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Editorial

This issue of Wood Design Focus features five articles on multi-story residential construction testing, analysis, and design and construction. Four of the articles were developed based on the NEES-Soft Project, a five-university, multi-industry project sponsored by the U.S. National Science Foundation, and one article details the design of a four-story cross laminated timber building in Northern Italy. All the articles have in common their contributions to the seismic design and analysis of wood buildings.

The first article by Bahmani et al. entitled "Performance-Based Seismic Retrofit of Soft-Story Woodframe Buildings" highlights the retrofit of a full-scale four-story woodframe building using direct displacement design techniques within the broader context of performance-based seismic design. The retrofit consists of two steel special moment frames at ground level and then several wood shear walls at each of the upper floors including a large intersecting shear wall detailed for shear transfer. Bahmani et al. presents test results of the retrofit thereby validating the performance-based seismic retrofit approach proposed for soft-story buildings. Then, a retrofit guideline developed by the Applied Technology Council and FEMA, known as FEMA P-807, is explained in the article by Pryor et al. Pryor et al. then goes on to discuss the details of the retrofitting of the same four-story woodframe building with another steel special moment frame designed to adhere to the FEMA P-807 guidelines, including test results. It is concluded that the FEMA P-807 procedure is a logical methodology which can benefit the engineering community and reduce seismic risk. In the third article Gershfeld et al. focuses on retrofit of soft-story woodframe buildings but examines a new concept known as distributed knee-braced frames which can be used as a supplement to a retrofit or by itself as a retrofit. Basic wood engineering and mechanics principles are applied for design and test results presented, which lead to the conclusion that the system appears promising as an alternative for retrofitting typical soft-story buildings. In the fourth article, Pang et al. highlights the method and test results for a relatively new test method known as pseudo-dynamic hybrid testing. A full-scale three-story hybrid test was conducted, also as part of the NEES-Soft Project, using this new approach, which was shown to work well. The bottom story was replaced with numerical retrofits in a computer model while the upper stories were tested physically with four large actuators in the laboratory. In the fifth and final article of this WDF issue, Vassallo et al. explains the details of the design of a four-story cross laminated timber building under construction in Northern Italy including numerical analysis using commercial software. The seismic design seeks to provide the ductility and energy dissipation by concentrating the deformation in the connectors since the CLT panels are essentially rigid.

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Performance-Based Seismic Retrofit of Soft-Story Woodframe Buildings

P. Bahmani, J. W. van de Lindt, S. E. Pryor, G. L. Mochizuki, M. Gershfeld, D. Rammer, J. Tian, and M. D. Symans

Abstract

Soft-story woodframe buildings are recognizable by their large garage openings at the bottom story which are typically for parking and storage. In soft-story buildings the relative stiffness and strength of the soft-story, usually the bottom story, is significantly less than the upper stories due to the presence of large openings which reduce the available space for lateral force resisting system components such as shearwalls. This leads to large inter-story drifts and potential collapse at the ground floor, before the upper stories inter-story experience significant drifts. In many cases the ground floor eccentricity, the distance between the center of rigidity and center of mass, of the soft-story is significant enough to develop considerable in-plane torsional moment in addition to the lateral force caused by the earthquake. A Performance Based Seismic Retrofit (PBSR) procedure can be used to effectively design retrofits that improve the performance of these at-risk buildings. This paper focuses on the PBSR methodology and the application and validation of this retrofit technique to a 4,000 sq. ft. full-scale four-story woodframe building tested at the University of California San Diego (UCSD) Network for Earthquake Engineering Simulation (NEES) outdoor shake table. The structure was retrofitted with various systems including a system that combined wood structural panel sheathing and Simpson Strong-Tie® Strong Frame® steel special moment frames. These types of retrofit techniques improve the performance of the soft-story building while accommodating existing architectural constraints of the building.

Introduction

As early as 1970, the structural engineering and building safety community recognized that a large number of two-, three- and four-story woodframe buildings designed with the first floor used either for parking or commercial space were built with readily identifiable structural deficiencies, referred to as a “soft story”. Often these buildings also have a strength deficiency when compared to the stories above, in which case they are also classified as “weak”. The majority of these multi-story woodframe buildings have large openings and few partition walls at the ground level. This open space condition results in the earthquake resistance of the first story being significantly lower than the upper stories. Thus, many of these multi-story woodframe buildings are susceptible to collapse at the first story during earthquakes. Furthermore, in-plane torsional moments and consequently rotational displacements can be induced when the center of rigidity (i.e. the point where seismic force is resisted) of a story does not coincide with the center of mass (i.e. the point where seismic force is applied). In this case, the building experiences additional displacement due to torsional moment, which causes more damage and increases the chances of collapse.

This paper presents the first generation of Performance-based seismic retrofit (PBSR) and resulting retrofit design using a combination of wood structural panel sheathing and Simpson Strong-Tie® Strong Frame® steel special moment frames. PBSR is essentially the same as performance-based seismic design (PBSD) with the obvious exception of the additional constraints on the design due to existing structural and non-structural assemblies. The PBSD method is a design methodology that seeks to ensure that structures meet prescribed performance criteria under seismic loads. In the PBSR, retrofits were installed such that the building meets the performance criteria at the DBE and MCE level and its torsional response reduces to an acceptable range. In this retrofit design methodology, retrofits are not limited to the bottom story (like

those of the FEMA P-807 retrofit methodology). They can also be applied to the upper stories to increase the strength of the building, leading better overall performance of the structure.

The seismic performance of the retrofitted building with PBSR procedure was evaluated numerically and validated by a full-scale four-story wood-frame building that was tested in summer 2013 at the NEES at UC San Diego outdoor shake table facility as part of the NEES-Soft project. The NEES-Soft project consists of a number of tasks including extensive numerical analysis, development of a performance-based seismic retrofit methodology, and a major testing program with testing at five university-based laboratories to better understand the behavior of these at-risk structures and the retrofit techniques. The listing of all the phases within the project can be found in the WDF article by Pryor et al in this issue. A full Journal paper from the WDF authors is forthcoming and a project report will be available at www.nees.org .

Performance-based Seismic Retrofit (PBSR)

In performance-based seismic retrofit (PBSR), which is a subset of performance-based seismic design (PBSD), the stiffness of the structure is distributed along its height and in the plane of each story such that a target displacement can be achieved under a specific seismic intensity, taking into account nonlinear behavior of the structure. The PBSR method presented herein can be used to retrofit existing buildings such that all stories meet the performance criteria; and it can be used to retrofit buildings that are weak under both translational forces and torsional moments.

Displacement-based design was originally proposed by Priestley (1998) and later modified by Filiatrault and Folz (2002) to be applied to wood structures. Pang and Rosowsky (2009) proposed the direct displacement design (DDD) method using modal analysis and later, Pang et al (2009) proposed a simplified procedure for applying the DDD method which was eventually applied to a six-story light-frame wood building and tested in Miki, Japan (van de Lindt et al., 2010) validating the simplified DDD procedure. Finally Wang et al (2010) extended the work of Pang et al. (2009) to allow correction as a function of building height. This design methodology determines the required lateral stiffnesses over the height of the building such that the building meets the target displacement defined by the building code. This method serves as the basis for a PBSR procedure by distributing the required in-plane stiffness of each story to eliminate the torsional response of the structure (i.e., reducing the in-plane eccentricity) (Bahmani and van de Lindt, 2012). However, for cases in which eliminating torsion cannot be achieved, PBSD that allows some level of torsional response can be used as the basis for design of retrofits for such buildings (Bahmani et al., 2013).

In torsionally unbalanced buildings, in-plane torsional moments, and consequently rotational displacements, can be induced when the center of rigidity of a story does not coincide with the center of mass. In this case, additional rotational displacements due to torsional imbalance should be taken into account whenever they occur. Figure 1(a) presents an N-story building with lumped masses of M_j for the jth story. The total displacement of the center of mass of the jth story is a summation of displacement due to lateral force ($\Delta_j^{Tns.}$) and displacement due to torsional moment ($\Delta_j^{Tor.}$). Eliminate of the torsional response of the structure can be achieved by distributing the retrofit in the plane of each story such that the retrofitted building becomes a structurally symmetric building (i.e., $\Delta_j^{Tor.} \approx 0$). However, if the torsion cannot be feasibly eliminated, the PBSR approach can be applied by assuming a ratio between the displacement caused by lateral force and torsional moments and then satisfying the assumption while applying the retrofit. A three-story torsionally unbalanced wood-frame building was retrofitted using PBSR methodology without eliminating torsion by van de Lindt et al. (2013).

In order to simplify the PBSR procedure, the structure can be modeled by an equivalent single degree of freedom system (Figure 1(b)). The effective weight (W_{Eff}) and lateral force distribution factors (C_v) can be calculated based on the approach outlined in NEESWood Report-05 (2009). The fundamental translational period of the building can be obtained from the displacement response spectrum which is developed based on the design spectral acceleration maps of ASCE7-10 (2010) and should be modified to take into account the effect of equivalent damping. The next step is to obtain the effective lateral stiffness, and consequently the distribution of the stiffness for lateral load resisting elements at each story. The last step is locating the lateral load resisting systems (i.e., shearwalls or other retrofit assemblies) such that the design satisfies the initial assumption

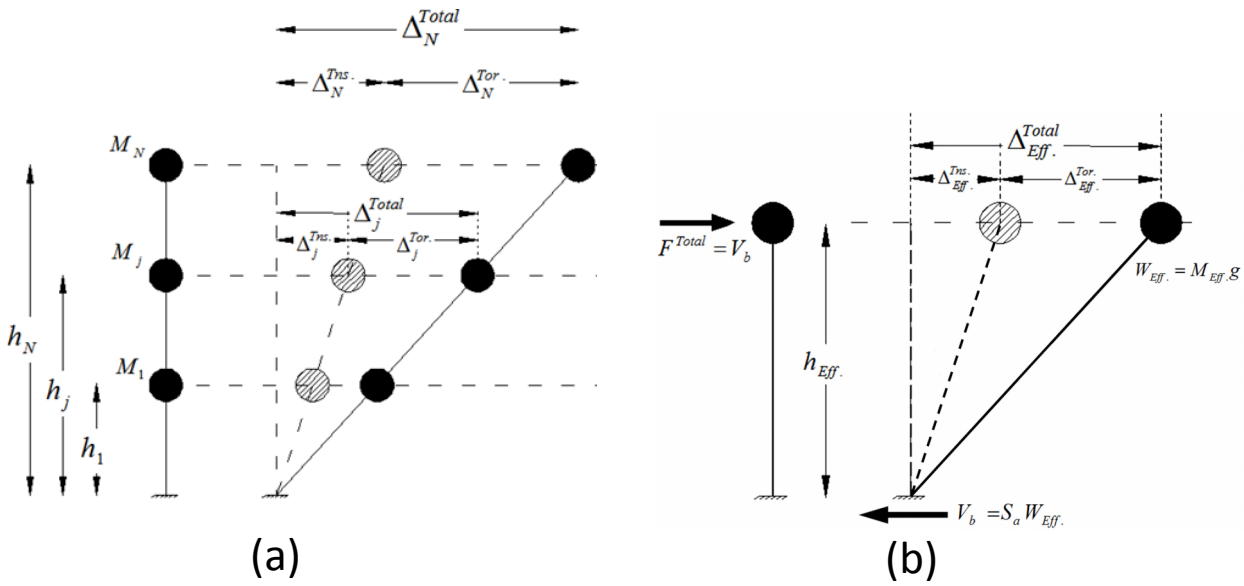

Figure 1. Translational and Torsional Displacements in a Torsionally Unbalanced Building (a) Multi-Story Building, (b) Equivalent SDOF Model of Multi-Story Building

that is made regarding the contribution of torsional response to the total displacement. If the contribution of torsional response is assumed to be close to zero (i.e., eliminating the torsion), then the lateral force resisting elements should be placed such that the CR and CM at each story become very close to each other at the target displacement. The required lateral stiffness can be provided by using the secant stiffness (at the target displacement) of the lateral force resisting elements (i.e., standard wood shearwall, steel moment frame, etc.).

The PBSR procedure described in this paper was applied to a four-story multi-family soft-story wood frame building with a soft-story at the ground level and was tested at the outdoor shake table at NEES at UC-San Diego. The building was designed to have less than 2% inter-story drift with 50% probability of non-exceedance (PNE) at all stories using PBSR methodology subjected to MCE level by eliminating torsional response of the building.

Shake Table Testing of a Full-Scale Four-Story Wood-frame Building

A full scale four-story building was constructed at the outdoor shake table facility at NEES at UC San Diego. On the ground floor, there was 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 and therefore have two essentially solid sides and two open sides. The test building was designed to replicate these conditions, thus making it, in many ways, a worst case scenario. The interior wall den-

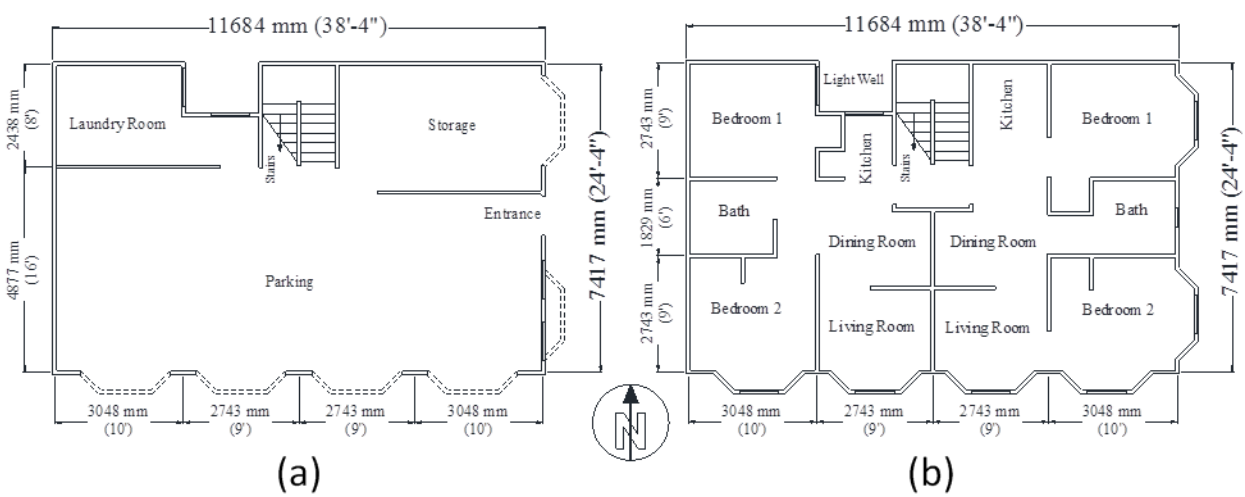

Figure 2. Floor Plans for the Four-Story Building: (a) Ground Story, (b) Upper Stories

Figure 3. (a) Completed 4-story 4000 sq-ft Building, (b) Isometric View of the Building.

sity in the upper stories was high, but this is in line with many soft-story woodframe buildings of that era. The outside was covered with horizontal wood siding (1x8 in. Douglas-Fir grade No 2 or BTR) with two 8d common nails connected to each vertical wall stud. The inside walls were covered with gypsum wall board instead of plaster. Figure 2 shows the ground floor and upper story floor plans for the building (plan dimensions are 24 ft x 38 ft). Each of the upper three stories had two two-bedroom apartment units as can be seen in Figure 2 (b). Figure 3 shows the finished building ready for shake table testing at the UCSD NEES laboratory.

Steel Special Moment Frame (SSMF) and Wood Structural Panel (WSP) Retrofits

In the PBSR procedure the objective was to design the building such that all the stories experience the same level of peak inter-story drift. This utilized the capacity of the upper stories to resist seismic loads and increases the probability of survival of the building under higher earthquake intensities. To achieve this goal, the four-story test building was retrofitted with a Simpson Strong-Tie Strong Frame steel special moment frame (SMF) at the ground level and 15/32" thick sheathing-rated plywood shear wall panels with different nail schedules and tie downs on the selected walls of the upper stories. The steel frames were designed and located such that they did not interfere with the intended use of the space (i.e. vehicle parking), or conflicted with any other architectural aspect of the building. Figure 4 presents the location of Strong Frames and wood shearwalls (SW) that were installed to retrofit the building. Simpson-Strong-Tie Anchor Tie-Down System (ATS) rods were used to transfer uplift forces, induced in the wood shear walls during the earthquake, to the foundation or in case of shear walls above the SSMF to the frame, (i.e. to provide overturning restraint). It should be noted that both the Strong Frame and wood shearwalls were placed such that the center of rigidity moved toward the center of mass at each story which effectively eliminated the concerns associated with torsional response of the structure. Figure 5 shows the Strong Frame, plywood panels, and ATS rods used to retrofit the building.

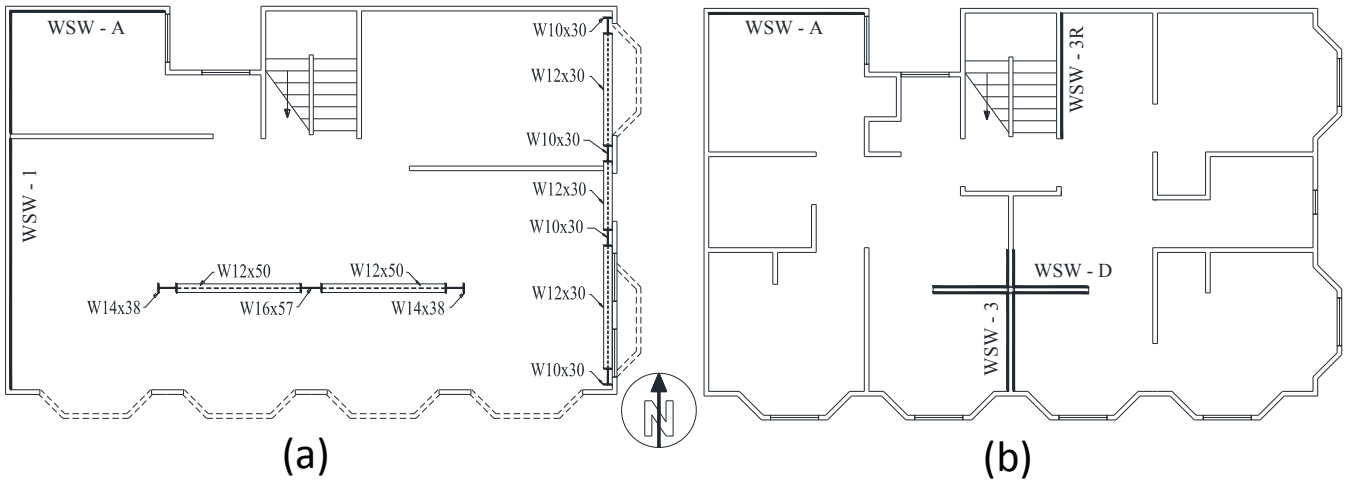

Figure 4. Location of the PBSR Retrofits: (a) Ground Level, and (b) Upper Stories

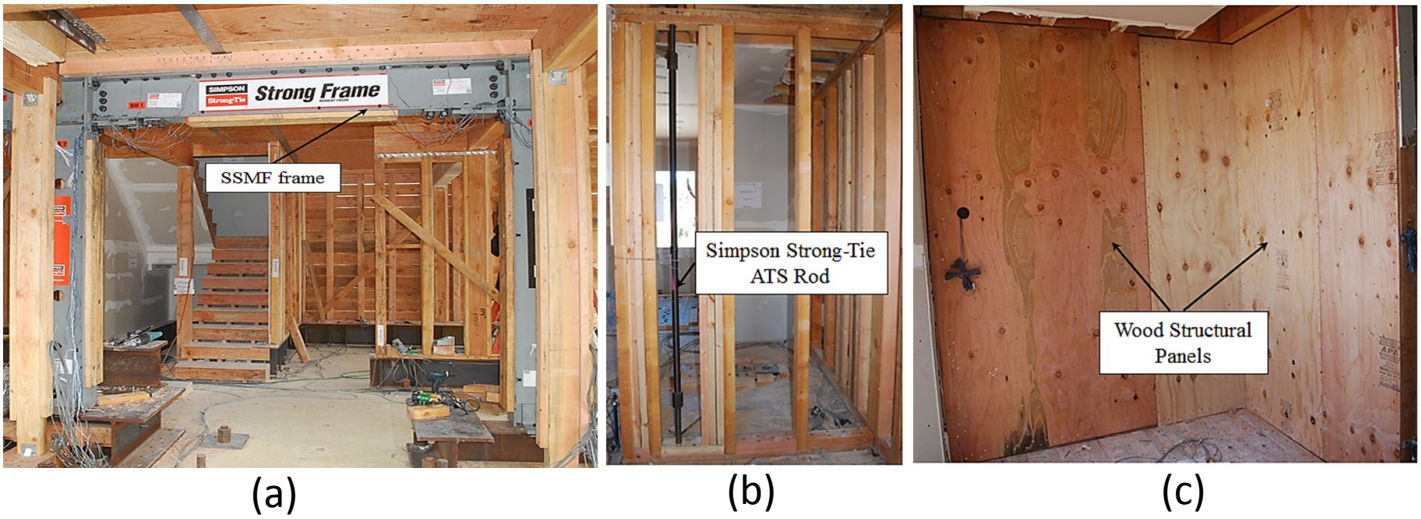

Figure 5. (a) East Span of Strong Frame Installed Parallel to the Motion of Shake Table, (b) ATS Rods and Stud Pack Inside Wood Shearwall, (c) Plywood Panels at Upper Stories

Figure 6 (a) shows the Simpson Strong Frame SMF that was installed at the ground floor parallel to the motion of the shake table and Figure 6 (b) presents backbone curves obtained from numerical pushover analysis for frames installed at the ground floor.

