SEISMIC ANALYSIS OF HIGH RISE STEEL FRAME WITH AND WITHOUT BRACING

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SUMMARY:
Presently, Indian standard codal provisions for finding out the approximate time period of steel structure is not considering the type of the bracing system. Bracing element in structural system plays vital role in structural behavior during earthquake. The pattern of the bracing can extensively modify the global seismic behavior of the framed steel building. In this paper the linear time history analysis is carried out on high rise steel building with different pattern of bracing system for Northridge earthquake. Natural frequencies, fundamental time period, mode shapes, inter story drift and base shear are calculated with different pattern of bracing system. Further optimization study was carried out to decide the suitable type of the bracing pattern by keeping the inter-story drift, total lateral displacement and stress level within permissible limit. Aim of study was to compare the results of seismic analysis of high rise steel building with different pattern of bracing system and without bracing system.

Keywords: Time history analysis, high rise steel building, bracing pattern

1. INTRODUCTION

Seismic Analysis is a subset of structural analysis and is the calculation of the response of a structure to earthquakes. Nowadays High Rise Steel frame building is well establishing in metro cities. For construction of high rise building bracing are constructed for stiffness and lateral load resistance purpose. Steel frame usually refers to a building technique with a “skeleton frame” of vertical steel columns and horizontal I-beams, constructed in a rectangular grid to support the floors, roof and walls of a building which are all attached to the frame. The development of this technique made the construction of the skyscraper possible. Bracings are strong in compression. Bracing with their surrounding frames has to be considered for increase in lateral load resisting capacity of structure. When bracings are placed in Steel frame it behaves as diagonal compression strut and transmits compression force to another joint. Variations in the column stiffness can influence the mode of failure and lateral stiffness of the bracing.

1.1. Recent Research Work

E.M. Hines and C.C. Jacob [2009] presented a paper on Eccentric braced frame system performance. The seismic performance of low-ductility steel systems designed for moderate seismic regions have generated new interest in the cost-effective design of ductile systems for such regions. Although eccentrically braced frames (EBFs) have a well-established reputation as high-ductility systems and have the potential to offer cost-effective solutions in moderate seismic regions, their system performance has not been widely discussed. Eccentrically Braced Frames (EBFs) are known for their attractive combination of high elastic stiffness and superior inelastic performance characteristics (AISC 2005). The University of California, Berkeley (UCB) under the direction of Professors Popov and Bertero conducted a test of two separate 0.3 scale shake table tests of Concentrically Braced Frame (CBF) and EBF dual systems (Uang and Bertero 1986, Whittaker et al. 1987, Whittaker et al. 1990). The design of shear links for the tower of the San Francisco-Oakland Bay Bridge East Bay self-anchored suspension span (McDaniel et al. 2003).

S.H. Chao and M.R. Bayat et.al [2008] studied on performance based plastic design of steel concentric braced frames for enhanced confidence level in China. Concentrically braced frames (CBFs) are generally considered less ductile seismic resistant structures than other systems due to the brace buckling or fracture when subjected to large cyclic displacements. This is attributed to simpler design and high efficiency of CBFs compared to other systems such as moment frames, especially after the 1994 Northridge Earthquake. However, recent analytical studies have shown that CBFs designed by conventional elastic design method suffered severe damage or even collapse. The three- and six-story Chevron type CBFs originally designed (Sabelli, 2000) as SCBF according to 1997 NEHRP design spectra (FEMA, 1997) and 1997 AISC Seismic Provisions (AISC, 1997) were used in this study.

R. Leon & R. DesRoches [2006] has done a research work on Behaviour of Braced Steel Frames with Innovative Bracing Schemes. Conventional bracing systems include typical diagonal and chevron bracing configurations, as well as innovative concepts such as strut-to-ground and zipper braced frames (Khatib et al. 1988, Bruneau et al. 1998). Seismic regulations and guidelines for the seismic design of CBFs can be found in the Structural Engineers Association of California (SEAOC) Recommended Lateral Force Requirements (SEAOC 1996), the International Building Code (IBC 2000), the NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings (BSSC 2000), and the AISC Seismic Provisions for Structural Steel Buildings (AISC 2002). Diagonal and chevron systems can provide large lateral strength and rigidity but do not provide great ductility as buckling of the diagonals leads to rapid loss of strength without much force redistribution (Goel 1992).

