STUDY REPORT

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Design of houses with vertical irregularity

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Preface

This report presents the results of a computer simulation study to quantify the extent that twisting will increase expected maximum wall deflections when New Zealand houses with vertical irregularity experience earthquakes. The effect of horizontal (plan) irregularity was investigated in BRANZ *Study Report* SR 171.

Acknowledgments

This work was funded by the Building Research Levy.

Note

This report is intended for standards committees, structural engineers, architects, designers and others researching earthquake resistance of houses.

Design of houses with vertical irregularity

BRANZ Study Report SR 210

SJ Thurston

Reference

Thurston SJ. 2009. 'Design of Houses with Vertical Irregularity'. BRANZ *Study Report 210*. BRANZ Ltd, Judgeford, New Zealand.

Abstract

This report presents the results of a computer simulation study to quantify the extent that torsion will increase wall deflections when New Zealand houses with vertical irregularity experience earthquakes.

Houses were modelled in a non-linear computer package and subjected to time-history earthquake loading. The floors and ceilings were modelled as rigid diaphragms. The walls were modelled as springs with load/deflection characteristics matched to wall test measurements. The maximum wall in-plane earthquake-induced deflections were plotted against the eccentricity of house mass.

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1. INTRODUCTION

Section 4.5.1 of NZS 1170.5 (SNZ 2004) states that a building shall be considered to have vertical irregularity when one of the following features apply:

- where the weight of any storey is more than 150% of the weight of an adjacent storey (weight irregularity)
- when the lateral stiffness of a storey is less than 70% of the stiffness of any adjacent storey, or less than 80% of the average stiffness of the three storeys above or below (stiffness irregularity). This type of irregularity leads to a soft-storey mechanism
- where a storey shear strength is less than 90% of that in the storey above (strength irregularity). This leads to a weak storey
- where the sum of the horizontal dimensions of the vertical elements of the primary structure in the direction under consideration in any storey is more than 130% of that in an adjacent storey (geometric irregularity).

The effect of vertical irregularity has been studied in depth by others (Chintanapakdee and Chopra (2004) and Pinto and Costa (1995)) for multi-storey construction. However, this study is only concerned with light timber-framed (LTF) low-rise buildings within the scope of NZS 3604 (SNZ 1999) called herein 'NZS 3604 type buildings'. These buildings are limited to a maximum of 10 m in height with a maximum of two storeys founded on piles or foundation walls with a part storey in a roof space being acceptable.

At design level earthquake loading, Thurston and Park (2004) showed that although standard NZS 3604 type buildings often experienced a soft-storey response vertical variations of mass, stiffness and strength did not give rise to excessive deflections. What was not examined, and what is the subject of this study, is the small subset of houses which 'step' up hillsides as shown in Figure 1. This particular structure is outside the scope of NZS 3604 as it is a three-storey structure on a piled foundation. However, as it is based on an actual building and illustrated the concepts being examined, it is used as the example analysed in this report.

The house shown in Figure 1 is built into the 'country' at two levels. This could be piles, retaining walls or concrete floor slabs. These are collectively referred to as 'hillside foundation elements' and may be very stiff under lateral load, thereby attracting large seismic forces and causing the house to 'twist'.

2. REPORT OUTLINE

This report presents the results of an analysis of a selected house which exhibits vertical irregularity using non-linear time-history earthquake computer simulation. Variables considered were eccentricity of the centre of mass at each floor level and pile stiffnesses (including 'hillside foundation piles'). It was found that deflections and forces were not sensitive to the mass eccentricity and the house showed little twist. This was attributed to the relatively uniform distribution of bracing resistances. However, forces in the 'hillside foundation elements' may be critical. To investigate this further additional computer runs were performed where the 'hillside foundation piles' were now modelled as being elastic and the variables considered were the elastic stiffness of these elements and the level of earthquake excitation.

The findings from the computer runs provided the basis for a proposed method for determining earthquake and wind design forces in such structures.

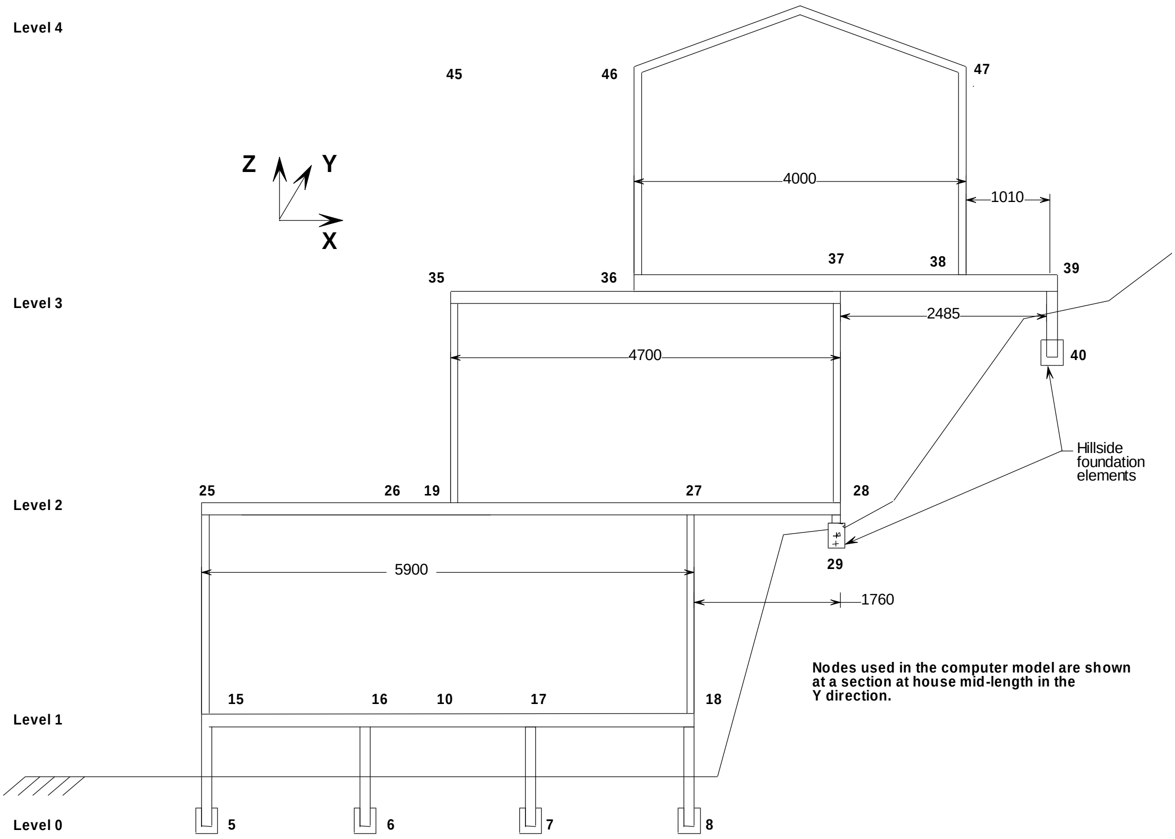

Figure 1. Section through analysed house

3. COMPUTER MODELLING

3.1 Computer model of 'idealised' house

Details of the model are given in Appendix A. The plan layout of walls and piles is implicit in the spring locations shown in Figure 9 to Figure 12 of Appendix A. Although an idealised layout of walls is used it provides a reasonable distribution of bracing strengths. All piles were given the same lateral stiffness properties, although in practice the hillside piles or hillside foundation elements may vary significantly from the base pile stiffnesses.

