



STUDY REPORT

No. 105 (2002)

Torsional Response of House Subfloor Systems

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The work reported here was jointly funded by the Building Research Levy and the Foundation for Research, Science and Technology from the Public Good Science Fund.



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ISSN: 0113:3675

Preface

This study forms the third phase of an investigation into the wind and earthquake racking resistance of timber piles used under New Zealand houses.

The first phase (see BRANZ Study Report No. 46) investigated the basis for the design values specified in the New Zealand Standard NZS 3604(1999) and also included some laboratory testing. The second phase (see BRANZ Study Report No. 58) reported on site measurements of the lateral strengths of piles when cast into the ground. Time history computer analytical simulation was used to determine earthquake static design loads from the averaged measured pinched hysteresis loops.

This third phase reports on supplementary tests from lateral loading of house bearer to foundation walls connections. It also describes a 'pushover' seismic analysis of the foundations of 20 conceptual buildings founded on piles and foundation walls. These included the more extreme eccentric situations which could be considered acceptable by NZS 3604:1999. Recommendations are made for suitable provision in NZS 3604:1999 to mitigate against torsion and on the design strength of lateral loads transmitted to foundation walls.

In the fourth and final phase of this project, the strength and stiffness of braced piles is investigated by both testing and theoretical means. This will be reported separately.

Acknowledgments

This work was jointly funded by the Building Research Levy and the Foundation for Research, Science and Technology from the Public Good Fund.

The author thanks Carter Holt Harvey for supply of timber used in this programme.

Readership

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

TORSIONAL RESPONSE OF HOUSE SUBFLOOR SYSTEMS

BRANZ Study Report SR 105 (2001)

S. J. Thurston

REFERENCE

Thurston, S.J. 2001. Torsional Response of House Subfloor Systems. Building Research Association of New Zealand. Study Report No. 105. Judgeford, Wellington.

ABSTRACT

NZS 3604:1999 stipulates the layout of bracing piles in a house piled foundation. This can still result in significantly non-symmetrical layouts. This paper describes a 'pushover' seismic analysis of the foundations of 20 conceptual buildings founded on piles and foundation walls. These included the more extreme eccentric situations which could be considered acceptable by NZS 3604:1999. Recommendations are made for suitable provisions in NZS 3604:1999 to mitigate against torsion and on suitable design strength for lateral loads transmitted to foundation walls.

This report presents test results from lateral loading of house bearers to foundation walls connections. It also discusses a field survey and test results to examine the maximum deflections a house founded on piles can sustain before rupture of building services occur.

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1. INTRODUCTION

This report presents test results from laterally loading house bearers to foundation wall connections. NZS 3604 (SNZ, 1999) stipulates the layout of bracing piles in a house piled foundation. This can still result in significantly non-symmetrical layouts. This paper describes a 'pushover' seismic analysis of the foundations of 20 conceptual buildings founded on combinations of piles and foundation walls. These included the more extreme eccentric situations which comply with the requirements of NZS 3604:1999. Recommendations are made for suitable provision in NZS 3604:1999 to mitigate against torsion and on variations to allowable lateral loads transmitted to foundation walls.

The report also discusses a field survey and test results to examine the maximum deflections a house founded on piles could sustain before rupture of building services would occur.

2. EARTHQUAKE RESPONSE OF PILED FOUNDATIONS

2.1 Earthquake Performance of New Zealand Houses

Although New Zealand has experienced uncommonly few damaging earthquakes in the last 100 years, timber framed-houses have generally performed well in these events. Earthquake structural damage has generally been limited to collapse of unreinforced brick chimneys (and to a lesser extent brick veneers), failure of piled foundations and, in a few instances, racking failure of lower storey walls. Wind damage to piled foundations is uncommon. When damaged in earthquakes, piled foundations have either failed at the connection between the top of the piles and the house or have developed a side-sway mechanism within the sub-floor with the piles rotating over until the house 'sat on the ground'. These issues were partially addressed in the 1978 edition of the non-specific light timber framed Standard NZS 3604 (SANZ, 1978). Engineering principles were more rigorously applied in the 1990 edition (SNZ, 1990). However, as no major earthquake has occurred in densely populated areas since the 1931 Napier earthquake, modern construction has not been tested to its nominated design strength.

Most new houses in New Zealand are designed and built in accordance with the New Zealand Standard for Timber Framed Buildings, NZS 3604:1999. This Standard specifies the earthquake and wind lateral load resistance required for all lateral load resisting elements, including house foundation systems, as a function of the house geometry and weight. The Standard also provides design strengths for various foundation configurations, e.g. the earthquake strength of an anchor pile is given as 120 bracing units (BUs) (i.e. 6 kN). A house designer must provide a foundation system such that the sum of the foundation element strengths equals or exceeds the lateral forces specified by the standard. There are also some requirements for the distribution of bracing elements.

2.2 Principles of Foundation Design

2.2.1 BRANZ design philosophy

NZS 3604:1999 uses an engineering rationale for the complete design process, from determination of design loads through to provision of load resisting elements with a traceable load path from the loaded element through to the foundations. It has long been acknowledged, however, that timber-framed buildings have greater resilience than expected from the application of strict engineering principles. Such reserve strength comes from the holistic response of the complete system which calls upon load sharing and composite action of both the structural and non-structural elements under lateral loading. This enhances the overall building performance. However, whilst true for the part of the house above floor level, piled foundation systems are unlikely to have significant reserve strength due to the lack of redundant elements.

Under serviceability level lateral loading, clause B1 of the New Zealand Building Code (NZBC) (BIA, 2001) requires that foundations should remain undamaged and have negligible permanent offset after the event. However, under design ultimate events it is acceptable for significant damage to occur provided collapse is avoided. It is then likely that the damage can be repaired quickly and at moderate cost. BRANZ design philosophy is to limit house deflections at ground floor level so damage to building services will be minor - particularly the sewerage system.

During an ultimate limit state event, the preferred failure mechanism should be ductile so that, although damage may occur, no abrupt collapse results. Yielding of the soil or ductile yielding of the connection between pile and house is ideal in this regard. Conversely, fracture of the pile or sudden rupture of the connection is undesirable and should be suppressed.

The horizontal forces induced in a house during an earthquake are a function of the stiffness of the house and the amount of yield before failure (once house lateral strength is reached). A structural ductility of 3 is assumed as the basis of the NZS 3604:1999 loading tables. The resisting systems must attain this level of post-elastic deformation to ensure no brittle failures occur.

2.2.2 Connection between house and top of piles

NZS 3604:1999 rates braced and anchor piles as having an ultimate lateral resistance of 6 kN. However, it requires the connection between the floor joist and bearer and also between the bearer and pile to have a strength of 12 kN per pile in the bracing direction. This is because the connector may be stiff up to 12 kN strength and then immediately rupture. The reserve between 6 kN and 12 kN is intended to prevent this undesirable brittle failure.

Thurston (1993) published a test procedure and related performance evaluation procedure for testing foundation system fixing devices. This includes an evaluation procedure which allows the use of connectors of less than 12 kN strength provided they exhibit adequate ductility when tested.

2.2.3 Soil strength

Pile foundation design to NZS 3604:1999 is only applicable for soil which has at least a (vertical) bearing capacity of 300 kPa below the underside of the pile. This can be determined by either general observation of the performance of nearby buildings or by performing Scala Penetrometer soundings at depths below the pile founding depth. No limitation is placed on soil strength over the depth of the pile - yet clearly it is this upper soil zone which determines pile lateral resistance.

2.2.4 Testing and derivation of design lateral strengths

(a) Round deep driven cantilever piles

Cocks et al. (1975) tested 1.8 m piles driven 1.2 m deep into silty clay. The average load per pile at 20 mm pile deflection, at three sites of soil bearing strength close to the minimum of 300 kPa allowed by NZS 3604:1999 was 8.2 kN, 6.5 kN and 5.4 kN. This is well in excess of the seismic design value of 1.5 kN (30 BUs) given in NZS 3604:1999.

(b) Tests of bolted connections

The strength of braced piles can be limited by the strength of the bolted connection between the brace and pile. Tests indicated the maximum lateral load per brace set at an angle of 45° was 12 kN (Thurston, 1993).

(c) *Tests at BRANZ*

Thurston (1996) tested a series of anchor and braced piles under simulated earthquake loading. Some were founded in minimum strength clay soils and others in loose sand. Earthquake design loads were directly determined by time history computer analysis using a synthetic ground motion with similar spatial characteristics to the design spectra in NZS 4203:1992 and elemental resistance derived from the test data. The results of this work were the basis of the enhanced bracing ratings for pile systems given in NZS 3604:1999.

2.2.5 Potential for pile flexural failure

It is important to avoid brittle failure of the piles themselves. The draft Timber Pile and Pole Standard (DZ 3605, SNZ, 2001) requires that 95% of 140 mm diameter and 125 mm square anchor piles have a flexural capacity of at least 6 kNm. Thus, a 0.6 m high anchor pile with a flexural strength of 6.0 kNm would be expected to fracture at the interface between pile and concrete footing at a horizontal load of $6/0.6 = 10$ kN. Such piles will therefore have a significant reserve capacity over the design value of 6 kN.

2.3 Current Research Program

The committee charged with reviewing NZS 3604:1990 and producing the current standard (NZS 3604:1999) identified the lack of basic information on pile performance – mainly measurement of strength and ductility of foundation systems and analysis of system performance (Shelton, 2001). This project was developed to address this need.

2.4 Earthquake Damage to Building Services

Previous BRANZ research (Thurston, 1993) had identified the need to quantify Serviceability Limit State (SLS) criteria for sub-floor elements – namely, the maximum lateral drift in the sub-floor which would avoid significant damage to building services and secondary elements.

Damage to gas, power, telephone, waste water and sewage services in piled houses needs to be avoided at SLS levels. BRANZ undertook a field survey of 14 typical modern homes founded on piles in the Wellington region. It became clear that the service systems used in these houses had ample flexibility with the exception of possibly the sewerage pipe exiting the house. BRANZ tested modern and historic sewage services by imposing the floor deformations expected in an earthquake. The systems used have varied considerably over the years. It was found that the modern PVC sewage systems with glued or ring joints and earthenware fired clay systems with rubber sleeves or rubber rings joining the pipes withstood the imposed sub-floor movement without damage even with a fully grouted junction at the toilet trap. Although not tested it is likely that systems using brittle pipes, and where all joints are rigidly grouted or wired, would not fare as well. More details of this testing are provided by Thurston (2001(b)).

For the purposes of the torsion study discussed later in this paper, the acceptable ultimate limit of pile deflections was set at 60 mm. Houses founded on flexible foundations may have lateral cyclic movement under commonly occurring wind, traffic etc excitation which may not result in structural damage but still annoys the house occupants. This disturbance is usually precluded by limiting deflections at serviceability level loads. The limit of 60 mm recommended above is not intended to be an acceptability criterion for annoying vibrations.

