MODIFICATION OF DESIGN FORCES IN ORDER TO AVOID BRACE YIELDING IN ECCENTRICALLY BRACED STEEL FRAMES

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ABSTRACT :

In capacity-based design of eccentrically braced steel frames according to AISC-Seismic Provisions (2005), while the links are sized for the load combinations specified by the related code, the members outside of the links are designed to resist the forces generated by fully yielded and strain hardened links. However, after carrying out inelastic static and dynamic analyses of 18 model frames designed according to LRFD (1999) and ASCE 7-05, it is observed that plastic hinges can develop at the braces of some of these frames before the limit state condition of AISC-Seismic Provisions is reached. Both static and dynamic inelastic analyses are carried out with the DRAIN-2DX program. One of the reasons of brace yielding is found out to be the distribution of inertia forces along the building height which is different from the distribution of the equivalent lateral design forces for high-rise frames (higher mode effects). The other reason is the difference of the distributions of the member forces in the elastic and inelastic frame analyses which the former is utilised during the design procedure. In this paper, the design forces of braces of such frames are modified to avoid brace yielding.

KEYWORDS: Eccentrically braced steel frame, capacity based design, design force, inelastic analysis.

1. INTRODUCTION

According to the capacity-based design of eccentrically braced frames (EBFs), inelastic action is restricted to the links and the elements outside of the links are designed to sustain the forces generated by the fully yielded and strain hardened links. Such kind of a design procedure, should guarantee that the links provide stable hysteretic behavior under extreme seismic events and preserve the integrity of the surrounding elements. The research carried out by Popov and his co-workers during the 1980s have shown that a properly designed and detailed link element can provide high deformation capacity. In addition, designing the diagonal braces, the columns, and the beam segments outside of the links to remain essentially elastic under the maximum forces delivered by the links preserves the integrity. In fact, preserving integrity allows taking advantage of the full inelastic capacity of the links.

In Seismic Provisions for Structural Steel Buildings (AISC, 2005), all of the fore mentioned principles are adopted. On the other hand, it is stated that limited yielding outside of the links, particularly in the beams, is sometimes unavoidable in an EBF. The yielding of beam segments does not seem problematic due to the beneficial effects of the floor slab which is not involved in the analytical model of the frame, but yielding of the braces is thought to be unacceptable since the brace cross sections are not mandated to be seismically compact in the code. On the other hand, seismically compactness limit of flanges and webs of HSS sections is drastic and uneconomic, so it is more preferable to increase brace design forces to avoid brace yielding under any circumstance.

In this research, the braces of the model frames satisfy the LRFD compactness condition (not the seismically compactness condition of AISC), nevertheless brace yielding occurred for some of the high-rise model frames with shear links before the links reach their full plastic capacity during inelastic dynamic analyses. Inelastic dynamic analyses of all the model frames are repeated by changing the scale of each earthquake record till one of the links of each frame reaches to its rotation capacity or till yielding of a brace. For each frame in which brace yielding occurred, the design procedures and inelastic analyses are repeated iteratively by small increase ratios of the design forces of all the braces till the yielding is prevented. Inelastic static analyses are also carried out.

2. MODEL FRAMES

18 EBFs are designed according to AISC Seismic Provisions (2005) and LRFD (1999). The loads and load combinations are compatible with ASCE 7-05. The spectral response acceleration diagram is given in Figure 1. The general properties of the model frames are given in Table 2.1 and Figure 2. 12 Number of model frames are composed of shear yielding links and 6 number of model frames have flexural yielding links.

