STUDY REPORT

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BRANZ TEST AND EVALUATION METHOD EM3-V3 FOR BRACING RATING OF WALLS TO NZS 3604

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Preface

The BRANZ P21 test method is currently used to obtain the bracing ratings of wall systems for low-rise buildings to meet the wind and seismic demand stipulated in the New Zealand timber framed building standard, NZS 3604. This report describes the basis of the proposed EM3-V3 and evaluation method. This is intended to replace the current BRANZ P21 test method.

The P21 wall bracing test and evaluation procedure was first published by BRANZ in 1979 and was revised in both 1982 and 1987. BRANZ *Technical Recommendation TR 10* revised the P21 evaluation method to bring it into line with the 1990 revision of NZS 3604, as the previous version of NZS 3604 (1984) was based on working stress design concepts whereas the 1990 version was in limit state format. Thurston and King (1992) discussed fundamental deficiencies in the methodology used in both the P21 and R10 procedures. A proposed revised method of test and evaluation of wall racking test results is discussed by Thurston and Park (2003).

The racking resistance of long walls with openings was investigated in BRANZ *Study Report 54*. Field measurements of the seismic performance of timber piles was reported in BRANZ *Study Report 58*. The equivalent ductility of residential timber buildings is investigated in BRANZ *Study Report 73*. BRANZ *Study Report 78* proposed a revised wall racking test and evaluation method but this was never adopted. A comparison of NZS 3604 predicted house strength and the measurements from a full-sized house cyclic racking test are described in BRANZ *Study Report 119*.

Acknowledgments

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Note

This report is intended for standards committees, structural engineers, architects, designers and others researching earthquake and wind resistance of low-rise buildings.

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BRANZ Study Report SR220

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Reference

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Abstract

This report presents the basis for changing the current BRANZ test and evaluation procedure used to establish wall bracing ratings. This is known as the BRANZ P21 test method and is used to obtain the bracing ratings of sheet-sheathed timber framed wall systems for houses, and other low-rise structures, to meet the wind and seismic demand stipulated in the timber framed building standard NZS 3604:1999. The demand loads in NZS 3604 were based on the loadings specified in the New Zealand loadings standard NZS 4203:1992, but are expected to be revised to be compatible with NZS 1170.5:2004. The revised wind and earthquake test and evaluation method (EM3-V3) was derived from engineering analysis to ensure the as-built house strength from walls assessed by EM3-V3 will achieve the NZS 1170.5 intent in a reliable but economical manner.

In the EM3-V3 method, the design seismic bracing strength of a test wall is obtained by factoring a selected peak resisted racking load by a two parameters, referred to as F1 and F2. The relationship between F1 and wall deflection was determined by computer simulation, using typical test hysteresis loops, and design level earthquake records. Factor F2 is called the 'systems factor' and was selected to represent the reliable strength enhancement of a total house compared to the sum of the racking strengths of individual bracing walls when isolated from the house and tested separately.

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1. INTRODUCTION

1.1 Bracing of New Zealand houses for wind and earthquake forces

Most New Zealand houses are built of timber stick framed construction and lined with plasterboard. Earthquake and wind demand loads are specified in NZS 3604 (SNZ, 1999). Various lining and cladding manufacturers publish bracing strengths for their wall systems which are currently based on the BRANZ P21 racking test. The P21 tests are performed on a representative length of wall with 'special' uplift restraints to simulate continuity of actual construction. For both major building axes a designer of a particular house: determines the predicted resistance of each discrete bracing wall (i.e. between door/window openings and corners) from manufacturers' published wall system bracing values; sums the resistances of all such walls; and ensures that this exceeds the NZS 3604 stipulated demand loads.

Earthquake loads in NZS 3604 have historically been based on that specified in the current loadings standard (e.g. SNZ, 1992). There was a significant increase in earthquake demand from NZS 3604:1990 (SNZ, 1990) to NZS 3604:1999 (SNZ, 1999) This created concern in the industry as there has been little field evidence that the average NZS 3604:1990 modern house would be inadequately braced when subjected to a design level earthquake. However, New Zealand has not experienced a large earthquake in an urban area since the 1931 Napier earthquake. At the time of writing NZS 3604 is to be revised with earthquake loads based on AS 1170.5 (SNZ, 2004) which may increase demand loads further. For the purposes of this report it was assumed that although the enhanced demand is technically justified, it is not desirable to be unduly conservative in the evaluation of wall bracing ratings.

In bracing calculations for houses submitted for approval to Territorial Authorities in New Zealand, plasterboard wall systems are most commonly used to meet the bracing demand. In the USA and Australia this is mainly achieved with plywood and orientated strand board (OSB) sheathed walls.

1.2 House bracing wall design philosophy

A bracing test evaluation method needs to take into account the return period of the design event and the likely life risk and property damage of the design event.

The design loads in NZS 4203 (SNZ, 1992) and NZS 1170.5 (SNZ, 2004) are based on the philosophy that there is only a 10% probability that New Zealand houses will experience an earthquake exceeding the design earthquake in any 50-year period.

A suitable design philosophy is considered to be that bracing walls only sustain minor cracks at serviceability limit state earthquakes, are readily repairable after a design ultimate event, and do not collapse in an extreme event.

2. BACKGROUND

2.1 The P21 test method and its deficiencies

The BRANZ P21 test was based on research at Forest Research Institute (Collins, 1975) and subsequent unpublished testing at BRANZ. The P21 wall bracing test and evaluation procedure was first published by BRANZ in 1979 (Cooney and Collin 1979) and was revised in both 1982 and 1987. This used a working stress approach where the bracing strength depended on the force resisted when it was cyclically loaded to 8 mm deflection. (Further loading ensured that there was adequate reserve strength and ductility for the ultimate loading case.)

NZS 3604 was revised in 1990 (SNZ, 1990) to reflect limit state design (LSD) philosophy. The BRANZ *Technical Recommendation TR 10* (King and Lim 1991) modified the P21 evaluation method to bring it into line with the 1990 revision, and this resulted in the bracing strength being generally dependent on the specimen ductility and maximum forces resisted.

A review of the P21 test and evaluation procedure was prompted by the following deficiencies:

1. The wall ductility is (mathematically) incorrectly evaluated in the P21 evaluation, (Thurston and King 1992). Although house walls tend to give pinched hysteresis loops when racked, the earthquake bracing rating is evaluated by assuming the system has elasto-plastic loops with an assigned ductility of 4 if the deflection at half the maximum load does not exceed the deflection at (maximum load)/4. As an example, consider Figure 1. Here the maximum load of 13 kN occurs at 16 mm deflection and the deflection at half maximum load (i.e. 6.5 kN) = 1.8 mm, giving an apparent ductility of 16/1.8 = 8.9 which is clearly excessive. As another example, the P21 method assigns a ductility of 2.0 to perfectly elastic walls rather than the correct value of 1.0. In contrast, the EM3-V3 method assigns a design seismic bracing rating as a function of the wall peak strength, shape of the hysteresis loops as identified by wall type, and also the deflection at which the peak strength is reached. This is expected to provide a more reliable assessment of wall design strength.

2. The strength of many walls tested using the current P21 test is limited by the P21 'special' end restraint and the wall consequently deflects by a rocking mechanism as shown in Figure 4. This is called the 'rocking strength'. Thus, the assessed wall racking strength is **not** a function of the characteristics of the wall fasteners and sheathing, but rather a function of the contrived end restraint as described in Section 3.6. However the actual racking strength of most house walls will be governed by their shear strength which is usually governed by fastener connection strength. EM3-V3 overcomes this dilemma by testing walls with both weak and strong uplift restraint as discussed in Section 7.

3. If a wall rocks in a P21 racking test then this mechanism acts like a ductile fuse to protect the rest of the wall and can prevent a non-ductile failure from occurring. This protection is unlikely to be replicated in the field. This feature in itself invalidates assessment of wall ductility in houses based on P21 tests. As the EM3-V3 method tests each wall construction with both a low (i.e. the current P21 end restraint) and full end restraint, and uses the lowest results from the two end conditions, the EM3-V3 method best simulates all realistic construction situations.

4. P21 methodology usually results in little extra bracing by the addition of a second sheathing as this construction usually results in the wall strength being governed by the rocking strength. In some overseas standards, e.g. in the USA, the addition of a second sheathing can double the bracing rating.

5. Compared with standards overseas, the bracing rating used in New Zealand for walls with non-ductile linings (such as standard plasterboard) is proportionally significantly higher compared to the rating of more robust sheathings (such as plywood or MDF). The term 'non-ductile' is intended to imply a relatively rapid drop-off in load at deflections greater than that at maximum racking strength

and with the deflection at this maximum racking strength being relatively low – say less than 20 mm.

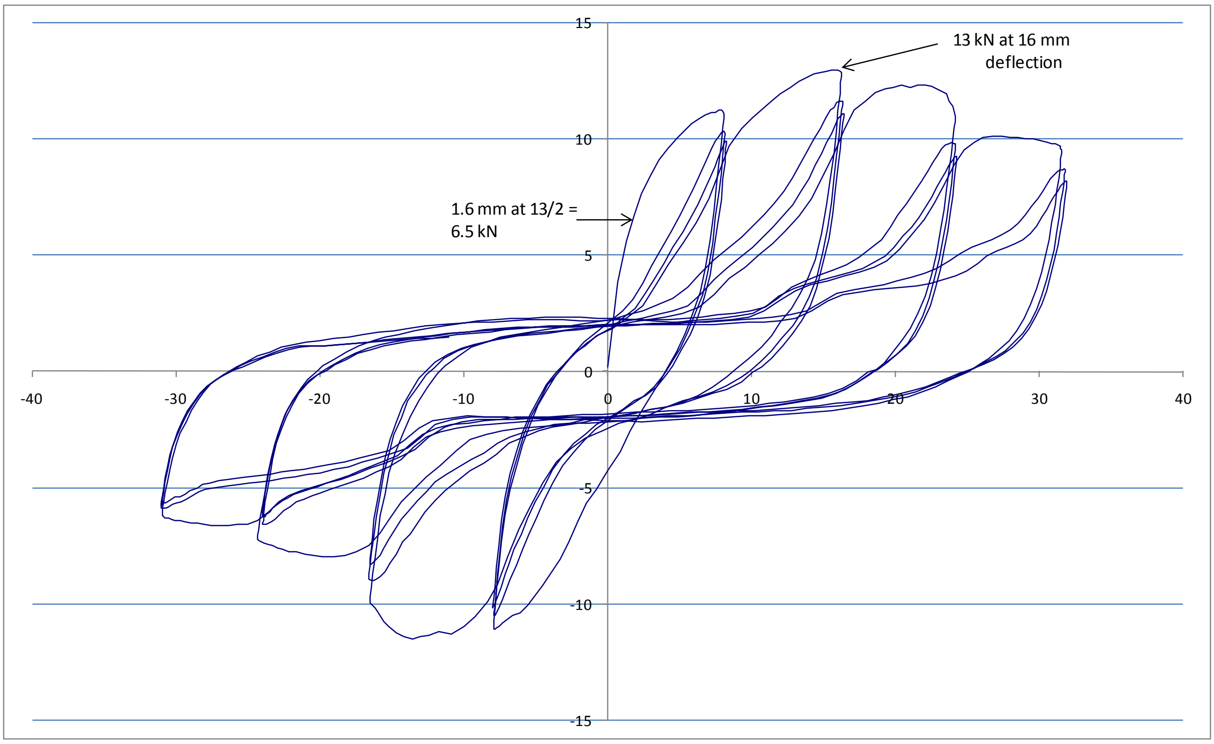

Figure 1. Sample P21 test result giving an apparent ductility of 8.9

2.2 History of EM3

The test method to replace P21 was allocated the working title 'EM3', an abbreviation of BRANZ Evaluation Method 3. EM3 has gone through several iterations with the milestones identified as EM3-V1, V2 and now V3. EM3-V3 is given in Appendix B.

2.3 EM3-V1

EM3-V1 was based on earthquakes corresponding to the NZS 4203 (SNZ, 1992) design spectra and was the subject of a paper by Thurston and Park (2003). The relationship between wall seismic mass, wall hysteresis loops and wall deflection under excitation from earthquakes corresponding to NZS 4203 spectra was found by a computer study where buildings were analysed by inelastic time history seismic analysis using the Ruaumoko 2D software (Carr, 2000) with the Stewart hysteresis element (Stewart, 1987). Davidson (2000) earthquakes were used for this simulation which were a modified form of measured earthquake records massaged so the spectra closely matched NZS 4203 spectra.

The analyses used a suite of earthquakes which had elastic spectra corresponding to the design elastic spectra of NZS 4203 (SNZ, 1992). 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. For each modelled structure, a series of computer runs was performed to compute the maximum deflection, Δ_{max}, for a range of the seismic weights, W. A hysteresis factor, F1, was then derived to account for this as described in Section 6.

Thurston and Park plotted F1 versus Δ_{max} for different basic hysteresis loops for single-storey buildings and this relationship averaged between plywood and plasterboard walls was adopted in the EM3-V1 method (Thurston and Park, 2003).

To account for the strength enhancement due to the presence of non-structural elements and also 'systems effects' they introduced a second factor, F2, which varied between 1.0 and 1.2 depending on the 'toughness' of the wall. The toughness was defined as the ability of the wall to deflect to 45 mm with only small strength loss.

2.3.1 EM3-V2

Following recommendations of a peer review meeting of industry leaders and technical experts to discuss EM3-V1, an application document was written which dealt with the application of the bracing test results and all discussion aspects were removed from the test method document. This was to allow code committees a choice of what factors/aspects they wished adopt. The resulting documents were called EM3-V2.

2.3.2 EM3-V3

A second peer review meeting of industry leaders and technical experts was held to discuss EM3-V2. There was vigorous discussion on the earthquake records used to derive the F1 factors, and it was considered by some that the energy level in the Davidson (2000) earthquakes (defined in Section 2.2.1) was excessive. As a draft version of the standard NZS 1170.5 (SNZ, 2002) was near to being adopted as a standard to replace NZS 4203 (SNZ, 1992), it was recommended that the computer analysis be repeated but using the earthquake selection method stipulated in the draft standard DZ 1170.5.

There was also vigorous discussion regarding the most suitable end restraint to be used in the test. It was recognised that walls in the lower-storey of two-storey walls experienced close to full uplift restraint. Top-storey and single-storey walls terminating at corners and window openings experience more than the 'three-nail' end restraint currently used in a P21 test. However results by Herbert and King (1998) indicate that walls terminating at doorways and free ends experienced approximately half that of the three-nail end restraint used in the P21 test. EM3-V3 (see Appendix B) continues to use the three-nail uplift restraint although it is conservative in many instances when the rocking deformation mode dominates.

3. OVERVIEW OF EM3

3.1 Proposed EM3-V3 test regime

An EM3 test, (see Appendix B), involves in-plane racking of a bracing wall specimen in a predetermined series of incrementally increasing cyclic deflections, in both push and pull directions. The deflections are to ±8 mm, ±15 mm, ±22 mm, ±29 mm, ±36 mm, with three cycles at each increment.

The peak force resisted during each first and third cycle (both push and pull) is extracted from the test data and is used in the evaluation procedure to determine the wind and earthquake bracing rating.

Typical hysteresis loops (load versus deflection plot) from such a test are shown in Figure 2.

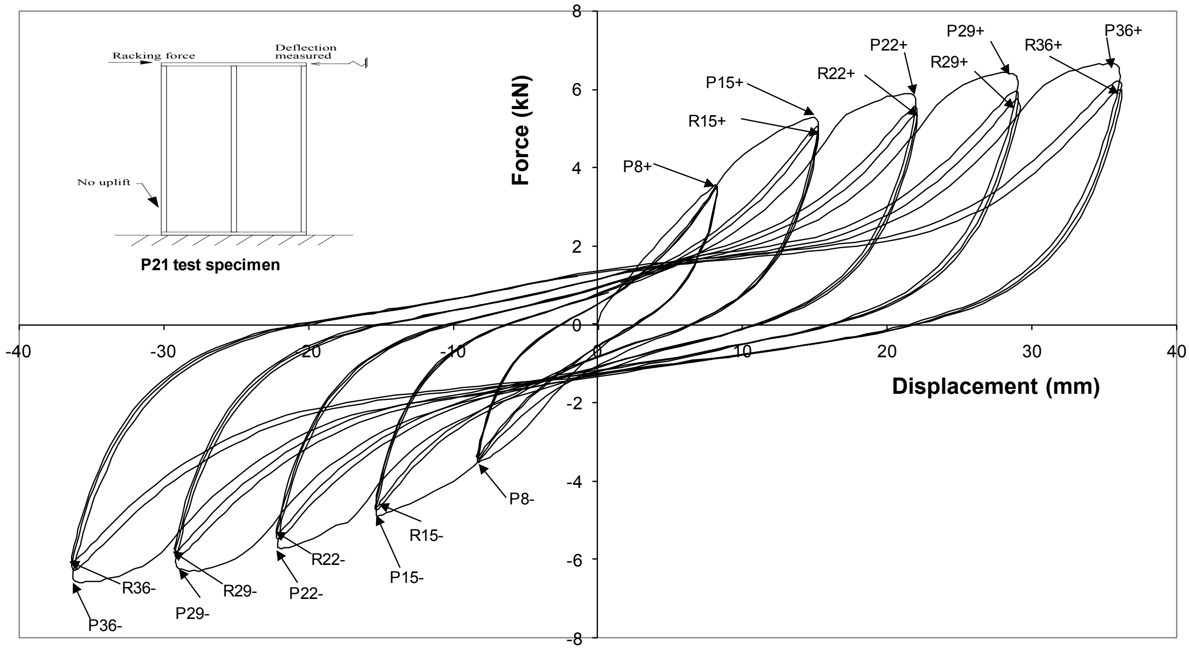

Figure 2. Hysteresis loops from a typical EM3-V3 test

To allow for continuity of construction (which cannot be directly simulated in a bracing test), three-nail special end restraints are used, as was the case in the P21 method.

Both EM3-V3 and the current P21 determine the bracing rating of a test wall for both serviceability (at the 8 mm deflection), and ultimate (between 15 and 36 mm deflections) limit states. The lowest of these two values is used as the bracing rating.

The bracing rating for wind is obtained directly from the peak load resisted, as described in Sections 4.1 (SLS) and 5.1 (ULS).

The seismic bracing rating is obtained by factoring a selected peak resisted load by two parameters, referred to as F1 and F2. Both these factors are described briefly below, and their derivation in more detail in Section 6.

Factor F1 is called the 'hysteretic factor'. The relationship between F1 and wall deflection was determined by computer simulation, using typical test hysteresis loops, and design level earthquake records. The equations used to determine F1 are described in Section 3.2. The methodology and greater detail are given in Section 6.

Factor F2 is called the 'systems factor' and was selected to represent the reliable strength enhancement of a total house compared to the sum of the racking strengths of individual bracing walls when isolated from the house and tested separately. The background to F2 is described in Section 3.3.

The respective bracing ratings are the lowest value derived from three replicate test walls. The implication of this is discussed in Section 3.3.1.

3.2 'Hysteretic effects' and F1 factor

The purpose of a bracing rating test is to allocate a wall bracing element a bracing rating resistance. This rating must be determined on the same basis that the demand load stipulated in NZS 3604 was determined. For the earthquake rating the difficulty is that the test is undertaken in a slow cyclic regime whereas the demand load is a simplified value derived from consideration of a building's response to random ground motion.

The equations used to determine F1 values are given below. A description of the computer analysis and methodology is given in Section 6. The BRANZ EM3 test method postulates that at ULS the seismic design resistance (SDR) of a sheathed timber framed wall may be given by:

$$SDR = F1 \times R_{\Delta}$$

where:

- R_{Δ} is the 3rd cycle peak load of a bracing wall in BRANZ EM3-V3 test at a given deflection Δ.
- F1 is a hysteretic factor which is derived by Eqn (2). It is a function of Δ.

NZS 3604 seismic demand forces are based on the loadings given in NZS 4203:1992. Shelton (2007) stated that the following formula was used to derive the demand earthquake load for Zone A (e.g. Wellington) in NZS 3606:1999:

V (Design lateral seismic base shear) = 0.241 W where W = building weight.

For Zone C, (e.g. Auckland) V = 0.1205 W.

Equating the demand, V, to the resistance, SDR, gives F1 = 0.241 W/R_{Δ} for Zone A and F1 = 0.1205 W/R_{Δ} for Zone C.

This report and EM3-V3 give the F1 factors for the loading currently stipulated in NZS 3604. If NZS 3604 changes the loadings tables when it adopts NZS 1170.5 (2004), then the F1 factors will need to be revised.

3.3 'Systems effects' and F2 factor

Full house tests reported by others, as summarised by Thurston (2003) and Fischer et al (2000), have reported that houses under racking load are stiffer and stronger than the sum of the designated individual bracing elements. This is attributed to load sharing and composite action of both the structural and non-structural elements. In addition, the taped and filled joints between plasterboard sheet lining at wall ends and ceiling junctions significantly increase wall racking strength, as illustrated in Figure 3. This is due to the increased uplift restraint at wall ends and by changing the deformation mechanism from the sheet rotating about its centroid, as shown in Figure 4(c), to close to pure translation along the bottom plate. This is acknowledged in EM3 with a 'systems effect' factor F2.

To investigate the 'systems effect' an existing experimental house on the BRANZ site was relined with plasterboard and cyclically racked to failure (Thurston, 2003). The report compared the actual house strength with the strength determined using the NZS 3604:1999 design provisions (i.e. summing wall strengths derived from P21 tests). The averaged cyclic strength of the lined house was 50% greater than that predicted based on summing all the component walls and assuming all the walls were restrained against uplift. Although it was recognised that this is but one example of a typical New Zealand house, it indicated that simple summing of all component bracing walls gives a conservative estimate of total house strength for single-storey structures.

The restraint against rocking of walls built into houses is often larger than simulated in a P21 test. This premise is supported by Thurston (2003) who found measured wall uplifts were very small, despite the walls having only single bottom-plate nails at 600 mm centres and fixings of internal walls being only into the sheet flooring.

Collier (2005) performed a series of racking tests on 3 m high x 3 m wide walls. These were fully restrained against rocking. By comparing the strength of walls with and without end corners and wall/ceiling junctions, Collier derived strength enhancement (i.e. F2) factors which exceeded 2.0.

EM3-V3 takes some account of this enhanced stiffness and strength (hereafter called 'systems effects'), by increasing the tested strength by a factor, F2, which is conservatively taken as 1.2.

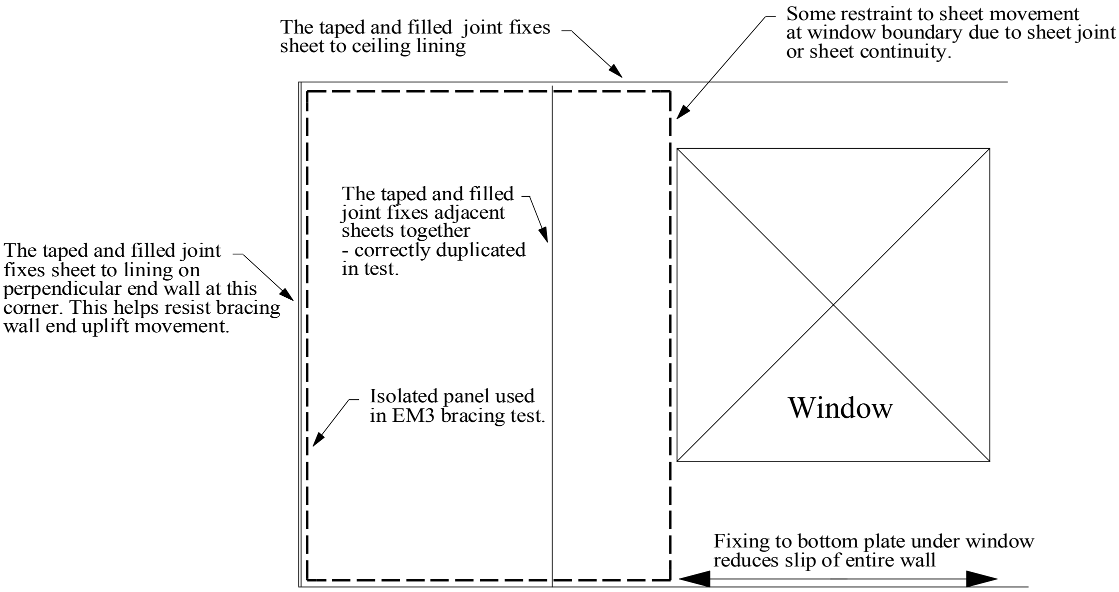

Figure 3. Restraint of wall lining in real buildings

3.3.1 Use of statistics in derivation of design strengths

Most standards for the derivation of structural design parameters take into account the statistical spread of test results. For example they may use the lower five percentile probability limit. However the P21 method derives the design strengths based on the average of three tests.

An evaluation based on the lowest of three test results is used in EM3-V3. Using complex statistical methods to assess strength variability was rejected as there are only three replicate tests conducted to derive a bracing resistance. Using a greater number of replicate tests than three was rejected due to the cost of testing.

Neither the P21 nor EM3-V3 methodologies take into account differences between site and laboratory construction i.e. strength reduction factors as used in material standards are assumed to be 1.0. This is justified by good historic performance of houses in major events, the rarity of such events, the low life risk in the event of failure and the expected economics of the average annual cost of providing additional house bracing against the average annual cost of damage from wind and earthquake attack.

3.4 Deflection compatibility

Houses in New Zealand are generally constructed with timber framed walls, each with a variety of 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 the 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 10-15 mm shear deflection and then drop in strength while plywood systems continue providing dependable and increasing resistance for 40-60 mm deflection. The proposed EM3-V3 method addresses this problem by assessing the bracing resistances in a deflection band (15-36 mm) to ensure at least moderate compatibility. However it is recognised that because different elements reach peak loads at different deflections, simply adding the EM3-V3 bracing strengths of each house element to obtain the complete house strength is inherently unconservative.

3.5 Damping

The hysteretic damping due to degradation of bracing walls is directly taken into account in the computer modelling used to derive the hysteretic factor F1 by using realistic hysteresis loops (Thurston and Park, 2003). When non-structural elements degrade under racking, they also contribute to damping of building motion. The computer analyses assumed a 5% damping ratio to account for this and other forms of damping (as also assumed in development of the NZS 4203 spectra (SNZ,1999). This is described in Section 6.

3.6 Purpose of 'special' uplift restraints

Many New Zealand house bracing walls are only nominally anchored to the foundation with bottom plate nailing (timber floors) or using short-fired fasteners (concrete floors). Other walls are more robustly held down using 'special uplift restraints' such as:

- Walls founded on timber floors use steel straps connecting the end studs to floor joists or joist blocking.
- Steel straps are wrapped around the bottom plate and fixed to the end studs to resist the tendency for the two to separate. The bottom plate is also anchored to the foundation which may be timber or concrete.
- Exterior sheathing may extend past the bottom plate and be fixed directly to boundary joists.

The special uplift restraints described above are part of the proprietary bracing system which is tested.

If bracing panels which are only nominally anchored to the foundation are isolated from the surrounding structure and laboratory tested under horizontal racking loads, without any 'special' end restraints to simulate continuity of actual construction, they usually only achieve a low racking strength. This is due to 'rocking' of the test wall about the bottom compression corner as shown in Figure 4(b). The associated uplift at the other (tension) end is due to either uplift of the bottom plate as the nails pull out of the timber floor or the studs lift from the bottom plate as the end nails withdraw. Systems with more robust hold-downs will experience less uplift in the EM3 test.

When bracing panels are built into a house, the wall sheathing, framing continuity and gravity effects provide resistance to uplifting, thereby reducing rocking effects and increasing the house racking stiffness. To simulate this a special end restraint was added to the P21 test set-up.

The current P21 method uses a three-nail special end restraint, which was designed to replicate the minimum nail fixing used at wall corner junctions. This ignores the additional uplift restraint due to the usual taped and filled joints in corners of plasterboard-lined houses.

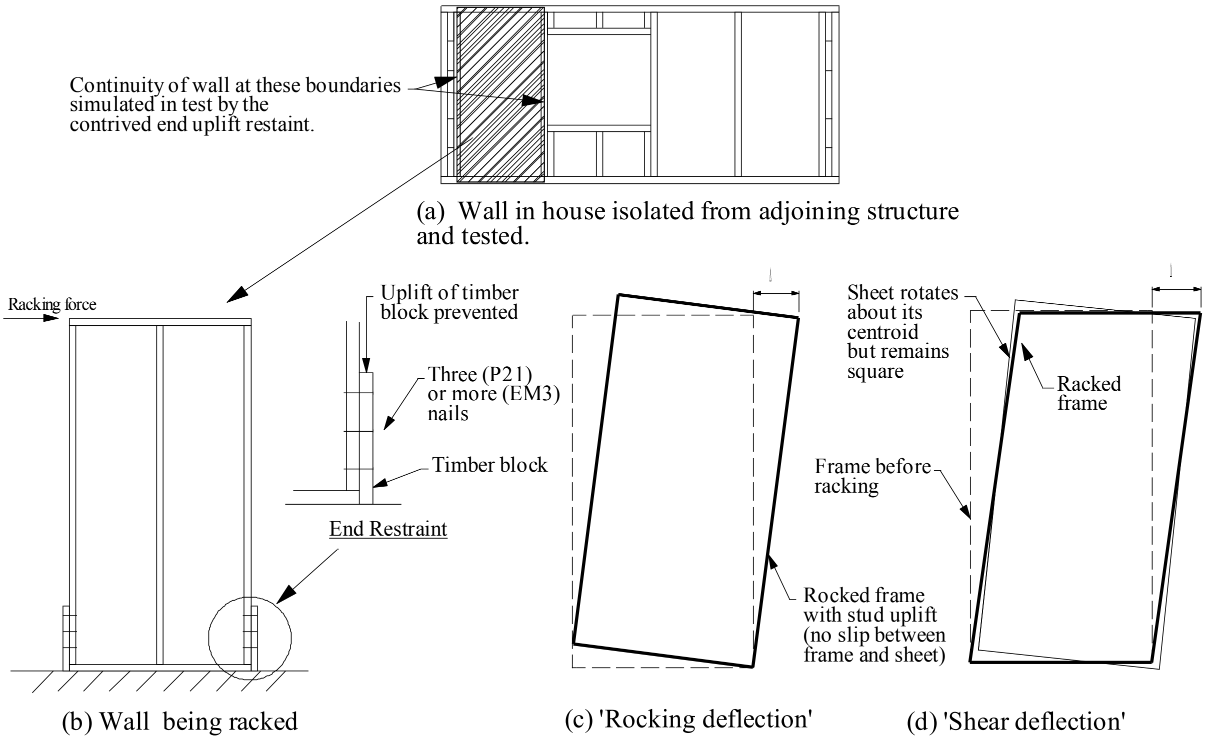

Figure 4. Components of racking wall deflection and sketch of the BRANZ P21 uplift restraints

Herbert and King (1998) reviewed wall racking tests used overseas. New Zealand appears to be the only country which attempts to simulate in-service continuity effects when testing isolated bracing panels by providing partial end stud 'special' uplift restraints. Other countries adopt either a full end stud hold-down (Japan and USA) by means of tie-rods or do not use any 'special' uplift restraints (Australia and the UK). Using no restraint can give unduly conservative results. Full uplift restraint is best for comparing the performance of sheathings under pure shear load. However it does not give an upper limit for when rocking will occur in actual construction, and does not simulate the potential of fixings from sheathing to bottom plate to pull-out perpendicular to the edge of the sheathing under rocking action.

It is difficult to select an appropriate level of uplift restraint to simulate actual construction. Too low a level and the bracing rating for walls without 'special uplift restraints' is too conservative, and the walls will get a rating which is purely a function of the 'special' uplift restraint and not the sheathing fixing strength. Too high and the racking strength may be overestimated for some walls. A low uplift restraint is appropriate for non-loadbearing walls adjacent to large openings in single-storey construction or building upper-storeys. A high uplift restraint is appropriate for loadbearing walls, walls near internal or external corners where plasterboard is fully stopped and in lower-storey walls. However it was decided to remain with the current three-nail uplift restraint, although this will be conservative much of the time.

4. BRACING RATINGS BASED ON SERVICEABILITY LIMIT STATE CRITERIA

Both EM3-V3 and the current P21 evaluation determine the design bracing rating of a test wall for both serviceability and ultimate limit state (ULS) criteria. The lowest of these two values is used. The ULS criteria are discussed in Section 5.

NZS 3604 demand loads are based on ULS criteria. Thus a serviceability limit state (SLS) rating must be obtained from the ratio of SLS/ULS demand loads from the loadings standard. The design bracing rating of a test wall, BR_s, based on serviceability criteria, is defined:

$$BR_s = \frac{\textit{Ultimate limit state demand load}}{\textit{Serviceability limit state demand load}} \; x \; \textit{Test load at serviceability deflection}$$

We can use the above equation to define LR (i.e. loading ratio)

Thus, BR_s = LR x *Test load at serviceability deflection* ………….. (1)

In Eqn (1), the limit state loads used are that given in the relevant loadings standard for wind or earthquake as applicable. Values of LR are calculated below for both P21 and EM3 test methods and for both wind and earthquake loading. This enables BR_s, based on serviceability criteria, to be determined from Eqn (1).

Both EM3-V3 and the current P21 evaluation methods have assumed that at the serviceability load the wall deflection should not exceed the deflection at which damage commences to plasterboard walls – taken as wall height/300 which is 8 mm racking deflection at the top plate level for a 2.4 m high wall.

4.1 Calculation for wind loading

4.1.1 Method used by P21

The P21 test method was based on a draft version of the standard NZS 4203:1992 (King and Lim, 1991) where LR = the ratio of ultimate wind force to serviceability wind force. The force is proportional to (wind speed).[2] Hence:

LR = $(\frac{\text{ultimate wind speed}}{\text{serviceability wind speed}})^2 = (\frac{1}{0.75})^2$ = 1.78.

Table 5.4.2 of the standard now gives the ratio, LR, as being: $(\frac{0.93}{0.75})^2$ = 1.54. However the original ratio of LR = 1.78 was retained in the P21 evaluation as it was considered that at low serviceability loads a house has large non-structural stiffening. Thus a systems factor of $\frac{1.78}{1.54}$ = 1.16 was effectively assumed for serviceability level wind loading.

4.1.2 Method used by EM3-V3

Table 3.3 of AS/NZS 1170.0 requires the serviceability level for loadings on houses to be set so that the annual probability of this or greater loading occurring is less than 1/25. Using the formula below (Table 3.1 of AS/NZS 1170:2 2002), and taking the ultimate wind speed as having a return period of 500 years, the ratio of serviceability wind to ultimate wind force, LR, is given by:

$LR = (\frac{51.08}{43.40})^2$ = 1.39 for wind zones A(1-7) and $LR = (\frac{44.98}{37.28})^2 =$ 1.46 for Zone W.

