Modelling the seismic response of light-timber-framed buildings

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ABSTRACT: Timber-framed buildings have become the focus of extensive research following their poor response in the Northridge and Kobe earthquakes. A series of research projects are being conducted by the California Universities for Research in Earthquake Engineering (CUREe) to improve the hazard mitigation for these buildings. In the past, a variety of analytical models have been developed to predict the seismic response of timber-framed buildings. Their accuracy is being assessed as part of an international benchmark, which is comparing their (blind) predictions of the response of a full-scale two-storey residential building tested on a shaketable.

This paper presents the method and results of the static and time-history modelling of the CUREe benchmark building. The building deformations were found to be similar to those expected for New Zealand buildings subjected to design level earthquakes.

1 INTRODUCTION

Over the last two decades or so, there has been considerable research carried out to investigate the performance of individual timber shear walls in resisting the lateral forces applied to timber structures from winds and earthquakes [e.g. International Timber (or Wood) Engineering Conference, 1988, 1991, 1994, 1997, 2000; Pacific Timber Engineering Conference, 1984, 1989, 1994, 1999]. To a lesser extent, there has also been research carried out to investigate the wind and seismic performance of complete houses, particularly in Japan.

In 1991, a workshop was held in the UK [Gupta & Moss 1991] to bring together a number of timber researchers from around the world with a view to arriving at a consensus on how best to share data from full scale house tests. It was not until after the Northridge earthquake in California that any large scale test programs have actually been carried out. An extensive research program [CUREe 2000] has been undertaken by CUREe (a consortium of Californian Universities), with tests ranging from nail joint tests to individual shear walls to shaketable tests of a full scale two-storey residential building [UCSD 2000]. As part of this project, a number of research groups from around the world were invited to participate in a blind prediction of the results of one phase of the research. This paper outlines the results of analyses carried out by the authors.

2 BUILDING DESCRIPTION

A two-storey residential building was constructed on the UC San Diego uni-axial shaketable and subjected to a series of vibration, static and earthquake tests. These tests were conducted at various stages of construction to study the effects of the different claddings and components. Low-level vibration tests were conducted before and after each test to monitor the change in natural period and damping. The building was reconstructed after some tests so the damage didn't affect the results of the subsequent tests.

Architectural details of the building chosen for the benchmark project are shown in Figure 1. The shaketable moves along a North-South axis, so the identical North and South walls only contribute to the torsional stiffness of the structure. The East and West walls were identical in the upper storey but there were different openings in the lower storey. The nail spacing for the cladding panels was reduced in the lower storey of the East wall so its strength was similar to that of the West wall. (The stiffness was different because the panel thickness was the same on all four walls.)