Initially the assumed pile and LTF shear wall load-versus deflection relationships were made non-linear to represent the performance of anchor piles measured by BRANZ. However, to determine the maximum load that can be transmitted to hillside foundation elements, some runs were made assuming a linear pile load-versus deflection relationship.

4. COMPUTED HOUSE DEFLECTIONS AND FORCES

Figure 2 plots the maximum horizontal deflection of the piles between Levels 0 to 1 versus mass eccentricity. As expected the deflection is very dependent on the pile stiffnesses. It is of interest to note that the deflections show little dependence on the mass eccentricity which implies the house is not susceptible to twist. This is considered to be due to the regular layout of piles and the resistance to twist provided by the hillside piles.

A plot of the maximum horizontal inter-storey deflection of the house walls in Figure 3 also shows the house deflections are also not susceptible to twist. This plot shows the greatest wall deflections generally occur when the piles are stiffer.

The greatest risk with houses like that in Figure 1 would appear to be that they would tear off their connection to the hillside foundation elements. Figure 4 and Figure 5 plot the maximum Y and X force respectively for the piles at Level 2 assuming a linear pile load-versus deflection relationship. Figure 6 to Figure 7 are similar plots but for Level 3, and Figure 9 is for forces in the bottom piles. Both the X and Y forces are sensitive to eccentricity and the pile shear forces that must be resisted are high. Up to a stiffness of 5400 kN/m the pile loads are sensitive to their stiffnesses and in this example the Y forces are greater than the X forces.

