SEISMIC PERFORMANCE ASSESSMENT OF MEGA-BRACED FRAME

A DISSERTATION

Submitted in partial fulfillment of the

requirements for the award of the degree

Of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With Specialization in Structural Dynamics)

By

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CANDIDATE'S DECLARATION

I hereby, declare that the work which is being presented in this dissertation report entitled, **"SEISMIC PERFORMANCE ASSESSMENT OF MEGA-BRACED FRAME"**, being submitted in partial fulfilment of the requirements for the award of degree of "Master of Technology" in "Earthquake Engineering" with specialization in Structural Dynamics, to the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, under the supervision of assistant Professor **Dr. P. C. Ashwin Kumar**, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out during the period of June 2018 to June 2019.

Place: Roorkee **BANITA**

Date: Enrollment no. 17526006

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge and belief.

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Date: Assistant professor

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Place: Roorkee

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ABSTRACT

Mega-braced frame is mainly preferred in designing of super-tall buildings, but to check the adequacy of mega-braced frame for low-rise, mid-rise, and high-rise buildings, seismic assessment of these buildings is necessary to performed. In this thesis, two types of bracing is adopted in designing, i.e., multistorey X-bracing and stacked bracing. A sample building is analysed by using software SAP2000 and design procedure is validated with the help of International building code (IBC). By following the same design procedure, all braced frame is analysed by linear method and nonlinear dynamic method. To investigate the collapse behavior of buildings, hinges are assigned in all structural members. In bracing members hinges are assigned manually by considering optimum ranges of slenderness ratio and width to thickness ratio of respective member. The mode of failure is important in comparing the performance of building for low and high seismic events. By comparing the results of nonlinear time history analysis performed on buildings, the braced frame which is more appropriate to execute good performance is selected in designing. The failure mechanism and plastic hinge formation explains details of the behavior of building, by comparing their results for all buildings, best performed bracing is determined for low-rise, mid-rise and high-rise braced frame. Interstorey drift ratio (IDR) has direct relation with behavior of structure component and ductility of structure. So this thesis explains the pattern of IDR at each storey level for all the structures analysed. On other hand the pattern and magnitude of forces coming on foundation level is compared for low-rise, mid-rise and high-rise structure. The column members which are deforming significantly during time history analysis, are compared with respect to the designed axial force.

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CHAPTER 1 INTRODUCTION

Earthquake is sudden violent shaking of the ground caused by the passage of seismic waves through rocks in earth crust. When the energy stored in Earth's crust is suddenly released, seismic waves propagates. Generally when rock masses are straining against each other, they generates fracture and slip. Earthquakes use to occur along geological faults and narrow zones because of movement of rock masses in relation to each other. Earth's crust is made of the huge tectonic plates, which construct major fault lines at their fringes.

Earthquakes usually affect large parts of the earth, and potentially can cause serious damage over large urban areas. The vulnerability to this natural disaster is increasing as urbanization and developments occupy more areas that are prone to the effects of significant earthquakes. Therefore, developing and implementing policies for seismic risk reduction in the areas prone to seismicity, are very essential for governments, which helps in minimizing the loss of life, damage of property, and both social and economic disruptions.

1.1 Impact of earthquake on building

During earthquake, motion of ground is frequent in nature. Therefore, due to seismic actions structure undergoes many reversals of stresses in small duration of earthquake. The building mass being designed controls seismic design in addition to the building stiffness, because earthquake produces inertia forces that are proportional to the building mass. While designing a structure behaving elastically during earthquakes without inducing damage, project becomes economically unviable. So, structure is designed such that deformations are intended to occur and structure can dissipate the energy absorbed during a seismic event. Therefore, the design method for earthquake-resistant building need to withstand

- Minor earthquake in which structure shouldn't produce any damage in its elements;
- Moderate earthquake inducing low damage in structural and non-structural component of a building;
- Severe earthquake inducing considerable damage in structural components of a building, but collapse condition should not reach.

Therefore, in seismic design of a structure there is a balance between project cost and destructions accepted in designing, such that the project becomes economical (Murty et al. 2012).

1.2 Importance of bracing

Apart from moment frame there are many other structural systems which are used to resist lateral loads coming on structure. Lateral load carrying capacity of a structure can be increased by providing structural walls, infills in frame and braced frame. By considering type of loading, section size of members, and design criteria any structural system is adopted in design to enhance stiffness of structure. The moment frame show remarkable deformations even in minor earthquakes. Because of its flexibility the strength in lateral direction is quite low, which results in higher ductility demands. For such higher level of ductility, an efficient designing can't be possible to adopt. Thus, it becomes necessary to make structure stiff with the help of braces to increase lateral load resisting capacity (Jagadeesh and Prakash 2016).

When a structural system exposes to a seismic event, the braces assigned in frame act as fuses in whole system. When deformations occur in braces, transfer of yielding becomes necessary from one storey to other storey to avoid concentration of stresses at a particular level. So lesser value of demand to capacity ratio is preferred while designing of braces.

1.3 Components of braced frame

1.3.1 Steel moment frames

Steel moment frame withstand all and impacts acting on it by the bending action of its members. These members are beam, column and connection between beam and column. The connection between beam and column is designed in such a way that it can resist joint moment and shear in panel zone of column. Cold rolled, hot rolled, built up members are used in designing the steel moment frames.

1.3.1.1 Ordinary moment frames

Ordinary moment frames are designed to take care of gravity load only and show good performance in low seismic region. After application of seismic forces ordinary moment frames can induce inelastic joint rotation up to a limit of 0.02, such that structural members and connections should not show any degradation of strength and stiffness before yielding. Thus, due to lower ductility demand these frames have response reduction factor of lower value.

1.3.1.2 Special moment frames

Special moment frames show good performance in high seismic region and used in those structure which require high level of ductility. After application of seismic forces, special moment frames can induce inelastic joint rotation up to a limit of 0.04 radians, such that structural members and connections should not show degradation of strength and stiffness before yielding. These frames have response reduction factor of higher value, and much stable behavior in nonlinear region.

1.3.2 Concentrically braced frames

Concentrically braced frames (CBFs) assemble its members such that the axes of the connecting members remain concentric at the joint, such that they can bear lateral loads coming on structure by behaving as a vertical concentric truss system. CBFs play an important role in steel structural system in high level of seismicity.

1.3.2.1 Ordinary concentrically braced frames

Ordinary concentrically braced frames (OCBFs) have a Response reduction factor (R) of value 3.0 and no special detailing for improving ductility, and used in areas of low seismicity. After application of seismic forces ordinary concentrically braced frames can induce inelastic joint rotation up to a limit of 0.02 radians, such that structural members and connections should not show degradation of strength and stiffness before yielding.

1.3.2.2 Special Concentrically Braced Frames

Special concentrically braced frames (SCBFs) are designed with relatively large Response Modification Factor, so as a consequence high relative inelastic damage is expected in severe earthquakes. While designing SCBFs, the ductile detailing and proportioning requirements should be ensured to achieve required inelastic deformations. SCBFs are efficient enough to have considerable lateral strength against minor earthquake, but during extreme earthquakes, to safeguard life safety and collapse prevention the inelastic deformation is governed by yielding in tension, buckling in compression, and post buckling deformation of the brace member. The requirement of ductility in structure and inelastic behavior depend upon the level of seismic hazard and the procedure of seismic design (Sabelli et al. 2013).

1.4 Configuration of braces in SCBF

The performance of a system is so much dependent on configuration of braces. There are many configurations of braces which are recommended to use, these configurations are identified in Figure 1.1.

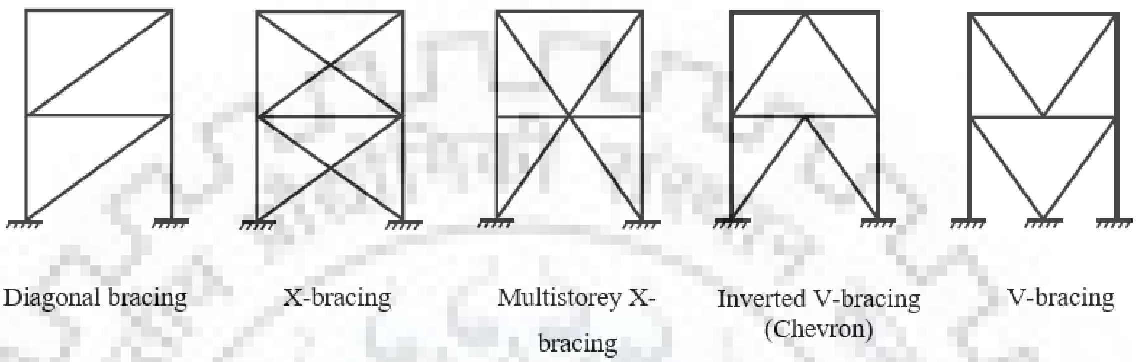


Figure 1.1 Various configuration of braced frame (Sabelli et al. 2013)

The behavior of a brace member is such that it buckles in compression and yields in tension. Initially the brace members have lesser buckling capacity in compression as compared with the yielding capacity in tension, but during successive cycles of yielding and buckling of a brace member, the buckling capacity is reduced to a greater extent due to initial inelastic deformations. So, the bracing systems are balanced in such a way that they have same lateral resistance for tension as well as compression in both directions. For matched tensile and compressive pairs diagonal bracing must be provided. There are some bracing configurations which can directly achieve this balance, like the X-bracing, multistory X-bracing and chevron bracing. X-bracing is most commonly used for shorter structures. In X-bracing the braces intersect at mid-length. Therefore, its capability to buckle is evaluated by considering half length of the brace member. In braced system the damage used to concentrate in some stories. But the multistory X-bracing are beneficial enough in providing a vigorous path to transfer storey shear originating in one storey to the nearby stories, and avoiding accumulation of stresses at one level. The shear transfer is possible even in later stages of buckling and fracture of braces, because the brace member which is in tension is helpful in transfer of shear from one storey to other storey. Chevron or inverted-chevron bracing has intersecting brace connections at mid-span of the beam, which produces large unbalanced forces and bending moments in the beam. K-braces intersect at mid-height of the column is expressly prohibited in AISC 341 (2016), because they create an unbalanced force in columns and induce high moments and deformations in columns prior to the beams. Thus, this results in failure of column before the beam failure and the collapse situation will trigger. Therefore, K-bracing is not allowed for the SCBF system (Sabelli et al. 2013; Razak et al. 2018).

1.5 Behavior of braces

1.5.1 Hysteretic behavior of a brace member

The brace member subjected to reversed cycles of axial loading shows the physical inelastic behavior, which plays an important role in designing ductile braced frames. The behavior of axially loaded members is commonly expressed in terms of the axial load (P), axial deformation (δ), and transverse displacement at mid-length (Δ). A simplified hysteretic curve for a brace member is presented in Figure 1.2.

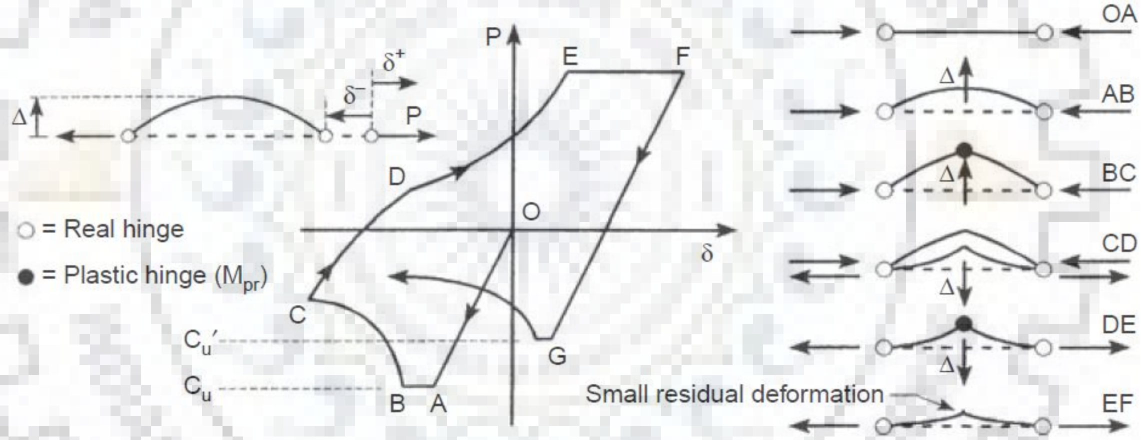


Figure 1.2 Hysteretic behavior of a brace under cycling axial loading (Bruneau et al. 2011)

The hysteretic behavior of a brace can be explained by discussing each segment of the curve shown above:

- OA: starting from the unloading condition brace is compressed in linearly elastic range. At point A brace will reach its buckling capacity (C_u).
- AB: slender brace member deflects laterally with an axial shortening to sustain applied axial load. This produces flexural moment which depends on axial load and displacement at any section from ends. In this case, the brace shows bilinear elasto-

plastic flexural behavior, in which under constant axial force transverse displacement further increases, at point B, when the plastic moment of the brace is eventually reached, plastic hinge forms.

- BC: axial resistance of brace drops, because moment at mid-length of brace can't go beyond of plastic moment capacity of brace, so increasing Δ decreases P.
- CD: Unloading occurs from point C to P = 0, after this when brace is loaded in tension, it shows elastic behavior.
- DE: Plastic hinge form at mid-length of brace, this induces plastic hinge rotations in reverse direction of BC-segment.
- EF: Tension yielding zone.
- FG: Unloading occurs from point F to P = 0. After this recompression is started with an initial deformation, and buckling capacity of brace reduces to C_u'. The elastic buckling plateau length reduces in each successive inelastic cycle.

1.5.2 Effect of slenderness ratio

- For larger slenderness ratio: By analysing the hysteretic curve for slender brace member, it's been concluded out that the OA-segment is small while the AB-segment is long. The segments OA and AB are given in Figure 1.2. Therefore, the hysteretic energy dissipation in compression is less for slender braces.
- For small slenderness ratio: By analysing the hysteretic curve for stocky brace member, it's been concluded out that OA- segment is long while AB-segment doesn't exist. The segments OA and AB are given in Figure 1.2. Therefore, the hysteretic energy dissipation in compression is high for stocky braces.

1.6 Performance based design

In performance based design procedure the structure is designed in such a way that it is able to achieve a specified performance level at a specified level of risk. The structure can withstand signified level of hazard at prescribed level of damage. Performance levels can be represented in categories as:

- Immediate Occupancy (IO): At this state the damage is quite low and there is no significant loss in the strength and stiffness after occurrence of an earthquake as compared with past earthquakes. The risk of injury at this level of damage is low and structure requires only minor repairing.
- Life Safety (LS): At this state of damage some members of structure get damaged, which does not produce hazard condition. Injuries occurred at this performance level creates minor life threatening risk. There is significant margin for partial or total collapse conditions and the structure requires repairing before occupancy.
- Collapse Prevention (CP): At this state there is no margin against collapse condition and significant loss in the strength and stiffness after occurrence of an earthquake as compared with past earthquakes. The structure shows large storey drift, but still can support gravity load. The structure cannot be repaired and not safe for the occupancy.

1.7 Objective

- To perform nonlinear time history analysis (NLTHA) to get to know about the failure mechanism, plastic hinge formation and performance level of each building analysed in this study.
- To understand the conventional design process and compare it with the results obtained from NLTHA.
- To determine which configuration of braces is best suitable for low-rise, mid-rise and high-rise structures, by comparing the performance of each structure at DBE and MCE level of seismicity.
- To check the adequacy of code in terms of designing column members using linear static method as prescribed in ASCE 341(2016).
- To understand the variation of slenderness ratio and width-to-thickness ratio on the response of brace and on the overall response of braced frames.

CHAPTER 2 VALIDATION OF DESIGN PROCEDURE

This sample building is taken from International Building Code IBC (2012) to validate the results during analysis, and this document shows procedure for the design of Special Concentrically Braced Frame (SCBF) building, that comply with the International Building Code and the AISC Seismic provisions for Structure Steel Buildings under seismic loading (AISC-341 2016). The SCBF system has been developed over several cycles of building codes as a moderately ductile system that can withstand moderate inelastic drift while maintaining strength.

2.1 Plan details and configuration of braces

Here the frames are located at the building perimeter, which is more efficient in controlling building torsion and ensuring redundancy. Braces are configured in a two story X configuration, which is advantageous in limiting beam flexural demands in the post buckled condition. Additionally the frames are offset at floor 5 and 6. This reduces the column overturning demand, which is beneficial for column size, base pate demand, and foundation demands. The plan layout and configuration of braces are shown in Figure 2.1 and Figure 2.2 respectively.

