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RESEARCH ON SEISMIC RESISTANCE OF CONVENTIONAL WOOD-FRAME BUILDINGS

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ABSTRACT

A research project initiated in 2004 by Forintek Canada Corp., in collaboration with Tongji University of Shanghai, China, seeks to provide data for the quantitative assessment of the seismic building code provisions for conventional wood-frame construction in both Canada and China. The research project comprises four parts: shake table tests of two two-storey buildings, cyclic tests of large shear wall configurations, simplified and detailed analytical studies, and application to codes and standards. The purpose of the paper is to describe the project and to present initial results of the shake table tests.

Shake table tests are described as carried out at Tongji University on two two-storey specimens, 6.0 m by 6.0 m in plan, to a progression of 3 seismic motions at nominal amplitudes of 0.1, 0.2, 0.4 and 0.55 g peak ground acceleration. The specimens were built according to Part 9 of the National Building Code of Canada, sheathed in oriented strand board, and augmented with additional weights. Specimen 1 was finished with gypsum wall board, while specimen 2 was not finished with gypsum wall board. Initial results of base shear, first-storey drift and changes in natural frequencies are reported and comparisons made between the results. The results demonstrate the considerable contribution of the gypsum wall board to the stiffness and strength of specimen 1. The other three parts of the research project are also briefly described.

Introduction

Wood-frame buildings in Canada and in some other countries can be designed and constructed in one of two ways: 1) by engineering principles of limit states design, and 2) for small buildings of limited area, occupancy and number of storeys, by conventional rules laid out in building codes and construction guides. In Canada the former is governed by Part 4 of the National Building Code of Canada (NBCC), the latter by Part 9 of NBCC. The results of designs by these two methods can differ substantially, especially in areas of higher seismic risk, and the resulting seismic behaviour can also be expected to vary accordingly.

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A number of major research programs on the seismic behaviour of wood-frame buildings have recently been carried out, among them the CUREE project in California (Fischer et al. 2001, Mosalam et al. 2002), and the Earthquake 99 project at the University of British Columbia (Ventura et al. 2002), both involving extensive shake table tests, component testing of shear walls and floors, development of analysis procedures and proposals for changes to design and construction. These studies have produced valuable insight into the seismic resistance of these widely used types of construction both in North America as well as in other seismically active countries such as New Zealand and Japan and recently China. While some cases of conventional houses were studied by these two projects, most of their attention was centred on engineered construction. The Forintek project, on the other hand, focuses entirely on the conventional method of house construction, the majority of small houses in North America being of that type. More specifically, this investigation concerns the seismic resistance of buildings designed and constructed according to Part 9 of the National Building Code of Canada (NBCC 2005).

The objective of this research project is to provide a quantitative basis for a more rational determination of the seismic behaviour of conventional wood-frame construction. This involves a determination of the differences in seismic behaviour between engineered and conventional construction and then narrowing the gap between these two approaches, thus achieving a more uniform seismic risk for these two types of construction. Towards this objective Forintek Canada Corp. is collaborating with Tongji University in Shanghai, China, on the experimental portion of the project.

The Forintek project on seismic resistance of conventional wood-frame housing consists of four main parts: 1) Shake table tests of two-storey specimens; 2) Shear wall tests on targeted geometric wall configurations; 3) Analytical studies of seismic resistance of houses; and 4) Implications for and applications to codes and standards.

This paper describes the research project and presents initial results of shake table testing of two symmetric two-storey specimens. Future work anticipated for the remaining portion of the project is also outlined.

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 specimens. Specimen 1 consisted of a two-storey 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). The specimen was founded on a steel grillage extension to the shake table. 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 and Figure 2 for details of the test specimens and loading procedure. Three different earthquakes were applied in three progressively larger steps of shaking intensity, from nominally 0.1g, 0.2 g, and 0.4 g peak table accelerations plus 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 storey was repaired or replaced.

For Specimen 2 the main objective was to assess the torsional behaviour of the specimen and to assess the difference in response between specimen 1 finished on the interior with gypsum wall board (GSB) and specimen 2 without the GWB. Otherwise the two-storey structures had the same physical dimensions and framing and added weights. Five phases of different opening configurations were investigated, the first two being symmetrical, the remaining three unsymmetrical. The specimen was subjected to the same sequence of ground motions as specimen 1, but the base input was kept nominally at or below 0.40 of gravity. After each test the specimen was inspected for damage and after each phase the damaged sheathing in the first storey was replaced.

