LHI Journal (2012) 3(1):79-82
http://dx.doi.org/10.5804/LHIJ.2012.3.1.79
http://lhi.lh.or.kr

An Assessment Study of Seismic Resistance of Two-story Wood-frame Housing by Shaking Table Tests

Chun Ni[1], Sang-Yeon Kim[2], Haijiang Chen[3] and Xilin Lu[4]

(Received January 2, 2012 / Revised January 26, 2012 / Accepted January 27, 2012)

ABSTRACT

While there exists a relatively large body of technical information for the engineered design of wood-frame buildings to resist seismic ground motions, the quantitative assessment of seismic resistance of conventional houses built by prescriptive requirements is less well understood. Forintek Canada Corp., in collaboration with other research and industry partners, has embarked on a research project to address this topic. This paper will report on the seismic shake table tests of a full-scale wood-frame building. The two-story specimen, 6m x 6m in plan, was built on the seismic shake table at Tongji University in Shanghai, China, according to Part 9 of the 1995 National Building Code of Canada and shaken uni-directionally in each of the two principal directions. Three different seismic table motions were applied at increasing peak ground motion amplitudes up to 0.40 and 0.50 g. The specimen was repaired after the above sets of seismic table motions, and successive runs were conducted for increased door openings. Measurements included specimen accelerations, displacements and anchorage forces. Static stiffness of the specimen was measured at low force levels, and natural frequencies were measured after each seismic loading stage by applying low-level random excitation. The results presented consist of the capacity spectra of the shake table tests, changes in specimen stiffness and natural frequencies with increasing seismic loading. These results and those from other recent shake table tests elsewhere will be compared with simplified engineering calculations based on codified values of strength, and on that basis preliminary conclusions will be drawn on the adequacy of the current code provisions and design guides in Canada and the USA for conventional wood-frame construction.

Keywords: Shake table test, wood-frame housing, seismic loading, peak ground motion

1. Introduction

Wood-frame construction is by far the most common structural type in North America for single-family houses and low-rise multi-family dwellings, constituting around 90% of all residential housing (Fischer et al., 2001).

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. Within these limits no detailed engineering calculations are required. Most 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 2308 of the International Building Code (IBC, 2003) for the USA, and for Canada in Part 9 of the National Building Code of Canada (NBCC, 1995).

Previous earthquakes in California, Japan and New Zealand have demonstrated that wood frame construction perform exceedingly well from a life-safety perspective. Compared to the number of houses strongly shaken, the number of people killed or seriously injured in wood frame houses is relatively low (Rainer & Karacabeyli, 1999). Shake table tests conducted at the University of California San Diego (Fischer et al., 2001) and in the University of British Columbia (Ventura et el., 2002) also demonstrated that residential wood frame buildings could withstand severe shaking without collapse.

It can be shown by simple engineering calculations, however, that the lateral strength of a number of conventional wood frame

1) Wood Engineering Scientist, Forintek Canada Corp, Canada (Main author: chun.ni@pfinnovations.ca)
2) Senior Research Fellow, LHI, Deajun, Korea (Corresponding author: sy-kim@lh.or.kr)
3) Professor, Tongji University, Shanghai, China
4) Post-doctoral Fellow, Tongji University, Shanghai, China

