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SEISMIC PERFORMANCE OF CONVENTIONAL WOOD-FRAME BUILDINGS

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SUMMARY

Results from shake table tests of full-scale specimens and cases of detailed non-linear analysis have shown that for the range of conventional wood-frame buildings tested, structural collapse is well above 0.5 g peak ground acceleration or spectral acceleration Sa(0.2) = 1.0. This corresponds in general with the observations of earthquake damage of wood-frame buildings that those without major weaknesses, such as weak or soft storeys and lack of anchor bolts, are able to survive ground shaking of the order of 0.5 and 0.6 g without collapse.

Parametric studies with code-prescribed static analysis of 1, 2 and 3 storey wood-frame buildings give lower relative seismic resistance for large and multi-storey buildings than for small square buildings. Those results should be taken as indicators of behavioural trends, rather than definitive final results since 1) the contributions of the finishing materials, such as exterior stucco and interior gypsum wall board, have not been included and 2) the static analysis method can only approximate the dynamic nature of the process.

INTRODUCTION

A large number of wood-frame houses in Canada, the USA, New Zealand and many other countries are built by conventional rules laid out in so-called prescriptive sections of building codes rather than in the portions where detailed engineering calculations are required. The prescriptive sections in Canada are those of Part 9 of the 1995 National Building Code of Canada [1] (NBCC 1995) and the upcoming 2005 edition [2] (NBCC 2005), those of the USA can be found in the Uniform Building Code 1994 (UBC 1994) and 1997 Editions (UBC 1997) [3] [4] and the International Building Code, 2000 Edition [5] (IBC 2000), and those of New Zealand in NZS 3604:1999 [6]. While not identical, the requirements in these codes follow similar principles. Among these codes, the provisions for seismic resistance are also similar except that the IBC 2000 ones are substantially more demanding. It needs to be pointed out that these codes are minimum requirements and that local practice or ordinances can, and sometimes do, require

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more than this minimum. The main characteristics of wood-frame buildings of conventional construction are described in standard references, e.g. [7].

In Canada, seismic design along engineering principles is carried out according to Part 4 of the NBCC and in California according to Chapter 16 of the UBC 1997 and the IBC 2000. All quantitative comparisons with engineered design will be carried out here in accordance with Part 4 of the upcoming 2005 edition of the National Building Code of Canada.

In order to assess the structural behavior trends of houses built under conventional or prescriptive rules of construction, such buildings are examined by codified engineering principles and by way of the results from experimental conditions of full-scale shake table testing. While some aspects of wood-frame houses, such as the seismic behaviour of finishing materials, are not yet well encompassed by this approach, this should nevertheless serve as a first step towards a rational basis for assessing the seismic resistance of houses built by conventional rules of construction. The ultimate test of a structure is, of course, the observed performance in actual earthquakes, both for engineered and conventional construction.

While the seismic performance of wood-frame buildings has been very positive in terms of life safety, the prediction of their performance by simple engineering calculations has so far not been totally successful. This can be attributed to the fact that these buildings consist of a large number of components of diverse geometry and structural properties, connected in a manner that is difficult to quantify. A high degree of non-linear behaviour at elevated levels of shaking further complicates the treatment of seismic response of these structures. Recent shake table tests of full-scale wood-frame buildings have brought new data and understanding of their seismic behaviour, and such results will be examined in this study. Much effort is also devoted to the detailed non-linear modeling of wood-frame buildings and this will be addressed as well. Finally, currently specified simple analysis procedures presented in building codes will be used to illustrate how building size and geometry could affect seismic beahaviour of buildings constructed by conventional rules.

OBJECTIVE AND SCOPE

The objective of this study is to compare the performance of wood-frame buildings built according to prescriptive construction rules with test results, with field observations in earthquakes, with detailed non-linear calculations and with design procedures that are specified for engineered buildings. The performance objective is that of "extensive damage" [8] which could also be termed the "state of near collapse". The seismic requirements are those of the upcoming 2005 National Building Code of Canada [1] and the Engineering Guide for Wood-Frame Construction [9], hereafter called "CWC design guide".

