Sigma J Eng Nat Sci, Vol. 41, No. 6, pp. 1121−1131, December, 2023

Sigma Journal of Engineering and Natural Sciences
Web page info: https://sigma.yildiz.edu.tr
DOI: 10.14744/sigma.2023.00075

Research Article

Seismic performance evaluation of eccentrically braced steel frame buildings

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ARTICLE INFO

Article history
Received: 18 October 2021
Revised: 22 November 2022
Accepted: 25 February 2022

Keywords:
Earthquake Resistant Design; Eccentrically Braced Frame; Steel Buildings; Response Reduction Factor; Seismic Performance

ABSTRACT

Eccentrically braced framing (EBF) systems have been in use for the past few decades, however, there seems to be little consensus on the amount of ductility capacity available in the EBF system, among code committees of different countries. This is reflected in the variation in the response reduction factors specified for EBF systems in different national design codes. Indian design code specifies a response reduction factor of 5 for EBF systems. Whether this much reduction in the design earthquake forces gives intended performance for the buildings with EBF system, needs to be verified. In the present study, an attempt has been made to verify the adequacy of the response reduction factor given in the Indian code for EBF buildings using the procedure given in FEMA P695. Adequacy is checked for 5 storey archetypes of EBF system having built-up sections and Indian Standard rolled steel sections as link members. The building models have been subjected to Incremental Dynamic Analysis (IDA) using a suite of 22 ground motions given in FEMA P695 to obtain the collapse margin ratio. The obtained collapse margin ratio has been adjusted for spectral shape correction and then checked against the acceptable value of collapse margin ratio given in FEMA P695 to verify the adequacy of the Response reduction factor used in the design.

Cite this article as: Khan NA, Bilal A. Seismic performance evaluation of eccentrically braced steel frame buildings. Sigma J Eng Nat Sci 2023;41(6):1121−1131.

INTRODUCTION

The use of steel as a construction material is increasing day by day facilitating the fabrication of structures with thinner sections and increased durability [1]. Steel has been particularly useful in the design of earthquake-resistant structures where imparting ductility to the structure is a major concern. The inherent strength combined with the ductility of steel enables the structures to satisfy the strength requirement during service loading conditions as well as ductility requirements during a seismic hazard. There are different types of steel structural framing systems that are used to render building safe from seismic forces, like moment-resisting framing system, concentrically braced framing system, eccentrically braced framing (EBF) system, buckling restrained brace framing system,

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This paper was recommended for publication in revised form by Regional Editor Chandramohan VP

Published by Yıldız Technical University Press, İstanbul, Turkey

steel plate shear walls, etc. The focus of this research is on studying the seismic performance of EBF systems.

In EBF systems, braces are connected to the beam with an offset, unlike the concentrically braced systems, thereby isolating a link element in the beam. Axial forces experienced by the braces during a seismic event are transferred to another brace or column through this link element. Under the action of brace forces, the link element undergoes inelastic deformation dissipating the energy imparted to the system and thereby preventing damage to the primary load-carrying beams and columns of the structure [2]. EBFs were first formally introduced in 1977 by Roeder and Popov [3], followed by several studies on the design and behavior of EBF systems under seismic loading [2,4]. Eccentrically braced frames are recognized as an efficient seismic load resisting system for achieving the stiffness and ductility demand of present-day building code provisions [5]. Along with having a high ductility capacity, EBF systems are easily repairable and replaceable after earthquakes and therefore beneficial in seismic regions [6]. In recent times, several researchers have focused their studies on eccentrically braced frames. Ashrafi and Imanpour [7] studied the seismic response of multi-tiered eccentrically braced frames. In his study, eccerntrically braced frames of different heights, relative teir heights and brace to beam connections were designed according to the Canadian design standard and the response was studied. Chacon et al. [8] presented a numerical study on stainless steel I-shaped links in EBF subjected to cyclic loading. A parametric analysis was also carried out to study the effect of variation of web slenderness, transverse stiffness and material properties. Hernandez and Carrera [9] carried out nonlinear analysis of 6 and 9 storey eccerntrically braced frame buildings to study the inelastic response of ductile eccentrically braced frames. Mohsenian et al. [10] studied the seismic performance of eccentrically braced frames of different heights under sequential earthquakes. In the 1990s EBF systems were recognized as high ductility systems by code committees all over the world and response reduction factors, R ranging from 4 to 7 were assigned to the EBF systems. However, the wide range of response reduction factor itself indicates the lack of consensus among different code committees on the ductility capacity available in EBF systems [11].

The drafting of the Indian standard code for earthquake resistant design of structures (IS 1893:2016) [12] has been done with assistance derived from the codes UBC 1994 [13], NEHRP 1991 [14], and NZS 4203:1992 [15]. The R-values in the NEHRP 1991 code are the same as given in ATC-3-06 [16], which were revised and designated as Rw in UBC 1994. In the report on structural response modification factors, ATC-19 [17] points out that the R-values specified in ATC-3-06 were based on a very little technical basis and were decided by the consensus of the committee keeping in view the following points: "1) General observed performance of like buildings during past earthquakes, 2) Estimates of general system toughness, 3) Estimates of the amount of damping present during inelastic response" (ATC-19). The ATC-19 report concluded that the specified values of R factors will not yield an equal level of risk for all the buildings as these values of seismic performance factors given in the codes do not address the variation in collapse performance due to differences in building periods, inelastic response capacity and seismic design category. This observation indicates the need to study and validate the performance of the EBF systems designed with response reduction factor value given in the Indian standard code. Therefore, The present study was aimed to study the adequacy of the response reduction factor given in the Indian codes for design of low rise EBF buildings.The check for adequacy requires a systematic comprehensive procedure for the estimation of seismic performance, such as documented in FEMA P695 [18].

