ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

A NEW MODEL FOR PLASTIC HINGE LENGTH OF RC COLUMNS AND EVALUATION OF SEISMIC ASSESSMENT DOCUMENTS

M.Sc. THESIS

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

BETONARME KOLONLAR İÇİN YENİ BİR PLASTİK MAFSAL BOYU MODELİ VE MEVCUT SİSMİK VERİLERİN DEĞERLENDİRİLMESİ

YÜKSEK LİSANS TEZİ

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OCAK 2015

Alvand-moshfeghi, a **M.Sc.** student of ITU **Institute of CIVIL ENGINERING** student ID 501111076, successfully defended the **thesis** entitled '**A NEW MODEL FOR PLASTIC HINGE LENGTH OF RC COLUMNS AND EVALUATION OF SEISMIC ASSESSMENT DOCUMENTS**", which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

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Date of Submission : 22 September 2014
Date of Defense : 20 January 2015

To my beloved parents,

FOREWORD

This study was performed under the supervision of the Prof. Dr. Alper ILKI. I would like to express my sincere thanks and appreciations for his invaluable support, guidance, encouragements, criticisms and insights throughout this study. It was a great honor and pleasure to work with him.
I would like to thank Mustafa CÖMERT, patiently checked my works and supported my research. In addition, I would like to thank Dr. Cem DEMİR, Ali Osman ATEŞ, Erkan TÖRE and all structural engıneerıng department of ITU because of their supports during whole my career.
I also would like to express my deepest thanks to my parents for their support, friendship, and understanding that they have provided for me through my life.

January 2015 Alvand MOSHFEGHI
(Civil Engineer)

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ABBREVIATIONS

DBYBHY	: Specification for Structures to Be Built in Disaster Areas in Turkey, 2007
T.D.Y.	: Assumption According to Turkish Seismic Design code
Eurocode	: Euro code 8: Design of Structures for Earthquake Resistance,
PEER	: Pacific Earthquake Research Center
Xtract	: Cross Section Analysis of Structural Components
RC	: Reinforced Concrete

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A NEW MODEL FOR PLASTIC HINGE LENGTH OF RC COLUMNS AND EVALUATION OF SEISMIC ASSESSMENT DOCUMENTS

SUMMARY

Application of displacement-based seismic design and analysis that is derived from plastic hinge method is still employed extensively to estimate the performance assessment procedures to evaluate the inelastic displacement demand and capacity. In other word, prediction of behavior of plastic hinges, subjected to large inelastic deformations, caused by extreme loads is used. These loads that affect structures significantly, such as earthquakes, play an important role in assessing the maximum stable deformation capacities of framed concrete structures.

In structural analysis, non-linear analysis is more preferable than linear analysis because of more realistic assessments. Because of dealing with high amount of effective factors on non-linear assessments, some assumptions and damage limit boundaries are required to assess the performance under seismic actions.

In this thesis, Turkish Seismic Design, code DBYBHY 2007 (Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelik) and Eurocode8 (Seismic Design of Buildings) are considered for assessing performance limits. Moreover, the most accepted previous models of plastic hinge length are employed, to investigate the accuracy of each. Finally, a new model proposed for determine the potential plastic hinge length of reinforced concrete columns.

In the first part, differences of codes, ways of approaches about the performance of reinforced concrete columns are assessed based on the results given by PEER database. For this purpose, damage limit states and boundaries are evaluated from codes.

In the second part, with comparing various plastic hinge length models, and the factors, which significantly influence plastic hinge length variation of reinforced concrete columns, it is tried to propose new model by improving the existing equations. The estimation of methods given by codes and the considered plastic hinge length models, are compared with each other, and with the experimental results. Finally, in this study, it is tried to extract a more accurate model for RC columns performance.

BETONARME KOLON LAR İÇİN YENİ BİR PLASTIK MAFSAL BOYU MODELI VE MEVCUT SISMIK BELGELERIN DEĞERLENDİRMESI

ÖZET

Elastik olmayan şekil değiştirme istek ve kapasitesin hesabında kullanılan performans değerlendirme yöntemleri, yer değiştirme kontrollü sismik tasarım ve plastik mafsal metodu kullanılarak bulunan analiz ile elde edilir. Başka bir deyişle, elastik olmayan şekil değiştirmelere maruz kalan plastik mafsal davranışının tahmin edilmesinde buyuk yuklar kullanılır. Bu yükler, deprem v.b, çerçeveli betonarme yapıların maksimum yer değiştirme kapasitesinin tayininde önemli rol oynar.

Yapıların analizinde, doğrusal olmayan analiz yönteminin doğrusal analiz yöntemine göre daha çok tercihedilmesi sebebi bu yöntemle gerçeğe daha yakın sonuçların alınabilmesidir. Yapılarda doğrusal olmayan performans değerlendirmesi sonuca doğrudan etki eden bir çok parametreye bağlıdır. Parametre sayısını fazlalığı ve işlemin karmaşıklığı işlemi basitleştirici bir takım varsayımlar yapılmasını gerektirir.

Yapı malzemelerinin doğrusal-elastik sınır ötesindeki taşıma kapasitesini gözönüne alarak, çok küçük olmayan yerdeğiştirmelerin denge denklemlerine ve gerekli olduğu hallerde geometrik uygunluk koşullarına etkilerini hesaba katmak suretiyle, yapı sistemlerinin dış etkiler altındaki davranışlarının daha yakından izlenebilmesi ve bunun sonucunda daha gerçekçi ve ekonomik çözümler elde edilmesi mümkün olabilmektedir.

Doğrusal olmayan malzemeden yapılmış sistemlerde, artan yüklerle birlikte iç kuvvetler de artarak bazı kesitlerde doğrusal elastik sınır aşılmakta ve bu kesitler dolayında doğrusal olmayan (plastik) şekildeğiştirmeler meydana gelmektedir. Doğrusal olmayan şekildeğiştirmeler genel olarak sistem üzerinde sürekli olarak yayılmaktadır. Buna karşılık kopma sırasındaki toplam şekildeğiştirmelerin doğrusal şekildeğiştirmelere oranının büyük olduğu sünek malzemeden yapılmış sistemlerde, doğrusal olmayan şekildeğiştirmelerin plastik mafsal adı verilen belirli kesitlerde toplandığı, bunun dışındaki bölgelerde ise sistemin doğrusal-elastik davrandığı varsayılabilir. Bu varsayım plastik mafsal hipotezi olarak isimlendirilmektedir. Plastik mafsal hipotezinin esas alındığı bir yapı sisteminin birinci mertebe teorisine göre hesabında oluşan plastik mafsal nedeniyle sistemin tümünün veya bir bölümünün mekanizma durumuna gelmesi taşıma gücünün sona erdiğini ifade eder.

Yapıların artan yükler altında, performansı ile ilgili daha gerçekçi bir tahmin için yapı elemanlarının elastik ötesi davranışları hakkında yeterli bilgiye sahip olunması geremektedir. Yapı elemanlarının elastik ötesi davranışları plastik mafsal hipotezi kullanılarak dikkate alınabilir. Plastik mafsal hipotezinde betonarme kesitin eğilme davranışı moment-eğrilik ilişkisi vasıtasıyla temsil edilebilir. Bu çalışmada moment-eğrilik ilişkileri belirlenirken sargılı, sargısız beton ve donatı çeliği davranışları için Mander et. Al. tarafından önerilen model kullanılmıştır.

Birinci kısım kapsamında, teorik hesaplar deney sonuçları ile karşılaştırılmıştır. Bu tez içerisinde, performans limitlerinin hesabında DBYBHY 2007 (Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelik) ve Eurocode8 (Binaların sismik dizaynı) adlı yönetmeliklerden faydalanılmıştır. Ayrıca, her iki yönetmeliğin de sonuçlarının tutarlılığının kontrolü için ilgili yönetmeliklerde önerilen plastik mafsal boyları kullanılmıştır.

Bu çalışmada, PEER veritabanında bulunan 306 adet dikdörtgen kolonlar içerisinden; belirli kriterlere sahıp olan 57 adet kolon seçilmiştir. Kolonlar seçilirken:

- Numunelerin konsol kolon (cantilever) olmasına,
- Numunelerin iyi sargılı olarak tasarlanmış olmasına,
- Kolon göçme modunun eğilme nedeniyle oluşmasına dikkat edilmiştir.

Göçme modunun kesme veya kesme–eğilme olduğu numuneler bu çalışmanın kapsamı dışındadır. Eğilme hasarında numunenin dayanımının %20 azaldığı veya basınç bölgesindeki boyuna donatının burkulduğu nokta göçme noktası olarak kabul edilmiştir. Her kolonun geometrik, eksenel yük ve donatı özellikleri hakkında bilgiler ekte sunulmuştur..

Türk Deprem Yönetmeliği 2007 (Bölüm 7)' ye göre hasar sınırları, minimum hasar sınırı (MN), can güvenliği hasar sınırı (CG) ve ileri hasar sınırı (GÇ) olarak kategorize edilmiştir. Benzer şekilde Eurocode 8'de de hasar bölgelere üç bölgeye ayrılmıştır. Buna göre; minimum hasar bölgesinde(LD), taşıyıcı elemanlarda belirgin bir plastikleşme söz konusu değildir ve bu elemanlar rijitlik ve dayanımlarını muhafaza etmektedır. Belirgin hasar bölgesinde (SD), yapıda belirgin hasar sözkonusu olmasına rağmen hala bir miktar rijitlik ve dayanım söz konusudur ve düşey taşıyıcı elemanlar düşey yükleri taşıyabilecek durumdadır. Göçme öncesi hasar bölgesinde (NC), binada ağır derecede hasar olmasına rağmen hala az miktarda rijitlik ve dayanım söz konusudur ve düşey taşıyıcı elemanlar düşey yükleri taşıyacak durumdadır.

Yönetmeliklerde kabul edilen hasar sınırları, deneysel çalışmalar dıkkatı alınarak aşağıdaki kriterler dikkate alınarak tanımlanan hasar sınırlari ile karşılaştırılmıştır. Bu tanıma göre hasarların oluştuğu deplasman ve mınımum hasar bögesinin maksimum yerdeğiştirme sınırı, akmaya karşı gelen yerdeğiştirme miktarının 1.2 katı, göçmeye ve beton dayanım kapasıtesinin %20 düştüğü noktaya karşı gelen yerdeğiştirme nihai yerdeğiştirme hasar sınırı ve ileri hasar bolgesinin yerdeğiştirme sınırı, nihai yerdeğiştirmenin %75'i olarak alınmıştır.

Sonuç olarak, Türk Deprem Yönetmeliği'nde malzeme gerimelerini bağlı olan, ve Eurocode 8'e göre ise kesitin kord dönmesine bağlı olarak tanımlanan performans tahmınlerı deneysel veritabanından elde edilen verilere göre konservatif bulunmuş ve sözü edilen yönetmeliklerin tahminlerinin tanımlanan sınır mıkararlarına yakınlığıt tayın edılmiştir. Bu karşilaştirma sırasında iki yönetmeliğin verdiği sonuçlar dikkate alınarak ileriki çalışmalar için önerilerde bulunulmuştur.

İkinci kısımda, plastik mafsal boylarının hesabında kullanılan modellerin ve betonarme kolonların plastik mafsal boylarını etkileyen faktörlerin kıyaslanması ile mevcut denklemleri geliştirecek yeni bir model öne sürülmeye çalışılmıştır. Yönetmeliklerle belirlenmiş olan hesap modelleri ve plastik mafsal boylarının hesabında kullanılan modeller birbirleri ve deney sonuçlarıyla kıyaslanmıştır. Son olarak, bu çalışma kapsamında, betonarme kolonların performanslarının tayinini daha tutarlı olarak ortaya koyacak yeni bir model ortaya çıkarılmıştır.

Bu bölümde, öncelikle daha önceki araştırmacıların önermiş olduğu formüllerle birlikte, DBYBHY 2007, EUROCODE 8-2008 ve MODELCODE 2010 yönetmelikleri karşılaştırılarak, en kabul edilebilir ve en gerçekçi sonuçlar veren, modelin ortaya çıkarılmasına çalışılmıştır. Toplam 22 kolon numunesi, aşağıda yer alan iki özelliğe sahip olacak şekilde, tezin birinci bölümünde analize konu olan 57 adet kolon arasından seçilmiştir:

- Analitik hesaplarla tahmin edilen moment-eğrilik, test sonuçlarından ortaya çıkan yanal yük-deplasman eğrileri, bire bir uyum göstermelidir.
- Numune dayanım kapasitesi deney sırasında yük-deplasman diyagramında, %15 ila %20 oranında azalma göstermelidir.

Yapılan araştırmalara göre plastik mafsal boyu eleman üzerinde bulunan eksenel yük, donatı oranı gibi parametrelere bağlıdır. Bu nedenle yönetmeliklerde ve bazı araştırmacılar tarafından, sadece geometriye bağlı olarak sunulan modeller hatalı sonuçlara sebep olabilir.

Son olarak, analitik hesaplar ve deney verilerinden elde edilen sonuçlar kullanılarak hesaplanan plastik mafsal boyuna en çok uyum gösteren model tespit edildi. Sonuç olarak plastik mafsal boyuna etkili olan parametrelerin de (eksenel yük vb.) etkisini göz önüne alarak yeni teorik model önerilmiştir. Önerilen teorik modelin geçerliliği veritabanında bulunmayan, literatürden elde edilmiş deney sonuçları kullanılarak değerlendirilmştir.

1. INTRODUCTION

1.1. Problem Statement And Scope of Study

Displacement based design and assessment approach that is based on plastic hinge theory is extensively employed to predict the inelastic seismic behavior. According to the available information in previous studies about this concept, most of analyses and tests are designed to investigate the performance of elements, in order to variation of plastic hinge. On the other hand, each research investigates plastic hinge under specific conditions, so a specific model cannot be responsive to other elements under other situations. Due to these mismatches, databases and literatures cannot comment about the plastic hinge, despite it's obvious importance.

In this study, columns with the same design type and efficiencies from the available database selected. First, tried to give a definition for the subject and analyze the existing documents based on a unit definition. To do so, different definitions of plastic hinge, which proposed before, reviewed. The accuracy of various models suggested before, evaluated by comparing the performance of columns and plastic hinge lengths (which calculated from theoretical analysis) and the measurements of real test results. In addition, to provide a practical way of plastic hinge application tried to assess design codes, so Turkish seismic design code and Eurocode considered for this manner.

1.2. Non-Linear Behavior Of RC Columns

To have a better prediction of structure behavior, prevent unaccepted critical deformations and control the life safety conditions, performance based design is one of the particular methods, which uses considerably. Although the probability of plastic hinge formation is very rare along a structure lifetime, it is necessary to have enough knowledge about inelastic deformations under significant seismic loads. Thus, plastic hinge concept plays an important role in structural engineering. Damages on structural members cause story displacements; main part of this displacement is due to inelastic rotations, which may be represented with hypothetical rotations of plastic hinges. As mentioned, the assumption of plastic hinge behavior is important toward seismic design or assessment of structurs.

1.3. Moment-Curvature Relationship

The flexural behavior of reinforced concrete cross-section can be studied by means of its moment-curvature relationship. The knowledge on the deformation relationships, of constituent materials and characteristic of a cross-section can lead predict the moment-curvature relationship.

Under increasing loads, the cover concrete starts to spall as the stress becomes equal to the concrete tensile strength. Due to height degradation, bending rigidity decreases along with curvature increase, although these changes are very tiny and can be neglected. As the cover concrete cracks, the curvature continues to increase linearly by the apply load increase, up to the commencement of plastic deformations formed. Plasticization may occurred in compressive fiber or tensile reinforcements, a little earlier than reaching the moment capacity of the concrete. Between plasticization and failure points, large deflections and cracks, with high curvatures are formed, massively influencing the behavior of structures.

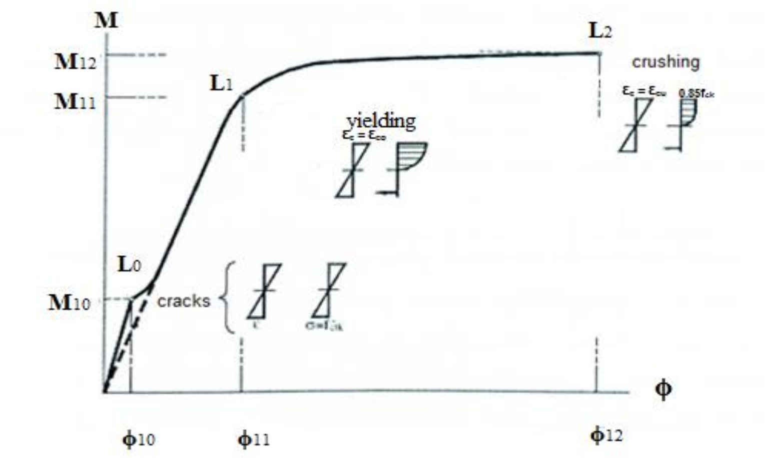


Figure 1.1 : Moment-curvature relation for reinforced concrete sections (Ozer, 2008).

1.4. Plastic Hinge Definition

While reinforced concrete columns support gravity load, they also experience large lateral displacements under high cyclic loads (such as high seismic loads), and severe damages occur at regions, where large inelastic deformations (curvatures) observed. The region that resists maximum moment under applied loads and modifies the largest deflections is called Plastic Hinge Region. The length of this region is referred as Plastic hinge length, L_p. Since the inelastic flexural deformations are generally concentrated around the plastic hinge region, a large curvature capacity is required in this region to withstand large lateral displacement demands.

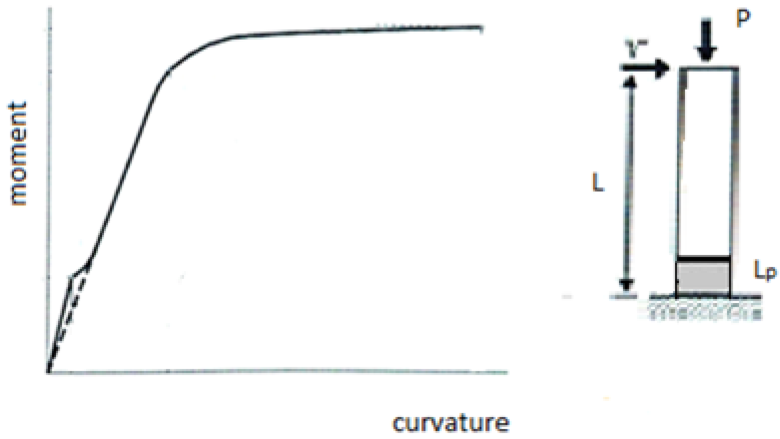


Figure 1.2 : Plastic hinge definition.

1.5. Curvature-Rotation

Deflections happen in elements under large moment changes. These deflections are illustrated as rotations in specific regions.

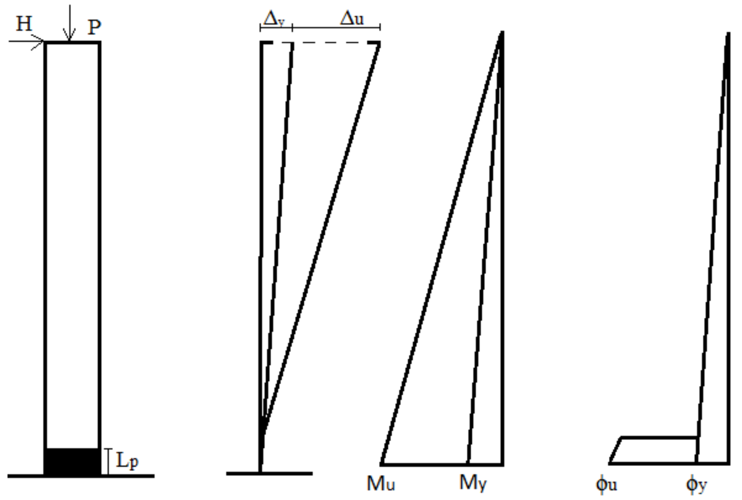


Figure 1.3: Moment-curvature relationship of RC columns under bending moment (Ozmen et al. 2007).

As it is shown in the figure1-3, damages are concentrated in a region and columns are able to show much large deflections due to plasticization. It is accepted that except the plastic hinge region all other parts behave linearly(figure 1-3-b). From the tests and cross section analyses, curvature distributions along the columns show a sudden increase beyond the plastic hinge region. Idealized model of curvature distribution is drawn, considering the mentioned conditions figure1-4.

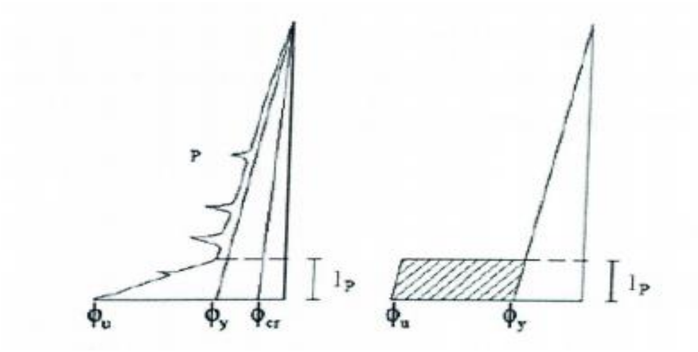


Figure 1.4 : Idealized curvature diagram (Celep 2004).

1.6. Curvature-Rotation-Displacement

The demand of plastic bending depends on the θ_p, the plastic rotation demand that is obtained in any section from push over analysis results or from area under the curve of curvature distribution along the column height:

$$\theta = \int_0^{lp} \varphi(x)\, dx \tag{1.1}$$

Or in simpler form:

$$\theta = \phi\, L_p \tag{1.2}$$

Here θ = rotation, ϕ= curvature and L_p = plastic hinge length

In this equation, the top displacement of columns can be obtained by integration of curvature from column's base to the end of the plasticized region. Paulay and Priestly were the first researchers that simplified the calculations by distributing the curvature along the columns height. Their rotation and deflection calculations in critical sections are summarized as follows:

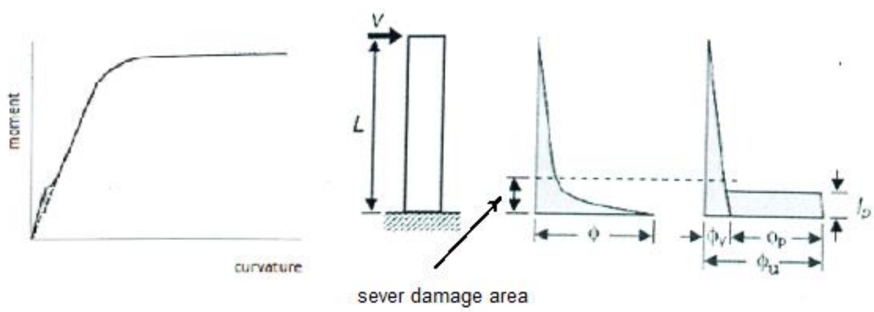


Figure 1.5 : Plastic hinge length and curvature distribution on cantilever columns.