Only buildings of regular geometric shapes are considered here, i.e. square or rectangular in plan and without setbacks in the vertical direction. Heavy roofs and floors with concrete topping are not included. Irregular shapes are also not addressed and these would need to be investigated separately.

PRINCIPLES OF SEISMIC RESISTANCE OF WOOD-FRAME BUILDINGS

The seismic resistance of wood-frame houses depends primarily on the three following properties of the building: 1) the shear resistance at the base; 2) shear resistance of the exterior and interior walls; and 3) the uplift resistance of the building at the base and at floors at every storey.

Base shear is the primary loading effect on the structure. For a wood-frame building the base shear has to be resisted at the foundations level since the foundation, usually constructed of concrete, moves with the ground and has been shown to resist seismic motion without significant distress. The movement at the interface between the concrete base and the wooden structure has to be resisted by anchor bolts from the concrete through the bottom sill of the exterior shear walls.

In most buildings the seismic resistance is provided by the exterior walls and to a lesser degree by "non-structural" items such as interior partitions and exterior finishes. Because the contributions to seismic performance of these non-structural items have not been well assessed quantitatively, they are largely discounted in seismic code requirements. For example, according to the upcoming 2005 edition of the National Building Code of Canada [2], when the force modification factor Rd = 3 is used in the analysis the contribution of interior gypsum wall board cannot be utilized.

Shear and uplift in a wall element are intimately connected by equilibrium relationships in that shear resistance requires resistance to uplift to prevent the walls from overturning. Such uplift resistance can be provided from three sources: 1) Hold down devices from the lower sill plate and from vertical studs to the foundation; 2) self weight or dead load of the wall and structure above the lower sill plate; and 3) shear transferred from walls that frame into to the shear wall in the perpendicular direction. The total uplift resistance can also derive from a combination of these. Lack of uplift resistance is not directly discernible in earthquake-induced damage. However, inadequate uplift resistance reduces the shear capacity of the shear walls and therefore reduces the seismic resistance of the building. Although anchor bolts through sill plates no doubt also contribute to uplift resistance, this could be at the expense of shear resistance of the sheathing that is attached to the sill plate. If uplift resistance is not adequate then the specified shear capacities have to be reduced [10] [11] or hold down devices installed at the ends of shear wall panels.

The approach of utilizing the shear capacity of walls perpendicular to the shear wall under consideration ultimately leads to the recognition that the building behaves like a box, in that elements not in the same direction as the primary resistance elements contribute to the seismic resistance of the entire structure. While in this study a number of simplifying assumptions in the analytical model have been made, the results permit a general overview of the seismic behaviour of wood frame buildings. The analysis employed here can then be extended by more sophisticated modelling and by further focused parametric studies.

RESULTS FROM FULL-SCALE SHAKE TABLE TESTS

A large number of static and dynamic tests on building components such as shear walls have yielded much useful data for the design of these items. However, their interaction with other building components is difficult to quantify unless full-scale buildings can be tested. Thus, shake table tests of full-sized building specimens constitute one further step towards greater realism in trying to understand and eventually predict the complex seismic behaviour of wood-frame buildings. In recent years a number of such shake table tests have been undertaken and some will be reviewed here.

University of California San Diego Tests

Description of Specimens

The test and specimens are described in detail by Fischer et al. [12]. The two-storey test structure with plan dimensions 16 ft by 20 ft (4.88 m by 6.10 m) was designed according to the UBC 1994 for UBC Zone 4 with Force Reduction Factor Rw = 8;

Size of specimen: length = 20'-0" (6.10 m) in E – W direction; width = 16'-0" (4.88 m), N – S.

Windows are located in second floor walls and normal to shaking on the first floor, no finishing on exterior or interior walls except for Phase 10, but masses were added to compensate for the weight of finishing materials.

Sheathing is 3/8 in. (9.5 mm) oriented Strand Board (OSB), 6" (150 mm) nail spacing.

