Benchmark Models for Experimental Calibration of Seismic Fragility of Buildings

by Andrei M. Reinhorn, Michael C. Constantinou and Dyah Kusumastuti,
University at Buffalo, State University of New York

Research Objectives

The seismic fragility of buildings is a performance measure, which is difficult to compute. Empirical evaluations require disasters, or field damage, for construction of such fragility, thus, analytical techniques were developed. However, the analytical tools need calibration for either the accuracy of prediction of response, or prediction of damage limit states in engineering terms, both components of fragility. The purpose of this research is to develop a benchmark model, which can be damaged for the above studies, but repairable using inexpensive means for further studies. Moreover, the model should be able to display damage due to irregularities, torsion, setbacks, or other types of damage. The current research was dedicated to the design of the benchmark model and preparations for construction.

The analytical project of MCEER (Tasks 1.5 and 2.5) explored several alternatives to develop fragility information. The fragility is the probability that the expected response of a structure, or component, will exceed a limit state during an expected level of ground shaking. A limit state usually represents, in the same terms as the response, a damage condition or a limitation of usage or other special condition. This analytical project is intended to develop a rational, simplified procedure to calculate fragility curves based on the simplified spectral approach, and verify it with more rigorous alternatives. The development requires:

• Calculation of the expected response after the onset of damage.
• Determining meaningful characteristics of “limit states” and their uncertainties, which are otherwise loosely defined by qualitative methods.
• Redefine the meaningful levels of ground shaking and their representation in fragility analysis.

Several procedures developed to evaluate seismic response and fragility use information at various degrees of detail regarding the structural model and ground motion. The level of detail of the information used determines the uncertainties involved in the resulting fragility curves. The fragility information is therefore developed along with the degrees of confidence.
in such information. The target of the analytical studies is a simple method based on nonlinear spectral analysis using linearized inelastic spectrum of suites of ground motions and its statistical distribution and a spectral inelastic capacity model representation of structure (obtained from static nonlinear analysis) in the presence of uncertainties. The method can be further simplified to use nonlinear spectral analysis using code type inelastic spectrum and its statistical distribution and simplified code spectral inelastic capacity model representation of the structure (obtained from static nonlinear analysis) in the presence of uncertainties.

The new method of analytical evaluation requires an experimental validation of the computational tool. Moreover, the definition of damage limit states in a structural system comprising many components (beams, columns, braces, joints, connections, etc.) requires quantification based on observed performance.

The availability of a large shaking table at the University at Buffalo made it possible to develop a series of models which can be tested and retested in various damage conditions. One 1:2 scale model, which was prepared and tested for evaluation of “toggle braces,” was also tested without any braces to provide data for the analytical verification of the computational tools. The model is a single story structure with story height of 6 feet 4 inches (approximately) and is shown in Figure 1. The horizontal span of the two identical frames for each direction is 8 feet 4 inches.

The gravity load applied is given by two concrete blocks with total weight of 32 kips. The detailing of connections and additional devices placed on the structure are given in the Appendix of Constantinou et al., 1997 or at http://civil.eng.buffalo.edu/users_ntwk/index.htm. The drawings show the plan view and elevations of the model, and detailing for connections and additional devices.

All MCEER researchers dealing with the analytical evaluation of fragility will use this research. The model will allow the testing of integration of innovative devices and systems in structures to improve their behavior for retrofit or repair. Moreover, since a benchmark model will be used, developers of computational tools will be able to calibrate their software based on common experimental evidence.
The main benchmark 1:3 scale model was designed to have various configurations of the lateral loading system, while the floor masses will remain rigid and undamaged. The model is being prepared for construction.