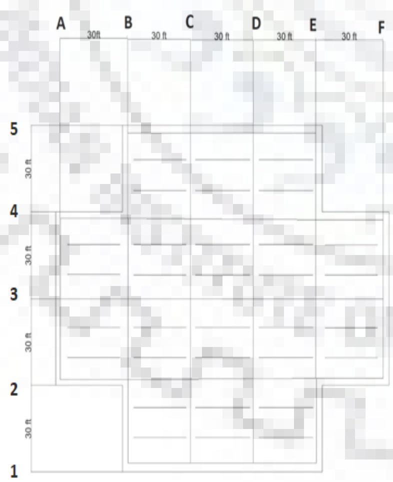


Figure 2.1 Plan of IBC sample building

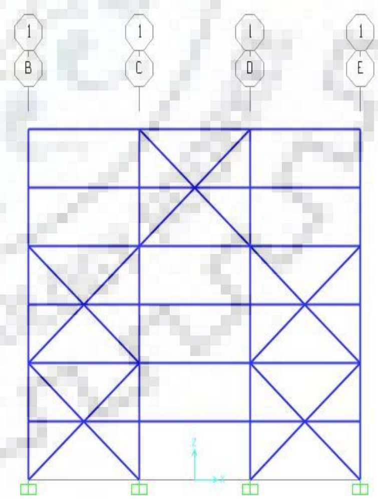


Figure 2.2 Elevation of IBC sample building

2.2 Design data

The data used in designing of braced frame is defined in Table 2.1 as follows:

Table 2.1 Design data of IBC sample building

Frame type	SCBF
Seismic category	D
Importance factor	1.0
Design spectral response acceleration parameter at short periods (S_{ds})	1g
Design spectral response acceleration parameter at 1 sec period (S_{d1})	0.6g
Response reduction factor (R)	6.0
Overstrength factor (Ω_0)	2.0
System deflection amplification factor (C_d)	5.0
Design provision	LRFD
Site location	Los Angeles
Risk category	2nd

2.3 Loads acting on building

2.3.1 Gravity load

Total gravity load coming on floor and roof of this building is defined in Table 2.2.

Table 2.2 Floor and roof load of IBC sample building

Level	Assembly	Unit weight (psf)	Area (ft^2)	Weight (kips)	Total weight (kips)
Floor	Floor	77.7	15,220	1183	1315
	External wall	19	6,990	133	
Roof	Roof	36	1,5220	548	656
	External wall	19	5,700	108	

W = 5(1315) + 656 = 7231kips.

Gravity Loads coming on the building is calculate with the help Assembly Weights of floors, roof and exterior wall given in ASCE-7 (2016) to determine total load coming on each member, which is given as:

Dead Load of floor = 77.7 psf, Dead Load of roof = 36 psf,

Dead Load of external wall = 19 psf, Live Load = 80 psf

2.3.2 Time period

Time period is calculated as

$T_a = C_t h^x = 0.02 \times 72^{0.75} = 0.49$ sec

2.3.3 Lateral load

If Period assumed to be 1.4 times T_a = 0.69 sec

By interpolating between time period of value 0.5 and 2.5, having corresponding values of "k" between 1 and 2 respectively, results in value of 1.095 as k.

$V = C_s \times W = 1205$ kips, and $C_s = \frac{S_{ds}}{(R/I)} = 0.167g$

Where, C_s should not cross the value $\frac{S_{d1}}{\left(\frac{R}{I}\right)T}$.So the lateral load is computed out as $F_x = C_{vx}V$

Where, $C_{vx} = \frac{W_x(h_x)^k}{\sum W_i(h_i)^k}$

Lateral load on each level can be calculated following the procedure explained above, and the results are tabulated in Table 2.3.

Table 2.3 Lateral load on each storey level of IBC sample building

Level	W_i (kips)	H_i (ft)	$W_iH_i^k$	C_{vx}	F_x (kips)	Frame shear(kips)
6th	656	72	70906	0.174	210	113
5th	1315	60	116413	0.286	344	186
4th	1315	48	91177	0.224	269	146
3rd	1315	36	66539	0.163	197	106
2nd	1315	24	42683	0.105	126	68
1st	1315	12	19982	0.049	59	32
Total	7231		407700		1205	651

2.4 Transfer mechanism of brace forces

The mechanism to transfer the brace forces corresponding to their expected strength can be defined by following two conditions:

1) Maximum Tension Force and Maximum Compressive Force condition is shown in Figure 2.3.

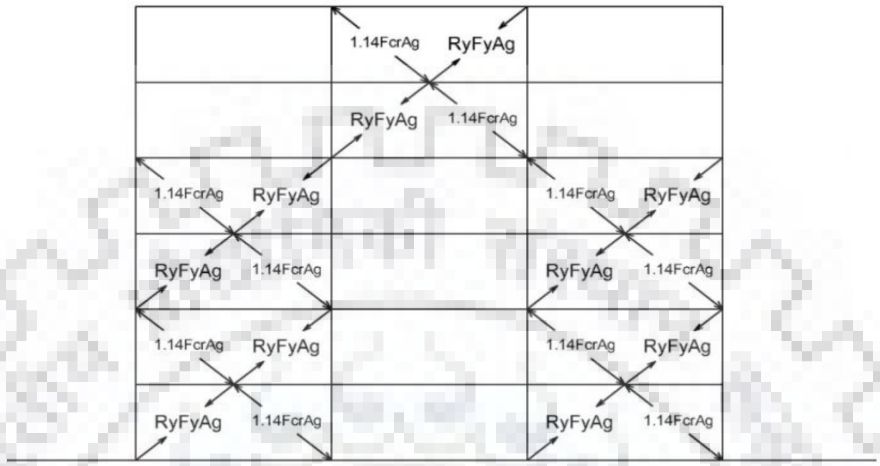


Figure 2.3 Maximum force condition in IBC sample building

2) Maximum Tension Force and Post Buckling Compression Force condition is shown in Figure 2.4.

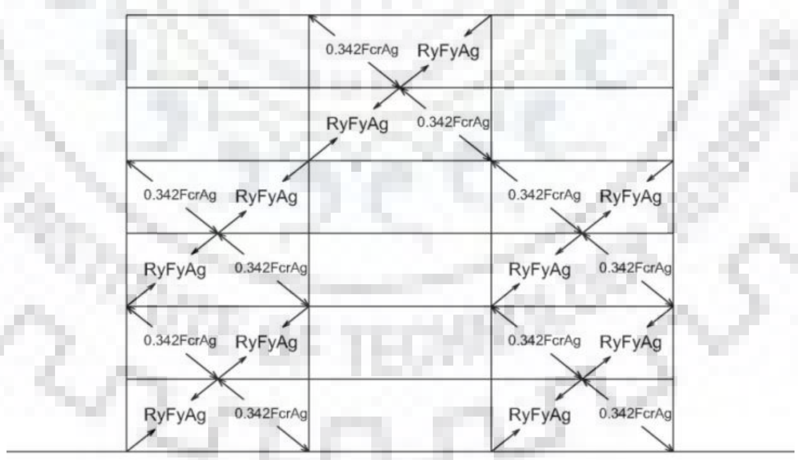


Figure 2.4 Post buckled condition in IBC sample building

2.5 Comparison of results

2.5.1 Brace forces:

As the frame with braces has been loaded with DL, LL, Lateral load, the effective section for braces can be determined after modeling in SAP. Round HSS sections are assigned as the sections for braces by AISC Manual (AISC/LRFD 2005). After fulfilling the required criteria for minimum radius of gyration (r) and depth to thickness ratio (D/t) the sections have been assigned, which are tabulated in Table 2.4.

Table 2.4 Brace seismic forces of IBC sample building

Storey level	Brace seismic Force (kips)	Brace seismic Force (kips)	Error (%)
	IBC	SAP	
6th	72	75	4.16
5th	191	198	3.6
4th	142	147	3.5
3rd	176	181	2.84
2nd	198	203	2.52
1st	208	216	3.84

The brace sections assigned in this building are defined in table below with its properties like radius of gyration (r), slenderness ratio (Kl/r), Depth to thickness ratio (D/t), maximum capacity (ϕ P_n).

Table 2.5 Properties of brace sections of IBC sample building

Storey level	Brace size	r (inch)	Kl/r	D/t	ϕ P_n (kips)
6th	HSS7×0.375	2.35	82	20.1	183
5th	HSS7.625×0.375	2.58	74	21.8	215
4th	HSS7×0.375	2.35	82	20.1	183
3rd	HSS7×0.375	2.35	82	20.1	183
2nd	HSS7.625×0.375	2.58	74	21.8	215
1st	HSS7.625×0.375	2.58	74	21.8	215

Table 2.6 Expected strength of brace section

Storey level	Brace size	Expected tensile strength (kips)	Expected compressive strength (kips)	Post-buckling strength (kips)	Due to over strength factor of 2
6th	HSS7×0.375	429	275	72	366
5th	HSS7.625×0.375	469	332	87	430
4th	HSS7×0.375	429	275	72	366
3rd	HSS7×0.375	429	275	72	366
2nd	HSS7.625×0.375	469	332	87	430
1st	HSS7.625×0.375	469	332	87	430

2.5.2 Column forces

The columns should be designed according to the dead load and live load of top floors and the vertical component of axial forces of attached braces. The columns should be able to transfer total loads with elastic behavior and they assumed not to enter to plastic region (Ghomi 2008).

Table 2.7 Axial force in column sections of IBC sample building

Storey level	Exterior Columns				Interior columns			
	Force (kips)		Error (%)	Section	Force (kips)		Error (%)	Section
	IBC	SAP			IBC	SAP		
6th	21	22	4.7	W12×40	300	307	2.33	W12×45
5th	69	71	2.8	W12×40	385	400	3.89	W12×45
4th	294	308	4.7	W12×45	491	503	2.44	W12×96
3rd	341	355	4.1	W12×45	570	604	5.96	W12×96
2nd	856	897	4.78	W12×96	640	668	4.37	W12×96
1st	903	947	4.87	W12×96	719	747	3.89	W12×96

The columns sections are assigned with respect to the maximum force corresponding to the three cases mentioned below, to make the structure safe in worst condition of loading.

- DL, LL, and brace forces corresponding to maximum expected strength
- DL, LL, and brace forces corresponding to post buckling strength
- DL, LL, and brace forces corresponding to over strength of braces

The values of column forces by IBC and by SAP analysis are tabulated in the Table 2.7, and by comparing the error corresponding to all the forces the structure is validated

2.5.3 Beam forces

The beams should be designed for flexural effects from direct transverse loading of that floor and the axial forces induced from horizontal component of braces forces (Ghomi 2008). Forces in beam sections is found out to be the maximum one out of the three combinations explained below:

- DL, LL, and brace forces corresponding to maximum expected strength
- DL, LL, and brace forces corresponding to post buckling strength
- DL, LL, and brace forces corresponding to over strength of braces

The value of beam forces from maximum of above three combinations is shown in Table 2.8:

Table 2.8 Forces in beam sections of the braced frame

Level	Left Panel		Middle Panel		Right Panel	
	Shear force (kips)	Bending moment (kips-ft)	Shear force (kips)	Bending moment (kips-ft)	Shear force (kips)	Bending moment (kips-ft)
6th	46	785	50	631	25	745
5th	105	1650	74	968	68	1567
4th	135	1907	129	1749	100	1836
3rd	128	2099	178	2480	160	2272
2nd	192	2799	150	2065	156	2714
1st	204	3269	126	1708	179	3178

Positive Bending Moments are Sagging Moments and Negative Moments are Hogging Moments. The beam on 5th level in mid panel experienced an axial compressive force of value 37 kips, due to different brace sections above and below the beam. The beam on 5th level in mid panel, when checked with respect to the results of IBC then we found percentage error which are tabulated in Table 2.9 as:

Table 2.9 Error in beam forces of IBC sample building

Type of force	IBC	SAP	Error (%)
Axial force	38 kips	37 kips	2.63%
Shear force	76 kips	74 kips	2.63%

CHAPTER 3 ANALYSIS OF LOW-RISE BRACED FRAME

A stable and safe building having stories up to six, having enough strength to withstand all loads and impact acting on it, comes under the category of low rise structure. Low rise braced frame is good enough to take care of horizontal load carrying capacity of the structure. A low rise structure provides a safer escape in case of emergency and hazard situations. These buildings are advantageous enough to resist natural impacts rather than high-rise buildings. The construction and maintenance costs are lesser as compared with mid and high-rise structure. While designing these buildings strength is the main criteria rather than rigidity and stability.

In interest to check the best performed configuration of braces in low rise buildings, it is very necessary to model the selected braced frame structures and analyse them for comparison for their performance. So in this thesis two types of bracings are selected for analysis, i.e. multistorey X-bracing and stack bracing.

3.1 Low-rise multistorey X-braced frame

Multistory X-bracing are beneficial enough in providing a vigorous path to transfer storey shear originating in one storey to the nearby stories, and avoiding accumulation of stresses at one level. The shear transfer is possible even in later stages of buckling and fracture of braces, because the brace member which is in tension is helpful in transfer of shear from one storey to other storey (Sabelli 2013).

3.1.1 Plan details and configuration of braces

The columns are designed with I-section, which is weak in bending about the minor axis so these are strengthen up by providing brace member about minor axis. In this symmetric planned building each series of columns is separated at distance of 20 ft. Hollow steel sections (HSS) which are circular in shape, are designed as brace members at an angle of 30.96 degree and located at the periphery of building. Beams and columns are of wide flanged I-sections and are tied up by brace members from storey-1 to storey-6 as shown in Figure 3.2. The plan layout and configuration of braces in the building are shown in Figure 3.1 and Figure 3.2 respectively.

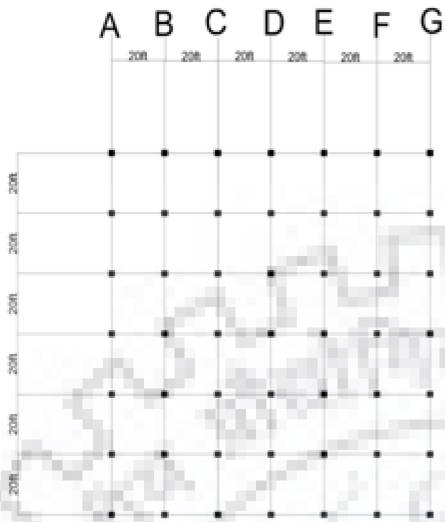


Figure 3.1 Plan of low-rise multistorey X-braced frame

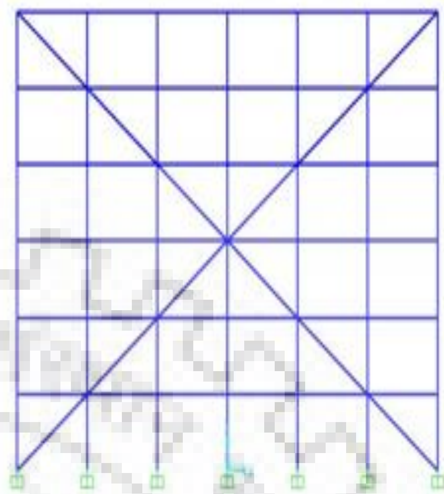


Figure 3.2 Elevation of low-rise multistorey X-braced frame

Section size of all members assemble in low-rise multistorey X-braced frame are tabulated as:

Table 3.1 Section sizes of members in low-rise multistorey X-braced frame

Members	**Sections**
Brace members	HSS8.625x0.625
Beam members	W18x40
Column members	W14x68

3.1.2 Modeling and Designing

Modeling is necessary for designing and analysing the structure. Building is modeled in SAP2000 with the help of plan and elevation details, sectional properties of structural members specified in AISC steel construction manual and load provisions specified in ASCE-7 (AISC/LRFD 2005; ASCE-7 2016). Beams and columns are rigidly connected to form a moment resisting frame (MRF) to take care of gravity load. The columns are supported to ground rigidly by providing fixed connection. The lateral load carrying capacity of frame is increased by assigning brace members, which are modeled as pinned. The exact connection with gusset plate is not modeled explicitly in this study. The energy dissipation capacity and ductility of SCBFs largely depend on the slenderness ratio and width-to-thickness ratio of braces. So while assigning hinges in brace members an optimum range of these parameters is been provided to achieve the enhanced seismic performance of SCBFs. (Kumar et al. 2015)

In this building braces are configured in pattern of multistorey X-bracing, which is advantageous in providing a robust path to transfer storey shear. In high seismic zones braced frames are preferred as compared with moment resisting frames. During earthquake concentrically braced frame (CBF) shows cyclic behavior, the same brace member experience successive cycles of yielding in tension and buckling in compression as shown in Figure 3.3.

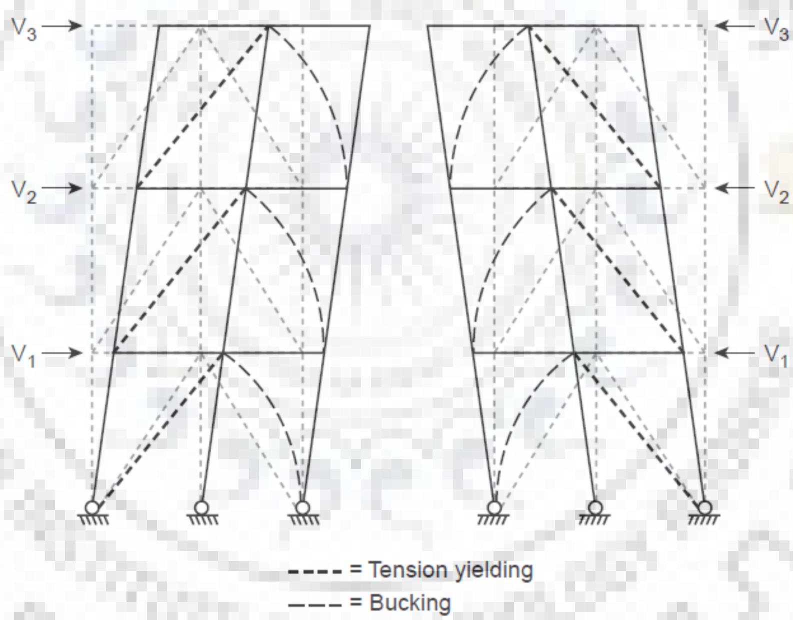


Figure 3.3 Inelastic behavior of CBF (Bruneau et al. 2011)

The impacts due to reversed cycle loading which is induced by earthquakes, results in repeated buckling and straightening of the member at the point of local buckling, which leads to low cycle fatigue. To survive an earthquake, the brace members must have ability to endure high inelastic displacements, in absence of much loss in strength and stiffness. The braces are energy dissipating elements in CBF. The design philosophy of braces includes a special design to sustain plastic deformations and to dissipate hysteretic energy in a stable manner by following successive cycles of yielding in tension and buckling in compression of braces, which further leads requirement of designing special concentrically braced frame (SCBF) over ordinary concentrically braced frame (OCBF). Because braces behave like the fuses in structure, plastic deformation occurs only in braces without damaging beams and columns in frame.

