ECCENTRICALLY BRACED FRAME DESIGN FOR MODERATE SEISMIC REGIONS

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ABSTRACT:

Factors influencing the design of eccentrically braced frames when the seismic load does not play a primary role in the member selection are discussed. The influence of seismic overstrength is considered. Twenty-two frames with chevron configuration of eccentrically braced lateral force resisting systems are designed to investigate the influence of structure height, link length and various levels of the ratio of wind to earthquake load. These structures were analyzed under monotonic and dynamic lateral loads. It is concluded that smaller link seismic overstrength factors may be used for brace and column design in moderate seismic regions than currently specified in the design standard. The lesser ductility demands experienced also suggest that the detailing requirements for short links can be relaxed.

KEYWARDS: Eccentrically braced frames, Moderate seismic loading

INTRODUCTION

Eccentrically braced frames (EBFs) can be detailed to provide ductile behavior under severe seismic load, and design provisions for these systems were introduced into Canadian design standards in 1989. The research on which these provisions are based was performed in the USA starting in the early 1980's, and was motivated by the needs for improved systems to resist seismic loading. Naturally the emphasis was on the behavior in regions where severe loading is to be expected, and design provisions therefore reflect this. Thus, the provisions of the Canadian steel design code [1] [2] specify one category of EBF, denoted Ductile Eccentrically Braced Frames (DEBF). This is unlike other defined categories of systems for steel and concrete structures which, for example for moment resisting steel frames (MRSF) include three categories, ductile, moderately ductile and those with limited ductility.

The systems with lower ductile capability must be designed for higher seismic load, since it is expected that there will be lower demands on the ductile performance, and ductile elements of the structures are therefore not subject to such severe detailing constraints as required for the dissipating elements in more ductile systems.

In severe seismic zones [7] or severe seismic geographical locations [8] of Canada the seismic load rather than wind load will usually participate in the load combination creating the controlling conditions under lateral load. In this case the resistance provided to the members in the lateral load resisting system will be adequate to resist wind load or any other associated loads such as seismic or anticipated safety related loads from industrial profiles, and the members can be detailed for ductility with the reasonable expectation that, should the design earthquake occur, the provided ductile features will be mobilized. In less severe seismic geographical locations, when wind load constitutes the controlling lateral load, the requisite resistance will exceed that required by seismic load considerations, and therefore less ductility demand may be anticipated. This is analogous to the behaviour of a nominally ductile system. Even for severe seismic level, there are various features of structural systems and their design (e.g. material factors used in design, minimum design requirements, capacity design, load combinations and the redistribution of forces arising from redundancy) that often lead to a lateral strength or force resistance that is considerably larger than that used as the basis for the design [8]. In such cases, the principle of design and philosophy is quite similar to this study as the design codes normally require that design must accommodate at least the type of lateral load resisting system and detailing that correspond to the seismic forces calculated for the building.

This paper studies the behaviour of EBF in moderate zones or lower seismic geographical locations, with the objective of examining the ductility demands and possible relaxation of some of the existing detailing requirements for DEBF.

EBF DESIGN PHILOSOPHY AND BEHAVIOUR

Severe Seismic Loading

Ductile performance of EBFs is based on yielding of the link - the short length of beam between the connections to the two braces. Other members are designed to remain elastic. The shearing force in the link, which is directly proportional to the storey shear, will cause yield in shear if the link is short or in bending for longer links. A beam section that can carry this link force is selected, with as little over design as possible. There will of course be some excess strength. The ratio of the beam resistance, V'_r (assuming a short link) to the factored load, V_f, is denoted the strength factor α. Values of α for a well design structure in a severe seismic geographical location can be expected in the range 1.0 to 1.10.

To confine inelastic action to the link other members of the frame are designed so that they will remain elastic when subjected to loads induced by yielding and strain-hardening links. As an estimate of the forces developed by link strain-hardening the link yield capacity is assumed to be increased by a factor K, which is in the range 1.25 to 1.5. Other members are therefore to be designed for forces which are αK times the forces induced by the factored storey shear force.