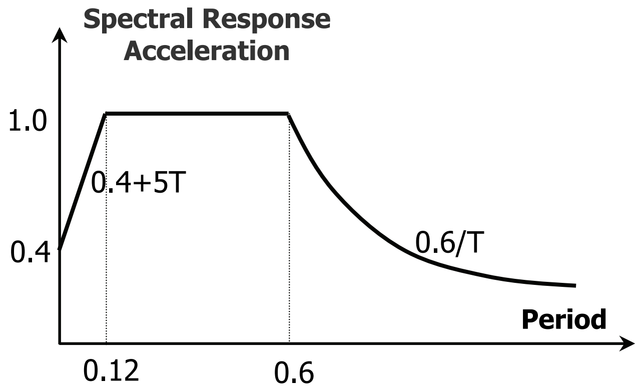

Figure 1 Spectral response acceleration diagram

Although, the frames are the components of symmetrical buildings in plan and there is no eccentricity between the mass and rigidity centers, the effect of the accidental torsion is added to be consistent with the code. All the framing except for the EBFs are simple. There is a clear separation of functions in which all of the horizontal loads are resisted by the EBFs. All the EBFs of a building are assumed to be identical. Steel types, those are convenient with the sections are chosen; A992 steel is used for the beams and the columns, A500-Grade B steel is used for the braces (McCormac and Nelson, Jr., 2003). The entire AISC wide-flange section database is assumed available for beams. Column and brace sections are chosen from W14 series and from rectangular hollow sections with equal depth and width values, respectively.

2.1. Brief Description of the Capacity-Based Design

In capacity-based design of eccentrically braced steel frames according to AISC-Seismic Provisions (2005), while the links are sized for the load combinations specified by the related code, the members outside of the links are designed to resist the forces generated by fully yielded and strain hardened links.
The required shear strength of the link V_u shall not exceed the design shear strength of the link given in Eqn.2.1:

$$\phi V_n \geq V_u \tag{2.1}$$

$$\phi = 0.90$$
$$V_p = 0.6 F_y A_w$$
$$A_w = (d_b - 2t_f) \times t_w$$

Here, V_n = Nominal shear strength of the link, equal to the lesser of V_p or $2M_p/e$; M_p = nominal flexural strength of the link; F_y =specified minimum yield stress of the steel; A_w = link web area; d_b = overall beam depth; t_f = thickness of flange; t_w = thickness of web.