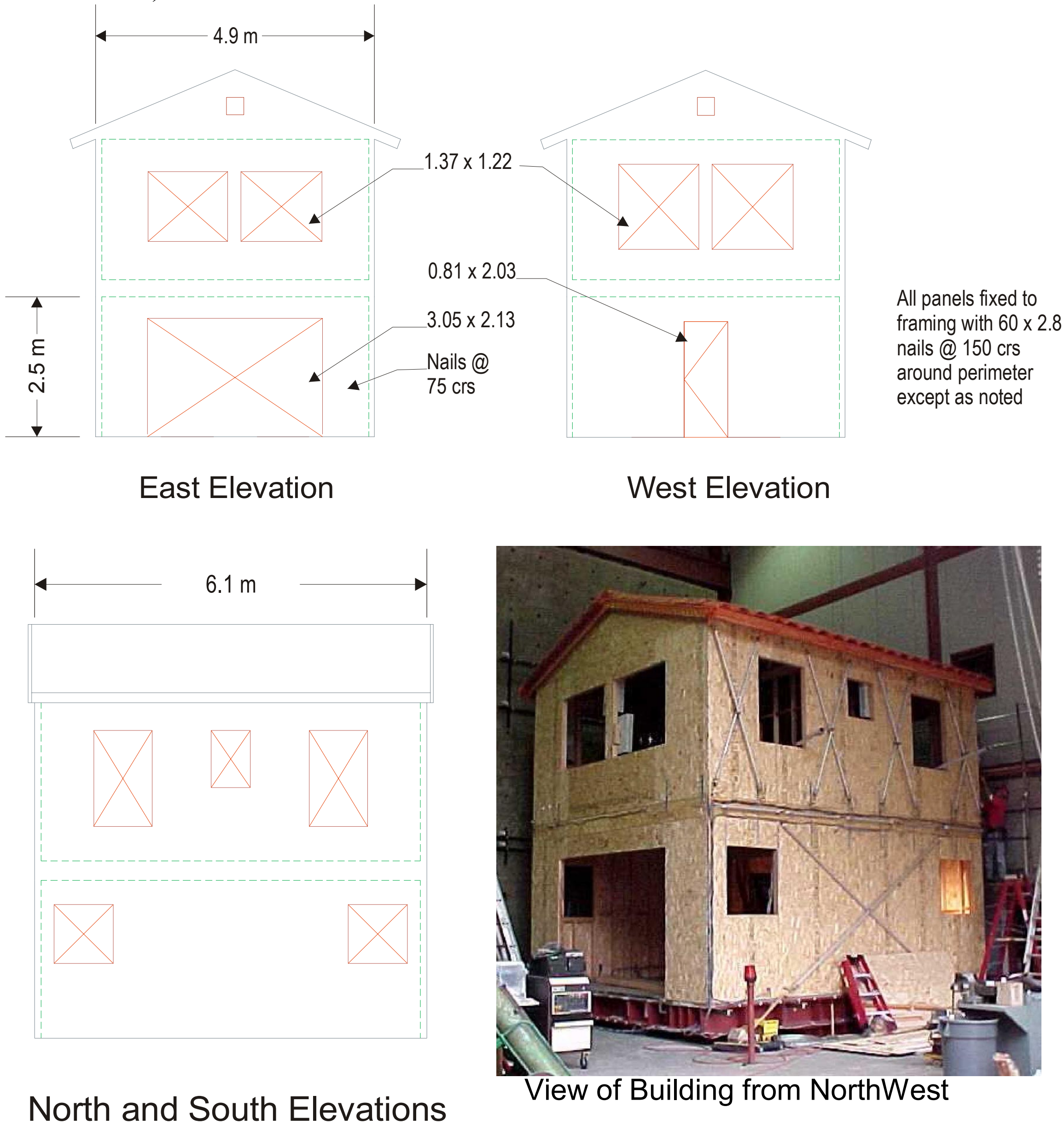


Figure 1 Architectural details of the UCSD Phase 9 Test Building

None of the internal walls were lined in the test building and there was no internal lining on the exterior walls. A series of weights were attached to the floor and walls of the upper storey so the total weight of the building was similar to a fully finished building.

3 MODELLING ASSUMPTIONS

As the interior walls were not lined, it was assumed that their frames would make no significant contribution to the overall stiffness and strength of the building.

The floor and roof diaphragms were assumed to be rigid, and their total weights were estimated to be 36 kN and 54 kN respectively.

Sheathing shear deformations were included in the analysis but hold-down deformations were assumed to be negligible because rigid connectors were installed at the corners and wall openings. Stud extension and contraction deformations were also assumed to be negligible.

3.1 *Material properties*

The building was clad with oriented strand board (OSB), a panel product manufactured by gluing strands of timber together. This has properties that are similar to medium density fibreboard (MDF) when load is applied parallel to the strands and is similar to plywood when loaded perpendicular to the strands.

The two most significant properties required for the analysis were the modulus of rigidity of the panels and the load-slip response of the nailed connection between the OSB panels and the timber framing. A small number of load-slip test results were provided by UCSD for the nailed connection. These had considerable scatter for both monotonic and reverse-cyclic tests but their average response was similar to nails in New Zealand plywood [Dean 1988] when loaded perpendicular to the strands. (The ultimate strength for the 2.8 mm diameter nails was 1 kN and the initial stiffness was 3.4 kN/mm.) The strength and stiffness were approximately halved for the tests with the load parallel to the strands.

3.2 *Static analyses*

For the static analysis, the building was modelled using single wall elements on each of the four sides of the building at each level as shown in Figure 2. The model was constructed within an Excel spreadsheet and used the Solver utility to balance the forces on the East and West walls against the applied force and to balance the torsion against the North and South wall forces.

