOIR-FOUNDATION MODEL

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Key Words: Analytical modal analysis, Dynamic characteristics, Experimental modal analysis, Prototype arch dam-reservoir-foundation model.

ABSTRACT In this study, dynamic characteristics of a prototype arch dam-reservoir-foundation model are determined using experimental and analytical modal analyses. For this aim, Type-1 arch dam presented in the literature is selected. And this arch dam is constructed in a laboratory with 6cm crest width and 60cm height considering reservoir and foundation. In experimental modal analysis, Type-1 arch dam is excited by an impact hammer and the frequency response functions are measured. To obtain frequency response functions of Type-1 arch dam, five sensitive accelerometers are used. These accelerometers are placed to crest of the arch dam. Measurements are done for empty and full reservoir of the arch dam. In analytical modal analysis, Type-1 arch dam-reservoir-foundation system is modeled using ANSYS software, and natural frequencies and mode shapes are determined for empty and full reservoir. It is stated from the study that, natural frequencies obtained from experimental and analytical modal analyses are almost closed the each other. In addition, the natural frequencies of the arch dam are changed approximately 20 % for empty and full reservoir cases.

INTRODUCTION

An arch dam is a solid concrete hydraulic structure curved in plan and possibly in elevation, which transmits a large portion of the water pressure and other loads by means of thrust (arch action) to the abutment utilizing the compressive strength of its material (USACE, 2003). Therefore, arch dams are required sophisticated engineering knowledge to design and construct. While arch dams have multiple benefits (balanced against financial and environmental costs), they can also present a risk to public safety and economic infrastructure. This risk happens from two sources: the possibility of a dam failure and the damage it would cause. Reasons such as age, construction deficiencies, inadequate maintenance, and seismic or weather events could be failure of dams (Nic, 2008). Especially, the failure of dams retaining large quantities of water poses a serious threat for life and property during earthquakes. Interaction between the dam and impounded water is an important factor affecting the dynamic response of arch dams during earthquakes. When an arch dam-reservoir system is subjected to a dynamic effect such as earthquake, hydrodynamic pressures in excess of hydrostatic pressures occur on the dam due to the vibration of the dam and water in the reservoir (Akköse et al, 2007). To reduce these risks, regular inspections are necessary to identify potential problems. One of the important inspections is to apply Experimental Modal Testing to existing dams to estimate their structural dynamic properties.

Experimental Modal Analysis (EMA) is one of the reliable methods to evaluate a structure's vibration properties. In this method, dynamic behavior of a structure is estimated through a number of natural frequencies obtained from Frequency Response Functions (FRF). In fact, the performance of numerical methods used to evaluate the seismic behavior of dams can only be truly assessed by correlation with experimental results obtained on existing structures. Experimental investigations such as forced vibration tests are extremely useful to this end (Ljung, 1987). In EMA; the structure is

excited by known input force (such as impulse hammer, and electrodynamics shaker) and responses of the structure are measured.

Last twenty years, some studies have been done to investigate the dynamic behavior of large dams using Experimental Modal Analysis. In these studies it is highlighted that although the physical conditions are hard as difficulties to excite of large dams, and test become too expensive; results obtained from EMA are very attractive and useful (Duron and Hall, 1988; Darbre, 1995; Cantieni, 2001). Such kind of conditions made to researchers to construct scaled prototype dam models to study experimentally (Wang and Li, 2006; Wang and Li, 2007, Mendes and Oliveira, 2007). However, when constructing scaled dam models it is very important to take great care in due to scaled models represent the real dam.

In this study, dynamic characteristics of a prototype arch dam-reservoir-foundation model are determined by experimental and analytical modal analyses. For this purpose, a prototype concrete arch dam-reservoir-foundation model is constructed in the laboratory conditions. Experimental and analytical modal analyses of the arch dam are performed to identify dynamic characteristics for empty and full reservoirs. And, experimental and analytical frequencies are compared to each other.

FORMULATION

In this study, the basic formulations of EMA and Lagrangian approach are given.

EXPERIMENTAL MODAL ANALYSIS

The Frequency Response Function (FRF) is a fundamental measurement that isolates the inherent dynamic properties of a structure. Natural frequencies, mode shapes and damping ratios are obtained from a set of FRF measurements. The FRF describes the input-output relationship between two points on a structure as a function of frequency. Since both force and motion are vector quantities, they have directions associated with them. Therefore, an FRF is actually defined between a single input DOF (point and direction), and a single output DOF (Schwarz and Richardson, 1999).

In EMA method, the structure is artificially excited with a forcing function in a point i and its response $Y_k(t)$ to this excitation is measured together with the forcing signal $X_i(t)$ (Figure 1). Transformation of these time signals into the frequency domain and calculation of all Frequency Response Functions (FRF) H_{ik} between the response and the forcing function time signals yields the Frequency Response Matrix, also referred to as Transfer Matrix, H(iw) (Figure 2) (Cantieni, 2004).

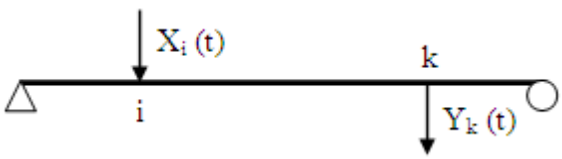


Fig. 1. EMA Scheme (Cantieni, 2004).

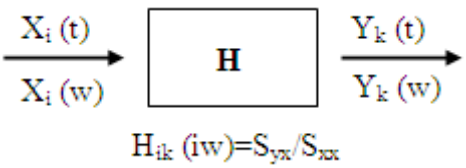


Fig. 2. Calculation of the FRF H_{ik} (Cantieni, 2004).

In this study, FRF measurements of the arch dam are obtained from impact hammer testing. Because, impact hammer testing is a fast, convenient, and cheap method to find

the modes of structures (Schwarz and Richardson, 1999). Also, in small and medium size structures, the excitation can be induced by an impulse hammer. But the selection of the hammer type can have a significant effect on the measurement acquired. The input excitation frequency range is controlled mainly by the hardness of the type selected (Avitabile, 2001). The schematic view of the impact hammer testing is shown in Figure 3 (Schwarz and Richardson, 1999).

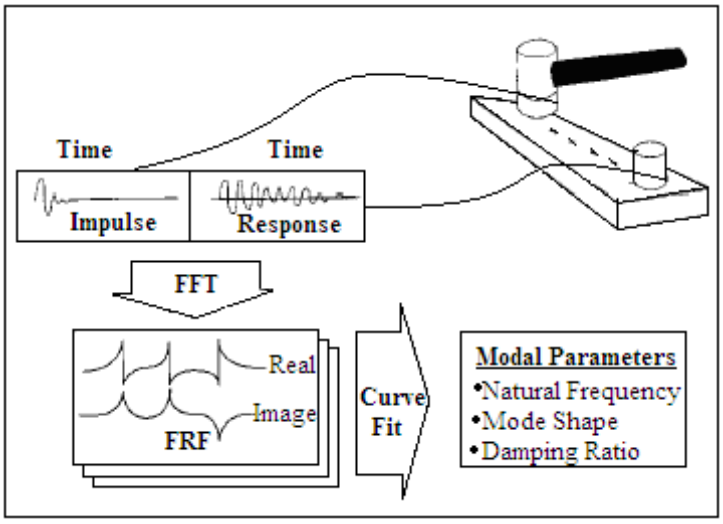


Fig. 3. The schematic views of the impact hammer testing (Schwarz and Richardson, 1999).

FLUID AND FLUID-STRUCTURE SYSTEMS BY LAGRANGIAN APPROACH

The formulation of the fluid system based on Lagrangian approach is given according to references (Wilson and Khalvati, 1983; Calayır, 1994). In this approach, fluid is assumed to be linearly elastic, inviscid and irrotational. For a general three-dimensional fluid, stress-strain relationships can be written in matrix form as follows,

$$\begin{Bmatrix} P \\ P_x \\ P_y \\ P_z \end{Bmatrix} = \begin{bmatrix} C_{11} & 0 & 0 & 0 \\ 0 & C_{22} & 0 & 0 \\ 0 & 0 & C_{33} & 0 \\ 0 & 0 & 0 & C_{44} \end{bmatrix} \begin{Bmatrix} \varepsilon_v \\ w_x \\ w_y \\ w_z \end{Bmatrix} \tag{1}$$

where P, C_{11}, and ε_v are the pressures, which are equal to mean stresses, the bulk modulus and the volumetric strains of the fluid, respectively. Since irrotationality of the fluid is considered like penalty methods (Bathe, 1996), rotations and constraint parameters are included in the stress-strain equation (Eq. 1) of the fluid. In this equation, P_x, P_y, P_z are the rotational stresses; C_{22}, C_{33}, C_{44} are the constraint parameters and W_x, W_y and W_z are the rotations about the cartesian axis x, y and z, respectively.

In this study, the equations of motion of the fluid system were obtained using energy principles. Using the finite element approximation, the total strain energy of the fluid system may be written as,

$$\pi_e = \frac{1}{2}\mathbf{U}_f^T \mathbf{K}_f \mathbf{U}_f \tag{2}$$

where $\mathbf{U}_f$ and $\mathbf{K}_f$ are the nodal displacement vector and the stiffness matrix of the fluid system, respectively. $\mathbf{K}_f$ is obtained by the sum of the stiffness matrices of the fluid elements in the following,

$$\left.\begin{aligned} \mathbf{K}_f &= \sum \mathbf{K}_f^e \\ \mathbf{K}_f^e &= \int_V \mathbf{B}_f^{eT} \mathbf{C}_f \mathbf{B}_f^e dV^e \end{aligned}\right\} \tag{3}$$

where $\mathbf{C}_f$ is the elasticity matrix consisting of diagonal terms in Eq. (2). $\mathbf{B}_f^e$ is the strain-displacement matrix of the fluid element.

An important behavior of fluid systems is the ability to displace without a change in volume. For reservoir and storage tanks, this movement is known as sloshing waves in which the displacement is in the vertical direction. The increase in the potential energy of the system due to the free surface motion using finite element method can be written as,

$$\pi_s = \frac{1}{2} \mathbf{U}_{sf}^T \mathbf{S}_f \mathbf{U}_{sf} \tag{4}$$

where $\mathbf{U}_{sf}$ and $\mathbf{S}_f$ are the vertical nodal displacement vector and the stiffness matrix of the free surface of the fluid system, respectively. $\mathbf{S}_f$ is obtained by the sum of the stiffness matrices of the free surface fluid elements in the following,

$$\left.\begin{aligned} \mathbf{S}_f &= \sum \mathbf{S}_f^e \\ \mathbf{S}_f^e &= \rho_f g \int_A \mathbf{h}_s^T \mathbf{h}_s dA^e \end{aligned}\right\} \tag{5}$$

where $\mathbf{h}_s$ is the vector consisting of interpolation functions of the free surface fluid element. ρf and g are the mass density of the fluid and the acceleration due to gravity, respectively. Also, kinetic energy of the system using finite element method can be written as,

$$T = \frac{1}{2} \dot{\mathbf{U}}_f^T \mathbf{M}_f \dot{\mathbf{U}}_f \tag{6}$$

where $\dot{\mathbf{U}}_f$ and $\mathbf{M}_f$ are the nodal velocity vector and the mass matrix of the fluid system, respectively. $\mathbf{M}_f$ is also obtained by the sum of the mass matrices of the fluid elements in the following,

$$\left.\begin{aligned} \mathbf{M}_f &= \sum \mathbf{M}_f^e \\ \mathbf{M}_f^e &= \rho_f \int_V \mathbf{H}^T \mathbf{H} dV^e \end{aligned}\right\} \tag{7}$$

where $\mathbf{H}$ is the matrix consisting of interpolation functions of the fluid element. If Eqs. (3, 5 and 7) are combined using the Lagrange's equation (Clough and Penzien, 1993), the following set of equations is obtained,

$$\mathbf{M}_f \ddot{\mathbf{U}}_f + \mathbf{K}_f^* \mathbf{U}_f = \mathbf{R}_f \tag{8}$$

where $\mathbf{K}_f^*$, $\ddot{\mathbf{U}}_f$ and $\mathbf{R}_f$ are the system stiffness matrix including the free surface stiffness, the nodal acceleration vector and time-varying nodal force vector for the fluid system, respectively. In the formation of the fluid element matrices, reduced integration orders were utilized. For the eight-noded three-dimensional fluid element, reduced integration order is chosen as (1x1x1) (Wilson and Khalvati, 1983).

The equations of motion of the fluid system (Eq. (8)), have a similar form with those of the structure system when Lagrangian approach is considered in the formulations. To obtain the coupled equations of the fluid-structure system, the determination of the interface condition is required. Because the fluid is assumed to be inviscid, only the displacement in the normal direction to the interface is continuous at the interface of the system. Assuming that the positive face is the structure and the negative face is the fluid, the boundary condition at the fluid-structure interface is,

$$\mathbf{U}_n^- = \mathbf{U}_n^+ \tag{9}$$

where $\mathbf{U}$ is the normal component of the interface displacement (Akkaş et al., 1979). Using the interface condition, the equations of motion of the coupled system to ground motion including damping effects are given by,

$$\mathbf{M}_c\ddot{\mathbf{U}}_c + \mathbf{C}_c\dot{\mathbf{U}}_c + \mathbf{K}_c\mathbf{U}_c = \mathbf{R}_c \tag{10}$$

in which $\mathbf{M}_c$, $\mathbf{C}_c$, and $\mathbf{K}_c$ are the mass, damping and stiffness matrices for the coupled system and $\mathbf{U}_c$, $\dot{\mathbf{U}}_c$, $\ddot{\mathbf{U}}_c$ and $\mathbf{R}_c$ are the vectors of the displacements, velocities, accelerations and external loads of the coupled system, respectively.

EXPERIMENTAL AND NUMERICAL APPLICATION

DESCRIPTION OF TYPE-1 ARCH DAM

In this study, Type-1 arch dam, which is one of the five arch dam types suggested in "Arch Dams" Symposium in England in 1968 (Arch Dams, 1968), is selected for the experimental and numerical application. Type-1 arch dam has the simplest geometry presented of the symposium. The geometrical properties of Type-1 arch dam are shown in Figure 4. As it is seen from Figure 4, Type-1 arch dam has a single curvature, constant radius and constant central angle. In addition, it has 6 unit height and 0.6 unit constant widths. To construct a prototype model, 1 unit is selected as 10cm. Therefore, dam height and crest width are designed as 60cm and 6cm, respectively.

3D finite element model of Type-1 arch dam is constituted by ANSYS (2008) software. Type-1 arch dam is developed considering reservoir and foundation. Reservoir of the dam is extended as 3H (H: dam height); foundation of the dam is extended as about H in the downstream and downward directions, and is extended as reservoir length in upstream direction. Such kind of modeling is appropriate to represent the dynamic behavior of concrete dams (USACE, 2003). Definitive dimensions of Type-1 arch dam-reservoir-foundation system are shown in Figure 5.

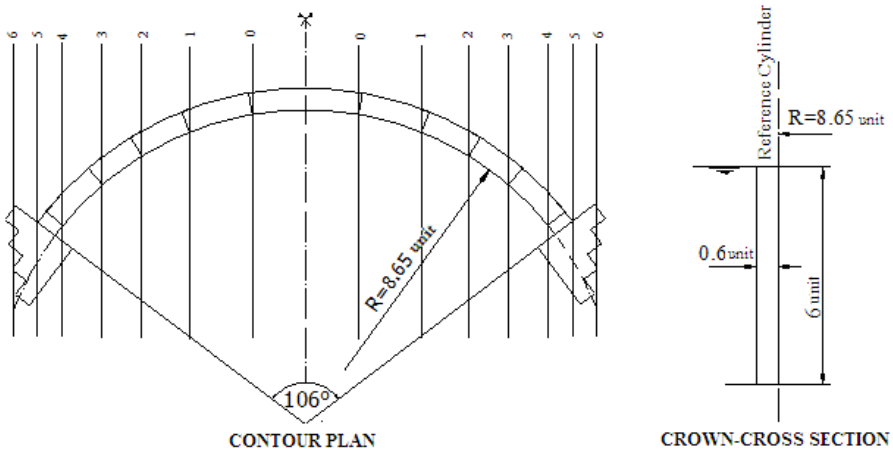


Fig. 4. Geometrical properties of Type-1 arch dam.

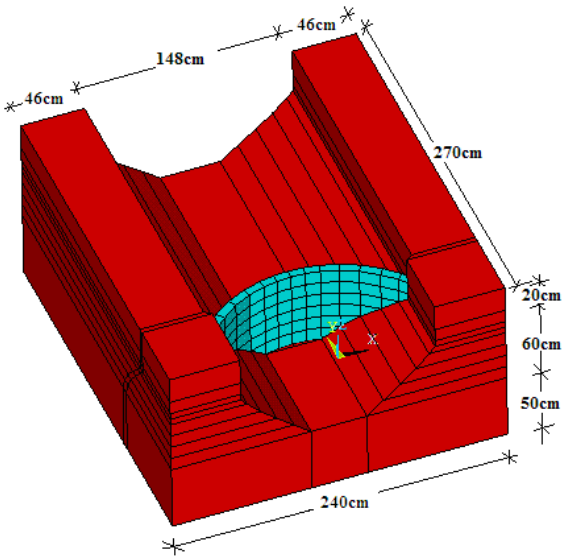


Fig. 5. 3D model of Type-1 arch dam-reservoir-foundation system and its dimensions.

Type-1 arch dam-reservoir-foundation system is constructed considering dimensions given in Figure 5. In the construction of the system, 6.5m^3 concrete are used approximately. Model is constructed working five days by three workers. Some pictures of the system are shown in Figure 6.

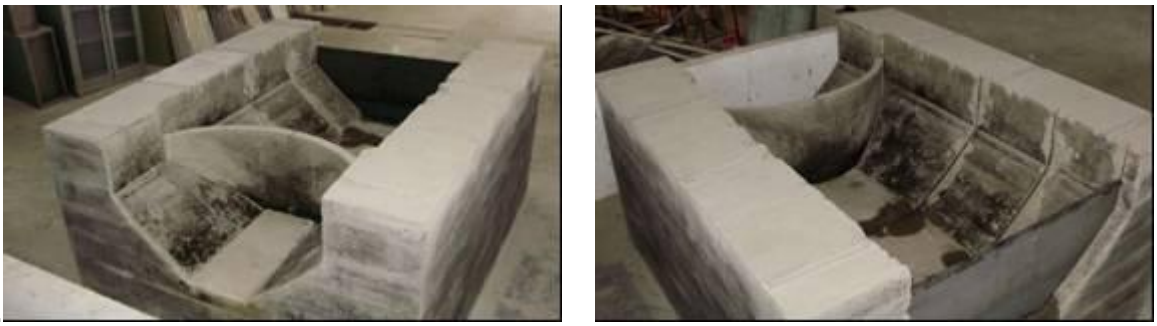


Fig. 6. Some pictures of Type-1 arch dam-reservoir-foundation system.

EXPERIMENTAL MODAL ANALYSIS

In experimental modal analyses, B&K 3560 data acquisition system with 17 channels, B&K 4507-B005 type uni-axial accelerometers and B&K 8206-002 type impact hammer are used as test equipment. The hammer has 2.27mV/N sensitivity and 4500N maximum force. Linear averaging method is selected during the tests and number of the samples is selected 5, in addition frequency span is selected 0-1600Hz. Tests are conducted to dam after 50 days of pouring concrete. FRFs of the measurements are obtained from PULSE (2006) software. In EMA, only the natural frequencies are determined due to optional tools are not found in the test equipment to determine mode shapes and damping ratios.

EMA tests are conducted to dam for empty and full reservoir. During the each of test, five accelerometers are located along to crest of the dam as seen in Figure 7. Some pictures from the tests related to empty and full reservoir given in Figure 8. Frequency responses of the input related to each case are shown in Figures 9 and 10. The natural frequencies of Type-1 arch dam obtained from the impact hammer testing related to empty and full reservoir are given in Table 1. It can be seen from Table 1, differences between empty and full reservoir are about 20%. Since the water is affected to behavior of dam, higher frequencies are not obtained. It means that the water averts the vibrations of the dam for higher modes.

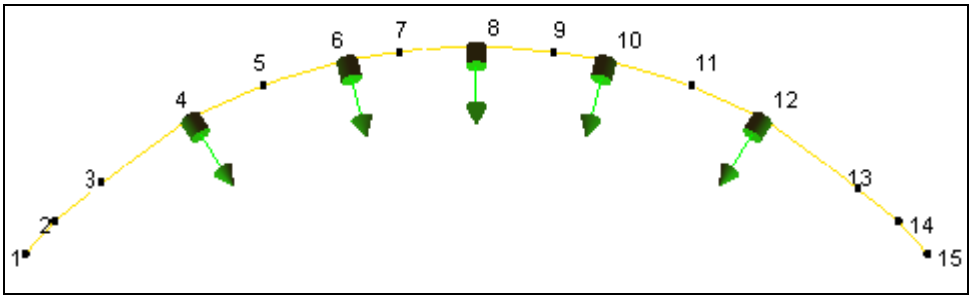
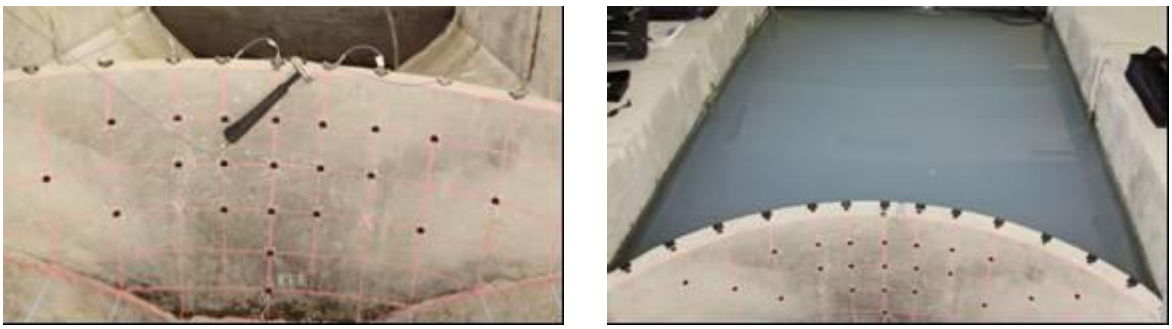
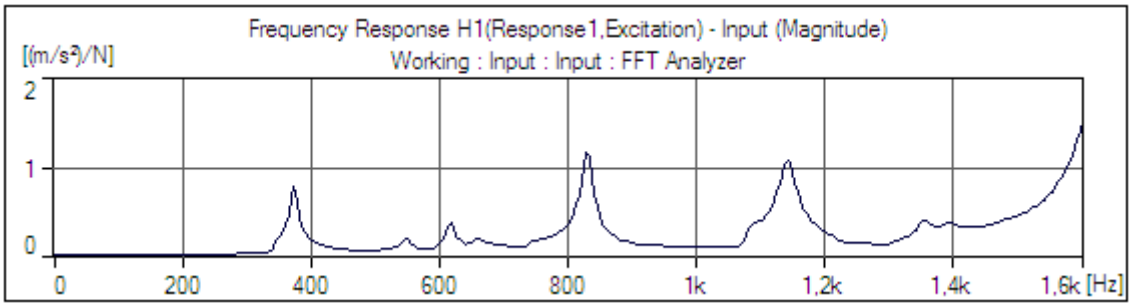


Fig. 7. Locations of accelerometers used in EMA tests.



a) Empty Reservoir b) Full Reservoir

Fig. 8. Some pictures related to (a) empty and (b) full reservoir tests.



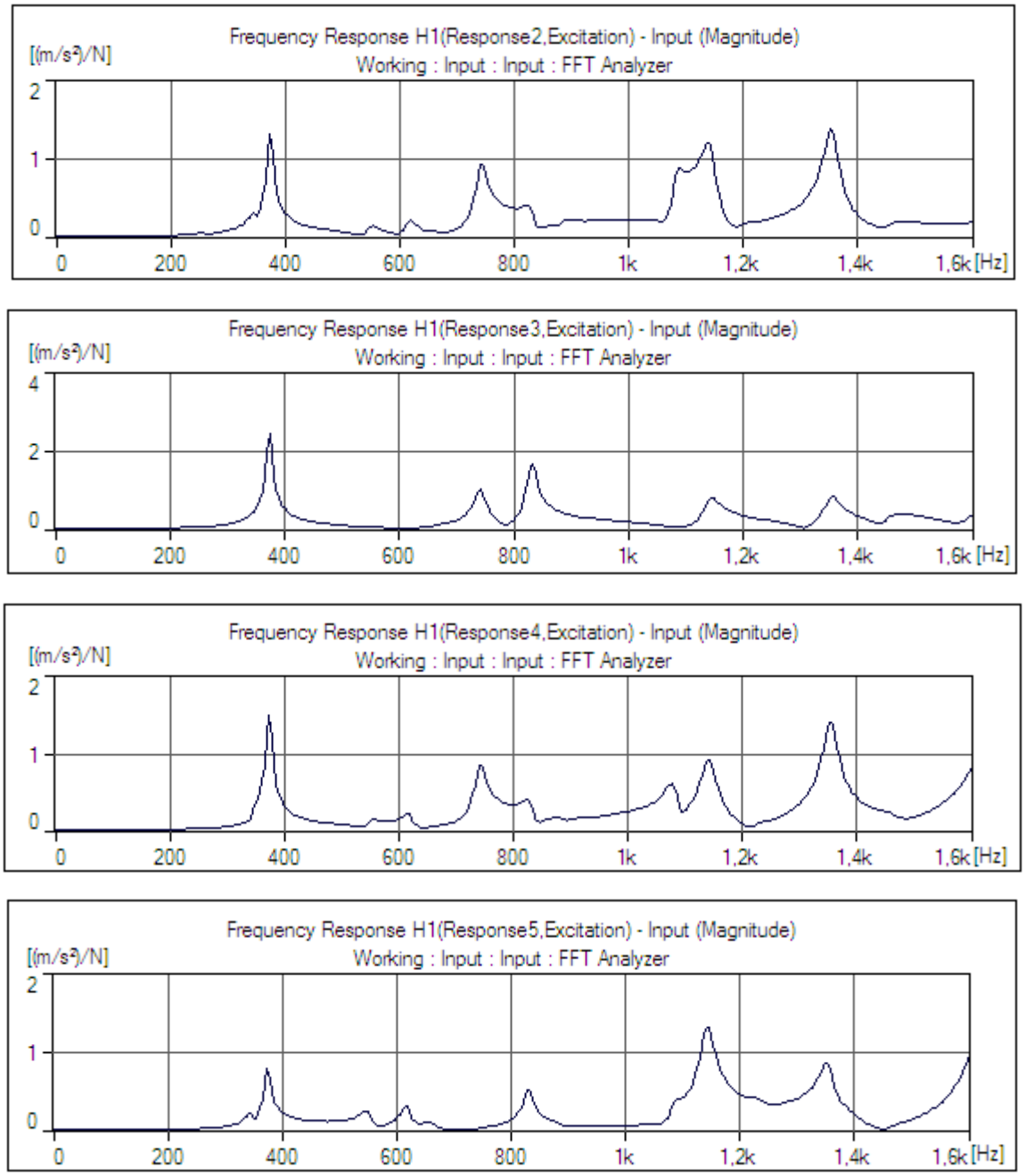
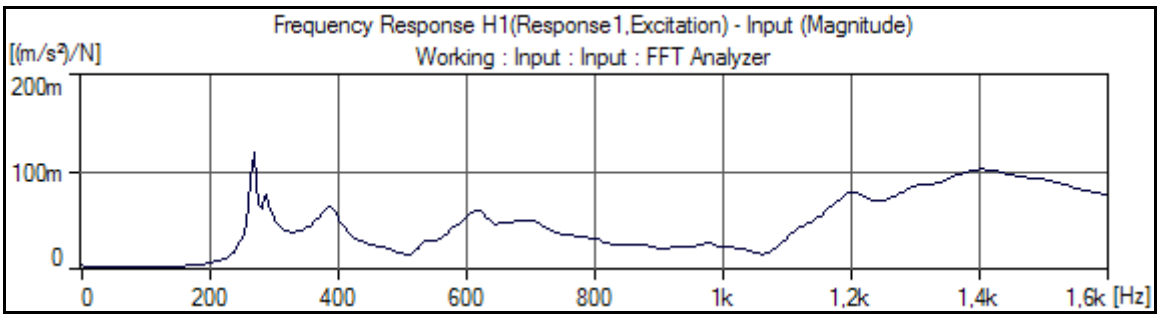


Fig. 9. Frequency responses of the input related to empty reservoir.



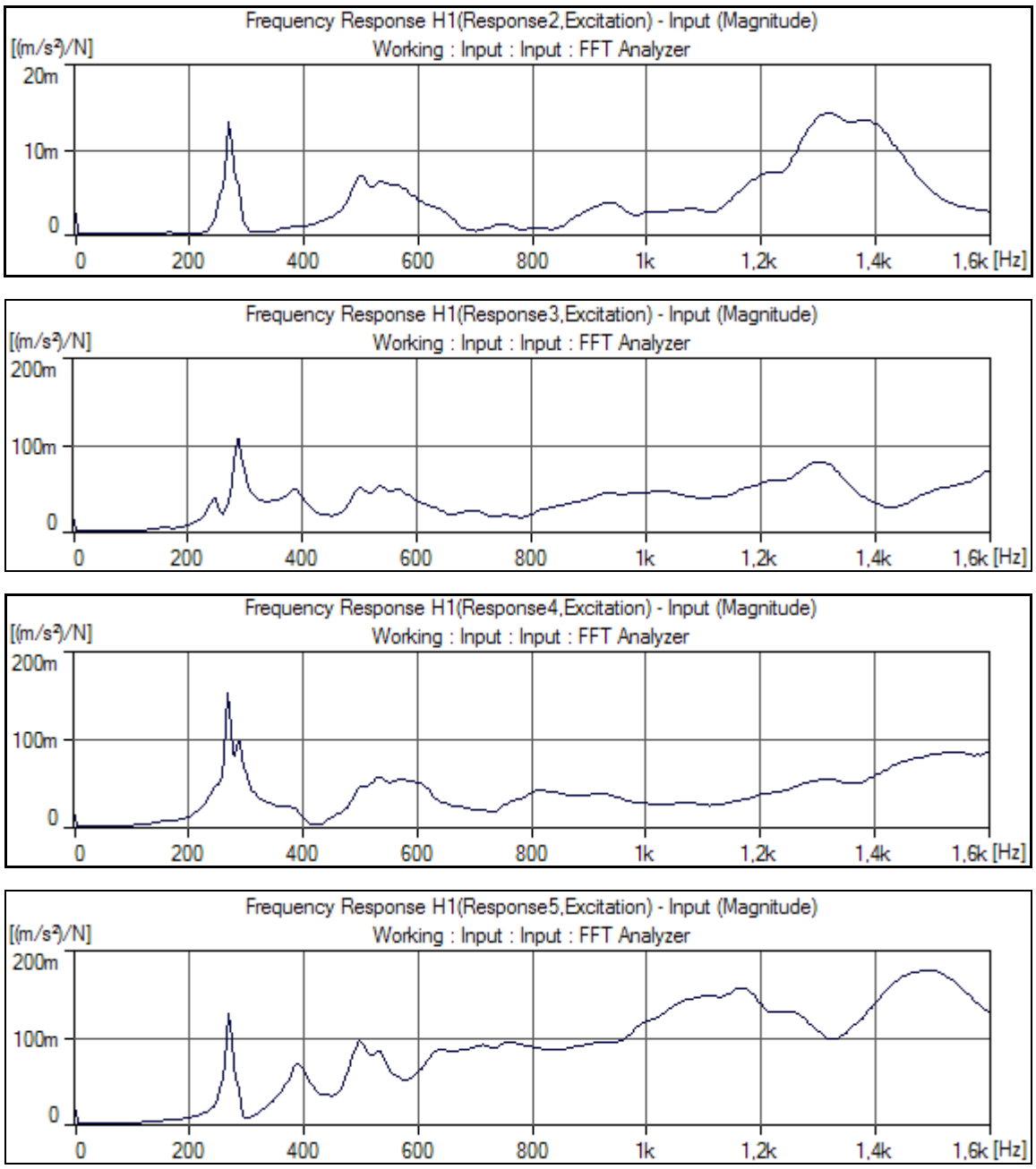


Fig. 10. Frequency responses of the input related to full reservoir.

Table 1. Experimental natural frequencies for empty and full reservoirs.

Mode	Empty Reservoir Frequency (Hz)	Full Reservoir Frequency (Hz)	Difference (%)
1	340	268	21
2	372	288	22
3	552	384	26
4	616	-	-
5	740	-	-
6	828	-	-
7	876	-	-
8	1140	-	-

ANALYTICAL MODAL ANALYSIS

The ANSYS (2008) finite element program is used to obtain dynamic characteristics of Type-1 arch dam, analytically. 3D finite element model of the dam is constituted using geometrical properties and dimensions given in Figures 4 and 5, respectively. The dam is modeled for empty and full reservoir water lengths. In the finite element model, solid elements for the dam and foundation, and fluid elements for the reservoir water are used. Both solid and fluid elements have 8-nodes and each node has three degree of freedom: x, y, z translations. The element types and material properties used in the analysis are summarized in Table 2. Material properties of Type-1 arch dam are determined by experimental methods such as uni-axial compression tests and ultrasonic velocity tests. To represent the interaction between the reservoir-dam and reservoir-foundation, coupled elements which have 0.001m length are used. Main objective of the coupled elements are hold equal the normal displacements between two reciprocal nodes.

Table 2. Material properties of dam-reservoir-foundation system.

	ANSYS Element Type	Material Properties		
		Elasticity Modulus (MPa)	Poisson Ratio	Mass Density (kg/m^3)
Dam	Solid 45	15500	0.2	2300
Reservoir	Fluid 80	2070	-	1000
Foundation	Solid 45	20000	0.2	2300

EMPTY RESERVOIR

3D Finite Element Model (FEM) of Type-1 arch dam for empty reservoir is shown in Figure 11. In this model, 2732 finite elements are used: 148 solid elements for the dam and 2584 solid elements for the foundation. First six natural frequencies and mode shapes of Type-1 arch dam are shown in Figure 12. Mode shapes are obtained as symmetrical, anti-symmetrical and vertical.

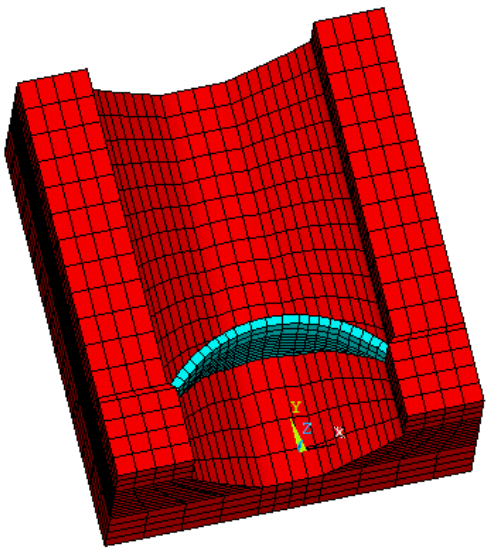
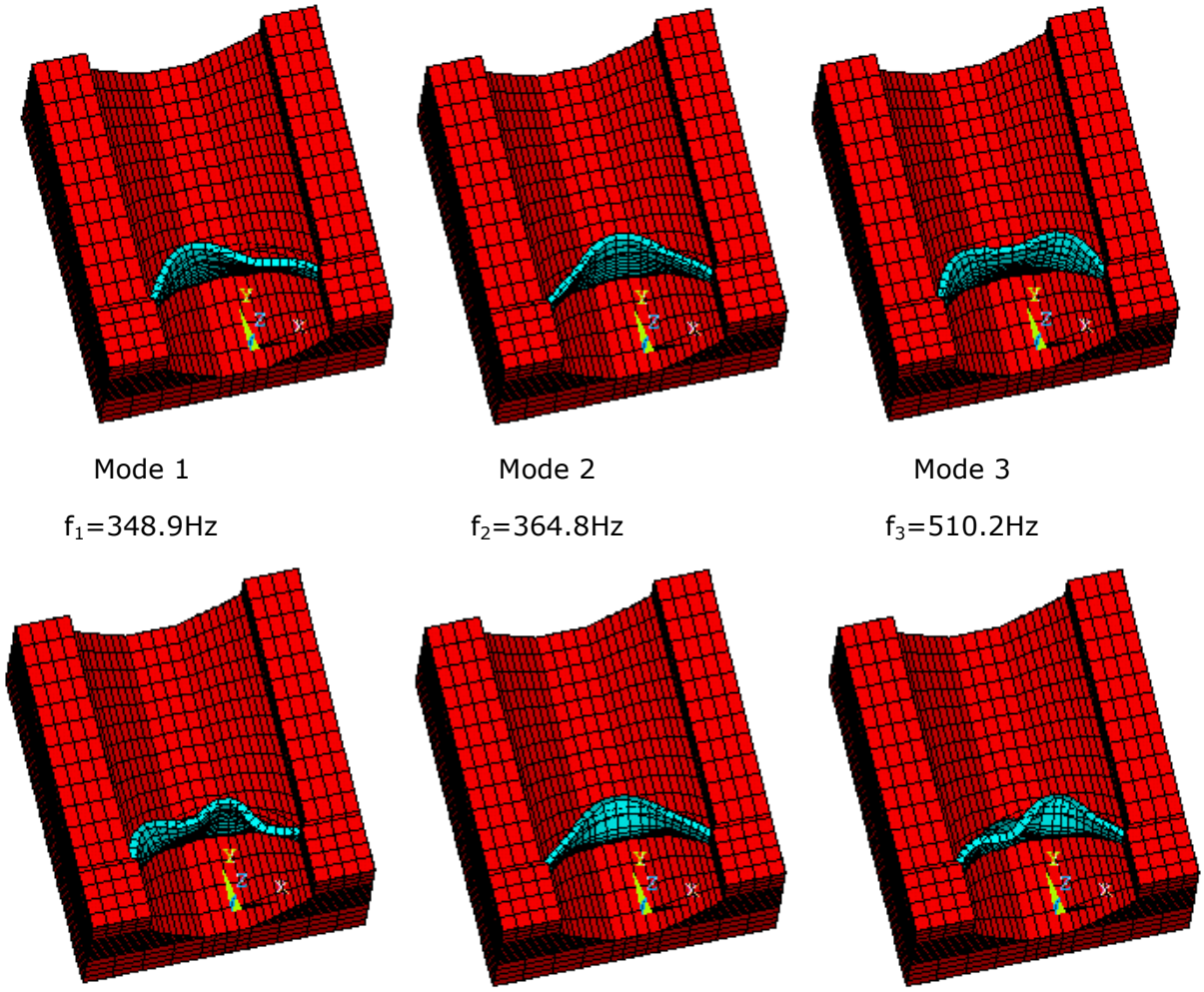


Fig. 11. 3D FEM of Type-1 arch dam for empty reservoir.



Mode 1 f_1=348.9Hz

Mode 2 f_2=364.8Hz

Mode 3 f_3=510.2Hz

Mode 4 f_4=658.5Hz

Mode 5 f_5=680Hz

Mode 6 f_6=873.7Hz

Fig. 12. Mode shapes of Type-1 arch dam for empty reservoir.

FULL RESERVOIR

3D finite element model of Type-1 arch dam for full reservoir is shown in Figure 13. In this model, 4508 finite elements are used: 148 solid elements for the dam, 2584 solid elements for the foundation and 1776 fluid elements for the reservoir. The first three natural frequencies and mode shape**s** of Type-1 arch dam are shown in Figure 14.

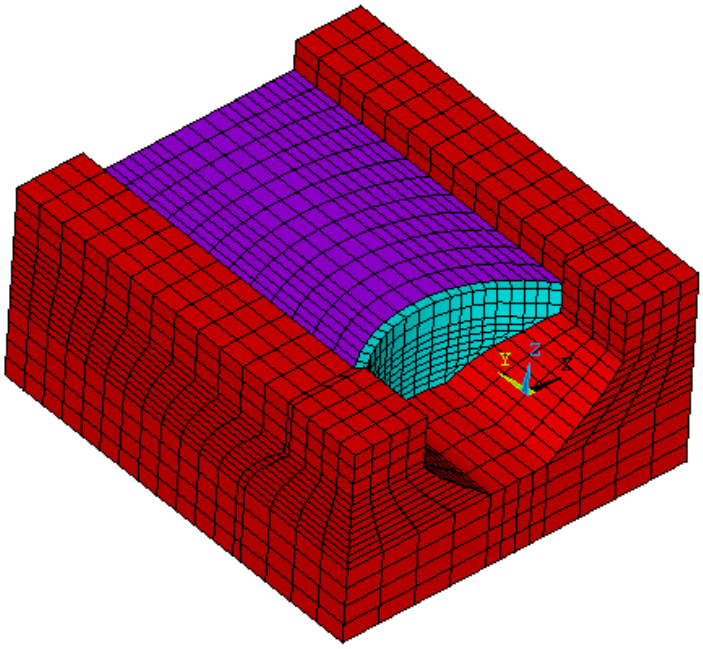
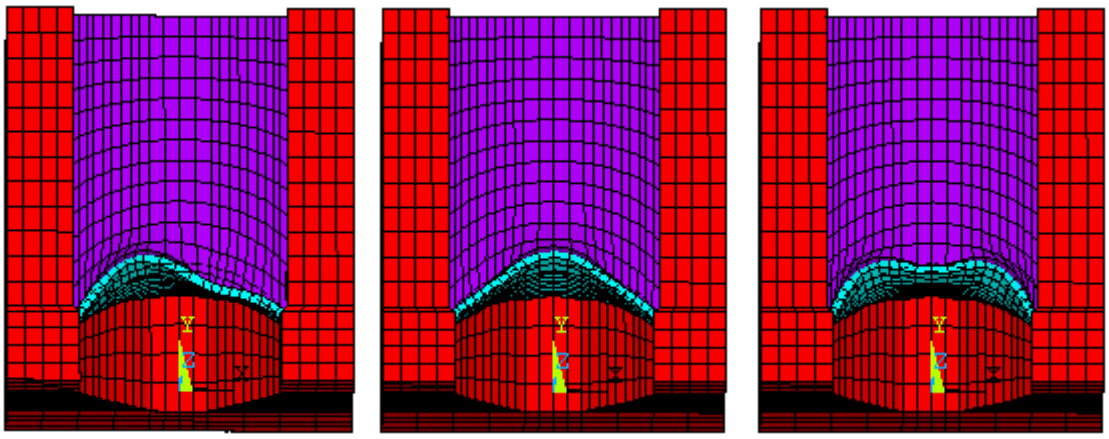


Fig. 13. 3D FEM of Type-1 arch dam for full reservoir.



Mode 1 Mode 2 Mode 3

f_1=250.8Hz f_2=252.9Hz f_3=371.4Hz

Fig. 14. Mode shapes of Type-1 arch dam for full reservoir.

COMPARISON OF EXPERIMENTAL AND ANALYTICAL FREQUENCIES

Experimentally and analytically identified natural frequencies are compared to each other for empty and full reservoir (Table 3). As it can be seen from Table 3, experimental and analytical frequencies are near to each other. Also the natural frequencies obtained for empty reservoir are approximately 20% bigger than those of full reservoir.

Table 3. Experimental and analytical natural frequencies for empty and full reservoirs.

Mode	Empty Reservoir		Full Reservoir	
	Experimental	Analytical	Experimental	Analytical

	Frequencies (Hz)	Frequencies (Hz)	Frequencies (Hz)	Frequencies (Hz)
1	340	348.9	268	250.8
2	372	364.8	288	252.9
3	552	510.2	384	371.4
4	616	658.5	-	-
5	740	680.0	-	-
8	828	873.7	-	-

CONCLUSIONS

In this study, dynamic characteristics of a prototype arch dam-reservoir-foundation system are determined using experimental and analytical modal analyses. For this aim, Type-1 arch dam presented in the literature is selected, and the arch dam is constructed in a laboratory considering reservoir and foundation. Impact hammer tests are conducted to arch dam for empty and full reservoir. Also, finite element analyses of the dam are performed. The following observations can be made from this study:

- Natural frequencies obtained from EMA and FE methods are near to each other.
- Natural frequencies obtained for empty reservoir are bigger approximately 20% than the full reservoir. It is highlighted that reservoir water effects can not be neglected in the seismic analyses.
- It is difficult to obtain more than 3-4 frequencies from EMA method when reservoir is filled with water. It can be stated that the reservoir water affect the dynamic behavior of the arch dam significantly.
- Mode shapes are obtained as symmetrical, anti-symmetrical and vertical from finite element analysis of the arch dam. Due to optional tools are not found in the test equipment, experimental mode shapes are not obtained.

ACKNOWLEDGMENTS

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GROUND MOTION SIMULATIONS OF THE 1999 MARMARA EARTHQUAKES

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Key Words: Ground motion, simulation, waveforms, North Anatolian Fault zone

ABSTRACT Evaluating the ground motions to which structures are exposed to during their lifetimes is the main step in earthquake-resistant design, retrofitting of the existing structures and reduction seismic risk. In regions with sparse or no seismic recordings are available, ground motion simulations become essential. In this study, the stochastic finite-fault technique is used to simulate strong ground motions of the 12 November 1999 Düzce earthquake. In this method, originally proposed by Beresnev and Atkinson (1997), the finite- fault plane is discretized into a number of rectangular subfaults, each of which is treated as a stochastic point source with an w-square spectrum. The contribution from each subfault is lagged properly and summed at the observation point to obtain the ground motion from the entire fault. Ground motion simulations are carried out using regional model parameters as well as local soil conditions. For taking the soil amplification account, we employ standard H/V ratios. The region-specific seismic model parameters are calibrated against the strong ground motion recordings of the 1999 Düzce earthquake. The results are expressed in terms of the simulated acceleration and velocity time histories, Fourier amplitude spectra and response spectra.

INTRODUCTION

Earthquakes are among the most destructive disasters which affect the urban areas all over the world. But it is possible to identify and reduce the losses earthquakes cause. Most of the systematic approaches for seismic loss reduction involve earthquake ground motions. Ground motion records, observed or simulated, play a critical role in estimating the shaking and the seismic damage. However, even in the most advanced seismic networks in the world, there can be missing or incomplete recordings. In addition, in seismically-active urban regions of no or sparse seismic networks, it is necessary to use simulated ground motions.

There are various methods for simulating strong ground motions in the literature. These can be grouped into two main categories as the stochastic and deterministic techniques. The challenging part of simulations is the different characters of the ground motions with respect to their frequency content. If the frequency content is dominantly higher than 1 Hz, the motion is said to be a high-frequency ground motion and can be simulated using stochastic techniques. On the other hand, if the motion is composed of relatively lower frequencies than 1Hz, it is called low-frequency ground motion and it can be simulated with deterministic techniques (Hanks and Mcguire, 1981). Majority of the deterministic techniques depend on the numerical solution of the wave equation, Green's functions or ray tracing (Atkinson and Somerville, 1994; Beresnev and Atkinson, 1997).

To simulate high frequency ground motions the stochastic method relies on the combination of a deterministic Fourier amplitude spectrum and random phase angles (Irikura, 1986; Kamae at al., 1998; Boore, 1983; Graves and Pitarka, 2004). The most common amplitude spectrum is the omega-square acceleration spectrum (Hisada, 2000; Askan and Bielak, 2004; Askan and Bielak, 2005). The stochastic techniques are

observed to match the far-field ground motion components of large earthquakes (Ugurhan and Askan, 2010).

In Turkey, on the western part of the North Anatolian Fault zone, there are numerous strong motion stations but the network in several other regions is still insufficient for continuous ground motion monitoring. Thus, it is necessary to simulate the strong ground motions of large earthquakes. In addition, synthetic ground motions are used commonly to study the source, path and site effects. The main objective of this study is to simulate strong ground motions of the 1999 Düzce earthquake using stochastic finite fault technique. The software FINSIM (Beresnev and Atkinson, 1997) is employed to simulate the earthquake. The meizoseismal region, fault plane and epicenter of the earthquake are shown in Figure 1.

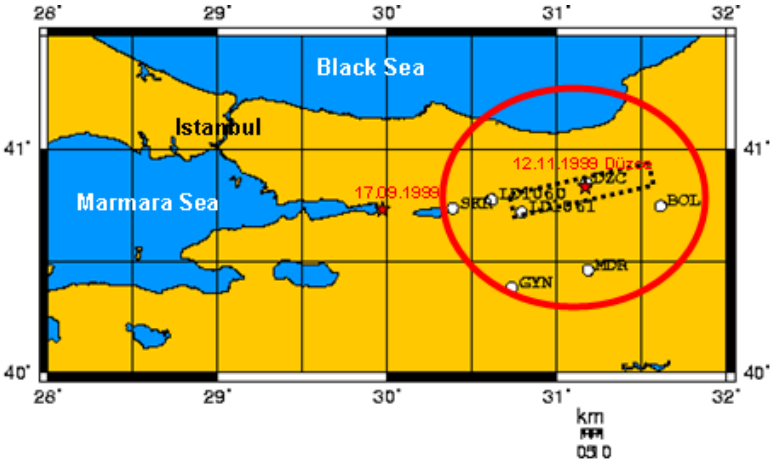

Fig.1. Düzce fault model of the 1999 Düzce earthquake and the nearby strong motion stations.

METHOD

The finite-fault technique used in this study divides the rectangular fault plane into smaller subfaults and each subfault is treated like a stochastic point source (Boore, 1983). The Fourier amplitude spectrum of a stochastic point source is the multiplication of the frequency-dependent filter functions of source, path and site effects. At a distance R from each subfault center, the S-wave acceleration amplitude is given as:

$$A(w) = S(w)P(w)D(w) \tag{1}$$

In this equation, w is the angular frequency, $S(w)$ is the omega-square source spectrum, $P(w)$ is the path-filter which is a function of the frequency-dependent quality factor and $D(w)$ is the site effects filter composed of spectral decay of high frequencies (kappa factor) and site amplification. This deterministic spectrum is combined with random phase angles to mimic the random behavior of the high-frequency ground motion.

The rupture on the finite fault is assumed to start from the hypocenter and the motion at a receiver is modeled by the addition of waveforms from each subfault in the time domain with proper time delays.

The most significant step in finite fault simulation is to determine the appropriate regional parameters of the earthquake of interest. The parameters of 1999 Düzce earthquake utilized here are adopted from previous studies of source inversions of the earthquake, as well as bedrock attenuation properties and geometrical spreading effects of the Marmara region (Umutlu et al., 2002; Durukal, 2002). The parameters employed for the 1999 Düzce earthquake simulations are displayed in Table 1.

Table 1. Source and path parameters used in the simulations.

Parameters	***Values***
Fault Orientation	Strike: 264° and Dip=64°
Fault Dimensions	65 x 25 km
M_w	7.1
Hypocenter Depth	12.5 km
Subfault Dimensions	5 x 5 km
Stress Drop	50 bars
S-wave Velocity	3500 m/sec
Crustal Density	2800 kg/m^3
Crustal Attenuation	$180f^{0.45}$
Geometric Spreading	1/R
Windowing Function	Saragoni-Hart

The site effects are critical components of ground motion simulations as they affect the amplitudes significantly. Thus it is important to realistically estimate the amplifications due to local soil layers. In this study, Horizontal to Vertical Ratio (H/V) method, also known as Nakamura's technique, is used for taking the site amplifications into account Nakamura (1989). Figure 2 displays the H/V ratios of the stations that are taken into account in this study.

The strong ground motion stations used in this study are shown in Table 2 with their codes, locations (latitudes and longitudes), hypocentral distances and NEHRP Site Classes [Web Ref. 1].

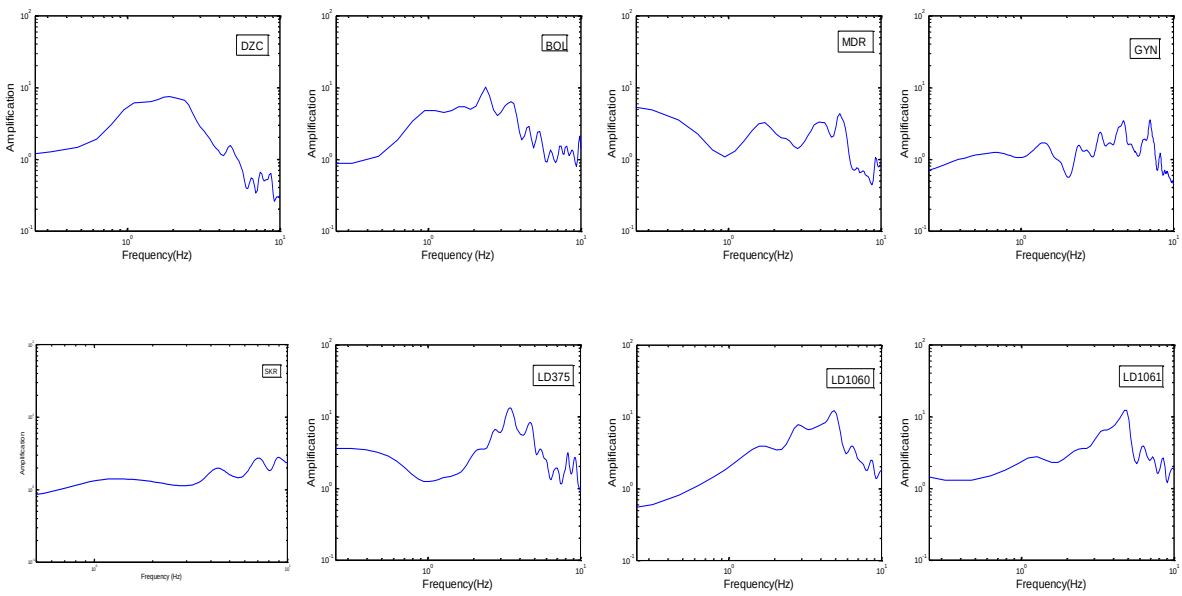

Fig. 2. H/V Ratios as Local Site Amplifications of the Strong Ground Motion Stations.

Table 2. Strong Ground Motion Stations.

Station Code	***Location***	***Latitude (°N)***	***Longitude (°E)***	***Hypocentral Distance (km)***	***NEHRP Site Class***
DZC	Düzce	40.85	31.17	12.7	D
BOL	Bolu	40.75	31.61	33.7	B
MDR	Mudurnu	40.46	31.18	30.9	B
GYN	Göynük	40.39	30.73	56.3	D
SKR	Sakarya	40.74	30.38	69.6	C
LD375	Karadere	40.74	30.88	26.5	B
LD1060	Karadere	40.78	30.61	49.8	B
LD1061	Karadere	40.72	30.79	34.6	C

RESULTS

To validate the ground motions obtained with the finite-fault technique, the synthetic ground motions are compared with the 1999 Düzce Earthquake mainshock recordings at the strong ground motion stations given in Table 2. The results of the simulations in terms of acceleration time histories and Fourier spectra are given in Figures 3 and 4, respectively. As observed in Figure 3, peak ground acceleration (PGA) values, duration of the recordings and S-wave arrival times are in close match with the observed values. It is observed in Figure 4 that the finite fault modeling gives close results in the high frequencies as expected but the results do not fully match with the observations in the lower frequencies especially below 1 Hz. In order to improve the match in the lower frequencies, an optimization study for the source parameters must be conducted and the effects of these parameters on the duration, amplitude and frequency content of simulations for frequencies lower than 1 Hz, must be investigated. The high frequency ground motions are observed to satisfactorily model the observed records in both time and frequency domains.

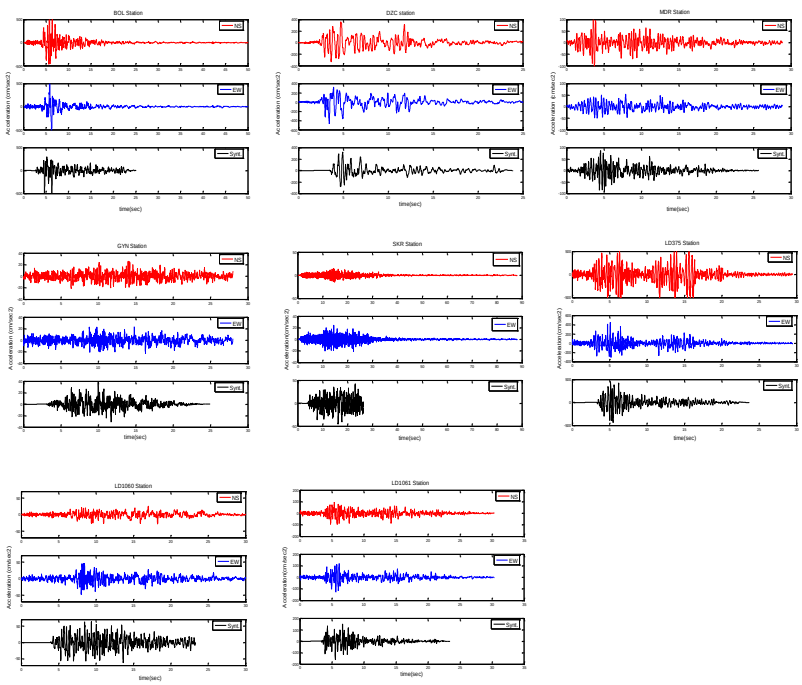

Fig. 3. Comparison of simulations with the observed records in time domain.

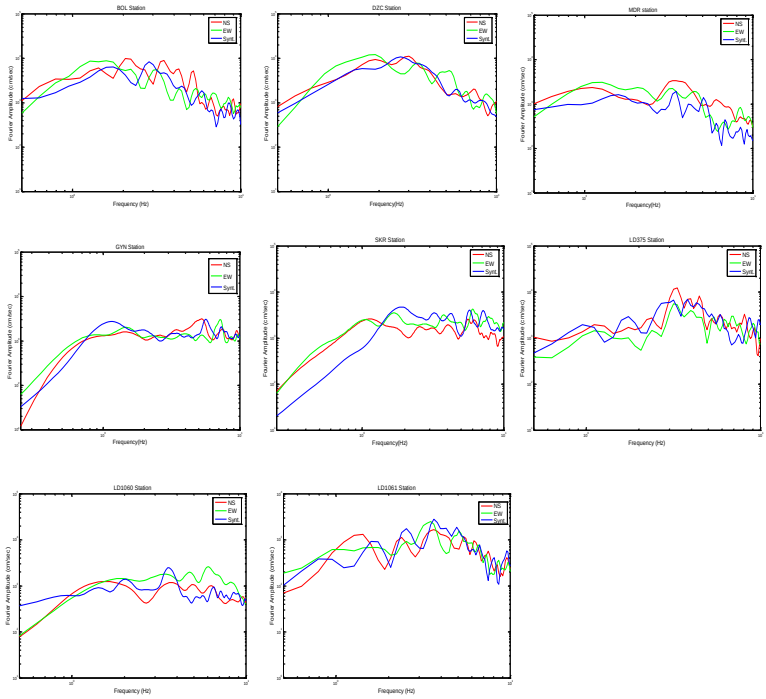

Fig. 4. Comparison of simulations with the observed records in frequency domain.

CONCLUSIONS

The strong ground motions of the 1999 Düzce earthquake are simulated using the stochastic finite-fault method. The source and crustal properties of the region are investigated, whereas site amplifications are represented in the form of H/V factors. The parameters employed in the simulations are validated with the mainshock recordings of the 1999 Düzce earthquake.

This study confirmed that the efficiency of finite fault modeling depends on the employed seismological parameters of the region of interest. However, the method produces reliable results that can be used in determining peak ground acceleration, peak ground velocity (PGV) and spectral acceleration (SA) values which the structures in the region of interest will experience during earthquakes of varying magnitudes.

ACKNOWLEDGMENTS

We thank Dr. Igor Beresnev for kindly providing us the software FINSIM for the finite-fault ground motion simulation.

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INVESTIGATION of EFFECT of SHEAR WALL CONTINIUTY to THE STRUCTURAL CAPACITY in RC BUILDINGS

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Key Words: Shear wall continuity, Structural capacity, Capacity curve.

ABSTRACT In this study, the relationship between the shear wall which continued from ground floor to top floor and the structural capacity was investigated in the multi-storey reinforced concrete buildings. For this purpose, fourteen different reinforced plane systems were selected. These structures consist of two plane frames, one of which is 5-strorey and the other is 7-storey. Moreover, 12 dual frame-shear wall systems, shear walls were located in middle axes, were also considered. Five of them were 5-storey and rests of them were 7-storey. Column sizes of the plane systems were chosen as 50/50 cm for 5-storey and 60/60 for 7-storey systems. Beam sizes of all the plane systems were chosen as 30/60 cm. Pushover analysis of all structures were performed and the capacity curves of them illustrated. The concrete classes, which were 20 MPa, 25 MPa and 30 MPa, were used in the analyses of structures. The capacity curves were obtained from the analyses and the areas under these curves were determined. The results revealed that increasing of the number of shear walled floor much more capacity in these structures. However, significant increases in the capacity were not observed after the height of shear wall reached certain level. SAP2000 structural analysis software was used in the analyses.

INTRODUCTION

Damages caused by earthquakes showed that structural performance and earthquake strength of existing structures have to be increased. The level of life safety performance in new structures has to be also provided. There are uncertainties in new structures and reinforced existing structures about their performance of earthquake. Therefore, the design of new structures and reinforcing of existing structures should be evaluated carefully.

Reinforced concrete shear walls increase the strength of tall buildings against lateral loads like earthquake and wind [Wang, 2001]. The effects of lateral loads become more important than vertical loads proportional to the height of structure. Lateral forces overload the structural elements of reinforced concrete buildings especially in the earthquake regions and increase second-order moments. Shear walls have much more bending rigidity than column elements are used in order to constrain the lateral displacement [Öztürk, 2005]. Moreover, it is determined that shear walls prevent the damages of non-structural elements [Celep, 2005, Atımtay, 2000].

In this study, the relationship between shear wall continuity along to building height and structural capacity was investigated. In this purpose, the pushover analyses of fourteen reinforced concrete plane frames have different concrete classes, shear wall continuity and numbers of story were performed and capacity curves were plotted. Calculating the areas under these curves, the energy absorption capacities of the structures were evaluated.

METHOD

Increasing of loads amplifies the internal forces. After the load values exceed the linear elastic limit in some beam column connections, the plastic deformation occurs in the buildings made from non-linear material. The plastic deformation concentrates some special sections called plastic section and the building behaves linear elastic in the other regions [Öncü, 2008]. This assumption is called as plastic hinge theory. In the case of being certain number of plastic hinge, the part of beam in the midst of two plastic hinges could displace without any increasing of load values. This situation is defined as mechanism.

Conventional design methods have certain criteria such as strength, ductility, usability damage control and shear limit. Performance based design in the current specifications is explained that the performance levels defines earthquake damages in the buildings. Performance based design have been substituting the conventional force based methods. It bases deformation in the design of new buildings and determination of earthquake performance of the existing buildings. The main tool of this method is static pushover analysis. The capacity curve obtained from pushover analysis shows the nonlinear variation of base shear force with the displacement of the top storey [Aydınoğlu, 2003].

Building is loaded by lateral forces which increased step by step until the top storey displacement reaches the target displacement. Lateral loads are applied together with vertical loads during the analysis procedure. The variation of the top storey displacement depending on the lateral load, called capacity curve, is demonstrated as a result of analysis. Therefore, damages, partial or total collapse conditions, weak elements, limit load and the displacement value at the moment of the collapse are specified. Besides, whether or not building is satisfied the performance level could be checked for earthquake hazard level [Korkmaz and Uçar, 2005].

In this study, shear walls used in the models was designed as equivalent column. Static pushover was applied until the system collapses in frames. Capacity curves were found depending on concrete classes, shear wall continuity and number of storey. The areas under these curves were calculated and some evaluations were made.

NUMERICAL STUDY

Fourteen different reinforced plane systems were selected for numerical studies. These structures consist of two plane frames, one of which is 5-strorey and the other is 7-storey. Moreover, 12 dual frame-shear wall systems, shear walls were located in middle axes, were also considered. Five of them were 5-storey and rests of them were 7-storey. The systems with C20, C25 and C30 concrete classes called as A, B and C models respectively (Figure 1-14). Column sizes of the plane systems were chosen as 50/50 cm for 5-storey and 60/60 for 7-storey systems. Beam sizes of all the plane systems were chosen as 30/60 cm. Pushover analysis of all structures were performed and the capacity curves of them illustrated (Figure 15-20). Areas under the capacity curves were calculated and changes of them according to A5, B5 and C5 models were represented in Table 1-6 and were compared with each other in Figure 21-26.

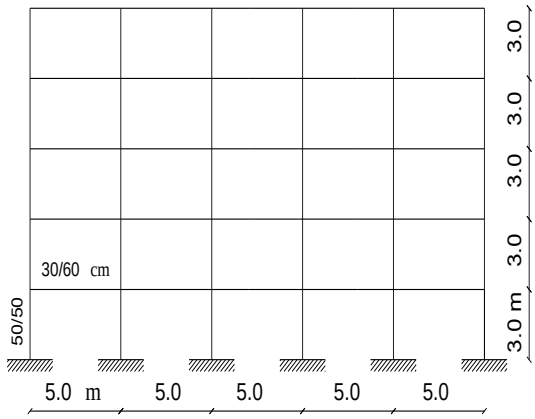

Fig. 1. A5, B5 and C5 models.

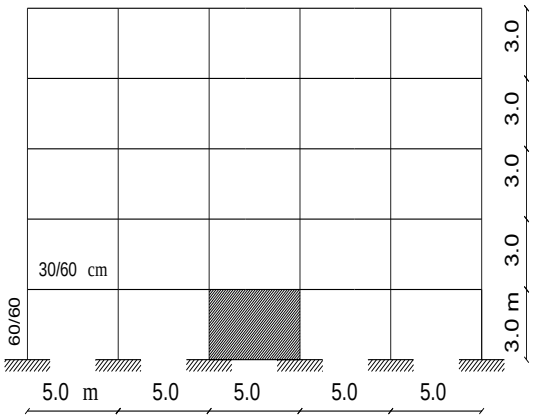

Fig. 2. A5–1, B5–1 and C5–1 models.

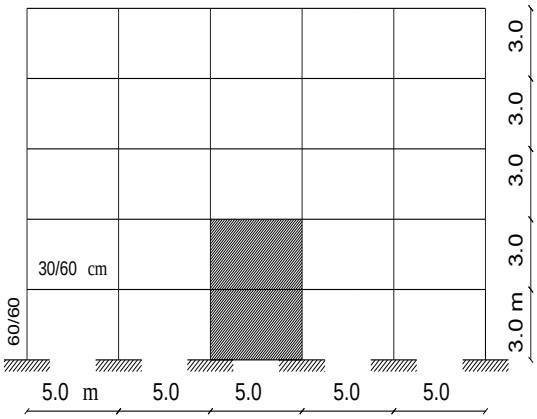

Fig. 3. A5–2, B5–2 and C5–2 models.

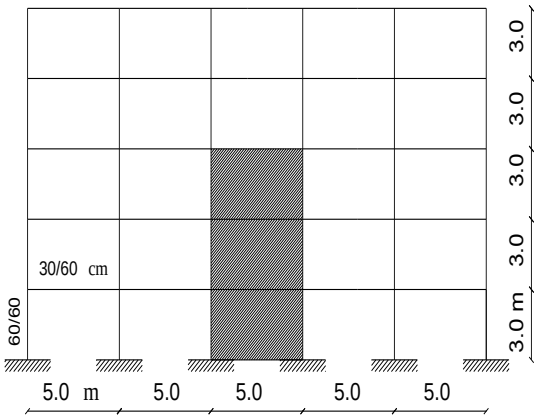

Fig. 4. A5–3, B5–3 and C5–3 models.

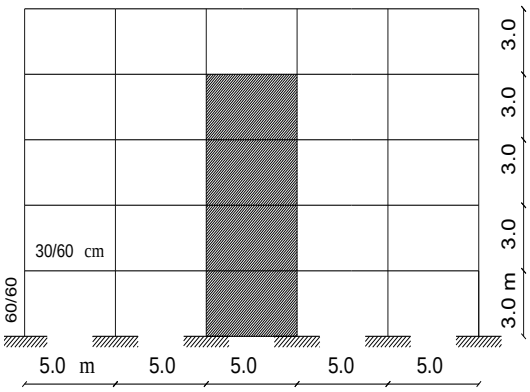

Fig. 5. A5–4, B5–4 and C5–4 models.

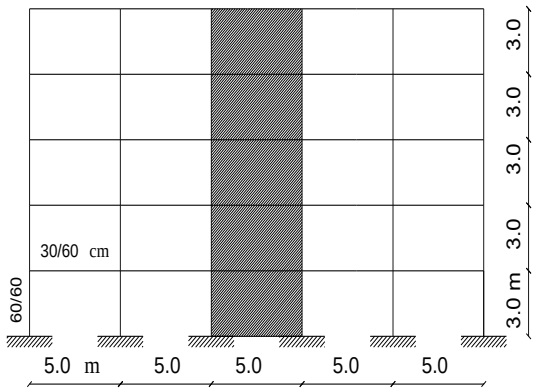

Fig. 6. A5–5, B5–5 and C5–5 models.

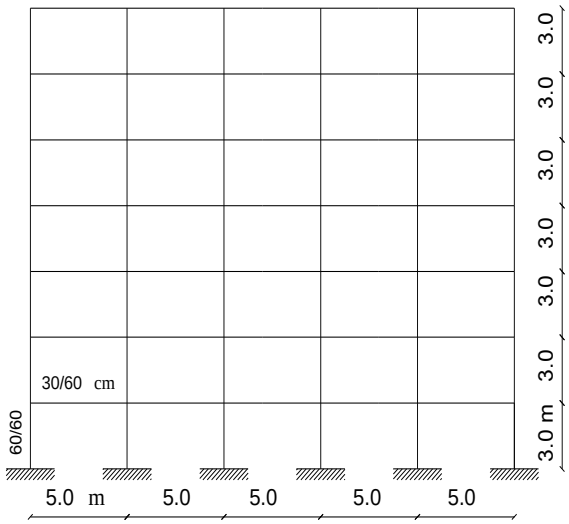

Fig. 7. A7, B7 and C7 models.

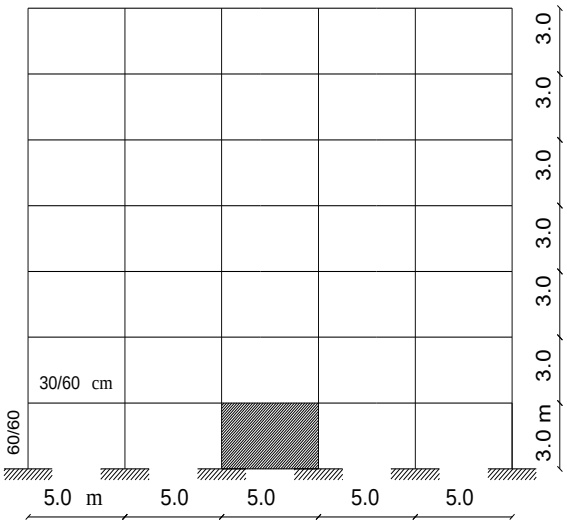

Fig. 8. A7–1, B7–1 and C7–1 models.

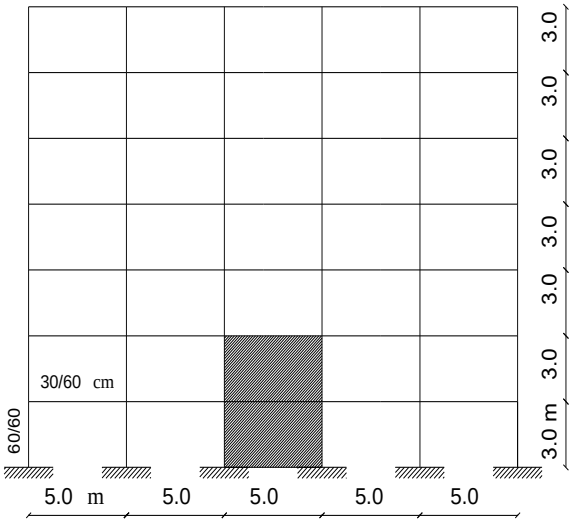

Fig. 9. A7–2, B7–2 and C7–2 models.

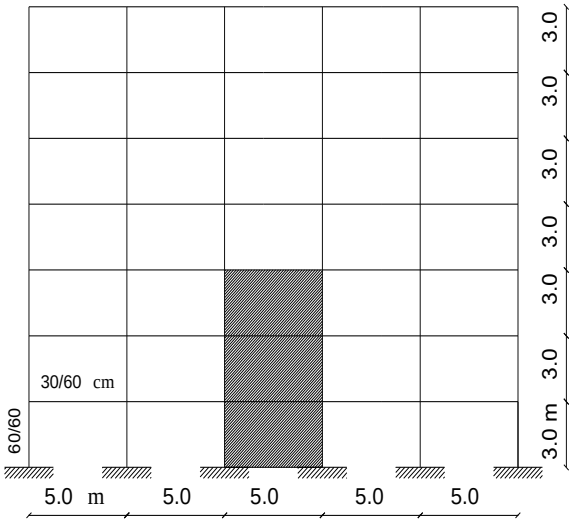

Fig. 10. A7–3, B7–3 and C7–3 models.

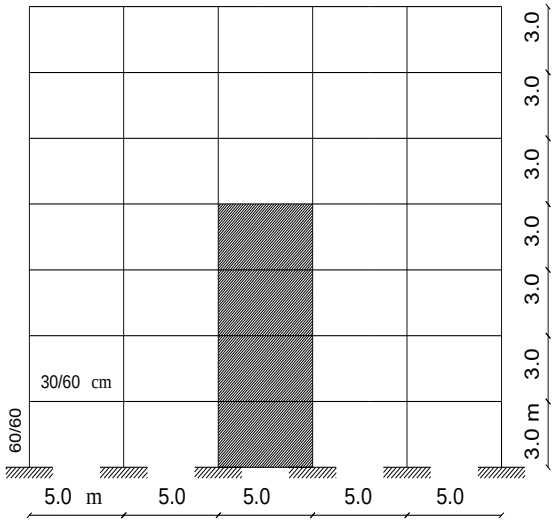

Fig. 11. A7–4, B7–4 and C7–4 models.

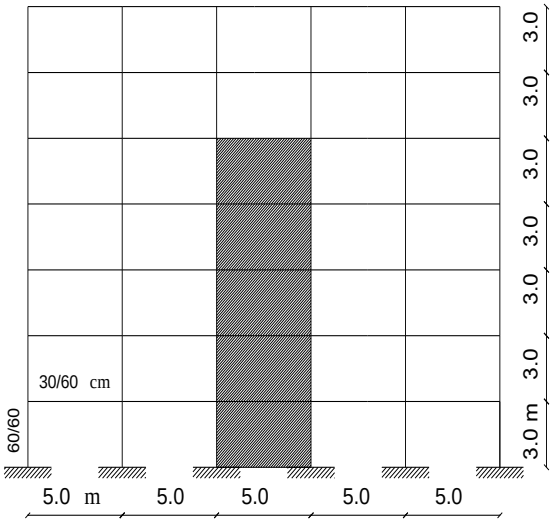

Fig.12. A7–5, B7–5 and C7–5 models.

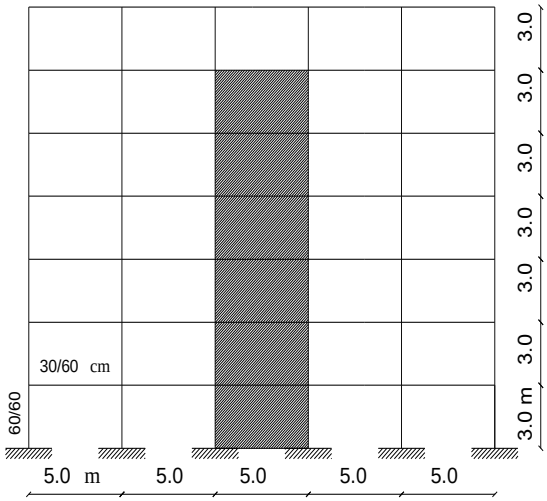

Fig. 13. A7–6, B7–6 and C7–6 models.

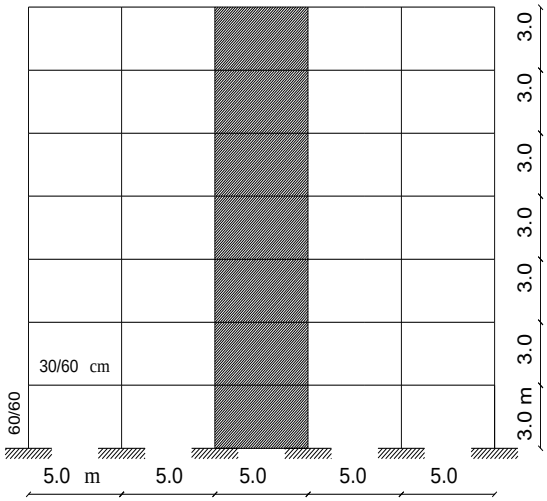

Fig. 14. A7–7, B7–7 and C7–7 models.

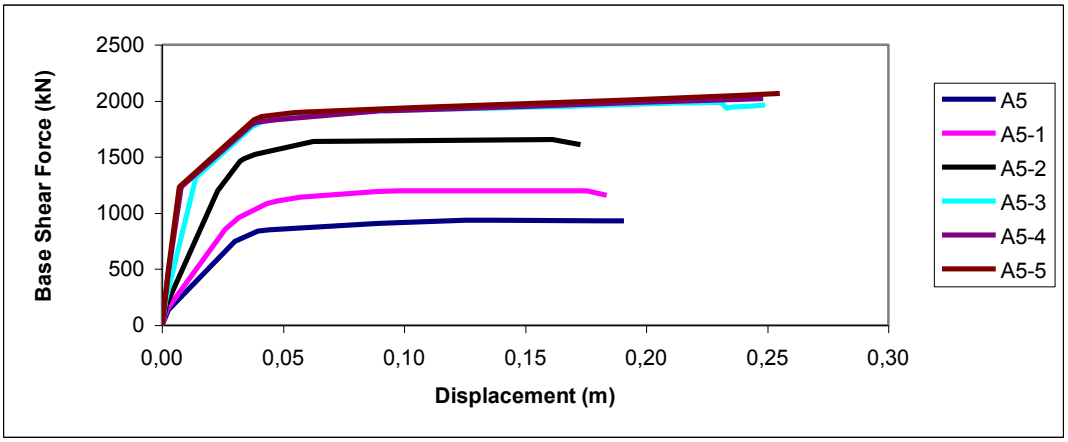

Fig. 15. Capacity curves of A5 models.

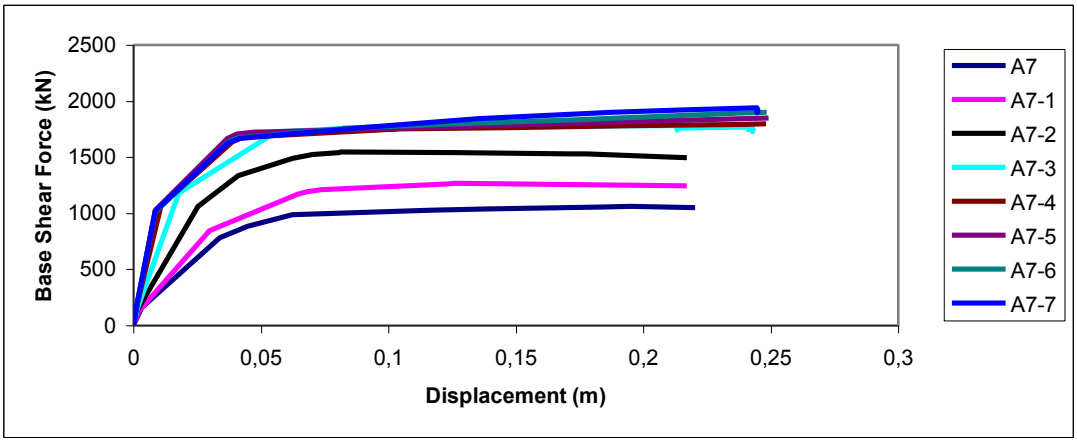

Fig. 16. Capacity curves of A7 models.

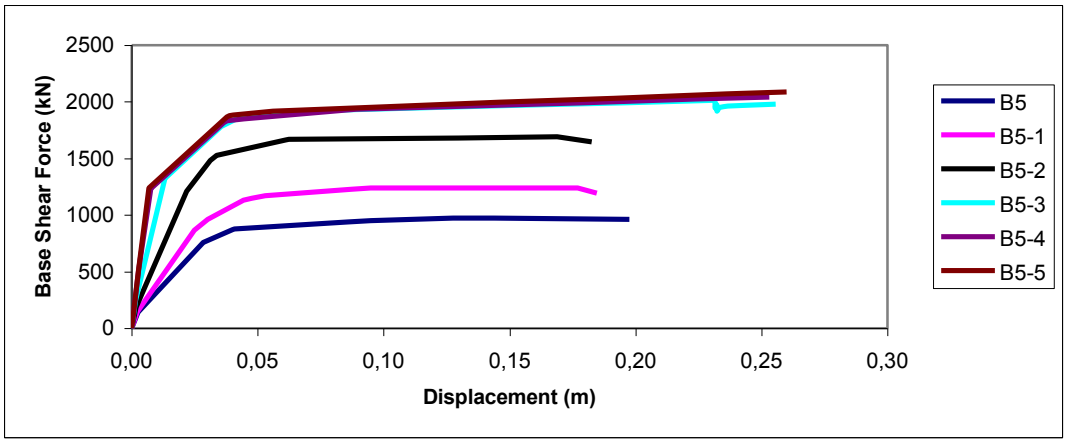

Fig. 17. Capacity curves of B5 models.

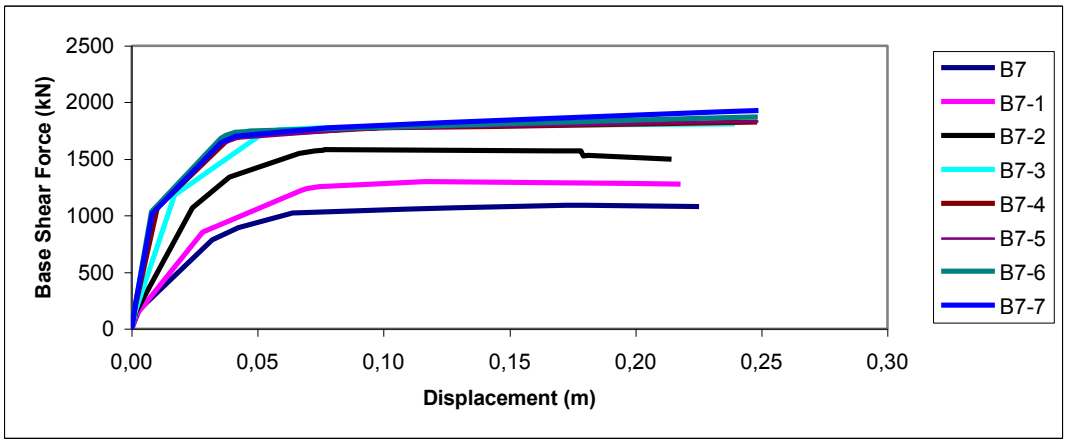

Fig. 18. Capacity curves of B7 models.

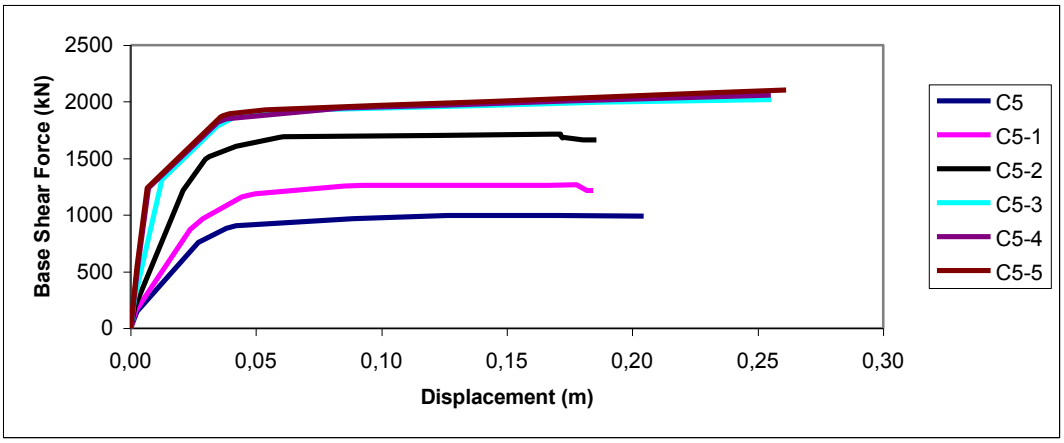

Fig. 19. Capacity curves of C5 models.

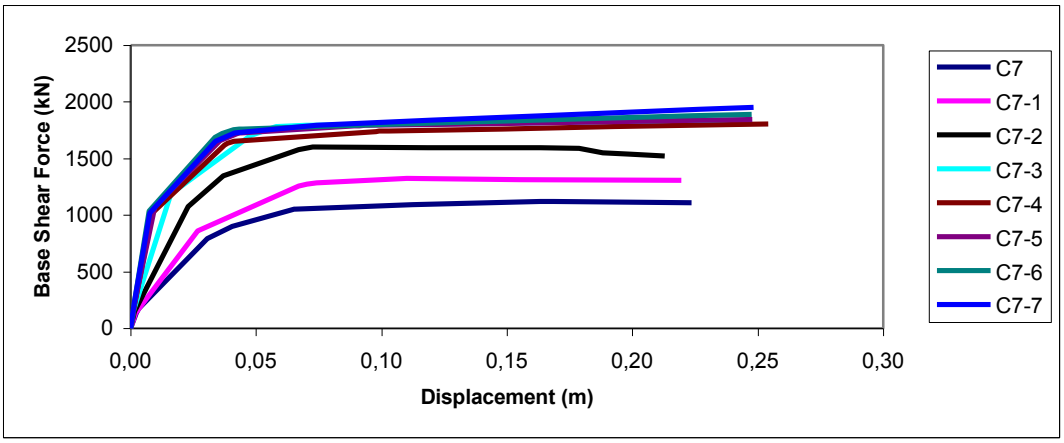

Fig. 20. Capacity curves of C7 models.

Table 1. Areas under the capacity curves of A5 models and percentage changes according to A5 model.

Structure Models	Areas Under the Capacity Curves (kNm)	Changing (%)
A5	157.7421	0.00
A5–1	194.7885	23.49
A5–2	256.2055	62.42
A5–3	454.6099	188.20
A5–4	458.5361	190.69
A5–5	479.9891	204.29

Table 2. Areas under the capacity curves of A7 models and percentage changes according to A7 model.

Structure Models	Areas Under the Capacity Curves (kNm)	Changing (%)
A7	203.2640	0.00
A7–1	238.5889	17.38
A7–2	299.8623	47.52
A7–3	391.4242	92.57
A7–4	411.2103	102.30
A7–5	421.1531	107.20
A7–6	422.8834	108.05
A7–7	424.1000	108.64

Table 3. Areas under the capacity curves of B5 models and percentage changes according to B5 model.

Structure Models	Areas Under the Capacity Curves (kNm)	Changing (%)
B5	170.5764	0.00
B5–1	202.5928	18.77
B5–2	278.2980	63.15
B5–3	473.3498	177.50
B5–4	474.4225	178.13
B5–5	495.7358	190.62

Table 4. Areas under the capacity curves of B7 models and percentage changes according to B7 model.

Structure Models	Areas Under the Capacity Curves (kNm)	Changing (%)
B7	216.321	0.00
B7–1	247.1523	14.25
B7–2	302.0514	39.63
B7–3	397.5016	83.76
B7–4	418.2043	93.33
B7–5	419.4505	93.90
B7–6	426.6004	97.21
B7–7	431.3618	99.41

Table 5. Areas under the capacity curves of C5 models and percentage changes according to C5 model.

Structure Models	Areas Under the Capacity Curves (kNm)	Changing (%)
C5	182.0384	0.00
C5–1	207.0061	13.72
C5–2	287.7707	58.08
C5–3	476.3739	161.69
C5–4	482.5385	165.08
C5–5	502.364	175.97

Table 6. Areas under the capacity curves of C7 models and percentage changes according to C7 model.

Structure Models	Areas Under the Capacity Curves (kNm)	Changing (%)
C7	220.1784	0.00
C7–1	254.3725	15.53
C7–2	305.0892	38.56
C7–3	403.3692	83.20
C7–4	421.887	91.61
C7–5	422.459	91.87
C7–6	429.1653	94.92
C7–7	435.3285	97.72

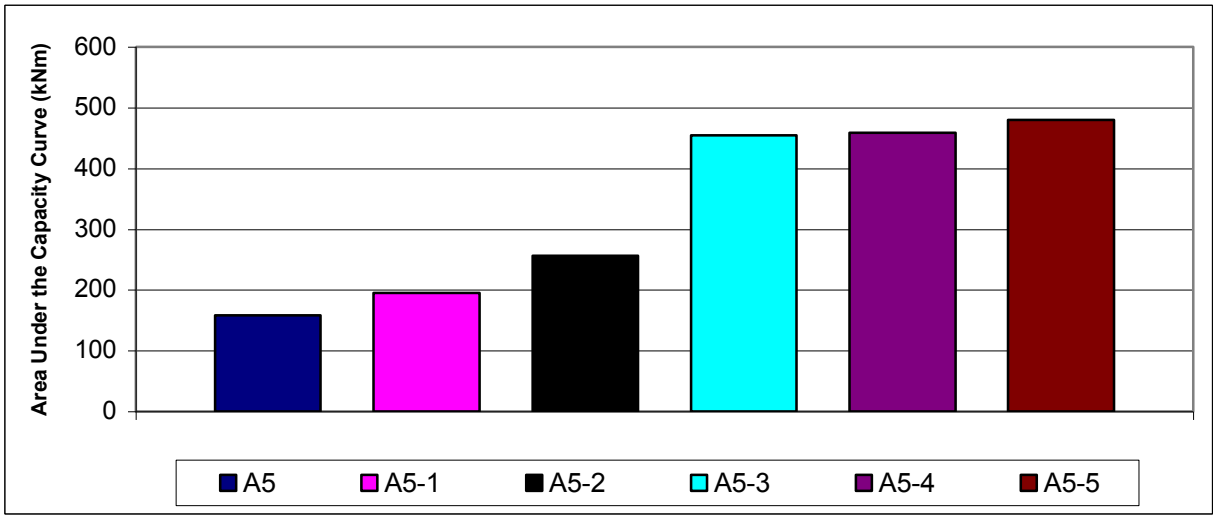

Fig. 21. Comparing of areas under the capacity curves of A5 models.

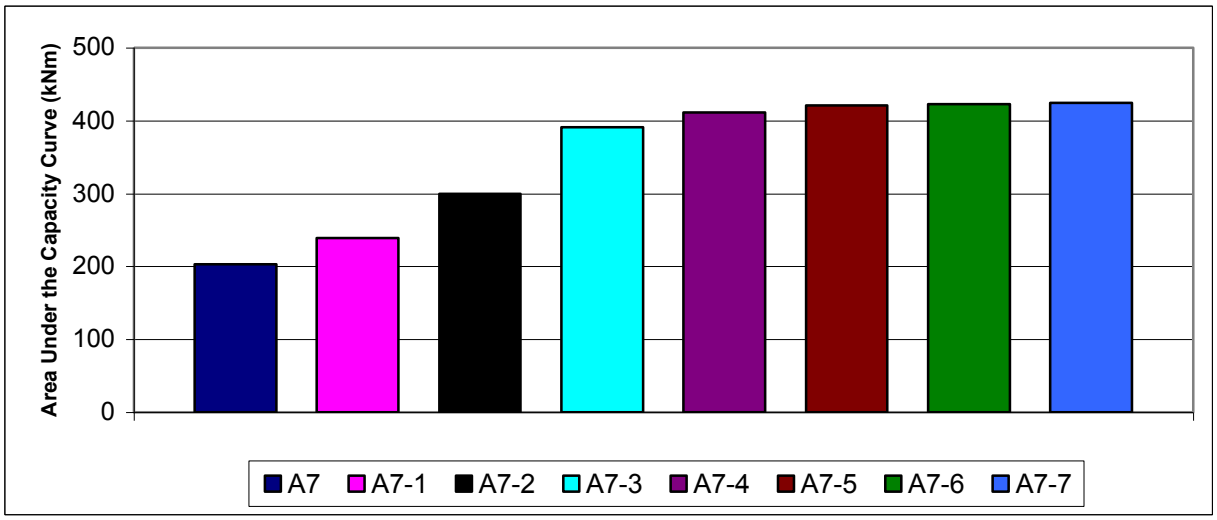

Fig. 22. Comparing of areas under the capacity curves of A7 models.

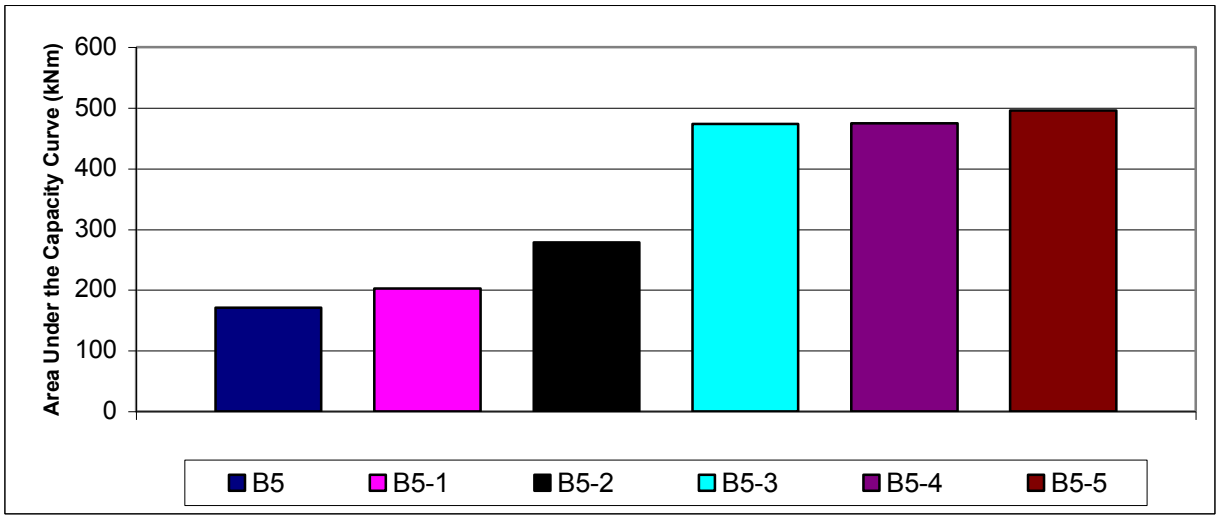

Fig. 23. Comparing of areas under the capacity curves of B5 models .

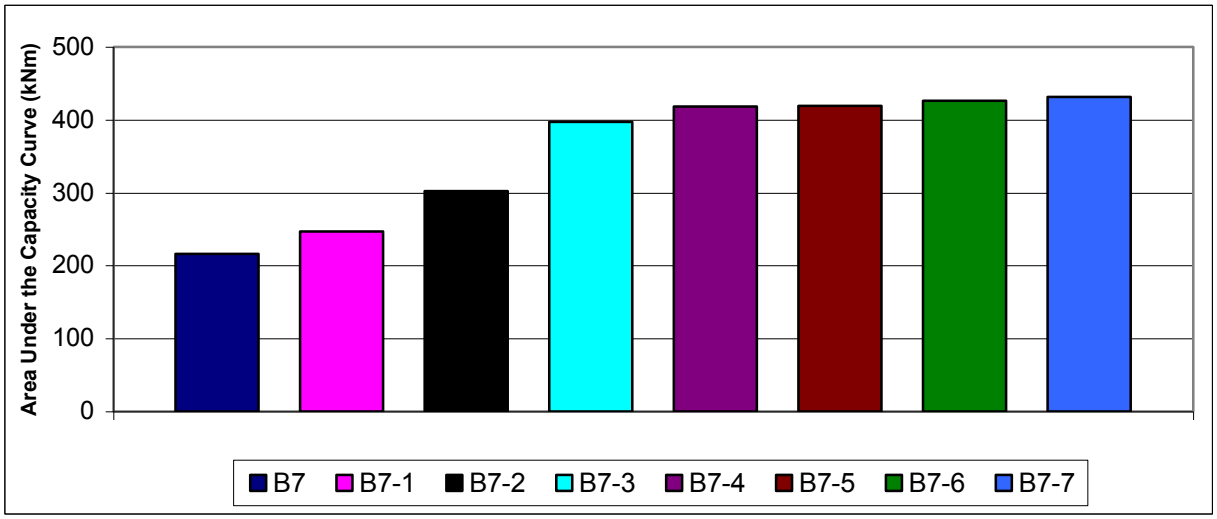

Fig. 24. Comparing of areas under the capacity curves of B7 models.

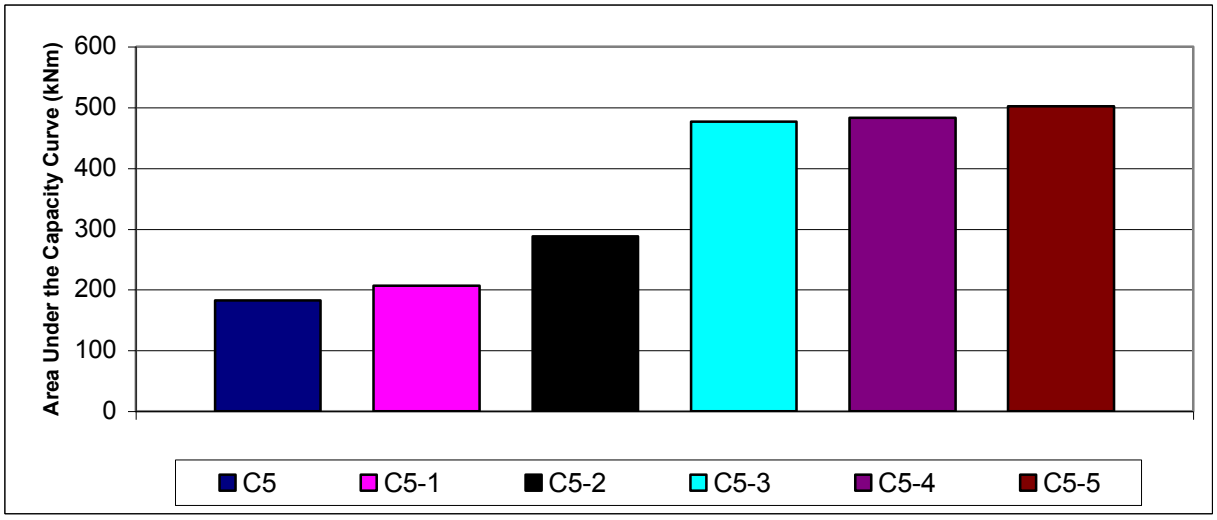

Fig. 25. Comparing of areas under the capacity curves of C5 models.

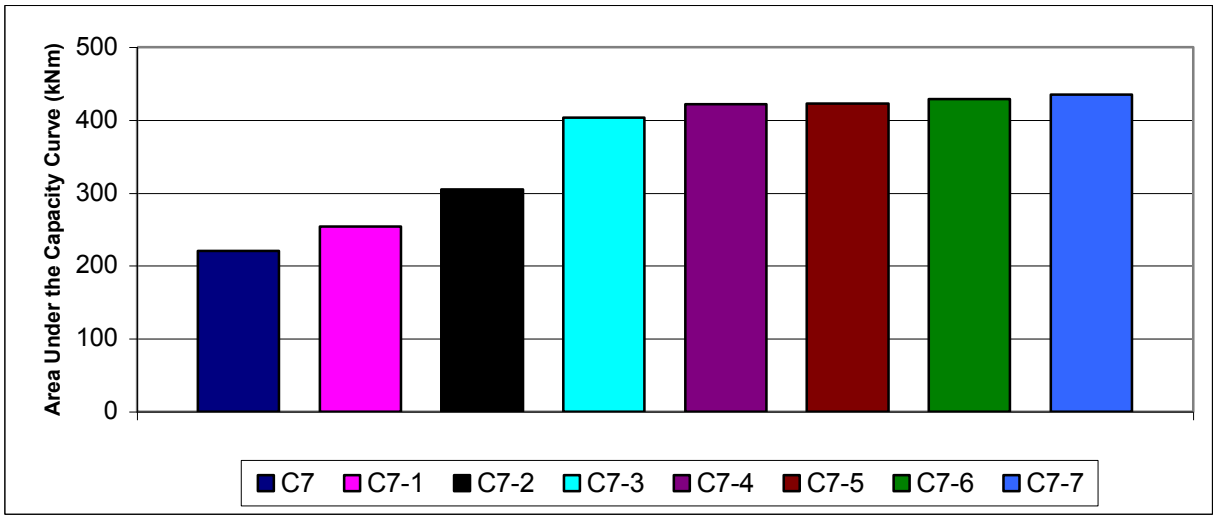

Fig. 26. Comparing of areas under the capacity curves of C7 models.

CONCLUSION

This study revealed that using of shear walls magnifies structural capacity significantly until the half height of building. However, the structural capacity slightly increases from this height. In accordance with our expectations, it is seeing that using of shear walls in all stories provides much more energy consumption.

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MUNZUR NEHRİ HAVZASINDA SULAMA SUYU İLE ENERJİ ÜRETİMİ ARASINDAKİ OPTİMAL İLİŞKİ

OPTIMAL RELATIONSHIP BETWEEN IRRIGATION AND ENERGY PRODUCTION IN THE MUNZUR RIVER BASIN, TURKEY

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Anahtar Kelimeler: Çok Barajlı Sistem, Çok Amaçlı Optimizasyon, Sulama Suyu, Enerji Üretimi, Ardışık Yaklaştırmalı Dinamik Programlama.

ÖZ Bu çalışmada, Munzur Nehri Havzası'nda Munzur Nehri üzerinde bulunan çok amaçlı ve çok barajlı bir su kaynakları sistemi üzerine uzun sureli planlama için enerji üretimi amaçlı optimizasyon modeli kurulmuştur. Optimizasyon tekniği olarak ardışık yaklaştırmalı dinamik programlama kullanılmıştır. Bu optimizasyon modelinde, ilk olarak, kurak dönemin aylık ortalama akımları kullanılarak ve sulama suyu parametrik olarak değiştirilerek güvenilir güç değerlerinin enbüyüklenmesi gerçekleştirilmiş ve minimum işletme seviyeleri bulunmuştur. Daha sonra, aynı modelde, güvenilir güç değerleri sulama miktarına göre kısıt olarak yerleştirilip, aylık ortalama akımlar kullanılarak ve sulama suyu parametrik olarak değiştirilerek toplam enerjinin enbüyüklenmesine çalışılmış ve normal işletme seviyeleri elde edilmiştir. Sonuç olarak, elde edilen işletme seviyelerine bakıldığında, optimizasyon süreci büyük hacimli barajlar tarafından kontrol edilmiş ve yönetilmiştir. Sulama miktarı arttığı zaman, güvenilir güç değerleri ve enerji faydası azalmakta, sulama faydası artmaktadır. Toplam faydanın en büyük olduğu yerde, optimum sulama suyu miktarı $60x10^6$ m^3 olarak elde edilmiştir.

GIRIŞ

Kullanılabilir kalitede su kaynaklarının, makul ekonomik çevrede temini ve topluma sunulması su kaynakları mühendisliğinin güncel sorunudur. Su hayati önem taşımakla birlikte modern toplumda tarımsal üretimi, sanayi kullanımı ve enerji üretimi açılarından büyük önem taşır. Ayrıca sınırlı su kaynakları giderek artan bir nüfus ile birlikte modern toplum talepleri karşısındadır ve su kaynakları yönetimi buna paralel bir dikkatle ele alınmalıdır.

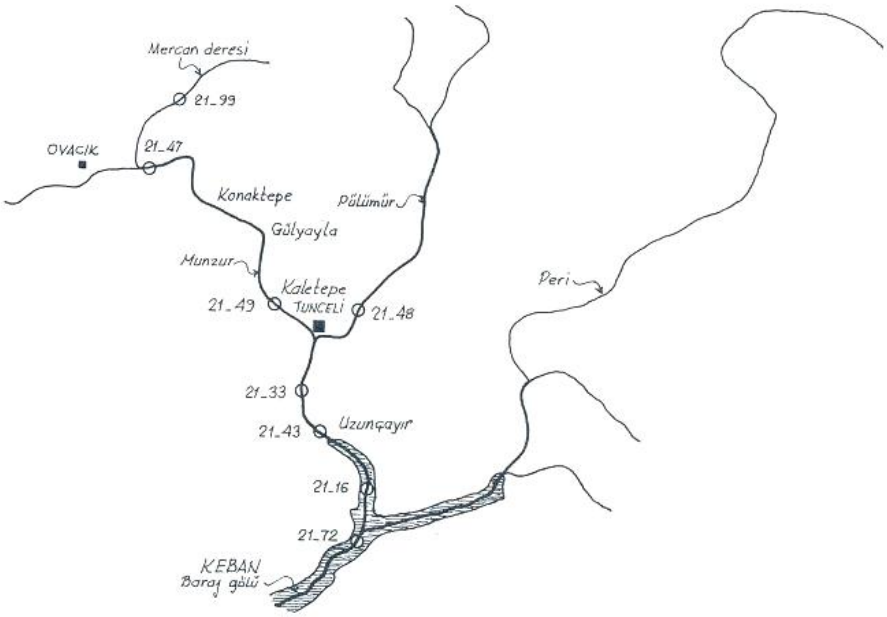
Tarımsal üretimin olduğu bölgelerdeki çok amaçlı, çok barajlı sistemlerde sulama için baraj etrafında bulunan tarım alanları enerji üretim faydasını beklenen seviyelerin altına çekebilmektedir. Bu durumla birlikte sulama - enerji faydasının arttığı durumları da gözlemek mümkün olabilmektedir. Bu durumlar için sulanacak alanlarının büyüklüğü, yıllık–aylık sulama ihtiyacı, ürün çeşitliliği, bu ürünler içerisinde enbüyük faydayı sağlayacak ürünlerin belirlenmesi gerekmektedir. Bu veriler ile birlikte, enerji üretimi amaçlı uzun süreli optimal işletme modeli geliştirilmiştir. Bu model içinde sulama miktarı sistemden parametrik olarak çekilmiş ve sulama faydası değerleri bulunmaya çalışılmıştır. Ek olarak bu faydanın enerji üretimi üzerindeki etkileri ile sulama–enerji üretiminin birlikte faydasının sonuçları da incelenmeye çalışılmıştır.

Labadie (2004), çok barajlı sistemlerde enerji optimizasyonu ve taşkın kontrolü gibi amaçları gerçekleştirmek için çeşitli yöntemlerin kullanıldığı çalışmalar üzerine dikkatleri çekmiştir. Güvel (1997), Sert ve Ark. (1982, 1983), Yakowitz (1982), Yeh (1985) ve Yurtal (1993), enerji optimizasyonu ile ilgili çalışmalarda, enerji üretim fonksiyonunun doğrusal olmaması nedeniyle, dinamik programlama optimizasyon ve simülasyon

tekniklerini kullanılmışlardır. Needham (2003), simülasyon, doğrusal ve doğrusal olmayan programlama optimizasyon teknikleri ile taşkın kontrolü ve sulama gibi amaçlar için sulama miktarı ve finansal açıdan çalışmalar yapmıştır.

Sert ve Ark.(1982), ardışık yaklaştırmalı dinamik programlama (DPSA) optimizasyon tekniği bir akarsu üzerindeki bir seri hidroelektrik tesisin optimal boyutlandırma ve işletmesi çalışmasında kullanmışlar. Daha sonra, Sert ve Ark.(1983), bu tekniği kullanarak Sakarya Havzası'nın enerji üretimini enbüyüklemek için benzetim yöntemleri ile birlikte ele almışlardır. Shim ve Ark. (2002), DPSA' yı Korea' de, Han Havzası'nda, gerçek-zamanlı taşkın kontrol işletmeleri için uygulamışlardır. Son olarak, Yi ve Ark. (2003), DPSA' yı, Lower Colorado Nehir Baraj Sistemi'nde saatlik optimal hidro-güç birimlerinin planlanmasına yardımcı olması amacıyla uygulamışlardır.

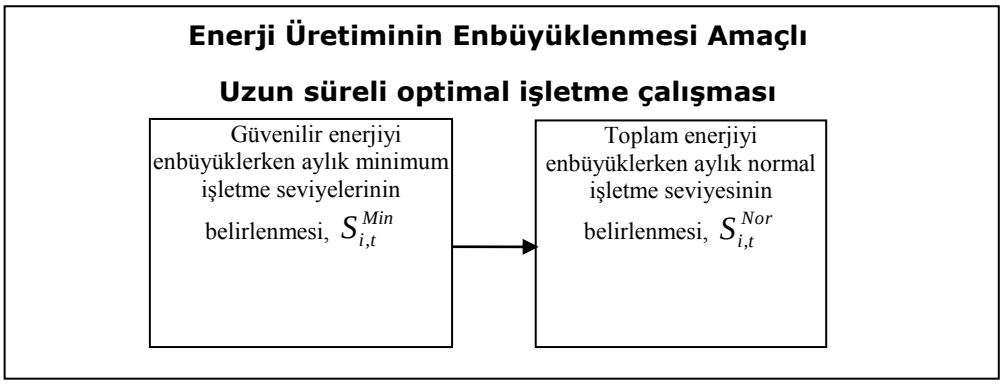
Ülkemizin Doğu Anadolu Bölgesi'nde yer alan Munzur Akarsuyu Havzası, kuzeyde Toros dağlarının devamı olan Munzur ve Mercan dağları, doğuda Bingöl Dağları, Batıda Erzincan ili ve güneyde ise Keban Baraj Gölü ve Elazığ ili arasında bulunmaktadır.Munzur Akarsu Havzası'na gelen yıllık yağış miktarı 550-1080 mm arasında değişmekte olup, karasal iklim sürmektedir. En az yağış yaz aylarında en çok yağış ise sonbahar ve kış aylarında düşmektedir. Yaz aylarında kuzey-batıdan esen kuvvetli rüzgarlarla iklim kurak geçmektedir. Kış aylarında ise rüzgar genellikle güney-batı tarafından hafif esmektedir. Kış ayları soğuk ve yağışlı, yaz ayları ise sıcak ve kurak geçer. En yüksek ısı derecesi +40 derece, en düşük ısı derecesi ise -23 derecedir. Yıllık ortalama nem %63 dür. Şekil 1' de, Munzur Nehir Havzasına ait şematik bir plan verilmiştir.



Şekil 1. Munzur Akarsu Havzası'nın şematik görünüşü.

Bu çalışmanın amacı, çok amaçlı ve çok barajlı bir su kaynakları sistemini matlab bilgisayar programını kullanarak modellemek, tanımlanan bu modele de sulama miktarının parametrik değişken olarak alındığı uzun sureli planlama için işletme optimizasyonu modeli kurmaktır. Optimizasyon modeli içerisinde, , kurak dönemin aylık ortalama akımları kullanılarak güvenilir güç değerleri enbüyüklenmiştir. Bu işlem sırasında sulama suyu sabit ve sıfır olarak seçilmiştir. Yapılan bu işlemin devamında, aynı modele elde edilen güvenilir güç değerleri tanımlanmış, aylık ortalama akımlar ile sulama miktarı parametrik olarak değiştirilerek toplam enerji enbüyüklenmiştir. Bu işlemlerde ilk olarak aylık minimum işletme seviyeleri ikincisinde ise, aylık normal işletme seviyeleri elde edilmiştir. Optimizasyon tekniği olarak ardışık yaklaştırmalı dinamik programlama seçilmiştir. Model Munzur Nehir Havzası'nda bulunan çok amaçlı ve çok barajlı bir su

kaynakları sistemine uygulanmıştır. Şekil 2' de, enerji üretiminin enbüyüklenmesi amaçlı uzun süreli optimal işletme çalışması süreci özetlenmiştir.



Şekil 2. Uzun süreli optimal işletme çalışması süreci.

SİSTEMİN MATEMATİKSEL MODELİNİN TANIMLANMASI

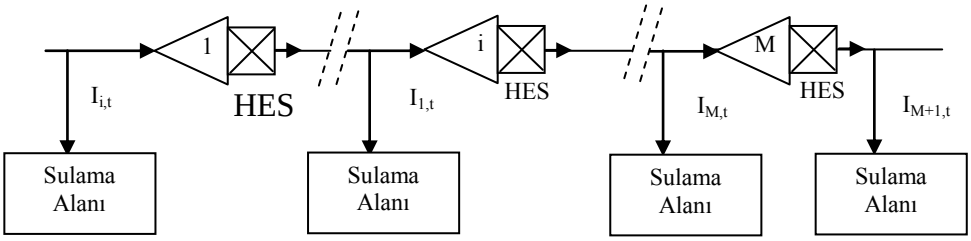
Bir akarsu üzerine kurulmuş çok sayıda baraj ve hidroelektrik santral (HES) tesisleri çok amaçlı ve çok barajlı bir su kaynakları sistemi olarak tanımlanabilir. Şekil 3' de, içerisinde sulama alanlarının yeraldığı çok barajlı bir su kaynakları sistemi şeması verilmiştir. Bu sistem içinde bulunan herhangi bir i-barajının t-zamanına ait işletilmesi ile ilgili değişkenleri Şekil 4' de gösterildiği gibidir. Burada, i=1,2…M; Baraj sayısını, t=1,2…KM; Ay/gün/saat olarak zamanı göstermektedir. Bu sisteme ait herhangi bir i-barajı için t-zamandaki su dengesi ilişkisi;

$$\frac{dS}{dt} = X_{i,t} - Y_{i,t} \tag{1}$$

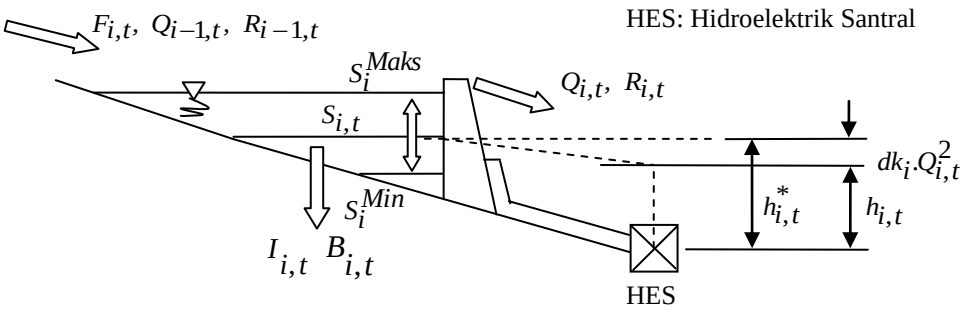
şeklinde yazılabilir. Burada,: $X_{i,t}$: Baraja giren akımları, $Y_{i,t}$: Barajdan çıkan akımları ve $\frac{dS}{dt}$: Barajda depolanan su miktarının zamanla değişimi olarak tanımlanmıştır. Ayrık zaman dilimi için su dengesi ilişkisi,

$$\frac{dS}{dt} = \frac{\Delta S}{\Delta t} \tag{2}$$

ile gösterildiğinde, buradan,



Şekil 3. Çok barajlı bir su kaynakları sisteminin şematik görünüşü.



Şekil 4. Herhangi bir barajın işletilmesi ile ilgili değişkenler.

$$\frac{\Delta S}{\Delta t} = X_{i,t} - Y_{i,t} \tag{3}$$

olarak yazılabilir. Burada, sisteme giren ve çıkan su miktarı,

$$X_{i,t} = F_{i,t} + Q_{i-1,t} + R_{i-1,t} \tag{4}$$

$$Y_{i,t} = Q_{i,t} + R_{i,t} + B_{i,t} + I_{i,t} - S_{i,t} \tag{5}$$

şeklinde olup, burada,

$S_{i,t}$: Barajda depolanan su miktarı,

$F_{i,t}$: Barajın havzasından gelen akım miktarı,

$Q_{i,t}$: Barajdan enerji üretimi için bırakılan akım miktarı,

$R_{i,t}$: Barajda dolu savaktan bırakılan akım miktarı,

$I_{i,t}$: Barajda sulamaya ayrılan akım miktarı ve

$B_{i,t}$: Barajdaki buharlaşma ve diğer kayıplar olarak tanımlanabilir.

Eğer $\Delta t = 1$ birim zaman olarak alınırsa, su dengesi ilişkisi,

$$\Delta S_i = S_{i,t+1} - S_{i,t} \tag{6}$$

$$S_{i,t+1} - S_{i,t} = F_{i,t} + Q_{i-1,t} + R_{i-1,t} - Q_{i,t} - R_{i,t} - B_{i,t} - I_{i,t} \tag{7}$$

şeklinde ifade edilebilir.

Çok amaçlı çok barajlı bir su kaynakları sisteminde her bir baraj için depolanan su miktarları, baraj maksimum ve minimum hacimleri ile sınırlanmış olmaktadır. Buna göre her bir barajda depolanmış su miktarı

$$S_i^{Min} \le S_{i,t} \le S_i^{Maks} \tag{8}$$

arasında olmakta, burada,

S_i^{Min} :Barajda depolanan minimum su miktarını

S_i^{Maks} :Barajda depolanan maksimum su miktarını göstermektedir.

Barajdan bırakılacak akımlar enerji üretim kapasitesine ve dolu savak kapasitesine bağlı olarak sınırlanabilir. Buna göre, barajdan bırakılan akımlar

$$0 \le Q_{i,t} \le Q_i^{Maks} \tag{9}$$

$$0 \le R_{i,t} \le R_i^{Maks} \tag{10}$$

olmaktadır. Burada,

Q_i^{Maks} : Barajdan enerji üretimi için bırakılabilecek maksimum su miktarını

R_i^{Maks} : Barajda dolu savaktan bırakılabilecek maksimum su miktarını

göstermektedir.

Diğer taraftan, barajlardan bırakılan akımların toplamına alt ve üst sınırlar tanımlanabilir. O zaman,

$$W_{i,t} \le (Q_{i,t} + R_{i,t}) \le W_i^{Maks} \tag{11}$$

arasında olup, burada,

$W_{i,t}$: Barajda akarsu yatağına bırakılması gereken kuraklık ve kirlilik kontrolü için minimum su miktarı

W_i^{Maks} : Barajda akarsu yatağına bırakılabilecek taşkın kontrolü için maksimum su miktarı olarak tanımlanmaktadır. Baraj etrafında bulunan tarımsal bölgelerde yapılacak sulama için ayrılan sulama miktarı değerleri minimum ve maksimum sulama miktarları arasında değişmektedir. Yani,

$$I_{i,t}^{Min} \le I_{i,t} \le I_{i,t}^{Maks} \tag{12}$$

şeklindedir. Burada,

$I_{i,t}^{Min}$: Barajdan sulamaya ayrılabilecek minimum su miktarını

$I_{i,t}^{Maks}$: Barajdan sulamaya ayrılabilecek maksimum su miktarını göstermektedir.

Her bir barajda enerji üretimi için bırakılan akımdan elde edilen güç hidroelektrik santral kurulu gücünü aşmaması gerekmektedir. Yani, ortalama güç,

$$P_{i,t} = k_i . Q_{i,t} . h_{i,t} \tag{13}$$

olup, $P_{i,t} \leq P_{k_i}$, olmalıdır. Burada, P_{k_i} : Barajda kurulu güç, k_i : Enerji üretim katsayısı ve

$h_{i,t}$: Ortalama net düşüdür. Barajlardaki ortalama su yüksekliği ise,

$$h^*_{i,t} = h\left(\frac{S_{i,t} + S_{i,t+1}}{2}\right) \tag{14}$$

şeklinde (ortalama depolanmış su miktarının bir fonksiyonu olarak elde edilmektedir. Cebri boru veya taşıma tünelindeki sürtünme kayıpları dikkate alındığında,

$$h_{i,t} = h^*_{i,t} - (h_f)_{i,t} \tag{16}$$

olmakta ve buradan

$$h_{i,t} = h^*_{i,t} - \alpha_i . Q^2_{i,t} . l \tag{17}$$

yazılmaktadır. Burada, $h^*_{i,t}$: Tüneldeki sürtünme kaybı düşünülmeden önce barajdaki ortalama su yüksekliği, $(h_f)_{i,t}$: Baraja ait cebri borudaki sürtünmeden dolayı yük kaybı, α_i : Barajda tünel çapına ve sürtünme özelliklerine bağlı bir katsayı ve l_i : Baraja ait cebri boru uzunluğu olarak tanımlanmaktadır. Burada, barajdaki $h_{i,t}$ yüksekliğine karşılık gelen depolama yüzey alanı belirlenip, bu değer buharlaşma yüksekliği ile çarpılarak buharlaşma miktarı belirlenmektedir.

ENERJİ ÜRETİMİ AMAÇLI UZUN SÜRELİ PLANLAMA İÇİN OPTİMİZASYON MODELİ

Çok amaçlı çok barajlı su kaynakları sisteminde uzun süreli planlama modelinde, zaman adımları olarak aylar kullanılmaktadır. Sistemin verilen boyutları için, ardışık yaklaştırmalı dinamik programlama (DPSA) modeli ile aylık kurak dönem akımları kullanılarak ve sulama miktarı sabit ve sıfır olduğu durum için güvenilir güç değerleri enbüyüklenmiş şekilde aylık minimum işletme seviyeleri($S^{Min}_{i,t}$) belirlenmektedir. Buradan elde edilen güvenilir güç değerleri modelde tek bir değer olarak tanımlanıp, sulama miktarı parametrik olarak değiştirilerek ve aylık ortalama akımlar ile toplam enerji enbüyüklenmekte ve aylık normal işletme seviyeleri ($S^{Nor}_{i,t}$) bulunmaktadır. Bunun için, modelde, amaç fonksiyonu, iki aşamalı olarak ele alınmaktadır

1. aşama, kurak dönemin aylık akımları ile güvenilir gücün enbüyüklenmesi şeklindedir. Yani,

$$Maks(Min \sum_{i=1}^{M} P_{i,t}) \tag{18}$$

olmaktadir.

2.aşama, aylık ortalama akımlarla toplam enerjinin enbüyüklennıesi şeklindedir. Yani,

$$Maks \sum_{t=1}^{KM} \left[(\sum_{i=1}^{M} P_{i,t} - P_G).p_s + P_G.p_G \right] \tag{19}$$

($Max \sum_{t=1}^{KM} \sum_{i=1}^{M} P_{i,t}$ ile eşdeğer)

$$\sum_{i=1}^{M} P_{i,t} \geq P_G \tag{20}$$

şeklinde olup, burada,

P_G : Enbüyüklenmiş güvenilir gücü (parametrik kısıt),

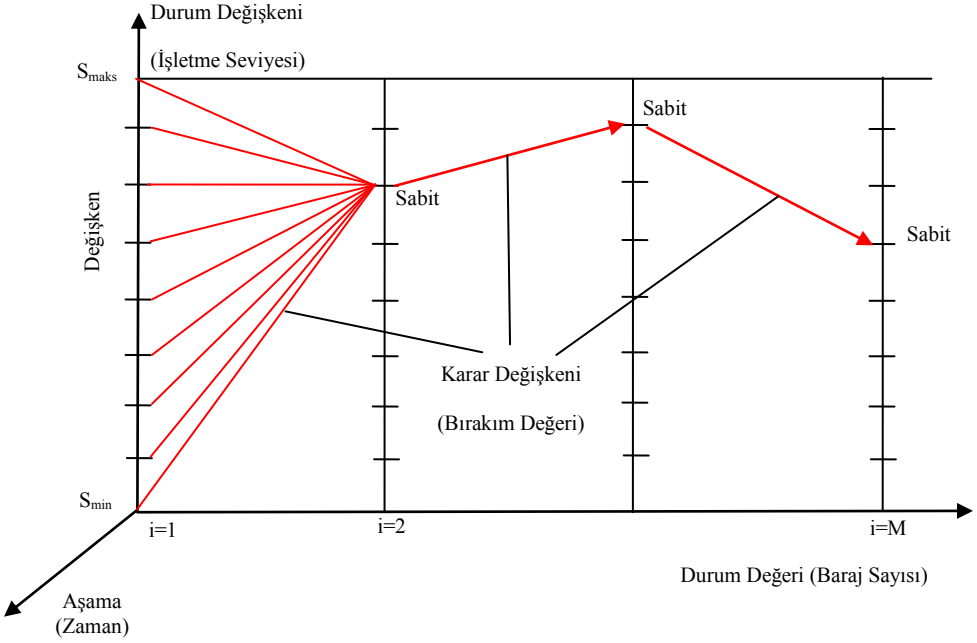
p_G : Güvenilir enerji birim fiyatını,

p_S : Sekonder enerji birim fiyatını göstermektedir. Güvenilir güç (P_G), kurak dönemin aylık akımları kullanılarak belirlenmekte ve aylık ortalama akımlara göre sekonder enerjinin enbüyüklenmesi ile toplam enerjinin enbüyüklenmesi sağlanmış olmaktadır.

Burada kullanılan enerji birim fiyatlarının sonuç üzerinde bir etkisi olmamakla birlikte, sadece toplam enerji faydasının değerini belirlemek için kullanılmaktadır. Yapısal denklemler olarak her bir baraj ve dönem için su dengesi ilişkisi, Denklem 7' de ifade edilmektedir. Sistemdeki kısıtlar ise, sisteme ait her bir barajdaki işletme seviyesinin değişimi, Denklem 8 ile gösterilmektedir. Barajdan bırakılacak akım miktarları, enerji üretimi ve dolu savaktan bırakılan akımlar, sırasıyla, Denklem 9 ve 10' da gösterildiği gibi tanımlanmaktadır.

ARDIŞIK YAKLAŞTIRMALI DİNAMİK PROGRAMLAMA (DPSA) OPTİMİZASYON TEKNİĞİ

Ardışık yaklaştırmalı dinamik programlama optimizasyon tekniği kullanmanın amacı, çok karar değişkenli bir dinamik programlama (DP) problemini, her birinde tek bir karar değişkeni olan DP ait problemlere ayrıştırmak ve böylece karar değişkenlerini teker teker ele alarak ana problemi çözmektir. Opan (2007), DPSA' da herhangi bir aşama için durum, durum değeri ve karar değişkeni için şematik gösterimini veren Şekil 5'i üretmiştir.



Şekil 5. DPSA' da herhangi bir aşamasındaki durum değeri ve durum-karar değişkeninin şematik gösterimi.

Bu metodun avantajı, n-boyutlu bir dinamik programlama problemini bir dizi tek boyutlu probleme indirgeyerek, hem hesaplama zamanını hem de bilgisayar bellek gereksinimlerini azaltmaktır. Çok barajlı ve çok amaçlı baraj işletmesinde uygulanan DP yöntemlerinin kullanıldığı bilgisayar belleği ve zamanı n-ye bağlı olarak üssel y-doğrusal bir artış göstermektedir. En iyi çözümün bulunabilmesi için başlangıç çözümünün dikkatli seçilmesi ve birden fazla başlangıç çözümü denenerek sonuçların karşılaştırılması gerekebilir. DPSA' da üç değişken tipi vardır. Bunlar:

1. Sistemin davranışını olduğu gibi tanımlayan ve sistemin herhangi bir andaki durumunu gösteren durum değişkenleri, (x, sistemde barajın işletme seviyesidir.)

2. Seçilen bir amaca göre sistemin kontrol edilebilir girdileri hakkında karar vermeyi sağlayan karar değişkenleri, (d, sistemde barajdan enerji üretimi için bırakılan ve dolu savaktan savaklanan su miktarlarıdır.)

3. Bu kararların verildikleri aralıkları belirleyen aşama değişkenleri, (t, sistemde işletme zamanını göstermektedir.)

şeklindedir.

Bu girdilerin bazı kısıtlara bağlı olarak aldıkları değer takımına politika denmektedir. Bu politikanın sistemin çıktıları üzerine etkisini belirleyen ölçüt ise, amaç fonksiyonu olarak tanımlanmaktadır.

PARAMETRİK DEĞİŞKEN YAKLAŞIMI

Çok barajlı sistemlerde barajlar birçok amaca hizmet etmektedir. Optimizasyon çalışmalarında aşağıda verildiği gibi çeşitli amaçlara ulaşılmaya çalışılmaktadır. Bunlar:

1. Enerji faydası ile sulama faydasının enbüyüklenmesi.

2. Enerji faydası enbüyüklenirken taşkın zararının enküçüklenmesi

3. Enerji ve sulama faydası enbüyüklenirken, taşkın zararının enküçüklenmesi

şeklinde olabilir. Ancak optimizasyon teknikleri içerisinde genelde amaç fonksiyonunda tek amacın fayda değeri gözetildiği için birden fazla amacın fayda değerlerinin amaç fonksiyonunda kullanılması ya hedef programlama optimizasyon tekniği ile yada parametrik değişken yaklaşımı ile mümkün olmaktadır.

Parametrik değişkenin herhangi bir optimizasyon yapısında kullanılması aşağıda verildiği gibidir:

1. Parametrik değişkenin alt ve üst sınırları arasında herhangi bir değer atanması.

2. Atanan bu değere karşılık fayda değerinin bilinmesi. (Örneğin: Parametrik değişken olarak kabul edilen sulama miktarına karşılık gelen sulama faydasının bilinmesi gibi)

3. Optimizasyon yapısı içerisinde yapısal denklemlerin içerisine parametrik değişken olarak konulan değişkenin herhangi bir değerine karşılık amaç fonksiyonunun ürettiği değerin belirlenmesi.

4. Parametrik değişken faydası ile amaç fonksiyonun ürettiği fayda değeri arasında bir ilişki ortaya çıkartılması,

şeklindedir. Tek amaç fonksiyonu olan optimizasyon yapısı içerisinde yapısal denklemlerin içerisine istenilen amaçlar doğrultusunda gerektiği kadar parametrik değişken kullanılabilir. Çok barajlı sistemlerde parametrik değişken olarak aşağıda verilen değişkenler kullanılabilir.

1. Enerji üretimi: Kurak Dönemlerde farklı sulama miktarlarına karşılık güvenilir güç parametrik olarak değişebilir. Bu çalışmada toplam enerjinin enbüyüklenmesi için optimizasyon modelinde güvenilir güç parametrik değişken ve kısıt olarak kullanılmıştır.

2. Sulama miktarı: Minimum ve maksimum sulama miktarları arasında sulama miktarı parametrik olarak değiştirilebilir. Bu çalışmada sulama miktarı parametrik değişken olarak kullanılmıştır.

3. Taşkın-Kuraklık Kontrolü: Taşkın için mansapta maksimum bırakım miktarına kadar, kuraklık için ise mansapta minimum ile maksimum bırakım miktarına kadar değerler atanarak taşkın ve kuraklık kontrolü parametrik olarak değiştirilebilir.

4. Kirlilik Kontrolü ve Ulaşım: Kirlilik kontrolü ve ulaşım, mansapta minimum ve maksimum bırakım miktarı arasında değerler atanarak parametrik değişken olarak kullanılabilirler.

OPTİMİZASYON MODELLERİ İÇİN UYGULAMA

Munzur Nehri Havzası'nda Munzur Nehri ana kolu üzerinde ardışık şekilde bulunan çok barajlı bir su kaynakları sistemi uygulama amaçlı olarak seçilmiştir. Seçilen çok barajlı sistemin şematik görünüşü, Şekil 6'da verilmektedir. Bu sisteme ait barajlardan Y. Konaktepe, Kaletepe, Uzunçayır Barajları enerji amaçlı ve biriktirmeli; Gülyayla ve Tünceli Barajları ise enerji amaçlı ve biriktirmesiz (nehir santrali) olarak planlanmışlardır. Tablo 1' de bu barajlara ait veriler ve Tablo 2' de, biriktirmeli barajlara ait hacim yükseklik ilişkileri verilmiştir. Şekil 6' da kritik döneme ait aylık kurak akımları verilmektedir. Bu kritik akım verileri uzun süreli optimal işletme modelinde kullanılarak ve sulama miktarı parametrik olarak değiştirilerek güvenilir güç değerleri enbüyüklenirken aylık minimum işletme seviyeleri elde edilecektir. Şekil 7' de 27 yıllık gözlenmiş akım verilerinde elde edilen aylık ortalama akımlar verilmiştir. Bu ortalama akım verileri uzun süreli optimal işletme modelinde kullanılarak, sulama miktarı parametrik olarak değiştirilerek ve enbüyüklenmiş güvenilir güç değerleri modele parametrik kısıt olarak yerleştirilerek toplam enerji enbüyüklenirken aylık normal işletme seviyeleri elde edilecektir.

Tablo 1. Munzur Nehri Havzası' nda bulunan barajlara ait veriler, [Temiz (2008)].

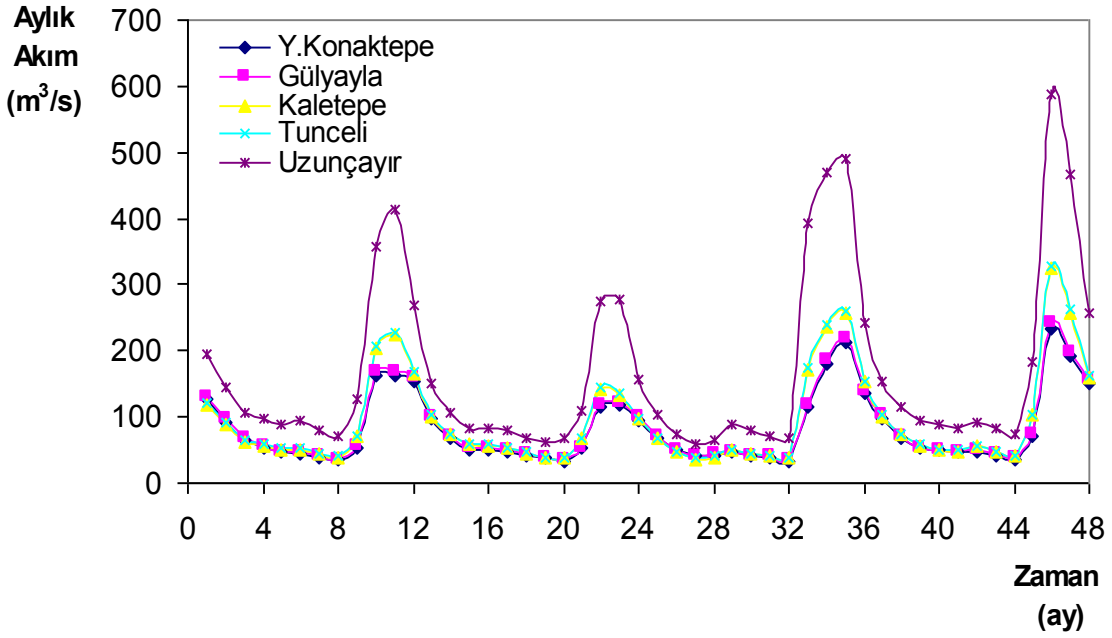
Barajlar	Y. Konaktepe	Gülyayla	Kaletepe	Tunceli	Uzunçayır
Yağış Alanı (km^2)	1086	1151	1586	1630	3335
Kurulu Güç (MW)	105	50	53	21	117
Baraj Yüksekliği (m)	110	25	65	25	58
Maksimum İşletme Seviyesi (m)	1240	1070	990	925	900
Minimum işletme seviyesi (m)	1170	1070	960.3	925	873.8
Maksimum Hacim (10^6 m^3)	456	6	47.5	5	303
Minimum Hacim (10^6 m^3)	20	6	10	5	40
Kuyruksuyu kotu (m)	1070	990	925	900	840
Enerji Üretim Kapasitesi (10^6 m^3)	200	200	250	260	500

Tablo 2. Barajlarda hacim yükseklik ilişkisi ($h=a.S^b+c$, h (m), S (10^6 m^3)), [Temiz (2008)].

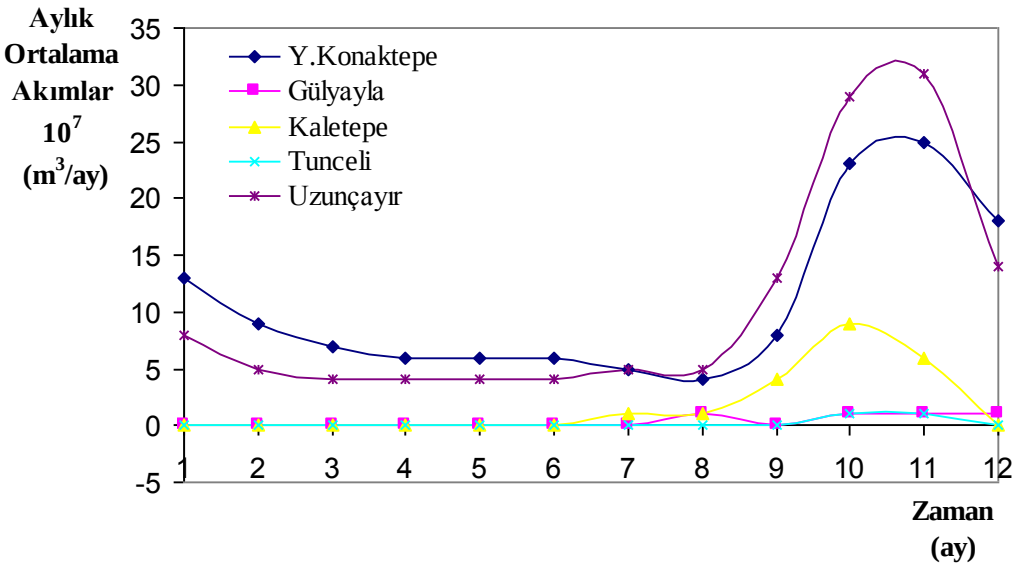
Barajlar	Y. Konaktepe	Gülyayla	Kaletepe	Tunceli	Uzunçayır
A	17.624	17.624	13.440	13.440	11.110
B	0.3005	0.3005	0.4134	0.4134	0.2850
C	60	55	0	0	2

Tablo 3. Barajlardaki buharlaşma değerleri (mm/ay), [Temiz (2008)].

Barajlar	Oc	Şub	Mar	Nis	May	Haz	Tem	Ağu	Ey	Ek	Kas	Ar
Y.Konaktepe	0	0	0	0	107.92	177.85	211.16	196.04	112.23	21.98	0	0
Kaletepe	0	0	0	0	107.92	177.85	211.16	196.04	112.23	21.98	0	0
Uzunçayır	0	0	0	0	41.49	171.50	232.45	231.98	141.86	8.18	0	0

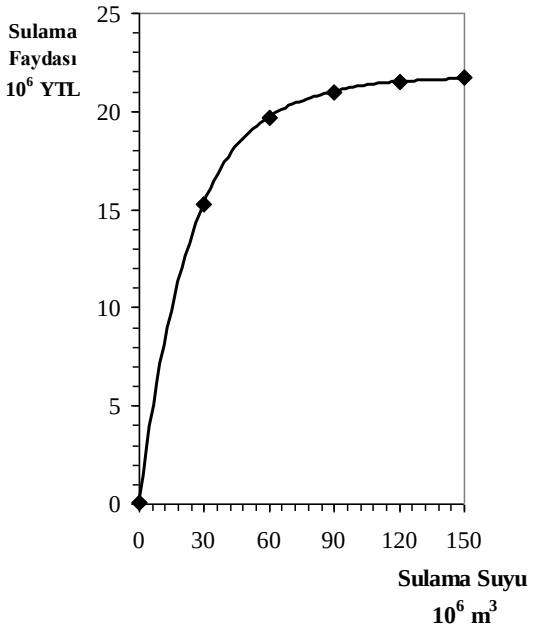


Şekil 6. Barajlara kritik dönemde havzasından gelen aylık kuraklık akımlar, [Temiz (2008)].



Şekil 7. Barajlara havzasından gelen aylık ortalama akımlar, [Temiz (2008)].

Sert ve Ark. (2007), Munzur Nehir Havzası' nda bulunan çok barajlı bir su kaynakları sistemi için birim sulama alanına düşen enbüyük faydaya sahip ürünleri (Domates, salatalık, biber ve patlıcan) belirledikten sonra, baraj etrafındaki mevcut tarım bölgeleri sulanarak bu ürünlerden oluşan toplam parasal net fayda değerlerini hesaplamışlardır. Buradan, Şekil 8' de verilen sulama miktarına karşılık sulama faydasının değişimini elde etmişlerdir.



