SEISMIC DESIGN OF NEW ZEALAND HOUSES

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ABSTRACT

This paper presents the basis for changing the current test and evaluation procedure used to establish bracing ratings. This is known as the BRANZ P21 test method and is used to obtain the bracing ratings of timber framed wall systems for houses, and other low-rise structures, to meet the wind and seismic demand stipulated in the light timber framing standard, NZS 3604:1999. The demand seismic loads in NZS 3604 were based on the loadings specified in the New Zealand loadings standard, NZS 4203:1992. This paper proposes a revised P21 test seismic evaluation (called EM3) so that houses constructed to NZS3604 do not exceed their wall deformation capacity when analysed against a suite of earthquake records compatible with NZS 4203:1992.

1. INTRODUCTION

Houses in New Zealand are generally constructed with light timber framed (LTF) walls each with a variety of wall lengths, sheathing and fastening systems. The result is many different bracing systems, each of which achieves peak bracing resistance at different deflections. This incompatibility precludes simple addition of peak strengths to obtain total lateral resistance. For instance, plasterboard (without fibreglass in the core) wall bracing systems generally reach peak resistance at 10–20 mm deflection and then drop in strength while plywood systems continue providing dependable and increasing resistance up to approximately 60 mm deflection.

In this study houses were analysed by inelastic time history seismic computer analysis. It was assumed that the houses were constructed with only one type of bracing wall, and these were computer modelled from experimental hysteresis loops. Houses are in fact an amalgam of different strength/stiffness walls and to ensure compatibility between different walls the proposed EM3 test and evaluation method specifies a small deflection range over which the walls are evaluated.

This study uses a suite of earthquakes which have elastic spectra corresponding to the design elastic spectra of Figure 4.6.1(b) in NZS 4203:1992 [1]. Computer models of single and two storey buildings with wall elements having pinched hysteresis loop shapes defined to cover the usual range of sheathed timber framed wall behaviour were analysed under excitation from these earthquakes and the results were checked against the predictions from NZS 3604:1999 [2].

2. BACKGROUND

Most New Zealand houses are designed and constructed using the New Zealand Standard for Timber Framed Buildings, NZS 3604:1999 [2]. The earthquake bracing tables in NZS 3604 were derived from the base shear prescribed in NZS 4203:1992 [1]. For $Z = 1.2$, $S_p = 0.67$, and $R = L_u = 1$, equations 4.6.2 and 4.8.1 of NZS 4203 result in the equation for demand seismic base shear force, V, used in NZS 3604, namely:

$$V = W \cdot C_h(T_1,\mu)\, S_p\, R\, Z\, L_u = 0.804\ C_h(T_1,\mu)\ W \qquad (1)$$

where

W = building weight

The seismic hazard acceleration coefficient, $C_h(T_1,\mu) = 0.3$ used to derive NZS 3604 demand bracing tables, was based on a recommendation of BRANZ [3] and approximately equates to an elasto-plastic ductility, $\mu = 3$, and a building period, $T = 0.4$ seconds. When inserted into Equation (1) this results in the relationship for Zone Z=1.2 (Wellington):

$$V = 0.241\ W \qquad (2)$$

Typically manufacturers have their wall sheathing systems tested using the BRANZ P21 [4,5] test method. The results are evaluated and wind and earthquake bracing ratings assigned. Builders/designers use the manufacturers' published ratings to ensure new house designs have sufficient lateral bracing to meet NZS 3604:1999 [2] demand loads. The evaluation method used to derive earthquake bracing ratings from the P21 test results is examined in this paper.

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There was a significant increase in earthquake demand bracing ratings from NZS 3604:1990 [6] to NZS 3604:1999 [2] based on analysis of past seismic events. This created concern in the industry as there is no field evidence that the average new house will be inadequately braced in a large earthquake. Thus, it is not desirable to be unduly conservative in the evaluation of wall bracing ratings.

3. PROPOSED NEW P21 TEST (CALLED EM3)

A draft revision of the P21 test procedure has been prepared and named the draft BRANZ EM3 test and evaluation method. Deam [7] describes in detail the difference between the P21 and the draft EM3 test method. The cyclic regime for the draft EM3 test is three cycles to each of ±(10, 15, 20, 25, 30, 35 and 55 mm deflection). To introduce compatibility between bracing wall systems, the draft EM3 evaluation of seismic strength is based on the third cycle peak loads at 25, 30 or 35 mm even though peak resistance may be attained outside this range. The draft EM test wall seismic strength is evaluated with Eqn. (3).

$$EM3_{\text{seismic rating}} = R_m.F1 \tag{3}$$

Where: R_m is the average of three tests of the average third cycle peak (push and pull) applied force at Δ_m

F1 is an appropriate factor. This paper investigates and recommends suitable values of F1.

The testing organisation may derive the seismic rating at the most advantageous of 25, 30 or 35 mm deflections.

4. TEST SPECIMEN BOUNDARY CONDITIONS

If bracing panels are isolated from the surrounding structure and laboratory tested under horizontal racking loads without any rocking restraint, they will generally uplift off the foundation beam at relatively low loads at the panel tension end (i.e., rock about one end of the panel) as shown in Figure 1. However, when panels are built into a house, the wall sheathing, framing continuity and gravity effects provide some resistance to uplifting, thereby significantly increasing actual house racking strength. A review of racking test methods used by others indicates that panel uplift is entirely restrained by external means, or not used at all (Thurston, [8]). The P21 test is unique in terms of worldwide test practices [8] in that it uses an intermediate method, i.e., a partial uplift restraint as shown in Figure 1. This effectively provides an uplift restraint of three 100 x 4 mm nails in shear.

Based on tests by Thurston [8] on long walls with openings, and further in-house tests at BRANZ and racking tests on existing buildings [9], it is expected that walls within buildings will generally have significantly greater uplift restraint than provided by the P21 uplift restraint in nearly all situations except where the bracing wall terminates at a doorway. Axial load on a wall and construction of a storey above is expected to significantly increase uplift restraint. Thus, the uplift restraint in the draft EM3 test is increased from 3 to 6 nails which is expected to provide a more realistic but still conservative estimate of average panel uplift restraint. The additional nail uplift restraint is expected to result in a higher bracing rating for stronger sheathing systems, whose bracing resistance currently tends to be governed purely by the P21 uplift strength rather than the shear strength of the sheathing fastening. Currently strong brittle sheathing systems without end straps can appear to exhibit ductile behaviour in the test, which is actually due to the low shear yield strength of the P21 uplift restraint, rather than the wall sheathing itself. This apparent ductility is unlikely to be replicated in a real structure. In this regard the "3-nail" restraint may have unrealistically benefited brittle systems (such as plasterboard) relative to tougher systems (such as plywood or MDF).

