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SEISMIC TESTING OF A FULL-SCALE WOOD STRUCTURE ON TWO SHAKE TABLES

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ABSTRACT

An impediment to the development of performance-based seismic design for woodframe buildings is the lack of understanding of the factors that affect the seismic behavior of woodframe structural systems. Few numerical seismic analysis models capable of considering all the factors influencing the seismic behavior for three-dimensional woodframe structures currently exist. Furthermore, only limited experimental data have been generated at the system level and never on a structure with realistic dimensions. This paper discusses the results of a shake table testing program on a full-scale woodframe structure conducted within the NSF-funded NEESWood Project.

The test structure considered was a full-scale two-story townhouse, having approximately 170 m^2 of living space with an attached two-car garage. It was assumed to be located on a level lot with a slab-on-grade and spread foundations and to have been built as a “production house” in either the 1980’s or 1990’s, located in either Northern or Southern California. The design was based on engineered construction. The size and weight of the test structure required for the first time the simultaneous use of the two three-dimensional shake tables at the Structural Engineering and Earthquake Simulation Laboratory (SEESL) at the University at Buffalo.

The testing program focused on the various construction elements that have significant influence on the seismic response of woodframe buildings. Five different testing phases were conducted to investigate the influence of the following elements on the seismic behavior: Phase 1 - Engineered wood structural (shear) walls alone; Phase 2 - Wood structural walls incorporating viscous fluid dampers; Phase 3 - Installation of gypsum wallboards to engineered wood structural walls; Phase 4 - Installation of gypsum wallboards to interior partition walls and ceilings; and Phase 5 - Installation of stucco as exterior wall finish.

KEYWORDS: Buildings, Wood structures, Seismic tests, Shake table tests

1. INTRODUCTION

While light-frame wood buildings have historically performed well with regard to life-safety requirements in regions of moderate-to-high seismicity, these types of low-rise structures have sustained significant structural and nonstructural damage in recent earthquakes. For example, the property loss to light-frame wood construction during the 1994 Northridge earthquake in California was estimated to be US $20 billion (Kircher et al. 1997) and greatly exceeded the losses associated with any other type of construction. One reason for this substantial level of damage is believed to be that current building code seismic design requirements for engineered wood construction around the world are not based on a performance-based design philosophy. Rather, wood elements are designed independently of each other without enough consideration of the influence that their stiffness and strength have on the other components of the structural system. Furthermore, load paths and dynamic response in light-frame wood construction arising during earthquake shaking are not well understood. These factors, rather than economic considerations, have limited the use of wood to low-rise construction and, thereby, have reduced the economical competitiveness of the wood industry in the U.S. and abroad relative to the steel and concrete industries.

A major impediment to the development of performance-based seismic design for light-frame wood buildings is the lack of a complete understanding of the factors that affect the seismic behavior of light-frame wood structural systems. Few numerical seismic analysis models capable of considering the multiple factors influencing the seismic behavior of three-dimensional light-frame wood structures currently exist (e.g. Tarabia and Itani 1997, Collins et al. 2005). Furthermore, only limited experimental seismic response data have been generated at the building system level (e.g. Filiatrault et al. 2002, Mosalam et al. 2002, White and Ventura 2007) and never on a light-frame wood building of realistic dimensions.

This paper discusses the results of a shake table testing program on a full-scale light-frame wood structure conducted within the NSF/NEES-funded NEESWood Project (http://www.engr.colostate.edu/NEESWood/). The main objective of this experimental study was to contribute toward a better understanding of the seismic behavior of light-frame wood structures typically built in North America. A significant task of this investigation focused on the effect of interior (gypsum wallboard) and exterior (stucco) finishes, applied to the surfaces of structural wood shear walls and to interior partition walls and ceilings, on the seismic response of the building.