A single degree of freedom structure, designed to resist a seismic design base shear V_E can be expected to undergo first yield when the lateral load is αV_E and would reach its ultimate load when the base shear is $\alpha K V_E$. For convenience consider the lateral load to be expressed as a seismic base shear factor λ_E times the base shear V_E. Fig. 1 shows the idealised relationship between the load factor and lateral displacement for the single degree of freedom system, Curves A, B and C represent the response of a system in which seismic load controls the design of the beam of the EBF. Line A represents perfectly plastic response if the link has exactly the strength to resist the seismic load. Line B is the corresponding response of a practical beam, having a resistance slightly higher than the force demand; by an amount α. Line C represents the probable real response, reflecting the development of strain hardening in the yielding link.

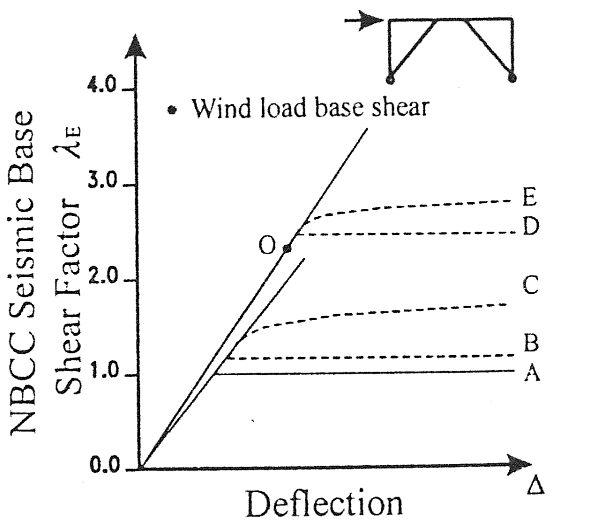

Figure 1 Idealized Lateral Load-Deflection Relations

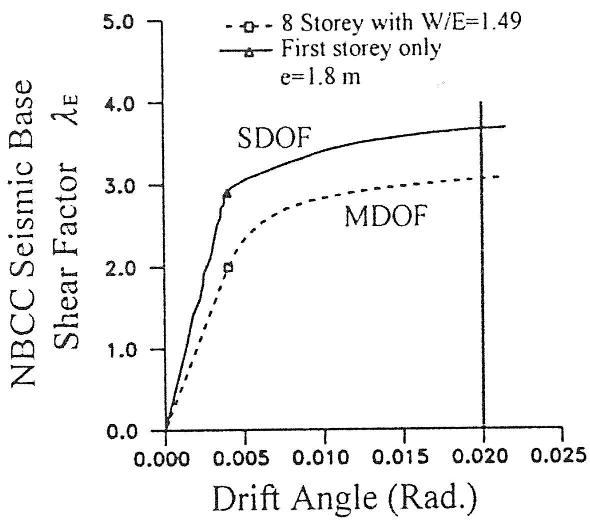

Figure 2 MDOF Response vs. SDOF Response

Seismic Loading in Moderate Seismic Geographical Locations

When seismic load does not control the size of the link beam, initial yield of the stiffer beam will occur at a higher lateral load. Consider the case where wind load requires the beam to be designed for the lateral load level corresponding to the point O in Fig. 1. Yield will occur at a higher load, and perfectly plastic or strain-hardening response would be represented by lines D and E in seismic responses, respectively.

The extent to which the strain-hardening response will rise above the perfectly plastic response will depend upon the maximum lateral deflection attained. It is recognised that the maximum displacements attained in an inelastic response are insensitive to the ductility available in the structure. Thus, the expected maximum deflections are independent of the force reduction factors used in the design. In view of this, it is to be expected that the maximum strain-hardening forces reached in the response E, compared with the initial yield value D, will be less than those in C relative to B.

In the case of a multi-degree-of-freedom system (MDOF), linear response will end when the first link yields. The subsequence response, lines C or E for example, will then incorporate, with the strain-hardening, the effects of changing stiffness as increasing numbers of links yield. Fig. 2 illustrates the lateral response to monotonically increasing lateral load of single and MDOF systems (the weaker MDOF response reflects the fact that some storeys are weaker than the one chosen as the SDOF frame).

The hypothesis that the strength enhancement due to strain-hardening will be reduced in the case when beams are designed for any reasons other than the seismic loads is important, since the forces for which the other members of the frame are designed derive from the mobilised link strain-hardening capacity. In the following, a number of structures are studied in order to examine this hypothesis.