FEMA P-695 provides a comprehensive procedure through which, the ability of a system to achieve the desired safety against collapse can be verified. In the present study, an attempt has been made to validate the value of the Response Reduction Factor specified in the Indian standard code for the design of EBF systems. A 5 storey EBF structure with the most optimal link length possible was used to validate the performance of the EBF system. The ratio of length of the link element to the length of the beam has been represented by the link-length ratio (e/L) throughout the paper. The most optimal linklength ratio was obtained through a parametric study on the link-length ratio by comparing the displacement ductility of the resulting EBF systems. Two types of link sections for the EBF structure were used in the study, rolled steel sections, and built-up sections. Rolled steel sections are readily available in the market but there is not much flexibility in the proportioning of flange and web dimension and thus constrain the design options. In the case of the built-up link section, it was assumed that there is no restriction on the selection of flange and web dimensions. The performance of the EBF structure having rolled steel link sections was compared with the EBF structure having built-up link sections, to study the effect of constraining the size of the link section on the performance of the EBF system.

DESIGN

Five storey dual EBF- moment resisting frame (MRF) buildings were modelled in SAP 2000. Initially, the link elements were modelled using the available rolled steel sections. In the latter part of the analysis, built-up steel sections were used for modelling the link elements in EBF buildings. The link sections of length varying from 15% to 40% of the beam length were created. The plan, elevation, and dimensions of the buildings are shown in Figure 1. Eccentric braces were provided in 2 of the 5 bays along the longitudinal direction, and, in both the bays in the transverse direction along the perimeter of the building.

Design guidelines developed by Singhal et al. [19] were used for the design of beams, columns, braces, and link elements. The section of the link was kept the same as that of the corresponding beam outside the link. The Link sections were designed to fail in shear by ensuring the links were shorter than 1.6 Mp/Vp, as shear links are known to have better plastic behavior than flexural and intermediate links [4]. The length of the links has been kept uniform along the height of the building. A uniform value of the link overstrength factor (the ratio of the link yield strength to link design shear force) is recommended for adequate link strength distribution along the height [20-22]. Therefore, a uniform overstrength factor (close to unity) has been kept in all the link members to minimize the subjective overstrength in the members outside the link. The buildings were designed for the seismic forces according to IS 1893:2016, assuming to be situated in Zone V on soft soil. A response reduction factor of 5, according to IS 1893:2016, was used in the design. Since IS 1893:2016 does not specify the capping period for correction of base shear in EBF, the approximate period specified in ASCE-7 [23] was used to calculate the base shear correction factor.

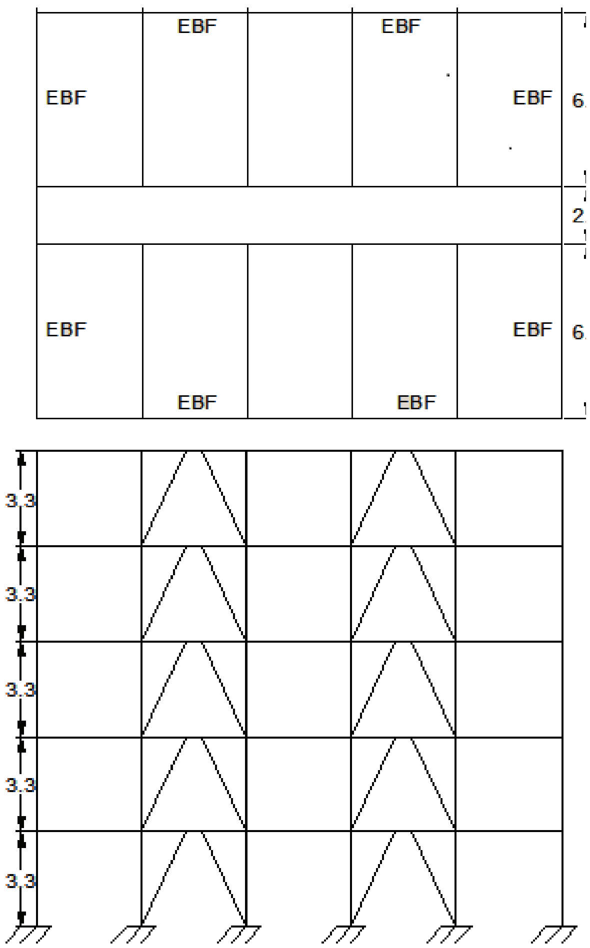


Figure 1. Plan and elevation of the modelled EBF building

MODELLING

Beams, columns, and links were modelled as frame elements having six degrees of freedom at each node. The nonlinear behavior of the frame elements was modelled with the help of lumped plasticity hinges at probable locations of yielding in the frame components. Flexural hinges were assigned at the expected locations of maximum bending moments in beams, and shear hinges were assigned in the links. Only shear and flexure action in the beams were considered as deformation-controlled actions as per the specification for EBF frame components given in ASCE 41-13 [24]. Nonlinear parameters were defined for the hinges using the values given in ASCE 41-13 [20]. The typical location of hinges have been shown in Figure 2, and the details of the hinges provided have been shown in Table 1. In ASCE 41-13, backbone curves as shown in Figure 3 are specified to simulate the strength degradation in the components as the inelastic deformation progresses. The values of parameters a, b and c are specified depending on the type of component and section properties. The relevant values of the parameters a, b, and c and performance limit states (acceptance criteria) for EBF components are given in Table 2. The strain hardening effect in steel was considered by increasing the peak strength capacity up to 1.1 times the yield strength.