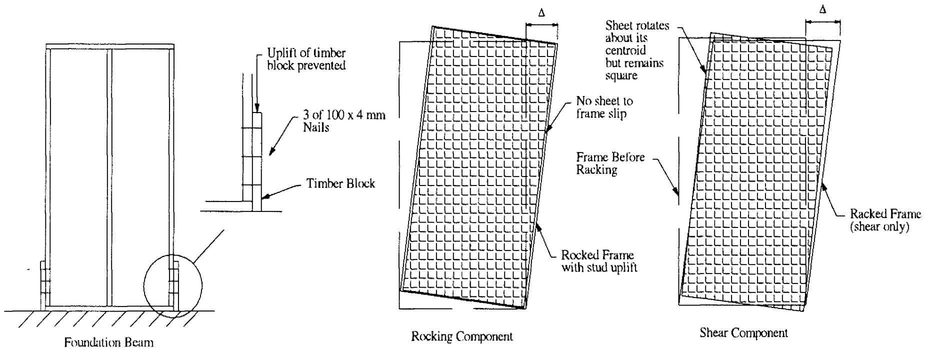


Figure 1. P21 End Restraints and Components of Deflection.

5. BRACING RATING FROM NZS 3604

Based on Eqn (2), a wall of bracing strength 5 kN is expected to restrain a seismic mass of 5/0.241 = 20.7 kN weight = 20.7/9.81 = 2.11 tonnes mass. An analysis of BRANZ commercial tests for a variety of wall linings and wall lengths to give a bracing rating of 5 kN gave an average secant stiffness at 0.5 peak load of approximately 500 kN/m and therefore its natural period can be calculated as $T = 2\pi\sqrt{(M/K)} = 2\pi\sqrt{(2.11/500)} = 0.41$ seconds which is close to the 0.4 seconds assumed by NZS 3604:1999 [2]. (See Section 2).

It can be expected that a house will be stiffer and therefore have a shorter natural period than 0.4 seconds due to the holistic response of the complete system. This is due to load sharing and composite action, of both the structural and non-structural elements, and the lateral restraint due to "rocking action" being small, owing to the transfer of house weights to the ends of the bracing elements. The enhanced stiffness and strength (hereafter called "systems effects"), is usually ignored when assessing house resistance to lateral forces. The computer simulation of earthquake shaking in this study ignores "systems effects", and it is expected that this will result in computed deflections greater than will occur in real structures. This will therefore result in a conservative design. It is noted that many non-structural elements will fail at relatively low house deflections.

Based purely on its bracing walls, a two storey house will have a calculated natural period significantly greater than 0.4 seconds (calculated for the single degree of freedom system above) and thus its seismic coefficient would be expected to reduce accordingly. However, soft storey effects are likely to make the two storey house perform more like a single degree of freedom (SDOF) system, rather than a two degree of freedom (2DOF) system. This is investigated later in the paper.

6. HOUSE BRACING WALL DESIGN PHILOSOPHY

It is expected that less than 10% of New Zealand houses will experience an earthquake exceeding the NZS 4203:1999 [1] design earthquake in any 50 year period. Historically, timber framed bracing walls have performed well in large earthquakes [13]. A suitable design procedure is considered to be that bracing walls are readily repairable after a design ultimate event, and do not collapse in an extreme event.

As noted above, "systems effects" add to the lateral strength. When walls and other items contributing to the "systems effects" fail, they are still likely to have contributed to the damping. However, the analyses in this study only assume 5% damping ratio (as also assumed in development of the NZS 4203 spectra [1]) and ignores "systems effects".

The strength of the walls is based on average data, rather than lower 5% data normally used by engineers. As with the P21 test method, averages were used with the EM3 method due to the good historic performance of building bracing walls and the difficulty therefore in requiring more bracing strength.

7. VERIFICATION OF DEMAND LOADS IN NZS 3604

After using information supplied from the 1990 NZS 3604 committee, BRANZ developed a spreadsheet to check whether the demand seismic loads in NZS 3604:1999 could be regenerated from the assumptions listed above. Weights from a standard sized house were calculated for the various combinations of light/medium/heavy wall and roof construction materials and the seismic loads calculated for each zone etc. There was generally good agreement, provided deductions were not made for window openings. There is scope to reduce the demand loads if these openings are considered.

8. EARTHQUAKE SUITES

The University of Auckland [14] provided BRANZ with the earthquake suite used for the analysis. These were five earthquake acceleration records from past events that were modified to match the NZS 4203:1992 spectra over its full range (as shown in Figure 2). In the analysis following, each earthquake acceleration record was scaled by 1.2 to correspond to the "Z = 1.2" factor of NZS 4203, applicable to the most severe seismic zone in New Zealand.

The computed deflections from the modified Parkfield earthquake were significantly greater than those computed for the other four earthquakes. Results for the other four earthquakes showed reasonable agreement. However, the average results from all five earthquakes were used in the analysis below. Computer 2D analysis was by the Ruaumoko computer program [10] available from the University of Canterbury.

9. SINGLE STOREY MODEL

This was a simple SDOF oscillator as shown in Figure 2. Damping was taken as 5% critical damping as also assumed in development of the NZS 4203 spectra [1]. The Stewart [11] model was used to describe the oscillator spring and was given properties to represent the hysteresis loops of either a plasterboard wall (Figure 3 (a)) or a plywood wall (Figure 3 (b)). These matched test results at BRANZ and are considered to represent the two extremes of shapes of loops for sheathed timber framed walls. Sensitivity analyses were also performed with variations of initial stiffness and damping.