DESCRIPTION OF STRUCTURES

A number of EBF structures located in a lower seismic geographical location of Canada were designed based on NBCC 1995 [7] and CSA S16-94 [1]. Those frames were then examined under the action of monotonic and dynamic loads. Note that as NBCC has been revised in 2005 [8] from 1995, some major changes have been made in the ways to determine seismic hazard and seismic loading. This would not significantly impact the overall design base shear of the buildings. Structures of four, eight and fourteen storeys were considered. In a moderate seismic geographical location, both gravity and wind load will have more influence on member selection than in a more severe seismic geographical location. In the designs considered, gravity load based on the design code was the same for all structures; specified wind load, on the other hand, will vary within the moderate seismic zone, and for this reason several different levels of wind load were considered. The ratio of *factored wind load base shear* to *factored seismic base shear,* W/E, was found to vary considerably within the moderate seismic zone selected, and structures with several different values of this ratio were considered. The ratio is a function of building size, and so different values apply to the different height buildings.

Table 1 Overstrength Factors Used for Brace/Column Design

Frames	W/E	e=1.8m	e=1.2m	e=0.8m	e=0.6m	e=0.4m
4 storey	0.35	1.25/1.15	1.25/1.15	-	1.40/1.25	1.70/1.25
8 storey	0.90	1.25/1.10	1.25/1.10	1.40/1.25		
	1.00	1.25/1.10	1.25/1.10	1.40/1.25		
	1.28	1.25/1.10	1.25/1.10	1.40/1.25		
	1.49	1.25/1.10	1.25/1.10	1.40/1.25		
14 storey	1.29	1.25/1.10	1.25/1.10	1.40/1.25		
	1.83	1.25/1.10	1.25/1.10	1.40/1.25		

The structures designed are described in Fig. 3. For each height of building, several link lengths were considered, varying from 1/3 to 1/15 of the bay width. Beams were continuous across the bay, and beams and braces were simply connected at the columns. Columns were continuous, with simply connected bases. Four W/E ratios were considered for the eight storey structure, two for the 14 storeys, and one for the four storeys. These reflect the fact that the four-storey frame is largely controlled by earthquake loads and the 14 storeys by wind loads, whereas only the eight-storey frame is very sensitive to the W/E ratios. In all, 22 frames were designed. All designs conformed to the requirements of Clause 27 of CSA S16-94[1], with the important exception that the overstrength factors, K, were varied.

Summary of cases studied:						
Cases	w*	W/E	Link length e (m) and (e/L)			
4 storey	0.37	0.67	1.8(1/3)	1.2(1/5)	0.6(1/10)	0.4(1/15)
8 storey	0.26	0.90	1.8(1/5)	1.2(1/8)	0.8(1/11)	
	0.31	1.00	1.8(1/5)	1.2(1/8)	0.8(1/11)	
	0.37	1.28	1.8(1/5)	1.2(1/8)	0.8(1/11)	
	0.41	1.49	1.8(1/5)	1.2(1/8)	0.8(1/11)	
14-storey	0.26	1.29	1.8(1/5)	1.2(1/8)	0.8(1/11)	
	0.37	1.83	1.8(1/5)	1.2(1/8)	0.8(1/11)	