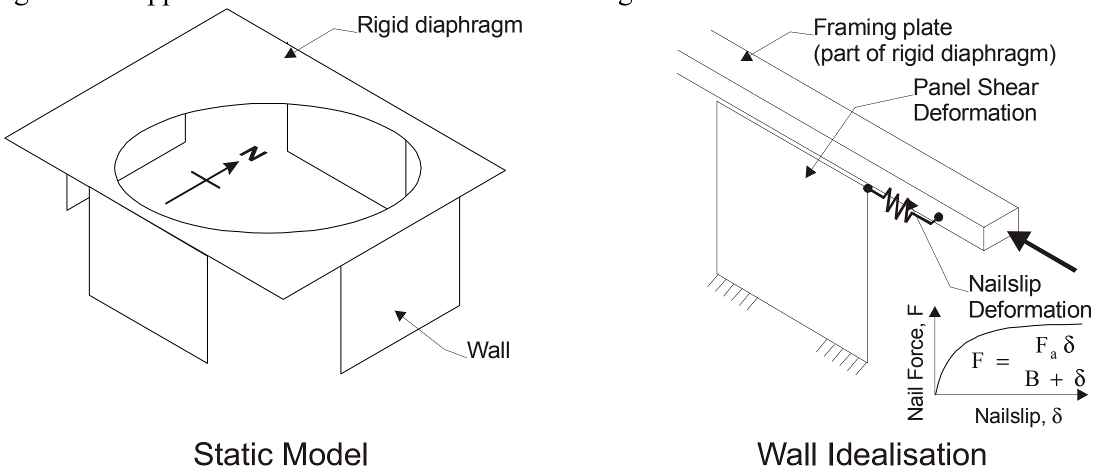


Figure 2 Static model of one level in the benchmark building

The wall strength-deformation responses were calculated using Equation 1 [Deam 1997] which approximates the wall strength, F, at deformation δ for a single panel of width, b, and aspect ratio, $\alpha = h/b$, that is attached to its perimeter framing with nails spaced, s, mm apart that have an asymptotic nail strength, F_a and nailslip e_n at $F_a/2$. (See Figure 2.)

$$F \approx \frac{1.1 F_a \, b/s \times \delta}{0.85 \times 2e_n (1+\alpha) + \delta} \tag{1}$$

The nailslip test responses provided for the analysis showed significant strength and stiffness differences between the perpendicular- and parallel-to-grain responses. The additional nailing along the vertical edges of the panels (assumed parallel-to-grain) was expected to provide a similar joint strength to that of the horizontal edge joint (perpendicular-to-grain).

The building was modelled using three types of wall. The openings in north and south walls were small so their strength and stiffness were calculated using the full wall length, giving 34 kN and 4.1 kN/mm respectively for both walls in both storeys. They are not subjected to large loads because the centre of mass is close to the centre of rigidity in both storeys. The panels were assumed to have an aspect ratio of $\alpha = 2$.

The large openings in the lower storey portions of the East and West walls would provide little coupling between the two halves of these walls so they were modelled as separate (identical) panels. The strengths of the two walls were similar (29 and 27 kN for East and West walls in the lower storey and 13.5 kN for the upper storeys). The West wall stiffness (3.8 kN/mm) was almost double that of the East wall stiffness (2.2 kN/mm) because the East wall panels had a greater aspect ratio of ($\alpha = 2.7$) and the sheathing stiffness was only half that of the West wall.

The upper storey openings in the East and West walls were slightly more complex to model because the panels below the openings provide additional stiffness. The panel aspect ratio was reduced to $\alpha = 1.0$ to represent the contribution of these panels below the openings. Only the two 0.92 m wide panels at the sides of the windows were assumed to contribute to the asymptotic strength.

3.3 *Dynamic analyses*

The dynamic analyses were carried out using the program "RUAUMOKO" developed at the University of Canterbury [Carr 2000].