Şekil 8. Sulama suyu değişimi ile sulama suyu faydası.

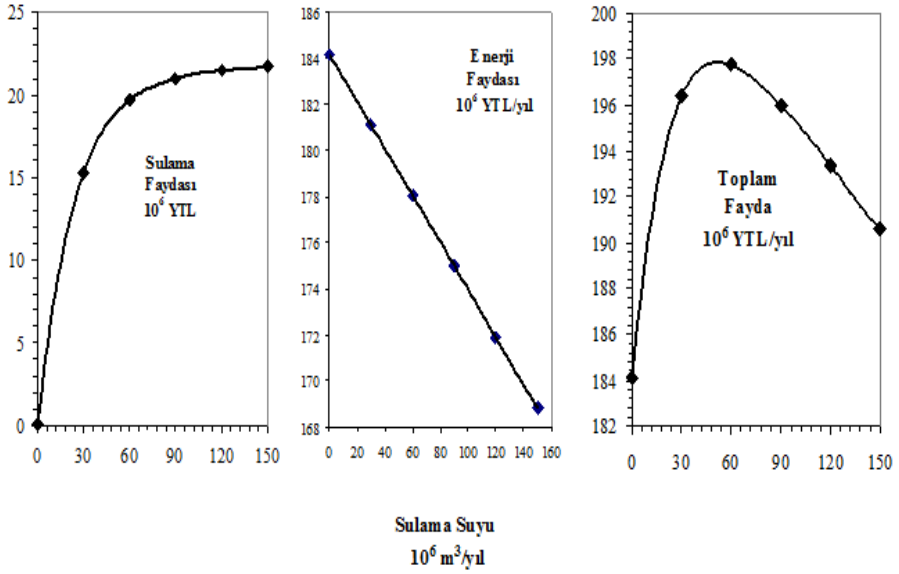
Sulama miktarının parametrik değişken olarak göz önüne alındığı uzun sureli planlama için işletme optimizasyonu sonucunda, su miktarına karşılık güvenilir/sekonder/toplam güç değerleri Tablo 4' de verildiği gibi elde edilmiştir. Burada, su miktarı arttığı zaman, enerji üretiminin azaldığı görülmektedir. Tablo 5' de, güvenilir/sekonder/toplam güç değerlerine ait birim fayda değerleri verilmektedir. Bu veriler kullanılarak, Şekil 9'da, sulama miktarına karşılık sulama, enerji üretimi ve toplam fayda değerleri elde edilmiştir. Burada, sulama miktarı artığı zaman, sulama faydası artmakta, enerji üretimi faydası azalmakta ve toplam fayda değeri su miktarının 60.10^6 m^3 olduğu yerde $197{,}751.10^6$ TL/Yıl olmakta ve en büyük değere ulaşmaktadır. Toplam fayda değerini en büyük yapan su miktarı optimum su miktarı olarak kabul edilmektedir.

Tablo 4. Sulama miktarına karşılık enbüyüklenmiş güvenilir/sekonder/toplam güç değerleri.

Sulama miktarı $I_{1,t}$ (10^6 m^3)				**Güvenilir Güç**	**Seconder Güç**	**Toplam Güç**
Haziran	**Temmuz**	**Ağustos**	**Toplam**	(MW/Yıl)	(MW/Yıl)	(MW/Yıl)
0	0	0	0	104,892	57,588	162,480
10	10	10	30	104,892	54,438	159,330
20	20	20	60	104,892	51,278	156,170
30	30	30	90	104,892	48,118	153,010
40	40	40	120	104,892	44,868	149,760
40	40	40	120	104,892	41,718	146,610

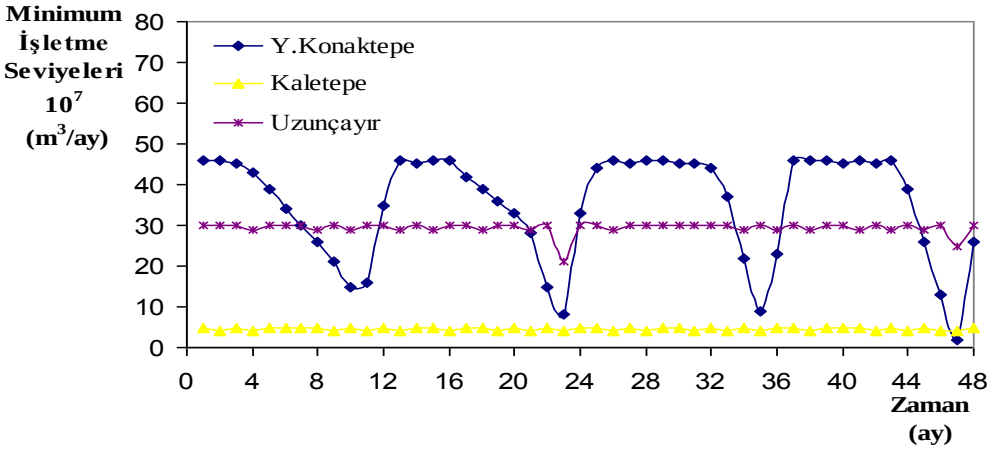
Tablo 5. Güvenilir/sekonder/toplam güç, enerji ve fayda değerler.i

	Güç (MW)	Enerji/Yıl (MW-saat)	Birim Fiyat (TL/kW-saat)	Fayda (10^6 TL/Yıl)
Güvenilir	104,892	918 853	0.14	128,639
Sekonder	57,588	504 470	0.11	55,491
Toplam	162.480	1 423 323	-	184.130

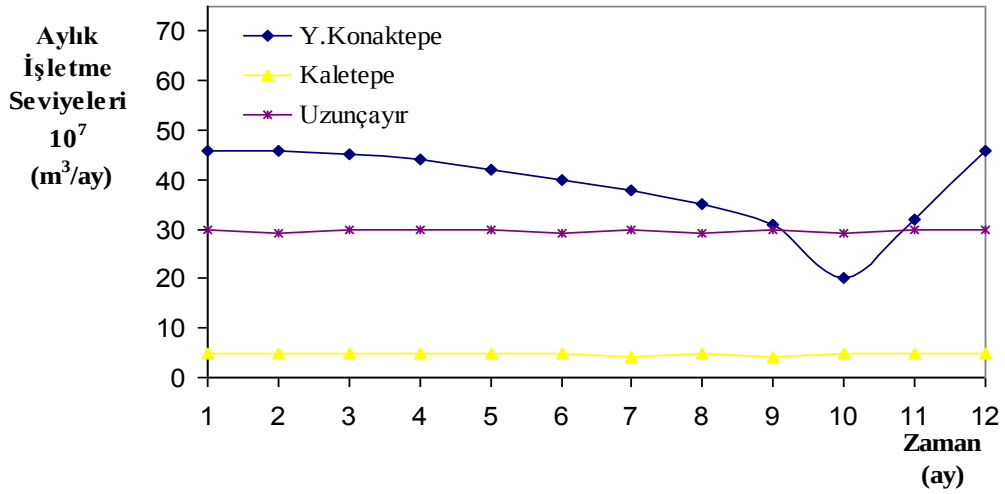


Şekil 9. Sulama miktarına karşılık sulama, enerji üretimi ve toplam fayda değerleri.

Sulama miktarı değerine ait aylık minimum işletme seviyeleri Şekil 10' de, aylık normal işletme seviyeleri ise, Şekil 11' de gösterilmektedir. Burada, elde edilen işletme seviyeleri incelendiğinde, en büyük işletme hacmine sahip barajların optimizasyon sürecini kontrol ettiği ve yönettiği görülmektedir.



Şekil 10. Barajlarda kurak dönemde optimum sulama miktarı ile oluşan aylık minimum işletme seviyeleri.



Şekil 11. Barajlarda optimum sulama miktarı ile oluşan aylık normal işletme seviyeleri.

SONUÇLAR VE ÖNERİLER

Sulama miktarının parametrik olarak ele alındığı enerji üretiminin enbüyüklenmesi amaçlı uzun süreli optimal işletme modeli Munzur Havzası'nda Munzur Nehri ana kolu üzerinde bulunan çok barajlı bir su kaynakları sistemine başarılı bir şekilde uygulanmıştır. Yapılan optimal işletme neticesinde, sulama miktarı artığı zaman, sulama faydası artmakta, enerji üretimi faydası azalmakta ve toplam fayda değeri su miktarının 60.10^6 m^3 olduğu yerde $197{,}751.10^6$ TL/Yıl olmakta ve en büyük değere ulaşmaktadır. Toplam fayda değerini en büyük yapan sulama miktarı optimum sulama miktarı olarak kabul edilmektedir. Optimum sulama miktarına karşılık elde edilen optimal işletme seviyeleri incelendiğinde, en büyük işletme hacmine sahip barajların optimizasyon sürecini kontrol ettiği ve yönettiği görülmektedir.

İleriki zamanlarda yapılacak çalışmalar olarak, optimizasyon modelinde, sulama miktarı için farklı senaryolar kullanılarak analizler yapılabilir.

TEŞEKKÜR

Mahmut Sert 1 Mayıs 2007 tarihinde vefat etmiştir. Bu tarihe kadar bu çalışmanın ortaya çıkartılması ve geliştirilmesi üzerine çok büyük katkılarda bulunmuştur. O olmadan bu çalışma asla yapılmazdı. Katkılarından dolayı kendisine çok teşekkür ederiz.

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OPTIMUM STACKING SEQUENCE OF SYMMETRIC HYBRID LAMINATED THIN PLATES FOR MAXIMUM FREQUENCY SEPERATION

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Key Words: Hybrid laminated plates, modified feasible direction method, frequency, optimal designs

ABSTRACT This paper deals with design of hybrid symmetric angle-ply laminated thin plates consisting of high-stiffness surface and low-stiffness core layers. The objective function is the maximization of the frequency separation of the laminates and the design variable is the fibre orientation. The classical laminated plate theory (CLPT) is used for the finite element solutions. Modified feasible direction (MFD) method is used for the optimization method. Finally, the effect of hybridization is investigated for various parameters of the laminate such as the aspect ratio, the number of layers and boundary conditions. Results are given for hybrid graphite-epoxy/glass-epoxy laminates.

INTRODUCTION

Laminated composite plates, one of the most basic structural elements, have been increasingly applied in many industry fields such as aviation, automobiles, marine and submarine vehicles, aerospace and mechanical/civil engineering etc. This is mainly due to the fact that these components, over the conventional and traditional structures, have great advantages of promising high stiffness and high strength to weight ratios and high rigidity. Hence, the pursuit of corresponding mechanical and mathematical models for predicting their behavior of dynamic characteristics, which is of great significance in modeling many mechanical parts in the afore-mentioned employment aspects besides other systems, has been an intensive research focus for several decades.

On the other hand, the maximum frequency problems are of practical importance in the design of laminates against resonance due to external excitation. The frequency of an external excitation can be placed either between zero and the fundamental frequency or in a gap between two consecutive higher-order frequencies depending on its magnitude.

Recently, fundamental frequency maximization of laminated plates has been investigated by some authors. Topal and Uzman, (2006) studied the maximization of the fundamental frequency of symmetrically laminated plates. The fibre orientation is considered as design variable. The effects of number of layers, boundary conditions, laminate thicknesses, aspect ratios and in-plane loads on the optimal designs are studied. Apalak *et al.*, (2008) carried out the layer optimization for maximum fundamental frequency of laminated composite plates under any combination of the three classical edge conditions. The optimal stacking sequences of laminated composite plates were searched by means of genetic algorithm. Abdalla *et al.*, (2007) studied maximization of the natural frequency of composite panels. The variation of the stiffness properties was parameterised using lamination parameters, and the fundamental frequency is predicted based on classical lamination theory. A new structural approximation scheme was introduced which is referred to as the generalised reciprocal approximation. The optimality conditions for the problem are formulated based on this generalised reciprocal approximation. Narita, (2003) proposed a layerwise optimization approach to determine the maximum fundamental frequency of laminated plates. Fukunaga *et al.*, (1994) examined the optimal laminate configurations of symmetric laminated plates for maximizing

fundamental frequencies. Diaconu *et al.*, (2002) examined the optimum laminate configurations for maximizing the fundamental frequencies of symmetrically laminated thick plates. Layup optimization was performed by introducing four out of-plane and two in-plane lamination parameters that characterize the out-of-plane and the transverse shear stiffnesses, respectively.

BASIC EQUATIONS

Consider a hybrid laminated plate of length *a*, width *b* and thickness *h* in the *x*, *y* and *z* directions, respectively. The hybrid laminate is composed of an even number of orthotropic layers (N) with fibre orientation θ_k (k=1, 2,…., N). The surface layers are made of a composite with high stiffness fibre reinforcements and the core layers of a composite with low stiffness reinforcements.

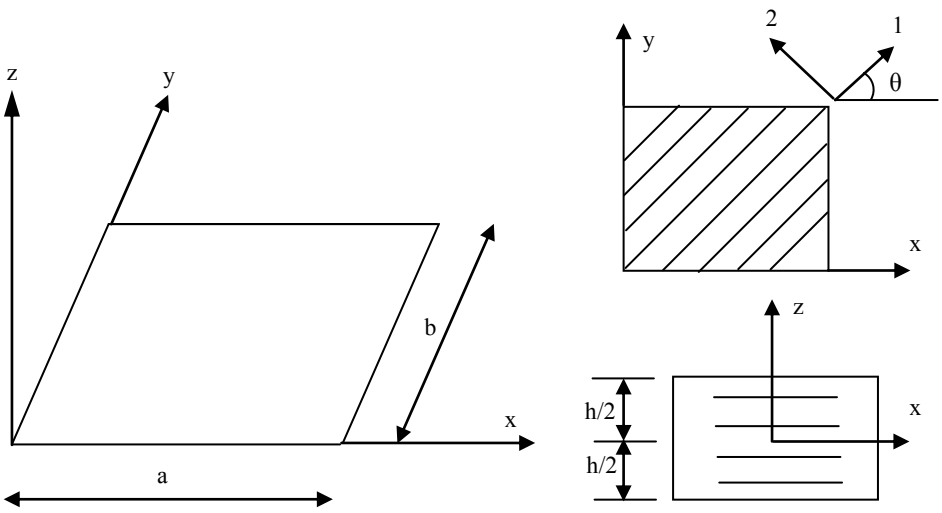

Fig. 1. Geometry of a laminated plate.

In this present study, the classical laminated plate theory is employed to analyse the problem and the following displacement field is assumed:

$$u(x,y,z) = u_o(x,y) - z\frac{\partial w}{\partial x}, \quad v(x,y,z) = v_o(x,y) - z\frac{\partial w}{\partial y}, \quad w(x,y,z) = w_o(x,y) \tag{1}$$

The membrane strains $(\varepsilon_x, \varepsilon_y, \gamma_{xy})$ and the bending curvatures $(\kappa_x, \kappa_y, \kappa_{xy})$ are defined as follows:

$$\{\varepsilon\} = \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = \begin{Bmatrix} u_{o,x} \\ v_{o,y} \\ u_{o,y} + v_{o,x} \end{Bmatrix}, \quad \{\kappa\} = \begin{Bmatrix} \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix} = \begin{Bmatrix} -w_{o,xx} \\ -w_{o,yy} \\ -2w_{o,xy} \end{Bmatrix} \tag{2}$$

The stress resultants (N_x, N_y, N_{xy}) and the moment resultants (M_x, M_y, M_{xy}) are defined as follows:

$$\begin{Bmatrix} N_x \\ N_y \\ N_{xy} \\ M_x \\ M_y \\ M_{xy} \end{Bmatrix} = \begin{bmatrix} A_{11} & A_{12} & A_{16} & B_{11} & B_{12} & B_{16} \\ & A_{22} & A_{26} & B_{12} & B_{22} & B_{26} \\ & & A_{66} & B_{16} & B_{26} & B_{66} \\ & & & D_{11} & D_{12} & D_{16} \\ & \text{sym} & & & D_{22} & D_{26} \\ & & & & & D_{66} \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \\ \kappa_x \\ \kappa_y \\ \kappa_{xy} \end{Bmatrix} \tag{3}$$

where A_{ij}, B_{ij} and D_{ij} denote extensional, coupling and bending stiffness coefficients of the laminates, respectively. A_{ij}, B_{ij} and D_{ij} can be calculated as follows

$$\left(A_{ij}, B_{ij}, D_{ij}\right) = \int_{-h/2}^{h/2} \bar{Q}_{ij}\left(1, z, z^2\right) dz \text{ (i, j=1, 2, 6)} \tag{4}$$

FINITE ELEMENT FORMULATION

In this study, four-noded Lagrangian rectangular elements having three degrees of freedom per node is used for the frequency analysis of laminated plates. Using the same shape functions associated with node i (i=1, 2,…, n), Φ_i, for interpolating the variables in each element, the displacement vectors can be written as

$$u = \sum_{i=1}^{n} \Phi_i(x, y) u_i \tag{5}$$

where u_i is the value of the displacement vector corresponding to the node i, and is given by

$$u = \left\{u_o^{(i)}, v_o^{(i)}, w_o^{(i)}\right\}^T \tag{6}$$

The problem of free undamped vibrations of a finite element discretized linearly elastic structure is defined by the eigenvalue problem

$$\left([K] - \lambda[M]\right)\{u\} = 0 \tag{7}$$

where $[K]$, $[M]$ and $\lambda = \omega^2$ are the stiffness matrice, mass matrice and eigenvalue, respectively. Eq. (7) is a set of homogeneous linear equations in the unknown displacements $\{u\}$. For non-trivial solution, the determinant is equal to zero and the eigenvalues correspond to natural frequencies of the laminated plates. The subspace iteration method is used for the frequency analysis.

OPTIMIZATION PROBLEM

The optimal design problems involve the selection of the optimal stacking sequence which yields the maximum fundamental frequency seperation of the hybrid angle ply laminated plates. Fibre orientations are taken as design variables and $\theta_k = (-1)^{k+1}\theta$ for $k \le N/2$ and $\theta_k = (-1)^k \theta$ for $k \ge N/2+1$ are considered. The optimal design problem can be stated mathematically as follows:

Find: θ,

Maximize: $\omega_i - \omega_{(i-1)}$, $\quad i = 2, 3$, (8)

Subject to $0^o \le \theta_k \le 90^o$

The modified feasible direction method (MFD) is used in the optimization stage. The flow chart of this method is given in Figure 2.

MFD

1. *q=0,* $X^q = X^m$
2. *q=q+1*
3. Evaluate objective function $F(X_i)$
4. Calculate gradient of the objective function $F(X_i)$
5. Find the usable-feasible direction S_q
6. Perform a one-dimensional search
   $X^q = X^{q-1} + \alpha\ S_q$
7. Check convergence. If satisfied, go to 8. Otherwise go to 2
8. $X^{m+1} = X^q$

Fig. 2. The flow chart of modified feasible direction method.

NUMERICAL RESULTS AND DISCUSSION

Consider multilayered hybrid laminate with graphite-epoxy outer layers and glass-epoxy inner layers. The material properties of the materials are given below.

- graphite-epoxy (T300/5280) $E_1 = 181\text{GPa},\ E_2 = 10.3\text{GPa},\ G_{12} = 7.17\text{GPa},\ \upsilon_{12} = 0.28$, $\rho = 1600\,\text{kg/m}^3$
- glass-epoxy (Scotchply 1002) $E_1 = 38.6\text{GPa},\ E_2 = 8.27\text{GPa},\ G_{12} = 4.14\text{GPa},\ \upsilon_{12} = 0.26$, $\rho = 1800\,\text{kg/m}^3$.

The dimensionless *i*th order frequency, ϖ_i is defined as

$$\varpi_i = (\omega_i b^2 / h)(\rho_o / E_o)^{1/2} \tag{9}$$

where ρ_o and E_o are reference values taken as $\rho_o = 1\text{kg/m}^3$ and $E_o = 1\text{GPa}$.

It is noted that placing the stronger material in the outer layers and the weaker material in the inner layers leads to a sandwich-type structure and provides the optimum material utilisation. In the calculations, the thickness *h* of the laminate kept constant. However, the thickness *t* of a single layer given by *t=h/N* varies as *N* changes. The reason for this procedure is to compare the performance of equal thickness designs with respect to the specific design objective.

Firstly, the effect of total number of layers is investigated on the optimal stacking sequence and maximum frequency seperations for simply supported square angle-ply laminated plates. The optimal stacking sequences are obtained for a given total number of layers (N) while the number of glass layers (N_{gl}). In this study, b/h=100 is taken. Figure 3 shows dependence of the maximum frequency seperation on the number of glass layers for 8-, 12- and 16- layered laminates for both $\varpi_2 - \varpi_1$ and $\varpi_3 - \varpi_2$. As seen, the maximum frequency seperation decreases as N_{gl} increases. It diminishes rapidly after the N_{gl} exceeds 3N/4 and N_{gl} is equal to N for $\varpi_2 - \varpi_1$ and $\varpi_3 - \varpi_2$, respectively. It is observed that $\varpi_2 - \varpi_1$ and $\varpi_3 - \varpi_2$ of an half-graphite/half-glass construction is only

5.78% and 3.88% less than those of the all graphite laminates, for 8-layered plates, respectively. Also, it can be seen that $\varpi_2 - \varpi_1$ and $\varpi_3 - \varpi_2$ of the all glass laminates are 48.18% and 32.29% less than those of the all graphite laminates for 8-layered plates, respectively. On the other hand, as seen from Figure 4, as the number of layer increases the maximum frequency seperation increases. The optimum fibre orientations are obtained $\theta_{opt} = 45^\circ$ and $\theta_{opt} = 0^\circ (90^\circ)$ for $\varpi_2 - \varpi_1$ and $\varpi_3 - \varpi_2$,, respectively.

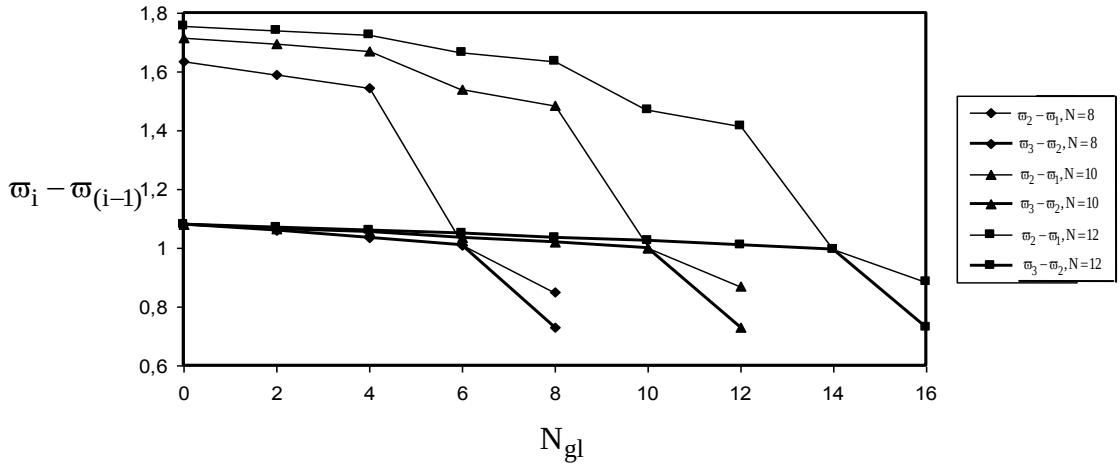

Fig. 3. The maximum $\varpi_i - \varpi_{(i-1)}$ versus the number of glass layers N_{gl} with a/b=1.

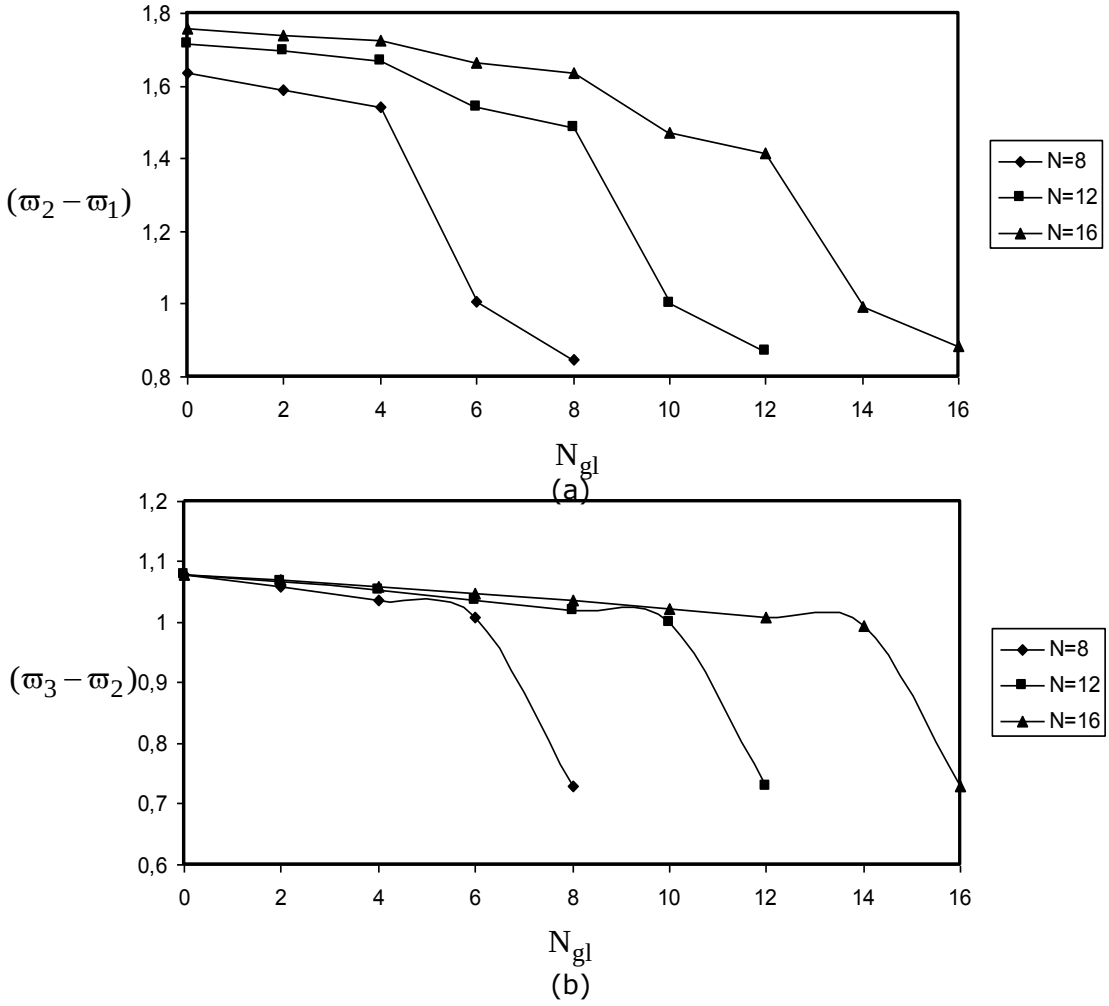

Fig. 4. Effect of number of layers on the maximum frequency separation.

In this study, effect of aspect ratio (a/b) on the optimum results is investigated for simply supported angle-ply laminated plates (N=8, b/h=100, $N_{gl} = N/2$). The maximum frequency seperation is investigated for only $\varpi_2 - \varpi_1$. As seen from Figure 5, as a/b ratio increases the maximum frequency seperation decreases. In Table 1, effect of a/b ratio on the optimum fibre orientations is given.

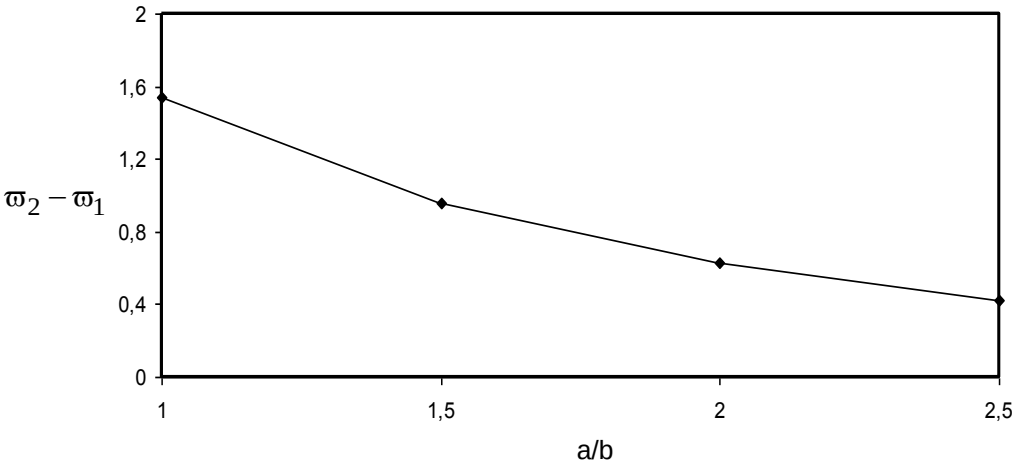

Fig. 5. Effect of a/b ratio on the maximum frequency separation.

Table 1. Effect of a/b ratio on the optimum fibre orientation.

a/b	$\theta_{opt}(^\circ)$
1	45,0
1,5	30,9
2	0,0
2,5	28,3

The optimization problem may be applied to laminated plates with any combinations of simple support (S), clamped support (C), and free edge (F). Four different combinations of the boundary conditions as (SSSS), (CSCS), (CCCC) and (FSCS) are considered. A clamped-simple-clamped-simple (CSCS) is a specimen with clamped on x=0 and x=a, and simple on y=0 and y=b, respectively. The maximum frequency seperation is investigated for only $\varpi_2 - \varpi_1$. Table 2 shows the effect of different boundary conditions on the optimal designs for square angle-ply laminated plates (N=8, b/h=100, $N_{gl} = N/2$). As seen from Table 2, as the degree of freedom decreases, the maximum frequency seperation increases.

Table 2. Effect of boundary conditions on the optimum design.

Boundary conditions	$\varpi_2 - \varpi_1$	$\theta_{opt}(^\circ)$
(SSSS)	1,5426	45,0
(CCCC)	1,9436	45,0
(CSCS)	1,6965	52,7
(FSCS)	1,2589	44,6

CONCLUSIONS

The problem of stacking sequence design for maximum frequency seperation is investigated for hybrid composite laminated plates consisting of high-stiffness surface and low-stiffness core layers. The effect of hybridisation, aspect ratio and boundary conditions on the optimal stacking sequence and maximum frequency seperation are investigated. It is shown that the maximum frequency seperation decreases as N_{gl} increases. As the number of layer increases the maximum frequency seperation increases. As a/b ratio increases the maximum frequency seperation decreases because of decrease of stiffness of the plate. The maximum and minimum frequency seperation occurs at (CCCC) and (FSCS) boundary conditions, respectively. This paper can be investigated for design of minimum material cost and the results can be compared.

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RESTRAINED SHRINKAGE: AN ENEMY CRYPTIC

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Key Words: Restrained Shrinkage, Reinforced Concrete, Structural Analysis, Crack

ABSTRACT Shrinkage, by wording, means loss of volume due either to a chemical process, or a thermal activity or due and mainly to the loss of free water from the composite material itself; all having distinct names, such as autogenous, thermal and drying shrinkage. Shrinkage may mainly be divided into two categories: free shrinkage and restrained shrinkage. Free shrinkage does not result in any internal force in the reinforced concrete members. Such a shrinkage usually takes place in statically determinate structures. Simply supported bridge beams, beams and columns of a pin supported type of precast construction or a single foundation member of a precast frame may be listed as the members of a statically determinate system. On the other hand, restrained shrinkage may yield forces well beyond the capacity of either the shrinking member or the restraining members. In case of a staged construction, where the shear walls are cast before the reinforced concrete floor slabs, the shear walls undergo free shrinkage throughout the curing period, while the slabs casted afterwards and attached to these cured members experience restrained shrinkage. In turn, lateral loads causing flexural moments and shears are observed on shears walls, while in plane tensile forces are observed in the slab plates during the curing period of the floor slab itself. The in-plane floor slab forces are consistent with the lateral stiffness of the vertical members. In other words, the stiffer the shear walls, the higher the in plane tensile stresses in the floor slabs. The forces, either moment, shear or in plane tension, may cause unexpected and immature failures.

Restrained shrinkage should be considered in the structural analysis in order to calculate the reinforcement required to carry the induced loads. Experimental and analytical research yielded that the shrinkage induced member forces results in cracks and the reinforcement crossing such cracks may reach yielding. It should be noted that the yielded reinforcement may no more create an increasing resistance to existing loads, moreover undergoes increasing unpredictable deformations.

Authors have experienced several structural overloads, sometimes failures, due to restrained or partially restrained shrinkage. The reinforcement amount and the distribution calculated through linear structural analysis performed without shrinkage loads is usually not enough in the case of shrinkage loading. The shrinkage cracking strength may well be calculated via the material properties of the concrete, while the shrinkage force is calculated through the shrinkage properties of the concrete and through the adverse environmental affects.

In this paper, failures due to shrinkage in some structures will be given and the means of shrinkage calculation in structural analysis will be discussed.

INTRODUCTION

In this paper case studies on restrained shrinkage and its adverse effects on reinforced concrete structures are discussed. National reinforced concrete design codes all over the world enforce the shrinkage analysis, in particular for the structures with thick concrete members. The current practice in structural analysis to include the shrinkage affects is very similar to the temperature analysis. In the subsequent part of this study, general

information on shrinkage will be summarized, design code approaches and the design equations will be highlighted and finally some case studies will be outlined.

SHRINKAGE MECHANISMS

Concrete is a strong and solid material that is composed of cement paste matrix and aggregates, such as gravel, and sand out of limestone, diabase, etc. Aggregates are used to fill the unit volume with strong and stiff materials, whereas the cement paste matrix, which is out of cement and water, takes the duty of uniting the aggregates and revealing a solid and strong composite material, which is concrete. Cement is a material that needs water for hydration. Hydration is a chemical reaction between cement and water that produces heat and sort of volume changes. The necessary amount of water for a through hydration is approximately 25% to 27% by weight of cement. However green concrete with this amount of water could not get its famous property, which is workability. Therefore, water content of green concrete is usually kept between 40% and 70%, by weight of the cement content. Compressive strength is one of other famous and demanded property of concrete. The higher the water to cement ratio (w/c), the lower the strength of the concrete.

Volume change of concrete, which can be named as the most adverse property of concrete along with the creep, may happen throughout the life of the concrete member, either in the form of shrinking or swelling cycles. Shrinkage, by wording, means loss of volume due either to a chemical process, or a thermal activity or due and mainly to the loss of free water from the porous composite material itself. Green concrete w/c ratio, ingredients of concrete such as cement type, silica fume existence, aggregate type and shape, curing conditions, ambient conditions such as evaporation speed, temperature and wind speed, and the ratio between the total area of drying face to the total volume of the drying member affects the shrinkage issue.

Concrete is faced with several types of shrinkage throughout its service life. Most common and known type is the plastic shrinkage. It is described as the water loss from concrete by evaporation and the rate of evaporation faster then bleeding water in concrete body to the surrounding environment resulting contraction. This phenomenon happens before concrete set and already in plastic phase.

Besides contraction in the plastic phase due to the loss of water, shrinkage may also be categorized into three groups after the set of concrete. Chemical shrinkage, which may be categorized as autogenous and the carbonation is the first group. Holt and Jansen explain that the autogenous shrinkage is associated with the loss of water from the capillary pores due to the hydration of the cement (Mokarem et all 2003). Autogenous shrinkage described as a self-desiccation event which creates empty pores within the hydrating paste structure and It is reported that considerable shrinkage may result due to the autogenous shrinkage (Bentz and Jensen 2004). Carbonation shrinkage is caused by the chemical reaction of various cement hydration products with carbon dioxide present in the air (Mokarem et all 2003). This kind of shrinkage also causes and to be blamed corrosion and rusting of reinforcing bars in concrete. Carbonation is generally considered a durability issue that takes a long time, in the order of many years, to affect a concrete structure (Holt 2001).

Second group of the shrinkage classification is the thermal shrinkage. It happens both in the early and late ages. During the setting process of concrete, the volume of concrete members increases due to the heat of hydration. The reach of maturity and the highest temperature developed due to hydration takes place approximately simultaneously. When the hydration stops, or slows down, the concrete member starts to shrink due to the loss of heat resulting thermal contraction, which is called the thermal shrinkage (Nejadi 2005). It is important to note that the tensile strength of green concrete hardening at such a high temperature level is much lower because of the shorter hydration process. Thus, temperature increase during the hydration period may not

cause cracking of concrete; rather the cracking results from the stresses that exceed the tensile strength of the material during the cooling period. The relationships between strength development, modulus of elasticity and the coefficients of thermal expansion and contraction are very important for the cracking of concrete (Ah-Sha et all 2001). It should be emphasized as a common knowledge that thermal expansion coefficient of concrete also varies in early age of concrete. Especially mass concrete and thick concrete sections are vulnerable to thermal restrained shrinkage.

The last item in the list is the drying shrinkage as commonly called just "shrinkage". Basically drying shrinkage is the loss of water from the concrete member. It refers to the evaporation of the non-hydrated free water from the composite after concrete set. In general, the plastic shrinkage may be described as drying shrinkage of concrete localized near surface at early ages. Drying shrinkage sustain several years with a decreasing rate like other kind of shrinkages. Drying shrinkage is also a partially reversible event.

RESTRAINED SHRINKAGE

Shrinkage causes length reduction in two dimensional freely supported concrete members. Moreover it does not originate stress in concrete unless it is restrained or has a relatively large volume. It is a well known fact that tensile stresses arise in concrete whenever the member is restrained from shrinking freely. Retrains can be either interior or exterior. Reinforcing bars embedded in concrete provide certain amount of restraint by it self. Large amount of volume thought to be an interior restraint situation. Inner portion of massive concrete may restrain the outer portion of it. Aggregates are also considered as interior restraints.

In general, shrinkage cracks are induced by exterior restraints, if the mass concrete is not the topic of the discussion. In case of a staged construction, where the shear walls are cast before the reinforced concrete floor slabs, the shear walls undergo free shrinkage throughout the curing period, while the slabs casted afterwards and attached to these cured members experience restrained shrinkage. In turn, lateral loads causing flexural moments and shears are observed on shear walls, while in plane tensile forces are observed in the slab plates during the curing period of the floor slab itself. It is obvious that the stiffer the shear wall, the higher the in plane tensile stresses in the floor slabs. The forces, either moment, shear or in plane tension, may cause unexpected and immature failures.

Early age shrinkage cracks, mostly caused by plastic shrinkage, can be quite wide on the outer surface of the concrete members reaching widths of 2 to 3 mm. But their width often decreases rapidly below the open surface. Plastic cracks typically do not exceed 10 mm in depth but may pass through the full depth of the member; however the mechanisms leading to the formation of plastic shrinkage cracking does not explain the full depth cracks. It is probable that the subsequent events including drying shrinkage and loading through the formwork replacement can cause the plastic shrinkage cracks to propagate (Transportation Research Board, 2006). Although the cracks placed on the outer surface of the concrete member seem to be harmless and unimportant, they can decrease the capacity of the member. It is reported that the first cracking strength and strains due to flexural loading are found to be significantly affected by the surface micro cracks which are due to the dry shrinkage (Limm et all 1999).

If flexure and shear forces are not dominant on member, stresses induced from restrained shrinkage control the behavior. In general shrinkage strains are bigger than concrete tensile strain capacity. Direct tension cracks caused by restrained deformations are induced by shrinkage and thermal strains and often penetrate completely through the cross-section of a member. They are one of the most difficult and troublesome peculiarities of structural concrete (Nejadi 2005).

SHRINKAGE PHENOMENON IN DESIGN CODES

Stresses and deformations due to shrinkage action in reinforced concrete structures are calculated similar to the contraction deformations through temperature analysis. Therefore one needs to alter the strains caused by the shrinkage to the temperature differences that will cause the same effects on structural deformations and member forces in the structural analysis. The fallowing expression (Eq.1) is used to change the shrinkage strain to the temperature difference.

$$\varepsilon = \alpha \quad \Delta T \tag{1}$$

where ε is the shrinkage strain, α is coefficient of thermal expansion for concrete and ΔT is temperature difference for the structural analysis model. The temperature load (ΔT) calculated using Eq1 is used in structural analysis being a part of factored load combination as implied by the national design codes. Resulting forces from the structural analysis are used to calculate the required amount of reinforcing bars not only for dead, live or earthquake loads but also for the adverse effect of the shrinkage action.

There are several methods derived so far to evaluate the shrinkage strains. Since there are many parameters affecting the shrinkage, and it is difficult to certainly evaluate the effect of every parameter on shrinkage, many simplifications and assumptions are made in shrinkage strain calculations. It can be claimed that quantifying and analyzing the time dependent volume changes, shrinkage, is more difficult than a strength based analysis.

Most popular and useful shrinkage strain calculation procedures are placed in Eurocode 2 (Eurocode 2 Editorial Group 2004) and ACI 209 (ACI Committee 209 1997). In the following paragraphs, the above mentioned code approaches along with the Turkish design and construction code TS500/2000 (TSE Structures Group 2000) formulas are discussed in brief. Expressions from Euracode 2 and ACI 209 are time sensitive equations. One can calculate shrinkage strain by using these codes for any given maturity day. In contrast, shrinkage strain evaluation method in TS500/2000 gives only long term values saying more than two or three years.

In Eurocode 2, total shrinkage strain is given as a composition of drying and autogenous shrinkage strains (Eq.2).

$$\varepsilon_{cs} = \varepsilon_{cd} \quad + \quad \varepsilon_{ca} \tag{2}$$

where ε_{cs} is the total shrinkage strain, ε_{cd} is the drying shrinkage strain and ε_{ca} is the autogenous shrinkage strain. Drying shrinkage strain ($\varepsilon_{cd,0}(t)$) is calculated with respect to environmental conditions, concrete strength, drying time and the dimensions of the structural member (Eq.3, Eq.4). Autogenous shrinkage strain, on the other hand, is evaluated with time and concrete strength (Eq.5, Eq.6, Eq.7).

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s) \quad k_h \quad \varepsilon_{cd,0} \tag{3}$$

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) \quad + \quad 0{,}04 \quad \sqrt{h_0^3}} \tag{4}$$

where:

$\varepsilon_{cd,0}$ is nominal or basic unrestrained shrinkage strain that can be determined either by using the equation in Annex B or by following Table 3.2 in the code

k_h is a coefficient depending on the size h_0 according to Table 3.3 in the code

t is the age of the concrete at the time considered, in days

t_s is the age of the concrete (days) at the beginning of drying shrinkage (normally this is at the end of curing)

h_0 is the notional size (mm) of the cross-section (=$2A_c/u$, A_c is the cross-sectional area and u is the perimeter of that part of the cross section which is exposed to drying)

$$\varepsilon_{ca}(t) = \beta_{as}(t) \quad \varepsilon_{ca}(\infty) \tag{5}$$

$$\varepsilon_{ca}(\infty) = 2{,}5 \quad (f_{ck} - 10) \quad 10^{-6} \tag{6}$$

$$\beta_{as}(t) = 1 - \exp(-0{,}2t^{0,5}) \tag{7}$$

where t is given in days.

ACI has a more simple method; however it is founded on lots of assumptions. Equation 8 is the basic expression for shrinkage strain ($(\varepsilon_{sh,})_t$) calculation in ACI 209.

$$(\varepsilon_{sh})_t = \frac{t^{\alpha}}{f + t^{\alpha}}(\varepsilon_{sh})_u \tag{8}$$

where t is the time from the end of the initial curing(in days), $(\varepsilon_{sh,})_u$ is the ultimate shrinkage strain. f (in days) and α are considered constants for a given member shape and size which define the time-ratio part. α coefficient is considered 1 for shrinkage. Normal ranges of $(\varepsilon_{sh,})_u$ and f constant have been founded to be 415 x10^{-6} to 1070x10^{-6} and 20 to 130 days respectively.

In TS500/2000, it is told that if one can not reach accurate and meaningful data, he can take the approximate values as shrinkage strain from the table in code namely Table 3.4. This method gives just the long term shrinkage strain values. It is based on equivalent member thickness, curing condition and ambient humidity data. Equivalent member thickness, l_e is determined same as h_0 (=$2A_c/u$) in Eurocode 2, as described above.

INVESTIGATIN of RESTRAINED SHRINKAGE EFFECTS IN STRUCTURES

Four cast-in-situ reinforced concrete structures, severely suffering from restrained shrinkage cracks are discussed as case studies in this part of the study. Identification information like cities, designing companies, and owner's names are kept confidential. It should be noted that, no shrinkage analysis was made during the project phase of these structures, resulting wild cracking in structural members. Linear elastic shrinkage analysis is performed for the structures and results are compared with the on-site observations.

SELECTED CASES

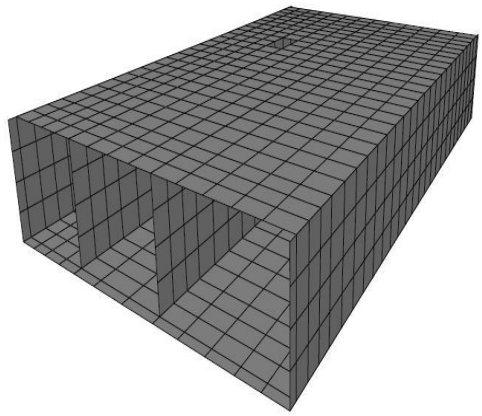
Basic views and properties of the structures are shown in Figure 1.

Case1 is a relatively massive and multi-storey transportation structure which is designed for general public services. It is a part of big structural complex that is separated with construction joints from each other. Design material properties are f_{ck}=30 MPa for concrete and f_{yk}=420 MPa for the reinforcing steel. Structure is composed of reinforced concrete shear walls, slab. There are three bays with lengths of 10.30m, 7.50m and 5.70m. The transverse dimension of the structure is 45.50m in plan. The story height is 7.55m. Other dimensional information is given in Figure 1. Field investigation was made after approximately three months from the first poring of concrete.

Case 2 is an eight storey apartment building. Reinforced concrete tunnel form construction technology was used to build the structure. Stairs were precast concrete. Plan dimensions are 38.60m and 23.70m with 2.80m storey height. Design material

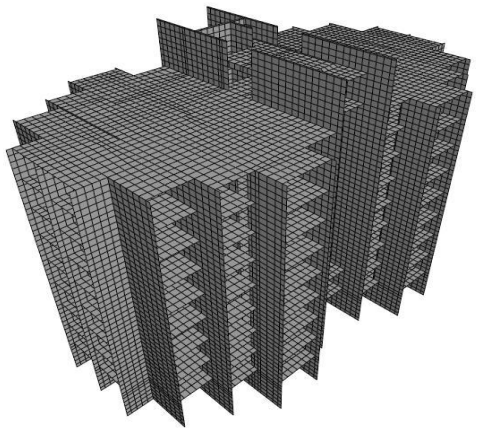
properties are f_{ck}=20 MPa for concrete and f_{yk}=420 MPa for reinforcing steel. Site investigation was made about one and a half year after the overall construction is completed. The very long, single piece reinforced concrete shear walls placed in the East-West direction may be the main peculiarity of the structure under investigation

Case1



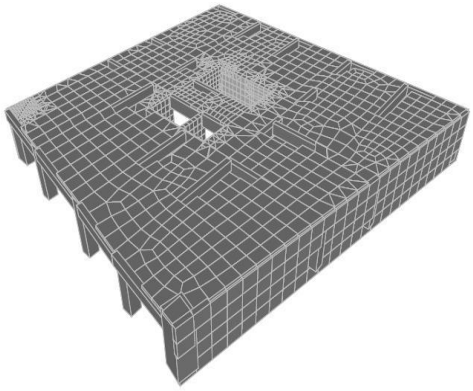
$t_{outer\ walls}$ = 130cm $t_{inner\ walls}$ = 150cm
t_{slab} = 50cm $t_{foundation}$ = 200cm

Case2



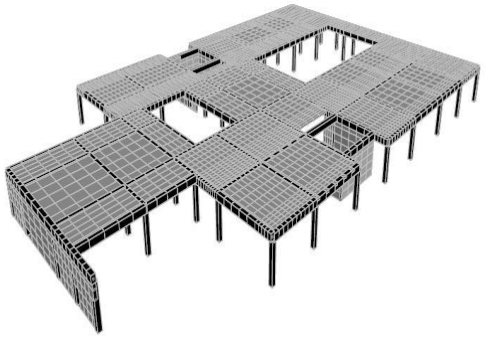
t_{slab} = 14cm $t_{shearwall}$ = 16cm

Case3



t_{slab} = 15cm $t_{shearwall}$ = 25cm
c_{column} = 105cm

Case4



t_{slab} = 15cm $t_{shearwall}$ = 25cm

Fig. 1. Views and properties of structures investigated.

<u>Case 3</u> is the basement of a 10 storey apartment building with elevator and stair well openings in the middle. Perimeter of the openings and two parallel sides of the building had been surrounded with shear walls. Plan dimensions are approximately 22.00m and 23.00m with 3.25m storey height. There are 1.05m x 1.05m columns placed at each corners of the building. The seismic loads are jointly carried by frames and shear walls. Design material properties are f_{ck}=25 MPa for concrete and f_{yk}=420 MPa for reinforcing steel. Investigation was done after couple of month of construction.

<u>Case 4</u> is the first floor of a public service structure that seismic loads are jointly resisted by frames and shear walls together. Plan dimensions are 48.00m and 30.00m, with a 3.00m storey height. There are two stair wells placed facing one another in the middle of the long direction. Stair wells had been formed with reinforced concrete shear walls. Design material properties are f_{ck}=30 MPa for concrete and f_{yk}=420 MPa for reinforcing steel. Structure was examined 6 months after the first poring of concrete.

SHRINKAGE ANALYSIS

Shrinkage strain data of the structures in all cases were derived according to Turkish reinforced concrete design and construction code, TS500/2000. The shrinkage strains

belonging to the structure in case 4 also calculated by using other codes that is Eurocode 2 and ACI 209 to observe the differences among the code expressions.

In the equivalent length calculations, interpolation was made if necessary and required as told in TS500/2000. Relative humidity was taken as %65 in case1, %50 in case2 and %80 in both case3 and case4. The equivalent length values are given in Table 1 accordingly appropriate curing conditions. ε_{cs} and ΔT data are also shown in Table 1.

Table 1. Equivalent length, shrinkage strains and equivalent temperature differences.

	Case1	Case2	Case3	Case4
l_e (mm)	500	150	150	200
ε_{cs} (x10^{-3})	-0,374	-0,400	-0,400	-0,400
ΔT (°C)	-38	-40	-40	-40

Thermal expansion coefficient is taken as 1x10^{-5}/°C in all structural models. Stress strain relationship under tension is thought to be linear for long term shrinkage. Simply fallowing the Hooke Law, whether cracking of concrete in the structures takes place or not can be judged. Table 2 shows the results. Concrete tensile strength (f_{ctk}) and elastic modulus (E) for each structure is determined by the formula in TS500/2000. It should be kept in mind that these shrinkage strains are occurred after at least two years according to TS500/2000.

Table 2. Stress evaluation.

	Case1	Case2	Case3	Case4
f_{ctk} (MPa)	1,917	1,565	1,750	1,917
E (Mpa)	31800	28535	30250	31800
$\sigma_{cs,\ Elastic}$ (MPa)	11,893	11,414	12,100	12,720
Cracking	Yes	Yes	Yes	Yes

Although creep in tension is also occurred in concrete and decrease these values by a certain amount, stresses due to shrinkage never take place below tensile strengths of concrete in all cases.

Moreover reinforcing bars have faced with extreme tension under no considerable loading like earthquake. It is obvious that capacities designed before the construction could never be reached during any important loading. In addition, it should be noted that the reinforcing steel has been fairly stressed even under service loads. In certain situations progressed shrinkage cracks in slabs may lead to noticeable vibrations, due to loss of flexural stiffness values.

Table 3. Shrinkage strains for case4 according to different codes.

Case4	Eurocode 2	ACI 209	TS500/2000
$\varepsilon_{cs,28}$ (x10^{-3})	-0,069	-0,073	---
$\varepsilon_{cs,730}$ (x10^{-3})	-0,248	-0,352	-0,400

Table 3 shows that shrinkage strains evaluated according to different codes and variant time periods for the structure in Case 4. TS500/2000 formula as mentioned before is not time sensitive. Hence shrinkage strain at early age could not be calculated using the TS500/2000 code. ACI and Eurocode expressions yield similar values. In long term evaluation of shrinkage strain in concrete, different results are obtained. Eurocode2 formula has given the least strain value still leads to stress above the limit of concrete tensile strength.

COMPARISON OF RESULTS WITH THE ON-SITE OBSERVATIONS

Computer models with the commercially available FEM program SAP2000 were made for the structures of 4 cases. Comparison of analysis results and in-situ observations is

made case by case. Informations provided on site investigations are varied for each study case.

In Case 1, excessive amount of cracks concentrated on the floor slabs in the middle of each bay through the long direction of slabs were observed. Moreover diagonal cracks located generally at the free edge to shear wall connection regions of slabs were observed. In certain locations of slabs, transverse cracks (in the short direction) were also formed through the bays. A detailed on-site investigation on crack propagation and crack width could be done. Carbonation shrinking products were observed in some cracks. It was measured that maximum crack width was 0,45mm. Analysis has shown that the stresses induced by shrinkage in the slabs are beyond the concrete tensile strength limit. Stresses in slab reinforcing bars are calculated as 260MPa at certain locations. This is rather a high value for the service loads and at a relatively early age of concrete. Furthermore shrinkage stresses in slabs were lead extra moments on outer shear walls in their weak axis. These shear walls were forced about %26-%39 of their moment capacities in their weak axis. In the point of an earthquake loading, the combined moments and forces may go beyond the section capacities resulting plastic hinges, where deformations are not easily predictable

In Case 2, shrinkage cracks concentrated on the two inline shear walls spanning in the East-West direction and on the slabs joining these shear walls. These shear walls are symmetric according to stair well axis. The cracks on these shear walls are located on the outer most sides at the foundation level and incline towards to the stair wells at the consecutive upper storeys. It is observed that these cracks also tend to develop properly through slabs at floor levels. Cracks were also observed in the beams and slabs parallel to the shear walls, located in the middle of the building joining the east and west building wings. These cracks on the intermediate slabs were mainly perpendicular to the axis of the shear walls and were followed through the slab cross-sectional height with same width. Resultant stresses of shrinkage analysis are in a good agreement with the observed situation. It can easily be noted that the locations where concrete tensile strength is exceeded are the same in the structural analysis and in the site investigation. In plane tensile strength of the shear wall was calculated as 1.02MPa in accordance with the supplied reinforcement. It was seen that this value exceeded in certain regions wherever cracking occurred in the shear wall. It is thought that reinforcement was reached to its yield limit at particular locations Tensile stress values on slab shell elements from the FEM analysis are 0,54MPa and 2,29MPa respectively for directions parallel and perpendicular to the shear wall axis. It explains that the cracks tend towards the direction perpendicular to the shear walls.

In Case 3, it is observed that the cracks are in diagonal directions near the corners of the building and they turn parallel to the shear walls placed on two opposite faces on the outer perimeter afterwards. Cracks were with the same width on both upper and lower faces of the slabs and beams. It is observed that the slab and the beams are designed with the minimum reinforcement ratio and bent-up bars are used both in beams and slabs. In some beams additional support reinforcement were used, the cut-off plane coinciding with the bending-up locations of the bet-up bars. The shrinkage cracks were mainly concentrated on these locations. The overall crack distribution is in good agreement with the locations of sudden change in reinforcement ratios. The tensile strength of the slab considering both the steel and concrete contributions is calculated as 2,87MPa in both directions. It is understand that several regions this limit was exceeded. Floor vibration was also reached to considerable amount.

In Case 4, there were crack on beams and slabs especially between shear walls placed at perimeter of stair wells. Cracks on beam were not induced by shear or flexure forces. They were at same line around the beam and has the same width throughout the cross section. They are concentrated at the section that additional support reinforcement is

cut off. The tensile strength of cracked beam section was 2,67MPa. It was seen that there was stresses beyond this limit even reached to yield in the analysis.

CONCLUSION

• Although shrinkage is mostly considered as a simple and unimportant issue, it can be fairly harmful for the serviceability of the structure either under service loads or under overloads such as earthquakes.

• Stresses in reinforced concrete members should be fairly low under service loads, however it is observed that they may reach to yield stresses due to shrinkage.

• The analysis results show that maximum stress locations induced by shrinkage and the crack locations and orientations observed in the field have close agreement.

• The Eurocode and ACI equations can be used for short term shrinkage analysis. It is observed that these two codes yield similar results for the Case Study presented herein. The TS500/2000 yield results are only for long term shrinkage. Moreover, TS500/2000 yields the highest strains among the codes considered in this study.

• Early and late age creep strains for concrete under tension must be considered in the shrinkage analysis, otherwise resultant stresses yields unrealistically too high values.

• The restrained shrinkage cracks cause unexpected reinforcement stresses before the structure meets with design load like earthquakes. Thus, design expected overall structural behavior and lateral load level may never be achieved.

• The large and rigid members in reinforced concrete structures must be considered as rigid supports for the slabs, and slab in-plane stresses must be calculated. The shrinkage induced cracking may be controlled only by appropriate reinforcement configuration and reinforcement ratio.

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STRUCTURAL MODELING AND EARTHQUAKE ANALYSIS OF OTTOMAN MINARETS

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Key Words: Historical Structures, Ottoman Minaret, Construction Techniques, Dynamic Analysis

ABSTRACT The conservation and restoration of historical structures are great challenges for the engineering community. Modeling and analyzing of historical structures require adequate knowledge of geometrical and material properties of the structure. A large number of historical minarets, those built during "Selçuk" and "Ottoman" periods and exist all over Turkey, were constructed as an integrated part with the mosque structural system or adjacent to the mosque main structure. Different types of construction materials were used for outer shells and inner stairs. In this study, the construction technologies and materials used for building minarets during Ottoman period are investigated. Different aspects in modeling and analysis of historical construction are reviewed. The finite element techniques are utilized to model the minaret's structural system using different elements for outer shells, inner stairs and different support conditions. The dynamic properties of different minaret models and the time history analyses results considering nodal displacements and elemental stresses are studied and compared.

INTRODUCTION

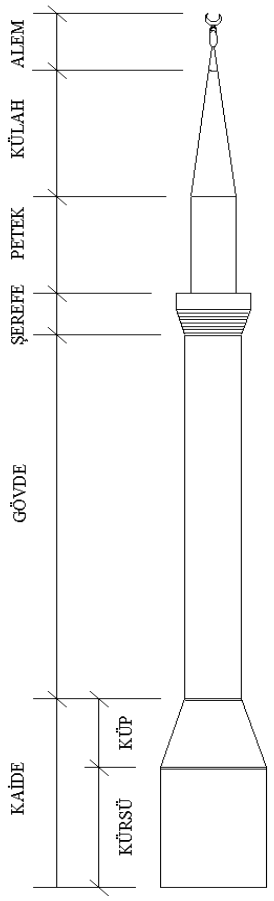
Minarets are used to read "EZAN" the human voices which reports pray time. Minarets in Tunisia, Persian, Algeria, Egypt and Syria are square or multi-angular and generally compose housing in addition to reading "Ezan". Because of housing function such minarets are heavy, low-storey tower structures and dissimilar appearance. This type of minarets is called the western minarets. Minarets in Iran, Iraq and eastern Anatolia are considered to form the eastern minarets group. Eastern minarets used only for reading "Ezan".

Local construction's materials were utilized for building minarets, for example, in Spain stone, in Northern Africa brick, in Cairo stone, in Arabia, Syria and Anatolia stone and brick, in Iraq, Iran and Afghanistan brick, in India stone and brick materials.

A large number of historical minarets exist all over Turkey built during "Selçuk" and "Ottoman" periods and the heights of the minarets are between 10-40 meters. The main materials used in these minarets are stone, brick and a kind of khorassan mortar. Khorassan mortar were prepared by mixing crushed brick, lime, sand or sand-sized fragments of bricks with water. During Ottoman period crushed brick and tile fragments in the size of chickpeas were added to the khorassan mortar.

Ottoman minarets have a special place in terms of format and generality among the other historical structures. In Great architect Sinan's works, Ottoman minaret had its classical form by using successive ladder assembly techniques in the outer walls and inner stairs. Survival of these minarets, that were constructed many centuries ago and opposed to many natural disasters such as earthquakes and storms, is the best indicator of the quality and stability of structural system of these minarets.

CLASSICAL OTTOMAN MINARET SECTIONS



Alem : Symbolic part with crescent-shaped element.

Külah : Cone-shaped part, generally covered with lead.

Petek : Includes the gate to the "Şerefe". In this section inner stairway continues 4-5 steps.

Şerefe : Place where muezzin read "EZAN" the call to prayer.

Gövde : Body part composed of exterior wall and interior spiral stairway.

Küp : The part that provides transition between the upper round part "Gövde" and the lower quadratic part "Kürsü".

Kürsü : Usually a square-based part that transfers the upper loads to the base. Entrance door to the minaret exist in this section.

Fig. 1. Ottoman minaret sections.

CONSTRUCTION TECHNIQUES OF OTTOMAN STONE MINARET

A systematic building technique with specified standards was adopted in constructing the minarets in the Ottoman Empire period. Many artisans were working in different stages of the construction. A description of these stages are given in Lomlu (1996) and summarized as below.

Stones to be used in the construction of minarets were prepared on the site. After completing the base and foundation of the minaret, the "Kürsü" section was built where entrance door was exist. A scaffold for the construction of minarets was created and superstructure construction was started. Each stone forming the outer shell and inner spiral stairs was a load carrying structural element and as a rule, the inner spiral stairs rise from the right side.

Each stone was connected to the upper and lower stones by iron tenons; and stones at the same elevation by iron anchors. Each step of stairway was composed of two stones and khorassan mortar at the middle (Figure 2).

One stone for each three steps in the stairway did not fully integrated with the wall (only 10-15 cm penetrated into the wall), and such stone played important role in balancing the stairway.

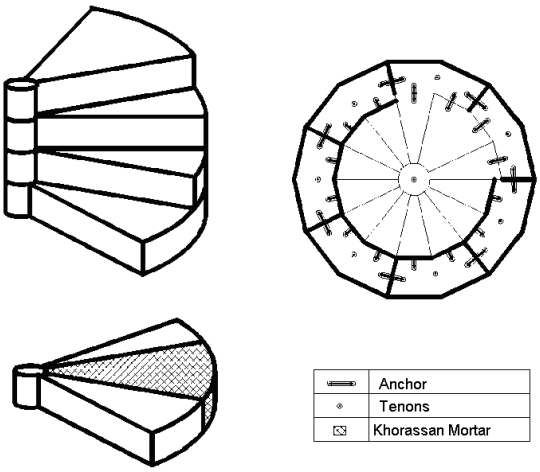
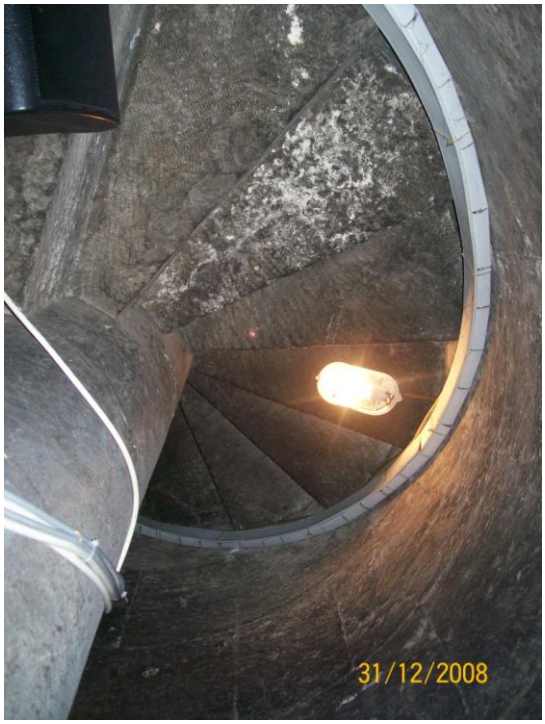


Fig. 2. Inner spiral stairs and connections

Picture 1. Anchor used to connect stones.



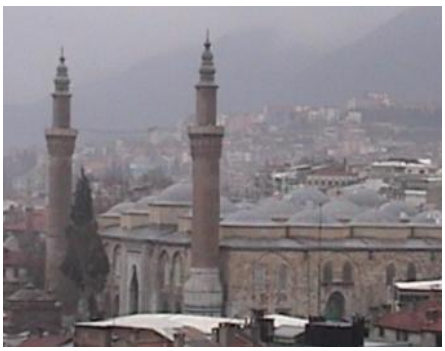
Picture 2. Inner spiral stairs and inner walls.

CLASSIFICATION OF THE OTTOMAN MINARETS

Ottoman minarets can be classified into three groups; before Great architect Sinan, from Architect Sinan to the tulip era and from the tulip era to the present (Eyice, 1963).

OTTOMAN MINARETS BEFORE ARCHITECT SINAN:

During this period the minaret had a thick cylindrical body. Bayezit, Murat Pasa (Picture 4), Koca Mustafa Pasha, Firuzağa (Picture 3), Atik Ali Pasa, Bursa Ulu (Picture 5) mosques' minarets are examples of this period.



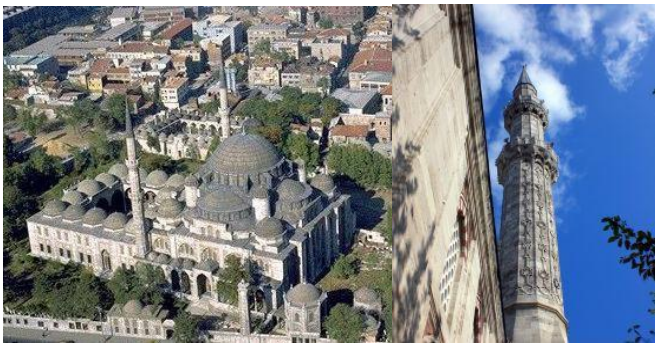
Picture 3. Firuzağa Mosque.

Picture 4. Murat Paşa Mosque.

Picture 5. Bursa Ulu Mosque.

OTTOMAN MINARETS FROM ARCHITECT SINAN TO TULIP ERA:

Starting with Architect Sinan, Ottoman minaret had reached its classical shape and the height of minarets had been considerably increased. In this period, the minaret and the mosque formed an integrated architectural composition. Şehzade (Picture 6), Süleymaniye (Picture 7), Selimiye, Sultanahmet, Yeni-Cami' minarets are examples of this period.



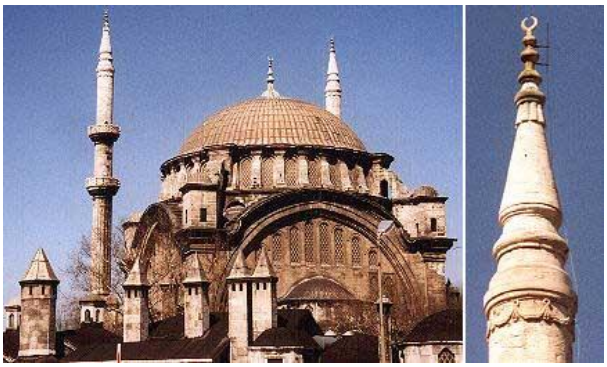
Picture 6. Şehzade Mosque and minaret.

Picture 7. Süleymaniye Mosque and minaret.

OTTOMAN MINARETS FROM TULIP ERA TO PRESENT:

This period minarets were quite decorative and thinner than those in the previous period. Yıldız (Picture 8), Nuru Osmaniye (Picture 9), Ortaköy, Beylerbeyi, Dolmabahçe Valide Sultan mosques' minarets are examples of this period.

Picture 8. Yıldız Mosque Minaret



Picture 9. Nuru Osmaniye Mosque and its minaret

MODELING AND ANALYSIS OF A MINARET

In this study, a simple geometry of a minaret modeled by different elements (solid, shell) is analyzed by finite element method. Geometric view and the size of the minarets are given in Figure 3.

Using different elements (frame, shell and solid), two models are prepared. Each model is analyzed with and without inner spiral stairways to examine the rule of the spiral stairway on the behavior of minarets structural system. Material properties of the models are taken to be isotropic with density of 2.2 ton/m3, elasticity modulus of 3000 MPa and poisson's ratio of 0.2. Finite element models of the minaret are shown in Figure 4.

Modal analyses for different minaret models are carried out to compute the dynamic modal properties and the linear time analyses are performed to compute the nodal displacements and elemental principal stresses.

Model 1: Outer walls, spiral stairway and the core are modeled with solid elements.

Model 2: Outer walls and spiral stairway are modeled with shell elements; core is modeled with frame elements.

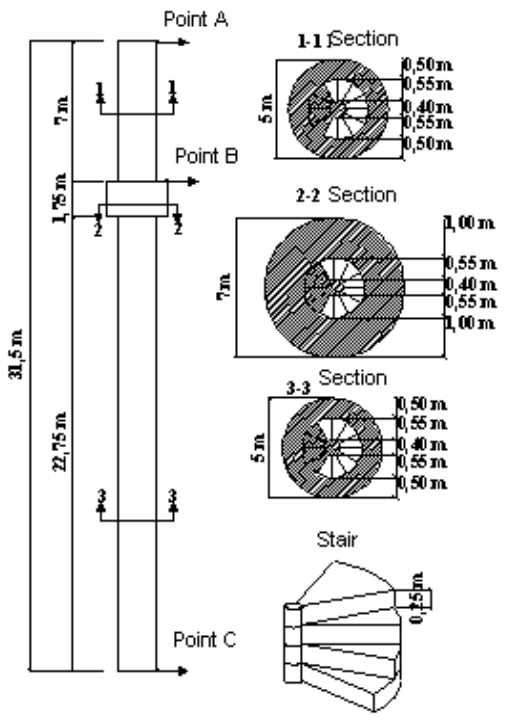


Fig. 3. Geometric view and dimensions of the system used in the analysis of the minaret.

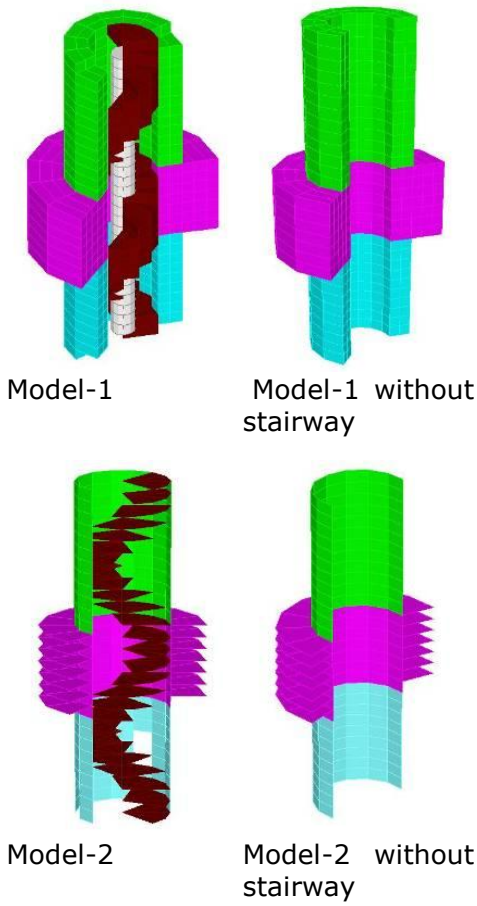


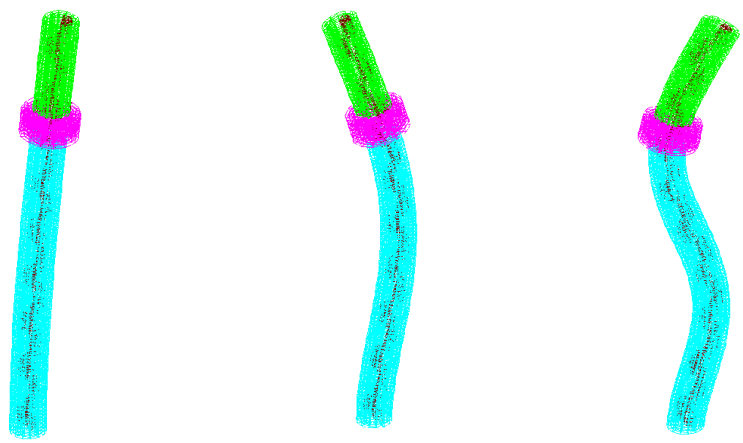
Fig. 4. Minaret models.

Eigenvalue analyses are performed to identify the dynamic properties of the minaret. The mode shapes of Model-1 are shown in Figure 5. Modal periods of the four models are given in Table 1. Modeling inner stairs increases the natural period of the system about %7 in Model-1, and %13 in Model-2. The main reasons of such difference are that modeling spiral stairway lead to increase of the model masses in Model-2 (shell elements), consequently system fundamental natural periods increase while in Model-1

(solid elements) the spiral stairway make increase in both the mass and the rigidity of the system.

Table 1. Modal periods of minaret models.

Mod	Period (Sec)			
	Model-1	Model-1 without stairway	Model-2	Model-2 without stairway
1	2.523	2.365	2.686	2.376
2	2.521	2.365	2.686	2.376
3	0.378	0.351	0.415	0.361
4	0.378	0.351	0.415	0.361
5	0.218	0.218	0.194	0.187
6	0.145	0.137	0.161	0.142
7	0.145	0.137	0.161	0.142
8	0.119	0.120	0.129	0.118
9	0.078	0.074	0.088	0.077
10	0.078	0.074	0.088	0.077
11	0.059	0.058	0.057	0.054
12	0.050	0.048	0.055	0.048



Mode 1 Mode 3 Mode 6

Fig. 5. Modal shapes of model-1.

Linear time history analyses are carried out for all the models. The selected input earthquake motion data is the acceleration component 090 recorded at Izmit station during Kocaeli Earthquake 1999. The time history analysis results are given in Table-2. For Model 2, where the minaret is modeled with shell and frame, existing of inner stairway leads to significant increase of both displacements at points A and B and stress at point C. The main reason of such increase is the major increase in system mass with negligible increase in system rigidity. For Model 2, where solid elements is used, including the stairway in modeling alter the analyses results either increasing or

decreasing. The main reason of such outcome is the major increase in both system mass and rigidity.

Table 2. Time History Analysis Results at Points A, B and C.

	Displacement at Point A		Displacement at Point B		Stress at Point C	
	Max.(m)	Min. (m)	Max.(m)	Min. (m)	Max.(KN/m2)	Min. (KN/m2)
Model-1 with stairs	0,1332	-0,1654	0,0919	-0,1129	1,418E+03	-1,654E+03
Model-1 without stairs	0,1561	-0,1533	0,1098	-0,1065	1,440E+03	-1,322E+03
Model-2 with stairs	0,1755	-0,2208	0,1239	-0,1524	1,953E+03	-2,144E+03
Model-2 without stairs	0,1540	-0,1520	0,1081	-0,1055	1,520E+03	-1,342E+03

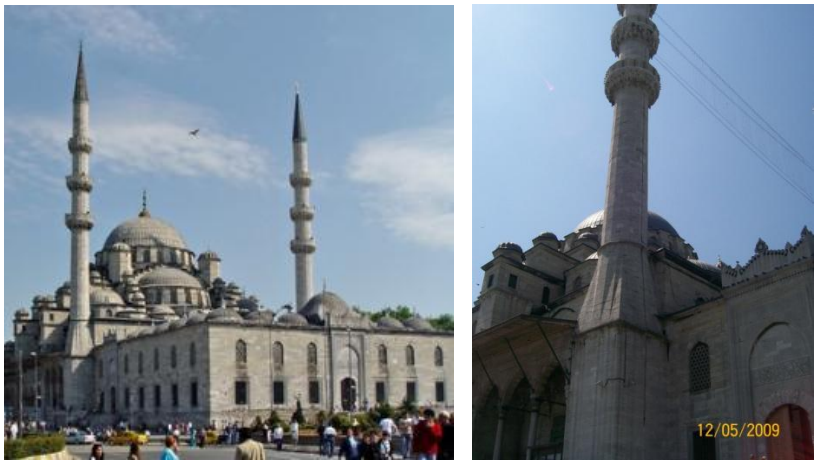
MODELING AND ANALYSIS OF A MINARET MODEL BY DIFFERENT SUPPORTING CONDITIONS:

In this study, minarets are categorized according to their boundary conditions into two groups: 1) Minarets built as stand alone separated part from the mosque structural system; for example Hagia Sophia (Picture 10); 2) Minarets built as integrated structure adjacent to the mosque main body (Yeni Mosque, see. Picture 11).

The effect of such boundary conditions on the structural behavior of the minaret is investigated. Details and finite element discretization of stand alone minaret model are prepared and shown in Figures 6 and 7, respectively. The second model considering the adjacency of the minaret to the mosque structure is prepared such that compression only spring elements with proper rigidity are applied to the lower part of the minarets to represent the existence of the mosque walls surrounded the minaret. The details of the model are shown in Figure 8.

The time history analyses are performed for both models under the same input motion and the maximum and minimum principal stresses distributions are shown in Figures 9 and 10. Comparisons between the two models results show that maximum stress values are increased about %20-30 when minaret is adjacent to the mosque.

Picture 10. Hagia Sophia.



Picture 11. Yeni Mosque.

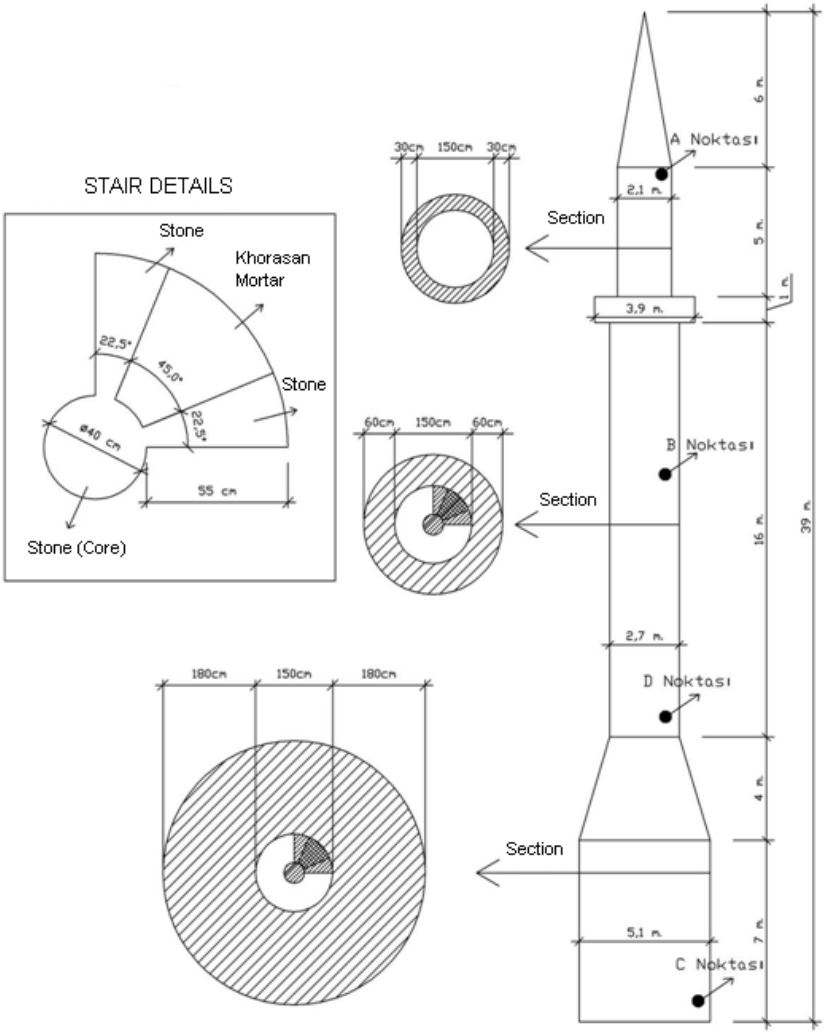


Fig. 6. Details of stand alone Minaret separated from the mosque main structure.

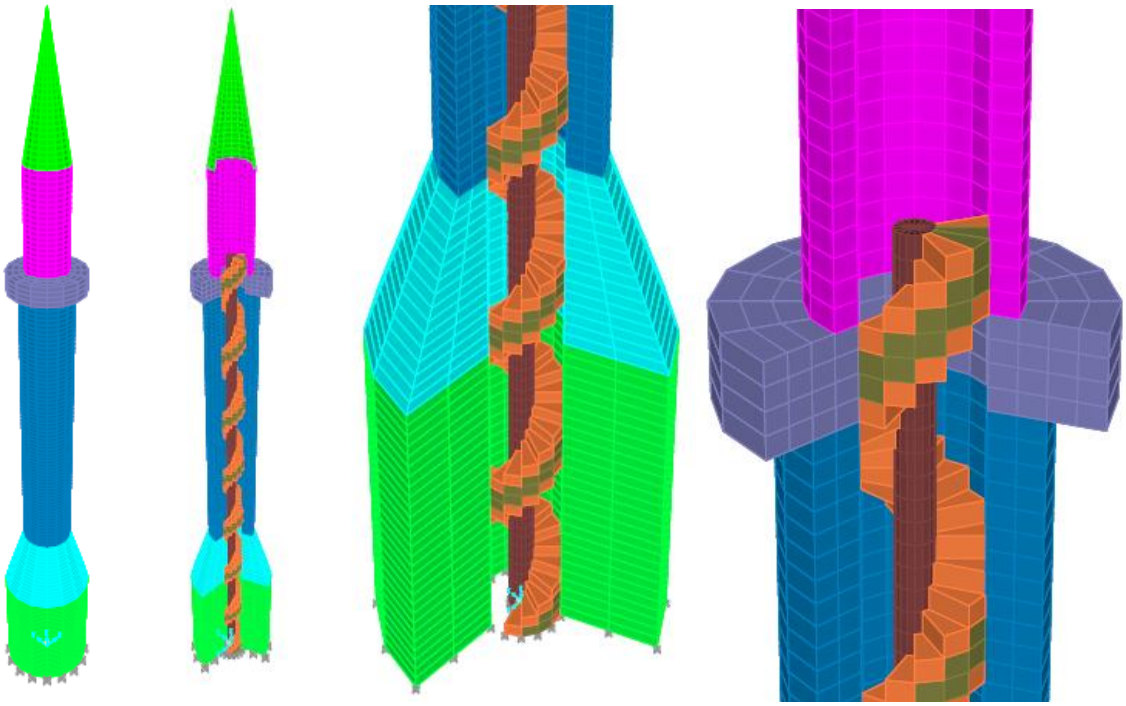


Figure 7. Finite element model of the stand alone Minaret.

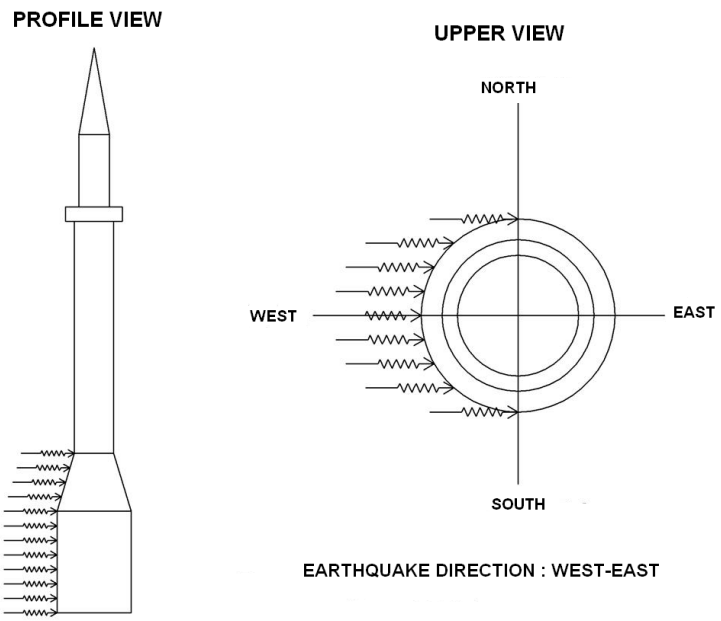


Figure 8. Placement of springs to account for the mosque adjacency boundary condition.

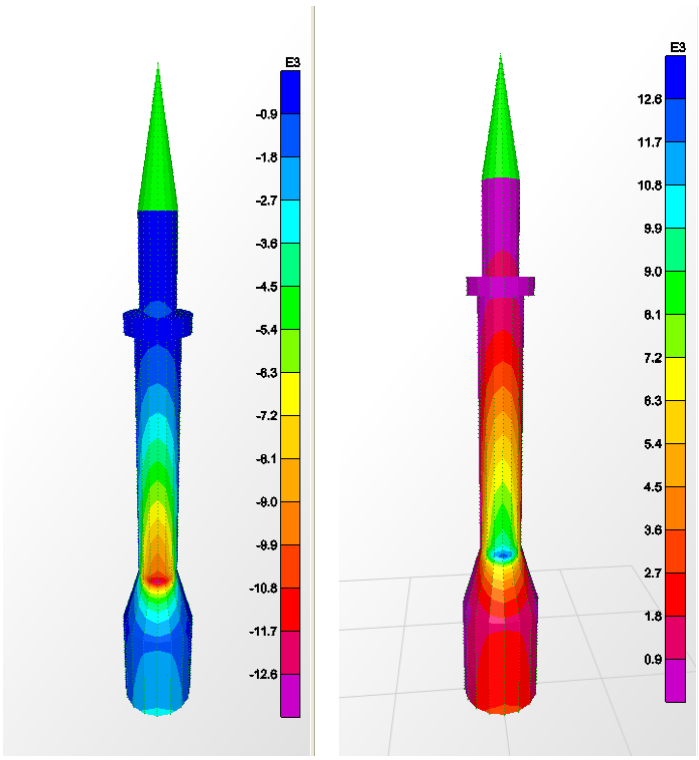


Figure 9. Maximum and minimum stress distribution from south view for the minaret separated from the mosque.

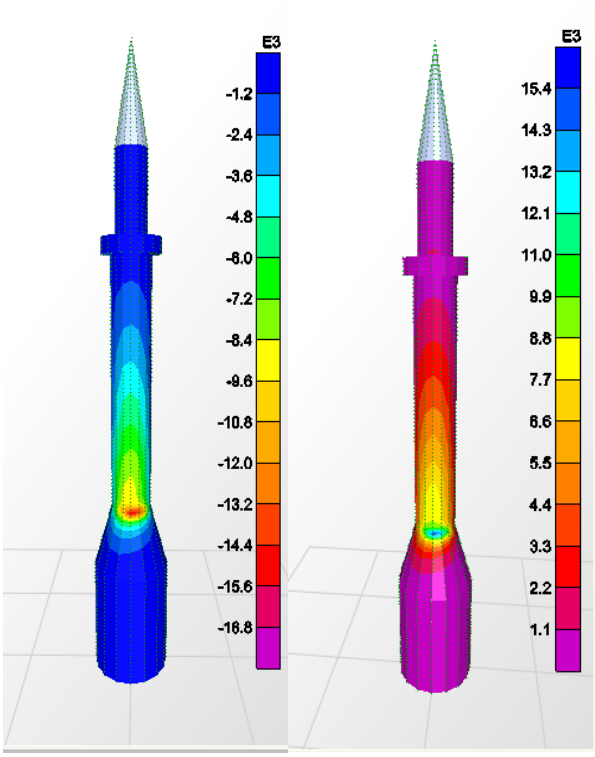


Figure 10. Maximum and minimum stress distribution from south view for the minaret adjacent to mosque.

CONCLUSIONS

Ottoman minarets have a special place in terms of format and generality among the other historical structures. Survival of these minarets that were constructed many centuries ago after many natural disasters such as earthquakes and storms is the best indicator of the good structural system of these minarets.

Modeling techniques and element type selection have important effects on the analyses results in the case of Minaret body integrated with the inner spiral stairway. Solid elements provide a proper connectivity between the upper and lower spiral layers of the stairway together with the minaret outer shell. Modeling with shell elements creates lack of connectivity among the different spiral stairway layers and with the outer shell.

According to the time history results, the maximum tension and compression stresses occur in the lower part of the minaret body at the connection of the "Küp" section.

Minarets constructed adjacent to the main structure have %20-30 more tension and compression stresses than stand alone minarets when subject to the same quake record.

ACKNOWLEDGEMENTS

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THE EFFECT OF THE POTENTIAL OF LIQUEFACTION SENSITIVITY ON DAMAGE: ISTANBUL (HISTORICAL PENINSULA)

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Key Words: Historical Penninsula (Istanbul), Liquafaction, microzonation, structure damage

ABSTRACT In this study, damage information on the historical artifacts and structures in the Historical Peninsula during the previous earthquakes and the liquefaction hazard of the region have been evaluated together with the seismic micro-zoning maps, and the effect of the ground behavior on the damages have been researched. Liquefaction potential micro-zoning maps of the study area have been generated using earthquake movements calculated in accordance with the 10% and 40% probability of exceedance in 50 years. In the study, geological and geotechnical features of the region have been examined in detail, and regions in which liquefaction sensitivity may occur have been determined with the help of the obtained geological maps and sections. Liquefaction analysis has been conducted in a total of 108 250m*250m cells which have liquefaction risk in accordance with the Simplified Method developed by Seed, et al., (1984, 1985), and the liquefaction potential of the region has been found as P_L (liquefaction potential index). Geographical information systems have been utilized in joining the damage information on the historical artifacts observed in the region (Erdik, et al., 2003), regional damage information on the earthquakes of 1509 and 1894 (Barka, 1991), damage information on 1999 Kocaeli Earthquake (Disaster Management Center Directorate) and liquefaction hazard seismic micro-zoning maps. It has been concluded that liquefaction may partially affect the observed damage.

INTRODUCTION

For the planning of new residential areas and assessment of the seismic vulnerability of the already built up areas during a probable earthquake, regions which are susceptible to soil liquefaction have to be determined and evaluated in detail. For this purpose, nowadays preparation of liquefaction hazard maps are increasingly being employed in seismic safety plans of authorities in order to mitigate the earthquake induced liquefaction hazards. Since liquefaction phenomenon is closely related with the geologic conditions, the liquefaction hazard maps should be based on geological maps. In this study the geologic and geotechnical conditions of old Istanbul (Fatih and Eminönü Provinces) were investigated in detail and a seismic microzonation study was performed with respect to liquefaction potential for probabilities of exceedance of 10% and 40% in 50 years. The geologic and tectonic features of the study area were determined utilizing the findings of soil investigation borings drilled in the past for various construction activities and the borings drilled within the framework of JICA project.

Seismic microzonation studies generally involve three stages: the assessment of seismicity and regional seismic hazard, identification of geological structure and fault features, and determination of the effects of local geotechnical conditions in the region. The local geotechnical effects are evaluated with respect to soil amplification, slope stability and liquefaction potential. The seismic hazard of the study area was estimated with respect to probabilities of exceedance of 10% and 40% in 50 years with a probabilistic approach at 250m intervals by utilizing the studies performed within the scope of GSHAP(1999) project and by using the computer code SEISRISK III (Bender vd., 1987). For the success of microzonation studies, the geological and fault structure of

the region should be evaluated correctly. Therefore, the geological maps including faulting systems and geological cross sections were prepared in detail utilizing the data from125 borings, surface geology observations and topographical information. The study area was divided into 250m*250m cells and representative soil profiles were created for each cell compatible with the geological maps and cross sections.

The microzonation maps regarding liquefaction potential were based on the assessment of liquefaction potential according to the simplified procedure recommended by Seed vd., (1984, 1985) for the coast lines of Haliç, Yenikapı and Kumkapı and along Vatan street where liquefaction is thought to be likely to occur due to the characteristics of soil layers.

GEOLOGY OF THE HISTORICAL PENINSULA

The geology and soil profile of the historical peninsula have been studied through surface geological investigations, 125 borings drilled for various purposes, and several trial pits opened in and around the area. The studies showed that two distinct formations outcrop on the peninsula. The older formation, constituting the bedrock in the area and known as "Trakya" formation, is Lower Carboniferous and represented by interbedded claystone, siltstone and sandstone (graywacke) (Kaya, 1971, Vardar and Bayraktar, 1993). The other formation, unconformably overlying the former, is a continuous sequence of deposits from Upper Oligocene to Upper Miocene, composed of loosely cemented and interbedded sand and clay with increased carbonate content in the upper levels resulting in intercalations of limestone and marl. This is a depositional sequence uniterruptedly continuous until end of Miocene (Yıldırım, 2002, 2003). The sequence, which has been described as a series of formations called "Gürpınar", "Çukurçeşme", "Güngören" and "Bakırköy" in the literature (Sayar, 1976), starts with widspread sand and gravel layers and continues with overconsolidated green clay layers. The clays, interbedded with sands, continued deposition intensely throughout Sarmatian and, in the upper levels, due to increased carbonate content in lacustrine environment, became interbedded with marl and argillaceous limestone (Fig. 1). The uppermost level of the sequence is represented by carbonaceous clay or sand layers of Upper Miocene (Pontian) (Baykal, 1974; Yıldırım, 2003).

Grayish brown graywackes belonging to Trakya formation outcrop near Aksaray, between Kumkapı and Sarayburnu, and on the steep slopes descending towards the Golden Horn, and are characterized by normal faults striking northwest-southeast and northeast-southwest. This sequence, consisting of sandstone, siltstone and claystone layers with lenses of gray limestone, dips westward, and is unconformably overlaid by Upper Oligocene – Upper Miocene deposits, which increase in thickness as they extent towards the west. Kırklareli formation of Middle Eocene – Lower Oligocene, which does not outcrop in the historical peninsula, has been encountered in some deep water wells in the southwest corner of the peninsula, at the location where the historical city walls reach the Marmara Sea, near Yedikule. The relevant borehole records indicate a 5 – 15 m thick clayey and sandy basal layer followed by marl and limestone belonging to Kırklareli formation. The location is significant in that it is the easternmost point where Kırklareli formation has ever been encountered.

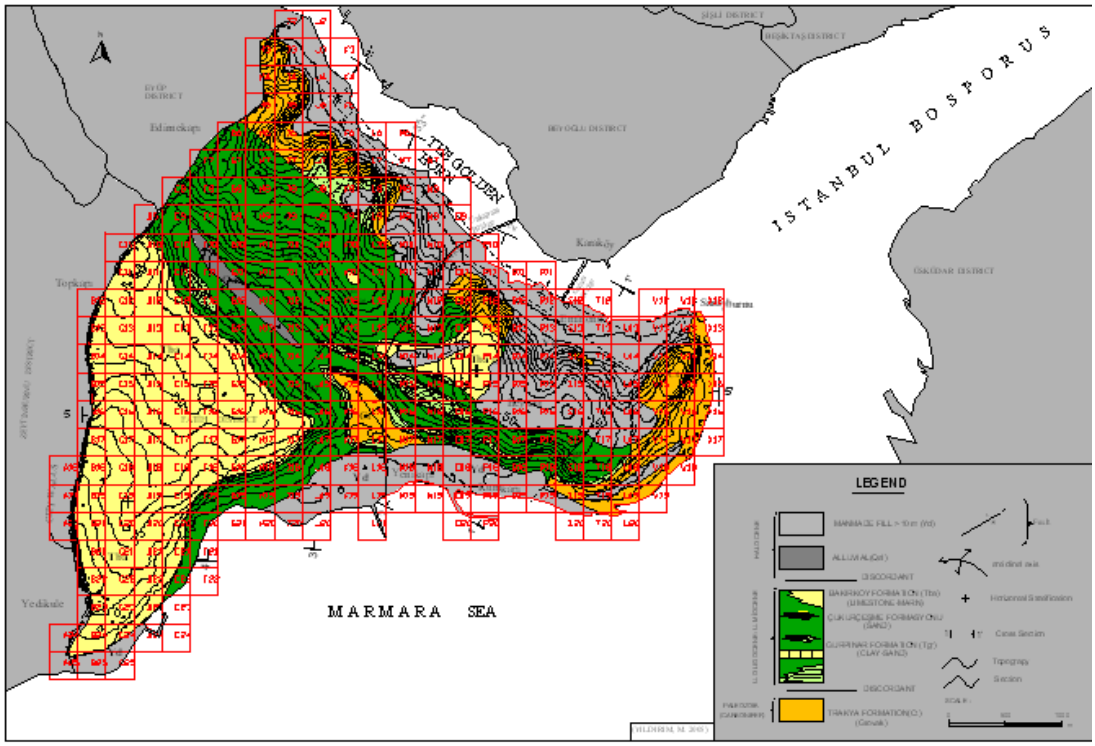


Fig. 1. Engineering Geological Map of the Historical Peninsula (Özaydin, et.al., 2005).

SITE RESPONSE ANALYSIS

In this study, site response analyses are conducted for historical peninsula by using the computer code EERA (Bardet et al., 1998) using the defined soil profiles and the simulated earthquake time histories (Papageorgiou et.al.,2000) obtained from the regional probabilistic earthquake hazard study (Erdik et.al., 2004) for each cell. From the results of one-dimensional (1-D) site response analysis, peak ground acceleration (PGA) values are determined for probabilities of exceedance of 10% and 40% in 50 years. The variation of peak ground surface accelerations in the region for probabilities of 10% and 40% in 50 years are shown in Figure 2 and Figure 3, respectively.

SITE CLASSIFICATION

The site classification of the region with respect to local soil conditions was performed by utilizing the data from 125 soil borings drilled by different agencies and companies. The soil boring logs included the description and thickness of soil layers and SPT results. The results of PS logging tests which were carried out in four deep borings drilled in Fatih and Eminönü provinces and two borings drilled in the vicinity of the study area within the scope of the project "The Study on a Disaster Prevention/Mitigation Basic Plan in Istanbul including Seismic Microzonation in the Republic of Turkey" carried out in 2001-2002 (JICA and IMM) were also utilized in site soil classification.

The site soil classification at each cell is determined according to both Turkish Earthquake Code (TEC,1998) and NEHRP (BSSC, 2001) by taking into consideration the soil profile and soil properties defined for each cell. The site classification maps according to each method are shown in Figure 4 and Figure 5.

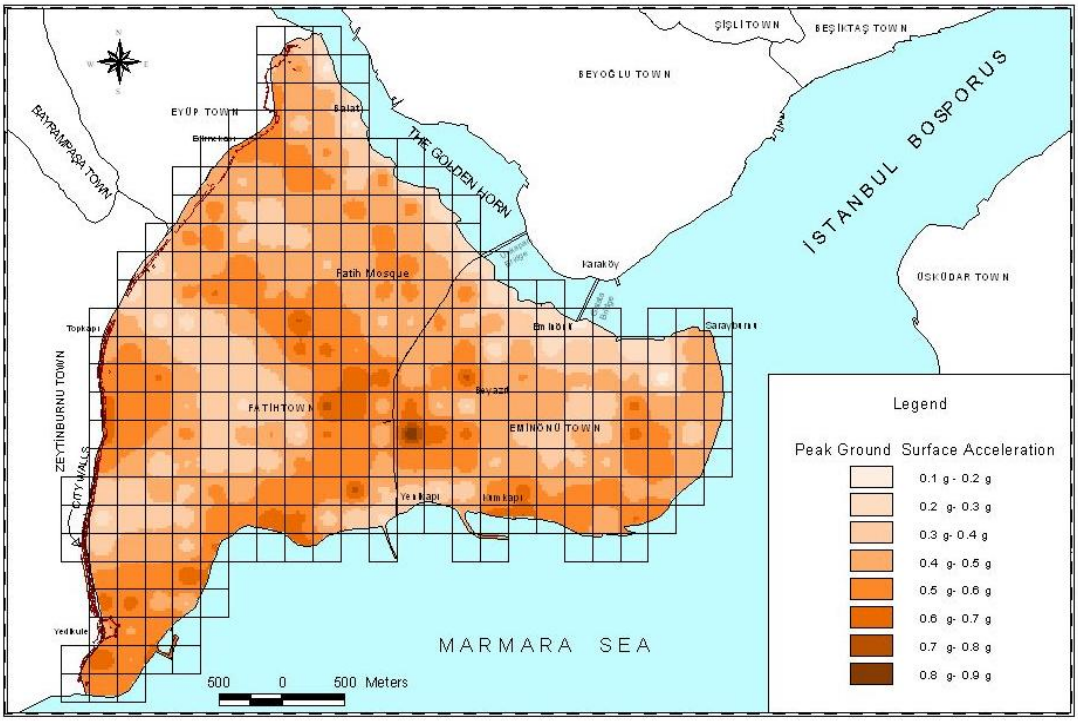


Fig. 2. The variation of peak ground surface accelerations in the region for probability of exceedance of 10% in 50 years (İnce, 2005).

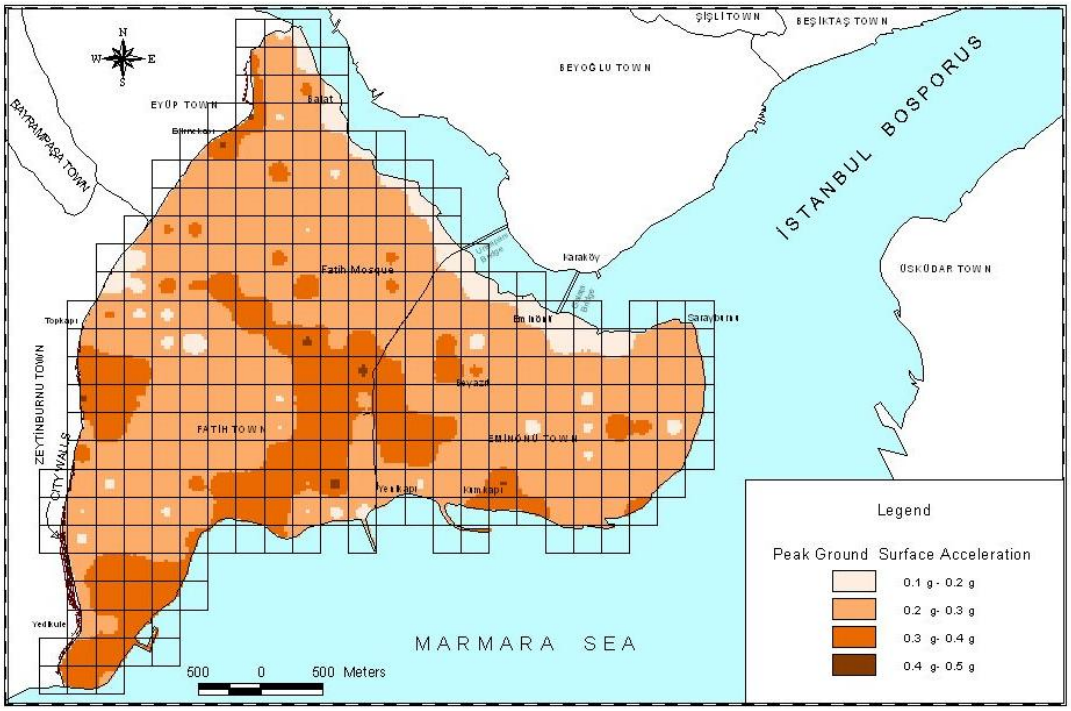


Fig. 3. The variation of peak ground surface accelerations in the region for probability of exceedance of 40% in 50 years (İnce, 2005).

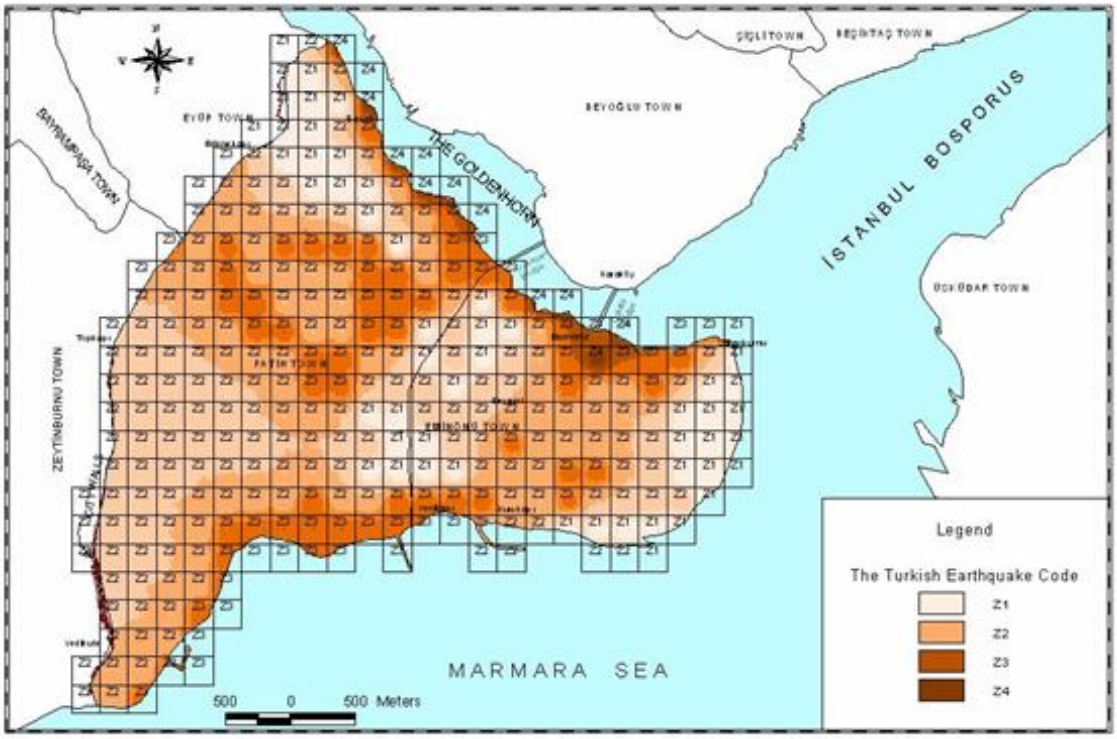


Fig. 4. Soil classification according to Turkish Earthquake Code (TEC,1998) (İnce, 2005).

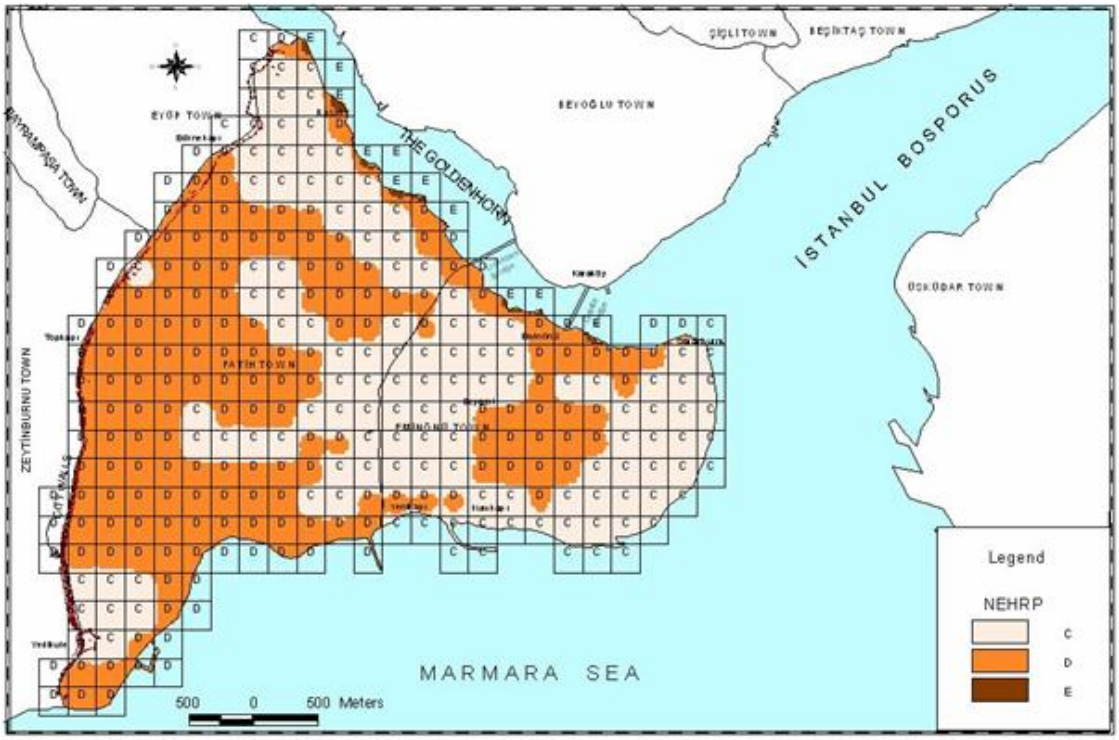


Fig. 5. Soil classification according to NEHRP (BSSC, 2001) (İnce, 2005).

LIQUEFACTION POTENTIAL OF THE REGION

In this study, the microzonation maps in terms of liquefaction susceptibility are developed for the regions covered with artificial fill and alluvial deposits in the peninsula based on the method developed by Youd et al.(2001) and Iwasaki et al. (1982) as recommended in the Microzonation Manual Part 2II (MERM, 2004). This widely accepted procedure is an extended version of SPT-based method proposed by Seed et al. (1984, 1985).

In the assessment of the study area for liquefaction hazard, areas which are covered with artificial fill and alluvium such as Vatan street (Yenibahçe brook) and Haliç coast, as well as Kumkapı and Yenikapı coast line along Marmara Sea are chosen for investigation. Other areas which are covered with Bakırköy formation, Gürpınar formation and Thrace formation comprise soils of low sand and silt content which are free of liquefaction risk. Liquefaction assessment is carried out in 108 cells which are identified as potential risk areas considering the top 20m of the soil profile, and liquefaction potential index is determined for each cell based on the procedure developed by Iwasaki et al. (1982).

The procedure developed by Iwasaki et al. (1982) predicts the liquefaction performance of the soil profile at a specific site to a depth of 20 m by employing a liquefaction potential index, P_L ,which is defined as

$$LPI = \int_0^{20} F(z)w(z)dz$$

where z is the depth below the ground water surface in meters,

$F_L=1-FS_L$ $\quad FS \leq 1$

$F_L=0$ $\quad FS>1$

$w_z=10-0,5z$

Table 1 shows the assessment of liquefaction hazard according to Liquefaction Potential Index. The study area is separated into three hazard level zones A_L, B_L and C_L. A_L represents areas where PL >15, B_L areas where 5>PL>15 and C_L areas where PL<5. In Figure 16, microzonation maps for liquefaction hazard corresponding to probabilities of exceedance of 10% and 40% in 50 years are shown. As can be seen from these maps, the shores of Marmara Sea and Golden Horn and a narrow band along the Vatan street are the most liquefaction susceptible areas in the historical peninsula. The liquefaction potential risk appears to be slightly higher with respect to the probability of exceedance of 10%, compared to that of 40%.

Table 1. Liquefaction potential according to Liquefaction Potential Index (LPI).

Liquefaction Potential	LPI
None	0
Low	0 < LPI <5
Medium	5<LPI <15
High	LPI > 15

1894 EARTHQUAKE REGIONAL STRUCTURE DAMAGE DATA

July 10, 1894 dated earthquake was the last greatest one having affected the region prior to August 17, 1999 Kocaeli earthquake. It was detected that the epicenter of the earthquake was 40.8 N and 29 E, approximately 60 kms away from study area and according to MSK scale, it had an intensity of X throughout the region. However there are some ambiguities concerning the intensity values of iso-intense regions which have been identified with numbers from one to four on iso-intense map of earthquake (Eginitis, 1894). While 1894 earthquake has been reported with M=6.7 magnitude and felt with VIII intensity in and around Istanbul (Barka, 1991), a group of researchers noted that the intensity felt was X (Gündoğdu, 1991). Despite this ambiguity concerning the value of intensity, since values are determined with respect to observed damages, they include the effects of local ground conditions as well.

In study area, damage data relevant to 1894 earthquake (Lav, 1994) have been compiled according to studies of several researchers. In present study, structures that have different levels of damage, have been given in Figure 6.

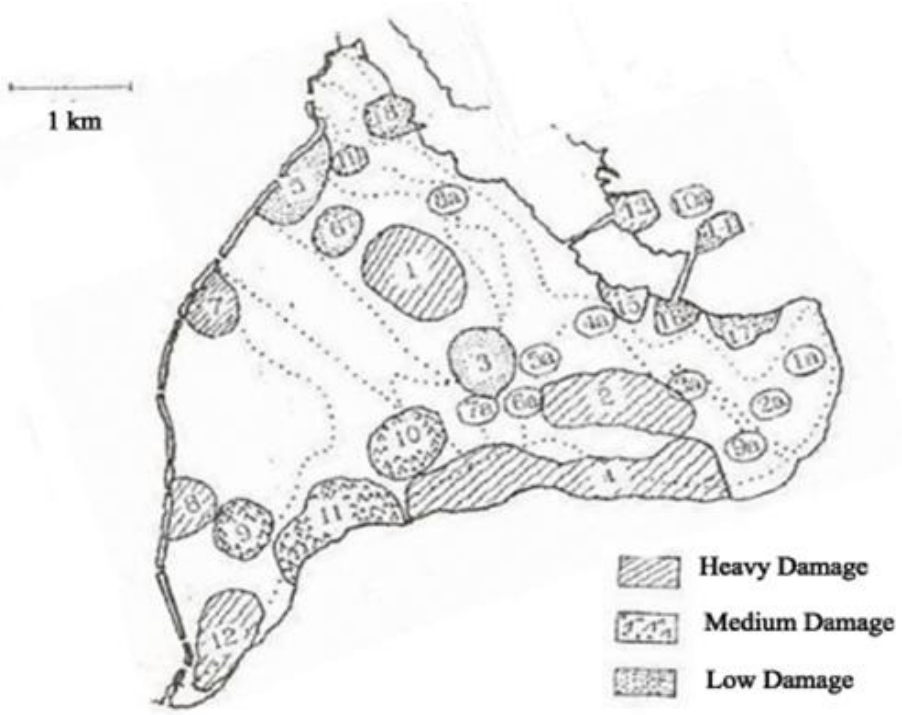


Fig. 6. Regional Damage Map of 1894 Earthquake (Lav, 1994).

Classification of damage levels as severe, mild and weak has been based on damage classification in a region and for each definition the values given between numbers zero and one have been added, divided into damage definition number and classification has been made according to the final values obtained (Lav, 1994).

In Figure 7. a) and b), liquefaction danger maps and damage distribution map have been illustrated together with respect to 10% and 40% chances of exceed for 50 years. Alongside Yenikapı and Kumkapı coastline where danger of liquefaction is high, severe damages are observed. Similarly around Yedikule where danger of liquefaction is high there are severe damages. Severely damaged region in Eminönü District Haliç coast is placed in A_L and B_L regions on liquefaction danger map. Except coastline of Fatih District throughout regions with high risks of liquefaction, generally there are no severe/mild damages. Of the weakly damaged regions, the ones on Haliç coastline are located in A_L

and B_L regions whereas the other weakly damaged regions are located in C_L region which has low risk potential in terms of liquefaction danger. In study area, it has been observed that the liquefaction might have been effective in the emergence of damages during 1894 earthquake throughout Marmara Sea and Haliç coast but liquefaction danger which might be expected around Vatan Street created no damage.

Generally evaluated, it surfaces that regions with severe damages are located in high risk regions on liquefaction maps and as a geological formation they are seen in regions with thick clay layers (Gürpınar formation) and regions around Marmara Sea coastline where thick fillings are present and the regions where Bakırköy formation has surfaced. It is seen that mildly damaged regions are in places where Gürpınar formation and certain regions of Bakırköy formation are. It emerges that along some regions in Haliç coastline where thick fillings are and Edirnekapı where thick clay layers are (Gürpınar formation) weak damages occurred. Places with no damage are located in regions Thracian formation surfaces. Overall, it is understood that damage distribution and liquefaction risk maps and geology maps in coordination except for some regions. Local ground conditions and liquefaction are generally effective in the emergence of damages throughout the region.

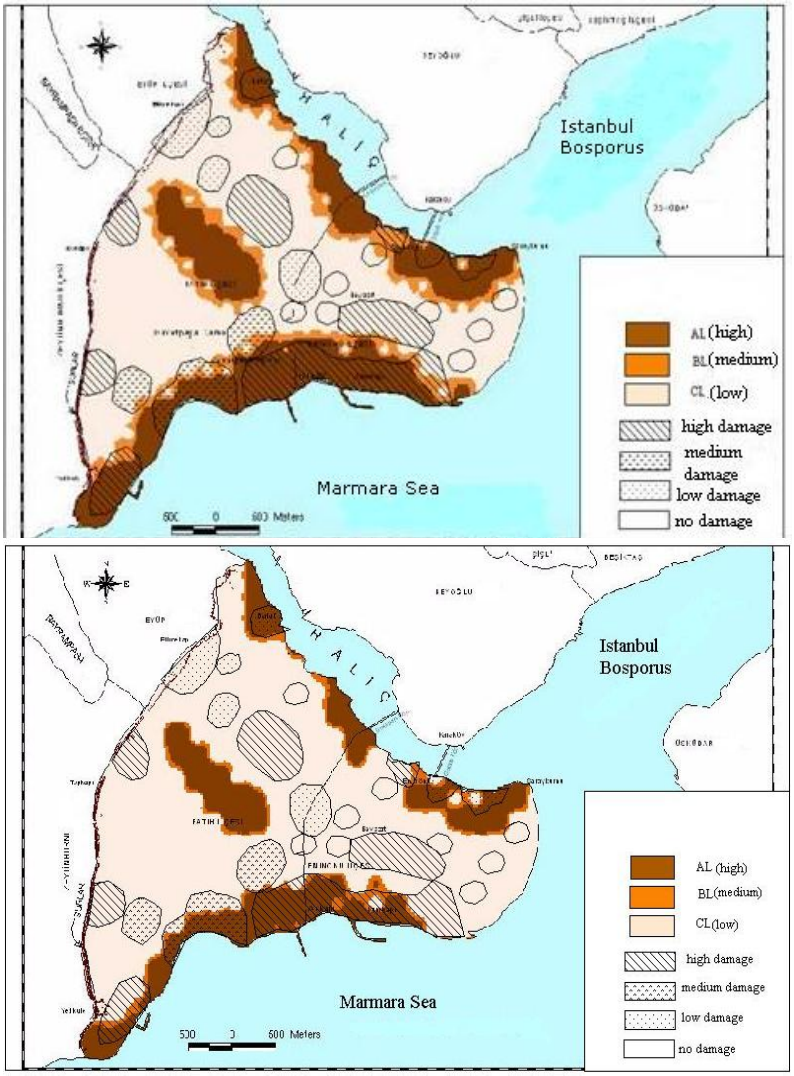


Fig. 7. a) and **b)** Comparison between damage observed in 1894 earthquake and liquefaction danger map prepared with respect to 10% and 40% chances of exceed for 50 years (İnce, 2005).

INFORMATION ON DAMAGE OCCURRED IN SIGNIFICANT HISTORICAL STRUCTURES

Study area Istanbul (Historical peninsula), is quite a rich region in terms of historical structures. Some of the significant historical structures are namely Santa Sophia, Topkapı Palace, Beyazıt Mosque, Fatih Mosque, Nur-u Osmaniye Mosque, Sultan Ahmet Mosque, Süleymaniye Mosque, Mihrimah Sultan Mosque, Şekerciler Caravanserai, Valide Caravanserai, Valide Mosque, Bali Paşa Mosque, Vezir Caravanserai. Information concerning the damage occurred in the historical structures at different earthquakes have been evaluated by(Erdik, et.al., 1999). In this study damages occurred in historical structures during earthquakes have been illustrated via a three dimensional matrix (Figure 8).

D1, D2, D3, D4 and D5 coding that are used to define damage, in line with European macro seismic scale (ESM, 1998) stand for weak, mild, severe, very severe damage and collapse. This study reveals that Fatih and Edirnekapı Mihrimah Sultan Mosques are adversely affected by earthquakes at all times. Information on damage on historical structures has been evaluated on earthquake micro zoning maps by displaying on ArcView 3.1 program(ince, 2005).

Having compared damages on historical structures and liquefaction maps (Figure 9. a) and b)) it emerges that mosques are located in general within regions which have no risk or minimum risk of liquefaction.

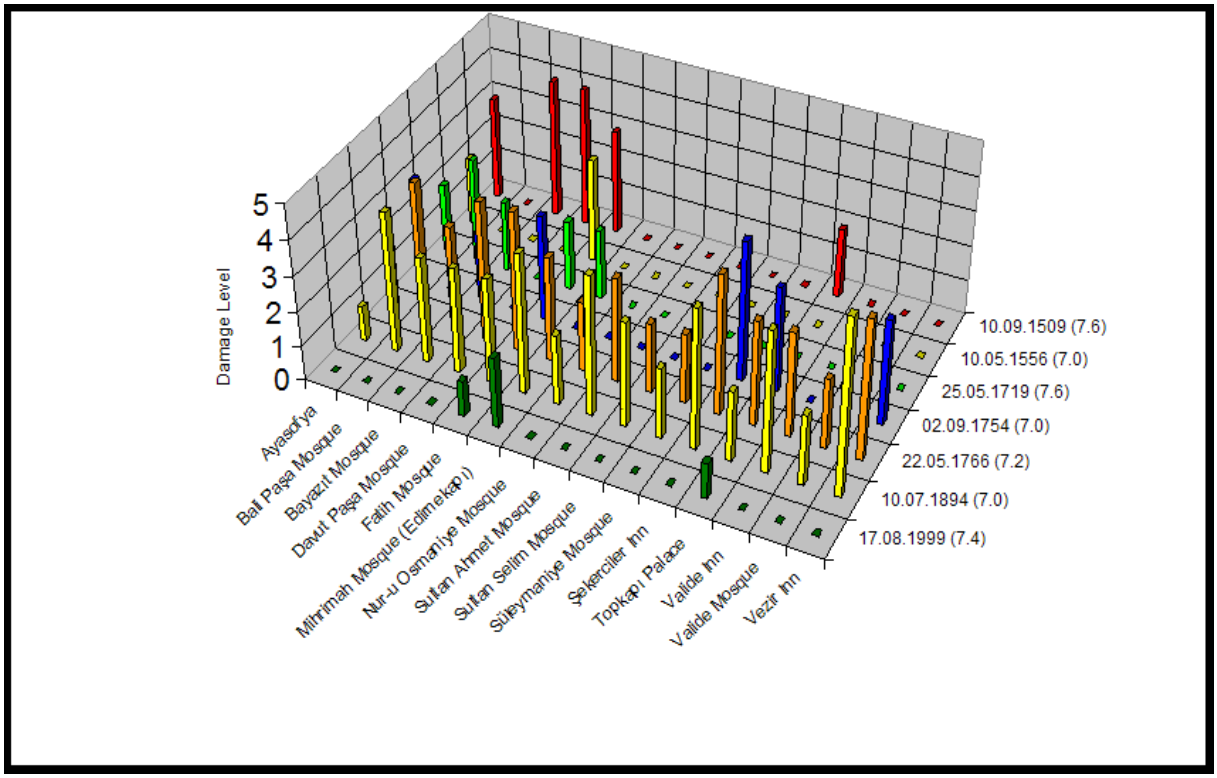
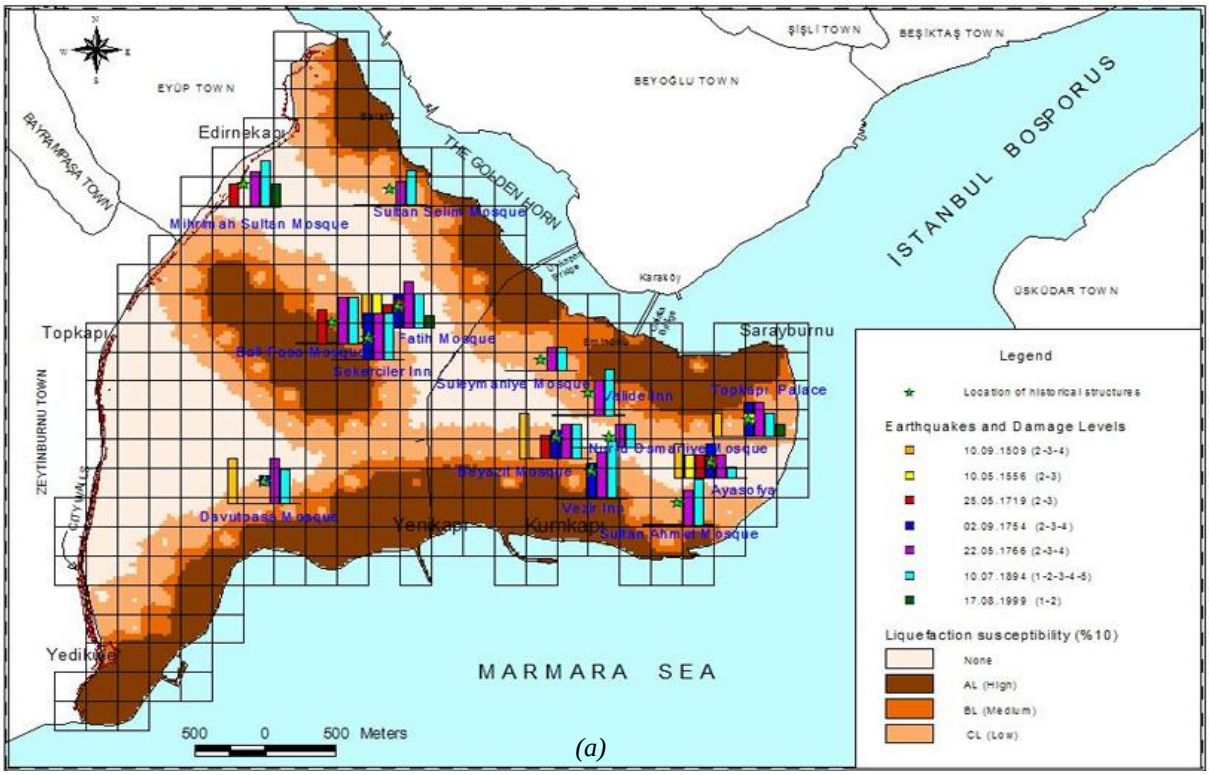


Fig. 8. Earthquake damages taken place in the historical structures within Old Istanbul (Fatih-Eminönü districts) Regions (Erdik, vd., 2001).



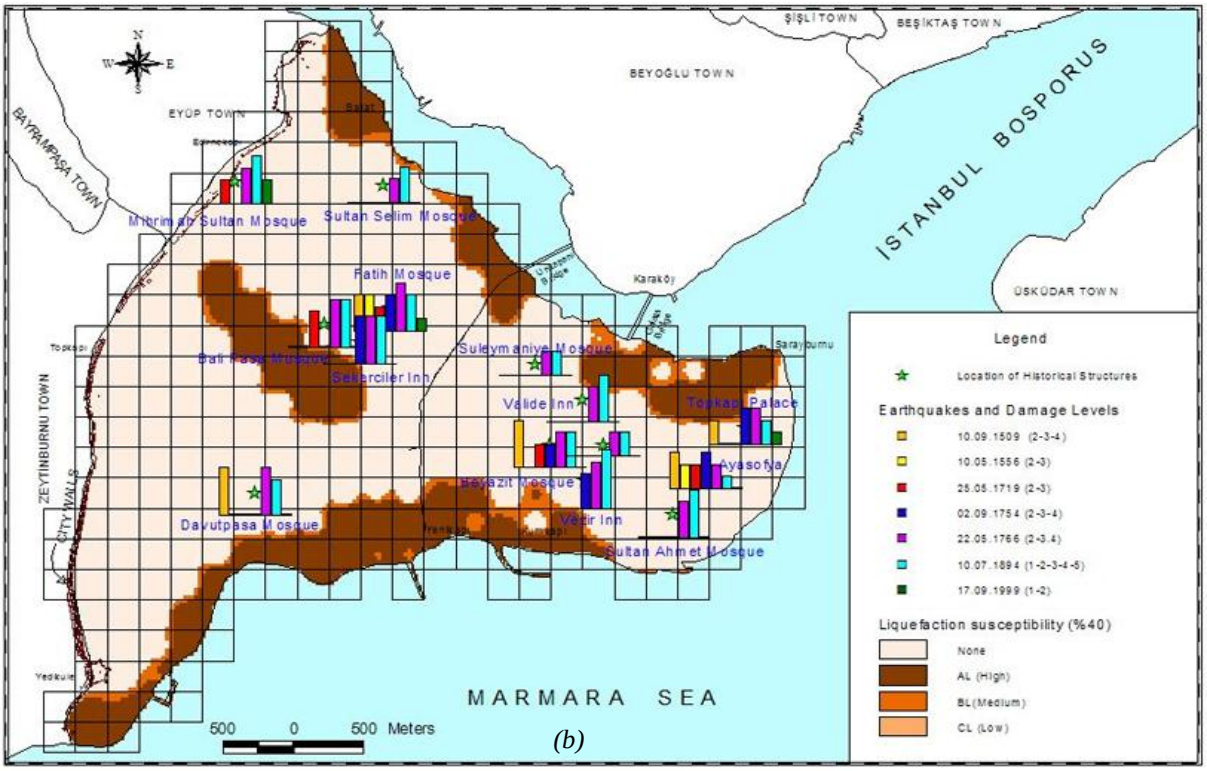


Fig. 9. a) and **b)** Comparison between damages taken place in the historical structures and liquefaction danger map prepared with respect to 10% and 40% chances of exceed for 50 years (İnce, 2005).

CONCLUSIONS

Overall, damage distribution and the risk of liquefaction of maps and geological maps, except for some regions they are in harmony can be seen. Damage in the area of the local soil conditions and liquefaction effects to be effective contributions may be mentioned.

ACKNOWLEDGMENTS

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BÇ-III(a) BETON ÇELİK ÇUBUKLARININ TÜRK DEPREM YÖNETMELİKLERİ AÇISINDAN İNCELENMESİ

THE EXAMINATION OF THE BC-III(a) REINFORCING STEEL BARS ACCORDING TO TURKISH SEISMIC CODES

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Anahtar Kelimeler: Beton Çelik Çubukları, Akma Dayanımı, Deprem Yönetmeliği, Kalite.

ÖZ Türkiye'de son yılarda meydana gelen büyük depremlerden dolayı yıkılan veya hasar gören yapılar üzerinde gerçekleştirilen araştırmalar sonrasında bu yapılarda taşıyıcı sistemin oluşturulmasında kullanılan malzemelerin kalitesinin kötü olduğu görülmüştür. Bu nedenle beton kalitesinin arttırılması için birçok çalışma yapılmıştır. Ancak, yapı taşıyıcı sisteminde beton kadar önemli olan beton çelik çubuklarının kalitesi yeterince dikkate alınmamış ve bu konuda fazla çalışma yapılmamıştır.

Bu çalışmanın amacı, elde edile deney raporlarına göre, akma dayanımı, çekme dayanımı ve kopma uzaması gibi mekanik özellikler dikkate alınarak beton çelik çubuklarının kalitesini istatistiksel olarak incelemektir. Ortadoğu Teknik Üniversitesi, İstanbul Teknik Üniversitesi ve Selçuk Üniversitesi malzeme laboratuarlarında çekme testine tabi tutulan çapları 8 mm ile 22 mm arasında değişen 5693 adet BÇ-III(a) beton çelik çubuğu numunesine ait deney raporları değerlendirilmiştir. Bu çalışmanın sonucunda, beton çelik çubuklarının mekanik özelliklerindeki değişkenliklerin yüksek olduğu ve geçmişe kıyasla beton çelik çubukların kalitesinin arttığı görülmüştür. Ayrıca, beton çelik çubuklarının % 42'sinin deprem yönetmeliğinin 1998 baskısındaki, % 21'inin ise 2007 baskısındaki koşulları sağlamadığı ortaya çıkmıştır.

GİRİŞ

Betonarme, beton ile çelik çubukların, birbirlerinin eksiklerini giderip beraber çalışacak şekilde bir araya getirilmesi sonucu ortaya çıkan kompozit malzemeye denilmektedir. Basınç dayanımı yüksek olan betonun eksikliği çekme dayanımının düşük olmasıdır. Betonun bu eksik tarafı, içerisine çekme dayanımı yüksek olan çelik çubuklar yerleştirilip bu iki malzeme arasındaki kenetlenmenin tam sağlanmasıyla giderilir. Bunun yanında, çelik çubukların önemli bir diğer görevi ise yapı elemanlarının sünekliğini arttırarak yapıya etkiyen düşey ve yatay yükler karşısında taşıyıcı yapı sisteminin göçme güvenliğini olumlu yönde arttırmaktır.

Betonarme bir yapı elemanın sergilemiş olduğu davranışı beton çelik çubukların yüzey özellikleri, kesitteki oranı, konum ve düzeni, imalat şekli ve en önemlisi bu çubukların mekanik özellikleri etkilemektedir. Bu özellikler, betonarme bir yapı elemanında beton çelik çubukların üzerine düşen görevi ne oranda yerine getirdiğini ortaya koymaktadır. Deprem etkisi altındaki betonarme yapılarda, donatı çelikleri büyük değerler alabilen çekme ve kesme gerilmelerine maruz kalabilmektedirler. Bu nedenle binalarda deprem etkisine maruz kalabilecek yapı elemanlarının içerisindeki çelik çubukların genel karakteristiklerinin bilinmesi ve ilgili standartları ne oranda sağladıklarının, dolayısıyla kalitesinin belirlenmesi, betonarme yapı elemanların davranışlarının analizi açısından önemli bir konu olarak karşımıza çıkmaktadır.

Üretim açısından kaliteli beton çelik çubukları denilince genellikle gelişmiş mekanik özelliklere sahip çelik çubuklar akla gelmektedir. Çelik çubuğun, çekme ve akma dayanımının yüksek olması, kopma uzamasının fazla olması gelişmiş mekanik özelliklere sahip olduğunu göstermektedir. Bir yapıda kaliteli beton çelik çubukların kullanılmasının nedenlerinden biri doğrudan yapı maliyeti ile alakalıdır. Buna örnek olarak: kaliteli beton çelik çubukları kullanıldığında, herhangi bir betonarme kesit için kullanılacak donatı alanı azalacak dolayısıyla yapıda kullanılacak beton çelik çubuğu miktarı düşecektir. Böylece işçilik masrafları azalacak, nakliye giderlerine daha az kaynak ayrılacaktır. Bununla birlikte üretim maliyeti dikkate alındığında, maliyet ile kalite arasında doğru orantı vardır. Yani üretim aşamasında kaliteli beton çelik çubuklar elde edilmesi üretim maliyetini arttırmaktadır. Bunun için kalite ve maliyet ikilisi arasında her zaman bir dengenin dikkate alınması gerekmektedir.

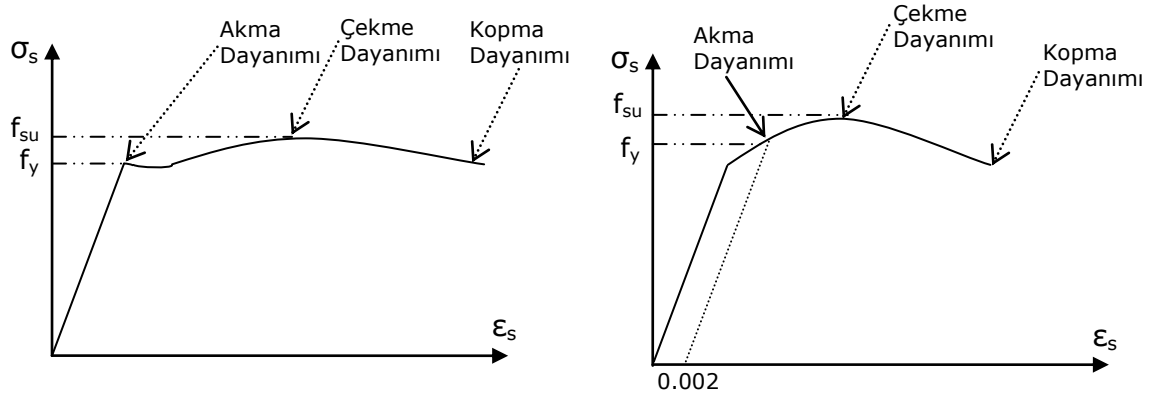
Bir yapıda kaliteli beton çelik çubukların kullanılmasının nedenlerinden bir diğeri ise yapı imal edilirken daha az miktarda donatı kullanılacağından yapı ağırlığı belirli bir oranda azalmaktadır. Bu da binaya etkiyen ölü yük değerini azaltacağı gibi aynı zamanda binaya etkiyen deprem yükünün miktarını da azaltacaktır. Bunun yanında bir yapı elamanı imal edilirken donatı aralığı beton döküm anında en büyük sorunlardan biri olarak karşımıza çıkmaktadır. Eğer donatılar arasındaki mesafe düşük ise vibratör kullanılsa bile betonun yapı elamanı içerisinde homojen dağılması zorlaşacak ve yapı elamanının bazı yerlerinde dayanım problemleriyle karşılaşılabilecektir. Gelişmiş mekanik özelliklere sahip beton çelik çubukları kullanıldığında daha az donatı gerekeceğinden donatılar arasındaki mesafe açılacak betonun daha rahat yerleşmesi sağlanacaktır. Ancak, üretim aşamasında herhangi bir çelik sınıfı için standartlarda belirtilen değerlerden daha fazla gelişmiş mekanik özelliklere sahip beton çelik çubuklarının üretilmesi durumunda, bu hesapta dikkate alınmadığı için yapıların güvenilirliğini azaltabilecek, göçmeye karşı güvenliğini olumsuz yönde etkileyecektir. Bu durum Türk Deprem Yönetmelikleri dikkate alınarak ileride ayrıntılı olarak ifade edilmiştir.

Bu çalışmada, bazı standart ve yönetmeliklerde S420a olarak gösterilen beton çelik çubuğu, Türkiye'de şu anda geçerli beton çelik çubuğu standardı olan TS 708'de (Beton Çelik Çubukları) belirtildiği şekilde BÇ-III(a) olarak gösterilmiştir. Bu çubuklar için çekme deneylerinden elde edilen değerlerin Türk Deprem Yönetmeliklerinde verilen sınır değerleri ne oranda sağladığı ile alakalı bir takım değerlendirmeler yapılacaktır.

BETON ÇELİK ÇUBUKLARIN MEKANİK ÖZELLİKLERİ

Beton çelik çubukların mekanik özellikleri, kalite açısından temel parametreler olarak çalışmalarda dikkate alınmaktadır. Bu çalışmada akma dayanımı, çekme dayanımı ve kopma uzaması gibi mekanik özellikler, beton çelik çubukların kalitesinin belirlenebilmesi için gerekli olacak temel parametreler olarak değerlendirilmiştir. Bu parametreler için sınır değerler donatı çelikleriyle ilgili standartlarda verilmiştir. Standartlarda öngörülen bu sınır değerlerle ilgili uygunluk kontrolü için beton çelik çubukları çekme deneylerlerine tabi tutulur. Çelik, çekme ve basınç altında benzer davranışlar gösteren bir yapı malzemesi olmasına rağmen çeliğin gerilme-birim şekil değiştirme özellikleri genellikle çekme deneyleri sonucunda bulunmaktadır.

Türk Standartlarında beton çelik çubukları imalat metoduna göre sınıflandırılırken sıcak haddeleme işlemiyle üretilenler, sıcak haddeleme esnasında ısıl işlem uygulanarak üretilenler, soğuk mekanik işlem uygulanarak üretilenler olarak üçe ayrılsa da uygulamada çelik donatılar bu sınıflar yerine soğukta işlem görmüş ve doğal sertlikte (sıcakta haddelenmiş) olarak ikiye ayrılmaktadır. Soğuk mekanik işlem uygulanarak gerekli dayanımlar elde edilmiş çelik, düşük ısıda çekilerek veya burularak elde edilmiş çeliklerdir (Doğangün, 2008).



a) Doğal sertlikteki çelik için σ-ε değişimi

b) Soğukta işlem görmüş çelik için σ-ε değişimi

Şekil 1. Doğal sertlikte ve soğukta işlem görmüş çelikler için tipik gerilme-birim uzama eğrileri.

