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THE DESIGN AND EVALUATION OF SEISMIC PERFORMANCE OF HIGH LEVEL OF DUCTILE RIGID FRAME

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MSc. THESIS
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Çağdaş ÖZDEMİR

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LIST OF SYMBOLS

AB	Area of base of structure
Ai	Web area of shear wall
Cm	Coefficient accounting for nonuniform moment distribution
Cs	Seismic response coefficient
Cvx	Vertical distribution factor
Cd	Deflection amplification factor
Di	Length of shear wall
E	Effect of horizontal and vertical earthquake induced forces
Eh	Horizontal earthquake forces
Em	Seismic load effect including overstrength factor
Emh	Effect of horizontal seismic forces including overstrength factor
Ev	Vertical seismic load
Fa	Short period site coefficient
Fcr	Critical stress
Fe	Elastic buckling stress
Fi	Portion of the seismic base shear
Fy	Specified minimum yield stress of the steel
Fv	long-period site coefficient
Fu	Specified minimum tensile strength
hi	Height of shear wall
hsx	Effect of horizontal seismic forces including overstrength factor
hn	Structural height
hsx	The story height below Levelx (hx-hx-1)
I	Importance factor
Ig	Moment of inertia of a horizontal structural element
Ic	Moment of inertia of a vertical boundary element taken perpendicular to the direction of the web plate line
Kx	Effective length factor for prismatic member
Ky	Effective length factor for prismatic member in y direction
k	Distribution exponent
Lb	Length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section
Lg	Length of horizontal element
Lc	Length of the column
Mpc	The sum of the nominal flexural strengths of the columns
Mpb	Expected flexural strength of the beam of the plastic hinge location

Mt	Torsional moment resulting from eccentricity between the location of center of mass and the center of rigidity
Muv	Additional moment due to shear amplification
Pe1	Elastic critical buckling strength of the member in the plane of bending
Pe,story	Elastic critical buckling strength for the story in the direction of translation being considered
Pn	Nominal compressive strength
Pu	Required axial strength using LRFD load combinations
Pstory	Total vertical load supported by the story using LRFD or ASD load combinations
Px	Total unfactored vertical design load at and above Level x
R	Response modification coefficient
rx	Radius of gyration for x direction
ry	Radius of gyration for y direction
RM	Coefficient to account influence of P-δ on P-Δ
SS	Mapped MCEg, spectral response acceleration parameter at short period
S1	Mapped MCEg, spectral response acceleration parameter at a period of 1s
SMS	The MCEg, spectral response acceleration parameter at short periods adjusted for site class effects
SDS	Design, spectral response acceleration parameter at a period of 1s
SM1	The MCEg, spectral response acceleration parameter at a period of 1s
SD1	Design, spectral response acceleration parameter at a period of 1s
Sa	Design response spectrum
Sh	The distance between the possible plastic hinge to the column floor
Ta	Approximate fundamental period of the building
Ts	SD1/SDS
TL	Long period transition period
To	0,2SD1/SDS
V	Total design lateral force or shear of the base
Vt	Design value of the seismic base shear
Vx	Seismic design shear in story x
W	Effective seismic weight of the building
Zb	Plastic section modulus of the beam
x	Number of shear walls in the building effective in resisting lateral forces
ΦE	Stability coefficient for P- Δ effects
P	A redundancy factor based on the extent of structural redundancy
ΩE	Effects of horizontal seismic force from V, FPX or FP
Ωo	Overstrength factor
δmax	Maximum displacement of Level x
δavg	The average of the displacements at the extreme points of the structure at Levelx
Δ	Design story drift
δx	Deflection of Level x at the center of the mass at and above Level x.
δxe	Deflection of Level x at the center of the mass at and above Level x determined by elastic analysis
Φmax	Maximum limit for the stability coefficient
B	Ratio of shear demand capacity for the story between Level x and x-1

LIST OF ABBREVIATIONS

ASD	Allowable Stress Design
CMS	Conditional Mean Spectrum
LRFD	Load and Resistance Factor Design
MCE	Mapped Considered Earthquake
SCBF	Special Concentrically Braced Frame
SMF	Special Moment Frame
UHS	Uniform Hazard Spectrum

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ABSTRACT

THE DESIGN AND EVALUATION OF SEISMIC PERFORMANCE OF HIGHLY DUCTILE RIGID STEEL FRAMES

Çağdaş ÖZDEMİR

Department of Civil Engineering

MSc. Thesis

Adviser: Assist. Prof. Devrim ÖZHENDEKCİ

This study is an attempt to investigate the effects of different ground motion sets on the structural response of steel special moment frames. Three ground motion sets are used which consist; near fault records with velocity pulses, near fault records without velocity pulses and far field records. The records of each set are scaled in order that their average spectrum approaches to the uniform hazard (design response) spectrum by following the rules provided in ASCE/SEI 07-10. The model frame is a component of lateral load resisting system of a ten story office building which is designed according to the Load and Resistance Factor Design of the ANSI/AISC 360-10. Applied capacity based design principles for providing global ductility are along with ANSI/AISC 341-10. The loads and load combinations are compatible with ASCE/SEI 7-10. The cross section and material properties are all compatible with USA codes. The seismic performance assessment of the model frame is carried out under the effect of above mentioned record sets via Nonlinear Dynamic Procedure (NDP) of ASCE/SEI 41-06 with the use of software PERFORM-3D. All of the structural elements' displacements and related member forces are post processed for each record during this assessment.

According to the results of the analyses, use of different record sets with distinctive characteristics do not affect the structural response much, in terms of satisfied seismic performance levels and mean values of the maximum story drift ratios. However, the dispersion of the story drift ratios are considerably higher for the near fault records with velocity pulses which can be attributed to the higher structural inelastic activity.

Keywords: Steel special moment frame, Seismic performance assessment, Nonlinear dynamic procedure, Ground motion set.

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ÖZET

SÜNEKLİK DÜZEYİ YÜKSEK RİJİT ÇERÇEVELERİN TASARIMI VE SİSMİK PERFORMANSLARININ DEĞERLENDİRİLMESİ

Çağdaş ÖZDEMİR

Yapı Mühendisliği Anabilim Dalı

Yüksek Lisans Tezi

Tez Danışmanı: Yrd. Doç. Dr. Devrim ÖZHENDEKCİ

Bu çalışmanın amacı farklı deprem setleri altında süneklik düzeyi yüksek çelik çerçevelerin yapısal tepkisini incelemektir. Toplamda hız darbeli yakın fay yer hareketi kayıtları, hız darbesiz yakın fay kayıtları, ve uzak fay yer hareketi kayıtlarından oluşan üç deprem seti kullanılmıştır. Tüm set kayıtları ASCE/SEI 07-10'da verilen yöntem uygulanarak her bir set için ortalama ivme spektrumu tasarim ivme spektrumuna yakınsayacak biçimde ölçeklendirilmiştir. Model çerçeve, 10 katlı bir ofis binasının bir deprem doğrultusu icin yatay yük taşıyıcı sistemini oluşturan iki süneklik düzeyi yüksek kenar çerçevesinden biri olarak seçilmiştir ve ANSI/AISC 360-10 şartnamesinde belirtilen LRFD tasarım kriterlerine göre tasarlanmıştır. Yeterli düzeyde yapısal sünekliğin sağlanabilmesi için uygulanan kapasite bazlı tasarım kriterleri ANSI/AISC 341-10'a uygundur. Yükler ve yük kombinasyonları ASCE/SEI 7-10'a uygundur. Profil kesitleri ve malzeme özellikleri Amerikan şartnamelerine uyumludur. Model çerçevenin sismik performans değerlendirmesi yukarıda belirtilen setler altında ASCE/SEI 41-06' da belirtilen doğrusal olmayan dinamik yonteme uygun olarak Perform 3D programıyla yapılmıştır. Bu değerlendirme esnasında tüm yapısal elemanların yer değiştirmeleri ve lüzumlu eleman kesit tesirleri her bir deprem kaydı icin okunmuş ve işlenmiştir.

Analiz sonuçlarına göre birbirinden son derece farklı özellikleri olan deprem setleri yapısal elemanların sergilediği sismik performans seviyelerini ve ortalama göreli kat ötelemesi değerlerini çok değiştirmemiştir. Öte yandan hız darbeli yakın fay yer hareketi kayıtlarını içeren deprem seti için yapılan cözümlemelerde göreli kat ötelemelerindeki saçılma göreli olarak fazla olmuştur, bunun sebebi ise bu karakterde deprem kayıtları için yapısal elemanlarda daha fazla elastik ötesi davranış meydana gelmesidir diye değerlendirilebilir.

Anahtar Kelimeler: Süneklik Düzeyi Yüksek Çelik Çerçeve, Sismik performans değerlendirmesi, Doğrusal olmayan dinamik yöntem, Yer hareketi setleri

CHAPTER 1

INTRODUCTION

1.1 Literature Rewiev

As an initial statement, some parts of this thesis is directly utilized during the preparation of a technical paper which is currently under revision and titled as; "Seismic Performance of Steel Special Moment Frames Subjected To Uniform Hazard Ground Motions" and the corresponding journal is The Structural Design of Tall and Special Buildings.

Nonlinear dynamic analysis of structures for seismic performance assessment is becoming more prevalent with the advancements in computer technology and with the accumulated data resources concerning the structural elements' behavior and ground motion records. However, the significant challenge to overcome for both practicing engineers and researchers is to establish the ground motion sets to be used. Unfortunately, there is currently no consensus in the earthquake engineering community on how to appropriately select and scale earthquake ground motions for the establishment of these sets [1]. There are a number of questions to be addressed such as: Which scaling technique provides the best fit for the existing case (scaling to a Uniform Hazard Spectrum-UHS, or the establishment of Conditional Mean Spectrum-CMS, etc?), At least how many records should be used?, How the orientation and directivity effects can be taken into consideration or should they be taken into consideration?, How many pulse-type motions can be used for each record set for near fault regions? Can the same scaling technique be applied to the pulse-type motions like the ordinary ones?... ,etc. There are a number of studies conducted in search for the answers of such questions [2,3,7,8] next to many other valuable work. Searching for the answers to these questions by excluding the sensitivity of the structural response to the variation of the ground motion parameters is impossible. Thus, some of the studies have their emphasis on structural behavior via studying the MDOF systems' inelastic response whereas some of them deal with the earthquake characteristics more deeply via studying the SDOF systems' spectra.

If the full distribution of structural response is required to be calculated, spectrum-matched motions can not be used. However seismic performance assessment of structures are generally carried out for a particular seismic hazard level in practice, besides spectrum matching decreases the dispersion of analysis results which in turn allows stable estimates of the response. Although it is disputable whether decreasing the dispersion is a realistic strategy, since inelastic response is nonlinearly related to the intensity of the input motions, and that intensities higher than the target spectrum produced disproportionately larger inelastic demand than the lower demand produced by lower intensities [13]. Thus, the median inelastic response under the effect of a record set with more variable spectra would be skewed towards higher values which would most probably result in bias of the response parameters from records spectra of which matched to a uniform hazard spectrum. Considering the previous work, there is a principal point of controversy whether spectrum-matched motions lead to unconservatively biased estimates of the median (or mean) response or not [2]. Actually, depending on the taken benchmark, some of the studies have showed an unconservative bias [15],whereas some of them did not[14].

1.2 Objective of the Thesis

It is also worth to note that nearly all of the previous studies have focused on the dispersion of story or interstory drift ratios. As far as known to the writers, effects of the use of various record sets fitting to a same uniform hazard spectrum on the seismic performance (assessed with nonlinear dynamic procedure - NDP) of building structures has not been studied. This is most probably because post processing the results of a number of nonlinear time history analyses of a multi story building is extremely tedious. However, the gained knowledge by the previous work about the story drift ratios can not be implemented into performance assessment procedures since the knowledge concerning the structural elements' displacements are needed directly. From this perspective, this study is an initial part of a going on research with emphasis on the effects of various record sets matched to a uniform hazard spectrum on the structural elements' displacements in terms of achieved seismic performance level rather than examining the dispersion of drift ratio. The authors have also been carrying out a companion study in order to relate the story drift ratios with structural element displacements in a systematic manner, the initial results are promising. This is relieving since it will be possible to implement the gained knowledge concerning the dispersion of drift ratios by the previous studies directly to the NDP procedure in this sense.

1.3 Hypothesis

Despite the above concerns about the ground motion selection and scaling procedures, nonlinear dynamic analyses are carried out generally following the limited rules provided by the codes in practice. Thus, the outcomes of the rules provided by the codes should also be studied thoroughly. Next to this, as mentioned before nearly all of the studies consisting structural analysis have dealt with story or inter story drift ratio variation, however in terms of seismic performance assessment the displacement of the whole structural elements should be examined. This study is such an attempt to comprehend the structural response of a model special moment frame. Actually, it wouldn't be too wrong to categorize this work as an investigation of the sensitivity of a steel special moment frame's response to different ground motion sets with distinctive characteristics. Furthermore, the rules provided by ASCE/SEI 7 -10 (2010) are adopted during the establishment of ground motion record sets.

CHAPTER 2

GENERAL INFORMATION

The model frame is one of the two perimeter special moment frames (SMF) of a ten story office building's lateral load resisting system in x direction (Fig.1). Furthermore, two perimeter special concentrically braced frames (SCBF) are assumed to resist to the lateral loads in y direction, however any investigation concerning those frames are not included in this work since the response of SCBF is beyond the scope of this paper. The model frame is designed according to the Load and Resistance Factor Design of the ANSI/AISC 360-10 [9]. Applied capacity based design principles for providing global ductility are along with ANSI/AISC 341-10 [10]. The loads and load combinations are compatible with ASCE/SEI 7-10 [11]. ASTM A992 steel material is used for both beams and columns, with 445 MPa (65 ksi) and 345 MPa (50 ksi), tensile and yield stresses, respectively. Dead load of the normal stories is 3.83 kN/m^2 (80 psf) including the weight of structural steel elements. Live load applied to the normal stories is 3.11 kN/m^2 (65 psf) including the weight of partition walls. Dead and live loads of the roof used for promenade purposes are 3.11 kN/m^2 (65 psf) and 2.87 kN/m^2 (60 psf), respectively. The site class which the sample building is assumed to be constructed on is C. The mapped maximum considered earthquake spectral response acceleration values at short and 1 second periods are 1.5g and 0.6g, respectively. Seismic design category of the sample building is D. Seismic response modification factor (R) is 8 and deflection amplification factor (C_d) is 5.5.

Strong column-weak beam considerations and especially the drift limits generally dominate the design process. The design lateral load distribution to the floor levels along the frame height is determined with the equivalent lateral load procedure of ASCE/SEI 7-10 [11]. The fundamental period of the model frame is found as 2.38 s. Assigned cross sections of all the beams and columns are chosen from W profiles. All of the columns from ground level to the 26 m (85 ft) height level are assigned as W14x93 and the remaining columns’ cross sections are W14x176. The whole beams of the 7^{th}-10^{th} stories are W18x71 whereas the remaining are W18x106. Gravity columns are also designed in order to be used for P-Delta modeling under the effect of "1.4D+1.6L" combination, next to the compactness and construction applicability considerations. The cross section assigned to the gravity columns is HSS 16x16x5/8.

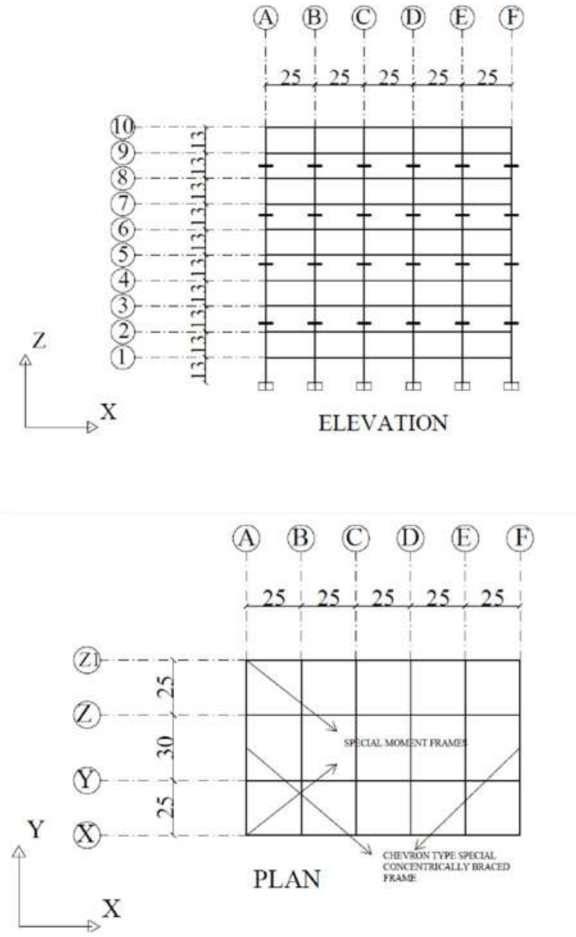


Figure 2.1 Plan and elevation section of the building

CHAPTER 3

GENERAL PROCEDURE FOR THE DETERMINATION OF THE DESIGN EARTHQUAKE LOAD AND RELEVANT COMBINATIONS ALONG WITH ASCE 07-10

3.1 Mapped Acceleration Parameters

There are two S_s and S_1 which shall be determined from the 0,2 and 1.0 s spectral response accelerations defined at ASCE 7-10, according to %g values S_s and S_1 values can be determined according to latitude and longitude information of relevant locations. S_1 is less than or equal to 0,04 and S_s is less than or equal to 0,15.

3.2 Site Class

Site classes can be classified as A, B, C, D, E or F according to soil properties. If there is no certain information about soil properties, site class D shall be used unless geotechnical data determines Site Class for F soils are present at the site.

Site Coefficients and Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameters

The MCE spectral response acceleration for short periods (S_{MS}) and at 1s (S_{M1}), adjusted for Site Class effects, shall be determined by Eqs 3-1 and 3-2, respectively.

$$S_{MS} = F_a \times S_{MS} \tag{3.1}$$

$$S_{M1} = F_v \times S_1 \tag{3.2}$$

S_s and S_1 are 'Mapped Acceleration Parameters' which are mentioned at 3.1

F_a and F_v are site coefficients based on the site class and mapped MCE spectral response acceleration values at short periods and at 1 s period, respectively and are provided in Tables 11.4-1 and 11.4-2 in ASCE 7/10

3.3 Design Special Acceleration Parameters

Design earthquake spectral response acceleration parameters at short period, S_{DS} and at 1s. period, S_{D1} shall be determined from Eqs 3-3 and 3-4, respectively. The reduction of the MCE ground motions by the 2/3 factor was introduced because the provisions in the Code provide a minimum safety margin of 1.5 against collapse and therefore include an inherent conservatism [16].

$$S_{DS} = \frac{2}{3} x S_{MS} \tag{3.3}$$

$$S_{D1} = \frac{2}{3} x S_{M1} \tag{3.4}$$

3.4 Design Response Spectrum

In fig 3-1, it is shown how to develop the design response spectrum curve and how to develop this curve are indicated. Accordingly, below formulae shall be used for the corresponding region.

According to 3-5 S_a shall be determined for.

$$S_a = S_{DS} x \left(0,4 + 0,6 \frac{T}{T_o} \right) \tag{3.5}$$

- If the periods greater than or equal to T_o and less than or equal to T_S, the design spectral response acceleration, S_a shall be taken equal to S_{DS}

- If the period is greater than T_S and less than or equal to T_L, S_a shall be taken as given by Eq3-6

$$S_a = \frac{S_{D1}}{T} \tag{3.6}$$

- If the period is greater than T_L, S_a shall be taken as given by Eq3-7;

$$S_a = \frac{S_{D1} x T_L}{T^2} \tag{3.7}$$

where

S_{DS} = the design spectral response acceleration parameter at

short periods

S_{D1} = the design spectral response acceleration parameter at

I-s period

T = the fundamental period of the structure, s

$$T_0 = 0.2x\frac{S_{D1}}{S_{DS}} \tag{3.8}$$

$$T_S = \frac{S_{D1}}{S_{DS}} \text{ and} \tag{3.9}$$

T_L = long-period transition period (s)

3.5 MCE Response Spectrum

It is determined by multiplying the design response spectrum by 1.5. In order to reduce the probability of exceedance this safety coefficient is used.

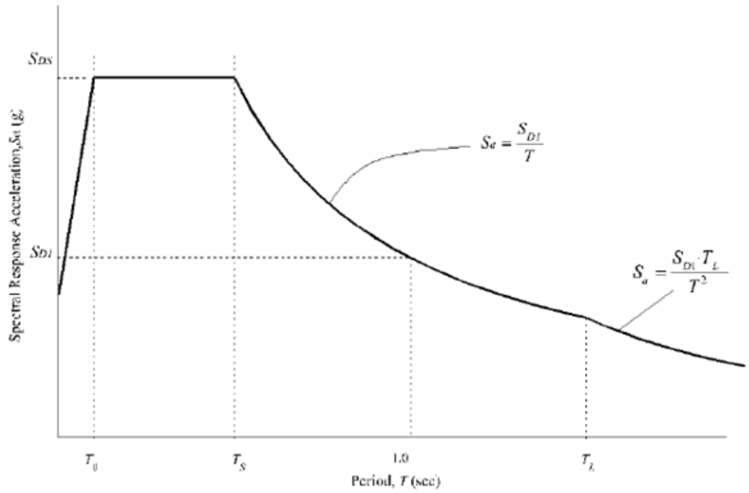


Figure 3.1 Response spectrum

3.6 Importance Factor and Occupancy Category

3.6.1 Importance Factor

An importance factor, I, shall be assigned to each structure in accordance with Table 3-1 based on the Occupancy Category from Table 1-1.

Table 3.1 Importance factor

Occupancy Category	*I*
I or II	1.0
III	1.25
IV	1.5

3.6.2 Protected Access for Occupancy Category IV

Where operational access is less than 10 ft from an interior lot line or another structure on the same lot, protection from potential falling debris from adjacent structures shall be provided by the owner of the Occupancy Category IV structure.

Table 3.2 Occupancy category

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \leq S_{DS} < 0.33$	B	B	C
$0.33 \leq S_{DS} < 0.50$	C	C	D
$0.50 \leq S_{DS}$	D	D	D

3.7 Seismic Design Category

Occupancy Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, S_1 is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. For the Occupancy Category IV in the same conditions, structure shall be assigned to Seismic Design Category F. All the other structures shall be categorized according to their occupancy category and S_{DS} and S_1 parameters.

