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SHAKE TABLE TESTING OF A SEVEN-STORY MIXED-USE CONDOMINIUM AT JAPAN'S E-DEFENSE

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ABSTRACT

Mixed-use construction is becoming more popular as urban revitalization of downtown areas is being pursued by cities throughout the Western U.S. The NEESWood project, a four-year five-university project, whose objective was to provide the technical support to safely increase the height of light-frame wood buildings to six or more stories through the development of a performance-based design philosophy, culminated with a series of tests called the Capstone tests. The Capstone building was a seven-story, approximately 12m x 18m (40-ft by 60-ft) condominium tower with 23 one- and two-bedroom living units for a total of 36 bedrooms and space to accommodate two retail shops at level one. Living space of the building was approximately 1350 square meters (14000 sq ft). The building is a steel special moment frame at story 1 and light-frame wood construction at levels 2-7 which includes continuous rod hold down system throughout its six levels of wood shear walls. On June 30, 2009 the building was subjected to the Canoga Park recording of the Northridge earthquake scaled to 1.16 times the design-basis seismic intensity level at Japan's E-Defense (Earthquake-Defense) shake table and then on July 6 and July 14 testing at three seismic intensity levels took place, culminating with the MCE scaling of the Canoga Park record. The shake table is the largest in the world with the platform measuring approximately 20m by 15m (65 feet by 49 feet). This paper presents the results of the shake table tests and an assessment of the overall building performance.

Introduction

Light-frame wood buildings represent the vast majority of the building stock in North America. Most of these buildings are single- and multi-family dwellings with a moderate

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percentage being light commercial construction. The significant progress researchers have made over the last decade to better understand the seismic response of light-frame wood buildings has essentially facilitated a change in building codes. Full-scale seismic tests, i.e. shake table testing, usually with the goal of understanding these systems better has only been performed a handful of times worldwide for wood frame buildings. 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 State University of New York at Buffalo's SEESL laboratory. Several other tests were performed but are not included here for brevity. This paper presents the results of a series of five shake table tests on a full-scale seven-story mixed-use apartment building (six-story wood frame building on top of a steel moment frame first story). The 1560m2 (16800 sq ft) building represents the largest shake table test performed worldwide and provided key information on experimental seismic testing of a full size wood frame building. The two immediate objectives were: (1) to provide a general understanding of how mid-rise wood frame buildings perform in a major earthquake since there are four to six story buildings in place in the Western U.S.; and (2) provide some level of validation for the performance-based seismic design philosophy developed within the NEESWood project by demonstrating good performance at high seismic intensity.

Test Specimen

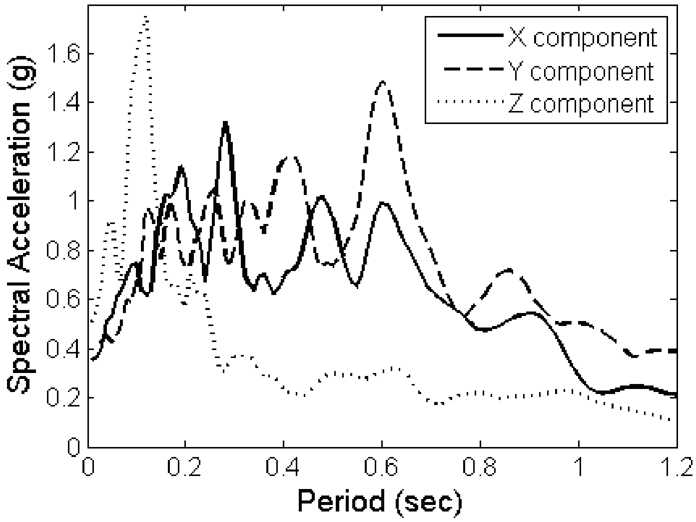
The building was approximately 18m x 12m (60ft x 40ft) in plan and just over 20m (66 ft) tall with the first story steel frame included. The floor plans for wood stories were similar except the top story which was opened up to provide one large two-bedroom unit. The steel moment frame was designed to be able to act as the lifting frame as well as the fixed test base once fully braced. The floor plans and elevation views of the test structure are presented in Figure 1. In this paper, the short direction of the building is designated as the X direction and the long direction as the Y direction. The shearwall selection was conducted using direct displacement design (DDD) (Pang and Rosowsky, 2007). The steel moment frame was designed to provide the strength and stiffness targets developed through the DDD process and utilized a new type of field-bolted beam to column connection. This new highly ductile connection precludes lateral-torsional buckling from becoming a failure mode in the beams, a necessary consideration because of the adjacent wood diaphragm's inability to provide the strength and stiffness necessary to brace beams in conventional steel special moment frames from this type of failure. A removable tension-only bracing system consisting of 2 inch diameter threaded rods was designed into the frame system to facilitate the wood-only portion of the project in tests three to five. This bracing also served as the web elements in the bi-directional truss system also designed into the frame to serve as the lifting mechanism to move the structure on and off the shake table.