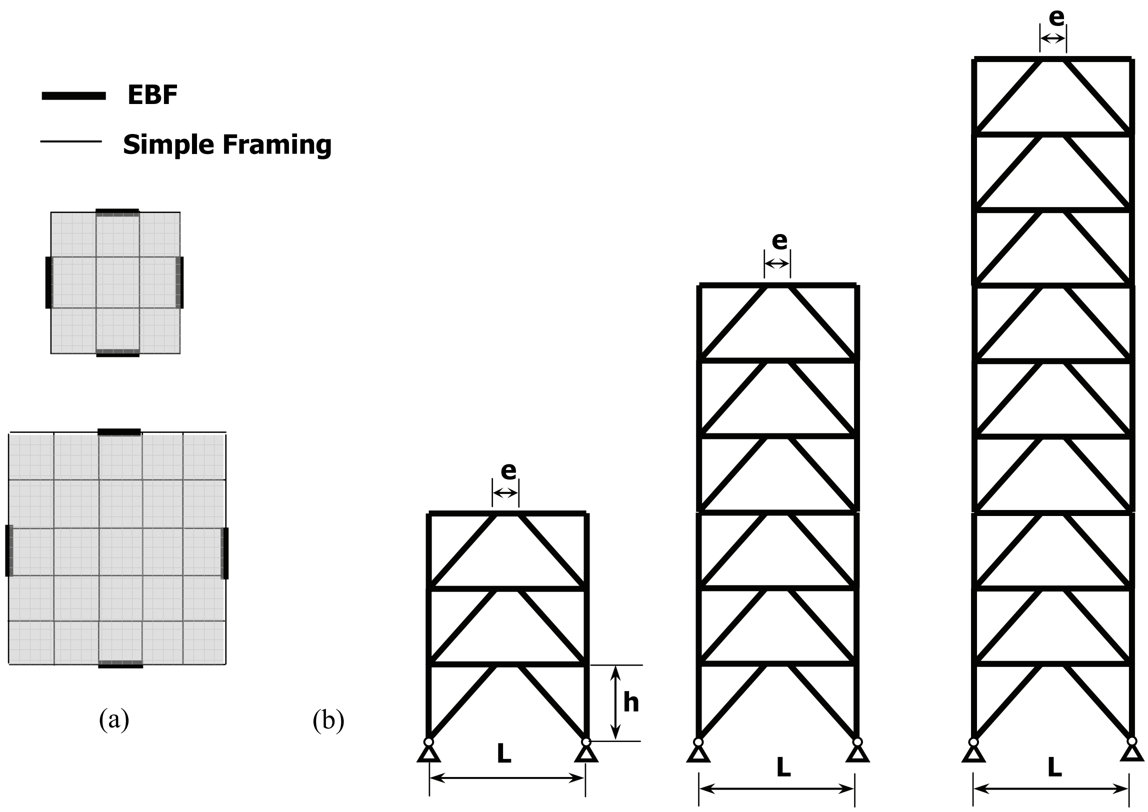

Figure 2 Properties of the model frames (a) Plan views of buildings (b) Elevation views of frames

Table 2.1 Properties of the model frames

Frame Name	Number of stories	Plan area (m^2)	Link yielding type	L (m)	e (cm)	h (m)
FR-01	3	600	SHEAR	7	90	3.5
FR-02	3	600	SHEAR	7	120	3.5
FR-03	3	800	SHEAR	7	90	3.5
FR-04	3	800	SHEAR	7	120	3.5
FR-05	6	600	SHEAR	7	90	3.5
FR-06	6	600	SHEAR	7	120	3.5
FR-07	6	800	SHEAR	7	90	3.5
FR-08	6	800	SHEAR	7	120	3.5
FR-09	9	600	SHEAR	7	90	3.5
FR-10	9	600	SHEAR	7	120	3.5
FR-11	9	800	SHEAR	7	90	3.5
FR-12	9	800	SHEAR	7	120	3.5
FR-13	3	250	FLEXURE	8	280	3.5
FR-14	3	250	FLEXURE	8	320	3.5
FR-15	6	250	FLEXURE	8	280	3.5
FR-16	6	250	FLEXURE	8	320	3.5
FR-17	9	250	FLEXURE	8	280	3.5
FR-18	9	250	FLEXURE	8	320	3.5

The required strength of the diagonal braces of an EBF can be taken as the forces generated by the following values of the link shear and the link end moment (AISC, 2005):

$$\text{Link shear} = 1.25 \times R_y \times V_p \tag{2.2}$$

$$\text{Link end moment} = e \times (1.25 \times R_y \times V_p)/2 \tag{2.3}$$

R_y = Ratio of the expected yield strength to the minimum specified yield strength and 1.25 is the factor considering the strain hardening.

For model frames, the beam segments and the link are a single continuous wide flange member, so the design strength of the beam can be increased by R_y. Therefore, the required strength of the beam segments can be determined based on the forces generated by the link forces given by the following simplified equations (AISC, 2005):

$$\text{Link shear} = 1.1 \times V_p \tag{2.4}$$

$$\text{Link end moment} = e \times (1.1 \times V_p)/2 \tag{2.5}$$

The above forces may be distributed to the brace and the beam segment joining at the link end, based on the distribution ratios of the global elastic analysis under the effect of lateral load. The other necessary ratios can be estimated by the elastic analysis as well. Consequently, the resultant design forces will be approximate, since they are based on the elastic range not the inelastic range.

The design procedure of columns is quite different. Columns of low rise EBFs with only a few stories should have the axial design strength to sustain the $1.25 \times R_y$ times the nominal shear strength of all the links above the level of the column under consideration. This is based on the premise that all the links simultaneously reach their maximum strength, but for multi storey EBFs, it is stated in the code that this has a smaller likelihood so a factor of 1.1 is permitted to be used instead of 1.25. For model frames of this study, a factor of 1.1 is used. During design of the columns amplified seismic load combinations are also applied without consideration of any concurrent flexural loads on the columns.

