

**ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF EARTHQUAKE  
ENGINEERING AND DISASTER MANAGMENT**

**SEISMIC PERFORMANCE ASSESSMENT OF A REINFORCED CONCRETE  
BUILDING**

**M.Sc. THESIS**

**Shahin DASHTI**

**Department of Earthquake Engineering and Disaster Management**

**Earthquake Engineering Programme**

**JULY 2014**



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YÖNETİMİ ENSTİTÜSÜ**

**BETONARME BİR BİNANIN DEPREM PERFORMANSININ BELİRLENMESİ**

**YÜKSEK LİSANS TEZİ**

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**TEMMUZ 2014**



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**Date of Defense : 23 July 2014**



*To my mother and father,*

*and everyone who recognizes knowledge as a saviour*



## **FOREWORD**

Nowadays in coexistence of human's community, to supporting against natural phenomenon as earthquake's destructives, the progressing and developing on science and information technology have been extended.

Scientific achievements in basic science studying, engineering knowledge and technical skills, also executive managements in standards, codes and enforceable instructions, caused possibility for this symbiosis.

In this M.S.c. thesis project seismic performance of a concrete structure is under consideration by static and dynamic (linear and nonlinear) method based on Code 2800, IR360, and FEMA356 which are seismic designing and rehabilitation codes.

Special thanks to respectable Prof. Dr. Kadir GÜLER for his plenteous helps and guidnesses and whole time assistance, and to my dear family because of their supports and compassion everytime in my whole life.

May 2014

Shahin DASHTI



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## ABBREVIATIONS

|                  |                                                                      |
|------------------|----------------------------------------------------------------------|
| <b>FEMA</b>      | : Federal Emergency Management Agency                                |
| <b>IR360</b>     | : Iranian code of seismic assessment and rehabilitaion of structures |
| <b>CODE 2800</b> | : Iranian Code of Practice for seismic Resistant Design of Buildings |
| <b>UBC1935</b>   | : Uniform Building Code 1935                                         |
| <b>TEC</b>       | : Turkish Earthquake Code 2007                                       |
| <b>LSP</b>       | : Linear Static Procedure                                            |
| <b>LDP</b>       | : Linear Dynamic Procedure                                           |
| <b>NSP</b>       | : Nonlinear Static Procedure                                         |
| <b>NSP</b>       | : Nonlinear Dynamic Procedure                                        |
| <b>ACI 318</b>   | : American Concrete Institute                                        |
| <b>ASCE</b>      | : American Society of Civil Engineering                              |
| <b>ASTM</b>      | : American Society of Testing and Materials                          |
| <b>IO</b>        | : Immidiet Occupancy                                                 |
| <b>LS</b>        | : Life Saftey                                                        |
| <b>CP</b>        | : Collapse Prevention                                                |



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# **SEISMIC PERFORMANCE ASSESSMENT OF A REINFORCED CONCRETE BUILDING**

## **SUMMARY**

Structural design with appropriate behavior and strength against lateral load has been one of the most controversial subjects in the field of structural engineering sciences which one of the most important of them is the seismic load or earthquake load. Basically standards that suggest linear analysis as the base of design are not satisfying criteria to guarantee structure's strength under the seismic forces. The reason is the differences between these standards attitude toward the nature of seismic load and the structure's behavior under these loads and the realistic condition.

However, during last decades, due to linear standards-designed-structures mostly have shown strength facing earthquake and have not collapsed, but always there was a question about how close is the behavior of designed structure to the real.

After Long-beach of America earthquake in 1993, for the very first time the idea of "structural performance expectance" raised among earthquake engineers society. After that in next edition of UBC1935 standard of USA the earthquake-coefficient for school designing increased and selected 10 times of structure's weight at least.

The structure which has been designed due to old standards is based on using the final strength of structure's elements that the earthquake force, assuming linear behavior for the structure, will decrease by behavior- coefficient equal to  $R$ . All efforts of engineers are to remove behavior coefficient,  $R$ , and simulate and finally analyze the structure and earthquake force without any decrease in performance. Providing three seismic parameters of the structure, including: Strength, stiffness and deformation capacity in proportion to linear and non-linear behavior of the structure is the main goal of next generation standards. The goal of seismic design is based on this performance which enables designers to design structures with predictable behavior.

This master thesis project includes linear and non-linear analysis related to reinforced concrete structure built and used in Ardebil- Iran. IRAN's earthquake standard with the code of 2800 has been used in linear analysis (static and dynamic) and FEMA 356 and IR360 have been used in both static and dynamic non-linear analysis.

In first section concepts have been discussed holistically to explain the necessity of using “performance based design “ and the basic of this branch of analysis.

Through second section problems related to analyze and design due to performance that has been mentioned in IR360 and FEMA356 have been discussed.

Through third section the analyzing and linear seismic design criteria due to Code 2800 standard have been studied and following that the study case structure has been analyzed and results of linear and non-linear analysis have been mentioned.

# **BETONARME BİR BİNANIN DEPREM PERFORMANSININ BELİRLENMESİ**

## **ÖZET**

Yapıların depreme dayanıklı tasarımı, yapı ve deprem mühendisliği konusunda çalışan uzmanların sürekli karşılaştığı önemli bir sorundur. Deprem etkisi yapılar için çoğunlukla en önemli yatay yük olarak ortaya çıkabilir. Aslında doğrusal davranışı esas alan ve tasarıma temel olarak yürürlüğe koyan yönetmelikler, yapıların titreşim etkisinde yeterli performans göstermesini sağlamak için yeterli bir yaklaşım olmayabilir.

Geçtiğimiz son bir kaç on yılda doğrusal analize göre tasarlanan yapıların pek çoğu hasar görmüş olsa da, depreme karşı beklenen davranışı göstermiş, ancak yapı davranışının doğru olarak tespit edilip edilmediği sorusu hala cevap beklemektedir.

Amerika Birleşik Devletlerinde 1933 ‘Long Beach’ depreminden sonra, ilk kez yapısal performansın belirlenmesinin önemi gündeme gelmiş ve buna paralel olarak, UBC’nin 1935’lerde güncellenmesinde okul yapılarının tasarımında deprem katsayısı bina ağırlığının en az yüzde onu mertebesine kadar deprem yükseltilmiştir.

Depremlerde yapısal hasarları azaltmak, ancak dikkatli tasarım yoluyla, ya da hasar görmüş yapılardan dersler alınması ile elde edilebilir. Yapıların depremlerde hasar görmesinin sınırlı tutulması amacıyla, güvenli tarafta kalacak şekilde kurallar konulmaktadır. Bu kurallardan yapıların inşası ve güçlendirilmesinde de faydalanılabilir.

Mevcut bir bina incelendiğinde, deprem performansının artırılması ve göçmesinin önlenmesi önemlidir. Taşıyıcı sistem güvenliği yeterli olmayan binaların onarım ve güçlendirilmesi gereklidir.

Geçmiş yönetmeliklerdeki tasarım ilkesi, taşıyıcı elemanların limit dayanımını esas almakta ve bu sebeple yapıya etkiyen kuvvetler davranış katsayısı dediğimiz “R” katsayısı ile azaltılmaktadır. İnşaat mühendislerinin tüm çaba ve çalışmaları “R” davranış katsayısını kullanmadan, yapının deprem kuvvetleri karşısındaki davranışını eksiksiz bir modelleme ile analiz edebilmek yönündedir. Üç temel dinamik parametre olan, dayanım, rijitlik ve sünek davranışın tanımlanması, yeni yönetmeliklerdeki doğrusal veya doğrusal olmayan analizlerin temelini oluşturmaktadır. Yapı davranışını temel alan dinamik etkiler altında tasarım, mühendislerin yapı davranışı hakkındaki öngörüsünün önemini artırmaktadır.

Sunulan bu yüksek lisans tez çalışması, betonarme çerçeve taşıyıcı sistem sahip bir binanın doğrusal ve doğrusal olmayan analizlerini içermektedir. Çalışmada, İran’ın Erdebil kentinde 20 sene önce inşa edilmiş ve kullanılmakta olan mevcut bir betonarme yapının doğrusal analizleri (statik ve dinamik analizleri) için İran Deprem Yönetmeliği’nden (Code 2800), doğrusal olmayan analizinde ise FEMA 356 ve IR360 yönetmeliklerinden yararlanılmıştır.

İncelenen bina, 8 katlı bir konut olarak tasarlanmıştır. Binanın 2. derece deprem bölgesinde (etkin yer ivmesi katsayısı=0.30) bulunduğu ve zemin sınıfının Z2 olduğu belirlenmiştir.

Birinci bölümde kısaca performansa dayalı tasarımdan yararlanma zorunluluğundan söz edilmektedir.

İkinci bölümde, FEMA 356 belgesi ve İR 360 Yönetmeliği’nde sözü edilen, performansa dayalı analiz ve tasarım sorunlarına değinilmiş ve bölümün son kısmında Türk Deprem Yönetmeliği 2007’de performans bazlı analiz ilkelerinden bahsedilmiştir.

Üçüncü bölümde, İran Deprem Yönetmeliği’nde (Code 2800) verilen kuvvet bazlı tasarım ve analiz ilkeleri açıklanmış, binanın doğrusal dinamik analizi ve tasarımı, Code 2800’de verilen açıklamalara göre gerçekleştirilmiş, doğrusal ve doğrusal olmayan analiz sonuçları verilmiştir.

Binanın sayısal analizleri için İran Deprem Yönetmeliği uygulanarak düşey ve yatay yükler altında SAP2000 yazılımı ile, üç boyutlu taşıyıcı sistem modeli kullanılmıştır. Daha sonra bütün yatay ve düşey yükler ile kolon ve kirişler uçlarında plastik mafsallar tanımlanmıştır. Bina hedef deplasmanı (yerdeğiştirmesi) bulunduğundan sonra, bu hedef yerdeğiştirmesi taşıyıcı sistem modelinin en üst katında simetrik olarak bir noktaya göre tarif edilmiş ve bina analiz edildikten sonra, tepe yerdeğiştirmesinin ne kadar olacağı belirlenmiştir.

Öncelikle, binanın mevcut taşıyıcı sisteminin güçlendirme ve rehabilitasyona ihtiyaç duyup duymadığı incelenmiştir. Daha sonra yapılan analizlerden bina taşıyıcı sisteminin güçlendirilmesi gerektiği tespit edilmiştir. Doğrusal olmayan analizler yapıldıktan ve hedef yerdeğiştirmesi belirlendikten sonra, plastik mafsallı dönmeleri incelenerek, güçlendirmeye ve güçlendirme yöntemine karar verilmiştir.

Doğrusal olmayan analizler olarak, itme analizi ve zaman tanım alanında analiz yöntemi kullanılmış ve her iki yöntem kısaca açıklanmıştır.

Doğrusal ve doğrusal olmayan analiz sonuçlarına göre, yapının yeterli performansa sahip olmadığı tespit edilmiş ve güçlendirilmesine karar verilmiştir.

Binanın güçlendirilmesi için FEMA 547 belgesinden faydalanılmış ve gerekli açıklamalar yapılmıştır.

Binanın güçlendirilmesi için iki farklı yöntem kullanılmıştır. Birinci yöntemde binanın mevcut betonarme taşıyıcı sistemine ek olarak, bazı çerçeveler arasına iki doğrultuda çelik profillerle oluşturulan çaprazlar yerleştirilmesi, bu uygulama çaprazların az ve çok sayıda olması şeklinde, ikinci yöntemde ise, bazı çerçeveler arasındaki bölme duvarlarının iki doğrultuda betonarme perdeye dönüştürülmesi ile güçlendirilmesi ve taşıyıcı sistem rijitliğinin artırılması öngörülmüştür.

Çelik çapraz profillerle güçlendirilen binada, çaprazlar çerçeveler arasında iki farklı şekilde (çapraz sayısının iki doğrultuda az ve çok olması) konumlandırılarak itme analizi yapılmış ve performans (kapasite) eğrileri iki ayrı durum için elde edilmiş ve karşılaştırma yapılmıştır. Betonarme perdelerle güçlendirmede ise, mevcut bina taşıyıcı sistemi ve güçlendirilmiş taşıyıcı sistemlerin itme analizinden elde edilen performans (kapasite) eğrileri karşılaştırılmıştır.

Son bölümde ise, iki farklı yöntemle güçlendirilen bina taşıyıcı sistemlerinin zaman tanım alanında analizi yapılarak performansları karşılaştırılmıştır. Bu sonuçlara göre binanın mevcut taşıyıcı sistemi için en uygun güçlendirme yönteminin betonarme perde ilavesi ile yapılan güçlendirme olduğuna karar verilmiştir.

## **1. INTRODUCTION**

Seismic Rehabilitation standards points to two main methods for structural analysis: Linear and non-linear. Each one of these two methods can be executed as static or dynamic analysis. According to the conditions determined by Rehabilitation standards, the linear method is usable in limited situations and actually the purpose of Rehabilitation standards is to make designers use non-linear methods (static or dynamic) in order to structural analysis. So the main difference between this standards and previous standards such as Code 2800 (Iran structural seismic design code) is embedded in using non-linear method. The principal motive of selecting non-linear method is the replacement of philosophy of performance-based designing instead of strength-based designing that has totally transformed the attitude toward the seismic design and the way of analyzing and designing structure. In the following, we will try to have a holistic express of this new attitude

### **1.1 The Major Problem With Linear Methods**

The main problem with these methods is that: the earthquake force assumed as a real force, like which dead loads, live, wind, etc. considered. While in fact, it is not true and what we will see is that earthquake loads have contracting nature and in fact, they have displacement nature more. Therefore, this makes no noticeable difference to design a structure under earthquake force equal to 0.2 or 0.3 of weights of that structure. Neither does overestimating the force will cause increasing safety factor and nor underestimating the earthquake force will necessarily cause to destroy the structure. Some people due to habit, as soon as seeing a collapsed building remind of lack of strength and say that the building not been calculated in the situation of earthquake. Investigations show that this is not true because seismic stability or instability is not just the function of structure strength and is very depended on the ability of structure to displacement tolerance.

## 1.2 Fundamental Differences Between Seismic Load And Earthquake Equivalent Static Load

Assent of the framework of the earthquake equivalent static analysis by engineering society during the past century does not mean that static forces estimate earthquake forces precisely. Conversely, results of measurement and scientific researches show that earthquake forces are maybe very larger than equivalent static forces.

The uncertainty of earthquake forces and their drastically increases beyond the standard values causes to this question that how are existing structures able to carry forces larger than themselves and basically how can we have a precise estimate of seismic loading with this much scattering and uncertainty . We should study the momentary nature of earthquake and the ability of structure in tolerating plastic deformations in order to find the answer of this important question. It worth noticing that if the strength of a structure under an earthquake load is less, the structure will reach to the plastic zone. The amount of reaching to this zone, which has known as ductility coefficient  $\mu$ , depends on structure strength to elastic force of earthquake.

The first time that researchers faced to this reality they worked on a question that how much ductility should be exist to make earthquake forces equal to standard`s forces. They calculated non-elastic response spectrum for different ductility coefficients and compared them with design spectrum of standards which one sample of that given below.

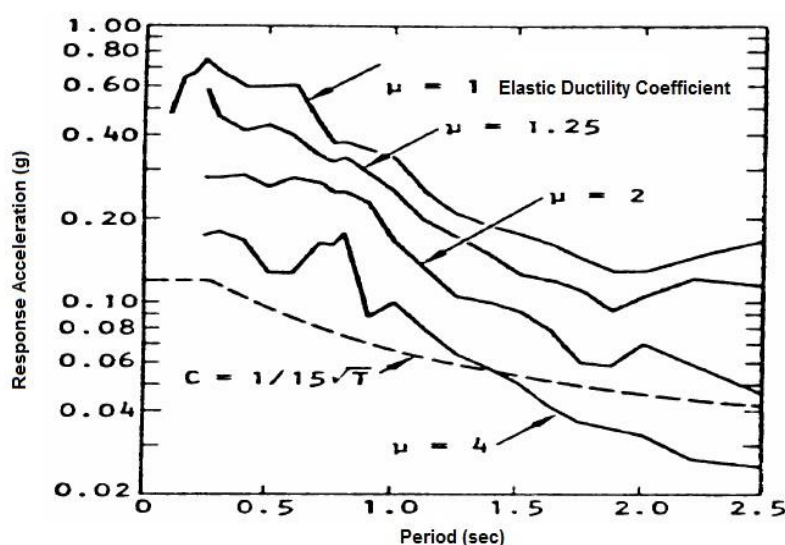


**Figure 1.1 :** The compare of non-elastic response spectra of El Centro earthquake with earthquake coefficient of UBC Code

shows that if ductility coefficient of structures with periods more than 1.5 sec is more than 4, the exerted earthquake force in El Centro earthquake is less than forces in American standard. While for the structures which are more rigid, ductility amount is not adequate and ductility coefficient should be larger, which sometimes meet 7 or more.

After that it's understood the role of ductility in decreasing earthquake force, in order to explain the standards of Codes should say that: In determining the earthquake forces, defined amount of ductility has been considered and for structures without ductility, should consider more forces so that with exerting less behavior factor, the difference between ductility of different structures will be observed.

It is observed that the magnitude of earthquake force can be expressed as a function of ductility proportion. The New Zealand Standard<sup>1</sup> for the first time expressed earthquake coefficient  $C$  as a function of ductility coefficient  $\mu$  as shown in following figure, with increasing ductility proportion, the earthquake magnification factor will be decreased.

Therefore, the safety factor will be equal to 1 and according to common definition, the structure lack safety margin. (Safety factor more than 1) and it is not clear that what was the actual criteria to determine the design forces.

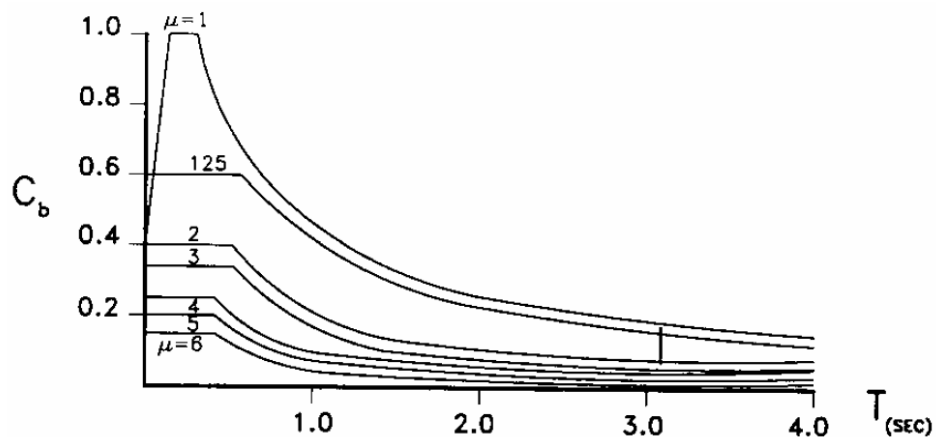


**Figure 1.2 :** The earthquake coefficient  $C$  as a function of ductility proportion  $\mu$  (standard of New Zealand's earthquake regulation)

<sup>1</sup> Code of Practice for General Structural Design and Design Loadings for Buildings, New Zealand standard NZS 4203, 1992.

During last two decades of 20th century, scientists started to seek for more logical criteria for seismic loading which has been continued to this time. Through these attempts they tried to exert both strength and ductility of a structure into loading criteria which one sample of that is mentioned in New Zealand's standard that observed that the earthquake force is considered as a function of ductility coefficient. In fact, designing according to the mentioned standard's condition explicitly guarantees that a structure's deformation under earthquake won't go beyond the permitted limits.

Since the very first of science of structure analysis, structural calculations in the last 18th, force and ductility considered as both sides of a coin and in the theory of structure's elastic behavior, each displacement is alterable to force. Therefore, during last two centuries the criteria of structural design has formed based on force and used. Even for a phenomenal like temperature that the main nature of that based on deformity, another force-like frame made (heating forces). Although this type of consideration is useful in so many cases, but in some cases this, lack a scientific reasons and engineering usability. For example earthquake engineering and seismic design of structures.

Force criteria based on following principal:

The created force in any points of the structure can go beyond the strength limits. Proportion of strength to force should be a number which is larger than 1 with a name of Safety factor.

Paulay<sup>2</sup> believes that the structures performance under earthquake load is like a chain, which has two type of links: brittle links and ductile links, which can yield. 1- Brittle links should designed in a way that will not overrun elastic zone under earthquake load. For the sake of this purpose, it is necessary for ductile links to have less strength so that yielding process will be concentrated in these links in a leaded way and brittle links will not damaged. This idea is used in **AASHTO** earthquake standards for designing bridge so that for ignoring states of brittle failure like axis and shear failure in columns and joints, and in order to lead yielding to making plastic hinge in columns, behavior factor for calculating joint's forces and axis and shear forces in columns will

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<sup>2</sup> Paulay, T, 1995, The philosophy and applications of capacity design, 2nd Intl Conf. On Seismology and Earthquake Eng., May 15-17, Tehran.

be considered equal to 1 while, for calculating bending moment of columns, the behavior factor which is equal to 5 has been suggested. So that we can guarantee that during the earthquake with bending mechanism, we will not have unwanted failure states in joints and elements.

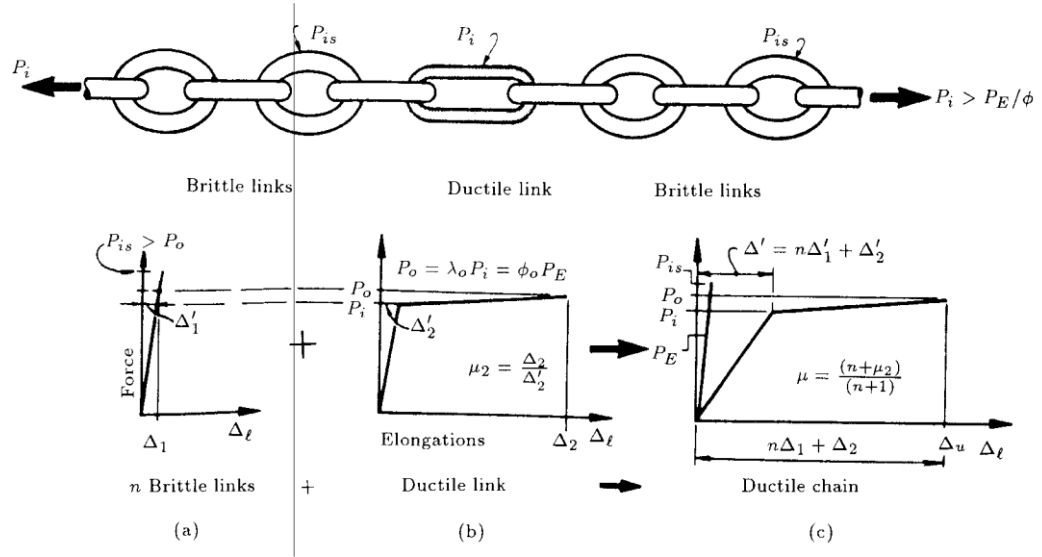


**Figure 1.3 :** Strength limits in a ductile chain

In Rehabilitation standards brittle and ductile elements are introduced with the title of elements controlled by force and displacement. Mostly this classifying is depends on force's behavior and displacement and due to Figure 1.4 it changes with proportion of  $e/g$  so that for proportions less than 2 for elements of the seismic system the will be counted as brittle element and for proportions more than that they will be counted as ductile element.

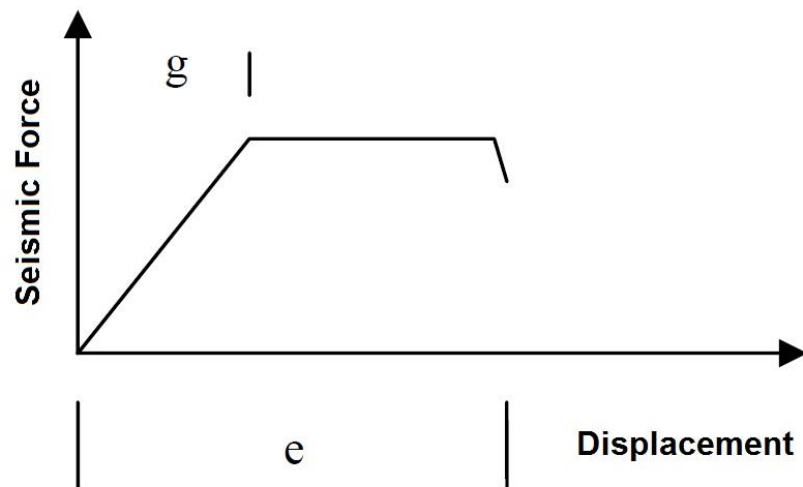


**Figure 1.4 :** Force-Displacement curve

The basis of performance based designing which in case study earthquake:

1. Force in brittle elements does not the yielding strength.
2. Displacement in ductile elements does not reach capacity of displacement.

It is observed that these conditions are just like **Paulay**'s conditions in safety chain philosophy. To create these conditions, the structure will be exposure to earthquake and will be permitted to reach the non-linear zone and yielding will occur in ductile elements. (Creating yielding mechanism in the structure)

In this state the force in brittle elements (which won't yield) will be calculated and compared with element's strength. Considering the necessary safety factor, the amount of force should not be more than element's strength. Here, in calculating strength, in order to have more reliability we consider lower limit. Moreover, the amount of deformation in ductile elements will be calculated and compared with permitted deformation. Permitted deformations for different elements provided in tables related to Rehabilitation standards.

Considering what mentioned until now, we should notice that:

1. It is meaningless to control created force with strength of ductile element, because the basic of eligible performance in extreme earthquakes is creating an appropriate yielding mechanism in structure, which will be in hand by yielding ductile elements. So, actually the created force in some of ductile elements will reach the limits of yielding strength of these elements and this not only won't causes structure to collapse, but also is a mean to ban structure's damage and collapse against earthquake.
2. In brittle failure modes, in every state, the created force should be less than strength even in ductile elements. For example in a bending element (beam) in concrete frame. Although created bending moment may reach the bending strength of the element (beam's bending yield) and due to this element controlling will not be according to strength but the bending deformation will be compared with that's permitted limit, but considering the brittle of shear failure's mode, the shear force resulted in the element should be less than shear strength of beam.

The basis of performance based design method:

Here is performance based design method in the rehabilitation standards is under consideration.

The main criteria consisted on:

- Determining the rehabilitation target based on performance level
- Estimating the level of earthquake hazard
- Defining structure model
- Determining analysis method (linear or non-linear)
- Estimating forces and non-elastic displacements and comparing with acceptance criteria.
- Rehabilitation

### **1.3 Purpose of Thesis**

The purpose of this M.Sc. Thesis is assessment seismic performance of a reinforced concrete frame structure by analyzing that due both linear and non-linear (static and dynamic) methods, which mentioned in earthquake codes and comparing results to rehabilitating the structure. The references standards, which used in this project, commonly are IRANIAN earthquake codes and FEMA356 code because of similarities between FEMA356 and IRAN's 2800 and IR360 codes.



## **2. PERFORMANCE BASED ASSESSMENT**

### **2.1 Rehabilitation Requirements According to FEMA 356 and IR 360 Standards**

#### **2.1.1 Design basis**

Seismic rehabilitation of buildings shall comply with the design criteria and procedures of mentioned standards for achieving the performance-based Rehabilitation Objectives the use of alternative performance-based criteria and procedures shall be permitted as approved by the code official.

#### **2.1.2 Seismic rehabilitation process**

Seismic rehabilitation of an existing building shall be conducted in accordance with the process outlined in following Sections. Before embarking on a rehabilitation design, seismic deficiencies shall have been identified through a prior seismic evaluation performed using an evaluation methodology considering a combination of building performance and seismic hazard.

#### **2.1.3 Review initial considerations**

The design professional shall review initial considerations with the building owner or code official to determine any restrictions that exist on the design of rehabilitation measures. Initial considerations shall include structural characteristics of the building, seismic hazards including geologic site hazards known to be present at the site, results of prior seismic evaluations, building use and occupancy requirements, historic status, economic considerations, societal issues, and local jurisdictional requirements.

#### **2.1.4 Select rehabilitation objective**

The building owner or code official shall select a seismic Rehabilitation Objective for the building as specified in Section 2.1.3. The selection of a Rehabilitation Objective

shall consist of the selection of a target Building Performance Level from a range of performance levels defined in Section 2.1.4 and on the selection of an anticipated Earthquake Hazard Level from a range of seismic hazards defined in Section 2.1.5.

#### **2.1.5 Obtain as-built information**

Available as-built information for the building shall be obtained and a site visit shall be conducted as specified in Section 2.2.1.

#### **2.1.6 Select rehabilitation method**

An applicable rehabilitation method shall be determined in accordance with relative sections.

#### **2.1.7 Perform rehabilitation design**

Rehabilitation measures shall be designed using the applicable rehabilitation method.

#### **2.1.8 Verify rehabilitation design**

The rehabilitation design shall be verified to meet the requirements of this standard through an analysis of the building, including rehabilitation measures. The analysis shall be consistent with the procedures for the applicable rehabilitation method. A separate analytical evaluation shall be performed for each combination of building performance and seismic hazard specified in the selected Rehabilitation Objective.

#### **2.1.9 Redesign unacceptable rehabilitation**

If the rehabilitation design fails to comply with the acceptance criteria for the selected Rehabilitation Objective, the rehabilitation measures shall be redesigned or an alternative rehabilitation strategy shall be implemented. This process shall be repeated until the design is in compliance with the acceptance criteria for the selected Rehabilitation Objective.

### **2.1.10 Prepare Construction documents of acceptable rehabilitation**

If the rehabilitation design meets the acceptance criteria for the selected Rehabilitation Objective, and the decision is made to proceed with the rehabilitation, construction documents shall be prepared and shall include requirements for construction quality assurance.

## **2.2 Rehabilitation Objectives**

A seismic Rehabilitation Objective shall be selected for the building, consisting of one or more rehabilitation goals. Each goal shall consist of a target Building Performance Level defined in Section 2.1.3 and an Earthquake Hazard Level defined 2.1.5. Goals shall be selected considering basic, enhanced, or limited objectives defined in Section 2.1.3.

### **2.2.1 Basic safety objective**

The Basic Safety Objective (BSO) is a Rehabilitation Objective that achieves the dual rehabilitation goals of Life Safety Building Performance Level (3-C) for the BSE-1 Earthquake Hazard Level and Collapse Prevention Building Performance Level (5-E) for the BSE-2 Earthquake Hazard Level.

### **2.2.2 Enhanced rehabilitation objectives**

Rehabilitation that provides building performance exceeding that of the BSO is termed an Enhanced Objective. Enhanced Rehabilitation Objectives shall be achieved using one or a combination of the following two methods:

1. By designing for target Building Performance Levels that exceed those of the BSO at either the BSE-1 or BSE-2 hazard levels, or both.
2. By designing for the target Building Performance Levels of the BSO using an Earthquake Hazard Level that exceeds either the BSE-1 or BSE-2 hazard levels, or both.

### **2.2.3 Limited rehabilitation objectives**

Rehabilitation that provides building performance less than that of the BSO is termed a Limited Objective. Limited Rehabilitation Objectives shall be achieved using Reduced Rehabilitation or Partial Rehabilitation specified in Section 2.1.3.3, and shall comply with the following conditions:

1. The rehabilitation measures shall not result in a reduction in the performance level of the existing building;
2. The rehabilitation measures shall not create a new structural irregularity or make an existing structural irregularity more severe;
3. The rehabilitation measures shall not result in an increase in the seismic forces to any component that is deficient in capacity to resist these forces;
4. All new or rehabilitated structural components and elements shall be detailed and connected to the existing structure in compliance with the requirements of this standard.

#### **2.2.3.1 Reduced rehabilitation objective**

Rehabilitation that addresses the entire building structural and nonstructural systems, but uses a lower seismic hazard or lower target Building Performance Level than the BSO, is termed Reduced Rehabilitation Objective. Reduced Rehabilitation shall be designed for one or more of the following objectives:

1. Life Safety Building Performance Level (3-C) for earthquake demands that are less severe (more probable) than the BSE-1;
2. Collapse Prevention Building Performance Level (5-E) for earthquake demands that are less severe (more probable) than the BSE-2;
3. Building Performance Levels 4-C, 4-D, 4-E, 5-C, 5-D, 5-E, 6-D, or 6-E for BSE-1 or less severe (more probable) earthquake demands.

#### **2.2.3.2 Partial rehabilitation objective**

Rehabilitation that addresses a portion of the building without rehabilitating the complete lateral-force resisting system is termed Partial Rehabilitation. A Partial

Rehabilitation shall be designed and constructed considering future completion of a Rehabilitation Objective intended to improve the performance of the entire structure.

### **2.3 Target Building Performance Levels**

A target Building Performance Level shall consist of a combination of a Structural Performance Level selected from the levels specified in Section 2.1.4.1 and a Nonstructural Performance Level selected from the levels specified in Section 2.1.4.2. The target Building Performance Level shall be designated alphanumerically in accordance with Section 2.1.4.3.

#### **2.3.1 Structural performance levels and ranges**

The Structural Performance Level of a building shall be selected from four discrete Structural Performance Levels and two intermediate Structural Performance Ranges defined in this section. The discrete Structural Performance Levels are Immediate Occupancy (S-1), Life Safety (S-3), Collapse Prevention (S-5), and Not Considered (S-6). Design procedures and acceptance criteria corresponding to these Structural Performance Levels shall be as specified in Section 2.4. The intermediate Structural Performance Ranges are the Damage Control Range (S-2) and the Limited Safety Range (S-4). Acceptance criteria for performance within the Damage Control Structural Performance Range shall be obtained by interpolating the acceptance criteria provided for the Immediate Occupancy and Life Safety Structural Performance Levels. Acceptance criteria for performance within the Limited Safety Structural Performance Range shall be obtained by interpolating the acceptance criteria provided for the Life Safety and Collapse Prevention Structural Performance Levels.

##### **2.3.1.1 Immediate occupancy structural performance level (S-1)**

Structural Performance Level S-1, Immediate Occupancy, shall be defined as the post-earthquake damage state that remains safe to occupy, essentially retains the pre-earthquake design strength and stiffness of the structure, and is in compliance with the acceptance criteria specified in this standard for this Structural Performance Level.

#### **2.3.1.2 Damage control structural performance range (S-2)**

Structural Performance Range S-2, Damage Control, shall be defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Immediate Occupancy Structural Performance Level (S-1).

#### **2.3.1.3 Life safety structural performance level (S-3)**

Structural Performance Level S-3, Life Safety, shall be defined as the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse in compliance with the acceptance criteria specified in this standard for this Structural Performance Level.

#### **2.3.1.4 Limited safety structural performance range (S-4)**

Structural Performance Range S-4, Limited Safety, shall be defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Collapse Prevention Structural Performance Level (S-5).

#### **2.3.1.5 Collapse prevention structural performance level (S-5)**

Structural Performance Level S-5, Collapse Prevention, shall be defined as the post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse in compliance with the acceptance criteria specified in this standard for this Structural Performance Level.

#### **2.3.1.6 Structural performance not considered (S-6)**

A building rehabilitation that does not address the performance of the structure shall be classified as Structural Performance Not Considered (S-6).

### **2.3.2 Nonstructural performance levels**

The Nonstructural Performance Level of a building shall be selected from five discrete Nonstructural Performance Levels, consisting of Operational (N-A), Immediate Occupancy (N-B), Life Safety (N-C), Hazards Reduced (N-D), and Not Considered

(N-E). Design procedures and acceptance criteria for rehabilitation of nonstructural components shall comply with the requirements.

#### **2.3.2.1 Operational nonstructural performance level (N-A)**

Nonstructural Performance Level N-A, Operational, shall be defined as the post-earthquake damage state in which the nonstructural components are able to support the pre-earthquake functions present in the building.

#### **2.3.2.2 Immediate occupancy nonstructural performance level (N-B)**

Nonstructural Performance Level N-B, Immediate Occupancy, shall be defined as the post-earthquake damage state that includes damage to nonstructural components, but building access and life safety systems—including doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems generally remain available and operable, provided that power is available.

#### **2.3.2.3 Life safety nonstructural performance level (N-C)**

Nonstructural Performance Level N-C, Life Safety, shall be defined as the post-earthquake damage state that includes damage to nonstructural components but the damage is non-life threatening.

#### **2.3.2.4 Hazards reduced nonstructural performance level (N-D)**

Nonstructural Performance Level N-D, Hazards Reduced, shall be defined as the post-earthquake damage state that includes damage to nonstructural components that could potentially create falling hazards, but high hazard nonstructural are secured and will not fall into areas of public assembly. Preservation of egress, protection of fire suppression systems, and similar life- safety issues are not addressed in this Nonstructural Performance Level.

#### **2.3.2.5 Nonstructural performance not considered (N-E)**

A building rehabilitation that does not address nonstructural components shall be classified as Nonstructural Performance Not Considered (N-E).

## **2.4 Designation of target building performance levels**

A target Building Performance Level shall be designated alphanumerically with a numeral representing the Structural Performance Level and a letter representing the Nonstructural Performance Level (e.g., 1-B, 3-C). If a Structural Performance Level other than Immediate Occupancy (S-1), Life Safety(S-3), or Collapse Prevention (S-5) is selected, the numerical designation shall represent the extent of structural damage expected in the selected Structural Performance Range.

### **2.4.1 Operational building performance level (1-A)**

To attain the Operational Building Performance Level (1-A), the structural components of the building shall meet the requirements of Section 2.1.4.1 for the Immediate Occupancy Structural Performance Level (S-1) and the nonstructural components shall meet the requirements of Section 2.1.4.2 for the Operational Nonstructural Performance Level (N-A).

### **2.4.2 Immediate occupancy building performance level (1-B)**

To attain the Immediate Occupancy Building Performance Level (1-B), the structural components of the building shall meet the requirements of Section 2.1.4.1 for the Immediate Occupancy Structural Performance Level (S-1) and the nonstructural components of the building shall meet the requirements of Section 2.1.4.2 for the Immediate Occupancy Nonstructural Performance Level (N-B).

### **2.4.3 Life safety building performance level (3-C)**

To attain the Life Safety Building Performance Level (3-C), the structural components of the building shall meet the requirements of Section 2.1.4.1 for the Life Safety Structural Performance Level (S-3) and the nonstructural components shall meet the requirements of Section 2.1.4.2 for the Life Safety Nonstructural Performance Level (N-C).

#### **2.4.4 Collapse prevention-building performance Level (5-E)**

To attain the Collapse Prevention Building Performance Level (5-E), the structural components of the building shall meet the requirements of Section 2.1.4.1 for the Collapse Prevention Structural Performance Level(S-5). Nonstructural components are not considered.

### **2.5 Seismic hazard**

Seismic hazard due to ground shaking shall be based on the location of the building with respect to causative faults, the regional and site-specific geologic characteristics, and a selected Earthquake Hazard Level. Seismic hazard due to ground shaking shall be defined as acceleration response spectra or acceleration time- histories on either a probabilistic or deterministic basis.

#### **2.5.1 General procedure for hazard due to ground shaking**

The seismic hazard due to ground shaking shall be defined for any Earthquake Hazard Level using approved spectral response acceleration contour maps of 5%-damped response spectrum ordinates for short- period (0.2 second) and long-period (1 second) response. The short-period response acceleration parameter,  $S_s$ , and the long-period response acceleration parameter,  $S_1$ , shall be determined as follows:

1. If the desired Earthquake Hazard Level corresponds with one of the mapped Earthquake Hazard Levels, obtain spectral response acceleration parameters directly from the maps.
2. If the desired Earthquake Hazard Level does not correspond with the mapped levels of hazard, then obtain the spectral response acceleration parameters from the available maps, and modify them to the desired hazard level.
3. Obtain design spectral response acceleration parameters by adjusting the mapped or modified spectral response acceleration parameters for site class effects.
4. If the desired Earthquake Hazard Level is the Basic Safety Earthquake 2 (BSE-2), obtain spectral response acceleration parameters.

5. If the desired Earthquake Hazard Level is the Basic Safety Earthquake 1 (BSE-1), obtain the spectral response acceleration parameters.
6. Using the design spectral response acceleration parameters that have been adjusted for site class effects, develop the general response spectrum.

### **2.5.2 BSE-2 Response acceleration parameters**

The design short-period response acceleration parameter,  $S_{xs}$ , and design response acceleration parameter at a one-second period,  $S_{x1}$ , for the BSE-2 Earthquake Hazard Level shall be determined using values of  $S_s$  and  $S_1$  taken from approved MCE spectral response acceleration contour maps and modified for site class.

Parameters  $S_s$  and  $S_1$  shall be obtained by interpolating between the values shown on the map for the response acceleration contour lines on either side of the site, or by using the value shown on the map for the higher contour adjacent to the site.

### **2.5.3 BSE-1 Response acceleration parameters**

The design short-period response acceleration parameter,  $S_{xs}$ , and design response acceleration parameter at a one-second period,  $S_{x1}$ , for the BSE-1 Earthquake Hazard Level shall be taken as the smaller of the following:

1. The values of  $S_s$  and  $S_1$  taken from approved 10%/50 year spectral response acceleration contour maps and modified for site.
2. Two-thirds of the values of the parameters for the BSE-2 earthquake hazard level.

### **2.5.4 Site-Specific BSE-2 response acceleration parameters**

The site-specific response acceleration parameters for the BSE-2 Earthquake Hazard Level shall be taken as the smaller of the following:

1. The values of the parameters from mean probabilistic site-specific spectra at the 2%/50 year probability of exceedance.
2. The values of the parameters from 150% of median deterministic site-specific spectra.

### **2.5.5 Site-Specific BSE-1 response acceleration parameters**

The site-specific response acceleration parameters for the BSE-1 Earthquake Hazard Level shall be taken as the smaller of the following:

1. The values of the parameters from mean probabilistic site-specific spectra at the 10%/50 year probability of exceedance.
2. Two-thirds of the values of the parameters determined for the BSE-2 Earthquake Hazard Level.

## **2.6 General Requirements**

### **2.6.1 As-Built information**

The configuration of the structural system, as well as the type, detailing, connectivity, material strength and condition of the structural elements comprising the building shall be determined in accordance with this section. Data shall also be obtained for all nonstructural elements of the building that affect the forces and deformations experienced by the structural elements during response to earthquake ground motion. This data shall be obtained from available drawings, specifications, and other documents for the existing construction, and shall be supplemented and verified by on-site investigations including nondestructive examination and testing of building materials and components.

#### **2.6.1.1 Building configuration**

The as-built building configuration information shall include data on the type and arrangement of existing structural elements and components of the gravity- and lateral-load-resisting systems, and the nonstructural components of the building that affect either the stiffness or strength of the structural elements, or the structural load path. The structural elements and components shall be identified and categorized as either primary or secondary.

The as-built building configuration shall be examined to identify the gravity and lateral load paths. These load paths shall be evaluated to identify seismic deficiencies prior to conducting a detailed rehabilitation design.

### **2.6.1.2 Component properties**

Sufficient as-built information shall be collected on component properties and their interconnection with other components to permit computation of strengths and deformation capacities. To account for any uncertainty associated with component as-built information, a knowledge factor  $K$  shall be used in the capacity evaluation.

### **2.6.1.3 Primary and secondary elements and components**

Data shall be collected to classify elements and components as primary or secondary. Data on primary and secondary elements and components shall be collected in sufficient detail to permit modeling and analysis of such components in accordance with the requirements of this standard.

## **2.6.2 Data collection requirements**

Data on the as-built condition of the structure, components, site, and adjacent buildings shall be collected in sufficient detail to perform the selected analysis procedure. The extent of data collected shall be consistent with minimum, usual, or comprehensive levels of knowledge as. The required level of knowledge shall be determined considering the selected Rehabilitation Objective and analysis procedure.

### **2.6.2.1 Minimum data collection requirements**

As a minimum, collection of as-built information shall consist of the following:

1. Information shall be obtained from design drawings with sufficient information to analyze component demands and calculate component capacities. For minimum data collection, design drawings need not be complete, but shall communicate the configuration of the gravity and lateral-force-resisting system and typical connections with sufficient detail to carry out linear analysis procedures. When design drawings are available, information shall be verified by a visual condition assessment.
2. In the absence of sufficient information from design drawings, incomplete or non-existent information shall be supplemented by a comprehensive condition assessment including destructive and nondestructive investigation.

3. in the absence of material test records and quality assurance reports, use of default material properties.
4. Information needed on adjacent buildings, shall be gained through field surveys and research of available as-built information.
5. Information on foundation and site related concerns shall be collected.

#### **2.6.2.2 Usual data collection requirements**

Usual collection of as-built information shall consist of the following:

1. Information shall be obtained from design drawings with sufficient information to analyze component demands and calculate component capacities. For usual data collection, design drawings need not be complete, but shall communicate the configuration of the gravity and lateral-force-resisting system and typical connections with sufficient detail to carry out any analysis procedure. When design drawings are available, information shall be verified by a visual condition assessment.
2. In the absence of sufficient information from design drawings, incomplete or non-existent information shall be supplemented by a comprehensive condition assessment including destructive and nondestructive investigation.
3. In the absence of material test records and quality assurance reports, material properties shall be determined by usual materials.
4. Information needed on adjacent buildings, shall be gained through field surveys and research of available as-built information.
5. Information on foundation and site related concerns shall be collected.

#### **2.6.2.3 Comprehensive data collection requirements**

Comprehensive collection of as-built information shall consist of the following:

1. Information shall be obtained from construction documents including design drawings, specifications, material test records, and quality assurance reports covering original construction and subsequent modifications to the structure. When construction documents are available, information shall be verified by a visual condition assessment.

2. If construction documents are incomplete, missing information shall be supplemented by a comprehensive condition assessment including destructive and nondestructive investigation.
3. In the absence of material test records and quality assurance reports, material properties shall be determined by comprehensive materials testing. The coefficient of variation in material test results shall be less than 20%.
4. Information needed on adjacent buildings, shall be gained through field surveys and research of available as-built information.
5. Information on foundation and site related concerns shall be collected.

### **2.6.3 Knowledge factor**

To account for uncertainty in the collection of as-built data, a knowledge factor,  $\kappa$ , shall be selected from Table 2.1 considering the selected Rehabilitation Objective, analysis procedure, and data collection process. Knowledge factors shall be applied on a component basis as determined by the level of knowledge obtained for individual components during data collection. When nonlinear procedures are used, data collection consistent with either the usual or comprehensive levels of knowledge shall be performed.

#### **2.6.3.1 Assumed values of knowledge factor**

Selected  $\kappa$  values shall be supported by data collection performed at any time prior to implementation of a rehabilitation strategy. It shall be permitted to perform an analysis in advance of the data collection process using an assumed value of  $\kappa$ , provided the value of  $\kappa$  is substantiated by data collection prior to implementation of the rehabilitation strategies. If the assumed value of  $\kappa$  is not supported by subsequent data collection, the analysis shall be revised to include a revised  $\kappa$  consistent with the data collected. If an analysis using an assumed value of  $\kappa$  will result in no required rehabilitation of the structure, the value of  $\kappa$  shall be substantiated by data collection before the analysis is finalized.

