RESEARCH PROGRAM ON THE SEISMIC RESISTANCE OF CONVENTIONAL WOOD-FRAME CONSTRUCTION

J. H. Rainer[1], X. Lu[2], C. Ni[3], H. Cheng[4], M. Follesa[5] and E. Karacabeyli[6]

ABSTRACT

Canada's wood products research institute, Forintek Canada Corp., in collaboration with Tongji University of Shanghai, China, and the Trees and Timber Institute, Italy, initiated a research program in 2004 on the seismic resistance of conventional wood-frame construction. The objective is to provide data needed for the quantitative assessment of the seismic building code provisions for this type of housing in both Canada and China. The research program is designed to generate complementary data to the experimental and analytical work carried out under the recently completed CUREE program in California, among others.

The program comprises shake table tests of two two-story buildings, cyclic tests of large shear wall configurations, simplified and detailed analytical studies, and application to codes and standards. Tests carried out at the Tongji University 4 m by 4 m shake table subjected two two-story specimens, 6.0 m by 6.0 m in plan, to a progression of 3 seismic motions at amplitudes of 0.1, 0.2, 0.4 and 0.55 g PGA. Both symmetrical and non-symmetrical configurations were tested; capacity spectra and changes in natural frequencies for the symmetric case are presented here.

Introduction

Wood-frame construction is by far the most common structural type in North America for single-family houses and low-rise multifamily dwellings, constituting around 90 % of all residential housing (Fischer et al. 2001). Wood-frame construction is also very common in New Zealand, Australia, Scandinavia, and is gaining acceptance in other parts of the world, including Japan, China and parts of Europe. In North-America wood-frame construction can be built either by conventional or prescriptive rules or by engineered design.

Conventional construction follows rules derived from experience for satisfactory behaviour, augmented by pre-engineered designs, as for example roof trusses and floor construction in terms of span tables. The rules specify, among other parameters, minimum member sizes, size and spacing of nails, and minimum and maximum permissible openings. Limitations are placed on this type of construction by building size, number of stories and type of occupancy, but within these limits no detailed engineering calculations are required. Most

[1]Rainer Dynamics, Surrey BC, V4P3A3, Canada
[2]Professor, State Key Laboratory for Disaster Reduction in Civil Engineering, Tongji University, Shanghai, China
[3]Wood Engineering Scientist, Forintek Canada Corp., Vancouver BC, V6T1W5, Canada
[4]Graduate Student, Dept. of Civil Engineering, Tongji University, Shanghai, China
[5]Trees and Timber Institute, 50019 Sesto Fiorentino, Italy
[6]Manager, Wood Engineering Dept., Forintek Canada Corp., Vancouver BC, V6T1W5, Canada

single-family houses and many multi-family buildings of up to three stories in height are built by conventional rules, except where more stringent requirements are in effect as in regions of high seismic activity or strong winds. The code requirements are prescribed in Section 2320 of the Uniform Building Code (UBC 1997) and Section 2308 of the International Building Code (IBC 2003) for the USA, GB 50005 – 2003 (GB 50005 – 2003) for China, and for Canada in Part 9 of the National Building Code of Canada (NBCC 2005). Since the NBCC does not contain seismic requirements in its Part 9, the guidelines issued by the Canadian Wood Council (CWC 2003) can be employed.

Engineered design proceeds along well-established principles of engineering, and codes and standards specify loadings on the structure and resistances of materials on the basis of limit states design principles. The current objective of most building codes is the ultimate limit state for life safety and the prevention of collapse,. In the USA, the seismic requirements are specified in Section 2307 of IBC (IBC 2003), in China in GBJ 11 – 89 (GBJ 11 – 89, 1994) and for Canada in Part 4 of the National Building Code of Canada (NBCC 2005). Work is also under way on various forms of performance-based design method but this will not be addressed here.

Simple engineering calculations would suggest that the lateral strength of a number of conventional wood-frame buildings would not be adequate for the seismic demand in high seismic areas. Yet with some notable exceptions such buildings have withstood significant ground shaking and did not collapse. This is generally attributed to redundancy of components and to the positive contribution of “non-structural” components such as partitions, gypsum wall board interior finishes and various exterior cladding. Recent full-scale shake table tests under the CUREE program (Fischer et al. 2001, Mosalam et al. 2002) and the University of British Columbia in Vancouver (Ventura et al. 2002), and numerous cyclic tests of shear walls and other components have provided useful data for characterizing the properties of wood-frame buildings. But they have not been able to explain in a quantitative manner the seismic behaviour of a complete building built by conventional rules.