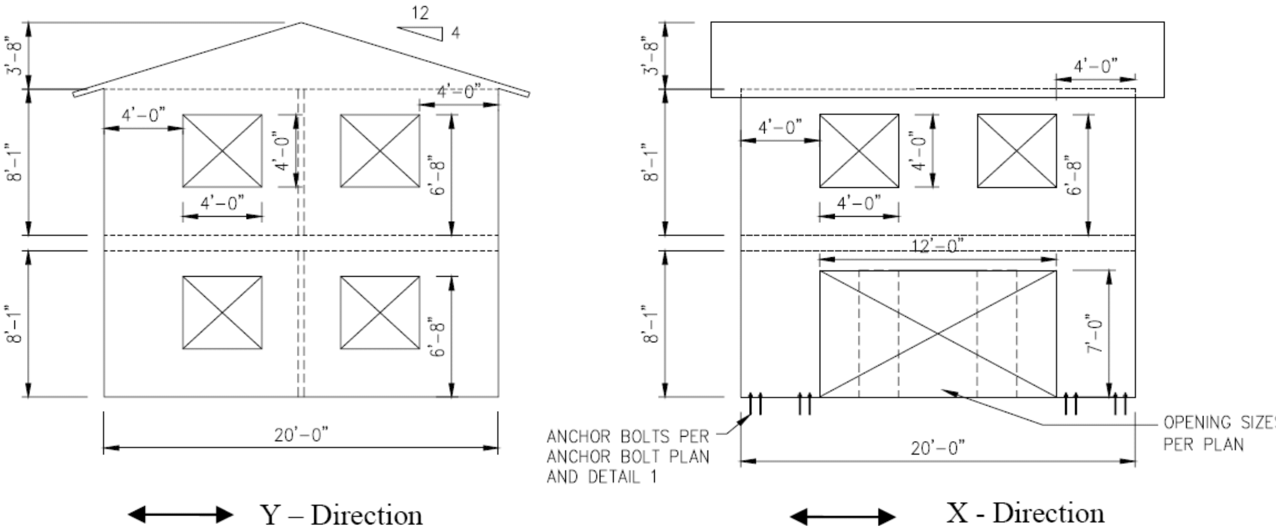

Fig. 1. Elevation of shake table test specimens and direction of shaking

buildings would not satisfy the seismic demand in higher seismic areas. Yet with some notable exceptions such buildings have withstood significant ground shaking and did not collapse. This is generally attributed 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 and numerous cyclic tests of shear walls and other components have provided useful data for characterizing the properties of wood frame buildings, but have not been able to explain the seismic behaviour of a complete building built by conventional rules.

This paper describes a research program undertaken by Forintek Canada Corp. in collaboration with other partners, and presents results from shake table tests of a conventional two-story wood frame building with the effort to quantify the performance of conventional wood frame buildings.

2. Research Program

The objective of this research program is to provide building codes with a quantitative basis for rational calculations of seismic behavior of conventional wood frame construction. For this purpose a collaborative research program was initiated by Forintek Canada Corp. with Tongji University in Shanghai, China. This research program focus on the seismic resistance by shake table tests.

Two two-story full-size wood frame buildings were tested on the shake table of Tongji University in Shanghai. The first specimen investigated the performance of a symmetric building with different braced wall lengths. For the second specimen, the main objective was to assess the torsion behavior and the role of interior finishing materials. Both specimens had the same physical dimensions and were subjected to the same sequence of ground motions.

Fig. 2. Two-story wood frame specimen

3. Experimental Research

3.1 Description of Test Specimens

A 6 × 6m two-story full-size wood frame specimen was built on an extended steel grillage on the 4 × 4m shake table. Figure 1 shows the elevations in X and Y directions. Both stories contained a load-bearing partition with a 1.8 m door opening.

The test specimen was built in accordance with the prescriptive requirements of the National Building Code of Canada (NBCC, 1995). Four phases representing increasing sizes of openings were investigated (Table 1). Three different earthquakes (Pasadena, El Centro, and an artificially generated Shanghai earthquake record) were applied in three progressively larger steps of shaking intensity, from 0.1g, 0.2g and 0.4g peak ground acceleration (PGA). For some tests, additional runs with peak acceleration levels of 0.55g were applied.

The test specimen was instrumented with 16 accelerometers, 1 at each corner of an exterior wall in the X and Y directions at each

Table 1. Shake table test matrix

Direction of shaking	Phase	Description	Braced wall length/side
X-dir (parallel to partition)	1	Remove one panel	4.8m (80%)
	2	Remove two panel	3.6m (60%)
	3	Remove three panel	2.4m (40%)
Y-dir (transverse to partition)	4	Fixed opening	3.6m (60%)

Table 2. Results of shake table tests

Phase	Size of opening in direction of shaking	Peak ground acceleration, (g)	Maximum first story drift, (mm)	Maximum first story drift ratio, (%)	Maximum base shear, (kN)	Natural frequency at end of run, (Hz)
1	1.2m door	0.10	1.48	0.061	16.42	4.44 / 4.44*
		0.21	2.80	0.115	30.25	4.44
		0.49	6.88	0.282	68.76	4.25
2	2.4m door	0.10	2.33	0.095	22.80	4.10 / 4.10*
		0.24	6.93	0.284	50.69	3.91
		0.38	13.8	0.567	78.57	3.56
3	3.6m door	0.11	3.25	0.133	22.92	3.56 / 3.66*
		0.19	7.87	0.323	44.78	3.32
		0.44	26.2	1.074	89.91	2.44
		0.59	75.4	3.090	107.2	1.46
4	2-1.2m windows	0.11	2.45	0.100	18.08	3.66 / 3.66*
		0.16	5.11	0.209	29.42	3.56
		0.36	18.1	0.743	68.3	3.13
		0.56	36.4	1.490	101.3	2.39

*Measured natural frequencies at the start of the phase.