2.5 Effects of Building Torsion on Pile Damage - Overview

A lack of symmetry in the stiffness of house pile systems was recognised as a potential source of large torsional effects in a seismic event. Such actions would result in twisting of the floor diaphragm and increasing the deformation demand on piles on the more flexible side.

Although design to the standard of NZS 3604:1999 does not directly consider the effects of torsion or the effect of combining foundation elements of different stiffnesses, the Standard does place some limitation on the distribution of foundation elements to minimise torsional effects.

This paper describes the seismic analysis of the foundations of 20 conceptual buildings founded on piles or foundation walls. These included the more extreme eccentric situations which could be considered acceptable under NZS 3604:1999.

3. EFFECT OF BUILDING TORSION ON PILE RESPONSE - PERCEIVED PROBLEM

Current practice for modern houses on flat sections is for houses to be founded on concrete slabs. Piled foundations remain in common use for pre-built homes and for houses on sloping sites to avoid the costly earthworks required to level the section or to keep the house elevated to achieve a desired view. However, by virtue of the ground slope, piles providing lateral bracing are often of different lengths and perhaps type. They may be combined with stiff foundation walls along portions of the house perimeter. These walls can be earth retaining walls, step up or step down walls at junctions where the house is partly founded on slabs (particularly garages) or timber-framed or concrete foundation walls where ground terrain warrants. These variations are likely to make the stiffness of the foundation system extremely non-symmetrical across the plan of the floor. Accidental eccentricity wherein the centre of mass is offset from the centre of area of the floor also needs consideration. Consequently under uni-directional earthquake excitation the floor is likely to twist due to torsional effects. The twisting of the floor diaphragm may result in excessive deflections resulting in pile failure in areas where the pile system was the least stiff. This is particularly likely where house lateral movement is restrained at junctions with concrete garage floors and the house rotates about this point.

4. REQUIREMENTS OF NZS 3604:1999

Section 5.4 of NZS 3604:1999 details the requirements for the distribution of sub-floor bracing elements. When a floor is not specifically considered to be a structural floor diaphragm, then bracing lines are required at a maximum of 6 m centres and each bracing line must be designed to have a design strength of at least 70 BUs, (i.e. 3.5 kN). In addition external bracing lines must have at least $10 \times L$ BUs where L is the length of the external wall in metres. Thus, as both braced piles and anchor piles are deemed to have an earthquake bracing rating of 120 BUs (i.e. 6 kN), only one of either pile type is required as a minimum on a 12 m long external wall or internal bracing line.

This project analyses various house foundation configurations which have close to the maximum eccentricity possible while still satisfying the requirements of NZS 3604:1999. This leads to recommendations on the appropriate layout of bracing elements to achieve satisfactory torsional behaviour.

5. ANALYSIS OF VARIOUS FOUNDATIONS UNDER RECTANGULAR HOUSES

5.1 Method of Analysis

The analysis was by Excel (MicroSoft, 2000) spreadsheet using the Excel ‘Solver’ tool and macros to speed arrival at a solution. Input data include coordinates of each pile and the pile type which defined the non-linear pile force versus deflection relationship assumed. Other data were the coordinates of the centre of mass and the maximum earthquake force to be applied. At each step of the analysis, the earthquake force was incrementally increased and the deflection at each pile calculated.

The solution proceeded by assuming a deflection of the floor diaphragm (Δ_{XCOM} , Δ_{YCOM}) at the centre of mass (COM) at coordinates (X_{COM} , Y_{COM}) and also diaphragm rotation (θ_{COM}) about the centre of mass. The deflection of a pile at coordinates (X_P, Y_P) can be calculated from simple geometry as ($\Delta_{XCOM} + \theta_{COM} \times (Y_P - Y_{COM})$, $\Delta_{YCOM} + \theta_{COM} \times (X_P - X_{COM})$). Hence the force at each pile can be calculated from the user defined pile stiffness properties. By summing the forces at each pile and also total torque from the pile group and by comparing with the assumed seismic force at a particular load step, the error in total X and Y force and torque was calculated. ‘Solver’ was automated by macro to adjust the assumed values of X_{COM} , Y_{COM} and θ_{COM} , until the error was negligible.

To consider the effects of variation of pile length due to ground slope on the pile stiffness, the spreadsheet used a user-defined stiffness factor (generally 1.0) at each pile. More information is given below in the particular analyses which used factors other than 1.0.

5.2 Input Data

5.2.1 Pile load versus deflection data

Pile load versus deflection behaviour was assumed to follow the ‘backbone’ curve joining peak load for the “repeat loops” of reverse cyclic loading as determined from field experiments by Thurston (1996). These curves were defined by the equation:

$$P = \frac{A \cdot \Delta}{B + \Delta^C} \dots\dots\dots(1)$$

Where: P = Applied load in kN

Δ = Pile deflection in mm

A, B, C = Constants

Table 1. Constants for pile stiffness or connection stiffness

<i>Pile Type</i>	<i>Direction</i>	<i>A</i>	<i>B</i>	<i>C</i>
Anchor	Any (in clay)	10.22	13.80	0.8835
Braced	Brace (in clay)	13.67	31.12	1.038
Stringer/wall	Perpendicular to Wall	2.36	0.105	1.0
Stringer/wall	Parallel to Wall	37.1	12.04	1.413

Values of A, B and C determined by Thurston (1996) for piles in soft clay close to the minimum soil strength allowed by NZS 3604:1999 are given in Table 1. The values have been corrected for anchor piles 600 mm above the ground and for braced piles, at 45° to the horizontal. The values for the stringer/wall are discussed later and represent the values for a 1 m long connection and are based on testing described in Appendix A. Figure 1 plots the above relationships.

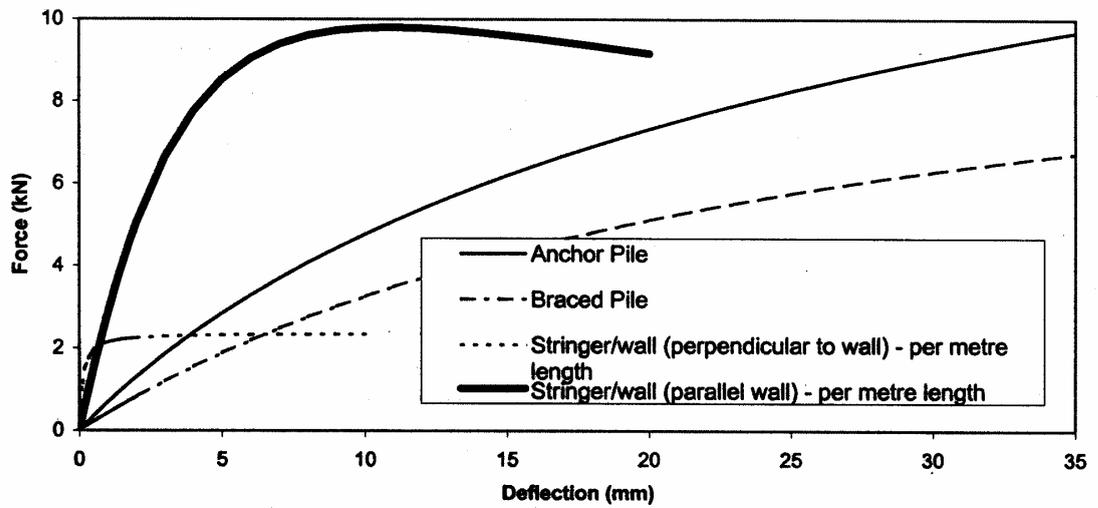


Figure 1. Plot of foundation element load versus slip relationship as defined by Table 1.

5.2.2 Retaining wall load versus deflection data

Various pile layout configurations were analysed for rectangular houses as depicted in Figure 2. A 6 m long masonry wall between garage concrete floor-slab and house timber floor was modelled in four of these analyses. This wall was shown as a thick line at the locations – see Figure 2.