The Research Program

The objective of this research program is to provide the Canadian and Chinese codes with a quantitative basis for the rational calculations of seismic behavior of conventional wood-frame construction. Towards this objective a collaborative research program was initiated by Forintek Canada Corp. with Tongji University in Shanghai, China, on the experimental phase of the program, while portions of the analytical program are carried out in collaboration with the University of Florence, Italy. This research program consists of four main parts: Shake table tests on two-story test houses, targeted shear wall tests, analytical approaches, and application to codes and standards.

Shake Table Tests

On the 4 m by 4 m shake table of Tongji University in Shanghai, tests were conducted on two wood-frame building specimen. Specimen 1 consisted of a two-story 6 m by 6 m house with 2 x 4 framing, sheathed with oriented strand board (OSB), and finished on the interior with gypsum wall board (GWB). Both stories contained a load-bearing partition with a 1.8 m door opening. Five phases, representing increasing sizes of exterior wall openings of 1.2 m, 2.4 m and

3.6 m were investigated, all except the final phase having symmetrical configurations. See Figure 1. Three different earthquakes were applied in three progressively larger steps of shaking intensity, from 0.1g, 0.2 g, and 0.4 g (nominal) peak table accelerations along with some additional tests with higher values. After each test the specimen was inspected for damage and after each phase the damaged sheathing in the first story was repaired or replaced.

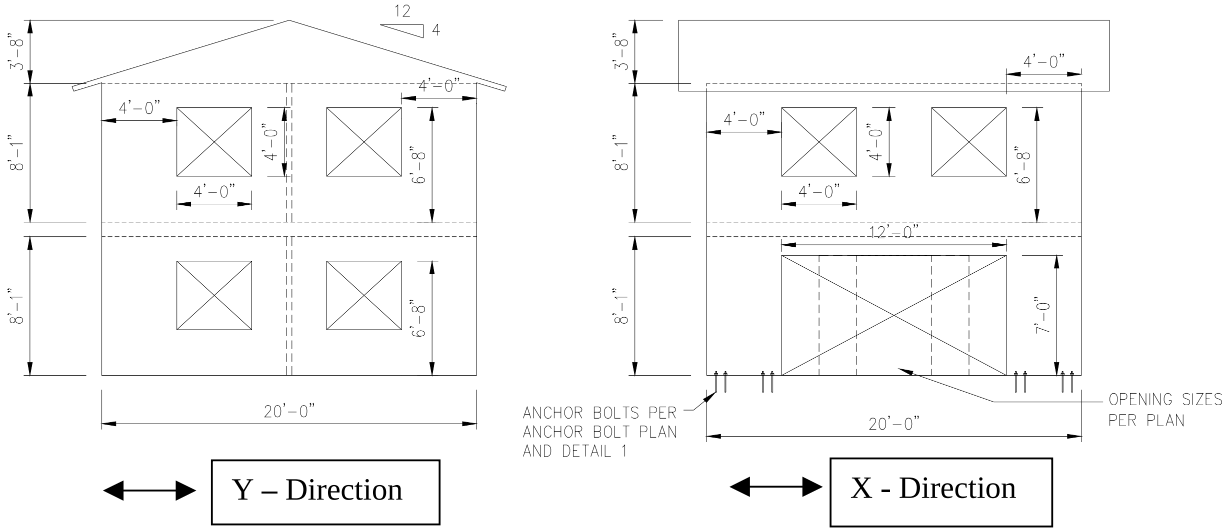

Figure 1. Elevations of shake table test specimens and directions of shaking.

For Specimen 2 the main objective was to assess the torsion behavior of the specimen. The two-story structure had the same physical dimensions and framing as the first specimen, but without interior GWB finishes. Five phases of different configurations were investigated, all except the first one being unsymmetrical. The specimen was subjected to the same sequence of ground motions as the first one, but no input greater than 0.40 of gravity was applied. After each test the specimen was inspected for damage and after each phase the damaged sheathing in the first story was replaced.

Shear Wall Tests

Two series of shear wall tests are carried out. The first investigates the influence of corner walls and of axial load on the shear capacity of the wall. Cyclic lateral in-plane loadings and ramp loadings are applied to the full-size wall specimen in order to obtain properties of walls under various constraint conditions.