**Table 2.1 : Data Collection Requirements**

| Level of Knowledge       |                                 |                     |                         |                  |                         |                  |                          |                          |
|--------------------------|---------------------------------|---------------------|-------------------------|------------------|-------------------------|------------------|--------------------------|--------------------------|
| Data                     | Minimum                         |                     | Usual                   |                  |                         |                  | Comprehensive            |                          |
| Rehabilitation Objective | BSO or Lower                    |                     | BSO or Lower            |                  | Enhanced                |                  | Enhanced                 |                          |
| Analysis Procedure       | LSP, LDP                        |                     | All                     |                  | All                     |                  | All                      |                          |
| Testing                  | No Tests                        |                     | Usual Testing           |                  | Usual Testing           |                  | Comprehensive Testing    |                          |
| Drawings                 | Design                          | Or                  | Design                  | Or               | Design                  | Or               | Construction             | Or Equivalent            |
|                          | Drawings                        | Equivalent          | Drawings                | Equivalent       | Drawings                | Equivalent       | Documents                |                          |
| Condition Assessment     | Visual                          | Comprehensive       | Visual                  | Comprehensive    | Visual                  | Comprehensive    | Visual                   | Comprehensive            |
| Material                 | From Drawings or Default Values | From Default Values | From Drawings and Tests | From Usual Tests | From Drawings and Tests | From Usual Tests | From Documents and Tests | From Comprehensive Tests |
| Properties               |                                 |                     |                         |                  |                         |                  |                          |                          |
| Knowledge Factor (k)     | 0.75                            | 0.75                | 1                       | 1                | 0.75                    | 0.75             | 1                        | 1                        |

## 2.6.4 Analysis procedures

An analysis of the building, including rehabilitation measures, shall be conducted to determine the forces and deformations induced in components of the building by ground motion corresponding to the selected Earthquake Hazard Level, or by other seismic geologic site hazards.

### 2.6.4.1 Linear procedures

Linear procedures shall be permitted for buildings, which do not have an irregularity. For buildings that have one or more of the irregularities linear procedures shall not be used unless the earthquake demands on the building comply with the demand capacity ratio (DCR) requirements.

$$DCR = \frac{Q_{UD}}{Q_{CE}} \quad (2.1)$$

Where:

$Q_{UD}$  = Force due to the gravity and earthquake loads

$Q_{CE}$  = Expected strength of the component or element

DCRs shall be calculated for each action (such as axial force, moment, and shear) of each primary component. The critical action for the component shall be the one with the largest DCR. The DCR for this action shall be termed the critical component DCR. The largest DCR for any element at a particular story is termed the critical element DCR at that story. If an element at a particular story is composed of multiple components, then the component with the largest computed DCR shall define the critical component for the element at that story.

The applicability of linear procedures shall be determined as follows:

1. If all component DCRs  $\leq 2.0$ , then linear procedures are applicable.
2. If one or more component DCRs exceed 2.0, and no irregularities are present, then linear procedures are applicable.
3. If one or more component DCRs exceed 2.0 and any irregularity is present, then linear procedures are not applicable, and shall not be used.

#### **2.6.4.2 Nonlinear static procedure**

The NSP shall be permitted for structures in which higher mode effects are not significant, as defined in this section. To determine if higher modes are significant, a modal response spectrum analysis shall be performed for the structure using sufficient modes to capture 90% mass participation. A second response spectrum analysis shall also be performed, considering only the first mode participation. Higher mode effects shall be considered significant if the shear in any story resulting from the modal analysis considering modes required to obtain 90% mass participation exceeds 130% of the corresponding story shear considering only the first mode response. If higher mode effects are significant, the NSP shall be permitted if an LDP analysis is also performed to supplement the NSP. Buildings with significant higher mode effects must meet the acceptance criteria of this standard for both analysis procedures, except that an increase by a factor of 1.33 shall be permitted in the LDP acceptance criteria for deformation-controlled actions (m-factors) building analyzed using the NSP, with or

without a supplementary LDP evaluation, shall meet the acceptance criteria for nonlinear procedures.

#### **2.6.4.3 Nonlinear dynamic procedure**

The NDP shall be permitted for all structures. An analysis performed using the NDP shall be reviewed and approved by an independent third-party engineer with experience in seismic design and nonlinear procedures.

#### **2.6.4.4 Acceptance criteria**

The acceptability of force and deformation actions shall be evaluated for each component. Each component shall be classified as primary or secondary and each action shall be classified as deformation-controlled (ductile) or force-controlled (non-ductile). Component strengths, material properties, and component capacities shall be determined, respectively.

The rehabilitated building shall be provided with at least one continuous load path to transfer seismic forces, induced by ground motion in any direction, from the point of application to the final point of resistance. All primary and secondary components shall be capable of resisting force and deformation actions within the applicable acceptance criteria of the selected performance level.

#### **2.6.4.5 Primary and secondary elements and components**

Elements and components that affect the lateral stiffness or distribution of forces in a structure, or are loaded as a result of lateral deformation of the structure, shall be classified as primary or secondary, even if they are not part of the intended lateral-force-resisting system.

#### **2.6.4.6 Primary elements and components**

Elements and components that provide the capacity of the structure to resist collapse under seismic forces induced by ground motion in any direction shall be classified as

primary.

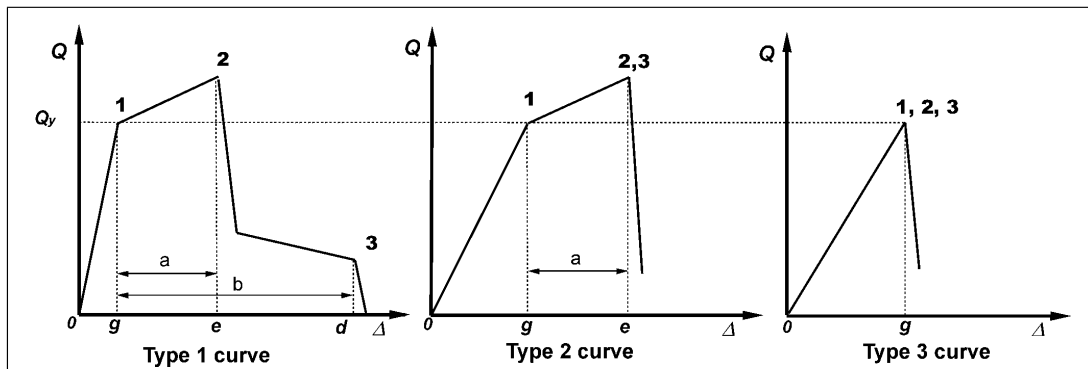


**Figure 2.1 : Component Force Versus Deformation Curves**

#### 2.6.4.7 Secondary elements and components

Other elements and components shall be classified as secondary.

#### 2.6.4.8 Deformation- and force-controlled actions

All actions shall be classified as either deformation- controlled or force-controlled using the component force versus deformation curves shown in Figure 2-1.

#### 2.6.4.9 Deformation-controlled and force- controlled behavior

The Type 1 curve depicted in Figure 2-1 is representative of ductile behavior where there is an elastic range (point 0 to point 1 on the curve) followed by a plastic range (points 1 to 3) with non-negligible residual strength and ability to support gravity loads at point 3. The plastic range includes a strain hardening or softening range (points 1 to 2) and a strength-degraded range (points 2 to 3). Primary component actions exhibiting this behavior shall be classified as deformation-controlled if the strain-hardening or strain- softening range is such that  $e > 2g$ ; otherwise, they shall be classified as force-controlled. Secondary component actions exhibiting Type 1 behavior shall be classified as deformation-controlled for any  $e/g$  ratio.

The Type 2 curve depicted in Figure 2-1 is representative of ductile behavior where there is an elastic range (point 0 to point 1 on the curve) and a plastic range (points 1 to 2) followed by loss of strength and loss of ability to support gravity loads beyond point 2. Primary and secondary component actions exhibiting this type of behavior

shall be classified as deformation-controlled if the plastic range is such that  $e > 2g$ ; otherwise, they shall be classified as force-controlled.

The Type 3 curve depicted in Figure 2-1 is representative of a brittle or non-ductile behavior where there is an elastic range (point 0 to point 1 on the curve) followed by loss of strength and loss of ability to support gravity loads beyond point 1. Primary and secondary component actions displaying Type 3 behavior shall be classified as force-controlled.

#### **2.6.4.10 Expected and lower-bound strength**

In Figure 2-1,  $Q_y$  represents the yield strength of the component. When evaluating the behavior of deformation-controlled actions, the expected strength,  $Q_{CE}$ , shall be used.  $Q_{CE}$  is defined as the statistical mean value of yield strengths,  $Q_y$ , for a population of similar components, and includes consideration of strain hardening and plastic section development. When evaluating the behavior of force-controlled actions, a lower bound estimate of the component strength,  $Q_{CL}$ , shall be used.  $Q_{CL}$  is defined as the statistical mean minus one standard deviation of the yield strengths,  $Q_y$ , for a population of similar components.

#### **2.6.5 Material properties**

Expected material properties shall be based on mean values of tested material properties. Lower bound material properties shall be based on mean values of tested material properties minus one standard deviation ( $\sigma$ ). Nominal material properties, or properties specified in construction documents, shall be taken as lower bound material properties. Corresponding expected material properties shall be calculated by multiplying lower-bound to translate from lower bound to expected values.

##### **2.6.5.1 Nonlinear procedures**

If nonlinear procedures are used, component capacities for deformation-controlled actions shall be taken as permissible inelastic deformation limits, and component capacities for force-controlled actions shall be taken as lower-bound strengths,  $Q_{CL}$ .

### **2.6.5.2 Linear procedures**

If linear procedures are used, capacities for deformation-controlled actions shall be defined as the product of m-factors and expected strengths,  $Q_{CE}$ . Capacities for force-controlled actions shall be defined as lower-bound strengths,  $Q_{CL}$ .

### **2.6.6 Rehabilitation strategies**

A Rehabilitation Objective shall be achieved by implementing rehabilitation measures based on a strategy of addressing deficiencies identified by a prior seismic evaluation. Each rehabilitation measure shall be evaluated in conjunction with other rehabilitation measures, and the existing structure as a whole, to assure that the complete rehabilitation scheme achieves the target Building Performance Level for the selected Earthquake Hazard Level. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated structure. The compatibility of new and existing components and elements shall be checked at displacements consistent with the demands produced by the selected Earthquake Hazard Level and geologic site hazards present at the site.

#### **2.6.6.1 Local modification of components**

Local modification of deficient components shall be permitted as an applicable rehabilitation strategy. Some existing buildings have substantial strength and stiffness, but some of their components may not have adequate strength, toughness, or deformation capacity to satisfy the Rehabilitation Objectives. An appropriate strategy for such structures may be to perform local modifications of components that are inadequate while retaining the basic configuration of the building's lateral-force-resisting system. Local modifications that can be considered include improvement of component connectivity, component strength, and/or component deformation capacity. This strategy tends to be the most economical rehabilitation approach when only a few of the building's components are inadequate.

Local strengthening allows one or more under-strength elements or connections to resist the strength demands predicted by the analysis without affecting the overall response of the structure. This could include measures such as cover plating steel beams or columns, or adding plywood sheathing to an existing timber diaphragm. Such

measures increase the strength of the element or component and allow it to resist more earthquake-induced force before the onset of damage.

Local corrective measures that improve the deformation capacity or ductility of a component allow it to resist large deformation levels with reduced amounts of damage, without necessarily increasing the strength. One such measure is placement of a confinement jacket around a reinforced concrete column to improve its ability to deform without spalling or degrading reinforcement splices. Another measure is reduction of the cross-section of selected structural components to increase their flexibility and response displacement capacity.

#### **2.6.6.2 Removal or lessening of existing irregularities**

Removal or lessening of existing irregularities shall be permitted as an applicable rehabilitation strategy. Removal or lessening of existing irregularities may be an effective rehabilitation strategy if a seismic evaluation shows that the irregularities result in the inability of the building to meet the selected Structural Performance Level. The results of analysis should be reviewed to detect existing irregularities. Stiffness, mass, and strength irregularities may be detected either by reviewing the results of a linear analysis, examining the distribution of structural displacements and DCRs, or reviewing the results of a nonlinear analysis by examining the distribution of structural displacements and inelastic deformation demands. If the distribution of values of structural displacements, DCRs, or inelastic deformation demands predicted by the analysis is nonuniform with disproportionately high values within one story relative to the adjacent story, or at one side of a building relative to the other, then an irregularity exists.

Such irregularities are often, but not always, caused by the presence of a discontinuity in the structure, as for example, termination of a perimeter shear wall above the first story. Simple removal of the irregularity may be sufficient to reduce demands predicted by the analysis to acceptable levels. However, removal of discontinuities may be inappropriate in the case of historic buildings, and the effect of such alterations on important historic features should be considered carefully.

Effective corrective measures for removal or reduction of irregularities, such as soft or weak stories, include the addition of braced frames or shear walls within the soft or

weak story. Torsional irregularities can be corrected by the addition of moment frames, braced frames, or shear walls to balance the distribution of stiffness and mass within a story. Discontinuous components such as columns or walls can be extended through the zone of discontinuity.

Partial demolition can also be an effective corrective measure for irregularities, although this obviously has significant impact on the appearance and utility of the building, and this may not be an appropriate alternative for historic structures. Portions of the structure that create the irregularity, such as setback towers or side wings, can be removed. Expansion joints can be created to transform a single irregular building into multiple regular structures; however, care must be taken to avoid the potential problems associated with pounding.

#### **2.6.6.3 Global structural stiffening**

Global stiffening of the structure shall be permitted as an applicable rehabilitation strategy. Global stiffening of the structure may be an effective rehabilitation strategy if the results of a seismic evaluation show deficiencies attributable to excessive lateral deflection of the building, and critical components do not have adequate ductility to resist the resulting deformations. Construction of new braced frames or shear walls within an existing structure are effective measures for adding stiffness.

#### **2.6.6.4 Global structural strengthening**

Global strengthening of the structure shall be permitted as an applicable rehabilitation strategy. Global strengthening of the structure may be an effective rehabilitation strategy if the results of a seismic evaluation show unacceptable performance attributable to a global deficiency in structural strength. This can be identified when the onset of global inelastic behavior occurs at levels of ground shaking that are substantially less than the selected level of ground shaking or large DCRs (or inelastic deformation demands) are present throughout the structure. By providing supplemental strength to such a lateral-force-resisting system, it is possible to raise the threshold of ground motion at which the onset of damage occurs. Shear walls and braced frames are effective elements for this purpose, but they may be significantly stiffer than the structure to which they are added, which requires their design to provide nearly all of the structure's lateral resistance. Moment-resisting frames, being more

flexible, may be more compatible with existing elements in some structures; however, such flexible elements may not become effective in the building's response until existing brittle elements have already been damaged.

#### **2.6.6.5 Mass reduction**

Reduction of mass in the building shall be permitted as an applicable rehabilitation strategy. Mass reduction may be an effective rehabilitation strategy if the results of a seismic evaluation show deficiencies attributable to excessive building mass, global structural flexibility, or global structural weakness. Mass and stiffness control the amount of force and deformation induced in a structure by ground motion. Reductions in mass can result in direct reductions in both the amount of force and deformation demand produced by earthquakes and, therefore, can be used in lieu of structural strengthening and stiffening. Mass can be reduced through demolition of upper stories, replacement of heavy cladding and interior partitions, or removal of heavy storage and equipment loads.

#### **2.6.6.6 Seismic isolation**

Seismic isolation of the building shall be permitted as an applicable rehabilitation strategy. Seismic isolation may be an effective rehabilitation strategy if the results of a seismic evaluation show deficiencies attributable to excessive seismic forces or deformation demands, or if it is desired to protect important contents and nonstructural components from damage. When a structure is seismically isolated, compliant bearings are inserted between the superstructure and its foundations. This produces a system (structure and isolation bearings) with a nearly rigid body translation of the structure above the bearings. Most of the deformation induced in the isolated system by the ground motion occurs within the compliant bearings, which are specifically designed to resist these concentrated displacements. Most bearings also have excellent energy dissipation characteristics (damping). Together, this results in greatly reduced demands on the existing elements of the structure, including contents and nonstructural components. For this reason, seismic isolation is often an appropriate strategy to achieve Enhanced Rehabilitation Objectives that include the protection of historic fabric, valuable contents, and equipment, or for buildings that contain important

operations and functions. This technique is most effective for relatively stiff buildings with low profiles and large mass. It is less effective for light, flexible structures.

#### **2.6.6.7 Supplemental energy dissipation**

Installation of supplemental energy dissipation devices shall be permitted as an applicable rehabilitation strategy. Installation of supplemental energy dissipation devices may be an effective rehabilitation strategy if the results of a seismic evaluation show deficiencies attributable to excessive deformations due to global structural flexibility in a building. Many available technologies allow the energy imparted to a structure by ground motion to be dissipated in a controlled manner through the action of special devices—fluid viscous dampers (hydraulic cylinders), yielding plates, or friction pads—resulting in an overall reduction in the displacements of the structure. The most commonly used devices dissipate energy through frictional, hysteretic, or viscoelastic processes. In order to dissipate substantial energy, dissipation devices must typically undergo significant deformation (or stroke), which requires that the structural experience substantial lateral displacements. Therefore, these systems are most effective in structures that are relatively flexible and have some inelastic deformation capacity. Energy dissipaters are most commonly installed in structures as components of braced frames. Depending on the characteristics of the device, either static or dynamic stiffness is added to the structure as well as energy dissipation capacity (damping). In some cases, although the structural displacements are reduced, the forces delivered to the structure can actually be increased.

### **2.7 Analysis Procedures**

This chapter sets forth requirements for analysis of buildings using the Systematic Rehabilitation Method. And specifies general analysis requirements for the mathematical modeling of buildings including basic assumptions, consideration of torsion, diaphragm flexibility,  $P - \Delta$  effects, soil-structure interaction, multidirectional effects, and overturning. Section 3.3 defines four analysis procedures included in this standard.

## **2.7.1 General analysis requirements**

### **2.7.1.1 Analysis procedure selection**

An analysis of the building shall be performed using the Linear Static Procedure (LSP), Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP), or Nonlinear Dynamic Procedure (NDP) selected based on the limitations.

### **2.7.1.2 Mathematical modeling**

A building shall be modeled, analyzed, and evaluated as a three-dimensional assembly of elements and components. Alternatively, use of a two-dimensional model shall be permitted if the building meets one of the following conditions:

1. The building has rigid diaphragms and horizontal torsion effects do not exceed the limits.
2. The building has flexible diaphragms.

If two-dimensional models are used, the three-dimensional nature of components and elements shall be considered when calculating stiffness and strength properties. If the building contains out-of-plane offsets in vertical lateral-force-resisting elements, the model shall explicitly account for such offsets in the determination of diaphragm demands. For nonlinear procedures, a connection shall be explicitly modeled if the connection is weaker, has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10%.

### **2.7.1.3 Horizontal torsion**

The effects of horizontal torsion shall be considered in accordance with this section. Torsion need not be considered in buildings with flexible diaphragms. The total horizontal torsional moment at a story shall be equal to the sum of the actual torsional moment and accidental torsional moment calculated as follows:

1. The actual torsional moment at a story shall be calculated by multiplying the seismic story shear force by the eccentricity between the center of mass and the center of rigidity measured perpendicular to the direction of the applied load. The center of mass

shall be based on all floors above the story under consideration. The center of rigidity of a story shall include all vertical seismic elements in the story.

2. The accidental torsion moment at a story shall be calculated as the seismic story shear force multiplied by a distance equal to 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

Effects of horizontal torsion shall be considered in accordance with the following requirements:

1. Increased forces and displacements due to actual torsion shall be calculated for all buildings.

2. The displacement multiplier,  $\eta$ , at each floor shall be calculated as the ratio of the maximum displacement at any point on the floor diaphragm to the average displacement ( $\delta_{\max} / \delta_{\text{avg}}$ ). Displacements shall be calculated for the applied loads.

3. Increased forces and displacements due to accidental torsion shall be considered unless the accidental torsional moment is less than 25 percent of the actual torsional moment, or the displacement multiplier  $\eta$  due to the applied load and accidental torsion is less than 1.1 at every floor.

4. If the displacement modifier  $\eta$  due to total torsional moment at any floor exceeds 1.50, two-dimensional models shall not be permitted and three-dimensional models that account for the spatial distribution of mass and stiffness shall be used.

5. When two-dimensional models are used, the effects of horizontal torsion shall be calculated as follows.

5.1. For the LSP and the LDP, forces and displacements shall be amplified by the maximum value of  $\eta$  calculated for the building.

5.2. For the NSP, the target displacement shall be amplified by the maximum value of  $\eta$  calculated for the building.

5.3. For the NDP, the amplitude of the ground acceleration record shall be amplified by the maximum value of  $\eta$  calculated for the building.

6. The effects of accidental torsion shall not be used to reduce force and deformation demands on components and elements.

Actual torsion is due to the eccentricity between centers of mass and stiffness. Accidental torsion is intended to cover the effects of the rotational component of the ground motion, differences between computed and actual stiffness, and unfavorable distributions of dead and live load masses.

The 10% threshold on additional displacement due to accidental torsion is based on judgment. The intent is to reward those building frames that are torsionally redundant and possess high torsional stiffness. Such structures are likely to be much less susceptible to torsional response than those framing systems possessing low redundancy and low torsional stiffness.

#### **2.7.1.4 $P - \Delta$ effects**

Buildings shall be evaluated for static  $P - \Delta$  effects and dynamic  $P - \Delta$  effects as specified as follow.

Static  $P - \Delta$  effects shall be included in linear or nonlinear analysis procedures as specified as follow respectively.

For linear procedures, the stability coefficient  $\theta$  shall be evaluated for each story in the building and for each direction of response using Equation (2.2).

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} \quad (2.2)$$

Where:

$P_i$  = Portion of the total weight of the structure including dead, permanent live, and 25% of transient live loads acting on the columns and bearing walls within story level  $i$

$V_i$  = The total calculated lateral shear force in the direction under consideration at story  $i$  due to earthquake response to the selected ground shaking level, as indicated by the selected linear analysis procedure

$h_i$  = Height of story  $i$ , which shall be taken as the distance between either the centerline of floor framing at each of the levels above and below, or the top of floor slabs at each of the levels above and below (or other common points of reference)

$\delta_i$  = Lateral drift in story  $i$ , in the direction under consideration, at its center of rigidity, using the same units as for measuring  $h_i$

When the stability coefficient  $\theta_i$  is less than 0.1 in all stories, the  $P - \Delta$  effects need not be considered. If the stability coefficient lies between 0.1 and 0.33, seismic forces and deformations in story  $i$  shall be increased by the factor  $1/(1-\theta_i)$ . When the stability coefficient  $\theta_i$  exceeds 0.33, the structure shall be considered unstable and the rehabilitation design modified to reduce the computed lateral deflections in the story to comply with this limitation.

For nonlinear procedures, static  $P - \Delta$  effects shall be incorporated in the analysis by including in the mathematical model the nonlinear force-deformation relationship of all elements and components subjected to axial forces.

Dynamic  $P - \Delta$  effects shall be included using the coefficient  $C_3$  defined for linear procedures.

#### **2.7.1.5 Component gravity loads for load combinations**

The following component gravity forces,  $Q_G$ , shall be considered for combination with seismic loads. When the effects of gravity and seismic loads are additive, the gravity loads shall be obtained in accordance with Equation (2-3).

$$Q_G = 1.1(Q_D + Q_L + Q_S) \quad (2.3)$$

When the effects of gravity and seismic loads are counteracting, the gravity loads shall be obtained in accordance with Equation (2-4).

$$Q_G = 0.9Q_D \quad (2.4)$$

Where:

$Q_D$  = Dead-load (action).

$Q_L$  = Effective live load (action), equal to 20% of the unreduced design live load, but not less than the actual live load.

$Q_S$  = Effective snow load (action) contribution to  $W$ .

#### **2.7.1.6 Verification of design assumptions**

Each component shall be evaluated to determine that assumed locations of inelastic deformations are consistent with strength and equilibrium requirements along the component length to verify that locations of potential inelastic action have been properly accounted for in the analysis. Each component shall also be evaluated for post-earthquake residual gravity load capacity by an approved rational analysis procedure that accounts for potential redistribution of gravity loads and reduction of strength or stiffness caused by earthquake damage to the structure.

#### **2.7.1.7 Overturning**

Structures shall be designed to resist overturning effects caused by seismic forces. Each vertical-force-resisting element receiving earthquake forces due to overturning shall be investigated for the cumulative effects of seismic forces applied at and above the level under consideration. The effects of overturning shall be evaluated at each level of the structure for linear procedures, or for nonlinear procedures.

Response to earthquake ground motion results in a tendency for structures and individual vertical elements of structures to overturn about their bases. Although actual overturning failure is very rare, overturning effects can result in significant stresses, as demonstrated in some local and global failures. In new building design, earthquake effects, including overturning, are evaluated for lateral forces that are significantly reduced (by an R-factor) from those that may actually develop in the structure.

For elements with positive attachment between levels that behave as single units such as reinforced concrete walls the overturning effects are resolved into component forces (e.g., flexure and shear at the base of the wall). The element is then proportioned with adequate strength using m-factors, where appropriate, to resist overturning effects resulting from these force levels.

Some elements, such as wood shear walls and foundations, may not be designed with positive attachment between levels. An overturning stability check is typically performed for such elements when designed using codes for new buildings. If the element has sufficient dead load to remain stable under the overturning effects of the design lateral forces and sufficient shear connection to the level below, then the design is deemed adequate. However, if dead load is inadequate to provide stability, then

hold-downs, piles, or other types of uplift anchors are provided to resist the residual overturning caused by the design forces.

In the linear and nonlinear procedures of this standard, lateral forces are not reduced by an R-factor, as they are for new buildings, so computed overturning effects are larger than typically calculated for new buildings. Although the procedure used for new buildings is not completely rational, it has resulted in successful performance. Therefore, it may not be appropriate to require that structures and elements of structures remain stable for the pseudo lateral forces used in the linear procedures in this standard. Instead, the designer must determine if positive direct attachment will be used to resist overturning effects or if dead loads will be used. If positive direct attachment will be used, then this attachment is treated just as any other element or component action.

However, if dead loads alone are used to resist overturning, then overturning is treated as a force-controlled behavior and the overturning demands are reduced to an estimate of the real overturning demands that can be transmitted to the element, considering the overall limiting strength of the structure.

There is no simple rational method available, shown to be consistent with observed behavior, to design or evaluate elements for overturning effects. The method described in this standard is rational, but inconsistent with procedures used for new buildings. To improve damage control, the full lateral forces used in the linear procedures of this standard are required for checking acceptability for performance levels higher than life safety.

When linear procedures are used, overturning effects shall be resisted through the stabilizing effect of dead loads acting alone or in combination with positive connection of structural components to elements or components below the level under consideration. Where dead loads alone are used to resist the effects of overturning, Equation (2-5) shall be satisfied:

$$M_{ST} > M_{OT} / (C_1 C_2 C_3 J) \quad (2.5)$$

Where:

$M_{OT}$  = Total overturning moment induced on the element by seismic forces applied at and above the level under consideration

$M_{ST}$  = Stabilizing moment produced by dead loads acting on the element

$C_1$ ,  $C_2$ , and  $C_3$  = Coefficients defined in FEMA356 and IR360 standard's section 3

$J$  = Coefficient defined in FEMA356 and IR360 standard's section 3

$M_{OT} / (C_1 C_2 C_3 J)$  need not exceed the overturning moment on the element, as limited by the expected strength of the structure. The element shall be evaluated for the effects of increased compression at the end about which it is being overturned. For this purpose, compression at the end of the element shall be considered a force-controlled action.

Alternatively, the load combination represented by Equation (2-6) shall be permitted for evaluating the adequacy of dead loads alone to resist the effects of overturning.

$$0.9M_{ST} > M_{OT} / (C_1 C_2 C_3 R_{OT}) \quad (2.6)$$

Where:

$R_{OT}$  = 10.0 for Collapse Prevention  
= 8.0 for Life Safety  
= 4.0 for Immediate Occupancy.

When nonlinear procedures are used, the effects of earthquake-induced uplift on the tension side of an element, or rocking, shall be included in the analytical model as a nonlinear degree of freedom. The adequacy of elements above and below the level at which uplift or rocking occurs, including the foundations, shall be evaluated for any redistribution of forces or deformations that occurs because of this rocking. Basis of the linear static procedure

If the Linear Static Procedure (LSP) is selected for seismic analysis of the building, the design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements shall be determined using a linearly elastic, static analysis. Buildings shall be modeled with linearly elastic stiffness and equivalent viscous damping values consistent with components responding at or near yield level. The pseudo lateral shall be used to calculate internal forces and system displacements due to the design earthquake.

### **2.7.2 Basis of the linear dynamic procedure**

If the Linear Dynamic Procedure (LDP) is selected for seismic analysis of the building, the design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements shall be determined using a linearly- elastic, dynamic analysis in compliance with the requirements of this section. Buildings shall be modeled with linearly elastic stiffness and equivalent viscous damping values consistent with components responding at or near yield level. Modeling and analysis procedures to calculate forces and deformations shall be in accordance with standard.

### **2.7.3 Basis of the nonlinear static procedure**

If the Nonlinear Static Procedure (NSP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. Mathematical modeling and analysis procedures shall comply with the requirements of FEMA356 or IR360.

#### **2.7.3.1 Modeling and analysis considerations**

The selection of a control node, the selection of lateral load patterns, the determination of the fundamental period, and analysis procedures shall comply with the requirements of this section. The relation between base shear force and lateral displacement of the control node shall be established for control node displacements ranging between zero and 150% of the target displacement ,  $\delta_t$  .

The component gravity loads shall be included in the mathematical model for combination with lateral loads as specified in past sections. The lateral loads shall be applied in both the positive and negative directions, and the maximum seismic effects shall be used for design. The analysis model shall be discretized to represent the load-deformation response of each component along its length to identify locations of inelastic action. All primary and secondary lateral-force-resisting elements shall be included in the model, as specified in past sections. The force-displacement behavior

of all components shall be explicitly included in the model using full backbone curves that include strength degradation and residual strength, if any.

#### **2.7.3.2 Control node displacement**

The control node shall be located at the center of mass at the roof of a building. For buildings with a penthouse, the floor of the penthouse shall be regarded as the level of the control node. The displacement of the control node in the mathematical model shall be calculated for the specified lateral loads.

#### **2.7.3.3 Lateral load distribution**

Lateral loads shall be applied to the mathematical model in proportion to the distribution of inertia forces in the plane of each floor diaphragm. For all analyses, at least two vertical distributions of lateral load shall be applied. One pattern shall be selected from each of the following two groups:

1. A modal pattern selected from one of the following:

1.1. Use of this distribution shall be permitted only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration, and the uniform distribution is used.

1.2. A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration. Use of this distribution shall be permitted only when more than 75% of the total mass participates in this mode.

1.3. A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate ground motion spectrum. This distribution shall be used when the period of the fundamental mode exceeds 1.0 second.

2. A second pattern selected from one of the following:

2.1. A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.

2.2. An adaptive load distribution the changes as the structure is displaced. The adaptive load Distribution shall be modified from the original load distribution using a procedure that considers the properties of the yielded structure.

#### 2.7.3.4 Idealized force-displacement curve

The nonlinear force-displacement relationship between base shear and displacement of the control node shall be replaced with an idealized relationship to calculate the effective lateral stiffness,  $K_e$  And effective yield strength,  $V_y$  of the building as shown in Figure 2.2 .

This relationship shall be bilinear, with initial slope  $K_e$  and post-yield slope  $\alpha$ . Line segments on the idealized force-displacement curve shall be located using an iterative graphical procedure that approximately balances the area above and below the curve. The effective lateral stiffness,  $K_e$  , shall be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure. The post-yield slope,  $\alpha$ , shall be determined by a line segment that passes through the actual curve at the calculated target displacement. The effective yield strength shall not be taken as greater than the maximum base shear force at any point along the actual curve.

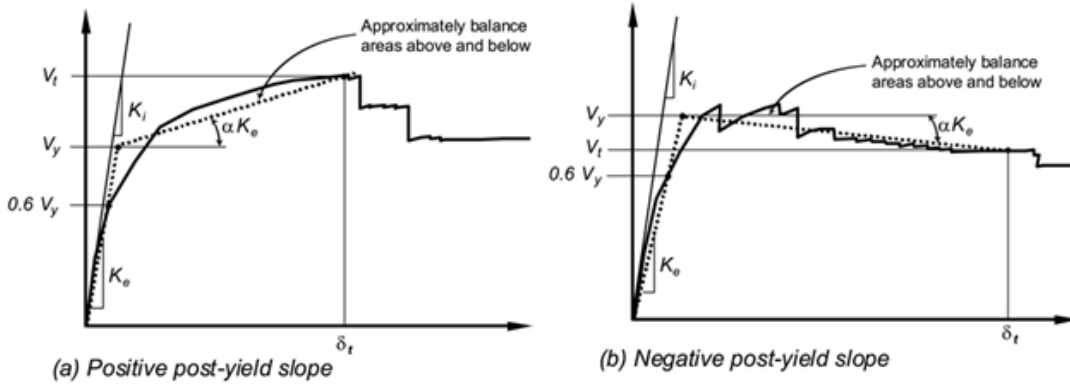


**Figure 2.2 : Idealized force-displacement curves**

#### 2.7.3.5 Period determination

The effective fundamental period in the direction under consideration shall be based on the idealized force- displacement curve. The effective fundamental period,  $T_e$ , shall be calculated in accordance with Equation (2-7):

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \quad (2.7)$$

Where:

$T_i$  = Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis

$K_i$  = Elastic lateral stiffness of the building in the direction under consideration

$K_e$  = Effective lateral stiffness of the building in the direction under consideration

### 2.7.3.6 Analysis of mathematical models

Separate mathematical models representing the framing along two orthogonal axes of the building shall be developed for two-dimensional analysis. A mathematical model representing the framing along two orthogonal axes of the building shall be developed for three-dimensional analysis. Independent analysis along each of the two orthogonal principal axes of the building shall be permitted unless concurrent evaluation of multidirectional effects is required.

### 2.7.3.7 Determination of forces and deformations

For buildings with rigid diaphragms at each floor level, the target displacement,  $\delta_t$ , shall be calculated in accordance with Equation (2.8) or by an approved procedure that accounts for the nonlinear response of the building. For buildings with non-rigid diaphragms at each floor level, diaphragm flexibility shall be explicitly included in the model. The target displacement shall be calculated as specified for rigid diaphragms, except that it shall be amplified by the ratio of the maximum displacement at any point on the roof to the displacement at the center of mass of the roof ( $\delta_{\max} / \delta_{cm}$ ).  $\delta_{\max}$  and  $\delta_{cm}$  shall be based on a response spectrum analysis of a three-dimensional model of the building. The target displacement so calculated shall be no less than that displacement

Given by Equation (2.8). No line of vertical seismic framing shall be evaluated for displacements smaller than the target displacement.

Alternatively, for buildings with flexible diaphragms at each floor level, a target displacement shall be calculated for each line of vertical seismic framing. The target displacement for an individual line of vertical seismic framing shall be as specified for buildings with rigid diaphragms, except that the masses shall be assigned to each line on the basis of tributary area. Forces and deformations corresponding to the control node displacement equaling or exceeding the target displacement shall comply with acceptance criteria.

### 2.7.3.8 Target displacement

The target displacement,  $\delta_t$ , at each floor level shall be calculated in accordance with Equation (2.8) and as specified:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \quad (2.8)$$

Where:

$C_0$  = Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system calculated using one of the following procedures:

- The first modal participation factor at the level of the control node;
- The modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement. This procedure shall be used if the adaptive load pattern defined in paragraph 2.2 of Section Lateral load distribution is used; or
- The appropriate value from Table 2-13.

$C_1$  = Modification factor to relate expected maximum inelastic displacements calculated for linear elastic response:

$$= 1.0 \text{ for } T_e \geq T_s$$

$$= [1.0 + (R-1)T_s / T_e] / R \text{ for } T_e < T_s$$

And not less than 1.0.

$T_e$  = Effective fundamental period of the building in the direction under consideration, sec.

$T_s$  = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

$R$  = Ratio of elastic strength demand to calculated yield strength coefficient calculated by Equation (2.9).

$C_2$  = Modification factor to represent the effects of pinched hysteresis shape, stiffness degradation, and strength deterioration on maximum displacement response. For linear procedures  $C_2$  shall be taken as 1.0.

$C_3$  = Modification factor to represent increased displacements due to dynamic P- $\Delta$  effects. For buildings with positive post-yield stiffness  $C_3$  shall be set equal to 1.0. For buildings with negative post-yield stiffness, values of  $C_3$  shall be calculated using Equation (2.10) .

$S_a$  = Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration.

$g$  = acceleration of gravity

he strength ratio  $R$  shall be calculated in accordance with Equation (2.9):

$$R = \frac{S_a}{V_y / W} \cdot C_m \quad (2.9)$$

Where  $S_a$  is defined above, and:

$V_y$  = Yield strength calculated using results of the NSP for the idealized nonlinear force-displacement curve developed for the building.

$W$  = Effective seismic weight.

$m$  = Effective mass factor from Table 3-1. Alternatively,  $C_m$  taken as the effective model mass calculated for the fundamental mode using an Eigenvalue analysis should be permitted.

Coefficient  $C_3$  shall be calculated in accordance with Equation (2.10) if the relation between base shear force and control node displacement exhibits negative post- yield stiffness.

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \quad (2.10)$$

Where  $R$  and  $T_e$  are as defined above, and:

$\alpha$  = Ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force- displacement relation shall be characterized by a bilinear relation as shown in Figure 2.2.

**Table 2.2 :** Values for effective mass factor  $C_m$

| No. of Stories | Concrete Moment Frame | Concrete Shear Wall | Concrete Pier-Spandrel | Steel Moment Frame | Steel Concentric Braced Frame | Steel Eccentric Braced Frame | Other |
|----------------|-----------------------|---------------------|------------------------|--------------------|-------------------------------|------------------------------|-------|
| 1-2            | 1                     | 1                   | 1                      | 1                  | 1                             | 1                            | 1     |
| 3 or more      | 0.9                   | 0.8                 | 0.8                    | 0.9                | 0.9                           | 0.9                          | 1     |

**Table 2.3 :** Values for modification factor

|                   | Shear Buildings                         |                            | Other Buildings  |
|-------------------|-----------------------------------------|----------------------------|------------------|
| Number of Stories | Triangular Load Pattern (1.1, 1.2, 1.3) | Uniform Load Pattern (2.1) | Any Load Pattern |
| 1                 | 1                                       | 1                          | 1                |
| 2                 | 1.2                                     | 1.15                       | 1.2              |
| 3                 | 1.2                                     | 1.2                        | 1.3              |
| 5                 | 1.3                                     | 1.2                        | 1.4              |
| 10+               | 1.3                                     | 1.2                        | 1.5              |

**Table 2.4 :** Values for modification factor  $C_2$

| Structural Performance Level | $T \leq 0.1 \text{ second}^3$ |                             | $T \geq TS \text{ second}^3$ |                             |
|------------------------------|-------------------------------|-----------------------------|------------------------------|-----------------------------|
|                              | Framing Type 1 <sup>1</sup>   | Framing Type 2 <sup>2</sup> | Framing Type 1 <sup>1</sup>  | Framing Type 2 <sup>2</sup> |
| Immediate Occupancy          | 1                             | 1                           | 1                            | 1                           |
| Life Safety                  | 1.3                           | 1                           | 1.1                          | 1                           |
| Collapse Prevention          | 1.5                           | 1                           | 1.2                          | 1                           |

1.Structures in which more than 30% of the story shear at any level is resisted by any combination of the following components, elements, or frames: ordinary moment-resisting frames, concentrically-

braced frames, frames with partially-restrained connections, tension-only braces, unreinforced masonry walls, shear-critical, piers, and spandrels of reinforced concrete or masonry.

2. All frames not assigned to Framing Type 1.

3. Linear interpolation shall be used for intermediate values of T.

#### **2.7.4 Basis of the nonlinear dynamic procedure**

If the Nonlinear Dynamic Procedure (NDP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load- deformation characteristics of individual components and elements of the building shall be subjected to earthquake shaking represented by ground motion time histories to obtain forces and displacements. Calculated displacements and internal forces shall be compared directly with acceptance criteria.

##### **2.7.4.1 Modeling and analysis considerations**

The modeling and analysis requirements specified for the NSP shall apply to the NDP, excluding considerations of control node and target displacements.

##### **2.7.4.2 Acceleration time histories**

Time history analysis shall be performed with no fewer than three data sets (each containing two horizontal components or, if vertical motion is to be considered, two horizontal components and one vertical component) of ground motion time histories that shall be selected and scaled from no fewer than three recorded events. Time histories shall have magnitude, fault distances, and source mechanisms that are equivalent to those that control the design earthquake ground motion. Where three recorded ground-motion time history data sets having these characteristics are not available, simulated time history data sets having equivalent duration and spectral content shall be used to make up the total number required. For each data set, the square root of the sum of the squares (SRSS) of the 5%-damped site-specific spectrum of the scaled horizontal components shall be constructed. The data sets shall be scaled such that the average value of the SRSS spectra does not fall below 1.3 (by 2800 and IR360 codes) times the 5%- damped spectrum for the design earthquake for periods

between 0.2T seconds and 1.5T seconds (where T is the fundamental period of the building).

Where three time history data sets are used in the analysis of a structure, the maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used to determine design acceptability. Where seven or more time history data sets are employed, the average value of each response parameter shall be permitted to determine design acceptability.

## 2.7.5 Acceptance criteria

### 2.7.5.1 Linear procedures

Deformation-controlled design actions  $Q_{UD}$  shall be calculated in accordance with Equation (2.11):

$$Q_{UD} = Q_G \pm Q_E \quad (2.11)$$

Where:

$Q_E$  = Action due to design earthquake loads calculated using forces and analysis models described in past sections.

$Q_G$  = Action due to design gravity loads as defined in past sections.

$Q_{UD}$  = Deformation-controlled design action due to gravity loads and earthquake loads.

Force-controlled design actions,  $Q_{UF}$ , shall be calculated using one of the following methods:

1.  $Q_{UF}$  Shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering load to the component under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building.
2. Alternatively,  $Q_{UF}$  shall be calculated in accordance with Equation (2.12).

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J} \quad (2.12)$$

Where:

$Q_{UF}$  = Force-controlled design action due to gravity loads in combination with earthquake loads

$J$  = Force-delivery reduction factor, greater than or equal to 1.0, taken as the smallest DCR of the components in the load path delivering force to the component in question. Alternatively, values of  $J$  equal to 2.0 in Zones of High Seismicity, 1.5 in Zones of Moderate Seismicity, and 1.0 in Zones of Low Seismicity shall be permitted when not based on calculated DCRs.  $J$  shall be taken as 1.0 for the Immediate Occupancy Structural Performance Level. In any case where the forces contributing to  $Q_{UF}$  are delivered by components of the lateral force resisting system that remain elastic,  $J$  shall be taken as 1.0.

#### 2.7.5.2 Acceptance criteria for linear procedures

Deformation-controlled actions in primary and secondary components and elements shall satisfy Equation (2.13).

$$m\kappa Q_{CE} \geq Q_{UD} \quad (2.13)$$

Where:

$m$  = Component or element demand modifier (factor) to account for expected ductility associated with this action at the selected Structural Performance Level.  $m$ -factors are specified in next Chapters

$Q_{CE}$  = Expected strength of the component or element at the deformation level under consideration for deformation-controlled actions

$\kappa$  = Knowledge factor defined in previous Sections

$Q_{CE}$ , the expected strength, shall be determined considering all coexisting actions on the component under the design loading condition by procedures specified in next chapters.

Force-controlled actions in primary and secondary components and elements shall satisfy Equation (2.14):

$$\kappa Q_{CL} \geq Q_{UF} \quad (2.14)$$

Where:

$Q_{CL}$  = Lower-bound strength of a component or element at the deformation level under consideration for force-controlled actions

$Q_{CL}$ , the lower-bound strength, shall be determined considering all coexisting actions on the component under the design loading condition.

### **2.7.5.3 Verification of design assumptions**

In addition to the requirements in previous Sections, the following verification of design assumptions shall be made. Where moments due to gravity loads in horizontally spanning primary components exceed 75% of the expected moment strength at any location, the possibility for inelastic flexural action at locations other than member ends shall be specifically investigated by comparing flexural actions with expected member strengths.

### **2.7.6 Nonlinear procedures**

#### **2.7.6.1 Design forces and deformations**

Component design forces and deformations shall be calculated in accordance with nonlinear analysis procedures.

#### **2.7.6.2 Acceptance criteria for nonlinear procedures**

Primary and secondary components shall have expected deformation capacities not less than maximum deformation demands calculated at the target displacement. Primary and secondary component demands shall be within the acceptance criteria for secondary components at the selected Structural Performance Level. Expected deformation capacities shall be determined considering all coexisting forces and deformations in accordance with next Chapter. The base shear at the target displacement,  $V_t$ , shall not be less than 80% of the effective yield strength of the structure,  $V_y$ .

Primary and secondary components modeled using the alternative simplified NSP analysis of previous Section shall meet the requirements of this section. Expected

deformation capacities shall not be less than maximum deformation demands calculated at the target displacement. Primary component demands shall be within the acceptance criteria for primary components at the selected Structural Performance Level. Demands on other components shall be within the acceptance criteria for secondary components at the selected Structural Performance Level. Expected deformation capacities shall be determined considering all coexisting forces and deformations by procedures specified in following chapter.

Primary and secondary components shall have lower- bound strengths not less than the maximum design forces. Lower-bound strengths shall be determined considering all coexisting forces and deformations by procedures specified in following chapter.

### **2.7.6.3 Verification of design assumptions**

In addition to the requirements in previous Sections, the following verification of design assumptions shall be made. Flexural plastic hinges shall not form away from component ends unless they are explicitly accounted for in modeling and analysis.

## **2.8 Concrete Structures**

This chapter sets forth requirements for the Systematic Rehabilitation of concrete lateral-force-resisting elements within a building. The requirements of this chapter shall apply to existing concrete components of a building system, rehabilitated concrete components of a building system, and new concrete components that are added to an existing building system.

**Table 2.5 :** Factors to translate lower bound material properties to expected strength material properties

| <b>Material Property</b>                   | <b>Factor</b> |
|--------------------------------------------|---------------|
| Concrete Compressive Strength              | 1.5           |
| Reinforcing Steel Tensile & Yield Strength | 1.25          |
| Connector Steel Yield Strength             | 1.5           |

### **2.8.1 Material properties and condition assessment**

Mechanical properties for concrete materials and components shall be based on available construction documents and as-built conditions for the particular structure.

Where such information fails to provide adequate information to quantify material properties or document the condition of the structure, such information shall be supplemented by materials tests and assessments of existing conditions. Material properties of existing concrete components shall be determined in accordance with next sections condition assessment shall be conducted in accordance with next sections. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor.

### **2.8.2 Properties of in-place materials and components**

The following component and connection material properties shall be obtained for the as-built structure:

1. Concrete compressive strength.
2. Yield and ultimate strength of conventional and pre-stressing reinforcing steel and metal connection hardware.

### **2.8.3 Component properties**

The following component properties and as-built conditions shall be established:

1. Cross-sectional dimensions of individual components and overall configuration of the structure.
2. Configuration of component connections, size of anchor bolts, thickness of connector material, anchorage and interconnection of embedments and the presence of bracing or stiffening components.
3. Modifications to components or overall configuration of the structure.
4. Current physical condition of components and connections, and the extent of any deterioration present.
5. Presence of conditions that influence building performance.