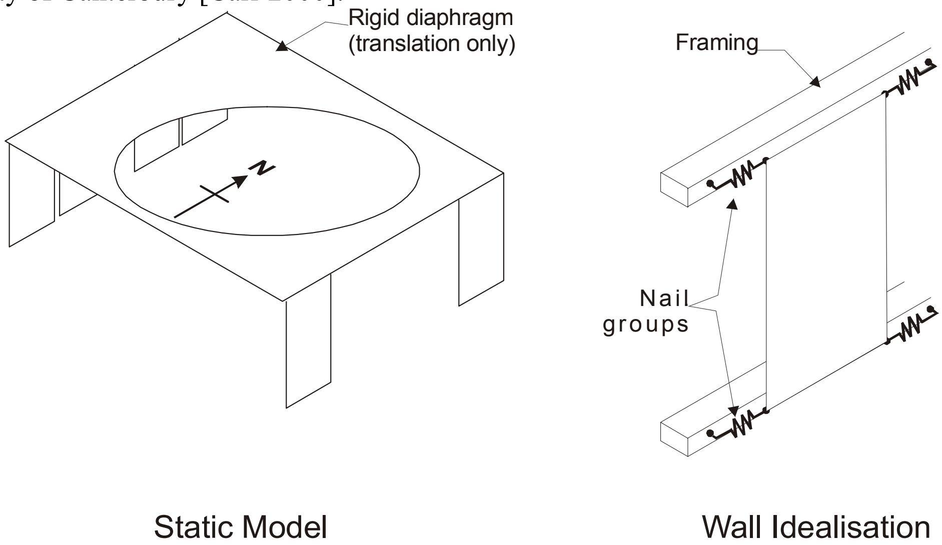


Figure 3 Dynamic model of one level in the benchmark building

For these analyses, it was assumed that the structure to be modelled consisted of the OSB panels on the outside of the building [UCSD 2000], together with the top and bottom plates and the studs at the edges of the panels. These studs were assumed to be double 100 x 50 studs for simplicity (this made some allowance for the fact that the intermediate studs with their nailing, were neglected). The panels above the door openings and above and below the window openings were neglected as they could be expected to provide only a minor contribution to the overall strength and stiffness.

The model used consisted of 100 × 100 mm timber frame members, fixed at the base, and rigidly connected to stiff beam members at the floor and roof levels to represent the floor diaphragm and the roof structure. These beam members were taken as being elastic with an elastic modulus of 10.3 GPa. The OSB panels were assumed to have a shear modulus value of 690 MPa. They were modelled as elastic quadrilateral elements. Since the program required the elastic modulus (E) and Poisson's ratio to be specified, it was necessary to calculate a value for E based on the required shear modulus and the upper limit in the program of 0.45 for Poisson's ratio. The nailed connection between the panels and the frame members was simulated by assuming that one quarter of the nails holding each panel could act at the corner of the panels. The accumulated nails were represented by spring members having hysteretic properties based on the cyclic nail joint test results (the Wayne Stewart hysteresis model in RUAUMOKO [Carr, 2000]) multiplied by the requisite number of nails. Not withstanding the variability, both within and between the monotonic perpendicular- and parallel-to-grain nailslip test responses, the properties used for the hysteresis model were based on the average from the cyclic parallel-to-grain nailslip test responses. The average value for the asymptotic strength was taken as 1 kN.

Initially, a 3D model was developed where, in order not to be too complex, each OSB panel was to be modelled as one element and the studs by a storey height member. This was later reduced to a simpler 2D model of the east and west walls. The loading on the roof was applied at the level of the top of the walls, rather than just above that level for the actual roof. Since there were no nodes in the model at the required level, the upper storey wall loads were distributed on the basis of 2/3rds to the roof level and 1/3rd to the floor level.

In view of the comparatively rigid nature of both the floor and roof structures, their diaphragm action was modelled by slaving the horizontal degrees of freedom at these two levels. Load transfer from the upper storey to the lower storey was achieved by slaving the vertical degrees of freedom of the appropriate nodes. In order to keep the analysis simple, the seismic mass at the roof and floor levels was distributed evenly to each of the four corners of the building.

4 RESULTS

4.1 *Static*

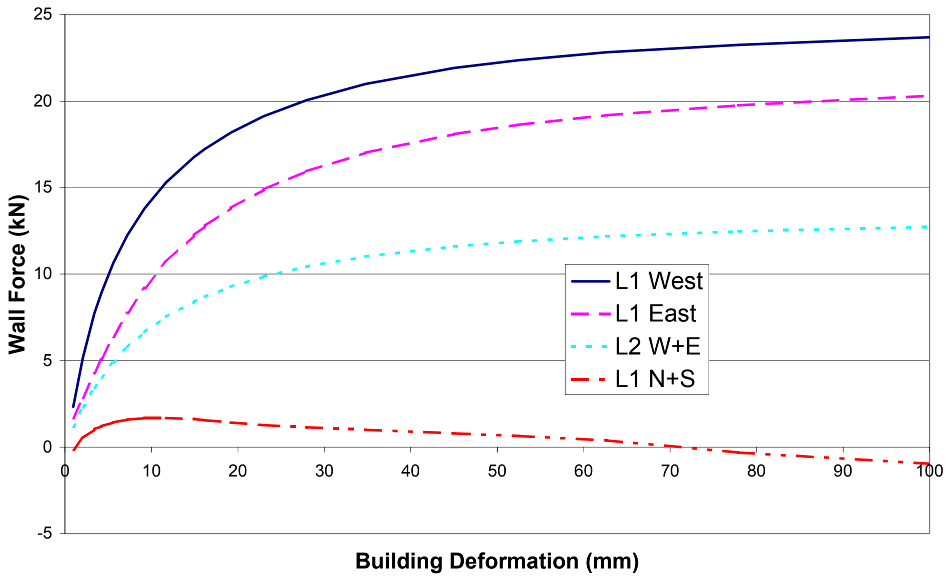
A natural frequency of 3.5 Hz was estimated using the Rayleigh method with an inverted triangular force distribution, with 2 percent of the total force applied at the roof level (cf 8 percent used in New Zealand). The calculated frequency was slightly lower than those reported in the International Benchmark Document [UCSD 2000] (see Table 1). The discrepancy could be due to differences between the connection test joints and those within the building (the number of nails in particular) or due to the effects of the panels above the openings that were not included in the model.

