

ENG 351-(EQC 2006/517)

Earthquake resistant design of tied-back retaining structures

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**EARTHQUAKE RESISTANT DESIGN OF
TIED-BACK RETAINING STRUCTURES**

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ABSTRACT

This report considers design procedures for tied-back retaining walls under earthquake loading. Tied-back retaining walls are becoming widely used in NZ to support permanent excavations on sloping sites in order to provide level building platforms for residential and commercial developments. They are also widely used to support excavations for roadways and other key infrastructure.

Very little guidance is available for the design of tied-back retaining walls to resist earthquake shaking. Little observational data on the behaviour of tied-back walls during earthquakes has been published, but, what there is suggests that they behave well.

A survey of New Zealand practice has showed that there is no consistency of approach and that most designers are relying on a range of different "black box" computer software with earthquake loading input simply as an additional horizontal force applied directly to the wall. The appropriateness of this approach is questionable because the full range of different failure modes is not necessarily addressed by the software nor is it always obvious what the software does.

In this study, a seismic design procedure for tied-back retaining walls was synthesized based on an existing, widely used, semi-empirical design procedure for gravity design of tied-back walls. The design procedure does not depend on specialist computer software.

The design procedure was tested by designing a range of case study walls and then subjecting them to simulated earthquakes by numerical time-history analysis using PLAXIS finite element software for soil and rock. The response of the walls to a variety of real earthquake records was measured including deformations, wall bending moments, and anchor forces.

From the results of these analyses, it was observed that all of the wall designs were robust and performed very well, including those designed only to resist gravity loads. In some cases large permanent deformations were observed (up to 400 mm) but these were for very large earthquakes (scaled peak ground acceleration of 0.6 g). In all cases the walls remained stable with anchor forces safely below ultimate tensile strength. Wall bending moments reached yield in some cases for the extreme earthquakes, but this is considered acceptable provided the wall elements are detailed for ductility.

Walls designed to resist low levels of horizontal acceleration (0.1 g and 0.2 g) showed significant improvements in performance over gravity only designs in terms of permanent displacement for relatively modest increases in cost. Walls designed to resist higher levels of horizontal acceleration (0.3 g and 0.4 g) showed additional improvements in performance but at much greater increases in cost.

Even when walls were designed to resist 100 percent of the peak ground acceleration of a particular earthquake record, significant permanent deformations were still observed.

A tentative, detailed design procedure is provided based on the results of the study.

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Support of the University of Canterbury, Department of Civil Engineering is also gratefully acknowledged, especially for access to library facilities and for the Senior Fellowship of the Principal Investigator.

DISCLAIMER

This report describes a research project carried out into the behaviour of tied-back retaining walls under seismic loading. The conclusions and recommendations contained within this report are based on a limited investigation as described in detail in the report. Pacific Geotech Limited and the Principal Investigator do not make any representations, express or implied, as to the accuracy, completeness, or appropriateness for use in any particular circumstances, of any of the information provided, requirements identified, or recommendations made in this report. Pacific Geotech Limited and the Principal Investigator do not accept any responsibility for the use or application of, or reliance on any procedures or other information, for any purpose or reason whatsoever.

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1 Introduction

Kramer [1996] has summarised the limited research available on this topic. Very few reports of the behaviour of tied back walls during earthquakes are available. Ho et. al. [1990] surveyed ten anchored walls in the Los Angeles area following the Whittier earthquake of 1987 and concluded that they performed very well with little or no loss of integrity.

Numerical analyses of tied-back walls have been performed by Siller and Frawley [1992] and Siller and Dolly [1992] who found that walls with stiff, more closely spaced anchors develop smaller and more uniform permanent displacements than walls with softer anchors and greater vertical spacing of anchors. Walls designed for higher static earth pressures were also found to develop smaller permanent displacement than walls designed to lower static pressures. Walls with higher initial anchor preloads were found to develop smaller permanent displacements than walls with lower preloads.

Fragaszy et. al. [1987] found that wall elements that extend into the foundation soils may be subjected to very high bending moments at the base because of phase differences in movements between the top and bottom of the wall. Inclined anchors extending below the base of the excavation may become highly stressed when the bonded end of the anchor embedded in soil moves out of phase with the wall face.

Detailed design guidance has been provided by Sabatini et. al. [1999] within a general design manual for tied-back walls prepared for the US Department of Transportation, Federal Highway Administration. This manual is in wide use within the US and is gaining increasing acceptance within New Zealand. They recommend the use of the pseudo-static so called Mononobe-Okabe method [Okabe, 1926; Mononobe and Matsuo, 1929] to calculate earthquake induced active earth pressures acting against the back face of a tied-back wall. A seismic coefficient from between one-half to two-thirds of the peak horizontal ground acceleration (0.5 PGA to 0.67 PGA) is recommended to provide a wall design that will limit deformations to small values acceptable for highway facilities.

Sabatini et. al. [1999] recommends that brittle elements of the wall system (the grout/tendon bond) should be governed by the peak ground acceleration “adjusted to account for the effect of local soil conditions and the geometry of the wall” and a factor of safety of 1.1 applied. Design of ductile elements, including the tendon, should be governed by the cumulative permanent seismic deformation. They recommend that, based on studies using Newmark type sliding wedge analyses, ductile elements should be designed using forces calculated by pseudo-static analysis using a seismic coefficient of 0.5 PGA with a factor of safety of 1.1 applied. The length of the ground anchors may need to be increased beyond that calculated for static design with the anchor bond zone located outside of the Mononobe-Okabe active wedge of soil.

The use of the Mononobe-Okabe method to calculate earth pressure for design of tied-back walls has the advantage of being straightforward and is widely used for design of gravity retaining walls. However, it is based on limiting equilibrium and the

development of an active failure wedge of soil that is at odds with the design procedure for static loads for tied-back walls. The recommendation to place the bond zone of the anchors behind the active soil wedge means that the wall is not free to move with the wedge, as assumed by the Mononobe-Okabe procedure.

1.1 Overview

This project has studied the performance of tied-back retaining walls by use of numerical time-history analysis using PLAXIS finite element software for soil and rock [Brinkgreve & Vermeer, 1988]. Too few field studies from actual earthquakes are available to make meaningful conclusions and testing of scaled down models on a shaking table is of limited use because of the impossibility of satisfying scaling laws without increasing the gravity field in a centrifuge. Numerical analysis of problems in geomechanics has become a recognised tool for exploring soil-structure interaction problems and is probably the only practical way to investigate the complexity of tied-back wall behaviour during earthquake shaking.

The project has focussed on developing a rational and practical design procedure then verifying the procedure by considering different case studies of tied-back walls. The case study walls were designed using the proposed procedure and then subjected to different earthquake time-histories using PLAXIS. The performance of each wall design was assessed for each earthquake by monitoring various key parameters including displacement, wall bending moments, and anchor forces.

After assessing the performance of the various wall designs, the proposed design procedure was critically assessed and final guidelines and recommendations made.

Every wall design case in practice is different in some way from every previous design. It was impossible within the constraints of time and budget to consider every possible wall circumstance. Instead, the case studies were based on the simple, case of a deep uniform sand soil deposit with suitably generic properties. This simplification is both necessary and desirable because it allows the basic trends in wall performance to be observed without “clutter” from a myriad of different parameters.

At the commencement of the project a survey was undertaken to identify available published design procedures and to identify current New Zealand practice. This information was used to identify the most rational design procedure and to clarify and refine such a procedure as necessary. The case study designs and analyses then were undertaken to prove or otherwise the efficacy and safety of the design procedure.

2 Design Procedures

2.1 Overview

Tied-back retaining walls were used originally as a substitute for braced retaining walls in deep excavations. Ground anchor tie-backs were used to replace bracing struts that caused congestion and construction difficulty within the excavation. Design procedures evolved from those developed for braced excavations and are typically based on the so-called “apparent earth pressure” diagrams of Terzaghi and Peck [1967] and Peck [1969]. These diagrams were developed empirically from measurements of loads imposed on bracing struts during deep excavations in sands in Berlin, Munich, and New York; in soft to medium insensitive glacial clays in Chicago; and in soft to medium insensitive marine clays in Oslo.

These original “apparent earth pressure diagrams” were not intended by the authors to be a realistic representation of actual earth pressures against a wall but to be “...merely an artifice for calculating values of the strut loads that will not be exceeded in any real strut in a similar open cut. In general, the bending moments in the sheeting or soldier piles, and in wales and lagging, will be substantially smaller than those calculated from the apparent earth pressure diagram suggested for determining strut loads.”[Terzaghi & Peck, 1967].

Since 1969, remarkably few significant modifications to this original work have been adopted in practice. More recently, Sabatini et. al. [1999] proposed a more detailed design procedure based on the apparent earth pressure approach intended specifically for pre-tensioned, tied-back retaining walls in a comprehensive manual prepared for the US Department of Transportation, Federal Highway Administration. This manual is in wide use within the US and is gaining increasing acceptance within New Zealand.

A detailed and well proven design procedure for walls under gravity loading is given in this manual which will be referred to throughout this report as the “FHWA procedure”. The manual also makes suggestions for design of tied-back walls to resist earthquake loading although a detailed procedure is not given.

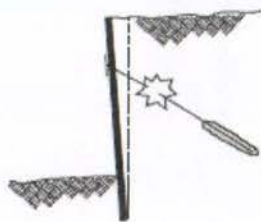
Increasingly, practitioners are relying on computer “black box” software to design tied-back walls with methodologies that range from fully elastic “beam-on-elastic-foundation” approaches to limiting equilibrium approaches. Caution is required when using “black box” software to ensure that all possible failure modes have been considered.

2.2 Gravity Design

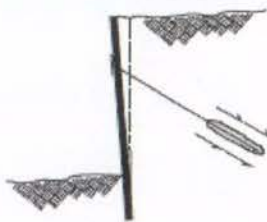
2.2.1 Possible modes of failure

Possible modes of failure for tied-back retaining walls are illustrated in cartoon fashion in Figure 2.2.1 (a). A complete design procedure needs to address each of these modes of failure

- a) **Tensile failure of tendon:** The range of tendon loads must be established with suitable margins for safety.
- b) **Grout/ground bond failure:** Generally this should always be established on site by proof testing given the difficulty in predicting the capacity and the dependence on installer skill and technique.
- c) **Tendon/grout bond failure:** Prevented by reference to proven/commercial anchor details.
- d) **Wall bending failure:** Actual wall moments are very difficult to predict because of the interaction between soil and structure stiffness and the non-linearity of soil stiffness. However, wall hinging does not necessarily create a mechanism provided the wall element is ductile.
- e) **Passive failure at foot of wall:** Insufficient embedment depth for continuous walls or soldier piles leads to passive failure of the soil immediately in front of the wall and instability of the wall and soil mass.
- f) **Forward rotation of wall:** Staging of excavation is necessary to prevent forward rotation of wall prior to anchor installation. Wall needs sufficient bending strength to resist cantilever moments for staged excavation. Anchors need to be of sufficient capacity and length to prevent forward rotation.
- g) **Bearing failure underneath wall:** Caused by downwards component of anchor force. Check axial capacity of soldier piles, or, bearing capacity of foot of continuous wall. Bearing loads may be reduced by reducing the anchor inclination as much as possible (15 degrees is a practical minimum).
- h) **Failure by overturning:** Essentially same as (f). Anchors need to be of sufficient capacity and length to prevent forward rotation.
- i) **Failure by sliding:** Possible mode for cohesionless soils. Factor of safety controlled by increasing depth of embedment of wall and/or soldier piles. Factor of safety calculated using limiting equilibrium "wedge" analysis.
- j) **Failure by rotation:** Possible mode for cohesive soils. Factor of safety controlled by increasing depth of embedment of wall and/or soldier piles. Factor of safety calculated using limiting equilibrium "Bishop" analysis or similar.



(a) Tensile failure of tendon



(b) Pullout failure of grout/ground bond



(c) Pullout failure of tendon/grout bond



(d) Failure of wall in bending



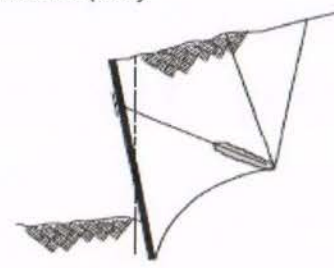
(e) Failure of wall due to insufficient passive capacity



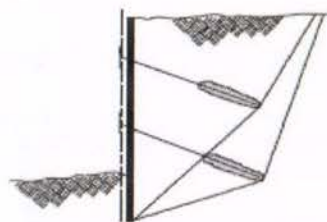
(f) Failure by forward rotation (cantilever before first anchor installed)



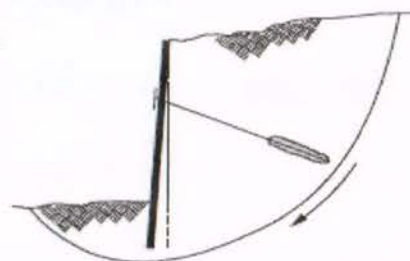
(g) Failure due to insufficient axial capacity



(h) Failure by overturning



(i) Failure by sliding



(j) Rotational failure of ground mass

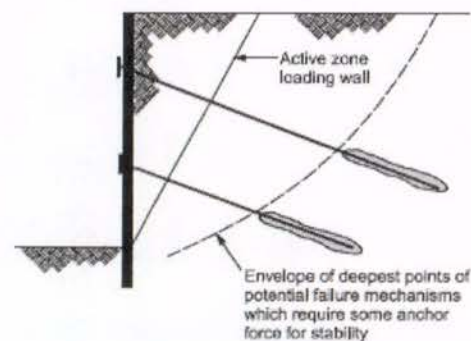
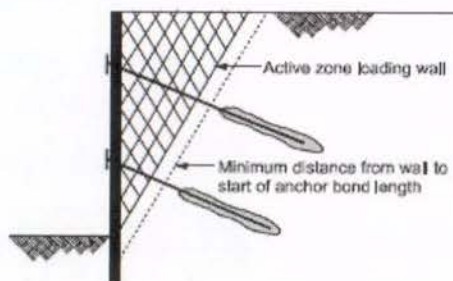


Figure 2.2.1 (a) Possible modes of failure for tied-back retaining walls [Sabatini et al., 1999].

2.2.2 Design procedure for sand

The following procedure addresses each of the above failure modes systematically (for the gravity load case) and is based on the FHWA procedure with minor modifications and clarifications where noted. It is assumed herein that the wall and retained soil are fully drained. This procedure is intended to be readily calculated by hand, although use of calculation software such as Mathcad or Excel will be useful for design iterations. Example calculations using Mathcad for the case studies are include in the appendices.

- a) **Initial trial geometry:** The depth of excavation and depth to each row of anchors needs to be estimated as a first step, based on experience or trial and error. Typically, for stronger soils, the first row will be at a depth of 2 m with subsequent rows at 5 m intervals.

- b) **Prepare apparent earth pressure diagram:** As shown in Figure 2.2.2 (a).

Note that K_A is calculated as follows: $K_A = \tan^2 \left(45 - \frac{\bar{\phi}}{2} \right)$.

The Rankine value of K_A is for frictionless walls but is used here by tradition because of the empirical nature of the apparent earth pressure formulation. Also, the wall will generally move downwards with any developing active soil wedge.

- c) **Calculate anchor design load:** As shown in Figure 2.2.2 (a).

- d) **Calculate wall base reaction, R:** As shown in Figure 2.2.2 (a).

- e) **Calculate wall section bending moment:** From the apparent earth pressure diagram as shown in Figure 2.2.2 (b). These methods are considered to provide conservative estimates of the calculated bending moments, but may not accurately predict the exact locations of the maxima. FHWA document recommends an allowable stress of $F_b = 0.55 F_y$ for steel soldier piles. For New Zealand design procedures using load and resistance factor design (LRFD) principles and for a strength reduction factor for steel sections of 0.8, an equivalent load factor of $\alpha = 0.8/0.55 = 1.45$ is implied. However, for consistency with NZS 4203 (see discussion elsewhere) a load factor of 1.6 was adopted for this study for the purpose of sizing wall structural elements.

- f) **Determine depth of embedment:** Calculate required depth of embedment for soldier piles to resist wall base reaction (R) using Broms [1965] or similar, or, for continuous walls using passive resistance from Coulomb theory or log-spiral theory such as NAVFAC DM-7. FHWA document recommends a factor of safety of 1.5 for these calculations. For this study, a strength reduction factor of 3 is applied to the Broms [1965] formulation because of the large plastic strains required to mobilise the full passive resistance. Use of this reduction factor was found to give realistic embedment depths consistent with avoidance of wedge failures and better control of displacements.

- g) **Check internal stability of the wall:** A possible internal failure mechanism is shown in Figure 2.2.2 (c), with an active failure wedge immediately behind the wall, a passive wedge immediately in front of the embedded toe of the wall, and the anchor(s) developing their ultimate capacity (taken to be the proven test capacity, normally 1.33 times the design load or 80 percent of the anchor tensile capacity).

The true factor of safety should be determined by reducing the assumed soil strength progressively in the calculations until the driving and resisting forces are just equal, i.e:

$$\text{Active force} = \text{Passive force} + \text{anchor ultimate force}$$

when the factor of safety against sliding is given by:

$$FS = \frac{\tan^{-1}(\phi)}{\tan^{-1}(\phi_{\text{reduced}})}$$

An iterative procedure is required to make this calculation, as shown in Appendix A using Mathcad.

No specific guidance on suitable factor of safety is given in the FHWA document but $FS > 1.3$ for gravity loading would seem to be a sensible value.

- h) **Check external stability of the wall:** External stability of tied-back retaining walls in sand is controlled by horizontal sliding of the wall with formation of an active soil wedge behind the wall and a passive wedge in front of the wall base, as shown in Figure 2.2.2 (c). The critical failure surface is assumed to pass immediately behind the anchor bond zone, as shown.

The same procedure was adopted for evaluating the factor of safety as described in g) above.

No specific guidance on suitable factor of safety is given in the FHWA document but $FS > 1.3$ for gravity loading would seem to be a sensible value.

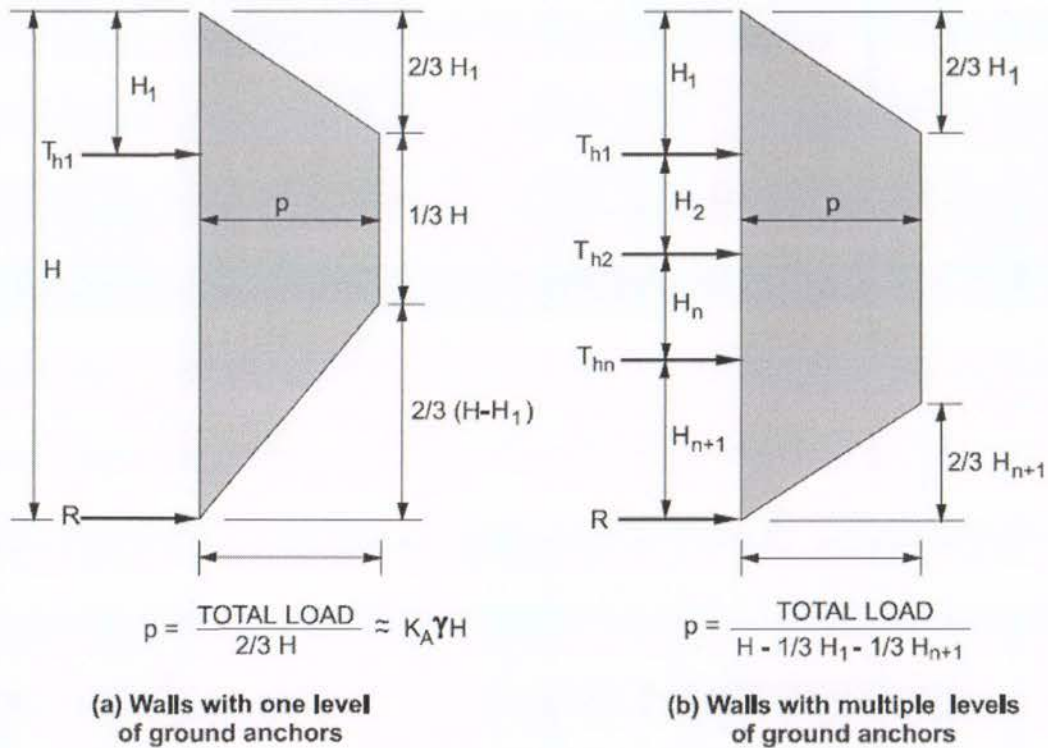
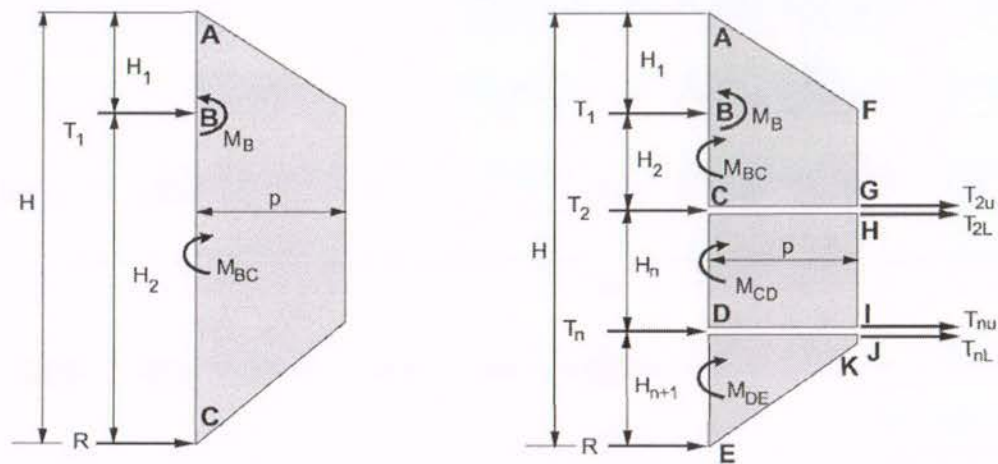


Figure 2.2.2 (a) Apparent earth pressure diagram for sand. [Sabatini et. al., 1999]



$$M_B = \sum M_B$$

M_{BC} = Maximum moment between B and C;
located at point where shear = 0

(a) Walls with one level of ground anchors

$$M_B = \sum M_B$$

$$M_C = M_D = M_E = 0$$

M_{BC} = Maximum moment between B and C;
located at point where shear = 0

M_{CD} ; M_{DE} : Calculated as for M_{BC}

(b) Walls with multiple levels of ground anchors

Figure 2.2.2 (b) Method for estimating wall bending moments for sand. [Sabatini et. al., 1999]

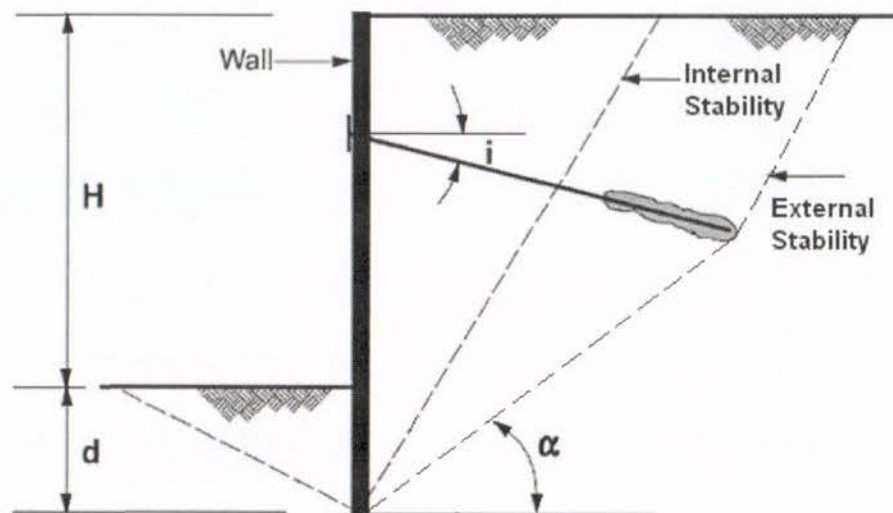


Fig 2.2.2 (c) Internal and external mechanisms for tied back walls.

2.3 Seismic Design

2.3.1 Overview

Little guidance is available for the design of tied-back retaining walls to resist seismic actions. Gravity retaining walls are normally designed using a pseudo-static approach: The active wedge of soil immediately behind the wall has an additional pseudo-static force component equal to the mass of soil within the wedge multiplied by acceleration. Typically, the resulting forces are resolved to derive a new critical wedge geometry and necessary wall pressure to achieve equilibrium, as in the Mononobe-Okabe (M-O) theory [Okabe, 1926; Mononobe and Matsuo, 1929].

For retaining walls that are rigid and unable to move sufficiently to allow soil yielding and development of a Rankine condition behind the wall (e.g. buried basements), a theoretical linear elastic solution for soil pressure derived by Wood [1973] is normally used to calculate dynamic soil pressure.

These two approaches represent, perhaps, an upper and lower bound of what the resulting dynamic soil load might be against a tied-back retaining wall.

The only published advice specific to design of tied-back retaining walls was found within the FHWA manual [Sabatini et. al., 1999]. FHWA recommend use of the pseudo-static Mononobe-Okabe (M-O) theory to design tied-back retaining walls but do not give a detailed procedure. Nor is such a procedure obvious because the recommended design procedure for tied-back walls under gravity loading is based on empirical "apparent earth pressure" diagrams.

The FHWA manual states that the design of brittle elements (e.g. the grout/tendon bond) should be governed by the peak force (i.e. corresponding to peak ground acceleration, PGA). Design of ductile elements (e.g. tendons, steel sheet piles, soldier piles) should be governed by cumulative permanent seismic deformation, or in lieu of such analysis, design should be based on 0.5 times the PGA. However, no advice is given as to how the “peak force” might be calculated.

Given that the anchor tendons are, effectively, long springs with little mass then there seems no reason why they should be subject to high peak forces and should respond only to elongation from gross movements within the soil mass.

Neither of the formulations (Wood or M-O) for calculating wall loads during shaking take any account of the flexibility of the wall and the likely kinematic effects and soil-structure interactions.

2.3.2 Mononobe-Okabe Equations

The M-O equations are an extension of the Coulomb equations based on considerations of equilibrium of a triangular shaped active (or passive) wedge of soil interacting with a sliding wall. The important assumption is made that the soil is yielding in shear along a planar failure surface at the base of the wedge with resolution then made of the resulting force polygon as shown in Figure 2.3.2 (a). An equivalent equation exists for the passive case, but, as for the Coulomb equation, it is inaccurate for walls with friction.

$$K_{AE} = \frac{\cos^2(\phi - \theta - \psi)}{\cos\psi \cos^2\theta \cos(\delta + \theta + \psi) \left[1 + \sqrt{\frac{\sin(\delta + \phi)\sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi)\cos(\beta - \theta)}} \right]^2}$$

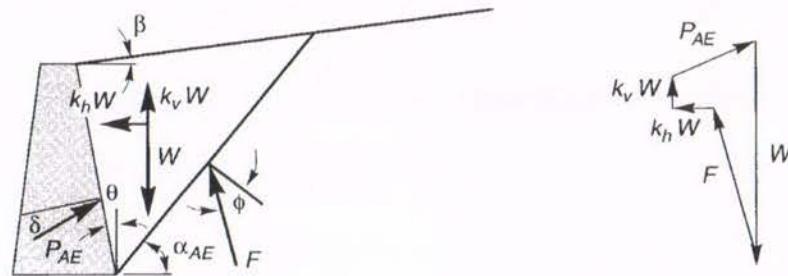


Figure 2.3.2 (a) Mononobe-Okabe equation for active case [Kramer, 1996].

2.3.3 Wood Procedure

Wood [1973] developed a procedure for estimating dynamic loads against smooth, rigid walls based on an assumption that the soil remains linear elastic and that the wall is completely rigid. While not intended originally for tied-back retaining walls but for rigid basements and the like, this procedure might be considered to given an “upper

bound" of the soil pressure that may develop for any given horizontal acceleration against the face of a retaining wall.

The dynamic component of thrust and overturning moment respectively are given by the following equations:

$$\Delta P_e = \gamma H^2 k_h F_p$$

$$\Delta M_e = \gamma H^3 k_h F_m$$

in which k_h = horizontal acceleration as a proportion of g , and F_m , F_p are factors given in Figures 2.3.3.1 (a) and (b) below. The ratio L/H in the Figures refers to length, L , in the horizontal direction for soil contained within a rigid box of depth, H , that was modelled by Wood. For tied-back retaining walls, L/H should be assumed to be infinite.

The point of effective application of the dynamic soil load is at a height above the base of the wall given by:

$$h_e = \frac{\Delta M_e}{\Delta P_e}$$

Typically, $h_e = 0.63H$.

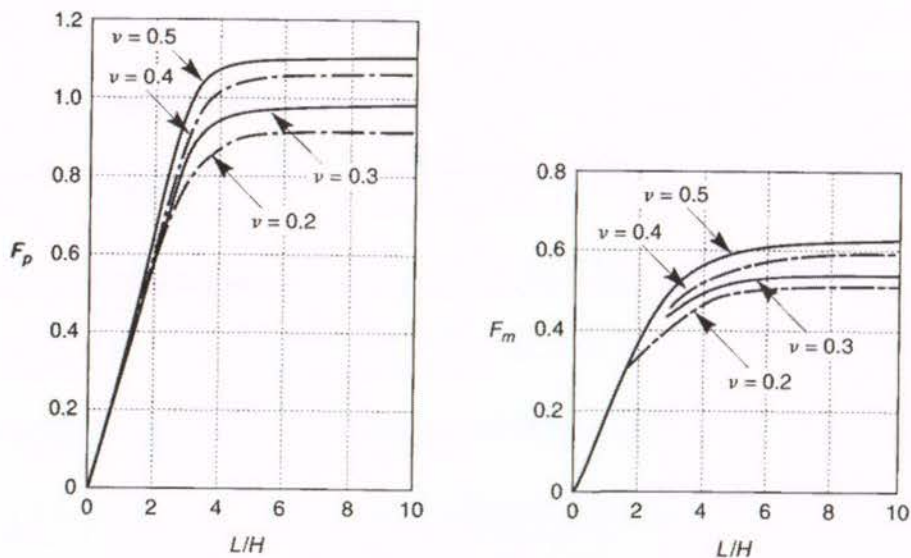


Figure 2.3.3.1 (a) and (b) Dimensionless thrust factor and moment factors. After Wood [1973]

2.3.4 Comparison between M-O and Wood factors

The additional nominal wall loading caused by a pseudo-static horizontal acceleration was calculated using either the M-O or the Wood equations as shown in Table 2.3.4 (a) for the case of walls in sand with $\phi = 35$ degrees.

Table 2.3.4 (a) Comparison of nominal wall loading caused by pseudo-static acceleration

Horizontal Acceleration	Wood k_h $\nu = 0.3$	Mononobe-Okabe $k_{ah} - k_a$ $\phi = 35$
0.1	0.1	0.06
0.2	0.2	0.13
0.3	0.3	0.21
0.4	0.4	0.31

At lower levels of acceleration, the M-O equation gives about $\frac{1}{2}$ the load of the Wood equation, increasing to $\frac{3}{4}$ at 0.4 g. The M-O equation is expected to give much lower loading because it assumes that soil shear strength is fully mobilised to resist the acceleration.

2.3.5 Practice in New Zealand

Given the paucity of guidance in the literature, it was decided to conduct a survey to find out how practitioners were designing tied-back walls to resist earthquakes in current practice.

Current practice in New Zealand was surveyed by conducting a series of personal interviews with senior staff in the largest practices and also from the author's experience in numerous design reviews. Little consistency in approach was evident, with most respondents relying on "black box" computer software that does not specifically consider earthquake loading.

The most commonly used software package is "WALLAP" [Copyright 2002, D.L. Borin, Geosolve, UK]. This software combines limiting equilibrium analysis to British and European standards to compute factors of safety coupled with a 1-D "beam on elastic foundation" or finite element analysis to compute wall element stresses and deformations.

Earthquake "loads" are typically being input as static loads applied to nodes. The calculation of the pseudo-static loads are made using either the M-O equations or the Wood [1973] analysis according to the judgement of the designer.

Typically, the free length of the anchors are located according to the inclination of the Coulomb, gravity only active soil wedge, with no increase to allow for the flattening of the active wedge under acceleration (at least one major consultancy).

(Note: A new version of "WALLAP" has recently been released which allows input of earthquake accelerations directly, although the methodology for computing earthquake response is not known).

2.3.6 Synthesized Design procedure

With no detailed procedure for the design of tied-back walls to resist earthquake loading available, it was necessary to synthesize a trial procedure. A procedure was synthesized based on the FHWA procedure for gravity loading by applying the following rationale.

1. Since the apparent earth pressure used for wall design in gravity loading is calculated based on K_a , the Rankine coefficient of active earth pressure, simply substitute K_{ae} , the M-O coefficient of active earth pressure under earthquake acceleration to calculate an equivalent apparent earth pressure for the earthquake design case.
2. Anchor free lengths are normally extended to beyond the location of the Coulomb active wedge slip plane when designing tied-back walls for gravity loading. Therefore, extend the anchor free length to beyond the equivalent M-O slip plane for earthquake loading.
3. The M-O equations should also be used when checking the external stability of a wall.

The following detailed procedure was adopted on a trial basis for the case studies examined in this project. Based on the results of the time history analyses, additional minor recommendations and improvements were made and these are included in the final recommended procedure of Section 4.

- a) **Initial trial geometry:** The depth of excavation and depth to each row of anchors needs to be estimated as a first step, based on experience or trial and error. Typically, for stronger soils, the first row will be at a depth of 2 m with subsequent rows at 5 m intervals.
- b) **Prepare apparent earth pressure diagram:** As shown in Figure 2.2.2 (a). Note that K_A is calculated using the M-O equation with the selected design pseudo-static acceleration. The wall is assumed to be frictionless (i.e. the wall is likely to move downwards with any active soil wedge).
- c) **Calculate anchor design load:** As shown in Figure 2.2.2 (a).
- d) **Calculate wall base reaction, R:** As shown in 2.2.2 (a)
- e) **Calculate wall section bending moment:** From the apparent earth pressure diagram as shown in Figure 2.2.2 (b). A load factor of 1.6 is recommended for the purpose of sizing wall structural elements using New Zealand standards.
- f) **Determine depth of embedment:** Calculate required depth of embedment for soldier piles to resist wall base reaction (R) using Broms [1965] (but

calculating K_p using the M-O equations), or, for continuous walls using passive resistance from M-O Okabe theory. A strength reduction factor of 3 is recommended to be applied to these calculations because of the large plastic strains required to mobilise the full passive resistance. Use of this reduction factor has been found to give realistic embedment depths consistent with avoidance of wedge failures and better control of displacements.

- g) **Check internal stability of the wall:** A possible internal failure mechanism is shown in Figure 2.2.2 (c), with an active failure wedge immediately behind the wall, a passive wedge immediately in front of the embedded toe of the wall, and the anchor(s) developing their ultimate capacity (taken to be the proven, test capacity, normally 1.33 times the design load or 80 percent of the anchor tensile capacity).

The true factor of safety may be determined by progressively reducing the assumed soil strength in the calculations until the driving and resisting forces are just equal, i.e:

$$\text{Active force} = \text{Passive force} + \text{anchor ultimate force}$$

when the factor of safety against sliding is given by:

$$FS = \frac{\tan^{-1}(\phi)}{\tan^{-1}(\phi_{\text{reduced}})}$$

For the earthquake load case using pseudo-static design, a minimum factor of safety of 1.1 is recommended, but not less than the factor of safety against external stability.