ANALYSIS

The most optimal link-length ratio (e/L) for the EBF archetypes under consideration was estimated through a parametric study. The most optimal link-length ratio was considered as the link-length ratio which provides the maximum displacement ductility for the EBF. To find out the link-length ratio corresponding to the maximum displacement ductility, models with e/L ratio increasing from 15% to 40%, were subjected to nonlinear static pushover analysis.

The same study was then repeated for the Indian standard rolled steel sections. Sections given in SP-6 [25] are more practical as these sections are rolled in the industry and more likely to be used for construction. A total of 34 cross- sections were selected from a list of 65 cross-sections which suited the criteria for plastic sections as specified in IS 800:2007 [26]. The details about the seismic analysis are given in [27-29]. The shear utilization ratio was kept close to unity. The link sections and the section of the beam outside the link were kept the same. For rolled steel sections, the capacity design could not be achieved for the e/L ratios of 20% to 35%, as the sections which could satisfy the flexural requirement for the beam outside the link while keeping the shear utilization ratio close to unity were not available among the selected 34 cross-sections. So the link length was reduced to decrease the demand on the link sections and make the criteria for selection less stringent.

Table 1. Deatils of the hinges provided in the building

Hinge Name	Type		Behavior
Axial	Axial	P	Force Controlled
beamsflex1	Interacting	P-M3	Force Controlled
beamsflex2	Interacting	P-M3	Force Controlled
beamsflex3	Interacting	P-M3	Force Controlled
beamsflex4	Interacting	P-M3	Force Controlled
beamsflex5	Interacting	P-M3	Force Controlled
Col	Interacting	P-M2-M3	Force Controlled
COL2_hinge	Interacting	P-M2-M3	Force Controlled
COL3_hinge	Interacting	P-M2-M3	Force Controlled
column_hinge	Interacting	P-M2-M3	Force Controlled
LH1	Shear	V2	Deformation Controlled
LH2	Shear	V2	Deformation Controlled
LH3	Shear	V2	Deformation Controlled
LH4	Shear	V2	Deformation Controlled
LH5	Shear	V2	Deformation Controlled
linkb1	Moment	M3	Deformation Controlled
linkb2	Moment	M3	Deformation Controlled
linkb3	Moment	M3	Deformation Controlled
linkb4	Moment	M3	Deformation Controlled
linkb5	Moment	M3	Deformation Controlled

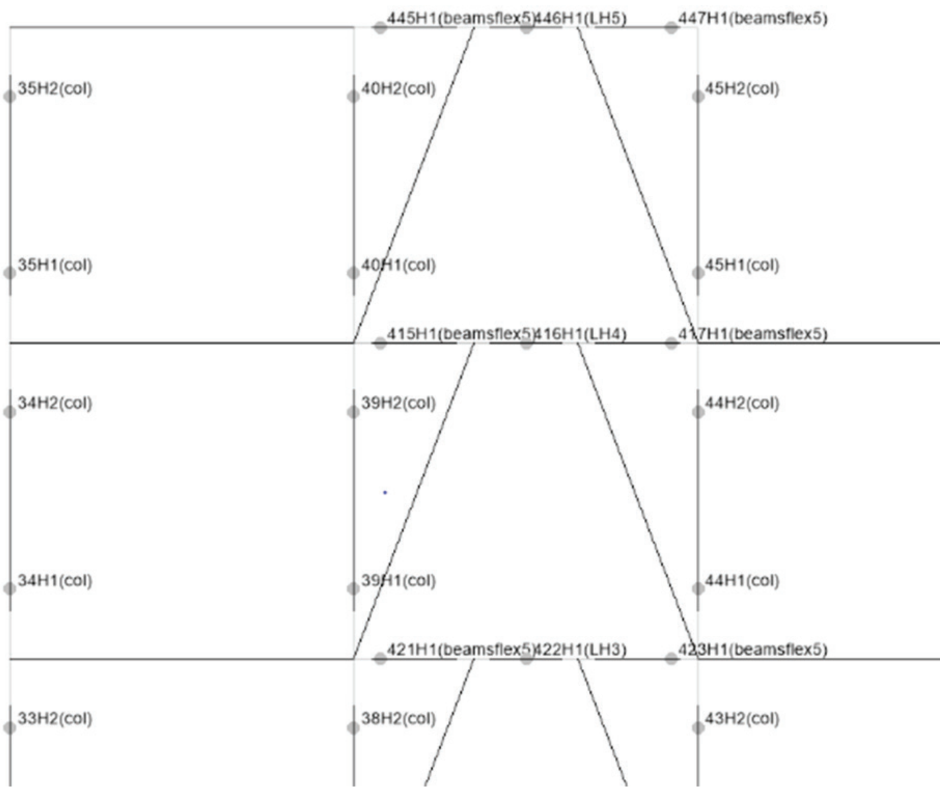


Figure 2. Typical locations of hinges in the EBF building.

