RESULTS OF EARLY COLLABORATIVE RESEARCH ON BEHAVIOR OF BRACED STEEL FRAMES WITH INNOVATIVE BRACING SCHEMES (ZIPPER FRAMES)

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ABSTRACT

The new Network for Earthquake Engineering Simulation (NEES), which was inaugurated in late 2004 in the USA, intends to transform the way earthquake engineering research is conducted. In this paper, the initial collaborative experiences with an early NEES project will be described and some early technical results reported. The vehicle used for the collaboration is the development of design provisions and analytical tools for zipper frames. Zipper frames are intended to improve on the behavior of conventional inverted-V-braced frames, which exhibit poor performance arising from the early buckling of the lower story braces. A zipper frame provides better performance by forcing simultaneous buckling of all braces. In this project, that behavior is tied to the use of an elastic hat truss at the top of the building, which prevents the formation of an overall collapse mechanism. The collaborative nature of the project arises from the need to test prototype and small scale model structures under both quasi-static and dynamic loading, and by the need to supplement such tests with much more economical subassemblage ones.

BACKGROUND TO COLLABORATIVE EFFORT

After many years of discussion and attempts at finding funding, the National Science Foundation started the Network for Earthquake Engineering Simulation (NEES) in 1998. This network was envisioned as the first collaboratory (collaboration+laboratory) in the area of engineering in the USA. A collaboratory has been defined as “... an organizational entity that spans distance, supports rich and recurring human interaction oriented to a common research area, and provides access to data resources, artifacts and tools required to accomplish key tasks.” 1 Collaboratories have been highly successful in areas such as astrophysics and geosciences, and their premise is that knowledge developed in this fashion adds up to much more than the sum of the individual parts. It is believed that such approach is also needed to solve the very complex problems associated with mitigating risks from seismic events. Fifteen equipment sites and one system integrator contract were awarded as part of the NEES collaboratory, with a total investment well in excess of $80 million.

In early 2002, as the facilities funded under the NEES program began to take shape, several groups of researchers in the USA began to form teams to test the early implementations of the software and hardware installed at some of the NEES sites. Several of the proposals from those teams, among these the one described herein, were successful at obtaining funding. All of these proposals had at least four commonalities: (a) they involved distributed groups of researchers with many of the principal investigators coming from outside NEES sites, (b) they involved multiple NEES sites to showcase how their testing capabilities improved simulation

1 For more information on collaboratories, see http://intel.si.umich.edu/cfdocs/si/research/researchprojects.cfm
capabilities, (c) they included a strong simulation component; and (d) they addressed a challenging but limited technical problem. The success of the NEES collaboratory will in large part depend on the collegial and productive sharing of NEES Equipment Sites among a wide variety of users. These users may include researchers from a variety of educational institutions, national laboratories, government agencies, the private sector, researchers at a NEES site, and combinations thereof. The utilization of the NEES facilities by such groups in integrated collaborative research projects is defined as “shared use” and is one of the two cornerstones of this new research approach. The other cornerstone is provided by the simulation part, which covers everything from advanced analysis in support of testing to active control of a subassemblage test at a remote laboratory with a combination of real-time experimental and analytical inputs coming from other sites.

The goals of the collaborative group on the behavior of zipper frames were:

- To develop a realistic prototype and scale model of a low-rise structure to serve as the benchmark structure for testing the validity of the zipper frame concept. It was originally envisioned that this could be achieved at $\frac{1}{2}$-scale. As will be discussed, the final structure had to be at $\frac{1}{3}$-scale to accommodate the payload limitations on the shake table available. The prototype structure was to be based on the SAC model studies so that meaningful comparisons between braced and unbraced frame systems could be made.

- To test full models of a typical zipper structure under both large seismic base excitations (shake table testing at the University at Buffalo (UB)) and quasi-static loads (Georgia Tech (GT)) to assess the fidelity of each testing method in following the prescribed load history. The load history for the shake table was to be chosen from the SAC suite of ground motions, appropriately scaled, and the deformations applied quasi-statically were to be taken directly from the shake-table measurements.