It is proposed to use a systems factor F2 = 1.3 for SLS wind. This is higher than the F2 = 1.0 proposed for ULS wind in Section 5.1. The reasons for the higher F2 factor at SLS are discussed in Section 5.1. When this systems factor is used with the LR ratios calculated above this will result in a modified ratio, LR, of 1.39 x 1.3 = 1.81 which is similar and slightly less conservative than the value of LR = 1.78 in the current P21 method. Hence wind bracing ratings already established to the current P21 method will not need to be re-evaluated for EM3-V3.

4.2 Calculation for earthquake loading

4.2.1 Method used by P21

From NZS 4203:1992, if it is assumed that $\mu = 1.25$ at SLS then the earthquake demand force at serviceability limit is given by:

$F' = C_h(T1,1.25) \times S_p \times R \times Z \times L_s \times$ building weight.

Shelton (2007) stated that NZS 3604:1999 ULS demand tables are obtained from the following equation:

$F = 0.3 \times S_p \times R \times Z \times L_u \times$ building weight ……… (2).

Hence the ratio of ultimate earthquake force to serviceability earthquake force, LR, is given by:

$LR = F/F' = (0.3 \times L_s / L_u)/C_h(T_1,1.25) = 0.3 \times 6/0.69 = 2.61$ (for a building of period T1 = 0.4 seconds and intermediate soils).

4.2.2 Method used by EM3-V3

From Section 5.2.1.2 and Eqn 5.2(1) of AS/NZS 1170:5 2004, the earthquake demand force at serviceability limit is given by:

$F' = C(T_1)\ S_p/K_\mu \times$ building weight $= C_h(T_1)\ ZR_sN(T_1,D)S_p/K_\mu \times$ building weight.

Hence $F' = 2.36 \times 0.4 \times 0.25 \times 1 \times 0.7/K_\mu \times$ building weight (for shallow soils in Wellington).

Taking $\mu = 1.25$ and $T_1 = 0.4$ (Wellington) results in $K_\mu = (0.25 \times 0.4)/0.7 + 1 = 1.14$.

Therefore $F' = 0.145 \times$ building weight (for shallow soils in Wellington).

For Wellington, Eqn (2) can be refined to be:

$F = 0.3 \times S_p \times R \times Z \times L_u \times$ building weight $= 0.3 \times 0.67 \times 1 \times 1.2 \times 1 \times$ building weight.

Hence the ratio of ultimate earthquake force to serviceability earthquake force for Wellington, LR, is given by:

$F/F' = (0.3 \times 0.67 \times 1.2)/0.145 = 1.66$ (for a building of period T1 = 0.4 seconds and intermediate soils).

As with the wind in Section 4.1.2, it is proposed to use a systems factor of 1.3 which will result in a modified ratio, LR, of 1.66 x 1.3 = 2.16 which is more conservative than the value of LR = 2.61 in the current P21 method.

5. BRACING RATING BASED ON ULTIMATE LIMIT STATE CRITERIA

5.1 Calculation for wind loading

The wall bracing rating for wind loading was simply taken as the average of the maximum ‘push and pull’ forces recorded during the racking tests. This makes no allowance for racking strength enhancement due to ‘systems effect’, i.e. the ‘systems factor’ called F2, was taken as 1.0. This is because a wall may be subjected to wind uplift and racking forces simultaneously which will increase the tendency of the wall to rock and hence may reduce its bracing strength. As this wind uplift is not simulated in the EM3-V3 racking test, it was assumed that the reduction in strength due to wind uplift was balanced by strength enhancement due to ‘systems effect’ i.e. F2 = 1.0. This is likely to be a conservative assumption, but was considered to be appropriate due to lack of information.

In Section 4.1.2, which considered EM3 wind serviceability criteria, it was assumed that F2 = 1.3 even though the walls could again be subject to uplift. This is because wind uplift is less likely to exceed gravity load at serviceability level winds and the 'systems effect' is likely to be greater at serviceability limit state than ULS. Thus EM3 uses F2 = 1.3 for wind at SLS and 1.0 at ULS.

5.2 Calculation for earthquake loading

An output of the EM3-V3 bracing tests is hysteresis loops of wall racking force versus deflection. The challenge is to use this information to determine the maximum seismic mass that can be carried by the test wall so that it will not rack beyond a maximum reliable deflection when subjected to design level earthquakes. This is achieved by assuming that the seismic strength of an isolated wall (i.e. when not built into a house) can be defined to be F1 x R where R is the average of the 'push' and 'pull' peak wall strength after three cycles to a selected deflection, Δ. F1 is a factor which is a function of wall deflection and was determined in this research using the procedure described in Section 6. The wall bracing rating for a wall when built into a house is taken as F1 x F2 x R, where F2 is a factor to account for wall strength enhancement when it is constructed into a building as against the isolated wall used in the racking test. F2 is taken as 1.2 as discussed in Section 3.3.

Summary of LR and F2 factors

Table 1 summarises the LR and F2 factors determined in Sections 4 and 5. Note: LR = 1.0 at ULS from definition.

Table 1. Summary of LR and F2 factors

Test Type	Serviceability limit state				Ultimate limit state	
	Wind		Earthquake		Wind	Earthquake
	LR	F2	LR	F2	F2	F2
P21 test	1.78	1.16	2.61	1.0	1.0	1.0
EM3 test	1.81	1.3	1.66	1.3	1.0	1.2

6. DETERMINATION OF F1 FACTORS

To determine values for the F1 factor, a four step procedure was followed:

(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 earthquakes selected using the criteria of NZS 1170.5. This was done for Wellington, Christchurch and Auckland to represent Zones A, B and C of NZS 3604 respectively. Walls were modelled in a non-linear dynamic structural analysis package – Ruaumoko 2D (Carr, 2000) – using the Stewart hysteresis element (Stewart, 1987), with parameters selected to provide a match with typical experimental hysteresis loops. A series of computer runs were performed to compute the maximum deflection, Δ_{max}, for an appropriate range of seismic weights, W, earthquake records, hysteresis loop shapes considered and for single and two-storey buildings.

(2). For each computer run, F1 values were found from $F1 = 0.241\ W/R_{\Delta}$ for Zone A and $F1 = 0.1205\ W/R_{\Delta}$ for Zone C. For this calculation the third cycle peak strength, R, at Δ_{max}, was determined from examination of the modelled hysteresis loops.

(3). A table of F1 and corresponding values of Δ_{max} was generated from Step (2).

(4). A best fit linear relationship between F1's and Δ_{max} was determined by regressional analysis. As slightly lower values of F1s were obtained from the single-storey analyses, these were the values used in the final recommendations which will therefore be slightly conservative for two-storey buildings.

For example if a wall has R_{Δ} = 10 kN at a test deflection of Δ = 25 mm, and F1 was computed to be 0.7 using the above methodology, then the evaluated design strength of the wall would be 10 x 0.7 = 7 kN. When used in Zone A (as defined in NZS 3604) the wall is assumed to deflect a maximum of 25 mm when subjected to the design NZS 1170.5 earthquakes for this zone if it carries a seismic weight of 7/0.241 = 29 kN.

This report and EM3-V3 give the F1 factors for the loading currently stipulated in NZS 3604. If NZS 3604 changes the loading demand tables when it adopts NZS 1170.5 (2004), then the F1 factors will need to be revised. If, as expected, the earthquake demand loads in NZS 3604 increase for Wellington, then the F1 factors will correspondingly increase.

6.1 Earthquake records used

6.1.1 GNS earthquakes (draft 1170.5) set

Details of the Geological and Nuclear Science (GNS) supplied earthquakes and method of scaling are given in Appendix A and summarised in Table 2. The target spectra, C(T), was obtained from DZ 1170.5 (SNZ, 2002) using the following equation:

$C(T) = C_h(T,1) \times R \times Z \times N(D,T)$ ……. (3).

Note that the zone factors, 'Z', are 0.4, 0.13 and 0.22 for Zones A, B and C of NZS 3604 respectively and thus differ from the 'Z' factors given in NZS 4203. The factor N(D,T) = 1.0 for buildings with a natural period less than 1.5 seconds, such as the buildings in this study. This is called the 'draft 1170.5' set.

6.1.2 GNS earthquakes (1170.5) set

Eqn 5.5(1) of NZS 1170.5 (SNZ, 2004) revised DZ 1170.5 (SA/SNZ, 2002) by requiring the target spectrum to be that given by Eqn (2) but factored by $(1+S_p)/2$. Clause 4.4 of NZS 1170.5 specifies S_p to be 0.7 for buildings with a ductility greater than 2.0 (such as houses). Output from these earthquakes is called the '1170.5' set.

This report was written when the requirements of the above paragraph were published. To avoid redoing all computer runs, only the most critical situations were re-run. Output is given in Sections 6.5.5 to Section 6.5.7. The remainder of the report gives the results for the 'draft standard' earthquake set.

Table 2. Summary of K_1 and K_2 factors provided by GNS

<table>
<tr><th>City Name</th><th>Site Condition</th><th>Earthquake Record</th><th>Principal Component</th><th>Scale Factor K_1</th><th>Scale Factor K_2</th></tr>
<tr><td rowspan="6">Wellington</td><td rowspan="3">Shallow</td><td>Tabas, Tabas 1978</td><td>TABAS78_2.acc</td><td>0.48</td><td rowspan="3">1.10</td></tr>
<tr><td>La Union, Mexico 1985</td><td>LAUNION85_1.acc</td><td>2.32</td></tr>
<tr><td>El Centro, Imperial Valley 1940</td><td>ELCETNRO40_2.acc</td><td>1.39</td></tr>
<tr><td rowspan="3">Deep</td><td>El Centro, Imperial Valley 1940</td><td>ELCENTRO40_2.acc</td><td>1.63</td><td rowspan="3">1.02</td></tr>
<tr><td>Joshua Tree, Landers 1992</td><td>JOSHUA92_2.acc</td><td>1.99</td></tr>
<tr><td>Llolleo, Chile 1985</td><td>LLOLLEO85_2.acc</td><td>0.79</td></tr>
<tr><td rowspan="6">Auckland</td><td rowspan="3">Shallow</td><td>Delta, Imperial Valley 1979</td><td>DELTA79_1.acc</td><td>0.53</td><td rowspan="3">1.00</td></tr>
<tr><td>Matahina, Edgecumbe 1987</td><td>EDGECUMBE87_2.acc</td><td>0.43</td></tr>
<tr><td>Bovino, Campo Lucano 1980</td><td>BOVINO80_1.acc</td><td>2.91</td></tr>
<tr><td rowspan="3">Deep</td><td>Korinthos-OTE, Aikon 1981</td><td>KORINTHOS_2.acc</td><td>0.61</td><td rowspan="3">1.13</td></tr>
<tr><td>Delta, Imperial Valley 1979</td><td>DELTA79_1.acc</td><td>0.62</td></tr>
<tr><td>Calipatra Fire Station, Imperial Valley 1979</td><td>CALIPATRA79_2.acc</td><td>1.71</td></tr>
<tr><td rowspan="3">Christchurch</td><td rowspan="3">Deep</td><td>70 Boshrooyeh, Tabas 1978</td><td>BOSTABAS78_2.acc</td><td>2.69</td><td rowspan="3">1.07</td></tr>
<tr><td>Taft, Kern County 1952</td><td>TAFT52_1.acc</td><td>1.56</td></tr>
<tr><td>Wrightwood, Lyttle Creek 1970</td><td>WTWLYTLE_2.acc</td><td>1.82</td></tr>
</table>

6.2 Hysteresis model

Computer simulation was performed using a non-linear time-history 2D computer package called Ruaumoko (Carr, 2000) as explained in Section 6. By varying the hysteretic model input parameters, experimental wall racking hysteresis loops that have been tested at BRANZ were matched to the Stewart element hysteresis loops for 1.2 m long walls lined with either standard plasterboard or medium-density fibreboard (MDF). The good match obtained is shown in Figure 5 and Figure 6 respectively.

A wall lined with a high performance plasterboard called Braceline, which is formulated as a bracing board and has fibreglass content in the core, was also racked although it used different fasteners than the standard plasterboard wall. A comparison of the Braceline hysteresis loops (with the load factored by 0.59) with those for standard plasterboard is given in Figure 7. Agreement is good. Hence it was considered that Braceline hysteresis loops were just a scaled version of standard plasterboard hysteresis loops, and thus there was no purpose in running computer simulations of both materials as the results would provide the same F1 values.

Similarly a plywood sheathed wall was found to give almost identical hysteresis loops as the MDF sheathing (see Figure 8). Thus only MDF and not plywood sheathing was modelled.

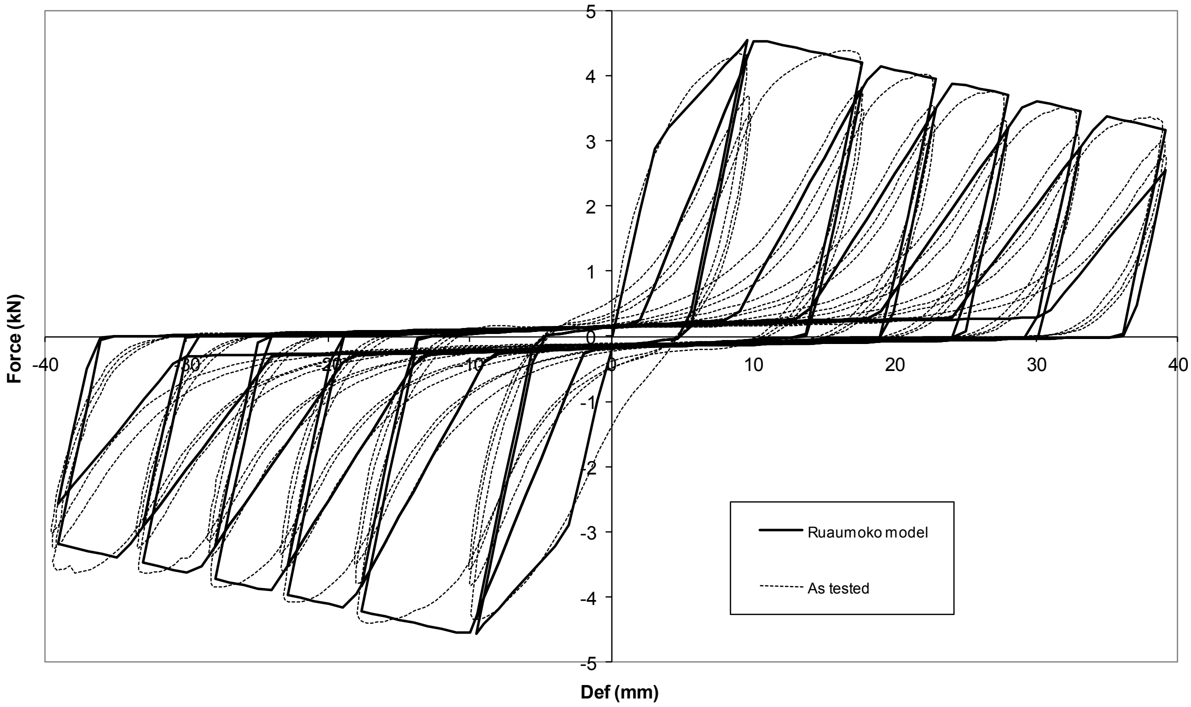

Figure 5. Comparison of test and matched hysteresis loops for standard plasterboard-lined walls

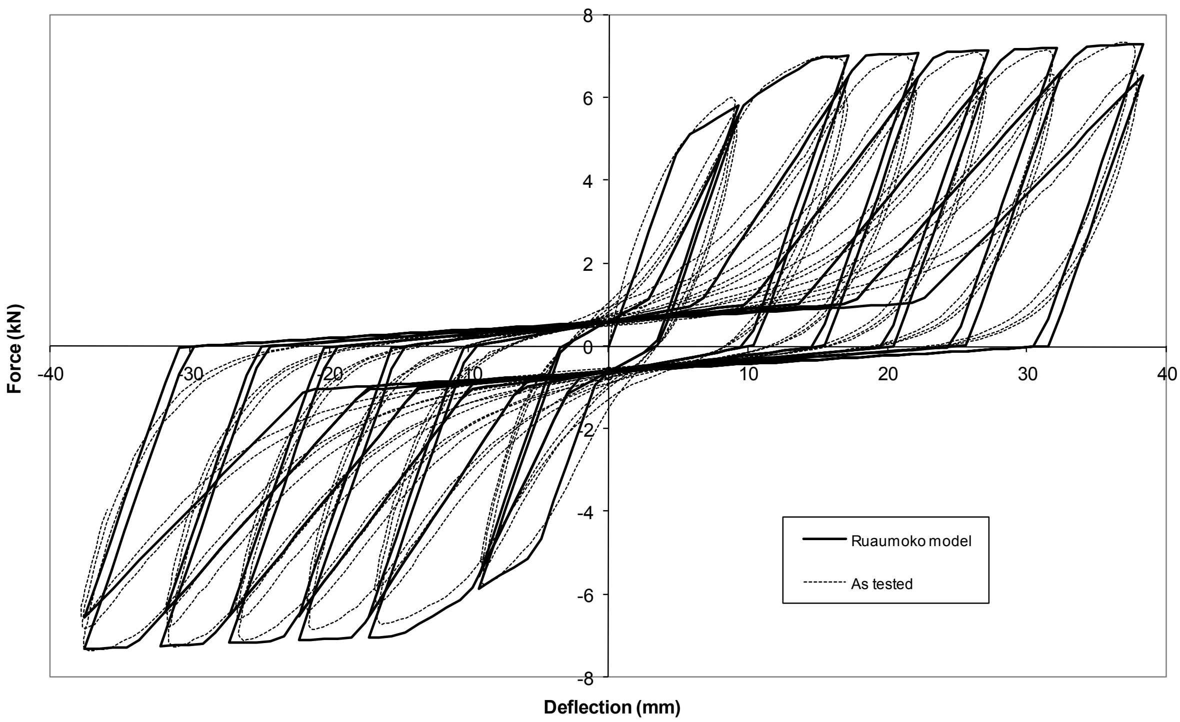

Figure 6. Comparison of test and matched hysteresis loops for MDF-lined walls

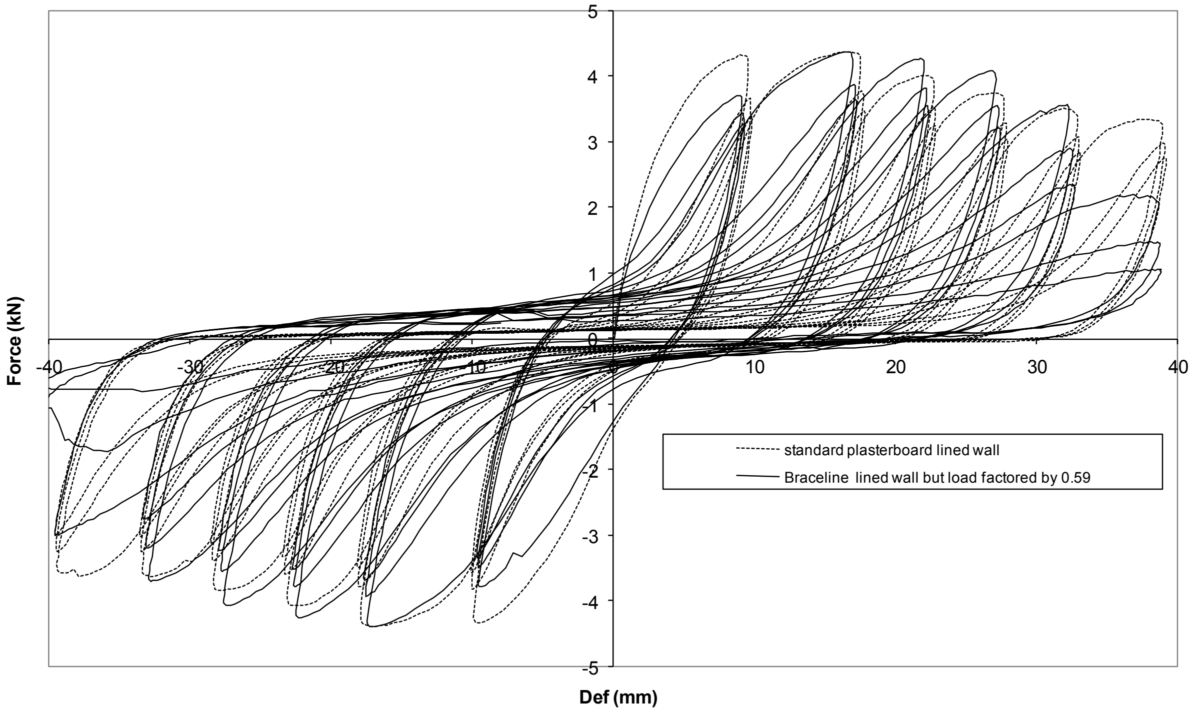

Figure 7. Comparison of hysteresis loops for standard plasterboard and Braceline-lined walls

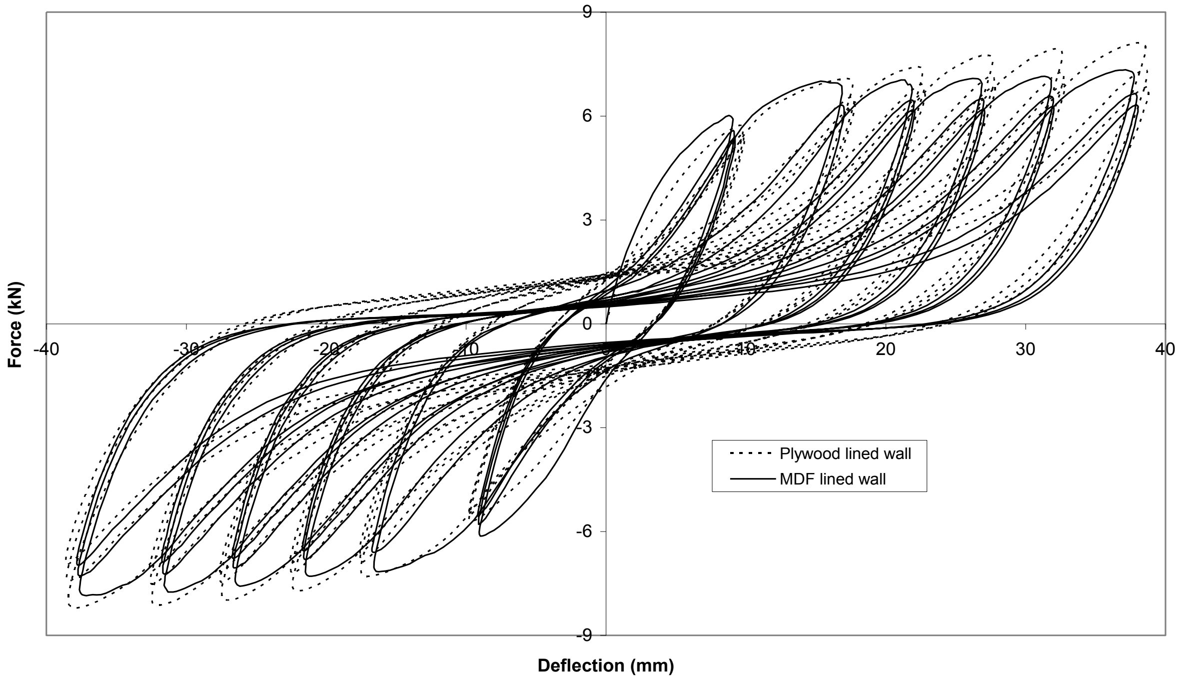

Figure 8. Comparison of test hysteresis loops for MDF and plywood-lined walls

6.3 Building models

NZS 3604 buildings are assumed to have a natural period ≤ 0.4 seconds (Shelton, 2007). NZS 1170.5 (SNZ, 2004) requires the design spectra to be matched for the period band $0.4T_1$ to $1.3T_1$. Thus the GNS records selected K_1 and K_2 to match the design spectra for the period band 0.16 seconds to 0.52 seconds.

The seismic weights used in the SDOF and 2DOF models using GNS earthquakes (which resulted in a computed first-storey level deflection in the range 15-36 mm) gave values of T_1 being between 0.15 to 0.28 seconds for Zone A and 0.22 to 0.4 seconds for Zone C. The natural period was determined as described `below.

The natural period, T, of a single degree of freedom (SDOF) system is given by:

$T = 2\pi\sqrt{\frac{W}{gK}}$ where W is suspended mass on a spring of stiffness K. The acceleration of gravity is given by g. The initial stiffness, K, used in the Stewart model was based on the deflection at approximately 67% peak load as shown in Figure 5 and Figure 6. Values of the first natural period, T, for the two-storey analysis were extracted from the Ruaumoko output data.

6.4 Single-storey computer analysis

6.4.1 GNS Wellington earthquakes (draft 1170.5) set

The factor F1 for Wellington has also been calculated using the GNS earthquakes for walls Standard plasterboard and MDF and for both Class C and D soil conditions. For each soil class three earthquake records were used and for each weight assumed in the analysis only the greatest deflection from the three earthquake runs was used. This is a requirement of NZS 1170.5 (SNZ, 2004). Results are shown in Figure 9 to Figure 12.

Best fit straight lines for this data in the range 15 to 35 mm are given in Eqns (5) to (8) and are also plotted in Figure 9 to Figure 12:

$F1 = 0.145 + 0.0151\Delta$ (Standard plasterboard lining, Class C, GNS EQs) …… (5)

$F1 = 0.0713 + 0.0167\Delta$ (Standard plasterboard lining, Class D, GNS EQs) …… (6)

$F1 = 0.162 + 0.00917\Delta$ (MDF lining, Class C, GNS EQs) …....(7)

$F1 = 0.353 + 0.00214\Delta$ (MDF lining, Class D, GNS EQs) ……(8).

6.4.2 GNS Auckland and Christchurch earthquakes (draft 1170.5) set

The factor F1 for Wellington has also been calculated using the GNS earthquakes for hysteresis loop envelope PLB-Hyst for both Class C and D soil conditions for Auckland and Class D for Christchurch. Results are shown in Figure 13 to Figure 15. Clearly the lowest values of F1 are for the Wellington region (or Zone A). The rest of this report only deals with this region.

6.4.3 Comparison of F1 values obtained from the single-storey analysis

6.4.3.1 Comparison for plasterboard-lined walls

Figure 16 compares F1 factors for single-storey structures lined with standard plasterboard. The most critical case for GNS earthquake records is for soil Class D. The writer recommends that this solution, Eqn (6), be adopted for plasterboard bracing walls in single-storey buildings if NZS 1170.5 is adopted in NZS 3604 without changes to the earthquake loading section.

6.4.3.2 Comparison for MDF clad walls

Figure 17 compares F1 factors for single-storey structures lined with MDF. The Class = C and Class = D GNS earthquake record lines cross. The writer recommends that the average of these two be adopted for the single-storey solution for sheathings other than plasterboard i.e. the average of Eqns (7) and (8).

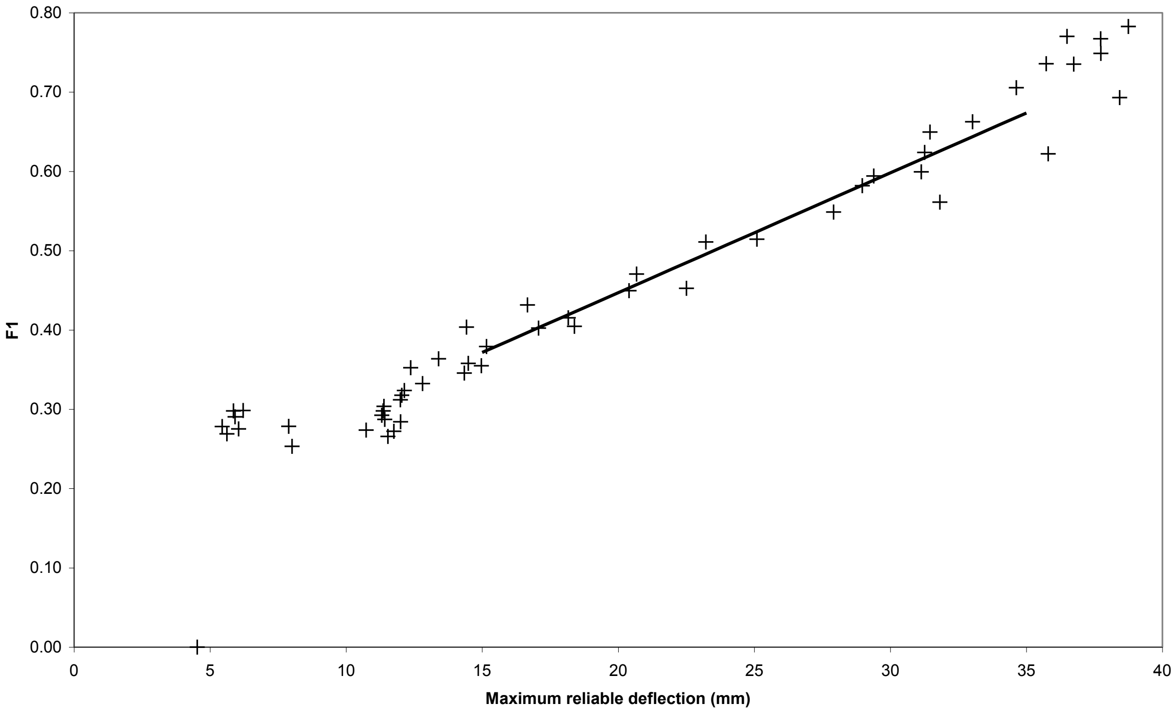

Figure 9. F1 factors for single-storey structures on Wellington Class C soils using the GNS earthquakes (draft 1170.5) set and hysteresis loops for standard plasterboard-lined walls

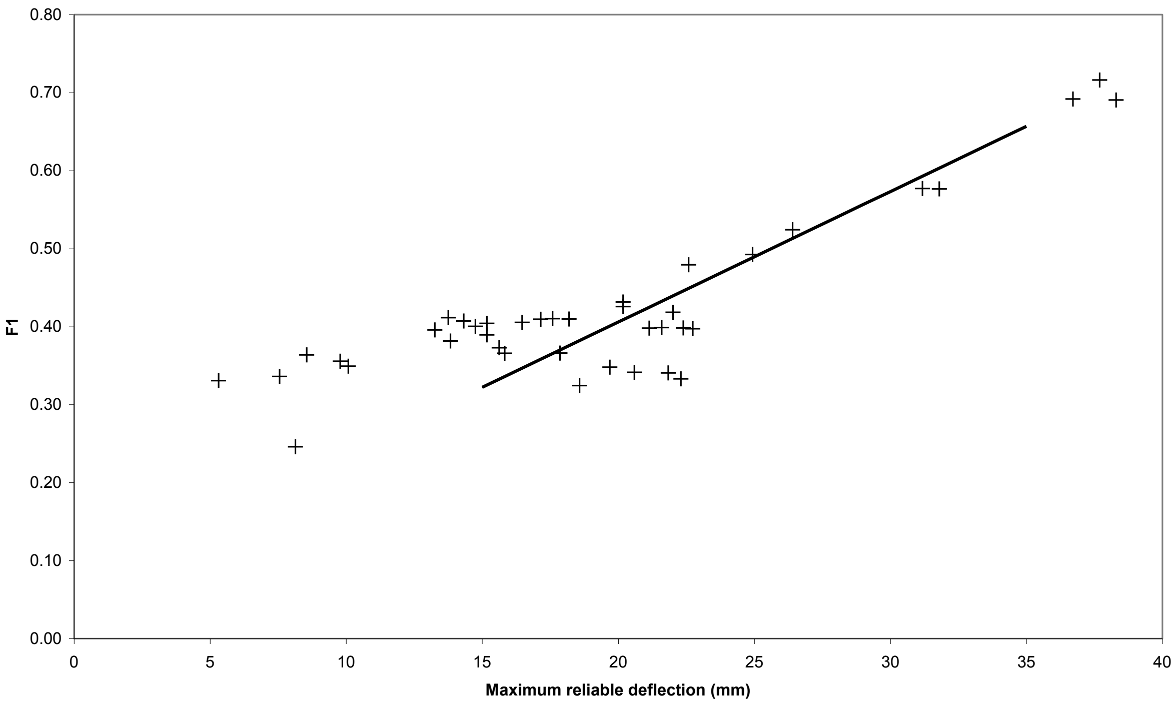

Figure 10. F1 factors for single-storey structures on Wellington Class D soils using the GNS earthquakes (draft 1170.5) set and hysteresis loops for standard plasterboard-lined walls

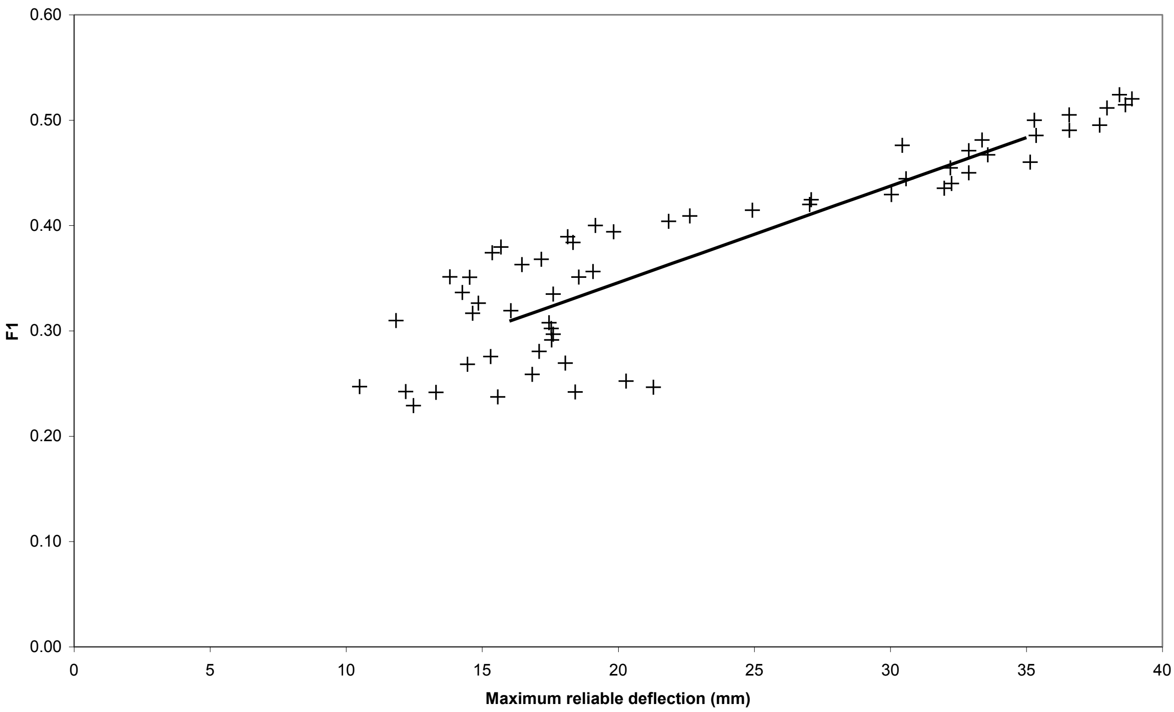

Figure 11. F1 factors for single-storey structures on Wellington Class C soils using the GNS earthquakes (draft 1170.5) set and hysteresis loops for standard plasterboard-lined walls