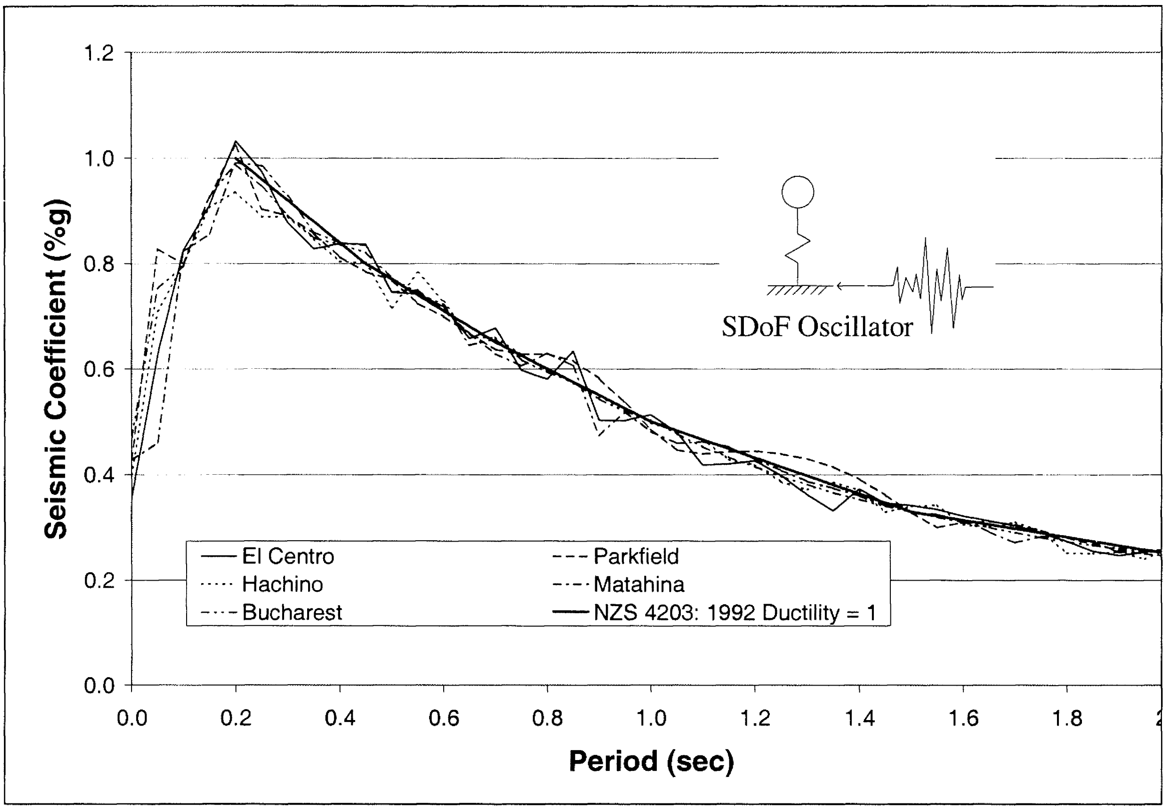
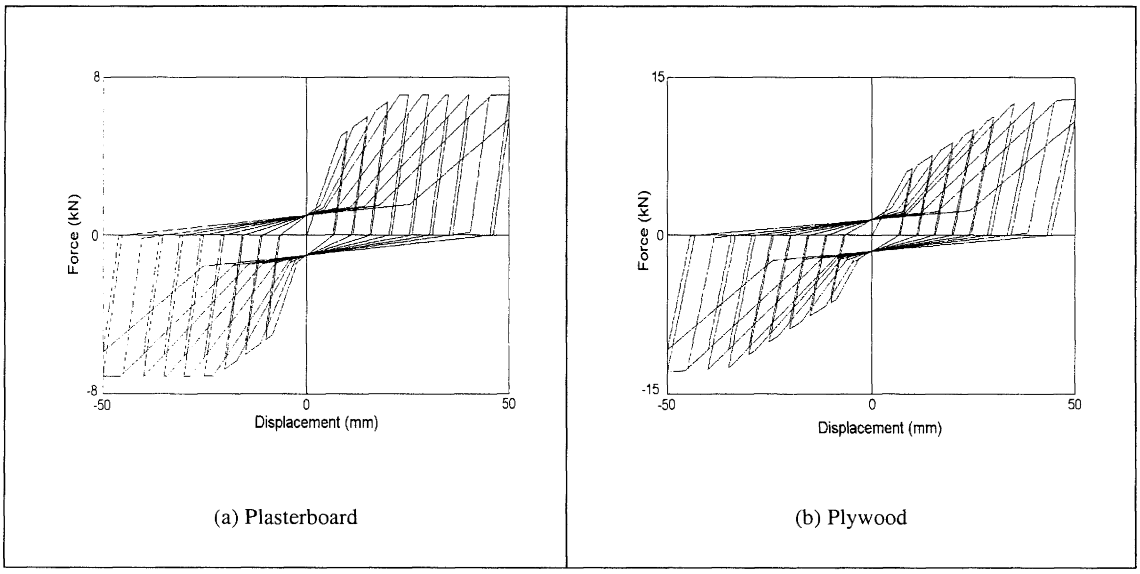


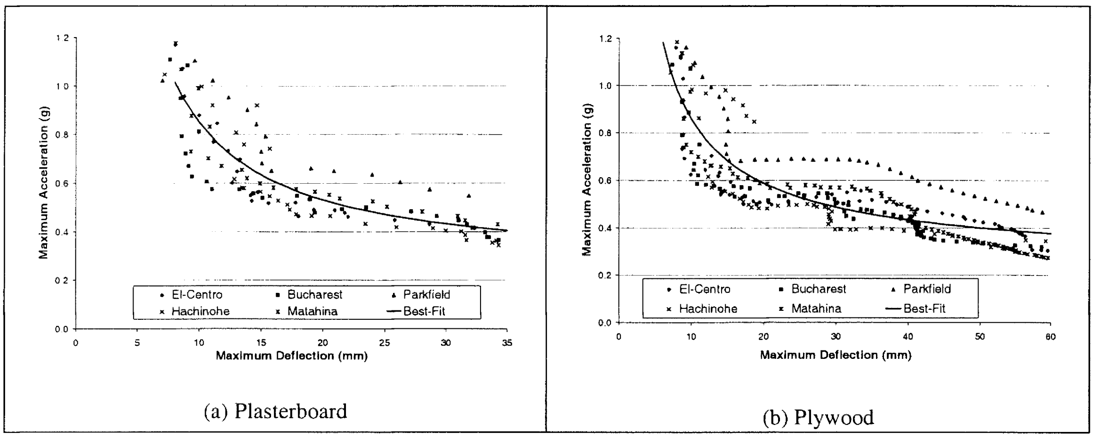
Figure 2. Comparisons of Acceleration Response Spectra from NZS 4203 and Five Earthquakes.



(a) Plasterboard

(b) Plywood

Figure 3. Plasterboard and Plywood Hysteresis Loops, as Stewart Springs, Input into Ruaumoko.