In chevron brace the unbalanced vertical forces that arise at the connections to the floor beams due to the unequal axial capacity of the braces in tension and compression. In order to prevent undesirable deterioration of lateral strength of the frame, the provisions require that the beam should possess adequate strength to resist this potentially significant post-buckling force redistribution. The adverse effect of the unbalanced vertical force at the beam-to-brace connections can be mitigated by adding zipper elements, as proposed by Khatib et al. (1988). If the compression brace in the first story buckles while all other braces remain elastic, a vertical unbalanced force is then applied at the middle span of the first story beam. The zipper elements mobilize the stiffness of all beams and remaining braces to resist this unbalance. The unbalanced force transmitted through the zipper elements increases the compression of the second story compression brace, eventually causing it to buckle.

P. Uriz and S.A. Mahin [2004] presented a paper on Seismic performance assessment of concentrically braced steel frames. The overall investigation includes systems that utilize conventional braces, buckling restrained braces and braces incorporating viscous damping devices. In the first part the same reliability framework as used to assess Special Moment Resisting Frame (SMRF) structures during the FEMA/SAC Steel Project was employed to assess the confidence with which Special Concentric Braced Frames (SCBF) and Buckling Restrained Braced Frames (BRBF) might achieve the seismic performance expected of new SMRF construction. In the second part, a test program to help improve modelling of SCBF systems is described, including the design of a nearly full-size, two-story SCBF test specimen. The confidence that a three story SCBF designed according to the 1997 NEHRP provisions is able to achieve the collapse prevention performance goal was less than 10% for all definitions capacity and a seismic hazard corresponding to a 2% probability of exceedance in 50 years. A similarly designed six-story BRBF was demonstrated to be much more reliable. The performance-based evaluation approach for characterizing and improving the performance of steel braced frames incorporating conventional bracing, buckling restrained braces, friction and hysteretic devices, and viscous dampers.

C.Y. Ho and G.G. Schierele [1990] published a journal paper Effect of configuration and lateral drift on High-rise space frames. Excessive lateral drift in high-rise frames can damage secondary systems, such as partitions walls; generate secondary column stress due to P-δ moments; and cause discomfort to building occupants under prolonged cyclical drift. Damage to secondary system can be controlled by reducing drift. The P-δ effect is most severe in moment resisting frames; the Uniform Building Code allows smaller seismic drift for moment resisting frames (0.3% story drift vs. 0.5 % for other systems). Design for wind or seismic forces are usually based on objectives to minimize lateral drift. To reduce lateral drift of high-rise buildings is an important design consideration in areas of high wind and/or seismic activity. The research presented here shows that selecting the most appropriate bracing system can substantially reduce drift with only minor cost differences. A sensitivity analysis, doubling cross- section areas of cap and belt truss braces, revealed only very minor drift reductions. The reductions ranged from a minimum of 1 % for the 20 story K-braced frame with cap truss, to a maximum of 7.6% for the same frame combining cap and belt trusses. Seismic forces tend to increase with the stiffness of a building. For example in the equivalent static formula for seismic base shear V=ZICW/R, the R factor is 12 for Special Moment Resisting Frames but varies between 4 and 8 for various bracing schemes. More research is needed to determine the effect of various framing configurations on lateral forces and drift in seismic situations.

From above study we can conclude that there is enough research on braced frame but mostly it is either experimental study or Finite element analysis of single bay two storey frame. Some macro model studies have been also done but limited to five to fifteen story 2D frame steel building. So there is scope to do Earthquake analysis on live project having more number of stories with 3D modeling (i.e. high rise framed building) for different kind of loading and to see the effect on both conditions i. e. with and without different bracing style. In this paper Earthquake analysis of a high rise moment resisting steel framed building is carried out for Northridge earthquake ground motion. The results are considered in terms of joint displacement, base shear reaction, modal period and checked against permissible values. The same steel frame building analyzed with different type of bracing patterns. The steel frame sections like columns, beams and bracings are optimized for various bracing pattern to reduce the cost by keeping the joint displacement and stress/capacity ratio within allowable limit.

2. STRUCTURAL MODELLING

For the analysis work, six models of high rise steel frame building (G+40) floors are made to know the realistic behavior of building during earthquake. The length of the building is 30m and width is 22m. Height of typical story is 3.5m. Column sizes changes first at 11storey and then at each 10 story. Building is symmetrical about X and Y-axis. Material concrete grade M20 is used, while steel Fe 250 (mild steel) is used. Modal damping 5% is considered. For consideration of diaphragm action Diaphragm is assigned at each floor. Analytical modeling that includes all components which influence the mass, strength and stiffness. The non-structural element and components that do not significantly influence the building behavior were not modeled. Beams and columns are modeled as frame element and joined node to nodes. The effect of soil structure interaction is ignored in analysis. The columns are assumed to be fixed at the ground level.