**Benchmark Model**

The model was designed to be built with three to five stories using mass simulation and has removable components, which can be replaced by advanced components of new materials or functions. The design of the model was based on the following principles:

- Have a series of relevant sacrificial structural systems which can be “safely damaged” to collapse;
- Include a secondary system to carry gravity loads for control of stability;
- Use replaceable parts, which require minimum reinvestment;
- Produce damaged conditions at low levels of shaking, which fit the capability of the shaking table;
- Provide for accommodating new systems for retrofit or upgrading;
- Provide for incorporating new methodology of rocking columns;
- Provide for instrumenting and monitoring structural characteristics beyond the onset of damage;
- Allow the model to be reconfigured as an irregular structure for studies of torsion, impact (pounding) and 3D behavior;
- Design the model with detailed form materials and parts available in the industry.

For more information on MCEER’s experimental/computational users network, see [http://civil.eng.buffalo.edu/users_ntwk/index.htm](http://civil.eng.buffalo.edu/users_ntwk/index.htm).
Three alternative configurations (see Figure 3) were detailed and their construction is expected to be completed in the summer of 1999. Figure 2 shows one alternative design of the model.

The benchmark model was designed with gravity load carrying rocking columns, which are not damaged in a seismic event (see Figure 4). The separation of the vertical and lateral load carrying system will be studied as an alternative for new construction or retrofit. The system can be realized in full scale without incurring large costs.

The structure is designed with replaceable side frames or other lateral load carrying systems. The connections are such that the load is transferred into the joint and does not affect the other components. The system was designed to yield at low levels of excitation and to develop near ultimate lateral capacity.

The model was evaluated using a nonlinear time history analysis based on IDARC2D (Valles et al, 1996) and an approximation based on a nonlinear spectral capacity procedure (Reinhorn, 1997). The approximate evaluation consists of using the composite nonlinear response spectrum derived from the Elastic Design Spectra (DARS), $S'_E$ vs. $S'_I$, to obtain the Inelastic Design Spectra (IDARS) [Reinhorn, 1997 - based on Krawinkler and Nasser]

$$Sd^I = \frac{Sd^E}{R} \left[ 1 + \frac{1}{c} \left( R^c - 1 \right) \right]$$

$$Sd^I = \frac{Sd^E}{R} \left[ 1 + \alpha \left( \frac{Sd^I}{u_y} - 1 \right) \right]$$

$$R = \frac{Sd^E W}{Q_y g} \quad u_y = \frac{Sd^E}{R}$$

$$c = \frac{T_0^a}{1 + T_0^a} + \frac{b}{T_0}$$

$$a = 1.0; \quad b = 0.37 \quad (\alpha = 2\%)$$

where the superscript $E$ indicates elastic, versus $I$ indicating inelastic, $Q_y$ is the lateral yield resistance, $W$ is the total weight and all other constants defined in the text or the relation. The spectral capacity is characteristic for any structure and was obtained from the actual base shear, $BS$, and the top floor displacement obtained from a nonlinear static procedure (IDARC2D) according to the following relations:

$$Q^* = \frac{BS}{\Gamma^2 g}; \quad u^* = \frac{u}{\Gamma \phi_1}$$

where $\Gamma$ and $\phi$ are the modal characteristics of the dominant mode.
The spectral characteristics $Q^*$ vs. $u^*$ were then compared to the $S_a$ vs. $S_d$ demand to estimate the response (Reinhorn, 1997).

A sensitivity analysis was performed on the three types of models to obtain an optimal construction and generate sufficient observable damage. Table 1 shows the influence of changing the floor weight from 40 to 53 mtons, i.e. 30%.

The apparent response ductility, $\mu$, changes by 10% when the weight increases by 30%. This is an inefficient way to increase inelastic response or for other designs to reduce the dynamic response.

The structural changes at the first floor using heavier steel shapes (S4 x 7.7 instead of S3 x 5.7) produce a reduction of 25% in the ductility (see Table 2). Although the strength increases substantially, the deformation reduces in smaller proportion.

The most sensitive parameter on the response and inelastic response is ground motion (see Table 3 and Figure 5). An increase in the ground motion produces almost proportional increases in the inelastic deformation. The sensitivity analysis shows that with the maximum shaking table input, a global ductility of 3 is feasible. This ductility translates to larger local ductilities and spread plasticity effects.