Description of Test Specimens

The test houses were built by a local contractor on a 6 m by 6 m 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

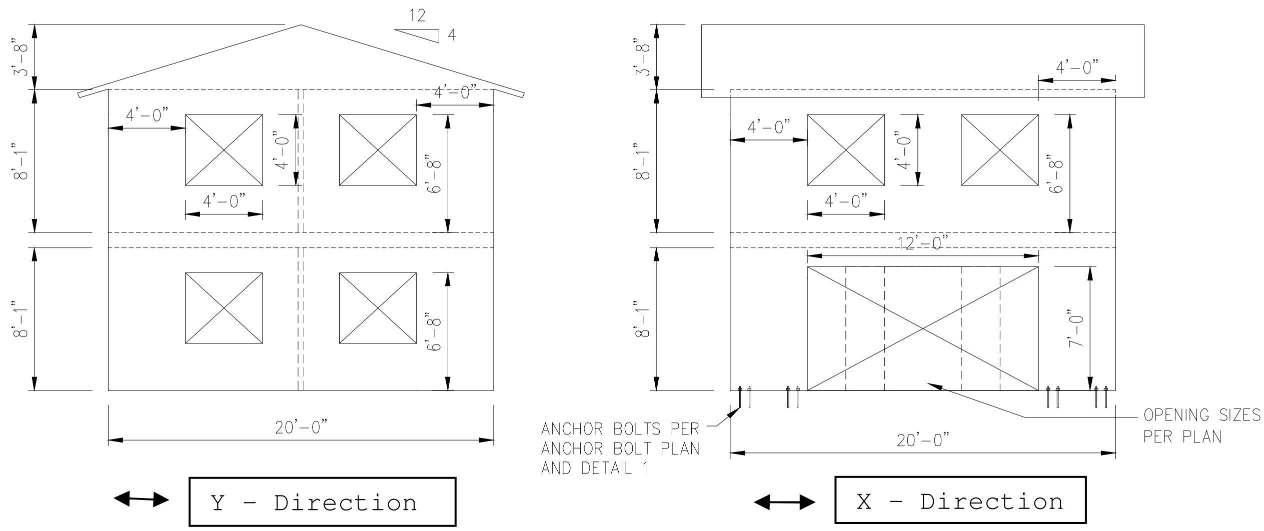

Figure 1. Elevations of shake table test specimens and directions of shaking.

Figure 2. View of test specimen 1.

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 about 11 m by 6 m, rather than the 6 m by 6 m base dimensions imposed by the extended size of the shake table. Thus additional weights of 6000 kg were added to the first floor of specimen 1 and 6092 kg to specimen 2, and 1600 kg to each of the roofs. The structure itself for specimen 1 weighed 5190 kg, and 3043 kg for specimen 2.

The two-storey 2 x 4 S-P-F 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). The sheathing was fastened with 65 mm (2 ½ in.) galvanized spiral nails of 3.2 mm diameter, spaced at 150 mm (6 in.) along the perimeter of the sheathing panels, 300 mm (12 in.) elsewhere. Floor construction was 19 mm (3/4 in.) tongue and groove (T&G) sheathing supported by 240 mm (9 ½ in.) I-joists at 400 mm (16 in.) o.c. 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; double anchor bolts were used at corners and door openings, but no hold-downs. Specimen 1 was finished on the inside and on both faces of the interior partition with 12.7 mm (1/2 in.) gypsum wall board (GWB), attached with 3.2 mm diameter screws 28 mm long at 200 mm spacing, taped and grouted. Specimen 2 had no GWB finish. All wood, joists, trusses, OSB, GWB and plywood originated in Canada and conform to Canadian CSA standards. Nailing schedule followed the Part 9 NBCC requirements (NBCC 2005). The door openings on one side of the specimen were progressively increased from 1.2 m, to 2.4 m, to 3.6 m for different phases of the test.

The test specimens were 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, at the first and second floors and at the gable. To measure uplift forces on anchor bolts, 12 relative displacement transducers were applied in the first storey 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 storey. 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

Before the shake table tests, static tests of the lateral stiffness were taken and the natural frequencies determined from low-level shaking as the door openings were progressively decreased from 3.6 m to 2.4 m to 1.2 m. Thereafter, seismic motions were applied at nominal levels of 0.1, 0.2 and 0.4 g peak table acceleration. For specimen 1, additional peak acceleration levels of 0.55 g were applied for some tests, and before each testing phase, between each amplitude level, and after each phase, low level band-limited white noise (< 0.1 g peak) was applied for the determination of natural frequencies and damping values. For specimen 2 the maximum peak table acceleration was nominally 0.40 g.