(a) Plasterboard

(b) Plywood

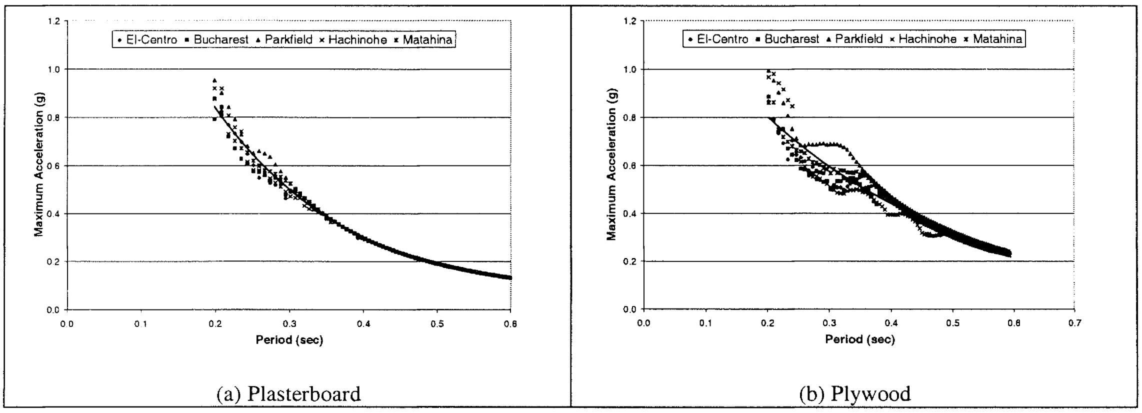
Figure 4. Maximum Acceleration versus Deflection for Single Degree of Freedom Structures.

The displacement constraint was that the computed deflections do not exceed 35 mm for plasterboard sheathed walls, and 60 mm for plywood sheathed walls. BRANZ has found that unreliable performance, often showing significant load decay, is likely at greater deflections. These limits are called "Max OK". Another limit is also used ("Max Rel") being 25 mm and 50 mm for plasterboard and plywood respectively. These are the estimated average values of wall deflection before significant load decay.

Time history analyses were performed on the SDOF oscillator for an incremental series of masses, using the five earthquakes and both the plasterboard and plywood models. In each run, the peak "acceleration", A_{max} was calculated (using A_{max} = maximum force/mass). A plot of A_{max} versus maximum deflection is shown in Figure 4. This also includes "best-fit" curves, which have been fitted to data for deflections not exceeding "Max OK". At low deflections the response of the walls is close to elastic (see Figure 3) and A_{max} becomes high. If an engineer chooses to estimate maximum wall deflections, before significant peak load resistance deterioration, by experimental means or otherwise, then a "base shear coefficient" can be derived from Figure 4.

A plot of A_{max} versus period calculated from $T = 2\pi\sqrt{(M/K)}$ is shown in Figure 5. This also includes "best-fit" curves, which have been fitted to data for deflections not exceeding "Max OK". The best-fit curves are compared with NZS 4203 spectra in Figure 6, which is likely to be more familiar to most engineers than the preceding plots. Higher periods imply greater masses and thus at higher periods the wall will deflect further in the design earthquake, and may exceed the effective deflection capacity of the wall. Hence the best-fit curves have only been plotted up to "Max OK" deflections.

For each Ruaumoko computer run the equivalent elasto-plastic ductility, μ, was calculated from maximum base acceleration coefficient and the elastic period using equations relating these two variables to the ductility, μ, given in Section C4.6.2 of NZS 4203. A plot of μ versus maximum deflection is shown in Figure 7. At "Max Rel" deflections, and if the data for the Parkfield earthquake is ignored, then Figure 7 indicates that a value of μ of 3 is appropriate for a SDOF plasterboard structure and μ of 2.5 for a plywood structure. The plasterboard exhibits greater ductility as the hysteresis backbone curves show greater deviation from the initial slope used in the model.



(a) Plasterboard

(b) Plywood

Figure 5. Maximum Acceleration versus Period for Single Degree of Freedom Structures.

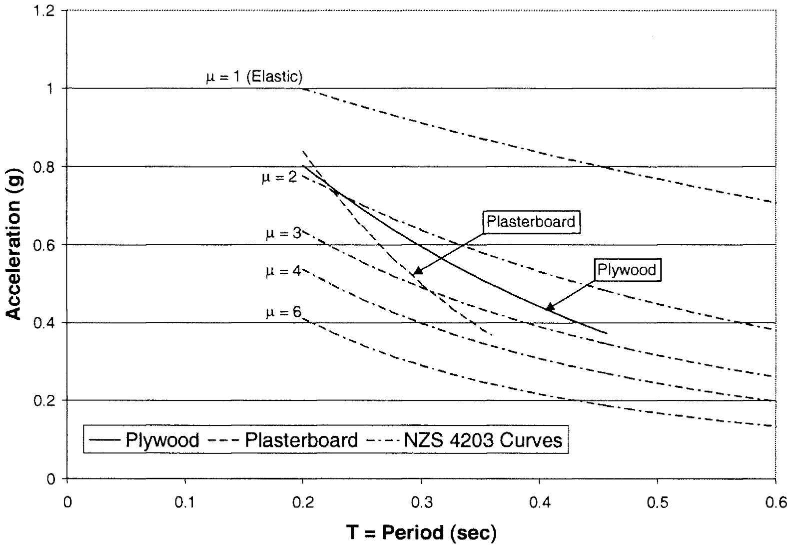
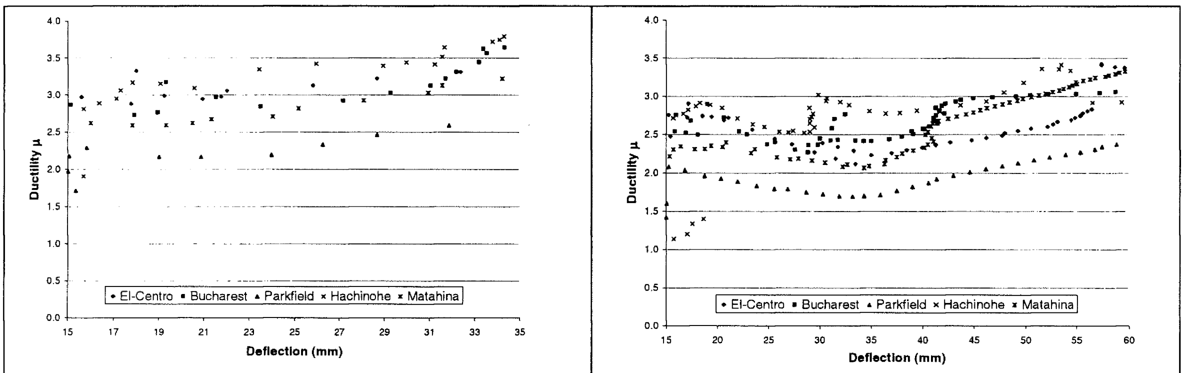