- h) **Set “free” length of anchor tendons:** The “free” length of the anchor tendons should extend beyond the active soil wedge defined by the M-O theory and originating at the base of the wall or the embedded soldier piles as indicated in Figure 2.2.2 (c).
- i) **Check external stability of the wall:** External stability of tied-back retaining walls in cohesionless soil is controlled by horizontal sliding of the wall with formation of an active soil wedge behind the wall and a passive wedge in front of the wall base, as shown in Figure 2.2.2 (c). The critical failure surface is assumed to pass immediately behind the anchor bond zone, as shown.

For the earthquake load case using pseudo-static design, a minimum “true” factor of safety of 1.0 based on mobilised soil shear strength is recommended.

- j) **Note:** When calculating passive soil resistance, the interface friction angle should be set to be no more than $\phi/2$. Use of higher values is not recommended because the resulting values of passive resistance will be unrealistically high.

3 Numerical Modelling of Case Studies

3.1 Introduction

No good case study data is available regarding the performance of tied-back retaining walls in real earthquakes. No data was found for physical model studies for tied-back retaining walls in simulated earthquakes. Modelling of geotechnical systems is difficult, in any case, because the laws of physical similitude require that model experiments be carried out either at very large scale, or, at small scale under high accelerations in a centrifuge.

Numerical modelling in geotechnical engineering has become an accepted research tool and is viewed as a practical substitute to physical modelling for many problems. For study of tied-back retaining walls under earthquake loading, numerical modelling may be the only practical method for realistic simulation given the complexities of the wall construction.

For this study, two representative tied-back wall designs have been modelled numerically: A simple wall with one level of tie-back anchors and a more complex wall with two levels of anchors. Simplified soil conditions have been chosen to be representative of real conditions. Obviously, in practice, much more complex stratigraphies are likely to be encountered, but the objective herein is to gain understanding of the fundamentals of wall performance without introducing confusion from complex stratigraphy.

Detailed design of the walls was made in accordance with the trial design procedure with slight variations and the performance of each under both static gravity and seismic conditions was determined using PLAXIS finite element software for soil and rock mechanics [Brinkgreve & Vermeer, 1988]. Earthquake performance was determined by subjecting each design to time histories of shaking from several real earthquake records scaled to different levels of peak ground acceleration (PGA).

3.2 Methodology

Three case studies were considered for this study, chosen to cover a range of typical scenarios:

- 1) Single row of anchors in deep sand soil (7 m high wall)
- 2) Two rows of anchors in deep sand soil (12 m high wall)
- 3) Two rows of anchors in deep sand soil (12 m high wall) with increased anchor free-length trialled

Tied-back walls up to about 7 m in height are usually able to be constructed with a single row of anchors. Such walls should be able to be designed using simple procedures with the wall structure being relatively stiff and without significant kinematic effects during earthquake shaking.

As walls become higher, with multiple rows of anchors, the wall elements become relatively more flexible and kinematic effects during shaking are likely to become more important. Verification of simple quasi-static design procedures for such walls is an important objective of this study.

Tied-back walls up to about 12 m in height are typically able to be supported by two rows of anchors. Walls greater in height than 12 m will usually require three or more rows of anchors and permanent walls of such height are not often encountered in practice. In this study, a 12 m high wall with two rows of anchors is studied under gravity and earthquake loading conditions. Walls greater in height than this should perhaps be the subject of special study if they are required to resist high seismic loads.

The uniform sand soil used for this study was intended to be representative of granular soil profiles in general. Obviously, much more complex stratigraphies will be encountered in practice, but the reason for simplifying the stratigraphy was to simplify the model as far as practicable to assist with interpretation of the results.

3.3 Time Histories

3.3.1 Overview

Three earthquake accelerogram records were selected to use as input motions for the time history analyses of this project:

- Loma Prieta Earthquake of 18 Oct 1989, $M_L = 6.9$, Dist = 43km, PGA = 215 cm/s/s
- Parkfield Earthquake of 28 Sept 2004, $M_L = 6.0$, Dist = 11.6km, PGA = 300.0 cm/s/s
- Sierra Madre Earthquake 28 Jun 1991, $M_L = 5.8$, Dist = 18.1km, PGA = 273.9cm/s/s

The objective in using multiple records was to include the influence of earthquake variability on wall performance.

All three records (shown in Figures 3.3.1 (a), (b), and (c)) are characterised by relatively modest values of PGA but they have a useful range of magnitudes for the sort of event that usually shows up in the de-aggregation of site specific hazard studies. The Sierra Madre record comes from a low magnitude and is characterised by a single strong pulse and some low level high freq noise. The Loma Prieta record corresponds to a larger magnitude and it contains several significant cycles of shaking but with lower PGA due to the greater distance. The Parkfield record is in-between: moderate magnitude, small distance, larger PGA, several cycles.

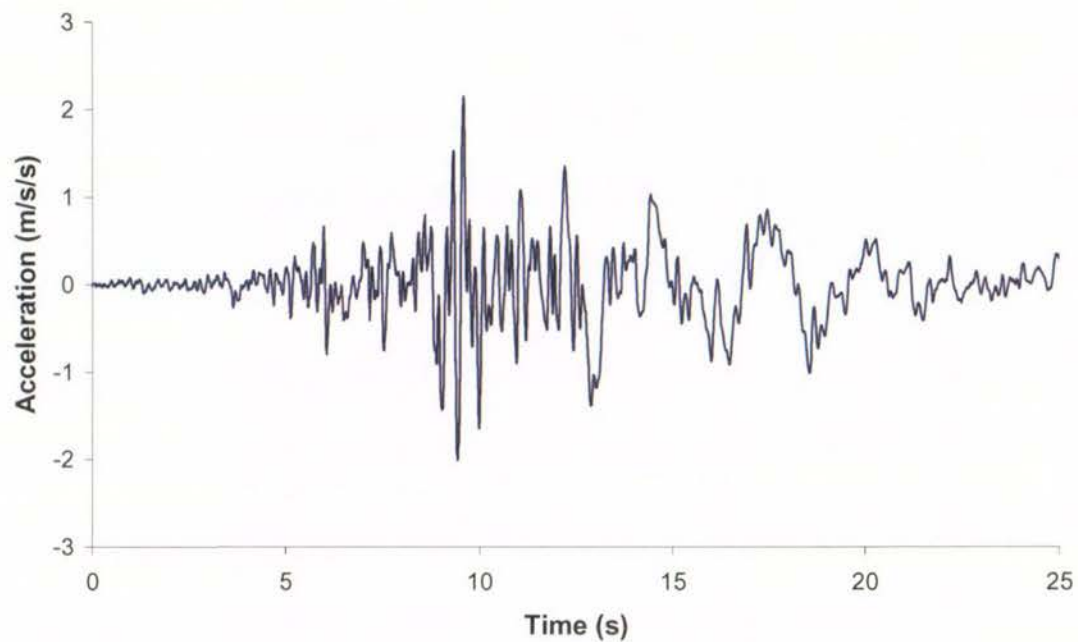


Figure 3.3.1 (a). Loma Prieta Accelerogram.

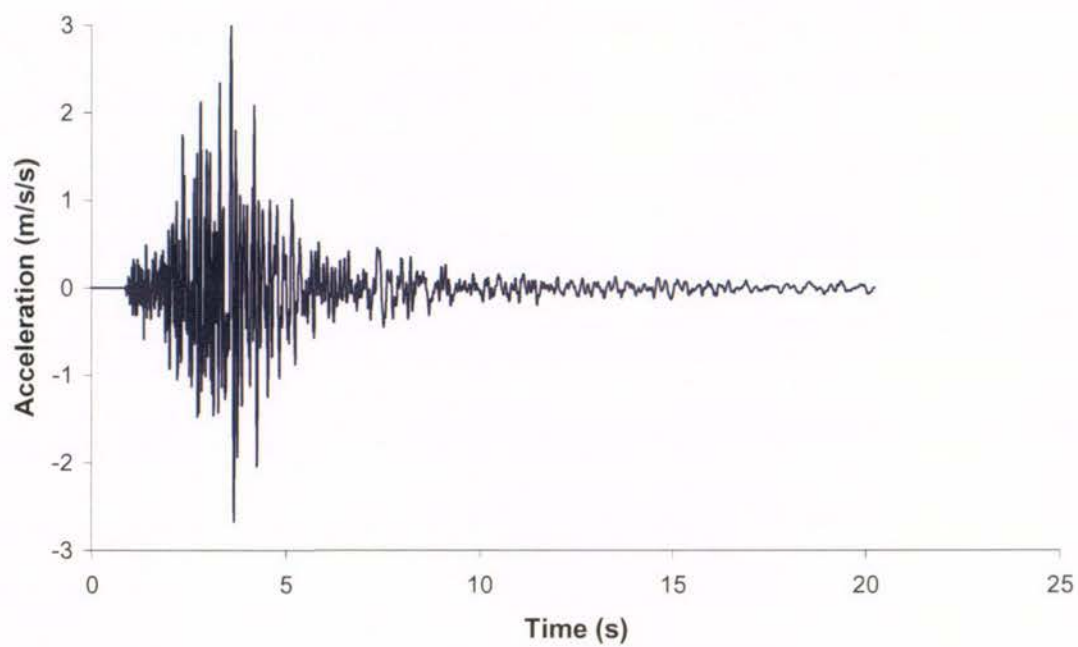


Figure 3.3.1 (b). Parkfield Accelerogram.

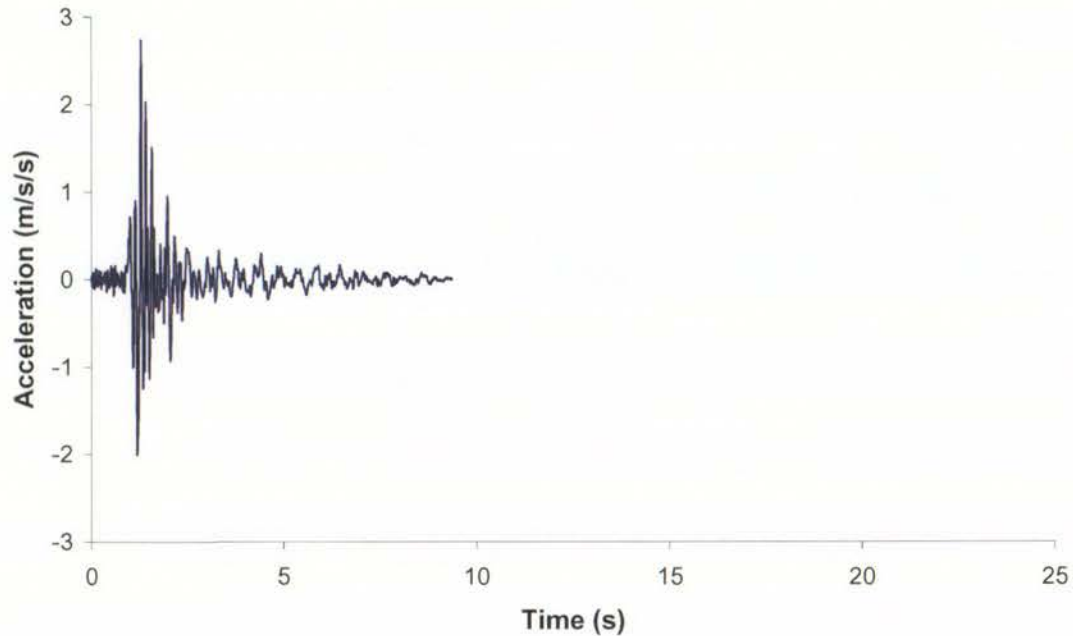


Figure 3.3.1 (c). Sierra Madre Accelerogram.

3.3.2 Scaling factors

All of the three records were recorded at the ground surface but were used as input motions to the base (effectively “bedrock”) of the PLAXIS numerical models used for the study. Each record was calibrated to each model by use of specific scaling factors determined experimentally by shaking model soil deposits using numerical time history analysis and then changing scale factors applied by trial and error until the desired peak ground acceleration (PGA) was obtained at the ground surface.

Because the effect on site response caused by excavating and constructing the tied-back walls is an integral part of the response being studied, the record calibration procedure was done on level ground soil deposits prior to excavating and constructing the walls.

The model soil deposit used for Case Study 1 (7 m high wall in sand) is shown in Figure 3.3.2 (a). The model was made extra wide (100 m) to allow for the accumulation of deformations close to the edge of the deposit caused by the PLAXIS energy absorbing boundaries. The depth (20 m) was judged sufficient to allow unrestricted development of the wall response while still being shallow enough to encourage a simple shear response of the model to the passage of incoming earthquake shear waves.

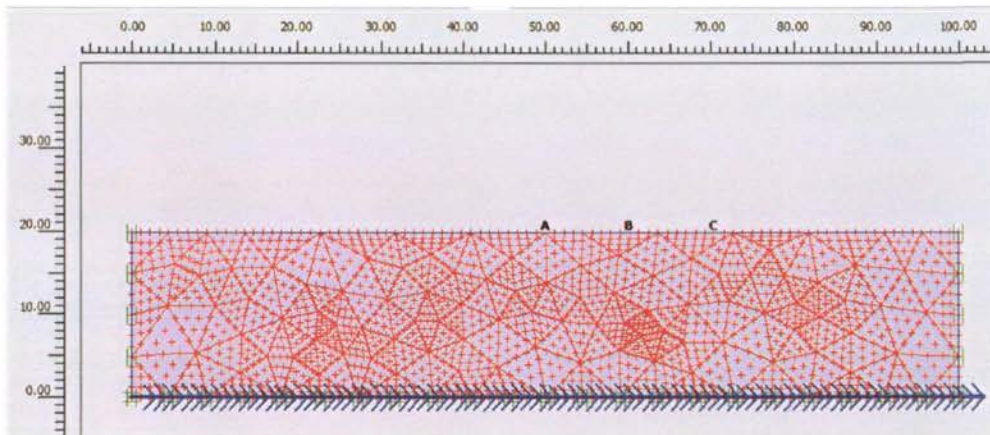


Figure 3.3.2 (a). Model Sand 1 used for calibrating earthquake record PGAs.

The resulting ground accelerations were monitored at three locations on the surface of the deposit, shown as points A, B, and C in Figure 3.3.2 (a). The values of PGA recorded at each of the three locations was averaged to eliminate small fluctuations in response at different locations on the surface. A typical result from one of the calibration analysis runs is shown in Figure 3.3.2 (b).

The energy absorbing boundaries used by PLAXIS allow permanent deformations in the modelled soil deposits near to the boundaries, as shown in Figure 3.3.2 (c). The width of the model was made sufficiently large (100 m) to prevent any effect on wall response.

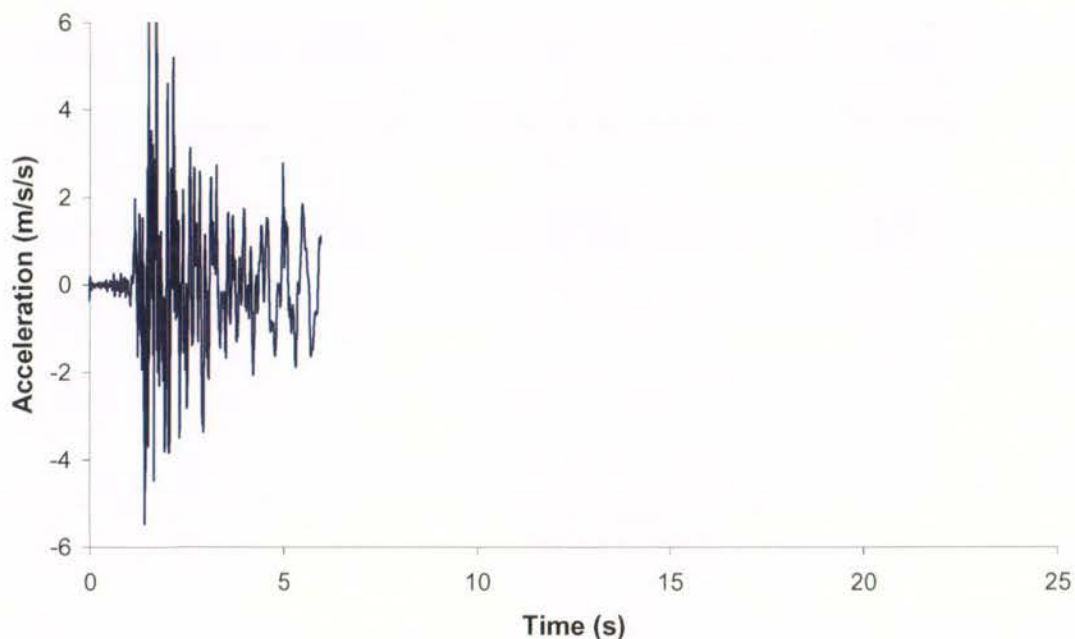


Figure 3.3.2 (b) Sierra Madre record scaled to give PGA of 0.6 g measured on surface of model Sand 1.

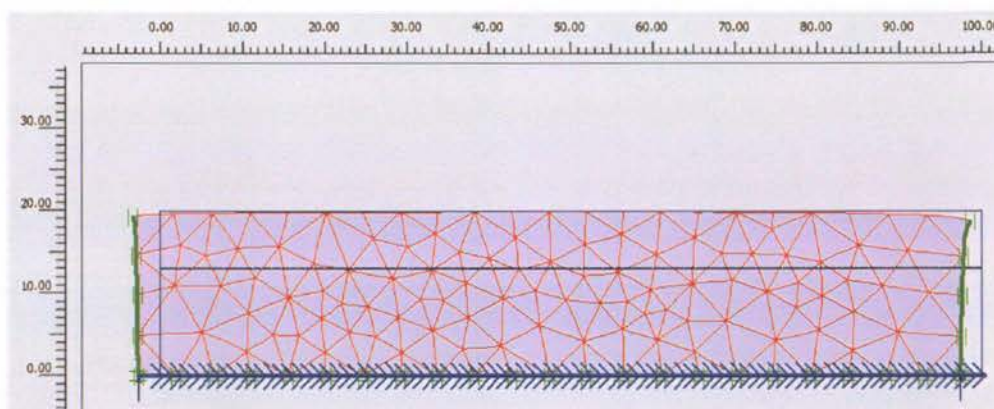


Figure 3.3.2 (c) Model Sand 1 after earthquake shaking.

The resulting scaling factors determined for model Sand 1 are shown in Table 3.3.2 (a) for the three different values of surface PGA selected for use in the study. The scaling factors do not represent a linear relationship between scaled base input record and surface PGA: The scaling factors increase markedly with increasing target surface PGA, presumably because of increasing soil non-linearity effects.

Table 3.3.2 (a). Scaling factors determined for model Sand 1.

Earthquake Record	Target PGA		
	0.2 g	0.4 g	0.6 g
Loma Prieta	0.22	0.53	0.97
Parkfield	0.22	0.46	0.77
Sierra Madre	0.30	0.69	1.11

For Case Study 2 and 3 (12 m high wall in sand) the depth of the soil deposit was increased to 25 m to maintain the same depth of soil beneath the excavation. The scaling factors for this soil deposit (Table 3.3.2 (b)) were slightly different from model Sand 1 because of the increased thickness of the soil deposit. Scale factors were only determined for the Loma Prieta record because this was found to give by far the greatest wall deformation response for Case Study 1.

Table 3.3.2 (2). Scaling factors determined for model Sand 2 and Sand 3

Earthquake Record	Target PGA		
	0.2 g	0.4 g	0.6 g
Loma Prieta	0.28	0.60	1.0

3.4 Case Study 1: Single Row of Anchors in Sand

3.4.1 Case study description

This case is for a 7 m deep excavation in sand. It is assumed that the water table has been drawn down to the base of the excavation. Typically, such an excavation would be made using concrete soldier piles with sprayed concrete facing for a permanent installation or galvanised steel UC sections with timber lagging. A single row of tie-back ground anchors is usually found to provide an economical solution with a two stage excavation process: Installation of soldier piles from the ground surface, excavation to 2 m depth, installation and stressing of the ground anchors, and final excavation to full depth.

A cross-section through the PLAXIS model is shown in Figure 3.4.1 (a). The depth to the first row of anchors was made 2 m based on experience leaving a further 5m deep excavation below. The anchor inclination is set at 15 degrees, about the flattest angle practicable. The bond length (yellow line, PLAXIS geogrid element) is set at 7 m which is typical for ground anchors in sandy soils assuming that multi-stage pressure grouting is utilised. The anchor free length (black line, PLAXIS node-to-node anchor), was determined using the FHWA gravity procedure.

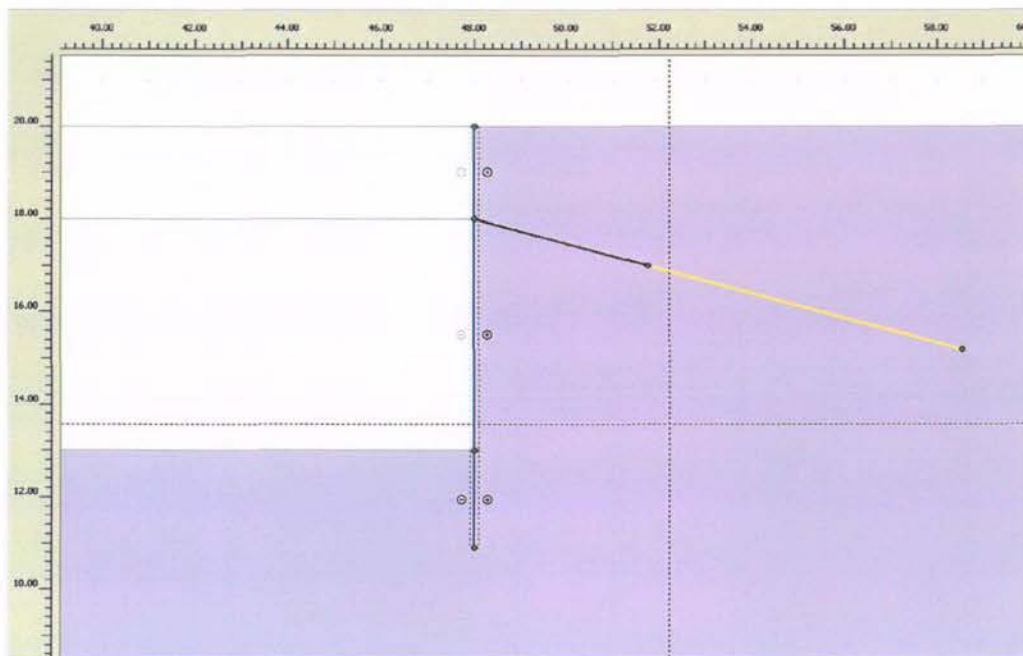


Figure 3.4.1 (a) PLAXIS model Sand 1: Gravity based design.

The assumed soil properties are given in Table 3.4.1 (a) and are considered to be typical for medium-dense sand.

Table 3.4.1 (a) Soil properties for case studies in sand.

Property	Symbol	Value
Density, unsaturated	γ_{unsat}	16 kN/m ³
Density, saturated	γ_{sat}	18 kN/m ³
Effective cohesion	c'	1 kN/m ²
Effective friction	ϕ'	35 degrees
Soil model		Hardening soil
Young's Modulus	E_{50}^{ref}	30 MN/m ³
Young's Modulus (unload/reload)	$E_{\text{ur}}^{\text{ref}}$	90 MN/m ³

3.4.2 Case 1a: Gravity design

Gravity design followed the FHWA gravity procedure described in Section 2.2.2. Detailed calculations are given in Appendix A and are summarised in Table 3.4.2 (a).

Table 3.4.2 (a). Design values for case study Sand 1a: Gravity design.

Design Parameter	Value
Apparent earth pressure, p	30 kN/m ²
Anchor design load (horizontal)	108 kN/m
Base reaction	30 kN/m
Negative bending moment (at anchor)	28 kNm/m
Maximum bending moment (below anchor)	45 kNm/m
ULS design bending moment, M^*	72 kNm/m

The wall structural elements were designed using these basic calculated design values with details given in Table 3.4.2 (b). For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections, etc.).

Table 3.4.2 (b). Design solutions for case study Sand 1a: Gravity design.

Design Solution	Value
Anchor cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	201 mm ² per anchor (2.01 strands per anchor)
Anchor free length ¹	3.9 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	94% ² of 200UC52.2
Depth of embedment	2.1 m

¹ Calculated using FHWA procedure. In practice, a minimum free length of 5 m is recommended for strand anchors.

² Section properties scaled for purpose of the study.

The embedment depth was determined for the design base reaction using Broms [1965] with a strength reduction factor of 1/3 (see Appendix A). The depth of embedment was verified by performing checks for internal and external stability as summarised in Table 3.4.2 (c). The full calculations are given in Appendix A.

Table 3.4.2 (c). Internal and external stability checks for case study Sand 1a: Gravity design.

Stability Case ¹	FS
Internal stability	1.39
External stability	1.82

¹ Refer Figure 2.2.2 (c)

3.4.3 Performance of Case 1a under gravity and pseudo-static loading

The performance of the tied-back wall designed using standard procedures considering only the gravity load case was determined by analysing the wall design using PLAXIS. First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor force were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability.

A summary of the main performance parameters is given in Table 3.4.3 (a), the bending moment distribution for the wall element is given in Figure 3.4.3 (a), and the collapse mechanism is illustrated in Figures 3.4.3 (b) and (c).

Table 3.4.3 (a) Performance of Case Study Sand 1a under gravity and pseudo-static loading

	Design Basis ULS	Final excavation FS=1.0	Wall first yield FS=1.38	Stability Limit FS=1.43	Max acceleration 0.21 g
Displacement (top of wall) (mm)	-	7	0	-12	491
Displacement (maximum) (mm)	-	26	94	148	630
Wall BM (at anchor) (KNm/m)	45	33	38	40	59
Wall BM (below anchor) (KNm/m)	72	25	72 ²	72 ²	72 ²
Anchor force (KN/m)	139 ¹	110	133	159 ¹	173 ³

¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load, so anchor load of 159 KN/m (92 percent of characteristic breaking load) exceeds the test load but anchor is considered unlikely to fail.

² Wall element is yielding

³ Anchor has reached UTS

The collapse mechanism of the wall under gravity loading appears to be hinging of the wall element with significant “bulging” of the wall into the excavation and development of an active soil wedge behind the wall. The strength of the wall element is, therefore, limiting the factor of safety, although the anchor tendon force is exceeding the desired ULS value and is approaching the characteristic breaking strength.

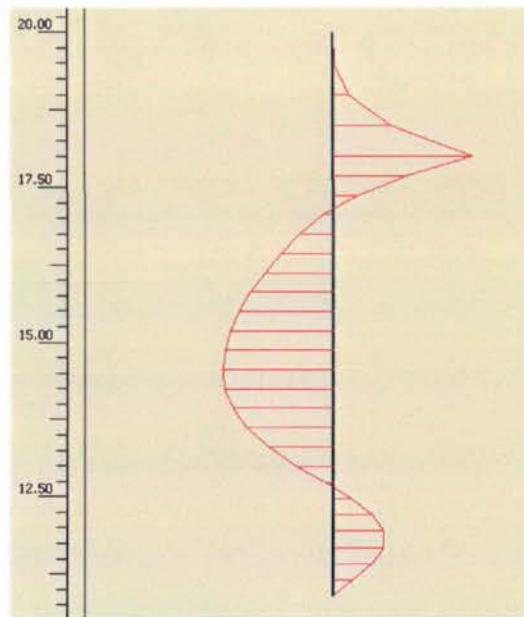


Figure 3.4.3 (a). Wall bending moment versus depth, $FS = 1$ (full soil strength).

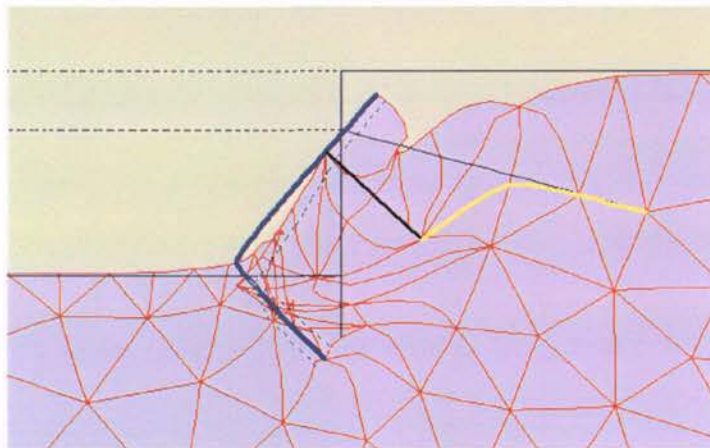


Figure 3.4.3 (b). Deformed mesh at onset of instability, $FS = 1.43$ (exaggerated scale).

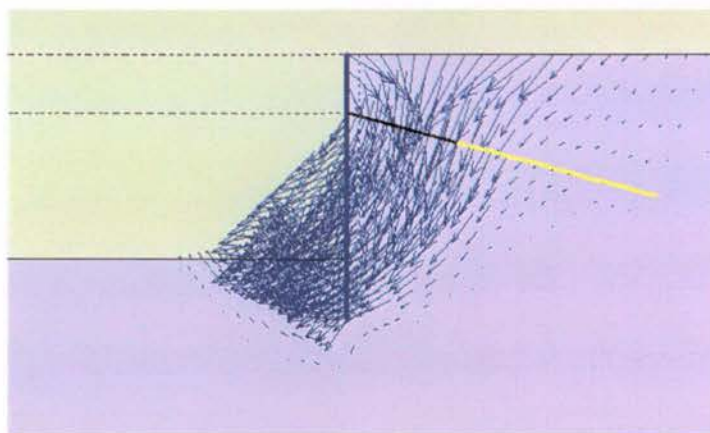


Figure 3.4.3 (c). Soil displacement vectors at onset of instability, $FS = 1.43$.

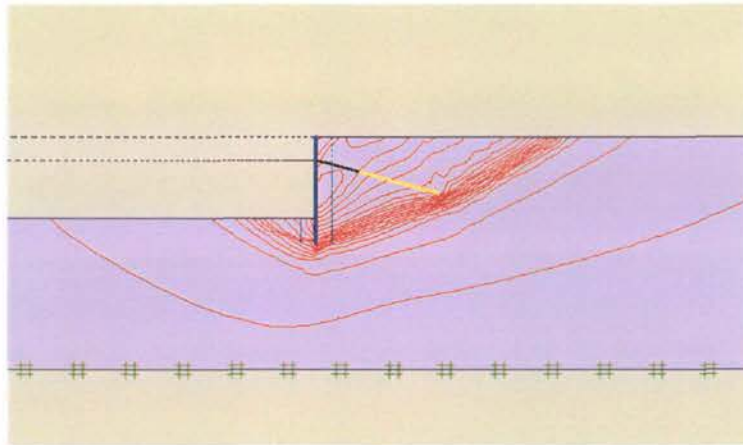


Figure 3.4.3 (d). Soil displacement countours at onset of instability, pseudo-static acceleration = 0.21 g

Under pseudo-static acceleration of 0.21 g, the wall is undergoing external stability failure (Figure 3.4.3 (d)) at about the same time as the anchor force reaches material ultimate tensile strength.

3.4.4 Evaluation of Case 1a under gravity loading

The factor of safety achieved in the PLAXIS analysis using “phi-c reduction” (lower bound) is considered satisfactory. Typically, acceptable factors of safety for slope stability analyses using limiting equilibrium methods of analysis (upper bound) are considered to be in the range from FS = 1.2 to FS = 1.5 for critical slopes.

The factor of safety determined for this case study (FS = 1.43) is close to the value calculated using the “by hand” limiting equilibrium procedure (internal stability, FS = 1.39).

The PLAXIS analysis suggests that it may be possible to improve the factor of safety by increasing the yield bending strength of the wall. However, experience shows that increasing the bending strength of the wall gives little improvement once the soil active wedge has developed.

The capacity of the wall under pseudo-static loading is surprisingly good, with failure occurring along the desirable external stability mechanism. The anchor force increases to reach UTS as displacements increase, but only after very large wall translational displacements are achieved (greater than 600 mm).

3.4.5 Performance of gravity design Case 1a under seismic loading

The performance of the gravity design under seismic loading was determined by applying the suite of three scaled earthquake time-history records to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g.

Wall performance is indicated primarily by outwards permanent displacement remaining after each earthquake “event”. For walls with a single row of tie-back

anchors, displacements are usually critical at two locations: At the crest of the wall and between the anchor and the base where the wall typically tends to “bulge” outwards. The bending moments in the wall elements were also monitored together with the anchor force. Results from all of the analyses for the gravity design are summarised in Figures 3.4.5 (a), (b), (c), and (d).

There was a very large difference in wall performance among the suite of three earthquake records: Displacements were modest for the Parkfield and Sierra Madre records (up to 32 mm at the crest and 93 mm at the “bulge”) but quite large for the Loma Prieta record (135 mm at the crest and 325 mm at the “bulge”). Wall bending moments remained comfortably below yield for the Parkfield and Sierra Madre records but were at yield at the end of the 0.6 g scaled Loma Prieta record. Tie-back anchor forces were barely affected by shaking for most of the runs but were increased by the 0.4 g and 0.6 g scaled Loma Prieta records. The anchor forces remained comfortably below the ultimate tensile capacity of the tendons in all cases.

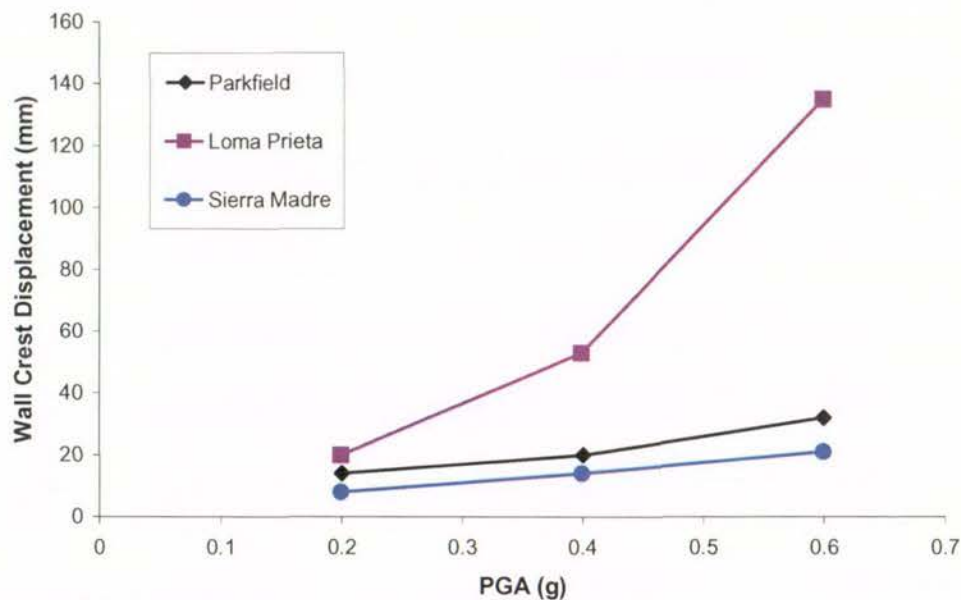


Figure 3.4.5 (a). Accumulated wall crest displacement after earthquake for gravity design.

