Damage Assessment of a Full-Scale Six-Story Wood-Frame Building Following Triaxial Shake Table Tests

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Abstract: In the summer of 2009, a full-scale midrise wood-frame building was tested under a series of simulated earthquakes on the world's largest shake table in Miki City, Japan. The objective of this series of tests was to validate a performance-based seismic design approach by qualitatively and quantitatively examining the building's seismic performance in terms of response kinematics and observed damage. This paper presents the results of detailed damage inspections following each test in a series of five shake table tests, and explains their qualitative synthesis to provide design method validation. The seismic test program had two phases. Phase I was the testing of a seven-story mixed-use building with the first story consisting of a steel special moment frame (SMF) and stories 2–7 made of light-frame wood. In Phase II, the SMF was heavily braced such that it effectively became an extension of the shake table and testing was conducted on only stories 2–7, making the building a six-story light-frame multifamily residential building instead of a mixed-use building. All earthquake motions were scalings of the 1994 Northridge earthquake at the Canoga Park recording station with seismic intensities ranging from peak ground accelerations of 0.22 to 0.88 g. The building performed quite well during all earthquakes with damage only to the gypsum wall board (drywall), no sill plate splitting, no nails withdrawing or pulling through the sheathing, no edge tearing of the sheathing, no visible stud splitting around tie-down rods, and reasonable floor accelerations. On the basis of damage inspection, it was concluded that it is possible to design this type of building and keep the damage to a manageable level during major earthquakes by utilizing the new design approach.
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CE Database subject headings: Damage; Earthquakes; Shake table tests; Full-scale tests; Experimentation; Multi-story buildings; Wood structures.

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Introduction

It is estimated that 80–90% of all structures in the United States are wood structures (Malik 1995). In California, virtually all single-family dwellings are wood-frame construction and in Los Angeles County, 96% of all buildings are wood-frame construction [Consortium of Universities for Research in Earthquake Engineering (CUREE) 1998]. Wood-frame construction is not only used for single- and multifamily dwellings, but also for low-rise commercial and industrial buildings. Historically, wood structures have performed extremely well with regard to protecting their occupants' safety during seismic events because of their high strength to weight ratio and highly nonlinear ductile behavior during earthquakes that enables them to dissipate energy. However, the energy-dissipating nonlinear behavior at times results in a level of damage that society deems unacceptable, i.e., the 1994 Northridge earthquake resulted in approximately US$40 billion in damage with half of that amount to wood-frame buildings, though most of this damage was in the nonstructural gypsum wallboard (GWB) (Schierle 2002a, b).

NEESWood was a four-year, five-university project that sought to develop a new design approach for taller wood-frame buildings in earthquake-prone areas. The project started in 2005 with the benchmark test of a full-scale two-story wood-frame townhouse on two triaxial shake tables acting in unison at the University of Buffalo Network for Earthquake Engineering Simulation (NEES) site (Christovasilis et al. 2007). The goal of the benchmark testing program was focused on the various construction elements that may have influenced the seismic response of wood-frame buildings and that should be considered in a performance-based seismic design. Several seismic tests were conducted at different stages of construction to examine this issue. The GWB was repaired between tests but sill plates were not repaired. Visual damage inspection was conducted after each seismic test and observations were reported in Christovasilis et al. (2007). Different damage states were correlated with the peak interstory drift recorded near the damaged location. For a low interstory drift (0.1–0.5%), hairline cracking of the sill plate was observed. This cracking grew and led to significant splitting of the first-floor sill plates at higher levels of

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interstory drift (greater than 2%), as one might expect. At this level, cracking was also observed in vertical studs connected to hold-downs. The GWB damage primarily included diagonal cracking at the corners in which there were wall openings, crushing of GWB corners, and buckling of GWB. Significant cracking and spalling of stucco was also observed all around the building. The most significant structural damage was the splitting of sill plates of the narrow wall piers of the garage. Correlations between interstory drift and observed damage in the 2006 benchmark building test were used as the various targets in the new performance-based seismic design (PBSD) approach, namely simplified direct displacement design (DDD) (Pang et al. 2010).