- To assess the robustness of different modeling methodologies in predicting the dynamic and static (pushover) behavior of the model structure. Several analysis programs, including OpenSees, ABAQUS, IDARC, and SAP2000, were used, along with variations within each program in modeling the non-linear effects due to buckling. It was assumed that each group would first work independently on this process, and that the results would then be compared and the differences in predictions would be clearly explained in terms of the varying assumptions used in each model. The modeling of the end conditions on the braces, their effect on the out-of-plane buckling, large deformations of the braces, and local damage due to high cyclic plasticity were considered to be among the most challenging issues for this portion of the work.

- To test both scaled models and full-scale prototypes quasi-statically to determine the ability of pushover analysis to predict dynamic behavior of braced frames. While the use of pushover analysis on moment-resisting frames has received considerable attention and the limitations of the technique are beginning to be understood in the context of such structural systems, the same cannot be said for braced frames.

Through coordinated work at the universities of Colorado at Boulder (CU) and California at Berkeley (UCB), to carry out development work to integrate and synchronize the fast hybrid pseudo-dynamic (FHPS) testing algorithms that will be used to test subassemblies at the two locations simultaneously. This testing intended to include a first-story set of braces at CU and a second-story set of braces at UCB, all linked through an advanced OpenSees simulation. It was anticipated that this task will require a significant amount of effort as this would be the first attempt to link tests and simulation at several sites under near real-time conditions. Due to the complexity of the models and the latency issues in the network, true real-time interaction will probably not be achieved, although the test will be relatively fast compared to the quasi-static work conducted at GT. The joint tests at CU and UCB were to be conducted using the displacements at key locations obtained during the shake table tests at UB.

To conduct tests at UCB on individual brace components to assess the ability of different pseudo-dynamic test algorithms to handle buckling of the brace and the associated changes in brace strength and stiffness.

To carry out a full-scale test at GT to validate the design provisions derived from the previous analytical and experimental work.

To develop a follow-up proposal dealing with active control and the use of shape memory alloys as a joint project by teams at GT and Florida A&M (FL A&M).

**BACKGROUND TO ZIPPER FRAMES**

As noted earlier, one of the characteristics of many of the initial NEES projects is that they address novel structural solutions. As more emphasis has been placed on increasing ductility and energy dissipation capability of all types of structures in modern codes, design provision for a new type of braced frame, labeled the Special Concentrically Braced Frame (SCBF), have been developed (Goel 1992, Bruneau et al. 1998). Within these provisions, the performance of Special Inverted-V-Braced Frames (SIVBF) was improved from that of
ordinary Inverted-V-Braced Frames (IVBF) by limiting width/thickness ratios, requiring closer spacing of stitches, and providing special design and detailing of end connections for the bracing members. However, SIVBFs still exhibit a typical braced frame design problem, i.e., the concentration of damage in the first floor, as evidenced by recent tests (Mahin and Patix, 2004). Upon continued lateral displacement, the first floor compression brace buckles and its axial capacity decreases while that of the tension brace continues to increase. This creates an unbalanced vertical force on the intersecting beam, resulting in a structural system that tends to concentrate inter-story drift in a single story, as shown in Fig. 1a. In order to prevent undesirable deterioration of lateral strength of the frame, the provisions require that the beam shall possess adequate strength to resist this potentially significant post-buckling force redistribution in combination with appropriate gravity loads (AISC 2002). This results in very strong beams, much stronger than would be required for ordinary loads.

The adverse effect of this unbalanced force can be mitigated by adding zipper columns, as proposed by Khatib et al. (1988) and shown in Fig. 1b. The intent of SIVBFs with zipper elements is to tie all brace-to-beam intersection points together, and force all compression braces in a braced bay to buckle simultaneously. This results in a better distribution of energy dissipation over the height of the building. However, instability and collapse can occur once the full-height zipper mechanism forms, as shown in Fig. 1b. This is due to the reduced lateral capacity of the frame after a full mechanism has formed (Tremblay and Tirca, 2003), and this drawback limits the potential applicability of this system. Moreover, from a seismic design standpoint, a capacity design approach for zipper frames will require that assumptions be made both as to whether the zipper column or the tension braces should be allowed to yield and as to what the desirable deformation mechanism should be. These questions have not been answered decisively as yet.

