AN EXPERIMENTAL STUDY OF DAMAGE DETECTION USING REMOVABLE BRACES

G. C. Archer¹ and C. C. McDaniel²

ABSTRACT

During recent renovations to a small (24’x48’) off-campus building, connections for four removable braces were added to the existing moment frames to facilitate simulated damage. As part of a NSF-NEESR grant, the authors have developed an Ultra-Low Forced Vibration Testing (UL-FVT) technique to determine the mode shapes, frequencies, and damping for low-rise structures using a small, portable, linear shaker (30 lb force) and highly sensitive accelerometers. The UL-FVT technique was applied to this building to determine these dynamic parameters for all possible brace configurations. While simple, the structure possesses several interesting structural features. The relatively small building has a straightforward and clearly visible structural steel system. However, the flexible roof diaphragm, the semi-rigid floor diaphragm, flexibilities in the foundation piers, and the exterior cladding present a challenge to the detection of the braces. The mode shapes are complex, non-symmetric and involve some participation from the transverse, longitudinal, and vertical directions in each mode. This paper presents both the layout and configuration of the physical structure including the removable braces, flexible foundations, diaphragm attachment, and the associated structural dynamic behavior found for all brace configurations using the UL-FVT technique. The complexities present in the structure preclude detecting the configuration of the braces through changes in the frequency alone. The authors use the mode shapes and comparisons to analytic models using a modified Modal Assurance Criteria (MAC numbers) to detect the brace configuration.

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An Experimental Study of Damage Detection using Removable Braces

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ABSTRACT

During recent renovations to a small (24’x48’) off-campus building, connections for four removable braces were added to the existing moment frames to facilitate simulated damage. As part of a NSF-NEESR grant, the authors have developed an Ultra-Low Forced Vibration Testing (UL-FVT) technique to determine the mode shapes, frequencies, and damping for low-rise structures using a small, portable, linear shaker (30 lb force) and highly sensitive accelerometers. The UL-FVT technique was applied to this building to determine these dynamic parameters for all possible brace configurations. While simple, the structure possesses several interesting structural features. The relatively small building has a straightforward and clearly visible structural steel system. However, the flexible roof diaphragm, the semi-rigid floor diaphragm, flexibilities in the foundation piers, and the exterior cladding present a challenge to the detection of the braces. The mode shapes are complex, non-symmetric and involve some participation from the transverse, longitudinal, and vertical directions in each mode. This paper presents both the layout and configuration of the physical structure including the removable braces, flexible foundations, diaphragm attachment, and the associated structural dynamic behavior found for all brace configurations using the UL-FVT technique. The complexities present in the structure preclude detecting the configuration of the braces through changes in the frequency alone. The authors use the mode shapes and comparisons to analytic models using a modified Modal Assurance Criteria (MAC numbers) to detect the brace configuration.

Introduction

In 1965, a class of undergraduate students [1] launched a project to utilize the rough terrain surrounding the Cal Poly, San Luis Obispo campus. Their solution was to span a ravine with a bridge-like steel structure similar to that of Mies Van Der Rohe’s Glass House. Over the years, the building has mostly been forgotten and has fallen into a state of disrepair. Fortunately, in 2011 another team of students [2] took on the task of revitalizing the building and have created a living structural dynamics laboratory. The Bridge House, as it's called, is supported on a slender concrete pier at each corner and spans 48 ft in the longitudinal direction and 24 ft in the transverse direction. It is vertically supported by a pair of longitudinal trusses that also serve as the lateral system in the longitudinal direction. In the transverse direction the lateral system originally consisted of moment frames. During the 2011 renovations, connections for four removable braces were added to the moment frames to facilitate simulated damage through the removal of the braces.

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While simple in layout, the structure possesses several interesting structural features. The relatively small building has a straightforward and clearly visible structural steel system. However, the flexible roof diaphragm, the semi-rigid floor diaphragm, flexibilities in the foundation piers, and the exterior cladding present a challenge to the detection of the braces. The mode shapes are complex, non-symmetric and involve some participation from the transverse, longitudinal, and vertical directions in each mode. The mode shapes and frequencies will be experimentally determined using forced vibration testing.

