8th Canadian Conference on Earthquake Engineering / 8ième Conférence canadienne sur le génie paraséismique
Vancouver – 1999

Seismic design considerations on wood frame structures

Karacabeyli, E.[1], Ceccotti, A.[2]

ABSTRACT

The results from the time-history non-linear dynamic analyses of a four-storey wood-frame platform structure are presented. These analyses are performed using a structural analysis program in which the in-elastic behaviour of the shear walls is represented by a hysteresis model that was based on reversed-cyclic test results on full-scale shear walls. The dynamic performance of this structure was evaluated by using twenty-eight accelerograms. The results confirm that the current Canadian seismic force modification factor (R=3), for the lateral resisting systems comprising of plywood nailed shear walls. The results also show that the presence of walls sheathed with GWB has a positive influence on the response of the structure which was designed considering only plywood shear walls. An alternate seismic force modification factor (R=2) which accounts for the contribution of the GWB in design is found to be appropriate.

INTRODUCTION

The lateral load resisting systems for most wood-frame buildings rely on nailed shear walls sheathed with plywood or oriented strand board (OSB). Related to their seismic design, a considerable amount of research was undertaken in consultation with the structural engineering community to address a number of issues such as confirmation of force modification (R) factors, the contribution of gypsum wall board (GWB) walls to the lateral load resisting system, design of shear walls and diaphragms with openings, the spacing of anchor bolts and placement of hold-down connectors. Based on the results from these research studies, a number of code change proposals have been submitted to the code committees for possible implementation.

For timber structures, the R factors were first implemented in the 1990 edition of the NBCC (CCBFC 1995) which stipulates that large residential and non-residential timber structures (exceeding 600 m^2 in building area and three storeys) shall be designed and detailed according to the Canadian Code for Engineering Design in Wood, CSA O86.1 (Canadian Standards Association 1994).

In order to assess the appropriateness of the R factors in the NBCC, and in EC8 (Eurocode 8, 1993), time-history dynamic analyses using twenty-eight earthquake accelerograms and a hysteresis model for the shear wall components were performed on a four-storey platform frame wood structure. This structure was recommended by the members of the Wood Frame Committee of the Structural Engineering Consultants of B.C. (SECBC, Continuing Education for Engineering & Architecture, UBC 1995). Based on these analyses, recommendations about R factors have been made. A comparison between the Canadian force modification factor and European seismic behaviour factor for lateral load resisting systems with nailed shear walls was also made.

TESTING

A comprehensive database was established at Forintek by testing wood-frame shear walls under monotonic and cyclic displacement schedules. The test program included wood frame shear walls sheathed with plywood, oriented strand board (OSB) and/or GWB. The detailed description and results of the test program are given in (Karacabeyli and Ceccotti 1996).

In establishing the skeleton curves for the hysteresis model for the shear walls, the effect of cyclic test schedule may play an important role. An examination of test results obtained with several cyclic test schedules revealed that the possible differences may be due to a) the different energy demand which depends on the magnitude and number of cycles; and b) the rate of loading (the velocity of the displacement). While a greater energy demand appears to result in a decrease in

[1] Wood Engineering Scientist, Forintek Canada Corp., 2665 East Mall, Vancouver, B.C. Canada, V6T 1W5

[2] Professor, Civil Engineering Dept., University of Florence, 3 Via di Santa Marta, Firenze 50139 Italy

maximum loads and ultimate displacements, a faster rate of loading results in an increase on those properties (Karacabeyli and Ceccotti 1998).

MODELLING

The approach for testing and analysis of the multi-storey timber structures is given in Figures 1 to 4. A detailed description of this approach is given in Ceccotti and Karacabeyli (1996).

The selected building was designed in accordance to the NBCC provisions for the City of Vancouver. In this paper, a two dimensional dynamic analysis of one of the shear walls parallel to the short dimension of the building was performed. Concrete floor topping was considered in the weight calculations. In the short direction, the building was symmetrical, and consequently torsional effects are not considered.