Wood shear walls were designed as stacked wall systems with a combination of steel rod hold-downs with mechanical shrinkage compensating devices at each end to prevent overturning, reduce uplift, and remove slack from the hold-down system that would otherwise develop from in-situ reductions in wood moisture content and natural settling of the structure. Full construction details for shear walls in the Capstone building can be found in Pei et al (2009). In addition to the as-built dead load, seismic mass was added to each story in the form of steel plates in order to bring the total floor seismic mass to the design level thereby accounting for all the insulation, exterior finish, plumbing, HVAC, and floor finish including soundproofing. The total weight of the building was 362 ton (797 kips).

Figure 1. Elevation, floor plan, and photo of the test specimen.

Seismic Tests

The seismic test program consisted of multiple shake table tests during three separate test days. The Canoga Park record from the Northridge earthquake was scaled and used for all the seismic tests. Figure 2 shows the spectral accelerations in the X, Y, and Z directions of the un-scaled ground motion and the scale factors for each test (the Y-component applied in the long direction of the building). During the first two tests, the steel special moment frame (SMF) at the base of the building was not braced and therefore participated in the testing. The SMF was braced for the test 3 through 5 allowing the test of the wood-only six-story building. Instrumentation consisted of a number of gage types ranging from tri-axial accelerometers to a 3-D optical LED tracking system on the exterior of the building. For instrumentation details the interested reader is referred to the forthcoming Capstone test report by Pei et al. (2009). A brief summary of the overall instrumentation plan is provided in Table 1.



Seismic Test Scale Factors

Test No.	Scale Factor	Probability of Exceedance *
1	0.53	50% in 50 years
2	1.40	5% in 50 years
3	0.53	50% in 50 years
4	1.20	10% in 50 years
5	1.80	2% in 50 years

* Probability of exceedance is the approximate value at Los Angeles, CA

Figure 2. Un-scaled pseudo-acceleration response spectra for the Canoga Park recording of the 1994 Northridge earthquake (5% damping).

Table 1. Summary of instrumentation plan for Capstone test.

Measurement	Location	Type	Number
Absolute acceleration	Each Floor	3D-acceleration	38
Diagonal shear wall drift	Selected shear walls	String Potentiometer	33
Out of plane diaphragm deformation	Third floor diaphragm	String Potentiometer	13
Shear wall end stud uplift	Selected shear walls	String Potentiometer	8
ATS hold-down strain	Selected shear walls	Strain Gage	78
Absolute displacement	Building exterior	3D Optical tracking	50

Experimental Results

White noise excitation was input in each direction of the building in order to identify the natural periods of the specimen before and after each seismic test. The natural period of the building varied from 0.41 sec to 0.49 sec depending on whether the SMF was braced and softening due to damage following earthquakes.

The averaged displacement at the centroid of the floor diaphragm was estimated based on the measurements from seven optical tracking markers at each floor level on the exterior of the specimen. The maximum roof displacements relative to the building base were measured to be

166mm for the 7-story building and 211mm for the 6-story wood-only structure. The maximum displacement occurred in the long direction of the floor plan, namely the Y direction. The building deformation shapes at the point in time of the maximum roof displacement levels in the X and Y directions during the MCE earthquake (test No. 5) are presented in Figure 3. The shape of the deformed grid was generated directly from the optical tracking sensor measurements and is exaggerated for clarity. Due to the presence of torsion, the maximum inter-story drift of some shear walls at the upper levels near the building corners slightly exceeded 3% during this seismic test.

