Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1662

The NEESWood Project in Review

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ABSTRACT

The NSF-funded NEESWood Project was a four-year, five-university research project whose objective was to develop a performance-based seismic design philosophy for mid-rise woodframe construction. The project began in 2005 and, by the end of 2006, the benchmark testing of a two-story townhouse structure had taken place at the University at Buffalo's SEESL shake table facility. This test included several shear walls designed with fluid dampers. A series of sub-assembly tests on shear walls with toggle-braced damping systems and half-scale base isolation tests followed. From 2005-2008, non-linear time history analysis software was developed that was based on existing concepts and software, and improved upon as part of the NEESWood effort. This software package, called SAPWood, had the dual purpose of being a research and design tool for later testing within the project as well as being available for use by practitioners. It was extended to include six degrees-of-freedom at each story and tri-axial excitation. In parallel, a detailed two-dimensional numerical model that enables the static and dynamic response analysis of two-dimensional light-frame wood buildings under unidirectional horizontal and vertical earthquake shaking was developed. From 2006-2008, the Direct Displacement Design (DDD) approach was extended to multi-story woodframe buildings, which is a key outcome of the project. The DDD approach was also extended for application to woodframe buildings with sliding seismic isolation systems. From 2007-2009 the effect of design code changes on societal risk were investigated within the project by using Los Angeles, CA as a test bed. Finally, in order to validate the DDD approach, the world's largest shake table test was conducted at Japan's E-defense laboratory in collaboration with numerous researchers and industry participants from the U.S., Japan, and Canada. The 1350 square meter, seven-story building was designed using the DDD concept, the development of which was completed in 2008. Shear transfer and continuous steel rod holdowns were designed based on probabilistic concepts using SAPWood. The building, termed the Capstone Building, was subjected to three levels of seismic intensity including a design-basis earthquake (DBE) and a maximum credible earthquake (MCE). This paper provides an overview of the entire NEESWood Project along with a discussion of key

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contributions to the seismic design of mid-rise (and low-rise) woodframe buildings and a candid assessment of the pros and cons of increasing the height of woodframe buildings.

Introduction

The NEESWood Project, funded through the U.S. National Science Foundation's George E. Brown, Jr Network for Earthquake Engineering Simulation, sought to safely increase the height of wood frame construction in seismic zones within the U.S. and around the world. The overall project consisted of 10 major tasks that were completed according to the schedule shown in Figure 1.

Task	Year 1				Year 2				Year 3				Year 4			
1. Numerical Analysis Tools (SAPWOOD)		1.1														
2. Seismic Protection Systems		2.1		2.2							2.3					
3. PBD Philosophy		3.1														
4. Testing			4.1				4.3							4.2		
5. Societal Risk / Decision Making			5.1													
6. Payload Projects			6.1								6.1				6.1	
7. Professional Advisory Committee (PAC)		7.1				7.1				7.1				7.1		
8. International Cooperation													8.1			
9. Outreach/Education	9.1															
10. Annual NEES Awardee Meetings				10.1				10.1				10.1				10.1

Fig 1. Schedule for ten major tasks within the NEESWood Project

Numerical Analysis Tools

Performance-based seismic design necessitates accurate response computation of woodframe buildings during earthquakes and thus numerical model development was a major part of the NEESWood project. A program for seismic response simulation was developed within the NEESWood project based on the earlier model developed by Folz and Filiatrault (2004). The accuracy of the models was verified through multiple shake table tests of various scales, including a number of low-rise building tests conducted at Colorado State University, isolated shear wall stack dynamic test data from Simpson Strong Tie's laboratory in Stockton, CA (Pei and van de Lindt, 2009), and the NEESWood Benchmark test (Christovasillis et al. (2007). The numerical model enables analysis of woodframe buildings with tri-axial ground motion and the assessment of hold-down forces which are modeled as an anchor element for wood shear walls. The numerical models were integrated into a software program called SAPWood (Seismic Analysis Package for Woodframe Buildings) which released its first version to public in 2007. The latest version of SAPWood (version 2.0) was used within the NEESWood project to design and analyze woodframe assemblies and buildings including the Capstone structure and will likely be released in 2010 once it is fully verified with the Capstone test data gathered in Miki, Japan during the summer of 2009.