2. DESCRIPTION OF TEST BUILDING

The full-scale test building considered in this study is one of the four California-style index buildings designed within the recently completed CUREE-Caltech Woodframe Project (Reitherman et al. 2003). It represents one unit of a two-story townhouse containing three units, having approximately 170 m^2 (1800 ft^2) of living space with an attached two-car garage. Figure 1 shows plan views of the first and second floor of the test building. The major structural components of the test building are identified on the figure and are described in detail by Christovasilis et al. (2007). The footprint of the test building is 7 m x 18 m. The height of the test building from the first floor slab to the roof eaves is 5.3 m and its total weight is 320 kN (32 metric tons).

All walls of the structure were built with 38 mm x 89 mm (2x4) Hem-Fir studs except for the North, South and West walls of the garage where 38 mm x 140 mm (2x6) studs were used. The exterior walls were covered on the outside with 22-mm thick stucco over 11-mm thick OSB sheathed shear walls and 12-mm thick gypsum wallboard on the inside. Eight penny common nails (3.3 mm in diameter x 63.5 mm long) with spacing of between 75 mm to 150 mm along panel edges and 150 mm along interior studs were used to connect the OSB sheathing to the wood framing. Construction details regarding the two-story townhouse building are given by Reitherman et al. (2003) and Christovasilis et al. (2007). The 12-mm thick gypsum wallboard panels were installed on all interior walls and ceiling surfaces and on both sides of interior partitions. The panels were oriented horizontally on the walls and fastened with #6-32-mm long drywall screws spaced at 400 mm along the vertical studs only (no fastening along the top and bottom plates). The ceiling panels were fastened with the same screws spaced at 300 mm on center. The stucco was attached to the wood framing by a galvanized 16-gage steel wire lath, fastened to the OSB sheathing and vertical studs by 38-mm long staples spaced at 150 mm on center. The construction of the building was conducted by professional contractors to replicate field conditions.

3. SHAKE TABLE TEST PROGRAM

3.1 Experimental Setup

The twin re-locatable, 50-ton, tri-axial shake tables of the Structural Engineering and Earthquake Simulation Laboratory (SEESL) at the University at Buffalo (UB) were utilized for the experiment. The two tables acting in unison were required to accommodate the size and weight of the full-scale test building. The 7 m x 7 m extension steel frames available on both of the UB-SEESL shake tables were connected together by a steel link structure to support the entire woodframe structure across the two shake tables with minimal vertical deflection. Threaded A-307 steel rods bolted to the existing extension frames were used as anchor bolts for the sill plates. A 60-mm thick layer of grout was installed on top of the steel base beneath the pressure treated sill plates to simulate the friction of the sill plate against a concrete foundation. Seismic holdowns were installed at the end of various first level narrow wall piers, as shown in Figure 1.

3.2 Testing Protocol

Multiple seismic tests were conducted for various configurations of the test building. Table 1 presents a summary of the five seismic test phases included in the test program and the corresponding configurations of the test building. Low amplitude white noise tests were also conducted between the seismic tests of each phase to determine the changes in the dynamic characteristics (natural periods, mode shapes and damping) of the test building as it experienced increasing levels of damage during each test phase. The building was repaired after each test phase to return the lateral load-resisting system to its original characteristics before the start of each subsequent test phase. These repairs included replacing some of the OSB panels, gypsum wallboards and wood studs. Note that all test phases were performed for a constant mass of the test building by incorporating ballast weights at the floor level for the test phases in which some of the wall finish materials were omitted. In this paper, only Phases 1, 3, 4 and 5 are discussed. Information on Test Phase 2, incorporating fluid dampers in selected locations of the test building, is available in Shinde et al. (2007). Test Phases 1, 3, 4 and 5 were designed to evaluate the effect of interior and exterior wall finishes on the seismic response of the test building. In Phase 1, the test building incorporated only the wood structural members without any wall finishes. In Phase 3, 12-mm thick gypsum wallboards were applied to the interior surfaces of the structural perimeter walls and to both sides of the two interior structural shear walls, located at the fist level of the test building in the North-South direction (see Fig. 1). In Phase 4, gypsum wallboards were also applied to all interior partition walls and ceilings. Finally, in Phase 5, 3-coat, 22-mm thick, stucco was applied to the exterior walls. Figure 2 shows photographs of the Phase 1 and Phase 5 test building ready for testing on the shake tables.

