Eccentric Braced Frame Design for Moderate Seismic Regions

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INTRODUCTION

Recent discussions related to the seismic performance of low-ductility steel systems designed for moderate seismic regions have generated new interest in the cost-effective design of ductile systems for such regions. Concentrically braced frames (CBFs) are prevalent in moderate seismic regions, both because of their high stiffness-to-weight ratio, and because of the ease with which they can be designed and evaluated by the equivalent lateral force method. Eccentrically braced frames (EBFs) can offer the same advantages as CBFs, while also providing significant ductility capacity, and greater flexibility with architectural openings. The office of LeMessurier Consultants designed several eccentrically braced frames for prominent buildings in the United States from the 1960’s through the 1980’s. For these structures, designed prior to modern seismic provisions, eccentric bracing was the most efficient means to developing a stiff lateral system that accommodated the architectural program. Since the branding of eccentric bracing as a high-seismic, high-ductility system in the 1990’s however, use of these schemes has tapered off to almost non-existent—restricting the potential both for working creatively with architects and for achieving moderate levels of ductility in the lateral force resisting system (LFRS).

This paper first discusses the recent transition from the 6th Edition to the 7th Edition of the Massachusetts State Building Code and its implications for the relationship between seismic loads and wind loads. Thereafter a theoretical 9-story EBF design is developed for Boston, Massachusetts according to the 2005 AISC Seismic Provisions for the purpose of comparing its tonnage to a lower ductility CBF. Special link details are introduced that allow reduced tonnage without compromising capacity design principles.

DESIGN BASE SHEARS IN MASSACHUSETTS

The recent adoption of the Massachusetts State Building Code 7th Edition in September 2008 provides an opportunity to contrast directly base shears due to wind and earthquake loads for a particular legal jurisdiction. For this exercise, a steel frame building was selected to match the plan dimensions of the buildings studied previously by Hines et al. [2008] (based on the SAC 9-story prototype) and pictured in Figure 1. As Figure 1 shows, the prototype for this study differs from the prototype in the previous study by locating its bracing bays in the building interior for the purpose of enhancing overturning performance. The prototype building is square with 30 ft x 30 ft bays and a 1 ft slab perimeter overhang. The first story is 18 ft high, and other stories are 13 ft high. Each direction has four bays of either CBF or EBF chevron bracing. Figure 1 represents conceptually, on the left of elevation, the CBF scheme and, on the right of the elevation, the EBF scheme. The seismic weight for a typical floor is 2160 kips, based on 85 psf for the floors and 25 psf for the façade. The seismic weight for the second floor is 2200 kips to account for additional façade. The seismic weight for the roof is 2210 kips, based on 80 psf
(concrete roof slab, no partitions and 5 psf for roofing), a 4 ft parapet added to the façade, and 200 kips for rooftop equipment.

Figure 2 plots base shear with respect to building height in stories for the prototype structure according to the 6th Edition of the Massachusetts State Building Code (1997-2008). Four curves show a wide range of base shears, with seismic forces clearly exceeding wind forces even for taller structures. Base shears are plotted according LRFD load combinations for the lateral load listed. For buildings with a long plan dimension similar to the prototype and a short plan dimension less than that of the prototype, the seismic forces would decrease with respect to the wind forces in the short direction.

Wind forces assume Exposure B and a load factor of 1.3, as specified by the 6th Edition. Seismic forces assume a soil factor of $S = 1.5$, and a load factor of 1.0. Figure 2 shows seismic forces for an EBF system (diamonds) and a CBF system (+ signs). Base shears for these two systems differ because of the difference in R-factor and the difference in fundamental period. Per common experience, fundamental periods for both systems plotted in Figure 2 were amplified by 1.6, according to Section 9.5.5.3.1 in ASCE 7-02, in order to reduce the base shear.