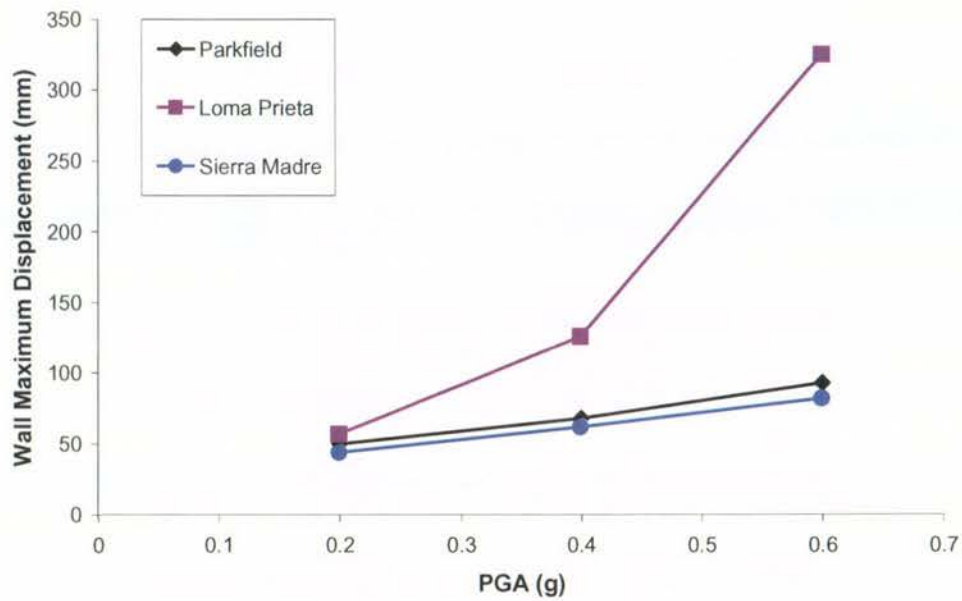


Figure 3.4.5 (b). Accumulated wall displacement below level of tie-back anchor after earthquake for gravity design.

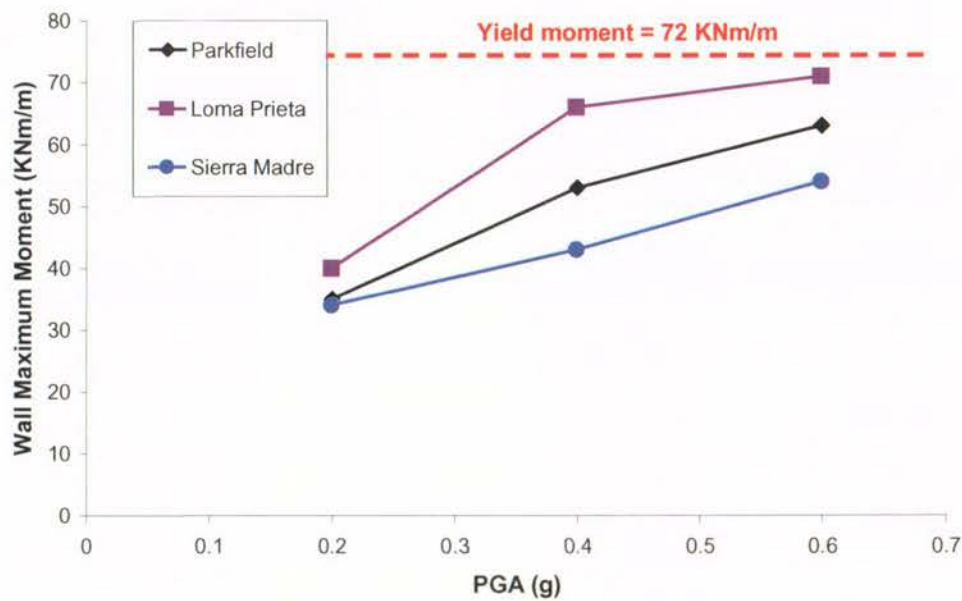


Figure 3.4.5 (c). Maximum wall bending moment after earthquake for gravity design.

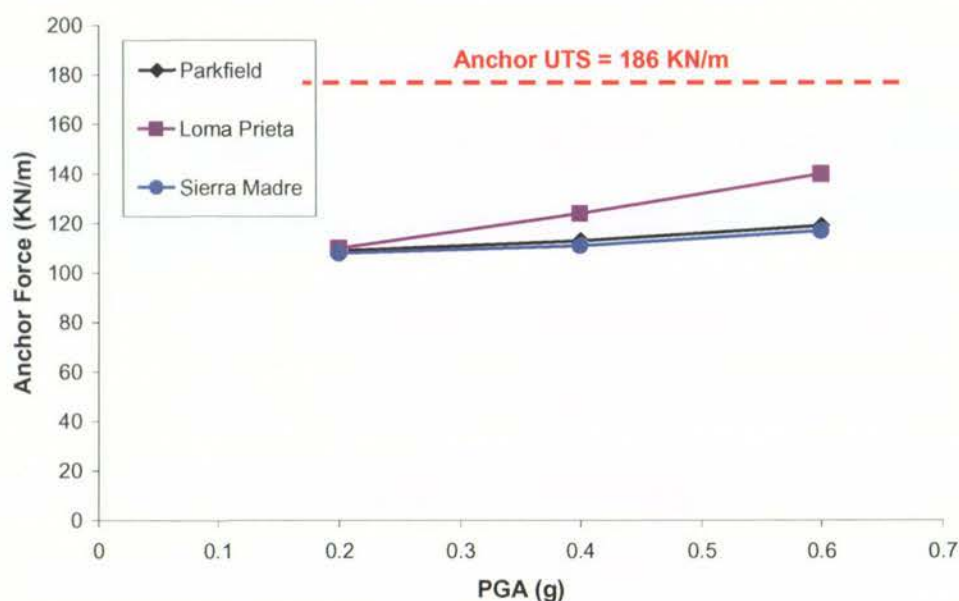


Figure 3.4.5 (d). Anchor force after earthquake for gravity design.

Generally, the performance of the wall was surprisingly good given that the design was for a standard gravity only procedure with no consideration of seismic effects. For the 0.2 g scaled records, displacements were all less than 57 mm even for the Loma Prieta record, and such a small displacement would be acceptable for most situations. Even at 0.4 g, displacements were limited to 126 mm for the Loma Prieta record and 68 mm for the other records, acceptable for many situations. At 0.6 g, the wall displacements for the Parkfield and Sierra Madre records were limited to 93 mm. Large displacements (up to 325 mm) and wall yielding occurred for the Loma Prieta record scaled to 0.6 g. However, the wall remained stable after the earthquake even though it would be considered badly damaged and the level of displacement might cause problems for supported or adjacent buildings and services etc. (Note that the PLAXIS model assumes that full ductility is available for the wall element).

The above discussion concerns the state of the wall after the earthquake. It is also important to know that the anchor forces do not exceed tendon capacity even instantaneously during earthquake shaking. The variation of anchor forces during Loma Prieta record scaled to 0.6 g is shown in Figure 3.4.5 (e) and show a sharp increase at about 10 seconds elapsed time, approximately coincident with the main acceleration pulse but only minor fluctuations otherwise. There is a small oscillation in anchor force that seems to be in phase with the significant oscillation in wall crest displacement.

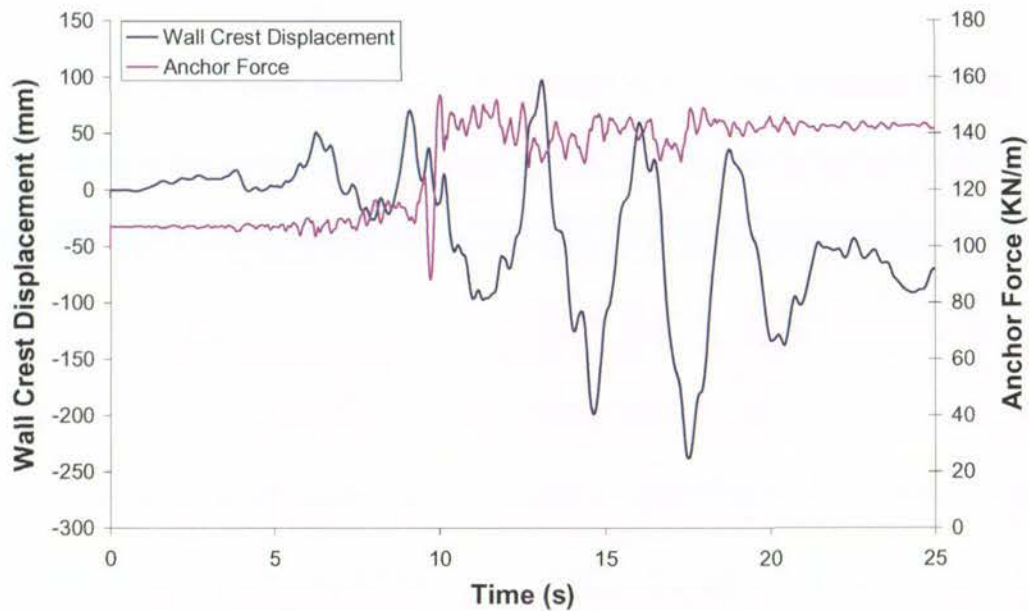


Figure 3.4.5 (e). Anchor force and wall crest displacement versus time for gravity design during 0.6 g Loma Prieta record.

3.4.6 Case 1b: M-O based design to 0.1 g

The wall for Case 1b was designed using the synthesized procedure for earthquake design outlined in Section 2.3.6. Detailed calculations are given in Appendix A and are summarised here:

Table 3.4.6 (a). Design values for case study Sand 1b: M-O based design 0.1 g.

Design Parameter	Value
Design acceleration	0.1 g
Apparent earth pressure, p	39 kN/m ²
Anchor design load (horizontal)	144 kN/m
Base reaction	40 kN/m
Negative bending moment (at anchor)	38 kNm/m
Maximum bending moment (below anchor)	60 kNm/m
ULS design bending moment, M^*	95 kNm/m

The wall structural elements were designed using these basic calculated design values as listed in Table 3.4.6 (b). For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections).

Table 3.4.6 (b). Design solutions for case study Sand 1b: M-O based design. 0.1 g.

Design Solution	Value
Anchor cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	268 mm ² per anchor (2.68 strands per anchor)
Anchor free length	4.4 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	72% ¹ of 250UC72.9
Depth of embedment	2.6 m

¹ Section properties scaled for purpose of the study.

The anchor free length for Case 1b was determined from the inclination of the M-O active wedge slip plane calculated for the soil strength *reduced* by the factor of safety for internal stability. This adjustment has the effect of increasing the free length to allow for uncertainty in soil strength parameter and ensure that the anchor free length extends beyond the active soil zone in all cases.

The results of the internal and external stability checks are given in Table 3.4.6 (c) and refer to the condition with pseudo-static horizontal acceleration of 0.1 g.

Table 3.4.6 (c). Internal and external stability checks for case study Sand 1b: M-O based design 0.1 g

Stability Case ¹	FS
Internal stability	1.23
External stability	1.31

¹ Refer Figure 2.2.2 (c)

3.4.7 Performance of Case 1b under gravity and pseudo-static loading

The performance of the tied-back wall designed using the M-O based design procedure was measured using the same PLAXIS modelling sequences as for the gravity design: First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor force were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability. Finally, a pseudo-static acceleration was applied and increased until the model became unstable. A summary of the main performance parameters is given in Table 3.4.7 (a) and the failure mechanism of the wall under pseudo-static loading is illustrated in Figure 3.4.7 (a).

Table 3.4.7 (a) Performance of Case 1b: M-O based design to 0.1 g under static and pseudo-static loading.

	Design Basis ULS	Final excavation FS=1.0	Onset of instability FS = 1.45	Design acceleration 0.1 g	Maximum acceleration 0.18 g
Displacement (top of wall) (mm)	-	3	-15	27	107
Displacement (maximum) (mm)	-	20	95	37	132
Wall BM (at anchor) (KNm/m)	-55	-41	-40	-44	51
Wall BM (below anchor) (KNm/m)	87	25	87 ²	40	66
Anchor force (KN/m)	181 ¹	136	142	137	148

¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load

² Wall element is yielding in bending.

As for the gravity only design (Case 1a), the collapse mechanism of the wall under gravity only loading appears to be hinging of the wall element with significant “bulging” of the wall into the excavation and development of an active soil wedge behind the wall (internal stability failure). The strength of the wall element is, therefore, limiting the factor of safety. In this case the anchor force hardly increases above its initial pre-load value and well below the test load.

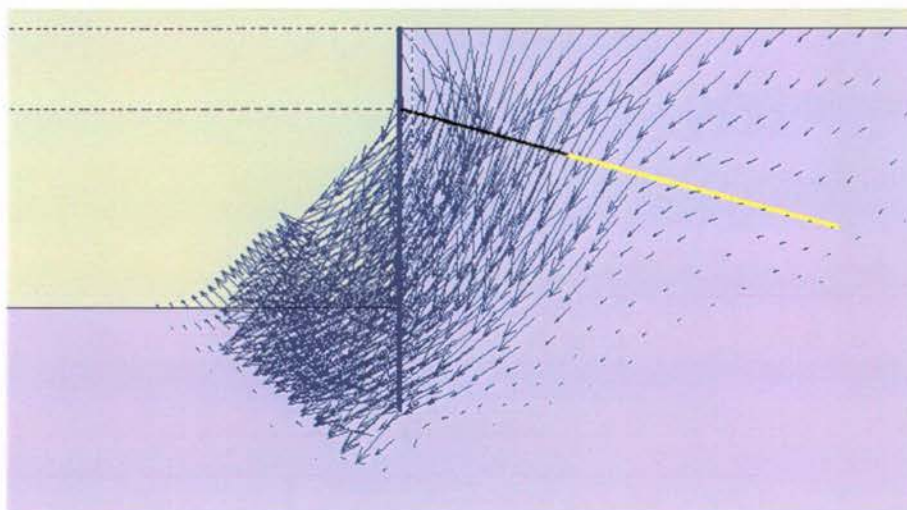


Figure 3.4.7 (a). Failure mechanism for Case 1b under gravity loading at FS = 1.45.

The wall achieved a maximum pseudo-static acceleration of 0.18 g. The factor of safety at the design acceleration of 0.1 g was found to be 1.27, slightly less than the value of 1.33 calculated using the limiting equilibrium external wedge analysis. The failure mechanism (Figure 3.4.7 (b)) was very like the assumed external stability limiting equilibrium failure model (Figure 2.2.2 (d))

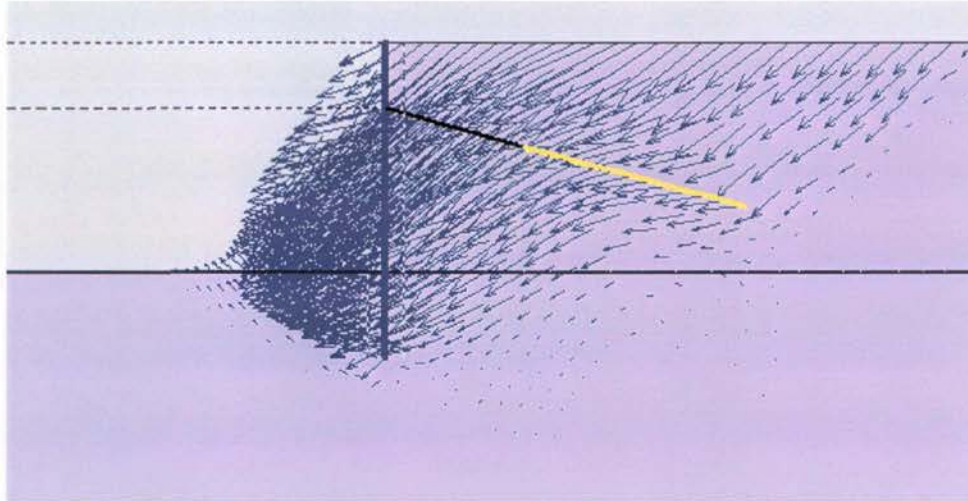


Figure 3.4.7 (b). Failure mechanism for Case 1b under pseudo-static loading of 0.1 g at FS = 1.23.

3.4.8 Performance of Case 1b under seismic loading

The performance of Case 1b under seismic loading was determined by applying the same suite of three scaled earthquake time-history records to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g, as for the gravity only design, Case 1a.

Results from all of the analyses for the Case 1b: M-O design to 0.1 g are summarised in Figures 3.4.8 (a), (b), (c), and (d).

The same trend was exhibited in wall performance among the suite of three earthquake records but overall the response was much superior to the gravity only design (Case 1a) with greatly reduced displacements: Displacements were modest for the Parkfield and Sierra Madre records (up to 22 mm at the crest and 59 mm at the "bulge") but still quite large for the Loma Prieta record (115 mm at the crest and 237 mm at the "bulge"). Wall bending moments remained comfortably below yield for the Parkfield and Sierra Madre records but were approaching yield at the end of the 0.6 g scaled Loma Prieta record. Tie-back anchor forces were barely affected by shaking for all of the runs and remained close to the initial pre-loading.

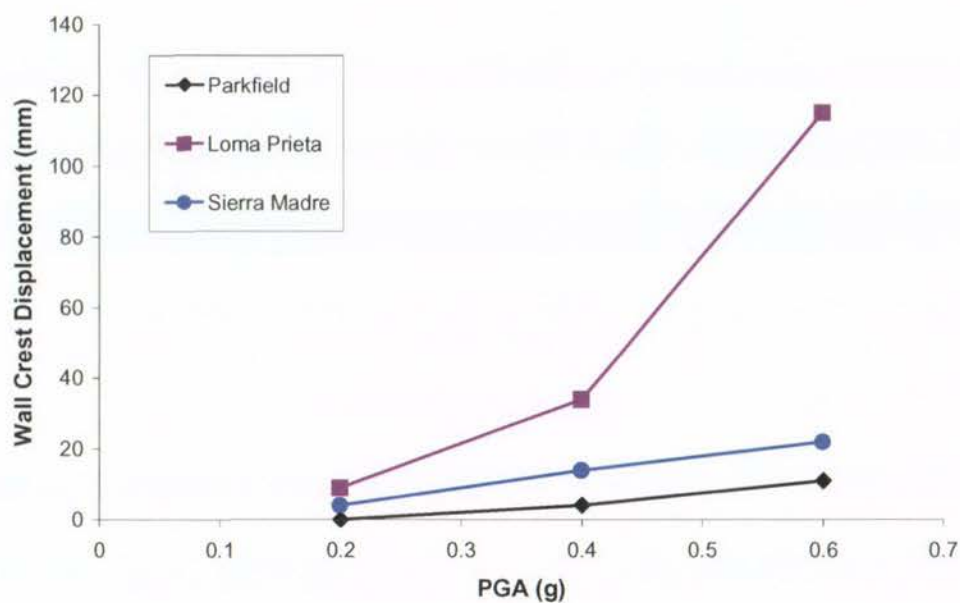


Figure 3.4.8 (a). Accumulated wall crest displacement after earthquake for Case 1b: M-O design to 0.1 g.

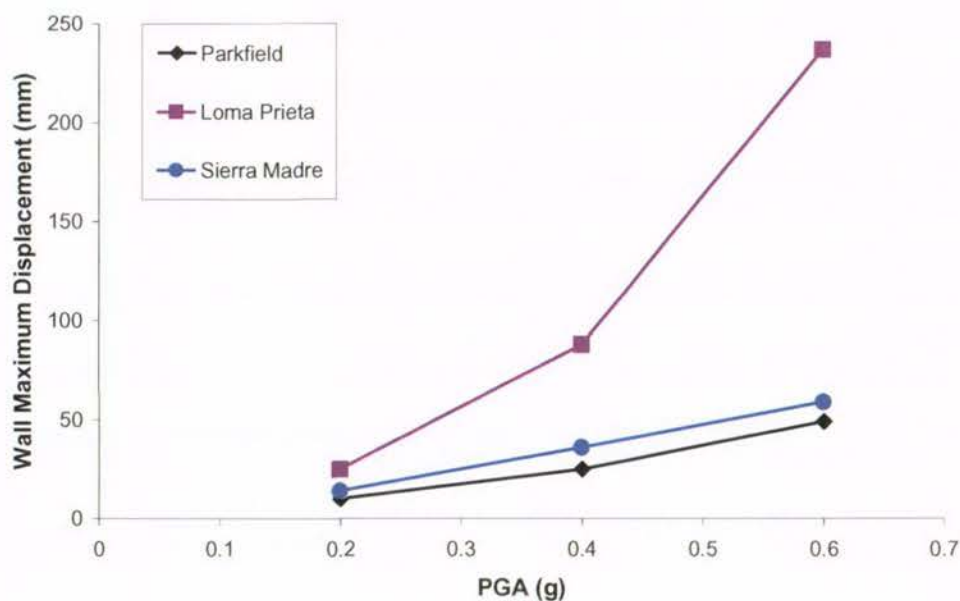


Figure 3.4.8 (b). Accumulated wall displacement below level of tie-back anchor after earthquake for Case 1b: M-O design to 0.1 g.

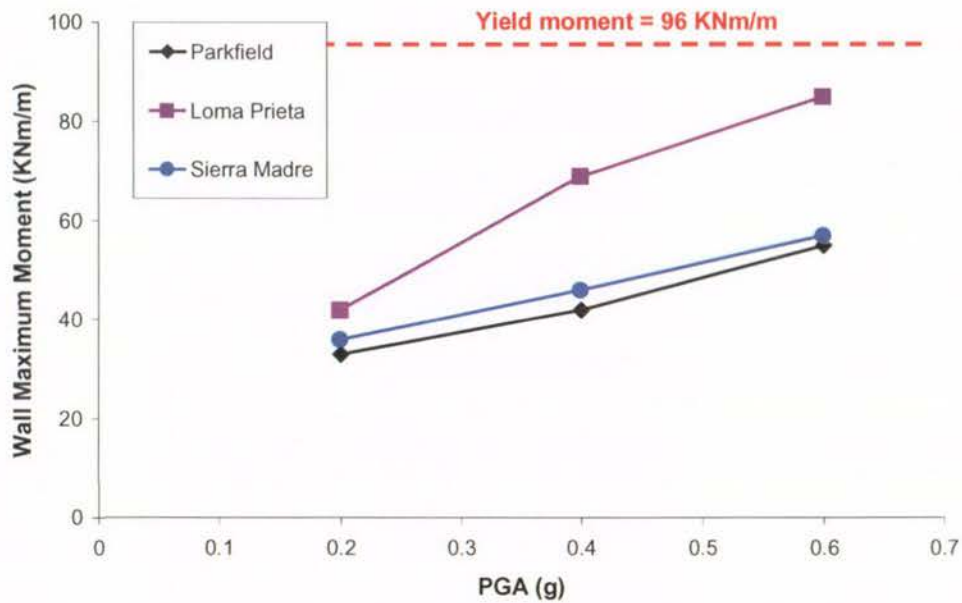


Figure 3.4.8 (c). Maximum wall bending moment after earthquake for Case 1b: M-O design to 0.1 g.

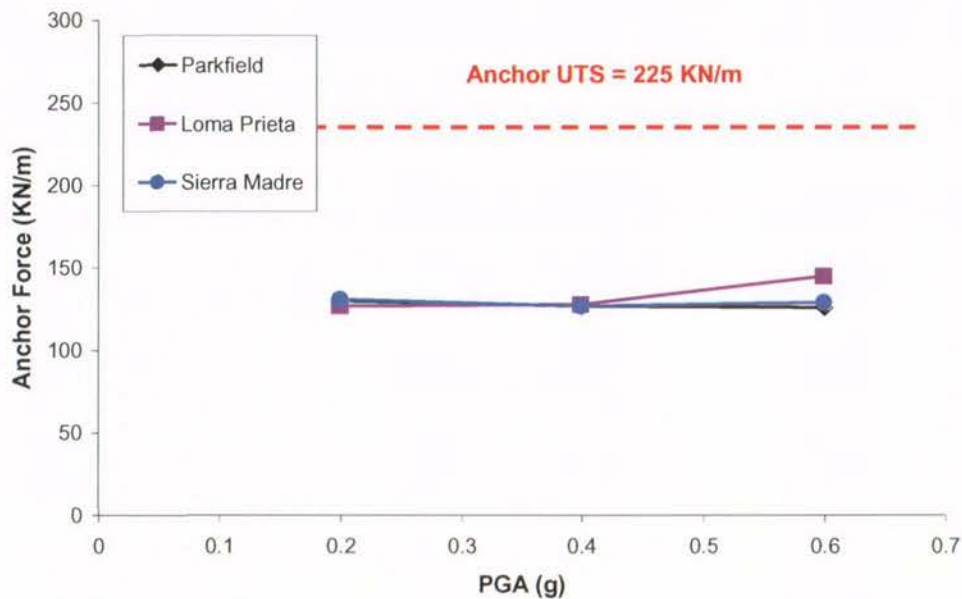


Figure 3.4.8 (d). Anchor force after earthquake for Case 1b: M-O design to 0.1 g..

Generally, the performance of the wall was good and showed a marked improvement in performance over the gravity only design although at the cost of significantly more

materials in both wall elements and anchors. A more detailed comparison among all the design cases is given in Section 3.4.18.

3.4.9 Case 1c: M-O based design to 0.2 g

The wall for Case 1c was designed using the synthesized procedure for earthquake design outlined in Section 2.3.6. Detailed calculations are given in Appendix A and are summarised here:

Table 3.4.9 (a). Design values for case study Sand 1c: M-O based design 0.2 g.

Design Parameter	Value
Design acceleration	0.2 g
Apparent earth pressure, p	43 kN/m ²
Anchor design load (horizontal)	158 kN/m
Base reaction	44 kN/m
Negative bending moment (at anchor)	42 kNm/m
Maximum bending moment (below anchor)	65 kNm/m
ULS design bending moment, M^*	105 kNm/m

The wall structural elements were designed using these basic calculated design values as follows. For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections).

Table 3.4.9 (b). Design solutions for case study Sand 1c: M-O based design. 0.2 g.

Design Solution	Value
Anchor cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	292 mm ¹ per anchor (2.92 strands per anchor)
Anchor free length	5.7 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	79% ² of 250UC72.9
Depth of embedment	2.8 m

¹ Section properties scaled for purpose of the study.

The anchor free length for Case 1c was determined from the inclination of the M-O active wedge slip plane calculated for the soil strength *reduced* by the factor of safety for internal stability. This adjustment has the effect of increasing the free length to allow for uncertainty in soil strength parameter and ensure that the anchor free length extends beyond the active soil zone in all cases.

The results of the internal and external stability checks are given in Table 3.4.9 (c) and refer to the condition with pseudo-static horizontal acceleration of 0.2 g.

Table 3.4.9 (c). Internal and external stability checks for case study Sand 1c: M-O based design 0.2 g

Stability Case ¹	FS
Internal stability	1.25
External stability	1.16

¹ Refer Figure 2.2.2 (c)

3.4.10 Performance of Case 1c under gravity and pseudo-static loading

The performance of Case 1c was measured using the same PLAXIS modelling sequences as for the gravity design: First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor force were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability. Finally, a pseudo-static acceleration was applied and increased until the model became unstable. A summary of the main performance parameters is given in Table 3.4.10 (a) and the failure mechanism of the wall under pseudo-static loading is illustrated in Figure 3.4.10 (a).

Table 3.4.7 (a) Performance of Case 1b: M-O based design to 0.2 g under static and pseudo-static loading.

	Design Basis ULS	Final excavation FS=1.0	Onset of instability FS = 1.57	Design acceleration 0.2 g	Maximum acceleration 0.22 g
Displacement (top of wall) (mm)	-	-3	-32	140	1040
Displacement (maximum) (mm)	-	12	163	140	1015
Wall BM (at anchor) (KNm/m)	67	52	52	64	68
Wall BM (below anchor) (KNm/m)	105	32	105 ²	86	105 ²
Anchor force (KN/m)	218 ¹	164	173	175	201

¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load, so anchor load of 159 kN/m (92 percent of characteristic breaking load) exceeds the test load but anchor is considered unlikely to fail.

² Wall element is yielding in bending.

As for the gravity only design (Case 1a), the collapse mechanism of the wall under gravity only loading appears to be hinging of the wall element with significant “bulging” of the wall into the excavation and development of an active soil wedge behind the wall. The strength of the wall element is, therefore, limiting the factor of safety. In this case the anchor force hardly increases above its initial pre-load value and well below the test load.

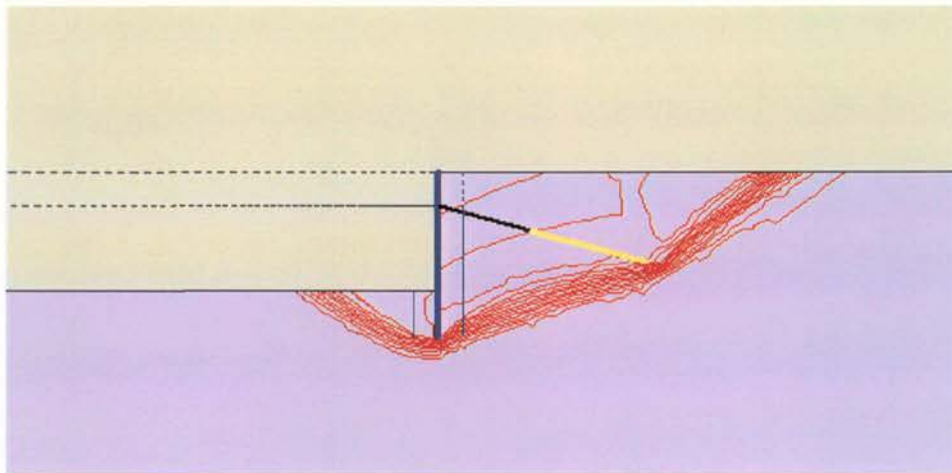


Figure 3.4.10 (a). Contours of incremental displacement at maximum pseudo-static acceleration of 0.22 g for Case 1c.

The wall achieved a maximum pseudo-static acceleration of 0.22 g with a failure mechanism that looks very like the assumed external stability limiting equilibrium failure model (Figure 2.2.2 (c)). The factor of safety at the design acceleration of 0.2 g was found to be 1.08, slightly less than the value of 1.16 calculated using the limiting equilibrium external wedge analysis.

3.4.11 Performance of Case 1c under seismic loading

The performance of Case 1c under seismic loading was determined by applying the suite of three scaled earthquake time-history records to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g, as for the gravity only design, Case 1a.

Results from all of the analyses for the Case 1c: M-O design to 0.2 g are summarised in Figures 3.4.11 (a), (b), (c), and (d).

The same trend was exhibited in wall performance among the suite of three earthquake records but overall the response was much superior to the gravity only design (Case 1a) with greatly reduced displacements: Displacements were modest for the Parkfield and Sierra Madre records (up to 18 mm at the crest and 47 mm at the “bulge”) but still quite large for the Loma Prieta record (29 mm at the crest and 106

mm at the “bulge”). Wall bending moments remained comfortably below yield for the Parkfield and Sierra Madre records but were close to yield at the end of the 0.6 g scaled Loma Prieta record. Tie-back anchor forces were barely affected by shaking for all of the runs and remained close to the initial pre-loading.

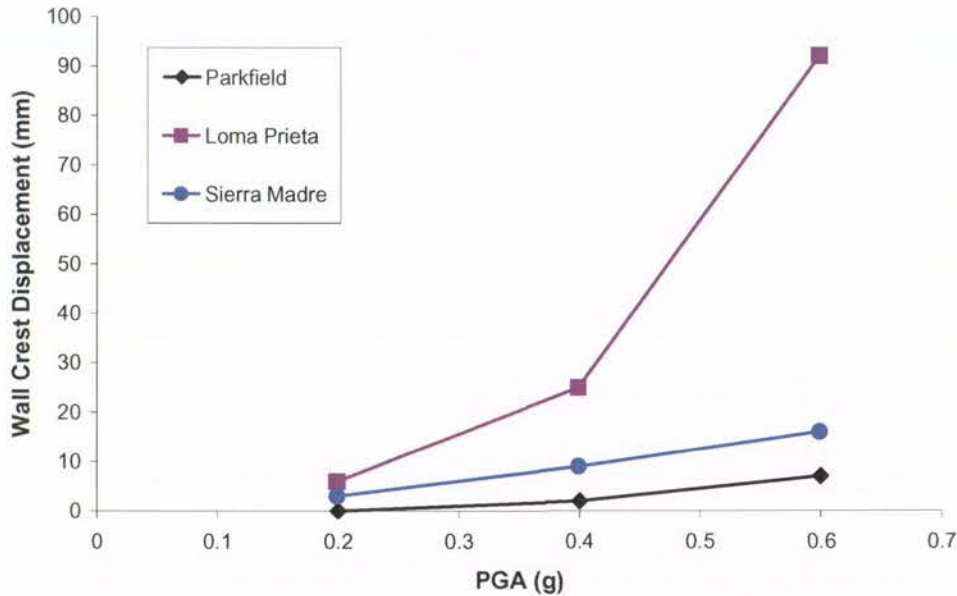


Figure 3.4.11 (a). Accumulated wall crest displacement after earthquake for Case 1c: M-O design to 0.2 g.

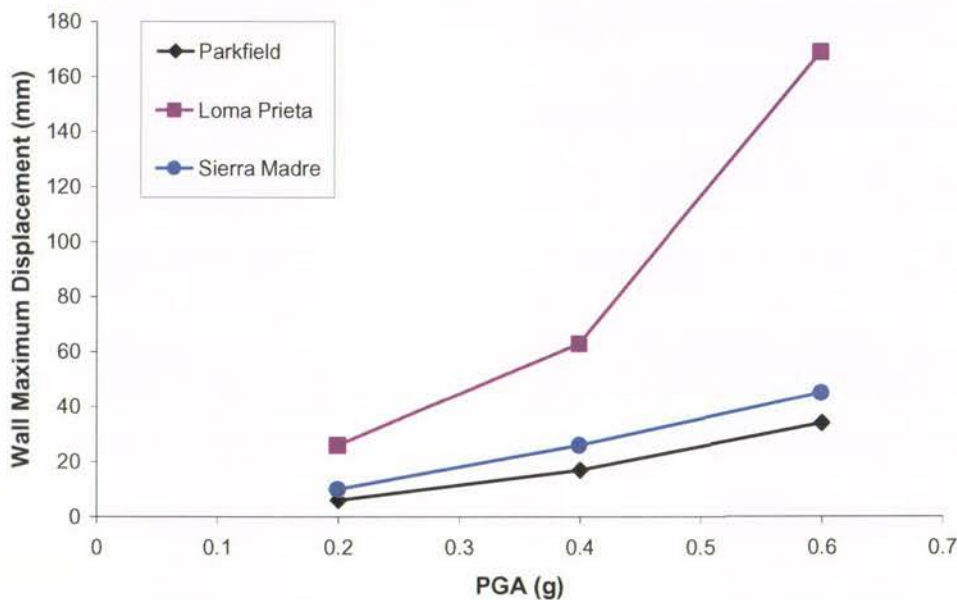


Figure 3.4.11 (b). Accumulated wall displacement below level of tie-back anchor after earthquake for Case 1c: M-O design to 0.2 g.