Where S_1 less than 0.75, the Seismic Design Category is is permitted to be determined from Table 11.6-1 in ASCE7/10.

3.8 Seismic Load Effects and Combination

3.8.1 Applicability

Structure members shall be designed according to this section. Seismic load effects are the axial, shear and flexural member forces which are caused from horizontal and vertical seismic forces.

3.8.2 Seismic Load Effect

The seismic load affect symbolized as 'E'. It is determined as

- $E = E_h + E_v$ or $E = E_h - E_v$, it depends on the combinations which are taken into account.

$E =$ Seismic load

$E_h =$ Effect of horizontal seismic forces.

$E_v =$ Effect of vertical seismic forces as defined.

3.8.2.1 Horizontal Seismic Load Effect

E_h is determined as

$$E_h = \rho\theta_E \tag{3.10}$$

Where

θ_E = effects of horizontal seismic forces from V or F_p . V is the seismic base shear force which can be calculated in accordance with equivalent lateral force method. F_p is the distributed seismic force to the stories.

Such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other.

ρ = Redundancy factor. A redundancy factor shall be assigned to the seismic force-resisting system in each of two orthogonal directions for all structures.

3.8.2.2 Vertical Seismic Load Effect

E_v is determined as ;

$$E_v = 0,2.S_{DS}.D \tag{3.11}$$

Where

S_{DS} = design spectral response acceleration parameter at short

periods obtained from ASCE 7/10.

D = effect of dead load

Exceptions:

- If the S_{DS} is equal to or less than 0,125, E_v is taken as zero.
- On the surface of foundation E_v is taken as zero on the calculations.

3.8.3 Seismic Load Combinations

Basic combinations for strength design and allowable stress design.

$$(1,2+0,2S_{DS}).D+\rho.Q_E+L+0,2.S$$
$$(0,9-0,2S_{DS}).D+\rho.Q_E+1,6H$$
$$(1,0+0,14S_{DS}).D+H+F+0,7.\rho.Q_E$$
$$(1,0+0,10S_{DS}).D+H+F+0,525.\rho.Q_E+0,75.L+0,75(L_r,S,R)$$

3.8.4 Seismic Load Effect Including Overstrength Factor

If the overstrength factor is required, Overstrength Factor is basically based on the fact that there is some reserve strength capacity because of the redistribution of the internal forces. It is implemented to calculations as follows:

$$E_m = E_{mh} + E_v \tag{3.12}$$

$$E_m = E_{mh} - E_v \tag{3.13}$$

where

E_m = seismic load effect including overstrength factor

E_{mh} = effect of horizontal seismic forces including structural

overstrength

E_v = vertical seismic load effect

3.8.4.1 Horizontal Seismic Load Effect with Overstrength Factor

The horizontal seismic load effect with overstrength factor symbolized as E_{mh} is calculated as:

$$E_{mh} = \Omega_o . Q_E \tag{3.14}$$

Where

Q_E=Effects of horizontal seismic forces from V or F_p

Ω_E=Overstrength factor

Exception

The E_{mh} can't be much more than the maximum force that can be developed in the analysis.

3.8.4.2 Load Combinations with Overstrength Factor

The combinations which contain both E_m and the other loads can be defined as;

Basic Combinations for Strength Design with Overstrength Factor

5. $(1.2 + 0.2S_{DS})D + \Omega_o Q_E + L + 0.2S$

7. $(0.9 - 0.2S_{DS})D + \Omega_o Q_E + 1.6H$

Notes:

- L in combination 5 is always equal to 0.5 in all conditions while Lo is equal or less than 100 psf.
- The load factor on H shall be set equal to zero in combination 7 if the structural action due to H counteracts that due to E. Where lateral earth pressure provides

resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance. [1]

Basic Combinations for Allowable Stress Design with Overstrength Factor

5. $(1.0+0.14S_{DS})D+H+F+0.7\Omega_{o}Q_{E}$

6 $(1.0+0.105S_{DS})D+H+F+0.525\Omega_{O}Q_{E}+0.75L$
$+0.75(L,S,R)$

8. $(0.6-0.14S_{DS})D+0.7\Omega_{O}Q_{E}+H$

3.9 Direction of Loading

3.9.1 Direction of Loading Criteria

The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. The procedures those should be carried out are based on the previously defined Seismic design categories are B, C, D, E and F.

3.9.2 Seismic Design Category B

In this category seismic design forces shall be applied independently in each of two orthogonal directions and orthogonal effects can be neglected in calculations.

3.9.3 Seismic Design Category C

Structures that are assigned to seismic design category C shall at least satisfy the requirements of seismic design category B. In the structures that have horizontal irregularity (which are shown in Table 12.3-1 in ASCE 7/10), designers should follow one of the following procedures.

Table 3.3 Horizontal structural irregularities

Table 12.3-1 Horizontal Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity:** Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F B, C, D, E, and F C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity:** Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity:** Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity:** Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one having a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offset Irregularity:** Out-of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.	12.3.3.3 12.3.3.4 12.7.3 Table 12.6-1 Section 16.2.2	B, C, D, E, and F D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel System Irregularity:** Nonparallel system irregularity is defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

a. Orthogonal Combination Procedure

Three methods can be used to analyze structures. Those methods are the equivalent lateral force analysis, the modal response spectrum analysis or the linear response history. In these methods loads shall be taken as applied independently in two orthogonal directions and then the most critical load effect shall be used in calculations. 100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular directions are used in combinations.

b. Simultaneous Application of Orthogonal Ground Motion

Linear response history, the nonlinear response history procedure or the nonlinear response history procedure shall be used in this method. The difference between 'Orthogonal Combination Procedure' and this method is; in this methods load pairs are applied simultaneously.

3.9.4 Seismic Design Categories D through F

Structures assigned to these categories shall, as a minimum conform to the requirements provided for Seismic Design Category C. Some additional information concerning the intersecting vertical elements of lateral load resisting systems of a building are also given in ASCE 7-10. Except as required by the standard's relevant chapter, 2D analyses are permitted for structures with flexible diaphragms

3.10 Analysis Procedure Selection

Selection of analyze methods depends on the structure's seismic design category, structural system, dynamic properties and regularity or the legal approval. In Table 12.6-1 in ASCE 7/10 the Permitted Analytical Procedures are given.

3.11 Modeling Criteria

3.11.1 Foundation Modeling

For purposes of seismic load determination, the structure can be considered is taken into account as fixed at the base.

3.11.2 Effective Seismic Weight

The effective seismic weight is symbolized as W and it includes the total dead load and other loads as listed below:

- %25 of the floor live load in areas used for storage.
- When partition is needed, total partition weight or a minimum weight of 10 psf of floor area is required. Whichever is greater.
- Total operating weight of equipment.

- If the flat roof snow load, P_f exceeds 30 psf, 20 percent of the uniform design snow load, regardless of actual roof slope.

3.11.3 Structural Modeling

Analyzes should be made for determining the displacement of the structural members exposed to applied loads. P-Delta effects should be considered. The stiffness and strength of elements should be included in calculations.

If there is a horizontal irregularity type as indicated by the relevant chapter of the standard at the structures, a 3D model shall be used. In 3D model three dynamic freedoms consists of translation in two orthogonal plan directions and one torsional rotation about the vertical axis shall be included at each level of the structure. Also as mentioned before, 3D model shall include the stiffness properties of members. Model should also include:

- Stiffness properties of concrete and masonry elements shall include the effects of cracked sections.

- For steel moment frame systems the contribution of panel zone deformations to overall story drift shall be included.

CHAPTER 4

DESIGN OF THE STRUCTURE

4.1 Seismic Base Shear

Seismic base shear, V is determined by this equation:

$$V = C_s.W \tag{4.1}$$

Where,

C, = the seismic response coefficient determined in accordance with Section 4.1.1

W = the effective seismic weight per Section 3.12.2

4.1.1 Calculation of Seismic Response Coefficient

C_s is seismic response coefficient. It is determined as:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \tag{4.2}$$

Where,

S_{DS} = the design spectral response acceleration parameter in the short period range as determined from Section 3.3

R = the response modification factor in the shown table. There are two reasons for using the response modification factor:

- In the hyper static structure systems the gap between one section's load capacity and the system's load capacity is much more than the isostatic systems.

- During the earthquake process period of the structure (T) will decrease. Because the structure's elasticity will increase.

Table 4.1 Design coefficient and factors for seismic force-resisting systems

Seismic Force-Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Special steel concentrically braced frames[f]	14.1	6	2½	5	NL	NL	35	NP	NP[h,k]
2. Special reinforced concrete shear walls	14.2	6½	2½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	14.4	3½	3	3	NL	NL	NP	NP	NP
5. Composite steel and concrete concentrically braced frames	14.3	5½	2½	4½	NL	NL	160	100	NP
6. Ordinary composite braced frames	14.3	3½	2½	3	NL	NL	NP	NP	NP
7. Ordinary composite reinforced concrete shear walls with steel elements	14.3	5	3	4½	NL	NL	NP	NP	NP
8. Ordinary reinforced concrete shear walls	14.2	5½	2½	4½	NL	NL	NP	NP	NP
F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	12.2.5.10 and 14.2	4½	2½	4	NL	NP	NP	NP	NP
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	12.2.5.2								
1. Special steel moment frames	12.2.5.5 and 14.1	2½	1¼	2½	35	35	35	35	35
2. Intermediate steel moment frames	14.1	1½	1¼	1½	35	35	35[h]	NP[h,i]	NP[h,i]
3. Ordinary steel moment frames	14.1	1¼	1¼	1¼	35	35	NP	NP[h,i]	NP[h,i]
4. Special reinforced concrete moment frames	12.2.5.5 and 14.2	2½	1¼	2½	35	35	35	35	35
5. Intermediate concrete moment frames	14.2	1½	1¼	1½	35	35	NP	NP	NP
6. Ordinary concrete moment frames	14.2	1	1¼	1	35	NP	NP	NP	NP
7. Timber frames	14.5	1½	1½	1½	35	35	35	NP	NP
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	14.1	3	3	3	NL	NL	NP	NP	NP

I = the occupancy importance factor determined in accordance with Section 3.7.1 from Asce 7-10.

The value of C_s must be in accordance with the following equation.

$$C_s = \frac{S_{D1}}{T.\left(\frac{R}{I}\right)} \text{ for } T \le T_L \tag{4.3}$$

$$C_S = \frac{S_{D1}.T_L}{T^2.\left(\frac{R}{I}\right)} \text{ for } T \ge T_L \tag{4.4}$$

C_S shall not be less than 0.01 (4.5)

If in the structural location, S_1 is equal or greater than 0,6g, C_S must be in accordance with this equation;

$$C_s = \frac{0,5.S_1}{\left(\frac{R}{I}\right)} \tag{4.6}$$

I and R is defined in table 12.8-1 in ASCE7/10 and

S_{D1} = the design spectral response acceleration parameter at a period of 1 .O s, as determined from Section 3.3

T = the fundamental period of the structure (s) determined in Section 12.8.2 in ASCE7/10

TL = long-period transition period (s) determined in Section 3.4

S_1 = the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 3.1

4.1.2 Soil Structure Interaction Reduction

It is up the some generally accepted procedures approved by the authority having jurisdiction.

4.1.3 Maximum S_s Value in Determination of C_s

C_S can be taken as 1,5 for S_S **,** for regular structures less than five story.

4.2 Period Determination

It is very important to use the deformational characteristics of the structural resisting member of the structure in the direction under consideration when we define the fundamental period of the structure, The fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period (Cu) from Table 12.8-1 in ASCE7/10 and the approximate fundamental period, Ta, determined from Eq.12.8-7 in ASCE7/10. As an alternative to performing an analysis to determine the fundamental period, T, it is permitted to use the approximate building period , Ta, calculated in accordance with approximate fundamental period is symbolized as T_a and it is determined as:

$$T_a = C_t h_n^x \tag{4.7}$$

where h , is the height in ft above the base to the highest level of the structure and the coefficients C, and x are determined from Table 12.8-2 in ASCE7/10.

In order to use approximate fundamental period, the structure model must not be exceed 12 story, structure must be consist of concrete or steel moment resisting frame and the story height is at least 10 ft.

$$T_a = 0.1N \tag{4.8}$$

N=Number of stories.

4.3 Vertical Distribution of Seismic Force

The lateral force F_x at any level can be determined by this equation.

$$F_x = C_{vx}.V \tag{4.9}$$

And

$$C_{vx} = \frac{w_x.h_x^{\ k}}{\sum_{i=1}^{n} w_i h_i^{\ k}} \tag{4.10}$$

where

C_{vx} = vertical distribution factor,

V = total design lateral force or shear at the base of the structure (kip or kN)

w_i and w_x = the portion of the total effective seismic weight of the structure (W) located or assigned to Level i orx

h_i and h_x, = the height (ft or m) from the base to Level i or x k = an exponent related to the structure period as follows: for structures having a period of 0.5 s or less, k = 1 for structures having a period of 2.5 s or more, k = 2 for structures having a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

4.4 Horizontal Distribution of Seismic Force

The horizontal distribution of forces V_x is determined by this equation:

$$V_x = \sum_{i=x}^{n} F_i \tag{4.11}$$

where

F_i = the portion of the seismic base shear (V) (kip or kN) induced at Level i.

The V_x is distributed to all the seismic force resisting vertical structure members in the framework based on consideration the lateral stiffness of the members.

4.4.1 Inherent Torsion

Because of the eccentricity between the location of rigidity and the center of mass, in the not flexible diaphragms, M_t (inherent torsional moment) occurs. In the flexible diaphragms the position and distribution of the masses that are supported should be taken into account when distributing the seismic forces.

4.4.2 Accidental Torsion

In addition to inherent torsional moment, the design should also include accidental torsional moments (M_{ta}) caused by the assumed displacement of the center of mass from its original by a distance equal to 5 percent of the total perpendicular direction of the building perpendicular to the direction of the applied forces.)

When earthquake forces are applied concurrently in two orthogonal directions, the 5 percent displacement of the center of mass should not be applied in both of the directions at the same time, but shall be applied in the direction that produces the greater effect.

4.4.3 Amplification of Accidental Torsional Moment

In the seismic design category C, D, E or F and also the irregularity type 1a or 1b torsional irregularity exist as defined in Table 12.3-1 in ASCE7/10. M_{ta} must be amplified factor must be defined by this equation.

$$A_x = \left(\frac{\delta_{\max}}{1,2.\delta_{avg}}\right)^2 \tag{4.12}$$

Where

$\delta_{\max}$ = The maximum displacement at Level x (in. or mm) computed assuming $A_x = 1$

δ_{avg} = The average of the displacements at the extreme points of the structure at Level x computed assuming $A_x = 1$ (in. or mm)

Exception: The conditions which are defined in 12.8.3 must be taken into account when defining to seismic forces.

4.5 Story Drift Determination

The design story drift, Δ, shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration.

The deflections of Level x at the center of the mass (δ_x) (in. or mm) shall be determined in accordance with the following equation: (5)

$$\delta_x = \frac{C_d.\delta_{xe}}{I} \tag{4.13}$$

Where

C_d = The deflection amplification factor in Table 12.2-1 in ASCE7/10. The deflection amplification factor is used to minimize the nonstructural damage caused by the moderate earthquakes.

δ_{xe} = The deflections determined by an elastic analysis.

I = The importance factor determined in accordance with Section 3.7.1

4.5.1 Minimum Base Shear for Computing Drift

It can be computed by the equation (4-1).

4.5.2 Period for Computing Drift

By using seismic design forces based on the computed fundamental period of the structure without the upper limit $(C_u T_u)$ specified in Section 4.2, elastic drifts shall be computed in accordance with the story drift limits of section 12.12.1 in ASCE7/10.

4.6 P-Delta Effects

Where the stability coefficient (θ) is equal to or less than 0.10, P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered.

$$\theta = \frac{P_x.\Delta}{V_x.h_{sx}.C_D} \tag{4.14}$$

Where

P_x P, = the total vertical design load at and above Level x (kip or kN); where computing P,, no individual load factor need exceed 1.0

Δ = the design story drift as defined in Section 12.8.6 occurring simultaneously with V, (in. or mm)

V_x = the seismic shear force acting between Levels x and x 1 (kip or kN)

h_{sx} = the story height below Level x (in. or mm)

C_D = the deflection amplification factor in Table 12.2-1 in ASCE7/10

The stability coefficient (θ) shall not be exceeding θ_{max}. θ_{max} is determined by this equation:

$$\theta_{max} = \frac{0.5}{\beta C_D} \le 0.25 \tag{4.15}$$

β is a ratio of shear demand to shear capacity between two consecutive stories. This ratio permitted to be 1.0, conservatively.

The incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis where the stability coefficient (θ) is greater than 0.10 but less or equal to θ_{max} .However, alternatively Displacements and member forces are permitted to be multiplied by $\frac{1.0}{1-\theta}$.

If the θ is greater to θ_{max}, it means the structure is potentially unstable and shall be redesigned.

If P-delta effects are included in an automated analysis, before checking θ value it must be divided by $(1+\theta)$ when comparing it with θ_{max}.

4.6.1 P-Delta Effects

The P-delta effects shall be determined in accordance with Section 4.6. The base shear used to determine the story shears and the story drifts shall be determined in accordance with Section 4.5.

4.6.2 Soil Structure Interaction Reduction

A soil structure interaction reduction is permitted where determined using Chapter 19 or other generally accepted procedures approved by the authority having jurisdiction.

4.7 Application of the General Procedure for design

In order to to determine the '*Approximate Fundamental Period (T_a)*' Eq (4-7) is used where:

$$T_a = C_t h_n^x \tag{4.7}$$

h_n =130 (Structural Height)

$$\left.\begin{array}{l} C_t = 0,028 \\ x = 0,8 \end{array}\right\} \text{Table 12.8-2 (ASCE 7-10)}$$

$T_a = 1.375$

The obtained period value is utilized for initial trial phase of the design process.

- Site Coefficients and Risk-Targeted Maximum Considered Earthquake Spectral Response Acceleration Parameters are defined as follows:

$$\left.\begin{array}{l} S_s = 1.5 \\ S_1 = 0.6 \end{array}\right\} \quad \text{Section 3.1 (ASCE 7-10)}$$

$$S_{ms} = F_a S_s = 1x1.5 = 1.5 \tag{3.1}$$

$$S_{m1} = F_v S_1 = 1.5x0.6 = 0.9 \tag{3.2}$$

- Design Spectral Acceleration Parameters are defined as follows:

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} x1.5 = 1 \tag{3.3}$$

$$S_{D1} = \frac{2}{3} x S_{M1} = \frac{2}{3} x0.9 = 0.6 \tag{3.4}$$

4.8 Assigned Initial cross sections for the initial phase of the design iteration

Table 4.2 Sections table

<table>
<tr><th>Story</th><th>Column</th><th>Beam</th></tr>
<tr><td>10</td><td rowspan="2">W14x176</td><td>W18x71</td></tr>
<tr><td>9</td><td>W18x71</td></tr>
<tr><td>8</td><td rowspan="2">W14x176</td><td>W18x71</td></tr>
<tr><td>7</td><td>W18x71</td></tr>
<tr><td>6</td><td rowspan="2">W14x193</td><td>W18x106</td></tr>
<tr><td>5</td><td>W18x106</td></tr>
<tr><td>4</td><td rowspan="2">W14x193</td><td>W18x106</td></tr>
<tr><td>3</td><td>W18x106</td></tr>
<tr><td>2</td><td rowspan="2">W14x193</td><td>W18x106</td></tr>
<tr><td>1</td><td>W18x106</td></tr>
</table>

4.8.1. Compactness for Columns

Flanges

$$\lambda_r = 0,56x\sqrt{\frac{E}{F_y}} = 0,56x\sqrt{\frac{29000}{50}} = 9,2 > \frac{b_f}{2t_f} = 5,06(W14x211), 5,97(W14x174) \quad (4.16)$$

Web

$$\lambda_r = 1,49x\sqrt{\frac{E}{F_y}} = 1,49x\sqrt{\frac{29000}{50}} = 35,9 > \frac{b_f}{2t_f} = 12,8(W14x211), 15,2(W14x174) \quad (4.17)$$

4.8.2. Compactness for Beams

Flanges

$$\lambda_r = 0,56x\sqrt{\frac{E}{F_y}} = 0,56x\sqrt{\frac{29000}{50}} = 9,2 > \frac{b_f}{2t_f} = 5,06(W18x106), 5,97(W18x71) \quad (4.18)$$

Web

$$\lambda_r = 1,49x\sqrt{\frac{E}{F_y}} = 1,49x\sqrt{\frac{29000}{50}} = 35,9 > \frac{b_f}{2t_f} = 28,5(W18x106), 34,1(W18x71) \quad (4.19)$$

4.9. Load Values

System load values are as follows:

$Area = 5000 ft^2 = 464.439 m^2$

$G_{Story} = 3.83 kN/m^2 (80 psf)$

$Q_{Story} = 3.11 kN/m^2 (65 psf)$

$G_{Roof} = 3.11 kN/m^2 (65 psf)$

$Q_{Roof} = 2.87 kN/m^2 (60 psf)$

$W_{Story} = 2139.903 kN (481 Kip)$

$W_{Roof} = 1777.64 kN (400 Kip)$

$W_{Total} = 21036.76 kN (4729.065 Kip)$

4.10. Equivalent Lateral Force Procedure

The Seismic Base Shear, V is defined along with Eq (4-1) as repeated follows:

$$V = C_s W \tag{4.1}$$

Limitations for coefficient of C_s are given below:

- $$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{1}{3,402x\left(\frac{8}{1}\right)} = 0,022046 \tag{4.2}$$

- $$C_s = \frac{S_{D1}xT_L}{T^2x\left(\frac{R}{I_e}\right)} = \frac{1x4}{3,402^2x\left(\frac{8}{1}\right)} = 0,025921 \tag{4.3}$$

- $$C_s = 0,044xS_{DS}xI_e = 0,044x1x1 = 0,044 \tag{4.20}$$

$$C_s = 0,5xS_1x\left(\frac{R}{I_e}\right) = 0,5*0,6*\left(\frac{8}{1}\right) = 0,0375 \tag{4.6}$$

Table 4.3 The story drift calculations

STORY	HEIGHT	W_iH_i	C_{vx}	F_x	Story Shear	δ_{xe}	δ_{xj}-δ_{xi}
10	130	5695592,05	0,22	39,60	0,00	4,14	0,20
9	117	5574110,20	0,22	38,76	39,60	3,94	0,33
8	104	4422428,74	0,17	30,75	78,36	3,61	0,44
7	91	3401783,48	0,13	23,65	109,12	3,17	0,46
6	78	2512790,18	0,10	17,47	132,77	2,71	0,45
5	65	1756164,03	0,07	12,21	150,24	2,26	0,47
4	52	1132757,39	0,04	7,88	162,45	1,79	0,49
3	39	643624,08	0,03	4,48	170,33	1,29	0,50
2	26	290143,58	0,01	2,02	174,81	0,79	0,48
1	13	74317,15	0,00	0,52	176,82	0,31	0,31
Total		25503710,87	1,00	177,34	177,34		

$$W_iH_i^k = 13x481^{1,965} = 74317,145Kip \tag{4.21}$$

The coefficient k is calculated as:

$$k = (T_a - 0.5)/2 + 1 = 1,965 \tag{4.22}$$

The coefficient k is used to calculate C_{vx} the vertical distribution factor.

$$C_{vx} = \frac{W_i H_i^k}{\sum_{i=1}^{n=1} W_i H_i^k} = \frac{74317,145}{25503711} = 0,002914 \tag{4.23}$$

F_x values are calculated as:

$$F_x = C_{vx} x V = 0,002914 x 177,3399 = 0,516764 kip \tag{4.24}$$

The seismic design story shear at any story (V_x) is calculated as:

$$V_x = \sum_{i=x}^{n} F_i \tag{4.25}$$

$$V_8 = \sum_{i=8}^{n} F_8 = 39,60427054 + 38,75955 = 78,36381767 kip$$

Story Drifts are taken from the Structural Analysis Program SAP 2000. The deflection at Level x (δ_x) is calculated as:

$$\delta_x = \frac{C_d x \delta_{xe}}{I_e} = \frac{5,5 * 0,314251}{1} = 1,728381 in \tag{4.26}$$

C_d is the deflection amplification factor in Table 12.12-1 in ASCE7/10 and I_e is the importance factor determined in accordance with section 3.7.1.