The $R = 5$ CBF system represented in Figure 2 was the most common steel frame lateral system designed in Massachusetts under the 6th Edition. While most structures in Massachusetts with $S = 1.5$ were considered Seismic Design Category C under the 6th Edition, this code required such a CBF system to be designed as an OCBF according to the 1992 AISC Seismic Provisions. $R = 3$ systems of any kind were not allowed under the 6th Edition. See [Hines et al. 2008] for a more detailed historical account of these Massachusetts code provisions.
FIGURE 2 – BASE SHEARS FOR WIND AND SEISMIC LOADS ACCORDING TO MASSACHUSETTS STATE BUILDING CODE 6TH EDITION.

As a chevron OCBF under the 1992 AISC Seismic Provisions, braces were required to satisfy seismic compactness requirements, and be designed for $1.5/0.8 = 1.9$ times the seismic force associated with the base shear plotted in Figure 2. Connections and columns were required to be designed for 2.0 times the seismic force associated with these base shears. These amplified forces essentially required the lateral system to be designed for twice the seismic force, a prescriptive requirement that made a brief and direct appearance in the 2002 AISC Seismic Provisions. In order to reflect more accurately the effects of the 6th Edition seismic design requirements on member forces, Figure 2 also includes a plot for the CBF with the base shear multiplied by a system overstrength factor of 2.0 (squares). During discussions of R = 3 systems, initiated by the 1997 AISC Seismic Provisions, this 6th Edition OCBF became known as the “R = 2.5 system”.

Figure 2 indicates that, under the 6th Edition, seismic forces controlled most braced frame member design for most building heights. Such high seismic base shears, combined with the exclusion of R = 3 structures and the history of concern for seismic design in Massachusetts, brought seismic design and detailing requirements to the forefront of many design discussions in Massachusetts. Interestingly enough, although EBF systems promised significantly lower base shears, their branding as “high-seismic systems” prevented their common use during the 6th Edition. Three circumstances contributed to the strength of this branding:

1. EBF seismic base shears would often be exceeded by wind base shears.
2. Capacity design requirements for EBF braces beams and columns would force member designs that significantly exceed designs required by wind loads.
3. Capacity design requirements for EBF beams outside of link regions limited design flexibility.

This branding held sway over not only the opinions of the design community, but also those of the fabrication community, which perceived shear links as inherently expensive to detail.
Figure 3 shows that under the 7th Edition of the Massachusetts State Building Code (2008-present) the motivation to pursue seismic detailing has diminished significantly. Wind forces now require a load factor of 1.6 instead of 1.3. Wind pressures have increased slightly, and constant leeward pressure is required in the 7th Edition where it was not in the 6th Edition. Figure 4 compares 6th and 7th Edition exposure B wind base shears directly. Figure 3 shows seismic base shears for Site Class D (approximately correlating to $S = 1.5$). This figure replaces the old “R = 2.5 system” with an R = 3 CBF system with no seismic detailing. The new OCBF system is not shown in Figure 3, however, with $R = 3.25$ and detailing consistent with the 2005 AISC Seismic Provisions, it is not hard to imagine that the 7th Edition OCBF curve and its consequences for chevron braced frame design exceed the R = 3 curve in this figure.

Figure 3 plots the R = 3 system assuming no allowed amplification to the structural period (solid squares) for comparison to the “R = 2.5 system” in Figure 2 and Figure 5. It seems consistent with the simplistic nature of $R = 3$ not to “sharpen one’s pencil” and develop an analytical assessment of the fundamental period in order to increase the period to the allowable $T = 1.7T_a$ (6th Edition allowed a maximum amplification of 1.6). Nevertheless, the 7th Edition does not enforce the use of the lower value for the fundamental period with $R = 3$ structures. Therefore, Figure 3 also plots an R = 3 CBF design assuming an amplified value of for the fundamental period ($\times$ signs). More sophisticated engineers would likely use this second approach in order to reduce tonnage. Lower forces reduce member sizes and the $R = 3$ provision excuses the structure from any special detailing requirements. Ironically, if $R = 3$ is thought to be a design that leverages a structure’s strength instead of its ductility, such “sophistication” implies the possibility of inferior collapse performance.
FIGURE 4 – BASE SHEARS FOR WIND LOADS ACCORDING TO MASSACHUSETTS STATE BUILDING CODE 6TH AND 7TH EDITIONS.