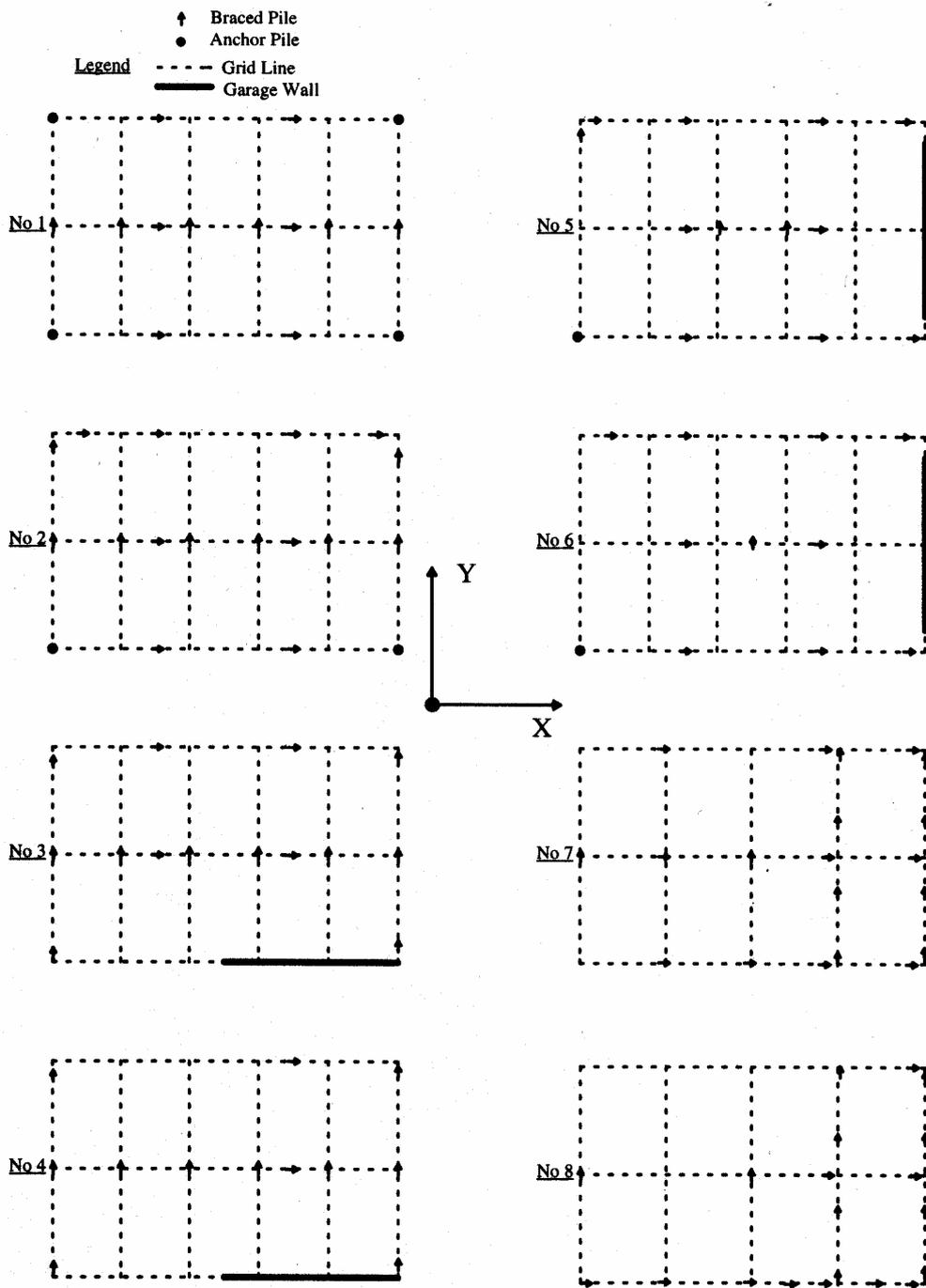


Figure 2. Pile configurations analysed for rectangular house

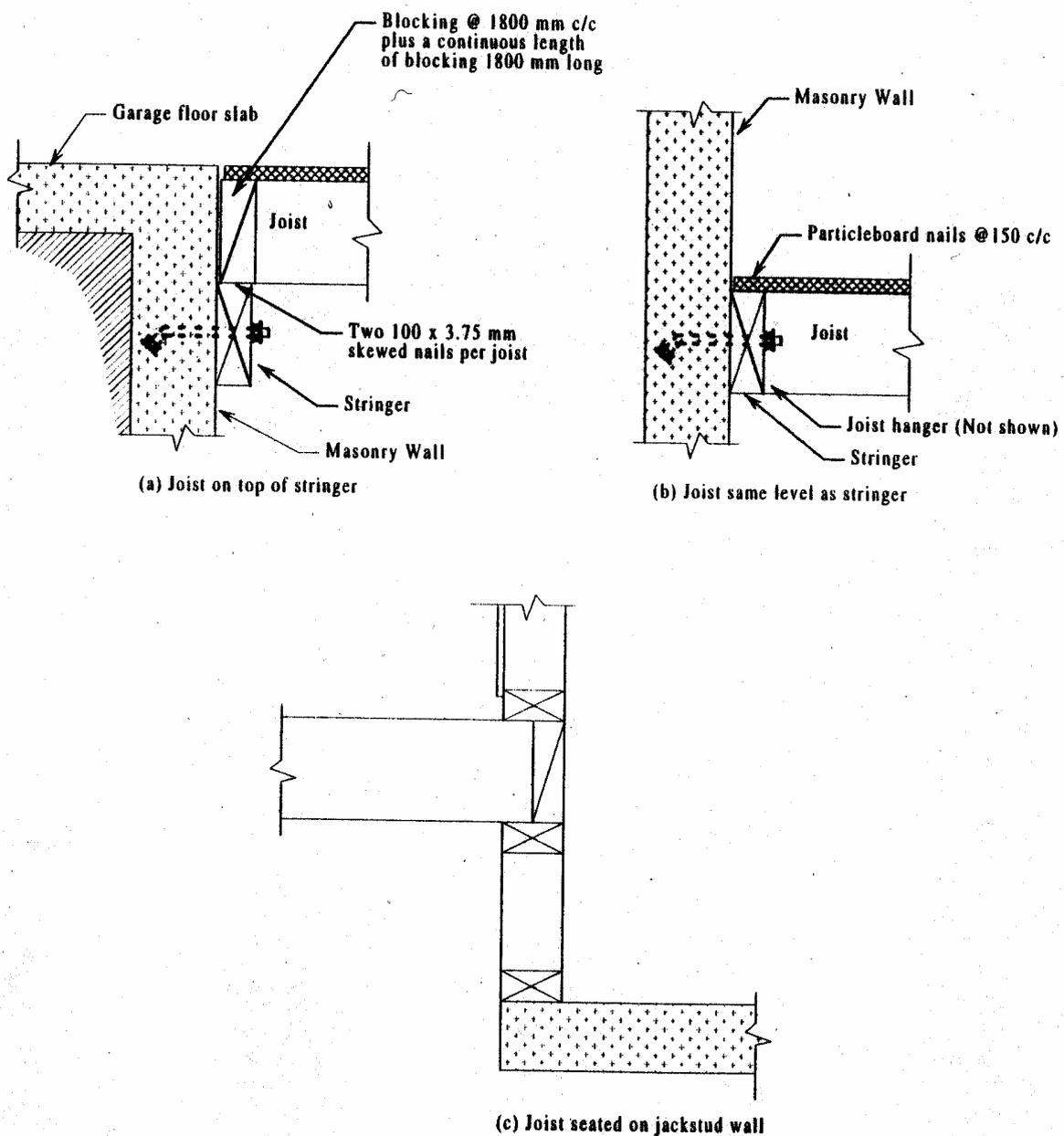


Figure 3. Fixings at junction of house floor on piles and garage

Typical details of the connection between the timber floor and the masonry wall for joists perpendicular to the wall are shown in Figure 3. In Figure 3(a) the joists are seated on and skew nailed to stringers. Blocking is used between some joists as stipulated in NZS 3604:1999. In (b) the joists are at the same level as the stringers and are connected by joist hangers. This is less common but is expected to result in a stronger connection because of direct fixing of the flooring. Because of this, and since this programme aims to quantify a lower bound solution, it is not considered further. It is also typical to fasten the house floor directly to a timber-framed garage internal wall. The most common method of achieving this is for the joists to be seated on a jackstud wall as shown in (c). The strength and stiffness of the construction is expected to be similar to that in (a) if the jackstud wall is well braced. A less common alternative is for a stringer to be bolted to the timber studs of the garage internal wall and for the joists to be seated on the stringer.

Where joists are perpendicular to the wall, the detail is similar to (b) except the stringer is an edge joist and no joist hangers are required. This is again expected to result in a stronger connection than in Figure 3(a) for load parallel to the masonry wall and hence was not considered further.

The major component of the movement at the retaining wall is expected to be slip between the stringer and wall. Based on work by Beattie (1998) and assuming six bolts between stringer and wall, the load versus slip relationship parallel to the stringer is given in Table 1. The load versus slip relationship perpendicular to the stringer is derived in Appendix A and assumes each joist is skew nailed to the stringer. An estimate of the relationship describing the resulting stiffness is given in Table 1 based on laboratory tests at BRANZ and by Dean (1988).

Where masonry walls resist seismic loads in the direction parallel to the walls, the horizontal forces need to be transmitted from the timber floor diaphragm through the blocking to the stringer and then through the bolted connection to the masonry wall. The weakest of these links will govern the strength of the load path. The strength of all these connections was measured as reported in Appendix A and is compared in Figure A.7. The weakest link turned out to be the nailed connection between floor diaphragm and stringer. Figure A.3 also plots the total system load versus deflection relationship and compares this with the NZS 3604:1999 design load. The concern is that the latter is significantly less than the total system strength. Appendix A derives a recommended design load.

5.2.3 Rectangular house NZS 3604:1999 compliant pile layout

Pile layouts used for the analysis of rectangular houses are shown in Figure 2. The arrows show the directions of the resistance of the piles and not the direction of applied loading. These layouts are described below and summarised in Table 2 and the grid and legend just below Table 2. Stiffness details and results of these analyses are given in Table 2 and Figures 3 to 6.

The house was assumed to be a single storey rectangular house (12 m x 7.5 m). It had light walls and heavy 30° pitch roof and was located in a very high wind zone and earthquake Zone A. From NZS 3604:1999 the required earthquake foundation bracing rating is $13.3 \times 12 \times 7.5 = 1197$ BU and the wind requirement is $117 \times 12 = 1404$ BU perpendicular to the ridge and $129 \times 7.5 = 968$ BU parallel to the ridge. NZS 3604:1999 gives the design strength of both braced and anchor piles as 120 BU for earthquake and 160 BU for wind. Earthquake loading governs and the required number of braced + anchor piles = $1197/120 = 10$ in each direction.

Description of pile layouts analysed

House No. 1A

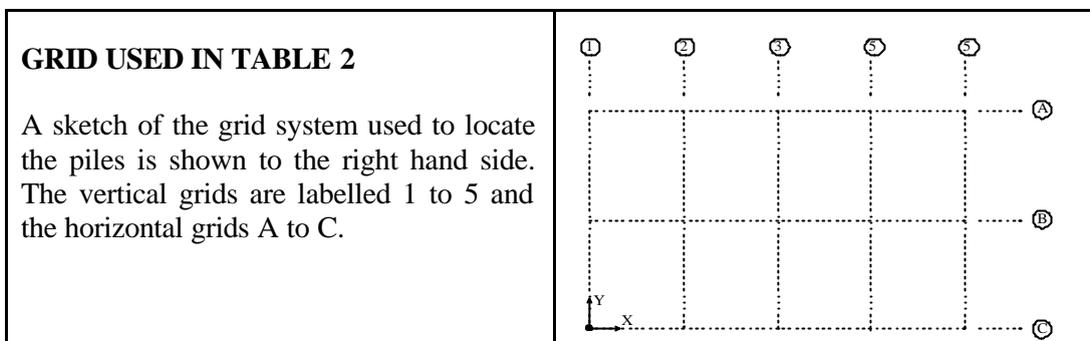
A symmetrical lateral load resisting pile layout was used with no accidental eccentricity as shown in Figure 1 and thus no twisting was expected under horizontal loading. Anchor piles were used in the corners and braced piles elsewhere.

Table 2. Summary of analysis results (rectangular shaped houses)

Run No	House No.	Features	EQ Dir.	Maximum Pile Deflection #	Ratio with Run1	Floor Rotation Radians x 1.0E-5
1A	1	Symmetrically placed piles. Anchor piles in all corners.	X	20.0	1.0	0.0
			Y	20.0	1.0	0.0
1B	1	Pile stiffness decreases by factor 2 over the width of the house – i.e. stiffness ratio of piles on row C:A=2 and row C:B=1.5.	X	29.6	1.5	31.3
1C	1	Pile stiffness decreases by factor 4 over the width of the house.	X	42.4	2.1	83.9
1D	1	Pile stiffness decreases by factor 2 over the length of the house.	Y	33.0	1.7	68.3
1E	1	Pile stiffness decreases by factor 4 over the length of the house.	Y	56.1	2.8	204
1F	1	Mass eccentric 0.75 m in Y direction = 0.1b.	X	21.1	1.05	28.5
1G	1	Mass eccentric 1.2 m in X direction = 0.1w.	Y	24.6	1.2	70.8
2	2	Same as Run 1A except the anchor piles along Line A replaced with braced piles.	X	23.5	1.2	12.7
3	3	As Run 2 but garage masonry wall added on Line C and piles in X direction on Line A and B changed to close to minimum – i.e. two piles = 240 BU.	X	18.1	0.9	138.5
4	4	As Run 3 but piles in X direction on Line A and B put at minimum - one pile = 120 BU.	X	24.3	1.2	188.6
5	5	As Run 2 but garage masonry wall added Line 5 and piles in Y direction on Line 1 and 3 changed to close to minimum – i.e. two piles = 240 BU.	Y	34.9	1.7	275
6	6	As Run 5 but piles in Y direction on Line 1 and 3 put at minimum - one pile = 120 BU.	Y	114.0 27.9*	5.7 1.4	886 214*
7	7	Symmetrical piles layout in X direction but maximum eccentricity in Y direction.	X	28.6	1.4	36.5
			Y	30.9*	1.5	219*
8	8	Maximum eccentricity in both X and Y direction.	X	52.8	2.6	397
			Y	39.7*	2.0	307*

Legend for Table 2.