Table 1. Test phases and building configurations.

Test Phase	Test Building Configuration
1	Wood structural elements only
2	Test Phase 1 structure with passive fluid dampers incorporated into selected wood sheathed walls
3	Test Phase 1 structure with 12-mm thick gypsum wallboard installed with #6-32-mm long screws @ 400 mm on center on structural wood sheathed walls
4	Test Phase 3 structure with 12-mm thick gypsum wallboard installed with #6-32-mm long screws on all walls (400 mm on center) and ceilings (300 on center)
5	Test Phase 4 structure with 22-mm thick stucco installed with 16 gage steel wire mesh and 38-mm long leg staples @ 150 mm on center on all exterior walls

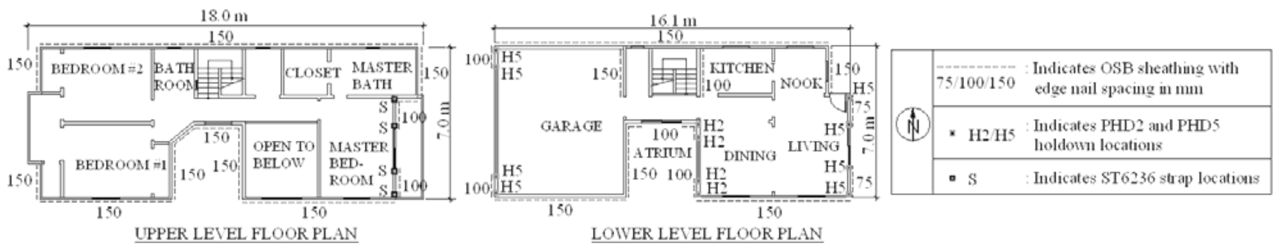

Figure 1. Plan views of test building.

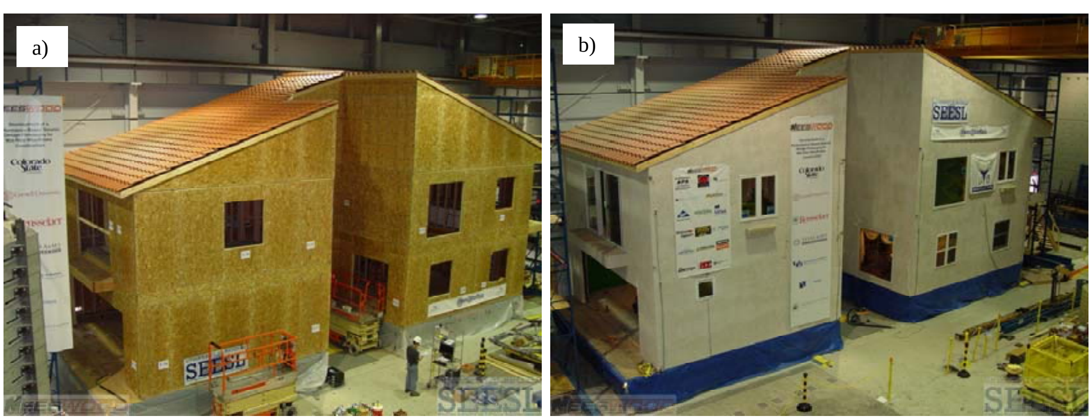

Figure 2. Test building on shake tables: (a) without wall finishes –Phase 1, and (b) with wall finishes –Phase 5.

3.4 Input Ground Motions

Two different types of tri-axial historical ground motions were used for the seismic tests: ordinary ground motions and near-field ground motions. The ordinary ground motions represented a Design Earthquake (DE) having a probability of exceedance of 10% in 50 years (10%/50 years), or equivalently, a return period of 475 years. The 1994 Northridge Earthquake ground motions recorded at Canoga Park, with an amplitude scaling factor of 1.20, were selected as the DE (Krawinkler et al. 2000). The near-field ground motions represented a Maximum Credible Earthquake (MCE) having a probability of exceedance of 2% in 50 years (2%/50 years), or a return period of 2475 years. The unscaled 1994 Northridge Earthquake ground motions recorded at Rinaldi were selected as the MCE (Krawinkler et al. 2000). Figure 3 presents the absolute acceleration response spectra at 5% damping for these two (unscaled) tri-axial seismic records.