The EBF system in Figure 3 has lower base shear values than wind for every building height except the 1-story configuration. The EBF curve increases linearly between 5 stories and 21 stories because it is driven by the 7th Edition’s minimum force requirement from ASCE 7-02. Figure 5 compares the 6th and 7th Edition seismic base shears for the CBF and EBF designs. The 7th Edition base shears are lower for all configurations due to the following: lower spectral acceleration values specified by the 2002 Hazard Maps produced by the United States Geological Survey (USGS); the attenuation of spectral acceleration according to $1/T$ in the 7th Edition as opposed to $1/T^{2/3}$ in the 6th Edition; and the use of $2/3$ of the Maximum Considered Earthquake (MCE) spectral acceleration in the 7th Edition. The relationship between these base shears appears ironic, when one considers the fact that the 7th Edition is based on a 2% in 50 year hazard while the 6th Edition is based on an approximately 10% in 50 year hazard. Both the changing attenuation relationships in recent years [Sorabella 2006] and the adoption of $1/T$ as a more realistic characterization of the constant velocity portion of the acceleration response spectrum (ARS) [BSSC 1998] contributed to this non-intuitive circumstance.

FIGURE 5 – BASE SHEARS FOR SEISMIC LOADS ACCORDING TO MASSACHUSETTS STATE BUILDING CODE 6TH AND 7TH EDITIONS.

The 7th Edition places most Site Class D structures in Design Category B, increasing the degree to which an East Coast engineer would tend to disregard the importance of earthquake hazard. At the same time, the 7th Edition places restrictions on $R = 3$ systems, including height
limits (as shown in Figure 3) and requiring connections to be designed for an amplified seismic force equal to twice the force used for member design. Ironically, some research indicates that R = 3 buildings under 100 ft high appear to be more vulnerable to collapse than taller buildings [Hines et al. 2008]. Figure 2 implies that the net result of the 7th Edition is to make wind even more dominant in the design of structural members but still to insist that structures be detailed for a minimum level of ductile capacity. Unfortunately, the type of detailing prescribed by both the 6th and 7th Editions has not been clearly established to produce its intended effect with a minimum impact on the cost of the structure. These two circumstances of reduced forces and increased detailing requirements motivate the design exercise and discussion that follows in this paper. Perhaps the branding of EBFs has been detrimental to lateral systems design in moderate seismic regions, and perhaps there is opportunity for innovation with systems whose forces are controlled by wind, but whose detailing should reflect consistently the principles of ductility and capacity design.

THE CASE FOR EBF DESIGN IN MODERATE SEISMIC REGIONS

The previous section listed three main circumstances that contributed to branding EBFs as high-seismic systems inappropriate for use in moderate seismic regions. This section addresses each circumstance and offers an alternative point of view.

1. **EBF seismic base shears would often be exceeded by wind base shears.**
   Indeed, under the 7th Edition, EBF base shears are exceeded by wind for all configurations except for that with 1-story. This means that the capacity design requirements discussed in the second item would generate artificially large braces and columns. While this is true, in the sense that the effective R-factor for the 9-story EBF design presented here is approximately 2.7 (see (1)), it also implies that EBF frames are inherently strong under expected seismic forces and therefore may not be required to be as ductile.

2. **Capacity design requirements for EBF braces beams and columns would force member designs that significantly exceed designs required by wind loads.**
   This circumstance can be mitigated by selecting link sizes that meet wind load requirements as closely as possible without compromising elastic drift requirements. For the 9-story design presented in the next section, this results in small W10x19 links at all floors except for the 2nd Floor. Link sizes are lengthened or shortened to match closely the expected shear demand from wind at a given story.

3. **Capacity design requirements for EBF beams outside of link regions limited design flexibility.**
   The small links proposed in this example necessitate that they be fabricated separately from the beams outside the link in order to allow them to act as girders. This has the advantage of strengthening these beams for capacity design purposes and stiffening them for drift control. The question becomes how these beam/link assemblies can be fabricated at reasonable expense.