Linear dynamic analysis i.e. time history analysis is used as per guideline given in IS-1893 (Part1). Northridge Earthquake time history is used and Maximum acceleration is applied at the base of building.

2.1. Studied Structural Configuration

Following two types of structural configuration is studied.
1. G+40 Steel Framed structure without bracing (MRF)
2. G+40 Steel Framed structure with different bracing patterns such as Diagonal brace-A, X-brace, K-brace, Knee brace and Diagonal brace-B

3. DETAILS OF THE BUILDING PLAN, MEMBER SIZE AND MATERIALS

3.1 Plan

Plan of the steel building which is used for the study is shown in figure 3.1.

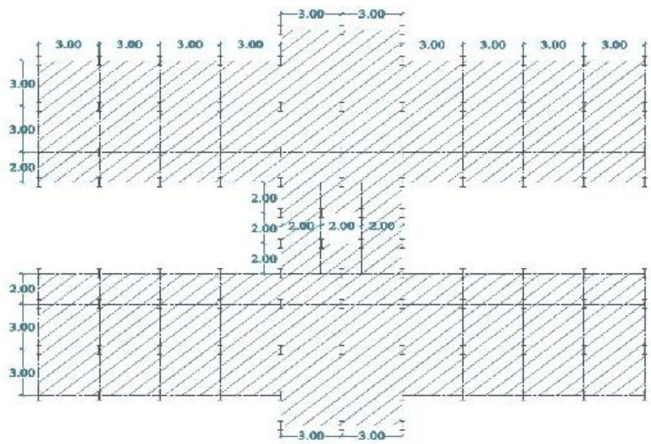


Figure 3.1: Plan of High rise steel framed building

3.2 Member size of the Beams, Columns and Bracing

Member size used for beams, columns and bracing are shown in table 3.1

Table3.1: Size of Beams, Columns and Bracings

Storey Level	Column Schedule		Beam Schedule		Bracing Schedule	
	Column No	Size	Beam No	Size	Bracing No	Size
G+10	C1	ISMB 600	B1	ISMB 550	BR1	ISA 200x150x15
11 to 20	C2	ISMB 500	B2	ISMB 450	BR2	ISA 200x100x15
21 to 30	C3	ISMB 400	B3	ISMB 350	BR3	ISA 150x150x15
31 to 40	C4	ISMB 300	B4	ISMB 250	BR4	ISA 150x115x15

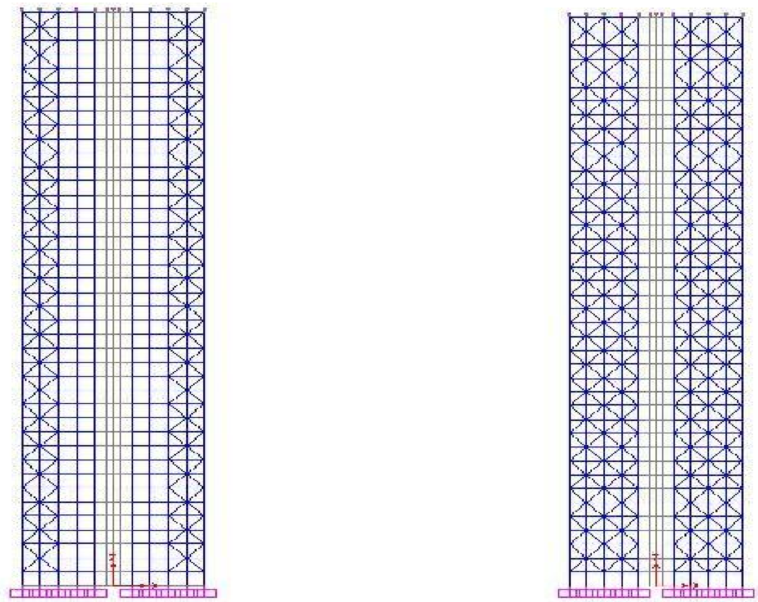
3.2 Material Properties used for analysis

Concrete- M 20, Density-2400 Kg/m^3, Young's Modulus E= 22360 N/mm^2, Shear Modulus 8000N/mm^2, Poisson's Ratio-0.2