In addition to the DE and MCE hazard levels, the Canoga Park ground motions were scaled to simulate hazard levels of 99.9%/50 years, 50%/50 years and 20%/50 years, associated with scaling factors of 0.12, 0.53, and 0.86, respectively. Five seismic test levels were considered during each phase of seismic testing. For each seismic test level, two seismic tests were conducted: one tri-axial (3D) test followed by one horizontal bi-axial (2D) test. Note that during Test Phases 1, 3 and 4, only Seismic Test Levels 1 and 2 were conducted in order to limit the damage of the test building to a repairable level. The structure was not repaired between test levels.

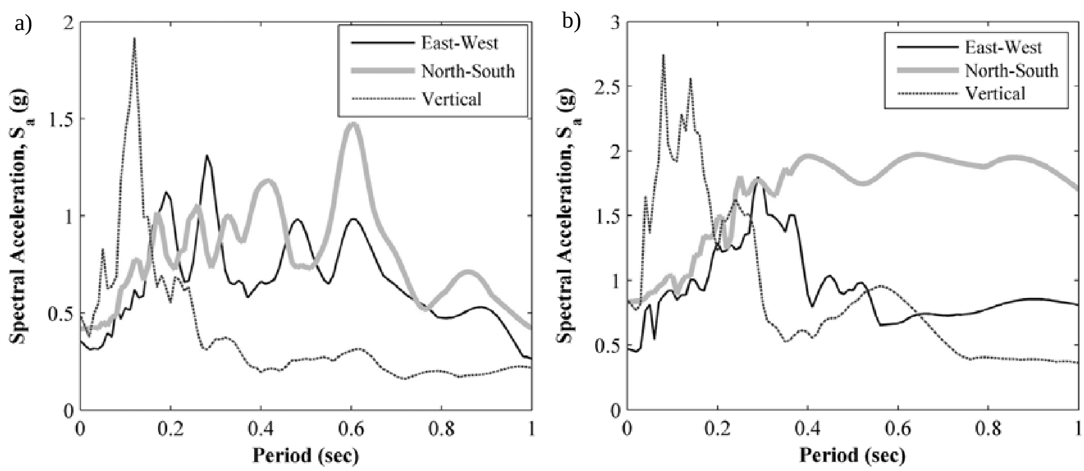

Figure 3. Absolute acceleration response spectra at 5% damping of earthquake ground motions used in seismic tests (a) Canoga Park Record, and (b) Rinaldi Record.

4. RESULTS OF AMBIENT VIBRATION TESTS

Before and after each seismic test, the dynamic properties of the test building were estimated by simulated ambient vibration tests. For this purpose, the test building was excited by a low-level white-noise base acceleration input having a flat (i.e. uniform) spectrum with 0.5–50 Hz frequency band and a Root Mean Square (RMS) amplitude of less than 0.10 g. The natural periods, mode shapes and associated modal damping ratios were determined through Transfer Functions (TFs) of the story acceleration response of the structure and the base motion. Thirty two horizontal accelerometers, located at the floor and roof levels of the test building, as well as four accelerometers located on the twin shake tables, were used to generate the TFs for each white noise test. The equivalent viscous damping ratios of the test building were determined using the well known half-power bandwidth method (see e.g. Clough and Penzien 1993) applied to the peaks of the TFs.