Rotation and displacement before yielding:

$$\theta_y = \frac{\phi_y \;\; L}{2} \tag{1.3}$$

$$\Delta_y = \theta_y \frac{2L}{3} = \frac{\varphi y \;\; L}{2}\frac{2L}{3} = \frac{\varphi y \;\; L^2}{3} \tag{1.4}$$

Ultimate rotation and displacement:

$$\theta = \theta_{y+} \theta_p \tag{1.5}$$

$$\theta_p = (\phi\text{-}\phi_y)L_p \tag{1.6}$$

$$\theta = \frac{\varphi y \;\; L}{2} + (\phi\text{-}\phi_y)L_p \tag{1.7}$$

$$\Delta = \Delta_y + \Delta_p \tag{1.8}$$

$$\Delta_p = (\phi\text{-}\phi_y)L_p\,(L\text{-}\ \frac{Lp}{2}) \tag{1.9}$$

$$\frac{\varphi y \;\; L^2}{3} + (\phi\text{-}\phi_y)L_p\,(L\text{-}\ \frac{Lp}{2}) \tag{1.10}$$

Here; Δ_y , Δ_p :yield and plastic displacements respectively, θ, Δ : ultimate rotation and displacements.

2. LITERATURE REVIEW

2.1.Experimental Models For Plastic Hinge Length

During previous years, there is little information about plastic hinge length based on laboratory tests. Table2-1 shows the models that proposed for determination of plastic hinge length from 1956 when Backer did very first test about this concept to 2010.

Table 2.1 : Plastic hinge length determination models

Researcher Reference	Researcher Reference Plastic Hinge Length Expression (pl)
1-Baker (1956)	k[(z/d)^0.25]d
2-Sawyer (1964)	0.25d+0.075z
3-Corley (1966)	0.5d+0.2√[d(z/d)]
4-Mattock (1967)	0.5d+0.05z
5-Park et al. (1982)	0.4h
6-Mander (1983)	Lpy+0.06z (Lpy=32√db)
7-Priestlley and Park (1987)	0.08z+6db
8-Paulay and Priestley (1992)	0.08z+0.022dbfy≥ 0.3dbl fye
9-Sheikh and Khoury (1993)	1.0h
10-Coleman and Spacone (2001)	Gf l [0.6f'c(ε20-εc+0.8f'c/Ec)]
11-Panagiotakos and Fardis (2001)	0.18L+0.021dbfy
12-Panagiotakos and Fardis (2001)	0.18z+0.021dbfy
13-Restrepo, Seible,Stephan,Schoettler(2006)	Lp=αHβfyedbl≤2βfyedbl (α is a yield spread coefficient and β is a strain penetration coefficient)
14-Berry (2008)	0.05L+(0.008fydb/√f'c)
15-Bae and Bayrak (2008)	Lp/h= [0.3(p/p0)+3(As/Ag)-1](z/h)+0.25≥0.25
16-Biskinis and Fardis	0.2h.[1+(1/3)min(9;Ls/h)] for cyclic loading h.[1.1+0.04min(9;Ls/h)] for monotonic loading

Nowadays due to the importance of plastic hinge topic on structure designs, cods tried to give practical solutions for engineers. This study effort investigate proposed solutions in Euro code, Turkish seismic code and some mostly acceptable researches, create better vision about codes and studies deficiency for this manner. This chapter defines summery of considered studies and codes.

2.1.1. Mattock 1964

In 1964, Mattock did his primary studies on plastic hinge concept and proposed an equation for plastic hinge length. In his studies, he used columns with concrete strength about 27.5 to 41.3 MPa, 324 to 413.6 MPa yielding strength steel reinforcements, and % 1 to % 3 transverse steel reinforcement ratios. Cross section dimensions of his studied columns are limited between 25.4 cm to 50.8 cm in width and z/d ratio (ratio of columns height to effective width) designed between 2.75 to 11. According his test results, he proposed L_p formula as:

$$L_p = \frac{d}{2}[1+(1.14\sqrt{\frac{z}{d}} - 1)\{1-(\frac{q-q'}{qb})\sqrt{\frac{d}{16.2}}\}] \qquad \textbf{(2.1)}$$

Here; q and q' are mechanical ratio of tensile and compression reinforcement, q_b : mechanical balance ratio of tensile reinforcement $((A_s/hxb)x(f'_c/f_y))$.

After a while in 1967 based on studies and analysis about same columns he simplified his L_p definition as:

$$L_p=\frac{d}{2} + 0.05\ z \quad \text{(all units are in inch)} \qquad \textbf{(2.2)}$$

2.1.2. Park and Priestley 1987

Park and priestly with improving the works did by Park, Priestly and Gill in 1982 proposed:

$$L_p = 0.08L + 6d_b \qquad \textbf{(2.3)}$$

Here; d_b: longitudinal reinforcement diameter and L: height of column from critical section to de zero bending point.

The column specimens had 550 mm width in cross sections and they were 1200 mm high with a span to depth ratio of 2.2. The main purpose of this study was to investigate the performance of tied columns, which designed according to the New Zealand code (DZ 3101:1978). The main test variables were the level of applied axial load and the amount of transverse reinforcement. The provided amount of transverse reinforcement in three of four column contained less transverse reinforcement than required amount proposed by the ACI 318-05 code ($0.63 \leq$ Ash/Ash, ACI ≤ 0.89). All specimens demonstrate very ductile behavior and displacement ductility capacity was observed at least 6 in all tests.

2.1.3. Sheikh And Khoury 1992,1994

Sheikh and Khoury (1993) run experimental research on six large scale, two normal and four high strength concrete columns specimens. The concrete columns had 184 cm height and width of 30.5 cm in cross section, resulting in a shear span to effective depth ratio of 6. The main test variables were the strength of concrete, axial load level, configuration and amount of transverse reinforcement. The primary objective of the investigation was to evaluate the confinement designed from ACI code. Following are some of the reported conclusions:

For the constant amount of confining reinforcement required by the ACI 318 code, lower strength concrete columns displayed better ductility than comparable columns made with high-strength concrete and tested under similar P/fc′Ag. However, for the same level of axial load measured as a fraction of Po, high-strength concrete and normal-strength concrete columns behaved similarly in terms of energy-absorption characteristics when the amount of tie steel in the columns was in proportion to the unconfined concrete strength. Conversely, the amount of confining reinforcement required for a certain column performance appeared to be proportional to the concrete strength as long as the applied axial load measured in terms of Po rather than fc′Ag.

• Confining reinforcement, designed according to the ACI code, can provide satisfactory behavior only for specific cases. Depending on the reinforcement detailing and axial load level, the code provisions may be unnecessarily conservative or unsafe.

From their test outputs they proposed;

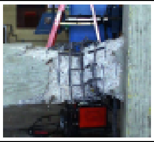
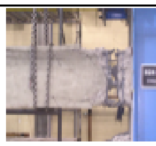
$$L_p = 1.H \qquad \textbf{(2.4)}$$

2.1.4. Sungjin Bae 2005

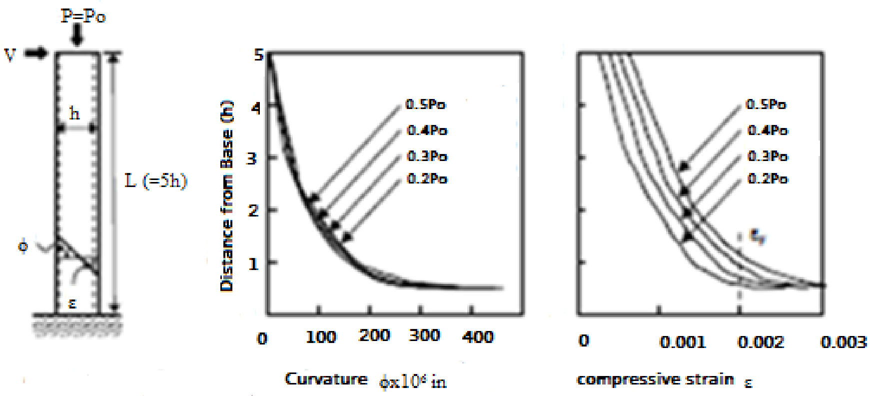
Experiments designed by Bae investigated the effect of axial load, opening to depth of the sections and longitudinal reinforcement ratio on the plastic hinge length variation. Column specimens had 61 cm X 44.4 cm square cross sections and reinforcement ratios are between percent0.72 to percentage2.05 according to the ASTM A370-03. His test results can summaries as fallow:

- From five column tests, observed plastic hinge length with specific conditioned mentioned on (Table 2.1); assumed as average plastic length in cycles at descending branch of concrete strength-strain diagram shown in table.

Table 2.2 :Evaluation of plastic hinge length by Sungjin Bae (2005).

Specimen	Cycle #	Equivalent Plastic Hinge Length l_p/h	Equivalent Plastic Hinge Length Average	P/P_o	L/h	Photograph
S24-2UT	14	0.64	0.66	0.5	5	
	17	0.66				
	20	0.70				
	23	0.68				
	26	0.61				
S17-3UT	14	1.02	0.91	0.5	7	
	17	0.89				
	20	0.82				
	23	0.80				
S24-4UT	14	0.57	0.49	0.2	5	
	17	0.52				
	20	0.50				
	23	0.43				
	26	0.44				
S24-5UT	14	0.50	0.47	0.2	5	
	17	0.50				
	20	0.44				
	23	0.41				

- The effect of axial load showed in Figure 2.2, although P/P0 changes has a little effect on curvature distribution diagram but tests illustrates highly tolerance on compressive deflections.



(a) Curvature Profile **(b) Compressive Strain Profile**

Figure 2.1 : Effect of axial load on curvature and compressive strain profiles (S.Bae 2005).

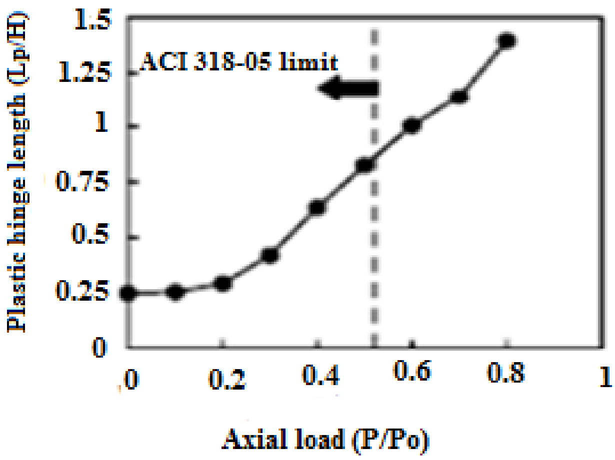


Figure 2.2 : Relationship between plastic hinge length and axial load (S.Bae 2005).

Bae consolidate with increasing of axial load ratio on columns the plastic hinge length increase. As it showed in fig L_p has intangible changes in low axial loads while in higher P/P0 rates increase of L_p become more significant. It is notable that linear changes on axial load ratio dose not mean the same behavior on plastic hinge length changes on a specific element.(Figure 2.3)

Bae considered section opening to depth of the section and longitudinal reinforcement ratio in his proposed formula. At certain axial load level plastic hinge length increased with the increase of section opening. Also longitudinal reinforcement ration increase effects L_p in ascending way. Figure 2.4 illustrates results.

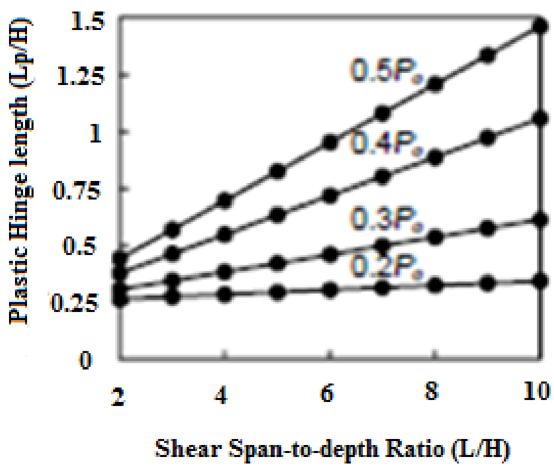


Figure 2.3 : Relationship between plastic hinge length and shear span-depth ratio (S.Bae 2005).

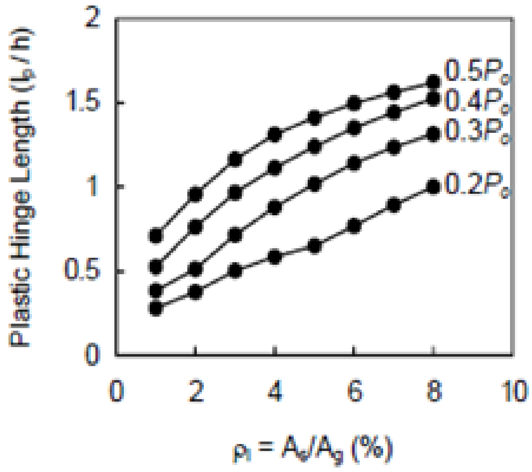


Figure 2.4 : Effect of longitudinal reinforcement (S.Bae 2005).

2.2.1. DBYBHY 2007

By Turkey republic published and approved from ISKAN minister DBYBHY 2007, include design codes to resistant structure buildings against seismic loads, performance assessment and retrofitting of existing substandard buildings, comments. Here is summery of the rules for performance of the existing buildings assessment that have been used in this code and you can find more details on chapter 7 (evaluation and retrofitting of the existing buildings) of Turkish seismic code 2007.

2.2.1.1.Damage limits

For ductile elements three limit conditions have been defined, (MN) minimum damage limit, (GV) safety limit and (GÇ) collapsing limit. Minimum damage limit refers to the beginning of the beyond elasticity behavior, safety limits defines as the behavior beyond elasticity that section is capable of safely ensuring the strength, and collapsing limit refers to ultimate limit for before collapsing behavior of the column.

If damages on elements do not reach MN are consider within the Minimum Damage region, those stayed between MN and GV boundary limits are within Significant Damage region, those passes GV boundary limit and stayed under the values of GÇ damage limit are define as Extreme Damage region and elements witch going beyond GÇ are considered as collapsed.(Figure 2.6)

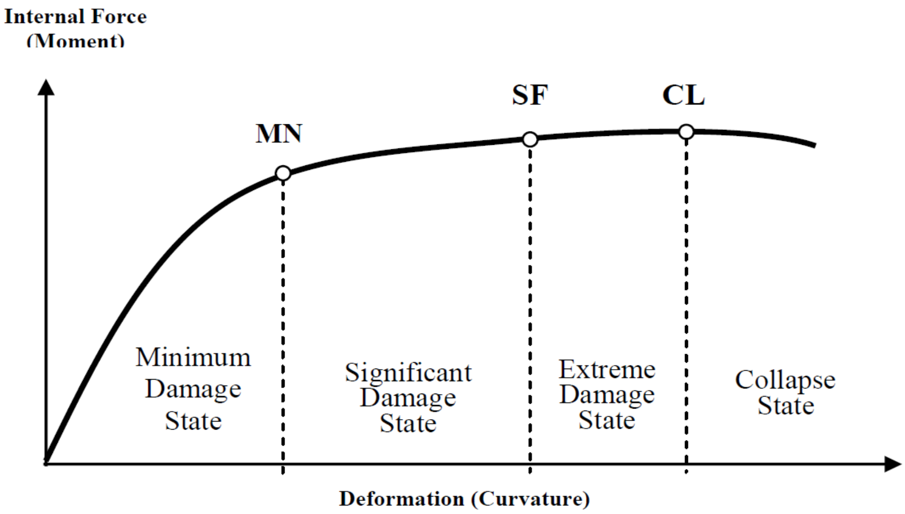


Figure 2.5 : Deformation curve under effect of internal forces. (DBYBHY 2007).

2.2.1.2.Deformation demands

The plastic bending demand depend on the θ_p plastic rotation demand that can obtained as a result of the repulsion analysis or obtained as an output information onto the calculation conducted within the time definition shall be calculated as follows;

$$\phi_p = \frac{\theta_p}{L_p} \tag{2.5}$$

$$L_p = 0.5H \tag{2.6}$$

The plastic hinge length of the cross section according to Turkish Seismic Code is taken as equation 2.6 and the ultimate curvature demand ϕ_u of the section obtained from adding the ϕ_y equivalent yield curvature with the ϕ_p plastic bending demand shows within the Equation 2.7. Yield curvature is defined from the bilinear momentum-curvature relationship obtained from the analysis conducted under the axial force demand of the section by means of using a reinforcement steel model that as well considers the strain hardening together with a concrete model chosen in accordance

$$\phi_u = \phi_y + \phi_p \tag{2.7}$$

Figure 2.5 illustrates the general principle of all calculations of structural performance limits that proposed by Turkish seismic code 2007.

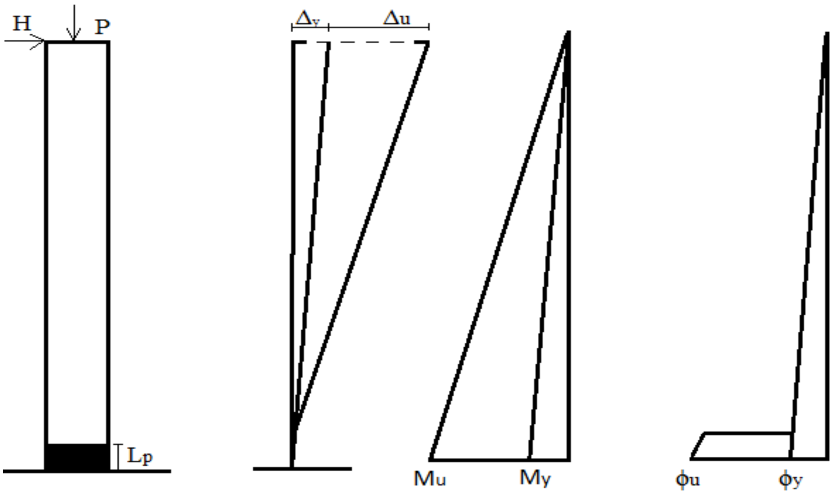


Figure 2.6 : Plastic hinge definition according to Turkish Seismic Code 2007.

2.2.1.3.Material behavior

Material parameters for moment-curvature analysis for evaluation with methods mentioned stress-strain curves for unconfined and confined concrete, is defined according to Mander et al. (1988) models. All information about this material model defined sections 4.

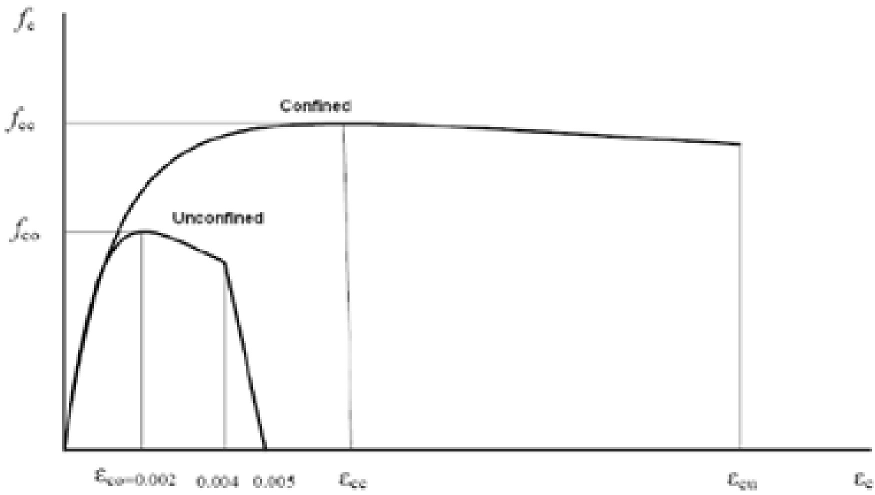


Figure 2.7 : Stress-strain relation for confined and unconfined concrete. (DBYBHY 2007)

The material models for unconfined and confined concrete in Turkish Seismic Code 2007, only differs from Mander et al. material models (1988) for maximum strain value of confined concrete. This value given as below;

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s\ f_{yw}\ \varepsilon_{su}}{f_{cc}} \quad \textbf{(2.8)}$$

Here; ρ_s= steel ratio, f_{yw}= yielding strength and ε_{su}=strain, of transverse reinforcements.

On the performance evaluation, stress-strain curves for reinforcement steel are given according to steel reinforcement quality in S220 and S420.the concerning information is showed in Table 2.2.

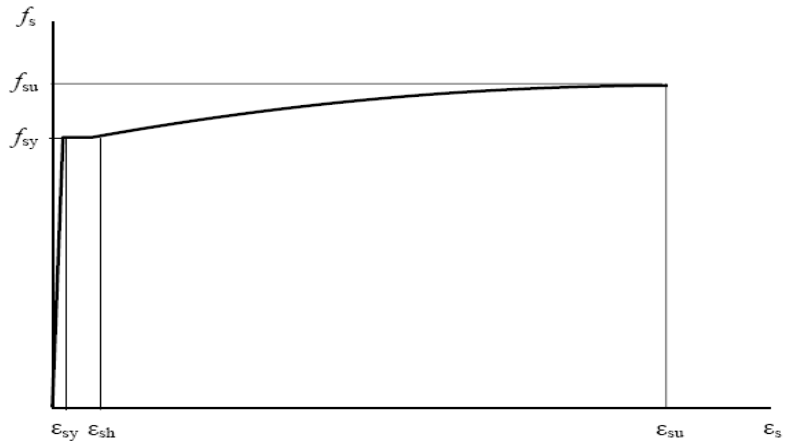


Figure 2.8 : Stress-strain relation for steel reinforcement. (DBYBHY 2007)

Table 2.3 : Information concerning reinforcement steel with quality S220 and S420

Quality	**f_{sy}**	**ε_{sy}**	**ε_{sh}**	**ε_{su}**	**f_{su}(Mpa)**
S220	220	0.0011	0.011	0.16	275
S420	420	0.0021	0.008	0.10	550

2.2.1.4.Determination of damage boundaries

The capacity of deformation for different sectional damages defines as the damage limits boundary.

For Minimum sectional damage boundary (MN), upper bounds of the concrete untie pressure deformation in the outer fiber of hoop and the reinforcement steel unit deformation demands:

$$(\varepsilon_{cu})_{MN} = 0.0035 \quad \textbf{(2.9)}$$

$$(\varepsilon_s)_{MN} = 0.01 \quad \textbf{(2.10)}$$

For safety bound (GV) upper bounds of the concrete unite pressure deformation in the outer fiber of hoop and the reinforcement steel unit deformation demands:

$$(\varepsilon_{cu})_{GV} = 0.0035 + 0.01(\rho_s/\rho_{sm}) \leq 0.0135 \quad \textbf{(2.11)}$$

$$(\varepsilon_s)_{GV} = 0.04 \quad \textbf{(2.12)}$$

For Section Collapse Bound (GÇ), upper bounds of the concrete unit pressure deformation in the outmost fiber of hoop and the reinforcement steel unit deformation demands:

$$(\varepsilon_{cu})_{GÇ} = 0.004 + 0.013(\rho_s/\rho_{sm}) \leq 0.018 \qquad \textbf{(2.13)}$$

$$(\varepsilon_s)_{GÇ} = 0.06 \qquad \textbf{(2.14)}$$

According to the DBYBHY 2007 part 3.3.4 (transverse reinforcement conditions) minimum steel reinforcement ration obtained as maximum amount of:

$$P_{sm} \geq 0.45\ [(A_c/A_{ck})-1](f_{cc}/f_{ywh}) \qquad \textbf{(2.15)}$$

$$P_{sm} \geq 0.12\ (f_{cc}/f_{ywh}) \qquad \textbf{(2.16)}$$

In these equations; ε_{cu}, the compressive strain of concrete; ε_s, the strain of reinforcement steel; ρ_s, current steel reinforcing ratio of special earthquake hoops and crossties in cross section; ρ_{sm}, the required steel reinforcing ratio of special earthquake hoops and crossties in cross section.