The following is an abbreviated list of the unidirectional shake table tests:

Phase 5: no openings on E and W side walls;
Phase 6: engineered construction with door and window openings; 3'-0" (0.91 m) door in middle of each E and W wall in short direction;
Phase 7: perforated shear wall design;
Phase 8: conventional construction in California;
Phase 9: engineered construction with large opening on E side;
Phase 10: as Phase 9 but with exterior stucco finish and interior GWB sheathing.

The essential construction features of test Phase 8 is presented in more detail since it is the most relevant one for the type of construction under consideration here:

Phase 8 - conventional construction: The framing followed conventional construction practice. Hold-down anchors were not used and inter-storey shear forces were transferred using nail connections of 3-16d (0.131" shank diameter) second storey sill plate nailing per stud bay and 2-8d (0.113" diameter) toe nailing through each block into the top plate of the first storey walls. First storey had a 3'-0" (0.91 m) door opening in each wall in the direction of shaking.

Shake Table Seismic Motion

Each test structure was subjected successively in the N – S direction to the ground motion record from Canoga Park of the 1994 Northridge Earthquake, scaled to 0.05, 0.22, 0.36 and 0.50 g peak ground acceleration (PGA). In addition, for Phases 9 and 10 the full record from the Rinaldi Recording Station of the 1994 Northridge Earthquake of 0.89 g PGA was also applied.

Evaluation of Test Results

Detailed results from the CUREE tests are presented by Fischer et al [12]. The following general observations and conclusions can be drawn:

1. The applied peak accelerations of 0.50 g for Phases 5 to 8 and 0.89 g for Phases 9 and 10 did not reach a "near-collapse" state, as can be deduced from the monotonically rising capacity spectra, the visual observation of damage, and the inter-storey drift in the first storey.
2. As might be expected, the amount of deflection increased with the degree of openings in the walls.
3. For the same PGA of up to 0.50 g, Phase 6 (the engineered construction), Phase 7 (the perforated shear wall design), Phase 8 (the conventional construction), and Phase 9 (the large opening) had very similar base shear forces: Phases 7, 8 and 9 also had very similar deflections. This would indicate that using hold-down connections only at the end of the walls do not significantly affect the seismic behaviour of this structure.
4. The stucco-finished structure with the large opening, Phase 10, was approximately twice as stiff as the fully sheathed OSB building, Phase 5; at 0.5 g PGA they had virtually the same base shear.
5. The inter-storey displacement ratio of the first storey is less than 2.0 % for all Phases subjected to 0.50 g PGA. For Phase 9 (with the large opening in the E-wall) the average inter-storey displacement ratio was 2.68 % for 0.89 g PGA. This easily satisfies the NBCC 1995 inter-storey drift requirements of < 2 % and the 2.5 % inter-storey drift limit proposed for NBCC 2005.
6. The stucco and GWB finishing in Phase 10 increased the stiffness of the structure significantly and reduced the lateral deflections by factors of 5 or more, reducing the damage to the finished structure substantially. In this series of tests the contributions of stucco and GWB were not evaluated separately.

Since the shake table specimens were not tested to collapse, the maximum PGA achieved can be taken as a lower bound of failure load. For the ultimate strength per unit length of wall, allowance will have to be made for the contribution of "non-structural" components, primarily sheathing with gypsum wall board (GWB) and exterior finishes such as stucco, neither of which were present in the Phases 5 to 9. The results of the tests indicate, however, that conventional construction incorporates adequate seismic resistance to achieve the ultimate limit state for ground shaking of at least 0.5 g PGA and well beyond that.