Maximum pile deflection # was calculated at a lateral force = 1197 BU. For the pile configurations where this force could not be resisted then the maximum deflection was calculated at half this force = 599 BU. This is identified by an * in the last column of the table.



House No. 1B and 1C

This was the same pile layout as in House 1A except it was assumed to be on a sloping foundation such that the pile stiffness reduced by a factor of $F = 2$ (House 1B) or 4 (House 1C) over the width of the house. (i.e., for $F = 2$, the stiffness of the centre row of piles was reduced by $1/1.5$ and the stiffness of the outside row of piles is reduced by $1/2$.) Under an X direction earthquake this results in the house twisting.

House No. 1D and 1E

This was the same pile layout as in House 1A except it was assumed to be on a sloping site such that the pile stiffness reduced by a factor of $F = 2$ (House 1D) or 4 (House 1E) over the length of the house. House twisting was expected under a Y direction earthquake.

House No. 1F

This was the same pile layout as in House 1A. However, the mass was assumed to be eccentric from the centre of area by $0.1 \times (\text{width of house}) = 0.75 \text{ m}$.

House No. 1G

This was the same pile layout as in House 1A. However, the mass is assumed to be eccentric from the centre of area by $0.1 \times (\text{length of house}) = 1.2 \text{ m}$.

House No. 2

This was the same pile layout as in House 1A except the anchor piles at the upper corners as drawn were each changed to braced piles in each direction. The ground was assumed to slope downwards at the upper side of the house and the pile height would consequently be too large for an anchor pile at these locations. This made the foundation stiffness slightly eccentric.

House No 3

This foundation system included a 6 m long retaining wall for a garage floor-slab, shown as a thick line in Figure 2. NZS 3604:1999 allows this wall to have a wind and earthquake design strength of $6 \times 300 = 1800 \text{ BU}$. If a structural floor diaphragm is not assumed then the exterior and interior bracing lines are required to have a bracing resistance of 10 BU/m (i.e. $10 \times 12 = 120 \text{ BU}$) which is satisfied by merely a single braced or anchor pile. However, a prudent designer would probably use at least two piles in each row as was used for this design. Thus, two braced piles were used along the upper and middle X direction grid lines.

House No. 4

This was the same as House 3 but used the minimum number of braced piles to comply with NZS 3604:1999 (namely 1) along the upper and middle X direction grid lines.

House No. 5 and 6

This was similar to House 3 and 4 respectively, except the garage retaining wall was placed at the end rather than the side of the house. House No 6 had the minimum number of braced piles while House 5 had two extra braced piles.

House No. 7

This house consisted solely of braced piles. A symmetrical layout of these piles was used to resist an X direction earthquake. However, the layout to resist a Y direction earthquake concentrated the piles near one end of the house, effectively giving the maximum pile non-symmetry which still satisfies NZS 3604:1999.

House No. 8

This was a more extreme situation than House 7. The layout to resist either an X or Y direction earthquake used the maximum eccentricity allowed by NZS 3604:1999.

6. SUMMARY OF FINDINGS FROM ANALYSIS OF RECTANGULAR HOUSE

Figure 4 shows the relationship between an X direction horizontal force at the centre of mass and the maximum pile deflection for each of the 8 houses except House 5 and 6. These were not included as they are not eccentric for this direction of earthquake. Figure 5 is similar but shows amount of twist (i.e. rotation) of the floor diaphragm. Figure 6 and Figure 7 show the same information but for a Y direction earthquake and for all houses except House 3 and 4. Again, these were not included as they are not eccentric for this direction of earthquake.

The standard symmetrical house (1A) deflected 20 mm in the design event and exhibited no floor rotation. Results from the other analyses are discussed below:

Variable pile stiffness across width or length of house

The house was assumed to be on a sloping site such that the pile stiffness (but not strength) reduced linearly by a factor of 2 or 4 over the width or length of the house. Test results on braced piles (Thurston, 2001 (a)) indicated that these were extreme scenarios. Maximum pile deflections and house rotations from these analyses are summarised in Table 3 and indicate significant increases when the factor is 4 (particularly where the change in stiffness was over the length of the house). However, maximum deflections remained within the limit of 60 mm set for this study.

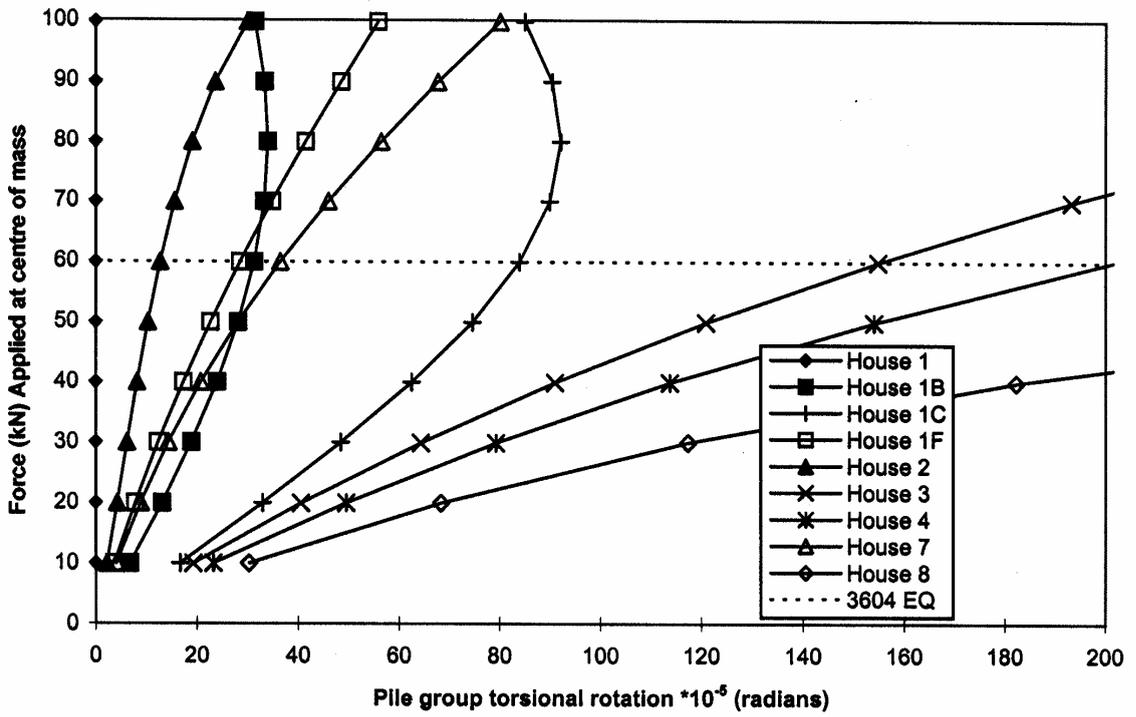


Figure 4. Maximum pile deflections versus X force – rectangular pile configuration

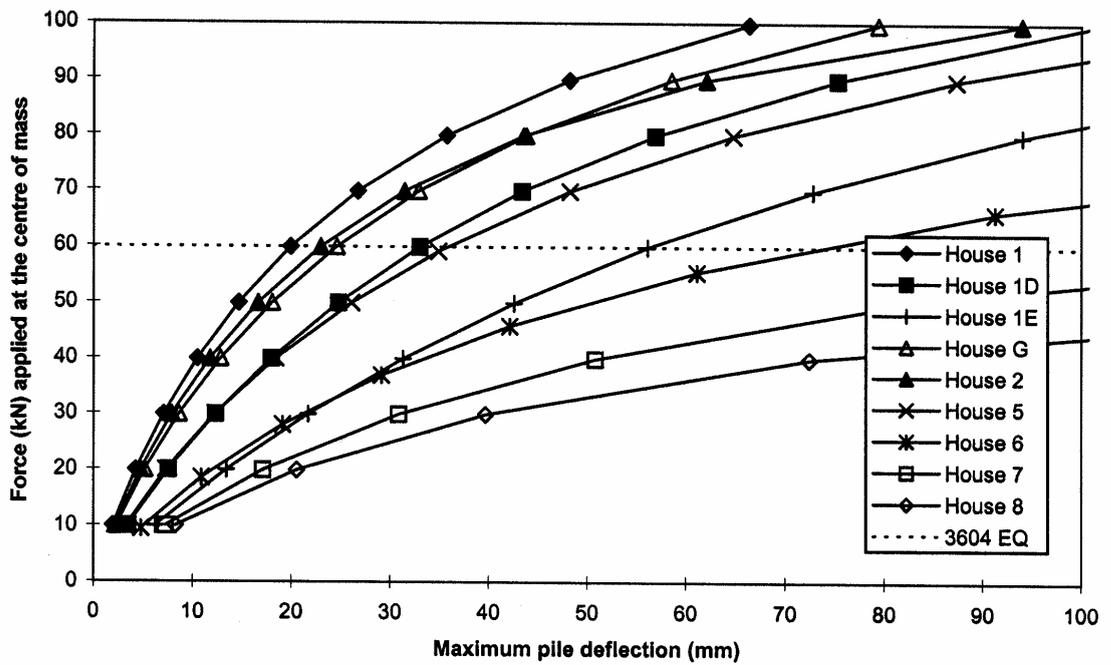


Figure 5. Floor rotation versus X force – rectangular pile configuration

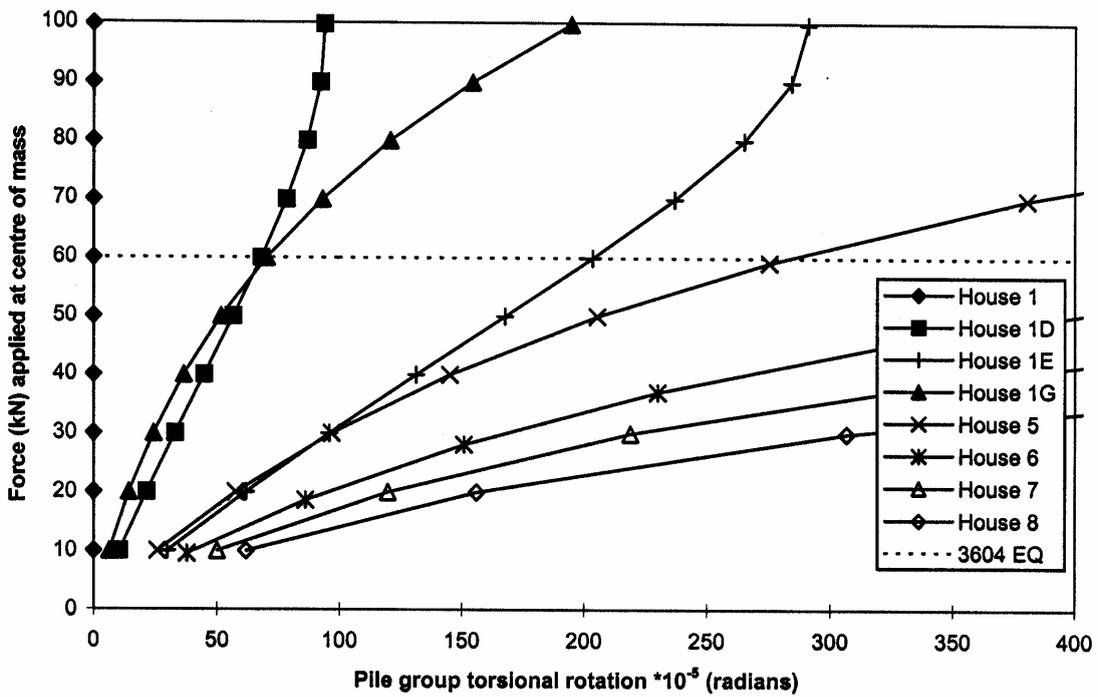


Figure 6. Maximum pile deflections versus Y force – rectangular pile configuration