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”. For specimen 1 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, and in the orthogonal direction (the Y direction) for Phase 4. Within each of Phases 2, 3 and 4 the three shake table motions were applied in succession without repairs to the specimen. For specimen 2 a similar procedure was followed for Phase 1 in the X direction and for Phase 2 in the Y direction, both with door openings of 1.2 m. For Phases 3, 4 and 5 the door opening on one wall was increased to 2.4 m, 3.6 m and 6.0 m, respectively, but results are not reported here.

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, and for specimen 2 in Table 2. Some results for specimen 1 have been presented previously (Rainer et al. 2006) but are included here for comparison purposes. For each testing phase and associated size of opening the actual peak table accelerations are shown for respective records and run numbers. Tables 1 and 2 also show the maximum drift ratio in the first storey 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.

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 storey drift ratio, (%)**	**Measured natural frequencies (Hz) in direction of shaking**	
							At start of Run	**At end of Run**
1	X	1.2 m door per wall	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 m door per wall	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 m door per wall	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

Table 2: Initial Results for Shake Table Tests of Specimen 2 – Symmetric Configuration.

Phase	Direction of shaking	Size of opening (m) in direction of shaking	Run nos.	Seismic record	Peak base accel., (g)	First storey drift ratio, (%)	Measured natural frequencies (Hz) in direction of shaking	
							At start of Run	At end of Run
1	X	1.2 m door per wall	2	Pasadena	0.10	0.20	3.61	
			3	El Centro	0.10	0.18		
			4	SHW2	0.08	0.14		3.62
			6	Pasadena	0.22	0.46	3.62	
			7	El Centro	0.20	0.40		
			8	SHW2	0.21	0.53		3.20
			10	Pasadena	-	-	3.20	
			10.1	Pasadena	0.34	1.25		
			11	El Centro	0.50	2.53		
			12	SHW2	0.45	3.51		1.17
2	Y	Two 1.2 m x 1.2 m windows per wall	15	Pasadena	0.10	0.12	3.52	
			16	El Centro	0.10	0.18		
			17	SHW2	0.08	0.17		3.22
			19	Pasadena	0.18	0.22	3.22	
			20	El Centro	0.19	0.37		
			21	SHW2	0.18	0.46		3.22
			23	Pasadena	0.21	0.47	3.22	
			23.1	Pasadena	0.39	1.26		
			24	El Centro	0.38	1.68		
			25	SHW2	0.38	1.48		1.17

Overview of Damage

The following general observations apply to the walls in the direction of loading for the first storey of specimen 1:

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 0.4 g 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 in Phase 2 to Phase 4, some nails at the bottom of the exterior wall were pulled through. In Phase 3, some nails near the corner of the OSB panels failed and after Run no. 37a the specimen was near collapse due to failure of a large number of sheathing nails. 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 storey 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 storey. 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.

The results of the performance of specimen 1 have shown that this structure can withstand seismic ground motions of 0.5 g and above, even when applied in sets of three successive records. This is in general agreement with shake table results of similar size and types of specimens (Fischer et al. 2001; Ventura et al. 2002). It is also in line with a survey of seismic performance of wood-frame houses in California, New Zealand and Japan, which concluded that except when major structural deficiencies were present these houses could withstand 0.5 to 0.6 g PGA without collapse (Rainer & Karacabeyli, 2000).

Specimen 2 exhibited the following damage pattern on the first storey walls in the direction of shaking:

1. For Phase 1, shaking in the X direction, at the 0.1 g excitation level no damage was visible. At the 0.2 g level, a few nails showed 1 to 2 mm withdrawal. At the 0.4 g level, over 40 % of the nails showed 1 to 4 mm withdrawal, and another 20 % more than that. Shear failure, pull-through of the nails and chip-out of the sheathing amounted to another 20 % of the nails.
2. For Phase 2, shaking in the Y direction, no damage was observed for the 0.1 and 0.2 g excitation levels. For the 0.4 g level, 20 % of the nails in the full panels showed withdrawal of 1 to 4 mm, and another 5 % more than that. Shear failure, pull-through of the nails and chip-out of the sheathing amounted to another 3 % of the nails.