3.1.3 Modal analysis

When a multi-degree of freedom system is excited by an external load system, it results in the dynamic response, which is computed from modal analysis. The total response of system determined by modal analysis is combination of responses of all modes. The equation of motion of multi degree of freedom system is like:

$$m\ddot{u} + c\dot{u} + ku = p(t) \tag{3.1}$$

The solution of above equation consists of spatial and temporal coordinates, thus the displacement vector can be defined as:

$$u(t) = \{\emptyset\}q(t) \tag{3.2}$$

By equation 3.1 and 3.2 the equation resulted as:

$$M_n\ddot{q_n} + C_n\dot{q_n} + K_n q_n = P_n(t) \tag{3.3}$$

Mass, stiffness, damping and load matrix are defined as:

$$M_n = \phi_n^T m \phi_n \tag{3.4}$$

$$K_n = \phi_n^T k \phi_n \tag{3.5}$$

$$C_n = \phi_n^T c \phi_n \tag{3.6}$$

$$P_n(t) = \phi_n^T p(t) \tag{3.7}$$

The total modal mass is account for at least 90% of total seismic mass, and the modes up to that point are taken into design consideration. The modal mass M_k of k^{th} mode can be defined as:

$$M_k = \frac{\left[\sum_{i=1}^{n} W_i \phi_{ik}\right]^2}{g\sum_{i=1}^{n} W_i (\phi_i)^2} \tag{3.8}$$

The modal analysis is performed on this building and its results are tabulated as:

Table 3.2 Modal analysis results of low-rise multistorey X-braced frame

Mode	Time period (sec)	Mass participation factor
1	0.41	0.86
2	0.16	0.08

3.1.4 Response spectrum analysis

The procedure to find out response spectra is given in ASCE-7 (2016), parameters S_S and S_1 shall be determined from the 0.2 and 1 sec spectral response accelerations respectively.

The MCER spectral response acceleration parameter for short periods (S_{MS}) and at 1 sec (S_{M1}), adjusted for Site Class effects, shall be determined by

$S_{MS} = F_a S_s$

$S_{M1} = F_v S_1$

Where, S_S is the mapped MCER spectral response acceleration parameter at short periods and S_1 is the mapped MCER spectral response acceleration parameter at a period of 1 sec

$S_{DS} = \frac{2}{3} S_{MS}$

$S_{D1} = \frac{2}{3} S_{M1}$

Spectral acceleration value:

$S_a = s_{ds}\left[0.4 + 0.6\frac{T}{T0}\right]$ for $T < T_0$

$S_a = \frac{S_{d1}}{T}$ for $T_s < T < T_l$

$S_a = \frac{S_{d1}T_l}{T^2}$ for $T > T_l$

Here, S_{DS} is the design spectral response acceleration parameter at short periods, S_{D1} is the design spectral response acceleration parameter at 1-s period and T is the fundamental period of the structure,

$T_0 = 0.2\,\frac{S_{d1}}{S_{ds}}$

$T_s = \frac{S_{d1}}{S_{ds}}$

T_l = long-period transition period (s)

By the method explained above Response spectrum can be plotted by choosing appropriate site-conditions and damping ratio. Here the soil is of class D and for steel structure the damping ratio is taken as 2 %.

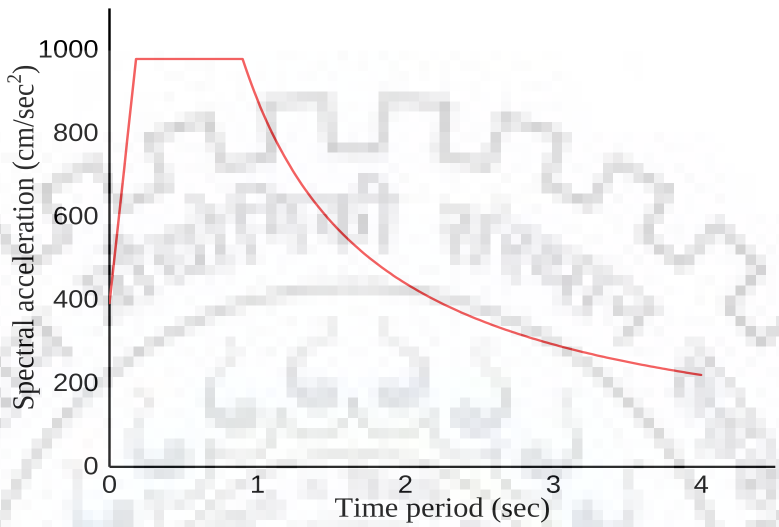


Figure 3.4 Response Spectrum at Los Angeles

The base shear determined by analysing the building by Response Spectrum method is tabulated as:

Table 3.3 Base shear of low-rise multistorey X-braced frame

	Base shear (kips)	
	Response spectrum analysis (V_B)	Equivalent static method (V_B')
X-direction	170.00	563.72
Z-direction	1569.55	1569.55

When the base shear determined by response spectrum method (V_B) is less than the base shear determined by equivalent static method (V_B'), the storey shear forces and base reaction is multiply by a multiplying factor, i.e., V_B'/ V_B Thus the base shear is corrected by a factor of 3.32.

3.1.5 Nonlinear time history analysis

The time history analysis determines the response of a structure due to forces, displacements, velocities or accelerations that vary with time. This method includes two versions, DIRECT INTEGRATION and MODAL SUPERPOSITION. For linear analysis appropriate method is modal superposition method, whereas for nonlinear analysis direct integration method is more satisfactory method. In direct integration method we get step by step solution of Equation of motion, which is basically described as:

$$M_n \ddot{q_n} + C_n \dot{q_n} + K_n q_n = P_n(t) \tag{3.9}$$

Where, M_n, C_n, K_n are the mass, damping, and stiffness matrices, respectively.

To study the behavior of special concentrically braced frames in dynamic condition, structure is analysed by nonlinear time history analysis (NLTHA). The analysis is performed in software SAP2000 by Hilber-Hughes-Taylor method, with α=0, β=0.25, ϒ=0.5 and a damping of 2 percent is provided.

3.1.5.1 Selection of ground motions

According to distance of recorded station measured from fault plane, ground motions are of two types i.e., near field records and far field records. Near field ground motions are measured at a distance less than 10 km, whereas far field records are measured at a distance greater than 10 km. In this study a total of 20 “far-field” ground motion records named as LA01-LA20 are selected to evaluate the dynamic response of framed structure. The further details of ground motions like site details, earthquake magnitude, distance of station from fault plane, peak ground acceleration (PGA), maximum incremental velocity (MIV) are shown in Table 3.4 (Morgen and Kurama 2008).

Table 3.4 Details of ground motions

Record name	Site location	EQ magnitude	Distance (km)	Scale factor	PGA	MIV (cm/sec)
LA01	Imperial valley, 1940, El Centro	6.9	10	2.01	0.46	89.2
LA02	Imperial valley, 1940, El Centro	6.9	10	2.01	0.68	96.1
LA03	Imperial valley, 1979, Array #05	6.5	4.1	1.01	0.39	103
LA04	Imperial valley, 1979, Array #05	6.5	4.1	1.01	0.49	74.8
LA05	Imperial valley, 1979, Array #06	6.5	1.2	0.84	0.30	106
LA06	Imperial valley, 1979, Array #06	6.5	1.2	0.84	0.23	81.7
LA07	Landers, 1992, Barstow	7.3	36	3.20	0.42	59.3
LA08	Landers, 1992, Barstow	7.3	36	3.20	0.42	71.9
LA09	Landers, 1992, Yermo	7.3	25	2.17	0.52	135
LA10	Landers, 1992, Yermo	7.3	25	2.17	0.36	76.8
LA11	Loma Prieta, 1989, Gilroy	7.0	12	1.79	0.66	79.7
LA12	Loma Prieta, 1989, Gilroy	7.0	12	1.79	0.97	88.9
LA13	Northridge , 1994, Newhall	6.7	6.7	1.03	0.68	138
LA14	Northridge , 1994, Newhall	6.7	6.7	1.03	0.66	132
LA15	Northridge , 1994, Rinaldi RS	6.7	7.5	0.79	0.53	124
LA16	Northridge , 1994, Rinaldi RS	6.7	7.5	0.79	0.58	165
LA17	Northridge , 1994, Sylmar	6.7	6.4	0.99	0.57	103
LA18	Northridge , 1994, Sylmar	6.7	6.4	0.99	0.82	139
LA19	North Palm Springs, 1986	6.0	6.7	2.97	1.02	100
LA20	North Palm Springs, 1986	6.0	6.7	2.97	0.99	150

3.1.5.2 Scaling of ground motion records

The ground motion records are scaled by SEISMOMATCH for a site in Los Angeles with a stiff soil profile of site class-D. These time histories are scaled to make their spectrum compatible with site response spectrum. The outcomes of nonlinear dynamic time history analysis are dependent on the actual ground motion records having proper characteristics of considered site soil conditions, site seismicity, and seismic demands. Ground motion records provide divergence in earthquake characteristics for all time histories, which are realistic in their average properties and individual properties. Target response spectrum is plotted by SEISMOMATCH software by providing appropriate details of site condition and code provisions of ASCE-7 (2016). Ground motions are scaled to make response spectrum of these selected ground motion compatible with target response spectrum.

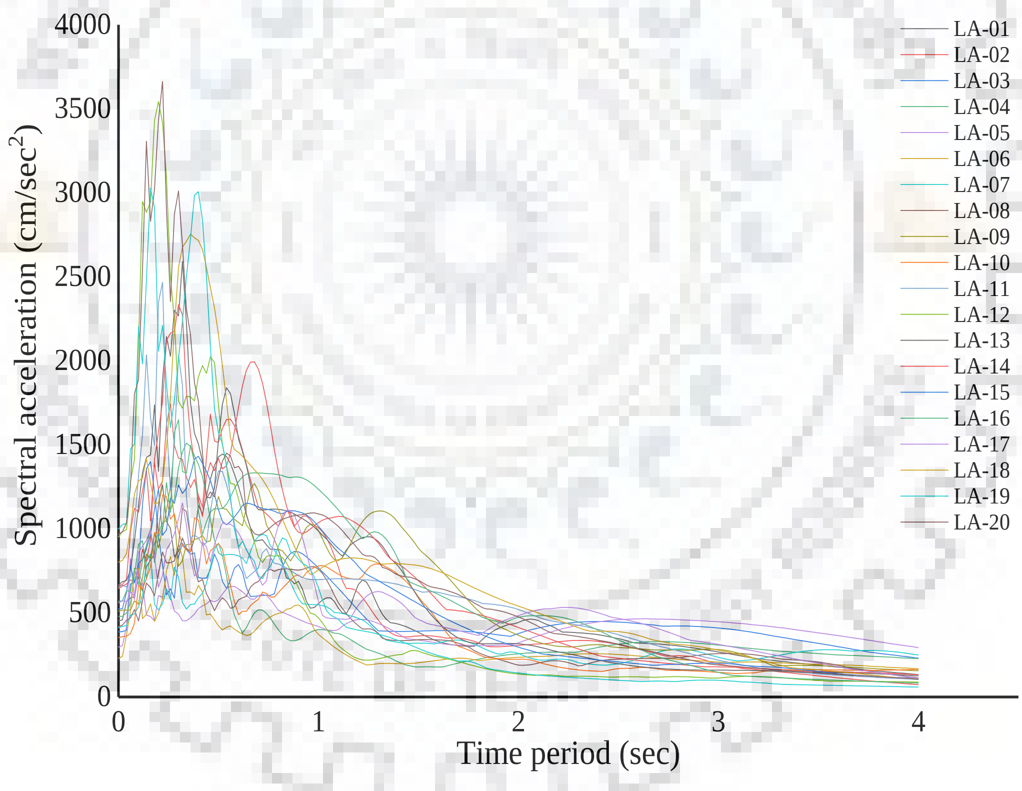


Figure 3.5 Response spectrum of selected ground motions before scaling

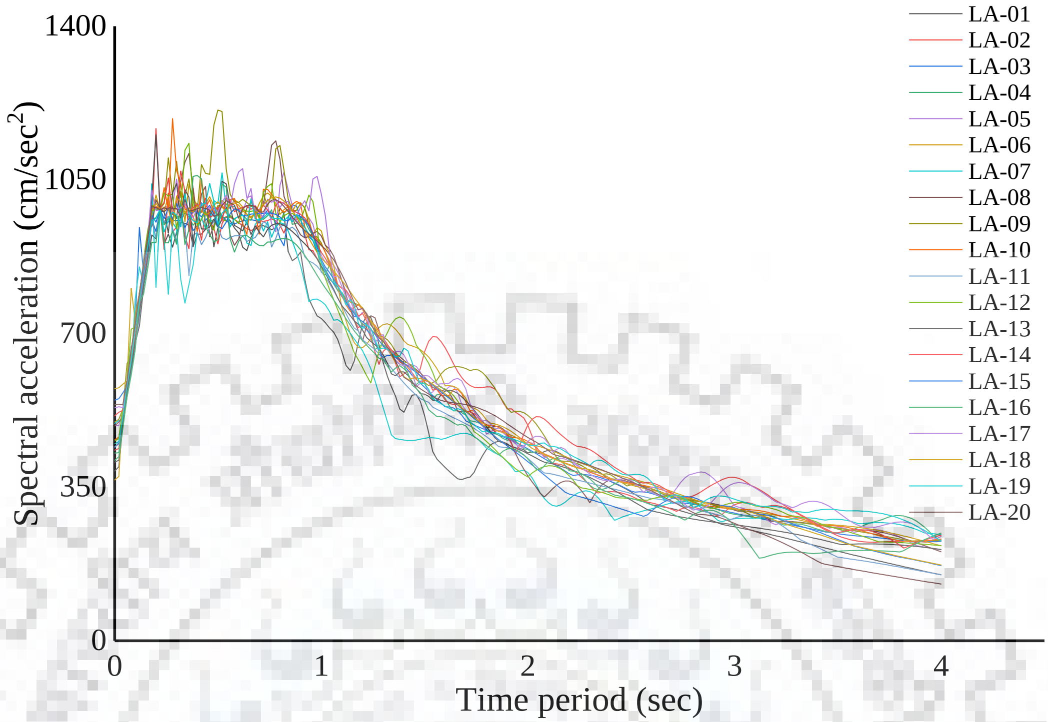


Figure 3.6 Response spectrum of selected ground motions after scaling

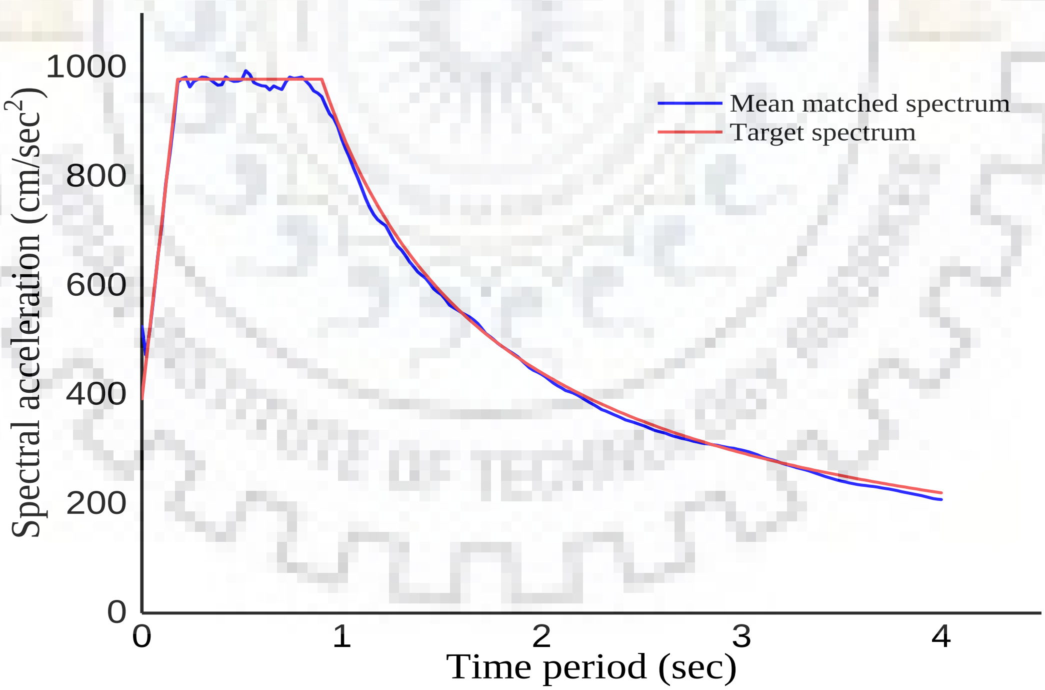


Figure 3.7 Mean matched response spectrum and target response spectrum

3.1.5.3 Results of nonlinear time history analysis

Special Concentrically Braced Frames (SCBFs) are special kind of concentrically braced frame, in which inelastic drift capacity is increased by special detailing. This system is basically an economical system for the low-rise structures located in highly seismic area. According to level of seismicity the inelastic drift is very important criteria to be considered in designing. A target inelastic deformation having storey drift of 2.5% is commonly assumed as corresponding to the condition earlier to brace fracture. When the storey drift gains a value of 5 %, potential collapse is estimated at the point. The drift limit of 5 % is arbitrarily chosen at the point when the connectivity between the frame, braces and begin to lose (Sabelli 2013). The inter story drift ratio (IDR), displacement of a storey with respect to adjacent storey divided by the storey height, plays an important role in seismic analysis of buildings. Because the structural components and story ductility demands have the reasonable correlation with inter story drifts (Vafaei 2015). The inter story drift profiles at DBE and MCE level is obtained from nonlinear time history analyses of the structure, which are presented in Figure 3.8 and Figure 3.9.