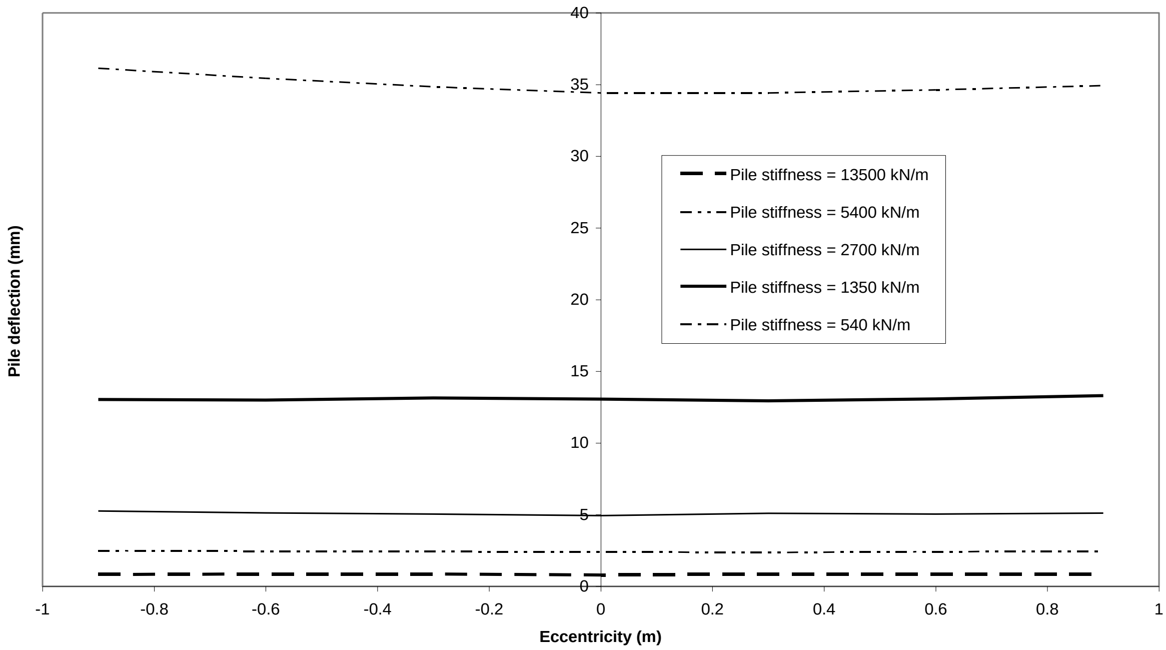

Figure 2. Deflection of piles between Levels 0 to 1 versus mass eccentricity

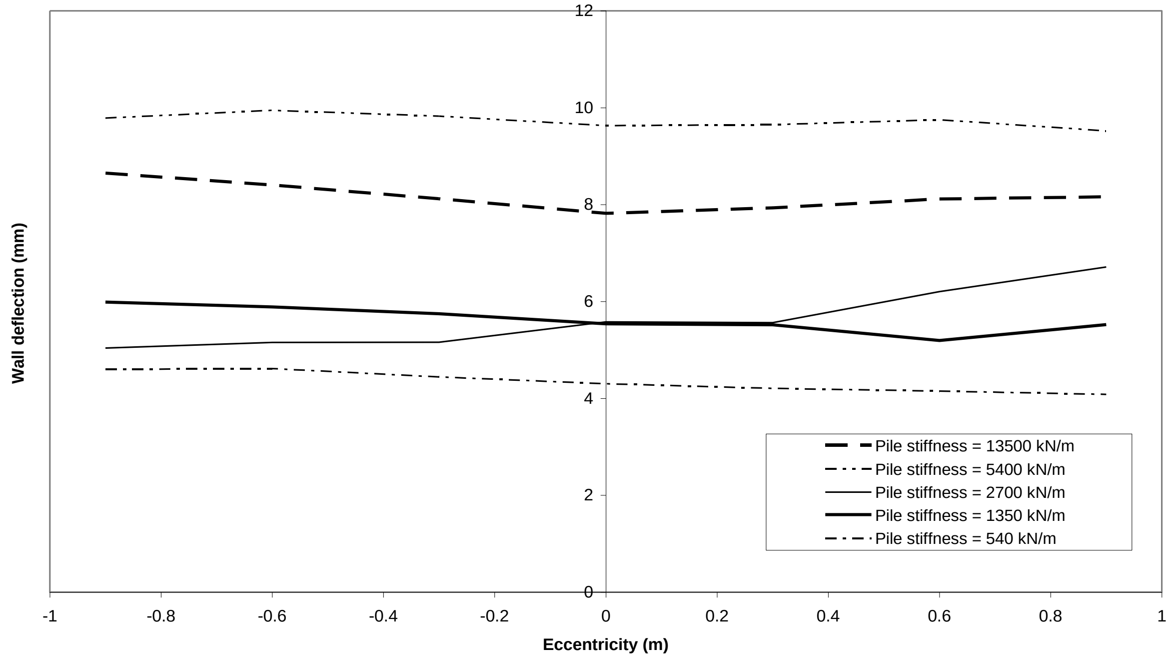

Figure 3. Maximum in-plane deflection of walls versus mass eccentricity

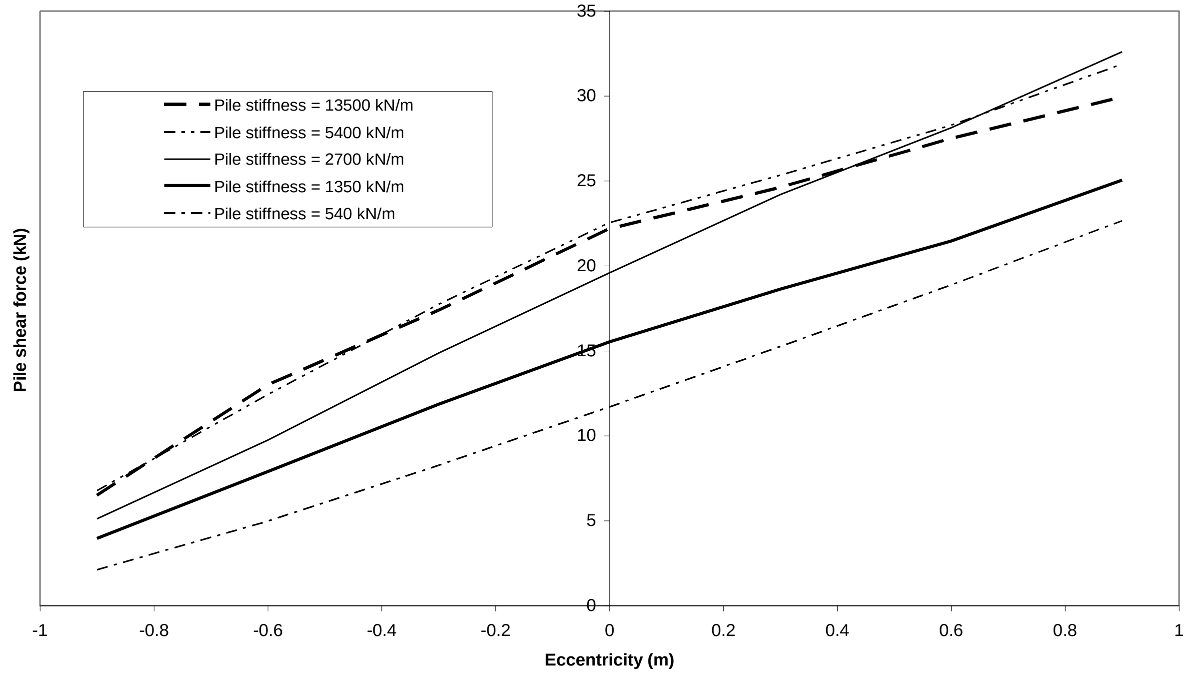

Figure 4. Maximum Y force in Level 2 piles versus mass eccentricity for elastic pile assumption

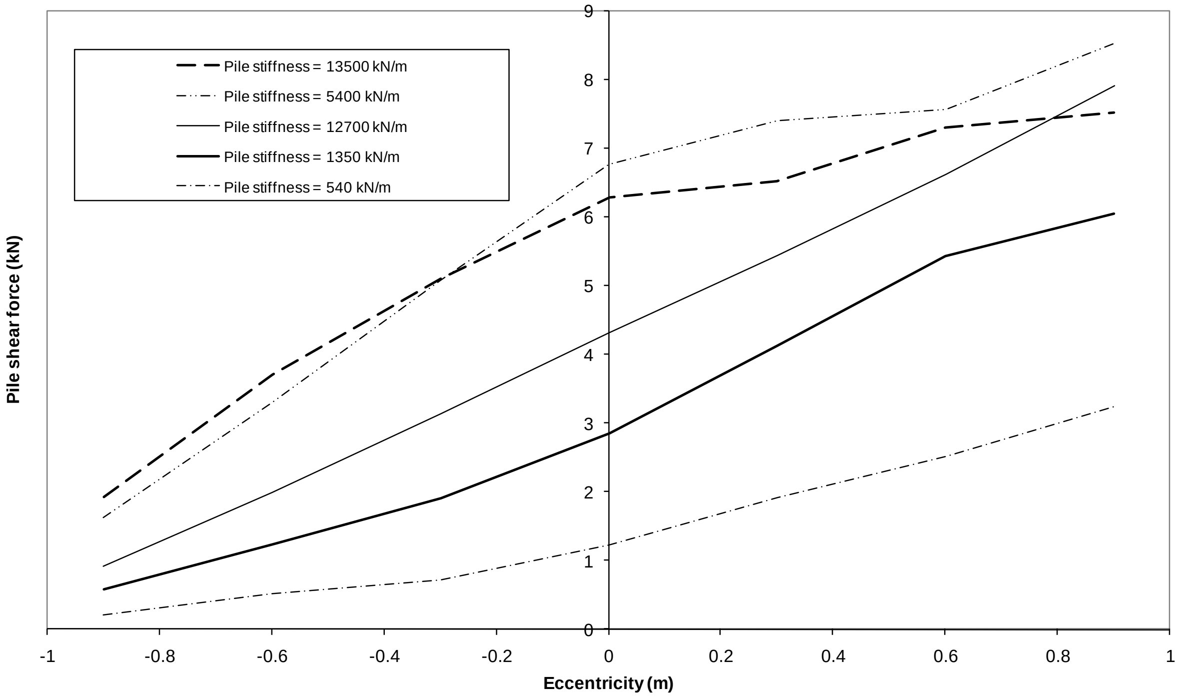

Figure 5. Maximum X force in Level 2 piles versus mass eccentricity for elastic pile assumption

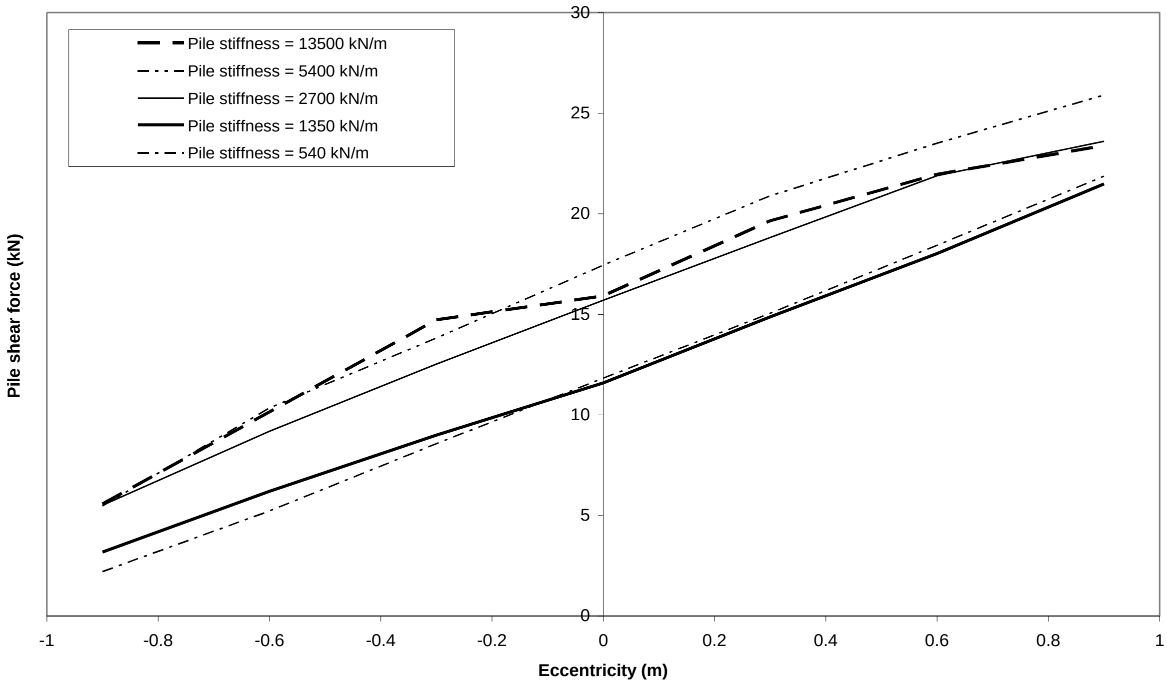

Figure 6. Maximum Y force in Level 3 piles versus mass eccentricity for elastic pile assumption