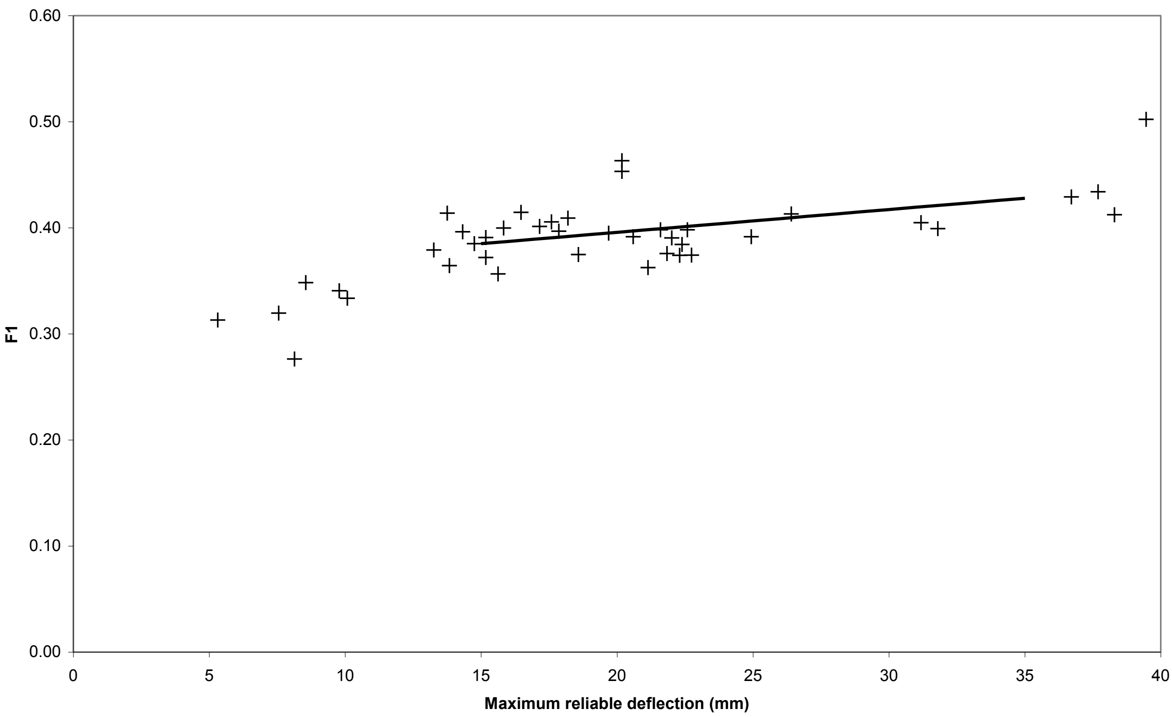

Figure 12. F1 factors for single-storey structures on Wellington Class D soils using the GNS earthquakes (draft 1170.5) set and hysteresis loops for MDF-lined walls

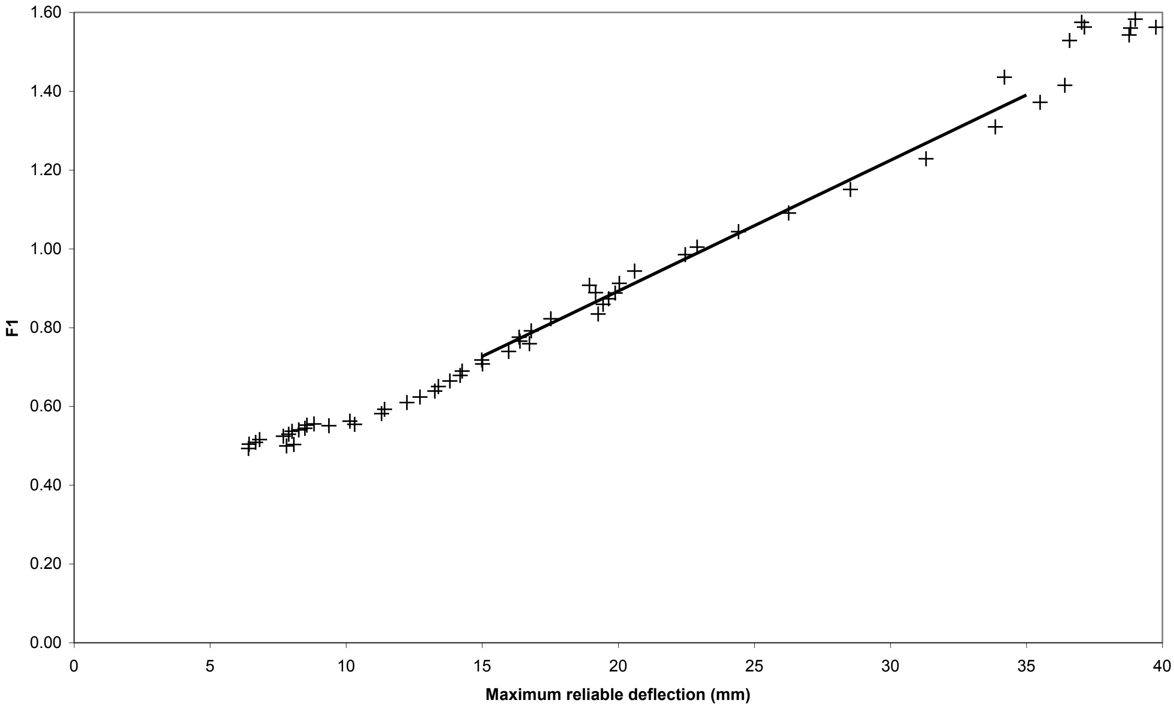

Figure 13. F1 factors for single-storey structures on Auckland Class C soils using GNS earthquakes (draft 1170.5) set and hysteresis loops for standard plasterboard-lined walls

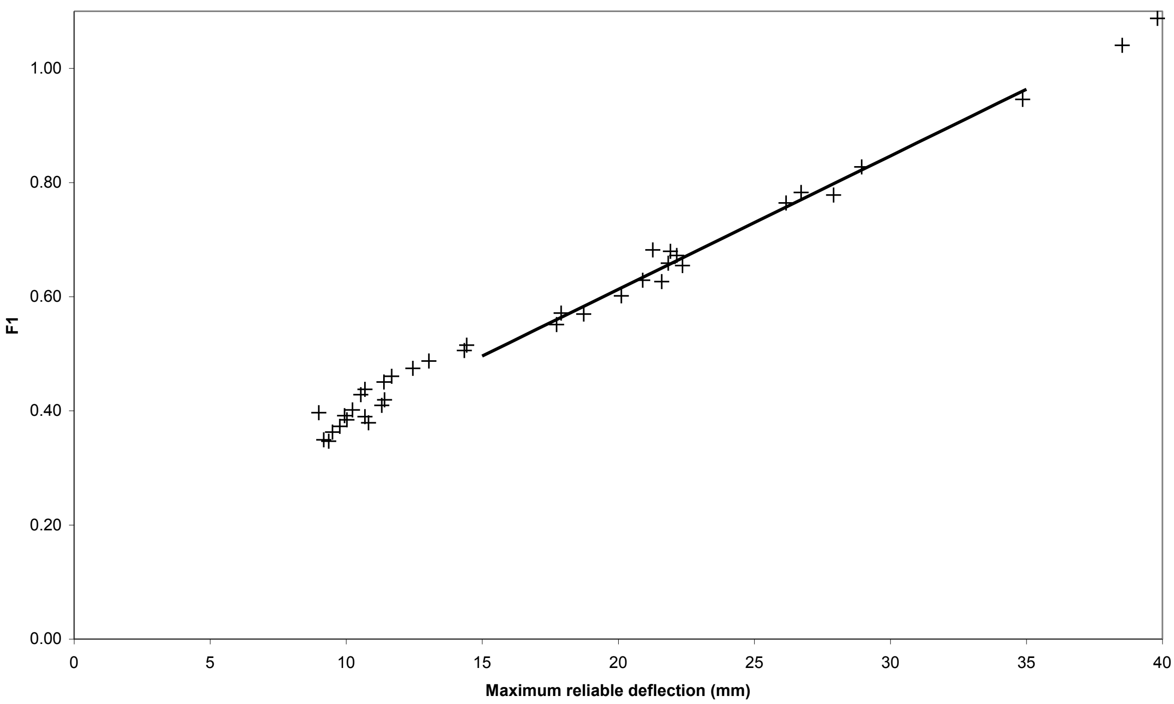

Figure 14. F1 factors for single-storey structures on Auckland Class D soils using the GNS earthquakes (draft 1170.5) set and hysteresis loops for standard plasterboard-lined walls

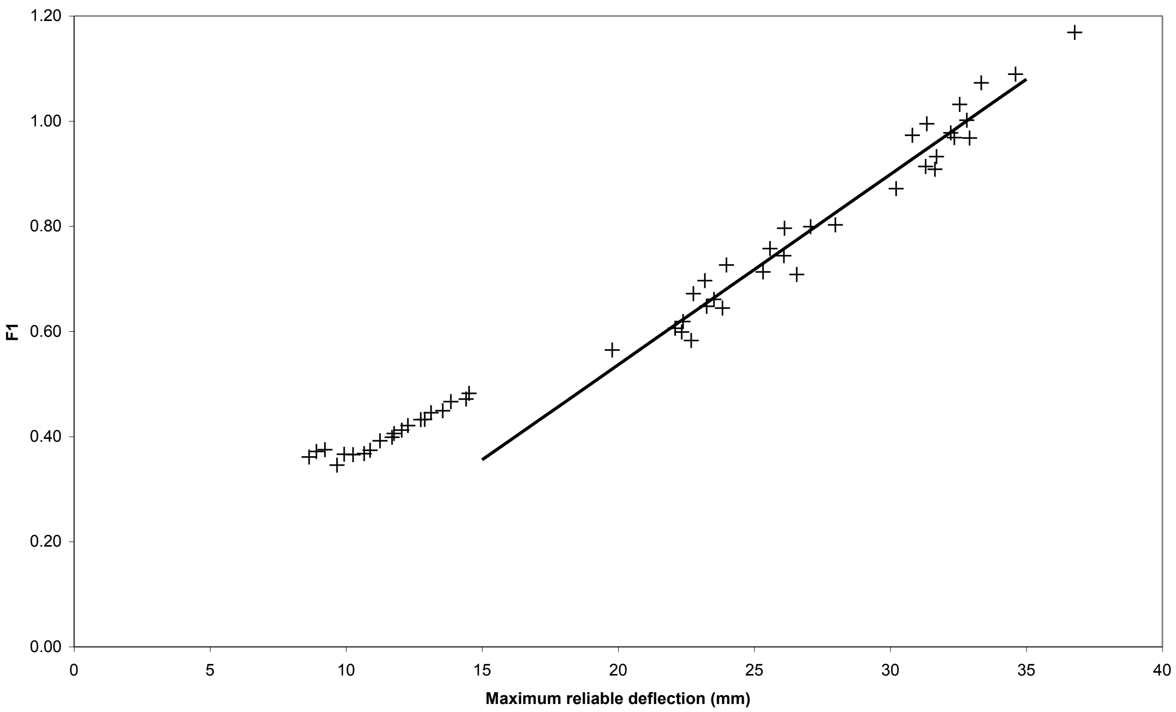

Figure 15. F1 factors for single-storey structures on Christchurch Class D soils using the GNS earthquakes (draft 1170.5) set and hysteresis loops for standard plasterboard-lined walls

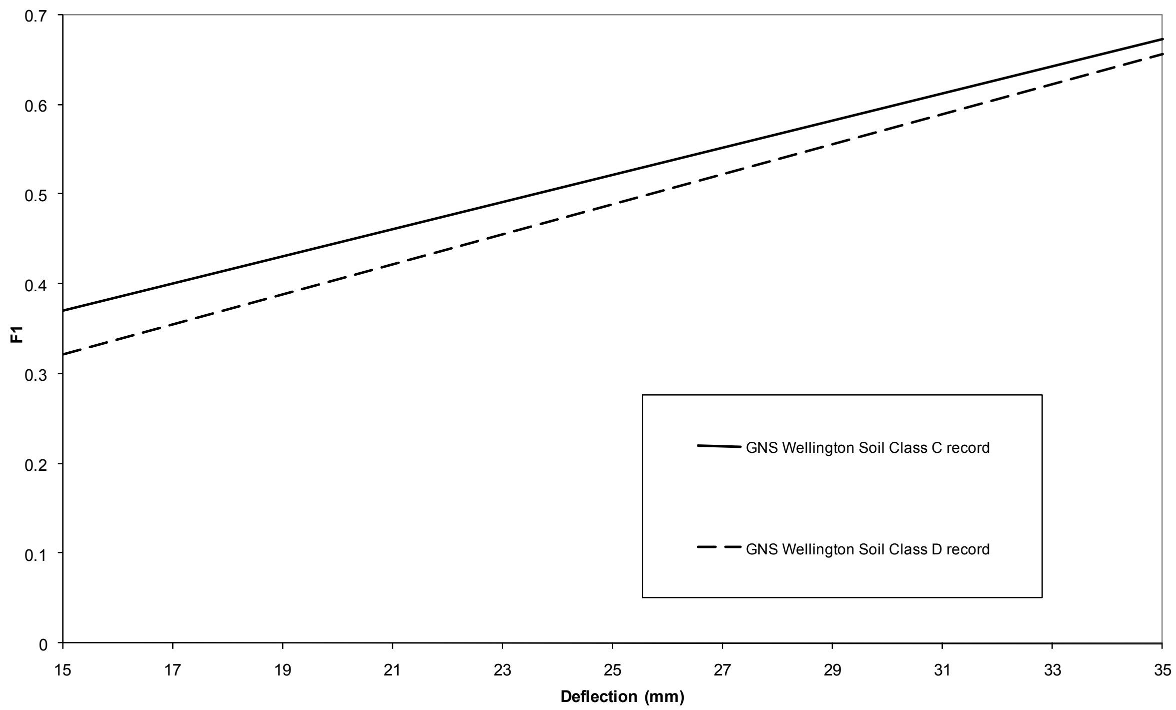

Figure 16. Comparison of F1 factors for Wellington single-storey structures with hysteresis loops for standard plasterboard-lined walls for GNS earthquakes (draft 1170.5) set

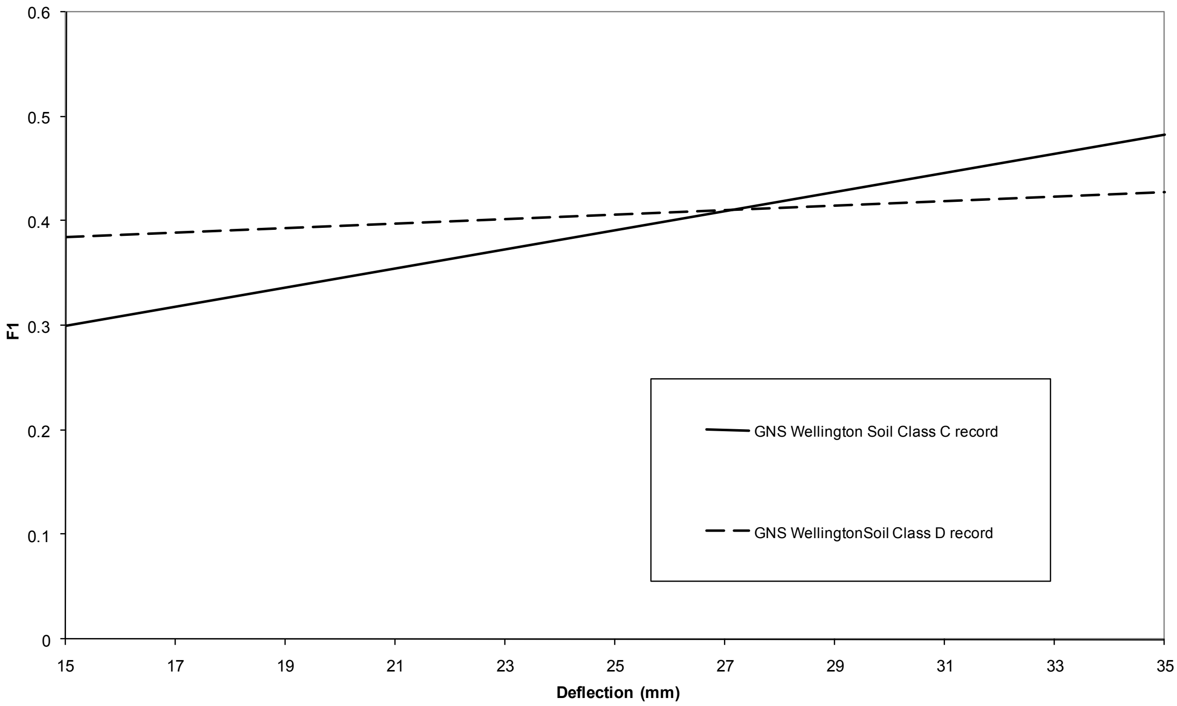

Figure 17. Comparison of F1 factors for Wellington single-storey structures with hysteresis loops for MDF-lined walls for GNS earthquakes (draft 1170.5) set

6.5 Two-storey computer analyses

6.5.1 Selection of masses and stiffness

A similar procedure was used to the single-storey analyses except only Zone A Class C and D were considered. (Zone B and C were found to be less critical in Section 6.4.) Three additional variables were considered:

1. The relative magnitudes of masses at first floor and roof levels.
2. The relative magnitudes of wall strength/stiffness between ground and first floor and first floor to roof.
3. Whether the strength of the first floor to roof walls is greater than required from NZS 3604 demand forces. (Note that if the total strength of walls is provided between first floor to roof, as between ground and first floor, then because the demand forces are greater in the lower-storey the top-storey walls will effectively remain elastic.)

Eight mass/strength/stiffness combinations between storeys were obtained for the analysis as follows:

First the NZS 3604 demand seismic tables were examined to select four representative structural types for a house having the same top and bottom-storey area. The chosen types are given in Table 3.

Table 3. Four cladding combinations and roof pitches considered for a two-storey house

Number	Lower-storey	Top-storey	Roof	Roof pitch
1	Heavy	Medium	Light	0-25º pitch
2	Heavy	Heavy	Heavy	26-45º pitch
3	Light	Light	Heavy	26-45º pitch
4	Heavy	Light	Heavy	26-45º pitch

Using the house weights assumed by NZS 3604 for the cladding combinations given in Table 3 and for the standardised house assumed by NZS 3604, the building weights calculated at each suspended floor level are given in Table 4. The earthquake demand forces published in Table 5:10 of NZS 3604:1999 for Zone A are also given in Table 4.

Table 4. Building weights and earthquake demand forces for the cladding combinations in Table 3

#	Wt roof	Wt/m^2 floor	Demand Level 1-2	Demand Level G-1
	kg/m^2	Kg/m^2	BU/m^2	BU/m^2
1	84	197	7.1	15.2
2	197	263	13.3	22.1
3	152	122	9.9	13.8
4	152	173	10.8	17.3

The ratio of floor weights to roof weights and Level G-1/Level 1-2 wall shear strength, obtained by taking ratios of values in columns in Table 4, are given in Table 5.

Initially it is assumed that the bracing strength actually provided equals the demand forces. Thus the heading for Column 4 of Table 4 becomes 'wall bracing strengths provided' in Table 5.

Table 5. Ratios obtained from Table 4

#	Weight ratio	Actual wall bracing strengths
	Floor/roof	Ratio Level G-1/ Level 1-2
1	2.345	2.141
2	1.335	1.662
3	0.803	1.394
4	1.138	1.602

However, for a nominally braced house, the strength of the walls in the top-storey are often greater than required. Assuming that the actual strength is up to 50% greater than required, but limited to being not greater than the strength in the ground to first floor level, four more combinations are obtained. This provided the eight solutions given in Table 6.

Table 6. Eight combinations of weights/strengths used in the two-storey house simulations

#	Weight ratio	Actual wall bracing strengths
	Floor/roof	Ratio Level G-1/ Level 1-2
1	2.345	2.141
2	1.335	1.662
3	0.803	1.394
4	1.138	1.602
5	2.345	1.427
6	1.335	1.108
7	0.803	1.000
8	1.138	1.068

6.5.2 F1 factors using GNS Wellington earthquakes (draft 1170.5) set and hysteresis loops for standard plasterboard-lined walls

For all eight combinations given in Table 6, the factor F1 has been calculated for standard plasterboard for both Class C and D soil conditions. For each soil class three earthquake records are used and for each weight assumed in the analysis only the greatest deflection from the three earthquake runs was used. This is a requirement of NZS 1170.5 (SNZ, 2004). Results are shown in Figure 18 and Figure 19 for Class C and D respectively.

Best fit straight lines for this data in the range 15 to 36 mm are given in Eqns (9) and (10) and are also plotted in Figure 18 and Figure 19:

F1 = 0.245 + 0.0111Δ (Standard plasterboard best fit, Class C, GNS EQs) ………(9)
F1 = 0.018 + 0.0185Δ (Standard plasterboard best fit, Class D, GNS EQs) ……….(10).

6.5.3 F1 factors using GNS Wellington earthquakes (draft 1170.5) set and hysteresis loops for MDF-lined walls

F1 has been calculated as per Section 6.5.2 but using the hysteresis loop envelope for MDF-lined walls. Results are shown in Figure 20 and Figure 21 for Class C and D respectively.

Best fit straight lines for this data in the range 15 to 35 mm are given in Eqns (11) and (12) and are also plotted in Figure 20 and Figure 21. A lower 25 percentile line is also shown but not used:

F1 = 0.211 + 0.0095Δ (MDF best fit, Class C, GNS EQs) ………. (11)
F1 = 0.221 + 0.0066Δ (MDF best fit, Class D, GNS EQs) ……….. (12).

6.5.4 Recommended F1 factors based on GNS earthquakes (draft 1170.5) set

6.5.4.1 Recommended F1 factors for plasterboard construction

Figure 22 presents the best fit F1 factors for standard plasterboard-lined walls for Class C and D soils for two-storey structures and the recommended F1 factors for single-storey structures. As the single-storey values are not much higher than the worst of the two-storey cases, the writer proposes that the single-storey recommendation be adopted for both single and two-storey.

6.5.4.2 Recommended F1 factors for construction for sheathings other than plasterboard

Similar to the above, but for MDF-lined walls, Figure 23 plots the best fit F1 factors for Class C and D soils for two-storey structures and the recommended F1 factors for single-storey structures. Again, as the single-storey values are not much higher than the worst of the two-storey cases, the writer proposes that the single-storey recommendation be adopted for both single and two-storey.

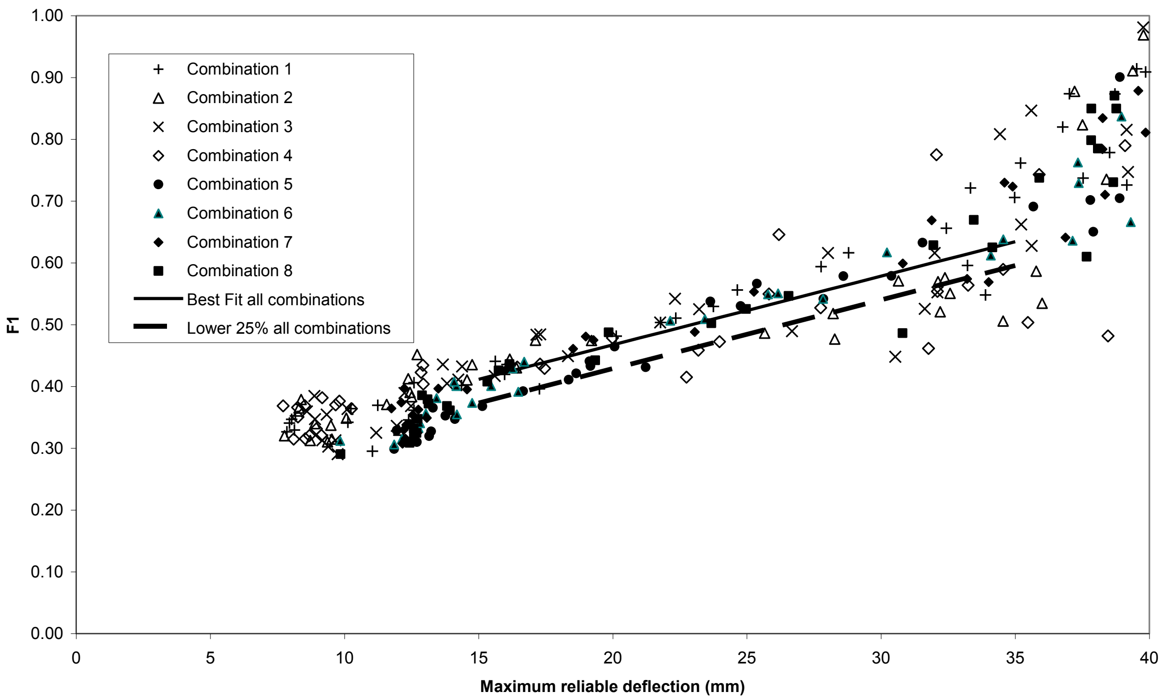

Figure 18. Comparison of F1 factors for bottom-storey walls of two-storey structures using hysteresis loops for standard plasterboard-lined walls and GNS Wellington (draft 1170.5) set Class C earthquakes

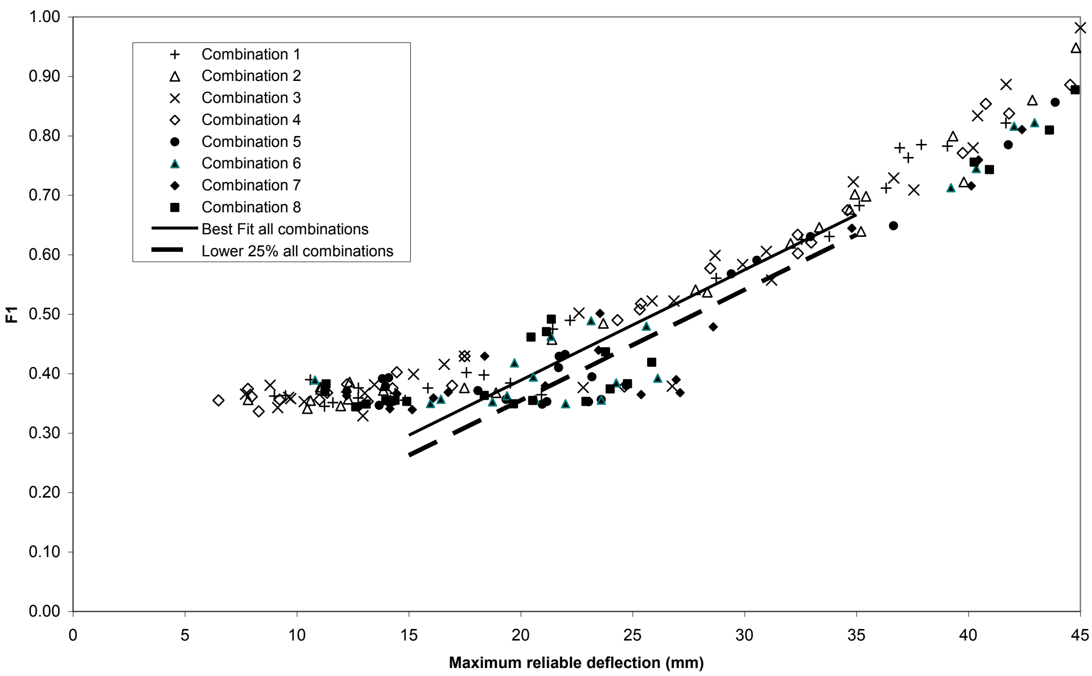

Figure 19. Comparison of F1 factors for bottom-storey walls of two-storey structures using hysteresis loops for standard plasterboard-lined walls and GNS Wellington (draft 1170.5) set Class D earthquakes

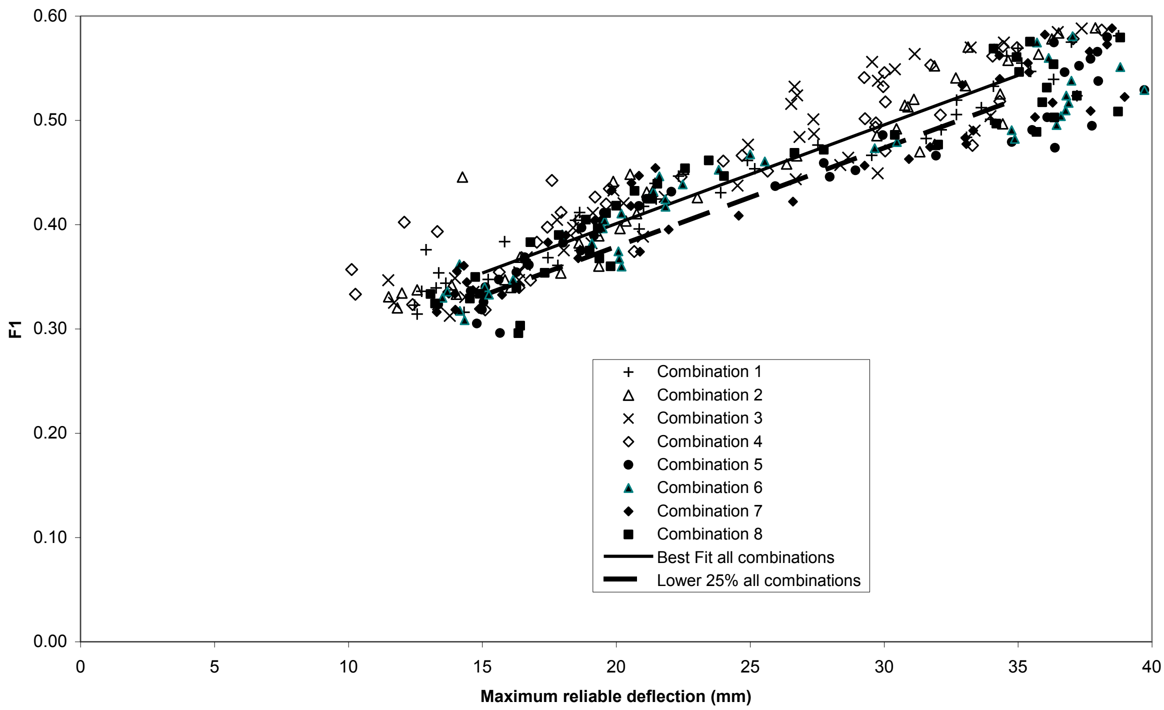

Figure 20. Comparison of F1 factors for bottom-storey walls of two-storey structures using hysteresis loops for MDF-lined walls and GNS Wellington (draft 1170.5) set Class C earthquakes

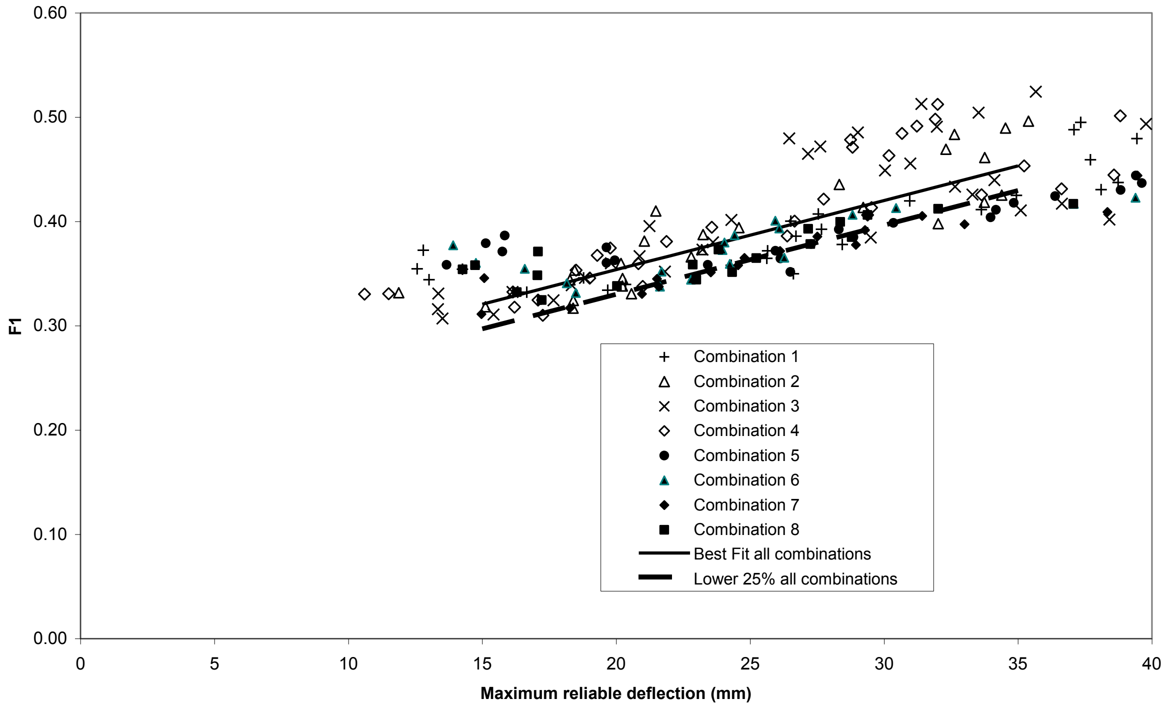

Figure 21. Comparison of F1 factors for bottom-storey walls of two-storey structures using hysteresis loops for MDF-lined walls and GNS Wellington (draft 1170.5) set Class D earthquakes

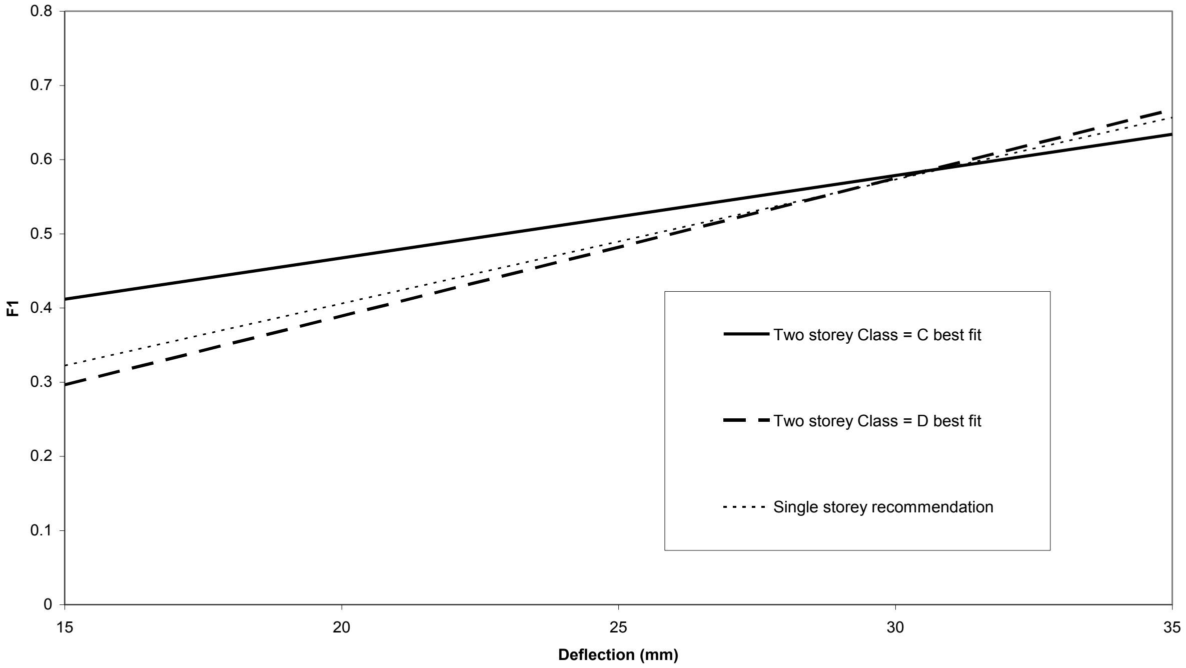

Figure 22. Comparison of F1 factors for bottom-storey walls of two-storey structures using hysteresis loops for standard plasterboard-lined walls

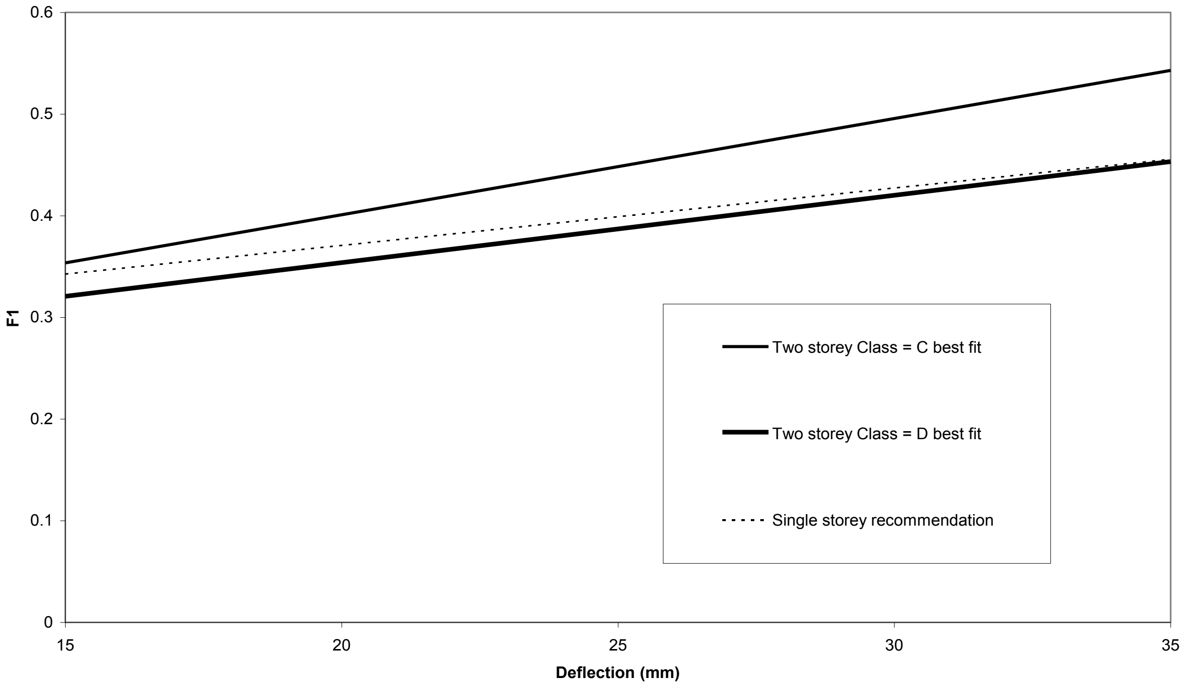

Figure 23. Comparison of F1 factors for bottom-storey walls of two-storey structures using hysteresis loops for MDF-lined walls

6.5.5 F1 factors using GNS earthquakes (1170.5) set

Equation 5.5(1) of NZS 1170.5 (SNZ, 2004) revised DZ 1170.5 (SNZ, 2002) by requiring the target spectrum to be as per DZ 1170.5 (SNZ, 2002) but factored by $(1+S_p)/2$. Clause 4.4 of NZS 1170.5 specifies S_p to be 0.7 for buildings with a ductility greater than 2.0 (such as houses). This section (Section 6.5.5) calculates F1 factors based on earthquakes factored by $(1+S_p)/2$ (i.e. the GNS earthquakes (1170.5) set) using the most critical runs identified in Section 6.