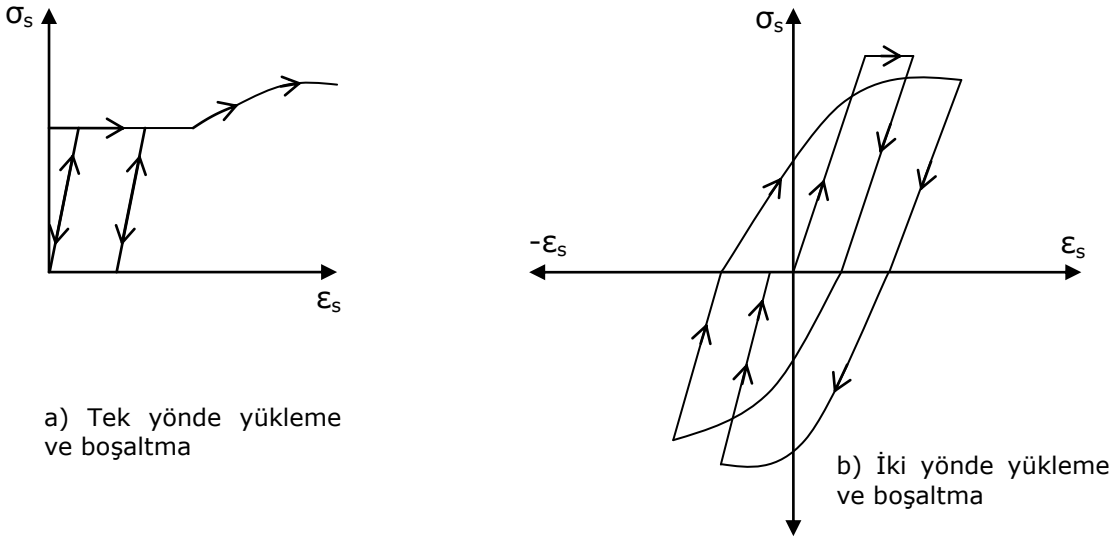
Şekil 1'de doğal sertlikteki çelik ve soğukta işlem görmüş çelik için deneysel olarak elde edilmiş olan tipik gerilme-birim uzama eğrileri gösterilmiştir. Şekilden de görüldüğü gibi bu iki sınıf çeliğin gerilme-birim deformasyon eğrileri birbirinden farklılıklar göstermektedir. Doğal sertlikteki çelikler ilk önce doğrusal bir davranış göstermekte daha sonra gerilmenin az değiştiği bir akma sahanlığına sahip olmaktadırlar. Doğrusal davranıştan akma sahanlığına geçiş akma sınırı ve bu sınıra karşı gelen gerilme akma gerilmesi (f_y) olarak ifade edilebilir. Temelde beton çelik çubukları bu gerilmeye bağlı olarak sınıflandırılırlar. Doğrusal davranış gösteren bölgede yük boşaltılırsa kalıcı deformasyon oluşmaz yani yükleme tamamen boşaltıldığında şekil değiştirme sıfırlanır. Akma sınırından sonra gerilmenin ve şekil değişmenin beraber arttığı bir bölge görülmektedir. Buna pekleşme bölgesi denilmektedir. Çekme deneyinde çeliğin kopmadan önce taşıyabildiği en büyük gerilme değerine çekme dayanımı denerek f_{su} ile gösterilir. Eksenel çekme altında yüklemeye devam edildiğinde çelik en zayıf bölgeden incelerek kopar. Donatının maksimum şekil değiştirmeye karşılık gelen ve donatının kopmadan hemen önce taşıdığı gerilme değerine kopma dayanımı denilmektedir. Kopma dayanımı her durumda maksimum gerilmeye eşit olmayabilmesine karşın bazı kaynaklarda maksimum gerilmeye karşı gelen dayanım olarak ifade edilmektedir.

Şekil 1-b'den görüleceği üzere soğukta işlem görmüş çelikler belirli bir noktaya kadar doğrusal bir davranış sergilemekte bu noktadan sonra elastik olma özelliğini kaybetmektedir. Doğrusal olan bu bölgenin eğimi doğal sertlikteki çeliğin eğimi ile aynı olmakta, yani bu iki tür çeliğin elastisite modülü yaklaşık olarak birbirine eşit olabilmektedir. Soğukta işlem görmüş çeliklerde belirli bir akma sınırı görülmediğinden %2 şekil değiştirmeye karşılık gelen gerilme değeri akma dayanımı olarak nitelendirilir.

Şekil 2-a'da sürekli ve tek yönlü (monotonik) yüklemede tekrarlanan yükleme altında elde edilen gerilme-şekil değiştirme diyagramı verilmiştir. Bu diyagramdan çeliğin tekrarlanan yük altında elastik sınırın ötesine geçildikten sonraki boşaltmada kalıcı şekil değiştirmeler meydana geldiği görülmektedir. Bununla birlikte çeliğe uygulanan gerilme akma sınırına ulaşmadan ortadan kalkarsa gerilme şekil değiştirme eğrisinin elastik parçası üzerinde geri dönülür.

Çelik davranışı, tersinir yükler altında gerilmenin işaret değiştirdiği durumlarda ise monotonik yüklemeden elde edilen davranıştan büyük farklılıklar gösterir. Deprem gibi yön değiştiren yükler söz konusu ve elastik sınır aşılıyorsa, gerilme-şekil değiştirme diyagramında çevrimsel davranış görülmektedir (Celep, 2009). Tersinir ve tekrarlanan yüklerden meydana gelen gerilme-şekil değiştirme diyagramı Şekil 2-b'de gösterilmiştir. Bu şekilden görüldüğü gibi akma sınırı geçildikten sonra eğer yüklemenin yönü

değiştirilirse meydana gelen ters işaretli gerilme altında akma gerilmesine ulaşılmadan gerilme-şekil değiştirme eğrisi doğrusallığını kaybetmekte daha az yük taşımaktadır. Bu durum "Bauschinger Etkisi" olarak ifade edilir. Bu etkiden dolayı tersinir yükler altındaki davranışın kestirilebilmesi için, donatı çeliğine daha önce uygulanan yük deformasyonlarının bilinmesi gerekmektedir. Bu nedenle çelik çubuklara uygulanan yük geçmişi önem kazanmaktadır (Ersoy ve diğ., 2003).



Şekil 2. Tekrarlanan ve tersinir yükler altında σ-ε eğrileri (Celep, 2009).

TÜRK DEPREM YÖNETMELİKLERİNDE BETON ÇELİK ÇUBUKLARININ MEKANİK ÖZELLİKLERİ

1998 yılında yayımlanan Afet Bölgelerinde Yapılacak Yapılar Hakkında Yönetmelikte beton çelik çubuklarının mekanik özellikleriyle ilgili bir takım değerlendirmeler yapılmıştır. Daha sonra 2007 tarihli Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelikte beton çelik çubuklarının mekanik özellikleri ilgili bazı sınırlandırmalar "Betonarme Binalar İçin Depreme Dayanıklı Tasarım Kuralları" bölümünde ele alınmıştır. Bu standarttaki sınırlandırma ve değerlendirmeler aşağıda verilmiştir:

a) Beton çelik çubuğun deneysel olarak bulunan akma dayanımı, ilgili çelik standardında öngörülen karakteristik akma dayanım değerinden en fazla % 30 fazla olabilir.

b) Deneysel olarak bulunan kopma dayanımı, yine deneysel olarak bulunan akma dayanımının minimum 1.15 katı olacaktır.

c) Etriye ve çiroz donatısı ile döşeme donatısı dışında, nervürsüz donatı çeliği kullanılamaz.

d) Binalarda kullanılan beton çelik çubuğun kopma uzaması en az %10 olacaktır.

e) S420'den (BÇ-III) daha yüksek dayanımlı donatı çeliği kirişsiz döşemelerde, dişli döşeme tablalarında, kirişli sistemlerin döşemelerinde, etriyelerde, bodrum katların çevresindeki dış perde duvarlarının gövdelerinde, deprem yüklerinin tümünün bina yüksekliği boyunca perdeler tarafından taşındığı ve aşağıda verilen koşulların (Denklem 1 ve 2) her ikisinin de sağlandığı binaların perde gövdelerinde kullanılabilir. Bunların haricindeki betonarme taşıyıcı sistem elemanlarında S420 çeliğinden yüksek dayanımlı bir çelik sınıfı kullanılamaz.

$$\frac{\sum A_g}{\sum A_p} \geq 0.002 \quad (1)$$

$$\frac{V_t}{\sum A_g} \leq 0.5 f_{ctd} \quad (2)$$

ΣA_g = Herhangi bir katta, göz önüne alınan deprem doğrultusuna paralel doğrultuda perde olarak çalışan taşıyıcı sistem elemanlarının enkesit alanlarının toplamı

ΣA_p = Binanın tüm katlarının plan alanlarının toplamı

V_t = Binaya etkiyen toplam deprem yükü (taban kesme kuvveti)

f_{ctd} = Betonun tasarım çekme dayanımı

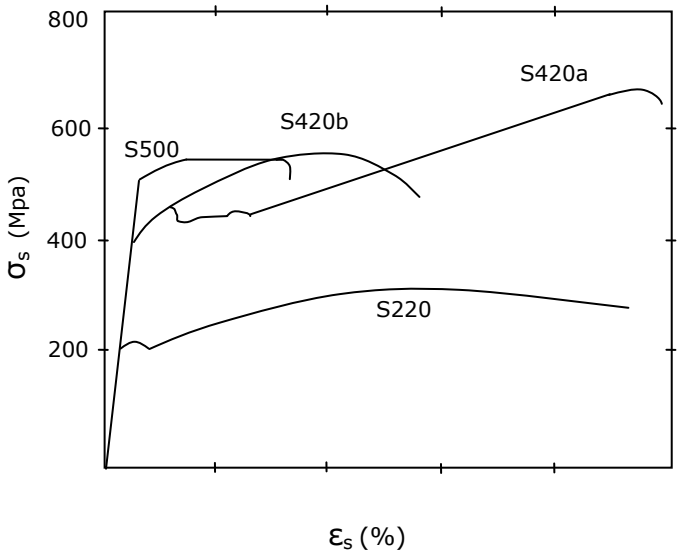
Afet Bölgelerinde Yapılacak Yapılar Hakkında Yönetmelik-1998 ile Deprem Bölgelerinde Yapılacak Yapılar Hakkında Yönetmelik-2007 arasında beton çelik çubuklarının mekanik özellikleri açısından bazı farklar vardır. Bu iki yönetmeliğe bu çalışmada Türk Deprem Yönetmelikleri denilecektir. Deprem Yönetmeliğinin 1998 tarihli baskısında deneysel olarak bulunan kopma dayanımının deneysel olarak bulunan akma dayanımına oranı 1.25'den fazla olması gerektiği öngörülmektedir. Bu oran 2007 tarihli Türk Deprem Yönetmeliğinde 1.15 değerine düşürülmüştür. Deprem Şurası-2004 Yapı Malzemeleri Komisyonu Raporunda bu oranın 1.25 olması gerektiği vurgulanmıştır. Bunun yanında, Türk Deprem Yönetmeliği-1998'de prefabrike yapı elemanlarında S420 çeliğinden daha yüksek dayanımlı öngerilmeli çeliklerin kullanılabileceği ifade edilmişken bu konu Türk Deprem Yönetmeliği-2007'de ele alınmamıştır.

TÜRK DEPREM YÖNETMELİĞİNDEKİ MEKANİK ÖZELLİKLERLE İLGİLİ KOŞULLARIN DEĞERLENDİRİLMESİ

Süneklik: bütün bir yapının, yalnızca bir yapı elamanının, bu elemanın herhangi bir kesitinin veya beton çelik çubuğu gibi bir malzemenin taşıma gücünde önemli bir azalma olmadan deformasyon yapabilme ve tekrarlı yükler altında enerji absorbe edebilme özelliğine denilmektedir. Deprem etkisi altındaki bir yapıda, enerjinin çoğu plastik mafsal oluşan kesitlerde tüketilmektedir. Dolayısıyla yapılarda süneklik plastik mafsallarla doğrudan ilgilidir. Plastik mafsal kesitte yük artmadığı halde şekil değiştirmelerin artması olarak tarif edilebilir.

Betonarme bir kesitte eksenel yük düzeyi azaldıkça süneklik artmaktadır. Bundan dolayı kirişlere kıyasla çok fazla eksenel yük taşıyan kolonların sünekliği kirişlerden çok daha düşüktür. Dolayısıyla betonarme çerçevelerin tasarımında plastik mafsalların kolonlar yerine kirişlerde oluşmasına dikkat edilmektedir. Deprem yönetmeliklerinde kirişlerin kolonlardan daha zayıf olarak projelendirilmesinin istenmesi, kolonun kirişe oranla daha az sünek davranış sergilemesinden kaynaklanmaktadır. Kolon kiriş birleşim bölgelerinde kirişlerin taşıma gücü momentlerinin kolonların taşıma gücü momentlerinden küçük olması durumunda güçlü kolon-zayıf kiriş ilkesi sağlanmaktadır. Aynı zamanda taşıyıcı sistem elemanlarının, yapıya etkiyen yükler altında eksenel yük ezilmesi veya kesme kırılması gibi ani ve gevrek kırılmaların önlenerek bu elemanların nihai güç tüketme kapasitelerine sünek bir davranışla ulaşmalarını sağlamak inşaat mühendisliğinde temel amaçlardan biridir.

Sünek davranışa sahip bir betonarme elemanda kullanılan beton çelik çubuklarının da sünek bir davranış sergilemesi gerekmektedir. Yüksek dayanıma sahip çeliklerin sünekliklerinin nispeten daha düşük olduğu bilinmektedir. Eğer çeliğin akma dayanımı soğukta işlem görerek arttırılmışsa kopma uzaması çok kısalmış ve akma platosu neredeyse kaybolmuştur (Şekil 3). Bundan dolayı deprem yönetmeliğinde S420a sınıfından daha yüksek dayanıma sahip donatı çeliği kullanılması yasaklanmıştır. Ayrıca %10'luk minimum kopma uzaması da beton çelik çubuğun sünek olmasıyla alakalı deprem yönetmeliğinde verilen önemli bir sınırlandırmadır.



Şekil 3. Değişik beton çelik çubukları için gerilme- şekil değiştirme eğrileri (Celep, 2009).

Beton çelik çubuklarının başta akma dayanımı olmak üzere mekanik özelliklerinin ilgili standartlarda verilen değerlerden daha düşük olması yapı elemanının dayanımının düşmesine güvenilirliğin azalmasına yol açacaktır. Çelik çubuğun akma dayanımının güvenilir olması, proje mühendisinin standartlarda belirtilen akma dayanımına dayanarak hesap yapması nedeniyle çok önemlidir. Bunun yanında standartlarda belirtilen akma dayanımının düşük olması gibi bu dayanımın yüksek olması da bir takım problemleri ortaya çıkarmaktadır.

Eğilme altındaki bir betonarme elamanın sünek bir davranış sergileyebilmesi için beton ezilmeden önce, çelik akmalı ve sabit yük altında çelik uzamaya devam etmelidir. Betonarme elemanın güç tüketimi betonun ezilmeye başlaması ile sonlanacak olduğundan donatı aktıktan sonra uzamaya devam etmeli ve betonun ezilmesi bu uzama miktarını mümkün olduğu kadar uzatacak şekilde sonradan oluşmalıdır. Eğer yapı elamanında kullanılan çeliklerin çekme testlerinden elde edilen "gerçek" dayanımları standartlarda öngörülenlerden fazla ise kesitte beton ezilmeye başlamadan önce donatının akmaya başlayacağı güvenliği azalmış olacaktır. Başka bir deyişle karakteristik akma dayanımı 420 Mpa olan beton çelik çubuğun çekme testlerinden elde edilen akma dayanımı 550 Mpa çıkması durumunda, proje mühendisinin sünek davranışa uygun olarak tasarımını yapmış olduğu yapı elamanının davranışı değişecektir. Bu durumda istenilen yerlerde plastik mafsallar oluşmadan önce ani ve gevrek eğilme veya kesme kırılması oluşabilecektir.

Bir diğer önemli neden de akma dayanımı standartta belirtilen nominal değerden daha yüksek olan çeliğin kullanılması sonucu yapı elemanının eğilme dayanımı artacaktır. Proje tasarımında hesaplanan momentlerden daha fazla moment taşıyan kirişler güçlü kolon-zayıf kiriş ilkesinin sağlanmasında olumsuzluklar yaratabilecektir. Bilindiği gibi, deprem yönetmeliğinde binaya etkiyen deprem yükleri altında taşıyıcı sistemde bir takım hasarların oluşması prensip olarak kabul edilmektedir. Ancak yapının çökme güvenliğinin tehlikeye girmemesi için hasarın sünek bölgelerde meydana gelmesi ve deprem enerjisinin bu sünek bölgelerde tüketilmesi gerekmektedir. Bundan dolayı plastik mafsallar kirişlerde oluşmalı kolonlarda plastik mafsal oluşmasına imkân verilmemelidir.

Yukarıda anlatıldığı gibi beton çelik çubukların gerçek akma dayanımları standartlarda yazılı olan nominal değerlerin üstünde olması durumunda güçlü kolon-zayıf kiriş yaklaşımında sorunlar çıkabilir ve kesitin kesme kapasitesinin eğilme kapasitesinden daha zayıf olduğu durumlar görülebilir. Bundan dolayı deprem yönetmeliğinde beton çelik

çubuklarının deneysel olarak elde edilen akma dayanımının karakteristik akma dayanımından en fazla % 30 fazla olabileceği belirtilmektedir.

Deprem Yönetmeliğinde deneysel olarak bulunan çekme dayanımının deneysel olarak bulunan akma dayanımına oranı minimum 1.15 olarak verilmiştir. Bu koşul ile sünek davranışa sahip yapı elamanı tasarlama amacına yönelik sünek beton çelik çubukların üretimi sağlanmaktadır. Bunun yanında, bu oran ne kadar büyük ise pekleşme bölgesindeki eğim artacak, tersinir ve tekrarlanan yükler altında oluşma ihtimali yüksek olan beton çelik çubuğun burkulması o oranda gecikecektir. Çekme dayanımının, akma dayanımına oranının 1.15 değerinden daha az olması durumunda projede tasarlanan güçlü kolon-zayıf kiriş yaklaşımında da problem çıkabilir.

BETON ÇELİK ÇUBUKLARIN KALİTESİNİN İNCELENMESİ

AMAÇ VE KAPSAM

Bu bölümde Türkiye'de son yılarda meydana gelen Erzincan, Dinar, Adana, Marmara, Afyon ve Bingöl depremlerinden sonra beton çeliği üretim kalitesinde bir iyileşme olmuş mudur sorusuna cevap aranmaya çalışılacak, özellikle Türk Deprem Yönetmeliklerinde öngörülen sınır değerleri, beton çelik çubukların ne oranda sağladığı araştırılacaktır.

Yapılan gözlemler sonucunda Türkiye'de son yıllarda yapılan bina ve diğer yapılarda düz demir (BÇ-I(a)) kullanılmadığı sonucu çıkartılmıştır. Bundan dolayı bu çalışmada düz demirler incelenmeyecektir. Ayrıca yeterli veri bulunmadığından IV-a donatı çubukları da araştırma kapsamı dışında bırakılmıştır. İnşaatlarda çok büyük bir oranda nervürlü (BÇ-III(a)) beton çelik çubukları kullanıldığından bu çalışmada bu çubukların akma, çekme dayanımları ve kopma uzaması yüzdelerinin, Deprem Yönetmeliklerindeki sınır değerleri ne oranda sağladıkları istatistiksel olarak belirlenecektir. Sonuç olarak geçmişten günümüze ne tür farklılıklar olduğu hakkında bir fikir edinilmeye çalışılacak ve Türkiye'de kullanılan veya ihraç edilen beton çelik çubukların kalitesi sorgulanacaktır.

Veri kaynağı, hangi fabrikada üretildiği belli olmayan BÇ-III(a) çelik çubuklardan alınan numunelerin çekme testine tabi tutulduktan sonra düzenlenen deney raporlarıdır. Bu çalışmada 8 mm ile 22 mm çapları arasında toplam 5693 adet BÇ-III(a) beton çelik çubuğu değerlendirilmiştir. Ortadoğu Teknik Üniversitesi (ODTÜ), İstanbul Teknik Üniversitesi (İTÜ) ve Selçuk Üniversitesi (SÜ) malzeme laboratuarlarında çekme testine tabi tutulan beton çelik çubuk numuneler için düzenlenen raporlardan yararlanılarak akma dayanımı, çekme dayanımı ve uzama oranlarının her bir beton çeliği çapına göre ortalama değerleri ve değişkenlik katsayıları hesaplanmıştır. Değişkenlik katsayısı standart sapmanın ortalama değere oranı olarak ifade edilmektedir.

VERİLERİN DEĞERLENDİRİLMESİ VE ELDE EDİLEN SONUÇLAR

Üniversite laboratuarlarına getirilen numuneler üzerinde yapılan çekme deneyleri dikkate alınarak, çeteleler oluşturulmuş, elde edilen sonuçlar tablolar halinde gösterilmiş ve grafik olarak yorumlanmıştır. Aşağıdaki tablo ve grafiklerde, Türkiye'de inşaatlarda kullanılan beton çelik çubuklarının akma dayanımı, çekme dayanımı ve uzama yüzdelerinin ortalama değerleri gösterilmiştir. Bunun haricinde akma, çekme ve uzama yüzdelerindeki değişkenlik katsayıları hesaplanmıştır. Ayrıca Türk Deprem Yönetmeliklerinde istenen, deneysel olarak bulunan akma dayanımı, ilgili çelik standardında öngörülen karakteristik akma dayanım değerinden en fazla % 30 olması ve deneysel olarak bulunan kopma dayanımının, yine deneysel olarak bulunan akma dayanımının minimum 1.15 katı olması gibi bazı sınırlandırılmalar bu bölümde incelenmiştir. Bu tablo ve grafikler ışığında Deprem yönetmeliklerdeki sınır değerlerin ne oranda sağlandığı belirlenerek Türkiye'de kullanılan donatı çeliklerinin mekanik özellikleri ve kalitesiyle ilgili bir takım sonuçlar çıkarılabilir.

Tablo 1'de BÇ-III(a) beton çelik çubuklarının akma dayanımına ait istatistiksel parametreler değişik çubuk çaplarına göre gösterilmiştir. Bu tablodan görüldüğü gibi elde edilen verilerin değerlendirilmesi sonucunda her bir çelik çapına ait ortalama akma dayanımı 477.36 Mpa ile 516.27 Mpa değerleri arasında değişmiş, çelik çaplarının ağırlıklı ortalaması ise 497.96 Mpa olarak bulunmuştur. Akma dayanımındaki değişkenlik katsayısı ise 0.098 ile 0.143 arasında yer almış, ağırlıklı ortalama değeri ise 0.134 olarak bulunmuştur.

Bu tabloda deneysel olarak ölçülen akma dayanımının karakteristik akma dayanımına oranı 1.30'dan fazla olan (f_y/f_{yk} >1.30) donatı çeliklerinin yüzdeleri, ortalama akma değerleri ve akma dayanımındaki değişkenlik katsayıları her bir çelik çapı için ayrı ayrı gösterilmiştir. Bütün çelik çaplarına ait, deneysel olarak ölçülen akma dayanımının karakteristik akma dayanımına oranı 1.30'dan fazla olan beton çelik çubuğu oranı ortalama olarak %20.7 değerini almıştır. Diğer bir deyişle, çekme testleri sonucunda bulunan akma dayanımları dikkate alındığında beton çelik çubukların ancak %79.3'ü Türk Deprem Yönetmeliklerinde istenilen bu koşulu sağlamıştır.

Tablo 1. BÇ-III(a) beton çelik çubukların akma dayanımına ait istatistiksel parametreler.

Çubuk çapı (mm)	**8**	**10**	**12**	**14**	**16**	**18**	**20**	**22**	**Bütün çaplar**
Numune sayısı	644	817	945	750	859	438	739	452	5693
Ortalama akma ($\bar{f}_y$)	477.36	500.02	504.52	481.90	493.26	492.28	516.27	505.81	497.96
Değişkenlik katsayısı	0.142	0.143	0.127	0.118	0.150	0.137	0.119	0.098	0.134
f_y/f_{yk}> 1.30 yüzdesi	13.4	25.1	25.5	10.4	17.8	16.0	28.0	19.2	20.7

Tablo 2'de, her bir çelik çapına göre ayrı ayrı olmak üzere çekme dayanımı, f_{su} ve çekme dayanımındaki değişkenlik katsayısı verilmiştir. Ayrıca deneysel çekme dayanımının deneysel akma dayanımına oranının 1.15 ve 1.25'den küçük olan beton çelik çubuğu yüzdeleri gösterilmiştir.

Bütün çelik çaplarının ağırlıklı ortalamaların hesaplanması sonucunda ortalama çekme dayanımı $\bar{f}_{su}$ =658.92 Mpa olarak bulunmuştur. Ayrı ayrı çelik çaplarına ait ortalama çekme dayanımı 648.97 Mpa ile 678.48 Mpa arasında değişmiş, büyük çaplı çelik çubukların ortalama çekme dayanımının diğerlerine kıyasla biraz daha yüksek olduğu görülmüştür. Ortalama değişkenlik katsayısı ise 0.115 olarak hesaplanmıştır. Bütün çelik çaplarının ağırlıklı ortalamaların hesaplanması sonucunda bulunan ortalama çekme dayanımının karakteristik çekme dayanımına oranı yaklaşık olarak f_{su}/f_{sk}=1.32'dir.

Deprem Yönetmeliği-2007'de deneysel olarak bulunan çekme dayanımının deneysel olarak bulunan akma dayanımına oranının, f_{su}/f_y en küçük 1.15 olması gerektiği ifade edilmiştir. Beton çelik çubukların %5.4'ü ise f_{su}/f_y=1.15 oranından daha küçük değerler alarak Deprem Yönetmeliği-2007'yi sağlamamıştır. Deprem Yönetmeliği-1998 dikkate alındığında ise çelik çubukların %42'si, bu yönetmelikte öngörülen f_{su}/f_y>1.25 koşulunu sağlamamıştır.

Tablo 2. BÇ-III(a) beton çelik çubuklarının çekme dayanımının istatistiksel parametreleri ve çekme dayanımının akma dayanımına oranları.

Çubuk çapı (mm)	8	10	12	14	16	18	20	22	Bütün çaplar
Numune sayısı	644	817	945	750	859	438	739	452	5693
Ortalama çekme ($\bar{f}_s$)	653.98	678.48	650.62	652.27	666.55	648.97	671.67	660.37	658.92
Değişkenlik katsayısı	0.128	0.110	0.107	0.111	0.129	0.091	0.104	0.087	0.115
f_{su}/f_y< 1.15 yüzdesi	6.8	8.7	9.3	5.7	3.4	3.2	4.3	2.7	5.4
f_{su}/f_y< 1.25 yüzdesi	28.9	37.5	5.6	41.1	41.3	40.2	54.8	49.6	42

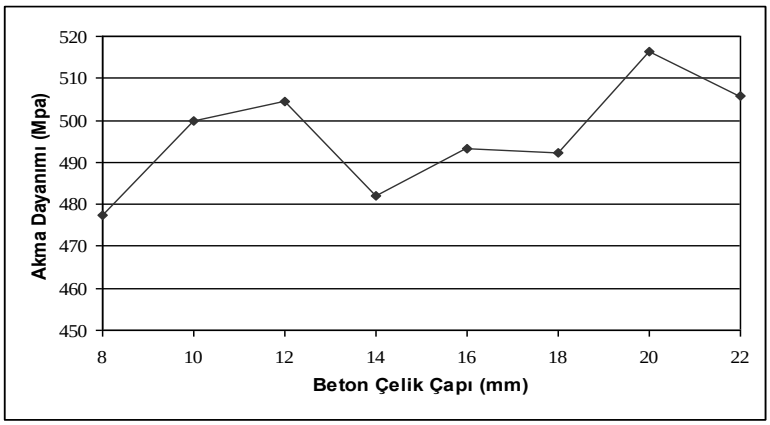
Tablo 3'te ise beton çelik çubuklarının uzama yüzdelerinin ve değişkenlik katsayılarının ortalama değerleri bu çalışmada dikkate alınan beton çelik çubuğu çaplarına göre gösterilmiştir. Bütün çelik çaplarının ağırlıklı ortalamaların hesaplanması sonucunda bulunan ortalama uzama oranı %19.04 olarak bulunmuştur. Her bir çelik çubuğu çapı için ortalama uzama oranı %18.01 ile %20.82 arasında değişmiştir. Değişkenlik katsayısı ise çelik çaplarına göre 0.133 ile 0.315 arasında yer almış, ağırlıklı ortalama değeri 0.184 olarak hesaplanmıştır.

Tablo 3'te dikkat çekici bir diğer husus da çelik çapı arttıkça uzama oranının düştüğü gerçeğidir. 8 mm çapındaki bir çubuğun ortalama kopma uzaması %20.82 iken 20 mm çapındaki bir çubuğun ortalama kopma uzaması ise %18.14'tür. Türk Deprem Yönetmeliklerinde kullanılan beton çelik çubuğunun kopma uzamasının en az %10 olması istenmektedir. Kopma uzamasının % 10'dan daha az olduğu çelik çubuk oranı bütün çelik çaplarının ağırlıklı ortalamaların hesaplanması sonucunda % 0.4 gibi küçük bir değer olduğu görülmüştür. Bunun yanında ortalama kopma uzamasının en yüksek olduğu 8 mm çapındaki beton çelik çubuklarda % 10 sınırlamasını sağlamayan çubuk sayısı yine % 2.9 gibi bir oranla diğer çubuk çaplarına kıyasla en yüksek olarak kendini göstermektedir.

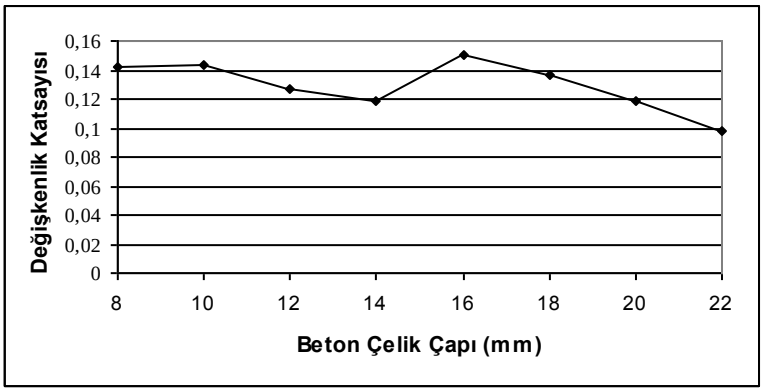
Tablo 3. BÇ-III(a) beton çelik çubuklarının uzama yüzdelerinin ortalama değerleri ve değişkenlik katsayıları.

Çubuk çapı (mm)	8	10	12	14	16	18	20	22	Bütün çaplar
Numune sayısı	644	817	945	750	859	438	739	452	5693
Ortalama uzama oranı (%)	20.82	19.06	19.68	19.66	18.95	19.03	18.01	18.14	19.04
Değişkenlik katsayısı	0.315	0.207	0.178	0.169	0.159	0.161	0.133	0.151	0.184
Uzama oranı<%10	2.9	0	0.2	0.1	0	0	0.2	0	0.4

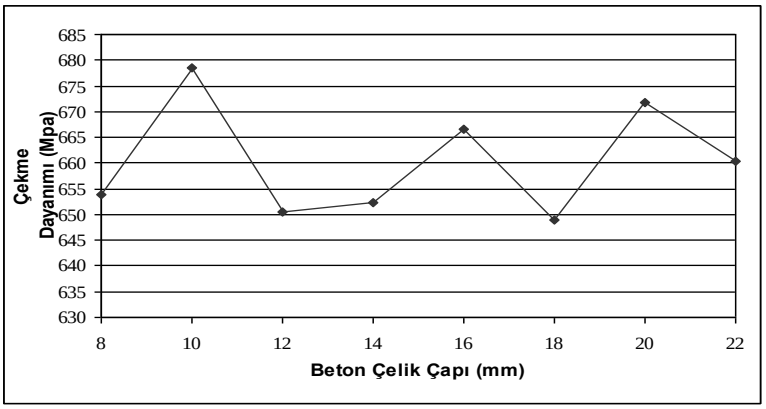
Beton çelik çubukların akma dayanımı çekme dayanımı ve uzama oranlarının ortalama değerleri ve bu parametrelerin ortalama değişkenlik katsayıları her bir çelik çapına göre Şekil 4-9'da grafik olarak gösterilmiştir. Bu şekillerden anlaşılacağı üzere akma ve çekme dayanımları çelik çapına çok fazla bağlı olmamakla beraber kopma uzaması çelik çapı arttıkça düşmektedir. Bunun yanında akma ve çekme dayanımlarındaki değişkenlikler çelik çaplarına göre incelediğinde dikkat çekici bir farklılık tespit edilememiş fakat uzama oranları söz konusu olduğunda beton çelik çapı arttıkça, bu parametredeki değişkenlik katsayısının açık bir şekilde azaldığı görülmüştür.



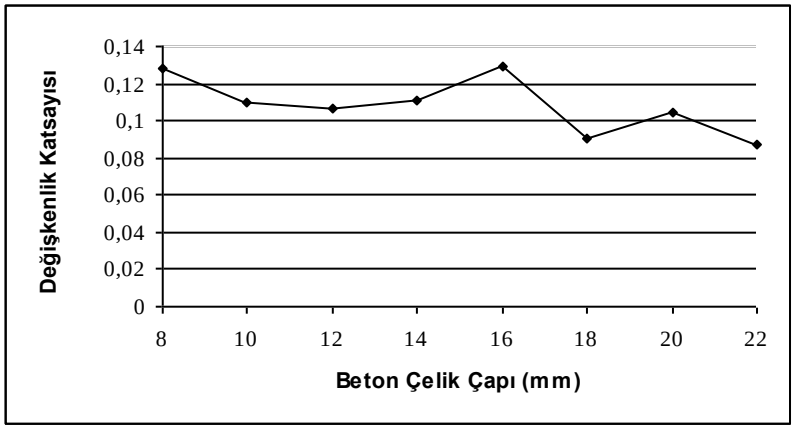
Şekil 4. Ortalama akma dayanımının (f_y) beton çelik çaplarına göre değişimi.



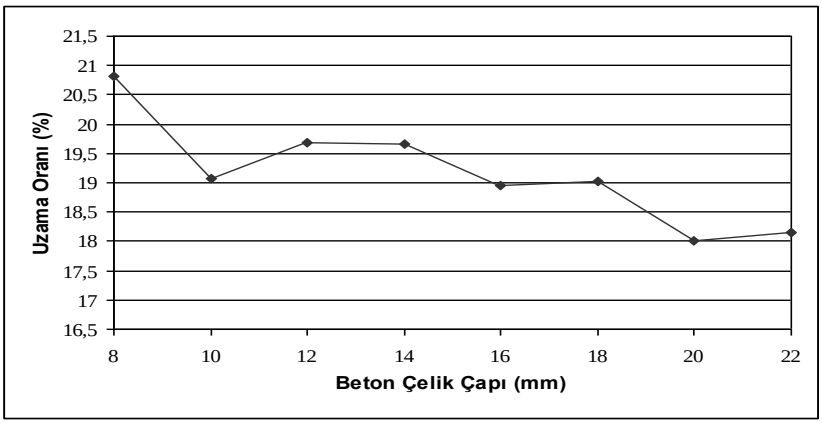
Şekil 5. Akma dayanımındaki değişkenlik katsayılarının beton çelik çaplarına göre değişimi.



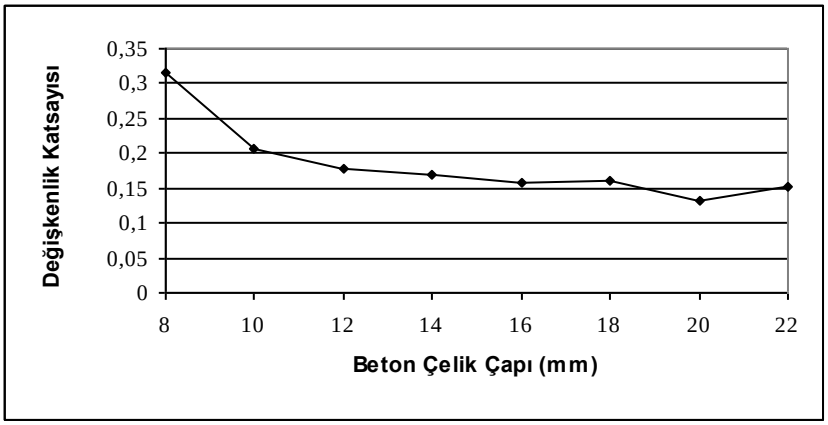
Şekil 6. Ortalama çekme dayanımının (f_{su}) beton çelik çaplarına göre değişimi.



Şekil 7. Çekme dayanımındaki değişkenlik katsayılarının çelik çaplarına göre değişimi.



Şekil 8. Ortalama uzama oranlarının beton çelik çaplarına göre değişimi.



Şekil 9. Uzama oranındaki değişkenlik katsayılarının beton çelik çaplarına göre değişimi.

SONUÇ VE DEĞERLENDİRME

Bu çalışmada beton çelik çubukların akma dayanımı, çekme dayanımı ve kopma uzaması oranları gibi mekanik özelliklerinin neler oldukları ifade edilmiş, çalışmanın altyapısını oluşturacak şekilde bu özellikler hakkında teorik bilgiler verilmiştir. Doğal sertlikte ve soğukta işlem görmüş çelikler için tipik gerilme-birim uzama ilişkileri incelenmiştir. Çelik davranışının gerek monotonik yükler altında gerekse tersinir yükler altında nasıl olacağı belirtilmiş, birbirlerinden farklılıkları ifade edilmiştir. Daha sonra, Türk Deprem Yönetmelikleri, beton çelik çubukların mekanik özellikleri açısından incelenmiş ve bu yönetmeliklerdeki sınırlandırmalar irdelenmiştir. Türk Deprem Yönetmeliği-1997'deki bazı koşulların, Türk Deprem Yönetmeliği-2007'deki koşullardan farklı olduğu görülmüştür. Bu yönetmeliklerde verilen koşulların deprem etkisi altındaki bir yapıda sünekliliği ve plastik mafsalların oluşumunu nasıl etkileyeceği ifade edilmiştir.

Çalışmada kullanılan veri kaynağı, ODTÜ, İTÜ ve SÜ İnşaat Mühendisliği Bölümü Malzeme laboratuarlarına çekme testine tabi tutulmak üzere getirilen ve hangi fabrikada üretildiği belli olmayan çelik çubuklardan alınan numunelerin kırılması sonucu düzenlenen deney raporlarıdır. Bu veri kaynağı kullanılarak beton çelik çubukların kalitesi, diğer bir deyişle Türk Deprem Yönetmeliklerindeki sınır değerleri ne oranda sağladığı istatistiksel olarak araştırılmıştır.

Elde edilen veriler incelendiğinde, akma dayanımı için öngörülen sınır değeri sağlayamayan beton çelik çubuğun çekme dayanımı için öngörülen sınır değerleri sağlayabildiği ortaya çıkmış; ama çekme dayanımı için öngörülen koşulu sağlayamayan numunenin minimum akma dayanımını da sağlayamadığı görülmüştür. Ayrıca,

numunelerin akma ve çekme dayanımları yükseldikçe kopma uzamalarının azaldığı gözlemlenmiştir.

Ortalama akma dayanımı 497.96 Mpa olarak bulunmuştur. Akma dayanımındaki ortalama değişkenlik katsayısı ise 0.134 çıkmıştır. Bu iki parametre dikkate alındığında geçmişe kıyasla beton çelik çubuğu kalitesinde bir artış olduğu söylenebilir. Değişkenlik katsayısının bu kadar büyük çıkması başka ülkelerin standartlarına göre üretilmiş çubukların ihraç fazlası olduğunda iç piyasaya BÇ-III(a) çelik çubuğu olarak pazarlandığının bir göstergesidir.

İncelenen çubukların %5.4'ü deneysel olarak ölçülen akma dayanımının deneysel olarak ölçülen çekme dayanımına oranı 1.15 değerinden küçük değerler alarak Deprem Yönetmeliği-2007'yi sağlamamıştır. Deprem Yönetmeliği-1998 dikkate alındığında ise çelik çubukların %42'si, bu standartta öngörülen deneysel çekme dayanımının deneysel akma dayanımına oranının 1.25'den büyük olması gerektiği koşulunu sağlamamıştır. Ayrıca beton çelik çubuğu numunelerinin %20.7'si, f_y/f_{yk} >1.30 değerinden büyük orana sahiptir. Diğer bir deyişle, çekme testleri sonucunda bulunan akma dayanımları dikkate alındığında %20.7'si 1998 ve 2007'de yayımlanan Türk Deprem Yönetmeliklerinin her ikisinde de istenilen bu koşulu sağlamamıştır.

Beton çelik çubukların akma dayanımı, çekme dayanımı ve kopma uzamaları beton çelik çubuğu çaplarına göre ayrı ayrı incelenmiştir. Bu inceleme sonucunda beton çelik çubuğu çaplarının akma ve çekme dayanımları üzerinde dikkat çekici bir etkisinin olmadığı tespit edilmiştir. Bir başka deyişle, çubuk çapı arttıkça akma ve çekme dayanımında artma veya azalma meydana gelmediği gözlemlenmiştir. Ayrıca, çap azaldıkça malzemenin kopma uzamasında artma tespit edilmiştir. Buradan da küçük çaplı beton çelik çubuklarının sünekliğe daha olumlu etki yaptığı sonucu çıkarılabilir.

Akma ve çekme dayanımlarındaki değişkenlikler çelik çaplarına göre ayrı ayrı değerlendirildiğinde ilgi çekici bir değişim gözlemlenememiş ama kopma uzama oranları söz konusu olduğunda beton çelik çapı arttıkça, bu parametredeki değişkenlik katsayısının açık bir şekilde azaldığı görülmüştür.

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THE HISTORICAL IMPROVEMENT, LITERATURE REVIEW OF STEEL PLATE SHEAR WALLS AND USING IN HIGH SEISMIC REGIONS

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Key Words: Steel plate shear wall; Lateral load resisting systems; Energy absorption capacity; Infill plate

ABSTRACT The main aim of this article is to summarize the valuable studies on steel plate shear walls and to offer thanks to all researchers who studied these subjects. Another aim was to give summarized information about using an alternative lateral load resisting system to the designers and researchers. This shear wall system was used in some buildings in Japan and North America as lateral load carrying system. Designers chose this system because of its advantages which are explained in this article. Our country is also located in a high seismic region so it is important to study such alternative and beneficial lateral load resisting systems. Steel plate shear walls consist of boundary frames which are infilled by stiffened or unstiffened thin steel plates. The beam to column connections of the frame can be moment resisting or shear type connection. The infill plate can be connected to the frame either directly or through fish plate which is welded or bolted.

Analytical and experimental studies on steel plate shear walls have been conducted since early 1970's. Most of the studies have been done in Japan, the United States of America and Canada. Early researches were conducted in Japan and a few years later such studies began in the United States and Canada. Today, it is possible to obtain tens of PhD theses and technical articles about steel plate shear walls. Early experimental and analytical studies in Japan were conducted with heavily stiffened plates and some designs have also been made this way. Early studies and designs of steel plate shear walls were the result of the opinion of preventing shear buckling of infill plates under lateral loading. With this opinion, it was impossible to use the post buckling strength and load-carrying strength (Behbahanifard, 2003).

Using thin plates in a shear wall to resist lateral loads is not a new idea. The history of steel plate shear walls has been known for 3 or 4 decades. This innovative lateral load resisting system has not only been studied as resarch but also has found applications area at the buildings in Japan, the United States of America and Canada (Ericksen, Sabelli, 2008).

THE DEFINITION OF THE STEEL PLATE SHEAR WALLS

This type of shear wall system is an innovative system of lateral load carrying systems. It is easily applicable in new buildings and can be used to retrofit old buildings instead of other traditional lateral load resisting systems such as reinforced concrete walls and steel braces. Steel plate shear walls consist of boundary columns, beams and an infill plate which are connected to the boundary frame. The experimental studies have been done mostly under cyclic lateral loading procedure with one bay and one, two, three or four-storey specimens. In some experimental studies shake table tests have also been used.

The mechanism of steel plate shear walls is classified to the resistence of buckling ability of the infill plates. The infill plate in stiffened steel plate shear walls resists full shear to be reached without buckling. The plate can be stiffened to prevent buckling of the infill plate in steel plate shear walls under cycling loading. Takahashi et al. (1973) recommended that stiffened steel plate shear walls be designed so that the shear panel

does not buckle elastically. They also recommended that inelastic buckling should be occur and be limited with stiffeners between stiffening elements. Stiffening increases the properties and effectiveness of the steel plate shear walls what the researchers want but it should be economical solution at the same time. Further researches that have conducted after a few years showed that the same benefits can be taken from unstiffened steel plate shear walls. The researches have been conducted in North America mostly in the unstiffened manner. The Japanese designers have used heavily stiffened infill plates in plate shear walls but most of the designers and researchers in North America used moderately stiffened or unstiffened plates. (Behbahanifard, 2003, Driver, 1997, Rezai, 1999).

Under lateral loading, out of plane buckling does not represent the failure of the load carrying mechanism. Under cyclic lateral loading procedure, the load resisting mechanism of the system changes from in-plane shear to diagonal tension after buckling. The shear stresses develop during the increase of the loads until the inclination of the plate and creating tension lines transfer the loads.

The idea of unstiffened steel plate shear walls came from Wagner (1931), who showed that thin aluminium shear plate used in aircrafts and supported by relatively stiff frame members develop a diagonal tension field after buckling. This idea and perspective of lateral load carrying mechanism was named "pure" tension field theory. In a plate, which supported relatively stiff boundary members, the tension field action creates additional shear capacity of the thin plate (Behbahanifard, 2003, Driver, 1997).

Even using steel plate shear wall systems is not very old, they have been recognized in some codes. This lateral load carrying system was used in some buildings in Japan and North America. Designers chose this system because of its advantages, which are explained in this article. Steel plate shear wall has been recognized by the National Building Code of Canada and Canadian Steel Design Standard since 1994. Similar provisions were included in FEMA 450 (NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures) in 2004. In 2005, the special plate shear wall was added to the AISC Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341- 05 (Ericksen, Sabelli, 2008).

There is an easy explanation of steel plate shear wall in Rezai (1999) "The steel plate system resembles a vertical plate girder where theoretical buckling strength of the plate panels is negligible and lateral loads are carried through post-buckling strength of the plate panels in combination with the frame action of the surrounding beams and columns." (Rezai, 1999).

The advantages of the steel plate shear panels have been compared below with the moment resistant frames, steel braces or reinforced concrete shear walls. The use of steel plate shear walls is more advantegeous than other lateral load carrying systems as:

- To use less steel
- To need smaller foundation dimensions
- To have more floor space (Rezai, 1999).

Another advantage of steel plate shear walls in comparison with steel braced lateral load resisting system is to show relatively stable and ductile behavior under cycling loading (Driver, 1997, Rezai, 1999). This ductile behavior of steel plate shear walls resulted from action between boundary members where it develops a diagonal tension field after buckling. Post-buckling behavior of infill plates provides the redundancy of lateral drift and acts as a good energy absorbing system.

The economical comparison of using reinforced concrete shear walls and steel plate shear walls, as lateral load carrying element, was reported after a design study in Vancouver,

Canada by Timler in 1998. Timler (1998) studied with an eight-storey office building. The advantages of steel plate shear walls are written below:

- The total cost of innovative system is 74 % of the traditional system
- The cost of the foundation for an innovative system is 63 % of the traditional system's foundation cost
- To compare the total weight of the building, the thickness of the infill steel plates at the bottom story is 8mm but the thickness of the reinforced concrete shear walls is selected 400mm.

It is clear to see that available floor space increases (Rezai, 1999, Timler, 1998).

At the beginning of 1980's, researchers started using the idea of utilizing the post-buckling strength of steel plates. One of the reasons to study this system is the difficulty of the application of the stiffened plate shear walls. A number of studies were performed in the early 1980's in Canada to investigate the idea of utilizing the post-buckling strength of steel infill plate. Thornburn et al.(1983), Timler and Kulak(1983), Tromposch and Kulak (1987) are some of these research reports. After these studies, an explanation of analysis and design of thin unstiffened steel plate shear walls was described in Canada's National Standart on Limit States Design of Steel Structures, CAN/CSA-S16. 1-94 (1994).

Some of the conducted theoretical and experimental studies on steel plate shear walls is shortly presented in Driver et al.(1997), Rezai (1999) and Behbahanifard(2003).

LITERATURE REVIEW OF STEEL PLATE SHEAR WALLS TAKAHASHI ET AL. (1973)

The first extensive research, design and test programme on the steel plate shear walls was conducted in the early 1970's by Japanese researchers. The report published by Takahashi et al. (1973) is the first of such studies as known. The objective of the studies was to investigate an alternative system to the traditional reinforced concrete shear walls as lateral load-carrying system. Takahashi et al. (1973) studied on two different test series. The first one is twelve one-storey and one bay specimen with varying plate thickness from 2.3 mm to 4.5 mm. The dimensions of the specimens are 2100mm in width and 900mm in height. The panels are connected to the stiff boundary members with high-strength bolts. One of these 12 panels was the control specimen which was without any stiffeners. The researchers concluded that the stiffened plates showed excellent behaviour and absorbed more energy than did the control specimen under cycling loading. The second one consisted of two full-scale tests which were taken from the design of a 32-storey building. One of these two specimens differed from the other with openings and the specimen with an opening was made of 6mm steel plate while the other one was 4.5mm. The specimens demonstrated very stable and ductile behavior and a large energy dissipation capacity. The difference between the behavior of unstiffened and heavily stiffened steel plate shear walls are shown in hysteretic curves below. More information can be obtain from Takahashi et al. (1973) (Rezai, 1999, Takahashi, Takemoto, Takeda and Takagi, 1973).

MIMURA AND AKIYANA (1977)

Mimura and Akiyana (1977) developed an expression for predicting the monotonic and cyclic behaviour of unstiffened steel plate shear plates that buckle prior to reaching their shear yield strength. The idea of utilizing the post-buckling strength of the infill plate was used the first time by Mimura and Akiyana. The monotonic behaviour of plate shear wall was superimposed from contribution of the behaviour of the infill plate and the boundary frame separately. The behaviour of the plate was taken from classical elastic plate buckling theory. If loads are beyond the elastic plate buckling load, the panel shear is

assumed to be resisted by diagonal tension field action. The assumption of this idea was taken from the derivation provided by Wagner (1931). The total load-deflection curves was obtained from the sum of the contributions of the infill plate and moment frame. Mimura and Akiyana have developed a theoretical model to show hysteretic behavior of unstiffened steel plate shear walls. This hysteretic curve was the basis of the models that a few years later were proposed by the researchers at the University of Alberta (Rezai, 1999).

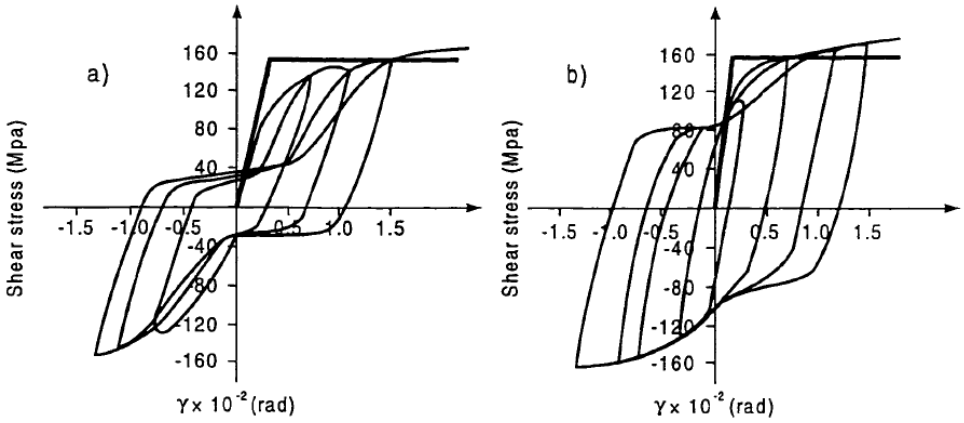

Fig. 1. The hysteretic curves of unstiffened (a) and heavily stiffened steel plate shear walls (Takahashi et al, 1973)) (Takahashi, Takemoto, Takeda and Takagi, 1973).

AGELIDIS AND MANSELL (1982)

The aim of this study is to calculate the costs and to show the feasibility of steel plate shear walls. For this reason they have studied the analyzing and designing of a 20-storey office building using steel plate shear walls in the service core. The system analyzed both manually and using a finite element computer program. The shear walls that were used in the service core were vertically stiffened only. The authors concluded that even though the cost of the steel core was more expensive than the concrete core, there could be some other benefits of this system which are explained in their report. Because of the high estimated cost of heavily stiffened steel plate shear walls, the system was not attractive to the designers until utilizing post-buckling strength to reduce the total cost of the steel plate shear walls (Agelidis and Mansell, 1982).

THORBURN, KULAK AND MONTGOMERY (1983)

Thorburn et al. (1983) developed an analytical model based on the theory of a pure diagonal tension field which are developed by Wagner (1931). This analytical model was developed for unstiffened steel plate shear walls. In this method the shear resistance of the panel prior to buckling was neglected and the idea of utilizing the post-buckling strength of the infill plate with the tension field was the main load carrying system. The strip model can be explained as series of parallel inclined members. Each of these members is pin-ended, tension only members. In this model the tension fields are idealized as strips. Each strip was idealized by an area equal to the width of strips multiplied by plate thickness (Thorburn and Kulak, 1983).

Thorburn et al. (1983) proposed an inclination angle to the strip model as in the following

$$\tan\alpha = \sqrt[4]{\frac{1+\frac{Lt_p}{2A_c}}{1+\frac{ht_p}{A_b}}}$$

Where, α: inclination angle

L: width

tp: plate thickness

Ab: cross-sectional area of beam

Ac: cross-sectional area of column

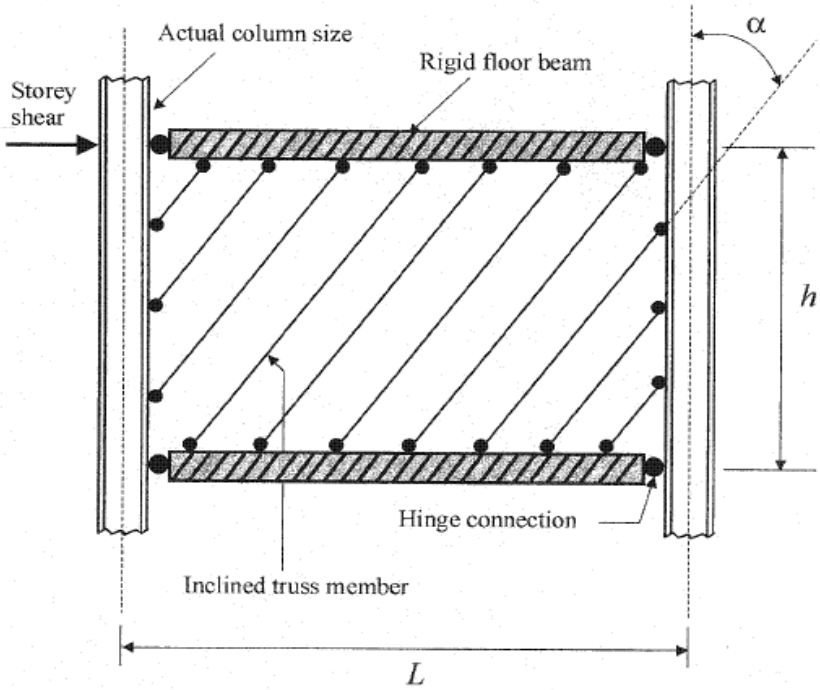

Fig. 2. Strip model improved by Thornburn and Kulak (1983) (Behbahanifard, 2003, Agelidis and Mansell, 1982).

TIMLER AND KULAK (1983)

Timler and Kulak (1983) studied a large scale, single storey steel plate shear wall specimen under cycling loading. The main aim of this study was to investigate the strip model developed by Thorburn et al. (1983). The thickness of the plate, used in specimen, was 5mm. The inclination angles of the tension fields, obtained from the strain gauges, were between 44° and 53°. The calculated value in the formula of Thorburn et al. (1983) was 51°. At the conlusion of the report, the strip model, developed by Thorburn et al. (1983), was found satisfactorily. Timler and Kulak (1983) revised the formula, developed by Thorburn et al. (1983), during their study. They added the bending effects of the boundary columns as following: (Timler and Kulak, 1983).

Where, α: inclination angle

L: width

tp: plate thickness

Ab: cross-sectional area of beam

Ac: cross-sectional area of column

Ic: moment of inertia of columns

TROMPOSCH AND KULAK (1987)

The authors conducted a large scale single storey steel plate shear wall test similar to the one tested by Timler and Kulak (1983). The difference was the typical bolted shear beam to column connections, thinner infill plates, stiffer beams and applied gravity loads. Specimens were subjected to lateral quasi-static loading at the top of the column. Up to the ultimate load, %67 of the loading device, cycling loading applied to the specimen. It was equal to 28 cycles and 17 mm maximum deflection. After this loading value, monotonic loading was applied to the specimen until the ultimate load carrying capacity of the specimen. At the end of the test, the maximum deflection was 71 mm. The behaviour of the system was very ductile and stable. The hysteretic loops were severly pinched because of using thinner plate and a flexible frame. To compare with the analytical results, researchers calculated a pushover analysis using the strip model formula developed by Thorburn et al. (1983). There was a good agreement between the hysteretic loops of pushover analysis and the results obtained from test. Researchers developed a new hysteretic behaviour for unstiffened steel plate shear walls similar to the one developed by Mimura and Akiyana (1977). One of the important conclusions of this test was that the rigid beam to column connections increase the energy absorption capacity of the system (Tromposch, Kulak, 1987).

SABOURI-GHOMI AND ROBERTS (1991, 1992)

Sabouri-Ghomi and Roberts (1991) studied a new dynamic analysis method for steel plate shear walls in England. The shear wall system idealized as a continuous cantilever vertical beam. The story masses and dynamic forces were assigned to each story with the time stepping technique. An improved formulation, which included shear and bending behaviour of shear wall, developed in their latter studies (Sabouri-Ghomi and Roberts, 1992). This study includes a series of small scale quasi-static tests to obtain the effects of the material nonlinearities in the analysis method. Six different plates, ranging from 0.54 mm to 1.23 mm thickness and 300 mm x 300 mm or 300 mm x 450 mm in size, were used in test. The thinnest plate was aluminium alloy and the others were steel. The boundary frame was pinned at the corners. All panels were subjected to tensile and compressive loading to two opposite diagonal corners. The hysteretic behaviour of the shear wall obtained from superimposed of panel and boundary members with the assuming plastic hinges at the top and bottom of the columns. Detailed figures and explaination can be found in this study. The authors developed their analytical model on five-storey steel plate shear wall but not validated with any experimental test results. (Sabori-Ghomi and Roberts, 1991, 1992).

YAMADA (1992)

Yamada (1992) conducted two small-scale (1/5) one-storey steel plate shear wall specimens in Japan. The dimensions were 600 mm in height and 1200 mm in width with a thickness of 1.2 mm and 2.3 mm. The infill plate welded to the rigid composite boundary members. The loading was applied monotonically in the horizontal direction. The lateral load resisting mechanism had diagonal tension fields after buckling such as the other studies on steel plate shear walls (Yamada, 1992).

CACCESE, ELGAALY AND CHEN (1993)

To investigate the effectiveness of post-buckling strength of the using steel infill plates, Caccese et al. (1993) conducted a series of quarter-scale, single-bay and three-storey unstiffened steel plate shear wall. Three of the specimens were built in moment resisting beam to column connections with 0.076mm, 1.90 mm and 2.66 mm infill plate thickness. Two of the specimens were built in shear connections with 0.076 mm and 1.90 mm plate thickness. Specimens were 1250 mm in width and 830 mm in height. They investigated the beam to column connection and width to thickness (slenderness) ratio under cycyling quasi-static loading. Displacement controlled loading protocol was used, which proposed

in ATC-24 (Applied Technology Council, 1992). Cycling loading was applied to the top of the shear wall until the maximum displacement of %2 drift at the top of the shear walls was gradually increased in eight steps. Each loading cycle was applied three times and specimens were subjected to a total of 24 cycles. At the end of the cycyles, the specimens were pulled monotonically until the displacement limit of the actuator. The results showed that the existence of the steel infill plate in a steel frame demonstrates the significant energy absorption capacity and initial stiffness to the system.

The authors concluded that the type of the beam to column connections had a minor effect on the overall behaviour of the hysteretic curves of thin steel plate shear walls. "This conclusion was discussed by Kulak et al. (1994) who pointed out that because of different plate thickness and material properties among the test specimens, plus a failed weld in one of the tests, made a direct comparison of the test impossible. Their assessment of the effect of connection type was therefore rejected." (Behbahanifard, 2003). The authors also concluded that the lateral load carrying mechanism was initiated by post buckling tension yielding and governed by plastic hinges at the bottom of the columns.

The researchers developed two analytical models after their experimental studies (Elgaaly et al. (1993)). The first one was a nonlinear finite element model which has material and geometric nonlinearities and the second one was the strip model which was developed by Thournbun et al. (1983). The stiffness and load carrying capasity results of the finite element model were significantly greater than to the obtained from experimental tests. The researchers concluded that after the studies on strip model, the inclination angle of tension fields had a minor effect on the capacity of the steel plate shear wall system [(Behbahanifard, 2003, Driver, 1997, Rezai, 1999, Caccese, Elgaaly and Chen, 1993).

XUE AND LUE (1994)

Xue and Lue (1994) examined a numerical investigation of different configurations of beam to column connections and infill plate to boundary member connections on steel plate shear walls. A three-bay twelve-storey frame with steel plate shear walls in the interior bay and moment resisting beam to column connections in the exterior bays were used for their studies. The frames for the interior bay were either fixed or shear connection type and the infill plates were connected either only girders or all the boundary members. For comparison, the lower bound and upper bound solutions were analysed. A frame with all fixed connections and with infill plate connected to all boundary members and prevented from buckling was used as the upper bound solution. A frame with shear beam to column connections in the middle frame with no infill plate was used as a lower bound solution. The dimensions of the exterior bays were 9144 mm and interior bay was 4572 mm. The first story was 3658 mm and the remainders were 3658 mm in height. The infill plate thickness were 2.8 mm in bottom four stories, 2.4 mm in the middle and 2.2 mm in the top four stories. The aim of the studies was to investigate the lateral stiffness of the system for different arrangements with the drift control. The total six frame-wall system which is explained above, was modeled using finite element method. They used UBC (1988) loading protocol to distrubute at each floor. They reported that as a conclusion, the stiffness of the structure increases significantly with the using infill plates. The effect of the beam to column connection type to the overall lateral stiffness is negligible. Using the shear beam to column connection at the shear wall and infill plates connected to the beams only showed the best performance.

The researchers (Xue and Lue, 1994b) also examined a parametric study to investigate the effect of the width to thickness ratio of the panel and width to height (aspect) ratio on the load-deformation behavior of the steel plate shear walls. The researchers reported that, in conclusion, the width to thickness ratio of the panel had no effect on the behaviour of the system while the width to height (aspect) ratio had a significant effect

on the load-deformation behavior of the steel plate shear wall (Xu and Lu, 1994a, Xu and Lu, 1994b).

DRIVER (1997)

A quasi-static cycling test on a half scale four storey unstiffened steel plate shear wall system was analysed by Driver at al. (1997). The objective of this study was to provide information about the behaviour of multi storey steel plate shear walls under severe cycling loading idealized with severe earthquake. The test specimen consists of four-storey, one-bay moment-resisting frame which infilled with steel plates. The plates were 4.8 mm for the first and second stories and 3.4 mm for the 3rd and 4th stories which were welded to the boundary columns and beams with fish plates. The first story was 1927 mm, the middle stories were 1829 mm and the 4th story was 1836 mm in height. The center-to-center column spacing was 3050 mm. The loading and deflection protocol were selected from the ATC recommendations (1992). The test specimen illustrated below was taken from the author's report.

The strip model, developed by Thournburn et al. (1983), was used to predict the behaviour of cycling testing. The author concluded that the nonlinear analysis was found to be in a good agreement with the experimental test results. 20 cycles of the total 30 loading cycles applied to the specimen were in the inelastic range.

The researcher also concluded that the hysteretic behaviour, shown in the figure below, indicates that the shear wall configuration tested possesses an extremely high degree of ductility. This quality is generally considered to be one of the most important for structures subjected to seismic loading. Even at the end of the test, the main ductile component of the shear wall, the infill panel, was still able to carry shears nearly equal to those resisted at the ultimate load level. The hysteretic curves were also very stable throughout the response and they did not show any sudden drops in capacity. They also reported that, the dissipated energy during the cycling loading was significantly greater than that obtained by similar steel plate shear walls but with shear type beam to column connections (Tromposch and Kulak 1987). Based on the result of this large scale four-storey test, it is conluded that the steel plate shear wall configuration tested represents an excellent lateral load resisting system for seismic loading (Driver, 1997).

The best predictions of internal member forces were obtained from the finite element model when the effect of geometric nonlinearities were included. Furthermore, the author recommended that the finite element model can be used by structural design offices to design steel plate shear wall systems.

AOYAMA AND YAMAMOTO (1984)

They conducted seven tests to investigate the behaviour of the stiffened steel infill plate which was connected to the reinforced concrete frame. The infill plates were connected to the reinforced concrete frame with studs and anchors. One bay, one storey and one-third scale specimens were used under cycling lateral loading with applied axial loads on columns. The effects of openings, in steel infill plates, on the behaviour of the system were also investigated. Generally, the specimens failed after loading up to the maximum deflection of h/50.

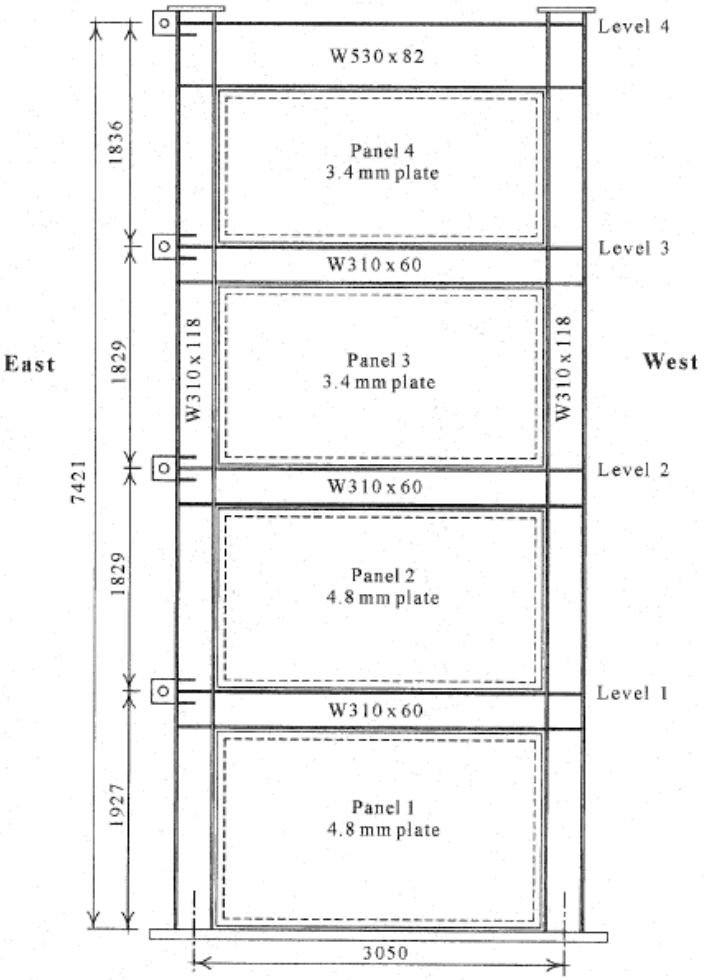

Fig. 3. Half scale four storey steel plate shear wall tested by Driver (1997) (Agelidis and Mansell, 1982).

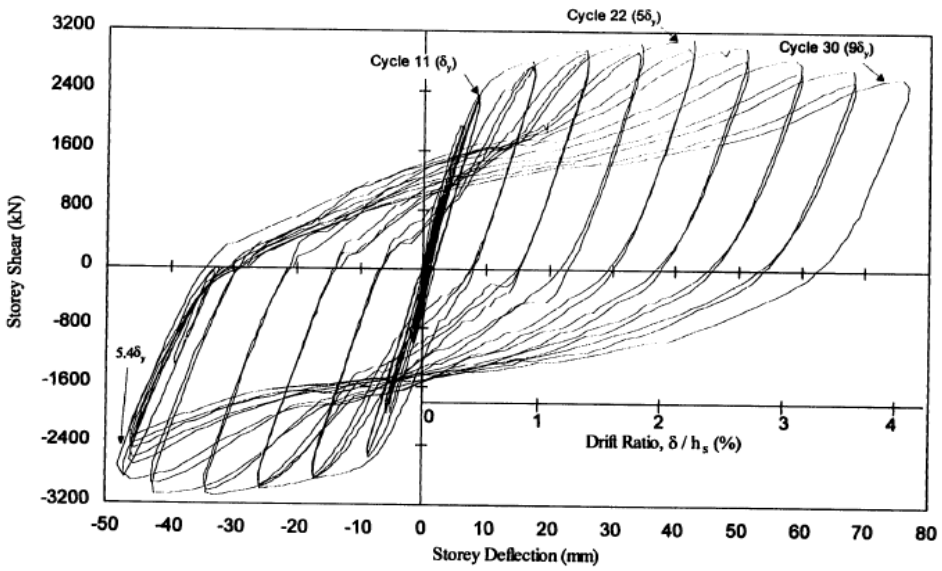

Fig. 4. Storey shear versus storey deflection of SPSW tested by Driver (1997) (Agelidis and Mansell, 1982).

The authors concluded that the infill plate helped the axial load carrying of concrete columns even if they formed shear cracks under loading. If a rectangular opening of %12 of the infill plate area exists, the effect of the location was negligible. The installing of infill steel plates to the existing reinforced concrete frames strengthened and stiffened the frame but ductility was limited with the failure of the anchors interface of the concrete frame and steel infill plate (Behbahanifard, 2003, Driver, 1997, Rezai, 1999).

ELGAALY AND LIU (1997)

They studied the orientation of strips of the infill plate under lateral cyclic loading. The earlier research of Elgaally et al. (1993) conluded that the distribution of tension fields in the infill plate was not distributed uniformly. Tension strips started yielding near the boundary members. The authors investigated, the concept of strip-gusset elements to obtain a uniform distribution of stresses on each strip. The gussets, which were named as a shear zone, are formed near the boundary members to yield under shear stress before buckling (Rezai, 1999, Elgaaly and Liu, 1997).

KULAK, KENNEDY, DRIVER AND MEDHEKAR (1999)

The authors studied a one-bay eight-storey unstiffened shear wall analytically. Their aim was to investigate the seismic performance of the system according to the codes and commonly used methods. They used the design base shear and load distribution protocol of National Building Code Of Canada (NBCC, 1995). The strip model proposed by Thournburn et al. (1983) was used to design steel plate shear wall. They studied on shear wall which had 8.0 m in width and 4.5 m for first story and the remained 7 stories were 3.6 m in height. The panel thickness of 4.8 mm used for each panel even the calculated thickness ranged from 3.33 mm for first panel to 0.66 mm for the top panel. The results were satisfactorily greater than the values recommended in code. A pushover analysis showed that the shear wall could resist nearly two times the code recommended base shear. The use of significantly greater infill plate thickness resulted in this over strength of the wall. They also conducted a nonlinear dynamic time history analysis using scaled eartquake records. The maximum interstory drift obtained from analysis was 0.009 while specified value of NBCC, 1995 was 0.02. The maximum shear was up to 2.97 times greater than the prescribed code values. The system showed a large energy absorption capacity reserve (Kulak, Kennedy, Driver, and Medhekar, 1999).

REZAI (1999)

To compare the cyclic behaviour of quasi static test and more realistic shake table test, researcher studied two small scale unstiffened steel plate shear wall specimens. The specimens were two one bay one storey and two one bay four storey and quarter scale. The centre-to-centre column spacing was 920 mm and the story height was 900 mm each. Moment resisting frame connections were used in the shear wall. The thickness of the infill plates was 1.5 mm for each panel and welded to the boundary members with welded fish plates such as Driver’s specimen (1997). Different time history types were applied to the specimen which studied with the shake table. The shake table test was governed by the limited capacity of the shake table. The single-storey specimens behaved very stiffly under cycling loading, compared to the unfilled frame and showed very good ductility, energy dissipating capacity and good stability under large deformations up to the ultimate loading. The limitation of the shake table prevented obtaining further inelastic response from the plate shear wall. Even under such limited conditions, researcher obtained and reported important information about the behaviour of steel plate shear walls under intense earthquake site-recorded ground motions. The researcher also studied the strip model, improved by Thournburn et al. (1983) and analytical finite element method to predict and compare the test results (Rezai, 1999).

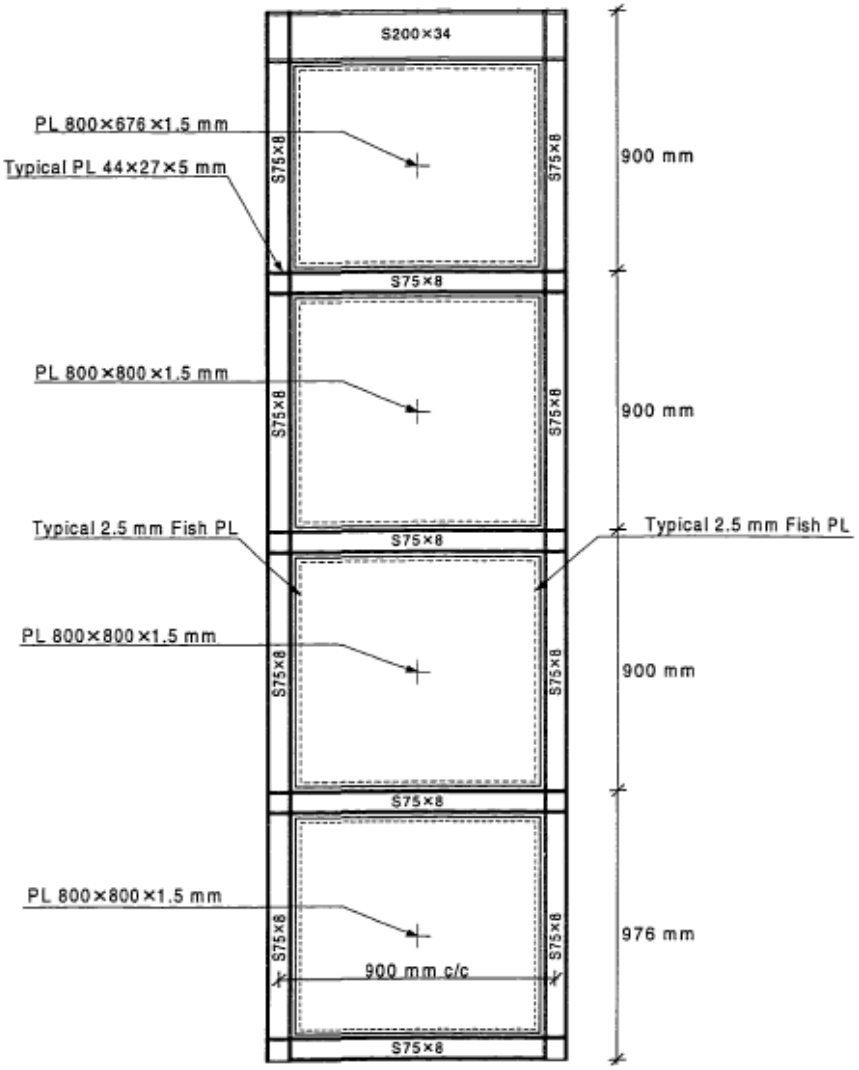

Fig. 5. Quarter scale four storey steel plate shear wall tested by Rezai (1999).

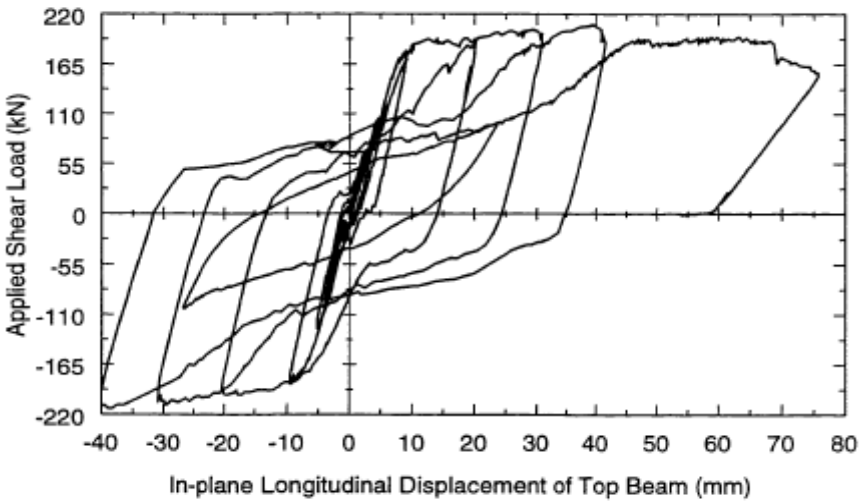

Fig. 6. Hysteretic curves of first single storey of SPSW tested by Rezai (1999).

LUBEL, PRION, VENTURA, REZAI (2000)

Lubel et al. (2000) studied a series of experimental and analytical investigations of single and four storey, one bay unstiffened steel plate shear walls similar to that conducted by Rezai (1999). The researchers paid attention to the local deformations of the columns. The inward column deformation resulted from plastic hinges at the top and bottom of the columns. This result was discussed by Montgomery and Medhekar (2001). They believed that the columns had inadequate stiffness and unusual geometric characteristics (Behbahanifard, 2003, Lubell, Prion, Ventura, and Rezai, 2000).

The researchers concluded that the design guidelines (Canadian Steel Design Standard CSA-S16-01;CSA, 2001) may not be directly applicable to some steel plate shear walls. They also criticized the provisions in some cases in the conclusion of their report (Lubell, Prion, Ventura, and Rezai, 2000).

ASTANEH-ASL AND ZHAO (2002)

Two half scale tests were conducted by Astaneh-Asl and Zhao (2002) to investigate the cyclic behaviour of a different style steel plate shear wall system developed by Skilling Ward Magnusson Barkshire of Seattle. The system consist of unstiffened steel plate shear wall and large concrete filled steel tubes. The steel tubes which filled with high strength concrete, were used for the exterior columns to carry the axial load and contribute to the shear resistence. The primarily lateral load resisting system was an unstiffened steel plate shear wall. The system behaved in a ductile manner in both specimens and absorbed up to the ultimate loading. The first specimen resisted a total of 79 cycles and 39 of them were in the inelastic range. The system was elastic up to the % 0.7 interstory drift. At the % 2.2 of interstory drift local buckling occurred in the interior columns. The specimen failed at the % 3.3 interstory drift and 4000kN base shear. The second specimen resisted a total of 29 cycles and 15 of them were in the inelastic range. The system was elastic up to the % 0.7 interstory drift. At the % 2.2 of interstory drift, the base shear reached 5451 kN and test was terminated. The specimens showed good ductility and good energy absorption capacity and concrete filled steel tubes remained elastic during the tests in both specimens (Astaneh-Asl, Zhao).

The research on unstiffened steel plate shear walls demonstrated the effectiveness of the system as a lateral load resisting system. The distinct advantages of the system are enhanced stiffness, strength and ductility, stable hystereses and a large capacity for plastic energy absorption. The system is also believed to be more economical than the equivalent reinforced concrete shear wall (Timler and Ventura, 1999) (Rezai, 1999).

MOHAMMAD R. BEHBAHANIFARD (2003)

The test specimen was the upper three storeys of the four-storey steel plate shear wall which tested by Driver et al. (1997). Most of the damage in the specimen tested by Driver was concentrated on the bottom strorey. Even the infill plate of the second storey of wall buckled and deformed plastically, the top three storeys remained without any significant damage. The first storey was removed with its beam and welded to the 90 mm thick base plate to provide the test specimen explained below. This specimen was tested under quasi static cycling loading with axial loading applied to the columns. Fourteen from a total of twentyfour cycles of loading were in the inelastic range. The author concluded that the specimen showed high initial stiffness, excellent ductility and energy absorption capacity and stable hysteretic loops.)

To estimate the shear capacity and the behaviour of steel plate shear wall specimen, a finite element analysis of the specimen was modeled. The 250 mm stroke jacks (125 mm limit for reversed loading) were used at the first and the second floors and a 380 mm stroke jack used at the top floor level with the capacity of 3300 kN. The applied loading protocol and the interstory drifts could be seen in the report.

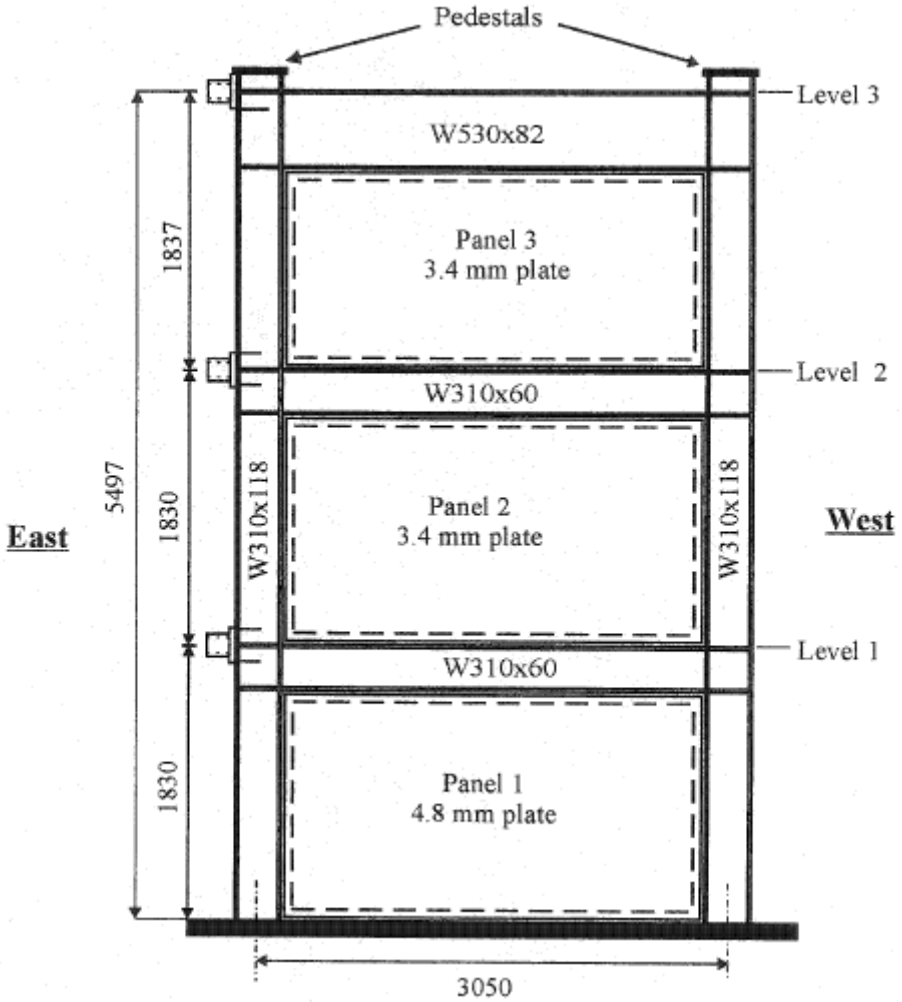

Fig. 7. Half scale three storey steel plate shear wall tested by Behbahanifard (2003)

The author discussed that, using thin plates as an infill plate, is not common but thicker plates (6 to 8 mm) are commonly used in steel plate shear walls. In order to avoid fabrication and installation problems of using thin plates, thicker plates are used more often, even a few milimeters are sufficient to absorb energy and to resist shear load. Behbahanifard (2003) recommended investigating thicker infill plates for future researchers. The researcher also recommended that the effect of the required openings and its location need to be investigated (Behbahanifard, 2003).

BERMAN, J. W., CELIK, O. C. , BRUNEAU, M. (2005)

The researchers studied a series of tests to investigate the comparison and superiority of the behaviour of braced frames and steel plate shear wall systems as a lateral load resisting system. The benefit of both systems was carried out with tens of experimental and numerical studies but there was no reported study to compare the comparison of the energy dissipating capacity, maximum displacement ductility, stiffness and some other results. Researchers studied six specimens and the aim of this study was to show the benefits of these different systems used in seismic retrofit of existing buildings. Four of these specimens were concentrically braced frames (shown below) and two of them were thin steel plate shear walls (one of them was flat the second was corrugated infill plate). All specimens were subjected to the same loading protocol (ATC-24). While the largest initial stiffness was obtained from the braced frame with steel studs, the largest ductilty and stable hysteretic curves provided by the flat steel plate shear wall specimen (Berman, Celik, Bruneau, 2005).

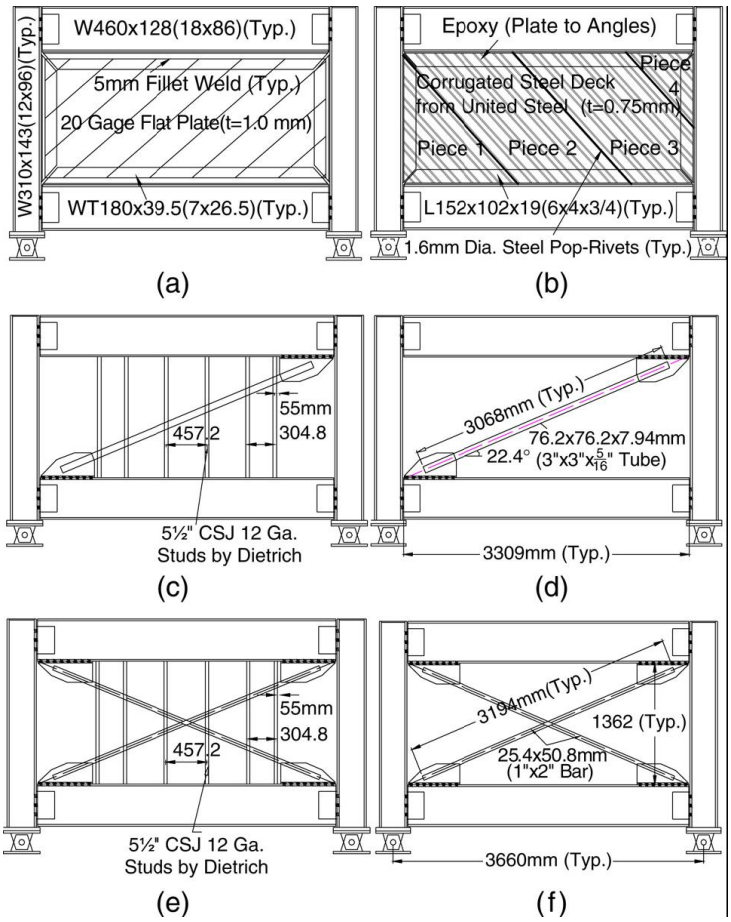

Fig. 8. Illustration of the specimens tested by Berman et al. (2005).

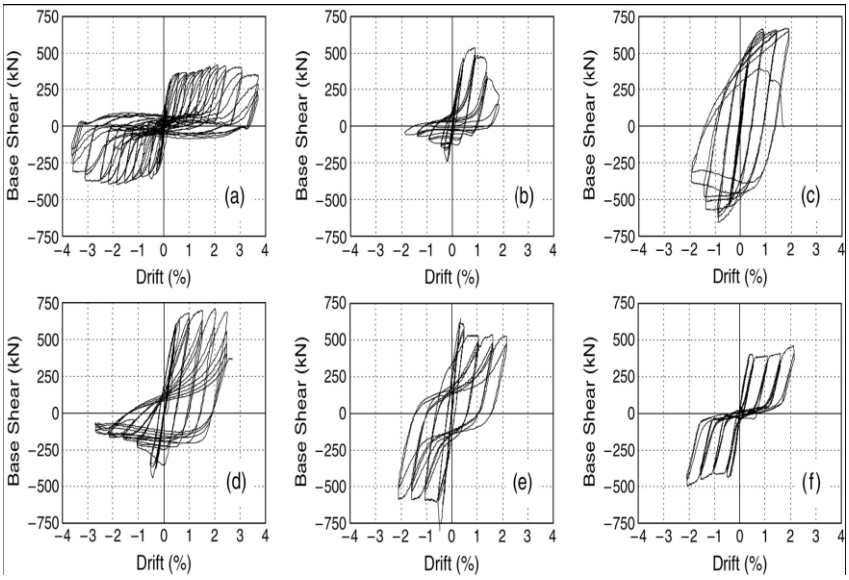

Fig. 9. Hysteretic curves of single storey specimens tested by Berman et al. (2005).

VATANSEVER, C. AND YARDIMCI, N. (2007)

Vatansever and Yardimci (2007) conducted numerical models for one bay single storey steel plate shear wall which were tested by Berman et al. (2005). The dimensions of the specimen were 3660 mm centre to centre column spacing and 1830 mm in height. 0.98 mm steel infill plate welded through fish plates. Vatansever and Yardimci (2007)

investigated numerical models (strip model and finite element model) using different commercial analytical computer programs. The load versus deflection curves were reported for numerical models and the earlier test results of Berman et al. (2005) (Vatansever, Yardımcı, 2005).

CONCLUSIONS

Every new research used the conlusions of reported studies of steel plate shear walls. For the future researchers it is especially important to read recent research in this subject. Availibility of using high speed computer programs make the analytical analyses easier. Research on steel plate shear walls has shown the effectiveness of the system as a lateral load resisting system with its large energy absorption capacity, ductility and initial stiffness. Researchers have also believed and demonstrated to be more economical than the traditional shear wall systems. After such valuable research, this innovative shear wall system should be used in high risk seismic zones by the designers.

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INTERNATIONAL EARTHQUAKE SYMPOSIUM AUGUST 17 – 19 KOCAELİ 2009

VARIATIONS ON SEISMIC RESPONSE VIA VISCOUS DAMPER PLACEMENT IN A FIVE-STOREY PLANAR BUILDING STRUCTURE

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Key Words: Viscous dampers, damper placement, seismic response, passive control

ABSTRACT In this study, without using any optimization method, a five-storey main structural model planar building, and 15 damper distributed sub-models are constructed and analyzed to understand the best suitable damper design in terms of some structural response parameters such as storey displacements, inter-storey drifts and storey accelerations. El Centro earthquake acceleration record is used in time history analyses. For the main structural model, a total damper capacity that is the sum of damping coefficient of the added dampers is chosen. Then, a damper whose damping coefficient presents total damper capacity is passed on each storey level. In other damper designs, the total damper capacity is divided by the damper number. More than one damper is placed on each storey sequentially, and is passed on the height of the structure. The results of analyses for the five-storey planar building structure point out that proper damper distribution relatively changes in terms of different response parameters.

INTRODUCTION

According to the conventional structural design, admissible structural seismic performance is succeeded in terms of the inelastic response which appears due to the effect of increase in energy. This situation also results in an indirect effect since the existence of inelastic deformations brings softening of the structural system which changes the absolute input energy itself. In fact, the increased flexibility acts as a filter which reflects a portion of the earthquake energy. It has been a common application to design structural systems, which are affected by a design level earthquake, for force and deformations. In current design practice, structural elements are allowed to yield under design force to absorb the earthquake energy without collapse. To accommodate large deformations associated with yielding, the structural elements are detailed to ensure sufficient ductility.

Recent destructive seismic events (i.e.Kobe 1995, Marmara 1999 and Düzce 1999, Sumatra 2004) in all over the world assigned the importance of mitigating these hazards for the design of new structures. These events demonstrated the reality in necessity of changing the seismic code requirements. Design for strength alone is not sufficient to prevent structural damage and collapse. Hence, there is a need for active, passive and semi-active control systems which are powerful tools to increase seismic performance of buildings. An active control system needs an external source power which controls actuators that apply forces to the structure in a prescribed manner. Meanwhile, passive control does not require an external power source. Passive devices impart forces that are developed in response to the motion of structure, and destructive energy is absorbed in the passive devices. In the semi-active control, control forces or damping characteristics can be changed by batteries. This situation gives convenience to control the structural response under the ever-changing external effects. Passive energy dissipating devices have been used to improve seismic response of both new and existing buildings. These systems are based on principles such as yielding of metals, frictional sliding, and phase transformation in metals, deformation of visco-elastic fluids and solids, and fluid orificing. These control devices are preferred because of increased flexibility, increased energy

absorption capacity, serviceability, increase in seismic safety, performance requirements, better utilization of material and cost saving, and their use for rehabilitation of existing structures.

Fluid viscous damper is the one in which passive dampers are commonly used. These devices involve the use of cylindrical piston immersed in a viscous fluid. Viscous fluid dampers, widely used in aerospace and military applications, have recently been adapted for building applications (Constantinou et al., 1993) as shown in Figure-1. Characteristics of these devices, which are of primary interest in structural applications, are the linear viscous response achieved over a broad frequency range, insensitivity to temperature and compactness in comparison to stroke and output force. It dissipates energy through movement of the piston in a highly viscous fluid. If the fluid is purely viscous, then the output force of the damper is directly proportional to the velocity of the piston.

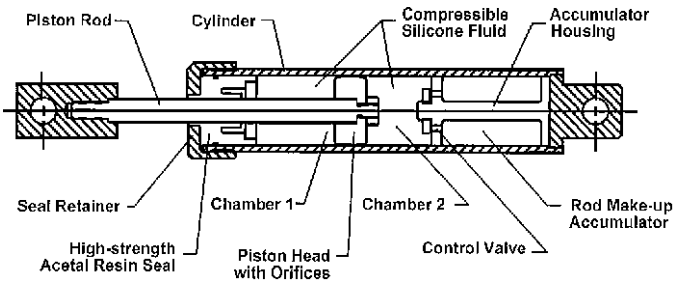

Fig. 1. Taylor devices fluid damper (Constantinou et al., 1993).

In addition, there is a need to define how many and/or where the dampers should be supplemented. There are some studies about allocation of the dampers in structures. Optimal damper distribution problem has been studied in a limited amount in the literature. The results of reducing the seismic response of multistorey shear type buildings with first storey damping are presented (Constantinou and Tadjbakhsh, 1983). The optimal damping of the system is essentially unaffected by the flexibility of the structure. A report was written about optimal damper location to the pattern that resulted in maximizing the first mode damping ratio (Ashour and Hanson, 1987). An analytical method was presented for finding the optimal positioning of a viscous damper for a linear conservative mechanical system on the basis of an energy criterion (Gürgöze and Müller, 1992) in which the calculation of optimal positioning of one damper was achieved analytically. In the same study, it was mentioned that if there was more than one damper, then the solutions should be performed numerically. An algorithm was introduced for finding the optimum sets of storey stiffness coefficients and damping coefficients of the dampers of an elastic planar shear building with viscous dampers (Tsuji and Nakamura, 1996). An analytical procedure was developed for redesign of structural systems with arbitrary damping systems for target transfer functions (Takewaki, 1997a). A systematic procedure was proposed for finding the optimal damper placement based on minimization of the sum of amplitudes of the transfer function of the interstorey drift at the undamped natural circular frequency of a structural system under a constraint on the sum of the damping coefficients of added dampers (Takewaki, 1997b). A non-monotonic optimal damper placement method was proposed for a 3-D shear building model, and a new algorithm was suggested to avoid negative damping coefficient (Takewaki, 1999a). An approach for simultaneous control of both displacement and acceleration was presented with stiffness-damping simultaneous optimization (Takewaki, 1999b). An optimal damper placement method was presented regarding the optimal damper placement for planar building frames using minimum transfer function (Takewaki, 2000). For planar building frames, a new objective function with base shear force based transfer functions was defined, and the optimal damper location and damper size were determined (Aydin et. al., 2007). A gradient-based optimization approach was used to solve optimal damper distribution problem (Mahendra and Moreschi, 2001). Genetic algorithm was used to find optimal size and location of

dissipation devices that were considered as both frequency-dependent and independent viscous and visco-elastic dampers in the structure (Mahendra and Moreschi, 2002). A simplified sequential search algorithm (SSSA) was presented for design of optimal damper configuration (Garcia and Soong, 2002). In this case, the regular building SSSA would generally lead to efficient configuration of dampers, particularly for low to medium rise buildings and for a number of dampers equal to or greater than 1.5-2 times the number of storeys. The determined damper distribution was shown to be sensitive to ground motion characteristics. Two optimal design methodologies were presented for passive added dampers based on active control theories; one approach was based on the H2 performance of the structure, whereas the other approach was based on the H∞ performance (Yang et al., 2002). Active control theories are used to determine damper allocations (Gluck et al., 1996, Agrawal and Yang, 2000a-b, Loh et al., 2000). A method was presented for optimal viscous damper design of regular as well as irregular yielding shear frames based on a global damage index under the realistic ground motion records (Lavan and Levy, 2005). In the damper optimization problem, top acceleration was defined as an objective function to find optimal damper placement (Cimellaro, 2007).

In order to obtain an optimal solution for the damper distribution, an objective function dependent on the structural response should be defined. An optimal solution upon damper placement relatively changes by the choice of objective function (Aydin et.al., 2007). There are some other relevant investigations in the literature. To understand the effect of distribution of dampers, in the first step, there is a need to know which damper placement mitigates which response of the structures.

This study helps to better understand the effects of viscous damper (VD) distribution in mitigating dynamic response of a five-storey planar building structure due to seismic excitation. It provides information regarding how many and/or where the dampers should be supplemented. An optimization method isn't used in this study. Hence, without using any optimization method, a main structural model that is a five storey planar building, and 15 damper distribution sub-models are constructed, and analyzed to understand the most suitable damper design in terms of some structural response parameters which are storey displacements, interstorey drifts and storey accelerations. El Centro NS (1940) earthquake acceleration record is used in time history analyses. For the main structural model, a total damper capacity that is the sum of damping coefficients of the added dampers is chosen. Then, a damper whose damping coefficient presents total damper capacity is passed on each storey level. In other damper designs, the total damper capacity is divided by the damper number. More than one damper is placed on each storey sequentially and is passed on the height of the structure. Finally, the results of analyses are discussed.

PLANAR BUILDING STRUCTURE WITH VISCOUS DAMPERS

Consider an n storey planar building frame as shown in Figure-2. Assume that viscous dampers are added at the mid-spans of the frame. The equations of motion of a planar building with no dampers subjected to base motion can be written as

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t) + \boldsymbol{C}_{nd}\dot{\boldsymbol{u}}(t) + \boldsymbol{K}\boldsymbol{u}(t) = -\boldsymbol{M}\boldsymbol{r}\ddot{u}_g(t) \tag{1}$$

where $\boldsymbol{M}$, $\boldsymbol{C}_{nd}$ and $\boldsymbol{K}$ represent mass, inherent structural damping and structural stiffness matrices, respectively. $\boldsymbol{r}$ is the influence vector, $\ddot{u}_g(t)$ is the horizontal acceleration of ground motion while $\ddot{\boldsymbol{u}}(t), \dot{\boldsymbol{u}}(t)$ and $\boldsymbol{u}(t)$ represent acceleration, velocity and displacement vectors, respectively.

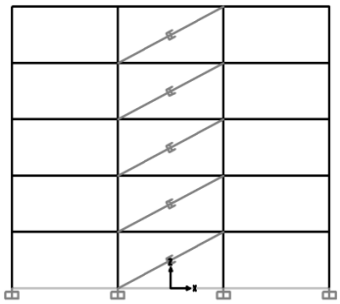

Fig. 2. n storey planar building frame with viscous dampers.

Structural damping matrix is defined as Rayleigh damping which is proportional to mass and stiffness matrices in the following form

$$\boldsymbol{C}_{nd} = \alpha \boldsymbol{M} + \beta \boldsymbol{K} \tag{2-a}$$

$$\alpha = 2\xi\omega_1 \qquad \beta = \frac{2\xi}{\omega_1} \tag{2-b}$$

where ω_1 is undamped fundamental natural frequency, ξ is known as critical damping ratio which corresponds to the lowest undamped fundamental natural frequency. An added viscous damper is also called as a manufactured damper. The relationship in between damping force and relative velocity can be written as

$$F_{md} = c_{md}\langle\dot{x}\rangle^{\eta} \qquad \langle\dot{x}\rangle^{\eta} = \begin{cases} \dot{x}(t)^{\eta} & \dot{x} \geq 0 \\ -(-\dot{x}(t))^{\eta} & \dot{x} < 0 \end{cases} \tag{3}$$

In earthquake engineering applications, a design with η=1 appears to be the most desirable. It results in essentially linear viscous behavior and Equation (3) is rewritten as

$$F_{md} = c_{md}\dot{x}(t) \tag{4}$$

When the dampers are added to the planar building structure, the equation of motion is written as

$$\boldsymbol{M}\ddot{\boldsymbol{u}}(t) + (\boldsymbol{C}_{nd} + \boldsymbol{C}_{md})\dot{\boldsymbol{u}}(t) + \boldsymbol{K}\boldsymbol{u}(t) = -\boldsymbol{M}\boldsymbol{r}\ddot{u}_g(t) \tag{5}$$

where $\boldsymbol{C}_{md}$ is called as the manufactured damping matrix. The time response of a structure with dampers subjected to an earthquake is numerically calculated by using Equation (5).

NUMERICAL EXAMPLES

To illustrate the effect of damper placement on planar steel building structures, a main model which is a 5-storey planar building structure is considered as shown in Figure-3. The model is constructed with a view to understand the structural responses in low-rise buildings.

The height of each storey is 4m and the span of each bay is 8m. Young's modulus is 2.0 10^{11} N/m^2 for both columns and beams. Shear deformations of the members are neglected. Only bending deformation in beams and both bending and axial deformations in columns are considered. The critical damping ratio of steel structures is considered as ξ=0.02 in the lowest undamped natural circular frequency.

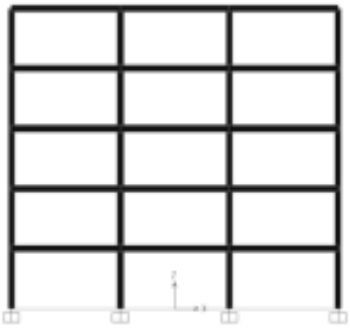

Fig. 3. 5-storey model planar building frame without viscous dampers.

A lumped mass of 51200 kg is placed on every interior node and a lumped mass of 25600 kg is placed on every exterior node. Every interior node is assumed to possess a mass moment of inertia of 5.46 10^5 kg.m^2 and every exterior node is assumed to possess a mass moment of inertia of 1.71 10^5 kg.m^2. Structural member properties are shown in Table 1.

Table 1. Member properties of 5-storey planar building model.

Model	Storey No	Beams		Columns	
		A(m^2) (10^{-4})	I(m^4) (10^{-5})	A(m^2) (10^{-4})	I(m^4) (10^{-5})
5-Storey	1st-2nd	756	383	756	383
	3rd-4th	683	353	683	353
	5th	365	205	365	305

The sum of damping coefficients of the viscous dampers is chosen as given in Table 2. Total damping coefficients of the added dampers for the model are equal to the values as shown in Table 2. It is possible to calculate the damping coefficient for a storey according to Equation (6) given below

$$c = \frac{\text{Total Damper Capacity}}{\text{Damper Number}} \tag{6}$$

Initially, a damper is placed on the first storey. A damper whose damping coefficient presents the total damping capacity is moved on to the fifth storey level afterwards for the five storey building model. Then, multi damper placement models shown in Figure 4 are investigated.

Table 2. Total damper capacity for the model.

Main Model	Sum of the damping coefficients of the added dampers (Ns/m)
5-Storey	1,47.10^7

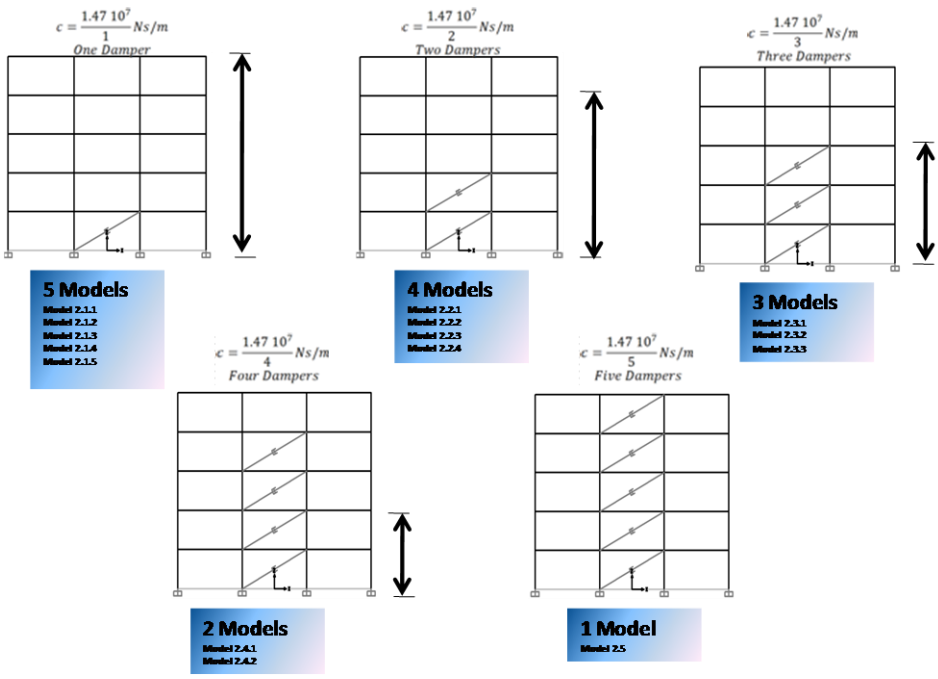

Fig. 4. Sub models for 5-storey main planar building model.

Numerical simulations were performed using the commercially available computer program SAP2000 (Computers and Structures Inc., 2000). El Centro NS (1940) ground motion record was used in the dynamic analyses.

Maximum storey displacement profiles, maximum interstorey drift profiles and maximum storey acceleration profiles are given in Figures 5-7 for all 16 models composed of one main model and 15 damper added sub models.

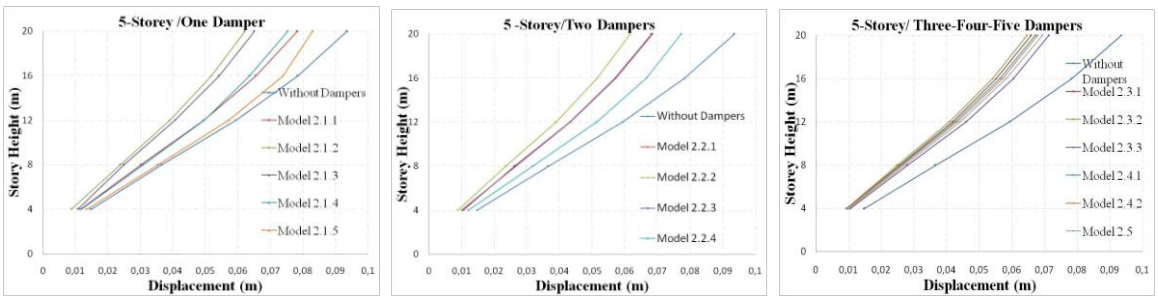

Fig. 5. Maximum storey displacement profiles for 5-storey planar building models.

It is observed that the best design in terms of maximum top displacement is Model 2.2.2 in which two dampers are placed to 2nd and 3rd storeys. Figure-5 emphasizes that dampers should be distributed to storeys where the interstorey drifts are larger compared to other storeys.

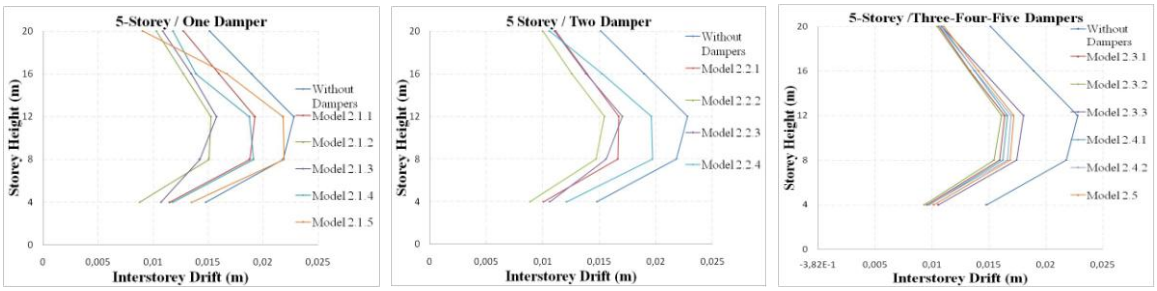

Fig. 6. Maximum interstorey drift profiles for 5-storey planar building models.

In Figure-6, it is shown that the best design in terms of maximum interstorey drift is either placement of a damper to that storey or placement of a damper to its neighbor storey for the one damper case.

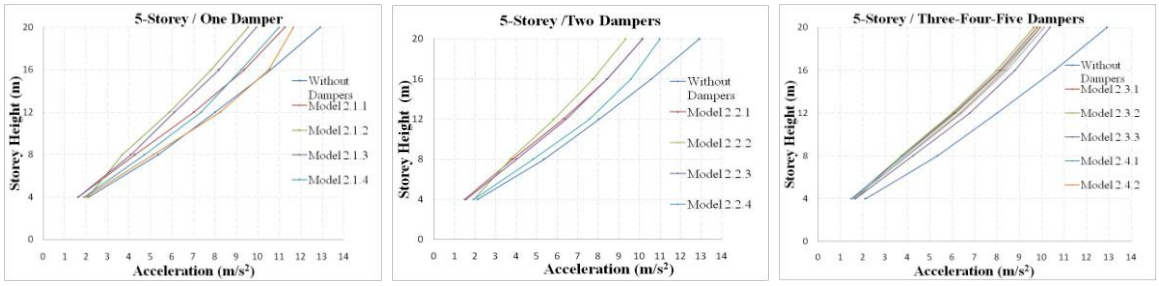

Fig. 7. Maximum storey acceleration profiles for 5-storey planar building models.

Figure-7 reveals that the best design in terms of maximum top acceleration is Model 2.2.2 in which two dampers are placed to 2nd and 3rd storeys.

DISCUSSION

As a result of this study, it is examined that the best design in terms of maximum top displacement is Model 2.2.2 in which two dampers are placed to 2nd and 3rd storeys. In addition, it is observed that dampers should be distributed to the storeys where the interstorey drifts are larger compared to other storeys. As an example, the best damper design reduces maximum top displacement by 35%. The best design in terms of maximum interstorey drift is either placement of a damper to that storey or placement of a damper to its neighboring storey. Damper designs reduce maximum interstorey drifts by 33% to 40%, accordingly. The best design in terms of maximum top acceleration is Model 2.2.2 as in the case of top displacement such that two dampers are placed to 2nd and 3rd storeys and maximum top acceleration is reduced by 28%.

CONCLUSIONS

For the 5-storey planar building model, the best damper distribution design is the same both in terms of top displacement, top acceleration and maximum interstorey drift values. For the model, the best design for top displacement occurs upon placement of dampers at storeys with high levels of interstorey drift. One damper placement is not effective; even though it reduces the interstorey drift of the storey at which it is located.

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VISUAL SEISMIC ASSESSMENT OF EXISTING REINFORCED CONCRETE BUILDINGS IN CYPRUS

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Keywords: Seismic Reliability, Physical Inspection, Reinforced Concrete,

ABSTRACT Since the existence of the Planet Earth, earthquakes occur and result by injury and death of thousands of people, structural losses and damages to the buildings and great economic losses to the countries. Although search and rescue is an important component, the most rational way to reduce the impact of the earthquakes is through proper mitigation and preparedness efforts. In the long term, from the economical view point, preparedness pays off response. Besides, the response activities never save human lives which have already been lost.

On architectural and engineering perspective, "Being Prepared" means "Design against Earthquake". Considering the architectural and engineering designs and buildings of last decades, it can be said that they have followed the new earthquake design codes and can be assumed to be Earthquake-Safe. But, what about the buildings being designed before relevant design codes? As a general statement, it has been thought that 80% of the existing buildings over the world have been built before relevant codes and needs to be seismically assessed and retrofitted in terms of seismic deficiencies.

As a part of the current dissertation study, the national standards of FEMA-United States, NRCC-Canada, EUROCODE-Europe, FIB-Europe explaining the guidelines for physical examination and seismic reliability assessment of the existing buildings have been discussed and compared for their applicability to Cyprus. It has been aimed to propose a methodology for the use of the experienced civil engineers and architects to assess and rank the seismic reliability of existing reinforced concrete buildings in Cyprus. There is a need to carry out a study specific for Cyprus on this issue, not just because Cyprus is on active region and is under the threat of earthquakes, it is also required because there are differences between the seismicity and construction systems of the countries which make them generate their own guidelines for seismic assessment.

In this study it is aimed to discuss the main principles of current national standards of USA, Canada and European Union and provide guidelines specific to Cyprus for the physical assessment of the existing reinforced concrete buildings.

INTRODUCTION

Although search and rescue is an important component, the most rational way to reduce the impact of the earthquakes is through proper mitigation and preparedness efforts. On the other hand, response activities never save human lives which have already been lost.

In order to reduce the impact of the disasters, it is therefore essential to concentrate our efforts on prevention and preparedness.

On architectural and engineering perspective, "Being Prepared" means "Design against Earthquake", which includes both New Builds and Assessment and Retrofitting of Existing Buildings. Considering the architectural and engineering designs of last decades, it can be said that they have followed the new earthquake design codes and can be assumed to be Earthquake-Reliable. But, what about the buildings being designed and built before that? Casualties and damage associated with older buildings which were designed and

constructed using codes that are not known to provide adequate seismic safety, are far worse and more than that for newer buildings which have been designed and built in accordance with more stringent code requirements. On a universal scale, the stock of older buildings built before 1980`s is believed to be 80 % of the existing building stocks in most cities. These huge numbers of building stocks are needed to be assessed in terms of seismic deficiencies and retrofitted to increase their seismic reliability.

Some main decisions were taken at the beginning of the study to be able to carry out a profound study under a narrower circumference. Although, earthquake is a global problem for almost all countries over the world, when it comes to risks of earthquake, it becomes a local problem due to seismicity, soil types and construction systems of the specific regions. So in additional to the decision of "pre-earthquake assessment", the region of Cyprus and most common construction material of the country, Reinforced Concrete and related structural systems, were decided to be studied by means of their influence to seismic deficiencies of the existing buildings.

AIMED STUDY AND ITS EXPECTED CONTRIBUTIONS TO THE LITERATURE

In the study it is aimed;

- To propose a methodology which would guide the professionals of the industry (mainly architects and civil engineers), to physically assess and rank the seismic reliability of the existing reinforced concrete buildings against earthquake.
- To ease the application of the relevant building codes.
- To tell both "what" to do and "how" to do.
- By limitations to be able to get in more detail regarding the material and system problems.

WORK CARRIED OUT TO DEVELOP THE PROPOSED METHODOLOGY AND DISCUSSION:

The Turkish Standards 2007 and Sucuoglu 2007 define 4 expected damage levels for the existing buildings as given in the Table 1. They also describe two main reliability classes by using the given damage levels and name them as High and Low Reliability Classes. The buildings determined to be in the high reliability class are accepted to be safe enough without any need of further investigation, where the ones in the low reliability class are required to be analyzed more intensely by means of analytical calculations and non destructive engineering tests. Then, it can be stated that there would be no need to analytically assess or do in situ material or strength tests to a

building which would probably have low or none damage during an earthquake. By means of rapid visual seismic assessment of buildings, it is possible to decide if the building is in low or none damage level or would need further analytical assessment studies and material tests to clarify it's seismic reliability.

Table 1. Damage levels and safety classes defined by Turkish Standards 2007 and Sucuoglu 2007.

Damage Level	Definition	Reliability Class
None	No damage or only capillary cracks on non-structural elements	HIGH
Low	Can be occupied following some repair of the	

	building's non-structural elements	
Moderate	Can be occupied following full retrofitting of the building's structural system	LOW
High	Demolished or cannot be used at any convenience	

Structural and non structural defects of reinforced concrete material (Mavilvaganam, Grattan-Bellew, Pernica, 2000,SEI/ASCE 11-99, 1999, Celikag, 2005) and current national and international pre-earthquake assessment methodologies (Sucuoglu, 2007, Institute for Research in Construction, 1993, Applied Technology Council-21, 1988, FIB-International Federation of Structural Concrete, 2003, Cyprus Civil Engineers and Architects Association, 1992, Senel, and et. al., 2007) have been studied to be able to determine performance parameters of the proposed study.

For determination of the cut off limits of reliability classes, studies of Sucuoglu 2007, Senel et. al. 2007 [10] and statistical data of 1999 Duzce Earthquake (Sucuoglu, 2007) have been examined as given in Table 2. Then, it is decided to use lower 40 % as the cut off limit for the low and high reliability classes, which would be under estimation for the current literature. Considering Cyprus building stock, which rarely exceeds 4 stories, and low seismicity risk of the region compared to Turkey, it is thought that the under estimation compared to the current literature would not create a problem in terms of seismic safety of existing buildings for the local of Cyprus.

Table 2. Comparison of current literature for determination of cut off limits of safety classes.

Damage Level	Senel et. al. 2007 [10]	Sucuoglu 2007[2]	1999 Duzce Earthquake [2]	Safety Class
None	30 %	58.5%	11.7 %	HIGH
Low	26 %		30 %	
Moderate	44 %	41.5 %	33 %	LOW
High			25 %	

Study is still carried out on existing literature and by means of case studies to finalize performance parameters, their coefficients and the cut off limit for High and Low Reliability Classes.

PROPOSED METHODOLOGY AND CASE STUDIES

The investigation forms of the proposed methodology are given in Figure 1, 2 and 3. The first two pages notes down structural deficiencies where the third page observes non structural ones in terms of their influence on seismic reliability. The coefficients of observed performance parameters of each section (structural and non structural) are multiplied among themselves and than added up together to receive the Overall Seismic Score.

The best (most reliable) and worst (least reliable) scores of the study are calculated as 2,196 and 0,120, respectively and the cut off of high and low reliability classes is calculated as 0,95 (lower 40 % of the assessments). For the beginning, 9 buildings in city

of Girne have been investigated by the proposed methodology and by two similar studies in the literature (Celikag, 2005, Institute for Research in Construction, 1993) to be able to evaluate the validity of the proposed study. In Table 3, the comparison of the results of these threefold investigations is given.

Table 3. Comparison of results of investigations of 9 buildings in city of Girne.

	USAGE	No. of STORIES	Year of Project or Construction	STRUCTURAL SCORE	NON STRUCTURAL SCORE	OVERALL SEISMIC SCORE	IRC [5]	ATC [6]
BEST SCORE	Dwelling	1 or 2	Post-Eq. Code	1,296	0,900	2,196	SEISMIC PRIORITY	
WORST SCORE	Immediate Occupancy	5 or 6	Pre-Eq. Code	0,002	0,119	0,120		
G001	Dwelling	5 or 6	Pre-Eq. Code	0,091	0,689	0,780	HIGH	HIGH
G002	Dwelling	1 or 2	Pre-Eq. Code	0,159	0,900	1,059	MED	LOW
G003	Dwelling	1 or 2	Pre-Eq. Code	0,054	0,516	0,570	MED	LOW
G004	School	1 or 2	Pre-Eq. Code	0,060	0,551	0,611	HIGH	HIGH
G005	School	1 or 2	Pre-Eq. Code	0,088	0,648	0,736	HIGH	HIGH
G006	School	1 or 2	Post-Eq. Code	0,231	0,800	1,031	MED	LOW
G007	Dwelling	5 or 6	Pre-Eq. Code	0,033	0,516	0,550	HIGH	HIGH
G008	Shopping and Dwelling	3 or 4	Post-Eq. Code	0,054	0,800	0,854	MED	HIGH
G009	Dwelling	1 or 2	Pre-Eq. Code	0,121	0,608	0,728	MED	HIGH

Fig. 4. Pictures of building G003.

Fig. 5. Pictures of buildings G009 and G008, respectively.

VISUAL SCREENING FORM FOR SEISMIC ASSESSMENT OF EXISTING REINFORCED CONCRETE BUILDINGS			
			p.1
Address:			**Item No:**
Design Year:	**Built Year:**	**Use:**	
Architectural Project:	**Date of Inspection:**		
Structural Project:	**Inspector:**		
Foundation Project:			
Sketch of Site:	**Photo:**		

STRUCTURAL ASPECTS			
Type of Structure			
	Non-Ductile Reinforced Concrete Moment Frames		1
	Concrete Shear Walls		0,9
	Dual System		0,95
	Reinforced Concrete Frame with Unreinforced Brick Infill Walls		0,8
Seismic Deficiencies of Building Structure			
	Problems with Load Path, Integrity, Redundancy and Inadequate Diaphragms	Beam Discontinuities	
		Strong Beam - Weak Column Connection	
		Connection of Beam to Beam	
		Eccentric connection of Beam to Column	0,8
		Large Spanning Slab	
		Mechanical and Electrical Shafts in Slabs	
		Large Spaces in Slabs (more than 1/3 of all slab area)	0,8
		No Beam Connection to Column	0,9
		Inadequate Stair-Slab Connection	0,9
		Non-orthogonal Axes of Structural Elements	0,8
		Existence of Split Floors	0,8
		Existence of Mezzanines	0,9
		Less Than 2 Vertical Planes of Framing or Shear Walls in Each Direction or Less Than 2 Bay for Frame	0,7

Fig. 1. Visual Screening Form, page 1.

				p.2
	Lack of Lateral Strength and Ductility .....Weak Story			0,8
	Lateral Stiffness..... Soft Story			0,8
	Building Irregularities due to Configuration Problems		Vertical Irregularity	0,7
			Horizontal Irregularity or Torsion	0,85
			Aspect Ratio (Length/Width Ratio) >5 ??	0,9
			Short Columns no is also important	0,8
			Heavy Cantilevers with beams	0,9
			Heavy Cantilevers without beams	0,8
	Pounding of Adjacent Buildings	Also for buildings with different blocks and heights	Semi-Detached Building on 1 side	0,9
			Semi-Detached Building on 2 side	1
			Neighbors buildings have different story height	0,9
			Existence of Filled Joint or Joint with Inadequate Width or No Joint at All	0,9
	Foundation Problems		Settlement	0,8
			Land Slide (Foundation on Slope)	0,8
			Existence of Basement	1,2
			Mat Foundation by project	1,2
			Continuous Foundation by project	1
			Single Foundation by project assume if no project	0,9
	Number of Stories Above Ground		1 or 2	1
			3 or 4	0,9
			5 or 6	0,8
Current Physical Condition of Building Structure				
	Deterioration of Reinforced Concrete Material That Creates Structural Problem		Active Cracks on Partition Walls	0,9
			Active Cracks on Structural Components (Beams, Columns, Shear walls or Slabs)	0,7
			Lack of Cover to Reinforcement	
			Cracks Above Ties and Reinforcement	
			Spalling Because of Corrosion of Reinforcement	0,85
			Delamination	
			Degradation	
			Cold Joints	
			Rust Stains	
			Plucked Surface	0,9
			Humidity	0,9
	Modifications, Damage and Repairs	Non-Structural Modifications	Modification to Partiton walls	0,95
		Structural Modification	Moderate	0,9
			Severe	0,8
	Visual Reinforced Concrete Material Quality	Good		1
		Moderate		0,8
		Poor		0,6
	Visual Finishing Material and Workmanship Quality	Good		1
		Moderate		0,9
		Poor		0,8
	Deterioration of Reinforced Concrete Building by Age – Time Dependent Deterioration	5-10 years old		0,9
		11-20 years old		0,85
		21-30 years old		0,8
		31-50 years old		0,7
Modification to Structural Score for Buildings Built Before 1993 Seismic Provisions				0,85
			STRUCTURAL SCORE (product of all)	

Fig. 2. Visual Screening Form, page 2.

VISUAL SCREENING FORM FOR SEISMIC ASSESSMENT OF EXISTING REINFORCED CONCRETE BUILDINGS			
			p.3
NON STRUCTURAL ASPECTS			
Falling Hazards to Life			
	Interior Non Structural Components (select at most 3)	Masonry Partitions (e.g. glass blocks, decorative stone blocks, etc.)	0,9
		Heavy Decorative Components	0,9
		Unfastened Shelves	0,9
		Large Non-Safety Glass Panels	0,9
		Blocked Corridors	0,85
		Heavy Lightning Equipment	0,9
	Exterior Non Structural Components (select at most 2)	Masonry Chimneys	0,9
		Free Standing Parapets	0,85
		Veneer, Stone or Precast wall panels	0,9
		Canopies over Exists and Walkways	0,9
Data Sufficiency			
	Intense:	Existence of architectural, structural and foundation projects and entry	1
	Moderate:	Entry to building but existence of no architectural project	0,9
	Limited:	No structural or foundation projects and no entry to the buildings	0,75
Building Importance			
	Buildings that should be occupied after an earthquake and have great importance to		0,75
	Buildings that people occupy for long hours during day time with high population		0,8
	Buildings that people occupy for short hours during day time with high population		0,85
	Buildings where high explosive and toxic materials are stored		0,75
	Other Buildings		0,9
Soil category for CYPRUS			
	Soil Category SC1	Rock of any characteristic, either shale-like or crystalline; or stiff soil conditions where the soil depth is less than 60 m and the soil types overlying rock are stable deposits of sand, gravels or stiffer clays	1
	Soil Category SC2	Deep cohesionless or stiff clay, including sites where the soil depth exceeds 60 m and the soil types overlying rock are stable deposits of sands, gravels or stiff clays. (In locations where soil properties are not known in sufficient detail to determine the soil category, or where the profile does not fit any of the three types, soil category SC2 shall be used.)	0,85
	Soil Category SC3	Soft-to-medium stiff clays and sands, characterised by 10 m or more of soft-to-medium stiff clay with or without intervening layers of sand or other cohesionless soils.	0,8
Seismicity of Region for CYPRUS			
	Seismic Zone 1	A_{max}=0.075	
	Seismic Zone 2		
	Seismic Zone 3		1
	Seismic Zone 4	A_{max}=0.100	0,75
	Seismic Zone 5	A_{max}=0.150	0,5
		NON-STRUCTURAL SCORE (product of all)	
		OVERALL SEISMIC SCORE	0
NOTES:			Detailed Evaluation Required? YES NO
	Best Score: 2,196	Limit of lower 40%:Low Safety Class	0,95
	Worst Score: 0,12		

Fig. 3. Visual Screening Form, page 3.

CONCLUSIONS

So far, 9 buildings (5 dwellings, 3 schools and 1 dwelling+shopping building) in city of Girne have been inspected by proposed methodology. It is planned to investigate hospital buildings and some other buildings also in Girne and in other cities of Cyprus, as each city locates on a region with different seismicity (Girne has low, Lefkosa has moderate and Gazi Magusa has high seismicity).

Preliminary results demonstrate that the proposed methodology give comparable seismic reliability estimations with the ones in the literature, although it also considers material, deterioration and some additional structural and nonstructural aspects.

In addition to this, proposed methodology give better estimations for some 1-2 stories buildings compared to the methods in the literature, as for buildings G003, G008 and G009.

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YIELD-CAPACITY-RISK RELATIONSHIPS FOR A RESERVOIR IN AN EARTHQUAKE

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Key Words: Reservoir, Capacity, Yield, Risk, Capacity Risk, Earthquake Risk.

ABSTRACT In the water resource systems; yield is defined as annual water amount supplied from the reservoir, capacity is defined as storage volume of reservoir and risk is defined as to obtain the same yield with lower capacity. The purpose of this study is to determine yield-capacity-risk relationship for reservoir storage in earthquake. For this purpose, firstly, the capacity values are calculated against to the various yield values of the reservoir storage. According to the data obtained from the calculations; it has been observed that capacity values are increased while the annual yield values are increasing and reduced while the annual yield values are reduced. Then, Earthquake forces acted on the reservoir in the earthquake can be reduced by changing capacity amount. Here, capacity with risk should be defined by changing the capacity amount to obtain the desired annual yield value. The desired annual yield value in the capacity's risk can be obtained from the capacity with different risk values. In this study; according to situations described as above, it has been observed that the capacity is reduced due to risk of earthquake and the desired annual yield value at the statement without earthquake can be obtained with the reduced capacity under the certain risk.

INTRODUCTION

In the water resources system stream flow has a stochastic feature. Flow has statistical feature as well as average amount. In the meantime, there has not been a relationship between elements of sequential flow series. These types of series are named as stochastic series. Changes in flow at the effort situation of taking advantage from stream flow can be changed by storage volume of dams. Active storage volume of dams is named as capacity in the water resources system that has dams. Thus, it can be said that active storage capacities of dams have stochastic feature.

Different methods that are used for designing of storage reservoir have advantages and/or disadvantages against each other (Bayazıt and Bulu, 1991). Although synthetic data have been used for designing of reservoir capacity, relation between storage reservoir characteristic as capacity-risk-yield can be used directly [5, 16].

The aim of this study is to determine yield-capacity-risk relationship for reservoir storage in earthquake. For this purpose, firstly, the capacity values are calculated against to the various yield values of the reservoir storage. According to the data obtained from the calculations; it has been observed that capacity values are increased while the annual yield values are increasing and reduced while the annual yield values are reduced. Then, Earthquake forces acted on the reservoir in the earthquake can be reduced by changing capacity amount. Here, capacity with risk should be defined by changing the capacity amount to obtain the desired annual yield value. The desired annual yield value in the risk of capacity can be obtained from the capacity with different risk values.

YIELD

Yield is defined as total water amount taken from dam as a result of arranging in the active storage reservoir of dam in a year period. There is a certain capacity for any yield. However obtaining the same yield for any yield is possible as well as obtaining different

yield from any capacity. In this situation, active storage capacity and yield of dam should be defined depending on a certain risk. Chosen capacity can provide a certain yield under a certain risk or certain yield can be obtained by a chosen capacity under a certain risk.

CAPACITY

Active storage volume of dams can be taken into consider as dam capacity. There is an annual yield for every capacity amount. There are lots of methods for determining capacity. Generally, dam capacity is determined by using relation between amount of water that arrives to dam and water amount that taken from dam. Taking into consider flood and drought situation, following steps are used for determining capacity.

1. Capacity that can provide annual yield with monthly average flow is determined depending on the predicted management strategies.

2. A certain flood/drought frequency period is chosen and by using critical term flow of this period, flood/drought control capacity is obtained.

3. Since flood control volume is discussed in wet season and drought control volume is discussed in dry season, total volume should not be achieved by sum of these but an appropriate combination.

Datas of long-term flow are important for obtaining capacity. Because observed flow is not enough long-term, requested long-term synthetic flow series can be derived by establishing their model, statistically. Thus, sufficient datas are provided for determining capacity.

RISK

In this study, risk is discussed as capacity risk (reducing capacity) and earthquake risk (occur possibility). Capacity risk is obtaining same yield with reducing capacity under certain risk. Forces that can affect dam in earthquake is decreased with decreasing in dam capacity under earthquake potential. In this situation, earthquake risk is discussed. It has been observed that capacity has been reduced and capacity risk has been increased with increasing in earthquake risk. That means, it is possible to obtain same yield with capacity risk that equal to earthquake risk.

CAPACITY RISK

When the linear relation between capacity and yield is investigated, it can be observed that there is an annual yield for every capacity value. However, at the risk capacity, it enables to obtain same yield with different capacity by reducing capacity. Capacity risk is

$$\text{Capacity Risk}=(\text{First Capacity}-\text{Last Capacity})/\text{First Capacity} \quad (1)$$

EARTHQUAKE RISK

Capacity risk is rised, when earthquake probability is increased due to the earthquake forces by reducing the reservoir capacity in an earthquake. This case is named as earthquake risk. When this risk is increased, reservoir capacity will be reduced and then, yield with reduced capacity will be worked to provide.

APPLICATION AND RESULTS

Monthly mean inflows of 10 historical years of the river with earthquake probability and annual yield (its value is 61 m^3/sec.) are given as shown in Table 1. In Table 1, value of the minimum inflow is 27 m^3/sec.

Table 1. Monthly mean inflows of 10 historical years of the river.

Months	Oct	Nov	Dec	Jan	Feb	March	April	May	June	July	August	Sep
I(m^3/s)	40	39	55	79	93	110	132	62	42	28	27	28
Days	31	30	31	30	28	31	30	31	30	31	31	30

CAPACITY-WATER HEIGHT RELATIONSHIPS

There has been a linear relationship between reservoir capacity and water height in the reservoir. When water height is increased, reservoir capacity is gone up. In Figure 1, variation of the capacity with water height in the reservoir.

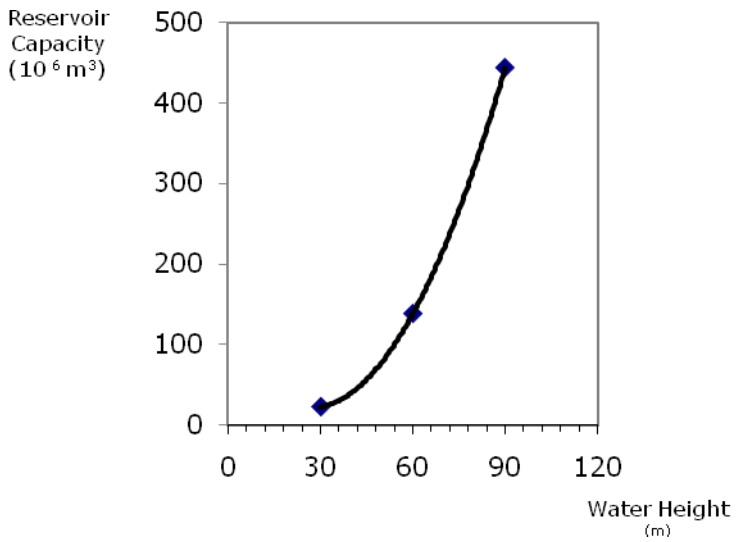

Fig. 1. Variation of the capacity with water height in the reservoir.

YIELD-CAPACITY RELATIONSHIPS

There has been a reservoir capacity to be obtained with each yield in the reservoir. Variation in Figure 2 is found by using values of the various yield and capacity and data in the Table 1. In the Figure 2, when reservoir capacity is increased, it is shown that reservoir yield was gone up.

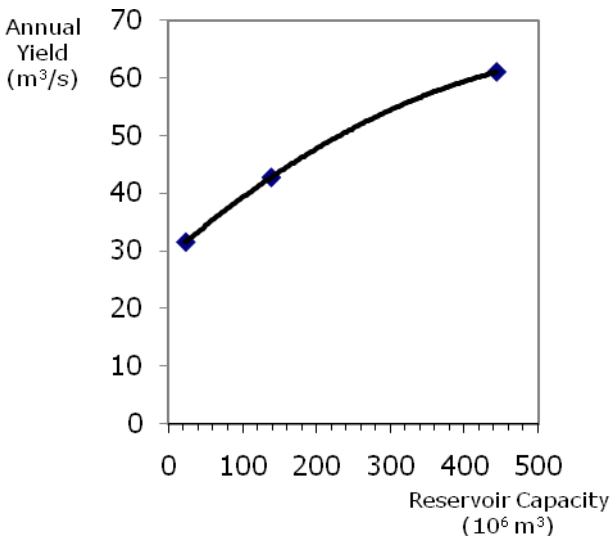

Fig. 2. Variation of the yield with capacity in the reservoir.

YIELD-CAPACITY-CAPACITY RISK RELATIONSHIPS

There has been a value of the reservoir capacity with any yield in the reservoir. However, it is possible to be obtained from the same yield by defining reservoir capacity with risk or to be provided yields in the various risks from any reservoir capacity. In this case, the effective storage capacity of the reservoir and the yield values of the reservoir can be defined by depending on a certain risk. Either a chosen reservoir capacity can be provided to a certain yield under a certain risk or a certain yield can be obtained from a certain risk to a chosen reservoir capacity in the reservoir. In Figure 3, the variation of the yield-capacity-capacity risk in the reservoir is shown. In Figure 3, when capacity risk is increased, reservoir capacity is reduced and it has been possible to provide with this capacity to the same yield under a certain capacity risk.

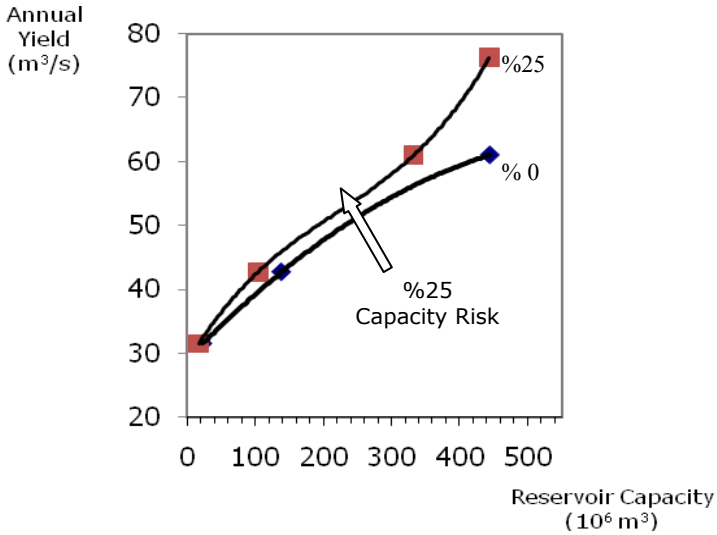

Fig. 3. Variation of the yield-capacity-capacity risk in the reservoir.

EARTHQUAKE RISK-WATER HEIGHT RELATIONSHIPS

Earthquake forces related to water in an earthquake are increased linearly with water height. Water height of the reservoir can be reduced to decrease these forces on the reservoir with earthquake risk. It is resulted to reduce the reservoir capacity when water height was decreased. In Figure 4, it is shown to the variation of the water height with earthquake risk. In Figure 4, when this risk is increased, the water height of the reservoir is reduced.

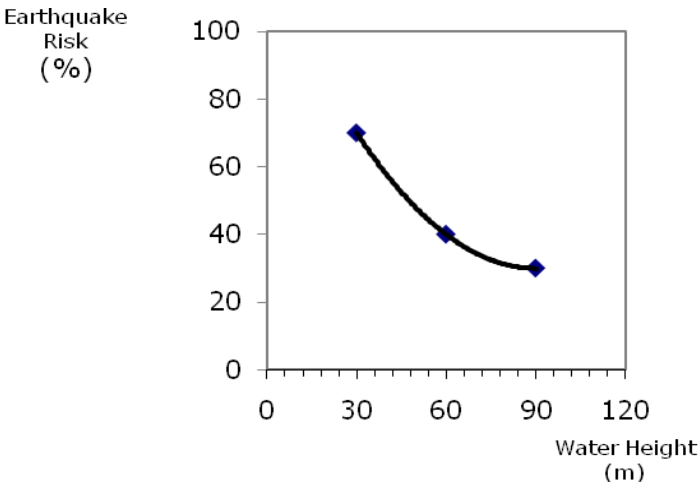

Fig. 4. Variation of the earthquake risk with water height of the reservoir.

EARTHQUAKE RISK-CAPACITY RISK RELATIONSHIPS

Capacity risk provided to the same yield by reducing reservoir capacity with earthquake risk is constituted. When earthquake risk is increased, capacity risk will be gone up. These risks are linear with each other. In Figure 5, the variation of the earthquake risk with capacity risk is shown.

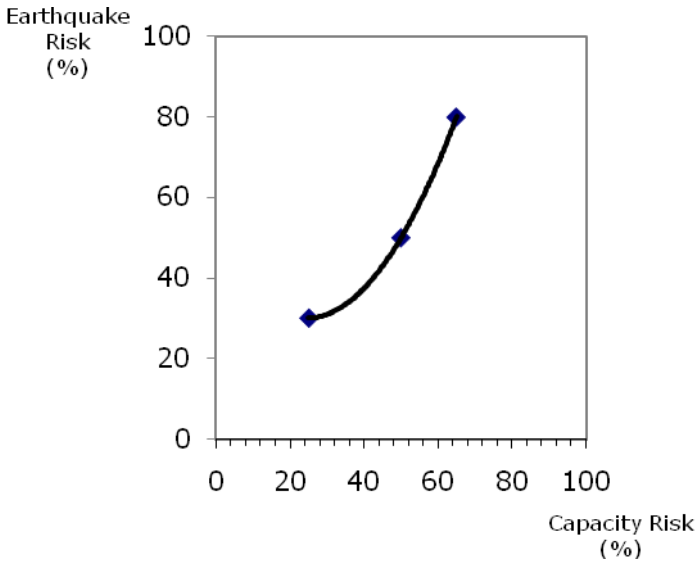

Fig. 5. Variation of the earthquake risk with capacity risk of the reservoir.