* w – wind pressure; W/E – base shear ratio of wind to earthquake

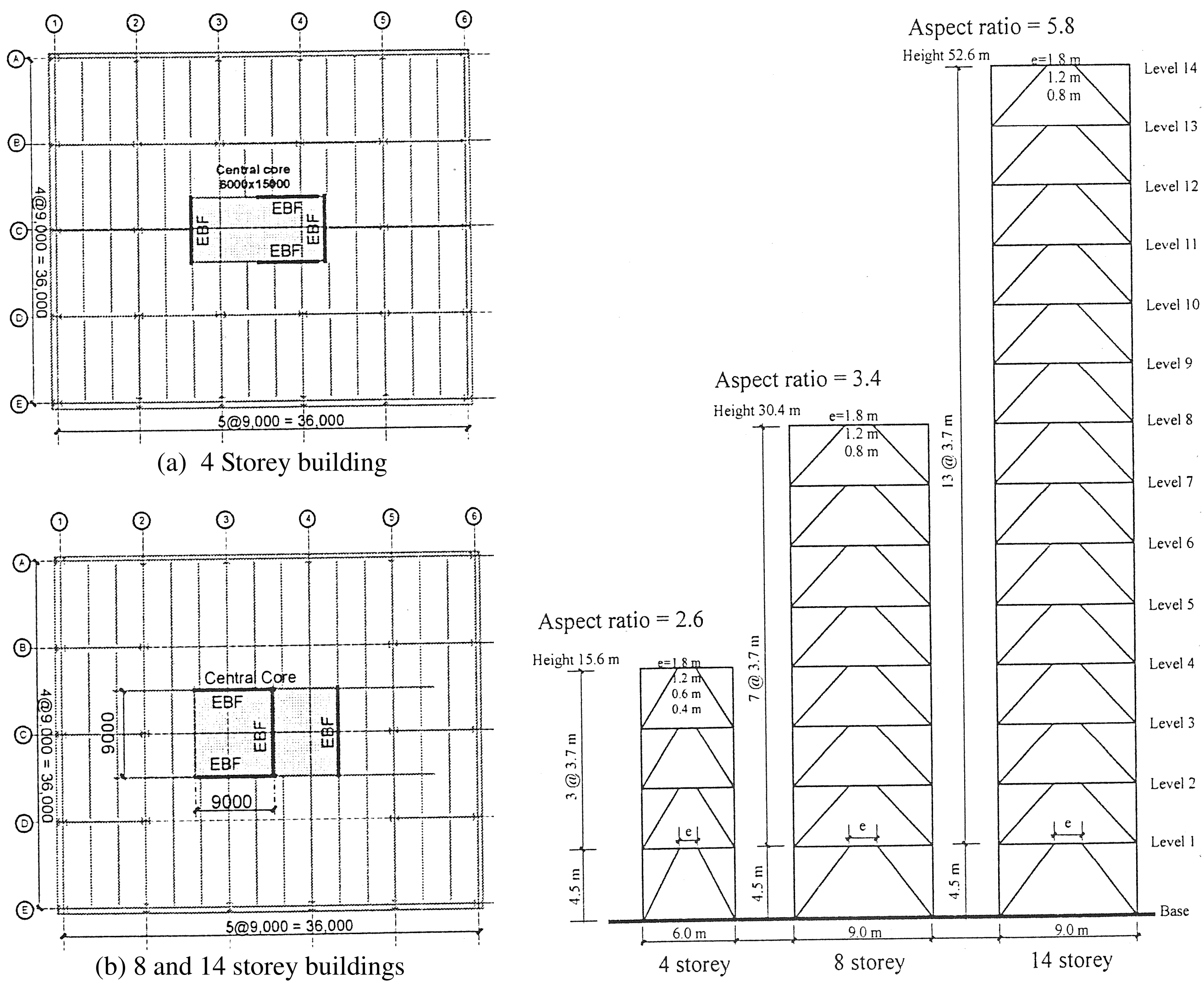

Figure 3 Details of Designed Frames

CSA S16-94 specifies two values of K. For brace and outer beam segment design K=1.5, and for column design K=1.25 (Note, CSA S16-01 (2005) [2] has updated by using $1.3R_y=1.43F_y$ for brace and outer beam segments except for the top two storeys and $1.15R_y=1.265F_y$ for typical column design, where $R_y=1.1F_y$). For the frames considered herein a trial-and-error procedure was followed in which reduced values of K were used both for brace and outer beam segment selection on the one hand, and columns on the other. The resulting structures were examined using inelastic time-history analysis for several earthquake records, and the maximum response forces were compared with the member resistances. K values were then modified until satisfactory responses were obtained. The overstrength factors, K, finally used are given in Table 1. Greatest reductions were found to be possible for the long links; very little variation was found for different W/E ratios. For one frame - (4 storey with e=0.4m) the brace factor needed exceeded the code value.

RESPONSE OF EBF STRUCTURES

Monotonic Loading

The variation of roof deflection with the Seismic Base Shear Factor, A, summarizes all monotonic load responses in Fig. 4. The vertical distribution of seismic load was taken as that specified for equivalent static seismic loads [7]. The ANSR program was used, and the links were modeled using the element proposed by Ric1es and Popov (1994) [9]. Other elements were modeled using the same connection and fixity assumptions as those used in design. All analyses terminated when the roof deflection reached 2 % drift. Two points are identified on each curve; one representing first yield of any of the links, and the second indicating when the code specified limit of link inelastic distortion was reached. The latter is seen to occur at a deflection considerably lower than the 2% value, although the difference between the corresponding loads is quite small.