Structural steel- Fe 250, Density-7850 Kg/m3, Young's Modulus E= $2.1x10^5 N/mm^2$, Shear Modulus 80000N/mm^2 Poisson's Ratio-0.3

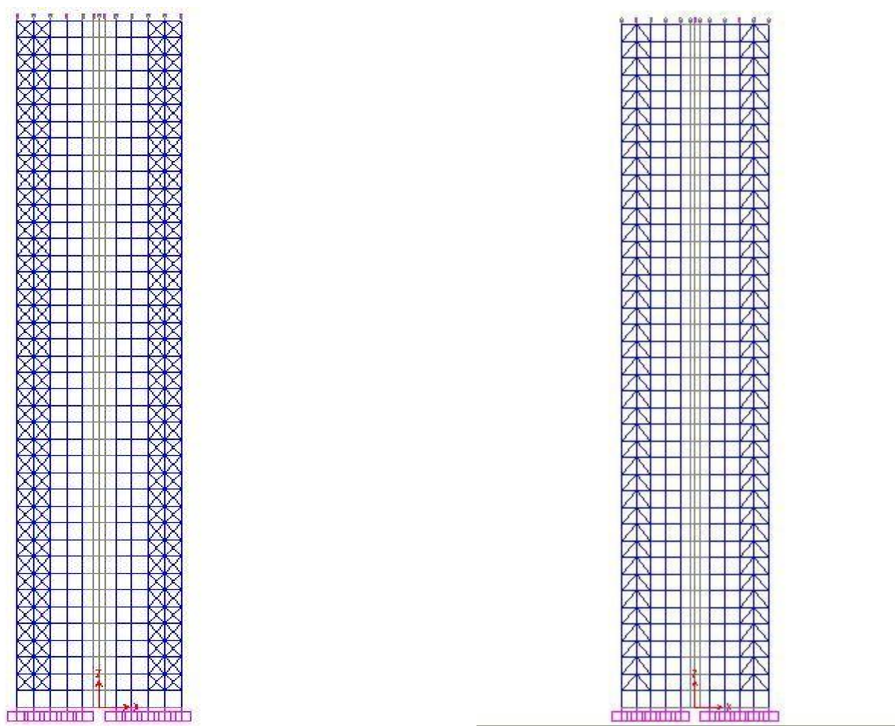
4. DIFFERENT TYPES OF BRACING PATTERNS USED IN THE STUDY

Different types of bracing pattern used in the study are shown in figure 4.1 to 4.4.



a) Diagonal Brace-A b) Diagonal Brace -B

Figure 4.1: Steel Framed Model of Building with a) Diagonal Brace-A and b) Diagonal Brace-B



a) X-Brace b) K- Brace

Figure 4.2: Steel Framed Model of Building with a) X-brace and b) K-brace

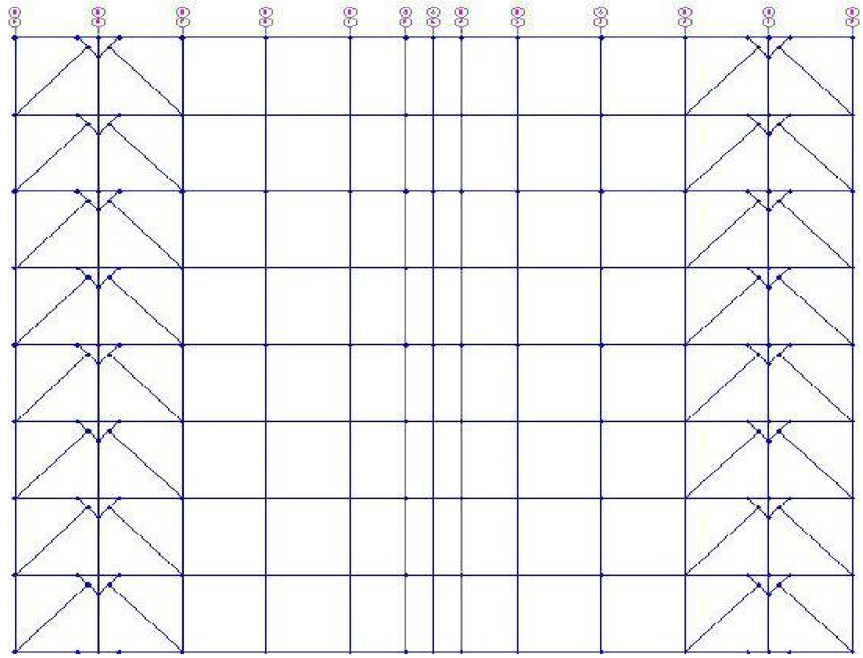


Figure 4.3: Steel Framed Model of Building with Knee brace

5. RESULTS

5.1Base Shear and Displacement

Results of base shear, storey lateral displacement in X and Y direction and roof level lateral displacement in X and Y direction are presented in the table no 5.1 to 5.4 and figure no 5.1 to 5.2