In this work, the disadvantages of a full-height zipper mechanism will be overcome by introducing a suspension system, labeled “suspended zipper frames,” as shown in Fig. 1c. In a suspended zipper frame, the top story bracing members are designed to remain elastic when the all other compression braces have buckled and the tension braces and zipper elements have yielded. Since the primary function of the suspended zipper struts is to sustain tension forces, and the suspended zipper struts support the beams at the midspan, the beams can be designed to be flexible. This results in significant savings in the amount of steel for the beams in SIVBFs with suspended zipper elements. Moreover, the force path is also so evident that a capacity design for all structural members is straightforward.

ANALYSES OF THE PROTOTYPE

Following a capacity design methodology, an inverted zipper frame was designed as the prototype structure for this project (Figure 2). The prototype structure was designed with a layout and loads similar to the 3-story SAC structure designed for Los Angeles (Maison and Kasai, 1999). It was assumed that the beam-to-column connections as well as brace-to-beam and zipper column-to-beam connections are pinned. Unfactored uniformly distributed roof dead loads of 1.69 kN/m, floor dead loads of 1.95 kN/m, and live loads of 0.40 kN/m were applied to the beams. The site is classified as site class D (soft soil) and its mapped spectral response acceleration at short periods (Ss,) and at 1 second period (S1) are 1.5g and 0.6g respectively. The seismic weight for this braced bay is 4820 kN, which is one sixth of the entire building seismic weight. The seismic loads were calculated based on the value of R equal to 6, the value of Ie equal to 1.5, and the provisions of IBC 2000. Accordingly, the seismic base shear was calculated to be 1204 kN, with the floor loads being 626 kN, 385 kN, and 193 kN from the third to first floor levels, respectively. The member sizes are listed on Table 1.
Table 1 – Member sizes.

<table>
<thead>
<tr>
<th>Story</th>
<th>Type</th>
<th>Prototype</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>Braces</td>
<td>HSS10x10x5/8</td>
<td>HSS3x3x3/16</td>
</tr>
<tr>
<td>2</td>
<td>Braces</td>
<td>HSS7x7x3/8</td>
<td>HSS2x2x1/8</td>
</tr>
<tr>
<td>1</td>
<td>Braces</td>
<td>HSS7x7x3/8</td>
<td>HSS2x2x1/8</td>
</tr>
<tr>
<td>3</td>
<td>Columns</td>
<td>W10x77</td>
<td>S4x9.5</td>
</tr>
<tr>
<td>2</td>
<td>Columns</td>
<td>W10x77</td>
<td>S4x9.5</td>
</tr>
<tr>
<td>1</td>
<td>Columns</td>
<td>W10x77</td>
<td>S4x9.5</td>
</tr>
<tr>
<td>3</td>
<td>Beams</td>
<td>W8x21</td>
<td>S3x7.5</td>
</tr>
<tr>
<td>2</td>
<td>Beams</td>
<td>W14x82</td>
<td>S5x10</td>
</tr>
<tr>
<td>1</td>
<td>Beams</td>
<td>W12x50</td>
<td>S3x5.7</td>
</tr>
<tr>
<td>3</td>
<td>Zipper</td>
<td>W8x48</td>
<td>HSS2x2x3/16</td>
</tr>
<tr>
<td>2</td>
<td>Zipper</td>
<td>W8x24</td>
<td>HSS1.25x1.25x 3/16</td>
</tr>
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</table>