SAPWood provides a good overall response, but within the wall and wall line of a building it does not fully capture the behavior of these subassemblies and their components. Therefore, to complement SAPWood, a new two-dimensional numerical model that enables the static and dynamic response analysis of two-dimensional light-frame wood buildings under unidirectional horizontal and vertical earthquake shaking was developed by Christovasilis and Filiatrault

(2009). This 2nd generation numerical model eliminates the limitations of existing 1st generation models while maintaining computational efficiency. The main features of the model include: 1) the formulation of a numerical building model based on nonlinear elements that effectively model the load-deformation characteristics of sheathing-to-framing wood connections and vertical load-transferring devices up to failure; 2) the proper simulation of shear and flexural modes of deformation of shear walls as well as contact/separation between framing members; 3) a co-rotational formulation to solve the equilibrium equations in the deformed configuration accounting for large rotations and displacements associated with rigid body motion, geometric nonlinearity, as well as P-Δ effects; and 4) an automatic mesh generator that maintains simplicity in model preparation and can accommodate various structural configurations. Figure 1 illustrates the deformed shape predicted by this new model for a two-story shear wall tested by Pardoen et al. (2003). The stud and plate uplifts and flexural deformation of the studs are clearly visible. More details on this model can be found in a companion paper of this conference (Christovasilis and Filiatrault 2010).

Seismic Test Program

In 2006, as part of the NEESWood project, Filiatrault et al (2010) conducted full-scale tri-axial tests of a two-story three-bedroom 160m^2 (1800 sq ft) townhouse (Fig. 1a) with an integrated two-car garage utilizing the twin shake tables at the Structural Engineering and Earthquake Simulation Laboratory at the University at Buffalo's NEES Site. This building, which was called the Benchmark structure, was designed in accordance with the 1988 Uniform Building Code and was intended to benchmark the seismic performance of existing buildings in California and other high seismic regions. The benchmark structure performed relatively well by protecting life safety of would-be occupants, but suffered substantial costly damage such as splitting of most of the sill plates (Christovasilis et al. 2007, Filiatrault et al. 2010). Filiatrault et al (2010) were also able to validate earlier observations that nonstructural elements such as gypsum wallboard and exterior stucco significantly increase the strength and stiffness and therefore contribute to the seismic resistance of woodframe buildings.

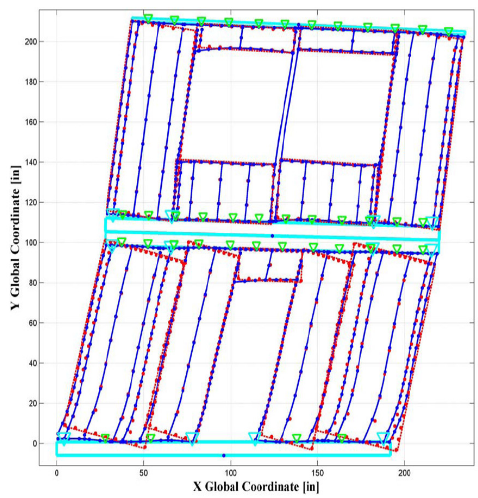


Fig. 2. Predicted Deformed Shape of a Two-Story Shear Wall tested by Pardoen et al. (2003) at Lateral Story Drift of 5% (1st Story) and 6% (2nd Story); Displacements magnified

Phase 2 of the NEESWood Benchmark structure test program involved implementation and evaluation of a seismic damping system with a chevron brace configuration (see Fig. 2b) (Shinde et al. 2007 and 2008b). The testing of such dampers within the NEESWood benchmark structure represents the first application within a full-scale woodframe building. Due to a number of factors, including the inherent flexibility in the connections of wood framing systems, engagement of the dampers was limited during these tests and thus the full effectiveness of the dampers was not realized. Based on what was learned from the Phase 2 testing, a new design for the modular damper walls with a toggle brace configuration was developed and tested within full-scale shearwalls (see Fig. 2c and 2d). The results demonstrated that the performance of the retrofitted walls with the toggle-braced assembly was significantly improved as compared to the conventional walls, partially as a result of increased damper engagement as compared to what was achieved in the benchmark structure testing. In addition, the displacement-based procedure that was developed as part of the general NEESWood project, and to be discussed subsequently, was modified for design of woodframed structures incorporating seismic damping systems (Shinde et al. 2008a).