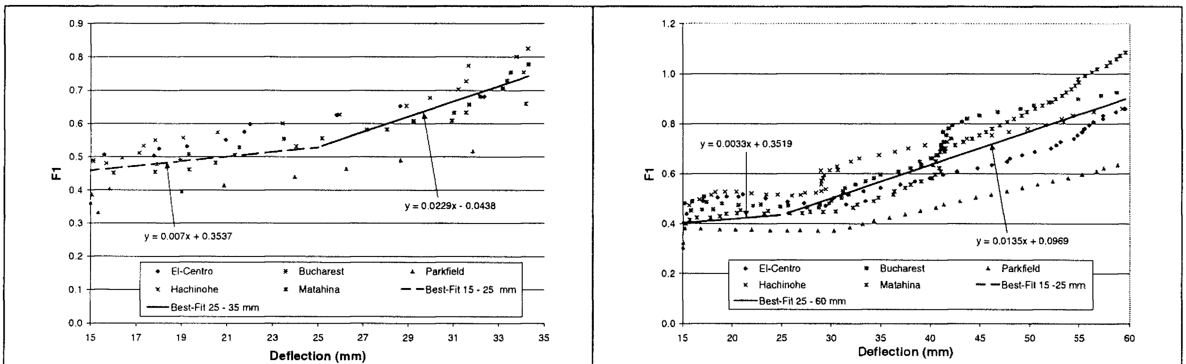
Figure 6. Comparison of Seismic Coefficient from Figure 4.6.1 of NZS 4203 and Ruaumoko Runs for Single Degree of Freedom Models.



(a) Plasterboard

(b) Plywood

Figure 7. Ductility versus Deflection for Single Degree of Freedom Structures.



(a) Plasterboard

(b) Plywood

Figure 8. F1 versus Deflection for Single Degree of Freedom Structures.

Based on equations (1 and 2) Fl was computed from 0.241W/R' where R' is the value of the third cycle peak load, R, at the maximum deflection, Δ, from the Ruaumoko run. Figure 8 gives plots of Fl versus Δ for both plasterboard and plywood. At any given Δ, the lowest values of Fl were for plywood and the best-fit curve shown is the adopted relationship recommended for use in the draft EM3 test.

10. TWO STOREY MODEL

The building is assumed to be a two-storey timber framed building founded on a concrete floor. Other options allowed in NZS 3604, namely a single or two-storey building on a subfloor, were not analysed as part of this study.

A two-degree of freedom (2DOF) Ruaumoko model (i.e. two masses) was developed and used to analyse the building subjected to five earthquakes, two sheathing materials and 29 values of total mass. For each combination, there were four ratios of floor to roof level masses and six first floor to second floor wall bracing combinations. Calculations were made of the ratios of floor to roof weights for NZS 3604 buildings with the same first floor area at each level, but with various combinations of a typical range of house construction materials. Based on this data the following floor to roof level mass ratios were used: (2.4, 1.66, 1.15 and 0.8). A realistic range of wall distributions are harder to cover as a building may have more bracing walls than required. Based on the bracing demands in Section 5 of NZS 3604 the basic ratio of number of bracing walls in the first storey relative to the second storey was taken to be 1.7. This ratio was factored by the following values to get six wall bracing ratios: 0.7, 0.85, 1, 1.2, 1.5 and 1.8.

It is assumed that the buildings analysed have walls in at least one storey that experience the maximum deformations to which they would have been assessed in the EM3 test. Thus, results were excluded where the maximum deformation of both storeys was less than 25 mm as this was outside the proposed range where the EM3 test is to be evaluated. Results were also excluded where the deflection of one storey exceeded "Max OK" (see Section 9). These deflection limits are called "2DOF Range". Another limit sometimes imposed was that no storey drift may exceed 35 mm and at least one storey must exceed 25 mm. This was called the "EM3 Compliant" deflection set and was selected to ensure all walls were evaluated near the same deflection to allow wall bracing strengths to be added to obtain total house bracing strength.

For data complying with the "2DOF Range", and if soft storey effect is defined as being when the maximum deflection of one storey is more than 2.5 x the maximum deflection of the other, then 12% of the runs developed a soft storey effect in the lower storey and 11% of the runs soft storey occurred in the upper storey of the plywood lined structures. The corresponding figures for plasterboard-lined walls are 28% and 14% respectively. Thus, a significant percentage (but less than half) of two storey houses in this study will develop a soft storey effect. A "soft-storey" building will tend to act as a single degree of freedom structure with most of the seismic deflection concentrating in the "soft-storey". In multi-storey structures this can lead to collapse but the risk is less in two-storey structures.

In each time history run, the maximum building "acceleration", A_{max}, was calculated (using A_{max} = maximum base shear/total building mass). A typical plot of A_{max} versus building first period is shown in Figure 9 for the "2DOF Range" data. The plot includes a "best-fit" curve. The best-fit curves for all earthquakes and materials have been compared with NZ 4203 spectra in Figure 10. The 2DOF results in Figure 10 are similar to that for the SDOF model given in Figure 6 but extends to greater periods.

For each inelastic time history analysis the equivalent elasto-plastic ductility, μ, was calculated from the maximum base acceleration coefficient and initial period. A plot of μ versus initial period for a plywood building and the modified El Centro earthquake is shown in Figure 11(a). This shows considerable scatter. The best-fit lines from all earthquakes and both materials are shown in Figure 11(b).

In Section 5 it was stated that a two storey building would be expected to have a lower seismic base coefficient than a single storey building, as its natural period would generally be higher. From each inelastic time history analysis, where the deflections were "EM3 Compliant", the base resistance, BR, was calculated from Fl x $R_{(bottom\ storey)}$, Fl was calculated from the maximum deflection in the bottom storey using the equations for the best- fit in Figure 8(b). The ratio of BR to the NZS 3604 demand load (0.241W – see Eqn. (2)) is the ratio of demand loading of 2DOF:SDOF structures, $Ratio_{2DOF:SDOF}$. The results are summarised in Table 1. The average value over all earthquakes was 0.88 for plywood sheathing and 0.76 for plasterboard. The average ratios for other materials are not expected to be > 0.9 and thus it is recommended that NZS 3604 bracing demands for two storey houses be reduced by 10%.

11. LOAD DISTRIBUTION

The authors wished to determine the appropriate vertical load distribution of base shear for buildings of this type.

Three load distributions were considered; (a) The distribution from equation 4.8.2 of NZS 4203 [1], using 92% of the lateral load with an additional 8% of the load directed to the top of the building, (b) equation 4.8.2 with 0.92 replaced by 1.0 and no additional load directed at the top, and (c) uniform distribution. For each of these the bracing requirement of the top storey, R'', was calculated from R'' = Fl x $R_{(top\ storey)}$. If W' is the load at the top of the building as determined using the above three assumed distributions then the shear force in the top storey = 0.241W' from Eqn. (2). The best distribution will have Target Shear, TS_R, = R''/(0.241W') closest to 1.0. However, if we wish to reduce total building shear by 10% then the target is TS_R, = 0.9, and hence is adopted.