Figure 4 shows the initial fundamental periods and mode shapes in each principal direction of the test building before the beginning of Test Phases 1, 3, 4 and 5, respectively. Not surprisingly, the fundamental periods of the test building are significantly longer in its transverse (North-South) direction than in its longitudinal (East-West) direction. The introduction of gypsum wallboard finishes on the structural walls in Test Phase 3 causes a reduction in the fundamental period of 9% and 5% along the transverse and longitudinal directions of the test building, respectively. These results indicate that introducing gypsum wallboard finishes on the interior surfaces of the structural walls increased the lateral stiffness of the test building. On the other hand, the introduction of similar gypsum wallboard finishes to all the interior partition walls and ceilings in Test Phase 4 had no effect on the fundamental periods and, thereby, the lateral stiffness of the test building (at least at low levels of shaking). This lack of positive effect can be attributed to the lack of structural connections between the interior partition walls and the floor and roof diaphragms of the test building.

The introduction of stucco on the exterior walls of the test building in Phase 5 causes a reduction in the fundamental period of 3% and 9% along the transverse and longitudinal direction of the test building compared to the Phase 4 configuration. In terms of equivalent lateral stiffness, Phase 5 exhibits an increase in lateral stiffness of 29% and 32% along the transverse and longitudinal directions, respectively, compared to the original Phase 1 building. For both directions, the deformations are concentrated in the first level of the test building, indicating the potential for a weak first story collapse mechanism. The fundamental mode shapes in the longitudinal direction are also affected by torsional response and by the in-plane shear deformations of the floor diaphragm in the stair core area between the two main units of the townhouse, particularly for the Phases 1 and 3. For Phases 4 and 5, the shear deformations of the diaphragm are reduced because of the in-plane stiffness provided by the gypsum ceilings.

Figure 5a illustrates the deterioration of the equivalent lateral stiffness in the transverse (North-South) direction of the test building through the various seismic tests conducted, assuming a single-degree-of-freedom response of the test building. Since the initial fundamental period is known, as well as the fundamental period measured after each seismic test, the normalized equivalent lateral stiffness, after each seismic test, can be calculated as a percentage of the initial lateral stiffness. The deterioration of the lateral stiffness is more pronounced for the Test Phase 1 configuration. The stiffness at the end of this phase dropped to less than 60% of the initial stiffness after Test Level 2. The lateral stiffness for the structures of Test Phases 3 and 4 was above 80% of their initial stiffness after Level 2 test; the corresponding value for the Test Phase 5 structure was above 90%. Even after the tri-axial DE Seismic Level 4 test, the lateral stiffness of the Test Phase 5 structure remained above 75% of its initial lateral stiffness. The deterioration was smaller when wall finishes were applied for the same level of simulated ground shaking. Note that the increase of the stiffness that is observed after the final tri-axial test of Seismic Level 5 of Test Phase 5 was due to the repair of damaged anchor bolts in the two walls on the West (garage wall) and East side of the first floor of the benchmark structure, prior to the execution of the Level 5 tri-axial test, which resulted in a stiffer structure. Figure 5b shows the variations of the first modal equivalent viscous damping ratio measured in the North-South direction of the test building after each seismic test conducted. The first modal damping ratios range from 10 to 20% of critical, with a mean value of around 15% of critical for all test phases.