YIELD-CAPACITY-EARTHQUAKE RISK RELATIONSHIP

Yield-capacity earthquake risk relationships due to a linear relationship between earthquake risk and capacity risk are constituted as shown in Figure 6, similar to the yield-capacity-capacity risk relationships. In Figure 6, it is shown that the same yield in the certain values of the earthquake risk was provided.

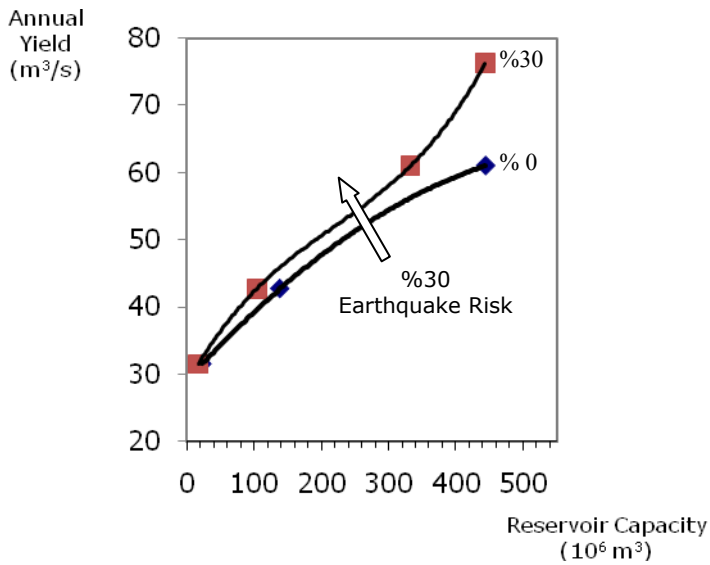

Fig. 6. Yield-capacity-earthquake risk relationships in the reservoir.

CONCLUSIONS

In this study, it has been worked to determine the yield-capacity-risk relationships and the results below are obtained. These are

- When water height is increased, reservoir capacity is gone up.
- When reservoir capacity is decreased, annual yield of the reservoir is reduced.
- When capacity risk is increased, reservoir capacity is reduced and it has been possible to provide with this capacity to the same yield under a certain capacity risk.
- When earthquake risk is increased, the water height in the reservoir is reduced.
- When earthquake risk is increased, capacity risk of the reservoir is gone up
- It is shown that the same yield in the certain values of the earthquake risk was provided.

ACKNOWLEDGEMENTS

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EFFECT OF ADDITION OF STEEL FIBERS ON CRACKING LOAD OF REINFORCED CONCRETE BEAMS UNDER THE EFFECT OF TORSION

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Key Words: Reinforced concrete, pure torsion, steel fiber, unit angle of twist, crack torque.

ABSTRACT Concrete beams subjected to torsion lose their rigidity totally with the first crack and than collapse. Longidutinal reinforcement applied to beams do not change the mentioned bahaviour. However with the shear reinforcement applied with longidutinal reinforcement effects the behaviour after crack and increases the torsional rigidity and effects the capacity according to reinforcement on the cracking load on cross section. In this study ten reinforced concrete beams are produced whose longidutinal and shear reinforcement ratios are constant. Compressive strength of the concrete, aspect ratio of the fibers and volumetric content of fiber are taken to be the variables of the experiments. Beams are subjected to torsion after the curing period and torsional moment versus unit angle of twist graphs are obtained for each specimen. It is observed that addition of steel fiber have no effect on cracking load of the specimens produced with normal strength concrete; whereas it increases the cracking torque of high strength concrete specimens.

INTRODUCTION

Beams are the structural members which mainly support the transverse loading through the flexural moment and the vertical shear. However, in some cases torsional response of beams may also control the overall structural behavior (Ersoy and Özcebe, 2001). For this reason, the torsional behavior of beams should be studied and comprehended as well as its shear or flexural behavior. This is especially important since the cracked torsional stiffness of a reinforced concrete beam may be much smaller than its uncracked stiffness.Realizing the importance of the problem, many researches (Hsu,1968 ; Fang and Shaiu, 2004) have been conducted on torsional behavior of reinforced concrete beams. The earliest experimental studies have investigated the effect of the presence of the reinforcement, both transverse and longitudinal, on torsional capacity and stiffness of the concrete beams. Thus, the torsional behavior of reinforced concrete beams have been compared with that of unreinforced (plain) concrete companions. It has been observed that the torsional behavior of concrete beams are linear until the first cracking torque. It has also been observed that the "uncracked" torsional stiffness of the beam, that is the slope of linear part of the torque twist $(T-\phi)$ diagram, is independent of the presence and amount of the reinforcement. In other words, up to the cracking point the torque-twist response of identical concrete and reinforced concrete beams are comparable. Surely, the reinforcement becomes effective after cracking, providing additional ductility, and even additional capacity, if proper reinforcement detailing is supplied. Since the torsional capacity i.e., the cracking torque, of an unreinforced concrete beam provides a limit state for torsional behavior of reinforced concrete beams, many analytical studies (MacGregor and Ghoneim, 1995 ; Bhatti and Almughrabi, 1996) have also been conducted to define the cracking torque. In these studies empirical expressions were proposed for the cracking torque level in terms of material and cross sectional properties of the beam. Advances in the material technology has pointed out the addition of steel fibers in concrete to improve the main characteristics of concrete, such as, stiffness, toughness and ductility. It should be noted that these improvements

closely affects the torsional capacity of a concrete beam. Thus, researches started to investigate whether the torsional capacity of a concrete beam can be increased with the addition of steel fibers in concrete, or not. Some compression tests using normal strength concrete with fiber reinforced specimens show that the addition of fibers may cause a decrease in compressive strength. However a considerable increase in tensile strength is observed in fiber reinforced specimens in split cylinder tests (El-Niema, 1993). Researches adding steel fiber into concrete in various aspect ratios and volume content give the corresponding stress strain curves relating to test specimens. As the volume content of the fiber increases, ductility under constant load increases in the diagrams. In other words, no difference is observed up to crack, but ductility increases afterwards with the increase in volume content of steel fibers (Rao and Seshu, 2005 ; Craig et all.,1986). When steel fibers are added to high strength concrete, the increase in fiber volumetric ratio also results in an increase in the compressive strength of the concrete, as well. However, when this addition exceeds a certain volumetric level, the increase in the strength becomes less (Song and Hwang, 2004

In the study presented here the volumetric steel fiber ratio, the aspect ratio of fibers and comressive strength of concrete are the selected variables. In the experiments, the cracking torsional capacity of reinforced concrete beams are investigated, with the variation of test parameters except the web reinforcement. In this study two different steel fiber aspect ratios, three different volumetric steel fiber ratios (one of them to be virgin) and two different compressive strength of concrete were used.

EXPERIMENTAL STUDY

Materials

Concrete mixtures are prepared by using crushed stone as coarse aggregate, sand as fine aggregate and cement with standard compressive strength of 42.5 MPa. In order to obtain the desired compressive strength, 30 MPa, superplasticizer is used in normal strength concrete mixtures and hyperplasticizer is used in high strength concrete mixtures in order to obtain a compressive strength of 60 MPa.

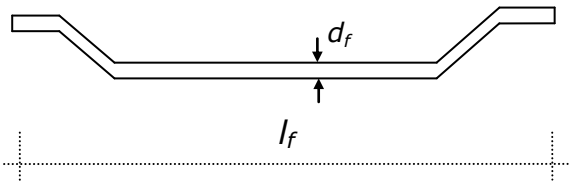

Fig. 1. Geometry of Steel Fibers.

Bent ended steel fibers having two different aspect ratios l_f / d_f = 40 and 55, corresponding to l_f and d_f dimensions of 30/0.75, 30/0.55 were used. Where l_f denotes fiber length and d_f denotes diameter of fibers. Geometry for steel fibers are given in Figure 1. d_w = 8 mm deformed bars are used as web reinforcement and d_l = 8 mm diameter deformed bars are used as longidutional reinforcement. The average yield strength for both longitudinal and web reinforcement is found to be f_{yk}=460 MPa.

Specimens

The crosssectional dimension of the test specimens are 150x200 mm with a length of 1900 mm. The longitudinal reinforcement ratio ρ_l is choosen as 0.0067 (4Φ8) and web reinforcement diameter is 8 mm and their center to center spacing is 200 mm which corresponds to shear reinforcement ratio of ρ_w=0.006 for all specimens. The shear reinforcement is set to be constant and the contrubition of longitudinal reinforcement and

the other variables to capacity is investigated. The tie-bar spacing outside the test region is decreased in order to force the failure of specimen to the test region. In those regions the tie-bar spacing is 50 mm from center to center. Concrete cover is set to be 5 mm for all specimens. The dimension of the test specimen and reinforcement layout are given in Figure 2.

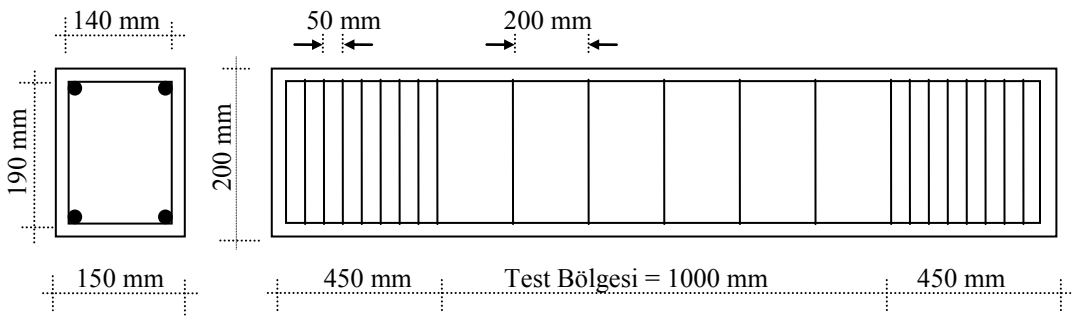

Fig. 2. Dimensions of the test specimen and the reinforcement layout.

It should also be noted that for each test specimen 12 standart cylindrical specimens with 150 mm diameter and 300 mm height are prepared and tested to obtain the compressive strength and split tensile strength of concrete.

The first three characters of a name of a specimens designate the compressive strength of the concrete type. NSC is used for normal strength and HSC is used for high strength concrete. Block F designates for the fiber aspect ratio F40, F55 designate for the aspect ratios 40 and 55 respectively. Block V designates the volumetric content of the fibers added, where V3 shows 0.3% and V6 shows 0.6% volumetric content. F blok is null and V is 0 while naming the specimens without SFR. Block V designates the volumetric content of the fibers added, where V3 shows 0.3% and V6 shows 0.6% volumetric content. F blok is null and V is 0 while naming the specimens without SFR. Specimens designation are given in Table 1.

Table 1. Test Specimens.

Specimens	Strength Type of Concrete	l_f/d_f	V_f (%)
NSCF00V0	NSC	-	-
NSCF40V3	NSC	40	0.3
NSCF40V6	NSC	40	0.6
NSCF55V3	NSC	55	0.3
NSCF55V6	NSC	55	0.6
HSCF00V0	HSC	-	-
HSCF40V3	HSC	40	0.3
HSCF40V6	HSC	40	0.6
HSCF55V3	HSC	55	0.3
HSCF55V6	HSC	55	0.6

NSC: Normal Strength Concrete HSC: High Strength Concrete

Test Results

The test specimens are subjected to uniform torsion in 28th day. Compression tests for concrete and tensile tests for steel used in the experiments are also performed at the same day. Graph of tension test is given in Figure 3. In order not to restrain the ends of the specimen from free rotation in the torsion tests, extension or contraction, the specimen is placed on roller supports aligned with the specimen.

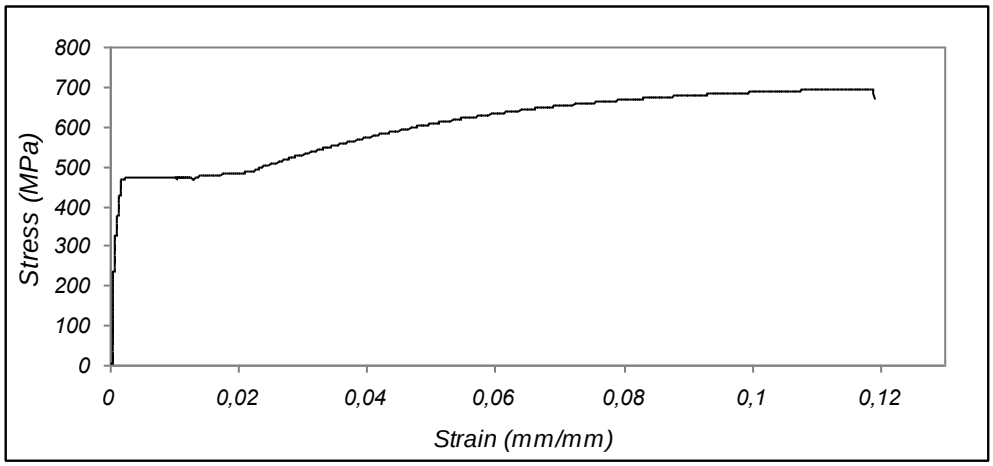

Fig. 3. Stress-Strain Curve for Steel Used in the Experiments.

Test data, torque (T) and unit angle of rotation (ϕ) are collected to a computer with the help of a data acqusition system and torque versus unit angle of twist graph ($T-\phi$) is obtained during the test.

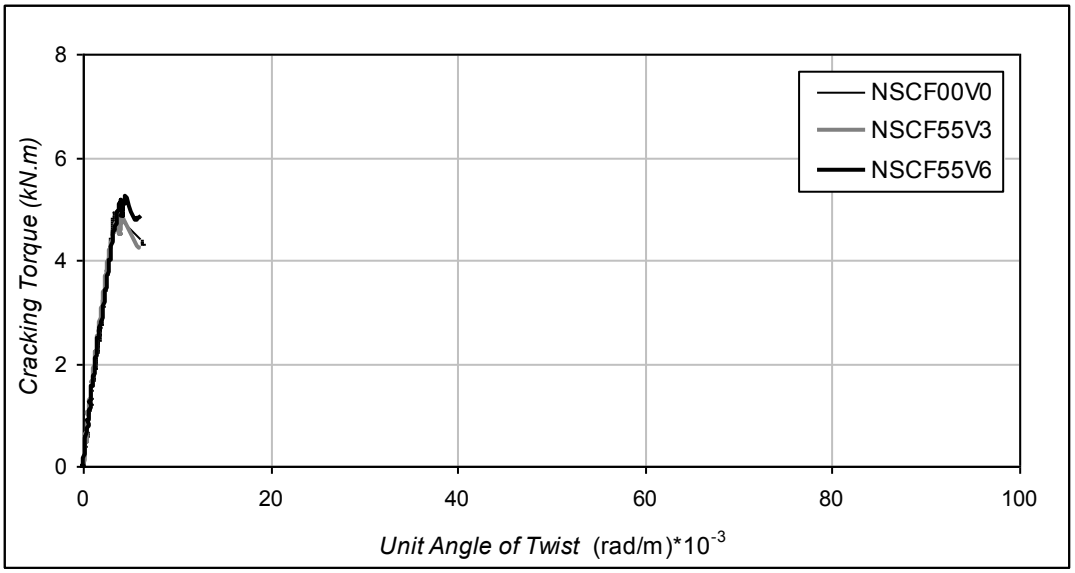

Fig. 4. ($T-\phi$) graph of specimens of normal strength concrete with l_f/d_f=55.

The data obtained from compession, split cylinder and torsion tests are given Table 2. In this table, f_{ck} designates the characteristic compressive strength of concrete, f_{cts} designates the split cylinder strength of concrete, T_{cr} designates the cracking torque of the specimen and ϕ_{cr} designates cracking unit angle of rotation.

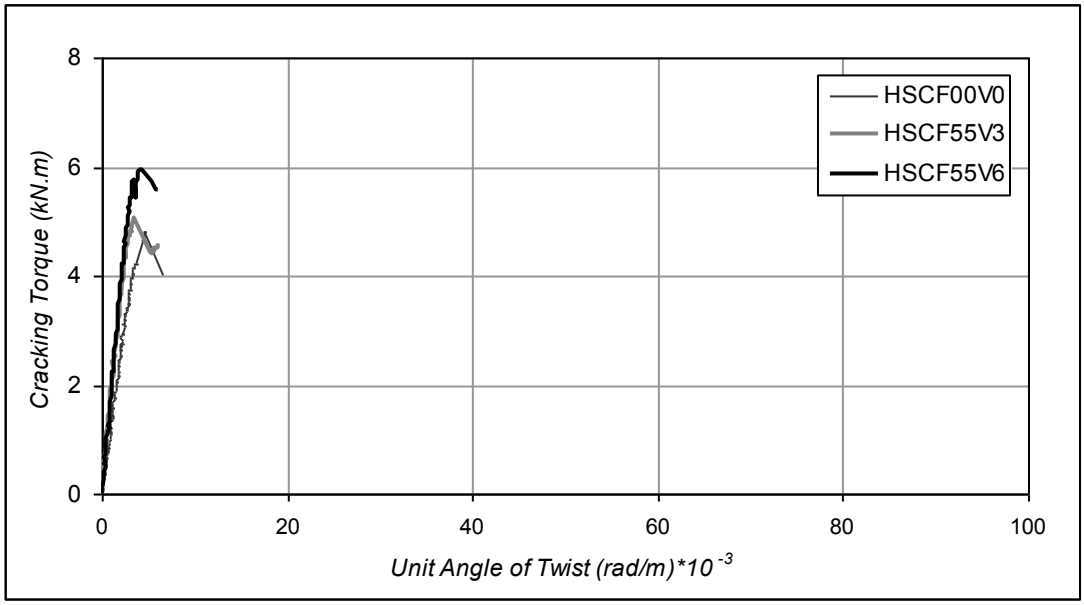

Fig. 5. ($T^{-\phi}$) graph of specimens of high strength concrete with l_f/d_f=55.

The data observed in Figure 4 and Figure 5 are given in Table 2. Increase in cracking torque for high strength concrete specimens are also seen in numerical values. The reason for this behaviour can be explained by the good bonding property of high strength concrete. The variations in cracking torques for the specimens are given in the last column.

Table 2. Test Data

Specimens	f_{ck} (MPa)	f_{cts} (MPa)	T_{cr} (kN.m)	ϕ_{cr} (rad/m)x10^{-3}	$T_{cr\text{-}variation}$* (%)
NSCF00V0	34.8	3.51	4.93	3.20	0.00
NSCF40V3	33.4	3.55	4.58	3.22	-7.10
NSCF40V6	31.3	3.35	4.62	3.52	-6.29
NSCF55V3	31.0	3.08	4.93	3.80	0.00
NSCF55V6	30.9	3.41	5.10	3.95	3.45
HSCF00V0	59.0	4.65	4.88	3.93	0.00
HSCF40V3	58.6	4.84	5.23	3.54	7.17
HSCF40V6	59.8	4.80	6.12	3.52	25.41
HSCF55V3	60.8	4.86	5.15	3.58	5.53
HSCF55V6	62.7	5.46	5.95	4.05	21.93

$$* \, T_{cr-\text{variation}} = \left(\frac{T_{cr-fiber} - T_{cr-virgin}}{T_{cr-virgin}}\right) * 100$$

The data in the last column of Table 2, is plotted as a graph in order to observe the cracking torque variations (Figure 6). According to this graph some effects (positive or negative) are seen in the cracking torques of normal stregth concrete specimens; however addition of steel fibers are not thought to be the reason for this variation. On the other hand, there is an increase of cracking torque values of high strength concrete specimens from 5% to 25% according to volumetric steel fiber ratio.

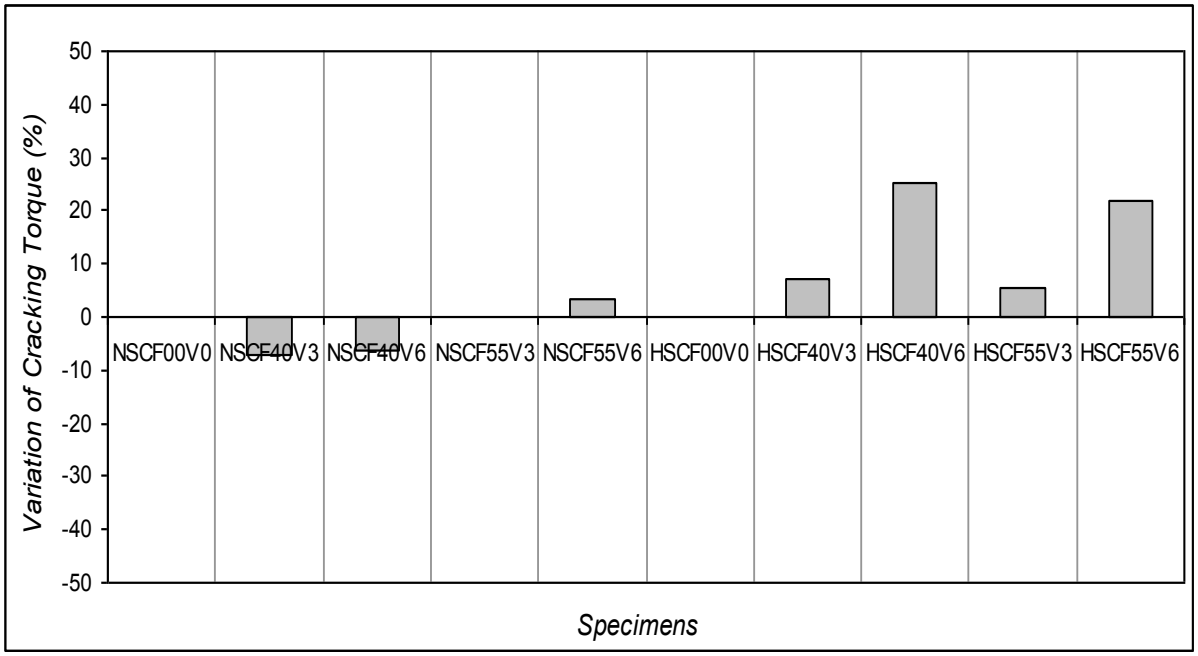

Fig. 6. Variations Obtained in Cracking Torques of Specimens.

CONCLUSIONS AND SUGGESTIONS

The below results are achieved provided that the data and variables of the test are used.

- Addition of steel fibers does not have a significiant effect on the cracking torque values of normal strength concrete specimens, but has an amount of 5% to 25% increase in high strength concrete specimens.
- Addition of steel fibers in high strength concrete delays the crack of the specimens.
- Addition of proper amount of steel fibers is thought to be useful in the cases where no crack or cracks in limitted widths are desired
- The profit obtained in this study and reliability of the results can be improved by the help of more number tests.

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FIRE PERFORMANCE OF CHEMICAL ADMIXTURES IN CEMENT MORTARS

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Key Words: Fire, superplasticizer, liquid latex, cement mortar

ABSTRACT In this study, fire performance of concrete with chemical admixtures was determined in this study. 54 cement mortar specimens with 50x50mm dimensions consist of superplasticizers (1 percent and 2 percent of cement volume) and liquid latex were produced. In the experimental study with liquid latex specimens including 1/3 and 1/1 ratio of latex/water were produced in addition to references specimens. All specimens except reference specimens were fired at 250°C and 500^0 C after the 28 days of cure time and end of the cooling period specimens were subjected to compressive tests. Compressive strength of mortar specimens were defined after the fire and loss of strength was investigated for each specimens. Compressive strength of the specimens exposed to elevated temperature and compressive strength of the reference specimens were compared. Thus, it was determined that effects of temperature reduce compressive strength of the concrete with superplasticizer. For the specimens consisting of liquid latex compressive strength loss was observed. However, loss of the specimens without admixture was more than the loss of the specimens without admixture.

INTRODUCTION

In recent years, concrete manufacturing has been increased rapidly with the development of technologies. Development of concrete manufacturing is provided by mineral and chemical admixtures. Materials as slag, fly ash, silica fume and stone dust similar with cement that stored in silo are named mineral admixtures. All mineral admixtures, except furnace slag, have not got the binding properties without any other binder as cement. However, like cement mineral admixtures behave as binder if they used with cement. Therefore, mineral admixtures provide usage of less cement. Chemical admixtures have a widespread use of enhancing the workability properties of concrete. Water reducers, superplasticizers, retarders, accelerators and air-entraining admixtures are the mainly chemical admixtures. Chemical admixtures provide advantages for the workability and pumping properties of concrete. Compressive strength of the concrete decreases significantly under the effects of temperature and fire [1]. On the other hand compressive strength of the lightweight concrete with superplasticizer or silica fume decreases under the effects of high temperature. There is no significant decreases at the lightweight and normal strength concrete until 100°C temperature as well insignificant increases are occurred. However, after 100°C temperature, loss of the compressive strength increases. Loss of the strength of both of two type concrete is significant at 800°C and 1000°C but compressive strength of lightweight concrete is more than the normal concrete [2]. Strength of the reinforcement steel is effected by the high temperatures, significantly. In the studies performed to determine the behavior of the reinforcement steel under the high temperatures, in the produced concrete with fly ash concrete cover was increased gradually and specimens were exposed to elevated temperatures until 1000°C. Strenght of reinforcement steel in specimens exposed to air cooling and tensile strenght of reinforcement steel in the specimen with fly ash, increased concrete cover and under high temperatures were compared. According to this result it was observed that tensile strenght decreased in two set-up but decreases are more for the reinforcement steel specimens exposed to high temperatures [3]. Strenght of the

concrete exposed to elevated temperatures are changed by air cooling and water cooling. In the studies, it was reported that in the specimens subjected to air cooling loss of the compressive strenght was less than the water cooled specimens [4]. Relation between color changing and compressive strenght in the concrete under high temperatures effects was investigated and relationship between these two parametres can be established [4,5].

In this study, specimens with superplasticizer or liquid latex were produced. These specimens were exposed to high temperatures. After cooling period specimens were subjected to compressive test.

EXPERIMENTAL STUDY

Materials

In this study, CEM42.4R cement producing in Hereke Nuh Çimento Plant was used. The properties of this cement were reported in Table 1. Sakarya River sand that aggregat size of 4 mm was used for the mortar mixtures. The properties of superplasticizer and liquid latex used as variable were reported in Table 2.

Table 1. Cement Properties.

CaO	64,51
SiO_2	20,14
Al_2O_3	4,50
Fe_2O_3	3,53
MgO	1,11
SO_3	2,39
Loss on ignition	2,80
Insoluble matter	0,69
Spesific weight (g/cm^3)	3,12
Spesific surface Blaine (cm^2/g)	3598

Table 2. Admixture Properties.

Superplasticizer	
Chemical Structure	Polycarboxylic Ether
Density (kg/l)	1,04 – 1,05
Chlorin content	< 0,1
Alkaline content	< 3,0
Liquid Latex	
Chemical Structure	Styrene Bütadien Emulsion
Density (kg/l)	1,015±0,01
pH	8-12
Freezing point	-5°C

Specimens

In experimental studies, for each mixture ratio three specimens of 50x50x50 mm were produced. For the each spesicific set-up compressive strenght was determined by calculating the mean of compressive strenght values obtained from the experiments. Specimens were prepared for sand/cement was 3 and the constant fluidity of about 110±2. For the first experimental set-up 27 specimen were prouced by using superplasticizer as variable and the admixture were added in 0%, 1% and 2% of the cement weight, respectively. For the second experimental set-up 27 specimen were prepared by using liquid latex as variable and the ratios of latex admixture and water used in mortar were chosen 0/1, 1/3 and 1/1, respectively. Mixture ratios in the experiments were showed in Table 3. Names and properties of the produced specimens were given in the Table 4 and also Table 4 showed the temperature that specimens were exposed to.

Table 3. Admixture Ratio of Cement Mortar.

Admixture Ratio (%)	C (kg/m^3)	K/C	E/C	s (mm)	Admixture Ratio (L/E)	C (kg/m^3)	K/C	E (kg/m^3)	(E+L)/C
0 S.P.A	500	3	0.50	110±2	0/1	400	3	200	0.50
1 S.P.A	500	3	0.41	110±2	1/3	400	3	150	0.50
2 S.P.A	500	3	0.37	110±2	1/1	400	3	100	0.50

S.P.A:superplasticizer, C:Cement, K: Sand, E: Water, s: Mortar Settling, L: Liquid latex

In Table 4, S and L determine chemical admixture type. The numbers at the side of the S (00, 01, 02) mean the amount of the superplasticizer in the cement by percentage. Meaning of the numbers at the side of the L (0/1, 1/3, 1/1) is ratio of the liquid latex and water. N determines the specimen was not exposed to heat just stored at the room temperature (25°C). M means that specimens were heated up to 250°C and the H means that specimens were heated up to 500°C. The numbers like 1-2-3 at the bottom of the table show different specimens had the same mixture ratio.

Tablo 4. Produced Specimens.

Specimen Name	S.P.A (Cement %)	T (°C)	Specimen Name	L/E	T (°C)
S00-N1	0	25	L0/1-N1	0/1	25
S00-N2	0	25	L0/1-N2	0/1	25
S00-N3	0	25	L0/1-N3	0/1	25
S00-M1	0	250	L0/1-M1	0/1	250
S00-M2	0	250	L0/1-M2	0/1	250
S00-M3	0	250	L0/1-M3	0/1	250
S00-H1	0	500	L0/1-H1	0/1	500
S00-H2	0	500	L0/1-H2	0/1	500
S00-H3	0	500	L0/1-H3	0/1	500
S01-N1	1	25	L1/3-N1	1/3	25
S01-N2	1	25	L1/3-N2	1/3	25
S01-N3	1	25	L1/3-N3	1/3	25
S01-M1	1	250	L1/3-M1	1/3	250
S01-M2	1	250	L1/3-M2	1/3	250
S01-M3	1	250	L1/3-M3	1/3	250
S01-H1	1	500	L1/3-H1	1/3	500
S01-H2	1	500	L1/3-H2	1/3	500
S01-H3	1	500	L1/3-H3	1/3	500
S02-N1	2	25	L1/1-N1	1/1	25
S02-N2	2	25	L1/1-N2	1/1	25
S02-N3	2	25	L1/1-N3	1/1	25
S02-M1	2	250	L1/1-M1	1/1	250
S02-M2	2	250	L1/1-M2	1/1	250
S02-M3	2	250	L1/1-M3	1/1	250
S02-H1	2	500	L1/1-H1	1/1	500
S02-H2	2	500	L1/1-H2	1/1	500
S02-H3	2	500	L1/1-H3	1/1	500

S.P.A.:Superplasticizer, T: Temperature, L: Liquid Latex, E: Water

Fig. 1. Oven Used in Experimental Studies.

Specimens were kept in molds for 24 hours after demoulding, the specimens were left for 27 days in a curing room at room temperature. After 27 days curing specimens were taken from water and stored at room temperature for drying. On the 28th day some of the specimens were exposed to 250 and 500oC temperatures for 3 hours in oven. Fig. 1. showed oven used in the experimental studies and changes at the applied temperatures were given at Fig. 2. When heating up period was ended oven was shut down and the specimens were taken out and left for cooling in standing water at room temperature. After cooling process specimens were dried and subjected to compressive tests.

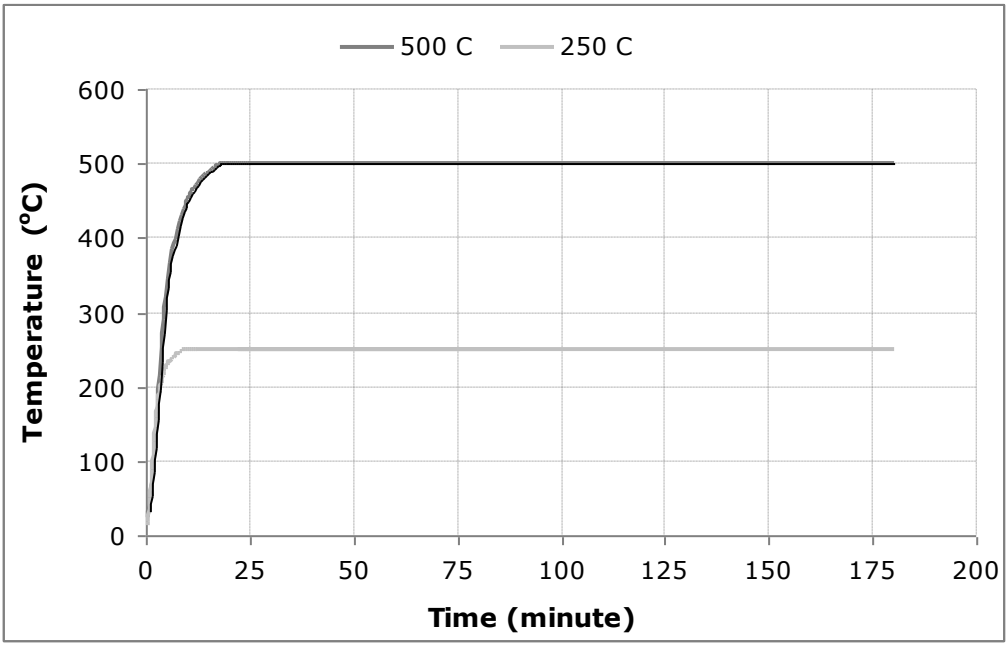

Fig. 2. Changes in the temperature applied to specimens.

Results

Produced specimens were exposed to high temperature, specimens heated up to high temperatures were subjected to cooling process. After cooling process specimens were dried in air and comperssive tests were performed. The results of the compressive tests of specimens added the superplasticizer were showed in Fig. 3. As seen from Fig. 3, compressive strenghts of the specimens without superplasticizer and were not subjected to heating up just left at room temperature were obtained with a value of 44 MPa. Compressive strenght of the specimen without admixture and were heated up to 250°C temperatures were determined with a value of 33 MPa. Compressive strenght of similar specimens heated up to 500°C temperatures were obtained with a value of 27 MPa. Compressive strenght of the specimen with 1% superplasticizer were determined with a value of 47 MPa at room temperature, 28 MPa at 250°C temperature and 19 MPa at 500°C temperatures. Compressive strenght values decreased by increased temperature. The comppressive strenghts of specimens with 2% superplasticizer admixture were obtained with a value of 49 MPa at room temperature. When the temperature was increased 250°C temperature compressive strenght value decreased to 29 MPa, at 500°C temperature compressive strenght decreased to 22 MPa. Relative strenght losses of the specimens with and without superplasticizer under temperature effect or not were given in Fig. 4.

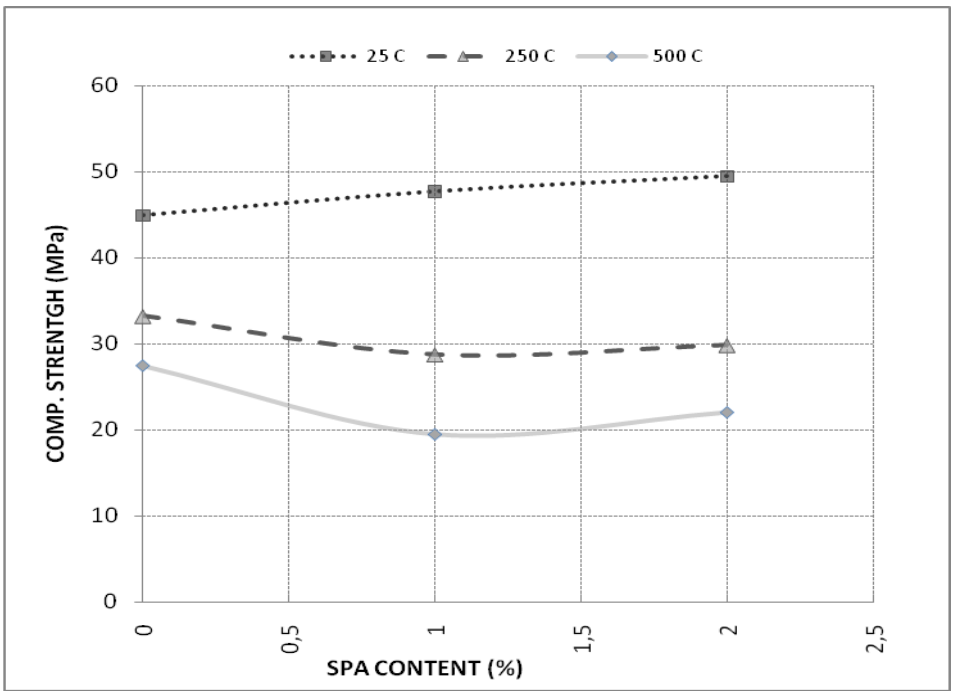

Fig. 3. Relation between superplasticizer content and compressive strenght.

According to Fig. 4, with acception of compressive strenght of the specimens without admixture at 25°C temperature as references it was obserbed that when temperature increased to 250°C and 500°C temperatures compressive strenght loss was 26% and 39%, respectively. Compressive strenght loss of the specimen with 1% superplasticizer at 25°C was accepted as references, compressive strenght loss for 250°C and 500° were 40% and 55%, respectively. Compressive strenght of the specimens with 2% superplasticizer for 250°C and 500°C were determined 40% and 59%, respectively.

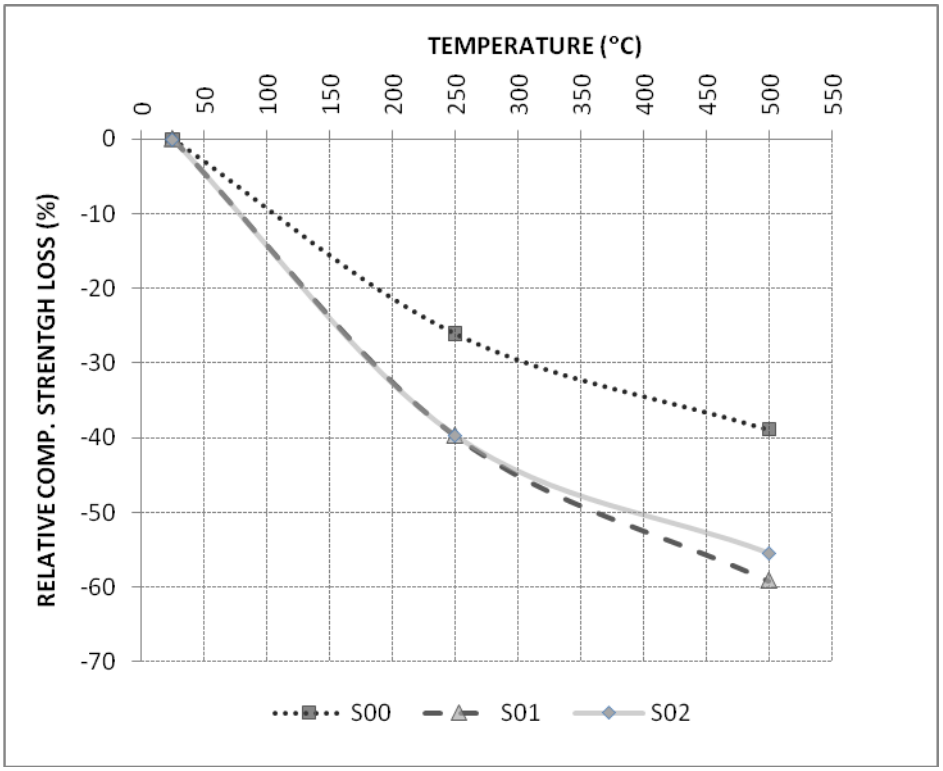

Fig. 4. Relation between superplasticizer and relative compressive strenght.

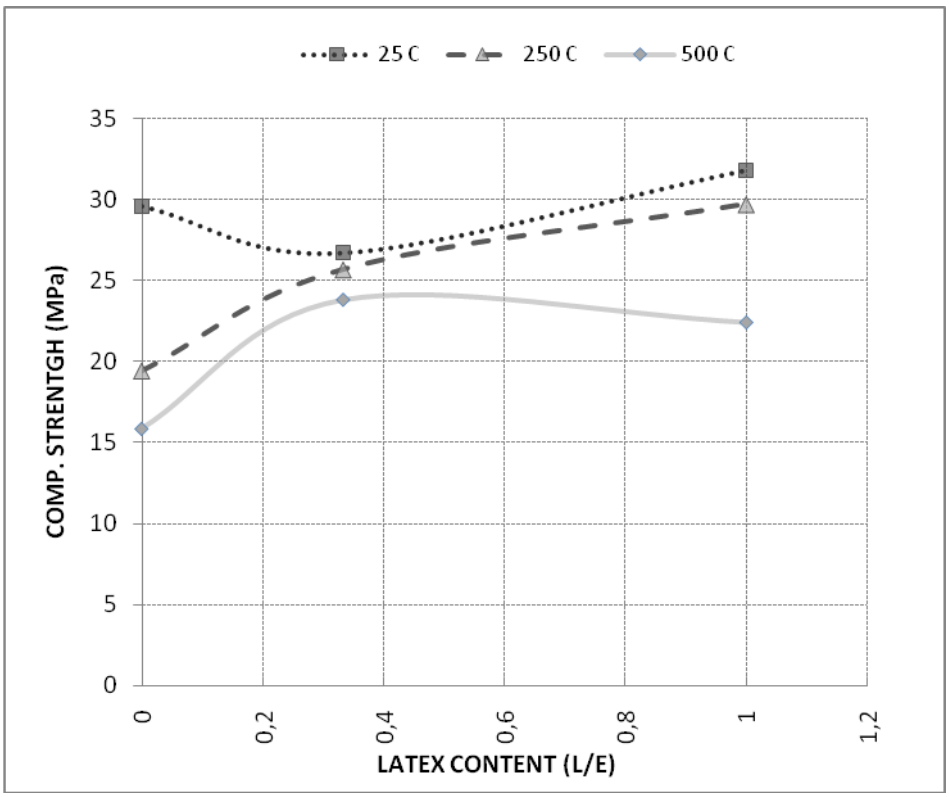

Fig. 5. Relation between liquid latex content and compressive strenght.

Fig. 5 and Fig. 6 showed the graphes of the specimens with liquid latex admixture. Due to the latex admixture compressive strenght obtained with increasing in temperatures were given in Fig. 5. Relation between temperature and relative strenght in case of containing latex admixture was showed in Fig. 6. According to Fig. 5, compressive strenght of the specimens without latex were reached 29 MPa, 19 MPa and 16 MPa for unheated (25°C), 250°C and 500°C temperatures, respectively. Compressive strenght were obtained from 1/3 latex specimens with a value of 26 MPa for unheated (25°C), 25 MPa at 250°C and 24 MPa at 500°C. For 1/1 latex specimens compressive strenght values were determined 31 MPa, 29 MPa and 22 MPa for unheated (25°C), 250°C and 500°C, respectively. Due to the Fig. 6, compressive strenght of the specimens without admixture was accepted as reference thus with increasing in the temperatures to 250°C and 500°C strenght loss was reached to 34% and 46%, respectively. For the 1/3 latex specimens when temperature increased to 250°C compressive strenght loss was determined 4% and at 500°C, 7% compressive strenght loss was observed. Compressive strenght loss of 1/1 latex specimens were obtained 7% at 250°C and 29% at 500°C. Considering all these data, it can be said that superplasticizer admixture is decreased the compressive strenght of the cement mortar exposed to high temperatures. Liquid latex admixture limits the strenght loss of the cement mortar more than cement mortar without admixture.

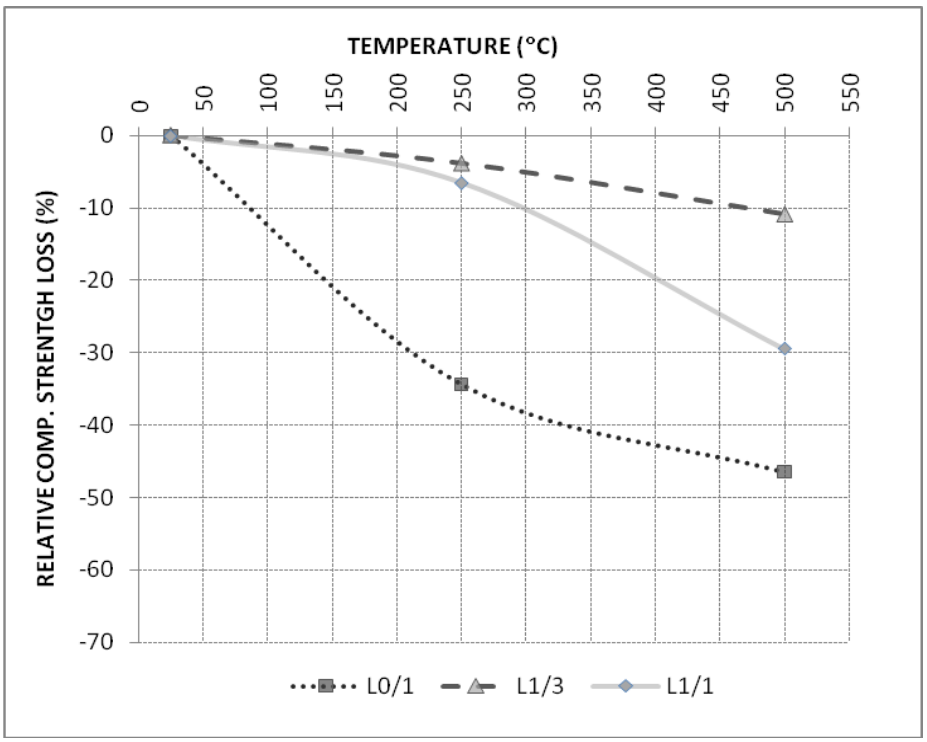

Fig. 6. Relation between liquid latex content and relative compressive strenght.

CONCLUSION

According to the test performed with the chosen variable, obtained results were given below;

- Compressive strenght of the cement mortar without admixture is decreased with increasing in temperature that specimens are exposed to. For the specimens without admixture compressive strenght loss was observed as 34% and 46% at 250°C and 500°C, respectively.

- Compressive strenght of the cement mortar with superplasticizer or liquid latex under the effects of high temperature is less than the cement mortar without admixture

- Relative compressiv strenght loss of the specimens with 1% or 2% were determined as 40% and 55% for 250°C and 500°C, respectively. These results show that they have similar fire performance.

- Although it was observed reducing in compressive strenght of the specimens used liguid latex at 1/3 or 1/1 ratio, it was determined that it caused less relative compressive strenght loss than the reference specimen.

- At 250°C, using liquid latex admixture at 1/3 ratio caused minimum compressive strenght loss.

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THE DESIGN AND EVALUATION STAGES OF LOCAL GNSS NETWORKS FOR MONITORING CRUSTAL MOVEMENTS

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Key Words: Optimization of GNSS Network, baseline optimization, second and first order optimization, quality and reliability criterions, optimization of quality and reliability, monitoring crustal movements.

ABSTRACT In this contribution, we propose a standard way to establish GNSS (Global Navigation Satellite System) networks and how we take under control them before not carrying out GNSS observations in the theory part.

Validation of the proposed way is applied on IZDOGAP (Project of monitoring Substructure of Natural Gas in Kocaeli via National Geodetic Networks and Land Information System) GNSS network in the example part.

INTRODUCTION

GNSS networks are widely used for deformation analyses in many Earth sciences, and generally it is not given heed to observation plan corresponding geometry of the network. Yet, GNSS network observation plan is also considered like classical networks (triangulation, trilateration and leveling network). In this study, how observation plan of a GNSS network should be constructed via solution of second order design with analytical method is explained. And, the optimum designed GNSS network is tested with criterions of quality and reliability.

The GNSS network design is carried out at two steps. First step is called network optimization (planning which baselines are observed) using the second order design and the reliability optimization, and second step is called baseline optimization (determining observation time span) in this paper. Approximate positions of the network points have been determined by hand-held GNSS or on 25000 scaled maps at the two stages. The network optimization is performed by using Taylor-Karman structured criterion matrix having properties of homogeneity and isotropy. The baseline optimization is performed with approximate positions of the points and satellite positions are computed from orbital parameters acquiring by internet (for example Yuma Almanacs).

In the design stage, the step of network optimization is to be main part, the other step in the stage is used to determine GNSS observation time span or to inspect the first step for quality and reliability according to the predicted time span. For instance, the time span was taken as 10-15 minutes in this study because of decreasing cost of GNSS campaign.

After the design stage, optimum observation plan tested for quality and reliability by baseline optimization will be carried out on working area. Baseline observations are made considering the observation plan and the predicted time span. Then, components (i.e. relative coordinates) of the baselines and their variance-covariance matrices are computed from processing phase and code observables and this step is called as baseline adjustment in the evaluating stage. After that, the components of baselines obtained from baseline adjustments are adjusted in the network according to the optimum observation plan again and this step is called as network adjustment in the evaluating stage. At the end of the evaluating stage, coordinates of the network points, quality and

reliability criterions are acquired, and the network is inspected for the quality and the reliability.

The coherence of the results of evaluating stage to those of designing stage was examined on IZDOGAP (Project of monitoring Substructure of Natural Gas in Kocaeli via National Geodetic Networks and Land Information System) GNSS network. IZDOGAP is founded by Scientific Research Project Unit of Kocaeli University and İZGAZ (İzmit Gas Distribution Industry and Trade Inc.).

The GNSS network has 106 points distributed at Kocaeli-Istanbul-Sakarya-Yalova-Bursa cities and 252 baselines observations after second order design according to Taylor-Karman structured criterion matrix and reliability optimization in the design stage. After the zero period GNSS observations of the designed network was completed on 14 July 2009, baseline adjustment and network adjustment were carried out. Moreover quality and reliability criterions of the network were computed and compared with those in which designing stage. And the results obtained are summarized on the network figures.

THEORY

Relative point positioning is basic evaluation way on GNSS observations for high precise. And, that is made in two different types. First type is single baseline solution; the second is multi baselines solution. In the single baseline solution, baselines in the network are considered as independent observation, not in the multi baseline solution. In this study, single baseline solution is chosen because of low number of GNSS receivers (is equal to 4 or 6 for six days only) and many observation sessions (92) (Craymer and Back, 1992; Han and Rizos, 1995). The single baseline solution is called as baseline adjustment. After the baseline adjustment, baseline vectors and their variance-covariance matrix are obtained. The vectors are adjusted in the network established with relative relationship among network points once more. That is called as network adjustment.

The baseline and network adjustment take place in the evaluation stage and the design stage of GNSS network. But, these are performed reversed order in the both stages, are going to be explained respectively.

EVALUATING GNSS OBSERVATIONS

Baseline Adjustment or the relative positioning is applied by differencing phase or/and code observables in GNSS. The differenced observables are Single Differences (SD), Double Differences (DD) and Triple Differences (TD). So, mathematical model for a baseline is established on the differenced observables. DD mathematical model is most popular one. A lot of commercial and academically softwares use it. For these reason, DD mathematical model is chosen for the baseline adjustment in the paper. The model for phase observable, are also necessary for precise positioning, is established as following manner.

$$E\{\underline{y}\} = \underline{A}\,\underline{a} + \underline{B}\,\underline{b} \qquad D\{\underline{y}\} = \sigma^2 \underline{Q}_y \quad (1)$$

Where E{*} and D{*} are expectation and dispersion operator respectively and $\underline{y}$ is the DD phase observations vector, $\underline{a}$ and $\underline{b}$ are unknowns DD ambiguities and baseline components and corresponding design matrix $\underline{A}$ and $\underline{B}$ respectively.

The baseline adjustment is performed in three-step processing procedure (Teunissen, 1995; Odijk, 2003; Kurt, 2003).

Step 1: *Float Solution:* In this step, ordinary Least Square (LS) solution carried out and all unknowns take real values and these are represented by "$\hat{*}$" sign.

$$\min_{a,b} = (\underline{y} - \underline{A}\,\underline{a} - \underline{B}\,\underline{b})^T \underline{Q}_y^{-1} (\underline{y} - \underline{A}\,\underline{a} - \underline{B}\,\underline{b})$$

$$\begin{bmatrix} \underline{\hat{a}} \\ \underline{\hat{b}} \end{bmatrix} = \begin{bmatrix} \underline{Q}_{\hat{a}} & \underline{Q}_{\hat{a}\hat{b}} \\ \underline{Q}_{\hat{b}\hat{a}} & \underline{Q}_{\hat{b}} \end{bmatrix} \begin{bmatrix} \underline{A}^T \underline{Q}_y^{-1}\,\underline{y} \\ \underline{B}^T \underline{Q}_y^{-1}\,\underline{y} \end{bmatrix} \quad (2)$$

$$\hat{\sigma}^2 = \underline{\hat{e}}^T \underline{Q}_y^{-1} \underline{\hat{e}} \,/\, (n - m - 3) \qquad \hat{e} = \underline{y} - \underline{A}\,\underline{a} - \underline{B}\,\underline{b} \quad (3)$$

Where, **n** and **m** represent number of observation and DD ambiguities respectively, and $\hat{e}$ is residual vectors for the float solution.

Step 2: *Ambiguity Resolution (ILS: Integer Least Squares):* In the step, the integer DD ambiguities are solved by search technique, which is performed in a hyper ellipsoid, is established from float DD ambiguities and their VC matrix acquiring by the float solution (Teunissen, 1995; Odijk, 2003; Kurt, 2003a, 2003b, 2003c, 2005).

$$\min_{a} = (\underline{\hat{a}} - \underline{a})^T \underline{Q}_{\hat{a}}^{-1} (\underline{\hat{a}} - \underline{a})$$

$$\breve{\sigma}^2 = \underline{\breve{e}}^T \underline{Q}_y^{-1} \underline{\breve{e}} \,/\, (n - 3) \quad \underline{\breve{e}}^T \underline{Q}_y^{-1} \underline{\breve{e}} = \underline{\hat{e}}^T \underline{Q}_y^{-1} \underline{\hat{e}} + (\underline{\hat{a}} - \underline{\breve{a}})^T \underline{Q}_a^{-1} (\underline{\hat{a}} - \underline{\breve{a}}) \quad (4)$$

Where, all parameters improving with the estimated DD ambiguities are represented by "$\breve{*}$" sign. For example, $\underline{\breve{a}}$ is a vector containing the DD integer values, and the vector is nearest distant to the float vector of DD ambiguities, and $\underline{\breve{e}}$ is a residual vector for the nearest integer vector and $\breve{\sigma}^2$ is its varyance. Validation test for the nearest integer of estimated DD ambiguities is performed by following F-test.

$$\breve{\sigma}^2 / \hat{\sigma}^2 <= F_{(0.95, n-3, n-m-3)} \quad (5)$$

While $\hat{\sigma}$ is determined experimentally in the design stage, it is acquired from the baseline adjustment using real GNSS observables in the evaluating stage.

Step 3: *Fixed Solution:* In this step, the baseline components are improved by the nearest integer vector. So these are gotten at millimeter precision at the end of the step.

$$\underline{\breve{b}} = \underline{\hat{b}} - \underline{Q}_{\hat{b}\hat{a}} \underline{Q}_{\hat{a}}^{-1} (\underline{\hat{a}} - \underline{\breve{a}}) \quad (6)$$

$$\underline{K}_{\breve{b}} = \breve{\sigma}^2 \underline{Q}_{\breve{b}} \qquad \underline{Q}_{\breve{b}} = \underline{Q}_{\hat{b}} - \underline{Q}_{\hat{b}\hat{a}} \underline{Q}_{\hat{a}}^{-1} \underline{Q}_{\hat{a}\hat{b}} \quad (7)$$

After baseline adjustments, we have the improved baseline vector ($\underline{\breve{b}}$) and their variance-covariance matrices ($\underline{K}_{\breve{b}}$). These are accepted as observations for the network adjustment that is next step of the valuation stage.

Network Adjustment is that coordinates of all network points are estimated within an adjustment using baseline vectors as observations. The mathematic model for a baseline is established with Eqs.(8) during the network adjustment.

$$\underline{\breve{b}}_{ij} + \underline{v}_{ij} = \underline{A}_{ij} \underline{x} \quad \underline{P}_{ij} = \underline{K}_{\breve{b}_{ij}}^{-1} \quad (8)$$

$$\underline{A}_{ij} = \left[\, \underline{0} \;\; \cdots \;\; -\underline{I}_i \;\; \cdots \;\; \underline{I}_j \;\; \cdots \;\; \underline{0} \,\right] \qquad \underline{x} = \left[\, \underline{X}_1 \;\; \cdots \;\; \underline{X}_i \;\; \cdots \;\; \underline{X}_j \;\; \cdots \;\; \underline{X}_p \,\right]^T$$

$$\underline{X}_k = \left[\, X_k \quad Y_k \quad Z_k \,\right]^T \quad k=1,\ldots,i,\ldots,j,\ldots,p$$

Where, p is number of network points, u=3p is number of unknowns, those are components of the network points, $\underline{x}$ is the vector of the network points, a design matrix $\underline{A}_{ij}$ for a baseline vector ($\underline{\breve{b}}_{ij}$) is a sub design matrix of the network design matrix, $\underline{I}$ is identity matrix by 3 times 3.

Mathematic model for all observed baselines is augmented with Eqs.(8) for all the network points and the model is given by Eqs.(9).

$$\underline{v}_\Delta = \underline{A}_\Delta \underline{x} - \underline{\ell}_\Delta \quad \underline{P} \qquad (9)$$

$$\underline{v}_\Delta^T \underline{P}\, \underline{v}_\Delta + \underline{x}^T \underline{x} = \min \qquad (10)$$

Using Least Squares according to the inner constraints solution, we have the adjusted points of the network and their variance-covariance matrix (for computing the quality measures) and redundancy matrix (for computing the reliability measures) from Eqs.(11), Eqs.(12) and Eqs.(13) respectively.

$$\underline{x} = \underline{Q}_X \underline{A}_\Delta^T \underline{P}\, \underline{\ell}_\Delta \qquad (11)$$

$$\underline{K}_X = m_0^2 \underline{Q}_X \qquad \underline{Q}_X = (\underline{A}_\Delta^T \underline{P}\, \underline{A}_\Delta)^+ \qquad (12)$$

$$\underline{R} = \underline{Q}_v \underline{P} \qquad \underline{Q}_v = \underline{P}^{-1} - \underline{A}_\Delta \underline{Q}_X \underline{A}_\Delta^T \qquad (13)$$

$$m_0^2 = \underline{v}_\Delta^T \underline{P}\, \underline{v}_\Delta \big/ (n - u + d)$$

where, b is number of observed baselines, n=3b is number of observations, those are components of the observed baselines, d is number of the datum defect (d=3 for GNSS networks) in the free-network adjustment (Kurt, 1996; Kurt et al., 1999, Açici et al. 2003, Konak et al., 2004, Kurt et al., 2009).

DESIGN STAGE FOR GNSS NETWORKS

Design stage is performed reversed order versus the evaluating stage. At first, observation plan (design matrix) is determined with precision and reliability optimization. And then, the design matrix is tested via baseline optimization.

The Networks Optimization

The network optimization is carried out at two steps. First step is precision (weight) optimization; the second step is reliability optimization.

The Precision Optimization uses artificial variance-covariance matrix, which is necessary for computing network quality measures (for examples error ellipses). The artificial variance-covariance matrix is called as criterion matrix. The criterion matrix is established according to Taylor-Karman structured criterion matrix having properties of homogeneity and isotropy (Figure-1). In the figure, the criterion matrix is represented on IZDOGAP GNSS Network.

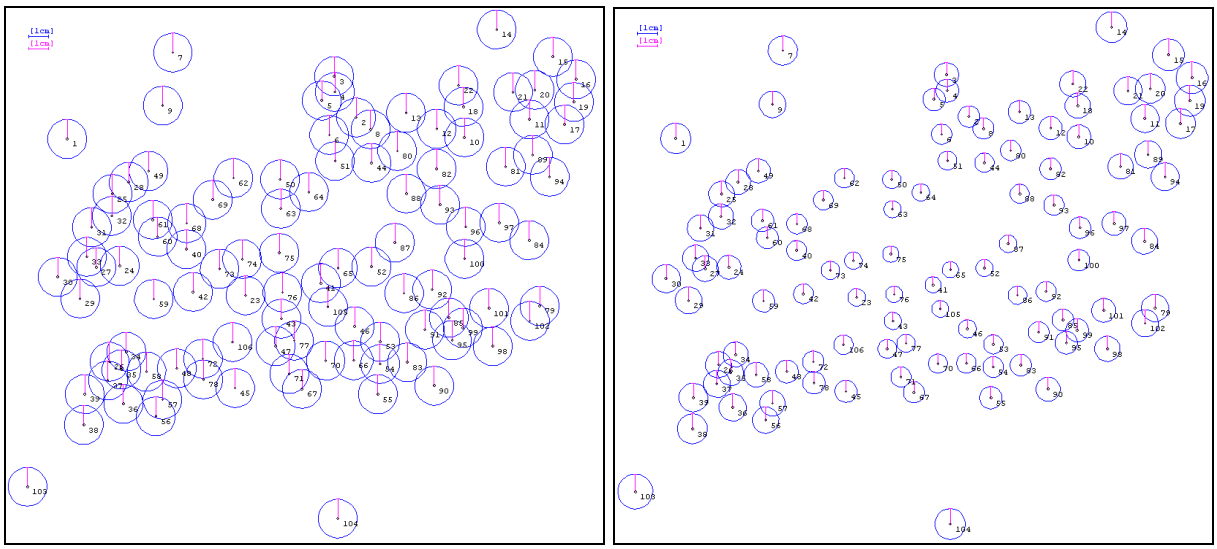
$$\underline{C}_x = \sigma^2 \underline{Q} \qquad (14)$$

Where, $\underline{C}_X$ is the artificial criterion matrix, σ^2 is a priori variance and $\underline{Q}$ is the artificial cofactors matrix of unknowns, too.

Constructing a maximum observation plan on matrix $\underline{A}_\Delta$ and using the criterion matrix, optimum weights are determined iteratively following formulas (Eqs.15). The solution is called as Um-solution, is second order design (precision or weight optimization) (Grafarend, 1985; Illner, 1986; Konak, 1994; Konak, 1995; Kurt, 1996; Kurt et al.1999; Konak et al., 2004; Konak et al., 2005; Kurt, 2009; Kurt and Konak, 2009).

$$\underline{p} = (\underline{A}_\Delta \, \underline{A}_\Delta^T * \underline{A}_\Delta^T \, \underline{A}_\Delta^T)^{-1} (\underline{A}_\Delta^T \Theta \underline{A}_\Delta^T)^T \underline{q} \qquad (15)$$

Where, $\underline{p} = \mathrm{vec}(\underline{P})$ and $\underline{q} = \mathrm{vec}(\underline{Q})$, and "*" and "Θ" are refer to Hadamart and Khatri-Rao products respectively. In the solution, mathematical meaningless weights, for example P_i <0.00 or >100, are removed from observation plan iteratively, this is called as *first reduction of maximum observation plan (or precision optimization)* in this study.



(a) **(b)**

Fig.1. (a) Experimental Criterion matrix ($\underline{C}_x$) for IZDODAP, **(b)** the criterion matrix transformed to free network datum $\underline{S}\,\underline{C}_x\,\underline{S}^T$ (error ellipses and precisions of heights of the network points, blue and pink respectively).

In the GNSS networks, maximum number of baselines (b_{max}) or number of observations ($3b_{max}$) is up to number of points in the network, and it can be determined $b_{max}=p(p+1)/2$ easily (p=106, b_{max}=5671 for the IZDOGAP network). But it is not necessary to plan all baselines the network. The maximum observation plan can be established under some restrictions. One of the restrictions can be chosen as all baselines in the network being shorter than 20km for ignoring the atmospheric effects on GNSS observables like as IZDOGAP. The restricted maximum observation plan on the network of IZDOGAP has b_{max}=1042 baselines, 1011 baselines ≤20km and 31 baselines >20km are in the network.

The Reliability Optimization is performed by using reliability measures. These are internal and external reliabilities. The internal reliability measures are partial redundancy number and threshold of undetectable outlier. These reliability measures and their thresholds are given following equations and equalities.

$$r_i = (\underline{R})_{ii} \qquad \geq 0.30 \text{ or } 0.50 \quad (16)$$

$$\Delta_i / \sigma_i = 4\sqrt{(\underline{P})_{ii} / (\underline{P}\,\underline{R})_{ii}} \qquad \leq 6 \text{ or } 8 \qquad (17)$$

i=1,2,…,3b

Where, r_i and Δ_i / σ_i are partial redundancy number and standardized threshold of undetectable outlier for i*th* observation, respectively.

The external reliability measure is a coefficient for effect of undetectable outlier on the unknowns, which are coordinates of the network points. And the measure is tested according to following threshold.

$$\delta_i = \Delta_i / \sigma \sqrt{(\underline{P})_{ii} - (\underline{P}\,\underline{R})_{ii}} \qquad \leq 8 \text{ or } 10 \qquad (18)$$

Where, δ_i is external centrality parameter (Öztürk and Şerbetçi, 1992). if $\underline{P}=\underline{I}$ and r_i=0.25, Δ_i/σ_i and δ_i are equal to 8 and 9.24 respectively. For this reason partial redundancy number (r_i) is very good chosen for reliability optimization than the others because of ensuring a simple presentation on the figures of the paper.

In this study, threshold of partial redundancy number is chosen as 0.90, and observations which their redundancy number are greater than 0.90 are also removed from the first reduced observation plan obtained by the precision optimization, and this part called as *second reduction of observation plan (or reliability optimization)* in this study.

Baseline Optimization for Testing Network Optimization

The baseline optimization used for inspecting the network optimization is last stage of the network optimization. It is realized by using the baseline variance-covariance gained from the network optimization, the approximate coordinates of the network points and ephemeredes of GNSS satellite. In the stage, the baseline variance-covariance is admitted to be criterion matrix and some measures decided by users are used to test the network optimization by the baseline adjustment explained previous part of the paper.

Varied users whose demands are different each other can choose different methods for testing their optimized GNSS network. In this study, five different measures are proposed. The all measures are worked on a time span of baseline occupation because all GNSS users usually plan their GNSS measurements with respect to the time spans that are generally predicted according to baseline length. For example, in the IZDOGAP we were going to intend that the time spans were equal to 10 or 15 minutes for baseline length shorter than and equal to 20km and that the time spans were increased proportionally by baseline length for the baseline bigger than 20km.

All proposed measures on time span can be used to test GNSS networks throughout some campaigns imagined. In the campaigns, design matrixes in the Eqs.(1) are established easily by using the approximate coordinates and GNSS ephemeredes acquired by internet [1]. So throughout campaigns, all possible observation sessions, are planed by users, can be examined to test the network optimization results according to the proposed measures following equations one by one.

$$\underline{\delta}^{(k)} = \mathrm{vec}\left[\underline{K}_{\breve{b}} - \breve{\sigma}^2 / \sigma^2\, \underline{\Sigma}_\Delta\right] \qquad \underline{\Sigma}_\Delta = \sigma^2 \begin{bmatrix} q_{\Delta X \Delta X} & 0 & 0 \\ 0 & q_{\Delta Y \Delta Y} & 0 \\ 0 & 0 & q_{\Delta Z \Delta Z} \end{bmatrix} \qquad (19)$$

$$\breve{\sigma} = \hat{\sigma}\sqrt{F_{(n-3,n-m-3,1-\alpha)}} \qquad k=1,2,\ldots,b$$

Where, b is the number of baseline in the network and the number of sessions in the campaigns, $\underline{\delta}^{(k)}$ is differences vector between the baseline variance-covariance ($\underline{K}_{\breve{b}}$ from Eqs.(7)) and baseline criterion matrix ($\underline{\Sigma}_{\Delta}$), which is a sub matrix for a baseline after the network optimization, σ is a expected standard deviation after the network adjustment, $\hat{\sigma}$ is a expected standard deviation of float solution of baseline adjustment and $\breve{\sigma}$ is a expected standard deviation of fixed solution after ILS.

Measure-1: $\underline{\delta}^{T}\underline{\delta}^{(k)} \rightarrow \min \quad \underline{\Sigma}_{\Delta} = \sigma^2 \underline{P}_{opt}^{-1}$

Measure-2: $\underline{\delta}^{T}\underline{\delta}^{(k)} \rightarrow \max \quad \underline{\Sigma}_{\Delta} = \sigma^2 \underline{P}_{opt}^{-1}$

Measure-3: $\underline{\delta}^{T}\underline{\delta}^{(k)} \rightarrow \min \quad \underline{\Sigma}_{\Delta} = \sigma^2 \underline{I}$ (20)

Measure-4: $\mathrm{RDOP} = \mathrm{tr}\{\underline{K}_{\breve{b}}\}/\breve{\sigma} \rightarrow \min$

Measure-5: $\underline{K}_{\breve{b}}^{(k)}$

Where, $\underline{P}_{opt}$ is a sub weight matrix for a baseline from the network optimization (Eqs.15), RDOP is relative dilution of precision, $\underline{I}$ is the identity matrix with 3 by 3.

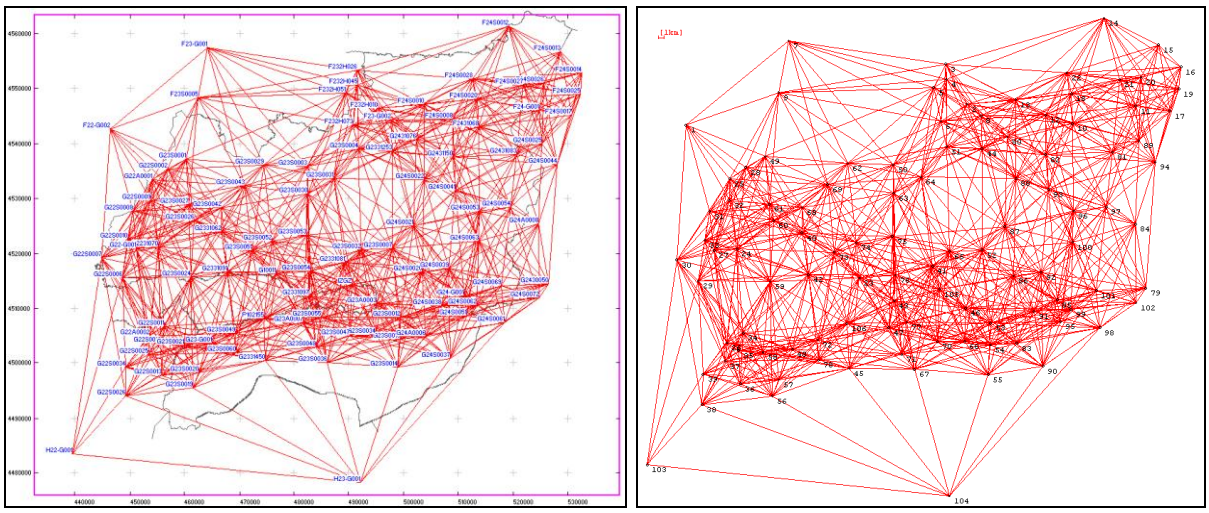
In this study, the baseline optimization using five different measures ensure to test the results (observation plan) of the network optimization for their validity in real life (Kurt, 2009; Kurt and Konak, 2009).

In the design stage, some standard deviation can be determined experimentally since real observation has been not carried out yet. In the paper, those were taken as $\sigma = 1.0\mathrm{cm}$ and $\hat{\sigma} = 0.3\mathrm{cm}$.

Examples

The coherence of the results of evaluating stage to those of designing stage was examined on IZDOGAP GNSS network. IZDOGAP is founded by Scientific Research Project Unit of Kocaeli University and IZGAZ.

The GNSS network has 106 points distributed at Kocaeli-İstanbul-Sakarya-Yalova-Bursa cities and 252 baselines observations after second order design according to Taylor-Karman structured criterion matrix and reliability optimization in the design stage. After the zero period GNSS observations of the designed network was completed on 14 July 2009, baseline adjustment and network adjustment were carried out. Moreover quality and reliability criterions of the network were computed and compared with those in which designing stage. And the results obtained are summarized on the network figures.



(a) **(b)**

Fig. 2. Maximum observation plan for the IZDOGAP GNSS network is according to names of the points **(a)** or according to point numbers **(b)**.

Design Stage of IZDOGAP Network

If the precision optimization (or second order design) was performed the maximum observation plan according to Eqs.(15), the first reduction was carried out iteratively on the plan.

a) Constructing Maximum Observation Plan for IZDOGAP Network

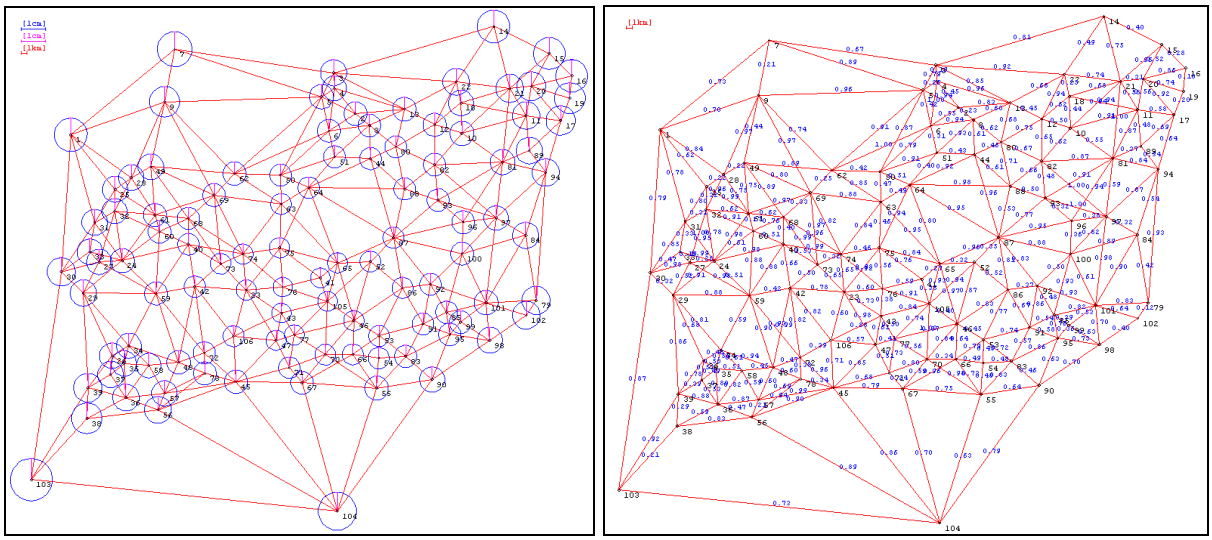
In the IZDOGAP GNSS networks, the maximum observation plan was established under a condition that all baselines in the network being shorter than 20km for ignoring the atmospheric effects on GNSS observables. The maximum observation plan on the network had b_{max}=1042 baselines (1011 baselines≤20km and 31 baselines >20km). In the network, lengths of baselines were changed with a range from $S_{min}=S_{(27-33)}$=2.381km to $S_{max}=S_{(104-103)}$=52.849km (Figure-2 and Table-1).

Table 1. Statistics of the IZDOGAP GNSS network.

Ranges for Baselines Lengths	**Maximum Observation Plan**	**After Precision and Reliability Optimization**
00km < Si <= 05km	54	54
05km < Si <= 10km	227	140
10km < Si <= 20km	730	46
SUM	**1011**	**240**
20km < Si <= 30km	24	9
30km < Si <= 40km	5	2
40km < Si <= 50km	1	0
50km < Si <= 60km	1	1
SUM	**1042**	**252**

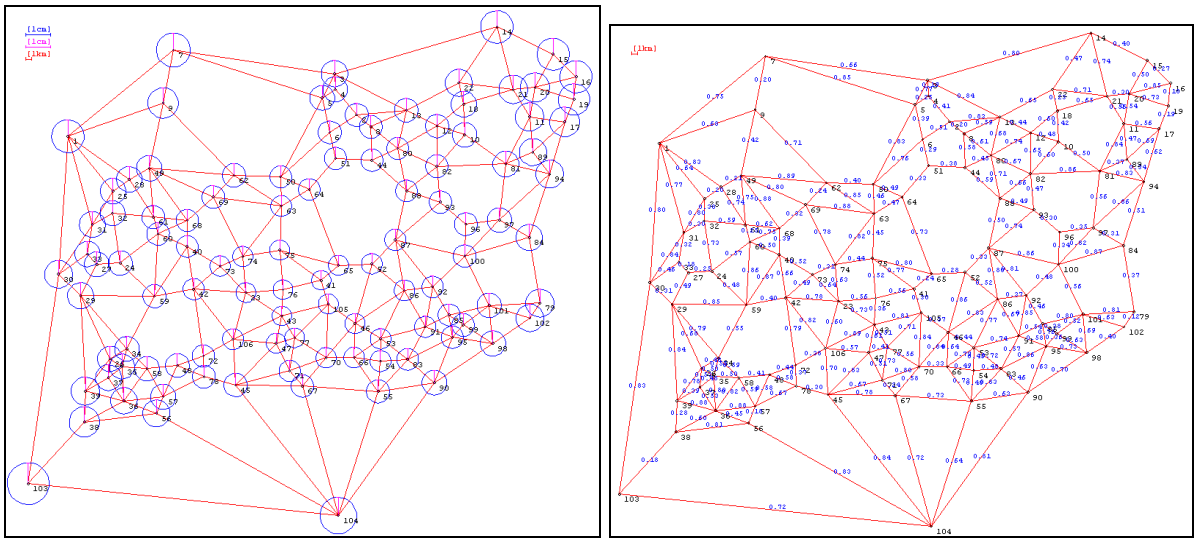
(b) The First and Second Reduction on IZDOGAP Network

The first reduction and the second reduction of the maximum plan was performed by Eqs.(15) iteratively (Figure-3, Table-2) and by Eqs.(16)-(18) (Figure-4, Table-1 and Table-2), respectively.



(a) **(b)**

Fig. 3, The first reduction of the maximum plans **(a)** Horizontal Error Ellipses (blue) and precisions of heights (pink), **(b)** Partial redundancy values of the observations.



(a) **(b)**

Fig. 4, The second reduction of the maximum plans **(a)** Horizontal Error Ellipses (blue) and precisions of heights (pink), **(b)** Partial redundancy values of the observations.

Table 2. Statistics for the first and second reduction stage of the maximum plan.

	Precision Optimization	**Time (h)**	**# of Obs. Removed** $p_i \leq 0$ || $p_i > 100$	**# of Obs. Remained** $0 < p_i < 100$	**Figures**
First Reduction	1. iteration	00:11:43	632	410	
	2. iteration	00:00:52	80	330	
	3. iteration	00:00:31	7	323	
	4. iteration	00:00:29	0	323	Figure-3
	Reliability Optimization	**Time**	$\mathbf{r_i \geq 0.90}$	$\mathbf{r_i < 0.90}$	
Second Reduction	5. iteration	~00:20:00	71	252	Figure-4
	Precision Optimization	**Time (h)**	**# of Obs Removed**	**# of Obs. Remained** $0 \leq p_i < 100$	
	6. iteration	00:00:19	0	252	Figure-4

Note: Computer used in this study has 4MB RAM and 1.7GHz CPU.

(c) Testing procedure for the reduced IZDOGAP network with baseline optimization

In reduced IZDOGAP GNSS network, the baseline optimization scenarios were used for a GNSS week and these are summarized in the Table-3 (Table-3). In the weeks both of two campaigns, variance-covariance matrices of every baseline were computed one by one at every session thorough the two imagined GNSS campaigns (Eqs.19-20). And, all measures of Eqs.(20) for baseline optimization were examined on the IZDOGAP network (Figure-5). It was seen that Measure-3 and Meausre-4 were gave almost same results for all sessions through the both campaigns. For this reason, the baseline optimization testing the results of the network optimization for IZDOGAP is summarized by using Meaures-1, Measures-2 and Measure-5 on the following figures in the paper.

Figure-6.a and 6.b, the error ellipses on the figures ensured to compare first and second measures after the baseline optimization in the Campaigns-I and in the Campaigns-II respectively. These were very good manner to examine the network optimization because of comparing best and worst solution among all sessions of both campaigns (Figure-6).

Table 3. Two GNSS campaigns imagined for testing the network optimization.

	Campaign I	**Campaign II**
Start of campaign	01-03-2009 Sunday / 08:00	26-04-2009 Sunday / 08:00
Finish of campaign	07-03-2009 Saturday/ 18:00	02-05-2009 Saturday/ 18:00
Ephemeris Files	YUMA Almanac Files in 1521 GNSS week	YUMA Almanac Files in 1529 GNSS week
Interval between two sessions	2200saniye(36.67dak)	2200saniye(36.67dak)
Number of sessions	252	252
Time Span for a Session	15 minutes	10 minutes
Epoch Interval	1 minute	1 minute
Working part in a day	24hours(3*8hours)	24hours(3*8hours)
Elapsed time during the baseline optimization	00:30:18 hours	00:20:45 hours

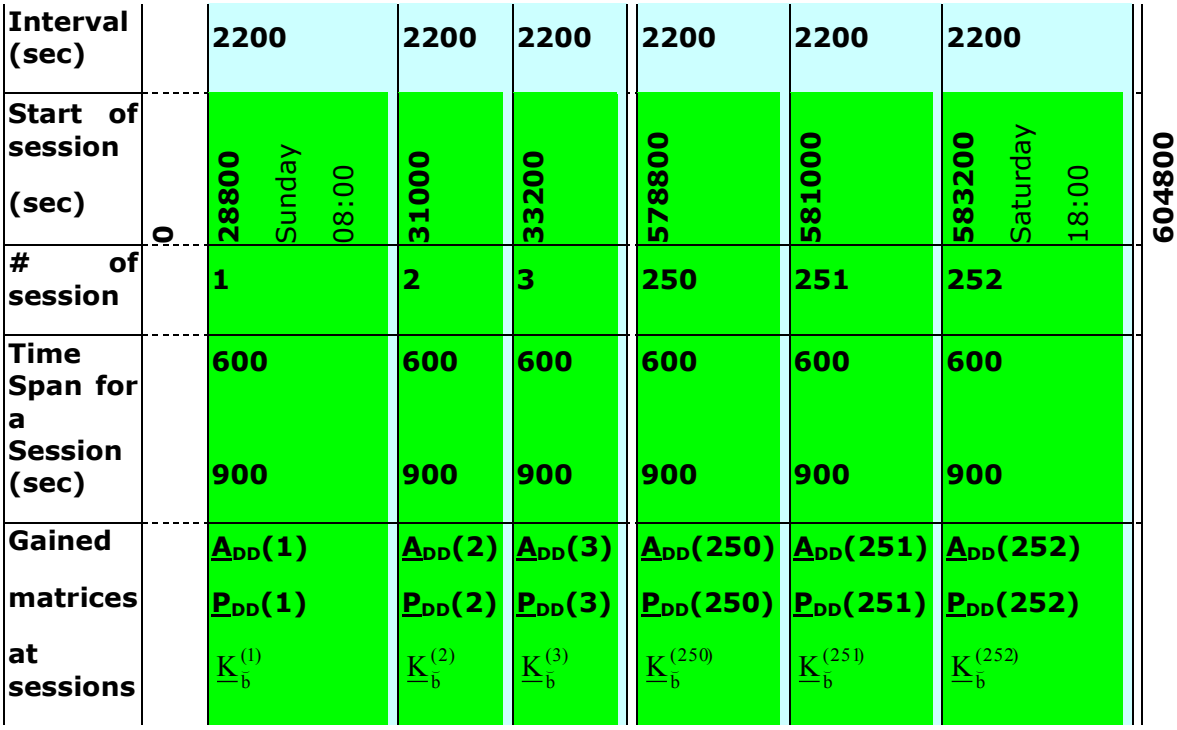
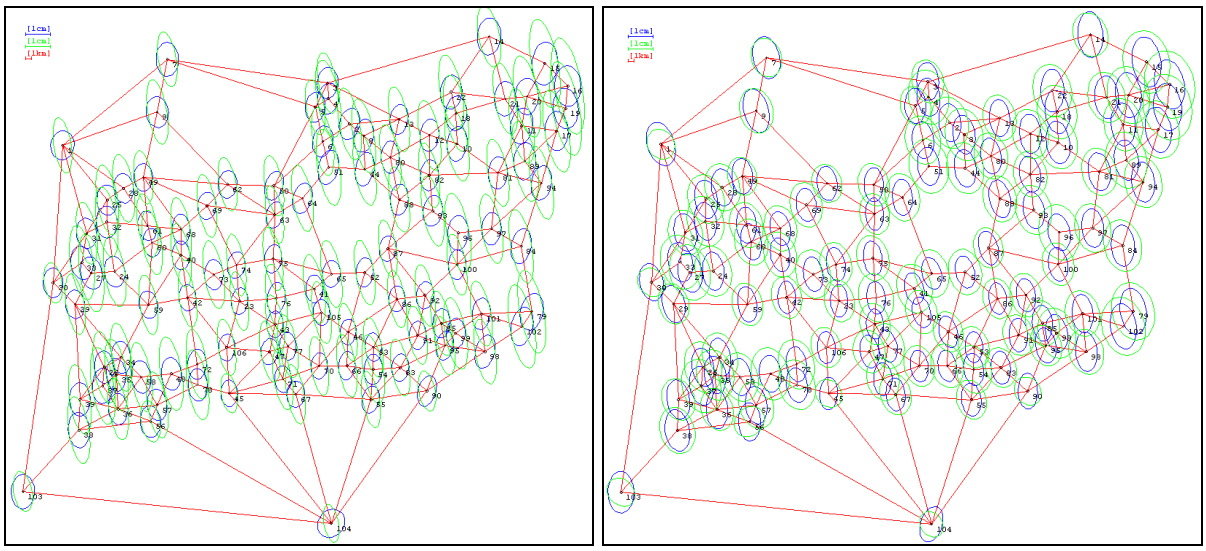
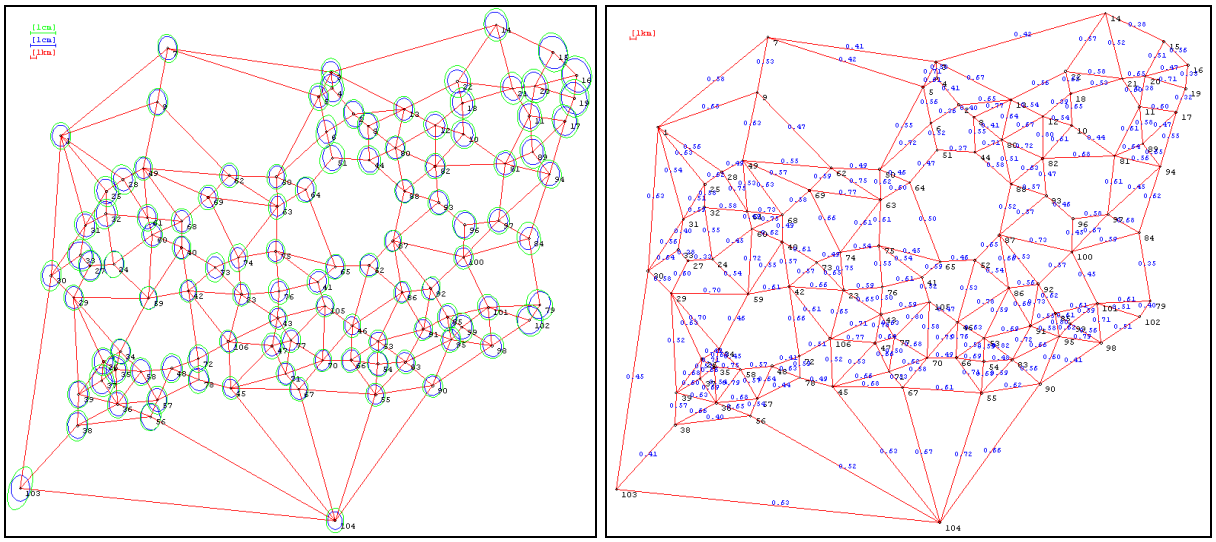


Fig. 5. Baseline optimization for testing procedure for the network optimization (the blue parts and green parts represent the intervals and time spans of sessions respectively).



(a) **(b)**

Fig. 6. Comparing measure-1 ($\underline{\delta}^T\underline{\delta}^{(k)} \rightarrow \min$) to mesure-2 ($\underline{\delta}^T\underline{\delta}^{(k)} \rightarrow \max$) for testing procedure by the baseline optimization for the network optimization in **(a)** for the campaign-I and **(b)** for the campaign-II.



(a) **(b)**

Fig. 7. (a) Comparing campaign-I (Blue) to campaign-II (green) according to the Measure-5. $\underline{K}_{\bar{b}}^{(k)}$, **(b)** Partial redundancy values of the observations.

Figure-7.a, the error ellipses on the figures ensured to compare two of the fifth measures in campaign-I and campaign-II.

The results of campaign-II are worse than those of the campaign-I (Figure-7.a). For this reason, in the Figure-7.b the values of r_i (i=1,2,..b) in campaign-II are used and all of them are greater than 0.25. Of course, it is clear that those of campaign-I are better than them.

In the measure-5, the variance-covariance matrix for baseline components for all sessions is computed according to Eqs.(7). For instance, variance-covariance matrix of the first baseline is equal to the variance-covariance matrix gained from the first

sessions, the second to the second and so on (Figure-5). So, all of the sessions in the campaigns are to be examined in this way. Therefore, the network optimization is to be examining via baseline optimization completely.

(d) Result for Design Stage

After baseline optimization, it was decided that the observation plan obtained from the network optimization could be applied in real life (Figure-4, Figure-5, Figure-6 and Figure-7) (Kurt, 2009; Kurt and Konak, 2009).

Evaluating Stage for Designed IZDOGAP Network after GNSS Observation

After the observation plan had been admitted to apply in reality, GNSS observations were carried out on the IZDOGAP network. The condition on baseline observations and adjustment are summarized at Table-4 (Table-4).

(a) Baseline Adjustment

Only, the baseline adjustment were realized in the commercial software (Table-4), and baseline components and their variance-covariance matrix were obtained by the software from Eqs.(1) to Eqs.(7). And, Loop closures on the network have been controlled every end of working day.

(b) Network Adjustment for Real Observation

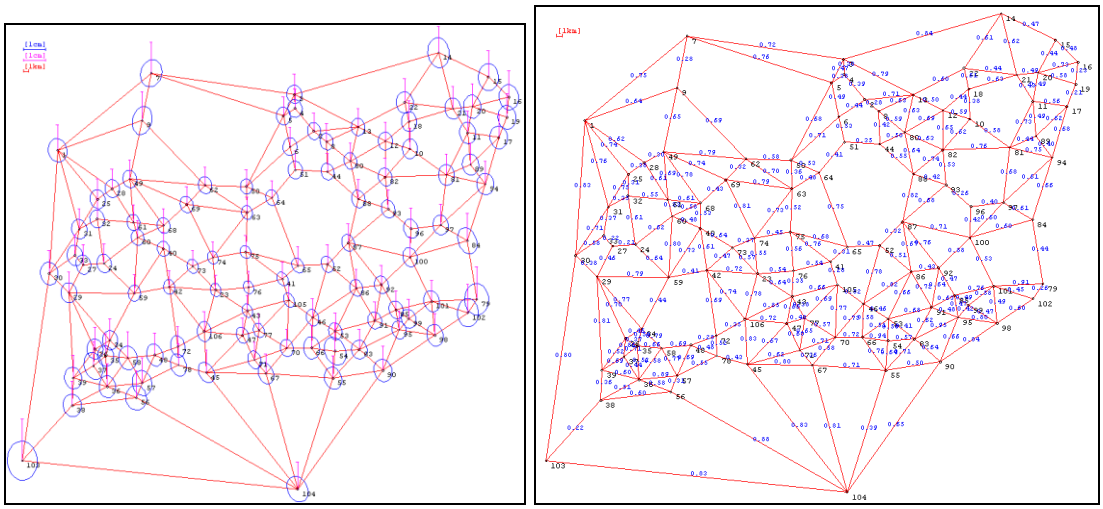
Network adjustment for the IZDOGAP Network was realized under inner constraint solution from Eqs.(8) to Eqs.(13). In the solution, number of the network points was p=106, number of unknowns (components of the network points) was u=3p=318, number of the baselines was b=252, number of observations (components of the observed baselines) was n=3b=756, and number of the datum defect was d=3.

Table 4. Observables and adjustments conditions and statistics for baselines.

	Conditions for Baseline Observations and Adjustment
Time spans of the sessions are between	15-45 minutes (the range is to be change with baselines length)
Elapsed time for all baseline observations is	1 month (the finished date is 14 July 2009)
Number of sessions is	92
Number of receivers in a session is	generally 4 (6 for only six day)
Receivers type used are	TOPCON TPS Hiper Series (GPS+GLONASS)
Baseline adjustment software used is	TOPCON Pinnacle 1.00
Ephemeris used in baseline adjustment are	Broadcast ephemeredes
Baseline adjustment type is	single baseline {because of number of GNSS receivers (4) and a lot of sessions (92)}
Mask angle is chosen as	15°

Baselines, which were linear independent and observed twice or more times, were adjusted before the final adjustment (or the network adjustment). And than, the final

adjustment was performed, and the error ellipses (Figure-8.a) and reliability measures (Figure-8.b) for the network were computed. The results of network adjustment on IZDOGAP network are summarized and compared with the results that were obtained from the designing stage on following figures.



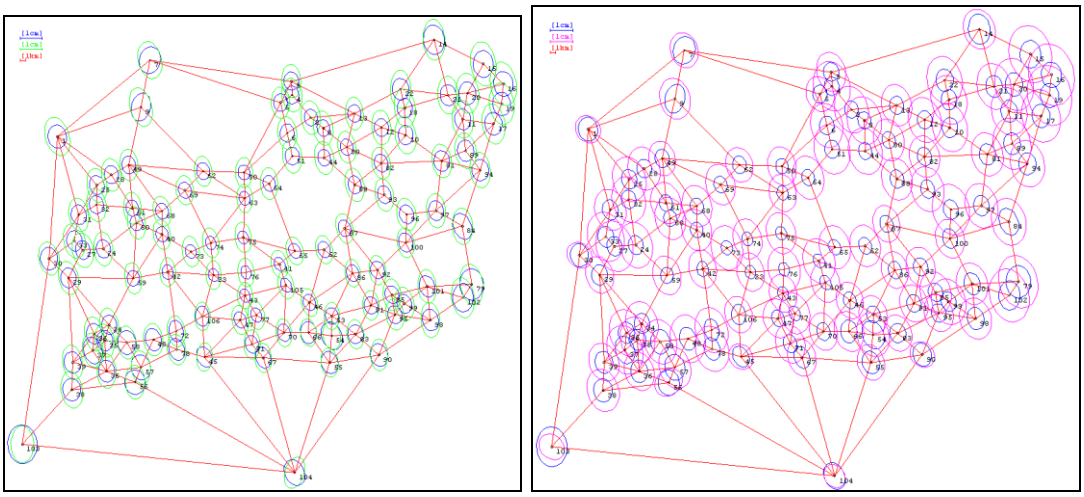
(a) **(b)**

Figure-8. (a) Horizontal Error ellipses (blue) and precisions of heights (pink), **(b)**Partial redundancy values of the observations (blue).

c) Comparing the evaluation stage to the design

Figure-9.a shows us a comparison between first measures ($\underline{\delta}^T\underline{\delta}$→min) in the campaign-II (imagined) that was worst in the design stage and the evaluating (real). Figure-9.b shows us the other comparison between second measures ($\underline{\delta}^T\underline{\delta}$→max) in the campaign-II and the evaluating, too.

Figure-10 shows us a comparison between the network optimization that was obtained from the precision and reliability optimization and the evaluating stage that was realized by the network adjustment from the real observations on the IZDOGAP network.



(a) **(b)**

Figure-9. (a) The green colored ellipses represent the result of first measure, **(b)** the pink colored ellipses represent the result of second measure in the Campaign-II and blue colored ellipses represent the result of real observations on the both figures

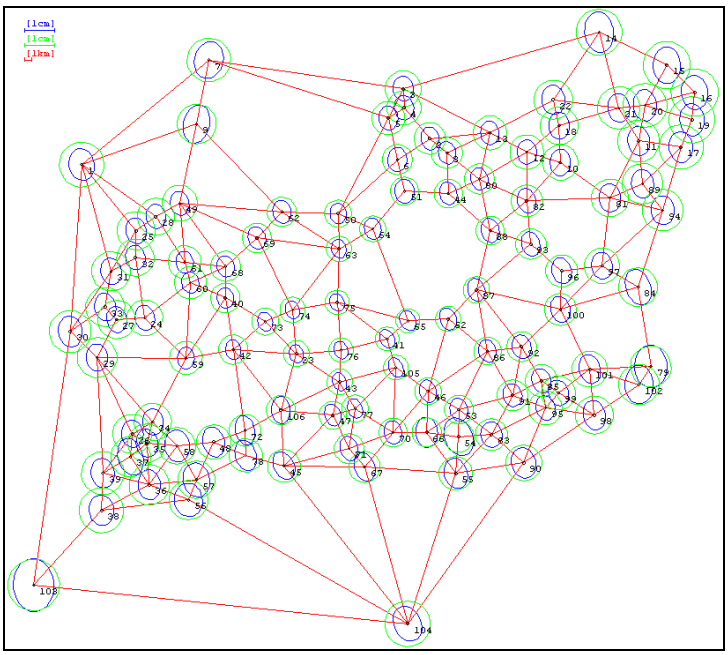


Fig. 10. Blue colored ellipses represent the result of real observations. Green colored ellipses represent the result of the network optimization (precision and reliability optimization).

CONCLUSIONS

In this contribution, we show a standard way to establish GNSS networks and how we take under control them before not carrying out GNSS observations. Furthermore, validation of the proposed way was applied on IZDOGAP GNSS network.

It is clear that, observation plan on a GNSS network should be taken account before GNSS observations campaign not started.

In the designing stage, while the network optimization for constructing observation plan is necessary, the baseline optimization is able to be useful for inspecting the result of the network optimization.

Yet, if the baseline optimization is carried out on a GNSS network acquired from the network optimization, a network designer will be able to determine the quality and reliability of her/his network before the observations on the working area.

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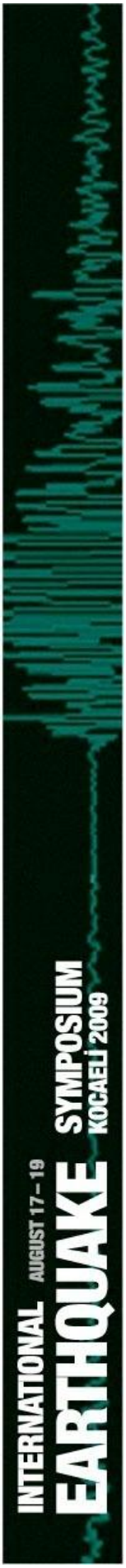

DISASTER-RISK MANEGEMENT

AFET YÖNETİMİ

DEFICIENCIES AND PROBLEMS THROUGH EMERGENCY ACTIONS AFTER EARTHQUAKES

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Key Words: Earthquake, Emergency Action, Hospital, Medical Buildings

ABSTRACT Türkiye is affected from natural disasters such as earthquake, landslide, water flood, fire, snow slide, underground water rise, according to meteorological effects and its location and ground properties. Earthquake is the most destructive disaster, devoted to researches and statistics on building damages, magnitude of disaster, types and significance of losses (life, good etc). The significance of emergency action arises, depending on to numerical data of life losses and injuries notably after earthquake. 1999 Kocaeli Earthquake is such an important indicator of increase of injury and life losses quantities and importance of emergency actions after a big earthquake. Main reasons of increase of injury and life losses quantities are building damages and problems and/or deficiencies through emergency actions. Some medical buildings' (hospitals) which were not regulated to "2007 Regulations about Buildings on Disaster Regions" (2007 Afet Bölgelerinde Yapılacak Yapılar Hakkındaki Yönetmelik) already deficient quantities were decreased by damages and collapses after Kocaeli Earthquake. After earthquake, necessary conditions such as treatments of injurers and protection of corpses couldn't provide in this region's hospitals due to their deficient quantity. In other words, deficient hospital quantity, increases panics, prevents survivals by emergency actions and finally causes the life losses. The deficiency of some disciplines' coordination arises as well as the deficiency of hospital quantity. Providing this coordination is important to make the emergency actions on necessary conditions and as fast as possible after earthquake. The main reason of deficiencies and problems after this Kocaeli Earthquake is the absence of an analytical system. Consequently, studies should be made devoted to medical buildings system after earthquake. This system should be taken on as a priority situation, to solve the treatment problems of injured survivors and emergency action needed people, to minimize the losses and invalidities and to prevent the epidemic diseases. The problems and deficiencies, which were mentioned before, after Kocaeli Earthquake will be studied by analysis and determinations in the scope of this paper. In this context, it is thought that these analysis and determinations will be efficacious to prevent the instauration of problems after the possible future earthquakes.

INTRODUCTION

Disaster is defined as a natural and human basis event which causes physical, economical and social losses for people, interrupts human life and activities therefore affects societies (Ergünay, 1996).

On the earth a lot of region is affected from natural disasters according to their location, ground properties, meteorological effects and strategically reasons. When we look at Türkiye, it is affected from natural disasters such as earthquake, landslide, water flood, fire, snow slide, underground water rise, according to meteorological effects and its location and ground properties (Ergünay, 1996). In the last 60 years the 500.000 of 600.000 structures were damaged from earthquakes, 120.000 of 600.000 structures were damaged from landslide and water flood. In this time when the building damages statistics are taken into account the damages of % 62 are occurred by earthquake, % 15 of occurred by landslide, % 12 of occurred by water flood, % 5 of occurred by rock fall,

% 4 of occurred by fire and % 2 of it occurred by snow slide which are the meteorological source and other disasters caused events (Songür, 2000). According to researches of General Directorate of Disaster Affairs Earthquake Research Department there have been one destructive earthquake occurred in every 9 months and in a year there have been 25 important water flood and 50 landslide occurred in Türkiye. The disasters occurred in Türkiye in between 1992-2003, shown in the Table-1, the total of injured people number were 55.802, and the total of dead toll were 19.964 due to the disasters (JICA, 2004).

Table 1. The disasters occurred in Türkiye in between 1992-2003, (JICA, 2004).

DATE	EVENT	HOUSELESS	AFFECTED NUMBER OF PERSONS	INJURED PEOPLE NUMBER	DEAD TOLL
13.03.1992	Earthquake (Erzincan)	95,000	250,000	3,850	653
1992 (14 Event)	Snow Slide (Güneydoğu)	11,600	30,000	53	328
13.07.1995	Muddy Flood (Senirkent-	2,000	10,000	46	74
01.10.1995	Earthquake (Dinar)	40,000	120,000	240	94
04.11.1995	Water Flood (İzmir)	6,500	300,000	117	63
21.05.1998	Water Flood (B.Karadeniz)	40,000	1,200,000	47	10
27.06.1998	Earthquake (Ceyhan)	88,000	1,500,000	1,600	145
17.08.1999	Earthquake (Kocaeli)	675,000	15,000,000	43,953	17,480
12.11.1999	Earthquake (Düzce)	35,000	600,000	4,948	163
03.02.2002	Earthquake (Sultandağı)	30,000	222,000	327	42
01.05.2003	Earthquake (Bingöl)	45,000	245,000	520	177
TOTAL		**1,078,200**	**19,494,300**	**55,802**	**19,964**

Earthquake is the most destructive disaster in Türkiye according to the statistics. Due to the earthquake region map which is currently in effect, the % 96 of Türkiye's land is in different ratio of risk of erthquake region and % 98 of population are living in this regions (M.A.K.R., 1997).

In this case emergency actions become more of an issue after earthquakes due to the number of injured people and dead toll. If health buildings are damaging which are necessary for emergency actions and treatment, it makes interventions difficult or impossible after earthquake. Besides this in Kocaeli and in Istanbul hospitals at existing situation they have got deficiencies bed numbers, after an earthquake increasing number of injured people and unusable situation of health buildings the bed numbers are decreasing. So for the injured people treatment does not provide.

DEFICIENCIES AND PROBLEMS THROUGH EMERGENCY ACTIONS AFTER EARTHQUAKES

Last century dead toll due to earthquake is over 83,000, number of injured people is over 120,000 in Türkiye (Şahin ve Sipahioğlu, 2002). In the Table-2 the earthquakes occurred

in Türkiye in between 1983-2009 with a magnitude over 5.0 can be seen. Only in the earthquakes occurred in Türkiye in between 1983-2009 with a magnitude over 5.0 the dead toll is 20.243, number of injured people is 58.180 [1]. 1999 Kocaeli Earthquake is such an important indicator of increase of injury and life losses quantities after a big earthquake. Only in Kocaeli Earthquake the dead toll was 17.480, the number of injured people was 43.953.

Table 2. The earthquakes occurred in Türkiye in between 1983-2009 with a magnitude over 5.0, [1].

DATE	EVENT	MAGNITUDE	INJURED PEOPLE NUMBER	DEAD TOLL
30.10.1983	Erzurum	6.8	1,142	1,155
18.09.1984	Erzurum-Balkaya	5.9	35	3
06.06.1986	Malatya-Sürgü	5.6	20	1
07.12.1988	Kars-Akyaka	6.9	11	4
13.03.1992	Erzincan	6.8	3,850	653
01.10.1995	Dinar	6.0	240	96
14.08.1996	Çorum-Amasya	5.4	6	-
27.06.1998	Adana-Ceyhan	6.3	1,041	145
17.08.1999	Kocaeli	7.4	43,953	17,127
13.09.1999	Kocaeli	5.8	-	-
20.09.1999	Marmara Denizi	5.0	-	-
05.10.1999	Marmaris-Muğla	5.2	-	-
12.11.1999	Bolu-Düzce	7.2	4,948	845
14.02.2000	Bolu-Yığılca	5.1	-	-
21.04.2000	Denizli-Honaz	5.0	-	-
06.06.2000	Çankırı-Orta	6.1	1,766	2
23.08.2000	Sakarya-Hendek	5.8	9	-
15.12.2000	Afyon-Sultandağı	5.8	547	6
27.01.2003	Tunceli-Pülümür	6.2	7	1
10.04.2003	İzmir-Urla	5.6	-	-
01.05.2003	Bingöl	6.4	520	176
25.03.2004	Erzurum-Çat	5.1	20	9
02.07.2004	Ağrı-Doğubeyazıt	5.1	32	18
03-04.08.2004	Muğla-Gökova Körfezi	5.0 - 5.4. - 5.0	-	-
11.08.2004	Elazığ-Sivrice	5.5	12	-
25.01.2005	Hakkari	5.5	5	2
12.03.2005	Bingöl-Karlıova	5.7	16	-
14.03.2005	Bingöl-Karlıova	5.9	-	-
20.12.2007	Ankara- Bala	5.6	-	-
TOTAL			**58,180**	**20,243**

Main reasons of increase of injury and life losses quantities are building damages and problems and/or deficiencies through emergency actions. Some medical buildings' (hospitals) which were not regulated to "2007 Regulations about Buildings on Disaster Regions" (2007 Afet Bölgelerinde Yapılacak Yapılar Hakkındaki Yönetmelik) already deficient quantities were decreased by damages and collapses after Kocaeli Earthquake.

After Kocaeli and Düzce Earthquakes İzzet Baysal State Hospital, Düzce State Hospital, Faculty of Medicine, Kocaeli State Hospital, SSK İzmit Local Hospital, SSK Kocaeli

Hospital and SSK Karamürsel Hospital were severely damaged and being unusable (Doğramacı, Koçak ve Ekiz, 2003), [2].

In Kocaeli hospitals at existing situation they have got deficiencies bed numbers, after an earthquake increasing number of injured people and the damaged and unusable situation of health buildings the bed numbers are decreasing. So for the injured people treatment and protecting dead people in hygienic conditions didn't provided. 20.000 injured people were transported to neighborhood province hospitals by air line, sea route and land route after the earthquake in a situation of emergency actions were crucial [3]. After Kocaeli Earthquake in the first ten days when the health buildings were checking it was detected that the % 20 of 51 health centers were furnishing services at damaged buildings and the % 25 of 51 health centers were furnishing services out of health centers [2]. So tents were pitched and containers were constructed instead of damaged health buildings for furnishing health services. But it can be said that this tents and containers were not suitable for furnishing health services in terms of health conditions.

In the Table-3 the "Necessary-(N)" and "Existing-(E)" number of beds in Kocaeli Hospitals according to the state information listed by years can be seen. "Necessary" number of beds are the needed bed numbers which is detected by the Ministry of Health for 10.000 (ten thousand) person. And the "Existing" numbers of beds are the bed numbers which is in use [2, 4]. In the table it can be seen that the number of beds are deficient even in a non-disaster situation in Kocaeli Health Buildings.

Table 3. The "Necessary-(N)" and "Existing-(E)" number of beds in Kocaeli Hospitals according to the State Information listed by years, [2, 4].

Corporation	**1996**		**2000**		**2001**		**2003**		**2006**
	(N) Number of Beds	**(E) Number of Beds**	**(N) Number of Beds**	**(E) Number of Beds**	**(N) Number of Beds**	**(E) Number of Beds**	**(N) Number of Beds**	**(E) Number of Beds**	**(E) Number of Beds**
Ministiry of Health	800	649	800	601	800	610	875	603	
SSK	763	829	763	639	763	695	863	917	
University	370	129	370	265	370	268	370	391	
Municipality	20	20	60	40	60	20	60	33	
Private	20	11	85	75	126	97	156	132	
Total	**1973**	**1638**	**2078**	**1620**	**2119**	**1690**	**2324**	**2076**	**2382**

In Istanbul excess of population and the earthquake could be occurred at any time it is thought of that the similar problems of Kocaeli will be happened due to the absence of an analytical system (managerial system) covering a time which is starting before earthquake and ending after earthquake. Next 30 years an earthquake with % 62 ± % 12 probabilities is awaited in Marmara Region according to the academic studies and their statistics made in 2002 [5]. The social and economical risks if there will be an earthquake (magnitude of 7.5) in Istanbul are expected; ~ 70.000 dead toll, 120.000 seriously injured, 400.000 minor injured and ~ 30 billion USD dollar economical losses [6]. Due to this scenario if a big earthquake could occur within 30 years in Marmara Region, it is important to know that the health buildings are adequate or inadequate for emergency actions in Istanbul. In the Table-4 the "Necessary-(N)" and "Existing-(E)" number of beds in Istanbul Hospitals according to the state information and the population listed by years can be seen. The existing bed number of Istanbul was 27.774 in 2006 when its population is over ten billion [4]. It has been seen that how the number of existing beds are deficient in proportion to population.

Table 4. The "Necessary-(N)" and "Existing-(E)" number of beds in Istanbul Hospitals according to the State Information listed by years, [4].

Year	Population	(E) Number of Beds	(N) Number of Beds	Bed Numbers for 10.000 People
2001	10,242,434	27,210	34,688	33,9
2002	10,553,738	27,827	34,831	33
2003	10,867,904	26,949	34,746	32
2004	11,184,865	28,084	35,153	31,4
2005	11,331,964	28,039		29,7
2006	12,573,836	27,774		22,1

Some of the health buildings in the table were not regulated to "2007 Regulations about Buildings on Disaster Regions" (2007 Afet Bölgelerinde Yapılacak Yapılar Hakkındaki Yönetmelik) so they were intended to reinforced. In the Table-5 the health buildings which will be reinforced / rebuild within ISMEP Project in Istanbul can be seen (Elgin, 2007).

Table 5. The Health Buildings which will be Reinforced / Rebuild within ISMEP Project, (Elgin, 2007).

Building Type	The number of buildings that are planned to be reinforced	The buildings that have been reinforced since September 2007
Hospital Buildings	130	4
Polyclinic Buildings	14	7
TOTAL	**144**	**11**

As it is seen in the table most of the health buildings are not reinforced yet. When an earthquake could occur at any time these health buildings will not be used and big problems will be occurring at the emergency actions. If we look at the existing damaged health buildings, existing bed numbers of hospitals and the population of Istanbul, it is seen that lack of health buildings for emergency actions will be very big extent when an earthquake will occur in Istanbul (magnitude over 7.0) in 30 years.

CONCLUSION

If health buildings are damaging which are necessary for emergency actions and treatment, it makes interventions difficult or impossible after earthquake. At the same time declining of the number of health buildings causes increasing of chaos in the environment and causes people to die who could survive with emergency actions. In Kocaeli earthquake dead people were waiting at the skating ring and the other areas because of the deficiencies of surgeries in the health buildings. It is not appropriate in terms of health conditions and it causes epidemics of diseases in the disaster area.

The problems likely to occur in health buildings after an earthquake are;

Deficiencies of health building numbers

The unusable situation, which occurred after earthquake, of the existing already damaged health buildings due to the lack of reinforcement

Deficiencies of essential units in health buildings like surgery, morgue and etc.

The damaged and unusable situation of some existing health buildings after an earthquake

The lack of necessary hygienic conditions in the health buildings

These problems are creating new disasters in the disaster areas. The inadequate building methods and systems to build the temporary health buildings which can offer necessary conditions after an earthquake for the replacement of damaged existing health buildings is one of the main problems. After Kocaeli Earthquake the shortcomings of emergency actions and deficiencies of systems were unfolded. As for Istanbul it is thought of for the expected potential earthquake larger problems will be occur because of population density. Consequently treatment problems of injured people and the people who need emergency action must be resolved immediately and a health constitution system must be established for minimizing life losses and disability, prevention of epidemic diseases that may occur after earthquake.

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MÜZELERDE YAPISAL OLMAYAN MALZEME KAYNAKLI RİSKLER

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1-İBB-Deprem Risk Yönetimi ve Kentsel İyileştirme Daire Başkanlığı

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Anahtar Kelimeler: Müzelerde Afet Yönetimi, Risk Azaltma, Kültürel Miras Risk Analizi, Kırılganlık.

ÖZ Afetler, ülkelerin kalkınmasını etkileyen ciddi bir sorun oluşturmaktadır. Afetler içinde de yapısal hasar görebilirlik ile can ve mal kayıplarına en fazla neden olan birincil afet depremdir. Tüm dünyayı tehdit eden potansiyel deprem riski, Türkiye'nin de %98 inde bulunmaktadır. KAFZ (Kuzey Anadolu Fay Zonu) ve DAFZ (Doğu Anadolu Fay Zonu) büyük deprem üretme aktivitesine sahiptir.

Yüzyıllardır önemli bir tarihi geçmişe sahip ve tüm insanlığın ortak mirası olan İstanbul, bünyesinde pek çok tarihi eseri (taşınır/ taşınmaz) barındırmaktadır. Bu eserlerin bulunduğu müzelerdeki olası risklerin azaltılması göz ardı edilmemesi gereken bir konudur. Son yıllarda gelişmiş ülkelerdeki müze yapılarında risk azaltıcı modern teknolojiler üzerinde yoğun çalışmalar yapılmaktadır.

Müzeler, tarihi eserleri geçmişten geleceğe taşıyan ve pek çok eseri bünyesinde barındıran kurumlardır. Müzelerde yer alan kültür mirasının koruma altına alınması ve olası afet risklerinin en aza indirilmesi, bu amaç için tüm imkanların seferber edilmesi sorumlulukla ele alınması gereken öncelikli bir konudur. Müzelerin olası bir afetin ardından da operasyonel faaliyetlerini kesintisiz devam ettirmesi sağlanmalıdır.

Yeni müzelerin inşasında ve halen faaliyetlerine devam eden mevcut müzelerde yapılacak olan yenileme ve risk azaltma çalışmalarında afet odaklı (deprem, yangın, sel vb.) analizlerin yapılması gereklidir. Bu çalışmada müze binalarında taşıyıcı özellikte bulunmayan "yapısal olmayan malzemeler" e yönelik riskler incelenerek, koruyucu tedbir önerileri sunulmuştur.

GİRİŞ

Binalarda farklı deprem büyüklüklerine göre meydana gelen sarsıntı sonucu yapısal olmayan malzemenin hasara uğraması, binanın pek çok fonksiyonunu işlevsiz hale getirmektedir. Önemsenmeyen ve maddi değeri çok az olabilecek önlemler alınmadığında maddi değeri çok büyük maddi hasarlara can ve mal kayıplarına neden olabilmektedir. Bu nedenle binalarda taşıyıcı yapı elemanlarıyla birlikte yapısal olmayan malzemelere de yönelik kırılganlık analizlerinin yapılması gereklidir. Bu analizlerden elde edilecek veriler doğrultusunda malzemeler, bina içinde bulunan objelerin ve insanların güvenliği açısından değerlendirilerek risk azaltma çalışmalarında kullanılmalıdır. Tüm riskler tanımlanarak risk etki derecelerine göre öncelikler belirlenmelidir.

Hastane gibi yapısal olmayan malzemelerin fazlasıyla kullanıldığı binalarda, bu malzemelerin maliyetleri bina maliyetinden fazla olabilmektedir. Müze gibi binalarda ise müze bünyesinde bulunan paha biçilmez eserlerin değeri bina maliyetinden yüksek olabilmektedir. Nasıl bir hastanenin afet anında faaliyetlerine kesintisiz devam etmesi beklenirse müzelerin de geçmişten bugüne kadar ulaşan tarihi değerleri aynı özenle gelecek kuşaklara aktarması beklenmelidir. Yapısal olmayan malzemelerin hasarları sergilenen eserler, müze personeli ve ziyaretçiler için potansiyel risk oluşturmaktadır. Bu nedenle yapısal olmayan malzemelere yönelik zarar azaltma çalışmalarına önem verilmeli, bu konuda metodolojiler ve teknikler geliştirilerek, titizlikle uygulanmalıdır.

Ayrıca müzelerin kesintisiz faaliyetlerine devam etmesi özellikle çocukların afet sonrası rehabilitasyon sürecinde müze ziyaretleri ve sanat terapileri aktivitelerinin sürekliliği açısından büyük önem taşımaktadır.

Türkiye'de ulusal ve uluslararası düzeyde konu ile ilgili çalıştaylar ve konferanslar düzenlenmektedir. 2007 yılında Müzelerin Depremden Korunması Uluslararası Konferansı İstanbul'da gerçekleştirilmiştir. (www.eqprotection-museums.org) Bu etkinliklerle konuyla ilgilenen otoritelerin, uzmanların dikkatini müzeleri tehdit eden deprem ve diğer doğal afet tehlikelerine çekebilmek ile bu afetlerin verebileceği risklerin en aza indirilmesi doğrultusunda bilinçlenme düzeyinin artırılması amaçlanmıştır.

Bu çalışmalar, ülkemizde tarihi objelerin ve müzelerin depremden korunmasına yönelik yapısal olmayan tedbirlerin alınmasına katkı sağlamışsa da uygulamalar henüz ulusal çapta istenilen düzeye ulaşmamıştır.

RİSK AZALTICI PLANLAMA:

Müzelerde risk değerlendirilmesi yapılırken, bu tehlikelerin kaynakları, bunlardan kimlerin zarar görebileceği, sonucundaki etkiler öngörülmelidir. Müzeler için hazırlanacak acil durum planlamasında öncelikle;

- Anahtar alanların belirlenmesi
- Müzenin bulunduğu alanın zeminin ve binanın durumu
- Ziyaretçi ve personelin can güvenliğini sağlayıcı önlemler
- Objelerin hasar riskini en aza indirici önlemler
- Ziyaretçi, personel ve müze objelerinin tahliye planlarının hazırlanması, konuları vurgulanmalıdır.

Mevcut müzelerde risk azaltma önlemlerini, yapısal ve yapısal olmayan önlemler olmak üzere iki kategoride toplamak mümkündür.

Yapısal önlemler; yapıyı oluşturan ana elemanların güçlendirilmesi ve diğer risk azaltıcı önlemlerin alınmasını içerir. Müzelerde yapısal olmayan tedbirleri ise;

- Ziyaretçi ve personele uygulanacak,
- Objelere uygulanacak,
- Yapıya uygulanacak tedbirler olarak sıralayabiliriz.

Yapıya uygulanacak önlemlerde tüm risk faktörleri göz önüne alınmalıdır. Yapının bulunduğu alanın deprem ve yangın gibi afetlere açık bir alan olup olmadığı analiz edilmelidir. Yapıda optimum dizaynı sağlayacak kriterler belirlenmelidir.

Öncelikle yapı sağlığının bir deprem mühendisi tarafından analiz edilmesi ve bu sonuca göre önceliklerinin saptanarak uygulama stratejilerinin ortaya konması gereklidir.

Yapısal olmayan malzemeler açısından ise "Müze kırılganlık analiz çalışması" ile müze binasının olası zayıf noktaları analiz edilerek, risk haritaları hazırlanmalıdır. Yapısal olmayan sistemlerin sağlığı özellikle sergileme ve depolama alanı gibi anahtar alanlarda izlenmelidir. Riskler, değerlendirmeleri içeren analiz formlarına işlenmelidir. Analizler kolay, orta ve zor uygulanabilir olmak üzere 3 şekilde kategorize edilmelidir (Ertürk, Sungay, 2004).

Bu çalışmada objeler ve binadaki yapısal olmayan malzemelerle ilgili anahtar alanlar olarak belirlenen müze bölümlerindeki riskler ele alınmıştır.

YAPISAL OLMAYAN RİSK ANALİZİ

DEPREM RİSKİ

Bir binada yapısal olmayan malzemeler; tavan kaplama malzemeleri, pencereler, ofis ekipmanları, dolaplar, raflar, ısıtma, havalandırma gibi hava şartlarını düzenleyici sistemler ve ekipmanları, elektrik ekipmanı, mobilyalar, lambalar, yangın söndürme sistemleri, su tankları, jeneratörler, duvarlara monte edilmiş elektronik aletler, vitrinler, işaret levhaları, panolar, bilgisayar ekipmanları vb. olarak tanımlanabilir. Bu malzemelerin bir kısmı bina inşaatının hemen akabinde binaya monte edilirken, bilgisayar, mobilya gibi malzemeler de sonradan kullanıcılar tarafından binaya getirilmektedir.

Yapısal olmayan bileşenler açısından müze yapısı ele alındığında depolama ve sergileme alanları ön plana çıkmaktadır. Öncelikle yaşam güvenliği ikincil olarak da objelerin güvenliği dikkate alınmalıdır. İyi sabitlenmemiş tavan elemanları, yangın, havalandırma ve ışıklandırma için kurulan kablolalama sistemleri vb insan yaşamını tehdit etmekle beraber objelerin de geri dönülmez hasarlarına neden olabilmektedir. Ağır vitrinlerin ve içindeki objelerin devrilmesi, camların kırılması, tavan sistemlerinin çökmesi, duvarlarda sergilenen objelerin yere düşmesi, yangın sistemlerinin yetersiz olması, yangına dayanıksız malzemenin kullanılması, düşen precast beton paneller, hasara uğrayan duvar veya korkuluklar (merdiven ve giriş-çıkış kapılarında), duvar yüzeylerine monte edilen malzemenin düşmesi risklerden bazılarıdır.

Malzemelerin sergilendiği kaideler ve etraflarında bulunan yapısal olmayan malzemeler objenin hasar görebilirliği göz önüne alınarak tasarlanmalıdır. Özellikle de müze olarak kullanılan tarihi binalarda bu tür sorunlar ciddi riskler oluşturmaktadır. Zira yapısal olamayan elemanlar sonradan yapıya montaj yapıldığından bilimsel verilere dayanarak kullanılması ve sürdürülebilirliğinin sağlanması gereklidir. Amerika Federal Acil Durum Yönetimi Teşkilatı olan FEMA'nın (Federal Emergency Manegement Agency) hazırladığı Yapısal Olmayan Deprem Hasarı Risklerinin Azaltılmasına yönelik hazırladığı raporlarda ve Amerika bina yapım kodlarında; yapısal olmayan pek çok malzemenin montajında mühendislik bilgisi gerektirdiği belirtilmiştir (Fierro, Freeman, Perry, 1995). Standartlarda yapısal olmayan malzemeler, bina tesisat üniteleri, mimari malzemeler ve mobilyalar başlıklarında değerlendirilmiştir. Buna göre dizel bir yakıt tankının montajında mühendislik gerektirirken, duvara su ısıtıcısı montajında ise gerektirmediği uygulamalarla açıklanmıştır.

Yapısal olmayan malzemelerin uygun olarak seçilmemesi ve montajının önemsenmeyişi ile her geçen gün gelişen teknolojilerle ortaya çıkan yeni malzemelerin yapıda kontrol edilememesi nedeniyle yaşadığımız depremlerde ciddi kayıplar meydana gelmektedir. İşçilik kalitesi, malzemede yaşlanma nedeniyle oluşan hasarlar da bunu tetiklemektedir. Ülkemizden bir istatistik verecek olursak 1999 Kocaeli depremindeki yaralanmaların %50 si ve ölümlerin %3 ü yapısal olmayan yapı elemanlarından kaynaklanmıştır (YOTA El Kitabı, 2005). Dünyanın pek çok yerindeki müzelerde afetlerin ardından müze binası ve koleksiyonlarda ciddi hasarlar oluşmaktadır. Bunlardan birkaç örnek verecek olursak; San Fransisko Körfezi bölgesindeki 8 müzede yapılan araştırmada 1989 Loma Prieta depreminin ardından 500.000 objeden 150 sinin hasara uğrayarak 10 milyon dolarlık bir kayıp oluştuğu, San Fransisko Asya Sanat Müzesinde 3 milyar dolarlık koleksiyondan 26 parçanın hasara görmesiyle 3 milyon dolarlık bir kaybın oluştuğu rapor edilmiştir (Fierro, Freeman, Perry, 1995). Yine 1990 daki Filipinler depreminde özellikle çanak çömlek koleksiyonunun hasarı rapor edilmiştir. 6 Nisan 2009 tarihinde İtalya'nın Abruzzi bölgesinde meydana gelen depremin ardından ICOM heyetinin 112 müzede yaptığı incelemede pek çok tarihi eserin hasara uğradığı, özellikle tarihi binalarda faaliyetlerini sürdüren müzelerin en ağır hasarı aldığı belirlenmiştir. Abruzzi'de bulunan tarihi binaların %25'i bu depremden etkilenmiştir [1].

Ülkemizdeki pek çok müze de faaliyetlerini tarihi yapılarda sürdürmektedir. 5747 sayılı

kanunla Fatih İlçesine bağlanan Eminönü'nde bulunan 16 müzenin tamamı tarihi bina niteliğinde olup, bina yaşları 100 yıl ile 1477 yıl arasında değişmektedir. Afetlere karşı kırılgan yapıda olan bu tür tarihi müzelerin bodrum katlarının olası bir afete karşı depo olarak kullanılması risk taşımaktadır. Yapılan yapısal olmayan malzeme analizleri sonucunda yapılarda objelerin düşmesi, tavan malzemelerinin düşmesi, kablo kaynaklı yangın, hatalı veya bakımsız montajlar, camların kırılması, kullanılan malzemenin deprem ve yangına karşı hasar görebilirliği genel risk faktörleri olarak değerlendirilmiştir (Kuzucuoğlu, 2008). Ayrıca bu tür binalar sürekli ve yüksek bütçeli restorasyon çalışmalarına ihtiyaç duymaktadır.

Vitrinlerin gerekli dayanıklılık ve kalınlıkta olduğu, malzeme seçimine ve bulunduğu konuma dikkat edilmelidir. Vitrinin bulunduğu kat seviyesindeki değişiklikler ve binanın taşıyıcı sistemine olan uzaklığı da objeleri etkileyecek önemli faktörler arasındadır. Objelerin vitrin içinde kayıp kırılmaması veya birbirleriyle çarpışmamaları için gerekli önlemler alınmalıdır. Tek tek objelerin taban izolasyonlarının yapılması yerine vitrinin taban izolasyonu tercih edilmelidir. Vitrin içindeki objeye ulaşma veya aydınlatma elemanlarına ulaşma objeye zarar vermeden yapılabilmelidir. Ağır çerçeveli tablolar, aynalar, taban yükseklik dengesi açısından denge problemi bulunan kırılgan karakteristik gösteren objelerin sabitlenmesi de risk azaltıcı tedbirler arasındadır.

Özellikle eski binalarda su boruları duvarların içine gömülmeden duvar yüzeyinden geçebilmektedir. Bu boru sistemlerinin sarsıntı sonucu kırılarak objeler ve depolama alanları üzerinde oluşturabileceği riskler unutulmamalıdır.

YANGIN RİSKİ

Müze koleksiyonları ve özellikle de bu koleksiyonların bulunduğu tarihi binalar yangın ve su baskını açısından çok kırılgan bir karakteristiğe sahiptir. Tarihi binalarda bulunan müzelerde boru ve kablolama sistemleri duvar yüzeyine sonradan monte edildiğinden gerek deprem gerekse yangın riski açısından tehdit oluşturmaktadır. Zayıf altyapıya sahip olan sistemin üzerine yeni eklemeler yapıldığında sistemin bunu kaldıramama ve kabloların ısınması nedeniyle yangına sebebiyet verme riski bulunmaktadır. Yangın söndürme sistemleri tarihi müze binalarının estetik ve mimari değerine en az etki edecek şekilde tasarlanmalıdır.

Mutfak, kafeterya alanları ve buralarda kullanılan malzemenin oluşturabileceği bağıl nem, sıcaklık, yangın, patlama vb. gibi riskler nedeniyle sergileme ve depolama alanlarından uzakta bulunmalıdır.

Müze binasındaki tüm kapılar, yangında hasar görmeyecek ve dışarıya doğru açılacak şekilde tasarlanmalıdır. Müze binası iç tasarımında yangının yayılmasını önleyici seperatörler ve yangını geciktirici malzemeler kullanılmalıdır. Tahliye koridorlarında tahliye yolunu engelleyecek ağır objelerin konumları, yangın söndürme sistemlerinin yerleri dikkatle seçilmelidir.

Etkili bir yangın hasarını en aza indirme stratejisi hayata geçirilmelidir. Hollandadaki Amerongen Kalesi için olası bir yangının ardından yapılacak tahliyeye yönelik hazırlanan plan detaylı olarak ele alınmıştır. Plan için yerel itfaiye yetkilileriyle irtibata geçilmiş, yangın söndürme araç ve malzemelerinin tedarik edilmesi, su temini, periyodik tatbikatlar konusunda işbirliğine gidilmiş, eğer 5 dakikalık, 15 dakikalık ve 30 dakikalık vakit varsa hangi objelerin tahliye edileceğine dair tahliye öncelikli planlar hazırlanmıştır (Stovel,1998).

ÇEVRESEL RİSKLER

Müzelerde sergilenen ve depolanan objelerin hasar oranlarını en aza indirecek veya durduracak çevresel koşulların sağlanması gereklidir. Kontrol edilmesi gereken faktörler;

- Sıcaklık

- Bağıl nem,
- Işık ve radyasyon
- Hava Kirliliği, gazlar ve partiküller
- Titreşim
- Vitrin yapısı
- Taşıma

Çevresel koşullar için ideal değerleri sağlayacak otomasyon sistemlerinin kurulması müzelerin sınırlı bütçeleri nedeniyle mümkün olmayabilir. Ancak bu şartları izleyebilecek otomasyon sistemlerine göre çok daha düşük maliyetli sensörlerin yerleştirilmesi objelerin ve müzelerin sağlığı açısından önemlidir. Yapısal olmayan malzeme içeriği nedeniyle malzeme ömrü süresince ortama gaz yayabilir. Off-gasing olarak tabir edilen gazın açığa çıkması sürecinde oluşan toksik maddeler insan veya obje sağlığını tehdit edebilir. Risk, sıcaklık ve güneş ışığı ile daha da artabilir. Bu gazlar sıva, vitrin, ara paneller, asma tavan, halılar üzerindeki boya, yapıştırıcı, dolgu malzemesi gibi kimyasallardan oluşur. Bu nedenle kullanılacak malzemenin toksik olmayan özellikte olması gereklidir. Müze yapılarında kullanılacak olan yapısal olmayan malzemelerin gaz salınımı riski açısından testten geçirilmelidir.

Vitrin ve pencere camlarının kırılması ile oluşabilecek riskler arasında olup. camlara film takılması ile güneş ışığı zararlarının önlenmesi, güvenlik ve deprem anında dağılıp hasar vermemesi açısından önemlidir.

Kalorifer kazanı gibi tertibatın binaya sarsıntı vermemesi için yalıtım yapılmalıdır. Özellikle depolama alanları bu tür teçhizatın bulunduğu odaların yanında yer almamalıdır.

Sıcaklık dalgalanmalarının yaşandığı iklim değişiklikleri sonucu müzeler için iklimlendirme, ısıtma ve aydınlatmadan kaynaklanan enerji maliyetleri yük haline gelirken diğer taraftan bu sistemlerin ihtiyacı olan enerji artışının da yangınlar için risk oluşturduğu göz ardı edilmemelidir (Kaya,2008) Yapıdaki potansiyel yangın kaynakları belirlenerek, eski elektrik tesisatı yenilenmelidir.

Vandalizme karşı eserlerin özel bariyerlerle korumaya alınması ve hırsızlığa karşı güvenlik tedbirlerinin azaltılması risk azaltıcı tedbirler arasındadır.

SONUÇ

Müze binalarında objelerin devrilme riski, tavan malzemelerinin düşme riski, yangın riski, iklimlendirme cihazları ve kablolama, asma tavan, yangın söndürme boruları ve teçhizatları, sıhhi tesisat boruları ve aydınlatma sistemi bağlantıları nedeniyle boru ve kablo sistemlerinden kaynaklanan riskler değerlendirilmiştir.

Yapısı nedeniyle kırılgan özellikteki objelere yönelik sismik izolasyon zorunluluğu ön plana çıkmaktadır.

Yapısal olmayan yapı elemanlarının olası bir afette hasarı hem kendi hem de çevresindeki nesneler için tehdit oluşturacaktır.

Dayanıklı, düşük maliyetli ve yangına dirençli malzemenin seçimi önemlidir. Sıcaklık, duman sensörleri ile otomatik şemsiyeleme sistemlerinin kurulması öncelikli ele alınması gereken konulardır.

Bağlama ve montajdan kaynaklı risklerin önüne geçilmesi için müze binalarındaki tüm boru sistemleri periyodik olarak gözden geçirilmelidir.

Müze binalarında sergileme alanları ziyaretçiler için sirkülasyon alanı olarak düzenlendiğinden yangının yayılmasının önlenmesi için gerekli boşluklar tasarlanmalıdır. Anahtar alanlar (sergileme ve depolama) ile tahliye koridorları yangına dirençli toksik olmayan boya ve malzeme ile kaplanmalı, yer malzemesi olarak kaymayı önleyici

malzeme seçilmelidir. Eserler taşınırken özellikle salon ve oda girişlerindeki eşiklerin kaldırılması da riski azaltacaktır. Olası bir tahliye anında da bu önlem faydalı olacaktır.

Dekoratif malzeme olarak kullanılan duvar kaplama elemanlarının bağlantıları düşme riski nedeniyle kontrol edilmelidir.

Heykel, uzun vazo gibi yükseklik ve taban alanı açısından dengesizlik oluşturan objelerde düşük maliyetli ve basit bir mekanizma ile çalışan sismik izolatörlerin kullanılmasına başlanarak, tüm müzelere yaygınlaştırılması sağlanmalıdır.

İstanbul Valiliğince yürütülen Dünya Bankası Kredisi destekli İstanbul Sismik Riski Azaltma ve Acil Durum hazırlık Projesi (İSMEP) projesinde tarihi binaların deprem performanslarının değerlendirilmesi açısından ilk etapta Topkapı Sarayı-Mecidiye Köşkü, Arkeoloji Müzesi- Ek ve Klasik bina ve Ayasofya Müzesi Müdürlüğü-Aya İrini anıtının proje kapsamına alınması önemli bir gelişme olarak tespit edilmiş olup, bu çalışmaların yanında müzelerde yapısal olmayan risklerin önlenmesi amacıyla da projeler hazırlanıp mevcut projeye entegre edilmesinin uygun olacağı değerlendirilmiştir.

Müzelerde sergileme ve depolama alanlarındaki her türlü afete karşı düşük maliyetli modern tekniklerin kullanılması için gerekli uzman desteği konusunda değişik meslek grupları ve STK lar ile koordine sağlanmalıdır.

Müze binasındaki iklimlendirme, ısıtma ve jeneratör merkezinde yangın ve deprem risklerine karşı önlemler alınmalıdır.

Türkiyede geçerli olan bina kodları da yapısal olmayan risklere göre yeniden düzenlenmelidir.

Yapısal olmayan malzeme kaynaklı risklerin en aza indirilmesi amacıyla yapılacak periyodik bakımlar, daha yüksek onarım veya yeniden yapım maliyetlerini önleyecektir.

Risk değerlendirme ve zarar azaltma çalışmalarına tüm müze personelinin ve idarecilerinin katılımı sağlanmalıdır.

2010 Avrupa Kültür Başkenti Projesi kapsamında kültür mirasına verilen önem bu tür projelerle vurgulandığından uluslararası kamuoyunun ilgisini çekecek ve tarihi eserleri koruma bilincinin toplumun her kesimine yayılması sağlanacaktır.

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NATIONAL-INTERNATIONAL DISASTER DATABASE SYSTEMS AND IMM-AKOM DISASTER INFORMATION SYSTEM: AKOMAS

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Key Words: Disaster database, information system, disaster archieve

ABSTRACT 2008 saw a marked increase in the number of deaths and economic losses compared to 2000–2007 yearly average. For example the death toll in 2008 was three times more than the annual average for 2000–2007 (ISDR, 2009).

This dramatic increase in human and economic losses from disasters in 2008 is alarming. By this reason, many of the organizations in the world have recognized the importance on systematic collection of information on disasters for their future emergency relief and planning activities. The collection of real time natural and technological disaster data is important for mitigating the risk in future. Also these kinds of data are important for the insurance sector. In recent years, the need of data on disasters is increasing; but there are no long term database on disasters or disaster information at real time and systematically published.

The most common international and commercial databases are; NatCatSERVICE and SwissRe CatNet, and publicly accessible EM-DAT and GLIDE. National databases are the newly developed systems such as; TUAA and AKOMAS.

"Natcat SERVICE" and "SwissRe CatNet" are private international level disaster database systems which are maintained by Reinsurance Companies. These kinds of database systems contain the available information from different type of disasters. Entries include dead, missing, injured and homeless, along with detailed accounting of insured and uninsured damages. But there is lack of public accessibility to these services.

Different kind of data in a database must be searchable, accessible, detailed, real-time and also must address the different sectors and researchers.

The Istanbul Metropolitan Municipality Disaster Coordination Center started to publish the disaster bulletins both for World and Turkey since 2005 at the AKOM Web page (http://www.ibb.gov.tr/sites/akom). AKOMAS system is a newly developed database system with detailed and searchable information.

In our study different international and national database systems are examined and the standards, service sectors and implementations in already developed and developing disaster databases are discussed.

INTRODUCTION

In 2008, 321 natural catastrophes and human related disasters caused 255,816 fatalities and led to economic losses of USD 181 bn. The death toll was three times higher than the annual average for 2000-2007 (ISDR, 2009).

The increase of the fatalities and the economic losses were mainly caused by two major events: Cyclone Nargis in Myanmar and Sichuan Earthquake in China.

Institutions in the world have recognized the importance of systematic collection of information in disaster risk reduction planning and relief activities. The collection of real time natural and technological disaster data is important for mitigating the risk in future.

Also these kinds of data are important for the insurance sector. In recent years, the need of data on disasters is increasing; but there are no long term database systems on disasters or disaster information at real time and systematically published.

In our study different international and national databases are examined and the standards, service sectors and implementations in already developed and developing disaster database systems are discussed.

METHOD

Today, the effect of disasters upon people and its costs to the countries are not clearly identified and not very well standardized.

The common disaster types within the international database systems are; earthquake, flood, extreme weather conditions, landslides, volcanoes, wild fires, epidemics, drought, avalanche and collapse. Some database systems also include man made and technological disasters such as; big traffic accidents, industrial accidents, terrorist attacks and nuclear events.

General criterias of the database systems are; 10 or more deaths, 100 or more affected and declaration of a state of emergency/call for international assistance.

Within the scope of the study a comparison between the 3 different non commercial, accessible disaster database; EM-DAT, Glide-number and Turkey National Disaster Archive System was done and details on the development of the AKOMAS database system of Istanbul Metropolitan Municipality will be discussed (Table 1).