Table 2 gives the average values of TS_R for buildings with "EM3 Compliant" deflections. The most critical case is "plywood", as the TS_R are highest, and for this sheathing it can be seen that assumption (b) gives the best but conservative fit for $TS_R = 0.9$.

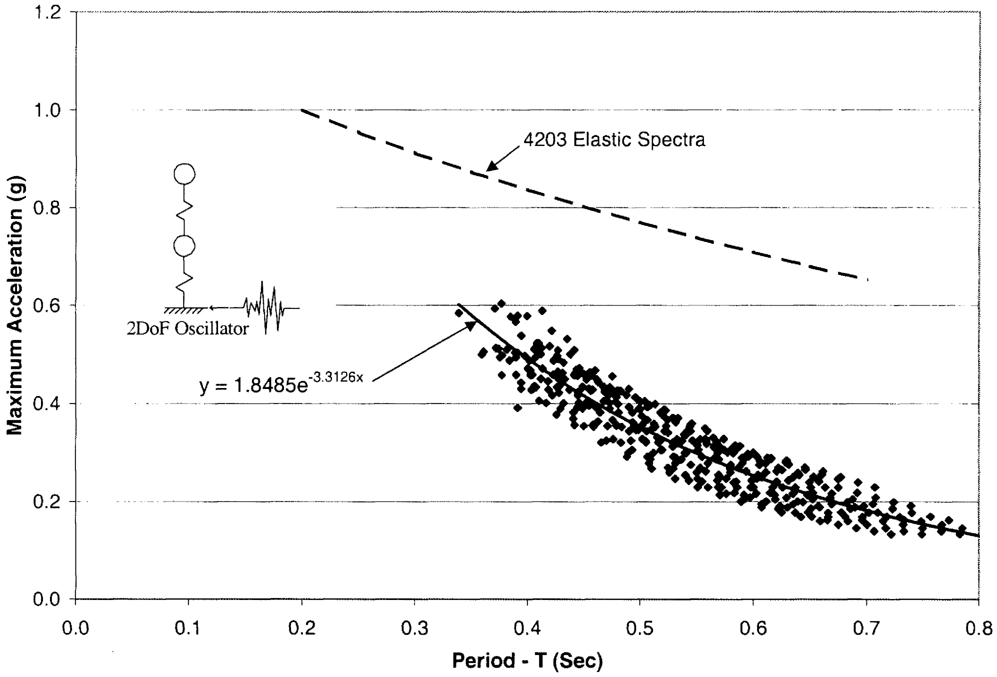


Figure 9. Maximum Acceleration versus Period for two degree of Freedom Structures Under Modified El Centro Earthquake.

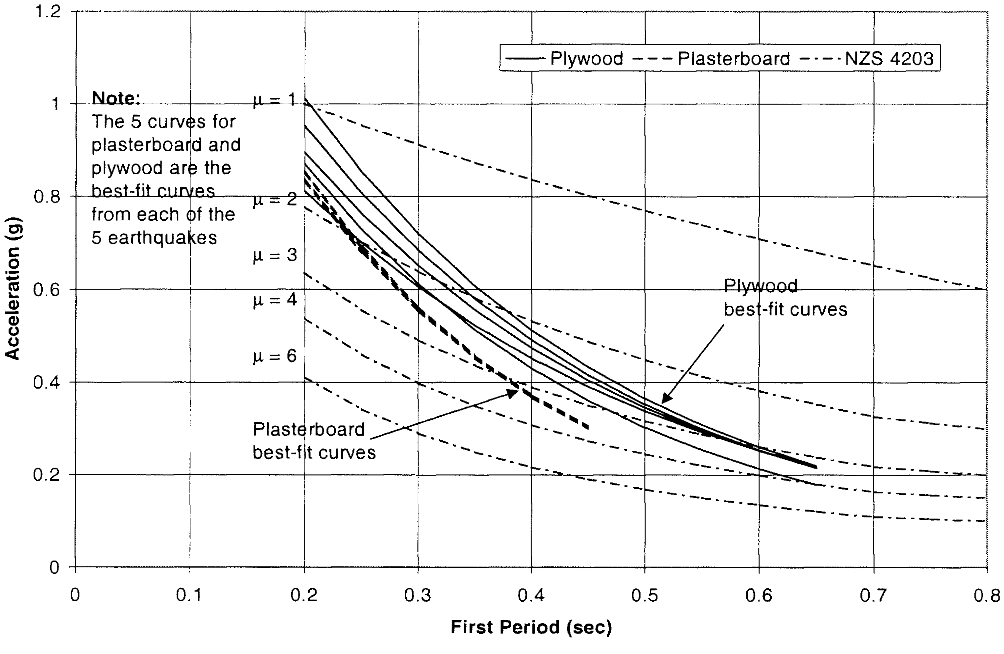
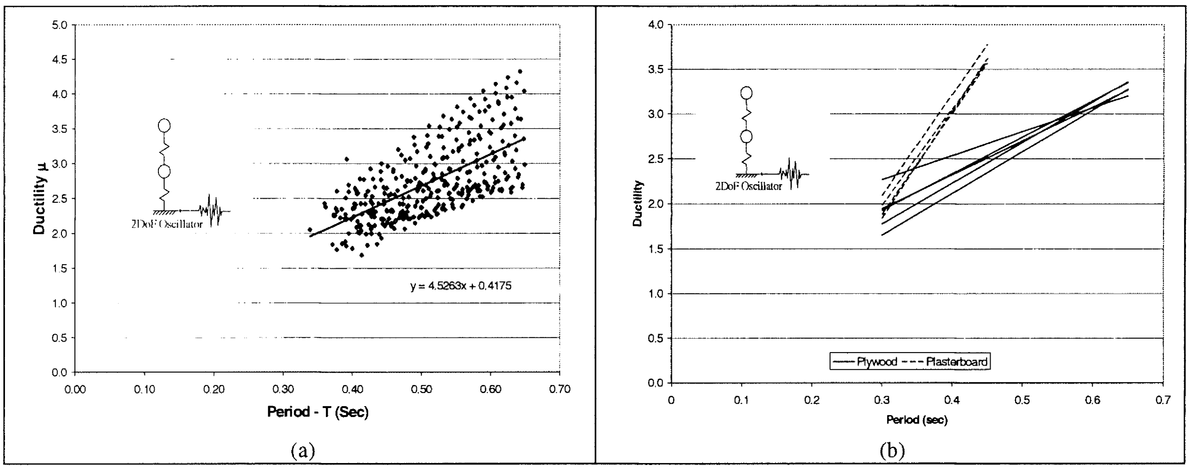


Figure 10. Comparison of Seismic Coefficient from Figure 4.6.1 of NZS 4203 and Ruaumoko Runs for two degree of Freedom Models.