Table 4.4 Story drift values

δ_x	**Story Drift**	**Story Drift Ratio**
22,78	1,10	0,01
21,68	1,83	0,01
19,85	2,42	0,02
17,43	2,53	0,02
14,90	2,47	0,02
12,43	2,61	0,02
9,82	2,71	0,02
7,11	2,75	0,02

Table 4.4 (cont'd)

4,36	2,63	0,02
1,73	1,73	0,01

Story drifts are divided into story height to obtain story drift ratios. All the ratios are under the upper limit of 0,02. Sections are acceptable.

4.11. P-Delta Effect

P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered where the stability coefficient (θ) as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta I_e}{V_x h_{xx} C_d} \tag{4.16}$$

Px = The total vertical design load at and above Level *x* (kip or kN); where computing *Px*, no individual load factor need exceed 1.0

Δ = The design story drift as defined in Section 12.8.6 (Asce 7-10) occurring simultaneously with *Vx* (in. or mm)

Ie = The importance factor determined in accordance with Section 3.7.1

Table 4.5 The axial forces of the columns

AXIAL FORCE							
Story	A	B	C	D	E	F	Total
10	67	132	133	67	132	133	664,08
9	146	283	287	146	283	287	1432
8	225	435	441	225	435	441	2202
7	304	587	595	304	587	595	2972
6	384	739	749	384	739	749	3744
5	463	890	904	463	890	904	4514
4	542	1042	1058	542	1042	1058	5284
3	620	1195	1212	620	1195	1212	6054
2	698	1349	1366	698	1349	1366	6826
1	773	1505	1520	773	1505	1520	7596

Table 4.6 The calculation table of the stability coefficient

STORY	Px	Δ	Ie	Vx	hxx	Cd	ϴ	β	ϴmax
10	664,08	4,14	1,00	39,60	156,00	5,50	0,08	1,00	0,09
9	1432,00	3,94	1,00	78,36	156,00	5,50	0,08	1,00	0,09
8	2202,00	3,61	1,00	109,12	156,00	5,50	0,08	1,00	0,09
7	2972,00	3,17	1,00	132,77	156,00	5,50	0,08	1,00	0,09
6	3744,00	2,71	1,00	150,24	156,00	5,50	0,08	1,00	0,09
5	4514,00	2,26	1,00	162,45	156,00	5,50	0,07	1,00	0,09
4	5284,00	1,79	1,00	170,33	156,00	5,50	0,06	1,00	0,09
3	6054,00	1,29	1,00	174,81	156,00	5,50	0,05	1,00	0,09
2	6826,00	0,79	1,00	176,82	156,00	5,50	0,04	1,00	0,09
1	7596,00	0,31	1,00	177,34	156,00	5,50	0,02	1,00	0,09

P_x =Total story axial forces

Δ = Story drifts

V_x =Story shear forces

h_{xx}=Story Height

$$\theta = \frac{7596x0.314251x1}{177.3399x156x5,5} = 0,015688 \quad (4.27)$$

β =This value can be taken as 1.0 conservatively.(ASCE7-10/12.8.7)

$$\theta_{\max} = \frac{0,5}{\beta C_d} < 0,25 \quad (4.28)$$

$$\theta_{\max} = \frac{0,5}{1x5,5} = 0,090909091$$

All the stability coefficient values are under the upper limit. All sections are acceptable.

4.12. Strong Column Weak Beam

Columns shall be designed to be stronger than the beams. In order to verify this check, shear forces on the beams shall be calculated. The combination of $(1.2 + 0.2S_{DS})$ D + L(+0.2S) is used. (4.29)

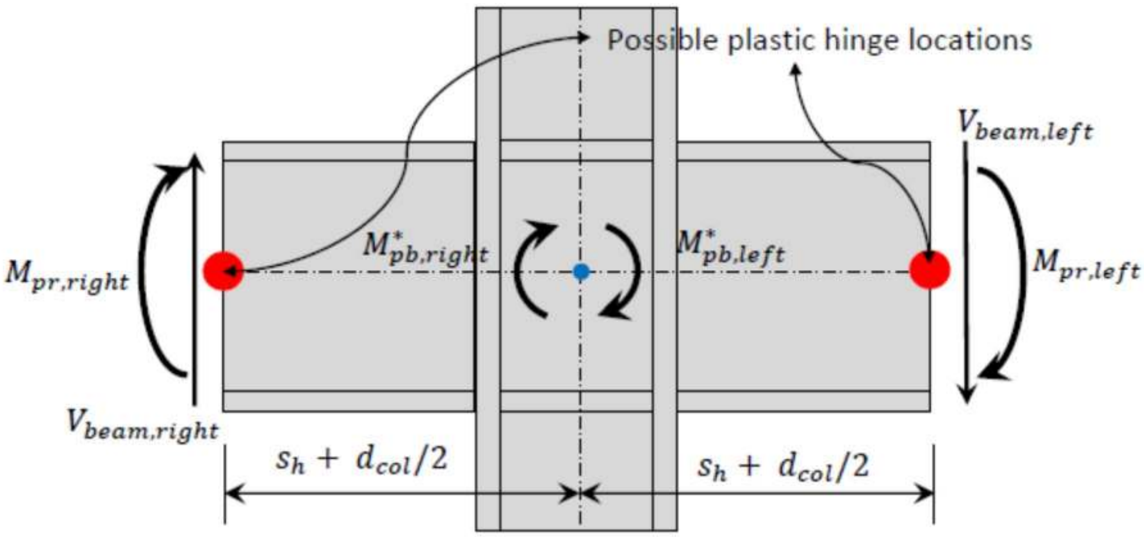


Figure 4.1 Beam-Column connection

$M_{pr} = C_{pr} F_y R_y Z_b$, results are given below; (4.30)

Table 4.7 M_{pr} values

sh	Lh	Fy	Fu	Ry	Zb	Mpr
9,25	341,5	50	65	1	146	7300
9,25	341,5	50	65	1	146	7300
9,25	341,5	50	65	1	146	7300
9,25	341,5	50	65	1	146	7300
9,35	341,3	50	65	1	230	11500
9,35	341,3	50	65	1	230	11500
9,35	341,3	50	65	1	230	11500
9,35	341,3	50	65	1	230	11500
9,35	341,3	50	65	1	230	11500
9,35	341,3	50	65	1	230	11500

$V_{Gravity} = 16.7237 kip$

$V_{beam} = 2[1.1 R_y M_p] / L_h + V_{Gravity} = 84.1130935 kip$ (For 1st story) (4.31)

$M_{uv} = V_{beam} x \left(S_h + \frac{d_{col}}{2} \right) = 285.9753$ (For 1st story) (4.32)

$S_h =$ The distance between the possible plastic hinge to the column face. The results of the structure are given below:

Table 4.8 M_{uv} values for grid A

Story	Beam	Column	Vgravity	Vbeam	sh	dcol/2	Muv
10	W18x71	W14x176	26,98	69,73	9,25	7,75	458,63
9	W18x71	W14x176	16,73	59,49	9,25	7,75	284,47
8	W18x71	W14x176	16,73	59,49	9,25	7,75	284,47
7	W18x71	W14x176	16,73	59,49	9,25	7,75	284,47
6	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98
5	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98
4	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98
3	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98
2	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98
1	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98

Table 4.9 M_{uv} values for grid B

Story	Beam	Column	Vgravity,left	Vgravity,right	Vbeam,left	Vbeam,right	sh	dcol/2	Muv,right	Muv,left
10	W18x71	W14x176	26,98	26,98	69,73	69,73	9,25	7,75	458,63	458,63
9	W18x71	W14x176	16,73	16,73	59,49	59,49	9,25	7,75	284,47	284,47
8	W18x71	W14x176	16,73	16,73	59,49	59,49	9,25	7,75	284,47	284,47
7	W18x71	W14x176	16,73	16,73	59,49	59,49	9,25	7,75	284,47	284,47
6	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98
5	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98
4	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98
3	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98
2	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98
1	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98

Table 4.10 M_{uv} values for grid C

Story	Beam	Column	Vgravity,left	Vgravity,right	Vbeam,left	Vbeam,right	sh	dcol/2	Muv,righ	Muv,left
10	W18x71	W14x176	26,98	26,98	69,73	69,73	9,25	7,75	458,63	458,63
9	W18x71	W14x176	16,73	16,73	59,49	59,49	9,25	7,75	284,47	284,47
8	W18x71	W14x176	16,73	16,73	59,49	59,49	9,25	7,75	284,47	284,47
7	W18x71	W14x176	16,73	16,73	59,49	59,49	9,25	7,75	284,47	284,47
6	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98
5	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98
4	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98
3	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98
2	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98
1	W18x106	W14x193	16,72	16,72	84,11	84,11	9,35	7,75	285,98	285,98

Table 4.11 M_{uv} values for grid D

Story	Beam	Column	Vgravity,left	Vgravity,right	Vbeam,left	Vbeam,ri	sh	dcol/2	Muv,right	Muv,left
10	W18x71	W14x176	26,98	26,98	117,16	116,87	9,25	7,75	458,63	458,63
9	W18x71	W14x176	16,73	16,73	128,81	128,60	9,25	7,75	284,47	284,47
8	W18x71	W14x176	16,73	16,73	128,76	128,60	9,25	7,75	284,47	284,47
7	W18x71	W14x176	16,73	16,73	128,73	128,60	9,25	7,75	284,47	284,47
6	W18x106	W14x193	16,72	16,72	163,37	162,98	9,35	7,75	285,98	285,98
5	W18x106	W14x193	16,72	16,72	163,28	162,98	9,35	7,75	285,98	285,98
4	W18x106	W14x193	16,72	16,72	163,21	162,98	9,35	7,75	285,98	285,98
3	W18x106	W14x193	16,72	16,72	163,12	162,98	9,35	7,75	285,98	285,98
2	W18x106	W14x193	16,72	16,72	163,04	162,98	9,35	7,75	285,98	285,98
1	W18x106	W14x193	16,72	16,72	162,82	162,98	9,35	7,75	285,98	285,98

Table 4.12 M_{uv} values for grid E

Story	Beam	Column	Vgravity,l	Vgravity,r	Vbeam,le	Vbeam,ri	sh	dcol/2	Muv,right	Muv,left
10	W18x71	W14x176	26,98	26,98	115,93	116,59	9,25	7,75	458,63	458,63
9	W18x71	W14x176	16,73	16,73	124,65	128,38	9,25	7,75	284,47	284,47
8	W18x71	W14x176	16,73	16,73	125,41	128,42	9,25	7,75	284,47	284,47
7	W18x71	W14x176	16,73	16,73	125,63	128,46	9,25	7,75	284,47	284,47
6	W18x106	W14x193	16,72	16,72	159,09	162,58	9,35	7,75	285,98	285,98
5	W18x106	W14x193	16,72	16,72	159,80	162,67	9,35	7,75	285,98	285,98
4	W18x106	W14x193	16,72	16,72	160,74	162,74	9,35	7,75	285,98	285,98
3	W18x106	W14x193	16,72	16,72	161,85	162,83	9,35	7,75	285,98	285,98
2	W18x106	W14x193	16,72	16,72	163,03	162,91	9,35	7,75	285,98	285,98
1	W18x106	W14x193	16,72	16,72	165,05	163,13	9,35	7,75	285,98	285,98

Table 4.13 M_{uv} values for grid F

Story	Beam	Column	Vgravity	Vbeam	sh	dcol/2	Muv
10	W18x71	W14x176	26,98	69,73	9,25	7,75	458,63
9	W18x71	W14x176	16,73	59,49	9,25	7,75	284,47
8	W18x71	W14x176	16,73	59,49	9,25	7,75	284,47
7	W18x71	W14x176	16,73	59,49	9,25	7,75	284,47
6	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98
5	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98
4	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98
3	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98
2	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98
1	W18x106	W14x193	16,72	84,11	9,35	7,75	285,98

$\sum M_{pb} =$ Expected flexural strength of the beam at the plastic hinge location.

$\sum M_{pb} = \sum(1{,}1 R_y F_{yb} Z_b + M_{uv})$, the results are given below: (4.33)

Table 4.14 M_{pb} values for grid A

Story	Beam	Column	Ry	Fyb	Zb	Mpb
10	W18x71	W14x176	1,21	50	146	10174,9
9	W18x71	W14x176	1,21	50	146	10000,8
8	W18x71	W14x176	1,21	50	146	10000,8
7	W18x71	W14x176	1,21	50	146	10000,8
6	W18x106	W14x193	1,21	50	230	15592,5
5	W18x106	W14x193	1,21	50	230	15592,5
4	W18x106	W14x193	1,21	50	230	15592,5
3	W18x106	W14x193	1,21	50	230	15592,5
2	W18x106	W14x193	1,21	50	230	15592,5
1	W18x106	W14x193	1,21	50	230	15592,5

Table 4.15 M_{pb} values for grid B

Story	Beam	Column	Ry	Fyb	Zb	Mpb,left	Mpb,right
10	W18x71	W14x176	1,21	50	146	10174,93	10174,93
9	W18x71	W14x176	1,21	50	146	10000,77	10000,77
8	W18x71	W14x176	1,21	50	146	10000,77	10000,77
7	W18x71	W14x176	1,21	50	146	10000,77	10000,77
6	W18x106	W14x193	1,21	50	230	15592,48	15592,48
5	W18x106	W14x193	1,21	50	230	15592,48	15592,48
4	W18x106	W14x193	1,21	50	230	15592,48	15592,48
3	W18x106	W14x193	1,21	50	230	15592,48	15592,48
2	W18x106	W14x193	1,21	50	230	15592,48	15592,48
1	W18x106	W14x193	1,21	50	230	15592,48	15592,48

Table 4.16 M_{pb} values for grid C

Story	Beam	Column	Ry	Fyb	Zb	Mpb,left	Mpb,right
10	W18x71	W14x176	1,21	50	146	10174,93	10174,93
9	W18x71	W14x176	1,21	50	146	10000,77	10000,77
8	W18x71	W14x176	1,21	50	146	10000,77	10000,77
7	W18x71	W14x176	1,21	50	146	10000,77	10000,77
6	W18x106	W14x193	1,21	50	230	15592,48	15592,48
5	W18x106	W14x193	1,21	50	230	15592,48	15592,48
4	W18x106	W14x193	1,21	50	230	15592,48	15592,48
3	W18x106	W14x193	1,21	50	230	15592,48	15592,48
2	W18x106	W14x193	1,21	50	230	15592,48	15592,48
1	W18x106	W14x193	1,21	50	230	15592,48	15592,48

Table 4.17 M_{pb} values for grid D

Story	Beam	Column	Ry	Fyb	Zb	Mpb,left	Mpb,right
10	W18x71	W14x176	1,21	50	146	10174,93	10174,93
9	W18x71	W14x176	1,21	50	146	10000,77	10000,77
8	W18x71	W14x176	1,21	50	146	10000,77	10000,77
7	W18x71	W14x176	1,21	50	146	10000,77	10000,77
6	W18x106	W14x193	1,21	50	230	15592,48	15592,48
5	W18x106	W14x193	1,21	50	230	15592,48	15592,48
4	W18x106	W14x193	1,21	50	230	15592,48	15592,48
3	W18x106	W14x193	1,21	50	230	15592,48	15592,48
2	W18x106	W14x193	1,21	50	230	15592,48	15592,48
1	W18x106	W14x193	1,21	50	230	15592,48	15592,48

Table 4.18 M_{pb} values for grid E

Story	Beam	Column	Ry	Fyb	Zb	Mpb,left	Mpb,right
10	W18x71	W14x176	1,21	50	146	10174,93	10174,93
9	W18x71	W14x176	1,21	50	146	10000,77	10000,77
8	W18x71	W14x176	1,21	50	146	10000,77	10000,77
7	W18x71	W14x176	1,21	50	146	10000,77	10000,77
6	W18x106	W14x193	1,21	50	230	15592,48	15592,48
5	W18x106	W14x193	1,21	50	230	15592,48	15592,48
4	W18x106	W14x193	1,21	50	230	15592,48	15592,48
3	W18x106	W14x193	1,21	50	230	15592,48	15592,48
2	W18x106	W14x193	1,21	50	230	15592,48	15592,48
1	W18x106	W14x193	1,21	50	230	15592,48	15592,48

Table 4.19 M_{pb} values for grid F

Story	Beam	Column	Ry	Fyb	Zb	Mpb
10	W18x71	W14x176	1,21	50	146	10174,93
9	W18x71	W14x176	1,21	50	146	10000,77
8	W18x71	W14x176	1,21	50	146	10000,77
7	W18x71	W14x176	1,21	50	146	10000,77
6	W18x106	W14x193	1,21	50	230	15592,48
5	W18x106	W14x193	1,21	50	230	15592,48
4	W18x106	W14x193	1,21	50	230	15592,48
3	W18x106	W14x193	1,21	50	230	15592,48
2	W18x106	W14x193	1,21	50	230	15592,48
1	W18x106	W14x193	1,21	50	230	15592,48

Table 4.20 Axial forces for combination $1,4D+L+2,5E$

Story	A	Grid	B	Grid	C	Grid	D	Grid	E	Grid	F	Grid
10below	20,39	20,39	54,37	54,37	55,48	55,48	55,11	55,11	56,16	56,16	34,95	34,95
9above	20,39	50,89	54,37	153,4	55,48	158,28	55,11	157,34	56,16	159,3	34,95	109,59
9below	30,5		99,03		102,8		102,23		103,14		74,64	
8above	30,5	62,53	99,03	243,89	102,8	252,86	102,23	251,59	103,14	252,81	74,64	196,89
8below	32,03		144,86		150,06		149,36		149,67		122,25	
7above	32,03	60,75	144,86	335,79	150,06	347,39	149,36	345,84	149,67	345,72	122,25	296,87
7below	28,72		190,93		197,33		196,48		196,05		174,62	
6above	28,72	42,64	190,93	428,67	197,33	441,97	196,48	440,15	196,05	436,9	174,62	413,8
6below	13,92		237,74		244,64		243,67		240,85		239,18	
5above	13,92	16,83	237,74	522,68	244,64	536,58	243,67	534,52	240,85	526,57	239,18	544,54
5below	2,91		284,94		291,94		290,85		285,72		305,36	
4above	2,91	25,54	284,94	617,99	291,94	631,1	290,85	628,91	285,72	615,98	305,36	679,23
4below	22,63		333,05		339,16		338,06		330,26		373,87	
3above	22,63	67,1	333,05	715,15	339,16	725,48	338,06	723,37	330,26	704,82	373,87	817,71
3below	44,47		382,1		386,32		385,31		374,56		443,84	
2above	44,47	111,31	382,1	814,09	386,32	819,74	385,31	817,9	374,56	793,27	443,84	957,52
2below	66,84		431,99		433,42		432,59		418,71		513,68	
1above	66,84	151,51	431,99	915,29	433,42	913,74	432,59	912,56	418,71	881,44	513,68	1091,43
1below	84,67		483,3		480,32		479,97		462,73		577,75	

It is necessary to calculate $\sum M_{pc}$. It is the sum of the nominal flexural strengths of the columns above and below the joint to the beam centerline. It is calculated as:

$$\sum M_{pc} = \sum Z_c (F_{yc} - P_{uc} / A_g) \tag{4.34}$$

Result table for $\sum M_{pc}$ is given below:

Table 4.21 Result table for M_{pc}

Story	A	B	C	D	E	F
10	15874,04	15664,12	15657,27	15659,55	15659,55	15653,07
9	31685,62	31052,36	31022,21	31028,02	31028,02	31028,02
8	31613,71	32000,00	30437,93	30437,93	30445,78	30445,78
7	31624,71	28986,69	29853,96	29853,96	30445,78	30445,78
6	35233,50	32820,81	32737,69	32737,69	33338,50	33338,50
5	35394,81	32233,25	33328,81	33328,81	33338,50	33338,50
4	35340,38	31637,56	33033,38	32737,69	32749,06	32749,06
3	35233,50	31030,31	30965,75	30965,75	32749,06	32749,06
2	34804,31	30411,94	30376,63	30376,63	32159,25	32159,25
1	34553,06	29779,44	29789,13	29789,13	32159,25	32159,25

Following condition should be satisfied:

$$\frac{\sum M_{pc}}{\sum M_{pb}} \tag{4.35}$$

Result table for the condition above is given below:

Table 4.22 Results for the condition

sh	Lh	Fy	Fu	Ry	Zb	Mpr
9,25	341,5	50	65	1	146	7300
9,25	341,5	50	65	1	146	7300
9,25	341,5	50	65	1	146	7300
9,25	341,5	50	65	1	146	7300
9,35	341,3	50	65	1	230	11500
9,35	341,3	50	65	1	230	11500
9,35	341,3	50	65	1	230	11500
9,35	341,3	50	65	1	230	11500
9,35	341,3	50	65	1	230	11500
9,35	341,3	50	65	1	230	11500

At the top stories, it is not necessary to satisfy this condition. As we can see at other stories this condition is satisfied.