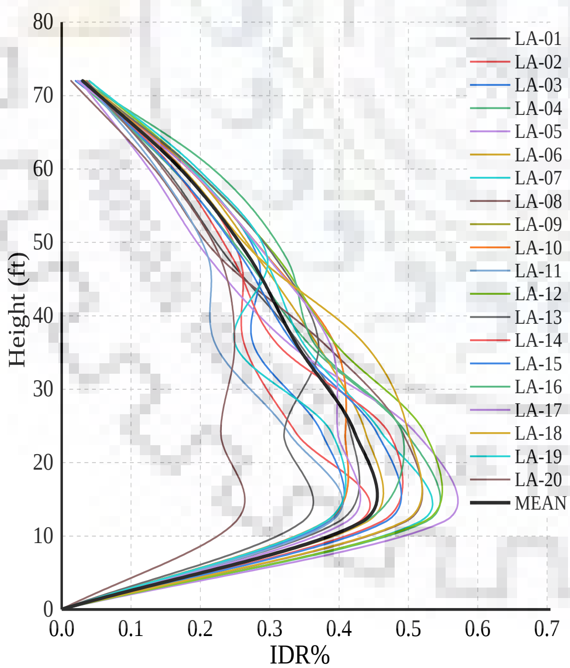


Figure 3.8 IDR profile of low-rise multistorey X-braced frame in DBE case

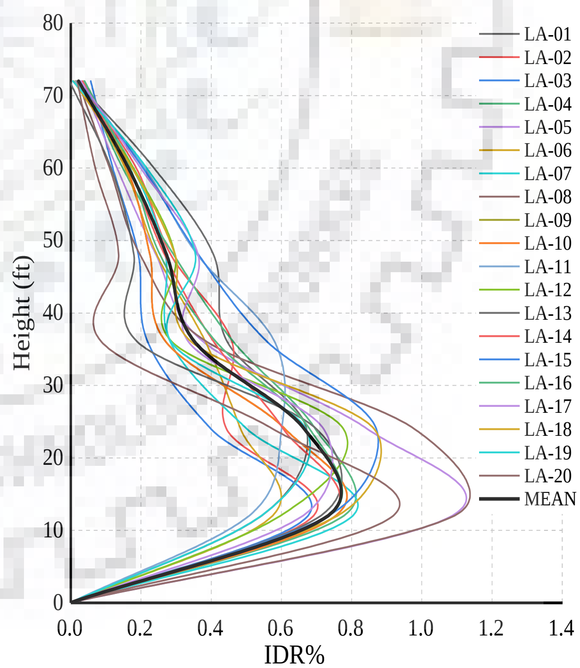


Figure 3.9 IDR profile of low-rise multistorey X-braced frame in MCE case

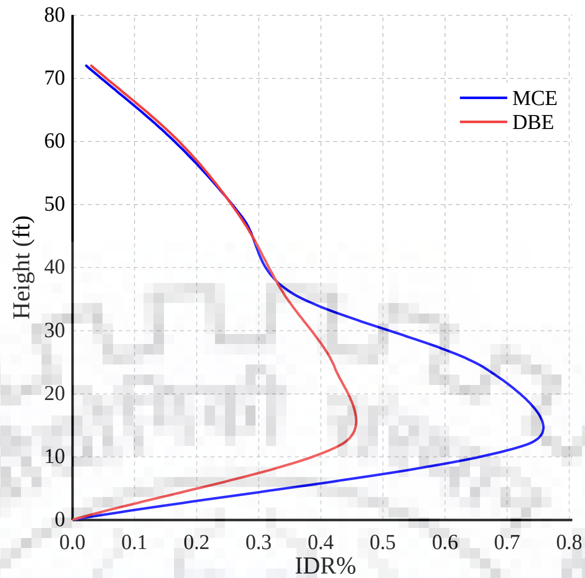


Figure 3.10 Mean IDR profile comparison between DBE and MCE case

The inter storey drift ratio (IDR) of building gains a maximum value of 0.55% at 1st storey level for the ground motion named LA-05 on DBE level, and a value of 1.1% at 1st storey level for the ground motion named LA-08 on MCE level. There are more concentration of inelastic demand at the lower stories that results in the higher value of inter storey drift which can be seen in the figures above. Behavior of building is almost identical for top 3 stories and having less inter storey drift as compared with the bottom three stories. The horizontal displacement of roof corresponding to that ground motion whose result is most severe is shown in Table 3.5

Table 3.5 Roof displacement of low-rise multistorey X-braced frame

Seismic hazard level	DBE level	MCE level
Roof displacement (ft)	0.21	0.334

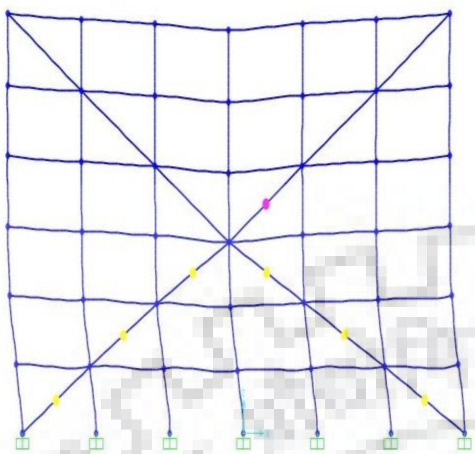


Figure 3.11 Plastic hinge formation and failure mechanism of low-rise multistorey X-braced frame in DBE case

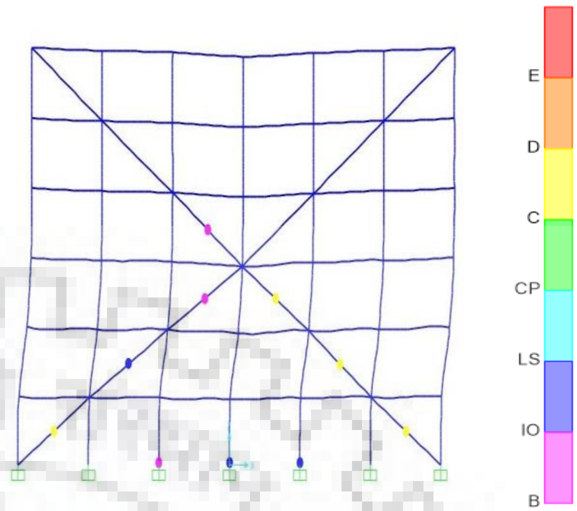


Figure 3.12 Plastic hinge formation and failure mechanism of low-rise multistorey X-braced frame in MCE case

The drift capacity of a structure depends upon buckling and yielding of brace members. The axial ductility is controlled by proportioning of braces and method of detailing used in designing. Less ductile response is preferred in designing of beams columns and their connections. High level of ductility is preferred for braces to have required level of performance by increasing drift capacity of structure. Braced frames are efficient enough to have considerable lateral strength against minor earthquake, but during extreme earthquakes, to ensure life safety and collapse prevention the inelastic deformation is governed by yielding in tension, buckling in compression, and post buckling deformation of the brace member The mechanism of plastic hinge formation is explained in Figure 3.11 and Figure 3.12 for DBE and MCE cases respectively. Structural performance on DBE level is such that the brace members are prior to take stresses and results in plastic hinge formation to dissipate the energy. At DBE level braces in bottom three stories are forming hinges in CD region and on fourth storey in IO region. Thus the inelastic demand is more concentrated at the lower level that finally results in comparatively high inter storey drift at lower storey level. Whereas at MCE level the hinges are resulting to form at support level of mid three columns and resulting into more inelastic drift.

3.2 Low-rise stack braced frame

In stacked braced frames the braces are located at same location in plan on each level of storey. These frames can develop high overturning forces, basically stacked braced frame are good enough in transferring the overturning moments to other bays. For designing of the elements which are interlinking these frames, it becomes critical to ensure that brace ductility remains the primary source of inelastic drift.

3.2.1 Plan details and configuration of braces

The columns are designed with I-section, having a uniform height of 12 ft and each series of columns is separated at distance of 20 ft. Beams and columns are of wide flanged I-sections. Hollow steel sections (HSS) which are circular in shape are designed as brace members at angle of 50.2 degree and 30.96 degree for exterior and interior panels respectively. Braced frames are located at only periphery of building. The plan layout and configuration of braces in the building are shown in Figure 3.13 and Figure 3.14 respectively.

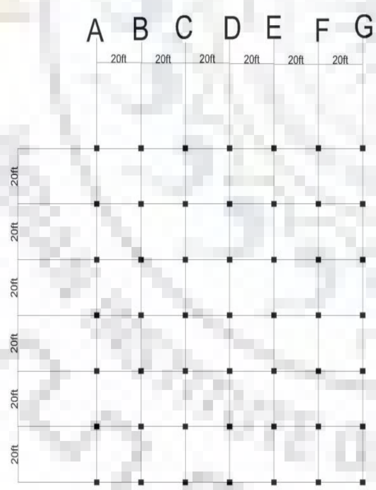


Figure 3.13 Plan of low-rise stack braced frame

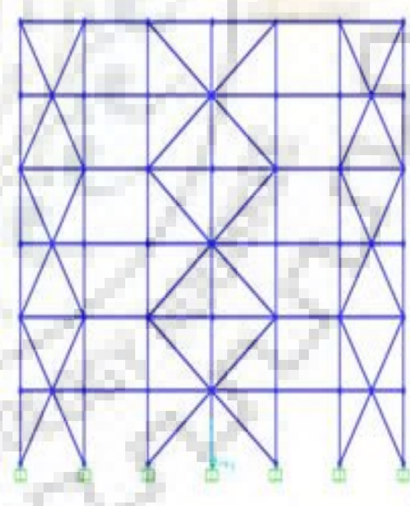


Figure 3.14 Elevation of low-rise stack braced frame

Section size of all members assemble in low-rise stack braced frame are tabulated as:

Table 3.6 Section sizes of members of low-rise stack braced frame

Members	Sections
Brace members in end panels	HSS8.625×0.5
Brace members in mid panels	HSS6.625×0.5
Beam members	W18×283
Column members	W14×428

3.2.2 Modal analysis

The results of structure analysed by modal analysis are tabulated as:

Table 3.7 Results of modal analysis of low-rise stack braced frame

Mode	Time period (sec)	Mass participation factor
1	0.31	0.83
2	0.10	0.11

3.2.3 Response spectrum analysis

The base shear determined by analysing the building by Response Spectrum method can be tabulated as:

Table 3.8 Base shear of low-rise stack braced frame

	Base Shear (kips)	
	Response Spectrum analysis (V_B)	Equivalent static method (V_B')
X-direction	221.73	563.72
Z-direction	2095.15	2095.15

When the base shear determined by response spectrum method (V_B) is less than the base shear determined by equivalent static method (V_B'), the storey shear forces and base reaction is multiply by a multiplying factor, i.e., V_B'/ V_B Thus the base shear is corrected by a factor of 2.54.

3.2.4 Nonlinear time history analysis

The inter story drift profiles at DBE and MCE level is obtained from nonlinear time history analyses of the structure are presented in Figure 3.15 and Figure 3.16.

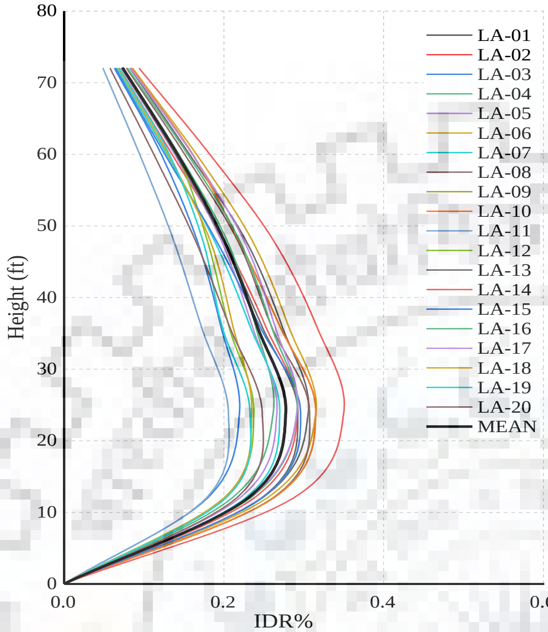


Figure 3.15 IDR profile of low-rise stack braced frame in DBE case

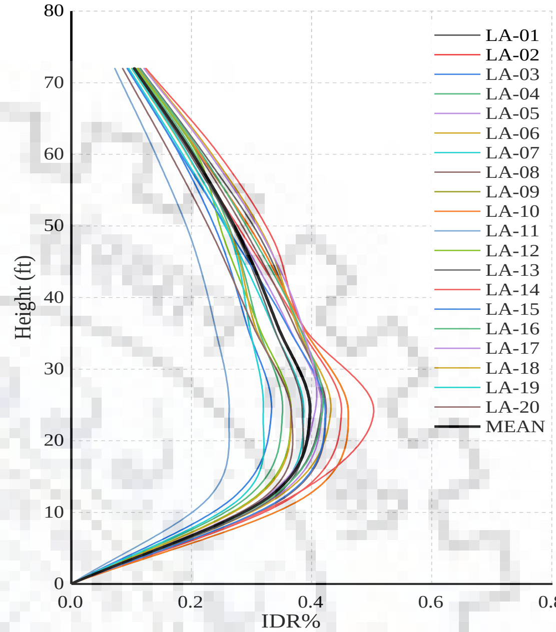


Figure 3.16 IDR profile of low-rise stack braced frame in MCE case

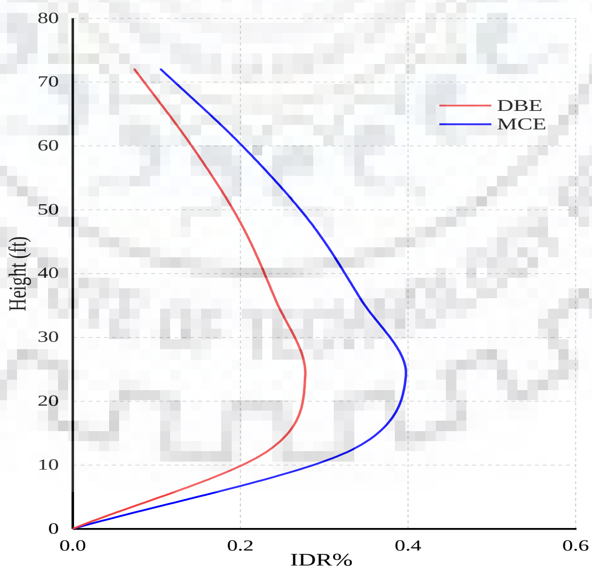


Figure 3.17 Mean IDR profile comparison between DBE and MCE case for low-rise stack braced frame

The inter storey drift ratio (IDR) of stack braced frame gains a maximum value of 0.35% at 2nd storey level for the ground motion named LA-14 on DBE level, and a value of 0.5% at 2nd storey level for the ground motion named LA-14 on MCE level. From Figure 3.17 it is clear that due to more concentration of inelastic demand at lower storey IDR is maximum at lower storey level as compared with stories at higher level. The horizontal displacement of roof corresponding to that ground motion whose result is most severe is shown in Table 3.9

Table 3.9 Roof displacement of low-rise stack braced frame

Seismic hazard level	DBE level	MCE level
Roof displacement (ft)	0.18	0.23

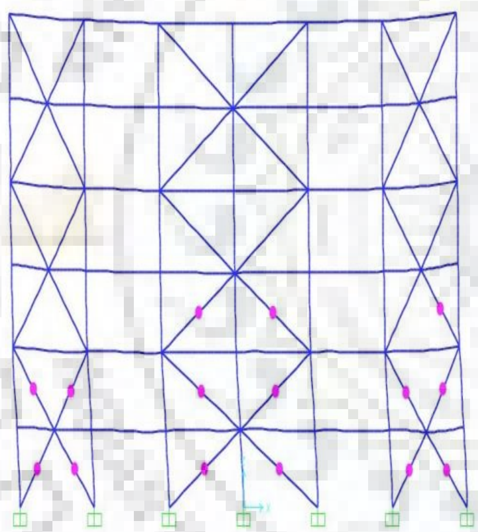


Figure 3.18 Plastic hinge formation and failure mechanism of low-rise stack braced frame in DBE case

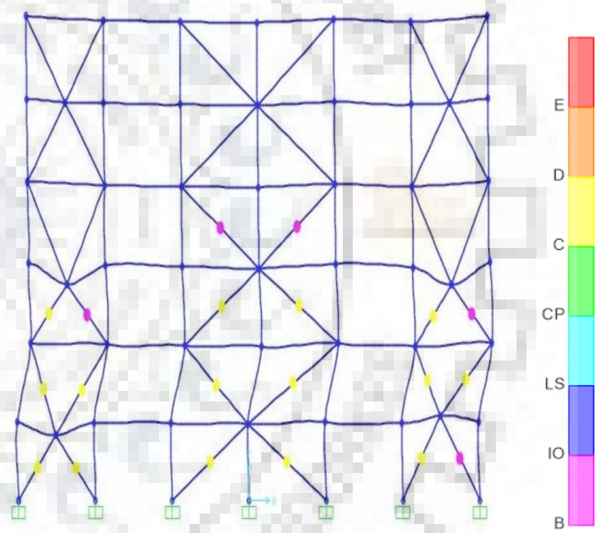


Figure 3.19 Plastic hinge formation and failure mechanism of low-rise stack braced frame in MCE case

Behavior of building can further be estimated by the pattern of plastic hinge formation. Building is behaving in such a way that most of the hinges are forming at lower level, thus the strength of braces on higher stories is not fully utilized. Braces are prior to take forces and form plastic hinges to dissipate energy, as shown in Figure 3.18 and Figure 3.19. In DBE case the category of all hinges is under Immediate Occupancy whereas in MCE case mostly hinges are in CD region for bottom three stories resulted as the higher accumulation of stresses.