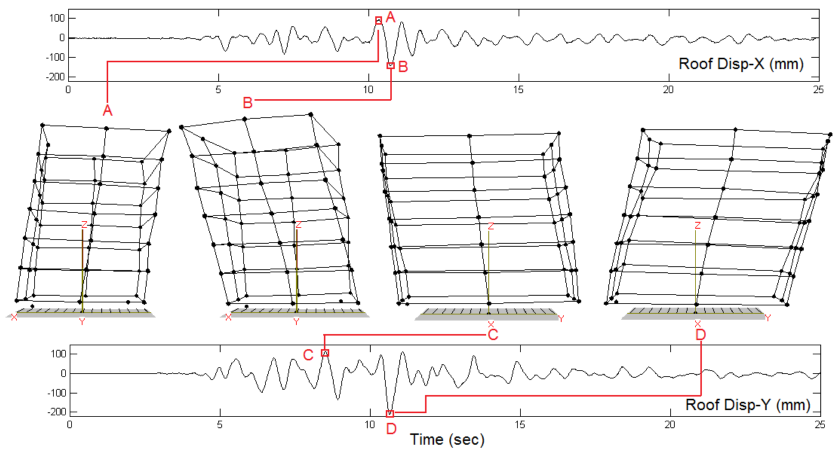


Figure 3. Time captures of the system deformation relative to the shake table during four points. in time for the LED optical tracking system.

The resulting inter-story drift of the Capstone building was calculated by subtracting the absolute displacement measurement between stories and dividing the value by the story height. The maximum values for the average inter-story drift are presented in Table 2.

Table 2. Averaged peak inter-story drift measured during the three tests.

Test No.	Direction	SMF (Story 1)	Wood Story					
			2	3	4	5	6	7
1	X	0.19	0.35	0.41	0.35	0.36	0.33	0.38
	Y	0.32	0.38	0.58	0.41	0.45	0.35	0.29
2	X	0.34	0.60	0.77	0.75	0.92	0.83	0.96
	Y	0.51	0.91	1.14	1.20	1.31	1.15	0.65
3	X	Fixed	0.26	0.35	0.29	0.30	0.36	0.40
	Y	Fixed	0.44	0.42	0.54	0.44	0.46	0.21
4	X	Fixed	0.49	0.63	0.64	0.77	0.64	0.88
	Y	Fixed	0.77	1.05	1.02	1.22	1.14	0.58
5	X	Fixed	0.84	0.97	0.89	1.10	1.00	1.35
	Y	Fixed	1.12	1.46	1.64	1.48	1.88	1.11

From the above table one can see that the heightwise distribution of inter-story drifts for the building under all seismic tests was close to uniform among the stories, which indicates the absence of a soft story mechanism. The drift of the SMF was measured to be less than the wood stories. Figure 4 presents the response time histories in the Y direction during seismic test No. 2

and 5 (strongest shake for each configuration) for the bottom story as well as for the story that had the largest peak inter-story drift. Interestingly, the maximum inter-story drifts were observed in the upper stories instead of the bottom story, which was, in fact, consistent with numerical model predictions performed prior to testing. 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 casualties due to the movement of heavy objects, e.g. furniture. The maximum floor lateral acceleration for the seven-story building was about 1.3g experienced on the seventh story during test 2. As for wood-only configuration, the maximum acceleration for the test 5 MCE earthquake on the highest occupied floor level (not the roof) was approximately 1.6 g.