4.13. Design of Beams

When designing the beams, it is supposed that the members are subjected to simple bending about one principal axis of the cross section because of the rigid diaphragm action. C_b , the lateral-torsional buckling modification factor for no uniform moment diagrams when both ends of the segment are braced is determined as follows:

$$C_b = \frac{12,5M_{\max}}{2,5M_{\max} + 3M_A + 4M_B + 3M_C} \tag{4.36}$$

Results for C_b value are given:

Table 4.23 C_b results for A-B

Story	Mmax	Ma	Mb	Mc	Cb
10	142,77	47,59	58,14	5,99	2,38
9	198,2	57,11	58,8	27,09	2,52
8	252,65	87,71	61,04	34,94	2,54
7	282,48	124,32	131,34	53,21	2,00
6	356,85	146,84	65,74	102,97	2,34
5	368,2	152,1	65,46	90,14	2,41
4	385,53	161,47	65,94	117,22	2,34
3	398,13	168,11	66,16	123,41	2,33
2	398,51	167,77	65,81	123,77	2,33
1	357,6	147,81	66,14	103,15	2,34

Table 4.24 C_b results for B-C

Mmax	Ma	Mb	Mc	Cb
160,59	40,9	47,74	20,11	2,59
218,43	61,57	55,03	39,1	2,56
267,68	86,08	54,95	63,74	2,50
295,9	100,11	54,9	77,91	2,48
373,9	139,26	55	116,86	2,43
382,19	143,23	54,89	121,07	2,43
394,05	148,93	54,74	127,08	2,42
400,62	151,95	54,56	130,45	2,42
394,8	148,83	54,42	127,61	2,43
345,98	123,9	54,07	103,37	2,45

Table 4.25 C_b results for C-D

Mmax	Ma	Mb	Mc	Cb
159,3	38,39	46,49	20,08	2,62
218,02	60,76	54,63	39,1	2,56
267,43	85,45	54,61	63,81	2,50
295,68	99,59	54,62	77,93	2,48
373,44	138,23	54,47	116,9	2,44
381,81	142,43	54,49	121,08	2,43
393,87	148,46	54,49	127,11	2,43
400,65	151,86	54,49	130,5	2,42
394,16	149,11	54,82	127,09	2,42
346,01	124,89	54,72	103,06	2,45

Table 4.26 C_b results for D-E

Mmax	Ma	Mb	Mc	Cb
159,3	38,39	46,49	20,08	2,62
218,02	60,76	54,63	39,1	2,56
267,43	85,45	54,61	63,81	2,50
295,68	99,59	54,62	77,93	2,48
373,44	138,23	54,47	116,9	2,44
381,81	142,43	54,49	121,08	2,43
393,87	148,46	54,49	127,11	2,43
400,65	151,86	54,49	130,5	2,42
394,16	149,11	54,82	127,09	2,42
346,01	124,89	54,72	103,06	2,45

Table 4.27 C_b results for E-F

Mmax	Ma	Mb	Mc	Cb
157,17	42,58	50	17,26	2,54
224,66	64,51	54,91	42,28	2,55
277,55	89,29	53,79	69,28	2,51
306,39	103,28	53,51	83,84	2,48
398,03	143,93	50,08	131,4	2,46
404,21	147,55	50,43	134,31	2,46
416,69	153,03	49,92	140,8	2,45
422,75	155,79	49,74	143,92	2,45
415,58	152,24	49,76	140,33	2,46
361,08	126,87	51,02	90,24	2,57

There are three limit states for lateral torsional buckling. These conditions are:

a) $L_p \geq L_b$

b) $L_r \geq L_b > L_p$

c) $L_b > L_r$

- **L_p and L_r**

Formulas for L_p and L_r are given below:

$$L_p = 1,76 r_y \sqrt{\frac{E}{F_y}} \tag{4.37}$$

$$L_r = 1,95 r_{ts} \frac{E}{0,7F_y} \sqrt{\frac{J_c}{S_x h_o} + \sqrt{\left(\frac{J_c}{S_x h_o}\right)^2 + 6,76\left(\frac{0,7F_y}{E}\right)^2}} \tag{4.38}$$

Table 4.28 Results for L_p

Story	Beam	r_y	Lp(in)	Lp(ft)
10	W18x71	1,7	72,06	6,00
9	W18x71	1,7	72,06	6,00
8	W18x71	1,7	72,06	6,00
7	W18x71	1,7	72,06	6,00
6	W18x106	2,66	112,75	9,40
5	W18x106	2,66	112,75	9,40
4	W18x106	2,66	112,75	9,40
3	W18x106	2,66	112,75	9,40
2	W18x106	2,66	112,75	9,40
1	W18x106	2,66	112,75	9,40

Table 4.29 Results for L_r

Story	Beam	rts	J	Sx	ho	Lr(ft)
10	W18x71	2,05	3,49	127	17,7	19,622
9	W18x71	2,05	3,49	127	17,7	19,622
8	W18x71	2,05	3,49	127	17,7	19,622
7	W18x71	2,05	3,49	127	17,7	19,622
6	W18x106	3,1	7,48	204	17,8	31,825
5	W18x106	3,1	7,48	204	17,8	31,825
4	W18x106	3,1	7,48	204	17,8	31,825
3	W18x106	3,1	7,48	204	17,8	31,825
2	W18x106	3,1	7,48	204	17,8	31,825
1	W18x106	3,1	7,48	204	17,8	31,825

L_b =25/5=5ft so the condition b is satisfied.

$L_b \leq 0,086x50xr_y = 7,31$ condition is satisfied.

$$M_n = M_p = Z_x F_y \qquad (4.39)$$

$\theta_b = 0,9$ (LRFD)

Table 4.30 Result table for $\theta_b x M_n$

Story	Beam	Z_x(in)	M_n=M_p(kipin)	M_n=M_p(kpft)	$\Theta_b x M_n$
10	W18x71	146	7300	608	547,5
9	W18x71	146	7300	608	547,5
8	W18x71	146	7300	608	547,5
7	W18x71	146	7300	608	547,5
6	W18x106	230	11500	958	862,5
5	W18x106	230	11500	958	862,5
4	W18x106	230	11500	958	862,5
3	W18x106	230	11500	958	862,5
2	W18x106	230	11500	958	862,5
1	W18x106	230	11500	958	862,5

The condition that has to be satisfied in this section is $\dfrac{M_u}{\theta_b x M_n} < 1$.

Table 4.31 The results for beam sections

M_u/ΘbxMn<1		Grid A-B		Grid B-C		Grid C-D		Grid D-E		Grid E-F	
Story	Beam	M_u	$\Theta_b x M_n$	M_u	$\Theta_b x M_n$	M_u	$\Theta_b x M_n$	M_u	$\Theta_b x M_n$	M_u	$\Theta_b x M_n$
10	W18x71	142,77	547,5	160,59	547,5	159,3	547,5	159,3	547,5	159,3	547,5
9	W18x71	198,2	547,5	218,43	547,5	218,02	547,5	218,02	547,5	218,02	547,5
8	W18x71	252,65	547,5	267,68	547,5	267,43	547,5	267,43	547,5	267,43	547,5
7	W18x71	282,48	547,5	295,9	547,5	295,68	547,5	295,68	547,5	295,68	547,5
6	W18x106	356,85	862,5	373,9	862,5	373,44	862,5	373,44	862,5	373,44	862,5
5	W18x106	368,2	862,5	382,19	862,5	381,81	862,5	381,81	862,5	381,81	862,5
4	W18x106	385,53	862,5	394,05	862,5	393,87	862,5	393,87	862,5	393,87	862,5
3	W18x106	398,13	862,5	400,62	862,5	400,65	862,5	400,65	862,5	400,65	862,5
2	W18x106	398,51	862,5	394,8	862,5	394,16	862,5	394,16	862,5	394,16	862,5
1	W18x106	357,6	862,5	345,98	862,5	346,01	862,5	346,01	862,5	346,01	862,5
Story	Beam	GRID A-B	GRID B-C	GRID C-D	GRID D-E	GRID E-F					
10	W18x71	0,26	0,29	0,29	0,29	0,29	DOĞRU				
9	W18x71	0,36	0,40	0,40	0,40	0,40	DOĞRU				
8	W18x71	0,46	0,49	0,49	0,49	0,49	DOĞRU				
7	W18x71	0,52	0,54	0,54	0,54	0,54	DOĞRU				
6	W18x106	0,41	0,43	0,43	0,43	0,43	DOĞRU				
5	W18x106	0,43	0,44	0,44	0,44	0,44	DOĞRU				
4	W18x106	0,45	0,46	0,46	0,46	0,46	DOĞRU				
3	W18x106	0,46	0,46	0,46	0,46	0,46	DOĞRU				
2	W18x106	0,46	0,46	0,46	0,46	0,46	DOĞRU				
1	W18x106	0,41	0,40	0,40	0,40	0,40	DOĞRU				

4.14. Design of Columns

Bending moment values of the columns those are going to be used in calculations are given below:

Table 4.32 Results for grid A

Columns	Mmax	Ma	Mb	Mc	Pu	Mabove	Mbelow
10	167,31	86,7	6,09	74,52	47,75	155,13	167,32
9	99,18	73,98	52,65	27,44	101,89	1,65	99,18
8	46,57	33,75	20,93	8,11	151,08	4,7	46,5
7	24,34	13,95	17,41	19	197,62	24,33	10,49
6	5,94	2,15	3,41	4,67	237,64	5,94	0,88
5	31,66	3,45	8,23	23,55	276,48	31,65	15,21
4	38,67	13,31	4,08	21,47	313,5	38,3	30,7
3	53,95	30,07	0	23,2	349,08	41,58	53,95
2	76,83	52,14	27,51	2,88	384,35	21,75	76,78
1	354,9	275,54	196,18	116,83	422,63	34,7	354,9

Table 4.33 Results for grid B

Columns	Mmax	Ma	Mb	Mc	Pu	Mabove	Mbelow
10	100,77	5,26	37,1	68,94	103,17	100,72	26,57
9	166,08	29,5	35,71	100,9	222,46	166,83	25,69
8	208,07	72,58	20,97	114,52	342,69	208,67	166,13
7	260,35	145,67	30,56	84,55	462,99	199,66	260,78
6	267,67	127,38	4,31	135,59	582,99	267,67	259,06
5	279,58	136,51	6,56	150	703,46	292,71	279,58
4	304,96	148,73	3,16	155,06	824,74	306,96	300,63
3	318,65	161,47	4,29	201,26	946,9	309,62	318,65
2	352,95	194,3	31,64	129,01	1069,91	289,66	352,95
1	438,3	288,41	138,51	11,38	1194,29	161,27	438,2

Table 4.34 Results for grid C

Columns	Mmax	Ma	Mb	Mc	Pu	Mabove	Mbelow
10	120,43	3,58	37,76	79,1	104,05	120,43	44,92
9	182,21	36,48	36,47	109,43	225,92	182,22	109,44
8	227,11	78,43	21,42	121,27	347,75	221,12	178,29
7	277,12	154,72	32,17	90,3	469,56	212,92	277,25
6	284,95	134,71	5,18	145,06	591,47	284,95	274,59
5	290,94	141,78	7,4	202,49	713,34	305,77	290,96
4	315,63	151,52	4,2	159,92	835,17	315,63	307,24
3	320,09	162,17	3,45	155,26	986,95	313,89	320,89
2	346,78	188,16	29,45	129,06	1078,69	287,68	346,78
1	435,57	287,99	140,39	7,2	1200,3	154,79	435,58

Table 4.35 Results for grid D

Columns	Mmax	Ma	Mb	Mc	Pu	Mabove	Mbelow
10	120,43	3,58	37,76	79,1	104,05	120,43	44,92
9	182,21	36,48	36,47	109,43	225,92	182,22	109,44
8	227,11	78,43	21,42	121,27	347,75	221,12	178,29
7	277,12	154,72	32,17	90,3	469,56	212,92	277,25
6	284,95	134,71	5,18	145,06	591,47	284,95	274,59
5	290,94	141,78	7,4	202,49	713,34	305,77	290,96
4	315,63	151,52	4,2	159,92	835,17	315,63	307,24
3	320,09	162,17	3,45	155,26	986,95	313,89	320,89
2	346,78	188,16	29,45	129,06	1078,69	287,68	346,78
1	435,57	287,99	140,39	7,2	1200,3	154,79	435,58

Table 4.36 Results for grid E

Columns	Mmax	Ma	Mb	Mc	Pu	Mabove	Mbelow
10	130,5	9,04	37,47	83,99	103,67	130,5	55,55
9	193,3	41,88	90,81	169,21	223,75	193,17	120,28
8	231,31	82,96	21,8	126,56	343,88	231,32	187,71
7	292,81	163,28	34,17	39,73	464,06	224,06	292,39
6	301,65	141,86	5,97	153,81	582,78	301,65	289,7
5	319,11	148,2	7,65	211,47	701,91	319,37	304,06
4	327,67	156,81	4,68	166,17	821,31	327,67	318,3
3	329,3	166,33	2,99	160,34	941,11	323,68	329,67
2	351,44	189,99	28,55	132,9	1061,58	294,34	351,44
1	436,53	288,13	139,74	8,66	1181,63	157,05	436,53

Table 4.37 Results for grid F

Columns	Mmax	Ma	Mb	Mc	Pu	Mabove	Mbelow
10	130,5	9,04	37,47	83,99	103,67	275,75	147,35
9	193,3	41,88	90,81	169,21	223,75	243,99	168,42
8	231,31	82,96	21,8	126,56	343,88	274,68	227,26
7	292,81	163,28	34,17	39,73	464,06	245,98	292,58
6	301,65	141,86	5,97	153,81	582,78	286,02	267,93
5	319,11	148,2	7,65	211,47	701,91	305,86	286,66
4	327,67	156,81	4,68	166,17	821,31	306,65	296,17
3	329,3	166,33	2,99	160,34	941,11	299,51	304,85
2	351,44	189,99	28,55	132,9	1061,58	279,6	351,72
1	436,53	288,13	139,74	8,66	1181,63	164,9	439,82

M_{max} = Maximum moment value on the girder.

M_a = Moment value at the point of 6, 25 ft on the girder.

$M_b =$ Moment value at the point of 12.5 ft on the girder.

$M_c =$ Moment value at the point of 18.75 ft on the girder.

$P_u =$ Axial force on the column.

$M_{above} =$ The above moment of end grid on the column.

$M_{below} =$ The below moment of end grid on the column.

- G_A & G_B *Values*

The A and B refer to the joints at the ends of the columns being considered. It is supposed that all

the joints are rigidly connected. The G value is calculated as:

$$G = \frac{\sum (E_c I_c / L_c)}{\sum (E_g I_g / L_g)} = \frac{\sum (EI / L)_c}{\sum (EI / L)_g} \tag{4.40}$$

$E_c =$ Elastic modulus of the column.

$I_c =$ The moment of inertia of the column.

$L_c =$ The unsupported length of the column.

$E_g =$ The elastic modulus of the girder.

$I_g =$ The moment of inertia of the girder.

$L_g =$ The unsupported length of the girder.

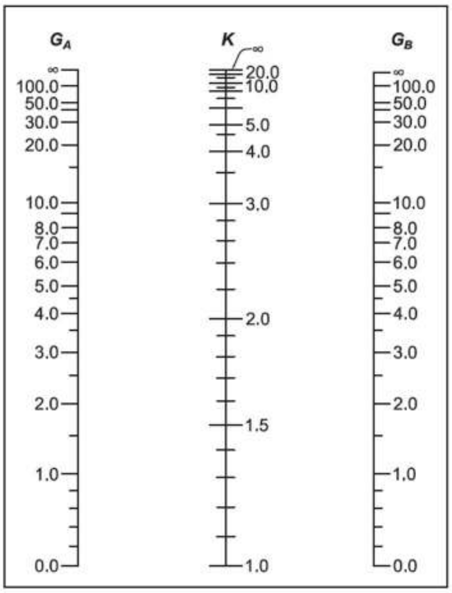
The result table of the G_A & G_B is given below which will be utilized during effective length calculations for compression of columns:

Table 4.38 The results for Grid A and F

Story	E_c	I_c	E_g	I_g	L_c	L_g	Ga	Gb
10	29000	2660	29000	1170	156	300	4,37	2,19
9	29000	2660	29000	1170	156	300	4,37	4,37
8	29000	2660	29000	1170	156	300	4,37	4,37
7	29000	2660	29000	1170	156	300	4,37	4,37
6	29000	3400	29000	1910	156	300	3,42	3,42
5	29000	3400	29000	1910	156	300	3,42	3,42
4	29000	3400	29000	1910	156	300	3,42	3,42
3	29000	3400	29000	1910	156	300	3,42	3,42
2	29000	3400	29000	1910	156	300	3,42	3,42
1	29000	3400	29000	1910	156	300	3,42	3,42

Table 4.39 The results for Grid B,C,D,E and F

Story	E_c	I_c	E_g	I_g	L_c	L_g	Ga	Gb
10	29000	2660	29000	1170	156	300	2,91	1,46
9	29000	2660	29000	1170	156	300	2,91	2,91
8	29000	2660	29000	1170	156	300	2,91	2,91
7	29000	2660	29000	1170	156	300	2,91	2,91
6	29000	3400	29000	1910	156	300	2,28	2,28
5	29000	3400	29000	1910	156	300	2,28	2,28
4	29000	3400	29000	1910	156	300	2,28	2,28
3	29000	3400	29000	1910	156	300	2,28	2,28
2	29000	3400	29000	1910	156	300	2,28	2,28
1	29000	3400	29000	1910	156	300	2,28	2,28



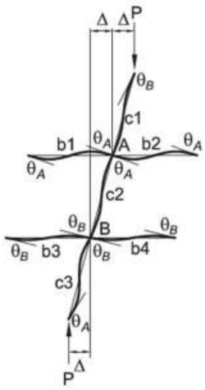


Figure 4.2 G_A & G_B values alignment chart

Şekil 1C-A-7.2. Alignment chart sidesway—uninhibited (moment frame).(AISC 360-10). The result table of $K_x L / r_x$ and $K_y L / r_y$ are given below:

Table 4.40 Grid A and F

Story	Columns	Kx	L(in)	r_x	KxL/rx
10	W14x176	2,2	156	6,43	53,37
9	W14x176	2,6	156	6,43	63,08
8	W14x176	2,6	156	6,43	63,08
7	W14x176	2,4	156	6,43	58,23
6	W14x193	2,2	156	6,50	52,80
5	W14x193	2,2	156	6,50	52,80
4	W14x193	2,2	156	6,50	52,80
3	W14x193	2,2	156	6,50	52,80
2	W14x193	2,2	156	6,50	52,80
1	W14x193	1,7	156	6,50	40,80

Table 4.41 Grid B,C,D,E and F

Story	Columns	Kx	L(in)	r_x	KxL/rx
10	W14x176	1,7	156	6,43	41,24
9	W14x176	1,9	156	6,43	46,10
8	W14x176	1,9	156	6,43	46,10
7	W14x176	1,8	156	6,43	43,67
6	W14x193	1,7	156	6,50	40,80
5	W14x193	1,7	156	6,50	40,80
4	W14x193	1,7	156	6,50	40,80
3	W14x193	1,7	156	6,50	40,80
2	W14x193	1,7	156	6,50	40,80
1	W14x193	1,5	156	6,50	36,00

4.14.1. The Coefficient of C_m

The coefficient of C_m is used to determine the maximum second order moments in compression members with no relative joints translation and no transverse loads between the ends of the member. It is calculated as:

$$C_m = 0,6 - 0,4 \frac{M_1}{M_2} \tag{4.41}$$

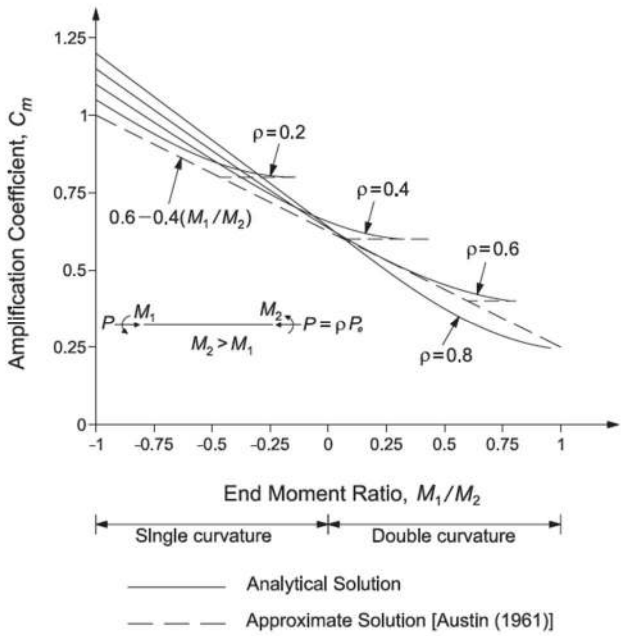


Figure 4.3 Calculation for the coefficient of C_m

Table 4.42 The result table for the coefficient of C_m

A		B		C		D		E		F	
Story	Cm	Story	Cm	Story	Cm	Story	Cm	Story	Cm	Story	Cm
10	0,06	10	0,11	10	-13,40	10	-13,40	10	0,11	10	0,06
9	0,23	9	0,19	9	-0,07	9	-0,07	9	0,19	9	0,23
8	0,19	8	0,17	8	-0,60	8	-0,60	8	0,17	8	0,19
7	0,22	7	0,29	7	-1,00	7	-1,00	7	0,29	7	0,22
6	0,20	6	0,16	6	0,01	6	0,01	6	0,16	6	0,20
5	0,20	5	0,16	5	0,20	5	0,20	5	0,16	5	0,20
4	0,20	4	0,11	4	-0,13	4	-0,13	4	0,11	4	0,20
3	0,19	3	-1,48	3	0,28	3	0,28	3	-1,48	3	0,19
2	0,22	2	-0,65	2	-0,87	2	-0,87	2	-0,65	2	0,22
1	-0,35	1	-0,36	1	-0,36	1	-0,36	1	-0,36	1	-0,35

4.14.2. The Coefficient of P_{e1}

P_{e1} is the elastic critical buckling strength of the member in major-axis of bending. It is assumed that no lateral translation at the member ends. It is calculated as:

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \qquad (4.42)$$

$EI =$ Flexural rigidity.