(a) (b)

Figure 11. Ductility versus First Period for 2 Degree of Freedom Structures.

Table 1. Ratio of Two-Storey: NZS 3604 Base Shear Demand

Sheathing	*Earthquake*	*Ratio*$_{2DOF:SDOF}$	*Standard Deviation*
Plywood	El Centro	0.95	0.12
Plywood	Hachinohe	0.75	0.07
Plywood	Matahina	0.86	0.15
Plywood	Bucharest	0.73	0.10
Plywood	Parkfield	1.09	0.14
	Average:	0.88	0.12
Plasterboard	El Centro	0.78	0.07
Plasterboard	Hachinohe	0.66	0.07
Plasterboard	Matahina	0.74	0.12
Plasterboard	Bucharest	0.68	0.10
Plasterboard	Parkfield	0.92	0.09
	Average:	0.76	0.09

Table 2. Ratio of Two Storey: NZS 3604 Upper Storey Shear Demand. (Standard deviations are given in brackets)

Sheathing	*Earthquake*	TS_R *Distrib. (a)*	TS_R *Distrib. (b)*	TS_R *Distrib. (c)*
Plywood	El Centro	0.83 (0.11)	0.87 (0.11)	1.23 (0.15)
Plywood	Hachinohe	0.65 (0.10)	0.69 (0.10)	0.98 (0.12)
Plywood	Matahina	0.79 (0.12)	0.84 (0.12)	1.18 (0.17)
Plywood	Bucharest	0.78 (0.15)	0.82 (0.15)	1.15 (0.21)
Plywood	Parkfield	1.00 (0.17)	1.05 (0.18)	1.47 (0.25)
	Average:	0.81	0.85	1.20
Plasterboard	El Centro	0.67 (0.06)	0.71 (0.06)	0.99 (0.11)
Plasterboard	Hachinohe	0.59 (0.08)	0.62 (0.08)	0.87 (0.12)
Plasterboard	Matahina	0.67 (0.08)	0.71 (0.09)	0.99 (0.15)
Plasterboard	Bucharest	0.66 (0.09)	0.70 (0.09)	0.98 (0.15)
Plasterboard	Parkfield	0.82 (0.08)	0.87 (0.08)	1.23 (0.14)
	Average:	0.68	0.72	1.01

12. SENSITIVITY ANALYSIS

The analyses above used the hysteresis loops shown in Figure 3 and assumed 5% damping. The shape of the loops was based on test results but the initial stiffness does vary in practice although the essence of the hysteresis shape remains. shows the influence on F1 when the initial stiffness is increased or decreased by 50% in respective Ruaumoko runs. Figure 13 shows the influence on F1 of variations in damping level. Both of these sensitivity analyses were performed on plywood SDOF walls under the modified El Centro earthquake. Both plots show significant, but not alarming, sensitivity to these initial assumptions. The sensitivity of damping on displacements starts to become severe from displacements greater than 40 mm.

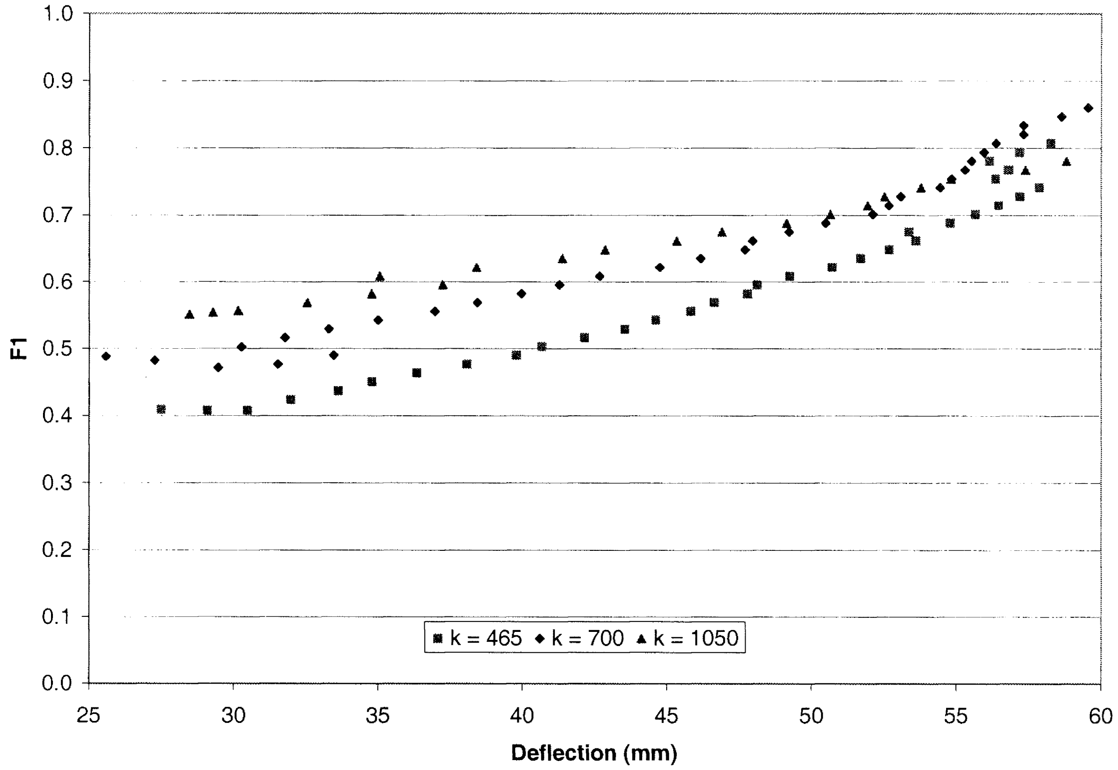


Figure 12. Sensitivity Analysis Showing Effect on F1 when Initial Stiffness, K, is Varied (modified El Centro earthquake and plywood SDoF model).