This paper presents both the layout and configuration of the physical structure including the removable braces, flexible foundations, diaphragm attachment, and the associated structural dynamic behavior found for all brace configurations using forced vibration testing. The complexities present in the structure preclude detecting the configuration of the braces through changes in the frequency alone. The authors use the mode shapes and comparisons to analytic models using a modified Modal Assurance Criteria (MAC numbers) to detect the brace configuration.

**Bridge House Description**

The Bridge House is located in the outdoor experimental construction laboratory inside of Poly Canyon, approximately one mile from the center of campus (see Fig. 1). The Bridge House vertical and lateral force resisting systems are composed of ordinary moment frames spanning 24 feet in the transverse (north-south, NS) direction and braced frames spanning 48 feet in the longitudinal (east-west, EW) direction. The exterior cladding is composed of plywood sheathing connected to the structural steel frame in a variety of ways, in some locations loosely connected and in other locations firmly tied into the exterior frame. The Bridge House columns and braces are HSS3x3x1/4 sections welded between two channels (C12x20.7) that also serve as the chord members of the truss system. The corner columns of the building are built-up sections composed of four HSS3x3x1/4 sections. The interior roof and floor beams, W8x31 sections spaced at 8 feet on center, are connected to the web of the truss chords with steel tabs. The roof diaphragm is comprised of a corrugated steel deck filled with rigid insulation topped with gravel. The floor diaphragm is composed of a corrugated steel deck filled with 3½-inch thick lightweight concrete. The building is supported at the four corners by 18-inch x 18-inch concrete piers. The embedment of each pier into the surrounding soil varies at each corner.

In 2011, a group of architectural engineering students rehabilitated the Bridge House [2] into a structural dynamics laboratory equipped for forced vibration testing for the undergraduate structural dynamics and seismic design and analysis courses as well as senior projects and masters projects. The key structural revision the students made was the addition of four removable braces (HSS3x2x3/16 sections) in the NS direction (Fig. 1), two on the west face of the building and two on the east face of the building, so that the layout of the lateral force resisting system can be modified to study the change in the structural dynamics and to simulate damage in the building. Plates were also added to the roof beams to allow for mounting of a linear shaker to dynamically excite the building.
Computational Model

To predict the natural frequencies and modes shapes of the structure for the various brace configurations, a three-dimensional computational model of the Bridge House was developed using commercial structural analysis software [3]. While care was taken in the creation of the model, it is important to note that only a reasonably accurate model was created – a model similar to that which would be used in practice. The vertical and lateral force resisting system structural steel members were modeled as frame elements (see Fig. 2). The floor and roof diaphragms were modeled to allow for diaphragm flexibility. The relative stiffness of the lateral force resisting system to the roof diaphragm in the NS direction ranged from 1:1.5 with none of the removable braces attached to 4.3:1 with all four of the removable braces attached. Consequently, as the number of attached braces increased the diaphragm flexibility increased. As a result the flexible roof diaphragm was modeled as a combination of 4-node plane stress elements and diagonal frame elements supported by the roof beams. The diagonal frame elements were added to account for the variability in the roof deck/roof beam connection. Forced vibration experiments [4] showed that three of the roof beams were not fully attached to the corrugated steel roof deck.

In order to accurately capture the modal properties of the Bridge House, the flexibility of the foundation piers needed to be modeled as well. Horizontal soil spring constants were determined experimentally by shaking the structure with all four removable braces attached. The lateral stiffness of the soil springs was found to be 205 kips/inch for each pier on the west side of the building and 153 kips/inch for each pier on the east side.
Models were created for the following removable brace configurations: a) no removable braces attached (none), b) the northwest removable brace attached (NW), c) the northwest and southwest removable braces attached (NW,SW), d) the northwest and northeast removable braces attached (NW,NE), e) the northwest, southwest and northeast removable braces attached (NW,SW,NE), and f) the northwest, southwest, northeast and southeast removable braces attached (NW,SW,NE,NW). In addition to the removable braces, a brace with an area of 0.1 in$^2$ was added to the east face of the Bridge House to account for stiffness resulting from an observed unintentional connection of the cladding to the exterior steel frame.