The reason why the damage in the X direction is more severe than in the Y direction is not immediately apparent. One might expect the opposite to happen, since the X direction has 4 fully sheathed panels per wall whereas in the Y direction the wall has 3 full panels.

Changes in natural frequency

For Phase 1 of specimen 1, no discernable change in frequency occurs for the 0.1 and 0.2 g level of the Pasadena motion alone, and only a small reduction for the 0.4 g level. As the door opening increases in Phases 2 and 3 and the sequence of the three records is applied, the initial natural frequencies become progressively smaller and the changes in natural frequency progressively larger for increasing values of peak table accelerations. Comparison of frequencies between Phase 2 and Phase 4 shows that with a comparable number of full-sized panels, (in this case 3 panels, 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 attributed 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.

Although the two specimens have the same geometry and the same added masses, the absence of GWB in specimen 2 results in significant changes in natural frequencies. While this reduction of mass from a total of 12788 kg for specimen 1 to 10739 kg for specimen 2 should by itself lead to an increase in frequency from 4.44 Hz to 4.84 Hz (in proportion to the square of the mass ratio), the frequency for specimen 2 actually decreased from the mass-adjusted 4.84 Hz to 3.62 Hz. This 25 % reduction in frequency can be attributed to the loss of stiffness from the GWB finish and demonstrates the significant contribution that the GWB makes to the stiffness of the specimen.

In the Y direction, a comparison between frequencies for Phase 4 of specimen 1 (Table 1) and Phase 2 of specimen 2 (Table 2) shows a smaller decrease in frequency, from 3.66 Hz to 3.52 Hz, respectively. This decrease is less pronounced since in the Y direction the partition does not contribute any stiffness, only some mass.

Relation between Maximum Base Shear and Drift

For the first storey of specimen 1, plots of maximum normalized base shear against maximum first-storey drift are shown in Figure 3 for the final run in each Phase. Of these, only the plots for “Pasadena” in Phase 1, Figure 3a, can be considered to be those of a single seismic motion; the others in Figure 3b are each the result of a succession of 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 response to the Pasadena record the plots of normalized base shear versus drift for Phase 1 – with the 1.2 m wall opening - is 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 0.4 g. For the 0.63 g PGA, a large reduction in slope occurs for a maximum first storey displacement of 43 mm or first storey drift of 1.7 %. In the Y direction, Phase 4, the plot is nearly linear with a slope that up to 0.2 g 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.

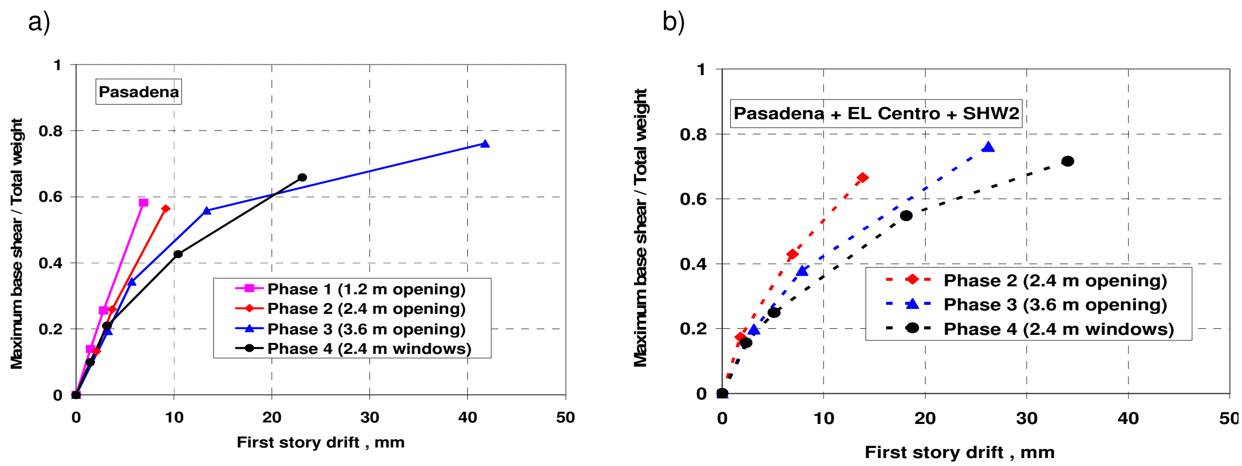

Figure 3. Plots of maximum base shear ratio versus first-storey drift for specimen 1 subjected to a) Pasadena record, and b) Pasadena + El Centro + SHW2 records.