2.2.2. Eurocode 8, 2003

The European committee of standard released Euro Code 8 in 2005 contain comments about existing building performance assessment and retrofitting application. Part 3 of the draft mentioned reinforced concrete, steel and composite structures calculation rules against seismic loads and performance assessment (part3: Assessment and Retrofitting of Buildings). The scope of this code is as follows:

- Provide criteria for the evaluation of the seismic performance of existing individual buildings.
- Necessary measures corrective, approach description.
- Set forth criteria for the design of retrofitting measures.

2.2.2.1. Damage limit states

The fundamental requirements refer to the damage state in the structure, in Eurocode defined through three Limit States, namely Near Collapse (NC), Significant Damage (SD), and Damage Limitation (DL). These Limit States shall be characterized as follows:

Limit States of Near Collapse (NC); the structure is heavily damaged, with low residual lateral stiffness and strength, although vertical elements are still sustaining against vertical loads. Most nonstructural elements have collapsed. Large permanent drifts are present. The near collapse structure would probably unable to survive other seismic loads, even of moderate intensity.

Limit States of Significant Damage (SD), the structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infill have not failed out-of-plane.

Moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity. The structure is likely to be uneconomic to repair.

Limit States of Damage Limitation (DL), the structure is only slightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infill's, may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible. The structure does not need any repair measures.

2.2.2.2.Determining the unit deformation

Chord rotation is stated as the deformation capacity of reinforced concrete members. According to Euro Code 8 chord rotation is the angle between the tangent to the axis at yielding end and the chord rotating at that end with the end of shear span (L_v=M/V=moment /shear at the end section). Other definition of chord rotation is equal to the element drift ratio, the deflection at the end of the shear span with respect to the tangent to the axis at the yielding end, divided by the shear span.

$$\theta_i = \frac{\delta_i}{L_{vi}} = \left|\frac{\Delta}{L} - \phi_i\right| \quad \textbf{(2.17)}$$

Here; θ_i, chord rotation; δ_i, deflection occurred because of difference of chord rotation and rotation of bottom joint (i); Δ the deflection of top joint (j); L is the height of the column; φ_i, rotation of bottom joint (i).

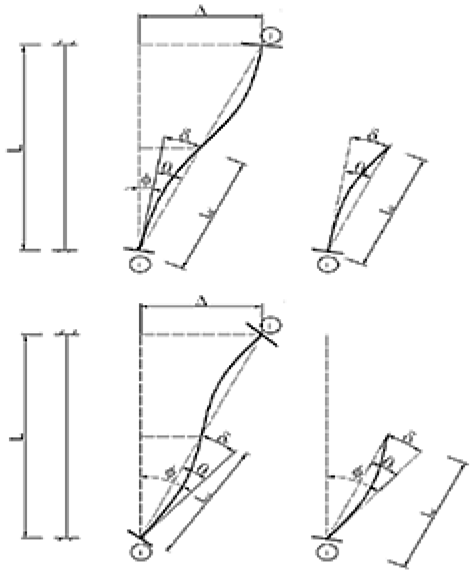


Figure 2.9 : Definition of chord rotation (Ozal 2005)

2.2.2.3.Ultimate chord rotation and curvature

According to Euro code 8 parts 2; When no test results are available for the direct estimation of the ultimate displacement, this estimation may be based on curvature integration along the member, where the ultimate curvature Φ_u at the plastic hinge of the member is taken as:

$$\Phi_u = \frac{\varepsilon_s - \varepsilon_c}{d} \quad \textbf{(2.18)}$$

d is the effective section depth, ε_s and ε_c are the reinforcement and concrete strains respectively (compressive strains negative), derived from the condition that either of the two or both have reached the following ultimate values:

• Compression strain of unconfined concrete ε_{cu1}= 0, 0035 (EN 1992-1-1 for $f_{ck} \leq 50$ MPa)

• Compression strain of confined concrete, corresponding to the first fracture of confining hoop reinforcement (Mander model)

$$\varepsilon_{cu,c} = 0.004 + \frac{1.4 \rho_s f_{ym} \varepsilon_{su}}{f_{cm,c}} \quad \textbf{(2.19)}$$

$\rho_s = \rho_w$ for circular spirals or hoops, $\rho_s = 2\rho_w$ for orthogonal hoops, and ε_{su}=ε_{uk}=0.075 is the characteristic value of the confining reinforcement steel elongation at maximum force (EN 1992-1-1 for Class C steel)

• Ultimate tensile strain of reinforcement $\varepsilon_{su} = \varepsilon_{uk} = 0.075$ (EN 1992-1-1, Class C steel)

The plastic rotation capacity $\theta_{p,u}$, and the total chord rotation θ_u of plastic hinges (see Fig.) may be estimated on the basis of the ultimate curvature Φ_u and the plastic hinge length Lp as follows:

$$\theta_U = \theta_y + \theta_{p,u} \tag{2.20}$$

$$\theta_{p,u} = (\Phi u - \Phi y)Lp(1 - \frac{Lp}{2l}) \tag{2.21}$$

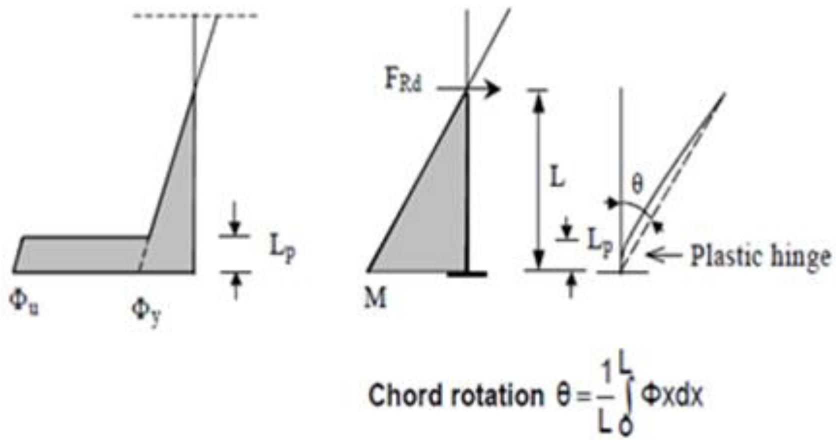


Figure 2.10 : Yield rotation assumption (Eurocode 2008)

$$\theta_y = \frac{\phi_y\ L}{3} \tag{2.22}$$

Both Φ_y and Φ_u should be assessed by means of a moment curvature analysis of the section under the axial load corresponding to the design seismic combination. When $\varepsilon_c \geq \varepsilon_{cu1}$, only the confined concrete core section should be taken into an account. Φ_y should be evaluated by idealizing the actual M-Φ diagram by a bilinear diagram of equal area beyond the first yield of reinforcement as shown in Fig 2.12

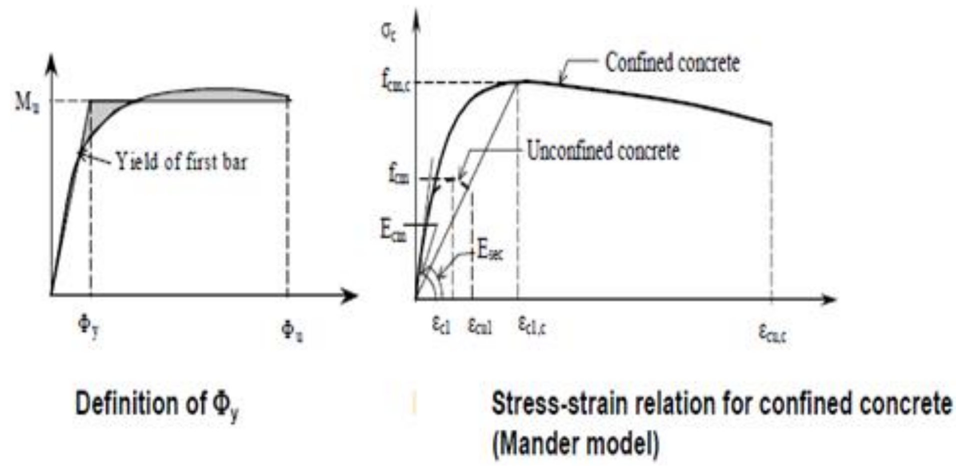


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Figure 2.11 : Stress-strain relation for combined concrete

The above estimation of the plastic rotation capacity is valid for piers with shear ratio α_s=L/d ≥ 3.0 For 1,0 ≤ α_s < 3,0 the plastic rotation capacity should be multiplied by the reduction factor $\lambda_{(\alpha s).}$

2.2.2.4.Determination of damage boundaries

The capacity for Limit state of damage limitation (DL) for beam and columns is given by chord rotation at yielding $\theta_{y,}$ evaluated as:

$$\theta_y = \varphi_y \frac{L_v + a_v z}{3} + 0.00135\left(1 + 1.5\frac{h}{L_v}\right) + \frac{\varepsilon_y}{d - d\prime}\frac{d_b f_y}{6\sqrt{f_c}} \quad \textbf{(2.23)}$$

Or from alternative expression for beam and columns;

$$\theta_y = \varphi_y \frac{L_v + \alpha_v z}{3} + 0.0013\left(1 + 1.5\frac{h}{L_v}\right) + 0.13\varphi_y \frac{d_b f_y}{6\sqrt{f_c}} \quad \textbf{(2.24)}$$

In this equation; φy, the yield curvature of the end section; αvz, the tension shift of the bending moment diagram (EN 1992-1-1: 2004, 9.2.1.3(2)); fy, the steel yield stress; fc, the concrete strength; _y, equal to fy/Es; d and d’, the depths to the tension and compression reinforcement; db, the diameter of the tension reinforcement.

The chord rotation for significant damage (SD) should be equal or less than 75% (θSD3/4θum) of the chord rotation calculated for limit state of near collapse (NC).

The limit chord rotation for near collapse (NC) should be equal or less than the value of the total chord rotation capacity at ultimate ($\theta_{NC} < \theta_{um}$) θ_{um}, of concrete members under cyclic loading may be calculated from the following expression:

$$\theta_{um} = \frac{1}{\gamma_{el}} 0.016(0.3^{v})\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.225} \left(\frac{Lv}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}(1.25^{100\rho_d}) \quad \textbf{(2.25)}$$

Where: $\Upsilon_{el:}$ is equal to 1.5 for primary seismic elements and to 1,0 for secondary seismic. h: is the depth of cross-section. L_V = M/V, is the ratio moment/shear at the end section. υ, is equal to ratio of N/bhf_c. ω and ω’, is the mechanical reinforcement ratio of longitudinal reinforcement. f_c, is the concrete compressive strength (MPa). f_{yw}, is the stirrup yield strength (MPa). ρ_{sx}, is the ratio of transverse steel parallel to the direction x of loading. ρ_d, is the steel ratio of diagonal reinforcement. α, is the confinement effectiveness factor, that may be taken equal to:

$$\alpha = \left(1 - \frac{S_h}{2b_0}\right)\left(1 - \frac{S_h}{2h_0}\right)\left(1 - \frac{\sum b_i^2}{6h_0 b_0}\right) \qquad \textbf{(2.26)}$$

Here; b_0 and ho, are the dimension of confined core to the centerline of the hoop; b_i, is the centerline spacing of longitudinal bars (indexed by i) laterally restrained by a stirrup corner or a crosstie along the perimeter of the cross-section. These definitions illustrates in Figure;

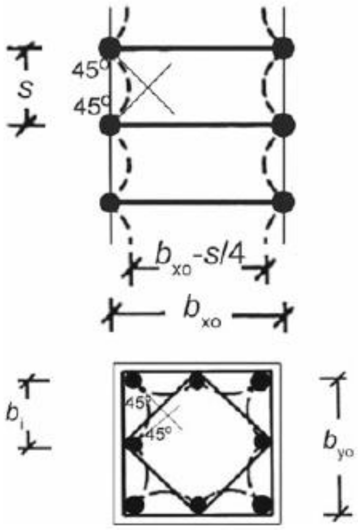


Figure 2.12 : Confined and unconfined parts over the cross-section and along a member with rectangular section (Fardis, 2009).

Confined and unconfined parts over the cross-section and along a member with square section and multiple ties (Fardis, 2009)

For ultimate chord rotation capacity evaluation and alternative expression may be used:

$$\theta_{um} = \frac{1}{\gamma_{el}}\left(\theta_y + (\phi_u - \phi_y)L_p\left(1 - \frac{0.5L_p}{L_v}\right)\right) \qquad \textbf{(2.27)}$$

2.2.2.5.Behavior of moment-curvature of cross section

In this section, all material behavior of structural materials is given according to Euro code 2: Design of concrete structures and Part 3: Assessment and retrofitting of buildings of Euro code 8. In calculations of thesis study, the unconfined concrete model is taken according to Section 3.1.5 of Euro code. The stress-strain relationship of unconfined concrete can be seen in (Fig) and material parameters are given in (Table) based on Table 3.1 of EN1992-1-1. For confined concrete, confinement of concrete results in a modification of effective stress-strain relationship: higher strength and higher critical strains are achieved.

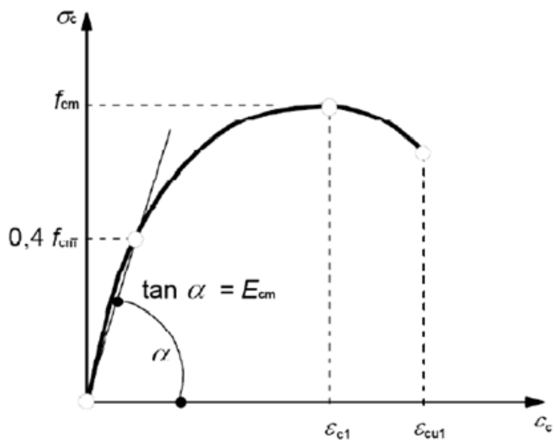


Figure 2.13 : Schematic representation of stress-strain relation for structural analysis (EN1992-1-1,2008).

Table 2.4 : Strength classes characteristics of concrete(EN1992-1-1,2004)

Strength Classes for Concrete									
$f_{ck(MPa)}$	12	16	20	25	30	35	40	45	50
$f_{cm(MPa)}$	20	24	28	33	38	43	48	53	68
$E_{cm(GPa)}$	27	29	30	31	33	34	35	36	37
$\varepsilon_{c1(\%)}$	1.8	1.9	2	2.1	2.2	2.25	2.3	2.4	2.45
$\varepsilon_{cu1(\%)}$					3.5				
$\varepsilon_{c2(\%)}$					2				
$\varepsilon_{cu2(\%)}$					3.5				

The other basic material characteristics may be considered as unaffected for design. In the absence of more precise data, the stress-strain relation shown in Fig (compressive strain shown positive) may be used, with increased characteristic strength and strains according to:

The strength of confined concrete is evaluated from;

$$f_{cc} = f_c\left[1 + 3.7\left(\frac{\alpha\rho_{sx}f_{yw}}{f_c}\right)^{0.86}\right] \qquad \textbf{(2.28)}$$

The strain at which the strength f_{cc} takes place is taken to increase over the value ε_{c2} of unconfined concrete as:

$$\varepsilon_{c2,c} = \varepsilon_{c2}\left[1 + 5\left(\frac{f_{cc}}{f_c} - 1\right)\right] \quad \textbf{(2.29)}$$

The ultimate strain of the extreme fiber of the compression zone is taken as:

$$\varepsilon_{cu2,c} = 0.004 + 0.5\varepsilon_{c2}\,\frac{\alpha\,\rho_{sx}\,f_{yw}}{f_{cc}} \quad \textbf{(2.30)}$$

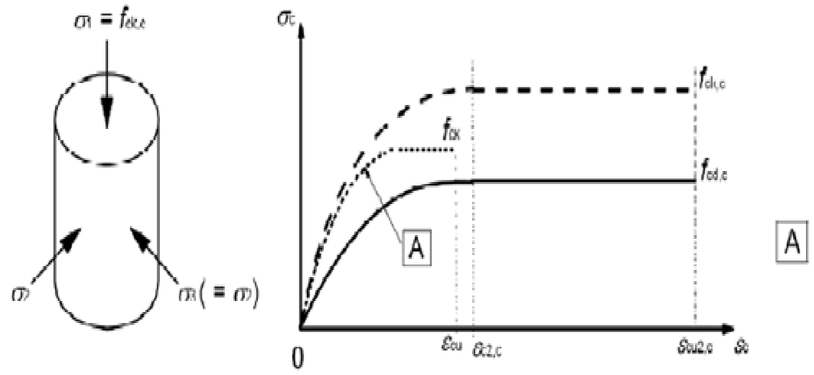


Figure 2-14 : Stress-strain relation for confined concrete (EN1992-1-1,2008).

2.2.2.6.The plastic hinge length

There are two expressions evaluated here are those given in Annex A of Part 3 of Euro code 8 for cyclic loading of members detailed for earthquake resistance:

• For members with detailing for earthquake resistance and without lapping of longitudinal bars approximately the section where yielding expected, L_{pl} may calculated from the following expression:

$$L_{pl} = 0.1L_v + 0.17h + 0.24\frac{d_{bl}\,f_y\,(MPa)}{\sqrt{f_c(MPa)}} \quad \textbf{(2.31)}$$

And; for members with detailing for earthquake resistance and no lapping of Longitudinal bars near the section where yielding is expected, L_{pl} may be calculated from the following expression:

$$L_{pl} = \frac{L_v}{30} + 0.2h + 0.11\frac{d_{bl}\,f_y\,(MPa)}{\sqrt{f_c\,(MPa)}} \quad \textbf{(2.32)}$$

Where h is the depth of the member and d_{bL} is the (mean) diameter of the tension reinforcement.

If the confinement model in EN1992-1-1: 2004 3.1.9 is used for calculations of the ultimate curvature of the end section, ϕ_u, and the value of L_{pl} from first equation is used, then the factor Υ_{el} therein may be taken equal to 2 for primary seismic and to 1.0 for secondary seismic elements. In this study, the confinement model given in Annex A, of Part 3 of Euro code 8 is used, together with second equation, and then the value of the factor Υ_{el} may be taken equal to 1.7 for primary seismic elements and to 1.0 for secondary seismic ones.

It is noteworthy that according to model code 2010 that is early condition of next Euro code plastic hinge length mentioned as:

$$L_p = 0.2h\ (1+ \tfrac{1}{3}\mathrm{MIN}\ (9\ , \tfrac{L_v}{h})\quad \text{for rectangular columns} \qquad \mathbf{(2.33)}$$

To have better vision of the code L_p considers as part of analysis ether.

3. PEER STRUCTURE PERFORMANCE DATABASE

The Pacific Earthquake Engineering Research (PEER) Center sponsored a project to create a Structural Performance Database (SPD) in 2003, consisting of 416 reinforced concrete columns that were tested by various researchers under cyclic lateral loads (PEER 2011). This project was assembled to service the earthquake engineering research community and may be accessed online by the public. The PEER Structure Performance Design contains columns of both rectangular and circular cross- section, but only cantilever rectangular columns considered in this study. Information available for these columns includes column geometry, material properties, reinforcing details, test configuration, axial load level, and force-displacement history of test data and references of documented tests. The database also gives specific displacement values at each of six damage states in specific columns. These damage states are: spoiling of the unconfined concrete, crushing of the confined concrete, buckling of longitudinal reinforcement, fracture of longitudinal reinforcement, fracture of transverse reinforcement and failure. These values were obtained from the documentation available for each test; therefore, not every column has values recorded for each of the noted six damage states.

3.1. Selected Columns

At The PEER SPD contains 306 rectangular columns; therefore in order to be considered relevant to this study, columns used for analyses were selected according to specific criteria. The criteria for chosen columns are as follows:

• Column failure type must be due to flexure. Columns resulting in shear failure or shear- flexure failure were eliminated. The flexural failure can be consider as longitudinal bar buckling and concrete crushes in compression part of the cross section. Concrete crush assumed as 20 percent strength loss of concrete strength capacity under loadings.

• Columns type must categorize as cantilever columns. This simplification is due to the representation of the column by its effective shear span, as shown in Figure, where the free-end in the simplified model represents the point of contra-flexure in the actual column. The inelastic region of a cantilever column when subjected to horizontal loading is located near the confined base, where the largest bending moment demands occur, and it is detailed for ductile response according to capacity design principles (Priestley et al. 1996). This is shown in Figure 2-3b,c where L is the effective shear span of the column. Since the response of a well-design column is mainly due to flexure, the behavior of the column may be extrapolated from its section behavior at the base represented by a moment-curvature response.

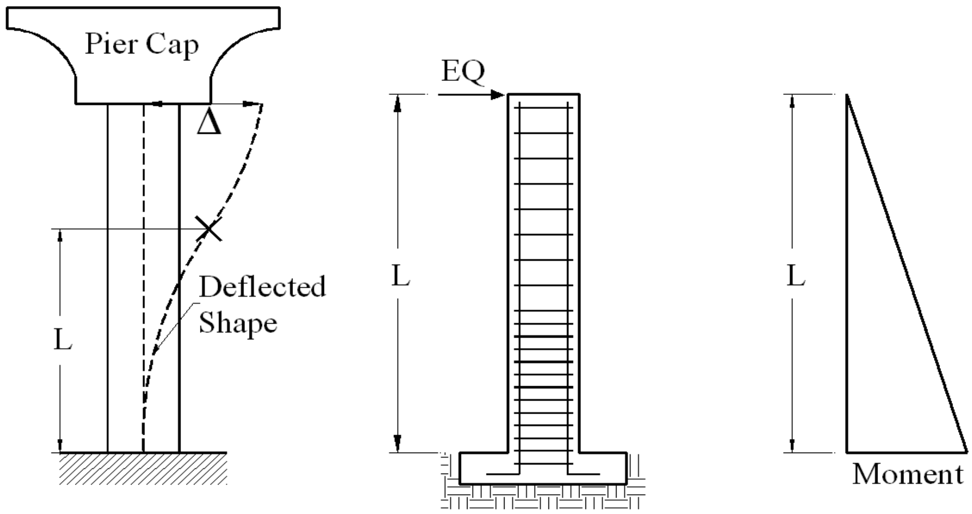
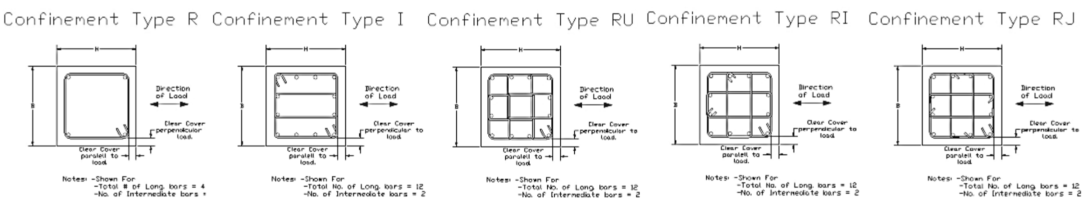


Figure 3.1: Cantilever column detail.