Table 2 . Modelling Parameters of EBF components

EBF components		Nonlinear modelling parameters for the components					
		a	**b**	**c**	**IO (Immediate Occupancy)**	**LS (Life Safety)**	**CP (Collapse Prevenstion)**
Shear Links		0.15	0.17	0.8	0.005	0.14	0.16
Beams	$\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{yw}}}$ $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{yw}}}$	$9\theta_y$	$11\theta_y$	0.6	θ_y	$9\theta_y$	$11\theta_y$
	$\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{yw}}}$ $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_{yw}}}$	$9\theta_y$	$9\theta_y$	0.2	$0.25\theta_y$	$3\theta_y$	$4\theta_y$

Where θ_y is the yield rotation

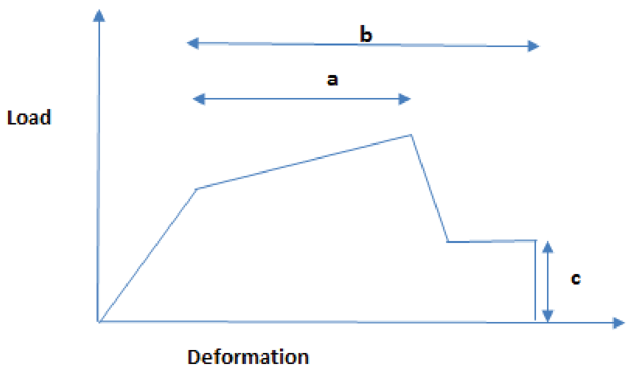


Figure 3. Force deformation curve for ductile components (ASCE 41-13)[20]

The buildings with the most optimal link-length ratio were then subjected to Incremental Dynamic Analysis (IDA) using the ground motion set given in FEMA P695. A set of 22 far-field ground motions are specified in the document for the collapse assessment of structural systems belonging to different seismic design categories. Mean spectra for 22 ground motion recordset has been shown in Figure 4. Before dynamic analysis, all the ground motions were normalized with respect to their respective peak ground velocities. The normalization factors and spectral acceleration for ground motions are shown in Table 3. After normalization, the ground motion recordset was scaled in such a manner that the median of spectral accelerations of ground motions matches with the target spectral acceleration at the fundamental period, T1, of the EBF structure under consideration. The ground motion intensity was defined in terms of the median of spectral intensities at the

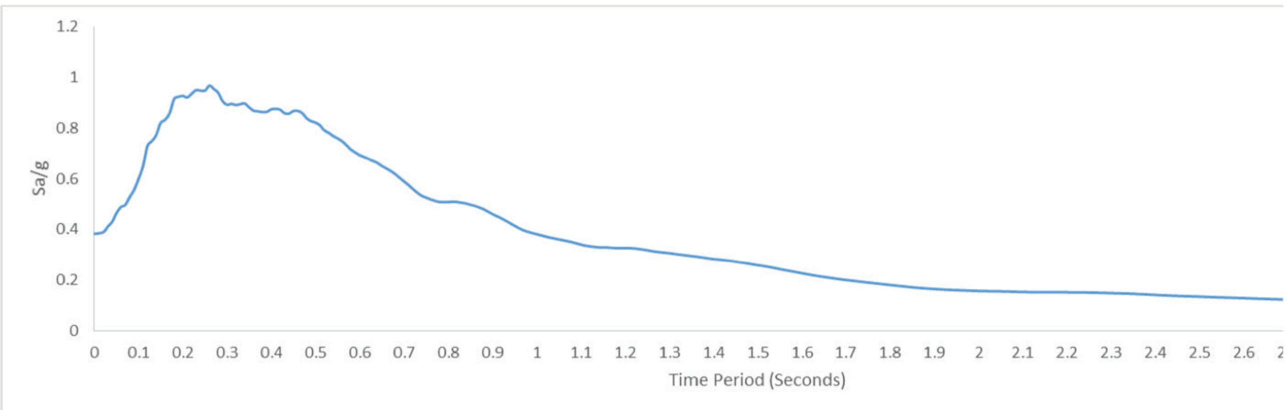
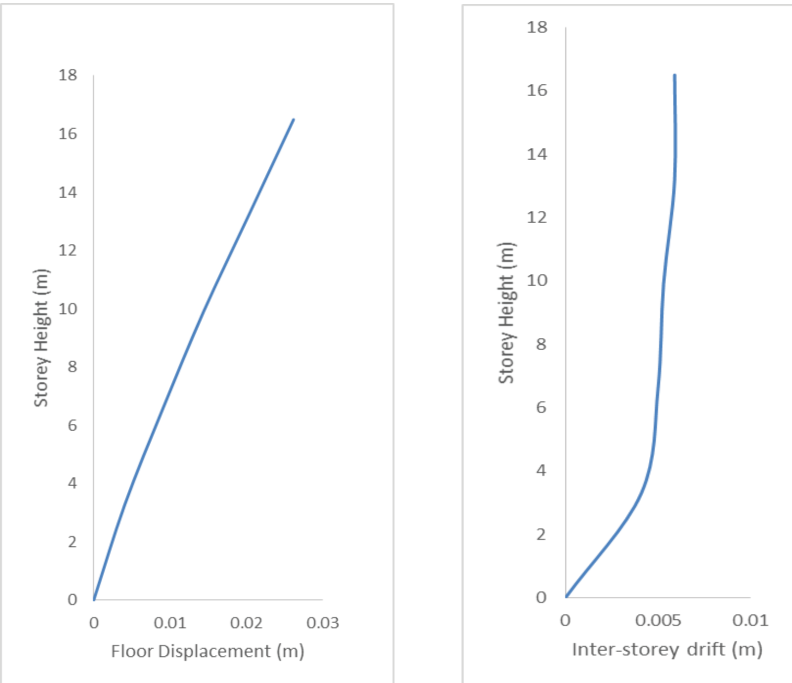


Figure 4. Mean spectra for 22 ground motion record set.