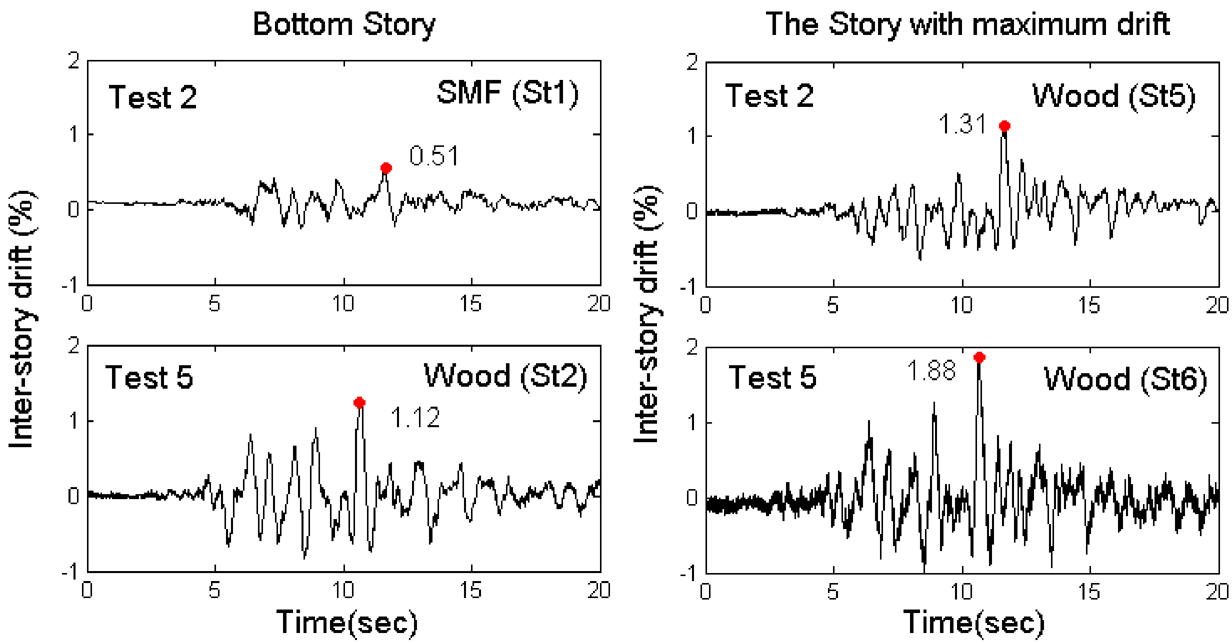


Figure 4. Time history plots for the average inter-story drifts.

The global hysteresis loops for the building are presented in Figure 5 and 6. 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. As the ultimate base shear capacity of the wood frame building, which was used in the performance-based seismic design, is about 2500 kN (562 kips), for the Y direction at seismic test 5 one can see that the test specimen resisted 1824 kN (410 kips) which is approximately 73% of the ultimate shear capacity.

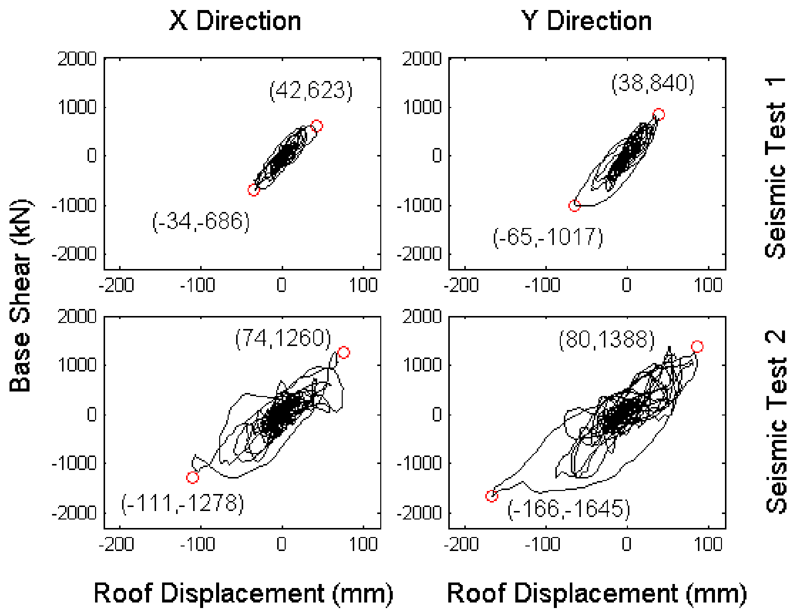


Figure 5. Global hysteresis for mixed-use configuration tests.

