6th International Conference on Advances in Experimental Structural Engineering
11th International Workshop on Advanced Smart Materials and Smart Structures Technology
August 1-2, 2015, University of Illinois, Urbana-Champaign, United States

FULL-SCALE EXPERIMENAL SEISMIC COLLAPSE STUDY of a 4-STORY WOOD-FRAME BUILDING

P. Bahmani[1], J.W. van de Lindt[2]

1 *Engineer, KPFF Special Projects Division, Seattle, Washington, United States.*
E-mail: pouria.bahmani@colostate.edu
2 *George T. Abell Distinguished Professor in Infrastructure, Civil and Env Engineering, Colorado State University, Fort Collins, Colorado, United States.*
E-mail: jwv@engr.colostate.edu

ABSTRACT
In the west coast of the United States there are a large number of wood-frame buildings with garage space at ground level, resulting in open fronts on one or two sides. Buildings of this archetype are generally referred to as soft-story buildings. In order to investigate the behavior of soft-story buildings a full-scale four-story woodframe building was tested at the NEES at UC-San Diego outdoor shake table. The objectives of the collapse testing phase of the NEES-Soft project were to (1) observe and document the nature of the soft-story collapse mechanism and (2) quantify the collapse drift for these types of soft-story wood-frame buildings. The collapse testing phase for the full-story building used minimal but key instrumentation to capture enough information for numerical model calibration. A series of uni-directional shake table tests was conducted eventually collapsing the building with two successive shakes of the Superstition Hills ground motion record scaled to 1.8g spectral acceleration. The building collapsed at approximately 19% inter-story drift at the first story. Finally, the current deformation capacity as it relates to retrofit of these types of buildings is examined and preliminary recommendations for new collapse prevention drift limits for older woodframe buildings are discussed.

KEYWORDS: *Woodframe, collapse, full-scale, shake table, NEES*

1. INTRODUCTION

The 1989 Loma Prieta and 1994 Northridge earthquakes in California caused extensive damage to multi-story woodframe buildings. The majority of woodframe buildings that suffered damage or collapse had large openings at the first story, typically for garage parking, and high density of partition walls in the floors above. This layout often results in first floor stiffness deficiency as compared to stories above and is classified as a soft story building. These deficiencies cause the earthquake resistance of the first story to be significantly lower than the upper stories resulting in a premature failure of the building under a moderate to large earthquakes because the majority of the building drift is at the first floor. A number of full-scale tests of woodframe buildings have been performed around the world over the last two decades with much of the body of work originating in Japan. A detailed summary of full-scale woodframe test programs is provided in a 2009 report prepared by the National Association of Home Builders Research Center. Several tests have specific relevance to the building examined herein. Filiatrault et al. (2002) tested a rectangular two-story house with an integrated one-car garage. As part of the NEESWood Project (Filiatrault et al., 2010) conducted full-scale tri-axial tests on a two-story three-bedroom 167 m^2 (1800 sq ft) townhouse with an integrated two-car garage using the twin shake tables at the State University of New York at Buffalo. The world's largest shake table test of a 1,300 m^2 (14,000 sq ft) six-story apartment building at Japan's E-defense facility in Miki, Japan was conducted as part of the NEESWood Project by van de Lindt et al (2010). None of these projects, however, tested buildings to collapse because of the complexities associated with this type of testing. In fact, full-scale collapse testing of woodframe buildings subjected to seismic load has been conducted only a few times worldwide. The laboratory equipment requirements and safety provisions needed for these types of tests can be quite complex and costly and are available in only a few laboratories worldwide. Sakamoto et. al (2002) discussed the planning of a series of tests on a full-scale two-story town house at the E-Defense laboratory in Miki, Japan as part of DAI-DAI-Toku project. The testing then occurred several years later at the Grand Opening of the laboratory. In 2004, a two-story Japanese conventional woodframe house was tested to investigate the collapse mechanism and predict the collapse margin for these types of buildings (Miyake et al., 2004 and Koshihara et al., 2004). The need for investigating the collapse of western style woodframe buildings in the United States is critical for the following reasons: (1) there is no data for full-scale mid-rise residential buildings subjected to large drifts; and (2) the tests in Japan were on buildings representative of conventional post and beam, not light woodframe construction which represents in excess of 80% of the U.S. building stock and 99% of residential stock in California. Thus, collapse testing of a full-scale building subjected to seismic loads was considered valuable to better understanding the collapse behavior of these types of buildings. This study is the first experimental test of its kind that helps to: 1) better understand the behavior of light woodframe buildings near and at collapse; 2) quantify the collapse displacement; and 3) investigate the collapse mechanism of soft-story buildings.