The study consisted of the following steps:

a) shear wall specimens were tested (Figure 1) under monotonic and cyclic displacement schedules;

b) a hysteresis model was fitted to the above cyclic test data (Figure 2). For systems containing shear walls sheathed with a combination of plywood and gypsum wall board (GWB), individual skeleton curves were superimposed. This method of superimposition has been shown (Karacabeyli and Ceccotti 1996) to be valid for displacements up to approximately 50 mm for monotonic, and up to 30 mm for the stabilized envelope (3rd in cyclic tests) curves (Figure 3);

c) the hysteresis model was employed in a time-step dynamic analysis for twenty-eight different earthquake accelerograms (Figure 4). The peak ground acceleration for each accelerogram was scaled upwards until the ultimate displacement is achieved. This acceleration is then called "A_u". The ultimate displacement, used as the collapse criteria, is defined as the displacement at 80 percent of the maximum load on the descending portion of the skeleton curve;

d) under the Canada/Japan Agreement in cooperation with Science and Technology, staff from Building Research Institute, Disaster Prevention Centre and Forintek Canada Corp. carried out shake table tests and pseudo-dynamic tests. The results of these tests confirmed that the theoretical model reasonably predicts the behaviour of a shear wall subjected to a selected earthquake record.

TIME-HISTORY DYNAMIC ANALYSIS

Time-history non-linear dynamic analyses was performed using a structural analysis program in which the in-elastic behaviour is modelled by a hysteresis model developed at the University of Florence (Ceccotti et.al. 1994). In the analysis, the skeleton curve for the hysteresis model is selected based on the 5th percentile (determined by assuming a 10 percent coefficient of variation and a normal distribution) of the first envelope curves obtained in the cyclic tests. No further adjustments for safety were used. The analyses were performed using twenty-eight earthquake accelerograms of which six were accelerograms from real earthquakes; the remaining twenty-two were modified accelerograms to fit the Vancouver area design spectrum.

Three design cases (Figure 5) were considered:

Case 1: R=3; Designed and analyzed <u>only</u> considering plywood shear walls. The effect of the GWB walls was neglected in the dynamic analysis.

Case 2: R=3; Designed <u>only</u> considering plywood shear walls. In the dynamic analysis, considered plywood shear walls and <u>accounted for</u> the effect of the GWB walls.

Case 3: R=2; Designed and analysed considering plywood and GWB shear walls. This case is proposed by the Wood Frame Committee of the SECBC (SECBC, Continuing Education for Engineering & Architecture, UBC 1995).

For Cases 2 and 3, the ratio of GWB to plywood walls was kept at 2.5, 2.5 and 5.0 for the first three storeys, respectively. No restriction was applied for the fourth floor. A 1 kN/m factored shear resistance for GWB walls is used in the design.

For Case 1, the European (Eurocode 8, 1993) seismic behaviour factors (q) for lateral resisting systems with nailed shear walls were also determined for the twenty-eight accelerograms. The factor "q" is calculated as the ratio of A_u and the acceleration which caused the yielding of the structure, A_y (as defined in CEN 1994).

The fundamental period of vibration of the structure (T_0) is calculated by using the NBCC, and also by dynamic analysis. The value of T_0^{NBCC}=0.2 sec was found to be much less than those (Figure 9) found for the three cases by dynamic analysis. In determining the design shear force, we used T_0^{NBCC}=0.2 sec.

RESULTS

The results of non-linear dynamic analysis are shown in Figures 6, 7 and 8 where the peak ground accelerations (A_u) that "caused" the inter-storey drift to reach the shear wall's ultimate displacement are shown against the twenty-eight accelerograms, and also against the Peak Ground Acceleration (PGA_{CODE}) given in the NBCC. These results lead to the following conclusions:

a) For Case 1, all values of A_u were found to be greater than PGA_{CODE} which confirms that the current force modification factor, R=3, is appropriate for plywood nailed shear walls. The median value of A_u is found to be three times the PGA_{CODE}.

b) For Case 2, most values of Au were found to be generally greater than those found for Case 1 suggesting that the existence of GWB walls did not impair the lateral resistance of the structure. In other words, GWB contributed positively to the response of the structure compared to Case 1 where only plywood shear walls were considered.

c) For Case 3, all values of A_u were also found to be greater than the PGA_{CODE} which shows that the alternate force modification factor, R=2 is appropriate. Although the median value of A_u was found to be smaller than that found for Case 1, the lower quartile values of A_u for Case 1 and Case 3 were similar. This is due to the smaller variability obtained in the results of Case 3.

d) Most values of q (Figure 9) were found to be greater than 3 which confirms that the seismic behaviour factor in Eurocode 8, q=3, for plywood or OSB nailed shear walls is appropriate. The median q was found to be between 5.0 and 6.0.