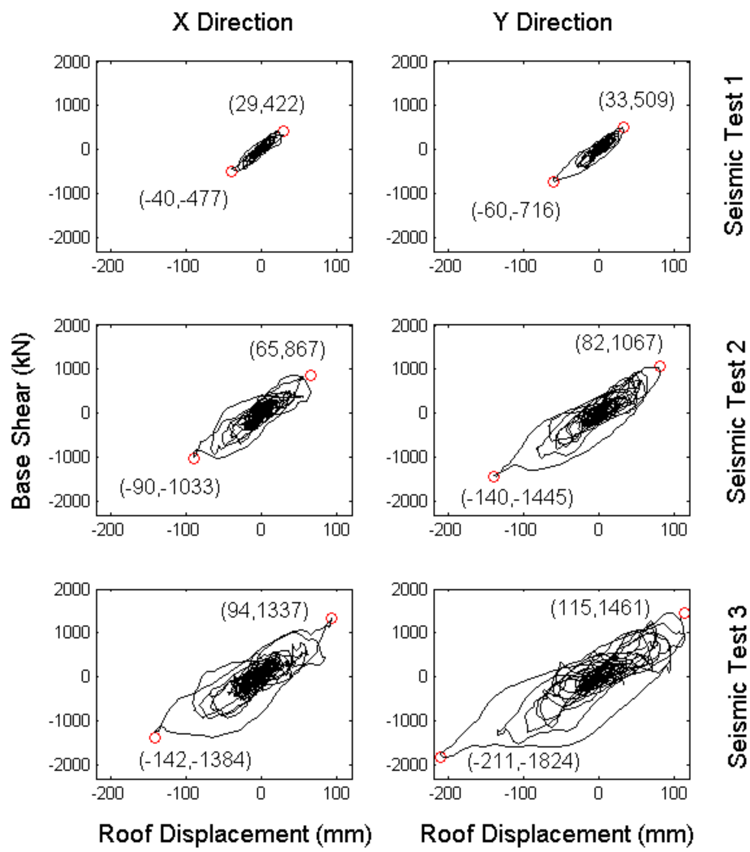


Figure 6. Global hysteresis for wood-only configuration tests.

Anchor tie-down rods (ATS) resisting shear wall overturning were instrumented for approximately half the building at the first wood story and were located at each end of each shear wall segment. The maximum uplift forces recorded during the tests indicated a high level of demand on the overturning resisting elements for multi-story wood frame buildings. The maximum tie-down tension forces for each test were listed in Table 3. Most of them (except for test 1) occurred at the same location as shown in Figure 7 with a circle. Figure 7 also shows the distribution of maximum forces for the rods throughout the floor during the MCE level seismic test (test 5). The spatial distribution of the peak rod forces was similar for other test levels while the value of the force decreased. To a large degree the behavior of the tie-down forces exhibited the expected alternating of tension from side to side as the building cycled during the test. The areas of largest observed holdown demand were next to very large openings in the wall line, with demands similar to that which would be obtained from a simple free body analysis of the stacked shear walls. In other areas where there was significant sheathed area above and below openings adjacent to the designated shear wall segments, the observed tie-down forces were less than that of a simple free body diagram analysis. Additionally, it appears that in some areas where two walls framed together in a ‘T’ fashion a composite behavior was achieved resulting in more than one designated tie-down rod resisting uplift at the end of the stem of the ‘T’. This is thought to be due in part to the glulam beam elements in the floor system used a shear collectors and bearing enhancers. More research is needed on this topic.

Table 3. Maximum tie-down rod tension force during tests.