**Experimental Testing**

In the past few years, the authors have developed and implemented [5, 6, 7] a unique type of Forced Vibration Testing (FVT) for low-rise building structures using ultra-low force amplitudes. This Ultra-Low FVT (UL-FVT) is accomplished by placing a small portable (~100 lb) harmonic shaker on the upper floors of the building and recording the resulting floor accelerations throughout the structure using highly sensitive accelerometers. The building structures tested to date include steel and concrete moment resisting frames, steel concentric braced frames, reinforced concrete moment resisting frames, and reinforced concrete shear walls. The majority of the structures were rectangular or L-shaped 2 and 3 story on-campus buildings with around 20,000 ft$^2$ of floor space [8]. The largest structure tested to date is a 5-story 180,000 ft$^2$ library building [9]. Since the Bridge House is considerably smaller (< 1,200 ft$^2$), the resulting accelerations are much larger and are well within the range of human perception. Fig. 3 shows the shaker being attached to the underside of the center beam at midspan. When activated, the shaker can put out a relatively constant 30 lb harmonic load over a wide range of frequencies. The resulting building accelerations are recorded using the accelerometers also shown in the right hand photo in Fig. 3.
Modal Frequency Comparison

To determine the first natural frequency in the NS direction, the direction in which the removable braces are engaged, the shaker was placed in the NS direction on the underside of the center roof beam and the resulting NS accelerations in an adjacent beam were measured over a range of frequencies. The test was repeated several times with a variety of brace configurations. The resulting frequency response graph is shown in Fig. 4. Clearly the drop in frequency as braces are removed is obvious and, as expected, the equivalent viscous damping (using the half-power band method [10]) is unchanged.

Unfortunately, the measured natural frequencies (4.15, 4.55, 4.9, 5.05, 5.47, 5.95 Hz) are not a great match for the computationally predicted frequencies (4.59, 5.26, 6.25, 5.92, 6.67, 7.35 Hz). The computational predictions range from 10-20% higher. The discrepancy is likely due to
uncertainty in the roof mass and roof diaphragm flexibility due to incomplete attachment to all roof beams as well as the modeling of the soil-structure-interaction. The goal of the work is to predict which braces are attached without prior knowledge of the structural condition. While great care was taken to provide an accurate computational model, the variation between the computational and experimental frequencies is well within the variation of the actual frequencies as braces are removed. Thus, observation of the natural frequencies alone is insufficient to predict the location of “damage”. For example, with the NW and NE braces attached the experimental frequency is 5.05 Hz. This lies between the computational results for no braces and for just the NW brace attached and completely misses the prediction of NW,NE as it should to be useful.

Mode Shape Comparison

It was hoped that observation of the mode shapes would lead to better predictors of the brace configuration (damage). To obtain the first NS mode shape for each of the brace configurations, the shaker was placed again in the NS direction on the underside of the center roof beam at midspan. The shaker was run at the natural frequency for the given brace configuration and the NS accelerations of each roof beam was recorded. The resulting accelerations were normalized and are presented in Fig. 5 alongside the computational prediction.

Figure 5. Experimental vs. computational first NS mode shapes for the brace arrangements.
A close look at Fig. 5 reveals much about the actual and predicted mode shapes. Even though the predicted and actual frequencies were 10-20% off, the predicted and actual mode shapes are remarkably similar. In all brace configurations, the first NS mode primarily involves a large rigid body translation and much smaller additional movement due to brace variation and diaphragm flexibility. This is typical of first mode shapes. Higher mode shapes would typically not involve much rigid body translations. The first three configurations consist of an unsymmetric arrangement of braces with more braces on the west side. Thus the resulting shapes are rotated counter-clockwise in plan. Curiously enough, for the next three configurations, which involve symmetric brace arrangements, both the actual and predicted mode shapes predict some overall rotation in plan. When all braces are attached, the rotation is counter-clockwise – leaning to the stiffer foundation on the west side. As the braces are removed the effect of the stiffer west side foundation is overtaken by the flexibility of the moment frame and the slight stiffening of the east frame due to the cladding mentioned earlier.