The benchmark model was designed here with steel frames, however, construction details were prepared to allow concrete frames to be used in a symmetric or non-symmetric way. Moreover, the model was designed to allow for mass eccentricity to permit testing of torsional effects and triaxial interaction between individual components. Computational models are currently being developed (Simeonov et al., 1999) which will require experimental validation within the global system.

Most importantly, the benchmark model should be able to verify and validate the new spectral capacity approach and its adequacy

---

**Table 1. Weight Sensitivity (Model 1 - 40 mtons, Model 2 - 53 mtons)**

<table>
<thead>
<tr>
<th>$T$ (sec)</th>
<th>$R$</th>
<th>$\mu$</th>
<th>$u^*/H$ (%)</th>
<th>$Q^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>1.41</td>
<td>2.75</td>
<td>2.63</td>
<td>5.4</td>
</tr>
<tr>
<td>Model 2</td>
<td>1.61</td>
<td>2.8</td>
<td>2.95</td>
<td>6.3</td>
</tr>
</tbody>
</table>

**Table 2. Sensitivity to Structural Changes for Single Bay Frame**

<table>
<thead>
<tr>
<th>$T$ (sec)</th>
<th>$R$</th>
<th>$\mu$</th>
<th>$u^*/H$ (%)</th>
<th>$Q^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>1.41</td>
<td>1.65</td>
<td>1.62</td>
<td>3.3</td>
</tr>
<tr>
<td>Model 3</td>
<td>1.16</td>
<td>1.25</td>
<td>1.26</td>
<td>2.8</td>
</tr>
</tbody>
</table>

**Table 3. Sensitivity to Input Ground Motion**

<table>
<thead>
<tr>
<th>PGA</th>
<th>$R$</th>
<th>$\mu$</th>
<th>$u^*/H$ (%)</th>
<th>$Q^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>1.65</td>
<td>1.62</td>
<td>3.3</td>
<td>0.16</td>
</tr>
<tr>
<td>0.25</td>
<td>2</td>
<td>1.95</td>
<td>4</td>
<td>0.17</td>
</tr>
<tr>
<td>0.3</td>
<td>2.45</td>
<td>2.25</td>
<td>4.6</td>
<td>0.17</td>
</tr>
<tr>
<td>0.35</td>
<td>2.75</td>
<td>2.63</td>
<td>5.4</td>
<td>0.175</td>
</tr>
<tr>
<td>0.4</td>
<td>3.1</td>
<td>3</td>
<td>6.1</td>
<td>0.18</td>
</tr>
</tbody>
</table>

---

**Figure 5. Spectral Response Evaluation of Model to Increasing Ground Motion**

---

*Developing Benchmark Models for Experimental Testing*
to new innovative protective systems, such as nonlinear dampers and active or semi-active systems. The model details will accommodate the addition of braces, removal of columns or other members, and so forth.

**Conclusion and Future Research**

The benchmark model (under construction) was also designed to reflect construction issues which are part of the MCEER hospital demonstration project. The model will be tested to determine the limit states associated with such construction and data interpretation will enable development of fragility information. The issues already identified are the irregularities in construction and distribution of floor weights, and the influence of nonstructural and architectural components. These can be simulated in the construction of the model for uncomplicated testing. The past experience of the authors with reuse of structural models enabled them to design the model with the capability to accommodate modern protective systems, while still producing inelastic nonlinear behavior.

The model will first be tested to obtain simple elastic and then inelastic behavior to calibrate the analytical tools. The results will be made available via the web to the experimental/computational users network of MCEER (http://civil.eng.buffalo.edu/users_ntwk/index.htm - temporary address). The computational models will also be presented for further reference.

The immediate future plans for this project are to construct and instrument the model, including motion and force sensors, with multiple usage for immediate testing of several steel frames. One particular configuration, a candidate for the first round of testing, is the multiple towers building (see Figure 3(c)). Preparations are currently being made for this testing.

**References**