The second series of cyclic shear wall tests concerns wall specimen with openings, similar to the walls of the shake table specimen. Restraints include anchor bolts and the presence of the second story wall. This will provide data for the analytical model of the shake table specimens and an opportunity to calibrate the model against the behavior of the test house under seismic shaking.

Analytical Approach

The analytical part consists of: 1) a simplified static analysis of the capacity of the structure to resist lateral seismic loads, and 2) a detailed step-by-step time history analysis of a three-dimensional building.

The simplified mathematical model comprises static calculations of the lateral load carrying capacity of the building, with stiffness and strength properties from pertinent shear wall tests and material standards. Calibration of the simplified method against the detailed method and the shake table test results will be carried out.

The detailed analytical model is based on the analysis method described by Ceccotti and Karacabeyli (2002). The deformational properties of shear wall elements are represented by springs whose stiffness properties have been calibrated against cyclic shear wall test data. The DRAIN-2DX program is employed for the step-by-step analysis to a seismic input. This model has been used successfully in the blind prediction of the seismic response of the building model tested at the University of California San Diego as part of the CUREE investigation (Folz and Filiatrault 2004). Objectives under the Forintek research program consist of further calibrations of the model against shake table test results so that the program can be used for parameter studies of conventional wood-frame buildings.

Application to Codes and Standards

The final part of this research program examines the adequacy of current seismic code provisions for conventional wood-frame construction and will provide code and standards committees with pertinent data for possible revisions where deemed necessary.

Shake Table Tests of Specimen 1 – Symmetrical Configuration

Description of Test Specimen

The test house was built by a local contractor on an extended steel grillage on the 4 m by 4 m shake table at Tongji University, Shanghai, China, in accordance with the prescriptive requirements of the 1995 National Building Code of Canada, augmented by the requirement of the Chinese seismic code of 0.5 times the floor design live load of 2.0 kPa for residential occupancy and 0.5 kPa for roof loading (GBJ 11 – 89, 1994). An additional portion of weights was added to simulate a building with plan dimensions of 10 m by 6 m, rather than the 6 m by 6 m base dimensions imposed by the extended size of the shake table. This resulted in loads of 6000 kg and 1600 kg being added in the form of weights bolted to the second floor and to the roof, respectively. The wood-frame house specimen itself weighed 4400 kg and the steel grillage 4500 kg, for a total mass on the shake table of 16500 kg.

The two-story 2 x 4 wood frame with studs at 400 mm (16 in.) was sheathed on the outside with 9.5 mm (3/8 in.) oriented strand board (OSB) and finished on the inside with 12.7 mm (1/2 in.) gypsum wall board (GWB), taped and grouted. The interior partition was finished with GWB on both faces. The OSB sheathing was fastened with 65 mm (2 ½ in.) galvanized twisted nails of 3.2 mm diameter, spaced at 150 mm (6 in.) along the perimeter of the sheathing panels, 300 mm (12 in.) elsewhere. The GWB was attached with 3.2 mm diameter screws, 28 mm long at 200 mm spacing. Floor construction was 19 mm (3/4 in.) T&G sheathing on 240 mm (9 ½ in.) I-joists at 400 mm (16 in.) spacing. The roof consisted of standard trusses at 61 mm (24 in.) spacing, sheathed with 11 mm (7/16 in.) plywood. Anchor bolts of ½ in. diameter at a nominal spacing of 1220 mm (48 in.) fastened the base of the specimen to the steel grillage. The door openings on opposite sides of the specimen were progressively increased from 1.2 m, to 2.4 m, to 3.6 m for Phases 1, 2, and 3, respectively.

The test specimen was instrumented with 16 accelerometers, 1 at each corner of an exterior wall in the X and Y directions at the base, the first and second floor ceilings, and 2 in orthogonal directions at each roof gable of the specimen. A total of 8 absolute displacement transducers were placed in the direction of shaking at the corners of the wall at the base, first and second floor and at the gable. To measure uplift, 12 relative displacement transducers were applied in the first story between the base plate and the stud at each exterior corner and partition intersection and at each door opening, and 4 at each corner in the second story. 16 load cells were inserted between the base plate and the nut, one at each end of each wall and two at intermediate points or at door openings.

Test Procedure

After some low level static lateral load tests to determine the stiffness of the test building, the seismic motions were applied at nominal levels of 0.1, 0.2 and 0.4 g peak table acceleration. For some tests, additional peak acceleration levels of 0.55 g were applied. Before each testing phase, between each amplitude level, and after each phase, low level white noise (<0.1 g peak) was applied for the determination of natural frequencies and damping values.