DATABASE SYSTEMS

INTERNATIONAL DATABASE SYSTEMS

CENTRE FOR RESEARCH ON THE EPIDEMIOLOGY OF DISASTERS (CRED): EM-DAT

Emergency Management Database (EM-DAT) is established in 1988 by the Centre for Research on the Epidemiology of Disasters at Belgium. It is an international database collecting information on natural and technological disasters. The database contains approximately 17,000 disaster entries [4,5].

EM-DAT aims to provide every statistical data on disasters and use of the sources including United Nations, NGO's, insurance companies, research institutions and media.

Criterias of disasters which are identified by CRED database are as follows;

- 10 or more death, 100 or more affected declaration of a state of emergency and call for emergency assistance.

EM-DAT codes the disasters with special identifications. The inquiries at the database are done according to these criterias; a unique 8 digits disaster number for each event includes the country code, disaster type and date i.e.: EQ-1999-0046-TUR). The data contains information on killed, injured, homeless people, affected number.

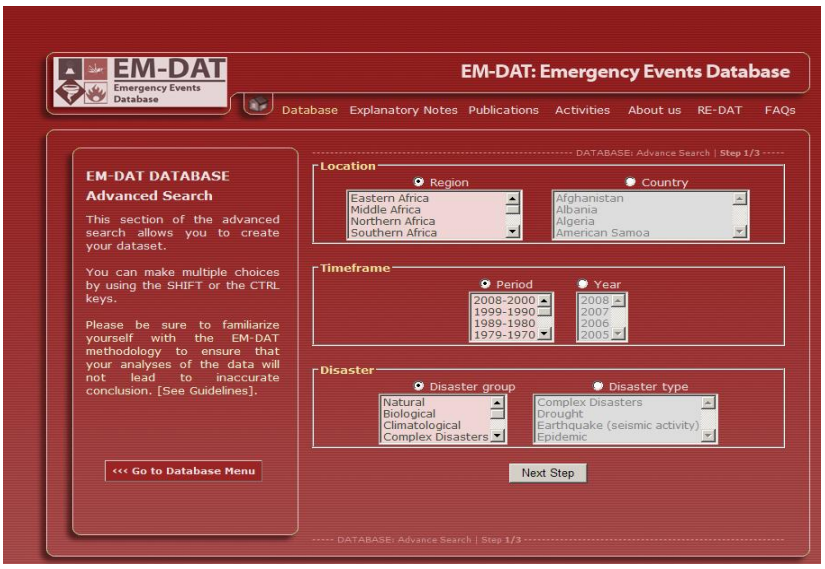

Fig. 1. EM-DAT Database (http://www.emdat.be/Database/terms.html).

ASIAN DISASTER REDUCTION CENTER (ADRC): GLIDE NUMBER

The Global Disaster Identifier Number (GLIDE) is established by ADRC in 1998 at Kobe, Japan for globally common Unique ID code for disasters [6].

The GLIDE Database is searchable by date, disaster type, country, injured/death number, evacuated number, duration of event, damage information and GLIDE number. The sources of the disasters are the United Nations and related agencies, Reuters and similar international media and NGO's.

The criterias for disasters are similar to CRED's;

- 10 or more death, 100 or more affected declaration of a state of emergency and call for emergency assistance.

GLIDE lists the disasters chronological and inquiries are available at the database as EMDAT's.

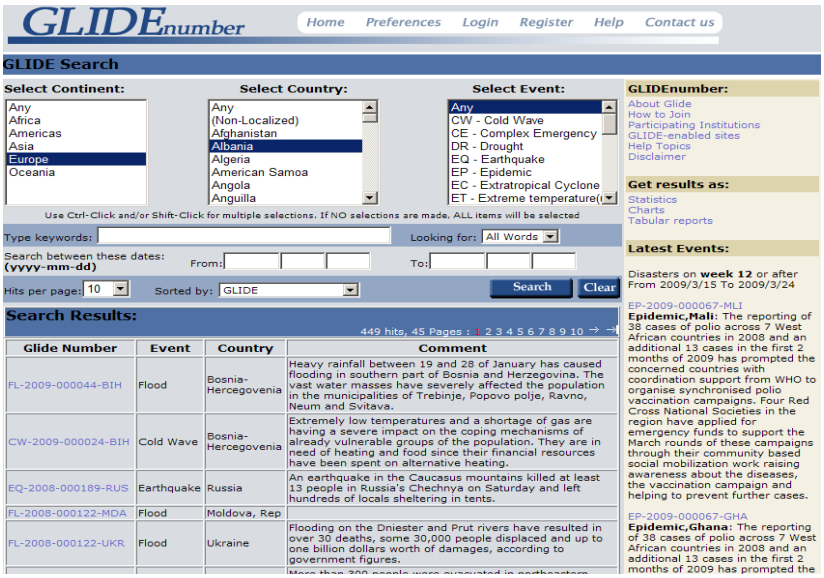

Fig. 2. GLIDE Database (http://gs.adrc.or.jp/glide/public/search/search.jsp).

NATIONAL DISASTER DATABASE SYSTEMS

GENERAL DIRECTORATE OF DISASTER AFFAIRS (AIGM): NDAT

The National Disaster Archive System of Turkey (NDAT) aims to put all disaster related data from Turkey including maps, reports, photos etc. into the web for public use [7].

By using the database queries on disaster type, date, map, and the place of the event can be done. System provides available documents within the General Directorate of Disaster Affairs such as archived report, maps, photographs, video etc [3].

Criterias for inclusion in the NDAT database are similar to the international criterias as follows;

- 10 or more death, 100 or more affected 50 or more injured declaration of a state of emergency and call for emergency assistance.

NDAT aims to numerate the disasters by using GLIDE number (for example; EQ-1999-0000046-TUR).

Tip	Il / İlçe / Köy	Tarih	Glide Number	Ölü	Yaralı	Etkilenen
Çığ				137	34	21858
Deprem				93069	76293	15915147
Heyelan				223	1	100743
Sel / Su Baskını				5	0	66433
Sis				0	0	0
Fırtına				0	0	0
Kaya Düşmesi				8	0	34278
Kuraklık				0	0	0
Kimyasal Kazalar				0	0	0
Orman Yangınları				4	0	0
Maden Kazaları				940	302	0
Şiddetli Yağış				0	0	0
Dolu				0	0	0
Kar				0	0	0
Don				0	0	0
Yıldırım				0	0	0
Trafik Kazası				1090	1630	0
Uçak Kazası				729	141	0
Deniz Kazası				31	0	0
Tren Kazası				372	1105	0
Diğer				87	77	0
Nükleer Kazalar ve Radyasyon Sızıntıları				0	32	0
Yangın				10	0	345

Fig. 3. National Disaster Archieve (http://www.afet.gov.tr/tuaa/).

ISTANBUL METROPOLITAN MUNICIPALITY-DISASTER COORDINATION CENTER DISASTER INFORMATION SYSTEM (IMM-AKOM): AKOMAS

Istanbul Metropolitan Municipality AKOM Disaster Information System (AKOMAS) is a newly established information system which is aimed to provide real time information on all types of natural and man-made disasters in the world and especially in Turkey [8].

Since 2005 AKOM was publishing the weekly list of disasters in the world and in Turkey. Starting by 2008, we initiated AKOMAS system to list the disasters with visual interactive maps (i.e location, date, type, effects etc.) not only for the disasters within Turkey but also for the disasters from the world as well and this makes the difference from other national systems. The aim of the AKOMAS mainly is to provide systematic information to public to support such as academic studies, insurance risk investigations, and mitigation projects.

Weekly lists of disasters are also shown on different maps for Turkey (Figure 4) and the World (Figure 5).

In addition, real time information for earthquakes in Turkey greater than 4 magnitude and earthquakes in the World greater than 6 magnitudes are sent by SMS to the related staff of IMM.

Fig. 4. Weekly map of disasters in Turkey (http://www.ibb.gov.tr/sites/akom/Documents/afet_felaket_harita_turkiye.html).

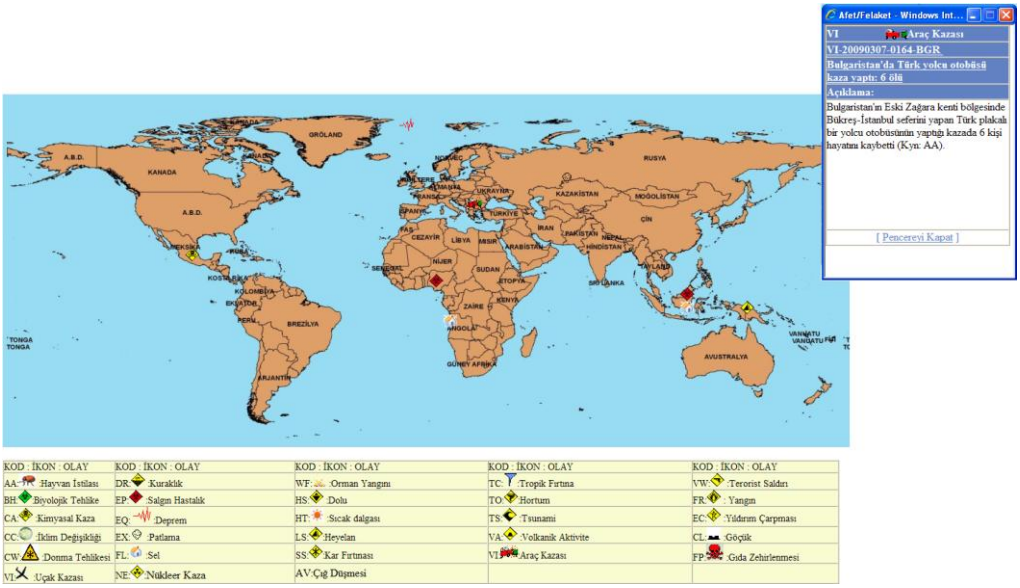

Fig. 5. Weekly map of disasters in the World (http://www.ibb.gov.tr/sites/akom/Documents/afet_felaket_harita_dunya.html).

General and detailed criterias of each disaster type are identified for inclusion in the AKOMAS database is similar to the international criterias (Table 2).

General criterias;

- 10 or more death, 100 or more affected 20 or more injured, declaration of a state of emergency and call for emergency assistance.

Detailed criterias are given in Table 1.

Table 1. Detailed AKOMAS Criterias by disaster types.

Disaster Type	Disaster Code	Death No	Injured No	Affected People No	Affected Area	Explanation
Vehicle Accident	VI	10 +	20 +	100 +	--------	Vehicle accident underground, aboveground and on road.
Mud Slide	MS	10 +	20 +	100 +	--------	
Environmental Pollution	EN	-------	-------	--------	--------	Every pollution event that affects the general life situation
Avalanche	AV	10 +	20 +	100 +	--------	Avalanche, which causes difficulties in transportation between two provinces or districts.
Earthquake	EQ	-------	-------	--------	--------	Every earthquake that caused significant property and/or personal injury + Earthquake, which strength is higher than 4.0 Richter scale
Other	OT	10 +	20 +	100 +	--------	Panic, insect infestations
Hail Storm	HS	--------	20 +	100 +	1 ha +	Every hail storm that affects the agricultural areas
Cold Wave	CW	10 +	20 +	100 +	--------	
Food Poisoning	FP	10 +	20 +	100 +	--------	
Collapse	CL	10 +	20 +	100 +	--------	
Climate Change	CC	-------	-------	--------	--------	Climate change in long term
Snow Storm	SS	10 +	20 +	100 +	--------	Avalanche, which causes difficulties in transportation between two provinces or districts and affects the general life
Sand Storm	SD	10 +	20 +	100 +	--------	Sand storm that affects the general life conditions
Drought	DR	-------	-------	100 +	--------	
Wild Fire	WF	--------	--------	--------	1 ha +	Every forrest or scrub fire
Explosion	EX	10 +	20 +	100 +	--------	
Epidemic	EP	--------	--------	--------	--------	Low number infection but potentionally can cause big plague or mass sickness
Flood	FL	10 +	20 +	100 +	--------	
Heat Wave	HT	--------	--------	100 +	--------	Temperature which is above 10 C° than the seasonal temperature.
Technological Disaster	AC	10 +	20 +	100 +	--------	
Terrorist Attack	VW	10 +	20 +	100 +	--------	
Land Slide	LS	10 +	20 +	100 +	--------	Slide, which causes difficulties in transportation between two provinces or districts
Tropical Storm	TC	10 +	20 +	100 +	--------	
Tsunami	TS	10 +	20 +	100 +	--------	
Volcanic Activity	VA	-------	-------	--------	--------	Every volcano activity, which could indicate eruption before stage
Fire	FR	10 +	20 +	100 +	--------	Fire events, which extend to big areas and causes high amount of economical losses.
Thunderstorm	EC	10 +	20 +	100 +	--------	

In AKOMAS system we code the disasters with special identifications according to the international norms like CRED criterias and numerate by Glide number (as EQ-20090131-0181-TUR).The first two digits of "EQ" is the disaster type, next 4 digits of 20090131 for the year and last 4 digits for the disaster number and last three digits for country code (Figure6).

Fig. 6. AKOMAS Archive System
http://www.ibb.gov.tr/sites/akom/Pages/akom_sorgu.aspx).

The inquiries at the database are done according to these criterias; 8 digits of disaster number, country code, disaster type and date (Figure 7).

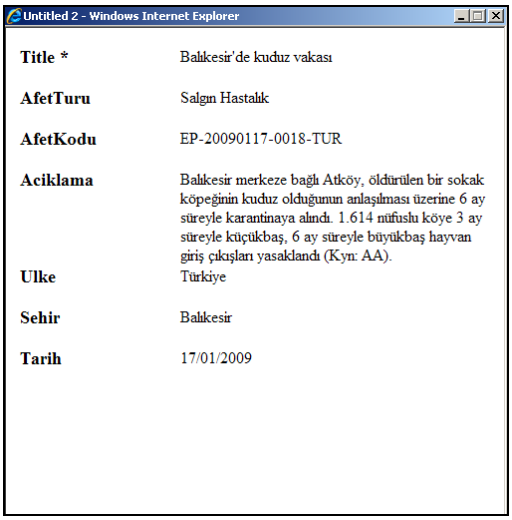

Fig. 7. AKOMAS Inquiry page.

In addition to AKOMAS information system, all the fires in the Istanbul city are being archived by the Directory of Fire Brigade of Istanbul Metropolitan Municipality according to the date, place of the event and type (Figure 8).

Fig. 8. Fire events listed by IMM Fire Brigade.

(http://www.ibb.gov.tr/sites/itfaiye/Pages/ItfaiOlayHaritasi.aspx).

Table 2. Comparison of the parameters between National and International Database Systems.

	EM-DAT	GLIDE	National Disaster Archive System (NDAT)	AKOMAS
Country	Belgium	Japan	Turkey	Turkey
Disaster Types	- Natural - Technological - Mixed disasters	- Natural - Technological - Mixed disasters	- Natural - Technological - Mixed disasters	- Natural - Man made disasters
Number of entries	17,000	4,340	3,314	1617*
Criteria	- 10 or more death, - 100 or more affected, - Declaration of a state of emergency.	- 10 or more death, - 100 or more affected, - Declaration of a state of emergency.	- 10 or more death, - 100 or more affected, - 50 injured, - Declaration of a state of	- 10 or more death, - 100 or more affected, - 20 injured, - Declaration of a state of emergency, - Call for emergency assistance.
Main Sources	UN sources	ADRC members	Official Sources	Anadolu Agency, Cihan Agency, IMM Fire Brigade
Other Sources	-NGO's -Research Centers -Insurance sources -Media -Official governmental sources	- Reuters etc. media organizations	-General Directorate of Highways - General Directorate of State Hydraulic Ways - General Directorate of Forestry - General Directorate of Protection and Control - General Directorate of Agriculture and Rural Affairs	- Governmental Sources - National Media - Natural Disaster Insurance Company - NDAT - BU Kandilli Observatory - EMDAT

			- General Directorate of Meteorology - General Directorate of Security	- GLIDE - USGS, EMSC - REUTERS
Accessibility	Free	Free	Free	Free
Web site	www.cred.be www.emdat.be	www.adrc.or.jp	www.afet.gov.tr/tuaa/	http://www.ibb.gov.tr/sites/akom
Update Frequency	Daily	Daily	-	Daily and Weekly

***** This number includes the disaster events of 2007 and later. The inputs of the data of 2005-2006 are continuing.

CONCLUSIONS

Since 2005, IMM AKOM- Urban Risk and Earthquake Monitoring Unit; is working to report the major events in Turkey and the World by disaster bulletins and maps. In this study the details of the AKOM information system is discussed. AKOMAS is developed to file and query the natural and manmade disasters which are affecting especially Istanbul, Turkey and its surrounding.

Another aim of this project is to provide a systematic database for disaster mitigation studies.

AKOMAS also share the visual and statistical information to users who are working on disaster related sectors. Another aim of AKOMAS is to provide information to the international community.

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SPACIAL SCALE EARTHQUAKE STATISTICS IN GEOMAGNETIC COORDINATES

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Key Words: Seismicity, Main Geomagnetic Field.

ABSTRACT The integrated studies involving seismology, geodynamics and geomagnetism are essential for advances in understanding of the dynamics of our planet. In this report, the results obtained in this direction in the Institute of seismology of Kazakhstan are presented. To obtain these results, the data on earthquakes with M≥4.0 were taken from the NEIC catalogue, and for each of the epicenters the parameters of the main geomagnetic field were estimated with using the International Geomagnetic Reference Field (IGRF) model. Then, spatial scale distribution of earthquake epicenters in relation to geometry of the main geomagnetic field was investigated. It is found: 1) local peaks of seismic activity in the northern and southern hemispheres are better organized according to geomagnetic latitude or angle of geomagnetic inclination then the geographic latitude; 2) the western and southern boundaries of the now-forming Somalian plate are defined very well using traced earthquakes, if one presents the map of epicenters not in the usual geographic coordinates but in a plot of geomagnetic inclination versus geographic longitude; 3) spatial location of three main seismotectonic areas: orogeny, subduction zones, and rift systems show systematization according to the angle of geomagnetic declination (D): for earthquake epicenters occurred in orogeny, D values are rather small (vary from ~ -1.0 to +5.0); for epicenters occurred in subduction zones, D values are rather large and positive, while they are rather large and negative for epicenters occurred in rift systems. It is concluded that geographical distribution of seismic zones at the Earth is controlled by the geometry of its main magnetic field.

INTRODUCTION

It is believed at present that the results of plate-tectonic forces are well evident in earthquakes activity: epicenters are located mainly within narrow bands, which coincide closely with the tectonic plate boundaries. For generation of plate tectonic forces the convection in the mantle is responsible (Bercovichi, 1998), which in turn is related to the processes into the fluid outer core (e.g. Hide et al., 2007). Simultaneously, the processes into the fluid outer core are responsible for generation of the Earth's main magnetic field (Glatzmaier and Roberts, 1995). Taking this into account, one may expect a coupling between spacial scale distribution of seismic zones and parameters of the main geomagnetic field. On the other hand, there are a lot of evidences that a few days/hours before the seismic shock the electromagnetic phenomena related to earthquakes are evident in the near space plasma parameters over the area of earthquake preparation and in magnetically conjugate area as well (e.g. Galper et al.,1995; Molchanov et al., 1995; Ondoh, 2008; Ouzounov and Freund, 2004; Sakhar et al., 2007; Ruzhin et al.,1998; Pulinets and Boyarchuk, 2004 and references in herein). To explain electromagnetic phenomena related to earthquakes, a model of lithosphere–atmosphere-ionosphere–magnetosphere coupling is under developing. This model is based partly on a concept of Global Electric Circuit (GEC) and suggests that an earthquake is an element of this circuit (e.g. Makarova and Shirochkov, 1998; Pulinets, 2009 and references in herein). It is known that a problem with operating of GEC is non conductive troposphere and stratosphere. It is considered at present that a source of ions in the near ground troposphere is air ionization produced due to release of

radioactive gases from the soil, while the principal source of atmospheric ions away from the boundary layer is galactic cosmic rays (Rycroft et al., 2000). The intensity of galactic cosmic rays reaching the top of the atmosphere is a function of the earth's magnetic field (Shea and Smart, 2006). Therefore, one again may expect a coupling between special distribution of seismic zones and geometry of the main geomagnetic field. The results presented in below confirm these expectations.

DATA AND METHOD

The data on earthquakes with magnitude M≥4.0 occurred at the globe in 1973-2006 (more than 241000 events) were taken from the NEIC catalogue [1]. Spacial scale distribution of epicenters is shown in figure 1, where the location of the now-forming Somalian plate is marked. It is seen that in geographic coordinates, the western and southern boundaries of Somalian plate are not well defined by traced earthquake epicenters.

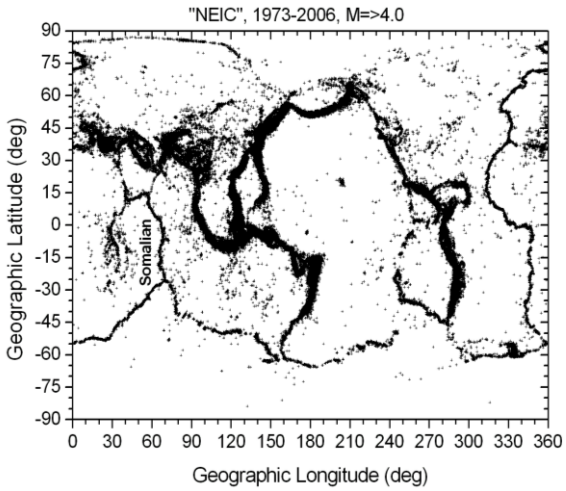

Fig. 1. The map of earthquake epicenters with M≥4.0 occurred at the globe during 1973-2006 [1].

For each of the epicenters presented in figure 1, the geomagnetic coordinates were calculated with using the well-known relations: $\sin \Lambda = \sin \lambda \sin \lambda_p + \cos \lambda \cos \lambda_p \cos (\varphi - \varphi_p)$, $\sin \Phi = \cos \lambda \sin (\varphi - \varphi_p)/\cos \Lambda$, where λ and φ are geographical latitude and longitude, and Λ and Φ are geomagnetic latitude and longitude, respectively; λ_p and φ_p are geographical coordinates of geomagnetic dipole poles. Also, for each of the epicenters, the parameters of the main geomagnetic field were estimated with using the International Geomagnetic Reference Field (IGRF) model, which computer codes are available at [2]. The IGRF is a series of mathematical models describing the Earth's main magnetic field and its secular variation. The model for each of the geomagnetic epoch comprises a set of spherical harmonic coefficients (called Gauss's coefficients) g_n^m и h_n^m, in a truncated series expansion of a geomagnetic potential function of internal origin:

$$V(r,\theta,\lambda,t) = R\sum_{n=1}^{N}\sum_{m=1}^{n}\left(\tfrac{R}{r}\right)\cdot\left(g_n^m(t)\cos m\lambda + h_n^m(t)\sin m\lambda\right)\cdot P_n^m(\cos\theta)$$

where *r, θ, λ* are geocentric coordinates (*r* is the distance from the centre of the Earth, *θ* is the colatitude, i.e. 90° - latitude, and *λ* is the longitude), *R* is a reference radius (6371.2 km); $g_n^m(t)$ and $h_n^m(t)$ are the coefficients at time *t* and $P_n^m(\theta)$ are the Schmidt semi-normalized associated Legendre functions of degree *n* and order *m*. The main field coefficients are functions of time and for the IGRF the change is assumed to be linear over five-year intervals. Because of the time variation of the field, a good accuracy of the model can only be produced for times when there is global coverage by satellites

measuring the vector field. This occurred in 1979-1980 years, when data of satellite MAGSAT were used, and from 1999, when data of Ørsted, CHAMP were used. It is considered [3] that the root mean square error for estimation of a module value of field is ~ 10 nT for 1980 and 2000 epochs. A 10 nT global rms vector error corresponds to global rms values of about 5, 5, and 7 nT for south, east, and radial geomagnetic components, respectively.

RESULTS

Distribution of number of earthquakes depending on geographic and geomagnetic latitudes and angle of geomagnetic inclination.

Figure 2 presents histograms for number of earthquakes in dependence on geographic latitude (2a), geomagnetic latitude (2b) and angle of geomagnetic inclination (2c).

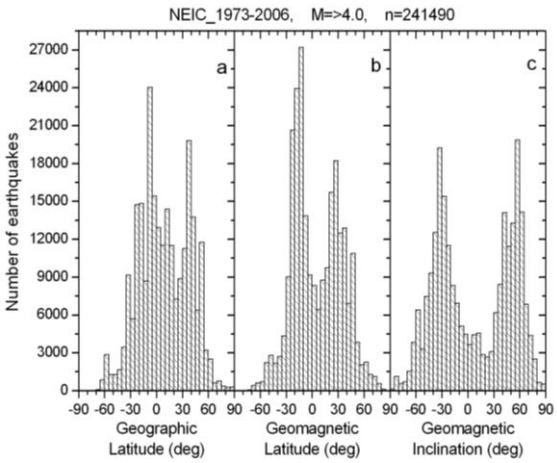

Fig. 2. Distribution of number of earthquakes versus geographic, geomagnetic latitudes, and angle of geomagnetic inclination (a-c, respectively).

Latitudinal variations of seismicity have been discussed by many authors (e.g. Levin and Chirkov, 1999; Fridman and Klimenko, 2002; and references in herein). It was found that epicenters density strongly decreases from equator to poles, described as a first approximation a curve of a square of distance up to an axis of rotation of the Earth. This result allowed authors to suggest that the mechanism of earthquake generation is related to the irregularity of the earth's rotation speed. Also, two local peaks of epicenters density seismicity located not symmetrically according to equator were revealed. In the northern hemisphere the peak was located near 30-39^{0}N, whereas in the southern hemisphere it was located near 0-9^{0}S. Fridman and Klimenko (2002) showed that asymmetry in peaks location takes place for both, the shallow and deep earthquakes. The results in figure 2a are in close agreement with Levin and Chirkov (1999). It is evident that number of earthquakes with M≥4.0 strongly decreases from equator to poles. The local peaks in northern and southern hemispheres are also evident in figure 2a, but they are better defined in figure 2b, where the distribution of number of earthquakes in dependence on geomagnetic latitude is shown. There is an asymmetry in peaks location according to geomagnetic equator: the center of hemispherical symmetry is displaced to the geomagnetic latitude ~ 5-10^{0}N. It is interesting to notice that during calculation ofd geomagnetic coordinates, the magnetic field was taken as centered dipole with symmetrically located poles. For epoch 2001 for example, the geomagnetic poles are chosen to be located at 79.397°S, 108.392°E and 79.397°N, 71.608°W. This is not in agreement with the real situation. According to the IGRF model for epoch 2001, the geomagnetic poles at the earth's surface are located at 79.0°N, 254.9°E and 64.7°S, 138.6°E. One may easily estimate that the center of the line connecting two geomagnetic poles is projected at latitude ~7.2°N that closely coincides with the center of symmetry in figure 2b. In figure 2c, the distribution of number of earthquakes

depending on the angle of geomagnetic inclination (I) is shown. This plot differs seriously from that one in figure 2a. There are two clear peaks in number of earthquakes depending on geomagnetic inclination. In the southern hemisphere (negative inclinations), where geomagnetic field lines come upward from the earth's surface, the maximum of seismicity occurs at I=~30^0. In the northern hemisphere (positive inclinations), where geomagnetic field lines come dawn into the surface, the maximum of seismicity occurs at I=~60^0. To more visualize a relationship between spatial scale distribution of epicenters and geomagnetic inclination, let us consider figure 3, where the map of epicenters is presented in a plot of geomagnetic inclination versus geographic longitude. Surprisingly, in figure 3, the western and southern boundaries of the now-forming Somalian plate are defined very well using traced earthquakes, while it was not evident in figure 1.

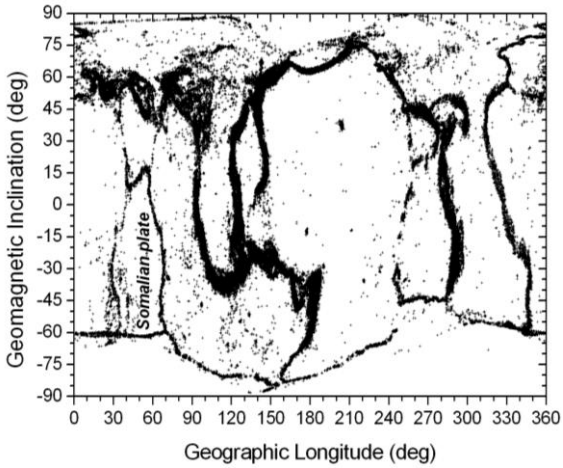

Fig. 3. Epicenters of earthquakes with $M \geq 4.0$ presented in a plot of geomagnetic inclination versus geographic longitude. The western and southern boundaries of the now-forming Somalian plate are defined very well using traced earthquake epicenters.

DISTRIBUTION OF NUMBER OF EARTHQUAKES DEPENDING ON GEOMAGNETIC DECLINATION

Figure 4 presents the global map of geomagnetic declination (D) for epoch 2000 [4]. Green lines in this figure show location of D=0 values. It is seen that the first D=0 line stretches from the south to the north through the American continent, and the second D=0 line stretches from Scandinavia through Europe, Africa, and India to East Asia, where it produces a loop and then comes through Indonesia and Australia to Antarctica. The D=0 lines divide the areas with positive (red) and negative (blue) declination values.

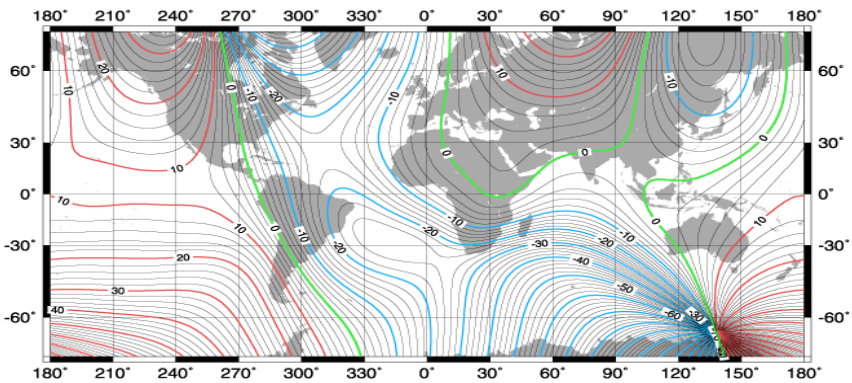

Fig. 4. Map of geomagnetic declination (D) for epoch 2000 [4] Green lines show zero, red – positive, and blue – negative D values.

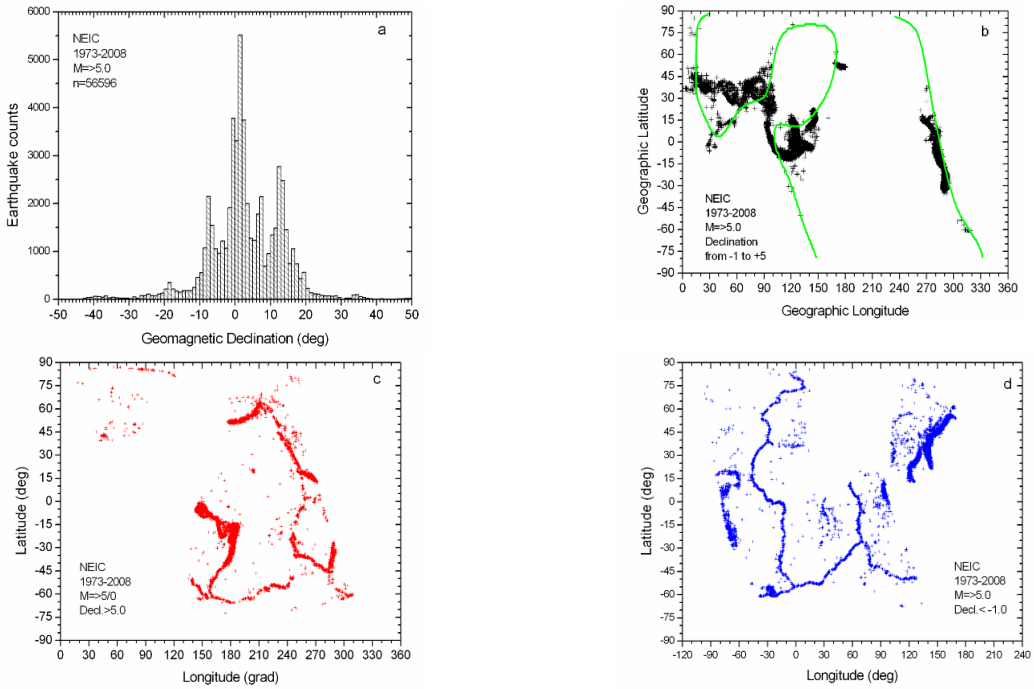

Fig. 5. (a) – Distribution of number of earthquakes in dependence on geomagnetic declination D; (b) - geographical location of epicenters which formed the main peak in (a) for 5.0 < D < -1.0, where green lines show location of D=0 for epoch 2000; (c, d) - geographical location of epicenters with D≥5.0 and D≤-1.0, respectively.

Earlier, Khachikjan et al. (2007, 2008) have reported on a coupling between spacial scale distribution of declination angles and earthquakes with M≥4.0 occurred in different tectonic areas. They showed that in epicenters belonging to continental orogeny, D-values vary around zero; in epicenters belonging to subduction zones, D - values are rather large and positive, while they are rather large and negative in epicenters belonging to rift systems. The goal of the present study is to carry out a similar analysis for stronger earthquakes (M≥5.0) which occurred at the globe in 1973-2008 (56596 events). The results are presented in figure 5. A histogram in (5a) shows three maxima in number of earthquakes depending on D. The central maximum is located around D=0, in addition, two weaker maxima are located at larger positive and negative D values. Figure 5b shows geographical location of epicenters which formed the main peak in (5a) at 5.0 < D < -1.0). Green lines in 5b mark D=0 lines. Keeping in mind a geological map, one may understand that the main peak in (5a) is formed by earthquakes occurred in areas of continental orogeny. Figure 5c shows geographical location of epicenters with large and positive declinations (D≥5.0). It is not difficult to understand that these earthquakes belong mainly to subduction zones which are located along the Pacific cost and partly in Alpine-Himalayan and Scandinavian regions. Figure 5d shows geographical location of epicenters with large and negative declination angles (D≤-1.0). One may conclude that these events occurred mainly in the rift systems at the bottom of the oceans. The results in figure 5 for earthquakes with M≥5.0 are in full agreement with the results obtained earlier for earthquakes with M≥4.0.

DISCUSSION AND CONCLUSION

The results in above paragraphs allow one to conclude that distribution of seismotectonic zones at the globe is related somehow to the geometry of the main geomagnetic field. On the definition, magnetic field may influence only electric currents or moving charge particles. Taking this into account, one may conclude that for generation of tectonic strains and deformations, which then results in earthquake activity, the electromagnetic forces may be responsible. It is suggested at present (e.g. Glatzmaier and Roberts, 1996; Aurnou et al., 1998) that the electromagnetic Lorenz force (Maxwell stresses) may

be responsible for the rotation of the inner core with respect to the mantle (super-rotation of the inner core). Also, (Karato, 1999) showed that the Maxwell-stress-induced flow at the boundary of the inner core results in a large strain that could cause the lattice preferred orientation of iron and, thus, the seismic anisotropy of the inner core that is observed in seismological observations. Khachikjan et all. (2007, 2008) presented very simple qualitative model for global distribution of magnetic part of Lorentz force, which may appear due to motion of charged particles in the geomagnetic field observed at the earth's surface. They showed in particular that due to vertically moved charged particles, the horizontal components of Lorentz force appear, and these components change theirs direction along the line of D=0, where the main amount of earthquakes occurred (figure 5a). It is clear that much work remains to further verify this speculative assertion and to find the physical processes linking main geomagnetic field with seismotectonic structures.

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THE MIXTURE MODELS FOR EARTHQUAKE RECURRENCE TIMES

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Key Words: Mixture Models, EM Algorithm, Earthquake Recurrence Time, NAFZ, AIC, Weibull, Exponential, Lognormal, Gamma

ABSTRACT In this paper, a mixture of distributions is proposed to modeling of earthquake recurrence times. We represent the time interval data between two successive earthquakes ($M \geq 5$) which were occurred in the area coordinated 39.000 – 42.000 North latitudes and 30.000- 40.000 East longitudes in North Anatolian Fault Zone (NAFZ) between 1900-2006 years with mixture of distributions such as Exponential, Gamma, Lognormal and Weibull. To show the effectiveness of mixture of distributions, firstly the earthquake recurrence times are modeled with pure probability distributions such as Exponential, Gamma, Lognormal and Weibull. Then the earthquake recurrence times are modeled with mixture of distributions such as Exponential, Gamma, Lognormal and Weibull. According to comparison, mixture of distributions is more suitable to represent the earthquake recurrence times than pure probability distributions. Under and over probability estimation problems in pure models for earthquake recurrence times are eliminated with using mixture distribution models. Mixture of which distribution is not important for modeling of earthquake recurrence times. On the other mixture of exponential distributions can be offered because parameter estimations can be obtained more easily than other mixture of distributions.

INTRODUCTION

Statistical methods have been widely used to model the large earthquakes recurrence times because earthquake is random events. There are many studies about the modeling earthquake recurrence times using statistical distributions which are Weibull distributions (Hagiwara 1974, Skyes and Nishenko 1984, Utsu 1984, Rikitake 1991, Parvez and Ram 1997, Newman et al. 2005, Yakovlev et al. 2006, Turcotte et al. 2007, Zoller and Hainzl 2007), Gamma distributions (Udias and Rice 1975, Utsu 1984) and lognormal distributions (Nishenko and Bulland 1987; Goes 1996, Shimazaki 2002).

In recent years, especially in the heterogeneous structure of the data model in the use of mixed distribution has become widespread. Frequency distribution in case of single-mode data with the standard model of probability distribution is useful and helpful. Mixture of distributions is even useful because it is applied to represent heterogeneous data set which there is evidence of multimodality or simply unimodality. Chau et al. (2005) represent the gait data with two component mixture normal distribution instead of with normal distribution.

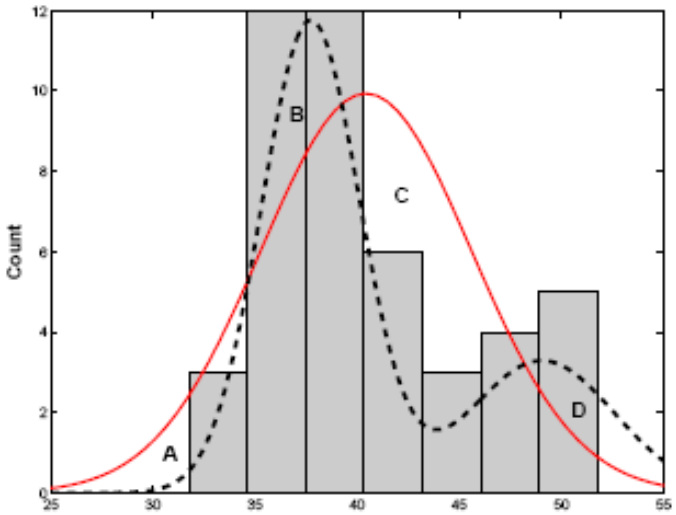

Fig.1. The Representation of the gait data with two components mixture normal distribution instead of with normal distribution(Chau et al., 2005).

Reviewed the literature in recently, studies on mixture of distributions approach is often encountered to desribe the data sets. Shepherd et al.(2000) showed that, the mixture of gamma distributions was clearly a better model for the tail moment distributions than either the log-normal or the pure gamma (including chisquare) distribution. By using Russian data for Q4 1998, Kolenikov and Aivazian(2001) showed that the mixture model, with additional weights accounting for non-response related to particular factors, performed beter than a standard lognormal one. Bucar et al. (2004) suggested to use finite Weibull mixture distributions for reliability approximation of an arbitrary system. Zhang and Liu (2006) used the mixture Weibull distribution to irregular diameter distributions of forest. Gong (2006) applied the mixture Weibull distribution to analyze the Fiat automobile data. Carta and Ramrez (2007) indicated that the mixture Weibull distribution provides a good fit for wind speed frequency distributions observed in the Canarian Archipelago. Consequently mixture distribution models are suitable for analysis of lifetime, system reliability analysis and modeling environmental events as wind speed and forest applications.

In this study, we represent the time interval data between two successive earthquakes (M>5) which were occurred in the area coordinated 39.000 – 42.000 North latitudes and 30.000- 40.000 East longitudes in North Anatolian Fault Zone (NAFZ) between 1900-2006 years with mixture of distributions such as Exponential, Gamma, Lognormal and Weibull.

MODELING EARTHQUAKE RECURRENCE TIME

Random variable T is assumed that refers to time interval between two successive earthquakes ($M \geq 5$). The probability density function and the cumulative distribution function of T is denoted by $f(t)$ and $F(t)$ respectively. The conditional probability of next earthquake between t and $t+\Delta t$ given the provided that it has no earthquake occured in elapsed time t since the last earthquake is denoted by $P(\Delta t \mid t)$ and defined by

$$P(\Delta t \mid t) = \frac{P(t < T \leq t + \Delta t)}{P(T \geq t)} \tag{1}$$

EARTHQUAKE RECURRENCE MODELS

There are several general statistical models frequently used for earthquake recurrence. In this study we start with four recurrence models to analyze the earthquake recurrence

times in NAFZ. Given $f(t)$ as the probability density function, $F(t)$ as the cumulative probability function, and t as the interval time, these models can be expressed as follows:

Exponential Distribution:

$$f(t) = \frac{1}{\lambda} e^{-\frac{t}{\lambda}} \quad t > 0 \quad \lambda > 0 \tag{2}$$

$$F(t) = 1 - e^{-\frac{t}{\lambda}} \tag{3}$$

$$E(t) = \lambda \tag{4}$$

Gamma Distribution:

$$f(t) = t^{\alpha-1} \frac{e^{-t/\beta}}{\beta^{\alpha}\Gamma(\alpha)} \quad t > 0 \quad \alpha, \beta > 0 \tag{5}$$

$$F(t) = \frac{\Gamma_x(\alpha)}{\Gamma(\alpha)} \tag{6}$$

$$E(t) = \alpha\beta \tag{7}$$

where $\Gamma_x(\alpha)$ is called an incomplete Gamma function and calculated by $\Gamma_x(\alpha) = \int_0^x t^{\alpha-1} e^{-t} dt$

Weibull Distribution:

$$f(t) = \frac{\alpha}{\beta}\left(\frac{t}{\beta}\right)^{\alpha-1} e^{-\left(\frac{t}{\beta}\right)^{\alpha}} \quad t > 0 \quad \alpha, \beta > 0 \tag{8}$$

$$F(t) = 1 - e^{-\left(\frac{t}{\beta}\right)^{\alpha}} \tag{9}$$

$$E(t) = \beta\Gamma\left(1 + \frac{1}{\alpha}\right) \tag{10}$$

Lognormal Distribution:

$$f(t) = \frac{e^{\left(-\frac{1}{2}\left\{\frac{\ln t - \mu}{\sigma}\right\}^2\right)}}{t\sigma\sqrt{2\pi}} \quad t > 0 \quad \mu, \sigma > 0 \tag{11}$$

$$F(t) = \Phi\{(\ln t - \mu)/\sigma\} \tag{12}$$

$$E(t) = e^{\mu + \frac{\sigma^2}{2}} \tag{13}$$

where Φ is cdf of normal probability distribution function and is defined by $\Phi = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{\frac{\ln t-\mu}{\sigma}} e^{-\frac{u^2}{2}} du$

MIXTURE BASED APPROACH TO MODEL LARGE EARTHQUAKES RECURRENCE TIMES

In finite mixture models assumed that the population consists of $g\,(\geq 2)$ distinct subgroups or subclasses. Finite mixture distribution function can written as

$$f(x|\psi) = \sum_{k=1}^{g} \pi_k f_k(x|\theta_k) \tag{14}$$

where the vector $\psi = (\pi, \theta)$ contains all unknown parameters $\pi = (\pi_1, \ldots, \pi_g)$ and $\theta = (\theta_1, \ldots, \theta_g)$ in the mixture model. The function $f_k(x|\theta_k)$ is called mixture component density function, $k = 1, \ldots, g$ for some parameter θ_k and π_k is called is called mixture weight of k th class has restriction that $\pi_k \in (0,1)$ and $\sum_{k=1}^{g} \pi_k = 1$.

PARAMETER ESTIMATION IN FINITE MIXTURE MODELS USING WITH EM ALGORITHM

In finite mixture models, the EM (Expectation-Maximization) algorithm has been used as an effective method to find maximum likelihood parameters estimation (McLachlan and Peel 2001). In EM framework, the observed data $t_1, \ldots, t_n$ is considered as an incomplete data and latent class variables $z_1, \ldots, z_g$ to be missing where $z_{ki} = z_k(x_i) = 1$ if observation t_i belongs to k th class and 0 otherwise for $k = 1, \ldots, g$ and $i = 1, \ldots, n$. The EM algorithm is applied to the mixture distributions by treating z as missing data. EM algorithm can be preceded with two steps, E- and M- steps.

In E step, to estimate the hidden variable vector $z_i = (z_{1i}, \ldots, z_{gi})$, Conditional expectation funciton $E(z_{ki}|t_i)$ is used.

$$\hat{z}_{ki} = E(z_{ki}|t_i) = \frac{\pi_k f(t_i|\theta_{ki})}{\sum_{s=1}^{g} \pi_s f_s(t_i|\theta_s)} \tag{15}$$

In M step, $E(z_{ki}|t_i)$ function which is calculated in E step is maximized under the conditional of on mixture weights as $\sum_{k=1}^{g} \pi_k = 1$. To estimate the mixture weights and parameters vectors which are denoted by $\pi = (\pi_1, \ldots, \pi_g)$ and $\theta = (\theta_1, \ldots, \theta_g)$ respectively, Lagrange method can be used. The estimated mixture weight of subgroup k is defined by

$$\hat{\pi}_k = \frac{1}{n} \sum_{i=1}^{n} \hat{z}_{ki}\,, \qquad k = 1, \ldots, g \tag{16}$$

In equation (14), if we choose $f_k(t|\theta_k)$ as an Exponential function of k th group for t observation vector, we can evaluate a mixture of exponential distributions and defined by

$$f(t|\psi)=\sum_{k=1}^{g}\pi_k\frac{1}{\lambda_k}e^{-\frac{t}{\lambda_k}} \tag{17}$$

In this form, the maximum likelihood estimator of λ parameters can be obtained with EM for k th subgroup as

$$\hat{\lambda}_k=\frac{\sum_{i=1}^{n}\hat{z}_{ki}t_i}{\sum_{i=1}^{n}\hat{z}_{ki}} \tag{18}$$

In equation (14), if we choose $f_k(t|\theta_k)$ as a Lognormal pdf of k kth group for observation vector t, we can evaluate a mixture of lognormal distributions and defined by

$$f(t|\psi)=\sum_{k=1}^{g}\pi_k\frac{e^{\left(-\frac{1}{2}\left\{\frac{\ln t-\mu_k}{\sigma_k}\right\}^2\right)}}{t\sigma_k\sqrt{2\pi}} \tag{19}$$

In this form, the maximum likelihood estimator of parameters μ and σ^2 can be obtained with EM for k th subgroup as

$$\hat{\mu}_k=\frac{\sum_{i=1}^{n}\hat{z}_{ki}\ln t_i}{\sum_{i=1}^{n}\hat{z}_{ki}} \tag{20}$$

$$\hat{\sigma}_k^2=\frac{\sum_{i=1}^{n}\hat{z}_{ki}\left(\ln t_i-\hat{\mu}_k\right)^2}{\sum_{i=1}^{n}\hat{z}_{ki}} \tag{21}$$

In equation (14), if we choose $f_k(t|\theta_k)$ as the Weibull pdf of k th group for observation vector t, we can evaluate a mixture of Weibull distributions and defined by

$$f(t|\psi)=\sum_{k=1}^{g}\pi_k\frac{\alpha_k}{\beta_k}\left(\frac{t}{\beta_k}\right)^{\alpha_k-1}e^{-\left(\frac{t}{\beta_k}\right)^{\alpha_k}} \tag{22}$$

In this form, the maximum likelihood estimator of parameters α and β can be obtained with EM for k th subgroup as

$$\hat{\alpha}_{k,(r+1)}=\hat{\alpha}_{k,r}+\frac{A_{k,r}+\left(1/\hat{\alpha}_{k,r}\right)-\left(C_{k,r}/B_{k,r}\right)}{\left(1/\hat{\alpha}_{k,r}^2\right)+\left(B_{k,r}D_{k,r}-C_{k,r}^2\right)/B_{k,r}^2}\qquad k=1,\ldots,g \tag{23}$$

$$\hat{\beta}_k=\left(\frac{\sum_{i=1}^{n}\hat{z}_{ki}t_i^{\hat{\alpha}_k}}{\sum_{i=1}^{n}\hat{z}_{ki}}\right)^{1/\hat{\alpha}_k} \tag{24}$$

where $A_{k,r} = \dfrac{\sum_{i=1}^{n} \hat{z}_{ki} \ln t_i}{\sum_{i=1}^{n} \hat{z}_{ki}}$, $B_{k,r} = \sum_{i=1}^{n} \hat{z}_{ki} t_i^{\hat{\alpha}_{k,r}}$, $C_{k,r} = \sum_{i=1}^{n} \hat{z}_{ki} t_i^{\hat{\alpha}_{k,r}} \ln t_i$ and $D_{k,r} = \sum_{i=1}^{n} \hat{z}_{ki} t_i^{\hat{\alpha}_{k,r}} (\ln t_i)^2$.

In equation (14), if we choose $f_k(t \mid \theta_k)$ as the Gamma pdf of k th group for observation vector t, we can evaluate a mixture of Gamma distributions and defined by

$$f(t \mid \psi) = \sum_{k=1}^{g} \pi_k t^{\alpha_k - 1} \frac{e^{-t/\beta_k}}{\beta_k^{\alpha_k} \Gamma(\alpha_k)} \tag{25}$$

In this form, for k th subgroup, the maximum likelihood estimator of parameters α and β can be obtained with EM as

$$\hat{\alpha}_k = \hat{\alpha}_{k-1} - \frac{\ln(\hat{\alpha}_{k-1}) - \psi(\hat{\alpha}_{k-1}) - \ln\left(\dfrac{\sum_{i=1}^{n} \hat{z}_{ki} t_i}{\sum_{i=1}^{n} \hat{z}_{ki}}\right) + \dfrac{\sum_{i=1}^{n} \hat{z}_{ki} \ln(t_i)}{\sum_{i=1}^{n} \hat{z}_{ki}}}{\dfrac{1}{\hat{\alpha}_{k-1}} - \psi'(\hat{\alpha}_{k-1})} \tag{26}$$

$$\hat{\beta}_k = \frac{\sum_{i=1}^{n} \hat{z}_{ki} t_i}{\hat{\alpha}_k \sum_{i=1}^{n} \hat{z}_{ki}} \tag{27}$$

where $\psi(.)$ and $\psi'(.)$ are a digamma and trigamma functions respectively.

CHOOSING NUMBER OF COMPONENTS IN MIXTURE MODELS

Defining number of components is the most important problem to represent data in mixture models. Number of components in mixture model can be defined by an information criterian. The most well known information criteria are Akaike's Information Criterion(AIC). The AIC criterion (Akaike, 1974) is defined by

$$AIC = -2 \ln L + 2r \tag{28}$$

where r is the number of free parameters in the finite mixture model.

APPLICATION

The North Anatolian Fault Zone (NAFZ) is one of the most prominent active earthquake zones in the world for approximately 12 million years**.** Just in the 20th century, 13 earthquakes with a magnitude 6.5 or higher have occurred along this fault that have killed thousands of people and caused extensive damage to infrastructure and buildings. The North Anatolian Fault Zone (NAFZ) is a 1600 km long geologic fault between the northern edge of the Anatolian Plate and the Eurasian Plate in northern Turkey. The fault zone display a fish-bone structure in the eastern part of its course by the Kırıkkale-Erbaa Fault Zone, the Almus Fault Zone and Taşova-Tekke Fault as splays. In this work, because of having similar characteristics, earthquakes with magnitudes equal 5 or higher are recurrence time data which were occurred in the area coordinated 39^0-42^0 North latitudes and 30-40 East longitudes in North Anatolian Faulth Zone (NAFZ) between

1900-2006 years (Yılmaz et al. 2004). This data set is taken from Bogazici University Kandilli Observatory and Earthquake Research Institute's database. Fig.2 shows the area of investigation. This data set consists of 159 earthquake recurrence times. The histogram of the earthquake($M \geq 5$) reccurrence times in NAFZ is given by Figure 3.

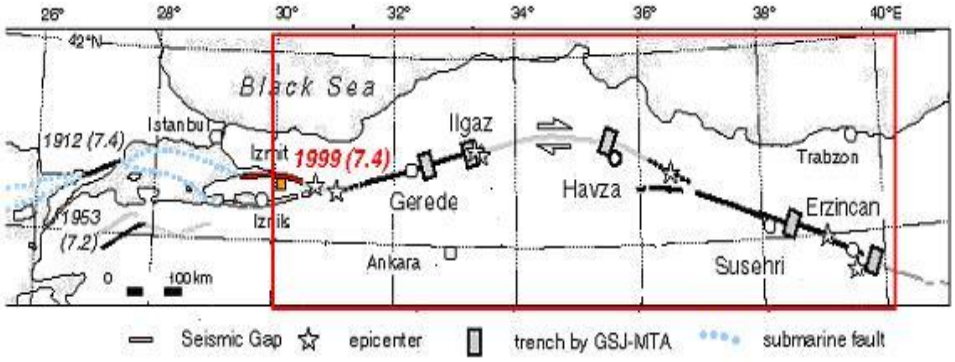

Fig. 2. The area coordinated 39.00^0 – 42.00^0 North latitudes and 30.00^0- 40.00^0 East longitudes in NAFZ. (Okumura, K., et al.1993).

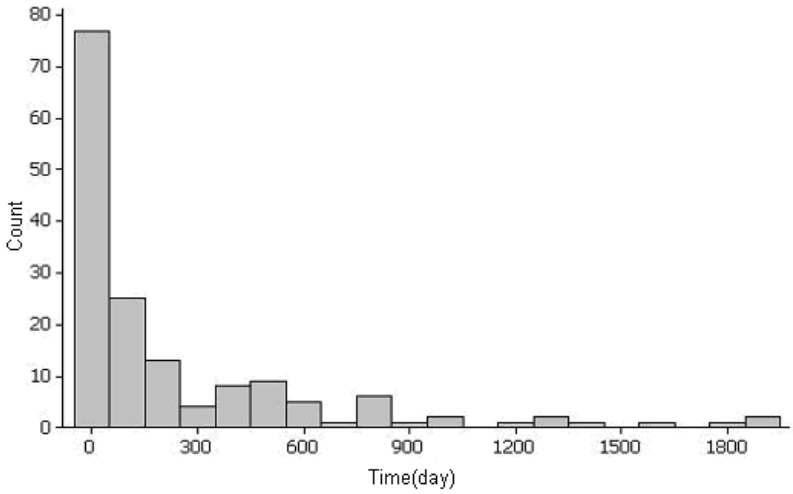

Fig. 3. The histogram of the earthquake($M \geq 5$) reccurrence times in NAFZ.

To show the effectiveness of mixture of distributions, firstly the earthquake recurrence times are modeled with pure probability distributions such as Exponential, Gamma, Lognormal and Weibull. The parameters estimations and mean recurrence time of these density functions given by Table 1.

Table 1. The parameters estimations and mean of recurrence times.

Distribution	Estimation of Parameter		Mean Recurrence Time
Exponential	$\hat{\lambda} = 239.3998$		239.3998
Gamma	$\hat{\alpha} = 0.2780$	$\hat{\beta} = 861.2100$	239.4164
Lognormal	$\hat{\mu} = 2.9665$	$\hat{\sigma} = 3.4582$	7677.6870
Weibull	$\hat{\alpha} = 0.3919$	$\hat{\beta} = 93.4320$	328.8856

It can be seen that modeling the earthquake recurrence times with different density function give us different characteristics. The best model to represent data set should be decided among them. Now, the earthquake recurrence times are modeled with mixture of distributions. In mixture model case choosing number of components is very important part. In this case number of components will be decided to AIC. To obtain the AIC values we need to compute values of log-likelihoods. The values of log-likelihood corresponding to different number of components are given by Table 2.

Table 2. The values of log-likelihood corresponding to different number of components.

Component Number	Exponential	Lognormal	Gamma	Weibull
1	-1030.00	-894.06	-875.46	-877.49
2	-892.99	-865.42	-868.99	-867.30
3	-863.18	-862.49	-866.99	-862.70
4	-857.81	-859.73	-864.94	-867.31
5	-857.81	-857.87	-855.75	-862.09

In AIC criterion, to find number of components in mixture model, the first local minimum is investigated. The AIC values computed with different number of components for mixture of exponential, lognormal, Gamma and Weibull density functions are shown in figure 4. According to figure 4, if the earthquake recurrence time data are modeled with mixture of exponential, lognormal, Gamma and Weibull density functions, the number of components are 4, 2, 2, and 3 respectively. The parameter estimations of mixture distributions and mean of the earthquake recurrence times are given in Table 3.

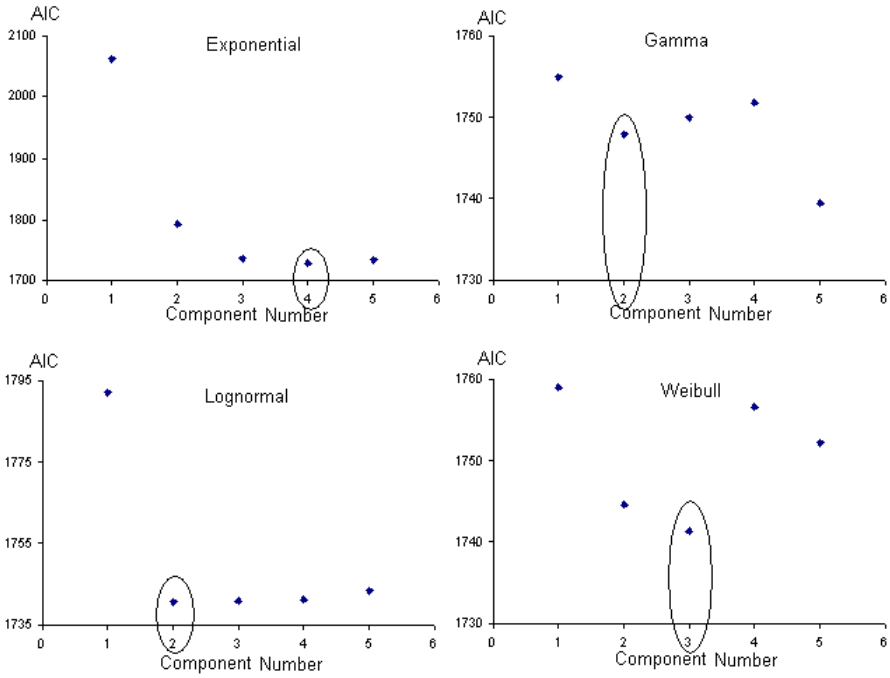

Fig. 4. The AIC values computed with different number of components, The optimum number of components is specified by circle.

Table 3. The parameter estimations of mixture distributions and mean of the earthquake recurrence times.

	k	$\hat{\lambda}_k$	$\hat{\pi}_k$	Component Mean	General Mean
Exponential	1	477.4017	0.4757	477.4017	239.4019
	2	2.7862	0.1614	2.7862	
	3	0.0620	0.1503	0.0620	
	4	55.7052	0.2126	55.7052	

	k	$\hat{\alpha}_k$	$\hat{\beta}_k$	$\hat{\pi}_k$	Component Mean	General Mean
Gamma	1	0.2685	502.9589	0.8365	135.0445	239.4111
	2	2.3835	324.4692	0.1635	773.3723	

	k	$\hat{\mu}_k$	$\hat{\sigma}_k$	$\hat{\pi}_k$	Component Mean	General Mean
Lognormal	1	0.1303	2.9779	0.4650	95.9900	295.0844
	2	5.4321	1.1972	0.5350	468.1290	

	k	$\hat{\alpha}_k$	$\hat{\beta}_k$	$\hat{\pi}_k$	Component Mean	General Mean
Weibull	1	0.36396	14.063	0.5986	62.0209	241.5957
	2	1.1939	475.41	0.3841	447.7894	
	3	65.751	1888.6	0.0173	1872.4460	

Mean recurrence times of mixture of distributions are similar to each other incase of one components density models. The effectiveness of mixture models is also shown in figure 5. In figure 5, the histogram estimations of the earthquake recurrence times in NAFZ with one component density functions and mixture models are shown. It can be seen from figure 5, the mixture of which distribution functions is not important to model earthquake recurrence times.

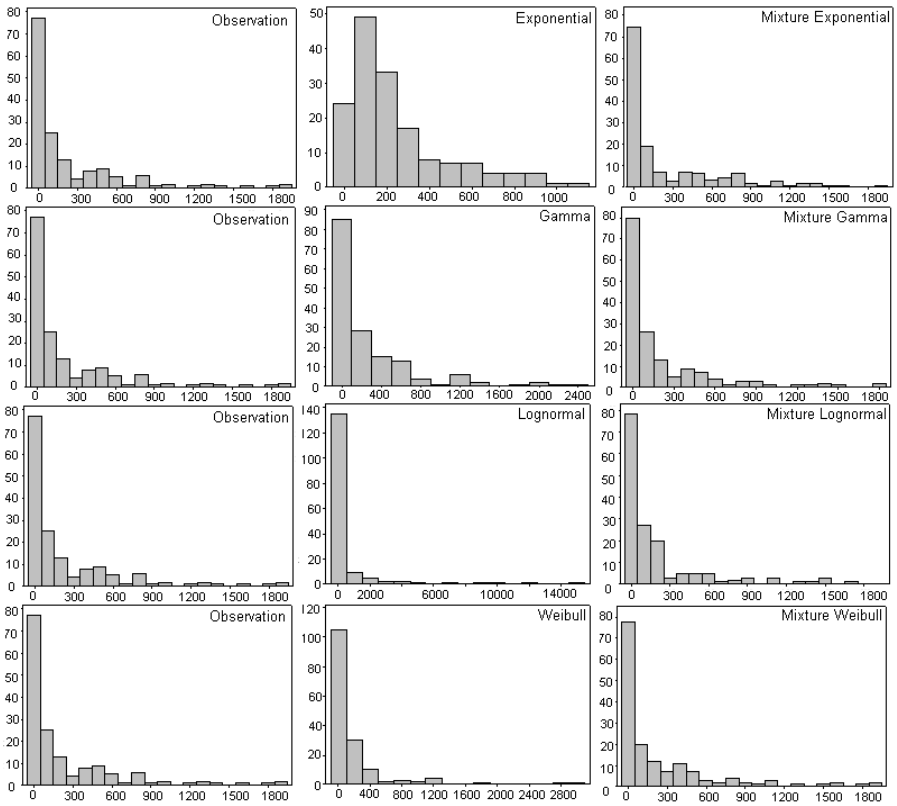

Fig. 5. The histogram estimations of the earthquake recurrence times in NAFZ with one component density functions and mixture models.

The cumulative distribution functions of pure distributions, mixture distributions and empirical distributions are shown in same plane with figure 6. It can be seen clearly that Mixture distributions is more successful than pure distributions for modeling the earthquake reoccurrence time data.

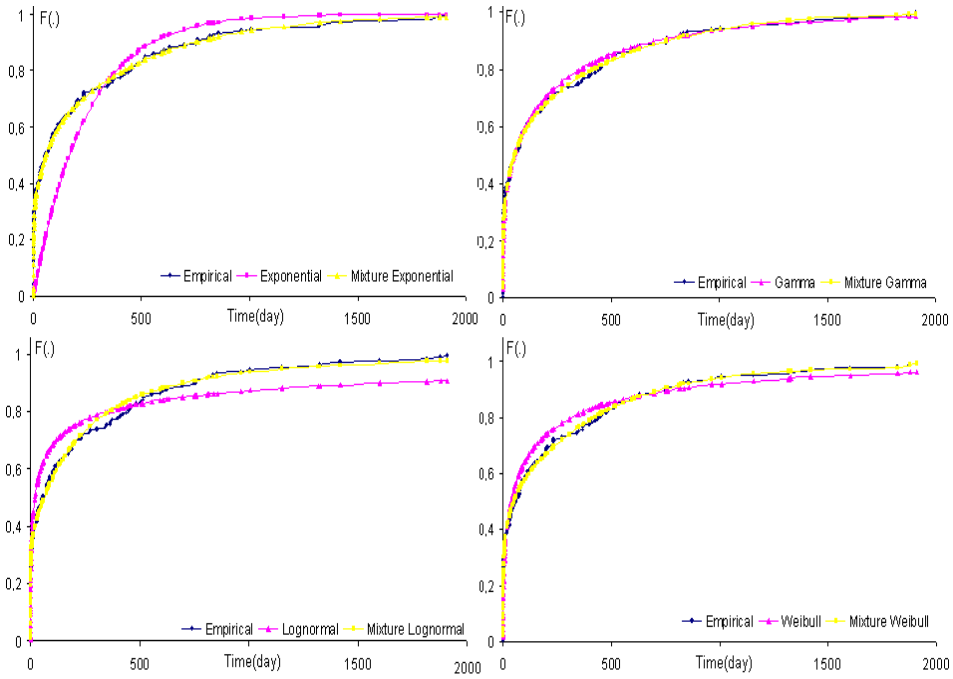

Fig. 6. Graphs cdf of pure distributions, mixture distributions and empirical distributions.

Risk design that shows occurrence risk of an earthquake having magnitude equal to 5 or higher in t day(s) after an earthquake occurred between 39^0-42^0 North latitudes and 30^0-40^0 East longitudes with magnitudes equal 5 or higher is given in Table 4.

Table 4. The occurrence risk of an earthquake having magnitude equal to 5 or higher in t day(s) after an earthquake occurred between 39^0-42^0 North latitudes and 30^0-40^0 East longitudes with magnitudes equal 5 or higher.

Time	**Exponential**		**Gamma**		**Lognormal**		**Weibull**	
(Day)	Pure	Mixture	Pure	Mixture	Pure	Mixture	Pure	Mixture
1	0.004	0.204	0.170	0.174	0.195	0.224	0.155	0.190
10	0.041	0.352	0.321	0.322	0.424	0.359	0.341	0.355
50	0.188	0.485	0.497	0.489	0.608	0.472	0.543	0.501
100	0.341	0.579	0.595	0.579	0.682	0.565	0.642	0.576
500	0.876	0.833	0.851	0.830	0.826	0.853	0.855	0.835
1000	0.985	0.941	0.939	0.941	0.873	0.936	0.920	0.944
1500	0.998	0.979	0.972	0.981	0.896	0.965	0.949	0.973
2000	1.000	0.993	0.987	0.994	0.910	0.978	0.964	0.997
2500	1.000	0.997	0.993	0.998	0.920	0.986	0.973	0.999

According to mixture models given in Table 4, the probability of an earthquake with magnitude 5 or above in 100 days after an earthquake with magnitude 5 or higher is between 0.565 and 0.579. But according to pure models in given Table 4, these

probability can be obtained between 0.341 and 0.682. It can be seen that probability estimations of the an earthquake with magnitude 5 or above in 100 days after an earthquake with magnitude 5 or higher is similar in mixture distribution models. Hence it is not important which distribution model used to model the earthquake for recurrence time data in mixture case.

We assume that an earthquake an earthquake having magnitude equal to 5 or higher is not occurred in 50 days after an earthquake occurred between 39^0-42^0 North latitudes and 30^0-40^0 East longitudes with magnitudes equal 5 or higher. In $50+\Delta t$ day(s), the conditional probability estimations of an earthquake having magnitude equal to 5 or higher are given in Table 5.

Table 5. The conditional probability estimations of an earthquake having magnitude equal to 5 or higher in $50+\Delta t$ day(s).

Δt	**Exponential**		**Gamma**		**Lognormal**		**Weibull**	
	Pure	Mixture	Pure	Mixture	Pure	Mixture	Pure	Mixture
50	0.188	0.182	0.195	0.177	0.190	0.176	0.217	0.150
150	0.466	0.381	0.413	0.374	0.363	0.412	0.432	0.339
450	0.847	0.676	0.704	0.668	0.557	0.722	0.682	0.668
700	0.946	0.808	0.814	0.801	0.629	0.824	0.772	0.811
1200	0.993	0.933	0.919	0.933	0.709	0.913	0.862	0.926
1950	1.000	0.986	0.974	0.989	0.770	0.959	0.921	0.994
2450	1.000	0.995	0.987	0.997	0.796	0.973	0.942	0.998

CONCLUSIONS

In this study, mixture of distributions approach for modeling of earthquake recurrence time is recommended. Mixture of distribution models is more suitable to represent the earthquake recurrence times than pure distribution models. Under and over probability estimation problems in pure models for earthquake recurrence times are eliminated with using mixture distribution models. Mixture of which distribution is not important for modeling of earthquake recurrence times. On the other mixture of exponential distributions can be offered because parameter estimations can be obtained more easily than other mixture of distributions.

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THE STATISTICAL EXAMINATION OF THE NATURAL DISASTERS AND THE LOSSES DUE TO THESE DISASTERS IN TURKEY

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Key Words: Natural Disaster, Earthquake, Landslide, Flood, Snow Avalanches.

ABSTRACT It is known that, Turkey has experienced significant economical and human loss after the natural disasters occurred in the past due to its geographical location and progression of urban development. For the determination of functional and effective natural disaster strategies, it is very important that the kinds of natural disasters which was came into existence in the past and loses caused by these disasters should be known. Natural disasters considered in this study are earthquakes, landslides, floods and snow avalanches which caused a large quantity of property and human losses in Turkey and the results of these natural disasters are shown in the tables. While making these tables, it is taken into consideration that where the natural disasters occurred in Turkey and how much loses took place in addition to the distribution of the number of the accommodation units affected natural disasters. With the help of these tables, a certain number of results are derived and different evaluations are made. As a result of these results and evaluations, it is seen that the large portion of Turkey have natural disaster risks and Turkey has lived a lot of human and property loses due to natural disasters which have occurred throughout its history.

INTRODUCTION

In a general sense, a disaster can be defined as the results of natural or human origin incidents, which cause physical, economic and social losses by interrupting or halting the social life and which the community cannot overcome through its own facilities. On the other hand, a natural disasters are losses that are caused by natural movements of earth and air suddenly or in a certain time period. Sometimes, human beings can be the effective factor having a triggering role in the creation of such disasters

In the annual disaster report of International Red Cross Committee (ICRC), depending on the results of the global warming, 427 natural disasters in 2007 in total were identified and the increase in the natural disasters were declared to be 70 % in comparison with the year 2004. Besides, it was reported that loss of lives were doubled by reaching from 600 thousand to 1.2 million in the decade between 1997-2006 in comparison with the decade between 1987-1996; natural disasters increased by 60 percent and the average annual number of human beings affected by these disasters increased from 230 million to 270 million [1].

In some of these natural disasters, serious number of deaths has occurred. In December 26, 2004 a huge earthquake caused big tsunami waves and these waves caused the death of more than 226000 people by affecting 16 countries (Alexander, 2007). Again in December 26, 2003 in a 6.4-magnitude earthquake occured in Bam city of Iran 35000 people out of 142000 people died and 80 % of the buildings collapsed (Naeim, et. al., 2005). Lately, in the 7.9-magnitude earthquake occurring in May 12, 2008 in China Sichuan, average people of 60000 died [2].

This situation makes one think of the fact that human beings are hopeless against the natural disasters and this hopelessness seems to continue in spite of the developing technology. Besides, some institutions share various suggestions about the precautions to be taken for the natural disasters and go towards a common work place regarding the

things that need to be done. In this respect, this caused an appearance of a new disaster policy in the international arena because of reasons such as the growth of amounts of needed aid, the tendency of risk taking of especially developing countries for economic reasons and becoming insensitive to disasters. The main reason of this new strategy is to take the concerns from the post-disaster to the pre-disaster phase, to take some precautions to decrease the risks and lastly to decrease social and economic problems along with the loss of goods and lives (Balamir, 2007).

Turkey is a country, which is always under the danger of natural disasters because of its being on one of the active seismic belts, its topographical structure and meteorological characteristics. Beyond the losses of lives and goods, natural disasters also cause the occurrence of health, shelter, food, labor and forensic problems. Because of the applications of urbanization and city planning, these problems appear even in a small-scale disaster. Therefore, the main aim in this study is to summarize the natural disasters that Turkey faces, in the light of the information and statistical data gathered.

It is a fact that such disasters as erosion, drought and global warming are natural disasters, which has not caused loss of lives until now in Turkey but which will affect the country in socio-economic respect. These natural disasters, which need to be mentioned specifically, are not discussed because of lack of space in this study. Natural disasters analyzed in this article are earthquake, landslide, flood and snow avalanche, which have caused the most loss of goods and lives until now.

GENERAL EVALUATION OF NATURAL DISASTERS

Natural disasters occuring in Turkey cause destruction and loss of lives seriously. Loss of lives, injured people, homeless and affected population and economical losses, which are caused by the natural disasters occuring since 1990 are summarized in Table 1.

Besides, the distribution of natural disasters based on the number of settlement units affected from a disaster, the number of events of disaster, the number of events given decision of conveyance and the number of effective conveyance housing are shown in Table 2. Here, the concept of effective conveyance housing is the number of housing whose conveyance is accepted after the natural disaster when the disaster is being approved as "effective". The effectiveness of a disaster is accepted by the Ministry of Public Works and Settlement based on the regulations about the Main Regulations Concerning the Effectiveness of Disasters on General Life (1968). In this regulation there is a passage stating that the disaster is effective on the general life of that place in the cases if: at least 1/10 houses in villages or subdistricts consisting of more than 100 housing, at least 20 buildings in districts having population up to 5000, 25 buildings in places whose population is between 5000-1000, 30 buildings in cities and districts having population of between 10000-30000, 40 buildings having a population of between 30000-50000, 50 buildings in places having a population more than 50000 collapse or become damaged so much to prevent housing. Also, in some cases as the 1/3 damage on agricultural products and damage on public utility services in that region (road, water, electricity, drainage, etc.) so badly to prevent usage or work again, the statement of "effectiveness on the general life" can be accepted.

Table 1. Property and human losses of the important natural disasters in the last years [The Ministry of Public Works and Settlement, the General Directorate of Disaster Affairs].

Locations of natural disasters	Years	Human losses	Injured People	Homeless	Affected population	Economical losses ($)
Earthquake Erzincan	1992	653	3.850	95.000	250.000	750
Snow Avalanche (14 event)	1992	328	53	11.600	30.000	25
Snow Avalanche (31 event) (East and Southeast)	1993	135	95	1.100	300	10
Mud Flowing (Sedirkent-Isparta)	1995	74	46	2.000	10.000	65
Earthquake (Dinar)	1995	94	240	40.000	120.000	100
Flood (İzmir)	1995	63	117	6.500	300.000	1.000
Earthquake (Çorum-Amasya)	1996	0	6	9.000	17.000	30
Flood (Batı Karadeniz)	1998	10	47	40.000	1.200.000	1.000
Earthquake (Ceyhan-Adana)	1998	145	1.600	88.000	1.500.000	500
Earthquake (İzmit Körfezi)	1999	17480	43.953	675.000	15.000.000	13.000
Earthquake (Düzce)	1999	763	4.948	35.000	600.000	750
Earthquake (Afyon-Sultandağı)	2002	42	327	30.000	222.000	95
Earthquake (Bingöl)	2003	177	520	45.000	245.000	135
TOTAL		19.964	55802	1078200	19494300	17460

Table 2. The distribution of the number of disaster events, effective convayence hausing and settlement units affected from a disaster since 1950 [The Ministry of Public Works and Settlement, the General Directorate of Disaster Affairs, Department of Disaster Survey and Damage Determination].

Disasters	**The number of settlement units affected from a disaster/ (%)**	**The number of events of disaster / (%)**	**The number of events given decision of conveyance**	**The number of effective conveyance housing (%)**
Landslide	4161 / (%34.18)	12794 / (%42.63)	6347	63969 / (%25.40)
Rock Falling	899 / (%7.38)	2769 / (%9.23)	1367	20836 / (%8.26)
Flood	1861 / (%15.23)	3873 / (%12.91)	2249	26081 / (%10.36)
Earthquake	2952 / (%24.25)	5267 / (%17.55)	4807	106838 / (%42.42)
Snow Avalanche	207 / (%1.7)	670 / (%2.23)	292	4112 / (%1.63)
Other Disasters	665 / (%5.46)	1076 / (%3.59)	658	8200 / (%3.25)
Multiple Disasters	1427 / (%11.73)	2067 / (%6.89)	1058	19102 / (%7.57)
Not Evaluated Disasters		1491 / (%4.97)	704	2723 / (%1.1)
Total	12172	30007	17482	251861

EARTHQUAKES

Turkey is situated on the Mediterranean, Alp, Himalayan seismic belt, which is an active one lying between Azor islands and South East Asia. Three major tectonic plates that are African, Arabian and Eurasian plates and two small plates such as the Aegean and Anatolian plates affect Turkey. Vast majority of earthquakes in Turkey take place in the North Anatolian Fault System, East Anatolian Fault System, Aegean Graben Systemand the Southeastern Anatolian Generation Overlap zones, which have boundaries with the plates around the Anatolian plate. In Turkey between 1988 and 1999, Global Positioning Studies (GPS) were made systematically. The results of these measurements, which provide important information about plate movements and annual displacement rates in the large fault zones, are summarized below [Ergünay, 2007; Country Strategy Report on Natural Disasters in Turkey, 2004].

- African plate moves in the direction of north-east at a speed of 10 mm per year relative to the Eurasian plate.

- Arabian plate moves in the direction of north, north-east at a speed of 23 mm per year.

- Anatolian plate moves in the direction of west at a speed of 25 mm per year relative to the Eurasian plate.

- Collapse basins of Western Anatolia move at a speed of 25 mm because of the north-east, south-west widening.

- Western Anatolia moves towards south-west at a speed of 30 mm per year.

In Turkey seismic zone map, places of expected earth acceleration of 0.40 g or more are defined as I. degree; of places between 0.30 g and 0.40 g are defined as II. degree; of places between 0.20 g and 0.30 g are defined as III. degree; of places between 0.10 g and 0.20 g are defined as IV degree and places less than 0.1 g are defined as V. seismic zone. Here, g is gravitational acceleration with a value of 9.81 m/sn2. In Table 3, how many people live in which seismic zone, how much space these zones cover, and the number of dams and industrial companies as a percentage according to the seismic zone are given.

Table 3. The percent of surface area, population, industry and the number of dams according to Turkey seismic zones map [The Ministry of Public Works and Settlement, the General Directorate of Disaster Affairs].

Seismic zone	Area (%)	Population (%)	Industry (%)	The number of dams (%)
I	42	45	51	46
II	24	26	25	23
III	18	14	11	14
IV	12	13	11	11
V	4	2	2	6

As seen in Table 3, only 4 % of our country's territory is on the V. degree seismic zone and 66 % is on the I. and II seismic zones. Besides, while only 2 % of the population live on the V. degree seismic zone, a vast majority of 71 % of the population live on the I. and II degree seismic zone. On the other hand, the fact that 51 % of the industrial companies and 46 % of the dams are situated directly on the I. degree seismic zone is a significant fact to be considered. Also, by stating that some province are on various degree seismic zones as well as the fact that even the center of city and the district may be on different seismic zones, in Turkey average of 35 provinces, 406 districts and 335 subdistricts can be said to be located on the I. degree seismic zone. In the earthquake zone II, there are 22 provinces and 176 districts and 152 subdistricts, and the number of provinces, districts and subdistricts in I and II degree seismic zones are 71 %, 69 % and 71 % respectively.

Among these earthquakes, there are three expected ones in the magnitude between 5.5 and 6 and the ones having magnitude between 5 and 5.5 will cause structural damage. The earthquakes greater than 6 will cause loss of lives. Especially in the Eastern Anatolia region while the earthquakes between magnitude 5 and 5.5 cause serious structural damage, it is observed that they even cause loss of life. While the size 6 to 7 earthquakes in our country recur for 3 to 7 years, earthquakes of greater magnitude 7 which cause severe loss of goods and lives are experienced in an average time period of

7-10 years (Arıoğlu et al., 2006). In table 4, major earthquakes in Turkey in the last century are displayed.

Table 4. The major earthqukes occured between 1900 and 2008 (Pampal ve Özmen, 2007).

Locations	Years	Magnitude	Human losses	Injured people	The number of buildings	Intensity (MSK)
Malazgirt	1903	6,7	2626	-	4500	IX
Mürefte	1912	7,3	216	466	5540	-
Hakkari	1930	7,2	2514	-	3000	X
Erzincan	1939	7,9	32962	-	116720	X-XI
Niksar-Erbaa	1942	7	3000	6300	32000	IX
Tosya-Ladik	1943	7,2	2824	-	25000	IX-X
Bolu-Gerede	1944	7,2	3959	-	20865	IX-X
Varto-Hınıs	1946	5,7	839	349	1986	VIII
Yenice-Gönen	1953	7,4	265	336	9670	IX
Varto	1966	6,9	2394	1489	20007	IX
Gediz	1970	7,2	1086	1260	9452	IX
Bingöl	1971	6,7	878	700	5617	VIII
Lice	1975	6,9	2385	3339	8149	VIII
Çaldıran	1976	7,2	3840	497	9552	IX
Erzurum-Kars	1983	6,8	1155	1142	3241	VIII
Erzincan-Tunceli	1992	6,8	653	3850	6702	VIII
Kocaeli	1999	7,4	17479	43953	77342	X
Bolu-Düzce	1999	7,2	763	4948	31197	X
Bingöl	2003	6,1	177	520	1029	VIII

As can be understood from Table 4, the number of earthquakes that happened between 1900 and 2008 and that caused severe loss of goods and lives is around 20 on average. And this means that our country will face earthquakes that cause severe loss of goods and lives every five years. A remarkable matter here is that when the values stated in Table 4 are compared with the losses in the developed countries, there are significant differences. That is, in the earthquakes happening in Japan in 2003 having magnitude of 7.6 and 8, some people in the first one and only one man in the second one died because of heart attacks.

FLOODS

Climate, vegetation, physiological characteristics and human activities in any region are the paremeters which become effective in water floods. While all the climactic characteristics may trigger the formation of flooding, rainfalls on the relevant basin are the main cause for the formation of flooding. At the same time, while the rainfall on a basin for a long time do not cause flooding, the reverse may occur if the same amount of rainfall fall on the same basin for a short time because a large amount of water cannot

be kept by plants and soil. The sudden snow melt resulting from instant extreme increase of the air temperature or the rainfall on a snow cover in a hot air, can cause the flooding. Vegetation is also one of the important reasons in the cause of flooding. In regions that are rich in terms of vegetation when compared to regions where vegetation is relatively weak, even the region would have a more humid climate, and more amount of rainfall, the possibility of flooding is less likely because of the vegetation's decreasing the water flow rate or causing water absorption. The reason why there is less flooding in the humid tropical zones in spite of the excessive rainfall is that those regions are ones rich in vegetation. A region's slope, soil permeability, soil characteristics, proximity to sea-distance, height above sea level and upgrade status such as mountain or hill distribution determine the physiological properties of that region and all these elements directly affect how often will the flooding be seen. Human activities such as bents and dams on the rivers, the lack of rainfall drainage system in cities or their inadequate structuring also effect the flooding there. The reason of the excessive loss of goods and lives in the floods occurring in Turkey is the intensive construction in the stream beds and closing of the rivers passing through the city.

Along with the fact that Turkey is a country facing the danger of flooding because of its geomorphologic structure, illegal structuring in stream beds, heavy snowfall seen in many of its region, and the rainfall it gets especially in spring; the decrease in the vegetation with the forest fires and deforestation especially in recent years increase the cases of flooding. Also seen in almost every region of the country, erosion causes an increase in the amount of water. Climate change is evaluated as the reason of the unexpected flooding but there are no exact data for Turkey about the effects these changes depending on the amount of rainfall, rainfall times, the intensity and duration of these rainfalls. The effects of the climate change on the basins that are vulnerable to dangers of flooding is an important subject to be studied.

Floods in Turkey generally occur after the long-lasting rainfalls that the soil becomes saturated and after the sudden downpour of rain after these. In arid or semi-arid regions, less vegetation, high slopes and sudden downpour of rain cause flooding and large amount of loss of goods and lives occur because of the flooding. Floods seen in Izmir and Isparta in 1995, in 1998 in Hatay and the Western Black Sea region and in 2007 in Diyarbakir, Sanlıurfa, Mardin and Batman are floods that caused by the sudden and severe rainfalls coming after the soil becomes saturated because of the continuous rainfalls. Within the natural disasters occurring in our country, flooding are the disasters where the greatest loss of goods and lives are seen after the earthquake. Each year, the economic loss caused by this disaster is estimated to be approximately 160 million TL (Sipahi, 2008). State Hydraulic Works (DSI) makes an investment on protection from flooding an average 35 million TL per year.

Statistical information about flooding is collected by General Directorate of State Hydraulic Works and published annually as yearbooks. Between the years 1955-2007, 1235 people died and 61000 housing became unusable because of flooding in Turkey. Since 1955, 4390 engineering construction has been done and approximately 3300 units of settlement have been protected from flooding by DSI [Ergünay, 2007; Country Strategy Report on Natural Disasters in Turkey, 2004]. In addition, the flood damage is divided as 45 % agriculture and animal husbandry, 32 % building and infrastructure, 7 % portable goods and vehicles and 16 % other losses. In Table 5, loss of goods and lives occurring in the floods between 1989 and 2007 are given.

Tablo 5. Property and human losses due to the floods occured between 1989 and 2006 [General Directorate of State Hydraulic Works].

Years	The number of floods	Human loss	Flood damage (ha)
1989	10	1	9500
1990	26	57	7450
1991	23	23	15770
1992	14	1	690
1993	2	---	60
1994	9	4	1680
1995	20	164	201100
1996	4	1	11000
1997	1	---	1390
1998	2	57	7000
1999	1	3	---
2000	4	---	8 066
2001	6	8	43297
2002	2	27	510
2003	21	7	64200
2004	19	3	25750
2005	33	14	13855
2006	19	45	85810

Due to the rapid increase in migration in Turkey, there has been an intense urbanization. With the urbanization, many areas are covered with asphalt or concrete roads and this decreases the land's water absorption capacity and rain flow rate increases. The risk of sudden flooding increase with the reasons such as the inadequate infrastructure systems of the settlement units, their lack of response to the needs, lack of making rainfall drainage systems or their remaining dysfunctional, illegal structuring in the river beds, not making the appropriate city plans and wrong usage of the land. In Table 6, cities that are most exposed to the risk of flooding are shown.

Table 6. The cities that are most exposed to the risk of flooding [General Directorate of State Hydraulic Works].

Rank	Cities	Annual frequency	The population exposed to the risk
1	İzmir	3,484	450000
2	Rize	1,841	55000
3	Kahramanmaraş	1,608	35000
4	Denizli	0,596	20000
5	Trabzon	0,508	32000
6	Antalya	0,408	400000

7	Kırıkkale	0,396	10000
8	Balıkesir	0,172	15000
9	Bartın	0,132	60000
10	Bitlis	0,132	10000
11	Sivas	0,132	10000
12	Van	0,132	70000
13	Batman	0,044	5000
14	Zonguldak	0,024	25000
15	Ankara	0,024	100000

LANDSLIDES

Landslide is the name given to the moving in the direction of the slope and replacement of the ground or artificial fillers under the influence of such factors as gravity, geology and water content. A region's climate, rainfall amount and how much rainfall has fallen in time, the presence of the risk of flooding, altitude characteristics, valley slopes, land topography, vegetation, soil properties, soil layers extension direction (landslide is more visible if the layers are parallel to the slope) are natural characteristics to be taken into account in terms of the existence of landslides. In addition to these natural features, some human activities have more effect on the occurrence of the landslides and the amount of losses than the natural characteristics. Incorrect placement planning, shanty settlements especially on prone areas in urban areas, not taking into account the characteristics of the land in the time of road constructions are negative human activities affecting the loss of goods and lives occurring after the landslides.

Often being in the form of slipping, landslides may be in the mixed form as sometimes being movements of fall, slide, or flow and combination of some of them. Therefore, it is hard to classify landslides under different names and different types but they can be classified in general as: a) Falls b) Slides c) Flows d) Flows of Clay in terms of the type of the movement of the soil. Landslides can be noticed before with a careful observation. Before the landslide occurs at a location, one or more of the indications stated below can be seen together:

- Occurrence of cracks in the building basements with the cleavages and partings, constantly widening cracks in the floors and walls of the building,

- Cracks and breaks in sewer channels, occurrence of cracks and leaks in the underground water pipes,

- Occurrence of water saturated areas, water sources and water leakage, which did not exist before,

- Bending of the trees, fences, and telephone poles towards the land’s downward slope,

- Observations of replacements of the fences.

In our country, lands that are exposed to the danger of landslides are the Eastern Black Sea Region, especially Trabzon and Rize provinces; in the Central and Western Black Sea Region, provinces of Kastamonu and Zonguldak. Cities where the least landslide is observed are Kırklareli, Mardin and Şanlıurfa. Total number of landslides occurred in Turkey between the dates 1950 and 2008 is 13746 (Gökçe et al., 2008). In Table 7, ten settlement units that are exposed to the risk of landslide are shown.

Table 7. The locations that are most exposed to the risk of landslide [The Ministry of Public Works and Settlement, the General Directorate of Disaster Affairs, Department of Disaster Survey and Damage Determination].

Rank	Cities	Locations	The number of the events	Disaster stricken
1	Karabük	Yenice	219	1434
2	Malatya	Hekimhan	121	1327
3	Bursa	İnegöl	179	1304
4	Trabzon	Maçka	231	1078
5	Bartın	Ulus	238	1015
6	Muş	Merkez	58	750
7	Ankara	Mamak	84	740
8	Bingöl	Merkez	75	720
9	Kars	Kağızman	65	720
10	Corum	Sungurlu	74	640

SNOW AVALANCHES

Snow avalanche can be defined as the rapid slide of the accumulated snow mass on the rugged and mountainous terrain in consequence of a movement starting with both internal and external triggering. Snowslide briefly is a flow motion of snow layer or layers in the direction of the slope gradient. As snow layers would have different characteristics from each other, snow avalanche sometimes occurs as a result of the slide of all the layers on the ground, or the slide of one or more layers' from the bottom layer or layers [Taştekin, A. 2007).

Along with the fact that, natural effects such as fresh snow depth, wind, shaping of the snow, temperature and vegetation cause the occurrence of the snowslide, some artificial effects also cause the occurrence of the snowslide. Among these artificial effects, activities that soldiers, hunters, local community and people who are interested in winter sports can be counted.

Natural factors that increase the risk of snow avalanche are summarized as items.

- If the air temperature falls suddenly after the snowfall, the interaction between the new snow layer and the old snow layer would be less and therefore the top layer would loosen when the air gets warm and increase the risk of snowslide.

- In the occurrence of snow storm, snow accumulation is seen on the slopes where the wind is relatively less. Snow, which is accumulated as waves on the top increase the risk a snowslide on the bottom.

- The more the slope, the higher the risk of snowslide.

- Slopes which have the most devastating effect and cause frequent snowslides are in the direction of northeast and southwest and the risk of snowslide in the shade is higher than in the sunny areas.

- As a result of more snow melting with the effects of warm weather and rain, amount of water on the snow cover increases. Accordingly, the friction between the snow layer and the ground decreases because the small objects that provide the roughness on the ground are kept under water. For this reason, more snowslides occur in spring.

Table 8. The distribution of the events of snow avalanches according to years (1990-2007) [The General Directorate of Disaster Affairs, the Avalanche Research-Development, Reconnaisance and Prevention Branch].

Years	Events	Human losses	Injured people	The number of effective conveyance housing
1990	6	4	1	47
1991	16	7		267
1992	157	443	108	1762
1993	60	139	95	146
1994	11	39	7	0
1995	6	10	2	68
1996	7	8	1	67
1997	10	16	3	88
1998	22	12	5	178
1999	15	11	5	31
2000	21	21	15	0
2001	11	2	1	120
2002	46	13	8	181
2003	67	15	10	153
2004	67	9	3	59
2005	54	8	8	125
2006	104	13	16	170
2007	159	4	13	32

RESULTS and SUGGESTIONS

Knowledge of natural disasters occurred in the past and the losses caused by such disasters are one of the important factors that will ensure the creation and practical application of country strategy for disasters. For this reason, significant disaster types such as earthquake, landslide, flood and snow avalanche are analyzed and the results caused by these disasters are summarized as tables. As can be understood from the tables, Turkey has faced significant losses in the natural disasters until now and the large part of whom is under the risk of disasters.

Many of the settlements in Turkey, in order to take advantage of the natural resources or the positional advantages, such as the nature of the presence of water, fertile land, transition points and regions of strategic military importance, are established in the areas where the risk of disaster is greater. This situation is the main reason of the greatness of the damages originating from disasters occurring, as summarized in previous chapters of the study. Beyond that, especially in the last 30 years in Turkey a rapid urbanization process has been experienced and the already disaster-risky bigger cities have grown because of the large portion of the country's population migrating from the rural areas to the cities with various socio-economic reasons and in this way distorted urbanization has shown itself. As a consequence of this stuation , with the increase of housing stocks that are built without engineering services, inadequate and uncontrolled, the number of losses after a potential natural disaster has increased in these cities. In reducing the risks of natural disasters, the greatest challenge that Turkey faces is to make the housing stock disaster-resistant, especially earthquake-resistant. Therefore, first of all, engineers who are equipped with sufficient knowledge and skills in the making of new disaster-resistant structures and strengthening existing structures are needed to be trained.

ACKNOWLEDGMENTS

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TSUNAMI DISASTER EVACUATION PLAN

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ABSTRACT Earth and Marine Sciences Institute of TUBITAK, Marmara Research Center took part in European Community 6th Framework project titled Scenarios for Hazard Induced Emergencies Management, SCHEMA. As part of this project, tsunami evacuation maps were studied, which is disseminated within the framework of this paper. First, the importance of tsunami for Turkey is mentioned, and then the purpose, the structure and the components of a tsunami evacuation plan are elaborated giving examples from Honolulu, Hawaii and Stromboli, Italy.

INTRODUCTION

A recent coseismic tsunami in Izmit Bay caused by the Kocaeli earthquake on 17 August 1999 has once more emphasized the fact that a tsunami might occur as the consequence of a large-scale earthquake. When the tsunamigenic potential of the Sea of Marmara is assessed, over 40 tsunamis have taken place during the course of an observational period of about 2000 years (Altinok and Ersoy, 2000). The areas in which such waves have concentrated are Izmit Bay, Istanbul coasts, Gemlik Bay, Kocaeli Peninsula and Gelibolu coasts (Figure 1) (FORESIGHT, D87). These facts point out to the necessity of availability of a tsunami evacuation plan in the hands of local authorities of the coastal towns to let them increase public awareness against tsunami.

In this paper, first the purpose of a tsunami evacuation plan will be explained, then structure and components of a tsunami disaster evacuation plan will be elaborated giving examples from the tsunami evacuation maps prepared for Honolulu, Hawaii and Stromboli, Italy.

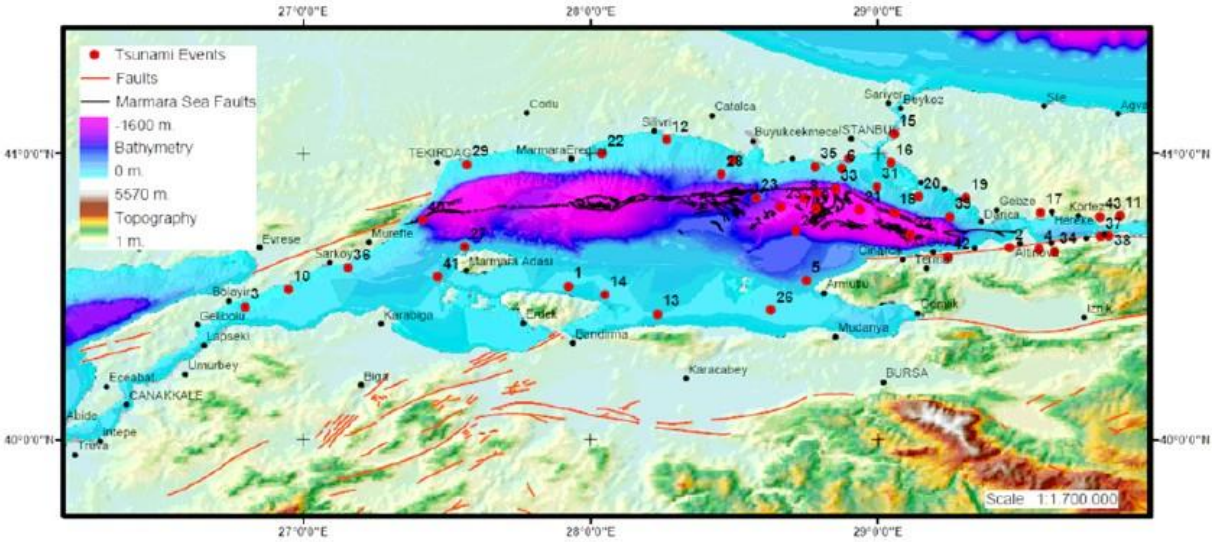

Fig. 1. Historical Tsunami events in the Marmara Sea (120-1999 AD).

PURPOSE OF TSUNAMI EVACUATION PLAN

Evacuation plan is an essential tool for any emergency situation. The way they have to be devised, distributed, tested and implemented are subjects of the disaster management strategy, namely of the disaster preparedness and of the emergency response.

As regards tsunami, evacuation from potentially attacked areas is the most obvious way to save lives. The main purpose of a tsunami evacuation plan is that of allowing people to reach a position safe from tsunami waves in a very short time and in an orderly way, avoiding chaos and panic, and avoiding as much as possible deadlock conditions, such as traffic jam and blocks. Conditions may be very different from place to place along the coast, and on a given place can be very time dependent, and moreover, evacuation capability can be age and gender dependent, business dependent, etc. Therefore, evacuation plans may cover a very broad range of sophistication, from very basic to very elaborated, depending on the means employed by the local authorities and by the expertise in terms of human resources they are able or willing to mobilize.

What is certain is that an evacuation plan must be devised for local coastal communities and adapted to any given specific coastal place. They cannot be implemented on a regional (i.e. supranational) or national basis, though many elements, especially as regards the actuators structure, i.e. the line of command and the designated authorities, can be defined on a national level (SCHEMA D1.1, 42/90)

STRUCTURE AND COMPONENTS OF AN EP

In strict sense, an Evacuation Plan (EP) consists

i) of the identification of the area that can be inundated by a tsunami along a given stretch of coast, and

ii) of the identification and of the related signaling of the routes that can be used by people (who at the time of the tsunami warning happen to be in such evacuation zone) to reach safe places from the waves either because they are too far from the coast (outside the evacuation zone) or because they are too elevated above the sea level (vertical shelters). In addition,

iii) the EP should indicate the authority responsible to order the people to evacuate, to confirm and/or to clear the evacuation conditions.

These are the basic elements of an EP, and it is relevant to stress that, though basic, these are difficult to devise without proper knowledge of tsunami physics, of the coastal morphology and occupation, and expertise. This is why tsunami EPs which should be a basic tool for any tsunami-prone coastal community, are not so common and are either non-existing or surrogated by a series of minimal recommendation, such as "in case you feel a shock and you are on the beach, leave the beach and go to higher ground".

Seen in the sense given above, an EP is a tool to determine the correct community response to the tsunami emergency. In a broader sense, however, one can consider within the EP even all those activities that serve to make it really efficient, i.e. the activities usually inscribed in the tsunami preparedness circle. Indeed, the coastal community must be properly informed of the EP that has been adopted by the authorities, with all the possible means, and must be trained to implement the plan by a series of programmed exercises and drills, which should be reiterated in order to test on one side the EP itself, and on the other side the reaction of people. Community exercises or targeted exercises (for specific population segments: e.g. schools, hospitals…) are reputed essential especially for infrequent and high-impact events like tsunamis.

Devising adequate and effective implementation plans requires a number of different skills. First of all, a pre-condition for an EP to be devised is the computation of an Inundation Map for the same area, which assesses the extension and probabilities of coastal zones that can be flooded by tsunami and damaged by water and waves. Capability of properly computing such maps is today more developed than in the past, but the task remains hard, especially owing to the uncertainties related to the tsunami sources and to the limited knowledge of the coastal bathymetry, often limiting the value of numerical tsunami simulations. It is also noted that often tsunami EP are connected to projects of tsunami vulnerability and risk assessment, though in principle they are a very different issue and could be at most included in the tsunami risk mitigation programs. More correctly, as already stated the tsunami EPs are classed within the category of the emergency response.

In recent years, there have been efforts to produce and test EPs in many coastal communities that have increased awareness and sensibility to tsunami threat and have agreed to implement tsunami programs. The Intergovernmental Coordination Group's of the International Oceanographic Committee (IOC) that have been established to coordinate the activities of the Tsunami Warning System (TWS) in the oceans around the world have not established explicit guidelines for the redaction of the EPs, though they recommend that all the IOC member states should adopt specific EPs for their tsunami-prone local communities, being aware that tsunami warnings are almost useless if the people receiving the messages are not educated to adopt the proper response, which is exactly what an EP has to specify. As an example of well defined and detailed EP, one can give the one elaborated by the Department of the Emergency Management of the Honolulu County, Hawaii. The EP is considered within the emergency response procedure that delineates the tasks and the responsibilities during the emergency phase of all the actors that have a role to play (e.g. the person in charge at the emergency scene, the Chief Executive Officer during an emergency, the Fire Brigade, the Health and Safety Department, the Medical Department personnel, the Vehicle/Transportation system responsible, etc). In addition, the EP comprehends a detailed list of instructions to the general public, specifying the action required for any level of warning, such as "If an Urgent Tsunami Warning is issued without the shaking associated with a local earthquake or tremor, immediately leave or vertically evacuate all tsunami evacuation zones." Another example of instruction to reach the vertical shelter selected for a given area is: "If you are in a tsunami evacuation zone and must evacuate immediately to a safer building, we have arranged for you to move to the (name of specific building). The (name of specific building) should provide you adequate protection from the approaching tsunami if you move to the third floor or above." These guidelines are made known to the population trough educational campaigns, leaflets, web pages, etc. Also special care is

taken of measurements to treat and govern the reaction of children and of the weakest segment of the population to disasters: "In a disaster, even the child who is usually competent and unafraid may react with fear and considerable anxiety to an event that threatens the family. Since adults also react emotionally with normal and natural fear to disaster, the child becomes terrified, taking parental fears as a proof that the danger is real…"

Evacuation zones, routes and tsunami shelters should be mapped in specific evacuation maps. Again, let us take the Honolulu County as an illustrative example. Here one of the 19 evacuation maps plotted by the Department of Emergency Management is given (Figure 2): it is the map 12 referring to the coastline between Waialua Bay and Sunset Beach, and includes the indication of the inundation zone, of all possible evacuation routes and of one vertical shelter. Evacuation maps stored in computer archives are useless to people on the coast. These maps have to be exhibited as posters along the beaches and distributed in coastal public places.

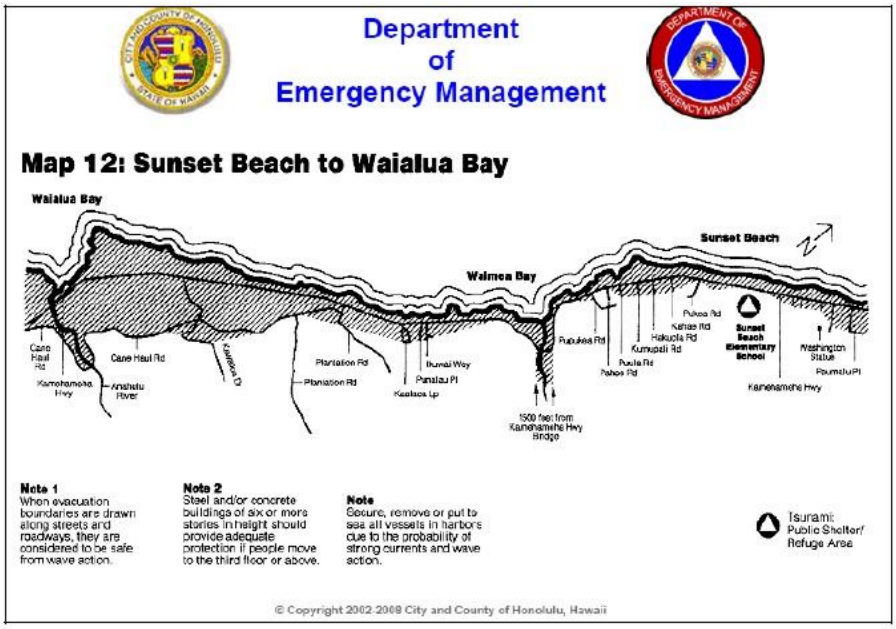

Fig. 2 Example of a tsunami evacuation map elaborated by the County of Honolulu, Hawaii (SCHEMA D1.1, 44/90).

But, most importantly, irrespective of the availability of the map on a beach, evacuation routes should be indicated with clearly recognizable and understandable tsunami signs. These signs should be used to signal the evacuation zone, also called the tsunami hazard zone, the evacuation route as well as the safe zone, also known as the safety site. Examples of such signs are taken from the Oregon Emergency Management Department, in recognition of the fact that Oregon was one the first states of USA that implemented tsunami preparedness and emergency plans for coastal communities (see Figure 3).

Fig. 3. Signs for evacuation route and for evacuation site placed on the Oregon coasts, USA (SCHEMA D1.1, 44/90).

As mentioned above, the coastal communities that have adopted an EP are much less than required, but in many countries in the Indian Ocean and in the Pacific Ocean one may find examples of tsunami protection strategies, including EPs. If one turns to the Euro-Mediterranean region, however, the situation is quite different, since, conversely, EPs are very rare. The first example of an adopted EP comes from Italy, and more specifically from the island of Stromboli that in 2002 was affected by a double tsunami. This was caused by two distinct landslides that a few minutes apart detached from the Sciara del Fuoco, on the northwestern flank of the volcano, during an eruptive crisis. The tsunamis, that attacked the coasts with waves up to 7-9 m high in several parts of the island, did not cause fatalities since they occurred in December, when the tourist season is over and coasts are unpopulated. The Civil Protection authorities gained awareness that repetition of such events is not improbable and set up a very local TWS, inclusive of a sophisticated monitoring network of the eruptive activity, of an acquisition, processing and decision centre placed in the island, of a set of sirens to disseminate warning to the population, and of an EP. Figure 4 shows the Evacuation Map in the areas named Ficogrande and Punta Lena on the northern coast of Stromboli, with the inundation area (yellow), the evacuation routes (green arrows), and the safety areas (green polygons), that are all located above the 10 m iso-line, which is estimated to be the maximum height attainable by tsunami waves at the coast (SCHEMA D1.1, 42-45/90).

Fig. 4. Evacuation Map for the northern coast of Stromboli, which is the most populated in summer season, but is almost totally uninhabited in winter. (SCHEMA D1.1, 45/90).

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URBAN RECONSTRUCTION PROCESS IN THE POST MARMARA EARTHQUAKE PERIOD

(Permanent Housing's Practices and Results)

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Key Words: Earthquake, Housing, Damage, Reconstruction, Disaster Management, Urban Reconstruction.

ABSTRACT Located on one of the major earthquake zones of the world, Turkey is under serious threat of earthquakes and suffers badly from them. Because housing and population level is very high in the regions on active faults, many lives and private properties were lost as result of the earthquakes. Underdevelopment, rapid and irregular urbanization contributed to the detrimental effects of the earthquakes, as well. One of the most developed and densely populated regions of Turkey, the western region was seriously damaged from the earthquake in 1999. The earthquake took many lives, caused injuries, economic losses and damages in buildings.

This paper aims to discuss not only the successful practices which eliminated the negative impacts of the earthquake so soon, but also the problems experienced during the creation of new urban areas.

INTRODUCTION

Natural disasters can suddenly destroy all the values established by human being within many years and is an important problem that today's world has to deal with. Nowadays, natural events increasingly turn into disasters by effects of quantitative and qualitative fast and unplanned urbanization, social, economical and political events.

Due to its geographical localization Turkey is faced with many disasters especially earthquakes. Also with the effects of fast and unplanned urbanization; important economical, social and environmental losses are emerging. Necessary financial resources that are needed to resolve these losses also effect the national development negatively. In Turkey, because of the laws that are in force; public resources are being used considerably to appease the negative effects and to provide new urbanization in a significant amount.

After the earthquakes that took place in August 17, 1999 and November 12, 1999, appease of the destruction and the rebuilding of the houses that were destroyed was conducted by public administrations. In the last few years, expenditures for the reconstruction works before during and after disasters have been financed with the credits provided by international financial bodies, besides public resources. In Turkey the private sector and individuals do not contribute sufficiently to the disaster losses. Determining damages, providing temporary settlement, determining where the new cities will be founded, for who the houses will be built for and all other operations are within the responsibility area of the public.

After the Marmara Earthquake, an important amount of effort had to be put in to rebuild the ongoing life, an important ratio of workforce and resources had to be used. Whether it is internal resources or works made by using external credits, these were important

performances to clear away the negative effects of the earthquake as soon as possible. The cities destroyed due to insufficient resources were rebuilt just as they were before the disaster in a short period of time. Despite many problems that happened, disaster victims were satisfied with the accommodation solutions that were provided within a short period of time. Providing to live in stable residences only two years after the earthquake has been evaluated as an important success. In this success, the solidarity culture of the Turkish public is an important supportive factor.

URBAN STRUCTURE OF TURKEY

Urbanization, in the strict sense, represents the increase of city numbers in quantity and the increase of population in cities (Keleş, 2006).Turkey is administratively divided into 81 states. In this division "more than 10 000 population" is a determinative factor. Industrialization and economic improvement is an important factor in the increase of the city population.

After the establishment of the Republic of Turkey, economic development and industrialization efforts increased; agricultural automation and excess workforce in agriculture forced people to search for life facilities in cities and therefore increased the population of the cities in the west. In the 1950's immigration from rural areas to cities increased gradually. Between the years of 1927-1997 population amount and population intensity increased 5 times more. By the year 2007 the city population ratio in comparison to the general population is 69%. However, because of the insufficient numbers of city houses stocks to accommodate this highly increasing city population, unlicensed, uncontrolled and unplanned housing increased greatly.

In the Turkish Republic the city planning works emerged from the new government as an anxiety for modernization, modernization of the cities and glamorization. Reaching the development level of the western countries, just like in every other field, has not been realized within the correct sensibility and therefore cities were just not prevented from squatter settlements.

This state of the cities and urbanization plays an important role in mass loss of lives and goods, economic and social losses during earthquakes or other natural disasters and technology based disasters. Thus, the reason that lies underneath the big destruction during the Marmara Earthquake is the unplanned and unhealthy urbanization.

EARTHQUAKE SITUATION IN TURKEY

Turkey is placed on the Alps –Himalaya seismic belt that starts from Azor Islands and reaches to Southeast Asia (Bagci, Gunruh And the others, 1991). It is known form historical facts that this geography has faced many big earthquakes since very old times until today; certain communities that used to live in Anatolia left their lands because of these reasons and also many civilizations disappeared completely (Bayindirlik ve İskân Bakanligi, 2000). Pamukkale, Troy and Ephesus are examples.

96% of the land in Turkey is in dangerous regions due to earthquakes and 98% of the population resides in these regions (Ozmen, Nurlu ve Guler, 1997), (Fig. 1.).

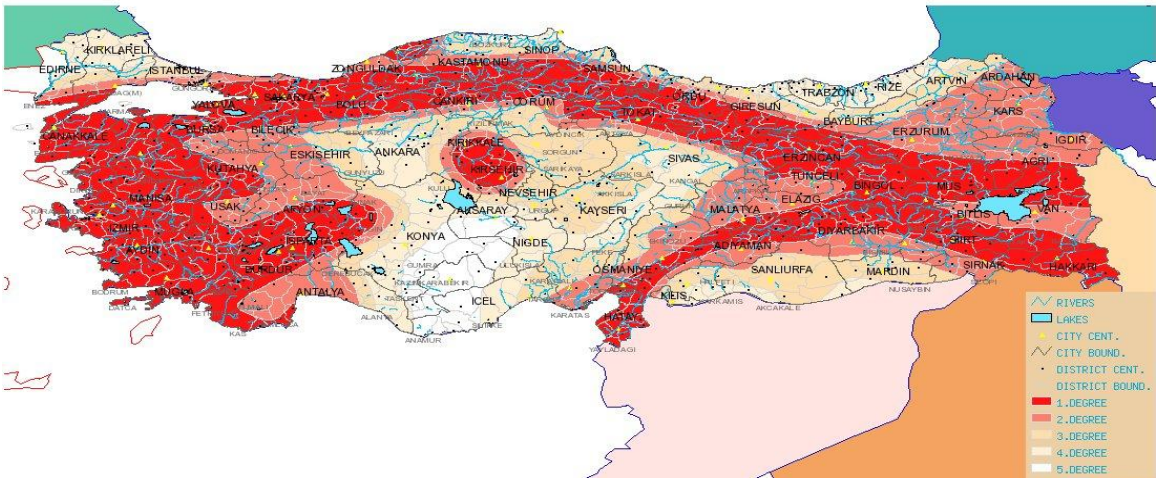

Fig. 1. Map of Earthquake Regions of Turkey (www.deprem.gov.tr (rewised)).

In Turkey, in every 10 months there is an effective earthquake and approximately in every 14 months destructive earthquakes happen (Tezcan and İpek, 1996). 130 damage causing earthquakes took place between the years of 1900-2006 and during these earthquakes records show that approximately 80.000 lives were lost, 600.000 structures collapsed or suffered extensive damage and 400.000 structures suffered mild damages (Keles, 2006).

According to these figures, in Turkey, a damage causing earthquake takes place every 7 months. Every year damage causing earthquakes cause approximately the destruction of 5844 buildings and the death of 982 people.

The Marmara Earthquake, as the century's biggest earthquake effected an area of 64.364 square meters (approximately). Kocaeli, Yalova, Bolu, Düzce and almost all of the cities of Sakarya are areas where earthquakes are the most effective.

GENERAL SPECIFICATIONS OF DISASTER MANAGEMENT

Dates of the legal arrangements regarding disaster management systems cannot be considered as very old. Since 1923, the establishment of the Republic until 1959, after each natural disaster a legal arrangement has been made about the disaster in question. The Disaster Law numbered 7269 omitted in the year 1959 covered arrangements such as recovering the damages from all of natural disasters, being prepared, intervention to disasters, recovering and rebuilding periods. This law, amongst its alike, it is the most advanced law and Turkey is one of the few countries that own a special law regarding natural disasters.

The main purpose of the law is; reducing the number of loss in lives and goods with economical losses caused by natural disasters, establishing an organization when a disaster occurs or after it to keep losses at a minimum level.

Disaster management in Turkey has a context that removes the negative effects that take place after the natural disasters; and duty and responsibility to rebuild life to its normal occurrence. Meeting the food and clothing demand, permanent and temporary sheltering, resolving the understructure problems and resolving all other problems after the disaster are within the responsibilities of the public administration. Because this approach focuses more on work and operations after the disaster rather than prevention of disasters, it is being highly criticized by scientific environments.

CORPORATION ATTRIBUTES OF DISASTER MANAGEMENT

The public administration structure of Turkey was established upon two main principles: Centralized Administrations and Local Administrations (Keles, 2004). The responsibilities of centralized management for natural disasters are to generate and develop national politics regarding the subject, substantiate the organizations that will implement these, administering legal arrangements. Providing the collaboration and coordination locally and nationally should also be realized by centralized administration.

Besides, the governors and district governors in the provinces; play an important role in the execution of works regarding disasters and development of emergency rescue and first aid works in cities and towns with the cooperation of public institutions, private institutions and voluntary establishments. Because of this, many public institutions intervene to many disasters with various regulations. In which case, within the disaster management system this leads to the presence of many appointed worker of the establishment and authorized personnel. In order to prevent this confusion, the works to gather the centralized system under one roof is still continuing.

The non effective stance of the local administrations in the years 2004-2005 when faced with a disaster have been changed within local administrative laws and the authority and responsibilities of the local administrations have been increased.

THE ECONOMICAL, SOCIAL AND ENVIRONMENTAL EFFECTS OF THE MARMARA EARTHQUAKE

The Marmara Earthquake affected the areas where population, industry and the metropolitan areas were located most intensely. This earthquake, since the 1906 San Francisco and 1923 Tokyo earthquakes until today, is considered as the biggest earthquake that took place in an industrial zone (Erdik, Durukal, 2002).

25% of the population, that is one out of every four person, lived in one of the seven cities that the earthquake affected. When looked from the point of urban population, these ratios increase to 31%.

In the Marmara Earthquake that took place in August 17, 1999 with 7.4 and November 12, 1999 with 7.2 magnitude, 18.243 people lost their lives, 48.901 people were injured and 376.379 houses and workplaces were damaged in various degrees.

It has been established that 112.724 of the houses and workplaces were demolished or severely damaged, 124.131 were moderately damaged and 139.524 were damaged minimally. Number of demolished or severely damaged houses was posted as 93.808 and workplace 15.944; moderately damaged houses number posted was 175.943 and workplace was 15.754. (Table 1.).

The social place of the region within Turkey and the national economy; caused the effects of the earthquake to be felt all over the nation very intensely. Among the direct economical damages caused by the earthquake; the halt of production for a period of time, stock of the capital, production capacity due to loss of workforce and therefore decrease of the established added value in the region, the reestablishment of the cities that were destroyed and the finance necessary to reestablish normal life flow are the first examples that can be counted.

The negative effect on the capital stock and national income is guessed to be around 9-13 billion Dollars according to the works that were made one month after the earthquake. The loss of the sectors that kept their presence in the region is between 3.076.000-6.476.000 American Dollars. According to this data the outmost loss was within the houses sector with 1.100.000-1.600.000 American Dollars.

Regarding to the sectors; damages and demolition within the energy, roads, railways, maritime line, airlines, pipe lines, communication, service sector (tourism, public buildings, security) industry, private sector and the agricultural sector affected the Turkish economy at a very important level.

Table 1. Distribution of the Total Damaged Houses and Workplaces in the Marmara Earthquake (Records of the General Directorate of Natural Disasters, 2006).

	Demolished – Severely Damaged		Moderately Damaged		Little Damage		
PROVINCE	Houses	Workplace	Houses	Workplace	Houses	Workplace	Total
Bolu	2334	219	6099	902	5767	1016	16337
Bursa	141	5	571	25	1371	68	2181
Düzce	16666	3873	10968	2573	13070	1605	48755
Eskişehir	90	21	167	18	398	32	726
İstanbul	3073	532	15102	2510	17870	2280	41367
Kocaeli	35845	5478	41091	5861	45606	6221	140102
Sakarya	24678	5478	18406	3764	27230	2699	82255
Yalova	13895	5146	14540	1159	12685	1885	49310
Zonguldak	108	60	311	3	952	9	1443
Karabük	0		76	0	106	2	184
Total:	**96.830**	**20.812**	**107.331**	**16.815**	**125.055**	**15.817**	**382.660**

Most of the public buildings and educational establishments in the region were demolished or became unusable after the earthquake. The disrupted highways also prevented help works.

The magnitude of the destruction disrupted the psychological and social structure of the habitants in the region, the social life was disrupted due to destructed and damaged houses, infrastructure etc.

Effects of the damages caused to established chemical factories and industrial establishments' leakages causing sea pollution, pollution of the debris and the filling of the sea with debris at the seaside of the Marmara Sea will continue for a long time.

URBAN DEVELOPMENTS AFTER THE MARMARA EARTHQUAKE

The Marmara Earthquake has directly put forward all of the wrongdoings regarding urbanization: unplanned and fast urbanization, wrong site selection, politics based on unearned income, amnesty of zoning, mistakes in the supervision systems, uncertainty in the professional and administrative responsibilities manifested themselves in this disaster. Many regulations were made in order to reduce the effects of the disaster and to reduce the negative effects in these regions during a possible disaster. Limitations were brought to the constructions of high rise buildings. Ground survey was made obligatory for areas that were planned for population and inspections were increased. Regulation changes have been made regarding constructions of earthquake resistant buildings. The strengthening of the damaged buildings in the regions affected by the earthquake is still continuing. The demolished or severely damaged houses were reconstructed in a collectively chosen area on suitable grounds. Moderately damaged buildings in the cities were optimized for a possible earthquake. Demolished or dangerous public buildings were demolished and reconstructed. Moderately damaged schools, hospitals and other public buildings were repaired.

After the earthquake storey restrictions in the old cities was enforced. However, these guidelines are being breached gradually. In this case new dangers are being invited.

After the Marmara Earthquake, as a state, a legal regulation has been made directed to mandatory earthquake insurance for sharing the general risk and not houses construction. Even though it was a legal obligation, certain citizens still have not insured their houses against earthquakes at a sufficient level.

Process of Determining Damages: After the Marmara Earthquake various establishments tried to determine the emerged damages (economical, social and spatial) of the earthquake in various ways. The determination works for the damages of the houses, workplaces and other public buildings are being conducted by technical teams of the Ministry of Public Works and Settlement. The damage determination reports, as a result of these works, will form a basis for the works of holder rights and aids that will be made.

Right to Ownership: In Turkey, for whose house suffers a damage caused by natural disaster, may obtain a long term credit from the government to repair their house and they also have the right to purchase an house that was constructed by the government. The first condition to acquire one of these house is that the dwelling has to be demolished or damaged in such a degree that it cannot be lived in. The owners of demolished or damaged house have to prove the situation. They may obtain credit help for only one house if they are the sole owners of the house. After an earthquake no help is being provided for people whose houses were demolished and lived on rent.

The credit that is provided for those whose houses were damaged to a point where it was unlivable has a grace period of two years and has to be paid back to the government treasury within twenty years in equal payments without interest. For those whose workplaces suffered damage may reconstruct their businesses with a credit they can procure from the government.

After the damages of the houses have been established, in order to profit from the credit loan provided by the government, within two months an application with a statement guaranteeing the pay back has to be omitted to the officials. Otherwise no help will be provided.

Developments Regarding Allocation of New Settlements; After the disaster three different approaches are in subject to the renewal of the cities. (**i**) Those who suffer a disaster may disperse into other settlements and thus the demolished settlements will be evacuated completely. (**ii**) The demolished place may be reconstructed at the same place. **(iii)** A new settlement may be constructed around the settlement in another territory. The third method was the approach adopted and applied after a disaster in Turkey. In a condition where houses are damaged or destroyed because of any type of natural disaster, the allocation of the new settlement areas for the establishment of new houses are determined by the technical team of the Ministry of Public Works and Settlement.

While allocating an appropriate place for new settlements, these places have to be appropriate for realizing settlement functionalities. If these houses are constructed in these chosen areas, as it can be predicted by the rightful owners necessary interest will be shown and other social criteria will form determinative factors. With these purposes, because of the houses that will be built due to the Marmara Earthquake a safe area of 1748 hectares has been chosen that will be sufficient in return of the 42 587 damaged houses.

In Turkey, for the reconstruction of the damaged buildings, instead of the old area where the debris is found, mass chosen newly appointed housing areas are being preferred. However, just as it happened after the Marmara Earthquake, because the new urban

areas are far from the old centers, because the social and economic life is not arranged properly, they are not places that are considered as very attractive. After 9 years has passed, some of the newly build city areas are moving towards city sites. (For example: City of Sakarya) In some cities the home owners prefer these places and build their own houses to where it used to be by themselves or repair them.

PERMANENT HOUSING APPLICATIONS AND THE FORMATION OF NEW URBAN AREAS

After the earthquake the main purpose of the actions is to bring all human activities, psycho-social life to a better lever than before the earthquake (Park, 1991). Within these activities largely scoped works such as, the reconstruction of all the damaged structures, rebuild the economically, socially and psychologically ruined society unity etc. takes place.

Even though the intensity of the earthquake varies the reconstruction of the physically ruined environment and the rebuilding of the new houses can be approached in two different ways; in the first method, the ruined social and economical balance is established supposedly by considering that no change has to be made to the socio economic structure. This restoration is made by reconstructing the physical surroundings. In the second method, especially in the developing countries, by considering the fact that disaster causes social destruction, the socio economic structure is diverted to a higher level of than before the earthquake.

A physical environment that necessitates such a social structure develops with it (Pehlivanlı, Ataman, 1997). The fact that physical plan realized during reconstruction for prevention of disasters or decreasing its effects can be used as an effective tool, makes it possible for disaster factor to start from national scale and integrate into every work plan at each level (Gundogdu, Ozer, Baki, 1999).

The reconstruction period is a period that is considered as possibility to seize for damage reduction precaution. During this period, dangerous zones are closed down for settlement and new arrangements are made for land use and decrement preventions such as construction measurements are included in the reconstruction period as a safer city formation is obtained. The education that continues during the reconstruction period, preparedness, will no doubt lay the foundation for a more ready social life with the formation works against the possible upcoming disasters.

For the more efficient use of the new city areas, the activation of social and economical life with new working areas has to be provided. During the first stages, the purpose of planning shelters, allocation of new settlement areas and physical planning is to provide necessary physical and social conditions for people, families, community safety and easy lifestyle. During this period the purpose is to help people as much as possible so that they can deal with their own lives without the need for help from others (Sphere Project Published, 2000).

Starting from the first minute of the earthquake, provisional settlement has to be solved and after permanent solutions have to be thought of. As the panic of the earthquake fades the necessity for sheltering will shadow forth.

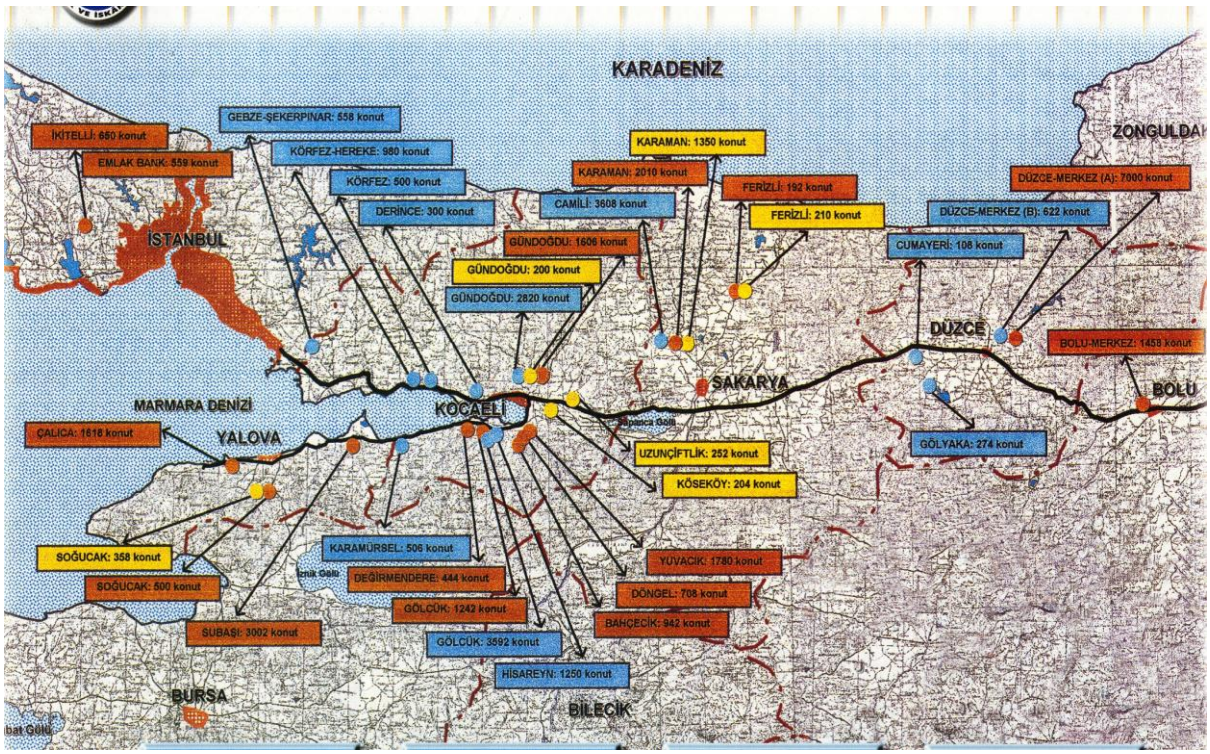

Fig. 1. New Urban Areas in Marmara Region (Ministry Of Public Work and Settlement's Magazine, 2006).

The sheltering problem caused by the earthquake in the turmoil that is formed is far from being a problem that can be handled by people. Sheltering solution has been primarily arranged within the responsibilities of the Ministry of Public Works and Settlement.

After the first few extreme days of the disaster the solution for sheltering process starts as short and long term solutions. As is known, the works focused on the sheltering problem that takes place in the highest frame of the necessity hierarchy is the most problematic period. However, seeing the house as a four walled place is disregarding its social function. After the earthquake the main purpose is to return to normal life standards in the shortest period of time.

During the process of allocating new permanent settlement areas three concurrent work has been realized. These are; (i) Allocating appropriate places to reconstruct new houses for the beneficiaries, (ii) realize the soil and geological surveys for environmental planning, (iii) Approval of the studies forming the Development Plan and when necessary provide technical help to the municipalities.

After the Marmara Earthquake tent cities were formed, 113.924 tents were built in 121 tent cities and families were placed here. As the seasons became colder and winter arrived, in a very short period of time (approximately within 3 months) the prefabricated houses were completed and the disaster victims were removed from the tents and placed here.

Table 2. Permanent Houses Constructed After the Marmara Earthquake (Resource: Records of the General Directorate of Natural Disasters, 2007).

PROVINCE	TOTAL CONSTRUCTED HOUSING NUMBER
KOCAELİ	17776
SAKARYA	8228
DÜZCE	8470
BOLU	1734
İSTANBUL	810
YALOVA	5476
TOTAL OF HOUSES CONSTRUCTED BY TENDER	42494
Purchased (Çerkezköy)	559
GENERAL TOTAL	**43053**

It is difficult to suddenly reconstruct thousands of places that have been torn down; it is a time consuming and expensive period. The construction of the disaster victims' houses begun at the beginning of the year 2000. In a short period of time such as two years, the construction of thousands of houses was completed. By 2002 the disaster victims started to settle in the houses.

The necessary financial resources in the construction of these houses have been compensated by various taxes. However, a demand for external credit was made because of insufficient resources. Within the demolished houses 2.574 of them were granted, 12.068 houses were made with the credit from World Bank, 15.502 houses with the credit of European Council Bank of Reconstruction, 3.050 houses with the credit of the European Investment Bank, 7.650 houses were constructed with internal resources (with the allowances in the budget) and the construction of 42 053 houses has been completed. The newly constructed buildings consist of 4 and 5 floors and have been constructed accordingly to the city planning with workplaces, park areas, parking spaces, schools, health centers ect.

An important portion of these permanent residences that were constructed with infrastructure and other public services were financed by external debts and equity capitals and their unit price sums up to approximately 42.000 American Dollars. The reimbursement price for twenty years without interest is approximately 12.000 US Dollars (Peynircioglu, 2004).

Table 3. Financial Resources of the Permanent Housing (Records of the General Directorate of Natural Disasters, Crisis Center).

Province	Grant	World Bank	European Investment Bank	Ministry Of Public Works And Settlement	Of European Council Bank Of Reconstruction	Total
Kocaeli	656	8480	1120		7520	17776
Sakarya	1488	2572	1000		3168	8228
Düzce		1004	466	7000		8470
Bolu					1734	1734
İstanbul				650	160	1928
Yaova	358				5118	5476
TOTAL	**3061**	**12056**	**2586**	**7650**	**17700**	**43053**

210 000 people profited from these houses that were put out to tender.

One of the ways that the government helped was that the disaster victims could received a credit equaling to half the amount of the building they had on the land they owned was worth. Approximately 30 000 people profited from this method 8000 houses were built.

Another help method was by providing a credit support for purchasing a completed houses from anywhere in the country. This amount is being determined each year by the Ministry. This method was initially implemented for the Marmara Earthquake. 6 000 people profited from this method and approximately the sheltering problem of 20 000 people was resolved.

Table 4. Housing Condition in the Marmara Region (Records of the General Directorate of Natural Disasters (Summary)).

Province	**Demolished Houses**	**Constructed Houses**	**Housing Defficiency**
Sakarya	24.588	8.264	16.324
Kocaeli	34.275	17.860	16.415
Yalova	14.113	5.478	8.635
Düzce	24.588	8.756	15.832
Bolu	2.532	1.734	798
Total	**91.853**	**42.902**	**58 004**

Due to different attributes of each city that was affected by the earthquake, the solution for each city differs.

EVALUATION

Even if because of a disaster, the fact that people have to move from places they have lived for years results are of dramatic chains of decisions. The vastness of the new settlement areas have been formatted according to the number of beneficiaries who

requested for permanent residences. However, unfortunately during the decision making of the allocated placed and houses types the participation of the owners was not sufficient.

Some of the risks related to each other, that could derive from changing places are as follows; unemployment, homelessness, landlessness, lapsing to end points, insufficient food, health problems, decrease in assets and deformation of the society. In these situations, approaches such as decreasing risks and actualizing of a successful reconstruction period, following an approach based on land, creating new employment fields are possible. Solely solving the sheltering problem will not be sufficient. Economical and social life also has to be solved according to the levels of modern lifestyle. Otherwise, the creation of *ghost towns* and its modern and technical ways will be realized. In many areas of the Marmara Region this situation may be observed.

The fact that the urban areas created after the earthquake are far from the city centers is a subject that is being criticized. However, residing in solid foundations and earthquake resistant buildings satisfy the residents. The criticism focus on the fact that the social and economic life was not arranged accordingly with the necessary infrastructure (Nakabayashi, Ichiko, 2004).

In order to construct resistant cities when faced with a natural disaster and risks, necessary information and accumulation has to be used effectively. Furthermore, because the life span of the cities is not limited with the life span of the residents they have to be sustainable and pass to the future. Because every city that is affected by the earthquake has a different characteristic, the solutions to the problems differ for each city; the solutions have to be various.

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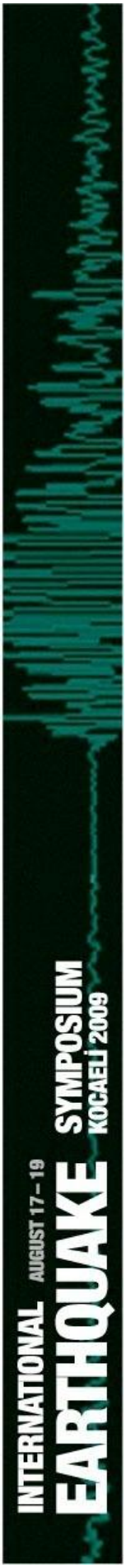

ARCHITECTURE

MİMARLIK

A REVIEW OF AUGUST 17 EARTHQUAKE'S EFFECTS ON HOUSING PRODUCTION AND DESIGN

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Key Words: Housing Construction, Housing Supply and Demand, Architectural Design, Urban Space.

ABSTRACT Two major earthquakes took place in Kocaeli and Duzce in 1999 causing a great loss life and possessions and these earthquakes effected not only the region socially, economically and culturally but the whole country. Besides the people that experienced the earthquake physically, the people effected psychologically by the media spread from the region started to approach the subject more aware compared to the times before the earthquake and watch the changes in building scale and city scale with more attention. The housing investments which have ceased, got started again in 2003 and gained acceleration compared to the era before the earthquakes. The supply and demand in urban housing differentiated not only in Istanbul and Kocaeli, but in the whole country.

In this paper differentiations in production, supply and demand of housing industry and changes in architectural design are reviewed with Istanbul, as the most important economical focus point, and Kocaeli as the city most effected from the earthquake, regarded as center. The houses built after the earthquake are assessed from the angle of design both in single building scale and urban scale and proposals are suggested

INTRODUCTION

There are periodic earthquakes in Eastern Marmara for centuries. Last ones took place on August 17 and November 12, 1999 effecting a large area, especially Yalova, Kocaeli, Duzce and partly Istanbul and caused heavy loss of lives and properties. According to the damage inspection studies after the earthquake, of the existing buildings 50% received heavy damage or collapsed, 20% needs repair and reinforcement, 10% has light damage. It is reported that only 20% of all buildings received no damage (Onel, 2002).

After the earthquake, the area entered a restructure period to meet the emerging need for houses and the permanent earthquake houses were given to their residents (Komurlu, Oztekin, 2007).

During this period of time people that had experienced the earthquake has got rid of the primary effects like insecurity, fear, panic and uncertainty, and started getting informed, thus tracked the publishes about the earthquake via media and asked for information about, with general phrase, the earthquake resistance of buildings, strength of buildings and prefered homes like this. A research that has been run in Kocaeli in 2005 shows that residents of Kocaeli still have conversations about the earthquake (10,3% of the attenders implied that they talk about it frequently and 74% occasionally), get information about the earthquake from television (73% of people), think these programs are both informing (67% of people), confusing (41,3%) and frightening (27,7%) (young people think informing, middle-aged confusing, old people and women frightening). According to the research 31% of Kocaeli residents implied that the thing that makes them relax is their trust to the safety of their homes, while 27,5% implied faith. 24,4% of Kocaeli residents implied that nothing made them relax about the earthquake and 8,9% implied that they aren't frightened of earthquake at all. These point out that

residents haven't reached the sufficient level of awareness (Es, Akın, 2005) and are misinformed mostly by stability speeches instead of living in safe buildings and environment.

It turns out that not only the people who has experienced the earthquake want and prefer living in earthquake resistant buildings, but also people who have anxiety and fear because of what they have watched or heard.

Thus after the earthquake, in the housing market freshened by globalization effecting our country and policies oriented for it companies producing houses began expressing that they are producing, both in project and production processes, buildings resistant to a drastic earthquake and acquired reports from universities for this. Using the reports in advertisements, these companies directed people to prefer them.

In this paper, today's -after the earthquake- housing's condition will be studied regarding Istanbul and Kocaeli cases which have suffered 1999 Marmara earthquake and expecting a new one with its reflections on architecture.

HOUSING AND HOUSE INDUSTRY IN TURKEY

It is obvious that housing deficit has risen because of our country's efforts oriented to being industrialized from 1950s to 2000 and policies causing urbanization and migration from villages to towns, and this, with the natural population growth, caused the production to be insufficient in urban areas, especially in industrialized areas. Considering the subject via numbers, the government has published legal regulations to provoke housing in order to meet the deficiency, but on the other hand left the concept of house, which is a general instrument of investment because of the lack of a general social security system, to market conditions and let annuity grow. The migration population that the government has ignored, had to meet their housing needs by themselves and so the cities suffered from illegal slums. The government legalized these solutions via dispensations laws and quality of houses and environment has been ignored (Yirmibesoglu, 2005), (Dulgeroglu, 1996).

Although, with the mass housing laws, there has been a major housing supply since 1980s, population per house units has dropped raising the need for new houses because of changes in social life i.e. large family model turn into nuclear family, people marry at old ages, more people keep away from getting married, more people get divorced, acceleration social activity (Bolat, 2007). Thus the supply doesn't meet the need in some regions and demand remains. Various amounts are mentioned for this demand (e.g. 400-600 thousands per year), even a total number of over 2 millions.

In an environment like this, as a result of frequent earthquakes there is heavy life loss, significant damage and collapses. Since the identity of physical environment is defined mostly by the houses and their relationship, i.e. house settlements, damages are concentrated on houses. 87,5% of the damaged buildings in Marmara earthquake are houses (Tas et. al., 2007).

1999 has been a milestone for housing. An important change in housing got started after Kocaeli centered earthquake (Bolat, 2007). This movement accelerated by government's legal regulations and especially the demand for living in earthquake resistant houses, moves fast this time with the guidance of policies aiming adaptation to globalization. With the legalization of mortgage, a structural change has started especially in housing finance. It can be expressed that this would cause major changes and institutionalization in real estate industry. Thus an increase in house loans is reported. According to the latest reports house loans have reached 30 billions TL and have increased 33% from January to December 2007 (Bolat, 2007).