2. BUILDING DESIGN AND CONSTRUCTION

The test building plan dimensions were dictated by the shake table size which was 7.6 m × 12.2 m (25 ft × 40 ft) resulting in plan dimensions of 7.3 m × 11.6 m (24 ft × 38 ft). Figure 1 shows the floor plans of the first story (soft-story) and upper stories. Each of the upper three stories had two two-bedroom apartment units as can be seen in Figure 1 (b). On the first floor, there was a garage space for four cars, a large laundry room, a storage room, and a light well. The light well was included since many of these buildings are surrounded by other buildings on two sides (i.e., north and west sides in the test building) and therefore have two essentially solid sides and two open sides. The building was soft and weak on two adjacent sides resulting in significant torsion.

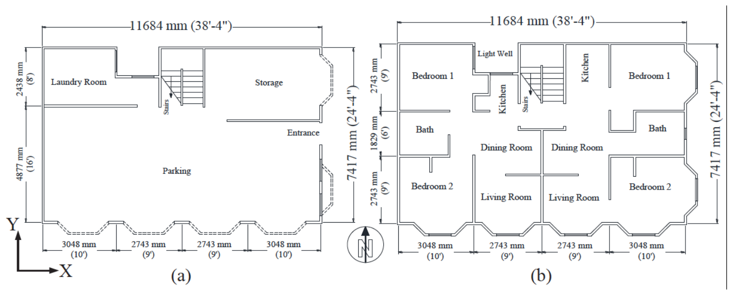

Fig 1: Comparison of the architecture for a soft-story wood building (a) in the San Francisco Bay Area, and (b) designed as part of the test building for the NEES-Soft project.

Figure 2 presents the construction sequence for the four-story woodframe building built on top of the shake table at NEES@UCSD. This is the largest and only shake table in the United States capable of conducting this type of test. In order to expedite the construction time, wall and floor framing assemblies were pre-fabricated and moved to the shake table once it became available for building erection. Interface steel framing beams for connecting the framing to the shake table were designed and fabricated at Colorado State University, shipped to the USCD site and installed on top of the shake table.

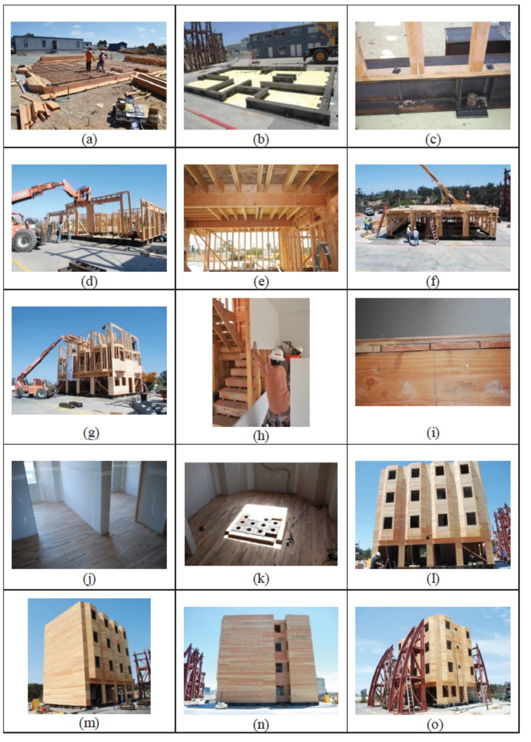

Fig 2: Floor Plans for the four-story test building: (a) First story, and (b) Upper stories.