University of British Columbia Tests

The Earthquake 99 project at UBC was carried out in parallel with the CUREE project. The specimens were of the "Box" building type, as designated by the CUREE project. Although detailed results of these tests are not available, two-storey test houses with OSB sheathing with and without stucco and those with horizontal board sheathing without stucco did not collapse when subjected to the Kobe – KJMA record of 0.6 g PGA, in addition to other seismic records [13]. Maximum inter-storey drifts were 0.7 % for OSB with stucco, 8.2 % for the horizontal board sheathing without stucco, and 2.3 % for the OSB without stucco. The 8.2% inter-storey drift associated with the two-storey house sheathed with horizontal boards (without stucco) can be considered as the "near collapse" state since horizontal board sheathing alone exhibits very large ductility but relatively low strength and stiffness. Note that North American conventional construction requirements do not recognize horizontal sheathing as lateral load resisting elements, although older buildings frequently used them. Although the test program did not include a test house with horizontal board sheathing together with stucco finish, the results of two-storey houses with OSB sheathed walls with and without stucco finish suggest that stucco finish would have significantly reduced the large inter-storey drift observed on walls with the horizontal boards without stucco. CUREE results also would support this extrapolation.

Japanese Tests

Kohara and Miyazawa [14] describe a series of shake table tests on six specimens of two-storey wood-frame houses with a floor area of 130 m^2 . Of these six, two are described in some detail in the above reference: Type B has a mortar finish and diagonal bracing, Type F has plywood sheathing and siding, a tile roof and hold-down devices at the four corners of the first floor. This latter corresponds most closely to a typical North-American wood-frame construction.

Type F was subjected to the components of the 1995 Kobe – JMA ground motions with 818 gal (cm/s^2) or 0.83 g horizontal, and 332 gal or 0.34 g vertical peak accelerations. The specimen exhibited the following behaviour: the four hold-down devices were deformed; nails of the plywood sheathing came out; a residual deformation of 1/30 radians was observed. The specimen did not collapse, nor did it appear to be near the point of collapse.

University of California Berkeley Tests

The building investigated at Berkeley as part of the CUREE Woodframe project consisted of a full-scale three-storey apartment building representative of the "tuck-under-parking" type [15]. In these types of buildings, some or all of the ground level is used for parking, resulting in one face of the first floor being open. This results in a torsionally sensitive structural configuration in the horizontal plane and a soft storey irregularity in the vertical plane on account of the first storey stiffness being substantially less than that of the upper storeys. Such building types have suffered considerable damage in previous earthquakes in California. During the 1994 Northridge earthquake some collapsed, others suffered near-collapse and damage. This series of tests, while significant for irregular types of wood-frame buildings, will not be considered in detail here since it is outside the scope of this paper. It is worth noting, however, that the building did not reach a near-collapse state until 0.5 g PGA.

CALCULATED SEISMIC CAPACITY OF WOOD-FRAME BUILDINGS

Two types of calculations of the seismic capacity of wood-frame buildings are considered here: 1) non-linear step-by-step analysis; and 2) static analysis as prescribed in the building code.

Non-linear step-by-step analysis

Because of the complex nature of the seismic behaviour of the multitude of components that constitute a wood-frame building, analytical studies are often not considered sufficiently reliable on which to base generally applicable design requirements unless relevant calibration studies have been carried out. Useful information and conclusions should be possible, however, when results from such calculations are combined with other corroborating evidence.

Ceccotti and Karacabeyli [16] carried out an analytical study of the seismic resistance of a four-storey multi-bay building designed for various combinations of plywood and GWB sheathing for various force reduction factors R as per the seismic design requirements of the NBCC 1995 for Vancouver, British Columbia. The various mathematical models were subjected to a number of ground motions, some actual recorded earthquakes, others artificial ones that were designed to represent plausible ground motions on firm soil for the Vancouver area. The following calibrations of the mathematical model were carried out:

1. The force-deformation relationships for the model of the shear walls are based on experimentally determined hysteresis curves of a cyclic loading protocol with increasing deformation amplitudes [16];
2. In a "blind" test conducted by the CUREE program at the University of San Diego, the same mathematical model was used to predict the failure of the two-storey wood-frame building described above which was to be subjected to seismic ground motions from the 1994 Northridge Earthquake on a shake table. The agreement between the experimental and the subsequent test results were excellent [17].

These calibrations provide a measure of assurance that the seismic response calculated with this model is fairly realistic.