So housing or with the much preferred naming real estate industry has performed a significant performance, much more than Turkey's economy in general. For example, in

2005 the gross national product has a growth rate of 7,6% while the construction industry's growth rate has reached 21,5% (Bolat,2007).

Public investments have progressed a lot about real estate and have become an important opponent to private enterprise and with the reconstructed TOKI have proven to be an important actor directing and regulating the industry (Bolat, 2007).

For expressing the state housing has reached in our country, Istanbul and Kocaeli has a specialty since these cities have suffered 1999 earthquake and have experienced housing in large scale since the earthquake.

HOUSING IN ISTANBUL AFTER THE 1999 EARTHQUAKE

As the largest city of our country and one of the numerous metropols in the world, Istanbul, with its potential about industry, education, culture and work opprtunities, have become an attraction point after 1950s under the effects of policies and experienced large domestic migration which caused fast urbanization. During this period the city has grown fast while changing socio-economically and physically, a huge and diversified demand for houses has appeared. While a portion of this demand has been met with illegal housing, some of it has been left to market provisions, the government has got involved to the process sometimes by financing and sometimes by producing. So illegal houses, apartments built by small investors, apartments built by housing cooperatives were added to low storey personally produced buildings coherent with protected historical pattern of the city built until 1950s. After 1980s mass houses built by mass house bureau and local administrations supported by this bureau, private enterprise investments and house bank's initiatives have been added to all.

But the house demand in the city couldn't be met, because of uncertainty and stagnation in house markets caused by the 1999 earthquake and the following economical depressions in 2000 and 2001 housing, in terms of quality and quantity, kept being a problem.

August 17, 1999 earthquake has been a turning point for the public for judging facts about socio-cultural, economic, physical and built environment. The new policies and legal regulations focusing house industry performed after the earthquake in Istanbul, that has been experiencing fast population growth and domestic migrations joined by social depressions, let residents' preferences appear in urban spaces in a short while, both in house production-supply and evaluation of house potential, geological studies oriented to structural stability, earthquake safety and building audition is emphasized and serious audition systems has got being applied. In brief, with the earthquake development dynamics of the city has changed (Yirmibesoglu, 2005a).

With getting involved in globalization after 2000, structure, population and economical progress of the city accelerated and exposure to globalization because of strategic location, the intention and effort to make it a finance and service center, with the necessity of changes introduced by the earthquake, caused a multi dimensional revolution, change and formation in house industry beginning in 2003 (Yirmibesoglu, 2005a).

The financial depression that appeared in the U.S. in the second half of 2006 effected housing market, banks and financial institutes, started to effect first Europe and then the rest of the world in 2007. Before this depression, the house industry in our country had its peak period from 2004 to 2007.

Between 2004 and 2008, with the recovery in the industry, various housing companies produced and sold a large number of houses. In this period TOKI (Housing Development Administration) produced a large number of houses for families with low and middle-low income and directed the construction companies produce for families with high and middle-high income via revenue sharing model.

With the new clauses in municipal law (09.07.2004) the concept of "urban transformation", which aims urban renewal, has been introduced to the public. With the "land and house production" and "urban transformation and development area" clauses of the law, authority of "providing uniform urbanization, constructing and selling houses and mass houses, obtaining land for these, confiscation, performing precautions for earthquake, preparing urban transformation and development projects" has been passed to municipalities.

During this, slums, which cover 60-70% of the city and historical parts of the city has been subject to reconstruction. Besides the single building scale reconstructions in slums, mass house production has been performed by big construction companies and municipal institutions, supported by TOKI of which duty scope has been expanded. It is observed that some parts of the historical textures of the city have gone through a renewal process after being assigned by new functions like tourism, shopping etc. and some of them have been restored in regional or single building scale in order to be used as residentials. A major part of these approaches need confiscations.

The large quantity of the houses supplied besides the large quantity demanded, an fast increase in loans borrowed from banks by the buyers and a growth in the number of companies selling houses with bank loans have been observed. According to this, in Istanbul, which has been in a period of enlargement and development, housing have been concentrated around highways and routes crossing Bosphorus and illegal house regions, areas subject to municipal plans until the earthquake. But after the earthquake, the concentration of housing has shifted to new settlements created in and out of the town center and to the borders (Yirmibesoglu, 2005a).

The dense demand for stable buildings has directed the new houses and settlements to the northern parts of the city, of which substantial terrain has been emphasized, and thus the green areas and drinking water supplies have been subject to damage.

On the other hand, the city getting larger and economically powerful needs new areas to work on and this triggers the appearance of new sub-centersone after another. These sub-centers are important for the sustainability of the urban texture and new settlements constructed for the people with high income and partly for the people with middle-high income have been hinged to these sub-centers. Thus the borders of the city move to east, west and partly to north. New houses and shopping centers are added to the reconstruction of the city.

Although planned development has been neglected since today, this reconstruction is carried on according to the plans (Dokmeci, Terzi, 2008). Nevertheless metropolitan municipal plans, which have been prepared after the earthquake, have been intervened and changed during the preparation phases, town center has been subject to high rise constructions and last green areas in the town center have been explicit to construction and these cause new problems.

The level of living quality presented by the new settlements that have been formed during this period is important for Istanbul and our country regarding the earthquake zone it is on and as a factor of competition between major cities worldwide considering the globalization (Bolen et al, 2009).

EXAMPLES OF PROJECTS BY COMPANIES BUILDING HOUSES IN ISTANBUL (PRIVATE&PUBLIC)

In this section the public (TOKI) and private companies' projects in Istanbul will be studied.

There have been numerous house constructions instructed by TOKI in Halkali, Istanbul and this region seems to be a new sub-center like Ikıtelli and Buyukcekmece.

ISPARTAKULE Project is a major project on a large area reserved for a number of construction companies with an act between TOKI and Municipality of Avcilar. It is supposed to be a satellite town with 45.000 houses to accomodate 200.000 people with middle or high income. Both in Halkali and Ispartakule, there are many construction companies building mass houses.

- Bosphorus City (Halkali),
- Olympiakent Houses (Halkali),
- Innovia Houses Project (Buyukcekmece),
- Toskana Valley Villa Project (Buyukcekmece),
- Spradon Houses (Bahcesehir),
- Loca Istanbul Houses Project (Bahcesehir),
- Uphill Houses (Bahcesehir),
- Cinarevler (Beylikduzu),
- Avrupa Konutlari TEM Houses Project (Kucukcekmece),
- Avrupa Konutlari Atakent I and II Projects (Kucukcekmece),
- Safir Residence and Shopping Mall Project (Levent),
- Uphill Houses and Uphill Court Residence Project (Atasehir),
- My World House and Residence Project (Atasehir),
- Incity House and Residence Project (Atasehir),
- Meridian Residence Project (Atasehir),
- Idealistkent House Project (Umraniye),
- Yenisehir House Project (Umraniye),
- Minimal Kent (Kurtkoy),
- Beyazevler (Kurtkoy),
- Tepepark Villas (Kurtkoy),
- Alizepark Houses (Kurtkoy),
- Trend House Project (Kurtkoy),
- Konsept Istanbul House Project (Kartal),
- Adres House Project (Tuzla-Aydinli),
- Dragos Royal Towers (Kartal),
- Evreka Residences and Houses (Kartal),
- Uprise Elit Houses (Kartal),
- Uplife Houses (Kartal),
- Mahalle Istanbul (Cekmekoy),

- Aqua City (Cekmekoy),
- Mesa Houses (Cengelkoy)

Following are some photos of these projects:

Fig. 1. Uphill Houses and Uphill Court Residence Project (Atasehir) (Ruveyda Komurlu Photo Archive).

Fig. 2. My World House and Residence Project (Atasehir) (Ruveyda Komurlu Photo Archive).

HOUSE PRODUCTION IN KOCAELI AFTER 1999 EARTHQUAKE

Industrialization in Kocaeli has started in 1930s. With the planned development between 1960 and 1975, the city experienced the second industrialization and after 1980 had the third industrialization with the private enterprise investments directed to Kocaeli from Istanbul also from where supervision, management and production have been instructed. Fast industrialization and domestic migrations of the last 35 years have brought a high rate of population growth and environmental problems thus the housing problem has been emphsized. Although industrialization and urbanization are acknowledged as engines for development, among the instruments of development industrialization and urbanization has not got proper reputation. The center of Kocaeli is in a relatively small space because of geographical restrictions and thus experiences infrastructure problems. But the surrounding of the center has casually developed and mixed with industry (Bolat, 2007). The area is placed on North Anatolian Fault and high rate of domestic migrations caused fast and unplanned, uncontrolled urbanization. Thus the city received heavy structural damage from August 17, 1999 earthquake. With fast urbanization and population growth in Kocaeli, after the earthquake in 1999 the differentitation in dwelling demands and trends have increased the quality and quantity needs for houses (Bolat, 2007) and the general changes in policies about urbanization and real estate have

effected the city of Kocaeli. The search for new settlement areas in Kocaeli to meet these changes has not ended yet. The new settlements, whether planned or not, cannot offer a solution for the socio-economic changes, building safety, social services, economical value of the house, the change in architecture, the needs of the different groups in the society (e.g. students, teenagers, disabled or old people) is going on (Bolat, 2007).

The building count performed in 2000 reveals that the number of houses in Kocaeli with a construction licence is 175.049. This is 83% of the total number of dwellings and it is the highest percentage in Turkey. In 2000, the number of houses without a construction licence is 34.037.

Table 1. Kocaeli House Stock 2000 (Building Count 2000, TUIK ve GYODER Research Report 4, November 2007, p:120).

INDICATOR	KOCAELI
Number of Buildings	140.613
Number of Houses	352.079
Number of Houses (Town centers)	209.086
Number of Houses With a Construction Licence	175.049
Number of Houses With a Usage Licence	90.027
Number of Houses Without a Construction Licence	34.037
Number of Houses With a Construction Licence / Total Number of Houses (%)	83.0
Number of Houses With a Usage Licence / Total Number of Houses (%)	43.0

The total number of construction licences given in Kocaeli betwee 2000 and 2005 is 50.168. Between 2001 and 2006, 6.000 buildings have been reported to be out of service and assuming only 3.000 buildings have been built without a construction licence as a result of intense inspections, it is predicted that the house stock in Kocaeli would be 256.254 in town centers and the population would be 226.000. Regarding these two numbers, it would be expressed that there is no need for new houses in Kocaeli. The house stock change that took place after 2000 is because of after earthquake enterprises and permanent house productions. In 2006 house stock data, temporary dwellings are disregarded (Gurlesel, 2007).

Table 2. Kocaeli 2006 House Stock Prediction (Town centers) (GYODER Research Report 4, November 2007, p:120).

INDICATOR	KOCAELI
Number of Houses (Town centers)	209.086
House Construction Licence (2000-2005)	50.168
Number of Houses Being Used (2001-2006)	6.000
Number of House Constructions Without Construction Licence (2001-2006)	3.000
House Stock in General 2006	256.254

But regarding the qualitative and judicial state of the existing houses after the earthquake and the houses not being used it can be expressed that there is need for new houses in Kocaeli. Thus considering the physical conditions and construction date indicators of the existing houses, according to the data of 2000, 12.500 houses need restoration and the existence of the need for new houses is emphasized.

Table 3. Physical State of Kocaeli House Stock 2000 (Building Count 2000, TUIK ve GYODER Research Report 4, November 2007, p:121).

INDICATOR	KOCAELI
Number of Houses	352.079
Number of Houses (Town centers)	209.086
Houses That Don't Need Restoration	144.269
Houses That Need Basic Repair and Restoration	50.181
Houses That Need Serious Repair and Restoration	10.454
Damaged and Planned To Be Destroyed	2.091
Unknown	2.090

On the other hand, considering the construction dates the total number of buildings older than 50 years in 2000 is 5.623 (approximately 10.000 houses). This number will extend to 13.862 (approximately 20.000 houses) till 2015 and with the exception of the ones with historical value, this number will be added to the number of new houses needed (Gurlesel, 2007).

Table 4. Kocaeli Building Production Dates 2000 (Building Count 2000, TUIK ve GYODER Research Report 4, November 2007, p:121).

End of Construction	Number of Buildings Used Totally for Dwelling	Number of Buildings Used Mostly for Dwelling
-1929	658	25
1930-1939	402	69
1940-1949	917	133
1950-1959	3.207	212
1960-1969	7.646	593
1970-1979	18.339	2.261
1980-1989	38.846	5.560
1990-2000	40.464	5.952
Bilinmeyen	867	222
TOPLAM	**111.346**	**15.027**

According to the data above, it may be expressed that the majority of the house stock in Kocaeli is in lack of quality, old, missing social services, economically at the end of lifespan, unhealthy, in lack of building safety and damaged because of earthquake, there

are unplanned settlements, it may be implied that quality and quantity of the houses cannot be stated properly because of missing certain qualities like permanent properties, economically effective lifespan, functionality, size, market price, rent wage, socio-economic state of dwellers, building ownership, number of people dwelling in the house, structural properties, access to socio-economic cultural services, heating system etc.

But approving that living in healthy, quality houses is a basic human right and Kocaeli is on a earthquake fault, the city needs renewal and transformation immediately. There are a lot of regions in Kocaeli that needs to go through urban transformation and renewal. Although new, spare areas to be constructed on should not be expressed as urban transformation instead of housing reserve area, constructing houses is not urban transformation but reconstruction or renewal, transformation aims renewal with assigning new functions, considering some clauses in the municipal law Kocaeli Metropolitan Municipality declared ten regions in Kocaeli as Urban Transformation Area and gave a start to the production of 20.000 houses via municipal company Kent Konut, some these have been finished.

The house stock that has been exposed to the earthquake but seem undamaged is ignored during house productions that are named transformation. Examining these housings, it may be reported that these are standard mass houses built all around Turkey, like permanent house constructions.

Although TOKI is planning to construct 10.000 houses, this amount is not sufficient and the prices are close to private enterprises'. Also there are complaints about quality by users.

Thus it may be implied that there is no plan in Kocaeli to set borders for urban transformation and general approach and priorities for development of the city. It is observed that partial solutions are applied during the process. This indicates that supply and demand relationship is more important than planning. But disaster risks, more from being a planning factor in city scale, should have the role of leading development (Akcan, 2007).

Below is the state of house industry and house market in Kocaeli.

1. Kocaeli construction and housing industry has been effected by the 1999 earthquake negatively. In the beginning the need for new houses appeared but then uncertainties like the new building code to be published, terrain conditions, number of floors etc. have ceased activity in the industry for a long period of time.

2. Permanent house constructions after the earthquake, evacuation of the risky zones and introducing new housing development zones have been the activities of the industry. In 2005-2006 the private enterprise has started house constructions.

3. After the earthquake Kocaeli Metropolitan Municipality got micro zoning terrain studies, terrain study reports have been prepared according to these. But since these reports were prepared on zone basis, some of the construction companies got terrain study reports for every single building they were planning to build. Moreover big construction companies mentioned these reports in their construction specifications and contracts.

4. After the earthquake new housing investments headed to northern, eastern and southeastern parts of the city where the terrain is stable. 80% of the new house constructions took place in these regions.

5. Because of the terrain properties the highest building permitted in Kocaeli is 4 storeys. So the new house productions are 2-3 storey single house groups.

6. There is no new area for house constructions in Kocaeli downtown. Because of this, Cumhuriyet Park and Rasathane Street are the most wanted and thus most expensive regions.

7. The need and demand for new houses in Kocaeli is predicted not to end. Especially in relation to the growth of household there is need for high quality houses and housing areas. It is predicted that Dongel, Bahcecik and Karsiyaka regions will be the new housing settlement areas in the near future.

8. There is a great need for social houses in Kocaeli. TOKI aims constructing 10.906 houses in central Izmit, Golcuk and Korfez. The period of urban transformation has started in Kocaeli and Metropolitan Municipality and its enterprise Kent Konut Inc. has started constructing high rise mass houses for the middle class besides the 2-3 storey buildings. Local authorities plan developing house production aiming 20.000 houses.

9. Housing industry in Kocaeli has not finished its reconstruction after the earthquake. There is need for quality, safe houses but on the other hand there needs to be produced low, middle and high class quality houses at the same time.

10. The rate of using housing finance systems is high in Kocaeli. There has been 4557 loans in 2005 and 5571 loans in 2006 (Gurlesel, 2007). This effects the ability of buying quality, earthquake safe houses and thus the demand.

EXAMPLES OF PROJECTS BY COMPANIES BUILDING HOUSES IN KOCAELI (PRIVATE&PUBLIC)

In this section the public (TOKI) and private companies' projects in Istanbul will be studied.

Kent Konut is a company founded by Kocaeli Municipality for housing investments and has numerous projects going on. Some of these have been planned and executed under the name of urban transformation. Some of these have been planned as mass houses and partly villas for people with high income.

The last 3 projects that Kent Konut has finished are Akca Kent, Izmit Kent and Gazanfer Kent. There are numerous projects going on, as listed in the table below. The biggest of these projects is urban transformation project and has a construction area of 130.000 squaremeters. The planned projects, listed in the table below, are Gebze housing project, Izmit-Cayirkoy project, Izmit Kent Residences and Basiskele Villa projects. The large sized urban transformation projects carried on by Kent Konut are subject to discussions by public and private institutions (Komurlu, 2009).

Table 5. Kent Konut Projects Going on in 2009.

Number	Project	Start Date	Finish Date	Number of Houses	Construction Area (sq.m.)
1	Derince-Yenikent Houses	08.02.2008	01.08.2009	120	16.591
2	Karamürsel-Dereköy Gazanferkent	17.08.2007	07.02.2009	144	25.674
3	Urban Transformation-II Region 3 (İzmitkent)	20.11.2008	13.05.2009	555	76.321
4	Urban Transformation-I 2nd	03.01.2008/17.06.2008	07.02.2010	384	56.805

	Phase (Region 3 and 4)				
5	İzmitkent II Phase (Region 4)	4th	19.03.2008	10.09.2009	676	61.835
6	Urban Transformation-I Region 2 (Umutkent)		01.02.2009	26.07.2010	152	28.772

Table 6. Kent Konut Inc. Planned Projects for 2009 (Kent Konut Inc., 2009).

Number	Project Name	Tender Date	Number of Houses	Construction Area (sq.m.)
1	İzmitkent Phase III. Region 2.	08.01.2009	700	56.697
2	Turgut Mah. (Residential and Commercial)	14.11.2008	30 Residences and Stores	5.746
3	Başiskele-Yuvacık Social Houses Project	February 2009	600	53.438
4	Başiskele-Yuvacık Villa Project	May 2009	76	10.032
5	Başiskele-Yeniköy Social Houses Project	February 2009	77+424	39.554
6	İzmit Kent Residences	October 2009	831	5.600
7	3rd Urban Transformation Project	September 2009	760	77.200
8	Gebze Housing Project	September 2009	1000	105.722
9	İzmit-Çayırköy Project Phase I	April 2009	2556	282.608

Examples of Kent Konut Inc. Projects:

Fig. 3. İzmit-kent 3, planned - İzmit-kent 1, construction going on.

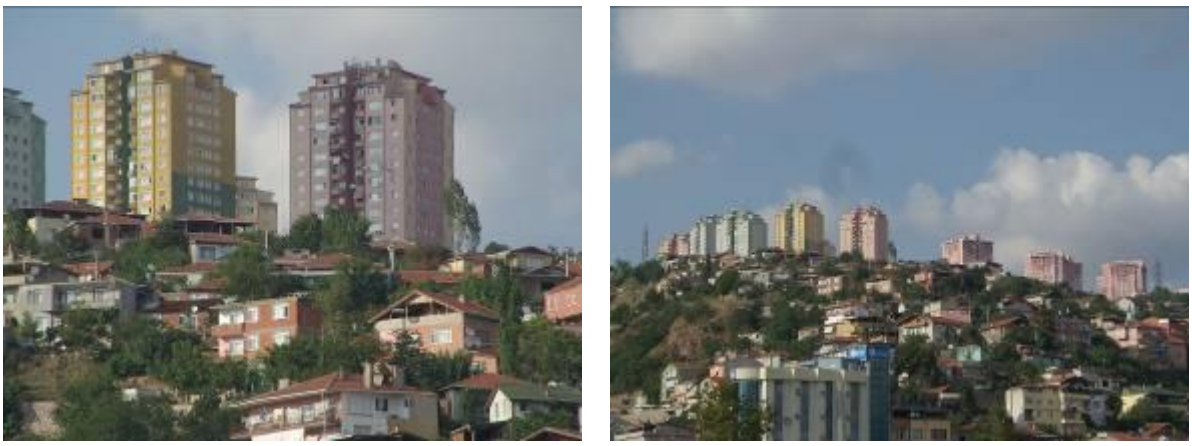

Fig. 4. İzmit-kent 1, (Ruveyda Komurlu Photo Archive).

Fig. 5. İzmit-kent 1, Photographs from the site (Ruveyda Komurlu Photo Archive).

Below are the examples of private enterprise constructions in Kocaeli.

- Armadakent Park Houses, consists of 4 phases. Construction of the first phase (in Yahya Kaptan) is over. Phases 2, 3 and 4 are going on.
- Armadakent Villas (in Yahya Kaptan)
- Armadakent Residences (in Yahya Kaptan)
- Ali Kahya Housing Project, planned as 1.000 houses.

- Saray Houses, Anemon Houses, one of the finished projects.
- Saray Houses Caglayan Houses, one of the finished projects.
- Saray Villas, one of the finished projects.
- Saray Houses Safir Project, construction going on.
- Bagcesme Lavinya Project, construction going on.

Below are photographs of some of these projects:

Fig. 6. Armadakent Houses Phase 4. Housing project, 3 storeys (Derya Bilgic Photo Archive).

Fig. 7. Armadakent Phase 4. Housing project, Residences (Derya Bilgic Photo Archive).

EFFECTS OF AFTER EARTHQUAKE CONSTRUCTION ON HOUSE DESIGN

The design properties of the houses constructed after the earthquake may be implied as follows.

- Some of the houses are villa type low storey buildings but some of them are high rise apartments.
- These houses are grouped together or organized in satellite city scale with the services like security, social services etc. included.
- Especially in Istanbul, construction of luxury high rise apartments and residences has started and the demand for these is intense.

- The majority of the residences are located above shopping malls and some of the houses have been designed including these shopping malls.
- There has been differentiation about area and number of rooms (2+1, 3+1, 4+1) in high rise apartments.
- 1+1 studio flats, that haven't been taken into consideration till now, have been built.
- Prestigious house design and star architects have got mentioned and house designs have been emphasized.
- On the other hand, houses constructed by TOKI and municipal companies have a standard and ordinary design.
- The studies and applications of restorations in old parts of the city which are subject to urban transformation cause arguments.
- It is reported that the dialogue between designers and civil engineers about the earthquake factor when building high rise buildings has developed.
- It is reported that traditional skeleton structure is still preferred in house constructions, but in high rise house constructions tunnel formwork is widely being used.
- Steel and wooden skeleton structures are constructed only in personal initiatives.
- Especially in private enterprise house constructions deluxe and expensive finishes are being preffered.

CONCLUSION

Remembering the earthquake reality with Eastern Marmara earthquakes, the housing investments ceased in our country until the acceleration in 2003. Below is the evaluation of housing investments in the two major cities which were hit by the earthquakes.

Housing has been left to market conditions. The government, instead of restoring the existing house stock with detailed studies, using the effect of earthquake has speeded up house production via mass house bureau and local administrations in the name of constructing earthquake safe houses and instead of preparing a detailed, integrated plan, encouraged production with local solutions and private enterprise followed this approach. While the cities growed with these productions, natural resources has been under threat and risks has appeared. There has appeared a perception that quality houses and environment is a privilege and the majority of the people has to live in unhelathy spaces.

Finally, it is obvious that deception of house production has not changed after the earthquake. Moreover, instead of overcoming the revealed problems, the house production perception and manners that cause new problems continued operation.

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Ruveyda Komurlu Photo Archive.

APPROACH OF ARHITECTURE HISTORY TO EARTHQUAKE: "OBJECTIVITY OF EARTHQUAKE PHENOMENON IN DESIGN AND CONSTITUTION OF COLLECTIVE MEMORY"

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ABSTRACT The question, how should "earthquake" phenomenon be included in reciting about the History of Architecture, was oriented to 40 students of 2nd Class, Architecture Department of the Faculty of Architecture and Design of Kocaeli University and requested to express their opinion in written with 1-2 pages. Although the approaches of the students were various, reciting of the Earthquake issue in the lessons of History of Architecture was requested by the majority of the students. At the same time, except for 1 student, remaining students agreed that earthquake theme should be included in the content of the History of Architecture lesson.

While the results of this survey and the view point of the students were underlining that earthquake issue had not been taken into consideration properly, this can force any Architecture Historian regarding that how this issue should be handled. It is very exciting that, earthquake phenomenon has not been considered commonly and even any information has not been given regarding the earthquakes of Turkey chronologically during the undergraduate education of architecture. While reciting about the modifications of the buildings that have important historical references such as Hagia Sophia and Grand Bazaar, adopting of an ordinary expression style for the executed modifications because of earthquake as if they had been performed as normal modification, consists another interesting dimension of the issue.

"Earthquake" phenomenon that could not establish a common point for "architecture" and "history", which are the main two components of Architecture History lesson should be discussed in the declaration, regarding that why could not it determine a history interval in the History of Turkish Architecture and why was not it included in the subjects of the Architecture History.

On the other hand, eras or structures have been included in the area of information because of establishing the subject of the Architecture History and being primarily products of the design, and determining of the historical time interval as being the first or single. In order to handle earthquake issue in History of Architecture as a time interval, both public and private structures should be the products of design after an earthquake and should have value rather than to "save the time" or "heal the wounds". In accordance with the concerns of the students about having inadequate information regarding structure (tectonic) issue at the practical information of architecture, it is attracted attention to emerge threat about the unidirectional perceptions about the demands that History of Architecture aims to cover the deficit in this area. When earthquake phenomenon was included in the issue, architecture (architectonic) has not been taken into consideration by the students as spatial, associative/lexical product of design, and this fact was assessed. A part from discussions regarding the structure should be "earthquake-resistant", how a structure should have take place in collective memory and to train the individuals, who ready for earthquake phenomenon and the success to place this issue in the memories, in other words importance of spatial, associative/lexical direction at the architectural dimension was struggled to be explained. Within this approach, it has been emphasized that explaining of the earthquake phenomenon to the next generations or against the struggle not to cause forget this fact,

positive role of the objective expression of architecture against the resistance of the public in order to forget this bad event, should be included in the information area of History of Architecture.

It has been known that, the role that will be played by the history of the architecture in order to constitute collective memory is more important than the reciting about the modifications of the structures in the past because of earthquake. While establishing collective memory about the earthquake phenomenon, contribution of the Architecture History is another important point. It has connection with the contributions of the designers upon the public or private housings after earthquake. In order to prepare the people against earthquake, a better way should be considered rather than threat the people by media without distressing and damaging joy of life of the people in our era and suggestions have been developing in the declaration for a public area, where "collective memory" can be established.

INTRODUCTION

A questionnaire was made for 40 second year students of University of Kocaeli Faculty of Architecture and Design, Department of Architecture, during the Architectural History class, which involved the question of how the phenomenon of "earthquake" should be covered, and the students were asked to express their views in written form on 1 to 2 pages. While the responses provided by the students were different, a similarity was observed in their approach to the matter. It was the opinion of the majority that Architectural History should be taught on the basis of the damages of the earthquake to the historical constructions and structures as well as the interventions to the structure in compliance with the structural forms of the era. With the exception of 1 student, it was an unanimous decision to include the earthquake phenomenon in the curriculum of Architectural History.

The need to investigate the reason of the fact that the earthquake phenomenon is not included in the Architectural History curriculum of Architecture students in the bachelor program, which is offered in this land, where many earthquakes have happened so far, becomes evident with the standpoints of the students, as evidenced by the results of the questionnaire?

The statement covered the reasons of the fact that the earthquake phenomenon is not included in the Architectural History curriculum, the lack of architectural understanding of the students who have requested the inclusion of architecture on the basis of its damage to the historical structures, and their difficulty in the comprehension of the difference between the structure (tectonic) and architecture (architectonic). As requested by the students, it was discussed whether the content of Architectural History classes consist of the studying of post-earthquake interventions to the old structures, and it was attempted to explain that the Architectural History classes can assume a more important role in the creation of the collective culture.

It is necessary to draw attention to the role of the "architectural design" factor in order for the Architectural History to bear this mission. If the Turkish earthquake history and post-earthquake architecture are not mentioned in Architectural History of the architecture programs, then we should look for the reason in the low level of contribution of the "architectural design". The role to be played by the design in the creation of the collective memory in public locations that will be constructed after the earthquake will be the main topic of the Architectural History in the future. It is also considered that the urban people who encounter the phenomenon of earthquake will create the collective memory in the public area to be created via the perception of the location. It will be suggested that the earthquake based public area projects that are handled with this perspective in mind to leave a message to the next generations and it will be revealed only then that it can define an interval of time, which will become a subject of the Architectural History.

REASONS OF THE FACT THAT EARTHQUAKES ARE NOT IDENTIFIED WITH A CERTAIN INTERVAL OF TIME WITHIN THE CONTEXT OF "ARCHITECTURAL HISTORY"

Collective memory is restricted by time. The width or narrowness of this time limit can be different for each community. It is not possible for the collective memory to directly appeal to the events and people that remain outside of these limits. The history comes to our help at that point, but old societies have all perished, and many events have lost our memories. The historian, whose area of interest is history, attempts to learn or teach us about this era with the help of the writings of contemporaries, the newspapers of the era, and the witnessing that has left their marks on official records for the events of the era. By doing that, the historian comes to conclusions depending on the perspective, which did not exist at the time (Halbwachs, 2009).

When the historian brings it up again, the deletion of the earthquake phenomenon from the collective memory would be prevented. The most important role in the vitalization of the collective memory, revival of the past events, and in making people remember all about the earthquakes with the memories of the past belongs to the historian. In the architectural education, on the other hand, it should be investigated why the earthquakes in Turkey are among the least discussed groups and are not involved in the information domain of the architectural history. Architectural history class, due to its subject and scope, aims to review the history and architecture of the periods, during which the historical changes were reflected on the architecture. These structures can be public or private, and be the product of a single person or multiple people. Just like general history, when there is nothing "new or different" in architectural history, then the history can not be written. Therefore the history is scaled by the new developments (Yürekli and Yürekli, 2004). Even if the damage of the earthquakes refers to destruction for this region or city, it has not cleared the path for the development of new methods or structural technologies in Turkey. In another words, the earthquakes have no definitive weights within the context of Turkish architectural history. For instance, even if the state intended to build the new residences with masonry in Istanbul after the earthquake of 1766, the population opted for the wood. Since the population considered the stones to be dangerous in a recurring earthquake, they went for a wooden structure instead (Afyoncu and Mete, 2000).

Even though the persistence with regard to masonry construction after the Tanzimat Reform has been frequently brought up throughout the nineteenth century after the fires in Istanbul, the implementation was slow due to the financial incapability of the public. Public's witnessing of the demolition of the masonry buildings during the earthquake of 1894 in despite of the durability against the fire has had a major influence on this standpoint. The construction system, which has been referred to as the "Frankish Masonry" and has costed 50% more has only been possible for the rich people (Ürekli, 1999). Therefore the popularization and implementation of the building technology that could be considered structurally more modern than the existing methods at the time could not be possible in despite of the destructive power of the earthquakes and other destructive disasters. The earthquake, as a forcing power, could not be deterrent with regard to the usage of different materials and technologies, and the construction of cheap and inadequate buildings have continued especially in public residences

The buildings that have been constructed after the earthquake are also not design products. They are all projects that aim to save the "moment" or the "day" with no connection to the past or future whatsoever and that have been constructed within the framework of common and rapid plans. Therefore it has been more important in these projects to come to a conclusion rapidly with contractors with good income and foremen that work for the state in such low-budget projects instead of focusing on the design. The approach "a roof over our heads" has prevented the taking of simple and fast measures and the lasting of building samples that were built after the earthquake.

While we have no data today of the residences that were rebuilt after the earthquake, which has affected the Marmara Region of Turkey at the end of the 19th century, the permanent residence types that emerged after the Marmara earthquake at the end of the 20th century will not define an interval of time within the context of the architectural history, since they do not differ from the residence types of the pre-earthquake period.

TENSE RELATIONSHIP OF THE EARTHQUAKE WITH THE ARCHITECTURE AND HISTORY

THE PHENOMENON OF "EARTHQUAKE" WITHIN THE CONTEXT OF THE ARCHITECTURAL HISTORY

Earthquakes have been chronologically sorted as soon as they have become a part of history. What has happened obviously becomes alive with the subjective narration of the historian within the context of the history, which is well beyond the chronological sorting. For instance activities that correspond to various points of the field of architecture are frequently referred just as the discussions that emerge due to house repairs between the tenants and the landlords, or damage assessments, repairs and wrongdoings of engineers. The records of the earthquake of 1766 occurred in the Marmara Region (Afyoncu and Mete, 2000) shares a similar pattern of activity with the earthquake of 1894, and their similarities continue with the effort of rescuing, the planning of the newly constructed buildings and the methods of entrepreneurship (Engin, 2000). At that point the similarity between the earthquakes of 1766 and 1894 with the earthquake of August 17th, 1999 is striking (Engin, 2000).

But the similarity and identity of the problems that are caused by the earthquakes of two different time points allows the mixture of the past and the time before that. It becomes quite difficult to determine the relatively older instance. It reveals the fragile nature of the bridge between the Turkish history of today and the history of the disaster time, and it constitutes the weakest point of the history component within the curriculum of "Architectural History". The phrase "Learning from history", which has been used extensively within the context of history discussions, proved ineffective after the earthquake of 1999. Another phrase, "history consists of reoccurrences" is also valid for a possible earthquake. Therefore the aspect of the history and today, which mix with each other and refer to the lack of valid processes, in a society, which sees history as something that only includes "learning" and "reoccurrence", is the reflection of societal acceptance. In order to "expect something" from the history, history should also provide strong references in the plans of today and tomorrow. But we can not create a reference point for the future if we feel a possible disaster as if it was the past. This is also an indication of our failure to realize that create the history with what we are doing today [1].

The relationship of the earthquake with the architecture is commonly considered by the majority (both by the public and by the insiders such as architects and engineers) to be with the structural interventions to the structure. It also reminds us of the interventions to the old structures that have undergone earthquakes. But the interventions to the building do not necessarily depend on the boundaries of the history. As long as the building stands, it will see interventions both today and in the future, and this will be taught to the students of all disciplines as the subject of today in the basic field of architecture. The duty of the Architectural History is not limited to the comparison of modern technologies with the former structural interventions. At that point we need to emphasize on the pioneer role of the Architectural History in the creation of the mass memory with regard to the earthquake. The negative effects of the earthquake, on the other hand, may lead to a rejection by the people who are forced to remember these unpleasant period, and the psychological effects of the earthquake obstructs the duty of the history to create a mass memory. What will eliminate the obstacle and simplify this obligation of the history at that phase is the objective nature of the architecture.

PROBLEMS IN THE ASSOCIATION OF STRUCTURE AND ARCHITECTURE WITH THE PHENOMENON OF EARTHQUAKE

The psychological effects of the earthquake on the minds within a historical context results in the distancing of the mind from what are considered to be unpleasant memories. Therefore a dilemma exists for remembering and forgetting about the earthquake with the stimulation of the feeling of "prevention" and "protection" with the consideration that this may happen again with the existing bad memories of the earthquake. They both secure the taking of the measures for the structures, and lead to the structural design by the architect in order to construct much more secure residences. Actually it is a part of the curriculum for a an architect candidate to be obligated to construct buildings that are resistant against natural disasters. In another words, it is the basic obligation of the architect to build a "solid" structure. But many people, which sadly include architectures, consider the relationship of the earthquake with the architecture as the construction of a solid structure. This common misperception of the society with regard to the relationship of the earthquake with the architecture is in fact related to the structure of the earthquake itself, and sometimes structures lose the battle against the earthquake regardless of how solid it has been. Well, how should the connection of the earthquake with architectural design be established? Before answering that question, we need to emphasize on the fact that structure and architecture should be reviewed separately, and elaborate on the subject.

While Construction is limited with the constructional and functional features of the subject (tectonic); architecture includes the spatial-aesthetical-semantic/associative views (architectonic) (Yücel, 2004). Constructions are performed alterations in the periods where they are exposed to the violence of earthquake and these interventions performed are necessary as a time passes over the construction or as it gets older. For that reason, old or new constructions are the objects which face structural alterations in order to be used also in the future. For that reason, all information areas and practices to be developed for the construction and structure are the interventions which have no connection with the past, which follow up the present belonging to the present period. However, architecture is the one which provides us to understand the historical specifications of the construction which contains the associative approaches of the construction as well as the aesthetical, cultural tendencies of the construction for the period when it has been built (Yücel, 2004).

Including the architects, there has been the discussion of earthquake over the tectonics in the society, for a long time. This has, no doubt, any objection, however, how much will it provide a decrease in the number of the buildings which collapses in the case of an earthquake? Despite this situation seems connected with architecture practices, the weakness of the collective memory and the defects in the theoretical side of this staff are in a dangerous level. Basing on architectonic, the public space projects where the earthquake concept has been included in the design has been neglected. This kind of public space projects may have a leading role in creating the collective memory. The public space projects which can be incorporated in the daily life and in which the physiological aspects of the earthquake has also been taken into account, will, with their discrepancies, form up another aspect of acquiring awareness towards earthquake.

THE ROLE OF "DESIGN" IN EARTHQUAKE'S HAVING A PLACE IN COLLECTIVE MEMORY

The Architecture History's being able to form up a "collective memory" is subject only by being able to realize the designs made for the earthquake. Creating of the collective memory is more difficult for the 21st century people, comparing to those of past. Abundance of speed and simulacrum causes the simulacrums to spread off rather than to gather them together, and as they spread off it causes the simulacrums to get lost in other words cause the memory to forget, rather than recall. Reaching the simulacrum

and meta plentifulness has required a period of 150 years. At the end of this process, forgetting offers us the first legal ground, over "too much to recall". All incidents experienced, important or not important, are equalized and "social acceptance" or "social memory loss" appears. In a world there numerous incident and numerous simulacrum are available, to remind the earthquake all the time through the destructiveness of the earthquake or to reflect this conceptual approach to the physical environment doubtlessly will make no use in providing the permanency of the earthquake concept in the mind, for the man of present time who lives in the middle of variety of simulacrums (Yazıcıoğlu, 2008). Instead of this, it looks better to make use of both the positive and the problematical sides of oblivion. Earthquake concept create a necrophobia, forgetting the death, thus forgetting the earthquake links the persons to the future, in that respect forgetting has a primary level importance in forming up future. On the other side, forgetting an earthquake which would occur now or in a few days means forgetting ourselves or our presence [2]. For that reason, the balance of remembering and forgetting is important in the transmission of the earthquake concept both to current generation and also to the next generations.

"MEMORY IS THE PLACE OF TIME", DESIGNING "MEMORY PLACES" FOR EARTHQUAKES

Doubtlessly, the spaces where the earthquake concept is included in the design will not have a "monument" or "museum" character and they will not be called with these names either. Nowadays, the monuments have lost their power to keep permanent memories; instead of this projects called "iconic construction" have been being produced rather than monuments. The spaces which are easily approved, which would form up an alternative against spaces such as museums, monuments and which would provide interactive interaction may cause the earthquake concepts to be left in memories. If it is assumed that the time is characterized not as a flow but as the memories of experienced paces and spaces, it must also be accepted that the spatial simulacrum has an important impact on the history (Harvey, 2006). Especially monuments will keep their subsistence as monuments, as long as they recall an event, person or an opinion and as long as the society finds this recall meaningful and important (Berger, 2007). The visitors of the buildings which has been constructed under the name "Earthquake museums" or "earthquake monument" has after now been as dead as a dodo and it is seen that the people who try to experience the space in an environment which has today's simulacrum and objects intensity, visit the buildings in groups and that common identities establish the spatialness of the social life (Akın, 2009). Environment – focused works, (by focusing on earthquake) fill again the lost cultural areas symbolically and offers historical counter memories, at the same time it can turn into touristic spaces and serve as an urban laboratory which gets in connection with the auditors in creation of innovator public art projects (Foster, 2009).

We know that in Turkey, many spaces are being used out of their function, whereas they have been established as earthquake museums and that the remaining ones, which are not used out of purpose, have a very little visitor flow [3]. Any project which desires to change the society in basic subjects such as difference, awareness and care [4], has to comprehend the complex network of the transformation of spatial and temporal understanding and practices (Harvey, 2006).

Converting into physical space the fact that the memory includes the records of both today and of the past and for that reason that it reserves all records in regard with the time, may provide anyone to design the earthquake concept both to be forgotten and to be remembered. Since any space to be designed for that purpose, beyond experiencing the now /today, determined a period in architectural aspect, will find a place among the subjects to be taught in Architectural History for the coming generations.

These spaces which have been fixated in order to create a social memory after the earthquake must be designs which are used by public and which assists today's withdrawn individual to create a common memory (Yazıcıoğlu, 2008). It is possible for the designers who could not be appointed in the construction of temporary residences after the earthquake, to provide the formation of different simulacrums in the mind, maybe by creating "common memory spaces" [5] and also by means of the spaces and objects they have designed, against the rapid changes of our age and plentifulness of simulacrum. While these "memory spaces" which will enable the remembrance of earthquake is an icon for today; they will form up a history of Architecture History for the next generations, by the meticulousness it has offered for remembering the earthquake concept for the first time and by the relationship it has established with the earthquake, by the innovations it has brought for the favor of the conceptual aspect of the space. It will have an important affect on alleviating the Architecture aspect's role of being the memory bank of the history, and even a concept's –which leaves bad memories such as earthquake- bravely recall by the present's and future's generations even if it has been forgotten.

CONCLUSION

The classes in architecture and the expectations from these classes with regard to the phenomenon of the architecture shows a single trend. In fact, structures of some classes and their social aspects, which are well beyond the architecture reveal a differentiation in the yield from the architectural products. It is, however, necessary to create this difference in the past and save it in the collective memory. The contribution of the Architectural History within the context of Architecture education to the creation of architecture in the collective memory can not be disregarded. But as mentioned before, past generations must have made an effort and reflected this on the architectural products in order for this to become a subject in the curriculum.

The interventions to the historical structures after the earthquakes define only a small portion within the information scope of the Architectural History. It can be considered within the area of specialty, which is developed on the structure, rather than the design, since it remains current due to its structural nature. Forefront positioning of the earthquake phenomenon as a design element in post-earthquake structures has surely an influence on the fact that it is now considered a part of the Architectural History curriculum. The design should not just be constructed in the production of residences that are earthquake-proof, but also in public areas, in which the earthquake will be remembered and/or forgotten. The designs that have made in order to establish a collective memory in Turkey will necessitate a discussion about the earthquake phenomenon in our Architectural History, and initiate a new era in the establishment of the relationship between the earthquake and the Architectural History.

ACKNOWLEDGMENTS

[1] I find it necessary to cite a quote from Stefan Behnisch, which goes "Buildings reflect the capabilities of the societies. Next generations will assess us with what we will have done" (see Arredamento Architecture, 2009/07-08, p.12 for this quote of Behnisch among others).

[2] Sanem Yazıcıoğlu, in her article titled "Time to Forget /To Forget Time", mentions that forgetting the present time indeed means forgetting ourselves. The opinion that the people act with the same reactions, against a possible or current earthquake, belongs to me.

[3] The earthquake museum which has been opened on 20 December 2001 in Bolu has decayed since its ceiling has cracked and due to water leakage (Radikal Gazetesi, 7 September 2002). According to a news published on 26.11.2007, it is written that the earthquake museum in Osmaniye has been let out of use. According to the news in

Zaman Gazetesi on the date 22.08.2006, the earthquake museum in Sakarya, which has a population of 700.000, has been visited by only 100 people.

[4] Here, I have used Harvey's expression in order to address the earthquake concept.

[5] Models and simulations where people can experience the earthquake, where the videos taken by means of camera under the ground or under the water are offered to the audiences by means of screens surrounding the space, which give an impression of underwater or underground, films and archives which tells the impacts of the earthquakes to occur in other parts of the earth, a map which indicates the tracks of the damage occurred in Kocaeli because of the earthquake, memory rooms which give sounds, dark spaces, use of the power of expression of the worried eyes, Tsunami simulation, fog effect, fire smell and hooter sound, appearing of social walls which are damaged and collapsed besides collapsing physical walls, theater for interactive games in respect with earthquake and preventing disasters, except from its function of, besides the destructive impacts of the earthquakes, the exposure of the unity and solidarity within the society, conference halls, cafes and parks where the effects are alleviated for the reminding to the people of the earthquake concept and the botanic garden which reconcile the people with the soil after the earthquake and other areas where plant growing is made, can be designed. (The building created in Kobe with the name "Disaster Reduction and Human Renovation" takes attention as a space where, rather than a museum, interactive communication is provided and the people are given awareness, however it describes a construction with closed spaces whose function has been fixated directly over the earthquake and all around the building has been covered with a dark colored wrap which makes a reference to the bereavement after the earthquake. Whereas, it can be provided that the spaces can be fixated which are similar to the spaces where today's people often visit in their daily lives, and that they visit the public spaces which are created for earthquake)

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EVALUATION OF CHANGES IN BUILDING CONSTRUCTION PROCESS (PLANNING-DESIGN-PRODUCTION) AFTER 1999 MARMARA EARTHQUAKE

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Key Words: Building Construction Process, Construction Law, Earthquake Legislation, Construction Audit Law.

ABSTRACT On August 17 and November 12, 1999 two major earthquakes took place in Eastern Marmara Region effecting Kocaeli, Duzce and Istanbul and caused a great loss life and possessions.

This was the first time a that big earthquake effecting a that wide area happened in our country and losses were mainly caused by buildings partially or totally damaged. This fact resulted in various researches and legal arrangements about the phases of building construction i.e. enterprise/planning, design, construction and usage.

However, the process of building construction involves various disciplines and policy makers and that rises the need for assessing the studies and precautions as a whole.

In this paper, revisions in laws and legislations effecting the process of building construction after the earthquake, reflections of the subject on architectural education, studies of professional chambers and various researchers will be assessed and their effect on construction process as a whole will be stated.

INTRODUCTION

According to the most general description, disasters are phenomenons caused by nature, technology or human beings causing physical, economical and environmental losses, effecting societies stopping or ceasing daily life or human actions and from which societies can not overcome by their own resources (Anon, 2007). Considering their origins, disasters are classified as;

- Geological originated disasters,
- Meteorological originated disasters,
- Technological originated disasters,
- Human originated disasters.

Earthquakes are geological originated disasters and can be described as vibration of earth's outer layers by seismic waves originating to energy accumulated in the relatively inner layers of earth bursting as a result of cracking techtonical plates. As a natural phenomenon, earthquakes, with their effects on settlements and built environment, may become a disaster causing loss lives and properties (Anon, 2007), (Anon, 2000).

Statistics point out that earthquakes are the most destructive of all disasters. The reason for this is, 66% of the land in our country is 1st and 2nd degree earthquake zone, in other words active fault zone, and 71% percent of the population lives on this part of the country (Anon, 2007). Official records of the 80 years passed from the foundation of the republic, accuracy of which are arguable, state that total lives lost because of the

earthquakes are 80.546, total buildings damaged are 541.018, most of which are houses, and calculations based on prices defined by Ministry of Public Works show that there is a loss of $100 billions on building scale, including the economical losses because of the damaged buildings total loss reaches $400-500 billions ($5 billions every year on the average).

Table 1. Distribution of Earthquake Risk Factors in Turkey.

Earthquake Zone	Surface Area (%)	Population (%)	Industry (%)	Dams (%)
Zone 1(pga ≥ 0.40 g)	42	45	51	46
Zone 2(pga = 0.30 - 0.39 g)	24	26	25	23
Zone 3(pga = 0.20 - 0.29 g)	18	14	11	14
Zone 4(pga = 0.10 - 0.19 g)	12	13	11	11
Zone 5(pga < 0.10 g)	4	2	2	6
Total	100	100	100	100

1999 Marmara Earthquake took place 10 years ago. With the effected area and the qualities of the settlements, with a high number of lives lost approximately 20.000 and the poverty loss, the disaster opened deep wounds on people and had a high impact on the economy.

Since settlements and the built environment is the subject of building construction, this situation points out that there are critical problems about building construction and the reasons for these should be examined from multiple points of view because both the single building constructions and the settlement scale organizations' building construction has multiple phases and actors. This examination should be carried out in multiple dimensions and for the use of decision makers and administers.

According to this approach, basic problems of building construction in our country are as follows:

The lack of meeting housing needs growing because of population increasing naturally and with migrations especially since 1950s.

Although constitution states government's role about residence and there are numerous legal regulations about this, administrations, ignoring these, leave housing supply to market conditions making it subject to private property. Besides these, following populist policies dispensation laws were published to legalize masses of residences and the community prioritized acquiring houses or gaining profit from houses (Çetiner, 2005).

Thus, most of the buildings suffered from 1999 Marmara Earthquake in İstanbul were illegal constructions.

Regarding these, it can be pronounced that 62% of the urban residence stock in Turkey is licenced, the ones with usage licence is 33% which means 67 ones of every 100 residence is illegal and almost half of the legal houses don't have a usage licence (Anon, 2008). On the other hand analyzing from angle of İstanbul which is subject to an earthquake that is about to happen, there are 206.500 buildings licenced in İstanbul till the end of year 2000 30% of which has applied for dwelling licence and 70% hasn't according to the municipal records. Regarding that there are 280.000 buildings in İstanbul, 61.950 buildings has dwelling licence but 798.050 buildings neither has no licence nor suitable for licencing (Anon, 2002).

Taking only licenced residences into consideration in our country, there is a need for 2,5 million houses in urban areas. According to datum that originates to TUIK, urban population will extend to 11 millions and number of residences needed will reach 2,9 millions. In Turkey, 600 thousands of residences are needed every year because of changes like population rise, migration and urbanization (Anon, 2008).

However it is obvious that considering only the licence is not right and enough for interpreting buildings as qualified. Because qualified buildings must have values like function, economy, stability and aesthetics which should be handled throughout the construction process in all planning, design, construction and usage phases.

But because of the earthquakes experienced, only the stability title of qualification has been taken into consideration and emphasized, relationship between the earthquake and the buildings' structure has been tried to set or its effect on forming the structure was expressed. The concept of "Earthquake Resistant Building Design", that became popular especially after the 1999 earthquake, expresses that the construction should stay stable in any circumstance and under any load but relates this only to the earthquake. Since almost whole of Turkey is earthquake zone, it is inevitable that loads being regarded while designing the constructional system and examining the problems that may occur under these loads should be studied regarding this so called ignored factor and the results needs to be evaluated scientifically. During the studies and scientific researches about the subject, this need causes more limitations and the scope of the research expands.

The earthquakes also implied that designers neglected working in cooperation with civil engineers when making decisions about structures.

Although it is inadequate and erroneous, the concept of earthquake resistant building design and stating it as a new expression that it is a new design scholarship which should be carried out parallel to civil engineering has been featured and expressed as a result of ignorance of the importance of structural design in order to construct buildings resistant to any impact and the necessity for admissions and regulations that guide architectural design to be in conformity with settlement decisions and applicable.

It has been expressed frequently that it would reduce the risk of damage to consider the earthquake as a new scientific point of view, that the society developed, to be in dialogue with engineering disciplines, to avoid faults in calculations carried out in engineering services, to provide a certain quality in labour and to use appropriate and adequate material in order to obtain safe structures and on the other hand to constitute an inspection mechanism consistent to laws (Cetiner, 2005).

From the settlement scale point of view, it has caused featuring and reassessment of numerous legal regulations and projects because of government's role of being constitutive regulator and decision maker about both production that consists multiple disciplines and the complex relationship between these and the effort of multiple decision makers, and the quality and guiding of applications that is supposed to be the result of these studies.

Because it became obvious that, more accurate the legal regulations about planning stages, less effect of the earthquake would come out and defects and faults in application of laws and voids in the laws caused big losses (Cetiner, 2005).

In this paper the changes in legal regulations effecting building construction directly or indirectly will be questioned, the changes in architectural education of universities, pointing the earthquake is to be assessed and the new studies carried out by chambers of profession is going to be commented considering the whole of building construction process.

LEGAL REGULATIONS PERFORMED IN TURKEY AFTER THE EARTHQUAKES IN GENERAL

Although its importance is obvious that laws, legislations and related regulations should be prepared and applied properly in order to eliminate various factors causing earthquake damage both in building and settlement scale, the government doesn't only perform its role about this, but rules it out by announcing dispensations which contradict with constitution's 56th and 57th clauses (Cetiner, 2005). This situation can be observed by examining the laws and legislations made since 1976.

The earthquakes in Lice (1975, 6,6), Muradiye (1976, 7,5), Palu (1977, 5,1), Biga (1983, 6,2), Erzurum (1983, 6,9), Balkaya (1984, 6,4), Dogansehir (1986, 5,9), Kars (1988, 6,9), Pulumur (1992, 5,8), Doganbey (1992, 6,0), Manisa (1994, 5,1), Dinar (1995, 6,1), Kigi (1995, 5,7), Mecitozu (1996, 5,6) and Antakya (1997, 5,6) caused enormous property and life losses because of irregular structures. Following are the laws eligible but ignored then (Oztekin et al., 2005).

On 01.07.1972, 6785 Building Code has been revised by the law 1605.

In 1976, with the law 1990, tearing down of illegal buildings arised between 1966 and 1976 has been stopped because of the dispensation announced for the 50th anniversary of the foundation of Turkish Republic (Oztekin et al., 2005).

With enactment published in the official journal on 15.06.1979, in order to provide economic house for the people with low income, it was planned to acquire land with expropriation and give these to cooperatives, mass housing institutions and small construction companies (Oztekin et al., 2005).

2487 Mass Housing Law published on 08.07.1981 is a big step for housing cooperatives. With 5% of the budget government support is provided for the first time. Rising the area limit for social houses to 100m2 and excluding small cooperatives of 25 or 30 houses are the negatives of this law (Anon, 1982).

With the law 2805, a clause in 6785 Building Code has been changed on 16.03.1983 featuring a dispensation and people were let to live in unhealty, illegal and physically deficient houses (Anon, 1982), (Anon, 1983).

On 18.11.1983 2960 Bosphorus Law has been published to protect Istanbul. The main flaw about this law is that it was local and it stated that there should be a separate law for every region to be protected (Oztekin et al., 2005).

With the law 2981 published on 24.02.1984, the complications of the law 2805 was tried to be overcome but improper formations like technical offices under oath were created for determination purposes. The law also degraded the plans to scale sketches and pronouncing correcting zoning plans instead of updating current ones causing a new conceptual confusion (Oztekin et al., 2005).

On 09.05.1985 3194 Building Code has been published. As stated in the first clause, the code was published in order to obtain settlements and buildings in these settlements suitable to conditions of plan, science, health and environment. In the second clause it is cited that all public and private buildings in municipal and contiguous areas are to be manufactured according to the code.

The regulation 7269 about the buildings to be built in disaster areas was published on 02.09.1997 and with this regulation, government moves its responsibility on to professionals and assuming that they don't do their job properly, explains them how to engineer and almost draws the architectural project (Oztekin et al., 2005).

The earthquakes in Karliova (1998, 5,0), Ceyhan (1998, 6,2), Golcuk (1999, 7,4), Duzce (1999, 7,2), Cankiri (2000, 6,1) and Sultandagi (2000, 5,8), which took place after these laws and regulations, caused a loss of 18.396 lives and 142.785 damaged buildings (Oztekin et al., 2005). As a result of these, it became obvious that we have to learn how to live with earthquake. Studies and precautions aiming earthquake gained importance.

One of these is decree law 587 named "Obligatory Earthquake Insurance Decree Law" that was published on 25.11.1999. In the 1st clause its intention was stated as "To determine bases and methods for obligatory earthquake insurances to be subscribed by building or land owners in order to sustain the damages caused by an earthquake". But since the law had no connection with the building legislations, was not convincing considering the government's practices before and lack of encouragements and sanctions (Balamir, 2000), according to Disaster Insurance Institute only 2.787.094 out of 12.988.669 residences applied for the insurance which is 21,5% and the majority of the appliers with 29,5% was from Marmara Region which was effected from the earthquake lately and suffered most (Anon, 2008).

"Decree Law About Building Audition" 595, which was published in 2000, is a major legal arrangement for rational auditing of building type constructions not inspected because of deficient "Building Code" 3194. Besides "Law About Engineering and Architecture" 601 and "Decree Law About Amendment on Law of Union of Turkish Architecture and Engineering Chambers" prepared according to this, mentions a concept like "Adequate Profession Person" and a system for applying this, with positive intentions. But as the Constitutional Court cancelled these laws, these progresses vanished. In order to fill this space, on 29.06.2001, the law that effects the building construction process the most, "Building Audition Law" 4708 is published hastily. Reminding the old and useless audition system in a way, this law was far from meeting the needs (Anon, 2002a). With this law authorization for audition is shifted from the municipalities to legal entities and thus has moved out from the extent of responsibility. So political institution cleared itself of charge. Furthermore buildings and facilities mentioned in the 26th and 27th clauses of Building Code are excluded (Anon, 2002b).

There are other laws and legislations about construction industry published other than these laws in the last years (Anon, 2008).

"Legislation About Buildings To Be Constructed in Earthquake Zone" published on 06.03.2007 and 2nd clause of "Law About Aids Performed with Cautions for Disasters Effecting Public Life" 7269 on 15.05.1959 among these, states the rules and minimum conditions for assessment and strengthening of all existing buildings and earthquake resistant design and construction of all public and private buildings to be built, changed, enlarged partly or totally in identified disaster zones.

It is expressed that this legislation is sufficient and even oversecure like legislation of 1998. The difference of 2007 legislation from 1998 is tht it includes repair and strengthening. Although it can be appraised that 2007 legislation was prepared ideally, it is renounced that practice of it would not be successful, engineers and architects would have difficulties in apprehension and comment, they would be in need of expertise on structural dynamics because terminology has entered the legislation and there are incohorencies (Yarar, 2002).

"Legislation About Buildings To Be Constructed in Earthquake Zone" published on 14.06.2007 and 2nd clause of "Law About Aids Performed with Cautions for Disasters Effecting Public Life" 7269 states the rules and minimum conditions for assessment and strengthening of all existing buildings and earthquake resistant design and construction of all public and private buildings to be built, changed, enlarged partly or totally in identified disaster zones.

"Legislation About Re-Finance of Credits Included in Housing Finance" published on 29.09.2007 and 38/a clause of "Securities Exchange Act" 2499 regularizes the bases of re-financing of credits included in housing finance.

The clauses about converting construction servitude to condominium ownership in "Law About Altering Condominium Ownership Law" published on 28.11.2007 and "Condominium Ownership Law" 634 published on 23.06.1965 has been changed.

With "Legislation About Application of Building Audition" published on 05.02.2008 basis about foundation and activities of building audition commission, building audition institutions and laboratories, architects, engineers and other stuff that will work in building audition institutions and laboratories, tasks and responsibilities of administrators, project authors, site managers, building owners and building audition institutions' associates, building audition service act and service fees, records of building audition institutions and architects and engineers working as auditer and inspector in them and certificating buildings (Anon, 2008).

Finally, it can be remarked that apart from not mentioning the earthquake reality in every law about construction in a country like ours' that experienced multiple earthquakes since its foundation, a complete building code hasn't been set up, so residences, offices, industrial or even public buildings constructed hasn't been audited properly. Instead of constructing healthy, audited, safe buildings, hasty, unaudited, unplanned productions took place. This is so unhealty that media executed sea sand as reason for deformations in structures. The technical responsibles of legal constructions' command in the subject has never been questioned (Oztekin et al., 2005). Despite all these legal arrangements, the need for a complete, contemporary building code covering all constructing and building actions residential or industrial, legal or illegal, public or private without any reservations regarding the regions or conditions all over the country is still present. The solution for this is instead of publishing dispensation for buildings constructed by gaps in laws or lack of audition and then try to meet the damage by dressing the wounds, political will should maintain every building fit to laws and legal system (Oztekin et al., 2005). Among the laws published after the 1999 earthquake, "Building Audition Law" and audition assignment are examined in detail because of their direct effect on construction process.

BUILDING AUDITION LAW 4708 AND AUDITION

Audition is the most important part of building phase of the building construction process. In 2200 BC in Babylonian King Hammurabi's laws it is stated that if a building collapses and the user dies, the constructor was to be executed. This is the punishment system of building audition and places the responsibility of the collapsed building directly on the constructor. Although operation of this system has changed throughout the past thousands of years, it is obvious that responsibilities and punishments are continued till today in legal frame and structural codes are developed in order to maintain strong buildings against disasters (Allen, 2004), (Cheeks, 2003).

On the other hand, despite developing building construction technologies, material quality and building codes and the fact that in industrial and developed countries buildings suffer less because of earthquakes or at least don't collapse totally, collapsing of buildings because of earthquakes in developing countries like ours points the problem caused by multiple reasons like approach of political authorities in planning scales, development plans, population growth, urbanization and urban renewal (Ozden, 2009). Although reality was obvious, with Building Audition Law 4708 published after 1999 Marmara Earthquake building audition is shifted to private institiutions from local administrations, every building covered by the law was to be audited by private building audition institutions operating with licence provided by the ministry, audition is executed according to an act signed by building owner and building audition institution. But it is reported that many building audition offices can't or don't operate properly (Kivircik,

2002), majority of the constructions go on unaudited like they used to be and the law is defective and deficient.

PROVIDING EARTHQUAKE SAFETY IN BUILDINGS AND THE ROLE OF THE ARCHITECT

Despite the fact that there are different risks that needed to be evaluated in settlement and urban scale, neighbourhood and building groups scale and finally single building scale and building audition process should consider these scales as a whole, according to the present legal constraints the architect that has single building audition assignment has the responsibility of auditing different phases of building production process like architectural project (1), structural system (2), building materials (3), close environment of the building and so (Ozden, 2009).

On the other hand, it is stated that the earthquake safety of a building can be evaluated according to certain issues like eligibility to both earthquake legislation and building code which means it was designed and constructed according to these and that it wouldn't take any damage from a light earthquake (i), it would take a damage that could be repaired from a medium scale earthquake (ii) or it would take damage but don't collapse from a heavy earthquake and prevent loss of life (Anon, 2002a).

The main problem about earthquake safety is not how architecture and engineering should get position during and after the disaster but clarifying what their role should be during design and construction and learning from these. Instead of "safe building", "safe building in a safe environment" should be comprehended and to obtain this, the subject must be handled on the basis of profession and must be translated to the public in order to make society accept. So the society would want and ask for the culture of living in a safe environment and prevent one's building's wouldn't endanger any other's from architect to engineer and from local authorities to administrations while arising a resident model seeking constitutional rights is needed (Kivircik, 2002).

The second major duty and responsibility for the architects is awareness about disaster and risk management. From this point of view, the duty of architect, as the decision maker for single or limited settlements and master of production process as designer, is not planning for the lot, area and quotient of which is determined by legislations nor developing the plans or the sections. Transferring the other issues to engineers, the architect should take care of the building as a whole. In addition, it is the architect that should realize and manage the disaster risks of buildings, group of buildings or environment (Kivircik, 2002).

In brief, obtaining earthquake safety doesn't require only building audition, but various studies from different disciplines examining the production as a whole (National Earthquake Council Report, 2002) and it is inevitable that architecture profession and architects should be emphasized. There are studies performed by both chambers of professions and universities to obtain a general project in order to obtain healthy buildings and environment by raising the standards of architectural education in universities, handling this education according to professional practice to cover disasters and earthquake consciousness, emphasizing vocational retraining including earthquake, trainings to be performed by universities or chambers of professions and providing professional architects to be trained too, in addition to architects, training professionals from other disciplines, describing professional practice and eligibility to give authorization and responsibility gradually according to education and experience (Kivircik, 2002).

CONCLUSION

After the 1999 earthquake, with multiple legislations and laws published by the government as the basic regulator and decision maker of building construction, the role of architects and engineers as decision makers in single building scale has changed

towards a better approach to the earthquake factor. It is observed that this change has brought new concepts about description and education of professions. Especially in architectural education, earthquake, although being a factor that should be taken into account during structural design, outrun all other factors and found its unique place in educational curriculum and new formations appeared in chambers of professions.

It is expressed that according to the legislations and laws that are published, a well designed active building audition system is necessary while the technical stuff in charge should be well educated and have a high moral standard in order to warrant the success of the audition system. But although importance of the education of the technical stuff is emphasized, it was missed that this was not enough.

Accordingly the government should

- pass the responsibility to above mentioned technical stuff while publishing legal regulations as its basic administrative function regarding disaster planning and disaster risk managements as a whole,
- supply disaster maps indicating terrestrial and other various conditions for all settlements, municipal plans and plans of earthquake resistant settlements indicating the standards,
- institutionalize the building audition and its quality on building and environmental qualities basis,
- quit populist wound dressing policies (Balamir, 2000b).

On the other, while passing the responsibility to technical stuff, the government should

- stop founding universities, faculties and departments one after the other or increasing the student capacities of these without preparing the educational infrastructure properly like the education policy of the recent years.
- reinforce the educational personnel of the existing institutions, close the institution if this is not possible, avoid technical personnel inflation by stopping production of inadequate personnel.
- disregard our country's conditions during Bologna process, determine the durations and qualities of higher education according to all related associates in addition to these conditions.
- implement professional adequacy. The system to provide this shouldn't be constructed to satisfy the majority, but rely on realistic evaluations, authorizations given should be reevaluated regularly, renewal should be according to performance.
- improve vocational education, make it mandatory if needed. It must be realized that superficial courses don't have much of an effect. These courses should be organized regularly and be evaluated with an exam in the end (Anon, 2002a).
- implement the architecture profession law regarding the architect's role and public responsibility about constructional and environmental quality.

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OPEN AREAS AND GREEN AREAS RISK ANYLSES AND MANAGEMENT ISTANBUL KUCUKCEKMECE DISTRICT MODEL

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Key Words: Küçükçekmece, earthquake, open areas, green areas.

ABSTRACT In emergency conditions, urban open areas and green areas are needed more than one reason. Areas which are conserved on housing such as active green areas (children's playground areas, park areas, sport areas), passive green areas, car parks, squares, building gardens are supposed to be in sufficient size and proximity, especially to be unoccupied. Open areas and Green areas which have vital importance urban renewal and urban transformation projects and applications in case of emergency conditions are supposed to be provided sufficient proximity and size to residential areas, to be continuous and these areas must be planned as easy accessible and controllable. Risk level of spatial distribution of open areas and green areas which will be used in emergency cases is supposed to evaluate whether they are adequate from the point of proximity and widthwise to regions which has high risk level and will be needed more in emergency cases, and repairing inadequacies with planning is supposed to become compulsory. Open areas and green areas are needed areas to improve special risk evaluation and management methods because of having function of creating safe and accessible evacuation areas when earthquake and other possible disaster are discussed, open and green areas safe and creating accessible evacuation areas besides physical functions, physiological functions, economic and ecologic functions, For this reason, available open and green areas at the level of Kucukcekmece district were elevated and determined adequateness and needs, depending on this, some suggestion made oriented providing needs and reducing the risks. Geographical information system is used to reduce the faults to the minimum level.

INTRODUCTION

In urban life, open areas and green areas necessity is a reality which can't be denied. When concretely perceived open areas and green areas have been planned in a continuously functional structure, they have contributed open area and green areas of the city. Within multifunctional and systematic planning open areas and green areas can be fulfilled the functions. The important thing about open areas and green areas; is not only their creature in the cities, also being planned systematically. At the same time contributing system has to be ensured both spatial dimension and time dimension (Ülger, 2005).

In this study; open areas and green areas of Küçükçekmece District, have been considered within system approach. Systematic planning of open areas and green areas that have important roles at life quality is a very complicated subject. Systematic planning of open areas and green areas that are important for the urban life quality means contribution of future macro form of the city. Aim of this research is to definite of open and green area system, to contribute macro scale proposals in Küçükçekmece District where unhealthy developments are seen, to make a database that can be helpful for the micro scale projects. In this study, important and big first step will be taken within systematic planning approach.

As a result of open area and green area analysis examination in Küçükçekmece District; for an effective open areas and green area usage and sustainable development model open areas and green area system principles and spatial construction are aimed.

This study also aims to propose means by which the green areas can be increased in number and the active and passive green areas in the city made more accessible to the public, as well as how they could be converted to more modern recreational areas. Küçükçekmece, a densely populated, working-class residential and light industrial district in central Istanbul, is an example that indicates that the planning of green areas should be re-evaluated so that both the emergency and recreational needs of people living in a particular area are met while a more enjoyable, relaxed, healthy and peaceful life is also provided for them.

This research has been carried out so that the green areas of the predominantly residential district of Küçükçekmece can meet the needs for green area utilizations that are of contemporary standards of the residents in the pre-earthquake period and also can serve as an area that can be used for planning for emergency utilizations and gathering during an earthquake and in the post earthquake period and that can function as areas that can be accessed quickly and will remove the residents from danger and that can be used as open areas.

METHOD

For contributing theory part of the project, literature study has been done. BİMTAŞ Planning Group data has been benefited. Project's statically data between 1980 and 1995 has been gained in Yıldız Aksoy's doctorate thesis in 2001, at İTU Science Institute. The conditions for periods between 2000 and 2007 were those derived by the project team working within the context of the Küçükçekmece Green Corridor Project. 1/5000 scale construction plan is taken as base data. For minimizing the errors geographic information system is used.

CONCLUSIONS

As the population of Küçükçekmece district continues to increase, the inadequacy of green areas becomes more prominent. However, due to the increasing concern of the municipality for the preservation of the environment, considerable improvements have been made in plans for the development of such areas in the course of the past four years. In spite of this, the existing active green areas in Küçükçekmece are inadequate in terms of both their sizes, numbers, equipping and overall quality.

After analysis it is shown as amount of available active and passive green area per person doesn't fit with the green area standards for getting a homogenous distribution of Küçükçekmece District at neighborhood level. For meeting the different age group demands, functional distribution of green areas is very important. But while examining functional distribution of green area situation in Küçükçekmece District; it is shown that there is no per person children playground.

By 1,5 m^2 children playground per person standard, in Küçükçekmece District there has to be 1008083 m^2 children playground. In Küçükçekmece District 1,5 m^2/per person standard adequacy and 1008083 m^2 children playground adequacy are shown.

According to 3 m^2 / per person in Küçükçekmece District, there has to be 2016165 m^2 sport area but there is 103774 m^2 available open sport area and 2054321 m^2 district stadium. According to 2 m^2/per person, there has to be 1344110 m^2 parks in Küçükçekmece District. There is available 1.477.571 m^2 park area in Küçükçekmece District, so that park norm adequacy is not appeared.

According to Housing and Deconstruction Ministry standards 2352193 m^2 city park is needed by 3,5 m^2/per person norm. According to this; in Küçükçekmece District 2,8 m^2 norm adequacy, and 1881754 m^2 city park adequacy are appeared (Table 1).

As the values have been couched above; in Küçükçekmece District functional distribution of available green area is not suitable for the recreational demands of the different age groups (Aksoy and ark., 2008)

Within 2 m^2 norm proposal for collection area per person in Küçükçekmece District, adequacy of collection areas at neighborhood level has been done. At the end of the examination, there are norm deficiency in İkitelli Mehmet Akif and Yeşilova Neigbourhood. In Küçükçekmece District 1058 collection areas are proposed and it makes 24893169 m^2. By the movement of 2 m^2 open area per person proposal norm, it shows the norm deficiency in district level.

At the scope of Küçükçekmece District Green Corridor Project, available open area and green areas are examined at district level; adequacy and demands are determinated, at that context proposals are made for meeting the demands and minimizing the risks.

In Küçükçekmece District open area distribution are considered as building gardens, open car parks and empty areas. At neighborhood level open area adequacy has determinated by proposal 2 m^2/per person norm.

For achieving open areas and green areas; seriously, important problems are determinated in İkitelli Mehmet Akif (524949 m^2), Kanarya (458767 m^2), Halkalı Merkez (334723 m^2) ve Tevfikbey (314129 m^2) District (Aksoy and ark., 2008).

New areas have to be created in places within open area and green area adequacy. Land gain by compulsory purchase or the other planning instruments (barter, redevelopment benefit access etc…) by 3194 Redevelopment Law and 2942 Compulsory Purchase Law and creating these lands as open area and green area can be done. Open area and green area in Küçükçekmece District, is examined at time aspectbut it is seen that there is no time continuosly that contributes the urban identity (Ünal, 1996).

As a result we can say that; using and transformation of before and after earthquakes for loading a determinative role to green areas as a part of urban landscape planning and urban planning; have to be taken as a design data (Ahern, 1995).

According to 1985 land use and standards 120767 m^2 passive green area and 0,4 m^2/per person area are available. Between 1990 and 1995 there is 0,3 m^2 per person. Between 1990 and 1995 7 m^2 per person standard couldn't been reached. At 2000 0,2 m^2 per person, at 2005 3,8 m^2 per person and at 2007 5,4 m^2 active green area per person has been considered (Aksoy, 2001).

At 2000, 2005 and 2007 active green area is under the 10 m^2 standard. When the quantity per person is considered, there is no structure which is suitable for open area and green time period continuous. So that time period continuous couldn't not be mentioned.

The green areas form a part of the urban and urban landscaping planning process and it is important that they provide a dominant and marked service in case of an emergency condition. Thus, they should be incorporated into urban planning at the very beginning (Nalbantoğlu and Güzer, 2000).

The green areas play a significant ecological, physical and economic role in public health and local habitation of people and they should be organized so that they also have effective functions after a disaster such as an earthquake.

Table 1. Status of Active Green Areas in the Küçükçekmece (Aksoy and ark. 2008).

Function	**Existing Situation (m²)**	**Population**	**Per person Area (m^2)**	**Turkey Ministry of Development and Housing Norms (Per Person Area m²)**	**Norm Deficiency (Per Person Area m²)**
Playgrounds for Children	-	672.055	-	1,5	1,5
Park Areas	1.477.571	672.055	2,2	2	-
City Parks	476.028	672.055	0,7	3,5	2,8
Open Sport Areas	103.774	672.055	0,2	2	1,8
District Stadium	2.054.321	672.055	3	1	-
Total Active Green Areas	**4.111.694**	**672.055**	**6,1**	**10**	**3,9**

Generally, the distribution of green areas in Istanbul and in Küçükçekmece district is not homogeneous and as a consequence of this they cannot be expected to provide for an alternative use during and after a disaster, such as an earthquake.

Due to this non-homogeneous distribution, the provisions for sheltering and similar organizations following an earthquake cannot be planned properly. The poor conditions of existing green areas, their inadequate sizes, poor furnishings, inadequate infrastructures, wrong choice of urban furnishings and plants limit their services following an earthquake.

The quality of a green area and how it is designed are just as important as its size. Thus, it is necessary to perform renovation studies and to improve qualities of these places to make them more attractive to the public. So, Küçükçekmece Municipality should make an effort in this direction.

The use of green areas as dumps for debris and rubble after an earthquake has an undesirable effect on the general psychology of people living close to those areas. At the present, since these areas are not secure against fires and similar problems arising in the course of an earthquake they cannot be used to diminish damages and problems in case of such an emergency condition (Nalbantoğlu and Güzer, 2000).

At district level, to protect natural site that take a little part, to increase their quality and quantity, using landscape reorganization as a data in open areas and place plans have to become a must. Sites not externalize nature and each other also in a harmony have to take place that will increase public life quality. For answering these demands, it will be enough to make open areas and green areas considerable (Önder and Aklanoğlu, 2006).

Municipalities have responsibilities as planning resource use, increasing life quality by preparing clean and healthy life place, preserving natural structures for preserving natural resources. These responsibilities shouldn't take place on the papers also they have to be practiced. In the context of sustainable development principles; to transfer natural resources (water, soil, air, nutrient) to the future, balanced use have to be existed (Önder and Aklanoğlu, 2006).

Planning and design studies have to be supported by effective management model. In management strategies, Küçükçekmece Municipality as district municipality has to be in coordination with Metropolitan Municipality without responsibility and authority confusion. In the frame of sustainable development principles, responsible municipalities have to use limited resources as energy, water, soil and stuffs effective. Bicycle and pedestrian transportation has to be considered (Çevik, 2006).

The other neighborhood of Küçükçekmece, green area reorganization has to be considered as Başakşehir Model; removed from the compact buildings, giving green area possibilities to public, breathing areas.

While construction is continue in Altınşehir, Beşyol, Fatih, Halkalı Merkez, İstasyon, Kayabaşı, Şahintepe, Şamlar Village, Atakent, Yarımburgaz District as developing settlement area and low density settlement in Küçükçekmece; preserving ecology and environment have to be paid attention.

Halkalı Agriculture School (Halkalı Landscape High School) can be one of the places where natural plant types, especially under rescue types could be cultivated. Some laboratories could be constituted for vegetation preserve and development.

In Küçükçekmece District beside the urban transformation projects, basin planning which includes all the ecologic structure has to be done.

Planning strategies have to be ecologic and sustainable for effective and sustainable use.

To evaluate the sites which are suitable for sectoral land use at planning processes; ecological rescue sites has to be considered, available uses that cause negative effects on soil, water, biotope, bioclimax potential have to be stopped, biological and ecological restoration have to be done (Çelikyay, 2006).

Küçükçekmece has ecologic, historic and cultural potential. Küçükçekmece Lake, one of the most important lagoon lakes in the World, has to be purified from the wrong land use pollution, pollution factors have to be consumed and wrong land uses have to be prevented.

Physical plan causes false land use without considering Küçükçekmece's natural resource. As a result of these negative effects environment pollution increases, natural resource capacities are forced and that causes risks at natural resource recycle ability.

It is necessary to develop a green system utilizing the existing green areas and those planned for the future. The district needs new green areas for a healthier, more aesthetic or in general, for a better quality of life psychologically, sociologically and economically. People should be able to relax, to discove the nature and relate themselves to their environment in those areas. The green areas should be located so that they are easily accessible without a traffic problem.

The earthquake of August 17, 1999 revealed the necessity of making fundamental alterations in the traditional landscape planning of Istanbul and particularly of its Zeytinburnu district.

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OPINIONS ABOUT THE EFFECTS OF THE EARTHQUAKES LEADING TO DESTRUCTION IN IZMIT, THE OTTOMAN CITY ON THE PHYSICAL CONSTRUCTION IN THE CONTEXT OF RELIGIOUS CONSTRUCTIONS

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Key Words: Izmit, the effects of the earthquakes on the urban construction, wooden interior dome, mosque architecture.

ABSTRACT Izmit had been exposed to earthquakes, most of which had had destructive intensity, as from the dates which it had been constructed with the name Nikomedia in year 264 before Christ and had been reconstructed again after these earthquakes. These activities performed for the purpose of removing the effects of earthquake in the city, had had an impact on the historical development of the city as well as its physical appearance.

Izmit was also confronted destructive earthquakes after it had fallen under the hegemony of Ottomans. The destructions caused by the earthquakes in question in the city and the reconstruction activities after them took place in various sources. In this paper, the effects of destructions generated by the earthquakes occurred in Ottoman Period on the physical appearance of the city are intended to be examined by means of published sources, old visual data and documents relating to the city. The determinations regarding the subject discussed will be attempted to be defined over religious constructions which can be perceived more easily in the physical construction. In this sense, yet another point to be emphasized particularly as well is that in the city where the wooden inner dome is observed in the religious constructions, whether the fact of earthquake plays a part in being trended to this design or not.

INTRODUCTION

Izmit, located on the North Anatolian fault line, had been exposed to many earthquakes from the Ancient Times when it was established with the name of Nikomedia, to the present. Naturally, the phenomenon of earthquakes is one of important factors affecting the city's historical development and the physical structure. One of the purposes of this study is to answer the question of where these earthquakes had caused destruction at most, and to determine the influence area (or areas) in the physical structure of these earthquakes. The review of the relevant data has brought to mind the question of how the earthquake affected the city’s silhouette in the context of the mosques, as the common approaches in the reconstruction of mosques after the earthquakes has been observed. The differences in the design of religious buildings may be considered to have less impact in the physical view. However, the mosques are one of the building types that come through at first glance in Islamic cities. The same is true for Izmit, the majority population of which was composed of Muslims during the period of Ottoman Empire.

As it is known, the negative effects caused by earthquakes are due to many reasons, and perhaps the most important of them is the soil conditions of the area affected by the earthquake. In this context, if the physical structure of the area where Izmit was located during the period of Ottoman Empire is addressed simply, the spreading area of the city is composed of wetland in the south, and the land available for agriculture in the north as

indicated in the map prepared in the end of the 19th century to show the swamp intended to be dried (Figure 1). Accordingly, the sloping northern part of the city is more available for residence, and has a soil structure that is more resistant to the earthquake.

Fig. 1. The Map that Shows the Land Structure of the Izmit Bay Surrounding Areas, (BOA, HRT.2279).

Before making relevant determinations about the topic, it is necessary to summarize briefly the physical development of Izmit as an Ottoman city. In Izmit, entered into Ottoman rule in 1337 definitely, the construction activities belongs to the early period had centered around the citadel belongs to the Byzantine period, and this suggest that the first residential settlements had been founded there. The trade area located outside the residential areas and usually under the castle in Ottoman urbanism was possibly in the district of "Yukarı Pazar" under the citadel in Izmit (Kaya, 2009). In addition, according to some sources, there was a small shipyard belongs the period of Byzantine period in Izmit (Uzunçarşılı, 1984). The physical development of the city has gained momentum as Ottoman rule began to strengthen in Anatolia. After the conquest of Istanbul, it became an important menzil (frequented) place along the road connecting the capital to Anatolia (Bostan 2001); also the need of supplies, timber and firewood needs of the capital were met from Izmit (Fıratlı, 1971). In the period of Süleyman the Magnificent, the expansion of the Baghdad Road that passes through İnönü Boulevard today and bringing the shipyard into force are among important factors supporting the development in respect of commerce. As a result of these developments, it is observed that the important reconstruction activities in the city, especially the commercial buildings, were centered on the region between Baghdad Road and the coast. Thus, Izmit, although it was going to be reshaped due to the different developments afterwards, maintained this particular structure composing of the residential areas in the north and the commercial buildings in the south in a general sense.

THE EFFECTS OF EARTHQUAKES ON PHYSICAL AREA AND THE RECONSTRUCTION ACTIVITIES IN THE MOSQUES AFFECTED BY EARTHQUAKES

During the Ottoman era, one of the devastating earthquakes in Izmit was occurred in 1509. According to the sources, this earthquake destroyed Suleiman Paşa Madrasa that is adjacent to Orhan Mosque located in citadel in the north, as well as five mosques and nearly three hundred houses (Öztüre, 1981). In the report, the location of destroyed mosques and houses has not been specified, but it has been stated that earthquake caused a damage in the shipyard, broke down the costal walls, and the waves caused by the earthquake affected the lower parts of the city (Öztüre, 1981). There is no extensive information about the reconstruction activities after the earthquake. However, Bedrettin el-Gazzi who had been in Izmit in 1530 mentioned about the debris of the buildings and Belon who had been in Izmit in 1548 stated that city was in disrepair. These statements indicate that the effects of earthquake did not disappear in a short time (Bostan, 2001).

Meanwhile, Izmit miniature drawn by Matrakçı Nasuhi, who participated the Iraq expedition of Süleyman the Magnificent in 1535, in a bird's-eye view according to the miniature understanding of that period, contains data about the city's physical structure (Figure 2). The miniature, in which the sea is not shown, pictures the city as the city walls destroyed from place to place. In the miniature, a religious building of hipped roof within the city walls is probably Orhan Mosque. It has been stated that the old plan was preserved in the restoration of the mosque, which was renewed in 1840 and has a wooden interior dome under a hipped roof today (Fıratlı, 1971). This indicates that the miniature in question was close to true. Other three mosques in the miniature had a dome; this might be accepted as evidence that there were domed mosques which were not destroyed by 1509 earthquake or build after the earthquake, if they were not depicted like that in order to emphasize that they were mosques. Another noteworthy point in miniature is the relative density of buildings in the regions above Orhan Mosque. As the density of buildings decreases in the bottom parts of the miniature, it can be said that the settlement is less dense in the south of the city.

Izmit was shaken by earthquake once again in 1567, and devastating earthquakes occurred in the 18th century. According to Silâhdar Fındıklılı Mehmet Ağa, eighty percent of the city and six mosques were destroyed, and also the shipyard was also damaged by

the earthquake, occurred in Izmit and its surroundings in 1719 (Silâhdar Fındıklılı Mehmet Ağa, 1969). According to another source, a large part of the city was destroyed, the Customs Building was flooded with sea water, Mehmet Bey / Fevziye Mosque and its premises were completely destroyed, Pertev Paşa Mosque was partially damaged by the earthquake (Bostan, 2001). As all of the mentioned structures were located near the coast, it is seen that earthquake caused more destruction in the south of the city. After this earthquake, the reconstruction activities in the city were organized by Damat Ibrahim Paşa between the years of 1724 and 1730 (Öztüre, 1981).

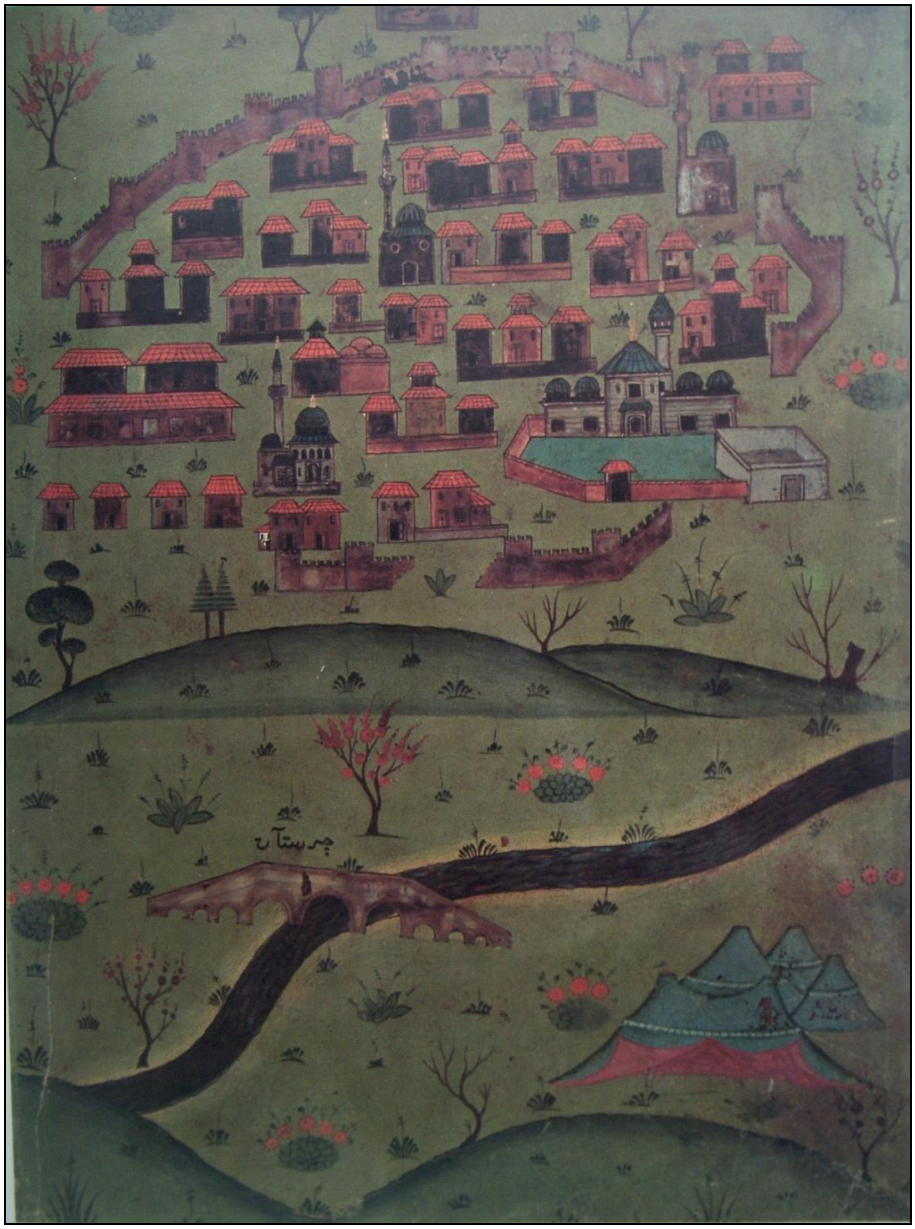

Fig. 2. Izmit Miniature Dated 1535, (Yurdaydın, 1976).

After the 1719 earthquake, the most recent visual document that gives an opinion about the city's physical structure is the panorama of Izmit drawn by Peyssonel in the 1745 (Figure 3). In this panorama, the city was pictured as spreading from the sloping terrain in the north to the eastward in the southern plain. The density of buildings was high in the north, but was relatively low in the east part of the southern plain. There was no settlement along the cost except the shipyard, and large and small buildings. IV. Murat Palace located at the hill in the west of the shipyard and the mosques located in certain

points of the city stand out in the silhouette of the city. When the panorama is examined carefully, it is seen that there are domed mosques in the upper parts of the city, built before or after the 1719 earthquake. From two mosques, shown in the eastern end, the domed one should be Pertev Paşa Mosque, and the one with a hipped roof should be Mehmet Bey / Fevziye Mosque. As it can be remembered, one of the mosques destroyed by the 1719 earthquake was Mehmet Bey / Fevziye Mosque built in the second half of the 16th century. The mosque which has a wooden interior dome under the hipped roof today has been renewed several times after being exposed to earthquakes and fires, and it has been rebuilt after the 1719 earthquake. According to archive documents, the money necessary for the construction of the mosque after the earthquake had been partially obtained by selling the old building's lead sheath, and the roof of the new building had been covered with tiles (Galitekin, 2002). Accordingly, the mosque probably built as a domed mosque in the past has been covered with a hipped roof after the 1719 earthquake as shown in the drawing of Peyssonel.

Fig. 3. The City of Izmit, 1745, (Peyssonel, 2005).

Izmit was shaken again by the 1766 and 1766 earthquakes in the same century, and the latter one caused a considerable damage in the city. The earthquake, occurred on 22 May 1766 and lasted in two minutes, destroyed all the buildings located along the cost, and made the shipyard unusable (Öztüre, 1981). IV. Murat Palace was also among the buildings destroyed by the earthquake (Ulugün, 2002). All of the structures in question were located in the south of the city. It indicates that the same parts of the city were affected by the earthquake as in the previous earthquakes. After almost 50 years, the reconstruction activities were begun again to eliminate the effects of earthquake. The reconstruction of the mosques affected by the earthquake was among these activities. For example, Çalık Ahmet Paşa / Yalı Mosque, which had been built as a domed mosque in the beginning of the 18th century, was built with a wooden roof after its roof was destroyed by the earthquake (Öztüre, 1981). Also Abdüsselam Mosque, which had been built as a domed mosque in the first half of the 16th century, was covered with a wooden roof (Öztüre, 1981). This mosque in question has taken its final shape after being repaired in the second half of the 19th century (Figure 4-5).

Fig. 4. Abdüsselam Mosque.

Fig. 5. Abdüsselam Mosque, Interior Dome.

During the Ottoman Empire era, Izmit was among the settlement areas affected in the first degree by the 1894 earthquake which affected a large area in Marmara region in the last period of the 19th century (Ürekli, 2000). This earthquake affected also the religious buildings as the other earthquakes, and caused damage in the public buildings such as İdadi (High School), İmaret and Maraşlı İptidai (Primary) Schools (BOA. MF. MKT. 229 / 33; MF. MKT. 219 / 30). The 1894 earthquake, apart from causing devastation in the city, affected the city's physical development by ending the reconstruction activities implemented during those years. For example, Selim Sırrı Paşha has initiated the road between Izmit and Bahçecik in the east side of the Izmit Gulf in 1893, the main purpose of which was to reach Bursa, but could not completed the road because of the 1894 earthquake (Yüce, 1945), (Öztüre, 1981).

One of the religious buildings destroyed by the 1894 earthquake was Mehmet Bey / Fevziye Mosque which had been destroyed also by the 1719 earthquake. The mosque was rebuilt by the Evkaf Nezareti (Ürekli, 2000). The mosque, which has taken its final shape after the 1894 earthquake, was rectangular nearly a square, and has two wooden interior dome, one of which is located in the portico and the other is located in the main place, under a hipped roof (Figure 6-8). Another mosque destroyed by the 1894 earthquake was Hüseyin Paşa Mosque. The mosque, which had been built as a domed mosque in the second half of the 18th century, was repaired and covered with a wooden roof after its dome was demolished (Ürkmez, 2007). Yalı / Çalık Ahmet Paşa Mosque, renewed after the 1766 earthquake, was destroyed again by the 1894 earthquake and was rebuilt again (Ürkmez, 2007). Both mosques are the examples of mosques in Izmit having a wooden interior dome under a hipped roof (Figure 9-12). The buildings affected by the earthquake were located in the south of the city again. This fact once more time confirms that the south of the city is a high risk earthquake zone in the city.

Fig. 6. Mehmet Bey / Fevziye Mosque.

Fig. 7. Mehmet Bey / Fevziye Mosque. Wooden Interior Dome in the Main Place.

Fig. 8. Mehmet Bey / Fevziye Mosque. Wooden Interior Dome in the Portico.

Fig. 9. Hüseyin Paşa Mosque.

Fig. 10. Hüseyin Paşa Mosque, Interior Dome.

Fig. 11. Çalık Ahmet Paşa/Yalı Mosque.

Fig. 12. Çalık Ahmet Paşa /Yalı Mosque, Interior Dome.

CONCLUSION

During the period of Ottoman Empire, the negative effects of these earthquakes, which ruined the city, depends on the severity of an earthquake as well as the relationship between structure and soil. The findings obtained show that earthquakes affected the wetland areas located in the southern part of the city at most. However, throughout the history of the city, this region has been used as mainly a business zone, due to the

commercial possibilities of the city. The mosques given as an example, which had a dome in their original forms and destroyed by earthquakes, are located entirely within this region. As it is known, the resistance of the domes to earthquake is directly related with their static conditions and ground-infrastructure-wall relationship. For example, Pertev Paşa Mosque which was built in the second half the 16th century, was not affected by earthquakes as much although it is located in the same area (Figure 13). This fact confirms that the buildings constructed according to the ground conditions and the right static conditions can be permanent.

There must be some reason that the domed mosques in Izmit were covered with a wooden roof after earthquakes. For example the above mentioned Mehmet Bey / Fevziye, Abdüsselam and Hüseyin Paşa Mosques, which were built as a domed mosque, were covered with a wooden roof after earthquakes. One of the common features of these mosques is that they have wooden interior domes under hipped roof, and these interior domes were possibly made during the reconstruction activities after earthquakes. In this case, it can be considered that usage of wooden interior dome under a hipped roof instead of a dome in the superstructure of mosques renewed after earthquakes may be due to the earthquake risk. However, this solution may be due to the fact that wood is abundant and cheaper in the region of Kocaeli that meet the fuel and timber needs of Istanbul; wooden domes may be preferred in order to alleviate the economic burden caused by earthquake. Besides this, it is known that wooden interior domes were used especially in the superstructure of weak masonry buildings and there are many similar examples belong to the 19th century in Anatolia (Tanyeli and Kuzucular). As a result of all these possibilities, it can be said that usage of wooden interior dome under a hipped roof in Izmit is based on earthquake probability, economic conditions and architectural style of this period.

Fig. 13. Pertev Paşa Mosque, 1895, (Photo Archive of German Archaeological Institute).

The main point here is that renewal of the buildings destroyed or damaged by earthquakes has affected the city's physical appearance. As it is stated above, all the mosques given as an example are located in the business zone in the south of the city. Thus mosques, which were built as a domed mosque since the 16th century, have ceased to be significant in the city's physical structure.

The renewal of the buildings destroyed or damaged by earthquakes according to the architectural style of that period instead of the original design is another dimension of the physical change. Mehmet Bey / Fevziye Mosque which was built in the classical period is one of the most prominent example of this. The mosque, which was destroyed by the 1719 and 1894 Earthquakes, was rebuilt according to the architectural taste of the 19th century after the 1894 Earthquake.

As a result, these comments are based on a review of the buildings affected by earthquake and renewed afterwards, and reconstruction activities. Indeed, there are other religious, public and civil buildings affected by earthquakes in the city. Some of these architectural structures were rebuilt, but some of them were completely eliminated. All these facts show once again that apart from socio-cultural and economic conditions, earthquakes have affected the physical appearance of the city, being exposed to one or more devastating earthquake almost each century, more than estimated.

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PRIORITIES AND PREFERENCES OF SOCIETY IN EARTHQUAKE RISK REDUCTION: A SOCIAL RESEARCH IN BAKIRKÖY

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Key Words: Risk mitigation, Social participation, Earthquake, Urban renewal, İstanbul.

ABSTRACT The interaction between three elements constituting space needs to be considered when the rearrangement of urban space is in question. These constituent elements are the Natural Environment, the Built Environment and the Social Environment. When at least one of these components is neglected to any degree, spatial relationship and the mutual interaction will deteriorate, and it will not be possible to establish a balanced system that operates in harmony.

For this particular reason, urban renewal and renovation projects intended to reduce disaster risks should not be regarded merely as an overall sum of construction projects. It is crucial to establish the relational balance between these three components and to facilitate harmony by minimizing conflicts in projects related to risk reduction, which require a series of new arrangements in urban space. In disaster risk reduction, an approach beyond the understanding of physical planning has emerged. This multi-constituent and integrated approach, called mitigation planning, is a system or executive scheme that encompasses all aspects of actions and programs of risk reduction.

This study specifically focuses on earthquake risks and the social component, which is a significant component of mitigation planning. A physical change anticipated for any kind of space will result in the element of humans, which brings space into existence, being affected directly and radically. The individual's and the society's ability to confront the physical changes in space with minimum damage or loss is related to the extent that they can accept/internalize these changes, which in return is related to the harmony with the subjective conditions that individuals and the society are situated in as well as the characteristic elements that shape their preferences. Therefore, a lot of internal and external factors such as psychological, demographic, and economic ones gain great importance for a good analysis. The determining factors underlying whether the individual will accept any hypothetical conditions about the future presented to them should be analyzed in cause-and-effect relationship, but more importantly, within the possible complex network of relationships.

Within this scope, a social research named "the Study of Determining Household Tendencies about Earthquake Risk Mitigation" was implemented in three quarters of Bakırköy district, which had been determined with a high risk of earthquake in the studies conducted after the 1999 earthquakes. The relatively old building stock in the area will help decrease the probable reactions against the renewal of the constructed environment. Under the light of these evaluations, this study owns a character of a social research which provides a groundwork for a probable future implementation on urban renewal within the scope of earthquake risk reduction, with its generous database.

INTRODUCTION

In today's urban renewal practices, public participation does not take place as it should. The authorities do not display enough effort or will to provide this, nor is there a valid institutional approach to serve as a model. Under certain circumstances, the society struggles to take part in the process with its own will. The success and sustainability of

urban renewal/renovation projects in the process of risk mitigation is closely related to primarily the users that will be directly affected by such projects and by the internalization of the projects by all segments of the society. Lewis and Mioch (2005) associated the success of this process with the way the society members are involved in the decisions that influence their lives, and they stated that the effects of a risk mitigation study without the consent of all members of the target community would be limited.

The concept of urban renewal with a focus on risk mitigation is a systematic and integrated planning approach consisting of seven components; namely, social political, administrative, legislative, financial, technical and environmental components. In this respect, in order to develop a tangible and feasible urban renewal model that cares for social peace and justice, the social component is an indispensable requirement in this model.

The more different segments of the society, which will be influenced by the planning participate in the process, the more support the plan and process will receive (FEMA, 2002). Field studies and surveys are highly effective tools in determining the approach toward possible risk mitigation practices. Besides, these tools enable a closer relationship with people encouraging them to be interested in the subject at hand (FEMA, 2003). To identify the local community whose participation is sought, it is crucial to make an intensive spatial analysis (Türkoğlu, 2003).

Accordingly, the purpose of the field study –within the focus on social peace and justice– is to determine the preferences and priorities of the households living in the defined field, along with the factors that influence these preferences and priorities.

The study has been constructed considering the requirements of a renewal/renovation project with the specific aim of mitigating earthquake risks. For this reason, it bears two points of departure: 1) mitigating earthquake risks, and 2) providing social participation in urban renewal practices.

At the phase of constructing this research project, a series of social studies on disasters and risks (Flynn et. al., 1999; İBB, 2002; FEMA, 2003; İBB, 2005; T.C. PUB, 2005b) have been investigated critically in terms of purpose and content. These studies have been utilized to contribute by evaluation of their negative and positive aspects during the process of forming the questionnaire and implementation method that have been developed which are in accordance with the purpose of this study.

A series of data have been considered in selecting the field. The first one is "the Project of Earthquake Risk Analysis Based on Ground-Structure Interaction in Bakırköy District". In this project, which was implemented by İstanbul University and İstanbul Technical University on Bakırköy Municipality (2003)'s demand between 2001 and 2003, 10162 buildings, corresponding to 96% of the buildings in the district, were examined and classified according to the level of risk (figure-1).

The second set of data is the "Feasibility Project for Reinforcement of Selected Housings" (T.C. PUB, 2005a) which is the continuation of the first project mentioned. In this project, implemented by the private sector in 2005 in cooperation between The World Bank, Project Implementation Unit of Turkish Prime Ministry-T.C.PUB and Bakırköy Municipality under the consultancy of Middle East Technical University, 369 buildings of high and very high risk groups were selected from the previous project, and they were assessed with structural analyses in terms of earthquake performance. With respect to utility-cost analyses, the requirement whether to pull down and reconstruct these buildings or reinforce them has been determined (figure-2).

The findings of both studies reveal that the selected quarters are located in highest risk group. Another set of data comes from the building age statistics taken from the building

inventory in 2000 by the Turkey Statistics Institution-TÜİK (figure-3). Building age is a significant factor influencing the level of risk. On the other hand, the proportional intensity of old buildings, which have completed or are about to complete their economically efficient lifespan, reduces the resistance against the implementation of renewal/renovation projects, thus offering an important opportunity in economic and social means and also disciplinary point of view. Building ages being above the district's average is a determining factor in field selection.

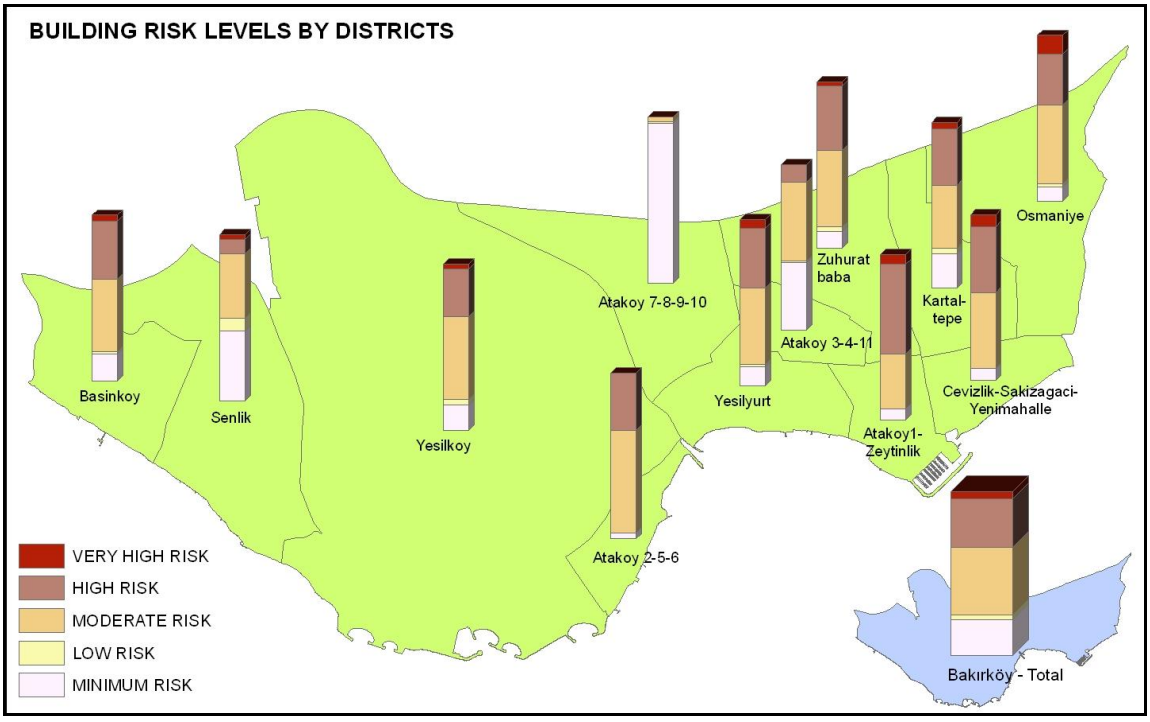


Fig.1. Distribution of building risk levels by quarters.

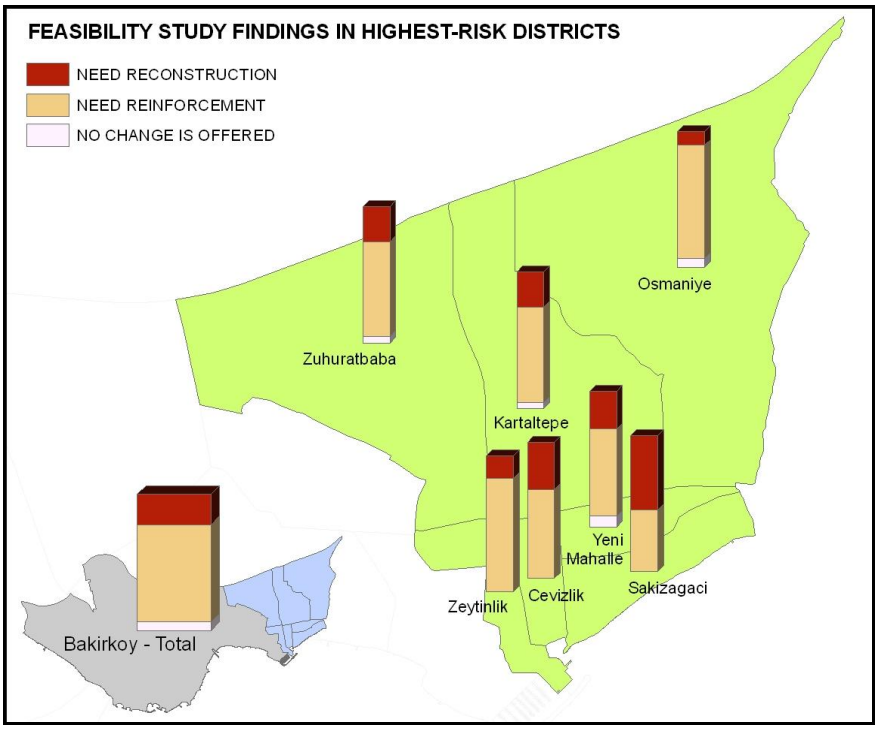


Fig. 2. Distribution of the Feasibility Study findings in highest-risk quarters.

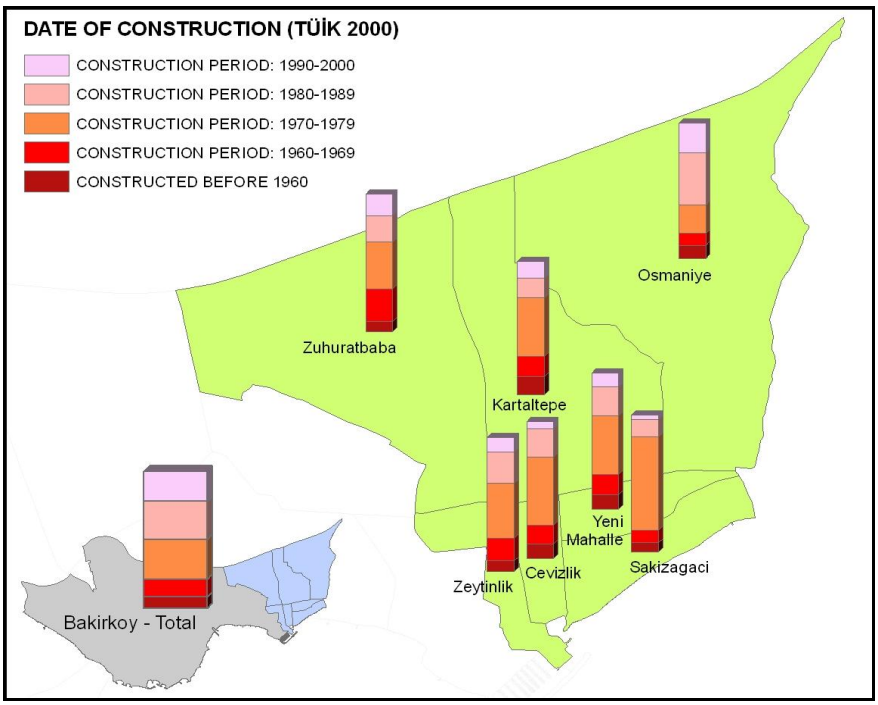


Fig. 3. Distribution of buildings according to the date of construction (TÜİK-2000).

Apart from these, other criteria determining field selection include the similarity of residential pattern, the spatial unity within identified borderlines, and mutual relationships between units. Following these evaluations, the study field has been determined to include Yeni, Sakızağacı, and Cevizlik quarters, located in south east parts of the district, and circumscribed by Sirkeci-Halkalı railway line, the Marmara Sea, Zeytinburnu district, and Ebuziya Street, which is the most important retail trade and pedestrian axis of the area (figure-4).

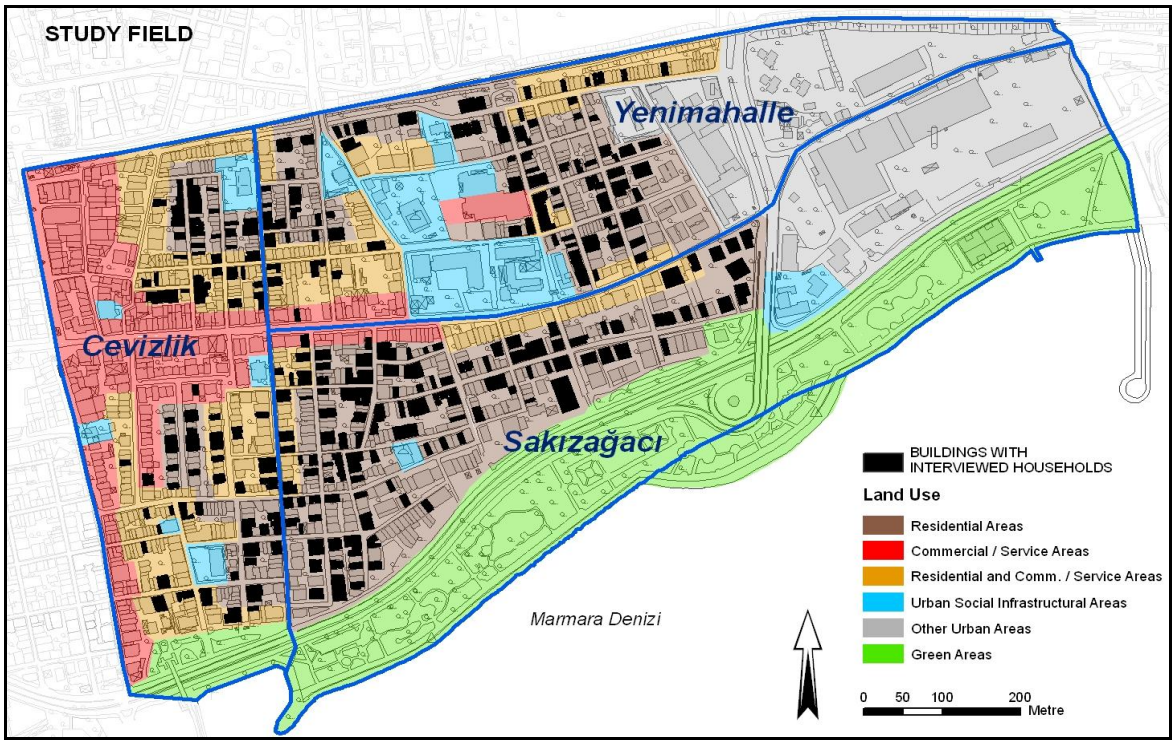


Fig. 4. The study field.

METHODOLOGY

This study was implemented at the dwellings, during June-August 2008 period, by using face-to-face interview method. 406 interviews were carried out, representing 1291 people in 382 buildings and 406 households. The main mass is 8844 which is the total count of buildings; the confidence interval is 95%, and the margin of error is 5% (table-1).

Table 1. The distribution of interviews among quarters.

Quarter	Sample		Main Mass		
	Count of Interviews	Rate	Count of Dwellings	Rate	Population
Cevizlik	**117**	**%28,8**	**2482**	**%28,1**	**5100**
Sakızağacı	**162**	**%39,9**	**3578**	**%40,4**	**7495**
Yeni	**127**	**%31,3**	**2784**	**%31,5**	**6919**
Toplam	*406*	*%100*	*8844*	*%100*	*19514*

The interviewers were organized in teams of two, one male and one female. The interviews were held at the dwellings with individuals who have financial income and/or are grown-up with the capability to represent the household. The respondents were asked over 100 questions excluding social and demographic parts. The duration of an interview varied greatly depending on the respondent's interest about the subject, level of knowledge, and personal characteristics. However, in general, one interview lasted for over 30 minutes.

SPSS statistical software version 11.5 was used for transferring the interviews to digital medium and for running the statistical analysis. Different multi-variable analysis techniques were utilized such as chi-square, one way ANOVA, Independent Sample T-test, Factor Analysis, and Multi-Linear Regression.

The survey presented four types of questions. The first type consisted of closed-ended questions, to which the respondents were able to give only one answer. However, the respondents could give more than one answer to the second type of questions. Ordering was yet another question type, for which respondents received assistant cards with alternatives written on each. Finally, the fourth type of questions were in the form of quinary Likert scale, which measured to what extent the respondents agreed with or preferred the given statements. Assistant cards were also used for this type of questions.

The questions were posed to the respondents under six headings in the following order:

1. Knowledge About The Subject Of Natural Disasters And Earthquake
2. Risk Perception
3. Preparedness
4. Opinions About Policies Of Risk Mitigation
5. General Information About The Household And The Housing
6. Information About The Building

The contents of the questions according to these six headings are given below:

<u>Knowledge About The Subject Of Natural Disaster And Earthquake:</u>

- Whether they have been victimized by any kind of natural disaster before,

- Whether they experienced Kocaeli earthquake in 1999,
- Their knowledge about post-earthquake studies carried out in İstanbul in general, and in Bakırköy district in particular,
- Through which means they have acquired information about the subject of earthquake,
- Which institution's or sector's statements do they rely on,
- Their knowledge about the risk level of the buildings they live in.

Risk Perception:

- Their level of anxiety about the listed disaster types,
- Their opinions as to when an earthquake is likely to happen in İstanbul,
- The frequency of their evocation of an earthquake,
- Major problems they associate with the concept of earthquake,
- The change in their anxiety level,
- Their perception levels about the threats of a possible earthquake.

Preparedness:

- The preparations the household is making in case an earthquake happens.

Opinions About Policies Of Risk Mitigation:

- The authorities in charge of risk mitigation actions,
- Which policies, incentives and means are preferred to what extent,
- What kind of sacrifices they can display,
- Participation in cost and cost sharing,
- Priorities in risk mitigation actions,
- Solutions of temporary housing,
- Policies regarding tenants,
- The announcement of risky buildings and the priorities about the conservation of building types,
- Social participation.

General Information About The Household And The Housing:

- Property ownership,
- The size of the household,
- Household income,
- The level of education,
- How long they have been in İstanbul and Bakırköy.

Information About The Building:

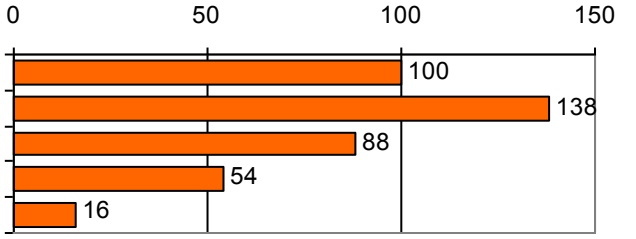
- The age of the building,
- The count of housings and the count of stories.

RESEARCH FINDINGS

In the region, the rate of property ownership was found to be 72% (Chart 6.6). However, the average monthly household income is relatively low (table-2). 60% of the households have a monthly income below 1500 TL. It is suggested that the high rate of retired population influences the general income level in the region. 31% of the respondents were determined to be retired.

Table 2. Average Household Income.

	Count of Observations	Rate (%)
Below 750 TL	100	25,3
750-1.500 TL	138	34,8
1.500-2.500 TL	88	22,2
2.500-5.000 TL	54	13,6
5.000-10.000 TL	16	4,0
Total	396	100
No Response	10	2,5



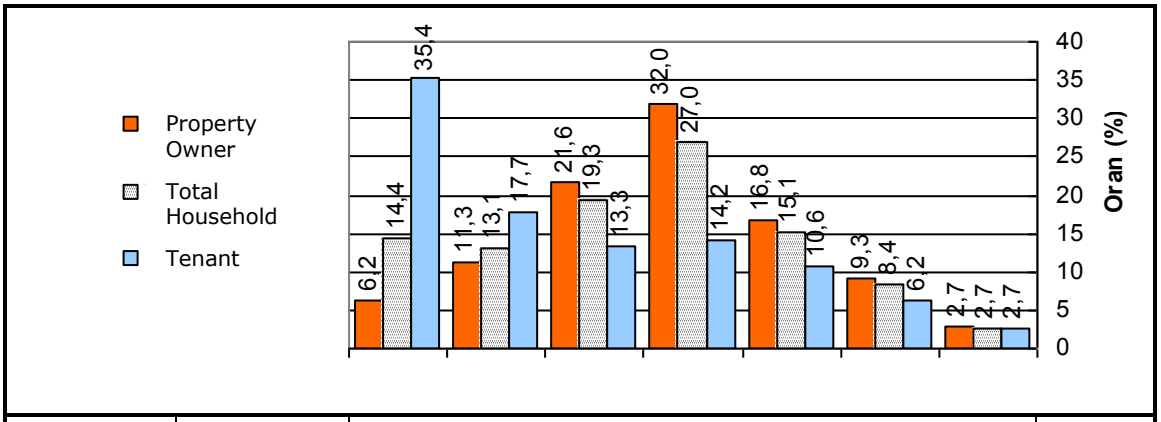
As of the date the research was implemented, approximately 89% of the households had lived in İstanbul at least for 20 years. The rate of people living in İstanbul for 30 or more years was 77%, and the rate of those living in İstanbul for more than 40 years reached 59%. When the duration of residing in Bakırköy was considered, the rates were 73% for 20 years and above, and 53% for 30 years and above. It is an expected result according to the process of urban development in the region. However, residing in the same building yielded opposite results, indicating that there was a relative mobility within the region for a number of reasons. It is concluded that the majority of the people in the research area have been settled in the city and in this region for a long time, and that their mobility has been limited to the immediate environment. This finding bears primary significance for the policies that might be adapted in possible urban renewal projects. While questioning the duration of residing in the city, the region and the building, the oldest individual who resides in the house was taken into account rather than the respondents themselves.

While evaluating the households' situation of being settled in the region in accordance with the property ownership, Chi-Square Independence Test was applied, and it was seen that there was a significant difference between tenants and housing owners. As table-3 shows, for households living in Bakırköy for less than 20 years, the amount of tenants are more than the amount of property owners, but it is observed that the situation is reversed as the duration of living in the area increases. The same result is also true – and more clearly – for the building in which the households lived during the research was being conducted (table-4).

Respondents were queried how much they trusted the related institutions and organizations when an earthquake is in question (table-5). The results reveal that governmental and administrative institutions are not reliable in public opinion. Local administrations, as well as central government and its related administrative offices were among the institutions with the lowest reliability index. Civil organizations also have a low level of reliability. It is observed that in public opinion academic circles receive the highest level of reliability. However, it should be noted that this reliability toward

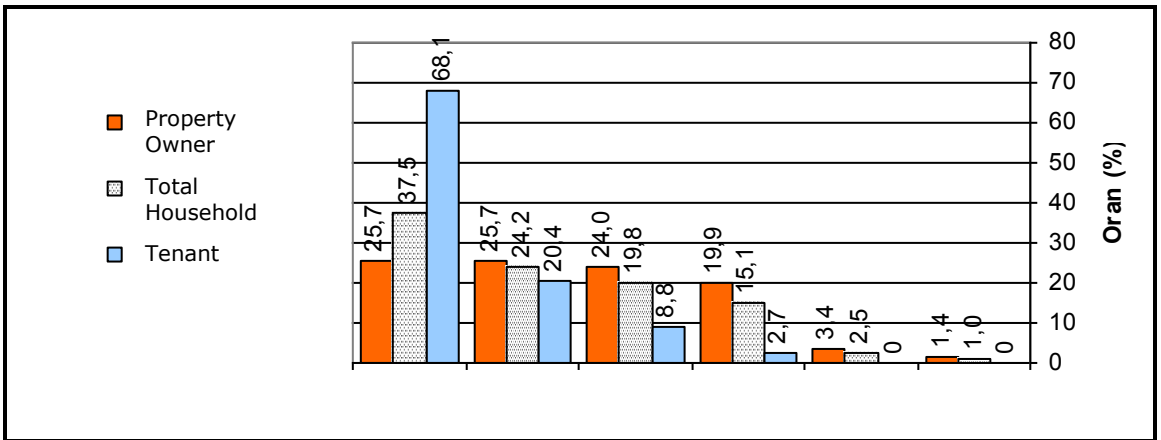
academic circles is only relatively high. This survey has also revealed that an important proportion of respondents do not have enough knowledge to state opinion about some of the listed institutions and organizations or the works they carry out. This situation could have resulted from the individuals themselves, but it should also be considered that the situation may be directly related to the degree of creating public opinion or taking part in current issues.

Table 3. Duration of residing in Bakırköy according to property ownership.



Property Ownership		Duration of Residing in Bakırköy (Year)							Valid Total
		0-9	10-19	20-29	30-39	40-49	50-59	60+	
Property Owner	Observation	18	33	63	93	49	27	8	291
	Rate (%)	6,2	11,3	21,6	32,0	16,8	9,3	2,7	100
Tenant	Observation	40	20	15	16	12	7	3	113
	Rate (%)	35,4	17,7	13,3	14,2	10,6	6,2	2,7	100
Total Household	Observation	58	53	78	109	61	34	11	404
	Rate (%)	14,4	13,1	19,3	27,0	15,1	8,4	2,7	100

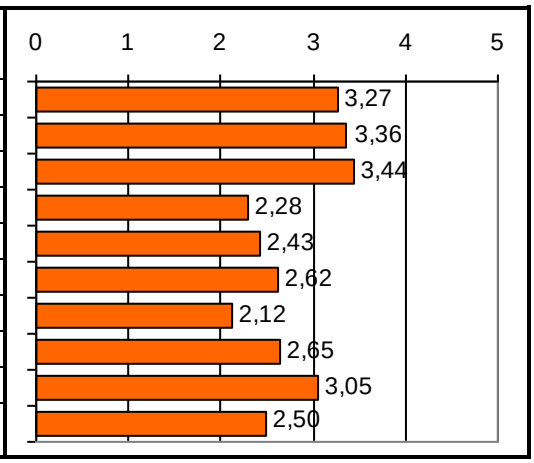
Table 4. Duration of residing in the same building according to property ownership.



Property Ownership		Duration of Residing in the same building (Year)						Valid Total
		0-9	10-19	20-29	30-39	40-49	50-59	
Property Owner	Observation	75	75	70	58	10	4	292
	Rate (%)	25,7	25,7	24,0	19,9	3,4	1,4	100
Tenant	Observation	77	23	10	3	0	0	113
	Rate (%)	68,1	20,4	8,8	2,7	0	0	100
Total Household	Observation	152	98	80	61	10	4	405
	Rate (%)	37,5	24,2	19,8	15,1	2,5	1,0	100

Table 5. Reliability towards Institutions – reliability index (average).

Institution	Reliability Index	Std. Dev.	N
Academics/Scientists	3,27	1,11	390
Tübitak	3,36	1,06	344
Kandilli Observatory	3,44	1,13	393
İstanbul Metropolitan Mun.	2,28	0,98	390
Bakırköy Municipality	2,43	1,02	389
Gen. Directorate of Dis. Affairs	2,62	1,01	318
Government / Ministries	2,12	0,99	385
Vocational Organizations	2,65	1,01	353
Non-Governmental Org.	3,05	1,04	378
Independent Experts	2,50	0,94	361



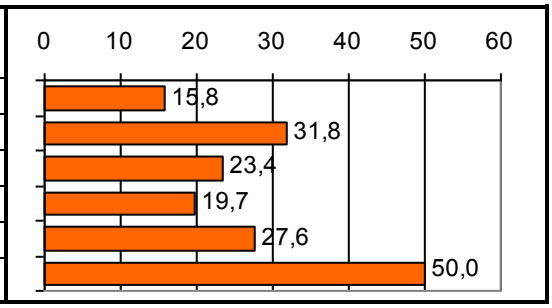
The research has also discovered that the respondents do not have enough information about the current or completed projects. When the question "Are you informed about projects implemented throughout İstanbul about earthquake?" was posed, 59% of the respondents gave the answer "no". When the respondents who replied "yes" to this question were asked to list a few of such projects, it was found out that only 13% of all respondents have correct knowledge about this subject. It has also been discovered that the respondents had very low level of knowledge about the earthquake projects implemented in their district following 1999.

Respondents were presented with a set of natural disasters and asked to state to what degree they are concerned that each disaster might affect İstanbul based on quinary Likert scale (1 = None, 5 = Excessively). As it had been anticipated, earthquake was determined to be the natural disaster arousing the highest level of concern, followed by drought. Considering the geographical condition of the study field, tsunami was expected to be a source of high concern, but it did not turn out to be so. For this reason, this finding is also considered as an indication that the households in the region do not have adequate level of awareness.

Respondents were posed a set of questions in order to determine the measures they took and the preparations they made to raise their awareness about the concept of earthquake and to decrease possible losses. As it can be followed from table-6, the rates of earthquake preparations are really low in the households.

Table 6. Readiness to earthquake of the households.

Activity of Readiness	Rate (%)
Meeting/Training about Natural Disasters	15,8
Family Plan	31,8
Earthquake Bag	23,4
First Aid/Rescue Training	19,7
Fastening the Furniture	27,6
Earthquake Insurance	50,0



Earthquake insurance has been determined as the most common precaution. However, earthquake insurance is not limited to Obligatory Earthquake Insurance which is a lawful statement. Respondents were asked whether they had earthquake insurance, regardless of which type. When other preparations were examined, tangible measures such as fastening the furniture and preparing an earthquake bag as well as having a family plan for the aftermath of the earthquake were found to be relatively high because such

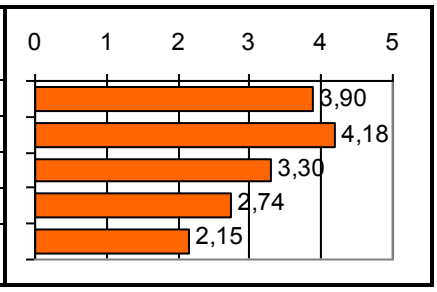
measures are presented to public's attention from time to time. On the other hand, training was discovered to be the most neglected preparation type.

In terms of providing earthquake safety in the dwelling, the highest responsibility seems to be placed on the contractor as the implementer, and on local administrations as supervisors.

One of the most crucial questions of this study is related to the preferred applications in order to live in a safer building in terms of earthquake threat. Respondents were asked to state to what degree they found the following applications acceptable in quinary Likert scale if the earthquake resistance of their building was low (1=I definitely don't prefer, 5=I definitely prefer). By calculating the average score of each application an index was created (table-7), which revealed that the most preferred application was to reconstruct the building in the same place. This was followed by reinforcement. The offer to provide a new building in another region in İstanbul or outside İstanbul was the least acceptable application since this would mean the transfer of zoning rights.

Table 7. Preferences about providing earthquake safety of the building in which households dwell.

Options	Index (Avr.)	Std. Dev.	N
Reinforcement	3,90	1,04	405
Reconstruction in the same place	4,18	1,04	405
Reconstruction in a nearby place	3,30	1,03	403
Reconstruction in İstanbul	2,74	0,99	405
Exchanging with a new housing outside İstanbul	2,15	1,00	406



Respondents were also asked which option they would primarily prefer among the options they were presented with. The replies seem to confirm the situation manifested by the previous chart. 61.1% of the respondents stated their primary preference to be the reconstruction of the building in the same place, 32.4% to be reinforcement.

The obvious result of this query is that respondents are determined not to leave the street or lane they live in. Even moving to a nearby place in the same area receives medium acceptance. Therefore, in the process of reorganizing urban space, it is highly important to establish policies, and to choose tools and methods of implementation according to the preferences mentioned above, so that a process based on reconciliation is maintained.

The purpose of queries about household preferences related to reinforcement or reconstruction should not lead to false conclusions that the actual implementations will be determined by the stated preferences. The purpose here is not to consult users about a decision that requires technical expertise, but rather to make users aware of the burden (psychological burden, concerning the family condition, financial condition, physical burden of moving, and etc.) awaiting them, and hence to check how much enthusiastic they are about solutions, as well as predicting the tendency to leave the environment they live in.

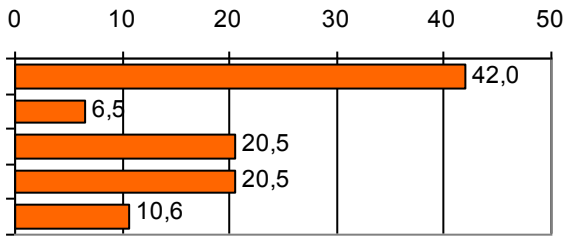
Whether to reinforce or reconstruct a building with weak earthquake resistance is currently a serious source of controversy among users. When the respondents were queried whether this controversial issue among users in the building could be reconciled or not, the rate of the ones with positive view seems to remain relatively low. Whereas respondents have been determined to hold firm opinions about the issue of reinforcement and reconstruction, the weak belief in reconciliation might be related to a number of factors including the fact that this issue has not been the subject of serious discussion among neighbors, that neighbors are not aware of each other's opinions, or that there

might be economic restrictions. This overall situation requires that the community opinion about the subject should be studied from various aspects, that the relationship between individuals should be organized to some extent, and that in this respect a unifying and guiding supreme administrative scheme should be developed.

In case of a possible reinforcement or reconstruction, who should undertake the cost is determined by various factors, which include primarily economic conditions, reliability towards the government, and opinions about the process of decision making. However, ideas related to sharing the cost and sharing the increased value appear to go opposite towards directions. When only property owners were asked what proportion of their budget they could spare for such implementations, the main determining factor is discovered to be related to economic conditions although other factors influencing cost share remain relevant (table-8).

Table 8. How much of your budget do you spare?

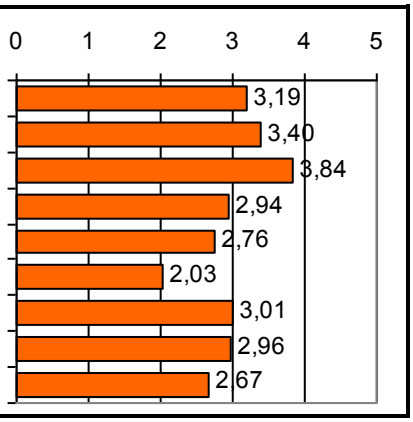
How much they spare	Count of Obs.	Rate (%)
None at all	123	42,0
Has doubts	19	6,5
Depends	60	20,5
According to their budget	60	20,5
Whatever is required	31	10,6
Total	293	100,00



Respondents were presented with a series of incentives and tools about the financial methods to utilize in case of a reinforcement or reconstruction, and they were asked to assess each of these items on a quinary likert scale based on their individual preferences (1=I definitely don't prefer, 5=I definitely prefer). The index created by calculating average scores of preferences is shown in table-9. As the chart indicates, the most preferred incentive is donation from the funds of the Institution of Natural Disaster Insurances (DASK), where Obligatory Earthquake Insurance premiums are also collected. On the other hand, the least preferred incentive and tool is securitization.

Table 9. Preferences about incentives and tools.

Incentives and Tools	Index (Avr.)	Std. Dev.	N
Long-term loan with low interest	3,19	1,24	399
Various tax reductions	3,40	1,17	393
Donation from dask funds	3,84	1,04	394
Exchanging the dwelling	2,94	1,07	400
Purchasing the dwelling	2,76	1,09	387
Securitization	2,03	0,83	343
Policies without increase in land value	3,01	1,22	341
Paying participation cost	2,96	1,07	395
A smaller dwelling	2,67	1,09	399



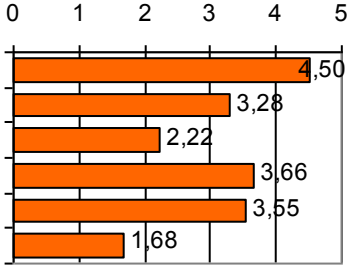
Within the scope of reorganizing the inhabited environment in terms of increasing its earthquake safety in unity with urban space, it will be necessary to evacuate the dwellers in the building that will be reinforced or reconstructed. When the fact that both property owners and tenants have lived in the region for a long time is taken into consideration, the problems that each group faces are subjects of different policies. While the changes in many aspects of life will mostly be temporary for property owners, it is possible that similar changes will be permanent for tenants in the same intensity. It should be a

primary responsibility of the government to protect the right of housing, which is a human and constitutional right, regardless of any conditions including property ownership. Under these circumstances, the situation of tenants should be considered separately and specifically to create a model around social peace and justice.

To this purpose, the survey asked respondents' opinions about this issue by presenting them with two separate sets of questions –the first being possible incentives for property owners, and the second being possible measures to decrease the problems awaiting tenants. As table-10 shows, respondents mostly preferred to live in a temporary housing provided in the nearby region among the incentives about temporary housing. Rental support is the second preferred incentive. But when the index values of options and standard deviations are taken into account together, it is clear to what extent the demand for the first preference is dominant. Respondents state with a definite expressions that they expect any kind of incentive about temporary housings. In general, it is maintained that the rental cost of the temporary housing as well as the moving costs should be covered within the scope of the project.

Table 10. Opinions related to temporary housing.

	Index (Avr.)	Std. Dev.	N
Temporary Housing in Nearby Environment	4,50	0,65	406
Temporary Housing Anywhere	3,28	0,99	406
Rental for Temp. Housing Be Paid by User	2,22	0,76	405
Rental Support Instead of Temp. Housing	3,66	0,85	402
Moving Cost Should Be Covered	3,55	0,99	405
No Need for any Support	1,68	0,66	406



Although the issue of temporary housing mainly concerns property owners, tenants' opinions were also asked. In order to determine whether there are any significant differences between property owners and tenants about their preference on the listed incentives, Independent Samples T-Test was applied. The results reveal that tenants go beyond individual measures about a subject that does not directly concern them and are able to empathize with property owners. This finding is considered as a positive condition to facilitate social peace and minimum controversy.

Also, the suggestions presented with a view to alleviating the problems tenants might face due to evacuation receive high support (table-11). In this respect, the tool that received the highest support include providing a rental housing in the nearby environment and financial support for a period of time in case of an increase in general cost. Providing priority and financial support for tenants to buy a new dwelling of their own constructed within the scope of risk mitigation program, also received considerable acceptance. Respondents are also in favor of providing rental housings in the mass housing areas constructed by the government throughout İstanbul. On the other hand, excluding tenants from the program completely and only compensating certain costs such as moving is clearly not accepted. Respondents firmly hold the belief that tenants should not be neglected.

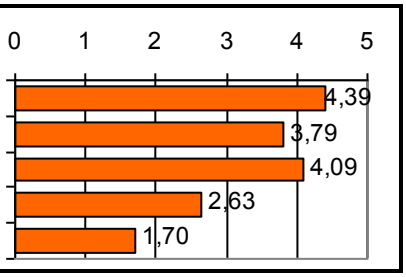
To examine whether there are any differences between the ideas of tenants and property owners regarding supports for tenants, as in the previous question group, Independent Samples T-Test was applied. According to the test results, tenants and housing owners have the same opinions about supports for tenants. This finding reveals that the empathy that the tenants establish with property owners is mutual.

Following the earthquakes of 1999, some projects focusing on earthquake risk mitigation were developed under the name of "urban transformation" in certain districts of İstanbul, primarily in Zeytinburnu, which were discovered to have highest earthquake risks (Jica,

2002). Apart from these, other projects are also brought to agenda, which do not have the primary aim to provide earthquake safety but rather to eliminate unhealthy housing stock of the city. These projects, which are again referred to as "urban transformation" projects, with some in the process of implementation, are generally shaped around the policies of global economy. In accordance with hosting congresses that gather global circles of immovable assets, such initiatives have intensified, going beyond local and national limits. In the regions where such "urban transformation" projects are implemented, inhabitants have acquired considerable level of awareness about this concept. In regions such as Tarlabaşı, Sulukule, Maltepe, and Sarıyer, inhabitants are struggling to activate the mechanism of social participation and not to leave the area they live in by means of local and democratic mass organizations.

Table 11. Opinions about the support for tenants.

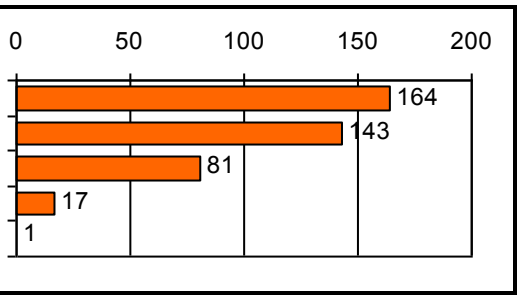
	Index (Avr.)	Std. Dev.	N
Rental House Nearby + Financial Support	4,39	0,72	404
Rental House Somewhere Else	3,79	0,80	404
Opportunities for Property Ownership	4,09	0,73	405
Only Moving Costs	2,63	1,00	404
No Support Is Necessary	1,70	0,65	403



In this respect, for the analysis of the current situation, it is important to understand what respondents think about "urban transformation" that has been on the agenda for a long time, though with more intensity from time to time, and more importantly whether they have knowledge about the subject. For this reason, respondents were queried in the simplest way possible with the question "Are you informed about urban transformation projects?" (table-12). While 40% of the respondents stated they had never heard of it, the rate of those with knowledge about the subject to varying degrees did not even reach 25%. It is an intriguing finding, which requires particular attention, that three fourths of the respondents are not informed about the mentioned concept and related projects.

Table 12. The level of knowledge about "urban transformation" projects.

Level of Knowledge	Count of Obs.	Rate (%)
Never heard of it	164	40,4
Heard of it but have no knowledge	143	35,2
Have little knowledge	81	20,0
Have enough knowledge	17	4,2
Know all the details	1	0,2
Total	406	100



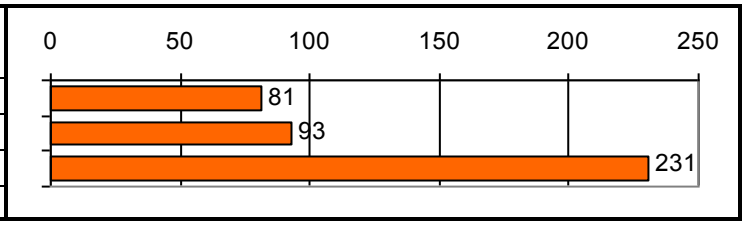
According to the current regulations, the public administration gives the information of a building's earthquake risk level only to those who can prove to be an owner or a dweller in that building. Otherwise, it is stated, speculations for unearned income could arise. On the other hand, if the risk level of a building ceases to be confidential information, it will be to the benefit of public good, fostering public health and safety. When respondents are queried, the claim of the public administration proves to be groundless considering that three fourths of the respondents are property owners. As high as 86% of the respondents state with most radical expressions that risky buildings be publicly announced.