(a) (b) (c) (d)

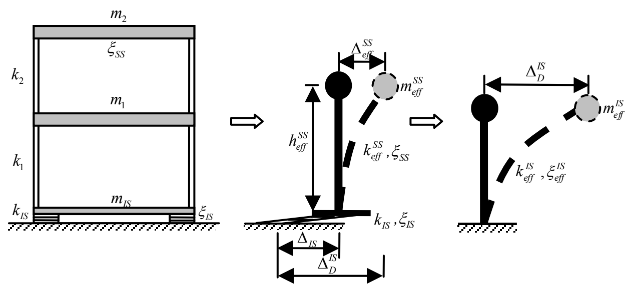
Figure 2 (a) Phase 5 Benchmark Structure (b) Chevron-braced Modular Damper Wall used in Phase 2 Benchmark Tests (c) Retrofitted Test Walls in Toggle-braced Damper Test and (c) Toggle-braced Modular Damper Wall

In 2008, as part of the effort to investigate the potential for application of seismic response modification devices to woodframe buildings, a half-scale base-isolated two-story residential building was tested (van de Lindt et al, 2009) (see Fig. 3a). The isolation system employed friction pendulum system (FPS) bearings and the test results demonstrated the significant benefit of isolating such structures. To facilitate future applications to woodframed buildings, two displacement-based methods for seismic design of light-framed wood structures with sliding isolation systems were developed. The validity of both displacement-based procedures was confirmed using results from nonlinear dynamic response-history analyses and experimental results from the aforementioned shaking table tests. The model of the FPS bearing was integrated into the SAPWood software package (Pei and van de Lindt, 2007) wherein it was combined with a non-linear flexible model of the woodframe superstructure. One of the displacement-based methods involves a conventional direct displacement-based design procedure in which a new approach has been taken to define an equivalent single-degree-of-freedom (SDOF) model for an isolated building structure with the computation of its design inter-story drift based on the relative contribution of the isolation system displacement and the effective displacement of the equivalent SDOF fixed-base superstructure, all without the need for modal analysis (see Fig. 3b). In addition, a simple and quick design procedure based on normalized modal analysis and generation of inter-story drift spectra has been developed. These more practical approaches have been developed with the objective of enabling designers to efficiently evaluate various options before making the final selection of isolation system parameters.

The final shake table test within the NEESwood project was the Capstone Test and focused on testing of a full-scale mid-rise woodframe condominium that was designed using the performance-based seismic design procedure summarized in the next section. The 1350 m^2

(14000 ft^2) building, whose elevation is shown in Figure 4a, represents the largest building ever tested on a shake table and provided key information on the seismic response of a full-scale mid-rise woodframe building. Ultimately this information will facilitate performance-based seismic design of mid-rise wood frame buildings around the world. The two immediate objectives of the Capstone Test were: (1) to provide a general understanding of how mid-rise woodframe buildings will perform in a major earthquake (since there are four and five story woodframe buildings that have already been constructed within the Western U.S.); and (2) provide some level of validation for the performance-based seismic design philosophy developed within the NEESWood project by demonstrating good performance for a high seismic intensity. Fig. 4b shows the building on the shake table and ready for testing. The performance of the building was consistent with the design criteria in that there was no structural damage while undergoing peak inter-story drifts of approximately 2% to 3%.