6.5.6 F1 factors using GNS Wellington earthquakes (1170.5) set for standard plasterboard-lined walls

The F1 factor for Wellington has been calculated using the GNS earthquakes for single-storey buildings lined with standard plasterboard for both Class C and D soil conditions. For each soil class three earthquake records were used and for each weight assumed in the analysis only the greatest deflection from the three earthquake runs was used. This is a requirement of NZS 1170.5 (SNZ, 2004). Results are shown in Figure 24 and Figure 25.

Best fit straight lines for this data in the range 15 to 36 mm are given in Eqns (13) to (14) and are also plotted in Figure 24 and Figure 25:

$F1 = 0.188 + 0.0177\Delta$ ……... (13)
(Standard plasterboard lining, Wellington Class C, GNS earthquakes (1170.5) set)

$F1 = 0.081 + 0.0206\Delta$ .... (14)
(Standard plasterboard lining, Wellington Class D, GNS earthquakes (1170.5) set).

The two equations are compared in Figure 28. Eqn (14) is the more critical of the two and is the recommended equation for bracing walls lined with standard plasterboard.

6.5.7 F1 factors using GNS Wellington earthquakes (1170.5) set for MDF-lined walls

The F1 factor for Wellington has also been calculated using the GNS earthquakes for single-storey buildings lined with MDF for both Class C and D soil conditions as per Section 6.5.6.

Best fit straight lines for this data in the range 15 to 36 mm are given in Eqns (15) to (16) and are also plotted in Figure 24 and Figure 25:

$F1 = 0.275 + 0.0098\Delta$ ……... (15)
(MDF lining, Wellington Class C, GNS earthquakes (1170.5) set)

$F1 = 0.339 + 0.0046\Delta$ .... (16)
(MDF lining, Wellington Class D, GNS earthquakes (1170.5) set).

Figure 29 compares F1 factors for single-storey structures lined with MDF. The writer recommends that the average of these two be adopted for the single-storey solution for sheathings other than plasterboard. This is the average of Eqns (15) and (16).

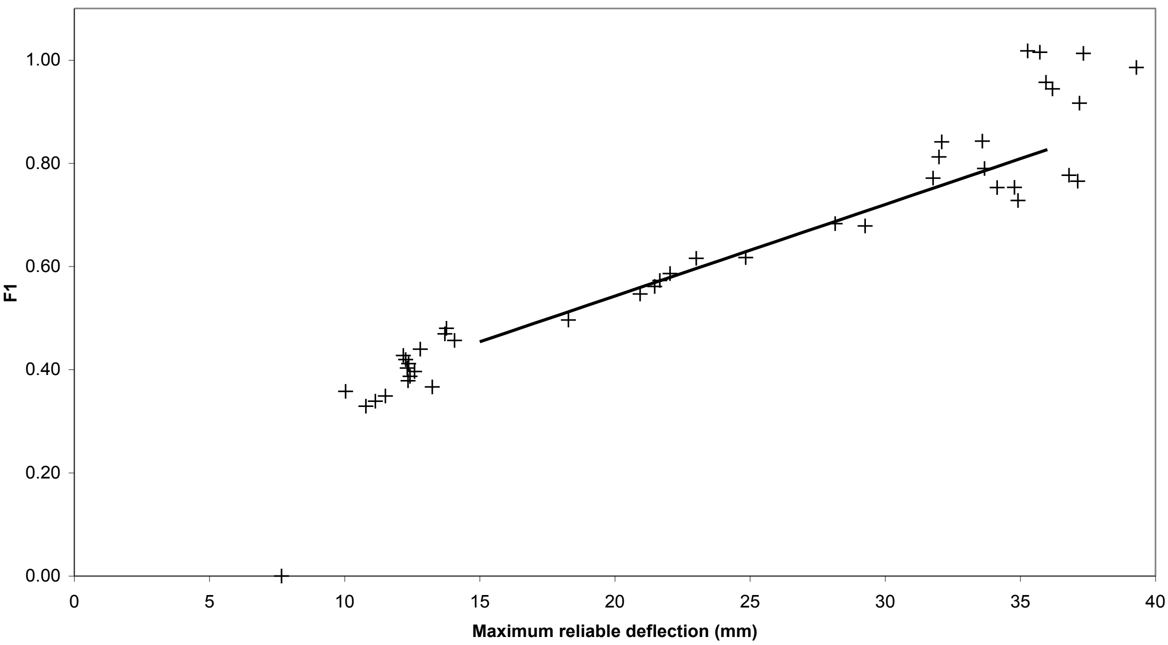

Figure 24. F1 factors for single-storey structures on Wellington Class C soils using the GNS earthquakes (1170.5) set and hysteresis loops for standard plasterboard-lined walls

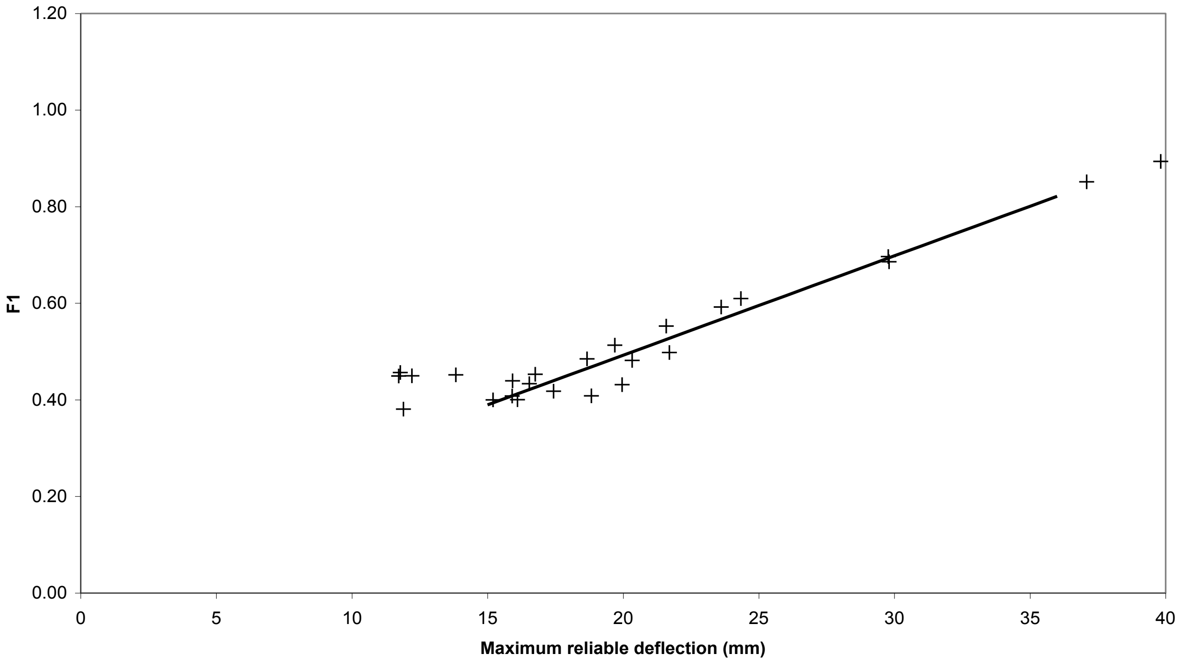

Figure 25. F1 factors for single-storey structures on Wellington Class D soils using the GNS earthquakes (1170.5) set and hysteresis loops for standard plasterboard-lined walls

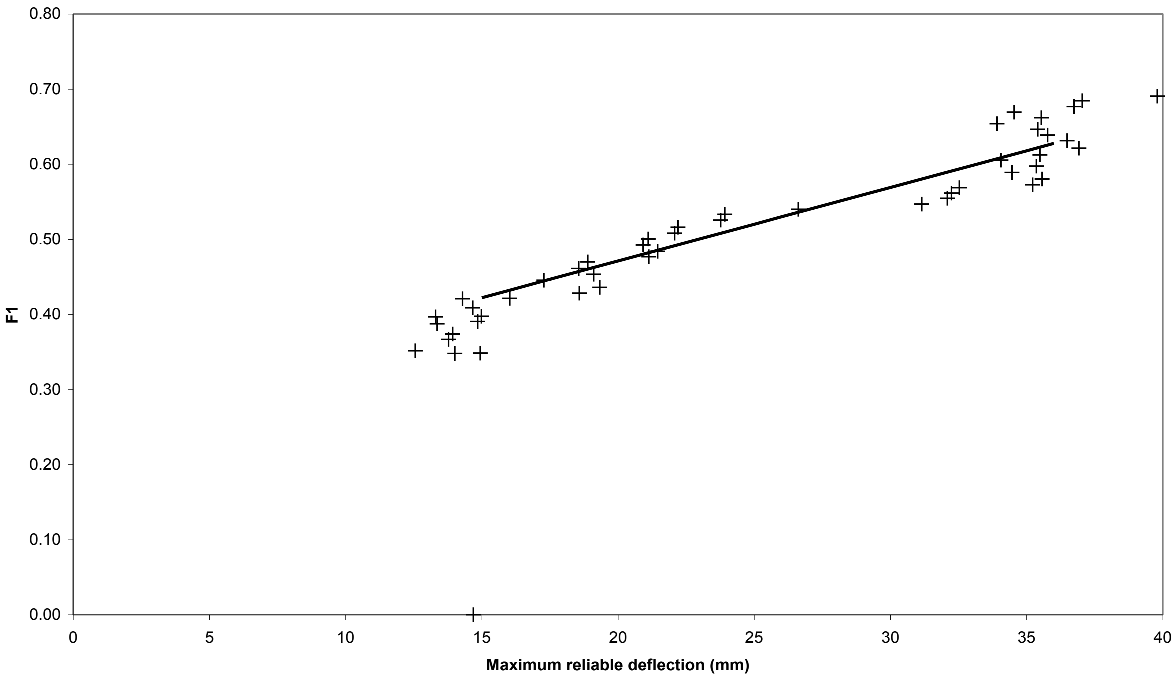

Figure 26. F1 factors for single-storey structures on Wellington Class C soils using the GNS earthquakes (1170.5) set and hysteresis loops for MDF-lined walls

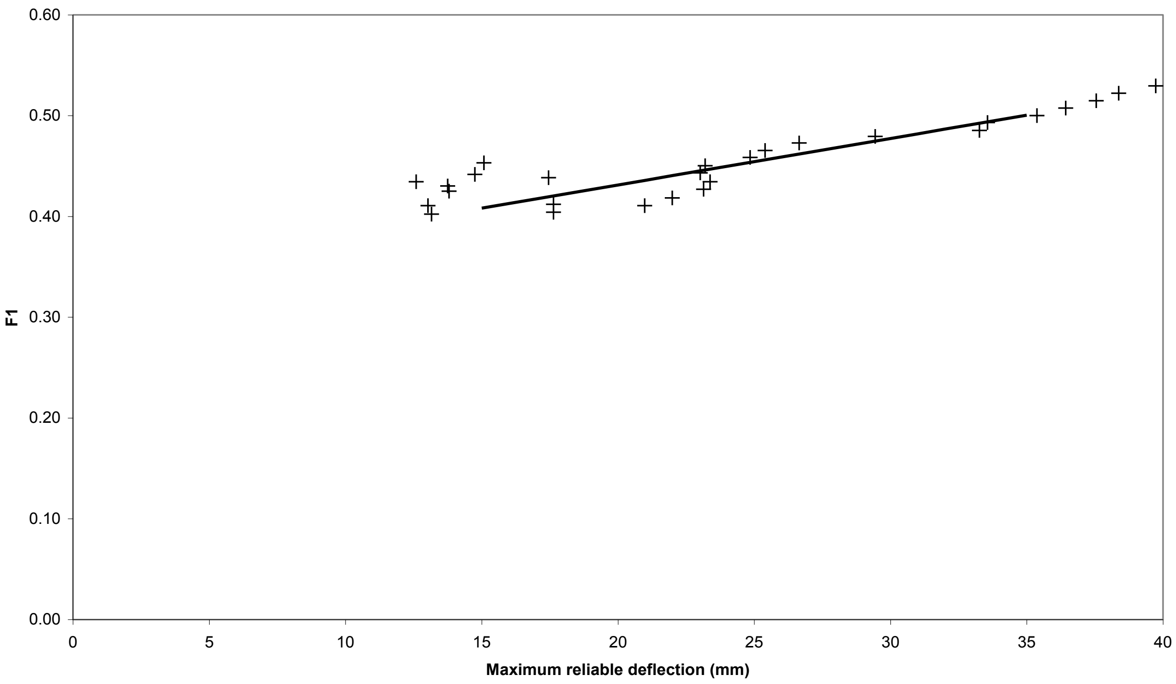

Figure 27. F1 factors for single-storey structures on Wellington Class D soils using the GNS earthquakes (1170.5) set and hysteresis loops for MDF-lined walls

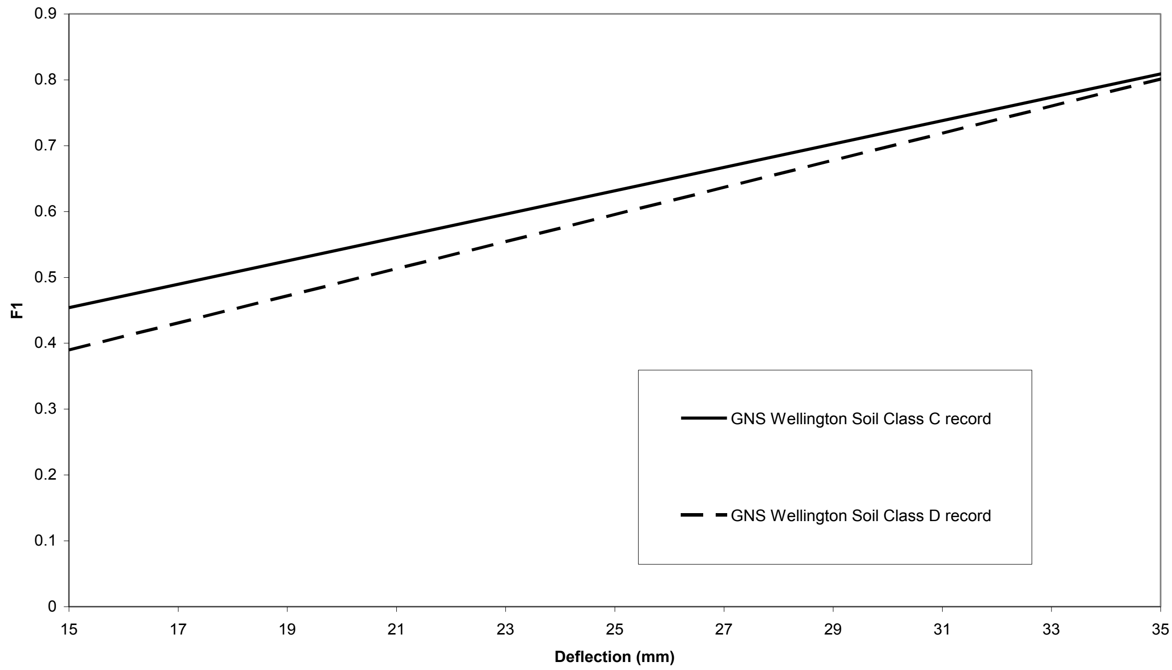

Figure 28. Comparison of F1 factors for Wellington single-storey structures with hysteresis loops for standard plasterboard-lined walls for GNS earthquakes (1170.5) set

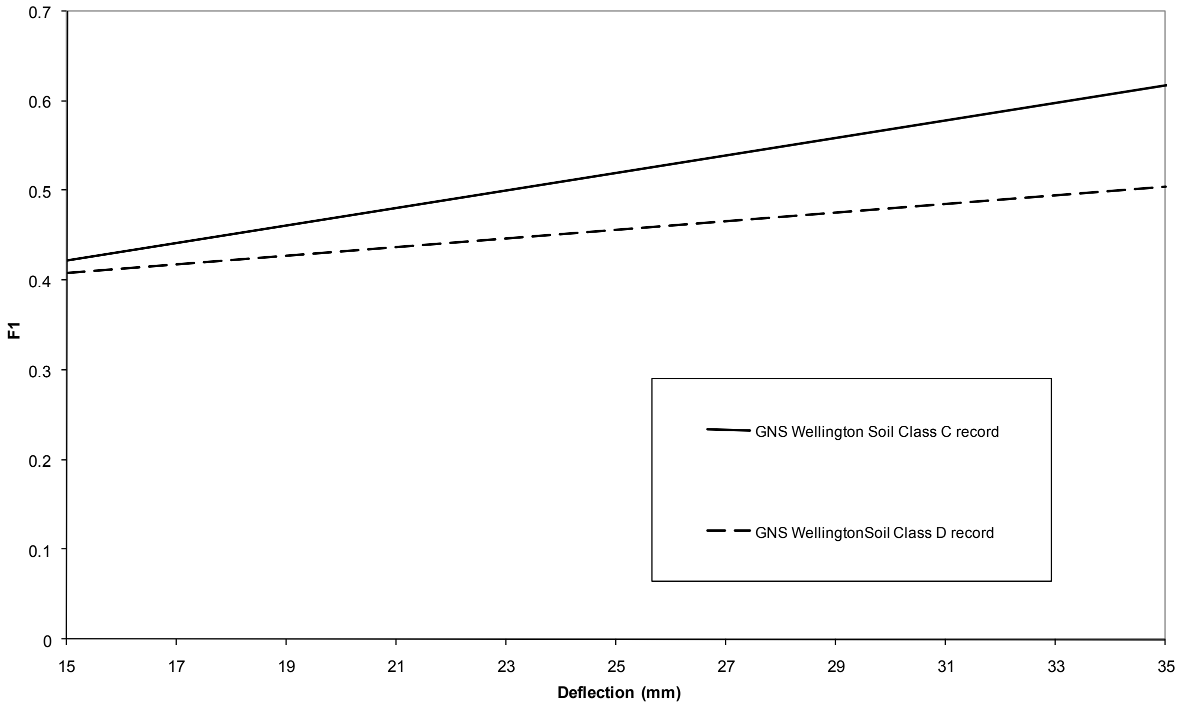

Figure 29. Comparison of F1 factors for Wellington single-storey structures with hysteresis loops for MDF-lined walls for GNS earthquakes (1170.5) set

6.6 Final summary of F1 factors

F1 factors to be used in EM3-V3 are plotted in Figure 30 and listed in Table 7. Linear interpolation may be used but not extrapolations as wall loads used must correspond to deflections in the range 15-36 mm. Note: NZS 3604 is currently based on NZS 4203. If it adopts AS 1170 and adjusts the earthquake loadings assumptions given in Section 5.2, then the F1 factors given below will need modification.

Table 7. F1 factors for EM3-V3

Wall	**Wall sheathing**	
Deflection	**Plasterboard**	**Other**
Δ (mm)	**Walls**	**Walls**
15	0.390	0.415
16	0.410	0.422
17	0.431	0.430
18	0.451	0.437
19	0.472	0.444
20	0.492	0.451
21	0.513	0.458
22	0.534	0.466
23	0.554	0.473
24	0.575	0.480
25	0.595	0.487
26	0.616	0.494
27	0.636	0.502
28	0.657	0.509
29	0.678	0.516
30	0.698	0.523
31	0.719	0.530
32	0.739	0.538
33	0.760	0.545
34	0.780	0.552
35	0.801	0.559
36	0.822	0.566

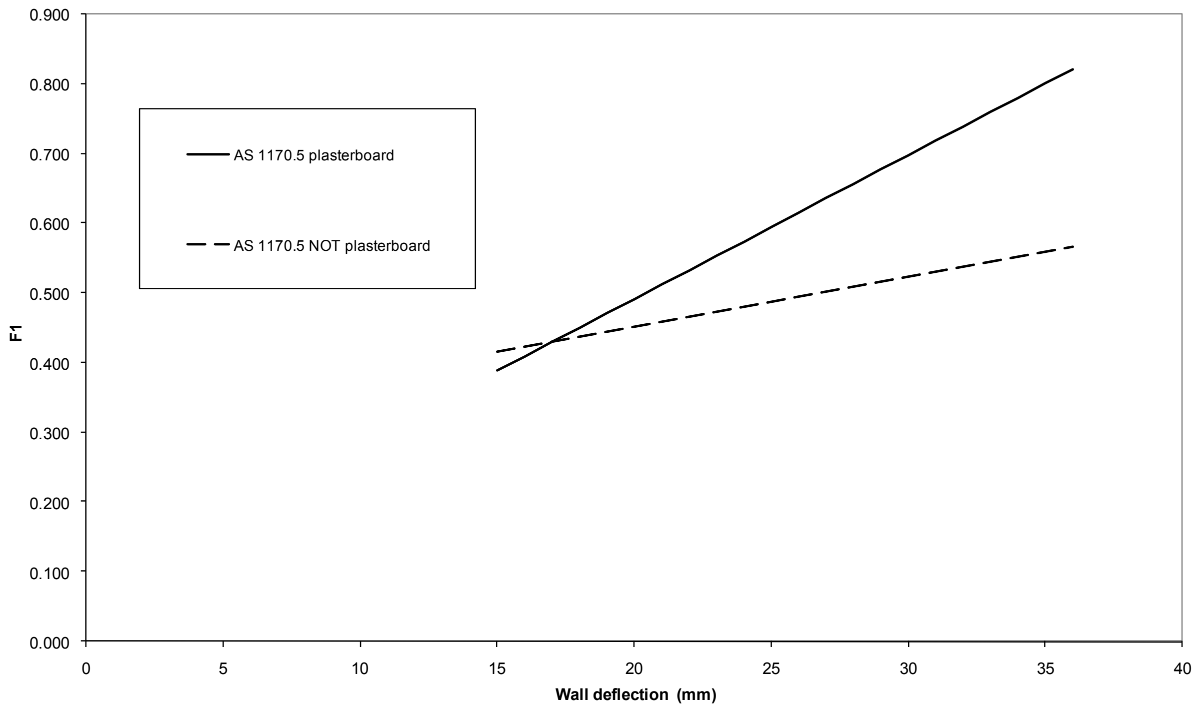

Figure 30. Final F1 factors for EM3-V3

7. COMPARISON OF THE BRACING RATINGS OBTAINED FROM THE CURRENT P21 METHOD AND THE PROPOSED EM3-V3 METHOD

It is important to be able to assess the changes in bracing rating if the current P21 test method is replaced with EM3-V3. To achieve this wall bracing ratings were determined using the evaluation methods listed in Table 8 based on actual test results representative of a number of different types of bracing elements and the EM3 method as proposed herein.

Table 8. Evaluations used to compare bracing ratings determined from the P21 and EM3-V3 methods

Method	Description
P21	Current P21 test evaluation method. The test specimen uses a 'three- nail' end restraint.
EM3	Tested under the P21 method but the data is evaluated using EM3-V3.

Results from the 11 walls analysed and the parameters used are given in Table 9. The percentage change in values from P21 to EM3 ratings is given in Table 10. Table 9 provides a brief description of the construction of each wall, states whether the wall lining was used on a single or both faces of the test wall, notes if end straps were used, and lists the parameters R_s, R_u, P and Δ_u extracted from the hysteresis loops. The values of K4 (for the P21 test method) and F1 (from the EM3-V3 methods) are listed. The bracing ratings determined using these numbers for both wind and earthquake and for the SLS and ULS criteria were calculated. Finally the bracing rating is listed, which is the least of the SLS and ULS criteria.

A discussion of the results of each wall is given below. This includes a cross-reference to plots of the P21 hysteresis loops from the test (which is the basis of the P21 and EM3 evaluations). The plots also include the hysteresis loops with the rocking deflection removed which is effectively the full rocking restraint situation. This is because discussion continues on whether full rocking restraint should be used for lower-storey walls.

The peak load R_u is evaluated for the 4th cycle with the P21 method, and the third cycle with the EM3 method, and so will vary with evaluation method even if corresponding points are used. However, as allowed by the EM3 method, to maximise the bracing ratings the EM3 peaks loads are often taken at greater deflections than the P21 method (i.e. past the peak R_u value) as the combined value of F1 x R_u was greater.

In each case the wind bracing ratings were the same irrespective of the evaluation method and are not discussed again.

Wall 1. See Figure 31. Bracing ratings P21/ EM3 = 107/99

This was a 0.6 m long wall lined with plasterboard on one side only and which had end straps. The P21 evaluation method assigned a ductility of 4 to this wall which appears from the shape of the hysteresis loops to be too high. Thus some reduction in earthquake bracing in the EM3 evaluations is expected.

Despite having end straps the wall rocking was mainly due to the wall end studs separating from the bottom plate. This rocking added significantly to the apparent ductility of the wall. Although this action tended to pull the lining from the bottom plate, thus weakening the wall, the observed damage to the wall was fairly uniform around the lining perimeter. Thus it is unlikely that walls tested with an increased rocking restraint would have achieved a significantly increased EM3 bracing rating.

Wall 2. See Figure 32. Bracing ratings P21/ EM3 = 165/153

This was a 0.6 m long wall lined with plasterboard on both sides and which had end straps. It had little damage at test completion and the main deformation mode was rocking. The EM3 ratings are likely to have been enhanced significantly if greater rocking restraints were used in the racking test.

Wall 3. See Figure 33. Bracing ratings P21/ EM3 = 94/70

This was a 1.2 m long wall lined with plasterboard on one side only and which had end straps. It had large damage in the plasterboard at fastener locations at test completion but had little rocking-induced deflection. Consequently the EM3 ratings would show little enhancement if greater end restraints were used.

Wall 4. See Figure 34. Bracing ratings P21/ EM3 = 133/139

This was a 1.2 m long wall lined with plasterboard on both sides and which had end straps. It only had small damage in the plasterboard at fastener locations at test completion as the strength of the wall was limited by the wall rocking strength rather than the shear strength. The EM3 ratings are likely to have been enhanced if greater rocking restraints were used in the racking test, but not to the same extent as Wall 2 as more degradation occurred with Wall 4.

Wall 5. See Figure 35. Bracing ratings P21/ EM3 = 80/64

This was a 1.2 m long wall lined with plasterboard on both sides and which had no end straps. The main deformation mode was rocking. The end studs of this wall separated from the bottom plate and pulled the lining off the bottom plate. The EM3 ratings are likely to have been enhanced significantly if greater rocking restraints were used in the racking test as this would have resisted the studs being separated from the bottom plate.

Wall 6. See Figure 36. Bracing ratings P21/ EM3 = 76/58

This was a 1.8 m long wall lined with plasterboard on one side and which had no end straps.
Comments are similar to that used given for Wall 3.

Wall 7. See Figure 37. Bracing ratings P21/ EM3 = 93/87.

This was a 2.4 m long wall lined with plasterboard on one side and which had no end straps. Comments are similar to that used given for Wall 5.

Wall 8. See Figure 38. Bracing ratings P21/ EM3 = 64/55

This was a 2.4 m long unlined wall lined with a special diagonal brace system. Although this wall showed some damage the load continued to rise with increased deflection and it exhibited little rocking. Consequently the original P21 is suitable for re-evaluation by EM3-V3.

The EM3 earthquake ratings are significantly lower that the P21 ratings, as although the peak loads occurred at high deflections the F1, coefficients were still low as it was a Type 2 (i.e. non-plasterboard) lined wall.

The wind bracing strength of this wall was governed by the serviceability rather than ULS criteria.

Wall 9. See Figure 39. Bracing ratings P21/ EM3 = 151/150

This was a 1.2 m long wall lined with fibre-cement sheets on one side and which had end straps.

This wall showed little-moderate damage in the fibre-cement at fastener locations at test completion. The strength of the wall was likely to have been limited by the shear strength rather than the rocking strength. The EM3 ratings are unlikely to be enhanced if greater rocking restraints are used in the racking test.

Wall 10. See Figure 40. Bracing ratings P21/ EM3 = 124/113

This was a 0.4 m long wall lined with plasterboard on one side and which had end straps. It had no visible damage in the plasterboard at fastener locations at test completion as the strength of the wall was limited by its flexibility (bending and rocking) and the resisted load increased with increased wall deflections. The EM3 ratings were likely to be enhanced if greater rocking restraints are used in the racking test.

Additional end restraint nails were then added at the end of the original P21 test specimen and further load cycles imposed. These hysteresis loops are also shown in Figure 40. The resisted load increased slightly at greater deflections with the rocking deformations removed.

Wall 11. See Figure 41. Bracing ratings P21/ EM3 = 163/157

This was a 1.2 m long wall lined with MDF on one side and which had end straps. The observations were similar to Wall 10. The resisted load increased at greater deflections with the rocking deformations removed.