The results of the numerical study by Ceccotti and Karacabeyli [16] show that for Case 1 design, R = 3 for plywood only, the median failure level of peak ground acceleration is 0.68 g, whereas the lower quartile is 0.50 g. For R = 3, plywood plus GWB, the median failure acceleration was found to be 0.68 g, the lower quartile 0.60 g; and for R = 2, plywood plus GWB, the median failure acceleration was 0.50 g, the lower quartile 0.45 g. This compares with design PGA of 0.23 g, the site-specific design ground acceleration with a 475 year return period as specified in the NBCC 1995. It also largely satisfies the PGA of 0.49 g for the state of "near collapse" for the upcoming NBCC 2005 zoning map having a 2500 year return period.

It needs to be noted, however, that this was an engineered building, but the methodology could be applied also to conventional construction.

Static analysis procedure

Aside from the size of the earthquake, three parameters play a key role in the seismic resistance of wood frame buildings: resistance to base shear, shear resistance of walls, and resistance to uplift and overturning. Of these three, the shear resistance is the most critical and only it will be treated here in some detail. The other two, anchor bolts and uplift resistance, are assumed to be adequately provided for.

Shear resistance of walls

The main structural properties and model assumptions for the static analysis are presented in the Appendix.

Conventional construction according to modern building codes requires braced walls, meeting minimum requirements with respect to sheathing length and type, to be spaced at regular intervals in the building. Traditionally, in high seismic regions, 25% of a braced wall supporting the roof and one storey had to be braced and 40% of a braced wall supporting the roof and two storeys had to be braced.

The relative shear resistance of a building can be expressed as the calculated resistance of the braced walls divided by induced seismic shear force calculated for a constant spectral acceleration. It is assumed that shear walls develop their full capacity. If the necessary conditions for full capacity are not present, the member capacities have to be reduced as specified in the CWC design guide [9] or the standard CSA O86-01 [11]. The relative shear resistance is given by

Relative shear resistance = wall capacity x 2 / (V - resistance of partitions)

Wall capacity is based on exterior wood sheathed walls with the minimum required percentage of sheathing and V is the calculated base shear force on the building for Sa = 1.0 g. An allowance could be made for the contributions of interior partitions to the seismic resistance of buildings. However, when Rd = 3 is used in the calculations as was done here, the NBCC 2005 [2] requires that the contribution to the seismic resistance of gypsum wall board on the partitions and on the interior face of shear walls be neglected.

The relative seismic resistance was used to compare the effects of building area, building height and building aspect ratio (the ratio of the building long dimension to the building short dimension). The comparisons are summarized in Figure 1. The following assumptions were used in the comparisons:

- Only wood structural sheathing on the exterior walls contributed to the braced wall shear resistance
- Wood structural sheathing was 9.5 mm thick with nails spaced at 150 mm at the panel edges
- 25% of the walls were sheathed on the bottom storeys of 1 and 2 storey buildings
- 40% of the walls were sheathed on the bottom storey of a 3 storey building.