3.3 Comparison between low-rise multistorey X-braced frame and stack braced frame

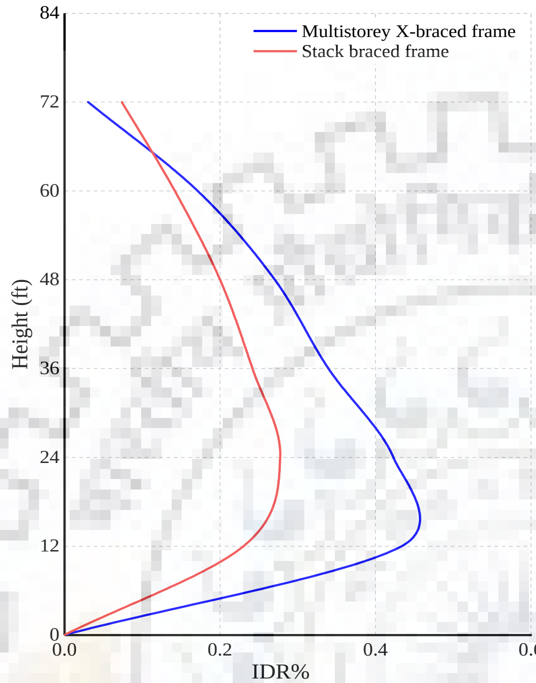


Figure 3.20 Comparison between low-rise braced frames on DBE level

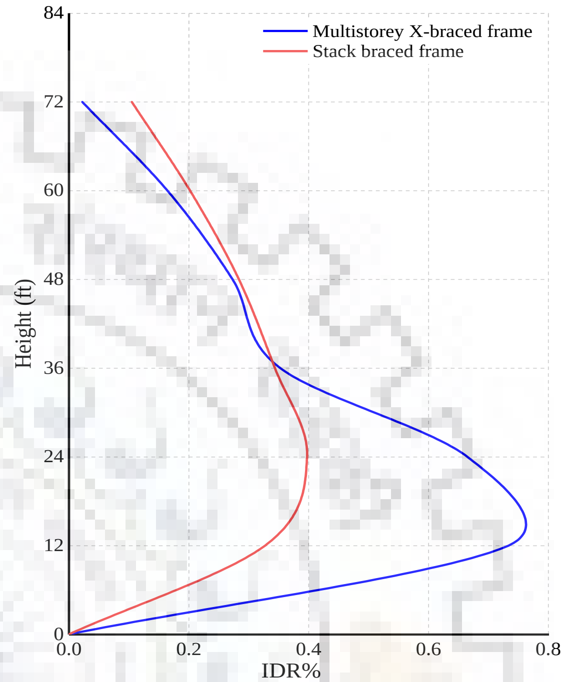


Figure 3.21 Comparison between low-rise braced frames on MCE level

Distribution of stresses in all stories is quite uniform in case of Stack Bracing, which results in uniformity in IDR profile. But in Multi Storey X bracing due to the higher accumulating stresses at lower storey level, structure experiences higher inter storey drift as compared with stack bracing. Stack bracing shows quite good performance than Multi Storey X bracing.

The columns located at ground storey are named C_1 to C_7 from left to right, the axial force, shear force and bending moment in these columns at the foundation level are defined in Table 3.10 and Table 3.11 for DBE and MCE cases. The values of forces and moments for multi-storey X braced frame are less as compared with stack braced frame. The axial forces coming on columns C_1, C_4, C_7 are less as compared with other columns, but variation is not that significant. In multistorey X-braced frame the axial force, shear force and bending moment are almost uniform in nature, which is not in stack bracing. The large variation in column axial forces affects the designing of foundation. In stack braced frame Shear force and moments are less for columns at extremity but high for other columns.

Table 3.10 Forces at foundation level for low rise structure at DBE level

	Multistorey X-braced frame			Stack braced frame		
Ground motion	LA-14			LA-14		
Columns	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)
C_1	194.40	17.80	156.15	1012.43	107.42	921.90
C_2	308.14	23.80	180.24	831.58	138.18	1021.51
C_3	314.22	23.16	178.49	819.08	134.02	1015.38
C_4	304.08	23.18	178.68	265.19	133.15	1007.77
C_5	342.52	23.15	178.70	856.54	137.85	1020.56
C_6	333.22	23.58	179.90	797.76	131.90	1007.68
C_7	200.73	21.45	171.23	1044.59	111.43	923.90

Table 3.11 Forces at foundation level for low rise structure at MCE level

	Multistorey X-braced frame			Stack braced frame		
Ground motion	LA-14			LA-14		
Columns	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)
C_1	291.61	26.70	234.22	1356.28	136.09	1190.09
C_2	462.21	35.71	270.36	1039.21	180.10	1342.49
C_3	471.34	34.75	267.73	998.62	172.07	1328.11
C_4	456.13	34.77	268.02	351.13	171.47	1319.15
C_5	513.79	34.73	268.05	1010.58	178.18	1335.57
C_6	499.84	35.37	269.85	1027.27	172.10	1328.56
C_7	301.1	32.18	256.84	1370.15	139.82	1195.12

CHAPTER 4 ANALYSIS OF MID-RISE BRACED FRAME

A stable and safe building having stories 7 to 12 having enough strength to withstand loads coming on it, comes under the category of mid-rise structure. The mid-rise building can be a way to achieve appropriate, transit-supporting densities without overwhelming the surrounding context. A mid-rise building has a vertical built form that is moderately taller than single family homes or horizontal multiple housing.

4.1 Mid-rise multistorey X-braced frame

4.1.1 Plan details and configuration of braces

The columns are designed with I-section, having a uniform height of 12 ft and each series of columns is separated at distance of 20 ft. Beams and columns are of wide flanged I-sections. Hollow steel sections (HSS) which are circular in shape are designed as brace members at angle of 30.96 degree for exterior and interior panels respectively. Braced frames are located at only periphery of building. The plan layout and configuration of braces in the building are shown in Figure 4.1 and Figure 4.2 respectively.

Figure 4.1 Plan of mid-rise multistorey X-braced frame

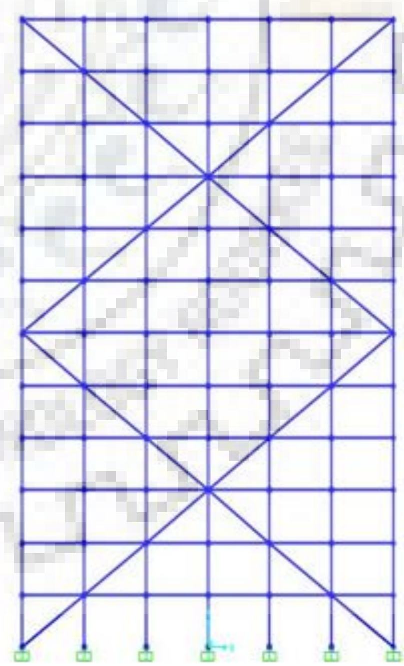


Figure 4.2 Elevation of mid-rise multistorey X-braced frame

Section size of all members assemble in mid-rise multistorey X-braced frame is tabulated as:

Table 4.1 Section sizes of brace and beam members of mid-rise multistorey X-braced frame

Members	Specification	Sections
Brace members	Storey (1-6)	HSS10×0.625
	Storey (6-12)	HSS8.625×0.5
Beam members	Storey (1-6)	W18×60
	Storey (6-12)	W18×50

Table 4.2 Section sizes of column members of mid-rise multistorey X-braced frame

Members	Specification	Exterior sections	Interior sections
Column members	Storey (1-6)	W14×233	W14×68
	Storey (6-12)	W14×132	W14×68

4.1.2 Modal analysis

The results of structure analysed by modal analysis are tabulated as:

Table 4.3 Results of modal analysis of mid-rise multistorey X-braced frame

Mode	Time period (sec)	Mass participation factor
1	0.82	0.78
2	0.29	0.12

4.1.3 Response spectrum analysis

The base shear determined by analysing the building by Response Spectrum method is tabulated as:

Table 4.4 Base shear of mid-rise multistorey X-braced frame

	Base Shear (kips)	
	Response spectrum analysis (V_B)	Equivalent static method (V_B')
X-direction	358.07	1179.11
Z-direction	3396.29	3396.29

When base shear by response spectrum (V_B) is less than the base shear by equivalent static method (V_B'), the storey shear forces and base reaction is multiply by a multiplying factor, i.e., V_B'/V_B Thus the base shear is corrected by a factor of 3.29.

4.1.4 Nonlinear time history analysis

The response of a structure due to forces, displacements, velocities or accelerations that vary with time is determined under the nonlinear time history analysis (NLTHA). To study the behavior of the structure in dynamic condition, structure is analysed by NLTHA by integrating the solutions in each time interval. The analysis is performed in software SAP2000 by Hilber-Hughes-Taylor method, with α=0, β=0.25, Υ=0.5 and a damping of 2 percent is provided. Ground Motions selected for analysis are same as selected in "Low-rise multistorey X braced frame". The ground motion records are scaled by SEISMOMATCH for a site in Los Angeles with a stiff soil profile of site class-D. These time histories are scaled to make their spectrum compatible with site response spectrum.

The inter story drift ratio (IDR), displacement of a storey with respect to adjacent storey divided by the storey height, plays an important role in seismic analysis of buildings. Because the structural components and story ductility demands have the reasonable correlation with inter story drifts. The inter story drift profiles at DBE and MCE level is obtained from nonlinear time history analyses of the structure are presented in Figure 4.3 and Figure 4.4.

The inter storey drift ratio (IDR) of multistorey X-braced frame gains a maximum value of 1.3% on 7th storey level for the ground motion named LA-13 on DBE level, and a value of 2.34% on 2nd storey level for the ground motion named LA-6 on MCE level. The horizontal displacement of roof corresponding to that ground motion whose result is most severe is shown in Table 4.5.

Table 4.5 Roof displacement of mid-rise multistorey X-braced frame

Seismic hazard level	DBE Level	MCE Level
Roof displacement (ft)	0.77	1.21

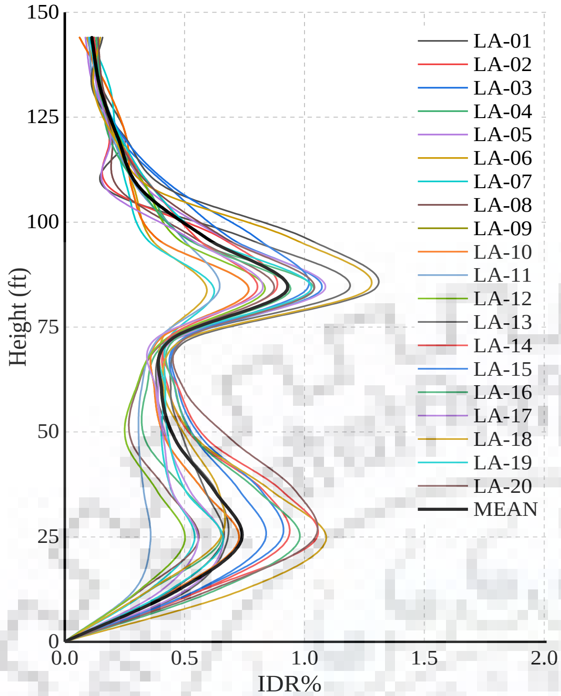


Figure 4.3 IDR profile of mid-rise multistorey X-braced frame in DBE case

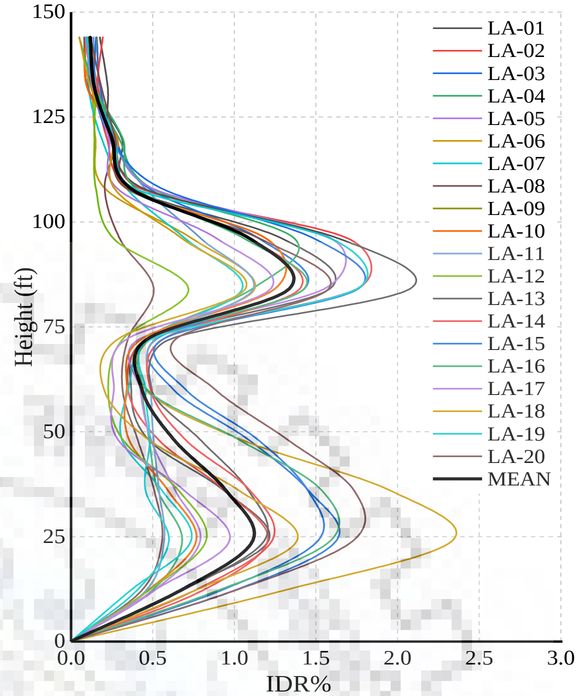


Figure 4.4 IDR profile of mid-rise multistorey X-braced frame in MCE case

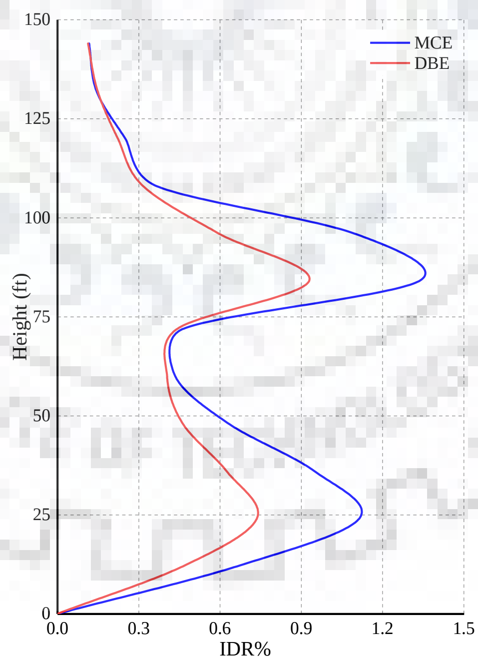


Figure 4.5 Mean IDR profile comparison between DBE and MCE case for mid-rise multistorey X-braced frame

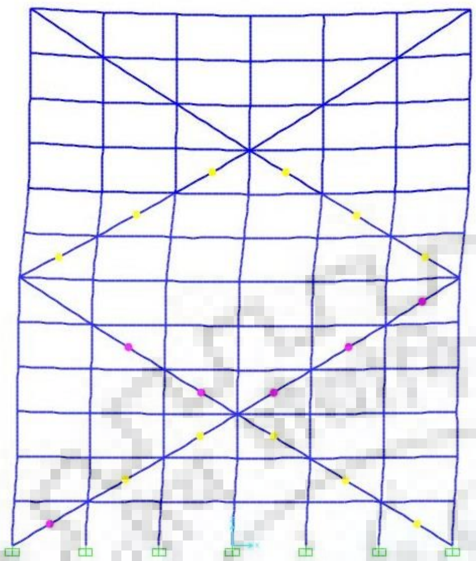


Figure 4.6 Plastic hinge formation and failure mechanism of mid-rise multistorey X-braced frame in DBE case

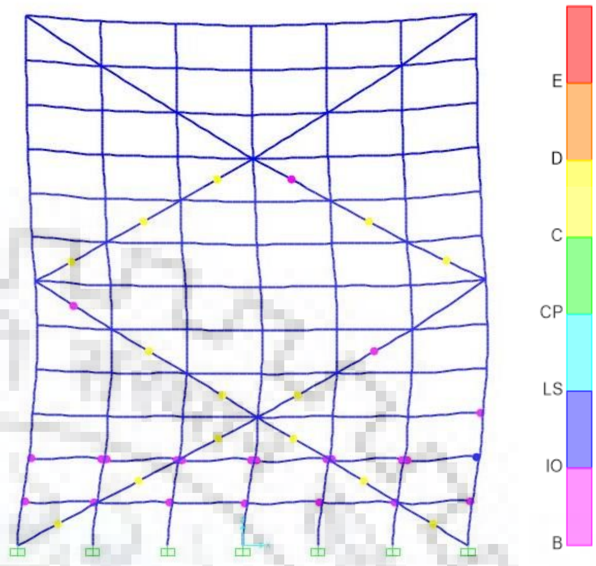


Figure 4.7 Plastic hinge formation and failure mechanism of mid-rise multistorey X-braced frame in MCE case for ground motion LA-06

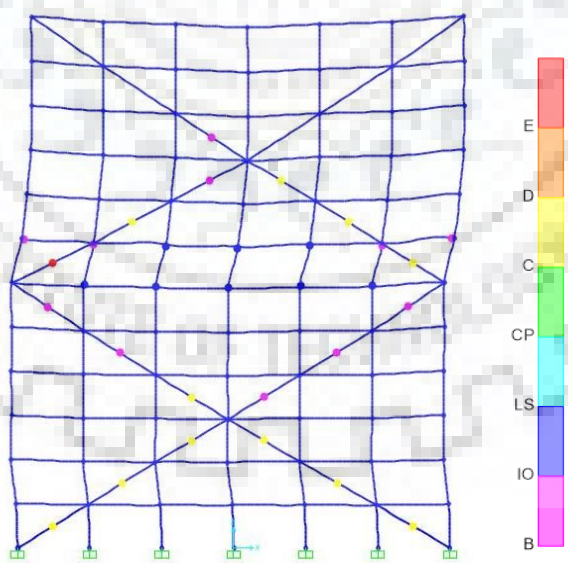


Figure 4.8 Plastic hinge formation and failure mechanism of mid-rise multistorey X-braced frame in MCE case-for ground motion LA-13

By observing the hinge mechanism in mutistorey X-braced frame as shown in Figure 4.8 for the ground motion LA-13 it is concluded out that hinge is forming firstly in braces then in beams and finally in columns at 7th storey level. The columns at 7th storey level are named as C_1^7, C_2^7, C_3^7, C_4^7, C_5^7, C_6^7, C_7^7 and the variation of forces in columns is defined in Table 4.6.