In order to test the retrofitted building, the building was subjected to the similar ground motions that were recorded during the 1989 Loma Prietaand 1992 Cape Mendocino earthquakes. The earthquakes were scaled to DBE and MCE levels with maximum spectral accelerations of 1.2g and 1.8g, respectively. Before and after each seismic test, a white noise test with a root mean square (RMS) amplitude of 0.05g was conducted to determine the fundamental period of the building and its modes shapes, and to obtain a qualitative feel for damage based on changes in building period. Figure 7 presents the building profile at its maximum deformations for five seismic tests along with a time-history response for the test with the highest response. It can be seen that all the stories experience less than 2% inter-story drift which meets the performance criteria (i.e., under 2% drift with only non-structural damages) and meet.

Conclusion

Overall the PBSR method was validated with the level of accuracy that would be expected for this type of testing. The peak inter-story drift response was approximately 2.5% at story 3 with the average of all stories being well under 2%. Full results will be presented in a forthcoming project report which will be available at www.nees.org in 2014.

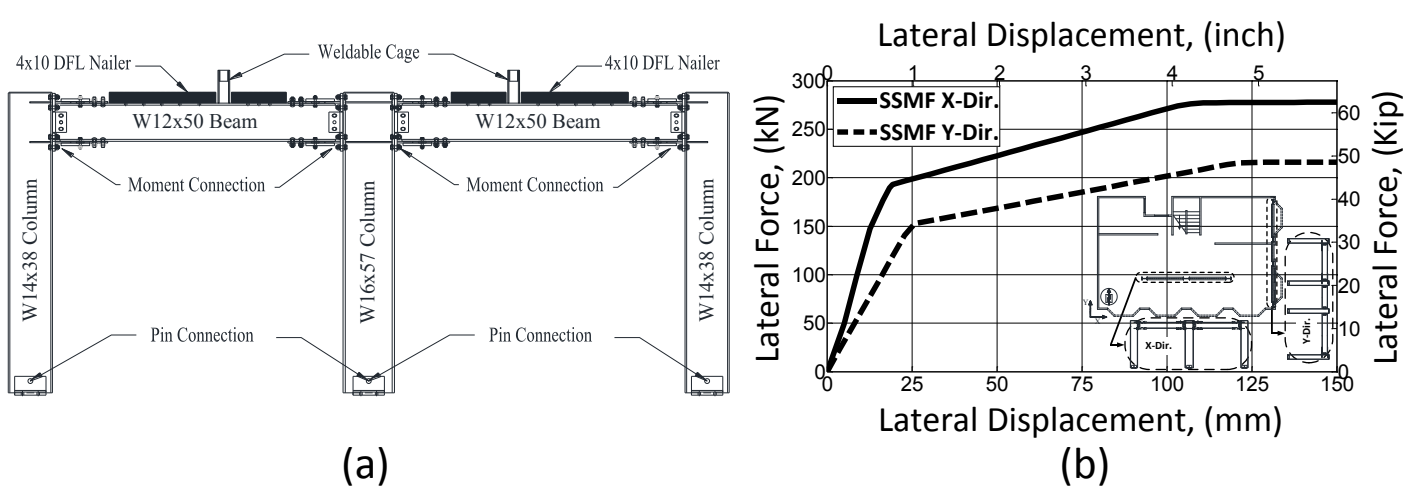

Figure 6. (a) Strong Frame SMF Installed Parallel to the Motion of Shake Table, (b) Backbone Curves of the Strong Frame SMFs Installed Parallel and Perpendicular to the Motion of Shake Table

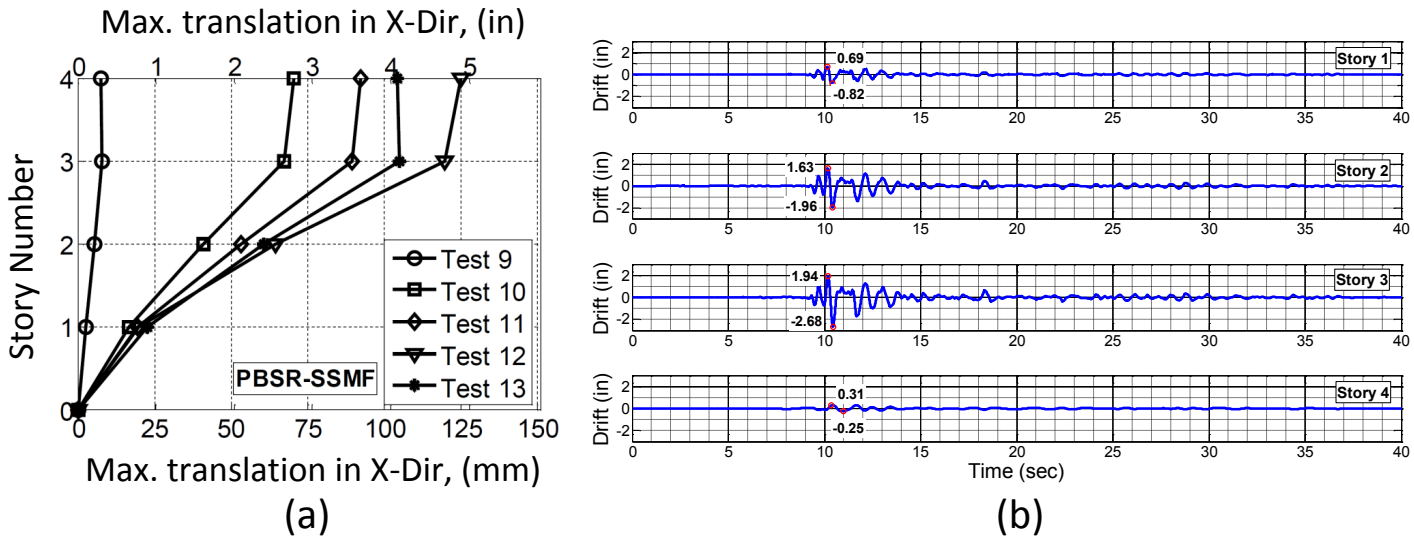

Figure 7. PBSR- Strong Frame SMF Retrofit. (a) Building Maximum Deformation Profile, and (b) Time-History Response to Cape Mendocino Earthquake Record with PGA of 0.89g

Acknowledgments

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Full-Scale Testing of Soft-Story Wood-Frame Buildings with Simpson Strong-Tie® Strong Frame® Steel Special Moment Frame Retrofits-

S. E Pryor, J. W. van de Lindt, P. Bahmani

Abstract

Throughout the high seismic regions of the western United States there are thousands of wood-frame buildings with a vulnerable first story weak- or soft-story condition. Major earthquakes around the world have repeatedly demonstrated the risk this kind of building poses to both life-safety and community resilience following an earthquake. In some areas, such as San Francisco, forward-thinking efforts are underway to mitigate this risk through mandatory retrofit ordinances. To investigate retrofit methodologies of these at-risk buildings further, the NEES-Soft project was undertaken. The NEES-Soft project, whose full title is "Seismic Risk Reduction for Soft-Story Wood-frame Buildings," is a five-university multi-industry three-year project which has many facets including improved nonlinear numerical modeling, outreach, design method development, and full-scale system-level experimental validation of soft-story retrofit techniques. This paper focuses on the successful FEMA P-807-based retrofit and test results for a 4,000 sq. ft. four-story full-scale soft-story wood building tested on the University of California San Diego (UCSD) Network for Earthquake Engineering Simulation (NEES) outdoor shake table and retrofitted using a combination of wood structural panel sheathing and Simpson Strong-Tie® Strong Frame® steel special moment frames.

Introduction

The NEES-Soft Project, whose full title is "Seismic Risk Reduction for Soft-Story Wood-frame buildings," is a five -university, multi-industry, NSF-funded project that has the objectives of: (1) enabling performance-based seismic retrofit (PBSR) for at-risk soft-story wood-frame buildings; and (2) experimentally validating the U.S. Federal Emergency Management Agency (FEMA) P-807 retrofit procedure (FEMA, 2012). ASCE 7-10 (ASCE, 2010) characterizes a "soft story" as having a stiffness that is significantly less than the story or stories above. Often these buildings also have a strength deficiency when compared to the story above, in which case they are also classified as "weak". If either of these deficiencies is particularly bad, they are also given the additional qualifier "extreme". In wood-frame buildings, these soft and weak stories typically occur at the first story as a result of the need for parking. Garage door openings and a lack of interior cross walls to facilitate the parking are usually responsible for creating this condition. In California, thousands of these buildings were built between 1920 and 1970 using construction practices not acceptable by today's codified standards.

The NEES-Soft project consists of a number of tasks including extensive numerical analysis, development of a performance-based seismic retrofit methodology, and a major testing program with testing at five university-based laboratories to better understand the behavior of these at-risk structures and the retrofit techniques. These include the following five test programs: Test Program 1: Real time hybrid testing (RTHT) of a 20-ft long wood wall with and without a toggle-braced damper assembly; University of Alabama Structural Engineering Laboratory; Test Program 2: Reversed cyclic testing of a light wood-frame distributed knee-brace (DKB) assembly for seismic retrofit; California State Polytechnic University San Luis Obispo Structures Laboratory; Test Program 3: Shake table testing of a wood-frame DKB assembly to collapse; Colorado State University Structural Engineering Laboratory; Test Program 4: Slow hybrid testing of a full-scale soft-story wood-frame building with various retrofits; Network for Earthquake Engineering Simulation (NEES) laboratory at the University at Buffalo; and Test Program 5: Shake table testing of a full-scale four-story soft-story wood-frame building with and without seismic

retrofit; Network for Earthquake Engineering Simulation (NEES) laboratory at University of California – San Diego. The remainder of this paper will focus on that portion of Test Program 5 used to experimentally validate the FEMA P-807 retrofit procedure using the Simpson Strong-Tie® Strong Frame® steel special moment frame for the vertical elements of the additional retrofit lateral force resisting system. A full Journal paper from the WDF authors is forthcoming and a project report will be available at www.nees.org .

FEMA P-807

Recognizing the high potential for collapse posed by the very large numbers of weak/soft first story wood-frame buildings in seismically active areas of particularly the western United States, FEMA worked with the Applied Technology Council to create a new set of guidelines entitled *Seismic Evaluation and Retrofit of Multi-Unit Wood-Frame Buildings With Weak First Stories* (FEMA P-807). In an effort to make retrofitting more affordable, P-807 takes several novel approaches for a retrofit guideline. First, it focuses on a specific building type and placing structural items for the retrofit in the first story only. Aside from making the retrofit work less expensive, it also increases the chance that tenants will not be required to relocate during the retrofit. Second, it focuses on improving the first story performance just enough to prevent collapse while at the same time not over strengthening it to the point that upper stories now become the collapse concern. Third, it explicitly considers the strengths of the nonstructural walls to evaluate their contribution to overall performance. Fourth, it does not require explicit structural analysis of the building in question, which could include potentially expensive nonlinear analysis. This is accomplished by comparing the building in question to surrogate models, which are hundreds of similar structures (with many variants each resulting in thousands of possibilities) for which their performance has already been assessed using nonlinear response history analysis. Finally, performance is evaluated in probabilistic terms based on estimated fragilities. This approach can lead to a wider range of mitigation performance options being available to the building owner voluntarily retrofitting or the jurisdiction requiring retrofitting.

Meeting a particular performance objective requires establishing the demand (hazard level), the resistance (performance level) and because of the probabilistic nature of P-807, the desired probabilistic limit that the performance level is exceeded given the hazard level. The default hazard level in P-807 is the Maximum Considered Earthquake (MCE), but any hazard level acceptable to the owner and jurisdiction may be used. As part of the City of San Francisco's mandatory retrofit ordinance, Administrative Bulletin 107 (AB-107) (SF, 2012) stipulates in B.1.1.1 that an acceptable hazard level is a spectral demand of $0.5S_{MS}$ (50% MCE) calculated in accordance with ASCE-05, but using a value of 1.3 for F_a in Site Class E locations. The default performance level for P-807 is what's referred to as the "onset of strength loss" (OSL). This is an interstory drift limit for the material being used that corresponds to the material being on the verge of not being able to resist lateral forces. For OSL two basic categories of material performance are considered in P-807: low ductility systems with OSL at 1.25% interstory drift, and high ductility systems with OSL at 4% interstory drift. OSL is also the performance level used in AB-107's implementation of P-807. Regarding the probability of exceedance (POE), AB-107 sets this at 30% but allows the POE to increase to 50% when the bottom story being retrofitted contains only parking, storage or utility uses or occupancies (B1.1.3).

Proper load path detailing in new construction is often one of the more challenging aspects of design, and the same can be true for retrofits. The analytical procedures in P-807 assume the floor diaphragm above the bottom story is adequate, the foundations below are adequate, and there are sufficient load transfer and load path elements and connections in place to allow the system to perform as desired. For the retrofitted building, these items must be checked and corrected where deficiencies are found to ensure the performance objective can be met.

Steel Special Moment Frame Retrofit

Because of the need to maintain access to first-story parking after the retrofit is complete, some type of moment-resisting framing system will often be needed. These frames may take the form of a cantilevered column element (pinned at the top, fixed or partially restrained at the bottom) or a traditional frame employing fixed, partially restrained, or pinned connections to the foundation. For the NEES-Soft study, pinned-base Simpson Strong-Tie Strong Frame steel special moment frames (SMF), ICC Evaluation Service report ESR-2802, were selected (ICC-ES, 2013). These frames are compatible with the high ductility class of retrofit material assumptions used in P-807 where OSL ≥ 4% interstory drift.

The Strong Frame SMF has several unique advantages that make it ideally suited to this particular application. First, the frame is easily assembled on site using bolted, not welded, beam-to-column connections. This elimi-

nates the fumes and potential fire hazard associated with field-welding traditional SMF beam-to-column connections. These field-installed bolts can be installed without special training or tools because they need only meet the snug-tight condition. This is achievable because qualification testing used only snug-tight beam-to-column bolts. The ability of the frame to perform with this installation has been repeatedly demonstrated in several full-scale shake table tests, both in the U.S. and in Japan (van de Lindt et al, 2011), in addition to those successfully performed in the NEES-Soft test program. Figure 1 shows the Strong Frame SMF installed in the first story of the four-story test building.

Figure 1. Strong Frame SMF Installed in First Story.

Another unique aspect of the Strong Frame SMF is the patented Yield-Link™ structural fuse element of the beam-to-column connection and the benefits it provides to a highly ductile steel frame implemented into a wood structure. Traditional steel SMF's used in seismic applications require beam bracing to prevent lateral-torsional buckling along the length of the beam, and also at the locations of plastic hinges in the beams (AISC, 2011). These flange braces not only require a minimum strength, but a minimum stiffness, and it is often difficult or impossible to supply the needed strength and stiffness in a diagonal flange brace when it is anchored up into a flexible wood floor system. In the Strong Frame SMF a plastic hinge does not form in the beam. Instead, a pair of Yield-Links, above and below the top and bottom beam flanges, respectively, forms the plastic hinge by absorbing the inelastic beam-column rotation demand so that the beams (and columns) remain essentially elastic. Knowing the maximum capacity of the connection, the beam can then be designed without flange bracing. A secondary but potentially important aspect of the Yield-Links is that they are not only bolted to the column, but also bolted to the beam. This means that following an event structural resiliency can be rapidly achieved by the easy replacement of the Yield-Links, should it be deemed necessary (note that Yield-Links were not changed out during multiple back-to-back seismic tests). Figure 2 shows the connection.

Figure 2. Strong Frame SMF Beam-to-Column Connection using Yield-Link Structural Fuses

Pinned bases were selected for the columns of the Strong Frame SMF for several reasons. First, they limit the forces transferred to the foundation, which contributes to less costly foundation remediation. Second, combined with the very simple mechanics of the beam-column connection employing the Yield-Link structural fuses, a high degree of confidence in the

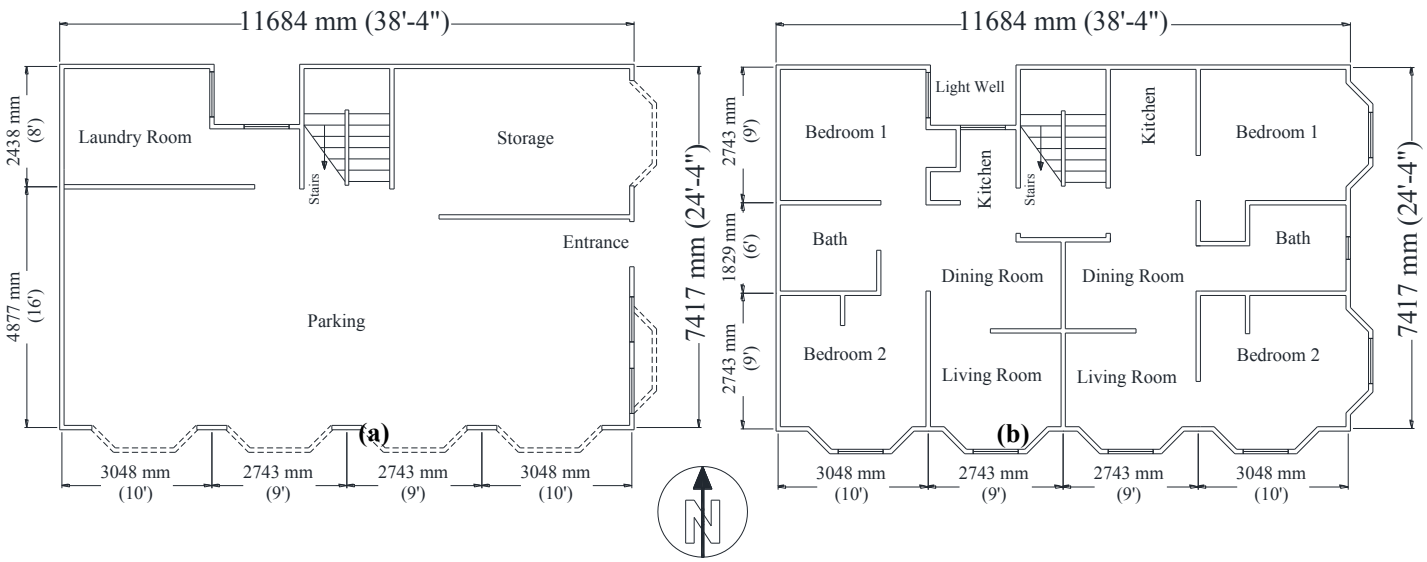

Figure 3. Floor Plans for the Four-Story Building. (a) First Story, (b) Upper Stories

maximum column base reactions is achieved. This in turn is helpful in designing the anchorage to the foundation and the foundation itself. Third, the pushover performance curve for the frame, needed for P-807, also gains a higher confidence due to the higher uncertainties of what is actually delivered when designating a "fixed" base vs. supplying an actual pinned base condition. Based on the required pushover curve determined from the P-807 analysis, Strong Frame SMF's were designed accordingly.