(a)



(b)

Figure 3 (a) Half-Scale Isolated Test Structure (b) Equivalent SDOF Model of Isolated Structure

(Fig. 4a)

(Fig. 4b)

Fig. 4. The elevation view of the Capstone test specimen (Fig 4a) and an aerial photograph of the Capstone test specimen ready for testing in July 2009 (Fig 4b).

Performance-Based Seismic Design Philosophy

Performance-based seismic design necessitates combining certain prescribed performance expectations with seismic intensity levels. These combinations, while prescribed, are subject to the input of building owners and overall stakeholders; thus any combination, provided it meets current design standards and represents an engineering solution, is acceptable. The NEESWood project team with input from the project advisory committee, defined the following four seismic intensity levels:

Level 1: Earthquake intensity having a 50% chance of being exceeded in 50 years. This corresponds to a 72-year return period.

Level 2: Earthquake intensity having a 10% chance of being exceeded in 50 years. This corresponds to a 475-year return period and *approximately* represents the Design-Basis Earthquake (DBE).

Level 3: Earthquake intensity having a 2% chance of being exceeded in 50 years. This corresponds to a 2500-year return period and represents the Maximum Credible Earthquake (MCE).

Level 4: Optional Near Fault: Un-scaled near-fault ground motions. This is an optional seismic hazard with its use depending on the location of a building with respect to the fault and/or the owner's desired performance expectation.

Each of the seismic intensity levels described above can be combined with a particular performance expectation. Table 1 presents the performance expectations from Christovasillis et al. (2007) which were developed based on the two-story Benchmark tests at the University at Buffalo.

Table 1: Performance Expectations (Derived from Christovasilis et al, 2007)

Performance Expectations	**Corresponding Peak Inter-story Drift (%)**	**Expected Damage to Wood Framing and OSB/Plywood Sheathing**	**Expected Damage to Gypsum Wall Board (GWB)**
Level A	0.1 – <u>1.0%</u>	Minor Splitting and cracking of sill plates (some propagation) Slight sheathing nail withdraw	Slight cracking of GWB Diagonal propagation from door/window openings Partial screw withdraw Cracking at ceiling-to-wall interface
Level B	1.0 - <u>2.0%</u>	Permanent differential movement of adjacent panels Corner sheathing nail pullout Cracking/splitting of sill/top plates	Crushing at corners of GWB Cracking of GWB taped/mud joints
Level C	2.0 - <u>4.0%</u>	Splitting of sill plates equal to anchor bolt diameter Cracking of studs above anchor bolts Possible failure of anchor bolts	Separation of GWB corners in ceiling Buckling of GWB at openings
Level D	4.0 - <u>7.0%</u>	Severe damage across edge nail lines, separation of sheathing Vertical posts uplifted Failure of anchor bolts	Large pieces separated from framing Entire joints separated and dislodged

Each performance level is specified by a probability of non-exceedance (NE) of an inter-story drift limit at a specified level of seismic hazard as shown in Table 2. For example, at seismic intensity level 1 the building should not exceed a median inter-story drift of 1%. At a level 3 seismic intensity the non-exceedance percentile was increased from the 50th percentile (i.e. median) to the 80th percentile because the 4% drift limit was close to what was felt to be the threshold between repairable damage and collapse. Thus it was assumed to be more critical during the design.

As part of the NEESwood project, Pang and Rosowsky (2007) extended the DDD approach to multi-story woodframe buildings. The basic idea of the approach was to account for the change in stiffness distribution that occurs over the story levels as the structure is damaged during the earthquake, thereby significantly reducing the possibility of the formation of a soft story mechanism. In the approach developed by Pang and Rosowsky (2007), initially one defines one or more inter-story drift limits for one or more corresponding seismic hazard levels. Then, stiffness and mass relative to the first story are estimated. Note that the procedure is iterative and therefore a good estimate of these ratios is all that is needed. Using these approximate values a normalized modal analysis is performed on a linear multi-degree-of-freedom system having these properties. Then, inter-story drift spectra are developed for each story and stiffness changed, if desired, to optimize the vertical distribution of maximum inter-story drift. Once the target drift profile and associated story stiffnesses are known, Pang and Rosowsky (2007) propose using a hysteretic backbone wood shear wall database and selecting the walls needed to achieve the stiffness computed earlier. The design can then be verified using the actual stiffness calculated for the selected shear walls. This procedure can be illustrated as the flowchart shown in Figure 5.

