EXPERIMENTAL SEISMIC RESPONSE OF A FULL-SCALE SIX- STORY WOOD APARTMENT BUILDING

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ABSTRACT: In July 2009 a full-scale mid-rise light-frame wood apartment building was subjected to a series of earthquakes at the worlds largest shake table in Miki, Japan. The test program consisted of two major phases: the first phase building consisted of a single story of steel with six stories of wood on top, and the second phase consisted of locking down the steel story and testing the six-story light-frame wood building. This paper focuses on the testing of the six-story light-frame wood building. The objectives of the test program were to (1) demonstrate that the performance-based design procedure developed as part of the NEESWood project worked, i.e. validate the approach; and (2) gain a better understanding of how mid-rise light-frame wood buildings respond in a major earthquake while providing a landmark data set to the seismic engineering research community. The building had 1350 square meters of living space and consisted of twenty-three apartment units: approximately half one-bedroom units and half two- bedroom units. The building was subjected to three earthquakes ranging from seismic intensities corresponding to the 72 year to the 2500 year event for Los Angeles, CA over a two week period. An anchor tiedown system, essentially steel rods running from the bottom of building to roof level at the ends of each shear wall, were used to prevent overturning and allow the shear walls to engage rather than uplift. The building, known as the NEESWood Capstone building, was instrumented with over 300 sensors and 50 LED optical tracking points to measure the component and global responses, respectively. In this paper the construction of the building is explained and the resulting seismic response in terms of base shears, wall drifts, global inter-story drifts, accelerations, and roof drifts are presented. Detailed damage inspection was performed following each test and those results are also presented in detail. The building was found to perform excellently with little damage even following the 2500 year earthquake. The global drift at roof level was approximately 0.25 meters and maximum inter-story drifts were under 3%.

KEYWORDS: shake table testing, seismic response, mid-rise building, light-frame wood, earthquake
1. INTRODUCTION

Light-frame wood buildings represent the vast majority of the building stock in North America. Most of these types of buildings are single- and multi-family dwellings with a moderate percentage being light commercial construction. Over the last decade progress has been made to better understand the seismic response of light-frame wood buildings. These advances have, in turn, resulted in the evolution of building codes for these types of buildings. Specifically, practical procedures that facilitate the move toward performance-based seismic design (PBSD) for light-frame wood buildings were developed within the research community (Pei and van de Lindt, 2009, Liu and van de Lindt, 2009, Pang and Rosowsky, 2009, etc.) in recent years based on simplified system models as well as nonlinear time history simulation methods. While these PBSD procedures were numerically verified to be able to adequately address performance objectives including structural drift, components damage, human comfort, and economic losses, their effectiveness in real applications can only be validated through the performance of structures designed using these procedures under real or simulated earthquakes. Full-scale seismic tests, i.e. shake table testing has only been performed a handful of times worldwide for wood frame buildings. During the CUREE Caltech Woodframe project, Filiatrault et al (2001) tested a rectangular two-story house with an integrated one-car garage. The structure was limited by the size of the available shake table, but provided a state-of-the-art data set nonetheless. The building was subjected to two 1994 Northridge recordings, i.e. the Canoga Park and Rinaldi motions. It was concluded that overall the performance was adequate and that non-structural finishes such as gypsum wall board (GWB) and stucco contributed significantly by increasing both the strength and stiffness of the system (Folz and Filiatrault, 2004). A three-story apartment building with a tuck-under garage was also tested as part of the CUREE-Caltech Wood Frame Project (Mosalam, 2003). The results of that series of tests confirmed that these types of structures were prone
to torsional response and subsequent soft-story collapse mechanisms. In 2006, as part of the NEESWood project (van de Lindt et al, 2006), Filiatrault et al (2009) conducted full-scale tri-axial tests of a two-story three-bedroom 160m² (1800 sq ft) townhouse with an integrated two-car garage utilizing the twin shake tables at the University at Buffalo’s SEESL laboratory. This building, termed the Benchmark structure, was designed to the 1988 Uniform Building Code (UBC, 1988) and was intended to benchmark the seismic performance of existing buildings in California and other high seismic regions. The benchmark structure performed relatively well by seemingly protecting life safety of would-be occupants, but suffered substantial and costly damage. Filiatrault et al (2009) was also able to validate the earlier conclusion that non-structural elements such as GWB and exterior stucco significantly increase the strength and stiffness thereby contributing to the improved seismic performance of wood-frame buildings. Most of these shake table tests were conducted with a focus on performance of the structure designed using traditional design specifications (with Phase II of the Benchmark test as an exception which utilized innovative shearwall damping devices). A series of shake table tests on a six-story light frame wood building designed using the PBSD methodology developed in the NEESWood project was conducted in 2009 at Japan’s E-defense shake table. The test program, namely the NEESWood Capstone test, served the purpose of validating the effectiveness of the PBSD procedure proposed as well as adding to the understanding of the dynamic behavior of mid-rise wood frame buildings during earthquake events.