Shake Table Testing of a Full-Scale Four-story Soft-Story Woodframe Building

The design of the four-story soft-story woodframe building for shake table testing at NEES@UCSD was meant to emulate existing buildings of this type in San Francisco. Figure 5 shows the first story and upper story floor plans for the building (plan dimensions are 24 ft x 38 ft). In addition to the first story parking spaces, there were 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 and therefore have two essentially solid sides and two more open sides. The test building was designed to replicate these conditions, thus making it, in many ways, a worst case scenario. The wall density in the upper stories was designed to be livable but dense since this is how many of the soft-story woodframe buildings of that era were designed. The exterior was sheathed with horizontal wood siding fastened to each vertical wall stud with two 8d common nails. The interior walls were sheathed with drywall instead of plaster. Each of the upper three stories had two two-bedroom apartment units as can be seen in Figure 3. Figure 4 shows the finished building ready for shake table testing at the UCSD NEES laboratory.

Figure 4. Completed 4-story 4000 sq. ft. Soft-Story Building Ready for Shake Table Testing

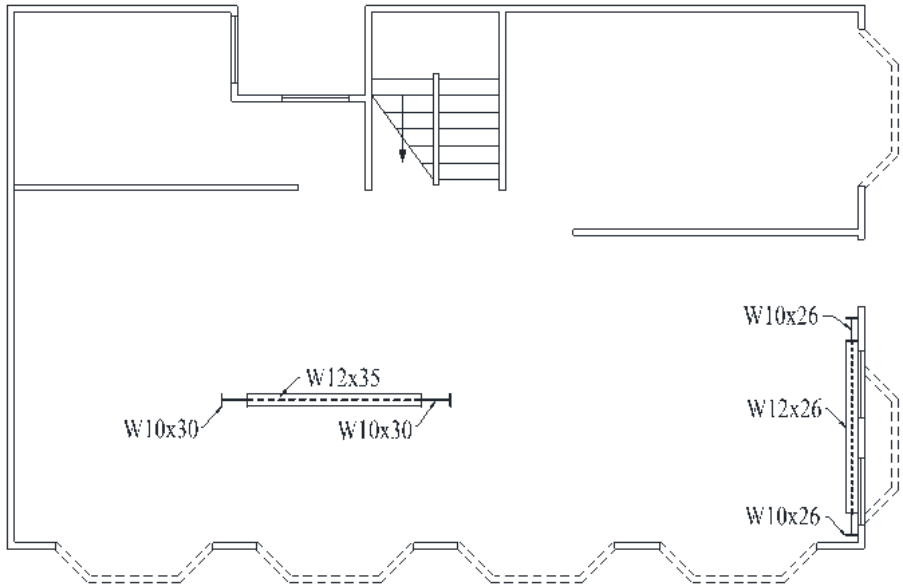

Figure 5. Location of Strong Frame SMF's in the First Story

test, white noise tests with root mean square (RMS) amplitude of 0.05g were conducted to determine the fundamental period of the building and its modes shapes, and to obtain a qualitative feel for damage based on changes in building period.

The Strong Frame SMF's were installed in the first story to provide adequate stiffness and strength to the soft story to meet the P-807 guideline requirements for 60% MCE. The frames were designed such that they could be installed efficiently into the existing building in a short period of time, which reduces the cost of the retrofit. Using the supplied 4x nailer bolted to the top flange of the Strong Frame beam, simple but adequate connections were made to the existing building framing to develop the proper load path into the frame. Figure 5 presents the location of the Strong Frames SMF's in the first story. Figure 6 (a) shows the Strong Frame SMF that was installed in the first story parallel to the motion of the shake table and Figure 6 (b) presents that backbone curve obtained from pushover analysis for frames installed in the first story.

Simulating existing structures, floors were sheathed with 1x6 boards placed at right angles to the 2x10 floor framing, with ¾" thick hardwood (unfinished Red Oak) floor strips placed at right angles to the 1x6's. Lacking performance data for this type of diaphragm construction, the bottom of the floor joists was sheathed with wood structural panels to ensure sufficient diaphragm strength. Because of the Strong Frame Yield-Link struc-

Soft-Story Shake Table Retrofit Testing, Results and Discussion

In the P-807 retrofit procedure, the goal is to achieve an acceptable performance by limiting the retrofit to the bottom story (i.e., soft story) to reduce the cost and time of retrofit. The building was tested using ground motions that were recorded after the 1989 Loma Prieta and 1992 Cape Mendocino-Rio earthquakes. The earthquakes were scaled to spectral accelerations (S_a) of 0.244 g and 1.1g, or ~14% and 60% of MCE (assumes S_{MS} = 1.8g for a typical San Francisco site). The smaller ground motions were used to ensure confidence in the numerical model prior to proceeding to the larger ground motions, which exceeded the required hazard level for the City of San Francisco's retrofit ordinance, AB-107, of 50% MCE. Before and after each seismic

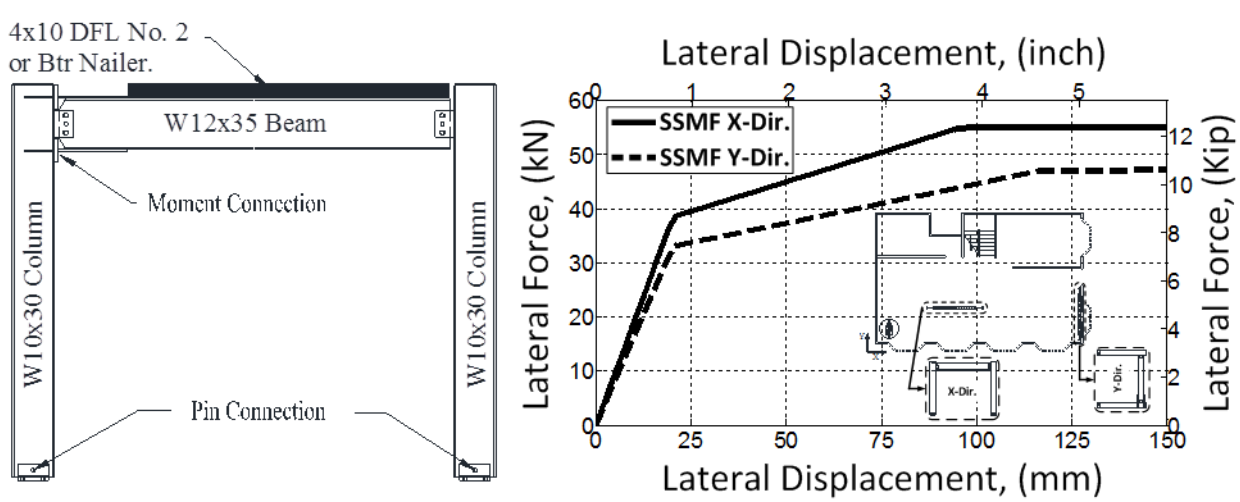

Figure 6. (a) Strong Frame SMF Installed Parallel to the Motion of Shake Table, (b) Backbone Curves of the Strong Frame SMF's Installed Parallel and Perpendicular to the Motion of Shake Table.

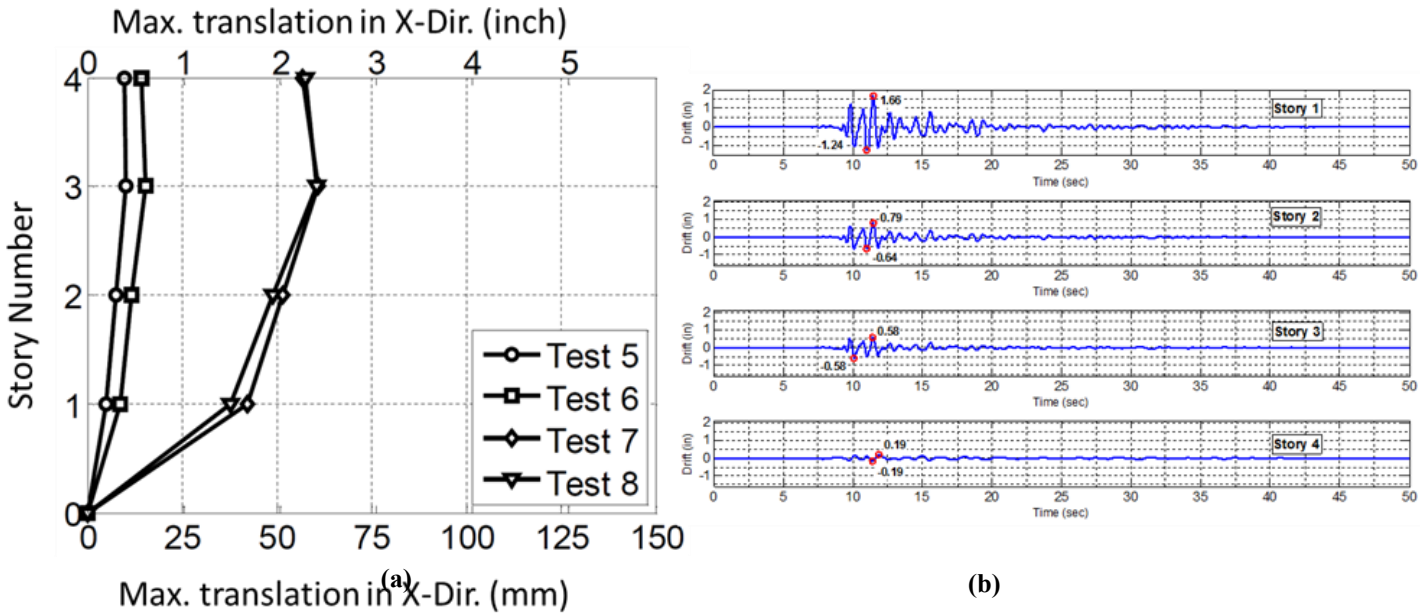

Figure 7. P-807 Strong Frame SMF Retrofit. (a) Building maximum deformation profile(b) Time-history response for all four stories during Test 7

tural fuse at the moment connections, the SMF beams were designed and installed without explicit flange bracing, one of the benefits of this specific system. Figure 7 presents the building profile in its maximum deformations for four seismic tests with a time-history response for the test with the highest peak ground acceleration (PGA).

Conclusion

The NEES-Soft test program results are still being interpreted and synthesized. Overall results, as presented for the full-scale building shake table test program discussed in this paper show that (1) the FEMA P-807 methodology is a logical engineering approach for seismic retrofit of these at-risk buildings when certain constraints such as first-story-only retrofits, are in place; and (2) the unique aspects of the Simpson Strong-Tie Strong Frame steel special moment frame allowed it to perform very well while still being easy to implement into the existing structure. Full project reports with all acknowledgements will be available in the two project reports in 2014 at www.nees.org .

Acknowledgments

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Retrofitting Soft-Story Wood-Frame Buildings with Distributed Knee-Braced (DKB) Frames

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Abstract

In 2012 the U.S. Federal Emergency Management Agency (FEMA) published FEMA P-807, Guidelines for Seismic Retrofit of Weak-Soft Story Wood-framed buildings and in 2013 the city of San Francisco adopted an ordinance mandating retrofit for soft-story wood frame buildings. Both of these are a part of an ongoing effort to address the life-safety and post-disaster management concerns associated with an anticipated significant seismic event on the West Coast and more specifically in the Bay area. The FEMA P-807 document was the formal outcome of the ATC 71.1 committee and the San Francisco ordinance of the San Francisco Community Action Plan for Seismic Safety (CAPSS). The physical validation of some of the FEMA P-807 recommendations and investigation of various retrofit options were some of the objectives of the NEES-Soft, NSF sponsored project that started in 2010 and is expected to be completed by 2014. The project is a five-university effort headed by John van de Lindt, PhD., and also involves design and code enforcement professionals, researchers and manufacturers. One of the retrofit options investigated was the use of wood-frame knee-braced frames that comprised a proposed Distributed Knee-Braced (DKB) System concept. The test results of the DKB system were positive and thus promise to offer an alternative solution for retrofitting typical soft-story buildings.

Introduction

The "soft-story" is a generally accepted term that describes buildings that exhibit a combination of various degrees of weak-story, soft-story and torsional structural irregularities at ground level. Soft-story wood-frame buildings are common occurrence on the West Coast of the United States and a disproportionally high number of them are located in the San Francisco Bay area. These buildings were built in the early 20th century and are typically mixed use or multi-family residential use. The lower level is intended for retail space or parking use and upper levels for residential use. Consequently, the wall density of the upper levels is significantly higher than the ground floor. This generally results in lower strength and stiffness of the ground floor relative to the upper levels. In addition, the ground floor walls have large openings and very few solid elements available to resist lateral loads. This imbalance in solid wall distribution results in high torsional irregularity.

Driven by public policy constraints, mandatory retrofitting of these buildings is limited to ground floor only retrofits. This approach is intended to prevent the disruption of the existing occupancies during retrofitting. The common approach to retrofitting ground floor involves: (1) identifying available existing lines of resistance, (2) adding new lines of resistance to eliminate or reduce torsional imbalance, (3) optimizing the strength and stiffness of the new ground floor lateral load resisting system to improve its load-deformation characteristics and (4) reinforcing of the foundation system to accommodate the new lateral load resisting sys-

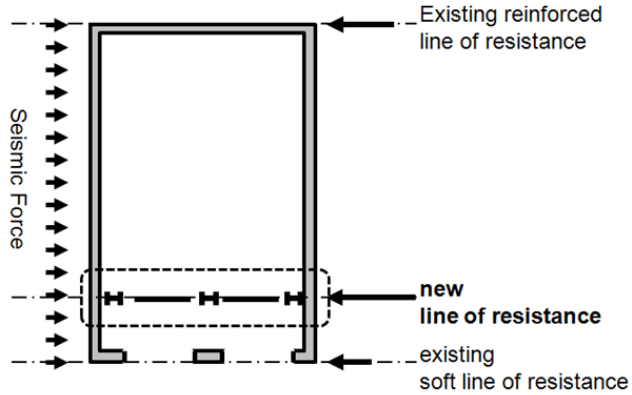

Figure 1. Rigid Frame or Cantilever Columns Retrofit Configuration

tem. Typically, the retrofit strategy involves adding some type of moment-resisting frame along weak/soft lines of resistance. This concentrates resistance into two or three lines of resistance and consequently it becomes necessary to reinforce the diaphragm and add a new foundation, typically a grade beam. This retrofit strategy is shown diagrammatically on Figure 1.

The proposed DKB system replaces or supplements a single "concentrated" line of resistance with multiple "distributed" (as few as two and as many as 20+) closely spaced lines of resistance across a portion of the depth of the ground floor. The DKB system consists of pairs of light wood-frame knee-braced frames matching the spacing of the floor joist (Figure 2). The DKB system engages existing wall studs and foundations previously not contributing to the lateral load resisting capacity of the building. The distributed nature of this retrofit decreases diaphragm spans and reduces diaphragm and foundation demand. The capacity and performance of a DKB system is easily predictable with clearly defined load deformation characteristics. This allows the designer to quickly design an individual knee-braced frame and easily adjust the DKB System capacity by varying the number and location of the individual frames. These features of the DKB system are likely to result in a reduction in design and construction time and cost.

Related Research

The recently completed (1) reversed-cyclic testing at the California State Polytechnic University, San Luis Obispo, (2) slow pseudo-dynamic hybrid testing at the NEES@Buffalo, and (3) shake table testing at Colorado State University, Fort Collins, helped to better understand the behavior of the DKB system and to validate the simplified design approach that predicts its performance. This appears to be the first time the light wood-frame knee-brace frame subjected to lateral loads was tested and its behavior quantified.

Design Considerations-Demand

The determination of the demand on the ground floor of the soft-story wood-frame building built with archaic materials and construction techniques could be guided by either EIBC (Existing International Building Code), ASCE-41 Rehabilitation of Existing Buildings or FEMA P -807 guidelines. While the EIBC offers a simplified approach and ASCE-41 offers a codified performance based design approach, FEMA P-807 guidelines offer a performance optimization approach that focuses on maximizing first floor performance based on limiting the level of damage on the upper floors. It estimates lateral

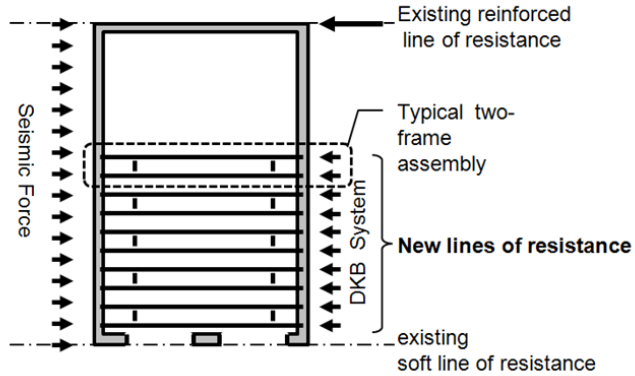

Figure 2. Distributed Knee-Braced (DKB) Retrofit Configuration.

load resisting capacity of all vertical elements of the upper floors that contribute to lateral load resisting capacity using fairly simple tools. It also accounts for torsional effects and estimates demand on the existing elements of the ground floor. It allows selecting retrofit elements with different load-deformation characteristics and locating them to maximize their contribution to improving performance.

For the purposes of this discussion, since the purpose of this paper is primarily the evaluation of the capacity of the DKB system, we will focus on determining the capacity of a single knee-braced frame and not include demand calculations and determination of total number of frames and location needed.

Knee-braced Frame Design

A soft-story 2-3 story wood-frame building observed in the field will typically have 2x4@16"o.c. stud walls supporting 2x10@16"o.c. floor joists aligned with each other, i.e. joists are located directly over the studs. The building might have multiple open bays thus allowing for reinforcement of single bay, double bays or selected bays. Regardless of the DKB system configuration, the knee-brace frame capacity is intended to be controlled by the capacity of the knee-brace connections. The following design steps illustrate the process of determining capacity of the individual knee-braced frame.

1. Knee-brace configuration

Locate knee-brace to stud connection at the center of the stud wall. The knee-brace run/rise ratio should be between 1:2 to 1:3 and is determined based on clearance required for the wall opening and non-interference with the occupancy use. The slope of the knee-brace will directly impact the horizontal loading the frame is able to resist.

2. Knee-brace to joist connection

The knee-brace to joist connection is intended to be designed as the frame capacity fuse. It is expected to control the failure mode of the frame, thus capacity protecting other elements of the system. To determine the capacity of this fuse select the geometry of the joist to knee-brace connection using NDS minimum nail spacing table C11.1.1.6.6. The suggested configuration is shown on (Figure 3). This is a critical connection and should be carefully installed and inspected. The calculation of the capacity of the knee-brace to joist connection is as following:

$Z'=Z(C_mC_tC_gC_\Delta)=97(1.0)(1.0)(1.0)(1.0) = 97$ lbs

$\varphi T_n= \varphi C_n= n\varphi K_f\lambda Z'= (6)(0.65)(3.32)(97$ lbs/nail$) = 1{,}255$ lbs

$\varphi T_n= \varphi C_n= 1{,}255$ lbs (this is also the capacity of the knee-brace to stud connection)

where:

- n = number of nails
- φ = resistance factor
- K_f = format conversion factor
- λ = time effect factor
- Z = reference lateral design value
- Z' = adjusted design value
- C_m = Wet service Factor (assume 1.0)
- C_t = Temperature Factor (assume 1.0)
- C_g = Group Action Factor (1.0 for dowel type fasteners, D<1/4")
- C_Δ = Geometry factor (1.0 for dowel type fasteners, D<1/4")

The horizontal force that a single braced frame with two knee-braces is capable of resisting with this fuse capacity is based on brace slope. For a slope of 1:2, the horizontal component is $\varphi T_n/2.23$, and for a 1:3 slope, horizontal component is $\varphi T_n/3.17$. The slope of 1:2.25 is used for this example. The ultimate horizontal load a frame can resist based on brace slope of 1:2.25 is calculated as following.