### **2.8.4 Condition assessment**

A condition assessment of the existing building and site conditions shall be performed as specified in this section. The condition assessment shall include the following:

1. The physical condition of primary and secondary components shall be examined and the presence of any degradation shall be noted.
2. The presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems shall be verified or established.
3. Other conditions including neighboring party walls and buildings, presence of nonstructural components, prior remodeling, and limitations for rehabilitation that may influence building performance shall be reviewed and documented.
4. A basis for selecting a knowledge factor in accordance with Section 2.4.2.3 shall be formulated.
5. Component orientation, plumpness, and physical dimensions shall be confirmed.

#### **2.8.5 Basis for the mathematical building model**

The results of the condition assessment shall be used to quantify the following items needed to create the mathematical building model:

1. Component section properties and dimensions.
2. Component configuration and the presence of any eccentricities or permanent deformation.
3. Connection configuration and the presence of any eccentricities.
4. Presence and effect of alterations to the structural system since original construction.
5. Interaction of nonstructural components and their involvement in lateral load resistance.

#### **2.8.6 Knowledge factor**

A knowledge factor ( $\kappa$ ) for computation of concrete component capacities and permissible deformations shall be selected in accordance with Section 2.2.1.4 with the following additional requirements specific to concrete components.

A knowledge factor, ( $\kappa$ ), equal to 0.75 shall be used if any of the following criteria are met:

1. Components are found damaged or deteriorated during assessment, and further testing is not performed to quantify their condition or justify the use of  $\kappa = 1.0$ .
2. Component mechanical properties have a coefficient of variation exceeding 25%.
3. Components contain archaic or proprietary material and the condition is uncertain.

### **2.8.7 Modeling and design**

Seismic rehabilitation of concrete structural elements of existing buildings shall comply with the requirements of ACI 318, except as otherwise indicated in this standard. Seismic evaluation shall identify brittle or low-ductility failure modes of force-controlled actions as defined in previous sections.

Evaluation of demands and capacities of reinforced concrete components shall include consideration of locations along the length where lateral and gravity loads produce maximum effects, where changes in cross-section or reinforcement result in reduced strength, and where abrupt changes in cross section or reinforcement, including splices, may produce stress concentrations, resulting in premature failure.

### **2.8.8 Stiffness**

Component stiffnesses shall be calculated considering shear, flexure, axial behavior and reinforcement slip deformations. Consideration shall be given to the state of stress on the component due to volumetric changes from temperature and shrinkage, and to deformation levels to which the component will be subjected under gravity and earthquake loading.

### **2.8.9 Nonlinear Procedures**

Where design actions are determined using the nonlinear procedures, component load-deformation response shall be represented by nonlinear load-deformation relations. Linear relations shall be permitted where nonlinear response will not occur in the component. The nonlinear load-deformation relation shall be based on experimental evidence or taken from quantities specified. For the Nonlinear Static Procedure (NSP),

use of the generalized load-deformation relation shown in Figure below or other curves defining behavior under monotonically increasing deformation shall be permitted. For the Nonlinear Dynamic Procedure (NDP), load-deformation relations shall define behavior under monotonically increasing lateral deformation and under multiple reversed deformation cycles as specified in this section.

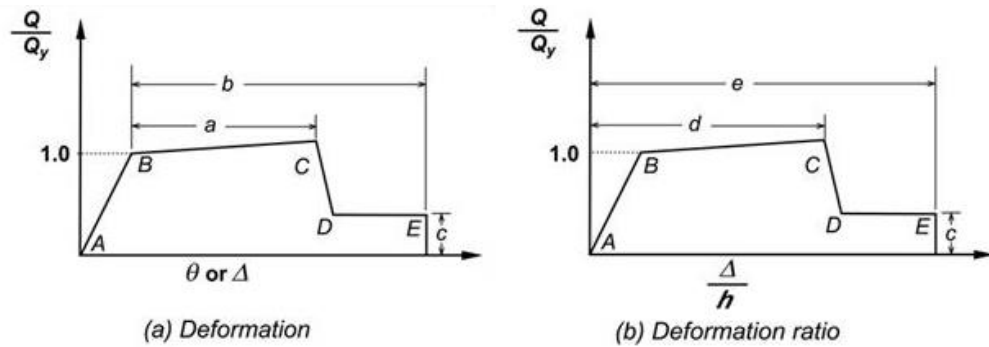


**Figure 2.3 :** Generalized force-deformation relations for concrete elements or components

The generalized load-deformation relation shown in Figure shall be described by linear response from A (unloaded component) to an effective yield B, then a linear response at reduced stiffness from point B to C, then sudden reduction in lateral load resistance to point D, then response at reduced resistance to E, and final loss of resistance thereafter. The slope from point B to C, ignoring effects of gravity loads acting through lateral displacements, shall be taken between zero and 10% of the initial slope unless an alternate slope is justified by experiment or analysis. Point C shall have an ordinate equal to the strength of the component and an abscissa equal to the deformation at which significant strength degradation begins. Representation of the load-deformation relation by points A, B, and C only (rather than all points A–E), shall be permitted if the calculated response does not exceed point C. Numerical values for the points identified in Figure 6-1 shall be as specified in next. Other load-deformation relations shall be permitted if justified by experimental evidence or analysis.

### 2.8.9.1 Strength and deformability

Actions in a structure shall be classified as being either deformation-controlled or force-controlled, as defined in previous Sections. Design strengths for deformation-controlled and force-controlled actions shall be calculated in accordance with next sections, respectively. Components shall be classified as having low, moderate, or high

ductility demands Where strength and deformation capacities are derived from test data, the tests shall be representative of proportions, details, and stress levels for the component and comply with requirements. The strength and deformation capacities of concrete members shall correspond to values resulting from earthquake loadings involving three fully reversed cycles to the design deformation level unless a larger or smaller number of deformation cycles is determined considering earthquake duration and the dynamic properties of the structure.

### **2.8.9.2 Deformation-controlled actions**

Strengths used for deformation-controlled actions shall be taken as equal to expected strengths,  $Q_{CE}$ , obtained experimentally, or calculated using accepted principles of mechanics. Expected strength is defined as the mean maximum resistance expected over the range of deformations to which the concrete component is likely to be subjected. When calculations are used to define expected strength, expected material properties should be used. Unless other procedures are specified in this standard, procedures specified in ACI 318 to calculate design strengths shall be permitted except that the strength reduction factor,  $\phi$  shall be taken equal to unity. Deformation capacities for acceptance of deformation-controlled actions calculated by nonlinear procedures shall be as specified in Sections 2.4.4 to Section 2.4.10 For components constructed of lightweight concrete,  $Q_{CE}$  shall be modified in accordance with ACI 318 procedures for lightweight concrete.

### **2.8.9.3 Force-controlled actions**

Strengths used for force-controlled actions shall be taken as lower-bound strengths,  $Q_{CL}$ , obtained experimentally, or calculated using established principles of mechanics. Lower-bound strength is defined as the mean minus one standard deviation of resistance expected over the range of deformations and loading cycles to which the concrete component is likely to be subjected. When calculations are used to define lower-bound strengths, lower bound estimates of material properties shall be used. Unless other procedures are specified in this standard, procedures specified in ACI 318 to calculate design strengths shall be permitted, except that the strength reduction factor,  $\phi$ , shall be taken equal to unity. For components constructed of lightweight

concrete,  $Q_{CL}$  shall be modified in accordance with ACI 318 procedures for lightweight concrete.

#### **2.8.9.4 Component ductility demand classification**

Where procedures in this chapter require classification of component ductility demand, components shall be classified as having low, moderate, or high ductility demands, based on the maximum value of the demand capacity ratio (DCR) defined in Section 2.3.2.1 and Section 2.3.2.2 for linear procedures, or the calculated displacement ductility for nonlinear procedures in accordance with Table 2.16.

#### **2.8.9.5 Flexure and axial loads**

Flexural strength and deformation capacity of members with and without axial loads shall be calculated according to the procedures of ACI 318 or by other approved methods. Strengths and deformation capacities of components with monolithic flanges shall be calculated considering concrete and developed longitudinal reinforcement within the effective flange width as defined in Section 2.4.3.1.

**Table 2.6 :** Component ductility demand classification

| Maximum value of DCR or displacement ductility | Descriptor                |
|------------------------------------------------|---------------------------|
| <2                                             | Low Ductility Demand      |
| 2 to 4                                         | Moderate Ductility Demand |
| >4                                             | High Ductility Demand     |

Strength and deformation capacities shall be determined considering available development of longitudinal reinforcement. Where longitudinal reinforcement has embedment or development length that is insufficient for development of reinforcement strength, flexural strength shall be calculated based on limiting stress capacity of the embedded bar as defined in Section 2.4.3.5.

Where flexural deformation capacities are calculated from basic principles of mechanics, reductions in deformation capacity due to applied shear shall be taken into

consideration. When using analytical models for flexural deformability that do not directly consider effect of shear, and where design shear equals or exceeds  $6\sqrt{f'_c A_w}$  where  $f'_c$  is in psi and  $A_w$  is gross area of web in square inches, the design value shall not exceed eighty percent of the value calculated using the analytical model.

For concrete columns under combined axial load and biaxial bending, the combined strength shall be evaluated considering biaxial bending. When using linear procedures, the design axial load,  $P_{UF}$ , shall be calculated as a force-controlled action in accordance with previous Sections. The design moments,  $M_{UD}$ , shall be calculated about each principal axis in accordance with previous Sections. Acceptance shall be based on the Equation (2.15):

$$\left( \frac{M_{UDx}}{m_x \kappa M_{CEx}} \right)^2 + \left( \frac{M_{UDy}}{m_y \kappa M_{CEy}} \right)^2 \leq 1 \quad (2.15)$$

where:

$M_{UDx}$  = design bending moment about x axis for axial load

$M_{UDy}$  = design bending moment about y axis for axial load

$M_{CEx}$  = expected bending moment strength about x axis

$M_{CEy}$  = expected bending moment strength about y axis

$m_x$  = m-factor for column for bending about x axis

$m_y$  = m-factor for column for bending about y axis

Alternative approaches based on principles of mechanics shall be permitted.

#### 2.8.9.6 Usable strain limits

Without confining transverse reinforcement, the maximum usable strain at the extreme concrete compression fiber shall not exceed 0.002 for components in nearly pure compression and 0.005 for other components unless larger strains are substantiated by experimental evidence and approved. Maximum usable compressive strains for confined concrete shall be based on experimental evidence and shall consider limitations posed by fracture of transverse reinforcement, buckling of longitudinal

reinforcement, and degradation of component resistance at large deformation levels. Maximum compressive strains in longitudinal reinforcement shall not exceed 0.02, and maximum tensile strains in longitudinal reinforcement shall not exceed 0.05.

#### **2.8.9.7 Shear and torsion**

Strengths in shear and torsion shall be calculated according to ACI 318 except as modified in this standard. Within yielding regions of components with moderate or high ductility demands, shear and torsional strength shall be calculated according to procedures for ductile components, such as the provisions in Chapter 21 of ACI 318. Within yielding regions of components with low ductility demands and outside yielding regions for all ductility demands, calculation of design shear strength using procedures for effective elastic response such as the provisions in Chapter 11 of ACI 318 shall be permitted.

Where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed not more than 50% effective in resisting shear or torsion. Where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed ineffective in resisting shear or torsion. For beams and columns in which perimeter hoops are either lap-spliced or have hooks that are not adequately anchored in the concrete core, transverse reinforcement shall be assumed not more than 50% effective in regions of moderate ductility demand and shall be assumed ineffective in regions of high ductility demand. Shear friction strength shall be calculated according to ACI 318, taking into consideration the expected axial load due to gravity and earthquake effects. Where rehabilitation involves the addition of concrete requiring overhead work with dry-pack, the shear friction coefficient  $\mu$  shall be taken as equal to 70% of the value specified by ACI 318.

#### **2.8.9.8 Development and splices of reinforcement**

Development of straight bars, hooked bars, and lap-spliced bars shall be calculated according to the provisions of ACI 318, with the following modifications:

1. Within yielding regions of components with moderate or high ductility demands as classified in Section 2.4.3.2, deformed straight bars, hooked bars, and lap-spliced bars shall meet the development requirements of Chapter 21 of ACI 318. Within yielding regions of components with low ductility demands, and outside yielding regions for all ductility demands, bars shall meet the development requirements of Chapter 12 of ACI 318, except requirements for lap splices shall be the same as those for straight development of bars in tension without consideration of lap splice classifications.

2. Where existing deformed straight bars, hooked bars, and lap-spliced bars do not meet the development requirements of (1) above, the capacity of existing reinforcement shall be calculated using Equation (2.16):

$$f_s = \frac{l_b}{l_d} f_y \quad (2.16)$$

where  $f_s$  = maximum stress that can be developed in the bar for the straight development, hook, or lap splice length  $l_b$  provided;  $f_y$  = yield strength of reinforcement; and  $l_d$  = length required by Chapter 12 or Chapter 21 (as appropriate) of ACI 318 for straight development, hook development, or lap splice length, except required splice lengths may be taken as straight bar development lengths in tension. Where transverse reinforcement is distributed along the development length with spacing not exceeding one-third of the effective depth of the component, it shall be permitted to assume the reinforcement retains the calculated maximum stress to high ductility demands. For larger spacing of transverse reinforcement, the developed stress shall be assumed to degrade from  $f_s$  to  $0.2 f_s$  at a ductility demand or DCR equal to 2.0.

3. Strength of deformed straight, discontinuous bars embedded in concrete sections or beam-column joints, with clear cover over the embedded bar not less than  $d_b$ , shall be calculated according to Equation (2.17):

$$f_s = \frac{2500}{d_b} l_e \leq f_y \quad (2.17)$$

Where  $f_s$  = maximum stress (in psi) that can be developed in an embedded bar having embedment length  $l_e$  (in inches),  $d_b$  = diameter of embedded bar (in inches), and  $f_y$

= bar yield stress (in psi). When  $f_s$  is less than  $f_y$  and the calculated stress in the bar due to design loads equals or exceeds  $f_s$ , the maximum developed stress shall be assumed to degrade from  $f_s$  to  $0.2 f_s$  at a ductility demand or DCR equal to 2.0. In beams with short bottom bar embeddings into beam-column joints, flexural strength shall be calculated considering the stress limitation of Equation (2.17).

4. For plain straight bars, hooked bars, and lap-spliced bars, development and splice lengths shall be taken as twice the values determined in accordance with ACI 318 unless other lengths are justified by approved tests or calculations considering only the chemical bond between the bar and the concrete.

5. Doweled bars added in seismic rehabilitation shall be assumed to develop yield stress when all the following conditions are satisfied:

5.1. Drilled holes for dowel bars are cleaned with a stiff brush that extends the length of the hole.

5.2. Embedment length  $l_e$  is not less than  $10 d_b$

5.3. Minimum spacing of dowel bars is not less than  $4 l_e$  and minimum edge distance is not less than  $2 l_e$ . Design values for dowel bars not satisfying these conditions shall be verified by test data. Field samples shall be obtained to ensure design strengths are developed.

#### **2.8.10 Rehabilitation—general requirements**

Upon determining that concrete elements in an existing building are deficient for the selected Rehabilitation Objective, these elements shall be rehabilitated or replaced or the structure shall be otherwise rehabilitated so that the element is no longer deficient for the selected rehabilitation objective. If replacement of the element is selected, the new element shall be designed in accordance with an approved building code.

Rehabilitation measures shall be evaluated in accordance with the principles and requirements of this standard, to assure that the completed rehabilitation achieves the selected Rehabilitation Objective. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated

structure. The compatibility of new and existing components shall be checked at displacements consistent with the selected performance level.

Connections required between existing and new elements shall satisfy requirements of this standard.

### **2.8.11 Concrete moment frames**

Concrete moment frames shall be defined as elements composed primarily of horizontal framing components (beams and/or slabs), vertical framing components (columns) and joints connecting horizontal and vertical framing components. These elements resist lateral loads acting alone, or in conjunction with shear walls, braced frames, or other elements in a dual system.

Frames that are cast monolithically, including monolithic concrete frames created by the addition of new material, shall meet the provisions of this section. Frames covered under this section include reinforced concrete beam-column moment frames, prestressed concrete beam-column moment frames, and slab-column moment frames. Precast concrete frames, concrete frames with infills, and concrete braced frames shall meet the provisions of this standard, respectively.

#### **2.8.11.1 Reinforced concrete beam-column moment frames**

Reinforced concrete beam-column moment frames shall satisfy the following conditions:

1. Framing components shall be beams (with or without slabs), columns, and their connections.
2. Beams and columns shall be of monolithic construction that provides for moment transfer between beams and columns.
3. Primary reinforcement in components contributing to lateral load resistance shall be nonprestressed.

Special Moment Frames, Intermediate Moment Frames, and Ordinary Moment Frames as defined in ASCE 7, shall be deemed to satisfy the above conditions. This classification shall include existing construction, new construction, and existing construction that has been rehabilitated.

The analytical model for a beam-column frame element shall represent strength, stiffness, and deformation capacity of beams, columns, beam-column joints, and other components of the frame, including connections with other elements. Potential failure in flexure, shear, and reinforcement development at any section along the component length shall be considered. Interaction with other elements, including nonstructural elements and components, shall be included.

Analytical models representing a beam-column frame using line elements with properties concentrated at component centerlines shall be permitted. Where beam and column centerlines do not intersect, the effects of the eccentricity between centerlines on framing shall be taken into account. Where the centerline of the narrower component falls within the middle third of the adjacent framing component measured transverse to the framing direction, however, this eccentricity need not be considered. Where larger eccentricities occur, the effect shall be represented either by reductions in effective stiffness, strength, and deformation capacity, or by direct modeling of the eccentricity.

The beam-column joint in monolithic construction shall be represented as a stiff or rigid zone having horizontal dimensions equal to the column cross-sectional dimensions and vertical dimension equal to the beam depth, except that a wider joint shall be permitted where the beam is wider than the column and where justified by experimental evidence. The model of the connection between the columns and foundation shall be selected based on the details of the column-foundation connection and rigidity of the foundation-soil system.

Action of the slab as a diaphragm interconnecting vertical elements shall be represented. Action of the slab as a composite beam flange shall be considered in developing stiffness, strength, and deformation capacities of the beam component model.

Inelastic action shall be restricted to those components and actions listed in Tables 2.17 through 2.19 (FEMA and IR360), except where it is demonstrated by experimental evidence and analysis that other inelastic action is acceptable for the selected performance level. Acceptance criteria shall be as specified in following sections.

### **2.8.11.2 Nonlinear static procedure**

Beams and columns shall be modeled using concentrated plastic hinge models or distributed plastic hinge models. Other models whose behavior has been demonstrated to represent the behavior of reinforced concrete beam and column components subjected to lateral loading shall be permitted. The beam and column model shall be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends. Where nonlinear response is expected in a mode other than flexure, the model shall be established to represent these effects.

Monotonic load-deformation relations shall be according to the generalized load-deformation relation shown in Figure 2.3, except that different relations shall be permitted where verified by experiments. The overall load-deformation relation shall be established so that the maximum resistance is consistent with the design strength specifications.

For beams and columns, the generalized deformation in Figure 2.3 shall be either the chord rotation or the plastic hinge rotation. For beam-column joints, the generalized deformation shall be shear strain. Values of the generalized deformation at points B, C, and D shall be derived from experiments or rational analyses, and shall take into account the interactions between flexure, axial load, and shear.

**Table 2.7 : Modeling parameters and numerical acceptance criteria for nonlinear procedures- reinforced concrete beams**

| Conditions |   |   | Modeling Parameters <sup>3</sup> |    |                         | Acceptance Criteria <sup>3</sup> |                |  |           |  |
|------------|---|---|----------------------------------|----|-------------------------|----------------------------------|----------------|--|-----------|--|
|            |   |   | Plastic Rotation Angle, radians  |    | Residual Strength Ratio | Plastic Rotation Angle, radians  |                |  |           |  |
|            |   |   |                                  |    |                         | Performance Level                |                |  |           |  |
|            |   |   |                                  |    |                         | IO                               | Component Type |  |           |  |
|            |   |   |                                  |    |                         |                                  | Primary        |  | Secondary |  |
|            |   |   | LS                               | CP | LS                      |                                  | CP             |  |           |  |
| a          | b | c |                                  |    |                         |                                  |                |  |           |  |

| i.Beam controlled by flexure <sup>1</sup> |                               |                               |       |       |     |        |       |       |       |       |
|-------------------------------------------|-------------------------------|-------------------------------|-------|-------|-----|--------|-------|-------|-------|-------|
| $\frac{\rho - \rho'}{\rho_{bal}}$         | Trans. Reinf.<br><sup>2</sup> | $\frac{V}{b_w d \sqrt{f_c'}}$ |       |       |     |        |       |       |       |       |
| ≤ 0.0                                     | C                             | ≤ 3                           | 0.025 | 0.05  | 0.2 | 0.01   | 0.02  | 0.025 | 0.02  | 0.05  |
| ≤ 0.0                                     | C                             | ≥ 6                           | 0.02  | 0.04  | 0.2 | 0.005  | 0.01  | 0.02  | 0.02  | 0.04  |
| ≥ 0.5                                     | C                             | ≤ 3                           | 0.02  | 0.03  | 0.2 | 0.005  | 0.01  | 0.02  | 0.02  | 0.03  |
| ≥ 0.5                                     | C                             | ≥ 6                           | 0.015 | 0.02  | 0.2 | 0.005  | 0.005 | 0.015 | 0.015 | 0.02  |
| ≤ 0.0                                     | NC                            | ≤ 3                           | 0.02  | 0.03  | 0.2 | 0.005  | 0.01  | 0.02  | 0.02  | 0.03  |
| ≤ 0.0                                     | NC                            | ≥ 6                           | 0.01  | 0.015 | 0.2 | 0.0015 | 0.005 | 0.01  | 0.01  | 0.015 |
| ≥ 0.5                                     | NC                            | ≤ 3                           | 0.01  | 0.015 | 0.2 | 0.005  | 0.01  | 0.01  | 0.01  | 0.015 |
| ≥ 0.5                                     | NC                            | ≥ 6                           | 0.005 | 0.01  | 0.2 | 0.0015 | 0.005 | 0.005 | 0.005 | 0.01  |

| ii.Beam controlled by shear <sup>1</sup> |  |  |       |      |     |        |       |       |       |      |
|------------------------------------------|--|--|-------|------|-----|--------|-------|-------|-------|------|
| Stirrup spacing ≤ d / 2                  |  |  | 0.003 | 0.02 | 0.2 | 0.0015 | 0.002 | 0.003 | 0.01  | 0.02 |
| Stirrup spacing > d / 2                  |  |  | 0.003 | 0.01 | 0.2 | 0.0015 | 0.002 | 0.003 | 0.005 | 0.01 |

| iii.Beam controlled by inadequate development or splicing along the span <sup>1</sup> |  |  |       |      |   |        |       |       |       |      |
|---------------------------------------------------------------------------------------|--|--|-------|------|---|--------|-------|-------|-------|------|
| Stirrup spacing ≤ d / 2                                                               |  |  | 0.003 | 0.02 | 0 | 0.0015 | 0.002 | 0.003 | 0.01  | 0.02 |
| Stirrup spacing > d / 2                                                               |  |  | 0.003 | 0.01 | 0 | 0.0015 | 0.002 | 0.003 | 0.005 | 0.01 |

| iv.Beam controlled by inadequate embendment into beam-column joint <sup>1</sup> |  |  |       |      |     |      |      |       |      |      |
|---------------------------------------------------------------------------------|--|--|-------|------|-----|------|------|-------|------|------|
|                                                                                 |  |  | 0.015 | 0.03 | 0.2 | 0.01 | 0.01 | 0.015 | 0.02 | 0.03 |

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.

2. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.

3. Linear interpolation between values listed in the table shall be permitted.

**Table 2.8 :** Modeling parameters and numerical acceptance criteria for nonlinear procedures- reinforced concrete columns

| Conditions                                                                                                                                                                                                                                                                                                                                                                                                | Modeling Parameters <sup>4</sup> |                               |                         | Acceptance Criteria <sup>4</sup> |                |       |           |       |       |       |    |    |
|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------|-------------------------------|-------------------------|----------------------------------|----------------|-------|-----------|-------|-------|-------|----|----|
|                                                                                                                                                                                                                                                                                                                                                                                                           | Plastic Rotation Angle, radians  |                               | Residual Strength Ratio | Plastic Rotation Angle, radians  |                |       |           |       |       |       |    |    |
|                                                                                                                                                                                                                                                                                                                                                                                                           |                                  |                               |                         | Performance Level                |                |       |           |       |       |       |    |    |
|                                                                                                                                                                                                                                                                                                                                                                                                           |                                  |                               |                         | IO                               | Component Type |       |           |       | LS    | CP    | LS | CP |
|                                                                                                                                                                                                                                                                                                                                                                                                           |                                  |                               |                         |                                  | Primary        |       | Secondary |       |       |       |    |    |
|                                                                                                                                                                                                                                                                                                                                                                                                           | a                                | b                             | c                       | LS                               | CP             | LS    | CP        | LS    | CP    |       |    |    |
| i.Columns controlled by flexure <sup>1</sup>                                                                                                                                                                                                                                                                                                                                                              |                                  |                               |                         |                                  |                |       |           |       |       |       |    |    |
| $\frac{P}{A_g f_c'}$                                                                                                                                                                                                                                                                                                                                                                                      | Trans. Reinf.2                   | $\frac{V}{b_w d \sqrt{f_c'}}$ |                         |                                  |                |       |           |       |       |       |    |    |
| ≤ 0.1                                                                                                                                                                                                                                                                                                                                                                                                     | C                                | ≤ 3                           | 0.02                    | 0.03                             | 0.2            | 0.005 | 0.015     | 0.02  | 0.02  | 0.03  |    |    |
| ≤ 0.1                                                                                                                                                                                                                                                                                                                                                                                                     | C                                | ≥ 6                           | 0.016                   | 0.024                            | 0.2            | 0.005 | 0.012     | 0.016 | 0.016 | 0.024 |    |    |
| ≥ 0.4                                                                                                                                                                                                                                                                                                                                                                                                     | C                                | ≤ 3                           | 0.015                   | 0.025                            | 0.2            | 0.003 | 0.012     | 0.015 | 0.018 | 0.025 |    |    |
| ≥ 0.4                                                                                                                                                                                                                                                                                                                                                                                                     | C                                | ≥ 6                           | 0.012                   | 0.02                             | 0.2            | 0.003 | 0.01      | 0.012 | 0.013 | 0.02  |    |    |
| ≤ 0.1                                                                                                                                                                                                                                                                                                                                                                                                     | NC                               | ≤ 3                           | 0.006                   | 0.015                            | 0.2            | 0.005 | 0.005     | 0.006 | 0.01  | 0.015 |    |    |
| ≤ 0.1                                                                                                                                                                                                                                                                                                                                                                                                     | NC                               | ≥ 6                           | 0.005                   | 0.012                            | 0.2            | 0.005 | 0.004     | 0.005 | 0.008 | 0.012 |    |    |
| ≥ 0.4                                                                                                                                                                                                                                                                                                                                                                                                     | NC                               | ≤ 3                           | 0.003                   | 0.01                             | 0.2            | 0.002 | 0.002     | 0.003 | 0.006 | 0.01  |    |    |
| ≥ 0.4                                                                                                                                                                                                                                                                                                                                                                                                     | NC                               | ≥ 6                           | 0.002                   | 0.008                            | 0.2            | 0.002 | 0.002     | 0.002 | 0.005 | 0.008 |    |    |
| ii.Columns controlled by shear <sup>1,3</sup>                                                                                                                                                                                                                                                                                                                                                             |                                  |                               |                         |                                  |                |       |           |       |       |       |    |    |
| All cases                                                                                                                                                                                                                                                                                                                                                                                                 |                                  |                               | —                       | —                                | —              | —     | —         | —     | 0.003 | 0.004 |    |    |
| iii.Columns controlled by inadequate development or splicing along the clear height                                                                                                                                                                                                                                                                                                                       |                                  |                               |                         |                                  |                |       |           |       |       |       |    |    |
| Hoop spacing ≤ d / 2                                                                                                                                                                                                                                                                                                                                                                                      |                                  |                               | 0.01                    | 0.02                             | 0.4            | 0.005 | 0.005     | 0.01  | 0.01  | 0.02  |    |    |
| Hoop spacing > d / 2                                                                                                                                                                                                                                                                                                                                                                                      |                                  |                               | 0                       | 0.01                             | 0.2            | 0     | 0         | 0     | 0.005 | 0.01  |    |    |
| iv.Columns with axial loads exceeding 0.7P <sub>0</sub> <sup>1,3</sup>                                                                                                                                                                                                                                                                                                                                    |                                  |                               |                         |                                  |                |       |           |       |       |       |    |    |
| Conforming hoops over the entire length                                                                                                                                                                                                                                                                                                                                                                   |                                  |                               | 0.015                   | 0.025                            | 0.02           | 0     | 0.005     | 0.01  | 0.01  | 0.02  |    |    |
| All other cases                                                                                                                                                                                                                                                                                                                                                                                           |                                  |                               | 0                       | 0                                | 0              | 0     | 0         | 0     | 0     | 0     |    |    |
| 1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.                                                                                                                                                                                                                                                      |                                  |                               |                         |                                  |                |       |           |       |       |       |    |    |
| 2. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming. |                                  |                               |                         |                                  |                |       |           |       |       |       |    |    |
| 3.To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.                                                                                                                                                                                                                                                                    |                                  |                               |                         |                                  |                |       |           |       |       |       |    |    |
| 4. Linear interpolation between values listed in the table shall be permitted.                                                                                                                                                                                                                                                                                                                            |                                  |                               |                         |                                  |                |       |           |       |       |       |    |    |

### 2.8.11.3 Nonlinear dynamic procedure

For the NDP, the complete hysteretic behavior of each component shall be modeled using properties verified by experimental evidence. The use of the generalized load-deformation relation described by Figure 2.3 to represent the envelope relation for the analysis shall be permitted. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics.

### 2.8.11.4 Strength

The maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, development, and other actions at all points along the length of the component under the actions of design gravity and earthquake load combinations.

### 2.8.11.5 Columns

For columns, the contribution of concrete to shear strength,  $V_c$ , calculated according to Equation (2.18) shall be permitted.

$$V_c = \lambda k \left( \frac{6\sqrt{f'_c}}{M/V_d} \sqrt{1 + \frac{N_u}{6\sqrt{f'_c} A_g}} \right) (0.8b_w h) \quad (2.18)$$

in which  $k = 1.0$  in regions of low ductility demand,  $0.7$  in regions of high ductility demand, and varies linearly between these extremes in regions of moderate ductility demand;  $\lambda = 0.75$  for lightweight aggregate concrete and  $1.0$  for normal weight aggregate concrete;  $N_u$  = axial compression force in pounds ( $= 0$  for tension force);  $M/V$  is the largest ratio of moment to shear under design loadings for the column but shall not be taken greater than  $3$  or less than  $2$ ;  $d$  is the effective depth; and  $A_g$  is the gross cross-sectional area of the column. It shall be permitted to assume  $d = 0.8h$ , where  $h$  is the dimension of the column in the direction of shear. Where axial force is calculated from the linear procedures of Section 2.3.2, the maximum compressive axial load for use in Equation (2.18) shall be taken as equal to the value calculated using Equation (2.4) considering design gravity load only, and the minimum compression axial load shall be calculated according to Equation (2.11). Alternatively,

limit analysis shall be permitted to be used to determine design axial loads for use with the linear analysis procedures. Alternative formulations for column strength that consider effect of reversed cyclic, inelastic deformations and that are verified by experimental evidence shall be permitted.

For columns satisfying the detailing and proportioning requirements of Chapter 21 of ACI 318, the shear strength equations of ACI 318 shall be permitted to be used. For beam-column joints, the nominal cross-sectional area,  $A_j$ , shall be defined by a joint depth equal to the column dimension in the direction of framing and a joint width equal to the smallest of (1) the column width, (2) the beam width plus the joint depth, and (3) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. Design forces shall be calculated based on development of flexural plastic hinges in adjacent framing members, including effective slab width, but need not exceed values calculated from design gravity and earthquake-load combinations. Nominal joint shear strength  $V_n$  shall be calculated according to the general procedures of ACI 318, as modified by Equation (2.19):

$$Q_{CL} = V_n = \lambda \gamma \sqrt{f'_c A_j} \quad (2.19)$$

in which  $\lambda = 0.75$  for lightweight aggregate concrete and 1.0 for normal weight aggregate concrete,  $A_j$  is the effective horizontal joint area with dimensions as defined above, and  $\gamma$  is as defined in Table 2-20.

#### **2.8.11.6 Acceptance criteria**

Calculated component actions shall satisfy the requirements. Where the generalized deformation is taken as rotation in the flexural plastic hinge zone in beams and columns, the plastic hinge rotation capacities shall be as defined by Tables 2.17 and 2.18. Where the generalized deformation is shear distortion of the beam-column joint, shear angle capacities shall be as defined by Table 2.19. For columns designated as primary components and for which calculated design shears exceed design shear strength as defined by Equation (2.18), the permissible deformation for the Collapse Prevention Performance Level shall not exceed the deformation at which shear strength is calculated to be reached; the permissible deformation for the Life Safety Performance Level shall not exceed three quarters of that value. Where inelastic action

Is indicated for a component or action not listed in these tables, the performance shall be deemed unacceptable. Alternative approaches or values shall be permitted where justified by experimental evidence and analysis.

#### **2.8.11.7 Rehabilitation measures**

Concrete beam-column moment frame components that do not meet the acceptance criteria for the selected rehabilitation objective shall be rehabilitated. Rehabilitation measures shall meet the requirements of Section which mentioned above and other provisions of this standard.

#### **2.8.11.8 Stiffness**

Nonlinear load-deformation relations shall comply with the requirements of previous Sections and the reinforced concrete frame requirements. Values of the generalized deformation at points B, C, and D in Figure 2.3 shall be either derived from experiments or approved rational analyses, and shall take into account the interactions between flexure, axial load, and shear. Alternatively, where the generalized deformation is taken as rotation in the flexural plastic hinge zone, and where the three conditions are satisfied, beam plastic hinge rotation capacities shall be as defined by Table 2.17. Columns and joints shall be modeled as described above.

For the NDP, the complete hysteretic behavior of each component shall be modeled using properties verified by experimental evidence. The relation of Figure 2.3 shall be taken to represent the envelope relation for the analysis. Unloading and reloading properties shall represent significant stiffness and strength degradation characteristics as influenced by prestressing.

#### **2.8.11.9 Strength**

Component strengths shall be computed according to the general requirements and the additional requirements. Effects of prestressing on strength shall be considered. For deformation-controlled actions, prestress shall be assumed to be effective for the purpose of determining the maximum actions that may be developed associated with nonlinear response of the frame. For force- controlled actions, the effects on strength

of prestress loss shall also be considered as a design condition, where these losses are possible under design load combinations including inelastic deformation reversals.

Post-tensioned concrete beam-column moment frame components that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements and other provisions of this standard.

## 2.8.12 Concrete shear walls

The provisions of this Section shall apply to all shear walls in all types of structural systems that incorporate shear walls. This includes isolated shear walls, shear walls used in dual (wall-frame) systems, coupled shear walls, and discontinuous shear walls. Shear walls shall be permitted to be considered as solid walls if they have openings that do not significantly influence the strength or inelastic behavior of the wall. Perforated shear walls shall be defined as walls having a regular pattern of openings in both horizontal and vertical directions that creates a series of pier and deep beam elements referred to as wall segments.

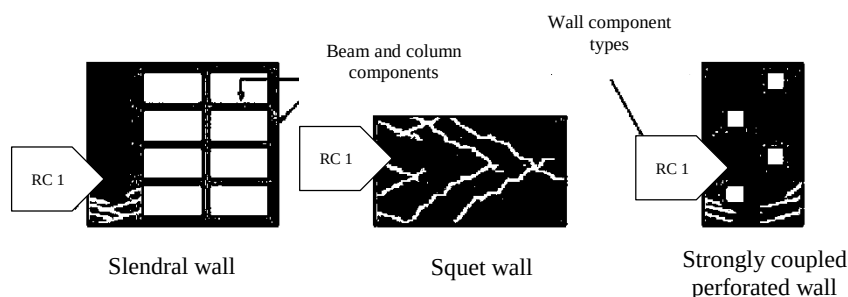


**Figure 2.4 : Cantilever wall mechanisms**

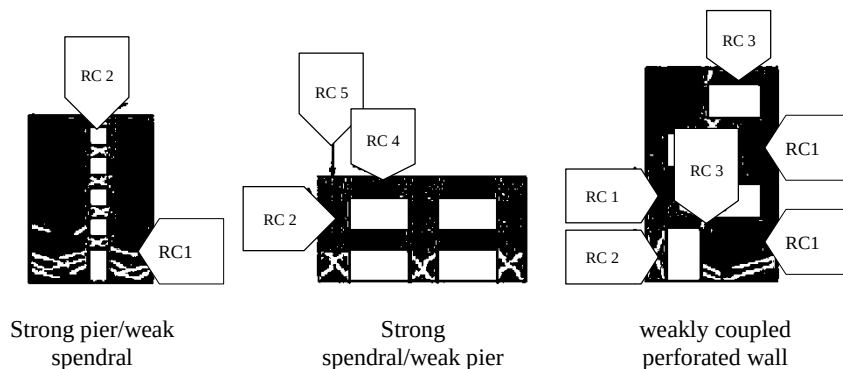


**Figure 2.5 : Pier/Spandrel mechanisms**

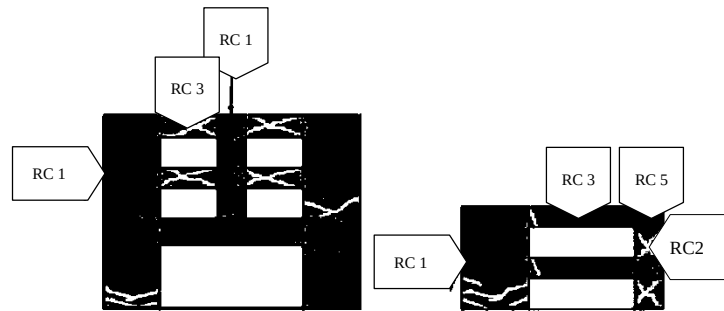


**Figure 2.6 : Mixed mechanisms**

**Table 2.9 : Reinforced concrete shear wall component types (from FEMA 306)**

| Component Type per <i>FEMA 306</i> |                                     | Description                                                                                                                                    | FEMA 356 Designation                                         |
|------------------------------------|-------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------|
| RC1                                | Isolated Wall or Stronger Wall Pier | Stronger than beam or spandrel elements that may frame into it so that nonlinear behavior (and damage) is generally concentrated at the base.  | Monolithic reinforced concrete wall or vertical wall segment |
|                                    | Weaker Wall Pier                    | Weaker than the spandrels to which it connects; characterized by flexural hinging top and bottom.                                              |                                                              |
| RC3                                | Weaker Spandrel or Coupling Beam    | Weaker than the wall piers to which it connects; characterized by hinging at each end, shear failure, sliding shear failure, etc.              | Horizontal wall segment or coupling beam                     |
| RC4                                | Stronger Spandrel                   | Should not suffer damage because it is stronger than attached piers. If this component is damaged, it should probably be re-classified as RC3. |                                                              |
| RC5                                | Pier-Spandrel Panel Zone            | Typically not a critical area in RC walls                                                                                                      | Wall segment                                                 |

#### 2.8.12.1 Monolithic reinforced concrete shear walls and wall segments

Monolithic reinforced concrete shear walls shall consist of vertical cast-in-place elements, either uncoupled or coupled, in open or closed shapes. These walls shall have relatively continuous cross sections and reinforcement and shall provide both vertical and lateral force resistance.

Shear walls or wall segments with axial loads greater than  $0.35 P_0$  shall not be considered effective in resisting seismic forces. The maximum spacing of horizontal and vertical reinforcement shall not exceed 18 inches. Walls with horizontal and

vertical reinforcement ratios less than 0.0025, but with reinforcement spacing less than 18 inches, shall be permitted where the shear force demand does not exceed the reduced nominal shear strength of the wall calculated.

### **2.8.13 Reinforced Concrete Columns Supporting Discontinuous Shear Walls**

Reinforced concrete columns supporting discontinuous shear walls shall be evaluated and rehabilitated to comply with the requirements of following Sections.

The analytical model for a shear wall element shall represent the stiffness, strength, and deformation capacity of the shear wall. Potential failure in flexure, shear, and reinforcement development at any point in the shear wall shall be considered. Interaction with other structural and nonstructural elements shall be included.

Slender shear walls and wall segments shall be permitted to be modeled as equivalent beam-column elements that include both flexural and shear deformations. The flexural strength of beam-column elements shall include the interaction of axial load and bending. The rigid-connection zone at beam connections to this equivalent beam-column element shall represent the distance from the wall centroid to the edge of the wall. Unsymmetrical wall sections shall model the different bending capacities for the two loading directions.

A beam element that incorporates both bending and shear deformations shall be used to model coupling beams. The element inelastic response shall account for the loss of shear strength and stiffness during reversed cyclic loading to large deformations. For coupling beams that have diagonal reinforcement satisfying ACI 318, a beam element representing flexure only shall be permitted.

The effective stiffness of all the elements shall be defined based on the material properties, component dimensions, reinforcement quantities, boundary conditions, and current state of the member with respect to cracking and stress levels. Alternatively, use of values for effective stiffness given shall be permitted. To obtain a proper distribution of lateral forces in bearing wall buildings, all of the walls shall be assumed to be either cracked or uncracked. In buildings where lateral load resistance is provided by either structural walls only, or a combination of walls and frame members, all shear walls and wall segments discussed in this section shall be considered to be cracked.

For coupling beams, the effective stiffness values for non-prestressed beams shall be used unless alternative stiffnesses are determined by more detailed analysis. The effective stiffness of columns supporting discontinuous shear walls shall change between the values given for columns in tension and compression, depending on the direction of the lateral load being resisted by the shear wall.

#### **2.8.13.1 Nonlinear static procedure**

Nonlinear load-deformation relations for use in analysis by nonlinear static and dynamic procedures shall comply with the requirements. Monotonic load-deformation relationships for analytical models that represent shear walls, wall elements, coupling beams, and columns that support discontinuous shear walls shall be in accordance with the generalized relation shown in Figure 2.3.

For shear walls and wall segments having inelastic behavior under lateral loading that is governed by flexure, as well as columns supporting discontinuous shear walls, the following approach shall be permitted. The load-deformation relationship in Figure 2.3 shall be used with the x-axis of Figure 2.3 taken as the rotation over the plastic hinging region at the end of the member shown in Figure 2.7. The hinge rotation at point B in Figure 2.3 corresponds to the yield point,  $\theta_y$ , and shall be calculated in accordance with Equation (2.20):

$$\theta_y = \left( \frac{M_y}{E_c I} \right) l_p \quad 2.20$$

where:

$M_y$  = Yield moment capacity of the shear wall or wall segment

$E_c$  = Concrete modulus

$I$  = Member moment of inertia

$l_p$  = Assumed plastic hinge length

For analytical models of shear walls and wall segments, the value of  $l_p$  shall be set equal to 0.5 times the flexural depth of the element, but less than one story height for shear walls and less than 50% of the element length for wall segments. For columns

supporting discontinuous shear walls,  $l_p$  shall be set equal to 0.5 times the flexural depth of the component.

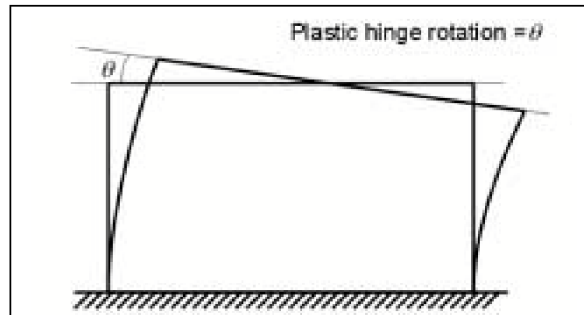


**Figure 2.7 :** Plastic hinge rotation in shear wall where flexure dominates inelastic response

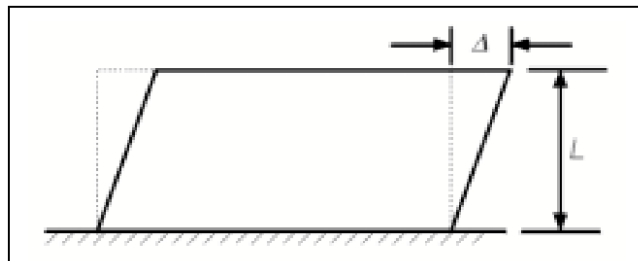


**Figure 2.8 :** Story drift in shear wall where shear dominates inelastic response

Values for the variables a, b, and c required to define the location of points C, D, and E in Figure 2.3(a), shall be as specified in Table 2.28.

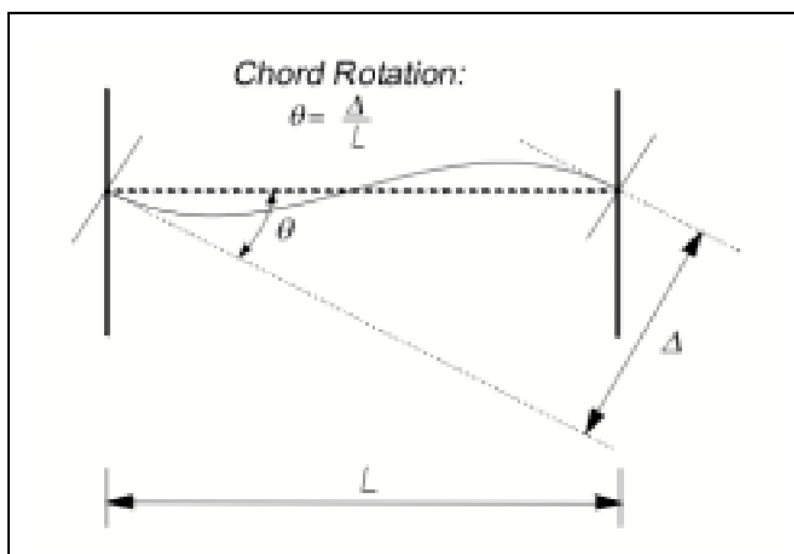


**Figure 2.9 :** Chord rotation for shear wall coupling beams

For shear walls and wall segments whose inelastic response is controlled by shear, the following approach shall be permitted. The load-deformation relationship in 2.3(b) shall be used, with the x-axis of Figure 2.3(b) taken as lateral drift. For shear walls, this drift shall be the story drift as shown in Figure 2.8. For wall segments, Figure 2.8 shall represent the member drift. Permitted. The load-deformation relationship in Figure 2.3(b) shall be used, with the x-axis of Figure 2.3(b) taken as the chord rotation as defined in Figure 2.9. Values for the variables  $d$ ,  $e$ , and  $c$  required to find the points C, D, and E in Figure 2.3(b), shall be as specified in Table 2.29 for the appropriate members. Linear interpolation between tabulated values shall be used if the member under analysis has conditions that are between the limits given in the tables.

#### **2.8.13.2 Nonlinear dynamic procedure**

For the NDP, the complete hysteretic behavior of each component shall be modeled using properties verified by experimental evidence. Use of the generalized load-deformation relation shown in Figure 2.3 to represent the envelope relation for the analysis shall be permitted. The unloading and reloading stiffnesses and strengths, and any pinching of the load-versus-rotation hysteresis loops, shall reflect the behavior experimentally observed for wall elements similar to the one under investigation.

#### **2.8.13.3 Acceptance criteria**

Shear walls, wall segments, coupling beams, and columns supporting discontinuous shear walls shall be classified as either deformation- or force-controlled. For columns supporting discontinuous shear walls, deformation-controlled actions shall be restricted to flexure. In other components, deformation-controlled actions shall be restricted to flexure or shear. All other actions shall be defined as being force-controlled actions.