2. CAPSTONE TEST PROGRAM

The NSF-funded NEESWood Project was a four-year, five-university project whose objective was to develop a performance-based seismic design philosophy for mid-rise woodframe construction. From 2006-2008, the Direct Displacement Design (DDD) approach (Pang and Rosowsky, 2007) was extended to multi-story woodframe buildings, which is a key outcome of the project. In order to validate the DDD approach, the world’s largest shake table test was conducted at Japan’s E-defense laboratory in collaboration with numerous researchers and industry participants from the U.S., Japan, and Canada. The shear wall selection of the 1350 square meter (14,000 sq ft), six-story building was conducted using the DDD method, the shear transfer and continuous steel rod holdowns were designed based on a specified non-exceedence probability using SAPWood (Pei and van de Lindt, 2007) simulation results. As a PBSD process, the performance target of the building was selected as combination of maximum allowable inter-story drifts and seismic intensity level, as is shown in Table 1.

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The building was approximately 18m x 12m (60ft x 40ft) in plan view and about 17m (56 ft) tall. The elevation views, presented in Figure 1, show the significant openings on all sides of the building requiring shear wall stacks in many locations. The floor plan for the first story is shown in Figure 2 and consisted of two small one-bedroom units (Unit A) and two two-bedroom units (Unit B). The floor plan for stories two through five were the same as story one with only a slight change to unit A since no entrance door to the building was needed at those levels. The top story, story 6, was modified from the other stories to include one large two-bedroom unit. This change in floor plan meant some of the shear walls in story 5 did not extend into story 6. For reference, the short direction of the building is designated as the X direction and the long direction as the Y direction.

In between the seismic tests, white noise excitation tests were conducted to detect possible softening of the structure from the measured building fundamental periods. In addition to the white noise system identification, preliminary evaluation of the test results was also conducted in between tests to ensure the safety of the following test. In total three seismic tests and five white noise tests were conducted during the test of the six-story wood frame structure.

3. STRUCTURAL DETAILS OF CAPSTONE STRUCTURE

Since the design of the building was not based on any existing code but completely through performance-based design method, there are some critical differences between the Capstone structure and a typical multi-story wood frame building in existence.

While typical joist system were used for all floors, glulam members were used extensively under all of the wall lines in order to handle the high level of seismically induced shear demand from the 2500 year earthquake. Some of the shear walls in the lower stories have massive compression stud packs consisting of multiple accelerations in the X, Y, and Z directions of the un-scaled Canoga Park record, with the Y-component (which has a higher PGA value) applied in the long direction of the building. The ground motion was scaled to the peak ground acceleration levels listed in Table 2 to represent seismic hazard levels with 50%, 10%, and 2% probability of exceedance in 50 years, which corresponds to return periods of 72, 475, and 2500 years, respectively.