Wind loads played only a minor role in the design of the four-storey frame, but nevertheless total frame overstrength in the range of 1.6 to 2.0 is evident. For the eight storey frames considerable differences between responses for the different W/E ratios is evident, except for the case with the shortest links e=0.8m. Wind drift played an important role in the 14 storey frame designs, and there is little difference between the responses for link lengths and W/E ratios examined. In nearly all cases, overstrength increases as W/E ratio increases, and also as the link length decreases.

The value of α for a beam will be high if the load controlling the choice of section is high compared with the seismic load in the beam. For a complete frame, a weighted average of the individual link beam α values can be obtained as $\alpha_{frame}= \Sigma\alpha_i V_{cum,i} /\Sigma V_{cum,i}$ where α_i is the link strength factor at storey i and $V_{cum,i}$ is the NBCC storey shear at storey i. Values computed in this way can be compared with the values obtained from the yield point indicated on each graph of Fig. 4. The average of the ratio of the former to the latter is 0.982 with COV 9.5%. The highest α_{frame} value from Fig. 4 is 2.90 for the 14-storey frame, and the lowest is 1.20 for the four-storey frame. The product of α_{frame} and the K factor used for each frame gives an estimate of the ultimate strength; these values corresponded quite well with the ultimate strengths evident in Fig. 4.

Dynamic Loading

Non-linear time-history analysis was performed on all structures, using six real earthquake records which were scaled to correspond to Zone 2 [7] [4]. Selected frames were also examined with these records scaled upward by a further 50 %. The same modelling was used as for the monotonic load analyses, and 3% damping was assumed. The peak response forces experienced by each member were examined. All members were modelled as beam-columns, and interaction between axial force and bending responses was examined at each step in the time-history, by combining and obtaining an interaction response value, representing a fraction of the resistance. Primary interest is in the maximum response values of the link forces, link deformations and forces in other members.

Maximum link moment and shearing force responses are summarised in Tables 2 and 3. These show ratios of maximum force to nominal yield strength as well as, in brackets, the maximum dynamic response as a percentage of maximum static response at 2 % roof drift. As expected, maximum moments occur in long links and maximum shears in short links. There is little evidence of any influence of the W/E ratio on these maximum link forces (see Table 3). Fig. 5 shows the distribution of maxima over the eight-storey frame height; there is greater variation for short links than for long links.

Table 2 Maximum Link Moment Response Ratios

Frames	W/E	e=1.8m	e=1.2m	e=0.8m	e=0.6m	e=0.4m
4 storey	0.35	1.15(96%)	1.21(98%)	-	1.25(87%)	1.23(99%)
8 storey	0.9	1.19(95%)	1.17(91%)	1.07(83%)		
	1	1.19(95%)	1.19(92%)	1.11(87%)		
	1.28	1.18(94%)	1.16(88%)	1.07(94%)		
	1.49	1.15(91%)	1.18(92%)	1.06(84%)		
14 storey	1.29	1.07(84%)	1.16(94%)	1.02(72%)		
	1.83	1.14(88%)	1.17(90%)	1.05(88%)		

Table 3 Maximum Link Shear Response Ratios

Frames	W/E	e=1.8m	e=1.2m	e=0.8m	e=0.6m	e=0.4m
4 storey	0.35	0.74(120%)	0.93(127%)	-	1.37(80%)	1.69(86%)
8 storey	0.9	0.66(72%)	1.03(79%)	1.22(76%)		
	1	0.69(79%)	1.04(80%)	1.32(81%)		
	1.28	0.84(91%)	1.08(88%)	1.43(89%)		
	1.49	0.77(72%)	1.06(80%)	1.32(79%)		
14 storey	1.29	0.95(88%)	1.01(98%)	1.24(80%)		
	1.83	0.95(85%)	1.03(76%)	1.27(81%)		