Table 9. Comparison of bracing ratings determined from the P21 and EM3-V3 methods

No.	Wall Type	Both faces ?	End straps ?	Type	R_s (kN)	R_u (kN)	P_u	Δ_u	F1 or K4	Bracing ratings in BU's/m					
										SLS		ULS		Combined	
										EQ	Wind	EQ	Wind	EQ	Wind
1	0.6 m long PB	No	Yes	P21	2.27	3.22	4.06	32	1.000	159	135	107	135	107	135
				EM3A	2.27	3.35	4.06	32	0.739	198	135	99	135	99	135
2	0.6 m long PB	Yes	Yes	P21	2.93	4.99	4.95	36	1.000	205	174	165	165	165	165
				EM3A	2.93	4.66	4.95	36	0.822	255	174	153	165	153	165
3	1.2 m long PB	No	Yes	P21	5.37	5.62	6.67	16	1.000	182	154	94	111	94	111
				EM3A	5.37	4.28	6.67	36	0.822	149	154	70	111	70	111
4	1.2 m long PB	Yes	Yes	P21	8.40	8.44	10.6	30	1.000	294	249	133	177	133	177
				EM3A	8.40	8.43	10.6	36	0.822	233	249	139	177	139	177
5	1.2 m long PB	Yes	No	P21	4.17	4.77	5.32	16	1.000	121	102	80	89	80	89
				EM3A	4.17	3.93	5.32	36	0.822	116	102	64	89	64	89
6	1.8 m long PB	No	No	P21	7.50	6.83	8.34	17	1.000	161	136	76	93	76	93
				EM3A	7.50	5.75	8.34	33	0.760	139	136	58	93	58	93
7	2.4 m long PB	No	No	P21	10.7	11.1	13.2	16	1.000	173	147	93	110	93	110
				EM3A	10.7	10.6	13.2	36	0.822	149	147	87	110	87	110
8	Diag. brace	N/A	No	P21	3.935	8.02	8.55	36	0.964	71	58	64	71	64	58
				EM3A	3.935	8.03	8.55	36	0.822	55	58	66	71	55	58
9	1.2 m fibre-	No	Yes	P21	6.42	9.07	10.5	29	1.000	225	190	151	175	151	175
				EM3A	6.42	9.15	10.5	36	0.822	178	190	150	175	150	175
10	0.4 m long PB	No	Yes	P21	1.36	2.49	2.65	36	1.000	143	121	124	132	124	121
				EM3A	1.36	2.5	2.65	36	0.822	113	121	123	132	113	121
11	1.2 m long	No	Yes	P21	5.66	9.76	10.6	36	1.000	198	168	163	177	163	168
				EM3A	5.66	9.74	10.6	36	0.822	157	168	160	177	157	168

Table 10. Extraction from Table 9 showing percentage change from P21 to EM3

No.	Wall Type	Both faces ?	End straps ?	Percent change from P21	
				EQ	Wind
1	0.6 m long PB	No	Yes	-8	0
2	0.6 m long PB	Yes	Yes	-7	0
3	1.2 m long PB	No	Yes	-25	0
4	1.2 m long PB	Yes	Yes	4	0
5	1.2 m long PB	Yes	No	-19	0
6	1.8 m long PB	No	No	-23	0
7	2.4 m long PB	No	No	-6	0
8	Diag. brace system	N/A	No	-15	0
9	1.2 m fibre- cement wall	No	Yes	-1	0
10	0.4 m long PB	No	Yes	-9	0
11	1.2 m long MDF wall	No	Yes	-3	0

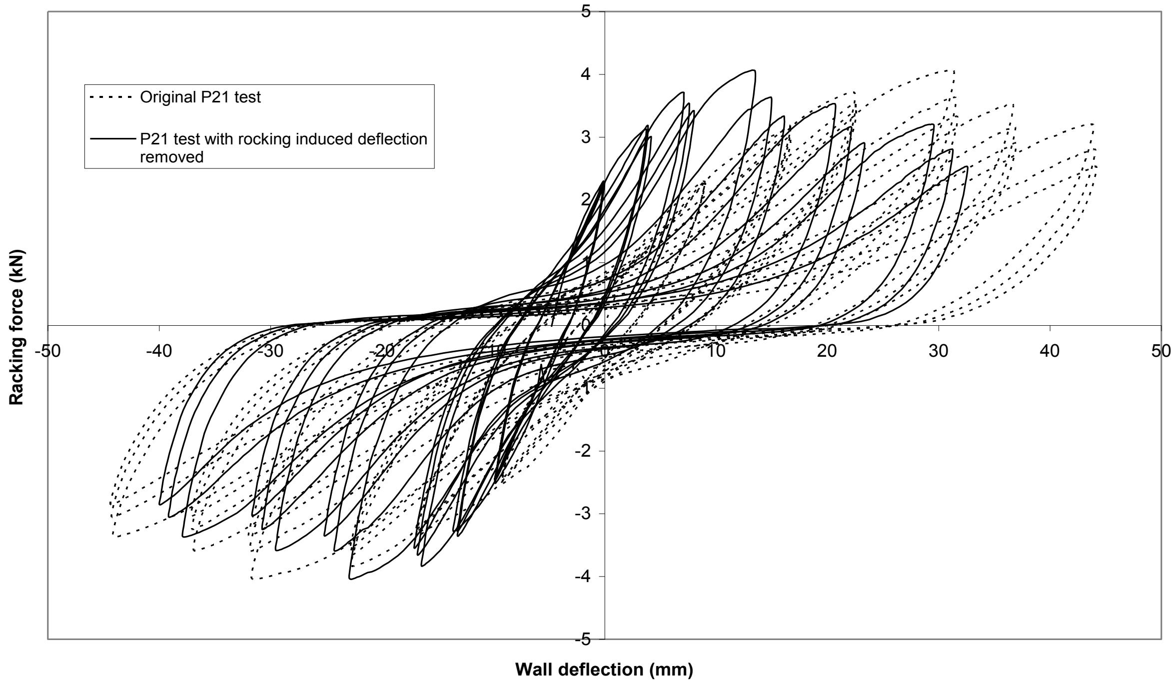

Figure 31. Wall 1 – 0.6 m long wall lined with plasterboard on one face with end straps

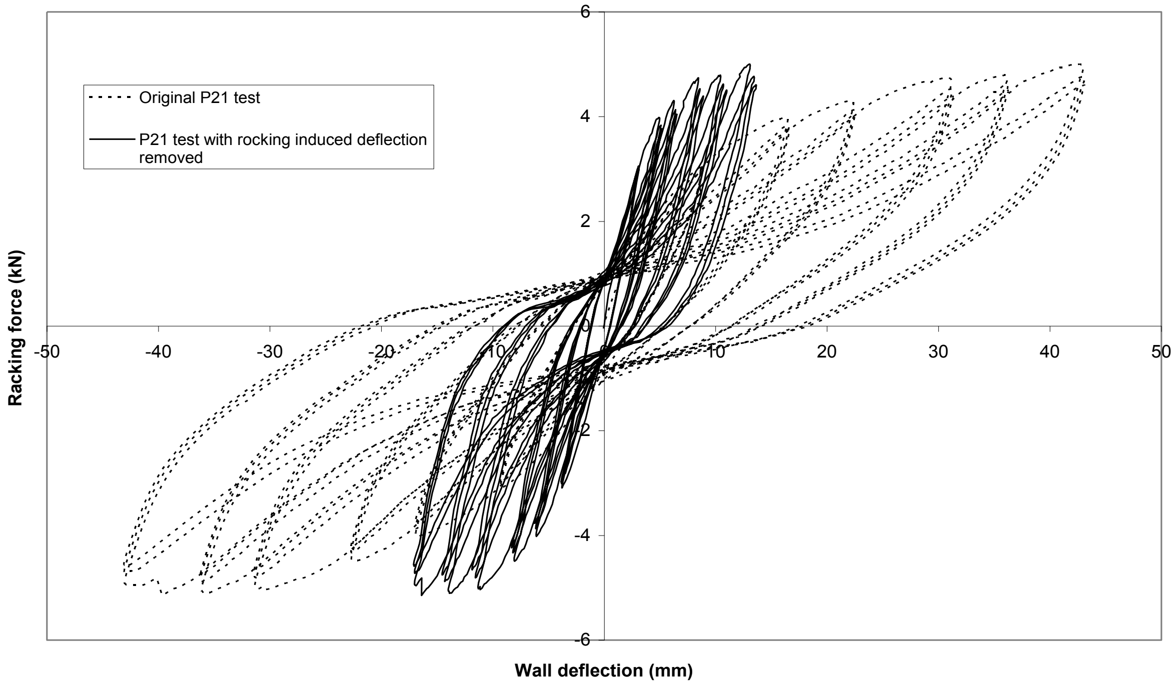

Figure 32. Wall 2 – 0.6 m long wall lined with plasterboard on both faces with end straps

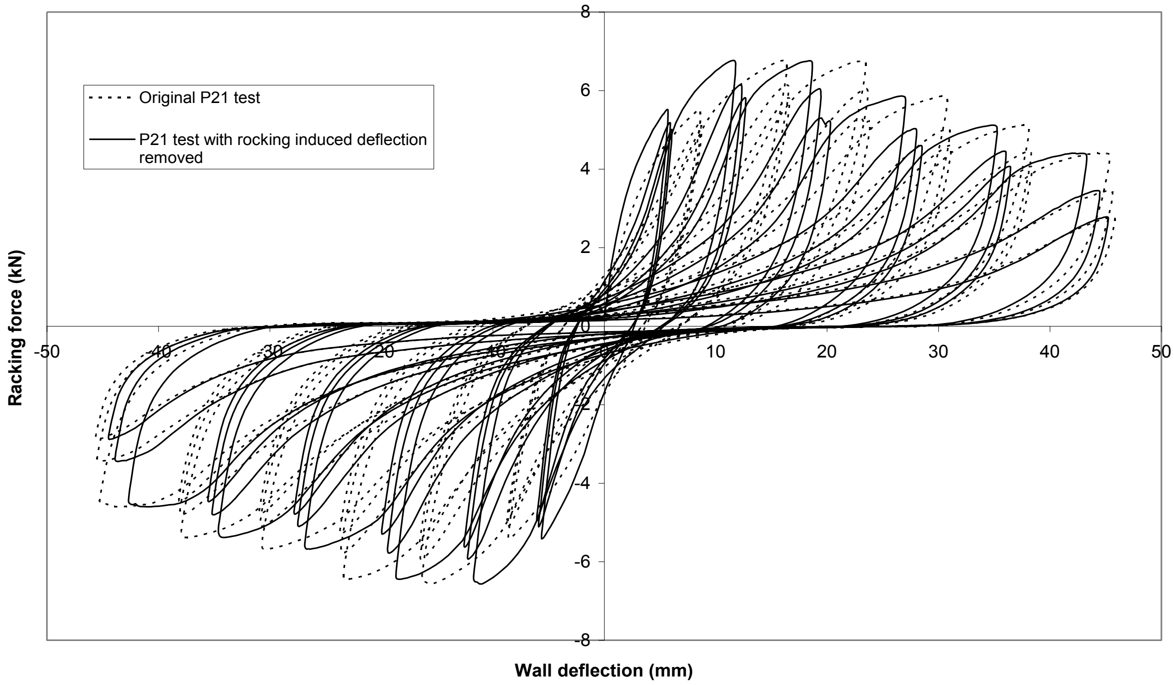

Figure 33. Wall 3 – 1.2 m long wall lined with plasterboard on one face without end straps

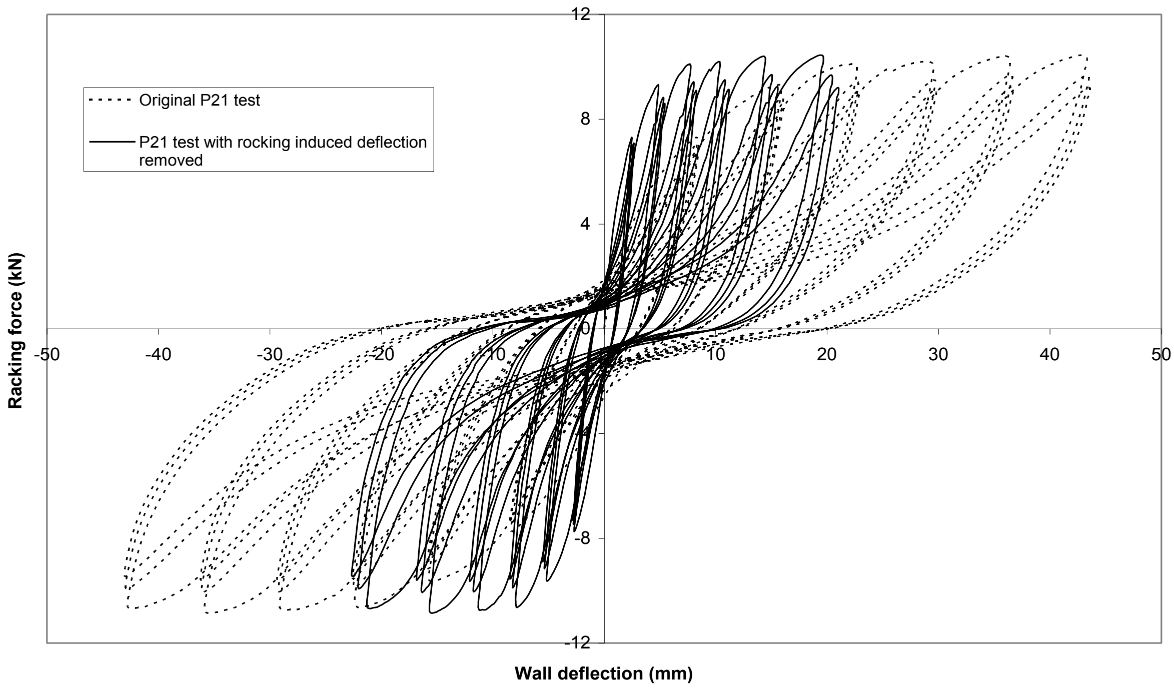

Figure 34. Wall 4 – 1.2 m long wall lined with plasterboard on both faces with end straps

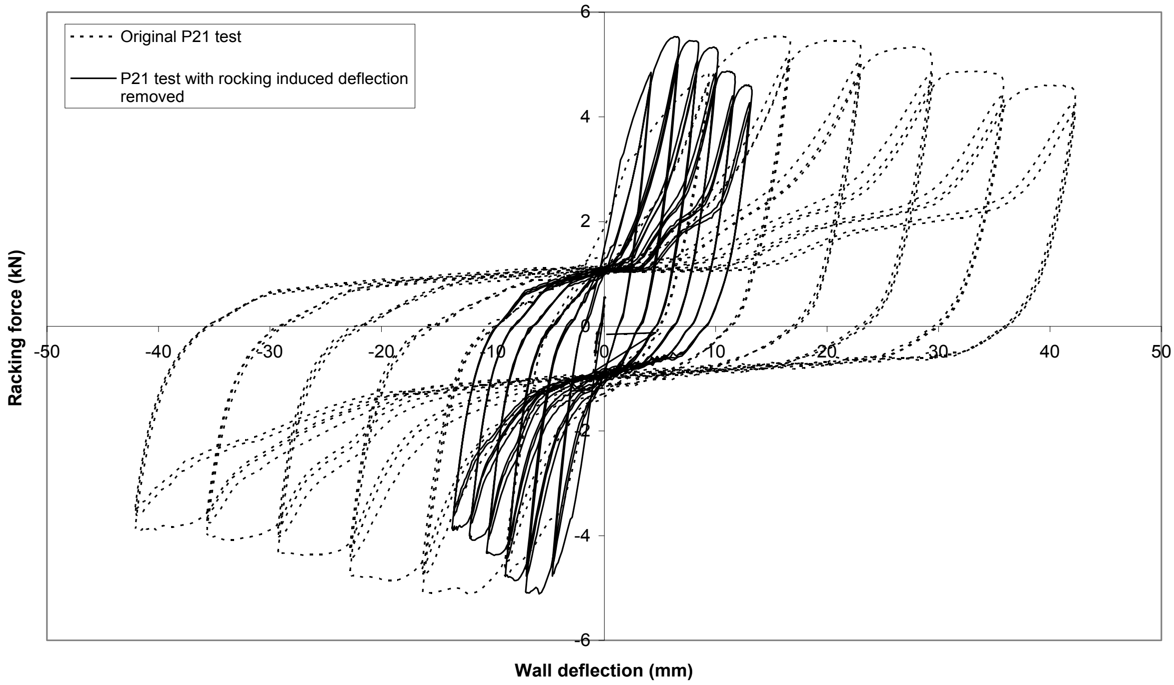

Figure 35. Wall 5 – 1.2 m long wall lined with plasterboard on one face with end straps

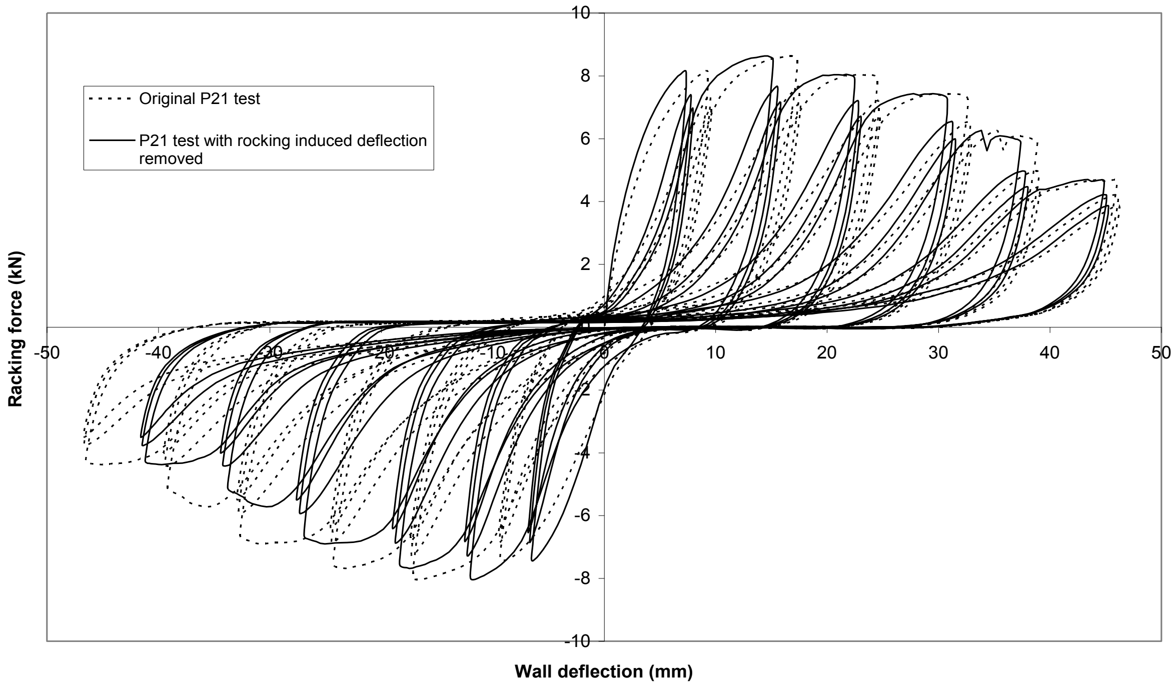

Figure 36. Wall 6 – 1.8 m long wall lined with plasterboard on one face without end straps

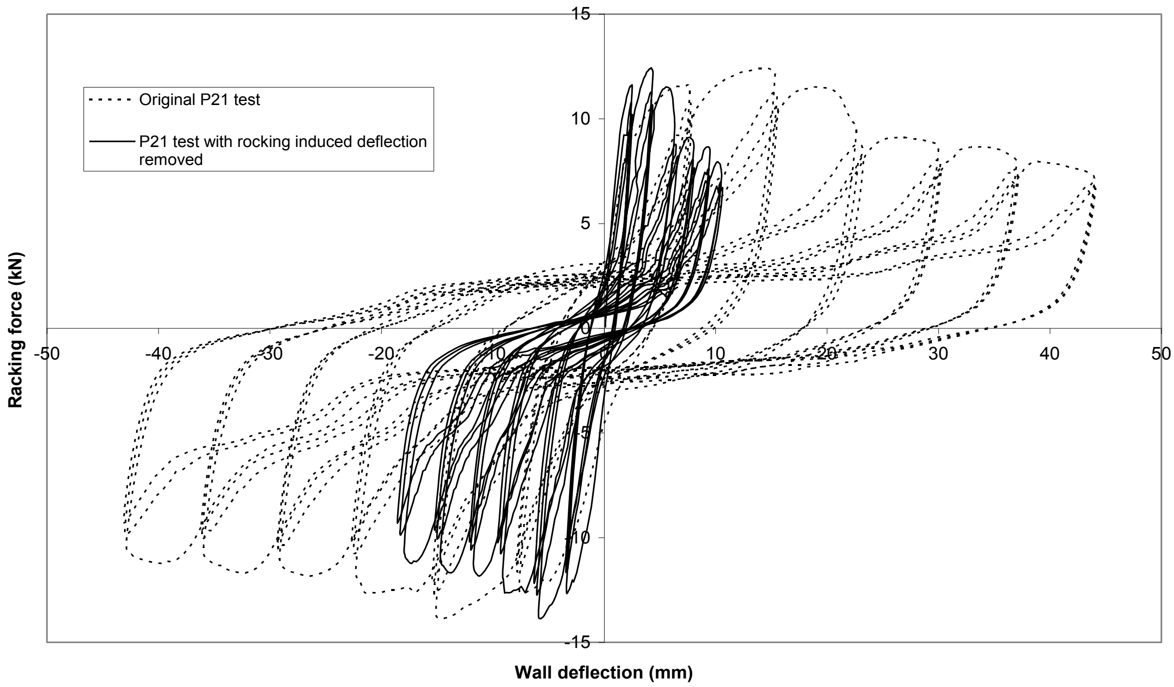

Figure 37. Wall 7 – 2.4 m long wall lined with plasterboard on one face without end straps

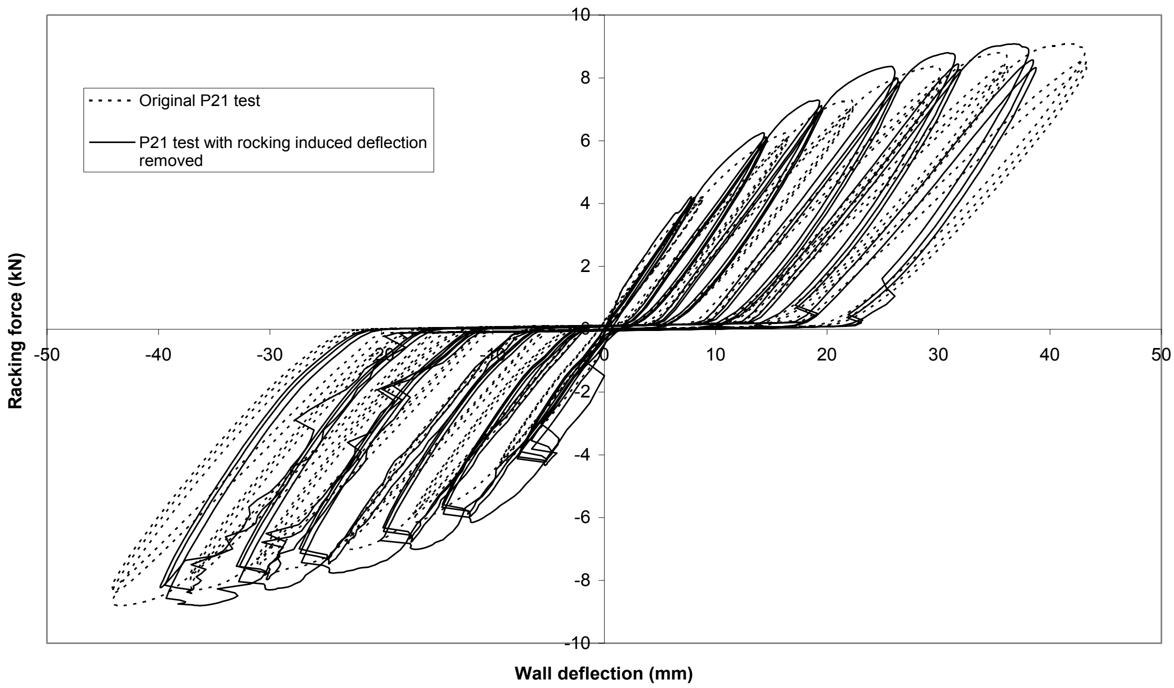

Figure 38. Wall 8 – 2.4 m long diagonally braced wall system

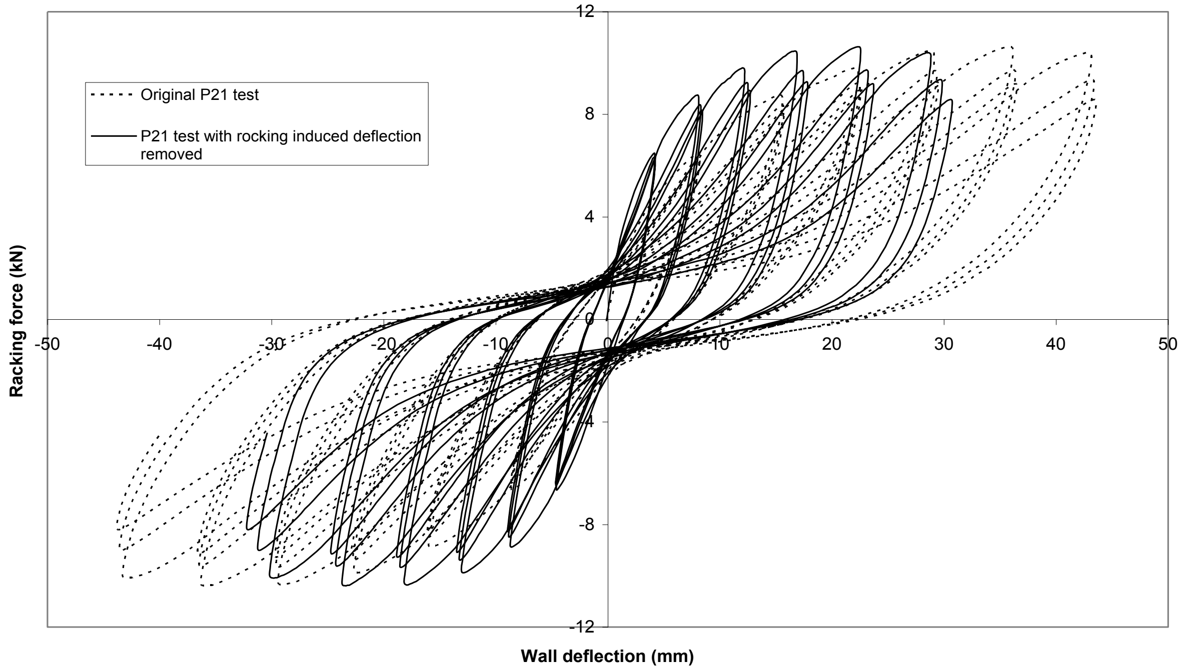

Figure 39. Wall 9 – 1.2 m long wall lined with fibre-cement sheet on one face with end straps

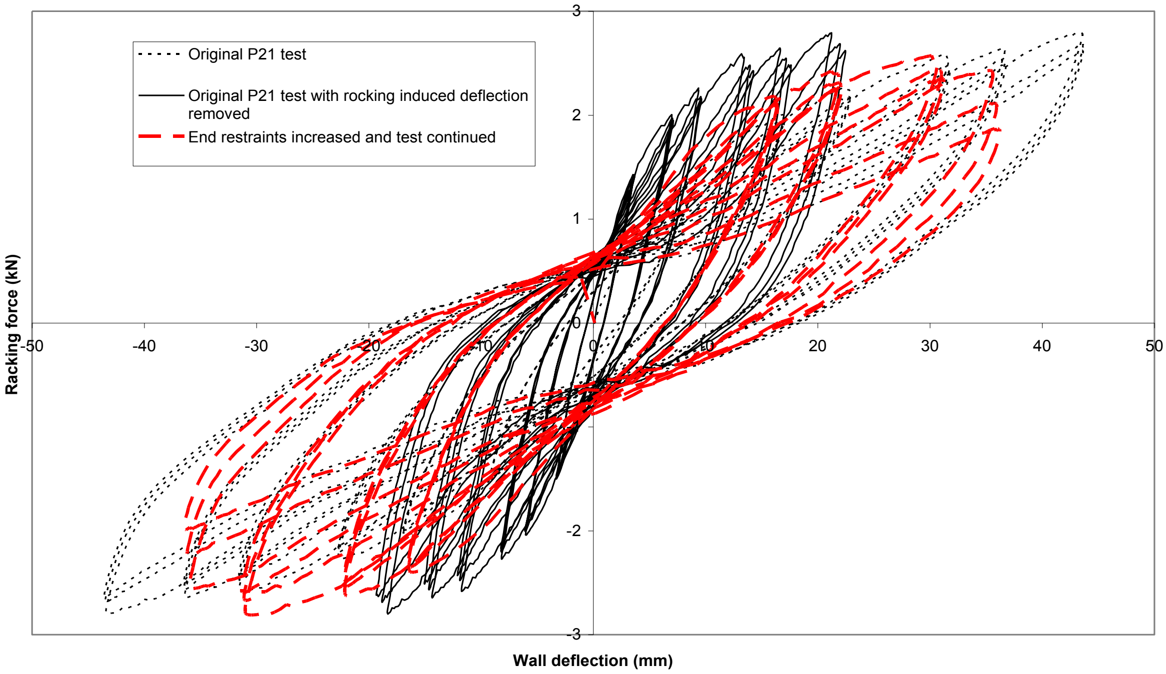

Figure 40. Wall 10 – 0.4 m long wall lined with plasterboard on one face with end straps

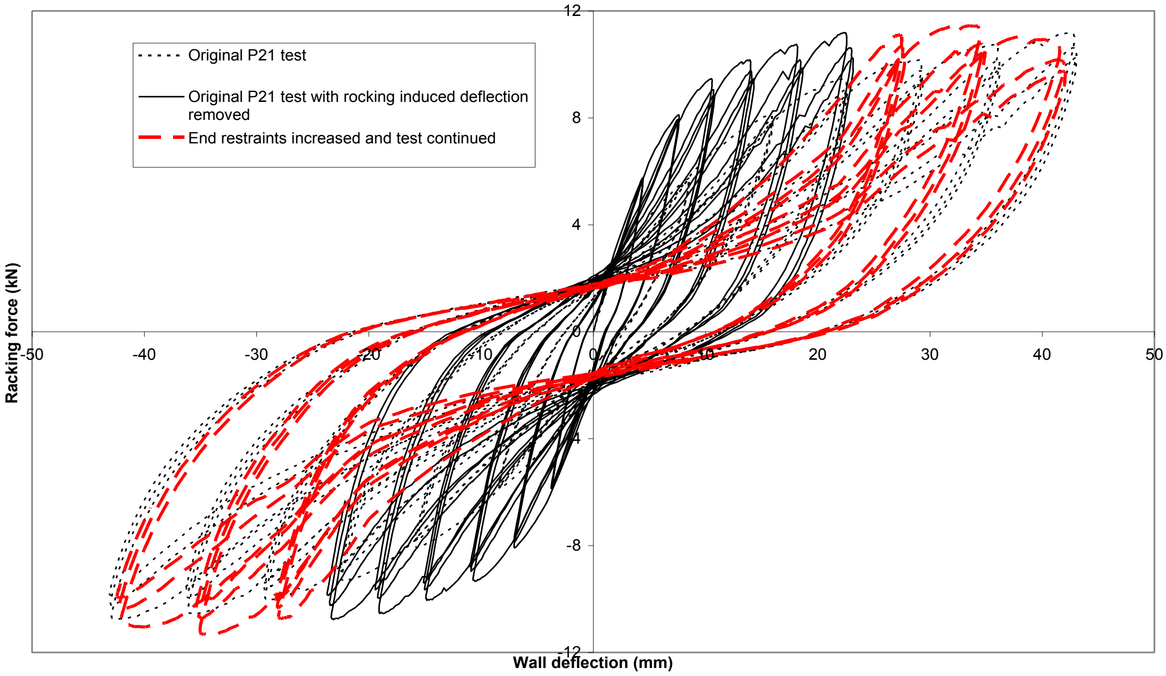

Figure 41. Wall 11 – 1.2 m long wall lined with MDF on one face with end straps

8. CONCLUSIONS

An EM3-V3 test and evaluation procedure has been recommended to replace the existing P21 method for determining bracing ratings of walls that may be used for the design of timber framed structures complying with the scope of NZS 3604. Building performance under design level winds and earthquakes is expected to be more accurately and reliably represented.

Comments on the relative magnitude of P21 and EM3-V3 bracing ratings are:

(1) The P21 and EM3 methods give the same wind bracing ratings.

(2) Walls which reach peak load near or greater than 36 mm wall deflection (when rocking deflections are subtracted) such as many MDF or plywood-lined walls will have similar bracing ratings with both the P21 and EM3-V3 tests and evaluation methods.

9. OTHER RECOMMENDATIONS

9.1 Wall lengths

Provided the criteria below are met, it is recommended that the bracing ratings per unit length of wall are applicable to walls of between 75% and 150% of the length of wall tested, except that tests on walls of length 2.4 m may be used for walls of any greater length.

(Note: the requirements of Section 9 mean that a test series on a 1.2 m long wall and a 2.4 m long wall will enable bracing ratings to be provided for wall lengths 900 mm and longer.)

9.2 Wall heights

Clause 8.3.1.4 of NZS 3604 (SNZ, 1999) stipulates that the bracing rating of panels be factored by 2.4/(element height in metres) with the proviso that walls of height less than 1.8 m shall be assumed to have a height of 1.8 m in application of this factor. The factor implicitly assumes all bracing panels are tested at a height of 2.4 m, and thus it is recommended that the actual test wall height be used in this equation. It also allows the bracing ratings of short walls to increase by a factor of up to 2.4/1.8 = 1.33. This is justifiable only if the panel strength is governed purely by rocking. If instead the panel strength is governed by fastener slip, as will be more common with the EM3 method and is usual for plasterboard panels with small head fasteners, then this relationship does not hold.

BRANZ considers that Clause 8.3.1.4 should be modified so that the formula is only applied to walls greater than 2.4 m high.

9.3 Specific design of non-NZS 3604 buildings

Non-NZS 3604 type buildings do not necessarily have the bracing wall ‘continuity’ or the total-building ‘systems effects’ or the damping inherent in NZS 3604 buildings.

All non-3604 buildings need to be designed to ensure that the predominant displacement mode is due to slip between sheathings and framing and that other brittle failure mechanisms are suppressed. Special end uplift restraints should be designed to resist panels overturning if the continuity features that usually exist in NZ 3604 buildings are not present.

Panel racking over-strength needs to be considered and other elements designed for the associated greater racking force using a capacity design procedure. The end uplift restraints, chords and sheathing need to be designed for this over-strength. A suitable over-strength factor needs to be chosen by the designer e.g. 1.5. Note: C5.2.4 in NZS 3603:1993 recommends an over-strength factor of 2.0.

Designers must ensure that bracing wall sheathing is fixed to framing on all edges, the top and bottom of the sheathing are fixed to a diaphragm at the (roof /ceiling/floor), and the bottom plate is held down and prevented from sliding.

The EM3 test will ensure the above walls, so designed, have an effective ductility of at least 3.0 at the assessed bracing deflection.

Published EM3 earthquake bracing ratings have been factored by F2 = 1.2. This is to simulate 'systems effects' and the strength enhancement in houses having taped and filled plasterboard joints on all building interior walls. This is discussed in Section 5. The designer should modify published test EM3 earthquake strengths to reflect his/her assessment of an appropriate 'systems effect' factor for the building being designed. For many engineered structures this will result in a downgrade of published ratings by 1.2. Published wind bracings ratings have not been factored by F2 and will not need downgrading.

Designers should be aware that the walls were assessed at 15-36 mm deflection and should design other aspects of the buildings to accommodate this movement.

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Selection of accelerograms for Wellington, Auckland and Christchurch

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Client Repor
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Septembe
2004

by Jian Zhang, Graeme McVerry, and Andrew King

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Prepared for

BRANZ

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CONTENTS

EXECUTIVE SUMMARY

Families of three earthquake accelerograms have been selected together with their associated record scale factors K_1 and family scale factors K_2 to be representative of the seismic hazard for Wellington, Auckland, and Christchurch for the following conditions:

Site conditions:	Wellington	Class C Shallow Soil and Class D Deep Soil.
	Auckland	Class C Shallow Soil and Class D Deep Soil.
	Christchurch	Class D Deep Soil.

Risk Factor: 475-year return period R=1.

Period Band: 0.16s to 0.52s, corresponding to $0.4T_1$ to $1.3T_1$, but with T_1 being not less than 0.4s

Structure: one storey with a period range for T_1 of 0.1s to 0.2s.

In addition, the accelerograms have been chosen to be representative of the magnitude, distance ranges and types of earthquakes that contribute significantly to the earthquake hazard at the sites for the spectral values of interest.

The selected accelerograms and scale factors K_1 and K_2 are listed in Tables 1, 2, and 3 for Wellington, Auckland, and Christchurch, respectively. The accelerograms multiplied by their associated K_1 and K_2 are appropriate to use to perform time-history analyses of structures in the period range up to 0.4s, including the required range of 0.1s to 0.2s.