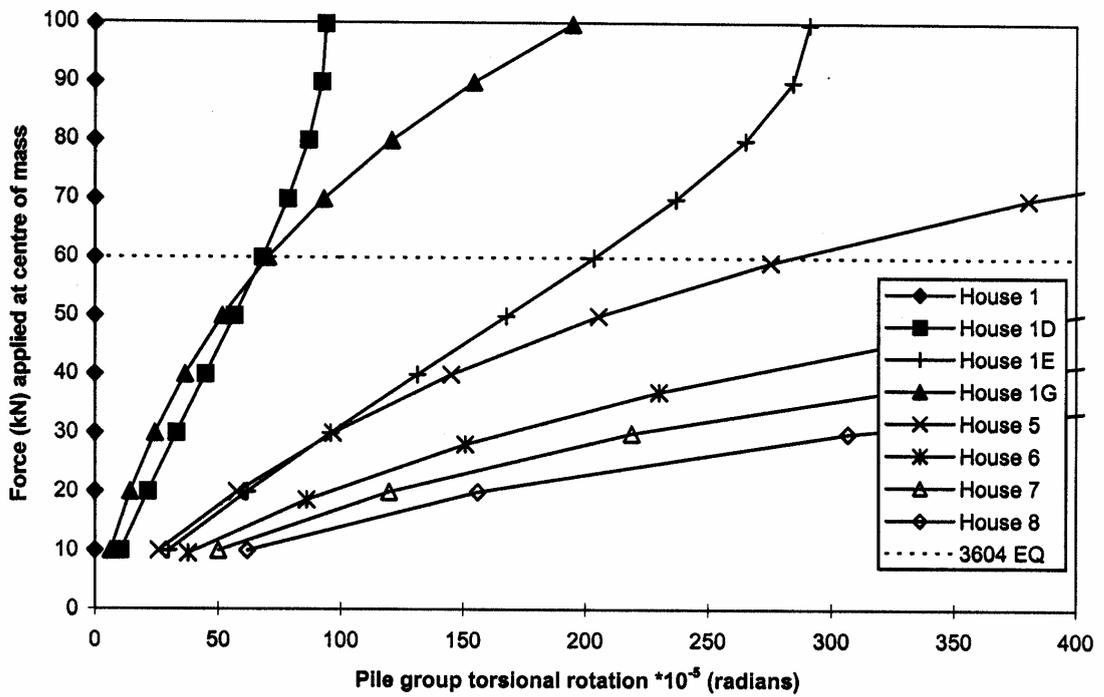


Figure 7. Floor rotation versus Y force – rectangular pile configuration

Table 3. Maximum pile deflections and house rotations with variations in site slope (rectangular house)

<i>House Analysis Number</i>	<i>Stiffness Reduction Factor</i>	<i>Reduced Across</i>	<i>Maximum Deflection (mm)</i>	<i>Maximum Rotation (Radians x1.0E-5)</i>
1B	2	Width	29.6	31.3
1C	4	Width	42.4	83.9
1D	2	Length	33.0	68.3
1E	4	Length	56.1	204

House mass offset from centre of area

The house mass was assumed to be offset from the centre of area by a distance 0.1 x (width of house) (Analysis 1F) or 0.1 x (length of house) (Analysis 1G). This resulted in maximum pile deflections of 21.1 and 24.6 mm for the two directions respectively which are relatively small increases from the 20 mm deflection for the concentric mass case. The eccentricity used was the design value used in the loadings standard (SNZ 1992).

Garage wall added to side of house

A 6 m long garage wall was added to the house foundation system and the number of braced piles in other bracing lines in the wall direction were reduced to two or one (with one being the minimum allowed by NZS 3604:1999. Maximum pile deflections and house rotations from these analyses are summarised in Table 4 and Figures 3 to 6 and indicate large maximum pile deflections for cases where the garage wall was at the end of the house. House 6 was unable to resist NZS 3604:1999 loads while the deflections for House 5 were just within the 60 mm limit (Figure 6). Where the wall was on the sides of the house, (House 3 and 4) the deflections at NZS 3604:1999 loads were not as high (Figure 6). However, the maximum load within the 60 mm limit was only 35% and 16% more than NZS 3604:1999 loads for the two and one pile scenarios, respectively.

It is concluded that the two braced pile scenario provided just acceptable results whereas the single braced pile scenario produced unacceptable results.

Table 4. Effect of garage wall on maximum pile deflections and house rotations (rectangular house)

<i>House Analysis Number</i>	<i>No. Braced Piles in Other lines</i>	<i>Garage Wall Direction (See Fig 1)</i>	<i>Maximum Deflection (mm)</i>	<i>Maximum Rotation (Radians x1.0E-5)</i>
3	2	X	18.1	138.5
4	1	X	24.3	188.6
5	2	Y	51.1	391
6	1	Y	114.0	886

Symmetry of pile layout

In House 7 the braced piles in the Y direction were concentrated at one end of the house to near the maximum extent allowed by NZS 3604:1999 while the layout in the X direction was near symmetrical. House 8 was the same as House 7 except that the braced piles in the X direction were also concentrated on one side of the house, to near the maximum extent allowed by the standard.

The analysis of House 7 and 8 as plotted in Figure 4 and Figure 6 indicate that the maximum pile deflection would exceed 60 mm before the resistance provided reached the NZS 3604:1999 demand levels. The ratio between the NZS 3604 level loading and the maximum pile rotation when the 60 mm limit was reached is summarised in Table 5. The analyses were then re-run as Analysis 7b and 8b where at least two piles were retained on each bracing line. The resisted load came within 6% of the NZS 3604 loading in both cases. Analysis 8b reached the NZS 3604 loading at 68 mm deflection which is not expected to result in excessive foundation damage. Thus, even for these extreme pile location non-symmetries, it is concluded that the two braced pile scenario provided close to acceptable results whereas the single braced pile scenario produced unacceptable results.

Table 5. Effect of pile symmetry on maximum pile deflections and house rotations (rectangular house)

<i>House Analysis Number</i>	<i>Ratio of Force at 60 mm to NZS 3604:1999 demand load.</i>	<i>Maximum Rotation (Radians x1.0E-5)</i>
7	71.6 %	434
8	60.4 %	469
7b	96.9 %	371
8b	94.0 %	389

7. ANALYSIS OF VARIOUS FOUNDATIONS UNDER ‘L’ SHAPED HOUSES

Pile layouts used for the analysis of ‘L’ shaped houses are shown in Figure 8. These layouts are described below and summarised in Table 6. Stiffness details and results of these analyses are given in Table 6. Figure 9 shows the relationship between an X direction horizontal force at the centre of mass and the maximum pile deflection for each of the three houses. Figure 10 is similar but for the amount of twist (i.e. rotation) of the floor diaphragm.

The house was assumed to be a single storey ‘L’ house with each long side being 12 m and each short side being 6 m giving a floor area of 108 m². It had light walls and heavy 34° pitch roof located in a very high wind zone and earthquake Zone A. From NZS 3604:1999, the required earthquake foundation bracing rating is 108 x 13.3 = 1436 BU and the wind requirement is 6 x 117 + 6 x 129 = 1476 BU. Earthquake loading governs as the resistance is deemed to be 160 BU under wind load and 120 BU under earthquake load (see Section 5.2.3) and the number of braced + anchor piles = 1436/120 = 12 piles in each direction.

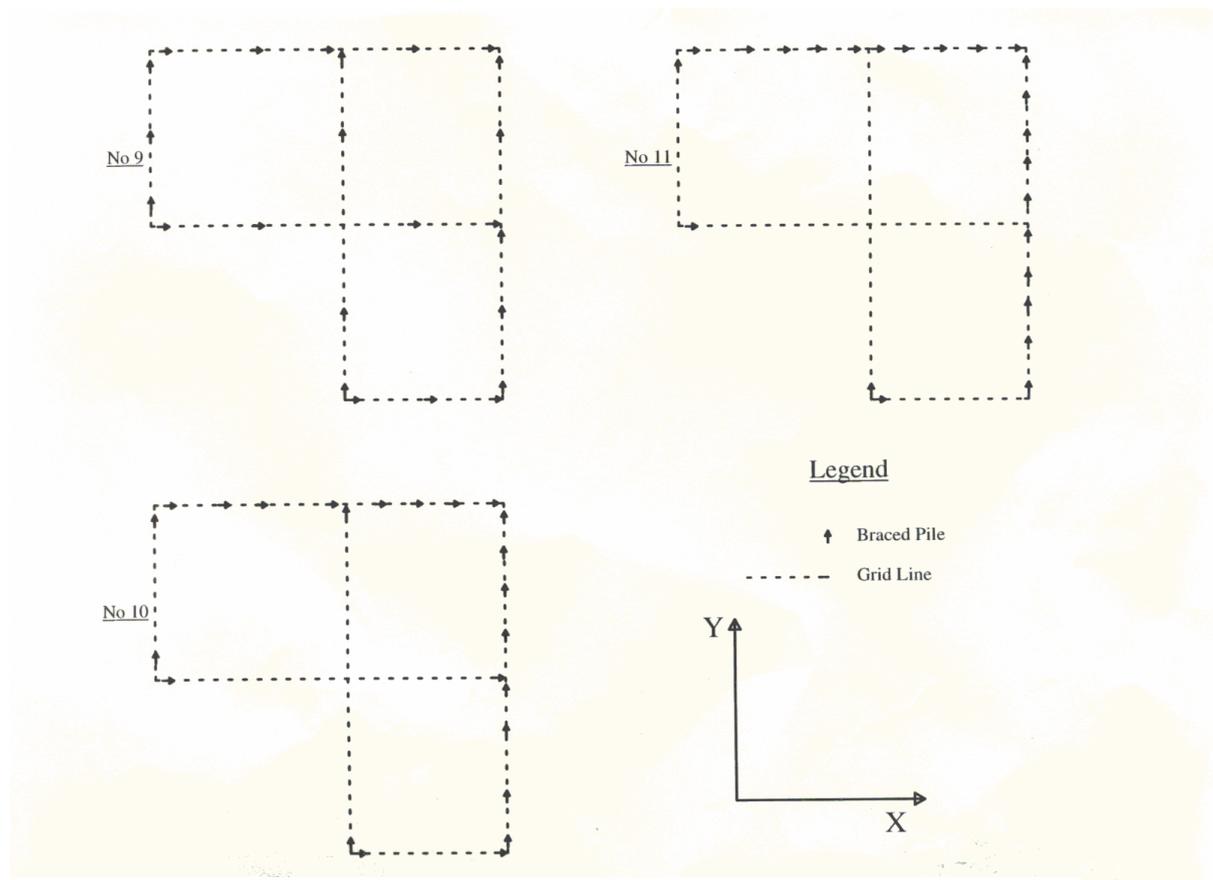


Figure 8. Pile configurations analysed for 'L' shaped house

Description of Pile Layouts Analysed.