At each amplitude level, three scaled shake table motions were applied: “Pasadena” of 1952, “El Centro” of 1940, and an artificially generated ground motion for the region of Shanghai, “SHW2”. The motions were applied uni-directionally in line with the partition and the walls with the door openings (the X direction) for Phases 1, 2 and 3 for openings of 1.2 m, 2.4 m, and 3.6 m, respectively, and in the orthogonal direction (the Y direction) for Phase 4. Within each Phase 2, 3 and 4 the three shake table motions for the runs presented were applied in succession without repairs to the specimen.

Results

The sequence of test runs and some results of the shake table tests for the symmetrical configuration of specimen 1 are presented in Table 1. For each testing phase and associated size of opening, actual peak base accelerations are shown for respective records and run numbers. Table 1 also shows the maximum drift ratio in the first story for each record and the natural frequency of the specimen at the start of each testing phase and at the end of the set of records applied.

Overview of Damage

After each set of seismic table motions at nominal peak values of 0.1, 0.2, 0.4 and some at around 0.55 g the specimen was inspected and any damage recorded. The following general

Table 1: Results for Shake Table Tests of Specimen 1 – Symmetric Configuration

Phase	**Direction of shaking**	**Size of opening (m) in direction of shaking**	**Run nos. ***	**Seismic record**	**Peak base accel., (g)**	**First story drift ratio, (%)**	**Measured natural frequencies**	
							At start of run, (Hz)	**At end of run, (Hz)**
1	X	1.2	3	Pasadena	0.10	0.086	4.44	4.44
			7	Pasadena	0.21	0.114	4.44	4.44
			11	Pasadena	0.49	0.279	4.44	4.25
2	X	2.4	15	Pasadena	0.10	0.085	4.10	
			16	El Centro	0.10	0.095		
			17	SHW2	0.08	0.074		4.10
			19	Pasadena	0.25	0.149	4.10	
			20	El Centro	0.20	0.192		
			21	SHW2	0.24	0.281		3.91
			23	Pasadena	0.44	0.372	3.91	
			24	El Centro	0.37	0.417		
			25	SHW2	0.38	0.562		3.56
3	X	3.6	28	Pasadena	0.11	0.132	3.66	
			29	El Centro	0.10	0.120		
			30	SHW2	0.08	0.127		3.56
			32	Pasadena	0.22	0.230	3.56	
			33	El Centro	0.20	0.221		
			34	SHW2	0.19	0.319		3.32
			36	Pasadena	0.42	0.54	3.32	
			37	El Centro	0.39	0.61		
			38	SHW2	0.44	1.06		2.44
			36a	Pasadena	0.63	1.70	2.44	
			37a	El Centro	0.59	3.03		1.46
4	Y	Two 1.2 m x 1.2 m windows per wall	41	Pasadena	0.10	0.059	3.66	
			42	El Centro	0.11	0.100		
			43	SHW2	0.08	0.096		3.66
			45	Pasadena	0.19	0.128	3.66	
			46	El Centro	0.20	0.201		
			47	SHW2	0.16	0.207		3.56
			49	Pasadena	0.37	0.42	3.56	
			50	El Centro	0.37	0.71		
			51	SHW2	0.36	0.74		3.13
			53	Pasadena	0.48	0.93	3.13	
			54	El Centro	0.56	1.47		
			55	SHW2	0.50	1.39		2.39

* Seismic shake table tests were carried out in the sequence listed

observations apply to the walls in the direction of loading for the first story:

1. At level 0.1 g, no visible damage was observed for any of the Phases.
2. At level 0.2 g, several nails at the bottom of exterior wall were pulled through in Phase 1; no visible damage was observed for other Phases.
3. At level 0.4 g, some OSB panels were compressed where they butted against adjacent panels at the corner edges. Some nails of exterior walls were pulled through the OSB in Phase 1, some nails withdrew slightly from the OSB in the other phases. The GWB had visible damage in each phase after the 0.4g level; the screws at the bottom of the GWB pulled through, GWB cracked at the screws and at the corner of window and door openings.
4. At level 0.55 g, some nails at the bottom of the exterior wall were pulled through in Phase 2 to Phase 4. In Phase 3, some nails near the corner of the OSB panels failed. In Phase 4, some nails of the exterior walls withdrew by a few millimetres.