$E =$ Modulus of elasticity (29000 ksi)

$I =$ Moment of inertia

$L =$ Length of member

$K_1 =$ Effective length factor.

Table 4.43 The result table for P_{e1}

A		B		C		D		E		F	
Story	P_{e1}	Story	P_{e1}	Story	P_{e1}	Story	P_{e1}	Story	P_{e1}	Story	P_{e1}
10	31252,9	10	31252,9	10	31252,9	10	31252,9	10	31252,9	10	31252,9
9	31252,9	9	31252,9	9	31252,9	9	31252,9	9	31252,9	9	31252,9
8	31252,9	8	31252,9	8	31252,9	8	31252,9	8	31252,9	8	31252,9
7	31252,9	7	31252,9	7	31252,9	7	31252,9	7	31252,9	7	31252,9
6	39947,3	6	39947,3	6	39947,3	6	39947,3	6	39947,3	6	39947,3
5	39947,3	5	39947,3	5	39947,3	5	39947,3	5	39947,3	5	39947,3
4	39947,3	4	39947,3	4	39947,3	4	39947,3	4	39947,3	4	39947,3
3	39947,3	3	39947,3	3	39947,3	3	39947,3	3	39947,3	3	39947,3
2	39947,3	2	39947,3	2	39947,3	2	39947,3	2	39947,3	2	39947,3
1	39947,3	1	39947,3	1	39947,3	1	39947,3	1	39947,3	1	39947,3

4.14.3. Multiplier B_1 for P-δ Effects

The B_1 multiplier is calculated as:

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \tag{4.43}$$

α=1,00(LFRD)

Table 4.44 The result table for B_1

A		B		C		D		E		F	
Story	β_1	Story	β_1	Story	β_1	Story	β_1	Story	β_1	Story	β_1
10	0,06	10	0,11	10	-13,42	10	-13,42	10	0,11	10	0,06
9	0,23	9	0,19	9	-0,07	9	-0,07	9	0,19	9	0,23
8	0,19	8	0,17	8	-0,60	8	-0,60	8	0,17	8	0,19
7	0,22	7	0,29	7	-1,01	7	-1,01	7	0,29	7	0,22
6	0,20	6	0,16	6	0,01	6	0,01	6	0,16	6	0,20
5	0,20	5	0,17	5	0,20	5	0,20	5	0,17	5	0,21
4	0,20	4	0,11	4	-0,13	4	-0,13	4	0,11	4	0,20
3	0,19	3	-1,50	3	0,28	3	0,28	3	-1,50	3	0,19
2	0,23	2	-0,65	2	-0,88	2	-0,88	2	-0,65	2	0,23
1	-0,35	1	-0,36	1	-0,36	1	-0,36	1	-0,36	1	-0,36

4.14.4. Multiplier B_2 for P-δ Effects

The B_2 multiplier is calculated as:

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{estory}}} \tag{4.44}$$

α=1,00(LFRD)

P_{story} = Total vertical load supported by the story.

$P_{e,story}$ = Elastic critical buckling strength for the story. $P_{e,story}$ is calculated as:

$$P_{e,story} = R_M \frac{HL}{\Delta_H} \tag{4.44}$$

B_2 multiplier is considered as 1,00 for 2D structure.

4.14.5. L_p and L_r Values

L_p & L_r values are determined as:

$$L_p = 1,76 r_y \sqrt{\frac{E}{F_y}} \tag{4.45}$$

$$L_r = 1,95 r_{ts} \frac{E}{0,7F_y} \sqrt{\frac{J_c}{S_x h_o} + \sqrt{\left(\frac{J_c}{S_x h_o}\right)^2 + 6,76\left(\frac{0,7F_y}{E}\right)^2}} \tag{4.46}$$

Table 4.45 The result tables for L_p & L_r

Story	Column	rts	J	Sx	ho	Lr(ft)
10	W14x176	4,55	26,5	281	13,9	352,30
9	W14x176	4,55	26,5	281	13,9	352,30
8	W14x176	4,55	26,5	281	13,9	352,30
7	W14x176	4,55	26,5	281	13,9	352,30
6	W14x193	4,59	34,8	310	14,1	356,04
5	W14x193	4,59	34,8	310	14,1	356,04
4	W14x193	4,59	34,8	310	14,1	356,04
3	W14x193	4,59	34,8	310	14,1	356,04
2	W14x193	4,59	34,8	310	14,1	356,04
1	W14x193	4,59	34,8	310	14,1	356,04

4.14.6. F_e Value

F_e value is the elastic buckling stress. F_e value is determined as:

$$F_e = \frac{\pi^2 E}{\frac{KL}{r}} \tag{4.47}$$

The result table for F_e is given below:

- *Grid A and F*

Table 4.46 The result table for F_e for grid A and F

Story	Columns	F_e
10	W14x176	100,37
9	W14x176	71,86
8	W14x176	71,86
7	W14x176	84,33
6	W14x193	102,56
5	W14x193	102,56
4	W14x193	102,56
3	W14x193	102,56
2	W14x193	102,56
1	W14x193	171,77

Table 4.47 The result table for F_e for grid B,C,D and E

Story	Columns	F_e
10	W14x176	189,87
9	W14x176	134,56
8	W14x176	134,56
7	W14x176	149,93
6	W14x193	171,77
5	W14x193	171,77
4	W14x193	171,77
3	W14x193	171,77
2	W14x193	171,77
1	W14x193	220,62

4.14.7. F_{cr} Value

F_{cr} is the critical stress. F_{cr} is determined as:

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}} \right] .F_y \tag{4.48}$$

The result table for F_{cr} is given below:

Table 4.48 The result table for F_{cr} for A and F

Story	Columns	F_{cr}
10	W14x176	40,59
9	W14x176	37,37
8	W14x176	37,37
7	W14x176	39,01
6	W14x193	40,77
5	W14x193	40,77
4	W14x193	40,77
3	W14x193	40,77
2	W14x193	40,77
1	W14x193	44,26

Table 4.49 The result table for F_{cr} for B,C,D and E

Story	Columns	F_{cr}
10	W14x211	44,78
9	W14x211	42,80
8	W14x211	42,80
7	W14x211	43,49
6	W14x257	44,26
5	W14x257	44,26
4	W14x257	44,26
3	W14x257	44,26
2	W14x257	44,26
1	W14x257	45,48

4.14.8. P_n Value

P_n is the nominal axial strength. P_n is determined as follows according to AISC LRFD Specification:

$$P_n = A_g F_{cr} \tag{4.49}$$

Table 4.50 The result table for P_n

Story	Columns	A_g	P_n
10	W14x176	51,8	2102,54
9	W14x176	51,8	1935,62
8	W14x176	51,8	1935,62
7	W14x176	51,8	2020,83
6	W14x193	56,8	2315,81
5	W14x193	56,8	2315,81
4	W14x193	56,8	2315,81
3	W14x193	56,8	2315,81
2	W14x193	56,8	2315,81
1	W14x193	56,8	2514,23

4.14.9. Interaction Equations

The condition that has to be satisfied for the members under combined effects of bending moment and axial force according to AISC LRFD Specification is:

$$\frac{P_u}{\theta_c P_n} + \frac{M_u}{\theta_b M_n} \leq 1 \tag{4.50}$$

Table 4.51 The result table for column sections for Grid A

Story	Columns	Result
10	W14x176	0,16
9	W14x176	0,14
8	W14x176	0,13
7	W14x176	0,13
6	W14x193	0,12
5	W14x193	0,16
4	W14x193	0,18
3	W14x193	0,21
2	W14x193	0,24
1	W14x193	0,45

Table 4.52 The result table for column sections for Grid B

Story	Columns	Result
10	W14x176	0,14
9	W14x176	0,27
8	W14x176	0,37
7	W14x176	0,47
6	W14x193	0,48
5	W14x193	0,55
4	W14x193	0,62
3	W14x193	0,69
2	W14x193	0,78
1	W14x193	0,86

Table 4.53 The result table for column sections for Grid C

Story	Columns	Result
10	W14x176	0,16
9	W14x176	0,28
8	W14x176	0,39
7	W14x176	0,49
6	W14x193	0,50
5	W14x193	0,56
4	W14x193	0,64
3	W14x193	0,71
2	W14x193	0,78
1	W14x193	0,86

Table 4.54 The result table for column sections for Grid D

Story	Columns	Result
10	W14x176	0,16
9	W14x176	0,28
8	W14x176	0,39
7	W14x176	0,49
6	W14x193	0,50
5	W14x193	0,56
4	W14x193	0,64
3	W14x193	0,71
2	W14x193	0,78
1	W14x193	0,86

Table 4.55 The result table for column sections for Grid E

Story	Columns	Result
10	W14x176	0,16
9	W14x176	0,29
8	W14x176	0,39
7	W14x176	0,50
6	W14x193	0,51
5	W14x193	0,58
4	W14x193	0,64
3	W14x193	0,70
2	W14x193	0,77
1	W14x193	0,85

Table 4.56 The result table for column sections for Grid F

Story	Columns	Result
10	W14x176	0,16
9	W14x176	0,29
8	W14x176	0,39
7	W14x176	0,50
6	W14x193	0,51
5	W14x193	0,58
4	W14x193	0,64
3	W14x193	0,70
2	W14x193	0,77
1	W14x193	0,85

4.15. Design of Gravity Columns

Gravity column design is also included in order to simulate the P-Delta effects on seismic performance of the model frame. Before designing gravity columns, loads that sustained by inner columns shall be defined.

Table 4.57 Load values for structures

G_{Story} =	3.83	kN/m²
Q_{story} =	3.11	kN/m²
G_{Roof} =	3.11	kN/m²
Q_{Roof} =	2.87	kN/m²

$$Area(m^2) = (7.62x(9.14x7.62)/2)/1000 = 63,87m^2$$

$P_u(kN) = 1.4x(3.83x63.87) + 1.6x(3.11x63.87) = 660.2967kN(148.44ksi)$ (Load to Normal Story)

Load to Roof Story=128.45 ksi

Total Load=1464.36 ksi

- *Section:HSS16X16-5/8*

$$b = 16in$$
$$t = 0.625in$$

- *Calculations for Section HSS16x16-5/8*

$$E_c = 29000ksi$$
$$I_c = 79.8in^4$$
$$E_g = 29000ksi$$
$$I_g = 1910in^4$$
$$L_c = 156in^4$$
$$L_{g(x-x)} = 300in$$
$$L_{g(y-y)c} = 300in$$
$$L_{g(y-y)b} = 360in$$

$$G_a = 2x\frac{\left(29000x\frac{79,8}{156}\right)}{\left(2x29000x\frac{1910}{300}\right) + \left(2x29000x\frac{1910}{360}\right)} = 0.0438$$

$$G_b = 0.0438$$

$$K_x = 1$$
$$K_y = 1$$
$$r_y = 2.52$$
$$r_x = 3.72$$
$$\frac{K_x L_x}{r_x} = 1x\frac{156}{3.72} = 41.93548 \qquad (4.51)$$
$$\frac{K_y L_y}{r_y} = 1x\frac{156}{2.52} = 61.90476$$
$$F_y = 50ksi$$
$$E = 29000ksi$$

$$F_e = \pi^2 x \left(\frac{29000}{41.94^2} \right) = 162.5901 kip$$

$$F_{cr} = (0.658^{(50/162.5901)})x50 = 43.9613 kip$$
$$A_g = 5.77 in^2$$
$$P_n = 5.77x43.9613 = 253.6567 kip$$
$$\theta_n x P_n / P_u \leq 1 = 0.0438 / (0.9x253.6567) = 0.6502$$
$$E / F_y = 29000 / 50 = 580$$

$$Compactness = 1.4\sqrt{580} = 33.72$$

$b / t = 1.667$ Condition is satisfied, section is acceptable.

CHAPTER 5

NONLINEAR ANALYSIS

A two dimensional model of the frame is constructed with software PERFORM-3D. P-delta effects are taken into consideration with the use of leaning column approach. The gravity columns are taken into consideration, but the orthogonal SCBFs' columns are neglected since their effect is expected to be negligible.

All inelastic actions of the structural elements are deformation-controlled and accordingly expected material properties are used for structural elements' strength calculations with an R_y factor of 1.1. Mean expected strength calculations are carried out using the procedures contained in Load and Resistance Factor Design Specification for Structural Steel Buildings, except that the strength reduction factor, ϕ, is taken as unity.

Model building's composite slab results in the rigid diaphragm action which causes the beams of the lateral load resisting system to work under flexure without axial force, or at least negligible axial force. On the other hand, columns work under both axial force and flexure.

The model frame is fully restrained, satisfying the condition that the joint deformations including the panel zone deformation do not contribute more than 10% to the total lateral deflection of the frame, and the connection is at least as strong as the weaker of the two members being joined. It is assumed that the expected shear strength of panel zones exceeds the flexural strength of the beams at a beam-column connection, and the stiffness of the panel zone is at least 10 times larger than the flexural stiffness of the beam, and accordingly direct modeling of the panel zone is not required. Rigid end offsets are taken into consideration to represent the effective span of the beam.

Beam flanges are restrained against lateral torsional buckling, so they are capable of fully plastifying. The cross sections of the structural elements are made of seismically compact components in order to allow the development of full plastic behavior with a certain level of ductility in a stable manner. Furthermore, story drift ratios obtained by the dynamic analyses are not too high which may result in beam-column connection damage and strength degradation. Consequently, strain-hardening elasto-plastic behavior with lumped plasticity approach is adopted without strength or stiffness degradation. A strain hardening slope of 3% of the elastic slope is used for beams and columns since there isn't a greater strain hardening slope justified by test data. Yield rotations of beams and columns are determined along with (Eq. 5-1) and (Eq.5-2) of ASCE/SEI : 41-06 (2007).

In the first step of the analyses building is exposed to the static load combination given in ASCE/SEI : 41-06 as 1.1(D+0.25L), where D and L are dead and live loads, respectively. Afterwards, nonlinear dynamic analyses are carried out on the deformed frame. The software PERFORM-3D is used for nonlinear time history analyses of the frame (CSI, 2006).

Table 5.1 Calculations

Story	Beam	Fy	Zx	Fu
10	W18x71	50	146	669,17
9	W18x71	50	146	669,17
8	W18x71	50	146	669,17
7	W18x71	50	146	669,17
6	W18x106	50	230	1054,17
5	W18x106	50	230	1054,17
4	W18x106	50	230	1054,17
3	W18x106	50	230	1054,17
2	W18x106	50	230	1054,17
1	W18x106	50	230	1054,17

$$F_u = 1.1xF_y xZ_x = (1.1x50x230)/12 = 1054.167kpft \tag{5.1}$$

Table 5.2 Result Tables

Story	Column	Fy	Zy	Fu2	Zx	Fu3	Ag(in)	Ag(in)	Fu	Fcr_{out}	Fu	Fcr_{inner}	Fu
10	W14x176	50	163	747,08	320	1466,67	51,80	621,60	2849	39,37	2243,03	44,90	2558,55
9		50	163	747,08	320	1466,67	51,80	621,60	2849	36,11	2057,46	41,63	2372,34
8	W14x176	50	163	747,08	320	1466,67	51,80	621,60	2849	36,11	2057,46	41,63	2372,34
7		50	163	747,08	320	1466,67	51,80	621,60	2849	36,94	2104,99	42,35	2413,07
6	W14x193	50	180	825,00	355	1627,08	56,80	681,60	3124	37,47	2341,32	43,35	2708,24
5		50	180	825,00	355	1627,08	56,80	681,60	3124	39,05	2439,66	43,35	2708,24
4	W14x193	50	180	825,00	355	1627,08	56,80	681,60	3124	39,05	2439,66	43,35	2708,24
3		50	180	825,00	355	1627,08	56,80	681,60	3124	39,05	2439,66	43,35	2708,24
2	W14x193	50	180	825,00	355	1627,08	56,80	681,60	3124	39,05	2439,66	43,35	2708,24
1		50	180	825,00	355	1627,08	56,80	681,60	3124	43,98	2748,18	45,18	2823,11

$$F_{u2} = F_y Z_y = (1.1x50x180)/12 = 825kpft \quad (5.2)$$

$$F_{u3} = F_y Z_x = (1.1x50x355)/12 = 1627.083kpft$$

$$F_u = 1.1xF_y xA_g = (1.1x50x681.6)/12 = 3124kpft(Tension) \quad (5.3)$$

$$F_u = 1.1xF_{crinner} xA_g = 1.1x45.1842x681.6 = 2823.11kpft(Compression)$$

$$F_u = 1.1xF_{crout} xA_g = 1.1x43.99x56.8 = 2748.18kpft$$

The procedure applied herein during the establishment of ground motion record sets is a similar version of the one reported by PEER Strong Motion Database Center (PEER, 2010) however “single record” s are used instead of “record pairs”. This is done because it is stated in ASCE/SEI : 07-10 that *where two-dimensional analyses are performed, each ground motion shall consist of a horizontal acceleration history, selected from an actual recorded event*. Though randomly-oriented pairs of motions is recommended to avoid causing a biased prediction of structural response in the literature (Haselton et al., 2012), there is not any obligation nor guidance for how this should be done in ASCE/SEI : 07-10 for two dimensional analysis. The quantitative measure used to evaluate how well a time series conforms to the design response spectrum is the mean squared error (MSE) of the difference between the spectral accelerations of the record and the design response spectrum. The rules provided by the ASCE/SEI 07-10 concerning the spectrum matching procedure are followed where a coded MATLAB script coded by Dr.Nuri Özhendekci is used to this aim. The upper bound for the scale factors of far field records and near fault records are 3.25 and 1.75, respectively. Unfortunately, any recommendation concerning the upper bound usage for these scale factors does not exist in the code. Though, it was stated that "*Ground motions with large changes in amplitude and frequency content introduced by spectrum matching should not necessarily be rejected from a ground motion set*" in a recent report (NEHRP, 2011), it is decided to bound these scale factors as strictly as possible in order not to alter the seed records' characteristics much. The coded MATLAB script enables the user to determine an upper bound for the scale factors in order the scaled records can be as close as possible to the seed records. After a number of trials above given upper bounds satisfy the best match for the records used in this work. Actually, trying not to alter the seed record's characteristics much may seem unnecessary, however it should be underlined that the basic objective of this paper is to study the effects of different record sets with some distinctive characteristics.

In order to study the behavior of the model frame under dynamic loads three different ground motion sets are prepared with the same hazard level of 10 % possibility of exceedance in 50 years. Accordingly, the whole structural elements of the model frame are expected to satisfy Life Safety (LS) performance level. Ground motion record sets have some distinctive characteristics; they are established as near fault records with velocity pulses (NF-WP), near fault records without velocity pulses (NF-WOP) and far field records. Each set has 7 scaled ground motion records, the average of which approximates the design response spectrum. In ASCE/SEI 7-10, it is stated that the appropriate records of events should have magnitudes, fault distances, and mechanisms that are consistent with the maximum considered earthquake. However, according to a study conducted by Iervolino and Cornell, *there is little evidence to support the need for a careful site-specific process of record selection by magnitude and distance* (Iervolino and Cornell, 2005). Thus, a specific scenario is not established, actually this was a must in order to approximate the design spectrum with the best fitting MSE values and with scaling factors lower than 4. Furthermore, the horizontal components of the whole database of the PEER Strong Motion Database Center for site class C is used during the iterations for the search of the best fit (http://peer.berkeley.edu, 2014). The records of NF-WP have closest distances to the fault rupture between 0.5-7 km, whereas the corresponding ones are between 4.5-13.9 for the NF-WOP set. For FF set, closest distance to the fault rupture is between 20.7-60 km. The magnitude of the whole records are between 5.5-7.62. The directivity is not considered as a classifying characteristics, so both fault normal and fault parallel components are selected randomly while attempting to obtain the best fitting average spectrum. Some of the basic properties of the earthquake records of these three sets can be found in Table 1. The response spectra and average spectrum of each record set and design response spectrum are also provided in Fig 5.1, Fig 5.2, Fig 5.3,

In the first step the building is exposed to combination of 1, 1(D+0,25L), after that dynamic earthquake forces are applied to the building. Here is the list of earthquakes which are applied;

1. Cape Mendocino
2. Chi-Chi Taiwan
3. Loma Prieta
4. Northridge-01
5. Nahanni-Canada
6. Coyote Lake
7. Morgan Hill

Near Fault-With Pulse (NF-WP)

8. Chi-Chi Taiwan
9. Northridge-01
10. Loma Prieta
11. Tabas İran
12. Nahanni Canada
13. Gazli-USSR
14. Baja California

Near Fault Without Pulse (NF-WOP)

15. Chi-Chi Taiwan
16. Coalinga-01
17. Loma Prieta
18. Hector Mine
19. Landers
20. Northridge-01
21. Kocaeli Turkey

Far Field (FF)

5.1. Near Fault Records with Velocity Pulses

The information table for the category of '*Near Fault with Pulse*' is given below:

Table 5.3 Information about earthquake sets

'YEAR'	'EVENT'	'MAG'	'NGAnums'	'Comp.'	**'SCALE'**
[1992]	' Cape Mendocino '	[7.0100]	[825]	'FP'	**[1.3191]**
[1999]	' Chi-Chi- Taiwan '	[7.6200]	[1515]	'FN'	**[1.7500]**
[1989]	' Loma Prieta '	[6.9300]	[779]	'FP'	**[1.3191]**
[1994]	' Northridge-01 '	[6.6900]	[1085]	'FN'	**[1.1639]**
[1985]	' Nahanni- Canada '	[6.7600]	[496]	'FP'	**[1.7500]**
[1979]	' Coyote Lake '	[5.7400]	[150]	'FN'	**[1.7500]**
[1984]	' Morgan Hill	[6.1900]	[451]	'FP'	**[1.1639]**