3.2. Confinement Details

To have a better vision of different confinement effects type on behavior of columns all confinement types are selected. According to peer column data base confinements are separated to 9 groups that details illustrates in fig;



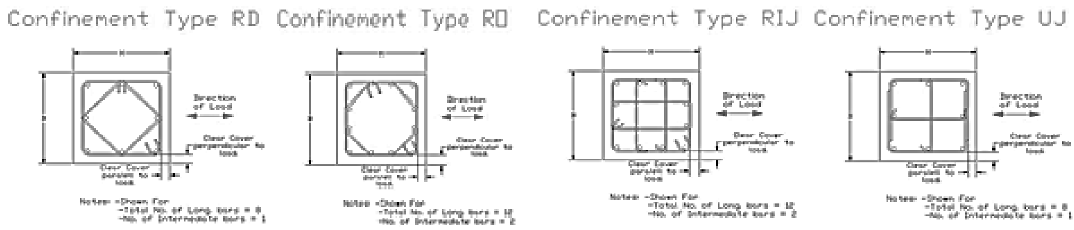


Figure 3.2 : Detail of transverse reinforcement of cross sections.

Lastly, columns without recorded limit state data were eliminated since they offer no comparison to the analyses performed. Force-Displacement Experimental Response:

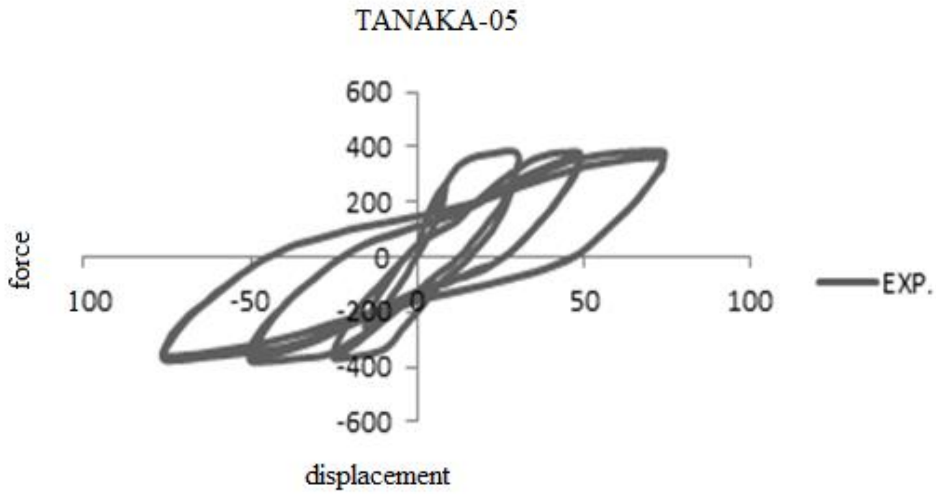


Figure 3.3 : Force-displacement relationship diagram (Peer Database).

The resulting 57 columns used in this study are shown in Table. The table presents information on each column's geometric properties, axial load and reinforcement properties. Properties shown include the column's aspect ratio (L/D), axial load ratio and reinforcement ratios. The axial load ratio is expressed as the percentage of load over the column's nominal crushing capacity:

Axial load ratio= $P/_{f_c'\ A_g}$

Where P is the axial force, f'_c is the concrete compressive strength and A_g is the gross cross sectional area of the column.

Table 3.1 states general information of the concerned columns; the details and section properties attached to Annex.

Table 3.1 : Database of analyzed features

NO	specimen name	BxH (M)	L (M)	F_c' (MPa)	Axial load (KN)
1	Tanaka and Park 1990.no5	0.55x0.55	1.65	32	968
2	Tanaka and Park 1990.no6	0.55x0.55	1.65	32	968
3	Tanaka and Park 1990.no7	0.55x0.55	1.65	32.1	2913
4	Tanaka and Park 1990.no8	0.55x0.55	1.65	32.1	2913
5	Park and Paulay 1990.no9	0.4x0.6	1.78	26.9	646
6	Ohno and Nishioka 1984,L1	0.4x0.4	1.60	24.8	127
7	Ohno and Nishioka 1984,L2	0.4x0.4	1.6	24.8	127
8	Ohno and Nishioka 1984,L3	0.4X0.4	1.6	24.8	127
9	Saatcioglu and Ozcebe 1989,U1	0.35X0.35	1	43.6	0
10	Saatcioglu and Ozcebe 1989,U3	0.35X0.35	1	34.8	600
11	Saatcioglu and Ozcebe 1989,U4	0.35X0.35	1	32	600

NO	specimen name	BxH (M)	L (M)	F_c' (MPa)	Axial load (KN)
12	Saatcioglu and Ozcebe 1989,U6	0.35X0.35	1	37.3	600
13	Saatcioglu and Ozcebe 1989,U7	0.35X0.35	1	39	600
14	Wehbe et al. 1998,A1	0.38x0.61	2.335	27.2	615
15	Wehbe et al. 1998,A2	0.38X0.61	2.335	27.2	1505
16	Wehbe et al. 1998,B1	0.38X0.61	2.335	28.1	601
17	Wehbe et al. 1998,B2	0.38X0.61	2.335	28.1	1514
18	Saatcioglu and Grira 1999,BG-1	0.35x0.35	1.645	34	1782
19	Saatcioglu and Grira 1999,BG-2	0.35x0.35	1.645	34	1782
20	Saatcioglu and Grira 1999,BG-3	0.35x0.35	1.645	34	831
21	Saatcioglu and Grira 1999,BG-4	0.35x0.35	1.645	34	1923

NO	specimen name	BxH (M)	L (M)	F_c' (MPa)	Axial load (KN)
22	Saatcioglu and Grira 1999,BG-5	0.35x0.35	1.645	34	1923
23	Saatcioglu and Grira 1999,BG-6	0.35x0.35	1.645	34	1900
24	Saatcioglu and Grira 1999,BG-7	0.35x0.35	1.645	34	1923
25	Saatcioglu and Grira 1999,BG-8	0.35x0.35	1.645	34	961
26	Saatcioglu and Grira 1999,BG-9	0.35x0.35	1.645	34	1923
27	Saatcioglu and Grira 1999,BG-10	0.35x0.35	1.645	34	1923
28	Mo and Wang 2000,C1-1	0.4x0.4	1.4	24.8	450
29	Mo and Wang 2000,C1-2	0.4x0.4	1.4	26.7	675
30	Mo and Wang 2000,C1-3	0.4x0.4	1.4	26.1	900
31	Mo and Wang 2000,C2-1	0.4x0.4	1.4	25.3	450
32	Mo and Wang 2000,C2-2	0.4x0.4	1.4	27.1	675
33	Mo and Wang 2000,C2-3	0.4x0.4	1.4	26.8	900

NO	specimen name	BxH (M)	L (M)	F_c' (MPa)	Axial load (KN)
34	Mo and Wang 2000,C3-1	0.4x0.4	1.4	26.4	450
35	Mo and Wang 2000,C3-2	0.4x0.4	1.4	27.5	675
36	Mo and Wang 2000,C3-3	0.4x0.4	1.4	26.9	900
37	Bechtoula, Arai and Watanabe2002,D1N30	0.25x0.25	0.625	37.6	705
38	Bechtoula, Arai and Watanabe2002,D1N60	0.25x0.25	0.625	37.6	1410
39	Bechtoula, Arai and Watanabe2002,L1D60	0.6x0.6	1.2	39.2	8000
40	Bechtoula, Arai and Watanabe2002,L1N60	0.6x0.6	1.2	39.2	8000
41	Bechtoula, Arai and Watanabe2002,L1N6B	0.56X0.56	1.2	32.2	6000
42	Takemura 1997,TEST1	0.4x0.4	1.245	35.9	157
43	Takemura 1997,TEST2	0.4x0.4	1.245	35.7	157
44	Takemura 1997,TEST3	0.4x0.4	1.245	34.3	157

NO	Spicemen name	BxH (M)	L (M)	F_c' (MPa)	Axial load (KN)
45	Takemura 1997,TEST4	0.4x0.4	1.245	33.2	157
46	Bayrak & Sheikh 2002 No.RS-9HT	0.25x0.35	1.842	71.2	2118
47	Bayrak & Sheikh 2002 No.RS-10HT	0.25x0.35	1.842	71.1	3111
48	Bayrak & Sheikh 2002 No.RS-11HT	0.25x0.35	1.842	70.8	3159
49	Bayrak & Sheikh 2002 No.RS-12HT	0.25x0.35	1.842	70.9	2109
50	Bayrak & Sheikh 2002 No.RS-15HT	0.25x0.35	1.842	56.2	1770
51	Bayrak & Sheikh 2002 No.RS-16HT	0.25x0.35	1.842	56.2	1819
52	Bayrak & Sheikh 2002 No.RS-17HT	0.25x0.35	1.842	74.1	2204
53	Bayrak & Sheikh 2002 No.RS-18HT	0.25x0.35	1.842	74.1	3242
54	Bayrak & Sheikh 2002 No.RS-19HT	0.25x0.35	1.842	74.2	3441
55	Bayrak & Sheikh 2002 No.RS-20HT	0.25x0.35	1.842	74.2	2207

NO	Spicemen name	BxH (M)	L (M)	F_c' (MPa)	Axial load (KN)
56	Bayrak & Sheikh 2002 No.RS-23HT	0.35x0.25	1.842	72.2	2085
57	Bayrak & Sheikh 2002 No.RS-24HT	0.35x0.25	1.842	72.2	3159

4. MATERIALS BEHAVIOR MODEL

The conditions that must be provided in structural engineering problems are listed as;

-equilibrium conditions,

-compatibility conditions,

-Stress-strain relation in materials,

Material behaviors are considered in the last step that has substantial effects on stress-strain relation and diagram. From this fact, to have a better comparison between column analyses, Mander et al. Model (1988) assumed as basis for all columns in this study.

4.1. Unconfined Material Behavior Model

J.B.Mander, M.J.N.Priestly and R.park in 1988 have developed a unified stress-strain model for confined concrete that can be applied to both circular and rectangular shaped transverse reinforcements. Fig shows the confined model, based on available equations. In this model, the concrete is assumed as unconfined when the effects of stirrups in confined case (that will be discussed in the next part) are neglected. In this manner, the effective confined pressure is taken zero; f_e=0. The curves equation will be like:

$$\sigma_c = \frac{fco \;\; x \;\; r}{r-1+x^r} \qquad \varepsilon_c<2\varepsilon_{c0} \qquad \textbf{(4.1)}$$

$$\sigma_c = f_{c0}(\frac{2r}{r-1+2^r})(1-\frac{\varepsilon c-2\varepsilon c0}{\varepsilon c\prime u-2\varepsilon c0}) \qquad 2\varepsilon_{c0}<\varepsilon_c<\varepsilon_{cu} \qquad \textbf{(4.2)}$$

$$x = \frac{\varepsilon c}{\varepsilon c0} \qquad \textbf{(4.3)}$$

$$r = \frac{Ec}{Ec-Esec} \qquad \textbf{(4.4)}$$

$$E_{sec} = \frac{fcc}{\varepsilon cc} \qquad \textbf{(4.5)}$$

$$E_c = 5000\sqrt{f_c} \quad (\text{MPa}) \qquad \textbf{(4.6)}$$

Where;ε_c=longitudinal compressive concrete strain, ε_{cc}=maximum strain in maximum stress, $\varepsilon_{c'u}$=ultimate strain of unconfined concrete, f_{cc} compressive strength of confined concrete, f_{co}=unconfined concrete strength, E_{sec}=tangent modulus of elasticity of concrete section.

The value of ultimate strain in unconfined model of Mander et al. is generally accepted as ε_{c0}=0.002. After the value of $2\varepsilon_{c0}$ the cover concrete starts to spoil, the falling branch part in the region is assumed to be linear up to zero stress, at the spalling strain ε_{sp}.

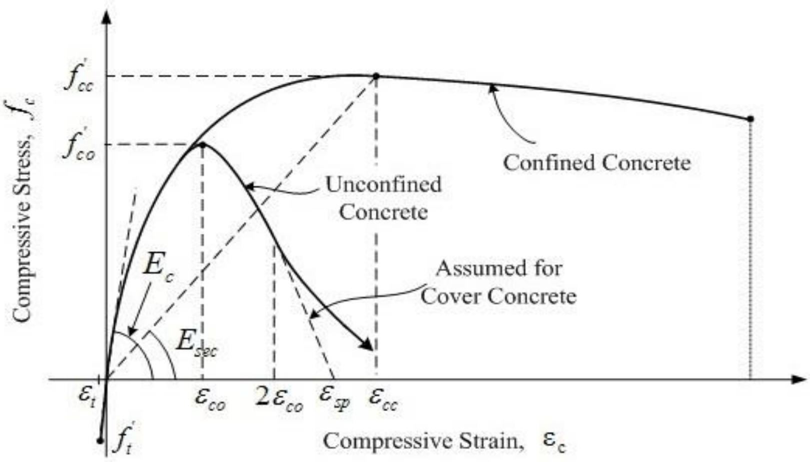


Figure 4.1 : Accepted concrete behavior model.

In this study, to have the same conditions in all column analysis, the strain values are assumed as:

ε_{c0}=0.002, $2\varepsilon_{c0}$=0.004, $\varepsilon_{c'u}$=0.005 (DBYBHY, 2007)

4.2. Confined Material Behavior Model

The confined model is obtained by multiplying confinement effectiveness coefficient λ_c to unconfined model. λ_c is defined as the function of the effective lateral confining pressure on the concrete section. The confined curvilinear equations are mentioned below;

$$\sigma_c = \frac{fcc\ x\ r}{r-1+x^r} \qquad \varepsilon_c<\varepsilon_{cu} \qquad \textbf{(4.7)}$$

$$\sigma_c = 0 \qquad \varepsilon_c>\varepsilon_{cu} \qquad \textbf{(4.8)}$$

$$\lambda_c = [2.245+\sqrt{\frac{7.94fe}{fc0}} - \frac{2fe}{fc0} - 1.254] \qquad \varepsilon_c<\varepsilon_{cu} \qquad \textbf{(4.9)}$$

$$f_{cc} = f_{c0}\ \lambda_c \qquad \varepsilon_c> \varepsilon_{cu} \qquad \textbf{(4.10)}$$

$$x = \frac{\varepsilon c}{\varepsilon cc} \quad \textbf{(4.11)}$$

$$\varepsilon_{cc} = \varepsilon_{c0}\,[1+5(\lambda_c-1)] \quad \textbf{(4.12)}$$

$$r = \frac{Ec}{Ec-Esec} \quad \textbf{(4.13)}$$

$$E_{sec} = \frac{fcc}{\varepsilon cc} \quad \textbf{(4.14)}$$

$$E_c = 5000\sqrt{f_c} \quad \text{(MPa)} \quad \textbf{(4.15)}$$

$$f_e = \frac{fex+fey}{2} \quad \textbf{(4.16)}$$

$$f_{ex} = k_e \frac{Ashx}{s\ b} f_{yh} \quad \text{(MPa)} \quad \textbf{(4.17)}$$

$$f_{ey} = k_e \frac{Ashy}{s\ b} f_{yh} \quad \text{(MPa)} \quad \textbf{(4.18)}$$

$$k_e = \frac{\left(1-\sum\frac{Wi^2}{6bh}\right)\left(1-\frac{s'}{2b}\right)\left(1-\frac{s'}{2\ ds}\right)}{1-\rho cc} \quad \textbf{(4.19)}$$

Where: A_{shx} and A_{shy} are the total area of the transverse bars running in the x and y directions respectively. s’: clear vertical spacing between spiral or hoop bars. f_{yh}: yield strength of the transverse reinforcement. f_e: average effective confinement compression. S: Center-to-center spacing or pitch of spiral or circular hoop, $k_{e:}$ the confinement effectiveness coefficient. ρ_{cc}: ratio of area of longitudinal reinforcement to area of core of section. d_s: diameter of spiral between bar centers. w_i: the i’th clear distance between adjacent longitudinal bars.

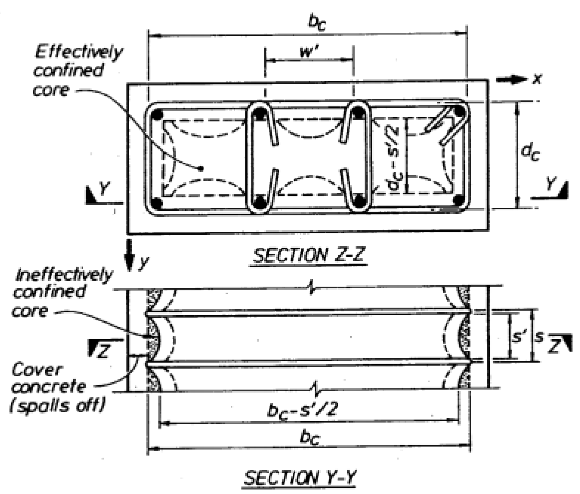


Figure 4.2 : Confined concrete detail.

4.3. Reinforcement Behavior Model

Mander in 1984, proposed the stress-strain relation can be regarded as a representative criterion for the various qualities of reinforcements.

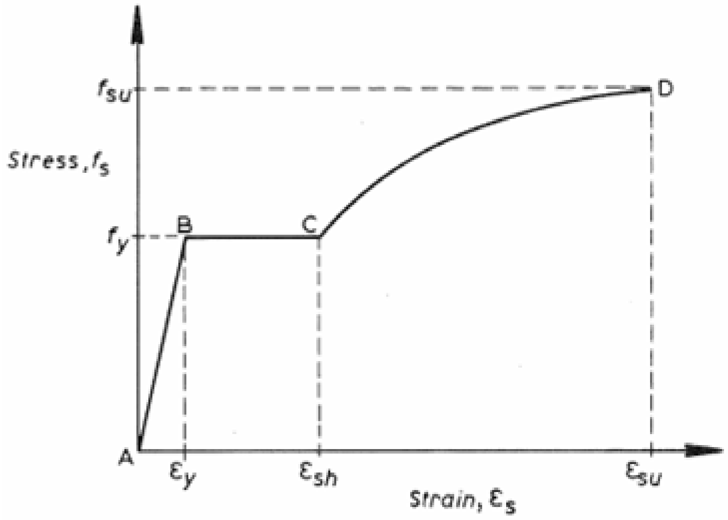


Figure 4.3 : Applied reinforcement behavior model.

In this model, the behavior of steel by yielding stress value of f_y is linear; in a higher amount of stress, the deflection of steel is being continued even in the constant stress, until it reaches the steel hardening state. The ultimate strain is reached after steel hardening, in ultimate stress value f_{su}. Based on a general acceptance, steel hardening is omitted and the deformation is continued up to the ultimate strain in constant stress f_y.

$$\sigma_s=E_s\,\varepsilon_s \qquad \varepsilon_s<\varepsilon_{sy} \tag{4.20}$$

$$\sigma_s=f_{sy} \qquad \varepsilon_{sy}<\varepsilon_s<\varepsilon_{sh} \tag{4.21}$$

$$\sigma_s=f_{su}-(f_{su}-f_{sy})\frac{(\varepsilon su-\varepsilon s)^p}{(\varepsilon su-\varepsilon sh)^p} \tag{4.22}$$

$$E_s=\frac{fsy}{\varepsilon sy} \tag{4.23}$$

$$P=E_{sh}\frac{(\varepsilon su-\varepsilon sh)}{(fsu-fsy)} \tag{4.24}$$

Where; σ_s=reinforcement stress, ε_s=reinforcement strain, ε_{sy}=strain of steel hardening, ε_{su}=ultimate strain of reinforcement, f_y=yield strength of reinforcement, f_u=ultimate strength of reinforcement, E_s=reinforcement elasticity modulus, E_{sh}=reinforcement hardening modulus.

5. NON-LINEAR ANALYSIS FOR REINFORCED CONCRETE COLUMNS

5.1. Moment-Curvature Analyze (X-tract)

In this study, the program of Xtract section analyzes, which is released by TRC software, is preferred. AVI FILE CREATOR (Xtract) allows the users to create a simulated animation of the section under the applied loading process, while simultaneously shows the graph of the response. This feature allows the user to export the results and to be able to use either in Power Point program or on the Internet. Furthermore, any loading type animations can be saved in an Avi file.

Some of the software abilities mentioned below;

Ability of bi-linearization of ideal moment-curvature response; In the previous versions of the XTRACT and UCFyber programs, the bi-linearization has been best fit with both the area under the moment curvature relation and the ultimate point, in the moment curvature response, history. In a recent released version of the program, the users are capable to choose both the post peak slope of the bi-linearization, and a user defined post peak slope.

- Ability to generate the plastic rotation capacity based on a code assumed as L_p. The user picks the plastic hinge length based on particular code and type, and enters the critical parameters; in addition, the calculated result is printed in the analysis report.
- Ability to fit the equation to the axial force-moment interaction diagram; in this analysis, the user can fit a cubic polynomial to the data. The equation parameters are printed along with the graph of the equation plotted against the actual detailed calculation of the points. The equation can be used on the other structural analysis programs that accept such inputs.

In this section, conditions and assumptions are considered and described at the same order as being applied in the analyses steps. No5 column analysis, which is tested by

Tanaka and Park in 1990, is described. All the required details of the columns are shown and extracted from peer column database, attached to Annex A.

Table 5.1 : Tanaka and Park 1990, NO5 column details (Peer Database)

specimen	F'c (MPa)	Axial load (KN)	B (mm)	H (mm)	L (mm)	Config.	Diameter (mm)	Total # Bars	Clear Cover (mm)	Fyl (MPa)	fsu (MPa)
Tanaka and Park No. 5	32	968	550	550	1650	C	20	12	40	511	675

a) Section definition: Sections are named according to the ''name of researcher-name of the column''. The analyses of the columns start from the template mode, which are based on the model of Mander et al. Confinement details and Cross sectional information are entered as reported in the tests and database.

b) Material definition: behavior, elasticity modulus, and properties connected to material property considered as the same as template mode of the program, selected from its startup. Additionally, the concrete strength value entered as reported in the database.

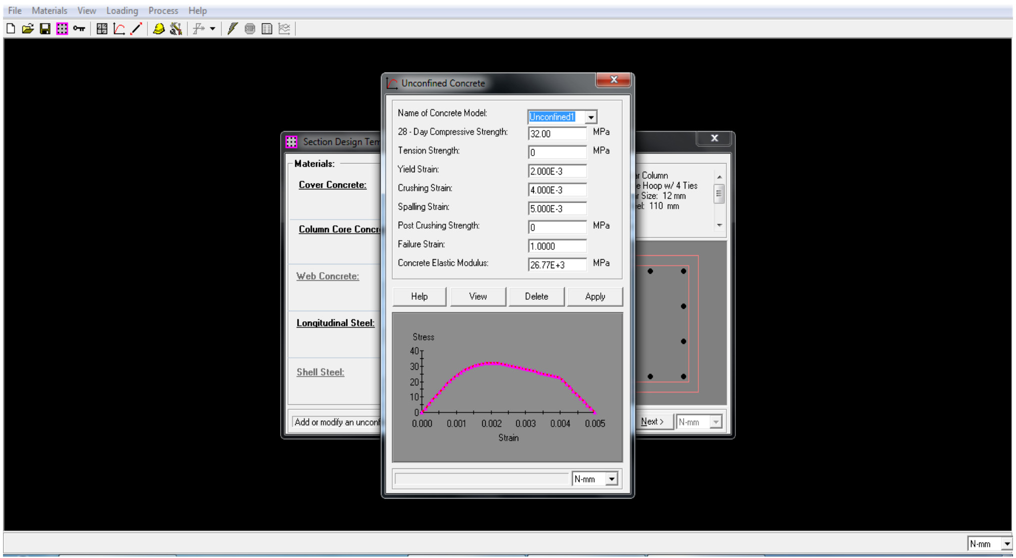


Figure 5.1 : Unconfined concrete definition (Xtract Analyses Program).