In the frame no visible damage occurred in Phases 1 to 4. The GWB in the second story developed cracks in the joints at the window openings at 0.4 g in Phase 4 and at 0.55 g in Phase 3; no other visible damage was observed in that story. In addition to some nails failing in the wall along the direction of loading, a few nails withdrew by a few millimetres in the walls perpendicular to the applied shaking at the 0.55 g level, indicating significant load transfer around the corner of the specimen.

Capacity Spectra

For the first story of the specimen, capacity spectra in the form of plots of maximum normalized base shear against maximum displacement are shown in Figure 2 for the final run in each Phase. Of these, only the capacity spectra for Phase 1 ("Pasadena") can be considered to be those of a single seismic motion, the other two being those of a succession of 2 or 3 seismic base motions without the specimen having been repaired. Also, Phase 4 represents shaking in the Y direction, whereas all others were shaken in the X direction.

For the Pasadena record the capacity spectrum for Phase 1, the 1.2 m wall opening, is very nearly linear, whereas for Phase 2, with the 2.4 m opening, a lower initial slope and greater softening behaviour at 0.4 g PGA is evident. For Phase 3, initially a further reduction in slope occurs up to 0.2 g PGA, then a more significant reduction at 0.4g. For the 0.63 g PGA, a significant reduction in slope occurs for a maximum first story displacement of 43 mm or story drift of 1.7 %.

In the Y direction, Phase 4, the capacity spectrum is nearly linear with a slope that up to 0.2 PGA is comparable to that of Phase 2, but then the slope reduces substantially to 0.4 g and further to 0.55 g PGA as the specimen weakens.

Changes in natural frequency

The changes in natural frequency from Table 1, plotted in Figure 3, follow the pattern of the capacity spectra and the observed damages. For phase 1, with the 1.2 m door opening, no discernable change in frequency occurs for the 0.1 and 0.2 g level of the Pasadena motion, and only a small reduction for the 0.4 g level. As the door opening increases in Phase 2 and 3, the initial natural frequencies become progressively smaller and the changes in natural frequency progressively larger for increasing values of peak table accelerations. Comparison of frequency between Phase 2 and Phase 4 shows that with a comparable number of full-sized panels, (in this case 3, for a total width of door openings of 2.4 m), the specimen in Phase 4 has a considerably lower frequency than that of Phase 2. This can be ascribed mainly to the stiffness of the partition along the X direction, a partition that is ineffective as a stiffening element in the Y direction. The progressively lower natural frequency as the level of shaking increases within each Phase is an indication of cumulative damage that the specimen has undergone, the modal stiffness being proportional to the frequency squared.