A pushover analysis was conducted within the same Excel spreadsheet. The eight wall forces were calculated as a function of their respective deformations. The Excel solver was used to find a set of deformations and corresponding wall forces that balanced the applied North-South forces and produced zero East-West forces. Torsional response was included in the analysis but the maximum force generated in the North and South walls was less than 2 kN (see Figure 2). The analysis was conducted with 58 percent of the base shear applied at Level 2.

The pushover analysis indicates that the largest deformations occur in the upper storey walls.

4.2 *Modal analyses*

For the 2D dynamic analyses, the original model had a fundamental (translational) frequency of 4.06 Hz. For the cases where the initial stiffnesses were altered to allow for the increasing crushing of the wood around the nails as the tests proceeded, the stiffnesses used gave the fundamental frequencies in Table 1.



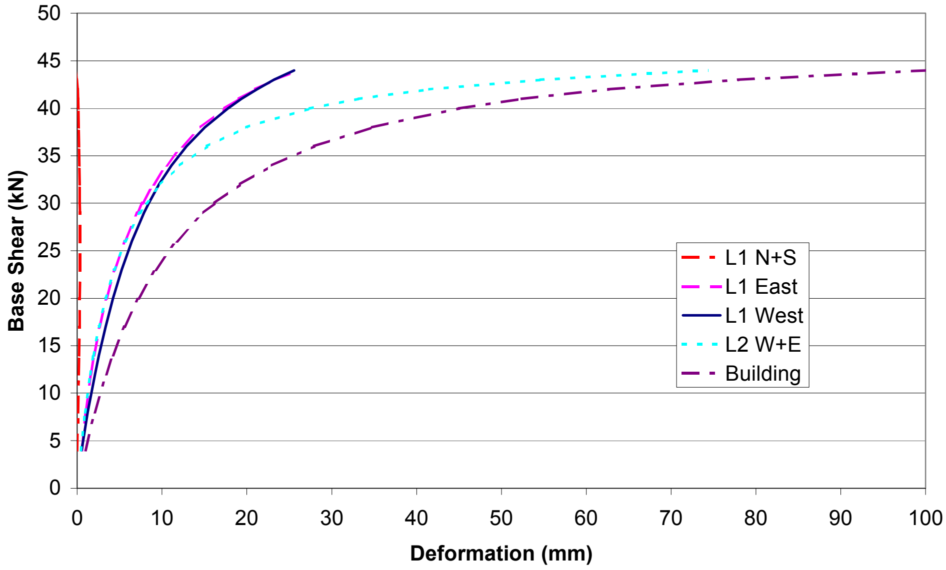


Figure 4 Static model results

Table 1 Comparison of the fundamental frequencies as modelled with those actually measured

Test	Modelled frequency (Hz)	Measured frequency (Hz)
Test 9.S.1	3.97	3.96
Test 9.S.2	3.97	3.91
Test 9.S.3	3.79	3.71
Test 9.S.3R	3.61	3.66
Test 9.S.4	3.50	3.42
Test 9.S.5	3.04	2.93

4.3 *Dynamic*

The analysis using the earthquake record for test 9.S.3 showed that the “nails” holding the lower corners of the upper storey panels had just begun to “yield” while for the records from tests 9.S.4 and 9.S.5, the yielding of the nails increased with the “nails” in these lower corners reaching a ductility of about 3.65 in the latter test.

The time histories for the horizontal displacements at the floor and roof levels for test 9.S.5 are shown in figure 5.