Table 5.1: Base shear in X-direction

Base Shear (KN)	For North Ridge Earthquake		
Type	Without Bracing	With Bracing	% Difference
Diagonal Brace A	7318.27	7998.44	8.50
X Brace	7318.27	8622.85	15.13
K Brace	7318.27	7994.75	8.46
Knee Brace	7318.27	8443.26	13.32
Diagonal Brace B	7318.27	10762.52	32.00

Table 5.2: Base shear in Y-direction

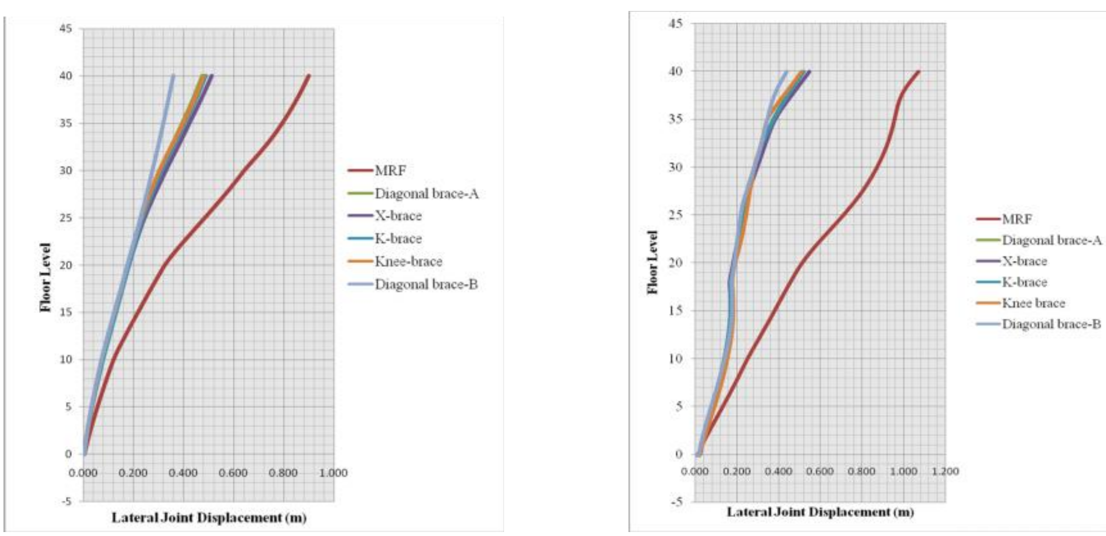
Base Shear (KN)	For North Ridge Earthquake		
Type	Without Bracing	With Bracing	% Difference
Diagonal Brace A	5035.03	5282.79	4.69
X Brace	5035.03	8115.77	37.96
K Brace	5035.03	5426.02	7.21
Knee Brace	5035.03	6097.91	17.43
Diagonal Brace B	5035.03	7271.40	30.76

Table 5.3: Storey displacement in X-direction

Floor level	Storey displacement in X-direction (m) for different types of bracings					
	Without Brace	Diagonal Brace-A	X Brace	K Brace	Knee Brace	Diagonal Brace-B
40th	0.900	0.475	0.513	0.489	0.481	0.360
35th	0.794	0.395	0.424	0.406	0.398	0.320
30th	0.643	0.305	0.330	0.313	0.303	0.276
25th	0.486	0.234	0.245	0.238	0.230	0.229
20th	0.325	0.180	0.183	0.181	0.175	0.176
15th	0.218	0.129	0.129	0.130	0.125	0.123
10th	0.121	0.078	0.077	0.078	0.072	0.072
5th	0.056	0.035	0.034	0.035	0.033	0.032
1st	0.013	0.008	0.008	0.008	0.008	0.008