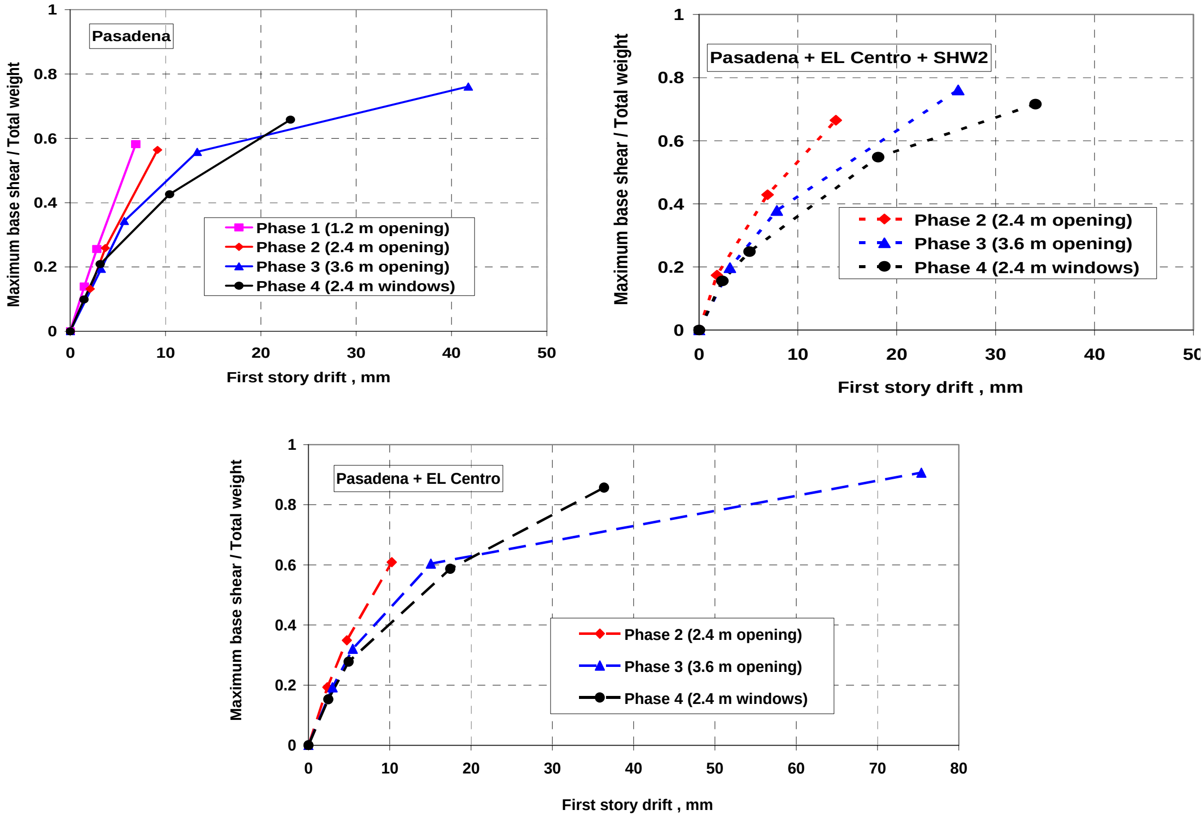

Figure 2. Capacity spectra for symmetrical Specimen 1 for three series of shake table motions.

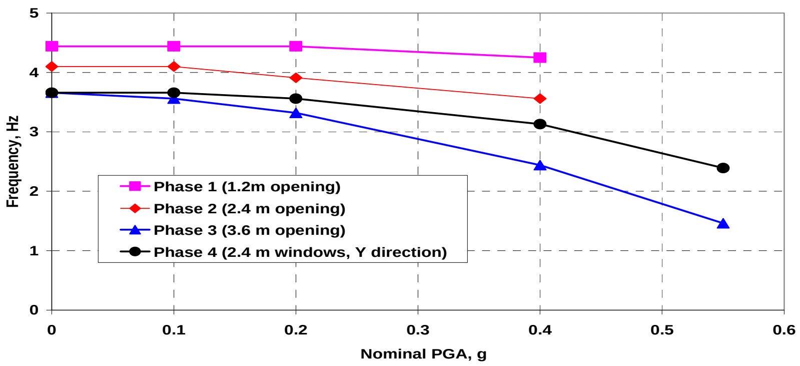

Figure 3. Change of natural frequency of Specimen 1 with level of seismic input.

Discussion

The results of this series of shake table tests show that a conventional wood-frame building of this construction and size can readily survive peak seismic motions of 0.5 g, even after repeated shaking. Even the specimen with 2 full panels of wall length withstood repeated applications of shake table motions of 0.4 g, although a final motion of 0.59 g PGA El Centro brought the specimen to the point of near-collapse. This result is in general agreement with behaviour observed in the shake table tests of the CUREE project at University of California San Diego (Fischer et al. 2001) and at the University of British Columbia (Ventura et al. 2002). It is also is in general agreement with the results observed in actual earthquakes in California, New Zealand and Japan, that wood-frame buildings without major structural deficiencies withstood seismic shaking of 0.5 to 0.6 g PGA without collapse (Rainer & Karacabeyli, 1999).

The challenge still remains of establishing a quantitative basis for explaining this behavior and to determine where the limits of geometry and material properties lie for achieving acceptable performance.

Summary and Conclusions

The research program carried out by Forintek Canada Corp. and its international partners on the seismic resistance of conventional wood-frame construction set out to establish quantitative means for determining the limits in design parameters for the seismic resistance of this type of construction. Shake table tests of two two-story house specimens were performed, and shear wall tests with various constraint conditions, an analytical phase for calculating seismic response, and applications to design guides and codes and standards are in progress or being planned.

The initial results presented show that the tested symmetric building specimen with progressively larger openings can withstand successive application of three different seismic ground motions in the order of 0.55g PGA. Interstory drift in the first story was generally below 1.0 % at table motions up to 0.4 g, some around 1.5 % and one at 3.0 % for the 0.55 g levels. The latter was judged to be very near the point of collapse. The results are consistent with those found by other investigators and the general conclusions from a survey of performance of wood-frame construction in past earthquakes.

Acknowledgements

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