$\varphi T_{nx} = \varphi C_{nx}= 1{,}255/2.46 = 510$ lbs (horizontal component of the brace force)

$F_u = 2x510(4'\text{-}½")/(8'\text{-}1") = 510$ lbs

For 1:2 slope $F_u = 1{,}255/2.23=562$ lbs

For 1:2.5 slope $F_u = 1{,}255/2.69 = 466$ lbs

For 1:3 slope $F_u = 1{,}255/3.16=397$ lbs

The remaining elements of the knee-braced frame would need to be designed using the knee-brace to stud connection capacity plus an additional 30% to ensure the fuse action of the connection.

$T_u = C_u = 1{,}255$ lbs x 1.3 $= 1{,}630$ lbs

The x and y components of the knee-brace force are:

$T_{ux} = C_{ux} = 1{,}630$ lbs / 2.46 $=$ 660 lbs

$T_{uy} = C_{uy} = 1{,}016$ lbs x 2.25 $= 1{,}485$ lbs

The corresponding horizontal force that single knee-brace frame with two knee-brace connections is capable of resisting based on 1:2.25 slope is as following.

$F_u= 2x660(4'\text{-}½")/(8'\text{-}1") = 660$ lbs

The 2x4 block placed between two mirror image knee-braced frames is important to avoid separation of the knee-brace from the joist and degrading load deformation characteristics of the connection.

3. Knee-brace to stud connection

The knee-brace to stud connection is to be designed to match the capacity of the knee-brace to joist connection. These two connections are intended to be the primary sources of ductility of the system and to provide capacity protection for all other frame components and connections. Select the geometry to accommodate 6-8d framing nails (0.131"x 3.25") in single shear at this

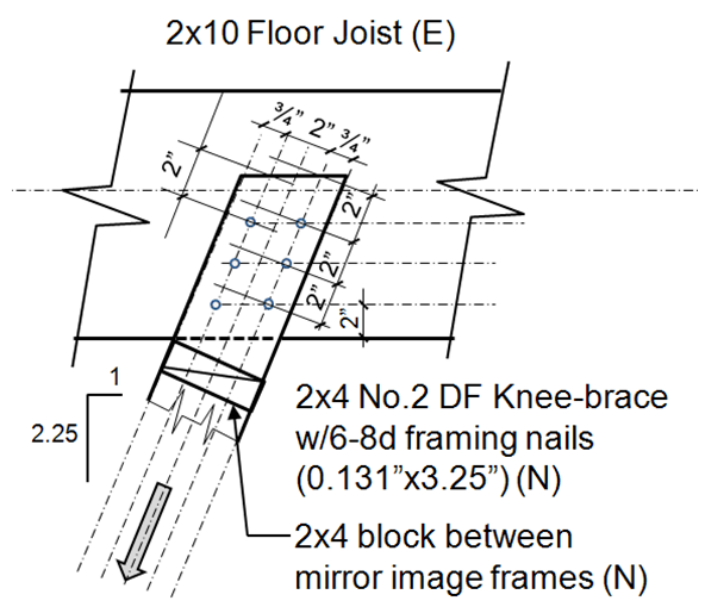

Figure 3. Knee-Brace to Joist Connection

connection as well (Figure 4). Note that the nailing spacing is very tight and placement of larger number of nails or larger nails may require pre-boring to meet spacing requirements. This is the most critical connection in the assembly and needs to be carefully installed and inspected. If the knee-brace or stud splitting occurs, the split components should be replaced.

4. Knee-brace

The knee-brace should also be checked for P_u =1,630lbs. The un-braced length of the knee-brace (assuming no finish provides lateral bracing) is between 50 - 60" depending on the points of bracing selected. If measured from interior face of the stud to the bottom face of the joist along the inner edge of the brace, the un-braced length is approximately 52". Check 2x4 No.2DF-L brace using LRFD as following.

fc = (1630)/5.25=350 psi

φF^*_{cn} = 0.9x2.4x1.0x1.15x1,350=3,353 psi

E_{min} = 0.85x1.76x580,000=867,680

$K_e l/d$= (52)/1.5=35

F_{cE}=(0.822)(867,680)/(35)^2=582 psi

F_{cE}/F^*c = 582/3,353=0.17

C_p=0.1 (0.73-0.73^2-0.21

F'_{cn}=3,353x0.16=536psi > 310 psi

DCR (Demand-Capacity Ratio) =310/536=0.58 (the brace is ok)

DCR = 0.93 if L of 60" is used (still ok)

5. Check joist capacity

Typically the joists would not be a critical element in the knee-braced frame. A simplified check could be quickly performed to confirm this. The joist in addition to uniform dead and floor live load will also resist vertical and horizontal concentrated loads from the knee-braces (note that the dead load is already present when the knee-brace is installed in a retrofit scenario and technically would already generate stresses due to dead load in the joist at full span). The existing 2x10 @ 16"o.c floor joists commonly used in soft-story buildings will usually have sufficient capacity to handle additional forces generated by the knee-brace and do not require any reinforcement.

6. Check capacity of the new stud assembly

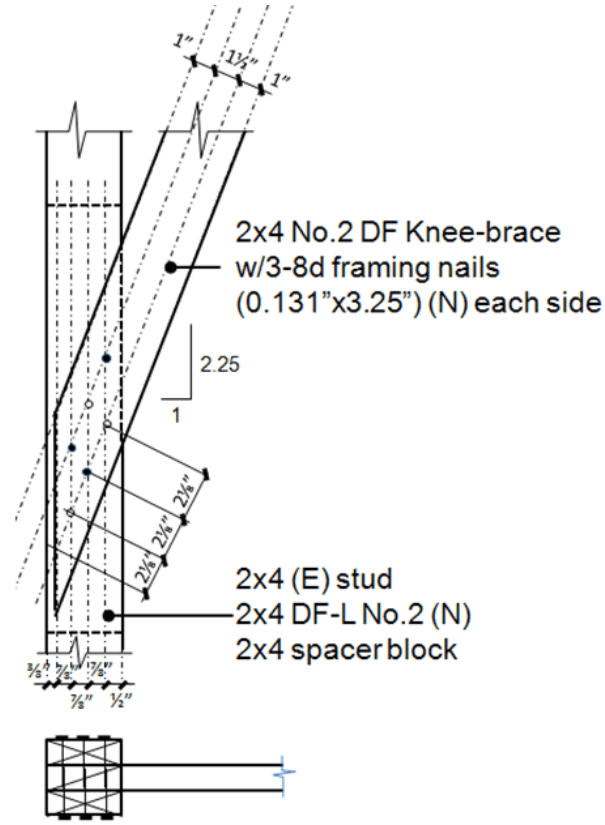

Figure 4. Knee-Brace to Stud Connection

The stud capacity check is critical. This is a primary load bearing component that the "fuse" is intended to protect. The stud would be checked for combined bending and increased axial load. Typically the load combination 1.2D+1.0L+1.0E will control the design. The cross-section that can be used for this check consists of three 2x4 studs, since combined loading technically occurs below or above the knee brace connection point depending on the direction of the lateral load. Detailed calculations will typically require that the existing stud is reinforced with an additional full height stud and two spacer studs below and above the connection.

7. Stud to bottom plate connection

The stud to bottom plate connection needs to accommodate a shear force and, if present, an uplift force. The shear force will typically be half of the total horizontal force, although this connection will likely have a sufficient number of end and toe nails to handle shear from lateral loads (2-16d end nails and at least 4-8d toe nails). One or two Simpson A-35 brackets should still be added to accommodate these loads. This avoids relying on withdrawal capacity for uplift forces and simplifies inspection.

8. Stud to Joist Connection

The stud to joist connection is critical. This connection must be able to transfer horizontal force and the uplift force generated by the knee brace. Horizontal force is half of the frame lateral load. The uplift loading would need to be determined using LRFD load combination 0.9D+E or ASD load combination (needed for hardware selection) 0.6D+0.7E. Typically, Simpson H2-A Hurricane ties would be a minimum requirement for this connection. If uplift is too high, additional hardware might be required. The failure of this connection will result in sudden collapse, which the "fuse" intends to protect against this failure mode as well.

The knee-braced frame design approach described above was used to design the test specimens for reversed cyclic and shake table tests (Figures 5,6,7 and 8).

Reversed-Cyclic Testing (Cal Poly San Luis Obispo)

The reversed-cyclic testing was performed at Cal Poly San Luis Obispo. The DKB system test structure was built to accommodate four knee-braced frames (4-frame DKB). It was retrofitted using the design approach outlined above. The testing was performed for two different configurations: 20ft and 10ft knee-braced frames (Figure 9 and Figure 10). The brace slope was approximately 2.5:1 for the test specimen (Gershfeld, 2013). The 3D Revit model of the test set up for a 20-ft frame is shown on Figure 11. The weights of approximately 1600 lbs to replicate additional two stories above were located over two exterior walls, but are not shown on the model for clarity.

Reversed-Cyclic Test Protocol, Results and Observations

The CUREE-Caltech loading protocol for deformation controlled quasi-static cyclic testing was selected for the DKB frame system testing (Figure 12). The results include hysteresis curves for both the 10ft and 20ft DKB

Figure 5. Knee-Brace to Joist Connection (Note Blocking Between the Two Knee-Braces)

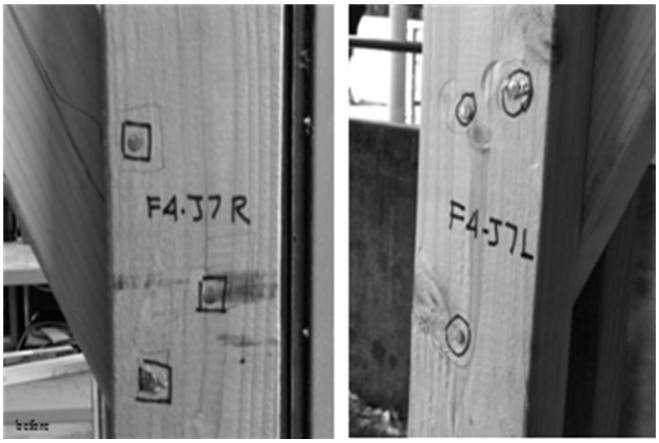

Figure 6. Knee-Brace to Stud Connection

Figure 7. Stud Assembly to Joist Connection with H2-A Left and Right Hurricane Ties

Figure 8. Stud to Bottom Plate Connection

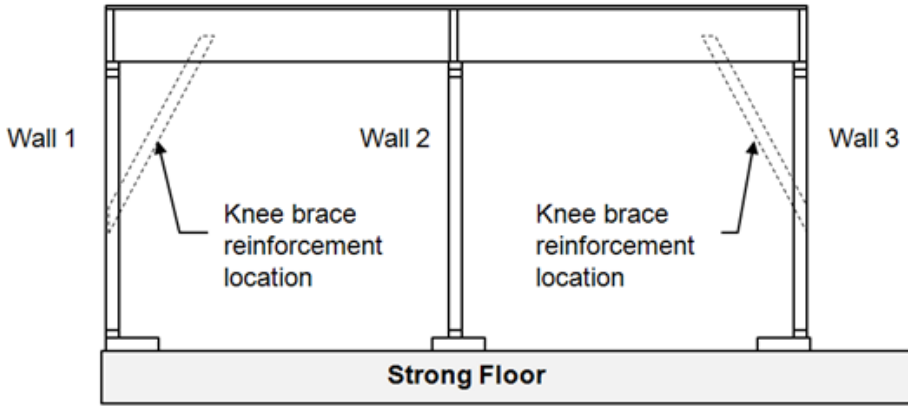

Figure 9. DKB System 20-ft Frame Configuration

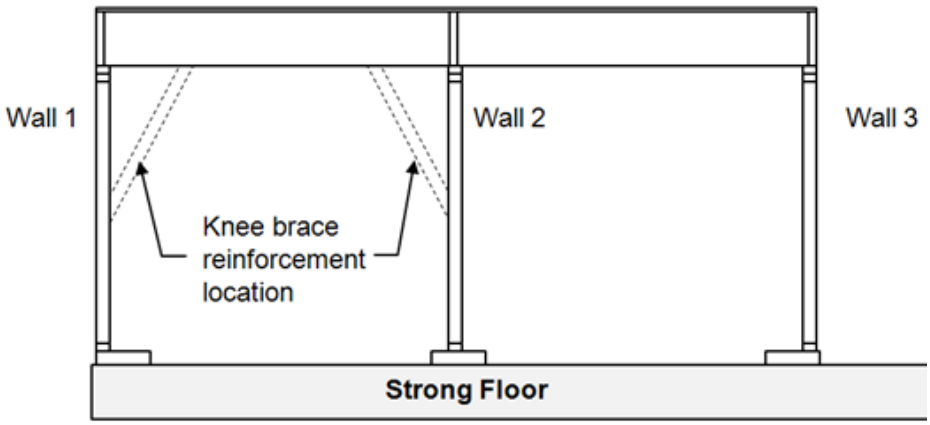

Figure 10. DKB System 10-ft Frame Configuration

four-frame systems (Figures 13 and 14). A summary of the 10ft and 20ft frame test data is provided in Table 1. The data shows little difference in the overall behavior between the two frame configurations. The frames were observed before and after the testing. Overall, there was no damage or significant displacement noted at the base connection nor at the stud to joist connection. All damage was concentrated at the knee-brace to joist and knee-brace to stud connections, as it was intended.

Shake Table Test Results and Observations

The shake table test was performed at Colorado State University. The four-frame DKB specimen for the shake table test was identical to the reversed-cyclic test shown in Figure 15. A large steel plate was bolted to the diaphragm to facilitate installation of seismic mass and a number of steel beams were welded to the plate to provide a total weight of 3200 lbs.

Instrumentation consisted of two string potentiometers attached to the floor diaphragm, and two linear variable differential transducer (LVDT) attached to each wall. The LVDT's were used to measure the relative movement between the sill plate and the base as well as the uplift of the stud. In addition, accelerometers were attached to the shake table and the roof diaphragm. The specimen was subjected to a number of design-based earthquakes (DBE) and maximum considerable earthquakes (MCE) ground motions, which are listed in Table 2.

Reduction in seismic intensity of the Superstition and Rinaldi (Figure 16) earthquakes was necessary because of the total shake table stroke length of 20 in. The DKB configuration performed well under first five ground motion records and no significant damage was observed during the damage inspection. The final test resulted in the collapse of the specimen due to failure of the knee-brace to joist connection (Figure 17). The inter-story peak displacement under Loma Prieta MCE ground motion was 3.29 in (2.9% story drift) with no residual deformation (Figure 18). The inter-story peak displacement under 85% Rinaldi earthquake was 13 in. (11.8% story drift) which resulted in an unstable structure ready to collapse (Figure 19).

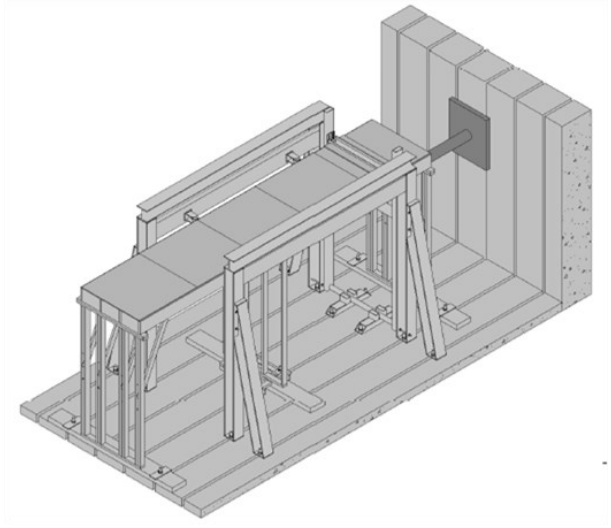

Figure 11. 20-ft frame 3D Revit Model Test Set-up.

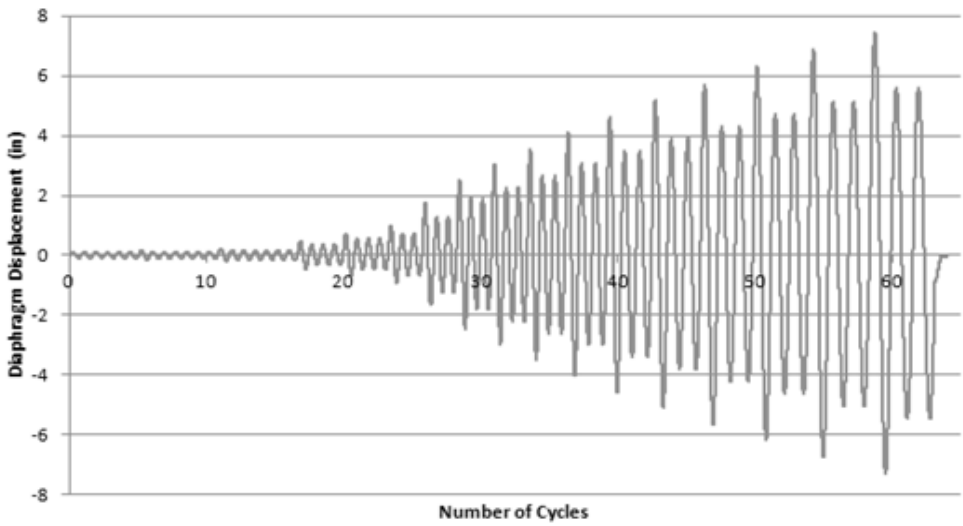

Figure 12. CUREE-Caltech Standard Protocol.