Table 4 Maximum Link Rotations

Frames	W/E		e=1.8m	e=1.2m	e=0.8m	e=0.6m	e=0.4m
4 story	0.35	$\gamma_{max,envelope}$	0.035	0.069		0.124	0.143
		$\gamma_{avg,envelope}$	0.921	0.044		0.090	0.078
		$\gamma_{max}/\gamma_{limit}$	58%	115%		108%	79%
		$\gamma_{avg}/\gamma_{limit}$	35%	73%		79%	43%
8 story	0.90	$\gamma_{max,envelope}$	0.056	0.040	0.031		
		$\gamma_{max}/\gamma_{limit}$	92%	65%	17%		
	1.00	$\gamma_{max,envelope}$	0.056	0.046	0.043		
		$\gamma_{max}/\gamma_{limit}$	92%	77%	23%		
	1.28	$\gamma_{max,envelope}$	0.054	0.038	0.052		
		$\gamma_{max}/\gamma_{limit}$	88%	63%	29%		
	1.49	$\gamma_{max,envelope}$	0.030	0.040	0.031		
		$\gamma_{max}/\gamma_{limit}$	50%	65%	17%		
14 story	1.29	$\gamma_{max,envelope}$	0.018	0.026	0.039		
		$\gamma_{max}/\gamma_{limit}$	28%	27%	22%		
	1.83	$\gamma_{max,envelope}$	0.028	0.030	0.031		
		$\gamma_{max}/\gamma_{limit}$	45%	31%	17%		