Preliminary results from a shake table tests confirmed the model predictions.

CONCLUSIONS

The results confirm the current Canadian seismic force modification factor (R=3) and the European behaviour factor (q=3) for lateral load resisting systems comprising of plywood nailed shear walls.

The results also show that the presence of walls sheathed with GWB has, in general, a positive influence on the response of the structure which was designed considering only plywood shear walls. An alternate seismic modification factor (R=2, recommended by the SECBC) which accounts for the contribution of the GWB walls in design is found to be appropriate.

REFERENCES

Canadian Commission on Building and Fire Codes. 1995. National Building Code of Canada. National Research Council Canada.

Canadian Standards Association. 1994. O86.1-94. Engineering Design in Wood (limit States Design). CSA, Etobicoke, Ont., Canada.

Ceccotti, A., and Karacabeyli, E. 1996. Testing and Analysis of Multi-storey Wood-frame Buildings. Proceedings, CIAS International Seminar on Evolution in Experimentation for Constructions. Malta.

Ceccotti, A., Vignoli, A., and Giordano, S. 1994. Seismic Tests on Full-Scale Timber Structures. Proceedings. Pacific Timber Engineering Conference; Goald Coast, Australia:1:232-240.

CEN TC124.117. 1994. Timber Structures - Test methods - Cyclic testing of joints made with mechanical fasteners. CEN, Bruxelles, Belgium.

EUROCODE 8. 1993. Design of Structures in Seismic Regions (draft). Draft ENV 1998-1-1 CEN, Bruxelles, Belgium.

Karacabeyli, E., and Ceccotti, A. 1998. Nailed Wood-frame Shear Walls for Seismic Loads: Test Results and Design Considerations. Paper submitted to Secretariat of Structural Engineers World Congress (to be held in San Francisco, USA).

Karacabeyli, E., and Ceccotti, A. 1996. Test Results on the Lateral Resistance of Nailed Shear Walls. Proceedings. International Wood Engineering Conference, New Orleans, USA. pp:V2,179-186.

SECBC, 1995. Conventional Wood Frame Multi-Unit Residential Buildings. Continuing Education for Engineering and Architecture, UBC, 1995. Proceedings of a Workshop on Seismic Design of Buildings. University of British Columbia.
Yasumura, M. 1991. Seismic Behaviour of Wood-Framed Shear Walls. Proceedings of CIB W18A. United Kingdom.

ACKNOWLEDGEMENTS

Forintek Canada Corp. would like to thank Natural Resources Canada (Canadian Forest Service), and industry members for their guidance and support for this research.

