COMPARISON OF LOW-DUCTILITY MOMENT RESISTING FRAMES AND CHEVRON BRACED FRAMES UNDER MODERATE SEISMIC DEMANDS

Timothy A. Nelson¹, Michael C. Gryniuk², and Eric M. Hines³

ABSTRACT

Non-linear analyses utilizing site specific ground motions were performed on four structural steel frames with geometry and loadings based on the SAC Joint Venture’s Boston structure: Pre-Northridge and Post-Northridge moment resisting frames, and strong and weak connection R=3 chevron braced frames. Although the two types of frames exhibited similar ultimate capacities, they are very different with respect to stiffness. Significant damage was observed in the braced frames under certain earthquakes, while almost no damage was observed in the moment frames. Because of this lack of damage, the behavior of the moment frames was further observed using elastic techniques. Response spectrum analyses on the moment frames revealed that the demands from a series of site specific ground motions for Boston rarely exceeded the demands imposed by the IBC maximum credible earthquake (MCE) response spectrum. The lower natural periods of the stiffer braced frames correspond to higher accelerations on the design response spectrum, resulting in a larger seismic demand than in the moment frames. This helps to explain why the braced frames showed many instances of non-linear behavior, while the moment frames responded almost entirely in the elastic range.

Introduction

As part of the SAC Joint Venture, moment frames were designed to resist seismic loads according to local building codes for Los Angeles (UBC 1994), Seattle (UBC 1994), and Boston (BOCA 1993). Multiple researchers have discussed the non-linear dynamic behavior of the Los Angeles and Seattle frames (FEMA 2000b; Gupta and Krawinkler 2000; Yun and Foutch 2000). To the authors’ knowledge, however, published discussion of the seismic performance of the Boston frames is not readily available in the open literature. This paper discusses the observed response of low-ductility structural steel frames subjected to site-specific ground motions in Boston, Massachusetts.

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Design and Modeling of the Frames

Design Considerations

The story heights and dead loads of the SAC Joint Venture’s 9-Story Boston structure were used for all frames in this study. The penthouse level of the structure was excluded for simplicity. The gravity frames of the structures were re-designed to account for the reduction in mass from the penthouse level and were used as equivalent moment resisting frames. The following frame designs and associated computer models are discussed in this study:

- **MF1** Pre-Northridge Moment Resisting Frame, “standard beam-to-column welded connections,” FEMA 267 not considered
- **MF2** Post-Northridge Ordinary Moment Frame, strong column/weak beam criteria satisfied, design complies with specifications of FEMA 267
- **BF1** Chevron Braced Frame with Strong Connection Capacity (SCC)
- **BF2** Chevron Braced Frame with Expected Connection Capacity (ECC)

The moment frame designs were obtained directly from the SAC Joint Venture and were controlled by wind loads (FEMA 2000b). This corresponds to a roof displacement of approximately \( h/400 \) based on Exposure C wind loads in Boston according to the 6th edition of the Massachusetts State Building Code. The braced frames were designed with a chevron brace configuration without additional seismic detailing per the R=3 criteria of the IBC2003. The basement level was excluded for simplicity. Column splices were located at floor levels 4 and 7 and the columns were assumed to be continuous at these points. The brace/column connections were assumed to be pins and the braced frame beams were assumed be continuous between columns and pin connected for design. The braces were assumed to be pin-connected at their ends. The limiting slenderness and width-to-thickness ratios were taken as \( kl/r \leq 200 \) and \( b/t \leq 35 \) respectively. The equivalent lateral force procedure was used for the distribution of design forces. Figures 1 and 2 display plans and elevations of the respective systems.

![Figure 1](image_url). Moment resisting frame plan and elevation (MF1 & MF2).
Modeling Assumptions

The moment frames and braced frames followed the same basic modeling assumptions. The expected steel yield strength was used for all members. This included a 10% increase for the 50ksi steel yield strength of the beams and columns and a 30% increase for the 46ksi steel yield strength of the braces. Furthermore, the approximate effective length of the braces was calculated to account for the depth of the columns and beams to which they were connected. This resulted in an average effective length factor of $k = 0.87$ and an additional increase in member capacity.