The nominal flexural strength of a shear wall or wall segment shall be used to determine the maximum shear force in shear walls, wall segments, and columns supporting discontinuous shear walls. For cantilever shear walls and columns supporting discontinuous shear walls, the design shear force shall be equal to the magnitude of the lateral force required to develop the nominal flexural strength at the base of the wall, assuming the lateral force is distributed uniformly over the height of the wall. For wall segments, the design force shall be equal to the shear corresponding

to the development of the positive and negative nominal moment strengths at opposite ends of the wall segment.

Design actions (flexure, shear, or force transfer at rebar anchorages and splices) on components shall be determined as prescribed in Analysis procedure. When determining the appropriate value for the design actions, proper consideration shall be given to gravity loads and to the maximum forces that can be transmitted considering nonlinear action in adjacent components. Design actions shall be compared with design strengths. Table 2.30 and 2.31 specify  $m$  values. Alternate  $m$  values shall be permitted where justified by experimental evidence and analysis.

#### **2.8.13.4 Nonlinear static and dynamic procedures**

In the design model, inelastic response shall be restricted to those elements and actions listed in Tables 2.28 and 2.29, except where it is demonstrated that other inelastic actions are justified for the selected performance levels. For members experiencing inelastic behavior, the magnitude of other actions (forces, moments, or torque) in the member shall correspond to the magnitude of the action causing inelastic behavior. The magnitude of these other actions shall be shown to be below their nominal capacities.

Components experiencing inelastic response shall satisfy the requirements and the maximum plastic hinge rotations, drifts, or chord rotation angles shall not exceed the values given in Tables 2.28 and 2.29, for the selected performance level. Linear interpolation between tabulated values shall be used if the member under analysis has conditions that are between the limits given in the tables.

#### **2.8.13.5 Rehabilitation measures**

Reinforced shear walls, wall segments, coupling beams, and columns supporting discontinuous shear walls that do not meet the acceptance criteria for the selected Rehabilitation Objective shall be rehabilitated. Rehabilitation measures shall meet the requirements.

**Table 2.10 : Modelling parameters and numerical acceptance criteria for nonlinear procedure members controlled by flexure**

| Conditions                                                                          | Plastic Hinge Rotation (radians)    |                                                           | Residual Strength Ratio | Acceptable Plastic Hinge Rotation (radians) |                |        |       |       |       |       |
|-------------------------------------------------------------------------------------|-------------------------------------|-----------------------------------------------------------|-------------------------|---------------------------------------------|----------------|--------|-------|-------|-------|-------|
|                                                                                     |                                     |                                                           |                         | Performance Level                           |                |        |       |       |       |       |
|                                                                                     |                                     |                                                           |                         | IO                                          | Component Type |        |       |       |       |       |
|                                                                                     | Primary                             |                                                           | Secondary               |                                             |                |        |       |       |       |       |
|                                                                                     | a                                   | b                                                         | c                       | LS                                          | CP             | LS     | CP    | LS    | CP    |       |
| i.Shear walls and wall segments                                                     |                                     |                                                           |                         |                                             |                |        |       |       |       |       |
| $\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'}$                                          | $\frac{Shear}{t_w l_w \sqrt{f_c'}}$ | Confined Boundary                                         |                         |                                             |                |        |       |       |       |       |
| $\leq 0.1$                                                                          | $\leq 3$                            | Yes                                                       | 0.015                   | 0.02                                        | 0.75           | 0.005  | 0.01  | 0.015 | 0.015 | 0.02  |
| $\leq 0.1$                                                                          | $\geq 6$                            | Yes                                                       | 0.01                    | 0.015                                       | 0.4            | 0.004  | 0.008 | 0.01  | 0.01  | 0.015 |
| $\geq 0.25$                                                                         | $\leq 3$                            | Yes                                                       | 0.009                   | 0.012                                       | 0.6            | 0.003  | 0.006 | 0.009 | 0.009 | 0.012 |
| $\geq 0.25$                                                                         | $\geq 6$                            | Yes                                                       | 0.005                   | 0.01                                        | 0.3            | 0.0015 | 0.003 | 0.005 | 0.005 | 0.01  |
| $\leq 0.1$                                                                          | $\leq 3$                            | No                                                        | 0.008                   | 0.015                                       | 0.6            | 0.002  | 0.004 | 0.008 | 0.008 | 0.015 |
| $\leq 0.1$                                                                          | $\geq 6$                            | No                                                        | 0.006                   | 0.01                                        | 0.3            | 0.002  | 0.004 | 0.006 | 0.006 | 0.01  |
| $\geq 0.25$                                                                         | $\leq 3$                            | No                                                        | 0.003                   | 0.005                                       | 0.25           | 0.001  | 0.002 | 0.003 | 0.003 | 0.005 |
| $\geq 0.25$                                                                         | $\geq 6$                            | No                                                        | 0.002                   | 0.004                                       | 0.2            | 0.001  | 0.001 | 0.002 | 0.002 | 0.004 |
| ii.Columns supporting discontinus shear walls                                       |                                     |                                                           |                         |                                             |                |        |       |       |       |       |
| Transverse reinforcement                                                            |                                     |                                                           |                         |                                             |                |        |       |       |       |       |
| Conforming                                                                          |                                     |                                                           | 0.01                    | 0.015                                       | 0.2            | 0.003  | 0.007 | 0.01  | n.a.  | n.a.  |
| Nonconforming                                                                       |                                     |                                                           | 0                       | 0                                           | 0              | 0      | 0     | 0     | n.a.  | n.a.  |
| iii.Shear wall coupling beams                                                       |                                     |                                                           |                         |                                             |                |        |       |       |       |       |
| Longitudinal reinforcement and transverse reinforcement3                            |                                     | -----S-----h-----e-----a-----<br>r-----<br>$t_w l_w f_c'$ |                         |                                             |                |        |       |       |       |       |
| Conventional longitudinal reinforcement with conforming transverse reinforcement    |                                     | $\leq 3$                                                  | 0.025                   | 0.05                                        | 0.75           | 0.01   | 0.02  | 0.025 | 0.025 | 0.05  |
|                                                                                     |                                     | $\geq 6$                                                  | 0.02                    | 0.04                                        | 0.5            | 0.005  | 0.01  | 0.02  | 0.02  | 0.04  |
| Conventional longitudinal reinforcement with nonconforming transverse reinforcement |                                     | $\leq 3$                                                  | 0.02                    | 0.035                                       | 0.5            | 0.006  | 0.012 | 0.02  | 0.02  | 0.035 |
|                                                                                     |                                     | $\geq 6$                                                  | 0.01                    | 0.025                                       | 0.25           | 0.005  | 0.008 | 0.01  | 0.01  | 0.025 |
| Diagonal reinforcement                                                              |                                     | n.a.                                                      | 0.03                    | 0.05                                        | 0.8            | 0.006  | 0.018 | 0.03  | 0.03  | 0.05  |

**Table 2.11 :** Modelling parameters and numerical acceptance criteria for nonlinear procedure members controlled by shear

| Conditions                                                                          | Total Drift Ratio (%), or Chord Rotation (radians)1 |          | Residual Strength Ratio | Acceptable Total Drift (%) or Chord Rotation (radians) |                |       |           |       |       |
|-------------------------------------------------------------------------------------|-----------------------------------------------------|----------|-------------------------|--------------------------------------------------------|----------------|-------|-----------|-------|-------|
|                                                                                     |                                                     |          |                         | Performance Level                                      |                |       |           |       |       |
|                                                                                     |                                                     |          |                         | IO                                                     | Component Type |       |           |       |       |
|                                                                                     |                                                     |          |                         |                                                        | Primary        |       | Secondary |       |       |
|                                                                                     | <i>d</i>                                            | <i>e</i> | <i>c</i>                | LS                                                     | CP             | LS    | CP        |       |       |
| i. Shear walls and wall segments                                                    |                                                     |          |                         |                                                        |                |       |           |       |       |
| All shear walls and wall segments                                                   | 0.75                                                | 2        | 0.4                     | 0.4                                                    | 0.6            | 0.75  | 0.75      | 1.5   |       |
| ii. Shear wall coupling beams                                                       |                                                     |          |                         |                                                        |                |       |           |       |       |
| Longitudinal reinforcement and transverse reinforcement                             | $\frac{Shear}{t_w l_w \sqrt{f_c'}}$                 |          |                         |                                                        |                |       |           |       |       |
| Conventional longitudinal reinforcement with conforming transverse reinforcement    | ≤ 3                                                 | 0.002    | 0.03                    | 0.6                                                    | 0.006          | 0.015 | 0.020     | 0.020 | 0.030 |
|                                                                                     | ≥ 6                                                 | 0.016    | 0.024                   | 0.3                                                    | 0.005          | 0.012 | 0.016     | 0.016 | 0.024 |
| Conventional longitudinal reinforcement with nonconforming transverse reinforcement | ≤ 3                                                 | 0.012    | 0.025                   | 0.4                                                    | 0.006          | 0.008 | 0.010     | 0.010 | 0.020 |
|                                                                                     | ≥ 6                                                 | 0.008    | 0.014                   | 0.2                                                    | 0.004          | 0.006 | 0.007     | 0.007 | 0.012 |

## 2.9 General Requirements According to Turkish Earthquake Code

Rules of calculation to be used in the assessment of performances of the existing buildings and building - type structures in earthquake zones under the impact of an earthquake, principles to be followed in decisions of strengthening, and principles of design for strengthening for buildings that decision of strengthening is made are defined in this section.

Rules given in this section do not apply to non-building-type structure. Furthermore, assessment and strengthening of registered buildings with historical and cultural value and monuments are outside the scope of this Regulation.

## **2.9.1 Data collection from building**

### **2.9.1.1 Limited information level in reinforced concrete buildings**

The measured drawings of the supporting system shall be prepared with field work. In case the architectural projects are present, these can be used as aids to the works for preparation of measured drawings. Information obtained must include the locations, axis openings, heights and dimensions of all the reinforced concrete elements and nonbearing walls, and must be sufficient for the creation of a calculation model for the building. Foundation system shall be determined by digging examination holes of sufficient numbers in- or outside the building. Short columns or similar irregularities in the building shall be entered on the floor plan and sections. Relation of the building with the neighboring buildings (separated, adjoining, jointing present / absent) shall be determined.

Projects or application drawings are not present. It is assumed that the amount of reinforcement in the reinforcement elements and details meet the minimum requirements for reinforcement for the date that the building was constructed. With the purpose of confirming this assumption, or to determine to what extent it is true, reinforcements and lengths of reinforcement laps shall be determined by scraping off the concrete covers of 10 % of bulkheads and columns and 5 % of beams, and at least one in each floor. Such scraping must be performed on the one-thirds of the lengths the columns and beams in the middle of the opening; however, same must be performed on the overlapping parts of at least three columns with the purpose of determining the length of overlapping. Scraped surfaces shall be covered afterwards with high-strength repair mortar. In addition, number of longitudinal and transverse reinforcement elements of 20 % of the elements that have not been scraped shall be determined using devices for reinforcement determining devices. The coefficient of actual reinforcement expressing the ratio of the amount of reinforcement the actually found in reinforced columns and beams to the minimum reinforcement shall be separately determined for columns and beams. This coefficient shall be applied to all the other elements that reinforcement has not been determined, and the possible amount of reinforcement shall thus be determined.

At least two samples of concrete (borehole sample) shall be collected from columns or bulkheads in each floor according to the conditions stated in TS-10465, and tests shall

be performed to determine the lowest pressure strength to be considered as the existing concrete strength. Reinforcement class shall be determined according to the visual examination as explained in the paragraph above, and the characteristic yield strength of the steel in this class shall be taken as the existing steel strength. In this examination, elements showing corrosion in reinforcement shall be marked on the plan, and this condition shall be taken into consideration in element capacity calculations.

#### **2.9.1.2 Medium level information in reinforced concrete buildings**

In case the reinforcement projects for the building are present, compliance of the geometry to the project shall be checked. In case there are no projects, then measured drawings shall be prepared by field work. Information obtained must include the locations in each floor, openings, heights and dimensions of all the reinforced concrete elements and nonbearing walls, and must be sufficient for the creation of a calculation model for the building. Information regarding the geometry of the building must include the details required for a precise description of the mass of the building. Short columns or similar irregularities in the building shall be entered on the floor plan and sections. Relation of the building with the neighboring buildings (separated, adjoining, jointing present/absent) shall be determined. Foundation system shall be determined by digging examination holes of sufficient numbers in- or outside the building.

In case project or manufacturing drawings are not present, then conditions in 7.2.4.2 apply; however, number of bulkheads, columns and beams that concrete covers shall be scraped off for the checking of reinforcement shall not be less than 20 % of the total number of columns in a certain floor, and 10 % of the number of beams, and at least two in each floor. In case project or manufacturing drawings are present, then conditions stated in 7.2.4.2 shall be applied with the numbers stated therein. In addition, number of longitudinal and transverse reinforcement elements of 20 % of the elements that have not been scraped shall be determined using devices for reinforcement-determining devices. The coefficient of actual reinforcement expressing the ratio of the amount of reinforcement the actually found in reinforced columns and beams to the minimum reinforcement shall be separately determined for columns and beams. This coefficient used to determine the capacities of elements cannot exceed 1. This coefficient shall be applied to all the other elements that

reinforcement has not been determined, and the possible amount of reinforcement shall thus be determined.

One concrete sample (borehole sample) shall be collected from columns or bulkheads in each floor not less than 3 in number, and also not less than 9 from the entire building and one sample from each 400 square meters for the tests to be performed according to the conditions stated in TS - 10465. The mean – standard deviation values found for the samples shall be taken as the existing concrete strength when calculating the capacities of the elements. Distribution of the concrete strength throughout the building can be checked with the readings of adjusted concrete test hammer to the borehole sample test results or similar undamaged examination tools. Reinforcement class shall be determined by visual examination on surfaces scraped as described in the paragraph above, and the characteristic strength of the steel of this class shall be taken as the existing steel strength in capacity calculations. In this examination, elements showing corrosion in reinforcement shall be marked on the plan, and this condition shall be taken into consideration in element capacity calculations.

#### **2.9.1.3 Comprehensive information level in reinforced concrete buildings**

Reinforcement projects for the building are present. Compliance with the projects of the actual geometry is checked with the measurements performed in the building. In case the projects show important conflicts with the measurements, then the projects are ignored, and the building is examined according to the rules for medium level information. Short columns or similar irregularities in the building shall be entered on the floor plan and sections. Relation of the building with the neighboring buildings (separated, adjoining, jointing present/absent) shall be determined. Information regarding the geometry of the building must include the required details for a precise description of the mass of the building. Foundation system shall be determined by digging examination holes of sufficient numbers in- or outside the building.

Details of elements: Detail projects of reinforcement of the building are present. Procedures stated in 7.2.4.2 for checking of the compliance of reinforcement with the project shall be applied on the same number of reinforcement elements. In addition, locations and numbers of longitudinal and transverse reinforcement elements of 20 % of the elements that have not been scraped shall be determined using devices for reinforcement determining devices. In case there are any conflicts between the project

and the application, then coefficient of actual reinforcement expressing the ratio of the amount of reinforcement the actually found in reinforced columns and beams to the minimum reinforcement shall be separately determined for columns and beams. This coefficient used to determine the capacities of elements cannot exceed 1. This coefficient shall be applied to all the other elements that reinforcement has not been determined, and the possible amount of reinforcement shall thus be determined.

One concrete sample (borehole sample) shall be collected from columns or bulkheads in each floor not less than 3 in number, and also not less than 9 from the entire building and one sample from each 200 square meters for the tests to be performed according to the conditions stated in TS - 10465. The mean – standard deviation values found for the samples shall be taken as the existing concrete strength when calculating the capacities of the elements. Distribution of the concrete strength throughout the building can be checked with the readings of adjusted concrete test hammer to the borehole sample test results or similar undamaged examination tools. Reinforcement class shall be determined on surfaces scraped as described in the paragraph above, and tests shall be performed on one sample for each steel class (S220, S420, etc.), and yield strength and breaking strength, and deformation characteristics of the steel shall determine and compliance with the project shall be determined. If it complies with the project, the characteristic strength of the steel used in the project shall be taken as the existing steel strength. If not, tests shall be performed on at least three more samples, and the least favorable value shall be taken as the existing steel strength in the element capacity calculations. In this examination, elements showing corrosion in reinforcement shall be marked on the plan, and this condition shall be taken into consideration in element capacity calculations.

**Table 2.12 :** Information level coefficients for buildings

| <b>Information level</b> | <b>Information level coefficient</b> |
|--------------------------|--------------------------------------|
| Limited                  | 0.75                                 |
| Medium                   | 0.90                                 |
| Comprehensive            | 1.00                                 |

### **2.9.2 Limits of damage in construction elements and areas of damage**

Elements that the damages with critical sections do not reach MN are within the Minimum Damage Region, those in-between MN and GV are within Marked Damage

Region, those in-between GV and GÇ are in Advanced Damage Region, and those going beyond GÇ are within Collapsing Region (Figure 2.10).

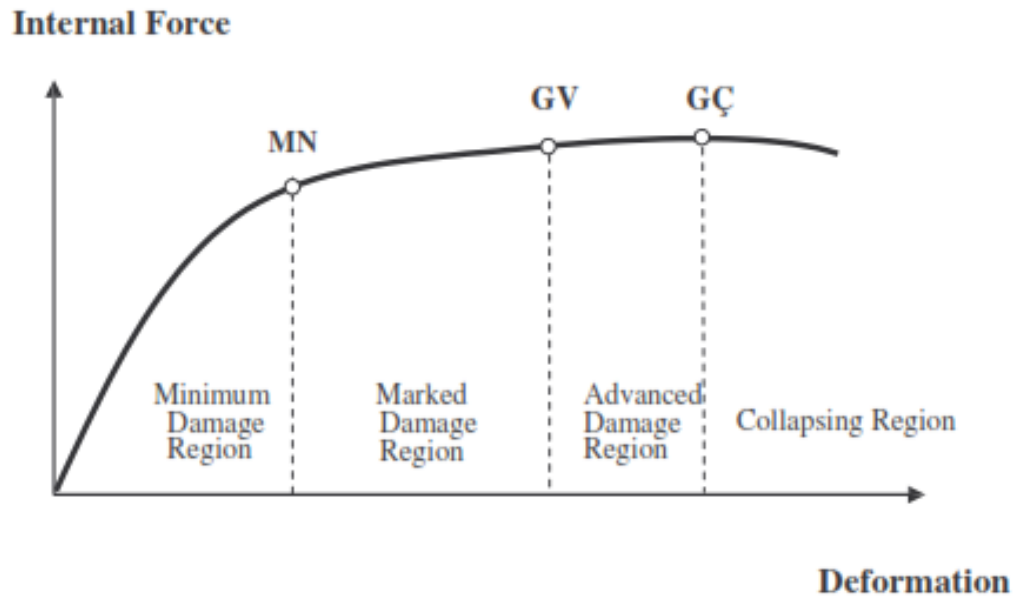


**Figure 2.10 : Damage limits in cross sections**

### 2.9.3 Determining the seismic performance of the building

The aim of the inelastic linear measurement methods to be used for determining the structural performances of the buildings under seismic effect and for the strengthening analyses is enabling the measurement of the plastic deformation volitions regarding the ductile behavior and inner force volitions concerning the brittle behavior for a given earthquake. Afterwards, the magnitudes of the mentioned volitions are compared with the deformation and inner force capacities that are defined in this section and structural performance evaluation shall be conducted both at sectional and building level.

The inelastic linear analysis methods included in the scope of this Regulation are Incremental Equivalence Seismic Load Method, Incremental Mode Combination Method and Measurement within the Scope of Time Definition Method. First two are the methods that shall be used for the Incremental Repulsion Analysis that is taken as a basis for determining the non - linear seismic performances and for the strengthening measurements included in this Regulation.

The steps that shall be taken during the inelastic non - linear performance evaluation conducted applying the Incremental Repulsion Analysis is summarized below.

(a) In addition to the general principles and rules defined in 7.4, the rules defined 7.6.4 shall be followed to idealize the non-linear behavior of the load-bearing system and construct the analysis model.

(b) Before applying the incremental repulsion analysis, a non-linear static analysis in which the gravity loads that are in accordance with the masses are taken into consideration shall be conducted. The results of this analysis shall be considered as the initial conditions of the incremental repulsion analysis.

(c) In case the incremental repulsion analysis is conducted via applying the Incremental Equivalence Seismic Load Method defined in 7.6.5, the “modal capacity diagram” belonging to the primary (dominant) mode the coordinates of which are defined as “modal translocation – modal acceleration” shall be derived. The modal translocation volition belonging to the primary (dominant) mode shall be set taking the elastic behaviors spectrum defined in 2.4 and the modifications applied on this spectrum in 7.8 for different exceeding probabilities together with the mentioned diagram into consideration. In the final step, the translocation, plastic deformation (plastic rotation) and inner force volitions that corresponds to the modal translocation volition shall be calculated.

(d) In case the incremental repulsion analysis is conducted via applying the Incremental Mode Combination Method defined in 7.6.6, modal translocation volitions will as well be derived together with the “modal capacity diagrams” for all modes taken into consideration and the translocation, plastic deformation (plastic rotation) and inner force volitions that will arise accordingly in the load-bearing system shall be calculated.

(e) The plastic rotation volitions that are calculated out of ductile sections shall be utilized for deriving plastic bending volitions and total bending volitions in accordance with 7.6.8. Afterwards, the unit deformation volitions of reinforced concrete sections, concretes and reinforcement steels shall be calculated accordingly. Performance evaluations regarding the ductile behavior in sectional level shall be conducted via comparing the mentioned values of volition with the unit deformation capacities defined in 7.6.9 for various damage thresholds. Moreover, the deformation volitions

calculated in terms of relative story drifts of the strengthened filled wall shall be compared with the deformation capacities defined in 7.6.10. The shear force volitions derived from the analysis on the other hand shall be compared with the capacities defined in 7.6.11 and performance evaluation regarding the brittle behavior shall be conducted at sectional level.

#### **2.9.4 Idealizing the Non - linear Behavior**

In order to idealize the inelastic non-linear behavior in respect to the materials, the models that proved valid in the literature can be utilized. However, due to its practicability and extensiveness in engineering practices, stacked plastic behavior model taken as a basis point for the inelastic non-linear analysis applied in the following sections. In a model that corresponds to the plastic support hypothesis in case of simple bending, it is assumed that the plastic deformations are formed in an evenly distributed manner all along the finite length zones in which the inner forces in the beam, column and frame-type load-bearing components idealized as stick components reach to their plastic capacities. The length of the plastic deformation zone referred to as plastic support length ( $L_p$ ) shall be taken as the half of the section length ( $h$ ) in the active direction ( $L_p = 0.5 h$ ). For the frames where  $H_w / w \leq 2.0$ , the plastic deformations under the bending effect will not be taken into consideration.

The length of the plastic deformation zones that undergo plastic deformation only under axial force shall be assumed to be equal to the open length of the related component.

The plastic section that represents the stacked plastic deformation should be positioned in the middle of the plastic deformation zone that is theoretically defined in 7.6.4.1. However, the approximate idealizations mentioned below can be allowed for practical applications.

(a) In columns and beams, the plastic sections can be placed right out of the column-beam confinement zone, in other words on the end zones of the net parts of columns or beams. However, the fact that plastic hinges may be formed as well in the beam bays due to the effects of the gravity loads should be taken into consideration.

(b) In reinforced concrete frames, plastic sections may be allowed to be placed on the bottom end zone of the frame zone for each floor. U, T, L or box-section frames should be idealized as a single frame all arms of which works cooperatively. In case there are rigid frames in the basement floors of the buildings, the plastic sections of this frames following on to the upper stairs of the building should be placed starting above from the basement.

The interaction diagrams that are defined in accordance with the rules defined in 7.4.11 shall be taken as the yield surfaces for the reinforced concrete sections that are plasticized under the effect of uniaxial or biaxial bending and axial force.

The information in the following paragraphs shall be considered regarding the inner force – plastic deformation correlations that shall be used in repulsion analysis model.

(a) The strain hardening effect (rise in the momentum in accordance with the rise in the plastic rotation) in the inner force – plastic deformation correlations can be left approximately (Figure 2.11a).

In such a case, the condition of the plastic deformation vector being approximately perpendicular to the yield surface, provided that the inner forces remain over the yield surface, shall be taken into consideration during the steps following the plasticizing of the sections under the impact of the uniaxial or biaxial bending and axial force.

(b) In case the strain hardening effect is taken into consideration (Figure 2.11b), the requirements the inner forces and the plastic deformation vector should satisfy during the repulsion steps following the plasticization in the sections under the effect of uniaxial or biaxial bending and axial force shall be defined in accordance with an appropriate strain hardening model to be obtained from the related literature.

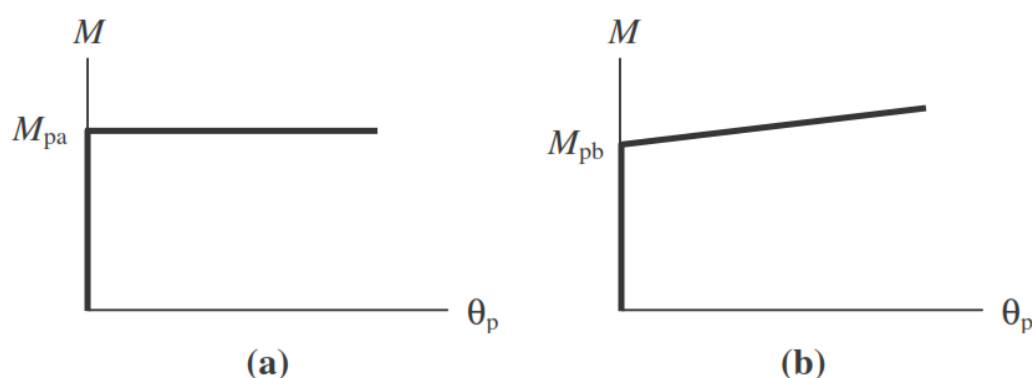


**Figure 2.11 : The strain hardening effect**

### 2.9.5 Section unit deformation capacities of reinforced concrete components

The seismic volitions obtained in accordance with 7.6.8 in terms of unit deformation volitions for concrete or reinforcement steel shall be compared with the unit deformation capacities given below so as to determine the performance of the load-bearing systems at sectional level.

The upper bounds (capacity) of deformation for different sectional damage thresholds for the ductile load-bearing system components that undergo plastic deformations are defined below:

The transverse reinforcements that consider the following:

(a) For Minimum Sectional Damage Boundary (MN), upper bounds of the concrete unit pressure deformation in the outmost fiber of the section and the reinforcement steel unit deformation volitions:

$$(\varepsilon_{cu})_{MN} = 0.0035 \quad ; \quad (\varepsilon_s)_{MN} = 0.010$$

(b) For Section Security Bound (GV), upper bounds of the concrete unit pressure deformation in the outmost fiber of hoop and the reinforcement steel unit deformation volitions:

$$(\varepsilon_{cg})_{GV} = 0.0035 + 0.01(\rho_s / \rho_{sm}) \leq 0.0135 \quad ; \quad (\varepsilon_s)_{GV} = 0.040$$

(c) 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 volitions:

$$(\varepsilon_{cg})_{GC} = 0.0004 + 0.013(\rho_s / \rho_{sm}) \leq 0.018 \quad ; \quad (\varepsilon_s)_{GC} = 0.060$$

Should be regulated as “special earthquake hoops and crossties”.

### 2.9.6 Determining the seismic performance of the building

The seismic performance of the buildings are related to the condition of the damages that are expected to come out under the effect of the earthquakes applied and is defined taking four different damage levels as basis. The seismic performance of the buildings are determined applying the calculation methods defined in 7.5 and 7.6 and deciding on the damage areas of the components. The rules to be applied so as to determine the

seismic performance of the buildings are given below. The rules given here are related to the reinforced concrete and prefabricated buildings. The rules to be applied for masonry buildings are given in 7.7.6.

#### **2.9.6.1 Ready for use performance level**

Onto the results of the calculations regarding all earthquakes applied in any floors, at most 10 % of the beams exceed the Significant Damage Zone and all other load-bearing components remain in the Minimum Damage Zone. Such buildings can be agreed to be in the Ready for Use Performance Level provided that the brittle damaged components, if any, are strengthened.

#### **2.9.6.2 Life safety performance level**

The buildings that satisfy the conditions mentioned below can be agreed to be in Life Safety Performance Level provided that the brittle damaged components, if any, are strengthened:

(a) As the result of the calculations made for each earthquake direction applies on each floor, at most 30 % of the beams except for the secondary ones (that does not take place in the horizontal load-bearing system) and at most the proportion of the columns defined in paragraph (b) can exceed the Advanced Damage Zone.

(b) The total contribution of the columns in the Advanced Damage Zone to the shear force that is borne by the columns in each floor should not exceed 20 %. For the top floor, the ratio of the total shear forces of the columns in the Significant Damage Zone to the total shear forces of all the columns at that floor can be at most 40 %.

(c) All other load - bearing components are in Minimum Damage Zone or Significant Damage Zone. However, the shear forces borne by the columns which exceeds the Minimum Damage Bound both in upper and lower sections for any floor should not be more than 30 % of the shear force borne by all columns of the floor (In calculation with linear elastic method, the columns that satisfy Equation (3.3) both in upper and lower confinement points are not included in this calculation).

#### **2.9.6.3 Pre - Collapse Performance Level**

The buildings that meet the conditions given below are agreed to be in the Pre - Collapse Performance Level provided that the fact that all components that are brittle damaged are in the Collapse Zone.

(a) Onto the results of the calculations regarding all earthquakes applied in any floors, at most 20 % of the beams except for the secondary ones (that does not take place in the horizontal load-bearing system) can enter the Collapse Zone.

(b) All other load-bearing components are in Minimum Damage Zone, Significant Damage Zone or in the Advanced Damage Zone. However, the shear forces borne by the columns which exceeds the Minimum Damage Bound both in upper and lower sections for any floor should not be more than 30 % of the shear force borne by all columns of the floor (In calculation with linear elastic method, the columns that satisfy Equation (3.3) both in upper and lower confinement points are not included in this calculation).

(c) Usage of the building under these circumstances poses threats towards life safety.

#### **2.9.6.4 Collapse Level**

If the building does not satisfy the Pre - Collapse Performance Level, it is in the Collapse Level. Usage of the building under these circumstances poses threats towards life safety.



### 3. SEISMIC PERFORMANCE ASSESSMENT OF A REINFORCED CONCRETE BUILDING

In this chapter seismic assessment of a reinforced concrete moment frame building has considered in both linear and nonlinear (static and dynamic) method and results will compared by each procedure due to FEMA356 and CODE 2800 or IR360 standards to obtain the most realistic performance of the structure.

#### 3.1 Building Structural Properties

The structure of building under consideration is a 8 story concrete moment frame with two-way slabs in floors, the properties of structure such as materials, sections, and etc. are as following sections.

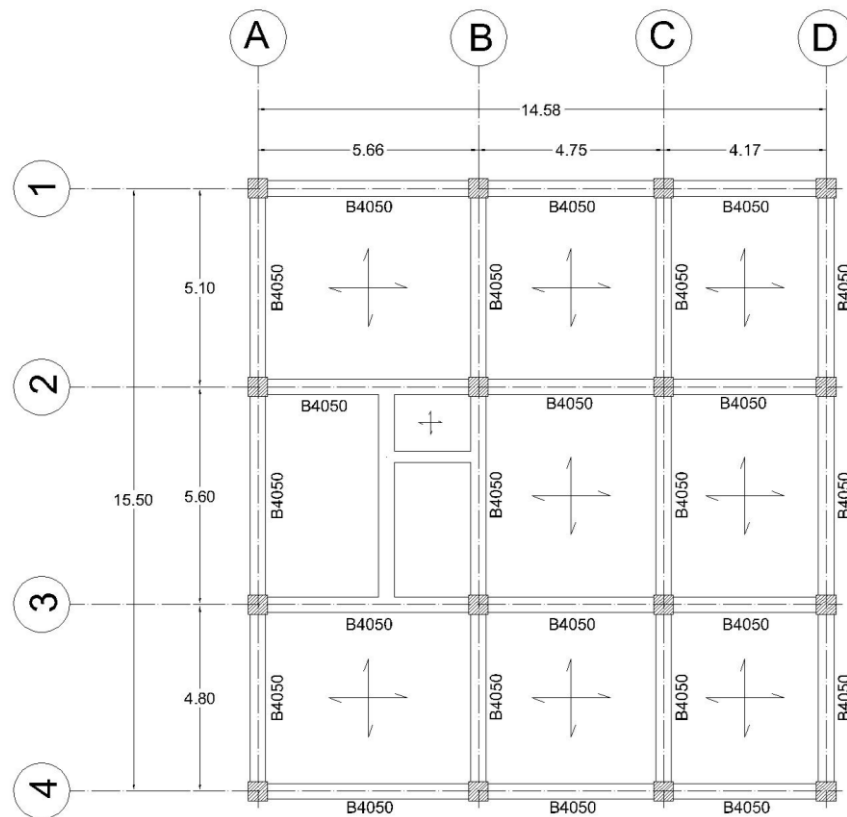


**Figure 3.1 :** Structural layout of the building

### 3.1.1 Material properties

Concrete: C25 concrete with a compressive strength of  $25 \text{ N / mm}^2$ , weight per unit volume of  $25 \text{ KN / m}^3$

Reinforcement : AIII deformed reinforcement with a yielding stress of  $460 \text{ N / mm}^2$ , ultimate stress of  $620 \text{ N / mm}^2$ , and weight per unit volume value of  $76.9 \text{ KN / m}^3$ .

### 3.1.2 Frame section preoperties of structure

Section of beams and columns are as following figures

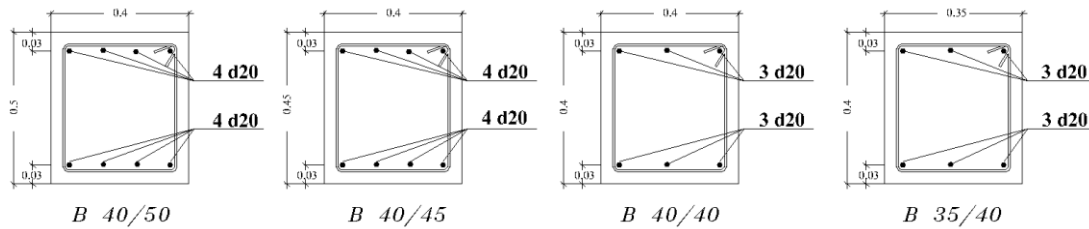


**Figure 3.2 : Beam sections**

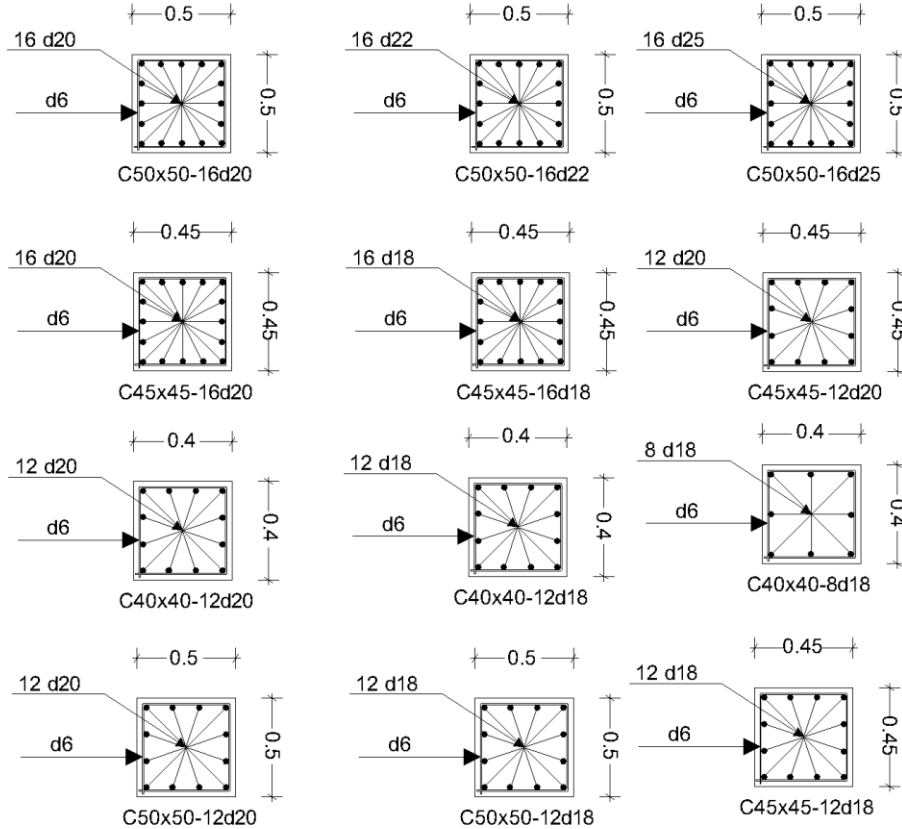


**Figure 3.3 : Column sections**

### 3.2 Geometry of structure

Geometry of the building under consideration is as follows:

Number of story: 8

Story height: ground floor 2.8m, other stories 3.2m

### 3.3 Building classification according to configuration

Buildings are classified as “Regular” or “Irregular”, based on their configuration as follows:

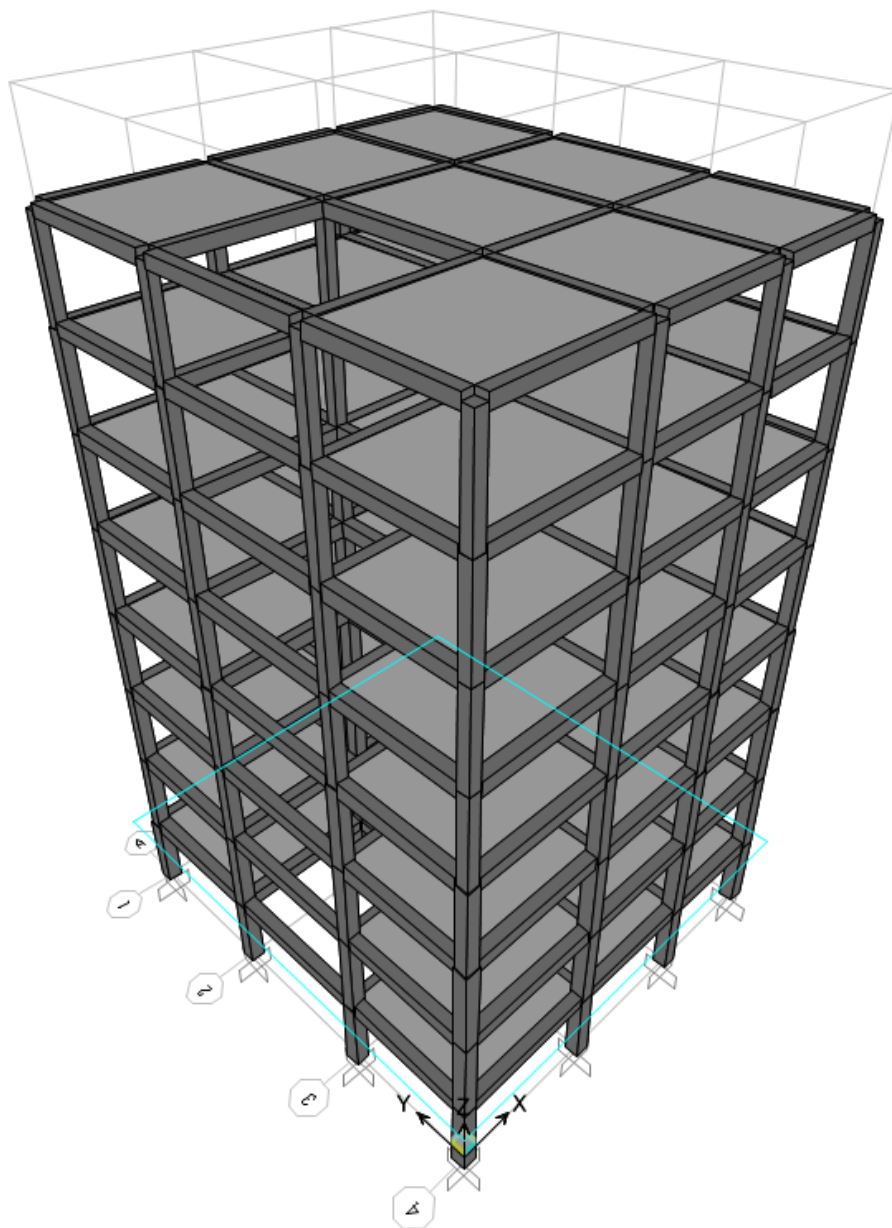


**Figure 3.4 : 3D View of structural system**

### 3.3.1 Regular buildings:

Buildings are “Regular” if they conform to all of the following criteria:

#### 3.3.1.1 Plan regularity:

1. The plan of the building shall be symmetric or almost symmetric about its principle axes, where the lateral load resisting elements are generally aligned with. In case there is any setback or projection, their dimension in each direction shall not exceed 25% of the respective building dimension in that direction.

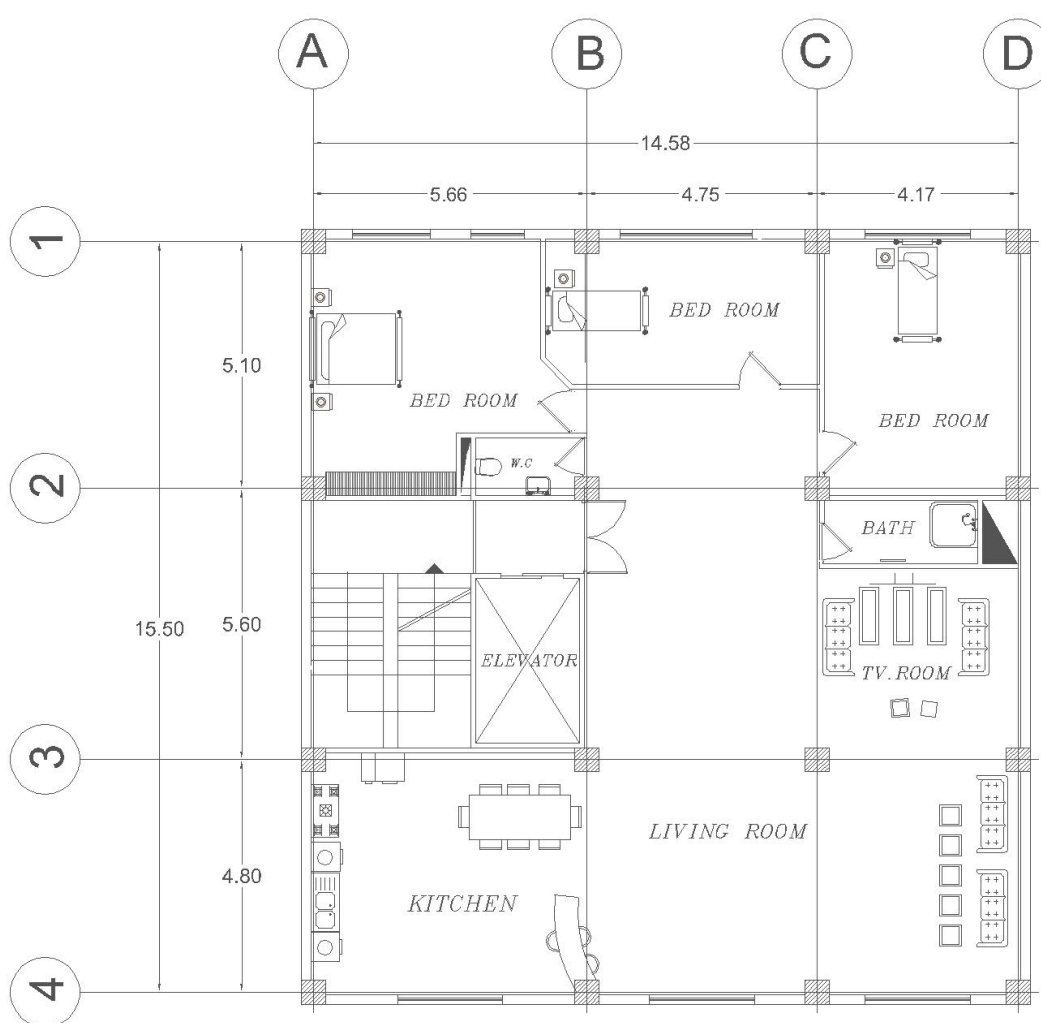


**Figure 3.5 :** Architectural layout of the building

2. In each story, the distance between the centers of mass and stiffness in each orthogonal direction shall not exceed 20% of the building dimension in that direction.

3. Abrupt variation in diaphragm stiffness relative to the adjacent stories shall not exceed 50%. Moreover, the total area of openings in each diaphragm shall not be greater than 50% of the total area of the diaphragm.
4. There shall be no discontinuity in the lateral load path toward the base, such as out-of-plane offsets of the lateral load resisting elements.
5. In each story the maximum drift, including accidental torsion, at one end of the structure shall not exceed 20% of the average of the story drifts of the two ends of the structure.

#### **3.3.1.2 Vertical irregularity:**

1. Distribution of mass over the height of the building shall be approximately uniform such that the mass of no story, except the roof and loft differ more than 50% of the mass of the story located below.
2. The lateral stiffness of each story shall not be less than 70% of that in the story above or 80% of the average stiffness of the three stories above. The story not complying with these, is considered a “flexible story”.
3. The lateral strength of each story shall not be less than 80% of that in the story above. The story strength is the total strength of all elements resisting the story shear in the direction under consideration. The story not complying with this, is considered a “weak story”.

#### **3.3.2 Irregular buildings**

An irregular building is a building not conforming to one or more of the provisions of Section 3.2.4.1.

### **3.4 Analysis Procedure**

Structural system of the building is analyzed by both linear and nonlinear (static and dynamic) method:

### 3.4.1 Linear static analysis

Static linear analysis proceed using gravitational and equivalent earthquake loads.

#### 3.4.1.1 Gravitational loads

Gravitational load cases defined and mounted as dead loads and live loads, values of each load given in Table 3.1 and Table 3.2.

**Table 3.1 :** Assigned loads for floors

| Story  | Dead load ( $KN / m^2$ ) | Live load ( $KN / m^2$ ) | Mass ( $KN / m^2$ ) |
|--------|--------------------------|--------------------------|---------------------|
| Story1 | 3.2                      | 2                        | 0                   |
| Story2 | 3.2                      | 2                        | 0                   |
| Story3 | 3.2                      | 2                        | 0                   |
| Story4 | 3.2                      | 2                        | 0                   |
| Story5 | 3.2                      | 2                        | 0                   |
| Story6 | 3.2                      | 2                        | 0                   |
| Story7 | 3.2                      | 2                        | 0                   |
| Story8 | 2.8                      | 2                        | 0.6                 |

**Table 3.2 :** Assigned loads for beam elements

| Story  | Dead load ( $KN / m$ ) | Live load ( $KN / m$ ) | Mass ( $KN / m$ ) |
|--------|------------------------|------------------------|-------------------|
| Story1 | 6                      | 0                      | 0                 |
| Story2 | 6                      | 0                      | 0                 |
| Story3 | 6                      | 0                      | 0                 |
| Story4 | 6                      | 0                      | 0                 |
| Story5 | 6                      | 0                      | 0                 |
| Story6 | 6                      | 0                      | 0                 |
| Story7 | 6                      | 0                      | 0                 |
| Story8 | 2                      | 0                      | 3                 |

### 3.4.2 Equivalent earthquake loads according to CODE 2800

The equivalent static lateral force shall be determined in accordance with the criteria of this section and shall be applied in opposite directions to the structure.