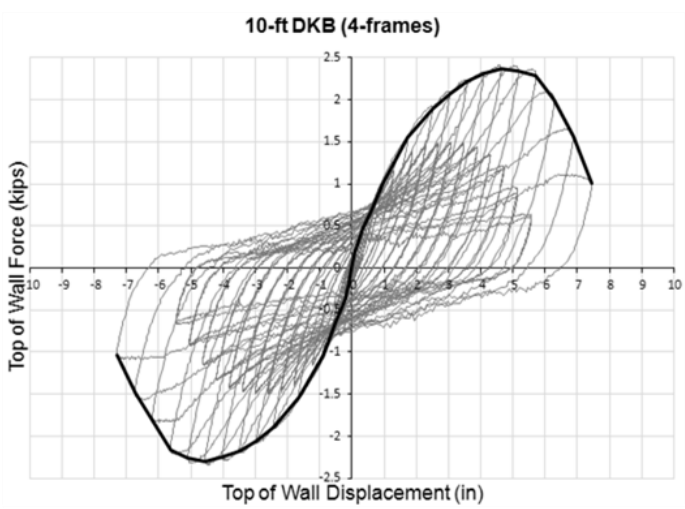

Figure 13. Hysteresis curve for 10-ft DKB Frame (4 Frames)

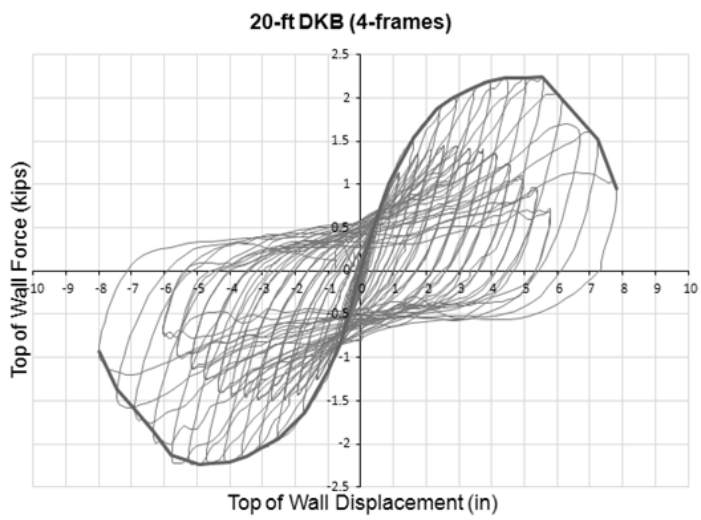

Figure 14. Hysteresis curve for 20-ft DKB Frame (4 frames)

Table 1. Load Deformation Curves Summary

Label	10 ft (4 Frames)		20 ft (4 frames)	
	(+)	(-)	(+)	(-)
F_{max} (kips)	2.41	2.34	2.24	2.24
D at F_{max} (in)	4.59	4.49	5.54	5.18
Drift at F_{max} (%)	4.51	4.41	5.44	5.09
D_{max} (in)	7.45	7.29	7.82	8.00
Drift at D_{max} (%)	7.30	7.20	7.66	7.86
F at D_{max} (kips)	0.98	0.99	0.95	0.93

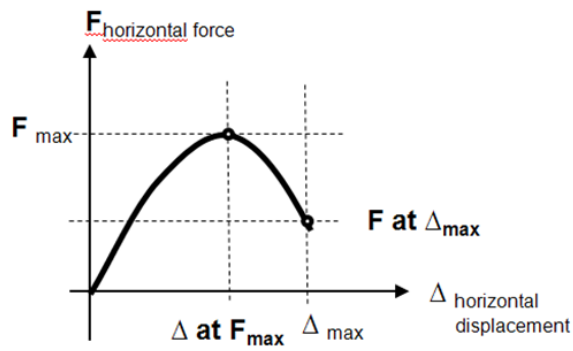

Figure 15. Undamaged Four-Frame DKB System Test Setup Prior to Testing

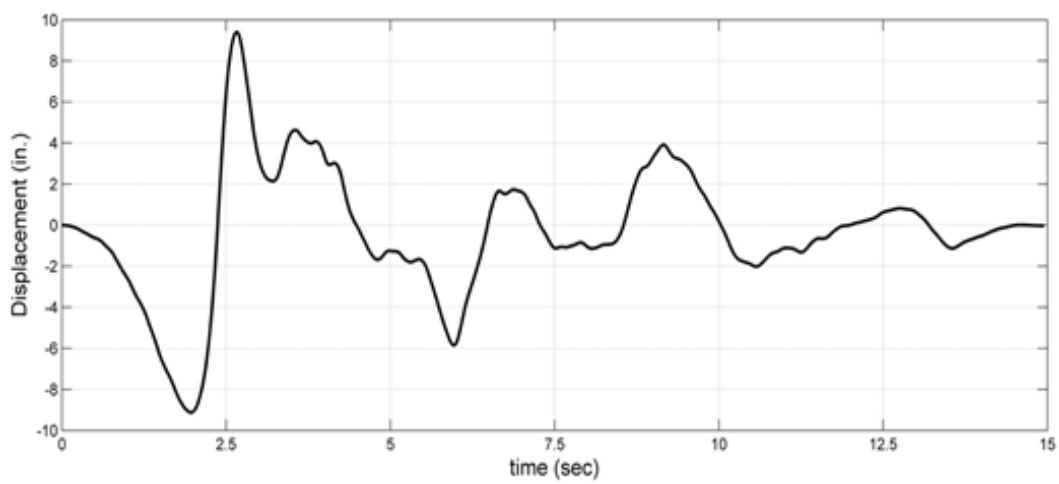

Figure 16 Ground Motion Record RINALDI85 MCE

Figure 17 Knee-Brace to Joist Nailed Connection Failure

Table 2. Ground Motions Testing Sequence

Test	EQ Name	Component
1	Loma Prieta	G03000 DBE
2	Loma Prieta	G03090 DBE
3	Loma Prieta	G03090 MCE
4	Cape Men.	RIO360 MCE
5	Superstition	70% B-ICC090 MCE
6	Northridge	85% RINALDI85 MCE

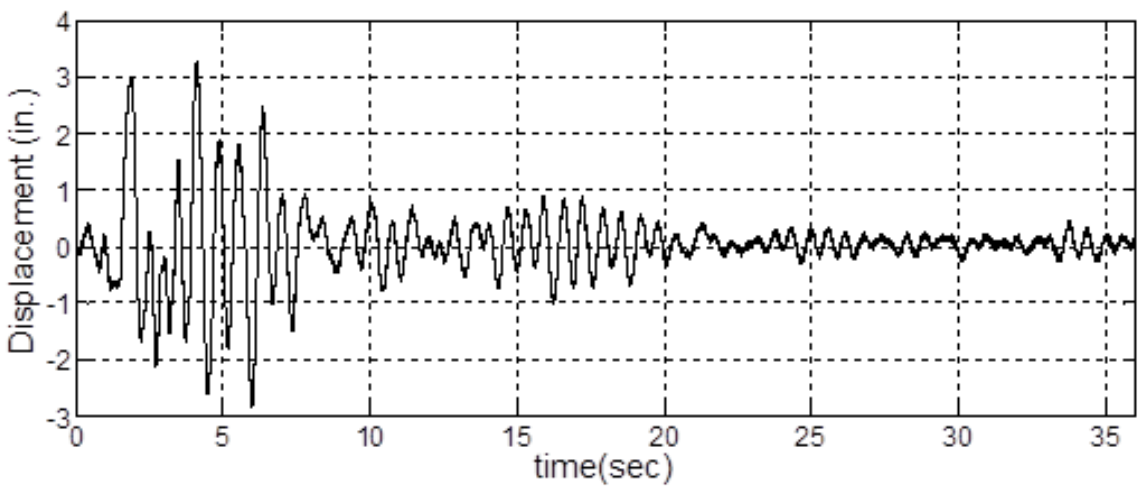

Figure 18. Inter-Story Displacement Time Histories for Test 3 (100% Loma-Prieta MCE)

Conclusions

The tested load-deformation characteristics of the DKB system indicate that it is a viable solution for retrofitting soft-story buildings. The simplified analysis and physical test data provide a good prediction of both system behavior and capacity. The system is proven capable of accommodating drifts of approximately 4.5% at peak capacity (0.56k per frame) and upwards of 7.5% drift while still supporting 0.24k (approximately 40% of the maximum frame capacity). The system successfully withstood multiple ground motions with no residual deformation at 3.29 in. displacement (2.9% drift) and failed at 85% Rinaldi with 13" displacement (11.8% drift). From a design perspective, because of the load fuse approach, the peak strength of this system can be predicted by simple calculations. The predicted capacity of a single DKB frame was 0.47k (1:2.5 brace slope) as compared to its peak test capacity of 0.56k (minimum of two frames are required). As such, the DKB system retrofit allows fairly simple adjustment to the total capacity of the lateral load resisting system by varying (1) the capacity of an individual frame, (2) the total number of frames, and (3) placement of frames.

The DKB system successfully addresses the constraints associated with retrofitting soft-story buildings (diaphragm reinforcement, addition of new foundation and providing favorable load-deformation characteristics) and has the potential of being a cost effective solution for a variety of soft-story building archetypes.

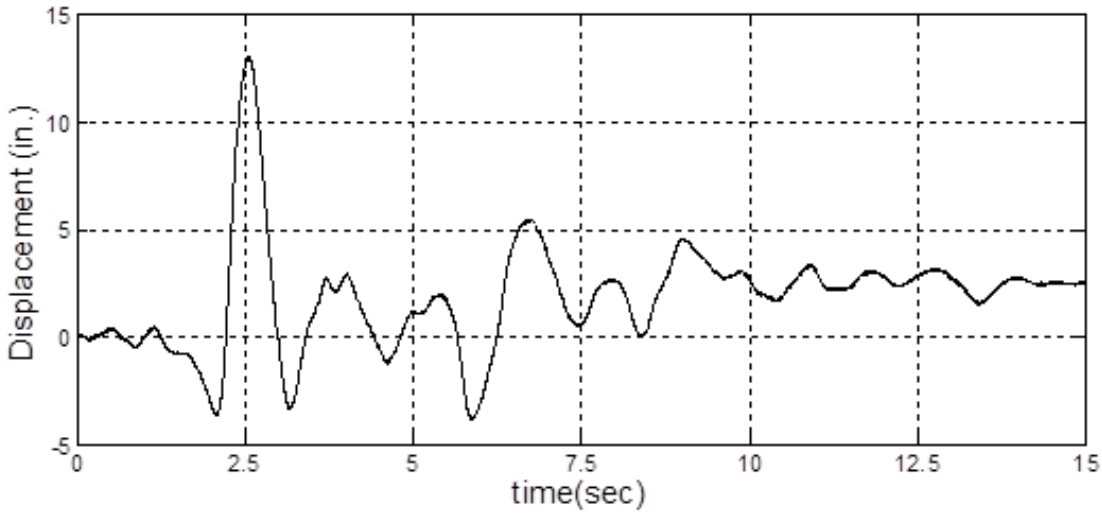

Figure 19. Inter-Story Displacement Time Histories for Test 6 (85% Rinaldi)

Acknowledgements

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Hybrid Testing of a Soft-Story Light-Frame Wood Building with Seismic Retrofits

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Abstract

Historically, two experimental methods have been utilized for testing the seismic performance of light-frame wood buildings, namely, quasi-static cyclic testing and shake-table testing. This paper presents the application of a relatively new testing method known as *pseudo-dynamic hybrid testing* to experimentally evaluate the effectiveness of different seismic retrofits to reduce the collapse risk of a full-scale three-story building with a soft-story deficiency in the ground floor. The full-scale three-story test building was one of the two full-size buildings tested as part of the NSF funded NEES-Soft project. Six different retrofits to strengthen the weak first-story were considered in the NEES-Soft project including cantilever column, steel moment frame, cross-laminated timber rocking panel, viscous fluid damper, shape memory alloy devices and distributed knee-braces. To evaluate the performance of each retrofit including the impact of each retrofit on the upper stories, the three-story hybrid building was divided into two fully coupled subassemblies (often termed substructures). The first story was numerically analyzed with different retrofits using a nonlinear model capable of large displacements, while the remaining part (i.e. upper stories) was constructed and physically tested using servo-hydraulic lab equipment. This paper provides an overview of the NEES-Soft hybrid test framework along with the results for one of the hybrid test phases.

Introduction

Light-frame wood buildings are the most common form of construction in the United States, in particular for residential buildings and low-rise commercial buildings. Modern engineered light-frame wood buildings using shear walls sheathed with plywood or oriented strand board (OSB) have been observed to perform well in terms of life-safety in design level earthquakes. However, in large parts of the US, the majority of residential buildings predate modern (seismic) building codes. In San Francisco, more than 80% of the residential buildings were constructed before 1970, prior to the introduction of modern seismic design provisions in building codes. Many of these older two- to four- story apartment buildings in San Francisco were built with tuck under parking with few interior walls in the ground floor (Figure 1). As a result, the lateral load carrying capacity of the ground floor may be significantly less than that of the upper stories, making the structure susceptible to pancake collapse of the first story in earthquakes. This structural deficiency is referred to as a “soft-story”. These soft-story town-homes and apartment buildings provide the principal form of housing in San Francisco. Recognizing the potential seismic hazards posed by these buildings, the Community Action Plan for Seismic Safety (CAPSS) project was initiated by the San Francis-

Figure 1. An Example Soft-Story Building in San Francisco, California (Photo: Mikhail Gershfeld).

co Department of Building Inspection (DBI) in 2008 to provide DBI and policymakers with an action plan to reduce earthquake risks in existing privately owned building stocks (ATC 2010a). The CAPSS study identified about 2,800 potentially vulnerable soft-story buildings with large perimeter wall openings at the ground floor in San Francisco. Without retrofit, it was estimated that 43% to 85% of these multi-story wood buildings will be deemed unsafe after a M_w 7.2 earthquake on the San Andreas fault, and about 25% of these buildings would collapse (ATC 2010b). About 58,000 people that reside in these soft-story buildings could be displaced from their homes after the quake and approximately 2,000 business entities housed in the ground floor would be severely damaged or destroyed. However, with proper retrofit measures, the CAPSS study estimated that the collapse risk of these soft-story buildings could be reduced to less than 1%. While the CAPSS study is a Bay Area study, it is well understood that this is a problem throughout many communities in California and likely many of the other seismic regions of the U.S.

A five-university multi-industry research project, known as the *NEES-Soft Project*, whose full title is "Seismic Risk Reduction for Soft-Story Woodframe Buildings", funded by the National Science Foundation via the Network for Earthquake Engineering Simulation (NEES), was initiated in 2011 (van de Lindt et al. 2012). The main vision of the NEES-Soft project is being accomplished through the development of an improved nonlinear numerical model, advanced retrofit methodology, and a series of full-scale experimental validation of soft-story retrofit techniques. Two full-scale soft-story buildings were tested as part of the NEES-Soft project. A three-story building was tested at the University at Buffalo NEES facility (NEES@UB) using a relatively new seismic experimental method known as the *pseudo-dynamic hybrid testing*. The other full-scale test was a shake table testing of a four-story building using the outdoor shake table at the NEES facility at University of California at San Diego (NEES@UCSD). The focus of this paper is on the application of hybrid testing to evaluate the effectiveness of different retrofits on the three-story building.

What is Hybrid Testing?

Historically, two experimental methods have been utilized for testing the seismic performance of lateral load resisting systems of light-frame wood buildings, namely, quasi-static cyclic testing and shake-table testing.

In quasi-static cyclic testing, one or more actuators are utilized to impose pre-determined displacement or force protocols to the test structure at a rate that is usually much slower than the actual rate that would be experienced by the test structure in an actual earthquake. While cyclic testing can be used to evaluate the performance of wood-frame building subassemblies such as shear walls, the test results could be affected by the choice of loading protocols. A displacement protocol with a large number of loading cycles will introduce premature fatigue failures that may not represent the actual loading cycles experienced in real earthquakes. In addition, a slow cyclic test cannot accurately capture the dynamic behavior or loading rate effect.

Shake-table testing provides a realistic way of simulating the vibration dynamics at the rate of earthquakes. Compared to a cyclic test, a shake-table test typically takes a significant amount of time and effort to setup; hence, full-scale shake table testing can be cost prohibitive in certain cases particularly when multiple test specimens are required. In addition, buildings that can be tested are usually limited by the size and payload of the shake table.

With the advancements made in earthquake experimental methods in recent years, an emerging testing method known as hybrid testing has seen increased attention. Hybrid testing allows the evaluation of the system behavior of a building by testing only part of the building using one or more actuators. This new testing method involves the creation of a *hybrid model* that consists of two complementary parts: (1) an *experimental substructure* - a physical test structure representing a portion of the full building, and (2) a *numerical substructure* – a computer model representing the remainder of the full building. Unlike the quasi-static cyclic test in which the displacement protocol is predefined, in a hybrid test, the displacement to be imposed to the physical substructure is computed during the test based on a fully coupled representation of the experimental and numerical substructures. The governing equation of motion for a structural system subjected to earthquake loading is given by Eqn. :

$$M\ddot{u} + C\dot{u} + f(u) = -M\ddot{u}_g$$

Where *M* and *C* are the mass and damping matrices; $\ddot{u}$ $\dot{u}$ and u are the acceleration, velocity and displacement vectors, respectively. $\ddot{u}_g$ is the input ground motion and *f(u)* is the restoring force vector which is displacement history dependent. For discussion purpose, consider a two-story stacked shear wall system as depicted in Figure 2 which consists of a physical wall in the first story and a numerical wall model in the second story. Con-

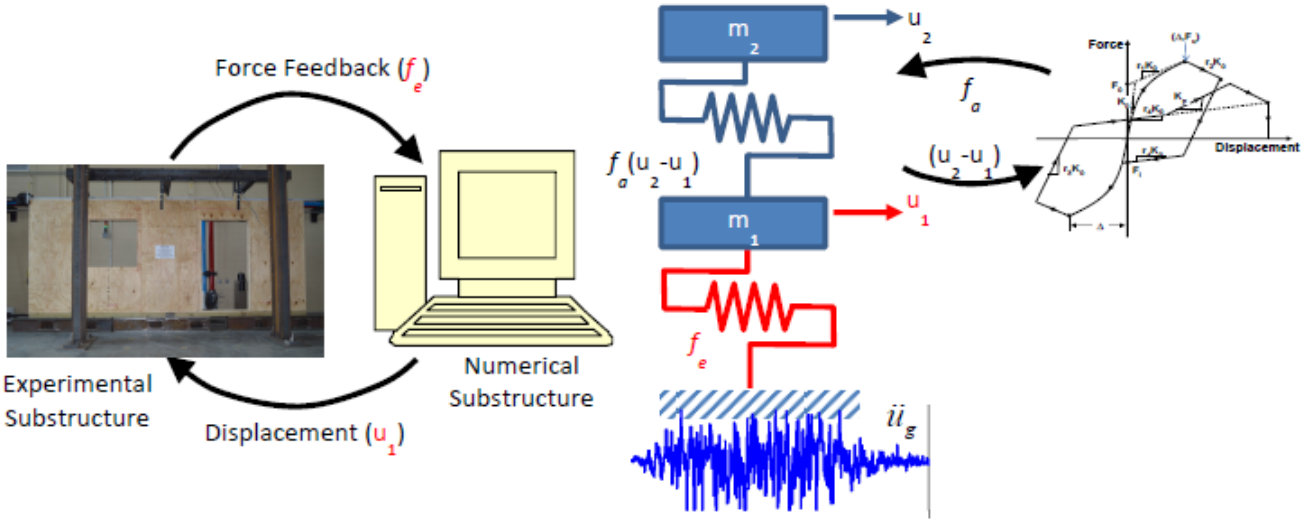

Figure 2. A Hybrid Simulation Model for a Two-Story Stacked Shear Wall System

sidering only the horizontal degrees-of-freedom (DOFs) (i.e. ignore P-delta effect), the equation of motion of the stacked shear wall system can be written as:

$$\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix}\begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \end{Bmatrix}+\begin{bmatrix} c_{11} & c_{12} \\ c_{21} & c_{22} \end{bmatrix}\begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \end{Bmatrix}+\begin{Bmatrix} f_e - f_a \\ f_a \end{Bmatrix}=-\begin{Bmatrix} m_1 \\ m_2 \end{Bmatrix}\ddot{u}_g$$

where f_a and f_e represent the restoring forces of the second-story analytical wall model and the first-story experimental wall, respectively. The two inertia masses (m_1 and m_2), the damping matrix, which can be computed using the mass and stiffness proportional Rayleigh damping model, and the input ground motion are defined numerically. The pseudo-dynamic hybrid test begins by solving Eqn. to obtain the displacements (u_1 and u_2) for the next time step. The restoring force of the second story wall is determined analytically using a hysteretic model which is part of the numerical substructure. The boundary condition between the numerical and experimental substructures is maintained by imposing the computed displacement u_1 on the top of the test wall (experimental substructure) using an actuator. Meanwhile, the restoring force measured from the first-story test wall is fed back to the numerical model (i.e. Eqn.) for solving the displacements for the next step. Thus, a *closed-loop experiment* is formed that integrates numerical and physical components into one structural response simulation to evaluate the system behavior.