Table 4.6 Variation of column forces for mid-rise multistorey X-braced frame

Columns	Column axial force (kips)	
	Response spectrum method	Time history analysis
C_1^7	217.95	291.62
C_2^7	228.05	314.14
C_3^7	241.94	354.68
C_4^7	232.05	320.64
C_5^7	241.94	416.45
C_6^7	228.05	317.10
C_7^7	217.95	298.05

4.2 Mid-rise stack braced frame

4.2.1 Plan details and configuration of braces

Figure 4.9 Plan of mid-rise stack braced frame

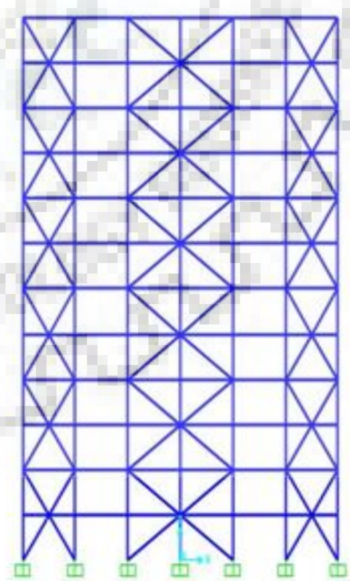


Figure 4.10 Elevation of mid-rise stack braced frame

The columns are designed with I-section, having a uniform height of 12 ft and each series of columns is separated at distance of 20 ft. Beams and columns are of wide flanged I-sections. Hollow steel sections (HSS) which are circular in shape are designed as brace members at angle of 50.2 degree and 30.96 degree for exterior and interior panels respectively. Braced frames are located at only periphery of building. The plan layout and configuration of braces in the building are shown in Figure 4.9 and Figure 4.10 respectively

Section size of all members assemble in mid-rise stack braced frame are tabulated as:

Table 4.7 Section sizes of brace members of mid-rise stack braced frame

Members	Specification	Sections	
		End panels	Mid panels
Brace members	Storey (1-6)	HSS6.625×0.5	HSS8.625×0.5
	Storey (6-12)	HSS5.500×0.5	HSS8.625×0.5

Table 4.8 Section sizes of beam and column members of mid-rise stack braced frame

Members	Specification	Sections
Beam members	Storey (1-6)	W18×46
	Storey (6-12)	W18×46
Column members	Storey (1-6)	W14×665
	Storey (6-12)	W14×311

4.2.2 Modal Analysis

The results of structure analysed by modal analysis are tabulated as:

Table 4.9 Results of modal analysis of mid-rise stack braced frame

Mode	Time period (sec)	Mass participation factor
1	0.74	0.76
2	0.25	0.13

4.2.3 Response spectrum analysis

The base shear determined by analysing the building by Response Spectrum method is tabulated as:

Table 4.10 Base shear of mid-rise stack braced frame

	Base shear (kips)	
	Response spectrum analysis (V_B)	Equivalent static method (V_B')
X-direction	391.74	1179.11
Z-direction	3909.17	3909.17

When the base shear determined by response spectrum (V_B) is less than the base shear determined by equivalent static method (V_B'), the storey shear forces and base reaction is multiply by a multiplying factor, i.e., V_B'/ V_B Thus base shear is corrected by a factor of 3.

4.2.4 Nonlinear time history analysis

The inter story drift profiles at DBE and MCE level is obtained from nonlinear time history analyses of the structure are presented in Figure 4.11 and Figure 4.12.

The inter storey drift ratio (IDR) of stack braced frame gains a maximum value of 0.78% at 3rd storey level for the ground motion named LA-14 on DBE level, and a value of 1.66% at 3rd storey level for the ground motion named LA-03 on MCE level. From Figure 4.13 it is clear that due to more concentration of inelastic demand at lower storey IDR is having maximum value as compared with stories at higher level.

The horizontal displacement of roof corresponding to that ground motion whose result is most severe is shown in Table 4.11

Table 4.11: Roof displacement of mid-rise stack braced frame

Seismic hazard level	DBE level	MCE level
Roof displacement (ft)	0.62	1.04

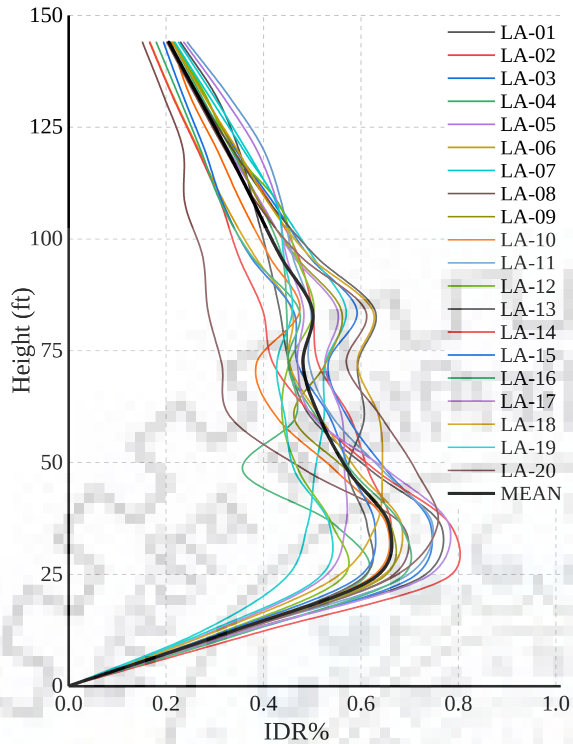


Figure 4.11 IDR profile of mid-rise stack braced frame in DBE case

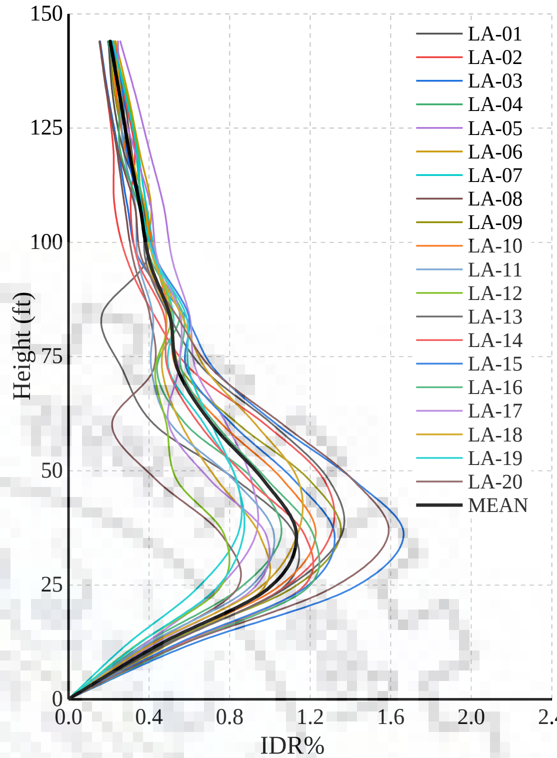


Figure 4.12 IDR profile of mid-rise stack braced frame in MCE case

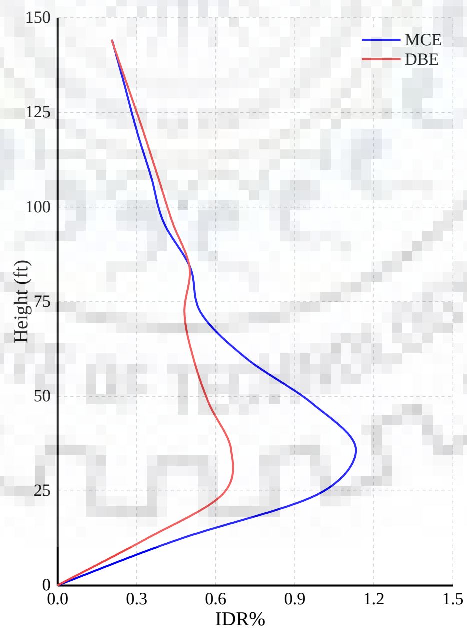


Figure 4.13 Mean IDR profile comparison between DBE and MCE Case for mid-rise stack braced frame

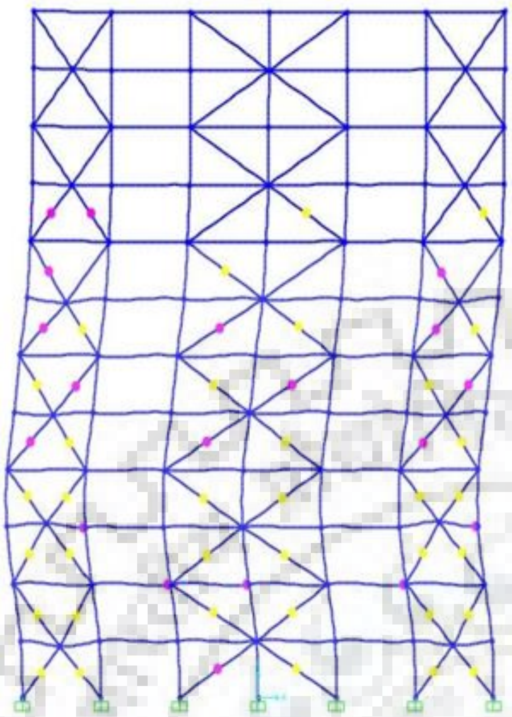


Figure 4.14 Plastic Hinge Formation and Failure Mechanism of mid-rise stack braced frame in DBE case

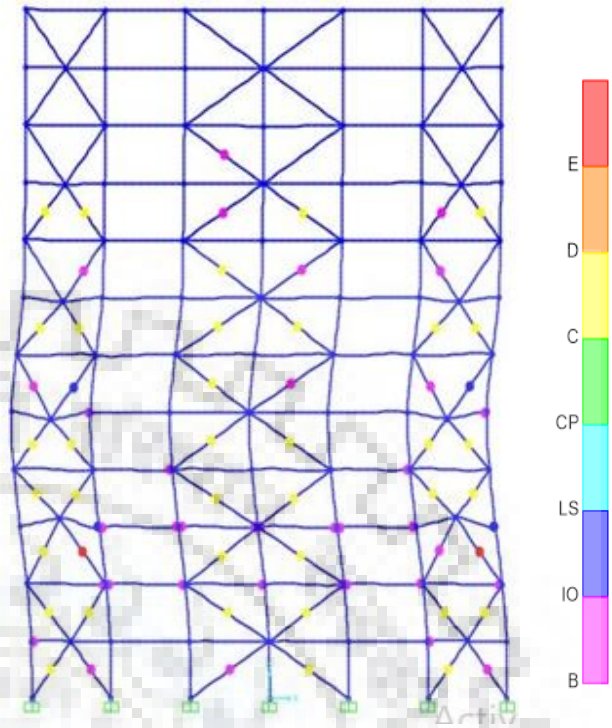


Figure 4.15 Plastic Hinge Formation and Failure Mechanism of mid-rise stack braced frame in MCE case

4.3 Comparison between mid-rise multistorey X-braced frame and stack braced frame

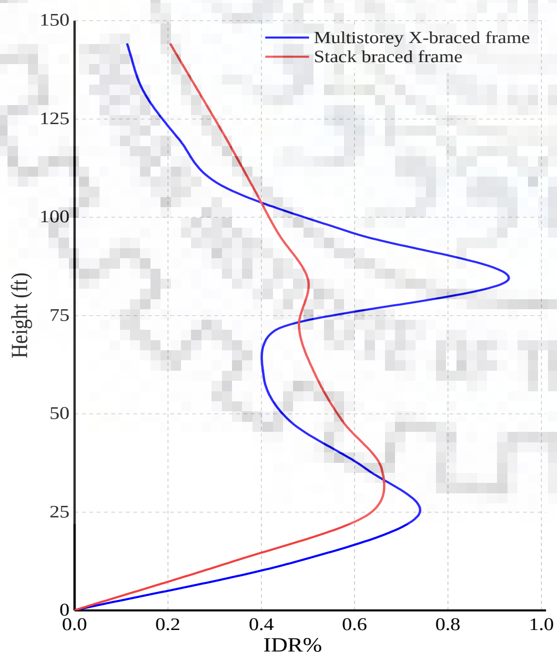


Figure 4.16 Comparison between mid-rise braced frames on DBE level

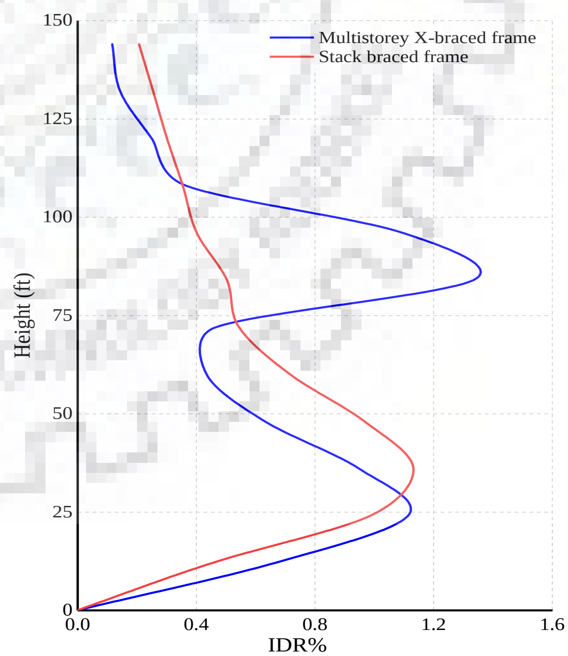


Figure 4.17 Comparison between mid-rise braced frames on MCE level

The columns located at ground storey are named C_1 to C_7 from left to right, the axial force, shear force and bending moment in these columns at the foundation level are defined in Table 4.12 and Table 4.13 for DBE and MCE cases.

Table 4.12 Forces at foundation level for mid-rise structure at DBE level

	Multistorey X-braced frame			Stack braced frame		
Ground motion	LA-14			LA-14		
Columns	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)
C_1	890.09	59.84	628.00	2048.71	115.04	1649.16
C_2	646.47	45.49	425.03	2140.87	107.66	1633.90
C_3	588.40	44.27	421.49	2221.32	117.69	1675.02
C_4	565.80	43.89	421.46	590.04	114.28	1660.32
C_5	593.02	43.90	421.65	2307.42	112.28	1650.94
C_6	604.15	45.25	425.43	1999.64	121.95	1690.35
C_7	1018.41	63.23	632.63	2220.29	104.93	1605.56

Table 4.13 Forces at foundation level for mid-rise structure at MCE level

	Multistorey X-braced frame			Stack braced frame		
Ground motion	LA-03			LA-03		
Columns	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)
C_1	1427.41	103.93	997.28	1996.50	124.48	2445.46
C_2	990.42	79.08	652.29	2508.81	106.69	2408.90
C_3	942.77	76.59	646.94	2288.77	131.09	2484.89
C_4	904.12	76.35	647.36	698.36	120.66	2463.98
C_5	897.43	76.57	648.01	2713.17	112.62	2450.63
C_6	972.81	77.99	652.58	1949.69	139.72	2509.71
C_7	1537.84	98.92	1004.02	2674.77	100.29	2355.53

CHAPTER 5 ANALYSIS OF HIGH-RISE BRACED FRAME

A stable and safe building having stories greater than 12, having enough strength to withstand all loads and impacts acting on it, comes under the category of high-rise structure. These structure can achieve allowed density of people without covering the entire area of site. Buildings with more height is more flexible with their overall design. For a variety of preferences these buildings can provide a variety in types of residential unit. High-rise buildings contribute to create a diverse and well-designed communities. High-rise buildings play an important role in assessment of the seismic vulnerability of megacities. Their assessment helps in determining the extreme demand in case of instability, collapse and need of evacuation. Therefore, high-rise buildings are intended to provide better performance not only for reliability against collapse but also for functionality.

