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An approach to seismic design of eccentrically braced frames

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ABSTRACT

An alternative approach to seismic design of eccentrically braced frames is presented. An iterative design procedure that includes a non-linear time history into design process is discussed and necessary steps are outlined. Analytical tools to improve the efficiency of the procedure are developed and described. The selection of the appropriate acceleration record to carry out the analysis is studied for Victoria, B.C. Methodology is developed to generate artificial records that reflect some local seismic condition of the selected location. The application of the procedure is illustrated for eight-story frame with chevron type of eccentric bracing. Results presented suggest that frames designed using this new approach compared to the ones designed in accordance with the present code requirements show improved behaviour, especially regarding the response of columns and braces.

INTRODUCTION

Eccentrically braced steel frames (EBFs) are designed to dissipate energy by inelastic deformations of short sections of the beams called links. Columns, braces and outer beam segments are expected to respond primarily elastically, and thus less ductile modes of failure such as column or brace buckling are inhibited.

Study of non-linear dynamic behaviour of several EBFs designed for different locations in Canada (Koboevic and Redwood, 1997) has indicated possible deficiencies in current design procedures. For structures in severe seismic zones higher link shear forces than those anticipated in design, led to yield in outer beam segments, extensive overloading of columns and of a small number of braces, all in upper storeys of the frame. The desirable feature of uniform energy dissipation over the height of the structure was not always achieved.

This paper presents a study of an alternative iterative design procedure for EBFs. The proposed approach incorporates non-linear time history analysis into the design process and leads to EBFs with more desirable seismic performance.

SUMMARY OF PROPOSED DESIGN PROCEDURE

The proposed procedure can be summarized as follows: Link sections are first selected to support the factored seismic load. Shear forces in links are calculated assuming rigid-plastic mechanism. All the other elements of the frame are designed for the relevant load combinations. Next, non-linear analysis is carried out for a selected ground motion record. Links and the outer segments of the beams are modeled as inelastic elements. Columns and braces are modeled as elastic since avoidance of distress in columns and braces is a design objective. The response of the members for which no yielding or loss of stability is admissible is then examined to find the peak response forces during the complete time history. If at any time step design requirements are not met, members are re-designed to provide adequate resistance. Re-analysis with the modified properties of the revised members is then carried out, followed by a check of the resistance and the revision of member sections. The procedure is repeated until convergence is reached, that is, all the members have the desired behaviour for the chosen earthquake record.

Some preliminary results for EBF designed using this procedure have been reported elsewhere (Koboevic and Redwood, 1997). The present paper addresses a more systematic approach to the application of the procedure. This involves improving the efficiency of the analytical tools and defining ways to select an appropriate earthquake record to use for the design process.

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IMPROVING THE EFFICIENCY OF THE PROCEDURE

The application of the above procedure was made practical by improving the efficiency of the analytical tools. The method has been implemented in three separate computer programs which have the following functions: (i) an non-linear time-history module, (ii) a post-analysis module which examines the response forces in elements, compares with resistances and chooses alternative sections if resistances are exceeded and (iii) a data preparation module to revise the input file for analysis, using the results of (ii) and a prepared input data template.

Non-linear time-history analysis is carried using the program ANSR-1 (Mondkar and Powell, 1975) employing the shear link element formulated by Ricles and Popov (1987, 1994). The programs were developed for use with this particular computer code but they could easily be adapted for other analysis programs.

Post-analysis module

Braces, columns and beam segments outside the links will generally carry both moments and axial forces (tension and compression) and are therefore treated as beam-columns when carrying out the verifications for strength and stability. The post-analysis program provides check of interaction equations for axial force and bending moment according to CSA (1994) design requirements. If at any time step the response ratio for a frame element considered exceeds allowable limits, a selection of sections with satisfactory resistance is provided and the user selects one of these. To ensure economical selection, only sections with response ratios between 0.85 and 1.00 are listed. All verifications for subsequent time steps are carried out with the updated section properties.

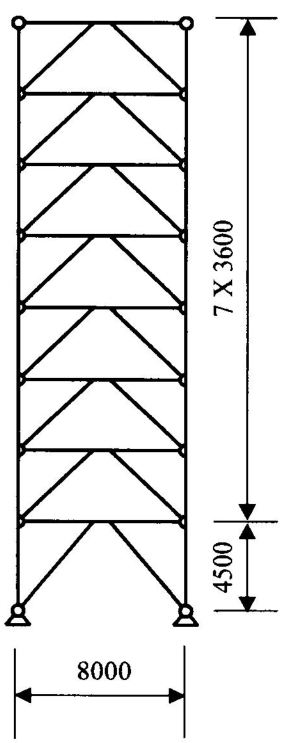


Fig. 1. EBF configuration

The program has been developed for chevron EBF configurations as illustrated in Fig. 1. The number of storeys can vary and column tiering can be imposed. Two different cases can be treated: Case 1, where all the elements other than links are expected to have elastic response, and Case 2, where yielding is also permitted in outer beam segments. Studies have demonstrated that some limited yielding in these elements can be accepted. The program provides several output files such as history of section selection, maximum response ratios for all considered elements, information on critical axial force and bending moment and their time of occurrence for each member, and the final sections selected. The process of member selection can be fully automated, but the authors prefer the greater flexibility and engineering input that interactive use offers.