3. COLLAPSE TEST PLANNING AND DESIGN

Prior to conducting the collapse test it was necessary to perform feasibility checks to determine if the shake table had sufficient capacity and if equipment and personnel safety might be compromised in any way. The shake table at NEES@UCSD can provide 1.2g base acceleration for a 400 ton payload. The total weight of the building including the steel interface framing was about 60.3 tons (135 kips) and the maximum ground acceleration that would be generated by the entire set of test ground motions was approximately 1.0g thus, the shake table was capable of accommodating the proposed collapse test.

The next challenging part of the collapse test was predicting the potential landing location of the collapsing building and if necessary reinforcing that area to avoid damaging the equipment underneath the shake table. Figure 3 presents the shake table with the building footprint shown in bold. The gap between the concrete slab and the shake table which allows the shake table to move, was covered by a 25.4 mm (1 in.) thick steel plate (labeled the steel platen in Figure 3) bolted to the top of the shake table and allowed to slide over the concrete slab. Temporary shoring of the concrete slab from underneath the shake table was provided by the UCSD site team, thus effectively reducing the span and corresponding slab maximum bending moment that might be caused by the impact of the building collapse. The shored slab and the steel plate also protected the actuators from possible damage. The building was expected to collapse entirely on its first floor and lean toward the south-east or south-west safety tower. The height of the first story was 2.7 m (9 ft); therefore, the collapse area was expected to be approximately 3.7 m (12 ft) from each side. In Figure 3(a), the shaded area represents the moving parts of the shake table assembly which includes the shake table itself and the steel platen attached to its west and east side. Figure 3(b) shows the south elevation view of the building (section A-A) erected on top of the shake table.

Three safety towers placed at each side of the building parallel to the motion of the shake table are shown in Figure 3(a). The distances from the face of the tower to the outside edge of the building was approximately 1.7 m (5.5 ft) allowing the building to collapse freely on the potential collapse area shown in Figure 3(a). The towers prevent excessive transverse movement of the building to protect the control room and other laboratory facilities located to the south side of the test building but would not affect the motion of the building. The concerns were effectively addressed, and the collapse test of the four-story building on top of the shake table was determined by the NEES-Soft project team and NEES@UCSD research staff and management to be feasible and safe.

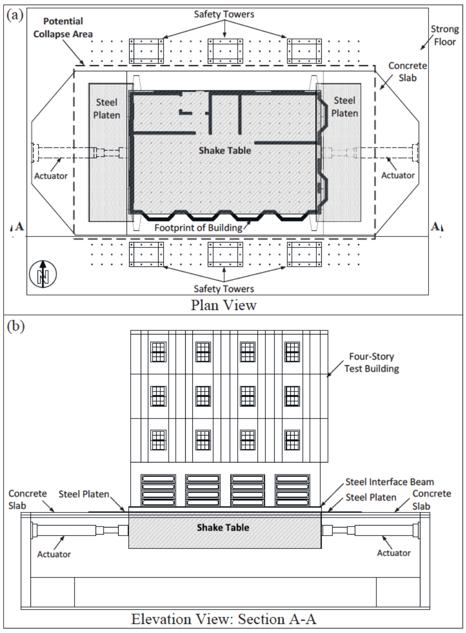

Fig 3: Elevation views of the test building: (a) East view, (b) South view, (c) West view, (d) North view.