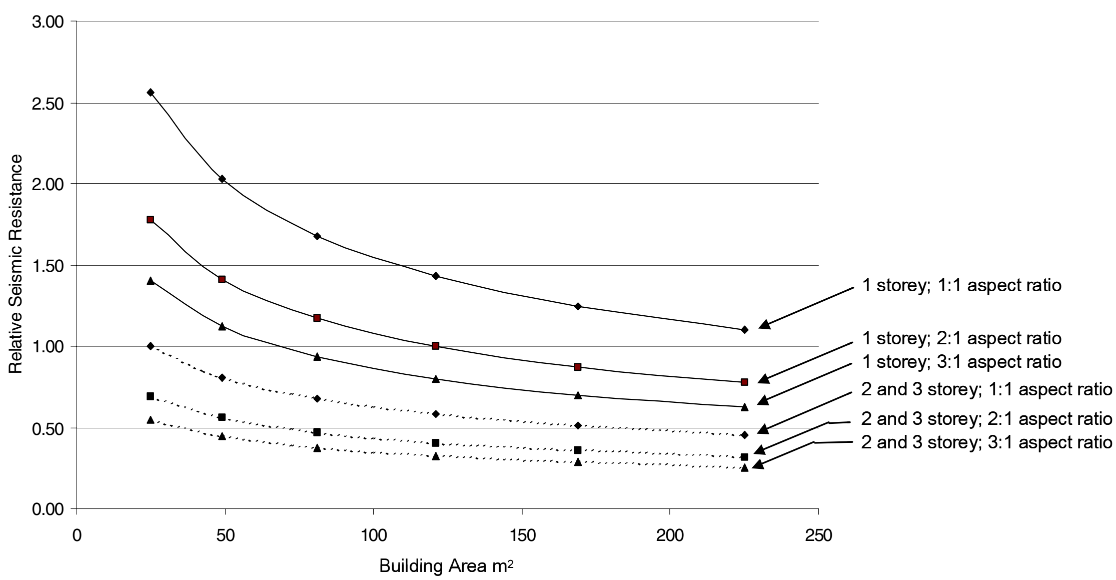

Figure 1: Relative Seismic Resistance of 1, 2 and 3 Storey Buildings Based on Static Calculations

The results show that large variations exist in the relative shear resistance of buildings. The 2 storey buildings with 25 % shear wall have essentially the same relative resistance as the 3 storey buildings with 40 % shear walls, but significantly less than the 1 storey building. As well, increasing building area and building aspect ratios decreases the relative resistance. These results also show that the various wood frame buildings have different margins of safety against collapse for the same assumed ground shaking and these margins of safety depend on the number of storeys, size of building and building aspect ratio. It needs to be emphasized, however, that any contributions of the interior and exterior finishes to the seismic resistance of the building have not been accounted for in this assessment.

COMPARISON WITH OBSERVED SEISMIC BEHAVIOUR

Full-scale shake table testing

The results of the shake table tests at the University of California San Diego (UCSD) for Phase 8, "conventional construction", [12] are compared with the results from the static analysis prescribed in the coming 2005 NBCC. Pertinent data is presented above with the description of the specimen for the UCSD test under "Results from full-scale shake table tests".

Length of shear wall calculated for Sa(0.2) = 1.0 and 150 mm nail spacing: S = 1.48 m per wall.
Available shear wall = 13'-0" = 3.46 m. Thus,

Sa max = 3.46 / 1.48 = 2.6 g

Since spectral acceleration Sa is approximately double the peak ground acceleration PGA,

PGA max predicted = 1.3 g (approx.)

Judging from the capacity spectrum of the UCSD results [12], the Phase 8 specimen could easily have withstood the 0.89 g shaking, just as Phase 9 did, and probably more. Although the test data is not available for a direct comparison, the result nevertheless point in the right direction.

Field observations of earthquake damage

The importance of the three main parameters listed above can be seen in observed behaviour of wood-frame buildings in earthquakes. Inadequate base shear resistance due to absence of anchor bolts was observed in a number of failures of houses sliding off the foundations in the San Fernando (California) earthquake of 1971, the Edgecumbe (New Zealand) earthquake of 1987 and the Loma Prieta California) earthquake of 1989. Since then, building codes have mandated anchor bolts and thus this type of failure should not occur in newer houses.

Lack of adequate shear resistance in the walls has led to a number of failures in the above earthquakes, as well as in the Northridge (California) earthquake of 1994 and the Kobe (Japan) earthquake of 1995. In the Loma Prieta earthquake, in the Marina district of San Francisco, and in the Northridge earthquake a number of multi-storey apartment blocks collapsed. This was attributed to the weak-storey effect in the lowest level of the building, where the seismic capacity was significantly lower than that of the rest of the building. Since the seismic shear of multi-storey buildings is largest at the base, failure will be initiated there and if too weak, collapse will follow. Again, building codes now address this issue by specifying a minimum percentage of wall length in the first storeys of two-storey buildings, and a larger minimum percentage of braced walls for three-storey buildings.