Test No.	1	2	3	4	5
Maximum Tension (kN)	245	542	231	497	768

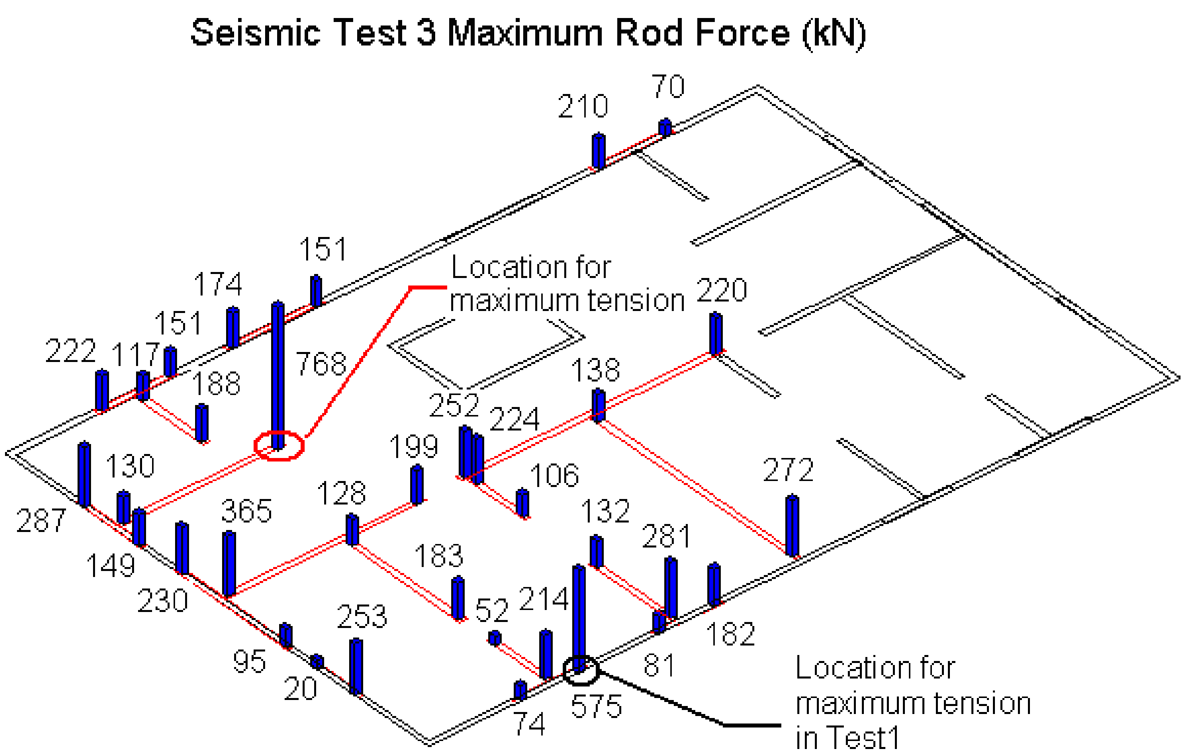


Figure 7. Maximum steel rod tension during test 5.

As mentioned earlier, the damage to the test specimen from the three seismic tests was not felt to be significant even for the 2500 year (MCE level) earthquake. There was no visible damage to any structural components or assemblies within 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 8. 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 the forthcoming report by Pei et al. (2009).

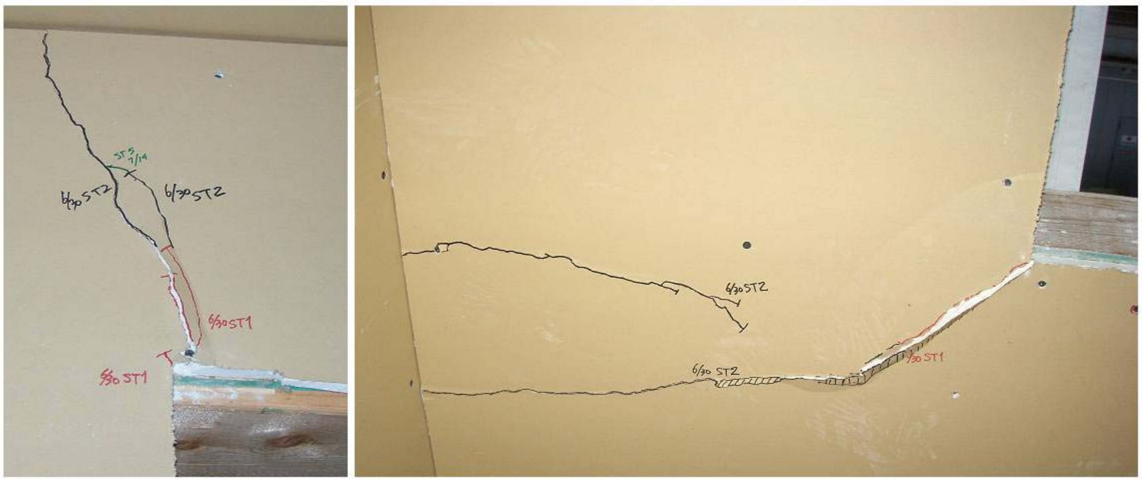


Figure 8. Typical damage observation around wall openings.

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Summary and Conclusions

A series of shake table tests on a seven-story mixed-use and then a six-story light-frame wood building were 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 maximum averaged inter-story drifts for the MCE test on the order of 2%. The damage to the structural and non-structural components of the building was very minor and easily repairable. Peak shear wall drifts at one corner slightly exceeded 3% during the MCE level test. The Capstone building performed very well and did not experience the start of 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 for occupant safety. Even with the approximately symmetric floor plan and evenly distributed seismic mass, considerable torsion was still observed during the seismic tests. It is clear that inclusion of torsion is needed within PBSD for mid-rise light-frame wood buildings. The hold down system employed in the design of the specimen serves the critical role of transferring uplift forces down to the foundation and thereby preventing overturning and controlling uplift. Although installed for each shear wall segment, tie-down rods from orthogonal walls may interact with each other to restrain uplift at the end of a given shear wall, and only at times did they act as semi-isolated shear wall stacks. In light of this observation, development of a system level design procedure for hold down rods used in multi-story woodframe construction is recommended.

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