4. GROUND MOTION RECORDS

To study the collapse mechanism and behavior of this type of at-risk building the building was subjected to a range of ground motions with different scaling. Three different ground motions with different intensities were selected. The selections were such that they would provide a range of earthquake records based on differences in ground displacement, even if the seismic intensity as determined through spectral acceleration was similar. The ground motions were then scaled to spectral accelerations ranging from S_a = 0.4g (33% of the design-based earthquake level) to S_a = 1.8g (maximum credible earthquake (MCE) level). The testing started with the Cape Mendocino-Rio station record with a PGA of 0.21g and ended with the Superstition Hills record with a PGA of 0.86g.

Figure 4 presents the response spectral acceleration of the ground motions used in the collapse test. It can be seen that the Loma Prieta and Cape Mendocino records significantly affect buildings when the fundamental period is less than 0.6 sec., whereas, the Superstition Hills ground motion has a substantial effect on the building at higher periods. Figure 4(d) presents the spectral acceleration of the three ground motions scaled to S_a = 1.8g. It can be seen that between the periods of T_n = 0.4 sec and T_n = 0.6 sec, which is the range of the periods for the retrofitted woodframe buildings when strength and stiffness is added, the maximum and minimum spectral accelerations are from the Cape Mendocino and Superstition Hills records, respectively. However, for a building with a period of greater than about T_n = 0.9 sec, which includes the unretrofitted building with a soft story tested here, the maximum spectral acceleration is clearly present in the Superstition Hills record. It can be seen from Figure 4(d) that the spectral acceleration corresponding to the Superstition Hills record was about 0.9g which has been shown to be enough to collapse typical soft-story buildings. Figure 5 presents the spectral displacements for the ground motions used in the collapse test. Superstition Hills has a very high spectral displacement for the periods higher than 1.0 sec. From Figure 5(g) it can be seen that the maximum ground displacement of the Superstition Hills record scaled to S_a = 1.8g is 277 mm (10.9 in.), having just displaced 170 mm in the other direction, then returning after the peak displacement to 250 mm. This type of ground motion is similar to the 1995 Kobe recording at the Takatori and JMA recording stations which have been used repeatedly in Japanese research projects, focused on studying collapse.

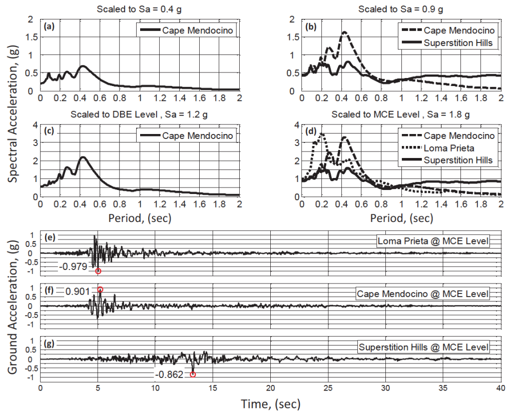

Fig 4: Construction sequence of the four-story woodframe building on top of the shake table.

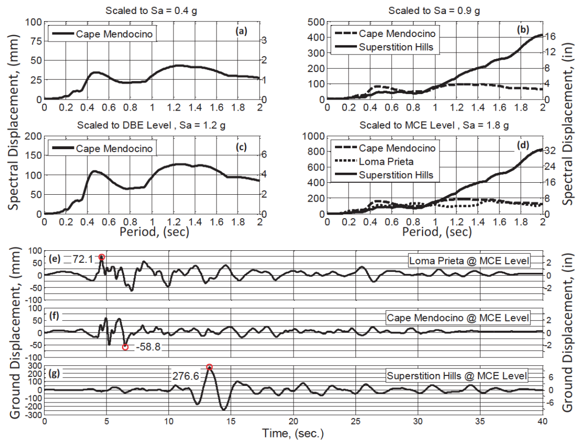

Fig 5: Position of the four-story building with its potential collapse area on top of shake table: (a) Plan view, and (b) Elevation view (Section A-A).