With the structural damage correlated to interstory drift, damage control in PBSD was believed to be achievable by limiting interstory drifts in midrise wood-frame building design. However, it was felt that this belief might be conservative, because the correlation between damage and interstory drift used in the design process was from a two-story building and the test in Japan was conducted on a much taller building. The method mentioned previously was developed by Pang and Rosowsky (2007) and built on the earlier studies for displacement-based design; see e.g., Priestley (1998) and Filiatrault and Folz (2002). Table 1 shows seismic intensities used in the Capstone test program. The detailed structural design, including shear transfer details and (continuous anchor tie-down) system design, was conducted after the DDD shear wall selection procedure using the Seismic Analysis Package for Wood-frame Structures (SAPWood) software (SAPWood for Windows Version 1.0). Seismic Test 2 had a hazard of 7% probability of exceedance in 50 years (7%/50) in an effort to engage the steel special moment frame to a greater extent than the 10%/50 was able to achieve in pretest computer simulations.

This paper presents the results of damage inspections following each test in a series of five shake table tests and explains their qualitative synthesis to provide design method validation for the DDD procedure. Damage is described qualitatively whereas response kinematics are described quantitatively. For each seismic intensity level shown in Table 1, a conclusion was reached as to whether the performance was better than expected, as expected, or poorer than expected.

Brief Description of Test Specimen

The Capstone test building was a six-story wood-frame apartment building with one additional steel moment frame (SMF) story at the bottom and a floor plan representative of a typical multistory residential condominium in California. It was constructed at full size with 23 living units for the wood portion providing approximately 1,340 m^2 (14,400 ft^2) of residential living space. The bottom story SMF had 223 m^2 (2,400 ft^2) of commercial retail space. Added mass consisted of 64 two-ton steel plates that were used to bring the test building to the design weight, which included dead load and applicable live loads. The dead load was estimated on the basis of a force-based design of the same architectural plan according to the International Building Code [International Code Council (ICC) 2009], including finishing materials such as floor tiles, insulation, air conditioning units, wiring, and plumbing. The total weight of the wood-only six-story building was 285 metric tons (628 kips). The total weight of the seven-story wood-steel hybrid building was 361 metric tons (797 kips). The test building was standard light-frame construction with several notable exceptions: (1) the stud packs were significantly larger than typical because of the height of the building and the seismic intensity used in the design, and (2) glulams were used (as drag struts) for all wood shear walls to transfer shear throughout the building. These glulams also served as headers over openings when needed. The first story had a clear height of 2.75 m (9 ft) and the other five stories had a clear height of 2.44 m (8 ft). Detailed information about the test building can be found in the NEESWood report by Pei et al. (2010).

There was no exterior finish material installed on the building, but an equivalent amount of mass was added to simulate the weight of exterior vinyl siding. It was felt after discussion with the project advisory committee that vinyl siding would not provide additional stiffness or strength to the specimen and could be neglected. Testing with exterior stucco was conducted by Filiatrault et al. (2010) and provided a significant increase in both strength and stiffness, thereby improving the overall performance of the building at all seismic test levels. For all interior walls, 13-mm (1/2 in.)-thick GWB was installed according to the California Building Code [California Building Standards Commission (CBSC) 2007] Table 25-G with #6 drywall screws at 300-mm (16-in.) spacing on both sides (i.e., not a "float" drywall installation) except for walls in the stairwell (which only had drywall on one side) and the double mid-ply walls. The GWB has historically been considered a nonstructural element in wood-frame design. However, damage assessment after the 1994 Northridge earthquake suggested that most of the loss was because of cracking and tearing of the GWB. Nails pulling out of the oriented strand board (OSB) also contributed to failures (Schierle 2002a, b). The drywall panels were purchased in Japan and were "normal" strength Japanese drywall panels (drywall in Japan is available in three different strengths). The drywall was installed with tape and putty on all joints except the wall-to-ceiling joints and corners. Finishing as many joints as possible was desirable to provide realistic damage inspection results. The elevation views of the test building are shown in Fig. 1. To identify damage to shear walls, each shear wall was provided an identifier, which is shown in Fig. 2. Over 300 channels of instrumentation were used to record the building response kinematics during each test. Fig. 3 shows an example of a diagonally mounted string potentiometer which was positioned on many of the shear walls to record deformation diagonally across the sheathing panel as a result of shear. The absolute displacements were measured using 50 optical tracking lights combined with seven high-resolution cameras as shown in Fig. 4. Thus, absolute displacement was only known for points on the exterior of the building and, thus, interior walls

Table 1. Seismic Intensities Used in Capstone Test Program

Seismic test	Test date	Structure	Hazard level (%)	Scaling factor	PGA (g)		
					X	Y	Z
1	June 30, 2009	Hybrid	50/50 years	0.53	0.19	0.22	0.26
2			7/50 years	1.40	0.50	0.58	0.69
3	July 6, 2009	Wood only	50/50 years	0.53	0.19	0.22	0.26
4			10/50 years	1.20	0.43	0.50	0.59
5	July 14, 2009		2/50 years	1.80	0.65	0.75	0.88