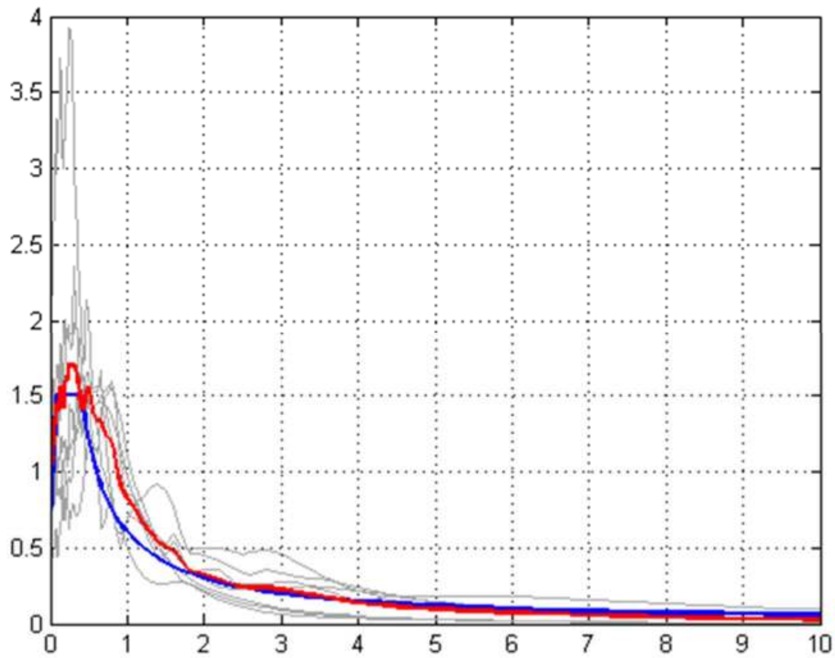


Figure 5.1 Near fault with pulse earthquake records

5.2. Near Fault Records without Velocity Pulses

The information table for the category of '*Near Fault without Pulse'* is given below:

Table 5.4 Information about earthquake sets

'YEAR'	'EVENT'	'MAG'	'NGAnums'	'Comp.'	**'SCALE'**
[1999]	' Chi-Chi- Taiwan '	[7.6200]	[1521]	'FN'	**[1.7500]**
[1994]	' Northridge-01 '	[6.6900]	[1004]	'FN'	**[1.4597]**
[1989]	' Loma Prieta '	[6.9300]	[802]	'FP'	**[1.7500]**
[1978]	' Tabas- Iran '	[7.3500]	[139]	'FP'	**[1.7500]**
[1985]	' Nahanni- Canada '	[6.7600]	[495]	'FN'	**[1.7500]**
[1976]	' Gazli- USSR '	[6.8000]	[126]	'FN'	**[1.4597]**
[1987]	' Baja California '	[5.5000]	[585]	'FN'	**[1.7500]**

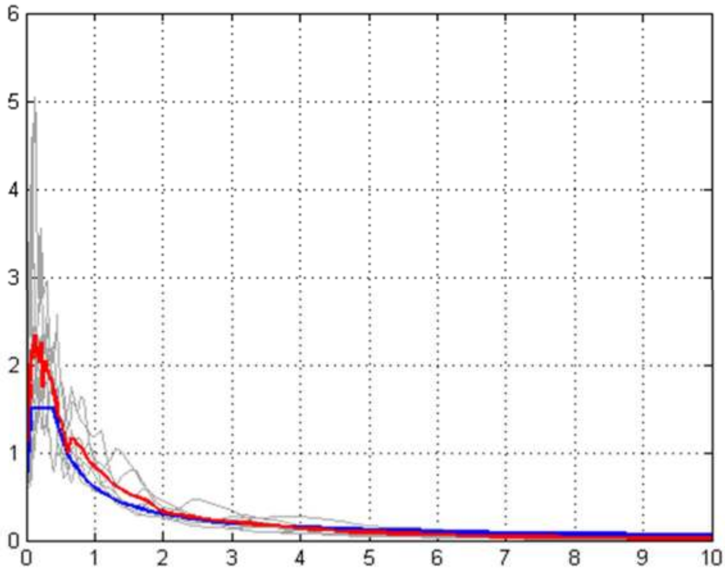


Figure 5.2 Near fault without pulse earthquake records

5.3. Far Field

The information table for the category of '*Far Field' is* given below:

Table 5.5 Information about earthquake sets

'YEAR'	'EVENT'	'MAG'	'NGAnums'	'Comp.'	**'SCALE'**
[1999]	' Chi-Chi- Taiwan	[7.6200]	[1484]	'FN'	**[3.2500]**
[1983]	' Coalinga-01	[6.3600]	[339]	'FN'	**[3.2500]**
[1989]	Loma Prieta	[6.9300]	[776]	'FP'	**[2.1283]**
[1999]	' Hector Mine '	[7.1300]	[1794]	'FP'	**[3.2500]**
[1992]	' Landers	[7.2800]	[838]	'FN'	**[3.2500]**
[1994]	' Northridge-01 '	[6.6900]	[963]	'FN'	**[1.6175]**
[1999]	' Kocaeli- Turkey	[7.5100]	[1163]	'FN'	**[3.2500]**

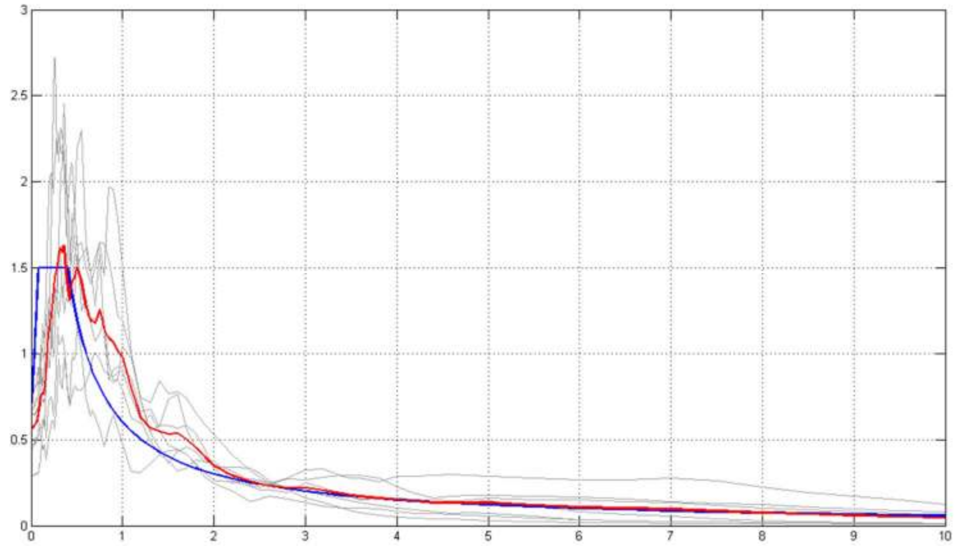


Figure 5.3 Far field eartquake records

CHAPTER 6

STORY DRIFT RATIOS AND SUMMARY OF ANALYSES RESULTS

The design story drift shall be determined as the difference of lateral deflection between the bottom and the top points of each story divided by the story height. Story drifts are determined from the nonlinear analysis. The drift values are taken as absolute values. In the graphic left values are story numbers and in the below drifts are shown as percentage. The distribution of maximum story drift ratios along the frame height, the average values of max drift ratios and the coefficient of variation of the drift ratios per each floor level are provided in Table 6.1, Table 6.2, Table 6.3 and Figure 6.1, Figure 6.2, Figure 6.3 Accordingly, all of the maximum story drift ratios are smaller than 0.04 which indicates that beam-to-column connections are not expected to undergo too much inelastic deformation which would result in a level of strength degradation that cannot be neglected during modeling phase of structural elements.

6.1. Story Drifts for Near Fault with Pulse Records

Table 6.1 Results for near fault with pulse

Story Level	' Cape Mendocino '	' Chi-Chi- Taiwan	' Loma Prieta	' Northridge-01 '	' Nahanni- Canada	' Coyote Lake	' Morgan Hill	Average
10	0,017	0,013	0,011	0,017	0,013	0,013	0,017	0,015
9	0,019	0,020	0,017	0,027	0,017	0,021	0,020	0,020
8	0,028	0,022	0,024	0,031	0,018	0,026	0,022	0,024
7	0,023	0,018	0,025	0,028	0,016	0,020	0,019	0,021
6	0,018	0,021	0,027	0,028	0,015	0,017	0,013	0,020
5	0,021	0,025	0,032	0,031	0,013	0,015	0,013	0,021
4	0,022	0,029	0,033	0,029	0,013	0,013	0,011	0,021
3	0,020	0,028	0,027	0,028	0,013	0,015	0,010	0,020
2	0,017	0,022	0,023	0,027	0,016	0,015	0,013	0,019
1	0,017	0,015	0,016	0,021	0,015	0,014	0,009	0,015

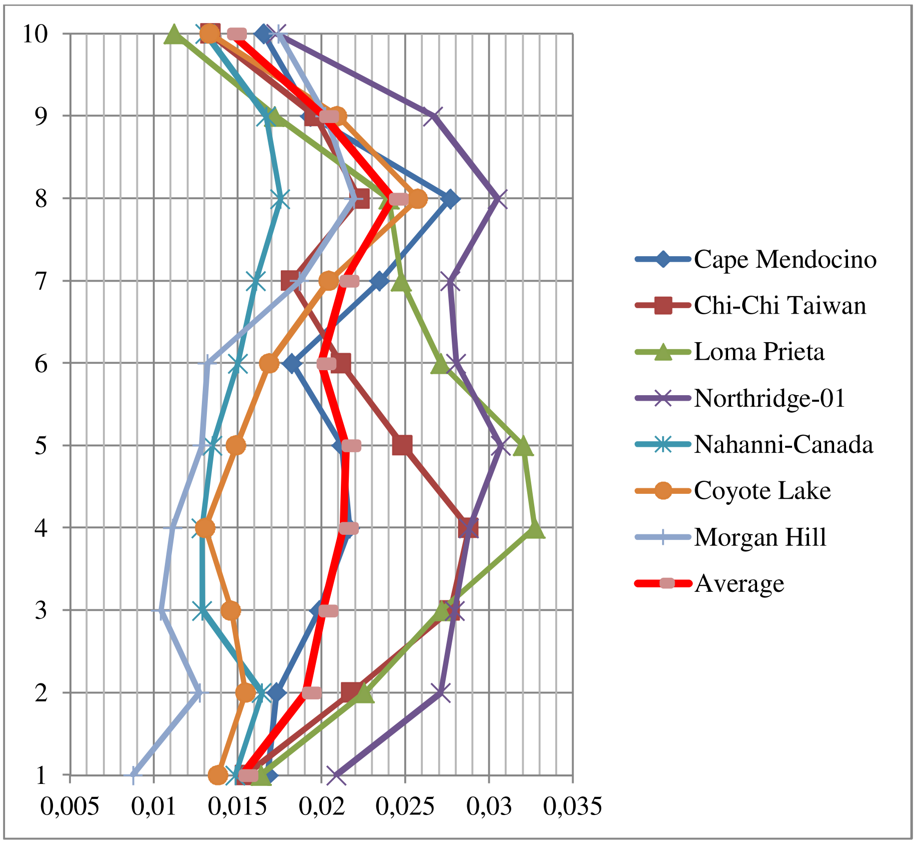


Figure 6.1 Results for near fault with pulse

6.2. Story Drifts for Near Fault without Pulse Records

Table 6.2 Results for near fault without pulse

' Chi-Chi- Taiwa	' Northridge-01 '	' Loma Prieta	' Tabas- Iran	' Nahanni- Canada	' Gazli- USSR	' Baja California	Average
0,017	0,020	0,014	0,012	0,018	0,018	0,016	0,016
0,022	0,024	0,015	0,016	0,022	0,027	0,017	0,020
0,023	0,027	0,016	0,017	0,030	0,029	0,024	0,024
0,021	0,029	0,017	0,018	0,027	0,022	0,020	0,022
0,019	0,021	0,016	0,017	0,022	0,014	0,021	0,018
0,017	0,019	0,016	0,018	0,022	0,017	0,021	0,019
0,021	0,025	0,018	0,015	0,017	0,022	0,019	0,019
0,023	0,026	0,020	0,019	0,020	0,020	0,023	0,022
0,021	0,023	0,021	0,019	0,021	0,021	0,024	0,022
0,018	0,019	0,019	0,016	0,021	0,014	0,024	0,019

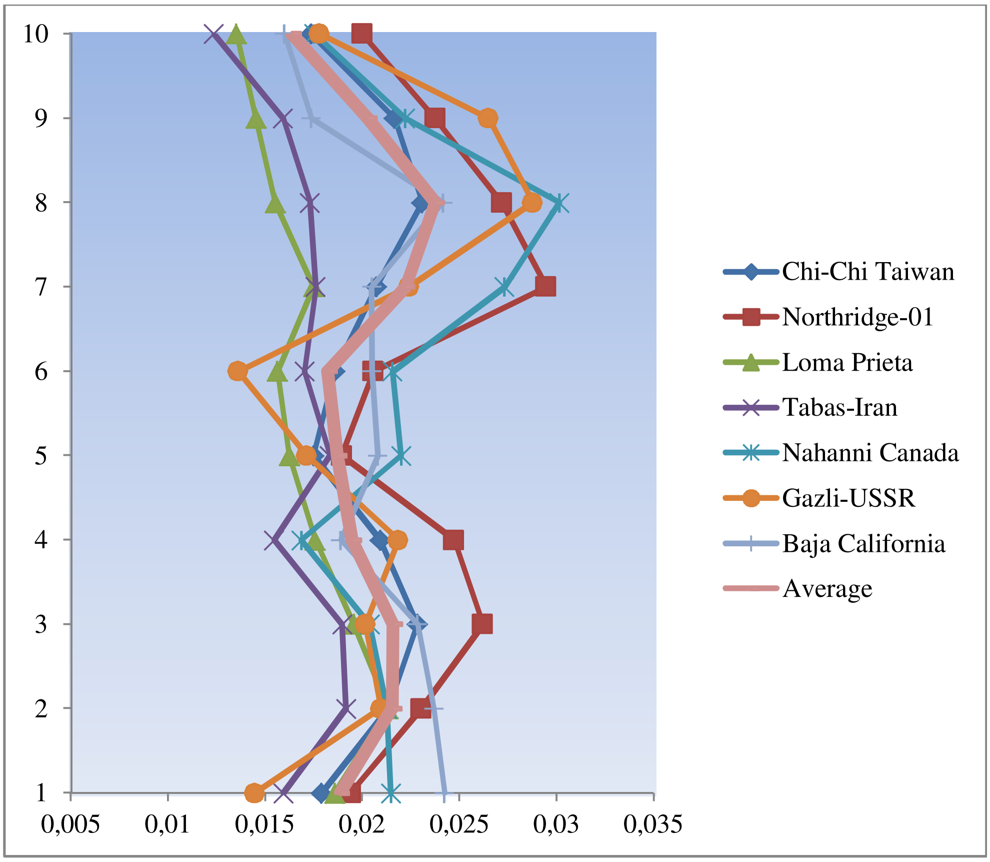


Figure 6.2 Results for near fault without pulse

6.3. Story Drifts for Far Field Records

Table 6.3 Results for far field

' Chi-Chi- Taiwar	' Coalinga-01	Loma Prieta	' Hector Mine '	' Landers	' Northridge-01 '	Kocaeli Turkey	Average
0,015	0,014	0,020	0,014	0,012	0,018	0,011	0,015
0,019	0,018	0,027	0,021	0,017	0,025	0,018	0,021
0,023	0,018	0,030	0,025	0,019	0,029	0,022	0,024
0,020	0,016	0,020	0,019	0,017	0,026	0,020	0,020
0,020	0,016	0,018	0,014	0,017	0,023	0,017	0,018
0,025	0,018	0,019	0,014	0,018	0,021	0,020	0,019
0,031	0,022	0,024	0,014	0,020	0,019	0,022	0,022
0,033	0,023	0,025	0,016	0,019	0,021	0,023	0,023
0,031	0,020	0,022	0,017	0,019	0,022	0,020	0,022
0,025	0,019	0,016	0,014	0,016	0,023	0,014	0,018

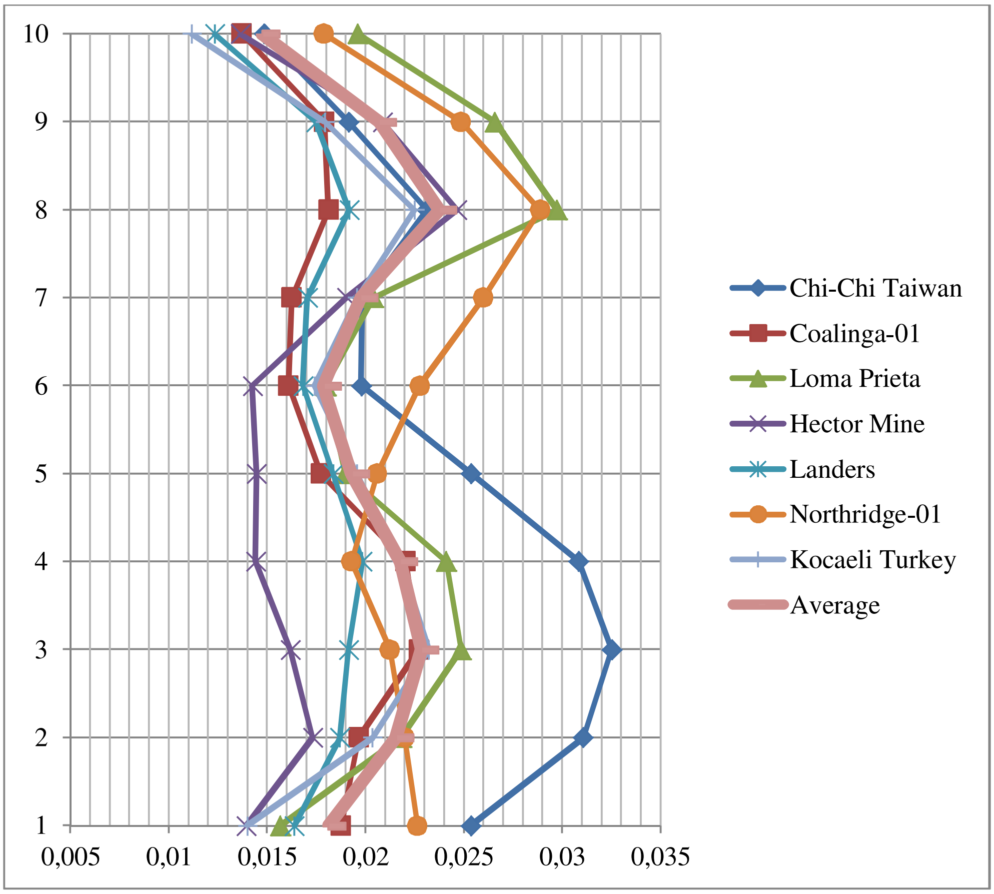


Figure 6.3 Results for far field

For NF-WP set the maximum average story drift is obtained at the 8th story with a value of 0.0242, also the peak value is obtained at the 4th story for the 779-FP record with a value of 0.0327. For NF-WOP set the maximum average story drift is obtained at the 8th story with a value of 0.0237 and the peak value is obtained at the 4th story for the 495-FN record with a value of 0.0301. For Far-Field set the maximum average story drift is obtained at the 8th story with a value of 0.0237, also the peak value is obtained at the 4th story for the 1484-FN record with a value of 0.0325. The average maximum drift ratio is observed at 8th story for the whole sets. The average drift ratios are very close for the top 3 stories. Actually, they are generally close to each other, the difference between the drift ratios of 3 sets for any floor level is smaller than 0.005. The average drift ratios of NF-WP records are the lowest for the bottom 3 stories and the coefficient of variation ratios are the considerably higher for the 6 stories from the base which indicates a high amount of dispersion. NF-WOP records have considerably lower coefficient of variation values for the 6 stories from the base. The coefficients of variation of all record sets are between 0.015 and 0.025 for top 4 stories.

6.4. Summary of Analyses Results

Results of the nonlinear analyses to be used for the seismic performance evaltuation procedure is provided via the following tables.