In order to more precisely compare two mode shapes, a modified modal assurance criteria (MAC) [6, 11] is employed. The MAC number represents a decimal percent of the correlation between two modes (1.0 would represent perfect correlation). Since the mass associated with the outside beams is approximately half that of the interior beams, the typical MAC number equation is modified to include the approximated mass matrix as a weighting factor. The mass-weighted MAC number for shapes $\phi_i$ and $\phi_j$ is given in Eq. 1:

$$MAC_{ij} = \frac{(\phi_i^TM\phi_j)^2}{(\phi_i^TM\phi_i)(\phi_j^TM\phi_j)}$$

The mass-weighted MAC numbers comparing the experimentally determined mode shapes with the computational predictions is given in Table 1. The MAC numbers are all close to unity due to the large rigid body contribution present in typical first mode responses as mentioned previously.

<table>
<thead>
<tr>
<th>Computational</th>
<th>None</th>
<th>NW</th>
<th>NW,SW</th>
<th>NW,NE</th>
<th>NW,SW,NE</th>
<th>NW,SW,NE,SE</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>0.9996</td>
<td>0.9958</td>
<td>0.9804</td>
<td>0.9990</td>
<td>0.9880</td>
<td>0.9969</td>
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<tr>
<td>NW</td>
<td>0.9960</td>
<td>0.9994</td>
<td>0.9939</td>
<td>0.9972</td>
<td>0.9977</td>
<td>0.9992</td>
</tr>
<tr>
<td>NW,SW</td>
<td>0.9753</td>
<td>0.9896</td>
<td>0.9990</td>
<td>0.9793</td>
<td>0.9970</td>
<td>0.9873</td>
</tr>
<tr>
<td>NW,NE</td>
<td>0.9998</td>
<td>0.9972</td>
<td>0.9832</td>
<td>0.9998</td>
<td>0.9901</td>
<td>0.9983</td>
</tr>
<tr>
<td>NW,SW,NE</td>
<td>0.9874</td>
<td>0.9968</td>
<td>0.9991</td>
<td>0.9904</td>
<td>0.9997</td>
<td>0.9955</td>
</tr>
<tr>
<td>NW,SW,NE,SE</td>
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<td>0.9988</td>
<td>0.9882</td>
<td>0.9995</td>
<td>0.9936</td>
<td>0.9995</td>
</tr>
</tbody>
</table>

The columns of Table 1 compare how the experimentally determined first NS mode shape for given brace configuration compares to the various computational predictions. The green entries down the diagonal represent the where the highest MAC number should be if the computational model correctly predicts the experimental setup. The off-diagonal orange entries represent errors in which another computational model better predicts the experimental shape. In four of the six
brace configurations, use of the MAC numbers alone correctly identifies the brace configuration. However, for the case of no braces attached and the case with both west side braces attached, use of the MAC number alone would lead to a false prediction.

For the experimental case in which no braces were attached, the computational symmetric case of one brace on each side (NW, NE) is slightly favored over no braces. The similarity between the mode shapes in both cases can be seen in Fig. 5. The experimental mode shapes are similar to each other with only a mild increase in diaphragm deformation as more braces are attached. The effect is picked up by the computational model. However the diaphragm flexibility is less pronounced in the computational model — leading to the misidentification. As mentioned previously, the attachment of the roof deck to the roof beams is uncertain. It is thought that a more detailed investigation and model of the roof deck may improve the results.

The other brace configuration misidentified by the MAC numbers is the case in which only two braces are attached to the west side (NW, SW). In Fig. 5 it is clear that the experimental shapes for the NW,SW case and the NW,SW,NE case are very similar with only a slight additional counter-clockwise rigid body rotation in the NW,SW case. The computational model has captured this additional rotation, but exhibits too much of it. Thus the MAC number suffers. It is likely that there exists additional stiffness from the cladding in the east moment resisting frame that has not been adequately represented in the computational model.

Conclusions

The use of the mass-weighted MAC numbers for the first NS mode shapes, obtained through forced vibration testing, with the shapes predicted by a reasonably accurate computational model was successful in identifying the brace configuration (damage). However, for two cases in which the computational model was not sufficiently accurate, an incorrect configuration exhibited a slightly better MAC number. At the same time it should be noted that the use of natural frequencies was unable to predict any brace configuration. A deeper investigation into how detailed a model is needed in order the correctly predict all brace configurations is warranted. In addition, it would be useful to test braces with smaller cross-sections to see if the results are similar.

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References