A complete iteration consists of running the ANSR-1 program, post-analysis program and data modification program in sequence. The three steps are automatically repeated until there are no more section modifications. The post-analysis module in combination with data preparation module significantly reduced the time needed to complete the design process since several iterations may be needed for convergence. The programs provided tools for efficient design and further study of the EBF seismic behaviour.

Sensitivity of the procedure to the choice of the initial structure

The sensitivity of the iterative procedure to the choice of initial structure is an important consideration. Ideally, for the same modeling assumptions and earthquake record chosen, the convergence should always be toward a unique structure regardless of initial frame member selection. The sensitivity of the procedure was studied on three different EBF frame configurations, with four, eight and fourteen storeys. Columns (a) and (c) of Table 1 summarize member sections of two initial structures for the eight-storey frame shown in Fig. 1. The first structure fully complies with strength, stiffness and ductility requirements of CSA (1994), hereafter denoted as code requirements. The second structure has link beams identical to those in the first structure, while the brace and columns sections previously selected for the bottom storey are kept uniform in all other storeys. Yielding of the outer beam segments was allowed so only brace and column sections were modified in the iterative procedure. The final sections selected for both structures are given in columns (b) and (d) of Table.1.

The structures show a lot of similarity, with difference in mass of less than 3%. A somewhat heavier top tier column section was selected in the second final structure, and a small difference is observed in brace sections at level 6. This variation is due to the range of acceptable response ratios that was set in the post-analysis program to define an economical selection. Similar results were obtained for four and fourteen storey frames.

Table 1. Convergence of design process

STOREY	(a) INITIAL STRUCTURE	(b) FINAL STRUCTURE	(c) INITIAL STRUCTURE	(d) FINAL STRUCTURE	
7-8	W200X52	W250X67	WWF450X228	W250X80	COLUMNS
5-6	W310X107	W310X118	WWF450X228	W310X129	
3-4	WWF350X176	WWF350X155	WWF450X228	WWF350X155	
1-2	WWF400X273	WWF450X228	WWF450X228	WWF450X228	
8	HSS178X178X10	HSS178X178X13	HSS305X305X11	HSS178X178X13	BRACES
7	HSS203X152X10	HSS254X152X13	HSS305X305X11	HSS254X152X13	
6	HSS254X152X11	HSS305X203X8	HSS305X305X11	HSS305X203X10	
5	HSS305X203X10	HSS305X203X10	HSS305X305X11	HSS305X203X10	
4	HSS305X203X11	HSS305X203X10	HSS305X305X11	HSS305X203X10	
3	HSS305X305X10	HSS305X203X10	HSS305X305X11	HSS305X203X10	
2	HSS305X305X10	HSS305X203X11	HSS305X305X11	HSS305X203X11	
1	HSS305X305X10	HSS305X305X11	HSS305X305X11	HSS305X305X11	

The study showed that for widely different initial frame members the iterative procedure, for a given frame geometry and earthquake record, always yield the same structure. It was concluded that the procedure is not sensitive to the initial frame member selection.

SELECTION OF ACCELERATION RECORD

Another important question is the selection of the appropriate earthquake record for which to carry out the process. An attempt was made to generate artificial records that would reflect relevant local seismic conditions. The following approach was adopted:

(i) Program EQDES (Tremblay,1994) is used to determine the earthquake magnitudes and distances that contribute most strongly to hazard at the selected location and are capable of producing peak horizontal acceleration (PHA) or velocity (PHV) equal to the design intensity of the site. For the location of interest (Victoria, B.C.) it was found that earthquakes of magnitude between 6.5 to 7.0 at distance from 50 to 75 km mainly contribute to PHV.

(ii) Earthquake records with appropriate combination of magnitude and distances are selected from Earthquake Strong Motion Database (NGDC,1996). Free-field accelerograms recorded on firm ground were the preferred choice. Since records were to be representative of Western North America, only those with low or intermediate a/v ratio are retained. A list of selected records is given in Table 2.

(iii) Response spectra for selected scaled records are calculated next. Accelerograms are scaled so that the peak ground velocity of the earthquake record corresponds to the one of the site, while maintaining the a/v ratio of the record unchanged. As can be seen in Fig. 2, response spectra derived for low a/v and intermediate a/v group of records show significant differences. It was decided to treat separately these two groups of records.