Fig. 1. Test building elevations

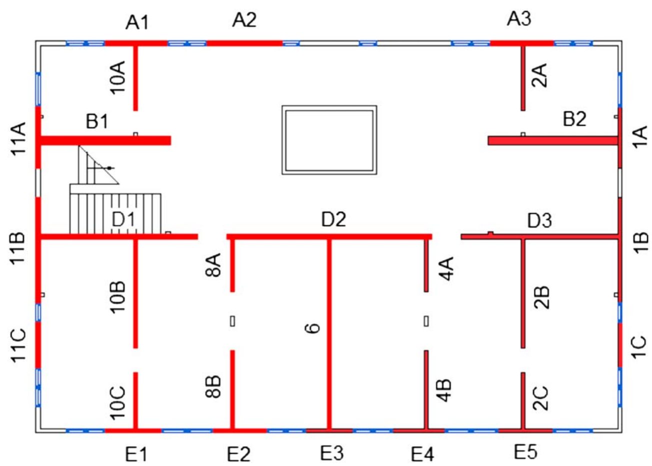

Fig. 2. Shear wall identification scheme

connected at an exterior wall line were assumed to approximately follow that displacement.

Description of Test Program

Recall that the Capstone test program consisted of two phases with five shakes in total, as presented in Table 1. The input ground motions were four scaled versions of the Canoga Park record from the 1994 Northridge earthquake, as shown in Fig. 5.

All five tests were triaxial tests that simulated all the ground motion components. There were no repairs or modifications of the structure during the entire testing program; therefore, any reported structural and or nonstructural damage includes propagation from previous tests, i.e., cumulative damage reporting over the entire study. This was not possible with the shake table occupation time provided for the project, because the building had a finished area equivalent to 1,200 standard GWB panels.

Test Results

This study focused on damage to the building and thus, only drift results are presented here. For other detailed response results, such

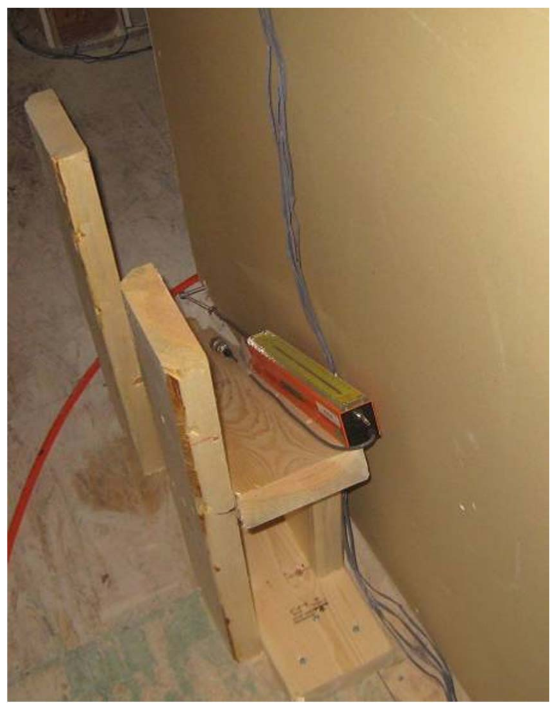

Fig. 3. Diagonal deformation measurements using stringpot

as forces and accelerations, the interested reader is referred to Pei et al. (2010). Table 2 presents the peak interstory drifts for each story, which was computed by averaging the seven optical tracking displacement measurements at each story diaphragm. Each row varies by story, and each column by seismic test for both horizontal directions. Table 3 presents these individual point interstory drifts that can be assumed to approximately represent the peak interstory drift for a single shear wall near that point. Torsional response of the building was observed to result in peak interstory drifts for individual walls (Table 3) significantly higher than the peak interstory drifts for the entire story (Table 2). For example, the peak interstory drift for the entire story for all tests and all story levels is 1.88% (Story 6, Seismic Test 5 in Table 2), whereas the peak interstory drift for an individual wall was 3.08% (Table 3) at the same location, thus indicating the presence of torsion in the building.