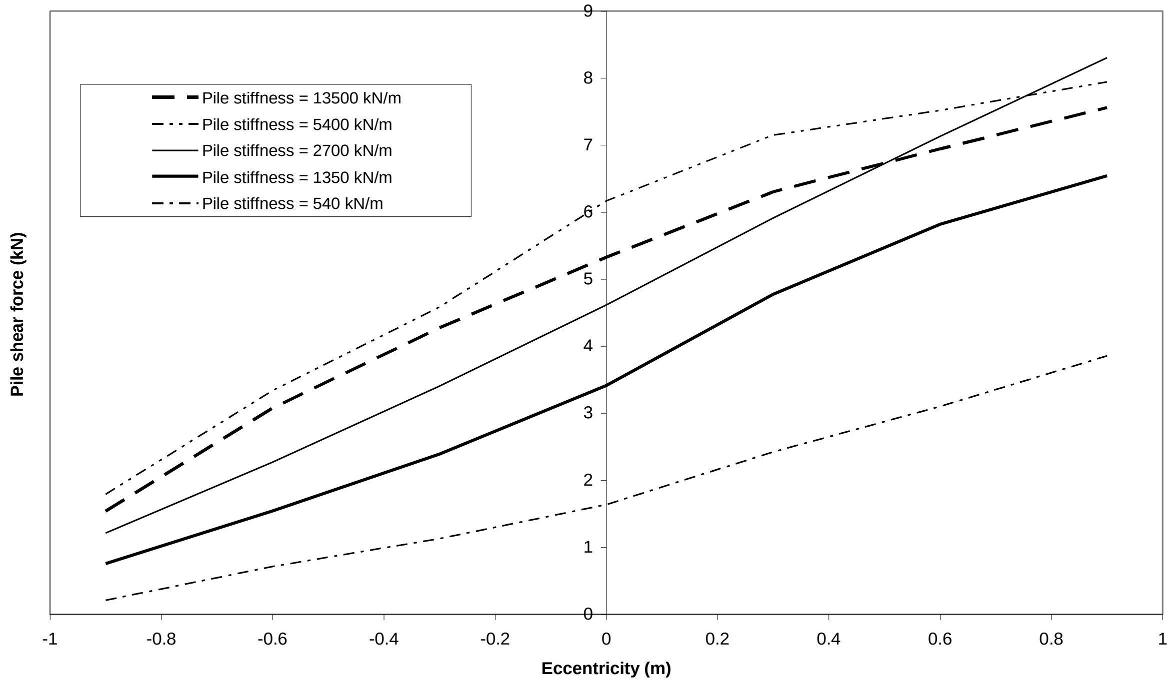

Figure 7. Maximum Y force in Level 3 piles versus mass eccentricity for elastic pile assumption

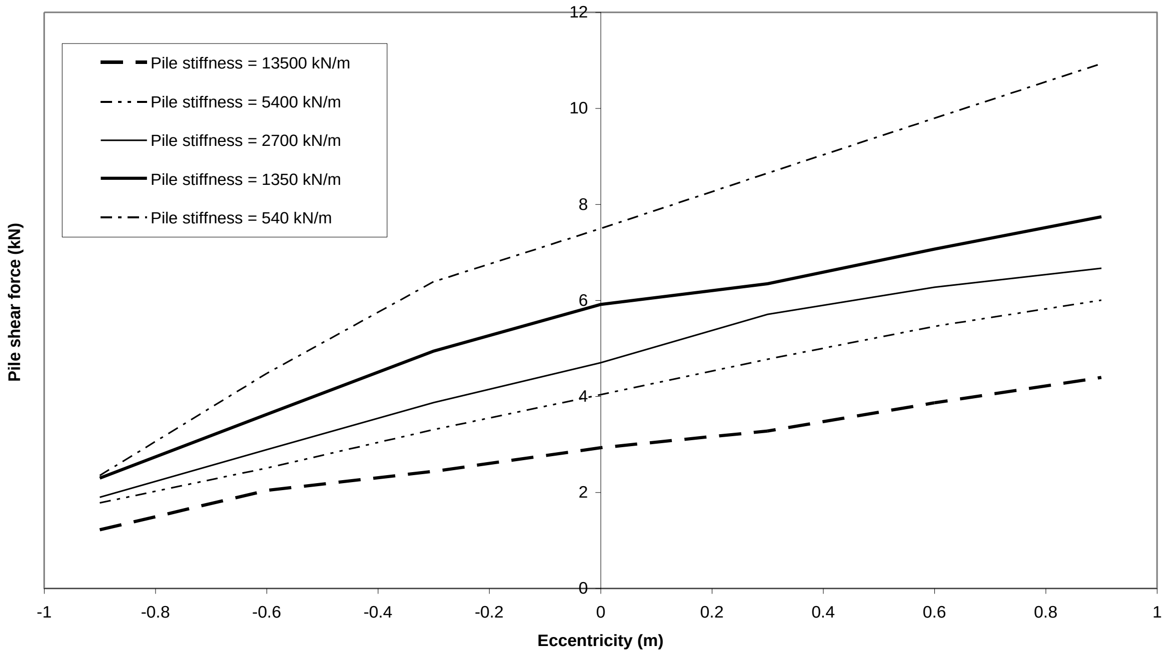

Figure 8. Maximum force in Level 0 to Level 1 piles versus eccentricity for elastic pile assumption

5. PROPOSED ANALYSIS METHOD

It is proposed that houses with vertical irregularity, such as shown in Figure 1, be designed using the methodology described in this section. This prescribes a method to determine:

(1) required house bracing wall demand loads

(2) required connection strength to 'hillside foundation elements'

(3) required strength of 'hillside foundation elements'.

It is imperative that items (2) and (3) do not fail in an earthquake and thus the design loads should assume a lower ductility than for failure of item (1).

It is recommended that NZS 3604 be amended to require houses with vertical irregularity to have the member and connection strengths specifically designed and that the method described in this section be cited as an Acceptable Solution.

5.1 Analysis method

It is proposed that a three-dimensional elastic static analysis be performed to determine the above forces which will vary from structure to structure. This may be reduced to a two-dimensional analysis if the 'pancake model' described in Appendix A is used. The minimum house member strengths are to be taken from the results of the analyses described below.

5.1.1 Building weights

The building weights to be used in the analysis may be taken from that assumed in NZS 3604 (Shelton 2007) and this is repeated below for completeness, including distribution of wall weights. Alternatively, the engineer may calculate the weights independently.

Application		***Dead load (kPa)***
Roof	*Light*	*0.2*
	Heavy	*0.6*
Ceiling		*0.24*
Walls	*Light*	*0.3*
	Medium	*0.8*
	Heavy	*2.2*
Partitions (based on floor area)		*0.2*
Floor		*0.6*

5.1.2 Bracing element stiffnesses

The effective stiffness of hillside bracing elements is a function of not only the stiffness of the elements but also the connection of the house to that element.

When designing a member or connection the forces used shall be based on the analysis assuming the maximum likely stiffnesses of that member group with all other members being assigned their minimum likely stiffnesses.

It is the responsibility of the designer to determine the appropriate stiffnesses to use. However, as a guide the following values are given below:

- Braced or anchor piles and connection between piles and superstructure. Maximum stiffness = 2000 kN/m per pile. Minimum stiffness = 300 kN/m per pile.
- Concrete and timber foundation walls including the connection between foundation walls and superstructure. Most of the flexibility is due to the connection. Maximum stiffness = 8000 kN/m per m length of wall. Minimum stiffness = 2000 kN/m per m length of wall.
- LTF bracing walls and connection between bracing walls and floor. Maximum stiffness = 2500 kN/m per m length of wall. Minimum stiffness = 500 kN/m per m length of wall.