5.1 High-rise multistorey X-braced frame

5.1.1 Plan details and configuration of braces

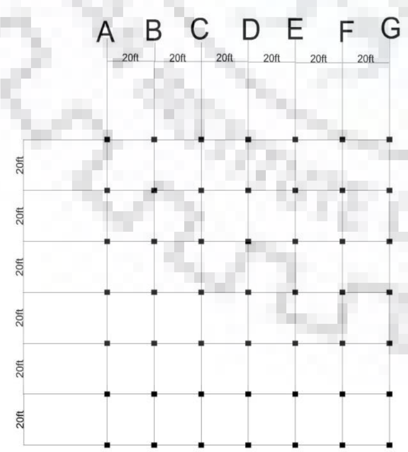


Figure 5.1 Plan of high-rise multistorey X-braced frame

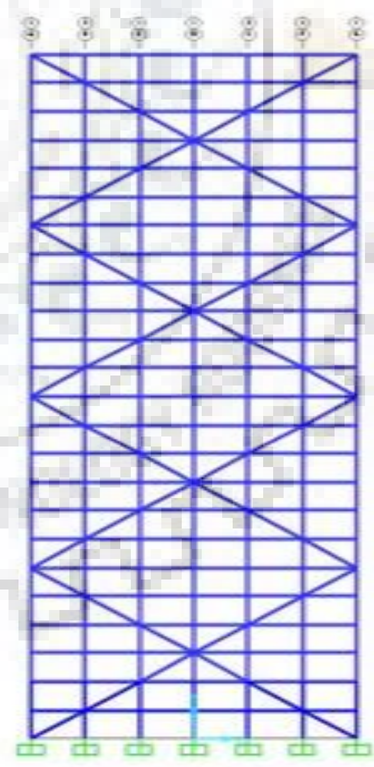


Figure 5.2 Elevation of high-rise multistorey X-braced frame

The columns are designed with I-section, having a uniform height of 12 ft and each series of column is separated at distance of 20 ft. Beams and columns are of wide flanged I-sections. Hollow steel sections (HSS) which are circular in shape are designed as brace members at angle of 50.2 degree and 30.96 degree for exterior and interior panels respectively. Braced frames are located at only periphery of building. The plan layout and configuration of braces in the building are shown in Figure 5.1 and Figure 5.2 respectively.

Section size of all members assemble in high-rise multistorey X-braced frame are tabulated as:

Table 5.1 Section sizes of brace members of high-rise multistorey X-braced frame

Specification	Sections	
	End panels	Other panels
Storey (1-12)	HSS16×0.75	HSS16×1.00
Storey (12-18)	HSS14×0.75	HSS14×1.00
Storey (18-24)	HSS12×0.60	HSS12×0.60

Table 5.2 Section sizes of beam members of high-rise multistorey X-braced frame

Specification	Sections
Storey (1-12)	Built-up section
Storey (13-18)	W36×800
Storey (19-24)	W36×529

Table 5.3 Section sizes of column members of high-rise multistorey X-braced frame

Specification	Sections	
	Exterior	Interior
Storey (1-6)	Built-up section	W14×550
Storey (7-12)	W36×880	W14×370
Storey (13-18)	W14×370	W14×342
Storey (19-24)	W14×500	W14×257

5.1.2 Modal analysis

The results of structure analysed by modal analysis are tabulated as:

Table 5.4 Results of modal analysis of high-rise multistorey X-braced frame

Mode	Time period (sec)	Mass participation factor
1	1.10	0.71
2	0.39	0.15
3	0.22	0.04

5.1.3 Response Spectrum Analysis

The base shear determined by analysing the building by Response Spectrum method are tabulated as:

Table 5.5 Base shear of high-rise multistorey X-braced frame

	Base Shear (kips)	
	Response spectrum analysis (V_B)	Equivalent static method (V_B')
X-direction	1249.48	2399.2
Z-direction	11428.77	11428.77

When the base shear determined by response spectrum (V_B) is less than the base shear determined by equivalent static method (V_B'), the storey shear forces and base reaction is multiply by a multiplying factor, i.e., V_B'/V_B Thus base shear is corrected by a factor of 1.92.

5.1.4 Nonlinear time history analysis

The response of a structure due to forces, displacements, velocities or accelerations that vary with time is determined under the nonlinear time history analysis (NLTHA). To study the behavior of the structure in dynamic condition, structure is analysed by NLTHA by integrating the solutions in each time interval. The analysis is performed in software SAP2000 by Hilber-Hughes-Taylor method, with α=0, β=0.25, Υ=0.5 and a damping of 2 percent. Ground Motions selected for analysis are same as selected in "Low-rise multistorey X-braced frame". The ground motion records are scaled by SEISMOMATCH for a site in Los Angeles with a stiff soil profile of site class-D. These time histories are scaled to make their spectrum compatible with site response spectrum. The inter story drift ratio (IDR), which is defined as the relative displacement between two consecutive story levels and normalized by the story height, plays an important role in seismic analysis of buildings, because of direct relation of IDR with the structural components and story ductility demands. The inter story drift profiles at DBE and MCE level is obtained from nonlinear time history analyses of the structure are presented in Figure 5.3 and Figure 5.4.

The inter storey drift ratio (IDR) of multistorey X-braced frame varies with a maximum value of 1% at 13th storey level for the ground motion named LA-18 on DBE level, and a maximum value of 1.3% at 19th storey level for the ground motion named LA-17 on MCE level. The horizontal displacement of roof corresponding to that ground motion, whose IDR is most severe is shown in Table 5.6.

Table 5.6: Roof displacement of building

Seismic hazard level	DBE level	MCE level
Roof displacement (ft)	1.92	2.25

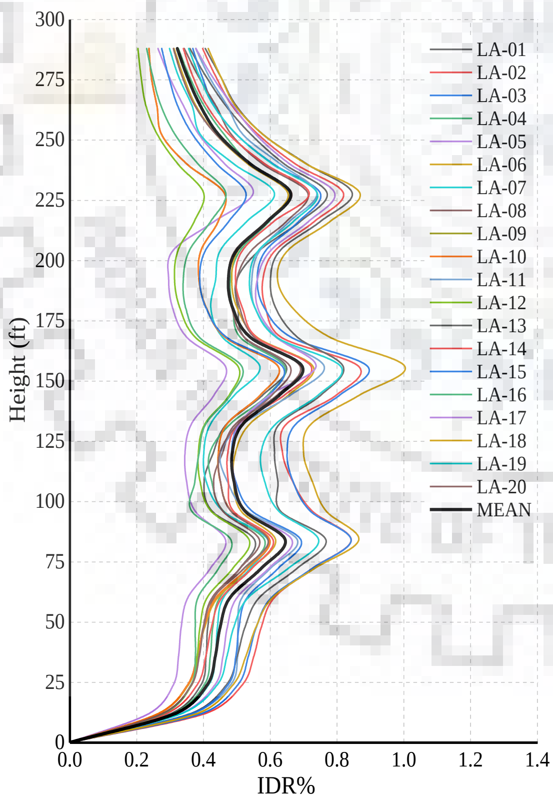


Figure 5.3 IDR profile of high-rise multistorey X-braced frame in DBE case

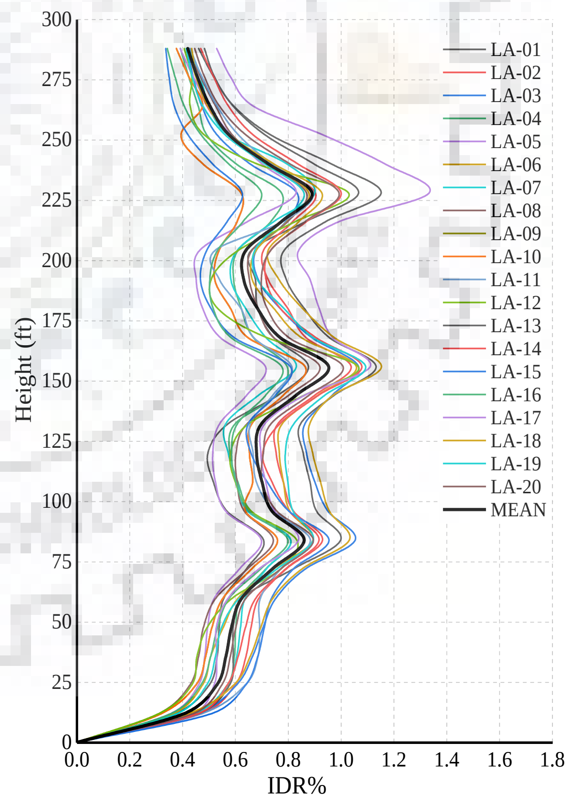


Figure 5.4 IDR profile of high-rise multistorey X-braced frame in MCE case

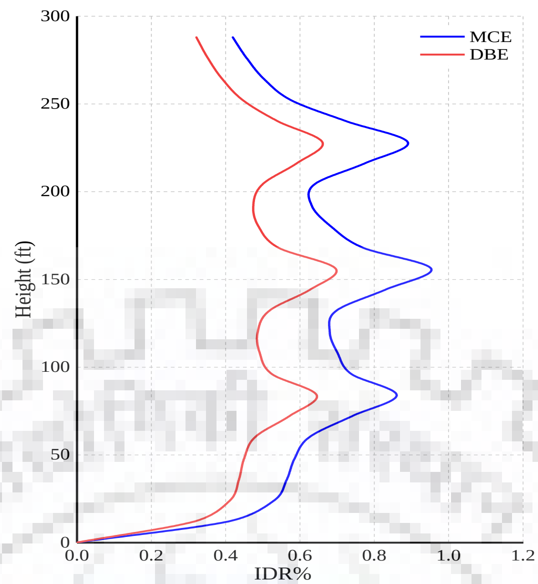


Figure 5.5 Mean IDR profile comparison between DBE and MCE case for high-rise multistorey X-braced frame

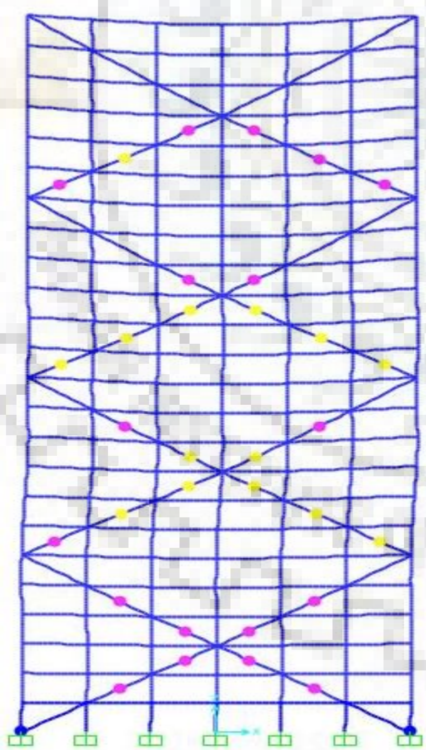


Figure 5.6 Plastic hinge formation and failure mechanism in high-rise multistorey X-braced frame in DBE case

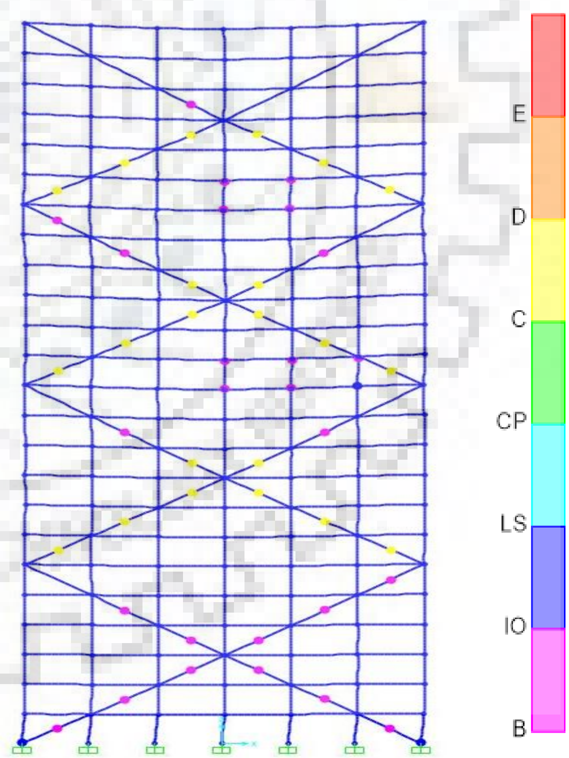


Figure 5.7 Plastic hinge formation and failure mechanism in high-rise multistorey X-braced frame in MCE case

By analysing the mechanism of plastic hinge formation it can be confer that braces are the first one to take load, showing successive cycles yielding in tension and buckling in compression depending on the direction of earthquake, and finally accommodates the hinge formation at mid length of brace member. As the seismic level increases columns at 13th and 19th storey accommodates hinges, due to accumulation of higher values of moments in the members at ending of X-bracing. Therefore, IDR profiles are having a maximum value at level of 13th and 19th storey on DBE and MCE level respectively.

The columns are named as C_1^{13}, C_2^{13}, C_3^{13}, C_4^{13}, C_5^{13}, C_6^{13}, C_7^{13} for 13th storey level and C_1^{17}, C_2^{19}, C_3^{19}, C_4^{19}, C_5^{19}, C_6^{19}, C_7^{19} for 19th storey level from left to right side. The forces in columns at 13th and 19th storey for ground motion LA-17 for MCE case are tabulated as:

Table 5.7 Variation of column forces for high-rise multistorey X-braced frame

Column axial force (kips)					
13th storey columns			19th storey columns		
Columns	Response spectrum method	Time history analysis	Columns	Response spectrum method	Time history analysis
C_1^{13}	1271.75	3650.58	C_1^{19}	429.62	861.40
C_2^{13}	730.32	1607.54	C_2^{19}	304.65	693.12
C_3^{13}	666.55	1075.63	C_3^{19}	305.99	590.77
C_4^{13}	614.83	628.97	C_4^{19}	282.98	317.45
C_5^{13}	666.55	1070.13	C_5^{19}	305.99	584.16
C_6^{13}	730.32	1699.96	C_6^{19}	304.65	692.91
C_7^{13}	1271.75	3991.38	C_7^{19}	429.62	925.73

5.2 High-rise stack braced frame

5.2.1 Plan details and configuration of braces

The columns are designed with I-section, having a uniform height of 12 ft and each series of columns is separated at distance of 20 ft. Beams and columns are of wide flanged I-sections. Hollow steel sections (HSS) which are circular in shape are designed as brace members at angle of 50.2 degree and 30.96 degree for exterior and interior panels respectively. Braced frames are located at only periphery of building. The plan layout and configuration of braces in the building are shown in Figure 5.8 and Figure 5.9 respectively.

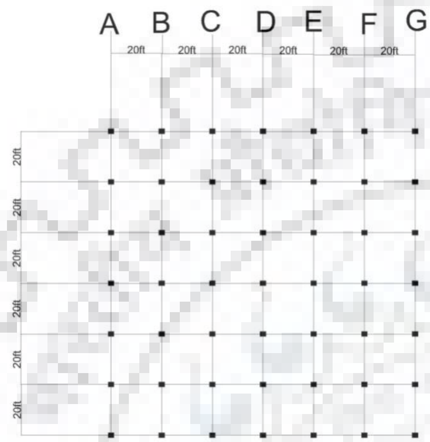


Figure 5.8 Plan of high-rise stack braced frame

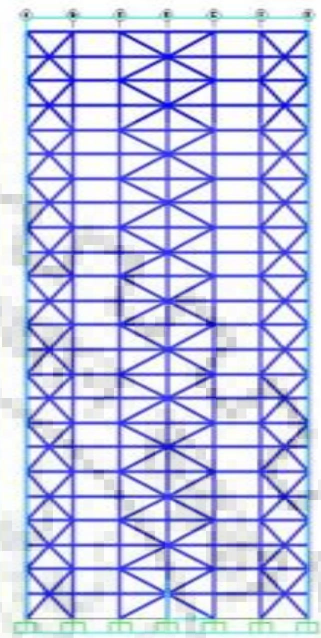


Figure 5.9 Elevation of high-rise stack braced frame

Section size of all members assemble in high-rise stack braced frame are tabulated as:

Table 5.8 Section sizes of brace members of high-rise stack braced frame

Specification	Sections	
	End panels	Other panels
Storey (1-6)	HSS8.625×0.625	HSS14×0.800
Storey (6-12)	HSS7×0.500	HSS14×0.750
Storey (12-18)	HSS6.875×0.500	HSS12×0.600
Storey (18-24)	HSS5×0.375	HSS10.750×0.500

Table 5.9 Section sizes of beam members of high-rise stack braced frame

Specification	Sections
Storey (1-12)	W36×800
Storey (13-18)	W36×441
Storey (19-24)	W36×361

Table 5.10 Section sizes of column members of high-rise stack braced frame

Specification	Sections	
	Exterior	Interior
Storey (1-6)	Built-up section	W14×550
Storey (7-12)	W36×880	W14×370
Storey (13-18)	W14×370	W14×342
Storey (19-24)	W14×500	W14×257

5.2.2 Modal analysis

The results of structure analysed by modal analysis are tabulated as:

Table 5.11 Results of modal analysis of high-rise stack braced frame

Mode	Time period (sec)	Mass participation factor	
		X-direction	Z-direction
1	0.990	0.657	0
2	0.370	0.178	3.40E-20
3	0.218	5.58E-17	0.642
4	0.213	0.058	5.09E-18
5	0.156	0.017	5.09E-17

5.2.3 Response spectrum analysis

The base shear determined by analysing the building by response spectrum method is shown in table below:

Table 5.12 Base shear of high-rise stack braced frame

	Base shear (kips)	
	Response spectrum analysis (V_B)	Equivalent static method (V_B')
X-direction	1085.513	2399.2
Z-direction	10007.557	10007.557

When the base shear determined by response spectrum (V_B) is less than the base shear determined by equivalent static method (V_B'), the storey shear forces and base reaction is multiply by a multiplying factor, i.e., V_B'/ V_B Thus base shear is corrected by a factor of 2.21.