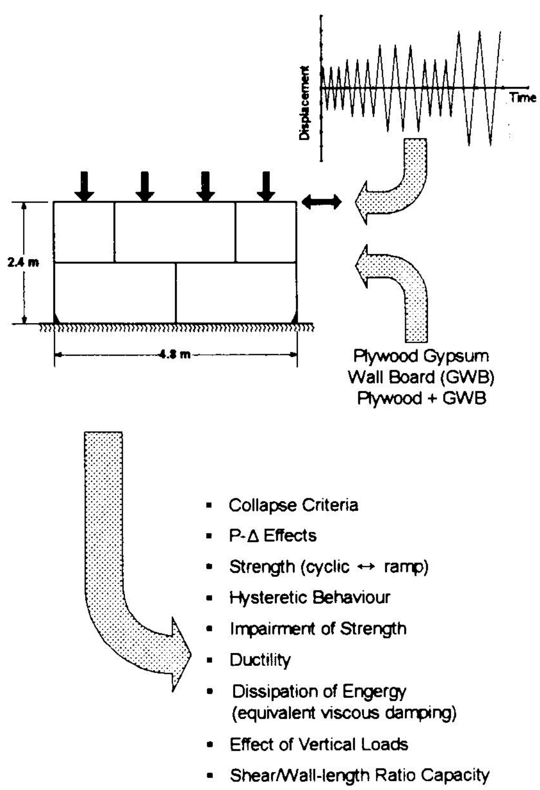

Figure 1 Cyclic testing of shearwalls

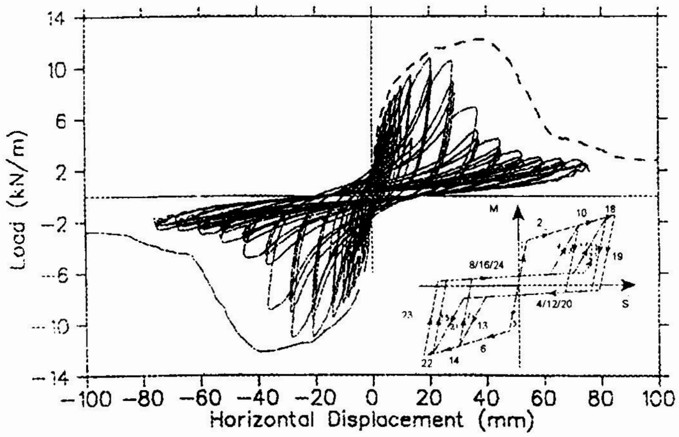

Figure 2 Typical monotonic and cyclic test data, the Hysteresis Model

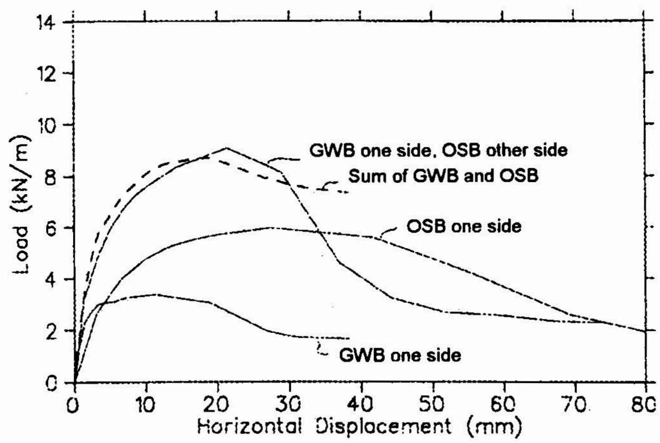

Figure 3 Contribution of gypsum wall board to the stablized (3rd envelope curves) load carrying capacity when used in combination with OSB