House No. 9

This house consisted solely of braced piles. The pile layout used is shown in Figure 9 and little twisting was expected under horizontal loading.

House No. 10 and 11

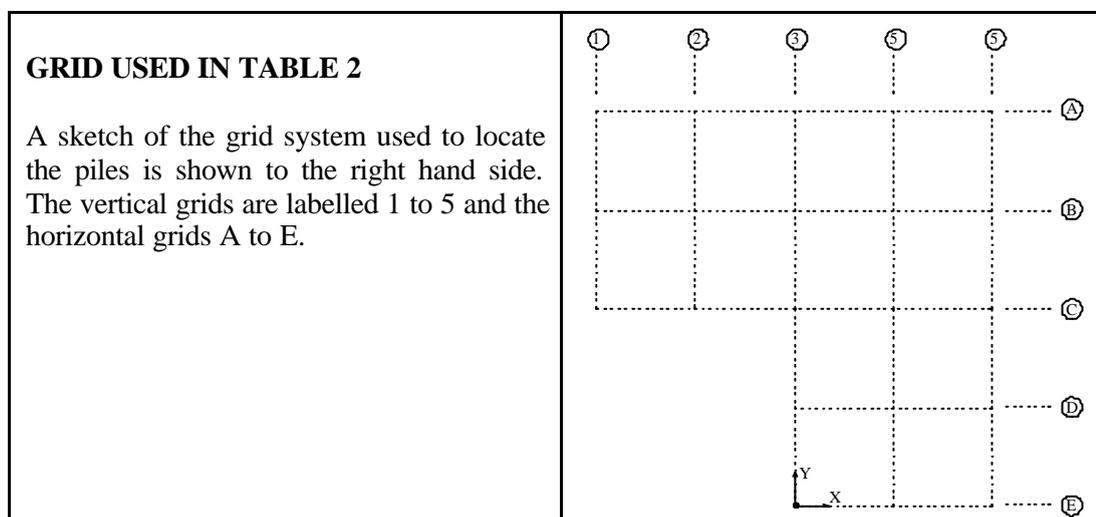
House 11 used the pile layout which concentrated the piles at one end and side of the house to the maximum extent allowed by NZS 3604:1999. House 10 was similar to House 11 except it used two rather than just a single braced pile as a minimum on any bracing line.

Table 6. Summary of analysis results ('L' shaped house)

Run No.	House No.	Features	EQ Dir.	Maximum Pile Deflection #	Ratio with Run 9	Floor Rotation Radians x 1.0E-5
9	9	Evenly distributed piles.	X	19.2	1.0	9.6
10	10	As Run 9 but piles placed to give near maximum eccentricity in both X and Y directions. Near minimum number of piles on Lines 1, 3, C and E – i.e. two piles.	X	28.5	1.5	96.9
11	11	As Run 9 but piles placed to give maximum eccentricity in both X and Y directions. Minimum number of piles on Lines 1, 3, C and E – i.e. one pile.	X	101.8 29.6*	5.3 1.5	668 177*

Legend For Table 6:

Maximum pile deflection # was calculated at a lateral force = 1197 BU. For the pile configurations where this force could not be resisted then the maximum deflection was calculated at half this force = 599 BU. This is identified by an * in the last column of the table.



8. SUMMARY OF FINDINGS FROM ANALYSIS OF 'L' SHAPED HOUSE

House 9 deflected 19.2 mm in the design event and exhibited small floor rotation. The maximum deflection from the analysis of House 10 (minimum of two piles on each bracing line) was 28.5 mm at NZS 3604:1999 loads. However, the resisted load was only 86% of NZS 3604:1999 at 60 mm deflection in the analysis of House 11. It is concluded that the two pile per line scenario resulted in acceptable deflections whereas the single braced pile per line scenario was too torsionally flexible.

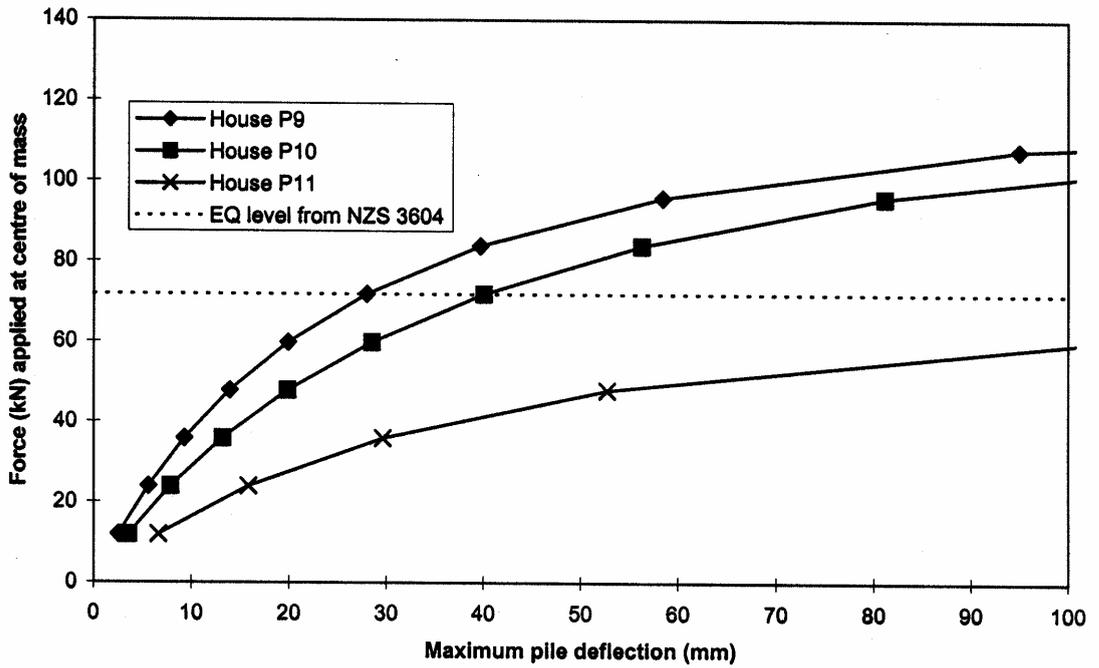


Figure 9. Maximum pile deflections versus X force – “L” shaped house

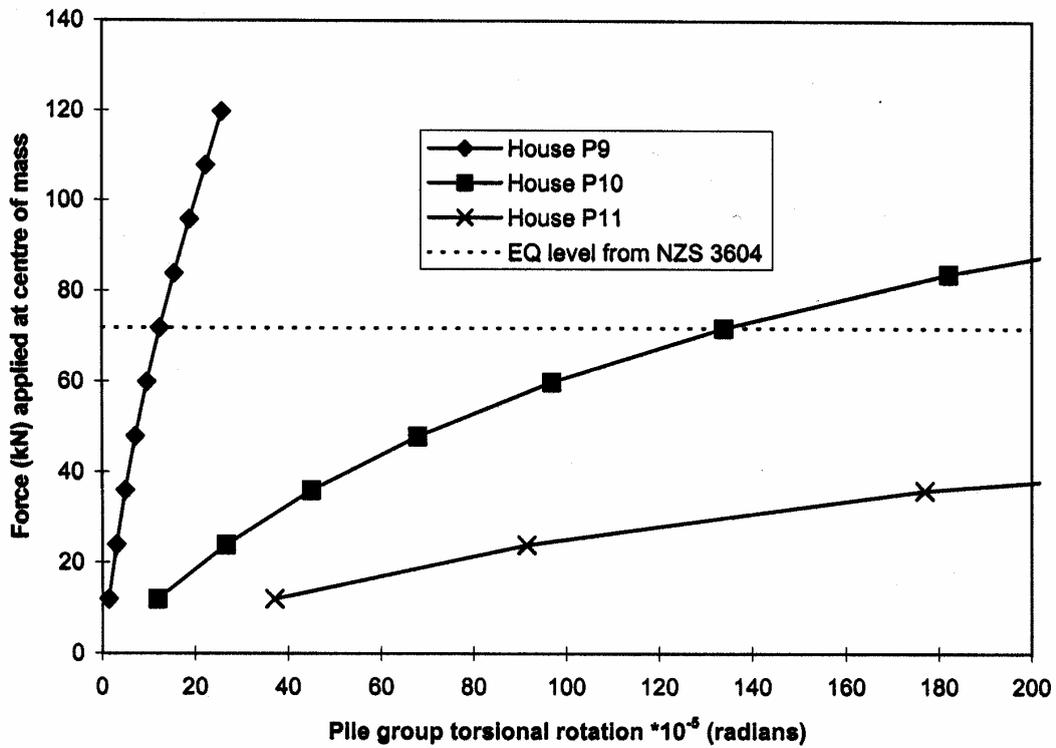


Figure 10. Floor rotation versus X force – “L” shaped house

9. CONCLUSIONS AND RECOMMENDATIONS

Section 5.4.2.3(a) of NZS 3604:1999 requires all bracing lines to have a bracing resistance of at least 10 BUs times the length in metres of the external wall. Based on the analysis in Sections 5 and 7 it is recommended that this minimum bracing rating requirement be doubled.

Table 5.11 of NZS 3604:1999 provides values for the bracing resistance of reinforced concrete and masonry walls. From the results of testing described in Appendix A, it would appear that the higher bracing ratings cannot be transmitted from the floor to the wall. It is recommended that where joists land on stringers, that continuous blocking be used between the joists over the stringers and the flooring be nailed to this edge at 150 mm centres. The strength of this nailing then becomes a limiting factor for the load transfer between the floor and stringer and it is recommended that the maximum bracing value of the masonry or concrete wall be limited to 150 BU/m length of wall unless extra nailing is provided.