NEHRP states that regular 3-dimensional structures may be modeled as 2-dimensional structures in each orthogonal direction (FEMA 2003). Figures 1 and 2 show that the SAC 9-story Boston structure is regular in plan, therefore a two-dimensional frame was used to represent half of the lateral force resisting system for each model. The vertical weights for the models were composed of the dead weight of the structure, including half of the partition weight, and half of the reduced live load weight, as is shown in Table 1. The lateral masses were then derived from the total weight.

Table 1. Vertical and lateral masses applied to the model (based on ½ structure’s weight).

<table>
<thead>
<tr>
<th>Floor #</th>
<th>1</th>
<th>2</th>
<th>3 - 9</th>
<th>Roof</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor Mass</td>
<td>1179 kips</td>
<td>1230 kips</td>
<td>1210 kips</td>
<td>1150 kips</td>
<td>12029 kips</td>
</tr>
</tbody>
</table>

For each model, the lateral mass was divided equally among the columns in the lateral force resisting system and lumped at each beam-column joint. The equivalent moment resisting frame was created to account for the lateral resistance offered by 2 interior column lines of the gravity system. A rotational strength of the simple shear connections was assumed based on experimental data (Liu and Astaneh-Asl 2000). Tributary vertical weights were applied directly to each node on the moment frames and were divided among the column and braced nodes for the braced frames.
Simplified models for frames MF1, MF2, BF1, and BF2 were constructed in the non-linear finite element program Ruaumoko (Carr 2004b). A bi-linear hysteresis rule with 1% strain hardening was applied to all beams and columns in all the models. A steel brace member hysteresis rule incorporating axial buckling behavior was used for the brace members (Remennikov 1997). This hysteresis rule, however, does not account for the fracture life of a cyclically loaded brace. Large changes in geometry during excitation affect a structure’s stiffness, and this was considered in the models by the Large Displacements option (Carr 2004a).

**Ground Motions for the Boston Area**

The IBC Site Class D ground motions used in the analyses were derived from a study for developing appropriate ground motion suites for the Boston area (Sorabella 2006). Refer to this document for a detailed discussion of Boston seismicity and the selection process utilized. To obtain desired Site Class D motions, a standard soil column analysis was used to amplify the motions from their original Site Class B characteristics. Figure 3 displays the average response spectrum for the 14 chosen ground motions, as well as the response spectrum for earthquake 3.

**Analysis**

**Initial Verification of the Model**

Periods and mass participation values for frames MF1 and MF2 were compared with the values that the SAC Project obtained (Table 2). The two sets of values match well. Mass participation values are in less agreement than the periods, but all discrepancies are small and are most likely due to differences in the modeling programs utilized. The good correlation shows that this study’s modeling techniques and assumptions are similar to those of the SAC Project.

![Figure 3. a) Average response spectrum for the 14 selected ground motions. b) Average response spectrum plotted against the earthquake # 3 response spectrum.](image-url)
Table 2. Period and mass participation comparison.

<table>
<thead>
<tr>
<th>Fundamental Mode</th>
<th>Moment Resisting Frames</th>
<th>Braced Frames</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MF1</td>
<td>Pre-Northridge SAC Results</td>
</tr>
<tr>
<td></td>
<td>Period (sec)</td>
<td>Mass Partic. (%)</td>
</tr>
<tr>
<td>1</td>
<td>3.16</td>
<td>77</td>
</tr>
<tr>
<td>2</td>
<td>1.18</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>0.699</td>
<td>3</td>
</tr>
</tbody>
</table>

Inelastic Time-History Analyses

Each of the four frames was subjected to the 14 site-specific ground motions and run through inelastic time-history analyses in Ruamoko. The results were examined for any instances of non-linear behavior and collapse.