#### 3.4.2.1 Base shear, V

The minimum base shear or the sum of the horizontal seismic loads, in each direction of the structure, shall be determined from:

$$V = CW \quad (3.1)$$

Where:

V= The shear force at base level..

W= The total seismic weight of building, that is equal to the total dead load plus a percentage of live and snow loads.

C= The seismic coefficient, that is determined from the following Equation :

$$C = \frac{ABI}{R} \quad (3.2)$$

Where:

A= Design base acceleration ratio (ratio of seismic acceleration to gravity acceleration,g).

B= Building response factor determined from the design response spectrum,

I=Importance factor

R= Building behavior factor

The design base shear shall not be considered less than:

$$V \geq 0.1AIW \quad (3.3)$$

**Table 3.3 :** Percentage of live and snow loads in the seismic weight of the building

| Location                                                  | Percentage of live load |
|-----------------------------------------------------------|-------------------------|
| Sloped roofs with a slope equal to or greater than 20%    | ----                    |
| Flat roofs or roofs with a slope less than 20%            | 20                      |
| Residential buildings, Offices, hotels and garages        | 20                      |
| Hospitals, schools, shopping centers and public Buildings | 40                      |
| Storages and Libraries                                    | 60                      |
| Water tanks and other liquid storage tanks and silos      | 100                     |

### 3.4.2.2 Design base acceleration ratio, A

The design base acceleration ratio shall be determined for various regions from Table-3.4, according to their level of seismicity.

**Table 3.4 :** Design base acceleration ratio for various seismic zones

| Zone | Description                                   | Design base acceleration(/g) |
|------|-----------------------------------------------|------------------------------|
| 1    | Very high level of relative seismic hazard    | 0.35                         |
| 2    | High level of relative seismic hazard         | 0.30                         |
| 3    | Intermediate level of relative seismic Hazard | 0.25                         |
| 4    | Low level of relative seismic hazard          | 0.20                         |

### 3.4.2.3 Building response factor, B

The building response factor represents the building response to the ground motion. This factor shall be determined from the following Equation or from Figure-3.4 and Figure-3.5:

$$\begin{aligned} B &= S + 1(T/T_0) & 0 \leq T \leq T_0 \\ B &= S + 1 & T_0 \leq T \leq T_s \\ B &= (S+1)(T_s/T)^{2/3} & T \geq T_s \end{aligned} \quad (3.4)$$

Where:

T= Fundamental period of vibration of the structure in the direction under consideration, in second

$T_0$  ,  $T_s$  and S = are parameters determined from the soil profile type and level of seismicity, as set forth in Table-3.5. Various soil types are described.

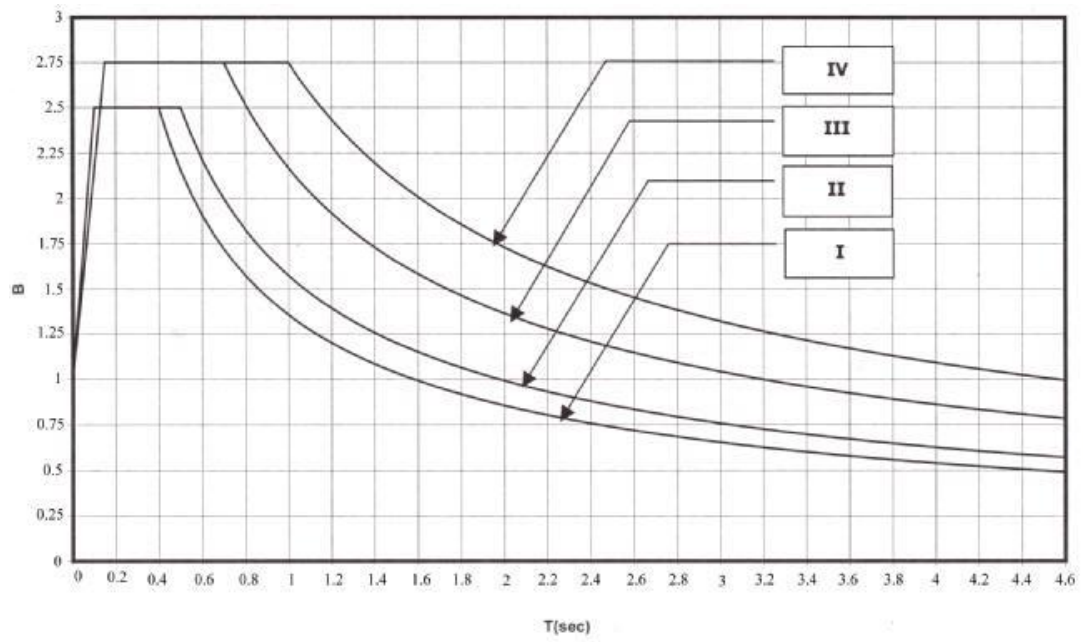


**Figure 3.6 :** Building response factor for zones of “Very High” and “High” levels of seismicity.

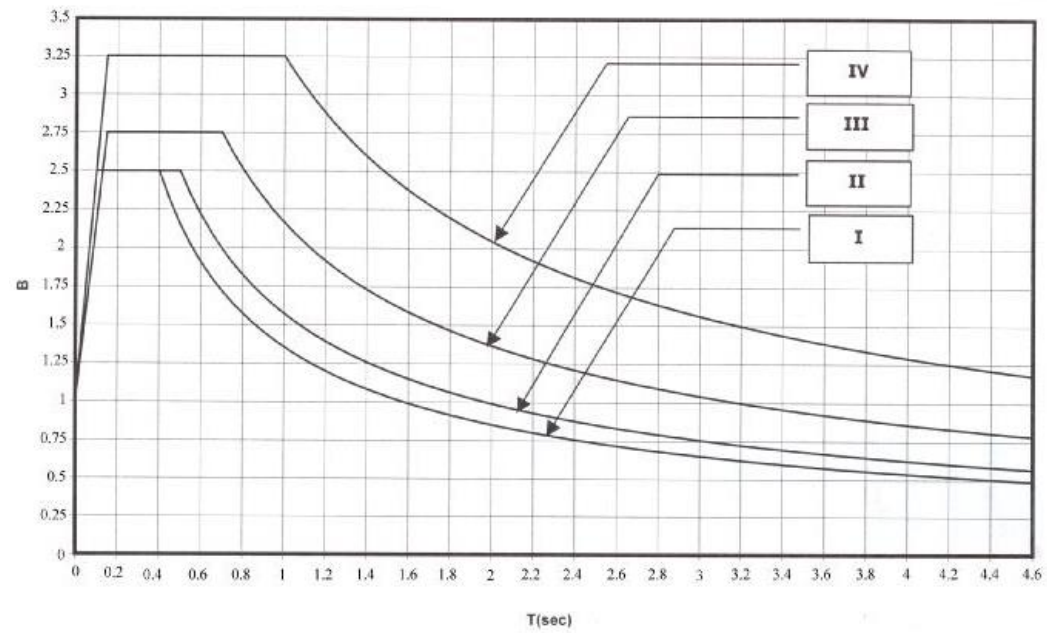


**Figure 3.7 :** Building response factor for zones of “Moderate” and “Low” levels of seismicity.

**Table 3.5 :** values for  $S$ ,  $T_0$  and  $T_s$ 

| Soil profile type | $T_0$ | $T_s$ | “Moderate” and “Low” seismicity | “High” and “Very high” seismicity |
|-------------------|-------|-------|---------------------------------|-----------------------------------|
|                   |       |       | $S$                             | $S$                               |
| I                 | 0.1   | 0.4   | 1.5                             | 1.5                               |
| II                | 0.1   | 0.5   | 1.5                             | 1.5                               |
| III               | 0.15  | 0.7   | 1.75                            | 1.75                              |
| IV                | 0.15  | 1.0   | 2.25                            | 1.75                              |

Soil profile types are classified according to Table 3.6:

**Table 3.6 :** Soil Profile classification

| Soil Profile Type | Description                                                                                                                                                                                                                                                                              | $V_s$ (m/sec)                            |
|-------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------|
| I                 | a) Igneous rocks (with coarse and fine grade texture), stiff sedimentary rocks and massive metamorphic rocks (Gneiss, crystal layers silicate rocks) and conglomerate.<br>b) Stiff soils (compact sand and gravel, very stiff clay) with a thickness more than 30 meters above bed rock. | $> 750$<br>$375 \leq \bar{V}_s \leq 750$ |
| II                | a) Loose igneous rocks (e.g. tuff), loose sedimentary rocks, foliated metamorphic rocks and in general rocks that have become loose and decomposed due to weathering<br>b) Stiff soils (compact sand and gravel, very stiff clay) having a thickness more than 30 meters above bed rock. | $375 \leq \bar{V}_s \leq 750$            |
| III               | a) Rocks that are disintegrated due to weathering<br>b) Soils with medium compaction, layers of sand and gravel with medium intragranular bond and clay with intermediate compaction.                                                                                                    | $175 \leq \bar{V}_s \leq 375$            |
| IV                | a) Soft deposits with high moisture content due to high level of water table<br>b) Any soil profile containing clay with a minimum thickness of 6 meters and a plastic index and moisture content exceeding 20 and 40 percent respectively.                                              | $< 175$                                  |

#### 3.4.2.4 Fundamental period of vibration, $T$

The fundamental period of vibration,  $T$ , shall be determined from the following Equation:

1. For buildings with moment resisting frame system;

1.1. If infill walls does not impose restraint on frame displacements:

- For steel frame:  $T = 0.08H^{3/4}$  (3.5)

- For reinforced concrete Frame:  $T = 0.07H^{3/4}$  (3.6)

1.2. If infill walls impose restraint on frame displacements,  $T$  shall be taken as 80 percent of the value calculated from the above Equation e.

2. For buildings with other systems:  $T = 0.05H^{3/4}$  (3.7)

In the above Equation ,  $H$  is the height of the building in meters, measured from the base level. If the weight of the loft is greater than 25 percent of the roof, its height shall be considered as well.

### 3.4.2.5 Building importance factor, $I$

The building importance factor shall be determined in accordance with the classification, from Table 5:

**Table 3.7 :** Building Importance Factor

| Building Classification | Importance Factor |
|-------------------------|-------------------|
| Group 1                 | 1.4               |
| Group 2                 | 1.2               |
| Group 3                 | 1.0               |
| Group 4                 | 0.8               |

Buildings are grouped into four categories based on their importance:

#### *Group1- Buildings with “Very High Importance”*

This group includes buildings that their post-continuous earthquake operation is of special importance and any discontinuity in this regard leads to an indirect increase in casualties and damages. Hospitals, clinics, fire stations, water supply and power plants, aviation control towers, communication centers including radio and TV , police stations, rescue centers and, in general, all buildings that are involved in rescue and help operation are in this category. Buildings and structures supporting

toxic and hazardous material whose failure may causes widespread important environmental damage in short and long term are considered in this group.

*Group2- Buildings with “High Importance”*

This group consists of the following:

a- Buildings whose damage results in great loss of life such as: schools, mosques, stadiums, cinemas and movie theatres, assembly halls, departmental stores, terminals or any enclosed area with a capacity greater than 300 people under one ceiling.

b- Buildings whose damage results in loss of national heritage. These include museums, libraries and other places where national documents and valuable items are preserved.

c- Industrial buildings and facilities whose failure, may result in environmental pollution or widespread fire, such as refineries, fuel storage tanks and gas supply centers.

*Group3- Buildings with “Moderate Importance”*

All buildings that are within the scope of this Code, except those included in other categories, fall in this group. These include residential, office and commercial buildings, hotels, multistory parkings, warehouses, workshops, industrial buildings, etc.

*Group4- “Buildings with Low Importance”*

This group consists of the following:

a- Buildings with low risk of damage and loss of life due to their failure such as agricultural warehouses and poultry farms.

b- Temporary buildings with an operational life of less than two years.

### **3.4.2.6 Building behavior factor, R**

The building behavior factor, R, represents the global characteristic of the structure such as ductility, redundancy and the inherent overstrength capacity. This factor shall be determined in accordance with the type of lateral- force-resisting system, from

Table 3.8. The values presented in this Table conform to the Allowable Stress Design method and the resulting forces shall be appropriately amplified when other design philosophies such as the Limit State or Strength Design methods are used. In cases that a structural system other than the ones listed in Table 3.8 is used, its corresponding R-value shall be determined from reliable codes.

**Table 3.8 :** Building Behavior factor, R, and maximum permissible building height, H<sub>m</sub>

| Structural System               | Lateral-Force, Resisting System                         | R  | H <sub>m</sub><br>(m) |
|---------------------------------|---------------------------------------------------------|----|-----------------------|
| A- Bearing Wall System          | 1- Special reinforced concrete shear walls              | 7  | 50                    |
|                                 | 2- Intermediate reinforced concrete shear walls         | 6  | 50                    |
|                                 | 3- Ordinary reinforced concrete shear walls             | 5  | 30                    |
|                                 | 4- Reinforced masonry shear walls                       | 4  | 15                    |
| B- Building frame System        | 1- Special reinforced concrete shear walls              | 8  | 50                    |
|                                 | 2- Intermediate reinforced concrete shear walls         | 7  | 50                    |
|                                 | 3- Ordinary reinforced concrete shear walls             | 5  | 30                    |
|                                 | 4- Reinforced masonry shear walls                       | 4  | 15                    |
|                                 | 5- Steel eccentric braced frame                         | 7  | 50                    |
|                                 | 6- Steel concentric braced frame <sup>[1]</sup>         | 6  | 50                    |
| C-Moment-resisting frame system | 1-Special reinforced concrete moment-resisting frame[2] | 10 | 150                   |
|                                 | 2- Intermediate reinforced concrete moment-             | 7  | 50                    |
|                                 | 3- Ordinary reinforced concrete moment-                 | 4  | ---                   |
|                                 | 4-Special steel moment-resisting frame[1]               | 10 | 150                   |
|                                 | 5- Intermediate steel moment-resisting                  | 7  | 50                    |
|                                 | 6- Ordinary steel moment-resisting                      | 5  | ---                   |
|                                 | 1-Special moment-resisting frame(steel or               | 11 | 200                   |
|                                 | 2- Intermediate concrete moment-resisting               | 8  | 70                    |
|                                 | 3- Intermediate steel moment-resisting                  | 8  | 70                    |
|                                 | 4- Special steel moment-resisting frame+                | 10 | 150                   |
|                                 | 5- Special steel moment-resisting frame+                | 9  | 150                   |
|                                 | 6- Intermediate steel moment-resisting                  | 7  | 70                    |
|                                 | 7- Intermediate steel moment-resisting                  | 7  | 70                    |

### 3.4.2.7 Vertical distributions of seismic forces

The Design Base Shear, V, determined from the provisions, shall be distributed over the height of the structure in accordance with Equation 3.8:

$$F_i = (V - F_t) \frac{W_i h_i}{\sum_{j=1}^n W_j h_j} \quad (3.8)$$

Where:

$F_i$  = The lateral force at level i

$W_i$  = The seismic weight of level i, that is equal to the dead plus a percentage of live load and half of walls and columns weight located immediately above and below this level.

$h_i$  = Height of the story of level i, from the base level

$n$  = Total number of stories above the base level

$F_t$  = A concentrated force at the top level that is determined from the following Equation :

$$F_t = 0.07TV \quad (3.9)$$

$F_t$  need not exceed 0.25V and may be considered as zero where T is 0.7 second or less.

If the weight of the loft is greater than 25 percent of the roof, the concentrated force  $F_t$  , shall be applied at loft level. Otherwise, it shall be applied at roof level.

The design story shear, determined from the vertical distribution of seismic forces as above, shall be distributed among the various vertical lateral-force-resisting systems in proportion to their rigidities. The shear due to horizontal torsion resulting from eccentricities of the applied design lateral forces at levels above any story, shall also be included. Where diaphragms are not rigid, the effect of their deformations shall also be considered in horizontal distribution of shears.

The torsional design moment at a given story, i, shall be determined from the following Equation :

$$M_i = \sum_{j=1}^n (e_{ij} + e_{aj}) F_j \quad (3.10)$$

Where:

$e_{ij}$  = Eccentricity between the lateral force at level j and the center of stiffness at level i (the horizontal distance between the center of mass at level j and center of rigidity at level i)

$e_{aj}$  = Accidental eccentricity at level j

$F_j$  = The lateral force at level j

All structural elements shall be designed for the torsional moment that induces the maximum effect.

For buildings up to 5 story or less than 18 meters in height, when the eccentricity of the lateral story shears in levels above the level under consideration are less than 5 percent of building dimension at that level, perpendicular to the direction of the force under consideration, the torsional moments need not be considered in design.

#### 3.4.2.8 Numerical solution for equivalent earthquake load

Building data:

$$H = 25.2m \rightarrow T = 0.07 \times (25.2)^{3/4} = 0.78 \text{ sec}$$

$$I = 1$$

$$R = 7$$

Building located in seismic zone of 2  $\rightarrow A = 0.3g$

$$\text{Soil type is II} \rightarrow \begin{cases} T_0 = 0.1 \text{ sec} \\ T_s = 0.5 \text{ sec} \end{cases} \quad \text{and } S = 1.5$$

$$T \geq T_s \rightarrow B = (1.5 + 1) \left( \frac{0.5}{0.78} \right)^{3/4} = 1.85 \quad C = \frac{0.3 \times 1.85 \times 1}{7} = 0.0792$$

$$W = 21501.578 \text{ kN} \rightarrow V = 0.0792 \times 21501.578 = 1702.92 \text{ kN}$$

$$0.1AIW = 0.1 \times 0.3 \times 1 \times 21501.578 = 645.03 \text{ kN} \leq 1702.92 \text{ kN}$$

$$V \geq 0.1AIW$$

### **3.4.3 Linear dynamic analysis**

In dynamic analysis procedures, the horizontal seismic load is determined from the dynamic response of building subjected to an appropriate “Ground Motion”. These procedures include the “Response Spectrum” method and the “Time History” method, defined in Sections 3.3.2.1 and 3.3.2.2.

#### **3.4.3.1 Response spectrum analysis**

In this method a linear dynamic analysis is performed and the vibration modes of the building is determined. The maximum modal response is then obtained from the design spectrum and combined by a statistical method to get the total response of the building.

In each orthogonal direction, a minimum number of three first modes, or all the modes with periods higher than 0.4 seconds or all the modes that include at least 90 percent of the participating mass of the structure, whichever is the greatest, shall be considered in the analysis.

The maximum member forces, displacements, story forces, story shears and base reactions for each mode shall be combined by recognized methods such as the square root of the sum of the squares (SRSS) or the Complete Quadratic Combination (CQC). For structures with plan irregularity or where torsional effects are considerable, the modal combination method shall include the interaction of modes. For these cases, the Complete Quadratic Combination (CQC) method may be used.

#### **3.4.3.2 Time history analysis**

Time history analysis is a method in which the dynamic response of a structure is determined at each increment of time, when the base is subjected to a specified ground motion time history. The analysis shall be performed using accepted principles of dynamic.

Each pair of accelerograms shall be applied simultaneously in both principal, orthogonal directions and the response parameters are determined at each time

increment. The final response of the structure at each time increment shall be the maximum response obtained from the analysis of each of the three-accelerogram pairs.

In linear time history analysis a 5 percent damping ratio may be assumed, unless a different value is shown to be consistent with the anticipated structural behavior.

#### **3.4.4 Story Drift**

The story drift at each story level is defined as the difference between the displacements at centers of masses in the stories above and below that level. These displacements are calculated for the design or service level earthquakes and are named accordingly.

The story drift at each story level is the displacement due to the applied lateral earthquake load, assuming a linear behavior for the structure. This parameter is named “Design story drift” and “Service story drift” for the design or service level earthquakes respectively.

The actual relative displacement or the inelastic drift, at each story level is the displacement that will be obtained if the actual nonlinear behavior of the structure is considered in the analysis. This behavior is noticeable only for the design earthquake level. If linear analysis is performed, the actual relative displacement may be determined using the following Equation :

$$\Delta_M = 0.7R.\Delta_w \quad (3.11)$$

Where:

$\Delta_M$  = The actual design story drift

$\Delta_w$  = The design story drift

$R$  = Building behavior factor

The actual design drift at the center of mass of each story shall not exceed the following values. The analysis used to determine  $\Delta_M$  shall consider  $P - \Delta$  effect in accordance with Section 3.3.4.

For structures with  $T < 0.7$  second:  $\overline{\Delta_M} < 0.025h$

For structures with  $T \geq 0.7$  second:  $\overline{\Delta_M} < 0.02h$

Where  $T$  is the fundamental period, and  $h$  is the story height of the structure.

$\overline{\Delta_M}$  is the actual design story drift including  $P - \Delta$  effect.

The Service story drift at each story level shall not exceed 0.005 times the story height. This limitation may be increased to 0.008 times the story height where the type, material and connection of nonstructural elements are such that they can withstand greater drift without considerable damage.

In calculating the design drift for reinforced concrete structures, the cracked moments of inertia may be taken according to the recommendation of Iranian Concrete Code (ABA). These values are  $0.35 I_g$  and  $0.7 I_g$  for beams and columns, respectively. For walls a value of  $0.35 I_g$  or  $0.7 I_g$  may be considered based on the amount of expected cracking. For Service level earthquake, these moments of inertia may be increase 1.5 times. At service level earthquake the  $P - \Delta$  effects may be disregarded.

### 3.4.5 $P - \Delta$ effect

$P - \Delta$  effect is the effect of axial forces in vertical members undergoing lateral displacement. This effect induces additional shears and bending moments in structural members, and consequently increases lateral deflection. This effect is also known as the secondary effect.  $P - \Delta$  effect shall be considered when the stability index,  $\theta_i$ , as determined by Equation (3.12), is greater than 10 percent.

$$\theta_i = \left( \frac{P \Delta_w}{V h} \right)_i \quad (3.12)$$

Where:

$P_i$  = Total sum of dead and live loads from level  $i$  to the last level,  $n$

$V_i$  = Sum of shear forces at level  $i$

$h_i$  = Story height at level i

In no cases the stability index,  $\theta_i$ , shall exceed  $\theta_{\max}$  max as determined from the following Equation . In such cases the structure may be unstable and its design shall be revised.

$$\theta_{\max} = \frac{1.25}{R} \leq 0.25 \quad (3.13)$$

### 3.5 Analysis in SAP2000

Structure has been modeled in SAP2000 by properties mentioned in previous sections and analyzed by equavalent static earthquake load procedure, load combination which used in analysis given in Table 3.9:

**Table 3.9 :** Load combinatins used in analysis

|               |                                |
|---------------|--------------------------------|
| <b>COMB1</b>  | 1.25DL                         |
| <b>COMB2</b>  | 1.25DL+1.5LL                   |
| <b>COMB3</b>  | DL + 1.2 LL + 1.2 EX + 0.36 EY |
| <b>COMB4</b>  | DL + 1.2 LL + 1.2 EX - 0.36 EY |
| <b>COMB5</b>  | DL + 1.2 LL - 1.2 EX + 0.36 EY |
| <b>COMB6</b>  | DL + 1.2 LL - 1.2 EX - 0.36 EY |
| <b>COMB7</b>  | DL + 1.2 LL + 1.2 EY + 0.36 EX |
| <b>COMB8</b>  | DL + 1.2 LL + 1.2 EY - 0.36 EX |
| <b>COMB9</b>  | DL + 1.2 LL - 1.2 EY + 0.36 EX |
| <b>COMB10</b> | DL + 1.2 LL - 1.2 EY - 0.36 EX |
| <b>COMB11</b> | 0.85DL + 1.2 EX + 0.36 EY      |
| <b>COMB12</b> | 0.85DL + 1.2 EX - 0.36 EY      |
| <b>COMB13</b> | 0.85DL - 1.2 EX + 0.36 EY      |
| <b>COMB14</b> | 0.85DL - 1.2 EX - 0.36 EY      |
| <b>COMB15</b> | 0.85DL + 1.2 EY + 0.36 EX      |
| <b>COMB16</b> | 0.85DL + 1.2 EY - 0.36 EX      |
| <b>COMB17</b> | 0.85DL - 1.2 EY + 0.36 EX      |
| <b>COMB18</b> | 0.85DL - 1.2 EY - 0.36 EX      |

#### 3.5.1 Linear static analysis

Linear static analysis has been proceed base on mentioned rules in Code 2800 and the results for anaylsis under gravitational and equavalent earthquak are as follow:

### 3.5.1.1 Element force results

For obtaining force results in each story a section cut has defined which dimension of this section cuts are given in Table 3.11 and results of analysis considering gravitational and equivalent loads are given in Table 3.12 for resultant forces and Table 3.13 for resultant bending moments in 1,2, and 3 direction which are equivalent to X,Y, and Z local directions.

**Table 3.10 :** Base shear in X and Y calculated by equivalent earthquake method

| LoadPat | Dir  | UserZ  | MaxZ | MinZ | C        | WeightUsed | BaseShear |
|---------|------|--------|------|------|----------|------------|-----------|
| Text    | Text | Yes/No | m    | m    | Unitless | KN         | KN        |
| EX      | X    | Yes    | 25.2 | 0    | 0.0792   | 21325.951  | 1689.015  |
| EY      | Y    | Yes    | 25.2 | 0    | 0.0792   | 21325.951  | 1689.015  |

To exporting datas, an envelope load combination including all existing combinations, which mentioned in Table 3.9 has been defined so this envelope load case has been selected as reference load case to obtaining maximum and minimum values for each type of loads in each element.

### 3.5.1.2 Displacements in control node

Relative displacements due to lateral load (in this case equivalent earthquake load) has been controlled in control node, which defined as a point in mass center of each floor, and given in Table 3.13, Table 3.15 and drift control for each story given in Table 3.14, Table 3.16.

**Table 3.11** : Defintion of section cuts for obtained results

| <b>Section Cut ID</b> | <b>Story</b> | <b>Point</b> | <b>X</b> | <b>Y</b> | <b>Z</b> |
|-----------------------|--------------|--------------|----------|----------|----------|
| <b>SCUT1</b>          | 1            | 1            | 0        | 0        | 0.1      |
|                       |              | 2            | 0        | 15.5     | 0.1      |
|                       |              | 3            | 14.58    | 15.5     | 0.1      |
|                       |              | 4            | 14.58    | 0        | 0.1      |
| <b>SCUT2</b>          | 2            | 1            | 0        | 0        | 2.9      |
|                       |              | 2            | 0        | 15.5     | 2.9      |
|                       |              | 3            | 14.58    | 15.5     | 2.9      |
|                       |              | 4            | 14.58    | 0        | 2.9      |
| <b>SCUT3</b>          | 3            | 1            | 0        | 0        | 6.1      |
|                       |              | 2            | 0        | 15.5     | 6.1      |
|                       |              | 3            | 14.58    | 15.5     | 6.1      |
|                       |              | 4            | 14.58    | 0        | 6.1      |
| <b>SCUT4</b>          | 4            | 1            | 0        | 0        | 9.3      |
|                       |              | 2            | 0        | 15.5     | 9.3      |
|                       |              | 3            | 14.58    | 15.5     | 9.3      |
|                       |              | 4            | 14.58    | 0        | 9.3      |
| <b>SCUT5</b>          | 5            | 1            | 0        | 0        | 12.5     |
|                       |              | 2            | 0        | 15.5     | 12.5     |
|                       |              | 3            | 14.58    | 15.5     | 12.5     |
|                       |              | 4            | 14.58    | 0        | 12.5     |
| <b>SCUT6</b>          | 6            | 1            | 0        | 0        | 15.7     |
|                       |              | 2            | 0        | 15.5     | 15.7     |
|                       |              | 3            | 14.58    | 15.5     | 15.7     |
|                       |              | 4            | 14.58    | 0        | 15.7     |
| <b>SCUT7</b>          | 7            | 1            | 0        | 0        | 18.9     |
|                       |              | 2            | 0        | 15.5     | 18.9     |
|                       |              | 3            | 14.58    | 15.5     | 18.9     |
|                       |              | 4            | 14.58    | 0        | 18.9     |
| <b>SCUT8</b>          | 8            | 1            | 0        | 0        | 22.1     |
|                       |              | 2            | 0        | 15.5     | 22.1     |
|                       |              | 3            | 14.58    | 15.5     | 22.1     |
|                       |              | 4            | 14.58    | 0        | 22.1     |

**Table 3.12 :** Section cut forces considering gravitational and equivalent earthquake loads

| Section Cut | Output Case | CaseType    | Step Type | F1       | F2       | F3       |
|-------------|-------------|-------------|-----------|----------|----------|----------|
| Text        | Text        | Text        | Text      | KN       | KN       | KN       |
| SCUT1       | Evelope     | Combination | Max       | 1997.77  | 1997.77  | 30549.04 |
| SCUT1       | Evelope     | Combination | Min       | -1997.77 | -1997.77 | 17298.11 |
| SCUT2       | Evelope     | Combination | Max       | 1944.12  | 1944.12  | 26566.10 |
| SCUT2       | Evelope     | Combination | Min       | -1944.12 | -1944.12 | 15024.11 |
| SCUT3       | Evelope     | Combination | Max       | 1829.17  | 1829.17  | 22599.35 |
| SCUT3       | Evelope     | Combination | Min       | -1829.17 | -1829.17 | 12761.13 |
| SCUT4       | Evelope     | Combination | Max       | 1652.98  | 1652.98  | 18632.59 |
| SCUT4       | Evelope     | Combination | Min       | -1652.98 | -1652.98 | 10498.14 |
| SCUT5       | Evelope     | Combination | Max       | 1417.45  | 1417.45  | 14675.15 |
| SCUT5       | Evelope     | Combination | Min       | -1417.45 | -1417.45 | 8241.485 |
| SCUT6       | Evelope     | Combination | Max       | 1123.92  | 1123.92  | 10771.89 |
| SCUT6       | Evelope     | Combination | Min       | -1123.92 | -1123.92 | 6021.67  |
| SCUT7       | Evelope     | Combination | Max       | 779.065  | 779.065  | 6955.59  |
| SCUT7       | Evelope     | Combination | Min       | -779.06  | -779.06  | 3860.99  |
| SCUT8       | Evelope     | Combination | Max       | 387.65   | 387.65   | 3222.09  |
| SCUT8       | Evelope     | Combination | Min       | -387.65  | -387.65  | 1756.61  |

**Table 3.13 :** Section cut bending moments considering gravitational and equivalent earthquake loads

| SectionCut | OutputCase | CaseType    | StepType | M1       | M2        | M3       |
|------------|------------|-------------|----------|----------|-----------|----------|
| Text       | Text       | Text        | Text     | KN-m     | KN-m      | KN-m     |
| SCUT1      | Evelope    | Combination | Max      | 236571.2 | -94206.1  | 19991.3  |
| SCUT1      | Evelope    | Combination | Min      | 98211.0  | -228612.6 | -19991.3 |
| SCUT2      | Evelope    | Combination | Max      | 205710.3 | -82848.9  | 19454.8  |
| SCUT2      | Evelope    | Combination | Min      | 86313.4  | -198802.1 | -19454.8 |
| SCUT3      | Evelope    | Combination | Max      | 175045.0 | -72340.1  | 18307.7  |
| SCUT3      | Evelope    | Combination | Min      | 75289.6  | -169160.7 | -18307.7 |
| SCUT4      | Evelope    | Combination | Max      | 144379.8 | -61427.6  | 16548.1  |
| SCUT4      | Evelope    | Combination | Min      | 63861.4  | -139519.3 | -16548.1 |
| SCUT5      | Evelope    | Combination | Max      | 113714.5 | -49917.9  | 14193.0  |
| SCUT5      | Evelope    | Combination | Min      | 51815.0  | -109904.2 | -14193.0 |
| SCUT6      | Evelope    | Combination | Max      | 83431.9  | -37908.1  | 11259.1  |
| SCUT6      | Evelope    | Combination | Min      | 39237.8  | -80715.8  | -11259.1 |
| SCUT7      | Evelope    | Combination | Max      | 53885.8  | -25338.3  | 7813.1   |
| SCUT7      | Evelope    | Combination | Min      | 26170.4  | -52157.7  | -7813.1  |
| SCUT8      | Evelope    | Combination | Max      | 24947.1  | -12026.7  | 3919.3   |
| SCUT8      | Evelope    | Combination | Min      | 12368.6  | -24192.7  | -3919.3  |

**Table 3.14 :** Story displacements in center of mass due to equivalent earthquake loads in X-direction

| <b>Joint</b> | <b>OutputCase</b> | <b>Story</b> | <b>CaseType</b> | <b>U1</b> |
|--------------|-------------------|--------------|-----------------|-----------|
| Text         | Text              |              | Text            | m         |
| 147          | EX                | 1            | LinStatic       | 0.004391  |
| 194          | EX                | 2            | LinStatic       | 0.014647  |
| 195          | EX                | 3            | LinStatic       | 0.025182  |
| 196          | EX                | 4            | LinStatic       | 0.035078  |
| 197          | EX                | 5            | LinStatic       | 0.045146  |
| 198          | EX                | 6            | LinStatic       | 0.053971  |
| 199          | EX                | 7            | LinStatic       | 0.061298  |
| 200          | EX                | 8            | LinStatic       | 0.066413  |

**Table 3.15 :** Story drift control in center of mass due to equivalent earthquake loads in X-direction

| <b>Joint</b> | <b>OutputCase</b> | <b>U1</b> | $\Delta_w$ (cm) | $\bar{\Delta}_m$ (cm) | h(cm) | 0.02h | <b>Drift Control</b><br>( $\bar{\Delta}_m < 0.02h$ ) |
|--------------|-------------------|-----------|-----------------|-----------------------|-------|-------|------------------------------------------------------|
| Text         | Text              | cm        |                 |                       |       |       |                                                      |
| 147          | EX                | 0.439128  | 0.439128        | 2.1517272             | 280   | 5.6   | Drift Accepted                                       |
| 194          | EX                | 1.464654  | 1.025526        | 5.0250774             | 320   | 6.4   | Drift Accepted                                       |
| 195          | EX                | 2.518184  | 1.05353         | 5.162297              | 320   | 6.4   | Drift Accepted                                       |
| 196          | EX                | 3.507778  | 0.989594        | 4.8490106             | 320   | 6.4   | Drift Accepted                                       |
| 197          | EX                | 4.51462   | 1.006842        | 4.9335258             | 320   | 6.4   | Drift Accepted                                       |
| 198          | EX                | 5.397056  | 0.882436        | 4.3239364             | 320   | 6.4   | Drift Accepted                                       |
| 199          | EX                | 6.1298    | 0.732744        | 3.5904456             | 320   | 6.4   | Drift Accepted                                       |
| 200          | EX                | 6.641328  | 0.511528        | 2.5064872             | 300   | 6     | Drift Accepted                                       |

**Table 3.16 :** Story displacements in center of mass due to equivalent earthquake loads in Y-direction

| <b>Joint</b> | <b>OutputCase</b> | <b>Story</b> | <b>CaseType</b> | <b>U2</b> |
|--------------|-------------------|--------------|-----------------|-----------|
| Text         | Text              |              | Text            | m         |
| 147          | EY                | 1            | LinStatic       | 0.004565  |
| 194          | EY                | 2            | LinStatic       | 0.015353  |
| 195          | EY                | 3            | LinStatic       | 0.026477  |
| 196          | EY                | 4            | LinStatic       | 0.036886  |
| 197          | EY                | 5            | LinStatic       | 0.047551  |
| 198          | EY                | 6            | LinStatic       | 0.056909  |
| 199          | EY                | 7            | LinStatic       | 0.06467   |
| 200          | EY                | 8            | LinStatic       | 0.070359  |

**Table 3.17 :** Story drift control in center of mass due to equivalent earthquake loads in Y-direction

| Joint | OutputCase | U2       | $\Delta_w$ (cm) | $\bar{\Delta}_m$ (cm) | h(cm) | 0.02h | Drift Control<br>( $\bar{\Delta}_m < 0.02h$ ) |
|-------|------------|----------|-----------------|-----------------------|-------|-------|-----------------------------------------------|
| Text  | Text       | cm       |                 |                       |       |       |                                               |
| 147   | EY         | 0.456543 | 0.456543        | 2.2370607             | 280   | 5.6   | Drift Accepted                                |
| 194   | EY         | 1.535343 | 1.0788          | 5.28612               | 320   | 6.4   | Drift Accepted                                |
| 195   | EY         | 2.647706 | 1.112363        | 5.4505787             | 320   | 6.4   | Drift Accepted                                |
| 196   | EY         | 3.688588 | 1.040882        | 5.1003218             | 320   | 6.4   | Drift Accepted                                |
| 197   | EY         | 4.755143 | 1.066555        | 5.2261195             | 320   | 6.4   | Drift Accepted                                |
| 198   | EY         | 5.69092  | 0.935777        | 4.5853073             | 320   | 6.4   | Drift Accepted                                |
| 199   | EY         | 6.467015 | 0.776095        | 3.8028655             | 320   | 6.4   | Drift Accepted                                |
| 200   | EY         | 7.035904 | 0.568889        | 2.7875561             | 320   | 6.4   | Drift Accepted                                |

### 3.5.2 Linear dynamic analysis

Linear dynamic analysis proceed according to design spectrum given in Code2800 as given in Figure 3.8

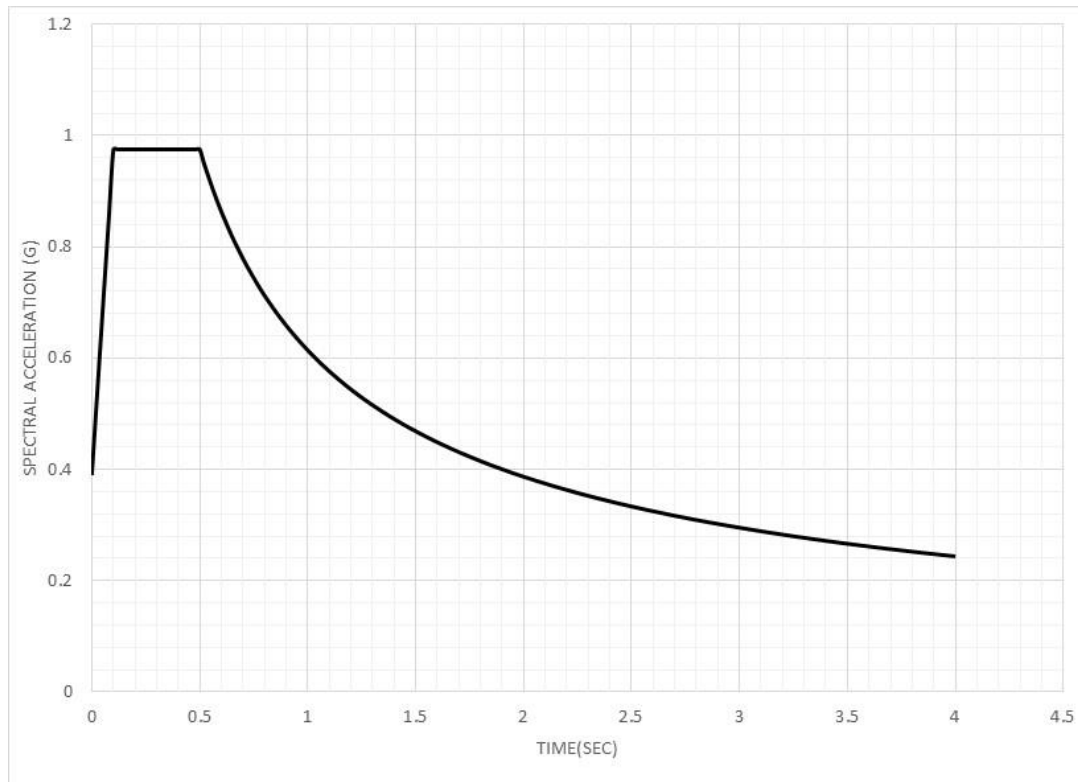


**Figure 3.8 :** Design acceleration response spectrum according to Code2800

Modal analysis proceed by ritz vector method considering  $P - \Delta$  effect and 90% target dynamic participation ratio in both direction and structure analyzed using modal

combination by design acceleration response spectrum method considering free vibration modes which resultant modal period, forces in section cuts in each direction given in following tables and figures.