5. TEST RESULTS

The four-story woodframe building was subjected to eight successive seismic tests with several different ground motion records scaled to spectral accelerations ranging from 0.4g to 1.8g. The logic of ASCE7-10 (2010) was used for scaling ground motions over the period of 0.08 to 1.5 seconds to ensure a fair comparison between retrofitted and unretrofitted test building during the entire NEES-Soft test program. In order to find the building mode shapes and their corresponding periods, a white noise test of 0.05g RMS was conducted before the first seismic test. White noise tests were not performed between all seismic collapse tests. The initial period of the building right before starting the seismic tests was T_n= 0.99 sec which was very close to the fundamental period calculated from the numerical analysis. Due to safety regulations, no damage inspection and repair was conducted between each consecutive test; therefore, the structural and non-structural damage accumulated during the entire collapse test program. It was observed that the period of the building increased significantly after the fourth shake due to permanent structural damage and was likely between 1.5 and 2.0 seconds. Figure 6(a) presents the complete back-to-back seismic tests with their corresponding time-history of ground acceleration, velocity and displacements measured directly from the shake table feedback output. Figure 6(b) shows the translational and torsional response of the first story recorded from the north- and south-string potentiometers.

The maximum ISD was about 95.9 mm (3.78 in.) and 102 mm (4.02 in.) with almost no observed residual drift for the Cape Mendocino and Loma Prieta ground motions scaled to MCE level, respectively. The building was damaged but not near collapse. The last three tests that led to collapse of the building were conducted by subjecting the building to the Superstition Hills ground motion. As shown in Figures 5(d) and 6(d), this ground motion has very high response spectral acceleration and displacement for periods larger than 1.0 sec. Furthermore, from Figure 6(a) it can be seen that the ground motion velocity and displacements of the Superstition Hills record are larger than those for the two other ground motions. From Figure 6(b), it can be seen that the building experienced very high translational displacement and rotational movement during these seismic tests. In fact, the building never even passed through its original equilibrium position further underscoring the severity of the damage sustained during the Superstition Hills shake. The building was then subjected to the same ground motion but this time with it again scaled to MCE intensity, S_a=1.8g, which led to collapse of the building. The shake table was stopped after the full collapse to protect the lab equipment rather than allow the collapsed building to potentially be damaged further. It can be seen from the last column in Figure 6(b) that the building experienced about 635.9 mm (25.0 in.) of translational displacement and about 11 degrees of rotational movement at the onset of collapse. Maximum displacements of 470 mm (18.5 in.) (i.e., ISD ratio= 19.3%) and residual displacements of about 350 mm (13.8 in.) (ISD ratio= 14.4%) when subjected to the first Superstition Hills earthquake were observed. The building had about 400 mm (15.7 in.) of residual displacement (i.e., ISD ratio= 16.4%) leaning toward the west at the start of the ground motion. Then, it moved slowly further to the west for the last time then moved toward the east and rotated about 11 degrees before hitting the safety towers. Since the maximum stroke of the string potentiometers was ±635 mm (±25 inch), the last recorded displacement was 635.9 mm (25.0 in.), and after reaching this displacement the string potentiometers were. Figure 7 shows the photos of the collapsed building from different angles. It can be seen that the building rotated substantially before it collapsed and hit the safety towers at the south-east corner of the building. Also, the first story was completely destroyed as expected in a soft-story collapse mechanism.