5. RESULTS OF SEISMIC TESTS

5.1 Global Hysteretic Responses

Figure 6a shows the global hysteretic responses (base shear force vs relative horizontal displacement at the center of the roof eave level) of the test building during Test Phases 1, 3, 4 and 5, respectively and under Seismic Test Level 2. The base shear was computed by summing the inertia forces at each level of the test building based on horizontal acceleration recordings. The maximum base shear and displacement achieved in each direction are indicated by circles on each graph. As expected, the lateral displacements in the transverse (North-South) direction are significantly larger than those in the longitudinal (East-West) direction. In Test Phase 1, the wood-only building experienced a peak roof displacement of 63 mm (1.3% building drift) in its transverse direction under the Seismic Test Level 2 representing excitation intensity of 44% of that expected for the Level 4 Design Earthquake (DE). The introduction of gypsum wallboard finishes on the structural walls in Test Phase 3 resulted in a significant reduction in transverse roof displacements (approximately 44% reduction compared to the wood-only building of Phase 1). The overall hysteretic response of the building in Test Phase 3 is also stiffer than that of Test Phase 1, indicating the important effects that the gypsum wallboard had in stiffening the structural walls. The introduction of gypsum wallboard on the interior partition walls and ceilings in Test Phase 4 resulted in a further reduction of 29% in roof displacements in the transverse direction (35 mm in Phase 3 vs 24 mm in Phase 4). Finally, the introduction of stucco on the exterior walls reduced the roof displacements even further to 18 mm; similar results are observed in the longitudinal direction. Note in Fig. 6a that only moderate pinching is observed for the transverse (North-South) direction in the wood-only Test Phase 1 building, while almost linear elastic responses are observed for Test Phases 3, 4 and 5. This result indicates that the wall finishes not only reduced the displacement response of the test building but changed also its overall hysteretic characteristics. Figure 6b shows the global hysteretic responses obtained with the complete (Phase 5) building under Test Levels 4 (DE) and 5 (MCE), respectively. In the transverse (North-South) direction, the maximum roof displacement reached 41 mm (0.8% drift) under the DE level and 101 mm (1.9% drift) under the MCE level. Note that the wood-only building of Phase 1 exhibited, under Test Level 2, a peak roof displacement larger than the Phase 5 building under the DE Test Level 4.

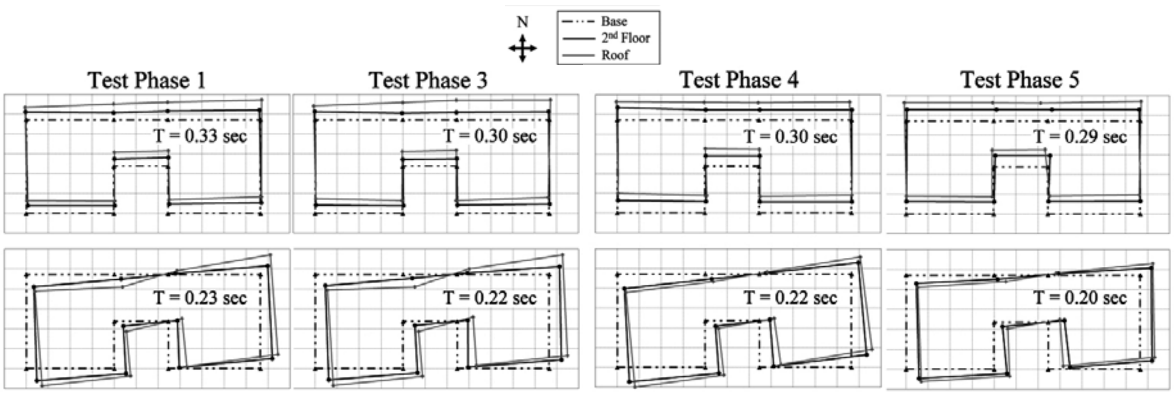

Figure 4. Initial natural periods and mode shapes of test building

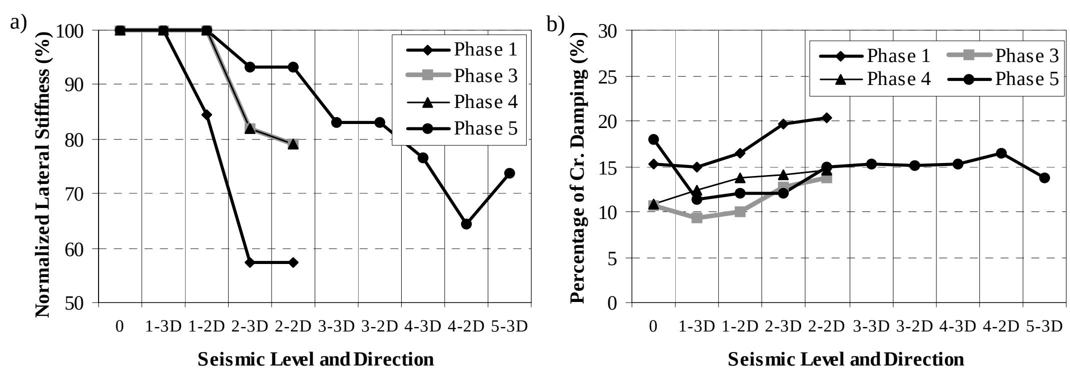

Figure 5. (a) Variations of normalized lateral stiffness in North-South direction of test building, and (b) Variations of first modal damping ratios in North-South direction of test building.