Table 6.4 Results for average element end rotations

SET1	A-B		B-C		C-D		D-E		E-F	
Story	i	j	i	j	i	j	i	j	i	j
10	0,011	0,009	0,009	0,009	0,009	0,009	0,009	0,009	0,009	0,010
9	0,016	0,015	0,015	0,016	0,015	0,016	0,015	0,016	0,014	0,017
8	0,020	0,023	0,019	0,024	0,019	0,024	0,019	0,024	0,019	0,026
7	0,019	0,024	0,020	0,025	0,020	0,025	0,020	0,025	0,020	0,026
6	0,015	0,016	0,014	0,016	0,013	0,016	0,013	0,017	0,013	0,020
5	0,016	0,017	0,015	0,018	0,015	0,018	0,015	0,018	0,015	0,020
4	0,017	0,019	0,016	0,019	0,016	0,019	0,016	0,019	0,016	0,021
3	0,018	0,018	0,016	0,018	0,016	0,018	0,016	0,018	0,016	0,020
2	0,017	0,016	0,015	0,016	0,015	0,016	0,015	0,016	0,015	0,018
1	0,018	0,014	0,015	0,014	0,014	0,014	0,015	0,014	0,015	0,017

SET 1	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,0019	0,0023	0,0029	0,0040	0,0029	0,0041	0,0029	0,0040	0,0029	0,0041	0,0017	0,0025
9	0,0025	0,0033	0,0037	0,0047	0,0037	0,0046	0,0037	0,0046	0,0037	0,0046	0,0025	0,0034
8	0,0029	0,0029	0,0041	0,0042	0,0042	0,0042	0,0042	0,0042	0,0041	0,0042	0,0029	0,0029
7	0,0039	0,0019	0,0059	0,0031	0,0058	0,0030	0,0059	0,0030	0,0060	0,0030	0,0040	0,0019
6	0,0029	0,0027	0,0046	0,0045	0,0046	0,0045	0,0046	0,0045	0,0046	0,0045	0,0028	0,0027
5	0,0024	0,0028	0,0043	0,0044	0,0044	0,0045	0,0043	0,0045	0,0043	0,0044	0,0028	0,0028
4	0,0029	0,0028	0,0045	0,0044	0,0044	0,0043	0,0045	0,0044	0,0044	0,0043	0,0030	0,0027
3	0,0029	0,0030	0,0045	0,0045	0,0043	0,0044	0,0044	0,0045	0,0045	0,0044	0,0028	0,0029
2	0,0030	0,0033	0,0051	0,0053	0,0051	0,0053	0,0051	0,0053	0,0051	0,0053	0,0030	0,0032
1	0,0162	0,0006	0,0161	0,0018	0,0165	0,0019	0,0164	0,0019	0,0163	0,0019	0,0142	0,0006

SET 2	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,0024	0,0026	0,0035	0,0046	0,0035	0,0046	0,0035	0,0046	0,0035	0,0046	0,0025	0,0026
9	0,0028	0,0034	0,0039	0,0044	0,0039	0,0045	0,0039	0,0045	0,0039	0,0045	0,0028	0,0032
8	0,0030	0,0034	0,0042	0,0046	0,0042	0,0046	0,0042	0,0046	0,0041	0,0046	0,0030	0,0033
7	0,0041	0,0021	0,0062	0,0032	0,0062	0,0032	0,0062	0,0032	0,0061	0,0072	0,0041	0,0021
6	0,0029	0,0031	0,0047	0,0048	0,0047	0,0048	0,0047	0,0048	0,0047	0,0048	0,0029	0,0031
5	0,0032	0,0033	0,0050	0,0050	0,0051	0,0050	0,0051	0,0050	0,0051	0,0050	0,0033	0,0032
4	0,0029	0,0035	0,0046	0,0053	0,0046	0,0053	0,0046	0,0051	0,0046	0,0053	0,0029	0,0035
3	0,0031	0,0033	0,0049	0,0052	0,0049	0,0053	0,0049	0,0052	0,0049	0,0052	0,0031	0,0033
2	0,0036	0,0031	0,0061	0,0052	0,0061	0,0052	0,0061	0,0052	0,0062	0,0053	0,0036	0,0031
1	0,0467	0,0014	0,0165	0,0020	0,0194	0,0019	0,0193	0,0043	0,0194	0,0019	0,0189	0,0022

SET 2	A-B		B-C		C-D		D-E		E-F	
Story	i	j	i	j	i	j	i	j	i	j
10	0,012	0,009	0,010	0,009	0,010	0,009	0,010	0,009	0,010	0,010
9	0,017	0,014	0,017	0,015	0,017	0,015	0,017	0,015	0,015	0,015
8	0,023	0,018	0,021	0,019	0,021	0,019	0,021	0,019	0,021	0,020
7	0,023	0,022	0,023	0,023	0,023	0,023	0,023	0,023	0,022	0,024
6	0,016	0,014	0,014	0,014	0,014	0,011	0,014	0,014	0,014	0,016
5	0,017	0,014	0,015	0,014	0,015	0,014	0,015	0,014	0,014	0,016
4	0,018	0,013	0,015	0,014	0,016	0,014	0,015	0,014	0,015	0,016
3	0,021	0,015	0,018	0,015	0,018	0,015	0,018	0,015	0,018	0,017
2	0,022	0,016	0,020	0,016	0,020	0,016	0,020	0,016	0,019	0,019
1	0,021	0,015	0,017	0,014	0,017	0,014	0,017	0,014	0,017	0,018

Table 6.4 (cont'd)

SET 3	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,0015	0,0024	0,0025	0,0040	0,0024	0,0040	0,0024	0,0040	0,0023	0,0040	0,0015	0,0024
9	0,0023	0,0029	0,0034	0,0044	0,0035	0,0044	0,0034	0,0044	0,0034	0,0044	0,0023	0,0031
8	0,0027	0,0029	0,0039	0,0041	0,0039	0,0042	0,0039	0,0041	0,0039	0,0041	0,0026	0,0028
7	0,0039	0,0019	0,0058	0,0030	0,0058	0,0030	0,0058	0,0030	0,0058	0,0030	0,0040	0,0018
6	0,0028	0,0026	0,0044	0,0044	0,0043	0,0045	0,0043	0,0045	0,0044	0,0045	0,0027	0,0026
5	0,0025	0,0025	0,0041	0,0046	0,0041	0,0047	0,0042	0,0047	0,0041	0,0047	0,0024	0,0029
4	0,0026	0,0030	0,0043	0,0048	0,0043	0,0048	0,0043	0,0048	0,0044	0,0048	0,0026	0,0030
3	0,0027	0,0030	0,0046	0,0049	0,0047	0,0048	0,0047	0,0049	0,0047	0,0049	0,0027	0,0030
2	0,0030	0,0028	0,0055	0,0050	0,0054	0,0051	0,0054	0,0050	0,0055	0,0050	0,0030	0,0029
1	0,0178	0,0005	0,0180	0,0017	0,0180	0,0017	0,0181	0,0017	0,0180	0,0017	0,0175	0,0013

SET 3	A-B		B-C		C-D		D-E		E-F	
Story	i	j	i	j	i	j	i	j	i	j
10	0,0097	0,0088	0,0086	0,0087	0,0086	0,0086	0,0086	0,0086	0,0086	0,0097
9	0,0165	0,0165	0,0158	0,0173	0,0157	0,0172	0,0157	0,0172	0,0151	0,0176
8	0,0221	0,0217	0,0208	0,0226	0,0208	0,0225	0,0208	0,0227	0,0200	0,0229
7	0,0223	0,0219	0,0213	0,0229	0,0213	0,0229	0,0213	0,0230	0,0203	0,0230
6	0,0163	0,0126	0,0142	0,0130	0,0141	0,0130	0,0141	0,0130	0,0137	0,0148
5	0,0163	0,0129	0,0146	0,0133	0,0146	0,0133	0,0146	0,0133	0,0140	0,0146
4	0,0191	0,0158	0,0171	0,0162	0,0171	0,0162	0,0171	0,0162	0,0165	0,0179
3	0,0219	0,0177	0,0199	0,0179	0,0199	0,0179	0,0199	0,0179	0,0194	0,0198
2	0,0216	0,0177	0,0192	0,0179	0,0192	0,0179	0,0192	0,0179	0,0188	0,0202
1	0,0148	0,0158	0,0169	0,0157	0,0169	0,0157	0,0169	0,0156	0,0169	0,0194

Table 6.5 Average axial force

SET 1	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	55,59	55,61	36,99	36,99	33,17	33,17	33,18	33,18	36,64	36,64	57,10	57,29
9	127,40	127,40	79,51	79,52	72,73	72,73	73,00	73,00	79,04	79,04	130,99	130,93
8	198,99	199,07	121,36	121,36	112,33	112,33	112,47	112,47	120,80	120,80	200,44	200,83
7	269,24	269,36	162,61	162,67	152,27	152,27	152,23	152,23	161,11	161,11	271,47	270,99
6	346,03	302,90	201,21	201,31	191,67	191,67	192,46	192,46	202,21	202,21	367,54	367,39
5	422,17	422,14	239,94	239,83	231,90	231,90	232,04	232,04	242,49	242,49	452,10	452,74
4	496,27	496,19	280,01	279,71	271,61	271,54	271,80	271,80	283,46	283,46	534,91	535,11
3	562,24	561,33	323,01	323,20	311,13	311,22	311,56	311,56	325,64	325,97	608,47	608,74
2	621,29	622,89	371,23	370,89	350,81	350,81	350,64	350,64	372,34	372,24	670,51	671,61
1	677,41	676,49	425,44	425,47	391,09	391,36	390,53	390,53	422,79	423,04	741,63	742,50

SET 2	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	60,84	60,81	37,97	37,96	33,30	33,30	33,24	33,24	36,64	36,57	57,12	57,13
9	132,74	133,19	79,95	80,27	73,14	73,14	73,16	73,16	79,60	79,48	128,63	128,57
8	197,93	140,54	122,13	121,99	112,84	112,84	113,03	113,03	121,26	121,14	193,94	193,57
7	255,23	265,53	162,59	163,06	152,87	152,87	152,80	152,80	161,26	161,13	257,71	257,56
6	337,09	352,34	203,83	203,71	192,46	192,46	192,57	192,57	202,20	202,17	351,11	351,59
5	438,74	438,89	243,30	243,07	232,13	232,13	232,47	232,47	244,01	243,66	440,07	440,07
4	525,54	526,11	283,84	285,33	271,80	271,80	272,30	272,30	284,21	284,09	526,90	525,29
3	610,81	612,23	329,51	329,74	311,40	311,40	311,63	311,63	328,71	328,56	606,77	608,60
2	699,27	701,07	380,07	380,17	350,90	350,90	351,67	351,67	376,66	376,86	682,73	684,70
1	789,60	788,34	432,43	432,54	391,41	391,41	391,66	391,66	426,66	426,77	748,23	751,89

SET 3	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	55,22	55,31	36,54	36,51	33,16	33,14	33,19	33,19	36,67	36,71	56,53	56,50
9	128,31	128,37	78,38	78,35	73,09	73,07	72,98	72,98	78,96	79,23	129,34	129,43
8	202,44	202,31	120,66	120,73	112,96	112,96	112,96	112,96	121,13	121,43	203,31	203,17
7	270,64	271,14	162,46	162,50	152,64	152,64	152,81	152,81	162,93	163,20	272,87	273,06
6	356,50	356,99	203,73	203,89	192,47	192,57	192,43	192,43	202,76	202,93	359,01	360,10
5	443,70	443,79	244,73	244,43	232,36	232,36	232,41	232,41	241,40	241,04	442,29	441,64
4	532,36	533,26	285,46	285,81	271,89	271,89	272,30	272,30	281,00	280,64	526,64	525,24
3	623,89	623,83	327,81	328,06	311,49	311,63	312,09	312,09	326,94	327,01	622,10	621,20
2	713,74	714,01	378,96	378,79	351,93	351,93	351,76	351,76	376,21	376,66	720,44	721,37
1	803,34	802,69	431,30	431,39	391,51	391,51	391,04	391,04	429,64	430,14	812,26	811,17

Maximum base shears obtained by nonlinear time history analyses are also provided in Table 6.6.

Table 6.6 Frame base cuts

SET1

Earthquakes	Maximum(kip)
' Cape Mendocino '	1504,4
' Chi-Chi- Taiwan '	1278,9
' Loma Prieta '	1208,4
' Northridge-01 '	1237,6
' Nahanni- Canada '	1209,8
' Coyote Lake '	1201,2
' Morgan Hill	1262,7

SET2

Earthquakes	Maximum(kip)
' Chi-Chi- Taiwan '	1231,2
' Northridge-01 '	1400,9
' Loma Prieta '	1377,
' Tabas- Iran '	1186,9
' Nahanni- Canada(NOP)	1280,3
' Gazli- USSR '	1264,4
' Baja California '	1317,6

SET3

Earthquakes	Maximum(kip)
' Chi-Chi- Taiwan	1273,1
' Coalinga-01	1300,9
Loma Prieta	1226,
' Hector Mine '	1141,7
' Landers	1179,2
' Northridge-01 '	1237,6
' Kocaeli- Turkey	1093,2

CHAPTER 7

LIMIT STATES AND SEISMIC PERFORMANCE EVALUATION

Nonlinear Dynamic Procedure per ASCE/SEI: 41-06 is applied according to which, calculated beam and column deformations should be lower than the inelastic deformation capacity. For each set, the average value of the maximum inelastic deformations of 7 records are retained and compared with the limit values for both ends of the whole elements. All of the elements (except for one column) satisfy the life safety performance level for all of the record sets. In order to give an idea the performance levels coinciding to the minimum displacement limits are provided for all of the elements. Satisfied performance levels in this manner and the demand/capacity ratio for the satisfied level are given in Table 7.5 and 7.13 for the beams and columns. For all of the sets most of the columns satisfy the Immediate Occupancy (IO) performance level. Only the bottom ends of the first story columns satisfy Life Safety (LS) performance level. Actually, for NF-WOP set the bottom end of the left column of the first story satisfies Collapse Prevention (CP) performance level. Moreover, for the NF-WOP set bottom ends of four columns of the second story and top end of one of the columns of the seventh story satisfy LS performance level. For all of the beams, both ends of most of the beams satisfy the LS performance level, whereas only the beams of the top stories satisfy IO level except for the left end of the left beam for NF-WOP set. Actually, considering the beams under the effect of NF-WP set, 82% of element ends' demand/capacity ratio percentages are below 15%. For the ones of NF-WOP set, 86% of element ends' demand/capacity ratio percentages are below 15%. It seems the distinctive characteristics of ground motion records do not affect the performance assessment procedures much, as long as the spectrum matching is applied.

7.1. Beams

The beams shall not exceed the limit states when it is subjected to appropriate load combinations.

7.2. Yield Rotations for Beams

The yield rotation (θ_y) was used to determine the limit.

Table 7.1 The Result table for beam

		Beams	Columns	Beams/Colmuns						Beams			
Column	Beam	Z_x	Z_x	F_{ye}	L_c	L_b	I_c	I_b	E	Θ_y	A_g	P_{ye}	P_{cl}
W14x176	W18x71	146	320	50	156	300	2140	1170	29000	0,011	51,8	2590	2102,54
	W18x71	146	320	50	156	300	2140	1170	29000	0,011	51,8	2590	1935,62
W14x176	W18x71	146	320	50	156	300	2140	1170	29000	0,011	51,8	2590	1935,62
	W18x71	146	320	50	156	300	2140	1170	29000	0,011	51,8	2590	2020,83
W14x193	W18x106	230	355	50	156	300	2400	1910	29000	0,010	56,8	2840	2315,81
	W18x106	230	355	50	156	300	2400	1910	29000	0,010	56,8	2840	2315,81
W14x193	W18x106	230	355	50	156	300	2400	1910	29000	0,010	56,8	2840	2315,81
	W18x106	230	355	50	156	300	2400	1910	29000	0,010	56,8	2840	2315,81
W14x193	W18x106	230	355	50	156	300	2400	1910	29000	0,010	56,8	2840	2315,81
	W18x106	230	355	50	156	300	2400	1910	29000	0,010	56,8	2840	2514,23

$$\theta_y = \frac{ZF_{ye}l_b}{6EI_b} \quad \text{(Eq.5.1 Asce7-10)} \qquad (7.1)$$

$$\theta_y = \frac{50x300x230}{6x29000x1910} = 0,010381$$

$$P_{ye} = A_g F_{ye} = 50x56.8 = 2840kip \qquad (7.2)$$

The action type of beams shall be considered as deformation-controlled. Limit states shall be computed as:

a) $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$

b) $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_{ye}}}$

c) Other

Table 7.2 Category results for beam sections

Beam	$b_f/2t_f$	h/t_w	F_{ye}	$52/\sqrt{F_{ye}}$	$418/\sqrt{F_{ye}}$	$65/\sqrt{F_{ye}}$	$640/\sqrt{F_{ye}}$	*Category*
W18x71	4,71	32,4	50	7,35	59,11	9,19	90,51	a
W18x71	4,71	32,4	50	7,35	59,11	9,19	90,51	a
W18x71	4,71	32,4	50	7,35	59,11	9,19	90,51	a
W18x71	4,71	32,4	50	7,35	59,11	9,19	90,51	a
W18x106	5,96	27,2	50	7,35	59,11	9,19	90,51	a
W18x106	5,96	27,2	50	7,35	59,11	9,19	90,51	a
W18x106	5,96	27,2	50	7,35	59,11	9,19	90,51	a
W18x106	5,96	27,2	50	7,35	59,11	9,19	90,51	a
W18x106	5,96	27,2	50	7,35	59,11	9,19	90,51	a
W18x106	5,96	27,2	50	7,35	59,11	9,19	90,51	a

7.3. Beam Limit States

Acceptance Criteria[14]				
Plastic Rotation Angle, Radians				
	Primary		Secondary	
IO	LS	CP	LS	CP
$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$

Figure 7.1 The acceptance criteria for limit states

Limit states for beam are given below:

Table 7.3 The limits for beam sections

	IO	LS	CP
Beam	Θ_y	$6\Theta_y$	$8\Theta_y$
W18x71	0,011	0,065	0,086
W18x71	0,011	0,065	0,086
W18x71	0,011	0,065	0,086
W18x71	0,011	0,065	0,086
W18x106	0,010	0,062	0,083
W18x106	0,010	0,062	0,083
W18x106	0,010	0,062	0,083
W18x106	0,010	0,062	0,083
W18x106	0,010	0,062	0,083
W18x106	0,010	0,062	0,083

Limit states for sets are given below;

Table 7.4 Seismic performance levels for beam sections

SET1	A-B		B-C		C-D		D-E		E-F	
Story	i	j	i	j	i	j	i	j	i	j
10	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
9	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
8	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
7	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
6	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
5	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
4	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
3	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
2	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
1	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS

SET2	A-B		B-C		C-D		D-E		E-F	
Story	i	j	i	j	i	j	i	j	i	j
10	LS	IO	IO	IO	IO	IO	IO	IO	IO	IO
9	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
8	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
7	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
6	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
5	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
4	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
3	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
2	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
1	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS

SET3	A-B		B-C		C-D		D-E		E-F	
Story	i	j	i	j	i	j	i	j	i	j
10	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
9	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
8	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
7	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
6	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
5	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
4	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
3	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
2	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS
1	LS	LS	LS	LS	LS	LS	LS	LS	LS	LS

Table 7.5 Demand/Capacity ratios are given below.

SET1	A-B		B-C		C-D		D-E		E-F	
Story	i	j	i	j	i	j	i	j	i	j
10	0,99	0,81	0,80	0,81	0,80	0,81	0,80	0,81	0,80	0,92
9	0,25	0,24	0,23	0,25	0,23	0,25	0,23	0,25	0,22	0,27
8	0,30	0,36	0,30	0,37	0,30	0,37	0,30	0,37	0,29	0,40
7	0,30	0,38	0,31	0,39	0,31	0,39	0,31	0,39	0,30	0,40
6	0,24	0,25	0,22	0,26	0,21	0,26	0,21	0,27	0,21	0,33
5	0,26	0,28	0,24	0,29	0,24	0,29	0,24	0,29	0,24	0,32
4	0,28	0,30	0,26	0,31	0,26	0,31	0,26	0,31	0,26	0,33
3	0,28	0,29	0,26	0,29	0,26	0,29	0,26	0,29	0,26	0,32
2	0,27	0,26	0,24	0,26	0,24	0,26	0,24	0,26	0,24	0,29
1	0,29	0,23	0,23	0,22	0,23	0,22	0,23	0,22	0,24	0,27

Table 7.5 (cont'd)

SET2	A-B		B-C		C-D		D-E		E-F	
Story	i	j	i	j	i	j	i	j	i	j
10	0,18	0,82	0,94	0,82	0,94	0,82	0,94	0,81	0,94	0,93
9	0,27	0,21	0,26	0,23	0,26	0,23	0,26	0,23	0,23	0,24
8	0,35	0,29	0,33	0,29	0,33	0,29	0,33	0,29	0,32	0,31
7	0,36	0,35	0,35	0,36	0,35	0,36	0,35	0,36	0,34	0,37
6	0,26	0,22	0,23	0,23	0,23	0,17	0,23	0,23	0,22	0,26
5	0,27	0,22	0,24	0,23	0,24	0,23	0,24	0,23	0,23	0,26
4	0,28	0,22	0,25	0,22	0,25	0,22	0,25	0,22	0,24	0,25
3	0,33	0,23	0,30	0,24	0,30	0,24	0,30	0,24	0,29	0,27
2	0,36	0,26	0,32	0,26	0,32	0,26	0,32	0,26	0,31	0,30
1	0,34	0,23	0,28	0,23	0,28	0,23	0,28	0,23	0,28	0,29

SET3	A-B		B-C		C-D		D-E		E-F	
Story	i	j	i	j	i	j	i	j	i	j
10	0,90	0,82	0,80	0,80	0,80	0,80	0,80	0,80	0,80	0,91
9	0,26	0,26	0,24	0,27	0,24	0,27	0,24	0,27	0,23	0,27
8	0,34	0,34	0,32	0,35	0,32	0,35	0,32	0,35	0,31	0,36
7	0,35	0,34	0,33	0,35	0,33	0,35	0,33	0,36	0,31	0,36
6	0,26	0,20	0,23	0,21	0,23	0,21	0,23	0,21	0,22	0,24
5	0,26	0,21	0,23	0,21	0,23	0,21	0,23	0,21	0,23	0,23
4	0,31	0,25	0,28	0,26	0,28	0,26	0,28	0,26	0,27	0,29
3	0,35	0,28	0,32	0,29	0,32	0,29	0,32	0,29	0,31	0,32
2	0,35	0,28	0,31	0,29	0,31	0,29	0,31	0,29	0,30	0,32
1	0,24	0,25	0,27	0,25	0,27	0,25	0,27	0,25	0,27	0,31

7.3.1. Results

Performance levels of Set 1 and 3 are almost symmetrical. Maximum IO demand/capacity ratio is at Set 1 is at Span E-F at 10th story (at j). Maximum LS demand/capacity ratio is at Span E-F at 7th story (at j). Maximum IO demand/capacity ratio is at Set 2 is at Span D-E at 10th story (at İ). Maximum LS demand/capacity ratio is at Span E-F at 7th story (at j). Maximum IO demand/capacity ratio is at Set 3 is at Span E-F at 10th story (at j). Maximum LS demand/capacity ratio is at Span E-F at 7th story (at j).

7.4. Columns

The columns shall not exceed the limit states when it is subjected to appropriate load combinations.

7.4.1. Yield Rotations for Columns

The yield rotation (θ_y) was used to determine the limit. It is determined as:

$$\theta_y = \frac{ZF_{ye}I_c}{6EI_c}\left(1 - \frac{P}{P_{ye}}\right) \tag{7.3}$$

Table 7.6 Yield rotations for columns

Set1	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066
9	0,0064	0,0064	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0064	0,0064
8	0,0062	0,0062	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0062	0,0062
7	0,0060	0,0060	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0060	0,0060
6	0,0058	0,0059	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0058	0,0058
5	0,0056	0,0056	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0056	0,0056
4	0,0055	0,0055	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0054	0,0054
3	0,0053	0,0053	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0052	0,0052
2	0,0052	0,0052	0,0058	0,0058	0,0058	0,0058	0,0058	0,0058	0,0058	0,0058	0,0051	0,0051
1	0,0050	0,0051	0,0056	0,0056	0,0057	0,0057	0,0057	0,0057	0,0056	0,0056	0,0049	0,0049

Set2	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,0065	0,0065	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066
9	0,0064	0,0064	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0064	0,0064
8	0,0062	0,0063	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0062	0,0062
7	0,0060	0,0060	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0060	0,0060
6	0,0058	0,0058	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0058	0,0058
5	0,0056	0,0056	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0056	0,0056
4	0,0054	0,0054	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0054	0,0054
3	0,0052	0,0052	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0052	0,0052
2	0,0050	0,0050	0,0057	0,0057	0,0058	0,0058	0,0058	0,0058	0,0058	0,0058	0,0050	0,0050
1	0,0048	0,0048	0,0056	0,0056	0,0057	0,0057	0,0057	0,0057	0,0056	0,0056	0,0049	0,0049

Set3	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066
9	0,0064	0,0064	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0064	0,0064
8	0,0062	0,0062	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0062	0,0062
7	0,0060	0,0060	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0060	0,0060
6	0,0058	0,0058	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0058	0,0058
5	0,0056	0,0056	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0056	0,0056
4	0,0054	0,0054	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0054	0,0054
3	0,0052	0,0052	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0052	0,0052
2	0,0050	0,0050	0,0057	0,0057	0,0058	0,0058	0,0058	0,0058	0,0058	0,0058	0,0049	0,0049
1	0,0048	0,0048	0,0056	0,0056	0,0057	0,0057	0,0057	0,0057	0,0056	0,0056	0,0047	0,0047

In order to consider the flexural action of the column is deformation-controlled, the condition of $P > P_{cl} > 0.5$ shall be satisfied.