Results for Pre-Northridge (MF1) and Post-Northridge (MF2) Moment Resisting Frames

The two moment frames performed favorably, that is, there were very few instances of non-linear behavior in either frame, and no collapses occurred. The only non-linear behavior in MF1 was a plastic rotation of 0.019 radians that occurred in Member 133 under earthquake # 3 (see Figure 1). There were no instances of non-linear behavior in MF2. The non-linear behavior occurring in Member 133 may be due to the effects of the adjacent gravity frame, pushing and pulling on the moment frame as the excitation occurs. Due to the lack of non-linear behavior in MF2, only MF1 will be discussed further.

Model MF1 does not consider the plastic rotational capacity of the beam-column joints, nor the brittle fracture behavior that could result from plastic rotation demands. It was necessary to separately determine whether connection strength was an issue in these models. To do this, the maximum plastic rotations of the beam-column joints in model MF1 were analyzed and compared to the expected plastic rotation capacities of the moment connections found from past Pre-Northridge connection tests. Testing performed on Pre-Northridge connections following the Northridge Earthquake showed that steel beam sections with similar depths to the beams in MF1 were able to achieve approximately 0.01 radians of plastic rotation (FEMA 2000a; Lee et al. 2000). In MF1, the only plastic rotation to exceed this limit was the 0.019 radian rotation in Member 133. Because this joint is located at the roof level, and because MF1 is quite flexible (natural period of 3.16 seconds), the collapse of the top story of the structure due to one potential brittle fracture is very unlikely.

Figure 4 presents the first three fundamental mode shapes for MF1, as well as a series of displaced shapes representing the response of a vertical line of columns under earthquake # 3. The semblance between the fundamental mode shapes and the displaced shapes suggests that MF1 exhibits behavior in at least the first three modes. The response spectrum in Figure 3
further supports this observation. Under earthquake # 3, the spectral acceleration for the natural period of MF1 is only approximately 50% of the IBC MCE acceleration at this period. Meanwhile, the second and third fundamental periods correspond to higher spectral accelerations that are closer to the design spectrum.

![Mode Shapes](image1)

The lack of non-linear behavior in MF1 is likely due to over-design. ASCE 7-02 estimates the natural period of MF1 to be 1.31 seconds, only 41% of the analytical period from the MF1 model. Seismic base shear varies inversely with period, so the resulting design base shear is 2.4 times greater than it would have been if the higher period was used. In terms of design, maximum periods are imposed to account for stiffness contributions of non-structural elements (FEMA 2003). In terms of assessment, however, it is difficult to determine which period is more appropriate. Figure 5 shows that the shear demands from the ground motion

Figure 4. Moment resisting frame MF1 subjected to earthquake # 3.

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response spectra rarely exceed the IBC MCE demand because the design shears are based on the lower period. The MF1 design was wind controlled, making the design base shear even larger.

Figure 5. Story shear demand (from response spec. analyses) normalized by design shears.

**Braced Frame Results**

Figure 6 shows a plot of the first story and seventh story inter-story drifts for the braced frame (BF1) under earthquake #3. Although collapse does not occur in the structure, significant permanent deformation is concentrated in the first and seventh stories. Strength degradation is a consequence of the buckling of braces at each level. The drift concentrations shown in Figure 6 are indicative of a concentration of inelasticity that is not re-distributed over the height of the structure. The total roof drift plot shows a gradual increase in drift for the duration of the earthquake. This plot has a strikingly different shape from the seventh story drift. The seventh story plot shows one large drift followed by a relatively constant drift. The first story plot shows a gradual drift on one side from approximately $t = 7s$ to $t = 20s$ when the structure “rights” itself and then begins to drift to the other side. In this case, looking solely at the roof drift does not give an accurate representation of the individual story behavior. It should be noted that this model assumes that the connections do not fracture. However, the overstrength of the brace is likely to exceed the capacity of the connections in the absence of seismic detailing.