Table 3. Normalization Factors and Spectral Acceleration for ground motions

Ground motion Record	PGV_{PEER}	NM	Spectral Acceleration at T_1 for 1st component	Spectral Acceleration at T_1 for 2nd component
953	57.2	0.65	0.82	1.37
960	44.8	0.83	1.15	0.82
1602	59.2	0.63	1.02	0.76
1787	34.1	1.09	0.44	0.95
169	28.4	1.31	0.62	0.91
174	36.7	1.01	0.38	0.63
1111	36	1.03	1.01	0.75
1116	33.9	1.10	0.61	0.79
1158	54.1	0.69	0.55	0.46
1148	27.4	1.36	0.27	0.23
900	37.7	0.99	0.48	0.27
848	32.4	1.15	0.66	1.34
752	34.2	1.09	1.06	1.00
767	42.3	0.88	0.65	0.54
1633	47.3	0.79	0.47	0.59
721	42.8	0.87	0.62	0.34
725	31.7	1.17	0.73	0.62
829	45.4	0.82	0.66	0.99
1244	90.7	0.41	0.27	0.32
1485	38.8	0.96	1.03	0.85
68	17.8	2.09	0.61	0.57
125	25.9	1.44	0.72	1.37
Median	37.2		0.64	0.75

PGV_{PEER}: geometric mean of PGVs of the two components of ground motion, NM: normalization factor for the ground motion



(a) **(b)**

Figure 5. (a) Floor displacements along the height of the building, (b) Variation of inter-storey drifts along the height of the building.

fundamental period T of the whole recordset rather than by the spectral accelerations of individual records. This way, the whole far-field recordset was representative of a suite of ground motions in which the spectral intensities of the individual records were dispersed about the median spectral intensity of the set. The intensity at which half of the ground motions induced collapse in the structure was obtained as the Median Collapse Intensity. The ratio of this Median Collapse Intensity to the Maximum Considered Earthquake (MCE) intensity obtained from the design spectrum given in the Indian seismic design code gave the Collapse Margin Ratio (CMR) for the EBF archetype under consideration. The CMR value was then adjusted for the spectral shape factor (FEMA P695) to obtain the adjusted collapse margin ratio. This adjusted collapse margin ratio value was checked against the acceptable value given in FEMA P695 for the corresponding established probability of collapse to verify the adequacy of the response reduction factor that has been used in the design of the EBF structure.

The acceptable value of the adjusted collapse margin ratio depends on total system collapse uncertainty and acceptable probabilities of collapse. Total system collapse uncertainty is determined by the quality of test data, the quality of design, and the quality of the model. The quality of test data used was considered to be of the "Good" category because it was referred from established standards (ASCE 41-13 and IS 1893:2016), in which thorough research and testing have been used to establish the test data. Quality of design requirements was also considered to be of the "Good" category as the model design guidelines were prepared by comparing widely accepted international codes and verified through nonlinear analysis of the designed models by Singhal et al. [19]. Model quality was considered to be "Fair" as the necessary details to model the nonlinear behavior of the components were incorporated, but the precise modelling of beam-column connections and foundation were missing. Total system collapse uncertainty was referred from Table 7-2c of FEMA P695 corresponding to "Good" quality of test data and design requirements and "fair" model quality. Then, the acceptable values of adjusted collapse margin ratio corresponding to the total system collapse uncertainty and established ranges of collapse probabilities were referred from Table 7-3 of FEMA P695.

RESULTS AND DISCUSSION

While designing the EBF systems, the maximum possible link length ratio for which the EBF system could be designed were obtained as the most optimal link-length ratios as they gave the maximum displacement ductility. If the link length were increased beyond these limits, the capacity-based design could not be achieved and the hinges were found to be formed outside the links. The floor displacements and inter-storey drifts along the height of the building have been shown in Figure 5. The Inter-storey drift ratios obtained for EBF with e/L ratio ranging from 15% to 35% have been shown in Table 4 and these values have been compared with the permissible values given in IS 1893:2016, ASCE 7 and AISC 360-10. The inter-storey drift ratios were found to be within the permissible limits. Results of pushover analysis obtained for different link length ratios have been presented in Table 5. The most optimal link length which provided the maximum ductility capacity was obtained as 35% of the beam length in both the directions in buildings with built-up link sections. The Capacity design for rolled steel link sections was achieved at the link length ratio of 15% in the longitudinal direction. Similarly, the link length (e/L) was reduced to 5% in the transverse direction so that section requirements could be at par with any of the 34 sections available for use. The link sections used in the EBF building under consideration are given in Table 6. The details of the rolled steel sections given in IS 808:1989 [30] which have been used as link elements in this study are given in Table 7. The performance point at Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) obtained from the pushover curves and the ductility demands have been given in Table 8. The pushover curves for e/L values ranging from 15% to 35% for built-up link elements have been shown in Figure 6. The formation of plastic hinges for built-up link elements in the structure have been shown for different link-length (e/L) ratios in Figure 7. The hinges in the fourth storey have reached collapse first for all link-length ratios (e/L) except for 15% as shown in Figure.7(a) in which the collapse has occurred in the links of the third storey. The collapses have been reached in the hinges of links first as desired for the EBF system. An exception has been observed in the case of 40% (e/L) as shown in Figure.7(f) where force-controlled