Respondents were asked whether they would actively take part in a local committee that would create common solutions with the participation of public institutions, universities and non-governmental organizations so as to provide a holistic approach towards

earthquake safety in the environment they live in. 23% of the respondents replied "maybe", while 57% "yes". This rate represents a majority that cannot be underestimated (table-13).

Table 13. Would you actively take part in a local organization about the issue of earthquake?

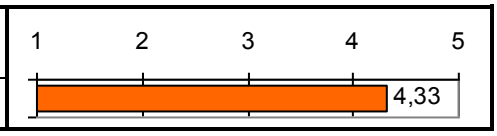
	Count of Observations	Rate (%)
No	81	20,0
Maybe	93	23,0
Yes	231	57,0
Total	405	100



The last question of the survey aims to measure social support, which is closely related to the main purpose of the study. The question was posed as follows: "How important is adapting the idea of Social Peace and Justice as the main principle of studies that will be implemented to mitigate earthquake risks to the success and applicability of these studies?" The index calculated by taking the average of the replies received on a quinary likert scale (1=Not important all, 5= Very important) has a value as high as 4.33 (table-14). 56% of the respondents assessed the subject to be "important" and 39% as "very important". It is clearly seen that the longing for social peace and justice strongly manifests itself also in studies for earthquake risk mitigation.

Table 14. How important is ensuring social peace and justice?

	Index (Avr.)	Std. Deviation	N
Social Peace and Justice	4,33	0,64	401



CONCLUSIONS

The most outstanding findings that should be noted about the households include residing in the region for a long time, the high rate of property ownership, and the relatively low income level. The majority of the tenants have been residing in the same region for a long period of time as well. The high rate of the retired people is one of the important factors influencing the low level of income in the region. These three socio-economic characteristics of the region provide two significant inputs to the process of political decision making –the first is the determination of the inhabitants not to leave the environment they live in, and second is their expectation of financial support from the government. These data were obtained based on the respondents' approach towards alternative actions and policies of risk mitigation.

Besides, all the data collected throughout the study point out that in any possible urban renewal/renovation project with a focus on risk mitigation, the public will display strong resistance against all models that involve moving away from the environment they live in. The inhabitants in the region are strongly attached to their environment to such an extent that they do not consider any concession even when temporary housing is in question, and that primarily expect a solution to the housing problem prior to implementation method or project finance. This finding is one of the most important and sensitive conclusions of the study.

The most acute finding about the physical environment is the high amount of buildings that are either close or have passed the economically efficient lifespan. Although this situation displays an important risk factor, it is also a significant opportunity to plan and foster extensive and integrated urban renewal process focusing on earthquake mitigation.

The inhabitants desire to see the leadership or reference of academic institutions and academics. This condition is crucial to maintaining public's reliance on and support to the project.

The inhabitants also object to the idea that if their housing is not safe for earthquakes, they should be the ones to cover all costs of physical and economic problems they are likely to face in any attempt to eliminate safety problems in the buildings as they do not have a chance to intervene in or supervise this situation. Apart from economic shortcomings, the lack of reliance on administrative bodies also enhances this reflex.

It is obvious in the study field that the longing for social peace and justice is also powerful in projects of earthquake risk mitigation. The most outstanding indication of the importance placed on providing social peace and justice appears to be the necessity not to put tenants in a difficult situation. It should be noted here that there is a mutual empathy between tenants and property owners on this particular issue. The public looks at the problem on a social level going beyond the individual level, which is a positive finding that social peace and a common ground for minimum controversy can be established. However, it is also discovered that there is no sufficient and extensive exchange of opinions among the society. For this reason, a supreme administrative structure is necessary, which will bring different segments of the society together, forming the basis for the inhabitants in the region to learn about each other's opinions, to be informed about the recent developments, to increase their awareness, to make decisions about their own futures, and to meet on the least common denominator. The society is ready to adopt such a model that will be constructed by collaboration between the government and civil agents. For if social participation is established, considerable progress will be made toward achieving social peace and justice.

In a probable urban renewal/renovation action, financial organization should be established on the grounds of not victimizing users with economic hardships, thus promoting social justice. A heterogeneous socio-economic structure is dominant in the region. By means of a flexible plan on the basis of maximum reconciliation, it is possible to put into practice more than one policy and implementation tool concurrently, and all the findings reveal a demand for this. To this purpose, it is crucial to organize a perfect risk management as a primary condition. The tool for such an organization is mitigation planning.

One of the most significant results of the research is the functional correlation between upper age, low education level, and low income. Unless a well-organized and participatory awareness raising and decision making process is put into practice, these subgroups will not be able to take part in the project implementations even in most favorable circumstances; as a result, it will not be possible to reflect their preferences, priorities and requirements in the model. That these groups are a part of the solutions is a criterion of social peace and justice.

All the findings point out to the requirement for raising the society's awareness and creating an organizational model that will put different social segments together with government/civil agents. In such an organization, the public should have a key role. In this respect, the policies followed by public rulers or those who are candidates to rule the public bear determining importance.

Another distinctive finding of the study shows that the society is open to and enthusiastic about collaboration to live in a safe environment, and that if the authorities can meet certain demands, the ground for a powerful reconciliation is obtainable.

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THE IMPORTANCE OF EARTHQUAKE RESISTANT BUILDING DESIGN ON ARCHITECTURAL EDUCATION

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ABSTRACT Globally increasing prosperity brings about growing consequence o in recent years, by decreasing the effects of earthquake hazards, well organized teams against earthquakes and earthquake resistant building design sensitive on society. The most important step of earthquake resistant building design is settled on during architectural design. One of the most important phases of architectural design is to identify structural system of the building. The structural system of building is decided by architects. The earthquake fact should always be taken into consideration due to building design in countries located on active seismic zones. It is known that, many collapsed or damaged buildings are recognized by their unsuitable structural system created during the architectural design stage. Such design errors usually can not be fixed by engineering calculations. In countries located on high seismic zones similar to Turkey, earthquake codes and specifications should be improved continuously. In addition to these legal regulations about earthquake resistant building design and construction, special attentions should be cared on architectural and engineering education. Earthquake resistant building design concepts should be taught to students of architecture considering the fact that "Earthquake is a reality of Turkey" instead of common criteria and examples. Curriculum of several architectural schools in Europe and Turkey, do not include compulsory "Earthquake Resistant Building Design" courses. As well as Gazi University Department of Architecture, a few school of architecture offer earthquake resistant building design elective courses in their curriculum. By questioning present circumstances of architectural education, the statement that "What should be done to provide more conscious and equipped earthquake resistant building design education" should be discussed. Earthquake resistant building design courses should take place as compulsory courses in the curriculum of schools of architecture. These courses should be adjoined to design projects and various visual materials, physical models must support to increase the students' perception of true structural behavior. Most of architectural students are visual learners, therefore teaching techniques including visual expression are the correct choice for architectural students. Graphical simulations of structural analysis software and simple physical models prepared by wood or metal pieces may be the best alternative for this teaching method. Consequently, architectural students appropriate to earthquake resistant building design theory will gain ability of the assessment and interpretation of the behavior of structural systems. In this study, the importance of earthquake resistant building design during the architectural education has been reminded. The weight of earthquake resistant building design concepts taken into account in architectural schools of Turkey is explored. Innovative teaching tools are proposed which provides improved earthquake resistance building design concepts to architectural students.

TURKEY AND EARTHQUAKES

Mankind has faced challenges of nature (eg. earthquake, flood, hurricane) many times so far. Most suffering loses recorded have been resulted from earthquakes.

Most of our country's territory is located on one of the world's most active seismic zones. For 58 years, 58 202 citizens have died and 122 096 have injured because of earthquakes in Turkey. Moreover nearly 411 465 buildings have collapsed or been heavily damaged. Briefly, 1003 citizens die and 7 094 buildings collapse per year in Turkey [1].

The casualties and damages of earthquakes are mainly caused by human faults. Because earthquakes have been happening as a natural phenomenon since the constitution of earth and they turn into "catastrophes" because of deficiencies and faults of humans. So it is very important to analyze the faults of mankind in order not to do them again. Human faults can be corrected but natural movements cannot be impeded (Bademli, 2002).

ARCHITECT AND EARTHQUAKES

When scientific researches and experiences regarding earthquakes are considered, it is concluded that one of the most important steps in building earthquake resistance structures is "architectural design". One of the most important steps of architectural design is to determine structural system of the building. The matter of earthquake should always be taken into consideration due to building design in countries located on active seismic zones such as Turkey. So architects should be aware of the fact that they are to pay attention to building earthquake resistance structures. Also they should always remember that earthquake is a fact of Turkey and it should be taken into consideration when building structures. However, it is a certain characteristic of Turkish nation to forget bad experiences. It is necessary to reverse this characteristic by means of education.

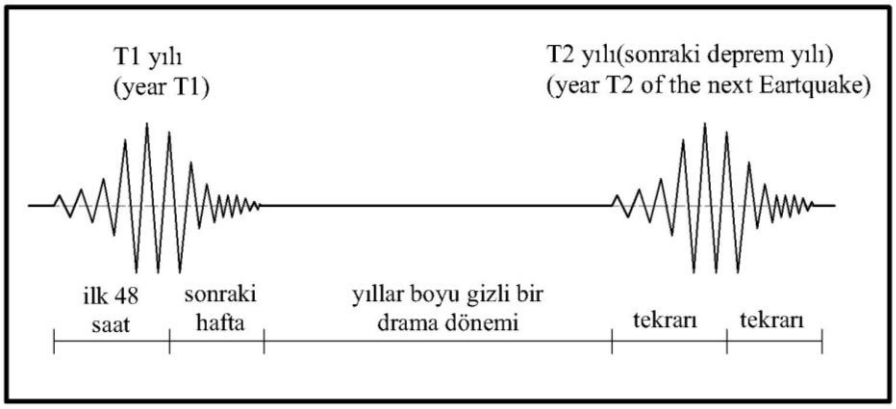

Fig. 1. The improvement of public awareness for earthquake by time (Karaesmen, 2002).

Earthquake happened in Turkey have displayed defects of structural system by doing big damages. When the structures collapsed during earthquakes are analyzed, it can be seen that architectural designs don't fulfill the criteria of earthquake resistance and structural system is forced. Moreover use of wrong and defect equipment and faults of structure and calculation should be considered. While the science and technology can reduce casualties of earthquakes in today world, deficiency of Turkey originates from lack of education.

Knowledge is the vital necessity for fight against earthquake. Main goal of study is to display the deficiency of "awareness of earthquakes" in architecture education and to search for the ways of solution.

CONCEPT OF EARTHQUAKE IN ARCHITECTURAL EDUCATION

Architectural education is a multidisciplinary which includes many various branches of science and art. The main goal of architecture is to construct the most convenient artificial environment for mankind. Architectural education process is based on examining and producing new solutions. The aim of theoretical and practical lessons is increase the quality of work produced according to design. Architectural design is not limited to only construct of location and design of building. Designer should be aware of all necessities that are related to the society which he serves. Also earthquake is a phenomenon that cannot be ignored.

THE PLACE OF EARTHQUAKE IN ARCHITECTURAL EDUCATION IN TURKEY

Recent earthquakes have shown us that it is crucial to take earthquake fact into consideration when building structures in countries located on active seismic zones like Turkey.

It is usually impossible for engineers to correct the deficiencies of structures not designed properly as earthquake resistant by architects and the structural system of buildings not designed with a proper plan for earthquakes. Architectural design is an important phase in designing earthquake resistance structures since the construct of building is made clear at the phase of architectural design and general principles are determined by architects. So it is significant the architect should take the fact of earthquake and its effects on structure into consideration.

After 1999 earthquake period, interest on lessons of "earthquake resistance building design" increased but these lessons were mostly given at graduate schools and in undergraduate schools-as optional lessons. It can be understood that not all of students are aware of these knowledge but only a part (Taş ve diğ., 2004).

In this study, schools giving architectural education in Turkey and TRNC are examined according to 2009 schedule. 6 out of 41 architecture departments couldn't be available for search. However, data obtained related to 35 architecture departments are displayed below:

Form 1. The universities whose schedule available "Earthquake Resistance Building Design" lesson.

LINE	UNIVERSITY	THE LESSON NAME AND CODE	THE STATUS OF LESSON		
			GRADUATE		UNDER GRADUATE
			MANDATORY	OPTIONAL	
1	Abant İzzet Baysal University	Deprem Mimarlığı		**x**	
2	Bahçeşehir University	ARCH 4912 Seismic Design		**x**	
3	Balıkesir University	MSD 2254 Depr. Day. Yapi Tas		**x**	
4	Çukurova University	MM 546 Depreme Dayanıklı Yapı Tasıyıcı Sistem Tasarımı İlkeleri			**x**

5	Dicle University	MİMS 330 Depreme Dayanıklı Yapı Tasarımı		x	
		MİM 525Depreme Dayanıklı Yapı Tasarımı İlkeleri		x	
6	Dokuz Eylül University	MİM 392 Depreme Karsı Yapıların Mimari Tasarım İlkeleri			x
7	Doğu Akdeniz University	ARCH 331 Tectonıcs Of Form Resıstant Structures		x	
8	Gazi University	M 411 Depreme Dayanıklı Yapı Tasarımı	x		
		MIM 624 Depreme Dayanıklı Yapı Tasarımı			x
9	Gebze Yüksek Teknoloji Enstitüsü	MIM 566 Depreme Dayanıklı Yapı Tasarımı İlkeleri			x
10	İstanbul Kültür University	ARC 003 Depreme Dayanıklı Yapı Tasarımı		x	
11	İstanbul Teknik University	MIM 388 Deprem Mühendisliği İlkeleri		x	
12	İzmir Ekonomi University	ARCH 411 Afete Dayanıklı Yapı Tasarımı		x	
13	İzmir Yüksek Teknoloji University	AR 434 Earthquakes and Structures (Depremler ve Yapılar)		x	
		AR 634 Taşıyıcı Sistemlerin Deprem Dayanımı			x
14	Mimar Sinan Güzel Sanatlar University	MIM 754 Depreme Dayanıklı Binaların Yapısal Oluşumu		x	
		MIM 654 Depreme Dayanıklı Yapı Tasarımı		x	
15	Orta Doğu teknik University	BS 535 Earthquake-resistant Building Design in Architecture (Mimarlıkta Depreme Dayanıklı Yapı Tasarımı)		x	

16	Uludağ University	MIM 3069 Depreme Dayanıklı Yapı Tasarımı		x	
17	Yakın Doğu University	Depreme Dayanıklı Yapılar		x	
18	Yaşar University	ARCH 457 Architecture and Earthquakes (Mimarlık ve Depremler)		x	
19	Yeditepe University	ARCH 567 Earthquake Resistant Architectural Design (Depreme Dayanıklı Bina Tasarımı)		x	
20	Yıldız Teknik University	MİM 3496 Tasarımda Deprem Faktörü		x	
		MİM 6419 Depreme Dayanıklı Yapı tasarımı			x

Form 2. The universities whose schedule doesn't include "Earthquake Resistance Building Design" lesson.

LINE	UNIVERSITY	THE LESSON NAME AND CODE
1	Anadolu University	NOT AVAILABLE
2	Beykent University	
3	Bozok University	
4	Erciyes University	
5	Haliç University	
6	İstanbul Bilgi University	
7	Karadeniz Teknik University	
8	Kocaeli University	
9	Lefke Avrupa University	
10	Maltepe University	
11	Mersin University	
12	Selçuk University	
13	Süleyman Demirel University	
14	Trakya Üniversitesi	
15	Uluslar arası Kıbrıs Üniversitesi	

Form 3. The universities whose schedules couldn't be reached.

IINE	UNIVERSITY	THE LESSON NAME AND CODE
1	Akdeniz University	COULDN'T BE REACHED
2	Eskişehir Osman Gazi University	
3	Girne Amerikan University	
4	Karabük University	
5	Niğde University	
6	Yüzüncü Yıl University	

When we look at 20 architecture departments giving this lesson: 15% of departments give this lesson to both undergraduate and graduate students and 70% give it only to undergraduate while the remaining 15 % give this lesson only to graduate students.

When we handle the situation of 35 architecture departments by considering it is important to give these lessons in undergraduate classes, only 2,86% of the departments give this lesson as mandatory and 54,28% of the departments give the lesson as optional while there cannot be seen this lesson in schedules of remaining 42,86%.

Consequently, just senior classes of Architecture Department of Faculty of Engineering and Architecture-Gazi University's schedule include "Earthquake Resistance Building Design" lesson as mandatory. It was also seen that a new graduate program on "Housing and Earthquake" started at Istanbul Technical University in 2001–2002 as non-thesis master program. This program aims at fulfilling the shortage of studies conducted regarding earthquake and housing in a disaster event and lack of knowledge. Also gathering knowledge about the subject and educating students of department about this subject are among the goals (Gülersoy ve diğ., 2002).

THE PLACE OF EARTHQUAKE IN ARCHITECTURAL EDUCATION IN USA

Architectural undergraduate and graduate programs in USA differs due to the various education systems of schools. Architectural education in universities are given in different structures such as Departments of Architecture, Schools of Architecture, Schools of Design, Colleges of Architecture. So each of them emphasize design, structural system knowledge and history of architecture at different levels. For instance, the affairs regarding earthquake are widely handled in University of South California while there is no place for earthquake in School of Design, Harvard University. This is because the west coastline of USA, which includes San Andreas Fault Line, is a seismic zone.

THE PLACE OF EARTHQUAKE IN ARCHITECTURAL EDUCATION IN EUROPEAN COUNTRIES

Most susceptible areas to earthquake in Europe are located on the basin of Mediterranean and Black Sea and along the active fault line between Eurasia and Africa continents (Greece, Italy and Eastern Balkans). In our research, we couldn't get certain data regarding education of earthquake resistance building design in Schools of Architecture in Europea. This subject is studied as an engineering lesson there.

ADVICES APPLICABLE ON EDUCATION

It's known that, in comparison to industraliazed countries, funds allocated for scientific research and technological development in Turkey is relatively low (Ulusal Deprem Konseyi, 2002). In Turkey, having the high risks of eartthquake danger as a whole, sensitiveness about architecture and engineering education is important as well as, continual development of earthquake regulations, earthquake resistant structure design and production.

In California, all architecture students have to be associated with earthquake resistant principles (State Seismic Safety Commission, 1991). Yet, in theoric courses, showing the former and latter videos about an earthquake and mentioning about earthquake are studied. In architecture syllabus, earthquake resistant structure courses must be among the required courses apart from engineering departments; it must be conducted not only theorically but also by using implementation projects and visual aids.

In architecture departments in Turkey, it is necessary to put the emphasize on consideration of earthquake resistant structures and interdisciplinary study skills, taking architecture design into consideration, structure frame must be associated with earthquake resistant as well as having the esthetic point of view. Utilizing the technology about earthquake resistant design studies has led to successful conclusions. Using structural analysis computer programs in bachelor and master education, it enables us to make synthesis by evaluating the data within (tools, system length, balance). In addition to bachelor and master education, to form earthquake consciousness and studio studies have an important role.having deep knowledge about earthquake resistant structures, is the first step of sanitary structure production.

Up to now, some studio studies are like folllowing:

1. ` 5 minutes to survive, years to recover2
2. UIA Summer School 2004. architects and disasters.
3. earthquake architecture studio program are successful studies.

CONCLUSIONS

Architecture is a solution based profession. In architecture education, technology, new concepts, and new perceptions must be evaluated, in a fast changing world, deficiencies must be resolved. Earthquake regulations must be developed, implementation of earthquake resistant structure design production regulations is important as well as sensitiveness on architecture education. One of the most important steps of architecture is conveyor system and it is determined by architectures as it is in architectural design.

Lastly education about earthquake resistant structure mustn't be limited to universities. Cooperation with media having the public's conscousness by educating the media staff is a vital part of the education.

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THE STRUCTURAL ANALYSES AND REINFORCEMENT STUDIES OF BURSA İNEBEY (EGNE BEG) MADRASAH

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Key Words: Historical monuments, reinforcement, structural analyses.

ABSTRACT The researcher of this paper want to understand the current state of the madrasah by using mechanical, physical and NDT (Non Destructive Tests) in-situ & in laboratory for the characterization of the materials used in the Madrasah. After the tests material properties are calculated for the composite materials which was used stone-mortar at walls and brick-mortar at vaults, domes and transition elements. The composite materials properties are adopted to the curent in situ model under the death and earthquake loads and current state of the madrasa is analysed. Acording to the results of the analyses the restoration of the madrasa and reinforcement studies are determined.

INTRODUCTION

The Inebey Madrasa is dated to the end of the 14th century in the first capital of the Ottoman Empire. Bursa had many commercial, educational, public and religious buildings between 14th and 15th centuries. Madrasa was built in the period of Sultan Yıldırım Bayezid I (1389-1402) by Subaşı Eyne Beg. But the exact construction date of the Madrasa is not known.

The aims of the study were the following:

a- To understand the characterisation of materials used in the madrasah by using physical, mechanical and Non Destructive Tests.

b- To perform the structural analyses of the Madrasah.

c- To determine the restoration and reinforcement studies of the Madrasah according to the structural analyses.

THEORY and METHOD

Inebey Madrasah is in Osmangazi District on the İnebey Street. It is used as a library of Hand Painted and Old Printed Works. The library belongs to the General Adminstration of Waqfs. The historical building is surrounded by İnebey Street at the north and Bükük (Ebuşahme) Street at the south. İnebey Hamam was built with the madrasah belonging to the İnebey Kulliye. Hamam is on the west side and Driving Course is on the east side of the building. According to the sources it is known that the wall close to the bath was opened 75 cm.in width during the 17th century because of the moisture caused by the Hamam.

According to a 1093 H. (1683 M) Ottoman Court Archives –Şeriye Sicili-it was restored in the 17th century and in 1965 by General Adminstration of Waqfs of Adminstration of Bursa Region.

It is reached to the madrasah by stairs from the İnebey Street. The entrance is from a rounded arched door under a pointed arched niche. The courtyard in the middle having dimension of 8.5x10 m. is surrounded by archades besides the rooms. The rooms are

situated on the east and west side, the classroom on the north, the entrance and the library are at the South direction (Fig.1).

Open madrasah with an iwan planned building is one floored except the library part. It is 32 m. on north-south direction and 24 m. on the east-west direction.

There are 12 pillars around the open courtyard having a dimension of 65 cm. with a hexagonal form. The pillars are connected to each other by pointed arches and iron tie bars between the arches and the rooms with the dimension of 4x4 cm. in thickness.

Rooms and archades are barrel vaulted except the front of the classroom's door and the front of the entranc eof madrasa in the archades.

The building was built with rubble stone, limestone, mortar, brick and timber in the masonry system (Fig.2,3).

The outer walls are 116 cm in thickness. Upper part of the pointed arches are surrounded by 12 piers. There are chain, water wave and hook motifed brick rows on the limestone are used between the brick rows. The walls are finished by five rowed saw-tooth eaves.

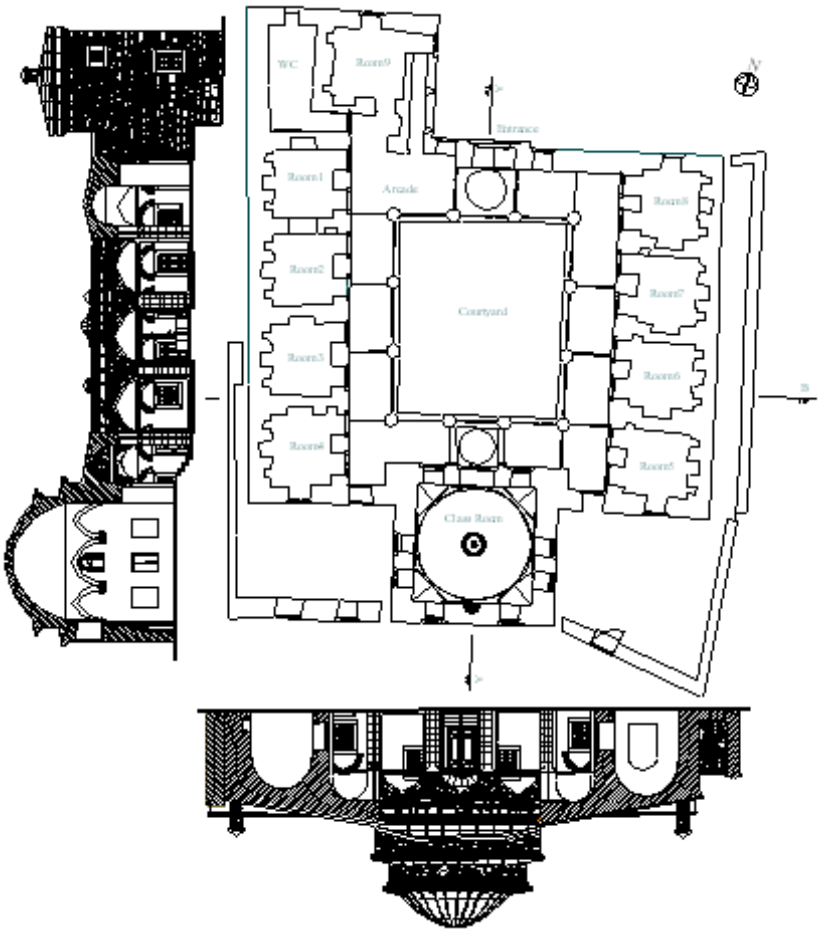

Fig.1. Analytical survey of plan and the two sections of the Madrasah.

Fig. 2. The North facade of the Madrasah.

Fig. 3. General view from the roof looking to the Courtyard.

The roof of the Madrasah had interventions several times by filling materials between and above the vaults (Fig. 4,5). The filling materials are separate layers which were consist of different materials such as clinker, limestone, brick pieces, concrete and reinforced concrete. This load was calculated 700 ton for 400m2 after taking off the filling material on and above the vaults. It was cleared that the roof of the Madrasah had loaded anormously which was 1,75ton/m2 in a period of time.

Fig.4. Different materials filling between and above the vaults.

Fig. 5. Layers of filling materials on vaults.

For the assesment of material properties of the madrasah, Non Destructive Test (NDT) Methods applied in situ. Tests are applied on walls and piers in order to find the mechanical properties of materials used in the Madrasah. Samples are also taken, cut and dried in an oven for physical assesments. Table 1 shows the physical and mechanical properties of the composites used in Madrasah.

Table 1. Mechanical and physical properties of Composites.

	Rubblestone-Mortar Composite	Brick-Mortar Composite
Compressive Strength	15,85 N/mm2	4,96 N/mm2
Tensile Strength	1,585 N/mm2	0,496 N/mm2
Modulus of Elasticity	15853 N/mm2	4963 N/mm2
Shear Modulus	6341 N/mm2	1985 N/mm2
Poisson's ratio	0,17	0,17
Unit Weight	23 N/mm3	16 N/mm3

Roof load was applicated to the finite element model which was prepared by using the SAP2000 program and the cracks of the Madrasah are investigated. Also modal analses were conducted to understand the behaviour of Madrasah under dynamic loads. The model of the Madrasah was consist of 6777 shell and 28 frame elements (Fig 6).

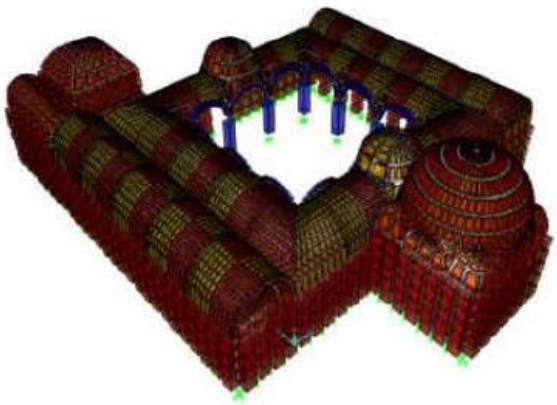

Fig. 6. Finite Element Model of the Madrasa.

The filling load on the vaults are causes tensile strengths on the edge of the vaults maximum 0,3-0,4 N/mm² but not exceeded the limit tensile strength of 0,496 N/mm² as shown in Figure 7,8 under dead loads. In-situ cracks seen irregularly on this area.

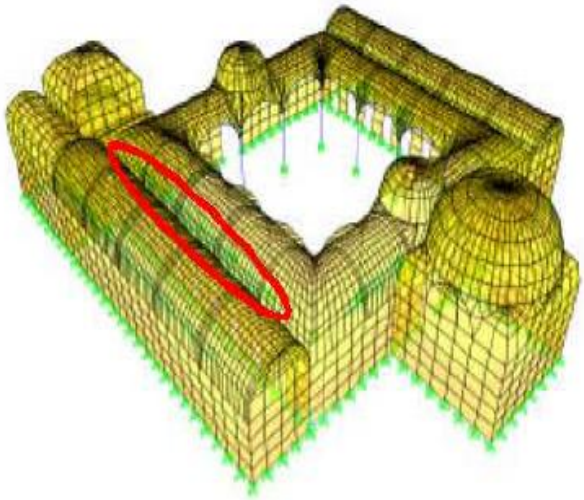

Fig. 7. Tensile stress areas between the vaults.

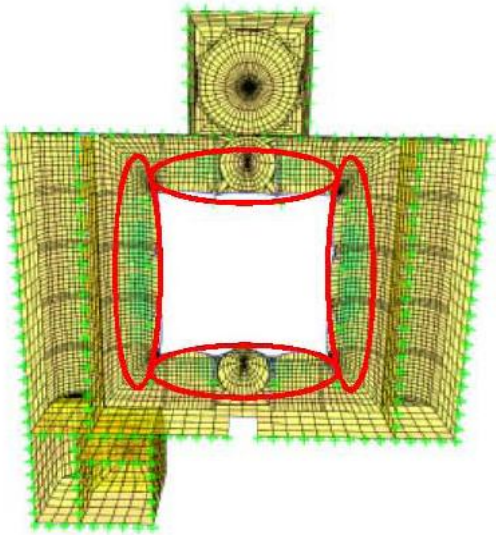

Fig. 8. Tensile stress areas under the vaults.

Under the vaults it was seen in the analyse program that the maximum tensile stresses has reached 0,43 N/mm2 which was close to the 0,496 N/mm² limit tensile stress value. In-situ there were main and regular cracks observed on this area which were same results in the analyses programme. Figure 9 and 10 shows the North-south direction, Figure 11 and 12 shows the east-west directions.

Fig. 9. In-situ cracks under vaults in North-South direction.

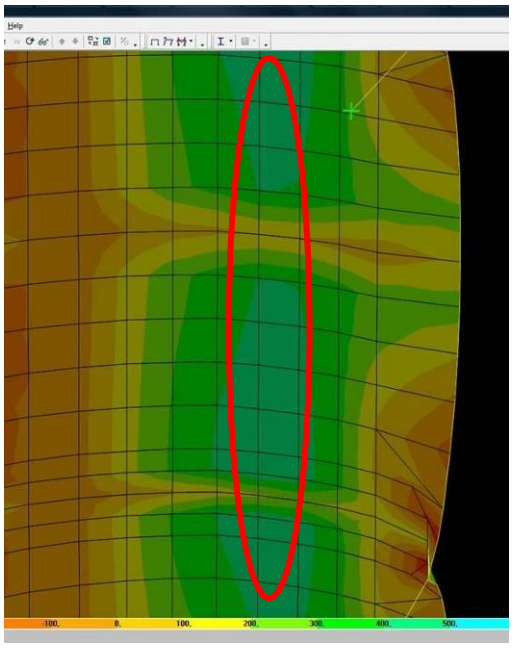

Fig. 10. Tensile stress areas under vaults in North-South direction.

Fig. 11. In-situ cracks under vaults in East-West direction.

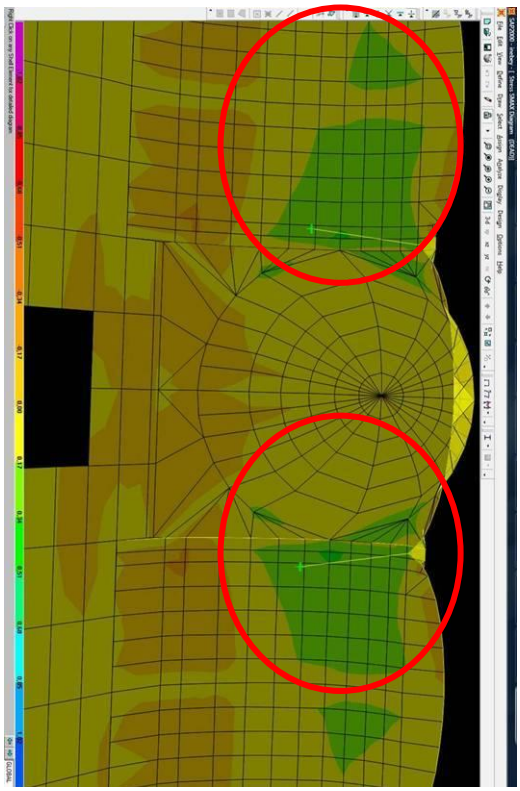

Fig. 12. Tensile stress areas under vaults in East-West direction.

The earthquake analysis was conducted using response spectrum approach. Madrasah is located on eartquake Zone 1, the most seismically risky regions in Turkey. In determining the earthquake loads, "Turkish Eartquake Code, 2007, Section 5" is used. Ground spectral accelaration coefficient was taken 0,4. The soil conditions were determined siltly clay material which equal to ground type Z2 and the characteristic spectrum periods were Ta=0.15sec. and Tb=0.40sec.

Tensile stresses were developed under the dead+earthquake loads between 4 N/mm2–7N/mm2 which was over then the tensile strength of the brick-mortar composite metarials used in superstructures and transition elements. In-situ continuous big cracks were observed under and centre of the vaults. Tensile stresses were reached around 7~9N/mm2 where large deformations on the edge of the vaults and the domes are seen.

Analyses results shows that the tensile stresses are exceeded limit values under earthquake loads on vaults and domes edges, under and center of the vaults and connection points of the vaults.

For decreasing the tensile stresses affects, the maximum tensile stresses areas which were also cracked areas covered with the FRP (fibre reinforced polymer) materials for strengtening the structure (Fig 13-16) FRP materials used under and edges of the vaults in 60cm width. CFRP (carbon fibre reinforced polymer) materials are used between coulumns and walls transfered the stresses to the walls and coulumns which are occured under the vaullts.

Fig. 13. FRP aplications at edge of the vaults.

Fig. 14. FRP aplications under the vaults.

Fig. 15. FRP aplications under the vaults.

Fig. 16. CFRP aplications between the coulumns and walls.

All cracks and gaps filled with the adequate injection material (Fig 17,20). After the reinforcement studies the injection material applied above the vaults, domes and edge of the vaults.

Fig. 17. Application of injection materials.

Fig. 18. Application of injection materials above the vaults.

Fig. 19. Application of injection materials above the domes.

Fig. 20. Application of injection materials edge of the vaults.

The roof system was completely changed by constructed the new wooden roof system (Fig. 21-24) respect to the safety of the structure. The new wooden roof structure is supperted on walls and coulumns with steel plates. Wooden trusses are arranged between the vaults.

Fig. 21. Supporting points for the wooden construction.

Fig. 22. Supporting points and the wooden construction.

Fig. 23. Perspective view of new wooden roof system.

Fig. 24. Wooden trusses between the vaults.

Also existing tie bars which were exposed to the corrosion and strengthened, timber beam gaps fulfill by the new timber materials.

CONCLUSIONS

The historical İnebey Madrasah is analysed by using SAP2000 program to determine the structural behaviour under the self-weight and earthquake loading. The existing material characteristics were found by physical and mechanical tests with the NDT Methods and laboratory tests. The filling materials weight above vaults and composite material characteristics adopted to the finite element model for structural analyses. Analyses results shows that the tensile stresses are exceeded limit values under earthquake loads on vaults and domes edges, under and center of the vaults and connection points of the vaults. For decreasing the tensile stresses affects, the heavy load on the vaults were removed and the maximum tensile stresses areas which were also cracked areas covered with the FRP (fibre reinforced polymer) materials for strengtening the structure. All cracks and gaps filled with the adequate injection material after the reinforcement studies. The injection material applied above the vaults, domes and edge of the vaults. The roof system was completely changed by constructed the new light wooden roof system. Also existing tie bars which were exposed to the corrosion cleaned and strengthened, timber beam gaps fulfill by the new timber materials during the restoration works, with the respect to the safety of the structure.

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UNSUITTABLE SETTLEMENT POLICIES IN EARTHUAKE RISKY CITIES: A CASE THROUGH İZMİR-KADİFEKALE LANDSLIDE REGION

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Key Words: Natural Hazard Planning, Landslide, Rock fall, Izmir- Kadifekale

ABSTRACT Many academically and practically study for 1999 Earthquake known in literature as "17 August Earthquake" has been made in Turkey, but reflections on the application of those studies couldn't go beyond a method of wound wrapping on the main editing for earthquake.

For this reason, neither policies including landslide, flood, and avalanche fall etc. separated from earthquake could be made, nor could comprehensive training on pre disaster, during disaster and after disaster be developed. Even though earthquake is at the centre of disaster prevention in Turkey, other possibilities in regards to geographical and climatic differences (avalanche possibilities in Eastern regions) or other inter activated disaster types (landslides after earthquake etc) are also a reality in Turkey.

Within the scope of this study, the İzmir City disaster-risk magnitude were revealed through the risk areas identification for Izmir and earthquake, landslide, rock fall and flood types of natural disasters in the landslide phenomenon were focused on. Konak-Kadifekale is of great importance for İzmir in the historical, social, cultural and geographical sense and Kadifekale is a sample of the city for the process of historical development within the urban erroneous policies that are questioned today, Kadifekale landslide area came to the last point of the concept of urban transformation is examined in this context.

INTRODUCTION

The earth is a highly dynamic planet, and most of the natural events show a wide range of variation through the time energy and material of environmental process. The extreme natural events are not considered hazards unless they cause death or damage to humans. A severe earthquake in a remote, unpopulated region is an extreme natural event of interest to seismologists, and no more.

Urban disaster risks and vulnerabilities are great problems for Turkey. The annual loss of life and property through disaster in the world's major metropolitan areas is increasing. Urban concentrations of the poor and less informed in environmentally fragile locations suffer the impact of disaster disproportionately. For example, the continued occupation of vulnerable locations in Turkey's metropolitan areas by law income Gecekondu developments will compound the inherent risks associated with high-density environments, in appropriate technologies, and inadequate infrastructure.

There are serious natural hazard risks in İzmir, which is a metropolis and third largest city of Turkey. Flood, earthquake, landslide and rock fall hazards have damaged to İzmir built up zone many times in the past. Especially, earthquake risk increases the hazard probability. But the competent authority cannot take main measures and precautions.

Izmir survived as a big city throughout its history of 5000 years and has been frequently renovated under geopolitical and geological influences. Izmir has been greatly affected by some disasters such as earthquakes, fires, epidemics and etc. Thus many edifices that would reflect historical background of the city did not survive until today and present

remains are generally few and known only by experts and the neighboring people (Local Agenda 21 in Izmir, 2003).

Natural environmental features of Izmir increase its natural hazard risks. Izmir has topography slopes that surround the city shape. Furthermore, soil geology is unsuitable for to settle down in built up area. On the other hand natural hazard risks increase because of the spread of the illegal urban settlements and the build feeble building types.

Geology, slope, rainfall and earthquake are reasons of to landslides. All of them are different importance for landslide risks in built up areas.

LANDSLIDES AND GEOLOGY

The relationship between landslides and soil; the most important factors related to the landslides are material properties, for example litlogy of soil, porosity, permeability, carbonate cement and soil mass properties such as stratified or massive character, fill material type, weathering grade, aperture, discontinuities and the length of the apertures. Additionally, types of soil and evaluation of the field observations are important elements.

The formations composed by such soils are effected by such properties and different landslides may occur. The change of seasons, climates, temperature, groundwater, erosion, heavy loading also help mass movements. Discontinuities of soil are one of the most important factors initiating landslides. Discontinuities of structure are second important factor for landslides.

LANDSLIDES AND PRECIPITATION

Rainfall is generally accepted as one of the chief factors controlling the frequency of landslides. The magnitude of its influence depends on climatic conditions, on the topography of the area, the geological structure of slopes, and the permeability and other properties of rocks and soils.

Slope movements generally occur in two periods of the year: in spring after thawing and in summer after heavy rains. A detailed analysis of sliding events reveals differences in the sensitivity of slopes to the effect of rainfall. The interval between the commencement of above-normal rainfall and the start of slope movement varies according to the permeability of the surface rocks and the type and form of the landslide induced (Avşar, 1997).

LANDSLIDES AND EARTHQUAKE

Especially, central European geologists have somewhat underrated the role of earthquakes in landslide phenomena, probably because there are practically no major earthquakes in these countries. During earthquakes with intensities higher than VIII (Intensity) according to the international MSC (magnitude scale), particularly that occurring in mountain regions, landslides and rock falls cause the largest damage to property and the greatest loss of life. (Zaruba, 1982)

The controlling factors of the rockslide were the local geological and morphological conditions and the large input of kinetic energy from the earthquake. In seismically active regions earthquakes are the predominant cause of slope failures. Seismically induced landslides occur in young tectonic mountain ranges and along active faults, particularly along the mobile marginal zones of continental blocks.

The main factor in provoking slope movements is the intensity of the shocks; it has been established that earthquakes of magnitude 6.5 cause large slope failures in all landslide-prone areas. Many fossil rockslides and rock falls in areas, which are now seismically

quiescent, show the characteristics of earthquake- triggered slope movements. Ancient earthquakes are very important problem for them.

METHODOLOGY

To minimize natural hazard risks is the planning conception is concretely discussed in methodological framework. Urban-scaled regional physical plans, land use plans, protection and improvement projects for old construction areas and new techniques are assured by setting relations with new techniques and natural hazard concept.

Development and Regional Plans: Basic principles for diminishing natural hazard effects consist of balanced allocation of the population, economical operations and avoiding agglomeration in certain regions, creating reliable environments, bearable for living. This is the reason that distorted urbanization is prevented. In other words, country leveled decisions be taken. National sources are researched and a relation brought up between economical and physical events. Local physical plans are supported with regional ones and consolidated. Crowding movements in metropolis, decrease of agricultural fields and constructing buildings in unfavorable alluvium lands, are facts, which increase natural hazard risks. Consequently, regional geology maps are determine inconvenient and natural hazard risky areas. Regional planning projects, which depend on mentioned regional geology maps, must be compulsory.

Sub- Regional Plans and Metropolitan Plans: Basic problem is that the necessity of making metropolitan plans according to the country and regional plans goes to a dead end from the very beginning, due to a lack of regional plans. A fact affecting the most natural hazard risks in metropolitan areas is the matter or borders of municipalities determined by the law nr. 5216. Although actual physical development area of metropolitan municipalities, today there is no one responsible and authoritative organization, which provides coordination between different municipalities and prepares metropolitan physical plans.

Due to shortages in laws, metropolitan municipality being unable to make changes in borders, controls and coordination around border areas are not provided, which results in broken, disordered situations. This of course produces uncontrolled and uninspected problems in respect of natural hazard effects. Insufficient organization level of the Ministry of Development and Reconstruction, responsible of making metropolitan plans, is one of reasons of non implementation of this process, too.

Local Implementation Plans: Local Implementation Plan as physical plans are known as basic physical plan in our country. Fixing the areas having natural hazard risky during planning process and limitative regionalization by these plans is quite possible. Compatibility between macro scaled plans, micro scaled plans and physical site are assured, in order to reach a reliable physical building and structure. Lack of relation between planning levels, missing of new strategies in planning process for preventing natural hazard effects, supervision, are the most important problems in existing physical planning practice.

EXAMPLES: IZMIR-KADIFEKALE LANDSLIDE AREA

Landslides are the most important mass movements over a widespread area in the Izmir built up area. This natural hazard may be extremely costly in destroyed regions even, they may cause lost of life. Especially, after the major earthquakes, a lot of landslide occurred in Izmir built up areas and caused big damage. So geological and topographical structure of Izmir are very risky for mass movement.

THE LANDSLIDE AREAS OF SOUTH SIDE OF THE IZMIR GULF

Landslide areas are classified into two parts in Izmir built up zone. First part is the north of Izmir Gulf, and the second part is the south of the Izmir Gulf. Balçova hills, Kadifekale

backs and Altındağ regions are dangerous areas in south of the Izmir Gulf. Uzundere – Kadifekale regions are known more dangerous than the others (Avşar, 1997).

The cretaceous detritics in the South of Izmir Gulf outcrop in the South line of Balçova-Güzelbahçe. The types and extent of landslides are quite different from the North of Gulf. The landslides in this district have taken place as circular, plain or wedge failures, which are observed as soil creeps, debris creeps and lateral in the South of Izmir Gulf outcrop in the eastern slope of Balçova. There are a lot of soils and debris flows developed and cut each other on the fossil landslide between Uzundere and Emreş hill. The landslides having plane and circular failures in Cretaceous detritics are noted in the northern slope of Teleferik. Narlıdere- Özmavikent site and Narkent site regions are active areas too (see appendix 1-2-3), (Working Group of Landslides- MTA Izmir Directorate, 2005).

Table 1. **Major Landslide and Rock fall Areas of İzmir Built up Zone (Ministry of Public Work and Settlement General Directorate of Izmir, 2005).**

No	Region	Damage Buildings	Natural Hazards type
1	Cigli- guzeltepe	440	Landslide
2	Cigli- Ornekkoy	18	Rock fall
3	Karsıyaka- Ornekkoy	50	Rock fall
4	Bayrakli- Cıcekkoy	20	Rock fall
5	Buca- Sakarya	44	Rock fall
6	Konak- Kocakapi	28	Rock fall
7	Konak- Gurcesme	10	Landslide
8	Asansor	54	Rock fall
9	Kadifekale	3162	Landslide
10	Altindag- Merkez	62	Landslide
11	Altindag- Kuyu	11	Landslide
12	Altindag- Camdibi	13	Landslide
13	Hakimiyeti Milliye	64	Landslide
14	Narlidere- Narkent	800	Landslide

Neogene volcanics are seen over the older than the other units in Kadifekale and its surroundings in these units. The circular types sliding have occurred in Kadifekale and its surroundings as lahars, debris flows and creeps.

Fossil, active and semi active landslides are taking place either circular or planer failures in the metropolitan area. Mass movements are more common in Neogene and Cretaceous detritics than volcanics. The lithological and structural properties effect the forming of the sliding surface in soil. Landslides occurred in the metropolitan area are generally complex structures. Flows and sliding cut off each other in the slide topography. They generally are observed on the fault zones and they're near surroundings or along the sharp and long valleys.

KADIFEKALE CASE AREA

Landslides and rock falls often occurred in Kadifekale and its surrounding area since 1960. In spite of the facts that, landslide areas were seen and were proposed as green areas in 1923 plans, these areas were determined as the development area in 1965 by the Town Council of Izmir. Mr. Erwin Lahan, who was a famous geologist, said the same problem for these parcels on 16.5.1950.

Fig. 1. The existing implementation plan and existing land use plan of Kadifekale landslide area.

Then the all hazard, reports (1959, 1962, 1977, 1980, 1981, 1990, 1992, 1996, and 1997) were prepared by the Ministry of Public Work and Settlement and new proposals were developed (see fig. 2). But always-political aspect of the problem has been important for local politicians. According to 2000 researches, the population of this region was 32.000 and politically this population level and vote of the residents were very important (see fig. 3- 4).

Fig. 2. The view of Kadifekale landslide area.

Fig. 3. The detail street views of Kadifekale landslide area.

Major points of these mentioned reports are summaried below:

26.10.1959 Dated Report: Detailed researches and analysis were realized in Kadifekale and then Mr. Mehmet Taşdemiroğlu who was a geologist of Ministry of Public Work and Settlement saw landslide areas. He reported that, "landslide hazard can be occurred in Kadifekale, for this reason, the problem areas which are at the south, north and east ridges of the Kadifekale, should be planned as a green areas." In spite of is serious warning, this region was not determined as a natural hazard zone and as a green area.

5.11.1962 Dated Report: In Kadifekale and Gürçeşme regions, landslides were watched over and the announcement of unavailability of region for building purposes were suggested by Mr. Mehmet Taşdemiroğlu again, but competent authorities had not take any measures. For this reason, constructions of squatters continued around the Kadifekale.

29.1.1977 Dated Report: In 1977, landslide occurred more seriously than the formers and new report was prepared. This report suggested that, 174 houses should be transported to the different regions and also this area should be decided as a natural hazard area. Moreover, 998, 1016, 989 and Rakım El Kutlu streets of north of the Altay district should be controlled. In addition, the border of landslide are of Kadifekale was determined in this report.

In that time, the Chamber of Civil Engineering office of Izmir published the proclamation on 21.3.1977. According to that report, "As now as, technique data related with landslide area have not been considered and build up areas of squatters have not been prevented by Ministry of Public Work and Settlement and Metropolitan Municipality. There are 1500 houses in Kadifekale and 500 of them are heavy damaged nowadays. The inhabitants are left living for their own fate, therefore, it seems that only the die off will stimulate the Ministries to take precaution. In our opinion, immediately, these areas should be decided as natural hazard areas, the habitants should be transported to different regions, and the new measures of build up area should be explained to habitants.

Because of this suggestions and reports, Kadifekale was decided a natural hazard region in the law, which is a number of 15319, on 23.2.1977 and Buca- Prevention Area, was separated as a new build up area for habitants.

26.5.1981 Dated Report: The damage effects of landslide in Kadifekale were researched and reported on between 8 and 25 April 1981. The report said that, there were 1116 heavy damaged houses, 134 middle damaged houses, 313 little damaged houses and 1959 strong houses in this region. 728 families were determined to have right to houses, and 382 families took new buildings. 346 families were examined once as much, so 197 of them got new buildings.

Although, the people got a new buildings, the authorities of municipality did not take a licenses/title-deeds of habitants who lived in Kadifekale (Politic worries caused this implementation). Therefore those plot owners did not stop the build new houses.

In 1992, DEU made a research in Kadifekale, they suggested that, "These areas should be natural hazard risky areas, subterranean waters cause the landslides. The people could not take permission for the buildup areas in active regions. According to that research, Topaltı area, HacıAli Efendi area, Akarcalı district, Yeşiltepe area, Yeşildere area were landslide areas in Kadifekale.

27.6.1997 Dated Report: İ.T.Ü. Faculty of Mineral Engineering and Ministry of Public Work and Settlement were researched together in Kadifekale district in 1997. These working groups prepared a report about 1000- 1014 streets and draw unavailable built up area on the plans of Kadifekale like the former reports.

In conclusion, all of the reports said that, "some special areas of Kadifekale district are more landslide risky areas than the other regions in Izmir and the risky areas should be controlled in detail. Squatters should be cleaned and these areas should be afforested." But, still there are a lot of squatters in Kadifekale and all of them are on the risky areas.

CONCLUSIONS

The importance of work and readiness for preventing or minimizing natural hazard effects in Turkey, belonging to Developing Country Group, is doubled due to risks increased by its geological and geographical conditions. Turkish Natural hazard Policy, relating internal social and economical conditions, together with foreign relation arrangements, should be examined, and new strategies in law, institution and application fields determined.

First of all, Turkey should give up a post-disaster "recovery" policy. New pre-disaster, then post-disaster strategies, supported by civil organizations, should be developed.

Another important matter is related with the revision of a building quality and supervision system. As the role of non-controlled construction in natural hazard impacts until nowadays is well known, necessary modifications should take place in building quality, building supervision field.

Professional skill and ethics, affecting a building quality, is one of the problem areas. Keeping professional people subject to a periodical theoretical examination is compulsory. Tight inspection of professional chambers is inevitable.

Some physical plans resulting from mentioned defects have a big role in increasing natural hazard risks. Due to political pressures and unconsciousness, physical plans are modified continuously and number of floors increased, so already limited urban utility fields decrease, additional floors affect seriously building real supporting capacity, creating thus higher risks of potential earthquake effects. Consequently, a general, transparent supervision thru people's participation should be assured by a tight control of revision plans, by inspecting systems giving such construction permits. In addition, plan revisions should comply with the whole, very small area revisions permitted.

Another plan, which named as "Improvement Plan", is another serious problem for existing built up areas of our cities, which are constructed illegally. Depending on the last Amnesty Law; disordered unreliable sites have been produced. Repeated amnesty laws have not only motivated unlicensed constructions, but have also created condensed planning problems, difficult to resolve. This kind of applications increases natural hazard damages hundred per cent. Applications of nowadays in the Development Improvement Plans should be ended, reliable and safe sites, new models, decreasing risks, should be created. Mass housing projects may be the most convenient solution to this.

In the implementation process of physical plans %40 of the lands are gotten from the landowners without and costs and are used for urban utility services. This constant rate is defined in Development Law and is used in everywhere. However, highly crowded areas taken into consideration, this proportion, regarding a number of users, remains insufficient. Increase of utility portion, proportional to a population density, is proposed by a wide section of people.

As a natural hazard concept and planning are so close one within the other, this brings together a natural hazard sensible plan understanding in plan approach and implementation revisions.

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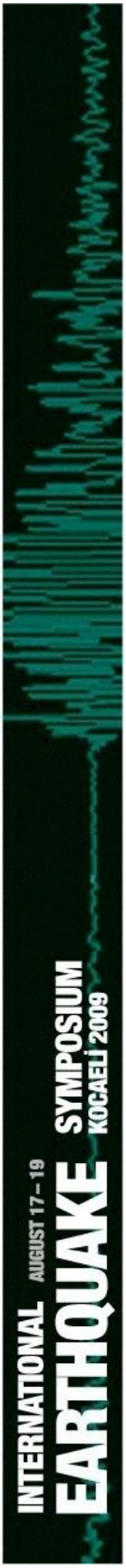

SOCIAL SCIENCES

SOSYAL BİLİMLER

A STUDY ON HIGH SCHOOL STUDENTS' ATTITUDES TOWARDS EARTHQUAKES: AN EXAMPLE OF GIRESUN

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Key Words: Earthquake, attitude, attitude scale, secondary education.

ABSTRACT Attitudes are described as "an individual's positive or negative reaction towards a stimulus". An individual at first gains knowledge about an attitude object; afterwards, expresses it as emotional reaction, and finally transforms it into a behavior. The aim of this study is to determine high school students' attitudes towards earthquakes. The sample of the research consists of totally 340 students in Giresun, Bulancak and Keşap. At first, a Likert type scale with 60 items was developed by the researchers to measure high school students' attitudes towards earthquakes. Validity and reliability of the scale was executed by the data obtained from the students. The students' general attitudes towards earthquakes and whether there are significant differences in terms of gender variable were investigated. Explanatory factor analysis was used for structure validity of the scale. After the factor analysis, 36 items were extracted from the scale. As a result, an Attitude Scale towards Earthquakes (ASTE) came out, which is composed of 24 items. Analysis showed that the scale consisted of four factors. Factor I which comprises 7 items was named "like and enjoyment". Factors II and III, each of which is composed of 6 items, was named "fear and anxiety" and "considering important" respectively. The last factor which consists of 5 items was named "interest". Finally, the scale with 24 items and 4 sub factors accounted for 50,102 % of the total variance. Internal consistency (Cronbach Alfa) coefficient was found to be . 87 (α = .87). These values showed that this scale was valid and reliable. Descriptive statistics and t-test were used to analyze the data. According to the findings, the students have positive attitudes towards earthquakes and there are significant differences in favor of female students. Teachers, researchers and curriculum developers were offered various suggestions.

INTRODUCTION

Turkey is one of the countries, which suffers from natural disasters because of its geological features, topographical structure and climate conditions (Ergunay, 2007: 2; Ozmen and oth. 2005), and it is a country in which there has been many losses of lives, properties and injuries. Nevertheless, in the last seven decades it is seen that earthquakes are listed in the first place among the natural disasters (Basbug, 2007: 124; Ozkul and Karaman, 2007: 251), it is followed by flood, land slip, rock fall and avalanche (Ceylan, 2007: 131).

Within this context, earthquakes are the first to come to mind because 2/3 of the physical damage is caused by aearthquakes in the last six decades. According to the currently validate Turkey Earthquake Zone Map, 96% of Turkey's land is under the risk of earthquake at various risk levels and 98% of the country's population maintain their lives in these places.

These rates clearly state that Turkey is a land which is openly under the risk of earthquakes (Activity Report of Natural Disasters Assurance Organization, 2004: 2). In Turkey which is situated on Alp-Himalayas Seismic Zone (Mediterrenean Earthquake Zone) (Sahin ve Sipahioglu, 2003: 42) 111 earthquakes occurred during the 20. Century (1900-2000) and 99391 persons lost their lives (Coburn & Spence, 2002: 4).

Despite the high number of loss of lives, the precautions taken to be protected from earthquake are not adequate in Turkey (Basibuyuk, 2004:1). At this point, as in the other countries where earthquakes occur frequently, Turkey is one of the countries which need to take crucial protective precautions. Even though scientific facts provide useful information about decreasing the risk of earhquakes, a large-scaled earthquake in a big city can be much more didactic (Bakir, 2004: 405). In fact, Turkish society has experienced such traumas caused by earthquake disaster in recent years. The earthquakes occurred lastly in Turkey have shown that even though seismic damage can be seen beforehand, intervention during the post-disaster are not adequate (Bakir, 2004: 405). In a country's development of individual and social conciousness, the culture of preparing for a disaster takes an important part. Education is one of the key tools to develop this tool (Shaw and oth. 2004; 39). In other words, educating activities play a critical role in solving the problems caused by natural disasters, especially earthquakes (Demirkaya, 2007: 38).

That people are informed about a matter affects their being ready for the coming risk (Dedeoglu, 2005: 1505). Accurate information helps people to change their behaviours and to question and research more in order to take precautions (Nathe, 2000:191).

In this context, one way of getting informed is about to being literate(Dedeoglu, 2005: 1505). In other words, education is one of the key tools in order to make a culture of being prepared for upcoming natural disasters (Shaw and oth. 2004; 39). In a society like Turkey where traditional values and attitudes are strong, education has the pioneering role in acquiring modern values and attitudes.

Likewise, in order to be get less harm, a necessity to be informed with proper attitudes against possible earthquakes in Turkey arises (Kasapoğlu and oth, 2004: 244–245). From this point, it is clearly seen that importance education in and out of school become visible. Accordingly, importance of education has been emphasized in different papers (Radu, 1993; Kuroiwa, 1993; Arya, 1993; Frew, 2002; Akt. Shaw and oth, 2004: 40; Andrews, 1998: 4). Shaw and Kobayashi (2001) stress that schools are important in raising awareness among students, teachers and parents (Akt. Shaw ve oth, 2004). That the main solution of the problem of decreasing the loss of lives, individual injuries and damage is education has been mentioned in various resources (Petal, 2004: 2; National Research Council, 1991: 17).

Shaw and others (2004: 41) state that education is a way of the key tools in order to prepare a culture of getting prepared and focusing on school-aged children is thought to be inevitable.

As indicated above, education has a major role in decreasing the damage caused by earthquakes. Therefore, it can be said that in a country like Turkey where earthquakes occur frequently the most important thing to do is raising up individuals who are aware and sensible about the earthquakes(Demirkaya, 2007: 38).

On the other hand, Karanci (2007) state that getting prepared for disasters increases the anxiety of disasters, expectation of losses, attitude of being prepared and risk perception.

Even though earthquakes are uncontrollable natural phenomenons (Slovic, Fisshoff and Lichtenstein, 1984), their damaging effects can be potentially decreased or restricted.

It is highly significant whether people see earthquakes and their effects are uncontrolable (McClure and oth, 1999: 241).

Peoples' getting informed about earthquakes and the ways of protection from them may help them to have positive attitudes towards the disaster and may give way to a change in behaviours to lessen the possible losses. This change of behaviour may help individuals to develop positive attitude and behaviours. In this way, the individuals who have positive attitudes towards earthquakes and their acquantices are possible to get

less harmed. Whereas, the ones who have a negative approach to earthquakes are proclaimed to be indifferent to the damages resulted from the earthquakes and increase the effects of damage; so that, the total damage will be more than in fact it is.

Attitudes like or dislike; are positive or negative conclusions or reactions to objects, occassions, or any other features of earth, abstract thoughts, or social policies (Atkinson and oth, 1999: 625). To be more frankly, attitudes are the tendencies of being positive or negative about an object, individual, organization or event (Ajzen, 2005: 3).

''An attitude generally makes people tend to behave against the attitude object. An individual who has a positive attitude towards an object will tend to behave posivitely, support the object, be close to it and help it. An individual who has a negative attitude towards an object will have the tendency of being indifferent, getting away from it, criticize it, or even hurt it''. (Hotaman, 1995, 13–16; Aydin, 2000; Akt. Demirkaya, 2007: 39).

Attitudes are learned through different ways: daily life, taking model, and learning. Once they are acquired, it is hard to change them, but in a systematic way it is possible to change them as well (Erdem and Anilan, 2000: 144).

The most effective way of acquiring attitudes is education. Effects of education on the attitudes are up to level of education (Morgan, 1993: 376).

However, attitudes cannot be measured with direct scanning, but they can be measured indirectly and behaviourally (Hogg and Vaughan, 2007: 205; Ajzen, 2005: 3; Morgan, 1993: 363).

It is useful to measure attitudes during the education process for predicting the learners' future behaviours by evaluating the attitudes measured at a specified time span, for determining their attitudes towards the conditions they are in, altering the attitudes or establishing new ones.

Hence, scientific attempt to depict the behaviours of individuals gives way to direct them towards better conditions with the help of regression.

When the students' attitudes towards earthquakes are mentioned, their feelings, thoughts and behaviours related to earthquakes are meant. In Turkey where losses of lives and properties caused by earthquakes are often experienced, knowing future adults' attitudes towards earthquakes has importance. In order to determine high school students' attitudes toward earthquakes, an attitude scale is needed.

In Turkey, there is not an attitude scale in order to measure especially high school students' attitudes. Keeping this need in mind, developing a scale to measure high school students' attitude toward earthquakes become one of the aims of this study. With the scale developed, it is expected to determine the students' attitudes toward earthquakes and variables related to positive or negative attitudes. This is expected to contribute to the studies in the field of attitudes toward earthquakes.

The main aim of this study is to determine high school students' attitudes toward earthquakes. In order to reach this aim developing a scale for high school students' attitudes is the second aim of this study.

THEORY (AND METHOD)

This study is based on descriptive method. As everybody knows descriptive studies define the situations accurate and precisely (Buyukozturk, 2009: 21).

POPULATION AND SAMPLE

Population of the study includes 1st, 2nd, and 3rd grade students in high schools in Giresun, Bulancak and Kesap during 2008–2009 academic years.

Sample of the study includes 340, randomly chosen, high school students who study in Kesap Anatolian Teacher Training High School (97 students), Bulancak High School (158 students) and Hamdi Bozbag Anatolian High School (85 students).

The students are chosen from different kinds of schools (like Anatolian High School, Anatolian Teacher Training High School, High School) in order to control the difficulties resulted from difference in student profiles. Besides, the students are chosen from different academic success groups and socio-economical groups on intent. The aim of having chosen so many students for sampling is gathering information from students who have different features, which has an increasing effect on reliability and validity of the study in fact.

TOOLS OF GATHERING DATA

Needed data for the study is gathered through questionnaires. The first part of the scale asks for personal information. In determining the questions in the poll previous studies and international papers on the topic have been looked through.

DEVELOPING SCALE

It is a fact that it is a difficult job to measure attitudes in the field of education. Therefore, researchers have developed many techniques. There is one way of developing scale in social sciences which was developed by Likert (1932) whose preparation, implementation and reply are easier. In such scales, participants determine the level of attitudes towards each item on the poll (Tezbasaran, 1997: 9). To be more frankly, participants mark their level of agreement for each statement (Anastasi and Urbina, 1997: 405). In such scales, there are statements, like *'completely agree', 'agree', 'I do not know' or 'I have no idea', 'I do not agree',* and *'I do not think so'.* These scales are graded like: 5 points for *'completely agree',* 4 points for *'agree',* 3 points for *'I do not know',* 2 points for *'I do not agree',* and 1 point for *'I do not think so'.* The negative statements are graded as just the opposite of the grades above. The total point is accepted as the participant's attitude towards that topic (Kagitcibasi, 1999). The most commonly used of these scales is the one developed by Likert attitude Scales (Tavsancil, 2005: 139; Ozgen and oth, 2007: 59). In order to determine the students' attitudes and behaviours, a 5-Likert type trial scale, which includes negative and affirmative items of 60, has been developed.

VALIDITY

For the content and aspect validity the attitude scale has been presented to the academicians in the departments of Geography, Geography Education and Turkish Language. Having retouched on the scale in accordance with the criticisizm, personal information questions have been added to the poll.

For the structure validity of the scale, it has been applied to 255 students as trial version. 245 of the gathered tests has been accepted as usable and included in the study. In order to investigate the structure analysis of the scale exploratory analysis has been applied.

In order to determine proficiency of the data gathered through factor analysis Kaiser-Meyer- Olkin (KMO) test is applied (Tavsancil, 2005: 50). If KMO value is higher than .60, it means that the data gathered through sampling is consistent with factor analysis. (Buyukozturk, 2007: 126). As this scale's KMO value is .871, it is acceptable. Secondly, having looked through Bartlett's Test of Sphericity (p=.00), it is agreed upon that the

data gathered is convenient for factor analysis. The higher the score of Barlett test is, the more significant it is (Tavsancil, 2005: 51).

At the end of the factor analysis, the items to be placed on the test, eigenvalue of the items 1, factor loading value of each item is minimum .30 are given emphasis and 36 items which are inconsistent with the values are omitted. First, the principal component analysis have been conducted for the remaining 24 items and then varimax rotation have been done. At the end of the procedure, it is found that the 24 items scale has four-factor structure. Then, Internal consistency Coefficient for each factor and the general reliability of the scale are computed. Last, the data from the four factors scale, common factor variance for the validity, factor loading values after rotation are presented in Table 1.

Table 1. Factor Loading Values After Varimax Rotation.

Factor 1			**Factor 2**			**Factor 3**			**Factor 4**		
Eigenvalue = 6.170			*Eigenvalue = 2.754*			*Eigenvalue = 1.841*			*Eigenvalue = 1.259*		
Variance %= 14.219			*Variance %= 13.731*			*Variance %= 11.657*			*Variance %= 10.495*		
IN	***AV***	***FLV***	***IN***	***AV***	***FLV***	***IN***	***AV***	***FLV***	***IN***	***AV***	***FLV***
1	*331*	*.481*	*2*	*423*	*.626*	*4*	*304*	*.406*	*3*	*424*	*.544*
5	*508*	*.626*	*7*	*527*	*.717*	*6*	*435*	*.498*	*8*	*625*	*.776*
9	*574*	*.679*	*12*	*546*	*.695*	*10*	*567*	*.751*	*11*	*445*	*.645*
13	*499*	*.651*	*14*	*532*	*.711*	*15*	*603*	*.703*	*16*	*392*	*.581*
17	*503*	*.663*	*19*	*545*	*.721*	*18*	*568*	*.734*	*24*	*454*	*.566*
20	*450*	*.614*	*22*	*627*	*.779*	*21*	*550*	*.686*			
23	*593*	*.699*									
IN: *Item Number*				***AV:*** *Average Variance*					***FLV:*** *Factor Loading Value*		

When Table 2 is looked into, it is seen that the attitude scale consists of four factors. There are seven items in the first one. Item number 1, 5, 9,13,17,20 and 23 include the subfactor of the scale 'love/like'. Factor loading values of these items are between .699-.481 and it means % 14.219 variance. Calculated Cronbach's Alpha (α) Coefficient for this sub-factor is .81.

Table 2. Sub-factors of Attitude towards Earthquake Scale (ATES).

Sub-scale	**Items**	**Factor Loading Value**	**Total Variance Explained**	**Cronbach's Alpha (α) Coefficient**
Love/Like	1., 5., 9., 13., 17., 20., 23.	.699-.481	% 14.219	.81
Fear/Anxiety	2., 7., 12., 14., 19., 22.	.779-.626	% 13.731	.81
Caring	4., 6., 10., 15., 18., 21.	.751-.406	% 11.657	.77
Interest	3., 8., 11., 16., 24.	.776-.544	% 10.495	.72
Total	**24 items**	**.779-.406**	**% 50.102**	**.87**

There are six items in the second sub- factor. Item number 2, 7, 12,14,19, and 22 include the subfactor of the scale 'fear/anxiety'. Factor loading values of these items are between .779-.626 and it means % 13.731 variance. Calculated Cronbach's Alpha (α) Coefficient for this sub-factor is .81.

There are six items in the third sub- factor. Item number 4, 6, 10, 15, 18, and 21 include the subfactor of the scale 'caring'. Factor loading values of these items are between .751-.406and it means % 11.657 variance. Calculated Cronbach's Alpha (α) Coefficient for this sub-factor is .77.

There are five items in the fourth sub-factor. Item number 3, 8, 11,16, and 24 include the subfactor of the scale 'interest'. Factor loading values of these items are between 776-.544 and it means % 10.495 variance. Calculated Cronbach's Alpha (α) Coefficient for this sub-factor is.72.

At this point scatter graphic of the scale should ne analyzied. The results are as follows as SPSS pack program shows:

When the scatter graphic is looked through, it is seen that the scale has a four-factor structure. At this point, 24 items scale is handled as four factors.

The factor loading values of 24 items in scale ranged from .779-.406. The factors on the scale meet %50.102 of total variance. This value explains the attitude towards earthquakes well.

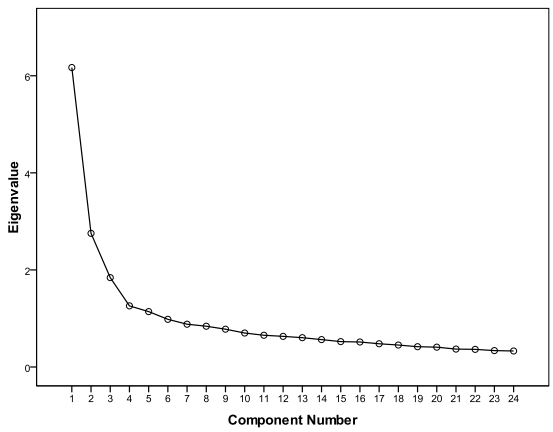

Figure 1. Scree Plot.

RELIABILITY

After checking structure validity, 36 of 60 items, whose internal consistency coefficient is low, have been omitted. Remaining 24 items', Attitude towards Earthquake Scale (ATES) Cronbach value is calculated .87.

When the results of ATES are looked through it is seen that the whole scale and the sub-factors have good internal consistency coefficients and it is accepted as a high-validate scale in the fields of education and social sciences.

At the end of the studies, there has become an attitude scale towards earthquakes with four sub-factors. 17 of the items on the scale have positive statements, and 7 of them have negative statements in order to prevent the participants' 'appreciate' tendencies, and these negative and positive statements are placed randomly. In evaluating the points on the scale the negative items (1., 6.,10.,15.,17.,18.,21. are reverse coded in order to supply unilaterality. The participants can have minimum 24 and maximum 120 points out of the scale. Higher points of the scale mean a positive attitude, and fewer points refer to negative sttitude. The total time given for scale is between 15-25 minutes.

DATA ANALYSIS

The data gathered through the scale are analysed with SPSS 17.0 (Statistical Package for the Social Sciences). In analysis of the data, descriptive statistics and independent t-test are used.

FINDINGS

Table 3. Attitude towards Earthquake Scale (ATES)'s Mean and Standard Deviation Values.

VARIANCE	N	MEAN	Sd
LOVE/LIKE	340	3,324	,777
FEAR/ANXIETY	340	3,712	,835
CARING	340	3,798	,777
INTEREST	340	3,005	,796
TOTAL	340	3,473	,574

When high school students' attitude towards earthquakes is scrutinized it is seen that their attitude is positive. This statement can be concluded from the points taken from both the subscales and whole scale. The results show that average of the scale is higher than $\bar{X}=3$ (Table 3).

Tablo 4. Attitude towards Earthquake Scale (ATES)'s Mean, Standard Deviation and t Values in terms of Gender.

VARIANCE	FACTOR	N	MEAN	Sd	t	p
LOVE/LIKE	FEMALE	190	3,329	,730	,134	,833
	MALE	150	3,317	,836		
FEAR/ANXIETY	FEMALE	190	3,881	,735	4,316	,000
	MALE	150	3,498	,905		
CARING	FEMALE	190	3,900	,729	2,762	,006
	MALE	150	3,668	,818		
INTEREST	FEMALE	190	2,997	,793	-,220	,826
	MALE	190	3,016	,802		
TOTAL	FEMALE	190	3,541	,545	2,464	,014
	MALE	150	3,387	,599		

Gender factor is also studied in high school students' attitudes towards earthquakes. There is not a significant gender difference in the results of t-test on the like/love sub-level (t=.134, p>.05). When fear/anxiety level is checked it is seen that gender means significance (t=4.316, p<.05). This has come from the higher points of the girls ($\bar{X}$ = 3.881, Ss = .735) than the boys ($\bar{X}$ = 3.498, Ss = .905). Gender is significant in the level of 'caring' (t=2.762, p<.05). This has come from the higher points of the girls ($\bar{X}$ = 3.900, Ss = .729) than the boys ($\bar{X}$ = 3.668, Ss = .818). In the 'interest' level it is seen that gender is not significant (t=-.220, p>.05). When the whole points are t-tested it is seen that gender factor is significant in the study (t=2.464, p<.05). This difference is resulted from the higher points of the girls ($\bar{X}$ = 3.541, Ss = .545) than the boys ($\bar{X}$ = 3.387, Ss = .599) (Tablo 4).

RESULTS AND SUGGESTIONS

In order to have a ppositive effect on attitudes towards earthquakes, determining the attitudes in pre, while, and post periods of the process is significant. (Guven and Uzman, 2006: 533). Accordingly prepared ATES will be able to be used easily by the researchers.

According to the results of this study:

• It is seen that high school students have a positive attitude towards earthquakes. The fact that there have been many earthquakes in Tukey and caused losses of properties and lives may have caused students to act more wisely about the subject. This positive attitude towards earthquakes may have a positive effect on their being readiness level for the disaster. Therefore, academic activities prepared for students may lead to success.

• When gender is concerned it is seen that there are significant differences on behalf of girls in sublevels of students' attitudes (especially fear/anxiety and caring). The fact that girls are more positive about the earthquakes may be a result of their being more sensitive about environmental affairs. However, many experimental studies show that girls have more sense of risk than boys. (Tomar, 2002: 49; Burger and oth, 2000; 157; Fordham, 2000: 7). Girls' being more anxious about the seismic danger can be explained with their being more vulnerable for the natural disasters. (Armas, 2006: 1230–1231; Krishnaraj, 1997). Likewise, many studies show that among the victims of disasters girls are seen to display more negative psychological mood and they are affected more negatively (Tomar, 2002: 49).

• Both the girls' and the boys' interest levels are seen to be lower than the other sublevels of the scale. Students' being less concerned about the topics related to earthquakes may be related with conducting the lessons.

Suggestions concerning the results of the study:

1. In attempting to maintain students' positive attitudes toward earthquakes, it is important for teachers to conduct their lessons in an enjoyable and attractive way. This approach will help students to learn the topics about earthquakes more temporarily and to develop attitudes which are affirmative. Therefore, it can be stated that the level of being prepared for the earthquakes can be developed among students and hence they will be able to lead a secured life.

2. In planning the training activities related to earthquakes gender factor should be taken into consideration.

3. In addition to education and training activities, media can be playing an important role in acquiring positive attitudes towards earthquakes. If media supports the activities with visual matearials such as documentaries and TV programs, a more positive attitude will be able to reached.

4. Special, in-service training activities related to earthquakes can be prepared for the staff by concerning organizations in order to be more effective on the subject with the students. During this training process, general information about earthquakes, precautions to be done before earthquake, ways of protection during earthquake, and overcoming the psychological effects of earthquakes can be taught.

5. It is thought that it will be better to emphasize the earthquake subjects in the areas under the high risk of earthquakes to help students to have a positive attitude towards it.

6. Last, it should be kept in mind that the study has been conducted in an area where the risk of earthquakes is quite low. So, the results can be different in high-risk areas. Therefore, it would be effective if the study is conducted in different risk leveled areas.

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"NATURAL DISASTERS COURSE" PROPOSAL FOR HIGHER EDUCATION UNDERGRADUATE PROGRAMS

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Key Words: Natural Disaster, Natural Disasters Course, Security Culture / Secure Life Culture

ABSTRACT This study was fulfilled towards the configuration of the Natural Disasters Course with the aim that the individuals in organizations aroused after the earthquake will gain skills towards the earthquake besides the professional training education in higher education programs and attain volunteerism spirit.

Disasters, with the broadest definition, are the technological or human structure based events result in physical, economical and social losses for people, and affect communities by stopping or interrupting the normal life and human facilities. (Ergünay, 1996)

Natural disaster program`s objectives are;

• To develop social skills to survive in extraordinary conditions through learning and teaching activities foreseen in undergraduate program,

• To evaluate volunteerism spirit of the higher education youth in updating and continuing emergency administration of Turkey,

• To systematize the collaboration of volunteer organizations and educated administrators that enforce the redevelopment process rapidly after the disaster,

• To provide unity of idea and application in the society through the individuals resisting the disaster and having the conscious of emergency by maintaining the security culture.

According to these objectives, education and teaching processes were determined by specifying anticipated behaviors for students` gain, units and topics, and a Natural Disaster Course Draft was devised.

Consequently, it is highly suggested that this study with the configuration of Natural Disaster Course in Higher Education Undergraduate Programs should aim urgent cases that requires individual intermeddling as well as the cases requires mass intermeddling by broadening and developing the study for further research.

INTRODUCTION

The situation that happened after the earthquake(s) occurred in Marmara Region in 1999 showed us that we are not ready for earthquake and the life following it. After a decade, we are in front of the history as a nation who lost the chance of giving the excuse 'we are not ready'. Now, we are more experienced in taking necessary precautions for before, during and after the disasters and we have more scientific savings against the earthquake and other natural disasters. Today, we shoulder the responsibility in the evaluation of world human beings' savings related with the issue of knowledge and skill shared pool.

When we add the possibilities of information sharing to this, it is obvious that our job is not hard. We should alter the options of structuring and application of this sharing.

In disastrous situations people come together regardless of their identities stemming from career, title, belongings, and status etc. They meet in a quite different geography with all stimuli that hurt. The power that stands in the breach from the picture of desperation, loneliness, vagueness and exhaustion can still be found in the repertoire in their brain and heart. Efforts like surviving and re-existing born from the behavior patterns which were loaded to them before disasters. Will of surviving will shape the energy of making his/her close people`s existence besides realizing one's self actualization. This unity of emotion and actuation will provide the mental schemas that have already been gained before. Thus, if these schemas are gained through the foreseen course content with theoretical and applied acquisition, being well equipped as a human being will stir up himself/herself. (Tankut, 2002; Öztürk, G. and Atalı,L.2002). It has been emphasized that disaster management should be included in human resources education with so many stages like prevention and harm reduction, preparedness, warning and forecasting, rescue and first aid, betterment and reconstructing in scientific studies (Marşap,2005).

A probable 'Natural Disasters' course in higher education undergraduate programs will aid the development of social responsibility consciousness and security culture of university youth besides self aiding skills.

PROBLEM

Disasters, with the broadest definition, are the technological or human structure based events result in physical, economical and social losses for people, and affect communities by stopping or interrupting the normal life and human facilities. (Ergünay, 1996)

Natural disasters, with the widest meaning, are the events that cause loss of lives and property. The first feature of a disaster is; it is natural, second; it causes losses of life and property, third; it happens in a very short time period and fourth; it cannot be prevented after it starts. Actually, it is known that some disasters may likely occur in some locations on earth. For example; the results of some disasters such as landslide, avalanche, flood and frost immediately and directly come out like the results of an earthquake. In some other disasters, the results are seen indirectly and after a long time like in drought (wikipedia.org).

Earthquake, drought, hail, heavy snow, blizzard, frost and icing, lightning, fog, flood, volcanic eruption, rock fall, mud flow, storm, hurricane, typhoon, tornado, landslide, tsunami (tidal waves), avalanche, meteor falling, extreme temp (so hot weather) and extreme cold, instant climate changes and natural radiation are in the information set of natural disasters. The development of security life culture has a significant importance on preventing loss of lives and property which are generally caused by the either natural oriented disasters and/or the disasters which have various effects by people. Such a school program has an effect like the rings on water on circle of individual, family, society and humanity.

Formal education is an important chance for the training of natural disasters knowledge. This knowledge- with a more responsible manner, faster, more optimal and aiming the gains leading to the behaviors which are closer to the true- have a value of an investment towards society by the help of youth.

RATIONALE OF THE STUDY

This study is based on preventive-developmental guidance understanding. It is also towards the configuration of the Natural Disasters Course with the aim that the individuals in organizations aroused before-during-after the earthquake will gain skills and knowledge required for the secure life culture and attain volunteerism spirit beside the professional teaching training education in higher education programs. With such

kind of approach the theoretical and applied configuration of the proposed course is given great importance.

THE STRUCTURE OF NATURAL DISASTERS COURSE

THE OBJECTIVES OF NATURAL DISASTERS KNOWLEDGE EDUCATIONAL PROGRAMS;

- To improve the surviving skills in extraordinary situations and leadership skills including values of human being through the educational activities that are foreseen in undergraduate programs.

- To evaluate volunteerism spirit of the higher education youth in updating and continuing emergency administration of Turkey (Öztürk,2008),

- To systematize the collaboration of volunteer organizations and educated administrators that enforce the redevelopment process rapidly after the disaster,

- To provide unity of idea and application in society by realizing "training of trainer" function in literature through the resistive individuals equipped with security culture and emergency situation consciousness against the disaster.

- According to these objectives, education and teaching processes were determined by specifying anticipated behaviors for students` gain, units and topics, and a Natural Disaster Course Draft was devised.

THE STUDENTS OF NATURAL DISASTERS COURSE ARE EXPECTED TO GAIN THE FOLLOWING BEHAVIORS:

- The student can define aims, scope and application situation of emergency education.

- The students` level of awareness on disasters that threat the society will increase.

- The students gain knowledge and skills of emergency.

- The students can produce risk taking projects and can improve practicing skills in the scale of themselves, families, classes, schools and society.

- The students see themselves as a part of the whole in spreading the disciplines of basic emergency awareness.

- The students develop values and attitudes towards the understanding and formation of security culture.

TEACHING PROCESSES;

- Making warm-up, aims and objective behaviors, topic patterns, educational situations live by considering summary and dimensions of evaluation will increase the efficiency of teaching in order for each session to form a self teaching package.

- In forming the educational situations towards the content; techniques like problem solving, presentation, project production and administration, controlled discussion, question-answer, role-play, drama, group games, individual and group-work are used.

- Audio-visual materials are used.

- Strength and practice knowledge are improved with scenarios of crisis- rehearsals of crisis organization.

- Development of communication skills is given importance by making use of active listening, empathy and effective communication techniques.

- Educational situation that contribute the development of life skills like decision making, developing secure behavior, creative problem-solving, time management, face to face communication, conflict management, adapting though stress, team work are planned and practiced.

- Precautions that increase the efficiency of sense organs in educational settings where theory and practice are performed together are taken.

- Topic experts are regarded as resource.

- Supervision settings trained in psychotherapy are needed to plan in case sharing of disaster victims' personal experiences is in question.

UNITS AND TOPICS (FORESEEN IN 13 MODULES AND 13 SESSIONS)

1) Arguing-debating the reasons for the need of Natural Disaster Course;

Meeting / Objectives / Expectations.

Restructuring the order and importance of topics with the group members.

2) Terminology; General approach towards the meaning and importance of the notions like Security culture / Insecure areas and situations / Disasters (natural , unnatural) / Crisis / Volunteerism / Democracy / Participation / Natural-unnatural dangers that threat civilians; Earthquake / Fire / Flood / Chemical, Biological, Nuclear dangers / Magnetic pollution and protection, etc.

3) Societies View over Natural Disasters: cultural differences of societies' view of natural disasters, regional differences in the same society, kinds of treatment against law, levels of esteeming scientific data, and judgment of personal experiences over natural disasters and secure life culture and effects in developing prejudice, anticipation and approach.

4) Types of natural disasters (Earthquake, drought, hail, heavy snow, blizzard, frost and icing, lightning, fog, flood, volcanic eruption, rock fall, mud flow, storm, hurricane, typhoon, tornado, landslide, tsunami (tidal waves), avalanche, meteor falling, extreme temp (so hot weather) and extreme cold, instant climate changes and natural radiation, reasons of happening, role of human factor and ways of developing secure life culture).

5) Leadership skills (Figure 1), Theory and practice of emergency-first aid and skills of surviving.

6) Decision Making Process and Secure Behavior: Growing up, developing and importance of decision making in our life, dimensions of decision making; data collection, developing and evaluating alternatives, importance of our values, risk taking, styles of decision making, understanding the roles of logic and intuition, and making individuals meet with decision making skills.

7) Cognitive and Psychological Importance and Methods of Preparations to Disaster;

Conscious of Insurance / Health / Law / Immediate Prevention / Collaboration-Division of Work and Solidarity.

Psychological problems that victims of disaster (child-young-adult) may face and precautions that can be taken against those problems.

8) Variables of developing Immediate Prevention Plan and Immediate Action Plan

9) Civil Defense Local Forces / Foundations and Institutions related with topic including existing central and country organizations.

Practicing stages of local organizations

Practices and precautions towards increasing social participation.

Importance of citizenship training in reducing the sufferings of disasters and models of education

10) Practices and precautions towards increasing the efficiency of emergency calls.

11) Relations with media in before disasters and in emergency situations.

*Evaluations of the first 12 hours / 12-24 hours / 24-36 hours / 36-72 hours / 72 and other hours,

*Evaluations of What / Who / When / Where / Why / How in order to form a public view before-during or after the disaster.

12) Scenarios of Rehearsal and Exercise

Samples of Simulation

13) Evaluation of the course with what we have learned, noticed and what we can do. Assignments, seminar, project proposal, examination, survey, pilot study, model proposal, practice exercises are the measuring instruments that can be used in evaluation of the course takers.

MEASURING AND EVALUATION;

- By the help of Constructivist View, planning and evaluation of warm-up, aims and objective behaviors, topic patterns, educational situations, summary and dimensions of evaluation will increase the efficiency of teaching in order for each session to form a self teaching package.

- A pre-test question list that contributes the realization of the objectives in teaching program is applied to the students. The same document is given as a post-test to the end of the term to examine the benefit of the students who participate to the program. If mid-evaluation of educational situations between two practices is done through periodical tests, an attitude towards "Whole Learning" will be followed then.

- Learning products of educational settings, where the education is realized with an attitude that gives importance to individual differences and "Multiple Intelligence", shouldn't regard only test results. Project reports, seminars, assignments, model proposals, practice samples, language of color-voice-line-body and verbal and written activities might be considered as the measuring instruments.

- "Certificate of Disaster Animator" might be given to those who hold the necessary success for the course.

RESULT and SUGGESTIONS

It is highly suggested that this study, with the configuration of Natural Disaster Course in Higher Education Undergraduate Programs should aim urgent cases that requires individual intermeddling as well as the cases requires mass intermeddling by broadening and developing the study for further research.

The volunteerism spirit of the higher education youth in meeting the needs of health, law and education of the children who lose their parents after the disaster may be increased in environments where international experiences are shared (Öztürk, 2002,2006).

Suggestions for this course are as follows;

- This paper should be accepted as a justified course by YÖK (High Education Board), in order to include "Natural Disasters" course in a term in higher education programs.

- As a result of a public opinion that will arouse after the presentation and the publication of this paper in our universities, "Natural Disaster" course should be included in the programs.

- Efforts towards making volunteerism spirit more professional should be performed with collaboration and coordination of universities with local nongovernmental organizations and related foundations in both theoretical and practical projects.

- Individuals who will take responsibility in "Natural Disasters" course should be those who have succeeded in Pre-service Educational Program in Kocaeli district, who have performed actively towards reformation of social life in earthquake areas. After that, the process should be started with academicians who have provided the institutionalization of these relations and students who have taken animation training and from there it should be distributed to other areas.

Skills of Education and Administration

Skills of Culture, Arts and Sports

Social Skills, Living Skills

Surviving Skills

Self-care Skills

Fig. 1. Leadership Skills (Öztürk,2006).

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DEPREMİN BÜYÜKLÜĞÜ VE ŞİDDETİ KONUSUNDAKİ YANILGILARIN KAVRAMSAL DEĞİŞİM MATERYALİ İLE GİDERİLMESİ

REMEDIATING MISCONCEPTIONS CONCERNING MAGNITUDE AND INTENSITY OF AN EARTHQUAKE THROUGH CONCEPTUAL CHANGE MATERIAL

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Anahtar Kelimeler: Deprem, büyüklük, şiddet, yanılgı

ÖZ Yerkabuğundaki kırılmalar veya kaymalar sonucu ortaya çıkan enerjinin yeryüzünde sarsıntılar meydana getirmesine deprem denir. Yeryüzündeki canlı ve cansız varlıklar bu sarsıntılardan etkilenir. Etkilenme derecesi ise depremin gücüne veya boyutuna bağlı olarak değişir. Depremin gücü iki yolla ölçülür. Bunlardan birincisi depremin büyüklüğü, diğeri de depremin şiddetidir. Araştırmalar depremin şiddeti ve depremin büyüklüğü kavramlarının sık sık karıştırıldığını veya birbirlerinin yerine kullanıldığını göstermiştir. Bu araştırmanın amacı da öğrencilerinin söz konusu kavramlar hakkındaki anlama düzeylerini artırmak ve yanılgılarını gidermeye yönelik bir öğretim materyali geliştirmek ve etkililiğini ölçmektir. Bu amacı gerçekleştirmek için metin, fotoğraf, grafik ve animasyon unsurlarını içeren web tabanlı bir öğretim materyali geliştirilmiştir. Materyal "büyüklük" ve "şiddet" kavramlarının ayırt edici özelliklerini ön planda tutularak tasarlanmıştır. Giresun Üniversitesi Eğitim Fakültesi Sosyal Bilgiler Öğretmenliği Programına devam eden 48 öğrenci araştırmanın örneklemini oluşturmuştur. Araştırma ön test – öğretim ve son test şeklinde dizayn edilmiştir. Ön test aşamasında yapılandırılmış mülakat ile öğrencilerin depremin şiddeti ve depremin büyüklüğü hakkındaki bilgi düzeyleri ve yanılgıları ölçülmüştür. Öğretim müdahalesi aşamasında, oluşturulan materyal kullanılarak sismolojideki büyüklük ve şiddet kavramları öğrencilere öğretilmeye çalışılmıştır. Son test aşamasında da ön test de kullanılan mülakat soruları öğrencilere tekrar sorulmuştur. Araştırmanın sonuçları, öğretimden sonra öğrencilerin anlam düzeylerinin arttığını ve daha önce sahip oldukları yanılgıların büyük ölçüde giderildiğini göstermiştir. Araştırma sonuçları dikkate alınarak öğrencilerin depremin gücü hakkındaki anlamalarını geliştirmeye yönelik tavsiyelerde bulunulmuştur.

GİRİŞ

Yerkabuğundaki kırılmalar veya kaymalar sonucu ortaya çıkan enerjinin yeryüzünü sarsması olarak tanımlanan deprem, bütün boyutları ile ciddiye alınması gereken doğal bir olaydır. Depremin oluşumu ve yeryüzündeki etkileri pek çok bilim dalını ilgilendirir. Sismoloji, jeofizik, jeoloji gibi fen bilimleri depremin oluşum mekanizmasıyla ilgilenmesinin yanında, iktisat, siyaset, sosyoloji, psikoloji, eğitim gibi sosyal bilimler de depremin zararlarının azaltılmasına yönelik çalışmalar yaparlar.

Deprem sırasında ve sonrasındaki kayıpları en aza indirmek için deprem öncesinde alınacak önlemler hayati öneme sahiptir. Yasal düzenlemeler, sarsıntılara dayanıklı yapı inşa edilmesi, bireylerde deprem bilgisi ve bilincinin oluşturulması deprem öncesinde alınabilecek başlıca tedbirlerdir. Deprem bilgisi ve bilincinin oluşturulması, formal ve informal eğitim süreci içerisinde yer almaktadır. Deprem hakkındaki yanılgıların tespit edilmesi ve giderilmesi bu sürecin önemli bir unsurunu oluşturur.

Gerek fen bilimlerinde, gerekse sosyal bilimlerde öğrencilerin, öğretmenlerin ve öğretmen adaylarının sahip oldukları yanılgıları tespit etmek amacıyla çok sayıda araştırmalar yapılmıştır. Farklı disiplinlerdeki bu araştırmaların pek çoğunun sonuçları; öğrencilerin, öğretmenlerin veya öğretmen adaylarının kavram yanılgılarına sahip olduklarını göstermiştir (Stavy, 1990; Barrow ve Haskins, 1996; Öcal, 2007; Bayram ve Cin, 2008).

Özdemir ve diğ. (2002), ilköğretim okulu öğrencilerinin deprem ve depremin zararlarından korunma yollarını öğrenme düzeylerini araştırmışlardır. Araştırma sonuçları, öğrencilerin depremin sebebi ve depremin oluşumu, deprem öncesinde alınacak tedbirlere ilişkin bilgi eksiklilerine ve kavram yanılgılarına sahip olduklarını göstermiştir.

Oğuz (2005) tarafından 5., 6., 7. ve 8. sınıfta okuyan toplam 823 Türk ve Amerikan öğrencinin katılımıyla yapılan çalışmanın sonuçları, Türk ve Amerikan öğrencilerinin depremi algılamaları arasında bazı konularda benzerlikler ve bazı konularda farklılıklar olduğunu göstermiştir. Ancak genel olarak, Amerikan öğrencilerinin deprem hakkındaki bilimsel bilgi düzeylerinin Türk öğrencilere göre daha yüksek olduğu ve Türk öğrencilerin daha fazla yanılgıya düştükleri görülmüştür.

Öcal (2007)'ın Sınıf Öğretmenliği ve Sosyal Bilgiler Öğretmenliği Anabilim Dalı 4. sınıf öğrencilerinin depremle ilgili bilgi düzeylerini belirlemek üzere yapmış olduğu araştırmanın sonuçları, öğrencilerin çoğunun kitle iletişim araçlarının olumsuz etkisi nedeniyle "deprem şiddeti" ile "deprem büyüklüğü" kavramlarını birbirine karıştırdığını göstermiştir.

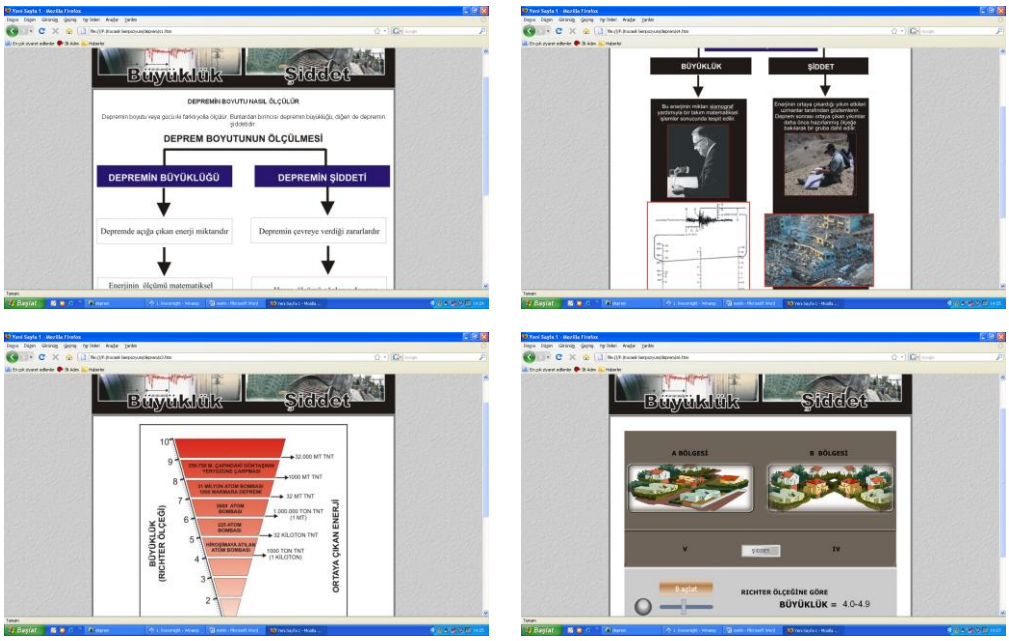
Bayram ve Cin (2008) tarafından sosyal bilgiler öğretmen adaylarının cinsiyet, sınıf düzeyi ve depreme maruz kalma değişkenlerine göre deprem hakkındaki yanılgılarını ve anlama düzeylerini belirlemek amacıyla yapılan çalışmanın sonuçları, öğrencilerin deprem ile ilgili bilgilerinin genelde yeterli olmakla birlikte yanlış öğrenmeler de içerdiklerini ortaya koymuştur.

Araştırmalar, yanılgılarının oluşmasında medya, dil, ders kitapları, öğretmen, sosyal çevredeki yanlış inanışlar (mit'ler) gibi pek çok faktörün etkili olduğunu göstermiştir (Trowbridge ve Mintzes 1985; Lawrenz, 1986; Wandersee ve diğ., 1994; Şahin, 2003; Yazıcı ve Samancı, 2003; Bayram ve Cin, 2008). Bu araştırmalar, kavram yanılgılarını gidermenin kolay olmadığını da göstermiştir. Bu yanılgıları gidermek için kavramsal değişim modelleri geliştirilmiş ve bunların etkililiği üzerine araştırmalar yapılmıştır (Posner et al, 1982; Dagher, 1994; Chambers ve Andre, 1997; Case ve Fraser, 1999; Akbaş, 2008) . Şimdiki araştırma, üniversite öğrencilerinin "depremin büyüklüğü" ve "depremin şiddeti" hakkındaki yanılgılarını tespit etmek ve bu yanılgıları gidermek için hazırlanan kavramsal değişim materyalinin etkililiğini ölçmek amacıyla gerçekleştirilmiştir.

YÖNTEM

Araştırma 2009 yılında Giresun Üniversitesi Eğitim Fakültesinde gerçekleştirilmiştir. Sosyal Bilgiler Öğretmenliği programı 1. sınıflara devam eden 48 öğrenci araştırmaya dahil edilmiştir (Erkek= 27, Kız= 21). Araştırma, ön test – öğretim - son test şeklinde dizayn edilmiştir. Ön test aşamasını gerçekleştirmek için yapılandırılmış bir mülakat ölçeği hazırlanmıştır. Ölçekte "depremin şiddeti" ve "depremin büyüklüğü" ile ilgili 6 soru yer almıştır. Sorular bu iki kavramın genel özellikleri ve ayırt edici özellikleri üzerine odaklanmıştır. Öğrencilerin konu hakkındaki ön bilgileri bu ölçek kullanılarak tespit edilmiştir. Öğretim aşamasında ise metin, fotoğraf, grafik ve animasyon unsurlarını içeren web tabanlı bir öğretim materyali kullanılmıştır. Materyal, "büyüklük" ve "şiddet" kavramlarının genel ve ayırt edici özelliklerini ön planda tutularak tasarlanmıştır (Şekil 1).

Bu aşama tamamlandıktan sonra ön testte kullanılan mülakat soruları öğrencilere tekrar sorulmuştur. Elde edilen veriler tanımlayıcı istatistik tekniği ile analiz edilerek her bir maddenin frekans ve yüzdeleri bulunmuştur. Kavramsal değişimin gerçekleşip gerçekleşmediğine öğrencilerin ön test ile son test arasındaki başarı farklılıkları dikkate alınarak belirlenmiştir.



Şekil 1. Öğretim Materyalinin Ekran Çıktısı.

BULGULAR

Araştırmadan elde edilen bulgular aşağıdaki tablolarda özetlenmiştir. Tablolar, öğrencilerin ön test ve son testlerdeki sorulara verdikleri cevaplarını içermektedir. Cevaplar gruplandırılmış, frekans ve yüzdelikleri bu tablolarda gösterilmiştir. Sorularla ilgisiz olan cevaplar "Gruplandırma Yapılamadı" şeklinde tablolarda belirtilmiştir. Boş bırakılan cevaplar ise "Cevap Yok" şeklinde kodlanmıştır.

Tablolara genel olarak bakıldığında öğrencilerin ön testte vermiş oldukları yanlış cevapların büyük bir kısmının son testte giderildiği görülmüştür. Öğrencilerin ön testteki sorulara verdikleri cevaplar çok fazla çeşitlilik göstermesine rağmen, son testteki sorulara verdikleri cevaplar daha az çeşitlilik gösterdiği de tablolardan anlaşılmaktadır. Tablolarda yer alan bulguların bir başka özelliği de "Gruplandırma Yapılamadı" ve "Cevap Yok" kategorilerinde yer alan öğrenci sayısının ön testte fazla, son testte az olmasıdır. Bütün bu göstergeler öğretimin ve öğretim materyalinin etkili olduğunu göstermektedir.

Tablo 1. Depremin Büyüklüğünün Tanımı ile İlgili Ön Test ve Son Test Sonuçları.

Depremin Büyüklüğünün Tanımı					
Ön Test Sonuçları			Son Test Sonuçları		
	Frekans	%		Frekans	%
Sarsıntının yayıldığı, etki ettiği alandır	18	37.5	Deprem sırasında açığa çıkan enerji miktarıdır	41	85.4
Oluştuğu bölgede meydana getirdiği zarar derecesidir	15	31.2	Depremin matematiksel ifadesidir	2	4.1
Deprem sırasında açığa çıkan enerji miktarıdır	5	10.4	Gruplandırma Yapılamadı	3	6.2
Gruplandırma Yapılamadı	4	8.3	Cevap Yok	2	4.1
Cevap Yok	6	12.5			

Tablo 2. Depremin Şiddetinin Tanımı ile İlgili Ön Test ve Son Test Sonuçları.

Depremin Şiddetinin Tanımı					
Ön Test Sonuçları			Son Test Sonuçları		
	Frekans	%		Frekans	%
Depremin yarattığı hasardır	22	45.8	Depremin çevresinde meydana getirdiği hasardır	42	87.5
Sarsıntının gücüdür	10	20.8	Sarsıntının gücüdür	2	4.1
Depremin matematiksel olarak hesaplanmasıdır	5	10.4	Kırılma sırasında açığa çıkan enerji miktarıdır	2	4.1
Kırılma sırasında açığa çıkan enerji miktarıdır	4	8.3	Kırılmanın büyüklüğüdür	1	2,1
Yeraltındaki dalganın, kırılmanın büyüklüğüdür	3	6.2	Gruplandırma Yapılamadı	1	2,1
Gruplandırma Yapılamadı	2	4.1			
Cevap Yok	2	4.1			

Tablo 3. Depremin Büyüklüğünün Ölçülmesi ile İlgili ön Test ve Son Test Sonuçları.

Depremin Büyüklüğü Ne İle Ölçülür					
Ön Test Sonuçları			Son Test Sonuçları		
	Frekans	%		Frekans	%
Sismograf ile	8	16.6	Rihter Ölçeği ile	43	89.3
Ölü, yaralı sayısı ve enkazın büyüklüğü ile	6		Sismograf ile	2	4.1
Rihter Ölçeği ile	3	6.2	Cevap Yok	4	8.3
Gruplandırma Yapılamadı	4	8.3			
Cevap Yok	28	58.3			

Tablo 4. Depremin Şiddetinin Ölçülmesi ile İlgili ön Test ve Son Test Sonuçları.

Depremin Şiddeti Ne İle Ölçülür					
Ön Test Sonuçları			Son Test Sonuçları		
	Frekans	%		Frekans	%
Rihter Ölçeği ile	14	29,1	Mercalli Ölçeği ile	42	83,3
Sismograf ile	6	12,5	Rihter Ölçeği ile	2	4,1
Bölgeye verdiği hasarla ile	1	2,08	Gruplandırma Yapılamadı	2	4,1
Gruplandırma Yapılamadı	3	6,25	Cevap Yok	2	4,1
Cevap Yok	24	50			

Tablo 5. Depremin Büyüklüğüne Etki Eden Faktörler İle İlgili ön Test ve Son Test Sonuçları.

Depremin Büyüklüğüne Etki Eden Faktörler					
Ön Test Sonuçları			Son Test Sonuçları		
	Frekans	%		Frekans	%
Tabakaların özellikleri, yerkabuğunun ve toprağın yapısı, fay hattının özellikleri	23	47,9	Fayların büyüklüğü, kırılmanın hızı, kırılmanın süresi, fayların gerilim düzeyi	42	87,5
Yanlış yapılaşma, binaların sağlamlığı	12	25	Fay hattının yeryüzüne yakınlığı	3	6,25
Gruplandırma Yapılamadı	3	6,25	Gruplandırma Yapılamadı	1	2,08
Cevap yok	10	20,8	Cevap yok	2	4,1

Tablo 6. Depremin Şiddetine Etki Eden Faktörler İle İlgili ön Test ve Son Test Sonuçları.

Depremin Şiddetine Etki Eden Faktörler					
Ön Test Sonuçları			Son Test Sonuçları		
	Frekans	%		Frekans	%
Fayların özellikleri	12	25	Nüfus, binaların dayanıklılığı, zemin özellikleri	40	83,3
Yerin yapısı ve tektonik özellikleri	11	22,9	Depremin oluşum süresi	3	6,25
Fayların yeryüzüne yakınlığı	6	12,5	Fayların yeryüzüne yakınlığı	2	4,1
Binaların yapısal durumu	4	8,3	Gruplandırma Yapılamadı	2	4,1
İnsanların eğitim durumu	2	4,1	Cevap yok	1	2,08
Gruplandırma Yapılamadı	1	2,08			
Cevap yok	12	25			

TARTIŞMA VE SONUÇ

Ön test sonuçları öğrencilerin çoğunun "depremin büyüklüğü" ile "depremin şiddeti" kavramlarını karıştırdıklarını göstermiştir. Bu kavramların medyada yanlış kullanımı, söz konusu yanılgının nedenlerinden biri olabailir. Basında yer alan "Yunanistan'da merkez üssü İyon denizinde Preveza sahil kenti kuzey açıklarında, Richter ölçeğine göre 4.2 şiddetinde (büyüklüğünde olması gerekir) bir deprem oldu", "İspanya'da bugün meydana gelen Richter ölçeğine göre 5.1 şiddetindeki (büyüklüğündeki olması gerekir) deprem paniğe neden oldu", "Adana'da Richter ölçeğine göre 3,4 şiddetinde (büyüklüğünde olması gerekir) deprem oldu" gibi haberler "büyüklük" ve "şiddet" kavramlarının karıştırılmasına neden olabilir.

Yanılgıların diğer bir nedeni de ders kitaplarına bağlanabilir. 2006 yılında Milli Eğitim Bakanlığı tarafından bastırılan ve Liselerde okutulan "Ortaöğretim Coğrafya 9" adlı kitapta yanılgılara neden olabilecek ifadelere rastlanmıştır. Örneğin, sayfa 110' da" Depremin şiddeti açığa çıkan enerji miktarına bağlıdır" ifadesi yer almıştır. Halbuki, depremde açığa çıkan enerji miktarı "büyüklük" ile açıklanması gerekirdi. Benzer şekilde, aynı kitapta sayfa 111'de "Depremin şiddetini tespit etmek için bazı ölçekler geliştirilmiştir ve bunlardan en yaygın olarak kullanılanı **Rihter** ölçeğidir" ifadesi bulunmaktadır. Halbuki, depremin şiddetini tespit etmek için Rihter ölçeği değil Mercalli ölçeği kullanılır.

Doğal Afetler ve deprem ile ilgili kazanımlar ortaöğretim coğrafya müfredatında yer almaktadır. Bu kazanımların gerçekleştirilmesi sırasında öğretmenin "depremin büyüklüğü" ile "depremin şiddeti" kavramlarına vurgu yapmasında fayda vardır. Öğrencilerin sahip oldukları yanılgılar tespit edilip düzeltilmediği taktirde uzun yıllar devam eder.

Sonuçlar, öğrencilerin "depremin şiddeti" ve "depremim büyüklüğü" kavramlarının farklı özelliklere sahip farklı iki kavram olduğunun farkında olmadıklarını göstermiştir. Ön test soruları öğrencilerin konu ile ilgili anlama seviyelerini ölçmesinin yanında söz konusu kavramların aynı kavramlar olmadığı yönünde farkındalık da geliştirmiştir. Bu durumdaki öğrenciler "depremin şiddeti" ve "depremin büyüklüğü" kavramlarını kelime anlamlarından yola çıkarak açıklamaya çalışmışlardır. Örneğin, depremin büyüklüğünü pek çok öğrenci "sarsıntının yayıldığı ve etki ettiği alan" ve "depremin meydana getirdiği zarar derecesi" olarak açıklamaya çalışmışlardır. Bu açıklamalar, öğrencilerin "depremin büyüklüğünü" daha çok "depremin şiddeti" ile açıklanabilecek bilgi ve kavramlarla açıklamaya çalıştıklarını göstermiştir. Son test sonuçları göstermiştir ki öğrencilerin yaklaşık %85 i depremin büyüklüğünü deprem sırasında ortaya çıkan enerji miktarı olarak görmüşlerdir.

Ön test sorularına verilen cevaplar, öğrencilerin depremin büyüklüğünün ve depremin şiddetinin ne ile ölçüldüğünü bilemedikleri veya yanlış bildiklerini ortaya koymuştur. Öğrencilerin en az yarısı depremin büyüklüğünün ve depremin şiddetinin ne ile ölçüldüğüne yönelik sorulara cevap vermemiştir. Depremin büyüklüğünün ne ile ölçüldüğünü yalnızca 3 öğrenci bilebilmiştir. Depremin şiddetinin ne ile ölçüldüğünü ise hiçbir öğrenci bilememiştir. Yine ön test sonuçları öğrencilerin "Mercalli Ölçeği'nden haberdar olmadıklarını, daha çok Rihter Ölçeği'ni bildikleri ve bu ölçeği hem depremin büyüklüğünü hem de depremin şiddetini ölçmede kullanıldığını düşündüklerini ortaya koymuştur. Son test sonuçları ise depremin büyüklüğünün ve depremin şiddetinin ölçümünde kullanılan ölçekleri büyük oranda öğrendiklerini ortaya koymuştur. Yani, öğrencilerin %89 u depremin büyüklüğünün Rihter Ölçeği ile ölçüldüğünü, %83'ünün de depremin şiddetinin Mercalli Ölçeği ile ölçüldüğünü öğrenmişlerdir.

Ön test sonuçları, öğrencilerin depremin büyüklüğüne ve şiddetine etki eden faktörler hakkındaki düşüncelerinin çok çeşitli olduğunu göstermiştir. Öğrenciler bu faktörleri bildiği halde hangi faktörün depremin büyüklüğüne, hangi faktörün de depremin şiddetine etki ettiğini bilememişlerdir. Yani, depremin şiddetine etki eden faktörleri depremin büyüklüğüne etki eden faktörler veya tersi olarak görmüşlerdir. Son test sonuçları ise öğrencilerin büyük bir kısmının depremin büyüklüğüne etki eden faktörleri yerkabuğunun iç özellikleri ilişkilendirebildiğini göstermiştir. Depremin şiddetine etki eden faktörleri ise depremin yeryüzünde meydana getirdiği etkiler ile ilişkilendirebildikleri görülmüştür.

Öğrencilerin ön bilgilerindeki eksiklikler ve yanlışlıklar oluşturulan öğretim materyali sayesinde büyük ölçüde giderilmiştir. Kavram öğretiminde ve kavramsal değişimde, kavramların ortak ve ayırt edici özelliklerinin bilinmesi oldukça önemlidir. Kavramların ortak özellikleri fazla, ayırt edici özellikleri az ve belirgin olmadığı durumlarda kavram öğretimi nispeten zor olur. Araştırmaya konu olan kavramların depremle ilgili olması, her iki kavramın da depremin boyutunu ölçme amacıyla kullanılması bu kavramların ortak özelliklerini oluşturmaktadır. Kavramsal değişim materyalinin de söz konusu kavramların hem ortak, hem de ayırt edici özelliklerine vurgu yapması öğrencilerin başarılarının en önemli nedeni sayılabilir.

Ortak özellikleri belirgin, ayırt edici özellikleri belirgin olmayan durumlarda araştırmamızda kullanılan kavramsal değişim materyalleri etkili olabilmiştir. Ancak, bu tür materyaller her durumda etkili olmayabilir. Bazı durumlarda öğrencilerde bilişsel çatışma yaratabilecek materyaller oluşturmak gerekli olabilir. Hangi tür materyalin kullanılması gerektiğini öğretilecek konunun özelliği ve öğrencilerin ön bilgileri belirler. Bu nedenle, depremle ilgili diğer olgu, olay ve kavramlarının öğretiminde hangi materyallerin oluşturulması gerektiği üzerine araştırmalar yapılmalı ve öğreticilerin hizmetine

sunulmalıdır. Teorikte mümkün olan bu öneri öğretilecek konuların fazla olması ve öğrenci özelliklerin çeşitlilik arz etmesi nedeniyle uygulamada mümkün olmayabilir. Araştırmacı öğretmen modeline ağırlık verilmesi ve öğretmenlerin uygun kavramsal değişim materyallerini kendilerin üretmesi en rasyonel çözümdür.

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THE EFFECTS OF 17 AUGUST MARMARA EARTHQUAKE ON THE TURKISH ECONOMIC CRISIS OF 2000 NOVEMBER/2001 FEBRUARY

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Key Words: Turkish Economy, Economic Crisis, Marmara Earthquake, Restructure Programme.

ABSTRACT In the year of 1994, Turkey experienced a major economic crisis. Following this, after the 17 August 1999 Marmara earthquake which is one of the disasters having a lot of human and economical losses in the world, the economical structure of Turkey which was affected by 1994 economic crisis was damaged in large quantities. This huge natural disaster together with other economical and social problems of Turkey cause a sequential crises of 2000 November and 2001 February. Thus, Turkey struggled to remove of the effects of Marmara Earthquake in addition to decrease of effects of 2000 November and 2001 February crises for a long time.

In this study, it is analyzed that how the Marmara earthquake effect the country economic structure and what type role this disaster undertake on the crises in that years in which Turkey already have a great number of problems, and also some evaluations are carried out. Finally, the research summarized that the economic effects of Marmara earthquake on the Turkish economic crisis of 2000/2001 are staded as figures with the help of different tables, considering macro-economic indicators, such as inflation, growth, balance of payments and public spending. Besides, it is emphasized that crises creates different chances for improve of the economic structures.

INTRODUCTION

Economic crises come first on the top of the list of the agenda of our country in the last thirty years. The matters such as economic crisis, the effects of the crises, the methods of struggle with the crisis results still continue from past to the present. The reasons such as our economy's being overwhelmed in crisis periods, natural disasters triggering the crisis, social and political disturbances called for majoring on the experienced crisis and looking for the solutions to get over the crisis.

Crises are experienced in our country and also in the world. All the crises prevent country's economic growth although their causes and results differ from one country to another. Therefore, crises generally negatively affect countries, enterprises and even social structure. In this regard, the politics to be applied and steps to be taken in order to provide consistency in economy are critically important. In recent years, countries experiencing crises are putting forward various solutions and programs supported by International Monetary Fund (IMF) to provide economic consistency.

Turkey met with Free Market Economy after 1980 and started to follow policies that are open to foreign markets. After the decisions taken on January 24,1980 along with the struggle with the economic imbalance, a new policy of economic development and growth was developed. After the April 5, 1994 decisions with the eartquake disaster and consecutive economic crises, Turkey experienced a difficult and a long process. Thus, the years 1994, 1999 and 2001 have a significant importance for Turkey's economy. Because economic crises experienced in the years of 1994, 2000, 2001 and Marmara earthquake caused serious damages in the country.

In this study, the effects of the earthquake on August 17 on October 2000 and February, 2001 crises, growth, inflation, balance of payments, budget balance and banking sector being the macro economic indicators of Turkey and banking sector are handled and they are analyzed through tables. Although it has been ten years since Marmara Earthquake on August 17, 1999 -being one of the biggest earthquakes in the world- its effects couldn't have been wiped out yet.

THE DESCRIPTION OF ECONOMIC CRISIS AND ITS CONTENT

One of the most significant economic problems that the countries face are crises nowadays. It is a reality that crises bring touble and danger to mind at the beginning. However, as well as they mean a dangerous period for the counties, they are also described as a turning point to create opportunities, too.

Altough crisis is described in many ways; it can be described as causing extensive damage to country from a macro point of view and to the firms from micro point of view on an uexpected time and basis in economy (Aktan and Şen, 2001; 1225). Crisis, in this meaning, is a sudden danger on an unexpected time for any reason whatsoever.

In another description, it is described as negative improvements harming to social and economic structure caused by some external factors that can not be controlled (Uludağ and Arıcan, 2003; 51). Countries have to be ready to the crises as they may emerge suddenly due to unexpected or external factors.

Crises experienced suddenly succeed in effecting not only the country but also the world since arising time by showing contagious effect such as flu bug. Even if the structures of the crises in the countries may differ, their results are also the same. Ecomic crises firstly cause macro economic destabilizing. The problems such as price inconsitencies, inflation, stagflation, and depression are major of these problems. Inflation has been the main problem that many of the countries dealt with from the crises periods up to now in the economic sphere. Economic crises bring along the imbalance of supply and demand with the inflation.

After the 1980s especially in developing countries productions of domestic industries failed to satisfy the needs as a result of liberal politics together with excessive loaning. In developing countries, productions' remaining incapable due to increasing demand and not being able to catch the technological improvements decreased countries' power of competing in the foreign markets (Sönmez, 2005; 184). Loaning deadlock that the countries face caused foreign imbalances affecting the balance of payments adversely.

Countries' foreign imbalances may derive from existing unfavorable variances between their exports and imports, inadequate productions, irregular capital movements, wrong foreign trade politics (Karluk, 2002; 160). These countries that can not compensate their expenses with foreign trade incomes tried to hold the balance with foreign loaning. For the developing countries domestic borrowing cause significant problems as well as foreign borrowing. In developed countries, the shortage of public revenues, the overage the public expenditures, not being able to tax the tax revenues effectively and public corporations' duty harms are the major reasons for the domestic borrowing. The crises in the country not only bring about the economic imbalances but they also cause social depressions. Unbalanced income distribution and social explosions are two of the primary problems.

After 1980s globalization and financial liberalization started to spread all over the world and significant fluctuation in capital movements have been experienced. In recent years, in all over the world and also in our country, financial and real crises have been experienced by the effects of globalization and financial integration process. Among the resons for he financial crises, we can take into account the weakness of capital structure

and banking sector and foreign capital investments negative effects especially on short term capital movements.

Crises may appear not only as economic crises but they can also arise as political, cultural, social and natural disaster crises. Therefore, countries on the one hand try to struggle with the crisis through politics applied in crisis period, on the other hand they carry out some stability programs studies in order to prevent new crises.

JANUARY 24, 1980 AND APRIL 5, 1994 DECISIONS AND AFTERWARDS

After 1980 Turkey met with Free Market Economy and started to follow outward-looking liberal economic politics. The economy which paused after 1980, speeding inflation, increasing loans and social and political chaos formed a basis for preparing stabilization program supported by International Monetary Fone (IMF) which was called as January 24, 1980 decisions (Kazgan, 2005; 192-193). It was aimed to take fundamental and extensive decisions in order to take out the economy from this inconsistency and marketeconomy to bring into force with January 24, 1980 Economic Stabilization Decisions (Karluk, 2002; 453). Altough the programs applied after January 24 Stability Decisions gave successful results in the first years generally, in the following years new crises were experinced again. Turkey almost got trough the 1990s in a period in which nearly in every three years a crisis took place. During this period, even though temporary stabilization programs were put into effect, they didn't have a permanent success and economy got into a serious narrowing period being negatively effected by these crises.

The factors carrying Turkish economy to April 5 such as chronical inflation, increasing external and internal loans, fast increase in government deficit, quick real rise in payments, Turkish Lira's gaining excesiveve value very fast, instabilities faced in foreign Exchange and financial markets gave rise to economic balances to shake. It was intended to decrease public deficits very fast, to reduce inflation and to add value to Turkish Lira and to perform structural reforms to increase export through April 5, 1994 decisions. By means of April 5 decisions that were put into effect, in very short time money and currency markets started to gain stability quite quickly, foreign exchange was taken under control and confidence for the Turkish Lira was provided. After the high rated devaluation, current accounts deficient turned out to be over and budget deficits were reduced. Narrowing the domestic demand slowed down the inflation's increasing speed for temporarily.

After April 5 decisions, with fiscal austerity politics, on the one hand economic progress was tried to be realized and on the other hand with the high rate interest external and internal borrowing were encouraged (Kazgan, 1995,234). Turkish economy entered to a short period of developing process after 1994 crisis. However after 1994 crisis, many of the structural reforms proposed in precautions couldn't been realized and public sector kept on having deficits in budget. Despite the fact that economy was getting over generally, it was seen that April 5 decisions couldn't reach the desired results.

In 1998, from Asia and Russia crises Turkey was negatively effected to a great extent and these crises resulted in decreasing of export incomes, narrowing of domestic demand and economic growth's slowing down (Özbilen, 2002;174). After the speedy spread crises in all over the world, Turkey's macro economic problems started to increase in spite of the stabilization programs. Together with this process, rising of inflation, interest, loaning and budget deficits ruined internal balance, capital outflows, liquidity crunch, uncertain foreign currency and increasing foreign trade deficit spoiled external balance, uncertainity in financial markets put banking sector in an awkward position and was effective in dragging Turkey to economic depression. As a result of this, in 1998 a medium term stabilization program was prepared.

THE EARTHQUAKE ON AUGUST 17, 1999 AND ITS EFFECT ON THE CRISES ON NOVEMBER 2000 AND FEBRUARY 2001

While this bad course was going on in the country, the Earthquake on August 17, 1999 became a great catastrophe. This earthquake which took place in the center of country's economy not only caused many losses of life and property but it also added a new problem to all the other matters. The program being applied in order to descrease the high inflation and to enable economic growth and in order not to be affected by global crisis was interrupted by earthquake disaster and the country entered a long and deep crisis period. The severe damage caused by the earthquake made it obligatory to revise the macroeconomic targets concerning the year.

The total cost of macroeconomic cost of the earthquake in 1999 is 19 billion dollars according to State Statics Institute (SPO) numbers. With refence to SPO, Marmara Earthquake caused the 8-10 percent loss of Gross National Product in the country. Due to the earthquake, particularly loss of life many other losses in habitation, trade, industry, infrastructure, tranportation, machine and production stocks took place. From this perspective, the social and economic life stopped and the economy collapsed that was already in a bad condition. It is found out that the repair cost of habitants that was ruined or destroyed, service sector and business and industry companies are 3.5 billion dollars (Morgil, 1999; 9).

Kocaeli, Sakarya, Yalova, İstanbul, Düzce and Bolu, where most of the life and property losses took place, are the industry and trade centres whose contribution to country's economy with their population and industries is very high. These cities called as the eartquake areas have 34.7% share in gross national product (GNP) according to SPO's 1999 data and their share in industry value added is 46.7%, and these areas in which the eartquake caused severe damage have important shares in especially petrochemical, textile, some industry products and motor vehicles production, assemblage and tyre industry. From this perspective, the earthquake are has experienced a reduction period in many sectors such as automative, retail, architecture sector and industry. Turkey's economy has entered a receSPOon process by the effects of temporary politics put forward against the crisis in 1998 and the eartquake in 1999.

Each sector's reaction against crisis and the earthquake was different. For instance, archtitecture and accordingly iron and stell sector lived its hardest period of receSPOon and they decreased. Acoordingly, many businesses' pause in production due to the earthquake in industry region has brought the biggest economic receSPOon after 1980 according to SPO's 1999 data. After the Eartquake in 1999, economy's becoming smaller gaining speed with businesses' stopping their productions was felt up to the crisis in 2001 densely and as a result production and usage capacity rates falling brought about a loss in GNP. As a conclusion, the eartquake's most significant macro economic effect on Turkey economy was experienced on GNP and generally the production decreased. The cities densely effected by Marmara Eartquake have a total 6.2% share in GNP, 14.1 and 3.9% share in export and import and their share in general budget incomes are 12.4%, and according to SPO's data there was a decrease of 6.1 in GNP (Önal, 1999; 2).

The economy of the country generally diminished due to Marmara Earthquake, the industry sector became smaller 8.3% while production industry narrowed 9.2%. Trade sector became one of those that were densely affected by the earthquake by decreasing 8.0%. Communication, transportation and tourism sectors are the ones that come first negatively affected by the earthquake (Selçuk and Yeldan, 2001; 483-488). In addition to earthquake's causing main losses in country's production, since the earthquake area was the hearth of Turkey's business life, especially in the number of qualified labor force many significant losses are experienced (Morgil, 1999; 8).

After the earthquake, banks and finace sector also suffered a lot. In the earthquake area, the systems of the banks collapsed, the trading values got narrowed depending on the

industry sector's decrease and their balance sheets were corrupted after the earthquake. Besides all these negative incidents, most of the banks greatly carried out their duties, and supported the earthquake victims. It was decided to postpone the loans provided by the banks but later it was seen that frozen or nonperforming loans increased. In 1998 financial problems such as nonperforming loans, increased losses on exchange and imposing state banks' duty losses on the public purse continued in 1999 and banking sector collapsed in a very short time.

This case of banking sector is closely related to insurance sector. After the crises in 1990s, the shareholders equities of the insurance sector weakened and technical problems rised. Increased competition after crises resulted in insurance premiums' decrease. Some of the capital of the firms in insurance sector belonging to banks and holding companies weakened insurance companies and banking sector indirectly. After the earthquake, insurance companies paying huge amounts of compensations in response to low premiums decreased this sector's profitability (Erdoğan, 2003, 80-87).

Some of Turkey's macro economic indicators between the years 1996–2002 are as in the following; The reduction started at the end of the year continued after the earthquake on August 17, 1999 and at the end of 1999 this reduction became negative -6.1%. After these succeive crises, when in 2001, application results in terms of growth deviated from the aims and became negative in -9.4%. After the applied program, the decrease in purchasing power -with the earthquake effect- decreased the general level of the prices, while the general inflation rate was above 80% in 1998, in 1999 this rate decreased to 65%. The current deficit's ratio to GNP was between 9,8 billion dollars and 4,9% at the end of the year 2000. As a result of the exchange rate politics resulting in overvaluation in Turkish Lira and money politics not finding the right way he export in 2000 increased to 32.7% in comparison with the export in the year 1999. As the industry value added share of the earthquake is about 45%, especially the intermediate goods export in the years 1999-2000 increased about 34%. In the year 2000 the export worth 53.98 billion dollars is realized. The amount of export in the year 2000 was 27,32 billion dollars and fell behind the targets and caused the increase of 89,12% in the foreign trade deficit. In the year 2001 the proportion of export's compensating the import increased to 75%. In the year 1999, the corruption in the budget increased and high reel interests, the rise in social security transfers and the earthquke caused the increase in budget expenses. According to 1997-1998 data of SPO the cities in the earthquake area constitute the 34% part of total added value in industry. Furtermore, 58% of tax revenues are gathered from this area in which industry and trade is very extensive. After the earthquake catastrophe, the growth of government deficits that were already large amounts brought forward additional taxes such as The Earthquake tax in 2000 and govermental economic business additional charges. The Central Bank had to change its money policy as a result of tax postpones, investment expenses and internal loan stocks increase although 5,7 billion dollars of additional earthquake tax was gathered in the year 2000. Government deficits permanently increased in 2001 and the ratio of Public Dept Necessity to Gross National Product (GNP) increased at the rate of 4,6, that is 16,4% proportionally to year 2000. (Table 1).

Table 1. Some Macroeconomic İndicators of Turkey (1996-2002).

	1996	1997	1998	1999	2000	2001	2002
Real Growth (%)	7,1	8,3	3,9	-6,1	6,3	-9,4	7,8
Inflation (CPI) (%)	80,4	85,7	84,6	64,9	39,0	68,5	29,7
Inflation(WPI) %)	85,9	81,8	71,8	53,1	32,7	88,6	30,8
Ratio of Exports to Imports	53,2	54,1	58,7	65,4	51,0	75,7	69,8
Foreign Trade Balance (Billion $)	-10,5	-15,3	-14,2	-10,4	-22,3	-4,4	-4,4
Current Account Balance / GDP (%)	-1,3	-1,3	1,0	-0,7	-4,9	2,3	-1,0
Consolidated Budget Balance (%)	291,0	81,0	65,0	147,5	40,4	122,4	36,8

Reference: T.R. Prime Ministry Undersecretariat of Treasury, the Treasury Statistics, 1980–2003, Ankara, 2004, s. 43–44.; T.R. Prime Ministry State Statistical Institute, 1923–2002 Statistical Indicators, SIS Press, Ankara, 2003, s. 445–616.

THE CRISES IN NOVEMBER 2000 AND FEBRUARY 2001

The process started with capital movements liberalising after 1994, not being able to convert the short term foreign capital movements to investments in the year 2000 and the sudden escape of foreign capital from the country in crisis periods accelerated financial crisis (Ertuna, 2005: 204-205). Asia and Russia crises in 1997–1998 rummaged the money and finance markets, as a result of these corruptions in the finance sector bankruptcies affected the economies of the countries and financial crises reached to large amounts (Stubbs, 2005; 199). Argentina experienced the crises heavier than the other Latin America countries after 2000 and faced with unemployment, poverty, economic narrowing and the escape of hot money (Berlin ve Trinkulas 2005; 14). In Turkey, by the negative effects of Russia and Southeastern Asia, high interest and economic recession went on and it was obeserved that problematic loans increased. Thus the profitability of the banks decreased, sunk credits rose and banking movements narrowed.

When these deformations in finance sectors in Turkey further increased towards the mid November, investors canalized to foreign currency by selling their stock certificates, overnight interests increased up to 400%, speedy melting foreign currency reserve of the central bank invited financial markets and stock markets into crisis (Ateş, 2004: 195–215).

Before the crisis in November 2000, although some alarm bells started to ring, not being able to take necessary precautions threw Turkey's economy into an inevitable crises. In November the crisis air replaced the positive expectations in the year 2000 Inflation Decrease Program. The day November 22, 2000 went down into history as 'Black Wednesday'.

While the crisis is being experienced in the country, Central Bank firstly transferred money the markets but as a result of insecurity this additional liquidity attempt couldn't prevent canalizing into foreign currency. In a short time, when Central Bank stopped liquidity entrance, overnight interests rose to 2000%, the prices of government bones constantly fell down and due to crisis 6 billion dollars went out from the markets. After the crisis, the targets of the money policy changed (Toprak, 2001; 184, 189).

After the November crisis, the decrease in interest rates was realized by the easiness of the additional reserve easiness from the IMF and short term capital entrance and the nervousness in the markets fell down for a short time. After 2000 November crisis, 2000 Inflation Decreasing Program application criterion were revised. Finance markets completely calmed down and growing speed slowed down towards the end of the year. People entered the year 2001 with pessimistic expectations. The crisis on February 21, 2001 was the successor of February 2001 crisis. Although November crisis was the significant indicator of the February crisis, necessary precautions couldn't been taken. In February, before the national treasury tender bid, political tensions such as the argument between the prime minister and the president caused the loss of trust to the program being applied. Since the non-confidence to the applied economic program after the financial crisis in November 2000, canalizing to the foreign currency increased by taking currency risk into account and thus February 2001 crisis was experienced.

At the beginning of the February 21, at one night the overnight repo interest rose at first up to 1500 % at first and then 7000%; 100 index of İstanbul Stock Exchange (ISE) fell down from 17000 to 7000. The foreign Exchange rate on February 19, which was 1$=686500 TL olan, increased up 920000TL on February 23 (Güloğlu, 2001; s.526).At the beginning of the year 2000, Iflation Decreasing Program that was put into effect, weakened by the increasing demand for currency in crisis in 2000 and the rise in interests and even if some precautions were taken in the November crisis and it collapsed after February 2001 crisis with the confidence to the markets completely disappeared.

Many banks open to the November crisis decreased their capital ratios by taking loans in high interest and they trapped under high costs. In February crisis, banks had to buy foreign currency from high rates by taking foreign currency risks (Toprak-Demir, 2001: 8-9). Especially after February 2001 crisis, the confusion in banking sector's capital structure remained and the need of borrowing of the public increased in order to restructure of public banks. Public sustained its borrowings over banking sector (İşcan, 2003: 85).

After November 2000 and February 2001 crises, Türkiye lived a heavy year. For instance, the economy narrowed at the rate of 9,4% and foreign trade deficit became 4,4 billion dollars. The current deficit had the surplus of 3.3 billion dollars but the inflation increased up to 86.6% in wholesale price index. The ratio of foreign borrowing to Gross National Product (GNP) rose to 77% and it was observed that there was a rise in internal loan stocks. Short term foreign capital declined and trust to TL already disappeared and GNP for per head decreased to 2123 in terms of dollars and these numbers indicated that the year 2001 was a bad year for the economy of Turkey (SPO, 2003).

After February 2001 crisis, in society loss of incomes and unemloyment increased and people's struggles to earn their livings reached to its advanced stage. Many businesses stopped their activities after February 2001 crisis and some of them closed. These crises brought about unemployment and production problems (www.bagımsızsosyalbilimciler.org /iktisatg.htm., 2002, p. 1). On Mach 14, 2001 The Transfer to the Strong Economy Program, which was a reform program directed to structural precautions, was put into practice by forming a new dimension to the crises solution programs.

In this program, floating Exchange rate system was passed from the system of depending on foreign currency. In the program, in short term money policy was attached

importance and it was intented to increase transparency and an independent central bank. In medium term, inflation was targeted and price stability was pointed. As for the long term, by the structural banking reforms a consistent growth was aimed (Serin and Bastı, 2001:1216). Turkey, in its struggle with the crises, covered a significant distance by the The Transfer to the Strong Economy Program. In the program, growth, reel sector, inflation targeting, privatization, Money and finance politics were reshaped in a strategic framework.

By transfer to the Strong Economy Program, reform studies containing a long period were started. This program became a hope for the Turkish society that had no more strength to stand after consecutively crises and earthquake catastrophe. The applications about the reforms show that Turkey did its part. Macroeconomic indicators showed that the objectives of inflation and growth rate were reached but problems such as GNP speed slowing down, increasing total loaning stocks, current account balances' constantly giving deficits dangerously, increasing foreign trade deficits and high rates of unemployment and rising state loaning stock indicated that there should be many revisions in the program.

CONCLUSION

It is a reality that from time to time, crises and natural disasters took place in our country and they may also happen in the future. Natural disasters and crises resemble eachother in terms of difficult to be predicted, sudden and being short term. Although crises do not cause so much loss of lives as natural disasters, yet many great losses are experienced in economic meaning. It is not posibleble to get rid of the loss of live pain after the earthquakes. Social, humane and economic effects of August 17 earthquake were felt in November 2000 and February, 2001 crises to a great extent. Turkey owned significant experience and high bills in both Marmara Earthquake and November 2000 and February 2001 crises. Therefore, required precautions should be taken against natural disasters and crises and plans should be made beforehand in order to lower the damages to the minimum. The earthquake negatively affected the country as much as crises in terms of its economic consequences. Turkey should learn lessons from the Marmara Earthquake. Governments' putting urgent action plans and programs into practice by not taking manipulative people into consideration will provide us to get rid of a probable earthquake with less losses of life and property. That's why strengthening or reconstruction of the buildings that were destroyed or ruined in the earthquake area should be prevented and it should be encouraged to transfer the trade centers and industries from the places that may be mostly affected from the earthquake. The buildings to be obligatory constructed should be erathquake-resistantly, having all the the facilities and should be designed in a way that not risking human life in the earthquake area being our centre of trade and industry, required controls should be repeated constantly. Under the leadership of the state, everbody should be made conscious about the earthquake and government promotion should not be cut off. All the facilities should be provided for the people and the institutions about the earthquake in both in the country and the abroad and necessary importance should be attached to their trainings and demonstrations. Today, when the global crisis had influence on Turkey, it should never be forgotten that crises conditions still survive and crises and earthquakes will not be the last ones. Thus, Turkey's durability against these kşnd of crises and shocks should be increased. In reconstruction program, necessary revisions should be made, especially the politics about the construction sector should be strenghtened, the quality and usage of the equipment to be used in the projects should be attached importance and the controls about this matter should be done so strictly. In urban transformation projects, the risk of eartquake should be taken into account, risk analysis should be done and wrong constructing shoul not be permitted. In the areas where the earthquake risk is high, initially socio-economic plans should be revised and old residential should be strengthened against the earthquake.

According to State Statistics İnstitiute (SSI) data, not only many of the people living in the earthquake area became homeless but 38.1% of them lost their jobs and became unemlpoyed. Thus, the promises given to the community should be kept and urgently new work areas that the crisis will create opportunity should be found. According to SSI qestionnaire, 55 % of the companies in the earthquake area did not take out a policy. In Turkey, required importance should be attached to the insurance sector in order to make it reach expected efficiency and power. Especially, many encouraging studies and trainings on earthquake insurance should be accelerated. The content of natural disaster insurance should be widened and Obligatory Earthquake Insurance should be made mandatory for each building owner. Particularly after 1980, three big economic crises and Marmara Earthquake showed that in Turkey various political and social problems take place being macro economic ones the major. Therefore, at first by determined programs a confident atmosphere should be created and then the suffers should be made forgotten in unity. By acting in this awereness, required precautions should be taken on risk management and alternative solutions should be produced without forgetting crises and earthquake disasters. In order to make Turkey a country that is developed, modern, easily adapted to globalization and catching the latest trends in technology, the necsessary strategies should be determined before the crises are experienced and all the opportunities should be evaluated.

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THOUGHTS AND KNOWLEDGE OF THE FIRST GRADE STUDENTS AT HIGH SCHOOLS ABOUT EARTHQUAKE

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Key Words: Earthquake Education, Science and Technology Lesson, Unit of Natural Processes, High School Students.

ABSTRACT In the science and technology courses, informations about earthquake are being given in the last unit of at 8 grades of the primary education. In this research, the thoughts of the first grade students in secondary schools have been examined in seven dimensions. The information in all dimensions has been examined to check if it changes according to gender of the students, earthquake experience and the type of the schools.

This research was done in Kocaeli-İzmit in every first grades of the secondary schools in Kocaeli Izmit. The 555 students were chosen by cluster sampling technique from the Anatolian, general and technical high schools. The data collection tool was a Likert type items consisting of three subgroups. Reliability and validity study was repeated, owing to passing time between two studies. A pilot study was done with 390 students except the sampling of this study. As a result of this pilot study some of the items were removed from the data collection tool. With the remaining items, Cronbach's alphas for internal consistency were 0.78 Collected data was analyzed with SPSS 15.program.

The results have indicated that the gender and the type of the school effects the results but the experience of the earthquake doesn't effect the earthquake knowledge. In the lights of these information the girls know much more than the boys about the earthquake, and the things to do during and after the earthquake. Moreover the students in Anatolian high schools has much more and true information than the students in technical high schools.

INTRODUCTION

THEORETICAL FRAMEWORK

As it is known, our country is on one of the most active earthquake region of the world. Like the past devastating earthquakes, in the future it is a fact that the earthquakes in the future also will cause many deaths and loss. According to the regions of earthquake map: the 92% of the whole country, 95% of the population, 98% of the industrial areas, 93% of the dams are under the earthquake danger. In the last 58 years, because of the earthquakes 58.202 people lost their life and 122.096 injured and approximately 411.465 building collapsed or heavily destroyed. As a result, earthquakes approximately Kill 1.003 people and destroy 7.094 buildings every year in Turkey [1].

The unwanted effects of the earthquakes arise from the social economical loss and psychological problems they caused. It is possible to prevent and lessen these losses. The loss is not the result of the earthquake indeed, it is because of the inefficient and insufficient precautions (Başıbüyük, 2004).

Many researches emphasize that the public ought to be educated to decrease the unwanted effects of the earthquakes (Ergünay, 1996; Hurnen & Mcclure, 1997; Erçetin, 2003; Taş, 2003; Başıbüyük, 2004; Kibici et all, 2005:161 ; Öcal, 2005). Goodlad thinks 'school is the most strategic part of this education' (As cited Erçetin, 2003). Students can learn and know what an earthquake is, how it happens, how it effects the society and

what it should be done to be preserved at schools. Earthquake education should start from the structure of the earth's crust and include earthquake, the mechanism of the earthquake, the probable loss of the earthquake and the ways to be protected (Öcal, 2005).