Table 2. Capstone structure design expectations

	Seismic hazard (Intensity) Level			
Performance Expectations	**Level 1**	**Level 2**	**Level 3**	**Level 4**
Level A (1%)	**50% NE**			
Level B (2%)		**50% NE**		
Level C (4%)			**80% NE**	
Level D (7%)				**50% NE**

The procedure developed for selection (design) of the wood shear wall system in a multi-story building developed by Pang and Rosowsky (2007) can be repeated for additional performance levels. Then, from the seismic demand on the shear walls, story shears and uplift forces may be computed for design. Their approach was verified using nonlinear time

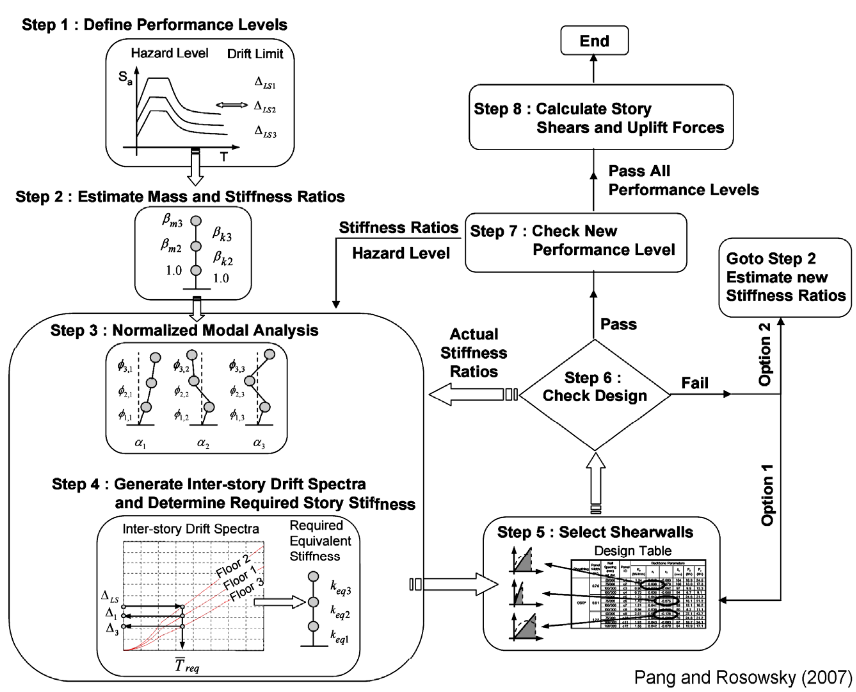


Fig. 5: Revised direct displacement design proposed by Pang and Rosowsky (2007)

history analysis. A simplified (non-iterative) version of their approach was then used to select the shear wall nailing schedule for the Capstone building of the NEESwood Project (Pang et al. 2009).

Loss Modeling

Another NEESWood project task has focused on how seismic loss modeling can be used to help recast seismic performance objectives for woodframe buildings in terms of economic loss, a measure that can be more directly useful to building owners than qualitative performance levels such as "life safety." A new seismic loss model for woodframe buildings was developed, incorporating the latest results from the other portions of the project. It uses SAPWood for the nonlinear dynamic analysis, for example, and fragility curve findings from the benchmark test. For an example individual building category, the model was applied to estimate losses as a function of ground motion intensity and building design. The results were examined to illustrate how they can be used to help define performance objectives and guide design to meet those performance objectives. At the time of this paper, the model is being applied to the entire city of Los Angeles to illustrate how loss modeling can be used to inform specification of loss performance objectives at a regional scale.