NEES-Soft Hybrid Simulation Framework

The hybrid testing method discussed in the previous section was applied to the full-scale three-story NEES-Soft building to evaluate the performance of six retrofits. The NEES-Soft hybrid testing setup is shown schematically in Figure 3. Focusing on evaluating the impact of each retrofit on the upper stories, the three-story hybrid building was divided into two complimentary parts: (1) the numerical substructure was the first story with different retrofits, which was modeled using a simulation package developed as part of the NEES-Soft project, called Timber3D (Pang et al. 2012); and (2) the experi-

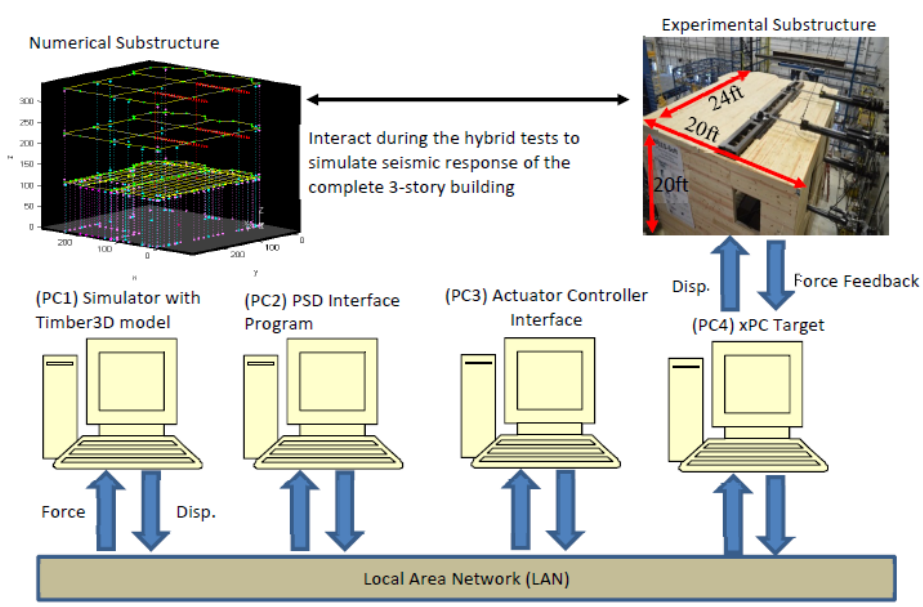

Figure 3. NEES-Soft Hybrid Testing Setup

mental substructure was the upper two stories that was constructed on the strong floor at the NEES@Buffalo facility and physically loaded using hydraulic actuators. This hybrid testing setup allowed the evaluation of different retrofits without the need to physically re-construct the first story multiple times. The effectiveness of different retrofits can be determined or quantified by examining the damages occur in the physical second and third stories. The hybrid testing setup consisted of five main components (see Figure 3): (1) the numerical model for the three-story structure hosted on PC1, (2) the hybrid testing controller programed in Matlab-Simulink hosted on PC2, (3) the actuator controller interface and data logger hosted on PC3, (4) the Matlab/xPCTarget running the hybrid testing controller in real time (hosted on PC4), and (5) the hydraulic actuators connected to the two-story experimental substructure.

Experimental Substructure

The NEES-Soft hybrid test building was designed with features to represent typical San Francisco Bay Area wood buildings constructed between 1920 and 1970. The complete test structure was a three-story light-frame wood building with a tuck-under parking garage in the first story. Figure 4 shows the floor plans of the ground floor and the upper two floors which had identical layout. Except for the walls around the stairwell, the ground floor had no other interior walls in order to remain open for vehicle parking, i.e. a garage.

As stated before, in the NEES-Soft hybrid tests, the first story was the numerical substructure; hence only the upper two stories were constructed on the strong floor in the NEES@Buffalo lab, serving as the physical substructure. The physical substructure was installed on a steel channel (MC6x15.3) which was bolted to the strong floor with 5/8 in. diameter threaded rods spaced at 24" on-center (Figure 5 upper left). All lumber used for the physical model was Douglas Fir-Larch. A 4x4 (DF-L No. 2) lumber was bolted onto the steel channel and served as a nailer for the physical substructure. The height of the physical building from the ground level to the roof, including the height of the steel channel and the 4x4 wood nailer, was 18.65 ft, and the height of numerical first story was 9.75 ft. The plan dimension of the building was approximately 20x24 ft (Figure 4). Various sizes and grades of Douglas Fir-Larch lumber was used to construct the physical substructure. The bottom plates of the second story walls (i.e. the first story of physical substructure) were fastened to the nailer using 16d common nails spaced at 16 in. o.c. The wall framing consisted of 2x4 (Stud grade) studs spaced at 16 in. o.c. with a 2x4 bottom plate and 2x4 double top plates. The floor joists were 2x10 (No. 2) spaced at 16 in. o.c. (Figure 5 lower left). To impose the desired translational and in-plane rotational motions the physical structure,

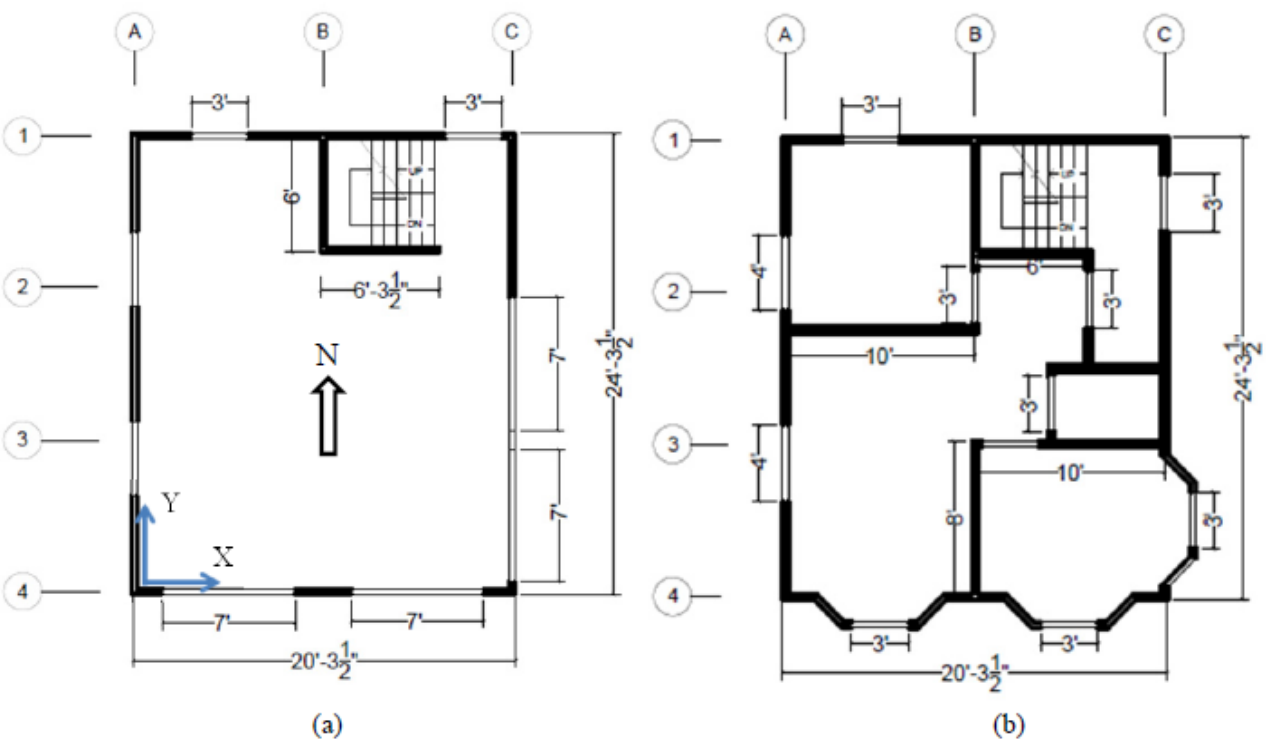

Figure 4. Floor Plans for the NEES-Soft Three-Story Hybrid Test Building; (a) Ground Floor; (b) Stories 2 and 3.

Figure 5. Construction of the NEES-Soft Hybrid Test Building

four 22-kip hydraulic actuators were attached to the test structure, with two on each diaphragm. These actuators can rotate 13° at the point of connections in both horizontal and vertical directions. The building exterior was covered with 1x10 (DF-L No. 1) horizontal wood siding fastened with two 8d common nails per stud. The interior walls and the inner side of the exterior walls were covered with ½ inch thick gypsum wall boards (GWB). Note that GWB was not a typical construction material for pre-1970's buildings. For this experimental study, GWB was selected to re-produce stiffness characteristic similar to that of stucco and plaster on wood lath.

Numerical Substructure

The Timber3D program was developed in Matlab/Simulink using a co-rotational formulation and the large displacement theory (Pang and Shirazi 2013). The floor and roof diaphragms of the three-story building are modeled using a two-node 12-DOF frame element with geometric nonlinearity (Figure 6). The 3D frame element has two nodes with six DOFs at each node (three translations in the element local x, y and z directions and three rotations about the element x, y and z axes). The lateral stiffness of shear walls and axial stiffness of wall studs are modeled using a zero-length 6-DOF link element (three relative translations and three relative rotations between two frame elements).

In order to reduce the computational time, a nodal condensation technique utilizing shape functions was used to reduce the size of the global matrix (Pang et al. 2012). Using this condensation technique, the size of the global stiffness matrix depends only on the number of frame elements. At each time step, the global stiffness matrix was assembled based on the rotated coordinate system of the individual frame and link elements (i.e. using a co-rotation formulation). The co-rotational formulation allowed more accurate predictions for the building behavior under large deformation.

System ID Test

Prior to each hybrid test, a System ID test was performed to determine the initial stiffness matrix of the experimental substructure. Figure 7 shows an example 4x4 stiffness matrix determined via the System ID test.

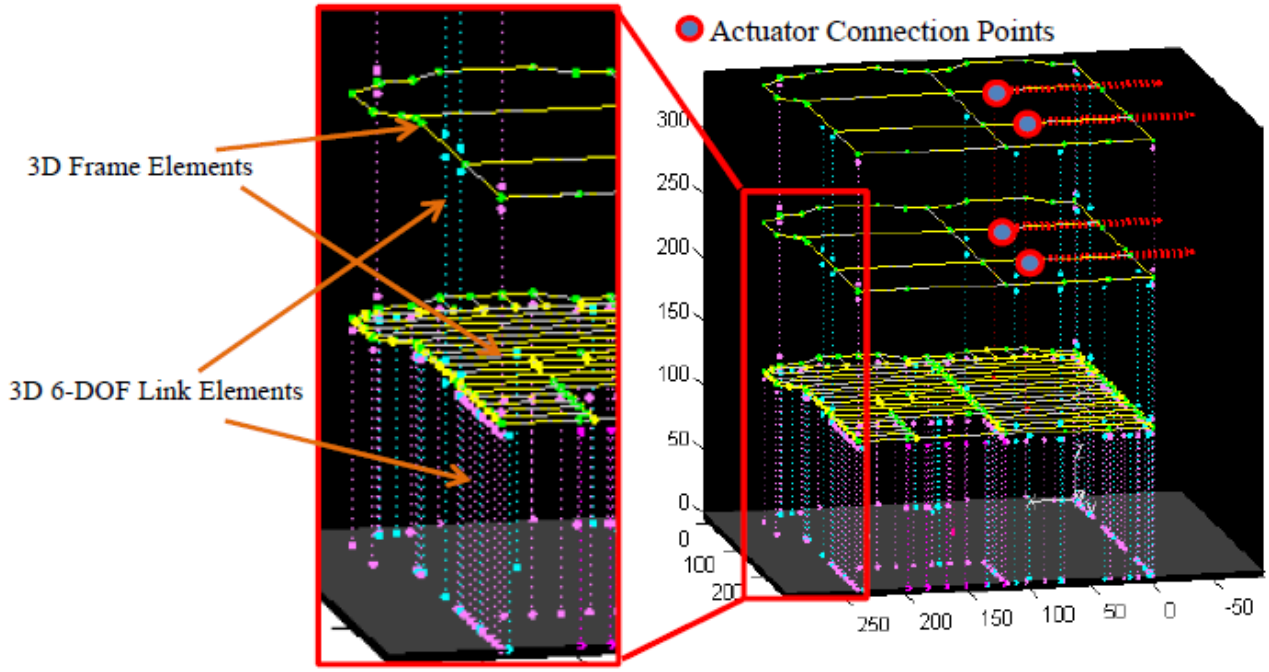

Figure 6. Numerical Substructure

The stiffness values of each column were determined by moving each actuator individually to displacements of +/-0.1" while holding the others in place and measuring the force feedbacks from all four actuators. This empirically determined stiffness matrix was used for three purposes: (1) to quantify the pre-test condition of the physical substructure; (2) to perform a pre-test numerical simulation; and (3) to determine the fundamental period of the full three-story building. These three were used in conjunction with a visual inspection of the physical test building to determine if the building was safe to continue for testing.

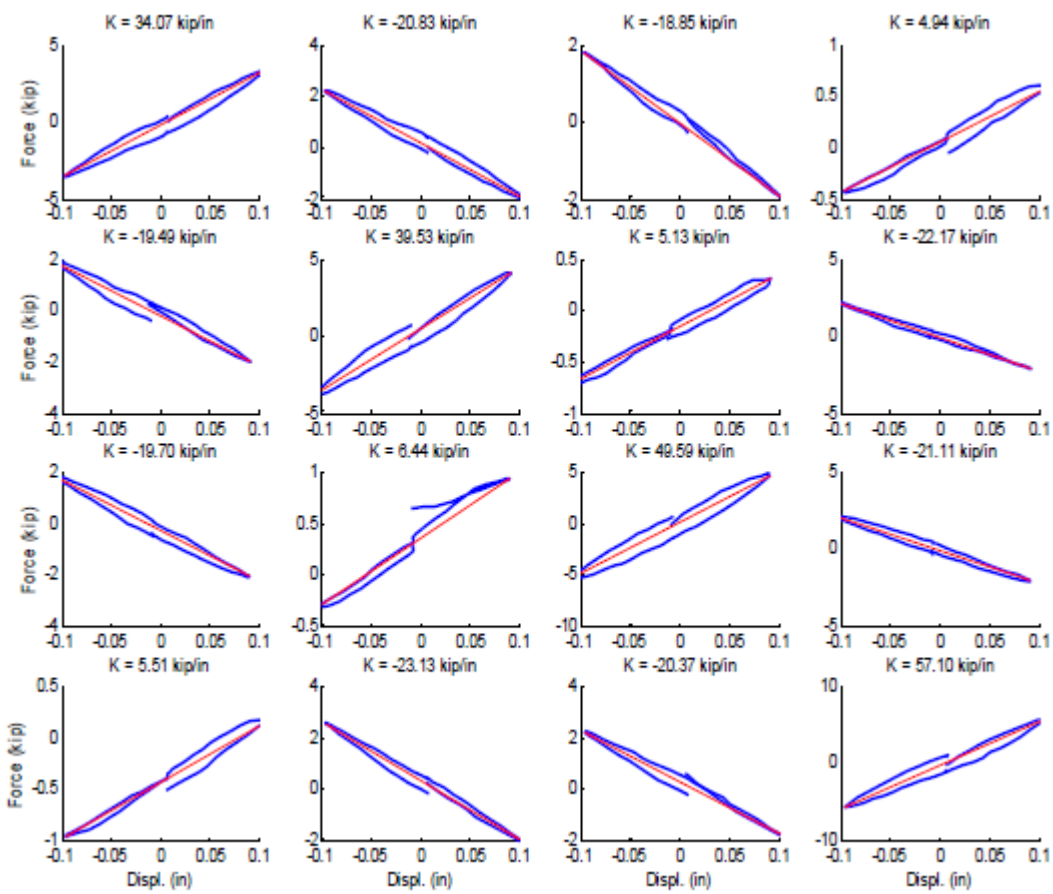

Figure 7. Example Stiffness Matrix Determined via the System ID Test

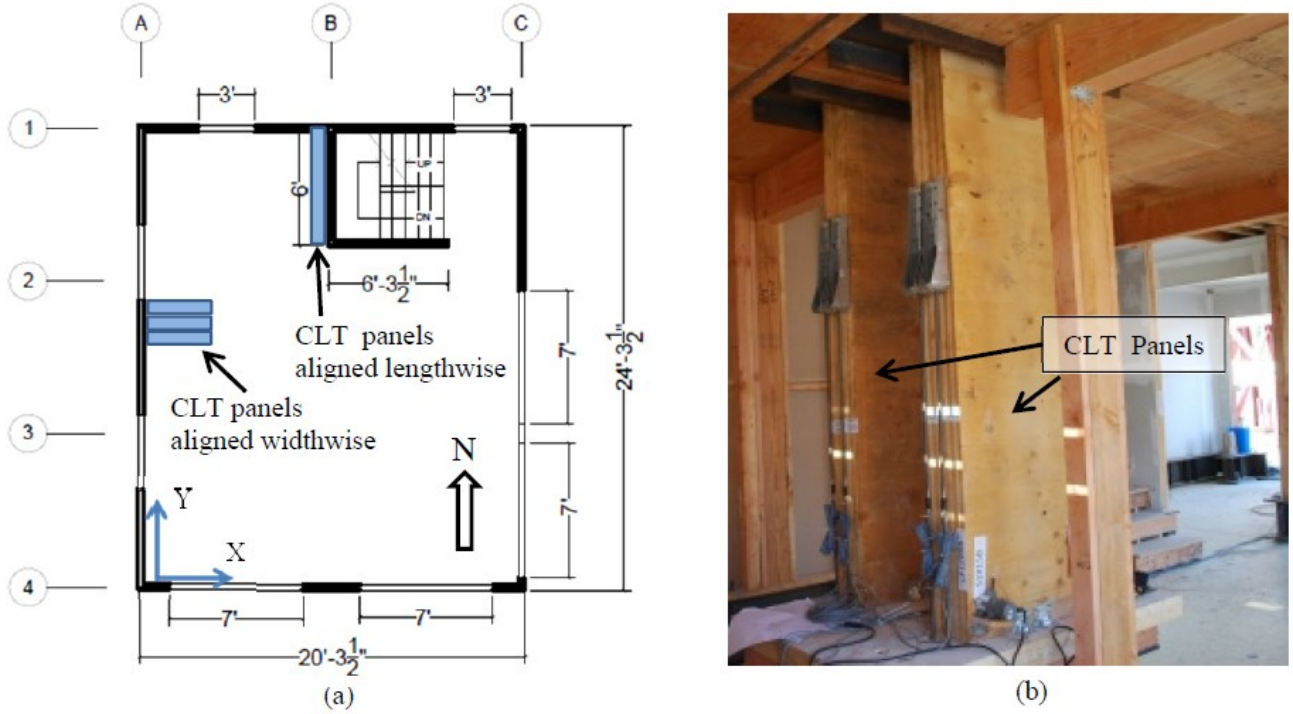

Figure 8. CLT Retrofit (a) Design Layout, and (b) CLT Retrofit for the Four-Story NEES-Soft Building

Hybrid Test Process

The hybrid test began by creating the numerical substructure model on PC1 using Timber3D. The numerical model contained the input ground motion, numerical masses, damping matrix of the full building as well as the hysteretic models of the first story with the retrofits being considered. Next, the Matlab/Simulink controller program was compiled and downloaded to the xPCTarget, which was hosted on PC4. A Matlab program, called *PSD*, was initiated next on PC2. The PSD program served as a coordinator between the hybrid testing controller running on the xPCTarget and the numerical simulation running on PC1. Note that the PSD program utilized the TCP/IP protocol and the Local Area Network (LAN) to communicate to both computers.

During the hybrid test, the equation of motion (Eqn. 1) was solved in PC1 to determine the displacement responses of the full building model including the four controlling DOFs which corresponded to the actuators' connection points (see Figure 6). Since the strong floor in the physical test setup was actually the first-floor diaphragm in the numerical model, the displacements relative to the first-floor diaphragm at the four controlling DOFs were computed as the actuators' commands. For each time step of the ground motion, the four actuators' commands were sent to the xPCTarget via the PSD program hosted in PC2. A ramp/hold loading pattern was programmed in the hybrid testing controller running on the xPCTarget to slowly move the four actuators to the target displacements using a designated ramp time of 1 second. Once the four actuators reached the target displacements, the restoring forces were measured and fed back to the numerical model hosted in PC1, again via the PSD program in PC2. The restoring forces were used to update the numerical model and to determine the displacement commands for the next time step. A modified implicit Newmark-β integrator that does not impose iterative displacements on the experimental substructure was used to solve the equation of motion. The process was repeated for the subsequent time steps until the end of the ground motions.

Select Results for CLT Retrofit Test

Six different retrofit techniques were evaluated via hybrid testing from June to October of 2013 which included: (1) a cross-laminated timber (CLT) rocking panel system, (2) distributed knee-brace (DKB) system, (3) cantilever column (CC), also known as inverted moment frame (IMF), (4) viscous fluid damper (VFD), (5) shape memory alloy (SMA) devices, and (6) steel moment frame (SMF). A total of 24 hybrid tests were performed. The complete details and results will be made available in a report archived on the NEES website (www.nees.org) in late 2014. The results for one of the CLT retrofit tests are discussed in the following section.