**Comparison of the Two Structural Systems**

It is apparent that the differences in flexibility between the moment frame and the braced frame are responsible for the variations in performance under the same earthquake ground motions. MF1 is able to undergo larger story drifts than BF1 without loss of strength. The MF1 roof drift and inter-story drift plots follow the same general paths, while soft story formation results in drift concentrations in BF1. An important difference in behavior can be seen in the deflected shapes between $t = 9.4s$ and $t = 9.9s$ for both frames. Without inelastic action, the moment frame is able to translate in opposite directions with relative ease, whereas the strength degradation of the seventh story of the braced frame results in an increased drift concentration at that story rather than a steady story translation.
Using SAP2000, a simple pushover analysis was performed on models MF1 and BF1 to quantify the differences in flexibility between the 2 types of frames. The loading patterns assumed first mode behavior and the roof displacements were monitored. The elastic stiffnesses were found to be 65 kips/inch for MF1 and 390 kips/inch for BF1. Thus, the braced frame is six times as stiff as the moment frame, further explaining why the responses of the two frames differed so greatly. The ultimate capacities of MF1 and BF1 were found to be approximately 1,120 kips and 1,150 kips respectively from the pushover analyses. Using these capacities, story shear demand to capacity ratios were calculated for MF1 and BF1 based on three different fundamental periods: the design period, the upper-bound design period, and the analytical period obtained from the finite element models. The demands were determined for these periods based on the MCE response spectrum and the earthquake # 3 response spectrum. Table 3 displays the ratios. This comparison assumes behavior purely in the first mode.
Table 3. Demand to capacity ratios for MF1 and BF1 based on various fundamental periods.

<table>
<thead>
<tr>
<th>Response Spectrum Utilized</th>
<th>Design Period</th>
<th>Upper-Bound Period</th>
<th>Analytical Period (finite element models)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MF1</td>
<td>BF1</td>
<td>MF1</td>
</tr>
<tr>
<td>T = 1.31 sec</td>
<td>1.78</td>
<td>2.78</td>
<td>1.10</td>
</tr>
<tr>
<td>T = 0.734 sec</td>
<td>5.56</td>
<td>3.18</td>
<td>1.02</td>
</tr>
</tbody>
</table>

In all cases but one, the BF1 demand ratio is higher than that of MF1. Since the braced frame is stiffer, it experiences higher accelerations and demands, as is shown in the non-linear analysis results. The only case where the BF1 demand is lower is with MF1 based on the design period and subjected to earthquake # 3. Earthquake # 3’s response spectrum has a spike in the vicinity of the design period (1.31 seconds) which results in a large increase in demand. This short-period spike tends to be a characteristic of east-coast ground motions (Sorabella et al. 2006). Based on the period determined analytically, the MF1 ratios are below 1.0, suggesting that this frame will never yield. The fact that the MF1 model did yield in the top story demonstrates that higher-mode behavior is occurring. The analytical ratios for BF1 are still above 1.0, although not to the extent that they are based on the design period.

Conclusions

Based simply on their differences in elastic stiffness, low-ductility braced frames appear to pose a greater threat to potential collapse than do moment frames of similar dimensions. The moment frames were never significantly excited at their natural periods by the site specific ground motions. The beam-column joint rotations remained very low, and in all cases but one, did not exceed 0.01 radians of plastic rotation. Although considered low-ductility systems, these moment frames possess excess capacity because wind stiffness design controlled the already over-designed shear capacity. The frames’ natural periods, as calculated by code, are noticeably lower than the analytical model periods, which makes a noticeable difference in design forces. The braced frames deflected much more than the moment frame structures, as they are stiffer structures, excited to a larger extent by the same ground motions. It is questionable whether or not these systems are appropriate for areas such as Boston that do not represent a high seismic hazard. From this research, the moment frames possessed enough strength and flexibility to remain essentially elastic throughout all earthquakes. The braced frames went inelastic in several instances, and it is clear that the low-ductility of this system is not adequate to resist the demands imposed by the suite of ground motions.

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References