In a previous survey of the performance of wood frame construction in earthquake [18] it was found that in a number of earthquakes in locations where wood- frame housing was prevalent (Canada, USA, New Zealand, some in Japan) the number of occupants killed in wood frame houses that collapsed was very small. From earthquake site-visit reports and available shaking data it was estimated that conventional wood frame houses could withstand seismic shaking with PGA of 0.5 to 0.6 g, provided that no major deficiencies were present such as weak first stories or lack of anchor bolts. Such a PGA would correspond approximately to Sa(0.2) equal to 1.0 to 1.2 g. The results of the present study also show that with varying minimum percentages of shear wall, some buildings should indeed be able to survive seismic shaking of Sa(0.20) equal to 1.0 g.

It is worth noting that the three-storey buildings, and to a lesser degree the two-storey ones, suffered the most serious distress in the last few earthquakes in California. This corresponds to the trend shown by the numerical calculations, that larger buildings with more storeys have lower relative resistance than one-storey buildings. Building codes have responded to that by specifying minimum percentages of shear walls for two and three-storey buildings of conventional wood-frame construction; the IRC 2003 requires engineering design for three-storey buildings in high seismic areas.

DISCUSSION OF RESULTS

Within the geometric and material parameters considered, the shake table tests of full-scale wood-frame specimens have shown that these type of structures are able to withstand seismic shaking with PGA of 0.5 g or Sa(0.2) of 1.0 g, and well above that. This is in general agreement with the observations and conclusions drawn from damage assessments in recent earthquakes. The parametric study using code-based static analysis agrees with this assessment for, one storey and square smaller buildings. However, the static analysis shows that the seismic demand is significantly higher in larger two- and three-storey buildings, particularly in buildings with large aspect ratios. The current Part 9 of NBCC 1995 does not differentiate among wood-frame buildings of various heights and aspect ratios and thus did not address this issue. The CWC design guide, however, published in 2001, and the upcoming NBCC 2005 take an important step in dealing with this potential deficiency by specifying a minimum of 25% shear walls in the first storey of two-storey buildings and 40% for three-storey ones. The UBC 1997 has similar requirements. Other recent building codes require an even higher percentage of shear walls for conventional wood-frame buildings. The IBC 2000 in the seismic zone $0.75 \le Sds < 1.0$ requires 37 % for one storey, 69 % for the first of two storeys, and engineered design for the first storey of three-storey buildings.

Since the assessment by the simplified static analysis does not include the contribution of interior partitions and exterior finishes, inadequate resistance capacities may be assigned to the walls. The shake table test results of the CUREE project and at the University of British Columbia have demonstrated that stucco increases the seismic resistance dramatically, and interior partitions undoubtedly also contribute. Neither contribution has so far been satisfactorily quantified or codified; this aspect clearly needs more attention.

SUMMARY AND CONCLUSIONS

Results from shake table tests of full-scale specimens and cases of detailed non-linear analysis have shown that for the range of conventional wood-frame buildings tested, structural collapse is well above 0.5 g PGA or Sa(0.2) = 1.0. This corresponds in general with the observations of earthquake damage of wood-frame buildings, that those without major weaknesses, such as weak or soft storeys and lack of anchor bolts, are able to survive ground shaking of the order of 0.5 and 0.6 g without collapse.

Parametric studies with code-prescribed static analysis of 1, 2 and 3 storey wood-frame buildings and with resistance properties specified in the CWC design guide give lower relative resistance values for large and multi-storey buildings than for small and one -storey buildings. Those results should be taken as indicators of behavioural trends, rather than definitive final results since 1) the contributions of the finishing materials, such as exterior stucco and interior gypsum wall board, have not been included and 2) the static analysis method can only approximate the dynamic nature of the process.

Improved quantitative assessment of exterior and interior finishes such as stucco and gypsum wall board and their contribution to the overall seismic performance of a wood-frame building are needed.