9-STORY EBF DESIGN FOR BOSTON, MASSACHUSETTS

This section describes in detail the design of a 9-story EBF structure based on the prototype configuration in Figure 1. Links are selected to resemble as closely as possible those tested by Okazaki and Engelhardt [2006]. These links were constructed from A992 steel in contrast to the
links tested in the 1980s that were constructed from A36 steel. The test units themselves were wide flange sections welded to end plates and then bolted to the test setup. The details provided later in this section are assumed to imitate the details of the actual test setup almost exactly, with the exception of thinner end plates proposed for this design. The 9-story prototype configuration has a fundamental period of $T = 1.87$ sec, a seismic weight of $W = 19530$ k, an LRFD seismic base shear of $V_E = 266$ k, and an LRFD wind base shear of $V_W = 702$ kips. Hence the effective R-factor with respect to base shear can be calculated as:

$$R_{\text{eff}} = R \left( \frac{V_E}{V_W} \right) = \frac{266}{702} = 2.7$$

(1)

Table 1 lists story shears from wind and seismic forces.

<table>
<thead>
<tr>
<th>Story</th>
<th>Height (ft)</th>
<th>$F_E$ (k)</th>
<th>$V_E$ (k)</th>
<th>$K_z$ (psf)</th>
<th>$q_z$ (psf)</th>
<th>$q_h$ (psf)</th>
<th>$p$ (psf)</th>
<th>$F_W$ (k)</th>
<th>$V_W$ (k)</th>
<th>$1.6V_W$ (k)</th>
<th>$1.6M_W$ (k)</th>
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<td>67.6</td>
<td>1.05</td>
<td>17.0</td>
<td>10.7</td>
<td>27.7</td>
<td>44.3</td>
<td>44</td>
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<td>0</td>
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<td>1.01</td>
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TABLE 1 – STORY SHEARS AND OVERTURNING MOMENTS DUE TO SEISMIC AND WIND LOADS

With 4 bays of bracing in each direction, the shear in a link due to wind load at a given story is calculated as

$$V_{\text{Link}} = \frac{1.6V_W}{4} \left( \frac{h_i}{D} \right)$$

(2)

where $h_i$ = story height below link, and $D$ = depth of braced bay. The allowable link eccentricity is calculated as

$$e_a = \frac{2dM_p}{V_L}$$

(3)

Table 2 shows that for drift control, the maximum link eccentricity is defined as $e_{\text{max}} = 72\text{in.}$.

<table>
<thead>
<tr>
<th>Story</th>
<th>$1.6V_W$ (k)</th>
<th>$V_L$ (k)</th>
<th>Link</th>
<th>$e_a$ (in.)</th>
<th>$E$ (in.)</th>
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<td>R</td>
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<td>W10x19</td>
<td>253</td>
<td>72</td>
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<td>157</td>
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<td>W10x19</td>
<td>74</td>
<td>48</td>
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<tr>
<td>7</td>
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<td>35</td>
<td>W10x19</td>
<td>56</td>
<td>48</td>
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<td>702</td>
<td>105</td>
<td>W16x36</td>
<td>55</td>
<td>48</td>
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TABLE 2 – LINK SELECTION
Figure 6 shows the equilibrium conditions used to proportion the braces to resist link overstrength, $\lambda$, in combination with dead and live loads for braces at the Roof level. The forces for this case are calculated as

$$\lambda M_p = (1.1)(1.25)(50\text{ksi})Z_x$$  \hspace{1cm} (4)

$$\lambda V_L = \frac{2\lambda M_p}{e}$$  \hspace{1cm} (5)

$$\lambda V_{Back} = \lambda V_L \left( \frac{e/2}{15\text{ft} - e/2} \right)$$  \hspace{1cm} (6)

where the links are controlled by bending. For links controlled by shear, $\lambda V_L$ is calculated directly as the overstrength shear capacity.