Discussion

The NEESWood project sought to provide the technical foundation to enable the performance-based seismic design of woodframe buildings in seismic regions of the U.S. and around the world. The full-scale system level tests within the project provided landmark data for two different types of structures: one being a low-rise structure designed based on an existing building code and the other being a mid-rise structure designed using PBSD. The performance of the mid-rise wood frame building, which was designed for an MCE level earthquake using the DDD procedure developed within the project, was shown to be quite good with no structural damage, thus providing some validation to the DDD procedure. However, the DDD procedure and subsequent sizing of overturning restraint (hold-down rods) and shear transfer mechanisms, is not yet in a form suitable for use by practitioners. All of the components of the DDD procedure calculations are available from the Capstone experimental data set and thus the combination of the procedure and data set represent a valuable contribution to the wood seismic practicing community.

The design of a mid-rise woodframe building using PBSD at the MCE level for the city of Los Angeles, CA resulted in a base shear capacity of about twice that of an IBC-designed building. Of course the building was designed to account for softening throughout the stories at a target inter-story drift level, and thus a direct comparison may not be possible. Regardless, this increase in shear demand translates directly to increased shear transfer forces and uplift and compressive forces at the shear wall end posts. This results in an increase in stud pack size for compressive load during wall racking, increased shear transfer hardware, and an increase in the labor to install these larger and/or additional components. In addition, if standard shear walls are used for a DDD design, a significant increase in fasteners is needed. This also requires additional labor for construction and installation. No detailed calculations have been performed for the cost increase in going from an IBC-design (DBE) to a DDD-design (MCE), but it is estimated be on the order of 15% to 20%. If the cost of land is included, this estimate decreases significantly for many urban locations. There are two major items for further consideration from here:

Item 1: The mid-rise woodframe building designed using the DDD procedure performed in accordance with the response predicted during the design phase. This performance was very good (no structural damage) and may be worth additional up-front cost to a building owner. However, there is clearly a point at which mixing material (e.g. wood and steel) would provide an equally performing but more economical design even with the additional engineering needed. This concept should be explored.

Item 2: The additional cost associated with designing and constructing a PBSD building is an issue that consistently arises. A detailed study that takes into account potential losses over time, time value of money, and up-front engineering and construction costs is needed.

Conclusions

The NEESWood Project has provided clear evidence that PBSD of mid-rise woodframe buildings can result in buildings that provide a desired level of performance for a specified seismic intensity level. However, as the height increases beyond about six stories and high levels of performance (e.g. continued occupancy at the MCE intensity level) are sought, other types of construction may be more economically viable (e.g. inclusion/mixing of other materials with wood and/or inclusion of seismic protection systems).

Acknowledgements

The material presented in this paper is based upon work supported by the National Science Foundation under Grant No. CMMI-0529903 (NEES Research) and CMMI-0402490 (NEES Operations). Any opinions, findings, and conclusions or recommendations expressed in this material are those of the author(s) and do not necessarily reflect the views of the National Science Foundation. Thanks to Weichiang Pang, Doug Allen, Kathryn Pfrefzschner, Hidemaru Shimizu, Izumi Nakamura, Chikahiro Minowa, Hiroshi Isoda, Mikio Koshihara, Kazaki Tachibana, Tomoya Okazaki, Kimberly Cronin, Tim Ellis, Steve Pryor, David Clyne, Simpson Strong Tie, U.S. Forest Product Laboratory, FP Innovations-Forintek Division, Maui Homes U.S.A., B.C. Ministry of Housing and Social Development, Stanley Bostitch, Strocal Inc., Structural Solutions Inc., Louisiana Pacific Corp., Natural Resources Canada, Forestry Innovation Investment, APA-The Engineered Wood Association, American Forest and Paper Association, Howdy, Ainsworth, Calvert Glulam, B&L Wholesale Supply Inc., Buffalo Plastering Inc., Erie Community College, Gambale USA, Georgia Pacific, Hartland Builders, MiTek Industries Inc., National Gypsum Company, NGC Testing Services, National Overhead Door Inc., Niagara Truss & Pallet LLC, Northeast Window and Door, Ridg-U-Rak Inc, and Taylor Devices Inc. Full acknowledgements and contributions can be found in the NEESWood Report series.

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