Note: To have a better comparison strains yield, crushing and spalling strains are assumed to be constant in all columns, equal to 0.002 for yielding strain, 0.004 for crushing and 0.005 for spalling (from DBYBHY 2007).failure strain remain 1.

Confinement strength is calculated from un-confinement details. According to manual of X-Tract program, the transverse reinforcement ratio are defined in two directions of the section as x-direction and y-direction. With this definition, the transverse reinforcing ratio in one of the two principal directions as x transverse steel reinforcing ratio (ρ_x), and the transverse reinforcing ratio in directions as y transverse steel reinforcing ratio (ρ_y), can be calculated by taking a 'cut' across the section in the x-direction. In this thesis study, the transverse reinforcement ratios of cross sections have been calculated by similar method with X-Tract program. This method is given below;

$$\rho_x = \frac{\sum A_v}{b_x s} \tag{5.1}$$

$$\rho_y = \frac{\sum A_v}{b_y\, s} \tag{5.2}$$

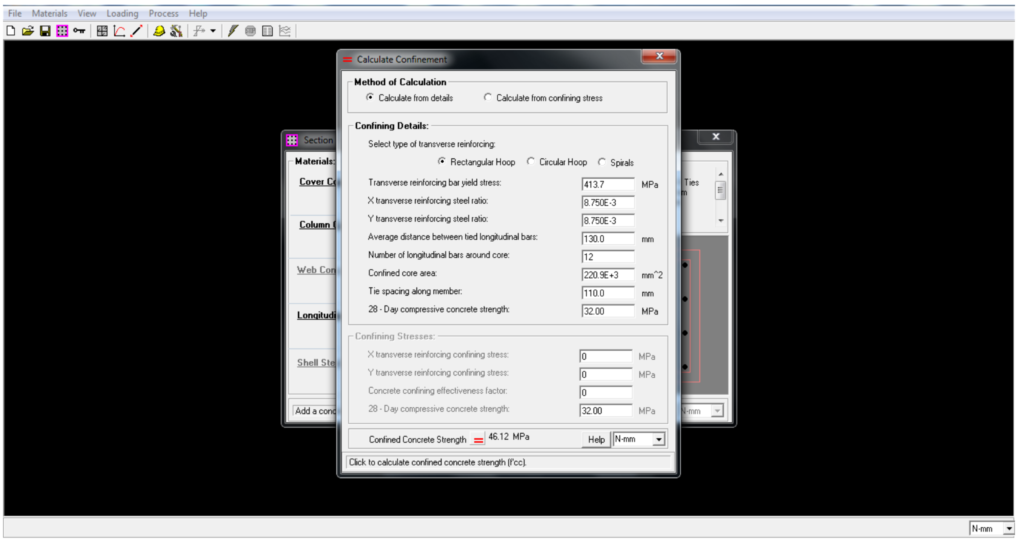


Figure 5.2 : Confined concrete design (Xtract Analaysis Program).

Steel behavior is also assumed as Mander reinforcement model. Again to make comparison under the same conditions, strains in hardening and failure points are assumed constant. Strains equal to 0.008 and 0.01, at strain hardening and failure strain respectively.

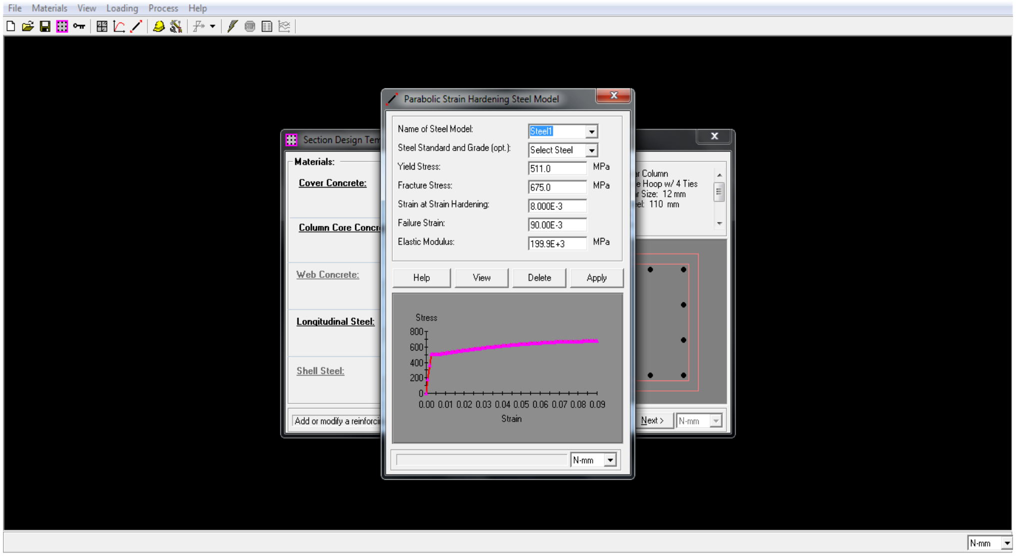


Figure 5.3 : Steel reinforcement definition (Xtract Analyses Program).

c) Application of axial load and moment direction: All experimented columns are tested in a direction, under the effect of different axial loads. The axial load amount and lateral load application direction are considered as reported for each column.

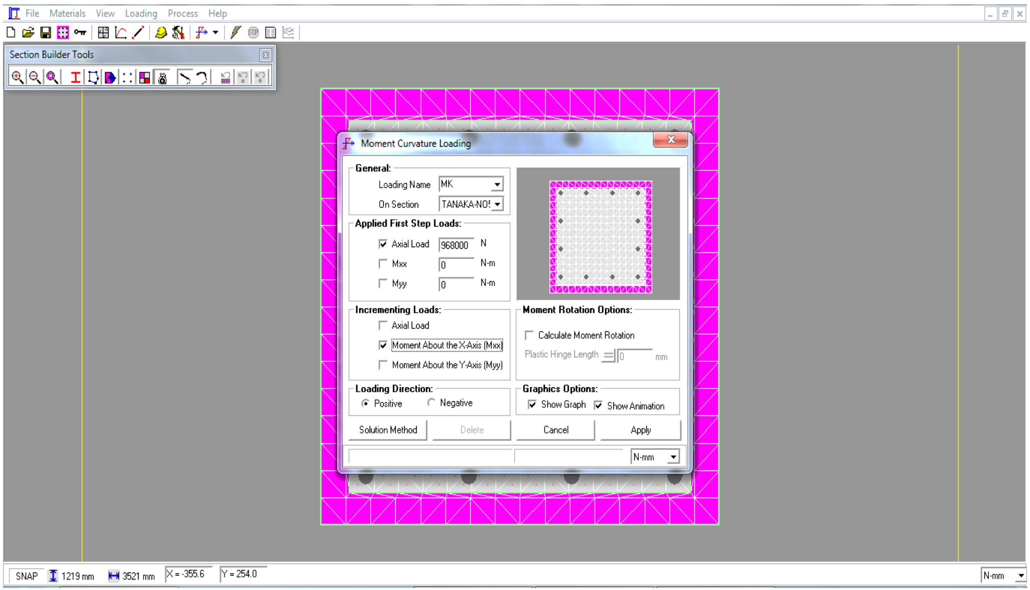


Figure 5.4 : Applied load definition (Xtract Analyses Program).

d) X-Tract section analysis helps in plotting moment-curvature relationship for a specific section. The output of the program shows as moment of the cross section which occurs the curvature on the cross section,

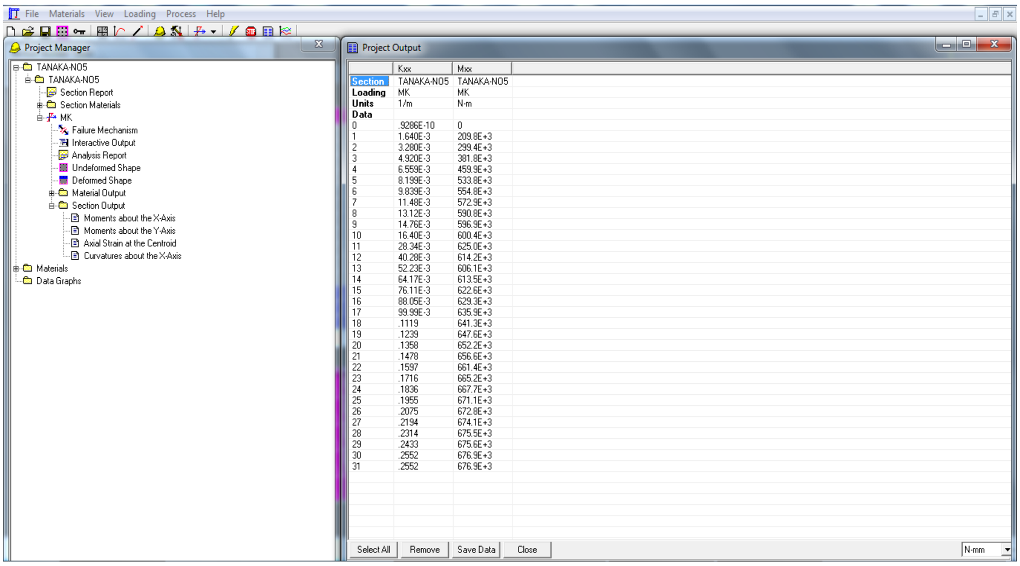


Figure 5.5 : Output moment-curvature from section analysis (Xtract Analaysis Program).

Analyzed reports and section properties of all studied sections are attached to the annex B.

5.2. Theoretical Procedure For Determination Of Nonlinear Force-Displacement Relationship

Extending cross section moment curvature relation to the entire element helps us to have a good vision of the element behavior under applied loads. In this manner, this study tries to compare the experimental force-displacement relation with converted form of moment-curvature relation, which is obtained from Xtract analysis. The converting steps are as below:

a) Plastic hinge assumption: As Turkish Seismic Code and Euro Code will be improved in near future, the codes conditions are selected as analyses default assumptions to investigate strong or weak points on performance assistance and damage limit states principles of each, as well as comparing to each other. More details about each code mentioned in chapter 3.

   According to DBYBHY 2007, plastic hinge length for rectangular columns and beams assumed as half the length of the effective depth of the cross section:

$$L_p = 0.5\ h \text{ (effective depth of section)} \qquad \textbf{(5.3)}$$

Based on Eurocode 8; for members with detailing earthquake resistance and no lapping of longitudinal bars near the section, where yielding is expected and considered in this thesis, L_p is calculated from the following expression:

$$L_p = \frac{L_v}{30} + 0.2h + 0.11\frac{d_{bl}\, f_y\,(\mathrm{MPa})}{\sqrt{f_c\,(\mathrm{MPa})}} \qquad \textbf{(5.4)}$$

New model code 2010, has proposed new formula for plastic hinge length. L_p in rectangular sections mainly depends on height and effective depth variation of elements, which is shown as below:

$$L_p = 0.2h\ (1+ \frac{1}{3}\min\ (9\ ,\ \frac{L_v}{h}) \qquad \textbf{(5.5)}$$

The notable point in this formula is the maximum range of plastic hinges that limited L_p to 0.8 of the effective depth length. The point of this formula can be in contradiction with some reported experiments that are mentioned in Literature Review.

b) Lateral force calculation: Moment is inferred as the effect of applied load in distance from the concerning point. According to this definition, the load capacity of a section can be calculate easily from moment amounts obtained from analysis. All details in this regard, described in chapter 3 (Plasticize and Plastic Hinge Definition).

$P = M/L$ (KNm/m) before plastic hinge occurs

$P = M/(L-L_p)$ from plastic hinge center to the load effected point **(5.6)**

Because of the plastic hinge occurrence, the moment capacity of undamaged section remains constant, in the case of increasing the applied load, the affected moment length is measured with a good approximation from the center of plastic hinge region, to the point of lateral load application.

c) Displacement calculation: As illustrated in Figure1.5, yield displacement can obtained from the area of curvature distribution, in linear the section multiplied by centroid of the area.

$$\Delta_y = \frac{\phi_y\, L^2}{3} \qquad \text{before plastic hinge occur}$$

$$\Delta = (\phi_u - \phi_y)\, L_p\, (L - \frac{L_p}{2}) + \frac{\phi_y\, L^2}{3} \qquad \text{after plastic hinge occur} \qquad \textbf{(5.7)}$$

As it noted before, the effect of plastic hinge occurrence on columns must considered.

d) Drift ratio: it can be defined as the ratio of top displacement of the element under the effect of external loads, by the length of the column or element. Drift ratio = Δ/L **(5.8)**

Here; $L_{v,}$ L= column's height from base to impact point of load, h=effective depth of cross section.

5.3. Performance Indicators And Corresponding Displacement Limits

According to Turkish seismic code 2007 (chapter 3), damage limits are categorized to minimum damage limit, safety damage limit, extremely damage limit and collapse state, with distinctive bounds. If the evaluated damages on elements do not reach the MN level, they are considered within the Minimum Damage region. Those stayed between MN and GV boundary limits are within Significant Damage region, those passing the GV boundary limit and stay under the values of GÇ damage limit are defined as Extremely Damage region and finally, the elements, which are going beyond GÇ, are considered as Collapsed. These boundaries are determined based on compressive and tensile strains, in element. In this manner, curvature-strain relation for concrete and steel that is obtained from Xtract section analyses program is used (Figure 5.6). The displacement is calculated from the corresponded curvature with limit boundaries strain.

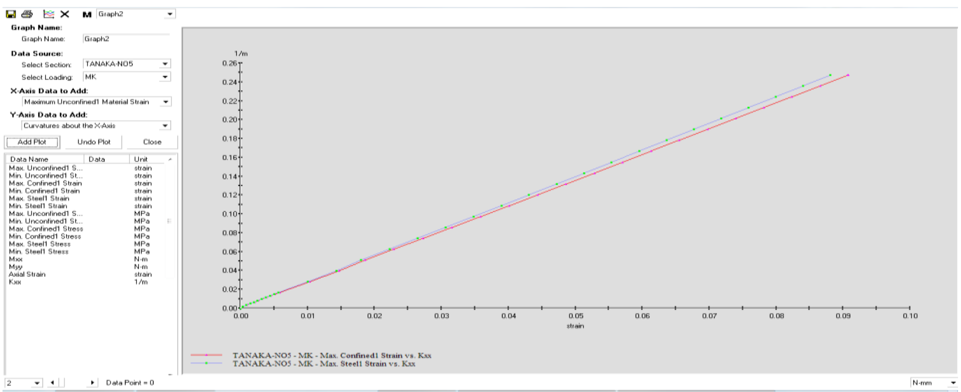


Figure 5.6 : Output stress-strain relation (Xtract Analaysis Program).

Euro Code categorizes damages to three groups. Near Collapse (NC), in which the structure heavily damaged and low residual in lateral stiffness and strength. Significant Damage (SD), the structure damaged significantly, with some residual lateral strength and stiffness, but vertical elements are still capable of sustaining vertical loads. Damage Limitation (DL) the structure lightly damaged, structural elements prevented from significant yielding and retaining their strength and stiffness properties. States of columns represented in terms of deformations and the corresponding capacity given by the chord rotation at yielding θy. From the alternative expressions for beams and columns:

$$\theta_y = \phi_y \frac{L_v + a_v z}{3} + 0.0013\left(1 + 1.5\frac{h}{L_v}\right) + 0.13\phi_y \frac{d_b f_y}{6\sqrt{f_c}}$$

The chord rotation for significant damages (SD) should be equal or less than 75% (θSD3/4θum) of the chord rotation calculated for the limit state of near collapse (NC).

For near collapse (NC) chord rotation limit should be equal or less than the value of the total chord rotation capacity at the ultimate point, ultimate chord rotation of concrete elements under cyclic loading can calculate from the following expression:

$$\theta_u = \frac{1}{\gamma_{el}}\left(\theta_y + \left(\phi_u - \phi_y\right)\right)L_p\left(1 - \frac{0.5L_p}{L_v}\right)$$

From investigations about experiments and reports, the damage limit states, which can correspond real performance of columns, assumed as below:

The minimum damaged region defined as a region that stays undamaged or without any significant damages under the applied loads. According to the experimental studies, investigations of test reports and photos, it is inferable that the damages take place slightly further than the yield deflection of an element. Subsequently, for the minimum damage region, boundary limit with a good approximation can considered as 1.2 Δ_y.

To obtain an experimental value for the yield displacement (Δy), from the resulting force-displacement diagram, a straight line from the point coordinate of 0.75*Vmax*, applied to the column, was extrapolated. The procedure is illustrated in the figure 5.7

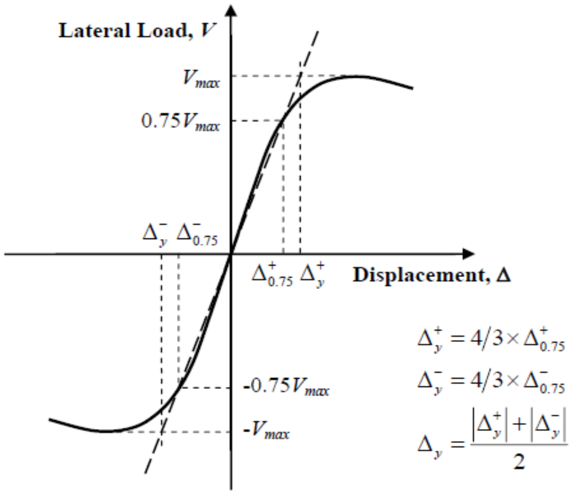


Figure 5.7 : Yield displacement assumption.

$\Delta_{\text{minimum damage}} = 1.2\ \Delta_{y\ \text{(according to definition)}}$ **(5.10)**

The average damage region refers to the region in which, significant damages have occurred with very small or no concrete strength loss. The average damage region boundaries, defined as percentage75 of collapse limit. Collapse damage region considered as a region, with significant damages in the column under applied loads. Concrete strength loss assumed for this region more than 15 to 20 percent in the descending branch of lateral load-displacement diagram.

$\Delta_{ave.} = \frac{3}{4}\Delta_{collapse}$ **(5.11)**

$\Delta_{collapse} = \Delta_{80\ \text{(displacement in \%80Vmax in descending branch)}}$ **(5.12)**

It is worth repeating that the coefficients, which have been applied in this study, are according to the observations (photos) and reports, being employed as the basis of the assessments.

5.4. Calculation of Plastic Hinge Length

Representing the plastic hinge definition (mentioned in section1), the displacement and curvature relation in plastic part, are obtained from equation;

$\Delta_u = (\phi_u - \phi_y)\ L_p\,(L - \frac{L_p}{2}) + \Delta_y$ **(5.13)**

According to this definition, it is possible to calculate the average plastic hinge length, when other parameters are estimated well. Yield curvature can simply be extracted from Xtrack analyzing program, which the details are described in section 5.2. Displacement of this curvature is computed from;

$$\Delta_y = \frac{\phi_y L^2}{3} \quad \textbf{(5.14)}$$

Since the experimental and theoretical studies are contrasting, some assumption must be considered as well.

In this study, it is tried to minimize columns that show unfavorable behaviors in both Xtract analysis and experimental tests results, which are reported by PEER Column Database. For this manner, 23 columns which show flexural failure after loss of core concrete strength about %15 to %20 in descending branch of stress-strain curve selected. Similarly, ultimate displacement is assigned as a point with %15 to %20 percent strength loss, in the experimental tests under applied loads.

6. EVALUATING PERFORMANCES OF ASSESSMENT DOCUMENTS

In this section, the output results of theoretical and experimental calculations are compared in a table. Here, the observed and assumed damage limits are compared with the calculated damage limits obtained from Turkish seismic code 2007 and Euro code 2008 separately, for each minimum, safety and collapse limits.

Columns with almost the same performance behavior between calculated force-displacement from analyses outputs, and the reported lateral load-displacement by researchers are considered. For this propose 51 cantilever columns are selected from database.

Table 6.1 shows the amount of calculated limits. Here, tried to challenge the accuracy of each code compared to analyses assumptions to figure out better and accurate limit states that provide both safety and economic conditions.

Table 6.1 : Boundaries of limit states

Specimen	T.S.C			EU.C			Exp.		
	MN	GV	GC	SD	DL	NC	LINEAR	3/4COL	COL
TANAKA-NO5	10	19	25	9.86	52.97	62.84	16	55.35	73.8
TANAKA-NO6	8	19	25.5	9.86	53.71	63.73	26.5	50.4	67.2
TANAKA-NO7	11	24	32	11.46	30.34	36.001	24	61.8	82.4
TANAKA-NO8	12	25	32	11.49	3.50	4.15	19	58.5	78
PARK-NO9	9	20	27	13.82	48.06	56.84	24	63	84
OHNO-L1	8.5	15	19	8.80	46.27	56.19	10.7	60.75	81
OHNO-L2	8.5	15	19	11.74	58.82	71.43	13	66	88
OHNO-L3	8.5	15	19	8.82	57.20	69.46	13.4	54.75	73
OZCEBE-U1	6	13	18	7.45	18.55	21.24	21	33.75	45
OZCEBE-U3	6	17	22	8.91	20.43	23.44	20	33.75	45
OZCEBE-U4	6	17	25	8.43	26.34	29.57	17.6	67.5	90
OZCEBE-U6	6	15	18	8.40	21.66	24.90	18	66	88
WEHBE-A1	14	30	34	19.31	25.50	31.24	40	85.5	114
WEHBE-A2	16	32	36	16.14	31.01	37.98	32	76.5	102
WEHBE-B1	13.5	29	38	15.21	27.63	33.87	44	120	160

Specimen	T.S.C			EU.C			Exp.		
	MN	GV	GC	SD	DL	NC	LINEAR	3/4COL	COL
WEHBE-B2	15.5	34	42	16.01	23.30	28.56	41	96	128
GRIRA-BG1	16	34	39	16.01	22.73	27.58	15	24	32
GRIRA-BG2	14	32	40	11.16	33.36	40.53	18	47.25	63
GRIRA-BG3	13.5	29	36	13.99	48.52	58.94	19	78.75	105
GRIRA-BG4	14	36	41	11.72	23.41	28.44	13	35.25	47
GRIRA-BG5	13	32	38	9.90	38.57	46.85	18	69	92
GRIRA-BG6	14	31	37	13.0	41.73	48.96	18	69	92
GRIRA-BG7	16	32	38	14.42	21.72	26.39	18	60	80
GRIRA-BG8	13.5	30	35	13.41	29.39	35.71	24.5	84	112
GRIRA-BG9	16	32	40	14.45	19.37	23.83	18	60	80
GRIRA-BG10	25	42	53	14.40	37.78	46.49	17.5	62.25	83
MO&WANG-C11	9	21	25	10.98	41.84	49.13	19	48.75	65
MO&WANG-C12	9	20	26	9.99	33.95	40.80	19	66.75	89
MO&WANG-C13	10	22	28	10.76	35.21	41.41	22.5	40.5	54
MO&WANG-C21	9	19	27	10.64	48.88	57.41	22.4	67.5	90
MO&WANG-C22	10	21	28	10.78	41.35	48.71	19	71.25	95
MO&WANG-C23	9.5	21	27	10.77	39.75	46.81	20	45	60
MO&WANG-C31	9	19	26	10.52	46.83	55.10	21	70.5	94
MO&WANG-C32	9.5	21	27.5	10.74	40.12	47.29	22.5	78	104
MO&WANG-C33	9.5	21	27	10.77	38.74	45.62	22	58.5	78
BECHTOULA-D1N30	4	8	11	4.474	17.34	19.98	5.12	10.5	14
BECHTOULA-D1N60	4	12	16	3.488	10.34	11.92	3.5	6.75	9
TAKEMURA-TEST1	6	15	18	5.94	12.15	14.23	9	60	80
TAKEMURA-TEST2	5	12	17	6.15	22.54	27.56	12.8	30.75	41
SHEIKH-RS9HT	15	32	40	11.91	27.05	33.62	18	60	80
SHEIKH-RS11HT	10	29	37	10.55	21.67	26.93	15	39.75	53

Specimen	T.S.C			EU.C			Exp.		
	MN	GV	GC	SD	DL	NC	LINEAR	3/4COL	COL
SHEIKH-RS12HT	6	15	18	11.96	17.68	21.98	12	60	80
SHEIKH-RS15HT	15	37	45	11.89	25.48	31.53	24	39.75	53
SHEIKH-RS16HT	15	35	42	11.81	20.63	25.53	19	60	80
SHEIKH-RS24HT	6	15	18	12.38	48.62	60.50	20	60	80
Ex.damage/proposed damage	0.59	0.47	0.45	0.62	0.63	0.56			

To have a better comparison, the ratio of experimentally assumed damage displacement to damage displacement proposed by codes, for each column calculated plotted in figures 6.1 to figure 6.6. Limit amounts for each column shows as points and accepted damage boundaries for any states as base line.