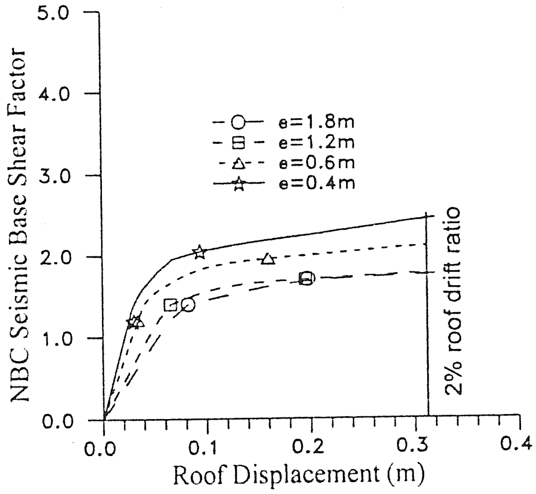

Figure 4a Base shear vs. Roof Displacement for Four Storey EBFs

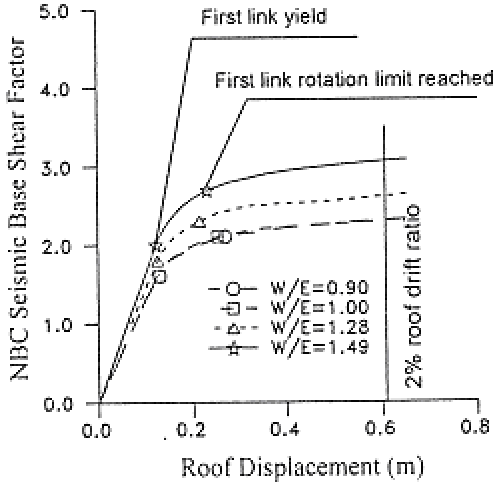

Figure 4b Base shear vs. Roof Displacement for Eight Storey EBFs with e=1.8m

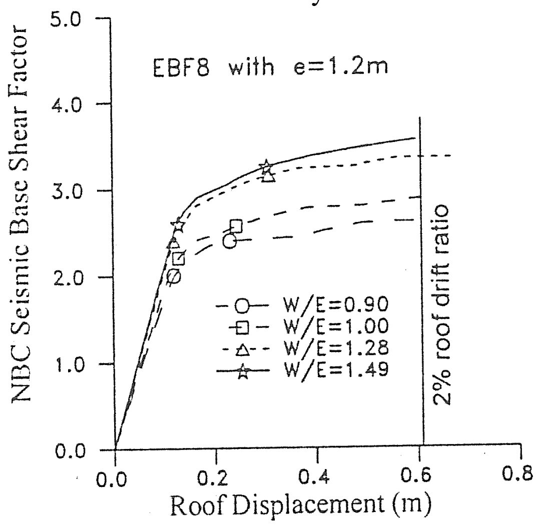

Figure 4c Base shear vs. Roof Displacement for Eight Storey EBFs with e=1.2m

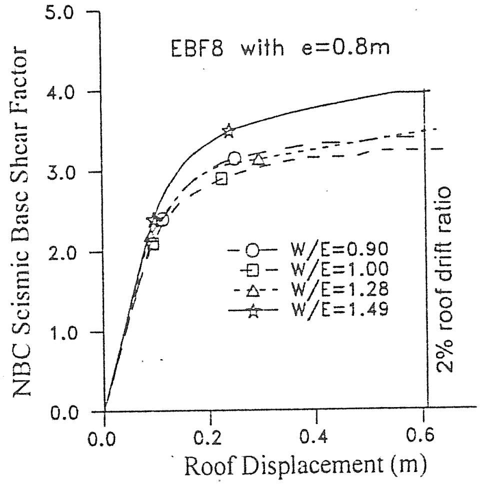

Figure 4d Base shear vs. Roof Displacement for Eight Storey EBFs with e=0.8m

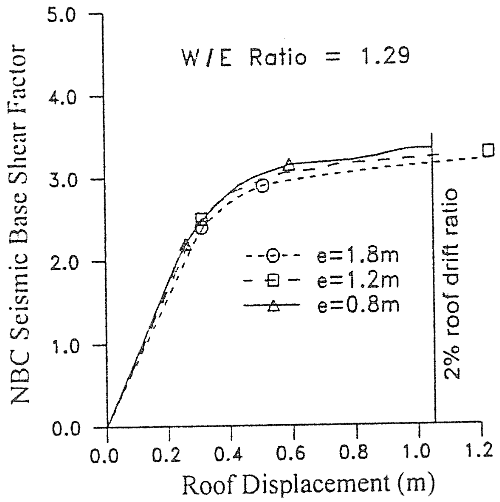

Figure 4e Base shear vs. Roof Displacement for Fourteen Storey EBFs

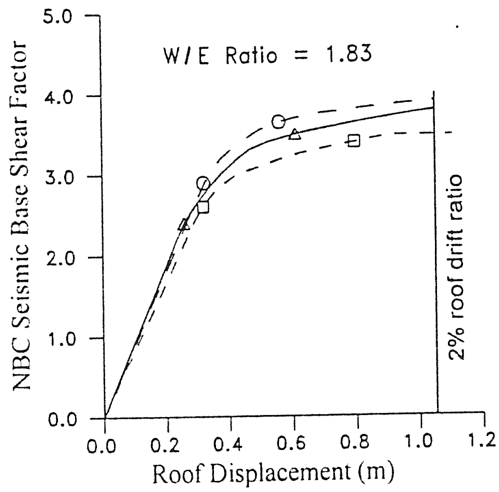

Figure 4f Base Shear vs. Roof Displacement for Fourteen Storey EBFs

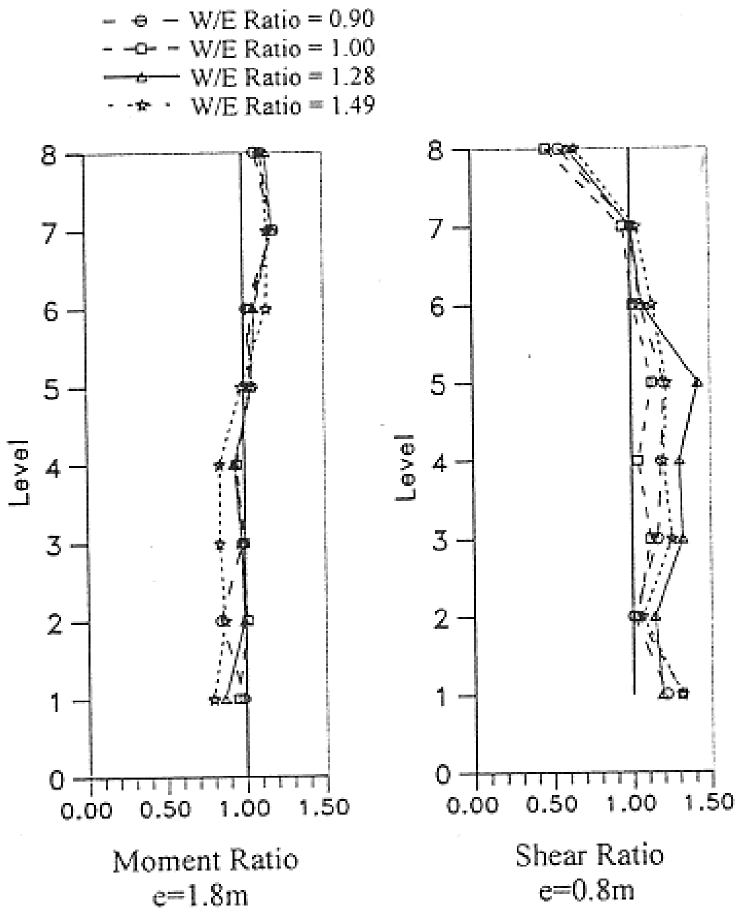

Figure 5 Link Forces in Dynamic Response
(Maximum from Six Ground Motions)