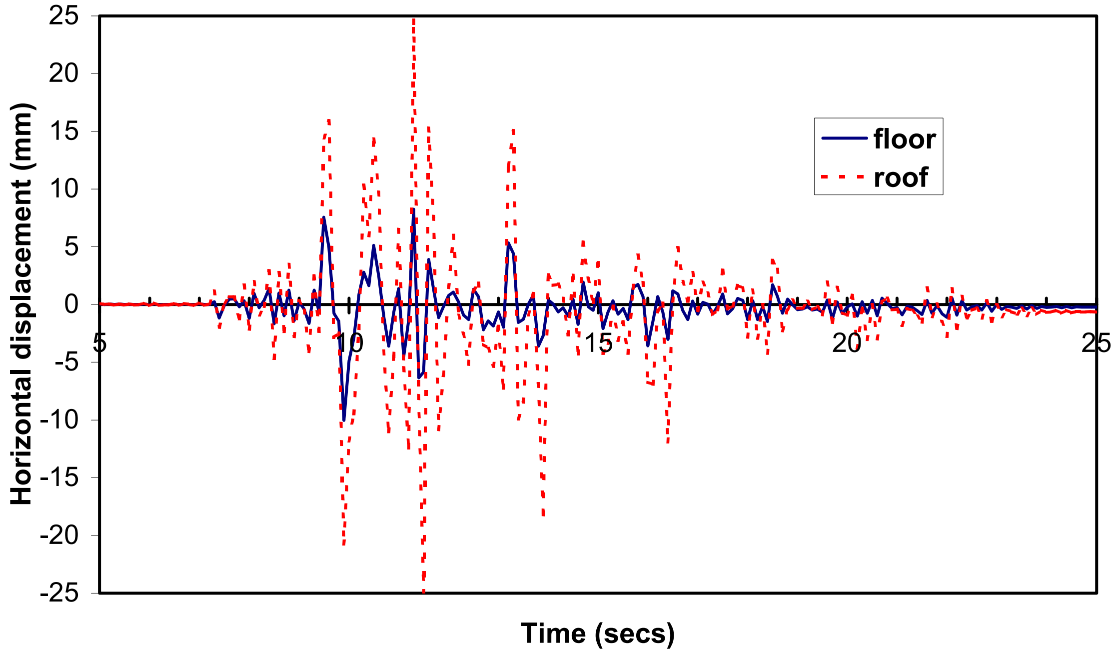


Figure 5 Horizontal displacement time-histories

The horizontal displacements at the floor and roof levels are given in Table 2 for the analyses using the same model (with a fundamental frequency of 4.06 Hz). The horizontal displacements at the floor and roof levels given in Table 3 are for the analyses using the models where the initial stiffness was varied to represent the increasing crushing of the wood around the nails during the tests.

Table 2 – Summary of displacements and accelerations from the dynamic analyses (using the same model for all cases)

Test	Floor level Displacement (mm)	Roof level Displacement (mm)
9.s.1	0.96	2.34
9.s.2	3.87	8.81
9.s.3	5.99	13.95
9.s.4	7.82	21.31
9.s.5	10.02	31.58

Table 3 – Summary of displacements and accelerations from the dynamic analyses (allowing for changes in the model to account for timber deformation, etc)

Test	Floor level Displacement (mm)	Roof level Displacement (mm)
9.s.1	1.02	2.39
9.s.2	4.21	9.61
9.s.3	7.87	18.45
9.s.3r	7.43	16.76
9.s.4	9.53	25.33
9.s.5	12.71	45.29

5 CONCLUSIONS

The static analysis showed that most of the deformation was parallel to the loading. The floor diaphragm and transverse walls significantly reduced the deformation of the more flexible wall.

However, there was minimal rotation, so the transverse walls were only lightly loaded. Most of the deformation occurred in the upper storey walls.

Deformations were similar to those expected for New Zealand buildings subjected to design level earthquakes. This makes the whole CUREe research programme complementary to that being carried out in New Zealand, and will allow the results to be readily translated into the New Zealand context.

The natural periods calculated in the dynamic analysis were similar to those supplied by UC San Diego [UCSD 2000]. The peak deflections during the time-history analyses were lower than the authors expected, based upon previous experience. This may be a result of the assumptions that the hold-downs and ties were rigid and that the floor diaphragm both within its own plane as well as through its depth.

6 ACKNOWLEDGEMENTS

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