Seismic Testing

Seven-Story Mixed-Use Steel and Wood Light-Frame Structure

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In June and July of 2009, a unique four-year test program was wrapping up in Japan. The NEESWood Capstone test program, largely a collaborative effort between project leader Colorado State University and industry partner Simpson Strong-Tie, subjected a seven-story building to severe ground shaking at E-Defense, the world's largest shake table facility in Miki, Japan. As the largest building ever tested on a shake table, the structure consisted of a ground level retail area framed with structural steel and six stories of wood light-frame construction featuring 23 residential units. The steel special moment frame (SMF) on the first floor utilized a new type of proprietary beam-column connection, and the entire structure was designed using a new direct displacement design methodology woven into a performance-based seismic design framework. The results show conclusively that, if properly designed and constructed, wood and mixed steel/wood can be great performers in mid-rise structures, even in areas of high seismicity.

Figure 1: Completed Structure on the Shake Table.

E-Defense

Following the devastating 1995 Kobe Earthquake, the Japanese government built the largest seismic testing facility in the world. Known as E(Earth)-Defense, the facility houses an enormous shake table capable of moving in three directions simultaneously. With surface dimensions of approximately 50 x 65 feet, the table can support test buildings weighing up to 2.5 million pounds and has a range of motion in excess of 6 feet in all directions. The Capstone building used up most of this space, with a footprint measuring 40 x 60 feet. The seven-story building was constructed during a 16-week period between February and June of 2009. Normally, test structures are constructed outside of the facility and then moved inside through mammoth sliding doors via a multi-wheeled crawler unit. The Capstone building, however, was too large to fit through the doors, so it was constructed inside the main test bay adjacent to the shake table.

The Need to Know

Wood light-frame construction represents the most common method of building single and multi-family residential units in the United States, and the push for "green" construction is leading to the use of wood in the commercial market as well. While wood has historically proven to be a good performer in seismic events, the ever-increasing use of engineered lumber and the prevalence of the use of wood in mid-rise construction have fundamentally changed the nature of these buildings compared to what was built 50 years ago.

These changes have led to structures with inherently less redundancy and a reduced lateral strength overall. The ability of engineered lumber to span longer distances in both floors and roofs has created an environment in which there are fewer interior walls. The desire for more openings in these remaining walls concentrates lateral demand into much shorter wall segments. Consequently, our need to understand the true systems-level behavior of these types of structures has increased substantially.

Over the last 12 years, a few projects have incorporated full-scale shake table tests of light-frame wood structures. The 2000 CUREE-Caltech Woodframe Project tested a modest two-story structure under uniaxial ground motion and was an important step in understanding nonlinear modeling issues as well as the influence of nonstructural finish materials. An earlier component of the NEESWood project tested a larger 1,800 square foot two-story townhouse on the shake tables at the University at Buffalo's SEESL laboratory. Utilizing triaxial ground motion input, this "benchmark" structure, as it was referred to, helped establish the expected seismic performance of California-type building stock designed in accordance with modern seismic codes – in this case the *1988 Uniform Building Code* (1988 UBC). While providing another data set to measure the success of predictive modeling, the test also confirmed and augmented the previous findings with respect to nonstructural finish materials adding strength, stiffness and damping – an inherent part of the largely empirical R factor used in today's building codes. In addition, the CUREE-Caltech Woodframe Project tested a three-story apartment building with "tuck-under" parking. This type of structure has one open side on the lowest level to facilitate parking. The tests confirmed that these types of structures are likely to experience torsional problems and/or soft story mechanisms, making them susceptible to collapse.

Along with other test programs, this early research has helped pave the way toward implementation of performance-based seismic design (PBSD) for light-frame mid-rise buildings. Inherent in the assumptions of PBSD is that structural modeling is accurate enough to warrant confidence that a building really will perform in accordance with the predictions. For wood light-frame structures, this is perhaps more difficult because the load path is not as discrete as it is in typical steel or concrete structures. The Capstone project, given the large size of the test structure and triaxial input motions, represents a quantum leap forward in providing a data set to test and refine researchers' abilities to predict structural response as well as validate new and existing construction methods.