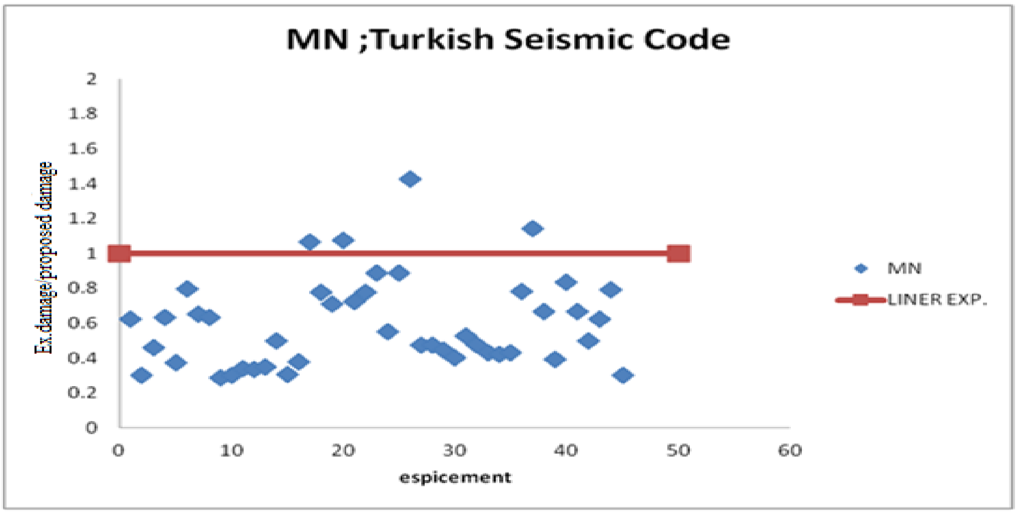


Figure 6.1 : Minimum damage evaluation (DBYBHY 2007)

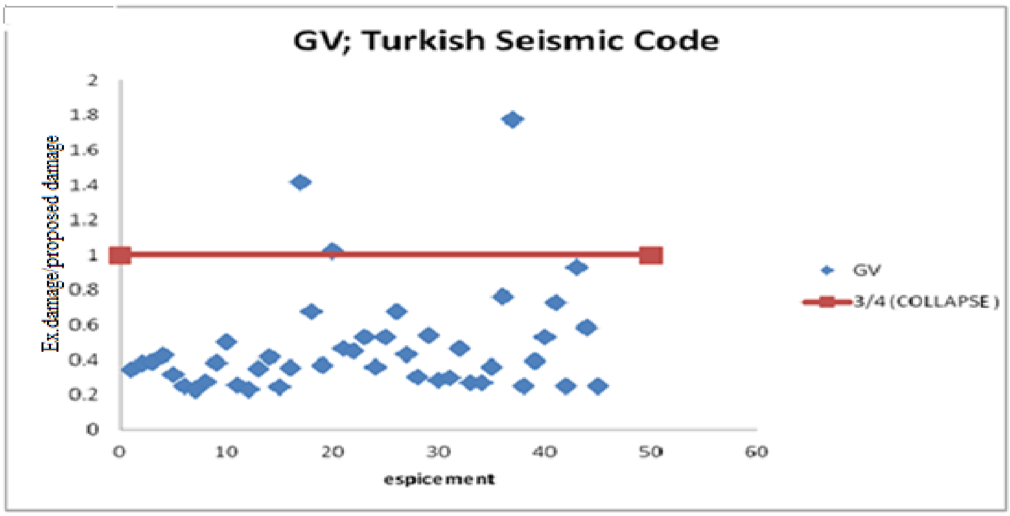


Figure 6.2 : Health safety damage evaluation (DBYBHY 2007)

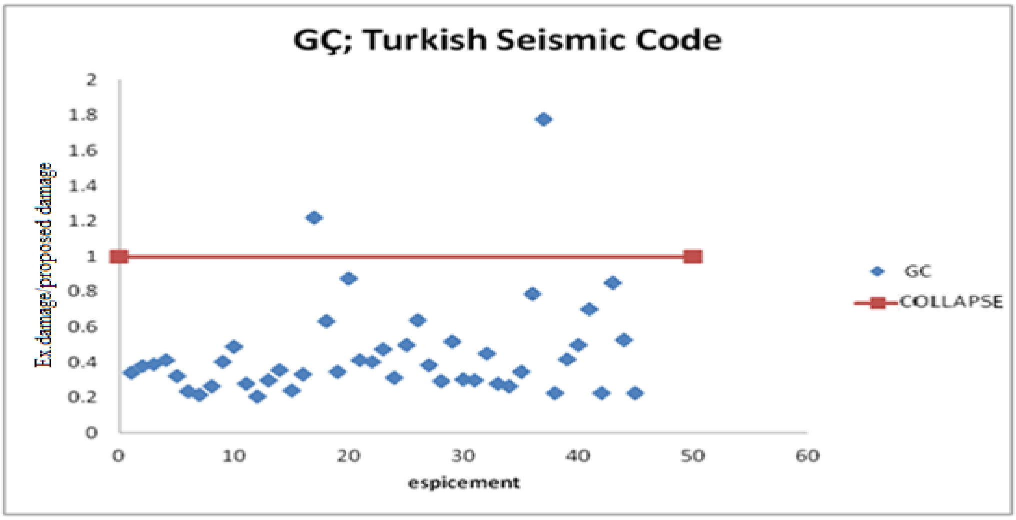


Figure 6.3 : Collapse limit damage evaluation (DBYBHY 2007)

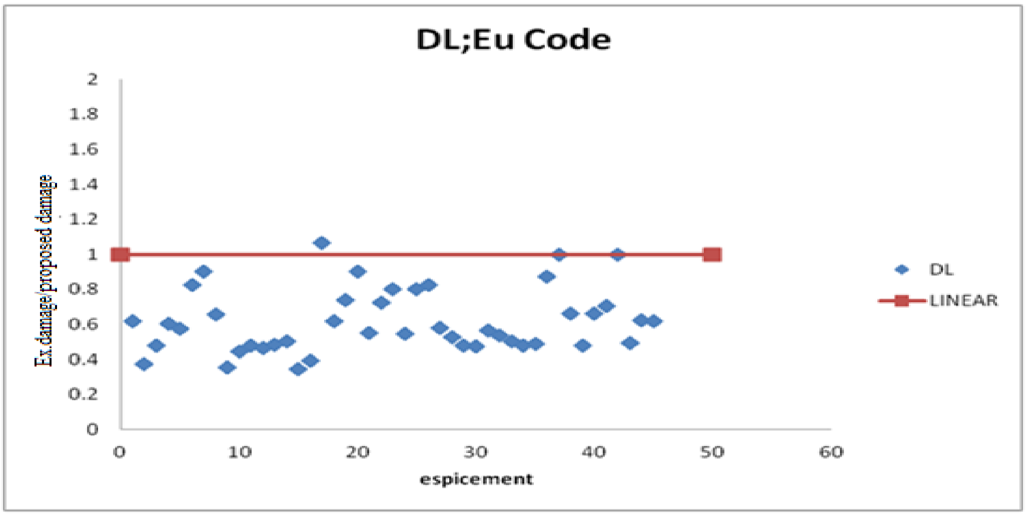


Figure 6.4 : Limited damage evaluation (Eucode 2008)

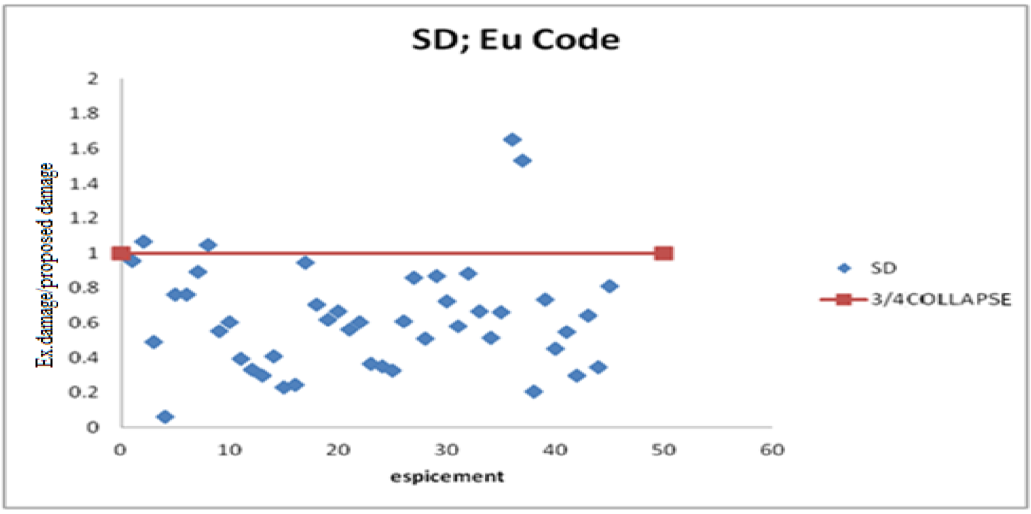


Figure 6.5 : Significant damage evaluation (Eucode 2008)

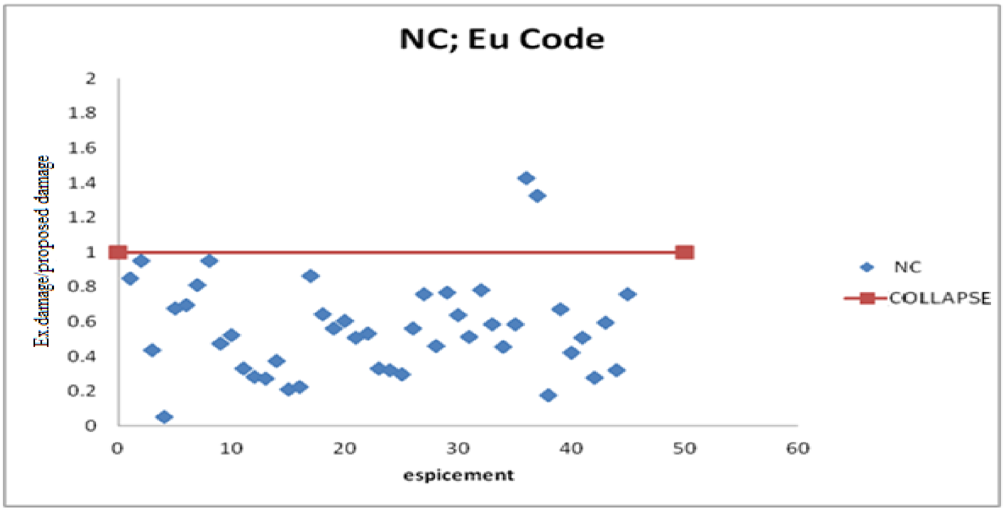


Figure 6.6 : Near collapse damage evaluation (Eucode 2008)

Each of these figures is indicating the performance assessment of columns, in different damage limit states, which proposed by the considered codes. The blue points of figures are representing the response of columns, evaluated by considering codes' damage limit boundaries and the red lines are evaluation criteria, defined to express the diversity of the columns response.

As it inferred from figures, the distance of points from base line illustrates the eccentricity of codes assumptions in limit state, from defined accepted limits. Due to the current explanation, as the points' situations lower, codes are evaluating, more conservative performance about the elements with corresponding properties.

As it is clear from the amounts, both codes are almost staying in conservative side. Investigation of the columns that exceeded the accepted damage limits, determined that the reasons of occurring higher damages prevented in the other parts of the codes, which is beyond this study subjects.

The performances of columns are directly dependent on the ratio of steel reinforcement, and steel strength. To explain the observation in details, Turkish Seismic code performance evaluation directly related to steel reinforcement ratio, which causes overestimation in the cases of containing excessive reinforcement ratios (multiple times the minimum design ratio of Turkish Seismic Code). The same overestimation happen in Eurocode evaluation, in the cross-sections, which high strength steel has been employed as transverse reinforcement. These direct relations are making the codes' evaluations conservative.

Overall it can be concluded that despite both codes are staying in conservative limits, the Turkish Seismic code present limits more safely and conservatively (about %40 to %50) than Euro code that presents more economical and realistic limits for performance states.

7. EVALUATION OF CALCULATED PLASTIC HINGE LENGTH AND A NEW SUGGESTTION FOR PLASTIC HINGE LENGTH

In this chapter, firstly by comparing the codes of TS.2007, EUROCODE 2008 and MODELCODE 2010, together with previous researches proposed formulas, it has tried to express the most realistic resulting formula, which are considered to be giving the most acceptable results. Afterwards, by investigating the influential factors, the formulas improved in a more comprehensive manner, in order to be employed in more cases that are general.

The total amount of 22 columns is selected from the database, having the following properties:

- compatibility between moment-curvature relation curves and lateral load- displacement curves,
- 15 to 20 percent decrease in the descending branch of concrete load-displacement.

In the following figures, the obtained results from codes and previous researches have shown.

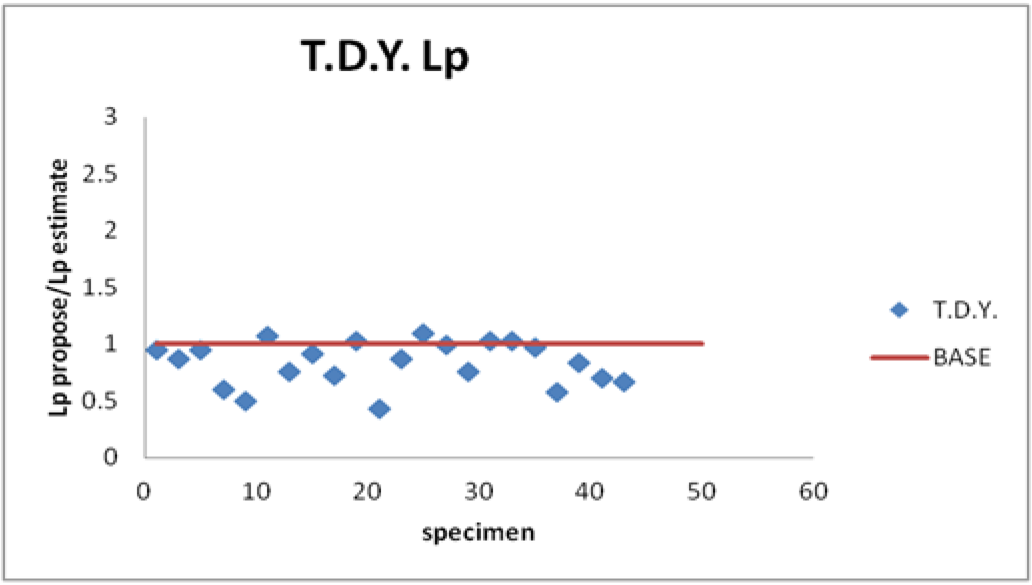


Figure 7.1 : Plastic hinge length determination, Turkish Seismic code 2007

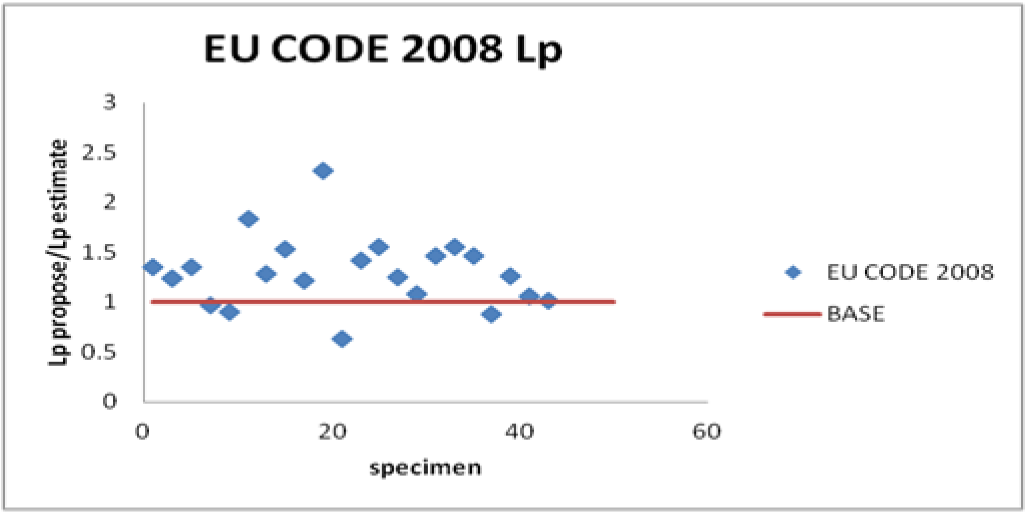


Figure 7.2 : Plastic hinge length determination, Eurocode 2008

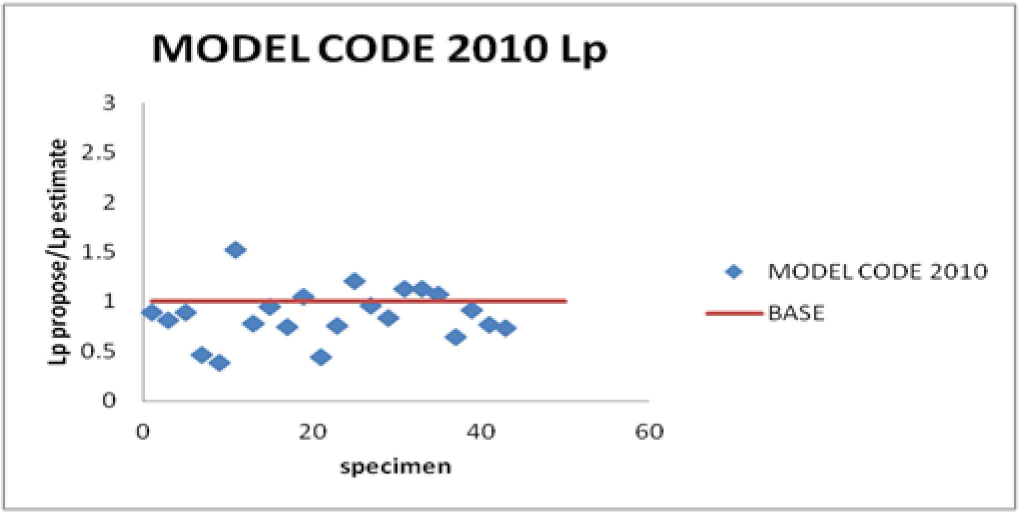


Figure 7.3 : Plastic hinge length determination, Model code 2010

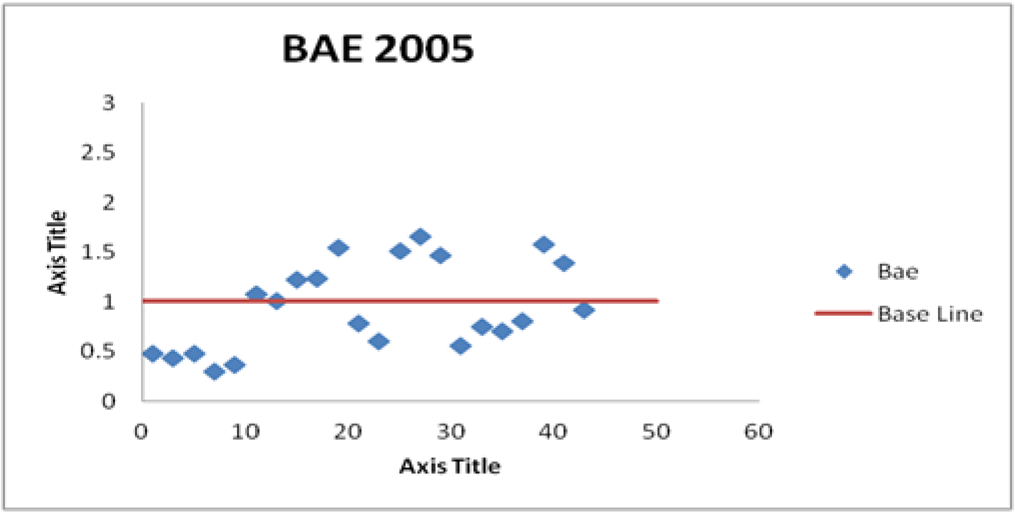


Figure 7.4 : Plastic hinge length determination, Sungjin Bae 2005

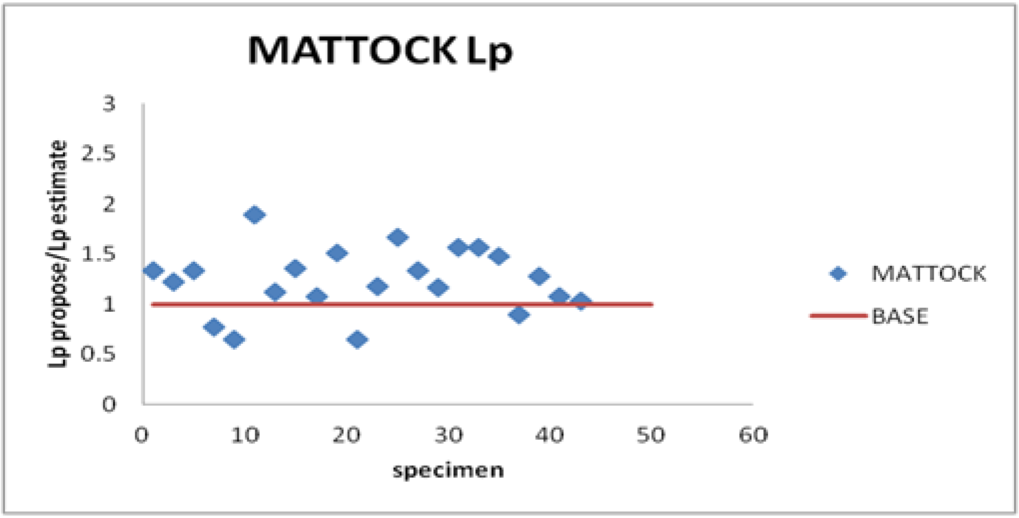


Figure 7.5 : Plastic hinge length evaluated, Mattock 1964

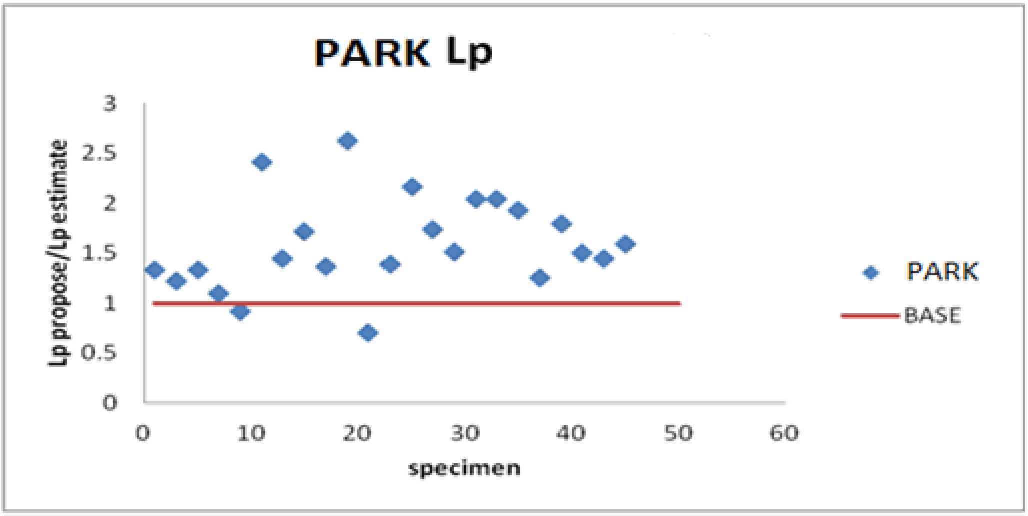


Figure 7.6 : Plastic hinge length determination, Park 1987

As aforementioned, blue points are showing the obtained results from columns, and the red line indicates the comparison criterion.