Table 4. Inter storey drift ratios for different link length ratio

e/L	Maximum Inter Storey Drift Ratio	Permissible Inter-storey Drift ratios as per:		
		IS 1893:2002	ASCE 7	AISC 360-10
15%	0.0052	0.0132	0.033	0.0275
20%	0.0056	0.0132	0.033	0.0275
25%	0.0062	0.0132	0.033	0.0275
30%	0.0065	0.0132	0.033	0.0275
35%	0.0073	0.0132	0.033	0.0275

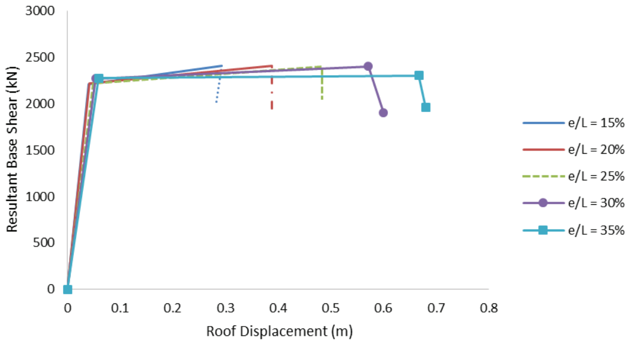


Figure 6. Pushover Curves for different e/L values using built-up link elements.

hinges have been formed in the beam outside the link in the lowermost storey even before the link hinges reached collapse. The median collapse intensity was obtained at 2.1g and 1.2g intensity for built-up link sections and for rolled steel link sections respectively after being subjected to increasing intensities of 22 Far Field ground motions. The MCE intensity was obtained from the design spectrum given in IS 1893:2016 for Zone V and type III soft soil. The formula for an approximate fundamental time period for steel buildings given in ASCE-7 was used for the calculation of time period of the EBF structure (as per the guidelines mentioned in FEMA P695) and an MCE intensity of 1.26g was obtained for the corresponding time period. The collapse margin ratio was calculated by taking the ratio of median collapse intensity to MCE intensity. The collapse margin ratio for EBF structures with rolled steel link sections was obtained as 0.952 and 1.67 for EBF structures with built-up link sections. The Collapse Margin Ratios obtained for the considered models were then adjusted for spectral shape effects. Fragility curves for the EBF building with built up sections and rolled steel sections have been shown in Figure 8.

The EBF building designed with rolled steel link sections had a fundamental period of 0.393s, ductility ratio of 6 and seismic design category SDC Dmax, the corresponding value of spectral shape factor of 1.28 was obtained from table 7-1b of FEMA P695. The Collapse Margin Ratio obtained from the Incremental Dynamic Analysis (IDA) results was multiplied with this spectral shape factor to obtain Adjusted Collapse Margin Ratio. Adjusted Collapse Margin Ratio for rolled steel sections was obtained as 1.2186. Similarly the EBF structure designed with built link sections had a fundamental period of 0.6827s, ductility ratio of 12 and seismic design category SDC Dmax, the corresponding value of spectral shape factor was obtained as 1.38. Adjusted Collapse Margin Ratio for the EBF structure designed with built up link section was obtained as 2.305.

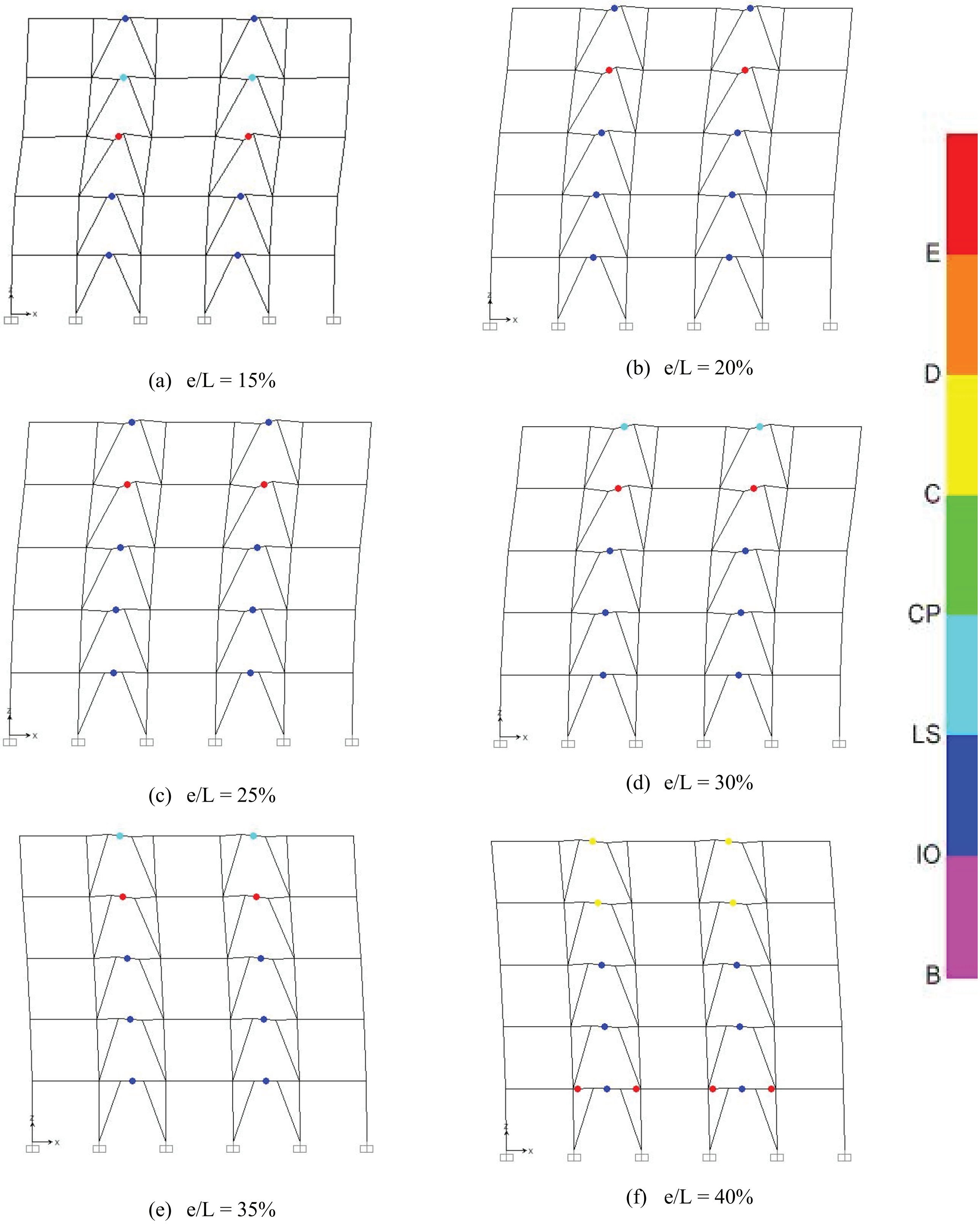
A value of 2.16 was obtained as the acceptable value of adjusted collapse margin ratio, ACMR10%, corresponding to the total system collapse uncertainty, $\beta_{TOT} = 0.600$ and 10% probability of collapse. By comparing this value of ACMR10% with the adjusted collapse margin ratio for EBF structures of built up section and rolled steel link sections, it was found that the EBF structure with the built-up link sections gives acceptable performance with the given R factor, whereas the EBF structure with rolled steel link sections resulted in unsatisfactory performance.