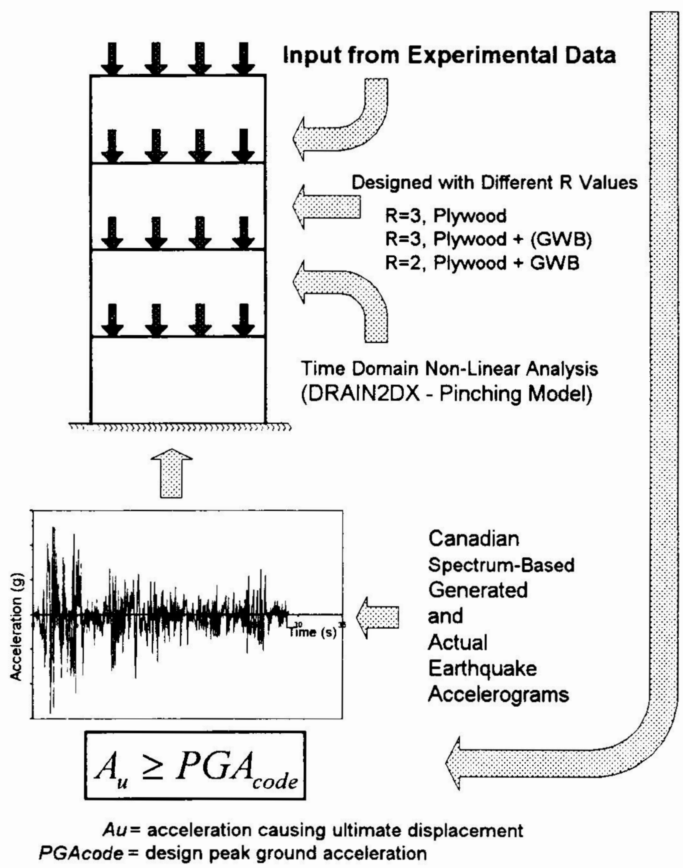

Figure 4 Dynamic analysis

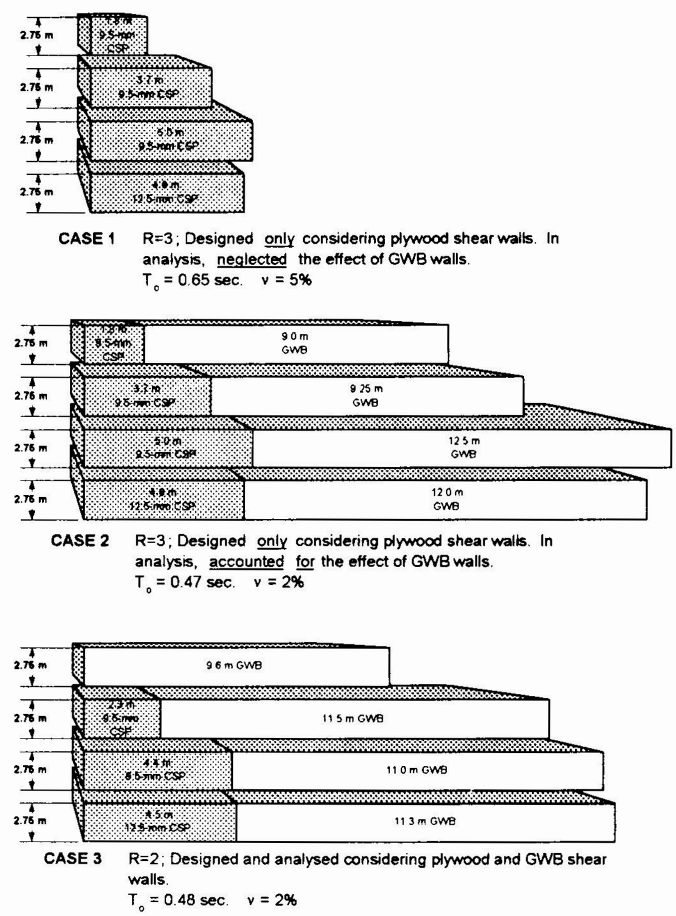

Figure 5 Three cases considered in the design and analysis

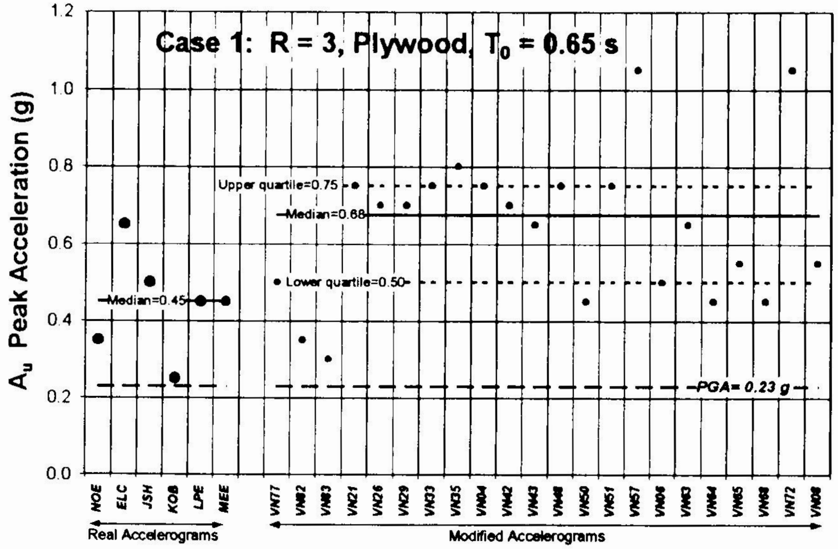

Figure 6. Results for Case 1