According to the figures, the proposed model of MATTOCK and PARK express the results in the most un-conservative way and Turkish Seismic Code and MODELCODE 2010 are giving out the most accurate results. According to observations in this study, despite the fact that MODELCODE has been tested by M. Fardis for a large number of RC columns with various properties, the diversity of this model' results, is larger than Turkish Seismic code proposed plastic hinge length model. Therefore, Turkish Standard gives the most acceptable results, among the most referred formulas.

As mentioned before the variation of plastic hinge length is directly related to performance behavior of sections but all the models which consider as best resulted models are generally determine the length of plastic hinge according to geometric conditions. The geometric condition dependent methods results same amounts for plastic hinge length for specific column under different conditions which make these models insufficient. As the next step of evaluation, it has been tried to investigate the variation of L_p, under the effect of the influential factors, which will be explained further. According to researches of Priestly 1987 and Sungjin Bae, 2005, the most influential factors can be indicated as:

Axial load ratio: P/P_0

Longitudinal steel reinforce ratio: ρ_s / ρ_g

Ratio of concrete strength to reinforce steel strength: f'_c / f_{yw}

In the other side shear effects assessed on the sections. The figure shows the effect of design shear force to shear capacity ratio on variation of plastic hinge length. According to the assumption all of the columns considered as flexural in failure behavior. The theoretical shear effects calculated as:

$$Vd \leq Vr \tag{7.1}$$

$$Vr = Vc + Vws \tag{7.2}$$

$$Vc = 0.8\ Vcr \tag{7.3}$$

$$Vcr = 0.65\ Fctd\ bw\ d\ (1-\gamma \frac{Nd}{Ac}) \tag{7.4}$$

$$Vws = \frac{Asw}{s} Fywd\ d \tag{7.5}$$

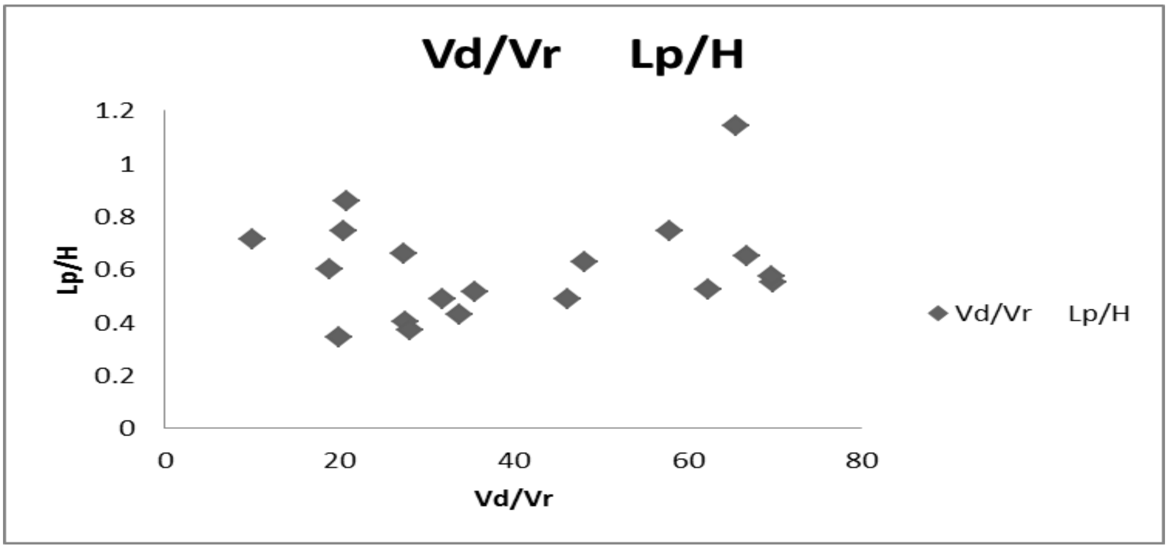


Figure 7.7: Shear effects on plastic hinge length variation.

As it showed in the figure for the columns that shear is ineffective and the ratio of design shear to the shear capacity of the section are less than 70 percent shear force seemed to be not effected on variation of plastic hinge length.

In the following figures, the factors way of affecting plastic hinge ratio (L_p/H) has been illustrated.

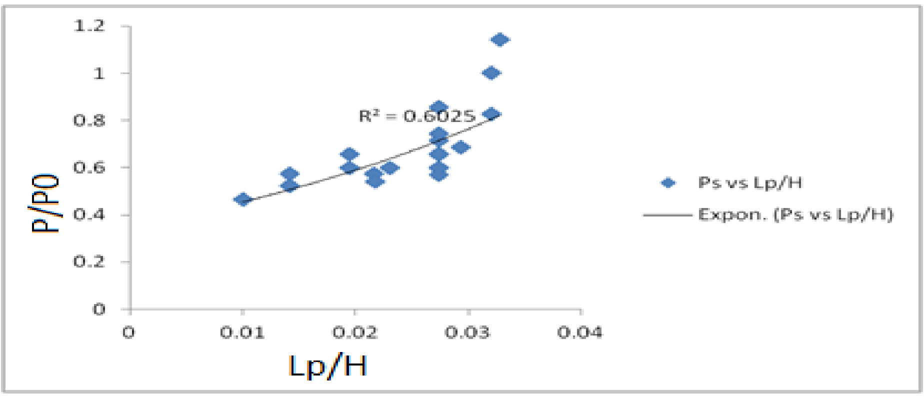


Figure 7.8: Plastic hinge variation according to axial load

As the figures indicate, by increasing the considered factors, the plastic hinge ratio (L_p/H), changes significantly. Increasing the axial load ratio and Steel reinforce ratio cause an increase in plastic hinge length, which is in accordance with S. Bae, 2005 researches. On the other hand, enhancement in ratio of concrete strength to steel reinforcement strength influences the plastic hinge length in a decreasing manner. Although these changes are significantly obvious, the factors are not behaving linearly, and the trends are illustrating parabolic.

In below figure, the plastic hinge length proposed by Bae, have been plotted on a figure against the increasing axial load.

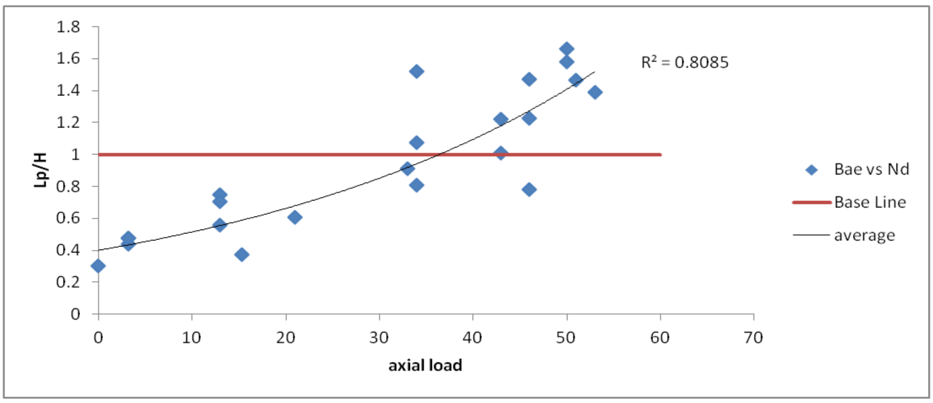


Figure 7.9 : Plastic hinge length Bae model variation according to axial load.

According to the figure, it can be seen that the proposed model of Bae, 2005, is giving reasonable results in the margin of force ratio between 35 to 45 percent. By decreasing the axial load ratio, the model responds conservatively, and by increasing this ratio, it behaves conversely. According to fig7-8 and fig7-9 comparisons, and studying the effective patters one by one in each model, it is understandable that S.Bae model consider the axial load effect on sections linearly.

Bae suggested the following formula as plastic hinge length:

$$L_p = \left[0.3\left(\frac{p}{p_0}\right) + 3\left(\frac{A_s}{A_g}\right) - 0.1\right] L + 0.25H \geq 0.25H \qquad (7.6)$$

According to the previous explanations, some points can be made about the above-suggested formula. The coefficient of 0.25 is dependent on axial load ratio variation. It can infer that by increasing the axial load ratio this coefficient decreases. According to the analyses of this study and investigations of Bae formula, in the range of considered axial load ratios (0 to 70 percent), the coefficient changes from 0.7 to 0. This new evaluation is not in contrast with Bae research results.

Moreover, the amount of -0.1 in the formula is considered as the resultant of the factors, which decrease the length of plastic hinge. The ratio of concrete strength devided by steel strength (Mechanical ratio), conducted as the most influential factor on decreasing the plastic hinge length. he following formula suggested:

$$L_p = \left[0.3\left(\frac{P}{P_0}\right) + 3\left(\frac{A_s}{A_g}\right) - 0.1\right] L + 0.7H \qquad 0\% \leq N_d < 10\%$$

$$L_p = \left[0.3\left(\frac{P}{P_0}\right) + 3\left(\frac{A_s}{A_g}\right) - 0.1\right] L + 0.5H \qquad 10\% \leq N_d < 30\%$$

$$L_p = \left[0.3\left(\frac{P}{P_0}\right) + 3\left(\frac{A_s}{A_g}\right) - 0.1\right] L + 0.25H \qquad 30\% \leq N_d < 50\%$$

$$L_p = \left[0.3\left(\frac{P}{P_0}\right) + 3\left(\frac{A_s}{A_g}\right) - 0.1\right] L \qquad 50\% \leq N_d \qquad N_d = \frac{P}{P_0}\,(\%) \qquad (7.7)$$

As it obvious from figures in proposed formula, variation of effective depth’s coefficient, changes according to the axial load. However, this coefficient decreases by the increase of axial load ratio, but these changes are not linear and the slope of changes decreases in higher load ratios.

The suggested formula evaluated through reviewing the previous analyzed columns and its credibility approved.

The following figures are indicating the evaluations.

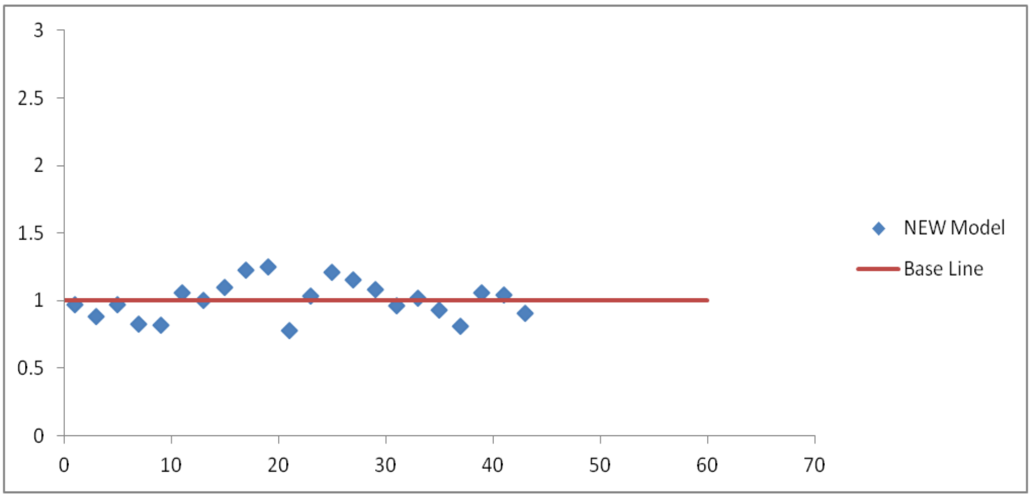


Figure 7.10 : Plastic hinge length evaluation according to correction model

According to the new proposed model, the axial load effects, varies the length of plastic hinge non-linearly and the geometry coefficient decrease by increasing the axial load. By taking the average of these formulas, it is possible to define the model written bellow as a model of plastic hinge length with a sufficient accuracy.

$$L_p = \left[0.3\left(\frac{P}{P0}\right)^2 + 2\left(\frac{A_s}{A_g}\right)\right] L + 0.3H \qquad (7.8)$$

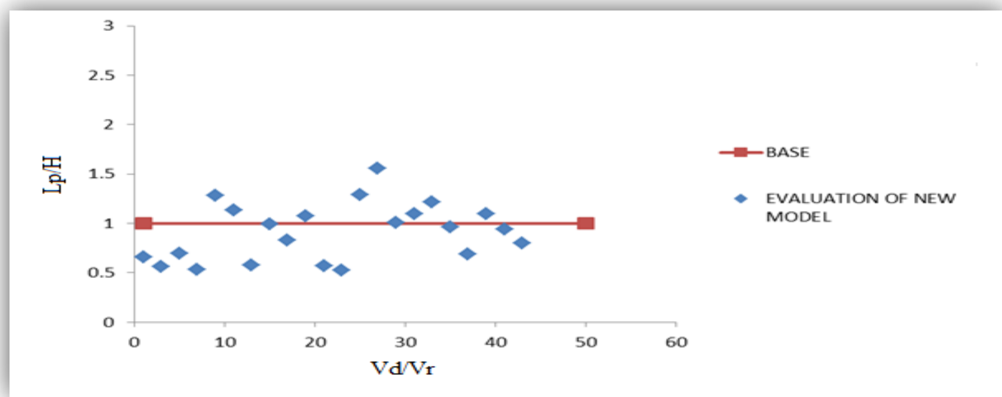


Figure 7.11 : Plastic hinge length evaluation according to proposed model

8. EVALUATION OF NEW MODEL

In this section, the new proposed model is going to evaluate on new columns with totally separate ways in order to evaluate its accuracy.

First, according to Portugal researchers Hugo Rodrigues, António Arêde, studies on behavior of concrete reinforcement columns behavior subjected to biaxial loadings; improved model formula results (which hand out plastic hinge length) is compared with damage zone length measurements that reported in the study. As mentioned before, the plastic hinge length refers to a zone under maximum moment, which cause maximum damages in the area. In the view of the points, the accuracy of new improved model compared with real damage measurements based on the same definition. Its noteworthy that in this study the columns subjected to uniaxial loading from the stronger side of the columns assessed (which results maximum Lp in a specific column).

The following table illustrates the columns details and measurements which reported in the study in addition to plastic hinge length calculated by the new improved model.

Table 8.1: Columns tested by H.Rodrigues

specimen	BxH (M)	L (M)	F_c' (MPa)	Axial load (KN)	P/P0 %	ρ_s	Average Measure(cm)
PB01-N01	0.2x0.4	1.2	48.35	375	4	0.00848	14.5
PB01-N05	0.3x0.4	1.2	21.4	300	12	0.00942	15.3
PB01-N09	0.3x0.5	1.2	24.39	300	12	0.01055	29
PB01-N10	0.5x0.3	1.2	23.3	300	8	0.01055	16
PB01-N13	0.3x0.3	1.2	21.57	210	1	0.01005	13

Table 8.2: Lp comparison

specimen	Lp T.D.Y.	Lp EU	Lp Model	Lp S.Bae	Lp New model	Lp Measured
PB01-N01	0.2	0.2	0.2	0.25	0.19	0.145
PB01-N05	0.2	0.25	0.2	0.25	0.15	0.15
PB01-N09	0.25	0.26	0.2	0.25	0.29	0.29
PB01-N10	0.15	0.26	0.22	0.25	0.155	0.16
PB01-N13	0.15	0.23	0.22	0.25	0.16	0.13

The as it can inferred from the table the accuracy of the new improved model is pleasing in comparison with average damage measurements.

9. CONCLUSIONS AND RECOMMENDATIONS

In this study, two codes of DBYBHY 2007, EUROCODE 2008 and one model of EUROCODE 2010 have been chosen to assess the performance evaluation of these codes. Total number of 57 columns was selected to conduct the investigation. Afterwards, among these 57 columns, 22 of them were selected based on the best-fitted the theoretical and experimental behavior of columns, and 15 to 20 percent decrease in descending branch of load-displacement curve. Subsequently, the plastic hinge length formulas from the most accepted previous models were selected to achieve the best satisfying model, compared with the experimental test reports.

The following points are the significant conclusions of this study:

- The limits criteria of DBYBHY 2007 and EUROCODE 8 are according to strains of structural material and chord rotation of structure member, respectively.
- Performance assessment models of the codes remain in conservative side when evaluating the columns with rectangular cross-section.
- Despite the fact that both codes are in conservative side, DBYBHY 2007 indicates results that are more conservative, which leads to less economical design approach. According to eccentricity of performance assessment outputs, shown in the following table, it can be inferred that EUROCODE 2008 is 40 to 50 percent is closer to the defined limits of damage states.

Table 9.1 : Determination of damage states

	DBYBHY 2007			EUROCODE 2008		
	MN	GV	GÇ	SD	DL	NC
Ex.damage/proposed damage	0.5	0.45	0.45	0.65	0.65	0.6

- In the evaluated results of each code there are a few points showing un-conservative behavior. By assessing those columns, it can be inferred that the performance assessment of Turkish seismic code is in direct relation with the longitudinal reinforcement ratio, which causes critical predictions in the columns with reinforcement ratio much higher than the proposed amounts. The same fact can be explained about EUROCODE 2008 when columns are designed employing high strength steel.

- Regarding the plastic hinge length, DBYBHY 2007, results in more reliable solutions, according to the plastic hinge length defined in this study.
- Despite the fact that the MODELCODE 2010 also gives out trustworthy results, but the diversity of its results are higher than Turkish Seismic code.
- The plastic hinge length was found to be a function of not only the geometry, but also in influenced by applied load, design details, and the employed materials properties.
- Axial load ratio has a direct influence on the variation of plastic hinge length. By increasing the axial load ratio in a specific column, plastic hinge length also increases. Although the influence is a direct, it is not linear one.
- In addition, there is also a direct relation between steel reinforcement ratio and the plastic hinge length. The slope of plastic hinge length variant decreases by increasing the ratio.
- By investigating the proposed formula of S. Bae, which is affected by te mentioned ratios, it is observed that the formula's results change affected by the axial load ratio. In fact, the axial load ratio has a significant effect on the geometry coefficient, which is a product of effective depth. This result is not in contrast with S. Bae studies.
- According to the studied experiments, the plastic hinge length can vary between 0.4H to H.
- According to the investigations, the following formula can be proposed as an improvement for the existing relation of plastic hinge length.

$$L_p = \left[0.3\left(\frac{P}{P0}\right)^2 + 2\left(\frac{A_s}{A_g}\right)\right]L + 0.3H \qquad (9.1)$$

To further, improve this study a few recommendations can also be suggested, which are listed as below:

- To improve the accuracy, more cases incorporating various properties must be studied.
- Although the effect of mechanical ratio (f'_c / f_y) is observed in most of the studied cases, due to lack of experimental studies about this subject, they were not considered in the results.
- Finally, it is necessary to design an experimental test, contain several real scale columns, to evaluate the effect of variation of axial load ratio, reinforcement ratio and the mechanical ratio, separately, based on a unite plastic hinge definition.