Table 5. Pushover analysis results for different link length ratios

e/L	Time Period T	Stiffness (kN/m)	Strength (kN)	Yield displacement (m)	Ultimate displacement (m)	Ductility Capacity
15%	0.576	58786	2207	0.0395	0.2934	7.428
20%	0.648	52351	2216	0.0413	0.3888	9.414
25%	0.654	47166	2216	0.049	0.4837	9.871
30%	0.705	42599	2268	0.0536	0.5714	10.660
35%	0.744	38088	2274	0.0592	0.6673	11.272

Table 6. Link Section used for rolled steel sections

Storey	e/L	Longitudinal direction	e/L	Transverse direction
5	15%	ISLB 225	5%	ISHB 150-1
4	15%	ISLB 325	5%	ISLB 225
3	15%	ISWB 350	5%	ISMB 250
2	15%	ISWB 400	5%	ISLB 300
1	15%	ISWB 400	5%	ISLB 300



(a) e/L = 15%

(b) e/L = 20%

(c) e/L = 25%

(d) e/L = 30%

(e) e/L = 35%

(f) e/L = 40%

Figure 7. Hinge formation in the EBF building for different link-length ratio (e/L) for built-up link elements.

Table 7. Details of the rolled steel sections used as link elements as per the Indian Standard [30](IS 808:1989)

Steel Section	Weight per Meter (kg)	Sectional Area (cm^2)	Depth of Section (mm)	Width of flange (mm)	Thickness of Flange (mm)	Thickness of Web (mm)
ISLB 225	23.5	29.92	225	100	8.6	5.8
ISLB 300	37.7	48.08	300	150	9.4	6.7
ISLB 325	43.1	54.9	325	165	9.8	7
ISMB 250	37.3	47.55	250	125	12.5	6.9
ISWB 350	56.9	72.5	350	200	11.4	8
ISWB 400	66.7	85.01	400	200	13	8.6
ISHB 150-1	27.1	34.48	150	150	9	5.4

Table 8. Performance points for different Link Lengths

e/L	Performance point at DBE	Performance point at MCE	Performance at DBE	Performance at MCE	Ductility Demand
15%	0.0785	0.1831	LS	LS	4.636
20%	0.0980	0.2218	LS	LS	5.371
25%	0.0996	0.2249	LS	LS	4.590
30%	0.1081	0.2382	LS	LS	4.445
35%	0.1130	0.2450	LS	LS	4.139

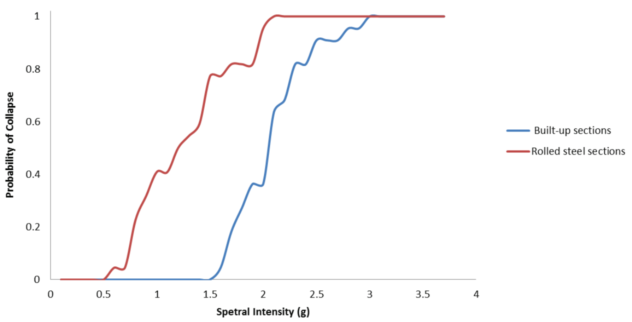


Figure 8. Fragility curves for the EBF building with built-up sections and rolled steel sections.

CONCLUSIONS

The adequacy of Response Reduction Factor specified in the Indian seismic design code has been verified using the procedure given in FEMA P695. It has been found from the study that acceptable performance is obtained only in the case of EBF structures with built up link sections, while the structure may not perform as desired if the rolled steel link sections are used. As the built up link sections used in the study are difficult to fabricate and rolled steel sections are usually employed in construction, the R factor given in IS 1893:2016 for EBF structure need to be revised to get the desired probability of collapse.

AUTHORSHIP CONTRIBUTIONS

Authors equally contributed to this work.