![Diagram of equilibrium conditions](image)

**FIGURE 6 – EQUILIBRIUM CONDITIONS FOR PROPORTIONING BRACE SIZE TO WITHSTAND LINK OVERSTRENGTH**

The case shown here is for a girder that carries beams at 10 ft on center. Vertical components for Dead and Live Loads are calculated assuming $D = 95$ psf, $L = 50$ psf, and assuming that all of the dead and live loads acting on the girder. Brace forces are calculated as

$$P_u = (1.2D + 0.5L + V_L + V_{Back} \frac{L_{Brace}}{h_t})$$  \hspace{1cm} (7)

Link yielding controls column design. Figure 7 shows the frame design and drifts calculated under wind loads. The total weight of a single EBF frame is 22 tons, approximately 1 ton lighter than an $R = 3$ CBF designed for the same building configuration. Figure 8 shows a shear link detail for the Roof Level. Bolting a weaker link together in the shop with a stronger beam allows for the selection of the smallest possible links for resisting wind forces. The larger beam outside the link easily satisfies the capacity design requirement that it remain elastic while the link yields. It also stiffens the building for better drift performance under wind loads, and it provides a reasonably sized member to act as a girder supporting other gravity framing. Finally, this larger outside beam provides several opportunities for bracing the shear link, and it represents almost exactly many of the large scale tests that have been performed to date.
In order to match as closely as possible the test data from Okazaki and Engelhardt, Figure 8 shows two-sided fillet welds between the link and end plate that are one-and-one-half times the size of the flange or web. Weld tabs are provided at the flange edges to “avoid introducing undercuts or weld defects at these edges” [Okazaki and Engelhardt 2007, p. 761]. End plates are specified as 1 in. thick both for the W10x19 link and the W16x36 outside beam, which is approximately 50% oversized from the requirements according to prying action under the
overstrength of a fully-plastic flange. These 1 in. plates are assumed to be smaller than the plates from previous tests (whose exact dimension is often not reported since it is part of the test setup). How well they would perform under plastic demands on the link flanges could be clearly demonstrated by further full-scale testing. Such testing could provide a basis for optimizing the end plate size, bolt design requirements, and stiffener requirements in the W16x36 outside beam. For instance, Figure 8 does not show horizontal stiffeners at in the W16x36 as the W10x19 flange locations because the 1 in. end plates acting together are thought to be adequately stiff and strong for transferring the load between flanges a the lower ductility levels required for Boston.

CONCLUSIONS

This design example for a theoretical 9-story building Boston demonstrates that an EBF can be designed to conform with the AISC 2005 Seismic Provisions without exceeding the weight of an R = 3 CBF. Expense incurred via capacity design requirements can be kept in check by selecting the smallest possible links to withstand wind forces. These links can be fabricated separately from the beams outside the link and bolted together as a single element in the shop. In the field, these built-up link beams and the braces can be erected in a manner similar to a typical CBF with no special detailing requirements. The extra fabrication effort required for the built-up link beams seems to be well worth the reliable safety benefits of such a robust seismic force resisting system.

Further testing is required for optimizing the end plate connections with respect to weight and weld requirements. Further testing could also determine practical connections between links and columns, and between braces and links. From a design point of view, the length of the links seems be limited by drift requirements, however further testing of continuous link beams with longer links and flange yielding outside the link region could help to create more latitude for designers in moderate seismic regions, where drift demands are expected to be significantly lower than in high seismic regions. Results reported by Engelhardt and Popov [1992] for beams outside of links that were overloaded axially (±0.7Py) and in bending, intentionally to violate capacity design principles, still allowed links to achieve approximately 0.02 radians of plastic rotation. Considering that the test setup for this study did not include a slab, that the links were framed into columns on one end, and that the poorly performing tests were designed with flexible braces to allow most of the moment to be taken by the beam; what was considered poor performance for high seismic regions may yet imply superior performance to low-ductility, low reserve capacity CBF designs in moderate seismic regions.

REFERENCES