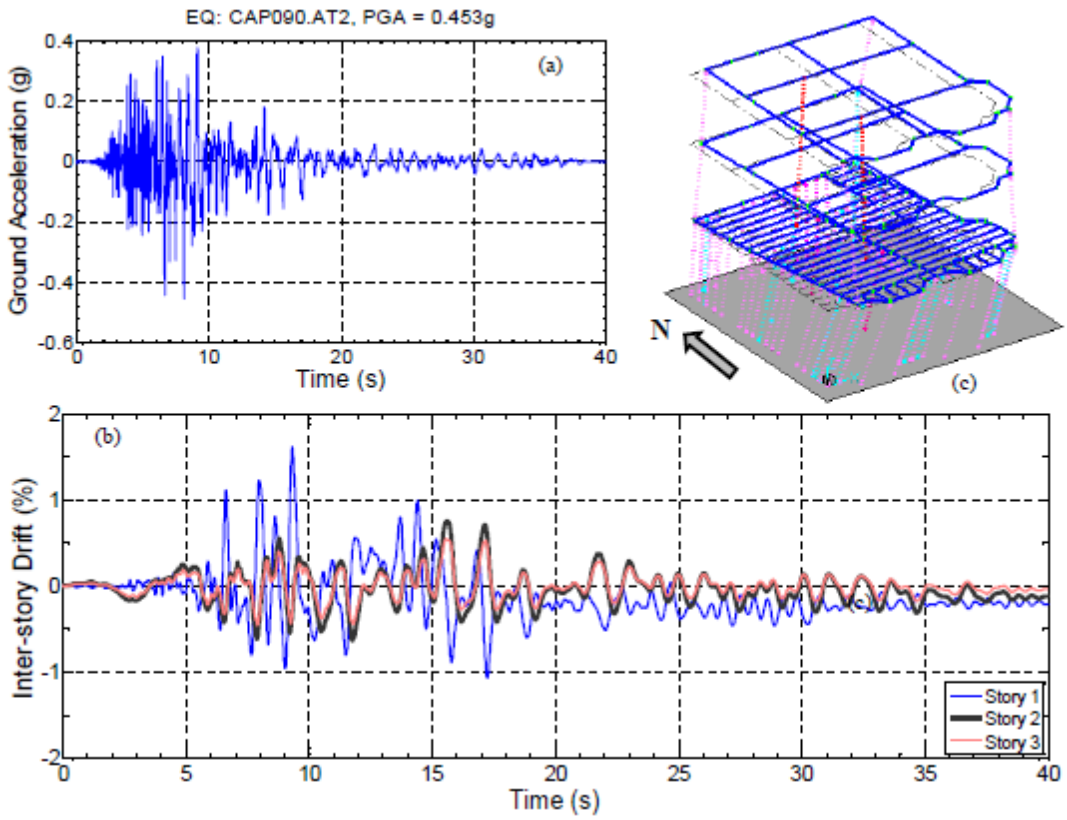

Figure 9. CLT Retrofit Test NSP1CLT02; (a) Input Ground Motion, 1989 Loma Prieta (Capitola, CAP090) Ground Motion Scaled to DBE Level; (b) Inter-Story Drift Time Histories at the South-West Corner of the Test Building; (c) Building Deformed Shape at Peak Drift

CLT Retrofit

The cross-laminated timber (CLT) rocking panel retrofit design followed the FEMA P807 guidelines (ATC 2012) and was implemented only in the ground floor (i.e. the numerical soft-story). The retrofit consisted of three 2-ft long CLT panels in the x-direction set adjacent to each other aligned widthwise, and three 2 ft. CLT panels in the y-direction aligned lengthwise (Figure 8a). An example field installation of a CLT retrofit for the NEES-Soft four-story shake table test building is shown in Figure 8b. That four-story test is described in another WDF article in this issue but illustrates the installation as well for this article since in this hybrid test, the rocking CLT panels were modeled numerically using the results of a wall-level rocking CLT test.

Figure 9 shows the results of one of the CLT retrofit hybrid tests (ID: NSP1CLT02). In this test, the building was subjected to the 1989 Loma Prieta (Capitola) ground motion (PEER Strong Motion Record ID: CAP090) scaled to the Design Basis Earthquake (DBE) level for a representative location in San Francisco Bay Area (scaled PGA=0.453g). The scaled earthquake was applied as a uniaxial ground motion parallel to the transverse direction (x-axis) of the building. As can be seen from Figure 9b, the maximum drift occurred in the first story reaching about 1.6% inter-story drift. The corresponding deformed shape of the three-story building when the first-story drift reached the peak is shown in Figure 9c. As expected, the drift along the garage door openings (Line 4) was larger than the drift along the back wall (Line 1). While noticeable torsional response was observed in the ground floor, the building did not collapse, thus achieving the collapse prevention retrofit objective.

Summary and Conclusion

A series of pseudo-dynamic (slow) hybrid tests were conducted at the NEES facility at the University of Buffalo using a new dynamic analysis package for light-frame wood buildings, called Timber3D, and a Matlab/Simulink controller algorithm developed specifically for slow hybrid tests. In this hybrid simulation, the first story was numerically analyzed with different retrofits while the upper two stories were constructed and physically tested. Six different retrofit techniques were evaluated. While the complete hybrid test results are still being analyzed and interpreted, the overall results show that the retrofit designs

following the FEMA P807 guidelines are effective at reducing the collapse risk of soft-story buildings in San Francisco Bay Area. In addition, the results also show that hybrid testing is a cost-effective alternative to shake table testing for evaluating the effectiveness of different retrofit techniques without the need to physically reconstruct the first story multiple times. The NEES-Soft hybrid test report and results will be available on the NEES website (www.nees.org) in 2014.

Acknowledgements

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Design of a Four-Story Cross Laminated Timber Building in Northern Italy

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ABSTRACT

Cross laminated timber structures are nowadays increasingly used in Italy, where a great number of low- to medium-rise buildings have been constructed mostly for residential purposes. However, in spite of this diffusion, there are still a lot of unresolved issues that design engineers have to face. This applies especially to seismic design, where several topics are still under discussion and are not yet completely covered by the Italian and European Building Code. This paper provides some answers to these issues, based on the actual design of a four-storey building in northern Italy.

Introduction

Cross Laminated Timber (CLT) structures are becoming more and more popular in Italy over the last years, especially for the construction of low- to medium-rise residential buildings, and particularly after 2011 when regulations were changed to remove limitations on the maximum number of storeys for timber buildings in earthquake-prone regions. CLT buildings offer substantial advantages over more traditional structural systems like masonry or reinforced concrete, such as: sustainability, speed of construction, energy efficiency, and above all seismic safety. Furthermore, the transportation cost of the CLT panels from the factories of the main European producers, which are located in Austria and Germany, is relatively low in northern Italy, thus leading to construction costs comparable to those of traditional masonry and reinforced concrete construction but with a better overall performance.

Despite the increasing popularity of CLT structures, there are still no design provisions in Europe specifically written for this kind of structural system. Design engineers often face difficulties in conducting the structural design of a CLT building, in particular the design against lateral forces (wind or seismic), as there are no adequate references and design methods to justify certain design choices made in order to meet the safety requirements prescribed by the current building codes. In addition, the numerical modeling of CLT structures is quite challenging because the lateral and gravity load resisting system is made of two-dimensional components (wall and floor panels) interconnected with various types of flexible connections. Thus, in order to provide a realistic schematization of the actual structure, the design engineer has to find a simplified, yet acceptable, approach to include the fundamental modes of deformation that contribute most to the response of CLT structures under lateral loads.

Description of CLT Buildings

CLT structures consist of wall and floor panels that are manufactured by bonding together a minimum of three laminations with perpendicular directions of the grains of each layer in order to acquire a two-dimensional structural component that provides strength and stiffness for in-plane as well as out-of-plane loads along both the spatial dimensions.

The erection process follows a platform-type of construction with walls of height equal to the inter-story height. Each wall assembly can be either made of a single CLT panel, provided that the length does not exceed the maximum transportable length of typically 16 m, or may be composed of a number of panels with a typical length of 2.5 m that renders the transportation more convenient and economical. In this case, the vertical step joints between wall panels are usually realized with the interposition of a wood-based multi-layer panel (e.g. cross-bonded laminated veneer lumber) that can be inserted in grooves cut inside the wall or on one of its faces, or alternatively with a half-lap joint. Either self-tapping screws with a diameter varying from 6 to 10 mm or nails of 3 mm diameter and spacing depending on the seismic

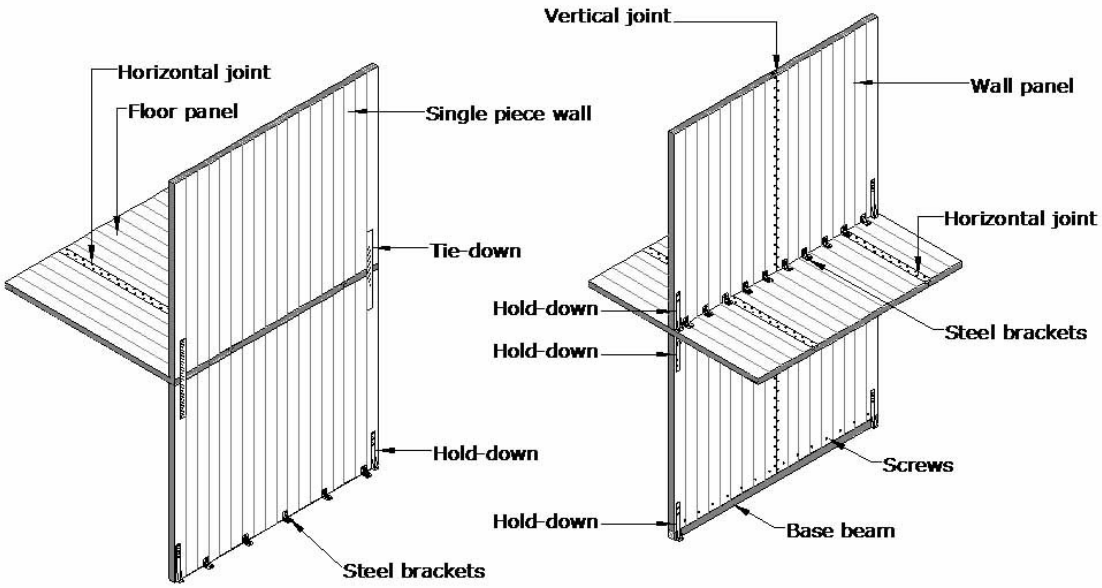

Figure 1. Walls and Floors in Platform-Type CLT Buildings (after Follesa et al. 2011)

loads are used for the vertical joints. The CLT walls exhibit high in-plane stiffness and strength and are connected to the foundation and to the floor panels with various mechanical fasteners (hold-down anchors, steel brackets, anchoring bolts, nails and screws) that restrain the wall against uplift and sliding. Uplift-restrain connections are typically placed at wall ends and at opening ends, while sliding-restrain connections are uniformly distributed along the wall length. Horizontal diaphragms are made of CLT timber panels connected through horizontal joints of the same type of the vertical step joints made with mechanical fasteners (screws or nails). The floor panels bear on the wall panels and are connected to them usually with self-tapping screws. The upper walls bear on the floor panels, and are connected to the lower walls using metal connectors similar to those used for the wall-foundation connection. Figure 1 illustrates the typical connections between walls of a lower and an upper storey to the floor diaphragm.

Design Recommendations for CLT Buildings in Seismic Regions

Research conducted over the last years has demonstrated that buildings with CLT walls composed of panels with a maximum length of 2.5 m, if designed to satisfy certain capacity-based criteria, have a greater level of ductility than buildings made of continuous walls and, therefore, a greater dissipation capacity of earthquake-induced energy (Ceccotti *et al.* 2007, Ceccotti and Follesa 2006). Moreover, Popovski *et al.* (2010) have shown that walls composed of narrow panels have lower stiffness and greater displacement capacity compared to a similar wall made of a single panel. Based on these observations, the seismic force modification factor *q*, used in the Eurocode to reduce the elastic acceleration response spectrum, can have a value as high as 3 for walls with panels that do not exceed 2.5 m in length. In the current version of Section 8 of Eurocode 8 (EC8 2005) there is no provision for CLT buildings. However, for *Glued wall panels with glued diaphragms, connected with nails and bolts*, which is the type of structure more similar as description to the CLT system, the upper limit of the behavior factor *q* is 2. Therefore, for compliance with the current building codes, this is the value currently used when designing CLT buildings.

The use of a seismic force modification factor implies the proper detailing of the ductile connections that will be dedicated to the dissipation of the energy induced by seismic loads. Moreover, following a capacity-based approach, non-ductile or non-dissipative connections as well as timber members should be designed with an overstrength and an overdesign factor to ensure that yielding in these areas will not occur prior to yielding of the dissipative connections.

Connections that should be designed for energy dissipation are:

1. nailed connections between wall and angle brackets, which are typically placed at a constant spacing at the wall-floor and wall-foundation joints ;
2. nailed connections between wall and hold-down anchors, typically placed at the wall ends; and
3. vertical step joints between wall panels of the same wall assembly.

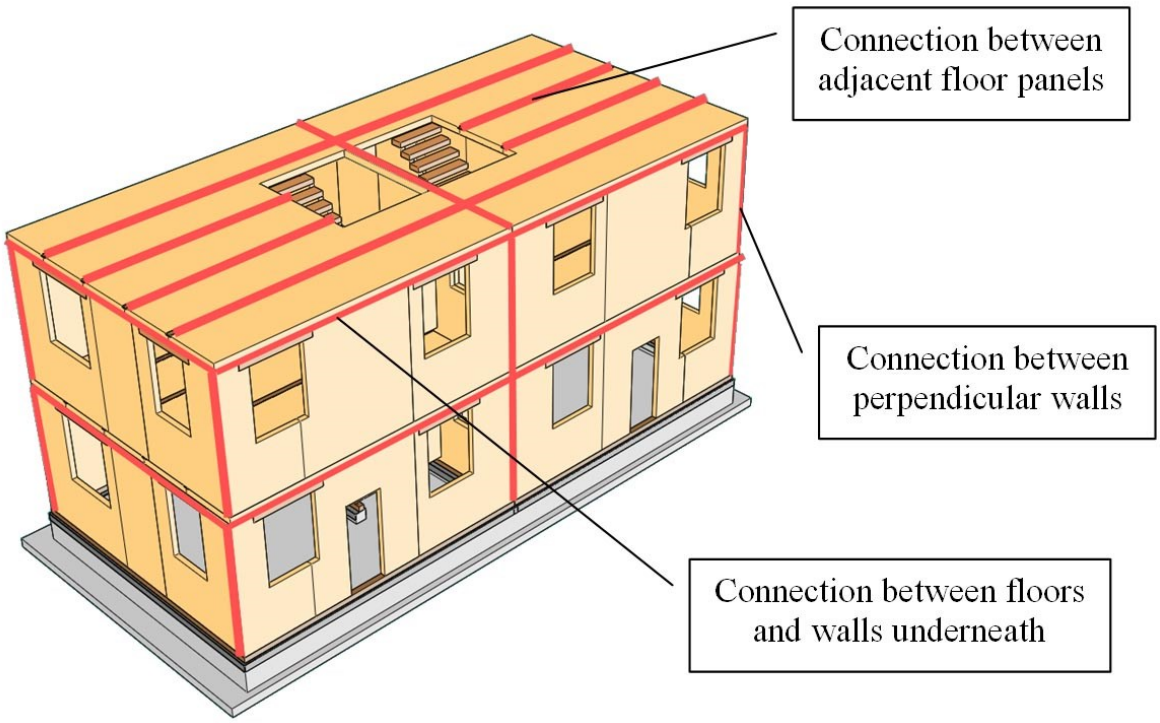

Figure 2. Connections to be Designed with Overstrength in Order to Fulfill the Capacity Design Criteria in CLT Buildings (Highlighted in Red)

Connections that should be designed to remain elastic, shown schematically in Figure 2, are:

1. connections between angle brackets and floor panels or foundation;
2. connections between hold-down anchors and floor panels or foundation;
3. horizontal step joints between floor panels, since it is important to achieve a high in-plane stiffness of the floor diaphragm and minimize the relative slip between the panels;
4. joints between floor diaphragms and walls underneath, typically realized with screws between 6 and 10 mm in diameter or nails of 3 mm in diameter and spacing dictated by the seismic load; and
5. joints between perpendicular walls, particularly at the building corners, so that the stability of the walls themselves and of the structural box is always guaranteed.

Timber and steel members that should be designed to remain elastic are:

1. All CLT panels, used for walls and floors;
2. The metal parts of all angle-brackets and hold-downs.

The overstrength factor applicable for timber connections should not be less than 1.3 and 1.6, for steel-to-timber and timber-to-timber connections respectively, according to a number of research studies (Jorissen and Fragiacomo 2011, Fragiacomo *et al.* 2011, Follesa *et al.* 2011). Since it may be impractical to use two different values of the overstrength factor, the greater value (1.6) can be conservatively used and was adopted in this study for metal connectors, screw joints, and CLT panels.

Apart from the capacity-based design of non-dissipative connections, the same approach has to be implemented for the non-ductile failure modes of the dissipative connections. For instance, the number of nails in a hold-down anchor should be selected so that the total characteristic resistance of the steel-to-timber connection, including the overstrength factor, is less than the characteristic resistance of the metal plate or the anchor bolt in tension, which has much lower displacement and energy dissipation capacity.

Numerical Modeling of CLT Buildings for Linear Analysis

The numerical modeling of CLT buildings for linear analysis can be conducted with any general finite element software package following the procedure described below. Three types of elements are utilized to represent the

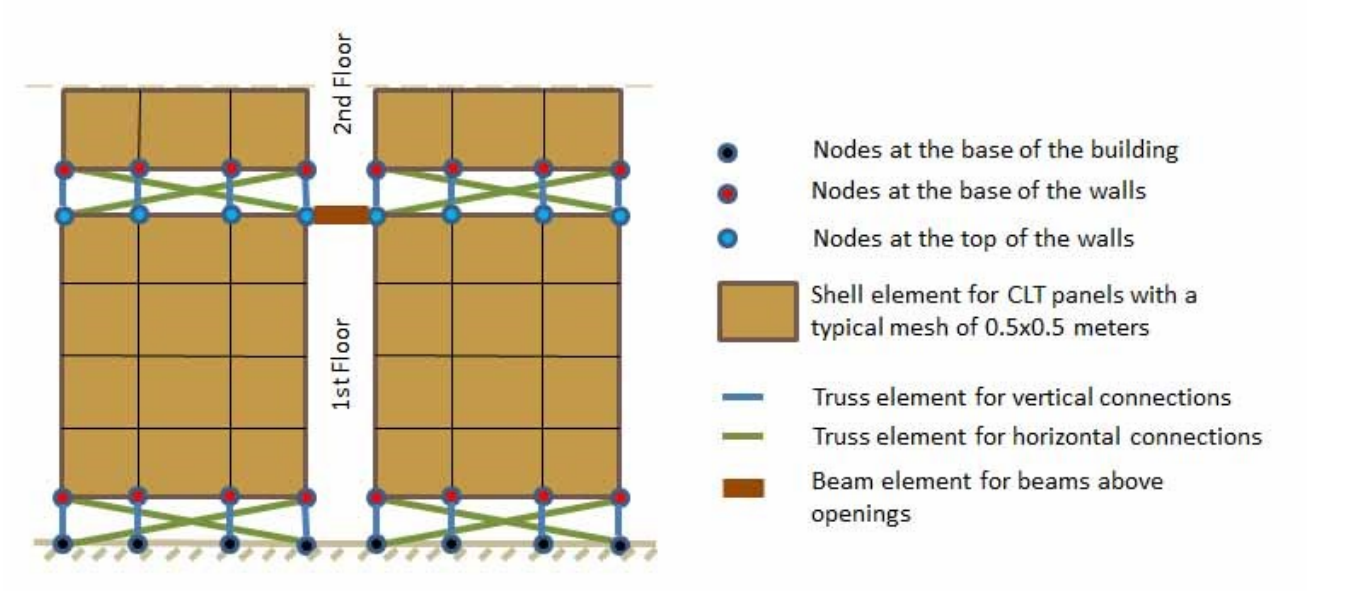

Figure 3. Typical Wall Schematization

structural components, namely: shell elements for the wall panels, truss elements for the panel-to-panel connections and beam elements for the lintels connecting walls above openings. The model described below is based on some simplified assumptions:

- floor diaphragms are assumed to be in-plane rigid while their out-of-plane stiffness is not considered;
- the connection between perpendicular walls is assumed to be rigid;
- the connection between floors and supporting walls is assumed to be rigid; and
- hold-down connectors are not explicitly modeled.