5.2.4 Nonlinear time history analysis

The inter story drift profiles at DBE and MCE level is obtained from nonlinear time history analyses of the structure are presented in Figure 5.10 and Figure 5.11.

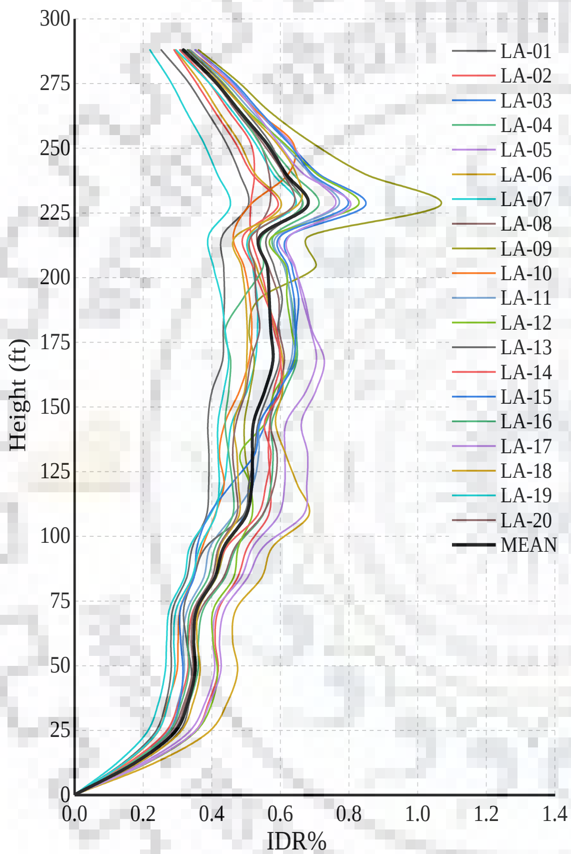


Figure 5.10 IDR profile in high-rise stack braced frame in DBE case

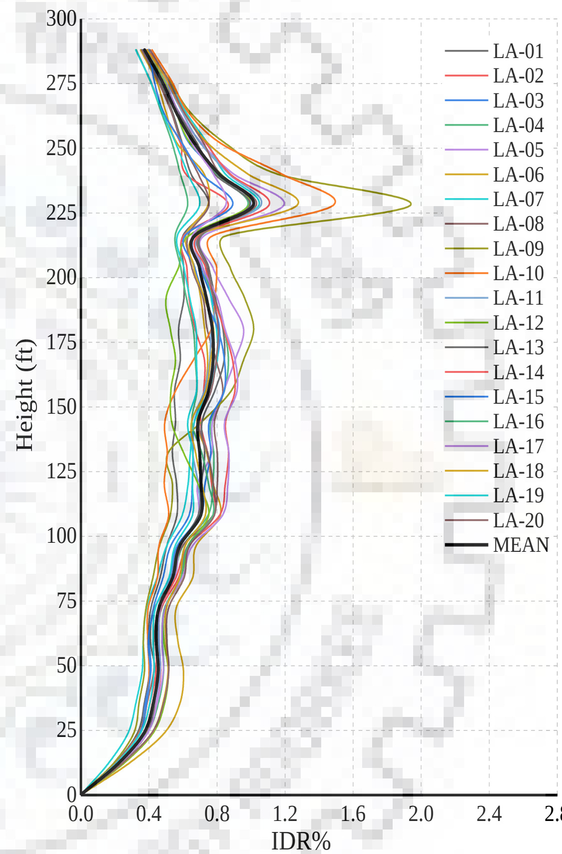


Figure 5.11 IDR profile in high-rise stack braced frame in MCE case

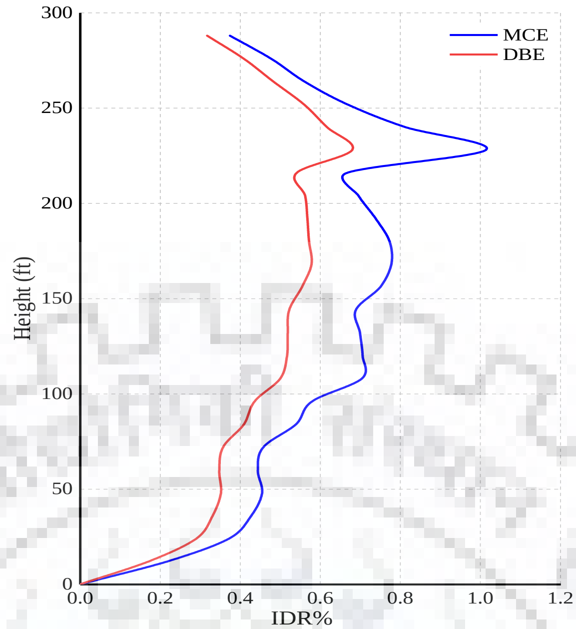


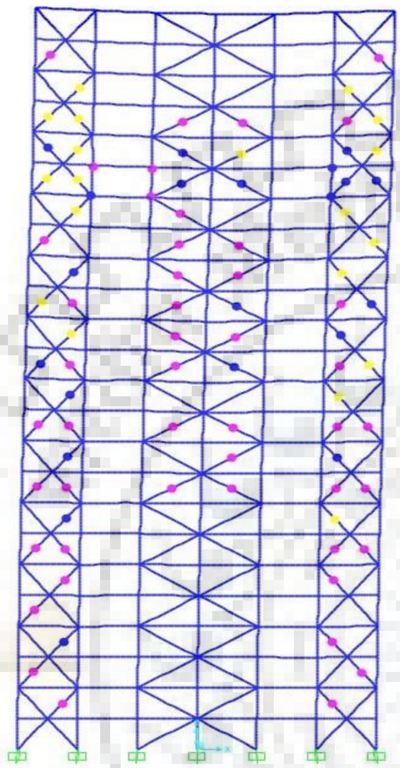
Figure 5.12 Mean IDR profile comparison between DBE and MCE case for high-rise stack braced frame

The inter storey drift ratio (IDR) of stack braced frame varies with a maximum value of 1.07% at 19th storey level for the ground motion named LA-9 on DBE level, and a maximum value of 1.93% at 19th storey level for the ground motion named LA-9 on MCE level. The horizontal displacement of roof corresponding to that ground motion whose result is most severe is shown in Table 5.13

Table 5.13: Roof displacement of high-rise stack braced frame

Seismic hazard level	DBE level	MCE level
Roof displacement (ft)	1.46	1.94

By analysing the mechanism of plastic hinge formation it can be confer that braces are the first one to take load, showing successive cycles yielding in tension and buckling in compression depending on direction in which building displaces, and finally accommodates the hinge formation at mid length of brace member. Due to the effect of higher modes stresses accumulate at storey level of 19th columns accommodate hinge formation. As the level of seismicity increases hinges are extended to form in columns of other stories also but on MCE level a drift at 19^{th} storey can be seen. Therefore, IDR profiles are having a maximum value at level of 19^{th} storey on both DBE and MCE level.



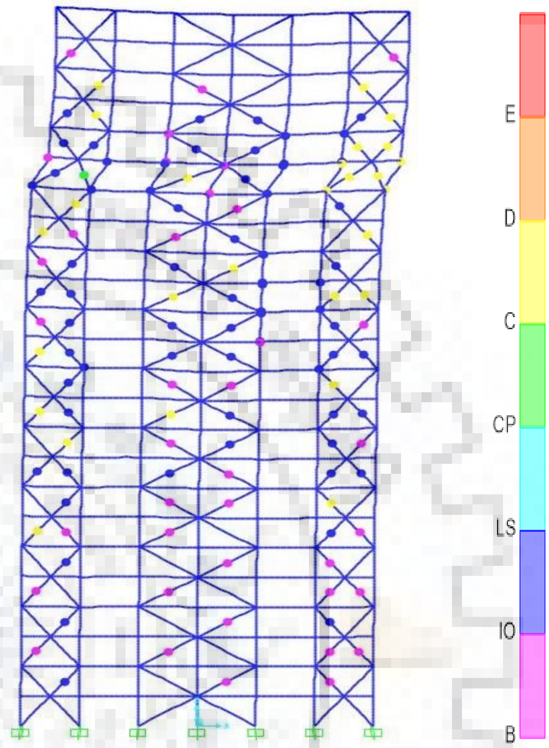


Figure 5.13 Plastic hinge formation and failure mechanism in high-rise stack braced frame in DBE case

Figure 5.14 Plastic hinge formation and failure mechanism in high-rise stack braced frame in MCE case

The columns at 19^{th} storey are named as C_1^{19}, C_2^{19}, C_3^{19}, C_4^{19}, C_5^{19}, C_6^{19}, C_7^{19}, from left to right, the forces in columns corresponding to ground motion LA-09 for DBE and MCE case are tabulated in Table 5.14.

Table 5.14 Variation of column forces for high-rise stack braced frame

Columns	Column axial force (kips)		
	Response spectrum method	Time history analysis	
		DBE	MCE
C_1^{19}	441.01	876.61	1033.93
C_2^{19}	487.11	868.36	918.06
C_3^{19}	219.13	330.24	389.34
C_4^{19}	342.24	378.83	446.82
C_5^{19}	219.13	365.33	462.12
C_6^{19}	487.11	904.16	1031.82
C_7^{19}	441.01	932.29	1135.80

5.3 Comparison between high-rise multistorey X-braced frame and stack braced frame

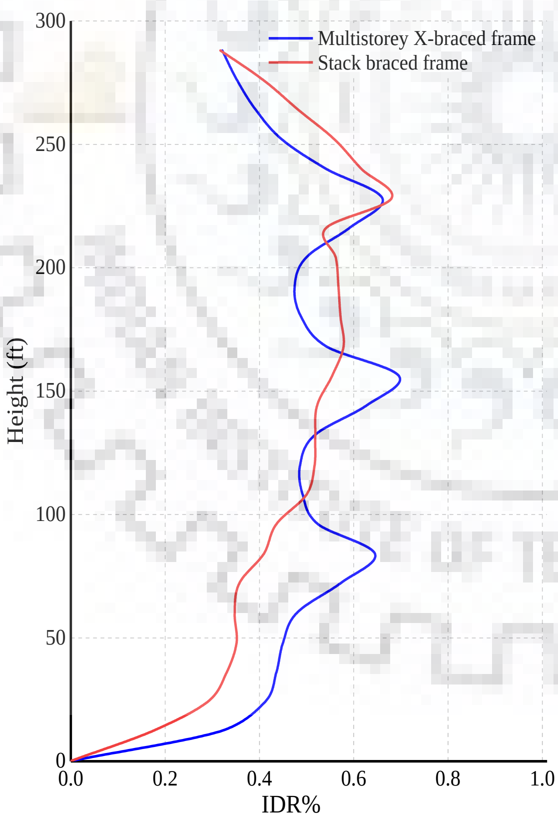


Figure 5.15 Comparison of high-rise braced frames on DBE level

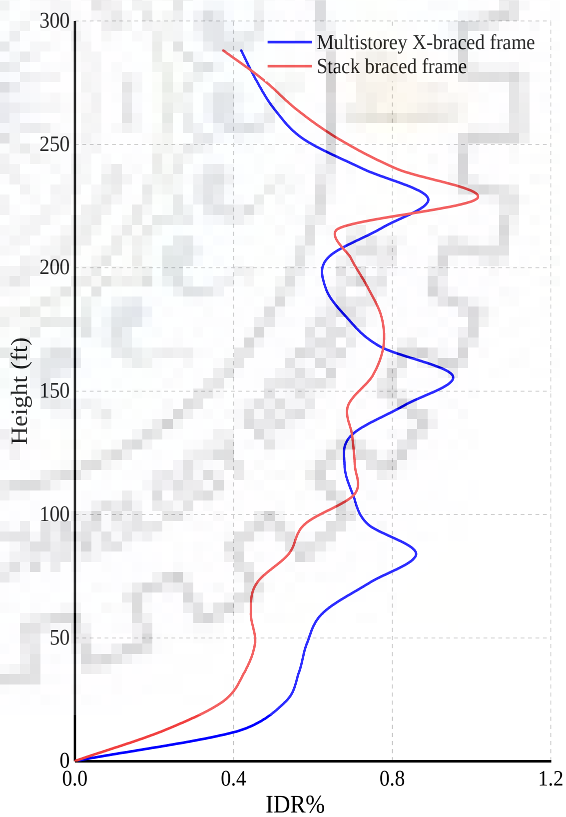


Figure 5.16 Comparison of high-rise braced frames on MCE level

Forces distributes more uniformly in case of stack bracing as compared with multi-storey X bracing which can be confirmed by pattern of IDR profiles. At lower levels of structure multi-storey X bracing is showing higher values of inter storey drift as compared with stack bracing, while at upper levels of structure stack bracing is showing higher values of inter storey drift as compared with multistorey X braced frame. But due to more uniform pattern of IDR profiles stack bracing is preferred in designing.

The columns located at ground storey are named as C_1, C_2, C_3, C_4, C_5, C_6, C_7 from left to right, the axial force, shear force and bending moment in these columns at the foundation level are defined in Table 5.15 and Table 5.16 for DBE and MCE cases. For multistorey X-braced frame axial force, shear force and moments are high for columns at ends than that of other columns, whereas in stack braced frame column forces and moments are of higher and lower values for alternate columns.

Table 5.15 Forces at foundation level for high-rise structure at DBE level

	Multistorey X-braced frame			Stack braced frame		
Ground motion	LA-09			LA-09		
Columns	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)
C_1	8980.25	1485.11	16454.03	8424.54	711.5	10316.30
C_2	2086.72	288.60	1813.70	606.02	40.81	240.22
C_3	1592.38	278.39	1774.93	6058.50	1062.09	13690.78
C_4	1184.50	281.14	1781.90	190.35	18.17	106.69
C_5	1528.38	279.77	1773.72	5794.72	1080.32	13714.02
C_6	1887.56	290.66	1807.74	614.65	40.60	237.22
C_7	8464.80	1516.42	16487.45	7961.42	732.40	10343.95

Table 5.16 Forces at foundation level for high-rise structure at MCE level

	Multistorey X-braced frame			Stack braced frame		
Ground motion	LA-09			LA-09		
Columns	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)	Axial Force (kips)	Shear force (kips)	Moment (kips-ft)
C_1	13470.38	2227.67	24681.05	9514.57	888.10	13424.61
C_2	3130.08	432.90	2720.55	626.35	54.00	316.42
C_3	2388.57	417.59	2662.40	6807.31	1348.05	17808.26
C_4	1776.75	421.71	2672.85	197.58	23.93	139.93
C_5	2292.57	419.65	2660.58	6589.63	1379.78	17849.38
C_6	2831.35	435.99	2711.61	677.06	53.39	310.90
C_7	12697.21	2274.64	24731.17	9511.26	935.21	13480.65

CHAPTER 6 CONCLUSIONS

In this thesis two types of bracing configuration is compared for low-rise, mid-rise, and high-rise braced frames. All structures are analysed by linear analysis and nonlinear dynamic analysis to compare the performance of both multistorey X-braced frame and stacked braced frame. In multistorey X-braced frame, braces at lower half level within each X-configuration are experiencing more damage than that of the braces at upper half level. Therefore, the inter storey drift ratio (IDR) profile is having zigzag pattern.

For low-rise multi storey X-braced frame, stresses are concentrating at lower stories and resulting into higher value of inter storey drift in those stories, whereas in low-rise stack braced frame inter-storey drift is showing a smooth and uniform pattern. If we compare the result, it can be easily concluded out that stack bracing is resulting less storey drift than that of multistorey X-braced frame. Because the structural components and storey ductility is directly related with IDR-profile, the best performance of a structure can be resulted out corresponding to stack braced frame. In multistorey X-braced frame the axial force, shear force and bending moment for the columns at foundation level, are almost uniform in nature, which is not in stack bracing. The large variation in column axial forces affects the designing of foundation in case of stack bracing.

For mid-rise braced frame, both type of bracing is showing same pattern of deformations at lower level. On other hand, for stories at mid-level of the building, multistorey X-bracing is following a zig-zag pattern, while stack bracing displays almost smooth pattern. But for the stories located at the upper level of structure, both bracing are producing smooth pattern of damage plus stack bracing is showing more inter-storey drift as compared with mutistorey X-bracing. Hence stack bracing is preferred over multistorey X-bracing, because it distributes the forces to other stories rather than concentrating at the same level. For multistorey X-braced frame the column forces and moments are higher for columns at extremity than that of other columns, whereas for stack braced frame forces and moments are almost uniform.

For high-rise braced frame the IDR pattern is quite similar with mid-rise braced frame. For multistorey X-braced frame axial force, shear force and moments are high for columns at ends than that of other columns, whereas in stack braced frame column forces and moments are of higher and lower values for alternate columns.

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