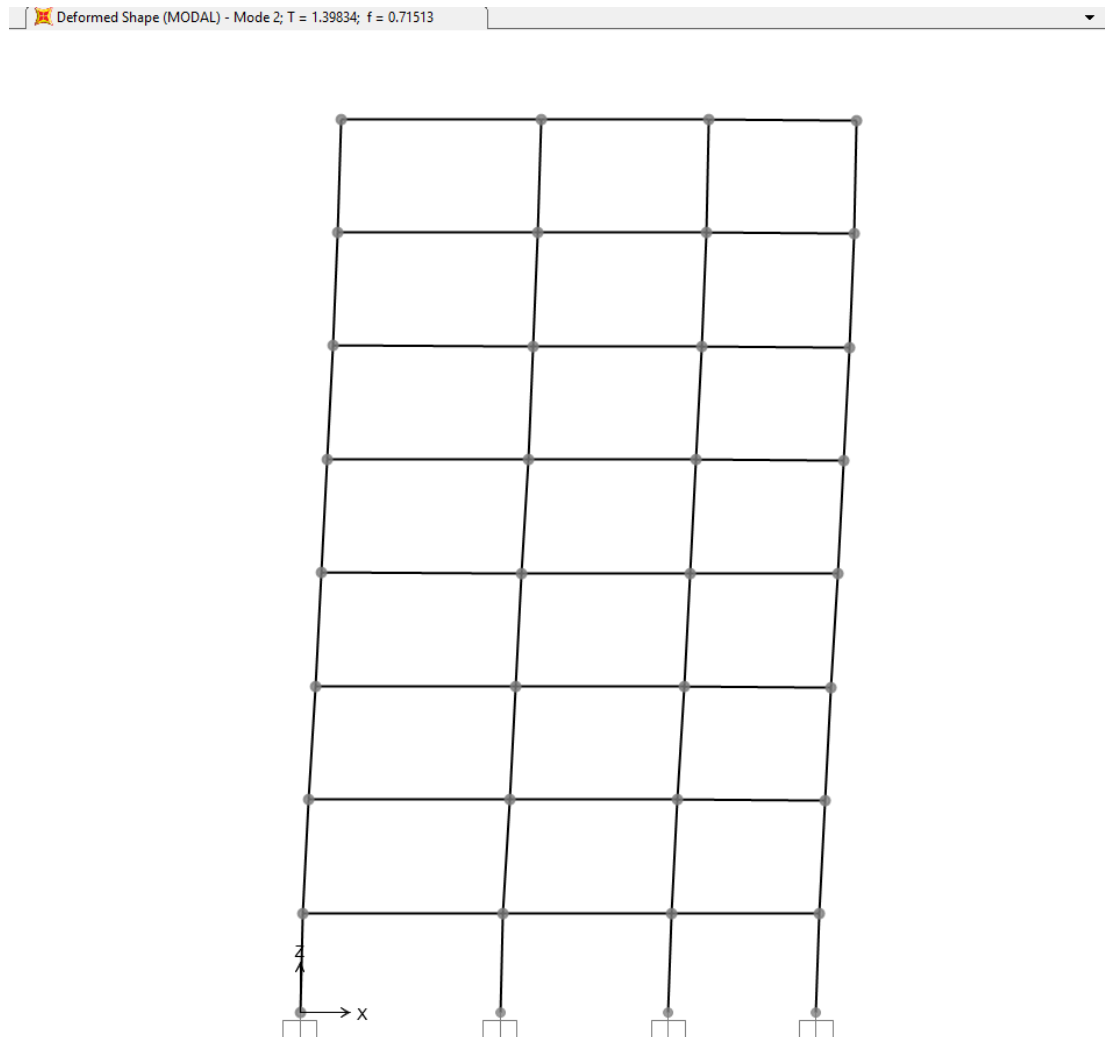


**Figure 3.9 :** First mode free vibration in X direction

Free vibration period:1.398sec

Free vibration frequency:0.715Hz

Participating Mass Ratio: 0.75118

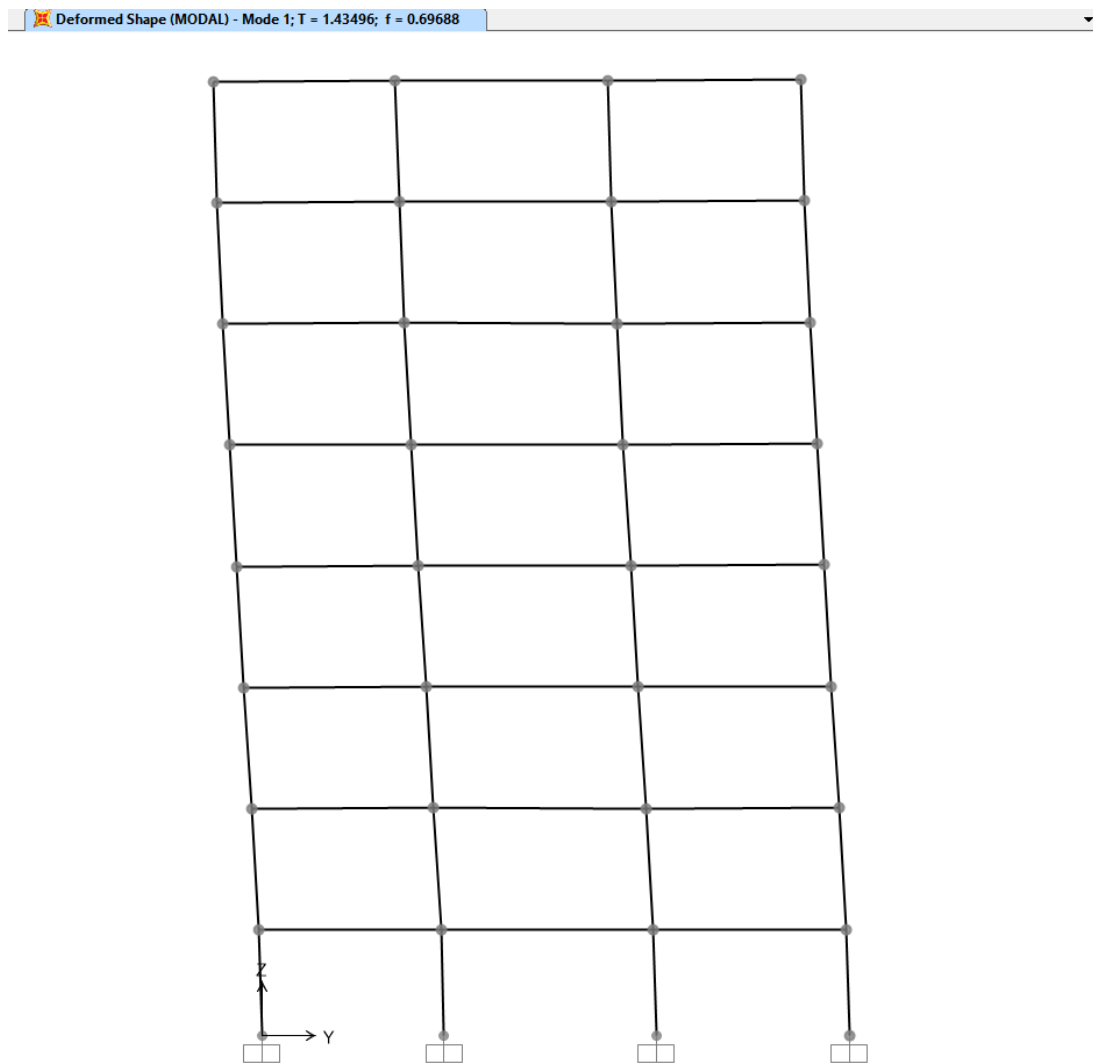


**Figure 3.10 :** First mode free vibration in Y direction

Free vibration period:1.434sec

Free vibration frequency:0.696Hz

Participating Mass Ratio: 0.75922

**Table 3.18** : Modal analysis results

| <b>Output Case</b> | <b>StepType</b> | <b>StepNum</b>  | <b>Period</b> | <b>Mass Ratio X (%)</b> | <b>Mass Ratio Y (%)</b> |
|--------------------|-----------------|-----------------|---------------|-------------------------|-------------------------|
| <b>Text</b>        | <b>Text</b>     | <b>Unitless</b> | <b>Sec</b>    | <b>Unitless</b>         | <b>Unitless</b>         |
| MODAL              | Mode            | 1               | 1.43496       | 0.00002551              | 0.759                   |
| MODAL              | Mode            | 2               | 1.398339      | 0.751                   | 0.759                   |
| MODAL              | Mode            | 3               | 1.248847      | 0.762                   | 0.76                    |
| MODAL              | Mode            | 4               | 0.518508      | 0.762                   | 0.872                   |
| MODAL              | Mode            | 5               | 0.50264       | 0.872                   | 0.872                   |
| MODAL              | Mode            | 6               | 0.453502      | 0.874                   | 0.872                   |
| MODAL              | Mode            | 7               | 0.302299      | 0.874                   | 0.914                   |
| MODAL              | Mode            | 8               | 0.2925        | 0.915                   | 0.914                   |
| MODAL              | Mode            | 9               | 0.265913      | 0.916                   | 0.914                   |
| MODAL              | Mode            | 10              | 0.20588       | 0.916                   | 0.939                   |
| MODAL              | Mode            | 11              | 0.200423      | 0.94                    | 0.939                   |
| MODAL              | Mode            | 12              | 0.182352      | 0.94                    | 0.939                   |
| MODAL              | Mode            | 13              | 0.154907      | 0.94                    | 0.957                   |
| MODAL              | Mode            | 14              | 0.151879      | 0.957                   | 0.957                   |
| MODAL              | Mode            | 15              | 0.128747      | 0.959                   | 0.957                   |
| MODAL              | Mode            | 16              | 0.121842      | 0.959                   | 0.967                   |
| MODAL              | Mode            | 17              | 0.11824       | 0.968                   | 0.967                   |
| MODAL              | Mode            | 18              | 0.106824      | 0.969                   | 0.971                   |
| MODAL              | Mode            | 19              | 0.102292      | 0.969                   | 0.982                   |
| MODAL              | Mode            | 20              | 0.101287      | 0.983                   | 0.982                   |
| MODAL              | Mode            | 21              | 0.084454      | 0.983                   | 1                       |
| MODAL              | Mode            | 22              | 0.083609      | 1                       | 1                       |
| MODAL              | Mode            | 23              | 0.018483      | 1                       | 1                       |
| MODAL              | Mode            | 24              | 0.014495      | 1                       | 1                       |

**Table 3.19** : Resulted forces in section cuts due to modal combination by design spectrum analysis for X-direction

| <b>SectionCut</b> | <b>OutputCase</b> | <b>CaseType</b> | <b>F1</b> | <b>F2</b> | <b>F3</b> |
|-------------------|-------------------|-----------------|-----------|-----------|-----------|
| <b>Text</b>       | <b>Text</b>       | <b>Text</b>     | <b>KN</b> | <b>KN</b> | <b>KN</b> |
| SCUT1             | SPECX             | LinRespSpec     | 1661.88   | 9.941     | 35.068    |
| SCUT2             | SPECX             | LinRespSpec     | 1611.7    | 8.791     | 26.775    |
| SCUT3             | SPECX             | LinRespSpec     | 1496.91   | 8.054     | 25.627    |
| SCUT4             | SPECX             | LinRespSpec     | 1344.22   | 7.326     | 23.341    |
| SCUT5             | SPECX             | LinRespSpec     | 1171.79   | 6.661     | 20.057    |
| SCUT6             | SPECX             | LinRespSpec     | 972.128   | 6.468     | 16.221    |
| SCUT7             | SPECX             | LinRespSpec     | 731.71    | 5.584     | 11.321    |
| SCUT8             | SPECX             | LinRespSpec     | 421.789   | 3.698     | 5.977     |

**Table 3.20 :** Resulted bending moments in section cuts due to modal combination by design spectrum analysis for X-direction

| SectionCut | OutputCase | CaseType    | M1      | M2        | M3        |
|------------|------------|-------------|---------|-----------|-----------|
| Text       | Text       | Text        | KN-m    | KN-m      | KN-m      |
| SCUT1      | SPECX      | LinRespSpec | 315.444 | 28523.688 | 13715.562 |
| SCUT2      | SPECX      | LinRespSpec | 266.27  | 24075.804 | 13308.02  |
| SCUT3      | SPECX      | LinRespSpec | 264.29  | 19119.832 | 12359.885 |
| SCUT4      | SPECX      | LinRespSpec | 260.697 | 14527.644 | 11094.932 |
| SCUT5      | SPECX      | LinRespSpec | 234.802 | 10384.453 | 9674.5278 |
| SCUT6      | SPECX      | LinRespSpec | 200.599 | 6727.0999 | 8039.5005 |
| SCUT7      | SPECX      | LinRespSpec | 140.53  | 3663.8459 | 6057.6218 |
| SCUT8      | SPECX      | LinRespSpec | 73.5796 | 1342.1212 | 3501.0238 |

**Table 3.21 :** Resulted forces in section cuts due to modal combination by design spectrum analysis for Y-direction

| SectionCut | OutputCase | CaseType    | F1    | F2      | F3     |
|------------|------------|-------------|-------|---------|--------|
| Text       | Text       | Text        | KN    | KN      | KN     |
| SCUT1      | SPECY      | LinRespSpec | 9.905 | 1661.79 | 22.108 |
| SCUT2      | SPECY      | LinRespSpec | 8.842 | 1611.73 | 23.57  |
| SCUT3      | SPECY      | LinRespSpec | 8.118 | 1497.63 | 21.137 |
| SCUT4      | SPECY      | LinRespSpec | 7.633 | 1345.35 | 19.348 |
| SCUT5      | SPECY      | LinRespSpec | 7.09  | 1172.92 | 16.342 |
| SCUT6      | SPECY      | LinRespSpec | 7.19  | 972.905 | 13     |
| SCUT7      | SPECY      | LinRespSpec | 6.148 | 733.03  | 9.042  |
| SCUT8      | SPECY      | LinRespSpec | 3.979 | 426.413 | 4.96   |

**Table 3.22 :** Resulted bending moments in section cuts due to modal combination by design spectrum analysis for Y-direction

| SectionCut | OutputCase | CaseType    | M1        | M2      | M3        |
|------------|------------|-------------|-----------|---------|-----------|
| Text       | Text       | Text        | KN-m      | KN-m    | KN-m      |
| SCUT1      | SPECY      | LinRespSpec | 28588.903 | 185.96  | 12221.055 |
| SCUT2      | SPECY      | LinRespSpec | 24138.223 | 174.943 | 11853.238 |
| SCUT3      | SPECY      | LinRespSpec | 19171.55  | 155.551 | 11016.157 |
| SCUT4      | SPECY      | LinRespSpec | 14567.622 | 139.684 | 9895.7603 |
| SCUT5      | SPECY      | LinRespSpec | 10415.577 | 113.4   | 8627.0715 |
| SCUT6      | SPECY      | LinRespSpec | 6752.249  | 96.5535 | 7155.1008 |
| SCUT7      | SPECY      | LinRespSpec | 3686.7335 | 58.1485 | 5386.8081 |
| SCUT8      | SPECY      | LinRespSpec | 1359.6054 | 36.4778 | 3124.9461 |

**Table 3.23 :** Story displacements in center of mass due to design spectrum in X-direction

| Joint | OutputCase | CaseType    | U1      |
|-------|------------|-------------|---------|
| Text  | Text       | Text        | m       |
| 147   | SPECX      | LinRespSpec | 0.00437 |
| 194   | SPECX      | LinRespSpec | 0.01451 |
| 195   | SPECX      | LinRespSpec | 0.02476 |
| 196   | SPECX      | LinRespSpec | 0.0342  |
| 197   | SPECX      | LinRespSpec | 0.04371 |
| 198   | SPECX      | LinRespSpec | 0.05207 |
| 199   | SPECX      | LinRespSpec | 0.05917 |
| 200   | SPECX      | LinRespSpec | 0.06435 |

**Table 3.24 :** Story drift control in center of mass due to design spectrum in X-direction

| Joint | OutputCase | U1       | $\Delta_w$ (cm) | $\bar{\Delta}_m$ (cm) | h(cm) | 0.02h | Drift Control<br>( $\bar{\Delta}_m < 0.02h$ ) |
|-------|------------|----------|-----------------|-----------------------|-------|-------|-----------------------------------------------|
| Text  | Text       | cm       |                 |                       |       |       |                                               |
| 147   | SPECX      | 0.436683 | 0.436683        | 2.1397467             | 280   | 5.6   | Drift Accepted                                |
| 194   | SPECX      | 1.451161 | 1.014478        | 4.9709422             | 320   | 6.4   | Drift Accepted                                |
| 195   | SPECX      | 2.476121 | 1.02496         | 5.022304              | 320   | 6.4   | Drift Accepted                                |
| 196   | SPECX      | 3.420227 | 0.944106        | 4.6261194             | 320   | 6.4   | Drift Accepted                                |
| 197   | SPECX      | 4.370626 | 0.950399        | 4.6569551             | 320   | 6.4   | Drift Accepted                                |
| 198   | SPECX      | 5.207128 | 0.836502        | 4.0988598             | 320   | 6.4   | Drift Accepted                                |
| 199   | SPECX      | 5.917164 | 0.710036        | 3.4791764             | 320   | 6.4   | Drift Accepted                                |
| 200   | SPECX      | 6.434886 | 0.517722        | 2.5368378             | 320   | 6.4   | Drift Accepted                                |

**Table 3.25 :** Story displacements in center of mass due to design spectrum in Y-direction

| Joint | OutputCase | CaseType    | U2      |
|-------|------------|-------------|---------|
| Text  | Text       | Text        | m       |
| 147   | SPECY      | LinRespSpec | 0.00454 |
| 194   | SPECY      | LinRespSpec | 0.01518 |
| 195   | SPECY      | LinRespSpec | 0.02599 |
| 196   | SPECY      | LinRespSpec | 0.03591 |
| 197   | SPECY      | LinRespSpec | 0.04598 |
| 198   | SPECY      | LinRespSpec | 0.05484 |
| 199   | SPECY      | LinRespSpec | 0.06237 |
| 200   | SPECY      | LinRespSpec | 0.06813 |

**Table 3.26 :** Story drift control in center of mass due to design spectrum in Y-  
direction

| Joint | OutputCase | U2       | $\Delta_w$ (cm) | $\bar{\Delta}_m$ (cm) | h(cm) | 0.02h | Drift Control<br>( $\bar{\Delta}_m < 0.02h$ ) |
|-------|------------|----------|-----------------|-----------------------|-------|-------|-----------------------------------------------|
| Text  | Text       | cm       |                 |                       |       |       |                                               |
| 147   | SPECY      | 0.453563 | 0.453563        | 2.2224587             | 280   | 5.6   | Drift Accepted                                |
| 194   | SPECY      | 1.518196 | 1.064633        | 5.2167017             | 320   | 6.4   | Drift Accepted                                |
| 195   | SPECY      | 2.598482 | 1.080286        | 5.2934014             | 320   | 6.4   | Drift Accepted                                |
| 196   | SPECY      | 3.591239 | 0.992757        | 4.8645093             | 320   | 6.4   | Drift Accepted                                |
| 197   | SPECY      | 4.597844 | 1.006605        | 4.9323645             | 320   | 6.4   | Drift Accepted                                |
| 198   | SPECY      | 5.484066 | 0.886222        | 4.3424878             | 320   | 6.4   | Drift Accepted                                |
| 199   | SPECY      | 6.23657  | 0.752504        | 3.6872696             | 320   | 6.4   | Drift Accepted                                |
| 200   | SPECY      | 6.812564 | 0.575994        | 2.8223706             | 320   | 6.4   | Drift Accepted                                |

### 3.5.3 Nonlinear static analysis

In FEMA-356 and IR360, several alternative invariant loading patterns are recommended for estimating equivalent seismic demands. In this study, two loading patterns are considered. These two patterns are permitted when more than 75 percent of the total mass participates in the fundamental vibration mode in the direction under consideration.

#### 3.5.3.1 Lateral load distribution

*“Load pattern considering static linear analysis (inverse triangular load pattern)”*

In this method lateral load for each story are proportionate by base shear distribution in height based on earthquake codes.in CODE 2800 standard lateral loads ditribution given as Equation 3.14:

$$F = V \times \frac{W \times h}{\sum_{t=1}^n W_t h_t} \quad (3.14)$$

In ASCE07 and IBC2006 and in FEAM356 and FEMA273 rehabilitation standards Equation 3.15 presented as lateral load distribution:

$$F_t = V \times \frac{W_t \times h_t^\kappa}{\sum_{t=1}^n W_t h_t^\kappa} \Rightarrow \kappa = \begin{pmatrix} 1 \rightarrow T \leq 0.5 \\ \left( \frac{T + 1.5}{2} \right) \rightarrow 0.5 \leq T \leq 2.5 \\ 2 \rightarrow T > 2.5 \end{pmatrix} \quad (3.15)$$

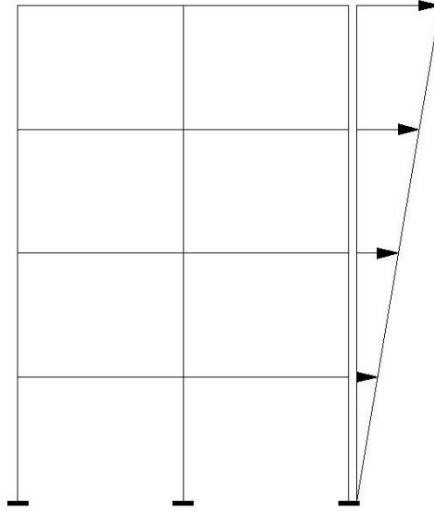


**Figure 3.11 :** Lateral load distribution proportionate to equivalent earthquake loads  
*“Modal distribution pattern”*

In this method lateral load distribution is proportionate to stories mass and displacement of mode shapes vector and expressed by Equation 3.16:

$$F_{ji} = \phi_{ji} m_{ji} \quad (3.16)$$

j : level number

i : mode number

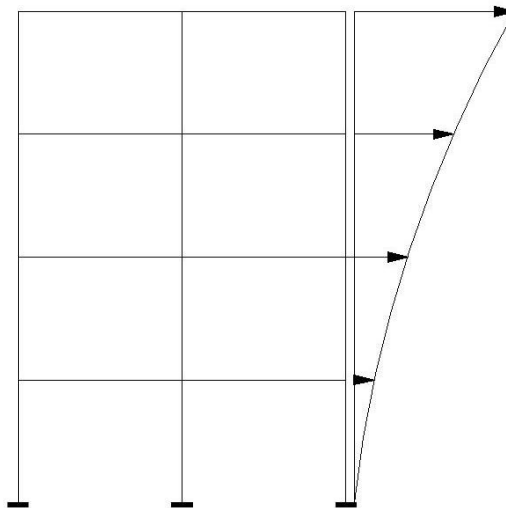


**Figure 3.12 :** Lateral load distribution proportionate to modal analysis

In this project both modal and inverse triangle load pattern applied to the structure model and the most critic results has been chosen as rehabilitation criteria, two

nonlinear gravitational load combination which proposed in FEMA and IR360 has been applied as initial conditions for this analysis:

$$\begin{aligned} PUSHG1 &= 1.1(Q_D + Q_L) \\ PUSHG2 &= 0.9Q_D \end{aligned} \quad (3.17)$$

And the other nonlinear loads applied as continue in state of loads which given in Equation 3.17, in Table 3.26 and Table 3.27 applied nonlinear static lateral loads are given :

**Table 3.27 :** Modal Lateral Load Pattern

| Load Case Name | Lateral Load Pattern | P_Deta Parameter | Initial Condition | Mode Number | Scale Factor | Direction (3.17) |
|----------------|----------------------|------------------|-------------------|-------------|--------------|------------------|
| PUSHMODEXG1    | Modal                | Yes              | PUSHG1            | 2           | 1            | X                |
| PUSHMODEXG-1   | Modal                | Yes              | PUSHG1            | 2           | -1           | X                |
| PUSHMODEYG1    | Modal                | Yes              | PUSHG1            | 1           | 1            | Y                |
| PUSHMODEYG-1   | Modal                | Yes              | PUSHG1            | 1           | -1           | Y                |
| PUSHMODEXG2    | Modal                | Yes              | PUSHG2            | 2           | 1            | X                |
| PUSHMODEXG-2   | Modal                | Yes              | PUSHG2            | 2           | -1           | X                |
| PUSHMODEYG2    | Modal                | Yes              | PUSHG2            | 1           | 1            | Y                |
| PUSHMODEYG-2   | Modal                | Yes              | PUSHG2            | 1           | -1           | Y                |

**Table 3.28 :** Inverse Tringle Lateral Load pattern

| Load Case Name | Lateral Load Pattern  | P_Deta Parameter | Initial Condition | Load Case | Scale Factor | Direction |
|----------------|-----------------------|------------------|-------------------|-----------|--------------|-----------|
| PUSHTRIXG1     | Equivalent Earthquake | Yes              | PUSHG1            | EX        | 1            | X         |
| PUSHTRIXG-1    | Equivalent Earthquake | Yes              | PUSHG1            | EX        | -1           | X         |
| PUSHTRIYG1     | Equivalent Earthquake | Yes              | PUSHG1            | EY        | 1            | Y         |
| PUSHTRIYG-1    | Equivalent Earthquake | Yes              | PUSHG1            | EY        | -1           | Y         |
| PUSHTRIXG2     | Equivalent Earthquake | Yes              | PUSHG2            | EX        | 1            | X         |
| PUSHTRIXG-2    | Equivalent Earthquake | Yes              | PUSHG2            | EX        | -1           | X         |
| PUSHTRIYG2     | Equivalent Earthquake | Yes              | PUSHG2            | EY        | 1            | Y         |
| PUSHTRIYG-2    | Equivalent Earthquake | Yes              | PUSHG2            | EY        | -1           | Y         |

### 3.5.3.2 Plastic hinge definition

In SAP2000 plastic hinge definitions base on FEMA 356 standard are available for use and this prjocet for RC columns(P-M-M) and beams(M3) has been used beacuase the values for plastic hinge definition are exactly same in both FEMA 365 and IR360 standards.

### 3.5.3.3 Target displacement

Target displacement has been calculated by Equation 2.8 but for calculating target displacement by SAP2000 and obtaining push over curve, related parameters applied in program as follow:

Target displacement calculation by Equation 2.8:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

|                         |                               |
|-------------------------|-------------------------------|
| $C_0 = 1.46$            | $B_x = 1.2596$                |
| $C_1 = 1$               | $B_y = 1.2384$                |
| $C_2 = 1$               | $S_{ax} = 0.37788g$           |
| $C_3 = 1$               | $S_{ay} = 0.37152g$           |
| $C_{mx} = C_{my} = 0.9$ | $T_e = 1.5\text{sec}$         |
| $T_x = 1.398\text{sec}$ | $\delta_{tx} = 26.7\text{cm}$ |
| $T_y = 1.434\text{sec}$ | $\delta_{ty} = 27.7\text{cm}$ |

Moreover, in SAP2000 FEMA356 parameters used for obtaining pushover curve as Figure 3.13:

Where IR\_SOIL\_2 is design acceleration response spectrum for soil type II base on 2800 standard, which given in Figure 3.8 and damping ratio of 5% and  $SF=0.3 \times 9.806=2.9418g$ .

### 3.5.3.4 Nonlinear static analysis results

For nonlinear static analysis mass center of the roof has been chosen as control point and applied in each load case of nonlinear static analysis, in Figure 3.13 and Figure

3.14, results of the most critic load case (PUSHMODEXG1) are shown both in 3D and in 2D view.

**Figure 3.13 :** Target displacement coefficients used in SAP2000 based on FEMA356

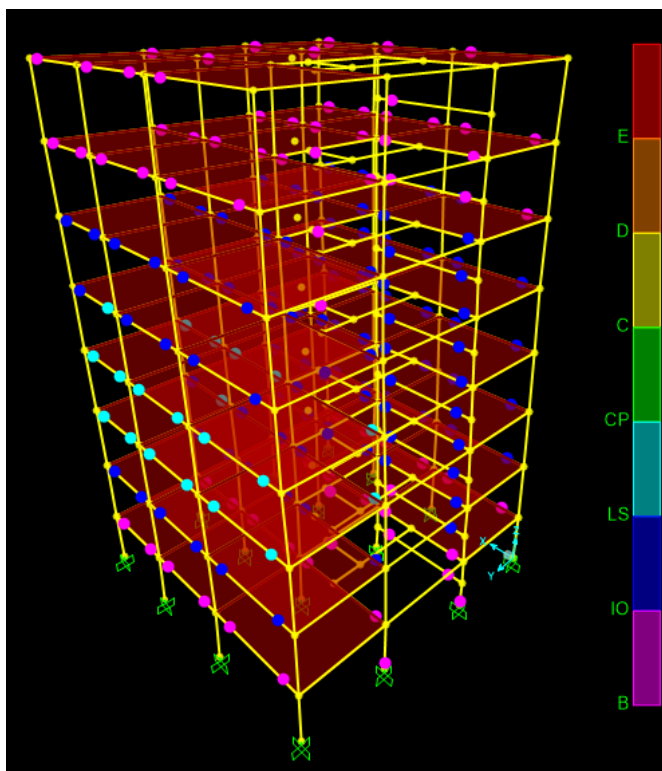


**Figure 3.14 :** Plastic hinge results in 3D view

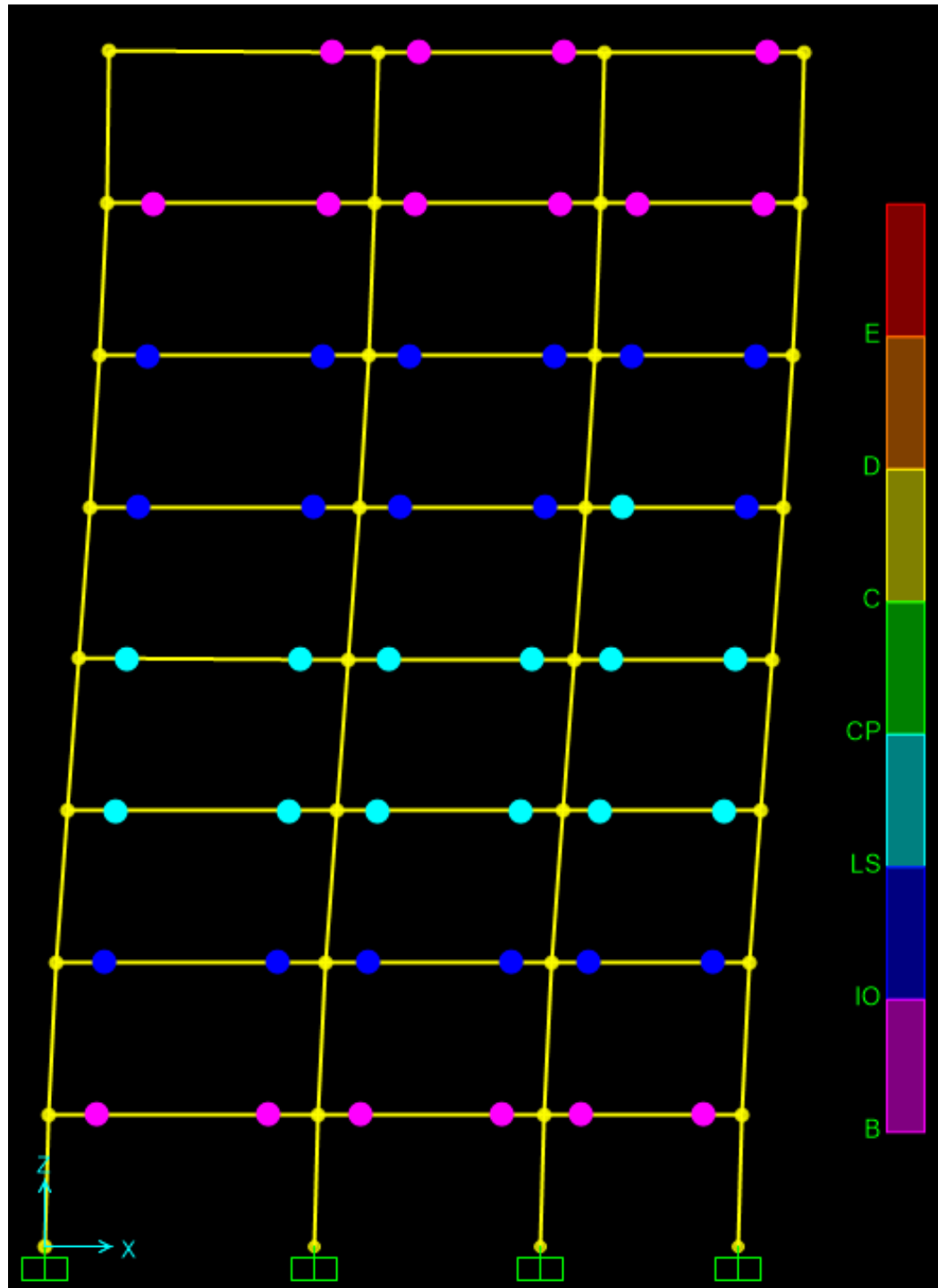


**Figure 3.15 :** Plastic hinge results in 2D view

As shown in Figure 3.14 and 3.15 plastic hinges are induced in story 3 and 4 in collapse prevention level. In Figure 3.16 pushover curve based on FEMA 356 and Figure 3.17 capacity spectrum based on ATC-40 ADRS method has been presented.

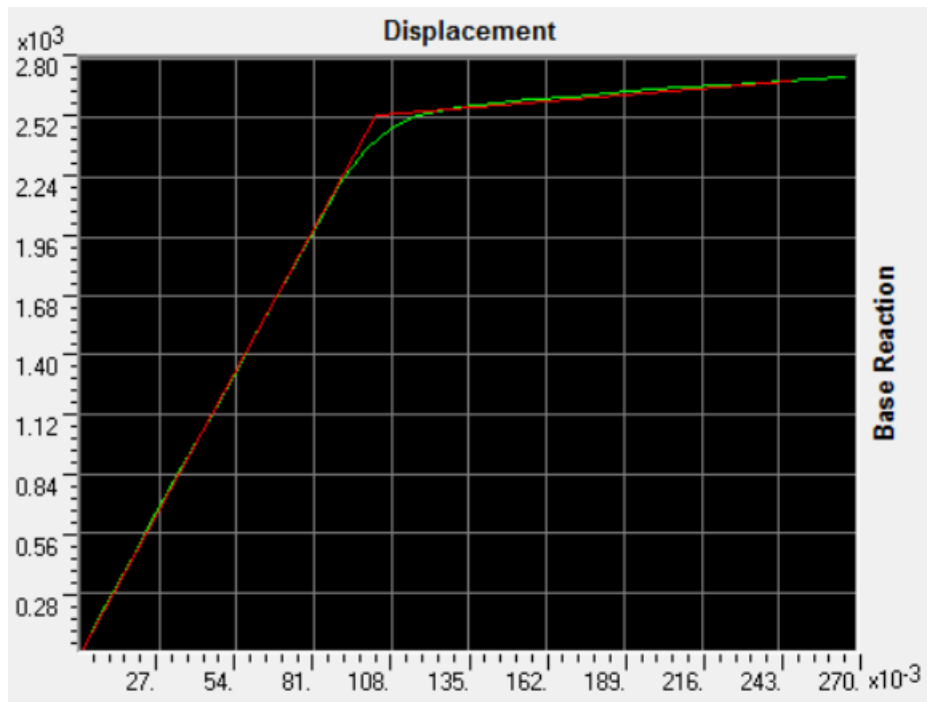


**Figure 3.16 :** Push over curve based on FEMA 356 method

Considering Figure 3.16, base shear (KN) and correspondig displacement (m) has shown for each step of push over analysis respectively.

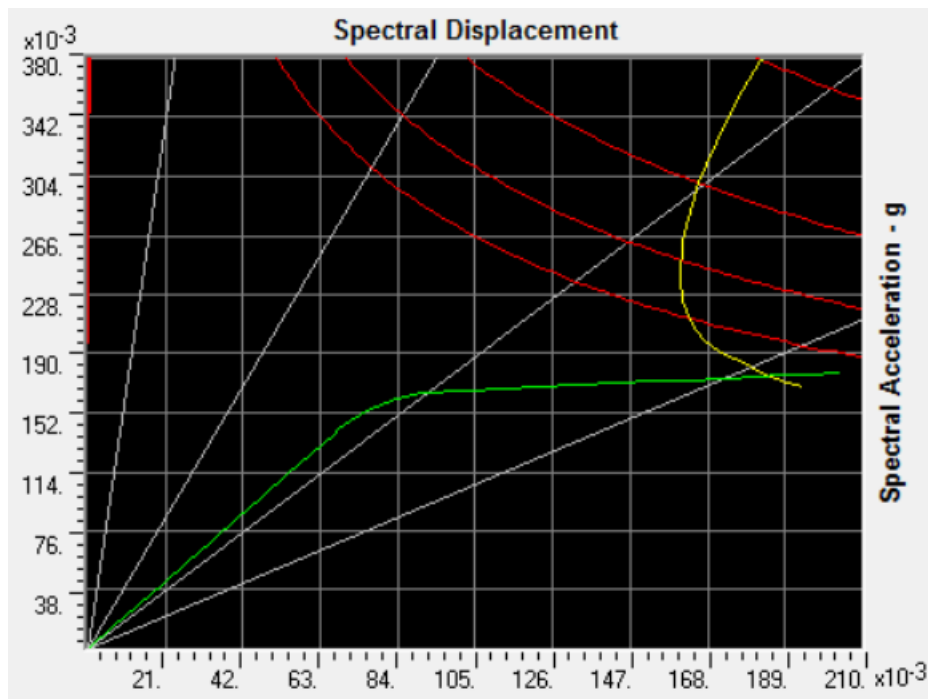


**Figure 3.17 :** Pushover capacity spectrum based on ATC-40

in Figure 3.17 the red lines are design acceleration response spectrum by various damping ratios (5%, 10%, 15% and 20%), yellow curve is known as demand

spectrum, the intersection point of demand spectrum and push over curve(green curve) is performance point, from ATC-40 capacity spectrum the values which given in below can be obtained in performance point:

Base shear (V): 2689.5 KN

Displacement due to shear force (D): 0.24m

Spectral acceleration (Sa): 0.176g

Spectral Displacement (Sd): 0.185m

Effective time (Teff): 2.060sec

Effective damping (Beff): 0.249

#### **3.5.3.5 Structure performance control**

In order to control the performance of building due to modal pushover load, the plastic behavior of hinges in columns and beams presented in Table 3.29 for each step of pushover analysis and corresponding base shear and displacement.

There is 43 hinges are reach to collapse prevention in target displacement so this structure should rehabilitated as following sections.

**Table 3.29 : Plastic hinge results in each step considering base shear(KN) and displacement(m)**

| Step | Displacement | BaseForce | AtoB | BtoIO | IOtoLS | LStoCP | CPtoC | CtoD | DtoE | BeyondE | Total |
|------|--------------|-----------|------|-------|--------|--------|-------|------|------|---------|-------|
|      | m            | KN        |      |       |        |        |       |      |      |         |       |
| 0    | -0.000692    | 0         | 688  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 1    | 0.008208     | 217.297   | 688  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 2    | 0.017108     | 434.594   | 688  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 3    | 0.026008     | 651.892   | 688  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 4    | 0.034908     | 869.19    | 688  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 5    | 0.043808     | 1086.488  | 688  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 6    | 0.052708     | 1303.787  | 688  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 7    | 0.061608     | 1521.086  | 688  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 8    | 0.070508     | 1738.382  | 688  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 9    | 0.079408     | 1955.682  | 688  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 10   | 0.088308     | 2172.982  | 687  | 1     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 11   | 0.089494     | 2201.766  | 682  | 6     | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 12   | 0.098742     | 2375.446  | 634  | 54    | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 13   | 0.108386     | 2477.575  | 583  | 105   | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 14   | 0.116985     | 2533.816  | 550  | 138   | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 15   | 0.131177     | 2570.056  | 527  | 161   | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 16   | 0.145098     | 2591.626  | 516  | 172   | 0      | 0      | 0     | 0    | 0    | 0       | 688   |
| 17   | 0.155979     | 2605.873  | 511  | 171   | 6      | 0      | 0     | 0    | 0    | 0       | 688   |
| 18   | 0.171258     | 2623.763  | 505  | 155   | 28     | 0      | 0     | 0    | 0    | 0       | 688   |
| 19   | 0.183556     | 2637.195  | 503  | 131   | 54     | 0      | 0     | 0    | 0    | 0       | 688   |
| 20   | 0.199162     | 2653.071  | 498  | 107   | 83     | 0      | 0     | 0    | 0    | 0       | 688   |
| 21   | 0.210271     | 2663.794  | 492  | 99    | 97     | 0      | 0     | 0    | 0    | 0       | 688   |
| 22   | 0.223744     | 2675.439  | 490  | 87    | 111    | 0      | 0     | 0    | 0    | 0       | 688   |
| 23   | 0.236154     | 2685.948  | 487  | 82    | 110    | 9      | 0     | 0    | 0    | 0       | 688   |
| 24   | 0.245054     | 2693.365  | 485  | 77    | 108    | 18     | 0     | 0    | 0    | 0       | 688   |
| 25   | 0.253954     | 2700.46   | 484  | 72    | 105    | 27     | 0     | 0    | 0    | 0       | 688   |
| 26   | 0.266308     | 2710.011  | 481  | 72    | 92     | 43     | 0     | 0    | 0    | 0       | 688   |

### 3.5.4 Nonlinear dynamic analysis

Nonlinear dynamic (Time-History) analysis is most natural method which compatible with physical behavior of structure while an earthquake occurs, this method can perform by two way: 1- using seven acceleration time record, 2- using three acceleration time record.

In first case, the average of results for each analysis should calculated to assessment the performance of structure, in second case the most critical results should be choosed

and structure's performance will be determined base on the choosen results.in this project second method is under consideration and 3 records has been used.

#### 3.5.4.1 General requirements

All accelerograms should be selected equivalent by main fault mechanism, almost same earthquake intensity, hypocentral distance to site, geotechnical and tectonical properties and compatibility with design spectrum, which mentioned in section 3.4.2.

Ground motion duration should be at least be 10 seconds or three times of structure's main natural vibration period, which is greater.

-Elastic resoponse spectrum for each earthquake should calculated in both X and Y direction by 5% damping ratio.

-For each earthquake the values of response spectrum in X and Y direction should combined by SRSS method between 0.2T-1.5T intervals (T: main natural vibration period of structure).

-Each couple accelerogram should scaled so that values of combined accelerograms in 0.2T-1.5T intervals not to be 10% less than 1.3 times of design acceleration response spectrum scaling factor for each earthquake should be equal in both direction.

Considering to standard's terms, which mentioned above three accelerograms selected for using in NDP for this project:

**Table 3.30 :** Accelerograms used in NDP

| Event           | Year | Station                       | Mag  | SF   |
|-----------------|------|-------------------------------|------|------|
| Imperial Valley | 1979 | EL Centro Array #13           | 6.53 | 1.14 |
| Loma Prieta     | 1989 | LGPC                          | 6.93 | 0.6  |
| Northridge      | 1994 | Jensen Filter Plant Generator | 6.69 | 0.4  |

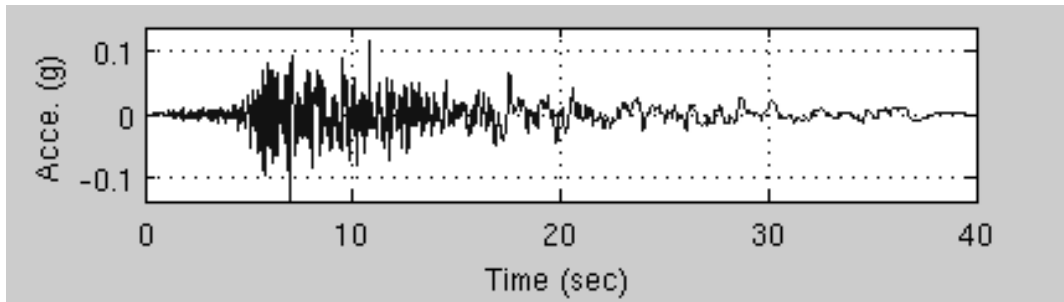


**Figure 3.18 : Imperial Valley (NS)**

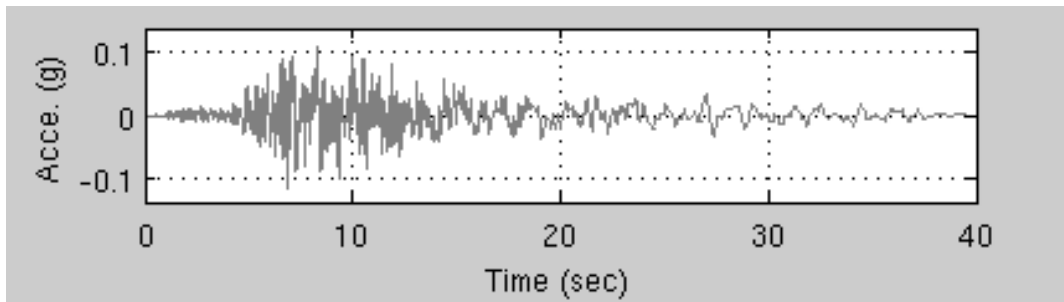


**Figure 3.19 : Imperial Valley (EW)**

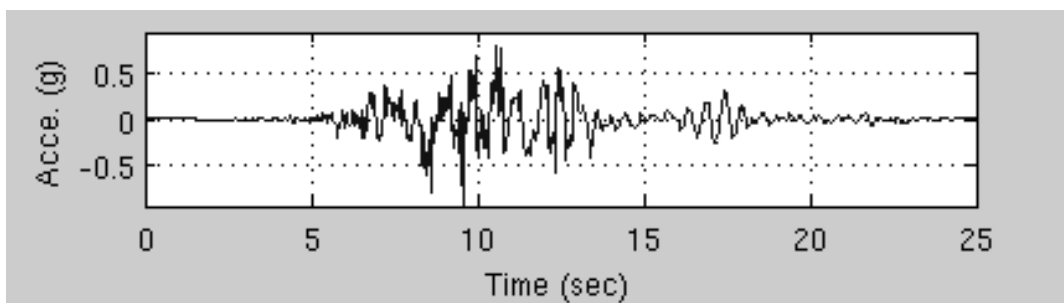


**Figure 3.20 : Loma Prieta (NS)**

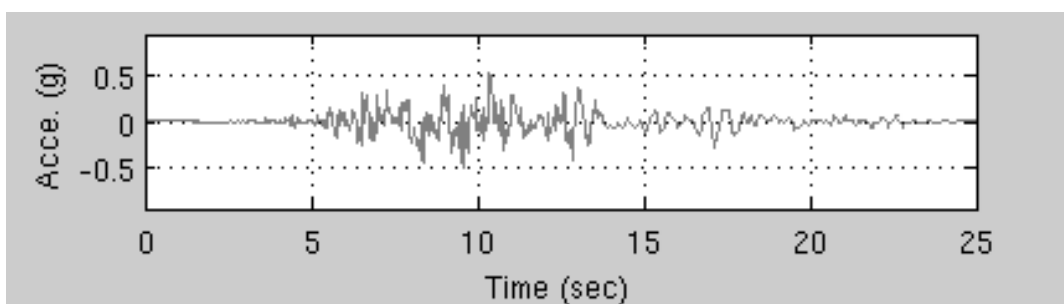


**Figure 3.21 : Loma Prieta (EW)**

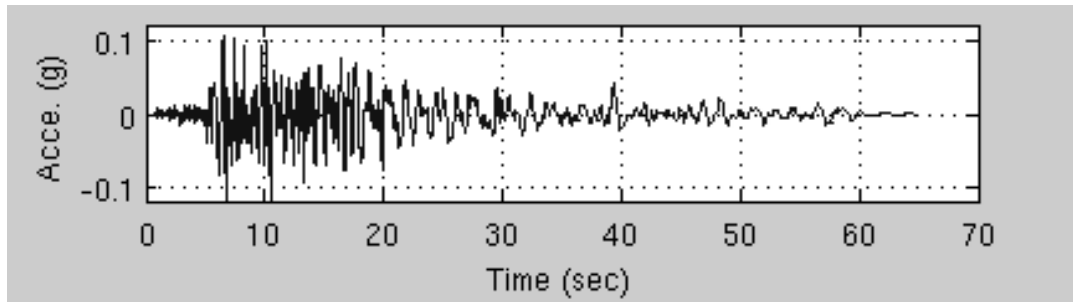


**Figure 3.22 : Northridge (NS)**

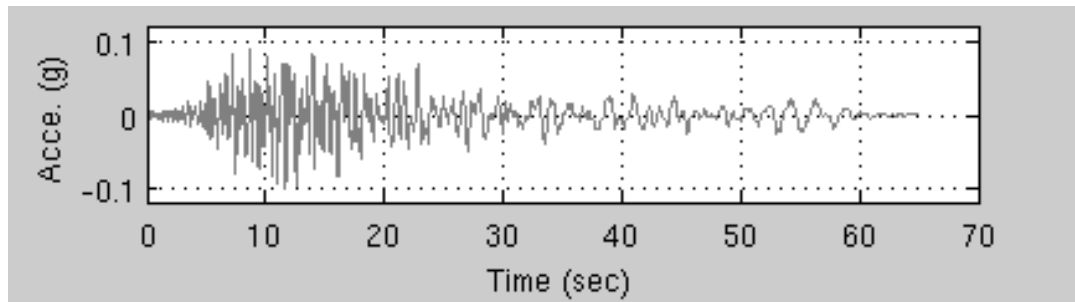


**Figure 3.23 : Northridge (EW)**

As shown in Figure 3.24, three combined and scaled earthquake data compared by their average and in Figure 3.25 compared by design spectrum based on IRAN and TURKEY earthquake codes.

#### **3.5.4.2 Nonlinear time-history analysis based on determined accelerograms**

All proposed accelerograms defined as a function in SAP2000 program. nonlinear time-history analysis has been proceed by considering nonlinear gravitational loads as initial condition and each nonlinear time-history case in continue of mentioned initial conditions and then each of these three earthquake records has been analyzed for both X and Y direction.

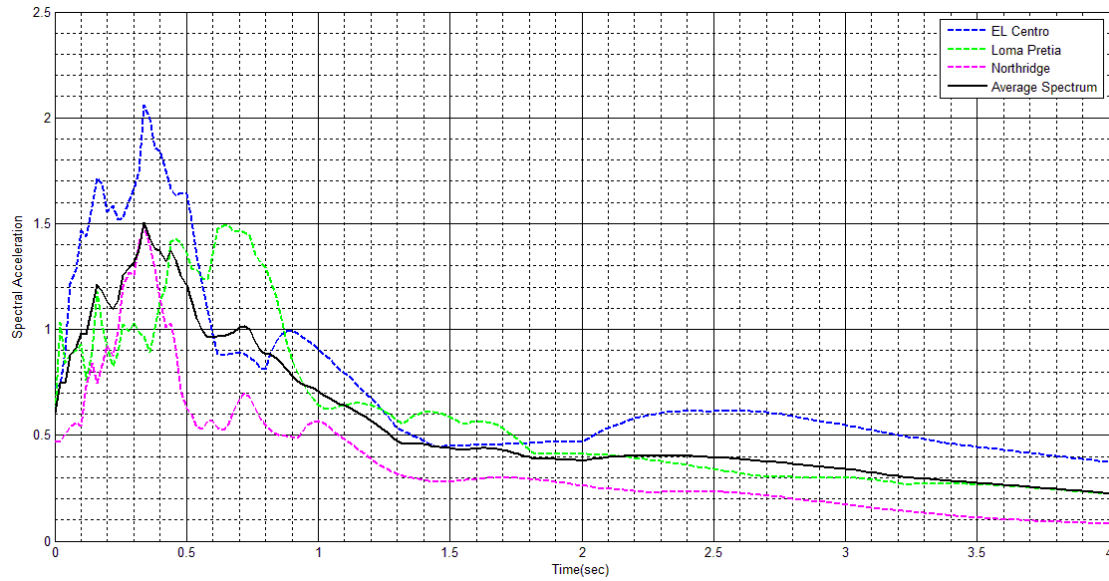


**Figure 3.24 :** Comparison between response spectrums and average spectrum

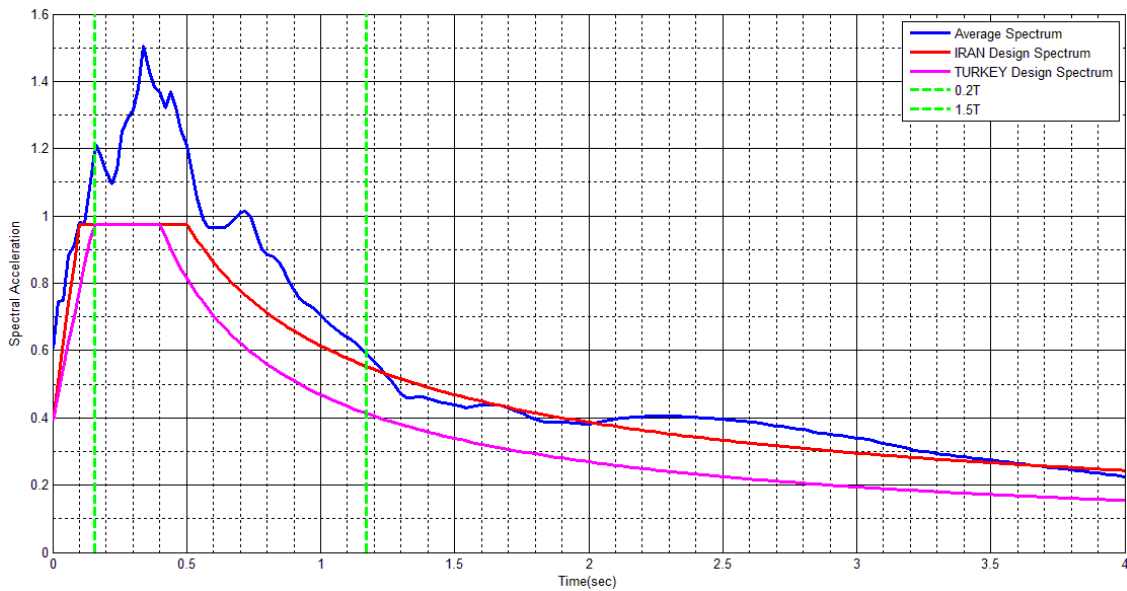


**Figure 3.25 :** Comparison between response spectrums and design spectrum based on both IRAN and TURKEY code

Analysis processed by direct integration as mathematical solution method, mass and stiffness proportional damping calculated as shown Figure 3.26 first period is the average of first three period of free vibration, and second period is average of the rest of free vibration's periods. so mass proportional coefficient calculated equal to 0.4037 and stiffness proportional coefficient calculated equal to 2.724E-03. For time integration parameters newmark method proposed by using  $\Gamma=0.5$  and  $\beta=0.25$ .