5.1.3 Lateral load coefficients and distribution

NZS 3604 assumed a house ductility factor, μ, of 3.5, a period, T_1, of less than 0.45 s, and that the house was at an intermediate soil site. A basic seismic hazard coefficient of $C_h(T_1,\mu) = 0.3$ was obtained from Table 4.6.1 of NZS 4203 (SNZ 1992).

Using a seismic performance factor, S_p, of 0.67, a risk factor, R, of 1.0, and a limit state factor, L_u, of 1.0, the lateral force coefficient, C, for the relevant seismic zones was tabulated by Shelton (2007) as:

Zone	Zone factor Z	Lateral force coefficient C
A	1.2	0.241
B	0.9	0.181
C	0.6	0.121

However, if the designer considers that hillside bracing elements have low ductility, then it may be more prudent to assume that $\mu = 1.25$.

The effective live load contributing can be taken as (= $\psi_a \times \psi_u$) = 0.134.

The vertical distribution between levels used the provisions of NZS 4203 (SNZ 1992) Clause 4.8:

$$F_1 = 0.92 \cdot Vx\left(\frac{W_i \cdot h_i}{\sum (W_i \cdot h_i)}\right)$$

6. REFERENCES

(1) Carr AJ. 2000. *Ruaumoko: Computer Program User Manual*. University of Canterbury, Christchurch, New Zealand.

(2) Chintanapakdee C and Chopra A. (2004). 'Seismic Response of Vertically Irregular Frames: Response History and Modal Pushover Analyses'. *Journal of Structural Engineering* 130(8): 1177-1185.

(3) Pinto D and Costa AG. (1995). 'Influence of Vertical Irregularities on Seismic Response of Buildings'. *Proceedings of 10th European Conference on Earthquake Engineering*. Balkema, Rotterdan, Holland.

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(7) Standards New Zealand. 2004. NZS 1170.5 *Earthquake Design Actions – New Zealand*. SNZ, Wellington, New Zealand.

(8) Stewart WG. 1987. *The Seismic Design of Shear Walls*. PhD Thesis, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

(9) Thurston SJ. 2001. 'Pseudo-dynamic Evaluation of Timber-framed Walls'. BRANZ *Study Report 109*. BRANZ Ltd, Judgeford, New Zealand.

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APPENDIX A COMPUTER MODEL USED

A.1 General description of computer model of analysed house

A computer model of the analysed house was examined using inelastic time-history computer runs with the well-known Ruaumoko (Carr 2000) software.

A section through the analysed house is shown in Figure 1. Each floor is a rectangular shape with the long side being 12 m. The building rests on 12 anchor piles on the building perimeter, with four on each side at 4 m centres and four at each end at 2 m centres. Internal piles resist only vertical load (i.e. not horizontal load) whereas the other piles resist both vertical and horizontal load.

A schematic depiction of the computer model of the analysed house is shown in Figure 9 to Figure 12. Each floor consisted of a 'cruciform' shape of very stiff members (representing a rigid floor/ceiling diaphragm) connected by springs to the floor above, the floor below or to ground supports. However, for clarity the springs to the floor above are not shown as these are shown in the floor above.

Examples of springs in model

- Spring 7-17 in Figure 9 is a spring between Point 17 on the cruciform of Level 1 to Point 7 on the ground. It represents the Y direction stiffness of the two end piles which have this X location. The ground is represented as a fully fixed support.
- Spring 1-11 in Figure 9 is a spring between Point 11 on the cruciform of Level 1 to Point 1 on the ground. It represents the X direction stiffness of the 4 end piles which have this Y location.
- Spring 22-12 in Figure 10 is a spring between Point 22 on the plan of Level 2 to Point 12 on the plan of Level 1. It represents the X direction stiffness of the internal wall between Level 1 and 2 which has this Y location.
- Spring 28-29 in Figure 10 is a spring between Point 28 on the cruciform of Level 2 to Point 29 on the ground. It represents the accumulated Y direction stiffnesses of the four 'hillside foundation piles' connecting Level 2 to the ground.
- Spring 22-50 in Figure 10 is a spring between Point 22 on the cruciform of Level 2 to Point 50 on the ground. It represents the X direction stiffness of the single 'hillside foundation pile' connecting Level 2 to the ground at this Y location.

The floor diaphragms are given the same 'Z' coordinate which simplifies the problem to a 'two degrees of freedom model'. This is the well-known 'pancake model' technique. A wall can therefore be represented as a horizontal spring being in the direction of the wall. An anchor pile can is represented as two horizontal springs, one in the X direction to represent the X direction stiffness of the pile and one in the Y direction to represent the Y direction stiffness of the pile. However, the software requires the springs to be of finite length. To meet this requirement the origin of the cruciform at Levels 2 and 4 was located at (-0.5 m, 0.5 m) whereas the origin at other levels was at (0,0).

Each floor mass was connected to the cruciform of that floor by a very stiff member and its location was varied between runs to investigate the relationship between house twist and eccentricity of house centre of mass. The mass was assigned a rotational inertia, I_r, which was calculated assuming the mass is uniformly distributed over the plan floor area. Thurston (2001) calculated this rotational inertia as:

$I_r = M(D^2 + B^2)/12$ where M = floor mass and D and B are the plan dimensions of the floor.

A.2 Hysteresis elements used in the computer model

The walls and piles were represented as springs and the cruciforms as very stiff beams. The springs were modelled using the Stewart (1987) hysteresis element shown in Figure 13. Parameters used are listed in Table 1. Values of F_u used for walls and K_0 for piles are defined for in Section A.3. The value of stiffness, K_0, used for walls was based on examination of BRANZ P21 tests of various plasterboard wall systems.

Table 1. Stewart hysteresis element spring properties

Spring property	Value assigned	
	Wall	**Pile**
Ultimate spring axial strength representing wall shear strength	F_u (defined in Section A.3)	6 kN
Spring initial stiffness K_0 (kN/m)	180 x F_u	90 x F_u
Secondary slope	$0.21K_0$	$0.4K_0$
Tertiary slope	Zero	Zero
Unloading slope K_u	1.3 K_0	1.3 K_0
Yield F_y	$0.58F_u$	$0.3F_u$
Intercept F_I	$F_u/6$	$F_u/8$
ALPHA	1.09	1.09
BETA	0.38	0.38

The shape of the hysteresis loops defined by Table 1 are plotted in Figure 14 for walls and are compared with test results for a 1.2 m long wall lined on both sides with standard 10 mm thick plasterboard.

The line joining the first cycle peak loads is herein called the 'parent' curve and the line joining the second and subsequent cycle peak loads is called the 'residual' curve. The 'parent' curve for the model hysteresis element shown in Figure 14 reached peak load at 14 mm deflection and retained this load at higher deflections, and in this respect slightly departed from the test specimen shown. On the other hand the 'residual' curve closely followed the test specimen plot.

The values in Table 1 used for piles were based on the shapes of the hysteresis loops defined by Thurston (SR 58). This used Fu = 6 kN and K_0 = 540 kN/m.

A.3 Properties assigned to walls and piles in the computer model

A.3.1 Member ultimate strengths (Fu)

The ultimate strength, Fu, of the piles and walls used in the computer analysis is summarised in Table 2. The basis for these numbers is described below.

(a) Piles

The external piles between Level 1 and the ground were all assumed to have a strength of 6 kN in both the X and Y directions. As there was 12 external piles the total strength in both directions was 72 kN.

(b) Walls Level 1-2

The house walls aligned in the Y direction were assumed to have an average bracing strength, Fu, of 2.8 kN/m over their entire length which is the same as assuming the walls had an average bracing strength of 4.0 kN/m and 30% of the walls were gaps (i.e. windows or doorway openings). As the two external walls in the Y direction were 12 m long, the house yield bracing strength was Fu = 12 x 2 x 2.8 = 67.2 kN.