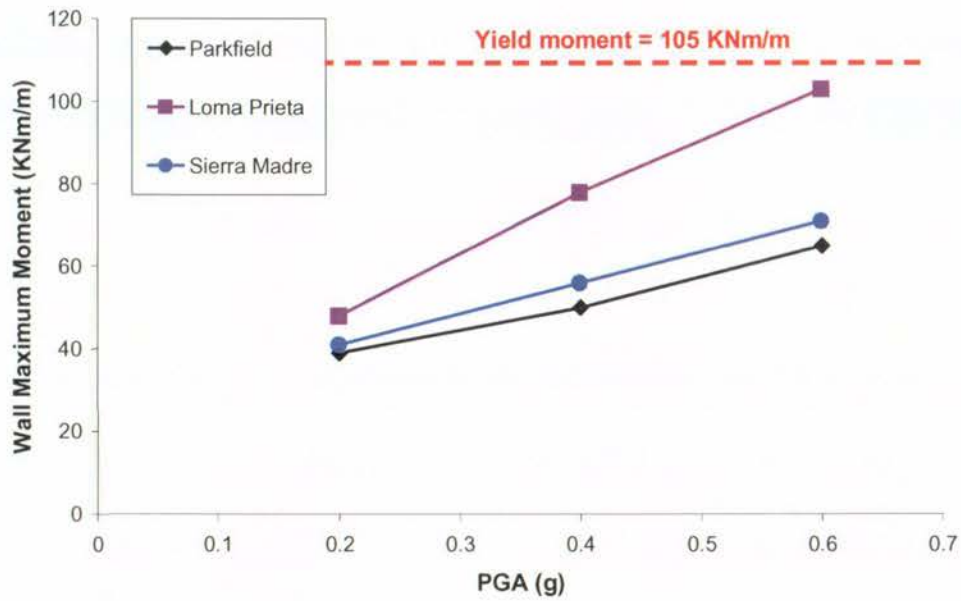


Figure 3.4.11 (c). Maximum wall bending moment after earthquake for Case 1c: M-O design to 0.2 g.

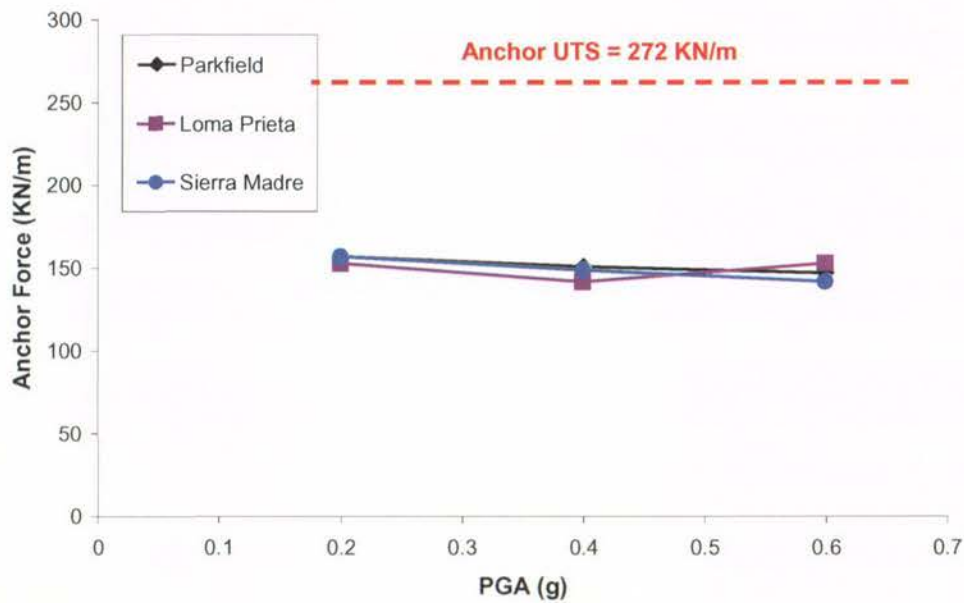


Figure 3.4.11 (d). Anchor force after earthquake for Case 1c: M-O design to 0.2 g..

Generally, the performance of the wall was good and showed a marked improvement in performance over the gravity only design although at the cost of significantly more materials in both wall elements and anchors. A more detailed comparison among all the design cases is given in Section 3.4.18.

3.4.12 Case 1d: M-O based design to 0.3 g

The wall for Case 1d was designed using the synthesized procedure for earthquake design outlined in Section 2.3.6. Detailed calculations are given in Appendix A and are summarised here:

Table 3.4.12 (a). Design values for case study Sand 1d: M-O based design 0.3 g

Design Parameter	Value
Design acceleration	0.3 g
Apparent earth pressure, p	52 kN/m ²
Anchor design load (horizontal)	191 kN/m
Base reaction	53 kN/m
Negative bending moment (at anchor)	50 kNm/m
Maximum bending moment (below anchor)	79 kNm/m
ULS design bending moment, M^*	126 kNm/m

The wall structural elements were designed using these basic calculated design values as follows. For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections).

Table 3.4.12 (b). Design solutions for case study Sand 1d: M-O based design 0.3 g.

Design Solution	Value
Anchor cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	356 mm ² per anchor (3.56 strands per anchor)
Anchor free length	8 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	95% ¹ of 250UC72.9
Depth of embedment	3.3 m

¹ Section properties scaled for purpose of the study.

The anchor free length for Case 1d was determined from the inclination of the M-O active wedge slip plane calculated for the soil strength *reduced* by the factor of safety for internal stability. This adjustment has the effect of increasing the free length to

allow for uncertainty in soil strength parameter and ensure that the anchor free length extends beyond the active soil zone in all cases.

The results of the internal and external stability checks are given in Table 3.4.12 (c) and refer to the condition with pseudo-static horizontal acceleration of 0.3 g.

Table 3.4.12 (c). Internal and external stability checks for case study Sand 1d: M-O based design 0.3 g

Stability Case ¹	FS
Internal stability	1.18
External stability	1.12

¹ Refer Figure 2.2.2 (c)

3.4.13 Performance of Case 1d under gravity and pseudo-static loading

The performance of Case 1d was measured using the same PLAXIS modelling sequences as for the gravity design: First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor force were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability. Finally, a pseudo-static acceleration was applied and increased until the model became unstable. A summary of the main performance parameters is given in Table 3.4.13 (a) and the failure mechanism of the wall under pseudo-static loading is illustrated in Figure 3.4.13 (a).

Table 3.4.13 (a) Performance of Case 1d: M-O based design 0.3 g under static and pseudo-static loading.

	Design Values (ULS)	End of Construction	Onset Instability, FS = 1.68	Design ³ acceleration 0.3 g	Maximum ³ acceleration 0.234 g
Displacement (top of wall) (mm)	-	-9	-108	-	181
Displacement (maximum) (mm)	-	8	194	-	232
Wall BM (at anchor) (KNm/m)	50	64	40	-	74
Wall BM (below anchor) (KNm/m)	79	28	126 ²	-	93

Anchor force (KN/m)	263 ¹	197	175	-	214
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¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load. Anchor pre-load = design load = 198 KN

² Wall element is yielding in bending.

³ Design pseudo-static acceleration not achieved by PLAXIS model (discussed below).

Case 1d was found to have a high factor of safety under gravity loading, as expected given the high design acceleration. The failure mechanism under gravity load appears to be similar to the gravity design with hinging of the wall element allowing an internal stability failure as illustrated in Figure 3.4.13 (a).

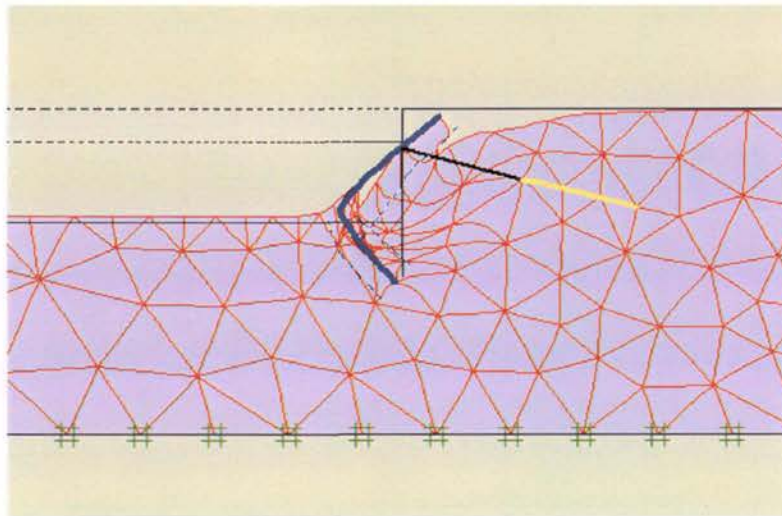


Figure 3.4.13 (a)

Under pseudo-static acceleration, the model became unstable at 0.235 g, much less than the design acceleration of 0.3 g. The reasons for the instability are unclear but do not appear to be caused by any weakness or shortcoming of the wall design. Rather, the model seems to be undergoing a deep seated failure, as illustrated in Figure 3.4.13 (b). It is possible that the instability is a numerical problem and a limitation of PLAXIS: Pseudo-static acceleration is, after all, an artificial loading case and not realistic.

It is impossible to predict the deep seated failure mechanism implied from the PLAXIS output (Figure 3.4.13 (b)) by using typical limiting equilibrium modelling, invoking "rigid" sliding blocks. The PLAXIS model, however, includes the elastic deformations as well as rigid body motions of the relevant soil blocks and a "hybrid" failure mechanism including both shear rupture of the soil along the planes indicated as well as compression of the soil mass on the left hand side of the soil block is indicated.

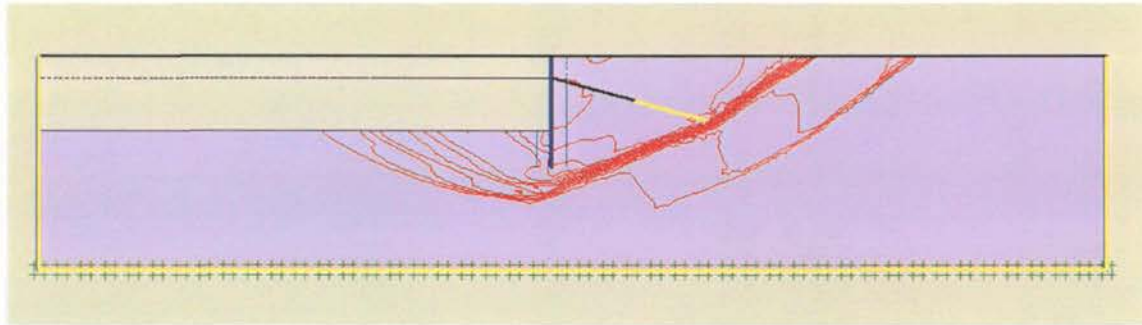


Figure 3.4.13 (b). Case 1d: Contours of incremental displacements at onset of instability, pseudo-static acceleration of 0.235 g.

A number of different analyses were attempted to try and eliminate modelling instability as a cause of the premature failure of the model, including varying the soil model (hardening soil model and Mohr-Coulomb models) and varying the soil boundary conditions (rigid boundaries and rotating, simple shear boundaries). All of these variations gave more-or-less the same outcome. The PLAXIS dynamic modelling system was also tried because of the greater inherent stability arising from the inclusion of soil inertia: The base acceleration of the model was increased to 0.3 g using a ramp function and then held constant. A very similar, deep seated failure mechanism was observed as shown in Figure 3.4.13 (c).

For one analysis, the anchor length was increased by 5 m in an effort to try and “push” the failure surface further back from the wall. While successful in moving the failure surface as desired, the maximum pseudo-static acceleration achieved was about the same.

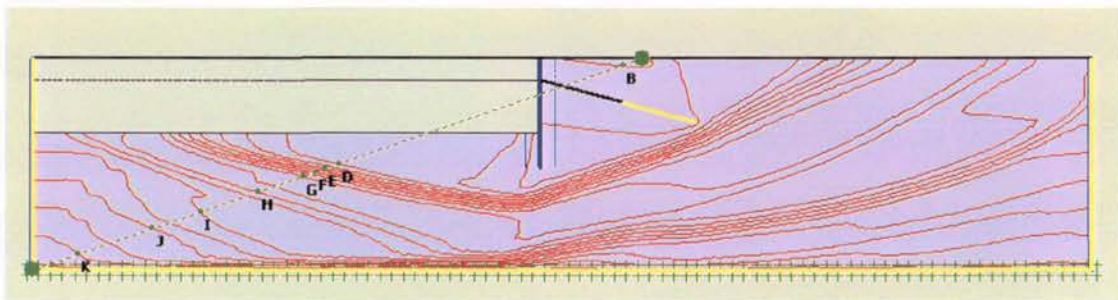


Figure 3.4.13 (c). Case 1d: Contours of incremental displacements at onset of instability, dynamic acceleration of 0.3 g.

Irrespective of whether or not the unexpected deep seated failure mechanism is real or a modelling effect, the wall system and supported soil remained stable although undergoing a rigid body translation.

3.4.14 Performance of Case 1d under seismic loading

The performance of Case 1d (M-O based design to 0.3 g) under seismic loading was determined by applying the same suite of three scaled earthquake time-history records to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g. Wall performance was determined by reference to the same indicators as for the previous cases as summarised in Table 3.4.14 (a) and Figures 3.4.14 (a), (b), (c), and (d).

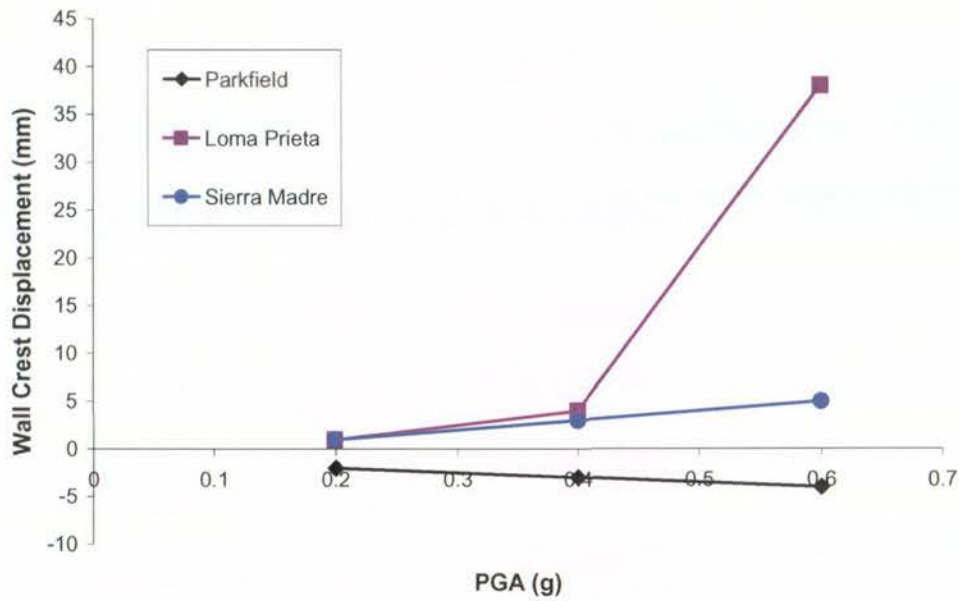


Figure 3.4.14 (a). Accumulated wall crest displacement after earthquake for Case 1d: M-O Design to 0.3 g.

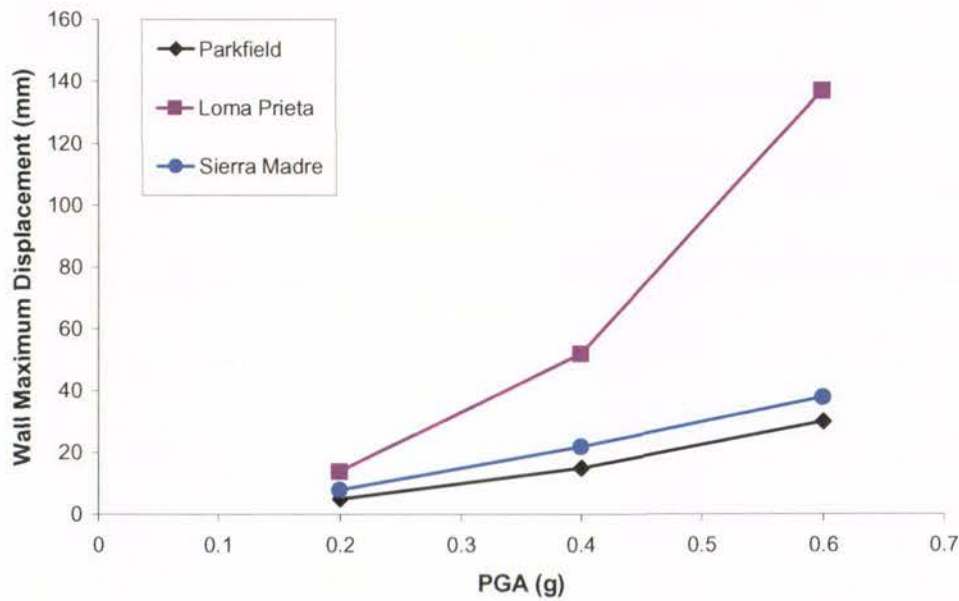


Figure 3.4.14 (b). Accumulated wall displacement below level of tie-back anchor after earthquake for Case 1d: M-O Design to 0.3 g.

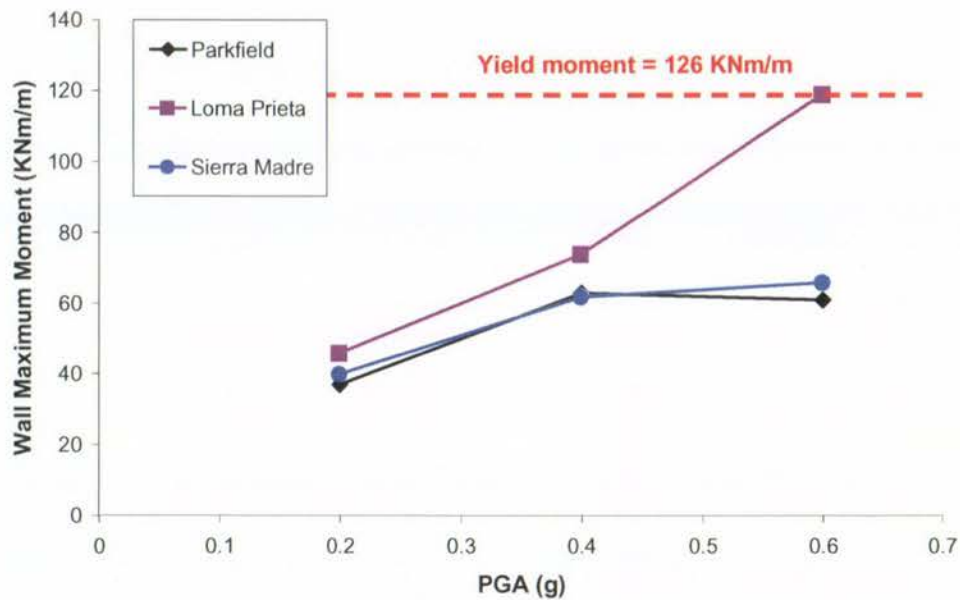


Figure 3.4.14 (c). Maximum wall bending moment after earthquake for Case 1d: M-O Design to 0.3 g.

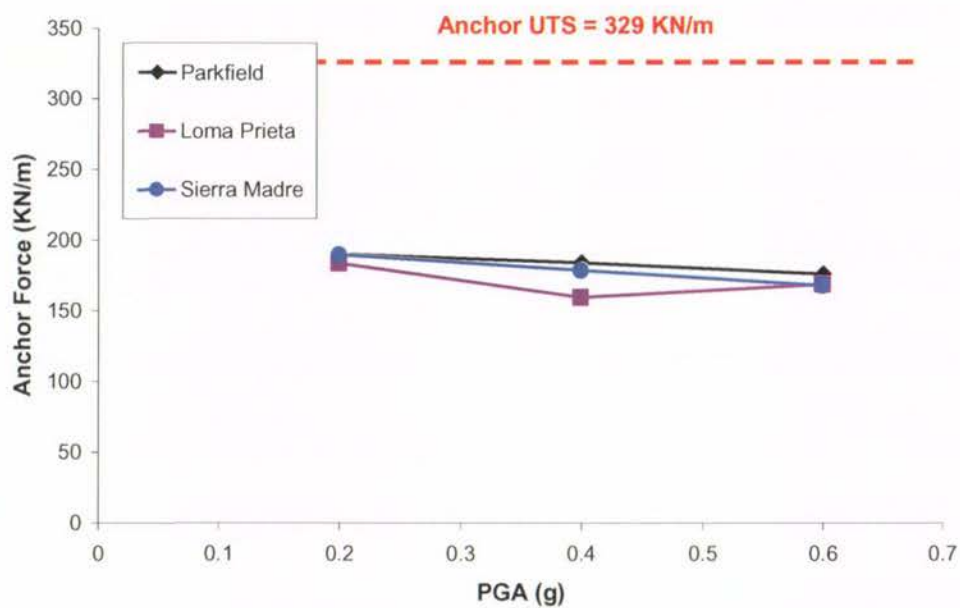


Figure 3.4.14 (d). Anchor force after earthquake for Case 1d: M-O Design to 0.3 g.

Generally, the performance of the wall was good. A more detailed comparison among all the design cases is given in Section 3.4.18.

3.4.15 Case 1e: M-O based design to 0.4 g

The wall for Case 1e was designed using the synthesized procedure for earthquake design outlined in Section 2.3.6. Detailed calculations are given in Appendix A and are summarised here:

Table 3.4.15 (a). Design values for case study Sand 1e: M-O based design 0.4 g

Design Parameter	Value
Design acceleration	0.4 g
Apparent earth pressure, p	63.4 kN/m ²
Anchor design load (horizontal)	232 kN/m
Base reaction	64 kN/m
Negative bending moment (at anchor)	61 kNm/m
Maximum bending moment (below anchor)	96 kNm/m
ULS design bending moment, M^*	153 kNm/m

The wall structural elements were selected using these basic calculated design values as follows. For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections).

Table 3.4.15 (b). Design solutions for case study Sand 1e: M-O based design 0.4 g.

Design Solution	Value
Anchor cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	430 mm ² per anchor (4.3 strands per anchor)
Anchor free length	9.2 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	99% ¹ of 250UC89.5
Depth of embedment	4.3 m

¹ Section properties scaled for purpose of the study.

For Case 1e the depth of embedment calculated using Broms method (3.8 m) had to be increased to 4.3 m to provide the desired minimum factor of safety against internal stability ($FS = 1.1$).

The anchor free length for Case 1e then was determined from the inclination of the M-O active wedge slip plane calculated for the soil strength *reduced* by the factor of safety for internal stability. This adjustment has the effect of increasing the free length to allow for uncertainty in soil strength parameter and ensure that the anchor free length extends beyond the active soil zone in all cases.

The results of the internal and external stability checks are given in Table 3.4.15 (c) and refer to the condition with pseudo-static horizontal acceleration of 0.4 g.

Table 3.4.15 (c). Internal and external stability checks for case study Sand 1e: M-O based design 0.4 g

Stability Case ¹	FS
Internal stability	1.1
External stability	1.0

¹ Refer Figure 2.2.2 (c)

A factor of safety of 1.0 for external stability is considered to be adequate for the pseudo-static design case. The external stability failure mechanism is considered to be the preferred mode of yielding for the wall since it is ductile and provides protection against overloading of the wall elements.

3.4.16 Performance of Case 1e under gravity and pseudo-static loading

The performance of Case 1e was measured using the same PLAXIS modelling sequences as for the gravity design: First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor force were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability. Finally, a pseudo-static acceleration was applied and increased until the model became unstable. A summary of the main performance parameters is given in Table 3.4.16 (a).

Table 3.4.16 (a) Performance of Case 1e: M-O based design 0.4 g under static and pseudo-static loading.

	Design Values (ULS)	End of Construction	Onset Instability, FS = 1.79	Design ³ acceleration 0.4 g	Maximum ³ acceleration 0.235 g
Displacement (top of wall) (mm)	-	-15	-83	-	>200
Displacement (maximum) (mm)	-	7	106	-	>200
Wall BM (at anchor) (KNm/m)	61	78	45	-	85
Wall BM (below anchor) (KNm/m)	96	36	153 ²	-	94
Anchor force (KN/m)	319 ¹	237	188	-	248

¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load. Anchor pre-load = design load = 240 kN

² Wall element is yielding in bending.

³ Design pseudo-static acceleration not achieved by PLAXIS model (discussed in Section 3.4.10).

Case 1e was found to have a high factor of safety under gravity loading, as expected given the high design acceleration. The failure mechanism under gravity load was similar to the other design cases.

3.4.17 Performance of Case 1e under seismic loading

The performance of Case 1e (M-O based design to 0.4 g) under seismic loading was determined by applying the same suite of three scaled earthquake time-history records to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g. Wall performance was determined by reference to the same indicators as for the previous cases as summarised in Table 3.4.17 (a) and Figures 3.4.17 (a), (b), (c), and (d).

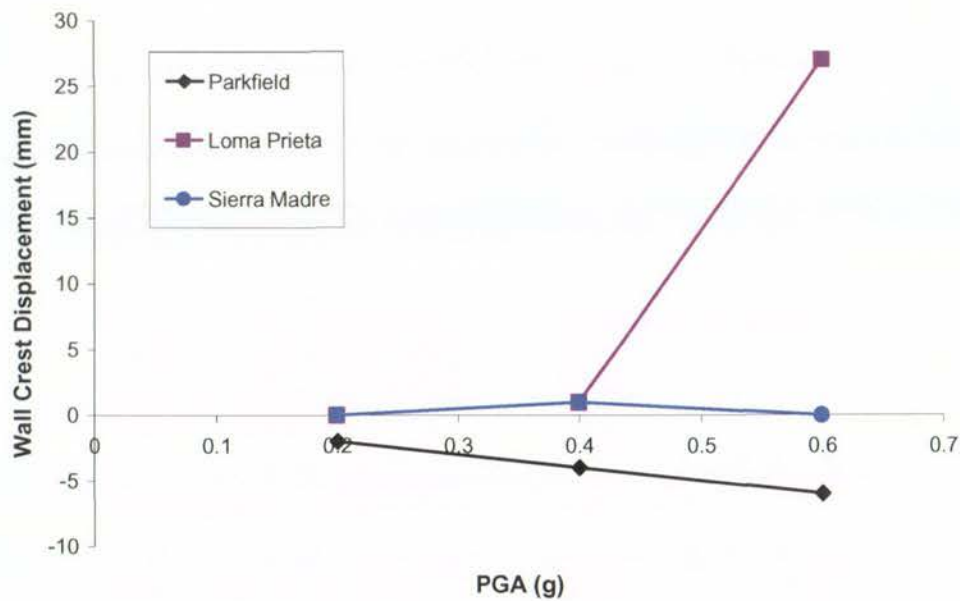


Figure 3.4.17 (a). Accumulated wall crest displacement after earthquake for Case 1e: M-O Design to 0.4 g.

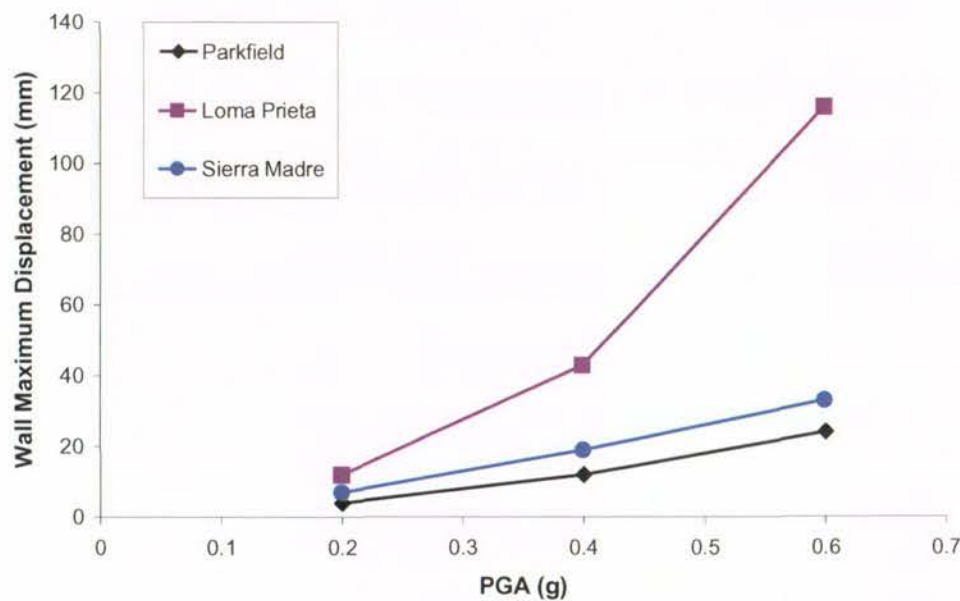


Figure 3.4.17 (b). Accumulated wall displacement below level of tie-back anchor after earthquake for Case 1e: M-O Design to 0.4 g.

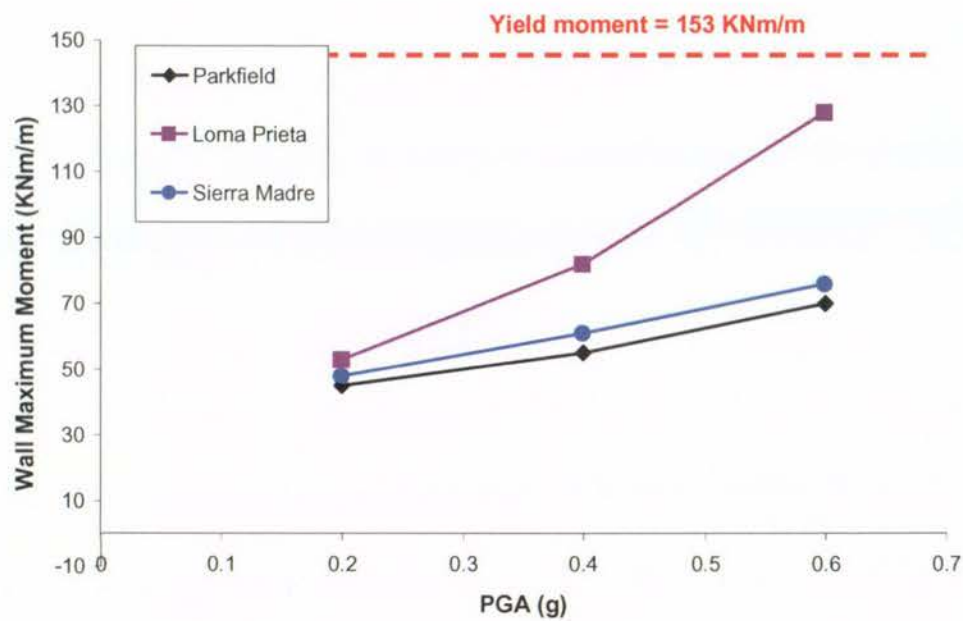


Figure 3.4.17 (c). Maximum wall bending moment after earthquake for Case 1e: M-O Design to 0.4 g.

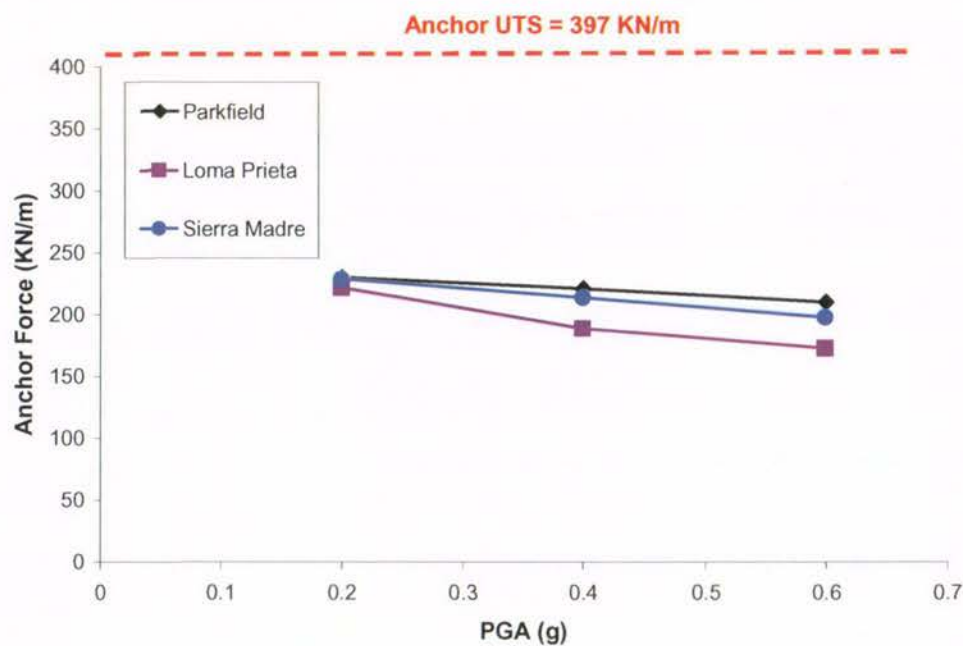


Figure 3.4.17 (d). Anchor force after earthquake for Case 1e: M-O Design to 0.4 g.

3.4.18 Comparison of design cases

For Case Study 1, a 7 m deep tied-back retaining wall in sand, five design variations were considered each with a different nominal design horizontal acceleration ranging from 0 g (gravity design) to 0.4 g. Each was designed using the synthesized design procedure based on the FHWA gravity design procedure. The resulting design values are compared in Table 3.4.18 (a) and the design solutions compared in Table 3.4.18 (b).

For the purposes of this research project, design solutions were perfectly optimised by taking crude proportions of whole steel sections or fractions of anchor strands. In real design cases section sizes can be optimised by changing spacing to some extent or simply rounding up to the next heaviest section.

From Table 3.4.18 (a), the increase in design apparent earth pressure is modest for the first increment of design acceleration to 0.1 g, but increases more rapidly with each subsequent step. For the greatest design acceleration of 0.4 g, the apparent earth pressure is more than doubled, resulting in more than doubling of the anchor force, base reaction, and bending moments.

Table 3.4.18 (a). Comparison of design values for case study Sand 1.

Design Parameter	Case 1a	Case 1b	Case 1c	Case 1d	Case 1e
Design acceleration	0 g	0.1 g	0.2 g	0.3 g	0.4 g
Apparent earth pressure, p (KN/m ²)	30	36	43	52	63
Anchor design load (horizontal) (KN/m)	108	131	158	191	232
Base reaction (KN/m)	30	36	44	53	64
Negative bending moment (at anchor) (KNm/m)	28	35	42	50	61
Maximum bending moment (below anchor) (KNm/m)	45	54	65	79	96
ULS design bending moment, M^* (KNm/m)	72	87	105	126	153

The design solutions for Case Study 1 are compared in Table 3.4.18 (b). The anchor and soldier pile sizes were optimised in an unrealistic way by taking proportions of whole member sizes. This optimisation was done to provide a more clear indication of trends for the purposes of the study.

For Case Study 1, the anchor free lengths were determined as follows: The angle of inclination of the M-O active wedge slip plane was calculated for the soil strength **reduced** by the factor of safety for internal stability (as shown in the calculations in Appendix A). The anchor free lengths were extended to a line drawn from the toe of the embedded soldier piles to the ground surface at the calculated angle of inclination

of the active slip plane. The objective of this procedure was to ensure that the anchor bond length was outside of any possible active shear zone.

Table 3.4.18 (b). Comparison of design solutions for case study Sand 1.

Design Solution	Case 1a	Case 1b	Case 1c	Case 1d	Case 1e
Design acceleration	0 g	0.1 g	0.2 g	0.3 g	0.4 g
No. strands using super strand (100 mm ²) anchors at 2 m centres, inclined 15 degrees	2.01	2.44	2.92	3.56	4.30
Anchor free length ¹	3.9 m	4.1 m	5.7 m	8 m	9.2 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	94% ² of 200UC52.2	98% ² of 200UC59.5	79% ² of 250UC72.9	95% ² of 250UC72.9	99% ² of 250UC89.5
Depth of embedment	2.1 m	2.4 m	2.8 m	3.3 m	4.3 m

¹ Calculated using recommended procedure. In practice, a minimum free length of 5 m is recommended for strand anchors.