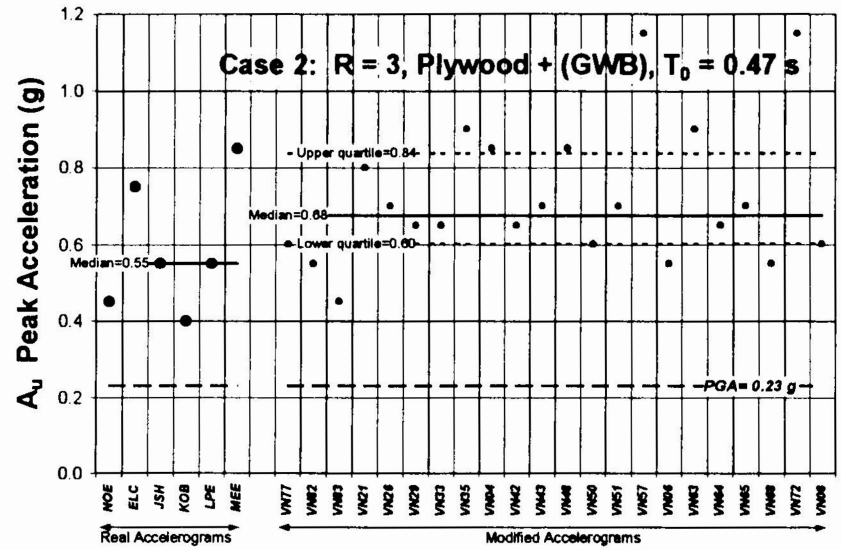

Figure 7. Results for Case 2

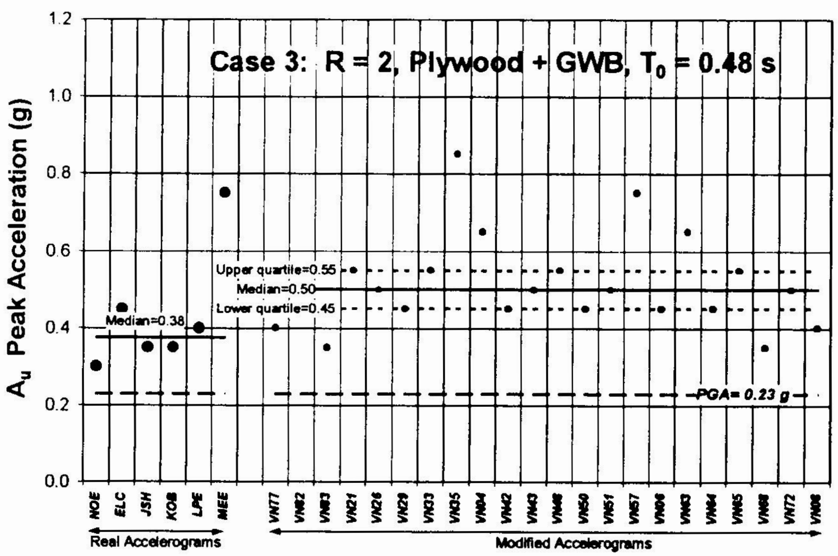

Figure 8. Results for Case 3

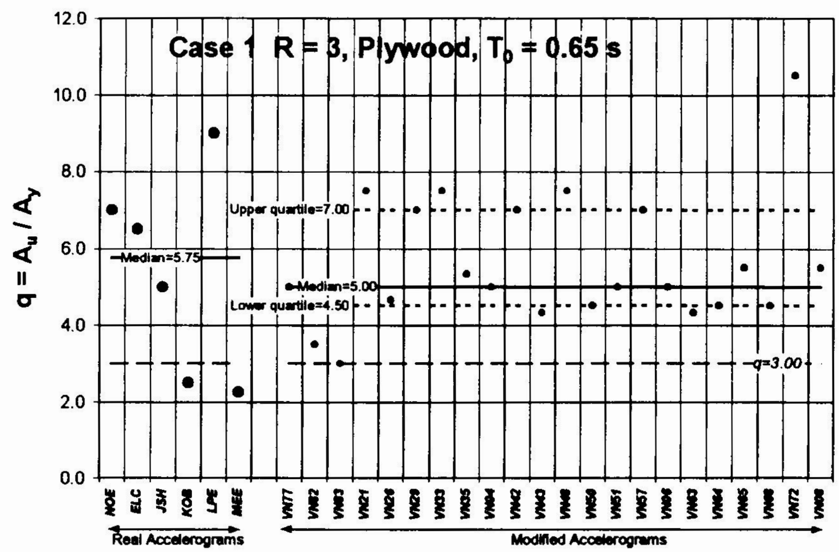

Figure 9. European behaviour factor for Case 1