In the X direction the two external walls are 6 m long giving a total of 6 x 2 x 2.8 = 33.6 kN. Then two internal walls were each taken as having a strength of 14 kN giving a total strength in the X direction of 61.6 kN.

(c) Walls above Level 2

- The walls between Levels 2 and 3 were given 80% of the strength of walls in Levels 1-2.
- The walls between Levels 3 and 4 were given 64% of the strength of walls in Levels 1-2.

Table 2. Earthquake resistances

Level	**Resistances (kN)**	
	Y direction	**X direction**
Piles 0-1	72	72
Walls 1-2	67.2	61.6
Walls 2-3	53.8	49.3
Walls 3-4	43.0	39.4

A.3.2 Weights

The dead plus live load weights assumed at each level is summarised in Table 3. These were calculated by assuming the house had a lightweight roof and wall cladding and a floor dead plus live load of 0.97 kPa.

Table 3. Weights assumed at each level

Level	**Weight (kN)**
1	85.9
2	79.3
3	69.9
4	37.8
Total	273.0

Taking the seismic load as 0.241 W as assumed by NZS 3604 for earthquake Zone A (Shelton) and using the distribution formula assumed by NZS 3604 (Shelton) the design shear load is calculated in Table 4 assuming no lateral load is removed from the structure at the piled connection to the country. This shear is compared to the strength provided and it can be seen that this strength just exceeds demand in the lower levels but there is significant reserve at Level 4.

Table 4. Lateral load distribution

						Ratio with strength	
Level	W	h	Wh	V	Shear (kN)	Y direction	X direction
1	85.9	0.8	68.8	3.8	65.8	0.91	0.91
2	79.3	3.4	269.7	15.1	62.0	0.92	1.01
3	69.9	6	419.6	23.4	46.9	0.87	0.95
4	37.8	8.6	325.1	23.4	23.4	0.54	0.59
sum=	**273.0**		**1083.2**	**65.8**			