10. REFERENCES

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Appendix A. Derivation of Load Versus Slip Equations for Connection Between Floor and Masonry Wall

A.1 Tests of Strength of Connection Between Particleboard Flooring and Timber

Dean (1988) presented plots from two tests of the shear force versus slip relationship between particle board flooring nailed to timber. The author manually extracted the peak points and fitted curves to them as per Eqn. 1 of Section 4.2.1. Results are given in Table A.1. The author also undertook similar tests for 6 specimens for load parallel and perpendicular to the sheet edge for both 60 x 3.15 annular groove flooring nails and 10g x 50 mm Posidrive Jet Screw screws using 12 mm edge distance. Results are also given in Table A.1 and best fit curves are plotted in Figure A.6.

**Table A.1: Coefficients for Particleboard Floor to Timber Attachment
(Repeat Cycling - Load per fastener)**

<i>Reference</i>	<i>Cycle</i>	<i>Fastener</i>	<i>Direction</i>	<i>A</i>	<i>B</i>	<i>C</i>
Dean (1998)	First	Nails	Parallel	1.119	0.344	0.912
Dean (1998)	Repeat	Nails	Parallel	1.400	0.848	1.234
Author	First	Nails	Parallel	2.443	1.438	1.087
Author	Repeat	Nails	Parallel	8.569	9.445	2.053
Author	First	Screws	Parallel	4.704	1.976	1.106
Author	Repeat	Screws	Parallel	10.853	7.654	1.670
Author	First	Nails	Perp.			
Author	Repeat	Nails	Perp.			

A.2 Tests of Strength of Connection Between Joists and Stringers

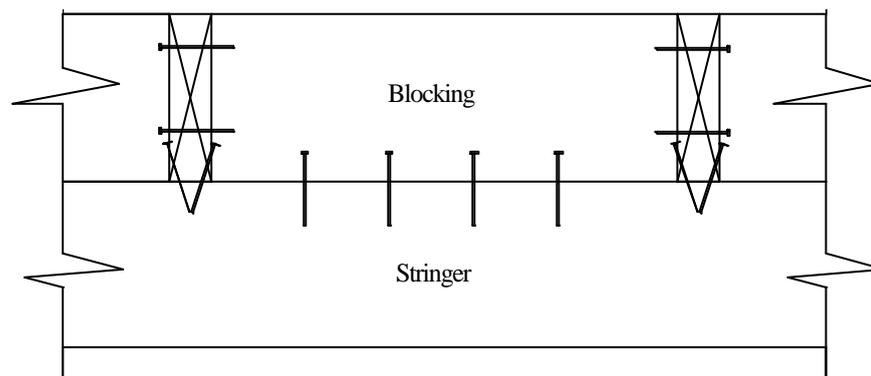


Figure A.1: Construction of Joint Tested

A.2.1 Test Setup

The author tested the strength of the nailed connection between nominal 200 x 50 mm kiln dried joists and stringers to simulate the construction shown in Figure 2(a). Tests were performed with the load parallel and perpendicular to the stringers. Test details can be seen in the photographs below. The joists were nailed to the stringers with a 100 x 4 mm skew nail each side as sketched above. Separate tests were done for construction including full depth blocking between the joists and without blocking. Two 100 x 4 mm nails were nailed through each joist into the ends of the blocking and four skew nails of the same size between blocking and stringer (all from the same side of the blocking). By subtracting the results from the tests with blocking from those without the effect of the blocking was determined. Table A.2 summarises the results determined for joists without blocking and for blocking alone. Figure A.7 plots the best fit curves.

**Table A.2: Coefficients for Joist to Stringer
(Repeat Cycling - Load per joist or blocking piece)**

<i>Member</i>	<i>Direction</i>	<i>A</i>	<i>B</i>	<i>C</i>
Joist	Parallel	53.48	172.65	1.898
Joist	Perpendicular	18.00	5.82	1.282
Blocking	Parallel	10.546	1.7796	1.089
Blocking	Perpendicular	15.16	26.35	1.137

A.2.2 Potential for the stringer to split

In the initial tests where the applied load was perpendicular to the stringer, the stringer split. To prevent this occurring the stringer was subsequently restrained in a manner which prevented the splitting. This section considers the theoretical strength of the stringer under this loading and the results are subsequently used in Section A.4 in examination of the load which can be transferred between floor diaphragm and garage wall in a direction perpendicular to the garage wall.

Timber is relatively weak when bent out of plane about the minor axis (cross-grain bending) and the timber design strength for this loading is usually taken as one third of the shear strength (Walford, 1999). This is used in the calculations below. Bier (1986) reported a fifth percentile bending strength of Radiata pine where some samples contained pith as 1.75 MPa and a mean strength of approximately 3 MPa. He reported that the characteristic tension strength given in CIB-W18 (1978) was 0.75 MPa and that the bending strength was likely to be twice this value. However, for construction where stringers are bolted at spacings significantly larger than joist spacing shear lag effects are likely to result in earlier failure. Splits and cracks in the timber due to drying or weathering will also reduce the strength.

NZS 3603 (SNZ, 1993) gives the characteristic shear strength of No. 1 FG Radiata pine as 3.8 MPa. Hence the maximum transverse force that can be resisted by a 6 m long 190 x 45 mm beam loaded at the top perpendicular to the beam and restrained at mid-height is:

$$Force = \frac{3.8 \times 6 \times 45^2 \times 2}{3 \times 6 \times 190} = 27 \text{ kN}$$

A.3 Strength of Bolted Connection Between Stringer and Masonry Wall

Beattie (1998) measured the load versus slip relationship between a stringer bolted to a foundation wall and the foundation wall under cyclic loading. The load versus slip relationship parallel to the stringer for a single bolt for third cycle loading is given in Table A.3.

**Table A.3: Coefficients for Stringer to Wall
(Repeat Cycling - Load per bolt)**

<i>Connection</i>	<i>Direction</i>	<i>A</i>	<i>B</i>	<i>C</i>
Bolted Joist/Stringer	Parallel	14.5	0.44	1

A.4 Total Load Versus Slip Relation for 6 m long Masonry Wall

The major components of the seismic movement of the floor at the masonry wall for construction as per Figure 2(a) are expected to be:

- (1) shear movement of the masonry wall;
- (2) slip between the stringer and wall;
- (3) slip between stringer and joists; and
- (4) slip between joists and flooring.

Figures 3 and 4 present the relationship between movement and force for diaphragm shear force transfer between floor and wall for the construction in Figure 2(a) for load parallel and perpendicular to the stringer respectively. The assumptions made were:

- (1) Movement (1) above is small.
- (2) M12 bolts between stringer and wall are at the maximum value of 1.6 m centres for joists spanning 3 m as per Table 6.7 of NZS 3604:1999 which results in 5 bolts over a 6m length.
- (3) Joists are at 400 mm centres.
- (4) Blocking is at a maximum of 1.8 m centres (Section 7.1.2.2 of NZS 3604:1999) and also a continuous length of a minimum of 1.8 m (Section 7.1.4.2(b) of NZS 3604:1999). This results in 8 lengths of blocking.
- (5) The flooring is nailed at 150 mm centres along its edge. This effectively assumes continuous blocking along the foundation wall which conflicts with (4) above. However, as transfer of load between flooring and joists is the weakest link, and the author considers continuous blocking to be desirable, continuous flooring nailing is assumed along the foundation wall.
- (6) Nailing between sheet and joists in the body of the sheet is ignored for load transfer between sheet and joists parallel to the foundation wall and assumed to be at 300 mm centres over a length of 3 m for load transfer between sheet and joists perpendicular to the foundation wall.

Items 24 of the components of floor movement (as listed above) were calculated for the construction shown in Figure 2(a) and the assumptions listed above. Figures A.8 and A.9 plot the relationship between shear force and movement for load parallel and perpendicular to the stringer respectively. The design strength of the masonry bracing element is given in Table 5.11 of NZS 3604:1999 as $6 \times 300 = 1800$ BU (or 90 kN) - also plotted in Figure A.8. The plot suggests this load will not be reached in practice as the connections will not be adequate to transmit this load to the masonry wall. Note that NZS 3604:1999 assumes the bracing strength of masonry walls is zero for loading perpendicular to the wall and thus NZS 3604:1999 design strength is not plotted in Figure A.9. However, Figure A.9 does plot the force at which the stringer is expected to split, calculated as described in Section A.2.

The load versus slip relationship used in the torsion analysis based on Eqn. 1 of Section 4.2.1 curve fit of the combined movement in Figures A.8 and A.9. is given in Table A.4. The joist splitting load has been taken as half that shown in Figure A.9 as shear lag effects and timber drying splitting are likely to result in earlier failure as noted in Section A.2.

Table A.4: Coefficients for 6 m Long Wall

<i>Member</i>	<i>Direction</i>	<i>A</i>	<i>B</i>	<i>C</i>
Joist	Perpendicular	14.18	0.1056	1.0
Joist	Parallel	222.6	12.04	1.413

Recommendations

Table 5.11 of NZS 3604:1999 provides values for the bracing resistance of reinforced concrete and masonry walls. From the results of this Appendix it would appear that the higher bracing ratings cannot be transmitted from the floor to the wall. It is recommended that where joists land on stringers that continuous blocking be used between the joists over the stringers and that the flooring be nailed to this edge at 150 mm centres. The strength of this nailing then becomes a limiting factor for the load transfer between the floor and stringer and it is recommended that the maximum bracing value of the masonry or concrete wall be limited to 150 BU/m length of wall unless extra nailing is provided. This is based on a strength of 45 kN for the 6 m long wall in Figure A.8 which is 150 BU/m.



Figure A.2: Load Perpendicular to Stringer Without Blocking



Figure A.3: Load Perpendicular to Stringer With Blocking.



Figure A.4: Load Parallel to Stringer Without Blocking.



Figure A.5 Load Parallel to Stringer With Blocking.