When we look at the researches about earthquake education; Barrow and Haskin's research named 'earthquakes don't change the university students' knowledge' shows that students knowledge about earthquake is just what they obtained through films or newspapers or TVs. Moreover, Ross and Shuell's study named 'student's beliefs about earthquake' revealed that the 6th grade student's lack of enough information about earthquake causes that the students keeps on wrong traditional beliefs and insufficient information they got from their public. In addition to these researches, Tsai's research named ' the thoughts about earthquake after the natural disaster in Taiwan' which examines fifty-two fifth grade students; reveals another truth that students explains the reasons of the earthquake very different from scientifically. The same students obtained much of their information about earthquakes through their families, films, and TVs (As cited Barnett et all, 2006).

According to Ross and Shuell, if the knowledge about earthquake has been taken from an informal environment they can learn wrong things (As cited Barnett et all, 2006). Barrow and Haskins (1993) students can have wrong information from TVs, movies and their environment.

There are some similar researches about students' earthquake knowledge at our country (Çelen & Üner, 2002; Başıbüyük, 2004; Oğuz, 2005; Öcal, 2005; Özgüven & Öztürk, 2006; Öcal, 2007; Şimşek, 2007).

Çelen & Üner (2002) in their study named 'the information comparison after and before the earthquake educations at medical high school students in Ankara' examined the students knowledge level after an earthquake education in a class. As a result; even with a short time earthquake education the students' knowledge about earthquake raise up and they indicated that earthquake education must be spread every classes of school.

Başıbüyük (2004) in his study named 'earthquake knowledge in adults and examining the effective factors' studied the adult's knowledge level, their view about causes for earthquake and to reduce the unwanted results of it. As a consequence, it is emphasized that by using national and local mass media the society should be educated and starting from the primary schools educative studies should be done about earthquake.

Oğuz (2005) in his study named 'surveying American and Turkish middle school students' existing knowledge of earthquakes by using a systemicnetwork' compare the Turkish and American students who are taking classes from five to eight and measured. He revealed that American students have a better knowledge of earthquakes than Turkish students.

Öcal (2005)'s study named 'evaluating the education in primary schools social sciences classes' which was done on social sciences teachers. This study clarified that many of the teachers didn't educated about earthquakes effectively during their license education. While preparing the primary school programme, the subjects about natural disasters especially the cities where earthquake may happen, should be considered and in these cities the earthquake subject should be taught in a detailed way. Again Öcal (2007)'s study about the nominee primary school teachers examined their knowledge level on earthquake. This study has done with 145 university students who has been still getting education at Gazi University. As a result of this study this teacher nominees haven't efficient earthquake knowledge.

Özgüven & Öztürk (2006)'s study named 'the effects of basic earthquake education on primary school students' revealed the truth that after 90 minutes basic earthquake education the level of earthquake knowledge is raised. As a result of this research, the

basic earthquake education is recommended to take place in Ministry of National Education Programme.

In Şimşek's (2007) research named 'students ideas about earthquake' the students at 1st, 2nd, 6th and 8th class examined and asked for their ideas about earthquake. None of these students could explain the earthquake scientifically. Similar to the other researches students said they obtained their information through TV programmes.

In an another research in Turkey there is a connection between the society's fate belief and the reason of the earthquake (Aydın & Esen, 1997). In Taiwan, Tsai (2001) who has made a study after the earthquake, students explained the cause of the earthquake as supernatural forces, cultural myths and sudden changes in gravity (As cited Barnett and et al, 2006). Even though the students have a formal education on earthquake they may have misunderstandings like this (Barrow & HasKins, 1993; Tsai, 2001; Oğuz, 2005; Şimşek, 2007).

In our study through primary school program, at eight class in science and technology class in subject 'natural processes' the basic principals about earthquakes that should be known by students and how can protected from earthquakes took part. As this unit is the last unit of the primary school program, the first classes of the secondary schools took part in this study. In this research group, knowledge and thoughts about earthquake, the comparison of the reason of earthquake with beliefs and other natural events, the to do list before and after the earthquake, the earthquake experiences and education, school type, the relations between these things examined and compared.

The Importance of the Study

Prior researches examined adults, teacher nominees, teachers and students knowledge about earthquakes. In all those studies it is especially emphasized that a formal earthquake education should take place in school program. Importance of this study, at eight grades of primary education in Science and Technology lessons earthquake is taken as a subject as a result of this the knowledge levels of the 1st grade students at high schools is examined in this study. Moreover this study can be taken as a pre-evaluation of the present primary schools program. In addition to these mentioned above, examine of the students knowledge, beliefs, behaviors, and teaching what to do before and after the earthquake is other differences of this study.

The Aim of Study

In this study, student's knowledge and thoughts examined according to air conditions, the to do list before and after and compared according to gender, experiences and school types they are situated.

Problem of the Study

What are the thoughts and knowledge of 1st grade students at high schools?

Sub-Problems

1. What are the knowledge and thoughts of the 1st grade students at high schools about air conditions, earthquake prediction, behaviors and things to do, to lessen the unwanted effects before, during and after the earthquake?

2. Do the thoughts and knowledge make difference according to gender?
3. Do the thoughts and knowledge of the students make difference according to their earthquake experience?
4. Do the students thoughts and knowledge makes change according to their school type?

METHOD

Method of the Study

At the study, selecting research sample, in terms of data collection and analysis was designed as a pure quantitative.

In the study, survey method is used. Survey models are a way of research which aims to describe a situation as it is in the past and present. The subject of the research is examined in the natural conditions as it is (Karasar,2007). In the research, singular survey model and comparative survey model are used together.

Universe and Sampling of the Study

The universe of the study is determined as all 1st grade students of the high schools in İzmit, Kocaeli.

While sampling groups from these students, cluster sampling, one of the probability-based sampling method is used. Cluster sampling is used when there is a naturally exist groups, artificially created groups for specific aims or created artificially for different aims but it shows similarity inside in few aspects (Yıldırım & Şimşek; 2006).

To strengthen the reflection ability of the groups, students are chosen from the different schools; Anatolian, general and technical high schools. In İzmit, the total high school students and their dispersion to the school types and their percentage are determined. The number of the participants and their dispersion to the school types are determined. From the school which has got more students, more students are taken as participants. As 10%of the high school students in İzmit, 555 students are chosen as participant.

For a pilot study before the research 390 students are examined, to test reliability and validity of the data collection device the pilot study is used.

Data Collection Tool and Data Collection

The data collection tool is taken with her permission from Oğuz (2005)'s degree doctor of philosophy thesis, which she used to measure the students' existing knowledge. But all the questions that Oğuz (2005) used in his research are not taken in this study. This tool is changed with Oğuz (2005)'s permission owing to the participants' ages are different from her study and passing time between two studies. The questions, students related the earthquake with air conditions, beliefs, earthquake predictions, and what to do after and before the earthquake are chosen and asked in different ways in the pilot study, too.

To test if the questions chosen for every headline is in the same dimension; by using SPSS programme exploratory factor analyze is done. Some of the questions are eliminated to gather all the questions under a headline. The research has 7 headlines and 37 questions. The students' thoughts about earthquake will be analyzed under every headline. reliability and validity study was repeated, owing to passing time between two studies. A pilot study was done with 390 students except the sampling of this study. As a result of this pilot study some of the items were removed from the data collection tool. With the remaining items, Cronbach's alphas for internal consistency were 0.78 collected data was analyzed with SPSS 15.program. This means the tool is reliable.

When the tool finally designed, in 2008-2009 education year at first semester the data collection tool applied to the participants.

Data Analysis

In the research, the questions under 7 headlines analyzed with descriptive analysis method. In this analysis, the students answers to the items of data collection tool ('I

agree, don't agree, I have no idea') percentage (%) and frequency (f) is counted. The reason of using this type of analyzing method is to determine misunderstanding in their knowledge and thoughts.

In the data collection tool, in seven headlines for every item according to the gender, earthquake experience and school type chi-square test used. The reason of the usage of this test is categorizing of the variable in data collection tool.

FINDINGS

In this part findings for every sub-problem of the study will be given one by one.

What are the thoughts and knowledge about earthquakes of the students at first grade in high schools?

The students' thoughts and beliefs are limited under seven dimensions and for every answer frequency and percentage values are counted and placed in Table 1. When we consider the answer about air conditions, students have generally true beliefs and knowledges about it. Answer about the foggy weather has the highest percentage which is true and 537 students (96, 8%) answered that foggy weather doesn't cause earthquakes. Answer about the earthquakes cause hot weather, 24% of the students agreed and 21, 3% of them have no idea. This item leads students into error and became the mostly wrong answered item. Items about movements are answered right by 448 (80,7 %) students that they do not agree, earthquake is a kind of hurricane. 37,3 % of the students agreed that earthquake is a loud noise coming from deep earth's crust and this is the students mostly wrong answered item. 55% of the students thought earthquakes cause dogs bark before they happen. 48,1% of them thought scientists can predict the earthquakes before and 56,6% of them thought animals can predict earthquakes before. 64% of the students do not agree the question that human beings can feel the earthquake before and this is the highly right answered item.

Table 1. Frequency and percentage values of the items in seven dimensions.

Air conditions		**Agree**		**No idea**		**Don't agree**	
	N	**%**	**f**	**%**	**f**	**%**	**f**
Earthquakes are caused by hot weather.	554	6,7	37	10,3	57	82,9	460
Earthquakes are caused by rain.	555	2,3	13	2,7	15	95,0	527
Earthquakes are caused by thunder.	555	1,1	6	4,0	22	95,0	527
Earthquakes are caused by foggy weather.	555	1,1	6	2,2	12	96,8	537
Earthquakes can cause heavy rains	555	5,9	33	13,7	76	80,4	446
Earthquakes can raise the temperature.	555	24,5	136	21,3	118	54,2	301
Movements							
Earthquakes are an eruption	555	15,0	83	20	111	65	361
Earthquakes are deep noises coming from under the ground.	555	37,3	207	21,1	117	41,6	231
Earthquakes are a kind of tornado.	554	7,7	43	11,4	63	80,7	448

Earthquakes are caused by tides.	554	15,0	83	18,2	101	66,7	370
Earthquakes are caused by bad vibrations.	555	18,0	100	19,5	108	62,5	347
Earthquakes can make the Earth turn faster.	555	7,2	40	13,7	76	79,1	439
Earthquake prediction							
Earthquakes can cause dog to bark just before it happens.	555	55,0	305	19,8	110	25,2	140
Scientist can accurately predict earthquakes before it happens.	555	48,1	267	30,3	168	21,6	120
Animals can predict earthquakes.	553	56,6	314	22,3	124	20,7	115
Some people can sense that earthquakes are about the happen.	554	14,2	79	21,6	120	64,0	355
Belief							
Earthquakes are the hand of God to punish people.	555	16,0	89	10,6	59	73,3	407
Earthquakes are disasters that happen only at night.	555	4,7	26	4,7	26	90,6	503
Earthquakes occur in winter.	555	1,6	9	2,7	15	95,7	531
Earthquakes are caused by construction workers taking down a building.	555	4,7	26	4,7	26	90,6	503
Earthquakes are caused by nuclear testing.	555	10,3	57	18,9	105	70,8	393
Before earthquake							
Before earthquakes we should build earthquakes resistance wood frame houses.	555	53,7	298	16,6	92	29,7	165
Before earthquakes we should move heavy objects to high shelves because heavy objects can withstand quakes.	555	16,0	89	11,2	62	72,8	404
Before earthquakes we should fasten water heaters to the wall.	555	37,5	208	19,1	106	43,4	241
Before earthquakes we should place beds by large windows so one can easily escape from the window.	555	11,2	62	9,2	51	79,6	442
Scientists can predict earthquakes.	555	30,6	17	36,	20	33,3	185

			0	0	0		
During earthquake							
During earthquakes we should hold on to something metal.	555	15,9	88	25,2	140	58,9	327
During earthquakes we should get under something sturdy like table or desk.	555	74,8	415	8,5	47	16,8	93
During earthquakes we should get near a window.	555	9,5	53	12,8	71	77,7	431
During earthquakes we should stand under a tree.	555	8,6	48	14,8	82	76,6	425
During earthquakes if you find yourself indoors it is usually better to rush outside.	555	18,6	103	18,0	100	63,4	352
After earthquake							
Earthquakes can move faults.	555	85,2	473	8,3	46	6,5	36
Earthquakes can make people have trouble walking.	555	54,2	301	19,6	109	26,1	145
Earthquakes can change the water level in wells.	555	61,8	343	22,7	126	15,5	86
Earthquakes can shape the physical features of land.	555	78,9	438	13,3	74	7,7	43
Earthquakes can damage trees.	555	73,0	405	16,8	93	10,3	57
Earthquakes can damage inside of buildings.	555	88,5	491	6,8	38	4,7	26

95,7% of the students do not agree that earthquakes happen in winter and 70,8 of them agree that nuclear experiments causes earthquakes. Surprisingly 16% of the students think earthquakes are a punishment from god for people. About the questions that are used to check the knowledge of the students about things before earthquake, students do not have true information. To the question which offers to move bed beside the window to run out easily, 442 (79, 6%) students did not agree and this was the most rightly answered question. 30, 6% of the students think scientists can guess the earthquakes before they happen. Students have right information about the things to do during the earthquake. 431 students (77, 7%) think that people shouldn't stay closer to the windows. In the same way, 76, 6% of them do not think that people should stay under a tree during the earthquake. 74, 8% of the students think people should stay under an enduring table or desk during the earthquake. Moreover, 63,4% of the students don't agree that during earthquake people should leave the buildings hastily. On the other hand this question was the most agreed question (18,6%) comparing to the others. The item which offers to hold a metal during an earthquake was the least agreed item. To the questions asking information about after an earthquake was rightly answered by most of the students. 88,5% of the students agreed that earthquakes can damage the inside of the buildings. 54,2% of the students do not agree that earthquakes causes people to walk hardly.

Do the knowledge and thoughts of the students change according to their gender?

The values about whether the knowledge and thoughts of the students change according to their gender is tested by using chi-square test, analyzed and tabled.

Table 2 shows the chi-square analysis of the answers that are given about air conditions.

Table 2. The results of chi-square test for "air condition" items according to their gender.

Items	Gender		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	P
1	F	f	20	27	265	312	2,24	2	,32
		%	6,4	8,7	84,9	100			
	M	f	17	30	195	242			
		%	7,0	12,4	80,6	100			
2	F	f	4	6	303	313	5,37	2	,07
		%	1,3	1,9	96,8	100			
	M	f	9	9	224	242			
		%	3,7	3,7	92,6	100			
3	F	f	1	13	299	313	3,94	2	,14
		%	0,3	4,2	95,5	100			
	M	f	5	9	228	242			
		%	2,1	3,7	94,2	100			
4	F	f	0	6	307	313	8,09	2	,03*
		%	0	1,9	98,1	100			
	M	f	6	6	230	242			
		%	2,5	2,5	95,0	100			
5	F	f	18	43	252	313	,05	2	,98
		%	5,8	13,7	80,5	100			
	M	f	15	33	194	242			
		%	6,2	13,6	80,2	100			
6	F	f	87	60	166	313	4,84	2	,09
		%	27,8	19,2	53,0	100			
	M	f	49	58	135	242			
		%	20,2	21,3	54,2	100			

When we examine the answers about air conditions related to earthquake, female students' knowledge and thoughts level is higher than male students. On the other hand, this difference only can be seen in 4th question (X^2 (2) = 8, 09; p< .05).

Table 3 shows chi-square analysis of the answers given about movements considering gender factor.

Table 3. The results of chi-square test for "movement" items according to their gender.

Items	Gender		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	P
1	F	f	34	62	217	313	10,08	2	,01**
		%	10,9	19,8	69,3	100			
	M	f	49	49	144	242			
		%	20,2	20,2	59,5	100			
2	F	f	120	74	119	313	4,68	2	,96
		%	38,3	23,6	38,0	100			
	M	f	87	43	112	242			
		%	36,0	17,8	46,3	100			
3	F	f	21	41	251	313	2,95	2	,23
		%	6,7	13,1	80,2	100			
	M	f	22	22	197	241			
		%	9,1	9,1	81,7	100			
4	F	f	47	59	206	312	,25	2	,88
		%	15,1	18,9	66,0	100			
	M	f	36	42	164	242			
		%	14,9	17,4	67,8	100			
5	F	f	56	56	201	313	1,24	2	,54
		%	17,9	17,9	64,2	100			
	M	f	44	52	146	242			
		%	18,2	21,5	60,3	100			
6	F	f	20	42	251	313	,81	2	,67
		%	6,4	13,4	80,2	100			
	M	f	20	34	188	242			
		%	8,3	14,0	77,7	100			

Generally, female students' knowledge level, considering their gender related to the answers about movements, is higher than male students. But this can only be seen in 1st answer (X^2 (2) = 10, 08; p< .05).

Table 4 The chi-square analysis results of the questions considering gender.

Table 4. The results of chi-square test for "earthquake prediction" items according to their gender.

Items	gender		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	F	f	172	56	85	313	2,41	2	,30
		%	55,0	17,9	27,2	100			
	M	f	133	54	55	242			
		%	22,7	22,3	55,0	100			
2	F	f	164	90	59	313	5,84	2	,054
		%	52,4	28,8	18,8	100			
	M	f	103	78	61	242			
		%	42,6	32,2	25,2	100			
3	F	f	169	72	70	311	1,92	2	,38
		%	54,3	23,2	22,5	100			
	M	f	145	52	45	242			
		%	59,9	21,5	18,6	100			
4	F	f	40	77	195	312	4,32	2	,12
		%	12,8	24,7	62,5	100			
	M	f	39	43	160	242			
		%	16,1	17,8	66,1	100			

Examining the answers given to 3rd question both male and female students by choosing 'I agree' answered wrong. At 2nd question shows male students have higher knowledge than female students at this subject. Moreover, from the students who answer 3rd question, female students' knowledge level is higher than male students but the difference between these answers have no meaningful relationship considering the students gender (X^2 (2)= 4.32; p>.05).

Table 5 shows chi-square analysis of the questions about beliefs. Considering the answers given about beliefs female students' knowledge level is higher than male students. The difference was meaningful at 1st question (X^2 (2)= 25,25; p< .01), at 3rd question (X^2 (2)= 11,18; p< .01), and 4th question (X^2 (2)= 12,32; p< .01).

Table 5. The results of chi-square test for "beliefs" items according to their gender.

Items	Gender		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	F	f	32	26	255	313	25,25	2	,00**
		%	10,2	8,3	81,5	100			
	M	f	57	33	152	242			
		%	23,6	13,6	62,8	100			
2	F	f	11	13	289	313	2,76	2	,25
		%	3,5	4,2	92,3	100			

	M	f	15	13	214	242			
		%	6,2	5,4	88,4	100			
3	F	f	1	5	307	313	11,18	2	,00**
		%	,3	1,6	98,1	100			
	M	f	8	10	224	242			
		%	3,3	4,1	92,6	100			
4	F	f	11	7	295	313	12,32	2	,00**
		%	3,5	2,2	94,2	100			
	M	f	15	19	208	242			
		%	6,2	7,9	86,0	100			
5	F	f	30	59	224	313	,39	2	,82
		%	9,6	18,8	71,6	100			
	M	f	27	46	169	242			
		%	11,2	19,0	69,8	100			

Table 6 shows chi-square analysis of the questions about before earthquake considering the students' gender.

Table 6. The results of chi-square test for "before earthquake" items according to their gender.

Items	Gender		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	F	f	176	46	91	313	2,50	2	,29
		%	56,2	14,7	29,1	100			
	M	f	122	46	74	242			
		%	50,4	19,0	30,6	100			
2	F	f	41	29	243	313	8,51	2	,014
		%	13,1	9,3	77,6	100			
	M	f	48	33	161	242			
		%	19,8	13,6	66,5	100			
3	F	f	121	64	128	313	2,01	2	,367
		%	38,7	20,4	40,9	100			
	M	f	87	42	113	242			
		%	36,0	17,4	46,7	100			
4	F	f	21	23	269	313	19,02	2	,00**
		%	6,7	7,3	86	100			

	M	f	41	28	173	242			
		%	16,9	11,6	71,5	100			
5	F	f	96	122	95	313	3,63	2	,16
		%	30,7	39,0	30,4	100			
	M	f	74	78	90	242			
		%	30,6	32,2	37,2	100			

Considering their gender and answers at 2nd answer (X^2 (2)= 8,51; p< .05) and 4th answer (X^2 (2)= 19,02; p< .01) female students' knowledge level is higher than male students. On the other hand, surprisingly at 1st ,3rd and 5th answers male students' knowledge level is higher than female students.

Table 7 shows chi-square analysis about during earthquake questions considering students' gender.

Table 7. The results of chi-square test for "during earthquake" items according to their gender.

Items	Gender		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	F	f	43	73	197	313	5,03	2	,08
		%	13,7	23,3	62,9	100			
	M	f	45	67	130	242			
		%	18,6	27,7	53,7	100			
2	F	f	228	28	57	313	1,46	2	,48
		%	72,8	8,9	18,2	100			
	M	f	187	19	36	242			
		%	77,3	7,9	14,9	100			
3	F	f	19	27	267	313	24,24	2	,00**
		%	6,1	8,6	85,3	100			
	M	f	34	44	164	242			
		%	14,0	18,2	67,8	100			
4	F	f	17	40	256	313	13,07	2	,00**
		%	5,4	12,8	81,8	100			
	M	f	31	42	169	242			
		%	12,8	17,4	69,8	100			
5	F	f	39	50	224	313	23,55	2	,00**
		%	12,5	16,0	71,6	100			
	M	f	64	50	128	242			
		%	26,4	20,7	52,9	100			

Generally, female students gave right answers to these items, but the difference at 3rd (X^2 (2)= 24,24; p< .01), and 4th (X^2 (2)= 13,07; p< .01) and 5th (X^2 (2)= 23,55 ; p< .01) item is meaningful.

Table 8 shows chi-square test results about after earthquake considering the students' gender.

Table 8. The results of chi-square test for "after earthquake" items according to their gender.

Items	Gender		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	F	f	272	23	18	313	1,60	2	,45
		%	86,9	7,3	5,8	100			
	M	f	201	23	18	242			
		%	83,1	9,5	7,4	100			
2	F	f	180	52	81	313	4,78	2	,09
		%	57,5	16,6	25,9	100			
	M	f	121	57	64	242			
		%	50,0	23,6	26,4	100			
3	F	f	199	63	51	313	2,76	2	,25
		%	63,6	20,1	16,3	100			
	M	f	144	63	35	242			
		%	59,5	26,0	14,5	100			
4	F	f	255	37	21	313	2,82	2	,24
		%	81,5	11,8	6,7	100			
	M	f	183	37	22	242			
		%	75,6	15,3	9,1	100			
5	F	f	240	48	25	313	5,86	2	,053
		%	76,7	15,3	8,0	100			
	M	f	165	45	32	242			
		%	68,2	18,6	13,2	100			
6	F	f	276	25	12	313	2,48	2	,29
		%	88,2	8,0	3,8	100			
	M	f	215	13	14	242			
		%	88,8	5,4	5,8	100			

At first 5 items considering the students gender, female students knowledge level is higher than male students but at 6th item male students' level is higher. On the other hand, students' answers according to their gender are not meaningful in all items.

Do the thoughts and knowledge of the students make difference according to their earthquake experience?

Students' knowledge and ideas are tested and analyzed with chi-square test. This test includes seven heads and applied in order to find the impact of a previous experience and it is schematized.

Table 9 shows the result of chi-square analysis with the factor of a previous experience on the items about air conditions.

Table 9. The results of chi-square test examining the impact of a previous experience upon their responds.

Items	experience an earthquake		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	Yes	f	32	46	390	468	,77	2	,68
		%	6,8	9,8	83,3	100			
	No	f	5	11	70	86			
		%	5,8	12,8	81,4	100			
2	Yes	f	12	12	445	469	,84	2	,66
		%	2,6	2,6	94,9	100			
	No	f	1	3	82	86			
		%	1,2	3,5	95,3	100			
3	Yes	f	4	18	447	469	1,62	2	,45
		%	,9	3,8	95,3	100			
	No	f	2	4	80	86			
		%	2,3	4,7	93,0	100			
4	Yes	f	6	12	451	469	3,41	2	,18
		%	1,3	2,6	96,2	100			
	No	f	0	0	86	86			
		%	0	0	100	100			
5	Yes	f	30	64	375	469	1,10	2	,58
		%	6,4	13,6	80,0	100			
	No	f	3	12	71	86			
		%	3,5	14,0	82,6	100			
6	Yes	f	118	99	252	469	,71	2	,70
		%	25,2	21,1	53,7	100			
	No	f	18	19	49	86			
		%	20,9	22,1	57,0	100			

When students' results are examined with the factor of a previous experience, the rate of the correct answers in first and third items are given more by the people who have experienced a big earthquake before than those who have never had such an experience before (p> .05). The other group who have not experienced an earthquake before have more correct knowledge and ideas in second, forth, fifth and sixth items than the experienced ones. However the difference is not significant for any items (p> .05).

Table 10 shows the result of chi-square test searching for the impact of their experiencing an earthquake before in terms of "movement".

Table 10. The results of chi-square test for "movement" items according to their previous experiences.

Items	experience an earthquake		Agree	No idea	Don't agree	Total	Chi-square testi		
							X^2	sd	p
1	Yes	f	73	88	308	469	3,25	2	,20
		%	15,6	18,8	65,7	100			
	No	f	10	23	53	86			
		%	11,6	26,7	61,6	100			
2	Yes	f	176	94	199	469	2,07	2	,37
		%	42,4	20,0	37,5	100			
	No	f	31	23	32	86			
		%	36,0	26,7	37,2	100			
3	Yes	f	34	52	382	469	1,35	2	,51
		%	7,3	11,1	81,6	100			
	No	f	9	11	66	86			
		%	10,5	12,8	76,7	100			
4	Yes	f	74	83	311	468	1,84	2	,40
		%	15,8	17,7	66,5	100			
	No	f	9	18	59	86			
		%	10,5	20,9	68,6	100			
5	Yes	f	87	93	289	469	1,08	2	,58
		%	18,6	19,8	61,6	100			
	No	f	13	15	58	86			
		%	15,1	17,4	67,4	100			
6	Yes	f	30	62	377	469	3,89	2	,14
		%	6,4	13,2	80,4	100			
	No	f	10	14	62	86			
		%	11,6	16,3	72,1	100			

When students' results are examined with the factor of a previous experience, the rate of the correct answers in first, second, third and sixth items which are about movement are given more by the people who have experienced a big earthquake before than those who have never had such an experience before (p>.05). However, this difference is not significant (p>.05).

Table 11 shows the result of chi-square test searching for the impact of their experiencing an earthquake before in terms of "earthquake prediction".

Table 11. The results of chi-square test for "earthquake prediction" items according to their previous experiences.

Items	experience an earthquake		Agree	No idea	Don't agree	Total	Chi-square testi		
							X^2	sd	p
1	Yes	f	226	142	101	469	,014	2	,99
		%	48,2	30,3	21,5	100			
	No	f	41	26	19	86			
		%	47,7	30,2	22,1	100			
2	Yes	f	260	107	100	467	1,52	2	,47
		%	55,7	22,9	21,4	100			
	No	f	54	17	15	86			
		%	62,8	19,8	17,4	100			
3	Yes	f	57	100	311	468	11,92	2	,00**
		%	12,2	21,4	66,5	100			
	No	f	22	20	44	86			
		%	25,6	23,3	51,2	100			
4	Yes	f	252	89	128	469	7,04	2	,03*
		%	53,7	19,0	27,3	100			
	No	f	53	21	12	86			
		%	61,6	24,4	14,0	100			

When their responds are associated with their experiences, the turnout is more for the experienced students than the others in second, third and forth items. On the other hand this difference is significant only for third (X^2 (2) =11,92;p<.01) and forth (X^2 (2) =7,04; p<.05). The inexperienced students are more only for the first item. However this difference is not significant (X^2 (2) =0, 14; p>.05).

Table 12 shows the result of chi-square test searching for the impact of their experiencing an earthquake before in terms of "belief".

Table 12. The results of chi-square test for "belief" items according to their previous experiences.

Items	experience an earthquake	Agree	No idea	Don't agree	Total	Chi-square testi		
						X^2	sd	p

1	Yes	f	73	54	342	469	2,71	2	,26
		%	15,6	11,5	72,9	100			
	No	f	16	5	65	86			
		%	18,6	5,8	75,6	100			
2	Yes	f	24	20	425	469	2,35	2	,31
		%	5,1	4,3	90,6	100			
	No	f	2	6	78	86			
		%	2,3	7,0	90,7	100			
3	Yes	f	8	12	449	469	,366	2	,83
		%	1,7	2,6	95,7	100			
	No	f	1	3	82	86			
		%	1,2	3,5	95,3	100			
4	Yes	f	22	19	428	469	2,72	2	,26
		%	4,7	4,1	91,3	100			
	No	f	4	7	75	86			
		%	4,7	8,1	87,2	100			
5	Yes	f	49	89	331	469	,122	2	,94
		%	10,4	19,0	70,6	100			
	No	f	8	16	62	86			
		%	9,3	18,6	72,1	100			

When students' results are examined with the factor of a previous experience, the rate of the correct answers in third and forth items which are about movement are given more by the people who have experienced a big earthquake before than those who have never had such an experience before. However this difference is not significant for non items (p>. 05). For the first second and fifth items, the people who have experienced a big earthquake before have more correct knowledge and ideas than those who have never had such an experience before (p>.05).

Table 13 the results of chi-square test for "before an earthquake" items searching for the impact their previous experiences.

Table 13. The results of chi-square test for "before an earthquake" items according to their previous experiences.

Items	experience an earthquake		Agree	No idea	Don't agree	Total	Chi-square tes		
							X^2	sd	P
1	Yes	f	252	77	140	469	,06	2	,97
		%	53,7	16,4	29,9	100			
	No	f	46	15	25	86			
		%	53,5	17,4	29,1	100			

2	Yes	f	72	53	344	469	1,06	2	,59
		%	15,4	11,3	73,3	100			
	No	f	17	9	60	86			
		%	19,8	10,5	69,8	100			
3	Yes	f	179	84	206	469	2,80	2	,25
		%	38,2	17,9	43,9	100			
	No	f	29	22	35	86			
		%	33,7	25,6	40,7	100			
4	Yes	f	53	42	374	469	,23	2	,89
		%	11,3	9,0	79,7	100			
	No	f	9	9	68	86			
		%	10,5	10,5	79,1	100			
5	Yes	f	135	175	159	469	5,03	2	,08
		%	28,8	37,3	33,9	100			
	No	f	35	25	26	86			
		%	40,7	29,1	30,2	100			

According to their past experiences , the people who have experienced a big earthquake have more correct knowledge and experiences than those who have never had such an experience before. However the difference is not significant for any items (p>.05).

Table 14 the results of chi-square test for "during an earthquake" items searching for the impact their previous experiences.

Table 14.The results of chi-square test for "during an earthquake" items according to their previous experiences.

Items	Experience an earthquake		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	Yes	f	73	114	282	469	1,92	2	,38
		%	15,6	24,3	60,1	100			
	No	f	15	26	45	86			
		%	17,4	30,2	52,3	100			
2	Yes	f	351	41	77	469	,48	2	,79
		%	74,8	8,7	16,4	100			
	No	f	64	6	16	86			
		%	74,4	7,0	18,6	100			
3	Yes	f	46	58	365	469	,65	2	,72
		%	9,8	12,4	77,8	100			

	No	f	7	13	66	86			
		%	8,1	15,1	76,7	100			
4	Yes	f	41	67	361	469	,58	2	,75
		%	8,7	14,3	77,0	100			
	No	f	7	15	64	86			
		%	8,1	17,4	74,4	100			
5	Yes	f	87	76	306	469	7,11	2	,03*
		%	18,6	16,2	65,2	100			
	No	f	16	24	46	86			
		%	18,6	27,9	53,5	100			

When students' results are examined with the factor of a previous experience, though the rate of the correct answers in first, third, forth and fifth items are given more by the people who have experienced a big earthquake before than those who have never had such an experience before, the difference is significant for only the fifth item (X^2 (2) =7,11;p<.05). In the second item the people who have not experienced respond more correctly than those who have had an experience before. However it is not significant, as well (X^2 (2) = ,48; p<.05)

Table 15 the results of chi-square test for " after an earthquake" items searching for the impact their previous experiences.

Table 15. The results of chi-square test for "after an earthquake" items according to their previous experiences.

Items	experience an earthquake		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	Yes	f	400	39	30	469	,04	2	,98
		%	85,3	8,3	6,4	100			
	No	f	73	7	6	86			
		%	84,9	8,1	7,0	100			
2	Yes	f	255	90	124	469	,44	2	,80
		%	54,4	19,2	26,4	100			
	No	f	46	19	21	86			
		%	53,5	22,1	24,4	100			
3	Yes	f	288	105	76	469	1,19	2	,55
		%	61,4	22,4	16,2	100			
	No	f	55	21	10	86			
		%	64,0	24,4	11,6	100			
4	Yes	f	367	64	38	469	,88	2	,64
		%	78,3	13,6	8,1	100			

	No	f	71	10	5	86			
		%	82,6	11,6	5,8	100			
5	Yes	f	341	79	49	469	,14	2	,93
		%	72,7	16,8	10,4	100			
	No	f	64	14	8	86			
		%	74,4	16,3	9,3	100			
6	Yes	f	413	31	25	469	3,00	2	,22
		%	88,1	6,6	5,3	100			
	No	f	78	7	1	86			
		%	90,7	8,1	1,2	100			

According to their previous experiences, when the results are examined it is observed that for first, second and sixth items are responded correctly by the people who have experienced an earthquake before than the inexperienced ones. However, the difference is not significant (p>.05). Third, forth and fifth items the people who have not experienced respond more correctly than those who have had an experience before. However it is not significant, as well (p>.05)

Do the thoughts and knowledge of the students make difference according to their school type?

Whether school kinds change students' knowledge and ideas or not is examined with chi-square test. This test includes seven heads and the points gotten from these heads are examined and schematized.

Table 16 shows the results of "air conditions" according to their responds and while doing this their schools are examined.

Table 16. The results of chi-square test for "air conditions" items according to their school type.

Items	School kind		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	P
1	Anatolian	f	3	4	71	78	10,83	4	,03*
		%	3,8	5,1	91,0	100			
	General	f	8	14	154	176			
		%	4,5	8,0	87,5	100			
	Technical-Vocatioanal	f	26	39	235	300			
		%	8,7	13,0	78,3	100			
2	Anatolian	f	1	2	75	78	1,73	4	,79
		%	1,3	2,6	96,2	100			
	General	f	3	6	168	177			
		%	1,7	3,4	94,9	100			

	Technical - Vocatianal	f	9	7	284	300			
		%	3,0	2,3	94,7				
3	Anatolian	f	6	4	68	78	4,31	4	,37
		%	7,6	5,1	87,3	100			
	General	f	7	4	166	177			
		%	4,3	2,3	93,6	100			
	Technical - Vocatianal	f	5	14	281	300			
		%	1,7	4,7	93,7	100			
4	Anatolian	f	7	1	70	78	3,44	4	,49
		%	9,1	1,3	89,6	100			
	General	f	165	6	6	177			
		%	93,2	3,4	3,4	100			
	Technical-Vocatianal	f	3	5	292	300			
		%	1,0	1,7	97,3	100			
5	Anatolian	f	2	7	69	78	4,94	4	,29
		%	2,6	9,0	88,5	100			
	General	f	10	23	144	177			
		%	5,6	13,0	81,4	100			
	Technical - Vocatianal	f	21	46	233	300			
		%	7,0	15,3	77,7	100			
6	Anatolian	f	18	14	46	78	7,83	4	,10
		%	23,1	17,9	59,0	100			
	General	f	33	37	107	177			
		%	18,6	20,9	60,5	100			
	Technical - Vocatianal	f	85	67	148	300			
		%	28,3	22,3	49,3	100			

According to their schools, for first, second and fifth items Anatolian High Schools, for third and forth items Technical and Vocational High Schools and for sixth item General High Schools are more successful. However it is found that the difference between the schools is not so significant.

Table 17 shows the result of items in terms of "movement".

Table 17. The results of chi-square tests for "movement" items which examines the impact of school kinds.

Items	School kind	Agree	No idea	Don't agree	Total	Chi-square test		
						X^2	sd	p

1	Anatolian	f	12	25	41	78	9.72	4	,04*
		%	15,4	32,1	52,6	100			
	General	f	30	31	116	177			
		%	16,9	17,5	65,5	100			
	Technical-Vocatianal	f	41	55	204	300			
		%	13,7	18,3	68,0	100			
2	Anatolian	f	22	15	41	78	12,08	4	,02*
		%	28,2	19,2	52,6	100			
	General	f	55	44	78	177			
		%	31,1	24,9	44,1	100			
	Technical-Vocatianal	f	130	58	112	300			
		%	43,3	19,3	37,3	100			
3	Anatolian	f	4	9	65	78	2,99	4	,56
		%	5,1	11,5	83,3	100			
	General	f	11	18	148	177			
		%	6,2	10,2	83,6	100			
	Technical-Vocatianal	f	28	36	235	299			
		%	9,4	12,0	78,6	100			
4	Anatolian	f	6	11	61	78	8,34	4	,08
		%	7,7	14,1	78,2	100			
	General	f	23	32	121	176			
		%	13,1	18,2	68,8	100			
	Technical-Vocatianal	f	54	58	188	300			
		%	18,0	19,3	62,7	100			
5	Anatolian	f	13	12	53	78	7,87	4	,10
		%	16,7	15,4	67,9	100			
	General	f	30	26	121	177			
		%	16,9	14,7	68,4	100			
	Technical-Vocatianal	f	57	70	173	300			
		%	19,0	23,3	57,7	100			
6	Anatolian	f	4	5	69	78	6,85	4	,14
		%	5,1	6,4	88,5	100			
	General	f	17	26	134	177			
		%	9,6	14,7	75,7	100			

	Technical-Vocatianal	f	19	45	236	300
		%	6,3	15,0	78,7	100

According to their current schools, when their responds are examined, though for second, forth and sixth items Anatolian High School students are more successful than those who are in Technical-Vocatianal and General High Schools, it is found that the difference is significant only for the second item (X^2 (4) =9,72; p<.05). Also at third and fifth items, General High Schools students have more accurate information than the others but it has been identified that the difference is not significant statistically.

Table 18 shows the chi-square test in terms of "earthquake prediction" according to their schools.

Table 18. The results of chi-square tests for "earthquake prediction" items which examines the impact of school kinds.

Items	School kind		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	Anatolian	f	46	12	20	78	1,28	4	,87
		%	59,0	15,4	25,6	100			
	General	f	97	35	45	177			
		%	54,8	19,8	25,4	100			
	Technical-Vocatianal	f	162	63	75	300			
		%	54,0	21,0	25,0	100			
2	Anatolian	f	30	25	23	78	5,33	4	,25
		%	38,5	32,1	29,5	100			
	General	f	89	49	39	177			
		%	50,3	27,7	22,0	100			
	Technical-Vocatianal	f	148	94	58	300			
		%	49,3	31,3	19,3	100			
3	Anatolian	f	40	15	22	77	6,336	4	,17
		%	51,9	19,5	28,6	100			
	General	f	101	35	41	177			
		%	57,1	19,8	23,2	100			
	Technical-Vocatianal	f	173	74	52	299			
		%	57,9	24,7	17,4	100			
4	Anatolian	f	9	7	61	77	13,97	4	,00**
		%	11,7	9,1	79,2	100			
	General	f	22	36	119	177			
		%	12,4	20,3	67,2	100			

	Technical-Vocatianal	f	48	77	175	300
		%	16,0	25,7	58,3	100

According to their current schools, when their responds are examined, although Anatolian High School students are more successful than Technical-Vocatianal and General High Schools, it is found that only the forth item is significant (X^2 (4) =13,97; p<.05).

Table 19 shows the chi-square test in terms of "belief" items according to their schools.

Table 19. The results of chi-square tests for "belief" items which examines the impact of school kinds.

Items	School kind		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	Anatolian	f	5	7	66	78	13,51	4	,00**
		%	6,4	9,0	84,6	100			
	General	f	26	13	138	177			
		%	14,7	7,3	78,0	100			
	Technical-Vocatianal	f	58	39	203	300			
		%	19,3	13,0	67,7	100			
2	Anatolian	f	0	1	77	78	9,413	4	,052
		%	,0	1,3	98,7	100			
	General	f	13	9	155	177			
		%	7,3	5,1	87,6	100			
	Technical-Vocatianal	f	13	16	271	300			
		%	4,3	5,3	90,3	100			
3	Anatolian	f	1	0	77	78	5,41	4	,25
		%	1,3	,0	98,7	100			
	General	f	4	3	170	177			
		%	2,3	1,7	96,0	100			
	Technical-Vocatianal	f	6	10	284	300			
		%	1,9	4,4	94,7	100			
4	Anatolian	f	3	0	75	78	8,812	4	,07
		%	3,8	,0	96,2	100			
	General	f	5	7	165	177			
		%	2,8	4,0	93,2	100			
	Technical-Vocatianal	f	18	19	263	300			
		%	6,0	6,3	87,7	100			
5	Anatolian	f	5	14	59	78	4,35	4	,36

		%	6,4	17,9	75,6	100
	General	f	20	27	130	177
		%	11,3	15,3	73,4	100
	Technical - Vocatianal	f	32	64	204	300
		%	10,7	21,3	68,0	100

Although Anatolian High School students are more successful than Technical-Vocatianal and General High Schools, it is found that only the first item is significant (X^2 (4)=13,51; p <.05).

Table 20 shows the chi-square test for the items on "before an earthquake".

Table 20. The results of chi-square tests "before an earthquake" which examines the impact of school kinds.

Items	School kind		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	Anatolian	f	30	15	33	78	11,40	4	,02*
		%	38,5	19,2	42,3	100			
	General	f	98	24	55	177			
		%	55,4	13,6	31,1	100			
	Technical-Vocatianal	f	170	53	77	300			
		%	56,7	17,7	25,7	100			
2	Anatolian	f	3	8	67	78	22,05	4	,00**
		%	3,8	10,3	85,9	100			
	General	f	45	18	114	177			
		%	25,4	10,2	64,4	100			
	Technical-Vocatianal	f	41	36	223	300			
		%	13,7	12,0	74,3	100			
3	Anatolian	f	42	14	22	78	14,50	4	,00**
		%	53,8	17,9	28,2	100			
	General	f	70	34	73	177			
		%	39,5	19,2	41,2	100			
	Technical-Vocatianal	f	96	58	146	300			
		%	32,0	19,3	48,7	100			
4	Anatolian	f	3	1	74	78	14,1	4	,00**

		%	3,8	1,3	94,9	100	2		
	General	f	25	19	133	177			
		%	14,1	10,7	75,1	100			
	Technical-Vocatianal	f	34	31	235	300			
		%	11,3	10,3	78,3	100			
5	Anatolian	f	24	21	33	78	6,86	4	,14
		%	30,8	26,9	42,3	100			
	General	f	53	74	50	177			
		%	29,9	41,8	28,2	100			
	Technical - Vocatianal	f	93	105	102	300			
		%	31,0	35,0	34,0	100			

In terms of first second forth and fifth items Anatolian High School students have more correct knowledge and ideas than Technical-Vocatianal and General High Schools while the difference is significant only in first (X^2 (4)=11,40; p<.05) second (X^2 (4)=22,05; p<.01) and forth (X^2 (4)=14,12; p <.01) items. Moreover, in this dimension it is observed that Technical-Vocatianal and General High Schools have more correct ideas and knowledge than Anatolian High School students (X^2 (4)=14,50; p <.01).

Table 21 shows the results of chi-square test for the questions on "during earthquake".

Table 21. The results of chi-square tests "during an earthquake" which examines the impact of school kinds.

Items	School kind		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	Anatolian	f	19	22	37	78	10,30	4	,04*
		%	24,4	28,2	47,4	100			
	General	f	33	40	104	177			
		%	18,6	22,6	58,8	100			
	Technical-Vocatianal	f	36	78	186	300			
		%	12,0	26,0	62,0	100			
2	Anatolian	f	34	4	40	78	83,29	4	,00**
		%	43,6	5,1	51,3	100			
	General	f	134	14	29	177			
		%	75,7	7,9	16,4	100			
	Technical -	f	247	29	24	300			

	Vocatianal	%	82,3	9,7	8,0	100			
3	Anatolian	f	3	3	72	78	11,68	4	,02*
		%	3,8	3,8	92,3	100			
	General	f	19	23	135	177			
		%	10,7	13,0	76,3	100			
	Technical - Vocatianal	f	31	45	224	300			
		%	10,3	15,0	74,7	100			
4	Anatolian	f	4	12	62	78	6,36	4	,17
		%	5,1	15,4	79,5	100			
	General	f	19	18	140	177			
		%	10,7	10,2	79,1	100			
	Technical - Vocatianal	f	25	52	223	300			
		%	8,3	17,3	74,3	100			
5	Anatolian	f	18	9	51	78	3,43	4	,49
		%	23,1	11,5	65,4	100			
	General	f	33	32	112	177			
		%	18,6	18,1	63,3	100			
	Technical - Vocatianal	f	52	59	189	300			
		%	17,3	19,7	63,0	100			

In terms of second, third, forth and fifth items Anatolian High School students have more correct knowledge and ideas than Technical-Vocatianal and General High Schools while the difference is significant only in second (X^2 (4)=83,29; p<.05) third (X^2 (4)=11,68;p <.01). It is also observed that for the first item Technical-Vocatianal High School students have more correct knowledge and ideas (X^2 (4)=10,30;p <.05).

Table 22 shows the results of chi-square test for the questions " after an earthquake"

Table 22. The results of chi-square tests " after an earthquake" items which examines the impact of school kinds.

Items	School kind		Agree	No idea	Don't agree	Total	Chi-square test		
							X^2	sd	p
1	Anatolian	f	68	5	5	78	1,27	4	,87
		%	87,2	6,4	6,4	100			
	General	f	151	13	13	177			

		%	85,3	7,3	7,3	100			
	Technical-Vocatianal	f	254	28	18	300			
		%	84,7	9,3	6,0	100			
2	Anatolian	f	44	13	21	78	2,24	4	,69
		%	56,4	16,7	26,9	100			
	General	f	90	35	52	177			
		%	50,8	19,8	29,4	100			
	Technical - Vocatianal	f	167	61	72	300			
		%	55,7	20,3	24,0	100			
3	Anatolian	f	49	21	8	78	3,42	4	,49
		%	62,8	26,9	10,3	100			
	General	f	104	43	20	177			
		%	58,8	24,3	16,9	100			
	Technical-Vocatianal	f	190	62	48	300			
		%	63,3	20,7	16,0	100			
4	Anatolian	f	64	8	6	78	1,26	4	,87
		%	82,1	10,3	7,7	100			
	General	f	139	26	12	177			
		%	78,5	14,7	6,8	100			
	Technical-Vocatianal	f	235	40	25	300			
		%	78,3	13,3	8,3	100			
5	Anatolian	f	63	6	9	78	7,21	4	,13
		%	80,8	7,7	11,5	100			
	General	f	125	30	22	177			
		%	70,6	16,9	12,4	100			
	Technical-Vocatianal	f	217	57	26	300			
		%	72,3	19,0	8,7	100			
5	Anatolian	f	71	3	4	78	5,79	4	,22
		%	91,0	3,8	5,1	100			
	General	f	149	16	12	177			
		%	84,2	9,0	6,8	100			

Technical-Vocatianal	f	271	19	10	300
	%	90,3	6,3	3,3	100

The results of the analysis of this part showed that there were not significant differences according to school types.

CONCLUSION AND DISCUSSION

While examining the results from seven different dimensions, it is determined that students generally have correct ideas and knowledge due to their responds about the relation between earthquakes and air conditions. The item which suggests earthquakes increases heat is agreed by one of four students while more than one of five students are undecided. This item is the most problematic item compared to the other items. The recent earthquakes have happened during summer seasons. And this can be accepted as the outcome of such an idea. According to the American Geological Research Foundation (USGS) which is the most esteemed foundation on earthquakes, the hot or moist air is not a sign of an earthquake. USGS clears this subject with these statements; "The first theory was given by Aristo in 400 BC which suggested that there is a correlation between heat and earthquakes". His argument is that hot air goes into the huge caves and this creates high pressure, thus large motions occur on the earth. This means, the air is hot and calm owing to the pressure. However, earthquakes happen under miles of soil in faults. Therefore, seismic activities can not be affected neither by wind nor the hot air. There has not been found a direct interaction between air condition and earthquakes" [2].

It is observed that students generally give correct answers to the second dimension of the items about movement. The students are mostly mistaken about the item of "Earthquake is a huge sound coming from the depth of soil" with thirty seven per cent attending.

Students' correct answers on air conditions and movement item can be the outcome of the eighth grade science and technology lessons' Natural Processes unit. In this unit, they learn what is earthquake, its impacts upon human life and its possible negatory results.

Most of the students give incorrect answers to the other dimensions which are based on the earthquake prediction. It is found that students believe that earthquakes can be predicted by scientists or animals. According to their ideas dogs bark before an earthquake. Here, the problem is that superstitions among society have a great impact upon the children. At school, they do not get correct information about these subjects and so, they get informal knowledge through public. Because of this, during eighth grade, students must be enlightened in science and technology lessons. "To predict a possible earthquake, panel limitations and its movement's characteristics must be determined" (MEB 2005). Though there is a notice in the educational programme, the result is worth of attraction. The news on media after an earthquake is also a factor. The news presents different factors which are not so much reliable for the audience [3; 4]. According to Şimşek (2007)'s researches, students obtain half of their information through television. Therefore, it is suggested that media should not broadcast information about earthquakes which are not scientific.

In terms of beliefs, students give correct answers. The most problematic item is "Earthquakes are the punishment of God." After earthquakes such sayings occurred and this spread quickly among public. In Aydın & Esen (1997)'s study named "the correlation between the high destruction of earthquakes and the concept of destiny", the question of the reasons of earthquakes are responded as holly fate and holly pain by more than half of the subjects. This finding is directly equal to the belief.

Most students were not able to respond to the questions about "before-earthquake items". Students must have talked about what should be done and what precautions should be taken during eighth grade in "We have a guest" unit (MEB 2005). Students who have talked and learned this subject before respond correctly to the items of "during an earthquake". The problematic item is "you must run away as soon as possible". Their most problematic item is "after an earthquake". Though this subject is given with educational programme they are not successful (MEB, 2005).

When we compare the answers according to their genders, female students have given more correct answer than male students. However, both male and female students have made a mistake to the questions "Why do dogs bark just before an earthquake?" and "Are the scientists able to estimate a possible earthquake?" and "Can the animals estimate an earthquake?" which are given as earthquake prediction items. Disaster education is given as another discipline owing to the reconstruction of primary school programmes. In primary schools first and fifth grade class teachers, sixth and eighth grade social sciences teachers and eighth grade science and technology teachers held an important position for the stability of primary school's earthquake education (Öcal, 2007). The students who got information from schools think more scientific. On the other hand, the students who got the information from their families, public, or environment are always mistaken (Tsai,2001). In eighth grade's science and technology lesson, Natural Process Unit, students acquire something and this study is applied to the first grade students who must have been educated before. Despite this, it is interesting that they still have such informal knowledge. This means their traditional learning is going on and they have various conceptive faults (Ross and Shuell, 1993).

Comparing other schools which are applied the same questionnaire; Anatolian High Schools are more successful. It is accepted that education is an important factor both in reducing the nugatory effects of earthquakes and increasing correct knowledge and ideas. Recent studies have shown that education level is an important factor. It is observed that well educated people behave more consciously and they are ready than those who are not educated well in terms of decreasing the damaging results of earthquakes (Rüstemli & Karancı, 1999; 95). According to Hurnen & McClure (1997), well educated people attend the studies of lessening the damages of earthquakes more. Because of these findings, it is possible to decrease the damages through education (As cited Barnett et all, 2006). Similarly, constant education of public will be helpful in case of a disaster (Çelen & Üner, 2002; Taş,2003; Karancı, 2007). In this sense, earthquake and ways of protection subjects should be taught in all levels of primary schools and with different materials and activities (Öcal,2005; Şimşek,2007).

Experiencing an earthquake before generally does not affect their correct knowledge and ideas. According to another research their dark experiences make people don't get more responsibilities (Fişek Okman & Kabasakal, 2008). Moreover, considering this result, while presenting units about these subjects, education programmes must consider various disasters and local disasters. For instance, for Kocaeli, Sakarya and İzmir, earthquakes must be stressed while landslide must be stressed in other places if there is such a risk (Öcal,2007).

RESULTS AND SUGGESTIONS

1. It is determined that girls' ideas and their knowledge about earthquakes are more correct than boys.

2. According to their current school Kinds, the students' ideas and their knowledge who are going to Anatolian schools are more correct compared to the other students who are attending other school types

3. It is determined that students' experiences on earthquakes are not a general condition that affect their ideas and knowledge about earthquakes

4. Beginning from primary school each student should be given a continuous and regular education on earthquakes.

5. To provide a satisfactory education on earthquakes, teachers also should have enough and correct knowledge about earthquakes. For this reason, future teachers in the level of university should be given "disaster education" lectures.

6. To determine science and technology's impact upon students' ideas and knowledge, similar hatching studies may be applied to the upper grades.

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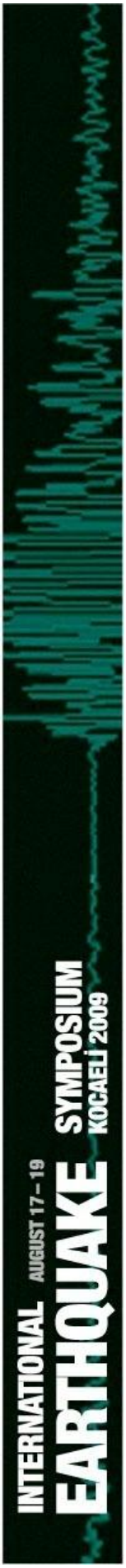
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HUKUK

THE EFFECTS OF EARTHQUAKES ON INCOME TAXES: A LEGAL PERSPECTIVE

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Key Words: Earthquake, income taxes, extension of periods, cancellation of tax

ABSTRACT An earthquake is a phenomenon which shows that Earth maintains its vitality. At the same time, this natural phenomenon is an event with social consequences under certain circumstances. Law regulates some social events according to their importance. One of the branches of law which contains arrangements related to earthquake is tax law.

There are general rules which regulate earthquake event as well as there are specific laws depending on the size and nature of the earthquakes. Within the distinction of adjective tax law and substantive tax law, there are both general and specific rules regarding the earthquakes. These rules aim to remove negative financial effects of earthquake on taxpayers.

Taxes are collected to cover public expenses according to persons' financial situations (powers). Indications of financial situations are incomes, wealths and expenditures of persons. Earthquakes do damages to taxpayer's financial situations. In other words, they reduce these resources. In such situations, it is necessary to abolish or reduce the taxpayers' obligations to tax administration due to economic and social reasons. On the other hand, the taxpayers who are not affected by earthquakes could face higher tax burden due to the increase in public expenses. In this study, normative effects of earthquakes on income taxes are aimed to be examined in Turkey.

INTRODUCTION

An earthquake can generally be expressed as sudden vibration that shakes the Earth's surface[1]. Collapsing inside of the Earth, fault line movements, volcanoes and artifical reasons can cause earthquakes. So, it is clear that an earthquake is primarily a natural event. For a moment, if we suppose that living things did not exist on the Earth, we would think that an earthquake only concerns the planet itself: Earthquakes caused by the interaction between floating planets formed on the layer of magma themselves will perhaps be accepted as an indicator of the liveliness of the planet in its navigation process. However, when it is thought that living things, including the human beings, also live on the planet, we can understand that the movement of the Earth's crust, as a result, is not a simple natural event at all. Besides by shaping the Earth's crust, an earthquake also affects all living species inside and on it. If we put other living beings aside, humans are mentally and physically affected and this situation causes some social consequences.

Law refers to the complete law rules which arrange the relations in society. An event is regarded as a legal event and therefore regulated by law as much as it concerns society. Generally, the events among the private people are the subject of private law, while the events related to state are subject of public law. An earthquake can also be subject of law if it has outcomes that affect society. An earthquake is regarded as a force majeure

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[1] Encyclopaedia Britannica Inc., 1973-1974, ***The New Encyclopaedia Britannica*** ("earthquakes", Helen Hemingway Benton, Publisher, Chicago) p. 68.

which affects the rights and obligations of the parties in a legal relation. In accordance with public order, safety precautions are taken to reduce and abolish the impacts of an earthquake. In this context, the Constitution, which regulates fundamental legal rules, regards natural disasters as a reason to declarate a state of emergency (Const. art. 119, 121, 148).

A tax system can be understood as a structure formed by taxes in force for a certain period of time in a society[2]. Taxes, which are collected in accordance with the persons' financial situations (powers) in order to meet the public expenses, are the most important of the public revenues. As a criterion for financial situation (power), the Turkish Tax System is based on persons' incomes, wealths and expenditures[3]. These issues are regulated in separate laws: Induvidual Income Tax Law (ITL) deals with the indivudials' profits and earnings, while Corporate Income Tax Law (CTL) deals with the incomes of corporations. The Real Estate Tax Law, a kind of wealth taxes, deals with buildings, lands and terrains; Gift and Inheritance Tax Law deals with non-reciprocal acquisitives; Motor Vehicle Tax Law deals with motor vehicles. As an expenses tax, Value Added Tax Law taxes generally subject to deliveries of goods, performances of services and importation of goods and services; Special Consumption Tax Law deals with the first deliveries of some goods.

The income taxes which are the subject of this article, are essentially regulated by two laws: The Induvidual Income Tax Law and Corporate Income Tax Law. The income tax taxes the gross income obtained from the income components (earnings and revenues) declared by law in a calendar year by a real person. The income components are ranked under seven headings in law; commercial incomes, agricultural incomes, wages and salaries, self-employment incomes, revenues from immovable properties, revenues from movable properties, other incomes and revenues (appreciation gains and incidential incomes). The concept of income seems to be closer to the net increase theory considering the other incomes and revenues component however considering the other income components the source theory seems to be more dominant in the ITL[4]. The corporate tax, on the other hand, taxes incomes of the legal entities and joint venture free from being legal entity. Therefore real persons are excluded from corporate tax. The legal entities are capital stock companies, cooperatives, public economic enterprises, and economic enterprises belonging to associations and foundations. The common name of these taxpayers is "corporation" and their income is called "corporate income". Both types of income taxes are based on the distinction of full tax liability and limited tax liability.

The event giving rise to the tax is the event which starts the relation between the state and the taxpayers or the tax responsibles. In other words, the event is the one in which tax laws indicate emerging of the tax receivable (TPL art. 19): This event sometimes occurs with a phemonenon and sometimes with a transaction. Persons are only regarded as taxpayers or tax responsibles as long as they experience the event giving rise to the tax declared by law. From the income taxes' point of view, the event giving rise to the tax is obtaining income for real persons while it is obtaining corporate income for corporates. When a real person begins to obtain an income or when a corporate begins to obtain corporate income a tax relation is founded. The event giving rise to the tax, from the real persons' point of view, is based on the components of income: The component of commercial income is based on commercial activities; the component of agricultral income is based on agricultral activities; the component of income from self employement is based on an any independent activity performed by a person; the wages and salaries are based on a dependent work; the component of revenue from movable

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[2] Turhan, Salih, 1993, ***Vergi Teorisi ve Politikası*** (Filiz Kitabevi, İstanbul) p. 340.
[3] Öncel, Muallâ-Kumrulu, Ahmet-Çağan, Nami, 2008, ***Vergi Hukuku*** (Turhan Kitabevi, Abr., Öncel-Kumrulu-Çağan, Ankara) p. 53.
[4] Öncel-Kumrulu-Çağan, p. 245.

propeties is based on leaving the use of them to the other persons; the component from immovable property and rights are based on leaving the use of them to the other persons; the component of other incomes and revenues, however, are based on accruement of some value increase and temporary perpormance of some income activities. In this way, the total income of real person consist of components of income obtained from various sources. As well as the corporate tax being formed from income components stated in income tax, corperate income essentially is determined with regard to provisions of commercial income (CTL art. 6/2).

Tax relation is maintained within the tax public order. In other words, obligations are given to persons by law[5]. The taxpayers (in its broad sense, the tax obligators[6]) are obliged to obey the duties imposed on them. The aim of these duties is to protect the tax receivable and to protect the public order related to it. Thus, the tax duties, starting before the event giving rise to the tax follow each other. The tax administration is the creditor side of the tax relation and the obligator is the debtor side (i.e. taxpayers, tax responsibles and other persons). The parties, subjects and obligations of parties in this relation are expressed in detail by the law. The tax obligator is required to fulfill all the obligations imposed by the law. In this respect, the Second Book of Tax Procedure Law (TPL) has been reserved for "Taxpayer's Obligators" (TPL art 153- reiterated art. 257)[7]. Tax payment is the substantive obligation, besides the procedural requirements imposed on the tax obligator[8]. When considered from debtor's side, the payment of tax debt, and when considered from creditor's side, the collection of the tax is the ultimate aim of the tax relation. Because, the law, in order to determine the tax receivable correctly and to collect and to protect it, regulates the obligations proposed. The tax office fulfills the tasks proposed by law against the tax obligator. The tax office implements the rules through its powers[9]: These powers are classifed as nondiscretionary and discretionary. Nondiscretiory power is the one in which the administration has right to fulfill an administrative act as indicated in law in advance[10]. Discretionary power is a solution form of implementation which is left to the administration. Administration has a self-determination power to decide about how to act in accordance with public interest[11]. Tax administration possesses some discretionary powers emerging from the law on earthquake as well as other issues.

THE GENERAL EFFECTS OF EARTHQUAKES ON INCOME TAXES

ITS IMPACTS ON ADJECTIVE TAX LAW

Earthquakes are considered as a force majeure which affects the relationship between tax administration and taxpayers or tax responsibles according to their income taxes. Force majeure is an unavoidable event that prevents to fulfill obligations of tax obligator[12]. Earthquake is considered as a force majeure which suspends the time periods

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[5] Mutluer, M. Kâmil, 2006, ***Vergi Genel Hukuku*** (İstanbul Bilgi Üniversitesi Yayınları, İstanbul) p. 126; Since the title of the article 73 of Constitution is "Tax Obligation", it exposes clearly the quality of this relation in terms of taxpayers.

[6] The concept of "tax obligator" has a broad meaning which covers all persons (including civil servants) who are obliged to obey the rules besides taxpayers and tax responsibles. See, Çağan, Nami, 1975, ***Vergi Hukukunda Süreler*** (Ankara Üniversitesi Yayınları, Ankara) p. 43; Öncel-Kumrulu-Çağan, p. 71).

[7] These obligations are stated as to submit declaration, to keep books (to obey the format of books, documents and registration order), to use documents, to submit the declaration of sowing and its account, to take a tax card and to preserve the books, documents and registers.

[8] Öncel-Kumrulu-Çağan, p. 71.

[9] Üstün, Ümit Süleyman, 2007, ***Türk Vergi Hukukunda İdarenin Takdir Yetkisi*** (Turhan Kitabevi, Ankara) p. 14.

[10] Gerçek, Adnan, 2006, ***Türk Vergi Hukukunda İdarenin Takdir Yetkisi*** (Yaklaşım Yayıncılık, Ankara) p. 10; Üstün, p. 16.

[11] Gerçek, p. 13; Üstün, p. 19.

[12] See. Arıca, Mehmet Nadir, 1989, ***Vergi Usul Kanunu*** (Ankara), p. 110; Karakoç, Yusuf, 2007, ***Genel Vergi Hukuku*** (Yetkin Yayınları, Ankara), p. 333; Öncel-Kumrulu-Çağan, p. 118; Şenyüz, Doğan, 2005, ***Vergi Hukuku*** (Genel Hükümler, Ekin Kitabevi, Bursa), p. 59.

in tax law. The effect of this event, in terms of the adjective tax duties, is on time periods. The periods for tax duties do not apply as long as the force majeure countinues; but with the end of the force majeure, the time periods restart[13]. As a result of this, obligations carried out in extended period are considered as valid and legal (TPL art. 13, 15). To regard an event as a force majeure, it must be obstructive for fulfilling the tax duities[14]. Whether the affect of an earthquake is an obstacle to fulfill the tax obligation is decided by the tax office. However, in principle, persons who are affected from force majeure is supposed to prove and certify the event. This is not necessary if the the events are known by the tax administration (TPL art. 15/2). When an event is regarded as a force majeure, the tax administration is obliged to accept that the time period is suspended[15]. The legal extension of time periods is not in administration's discretionary power whereas deciding the event as a force majeure is in administrative discretion. However, the power of administration for an event whether it's regarded as a force majeure or not is an administrative discretion[16].

If the formal tax obligations are not fulfilled in time, due to the force majeure, the time period freezes, but if the obligations are carried out within the extended period, the tax penalties (administrative offences) are regarded as not commited (TPL art. 373). Because a force majeure is legal excuse for tax obligator[17].

Ministry of Finance announced that the earthquake on August 17, 1999 was known by everyone and it declared this event as a force majeure[18]. Some amendments were made in the TPL after this earthquake [19]: Primarily, the Ministry of Finance was authorized to announce the state of force majeure (TPL art. 15/3). This power is regarded as discretionary. In this contex, the administration can announce a force majeure in a specific location (as a province, district or region) and can determine those who cannot fulfill their tax duties (4731. art. 4/a). Later, the coverage of this discretiory power was enlarged as follows: "consolidation of the tax returns (considering tax types and work places), determining new periods to submit tax return and abolition of submitting tax return obligation " (5228. art. 1)[20].

ITS IMPACTS ON MATERIAL TAX OBLIGATIONS

Earthquakes have three effects on the payment of the tax which is accepted as material tax obligation. The first is the extension of tax payment period; the second one is the effect on the tax base and tax receivable, the last one is cancellation of the tax.

1. Extension of the Tax Payment Period

A force majeure is not only a reason for the periods of formal tax obligations but also for the payment periods of tax debt which is material tax obligation. The Ministry of Finance was given the power to extend the tax payment period after the earthquakes on August 17, 1999 and on November 12, 1999 (TPL art. 111/6 which is amended by 4731. art.

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[13] Akdoğan, Abdurahman, 2008, ***Vergi Hukuku ve Türk Vergi Sistemi*** (Gazi Kitabevi, Ankara), p. 51; Arıca, p. 117, 133; Bilici, Nurettin, 2009, ***Vergi Hukuku*** (Seçkin Kitabevi, Ankara), p. 83; Çağan, p. 139; Kaneti, Selim, 1986/1987, ***Vergi Hukuku*** (İstanbul Üniversitesi Yayınları, İstanbul), p. 89; Karakoç, p. 333, 337; Kırbaş, Sadık, 2006, ***Vergi Hukuku*** (Siyasal Kitabevi, Ankara), p. 136; Mutluer, p. 171; Özbalcı, Yılmaz, 1999, ***Vergi Usul Kanunu Yorum ve Açıklamaları*** (Oluş Yayınları, Ankara), p. 166; Şenyüz, p. 59.
[14] Çağan, p. 148; Karakoç, p. 333; Öncel-Kumrulu-Çağan, p. 118; Özbalcı, p.169; Şenyüz, p. 60.
[15] Karakoç, p. 335; Öncel-Kumrulu-Çağan, p. 120.
[16] Çağan, p. 153; Üstün, p. 269.
[17] See. Arıca, p. 139; Karakoç, p. 337; Kırbaş, p. 172; Mutluer, p. 322; Oktar, S. Ateş, 2009, ***Vergi Hukuku***, Türkmen Kitabevi, İstanbul, p. 86; Öncel-Kumrulu-Çağan, p. 120, 219; Saban, Nihal, 2006, ***Vergi Hukuku***, Beta Yayınları, İstanbul, p. 273.
[18] Tax Procedure Law General Communiqué, No. 277 (OG: 23803-01.09.1999).
[19]The Cancellation of Tax Debts and Tax Penalties for Taxpayers Having Damaged from the Esrtquakes on August 17, 1999 and November 12, 1999 and Amendements About the Tax Procedure Law, Value Added Tax Law, Law of Fees and Organized Industrial Zones Law, No. 4731 (OG: 30.12.2001-24626).
[20] The law No. 5228 on Making Amendments in Some Laws and Decree Law No. 178 (OG: 31.07.2004-25539).

4/B). Taxes are normally paid within the tax payment period determined by the law. However, in the case of a force majeure announced by tax administration, the normal payment period extends. In the extension of payment period, due time is the day on which the extension period is over. In places like cities, districts and regions, where a force majeure is announced, the administration can extend the payment period up to one year from the end of the due time (if this period starts after the disaster). When the administration extends the payment period, it has three types of discretionary power[21]: Different time periods can apply for city centres, distircts and communs, the period of payment can be extended according to the degree of damage of the earthquake; different extended periods can be determined for different types of taxes.

2. The Effect of the Earthquake on the Tax Base and the Tax Receivable

a. Income Tax

An earthquake damages the commercial and agricultural enterprises. These damages are taken into account when the tax base is calculated, therefore the amount of income tax is also affected.

1) The losses of the assets of commercial (and industrial) enterprises: An enterprise consists of movable and immovable assets. Buildings, commodities and receivables are defined as physical assets, the name of the bussiness and trademark right are defined as non-physical assets. Both movable and immovable assets can be damaged by the earthquake. The assessment of the damages of movable assets is easy. If the movable assets are damaged, the accounting value of the enterprise changes, so does the tax base[22].

The decrease in the value of the assets of an enterprise is assessed by "valuation" criteria. Valuation is defined as the assessment of the economical assets related to the tax base (TPL art. 258). The bases of valuation can be summarized as follows; 1) for valuation, only the dates stated in the law are taken into consideration; 2) each item is valued separately; 3) each item is valued according to its own criteria (imputed value or cost value) (TPL art. 259, 260)[23]. In this context, for real estates and fixtures "cost value"[24] criterion is used (TPL art. 269, 273). For commodities, in principle, cost value is accepted (TPL art. 274, 275). However, if the value of the commodities decreases significantly because of disasters (such as fire, earthquake or flood), the valuation is done through "imputed cost"[25] (TPL art. 278). In this way, the loss assessment is done more realistically. Also the unusual junks, cotton wastes and scraps are evaluated by their imputed costs. The breaks, cracks and bursts on merchandise due to the earthquake indicates the damage. We can explain the drastic decrease in the value of the commodities as the difference in the assesment of the commodities by their owners according to their value before and after the damage. In this case, it must be proven that the loss of value is due to the earthquake. Since the damage on the commodities affects the tax base, it is compulsory that the assessment of the damage must be carried out by the assessment commission (TPL art. 74).

Another procedure when accounting the damage is that the financial values, which have deprication and erosion features and used more than a year, are recorded as an expense, according to the provisions of law, which means amortisation (TPL art. 313; ITL. art.

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[21] See. Üstün, p. 270.
[22] Mutluer, p. 178; Özbalcı, p. 351.
[23] Kaneti, pp. 171-172; Valuation criteria are stated as cost value, fair value, disposal value, nominal value, tax value, current value, examplar value and price (TPL art. 261).
[24] Cost value is defined as, acquisition of an economic asset or else for the increase in its value and associated all espenses with it (TPL art. 262).
[25] Imputed value is defined as the value of an asset which has no actual value or whose value is not unknown or value can not be assessed when the asset is sold on the value day in accordance with an examplar (TPL art. 267/I).

40/7). Depreciable properties may be damaged in an earthquake. Yet their values may disappear partially or completely much earlier than their usual expiration period[26]. As this is an exceptional case, it is accepted a reason for an extraordinary amortization[27]. The extraordinary amortisation rates are determined by the Ministry of Finance (TPL art. 317). If the damages of earthquake–damaged depreciable values are compensated by the insurance company, the amortization sum total is subtracted from the sum of indemnity paid by the insurance company. Then, the balance is entered into the profit and loss account (TPL art. 329).

2- The loss of assets at an agricultural enterprise: The economical properties of an agricultural enterprise are basically real estates, crops and animals. For the real estates and permanent goods, the cost price measurement is used (TPL art. 269, 273). Agricultural products are evaluated by means of "cost price" (TPL art. 276). Besides being evaluated by the cost price, the registered animals are also evaluated by the imputed value when the determination of the cost price is impossible. This value is formed of average cost price fixed by the agricultural income commission (TPL art. 277). However, account book and registration order are not compulsory for the establishments that do not exceed the average enterprise size measurements, so determination of damages becomes very difficult for those enterprises.

3- The damages in immovable properties, other incomes and revenues: When calculating these income components, repairment expenses of the damages caused by earthquake can be taken into consideration (ITL. art. 74/7, reiterated art. 81 and 82)

4- The physical damage of real person: Physical damages are important for the self-employement income and wage (or salary) income. Because of the loss of work force, these incomes are subject to a tax discount (ITL. art. 31).

b. Corporation Tax

The incomes of a corporation are composed of income components declared in the ITL. In other words, commercial incomes, agricultural incomes, self-employment incomes, wages and salaries, revenues from immovable properties, revenues from movable properties, other incomes and revenues are the income resources of corporations (CTL. Art. 1/2). However, for the determination of the corporation income, it is indicated that the rules relevant to commercial income in the Induvidual Income Tax Law applies (CTL. art. 6). Corporations are taxed on earned net corporation incomes within a fiscal year. If a corporation is engaged in an agricultural activity, then the last subclause of the article 59 of the Induvidual Income Tax Law is imposed. In case of earthquake, there is no difference between a corporation and a commercial enterprise of a real person as far as the determination of economical values and amortization are concerned. Corporations, like the real persons, are also subject to valuation and amortization (TPL art. 269-290; 317)

3- Cancellation of Tax

a. In General

Cancellation means erasing a written thing by crossing it out[28]. In the Tax Law, cancellation, irrelevant to the loss of operating activity, is to erase the tax receivable and tax penalty directly (TPL art. 115/I, 1 and 2). Cancellation is an administrative act that abolishes the tax debt and tax penalty[29]. Tax debt should belong to a taxation period

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[26] Kaneti, p. 188; Oktar, p. 311.
[27] Mutluer, p. 195; Öncel-Kumrulu-Çağan, p. 247.
[28] Türk Dil Kurumu, 1998, ***Türkçe Sözlük*** (Vol. 2, Ankara, The article of "terkin"), p. 2198; Devellioğlu, Ferit, 2002, ***Osmanlıca-Türkçe Ansiklopedik Lûgat*** (The article of "terkin", Aydın Kitabevi, Ankara), p. 1086.
[29] Karakoç, p. 429; Kızılot, Şükrü, 1994, ***Vergi Usul Kanunu ve Uygulaması*** (Vol. 7, Yaklaşım Yayınları, Abr. VUK, Vol. 7), Ankara, p. 1015; Oktar, p. 125; Öncel-Kumrulu-Çağan, p. 149; Saban, p. 151.

before the earthquake and it should be the one which has not been paid, yet[30]. The taxes which were due before the earthquake and thus having been paid are not regarded as a debt[31]. Because of this, the receivables exceeding the taxation period and not being collected and the penalties belonging to this period should be considered under the coverage of cancellation. Cancellation aims to ease the situation of the taxpayer who has lost this financial power by erasing his tax debt[32]. Cancellation is applied for the recovery of the mistake and for the judication as well as for the disasters (TPL art. 115).

In the case of disasters, the tax debts and tax penalties related to the damaged income resources are cancelled (TPL art. 115/I, 1). Within this context, it is possible to think that the cancellation of the tax debts due to the earthquakes can be considered as renouncement of receivables[33]. This issue is imposed by the Tax Procedures Law with Sequence Number 1[34]. In order to determine the tax debts and tax penalties, it is necessary to know when (in other words the taxation period) the debts of income taxes arise within the context of income taxes. Basically, for the income tax and corporation tax, the taxation period is one year which is the past calender year (ITL. art. 108). The event which arises the tax is completed on the last day of the past calender year. Income taxes are based on the declaration, which are made by annual tax return. Real persons are obliged to declare their income on 25th of March and the corporations on 25th of April of the fiscal year following the taxation period. Once the income is declared, account receivable is accrued and becomes payable within the period of payment. The last payment day is also called the due date of payment. Income tax can be paid at two equal installements (in March and July) while the corporations tax can be paid until the last day of April when the tax return is submitted.

b. The Conditions of Cancellation

1) Material Conditions

a) Occurrence of an earthquake: Earthquake is regarded as a cause which activates the tax law and creates taxational results for the damages taxpayer suffers. Tax office has got the power to determine whether an earthquake is an obstacle for undertaking the taxational responsibilities or not.

b) The existence of tax debt and tax penalty in case of earthquake: In the earhquake-based tax cancellation, tax debts and tax penalties directly related to the damages caused by the earthquake, are cancelled (TPL art 115/I, 1). At this point, determination of the coverage of tax debt which are going to be cancelled becomes important: The Law clearly mentions the cancellation of the tax debts and tax penalties related to the "income resources". However, which taxation period's tax debts and penalties are going to be cancelled and at what stage is ambiguous. Because of this, from the income taxes' point of view, it's important to determine the taxation period which is completed before the earthquake. Then, accepting the taxes of incomes and the unpaid part of the tax penalties of those incomes within the cancellation is suitable for the terminological interpretation of the law. Tax claims of each taxation period can be at different stages for different taxpayers. In other words, for those taxpayers who don't submit their tax returns in time or who file a lawsuit against assessment, it is absolute that the phase of accruement will come later. Accordingly, for the cancellation, the accrual of tax in time or

[30] Tax Procedures Law Communiqué, Sequence No. 1 (OG:13.03.1950-…); Arıcı, p. 435; Kızılot, VUK, Vol. 7, p. 1024, 1031; Özbalcı, p. 353.
[31] Kızılot, VUK, Vol. 7, p. 1031; Öncel-Kumrulu-Çağan, p. 151; Özbalcı, p. 353.
[32] Kızılot, VUK, Vol. 7, p. 1016; Mutluer, p. 154.
[33] Karakoç, p. 431; Oktar, p. 125; Öncel-Kumrulu-Çağan, p. 150.
[34] Although the Tax Procedure Law Communiqué No. 1 is regulated according to article 105 of the Tax Procedure Law No. 4532, it is still applied because the article 115 of the Tax Procedure Law No. 213 has the same meaning within it.

later, or arrangement of tax penalty later should not be important[35]. When the cancellation is issued, the accruement of a claim is not indicated as precondition in the law. That is to say, it is possible that tax receivable of the previous taxation period can be accrued and cancelled afterwards[36]. Within this context, at first the tax is accrued and then the tax cancellation is made. Even if they originate before an earthquake, the taxes which have not been accrued yet are under the coverage of cancellation. Thus, it is to be thought that the tax debts which are in effluction of time for taxation and the penalties belonging to this period are in the process of cancellation[37].

Tax Procedure Law is applied to the cancellation of both tax debts and penalties in the collection phase (Public Receivables Collection Procedure Law, RCPL art. 105). It is essential that the tax receivables in the stage of tax recovery should be accrued and should be related to the income source damaged by the disaster. The condition of accrument is vital in this period, moreover the executive collection process stars for the accrual and unpaid debts in due time. Therefore, the condition of "accrueing" necessary in article 115 of TPL which is also a condition in article 105 RCPL is only for the tax receivables in the executive collection process.

c) Damages for the taxpayer's "general assets" in the earthquake in certain rates: According to the income and corporation tax "the taxpayer who loses at least one-third of his/her assests" can benefit from the cancellation. This can also be called "common loss condition". The taxpayer's "economical impossibility", due to the damage of his all assests, is estimated. For this reason, first of all, the quality and values of his general assests must be searched. Assests include not only the taxpayer's movable and immovable assets but also his rights and benefits[38]. For the cancellation, the values of the assests preceding and following the earthquake must be determined whether the general assest decreases at least in the rate of one-third. If there is at least one-third loss of the general asset, it has to be accepted that the general damage condition of the cancellation (the damage degree in general assets) has fulfilled. Indemnifications, owing to the loss, obtained from the insurance are not important when the loss degree in the general asset is estimated[39]. This is because of the fact that the impact of the earthquake is measured in the loss of general assets.

d) The damage of the income source's loss for the cancellation of the tax debt and penalty: Cancellation procedure does not apply if taxpayers loses only one third of his assets but also the loss of the income source which is caused to tax debt and tax penalty is required[40]. This can be called "condition of private loss" because the relation between the debt which will be cancelled and the income source must only be seen. It is possible

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[35] Cf. Arıcı, p. 433, 436, 440; Bilici, p. 121; Kaneti, p. 128; Kızılot, VUK, Vol. 7, p. 1015, 1016, 1021; The taxes which belong to pre-earthquake taxation period and which were assessed ex officio and supplementary are also under this coverage (Kızılot, VUK, Vol. 7, p. 1023).

[36] Özbalcı, p. 353.

[37] Öncel-Kumrulu-Çağan, p. 151.

[38] Tax Procedures Law General Communiqué, No. 1.

[39] Cf. Kaneti, p. 129; Kızılot, VUK, Vol. 7, p. 1026; Özbalcı, p.349; Yet, the Council of State has the oppinion that insurance indemnity does not effect the degree of damage (See. Kızılot, VUK, Vol. 7, p. 1026, footnote. 27).

[40] Şenyüz, p. 94; The title of the Article 104 was stated as "tax amnesty" in the Draft Tax Procedure Law No. 5432 (1949). The expression of "income sources" was not written in the draft, but it was stated that tax debts and penalties could be cancelled partially or totally by the Ministry of Finance according to the degree of damage determined following an administrative investigation. The number and title of the article were altered to 105 and "cancellation of tax" in the Interm Comitte report dated April 16, 1949, decision 1/174 on the basis and decision number 5. (T.C. Maliye ve Gümrük Bakanlığı, Gelirler Genel Müdürlüğü, 1988, ***Türk Vergi Kanunları Gerekçeleri***, Vol. 3, Ankara, p. 48, 130). The purpose of above amendement was stated as "…the tax to be cancelled is restricted to income sources which are damaged by those disasters…". "Restriction" (hasr) means confinement, putting something into a limited place (Devellioğlu, p. 336). As a result, the Interim Comittee based its legislation intension of the amendement specifialy on "income sources". In other words, the damages to the income components in the income taxes is under this coverage. However, non-reciprocal acquisitives in gift and inheritance tax should be considered as an income although they do not come from a permanent source.

to see that the loss in a commercial and agricultural enterprise or the loss in a real estate capital can be as damages of a source of income. So, we must consider income taxes under this scope since they are related to the source of income[41]. In this context, despite the damage of all the income sources, if one-third of the general asset is not damaged, the process of cancellation can not be applied[42]. Also no cancellation can be applied if one-third of the income source is not damaged in spite of the damage of all the general asset[43].

e) *Calculating the amount to be cancelled in the rate of the damage degree*: The damage degree of the income source (the rate of asset having been damaged) is the rate which will be used in order to estimate the amount of tax debt or tax penalty to be cancelled[44]. The record has to be demolished by subtracting the amount of the cancellation from the amount of tax charge which is accrued (or will be accrued)[45].

2) Formal Conditions

a) *Application for Cancellation*: There is no provision regarding the start of cancellation procedure in the law. But, according to Tax Procedure Law Communiqué, No. 1, the first possibility, applying for the cancellation to the tax administration[46]. It is possible to apply for the cancellation of the uncollected tax receivables and tax penalties related to the pre-earthquake taxation period until the collection prescription of the tax[47]. The second possibility has been thought for common disasters: The tax office informs the taxpayers about the cancellation process via appropriate means. The last possibility is for the taxpayers who are known by the tax office when they are exposed to an earthquake. A taxpayer's declaration of the cancellation is provided by the tax office to make the taxpayer's declaration valid during these cancellation processes so that the taxpayer can declare his cancellation.

b) Determining the values of loss in general assets and source of income by local tax assessment commissions: After being signed by the head of principal treasury or fiscal director of a district, the application documents are presented to the boards of administration for the necessary examinations[48]. The Local boards of administration estimate the amount of damaged general assets (determining their movable, immovable assets and their rights and benefits), its damage degree and the damage degree of the source of income according to the current value[49]. The authorised administrative boards are supposed to send back the document that include the view on the amount which has to be cancelled from the tax debts and tax penalties[50].

c) *Cancellations by the tax office:* Even though the cancellation power, in principle, has been given to the Ministry of Finance, it can be handed over to a local authority. Cancellation procedures are made according to the damage and damage values fixations of local districts (TPL art. 115)[51]. Transactions to be done by local districts about this subject are regarded as preparatory process which causes a cancellation done by tax offices. For this reason, fixation process is not notified to the person who suffers from the earthquake. Furthermore, this process is only accepted to be the reason of the cancellation process. Thus, local districts processes which are preliminary procedure on

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[41] See. Özbalcı, p. 351, 352; Şenyüz, p. 95.
[42] Arıcı p. 435; Özbalcı, p. 347.
[43] Özbalcı, p. 348.
[44] Özbalcı, p. 349.
[45] Uluatam, Özhan-Mehtibay, Yaşar, 2000, ***Vergi Hukuku*** (İmaj Yayıncılık, Ankara), p. 149.
[46] Kaneti, p.129.
[47] Tax Procudure Law General Communiqué No.1; Kızılot, VUK, Vol. 7, p. 1037; Özbalcı, p. 357.
[48] Tax Procudure Law General Communiqué No.1.
[49] Kaneti, p. 129.
[50] Kızılot, VUK, Vol. 7, p. 1039.
[51] See. Danıştay İBKK, 28.06.1979 T. 1976/1 E. 1979/10 K. (***Danıştay Dergisi***, Y. 10, No. 36-37, 1980, p. 189).

its own must not be treated to an action for annulment[52]. However, tax office's process about the cancellation rejection based on the procedure mentioned earlier (since this process is executive and conclusive) can be counterpleaded for cancellation[53]. The contradiction to law of the fixation process by local districts can be put forward.

If a deffered tax asset which must be within the context of cancellation is paid after the earthquake, this claim is refused and refunded to the owner because of the fact that this debt belongs to the pre-earthquake period[54].

THE EFFECTS OF THE EARTHQUAKES ON AUGUST 17, 1999 AND NOVEMBER 12, 1999 ON INCOME TAXES

ITS EFFECTS ON ADJECTIVE TAX LAW

1. Casualty Document Facilities

The first arrangement after August 17, 1999 Earthquake was about the losses of documents and instruments which were a proof of tax relation[55]: Because documents and instruments are a proof for the activities of taxpayers, they are vital instruments in tax relation. For this reason, taxpayers have to apply to the competent courts from the date that they realize the lost documents and instruments so that they can obtain a document (casualty document) which certify the situation. This is the period of limitation[56]. The court could also require the collection of evidences when needed. It is in the discretion of the court whether to require the collection of evidences. The authority to issue the casualty document is given to the provincial administration board and the district administration board besides the courts in places which are exposed to earthquakes. This power given to boards is limited to prepare the casualty document, and does not include cancellation. This kind of requests about deleting tax debt are decreed by the local tax administration[57]. Thus, the option of receiving casualty document from the administration boards is offered to the taxpayers beside courts. Taxpayers who do not take the casualty document from courts and administration boards are considered as an avoidance of submitting their account books and documents. Thanks to this amendment, the main rules related to taking casualty document in case of a disastrous earthquake are put aside. An exceptional practice has been in force which extends the application time period and also increases the number of places where the document is issued.

2. Declaration of Force Majeure

The earthquake is shown among the reasons of force majeure (TPL m. 13). Because of the earthquake on August 17, 1999, Ministry of Finance declared state of force majeure with the Tax Procedure Law General Communiqué No. 277 so as to apply the effect of earthquake on periods[58]. With this communiqué, the places affected by the earthquakes are classified into two groups: The first one is "persons who lives in cities which are affected intensively from the earthquake". On the Communiqué, the centres and districts of Kocaeli, Sakarya and Yalova provinces and Düzce, Gölkaya, Gümüşova, Cumayeri and Çilimli districts of Bolu are considered as provinces that are affected intensively from

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[52] See. Kızılot, VUK, Vol. 7, p. 1038.
[53] Karakoç, p. 433; Özbalcı, p. 358.
[54] Kızılot, VUK, Vol. 7, p. 1040; Öncel-Kumrulu-Çağan, p. 151.
[55] This conveniences were added to the temporary Article 15 of the Law Related to Aids and Measures Effective in Public Life Because of Disasters No. 7269 by the Article 3 of the Decree Law No. 574 dated August 31, 1999.
[56] See. Karakoç, s. 313.
[57] Council of State upheld the decision of the court of first instance on the issue that the applications regarding the cancellation of the tax debpts must be finalised by the tax Office and that the board of district administration must inform the tax Office about the loss of taxpayer's asset (whether it lost its value or not or how much loss it had). Council of State held above decision as a result of the denial of taxpayer's application to the board of administration (4. D. 29.06.2004 T. 2003/1944 E. 2004/1575 K. ***Danıştay Bilgi Bankası***, http://danistay.gov.tr)
[58] Tax Procudure Law General Communiqué No. 277 (OG: 23803-01.09.1999).

earthquake. The declaration of force majuere applies to all taxpayers who has taxpayer entry regardless of having been damaged. The second one is "experiencing a damage from earthquake": The existence of force majeure is accepted for only the taxpayers having experienced the damage of the earthquake among the taxpayers who have taxpayers entry in İstanbul, Eskişehir and Bursa provinces taxpayers. Within the context of these two criteria taxpayers are not obliged to pay taxes and submit their declarations within the period of existence of force majeure.

ITS EFFECTS ON MATERIAL TAX LAW

1. The Extension of Payment Period, The Postponement of the Payment and the Installment Facilities

Following the announcement of force majeure, the authority to extend the period of payment of tax receivables, tax penalty and accessory receivables of which due after the earthquake are given to Ministry of Finance on August 17, 1999 by the Decree Law No. 576 (art. 2)[59]. This power has been extended to include the earthquake of the November 12, 1999, declaration of force majeure also applies to this earthquake[60]. In this decree, the authority "to determine various periods about cities, city centres, districts, communs, taxpayers experincing damage from the earthquake and types of tax" is also given besides "extending the period of payment".

Ministry of Finance extended the end date of force majeure by the General Communiqué No. 277 based on Decree Law No. 576[61]. The due time of the tax receivables, tax penalties and accesorry receivables is between August 17, 1999 and September 30, 1999 . And it is also included in this communiqué. The cities and districts in which force majeure rule applies, are mentioned in Tax Procedure Law General Communiqué No. 282 about ending the force majeure stressing upon the fact that the earthquake damaged these places in different sizes[62]: According to this communiqué, the state of force majeure ends on January 10, 2000 for the taxpayers who have taxpayers entry out of the centres of Sakarya, Bolu and Düzce provinces and the districts of Gölcük and Karamürsel. In this case, they are supposed to have started their active work by the date of publication of Communiqué or they must have been active before the earthquake (the provision of article 14 of the TPL is considered). In places where the force majeure ends, the due date of submitting the tax return will be calculated by adding the period which are not applied between August 17, 1999 and January 10, 2000 (including the date January 11, 2000). The tax returns which will be submitted according to the calculated period above are accepted as given in the legal period. The state of force majeure for the taxpayers having tax payment entry in the centres of Sakarya, Bolu and Düzce provinces and their districts and Karamürsel and Gölcük districts of Kocaeli Province expired on May 31, 2001[63]. The due time for these taxpayers to submit their declarations is the end of working hours of June 30, 2001[64]. Since the payment of taxes, within the same period, that accrue according to these declarations will cause distress for taxpayers, it is stated that the due times will be announced later by Ministry of Finance according to the Decree Law No. 576. Since there is the same distress about the payments of debts of which payment period is extended by the Collection General Communiqués No. 406 and 407, it is also stated that the periods of payments declared on the communiqué will be

[59] Decree Law No. 576 Regarding the Regulation of the Natural Disasters Aids and the Extention of Tax Payment Periods and Making Amendements in Some Laws (OG: 23.09.1999-23825).
[60] Article 7 of the Decree Law, No. 589.; Tax Procudure Law General Communiqué No.282 (OG: 21.12.1999 – reiterated no 23913).
[61] See, Collection General Communiqué, No.406 (OG: 24.09.1999-23826) and Collection General Communiqué, No. 407 (OG: 23.09.1999-23855).
[62] Tax Procudure Law General Communiqué, No. 282 (OG: 21.12.1999-reiterated art. 23913).
[63] Tax Procudure Law General Communiqué, No. 295 (OG: 14.05.2001-24402).
[64] The deadline to submit the tax return for income tax extented to the end of working hours on July 31, 2001 for the taxation period of years 1999 and 2000, and it was August 31, 2001 for corporate tax accordingly (Tax Procudure Law General Communiqué No. 296 (OG: 15.06.2002-24419).

announced later. Ministry of Finance provided the taxpayers, who have diffuculty in payments, to benefit from the facilities by extending the postponement and installment of their debts at various times by the Collection General Communiqués[65].

Apart from the Decree Law No. 576, the Tax Peace Law No. 4811 also accepts the last day of February in 2003 as the due date of income tax and corporate tax receivables which have not been paid yet until the date of commencement of Tax Peace Law and extended periods because of the earthquakes on August 17, 1999 and November 12, 1999 (4811. art. 2/5)[66]. All taxes which are not paid on their due date and whose due date is not over and related default interest and delinquent tax due are calculated again according to the total commodity price index and a new installment plan is made (in February, April, June, August, October, December in 2003 and in February, April and June in 2004) on condition that they are paid in nine equal installments. Instead of default interest and delinquent tax due, accesory receivable's calculation is based on the total commodity price index and calculated total receivables could be paid in nine equal installments (4811. art. 2/1, a).

2. The Special Cancellation of Tax Receivable

Following the earthquakes on August 17, 1999 and November 12, 1999, since the article 115 of the TPL was not sufficient to solve economic and social problems of the taxpayers, a new law called "The Cancellation of Tax Debts and Tax Penalties for Taxpayers Having Damaged from the Earthquakes on August 17, 1999 and November 12, 1999 and Amendements About Tax Procedure Law, Value Added Tax Law, Law of Fees and Organized Industrial Zones Law No. 4731" was put into force[67] and a related communiqué was prepared according to article 1 and 2 and 2 of this law[68]. The Law covers other taxes together with income taxes. Therefore, we can examine the special cancellation rules in terms of place, person, subject and time in income taxes:

a) Application in terms of place: In terms of place the law covers "the cities damaged by the earthquake" which occured on the dates of August 17, 1999 and November 12, 1999 (4731. art. 1). Whereas, the state of force majeure has been based on two different criteria in respect to place due to the earthquake on August 17, 1999 in Tax Procedure Law General Communiqué No. 277: The places damaged intensively by the earthquake and the places damaged by the earthquake only . The places damaged intensively by the earthquake are as follows; 1) the centers and districts of Kocaeli, Sakarya and Yalova provinces, 2) the districts of Düzce, Gölkaya, Gümüşova, Cumayeri and Çilimli of Bolu province. Determination of places "damaged only by the earthquake" is needed in the Law No. 4731 apart from the communiqué. This defination clearly consists of the provinces damaged intensively by the earthquake and the provinces damaged by the earthquake only. In other words, Kocaeli, Sakarya, Yalova, Istanbul, Eskişehir and Bursa provinces together with Çilimli, Gölkaya, Gümüşova and Cumayeri distiricts and centers of Düzce and Bolu provinces will be considered in this context[69]. The state of force majeure was accepted for all taxpayers having taxpayer entries in places effected intensively by the earthquake. This is a legal presumption, thus, proving and claiming the contrary is not possible. Taxpayers having taxpayer entries could benefit from the state of force majeure on condition that they prove their damage in places damaged only by

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[65] The number and publication dates of the Collection General Communiqués in the Official Gazette are as follows: 414-06.02.2001; 418-07.08.2001; 419-16.10.2001; 420-15.12.2001; 423-14.02.2002; 425-21.05.2002; 426-05.07.2002; 427-15.09.2002).

[66] Tax Peace Law, No. 4811 (OG: 27.02.2003-25033).

[67] (OG: 30.12.2001-24626).

[68] General Communiqué on Cancellation of the Tax Receivables and the Tax Penalties Belonging to the Taxpayers Damaged from the Earthquakes on August 17, 1999 and November 12, 1999, No:1 (OG: 07.04.2002-24719).

[69] See. For the title of "With Respect to the Periods" in General Communiqué on Cancellation of the Tax Receivables and the Tax Penalties Belonging to the Taxpayers Damaged from the Earthquakes on August 17, 1999 and November 12, 1999, No:1 (OG: 07.04.2002-24719).

the earthquake. Tax obligations are in progress for taxpayers who have not experienced damage in these places. Taxpayers who are out of the places that are damaged intensively by the earthquake and damaged only by the earthquake are excluded from the state of force majeure. In other words, they are subject to general rules of TPL, even if they are effected from the earthquake.

b) Application in terms of person: The Law indicates the taxpayers having taxpayer entries in these damaged cities after determining the boundaries of application in terms of places. The fact of having taxpayer entry in these places is taken as a criterion also in the declaration of force majeure. However, taxpayers are classified into two groups in terms of application of cancellation as taxpayers whose debts will be cancelled conditionally and unconditionally: 1) Taxpayers whose debts will be cancelled unconditionally are all taxpayers whose entries can not be found partially or absolutely in city and district centres[70] and taxpayers who reside in Gölcük district of Kocaeli. Taxpayers whose debts will be cancelled unconditionally are not obliged to apply for cancellation. Tax offices are required to proceed with the ex officio cancellation[71]. 2) As for the taxpayers whose debts will be cancelled conditionally are those who lose at least 10% of their assets and those having reached their accrual and collection registers on electronic environment.

Considering the date of earthquake, taxpayers are required to declare the statement which certifies that they have at least 10% of damage. This statement must also iclude a cancellation request form evaluating their movable and immovable assets according to current value before and after the earthquake[72]. Also, the taxpayers could reduce their debts which depend on a document and add the amount of their rights and benefits to the total assets. A specific application period and a formal application is a condition which required for taxpayers whose debts will be cancelled conditionally in order to benefit from special cancellation. Taxpayers who have a damage of 10% in their assets are required to apply their tax offices until July 1, 2002.

Special cancellation process is applied by the tax office upon the declaration of taxpayers. It could be examined whether the taxpayer has a loss of 10% on his asset in an examination which will be carry out in the future. If it is understood that the taxpayer does not have a loss of 10% on his asset, the tax office levies his income taxes for that year and notifies him.

c) Application in terms of subject: The Law covers all taxes, duties and fees subject to TPL and related tax receivables, penalties, default interests, delinquent tax due, share of fund and additional share for education. As a result, in terms of subject, the Law No. 4731 is more comprehensive than the cancellation rule in article 115 of TPL. Income taxes (ITL and CTL) and dependent penalties and accessory receivables are under this scope. In context of income tax and corporate tax specific cancellation rules are put into two groups when taxation periods are taken into account[73]. In the first group, the tax

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[70] Following the earthquakes dated August 17, 1999 and November 12, 1999 it was stated that the Office of which registers were unattainable were Sakarya Province Centre, Düzce Province Centre and Kaynaşlı District and Gölcük District of Kocaeli (See. General Communiqué on Cancellation of the Tax Receivables and the Tax Penalties Belonging to the Taxpayers Damaged from the Earthquakes on August 17, 1999 and November 12, 1999, No:1 (OG: 07.04.2002-24719).

[71] General Communiqué on Cancellation of the Tax Receivables and the Tax Penalties Belonging to the Taxpayers Damaged from the Earthquakes on August 17, 1999 and November 12, 1999, No:1 (OG: 07.04.2002-24719).

[72] Council of State reversed the decision of the court of first instance stating that assets which are not subject to land registration and exclusive property could not be accounted in the total sum of asset which would be taken as a degree of damage (Dan. 3. D. 02.12.2004 T. 2003/2028 E. 2004/2995 K, ***Danıştay Bilgi Bankası***, www.danistay.org.tr).

[73] Regarding the taxation periods, monthly declaration means month July, 3-month declaration means months May, June and July and annual declaration means 1998 fiscal year for the periods before 17.08.1999. Special accounting periods in 1999 (either before or after the earthquake) and the ones in 2000 were taken into consideration as taxation periods before the earthquake (See. General Communiqué on Cancellation of the Tax

receivables which belong to previous taxation period and in the second group, the income tax of 1999 and tax witholdings accrued in the year of 2000 are regulated:

1) The tax debts accrued in pre-earthquake taxation periods:

a) The cancellation of the taxes which are under the coverage of TPL (including income tax and corporate tax) and those which accrued in pre-earthquake taxation periods, but have not been paid or taxes which belong to taxation periods before these dates but not due in the date of earthquake, the penalties of the taxes above, their defaults interest and delinquent tax due are prescribed: Income tax and corporate tax are those which are based on declaration. The law indicates that only those debts, which accrued through declaration in other words in pre-earthquake taxation period can be cancelled. By doing so, the law makes a distinction between the debts mentioned above and the debts of taxpayers who have not made declaration yet. Thus, without mentioning the techniques of taxation such as tax assessment, tax accrual and collection of tax, just by taking the taxation period into consideration, the regulation of cancellation of income and corporate tax debts of the year 1998 would be much more suitable to the concept of tax equity.

b) For the cancellation, all of the tax payers gathered in two groups in the cities which suffered from the earthquake are considered. So, all tax receivables, tax penalties and default interests of taxpayers whose tax debts are cancelled conditionally or unconditionally.

c) Tax assessment which belongs to pre-earthquake taxation period can not be implemented. If the taxes of these periods are assessed and, although accrued, have not been paid until (December 30, 2001) the date in which the law No. 4731 put into effect, are cancelled: The abolishment of the power of tax assessment in pre-earthquake taxation period means that the administration can not assess the taxes of this period by declaration, ex officio assessment, additional assessment and administrative assessment.

On the other hand, for these debts, from the due time to the date of cancellation, no additional receivables are demanded under the name of default interest and delinquent tax due (4731. art. 1, 5/c). Only in this case, the elimination of the power of assessment is not considered as an obstacle for the implementation of the articles 359 and 367 of TPL. Nevertheless, although covered by cancellation, the collected tax receivables are not refused or refunded to provide the equality (4731. art. 5/b).

d) In the cancellation process the Ministry of Finance is authorised for the identification of procedures and principles. Tax offices are authorised for the conclusion of cancellation process (4731. art. 1/5, a). The procedures and principles considering the cancellation issues and the deadline for application are determined by the Ministry of Finance. The taxpayers are obliged to apply to the tax office in order to benefit from the rights given by the law.

2) The debts of the taxation period in which the earthquake occrurred and the debts of the post-earthquake period: The annual income tax and coporate tax with regard to the year of 1999, advance taxes which belong to the post- earthquake period of the year mentioned above and all the witholdings of income tax by corporations in 2000 (with refer to ITL, art. 94/6, b) are cancelled (4731. art. 2). Also, the taxes of these taxpayers which had accrued relating to the year 2000 and the witholdings of income tax by corporates of the year 2001 can be cancelled on the basis of rates determined by the Council of Ministers according to the provinces and distriscts. Temporary taxes and witholding taxes which was appropriated from the annual income tax and corporate tax

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Receivables and the Tax Penalties Belonging to the Taxpayers Damaged from the Earthquakes on August 17, 1999 and November 12, 1999, No:1 (OG: 07.04.2002-24719).

relating to the years 1999 and 2000 are refused and refunded[74]. In the workplaces, four times higher amount of tax allowance for the employees also have applied for cancellation. Ministry of Finance determines the procedures and principles of this type of cancellation .

3. The Levy of Additional Income Taxes

The second regulation that has been put into force after the earthquakes on August 17, 1999 and November 12, 1999 is about additional income taxes. Additional taxes were levied with the Law No. 4481 titled "The Law Concerning the Establishment of Liability and Making Amendements in Some Tax Laws in order to Eliminate the Economic Loss Caused by the Earthquakes in the Marmara Region on August 17, 1999 and November 12, 1999"[75]. The Law imposes "additional income tax" and "additional corporate tax" on income tax and corporate taxpayers who were not effected by the earthquakes. The taxpayers who were effected by the earthquakes are exept from that taxes.

The following persons are subject to additional income and corporate tax; 1) The income and corporate taxpayers who declared their annual incomes with an annual tax return in 1998, 2) The income taxpayers whose commercial and self-employment incomes were determined on lump sum basis in 1998, 3) The employees whose total incomes exceeded twelve billion Turkish Liras in 1998 and the employees who received wage when the law come into force and 4) The law applies to the taxpayers who received interest income from the Governmental Debt Securities which were issued before February 1, 1999 (Private Series Government Bonds and foreign currency bonds are excluded), as of January 1, 2000 (4481. art. 1) [76]. Those who are exempted from additional income tax are as follows: 1) taxpayers whose working places are specified by law and wage earners; 2) those who have lost at least 10% percent of their assets; 3) those whose spouse or whose first degree relative had died (4481. art. 3)

The subject of additional income tax is the income of the year 1998 [77] and is applied to the tax base of 1998 with the rate of 5 %[78]. In other words, additional income taxes are based on the incomes of previous year. Those who declared losses in 1998 and those who benefited from investment allowance (although it was repealed later) are exempted from additional income tax79. However, those who are taxed on lump sum base and those who are self employed are considered to be additional income taxpayers80. 12 billion Turkish Liras from their wage income was exempted from additional tax81. Since,

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[74] See. General Communiqué on Cancellation of the Tax Receivables and the Tax Penalties Belonging to the Taxpayers Damaged from the Earthquakes on August 17, 1999 and November 12, 1999, No:1 (OG: 07.04.2002-24719).

[75] (OG: 26.11.1999-reitreated no. 23888); The case which was filed for the cancellation of some of the articles of Law 4481, was denied by the Constitutional Court (Constitutional Court, 28.03.2001 T. 1999/51 E., 2001/63 K., http://anayasa.gov.tr) .

[76] For the opinion that retroactive interest tax is contrary to tax equity, See. Sonsuzoğlu, Elif, "Depremler Nedeniyle 4481 Sayılı Yasayla Getirilen Ek Vergi Mükellefiyetleri" ***Vergi Sorunları***, Nisan, 2000, p.139; For the opinion that interest tax is an unreal retroactive tax, See. Ortaç, F. Rifat, "Ekonomik Kayıpları Gidermek Amacıyla Düzenlenen Ek Vergi uygulamasının Vergi Hukuku Yönünden Değerlendirilmesi", (***Türk Hukuk Dünyası***, Mayıs, 2000, Y. 1, No. 1), p. 33.

[77] Additional taxes are thought to have been contrary to the priciples of state of law since they are based on the event giving rise to the tax which was completed in the previous year (See. Ortaç, p. 32; Sonsuzoğlu, s. 149; Also, see the counter vote justification opinions in the decision of Constitutional Court). In 1998, Constitutional Court held the opinion that it is not right to claim that the principles of taxation according to financial power and fair distrubition of the tax burden are not taken as a basis for the same year's (1998) additional taxation because of the fact that the special situations and financial powers of the taxpayers are taken care of while counting income taxes and corporate taxes.

[78] As it covers those who gave a declaration in 1998, the application of the flat rate of 5% is criticized because it does not take the financial strengt into consideration (See. Sonsuzoğlu, p. 142, 150).

[79] Sonsuzoğlu, p. 139, 151.

[80] Sonsuzoğlu, p. 139.

[81] It is criticized in terms of tax justice that the exception limit of the wage income is high and despite the height is not subject of additional tax (Ortaç, p. 32). Furthermore the different applications between the real estate capital of the annuity which is subject to tax deductions and the real estate capital of the annuity which

the law related to additional income taxes was put into force on November 26, 1999, for the law that was in force on that date applies for not fulfilling the obligations82. The retroactivity of additional tax issue is a matter af discusssion: In terms of tax laws, retroactivity is regarded as real and unreal . In the real retroactivity, tax law applies to a tax event that was given rise before the law. Whereas in the unreal retroactivity tax law applies to an event which has not been completed yet83. According to the law numbered 4481, apart from taxes which were accrued on the basis of the incomes and corporate incomes obtained in 1998 and paid in 1999, an addional income tax and corporate tax are paid depending upon the event giving rise to the same tax. As the event causing the additional tax is completed at the end of 1998, these taxes are subject to real retroactivity84. Increasing the tax burden belonging to the the past periods causes damages to the legal reliability85.

The additional income taxes are declared within one month from the date that the law into force and has to be paid in six installments, where the first one has to be paid in declaration period.

CONCLUSION

The rules relating to earthquakes in terms of income tax are mentioned in TPL, ITL and CTL. Earthquakes affect income taxes in two ways:

It facilitate the obligations of those who were exposed to the earthquake. In this context, the time periods for taxpayers to fullfil their material and formal duties are extended. In the taxation of commercial and agricultural incomes, the base is calculated by deducting the losses. So that, the negative impact of the earthquake on the tax debt is removed. In addition, with the realization of the conditions stated in the law, the tax debts of taxpayers of past taxation periods and penalties are cancelled in proportion to their income sources. Special regulations are required considering the size of the damage caused by earthquakes. Because of the earthquakes of August 17, 1999 and November 12, 1999 comprehensive provisions in favor of taxpayers are made in terms of periods and cancellations according to general regulations. Moreover there have been amendments in the article 15 and 111 of TPL about the powers of the administration. In other words, large earthquakes there have been changes in general tax law.

The second effect of earthquakes are on taxpayers who were not exposed to earthquakes. Public losses caused by earthqakes are ultimately covered by tax revenues. Therefore, the losses which the state can not cover with the normal income tax must be met by additional taxes. However while additional taxes were levied in case of earthquake, the principles of social law state should be taken into account.

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is not subject to tax deduction is criticized in the same way (See. Sonsuzoğlu, p. 140, 141). For the view that the exception limit was specified higher because it was paid from wage incomes of the current year. See. Ortaç, p. 33.

[82] In the decision of the Council of State (4. D dated 20.10.2004 , No E. 2004/1052, 2004/2033 K.), it was decided that those who did not give any declaration because of tax surcharge should be given "tax loss penalty" instead of "defects penalty" (valid in 1998) because it was removed in 1999 (See. ***Danıştay Bilgi Bankası***, http://www.danistay.gov.tr).

[83] See. Aktan, Coşkun Can-GENCEL, Ufuk, 2003, "Türkiye'de Geriye Doğru Vergileme ve Verginin Hukukiliği İlkesinin İhlali", ***Dokuz Eylül Üniversitesi Sosyal Bilimler Enstitüsü Dergisi,*** Vol. 5, No. 1, http://www. bayar.edu.tr, p.4); Kaneti, p. 41; Karakoç, p. 141; Ortaç, p. 32; Öncel-Kumrulu-Çağan, p. 47.

[84] Aktan-Gencel, p. 14, 18; Ortaç, 32; Üstün, s. 90.

[85] See. Aktan-Gencel, p. 6; Kaneti, p. 42; Karakoç, p. 142; the economic balance tax which brought a 10% additional tax burden to the income and corporate taxes of 1993 is shown as an example of real retroactivity in 1994. However, the Constitutional Court does not consider the economic balance tax as an additional tax but a tax of its own and a real retroactivity by declaration of the tax event (Karakoç, p. 143, footnote 85; Saban, p. 28, footnote 97).

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17 AĞUSTOS DEPREMİNİN ÇALIŞMA HAYATINDA CİNSİYET AYRIMCILIĞINA ETKİLERİ

IN THE WORKING LIFE AUGUST 17 EARTHQUAKE EFFECTS OF GENDER DISCRIMINATION

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Anahtar Kelimeler: 17 Ağustos, Çalışma Hayatı, Cinsiyet Ayrımcılığı, Kadınlar

ÖZ 17 Ağustos 1999 tarihinde meydana gelen Marmara depreminden en çok zarar görenler hiç kuşkusuz kadınlar olmuştur. Çalışma hayatında zaten yeterince aktif rolü olmayan kadınların depremin de etkisiyle işgücü piyasasına katılma oranları daha da düşmüş ve bu durum yoksulluğun yaygınlaşmasına sebep olmuştur. Birçok işveren, işçi seçiminde cinsiyet ayrımcılığı yapmakta ve erkek işçileri tercih etmektedir. Bunun başlıca nedenlerinden biri, kadının iş yaşamında ev yaşamındaki aktif rolü oynayamayacağının düşünülmesi, hamilelik dönemi ve çocuk gibi nedenlerle de yeterli verimin alınamayacağı varsayımıdır. Çalışma hayatında kadına karşı ayrımcılık oluşturan nedenlerin temelinde sosyo-kültürel etkenler yatmaktadır. ILO'nun cinsiyet ayrımcılığına ilişkin kararlarına ve Anayasamızdaki eşitlik ilkesine rağmen kanun önünde eşitlik, uygulamada cinsiyetler arasındaki eşitsizlikleri ne yazık ki çözememektedir. Cinsiyete dayalı işbölümü de kadınların mağduriyetine yol açmaktadır. Bu nedenle, kadınların sosyal güvenliği alanında elde edilen temel hakların tehlikeye düştüğünü ve kadınların yaşam kalitesinin azaldığını söylemek yerinde olacaktır.

Çalışmamızda 17 Ağustos 1999 tarihinde meydana gelen Marmara depreminden en çok etkilenen kadınların çalışma hayatındaki durumu, kadın istihdamını artıracak projeler ve cinsiyet ayrımcılığı yasağına ilişkin yasal düzenlemelerin neden uygulanmadığı konuları incelenmiştir. Bu inceleme sonucunda, Türkiye İstatistik Kurumu'ndan (TÜİK) ve İŞKUR'dan elde edilen veriler doğrultusunda, Kocaeli'de ve depremden etkilenen yakın illerde yıllar itibariyle kadınların işsizlik oranının arttığı saptanmıştır.

GİRİŞ

Toplumsal cinsiyet, kadın ve erkeğin sosyo-kültürel açıdan tanımlanmasını, toplumun kadın ve erkeği birbirinden ayırt etme biçimini ve onlara verdiği toplumsal rolleri ifade etmektedir. Bir başka ifade ile toplumsal cinsiyet; cinsiyetlerden birine yüklenen kimliklere, statülere, rol ve sorumluluklara dayanan ilişkiyi tanımlamaktadır (Gündoğan, 2008; 89 ve Erdem/Şahin, 2008; 45). Çalışma hayatındaki cinsiyet ayrımcılığı da bu noktada ortaya çıkmaktadır. Çünkü kadınlara karşı ayrımcılığın temel nedenlerinden biri, toplumda kadınlara ve erkeklere farklı görevlerin verilmesi bu bağlamda da erkeklerin meslek yaşamında, kadınların ise aile yaşamında aktif rolleri bulunduğunun düşünülmesidir (Onaran Yüksel, 2000; 12 ve Erdem/Şahin, 2008; 43). Toplumsal cinsiyete dayalı işbölümü, kadınları ev işleri, çocuk, hasta, yaşlı bakımından sorumlu tutmakta ve kadınların işgücü piyasasına girmesini, iş aramasını büyük ölçüde engellemektedir. Bununla birlikte Türkiye'de kadın işgücü talebinin düşük olması ve işgücü piyasasında cinsiyete dayalı mesleki ayrımcılığa bağlı olarak kadınlara sunulan işlerin sınırlılığı, koşulların ağırlığı ve ücretlerin azlığı da çalışmayı kadınlar açısından cazip olmaktan çıkarmaktadır (Toksöz, 2007; 58/Tunalı, 2003; 109-110). Bu etkenlerin sonucu olarak da kadınların işgücüne katılma oranı erkeklere göre daha düşük olmaktadır. İstatistikler incelendiğinde, 1990-2000 arası dönemde gerçekleşen krizlerin ve 17

Ağustos 1999 tarihinde meydana gelen Marmara depreminin Kocaeli'de istihdamı olumsuz etkilediği görülmektedir.

Çalışmamızda, Türkiye İstatistik Kurumu'ndan (TÜİK) ve İŞKUR'dan elde edilen veriler doğrultusunda, 17 Ağustos 1999 tarihinde meydana gelen Marmara depreminin Kocaeli ilinde kadın ve erkeğin çalışma hayatına katılma oranları üzerindeki etkisi incelenmiştir.

KOCAELİ'DE YILLAR İTİBARİYLE CİNSİYETE GÖRE İŞGÜCÜ DURUMU

Tablo 1. Kocaeli İlinde Cinsiyete Göre İşgücü Durumu, 1980-2000.

	Toplam Nüfus	**12 ve Daha Yukarı Yaş Oranı**	**İşgücü %**	**İstihdam %**	**İşgücüne Katılma Oranı %**	**İşsizlik Oranı %**	**İşgücüne Katılma Oranına Dahil Olmayan Nüfus %**
1980	596.899	70.5	38.43	36.44	54.5	5.2	31.73
1985	742.245	71.5	37.89	35.50	53.0	6.3	33.40
1990	936.163	74.0	39.78	37.04	53.8	6.9	34.16
2000	1.206.085	78.0	45.48	41.70	58.1	8.3	32.73
Erkek							
1980	311.837	71.16	56.12	52.83	78.9	5.9	14.45
1985	383.691	71.90	54.52	50.54	75.8	7.3	17.17
1990	486.436	74.38	56.34	52.02	75.7	7.7	18.03
2000	625.309	78.43	58.14	52.97	74.1	8.9	20.29
Kadın							
1980	285.062	69.8	19.09	0.57	27.4	3.0	50.63
1985	358.554	71.0	19.40	0.70	28.3	3.5	50.77
1990	449.727	73.5	21.87	20.83	29.8	4.7	51.61
2000	580.776	78.0	31.85	29.56	40.8	7.2	46.15

Kaynak: DİE, 2000 Genel Nüfus Sayımı, Kocaeli 2002

Yukarıdaki tabloda görüldüğü gibi 1980-1990 arası işgücü ve istihdam artış hızları, 1990-2000 arası dönemden daha yüksektir. Bu durum 1990-2000 arası dönemde gerçekleşen krizlerin ve 17 Ağustos 1999 tarihinde meydana gelen Marmara depreminin istihdamı olumsuz etkilemesinden kaynaklanmaktadır. Ayrıca 1990-2000 arası dönemde erkeklerde işsizlik oranı 7.7'den 8.9'a çıkarken, kadınlarda 4.7'den 7.2'ye çıkmıştır. Dolayısıyla kadınların olumsuz koşullardan daha çok etkilendiğini söylemek yerinde olacaktır.

Tablo 2. Kocaeli İlinde Kayıtlı İşsizlerin Cinsiyet Dağılımları, 1996-2003.

Yıllar	**Erkek**	**Kadın**
1996	8355	1445

1997	10.743	1553
1998	10.216	1918
1999	13.144	2709
2000	23.674	5028
2001	17.591	3821
2002	15.011	3482
2003	22.826	4723

Kaynak: (Lordoğlu/Demirdizen Çevik, 2007; 36).

Tabloda görüldüğü gibi, 1999 yılında meydana gelen Marmara depreminden sonra, 2000 yılında erkeklerde ve kadınlarda işsizlik bir önceki yıla göre önemli ölçüde artmıştır. 1999 yılı Marmara depremi, Kocaeli istihdamında ciddi sorunlar yaratmıştır. Çünkü deprem sonucu 11.000 işyeri yıkılmış ve 139.000 kişi işsiz kalmıştır. Bölgede yapılan bir araştırmanın sonucuna göre, çeşitli meslek grupları arasında deprem nedeni ile işten çıkarılan işçi ve memur oranının % 44.5 olduğu saptanmıştır (Lordoğlu/Demirdizen Çevik, 2007; 40).

2001 ve 2002 yıllarında ise, erkek ve kadın işsizliğinde azalma olurken, 2003 yılında işsizlik her iki taraf açısından tekrar artış göstermiştir. Bu artışta ekonomik krizin etkisinin olduğu söylenebilir.

Tablo 3. Kocaeli, Sakarya, Düzce, Bolu, Yalova İllerinde Cinsiyete Göre İşgücü Durumu, 2004-2008 (15 ve Daha Yukarı Yaş).

	İşgücü	**İstihdam**	**İşgücüne Katılma Oranı %**	**İşsizlik Oranı %**
Toplam				
2004	923.000	812.000	43.5	12.0
2005	963.000	855.000	46.2	11.2
2006	1.033.000	922.000	47.9	10.7
2007	1.000.000	883.000	45.6	11.6
2008	1.082.000	965.000	47.5	10.8
Erkek				
2004	728.000	636.000	68.3	12.6
2005	740.000	653.000	70.8	11.7
2006	788.000	711.000	72.4	9.7
2007	769.000	688.000	70.7	10.5
2008	805.000	726.000	71.6	9.9
Kadın				
2004	195.000	176.000	18.5	9.6

2005	224.000	202.000	21.5	9.7
2006	245.000	211.000	22.9	13.8
2007	231.000	195.000	20.9	15.3
2008	276.000	239.000	24.0	13.4

Kaynak: TÜİK, Hanehalkı İşgücü İstatistikleri, İBBS, 2006; Hanehalkı İşgücü İstatistikleri 2007; Bölgesel Göstergeler 2008

Not: 2009 yılı verileri mevcut olmadığından tabloda gösterilememiştir.

Son yıllara ilişkin olarak depremden etkilenen illerdeki istatistikler incelendiğinde; erkeklerdeki işsizlik oranının azaldığı, kadınlardaki işsizlik oranının ise arttığı görülmektedir. Bununla birlikte, tablo 3'te 2008 yılında erkeklerde % 9.9 olan işsizlik oranı, 1999 yılında meydana gelen Marmara depreminden sonra işsizliğin önemli oranda arttığı tablo 1'de gösterilen 2000 yılındaki % 8.9 olan işsizlik oranından daha yüksektir. Kadınlarda ise, 2000 yılında işsizlik oranı % 7.2 iken 2008 yılında bu oran % 13.4'tür. Erkeklerde 2000 ile 2008 yılları arasında işsizlik oranları arasındaki fark azalırken, kadınlarda artmaktadır. Dolayısıyla depremin ve ekonomik krizin kadınları daha çok etkilediği söylenebilir. Bu durum kadın istihdamını artıracak projelerin daha etkin uygulanması gerektiği sonucunu ortaya çıkarmaktadır.

KADIN İSTİHDAMINI ARTIRACAK PROJELER

● Eğitimde fırsat eşitliği sağlanmalıdır. Okuma-yazma bilmeyen yetişkin kadın oranının azaltılması için kadına yönelik çalışan sivil toplum kuruluşları ile işbirliği yapılmalıdır (Balkır, 2006; 18).

● İşsizlik sigortası fonunda biriken kaynakların bir kısmı genç kız ve kadınların mesleki eğitiminde kullanılmalıdır.

● İşverenlerden görüş alarak Kocaeli'de işgücü piyasasının ihtiyacı olan alanlarda kadınlara mesleki eğitim kursları verilmelidir. Bu bağlamda bazı illerde kadınlara yönelik çeşitli mesleki eğitim kursları olan kuaförlük, dikiş, kilim, halı, bilgisayar, tasarım vb. alanlarda eğitim verilmekte, çeşitli atölyeler (bebek üretimi, kuru gıda, dikiş atölyeleri, restoran vb.) kurulmakta ve toplu satış yerleri açılmaktadır (Toksöz, 2007; 74 ve Toksöz, Kadın İstihdamı; 73).

● Kadınlar için yeni iş alanları yaratmaya çalışılmalı, bu konuyla ilgili olarak hükümet tarafından yatırımlar için teşvikler verilmelidir (Berber/Yılmaz Eser, 2008; 14).

● Çocuk sahibi kadınlar açısından çocuk bakımı için, kamusal hizmet kurumları kurulabilir ya da işyerlerinde kreşler açılabilir (Berber/Yılmaz Eser, 2008; 14-15).

● Sendikalar kadın üyelerinin çıkarlarını daha etkin savunmalıdır.

● Kadınların kayıt dışında çalıştırılma eğilimleri yüksektir. Çünkü kadınların yoğun olarak çalıştıkları tarım, tekstil ve hazır giyim sektörlerinin özellikleri, evde çalışmayı ve kayıt dışılığı etkin kılmaktadır (Çakır, 2008; 36). Sektörel dağılım açısından ülkemizde istihdama katılan kadınların %48.5'i tarım sektöründe, %14.4'ü sanayi sektöründe, %37.1'i ise hizmet sektöründe yer almaktadır (Çakır, 2008; 29). Dolayısıyla kadın işgücünün en fazla kullanıldığı sektör tarım sektörüdür. Tarım İş Yasası da kadınlar açısından önemli bir ihtiyaçtır (Gündoğan, 2008; 115).

● Kocaeli bir sanayi kenti olduğu için kırsal alandan sürekli göç almaktadır. Bu nedenle kentin kırsal alanlarında istihdam alanları yaratmak üzere organik tarım, ve doğal ürünler yetiştiriciliği gibi kırsal kesimde önemli ve katma değeri yüksek alanlar istihdamı artırmak amacıyla değerlendirilebilir (Balkır, 2006; 18-19).

● Kadınların işgücüne katılımları Türkiye'nin geri kalmış bölgelerinde oldukça zordur. Dolayısıyla kadın istihdamı politikalarının, bölgesel özellikleri göz önünde tutması ve yerel ihtiyaçlara göre şekillenmesi gerekmektedir (Toksöz, 2007; 59). Bu bağlamda GAP Bölge Kalkınma İdaresi tarafından yürütülen "Güneydoğu Anadolu Projesi" kapsamlı bir kalkınma projesidir. Başlıca hedefleri de kırsal kalkınmanın sağlanması, bölgesel gelişmişlik farklılıklarının ortadan kaldırılması, halkın yaşam standartlarının yükseltilmesi, verimliliğin artırılması ve yerel ekonominin istihdam gücünün artırılması olarak tanımlanmıştır. GAP kapsamında kadınlar, genç kızlar ve çocuklar öncelikli hedef gruplarını oluşturmaktadır (Toksöz, 2007; 73).

● Eğitim, bölgesel dengesizlikleri giderme konusunda son derece önemlidir. Bu nedenle geri kalmış bölgelerde yatılı okullar açarak, öğrencilere çeşitli burslar vererek eğitimin artırılması sağlanmaya çalışılmalıdır. Eğitimin artırılması, eğitimli bir kadının daha eğitimli ve bilinçli çocuklar yetiştirmesini, daha az çocuk yaparak nüfus artış hızının denetim altına alınmasına yardımcı olmasını sağlayacak ve bu durum sadece işgücü piyasası için değil, uzun vadede ülke ekonomisi için de son derece yararlı olacaktır. Dolayısıyla eğitim tek yönlü olarak ele alınmamalıdır (Berber/Yılmaz Eser, 2008; 14).

● Son yıllarda Türkiye'nin çeşitli illerinde girişimci kadın ve iş kadını dernekleri kurulmaktadır. Bunlara örnek olarak, Adıyaman'da Anadolu Kadınlar Derneğini (AKAD), Diyarbakır'da İş Kadınları Derneğini (DİKAD) ve Güneydoğu İşkadınları Derneğini, Siirt Kurtalan Kadın Dayanışma Derneği sayılabilir. Bu dernekler kadınlara çeşitli alanlarda beceri kazandırıcı kursların yanı sıra girişimcilik eğitimi vermekte, kadınların evde ürettikleri el ürünlerini sergileyip satışa sundukları tezgahlar, pazarlar açmaktadırlar (Toksöz, 2007; 74). Kocaeli'de de 17 Ağustos depreminden sonra bir grup gönüllü kadın bir araya gelerek Kadın Dayanışma Vakfı'nı (KADAV) kurmuştur. Bu kuruluşlar, kadınların bir araya gelmesi ve gelir getirici etkinliklerde bulunması açısından önemlidir.

CİNSİYET AYRIMCILIĞI YASAĞI

Cinsiyet ayrımcılığı yasağına ilişkin yasal düzenlemelere rağmen bu yasağın etkin bir şekilde uygulanmaması sorunu önemlidir. Eşit haklara sahip olma ve cinsiyet ayrımcılığı yasağının hukuki dayanakları uluslararası dayanaklar ve ulusal dayanaklar olmak üzere ikiye ayrılmaktadır (Onaran Yüksel, 2000; 41 vd). Uluslararası dayanaklar; BM'lerin Kadınlara Karşı Her Türlü Ayrımcılığın Önlenmesi Sözleşmesi, Uluslararası Çalışma Sözleşmeleri ve Tavsiye Kararları, İnsan Hakları Evrensel Bildirgesi, Avrupa İnsan Hakları Sözleşmesi ve Avrupa Sosyal Şartıdır. Avrupa Birliği Hukuku Mevzuatına baktığımızda ise Avrupa Topluluğu Antlaşması, Maastricht ve Amsterdam Antlaşmaları, 75/117 sayılı Kadın ve Erkekler için Ücret Eşitliği Yönergesi, 76/207 sayılı İşe Alınma, Mesleki Eğitim ve İlerleme ve Çalışma Koşullarında Kadın ve Erkeğe Eşit Davranma Yönergesi, 79/7 sayılı Sosyal Güvenlik Alanında Kadın ve Erkeğe Eşit Davranma Yönergesi, 86/378 sayılı İşyerlerinde Uygulanan Sosyal Güvenlik Sigortalarında Kadın ve Erkeğe Eşit Davranma Yönergesi, 92/85 sayılı Hamile, Loğusa veya Emziren Kadınların İşyerinde İş güvenliğinin ve Sağlığının Korunması Hakkında Yönerge, Konsey Kararları ve Komisyon Tavsiyeleri yer almaktadır. Ulusal dayanaklar ise, 1982 Anayasasının 10. maddesinde düzenlenen eşitlik ilkesi, 4857 sayılı İş Kanununun 5. maddesi, ve 2821 sayılı Sendikalar Kanununun 33. maddesidir.

1982 Anayasasında önce eşitlik ilkesine yer verilmiş, ardından bu özelliklerin farklı muameleye esas olamayacağı belirtilmek suretiyle ayrımcılık yasağı vurgulanmıştır. Eşit durumda olanlar arasında objektif bir neden olmadan farklı muamele yapılması ayrımcılık anlamına gelmekte bu nedenle de eşitlik ilkesi ile ayrımcılık yasağı arasında sıkı bir bağ bulunmaktadır (Caniklioğlu, 2006; 643-644). Anayasanın 10. maddesine göre, "Herkes, dil, ırk, renk, cinsiyet, siyasi düşünce, felsefi inanç, din, mezhep ve benzeri sebeplerle ayırım gözetilmeksizin kanun önünde eşittir". Aynı maddenin ikinci fıkrasına göre de, "Hiçbir kişiye, aileye, zümreye veya sınıfa imtiyaz tanınamaz". Böylece, temel hak ve özgürlüklerin güvencesi olan eşitlik ve eşitliğin uygulamada bir görünüm biçimi olan

ayrımcılık yasağı, anayasal düzeyde ve genel esaslar içinde ifadesini bulmuştur (Caniklioğlu, 2006; 644).

2004 yılında Anayasanın 10. maddesine bir fıkra eklenmiş ve kadınlarla erkeklerin eşit haklara sahip olduğu belirtilerek, devletin bu eşitliği sağlamakla yükümlü olduğu esasına yer verilmiştir. Dolayısıyla mevzuatımızda 1982 Anayasasının 10. maddesi, cinsiyet ayrımcılığını yasaklayan en üst hukuk normudur.

Türk İş Hukuku mevzuatı açısından ise, 4857 sayılı İş Kanununun 5. maddesinde eşit davranma ilkesi düzenlenmiş ve bu madde ile cinsiyet ayrımcılığı yasağı getirilmiştir. Bu maddeye göre, "İş ilişkisinde dil, ırk, cinsiyet, siyasal düşünce, felsefi inanç, din ve mezhep ve benzeri sebeplere dayalı ayrım yapılamaz… Aynı veya eşit değerde bir iş için cinsiyet nedeniyle daha düşük ücret kararlaştırılamaz. İşçinin cinsiyeti nedeniyle özel koruyucu hükümlerin uygulanması, daha düşük bir ücretin uygulanmasını haklı kılmaz…" 1475 sayılı eski İş Kanununun 26. maddesi benzer işi yapan işçiler arasında cinsiyet nedeniyle ücret ayrımcılığı yapılamayacağını belirtmiş iken, 4857 sayılı yeni İş Kanunu ile getirilen düzenleme cinsiyet ayrımcılığını sadece ücretler bakımından değil, iş ilişkisinin bütün süreçleri bakımından yasaklamıştır. Dolayısıyla iş sözleşmesinin yapılması, uygulanması ve sona ermesi sırasında cinsiyet ayrımcılığı yapılması yasaklanmıştır.

2821 sayılı Sendikalar Kanununun 33. maddesinin son fıkrasında "Sendika ve konfederasyonlar faaliyetlerinden yararlandırmada üyeleri arasında eşitliğe uymak zorundadırlar" hükmüne yer verilmiştir. Dolayısıyla bu madde de, 1982 Anayasamızdaki eşitlik ilkesinin bir yansımasıdır.

T.C. Devleti, Uluslararası Çalışma Örgütünün (ILO) iki çalışma sözleşmesini (100 sayılı Eşit Değerde İş İçin Erkek ve Kadın İşçiler Arasında Ücret Eşitliği Hakkında Sözleşme ve 111 sayılı İş ve Meslek Bakımından Ayrımcılığa İlişkin Sözleşme) ve Birleşmiş Milletlerin Kadınlara Karşı Her Türlü Ayrımcılığın Önlenmesi Sözleşmesini usulüne uygun olarak onaylayarak iç hukukumuza dahil etmiştir ve yürürlüğe koymuştur (Onaran Yüksel, 2000; 63). Bu sözleşmeler, usulüne uygun olarak yürürlüğe konulduğundan kanun hükmündedir. 1982 Anayasasının 90. maddesi "… Usulüne göre yürürlüğe konulmuş milletlerarası antlaşmalar kanun hükmündedir. Bunlar hakkında Anayasaya aykırılık iddiası ile Anayasa Mahkemesine başvurulamaz" demektedir. Dolayısıyla tüm Devlet organları, idari makamları ve tüm yargı yerlerini bağlayıcı hükümler içermektedir. Bununla birlikte bir uluslararası antlaşmanın iç hukukta doğrudan uygulanabilmesi, o antlaşmanın doğrudan uygulanabilir nitelikte olmasına ve onaylayan ülkenin hukuk düzeninin doğrudan uygulamaya olanak tanımasına bağlıdır (Güzel, 1997; 23). Bu nedenle bir ILO sözleşmesi, içeriği açısından doğrudan uygulanabilir nitelikte olsa dahi sözleşmeyi onaylayan üye devletin iç hukuk düzeninin doğrudan uygulamaya olanak tanıyıp tanımadığına bakmak gerekir.

Bir sözleşmenin iç hukuk düzeninde, iç hukuk normu olarak doğrudan uygulanabilir olup olmadığı tarafların sözleşme ile güttükleri amaca, sözleşme kurallarının konusuna ve sözleşme hükümlerinin şekline ve yazılış biçimlerine bakılarak belirlenebilir (Onaran Yüksel, 2000; 65/Güzel, 1997; 23-25). Uluslararası antlaşma niteliğindeki ILO sözleşmeleri onaylanmalarını takiben kanun hükmündedir. Onaylanan bir ILO sözleşmesi doğrultusunda iç hukukumuzda gerekli yasal düzenleme yapılmadan ILO sözleşmesinin doğrudan uygulanması mümkün değildir (Süzek, 2006; 74/Güzel, 1997; 28). Devlet, ILO sözleşmesini onaylamakla iç mevzuatını o sözleşmeye uyumlu hale getirme yükümlülüğü altına girer (Süral, 2001; 1305). ILO sözleşmesinin salt onay işlemi ile birlikte iç hukukta uygulanabilmesi için başka hiçbir düzenlemeyi gerektirmeyecek kadar açık, kesin, somut, olaya doğrudan uygulanabilecek nitelikte bir hüküm getirmiş olması zorunludur (Süzek, 2006; 74). Yargıtay da (21. Hukuk Dairesi, 18.10.1996 T., 1996/2261 E., 1996/5790 K.) onaylanan ILO sözleşmesinin onaylanmakla doğrudan uygulanabilirlik kazanmayacağı, bu sözleşme doğrultusunda iç hukukumuzda düzenleme yapılması gerektiği görüşündedir (İşveren Dergisi, Mayıs 1997; 15).

İstihdamda cinsiyete dayalı ayrım yasağını, kadın erkek eşitliğini öngören ILO'nun 100 ve 111 sayılı Sözleşme hükümleri içerdiği konularda devletlere ulusal koşullara ve uygulamalara uygun yöntemlerle politikalar oluşturma, önlemler alma ve yasal düzenlemeler yapma yükümlülüğü getirmiştir (Onaran Yüksel, 2000; 67/Güzel, 1997; 10). Bu çerçevede Sözleşme hükümlerinin doğrudan uygulanabilir nitelikte olmadıkları açıktır (Onaran Yüksel, 2000; 67). Dolayısıyla Sözleşme hükümlerinin esnek bir yapıya sahip olmaları da doğrudan uygulanma kabiliyetini büyük ölçüde zayıflatmaktadır (Onaran Yüksel, 2000; 67/ Süzek, 2006; 74) .

SONUÇ

Kadınların işgücüne katılım oranlarının düşük olması, Türkiye'nin istihdam alanındaki en önemli sorunlarından biridir. Özellikle AB ülkeleri ile karşılaştırıldığında, Türkiye'deki kadın işsizlik oranlarının son derece yüksek olduğu göze çarpmaktadır (AB ülkeleri için bkz. İzmit Ticaret Odası, Yayın No: 2006/6; 71-72). Türkiye İstatistik Kurumu'ndan (TÜİK) ve İŞKUR'dan elde edilen bölgesel istatistikler incelendiğinde ise, 1990-2000 arası dönemde gerçekleşen krizlerin ve 17 Ağustos 1999 tarihinde meydana gelen Marmara depreminin Kocaeli'de istihdamı olumsuz etkilediği görülmektedir. Çünkü deprem sonucu, 11.000 işyeri yıkılmış ve 139.000 kişi işsiz kalmıştır. Bölgede yapılan bir araştırmanın sonucuna göre, çeşitli meslek grupları arasında deprem nedeni ile işten çıkarılan işçi ve memur oranının % 44.5 olduğu saptanmıştır. Ancak, depremin yaraları sarılmasına rağmen, son yıllara ilişkin olarak Kocaeli'ye ve depremden etkilenen yakın illere ait istatistikler incelendiğinde; erkeklerdeki işsizlik oranının azaldığı, kadınlardaki işsizlik oranının ise arttığı görülmektedir. Bununla birlikte 2008 yılında erkeklerde % 9.9 olan işsizlik oranı, 1999 yılında meydana gelen Marmara depreminden sonra işsizliğin önemli oranda arttığı 2000 yılındaki % 8.9 olan işsizlik oranından daha yüksektir. Kadınlarda ise, 2000 yılında işsizlik oranı % 7.2 iken 2008 yılında bu oran % 13.4 olmuştur. Erkeklerde 2000 ile 2008 yılları arasında işsizlik oranları arasındaki fark azalırken, kadınlarda artmaktadır. Dolayısıyla depremin ve ekonomik krizin kadınları daha çok etkilediği söylenebilir. Bu durum kadın istihdamını artıracak projelerin daha etkin uygulanması gerektiği sonucunu ortaya çıkarmaktadır.

Çalışma hayatında kadın erkek eşitliğinin sağlanması gereğinin üç temel nedeni bulunmaktadır. Bunlar (Süral, 2001; 1290):

- Kadın ve erkek eşittir. Dolayısıyla cinsiyetler arasında ayrımcılık yapılması insan hakları sorunu olarak ortaya çıkmaktadır.

- Kadın-erkek eşitliği, sosyal adaletin gereğidir.

- Sosyal ve ekonomik gelişmenin sağlanması için hem kadının hem de erkeğin çalışma hayatına katılması gerekmektedir.

Çalışma hakkı, anayasal bir haktır. Anayasa madde 49'a göre, "Çalışma, herkesin hakkıve ödevidir". Bu nedenle çalışmak kadının da hakkıdır. Toplumun yarısını oluşturan kadınların çalışma hayatında, siyasette ve ekonomide daha etkin rol alması, yoksulluğun azalması, toplumsal refahın çoğalması, kadınların maddi olanaklarıyla bilinçli, sağlıklı, topluma faydalı bireyler yetiştirmesi, ülkenin kalkınması, demokratik saygınlığının artması ve yüksek medeniyete ulaşması açısından önemlidir.

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