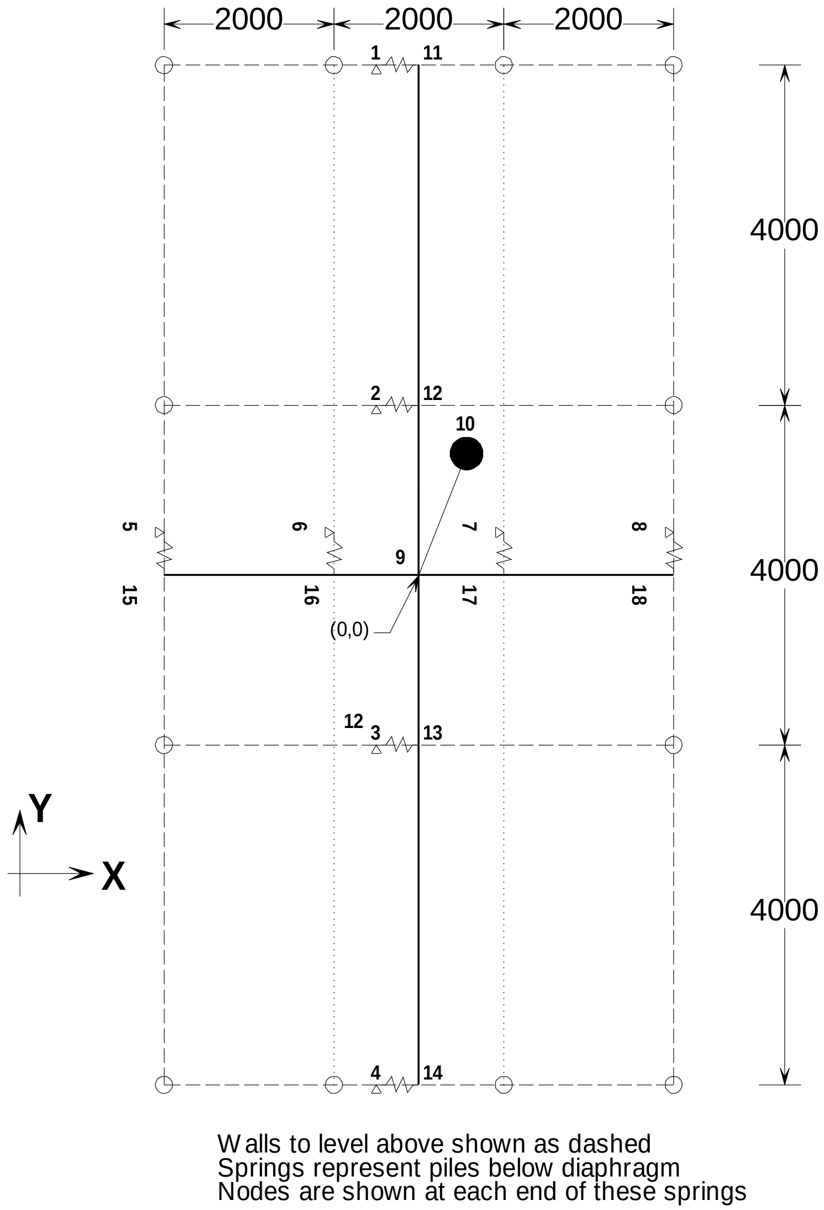


Figure 9. Computer model for Level 1 diaphragm

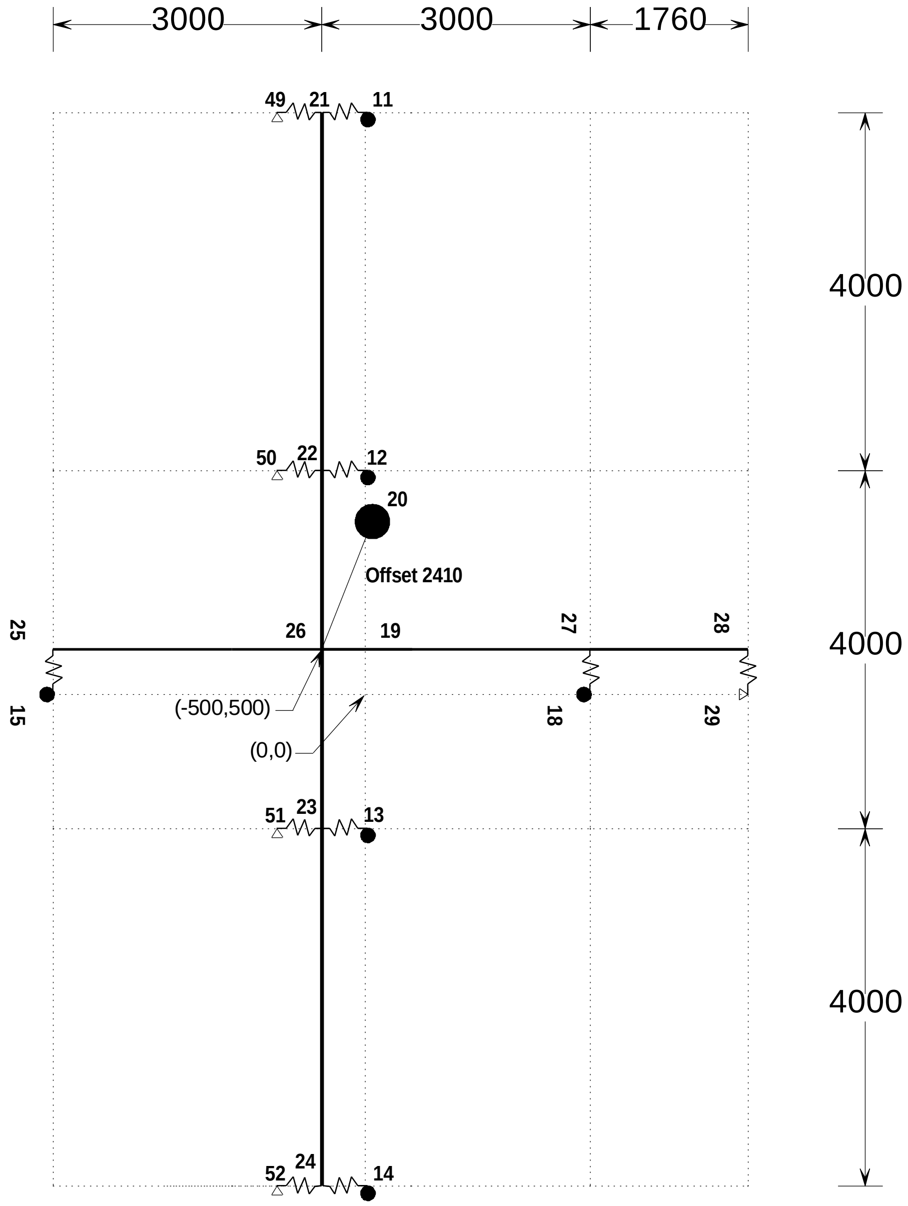
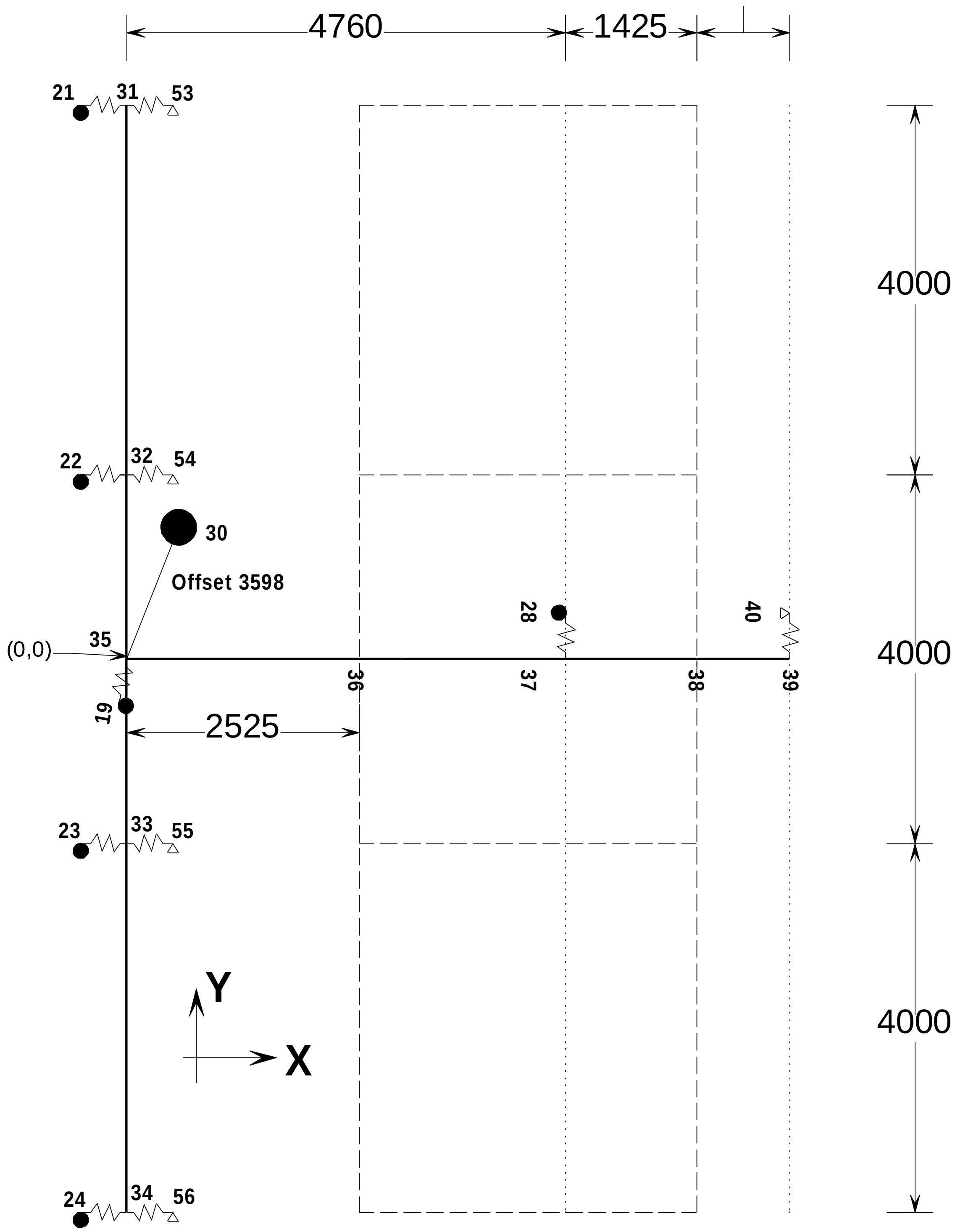


Figure 10. Computer model for Level 2 diaphragm



Walls to the level above shown are shown as dashed lines
Springs with a circle at one end represent walls to level below
Springs with a triangle at one end represent "hillside" piles
Node numbers are shown at each end of the springs