² Section properties scaled for purpose of the study.

The variation of anchor design, both free length and number of strands is plotted versus design acceleration in Figure 3.4.18 (a) and shows a more-or-less linear increase with increase in design acceleration. The variation of soldier pile design, both section weight and depth of embedment is shown in Figure 3.4.18 (b) and shows a non-linear, compounding increase with increase in design acceleration.

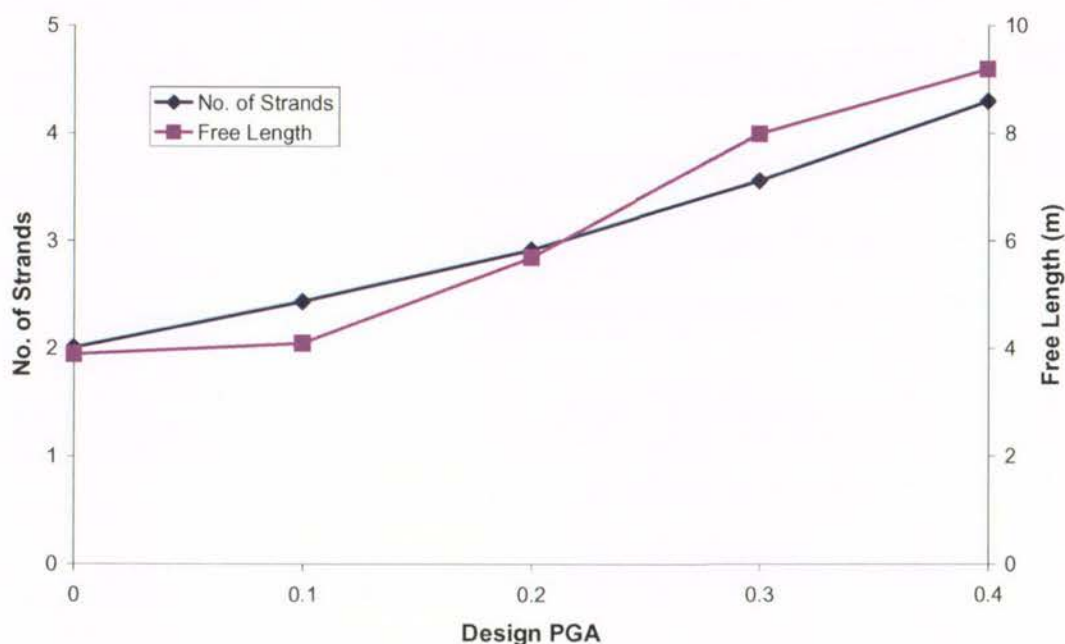


Figure 3.4.18 (a) Variation of anchor design parameters with design acceleration.

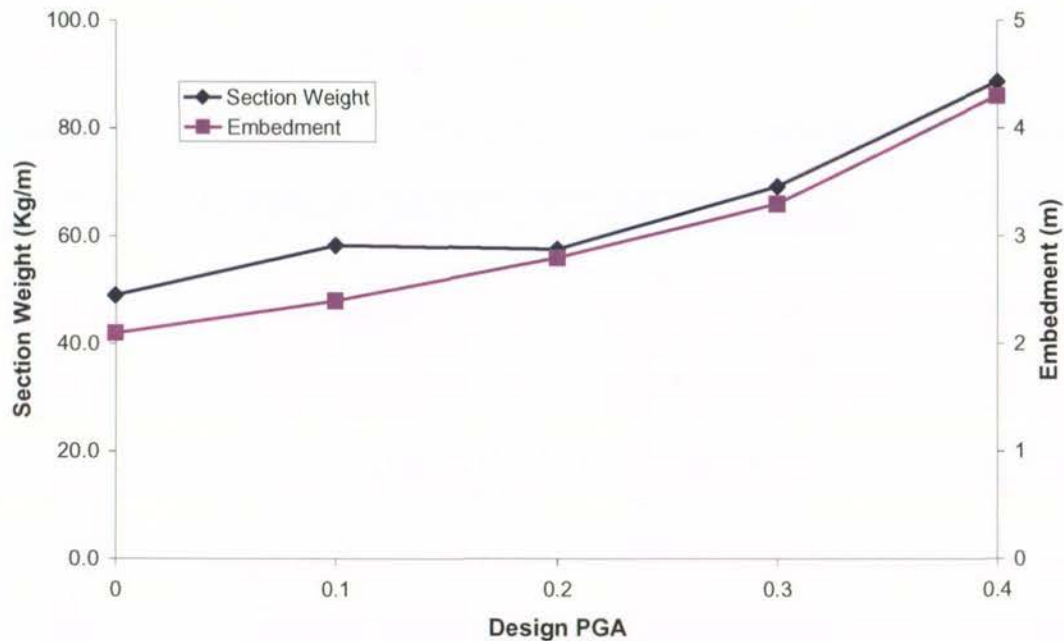


Figure 3.4.18 (b) Variation of soldier pile design parameters with design acceleration.

A crude cost index was derived for comparative purposes for both the soldier piles and the anchors. For the soldier piles the index was calculated by multiplying the section weight/m times the pile length (wall height plus embedment) and for the anchors by multiplying the number of strands times the anchor length (free length plus the bond length of 7 m). These indices were normalised by dividing by the values for the gravity only (0 g) designs.

These cost indices for soldier piles and anchors were kept separate because the comparative cost of anchor installation and soldier pile installation will depend on site specific factors.

A cost-performance comparison is made in Figure 3.4.18 (c) by plotting the cost indices with the average wall displacements. The wall displacements were averaged for each of the three earthquake time histories considered (Loma Prieta, Sierra Madre, and Parkfield), with separate curves shown for each of the three scales of peak ground acceleration considered (0.2 g, 0.4 g, 0.6 g)..

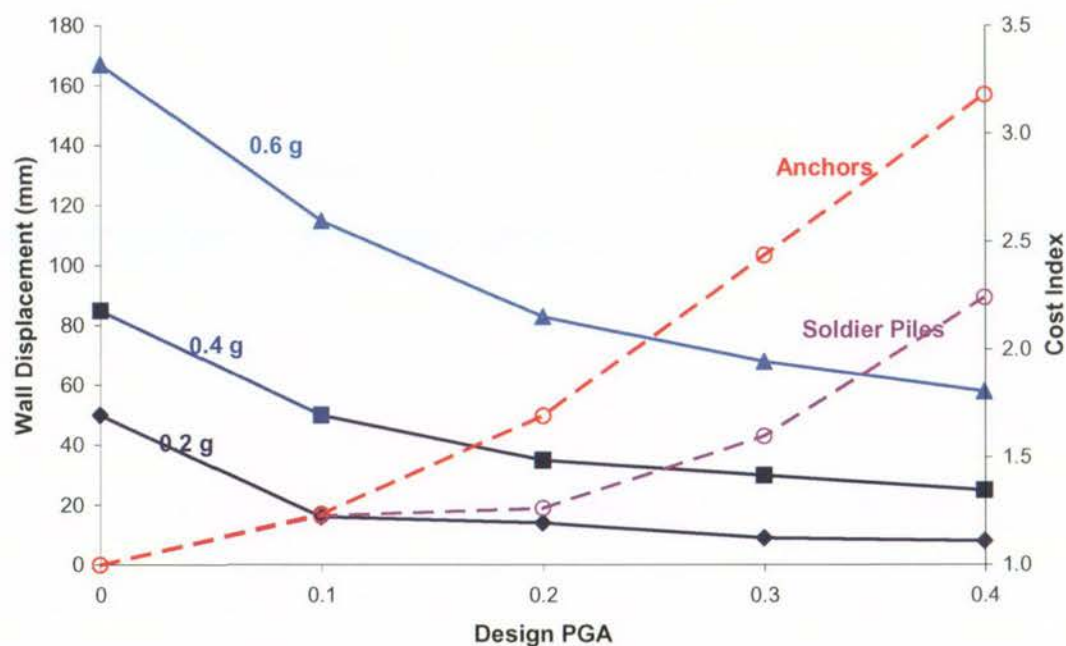


Figure 3.4.18 (c) Cost-Performance summary for Case Study 1.

3.4.19 Conclusions

The curves from Figure 3.4.18 (a) show that as the wall was designed to resist greater levels of quasi-static horizontal acceleration the wall performance in terms of permanent displacement improved significantly for all levels of earthquake shaking. However, the cost of the wall also increased substantially, especially for higher levels of design acceleration.

The greatest benefit-cost ratio was for Case 1b where the wall was designed to resist a horizontal acceleration of 0.1 g, resulting in a cost increase of about 20 percent and a reduction in permanent displacement ranging from 68 percent for the 0.2 g earthquakes to 31 percent for the 0.6 g earthquakes.

Increasing the design acceleration to 0.2 g (Case 1c) increased cost by a further 25 percent and gave a further reduction in permanent displacement from 12 percent for the 0.2 g earthquakes to 30 percent for the 0.4 g and 0.6 g earthquakes.

These benefit-cost ratios indicate that the optimum design is probably gained by making the design acceleration about $\frac{1}{2}$ the PGA of the design earthquake (e.g. for design earthquake with PGA = 0.2 g make the design acceleration 0.1 g, and for a design earthquake with PGA = 0.4 g make the design acceleration 0.2 g). Such a recommendation would be in keeping with accepted practice which is to design retaining walls to resist pseudo-static acceleration of between $\frac{1}{2}$ and $\frac{1}{3}$ of the design earthquake PGA.

3.5 Case Study 2: Two Rows of Anchors in Sand

3.5.1 Case Study Description

This case is for a 12 m deep excavation in sand. It is assumed that the water table has been drawn down to the base of the excavation. Typically, such an excavation would be made using concrete soldier piles with sprayed concrete facing for a permanent installation or galvanised steel UC sections with timber lagging. Two rows of tie-back ground anchors is usually found to be the most economical solution for a 12 m high wall, requiring a three stage excavation process: Installation of soldier piles from the ground surface, excavation to 3 m depth, installation and stressing of the first row of anchors, excavation to 8 m depth, installation and stressing of the second row of anchors, and final excavation to depth.

A cross-section through the PLAXIS model is shown in Figure 3.5.1 (a). The anchor spacing was optimised with the depth to the first row of anchors at 3 m and the second row at 8 m deep. As for Case Study 1, the anchor inclination is set at 15 degrees, about the flattest angle practicable. The bond length (yellow line, PLAXIS geogrid element) is set at 7 m which is typical for ground anchors in sandy soils assuming that multi-stage pressure grouting is utilised. The anchor free length (black line, PLAXIS node-to-node anchor), was determined using the trial design procedure.

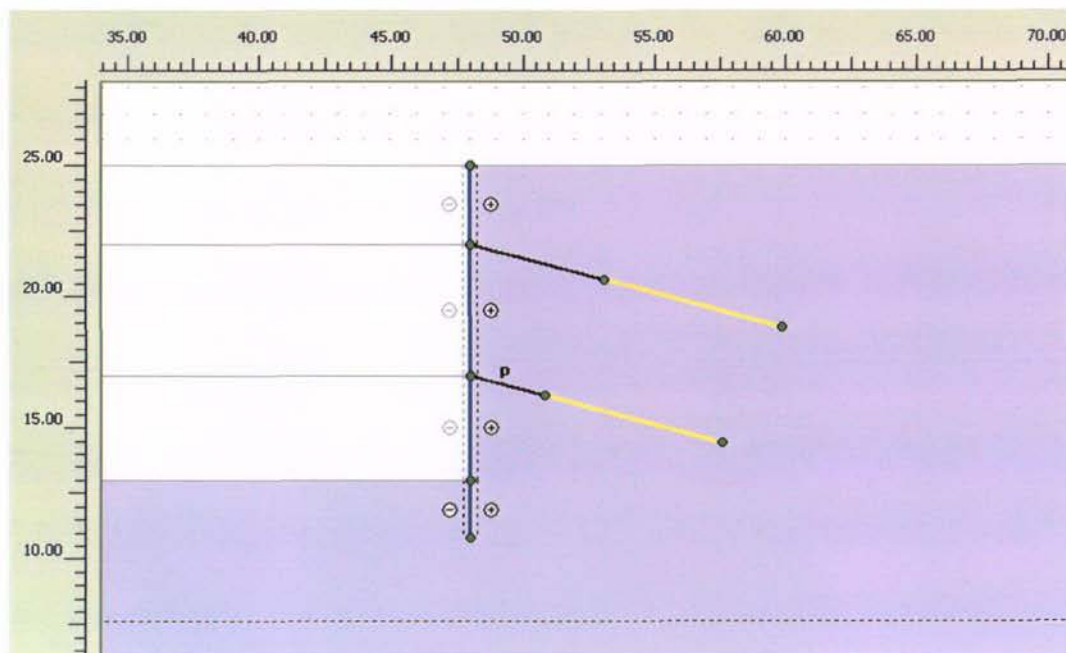


Figure 3.5.1 (a) PLAXIS model Sand 2: Gravity based design.

The soil properties were the same as for Case Study 1, considered to be typical for medium-dense sand, with the properties as given in Table 3.5.1 (a)

Table 3.5.1 (a) Soil properties for case studies in sand.

Property	Symbol	Value
Density, unsaturated	γ_{unsat}	16 kN/m ³
Density, saturated	γ_{sat}	18 kN/m ³
Effective cohesion	c'	1 kN/m ²
Effective friction	ϕ'	35 degrees
Soil model		Hardening soil
Young's Modulus	E_{50}^{ref}	30 MN/m ³
Young's Modulus (unload/reload)	$E_{\text{ur}}^{\text{ref}}$	90 MN/m ³

3.5.2 Case 2a: Gravity design

Gravity design followed the synthesized design procedure described in Section 2.3.6. Detailed calculations are given in Appendix B and summarised here:

Table 3.5.2 (a). Design values for case study Sand 2a: Gravity design.

Design Parameter	Value
Apparent earth pressure, p	42 kN/m ²
Anchor 1 design load (horizontal)	188 kN/m
Anchor 2 design load (horizontal)	185 kN/m
Base reaction	31 kN/m
Cantilever bending moment (at anchor 1)	91 kNm/m
Maximum bending moment (at anchor 2)	91 kNm/m
ULS design bending moment, M^*	145 kNm/m

The wall structural elements were designed using these basic calculated design values as follows. For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections).

Table 3.5.2 (b). Design solutions for case study Sand 2a: Gravity design.

Design Solution	Value
Anchor 1 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	350 mm ² per anchor (3.5 strands per anchor)
Anchor 2 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	344 mm ² per anchor (3.44 strands per anchor)
Anchor 1 free length	5.3 m
Anchor 2 free length ¹	2.9 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	94% ² of 250UC89.5
Depth of embedment	2.2 m

¹ Calculated using FHWA procedure. In practice, a minimum free length of 5 m is recommended for strand anchors.

² Section properties scaled for purpose of the study.

The anchor free length for Case 2a was determined from the inclination of the M-O active wedge slip plane calculated for the *full* soil strength with no reduction to allow for uncertainty in soil strength parameters. The active wedge was assumed to extend from the toe of the embedded soldier piles unlike for the FHWA procedure which assumes that the active wedge extends from the base of the excavation.

Table 3.5.2 (c). Internal and external stability checks for case study Sand 2a: Gravity design.

Stability Case ¹	FS
Internal stability	1.32
External stability	1.32

¹ Refer Figure 2.2.2 (c)

3.5.3 Performance of Case 2a under gravity and pseudo-static loading

The performance of Case 2a designed using the synthesized procedure but considering only the gravity load case was determined by analysing the wall design using PLAXIS. First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor forces were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability. A summary of the main performance parameters is given in Table 3.5.3 (a), the bending moment distribution for the wall element is given in Figure 3.5.3 (a), and the collapse mechanism is illustrated in Figures 3.5.3 (b) and (c).

Table 3.5.3 (a) Performance of Case Study Sand 2a under gravity and pseudo-static loading

	Design Basis ULS	Final excavation FS=1.0	Stability Limit FS=1.32	Maximum pseudo-static acceleration 0.11 g
Displacement (top of wall) (mm)	-	48	180	127
Displacement (maximum) (mm)	-	57	189	117
Wall BM (at anchor 1) (KNm/m)	-145	-76	-77	-77
Wall BM (below anchors) (KNm/m)	145	75	118	125
Anchor 1 force (KN/m)	259 ¹	197	214	197
Anchor 1 force (KN/m)	255 ¹	192	292 ¹	234

¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load, so anchor load of 292 KN/m (92 percent of characteristic breaking load) exceeds the test load but anchor is considered unlikely to fail.

The collapse mechanism of the wall appears to be external failure with a large active wedge incorporating the entire wall and anchorages pushing up a small passive wedge into the excavation.. The modest factor of safety (FS = 1.32) might be improved by increasing the depth of embedment of the soldier piles.

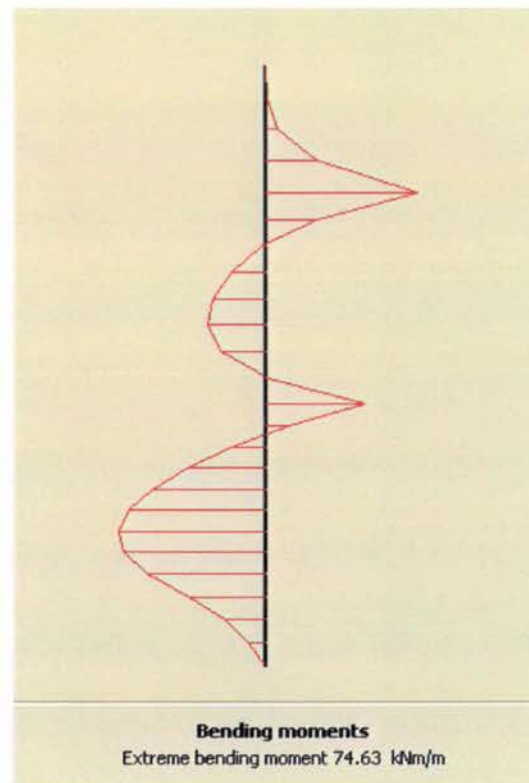


Figure 3.5.3 (a). Wall bending moment versus depth, FS = 1 (full soil strength).

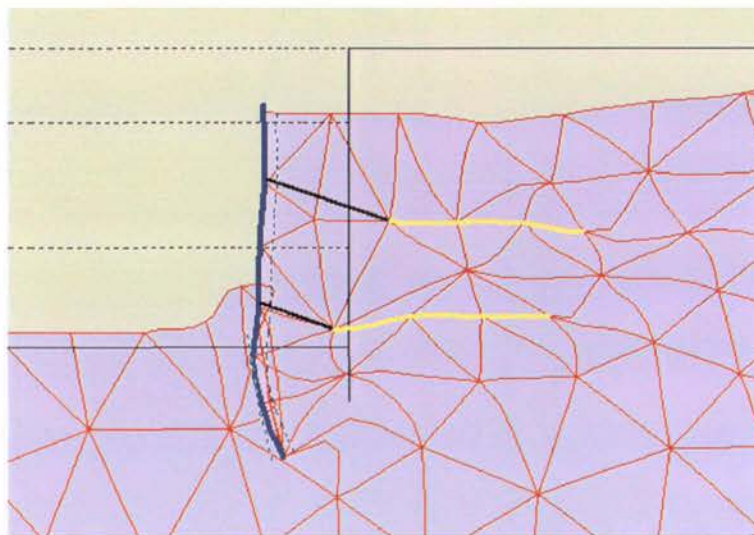


Figure 3.5.3 (b). Deformed mesh at onset of instability, FS = 1.32 (exaggerated scale).

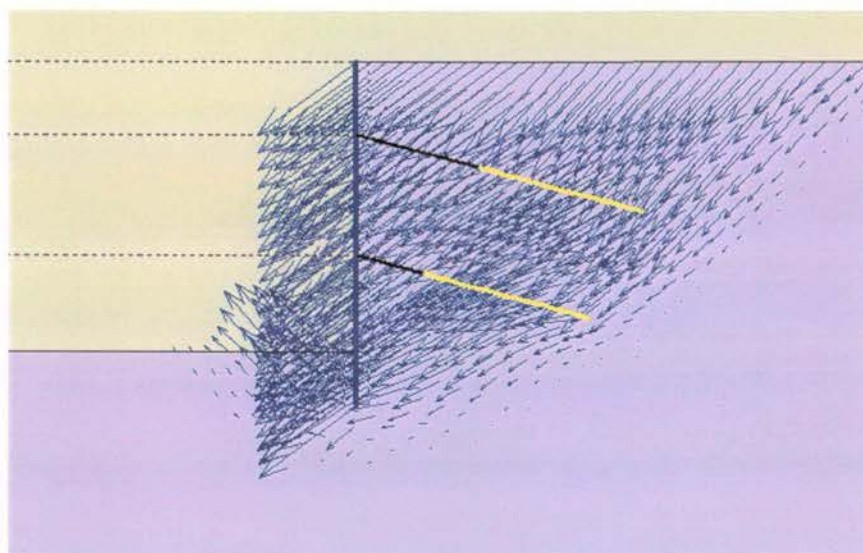


Figure 3.5.3 (c). Soil displacement vectors at onset of instability, $FS = 1.32$.

3.5.4 Evaluation of Case 2a under gravity loading

The factor of safety achieved in the PLAXIS analysis using “phi-c reduction” (lower bound) is the same value estimated using the limiting equilibrium, wedge analyses and is considered satisfactory. Typically, acceptable factors of safety for slope stability analyses using limiting equilibrium methods of analysis (upper bound) are considered to be in the range from $FS = 1.2$ to $FS = 1.5$ for critical slopes.

The factor of safety determined for this case study ($FS = 1.32$) is close to the value calculated using the limiting equilibrium procedure (internal and external stability, both $FS = 1.32$).

The PLAXIS analysis suggests that it may be possible to improve the factor of safety by increasing the depth of embedment of the soldier piles. A prudent designer may choose to do this.

3.5.5 Performance of Case 2a under seismic loading

The performance of Case 2a, gravity only design, under seismic loading was determined by applying only one scaled earthquake time-history record (Loma Prieta) to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g. This record was determined from Case Study 1 to be much more critical than the other earthquake time histories.

Wall performance is indicated primarily by permanent displacement (always outwards) remaining after each earthquake “event”. For the wall of Case Study 2a, the displacement was maximum at either the crest or near to the base below the second row of anchors where the wall typically tends to “bulge” outwards.

The bending moments in the wall elements were critical at either the top row of anchors or below the second row of anchors (the “bulge”) and these were also

monitored together with the anchor forces. Results from all of the analyses for the Case 2a are summarised in Figures 3.5.5 (a), (b), (c), and (d).

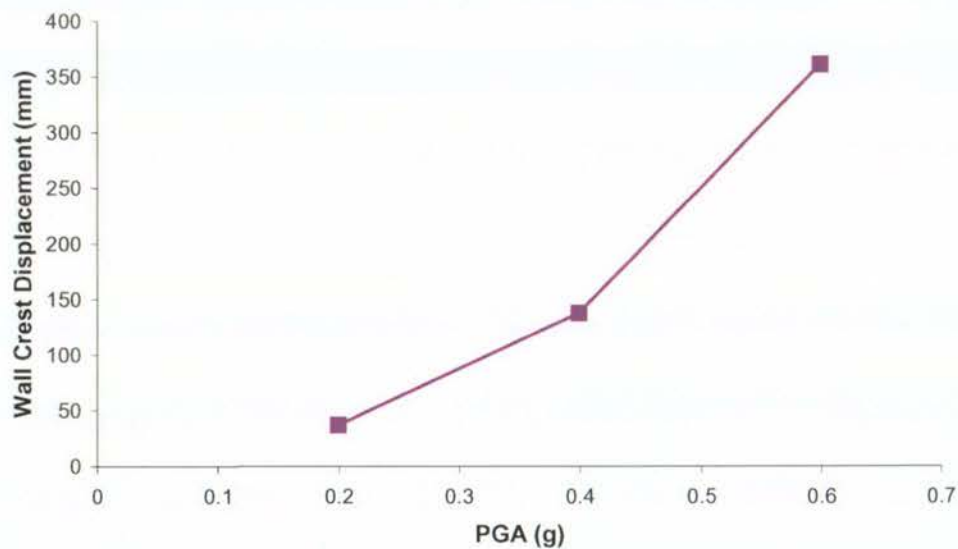


Figure 3.5.5 (a). Accumulated wall crest displacement after Loma Prieta earthquake for gravity only design.

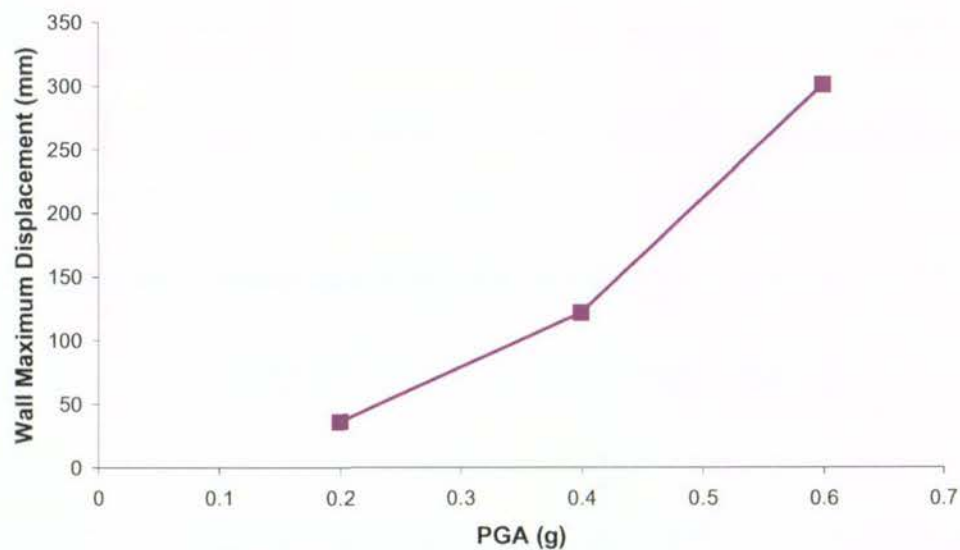


Figure 3.5.5 (b). Accumulated wall displacement below level of tie-back anchor after Loma Prieta earthquake for gravity only design.

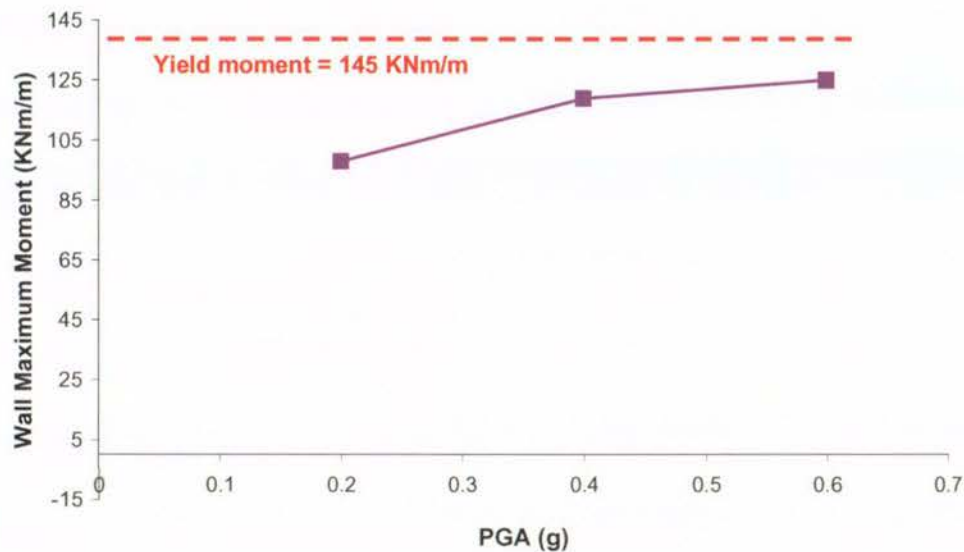


Figure 3.5.5 (c). Maximum wall bending moment after Loma Prieta earthquake for gravity only design.

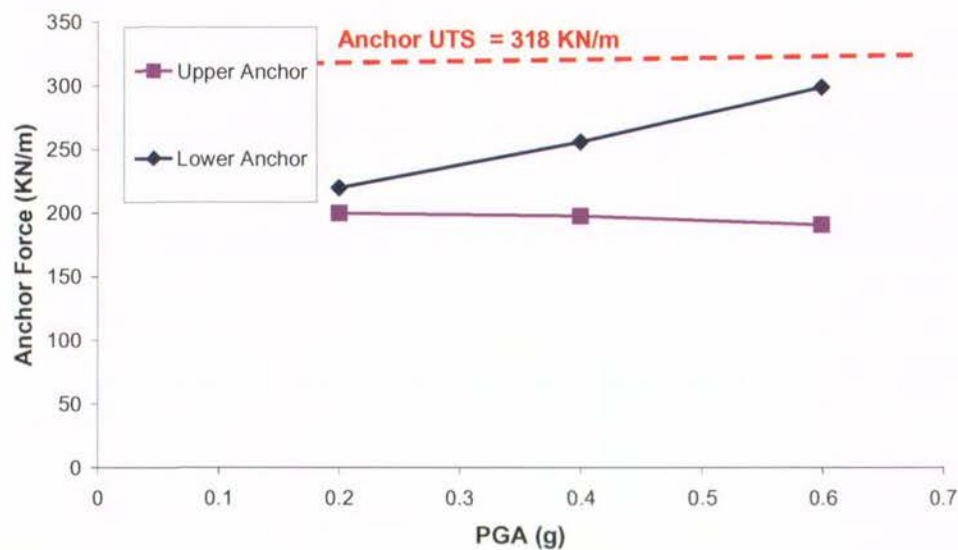


Figure 3.5.5 (d). Anchor force after Loma Prieta earthquake for gravity only design.

Generally, the performance of the wall was surprisingly good given that the design was a standard gravity only procedure with no consideration of seismic effects. For the 0.2 g scaled record, displacements were very modest at less than 40 mm, increasing to 138 mm at 0.4 g, but becoming excessive at 361 mm at the wall crest for

the 0.6 g scaled time history. Wall moment and the lower anchor force were also increasing to quite high levels by the end of the 0.6 g record.

A more detailed comparison among all of the Case 2 design cases is given in Section 3.5.12.

3.5.6 Case 2b: M-O based design to 0.1 g

Design of Case 2b followed the synthesized design procedure described in Section 2.3.6. Detailed calculations are given in Appendix B and summarised here:

Table 3.5.6 (a). Design values for case study Sand 2b: M-O design to 0.1 g.

Design Parameter	Value
Design acceleration	0.1 g
Apparent earth pressure, p	51 kN/m ²
Anchor 1 design load (horizontal)	229 kN/m
Anchor 2 design load (horizontal)	224 kN/m
Base reaction	38 kN/m
Cantilever bending moment (at anchor 1)	110 kNm/m
Maximum bending moment (at anchor 2)	110 kNm/m
ULS design bending moment, M^*	176 kNm/m

The wall structural elements were designed using these basic calculated design values as follows. For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections).

Table 3.5.7 (b). Design solutions for case study Sand 2b: M-O design to 0.1 g.

Design Solution	Value
Anchor 1 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	426 mm ² per anchor (4.2 strands per anchor)
Anchor 2 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	426 mm ² per anchor (4.2 strands per anchor)
Anchor 1 free length	6.3 m
Anchor 2 free length ¹	3.6 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	83% ² of 310UC96.8
Depth of embedment	2.5 m

¹ Calculated using FHWA procedure. In practice, a minimum free length of 5 m is recommended for strand anchors.

² Section properties scaled for purpose of the study.

The anchor free length for Case 2a was determined from the inclination of the M-O active wedge slip plane calculated for the **full** soil strength with no reduction to allow for uncertainty in soil strength parameters. The active wedge was assumed to extend from the toe of the embedded soldier piles unlike for the FHWA procedure which assumes that the active wedge extends from the base of the excavation.

Table 3.5.7 (c). Internal and external stability checks for case study Sand 2b: M-O based design 0.1 g

Stability Case ¹	FS
Internal stability	1.31
External stability	1.13

¹ Refer Figure 2.2.2 (c)

3.5.7 Performance of Case 2b under gravity and pseudo-static loading

The performance of Case 2b was determined using the same PLAXIS modelling sequences as for the gravity design: First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor force were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability. Finally, a pseudo-static acceleration was applied and increased until the model became unstable. A summary of the main performance parameters is given in Table 3.5.7 (a) and the failure mechanism of the wall under pseudo-static loading is illustrated in Figure 3.5.7 (a).

As for the gravity only design (Case 2a), the failure mechanism of the wall under both gravity only loading and pseudo-static loading appears to be external stability with formation of a large active wedge of soil encompassing the wall and both anchors, as shown in Figure 3.5.7 (a)

Table 3.5.7 (a) Performance of Case 2b: M-O based design to 0.1 g under static and pseudo-static loading.

	Design Basis ULS	Final excavation FS=1.0	Onset of instability FS = 1.42	Design acceleration 0.1 g	Maximum acceleration 0.14 g
Displacement (top of wall) (mm)	-	30	268	107	458
Displacement (maximum) (mm)	-	43	289	110	423
Wall BM (at anchor 1) (KNm/m)	176	-91	-100	-93	-100
Wall BM (below anchor) (KNm/m)	176	91	144	127	174
Anchor 1 force (KN/m)	315 ¹	228	246	238	240
Anchor 2 force (KN/m)	315 ¹	224	357 ¹	256	288

¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load. Anchor force of 357 KN exceeds ULS load but is still less than anchor UTS of 394 KN.

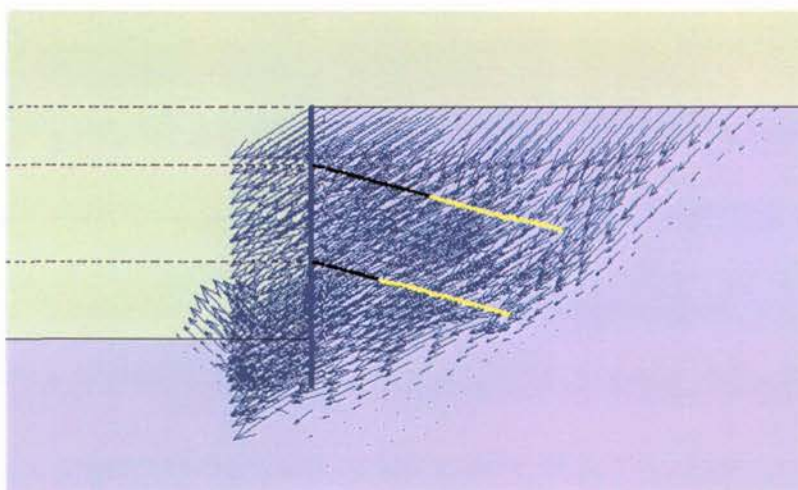


Figure 3.5.7 (a). Failure mechanism for Case 2b under gravity loading at FS = 1.42.