2.2. Link Types

The choice of the link length will effect the yielding behavior of the link as given in Eqn. 2.6:

$$\begin{aligned} & e \le \frac{1.6M_p}{V_p} && \Rightarrow \textit{shear yielding} \\ & e \ge \frac{2.6M_p}{V_p} && \Rightarrow \textit{flexural yielding} \\ & \frac{1.6M_p}{V_p} < e < \frac{2.6M_p}{V_p} && \Rightarrow \textit{combination} \end{aligned} \tag{2.6}$$

2.3. Coded Design Program

The chosen sections are the minimums, which satisfy the necessary conditions. This was achieved by a coded design program. The reader is referred to (Özhendekci and Özhendekci, 2008) for the basic characteristics and the algorithm of the program.

3. INELASTIC ANALYSES

Inelastic dynamic analyses of each of model frames are performed under 20 SAC ground motions with DRAIN-2DX whose input files are prepared automatically by the program coded. These are the ground motions of Los Angeles with 2% probability of exceedance in 50 years. Each earthquake record for each frame is scaled by the program by modifying the DRAIN input files in an automated manner until one of the links reaches the code-based limit rotation angle or brace yielding. Inelastic static analyses are also performed till the limit states; however brace yielding does not occur during pushover analyses. Modified plastic link-rotation equation including cumulative vertical displacements of the column ends is used in the inelastic analyses (Özhendekci and Özhendekci, 2008).

3.1. Inelastic Link Element Model

Richard and Uang's link element model is utilised in this study, the accuracy of which is verified by Richards with a correlation study comparing the analytical and experimental results for some of the UTA links with A992 steel (Richards, 2004). According to this model and the link section properties, links can develop shear strengths higher than the ones used during the design of braces (Eqns. 2.2, 2.3). The shear strength and vertical displacement relationship of the link element model is given in Figure 3. Although, aforementioned design forces are consistent with the code, it is also stated that the strain hardening ratio of the link shear forces may be in excess of the given values, and the designers can use strain hardening ratios higher than 1.25 for braces. According to the code, one can also use inelastic analyses directly in order to accomplish the design philosophy of the EBFs, but such kind of a design procedure is rather time-consuming and needs a good knowledge of building simulation for inelastic behaviour. Hence, it seems reasonable to provide increase ratios for design forces of braces based on the inelastic analyses of various frames under various earthquake loads. These increase ratios will render possible a safe and simple elastic design procedure.

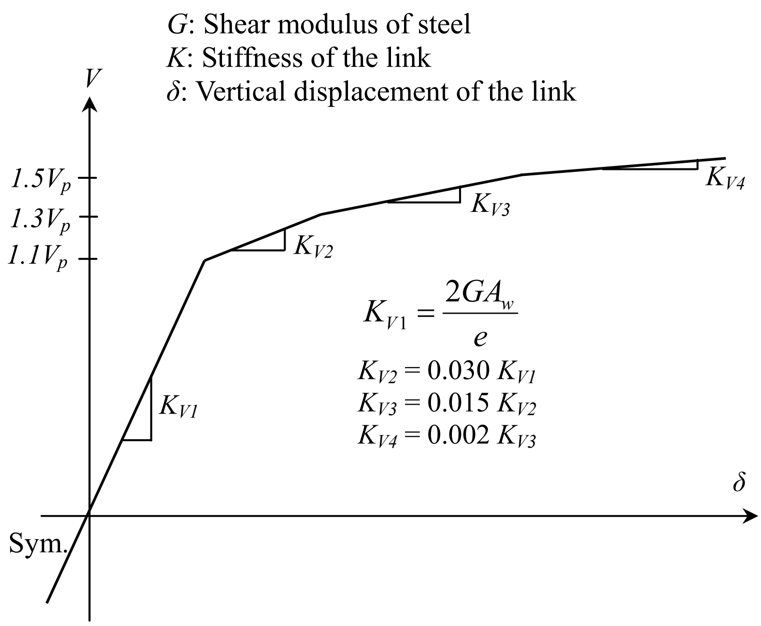

Figure 3 Shear-displacement relationship of the link model (Richards and Uang, 2004)