**Mass and Stiffness Proportional Damping**

Damping Coefficients

☐ Direct Specification

☒ Specify Damping by Period

☐ Specify Damping by Frequency

|        | Period                              | Frequency                     | Damping                           |
|--------|-------------------------------------|-------------------------------|-----------------------------------|
| First  | <input type="text" value="1.3607"/> | <input type="text" value=""/> | <input type="text" value="0.05"/> |
| Second | <input type="text" value="0.1958"/> | <input type="text" value=""/> | <input type="text" value="0.05"/> |

**Figure 3.26 : Mass and stiffness proportional damping**

### 3.5.4.3 Load case definition

Load cases applied to the computer model considering scale factors as shown in Table 3.31, and for eccentricity in each load case 30% of the other lateral component of that earthquake record considered.all records multiplied by 9.806 to convert (g) to ( $cm/sec^2$ ) .

**Table 3.31 : Load case definition in SAP2000**

| Name         | Load Type    | Analysis Type | Initial Condition | Function                         | Scale Factor |
|--------------|--------------|---------------|-------------------|----------------------------------|--------------|
| NORTH1-1     | Time-History | Nonlinear     | PUSHG1            | Northridge NS+0.3Northridge EW   | 0.4x9.806    |
| NORTH1-2     | Time-History | Nonlinear     | PUSHG1            | Northridge EW+0.3Northridge NS   | 0.4x9.806    |
| NORTH2-1     | Time-History | Nonlinear     | PUSHG2            | Northridge NS+0.3Northridge EW   | 0.4x9.806    |
| NORTH2-2     | Time-History | Nonlinear     | PUSHG2            | Northridge EW+0.3Northridge NS   | 0.4x9.806    |
| EL CENTRO1-1 | Time-History | Nonlinear     | PUSHG1            | EL CENTRO NS+0.3EL CENTRO EW     | 1.14x9.806   |
| EL CENTRO1-2 | Time-History | Nonlinear     | PUSHG1            | EL CENTRO EW+0.3EL CENTRO NS     | 1.14x9.806   |
| EL CENTRO2-1 | Time-History | Nonlinear     | PUSHG2            | EL CENTRO NS+0.3EL CENTRO EW     | 1.14x9.806   |
| EL CENTRO2-2 | Time-History | Nonlinear     | PUSHG2            | EL CENTRO EW+0.3EL CENTRO NS     | 1.14x9.806   |
| LOMA 1-1     | Time-History | Nonlinear     | PUSHG1            | LOMA PRIETA NS+0.3LOMA PRIETA EW | 0.6x9.806    |
| LOMA 1-2     | Time-History | Nonlinear     | PUSHG1            | LOMA PRIETA EW+0.3LOMA PRIETA NS | 0.6x9.806    |
| LOMA 2-1     | Time-History | Nonlinear     | PUSHG2            | LOMA PRIETA NS+0.3LOMA PRIETA EW | 0.6x9.806    |
| LOMA 2-2     | Time-History | Nonlinear     | PUSHG2            | LOMA PRIETA EW+0.3LOMA PRIETA NS | 0.6x9.806    |

### 3.5.4.4 Analysis Results

After proceed nonlinear time-history analysis for each earthquake and comparing results, the most critical load case has selected as criterion to control sturcture performance, which in this project the NORTH1-2 selected as most critical load case so results for this load case presented as follow.

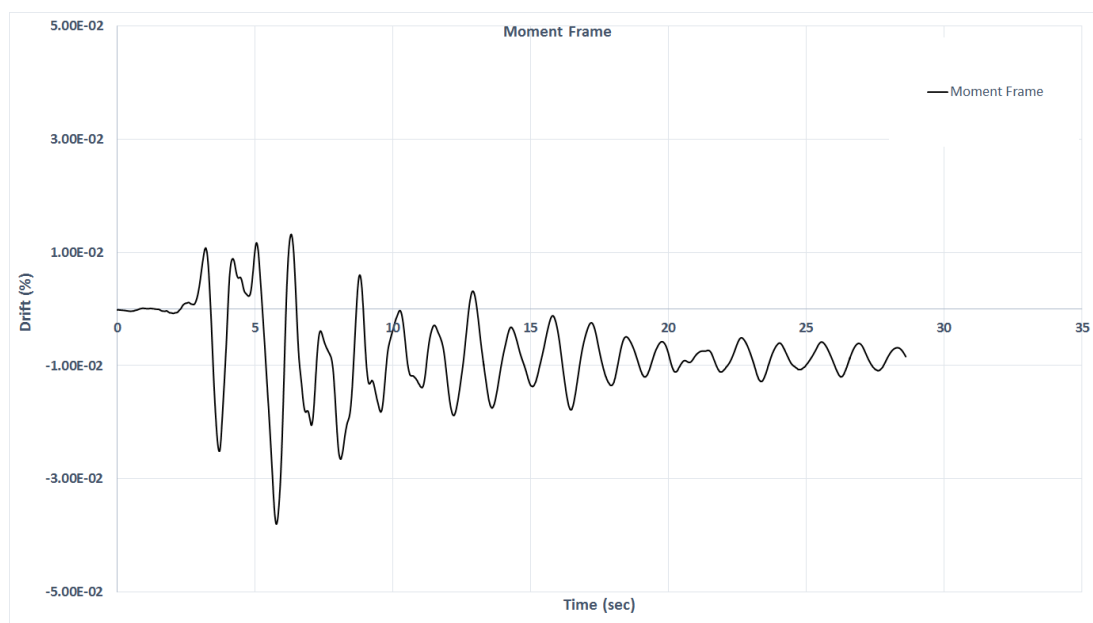


**Figure 3.27 : Story drift for roof level**

According to IR360, standard sroy drift for nonlinear analysis could be 1.2 times of story drift in linear analysis in this project maximum drift should be  $1.2 \times 0.02 = 0.024$  but as shown in Figure 3.27 drift exceeds 0.03 in negative direction so the structure should rehabilitated due to nonlinear time-history analysis too.

### **3.6 Seismic Rehabilitation of Building**

Rehabilitation design of this building will be done based on FEMA 547 code chapter 12, which covers moment concrete frame buildings (TYPE C1).

#### **3.6.1 Detailed description of techniques**

##### **3.6.1.1 Adding steel braced frame**

Deficiencies addressed by the rehabilitation technique: Inadequate global shear capacity, inadequate lateral displacement (global stiffness) capacity. In this project, inadequate lateral displacement is considered.

Addition of steel braces to an existing concrete moment frame building is a method of adding strength and/or stiffness to the structural system. The steel braces can be added without a significant increase in the building weight.

Most designs of braced frame retrofits will be governed by maintaining drifts within the range of acceptability for the existing concrete elements. This can be accomplished by setting up a model that includes both the stiffness of the braced frame and of the concrete frame and meeting acceptability requirements for the displacements (or pseudo forces) in the concrete elements. Some engineers prefer to consider only the braced frames as a new lateral system, determine real drift demand for that system, and then check that drift for acceptability superimposed on the existing frame.

The new braces may be located on the exterior or interior of the building. An exterior location generally allows for easier construction access and perhaps less cost, but is visible, exposed to the environment and probably will impact exterior building finishes. Braces placed parallel to the façade can be connected to the exterior faces of perimeter spandrel beams, perimeter moment frames or edges of floor and roof diaphragms relatively easily, but will most likely cross in front of some windows. Alternatively, exterior bracing may be placed as buttresses, perpendicular to the existing façade. This configuration will probably require more extensive new collectors to deliver lateral forces from the diaphragms but may allow creation of new stair or elevator shafts, or perhaps additional floor area. For projects that include

expansion of or additions to the existing building, the new braces could be located in the adjacent new construction, tied to the existing building.

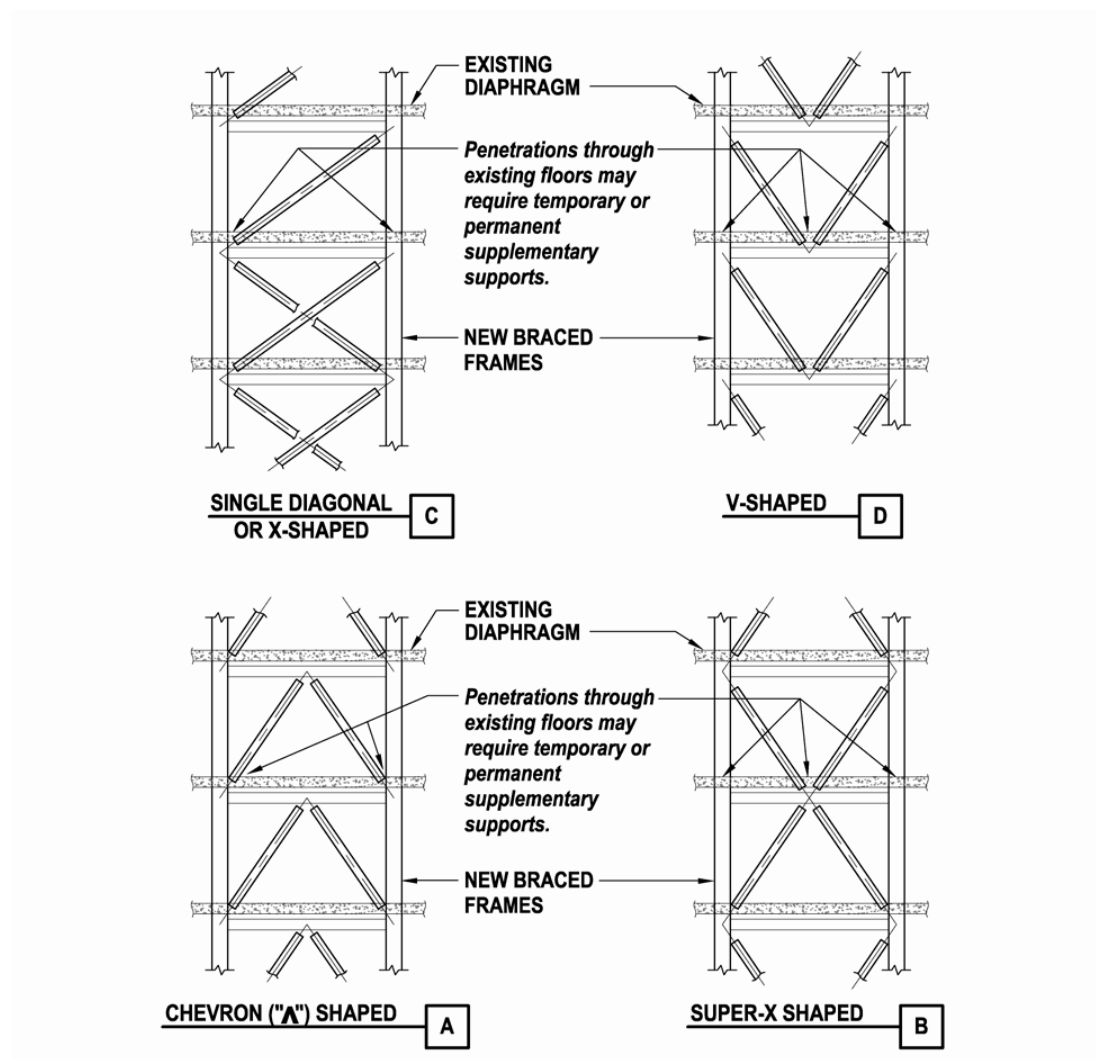


**Figure 3.28** : Typical braced frame configurations

Interior braces will most commonly be located along existing frame lines, particularly at moment frame bays. This will allow for best use of any existing diaphragm chords and collectors and for best moment frame – braced frame interaction. In some cases however, interior braces will be located offset from existing column-frame lines to minimize direct impact on existing structural or architectural components or to simplify the frame-diaphragm connections.

The addition of new braced frames to a building will always impact the architectural character and functional uses of the building to some degree. Selection of preferred brace locations must be made considering these issues, such as space layout, corridor

locations, doorways, windows, main M/E/P distribution runs, as well as the structural or construction considerations.

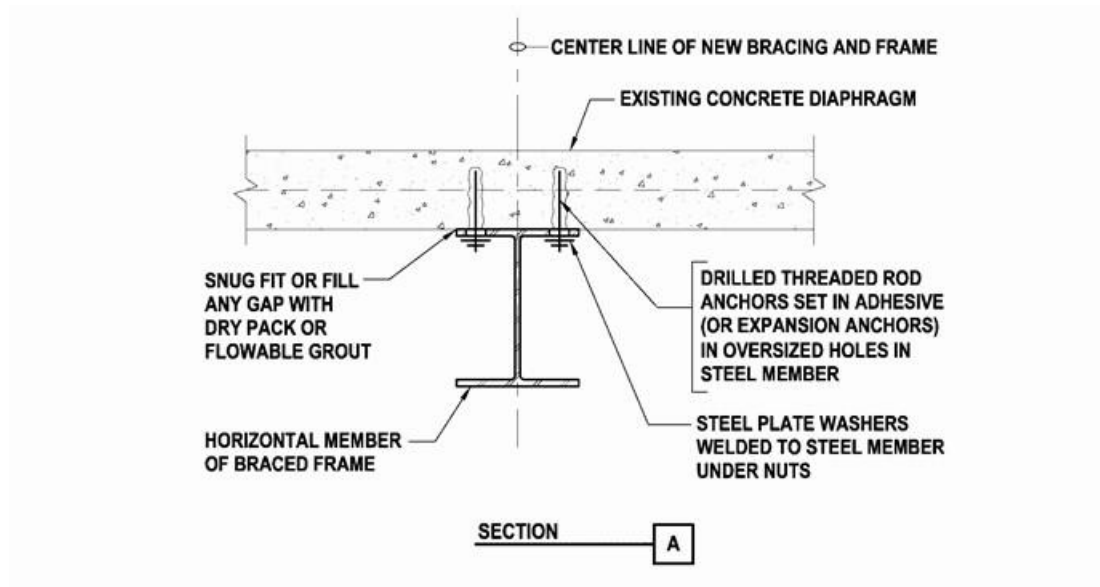


**Figure 3.29 :** Typical connection to concrete diaphragm

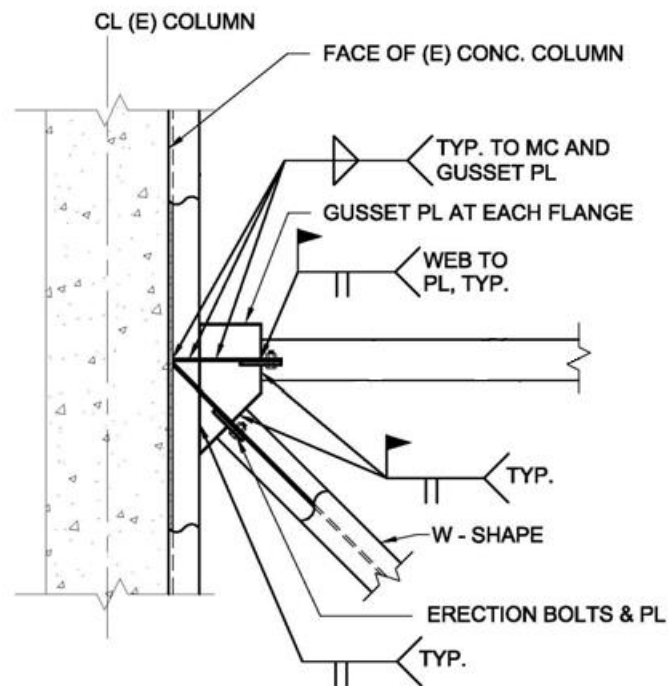


**Figure 3.30 :** Elevation

The cost and level of disruption associated with installation of steel bracing is generally less than that of shear walls. The number of penetrations that need to be cut through the existing concrete structure and of drilled dowels and anchors may be less than for the shear wall alternative, and the work is generally not as wet or messy. Also, it will not be necessary to prepare any existing concrete surfaces that will be in contact

with new steel members. The new members are discrete and welded or bolted connections are localized. However, there will be noise and vibrations resulting from the required cutting and drilling that will make continued occupancy difficult. Invariably, some architectural and M/E/P system components will require relocation or replacement.

### **3.6.1.2 Adding reinforced concrete shear wall**

Deficiencies Addressed by the Rehabilitation Technique: Inadequate global shear capacity, inadequate lateral displacement (global stiffness) capacity. In this project, inadequate lateral displacement is under consideration.

Addition of shear walls to an existing concrete frame building is a common method of adding significant strength and/or stiffness to the structure. The new walls may be of cast-in-place concrete, shotcrete or fully grouted concrete masonry unit (CMU) construction.

A primary design consideration is determination of whether or not the existing concrete frames may be used as an effective part of a combined system. Are the existing frame columns strong enough and/or well detailed enough to serve as the chord/boundary member of a shear wall without improvement? Are the frame beams detailed well enough to serve as coupling beams or to be incorporated into the wall itself? These considerations may limit the choices of wall-frame physical relationship: that is, should the walls be placed 1) within the plane of the existing concrete frames, 2) as vertically continuous walls alongside of, and joined to, the existing frames, or 3) as separate vertical elements independent of the frames? The first alternative is often best avoided as noted in the Detailing Considerations discussion below. Considering alternates 2 and 3, it must be determined if the frames are capable of becoming part of the shear wall (primarily as chord elements) or if it is beneficial to prevent direct interaction by placing the shear walls free of the existing concrete frame elements. In some cases, it is not feasible to stiffen the building into the range of acceptable deformation of

the existing frames, and improvement in deformation capacity may be required in addition to the addition of new walls.

: The new walls may be placed on the exterior or interior of the building. An exterior location generally allows for easier construction access and perhaps less cost, but is visible, exposed to the environment and may impact exterior building finishes. Walls placed parallel to the façade can be connected to the exterior edges of floor and roof diaphragms or perimeter concrete frames relatively easily, but will most likely require closure or reduction in size of some windows. Alternatively, exterior walls may be placed as buttresses perpendicular to the existing façade. This configuration will probably require more extensive new collectors to deliver lateral forces from the diaphragms but may allow creation of new stair or elevator shafts, or even of additional floor area. For projects that include expansion of or additions to the existing building, the new walls could be located in the adjacent new construction.

Interior walls located along frame lines, particularly at moment frame bays will often allow for best use of any existing diaphragm chords and collectors. Beams that frame directly into the ends of new walls may behave like coupling beams as described above. In some cases, interior walls are better located offset from existing column-frame lines to minimize direct impact on existing structural or architectural components or to simplify the wall-diaphragm connections.

The addition of shear walls to a building will always impact the architectural character and functional uses of the building. Selection of preferred wall locations must be made considering these issues, such as space layout, corridor locations, doorways, windows, main M/E/P distribution runs, as well as the structural or construction considerations.

The most significant detail associated with installing a new shear wall in a concrete building is the connection at the top of the new wall to the underside of the existing concrete diaphragm overhead. The construction joint must be made tight, without any gapping, to facilitate transfer of shear forces from the overhead diaphragm into the new wall below and to minimize the possibility of joint slip.

The vertical dowels must be sufficient to transfer forces from the existing diaphragm and from the new wall above (if it exists), to the lower wall. Shears can also be transferred across this joint with large diameter pipes or structural shapes. The holes made through the existing slab must serve not only to install the dowels, but also to allow for placement and consolidation of the wall concrete. The concrete head created

by placement up to the top of slab coupled with cleaning and roughening the existing concrete contact surface by either sandblasting or chipping will provide the best joint available.

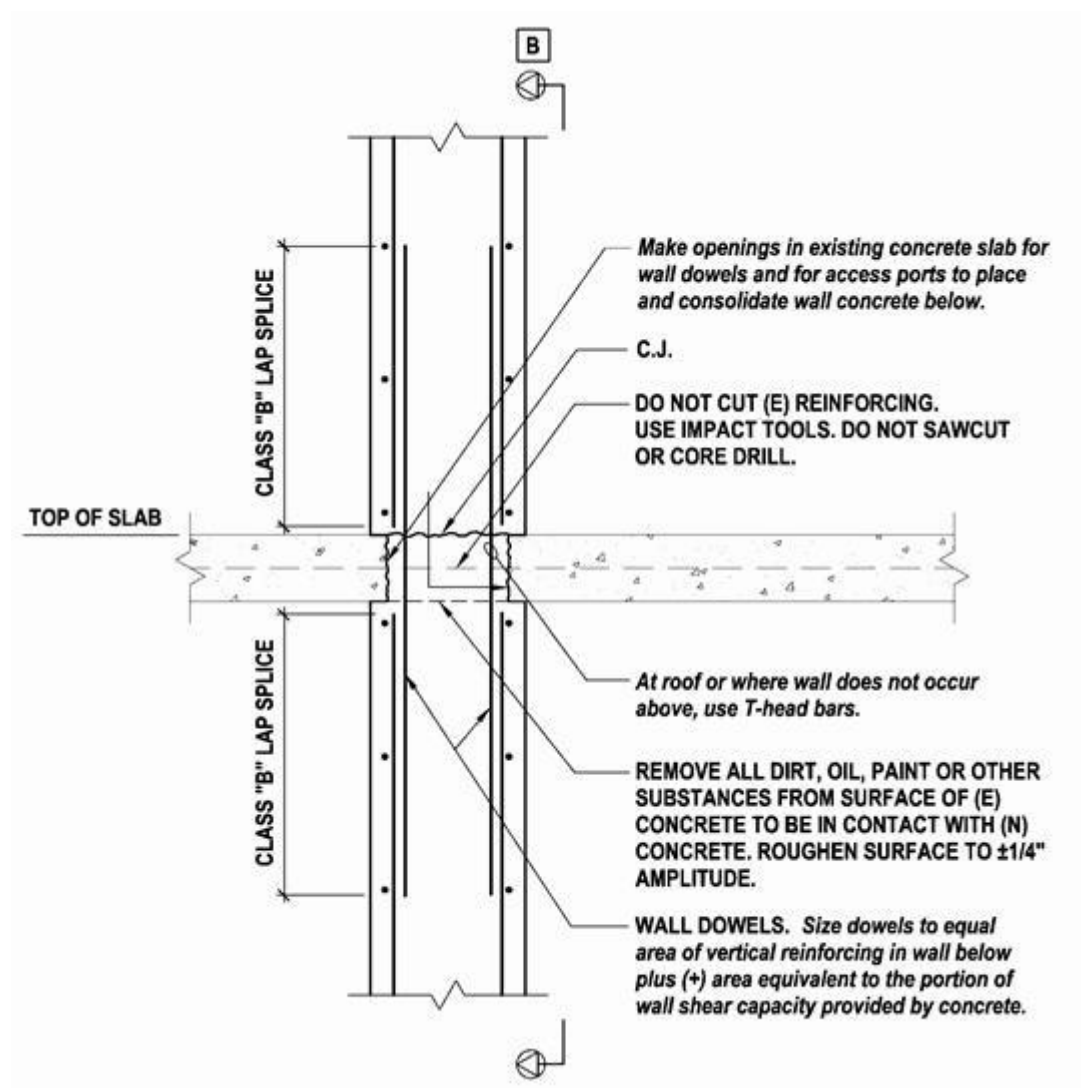


**Figure 3.31** : Concrete wall connection to concrete slab

The larger holes through the slab will also be more like intermittent shear keys. The holes should be drilled or made with impact tools instead of saws or core drills to avoid cutting or damaging existing slab reinforcement. Prior to cutting the holes, temporary shores may be required below the slab along each side of the row of holes. The concrete should be placed through the slab openings into the forms below, up to top of slab, to provide some head on the joint at the underside of the diaphragm.

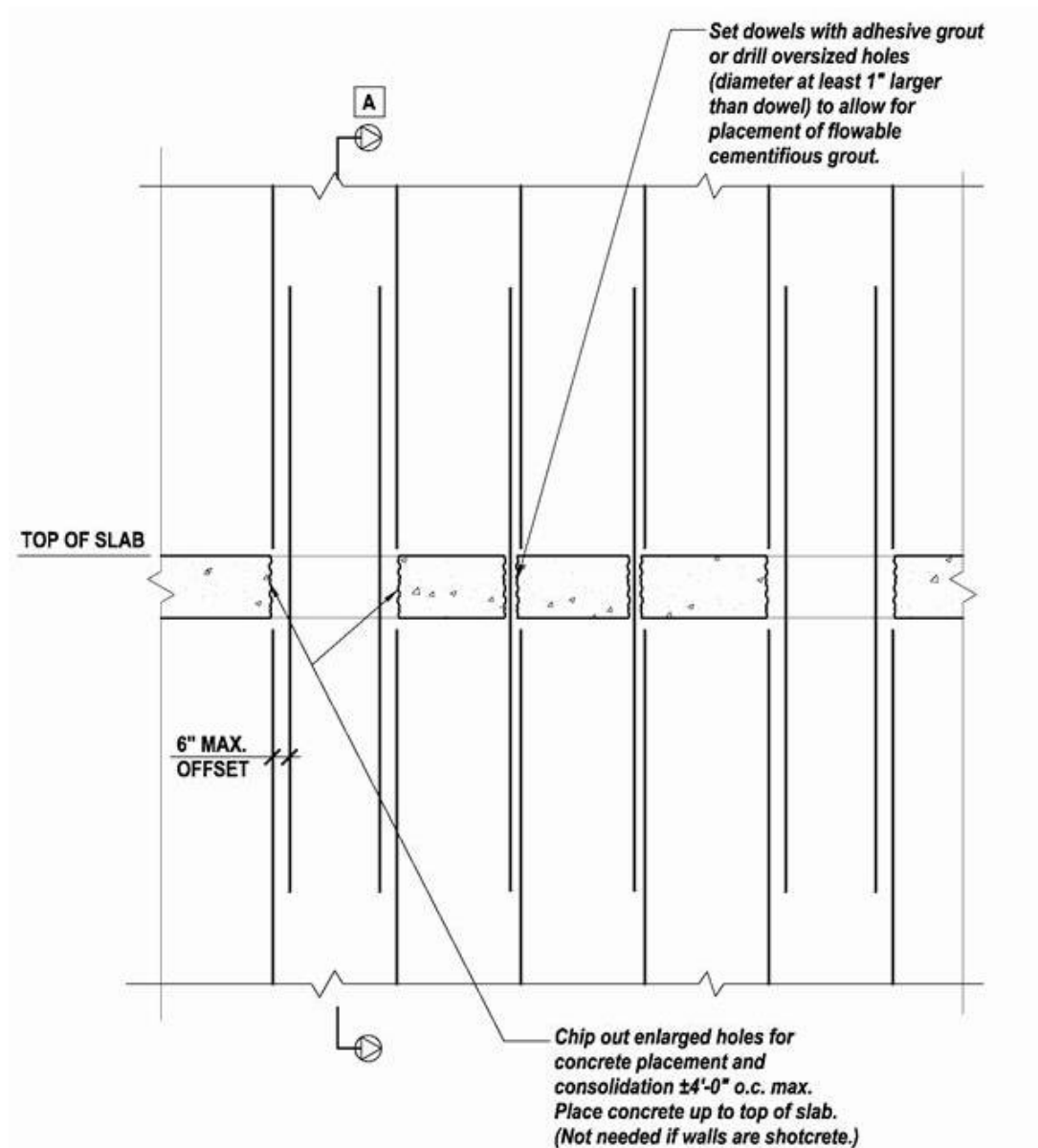


**Figure 3.32 :** Concrete wall connection to concrete slab (partial elevation view)

Construction of new shear walls in an existing building can be very disruptive to any building occupants. Noise, vibration, and dust associated with many operations, especially cutting holes through and drilling dowels into concrete, can be transmitted throughout a concrete structure. Placing cast-in-place concrete is a wet process and very messy. Also, excavation and drilling operations and the use of mechanized and/or truck mounted equipment associated with installation of new foundations can be very disruptive.

### 3.6.2 Rehabilitation using braces

In this project structure rehabilitated by two methods which described in section 3.5.1.1 and section 3.5.1.2 and analysis results will given in following sections.

#### 3.6.2.1 Braces location

In this project adding X-braces in both direction proposed and different sections has tested and eventually 2UPN300 has been chosen as appropriate section, location of braces shown in Figure 3.33 and Figure 3.34, in plan.

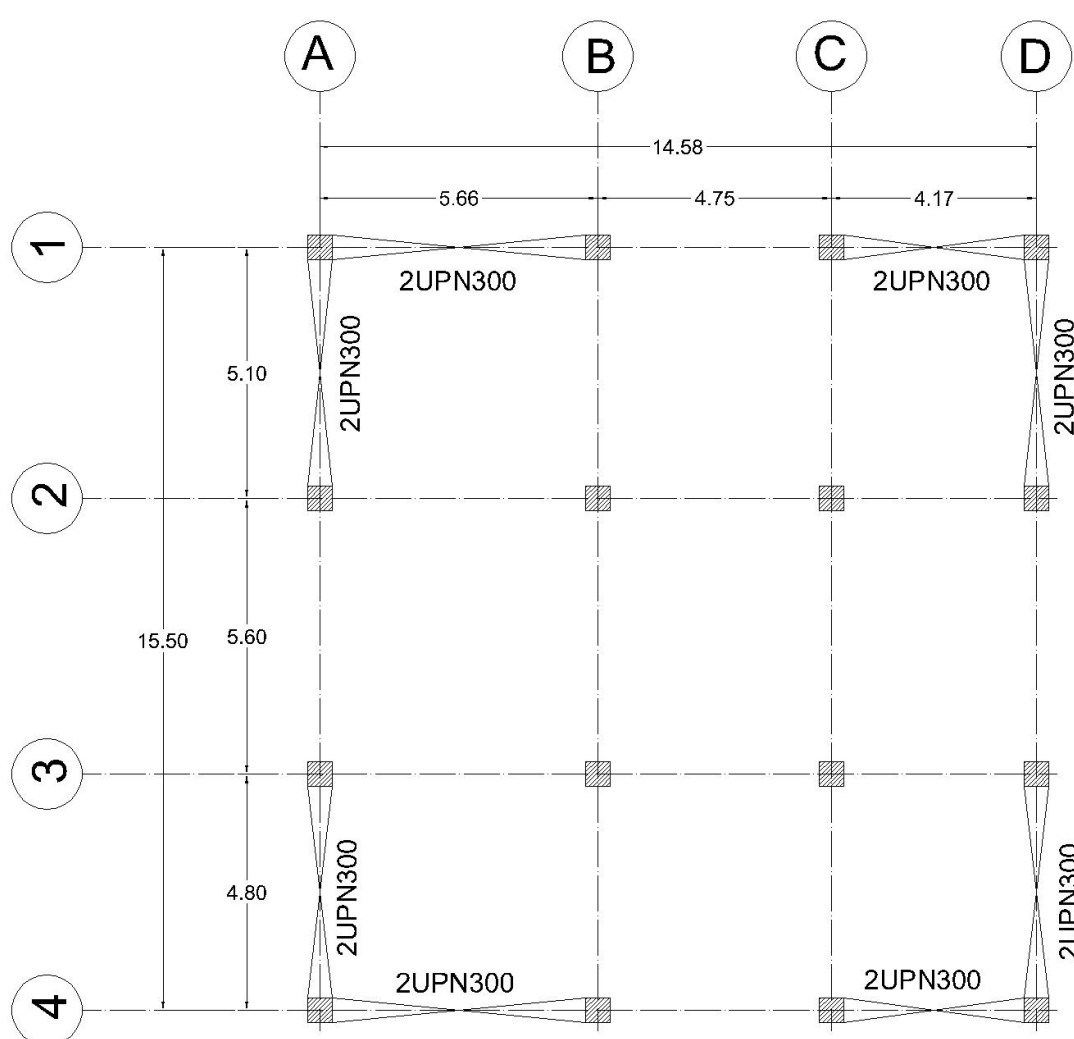


**Figure 3.33 : Location of braces in plan (Type 1)**

#### 3.6.2.2 Section properties

Sections are include two UPN300 steel profile, and their mechanical properties are given in Figure 3.35.

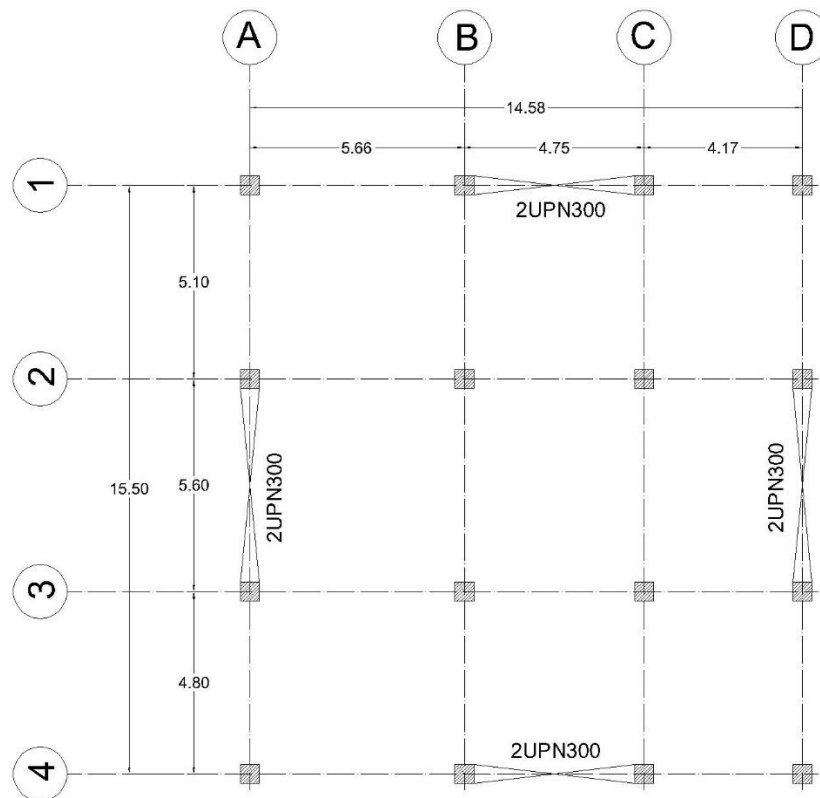


**Figure 3.34 : Location of braces in plan (Type 2)**

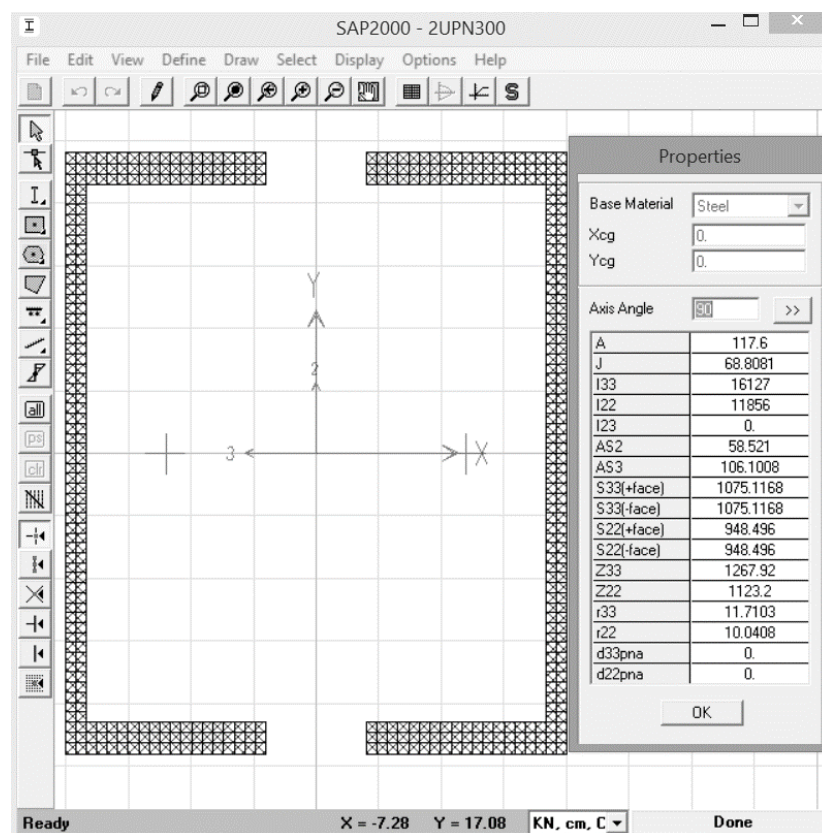


**Figure 3.35 : 2UPN300 profile and section properties**

### 3.6.2.3 Plastic hinge definition for 2UPN300 sections

To defining plastic hinges for steel braces, data which given in Table 3.32, from FEMA356 and IR360 standards used. Since in this project 2UPN section has been used so from Table 3.32, depend on buckling state, one of sections (d) or (e) are should be used for compression. So first step in plastic hinge definition is determining buckling state of section therefore radius of gyration the section will be taken from Figue3.35.

Properties which given in Figure 3.35 as  $r_{22}$  for out-of-plane and  $r_{33}$  for in-plane, then effective length ratio (K) should determined due to IR360 standard which for X-bracing system which barces intersects each other in the middle of their length K is equal to 0.5 for in-plane buckling and 0.67 for out-of-plane buckling.

Determining buckling state:

$$\Rightarrow \left\{ \begin{array}{l} \text{Out -of -Plane} : \frac{K}{r_{22}} = \frac{0.67}{10.0408} = 0.0667 \\ \text{IN -Plane} : \frac{K}{r_{33}} = \frac{0.5}{11.7103} = 0.0426 \end{array} \right\}$$

So out-of-plane buckling is more critical than in-plane state and the values in section (e) which is related to sections with out-of-plane bucklinge should be used for plastic hinges definition given as follow.

Modelling parameters:

Acceptance criteria:

$$a = 0.5\Delta_c$$

$$IO = 0.25\Delta_c$$

$$b = 8\Delta_c$$

$$LS = 4.0\Delta_c$$

$$c = 0.2$$

$$CP = 6.0\Delta_c$$

In addition, for brace elements in tension due to Table 3.32:

Modelling parameters:

Acceptance criteria:

$$a = 11\Delta_T$$

$$IO = 0.25\Delta_T$$

$$b = 14\Delta_T$$

$$LS = 7.0\Delta_T$$

$$c = 0.8$$

$$CP = 9.0\Delta_T$$

**Table 3.32 : Modeling parameters and acceptance criteria for nonlinear procedures—structural steel components**

| Component/Action                                                                                                               | Modeling Parameters                 |      |                         | Acceptance Criteria |         |     |           |      |
|--------------------------------------------------------------------------------------------------------------------------------|-------------------------------------|------|-------------------------|---------------------|---------|-----|-----------|------|
|                                                                                                                                | Plastic Deformation                 |      | Residual Strength Ratio | Plastic Deformation |         |     |           |      |
|                                                                                                                                |                                     |      |                         | IO                  | Primary |     | Secondary |      |
|                                                                                                                                | a                                   | b    | c                       |                     |         | LS  | CP        | LS   |
| Braces in Compression(except EBF braces)                                                                                       |                                     |      |                         |                     |         |     |           |      |
| a. Double angles buckling in-plane                                                                                             | 0.5Δc                               | 9Δc  | 0.2                     | 0.25Δc              | 5Δc     | 7Δc | 7Δc       | 8Δc  |
| b. Double angles buckling out-of-plane                                                                                         | 0.5Δc                               | 8Δc  | 0.2                     | 0.25Δc              | 4Δc     | 6Δc | 6Δc       | 7Δc  |
| c. W or I shape                                                                                                                | 0.5Δc                               | 8Δc  | 0.2                     | 0.25Δc              | 5Δc     | 7Δc | 7Δc       | 8Δc  |
| d. Double channels buckling in-plane                                                                                           | 0.5Δc                               | 9Δc  | 0.2                     | 0.25Δc              | 5Δc     | 7Δc | 7Δc       | 8Δc  |
| e. Double channels buckling out-of-plane                                                                                       | 0.5Δc                               | 8Δc  | 0.2                     | 0.25Δc              | 4Δc     | 6Δc | 6Δc       | 7Δc  |
| f. Concrete-filled tubes                                                                                                       | 0.5Δc                               | 7Δc  | 0.2                     | 0.25Δc              | 4Δc     | 6Δc | 6Δc       | 7Δc  |
| g. Rectangular cold-formed tubes                                                                                               |                                     |      |                         |                     |         |     |           |      |
| $1.\frac{d}{t} \leq \frac{90}{\sqrt{F_y}}$                                                                                     | 0.5Δc                               | 7Δc  | 0.4                     | 0.25Δc              | 4Δc     | 6Δc | 6Δc       | 7Δc  |
| $2.\frac{d}{t} \geq \frac{190}{\sqrt{F_y}}$                                                                                    | 0.5Δc                               | 3Δc  | 0.2                     | 0.25Δc              | 1Δc     | 2Δc | 2Δc       | 3Δc  |
| $3.\frac{90}{\sqrt{F_y}} \leq \frac{d}{t} \leq \frac{190}{\sqrt{F_y}}$                                                         | Linear interpolation shall be used. |      |                         |                     |         |     |           |      |
| h. Circular hollow tubes                                                                                                       |                                     |      |                         |                     |         |     |           |      |
| $1.\frac{d}{t} \leq \frac{1500}{F_y}$                                                                                          | 0.5Δc                               | 9Δc  | 0.4                     | 0.25Δc              | 4Δc     | 6Δc | 5Δc       | 8Δc  |
| $2.\frac{d}{t} \geq \frac{6000}{F_y}$                                                                                          | 0.5Δc                               | 3Δc  | 0.2                     | 0.25Δc              | 1Δc     | 2Δc | 2Δc       | 3Δc  |
| $3.\frac{1500}{F_y} \leq \frac{d}{t} \leq \frac{6000}{F_y}$                                                                    | Linear interpolation shall be used. |      |                         |                     |         |     |           |      |
| Braces in Tension (except EBF braces)                                                                                          | 11ΔT                                | 14ΔT | 0.8                     | 0.25ΔT              | 7ΔT     | 9ΔT | 11ΔT      | 13ΔT |
| Beams, Columns in Tension (except EBF beams, columns)                                                                          | 5ΔT                                 | 7ΔT  | 1.0                     | 0.25ΔT              | 3ΔT     | 5ΔT | 6ΔT       | 7ΔT  |
| 1. Δc is the axial deformation at expected buckling load.<br>2. ΔT is the axial deformation at expected tensile yielding load. |                                     |      |                         |                     |         |     |           |      |

Next step is calculating values which given above based on force-deformation diagram including A,B,C,D,E coordinate , in Figure , a generalized force- deformation diagram of steel elements is shown,in this diagram  $Q$  is generalized force ,  $Q_y$  is first yield strength in element,  $\Delta$  and  $\theta$  are deformations.the diagram produces elastic and plastic behavior for a steel element.

In this diagram, point A is zero for both force and deformation; in point B the ratio of  $Q$  to  $Q_y$  is equal to 1, means that the first yielding occurs, in this point deformation assumed to be zero. Form point B to C effects of strain hardening, considered by 3% slope.

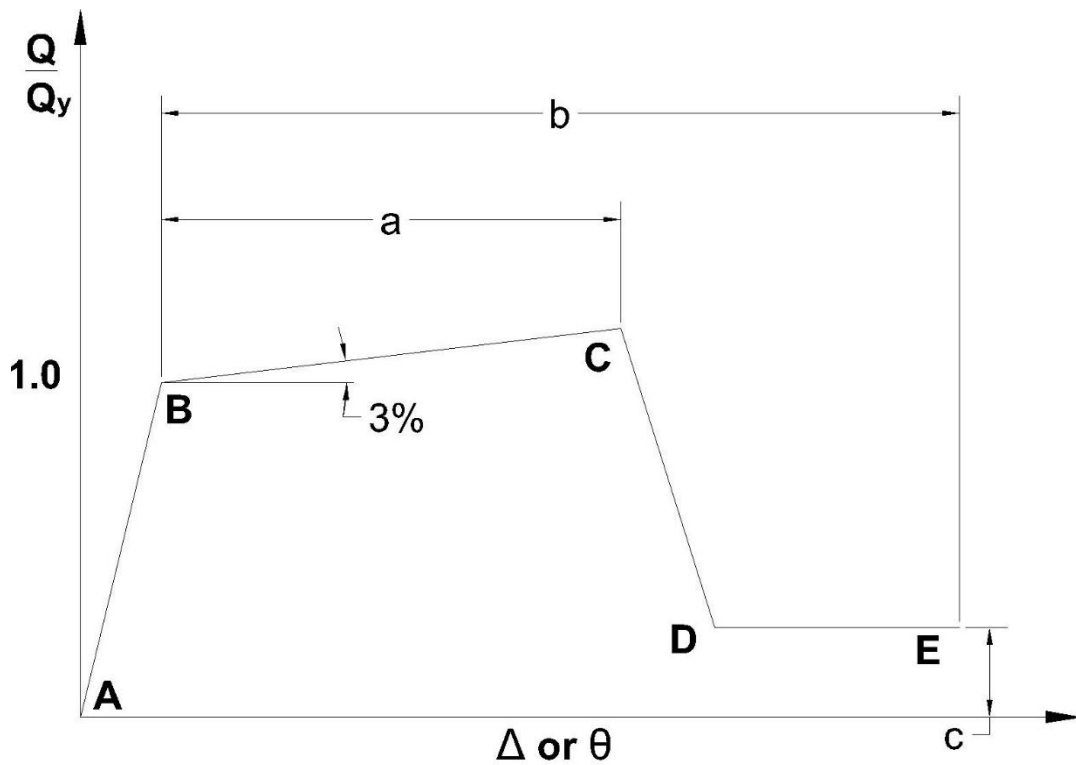


**Figure 3.36 :** Generalized force-deformation diagram

So plastic hinge values are as follow:

For element in compression:

$$\begin{cases} \Delta_A=0 \\ F_A=0 \end{cases} \quad \begin{cases} \Delta_B=0 \\ F_B=1 \end{cases} \quad \begin{cases} \Delta_C=0.5 \\ F_C=1+0.5(0.03)=1.015 \end{cases} \quad \begin{cases} \Delta_D=0.5 \\ F_D=0.2 \end{cases} \quad \begin{cases} \Delta_E=8.0 \\ F_E=0.2 \end{cases}$$

For element in tension:

$$\begin{cases} \Delta_A=0 \\ F_A=0 \end{cases} \quad \begin{cases} \Delta_B=0 \\ F_B=1 \end{cases} \quad \begin{cases} \Delta_C=11 \\ F_C=1+11(0.03)=1.33 \end{cases} \quad \begin{cases} \Delta_D=11 \\ F_D=0.8 \end{cases} \quad \begin{cases} \Delta_E=14 \\ F_E=0.8 \end{cases}$$

This calculated values are in fact coordinates of A,B,C,D,E points in force-deformation diagram.

Calculating allowable strength and deformation for mentioned section due to IR360 standard:

$F_a$ : taken from linear analysis of structure from SAP2000

$$\Rightarrow C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \frac{6440}{2400} = 131.5$$

$$\Rightarrow B = \frac{1}{1 + \frac{Kl/R}{2C_c}}$$

$$\Rightarrow F_{as} = B \times F_a$$

$$\Rightarrow P_c = 1.7 \times F_{as} \times A$$

$$\Rightarrow \Delta_c = \frac{P_c \times l}{E \times A}$$

$$\Rightarrow P_T = F_{ye} \times A$$

$$\Rightarrow \Delta_T = \frac{P_T \times l}{E \times A}$$

Therefore, these components calculated in excel program for each brace in each axis and story and results presented in Table 3.33.