Initially the intent was to use only one bay for the braces; this proved impractical and the decision was made to turn one of the long unbraced spans in the original SAC structure into two zipper braced spans. The performance of the structure was then assessed with the aid of nonlinear pushover and dynamic time history analyses using OPENSees models. Initially the elements chosen to model the braces were quite crude phenomenological ones, but progressively more realistic ones have been used. Figure 3 shows a comparison of two of the initial pushover analyses conducted independently at two sites, emphasizing the range of differences that can be obtained by different but entirely reasonable modeling assumptions. The upper line shows a model that includes a rapid loss of strength with buckling and larger strength due to strain hardening assumptions.
The second pushover analysis of the model, shown in Fig. 3, can be used to illustrate the overall desirable mechanism for zipper frames. In stage 1, the structure is linearly elastic and the maximum base shear force is about 1512 kN (340 kips). This is significantly larger than the design base shear, which corresponded to 1205 kN (270 kips). When the first story compression brace buckles, the structure enters stage 2. When the second story compression brace buckles the structure enters stage 3 and the maximum base shear is reached. At this stage the maximum base shear is 1780 kN (400 kips) or about 1.48 times the design base shear. Once the first story tension brace yields, the structure goes into stage 4, which is characterized by a slow loss of strength as the first story brace continues to yield. When the compression brace reaches its constant post-buckling value and its stiffness becomes a small positive value, the structure enters stage 5. In the stage 6, the structural strength decreases again due to the yielding of the second story tension brace. Once the second story compression brace reaches the minimum post-buckling strength, the structure enters stage 7 and its strength increase slightly. This phenomenon is consistent with the mechanism shown in Fig 1c. The results of this pushover analysis indicate that the design procedure results in a structure that is ductile and not susceptible to large losses of lateral strength and stiffness as conventional braced frames are.

The differences for non-linear time history analysis are even more marked, indicating substantially different mechanism, locations of yielding and residual deformations depending on the model assumptions. The suite of 20 ground motions LA21 through LA40 from the SAC project were used to determine the ground motion most likely to damage the structure at reasonable load levels. The LA21 and LA22 (Kobe) ground motions at their full value was deemed the most critical, and Figure 4 shows the sequence of brace yielding, zipper strut buckling, and beams and/or columns full-or partial-section yielding when the 3-story SAC model building is subjected to the Kobe earthquake (LA21). The solid rectangles represent brace’s buckling, while the solid circles represent braces yielding, zipper struts yielding, and columns or beams yielding. Beginning about 7.85 sec., when the largest demand occurred, the structure moved left, and all left compression braces below the top level buckled. The first- and second-story tension braces then yielded, followed immediately by the yielding of the upper zipper column.

At time equal 9.22 sec during the Kobe earthquake, the top floor reached its maximum drift of 364 mm (14.32 in.). At this point, interstory drifts corresponded to 0.38%, 3.32% and 5.48% for the third through the first floor respectively. Although the deformation is concentrated in the first floor, as expected, the distribution of interstory drifts is reasonable. The final pushover and non-linear time history analyses have been carried out with a sophisticated model that includes the possibility of fracture due to low-cycle fatigue (Mahin and Patix, 2004).

Fig. 4 - Time of formation of buckles and hinges

SCALING OF THE PROTOTYPE FOR SHAKE TABLE TESTS

A most challenging part of the initial work was the development of a scale model that could be tested at all four sites without requiring extensive modification of existing facilities or the acquisition of any new loading equipment. The main limitation was the payload capacity at the UB shake table. The SAC prototype building that was chosen for this study consists of a large floor plan with lateral resistance concentrated on just a few locations; thus the tributary mass of about 5000 kN assigned to each of the zipper frames was quite large, and well above maximum table capacity (about 250 kN). From this limitation alone, it became clear that the initially proposed ½-scale would not be achievable, and that something closer to a ⅓-scale would be needed. In addition, a substantial problem was the availability at that scale of combinations of small rolled steel members that would preserve the strength and stiffness ratios required to maintain the expected failure mechanisms. After numerous design iterations, the sizes shown in Table 1 were selected for the model. The final scaling relations used are shown in Table 2.
Table 2 Summary of scale factors for earthquake response of the model