Table 3.1 Properties of the earthquake records

Record Name	Magnitude	Distance (km)	Duration (s)	PGA (cm/s^2)
1995 Kobe	6.9	3.4	59.98	1258.00
1995 Kobe	6.9	3.4	59.98	902.705
1989 Loma Prieta	7	3.5	24.99	409.95
1989 Loma Prieta	7	3.5	24.99	463.76
1994 Northridge	6.7	7.5	14.945	851.62
1994 Northridge	6.7	7.5	14.945	925.29
1994 Northridge	6.7	6.4	59.98	908.70
1994 Northridge	6.7	6.4	59.98	1304.10
1974 Tabas	7.4	1.2	49.98	793.45
1974 Tabas	7.4	1.2	49.98	972.58
Elysian Park (simulated)	7.1	17.5	29.99	1271.20
Elysian Park (simulated)	7.1	17.5	29.99	1163.50
Elysian Park (simulated)	7.1	10.7	29.99	767.26
Elysian Park (simulated)	7.1	10.7	29.99	667.59
Elysian Park (simulated)	7.1	11.2	29.99	973.16
Elysian Park (simulated)	7.1	11.2	29.99	1079.30
Palos Verde (simulated)	7.1	1.5	59.98	697.84
Palos Verde (simulated)	7.1	1.5	59.98	761.31
Palos Verde (simulated)	7.1	1.5	59.98	490.58
Palos Verde (simulated)	7.1	1.5	59.98	613.28

4. RESULTS

Brace yielding does not occur in any of the model frames with flexural links during the inelastic analyses. It occurs at 4 frames among the EBFs with shear links during inelastic dynamic analyses. These frames are **FR-06**, **FR-08**, **FR-10** and **FR-12** all of which are high-rise and have relatively long shear links. The earthquake records causing the brace yielding and the number of them are different for each EBF. The level of the storey of the yielding brace can also change from earthquake to earthquake. One of the reasons of brace yielding is the distribution of inertia forces along the building height which is different from the distribution of the equivalent lateral design forces for high-rise frames. The other reason is the difference of the distributions of the member forces in the elastic and inelastic frame analyses which the former is utilised during the design procedure.

For some of the earthquake records, brace yielding occurs when the link barely reaches 75% of its rotation capacity (0.06 rad). After repeating the designs and inelastic analyses iteratively by small increase ratios of design forces of all the braces of each frame, the conservative increase ratio is found out to be 10%. Namely, multiplying the Eqns. (2.1) and (2.3) with 1.1 will prevent the yielding of braces and it will be possible to take advantage of the full inelastic capacity of the links.

5. CONCLUSIONS

Consequently, it is not safe to permit the yielding of braces as long as they do not satisfy the seismically compactness condition. Besides, yielding of braces is not a favourable thing which decomposes the integrity of the surrounding framing outside the links where the links are in inelastic range. Being consistent with the design philosophy of EBFs: *Inelastic action is restricted to the links, and the elements outside of the links are designed to sustain the forces generated by the fully yielded and strain hardened links till they reach their rotation capacity* is the way to handle the problem easily. In this research, the braces are chosen from the sections which satisfy the LRFD compactness limit and 10% increase for the design forces of braces of EBFs with shear yielding links is proposed. In addition, it can be reasonable to choose noncompact and slender sections as long as they can sustain the forces generated by the fully yielded and strain hardened links till the links reach their rotation capacity. However above mentioned increase factor should also be investigated with noncompact and slender sections.

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