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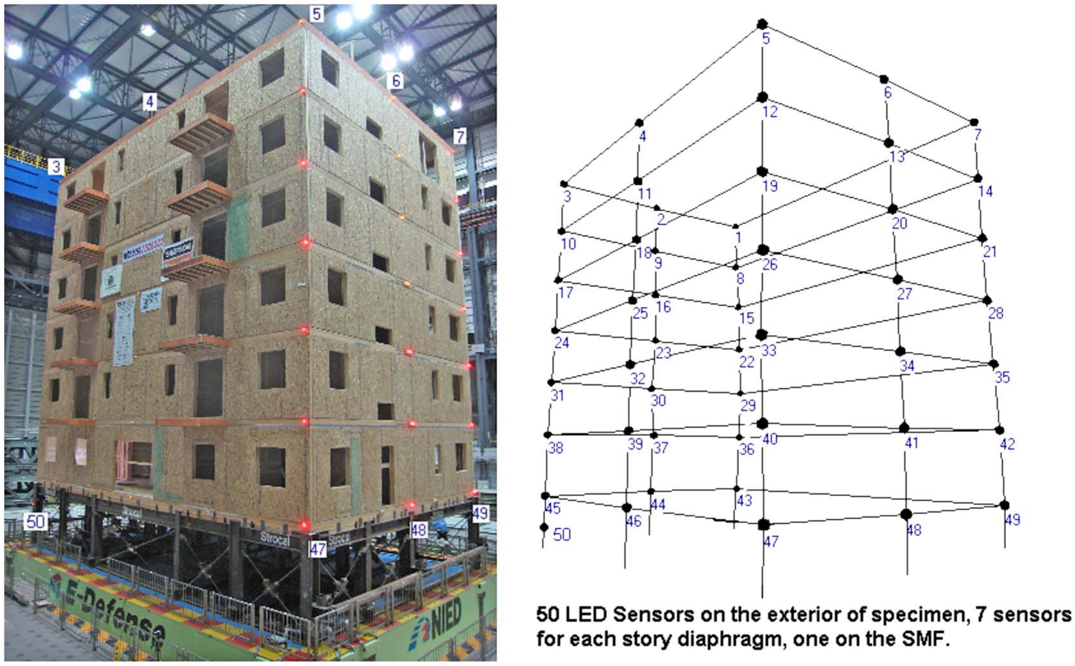

Fig. 4. Optical tracking system to record absolute displacements during testing

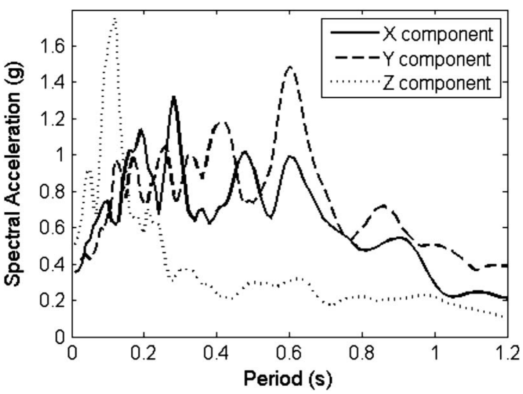

Fig. 5. Acceleration response spectra of unscaled Canoga Park record for 5% damping

Damage Inspection and General Observation

An inspection group of approximately 40 people performed damage inspection immediately following each test. Teams of two people were each assigned one quadrant of one story in the building. Different colored felt-tip markers, similar to the marking schemes used in typical concrete seismic testing, were used after each test to show the propagation of GWB cracks as a result of subsequent shakes. No edge tearout of GWB was primarily attributed to the fact the all edges of GWB had large edge distances. These large distances were possible because it was an engineered building (as opposed to a prescriptively built wall, in which generally only one stud is behind one or two edges of GWB; hence, small-edge distances for fasteners) and each GWB edge had more than one stud behind it providing large edge distances. Overall damage was limited to what has historically been described as nonstructural damage, i.e., cracks in GWB. Most of the cracking was found near window and door openings and propagated diagonally outward, similar to typical damage found in GWB following a moderate earthquake (Schierle 2002a, b). Fig. 6 shows typical cracks observed as a result of the five shakes. Sill plates did not split anywhere in the building, which is unusual for this ground motion intensity level. This is believed to be the result of the 6-mm (1/4 in.)-diameter self-tapping screws used to anchor the shear wall sill plates through the floor sheathing and into glulams, and served as shear collectors rather than the typical 16 mm (5/8 in.) diameter bolts. Fig. 7(a) shows the end of the sill plates for one wall on each story that experienced average or above average drift. The slight damage seen to the Story 3 sill plate was the

Table 2. Maximum Interstory Drift (%) for All Tests

Seismic test →	1		2		3		4		5	
Direction → Story ↓	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*
1	0.19	0.32	0.34	0.51	0.12	0.10	0.16	0.28	0.31	0.34
2	0.35	0.38	0.60	0.91	0.26	0.44	0.49	0.77	0.84	1.12
3	0.41	0.58	0.77	1.14	0.35	0.42	0.63	1.05	0.97	1.46
4	0.35	0.41	0.75	1.20	0.29	0.54	0.64	1.02	0.89	1.64
5	0.36	0.45	0.92	1.31	0.30	0.44	0.77	1.22	1.10	1.48
6	0.33	0.35	0.83	1.15	0.36	0.46	0.64	1.14	1.00	1.88
7	0.38	0.29	0.96	0.65	0.40	0.21	0.88	0.58	1.35	1.11

Note: Values are average value at centroid of floor plan.