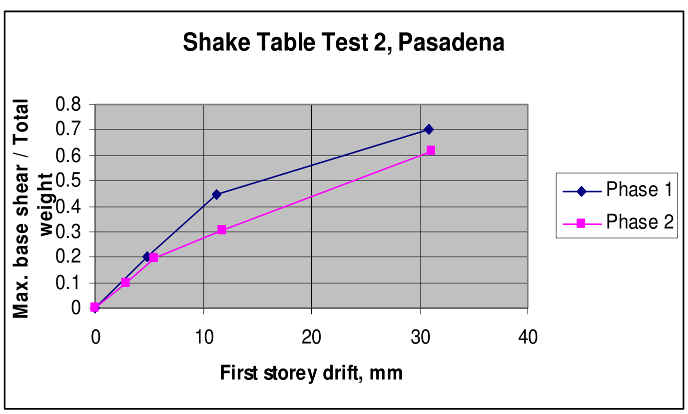

Figure 4. Plots of maximum base shear ratio versus first-storey drift for specimen 2 subjected to Pasadena record.

Figure 4 shows plots of normalized base shear versus first-storey drift for Phases 1 and 2 of specimen 2 after having been subjected to the respective amplitudes of the Pasadena record. A comparison of Figure 3a and Figure 4 shows that the initial slope of the plot for Phase 1 and Phase 4 of specimen 1 in Figure 3a is substantially greater than corresponding slopes for Phase 1 and Phase 2 of specimen 2 in Figure 4, indicating a greater stiffness for specimen 1. The deviation from the essentially linear behaviour occurs at larger base shear ratios for specimen 1 than for specimen 2. These differences in deformational behaviour can be attributed mainly to the absence of GWB finish in specimen 2.

Shear Wall Tests

Two series of in-plane monotonic and cyclic shear wall tests have been carried out under this research project and a third one is in the planning stage.

Series 1 investigates the influence of corner walls and of axial load on the shear capacity of the wall. Tested were walls of 2x4 framing sheathed with OSB, 8' x 20' in size with various openings. Some walls were augmented by 4' long perpendicular corner walls and by dead load. Horizontal cyclic and monotonic in-plane loadings were applied to obtain deformational properties of walls under various constraint conditions. The results have demonstrated that corner walls enhance the shear capacity of conventional braced wall lines. For further details see Cheng et al. (2006).

Series 2 of the shear wall tests concerned the effect of additional stiffeners added to the top of the wall. 8' x 20' walls were tested with a hinged loading beam and then with a stiff beam added on top. Two tests were also carried out on two-storey specimens. The results have demonstrated that the stiffening effects of a beam or a second storey substantially increases the shear resistance of conventional braced wall lines. For further details see Liu et al. (2006).

Series 3 of the shear wall tests is planned to yield further quantitative data of the effect of various forms of stiffeners. A new racking device will be employed that minimizes the constraint effects of the loading bar of the standard test procedure. This will permit a better quantification of the stiffening effects of corner walls, second storey and dead load effects.

Analytical Modeling

The analytical part of the research project consists of simplified static analyses of the capacity of the structure to resist lateral seismic loads, and detailed step-by-step time history analysis of three-dimensional buildings. The latter will include improvements to the analysis method described by Ceccotti and Karacabeyli (2002). Calibration of the simplified method against detailed methods and the shake table test results will also be carried out.

Application to Codes and Standards

The final part of this research project will examine the adequacy of current seismic code provisions and design guides for conventional wood-frame construction, e.g. the 2005 NBCC, the Canadian Wood Council Design Guide (CWC 2004), the Chinese Code for Design of Timber Structures (GB 50005-2003), as well as others, and will provide code and standards committees with recommendations for revisions where deemed necessary.

Summary and Conclusions

The research project 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-storey house specimens were described, and outlines of shear wall tests with various constraint conditions, an analytical phase for calculating seismic response, and applications to design guides and codes and standards were presented.

The initial results from the shake table tests show that the tested symmetric building specimen with progressively larger openings could withstand successive application of three different seismic ground motions in the order of 0.55 g PGA. Inter-storey drift in the first storey was generally below 1.0 % at table motions up to 0.4 g, some around 1.5 % and around 3.0 % for the 0.5 g levels. Comparison of response for specimens 1 and 2 shows that gypsum wall board (GWB) finish contributes substantially to the stiffness of the structure.

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.

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