REFERENCES

1. National Building Code of Canada (NBCC), 1995 Edition, National Research Council of Canada, Ottawa ON.
2. National Building Code of Canada (NBCC), Draft of the 2005 Edition, Ottawa ON, (to be published).
3. Uniform Building Code, 1994 Edition, International Conference of Building Officials, Whittier, CA.
4. Uniform Building Code, 1997 Edition, International Conference of Building Officials, Whittier, CA.
5. International Building Code (IBC), 2000 Edition, International Code Council, Falls Church, VA.
6. New Zealand Standard NZS 3604:1999. Timber Framed Buildings. Standards New Zealand, Wellington, New Zealand.
7. Kesik TJ. "Canadian Wood-frame House Construction." Canada Mortgage and Housing Corp. (CHMC), Ottawa ON.
8. Commentary J. "Design for seismic effects." Draft of Supplement to National Building Code of Canada 2005 (Part 4), National Research Council of Canada, Ottawa ON, (to be published).
9. Engineering Guide for Wood Frame Construction, Canadian Wood Council (CWC), Ottawa ON, 2001.
10. Ni C. and Karacabeyli E. "Capacity of Shear Wall Segments without Hold-Downs." Wood Design Focus, Summer 2002, 10 – 17.
11. CSA O86-2001. Engineering Design in Wood (Limit States Design) – Structures (design). Canadian Standards Association, Toronto ON.
12. Fischer D, Filiatrault, A, Folz B, Uang C-M and Seible F. Shake Table Tests of a Two-Story Woodframe House. Department of Structural Engineering, University of California, San Diego; CUREE publication No. W-06, 2001.
13. Ventura CE, Taylor GW, Prion HGL, Kharrazi MHK and Pryor S. Full-Scale Shaking Table Studies of Woodframe Residential Construction. Proceedings, 7th US National Conference on Earthquake Engineering, July 21 – 25, 2002, Boston, MA, p.528.
14. Kohara K and Miyazawa K. Full-Scale Shaking Table Test of Two Story Wooden Houses. Proceedings of the International Conference on Timber Engineering, 1998, Montreux, Switzerland, Vol. 2, p. 548-555.
15. Mosalam KM, Mahin S, and Naito C. Shake Table Test of a Multi-Story Apartment Building with Tuck-Under Parking. University of California, Berkeley, CUREE Publication No. W-19, 2002.
16. Ceccotti A and Karacabeyli E. Validation of seismic design parameters for wood-frame shearwall systems. Can. J. Civ. Eng., 2002; 29(3): 484 – 498.

17. Folz B, Filiatrault A, Uang C-M and Seible F. Blind Predictions of the Seismic Response of a Two-Story Woodframe House: An International Benchmark. Division of Structural Engineering, University of California, San Diego, Report No. SSRP – 2001/15, 2001.
18. Rainer JH and Karacabeyli E. Performance of wood-frame construction in past earthquakes, Proceedings, 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 30 Jan. – 4 Feb. 2000, Paper no. 2454.

APPENDIX

Structural properties and model assumptions for static analysis

Building and Design Parameters:

Storey Height =3 m
Roof Snow Load = 1 kPa (0.25 kPa for seismic)
Roof Dead Load = 0.5 kPa
Floor Dead Load = 0.5 kPa
Wall Dead Load = 0.32 kPa
Floor Partition Load = 0.5 kPa

Exterior Walls: Braced Walls on Foundations

S-P-F studs spaced at 400 mm
9.5 mm thick exterior sheathing (Oriented strand board (OSB) or plywood)
2 in. common (2.84 mm dia.) sheathing nails
Sheathing nails spaced at 150 mm at panel edges
Exterior panel edges blocked

Seismic Forces

According to NBCC DRAFT 2005, base shear (V):

Period T = 0.05 (hn)^3/4
= 0.11 s, 0.19 s, 0.26 s for 1, 2, 3 storey buildings, respectively,
Soil Class C, Fa=1.0, Rd = 3, Ro = 1.7, I = 1.0, spectral acceleration Sa =1.0
S (0.2) = Fa x Sa(0.2) = 1 x 1 = 1
Base shear V = 2/3 S(0.2) I W/(Rd Ro) = 0.131 W