Table 1 Selected accelerograms and K_1 and K_2 for Wellington

Site Conditions	Station Name	Event	Date	Magnitude	Distance (km)	K_1	K_2
Shallow Soil	Tabas	Tabas	1978.9.16	7.4	1.2	0.48	1.10
	La Union	Mexico	1985.9.19	8.1	16.0	2.32	
	El Centro	Imperial Valley	1940.5.19	7.0	12.0	1.39	
Deep Soil	El Centro	Imperial Valley	1940.5.19	7.0	12.0	1.63	1.02
	Joshua Tree	Landers	1992.6.28	7.23	11.3	1.99	
	Llolleo	Chile	1985.3.3	7.9	61?	0.79	

Table 2 Selected accelerograms and K_1 and K_2 for Auckland

Site Conditions	Station Name	Event	Date	Magnitude	Distance (km)	K_1	K_2
Shallow Soil	Delta	Imperil Valley	1979.10.15	6.5	32.7	0.53	1.0
	Matahina	Edgecumbe	1987.3.3	6.5	18.9	0.43	
	Bovino	Campo Lucano	1980.9.23	6.8	39.0	2.91	
Deep Soil	Korinthos-OTE	Aikon	1981.	6.6	10.0	0.61	1.13
	Delta	Imperial Valley	1979.10.15	6.5	32.7	0.62	
	Calipatra	Imperial Valley	1979.10.15	6.5	23.0	1.71	

Table 3 Selected accelerograms and K_1 and K_2 for Christchurch

Site Conditions	Station Name	Event	Date	Magnitude	Distance (km)	K_1	K_2
Deep Soil	Boshrooyeh	Tabas	1978.9.16	7.4	34.0	2.69	1.07
	Taft	Kern County	1952.7.21	7.4	42.0	1.56	
	Wrightwood	Lytle Creek	1970.9.12	5.3	13.0	1.82	

1.0 INTRODUCTION

This report has been prepared following instructions received from BRANZ to select accelerograms to be used for earthquake time history analysis of structures at Wellington, Auckland, and Christchurch according to the following parameters:

Site conditions:	Wellington	Class C (Shallow Soil) and Class D (Deep Soil).
	Auckland	Class C (Shallow Soil) and Class D (Deep Soil).
	Christchurch	Class D (Deep Soil).

Risk Factor: 475-year return period R=1.

Spectral-matching Period Band: $0.4T_1$ to $1.3T_1$, with T_1 taken as 0.4s for shorter periods

Structure: one storey with a period range for T_1 of 0.1s to 0.2s.

The essence of the work is to select three accelerograms appropriate for meeting the above conditions for Wellington, Auckland, and Christchurch. The results will be provided as recommendations of these accelerograms and their scalings for the three regions for structures with fundamental periods T_1 up to 0.4s.

2.0 HAZARDS FROM EARTHQUAKES

The selected accelerograms for each site are intended to represent as closely as possible the magnitude, distance and focal mechanism of the earthquake source(s) that make main contributions to the rate of exceedance of the draft code spectrum at a site for Risk Factor R=1, as well as being recorded at a station with appropriate site conditions. These characteristics are referred to in the draft Loadings Standard as the "seismic signature" of the site. To this end, it is useful to understand the tectonic setting of New Zealand and the types of earthquakes from different tectonic locations, for example: crustal, subduction interface and subduction slab.

2.1 Tectonic Setting of New Zealand

New Zealand straddles the boundary of the Australian and Pacific plates, where relative plate motion is obliquely convergent across the plate boundary. The relative plate motion is expressed in New Zealand by the presence of many active faults, a high rate of "small-to-moderate" earthquakes (M<7), the occurrence of many "large" earthquakes (M7-7.9) and one "great" earthquake (M>8) in historical time. A southeast-dipping subduction zone lies at the far south-western end of the country ("Fiordland subduction zone" in Figure 2.1). It is linked to a major northwest-dipping subduction zone in the eastern North Island ("Hikurangi subduction zone") by a 1000 km long zone of right-lateral oblique slip faults ("Axial tectonic belt"). Essentially all of the relative plate motion is accommodated by the faults of the axial tectonic belt in the area between the Fiordland and Hikurangi subduction zones.

The Hikurangi subduction interface dips beneath the eastern North Island. Only one large (M_w>7) earthquake and no great (M_w>8) earthquakes are known to have been produced by the Hikurangi subduction zone in historical times (since 1840), and so little is known about the earthquake potential of this feature. The Fiordland subduction zone trench is located immediately offshore from Fiordland and the subducting slab dips steeply southeast beneath Fiordland. Some of the highest rates of seismicity in the country occur within the dipping slabs of the subduction zones. Frequent moderate earthquakes also occur above the Fiordland subduction zone, and to a lesser extent above the Hikurangi subduction zone.

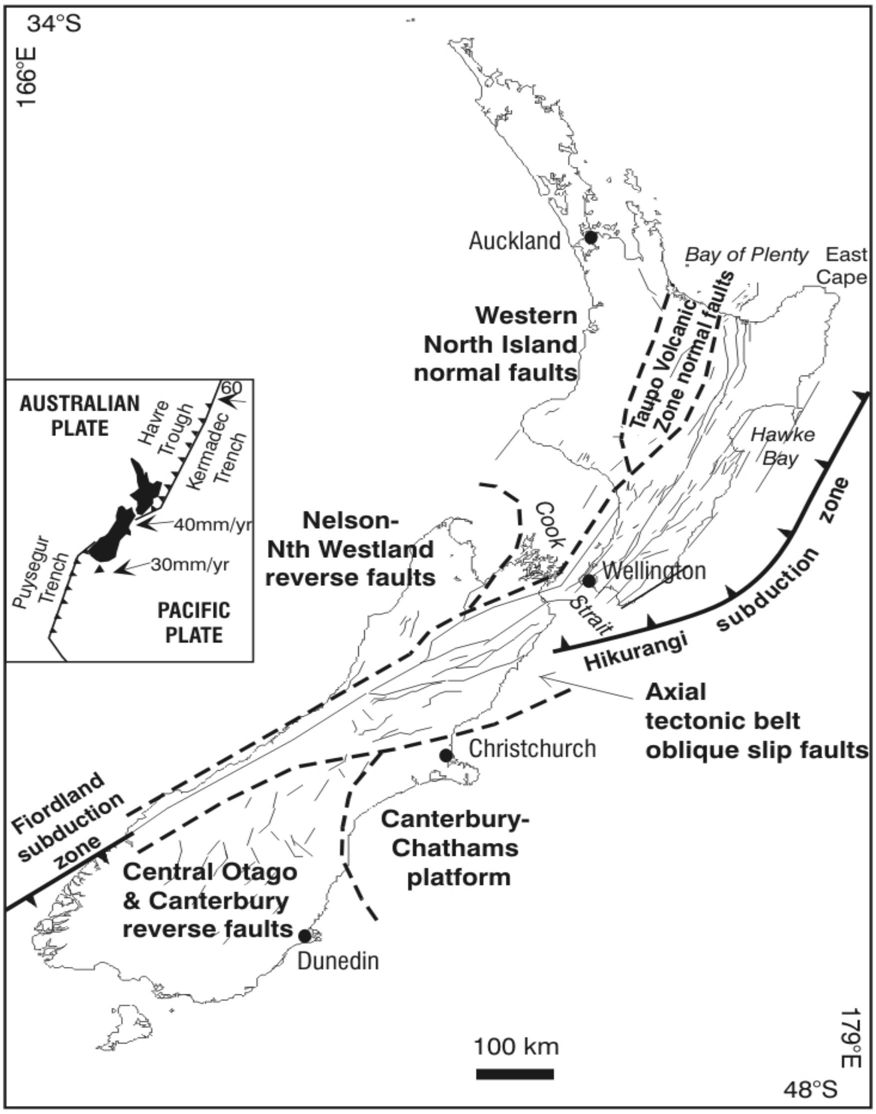


Figure 2.1 Tectonic setting of New Zealand

The axial tectonic belt is a zone that is composed of right-lateral strike-slip motion and compression. Many moderate or larger earthquakes have occurred within the axial tectonic belt in historic time, including New Zealand's two largest historical earthquakes (the Mw8.1-8.2, 1855 Wairarapa earthquake, and Mw7.8 Hawke's Bay earthquake). The axial tectonic belt includes the Alpine Fault. This fault accommodates virtually all of the relative plate motion in the central South Island, but has not produced any large or great earthquakes in historic time. Geologic data provide evidence for the occurrence of great earthquakes on the Alpine Fault with return times of about 300 years.

2.2 Regional Seismicity

Seismicity in New Zealand varies regionally from moderate to very high on a world scale. Wellington lies in one of the most active of New Zealand's seismic regions and Auckland in one of the least active. Dunedin lies in a region that is a little more active than the Auckland region. Activity in the Christchurch area is intermediate between that of Wellington and Dunedin. These differences are illustrated by Figure 2.2, which shows the locations of the major earthquakes that have occurred in the New Zealand area since 1840.

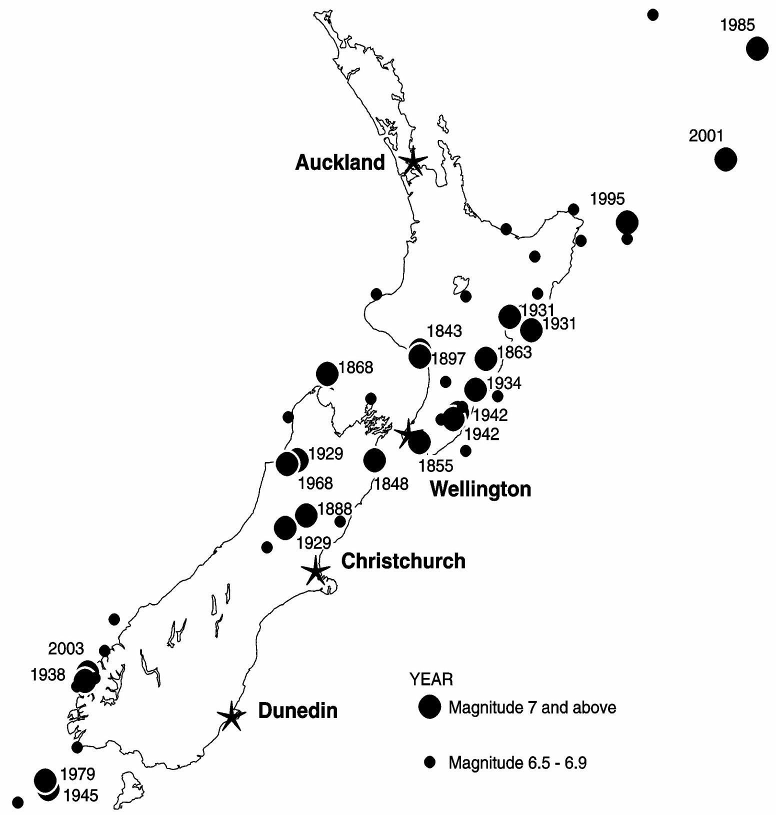


Figure 2.2 Occurrence of large earthquakes in New Zealand since 1840.

From the characteristics of seismicity at Wellington, Auckland, and Christchurch, it is known that the seismic hazard is dominated by normal faulting in Auckland, reverse faulting in Christchurch, and by the strike-slip Wellington Fault in Wellington.

3.0 THE NATIONAL SEISMIC HAZARD MODEL

The National Seismic Hazard Model (Stirling et al., 2000, 2002) has been used as the basis of several of the design spectra presented in this report. The model is based on a uniform hazard approach wherein the earthquake hazard is assessed in terms of a nominated annual probability of exceedance so that the results are consistent between locations with New

Zealand. The model was used as the basis for the seismic coefficients and zone factor maps for New Zealand in the Draft Australia-New Zealand Loadings Standard DR 1170.4/PPC2 (Standards Australia/Standards New Zealand, 2002).

In the National Seismic Hazard Model, the seismicity of New Zealand is modelled by a combination of fault sources that produce earthquakes of a specified magnitude and average recurrence interval and a grid of distributed point sources with a range of earthquake magnitudes satisfying Gutenberg-Richter (G-R) magnitude-recurrence interval relations (Gutenberg & Richter, 1944). The distributed point sources model the historical seismicity that is not associated with known faults. The values of the parameters of the G-R relations for each of the distributed sources have been derived from the historical earthquake catalogue, while the parameters of the fault sources have usually been estimated from geological studies.

In the National Seismic Hazard Model, the three-dimensional grid of point sources has a horizontal spacing of about 10km, with layers at depths of 10, 30, 50, 70, and 90km. The historical record of earthquakes is used to estimate the recurrence rate of moderate-to-large magnitude (M5.25 up to a regionally-varying maximum “cutoff” magnitude, Mcutoff) “distributed” earthquakes assigned to this grid.

The selection of the lower-bound magnitude of 5.25 dates from the development of the current New Zealand Loadings Standard NZS4203:1992 (Standards New Zealand, 1992), when the code committee “considered that structures designed in accordance with current design code standards would not be susceptible to damage from the effects of earthquakes with magnitudes less than 5.25” (Matuschka et al., 1985). At about the same time, magnitude 5.0 was adopted as the usual lower-bound magnitude in probabilistic seismic hazard analysis in U.S. practice for similar reasons (e.g. Reiter 1990, pp.201-203), and has been retained to the present (e.g. Frankel et al., 2002). We have retained a lower-bound magnitude of 5.25 rather than moving to the U.S. practice of 5.0 because the McVerry et al. (2000) attenuation model used in the model produces unusually high response spectrum accelerations around the spectral peaks in the 0.1-0.3s range at short distances for M<5.25.

The upper-bound “cutoff” magnitude used to calculate the Gutenberg-Richter distributed seismicity rates is set according to the tectonic regime of the area. In some regions its selection acknowledges that very large earthquakes have occurred on previously unknown “blind” or “hidden” faults (e.g. 1931 Hawkes Bay earthquake, M7.8). Elsewhere, the upper-bound cutoff magnitude is considerably lower, because it has been assessed that large magnitude earthquakes either do not occur in a region or are accommodated by the fault sources. For the Auckland region, the maximum cutoff magnitude is taken as 7.0.

The National Seismic Hazard Model incorporates the latest ground motion attenuation relationships (McVerry et al., 2000). An important feature of the new attenuation relationships is that they take into account the different tectonic types of earthquakes in New Zealand (i.e. crustal, subduction interface, and dipping slab). The attenuation relationships for crustal earthquakes have further subdivisions, through mechanism terms, for different types of fault rupture (strike-slip, normal, oblique/reverse and reverse). They also cater for several site conditions that are defined in terms of Classes A/B, C and D of the Draft Loadings Standard. The attenuation expressions have been derived from a database incorporating all available New Zealand strong-motion response-spectral data supplemented by a representative selection of overseas near-source peak ground acceleration data. Both the crustal and subduction zone attenuation relations have been derived by modifying models from other parts of the world to obtain better fits to the supplemented New Zealand database. The crustal model was modified from the Abrahamson & Silva (1997) model that was derived from mainly western US data, while the subduction zone expression was modified from the Youngs et al. (1997) expression derived from subduction zone earthquakes around the world.

4.0 DERIVATION OF TARGET (ACCELERATION) SPECTRA

In this report, target spectra for shallow and deep soil sites are directly derived from loadings standard (Standards Australia/Standards New Zealand, 2002) and are given in equation 4.1

$$C(T) = C_H(T)\ Z\ R\ N(D,T) \tag{4.1}$$

where

$C_H(T)$ = Spectral shape factor (based on uniform hazard) for the appropriate site class

Z = Zone factor, a seismicity-related scale factor

R = Risk factor, a factor related to the target return period for the limit state and building importance category

$N(D,T)$ =Near-fault factor, a period-dependent factor reflecting forward-directivity effects from active fault.

T =fundamental period of structures

Two implications were made in the approach used in the Loadings Standard, namely:
a) the spectral shape factor for a given site class is the same throughout New Zealand,
b) risk factor as a function of the annual probability of exceedance is independent of location.

In the present report, risk factor R equals to 1 for 475-year return period and zone factor Z equals to 0.40, 0.13 and 0.22 for Wellington, Auckland, and Christchurch, respectively.

Near-fault factor is used to account for forward-directivity effects in the range of 0-20km from a major active fault, and the following formula for calculating the near-fault factor are given in the Loadings standards.

Code Near-Fault Factor

$$
\begin{aligned}
&= 1.0 && T = 1.5s \\
&= 1 + 0.24\,(T - 1.5) && 1.5s < T = 4s \qquad (4.2) \\
&= 1.6 + 0.12\,(T - 4) && 4s < T = 5s \\
&= 1.72 && 5s < T
\end{aligned}
$$

Equation 4.2 shows that near source effects are considered only for spectral periods larger than or equal to 1.5s, and the near-source factor is not relevant to the present study because the accelerograms selected here are for short period structures only. The spectral shape factors, $C_H(T)$, for Class Site C and D are listed in Table 4.1.

Table 4.1 Spectral shape factors, $C_H(T)$, for Class Site C and D

Period, T (second)	Spectral shape factor $C_h(T)$ (g)	
	Site Subsoil Class	
	C Shallow Soil	D Deep or Soft Soil
0.0	1.33	1.12
0.1	2.93	3.00
0.2	2.93	3.00
0.3	2.93	3.00
0.4	2.36	3.00
0.5	2.00	3.00
0.6	1.74	2.84
0.7	1.55	2.53
0.8	1.41	2.29
0.9	1.29	2.09
1.0	1.19	1.93
1.5	0.88	1.43
2.0	0.66	1.07
2.5	0.53	0.86
3.0	0.44	0.71
3.5	0.32	0.52
4.0	0.25	0.40
4.5	0.20	0.32

5.0 SELECTION OF ACCELEROGRAMS AND SCALE FACTORS K_1 AND K_2

The selection of accelerograms is based on the deaggregated results of the acceleration spectra for a 475-year return period for shallow and deep soil sites. When we select accelerograms, four factors are considered, i.e. earthquake magnitude, shortest distance from a site to fault rupture plane, site conditions and focal mechanism. A database of accelerograms consistent with the deaggregated results for Wellington, Auckland, and Christchurch has been constructed from world-wide strong motion records. From this database, we are able to select accelerograms with most desirable values for K_1 and K_2.

For the two horizontal components of each recommended accelerogram, K_1 has been calculated from the best least-square matches between the accelerogram spectra and the target spectra in logarithm scale. The logarithm of the spectrum is used because it is found that spectral accelerations are generally close to log-normally distributed. Before selecting accelerograms, a number of fitted periods (T_{fit}), 0.4s, 0.5s, 1.0s, 1.5s, 2s, 2.5s, 3s, 4s, and 6s, have been chosen, and these T_{fit} are assumed to be structural fundamental periods. For each of these T_{fit}, a fitted period range of $0.4T_{fit}$ to $1.3T_{fit}$ has been determined based on the conditions in Section 1. To select appropriate accelerograms, RMS errors in logarithm scale over the period ranges of $0.4T_{fit}$ to $1.3T_{fit}$ are calculated, and then converted to factors 10^{RMS1} and 10^{RMS2} for the two components. Values less than 1.2 for these two factors represent exceptional matches, values between 1.2 and 1.3 are very good, values between 1.3 and 1.4 are good, and values between 1.4 to 1.5 are marginally acceptable. A value larger than 1.5 indicates that the accelerograms have the wrong spectral shape over the period range of interest. To show the quality of fit, the 5% damped elastic acceleration response spectra of the scaled accelerograms are plotted against the target spectra for a fitted period of 0.4s in following Sections. The reason for using 0.4s period is because the Loadings Standard requires that using T_{fit}=0.4s determines K_1 if T_{fit} is less than 0.4s.

K_2 is a family scale factor which is required to ensure that for every period in the period range of interest the spectrum from at lease one record exceeds the target spectrum. The rule for selecting K_2 is: if K_2 is in a range of 1.0 and 1.3, the selection for the principal and secondary components may be confirmed; if K_2 is less than 1.0, set K_2=1.0; and if K_2 is larger than 1.3, the selected principal components should be reconsidered.

5.1 Selected Accelerograms and Scaling Factors K_1 and K_2 for Wellington

To derive the contribution from different earthquake sources to the seismic hazard in Wellington, deaggregation analyses for the seismic hazard for a 475-year return period have been performed. The contributions of major active faults and distributed seismicity have been calculated as a function of magnitude and shortest distance to the rupture plane. Focal mechanisms for these active faults are listed in the report of Stirling et al (2000).

Deaggregated results for a period 0.4s are shown in Fig.5.1.1. The contributions of the main active faults are listed in Table 5.1.1 which shows that the Wellington-Hutt Valley segment of the Wellington fault makes the maximum contribution to the seismic hazard. Also note that the Hikurangi subduction zone makes a contribution to the hazard level.

Accelerograms have been selected and are listed in Table 1, together with corresponding record-scaling factor K_1 and family-scaling factor K_2. The quality for selected K_1 fitted to the period range of 0.16s to 0.52s is shown in Figs. 5.1.2 and 5.1.3, and for K_2 is shown in Fig.5.1.4. Figs. 5.1.2 and 5.1.3 suggest a good fit by the scale factor K_1 in the period range of 0.16s to 0.52s. Fig. 5.1.4 suggests a good fit by scale factor K_2. Note that near-fault factor is not relevant to the short period structures considered in this study.

For shallow soil site conditions, the accelerograms representing the contributions of the main active faults for Wellington are the La Union record from the 1985 Michoacan earthquake with a magnitude of 8.1, the El Centro record from the 1940 Imperial Valley earthquake with a magnitude of 7.0 and the Tabas record from 1978 Tabas earthquake with a magnitude of 7.4. For deep soil site conditions, the Joshua Tree record is from the 1992 Landers earthquake with a magnitude of 7.2, the Llolleo record is from the 1985 Chile earthquake with a magnitude of 7.9 and the El Centro record is from the 1940 Imperial Valley earthquake with a magnitude of 7.0. For each of the selected earthquake records, a more detailed description is given below:

For shallow soil site conditions:

The La Union record was recorded on a rock site above the rupture plane at a distance of 16km (Singh *et al.*, 1988). This record is taken as representative of a great subduction zone earthquake affecting Wellington for shallow soil site conditions. Records from crustal earthquakes exceeding magnitude 8 are not available. Therefore this M_w=8.1 event is also taken as the representative of the Wairarapa Fault earthquake event.

The El Centro 1940 record, recorded at a distance of 12km from the rupture plane in a magnitude 7.0 strike-slip earthquake, is taken as representative of Ohariu fault and Pukerua-Shepherds Gully fault earthquakes. It is a strong near-source record, fits target spectra very well, and has a wide period range of constant spectral velocity, although it was recorded on a deep soil site rather than a shallow soil site.

The Tabas record was recorded at a distance of 1.2 km from the rupture plane in a magnitude 7.4 earthquake. The record as provided has been processed to retain more low-frequency content than usual so that the forward-directivity pulse is retained. This record is recommended to represent the possible effect of an earthquake on the Wellington-Hutt Valley segment of the Wellington fault.

For deep soil site conditions:

The Joshua Tree record was recorded at a distance of about 11km from the rupture plane on a stiff soil site. This record is representative of the response of a deep soil site to an earthquake on the Wellington fault, the Ohariu fault and the Pukerua-Shepher Gully fault. The Joshua Tree record shows a backforward-directivity motion, rather than a forward-directivity motion.

The Llolleo record was recorded during a subduction zone earthquake at a distance of about 61km from the rupture plane on a deep soil site. This record is selected to represent the contribution from a Hikurangi subduction zone earthquake at a deep soil site in Wellington.

The El Centro record has been used for shallow soil site conditions, but it was recorded on a deep soil site. The El Centro record is appropriate for an earthquake source to represent the seismic hazard of the Ohariu fault and the Pukerua-Shepherds Gully fault. Note that this record has a wide constant-velocity period band.

Table 5.1.1 Earthquake Source Contribution for Wellington

Fault Name	Magnitude & distance from source	Contribution(%)
Wellington-Hutt Valley segment of Wellington Fault	M7.3@0.5-2km	38
Ohariu	M7.4@5-10km	6
Pukerua-Shepherds Gully	M7.2@10km	3
Wairarapa	M8.1@20km	15
Hikurangi Subduction Zone-Wellington segment	M7.8-8.4@20km	6

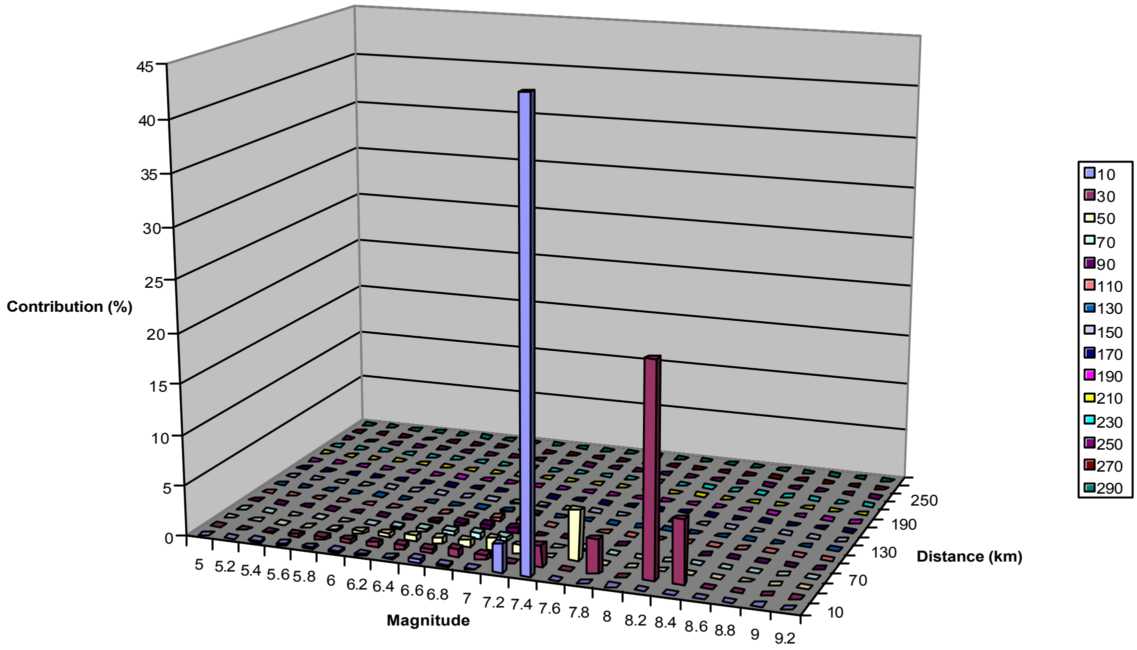
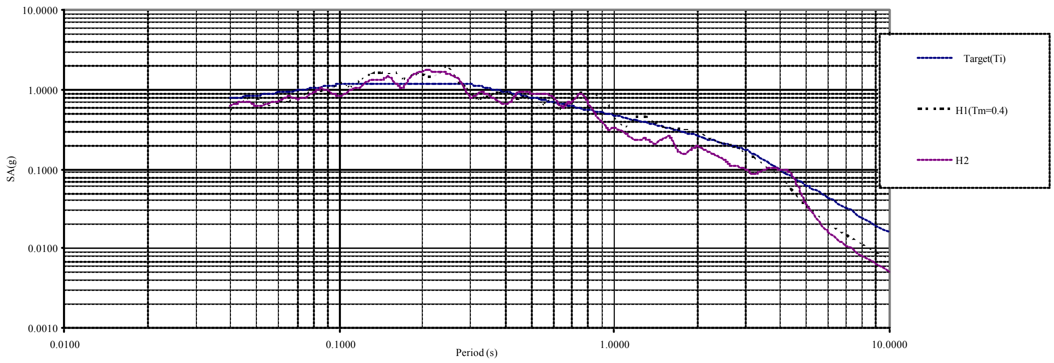


Figure 5.1.1 Deaggregated result for Wellington shallow soil sites at a period 0.4s. Wellington fault and Wairarapa fault dominate the hazard level.

MATCHING CODE SPECTRUM Tfit=0.4s
TABAS TABAS78 matching Wellington Shallow Soil

Australia/NZ code spectrum with (1/T)^0.75 branch matched from 0.4T(fit) to 1.3T(fit)

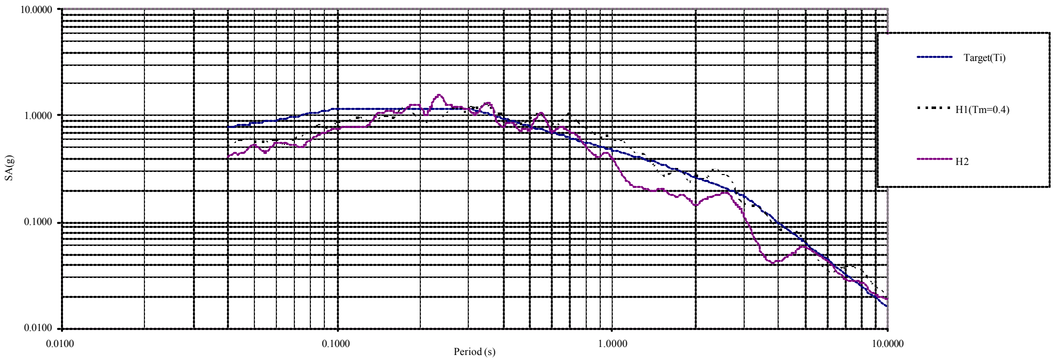
MSF= 1.00 Rock pga Z= 0.40 R= 1.00 Scaling factors Kfirst= 0.56 Ksecond= 0.48 RMS1= 0.100 RMS2= 0.109
Standard spectrum Rrup (km) = 2.00 Matching= Primary Match



MATCHING CODE SPECTRUM Tfit=0.4s
LAUNION LAUNION85 matching Wellington Shallow Soil

Australia/NZ code spectrum with (1/T)^0.75 branch matched from 0.4T(fit) to 1.3T(fit)

MSF= 1.00 Rock pga Z= 0.40 R= 1.00 Scaling factors Kfirst= 2.32 Ksecond= 2.45 RMS1= 0.067 RMS2= 0.058
Standard spectrum Rrup (km) = 2.00 Matching= Primary Match



MATCHING CODE SPECTRUM Tfit=0.4s
El Centro ELCENTRO40 matching Wellington Shallow Soil

Australia/NZ code spectrum with (1/T)^0.75 branch matched from 0.4T(fit) to 1.3T(fit)

MSF= 1.00 Rock pga Z= 0.40 R= 1.00 Scaling factors Kfirst= 2.01 Ksecond= 1.39 RMS1= 0.125 RMS2= 0.102
Standard spectrum Rrup (km) = 2.00 Matching= Primary Match

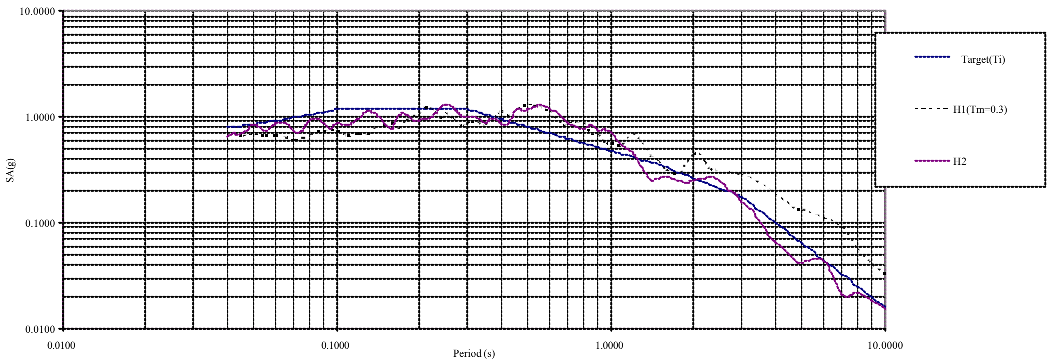
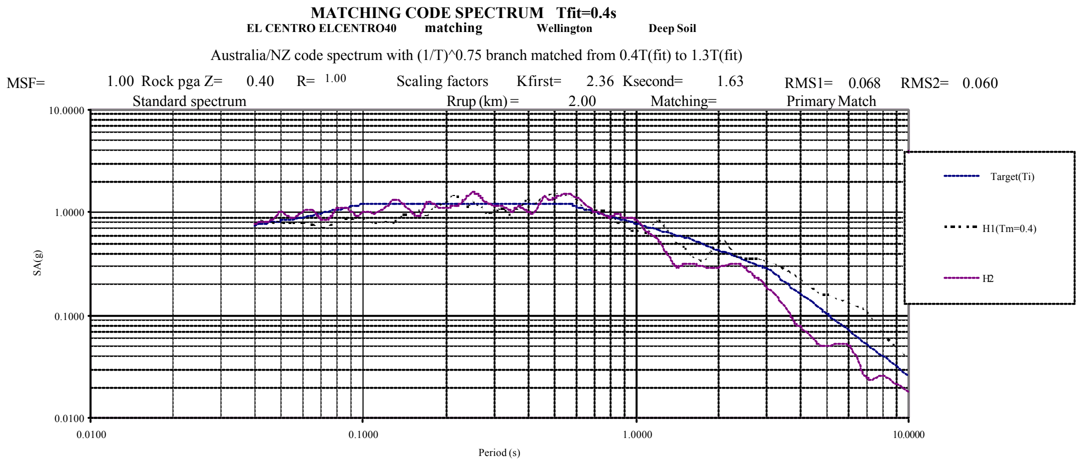
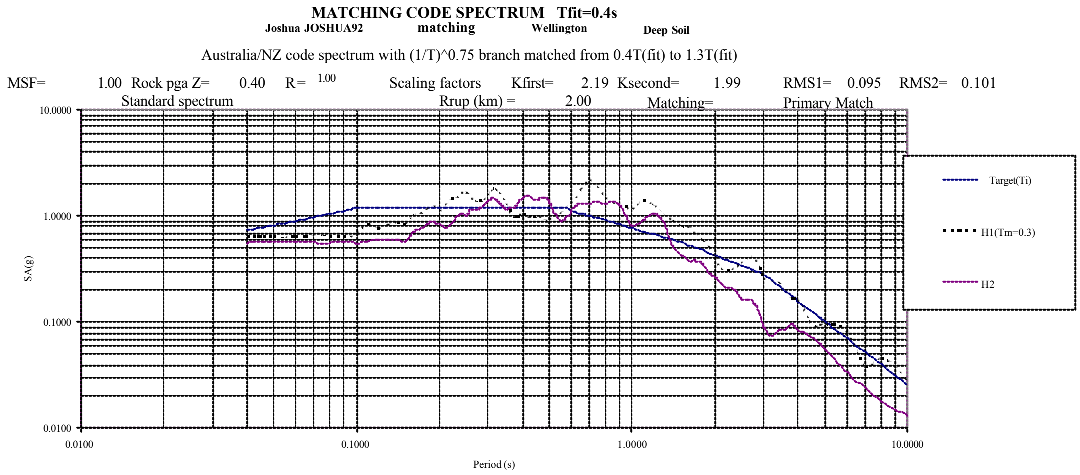


Figure 5.1.2 Matching the target spectrum at Tfit=0.4s for shallow soil site. Tabas, La Union, and El Centro earthquake records have been selected as the required accelerograms. K1 is represented by Kfirst and Ksecond for two horizontal components.