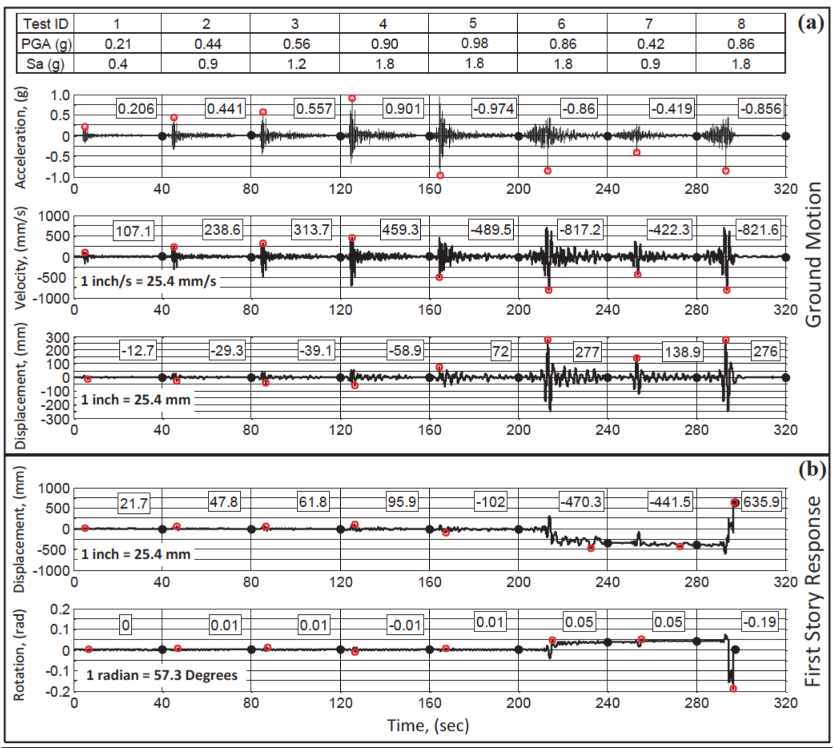

Fig 6: Time-history of ground motions and corresponding responses of the building during consecutive seismic tests.

(a) (b)

Fig 7: Photos of the collapsed building: (a) South-West view, and (b) North-East view.

6. CONCLUSIONS

The test building was designed to represent a corner building on a typical Bay Area street, which can be viewed as a worst case scenario among these types of soft-story buildings although not a typical. The building was subjected to three different ground motions scaled to four different scale factors. The following conclusions were reached as a result of this experimental program:

1) The maximum inter-story drift (ISD) ratio experienced at the first story before the collapse was 19.3% and maximum residual displacement was 16.4% just before the last test.
2) This experimental test confirmed that the upper limit of the collapse drift for the test building is close to 19% ISD ratio, likely between 14% and 19%; however, it was observed that the building was unrepairable and uninhabitable when it reached approximately 14% ISD ratio.
3) It was observed that the building collapsed over its soft-story (i.e., first story) and the upper stories moved as a rigid body on top of the first story during all earthquakes. The building was weak and soft in both directions at the first story which caused rotation toward the street (south) side when it collapsed.

ACKNOWLEDGEMENTS

This material is based upon work supported by the National Science Foundation under Grant No. CMMI-1041631 and 1314957 (NEES Research) and NEES Operations. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the investigators and do not necessarily reflect the views of the National Science Foundation. A sincere thank you to Simpson Strong Tie for their financial, personnel, and product support throughout the project, including the engineering support for the SMF in San Diego. The authors kindly acknowledge the other senior personnel of the NEES-Soft project: David V. Rosowsky at University of Vermont, Andre Filiatrault at University at Buffalo, Shiling Pei at South Dakota State University, David Mar at Tipping Mar, and Charles Chadwell at Cal-Poly; the other graduate students participating on the project: PhD student Jingjing Tian (RPI), MSc Students Chelsea Griffith (WMU), and Jason Au (CPP); and the practitioner advisory committee: Laurence Kornfield, Tom Van Dorpe, Doug Thompson, Kelly Cobeen, Janiele Maffei, Douglas Taylor, and Rose Grant. A special thank you to all of the REU students Sandra Gutierrez, Faith Silva, Gabriel Banuelos, Rocky Chen, Connie Tsui. Others that have helped include Asif Iqbal, BRANZ; New Zealand, Vaishak Gopi, Ed Santos, Tim Ellis, and Russell Ek. Finally, our sincere thank you to NEES and all site staff and site PI's, Jose Restrepo, Enrique Luco, and Joel Conte at NEES@UCSD for their help and support getting the tests ready.

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