The Test Program

The Capstone testing consisted of two phases. Phase I tested the response of the full seven-story mixed-use steel/wood structure. This included two tests on June 30, 2009, utilizing the Canoga Park ground motion recorded during the 1994 Northridge earthquake. The first test used a ground motion scaled to 60% of the original record, while the second test used a ground motion scaled to 140% of the original. This second test corresponded to a 5% in 50 year level event, slightly higher than the Design Basis Earthquake (DBE). Following the second test, the steel special moment frame was reconfigured for Phase II as a robust braced frame via activation of a very stiff bracing system that had been designed for the job. Because of the added bracing participation of the moment frame was locked out of the response in Phase II so that the upper six stories of light-frame construction could be studied on their own.

Phase II testing also utilized the Canoga Park record at various amplification levels. Tests three and four were run at 60% and 120% of original, respectively, but the final test five on July 14 subjected the structure to 180% of the original record. This level of scaling represented the 2% in 50 year return interval event, or Maximum Considered Earthquake (MCE). Between each test, the structure was carefully inspected to see if structural repairs were needed, and in each case they were not.

Figure 2: Wood Nailers Being Installed on Steel Frame.

Design and Construction

The steel special moment frame utilized a new beam-column connection designed to address several issues. First, steel special moment frames (SMF) require bracing to prevent lateral-torsional buckling at plastic hinges, along the length of the beam and at concentrated loads. To work properly, this bracing needs both strength and stiffness, as required by the American Institute of Steel Construcction's (AISC) *Seismic Provisions*, with the stiffness requirement often overlooked because it is actually given in Appendix A of the AISC *Specification for Structural Steel Buildings*. When the floor diaphragm adjacent to the steel beam in a SMF is built using wood, it is nearly impossible to meet both requirements when a detailed investigation of available strength and stiffness is conducted. The new connection addresses this by moving the plastic hinge out of the beam itself, thereby allowing the beams to be designed to remain elastic and unbraced. The second function of this connection is to limit the damage to easily replaceable "fuses". Since these moment-transferring fuses connect beam to column using snug-tight bolts, not only is field erection simple, field repair following an earthquake can be very rapid and supports the resilient structure concept. Transfer of shear and overturning forces from the upper wood structure to the steel frame was accomplished with traditional bolted wood nailers and welded steel brackets to receive the tie-down rods from the shear walls above.

In many respects, the six-story wood structure was essentially the same as typical light-frame construction, with a few exceptions. Walls were framed with 3x6 sill plates, 2x6 studs, and double 2x6 top plates. Floors were framed with 9.5-inch deep I-joists. Shear walls were designed using the segmented approach, and boundary members were comprised of multiple (up to 15) 2x6s spaced about each side of the anchor tie-down system (ATS). Shear transfer made widespread use of ¼-inch diameter self-drilling screws to anchor sill plates to the rim board below. In this case, the "rim board" was actually a glulam member nearly the same width of the wall and occupying the full floor cavity. Gravity and overturning compression forces were very large and the glulams facilitated the needed force transfer through the depth of the floor into the walls below. They also functioned as horizontal collectors in the diaphragm to help complete the load path. One line of shear resistance also contained a new type of experimental, high-strength wood shear wall system known as Mid-ply, the concept for which was developed by Canada's FPInnovations-Forintek. As is to be expected for a building of this type designed for high seismicity, sheathing in the lower stories covered both sides of most walls with nail spacing of two to three inches on center.

The performance-based design objective for the response of the building under the MCE level shaking was for only a 20% chance that average interstory drift would exceed 4%. This level of drift is well into the inelastic range of response of the shear walls and requires new tools to evaluate. The *Direct Displacement Design* procedure developed as part of the NEESWood project was created to do this in a manner that could be implemented in the design office. By using these procedures to look at the nonlinear response of the building, and targeting performance objectives at the true expected drift levels, the designer is better equipped to assure that strength and stiffness is spread over the height of the structure to prevent story drift from accumulating in any one level.

Results

More than 200 sensors of various types were used to collect data on the performance of the NEESWood Capstone building. Optical tracking was employed to obtain data on the gross external movement of the building at each level. Strain gauges determined tie-down forces, accelerometers tracked floor accelerations, and string potentiometers measured shear deformation and post movement in the structure.

In tests one to four, maximum average interstory drift did not exceed 1.25% in any level. In the final MCE level shake of test five, maximum average interstory drift was just under 2%, with the largest drift in any wall line just over 3%. Damage was primarily nonstructural in nature and consisted of drywall cracks around openings. These results indicated very good performance and satisfied the performance objectives, demonstrating both the benefits and utility of *Direct Displacement Design*.