Figure 11. Computer model for Level 3 diaphragm

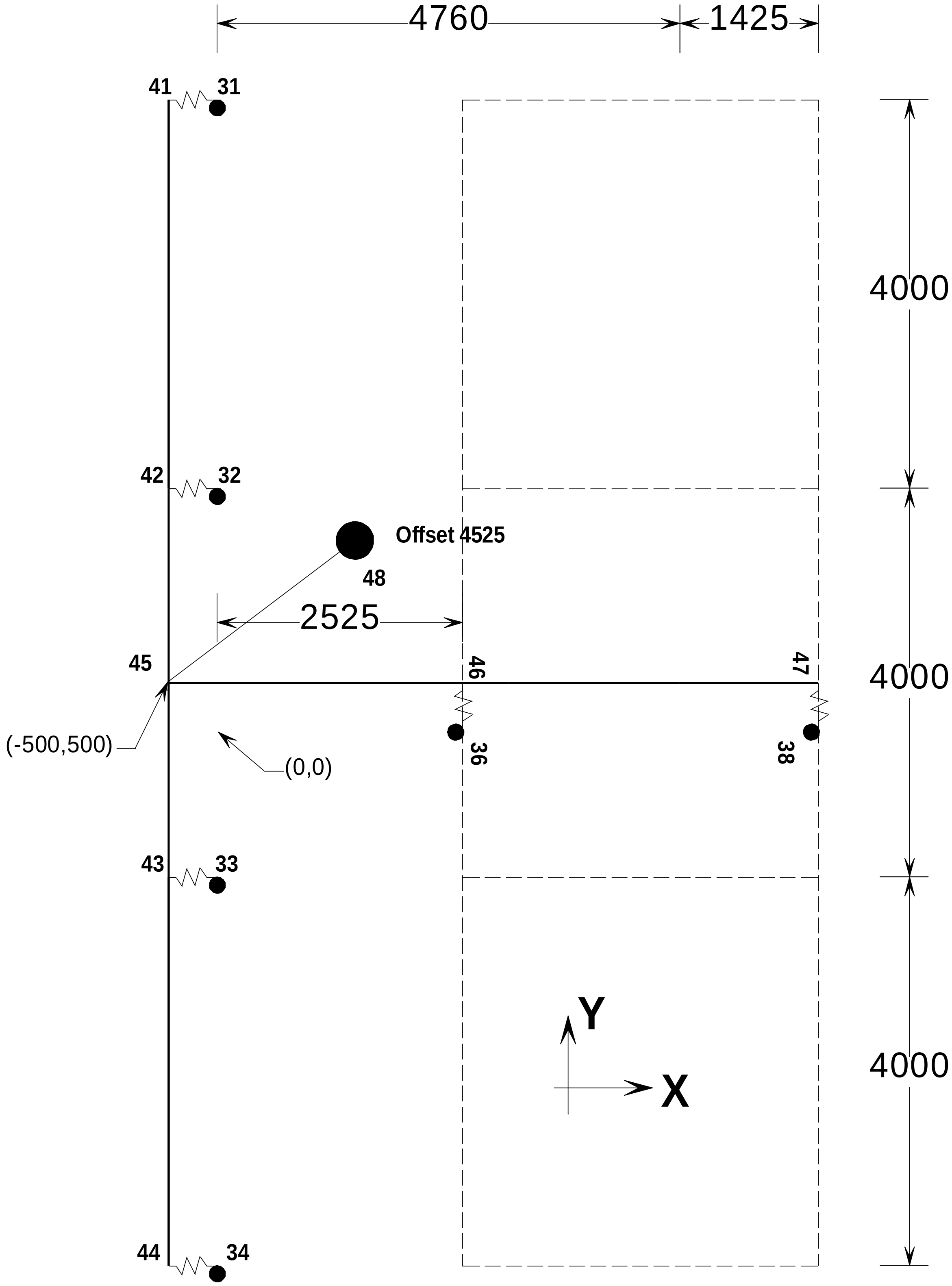


Figure 12. Computer model for Level 4 diaphragm

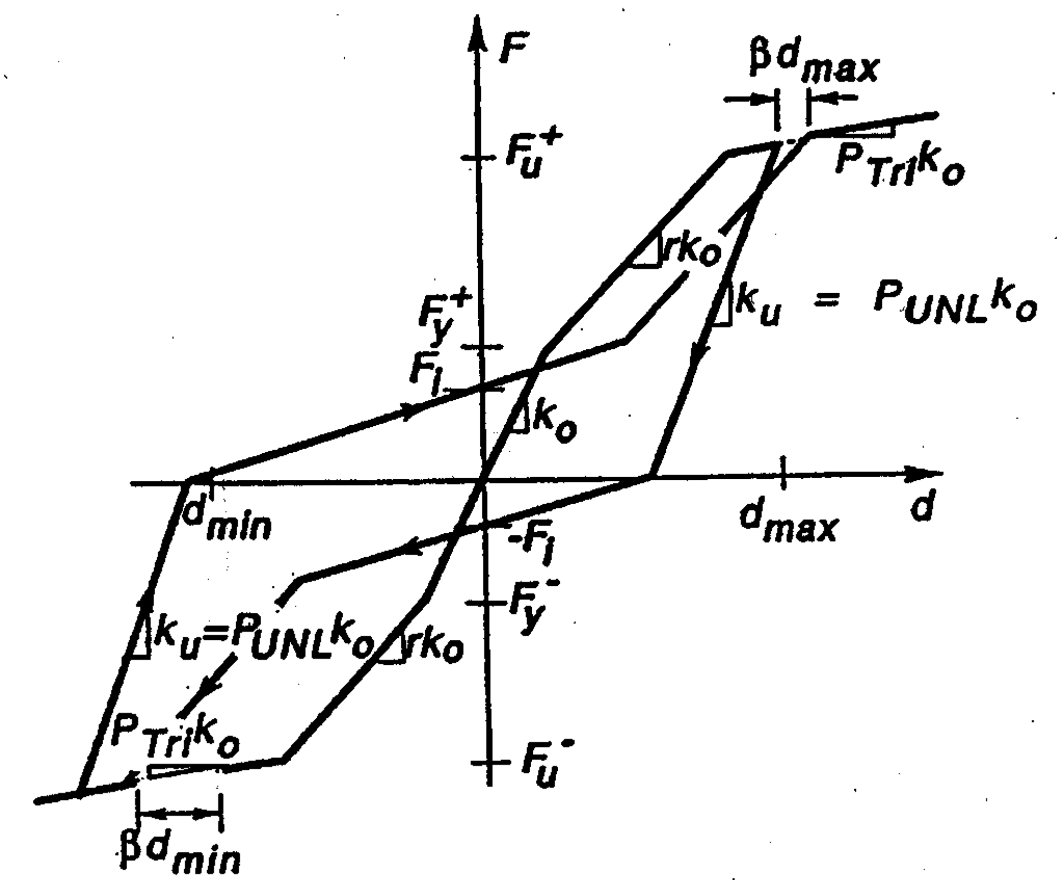


Figure 13. Terminology for Stewart hysteresis element

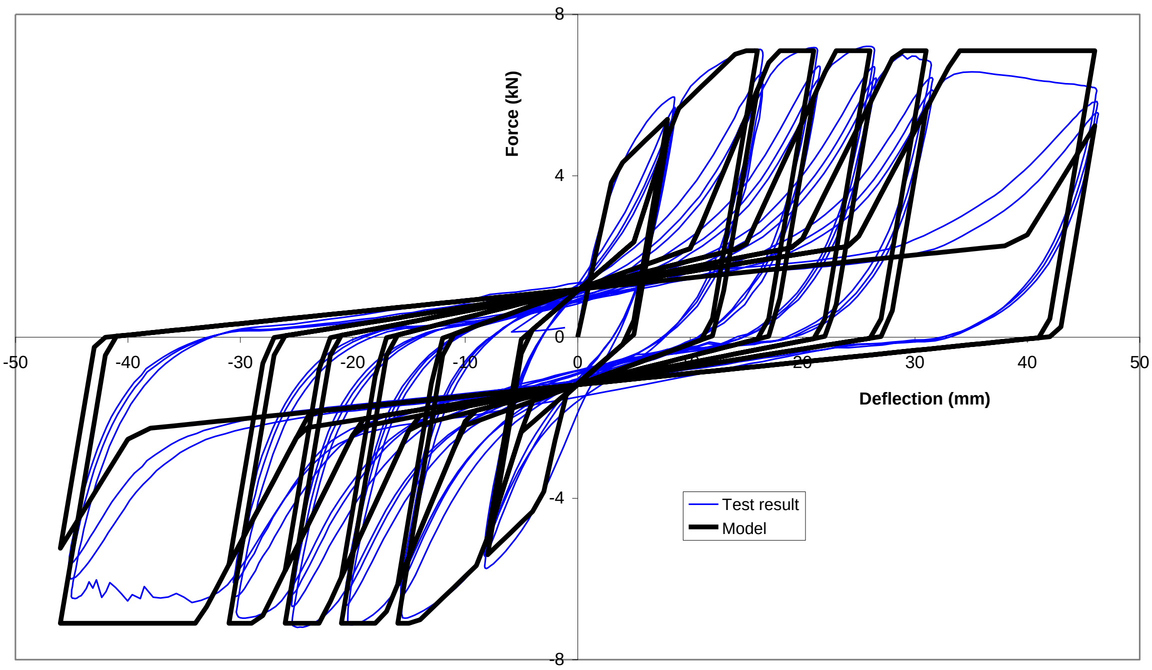


Figure 14. Comparison of Stewart hysteresis model and wall test data