The wall achieved a maximum pseudo-static acceleration of 0.14 g. The factor of safety at the design acceleration of 0.1 g was found to be 1.13, exactly the same as

that calculated using the limiting equilibrium external wedge analysis. The failure mechanism (Figure 3.5.7 (b)) was very like the assumed external stability limiting equilibrium failure model (Figure 2.2.2 (c))

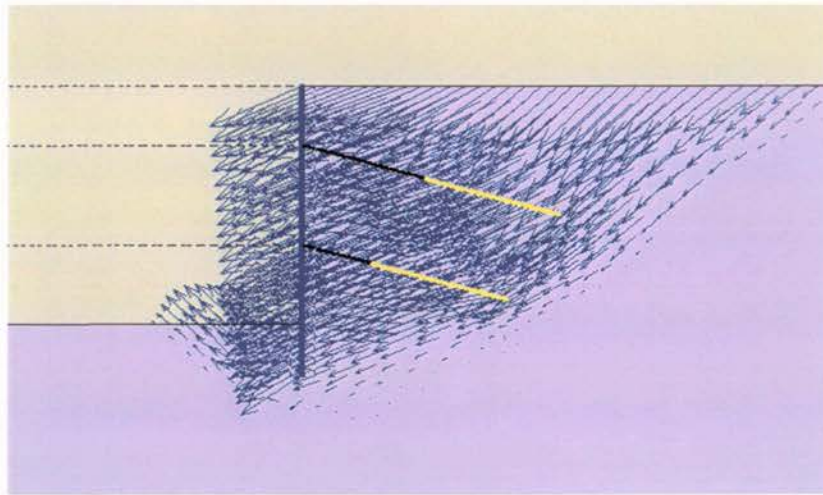


Figure 3.5.8 (a). Failure mechanism for Case 2b under pseudo-static loading of 0.1 g at FS = 1.13.

3.5.8 Performance of Case 2b under seismic loading

The performance of Case 2b under seismic loading was determined by applying only one scaled earthquake time-history record (Loma Prieta) to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g. Results from all of the analyses for the Case 2b are summarised in Figures 3.5.8 (a), (b), (c), and (d).

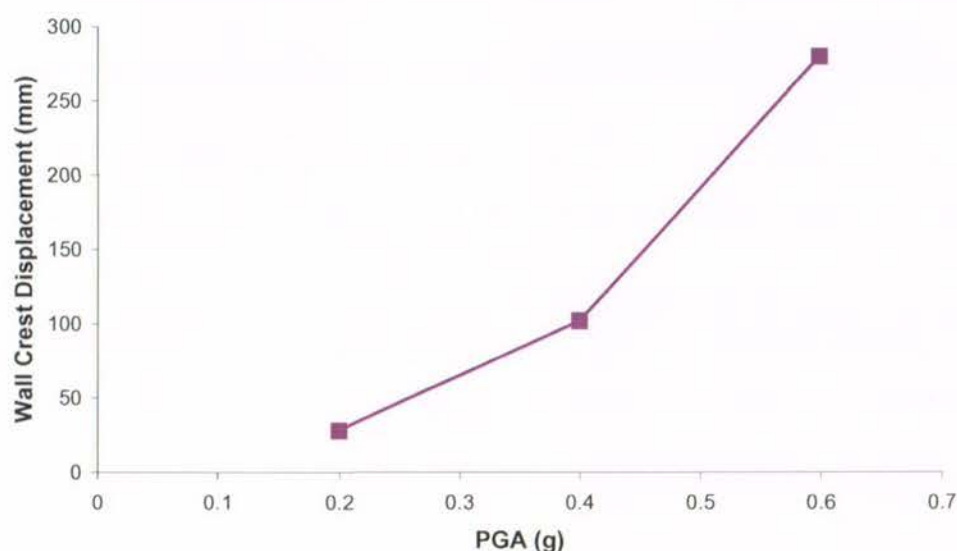


Figure 3.5.8 (a). Accumulated wall crest displacement after Loma Prieta earthquake for 0.1 g design.

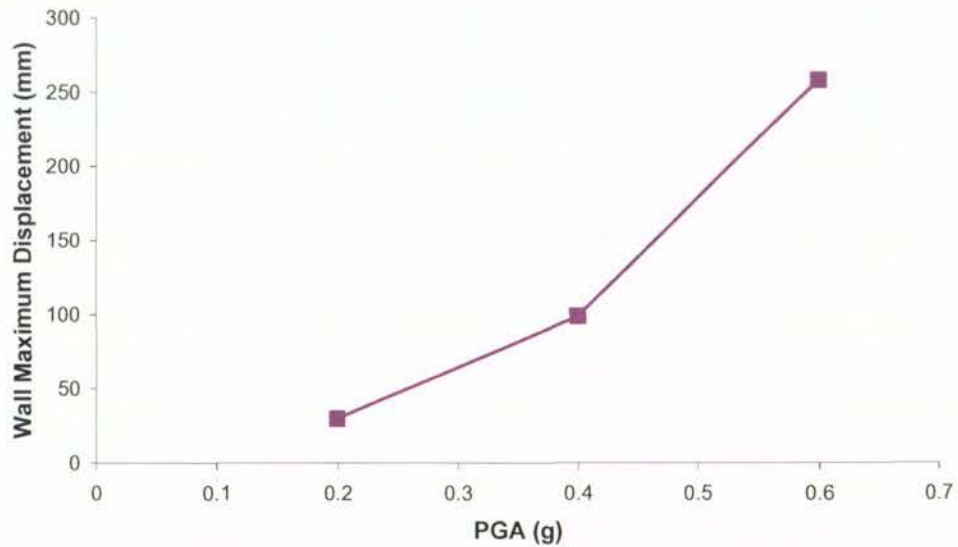


Figure 3.5.8 (b). Accumulated wall displacement below level of tie-back anchor after Loma Prieta earthquake for 0.1 g design.

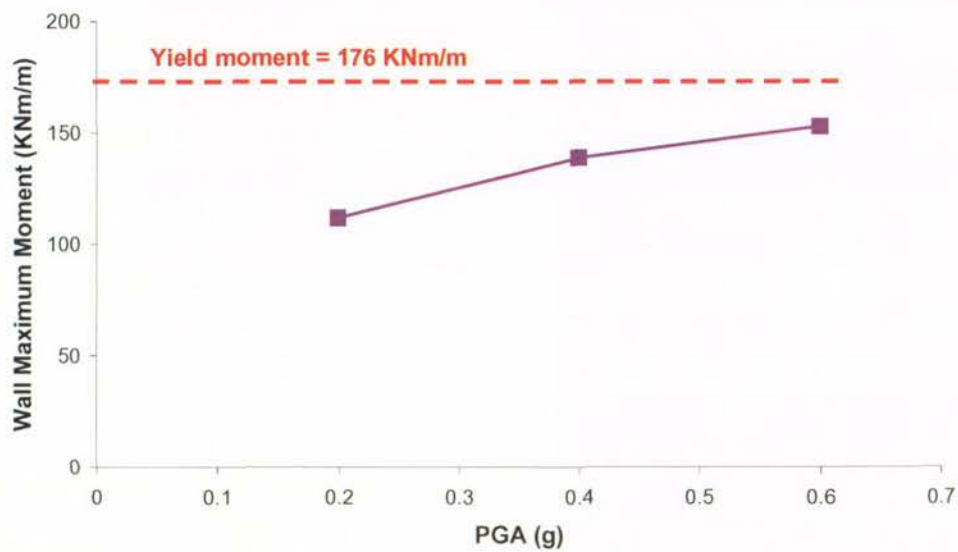


Figure 3.5.8 (c). Maximum wall bending moment after Loma Prieta earthquake for 0.1 g design.

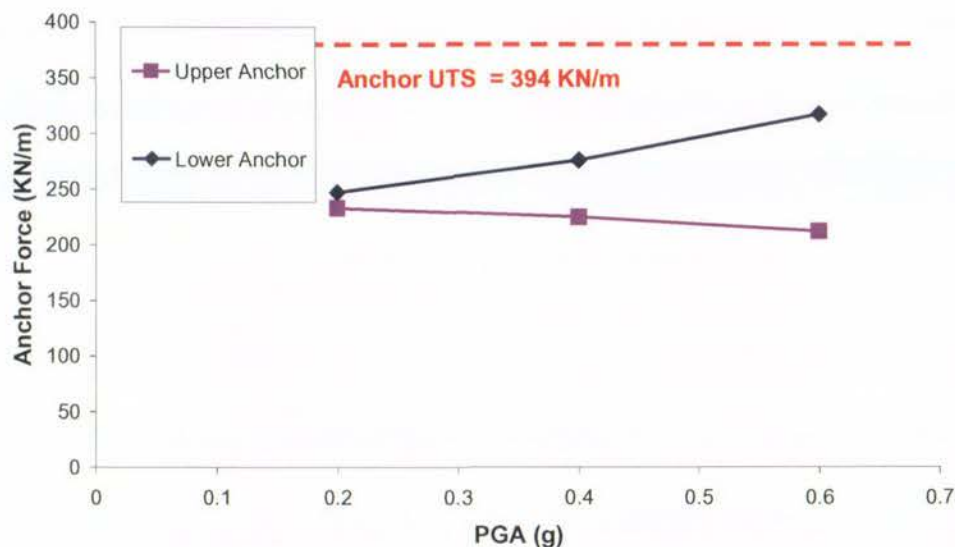


Figure 3.5.8 (d). Anchor force after Loma Prieta earthquake for 0.1 g design.

The performance of the wall can be seen to be a significant improvement over Case 2a, the gravity only design. A more detailed comparison among all of the designs for Case Study 2 are given in Section 3.5.12.

3.5.9 Case 2c: M-O based design to 0.2 g

Design of Case 2c followed the synthesized design procedure described in Section 2.3.6. Detailed calculations are given in Appendix B and summarised here:

Table 3.5.9 (a). Design values for case study Sand 2c: M-O design to 0.2 g.

Design Parameter	Value
Design acceleration	0.2 g
Apparent earth pressure, p	61 kN/m ²
Anchor 1 design load (horizontal)	276 kN/m
Anchor 2 design load (horizontal)	271 kN/m
Base reaction	46 kN/m
Cantilever bending moment (at anchor 1)	133 kNm/m
Maximum bending moment (at anchor 2)	133 kNm/m
ULS design bending moment, M^*	213 kNm/m

The wall structural elements were designed using these basic calculated design values as follows. For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections).

Table 3.5.9 (b). Design solutions for case study Sand 2c: M-O design to 0.2 g.

Design Solution	Value
Anchor 1 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	514 mm ² per anchor (5.1 strands per anchor)
Anchor 2 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	514 mm ² per anchor (5.1 strands per anchor)
Anchor 1 free length	7.7 m
Anchor 2 free length ¹	4.5 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	101% ² of 310UC96.8
Depth of embedment	2.9 m

¹ Calculated using FHWA procedure. In practice, a minimum free length of 5 m is recommended for strand anchors.

² Section properties scaled for purpose of the study.

The anchor free length for Case 2a was determined from the inclination of the M-O active wedge slip plane calculated for the **full** soil strength with no reduction to allow for uncertainty in soil strength parameters. The active wedge was assumed to extend from the toe of the embedded soldier piles unlike for the FHWA procedure which assumes that the active wedge extends from the base of the excavation.

Table 3.5.9 (c). Internal and external stability checks for case study Sand 2c: M-O based design 0.2 g

Stability Case ¹	FS
Internal stability	1.26
External stability	1.00

¹ Refer Figure 2.2.2 (c)

The factor of safety against external stability (FS = 1.0) is very low, but since external stability is considered the most desirable failure mechanism for earthquake overload it is considered acceptable. In practice, a minimum value of FS = 1.1 is recommended to allow for soil strength uncertainty.

3.5.10 Performance of Case 2c under gravity and pseudo-static loading

The performance of Case 2c was determined using the same PLAXIS modelling sequences as for the gravity design: First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor force were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability. Finally, a pseudo-static acceleration was applied and increased until the model became unstable. A summary of the main performance parameters is given in Table 3.5.10 (a).

The failure mechanism of the wall under gravity only loading was rupture of the lower anchor at just above $FS = 1.5$ (factor of safety on soil shear strength). For pseudo-static loading, the failure mechanism appears to be external stability with formation of a large active wedge of soil encompassing the wall and both anchors.

Table 3.5.10 (a) Performance of Case 2c: M-O based design to 0.2 g under static and pseudo-static loading.

	Design Basis ULS	Final excavation FS=1.0	Onset of instability FS = 1.49	Maximum acceleration 0.14 g
Displacement (top of wall) (mm)	-	21	616	133
Displacement (maximum) (mm)	-	32	641	123
Wall BM (at anchor 1) (KNm/m)	213	-117	-121	-110
Wall BM (below anchor) (KNm/m)	213	77	130	143
Anchor 1 force (KN/m)	380 ¹	285	280	288
Anchor 2 force (KN/m)	380 ¹	286	455 ¹	317

¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load. Anchor force of 455 KN exceeds ULS load but is still less than anchor UTS of 475 KN.

The maximum value of pseudo-static acceleration achieved by the model was 0.14 g, much less than the design value of 0.2 g. As for Case 1d and Case 1e, PLAXIS seems

unable to model large values of pseudo-static acceleration without generating deep seated shear failures through the entire soil deposit, as shown in Figure 3.5.10 (a).

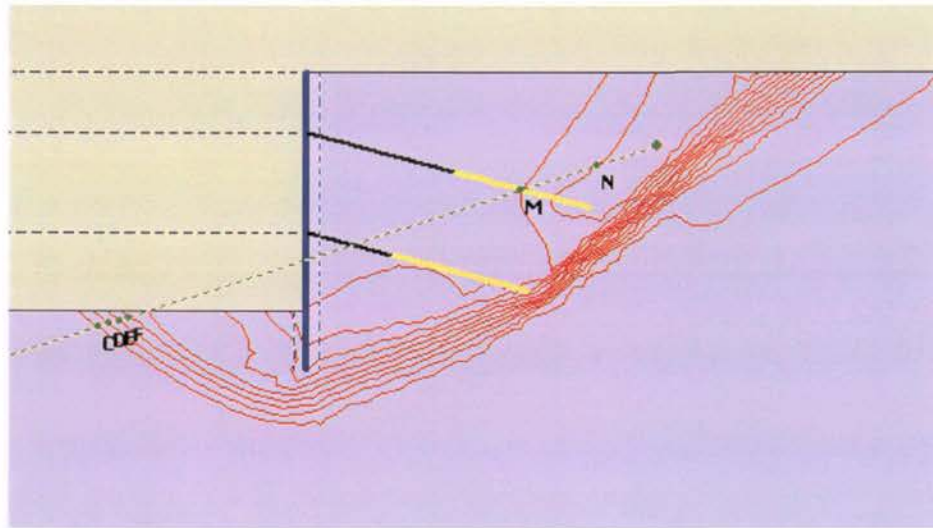


Figure 3.5.10 (a). Deep seated failure mechanism for Case 2c at horizontal acceleration of 0.14 g.

3.5.11 Performance of Case 2c under seismic loading

The performance of Case 2c under seismic loading was determined by applying only one scaled earthquake time-history record (Loma Prieta) to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g. Results from all of the analyses for the Case 2c are summarised in Figures 3.5.11 (a), (b), (c), and (d).

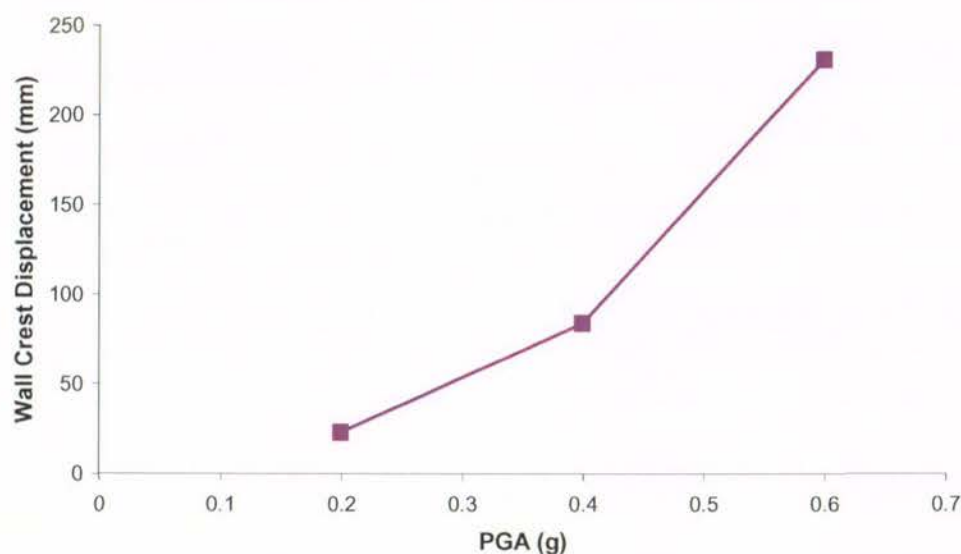


Figure 3.5.11 (a). Accumulated wall crest displacement after Loma Prieta earthquake for 0.2 g design.

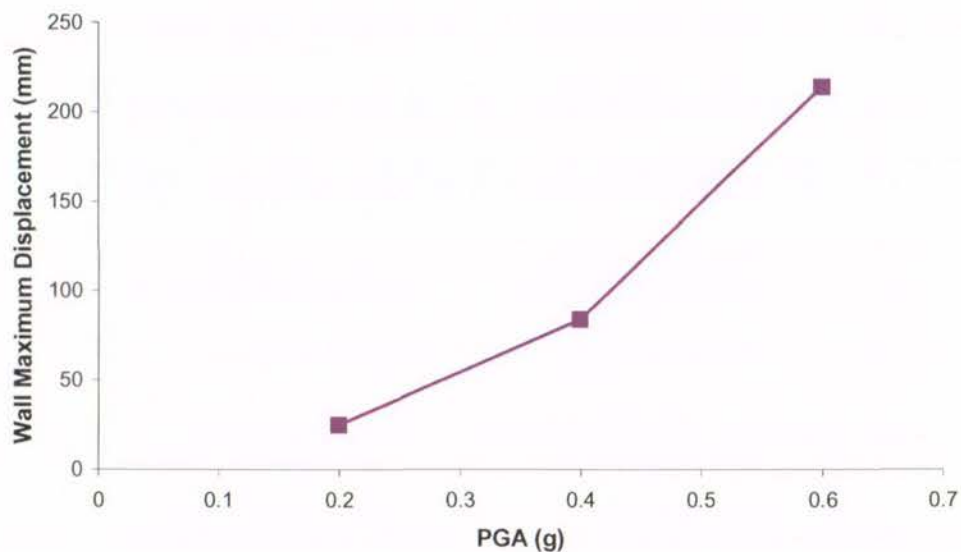


Figure 3.5.11 (b). Accumulated wall displacement below level of tie-back anchor after Loma Prieta earthquake for 0.2 g design.

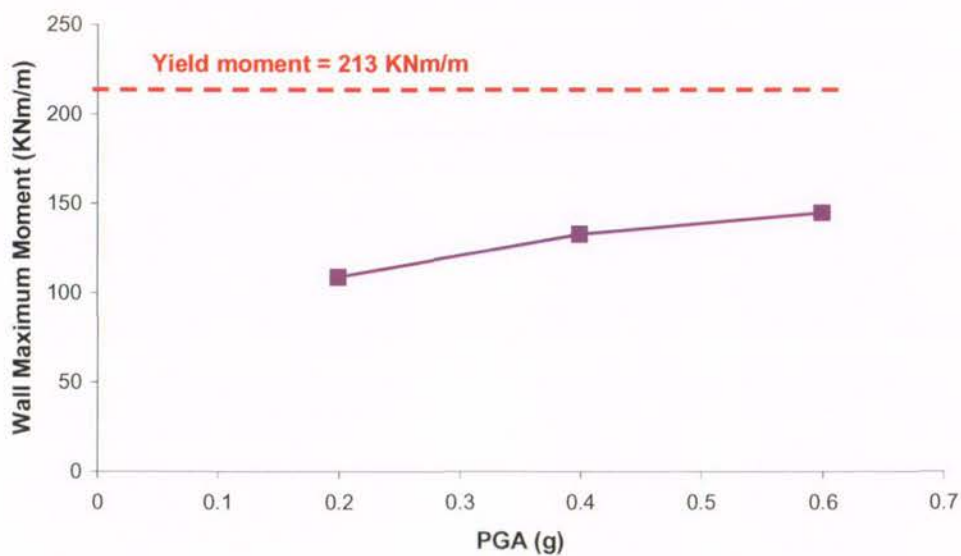


Figure 3.5.11 (c). Maximum wall bending moment after Loma Prieta earthquake for 0.2 g design.

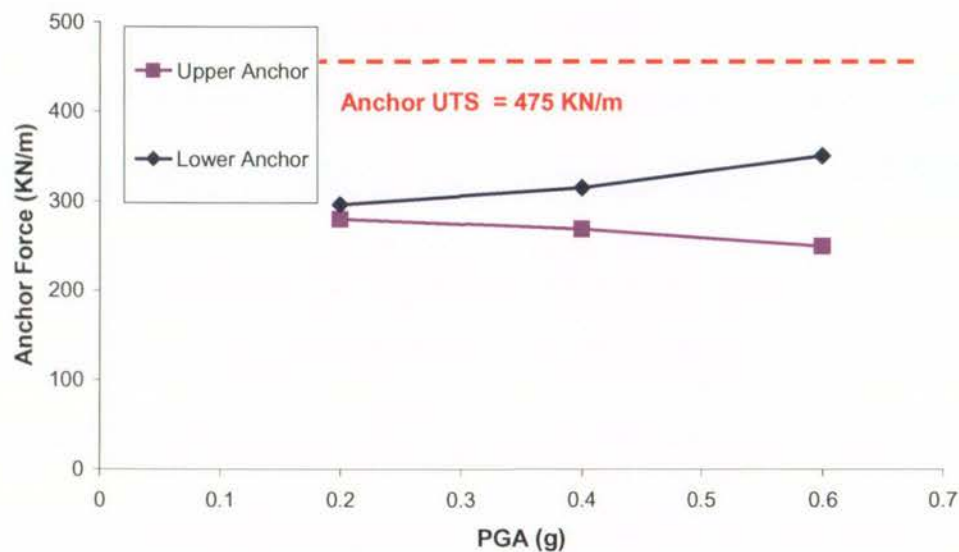


Figure 3.5.11 (d). Anchor force after Loma Prieta earthquake for 0.2 g design.

3.5.12 Comparison of design cases

For Case Study 2, a 12 m deep tied-back retaining wall in sand, three design variations were considered each with a different nominal design horizontal acceleration ranging from 0 g (gravity design) to 0.2 g. Each was designed using the synthesized design procedure based on the FHWA gravity design procedure. The resulting design values are compared in Table 3.5.12 (a) and the design solutions compared in Table 3.5.12 (b).

Wall design accelerations were not extended to beyond 0.2 g because of the conclusions reached in Case Study 1 which showed that optimum benefit cost ratios were obtained for 0.1 g and 0.2 g designs.

For the purposes of this research project, design solutions were perfectly optimised by taking crude proportions of whole steel sections or fractions of anchor strands. In real design cases section sizes can be optimised by changing spacing to some extent or simply rounding up to the next heaviest section.

From Table 3.5.12 (a), the increase in design apparent earth pressure increases steadily as the design acceleration is increased resulting in a steady increase in anchor loads and wall design moments.

Table 3.5.12 (a). Comparison of design values for case study Sand 2.

Design Parameter	Case 2a	Case 2b	Case 2c
Design acceleration	0 g	0.1 g	0.2 g
Apparent earth pressure, p (KN/m ²)	42	51	61
Anchor 1 design load (horizontal) (KN/m)	188	229	276
Anchor 2 design load (horizontal) (KN/m)	185	224	271
Base reaction (KN/m)	31	38	46
Cantilever bending moment (at anchor 1) (KNm/m)	91	110	133
Maximum bending moment (at anchor 2) (KNm/m)	91	110	133
ULS design bending moment, M* (KNm/m)	145	176	213

The design solutions for Case Study 2 are compared in Table 3.5.12 (b). The anchor and soldier pile sizes were optimised in an unrealistic way by taking proportions of whole member sizes. This optimisation was done to provide a more clear indication of trends for the purposes of the study.

For Case Study 2, the anchor free lengths were determined as follows: The angle of inclination of the M-O active wedge slip plane was calculated for the *nominal* soil strength. The anchor free lengths were extended to a line drawn from the toe of the embedded soldier piles to the ground surface at the calculated angle of inclination of the active slip plane. This approach is much simpler than that used in Case Study 1 where the soil friction angle was reduced by the factor of safety calculated for internal stability. The benefit of using longer free-lengths (flatter angle of inclination) was tested in Case Study 3 where the angle of inclination was flattened by five degrees.

Table 3.5.12 (b). Comparison of design solutions for case study Sand 2.

Design Solution	Case 2a	Case 2b	Case 2c
Design acceleration	0 g	0.1 g	0.2 g
Anchor 1: No. strands using super strand (100 mm ²) anchors at 2 m centres, inclined 15 degrees	3.5	4.2	5.1
Anchor 2:	3.4	4.2	5.1
Anchor 1 free length ¹	5.3 m	6.3 m	7.7 m
Anchor 1 free length ¹	2.9 m	3.6 m	4.5 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	94% ² of 250UC89.5	83% ² of 310UC96.8	101% ² of 310UC96.8
Depth of embedment	2.2 m	2.5 m	2.9 m

¹ Calculated using recommended procedure. In practice, a minimum free length of 5 m is recommended for strand anchors.

² Section properties scaled for purpose of the study.

The variation of anchor design, both free length and number of strands is plotted versus design acceleration in Figure 3.5.12 (a) and shows a more-or-less linear increase with increase in design acceleration. The variation of soldier pile design, both section weight and depth of embedment is shown in Figure 3.5.12 (b) and also shows a more-or-less linear increase with increase in design acceleration.

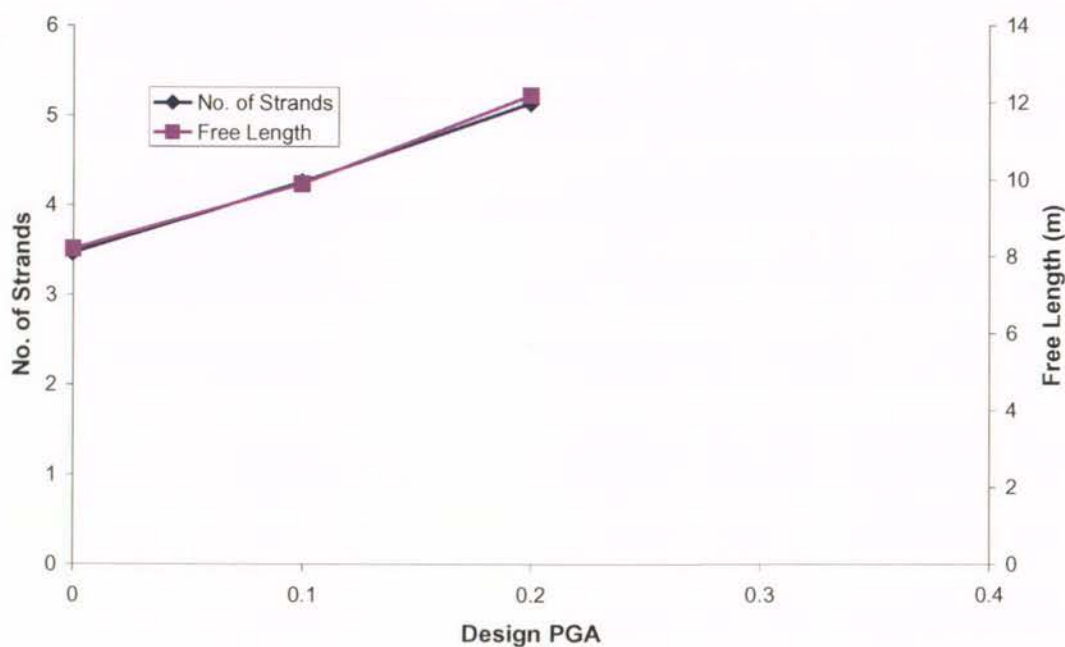


Figure 3.5.12 (a) Variation of anchor design parameters with design acceleration.

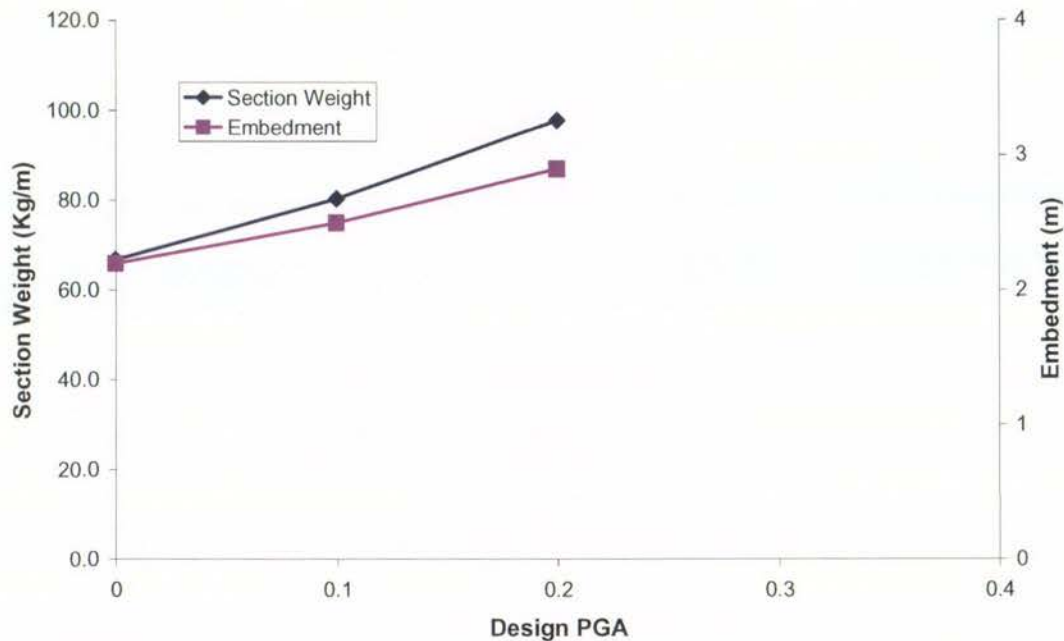


Figure 3.5.12 (b) Variation of soldier pile design parameters with design acceleration.

A crude cost index was derived for comparative purposes for both the soldier piles and the anchors. For the soldier piles the index was calculated by multiplying the section weight/m times the pile length (wall height plus embedment) and for the anchors by multiplying the number of strands times the anchor length (free length plus the bond length of 7 m). These indices were normalised by dividing by the values for the gravity only (0 g) designs.

These cost indices for soldier piles and anchors were kept separate because the comparative cost of anchor installation and soldier pile installation will depend on site specific factors.

A cost-performance comparison is made in Figure 3.5.12 (c) by plotting the cost indices with the average wall displacements. The wall displacements were for the single earthquake time history considered: Loma Prieta with separate curves shown for each of the three scales of peak ground acceleration considered (0.2 g, 0.4 g, 0.6 g). The wall crest displacement and “bulge” displacements were averaged.

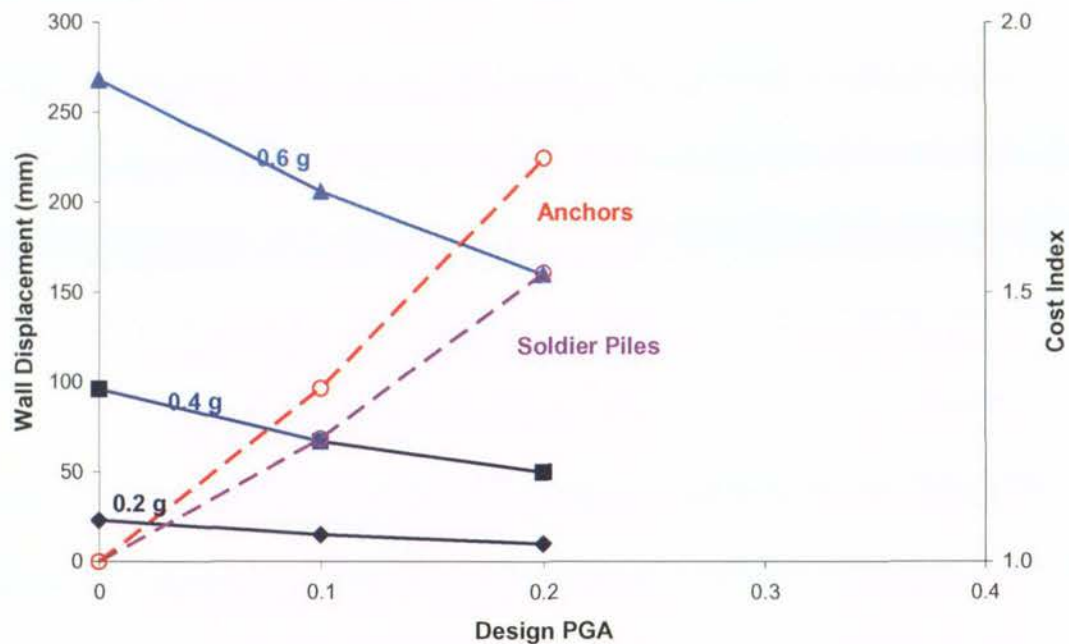


Figure 3.5.12 (c) Cost-Performance summary for Case Study 2.

3.5.13 Conclusions

The curves from Figure 3.5.12 (a) show that as the wall was designed to resist greater levels of quasi-static horizontal acceleration the wall performance in terms of permanent displacement improved significantly for all levels of earthquake shaking. However, the cost of the wall also increased substantially.

The benefit-cost ratio was about similar for both Case 2b (0.1 g design acceleration) and for Case 2c (0.2 g design acceleration): For Case 2b there was a cost increase of about 25 percent and a reduction in permanent displacement ranging from 35 percent for the 0.2 g earthquake to 23 percent for the 0.6 g earthquake. For Case 2c there was a further cost increase of about 28 percent and a reduction in permanent displacement ranging from 33 percent for the 0.2 g earthquake to 22 percent for the 0.6 g earthquake.

3.6 Case Study 3: Two Rows of Anchors in Sand with Extended Anchors

3.6.1 Case Study Description

This case is for the same 12 m deep excavation in sand of Case Study 2. All aspects of the wall were kept the same except for the length of the tie-back anchors which were set as follows:

1. For Case Study 2, the anchor free lengths were determined from the inclination of the M-O active wedge slip plane calculated for the **full** soil strength with no reduction to allow for uncertainty in soil strength parameters. The active wedge was assumed to extend from the toe of the embedded soldier piles unlike for the FHWA procedure which assumes that the active wedge extends from the base of the excavation.
2. For Case Study 3, the anchor free lengths were extended by decreasing the inclination from the horizontal of the active wedge slip plane by **five degrees**. The purpose of this increase was to increase the anchor free-lengths to ensure that they remain outside of the soil active wedge even during more extreme earthquake accelerations, possibly improving wall performance.

The anchor bond lengths were kept the same throughout this study at 7 m.

A cross-section through the PLAXIS model is shown in Figure 3.6.1 (a), which is essentially the same as for Case Study 2 apart from the anchor lengths. the soil properties were kept the same as for the previous case studies.