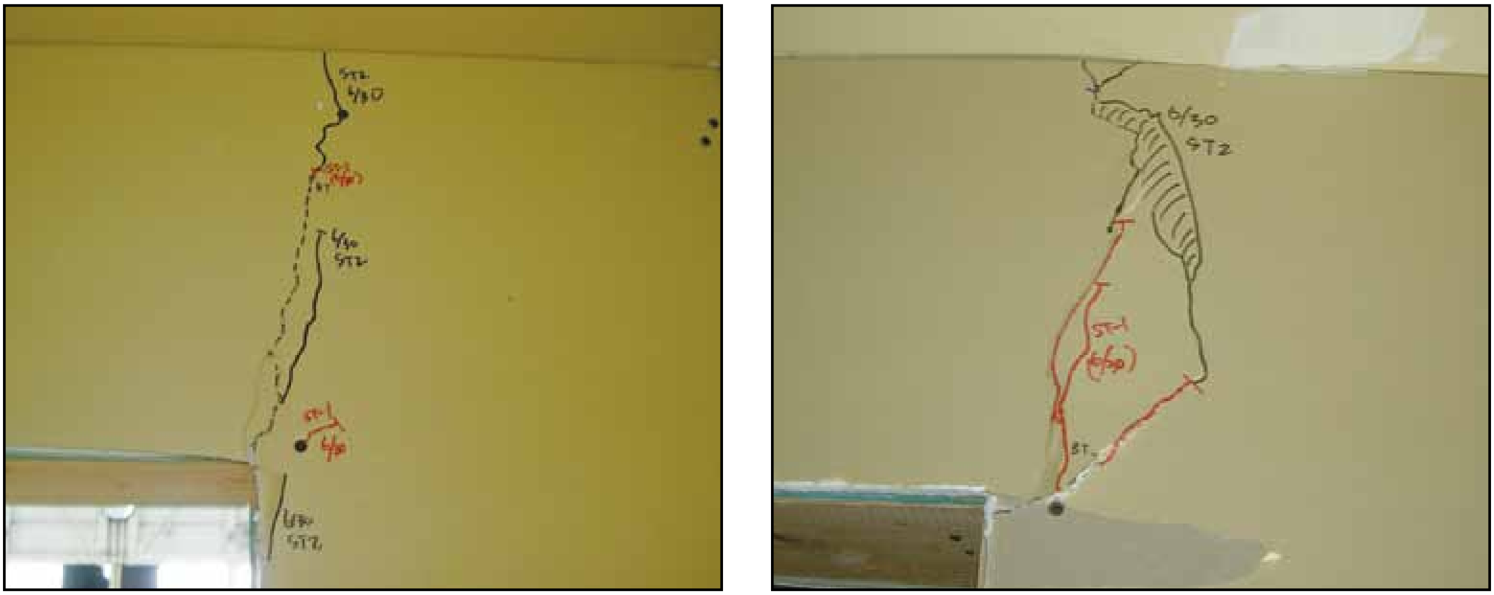

Figure 3: Gypboard Damage at Door (L) and Window (R).

One area of keen interest in the project was the data collected on tie-down forces. While a full account is not possible in this article, a few highlights are in order. First, overall response of the shear walls was not consistent and varied between segmented and perforated. As mentioned earlier, a segmented approach to the shear walls was used. This produced tie-down design demands that we expected might be conservative. This was in fact the case, where the walls behaved in a more perforated manner. However, there also were cases where the predicted demands and the measured demands were actually very close to the anticipated large tie-down forces. 130 kips was measured in the lowest rods in a six-story tall shear wall stack comprised of typical double and single-sided wood structural panel sheathed shear wall construction. The side-by-side Mid-ply walls generated a combined maximum uplift of 170 kips. Both of these results were very close to what was predicted.

Conclusions

Currently, the province of British Columbia in Canada is allowing light-frame wood construction up to six stories. The trend toward urban densification is leading to increased popularity of mixed-use mid-rise buildings, and the combination of bolted steel special moment frames and wood light-frame construction shortens the construction cycle and reduces cost. A road map for building this type of structure to withstand severe earthquakes has been outlined by the NEESWood project. Specifically, the design and construction methods used in the Capstone program, including the *Direct Displacement Design* methodology developed within the NEESWood project and numerous structural detailing configurations, have proven to be very effective in delivering superior structural performance under severe ground shaking. The NEESWood Capstone tests have pushed the boundaries of where light-frame construction is typically used and will serve as a foundation for future implementation of mid-rise wood frame construction in seismic regions.▪

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