Table 5.4: Storey displacement in Y-direction

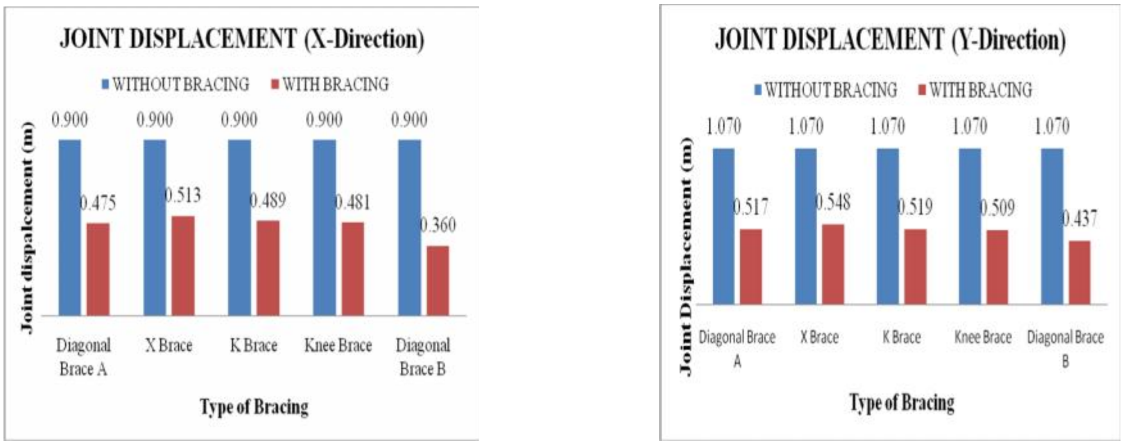
Floor level	Storey displacement in Y-direction (m) for different types of bracings					
	Without Brace	Diagonal Brace-A	X Brace	K Brace	Knee Brace	Diagonal Brace-B
40th	1.070	0.517	0.548	0.519	0.509	0.437
35th	0.956	0.373	0.383	0.374	0.346	0.345
30th	0.872	0.283	0.297	0.283	0.284	0.284
25th	0.710	0.225	0.229	0.224	0.244	0.217
20th	0.512	0.187	0.182	0.186	0.191	0.187
15th	0.380	0.169	0.169	0.169	0.183	0.177
10th	0.251	0.139	0.141	0.138	0.154	0.141
5th	0.132	0.101	0.081	0.081	0.093	0.076
1st	0.033	0.029	0.030	0.030	0.028	0.026



a) Storey Displacement in X-direction

b) Storey Displacement in Y -direction

Figure 5.1: Storey Displacement in a) X-direction and b) Y-direction



a) Roof Displacement in X-direction

b) Roof Displacement in Y-direction

Figure 5.2: Roof Displacement in a) X-direction and b) Y-direction

5.2 Time Period

Time period as per as per IS 1893 (Part 1):2002 clause No. 7.6.1, is equal to
$T_a = 0.085\ h^{0.75}$ for Steel frame building (here, h is the height of building in m)
$= 0.085 \times (143.5)^{0.75}$
$= 3.52$ sec

Time period from modal analysis is presented in table no 5.5

Table 5.5: Time Period from Modal Analysis

Type of Bracings	Modal Period For 1st Mode Shape		
	Without Bracing	With Bracing	% Difference
Diagonal Brace A	6.89	2.47	64.18
X Brace	6.89	2.50	63.76
K Brace	6.89	2.46	64.27
Knee Brace	6.89	2.53	63.34
Diagonal Brace B	6.89	2.43	64.65

5.3 Optimization of frame section

Frames are further analyzed for cost optimization by keeping the roof joint displacement and stress level in the members within the permissible limit. Results are presented in table no 5.6

Table 5.5: Column and Bracing dimensions before and after optimization

Column Section		Bracing Section	
Member before Optimization	Member after optimization	Member before Optimization	Member after optimization
ISMB 600	ISMB 350	ISA 200x150x15	ISA 200x150x12
ISMB 500	ISMB 300	ISA 200x100x15	ISA 150x115x12
ISMB 400	ISMB 250	ISA 150x150x15	ISA 125x95x12
ISMB 300	ISMB 225	ISA 150x115x15	ISA 100x75x10

6. CONCLUSION

The result of the present study shows that bracing element will have very important effect on structural behavior under earthquake effect. From the tables it shows that due to bracings in both direction base shear increases up to 38%. The displacements at roof level of the building with different bracing style is reduces from 43% to 60%. Modal time period is also reduced up to 65%. The diagonal brace-B shows highly effective and economical design of bracing style.

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