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Table 3. Maximum Interstory Drift (%) at Any Individual Wall Line for Each Story During Seismic Tests 1–5

Seismic test →	1		2		3		4		5	
Direction → Story ↓	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*
1	0.43	0.45	0.70	0.69	0.56	0.37	0.58	0.40	0.97	0.52
2	1.70	1.37	1.48	1.11	0.91	0.64	1.09	1.02	1.40	1.50
3	1.88	2.04	1.87	1.50	1.25	0.82	1.20	1.41	1.90	2.10
4	0.66	0.78	1.28	1.63	0.69	0.83	1.21	1.52	1.57	2.08
5	0.73	0.77	1.40	1.64	0.71	0.76	1.69	1.69	1.62	2.35
6	0.94	0.63	1.43	1.98	0.82	0.80	1.29	1.90	1.82	3.08
7	0.82	0.65	1.40	0.90	0.78	0.63	1.18	0.83	1.78	2.18

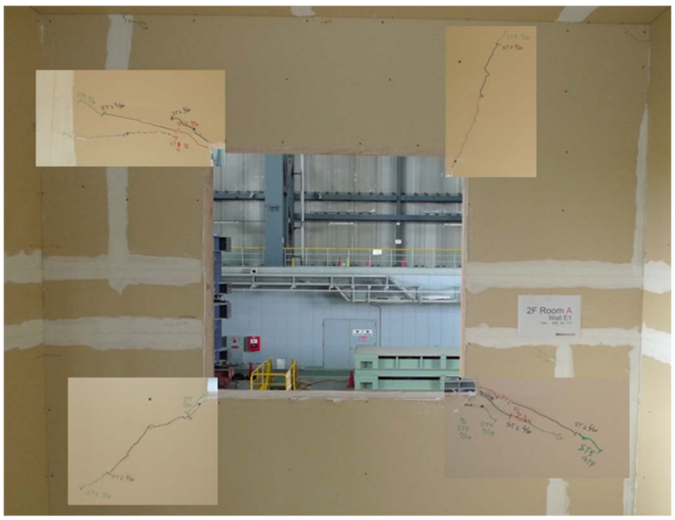

Fig. 6. Example of cracking propagation from the corners of a window opening after five simulated earthquake tests

result of toe-nailing during construction and was marked as pretest damage. Fig. 7(b) shows the top of the wood screws after removing OSB on one side of the shear wall following the final seismic test. Other than the damage to GWB, no additional damage was observed, i.e., no nail withdrawal, no nail pull-through in the sheathing, and no visible stud splitting around tie-down rods. In a typical shear wall, testing without GWB failures like these are seen. But the full-scale, six-story Capstone building did not see any of these failures. This is because in shear wall testing, shear deformation is dominant, whereas in a full-scale and especially a six-story building, other deformations (such as bending and axial) are also present. Also, tie-down rods that extended the whole height of the building resisted most of the overturning forces.

Correlation of Observed Damage to Intensity and Drift

Intensity Level Correlation

Although damage was limited to dry wall cracks only, correlation of damage to the seismic intensity test level can be identified during all five tests. In the test at Seismic Intensity Level 1, only very minor cracking was observed around the openings, as shown in Fig. 8. In many of these cases, the cracks were small enough to have been covered/filled with paint. As was anticipated, Seismic Intensity Level 2+, which slightly exceeded the design-basis earthquake (DBE), caused more damage. The cracking was definitely detectable at many locations, but was still limited to the GWB. At this seismic intensity level, it became apparent that the stories with the larger interstory drift levels had longer, wider, and at times more numerous GWB cracks. Fig. 9 shows one of the more noticeable crack patterns that occurred above several doorways after Seismic Test 2. The diagonal pen marking indicates a section of the GWB that may have slightly pulled away, or in which the paper was loosened. Seismic Test 3 was a smaller shake, but with the steel SMF locked down, i.e., testing only the six-story wood building. During this small shake, many locations had no propagation of crack damage. For example, Fig. 10 shows a vertical crack above the doorway that occurred during Seismic Test 2 but did not widen or spall during Seismic Test 3. Because of the way the tests were performed, it is difficult to tell if the 50%/50-year earthquake used in Seismic Test 3 would have caused any appreciable damage even to the GWB. It is likely there would have been only hairline cracking such as that shown in Fig. 11.