Shell elements with membrane and bending stiffness are used for the CLT wall panels with a typical mesh of 0.5x0.5 meters. These elements are defined with the same length, height and thickness as the associated CLT wall panels. Orthotropic material properties are defined based on the orthotropic properties of the wood class which the boards of the CLT walls are made of and the number of layers and their associated thickness and grain direction. The equivalent modulus of elasticity along the two principle directions is calculated as suggested in Blass and Fellmoser (2004) that is based on the composite theory. The shear modulus in the plane of the wall is typically equal to the shear modulus of the boards or can be reduced by 10% if no edge bonding between the boards of each layer is applied (Fragiacomo *et al.* 2011).

A pair of horizontal cross truss elements is used to connect each wall to the foundation as well as to the wall of the upper floor where applicable. The section and material properties of the trusses are computed based on the horizontal stiffness of the connections used to transfer the shear force from the wall to the floor diaphragm below. Typically, these connections consist of angle brackets or a pair of inclined screws at a certain spacing. Connections between walls and upper diaphragms are considered as rigid, thus are not simulated, since they are typically over-sized, namely designed for the overstrength of the dissipative connectors in order to satisfy capacity design requirements, but also because floor diaphragms are not explicitly modelled but just schematized using a kinematic constraint of rigid floor.

Vertical truss elements are used to simulate the deformability of the floor diaphragms along their thickness, namely perpendicular to their plane, as the walls bear on the floor panels. Thus, the modulus of elasticity perpendicular to grain is selected for the isotropic material properties of these vertical truss elements. Since the shell elements are meshed with a grid of 0.5 meters length, the cross sectional area of the vertical trusses is equal to 0.5 meters times the thickness of the wall above. At the foundation level, the modulus of elasticity of concrete is used for the isotropic material properties. Forces in the vertical truss elements of a wall are then utilized to calculate the tensile forces for the design of the hold-downs, typically installed at each end of the wall to resist overturning moments from horizontal seismic loads. It should be noted that although hold-down anchors play a major role in the actual lateral stiffness and strength of

the wall, they are not explicitly simulated in the linear numerical model, mainly due to the nonlinear nature of their response that exhibits markedly different stiffness in compression (where there is contact between wall and floor panels) and in tension (where only the hold-downs resist).

Figure 3 illustrates a typical schematization of a pair of CLT wall panels and the connections at the base of the building as well as the connections with the upper floor walls. With this representation, the in-plane shear forces transmitted from the walls to the walls underneath can be directly obtained from the axial forces of the horizontal truss elements, while the uplifting vertical forces are obtained from the tensile forces in the vertical truss elements. A rigid diaphragm constraint is used to constrain all nodes at the same level in Figure 3. It should be noticed that a separate constraint is used to constraint the nodes at the bottom of the wall and at the top of the wall underneath.

Design Procedure and Numerical Modeling of the CLT Building

Description of the Building

The CLT structure presented in this paper is a four-storey residential building that contains six apartments and has plan dimensions of about 19 x 12 m and average height of 13 m. The building is located at Arsago Serpio at Varese in northern Italy and is being constructed by *Montagnoli Evio srl* at the time of writing of this paper. Figure 4 illustrates two renderings of the structural components of the building and two external views of its current state.

Wall diaphragms are made of *XL 5-layer 130 mm* panels for the first two stories and *XL 5-layer 95 mm* panels for the two upper stories, where *XL* is used to denote the use of long panels, as opposed to the use of narrow panels with vertical joints. The floor diaphragms are constructed with *5-layer 144 mm* panels and the roof diaphragm with *3-layer 66 mm* panels.

Gravity Loads and Seismic Weight

Gravity loads for the seismic combination were estimated based on the structural and non-structural elements. The dead loads G of external and internal walls are 1.85 kPa and 1.91 kPa, respectively, for the first two stories and 1.68 kPa and 1.74 kPa for the two upper stories. Dead loads G of the floor and roof diaphragm are 2.45 kPa and 1.03 kPa, respectively, where the roof loads refer to the inclined area. The live loads Q for the floors are 2.00 kPa for residential use and 4.00 kPa for the

Figure 4. Rendering of the structural components of the CLT building (left) and external views of the current state of the building (right)

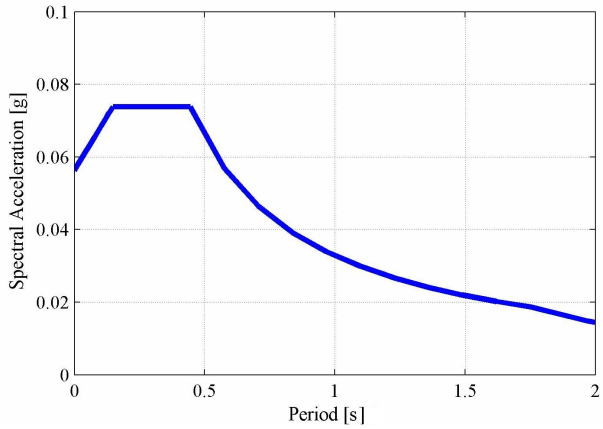

Figure 5. Design Response Spectrum for Arsago Serpio at Varese for 5% Damping

balconies while for the roof diaphragm no accidental load is considered for the seismic combination. Based on these gravity loads, Table 1 lists the total dead and live loads as well as the seismic weight of each floor of the building. The total seismic weight is W = 3990 kN.

Design Spectrum

The design response spectrum for Arsago Serpio at Varese, illustrated in Figure 5, for a 10% probability of exceedance in 50 years was calculated based on the Italian National Building Code (Norme Tecniche per le Costruzioni – NTC 2008) with the following parameters:

- nominal life of the structure equal to 50 years;
- design ground acceleration a_g = 0.038 g;
- soil factor S = 1.5 for ground type C;
- amplification factor F_0 = 2.62
- lower limit of the period of constant spectral acceleration branch T_B = 0.15 s;
- upper limit of the period of constant spectral acceleration branch T_C = 0.45 s;
- value defining the beginning of the constant displacement response range of the spectrum T_D = 1.75 s;
- seismic force modification factor q = 2.

Numerical Model of the Four-Story CLT Building

The three-dimensional numerical model of the four-story building was implemented in the widespread software package for structural analysis SAP2000 (CSI 2007), while a pre- and post-processing software specifically developed by Tecnisoft (Modest-Ver.8.1 2013) was used to aid in the implementation. Figure 6 illustrates the numerical model of the building extracted from the pre-processing software.

The properties of the horizontal truss elements were defined based on the results of a preliminary analysis and design that was conducted using a simplified model developed in Matlab (Mathworks 2009), a programming language for technical computing. This simplified model is based on the so-called *pancake modeling*, introduced by Folz and Filiatrault (2004), that considers floor diaphragms to be rigid bodies that translate and rotate in the horizontal plane. Out of plane rotations and deformations of the diaphragms are not considered, thus, wall elements transfer horizontal forces between floors neglecting the overturning response and the associated uplift forces. In other words, the simplified model yields only the shear force transferred by each wall. This modeling approach is justified because hold-downs are not explicitly considered in the final finite-element model, so no design for uplift forces is needed in the preliminary stage. The horizontal stiffness of each wall is proportional to its length and although the user can define different stiffness per length for each wall, it is common practice that the same value is used for all wall elements at this preliminary stage. The actual value of the distributed stiffness is not important since only the lateral force method of analysis is conducted for the preliminary design and the fundamental period is calculated from the code equation provided by Eurocode 8 for structures with shear walls. The estimated period of the structure in this case is:

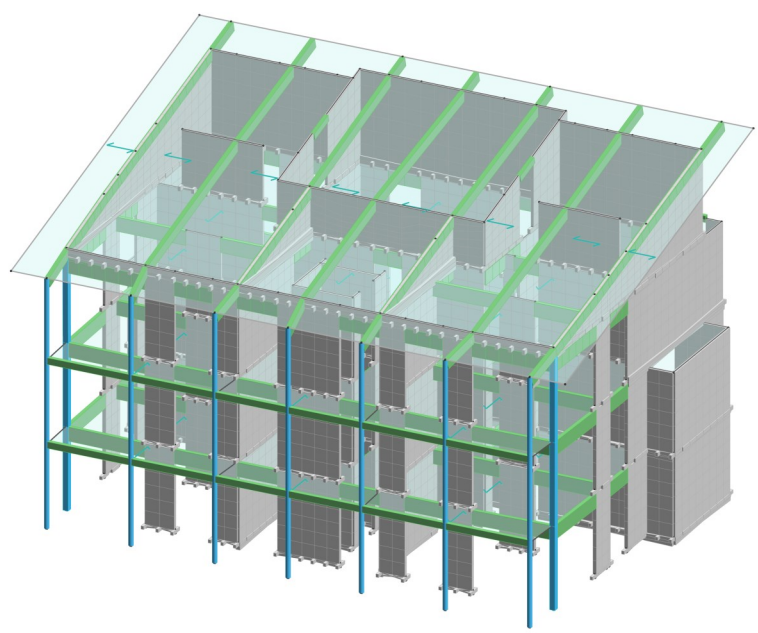

Figure 6. Numerical Model of the Four-Story CLT Building

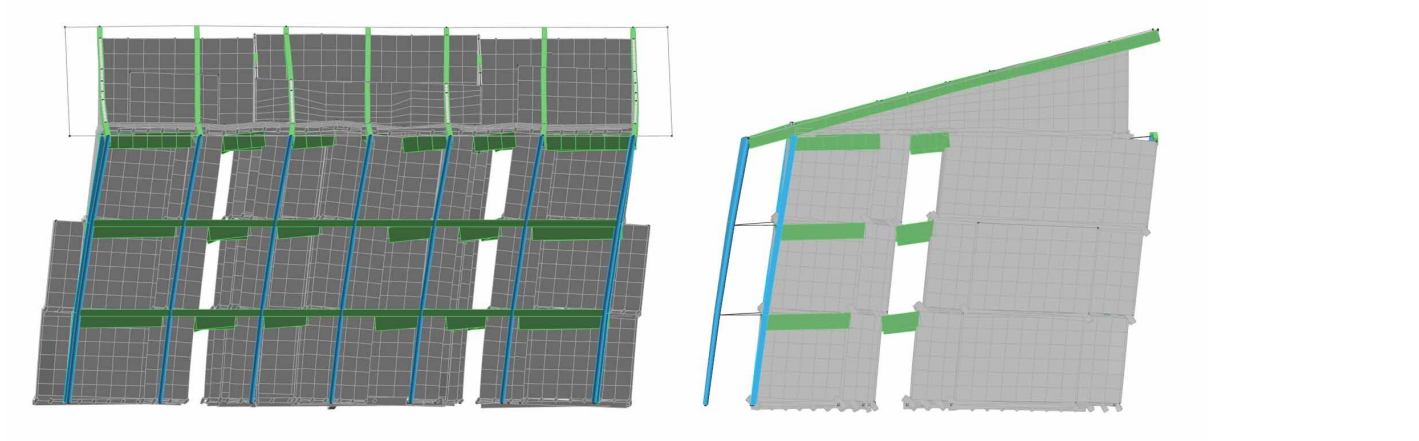

Figure 7. First (Left) and Second (Right) Mode Shapes of the Building

$$T_1 = C_T \cdot H^{3/4} = 0.05 \cdot 13^{3/4} = 0.34\,\text{sec} \quad (1)$$

where C_T may be assumed equal to 0.05, H is the height of the building in meters, and T_1 is measured in seconds.

Based on the results of the preliminary analysis and design, the numerical model was built incorporating the actual stiffness contribution of the angle brackets prescribed at the preliminary stage in the material properties of the horizontal springs of each wall assembly. This model was then used to perform a modal response spectrum analysis of the structure, although for this building that meets the regularity criteria in plan and elevation the lateral force method of analysis could be alternatively applied according to Eurocode 8.

Table 2 lists the fundamental periods and the associated mass participation factors for each mode shape of the structure and Figure 7 illustrates the first two mode shapes. The first mode shape, with a period of 0.31 sec that is similar to the 0.34 sec computed with the code equation, is related to translation along the long direction of the building. Although this may not seem logical, it is justified because along this direction there are a lot of window and door openings and the lateral stiffness of the building is lower compared to the stiffness along the short direction. The second mode shape, with a period of 0.29 sec, is related to translation along the short direction of the building.

Analysis and Design Results

The modal response spectrum analysis was conducted for 16 load combinations accounting for bidirectional and torsional effects. The bidirectional effects were considered by applying 100% of the design spectrum in one direction and 30% of it in the perpendicular direction while accidental eccentricity was introduced by translating the floor masses by 5% of the building's length in each direction. Table 3 lists the shear forces at each story along the two orthogonal directions of the building. The results from the modal response spectrum analyses for each wall are summarized graphically in the following figures. Figure 8 illustrates the maximum shear force per length, Figure 9 shows the maximum uplift force while Figure 10 displays the maximum axial compressive force per length for the seismic combinations.

The shear forces shown in Figure 8 were used to calculate the required number of angle brackets at the base of the walls and the required spacing of the screws used for the floor-to-wall connections at the top of the walls. Two types of angle brackets were used in the design of the building, which are manufactured by Rotho Blaas (2012): the WVS90110 that was used for the shear-transferring

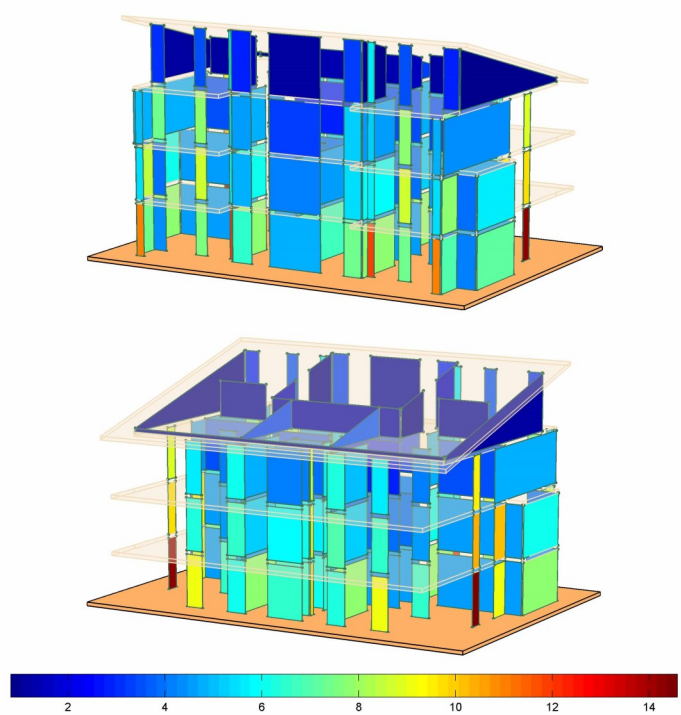

Figure 8. Shear Force per Unit of Length for the Different Walls in kN/m

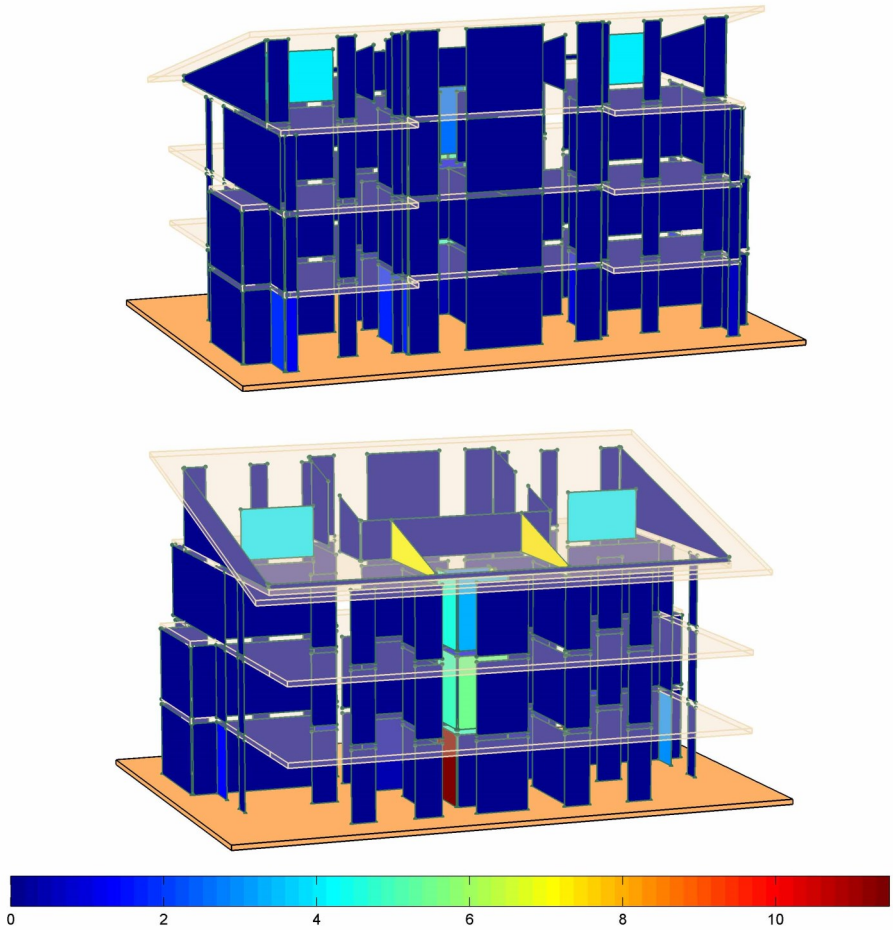

Figure 9. Uplift Force for Each Wall in kN

connections of the 1st story, and the WB90 that was used for the connections of the upper stories. Table 4 summarizes the main properties of the angle brackets. The design strength was taken equal to the characteristic strength of the connection reduced by 20% to account for strength degradation under cyclic loads, as prescribed by NTC (2008). Self-tapping screws f8x300 and f8x220 were prescribed for the connection of the floor and the roof panels, respectively, to the supporting wall panels. The design strength per connector, equal to 2.77 kN and 2.71 kN, respectively, was taken equal to the characteristic strength divided by a safety factor equal to 1.5, according to NTC (2008). To satisfy the capacity design criteria, the shear force was multiplied by an overstrength factor of 1.6 and by an overdesign factor that depends on the ratio between the shear strength, provided by the angle brackets, and the design shear yielded from the analysis. Self-tapping screws f8x220, with a design strength of 2.71 kN, were also prescribed for the connection between perpendicular walls. This type of connection was designed for a force equal to the maximum shear force per unit length between the two walls multiplied by the length of the connection and further multiplied by the overstrength (1.6) and the overdesign factor.

The uplift forces shown in Figure 9 were used to calculate the required number of tie-down connectors at the ends of each wall. One type of hold-down, the WHT340, manufactured by Rotho Blaas (2012), was used for the first story and a pair of hold-downs were used for the internal walls of the upper stories. In addition, a steel strap with dimensions of 100x1000x1.5 mm was used to restrain external walls of the upper stories against uplift. Table 5 summarizes the main properties of the hold-down and the steel strap. Again, the design strength was calculated as 80% of the characteristic strength of the connection, similarly to the angle brackets.

Finally, each wall panel was verified against shear and against axial force including axial instability, considering pinned connections of the walls to the floor above and below for out-of-plane bending. Figure 11 illustrates a photo taken during the construction of the building that shows the connections at the base of an internal wall at an upper story.

Conclusions

This paper provided design recommendations and a modeling approach for the design of Cross Laminated Timber (CLT) buildings in seismic regions. In addition, the seismic analysis and design procedure of a four-story CLT building located in northern Italy was briefly presented and discussed. The results from the modal response spectrum analysis were graphically illustrated and the characteristics of the main type of connections used to connect the structural components were provided. The material presented can be useful to structural engineers designing CLT buildings as it can provide a guidance for ensuring the required global level of ductility of the structure with energy dissipation concentrated in the desired structural components.

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