**Table 3.33 : Plastic hinge values for 2UPN300 section**

| Section    | 2UPN300     | Section    | 2UPN300     | Section    | 2UPN300     | Section    | 2UPN300     |
|------------|-------------|------------|-------------|------------|-------------|------------|-------------|
| Brace Type | X           | Brace Type | X           | Brace Type | X           | Brace Type | X           |
| Axe        | A-3~4,D-3~4 | Axe        | A-1~2,D-1~2 | Axe        | 1-A~B,4-A~B | Axe        | 1-C~D,4-C~D |
| Fa         | 978.643     | Fa         | 958.682     | Fa         | 921.244     | Fa         | 1019.066    |
| Fye        | 2640        | Fye        | 2640        | Fye        | 2640        | Fye        | 2640        |
| A          | 117.6       | A          | 117.6       | A          | 117.6       | A          | 117.6       |
| l          | 576.89      | l          | 602.08      | l          | 650.2       | l          | 525.6       |
| K          | 0.67        | K          | 0.67        | K          | 0.67        | K          | 0.67        |
| r          | 10.0408     | r          | 10.0408     | r          | 10.0408     | r          | 10.0408     |
| Kl/r       | 38.49457215 | Kl/r       | 40.17544419 | Kl/r       | 43.38638356 | Kl/r       | 35.07210581 |
| Cc         | 131.5       | Cc         | 131.5       | Cc         | 131.5       | Cc         | 131.5       |
| B          | 0.872320845 | B          | 0.867484505 | B          | 0.858393238 | B          | 0.88233684  |
| Fas        | 853.6906889 | Fas        | 831.6417798 | Fas        | 790.7896206 | Fas        | 899.1594744 |
| Pc         | 170669.8425 | Pc         | 166261.8246 | Pc         | 158094.6609 | Pc         | 179759.9621 |
| $\Delta c$ | 0.398678836 | $\Delta c$ | 0.405340619 | $\Delta c$ | 0.416234    | $\Delta c$ | 0.382579511 |
| PT         | 310464      | PT         | 310464      | PT         | 310464      | PT         | 310464      |
| $\Delta T$ | 0.725233143 | $\Delta T$ | 0.756900571 | $\Delta T$ | 0.817394286 | $\Delta T$ | 0.660754286 |

Final step is definig the calculated values in SAP2000 program. Figure 3.36 represents a plastic hinge definition in SAP2000 for example.

Frame Hinge Property Data for A,D(3-4) - Axial P

Edit

Displacement Control Parameters

| Point | Force/SF | Disp/SF |
|-------|----------|---------|
| E-    | -0.2     | -8.     |
| D-    | -0.2     | -0.5    |
| C-    | -1.015   | -0.5    |
| B-    | -1.      | 0.      |
| A     | 0.       | 0.      |
| B     | 1.       | 0.      |
| C     | 1.33     | 11.     |
| D     | 0.8      | 11.     |
| E     | 0.8      | 14.     |

☐ Symmetric

Load Carrying Capacity Beyond Point E

☒ Drops To Zero

☐ Is Extrapolated

Scaling for Force and Disp

|                                                                      | Positive | Negative  |
|----------------------------------------------------------------------|----------|-----------|
| <input type="checkbox"/> Use Yield Force Force SF                    | 310464.  | 170669.84 |
| <input type="checkbox"/> Use Yield Disp (Steel Objects Only) Disp SF | 0.7252   | 0.3987    |

Acceptance Criteria (Plastic Disp/SF)

|                                              | Positive | Negative |
|----------------------------------------------|----------|----------|
| <input type="checkbox"/> Immediate Occupancy | 0.25     | -0.25    |
| <input type="checkbox"/> Life Safety         | 7.       | -4.      |
| <input type="checkbox"/> Collapse Prevention | 9.       | -6.      |

☐ Show Acceptance Criteria on Plot

Type

☒ Force - Displacement

☐ Stress - Strain

Hinge Length

☐ Relative Length

Hysteresis Type And Parameters

Hysteresis Type

No Parameters Are Required For This Hysteresis Type

OK Cancel

**Figure 3.37 : Plastic hinge definition in SAP2000**

### 3.6.3 Rehabilitation using concrete shear walls

Adding concrete shear walls is second rehabilitation method for this structure to increase global stiffness capacity.

#### 3.6.3.1 Shear walls location

Shear walls are located as presented in Figure 3.37.

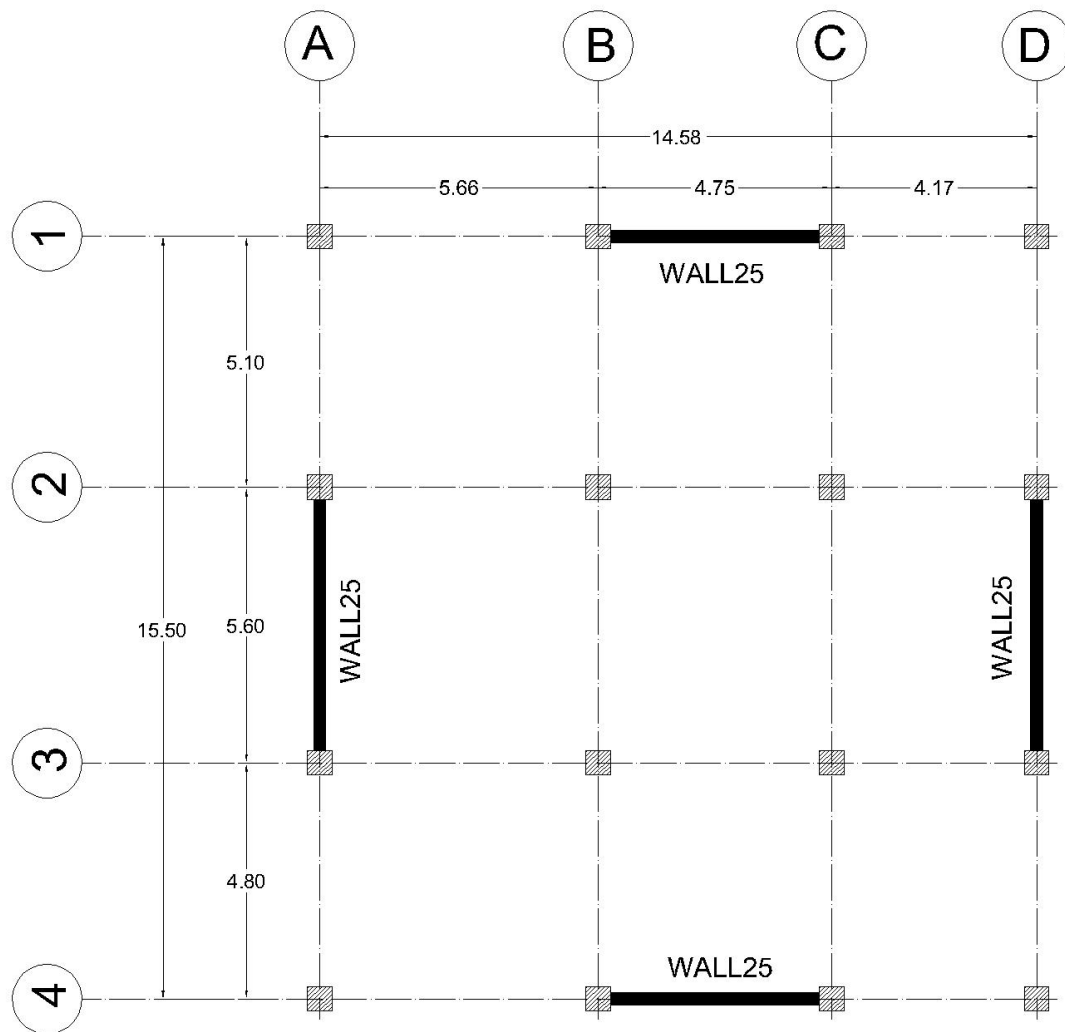


**Figure 3.38 : Shear walls location in plan**

### 3.6.3.2 Shear wall section definition in SAP2000

In SAP2000 program shear walls could be defined by layered/nonlinear shell, in this method every part of section such as rebars and concrete, will be defined as a layer of section and plastic behavior for each layer is definable. In Figure 3.38, definition of a shear wall section is presented for example.

| Layer Name | Distance | Thickness | Type     | Num Int. Points | Material | Material Angle | Material S11 | Material S22 | Behavior S12 |
|------------|----------|-----------|----------|-----------------|----------|----------------|--------------|--------------|--------------|
| ConcM      | 0.       | 0.25      | Membrane | 1               | Concrete | 0.             | Linear       | Nonlinear    | Linear       |
| TopBar2M   | 0.089    | 0.000377  | Membrane | 1               | Rebar    | 90.            | Nonlinear    | Inactive     | Linear       |
| BotBar2M   | -0.089   | 0.000377  | Membrane | 1               | Rebar    | 90.            | Nonlinear    | Inactive     | Linear       |
| ConcP      | 0.       | 0.25      | Plate    | 2               | Concrete | 0.             | Linear       | Linear       | Linear       |

**Figure 3.39 : Layered/nonlinear shear wall section definition**

### 3.6.4 Nonlinear static analysis

In this section results of static nonlinear analysis are presented and results between non-rehabilitated structure and two rehabilitation will be compared.

#### 3.6.4.1 Braced frame analysis results

The results for structure which is rehabilitated by braced frame mentioned in section 3.5.2 are presented. In Figure 3.39, 3D view of model is shown and almost all hinges are in immediate occupancy range except just one hinge in a beam.

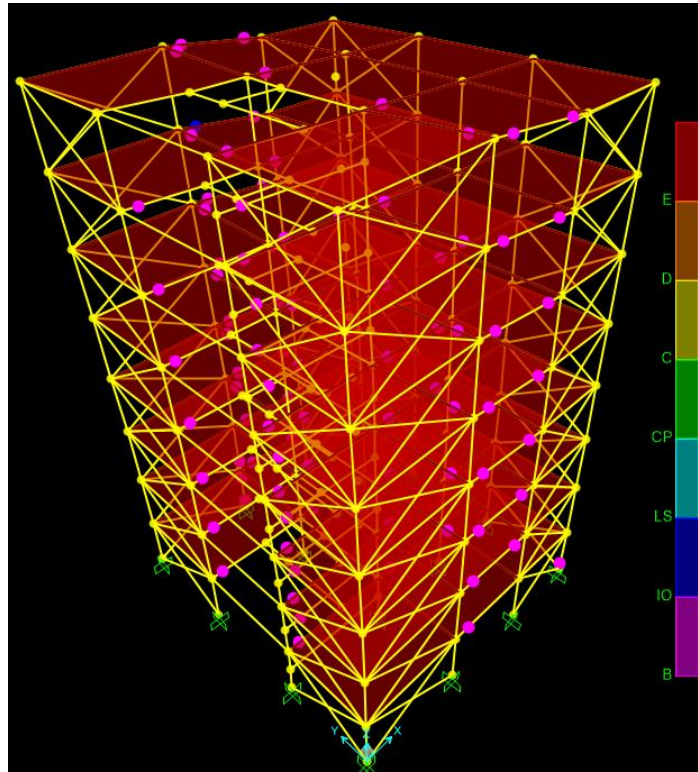


**Figure 3.40 :** Hinges results for structure rehabilitated by braced frame (Type 1) in 3D view

In Figure 3.40, the axis with most critical results presented, and as shown in figure, hinges results are acceptable due to static nonlinear analysis.

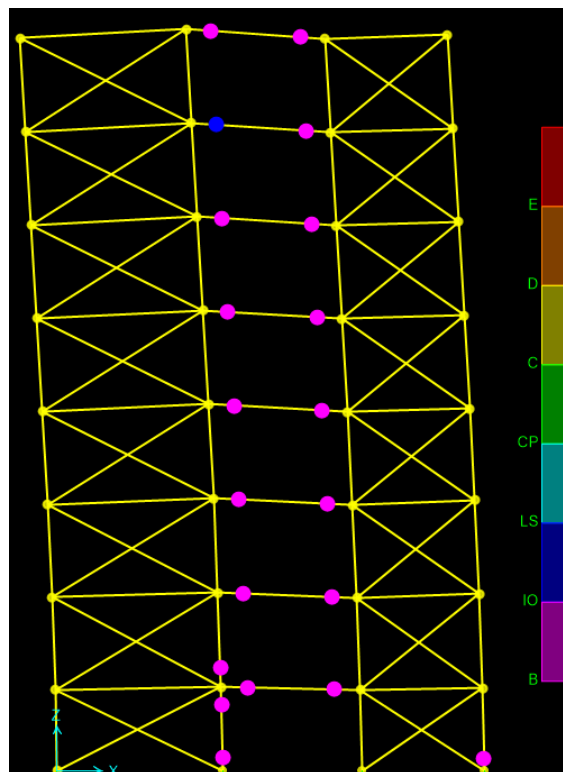


**Figure 3.41 :** Plastic hinges results in most critical axis (Type 1)

In Table and Table 3.34, hinge results for every step of analysis for adding braces and shear walls respectively are also given.

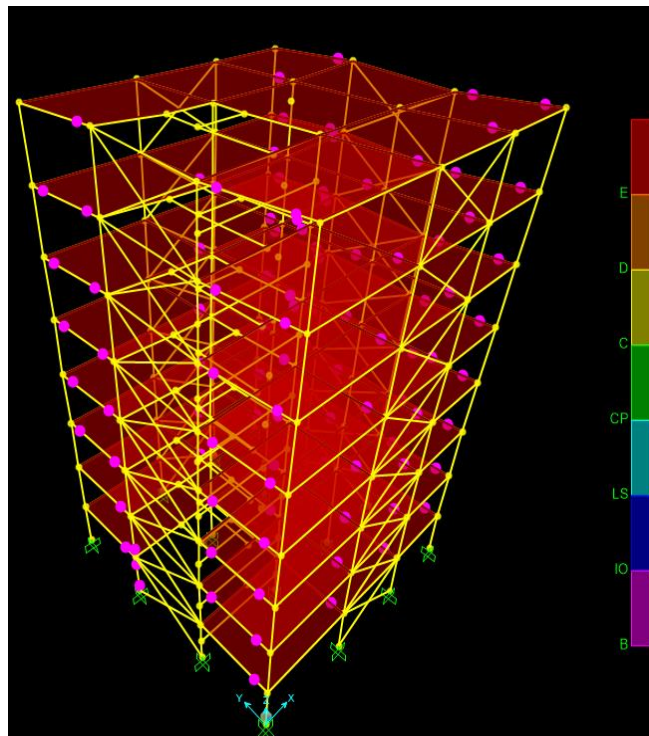


**Figure 3.42 :** Hinges results for structure rehabilitated by braced frame (Type 2) in 3D view

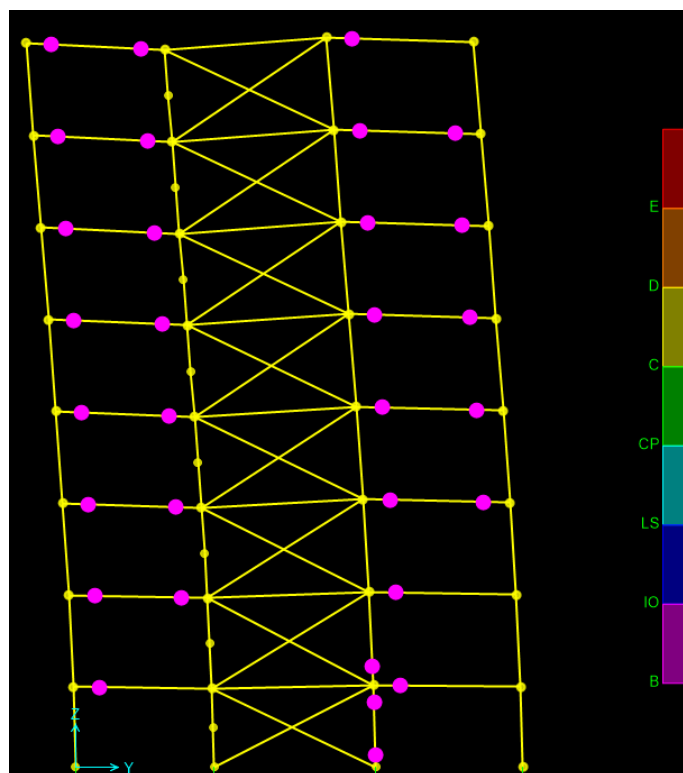


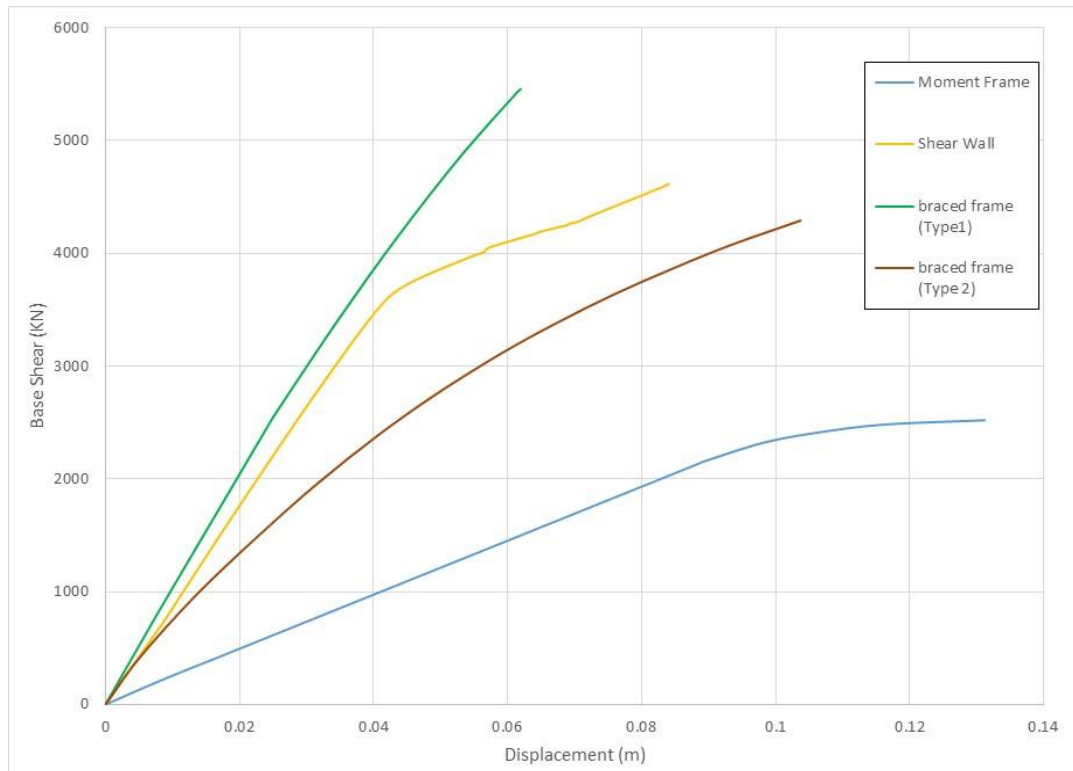
**Figure 3.43 :** Plastic hinges results in most critical axis (Type 2)

**Table 3.34** : Plastic hinge results for structure rehabilitated by adding braces

| Step | Displacement | BaseForce | AtoB | BtoIO | IOtoLS | LStoCP | CPtoC | CtoD | DtoE | BeyondE | Total |
|------|--------------|-----------|------|-------|--------|--------|-------|------|------|---------|-------|
|      | m            | KN        |      |       |        |        |       |      |      |         |       |
| 0    | 0.000227     | 0         | 816  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 816   |
| 1    | -0.008073    | 853.755   | 816  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 816   |
| 2    | -0.016373    | 1707.513  | 816  | 0     | 0      | 0      | 0     | 0    | 0    | 0       | 816   |
| 3    | -0.024673    | 2561.273  | 815  | 1     | 0      | 0      | 0     | 0    | 0    | 0       | 816   |
| 4    | -0.025072    | 2601.98   | 811  | 5     | 0      | 0      | 0     | 0    | 0    | 0       | 816   |
| 5    | -0.034532    | 3461.042  | 790  | 26    | 0      | 0      | 0     | 0    | 0    | 0       | 816   |
| 6    | -0.043705    | 4237.1    | 761  | 55    | 0      | 0      | 0     | 0    | 0    | 0       | 816   |
| 7    | -0.052785    | 4940.692  | 745  | 71    | 0      | 0      | 0     | 0    | 0    | 0       | 816   |
| 8    | -0.061387    | 5531.691  | 726  | 89    | 0      | 0      | 0     | 1    | 0    | 0       | 816   |
| 9    | -0.061962    | 5563.176  | 725  | 90    | 1      | 0      | 0     | 0    | 0    | 0       | 816   |
| 10   | -0.053636    | 4706.594  | 725  | 90    | 1      | 0      | 0     | 0    | 0    | 0       | 816   |

### 3.6.4.2 Added shear walls results

The results for structure which rehabilitated by shear walls mentioned in section 3.5.3 are presented. In Figure 3.42, 3D view of model shown and almost all hinges are in immediate occupancy range.

**Figure 3.44** : Comparison between rehabilitation methods due to pushover curve

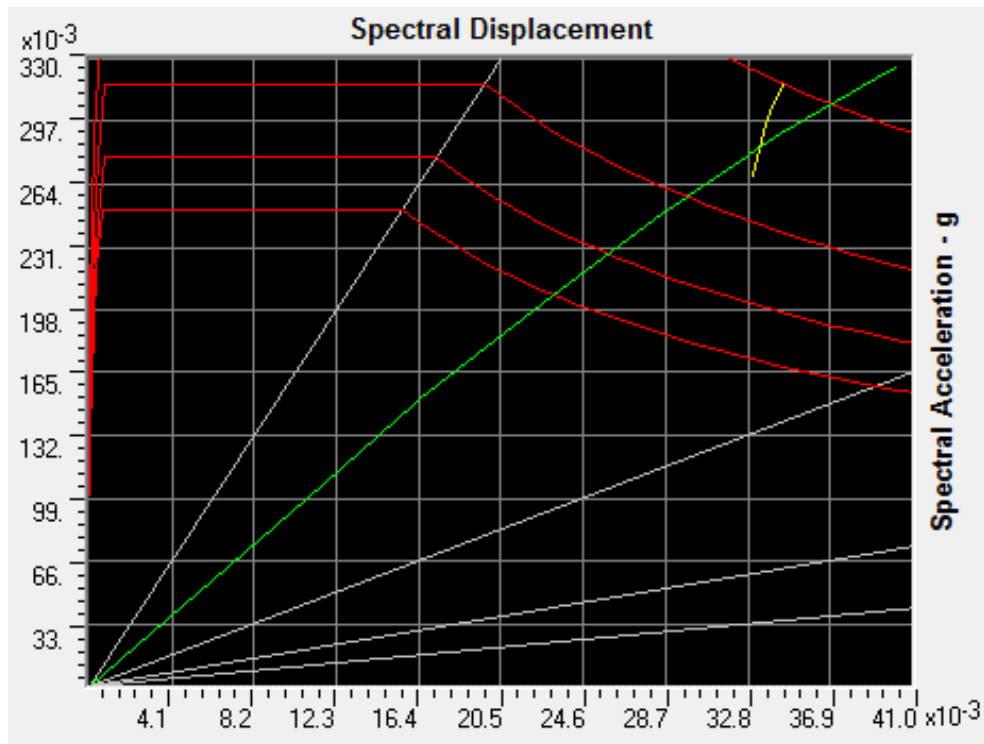


**Figure 3.45 :** Pushover capacity spectrum based on ATC-40 for bracing system

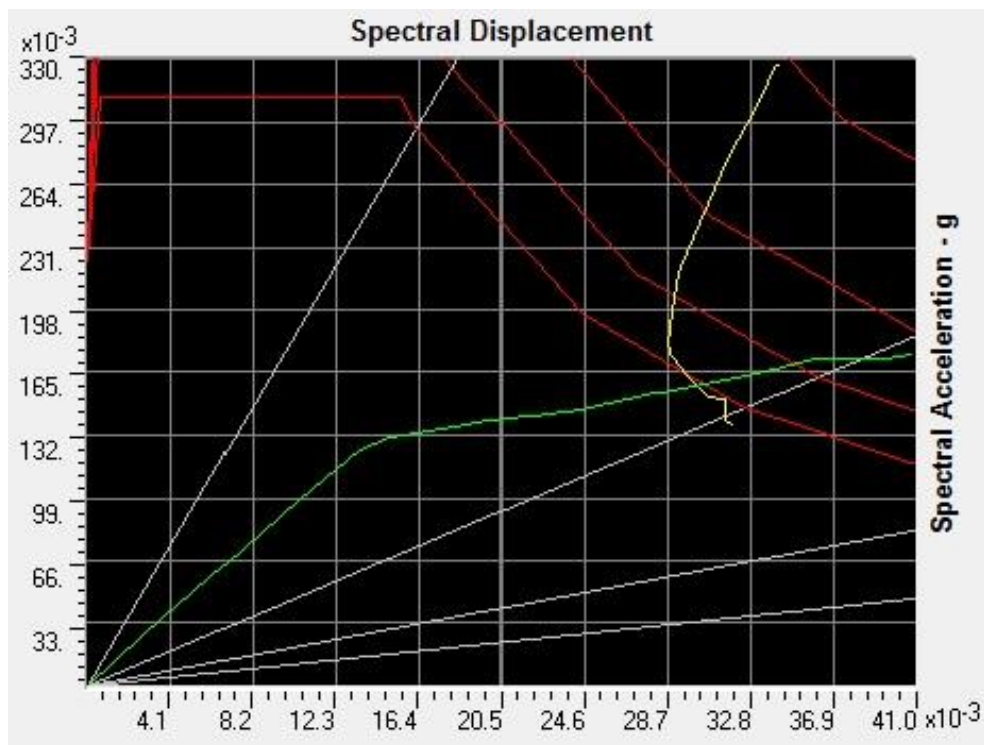


**Figure 3.46 :** Pushover capacity spectrum based on ATC-40 for concrete shear wall system

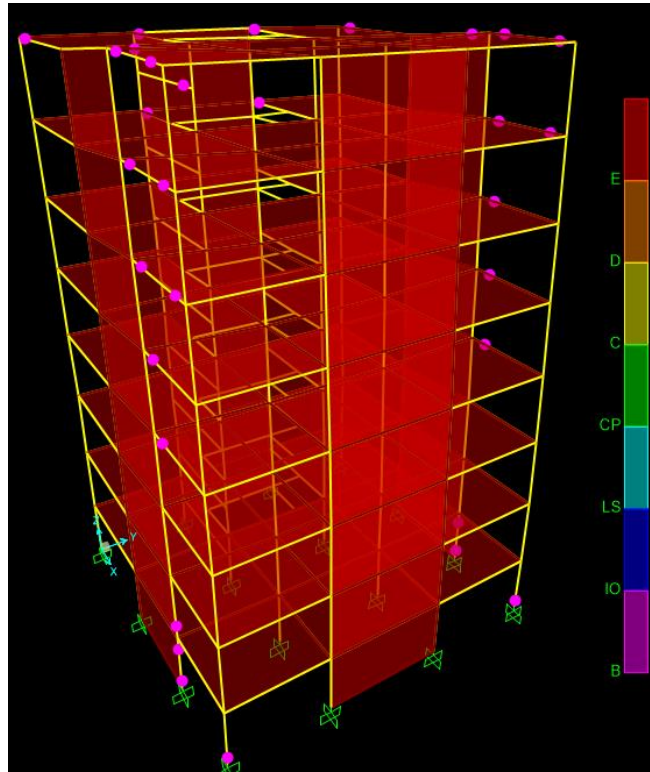


**Figure 3.47 :** Hinges results for structure rehabilitated by braced frame in 3D view

In Figure 3.46, the axis with most critical results presented, and as shown in figure, hinge results are acceptable due to static nonlinear analysis.

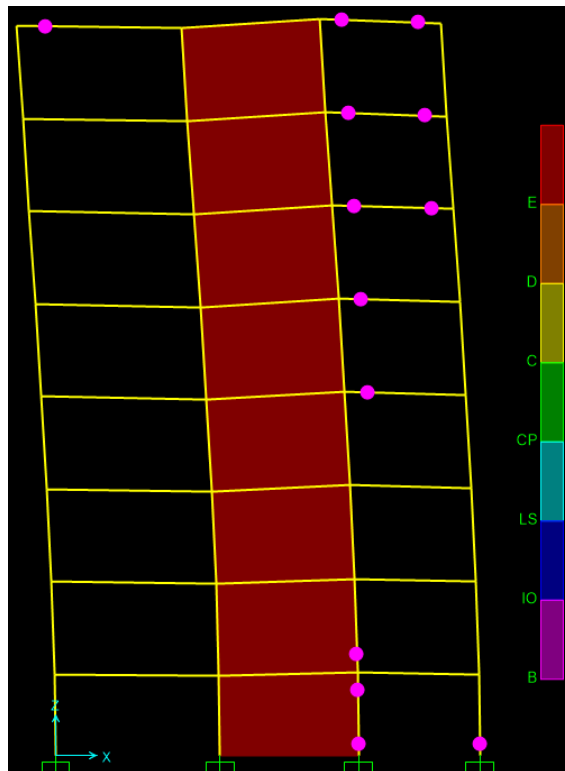


**Figure 3.48 :** Plastic hinges results in most critical axis

**Table 3.35 : Plastic hinge results for structure rehabilitated by adding shear walls**

| Step | Displacement | BaseForce | AtoB | BtoI | ItoLS | LStoCP | CPtoC | CtoD | DtoE | BeyondE | Total |
|------|--------------|-----------|------|------|-------|--------|-------|------|------|---------|-------|
|      | m            | KN        |      |      |       |        |       |      |      |         |       |
| 0    | -0.000651    | 0         | 688  | 0    | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 1    | -0.008951    | 772.974   | 688  | 0    | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 2    | -0.011719    | 1030.771  | 687  | 1    | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 3    | -0.020599    | 1851.236  | 682  | 6    | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 4    | -0.031587    | 2830.215  | 679  | 9    | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 5    | -0.040551    | 3568.118  | 667  | 21   | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 6    | -0.044987    | 3796.885  | 657  | 31   | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 7    | -0.054283    | 4042.207  | 644  | 44   | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 8    | -0.056383    | 4086.963  | 635  | 53   | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 9    | -0.057309    | 4132.397  | 634  | 54   | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 10   | -0.064281    | 4258.762  | 626  | 62   | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 11   | -0.064726    | 4272.32   | 626  | 62   | 0     | 0      | 0     | 0    | 0    | 0       | 688   |
| 12   | -0.068668    | 4330.692  | 613  | 73   | 2     | 0      | 0     | 0    | 0    | 0       | 688   |
| 13   | -0.069394    | 4350.122  | 613  | 74   | 1     | 0      | 0     | 0    | 0    | 0       | 688   |
| 14   | -0.070613    | 4365.786  | 610  | 77   | 1     | 0      | 0     | 0    | 0    | 0       | 688   |
| 15   | -0.072297    | 4413.192  | 603  | 83   | 2     | 0      | 0     | 0    | 0    | 0       | 688   |
| 16   | -0.083126    | 4679      | 571  | 109  | 8     | 0      | 0     | 0    | 0    | 0       | 688   |

### 3.6.5 Nonlinear time-history analysis

Nonlinear time-history analysis proceed for rehabilitated structure model in both added braced frame and shear wall method, drift results given in Figure 3.44 and Figure 3.45. There is shows that drifts are acceptable for both methods but using shear walls is almost suitable.and results compared by each other in Figure 3.46 for drift control in control node (mass center of roof).

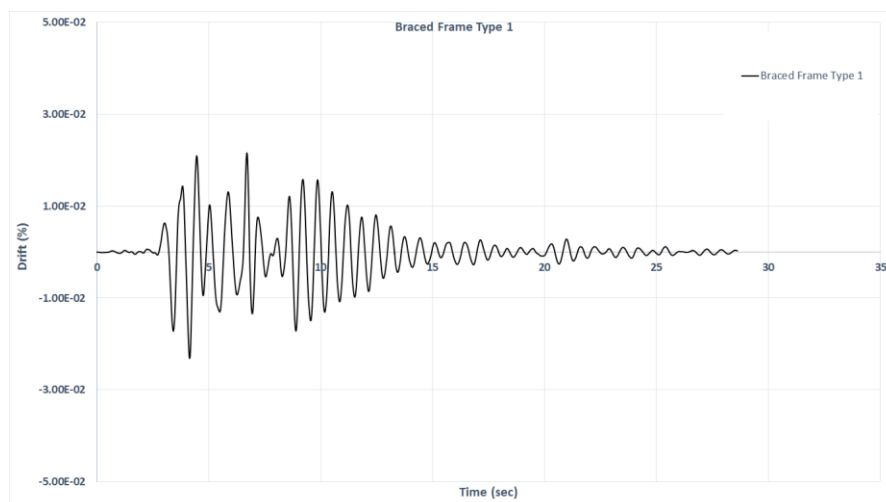
**Figure 3.49 : Drift control due to nonlinear time-history analysis for braced frames Type 1**

**Table 3.36 :** Comparison between strengthening methods based on modal analysis results

| StepNum  | Bracing Type 1 |          |          | Bracing Type 2 |          |          | Shear wall |          |            |
|----------|----------------|----------|----------|----------------|----------|----------|------------|----------|------------|
|          | Period         | SumUX    | SumUY    | Period         | SumUX    | SumUY    | Period     | SumUX    | SumUY      |
| Unitless | Sec            | Unitless | Unitless | Sec            | Unitless | Unitless | Sec        | Unitless | Unitless   |
| 1        | 0.655482       | 0.003015 | 0.681    | 0.889231       | 0.008002 | 0.748    | 0.68382    | 0.651    | 0.00001838 |
| 2        | 0.642308       | 0.691    | 0.684    | 0.876797       | 0.759    | 0.756    | 0.602871   | 0.651    | 0.649      |
| 3        | 0.399233       | 0.691    | 0.685    | 0.622601       | 0.759    | 0.756    | 0.460895   | 0.651    | 0.649      |
| 4        | 0.184943       | 0.704    | 0.875    | 0.295857       | 0.76     | 0.9      | 0.160852   | 0.85     | 0.649      |
| 5        | 0.18449        | 0.89     | 0.889    | 0.293402       | 0.901    | 0.901    | 0.140541   | 0.85     | 0.855      |
| 6        | 0.137715       | 0.89     | 0.889    | 0.209426       | 0.901    | 0.901    | 0.125579   | 0.85     | 0.855      |
| 7        | 0.125217       | 0.89     | 0.889    | 0.162838       | 0.902    | 0.943    | 0.112598   | 0.85     | 0.855      |
| 8        | 0.12239        | 0.89     | 0.889    | 0.1622         | 0.943    | 0.943    | 0.109953   | 0.85     | 0.855      |
| 9        | 0.12061        | 0.89     | 0.889    | 0.138347       | 0.943    | 0.943    | 0.109131   | 0.85     | 0.855      |
| 10       | 0.11313        | 0.89     | 0.889    | 0.122474       | 0.943    | 0.943    | 0.102975   | 0.85     | 0.855      |
| 11       | 0.10205        | 0.89     | 0.889    | 0.120604       | 0.943    | 0.943    | 0.090263   | 0.85     | 0.855      |
| 12       | 0.097012       | 0.89     | 0.89     | 0.115765       | 0.943    | 0.964    | 0.08562    | 0.85     | 0.855      |
| 13       | 0.091737       | 0.948    | 0.891    | 0.115175       | 0.965    | 0.964    | 0.07917    | 0.85     | 0.855      |
| 14       | 0.091435       | 0.949    | 0.949    | 0.101813       | 0.965    | 0.965    | 0.075231   | 0.85     | 0.856      |
| 15       | 0.084597       | 0.949    | 0.95     | 0.092883       | 0.971    | 0.966    | 0.071059   | 0.923    | 0.856      |
| 16       | 0.079322       | 0.949    | 0.95     | 0.090834       | 0.972    | 0.977    | 0.06352    | 0.923    | 0.91       |
| 17       | 0.061986       | 0.974    | 0.95     | 0.086994       | 0.979    | 0.978    | 0.061457   | 0.923    | 0.928      |
| 18       | 0.061668       | 0.974    | 0.975    | 0.075335       | 0.98     | 0.987    | 0.054212   | 0.923    | 0.928      |
| 19       | 0.047687       | 0.979    | 0.984    | 0.072984       | 0.991    | 0.989    | 0.043841   | 0.959    | 0.928      |
| 20       | 0.047428       | 0.989    | 0.988    | 0.063673       | 0.992    | 0.997    | 0.039746   | 0.959    | 0.962      |
| 21       | 0.037558       | 0.991    | 0.997    | 0.061829       | 0.999    | 0.999    | 0.030142   | 0.985    | 0.962      |
| 22       | 0.037204       | 0.999    | 0.999    | 0.055928       | 1        | 1        | 0.028116   | 0.985    | 0.986      |
| 23       | 0.018709       | 0.999    | 1        | 0.014509       | 1        | 1        | 0.019275   | 0.999    | 0.986      |
| 24       | 0.016811       | 1        | 1        | 0.012675       | 1        | 1        | 0.018879   | 0.999    | 1          |

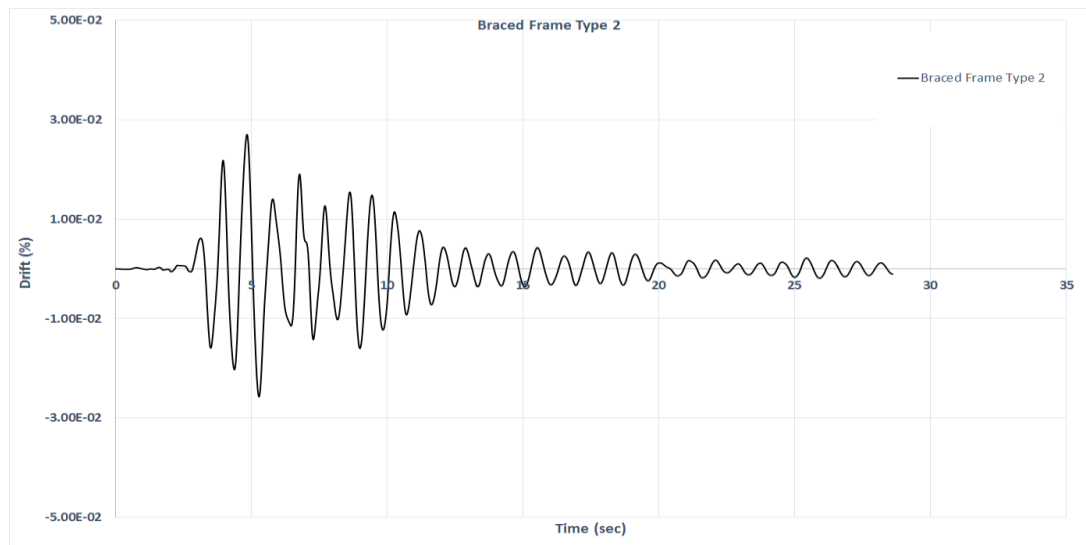


**Figure 3.50 :** Drift control due to nonlinear time-history analysis for braced frames Type 2

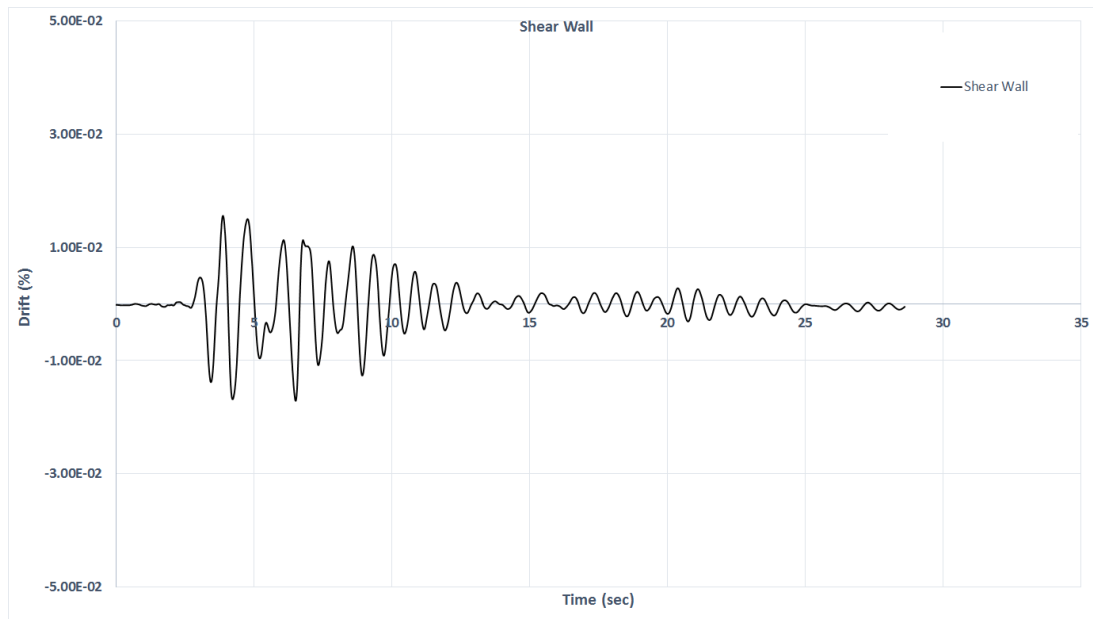


**Figure 3.51 :** Drift control due to nonlinear time-history analysis for shear walls

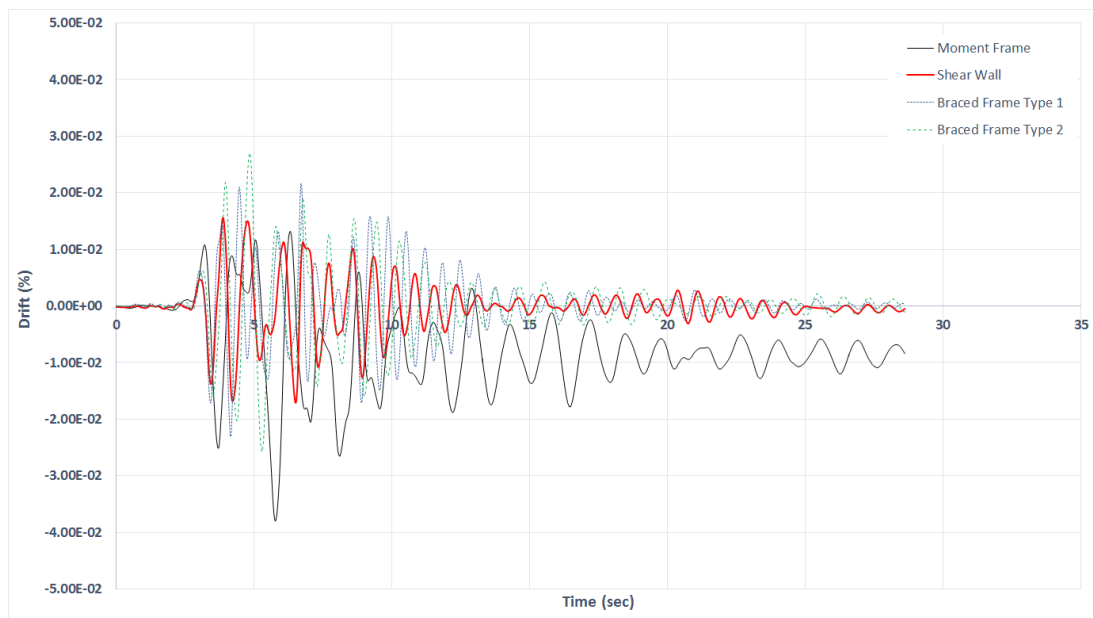


**Figure 3.52 :** Comparison between strengthening methods

#### 4. CONCLUSION

Through this study, seismic performance assessment of an eight story reinforced concrete building has been investigated. The building designed and constructed by using reinforced concrete moment frames as a lateral load resistance system according to the old version of IRAN's earthquake standard. Sectional analysis of the structural elements is carried out by using the allowable stress method. In order to assess the performance of the building, structural system has been modeled in SAP2000 ver.16 computer program and analyzed according to CODE2800 (ver 2013).

For linear static analysis, equivalent earthquake load has been calculated considering the seismic zone and soil profile type classifications for the region, which the building located, and for the linear dynamic analysis, modal combination method has been used considering the acceleration design spectrum for soil type 2 and seismic zone 2 according to the CODE2800. Free vibration modes combined by mentioned design acceleration spectrum and under this load case, the resulted forces are compared with allowable stress of the structure.

According to both methods, which mentioned above, using the equivalent earthquake method, the base shear was equal with 1998 KN and the maximum displacement of the building in the roof story was 7 cm, in modal analysis, fundamental period of free vibration of the existing building was 1.43 sec in X direction and 1.39 sec in Y direction.

In the next step, performance based analysis of the building has been considered. So in order to assess the performance of the structural system of the building two methods have been used: 1- nonlinear static analysis (push over), 2- nonlinear dynamic (time-history) analysis.

Pushover analysis results shows that the number of plastic hinges which reach collapse prevention limit state are 43 hinges and the target top displacement was equal with 26 Cm. Furthermore the base shear was 2710 KN. On the other hand nonlinear time-

history analysis results shows that the maximum drift percentage was equal to 3.8 % and this value is not acceptable according to the IRAN's earthquake standard . Hence according to the analysis results the lack of global rigidity and inadequate lateral displacement was observed in the building and the building need to be retrofitted.

Two different retrofitting strategy has been proposed to gain an acceptable seismic performance level based on FEMA 547 pre-standard.

The first strengthening method is adding steel braces to the existing RC frames in different locations which defined in two types, and the second method is adding shear wall system which applied to the structural system of the building.

The FEM of the building were modified and two retrofit models were created and analyzed using the same analysis methods used on the nonretrofitted model and results were reported. Comparing analysis results for the proposed rehabilitation methods the following findings were obtained:

According to pushover analysis the base shear carrying capacity of the building were increased by about two times when the building retrofitted with steel bracing Type 1 (i.e it increase from 2700kN for non-rehabilitated model to 5400 KN for retrofitted model with bracing Type 1 ). Also it increase by about 1.5 time in case of retrofitted with steel bracing Type 2 and shear wall system (i.e it becomes 4200kN and 4600 kN respectively)

The number of plastic hinges which formed in the building are : 90 hinges in Immediate Occupancy range and 1 hinge in the Life Safety range for bracing type 1, 116 hinges in Immediate Occupancy range and 13 hinge in Life Safety range for bracing type 2 and 109 hinges in Immediate Occupancy range and 8 hinge in Life Safety range for the added shear wall system.

Considering the nonlinear time-history analysis, the drift percentage which was 3.8% in non-rehabilitated model, decreased to 2.3% in bracing Type 1, to 2.7% in bracing Type 2 and to 1.71% in added shear wall method.

Finally, according to the performance of the structural system which analyzed under both nonlinear static and nonlinear dynamic loads, it is recommended to use shear walls or steel braces Type 1 to retrofit the case study building. Furthermore the results

which presented in Figure 3.50 , shows that the drift of the building which retrofitted by shear walls is more suitable, in the other side in point of economics, using shear walls is more reasonable than using braces, and shear walls are more applicable to retrofit the reinforced concrete building.



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