Table 7.7 $P > P_{cl}$ ratio for columns

Set1	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,03	0,03	0,02	0,02	0,02	0,02	0,02	0,02	0,02	0,02	0,03	0,03
9	0,07	0,07	0,04	0,04	0,04	0,04	0,04	0,04	0,04	0,04	0,07	0,07
8	0,10	0,10	0,06	0,06	0,06	0,06	0,06	0,06	0,06	0,06	0,10	0,10
7	0,13	0,13	0,08	0,08	0,08	0,08	0,08	0,08	0,08	0,08	0,13	0,13
6	0,15	0,13	0,09	0,09	0,08	0,08	0,08	0,08	0,09	0,09	0,16	0,16
5	0,18	0,18	0,10	0,10	0,10	0,10	0,10	0,10	0,10	0,10	0,20	0,20
4	0,21	0,21	0,12	0,12	0,12	0,12	0,12	0,12	0,12	0,12	0,23	0,23
3	0,24	0,24	0,14	0,14	0,13	0,13	0,13	0,13	0,14	0,14	0,26	0,26
2	0,27	0,27	0,16	0,16	0,15	0,15	0,15	0,15	0,16	0,16	0,29	0,29
1	0,27	0,27	0,17	0,17	0,16	0,16	0,16	0,16	0,17	0,17	0,29	0,30

Table 7.7 (cont'd)

Set2	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,03	0,03	0,02	0,02	0,02	0,02	0,02	0,02	0,02	0,02	0,03	0,03
9	0,07	0,07	0,04	0,04	0,04	0,04	0,04	0,04	0,04	0,04	0,07	0,07
8	0,10	0,07	0,06	0,06	0,06	0,06	0,06	0,06	0,06	0,06	0,10	0,10
7	0,13	0,13	0,08	0,08	0,08	0,08	0,08	0,08	0,08	0,08	0,13	0,13
6	0,15	0,15	0,09	0,09	0,08	0,08	0,08	0,08	0,09	0,09	0,15	0,15
5	0,19	0,19	0,11	0,10	0,10	0,10	0,10	0,10	0,11	0,11	0,19	0,19
4	0,23	0,23	0,12	0,12	0,12	0,12	0,12	0,12	0,12	0,12	0,23	0,23
3	0,26	0,26	0,14	0,14	0,13	0,13	0,13	0,13	0,14	0,14	0,26	0,26
2	0,30	0,30	0,16	0,16	0,15	0,15	0,15	0,15	0,16	0,16	0,29	0,30
1	0,31	0,31	0,17	0,17	0,16	0,16	0,16	0,16	0,17	0,17	0,30	0,30

Set3	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,026	0,026	0,017	0,017	0,016	0,016	0,016	0,016	0,017	0,017	0,027	0,027
9	0,07	0,07	0,04	0,04	0,04	0,04	0,04	0,04	0,04	0,04	0,07	0,07
8	0,10	0,10	0,06	0,06	0,06	0,06	0,06	0,06	0,06	0,06	0,11	0,10
7	0,13	0,13	0,08	0,08	0,08	0,08	0,08	0,08	0,08	0,08	0,14	0,14
6	0,15	0,15	0,09	0,09	0,08	0,08	0,08	0,08	0,09	0,09	0,16	0,16
5	0,19	0,19	0,11	0,11	0,10	0,10	0,10	0,10	0,10	0,10	0,19	0,19
4	0,23	0,23	0,12	0,12	0,12	0,12	0,12	0,12	0,12	0,12	0,23	0,23
3	0,27	0,27	0,14	0,14	0,13	0,13	0,13	0,13	0,14	0,14	0,27	0,27
2	0,31	0,31	0,16	0,16	0,15	0,15	0,15	0,15	0,16	0,16	0,31	0,31
1	0,32	0,32	0,17	0,17	0,16	0,16	0,16	0,16	0,17	0,17	0,32	0,32

7.4.2. Columns Flexure

The action type of beams shall be considered as deformation-controlled. Limit states shall be computed as:

d) $$\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}} \text{ and } \frac{h}{t_w} \leq \frac{300}{\sqrt{F_{ye}}} \tag{7.4}$$

e) $$\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}} \text{ and } \frac{h}{t_w} \geq \frac{460}{\sqrt{F_{ye}}} \tag{7.5}$$

f) Other

Table 7.8 Categories for column section

Column	$b_f/2t_f$	h/t_w	F_{ye}	$52/\sqrt{F_{ye}}$	$300/\sqrt{F_{ye}}$	$65/\sqrt{F_{ye}}$	$460/\sqrt{F_{ye}}$	Category
W14x176	5,97	12,8	50	7,35	42,43	9,19	65,05	a
W14x176	5,97	12,8	50	7,35	42,43	9,19	65,05	a
W14x176	5,97	12,8	50	7,35	42,43	9,19	65,05	a
W14x176	5,97	12,8	50	7,35	42,43	9,19	65,05	a
W14x193	5,45	13,7	50	7,35	42,43	9,19	65,05	a
W14x193	5,45	13,7	50	7,35	42,43	9,19	65,05	a
W14x193	5,45	13,7	50	7,35	42,43	9,19	65,05	a
W14x193	5,45	13,7	50	7,35	42,43	9,19	65,05	a
W14x193	5,45	13,7	50	7,35	42,43	9,19	65,05	a
W14x193	5,45	13,7	50	7,35	42,43	9,19	65,05	a

Acceptance Criteria[14]				
Plastic Rotation Angle, Radians				
	Primary		Secondary	
IO	LS	CP	LS	CP
$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$

Figure 7.2 The acceptance criteria for Limit States

- *Immediate Occupancy*

Table 7.9 Limit states for columns for immediate occupancy

Set1	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066
9	0,0064	0,0064	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0064	0,0064
8	0,0062	0,0062	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0062	0,0062
7	0,0060	0,0060	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0060	0,0060
6	0,0058	0,0059	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0058	0,0058
5	0,0056	0,0056	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0056	0,0056
4	0,0055	0,0055	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0054	0,0054
3	0,0053	0,0053	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0052	0,0052
2	0,0052	0,0052	0,0058	0,0058	0,0058	0,0058	0,0058	0,0058	0,0058	0,0058	0,0051	0,0051
1	0,0050	0,0051	0,0056	0,0056	0,0057	0,0057	0,0057	0,0057	0,0056	0,0056	0,0049	0,0049

Set2	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,0065	0,0065	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066
9	0,0064	0,0064	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0064	0,0064
8	0,0062	0,0063	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0062	0,0062
7	0,0060	0,0060	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0060	0,0060
6	0,0058	0,0058	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0058	0,0058
5	0,0056	0,0056	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0056	0,0056
4	0,0054	0,0054	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0054	0,0054
3	0,0052	0,0052	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0052	0,0052
2	0,0050	0,0050	0,0057	0,0057	0,0058	0,0058	0,0058	0,0058	0,0058	0,0058	0,0050	0,0050
1	0,0048	0,0048	0,0056	0,0056	0,0057	0,0057	0,0057	0,0057	0,0056	0,0056	0,0049	0,0049

Set3	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066	0,0066
9	0,0064	0,0064	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0065	0,0064	0,0064
8	0,0062	0,0062	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0064	0,0062	0,0062
7	0,0060	0,0060	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0063	0,0060	0,0060
6	0,0058	0,0058	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0062	0,0058	0,0058
5	0,0056	0,0056	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0061	0,0056	0,0056
4	0,0054	0,0054	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0060	0,0054	0,0054
3	0,0052	0,0052	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0059	0,0052	0,0052
2	0,0050	0,0050	0,0057	0,0057	0,0058	0,0058	0,0058	0,0058	0,0058	0,0058	0,0049	0,0049
1	0,0048	0,0048	0,0056	0,0056	0,0057	0,0057	0,0057	0,0057	0,0056	0,0056	0,0047	0,0047

- *Life Safety*

Table 7.10 Limit states for columns for life safety

Set1	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,039	0,039	0,040	0,040	0,040	0,040	0,040	0,040	0,040	0,040	0,039	0,039
9	0,038	0,038	0,039	0,039	0,039	0,039	0,039	0,039	0,039	0,039	0,038	0,038
8	0,037	0,037	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,037	0,037
7	0,036	0,036	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,036	0,036
6	0,035	0,036	0,037	0,037	0,037	0,037	0,037	0,037	0,037	0,037	0,035	0,035
5	0,034	0,034	0,036	0,036	0,037	0,037	0,037	0,037	0,036	0,036	0,033	0,033
4	0,033	0,033	0,036	0,036	0,036	0,036	0,036	0,036	0,036	0,036	0,032	0,032
3	0,032	0,032	0,035	0,035	0,035	0,035	0,035	0,035	0,035	0,035	0,031	0,031
2	0,031	0,031	0,035	0,035	0,035	0,035	0,035	0,035	0,035	0,035	0,030	0,030
1	0,030	0,030	0,034	0,034	0,034	0,034	0,034	0,034	0,034	0,034	0,029	0,029

Set2	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,039	0,039	0,040	0,040	0,040	0,040	0,040	0,040	0,040	0,040	0,039	0,039
9	0,038	0,038	0,039	0,039	0,039	0,039	0,039	0,039	0,039	0,039	0,038	0,038
8	0,037	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,037	0,037
7	0,036	0,036	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,036	0,036
6	0,035	0,035	0,037	0,037	0,037	0,037	0,037	0,037	0,037	0,037	0,035	0,035
5	0,034	0,034	0,036	0,036	0,037	0,037	0,037	0,037	0,036	0,036	0,034	0,034
4	0,032	0,032	0,036	0,036	0,036	0,036	0,036	0,036	0,036	0,036	0,032	0,032
3	0,031	0,031	0,035	0,035	0,035	0,035	0,035	0,035	0,035	0,035	0,031	0,031
2	0,030	0,030	0,034	0,034	0,035	0,035	0,035	0,035	0,035	0,035	0,030	0,030
1	0,029	0,029	0,034	0,034	0,034	0,034	0,034	0,034	0,034	0,034	0,029	0,029

Set3	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,039	0,039	0,040	0,040	0,040	0,040	0,040	0,040	0,040	0,040	0,039	0,039
9	0,038	0,038	0,039	0,039	0,039	0,039	0,039	0,039	0,039	0,039	0,038	0,038
8	0,037	0,037	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,037	0,037
7	0,036	0,036	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,038	0,036	0,036
6	0,035	0,035	0,037	0,037	0,037	0,037	0,037	0,037	0,037	0,037	0,035	0,035
5	0,034	0,034	0,036	0,036	0,037	0,037	0,037	0,037	0,036	0,036	0,034	0,034
4	0,032	0,032	0,036	0,036	0,036	0,036	0,036	0,036	0,036	0,036	0,032	0,032
3	0,031	0,031	0,035	0,035	0,035	0,035	0,035	0,035	0,035	0,035	0,031	0,031
2	0,030	0,030	0,034	0,034	0,035	0,035	0,035	0,035	0,035	0,035	0,030	0,030
1	0,029	0,029	0,034	0,034	0,034	0,034	0,034	0,034	0,034	0,034	0,028	0,028

- *Collapse Prevention*

Table 7.11 Limit states for columns for collapse prevention

Set1	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,052	0,052	0,053	0,053	0,053	0,053	0,053	0,053	0,053	0,053	0,052	0,052
9	0,051	0,051	0,052	0,052	0,052	0,052	0,052	0,052	0,052	0,052	0,051	0,051
8	0,050	0,050	0,051	0,051	0,051	0,051	0,051	0,051	0,051	0,051	0,049	0,049
7	0,048	0,048	0,050	0,050	0,050	0,050	0,050	0,050	0,050	0,050	0,048	0,048
6	0,047	0,047	0,049	0,049	0,049	0,049	0,049	0,049	0,049	0,049	0,046	0,046
5	0,045	0,045	0,049	0,049	0,049	0,049	0,049	0,049	0,049	0,049	0,045	0,045
4	0,044	0,044	0,048	0,048	0,048	0,048	0,048	0,048	0,048	0,048	0,043	0,043
3	0,043	0,043	0,047	0,047	0,047	0,047	0,047	0,047	0,047	0,047	0,042	0,042
2	0,041	0,041	0,046	0,046	0,046	0,046	0,046	0,046	0,046	0,046	0,041	0,041
1	0,040	0,040	0,045	0,045	0,046	0,046	0,046	0,046	0,045	0,045	0,039	0,039

Table 7.11 (cont'd)

Set2	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,052	0,052	0,053	0,053	0,053	0,053	0,053	0,053	0,053	0,053	0,052	0,052
9	0,051	0,051	0,052	0,052	0,052	0,052	0,052	0,052	0,052	0,052	0,051	0,051
8	0,050	0,051	0,051	0,051	0,051	0,051	0,051	0,051	0,051	0,051	0,050	0,050
7	0,048	0,048	0,050	0,050	0,050	0,050	0,050	0,050	0,050	0,050	0,048	0,048
6	0,047	0,046	0,049	0,049	0,049	0,049	0,049	0,049	0,049	0,049	0,046	0,046
5	0,045	0,045	0,049	0,049	0,049	0,049	0,049	0,049	0,048	0,048	0,045	0,045
4	0,043	0,043	0,048	0,048	0,048	0,048	0,048	0,048	0,048	0,048	0,043	0,043
3	0,042	0,042	0,047	0,047	0,047	0,047	0,047	0,047	0,047	0,047	0,042	0,042
2	0,040	0,040	0,046	0,046	0,046	0,046	0,046	0,046	0,046	0,046	0,040	0,040
1	0,038	0,038	0,045	0,045	0,046	0,046	0,046	0,046	0,045	0,045	0,039	0,039

Set3	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,052	0,052	0,053	0,053	0,053	0,053	0,053	0,053	0,053	0,053	0,052	0,052
9	0,051	0,051	0,052	0,052	0,052	0,052	0,052	0,052	0,052	0,052	0,051	0,051
8	0,049	0,049	0,051	0,051	0,051	0,051	0,051	0,051	0,051	0,051	0,049	0,049
7	0,048	0,048	0,050	0,050	0,050	0,050	0,050	0,050	0,050	0,050	0,048	0,048
6	0,046	0,046	0,049	0,049	0,049	0,049	0,049	0,049	0,049	0,049	0,046	0,046
5	0,045	0,045	0,048	0,048	0,049	0,049	0,049	0,049	0,049	0,049	0,045	0,045
4	0,043	0,043	0,048	0,048	0,048	0,048	0,048	0,048	0,048	0,048	0,043	0,043
3	0,041	0,041	0,047	0,047	0,047	0,047	0,047	0,047	0,047	0,047	0,041	0,041
2	0,040	0,040	0,046	0,046	0,046	0,046	0,046	0,046	0,046	0,046	0,040	0,040
1	0,038	0,038	0,045	0,045	0,046	0,046	0,046	0,046	0,045	0,045	0,038	0,038

Limit States for sets are given below:

Table 7.12 Limit states for columns

SET 1	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
9	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
8	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
7	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
6	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
5	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
4	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
3	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
2	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
1	LS	IO	LS	IO	LS	IO	LS	IO	LS	IO	LS	IO

SET 2	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
9	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
8	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
7	IO	IO	IO	IO	IO	IO	IO	IO	IO	LS	IO	IO
6	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
5	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
4	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
3	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
2	IO	IO	LS	IO	LS	IO	LS	IO	LS	IO	IO	IO
1	CP	IO	LS	IO	LS	IO	LS	IO	LS	IO	LS	IO

Table 7.13 (cont'd)

SET 3	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
9	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
8	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
7	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
6	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
5	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
4	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
3	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
2	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO	IO
1	LS	IO	LS	IO	LS	IO	LS	IO	LS	IO	LS	IO

Table 7.13 Demand/Capacity ratios

SET 1	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,28	0,35	0,43	0,61	0,43	0,61	0,43	0,61	0,43	0,62	0,26	0,37
9	0,39	0,52	0,56	0,72	0,57	0,71	0,57	0,70	0,57	0,71	0,39	0,53
8	0,48	0,46	0,65	0,66	0,65	0,65	0,65	0,66	0,65	0,66	0,47	0,46
7	0,66	0,32	0,94	0,49	0,92	0,48	0,93	0,48	0,95	0,48	0,67	0,32
6	0,49	0,45	0,75	0,73	0,74	0,73	0,74	0,73	0,74	0,73	0,49	0,47
5	0,43	0,50	0,71	0,72	0,71	0,74	0,71	0,74	0,72	0,73	0,50	0,50
4	0,52	0,52	0,75	0,73	0,74	0,72	0,74	0,73	0,74	0,73	0,55	0,51
3	0,54	0,56	0,76	0,77	0,74	0,75	0,75	0,76	0,76	0,75	0,54	0,56
2	0,57	0,63	0,88	0,92	0,87	0,91	0,87	0,91	0,89	0,93	0,60	0,63
1	0,53	0,12	0,48	0,33	0,48	0,33	0,48	0,33	0,48	0,33	0,48	0,13

SET 2	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,36	0,40	0,52	0,69	0,53	0,69	0,52	0,69	0,52	0,69	0,39	0,40
9	0,44	0,53	0,60	0,68	0,59	0,68	0,60	0,68	0,60	0,69	0,44	0,51
8	0,49	0,53	0,65	0,72	0,65	0,72	0,65	0,72	0,65	0,72	0,49	0,54
7	0,68	0,36	0,99	0,51	0,99	0,50	0,99	0,51	0,98	0,19	0,69	0,35
6	0,49	0,54	0,76	0,77	0,76	0,77	0,76	0,78	0,76	0,78	0,50	0,53
5	0,58	0,58	0,83	0,83	0,83	0,82	0,83	0,82	0,84	0,82	0,59	0,58
4	0,53	0,64	0,77	0,89	0,76	0,88	0,77	0,86	0,76	0,88	0,54	0,65
3	0,59	0,63	0,84	0,88	0,83	0,89	0,84	0,88	0,84	0,90	0,60	0,63
2	0,72	0,62	0,18	0,91	0,18	0,90	0,18	0,90	0,18	0,92	0,71	0,61
1	1,22	0,29	0,49	0,35	0,57	0,34	0,56	0,75	0,57	0,34	0,65	0,45

SET 3	A		B		C		D		E		F	
Story	i	j	i	j	i	j	i	j	i	j	i	j
10	0,23	0,37	0,37	0,61	0,36	0,61	0,36	0,61	0,35	0,61	0,23	0,36
9	0,36	0,46	0,53	0,68	0,53	0,68	0,53	0,68	0,53	0,68	0,36	0,49
8	0,44	0,47	0,61	0,65	0,61	0,65	0,60	0,65	0,61	0,65	0,42	0,46
7	0,65	0,32	0,93	0,48	0,93	0,48	0,92	0,48	0,93	0,48	0,66	0,31
6	0,47	0,45	0,72	0,72	0,70	0,73	0,70	0,73	0,72	0,72	0,47	0,45
5	0,45	0,45	0,67	0,77	0,68	0,77	0,68	0,77	0,68	0,77	0,43	0,52
4	0,49	0,56	0,73	0,81	0,72	0,81	0,72	0,81	0,73	0,81	0,48	0,56
3	0,53	0,57	0,79	0,84	0,79	0,82	0,80	0,83	0,80	0,84	0,53	0,58
2	0,61	0,57	0,95	0,87	0,94	0,87	0,93	0,86	0,95	0,88	0,61	0,58
1	0,62	0,11	0,53	0,31	0,52	0,30	0,53	0,30	0,53	0,31	0,62	0,27

7.4.2.1. Results

Performance values of Set 1 are almost symmetrical. Inner columns demand/capacity is higher than outer columns. Maximum IO demand capacity ratio at Set 1 is at Grid E-2th story (at j). Maximum LS demand capacity is at Grid C- 1st story (at i). Maximum IO demand capacity ratio at Set 2 is at Grid E-2th story (at i). Maximum LS demand capacity is at Grid A- 1st story (at i). Maximum IO demand capacity ratio at Set 3 is at Grid B-2th story (at i). Maximum LS demand capacity is at Grid D- 1st story (at i).

CHAPTER 8

RESULTS AND DISCUSSIONS

Considering the model frame and the record sets used in this study, the use of NF-WP, NF-WOP and FF record sets do not affect the structural response much in terms of seismic performance level satisfied and also the mean values of the maximum story drift ratios. However, the dispersion of the maximum story drift ratios is the highest for NF-WP set and it is the lowest for the FF set except for the 7^{th}, 8^{th} and 9^{th} floor levels. It is obvious that the NF-WP set causes higher variation in deformations from record to record, but this variation does not affect the seismic performance level and mean deformation values much. Actually, though there isn't a specific recommendation about pulse-type ground motions in ASCE/SEI 7-10, the existing studies in the literature recommend the use of sets with a limited number of pulse-type ground motions. Furthermore, scaling procedures developed for ordinary motions may not be appropriate for pulse-type motions [2]. Nonetheless, this work can be accepted as an extreme case, with NF-WP set the whole records of whose are pulse-type. The results seem interesting since the effects of record sets with different characteristics are much lower than expected in terms of satisfied performance levels of structural elements. It is obvious that this study should be extended to other structural types and different record selection and scaling techniques, however this attempt is thought to be beneficial for especially the practicing engineers following the code based procedures.

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