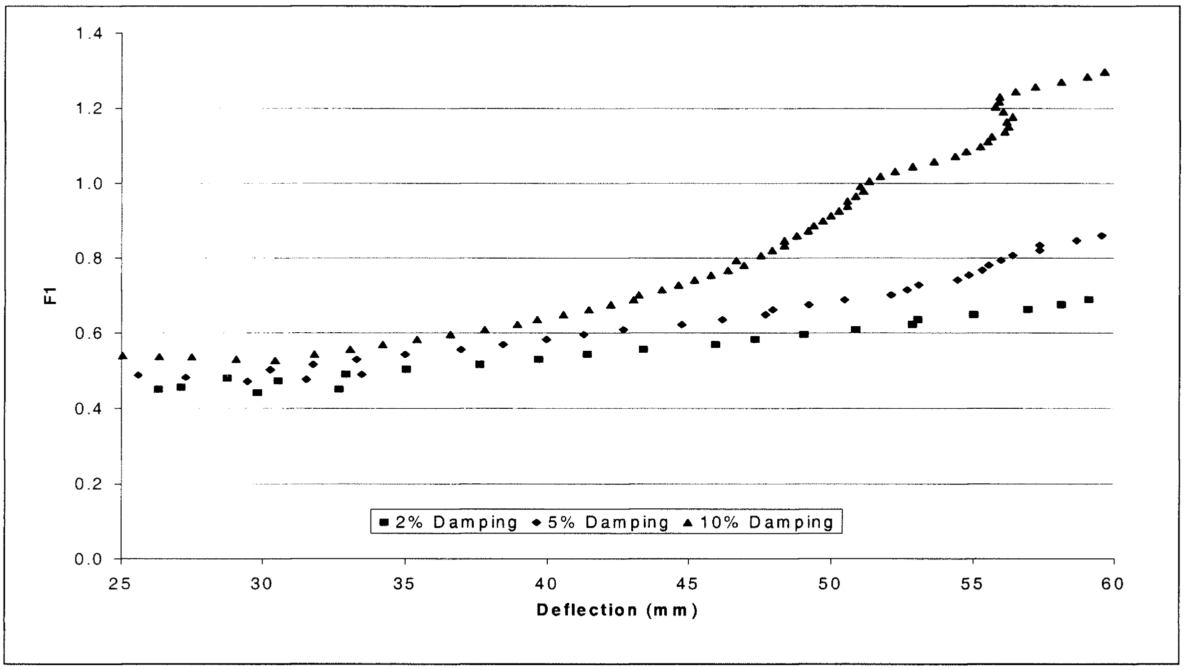


Figure 13. Sensitivity Analysis Showing Effect on F1 when Level of Damping is Varied (modified El Centro earthquake and plywood SDoF model).

13. OTHER EFFECTS

It is desirable that houses have the ability to survive larger earthquakes than the design earthquake even if damage is severe. As noted above, timber framed houses with sheathed wall bracing panels have performed well in past earthquakes due to "systems effects" and probably larger damping than assumed in the analyses above. BRANZ considers that it is reasonable to assume up to 20% of the overall bracing resistance can be apportioned to these combined effects, provided the system has adequate reserve strength at the ultimate deflection (taken as 55 mm). Thus it is proposed to introduce another factor F2 to the bracing strength into equation (1) as given in Eqn. (4) with F2 being defined in Eqn. (5):

$$EM3_{\text{seismic rating}} = R_m F1.\ F2 \qquad (4)$$

$$1.0 \leq F2 = 1.3\ R_{55}/R_m \leq 1.2 \qquad (5)$$

Where R_{55} and R_m are the average third cycle peak loads at 55 mm (R_{55}) and 25, 30 or 35 mm (R_m).

NZS 3604 specifies bracing wall layout to minimise torsion effects. Building twisting can be expected to increase seismic demand on some walls. It is proposed that torsional effects be the subject of a separate study.

14. SPECIFIC DESIGN OF LOW RISE BUILDINGS

For the specific design of one and two storey buildings with bracing walls having similar shaped hysteresis loops to the plasterboard and ply walls analysed in this report, it is suggested that the designer "determine" the relationship between third cycle peak load, R, and deflection of each wall (such as by test or manufacturer data). The lateral resistance of each floor, F_{floor}, is calculated from F2 x F3 x Σ(R x F1) for a range of inter-storey deflections, Δ, where Δ is less than the maximum allowable inter-storey drift but not greater than 50 mm. Both R and F1 are a function of Δ and are summed over all bracing walls at a given level. The maximum value of F_{floor} for the trialed Δ, at each floor level is then selected. The lateral forces are then calculated using C = 0.3 and distributed according to equation 4.8.2 with 0.92 replaced by 1.0 as discussed in Section 10. The designer then simply ensures that the bracing resistance exceeds the demand forces at each level. F3 is taken = F4 x F5 and can be taken to be 1.0 for buildings with "systems effects" and torsion demands equivalent to a typical house. This can be assumed if the ratio of bracing walls to total walls < 0.7 and the perimeter bracing walls are distributed according to NZS 3604. For a building with little strength enhancement due to "systems effects" we recommend F4 to be taken as 0.7 – 0.9. The designer should deduce F5 to take into account torsion effects and should be taken as 0.5 – 1.0.

The above procedure may also be used with a mixture of masonry and timber framed walls but it is imperative that the hysteresis loops for the masonry walls also include the connection stiffness to transfer the loads from the diaphragm to the wall. Beattie [12] found this to be the prime deflection source for masonry walls.

More work is required to determine if the above procedure is applicable to three and four storey buildings.

15. SUMMARY AND RECOMMENDATIONS

Models of single and two storey houses have been analysed under five earthquakes corresponding to NZS 4203:1992 [1] spectra and the resistance exhibited by the analysis related to the demand loads in NZS 3604 [2]. Average test data rather than lower 5th percentile data was used and this was justified on the basis that house structures included many "systems effects" that enhanced their actual seismic resistance, but are not taken account of in the analyses and on the good record of house performances in past earthquakes. It is recommended that:

- Demand seismic loads in NZS 3604 be reduced to account for reduction of building weight due to openings in exterior walls as discussed in Section 7.
- The demand seismic loads in NZS 3604 be further reduced by 10% for structures greater than one storey largely due to increased building natural period.
- The demand seismic loads in NZS 3604 be based on distributing the base shear in eqn. 4.8.2 of NZS 4203 with 0.92 replaced by 1.0. (i.e. according to the mass distribution and without additional roof level concentrated load.)
- A draft EM3 test and evaluation method was recommended with a stipulated cyclic regime, an increased P21 end restraint and a seismic evaluated resistance given by F1 x F2 x R where R was the average of the push and pull residual loads in the deflection range 25-35 mm and from three tests. Values of F1 and F2 were recommended. This is expected to give more realistic modelling of full house performance and will bring New Zealand more in line with overseas relative values for the ratio of plasterboard/plywood wall strengths.
- For low rise structures constructed from disparate walls which exhibit pinched hysteresis loops, and which cannot be directly designed from NZS 3604, a Specific Design Method is described.

16. ACKNOWLEGEMENTS

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