DATA AVAILABILITY STATEMENT

The authors confirm that the data that supports the findings of this study are available within the article. Raw data that support the finding of this study are available from the corresponding author, upon reasonable request.

CONFLICT OF INTEREST

The author declared no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

ETHICS

There are no ethical issues with the publication of this manuscript.

REFERENCES

[1] Baddoo NR. Stainless steel in construction: A review of research, applications, challenges and opportunities, J Constr Steel Res 2008;64:1199−1206. [CrossRef]

[2] Hjelmstad KD, Popov EP. Characteristics of eccentrically braced frames, J Struct Eng 1984;110:340−353. [CrossRef]

[3] Roeder CW. Inelastic behavior of eccentrically braced steel frames under cyclic loading, UCB/EERC-77/18, University of California, Berkeley, August, 1977, pp. 308.

[4] Popov EP. Engelhardt MD, In Seismic Eccentrically Braced Frames, 1988;10:321–354. [CrossRef]

[5] Okazaki T, Engelhardt MD, Drolias A, Schell E, Hong JK, Uang CM, et al. Experimental investigation of link-to-column connections in eccentrically braced frames. J Constr Steel Res 2009;65:1401–1412. [CrossRef]

[6] Ioan A, Stratan A, Dubina D, Poljanšek M, Molina FJ, Taucer F, et al. Experimental validation of re-centring capability of eccentrically braced frames with removable links. Eng Struct 2016;113:335–346. [CrossRef]

[7] Hines EM, Jacob CC. Eccentric Braced Frame System Performance, In ASCE Conf Proc. 2010;369:121. [CrossRef]

[8] Ashrafi A, Imanpour A, Seismic response of steel multi-tiered eccentrically braced frames. J Constr Steel Res 2021:181. [CrossRef]

[9] Chacón R, Vega A, Mirambell E. Numerical study on stainless steel I-shaped links on eccentrically braced frames, J Constr Steel Res 2019;159:67–80. [CrossRef]

[10] Hernández ET, Carrera SG, Inelastic response of ductile eccentrically braced frames, J Build Eng 2019;26:100903. [CrossRef]

[11] Mohsenian V, Filizadeh R, Hajirasouliha I, Garcia R. Seismic performance assessment of eccentrically braced steel frames with energy-absorbing links under sequential earthquakes. J Build Eng 2021;33:101576. [CrossRef]

[12] IS 1893 (Part 1):2016, Criteria for Earthquake Resistant Design of Structures Part-1: General Provisions and Buildings, Bureau of Indian Standards, New Delhi, India, 2016.

[13] UBC, Uniform Building Code, In International Conference of Building Officials, Whittier, California, USA, 1994.

[14] NEHRP, NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, Part 1: Provisions, Report No FEMA 222, Federal Emergency Management Agency, Washington, D.C., USA, January 1992.

[15] NZS 4203: 1992, Code of Practice for General Structural Design and Design Loadings for Buildings, Standards Association of New Zealand, Wellington, New Zealand, 1992.

[16] ATC, Tentative Provisions for development of seismic regulations for buildings, ATC-3-06 Report, Applied Technology Council, Redwood City, California.

[17] ATC, ATC 19 Structural Response Modification factors, Applied Technology Council, 1995, 1–69.

[18] FEMA P695, Quantification of building seismic performance factors, Federal Emergency Management Agency, Washington, D.C., USA, 2009, 421.

[19] Singhal A, Singh Y, Seismic Performance of Eccentrically Braced Frame (EBF) Buildings, In Adv Struct Eng Dyn (ed. Matsagar, V.), Vol. two, Springer India, New Delhi, 2015, pp. 921-932. [CrossRef]

[20] Bosco M, Rossi PP, Seismic behaviour of eccentrically braced frames. Eng Struct 2009;31:664–674. [CrossRef]

[21] Popov EP, Ricles JM, Kasai K, Methodology for optimum EBF link design, In Proceedings of the 10th world conference on earthquake engineering, 1992:3983–3988.

[22] Kasai K, Han X, New EBF design method and application: Redesign and analysis of US_Japan EBF, In Proceedings of the international workshop and seminar on behaviour of steel structures in seismic areas. 1997:242–249.

[23] ASCE/SEI 7, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers.

[24] ASCE/SEI 41-13, Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers, 2014.

[25] SP-6 (Part 1): 1964 (Reaffirmed 1998), Handbook for Structural Engineers No. 1, Structural Steel Sections, Bureau of Indian Standards, New Delhi, India, 1998.

[26] IS 800:2007, General construction in steel - Code of practice, Bureau of Indian Standards, New Delhi, India, 2007.

[27] Mohammad Z, Bilal A, Baqi A, Effect of Base Isolation on the Seismic Performance of Hill Buildings, Earthquakes and Structures, Lecture Notes in Civil Engineering, vol 188, Springer, Singapore, 2022. [CrossRef]

[28] Bilal A, Mohammad Z, Non-linear analysis of base-isolated building having optimized number of base isolators, Earthquakes and Structures, Lecture Notes in Civil Engineering, vol 188, Springer, Singapore, 2022. [CrossRef]

[29] Bilal A, Agarwal P, Sadique MR, Performance Evaluation of Base Isolated Building, In: Kumar Shukla S, Raman SN, Bhattacharjee B, Bhattacharjee J (eds) Advances in Geotechnics and Structural Engineering, Lecture Notes in Civil Engineering, vol 143, Springer, Singapore, 2021. [CrossRef]

[30] IS 808:1989, Dimensions for hot rolled steel beam, column, channel and angle sections (third revision).