<table>
<thead>
<tr>
<th>Seismic hazard (Intensity) Level</th>
<th>Drift limit 50% in 50 yr</th>
<th>10% in 50 yr</th>
<th>2% in 50 yr</th>
<th>Near Fault</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level A (1%)</td>
<td>50% NE</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Level B (2%)</td>
<td>50% NE</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Level C (4%)</td>
<td>80% NE</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Level D (7%)</td>
<td>50% NE</td>
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The seismic testing program consisted of three levels of seismic intensity including a design-basis earthquake (DBE) and a maximum credible earthquake (MCE). Northridge ground motion recorded at the Canoga Park station which was used throughout the tests with different scale factors. Figure 3 shows the spectral accelerations in the X, Y, and Z directions of the un-scaled Canoga Park record.
2x6 studs lumped together at the ends near the hold-down devices. These studs were designed to resist mostly the dynamic compressive load induced by the racking behavior of shear walls as well as gravity. Some of these stud packs interfered with the installation of shear connector on the top and sill plates of the walls. In order to resolve this confliction, some of these packs were installed after the walls were erected in place with shear connection installed. A typical wall configuration for the Capstone building was shown in Figure 4.

Continuous tie-down system (Simpson ATS) was utilized for every shear wall in the structure. These ATS rods ran through the stories and integrated individual walls into shearwall stacks. The system is more effective in controlling uplift (overturning) of the wall systems than traditional hold-down system and is deemed necessary for multi-story wood frame construction, as it was shown in Figure 5. After the ATS system was installed, the sheathing panel for shearwall was attached and nailed following the nailing pattern specified in the design. Then drywall boards were attached with drywall screws at 16” spacing (including top and bottom, i.e. not a floating construction style). The ceiling was also installed since the drywall panels contribute to structural stiffness and mass. Finally additional seismic mass in form of steel plates (shown in Figure 6) were added to the floor in order to simulate the design weight for the building.

The NEESWood Capstone test structure was heavily instrumented with over 300 channels of strain, deformation, and acceleration measurements connected to the high speed data acquisition system at E-defense. The absolute displacement measurements were obtained using 50 three-dimensional optical tracking sensors attached to the exterior of the building whose motion during the test was captured and processed using a 3D motion tracking system consisting of multiple high-speed cameras and special software programs. A list of instrumentation installed is shown in Table 3. The completed structure ready to test is shown in Figure 7.
3. STRUCTURAL RESPONSE DURING TESTS

As mentioned above, over 350 measurements were being recorded during each test, and a summary and discussion of the experimental response is provided below.

3.1 NATURAL PERIOD OF THE BUILDING

White noise excitation was input in each direction of the building in order to identify the natural periods of the structure before and after each seismic test. There was no significant change in the building fundamental period at each stage of the test program. The natural period of the building was close to 0.41 sec in the beginning of the test. The period increased to 0.49 sec after the MCE earthquake. The main reason for this change in natural period was the damage it received from the shaking. This magnitude of change in the natural period indicated that the damage to the building is quite limited, which confirmed the damage assessment results after the tests. The building period under the white noise excitation in both directions agree well with one another indicating a similar stiffness for both directions, which was consistent with the performance-based seismic design approach which had the goal of providing the same stiffness and strength in both directions.

3.2 GLOBAL RESPONSES

The averaged displacement at the centroid of floor diaphragm can be estimated based on the measurements from 7 optical tracking markers for each floor. The maximum roof displacements relative to the shake table were measured to be 60mm, 140mm, and 211mm for seismic intensities 1, 2, and 3 respectively. The maximum displacement occurred in the long direction of the floor plan, namely the Y direction. The time history responses of the roof level for all seismic tests were shown in Figure 8.

![Figure 8: Roof level displacement responses](image)

Although the building was designed to be symmetric and added seismic mass was distributed approximately uniformly over the building, torsional response was clearly observed during testing. The torsional response was in synchronization with the lateral response of the building, which means the time point at which the torsion reached maximum value is very close to the occurrence of the maximum value of one of the lateral responses. Due to the presence of torsion, the maximum inter-story drift of some shear walls near the building corners slightly exceeded 3% during the level 3 seismic test. Figure 9 showed the measured torsional response at the roof level in level 3 test, as well as the shape of the deformed floor diaphragm (exaggerated for clarity) at different point in time history.