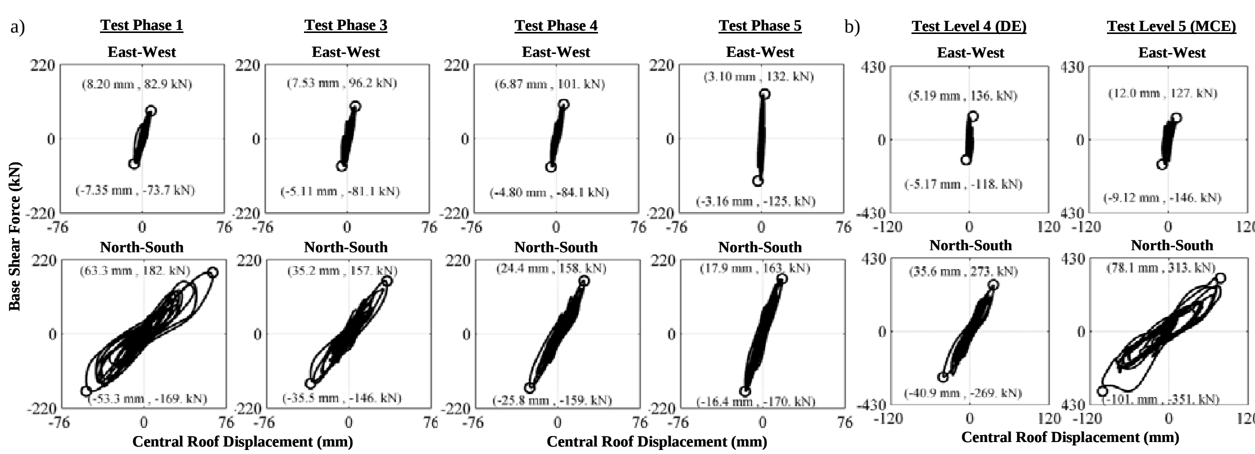

Figure 6. Global hysteretic responses of test building, (a) Test Level 2, and (b) Test Phase 5

5.2 Response of Garage Wall Line

The seismic response of the test building in its transverse (North-South) direction was significantly influenced by the response of the garage wall line at the first level. The narrow wall piers (aspect ratio of 2.5:1) on each side of the garage opening compounded by the significant torsional response of the building under high intensity shaking, caused this garage wall line to experience the largest inter-story drifts.

Figure 7a shows the inter-story drift time-histories measured along the garage wall line during Test Phases 1, 3, 4, and 5, respectively and under Seismic Test Level 2. The garage wall line of the wood-only building of Phase 1 experiences a peak relative displacement of 42 mm (1.5% inter-story drift) which corresponds to 65% of the total building drift developed in the transverse direction during this test (see Fig. 6a). This result indicates that most of the transverse lateral displacements of the test building in the garage wall line occurred at the first level, which suggests a possible soft-story collapse mechanism under higher amplitude base excitations. Note that this conclusion is valid for the garage wall line only under high level of excitations. For some other wall lines in the building, the second story inter-story drifts could be greater than the first story in some cases under lower amplitude excitation. Again, the introduction of gypsum wallboard finishes on the structural walls in Test Phase 3 caused a significant reduction in the peak drift experienced by the garage wall line (42% reduction compared to the wood-only building of Phase 1). The response of the Test Phase 4 building, however, is almost identical to that of Phase 3. This can be explained by the fact that very little interior partition wall lines were incorporated in the first level of the test building (see Fig. 1). The incorporation of exterior stucco finish also caused a significant reduction in the peak drift experienced by the garage wall line (66% reduction compared to the wood-only building of Phase 1 and 42% reduction compared to the Phase 3 building).