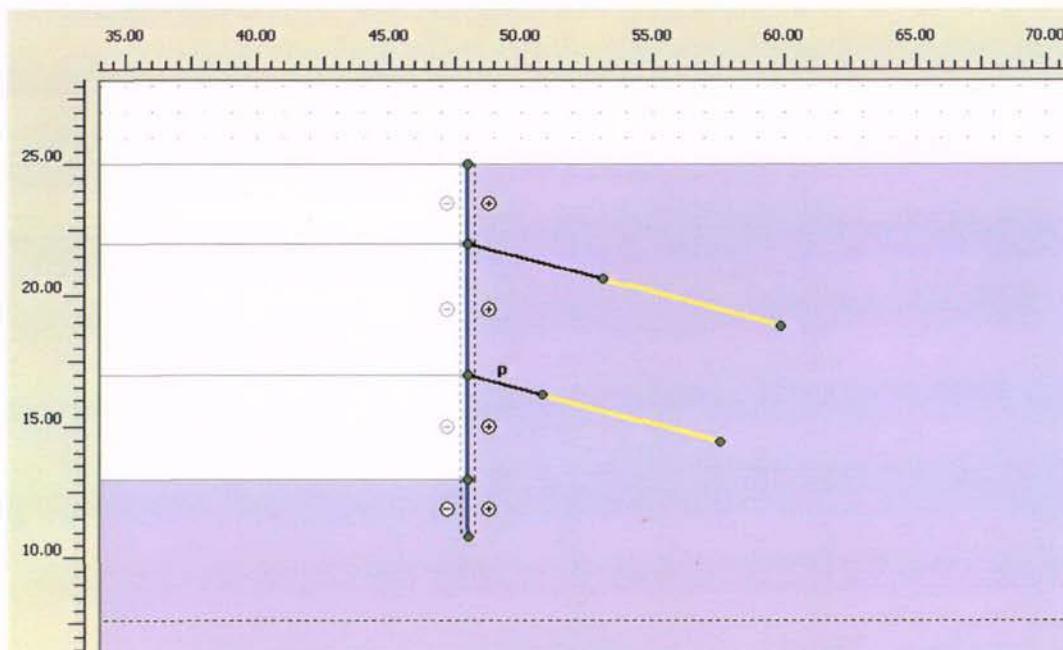


Figure 3.6.1 (a) PLAXIS model Sand 3: Gravity based design.

3.6.2 Case 3a: Gravity design

Gravity design followed the synthesized design procedure described in Section 2.3.6. Detailed calculations are given in Appendix C and summarised here:

Table 3.6.2 (a). Design values for case study Sand 3a: Gravity design.

Design Parameter	Value
Apparent earth pressure, p	42 kN/m ²
Anchor 1 design load (horizontal)	188 kN/m
Anchor 2 design load (horizontal)	185 kN/m
Base reaction	31 kN/m
Cantilever bending moment (at anchor 1)	91 kNm/m
Maximum bending moment (at anchor 2)	91 kNm/m
ULS design bending moment, M^*	145 kNm/m

The wall structural elements were designed using these basic calculated design values as follows. For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections).

Table 3.6.2 (b). Design solutions for case study Sand 3a: Gravity design.

Design Solution	Value
Anchor 1 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	350 mm ² per anchor (3.5 strands per anchor)
Anchor 2 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	344 mm ² per anchor (3.44 strands per anchor)
Anchor 1 free length	6.3
Anchor 2 free length ¹	3.5 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	94% ² of 250UC89.5
Depth of embedment	2.2 m

¹ Calculated using FHWA procedure. In practice, a minimum free length of 5 m is recommended for strand anchors.

² Section properties scaled for purpose of the study.

The anchor free length for Case 3a was determined from the inclination of the M-O active wedge slip plane calculated for the *full* soil strength *less five degrees*. The active wedge was assumed to extend from the toe of the embedded soldier piles unlike for the FHWA procedure which assumes that the active wedge extends from the base of the excavation.

The factor of safety against external stability was increased from 1.32 to 1.39 by increasing the anchor free lengths (Table 3.6.2 (c)).

Table 3.6.2 (c). Internal and external stability checks for case study Sand 3a: Gravity design.

Stability Case ¹	FS
Internal stability	1.32
External stability	1.39

¹ Refer Figure 2.2.2 (c)

3.6.3 Performance of Case 3a under gravity and pseudo-static loading

The performance of Case 3a designed using the synthesized procedure but considering only the gravity load case was determined by analysing the wall design using PLAXIS. First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor forces were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability. A summary of the main performance parameters is given in Table 3.6.3 (a), the bending moment distribution for the wall element is given in Figure 3.6.3 (a), and the collapse mechanism is illustrated in Figures 3.6.3 (b) and (c).

Table 3.6.3 (a) Performance of Case Study Sand 3a under gravity and pseudo-static loading

	Design Basis ULS	Final excavation FS=1.0	Stability Limit FS=1.35	Maximum pseudo-static acceleration 0.12 g
Displacement (top of wall) (mm)	-	42	180	125
Displacement (maximum) (mm)	-	52	189	115
Wall BM (at anchor 1) (KNm/m)	-145	-76	-77	-77
Wall BM (below anchors) (KNm/m)	145	76	118	117
Anchor 1 force (KN/m)	259 ¹	196	214	199
Anchor 1 force (KN/m)	255 ¹	192	318 ¹	237

¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load. Anchor load of 318 kN/m (100 percent of characteristic breaking load) is limiting the final stability of the wall.

The collapse mechanism of the wall appears to be internal failure with rupture of the lower anchor.

3.6.4 Evaluation of Case 3a under gravity loading

The factor of safety achieved in the PLAXIS analysis using “phi-c reduction” (lower bound) is greater than the value estimated using the limiting equilibrium, internal stability, wedge analysis and is considered satisfactory. Rupture of the lower anchor is an undesirable failure mechanism and results from increasing the anchor free length.

It would be desirable to increase the anchor size to prevent this failure mode.

3.6.5 Performance of Case 3a under seismic loading

The performance of Case 3a, gravity only design, under seismic loading was determined by applying only one scaled earthquake time-history record (Loma Prieta) to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g. This record was determined from Case Study 1 to be much more critical than the other earthquake time histories.

Wall performance is indicated primarily by permanent displacement (always outwards) remaining after each earthquake “event”. For the wall of Case Study 3a, the displacement was maximum at either the crest or near to the base below the second row of anchors where the wall typically tends to “bulge” outwards.

The bending moments in the wall elements were critical at either the top row of anchors or below the second row of anchors (the “bulge”) and these were also monitored together with the anchor forces. Results from all of the analyses for the Case 3a are summarised in Figures 3.6.5 (a), (b), (c), and (d).

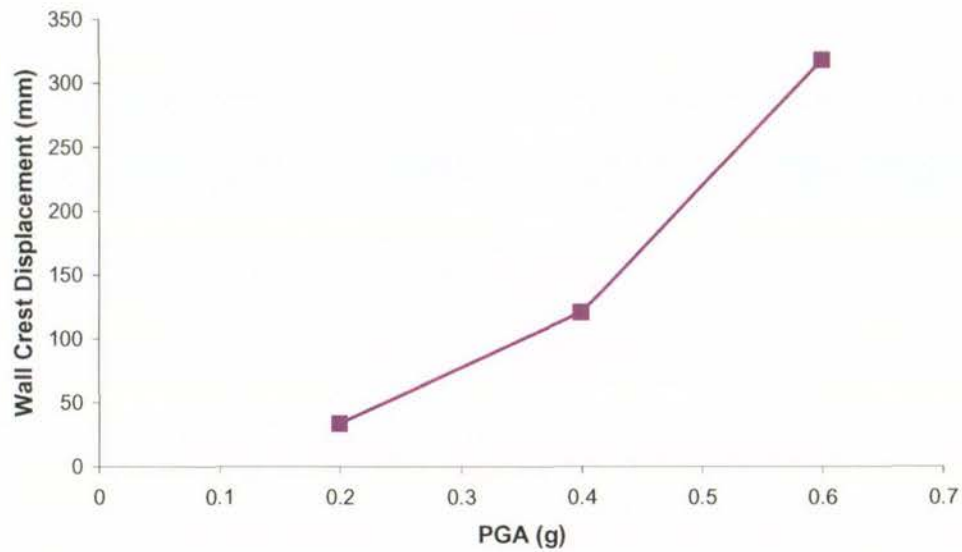


Figure 3.6.5 (a). Accumulated wall crest displacement after Loma Prieta earthquake for gravity only design.

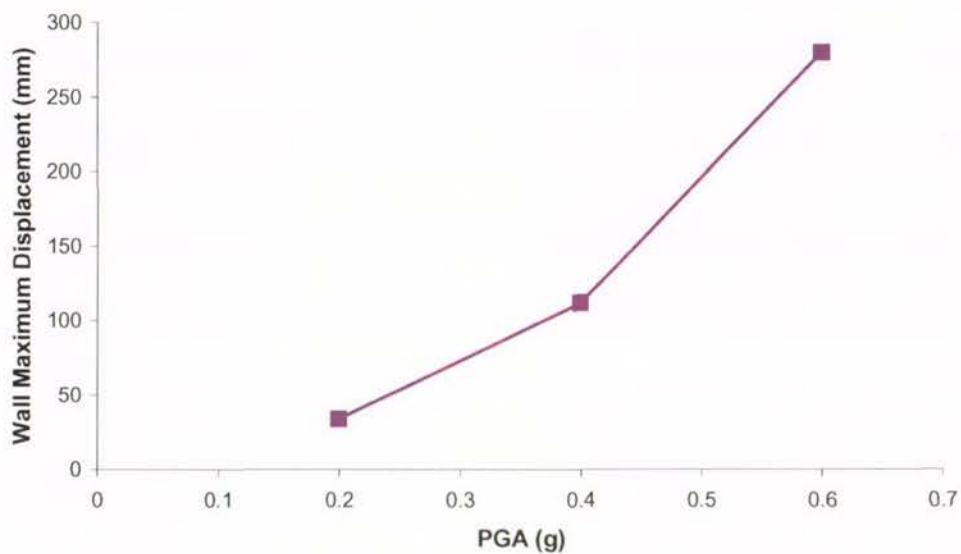


Figure 3.6.5 (b). Accumulated wall displacement below level of tie-back anchor after Loma Prieta earthquake for gravity only design.

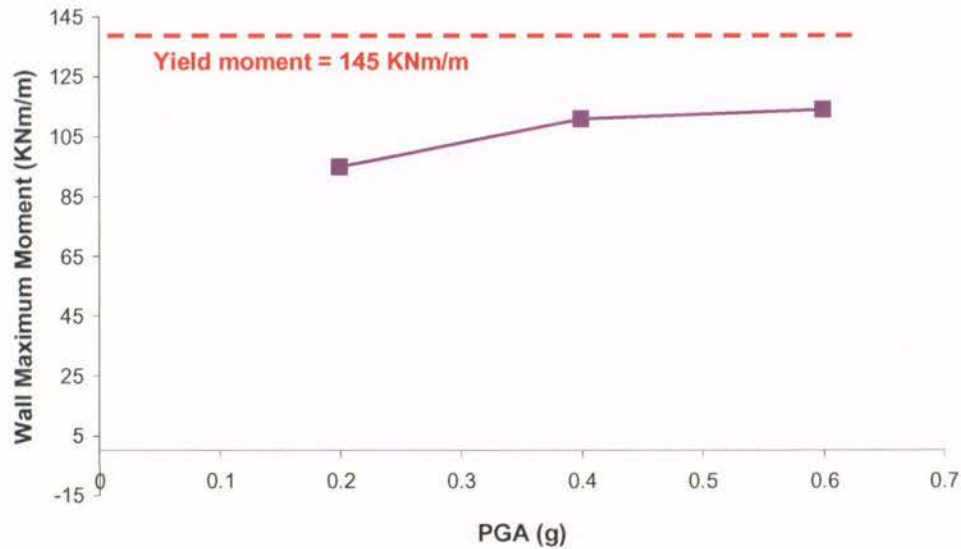


Figure 3.6.5 (c). Maximum wall bending moment after Loma Prieta earthquake for gravity only design.

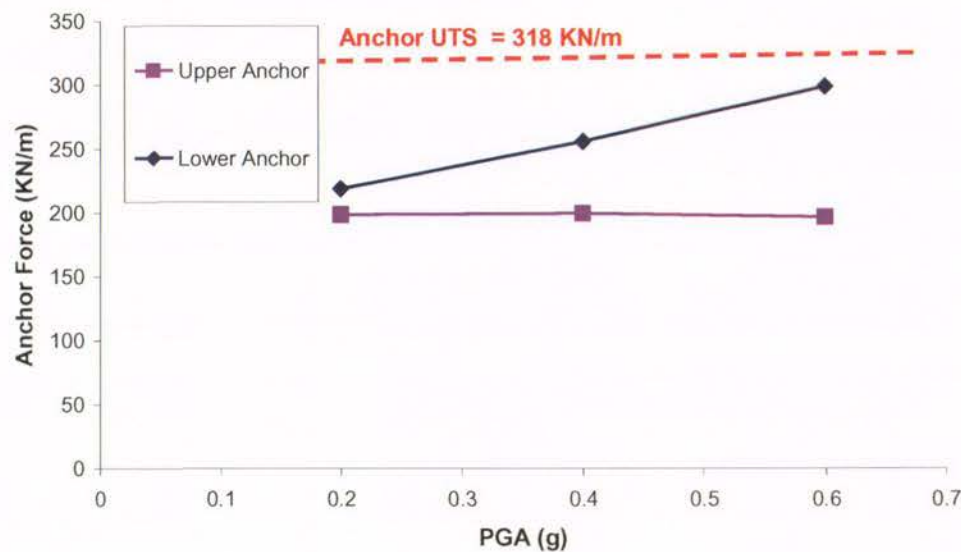


Figure 3.6.5 (d). Anchor force after Loma Prieta earthquake for gravity only design.

Generally, the wall deformations were significantly improved over Case Study 2 with the shorter anchor free lengths.

A more detailed comparison among all of the Case 3 design cases is given in Section 3.6.12.

3.6.6 Case 3b: M-O based design to 0.1 g

Design of Case 3b followed the synthesized design procedure described in Section 2.3.6. Detailed calculations are given in Appendix C and summarised here:

Table 3.6.6 (a). Design values for case study Sand 3b: M-O design to 0.1 g.

Design Parameter	Value
Design acceleration	0.1 g
Apparent earth pressure, p	51 kN/m ²
Anchor 1 design load (horizontal)	229 kN/m
Anchor 2 design load (horizontal)	224 kN/m
Base reaction	38 kN/m
Cantilever bending moment (at anchor 1)	110 kNm/m
Maximum bending moment (at anchor 2)	110 kNm/m
ULS design bending moment, M^*	176 kNm/m

The wall structural elements were designed using these basic calculated design values as follows. For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections).

Table 3.6.6 (b). Design solutions for case study Sand 3b: M-O design to 0.1 g.

Design Solution	Value
Anchor 1 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	426 mm ² per anchor (4.2 strands per anchor)
Anchor 2 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	426 mm ² per anchor (4.2 strands per anchor)
Anchor 1 free length	7.4 m
Anchor 2 free length ¹	4.2 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	83% ² of 310UC96.8
Depth of embedment	2.5 m

¹ Calculated using FHWA procedure. In practice, a minimum free length of 5 m is recommended for strand anchors.

² Section properties scaled for purpose of the study.

The anchor free length for Case 3a was determined from the inclination of the M-O active wedge slip plane calculated for the *full* soil strength *less five degrees*. The active wedge was assumed to extend from the toe of the embedded soldier piles unlike

for the FHWA procedure which assumes that the active wedge extends from the base of the excavation.

The factor of safety against external stability was increased from 1.13 to 1.18 by increasing the anchor free lengths (Table 3.6.6 (c)).

Table 3.6.6 (c). Internal and external stability checks for case study Sand 3b: M-O based design 0.1 g

Stability Case ¹	FS
Internal stability	1.31
External stability	1.18

¹ Refer Figure 2.2.2 (c)

3.6.7 Performance of Case 3b under gravity and pseudo-static loading

The performance of Case 3b was determined using the same PLAXIS modelling sequences as for the gravity design: First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor force were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability. Finally, a pseudo-static acceleration was applied and increased until the model became unstable. A summary of the main performance parameters is given in Table 3.6.7 (a).

The failure mechanism of the wall under both gravity only loading and pseudo-static loading appears to be external stability with formation of a large active wedge of soil encompassing the wall and both anchors.

Table 3.6.7 (a) Performance of Case 3b: M-O based design to 0.1 g under static and pseudo-static loading.

	Design Basis ULS	Final excavation FS=1.0	Onset of instability FS = 1.45	Design acceleration 0.1 g	Maximum acceleration 0.14 g
Displacement (top of wall) (mm)	-	28	148	66	175
Displacement (maximum) (mm)	-	38	192	61	162
Wall BM (at anchor 1) (KNm/m)	176	-96	-107	-95	-95
Wall BM (below anchor) (KNm/m)	176	78	146	117	145
Anchor 1 force (KN/m)	315 ¹	237	264	240	242
Anchor 2 force (KN/m)	315 ¹	237	377 ¹	260	282

¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load. Anchor force of 377 KN exceeds ULS load but is still less than anchor UTS of 394 KN.

The wall achieved a maximum pseudo-static acceleration of 0.15 g. The factor of safety at the design acceleration of 0.1 g was found to be 1.18, exactly the same as that calculated using the limiting equilibrium external wedge analysis. The failure mechanism was very like the assumed external stability limiting equilibrium failure model (Figure 2.2.2 (c)).

3.6.8 Performance of Case 3b under seismic loading

The performance of Case 3b under seismic loading was determined by applying only one scaled earthquake time-history record (Loma Prieta) to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g. Results from all of the analyses for the Case 2b are summarised in Figures 3.6.8 (a), (b), (c), and (d).

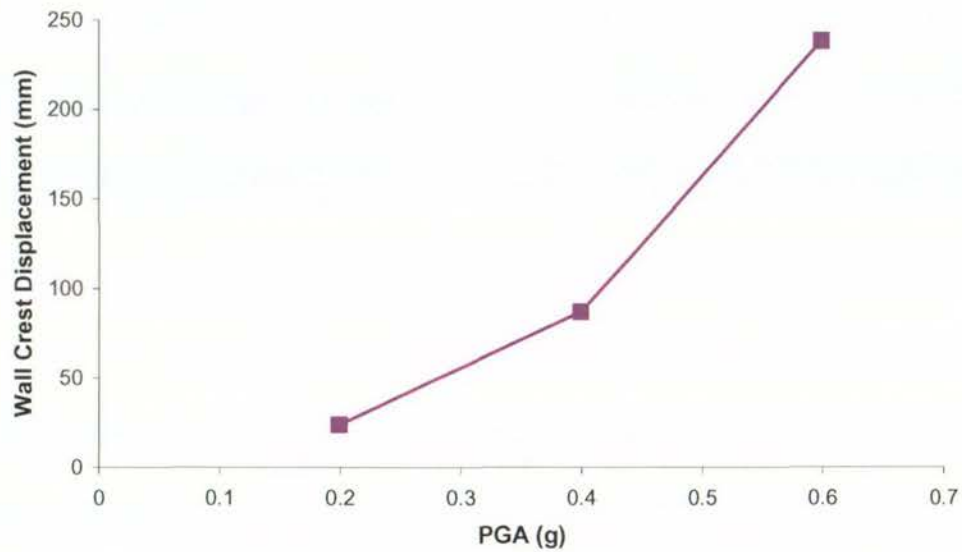


Figure 3.6.8 (a). Accumulated wall crest displacement after Loma Prieta earthquake for 0.1 g design.

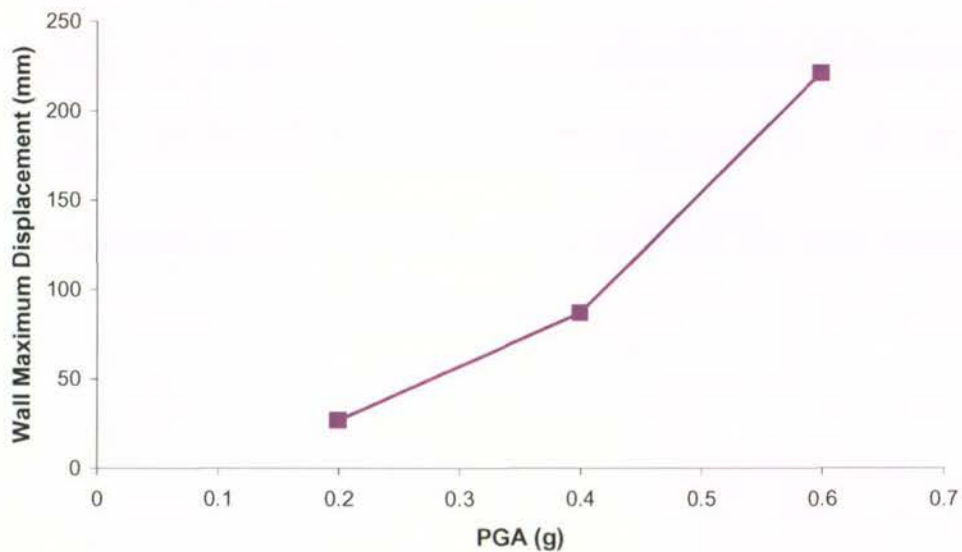


Figure 3.6.8 (b). Accumulated wall displacement below level of tie-back anchor after Loma Prieta earthquake for 0.1 g design.

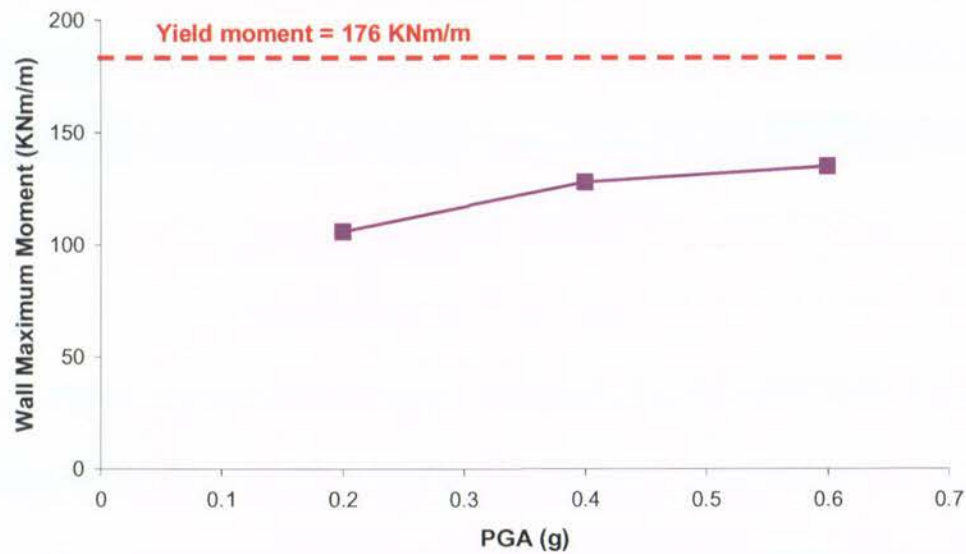


Figure 3.6.8 (c). Maximum wall bending moment after Loma Prieta earthquake for 0.1 g design.

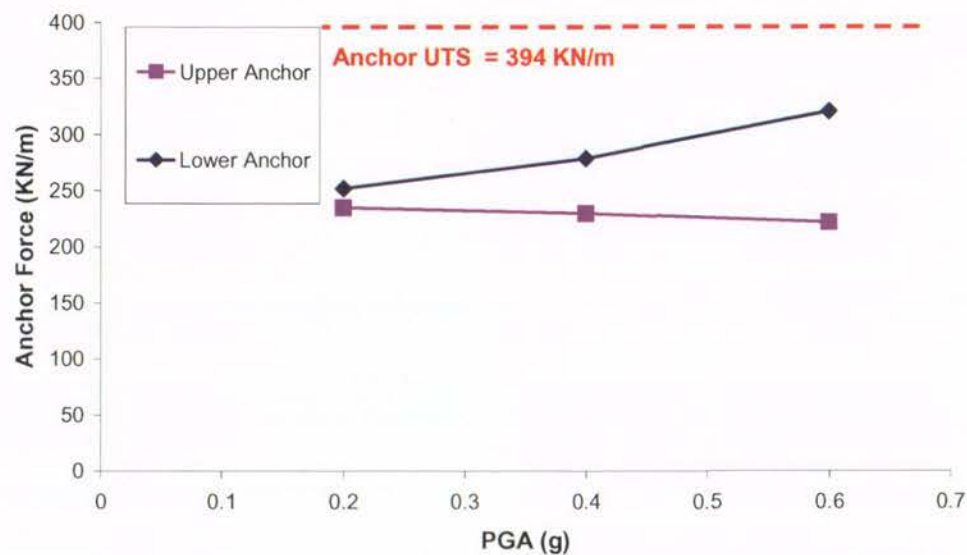


Figure 3.6.8 (d). Anchor force after Loma Prieta earthquake for 0.1 g design.

Generally, the wall deformations were significantly improved over Case Study 2 with the shorter anchor free lengths.

A more detailed comparison among all of the Case 3 design cases is given in Section 3.6.12.

3.6.9 Case 3c: M-O based design to 0.2 g

Design of Case 3c followed the synthesized design procedure described in Section 2.3.6. Detailed calculations are given in Appendix B and summarised here:

Table 3.6.9 (a). Design values for case study Sand 3c: M-O design to 0.2 g.

Design Parameter	Value
Design acceleration	0.2 g
Apparent earth pressure, p	61 kN/m ²
Anchor 1 design load (horizontal)	276 kN/m
Anchor 2 design load (horizontal)	271 kN/m
Base reaction	46 kN/m
Cantilever bending moment (at anchor 1)	133 kNm/m
Maximum bending moment (at anchor 2)	133 kNm/m
ULS design bending moment, M^*	213 kNm/m

The wall structural elements were designed using these basic calculated design values as follows. For the purposes of this research project, design solutions were perfectly optimised, whereas for everyday design it would be necessary to select from standard products (e.g. standard anchor configurations, stock steel sections).

Table 3.6.9 (b). Design solutions for case study Sand 3c: M-O design to 0.2 g.

Design Solution	Value
Anchor 1 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	514 mm ² per anchor (5.1 strands per anchor)
Anchor 2 cross-section (using super strand anchors at 2 m centres, inclined 15 degrees))	514 mm ² per anchor (5.1 strands per anchor)
Anchor 1 free length	8.7 m
Anchor 2 free length ¹	5.1 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	101% ² of 310UC96.8
Depth of embedment	2.9 m

¹ Calculated using FHWA procedure. In practice, a minimum free length of 5 m is recommended for strand anchors.

² Section properties scaled for purpose of the study.

The anchor free length for Case 3c was determined from the inclination of the M-O active wedge slip plane calculated for the *full* soil strength *less five degrees*. The active wedge was assumed to extend from the toe of the embedded soldier piles unlike for the FHWA procedure which assumes that the active wedge extends from the base of the excavation.

The factor of safety against external stability was increased from 1.00 to 1.04 by increasing the anchor free lengths (Table 3.6.2 (c)).

Table 3.6.9 (c). Internal and external stability checks for case study Sand 3c: M-O based design 0.2 g

Stability Case ¹	FS
Internal stability	1.26
External stability	1.04

¹ Refer Figure 2.2.2 (c)

3.6.10 Performance of Case 3c under gravity and pseudo-static loading

The performance of Case 2c was determined using the same PLAXIS modelling sequences as for the gravity design: First, the construction sequence was modelled and the wall deformations, wall element bending moments, and anchor force were analysed. Then, the soil strength was progressively reduced (using PLAXIS “phi-c reduction” procedure) to determine the variances of structural performance with reduction of soil strength and to determine the factor of safety against instability. Finally, a pseudo-static acceleration was applied and increased until the model became unstable. A summary of the main performance parameters is given in Table 3.6.10 (a).

The failure mechanism of the wall under gravity only loading was by the external stability mechanism. For pseudo-static loading, the failure mechanism appears to be external stability with formation of a large active wedge of soil encompassing the wall and both anchors.

The maximum value of pseudo-static acceleration achieved by the model was 0.18 g, less than the design value of 0.2 g. As for Case 2c, PLAXIS seems unable to model large values of pseudo-static acceleration without generating deep seated shear failures through the entire soil deposit, as shown in Figure 3.5.10 (a).

Table 3.6.10 (a) Performance of Case 3c: M-O based design to 0.2 g under static and pseudo-static loading.

	Design Basis ULS	Final excavation FS=1.0	Onset of instability FS = 1.54	Maximum acceleration 0.18 g
Displacement (top of wall) (mm)	-	18	122	764
Displacement (maximum) (mm)	-	29	176	676
Wall BM (at anchor 1) (KNm/m)	213	-120	-120	-126
Wall BM (below anchor) (KNm/m)	213	74	173	173
Anchor 1 force (KN/m)	380 ¹	289	301	300
Anchor 2 force (KN/m)	380 ¹	280	475 ¹	344

¹ ULS capacity of anchor may be assumed to be the anchor test load, normally set at 80 percent of tendon characteristic breaking load

3.6.11 Performance of Case 3c under seismic loading

The performance of Case 2c under seismic loading was determined by applying only one scaled earthquake time-history record (Loma Prieta) to the PLAXIS model over a range of increasing PGA's: 0.2 g, 0.4 g, and 0.6 g. Results from all of the analyses for the Case 2c are summarised in Figures 3.5.11 (a), (b), (c), and (d).

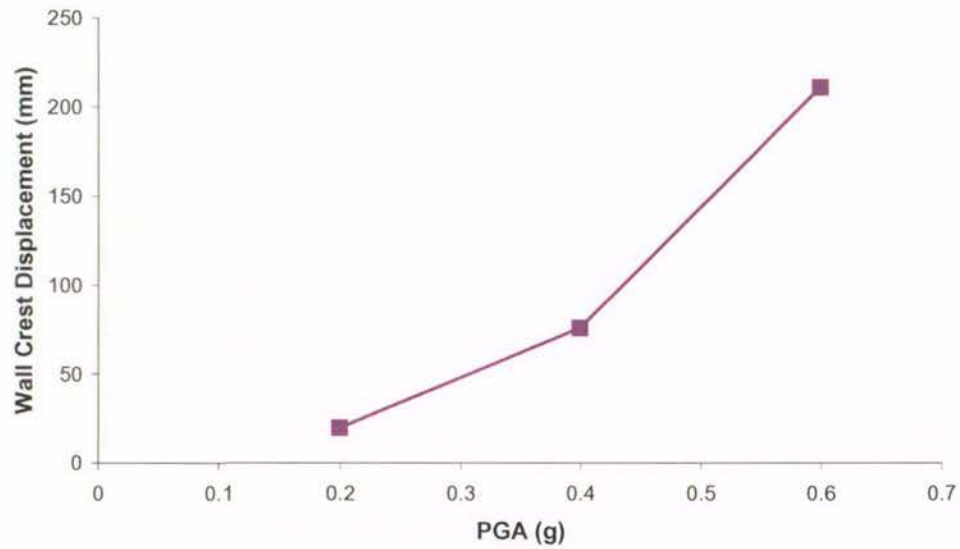


Figure 3.6.11 (a). Accumulated wall crest displacement after Loma Prieta earthquake for 0.2 g design.

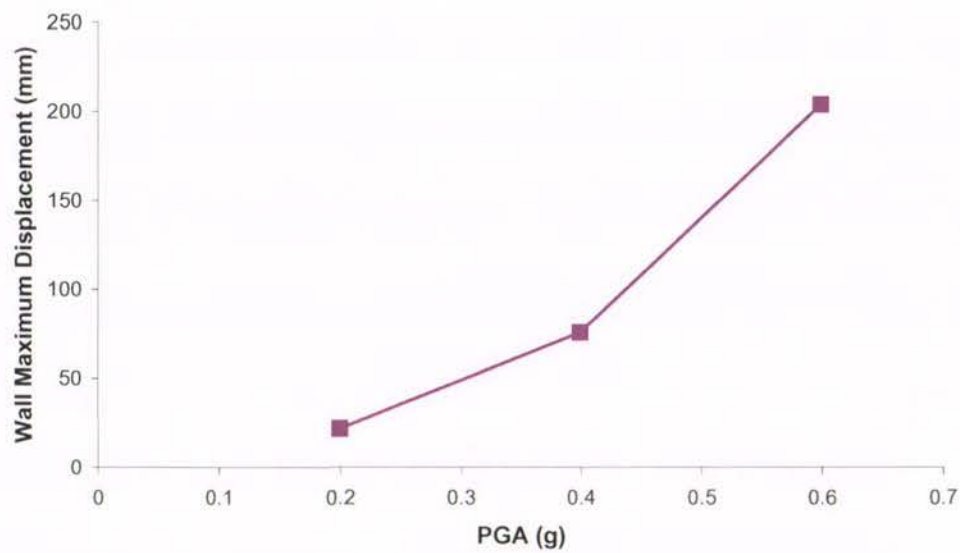


Figure 3.6.11 (b). Accumulated wall displacement below level of tie-back anchor after Loma Prieta earthquake for 0.2 g design.

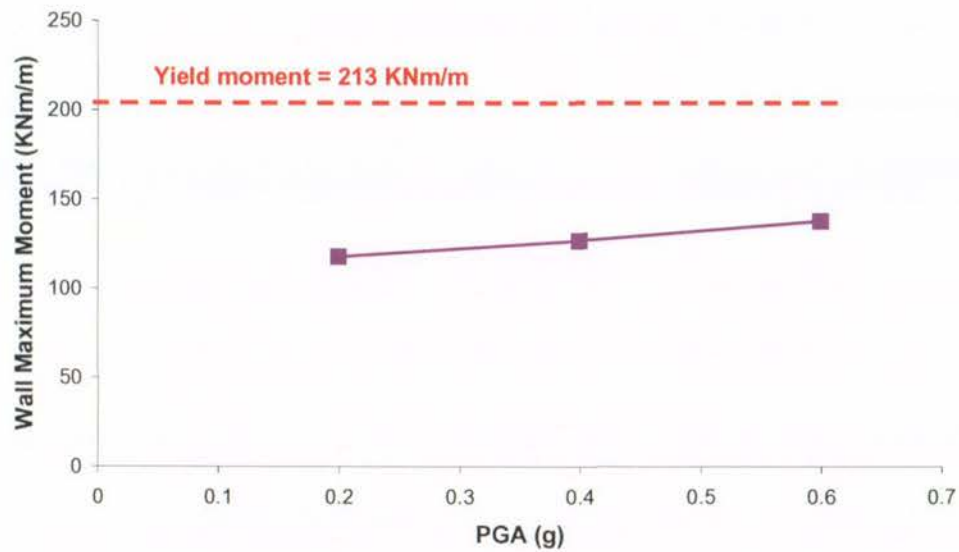


Figure 3.6.11 (c). Maximum wall bending moment after Loma Prieta earthquake for 0.2 g design.

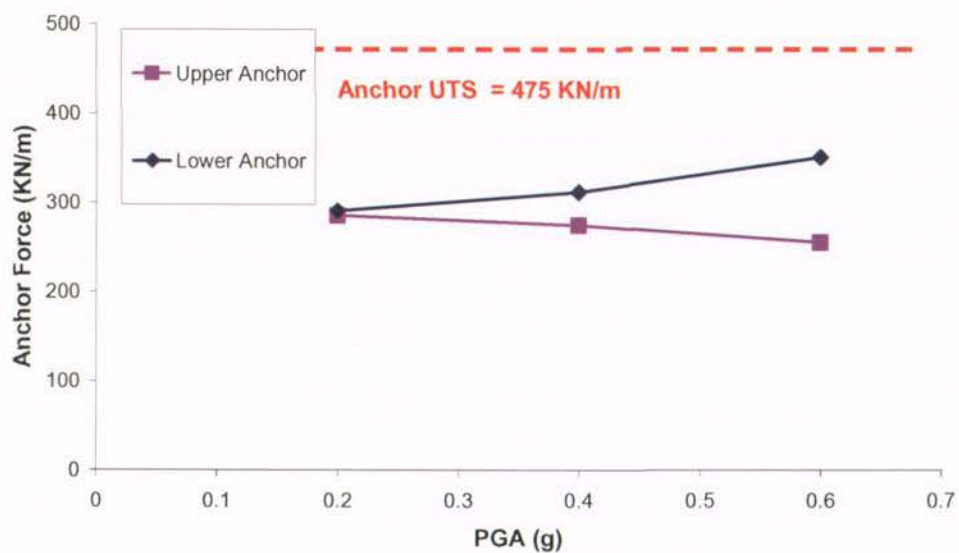


Figure 3.6.11 (d). Anchor force after Loma Prieta earthquake for 0.2 g design.