<table>
<thead>
<tr>
<th>Relationship</th>
<th>Target Factor</th>
<th>Actual Factor (*)</th>
<th>Level 3</th>
<th>2</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Loading</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Force</td>
<td>$S_Q = S_e S_i^2$</td>
<td>9</td>
<td>9</td>
<td>Column area 12.9</td>
<td>12.9</td>
</tr>
<tr>
<td>Acceleration</td>
<td>$S_a = 1$</td>
<td>0.5</td>
<td></td>
<td>Column inertia 94.4</td>
<td>96.4</td>
</tr>
<tr>
<td>Time</td>
<td>$S_t = [S_e S_i^{-1}]^{1/2}$</td>
<td>1.732</td>
<td>2.45</td>
<td>Beam area 3.71</td>
<td>13.71</td>
</tr>
<tr>
<td><strong>Geometry</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Linear dimension</td>
<td>$S_L = 3$</td>
<td>3</td>
<td>3</td>
<td>Beam inertia 30.12</td>
<td>186.7</td>
</tr>
<tr>
<td>Area</td>
<td>$S_A = S_L^2$</td>
<td>9</td>
<td>(*)</td>
<td>Brace area 11.11</td>
<td>14.75</td>
</tr>
<tr>
<td>Moment of Inertia</td>
<td>$S_m = S_e^4$</td>
<td>81 (*)</td>
<td></td>
<td>Brace inertia 123</td>
<td>345</td>
</tr>
<tr>
<td><strong>Material properties</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modulus</td>
<td>$S_e = 1$</td>
<td>1</td>
<td>1</td>
<td>Zipper area 12.2</td>
<td>6.50</td>
</tr>
<tr>
<td>Mass</td>
<td>$S_m = S_Q S_a$</td>
<td>9</td>
<td>18</td>
<td>Zipper inertia 136.2</td>
<td>105.8</td>
</tr>
</tbody>
</table>

Figure 5 shows the specimen on the shake table before all the masses were added. Critical details of the connection regions for the first story braces are shown in Figs. 6 and 7. Both show the implementation of a new requirement in US seismic codes to have an uninterrupted area at least two times the gusset plate thickness in length perpendicular to the end of the braces to allow for a better prediction of the out-of-plane buckling capacity.

Fig. 6 - Second-story bottom brace connection

Fig. 7 - Second-story top brace-zipper

Fig. 5 - Photograph of the test frame on the shake table.
PRELIMINARY RESULTS

The first test was successfully conducted on the shake table in December 2004. A preliminary base shear vs. interstory displacement plot is shown in Fig. 8. The system appears to produce ductile behavior with only moderate losses of strength at the instant of buckling. The large out-of-plane displacements observed as the bottom story braces buckled are shown in Fig. 9. This unique set of data, obtained through the use of a Krypton measurement system, will be extremely helpful in calibrating the proposed models.

Fig. 8 – Base shear vs. interstory drift at first floor (1 in. displacement = 2.5% story drift).

Fig. 9 – Displacement of the bottom left brace with cycling.
COLLABORATION TOOLS AND EXPERIENCES

The team has kept in close contact by sharing all information by email. As currently structured, all email goes to all researchers, regardless of whether they are directly impacted or not by the material under discussion. While this sometimes results in unnecessary email being received by the researchers, it has created a great feeling of cooperation and significantly increased the trust level amongst the collaborators. During critical periods of the work (i.e., as the scale model was being developed), teleconferences at least every other week were conducted. Although the teleconferencing does not always work well, the immediacy of the medium led to critical decisions being reached quickly, substantially speeding the process. The team has gotten together once (in May 2004) and expects to meet face-to-face at least once a year. In addition, several of the collaborators attended the first shake table test at UB.

There has been great cooperation amongst the graduate students working on the project; for example, the UCB students have been extremely helpful in clarifying the inner working of OPENSees and the UB students and faculty were invaluable in dealing with the fabricating of the specimens. Some of the limitations of the current NEES software, primarily, the relatively poor video streaming and the lack of a robust real-time data viewer were noted during the first test. Finally, it was noted that remote viewers of the shake table tests drifted away quickly when the tests were stopped to rearrange the loading system as there is currently little capability of informing remote viewers of changes in the schedule or alerting them of exactly when a test run is about to start. The latter is critical as a shake table run may take just a few seconds and may be missed by a remote viewer that is not paying 100% attention to his/her computer screen.

SUMMARY AND CONCLUSIONS

The proposed design strategy results in suspended zipper frames having more ductile behavior and higher strength than ordinary zipper frames. So far the first shake table test at the University of Buffalo has been carried out. From the preliminary experimental results, the first- and second-story braces buckled and yielded as simulated in OpenSEES. The structure showed a good spread of yielding and buckling, resulting in different behavior from the concentration of damage that appears in special concentrically braced frames. However, it is still not clear how to interpret some of the brace data obtained after entering the non-linear range. The reversed cycle loading tests at Georgia Tech will overcome this issue and verify the load path, i.e., the sequence of member failure, when the structure is subjected to the lateral forces.

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