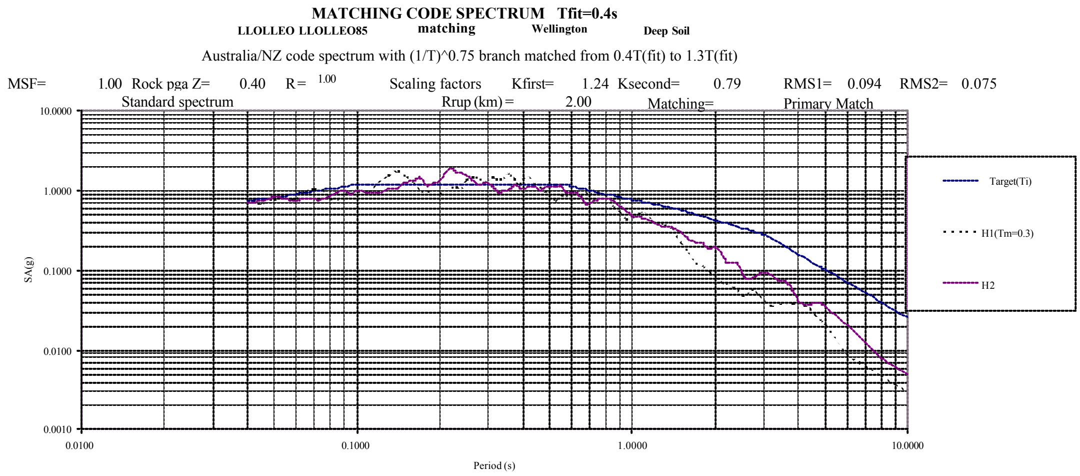
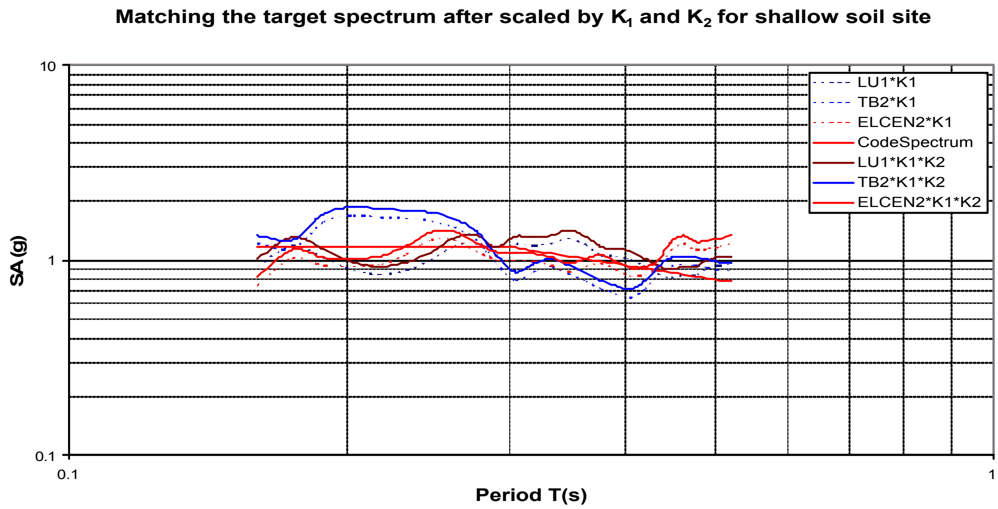


Figure 5.1.3 Matching the target spectrum at T_{fit}=0.4s for deep soil site. El Centro, Joshua Tree, and Llolleo earthquake records have been selected as the required accelerograms. K_1 is represented by K_{first} and K_{second} for two horizontal components.



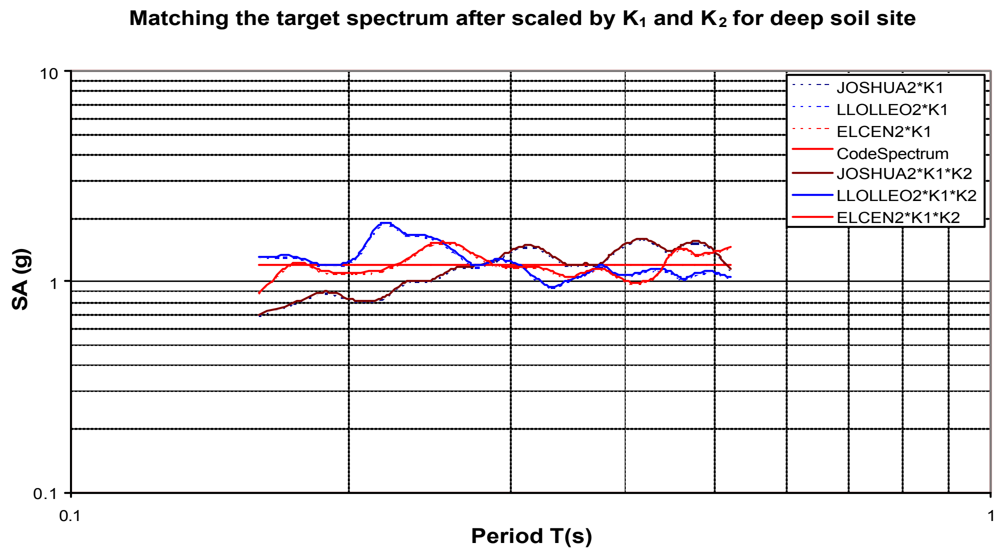


Figure 5.1.4 Comparison of scaled spectra by K_1 and K_2 with the target spectrum at shallow and deep soil sites.

5.2 Selected Accelerograms and Scaling Factors K_1 and K_2 for Auckland

Similar to Section 5.1, deaggregation analyses are performed for Auckland for 475-year return period at shallow and deep soil site conditions. Fig.5.2.1 shows the deaggregated result for a period of 0.4s and Table 5.2.1 lists the contribution of main faults and distributed seismicity.

Selected accelerograms are listed in Table 2, together with corresponding scale factors, K_1 and K_2. Comparisons between the target spectrum and scaled spectra by K_1 for the selected accelerograms are shown in Figs. 5.2.2 and 5.2.3. Further, comparisons between the target spectra and scaled spectra by K_1 and K_2 for the period range of interest are shown in Fig.5.2.4. Figs. 5.2.2 and 5.2.3 show a good fit by scale factor K_1 at a period range of 0.16s to 0.52s.

Fig.5.2.4 suggests that the matching is reasonable between the target spectrum and scaled spectra by scale factors K_1 and K_2.

In the draft AUS/NZ Loadings Standard 1170.4, a magnitude 6.5 earthquake at a distance of 20 km is representative of the seismic hazard in Auckland. This is analogous to the seismic hazard from the Wairoa North fault and the Kerepehi North fault, as well as the distributed seismicity. These hazard sources are a key factor considered in the selection of accelerograms.

For shallow soil site conditions, the selected accelerograms are: the Delta record from the 1979 Imperial Valley earthquake with a magnitude 6.5, the Bovino record from the 1980 Campano Lucano earthquake with a magnitude 6.8, and the Matahina record from the 1987 Edgecumbe earthquake with a magnitude 6.5. For deep soil site conditions, the selected accelerograms are: the Korinthos-OTE record from the 1981 Alkion earthquake with a magnitude 6.6, the Calipatra record from the 1979 Imperial Valley earthquake with a magnitude 6.5, and the Delta record from the 1979 Imperial Valley earthquake with a magnitude 6.5. These records are explained in detail below:

For shallow soil site conditions:

The Delta record at a distance of 32km from the rupture plane on a deep soil site is representative of the Wairoa North fault earthquake, and also representative of the M6.5@20km event used in the draft AUS/NZ Loadings Standard. The distributed seismicity listed in Table 5.2.1 is also represented by the Delta record. Seismic hazards in Auckland are dominated by earthquakes on normal fault. Compared with other focal mechanisms, a ground motion from a strike-slip event is a better representative of a normal fault event than any of the other focal mechanisms.

The Bovino record was recorded at a distance of 39km from the rupture plane on a shallow soil site in a normal fault earthquake. It is a possible representative of the Kerepehi North fault earthquake and some distributed seismicity.

The Matahina record was recorded at a distance of 19km to the rupture plane on a rock site. The Matahina station is located at the bottom of the Matahina dam with alluvial gravels of 15m deep. This is a sole New Zealand record selected in the dataset to represent the M6.5@20km event.

For deep soil site conditions:

The Korinthos-OTE record is from a normal fault earthquake at a distance of 10km from the rupture plane on a deep soil site. This record is a representative of M6.5@20km event, the Wairoa North fault earthquake, and some distributed seismicity.

The Calipatra record was recorded in a strike-slip earthquake at a distance of 23km from the rupture plane on a deep soil site. This record is a possible representative of the Wairoa North fault earthquake.

The Delta record is also used for deep soil site conditions. The Delta record fits the target spectra for shallow and deep soil sites for Auckland quite well.

Table 5.2.1 Earthquake source contribution for Auckland shallow soil site

Fault Name	Magnitude & distance from source	Contribution(%)
Wairoa North Fault	M6.6@27km	1
Kerepehi North Fault	M6.7@62km	2
Distributed Seismicity	M5.1-5.9@0-20km	6
Distributed Seismicity	M5.3-5.9@20-40km	7
Distributed Seismicity	M5.9-6.7@20-40km	12

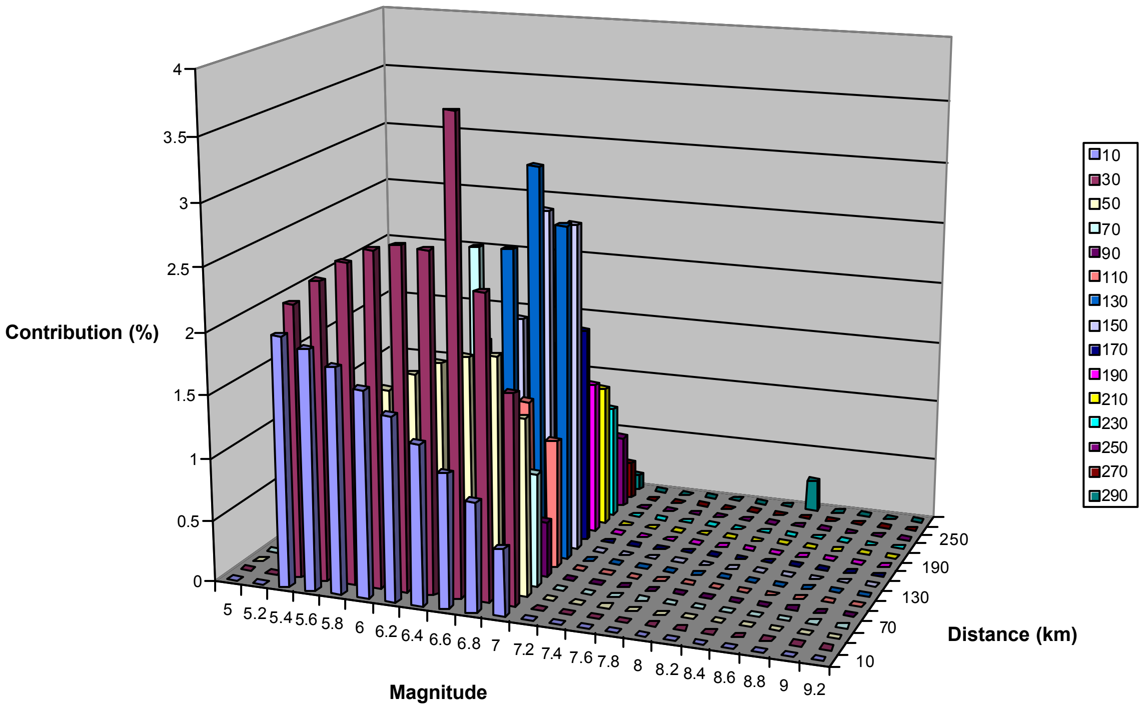
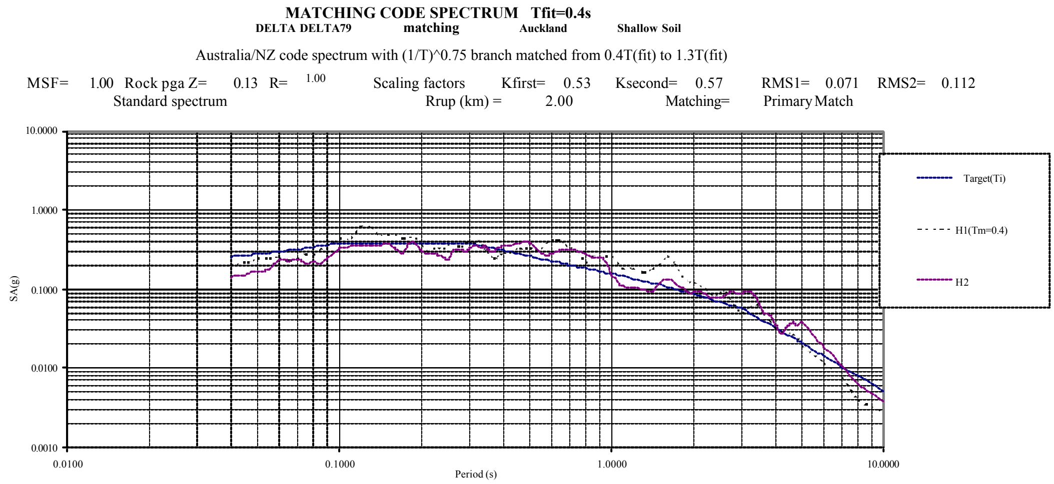
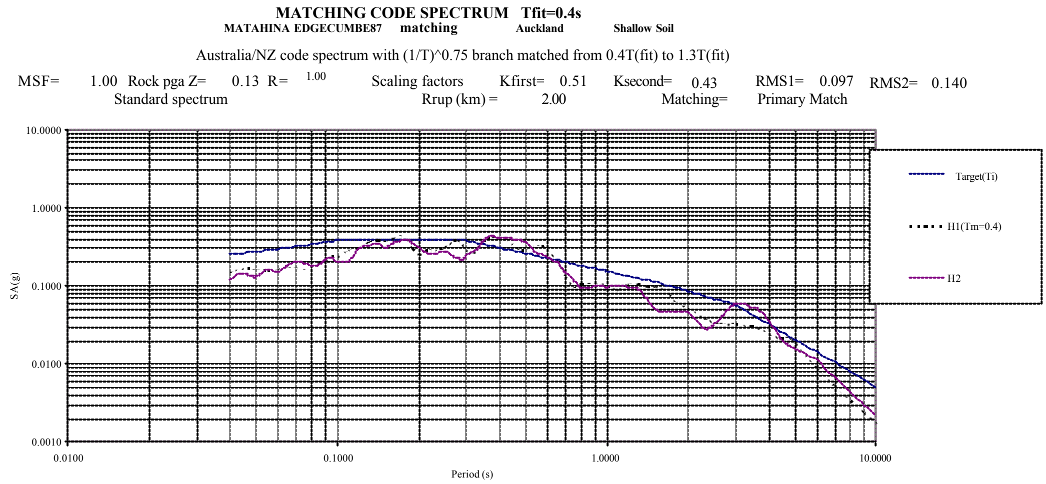


Figure 5.2.1 Deaggregated result for Auckland shallow soil site at a period of 0.4s. The contribution of background seismicity is important.





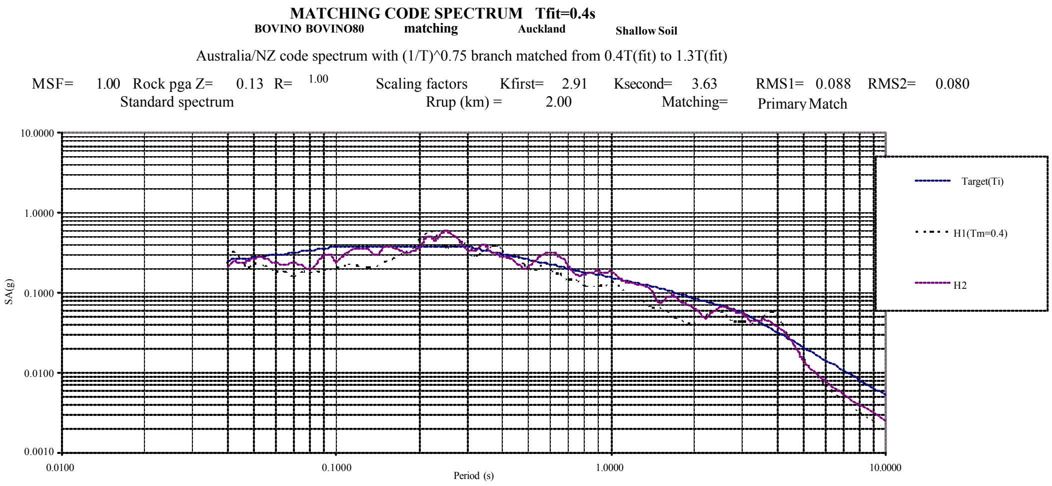
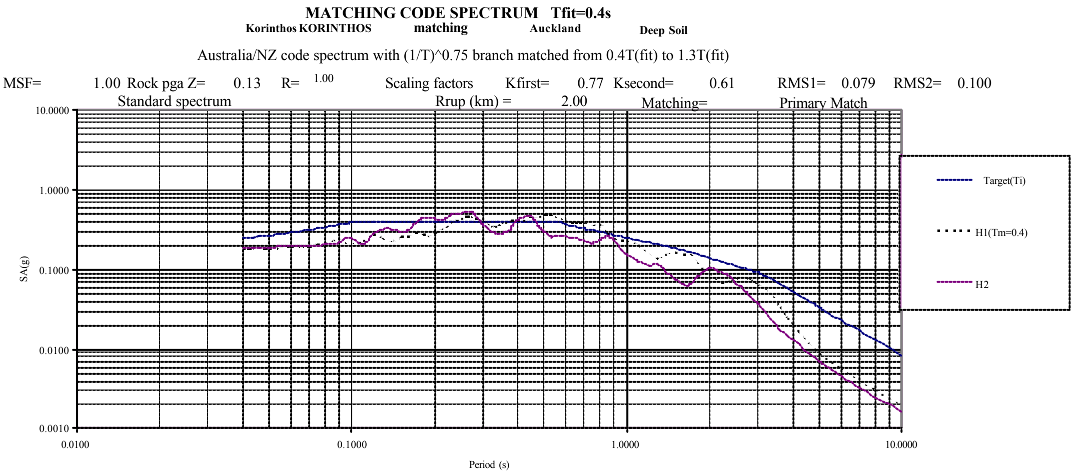
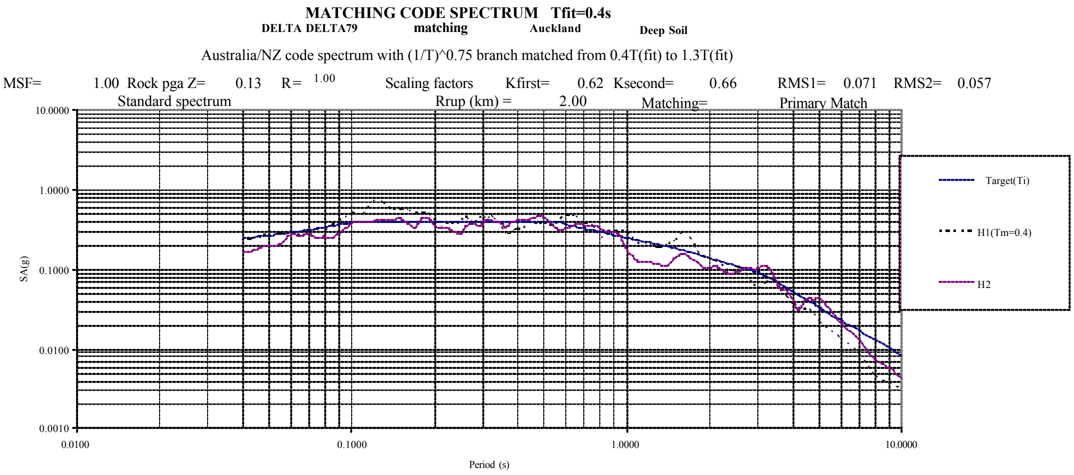


Figure 5.2.2 Matching the target spectrum at T_{fit}=0.4s for shallow soil site. Delta, Matahina, and Bovino earthquake records have been selected as the required accelerograms. K_l is represented by K_{first} and K_{second} for two horizontal components.





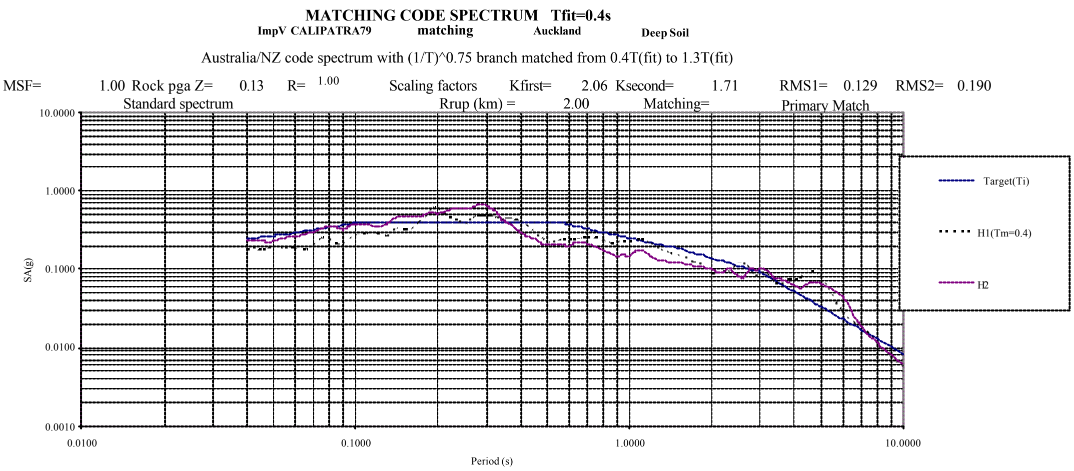
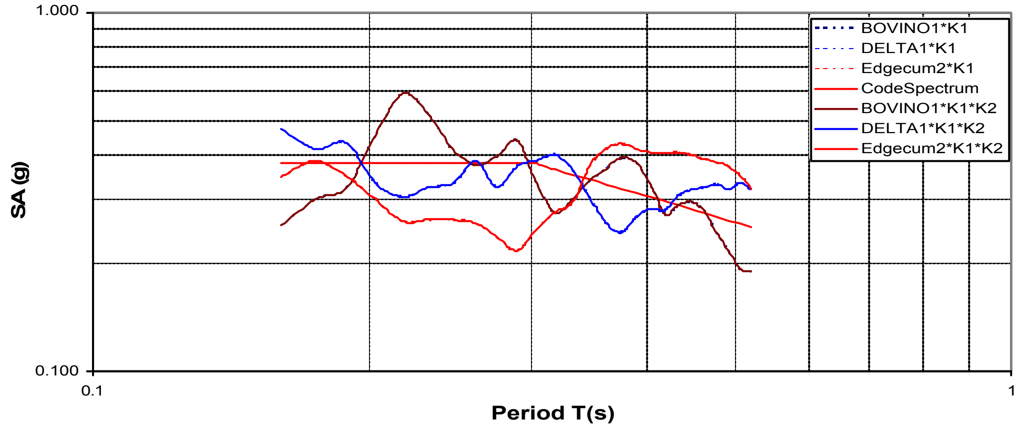


Figure 5.2.3 Matching the target spectrum at T_{fit}=0.4s for deep soil site. Korinthos-OTE, Delta, and Calipatra earthquake records have been selected as the required accelerograms. K_l is represented by K_{first} and K_{second} for two horizontal components.

Matching the target spectrum with spectra scaled by K_1 and K_2 for shallow soil site



Matching the target spectrum with spectra scaled by K and K for deep soil site

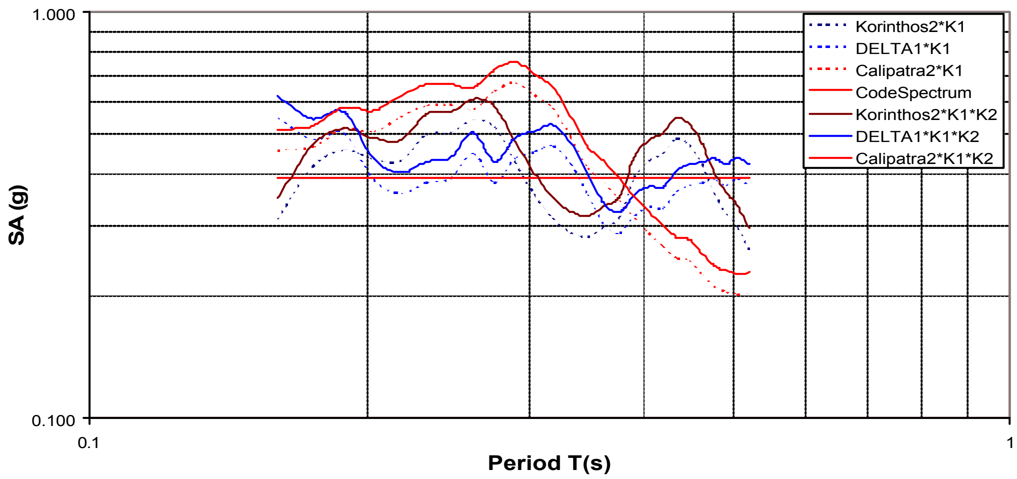


Figure 5.2.4 Comparisons between the target spectrum and spectra scaled by K_1 and K_2 for selected accelerograms at shallow and deep soil sites.

5.3 Selected Accelerograms and Scaling Factors K_1 and K_2 for Christchurch

The deaggregated results for 475-year return period are shown in Fig.5.3.1 for deep soil sites at a period of 0.4s and Table 5.3.1 lists earthquake source contributions from main active faults and distributed seismicity. The values of K_1 and K_2 associated with the selected accelerograms are listed in Table 3. Comparisons between the target spectrum and spectra scaled by K_1 are shown in Fig.5.3.2. Comparisons between the target spectrum and spectra scaled by K_1 and K_2 are shown in Fig.5.3.3. Table 3 shows that the selected scale factors K_1 and K_2 meets the conditions for the period range of interest. The selected accelerograms are described below:

The Boshrooyeh record was from in the 1978 Tabas reverse-slip earthquake with a 7.4 magnitude at a distance of 34km from the rupture plane on a deep soil site. In the present study, this record is considered as a representative of the Ashley fault and Omihi fault earthquakes which have a reverse faulting mechanism.

The Taft record is from the 1952 Kern County earthquake with a magnitude of 7.4 at a distance of 42km from the rupture plane on a alluvial deposit site. This record fits the target spectrum with K_1=1.56 and is a possible representative of the Ashley and Omihi fault earthquakes. This record is also a possible representative of the Alpine fault earthquake, although the focal mechanism of the Kern County earthquake is reverse slip. However, it is impossible to find records at present for a magnitude 8.0 strike-slip earthquake.

The Wrightwood accelerograms was recorded during the 1970 Lytle Creek earthquake with a magnitude of 5.3 at a distance of 13km from the rupture plane on a deep soil site. This record is a representative of the distributed seismicity. The selection of this record is because the contribution of distributed seismicity to Christchurch seismic hazards is high and this contribution should be included in at least one of the selected accelerograms.

Table 5.3.1 Earthquake source contribution for Christchurch deep soil site

Fault Name	Magnitude & distance from source	Contribution (%)
Ashley fault etc	M7.2@30km	14
Omihi fault etc	M6.7-7.5@40-60km	26
Alpine fault	M8.0@120km	14
distributed Seismicity (only short periods)	M5.0-6.5@0-40km	11

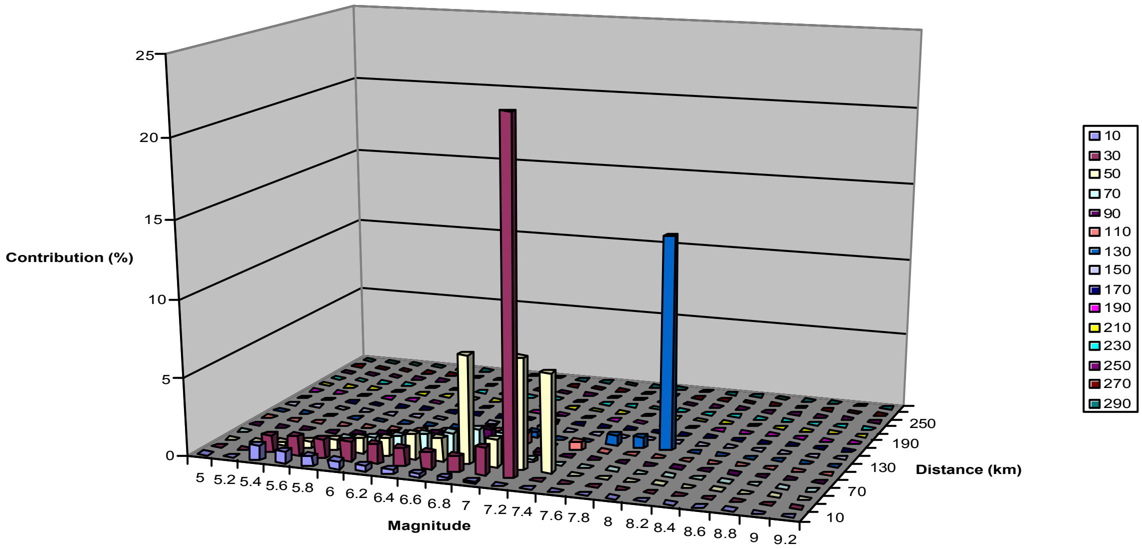
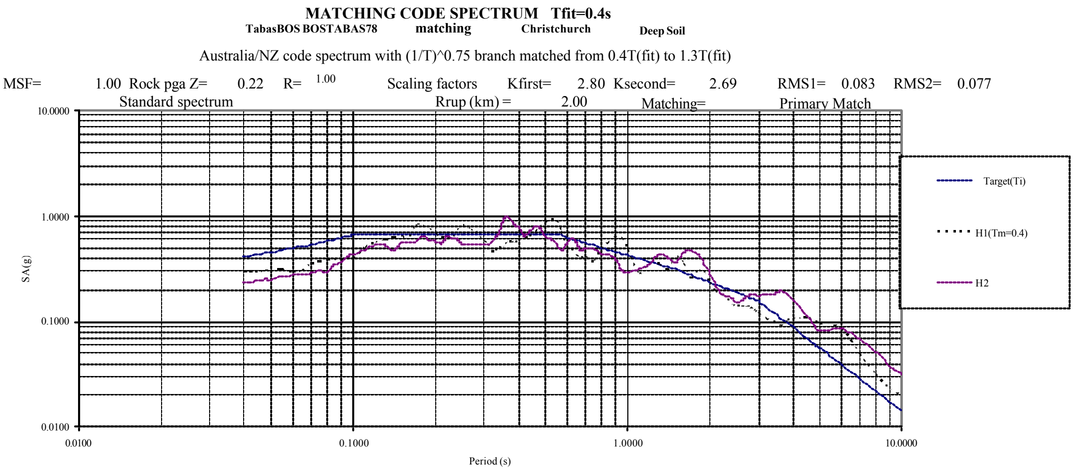
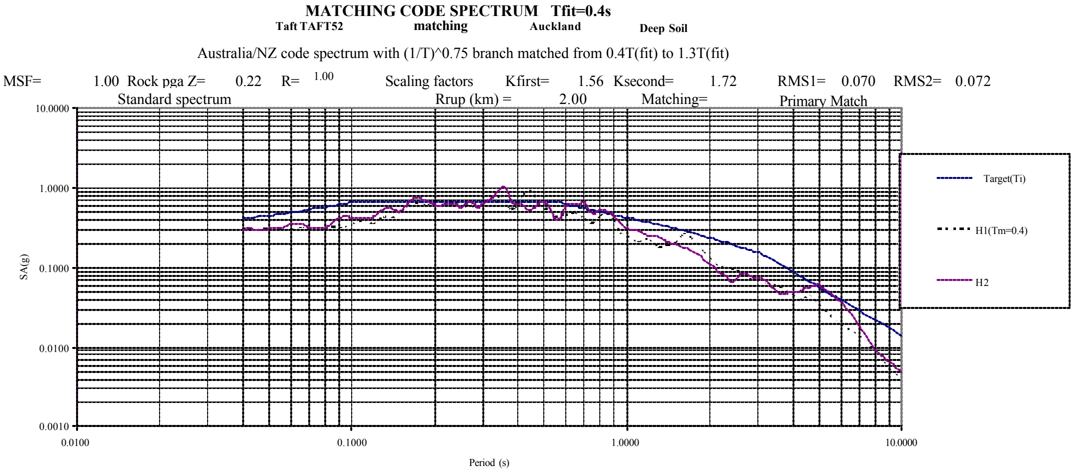


Figure 5.3.1 Deaggregated result for Christchurch deep soil site at a period of 0.4s.





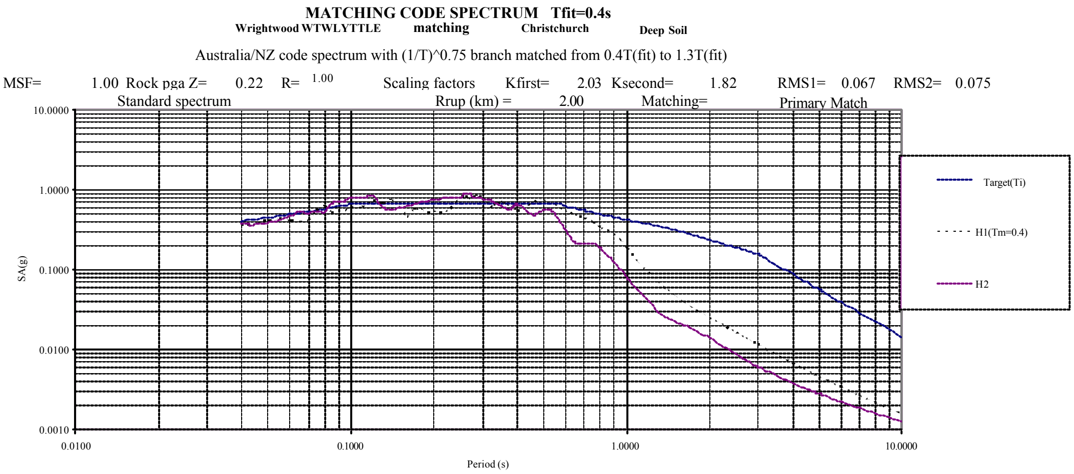


Figure 5.3.2 Matching the target spectrum at T_{fit}=0.4s for deep soil site. Boshrooyeh, Taft, and Wrightwood earthquake records have been selected as the required accelerograms. K_1 is represented by K_{first} and K_{second} for two horizontal components.