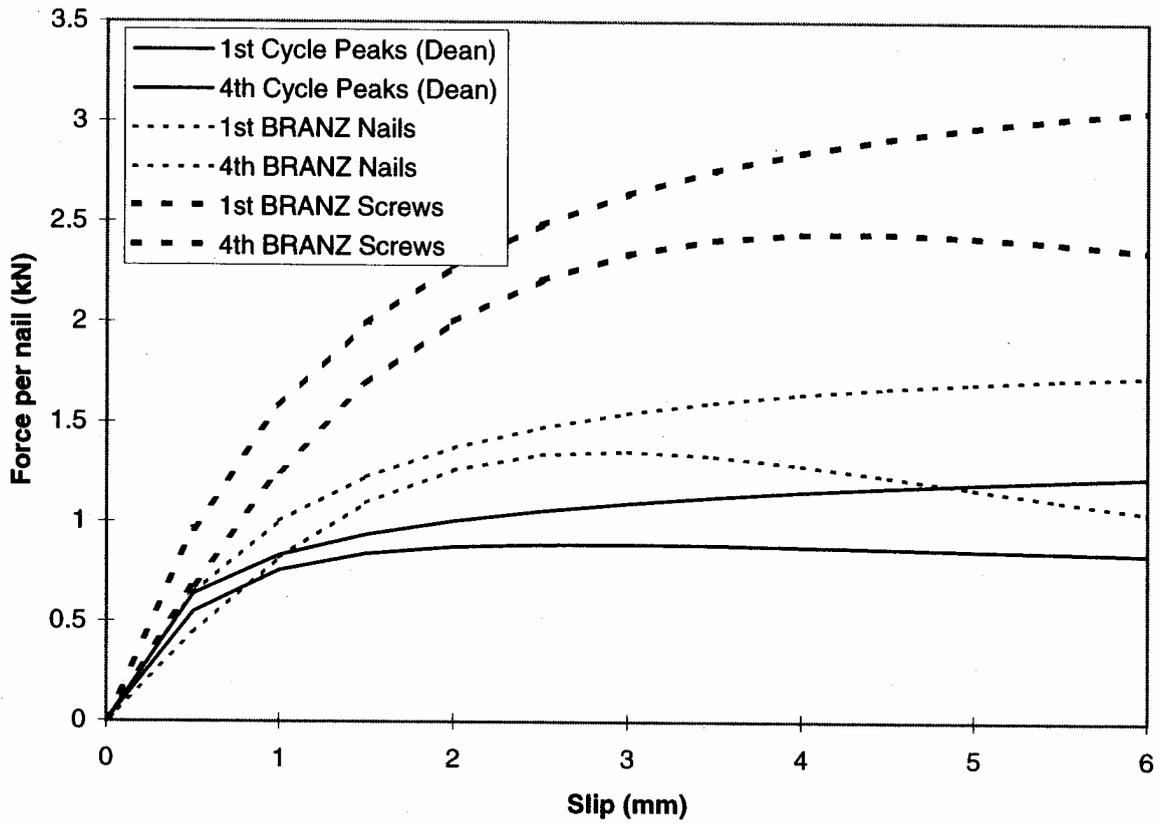


Figure A.6: Comparison of Nail Slip Results for BRANZ and Dean

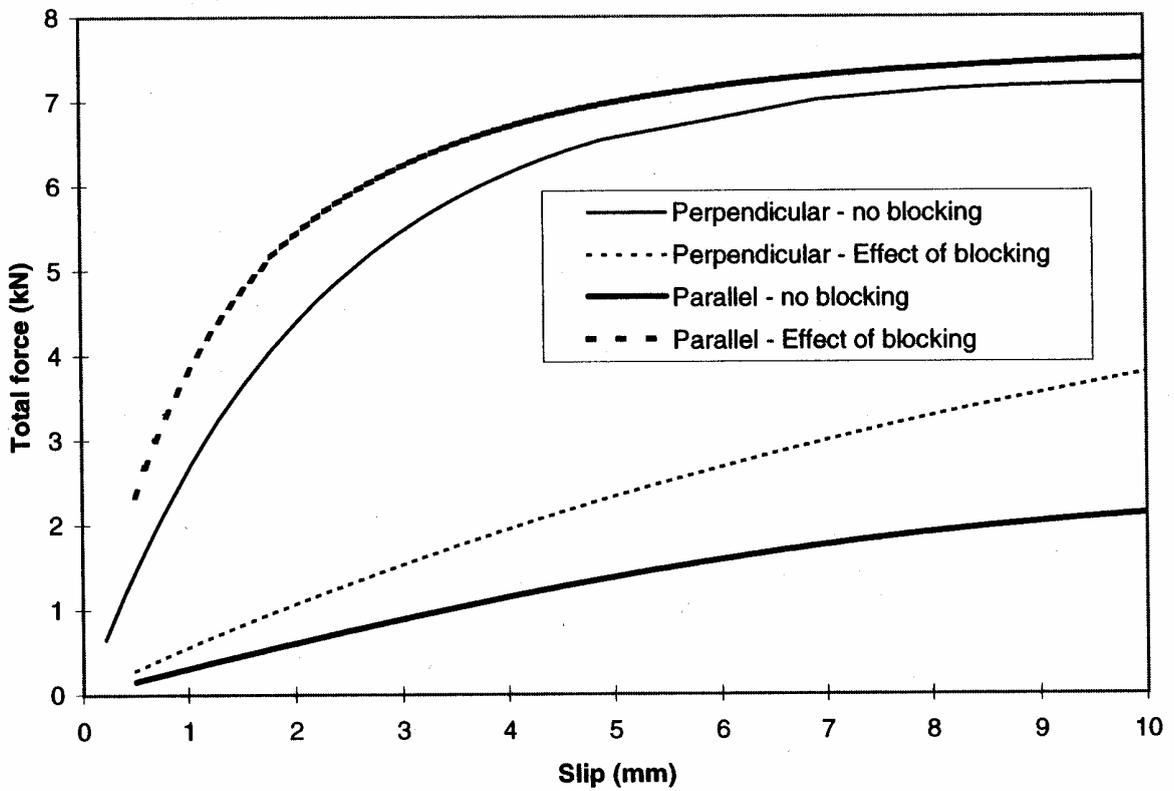


Figure A.7: Load Versus Deflection for Joists Perpendicular

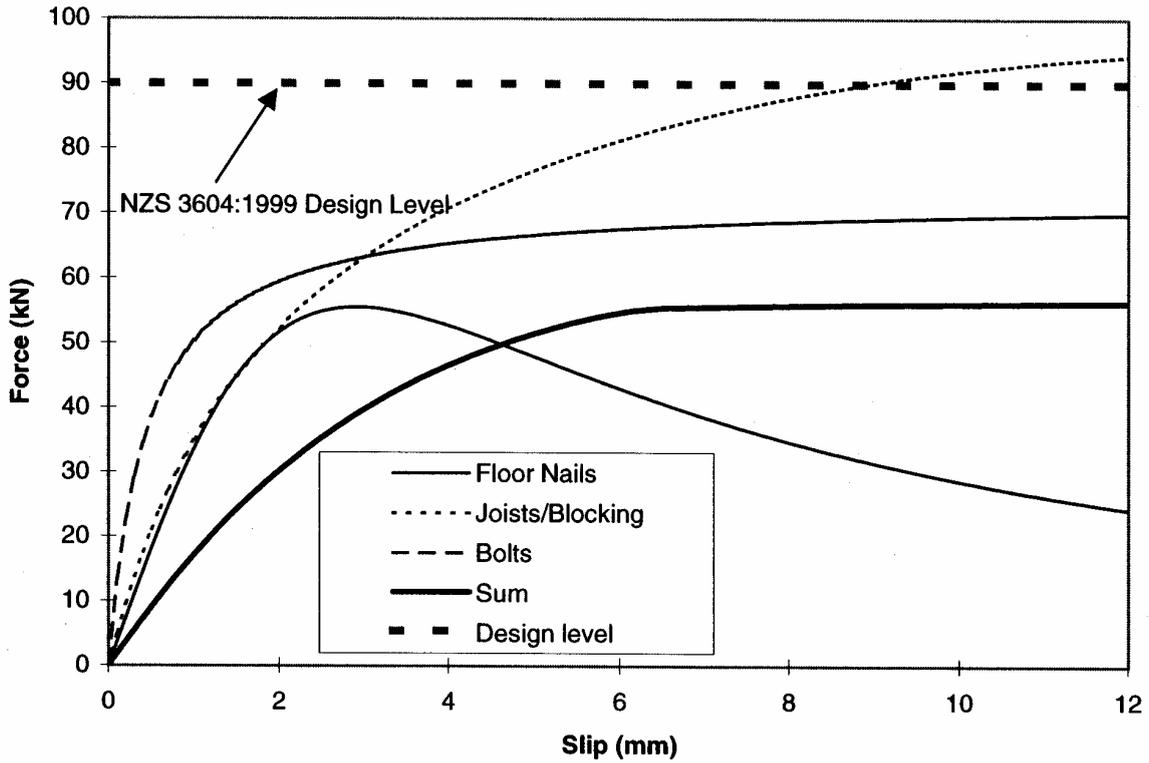


Figure A.8: Movement Between Wall and Floor – Force Parallel to Stringers

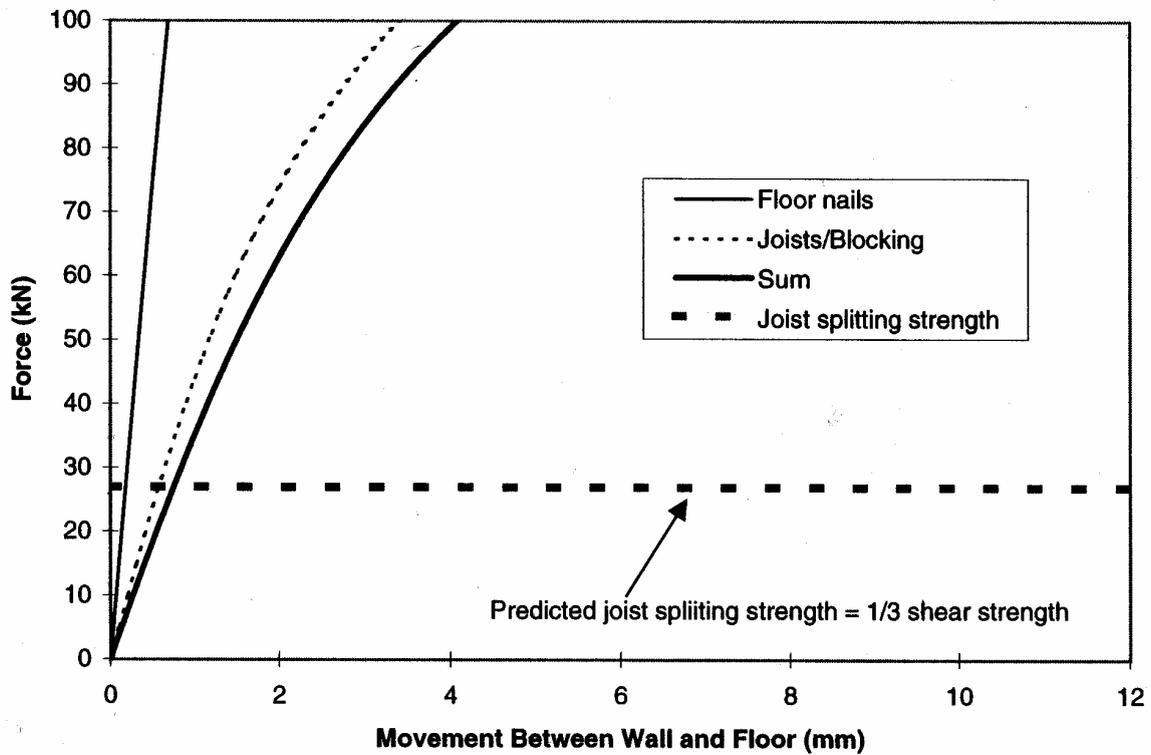


Figure A.9: Relationship Between Force and Movement – Force Perpendicular to Stringers.