Seismic Test 4 (ST4) was a DBE level earthquake and resulted in more GWB damage. The start of some GWB spalling can be seen in the photograph. In general, Seismic Test 4 propagated existing cracks and caused some smaller cracks to branch off from the main cracks. Fig. 12 shows some new cracking near one of the corner GWB screws, indicating significant racking in the panel after ST4. The propagation discussed previously was observed routinely, as shown in Fig. 13. The "ST4" marking is for both the upward and downwardly propagating cracks. Repairs (structural or to the GWB) between tests were not made; thus, conclusions and correlations discussed in this paper only seek to better understand the trends.

The final test [Seismic Test 5 (ST5)] in the Capstone test program was a maximum credible earthquake (MCE) level corresponding to a 2%/50-year event, statistically giving a return period of approximately 2,500 years. In ST5, crack propagation from the top corner of an opening connected to another crack propagation from a GWB screw. This was observed for walls that experienced the largest interstory drifts, i.e., near 3%, but not all. An example of this type of damage is shown in Fig. 14. In some places a crushed corner was observed, as shown in Fig. 15. A more interesting occurrence was the substantial cracks that were observed to be new and not propagating from existing cracks from the previous tests. This only occurred as a result of ST5 and an example is shown in Fig. 16.

Drift Correlation

An effort is made here to qualitatively correlate damage with drift. Initially, Story 2 had a smaller interstory drift during each of the tests than Story 6 (Tables 2 and 3), yet the amount of damage was about the same at both stories. Figs. 17 and 18 show very typical damage for Story 2 after ST5. Fig. 17 shows some light spalling, but in general the cracks did not extend all the way to the panel seam or nearby wall corners. Fig. 18 shows two cracks failing to

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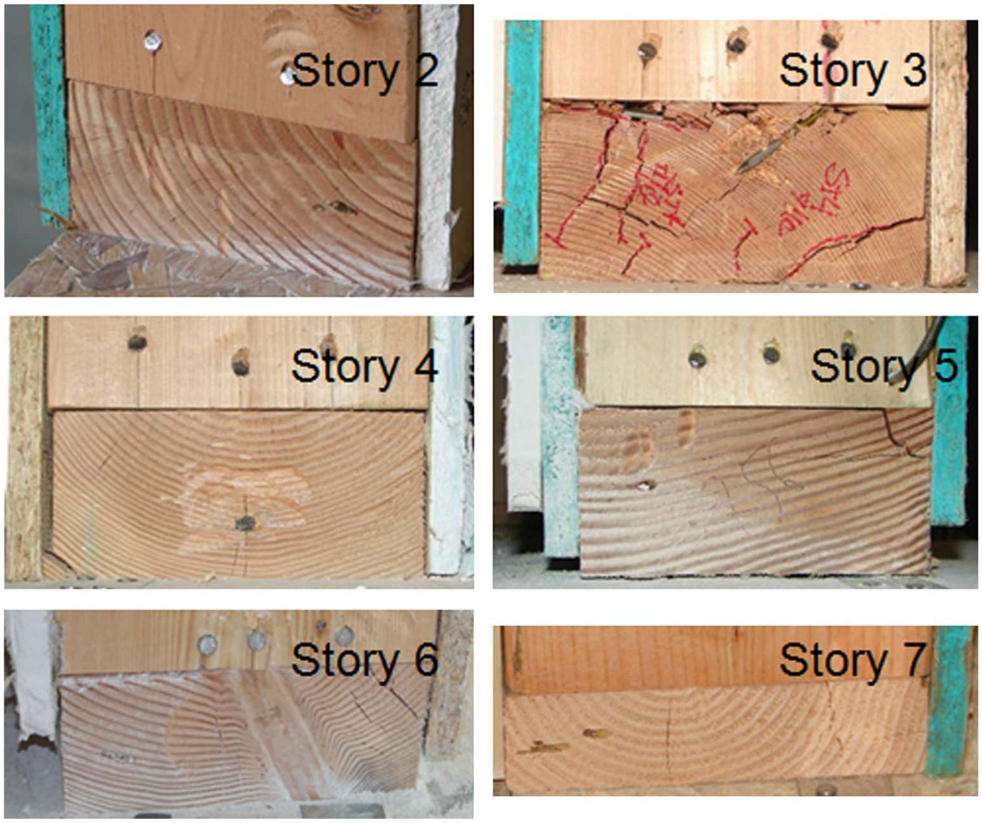