(iv) Smooth pseudo-velocity spectra are constructed following the approach described by Cleveland (1979). Program SIMQKE (1976) is employed to generate artificial records to match targeted spectra. For each spectrum three different acceleration records were generated. Each earthquake simulation is unique as the seed number is changed for each generation. In addition, an artificial record matching NBCC pseudo-velocity design spectrum was produced for purpose of comparison. A Study reported by Christopoulos (1998) provided guidance in determining the appropriate duration and intensity functions. Data derived for the historical set of records were incorporated in evaluation of input parameters. The analysis of frequency content of the historical accelerograms indicated a predominance of low frequencies (0-5Hz) and these were the frequencies represented in the simulations. An attempt was made to obtain simulated records with peak ground velocity equal to the one at site (PGV= 0.3m/s) and the a/v ratio within the limits range used to classify accelerograms ($1.2 < a/v < 0.8$ and $a/v < 0.8$ for low and intermediate groups respectively)

Table 2. Summary of selected historic earthquake records

Record, date (D/M/Y), recording site	Abbreviation	Component	PHA (g)	PHV (m/s)	a/v	
Alaskan subduction eq., 15/10/1965, Kodiac Naval Stat.	AL1	N260E	0.022	0.033	0.67	Low a/v
Loma Prieta eq., 18/10/19989, Crystal Springs reservoir	LPC1	137	0.117	0.171	0.68	
Loma Prieta eq., 18/10/19989, Crystal Springs reservoir	LPC2	227	0.108	0.187	0.58	
Coalinga eq., 05/02/1983, Parkfield, Goldhill 2W	C1	East	0.074	0.121	0.61	
Coalinga eq., 05/02/1983, Parkfield, Goldhill 2W	C2	West	0.083	0.115	0.72	
Loma Prieta eq., 18/10/19989, Stanford University lab.	LPS1	270	0.202	0.367	0.55	
Alaskan subduction eq., 15/10/1965, Kodiac Naval Stat.	AL2	N350E	0.017	0.019	0.89	Intermediate a/v
Milford Sound eq., 04/05/1976, Milford Sound hotel	MS1	N49E	0.080	0.083	0.96	
Milford Sound eq., 04/05/1976, Milford Sound hotel	MS2	S41E	0.090	0.100	0.90	
Northridge eq., 17/01/19994, Griffith observatory	NGO1	270	0.297	0.257	1.15	
Northridge eq., 17/01/19994, Griffith observatory	NGO2	360	0.167	0.139	1.20	
Northridge eq., 17/01/19994, Huntington beach	NHB1	360	0.120	0.111	1.08	
Northridge eq., 17/01/19994, Hungtington beach	NHB2	270	0.112	0.104	1.08	
Loma Prieta eq., 18/10/19989, Stanford University lab.	LPS2	360	0.288	0.284	1.01	

Comparison of historic and generated accelerograms

To validate if the generated artificial records can be considered representative of Western North American events, a number of indices characterizing earthquake records are calculated and compared to historical ones. Further evaluation of generated records was based on the results of non-linear analysis of an EBF structure. The response of links is calculated in terms of maximum induced shear forces and maximum range of shear deformations (i.e. the sum of maximum positive and maximum negative shear strains). The mean values are compared for two sets of records with different a/v characteristics as shown in Table 3. Results obtained indicate good agreement of structural non-linear response parameters for historic and generated records.

Table 3. Comparison of response parameters for historic and generated records

STOREY	HISTORIC RECORDS				GENERATED RECORDS			
	(a) LOW a/v		(b) INTERMEDIATE a/v		(c) LOW a/v		(d) INTERMEDIATE a/v	
	Mean V_{max}/V_r	Mean Max. range γ (rad)	Mean V_{max}/V_r	Mean Max. range γ (rad)	Mean V_{max}/V_r	Mean Max. range γ (rad)	Mean V_{max}/V_r	Mean Max. range γ (rad)
8	1.73	0.110	1.85	0.180	1.77	0.118	1.89	0.202
7	1.83	0.109	1.71	0.080	1.79	0.108	1.71	0.086
6	1.73	0.085	1.51	0.046	1.74	0.075	1.56	0.050
5	1.59	0.060	1.25	0.019	1.58	0.042	1.32	0.018
4	1.60	0.056	1.24	0.017	1.58	0.045	1.30	0.013
3	1.57	0.034	1.30	0.012	1.53	0.030	1.28	0.008
2	1.63	0.054	1.43	0.022	1.60	0.044	1.31	0.040
1	1.62	0.069	1.44	0.039	1.67	0.060	1.32	0.030