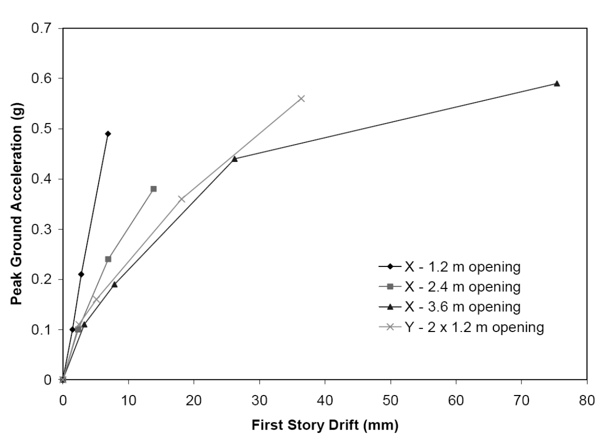

Fig. 3. PGA vs. max first story drift

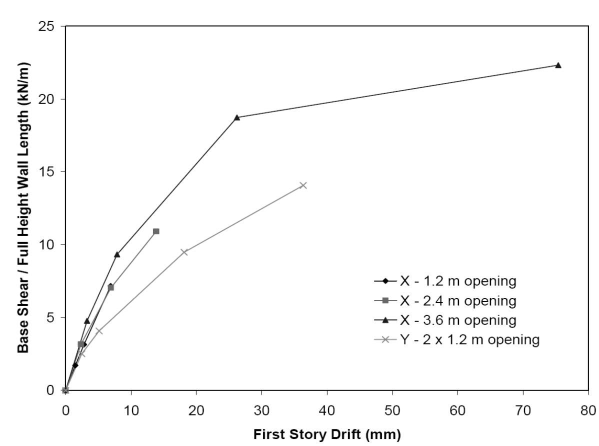

Fig. 4. Base shear vs. max 1st story drift

floor, and 2 in orthogonal directions at each roof gable. A total of 8 displacement transducers were placed at the corners of the wall at the base, first and second floor and at the gable. To measure uplift, 12 displacement transducers were applied in the first and second story. 16 load cells were inserted between the sill plate and the nut.

3.2 Test Results

The maximum story drift and story drift ratio in the first story, the maximum base shear and the natural frequency of the specimen are provided in Table 2.

After each set of seismic table motions at nominal peak values of 0.1, 0.2, 0.4 and some at around 0.55g the specimen was inspected for any damage. The following general observations apply to the walls in the direction of loading for the first story:

- At level 0.1 g, no visible damage was observed for any of the phases.
- At level 0.2g, several nails at the bottom of exterior wall loosened.
- At level 0.4g, some OSB panels were compressed.
- At level 0.55g, more nails at bottom were withdrawn or pulled through OSB panels.

The relationship between the PGA and the maximum first story drift is shown in Figure 3. The maximum base shear versus the maximum first story drift is shown in Figure 4. As noticed from the Figures, for phase 1, which has two 2.4m full-height wall segments on each side of the exterior wall, the response of the first story is still in a near elastic range at peak ground acceleration of 0.49g. For phase 2, the interstory drift is almost double that of phase 1. For phase 4, the initial stiffness is substantially smaller than that of phase 2. This indicates that the interior partition wall along the X direction has a significant influence on the initial stiffness of the building. Under the same peak ground acceleration, phase 4 experienced greater interstory drift than phase 2. For phase 3, large interstory drift was observed at peak ground acceleration of 0.59g, indicating the building is close to collapse.

4. Conclusion

The test results show that the symmetric two-story building specimen can withstand successive application of three different seismic ground motions in the order of 0.55g PGA. For all the phases, the maximum interstory drift in the first story was below 1.0% for PGA up to 0.4g. For phases 3 and 4, the maximum interstory drift reached 3.0% and 1.5% at the nominal PGA of 0.55g, respectively. As nailed shear walls can usually withstand a maximum interstory drift of 4% (Karacabeyli and Ceccotti, 1996), it is judged that the phase 3 building is near the point of collapse at the PGA of 0.59g. 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 in general agreement with the results observed in actual earthquakes in California, New Zealand and Japan (Rainer & Karacabeyli, 1999).

Acknowledgements

The contributions of the staff of the shake table facility of Tongji University for their efficient and competent performance of the tests are appreciated. Financial support from Forestry Innovation Investment BC and the Canadian Wood Export Program is gratefully acknowledged.

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