(a)

(b)

Fig. 7. (a) Sill plates remaining intact (i.e., no splitting) following all five shakes; (b) close-up of sill plate after removing wall sheathing

Fig. 8. Minor crack on Wall E4, Story 2 (over window opening) after Level 1 shaking

Fig. 9. Crack on Wall D1, Story 4 (over door opening) after Seismic Test 2

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Fig. 10. No new crack on Wall 2A, Story 7 (over door opening) after Seismic Test 3; the crack shown was developed during Seismic Test 2

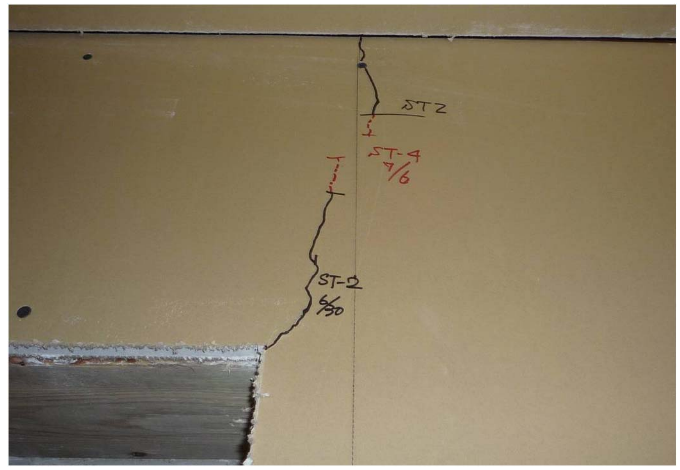

Fig. 13. Crack on Wall 2A, Story 5 (over door opening) after Seismic Test 4

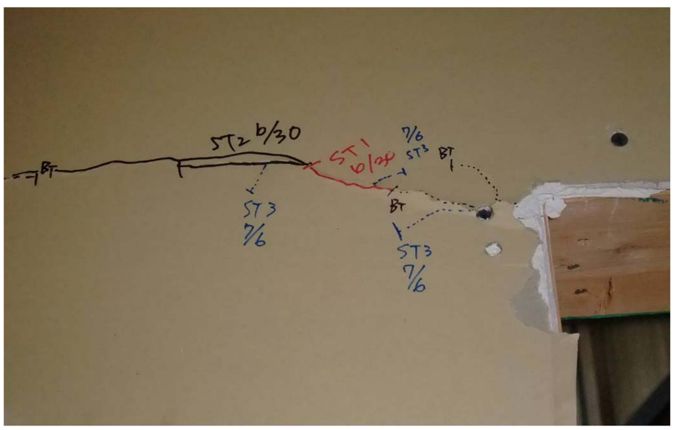

Fig. 11. Crack on Wall A3, Story 2 (over window opening)