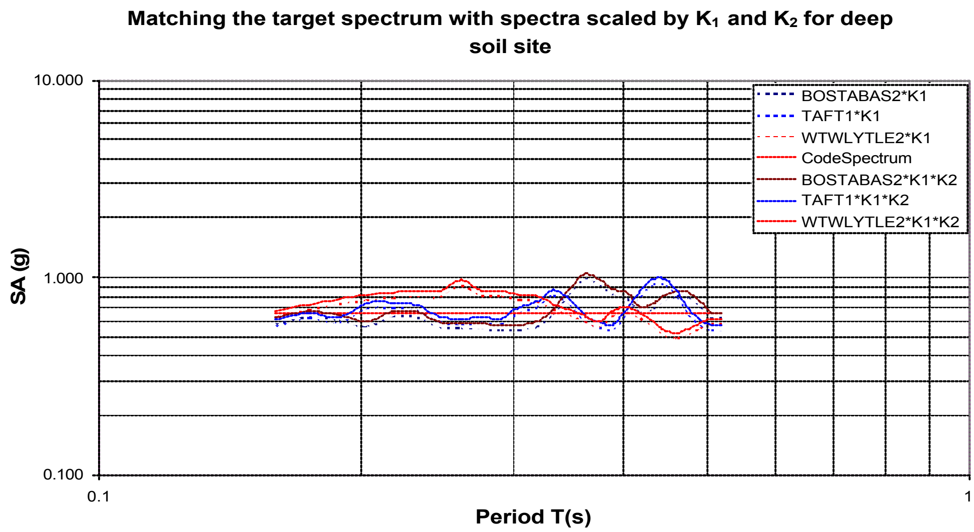


Figure 5.3.3 Comparison between the target spectrum and spectra scaled by K_1 and K_2 for selected accelerograms at deep soil site.

6. 0 SUMMARY

To select accelerograms appropriate for shallow and deep soil sites for Wellington, Auckland and Christchurch, probabilistic seismic hazard analyses for 475-year return period have been performed. Contributions from major faults and distributed seismicity have been derived based on the deaggregation analyses for hazard level from the 475-year return period. Deaggregation analyses show that seismic hazard level is dominated by the Wellington fault and the Wairarapa fault in Wellington, dominated by distributed seismicity, the Wairoa fault and the Kerepehi fault in Auckland. In Christchurch, seismic hazard level is dominated by the

Ashley fault, the Omihi fault and the Alpine fault, but for short period, the distributed seismicity makes a significant contribution.

The selection of accelerograms is based on earthquake magnitude, the shortest distance to the fault rupture plane, the focal mechanisms of the faults and site conditions. Sometimes distributed seismicity also makes a significant contribution to seismic hazard level. The selected accelerograms are a best possible representative of the above factors and the scale factors K_1 and K_2 associated with the selected accelerograms have met the conditions in the Loadings Standard.

The selected accelerograms are listed in Table 1 for Wellington, Table 2 for Auckland, and Table 3 for Christchurch, together with the associated scale factors K_1 and K_2. Comparisons between the target spectra and the spectra scaled by K_1 and K_2 show that the matching in the period range of 0.16s to 0.52s is reasonable. Note that some of the accelerograms are used for both shallow and deep soil sites.

7.0 REFERENCES

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Appendix B

EM3_V3

(an update of P21. A bracing wall test and evaluation procedure)

Revision note:

This is an adaptation of the current P21 (1988), and TR10 (1991), to incorporate current best practice and prepare for publication and citation in NZS 3604. NZS 3604 is currently under revision, and due for publication in late 2010.

1 Scope and use

This procedure is to be used to determine the dependable racking resistance of timber-framed walls sheathed with sheet material. Bracing ratings evaluated in accordance with this procedure are intended to provide bracing resistance compatible with the Timber Framed Building Standard, NZS 3604.

This procedure is not intended to be used for evaluation of the performance of concrete or masonry walls, steel-framed walls, or timber framed walls sheathed with boards, plank and post construction, or prefabricated or panellised construction.

A bracing panel fixed to a concrete floor slab may behave in a different manner to the same panel fixed to a timber floor. The bracing ratings derived are only applicable to the construction tested.

(Note: Refer to Section 14 for application of this procedure, which discusses these issues).

2 Test and evaluation objectives

The test is intended to evaluate the performance of wall bracing elements and their fixings when subjected to in-plane racking load. Such performance includes consideration of:

a) Adequate strength to withstand the maximum likely wind and earthquake loads.
b) Adequate stiffness to avoid excessive deflections.
c) Adequate elastic recovery after loading to prevent unacceptable permanent deflection.
d) Resistance to repeated loading, and demonstration of ductility and reserve of strength so that earthquake energy can be adequately dissipated.

The procedure has been developed so that, regardless of the form of construction, walls possessing a given number of bracing units of racking resistance will give equivalent performance in service.

3 Definitions

Bracing rating (earthquake)	The bracing rating derived from analysis of loads resisted by three test bracing walls as defined in Section 12.1, expressed in BU.
Bracing rating (wind)	The bracing rating derived from analysis of loads resisted by three test bracing walls as defined in Section 12.2, expressed in BU.
Bracing unit (BU)	A unit of force used to value the overall bracing performance of a panel tested to the P21 test method. By definition, 1 kN = 20 Bracing units. It is also used in NZS 3604 to express the magnitude of wind and earthquake bracing demand.
Cyclic displacement set	A set of three displacement cycles to a designated displacement, first in a positive direction, and then in a negative direction.
Notes	Notes within this document are informative only, and are not a mandatory part of the verification procedure.
Sheet lining	Wall lining material formed from large (typically 2.4 m x 1.2 m) sheets, fixed to the frame with discrete fixings (screws or nails).
Supplementary uplift restraints	Devices attached to each end of the specimen to provide a level of uplift restraint that can reasonably be anticipated in service.
Timber-framed wall	A wall built of spaced timber stick members and lined on one or both faces. Lateral resistance of a bracing wall is achieved through the in-plane resistance of sheet lining or cladding fixed to the face of the framing elements.

4 Notation

H	Overall height of test specimen (mm).
$\mathbf{P_{\Delta}, P_{8}, P_{15}, P_{22}, P_{29}, P_{36}}$	Average of the 'push' and 'pull' loads resisted at the denoted subscripted displacement. (kN) Defined in Section 12.
$\mathbf{R_{\Delta}, R_{8}, R_{15}, R_{22}, R_{29}, R_{36}}$	Average of the 'push' and 'pull' residual loads resisted at the denoted subscripted displacement. (kN) Defined in Section 12.
$\mathbf{F1_{\Delta}}$	Deflection dependent non-linearity factor, given in Table 1.
F2	Systems factor, allowing for additional construction not accounted for in a bracing calculation. Taken as 1.2 on the evidence of experimental studies.
$\mathbf{EQ_{\Delta}}$	Calculation parameter being the ultimate earthquake rating of a single wall at a target displacement, Δ. (kN) Defined by the equation in Section 12.1.
$\mathbf{BR_{EQ}}$	Single wall indicative earthquake rating. (BU)

Defined in Section 12.1.

BR_W Single wall indicative wind rating. (BU)
Defined in Section 12.2.

5 Principle

The bracing rating of a specified bracing wall system is determined by experimentally subjecting three full-scale walls to an incremental series of cyclic lateral displacements, and by ascertaining the load that the wall resists within a defined displacement range from measurements of the resisted force and wall deformations. The test arrangement is illustrated in Figure 1.

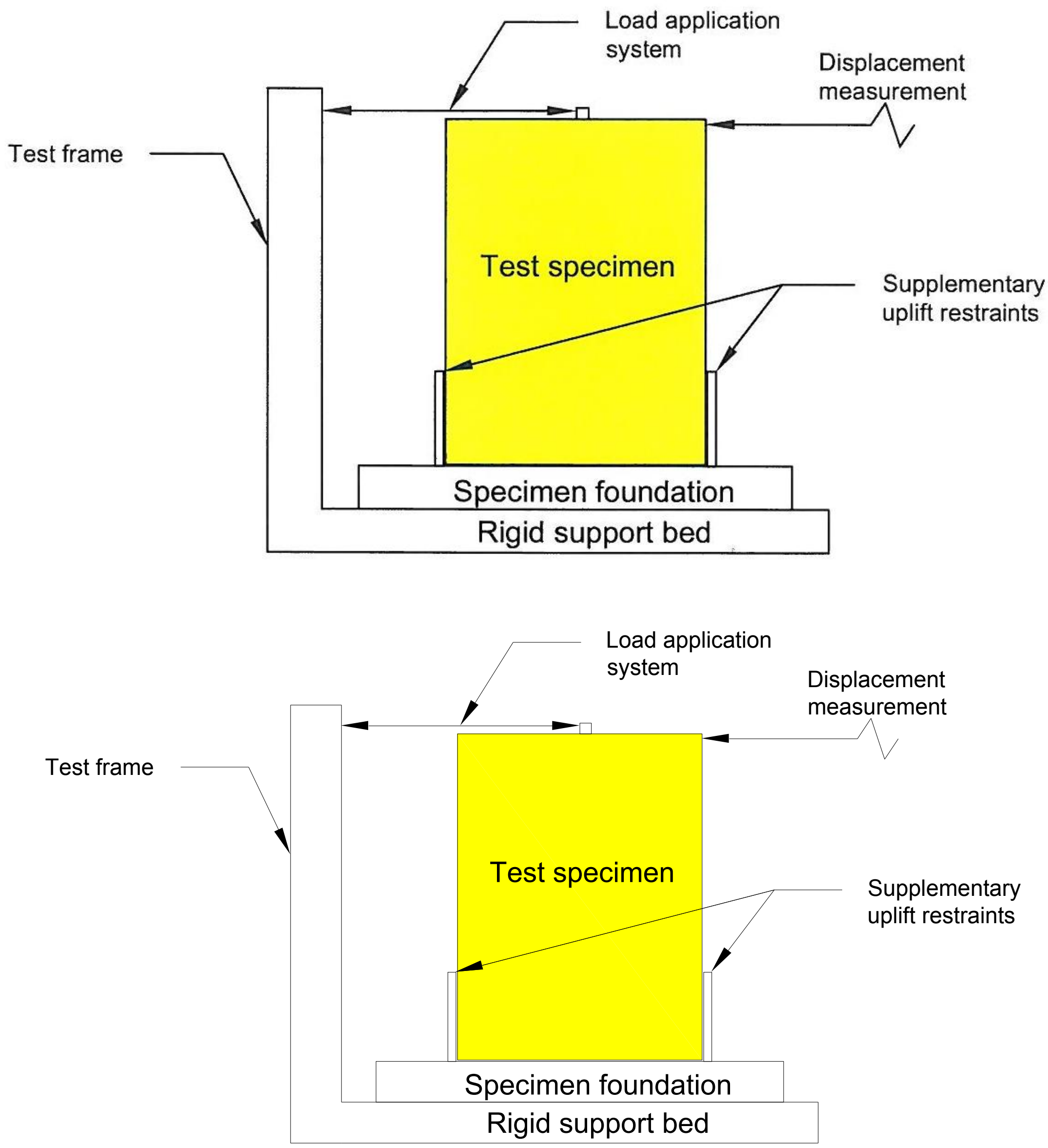

Figure 1. Test arrangement

This test procedure derives a bracing rating for both wind and earthquake as the lower of the rating derived from serviceability criteria and ultimate limit state criteria.

The serviceability criterion is a function of the average force resisted by the wall at a nominated top plate displacement. It is deemed that damage, commensurate with serviceability level loading, is satisfactory up to, but not exceeding, this displacement. The ultimate limit state resistance to earthquake action is the maximum force resisted during the third cycle of a displacement series within the defined deformation range, factored by F1 and F2. The ultimate limit state resistance to wind action is the maximum force resisted within the defined deformation range.

The approach has been taken to evaluate wall bracing performance within a defined deformation range to allow racking resistances of different wall types and lengths within a building to be added.

Vertical loads and vertical restraint from the surrounding structure improve the racking stiffness of a section of wall by reducing its rotation about its base. The amount of improvement for a given degree of vertical loading is related to the stiffness and strength of the wall-to-floor attachments in relation to the shear stiffness of the complete wall section. Provision is made in the test method for testing the wall under vertical end restraint representative of the restraint provided by the adjacent construction in a building.

6 Test apparatus and set up

To allow unrestricted observation of the wall while the test is in progress, to permit dead loading of the top edge of the wall when required, to allow for the effects of the dead weight of the wall itself, and to minimise unwanted frictional effects, the wall must be tested in the vertical position.

The following apparatus is required to conduct this test:

6.1 Test frame

a) A test frame of sufficient size that full scale specimens with a minimum height of 2.4 m and length equal to that dictated by the test specification, together with ancillary supports (eg foundation beams etc) can be installed.
b) The frame shall be built in a manner such that it does not provide artificial restraint during the load cycles, and does not restrain the specimen in any way such that higher bracing values may result.
c) The frame shall include a rigid support bed that allows the wall to be fixed at the base according to its specification and using supplementary uplift restraints if appropriate (see Section 10.4). The support bed shall be sufficiently rigid that the attachment points to any uplift restraints do not displace any more than 0.1 mm at any stage during the test.
d) The frame shall have provision for a lateral restraint mechanism to prevent significant out-of-plane distortion along the top of the specimen. The mechanism used to control out-of-plane distortion of the specimen as it is laterally displaced should be located at each end of the top of the specimen. The mechanism shall not restrain the specimen from in-plane movement, but shall be sufficiently rigid that it will not permit the specimen to distort more than 5 mm out-of-plane over the expected load range.

6.2 Load application system

e) An actuator of sufficient capacity and travel to fail the specimen under the intended action at the prescribed loading rate. It shall be controlled (manually or automatically) so as to follow the nominated test sequences at the prescribed loading rate.

f) The load application system must only impose horizontal in-plane load to the specimen top plate and not restrict any other specimen deformation.

(Note: It is recommended that the loading system consist of a hydraulic actuator, load cell(s) and a connector having a universal joint at the actuator end, and a horizontal pin at the specimen end. Thus, the wall may lift (rock) without restraint and have limited lateral movement without imposing shear forces across the load cell).

6.3 Measurement system

g) The applied load shall be monitored by a load cell calibrated to Class 1 in accordance with International Standard EN ISO 7500-1 over the load range 0.5 to 1.1 times the peak load resisted by the test specimen.

(Note: It is recommended that the applied load be monitored by two independent systems, such as two load cells in series. Checks should be made to ensure the forces registered by the independent systems are within 0.3 kN at all stages of the test).

h) Measurements of in-plane deflection shall be made at the top of the specimen. The method of measurement shall be such that it will not be affected by either vertical movement at the point of measurement, or by test rig deflections under load. The displacement measurements shall be accurate to within 2 mm at all stages of the test.

i) Horizontal slip of the bottom of the wall relative to the foundation shall be monitored during the test, so that it can be allowed for if it occurs.

6.4 Recording system

j) Load and deflection measurements shall be recorded so that a complete plot of load verses deformation (as prescribed in Section 11.2) may be made. Deflections at zero load and loads at zero deflection shall also be recorded.

(Note: Automatic direct plotting of load verses deflection is desirable.)

7 Sampling

A minimum of three specimens, built to the same specification, shall be tested.

Timber framing members shall be of a grade, and shall have dimensions that are representative of that to which the rating is to be applied. Sheathing materials and fasteners supplied for testing shall be representative of those being manufactured.

8 Test specimens

The specimens shall as far as possible be representative of the minimum specified construction with respect to dimensions, material and fasteners. In particular, the specimen length shall be the length for which the performance rating is sought.

8.1 *Bracing system specification*

The system proprietor shall provide a detailed specification for the bracing system to be tested. This shall include the following information:

a) The specification for the frame construction, which should normally be in accordance with NZS 3604. Unless otherwise specified as part of the bracing system, the following frame construction details shall be used to ensure consistent results between laboratories. Timber framing shall be kiln dried Radiata pine, graded MSG8 or VSG8 at a maximum moisture content of 18 % (verification by meter is acceptable). Framing members shall be the maximum spacing and minimum cross section that may be used in actual construction for the scope of application. The plates shall be nailed to the studs with two 90 x 3.15 power driven nails. No dwangs (nogs) shall be used unless required by the system specification.
b) The sheathing parameters, including the sheet thickness, density and orientation, the face(s) of the frame onto which the sheathing is to be installed.
c) The fastener type (including base material, head and shank diameters, shank length and surface finish), the fastener spacing around the edges of the specimen and within the body of the specimen, and the fastener locations relative to the corners of the sheathing.

(Note: Lining shall not be glued to the framing unless the adhesive has been assessed as having a durability in this application of greater than 50 years.)

d) The specification of any jointing system that provides continuity between sheets. Where appropriate, this specification shall include the method of stopping, the stopping compound(s) to be used, the application method and the joint reinforcement (if any).
e) Full details of any uplift restraints that are part of the bracing system.
f) Specification of the fixing to timber or concrete floors where different to the provisions of NZS 3604.

These details are to be included in the test report to the extent that the bracing wall to be tested can be clearly identified and replicated.

8.2 *Specimen construction*

Each specimen shall be built in accordance with the system specification and the manufacturer's instructions. An average standard of workmanship should be achieved.

Where the test specimen is simulating continuous construction, the specimen length shall be based on the nominal sheet size/length plus the thickness of the stud. Otherwise the specimen length shall be equal to the sheet size/length.

(Note: This enables the test to simulate the effect of butt jointing lining sheets over a single stud. The "rated" length will be the nominal length, exclusive of the additional stud thickness.)

Where the specimen length is equal to or less than half the manufactured width of the sheet, the three test specimens shall be constructed with the following edge arrangements (in the orientation as listed with respect to the position of the actuator):

- Manufactured edge/cut edge
- Cut edge/cut edge

- Cut edge/manufactured edge.

Where the specimen length includes a sheet greater than half the manufactured width, the three test specimens shall be constructed with the following edge arrangements:

- Cut edge adjacent to the actuator
- Cut edge at the opposite end
- Cut edge in the position giving the lowest result from the previous two cases.

Fixings shall be installed to a similar depth/tightness as will occur on site. For example plasterboard fasteners must be sufficiently embedded below the sheet surface to allow stopping to take place. Holding down bolts/screws shall not be tightened to a greater extent than expected on site.

Dimensional tolerances shall be consistent with those resulting from good trade practice (this is satisfied as being ±5 mm for any nominated dimension except for ±2 mm for fastener edge distance).

9 Conditioning

The specimens shall be constructed and tested in a condition representative of the anticipated construction and in-service situation, e.g., frames assembled with green timber shall be allowed to dry to the anticipated in-service moisture content (18 % maximum) before testing. Temperature and humidity conditions during construction and test shall be recorded by the testing agency, but need not be reported in detail.

(Note: Materials may be conditioned to 20^{0}C and 65% relative humidity prior to testing)

10 Test arrangement

10.1 Specimen foundation

For the simulation of a timber floor system, the specimen foundation shall consist of a timber beam, at least 90 mm wide and 90 mm deep, with 20 mm particle board flooring strip attached to the top face, with pairs of 60 mm flooring nails at 200 mm centres.

For the simulation of a concrete floor system, the specimen foundation may be either the same timber beam as above, or alternatively, a concrete beam shall be used.

The specimen foundation shall be securely fixed to the specimen support bed in order that the effects of bending flexibility of these members are eliminated as much as practicable.

10.2 Specimen installation

The method of fixing the base of the specimen to the foundation shall be consistent with that expected to be encountered in service for the intended scope of application.

In a test simulating timber floor construction, the bottom plate shall be fixed to the foundation by three 90 x 3.15 power driven nails at 600 mm centres, commencing 50 mm inside the end studs. Alternatively, the proprietor's specification (see Section 8.1) shall be used. This must form part of the construction specification.

In a test simulating concrete floor construction, the bottom plate shall be fixed as specified by the test proprietor. However the system specification must comply with NZS 3604, including the maximum spacing of bottom plate anchors, and the minimum shear and uplift capacity of anchors as required by Section 7.5.12.4.

If a timber foundation is used to simulate a concrete slab, then the anchors fixing the bottom plate to the concrete foundation in actual construction shall be simulated by through-bolts or coach screws in the bracing test.

Square washers, 50 x 50 x 3 mm, shall be used between the anchor heads/nuts and the timber.

(Note: It is recommended that the lining sheet (s) be fixed to the bottom plate only after the bottom plate itself is fixed to the specimen foundation. This is the sequence which will be used in normal construction practice, and prevents potential damage to the sheet caused by the fixings if movement of the bottom plate occurs when fixing it to the specimen foundation.)

10.3 Load application

The attachment of the load application system to the specimen shall be located at mid-length of the specimen to minimise any tilt effects that can occur if the wall rocks under load.

10.4 Supplementary uplift restraints

A supplementary uplift restraint may be used at each end of the test specimen, as shown in Figure 1, if the testing laboratory considers it appropriate. Otherwise no supplementary uplift restraint shall be used.

Construction details of the restraint are shown in Figure 2. The nails to be used in the restraint are 90 x 3.15 power driven nails. The timber block is to be from the same timber (species and grade) as the frame.

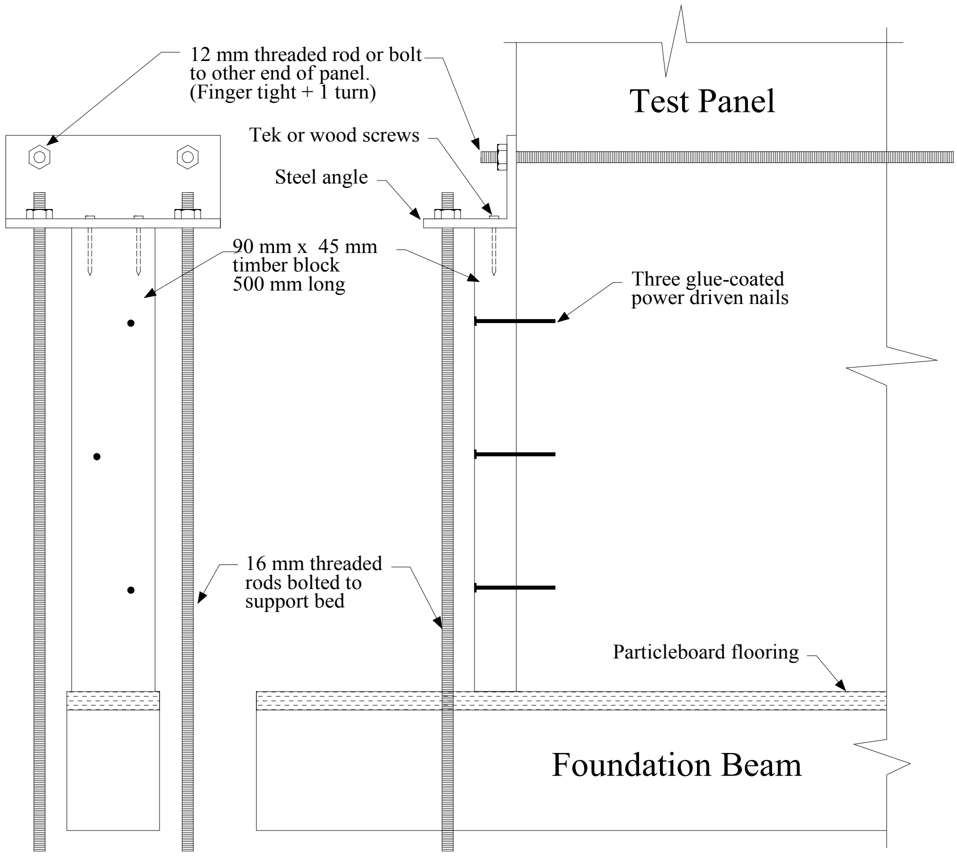

Figure 2. Supplementary uplift restraint

11 Test procedure

11.1 Cyclic test loading protocol

Each specimen shall be subjected to three cycles of in-plane displacement at top plate level to each of ±(H/300 + 1) mm, ±15 mm, ±22 mm, ±29 mm, ±36 mm, and ±43 mm as illustrated in Figure 3, (where H is the height of the specimen). Target displacements are to be met within a tolerance of ±2 mm, measured between top plate and the base of the specimen.

(Note: the specified target top plate displacement of H/300 + 1, is to ensure that data will always be available for evaluation at H/300. For example, see 6.3. i)

Where the bracing element is unsymmetrical, (for example, see Section 8.2), the first two specimens shall be displaced in the "weak" direction first, and the "strong" direction first respectively, and the 3rd specimen in the direction giving the lowest result in the first two tests.

The rate of applied displacement shall be within the range 1 to 5 mm per second.

(Note: if sinusoidal displacement is used, then the average rate shall be in the range 3 to 4 mm per second)

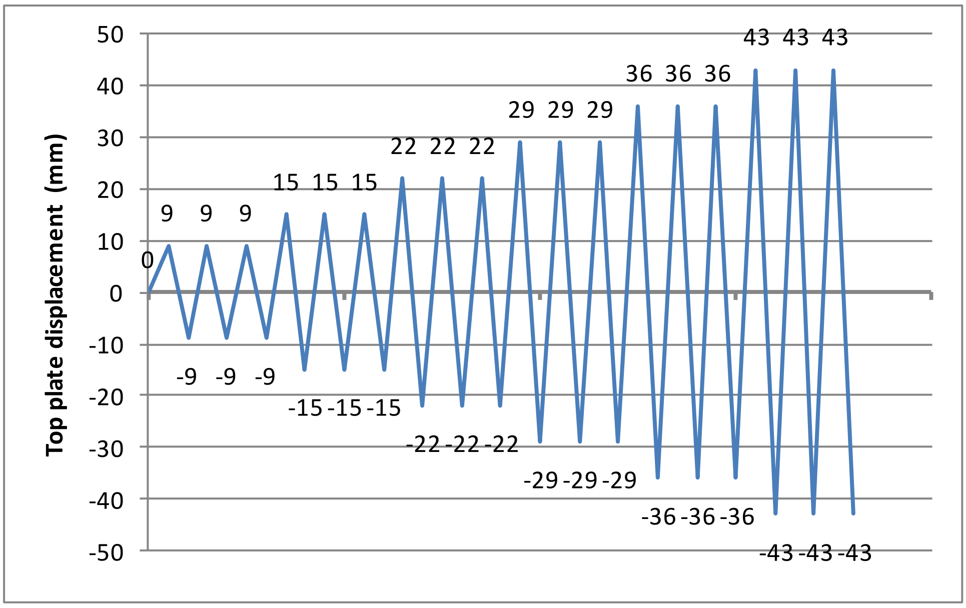

Figure 3. Cyclic load protocol. (example shown for a 2,400 mm high specimen)

11.2 Data recording

During the test, measure and record the following data:

a) The force applied to the specimen, and the relative displacement between top and base of the specimen, sampling at a rate of at least three readings per second.

b) Observations of the condition of sheet and fasteners which attach the sheathing to the frame.

c) Record the dominant mode of deformation and any observed distress or damage in the test specimen.

d) A description of the mode of failure experienced by the specimen.

12 Results evaluation

At each target top plate displacement in the "push" direction, Δ (= +8 mm, +15 mm, +22 mm, +29 mm and +36 mm), determine the loads recorded on the first cycle as $+P_{\Delta}$, and on the third cycle as $+R_{\Delta}$, as illustrated in Figure 4. Do the same for the "pull" direction as $-P_{\Delta}$, and $-R_{\Delta}$ respectively. Where the value of the load in one direction is more than 20% greater than the load in the other, assign it a value of 1.20 times the lower value.

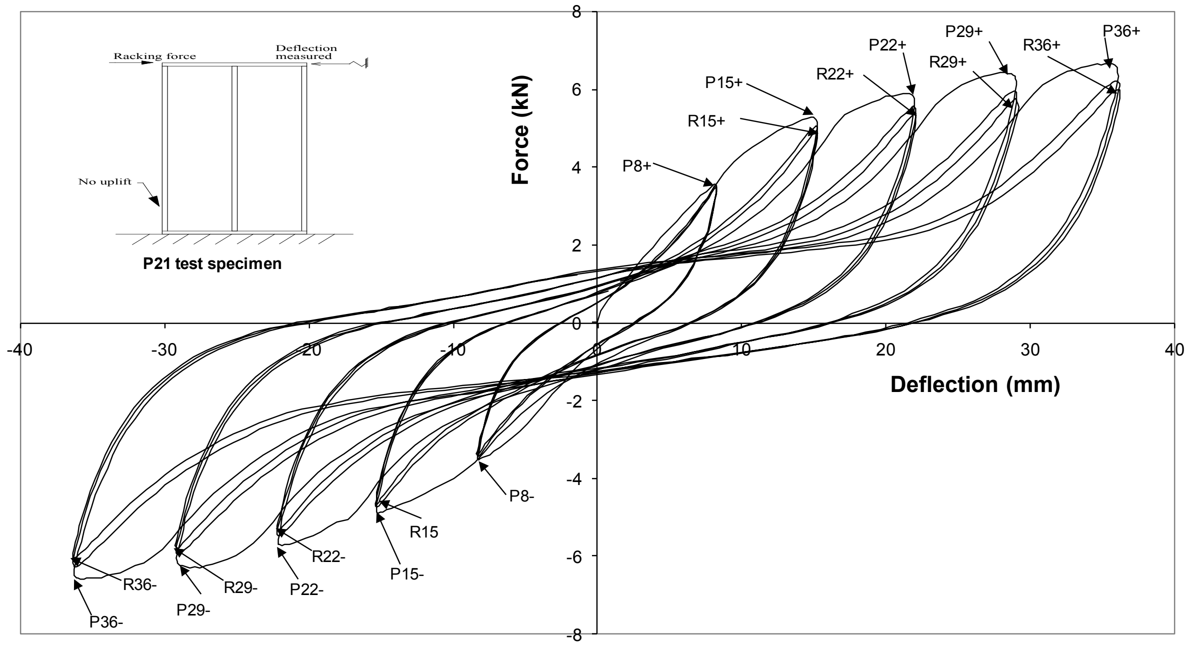

Figure 4. Load measurements

Average the absolute values of these loads for each of the displacements to give P_{Δ} for the first cycles, and R_{Δ} for the third cycles.

12.1 Earthquake rating

For each target displacement Δ, calculate $F1_{\Delta}$ from Table 1. Type 1 factors shall be used for walls clad with paper faced gypsum plasterboard with continuity at the joints. Type 2 factors shall be used for all other systems, including combination systems. If the contribution of calculated wall rocking displacement exceeds 30% of Δ actual, then Type 2 factors from Table 1 shall be used.

(Note: Refer to SR220 for derivation of these factors)

If other target displacements are used, values of $F1_{\Delta}$ may be obtained by linear interpolation.

Wall displacement Δ (mm)	$F1_{\Delta}$ factors Wall sheathing Type 1	Type 2
15	0.390	0.415
22	0.534	0.466
29	0.678	0.516
36	0.822	0.566

Table 1. F1 factors.

For the four target displacements Δ = 15, 22, 29, 36 mm, calculate $EQ_{\Delta} = F1_{\Delta} \times F2 \times R_{\Delta}$. Take F2 = 1.2.

Compute the earthquake bracing rating BR_{EQ} for each test specimen, in units of BU's, as being 20 times the lesser of the following two values:

- Maximum value of EQ_{15}, EQ_{22}, EQ_{29} and EQ_{36}
- $P_8/0.463$.

The earthquake bracing rating for the wall system/element is the minimum value calculated from the three replicate test specimens.

(Note: for the derivation of the value 0.463, and the value of 1.2 to be used for the systems factor F2, refer to SR 220)

12.2 Wind rating

Compute the wind bracing rating BR_W for each test specimen, in units of BU's, as being the lesser of the following two values:

- Maximum value of P_{15}, P_{22}, P_{29} and P_{36}
- $P_8/0.563$.

The wind bracing rating for the wall system/element is the minimum value calculated from the three replicate test specimens.

(Note: for the derivation of the value 0.563, refer to SR 220)

13 Reporting

The report shall contain the following information:

1. The name of the testing agency performing the tests.
2. The name of the person responsible for the test.
3. The location and dates over which the testing was undertaken.
4. Full details of the construction specification used, including any limitations that may be placed on the scope of application that may apply to the bracing system. Include full details of the installation specification used.
5. Framing timber type, grade and moisture content.
6. Sheathing type (eg plasterboard, plywood etc), thickness and density (if applicable).
7. Sheathing fixings, (type, material, shank and head diameter) fixing pattern and edge distances.
8. Curing time for jointing compound (if applicable).
9. Details of the fixing of the bottom plate to the foundation.
10. Details of the means by which uplift of the ends of the specimen is controlled.
11. Description of the mode of failure.
12. Plot of applied load against top plate horizontal displacement.
13. Tabulation of values of R_8, R_{15}, R_{22}, R_{29}, R_{36}, P_8, P_{15}, P_{22}, P_{29} and P_{36}.
14. Calculation sheet showing derivation of bracing values for earthquake and wind rating.
15. Photographs and drawings.

(Note: In general, the report should include sufficient detail to enable a third party to duplicate the construction, the testing and produce similar results)

14 Application of Bracing ratings

This procedure produces bracing ratings intended to be compatible with NZS 3604, which applies only to modest sized timber buildings with well distributed structural elements. As a result the derived ratings are not based on characteristic or "dependable" values as used with other material standards (for example NZS 3603, NZS 3404 etc), and the F2 systems factor assumes "typically" distributed bracing elements. When bracing ratings are used for the specific structural design of a building outside the scope of NZS 3604, equating ratings with loads derived from AS/NZS 1170 together with bracing elements of other materials, engineer designers should be aware of these crucial differences, and should make appropriate allowance for the resistance of critical structural elements.

This procedure produces bracing ratings for stick framed timber walls with sheet lining, as stated in Section 1. Testing agencies wishing to use it for other bracing systems should be aware that the F1 non-linearity factors were derived for these types of systems only. Other bracing systems (for example masonry, steel framed walls etc) will have different hysteretic behaviour, thus invalidating the basis of the F1 factors. To use this procedure for these types of systems would require a derivation process similar to that described in BRANZ SR220 to arrive at suitable values of F1 factors.

The bracing ratings evaluated using this test procedure apply only to bracing walls built in accordance with the test system specifications. Changes to any of the parameters listed in Section 8.1 would render the rating invalid. Therefore, publication of bracing data based on this procedure must give full details of all these parameters, including fixing to the floor (concrete or timber as appropriate).

Bracing ratings using this procedure are intended to be constructed in buildings within the scope of NZS 3604. Systems producing high ratings will require resistance to hold down reactions that may not be able to be provided by a typical timber framed buildings. For this reason ratings above 110 BU/m for timber floors or 150 BU/m for concrete floors should be published with caution. Refer to Clause xxxx of NZS 3604.

(Note: *this is written before the first public comment draft of the revised document is available, so no clause reference is included).*

Ratings derived using this evaluation method may be used for a bracing system of length within +100% of the tested specimen length. Beyond this, the system behaviour and failure mode is likely to be sufficiently different that a separate test is required.