Figure 7b shows the inter-story drift time-histories measured along the garage wall line of the completed Test Phase 5 building under Seismic Test Levels 4 (DE) and 5 (MCE), respectively. The Test Phase 5 building experienced peak relative displacements at the garage wall line of 33 mm (1.2% inter-story drift) and 86 mm (3.1% inter-story drift) under the DE and MCE levels, respectively. Note again that the wood-only Phase 1 building experienced higher drifts at the garage wall line under Test Level 2 (44% DE) than the complete Test Phase 5 building under Test Level 4 (100% DE). This result again underscores the significant contribution of the wall finishes in improving the seismic response of the test building.

6. CONCLUSIONS

The shake table testing of a full-scale, light-frame wood building conducted in this study has provided an opportunity to study various parameters that influence the seismic response of light-frame wood buildings. This paper has concentrated on the effect of interior and exterior wall finishes during earthquake excitation. Based on the experimental results obtained, it can be concluded that the installation of gypsum wallboard to the interior surfaces of structural wood sheathed walls improved substantially the seismic response of the test building. The application of exterior stucco further improved the seismic response of the test building, particularly in its longitudinal direction, where the shear response of the wall piers dominated.

These shake table test results provide the evidence of the significant influence that wall finish materials have on the behavior of lateral load-resisting systems in light-frame wood construction. The development of a performance-based seismic design methodology that takes into account the effect of wall finish materials is urgently needed. Several issues need to be addressed before these materials can be effectively considered in design. For example, the method of attachment of stucco to the wood framing should be examined in order to evaluate current practices and possibly develop improved attachment methods that could mobilize the lateral stiffness and strength of stucco for the sequence of earthquakes that a light-frame wood building could experience during its life span.

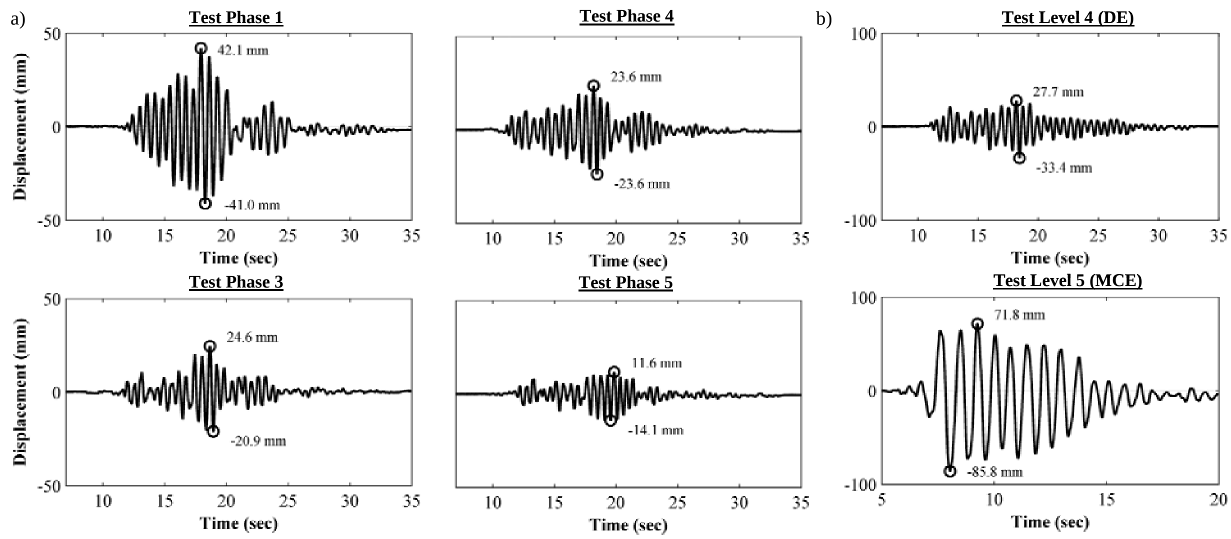

Figure 7. Response of garage wall line, (a) Test Level 2, and (b) Test Phase 5

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