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12. APPENDICES

No.	Specimen Name	COMMENTS	f'c (MPa)	Axial Load (kN)	P-Δ	B (mm)	H (mm)	L (mm)	Config
1	Tanaka and Park 1990, No. 5		32	968	2	550	550	1650	C
2	Tanaka and Park 1990, No. 6		32	968	2	550	550	1650	C
3	Tanaka and Park 1990, No. 7		32.1	2913	2	550	550	1650	C
4	Tanaka and Park 1990, No. 8		32.1	2913	2	550	550	1650	C
5	Ohno and Nishioka 1984, L1	cover scaled from drawing	24.8	127	4	400	400	1600	C
6	Ohno and Nishioka 1984, L2	cover scaled from drawing	24.8	127	4	400	400	1600	C
7	Ohno and Nishioka 1984, L3	cover scaled from drawing	24.8	127	4	400	400	1600	C
8	Saatcioglu and Ozcebe 1989, U1		43.6	0	2	350	350	1000	C
9	Saatcioglu and Ozcebe 1989, U3		34.8	600	2	350	350	1000	C
10	Saatcioglu and Ozcebe 1989, U4		32	600	2	350	350	1000	C
11	Saatcioglu and Ozcebe 1989, U6		37.3	600	2	350	350	1000	C

Longitudinal Reinforcement Data								
No.	**Specimen Name**	**Diameter Corner (mm)**	**Diameter Interm. (mm)**	**Total # Bars**	**Clear Cover (mm)**	**Reinf. Ratio (Calc.)**	**fyl Corner (Mpa)**	**fsu long. Corner (MPa)**
1	Tanaka and Park 1990, No. 5	20	20	12	40	0.0125	511	675
2	Tanaka and Park 1990, No. 6	20	20	12	40	0.0125	511	675
3	Tanaka and Park 1990, No. 7	20	20	12	40	0.0125	511	675
4	Tanaka and Park 1990, No. 8	20	20	12	40	0.0125	511	675
5	Ohno and Nishioka 1984, L1	19	19	8	31.5	0.0142	362	-
6	Ohno and Nishioka 1984, L2	19	19	8	31.5	0.0142	362	-
7	Ohno and Nishioka 1984, L3	19	19	8	31.5	0.0142	362	-
8	Saatcioglu and Ozcebe 1989, U1	25	25	8	22.5	0.0321	430	-
9	Saatcioglu and Ozcebe 1989, U3	25	25	8	22.5	0.0321	430	-
10	Saatcioglu and Ozcebe 1989, U4	25	25	8	22.5	0.0321	438	-
11	Saatcioglu and Ozcebe 1989, U6	25	25	8	26	0.0321	437	-

Transverse Reinforcement Data									
No.	**Specimen Name**	**Type of Confinement**	**Type of Confinement**	**Nv**	**Bar Dia. (mm)**	**Spacing (mm)**	**Vol. Trans. Reinf. Ratio**	**Fyt (Mpa)**	**fsu Trans**
1	Tanaka and Park 1990, No. 5	RJ	6	4	12	110	0.017	325	429
2	Tanaka and Park 1990, No. 6	RU	8	4	12	110	0.017	325	429
3	Tanaka and Park 1990, No. 7	RJ	6	4	12	90	0.0208	325	429
4	Tanaka and Park 1990, No. 8	RU	8	4	12	90	0.0208	325	429
5	Ohno and Nishioka 1984, L1	R	2	2	9	100	0.0032	325	-
6	Ohno and Nishioka 1984, L2	R	2	2	9	100	0.0032	325	-
7	Ohno and Nishioka 1984, L3	R	2	2	9	100	0.0032	325	-
8	Saatcioglu and Ozcebe 1989, U1	R	2	2	10	150	0.0085	470	-
9	Saatcioglu and Ozcebe 1989, U3	R	2	2	10	75	0.0169	470	-
10	Saatcioglu and Ozcebe 1989, U4	R	2	2	10	50	0.0254	470	-
11	Saatcioglu and Ozcebe 1989, U6	RJ	6	6	6.4	65	0.0195	425	-

No.	Specimen Name	COMMENTS	f'c (MPa)	Axial Load (kN)	P-Δ	B (mm)	H (mm)	L (mm)	Config
12	Wehbe et al. 1998, A1	Irregular tie configuration #3 cross ties perpendicular to load.	27.2	615	1	380	610	2335	C
13	Wehbe et al. 1998, A2	Irregular tie configuration #3 cross ties perpendicular to load.	27.2	1505	1	380	610	2335	C
14	Wehbe et al. 1998, B1	Irregular tie configuration #3 cross ties perpendicular to load.	28.1	601	1	380	610	2335	C
15	Wehbe et al. 1998, B2	Irregular tie configuration #3 cross ties perpendicular to load.	28.1	1514	1	380	610	2335	C
16	Nosho et al. 1996, No. 1		40.6	1076	3	279.4	279.4	2134	C
17	Saatcioglu and Grira 1999, BG-1		34	1782	3	350	350	1645	C
18	Saatcioglu and Grira 1999, BG-2		34	1782	3	350	350	1645	C
19	Saatcioglu and Grira 1999, BG-3		34	831	3	350	350	1645	C
20	Saatcioglu and Grira 1999, BG-4		34	1923	3	350	350	1645	C
21	Saatcioglu and Grira 1999, BG-5	Transverse reinforcement welded grid	34	1923	3	350	350	1645	C
22	Saatcioglu and Grira 1999, BG-6	Transverse reinforcement welded grid	34	1900	3	350	350	1645	C

Longitudinal Reinforcement Data								
No.	**Specimen Name**	**Diameter Corner (mm)**	**Diameter Interm. (mm)**	**Total # Bars**	**Clear Cover (mm)**	**Reinf. Ratio (Calc.)**	**fyl Corner (Mpa)**	**fsu long. Corner (MPa)**
12	Wehbe et al. 1998, A1	19.1	19.1	18	28	0.0222	448	731
13	Wehbe et al. 1998, A2	19.1	19.1	18	28	0.0222	448	731
14	Wehbe et al. 1998, B1	19.1	19.1	18	25	0.0222	448	731
15	Wehbe et al. 1998, B2	19.1	19.1	18	25	0.0222	448	731
16	Nosho et al. 1996, No. 1	15.875	15.875	4	25.4	0.0101	407	659
17	Saatcioglu and Grira 1999, BG-1	19.5	19.5	8	29	0.0195	455.56	660
18	Saatcioglu and Grira 1999, BG-2	19.5	19.5	8	29	0.0195	455.56	660
19	Saatcioglu and Grira 1999, BG-3	19.5	19.5	8	29	0.0195	455.56	660
20	Saatcioglu and Grira 1999, BG-4	19.5	19.5	12	29	0.0293	455.56	660
21	Saatcioglu and Grira 1999, BG-5	19.5	19.5	12	29	0.0293	455.56	660
22	Saatcioglu and Grira 1999, BG-6	29.9	29.9	4	29	0.0229	477.78	700

Transverse Reinforcement Data									
No.	**Specimen Name**	**Type of Confin ement**	**Type of Confin ement**	**Nv**	**Bar Dia. (mm)**	**Spacin g (mm)**	**Vol. Trans. Reinf. Ratio**	**fyt (Mpa)**	**fsu Trans**
12	Wehbe et al. 1998, A1	RJ	6	4	6	110	0.0037	428	738
13	Wehbe et al. 1998, A2	RJ	6	4	6	110	0.0037	428	738
14	Wehbe et al. 1998, B1	RJ	6	4	6	83	0.0048	428	738
15	Wehbe et al. 1998, B2	RJ	6	4	6	83	0.0048	428	738
16	Nosho et al. 1996, No. 1	R	2	2	6.35	228.6		351	390
17	Saatcioglu and Grira 1999, BG-1	*RI	4	3	9.53	152	0.01	570	680
18	Saatcioglu and Grira 1999, BG-2	*RI	4	3	9.53	76	0.02	570	680
19	Saatcioglu and Grira 1999, BG-3	*RI	4	3	9.53	76	0.02	570	680
20	Saatcioglu and Grira 1999, BG-4	*RI	4	4	9.53	152	0.0133	570	680
21	Saatcioglu and Grira 1999, BG-5	*RI	4	4	9.53	76	0.0266	570	680
22	Saatcioglu and Grira 1999, BG-6	*RI	4	4	9.53	76	0.0266	570	680

No.	Specimen Name	COMMENTS	f'c (MPa)	Axial Load (kN)	P-Δ	B (mm)	H (mm)	L (mm)	Config
23	Saatcioglu and Grira 1999, BG-7	Transverse reinforcement welded grid	34	1923	3	350	350	1645	C
24	Saatcioglu and Grira 1999, BG-8	Transverse reinforcement welded grid	34	961	3	350	350	1645	C
25	Saatcioglu and Grira 1999, BG-9	Transverse reinforcement welded grid	34	1923	3	350	350	1645	C
26	Mo and Wang 2000,C1-1		24.9	450	3	400	400	1400	C
27	Mo and Wang 2000,C1-2		26.7	675	3	400	400	1400	C
28	Mo and Wang 2000,C1-3		26.1	900	3	400	400	1400	C
29	Mo and Wang 2000,C2-1		25.3	450	3	400	400	1400	C
30	Mo and Wang 2000,C2-2		27.1	675	3	400	400	1400	C
31	Mo and Wang 2000,C2-3		26.8	900	3	400	400	1400	C
32	Mo and Wang 2000,C3-1	Atypical Transverse Reinforcement, spliced longitudinal reinforcement	26.38	450	3	400	400	1400	C
33	Mo and Wang 2000,C3-2	Atypical Transverse Reinforcement, spliced longitudinal reinforcement	27.48	675	3	400	400	1400	C

Longitudinal Reinforcement Data								
No.	**Specimen Name**	**Diameter Corner (mm)**	**Diameter Interm. (mm)**	**Total # Bars**	**Clear Cover (mm)**	**Reinf. Ratio (Calc.)**	**fyl Corner (Mpa)**	**fsu long. Corner (MPa)**
23	Saatcioglu and Grira 1999, BG-7	19.5	19.5	12	29	0.0293	455.56	660
24	Saatcioglu and Grira 1999, BG-8	19.5	19.5	12	29	0.0293	455.56	660
25	Saatcioglu and Grira 1999, BG-9	16	16	20	29	0.0328	427.78	675
26	Mo and Wang 2000,C1-1	19.05	19.05	12	34	0.0214	497	592
27	Mo and Wang 2000,C1-2	19.05	19.05	12	34	0.0214	497	592
28	Mo and Wang 2000,C1-3	19.05	19.05	12	34	0.0214	497	592
29	Mo and Wang 2000,C2-1	19.05	19.05	12	34	0.0214	497	592
30	Mo and Wang 2000,C2-2	19.05	19.05	12	34	0.0214	497	592
31	Mo and Wang 2000,C2-3	19.05	19.05	12	34	0.0214	497	592
32	Mo and Wang 2000,C3-1	19.05	19.05	12	34	0.0214	497	592
33	Mo and Wang 2000,C3-2	19.05	19.05	12	34	0.0214	497	592

Transverse Reinforcement Data									
No.	**Specimen Name**	**Type of Confinement**	**Type of Confinement**	**Nv**	**Bar Dia. (mm)**	**Spacing (mm)**	**Vol. Trans. Reinf. Ratio**	**fyt (Mpa)**	**fsu Trans**
23	Saatcioglu and Grira 1999, BG-7	*RI	4	4	6.6	76	0.0126	580	720
24	Saatcioglu and Grira 1999, BG-8	*RI	4	4	6.6	76	0.0126	580	720
25	Saatcioglu and Grira 1999, BG-9	*RI	4	4	6.6	76	0.0126	580	720
26	Mo and Wang 2000,C1-1	RJ	6	4	6.35	50		459.5	576.5
27	Mo and Wang 2000,C1-2	RJ	6	4	6.35	50		459.5	576.5
28	Mo and Wang 2000,C1-3	RJ	6	4	6.35	50		459.5	576.5
29	Mo and Wang 2000,C2-1	RI	4	4	6.35	52		459.5	576.5
30	Mo and Wang 2000,C2-2	RI	4	4	6.35	52		459.5	576.5
31	Mo and Wang 2000,C2-3	RI	4	4	6.35	52		459.5	576.5
32	Mo and Wang 2000,C3-1	RI	4	4	6.35	54		459.5	576.5
33	Mo and Wang 2000,C3-2	RI	4	4	6.35	54		459.5	576.5

No.	Specimen Name	COMMENTS	f'c (MPa)	Axial Load (kN)	P-Δ	B (mm)	H (mm)	L (mm)	Config
34	Mo and Wang 2000,C3-3	Atypical Transverse Reinforcement, spliced longitudinal reinforcement	26.9	900	3	400	400	1400	C
35	Bechtoula, Kono, Arai and Watanabe, 2002, D1N30		37.6	705	2	250	250	625	C
36	Bechtoula, Kono, Arai and Watanabe, 2002, D1N60		37.6	1410	2	250	250	625	C
37	Bechtoula, Kono, Arai and Watanabe, 2002, L1D60		39.2	8000	2	600	600	1200	C
38	Bechtoula, Kono, Arai and Watanabe, 2002, L1N60		39.2	8000	2	600	600	1200	C
39	Bechtoula, Kono, Arai and Watanabe, 2002, L1N6B		32.2	6000	2	560	560	1200	C
40	Takemura and Kawashima, 1997, Test 1 (JSCE-4)	Axial Load = 0.027 Agf'c	35.9	157	1	400	400	1245	C
41	Takemura and Kawashima, 1997, Test 2 (JSCE-5)	Axial Load = 0.027 Agf'c	35.7	157	1	400	400	1245	C
42	Takemura and Kawashima, 1997, Test 3 (JSCE-6)	Axial Load = 0.027 Agf'c	34.3	157	1	400	400	1245	C
43	Bayrak & Sheikh, 2002, No. RS-9HT	Cyclic F-D data not available	71.2	2118	3	250	350	1842	C

Longitudinal Reinforcement Data								
No.	**Specimen Name**	**Diameter Corner (mm)**	**Diameter Interm. (mm)**	**Total # Bars**	**Clear Cover (mm)**	**Reinf. Ratio (Calc.)**	**fyl Corner (Mpa)**	**fsu long. Corner (MPa)**
34	Mo and Wang 2000,C3-3	19.05	19.05	12	34	0.0214	497	592
35	Bechtoula, Kono, Arai and Watanabe, 2002, D1N30	12.7	12.7	12	18.5	0.0243	461	634.3
36	Bechtoula, Kono, Arai and Watanabe, 2002, D1N60	12.7	12.7	12	18.5	0.0243	461	634.3
37	Bechtoula, Kono, Arai and Watanabe, 2002, L1D60	25.4	25.4	12	44.5	0.0169	388	588
38	Bechtoula, Kono, Arai and Watanabe, 2002, L1N60	25.4	25.4	12	44.5	0.0169	388	588
39	Bechtoula, Kono, Arai and Watanabe, 2002, L1N6B	25.4	25.4	12	24.5	0.0194	388	588
40	Takemura and Kawashima, 1997, Test 1 (JSCE-4)	12.7	12.7	20	27.5	0.0158	363	-
41	Takemura and Kawashima, 1997, Test 2 (JSCE-5)	12.7	12.7	20	27.5	0.0158	363	-
42	Takemura and Kawashima, 1997, Test 3 (JSCE-6)	12.7	12.7	20	27.5	0.0158	363	-
43	Bayrak & Sheikh, 2002, No. RS-9HT	19.54	19.54	8	20	0.0274	454	700

Transverse Reinforcement Data									
No.	**Specimen Name**	**Type of Confinement**	**Type of Confinement**	**Nv**	**Bar Dia. (mm)**	**Spacing (mm)**	**Vol. Trans. Reinf. Ratio**	**fyt (Mpa)**	**fsu Trans**
34	Mo and Wang 2000,C3-3	RI	4	4	6.35	54		459.5	576.5
35	Bechtoula, Kono, Arai and Watanabe, 2002, D1N30	RU	8	4	4	40		485	606.1
36	Bechtoula, Kono, Arai and Watanabe, 2002, D1N60	RU	8	4	4	40		485	606.1
37	Bechtoula, Kono, Arai and Watanabe, 2002, L1D60	RU	8	4	12.7	100		524	673
38	Bechtoula, Kono, Arai and Watanabe, 2002, L1N60	RU	8	4	12.7	100		524	673
39	Bechtoula, Kono, Arai and Watanabe, 2002, L1N6B	RU	8	4	12.7	100		524	673
40	Takemura and Kawashima, 1997, Test 1 (JSCE-4)	R	2	2	6	70		368	-
41	Takemura and Kawashima, 1997, Test 2 (JSCE-5)	R	2	2	6	70		368	-
42	Takemura and Kawashima, 1997, Test 3 (JSCE-6)	R	2	2	6	70		368	-
43	Bayrak & Sheikh, 2002, No. RS-9HT	RD	3	3.41	11.3	80	0.0344	542	-

No.	Specimen Name	COMMENTS	f'c (MPa)	Axial Load (kN)	P-Δ	B (mm)	H (mm)	L (mm)	Config
44	Bayrak & Sheikh, 2002, No. RS-10HT	Cyclic F-D data not available	71.1	3111	3	250	350	1842	C
45	Bayrak & Sheikh, 2002, No. RS-11HT	2 Sizes of Transverse Reinforcement	70.8	3159	3	250	350	1842	C
46	Bayrak & Sheikh, 2002, No. RS-12HT	Cyclic F-D data not available	70.9	2109	3	250	350	1842	C
47	Bayrak & Sheikh, 2002, No. RS-15HT	Cyclic F-D data not available	56.2	1770	3	250	350	1842	C
48	Bayrak & Sheikh, 2002, No. RS-16HT	Cyclic F-D data not available	56.2	1819	3	250	350	1842	C
49	Bayrak & Sheikh, 2002, No. RS-17HT	Cyclic F-D data not available	74.1	2204	3	250	350	1842	C
50	Bayrak & Sheikh, 2002, No. RS-18HT	Cyclic F-D data not available	74.1	3242	3	250	350	1842	C
51	Bayrak & Sheikh, 2002, No. RS-19HT	Cyclic F-D data not available	74.2	3441	3	250	350	1842	C
52	Bayrak & Sheikh, 2002, No. RS-20HT	Cyclic F-D data not available	74.2	2207	3	250	350	1842	C
53	Bayrak & Sheikh, 2002, No. WRS-21HT	Cyclic F-D data not available	91.3	3755	3	350	250	1842	C
54	Bayrak & Sheikh, 2002, No. WRS-22HT	Cyclic F-D data not available	91.3	2477	3	350	250	1842	C

Longitudinal Reinforcement Data								
No.	**Specimen Name**	**Diameter Corner (mm)**	**Diameter Interm. (mm)**	**Total # Bars**	**Clear Cover (mm)**	**Reinf. Ratio (Calc.)**	**fyl Corner (Mpa)**	**fsu long. Corner (MPa)**
44	Bayrak & Sheikh, 2002, No. RS-10HT	19.54	19.54	8	20	0.0274	454	700
45	Bayrak & Sheikh, 2002, No. RS-11HT	19.54	19.54	8	20	0.0274	454	700
46	Bayrak & Sheikh, 2002, No. RS-12HT	19.54	19.54	8	20	0.0274	454	700
47	Bayrak & Sheikh, 2002, No. RS-15HT	19.54	19.54	8	20	0.0274	454	700
48	Bayrak & Sheikh, 2002, No. RS-16HT	19.54	19.54	8	20	0.0274	454	700
49	Bayrak & Sheikh, 2002, No. RS-17HT	19.54	19.54	8	20	0.0274	521	692.3
50	Bayrak & Sheikh, 2002, No. RS-18HT	19.54	19.54	8	20	0.0274	521	692.3
51	Bayrak & Sheikh, 2002, No. RS-19HT	19.54	19.54	8	20	0.0274	521	692.3
52	Bayrak & Sheikh, 2002, No. RS-20HT	19.54	19.54	8	20	0.0274	521	692.3
53	Bayrak & Sheikh, 2002, No. WRS-21HT	19.54	19.54	8	20	0.0274	521	692.3
54	Bayrak & Sheikh, 2002, No. WRS-22HT	19.54	19.54	8	20	0.0274	521	692.3

Transverse Reinforcement Data									
No.	**Specimen Name**	**Type of Confinement**	**Type of Confinement**	**Nv**	**Bar Dia. (mm)**	**Spacing (mm)**	**Vol. Trans. Reinf. Ratio**	**fyt (Mpa)**	**fsu Trans**
44	Bayrak & Sheikh, 2002, No. RS-10HT	RD	3	3.41	11.3	80	0.0344	542	-
45	Bayrak & Sheikh, 2002, No. RS-11HT	RD	3	3.41	11.3 & 15.96	80	0.0543	542 & 463	-
46	Bayrak & Sheikh, 2002, No. RS-12HT	RD	3	3.41	11.3	150	0.0183	542	-
47	Bayrak & Sheikh, 2002, No. RS-15HT	RD	3	3.41	11.28 379	100	2.75	465	630.9
48	Bayrak & Sheikh, 2002, No. RS-16HT	RD	3	3.41	11.28 379	150	1.83	465	630.9
49	Bayrak & Sheikh, 2002, No. RS-17HT	RD	3	3.41	8	75	1.83	1360	1488
50	Bayrak & Sheikh, 2002, No. RS-18HT	RD	3	3.41	8	75	1.83	1360	1488
51	Bayrak & Sheikh, 2002, No. RS-19HT	RD	3	3.41	11.1	75	3.54	1402	1564
52	Bayrak & Sheikh, 2002, No. RS-20HT	RD	3	3.41	11.1	140	1.9	1402	1564
53	Bayrak & Sheikh, 2002, No. WRS-21HT	RD	3	3.41	11.28 379	70	3.92	465	630.9
54	Bayrak & Sheikh, 2002, No. WRS-22HT	RD	3	3.41	11.28 379	70	3.92	465	630.9

No.	Specimen Name	COMMENTS	f'c (MPa)	Axial Load (kN)	P-Δ	B (mm)	H (mm)	L (mm)	Config
55	Bayrak & Sheikh, 2002, No. WRS-23HT	Cyclic F-D data not available	72.2	2085	3	350	250	1842	C
56	Bayrak & Sheikh, 2002, No. WRS-24HT	Cyclic F-D data not available	72.2	3159	3	350	250	1842	C
57	Saatcioglu and Ozcebe 1989, U2	Cyclic F-D data not available	30.2	600	2	350	350	1000	C
58	Mattock & Wang, 1984, C210-S0	Cyclic F-D not available, monotonic, beam, check P-delta	23	0	2	150	350	630	C
59	Mattock & Wang, 1984, C210-S1	Cyclic F-D not available, monotonic, beam, check P-delta	23	95	2	150	350	630	C
60	Mattock & Wang, 1984, C210-S2	Cyclic F-D not available, monotonic, beam, check P-delta	23	533	2	150	350	630	C
61	Mattock & Wang, 1984, C210-S4	Cyclic F-D not available, monotonic, beam, check P-delta	23	667	2	150	350	630	C
62	Mattock & Wang, 1984, C210-S6	Cyclic F-D not available, monotonic, beam, check P-delta	23	785	2	150	350	630	C
63	Mattock & Wang, 1984, C105-S3	Cyclic F-D not available, monotonic, beam, check P-delta	21.5	324	2	150	350	315	C
64	Mattock & Wang, 1984, C105-S6	Cyclic F-D not available, monotonic, beam, check P-delta	21.5	647	2	150	350	315	C
65	Mattock & Wang, 1984, C205-S0	Cyclic F-D not available, monotonic, beam, check P-delta	21.5	0	2	150	350	630	C

Longitudinal Reinforcement Data								
No.	Specimen Name	Diameter Corner (mm)	Diameter Interm. (mm)	Total # Bars	Clear Cover (mm)	Reinf. Ratio (Calc.)	fyl Corner (Mpa)	fsu long. Corner (MPa)
55	Bayrak & Sheikh, 2002, No. WRS-23HT	19.54	19.54	8	20	0.0274	521	692.3
56	Bayrak & Sheikh, 2002, No. WRS-24HT	19.54	19.54	8	20	0.0274	521	692.3
57	Saatcioglu and Ozcebe 1989, U2	25	25	8	22.5	0.0321	453	-
58	Mattock & Wang, 1984, C210-S0	28	28	4	15	0.0469	361.4	
59	Mattock & Wang, 1984, C210-S1	28	28	4	15	0.0469	361.4	
60	Mattock & Wang, 1984, C210-S2	28	28	4	15	0.0469	361.4	
61	Mattock & Wang, 1984, C210-S4	28	28	4	15	0.0469	361.4	
62	Mattock & Wang, 1984, C210-S6	28	28	4	15	0.0469	361.4	
63	Mattock & Wang, 1984, C105-S3	28	28	4	15	0.0469	359.9	
64	Mattock & Wang, 1984, C105-S6	28	28	4	15	0.0469	359.9	
65	Mattock & Wang, 1984, C205-S0	28	28	4	15	0.0469	359.9	

Transverse Reinforcement Data									
No.	Specimen Name	Type of Confinement	Type of Confinement	Nv	Bar Dia. (mm)	Spacing (mm)	Vol. Trans. Reinf. Ratio	Fyt (Mpa)	fsu Trans
55	Bayrak & Sheikh, 2002, No. WRS-23HT	RD	3	3.41	11.3	80	0.0344	542	-
56	Bayrak & Sheikh, 2002, No. WRS-24HT	RD	3	3.41	11.3	80	0.0344	542	-
57	Saatcioglu and Ozcebe 1989, U2	R	2	2	10	150	0.0085	470	-
58	Mattock & Wang, 1984, C210-S0	R	2	2	6	80		354	
59	Mattock & Wang, 1984, C210-S1	R	2	2	6	80		354	
60	Mattock & Wang, 1984, C210-S2	R	2	2	6	80		354	
61	Mattock & Wang, 1984, C210-S4	R	2	2	6	80		354	
62	Mattock & Wang, 1984, C210-S6	R	2	2	6	80		354	
63	Mattock & Wang, 1984, C105-S3	R	2	2	6	160		354	
64	Mattock & Wang, 1984, C105-S6	R	2	2	6	160		354	
65	Mattock & Wang, 1984, C205-S0	R	2	2	6	160		354	

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