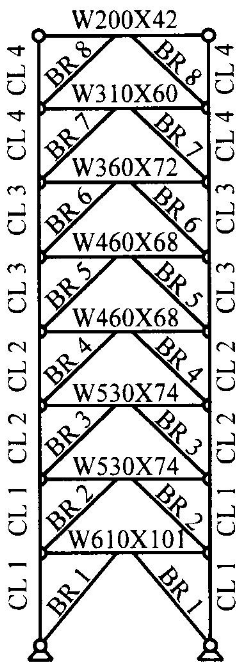
ILLUSTRATION OF THE APPLICATION OF THE PROCEDURE

The application of the procedure will be illustrated with an example of eight-storey EBF located in Victoria B.C. Initial sections, selected to fully comply with strength and ductility design code requirements, are illustrated in Fig. 2 (a). Non-linear response of this structure was then examined for the historic records listed in Table 2. Columns and braces were modeled as elastic. The values of link response parameters are those indicated in columns (a) and (b) of Table 3. The extent of overload of braces and columns was also evaluated and the results for all fourteen historic records are summarized in Table 4. The extent of overload was monitored separately for those two element groups. It is expressed in terms of the summation of the number of time increments (each of 0.04s, this being the frequency with which the output was saved) during which any column or brace in the frame was subjected to forces greater than their nominal resistances for a particular earthquake record.

Table 4. Duration of excess loading: code-based designed eight-storey frame (Victoria, B.C)

	LOW a/v						INTERMEDIATE a/v							
	AL1	LPC1	LPC2	C1	C2	LPS1	AL2	MS1	MS2	NGO1	NGO2	NHB1	NHB2	LPS2
BRACES	47	357	37	81	182	23	103	52	41	55	104	83	84	51
COLUMNS	343	259	0	712	1	26	4	13	1	461	238	144	73	55

This structure was then re-designed using the previously described procedure. The assumption was made that yield in the outer beam segments is acceptable. Design was carried out based on one of the artificial records generated to match smooth response spectrum for low a/v historic records. The generated accelerogram had peak ground acceleration of 0.2g and peak ground velocity of 0.3 m/s. Four iterations were needed to obtain the final structure. The selected sections are illustrated in Fig 2(b).



	(a) initial sections	(b) sections after iterative procedure
CL1	WWF400X273	WWF400X273
CL2	WWF350X176	WWF350X192
CL3	W310X107	W310X143
CL4	W200X52	W250X67
BR1	HSS305X305X10	HSS305X305X10
BR2	HSS305X305X10	HSS305X203X11
BR3	HSS305X305X10	HSS305X203X11
BR4	HSS305X203X11	HSS305X203X13
BR5	HSS305X203X10	HSS305X203X13
BR6	HSS254X152X11	HSS305X203X10
BR7	HSS203X152X10	HSS254X152X11
BR8	HSS178X178X10	HSS178X178X13

Fig. 2. Summary of selected sections for eight-storey EBF (Victoria, B.C)

For the chosen record, columns and braces have the desired elastic response. It was of interest to see if columns and braces would exhibit similar behaviour under another excitation. The non-linear analysis of the re-designed structure was carried out for the set of historic records and results are presented in Table 5.

Table 5. Duration of excess loading: re-designed eight-storey frame (Victoria, B.C)

	LOW a/v						INTERMEDIATE a/v							
	AL1	LPC1	LPC2	C1	C2	LPS1	AL2	MS1	MS2	NGO1	NGO2	NHB1	NHB2	LPS2
BRACES	1	212	1	30	75	0	5	0	0	0	2	1	8	4
COLUMNS	0	5	0	0	0	0	0	0	0	15	16	9	3	2

The structure obtained using the iterative procedure has experienced much less distress in columns and braces compared to the structure designed according to the current code requirements. The proposed designed procedure does not adhere strictly to capacity design principles, since the design forces are based on the maximum link force developed for a selected earthquake record, and not on link capacity. It does however lead to improved dynamic response, and may usefully be combined with an examination of the structure from the viewpoint of capacity design.

It should be noted that some reduction of inelastic shear deformation of links was observed, but maximum link shear forces still exceed those anticipated by the code ($1.5V_r$), mainly in the upper storeys of the frame. Additional studies are necessary to evaluate realistic magnitudes of the inelastic shear force that the link can develop.

CONCLUDING COMMENTS

A design procedure to improve overall seismic response of eccentrically braced frames has been presented. The approach is based upon the seismic response of the frame and incorporates non-linear time history analysis. Basic steps of the method are outlined, and analytical tools developed to enhance the efficiency of the procedure are discussed. It is shown that the procedure is not sensitive to the initial member selection. The methodology to select an appropriate earthquake record to use in the analysis is suggested. The application of the procedure is examined for an eight-storey EBF. The proposed approach led to a significant reduction in overload of the members other than link beams compared with the code-based design. Verification of the proposed procedure for different frame heights and seismic locations is the subject of on-going study.

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