Maximum link rotations are given in Table 4. Since the code specified limit varies with beam section, the values are also expressed as proportions of the relevant limit value. Except for the four-storey structure, all maxima are lower than their limit, and for the eight and fourteen storey frames they are less than 30% of the limit for short links. Only for intermediate and long links are they a significant proportion of the limit. The four-storey structure has high link deformations; the excessive values occur at one level, with other levels respecting the limits.

Maximum response forces in braces and columns were all less than the member resistances, indicating that with the reduced link overstrength factors satisfactory performance is achieved. Beam outer segments experienced some yield at the link-brace end. This was associated with shorter links, suggesting that the greater gravity load effect due to the longer outer segment combined with the seismic loads caused this yield. Most plastic hinges at this location underwent small rotations, although a small number were comparable with the link rotations.

Several eight storey frames were examined under more severe earthquakes (a factor of 1.5 times the Zone 2 PGV [7]). Maximum link forces increased only slightly and caused no brace overload. Link deformations were more significantly affected: flexural link maximum deformations reached twice the code specified limit in some cases, whereas for short links they were within the limits.

Detailing of Link

In view of the provisions in the design code and the above analytical results (Table 4), less ductile details of a link may be sufficient with the lower maximum response values expected for some of the eight and fourteen storey frames. On the other hand the high responses for the four storey frames suggest that the link detailing for these needs to comply with the current code provisions. For the eight storey frames with 1.8m link length, the link maximum response rotations are not significantly lower than their rotation limits. Because of greater uncertainties in predicting the rotation capacities of long links[3], relaxation of link detailing for moment links in moderate seismic zones is not recommended. The links with intermediate lengths combine bending with shear behaviour, and this complex situation is currently less adequately modelled than for short or long links. Therefore, no suggestion is made that the detailing for this type of link should be relaxed from the current code requirements until further studies are made.

Link stiffener spacing for the eight storey EBFs with 0.8m link lengths was examined using criteria for the initiation of web buckling [5] and based on the maximum responses to moderate seismic loads. In Clause 27 of CSA-S16-01 (2005)[2], spacing of equally spaced stiffeners within a link is expressed as $a=\delta w-0.2d$ (where w is web thickness, and d is depth of link section). δ is 30 for a shear link with rotation angle of 0.08 radian when $e\leq 1.6M_p/V_p$, or 52 when the rotation angle is 0.02 radians or less.

The maximum link rotation value obtained for each of the frames studied was used to estimate the necessary stiffener spacing based upon the above-mentioned code provisions. The results show that for the EBFs with W/E ratios, 0.9, 1.0 and 1.49, the values of spacing, a, are large, i.e., a/e>0.78. On the other hand, for the EBF with W/E ratio 1.28, the ratio a/e is 0.36, suggesting that δ may be taken as 60 for shear links when seismic load is not in control. This value is quite conservative.

CONCLUSIONS

The scope of this study is limited to three different height structures with identical floor plan, and is based on the predicted behavior under the action of a limited number of ground motions. Within this scope, it is shown that in a moderate seismic geographical location satisfactory EBF designs can be achieved with lower link overstrength factors than currently specified in the Canadian Standard. The longer the link the lower the factor that can be justified. There is little evidence that the overstrength factors depend on the variation in wind load within moderate seismic geographical locations. Analyses of several frames under very severe earthquake loads showed that braces would remain stable but link deformations for long links would exceed the code limits. The small link deformations experienced by short links indicates the possibility of reduced requirements for link web stiffening for these links in moderate seismic geographical locations.

ACKNOWLEDGEMENTS

This work was supported by the Steel Structures Education Foundation and the Fonds pour la Formation de Chercheurs et l' Aide a la Recherche of the Province of Quebec., Professor Richard Redwood, McGill University, Professor K. Kasai, Tokyo Institute of Technology and Professor, J. Ricles, Lehigh University are thanked for their interest and provision of software. The advice of Alfred Wong, P.Eng., CISC, concerning structural configurations is also gratefully acknowledged.

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