Fig. 14. Crack on Wall A3, Story 4 (over window opening) after Seismic Test 5

Fig. 12. New crack on Wall D1, Story 3 (corner crushing) after Seismic Test 4

Fig. 15. Crack on Wall D2, Story 6 (corner crashing) after Seismic Test 5

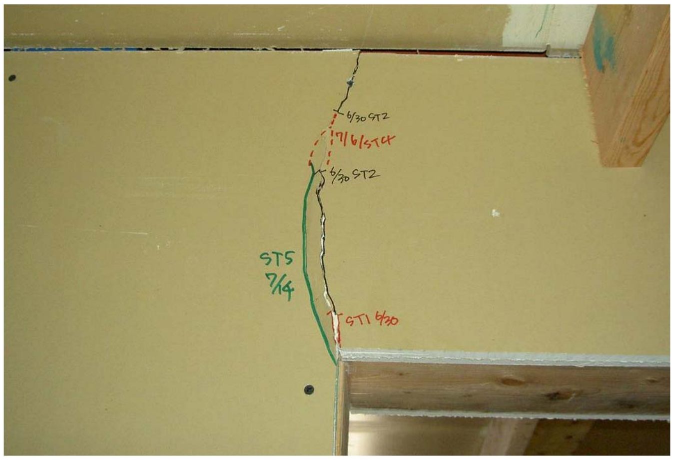

Fig. 16. New crack on Wall 2A, Story 7 (over door opening) after Seismic Test 5

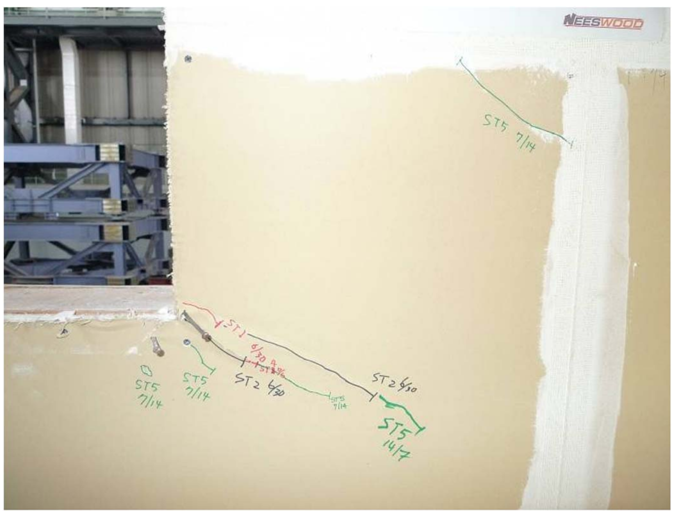

Fig. 17. Wall E1; cracks did not extend all the way to the panel edge after Seismic Test 5

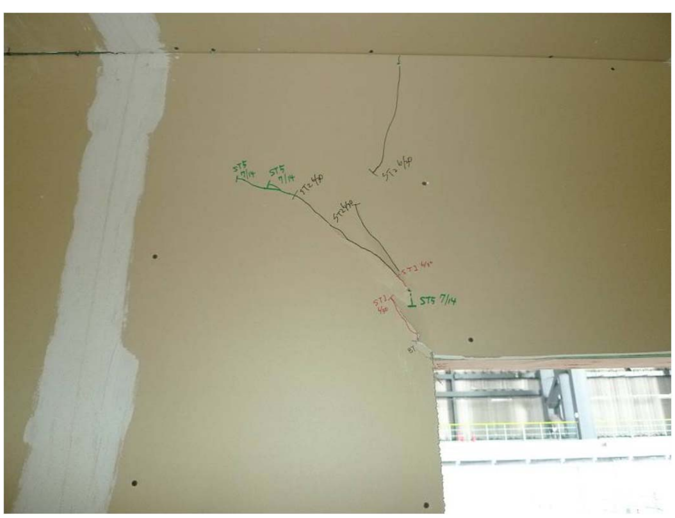

Fig. 18. Story 2, Wall E2; cracks failing to connect together after Seismic Test 5

Fig. 19. Story 6, Wall E1; cracks develop all the way through panel after Seismic Test 5

Fig. 20. Story 6, Wall E2; cracks develop all the way through panel for all corners after Seismic Test 5

connect following ST5 at Story 2. For some of the damaged GWB in this study, cracks tended to propagate until they connected to another crack, particularly when there was significant interstory drift caused by a combination of shear and bending. This was typical on lower stories of the building because the overall interstory drifts were lower. Comparatively, consider Story 6 following the series of five shakes. The cracks tended to propagate all the way through the panel as shown in Fig. 19. Fig. 20 shows this from a full-panel vantage point. Further, in Story 6, crushing of the panel corners was more routinely observed, as shown in Fig. 21. Most of the damage in lower stories was attributed to shear deformation,

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Fig. 21. Crushing of corners also seen at Story 6, Wall E5 after seismic test

whereas in upper stories the damage was attributed to a combination of shear and bending.

Conclusions

This paper presents the results of damage inspections following a series of simulated earthquake tests of a full-scale, six-story, wood-frame building and qualitatively correlates the interstory drift with damage. The building performed quite well during all earthquakes, including the MCE level earthquake corresponding to a 2%/50-year event with damage only to the nonstructural elements, i.e., GWB (drywall). There was no damage to any wood components of the building, including no sill plate splitting, no nail withdrawal or nails pulling through the sheathing, no edge tearing of the sheathing, and no crushing of the wood under tie-down rod-bearing plates.

There was good correlation of the overall interstory drift with damage. However, much of the drift in the upper building stories was attributed to global bending of the building and not solely shear deformation at the individual shear wall level. This results in a blending (some portion shear and some portion bending) of drifts between 1 and 3% as the story level increases in the building. However, there appears to be a general trend and thus correlating damage with overall interstory drift is an attainable goal, at least in the less-than-3.5% interstory drift range, because a general correlation was observed. Therefore, controlling drifts in engineering design can work for damage control in midrise wood-frame buildings, but should not be quantitatively extracted from shear wall tests or low-rise shake table tests. The effect of preexisting damage attributed to previous tests was neglected in the qualitative correlations discussed in this paper.

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