![Figure 9: Torsional response measured at roof level](image)

With the seismic mass known and the floor acceleration measured, the time history for each story shear can be calculated. The global hysteresis of the building, defined as the base shear versus roof displacement, is presented in Figure 10. Like the roof displacement records, similar behavior from one seismic intensity level to the next is observed since the same ground motion (Northridge-Canoga Park) was scaled for each test. The ultimate base shear capacity of the building was designed to be approximately 2500 kN (562 kips), and from inspection of the hysteresis one can see that for the Y direction at intensity level 3 the test structure resisted 1824 kN (410 kips) which is approximately 73% of the ultimate base shear capacity. Figure 11 shows the normalized base shear (normalized by the seismic weight of the building) as a function of spectral acceleration at the building period.

![Figure 10: Global hysteresis for all test levels](image)

![Figure 11: Normalized base shear](image)
3.3 INTER-STORY DRIFT RESPONSES

The distribution of inter-story drift of the building under all three seismic test is close to uniform among the stories, which indicates the absence of the soft story mechanism observed in many wood frame buildings during large earthquakes (such as the 1994 Northridge and the 1995 Kobe earthquakes). The approach used to design the Capstone test structure which is outlined in Pang et al (2009) vertically distributes the shears according to the deformed state of the structure essentially eliminating or at least drastically reducing the probability of a soft story being present. Figure 12 presents the response time histories in the X and Y direction at all three seismic intensity levels for the first story as well as for the story that had the largest transient drift. Interestingly, the maximum drifts were observed in the upper stories instead of the bottom story.

![Figure 12: Inter-story drift response from seismic test 3.](image)

3.4 FLOOR ACCELERATION

Another major concern for multi-story buildings is the safety issues related to the high lateral accelerations in the upper stories that may result in occupant injury or even casualty due to heavy objects, e.g., furniture. The average acceleration that will be experienced by the occupants during the test was calculated by spatially averaging the acceleration measured by the five sensors installed at each floor level. It turned out the maximum lateral acceleration for the DBE level earthquake on the top story was about 1.3 g. The maximum acceleration for the MCE earthquake on the top floor was close to 1.6 g.

3.5 ATS ANCHOR TIE-DOWN SYSTEM

Recall from the earlier discussion on instrumentation that the anchor tie-down rods were instrumented for force measurements. Figure 13 shows the distribution of maximum measured differential forces for the rods throughout the first floor during the level 3 seismic test. The spatial distribution of the peak rod forces was similar for other test levels while the value of the force decreased. With the maximum uplift measured at more than 768 kN, it was quite clear that a high strength continuous hold down system must be in place for this type of building to survive a major earthquake such as the level 3 seismic intensity. Another observation from the rod force data is that the maximum hold-down forces do not necessarily occur at the same time.

![Figure 13: Maximum rod forces measured in seismic test 3 (kN).](image)

3.5 OBSERVED DAMAGE

As was mentioned earlier, the damage to the test structure from the three seismic tests was not felt to be significant even for the 2500 year earthquake. There was no visible damage to any structural components or assemblies of the building, with damage limited to the gypsum wall board (GWB). The GWB damage was observed primarily around the corners of openings as illustrated by the post-shake photographs in Figure 14. The damage and its correlation to inter-story drifts will be presented in its entirety in a forthcoming paper by several of the authors and can be found detailed in Pei et al. (2009).

![Figure 14: Observed GWB damage after seismic test 3](image)

4 CONCLUSIONS

A series of three shake table tests on a six-story light-frame wood building was completed in July 2009 in Miki, Japan. Designed with the performance based design procedure developed within the NEESWood project, the building was able to achieve very good performance under both DBE and MCE level
earthquakes, with spatially averaged inter-story drifts on the order of 2%. The damage to the structural and non-structural components of the building was very minor, all repairable. Peak shear wall drifts at one corner slightly exceeded 3% for the MCE level test. The Capstone building performed very well and did not experience a soft story mechanism at any of the test levels. The averaged floor accelerations were felt to be reasonable at the higher story levels, although objects would still need to be anchored as recommended by FEMA. Even with the approximately symmetric floor plan and evenly distributed seismic mass, considerable torsional response was still observed during the seismic tests. Inclusion of torsion is needed within PBSD for mid-rise light-frame wood buildings. The hold down system employed in the design of the structure serves the critical role of transferring uplift forces down to the foundation and thereby preventing overturning.

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