3.6.12 Comparison of design cases

For Case Study 3, the designs of case Study 2 were altered by increasing the free lengths of the anchors as explained in Section 3.6.1. All other aspects of the wall designs were kept the same.

The key design values are the same as for Case Study 2 as listed in Table 3.5.12 (a).

The design solutions for Case Study 3 are the same as for Case Study except for the variances in anchor free lengths as highlighted in Table 3.6.12 (a).

Table 3.6.12 (b). Comparison of design solutions for case study Sand 2 and Sand 3.

Design Solution	Case 2a Case 3a	Case 2b Case 3b	Case 2c Case 3c
Design acceleration	0 g	0.1 g	0.2 g
Anchor 1: No. strands using super strand (100 mm ²) anchors at 2 m centres, inclined 15 degrees	3.5	4.2	5.1
Anchor 2:	3.4	4.2	5.1
Anchor 1 free length ¹	5.3 m 6.3 m	6.3 m 7.4 m	7.7 m 8.7 m
Anchor 1 free length ¹	2.9 m 3.5 m	3.6 m 4.2 m	4.5 m 5.1 m
Soldier piles (UC sections set in 450 mm diameter concrete @ 2 m crs)	94% ² of 250UC89.5	83% ² of 310UC96.8	101% ² of 310UC96.8
Depth of embedment	2.2 m	2.5 m	2.9 m

¹ Calculated using recommended procedure. In practice, a minimum free length of 5 m is recommended for strand anchors.

² Section properties scaled for purpose of the study.

A comparison between the displacement performance of the Case Study 2 and Case Study 3 anchors is given in Figure 3.6.12 (a). The effect of increasing the anchor free length was to reduce wall displacements by between 10 percent and 30 percent with an average of 16 percent for all cases.

A crude cost index was derived for comparative purposes for both the soldier piles and the anchors. For the soldier piles the index was calculated by multiplying the section weight/m times the pile length (wall height plus embedment) and for the anchors by multiplying the number of strands times the anchor length (free length plus the bond length of 7 m). These indices were normalised by dividing by the values for the gravity only (0 g) design of Case Study 2 (shortest anchor free lengths). these cost indices are plotted for all Case Study 2 and case Study 3 designs in Figure 3.6.12 (b).

There was no cost variance between Case Study 2 and Case Study 3 for soldier piles, but there was about a 10 percent increase in anchor costs for Case Study 3.

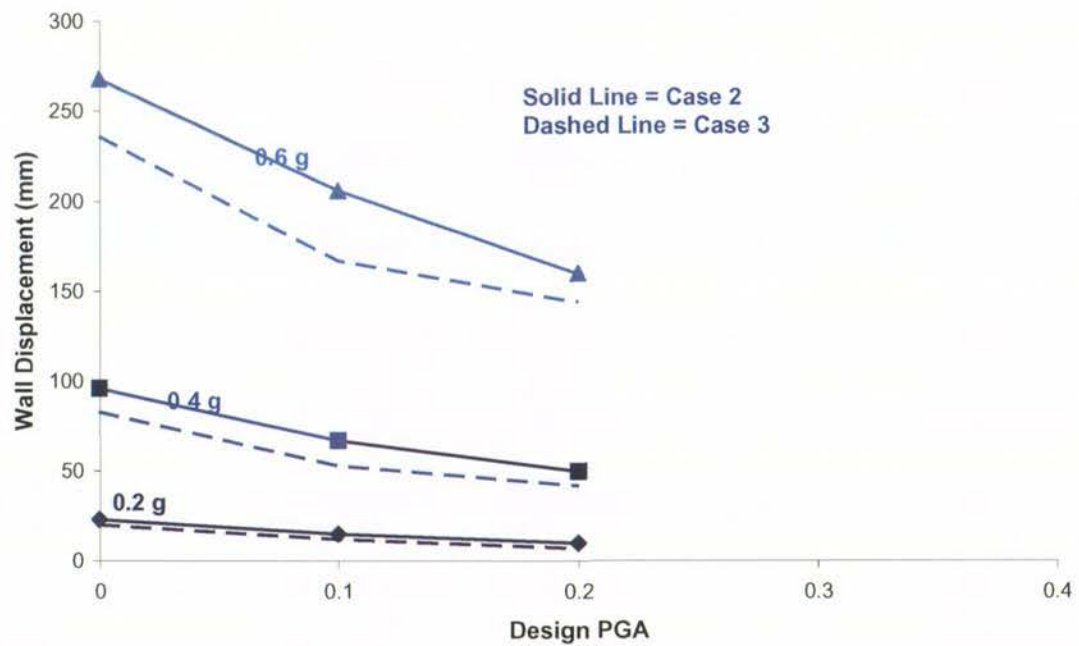


Figure 3.5.12 (a) Performance comparison summary for Case Studies 2 and 3.

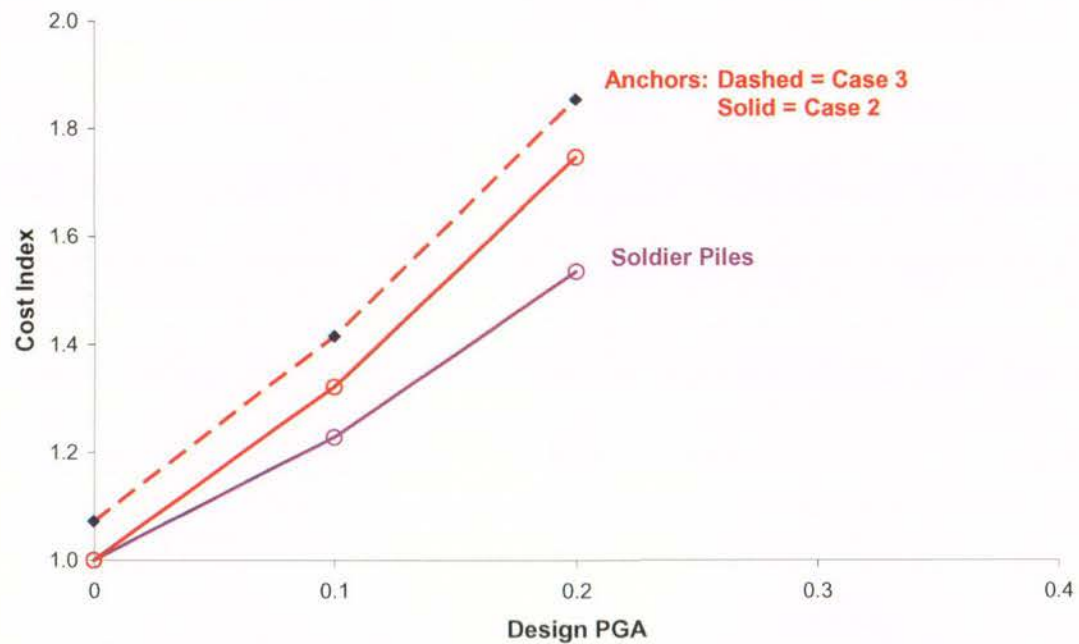


Figure 3.5.12 (b). Figure 3.5.12 (a) Cost comparison summary for Case Studies 2 and 3.

3.6.13 Conclusions

A proposal to increase the anchor free lengths according to the following procedure was tested:

1. For Case Study 2, the anchor free lengths were determined from the inclination of the M-O active wedge slip plane calculated for the *full* soil strength with no reduction to allow for uncertainty in soil strength parameters. The active wedge was assumed to extend from the toe of the embedded soldier piles unlike for the FHWA procedure which assumes that the active wedge extends from the base of the excavation.
2. For Case Study 3, the anchor free lengths were extended by decreasing the inclination from the horizontal of the active wedge slip plane by *five degrees*. The purpose of this increase was to increase the anchor free-lengths to ensure that they remain outside of the soil active wedge even during more extreme earthquake accelerations, possibly improving wall performance.

The increase in free length resulted in an improvement in wall displacement response averaging 16 percent for a small increase of 10 percent in anchor cost. Anchor forces during and after earthquake shaking remained much the same and well below anchor ultimate tensile strengths.

Increasing the free length of the lower anchor resulted in anchor tensile failure becoming the critical failure mechanism for gravity loading. However, the factor of safety against failure under gravity loading was improved.

4 Design Guidelines

4.1 Overview

One of the main objectives for this study was to develop workable design guidelines for use by practicing engineers. As such they need to be as simple as possible without overlooking or over-simplifying key aspects of performance or safety of tied-back walls.

The approach adopted has been to build on an existing design procedure that is well proven and in wide use: The apparent earth pressure diagrams of Terzaghi and Peck [1967] subsequently updated, revised, and re-published in the FHWA Geotechnical Engineering Circular No. 4. [Sabatini. et. al., 1999].

This study has investigated several aspects of the performance of tied-back walls designed using the FHWA including the factor of safety against instability under gravity loads as well as the seismic performance. This study has shown that the FHWA design procedure is sound, at least for the case studies examined, but proposes some minor improvements and clarifications to the procedure as follows:

1. The line for setting the anchor free-lengths should extend from the embedded base of the wall, not from the base of the excavation. The reason for this recommendation is that the PLAXIS analysis indicates that active soil failure surfaces that develop pass below the embedded portions of the wall. The bond zone of the anchors should be placed outside of the active soil wedge. (This approach was used in all of the case studies in this report).
2. Further, it is recommended that the assumed angle of inclination of the active wedge should be calculated using the Mononobe-Okabe theory [Okabe, 1926; Mononobe and Matsuo, 1929]. An additional flattening by five degrees is recommended to provide some "buffer" against variability and uncertainty in soil parameters and to improve the seismic performance.
3. The depth of embedment of soldier piles or continuous wall elements should be calculated using Broms's [1965] theory with a factor of safety of 3. The procedure for calculating depth of embedment is not clearly stated in the FHWA procedure. A factor of safety of 3 when using Broms's theory to calculate passive lateral resistance of piles is commonly recommended because of the large deformations required to generate passive soil resistance.
4. Final checks of wall designs should be made using limiting equilibrium methods to verify both the internal and external stability. This process is alluded to in the FHWA report but no clear guidance is given. (Detailed example calculations for the case studies are appended to this report.)

The recommendations given in this report are based on detailed analysis of a limited number of case studies with greatly simplified soil conditions and so should be considered tentative.

No consideration is given in the recommended design procedure for directly considering the likely deformations. The limited case study analyses indicate that deformations will be “reasonable” for many situations. Of course “reasonableness” of deformation will depend very much on the particular context of each individual wall being designed. The designer must consider which situations will be more critical to deformation and carry out appropriate analysis.

4.2 Seismic Design Accelerations

The proposed design procedure for tied-back retaining walls is based on a “pseudo-static” approach where the equilibrium of the structure and internal stresses are assessed considering a static horizontal component of acceleration in addition to gravity. Generally, it is uneconomic or even impracticable to design walls to resist very high values of “quasi-static” acceleration. Instead, reduced values of acceleration, as of $1/3$ to $1/2$ the design peak ground acceleration, are considered [e.g. Kramer, 1996].

The rationale for using such low design values of horizontal acceleration is that the PGA occurs only for brief instants of time (in the millisecond range) during an earthquake. The soil mass is considered to behave in a “ductile” fashion - yielding briefly during these peaks with little accumulation of strain or strain-softening effects. The structural elements of the wall do not, in general, feel the effect of the momentary peak accelerations and respond mainly to the soil deformations.

In general, the lower the value of the selected “pseudo-static” acceleration, the greater will be the deformation of the wall and retained soil after an earthquake. However, an important conclusion from this study is that substantial reductions of deformation were found when walls were designed to resist even modest levels of pseudo-static acceleration (as low as 0.1 g). Further reductions in deformation with increasing levels of design acceleration were observed but became increasingly modest as the design acceleration was increased. Even when walls were designed to resist the full peak ground accelerations (100 percent of PGA) significant deformations still occurred.

Based on the results of this study and previous published recommendations, the following recommendations are made for minimum design accelerations for tied-back retaining walls:

1. Tied-back walls should be designed to resist a minimum pseudo-static horizontal acceleration of 0.1 g. This value is considered a sensible minimum value and was shown in this study to give very good improvements in earthquake performance for a modest increase in wall cost. As an additional benefit, the factor of safety against instability under gravity loading is also increased.
2. For higher seismic hazard zones or for structures of greater importance, tied-back walls should be designed to resist a minimum pseudo-static acceleration of $1/3$ PGA to $1/2$ PGA of the design earthquake.

Where deformations are critical it may be necessary to use still higher values of pseudo-static acceleration. However, it rapidly becomes impractical to design walls to resist very high accelerations and it appears impossible to reduce deformations below certain limits.

4.3 Proposed Design Guidelines for “Sand” soils

The following is a detailed recommendation for designing tied-back retaining walls in “sand” soils. Examples of the calculations are given for the case studies in the Appendices.

- a) **Initial trial geometry:** The depth of excavation and depth to each row of anchors needs to be estimated as a first step, based on experience or trial and error.
- b) **Prepare apparent earth pressure diagram:** As shown in Figure 4.3 (a). Note that K_A is calculated using the Mononobe-Okabe equation with the selected design pseudo-static acceleration. The wall is assumed to be frictionless (i.e. the wall is likely to move downwards with any active soil wedge).
- c) **Calculate anchor design load(s):** As shown in Figure 4.3 (a).
- d) **Calculate wall base reaction, R:** As shown in Figure 4.3 (a).
- e) **Calculate wall section bending moment:** From the apparent earth pressure diagram as shown in Figure 4.3 (b). These methods are considered to provide conservative estimates of the calculated bending moments, but may not accurately predict the specific locations of the maximum. FHWA recommends an allowable stress of $F_b = 0.55 F_y$ for steel soldier piles. For New Zealand design procedures using load and resistance factor design (LRFD) principles and for a strength reduction factor for steel sections of 0.8, an equivalent load factor of $\alpha = 0.8/0.55 = 1.45$ is implied. However, for consistency with NZS 4203 a load factor of 1.6 is recommended for the purpose of sizing wall structural elements. (This procedure was found to be suitably conservative for the case studies in this report).
- f) **Determine depth of embedment:** Calculate required depth of embedment for soldier piles to resist wall base reaction (R) using Broms [1965] (but calculating K_p using the Mononobe-Okabe equations including the design acceleration), or, for continuous walls using passive resistance from Mononobe-Okabe theory. A strength reduction factor of 3 is recommended to be applied to these calculations because of the large plastic strains required to mobilise the full passive resistance.
- g) **Check internal stability of the wall:** A possible internal failure mechanism is shown in Figure 4.3 (c), with an active failure wedge immediately behind the wall, a passive wedge immediately in front of the embedded toe of the wall, and the anchor(s) developing their ultimate capacity (taken to be the

proven, test capacity, normally 1.33 times the design load or 80 percent of the anchor tensile capacity).

The true factor of safety may be determined by progressively reducing the assumed soil strength in the calculations until the driving and resisting forces are just equal, i.e:

$$\text{Active force} = \text{Passive force} + \text{anchor ultimate force}$$

when the factor of safety against sliding is given by:

$$FS = \frac{\tan^{-1}(\phi)}{\tan^{-1}(\phi_{\text{reduced}})}$$

For the earthquake load case using pseudo-static design, a minimum factor of safety of 1.2 is recommended, but not less than the factor of safety against external stability.

- h) **Set “free” length of anchor tendons:** The “free” length of the anchor tendons should extend beyond the active soil wedge defined by the Mononobe-Okabe theory flattened by five degrees, originating at the base of the wall or the embedded soldier piles as indicated in Figure 4.3 (c).
- i) **Check external stability of the wall:** External stability of tied-back retaining walls in cohesionless soil is controlled by horizontal sliding of the wall with formation of an active soil wedge behind the wall and a passive wedge in front of the wall base, as shown in Figure 4.3 (c). The critical failure surface is assumed to pass immediately behind the anchor bond zone, as shown.

For the earthquake load case using pseudo-static design, a minimum “true” factor of safety of 1.1 based on mobilised soil shear strength is recommended.

- j) **Note:** When calculating passive soil resistance, the interface friction angle should be set to be no more than $\phi/2$. Use of higher values is not recommended because the resulting values of passive resistance will be unrealistically high.

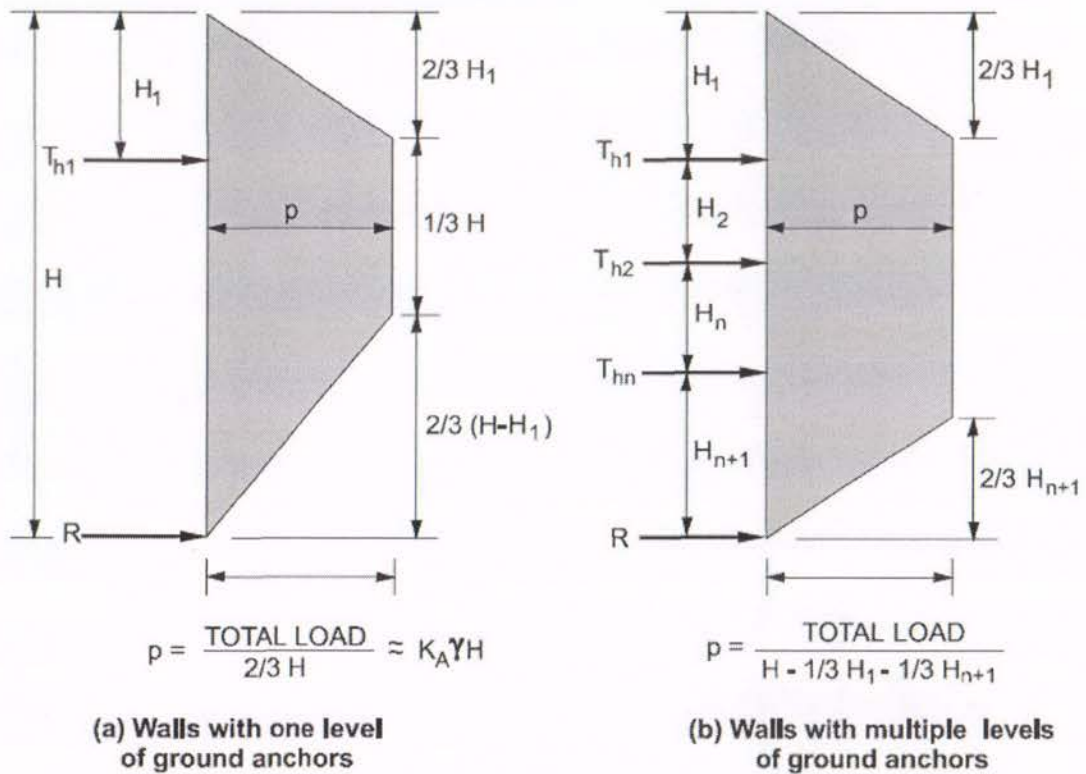


Figure 4.3 (a). Apparent earth pressure diagram for cohesionless soils.

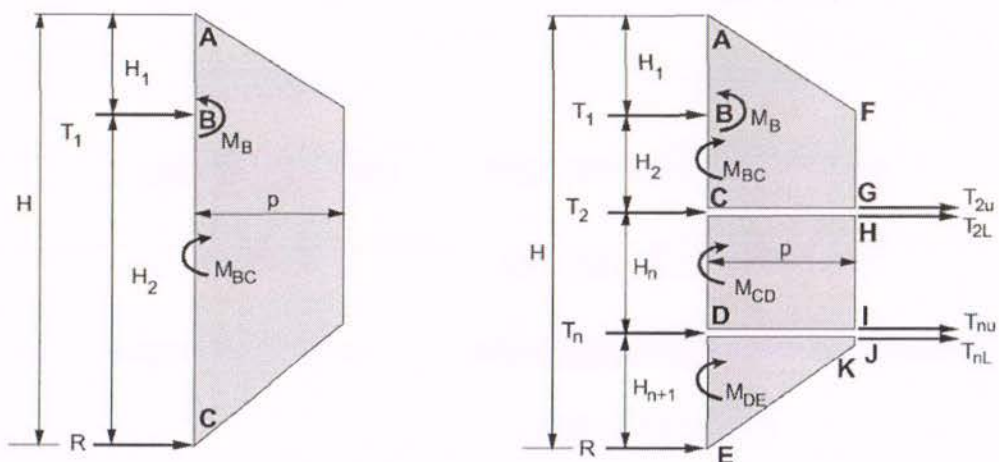


Figure 4.3 (b). Estimation of wall element bending moments.

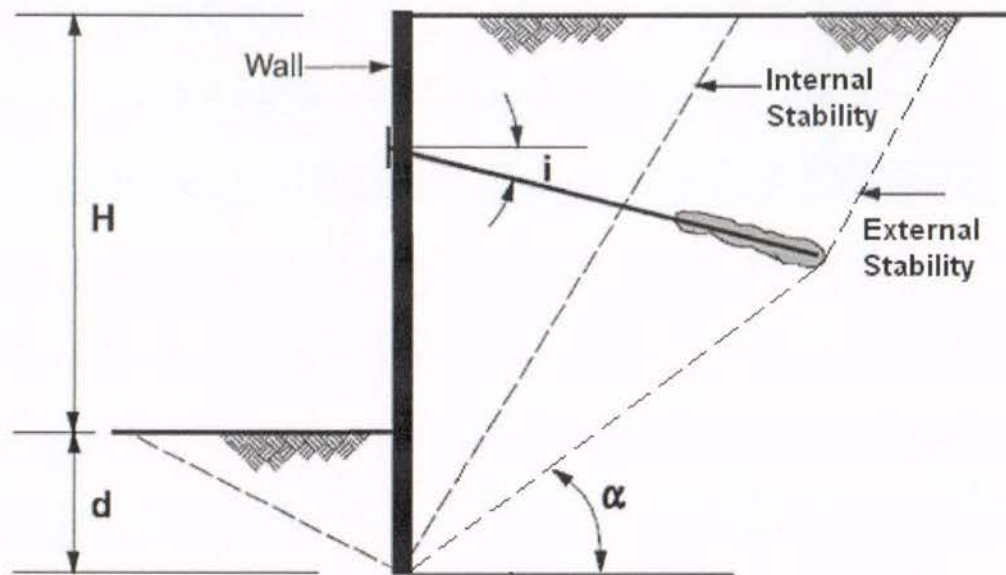


Figure 4.3 (c). Internal and external stability mechanisms.

5 Summary and Conclusions

Very little guidance is available for the design of tied-back retaining walls to resist earthquake shaking. Little observational data on the behaviour of tied-back walls during earthquakes has been published, but, what there is suggests that they behave well.

A survey of New Zealand practice has showed that there is no consistency of approach and that most designers are relying on a range of different “black box” computer software with earthquake loading input simply as an additional horizontal force applied directly to the wall. The appropriateness of this approach is questionable because the full range of different failure modes is not necessarily addressed by the software nor is it always obvious what the software does.

Methods for calculating the input additional earthquake “loading” on the wall also were found to vary from the lower bound, Mononobe-Okabe approach (a variation of Coulomb’s method for calculating static loads on retaining walls) where the soil is assumed to be in a fully yielding “Rankine” state to the upper bound Wood approach where the wall is assumed to be rigid and the soil to remain fully elastic.

The commonly used software packages do not give guidance as to the length of anchors required, especially the “free length” of the anchors. This study has shown that the anchor lengths are very important in determining the wall response and that they should be lengthened as the design acceleration of the wall increases.

The focus of this study has been to develop a pragmatic, practical design procedure that produces safe and economical designs and that does not depend on “black box” software. As a starting point, a well established design procedure for tied-back retaining walls under gravity loading was adopted (FHWA procedure, based on the semi-empirical “apparent earth pressure” method of Terzaghi and Peck) and verified for different, simplified case studies, using PLAXIS finite element analysis software. The analyses showed that this design procedure produced walls with adequate, but not excessive, factors of safety against instability (FS = 1.38 for Case Study 1, 7 m high wall with one row of anchors, FS = 1.32 for Case Study 2, 12 m high wall with two rows of anchors).

The case studies assumed a generic, uniform “sand” soil with average properties modelled in PLAXIS using the hardening soil model.

These standard, gravity designs then were subjected to simulated earthquakes, scaled to different values of peak ground acceleration ($PGA = 0.2\text{ g}$ to 0.6 g) by using numerical time history analysis. The walls performed surprisingly well considering that they were not specifically designed to resist earthquakes: Wall displacements became quite significant (worst case of 350 mm for the 7 m high wall under Loma Prieta record scaled to 0.6 g) for the extreme earthquake “events”, but, the walls remained stable, anchor forces remained well within acceptable limits, and the soldier piles only reached yield in one case (7 m high wall, Loma Prieta record scaled to 0.6 g).

The PLAXIS time histories showed that the structural elements were little affected by the high peak ground accelerations. Anchor forces generally varied little during the earthquakes, and showed little or no response to the large instantaneous variation in ground acceleration, even for the very high peaks. Instead, anchor forces seemed to respond more to the gross deformations of the soil mass, either increasing slightly, or, in some case, decreasing. Likewise, soldier pile bending moments showed little response to the instantaneous variation in ground acceleration but showed a more steady increase in bending moment towards the base of the wall as the gross deformation of the soil mass increased during an earthquake.

Given the good performance of the walls designed using the gravity only FHWA procedure, it was decided to use this procedure as the basis for an earthquake design procedure. The FHWA document recommends the use of the pseudo-static Mononobe-Okabe theory to design tied-back walls to resist earthquakes but does not give a detailed procedure. Nor is such a procedure obvious because the recommended design procedure for tied-back walls under gravity loading is based on the semi-empirical "apparent earth pressure" diagrams of Terzaghi and Peck. However, for sands the apparent earth pressure is assumed to be proportional to K_a , the Rankine active earth pressure and it was assumed that the equivalent apparent earth pressure for the earthquake design case might be proportional to K_{ah} , the Mononobe-Okabe value for active earth pressure.

Of equal importance was the evident need to also extend the anchor free-lengths to beyond the active soil zone immediately behind the wall. The Mononobe-Okabe theory also provides a means for calculating the location of the active zone.

A new design procedure was synthesized from the FHWA procedure incorporating the use of the Mononobe-Okabe theory. The case study walls then were re-designed using the new procedure for various design accelerations from 0.1 g to 0.4 g and tested by running them through the same PLAXIS numerical earthquake simulations. Significant improvements in performance, reductions in wall deformation mainly, were observed even for walls designed to resist low acceleration (0.1 g).

As walls were designed to resist greater levels of horizontal acceleration, wall displacements continued to reduce, but at a decreasing rate. Even when a wall was designed to resist 100 percent of the PGA of an earthquake, it still accumulated significant permanent displacement by the end of the shaking.

There was a great range in wall displacements among the three different earthquake records modelled (Loma Prieta, Parkfield, Sierra Madre), with variations of as much as 300 percent.

The greatest benefit-cost ratio was found for the walls designed to resist the low level (0.1 g) accelerations. The additional cost of building the case study walls to resist 0.1 g was modest (about 25 percent) for a good reduction in wall deformation (about 30 percent). The cost of increasing resistance beyond 0.2 g starts to increase very rapidly with only modest reductions in wall deformation observed.

Designing walls to resist even a low level of acceleration (0.1 g) had the additional benefit of significantly increasing the factor safety against instability for gravity loading.

The benefit-cost ratios established for the 7 m high wall (Case Study 1) indicate that the optimum design is probably gained by making the design acceleration about $\frac{1}{2}$ the PGA of the design earthquake (e.g. for design earthquake with PGA = 0.2 g make the design acceleration 0.1 g, and for a design earthquake with PGA = 0.4 g make the design acceleration 0.2 g). Such a recommendation would be in keeping with accepted practice which is to design retaining walls to resist pseudo-static acceleration of between $\frac{1}{2}$ and $\frac{1}{3}$ of the design earthquake PGA.

For the higher, 12 m walls, the study showed similar trends, although the benefit-cost ratio was about the same for walls designed to resist 0.1 g and 0.2 g horizontal acceleration.

The effect of increasing the anchor free length to beyond the active wedge slip plane (flattening the slip plane by five degrees) was studied for 12 m high walls in Case Study 3. The increase in anchor free length resulted in an improvement in wall displacement response averaging 16 percent for a small increase of 10 percent in anchor cost. Anchor forces during and after earthquake shaking remained much the same and well below anchor ultimate tensile strengths.

Increasing the free length of the lower anchor resulted in anchor tensile failure becoming the critical failure mechanism for gravity loading. However, the factor of safety against failure under gravity loading was improved.

This study has demonstrated that use of the PLAXIS finite element software with the dynamic analysis module is a useful tool for studying the performance of tied-back retaining walls and, probably, other complex problems in soil-structure interaction. The only difficulty experienced with the software was the inability to analyse deep excavations under high pseudo-static accelerations. However, this is not considered a serious limitation since such situations are somewhat artificial and divorced from practical reality.

This study has considered a limited range of case studies, and for walls greater in height than the 12 m considered, it is strongly recommended that a special study using PLAXIS analysis or similar be considered during the design process. The trends from the case studies in this report suggest that as walls get higher, the factors of safety reduce and the safety of the proposed design procedure has not been confirmed for such extrapolations.

A detailed, recommended design procedure for design of tied-back retaining walls with earthquake loading is given in Section 4. This recommended procedure is based on detailed analysis of a limited number of case studies with greatly simplified soil conditions and so should be considered tentative. On the other hand, it is based closely on a well proven gravity design procedure.

No consideration is given in the recommended design procedure for directly considering the likely deformations during shaking. The limited case study analyses indicate that deformations will be "reasonable" for many situations. Of course "reasonableness" of deformation will depend very much on the particular context of each individual wall being designed. The designer must consider which situations will be more critical to deformation and carry out appropriate analysis.

It is recommended that walls should be designed to resist a minimum pseudo-static acceleration of 0.1 g. Based on the case studies, this level of acceleration was shown to give a good improvement in wall performance for only a modest increase in cost. The factor of safety against failure under gravity loading was also shown to be significantly enhanced.

Little benefit was found from designing walls to resist the maximum peak ground acceleration (PGA) for a given earthquake: The improvements in wall performance were found to become more modest once the design acceleration exceeded about 0.2 g. Even when walls were designed to resist 100 percent of the earthquake PGA, significant wall deformations still occurred.

Attempts to design tied-back retaining walls to resist very high levels of horizontal acceleration become difficult in any case.

The observations by Ho et. al. [1991] of little damage to tied- back retaining walls is understandable given the conclusions from this study: The walls studied herein were found to be robust even when not specifically designed to resist earthquake shaking.

The observation of Frigaszy et. al. [1987] that wall elements extending into the foundation soils may be subjected to very high bending moments is not supported by this study. Adequate depth of embedment for all the wall studied was found to be a critical aspect of wall performance since it governs both the internal and external stability of a wall. Walls embedded into stiff soils but supporting softer soils may expect to have more severe concentrations of bending at the interface but this was not explored in this present study.

Sabatini et. al [1999] recommended that brittle elements of the wall system, specifically the grout tendon bond should be governed by the peak ground acceleration. This present study has shown that anchor forces are little affected by even very high peak ground accelerations and normal anchor detailing and testing should be adequate.

Most researchers to date have focused on calculating a “pressure” to be applied to walls arising from the earthquake shaking, perhaps failing to view the wall and soil mass response in a holistic way. The results of this study indicate that there are complex interactions between the retained soil mass and the wall elements that contribute to a greater then expected resilience for walls but at the expense of deformations that seem impossible to reduce below certain levels.

6 *Recommendations for Future Research*

The recommendations given in this report are based on detailed analysis of a limited number of case studies with greatly simplified soil conditions. The effect of varying soil conditions such as soft retained soil overlying a much stiffer foundation need further investigation.

Also, the study should be extended to look at higher walls. As walls increase in height their complexity increases because of the need for multiple rows of anchors. They also become more flexible (relatively) and there will be more kinematic interaction with incoming seismic waves during earthquakes.

The case studies considered were for steel soldier pile walls and the results are considered applicable to reinforced concrete soldier pile walls, continuous concrete walls, and also to steel sheet pile walls. Timber poles are also commonly used for tied-back retaining walls of moderate height and require specific consideration because of their more limited ductility.

Great variances in wall displacement were observed for the different earthquake time history records considered in this study. It would be useful to understand the reasons for those variances so that it is possible to better predict wall deformation.

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Appendix A

A.1 Design calculations for case study Sand 1a – Gravity based design

1. Calculation of K_a

$k_h := 0$	horizontal acceleration in g	$\theta := \text{atan}(k_h)$	$\theta = 0$
$\beta := 0\text{-deg}$	slope of the back of the wall		
$i := 0\text{-deg}$	slope of the backfill		
$\phi := 35\text{-deg}$	angle of internal friction		
$\delta_i := 0$	angle of interface friction		

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi - \theta - i)}{\cos(\delta_i + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.476$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_i + \theta) \cdot D} \quad K_{AE} = 0.271$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_i) K_{AE} \quad K_{AEH} = 0.271$$

2. Calculation of apparent earth pressure (Refer to Figure 2.2.2 (a). Units = KN/m^2)

$H_{\text{wall}} := 7$	Height of wall	
$H_1 := 2$	Distance to anchor	
$K_A := 0.271$		
$\gamma := 16$		
$\text{Load} := 0.65 K_A \cdot \gamma \cdot H_{\text{wall}}^2$		Load = 138.102
$p := \frac{\text{Load}}{0.667 H_{\text{wall}}}$		$p = 29.578$

3. Calculation of anchor design load and reaction force required at base of wall.

$$L_a := 2 \cdot \frac{H_1}{3} \quad L_a = 1.333$$

$$L_b := \frac{H_{\text{wall}}}{3} \quad L_b = 2.333$$

$$L_c := 2 \cdot \frac{H_{\text{wall}} - H_1}{3} \quad L_c = 3.333 \quad L_{\text{check}} := L_a + L_b + L_c \quad L_{\text{check}} = 7$$

$$M_R := p \cdot \left[\frac{L_c^2}{3} + L_b \cdot \left(\frac{L_b}{2} + L_c \right) + \frac{L_a}{2} \cdot \left(\frac{L_a}{3} + L_b + L_c \right) \right] \quad M_R = 540.628$$

$$T_{\text{anchor}} := \frac{M_R}{H_{\text{wall}} - H_1} \quad T_{\text{anchor}} = 108.126$$

$$R_{\text{base}} := \text{Load} - T_{\text{anchor}} \quad R_{\text{base}} = 29.976$$

4. Calculation of cantilever moment in wall element above anchor

Cantilever moment

$$L_{a2} := \frac{H_1}{3} \quad L_{a2} = 0.667$$

$$M_c := p \cdot \left[\frac{L_a}{2} \cdot \left(\frac{L_a}{3} + L_{a2} \right) + \frac{L_{a2}^2}{2} \right] \quad M_c = 28.483$$

5. Calculation of maximum bending moment in wall element below anchor

$$V_{La} := p \cdot \frac{L_a}{2} \quad V_{La} = 19.719$$

$$V_{H1} := V_{La} + p \cdot (H_1 - L_a) \quad V_{H1} = 39.438$$

$$Z_{\text{vzero}} := \frac{T_{\text{anchor}} - V_{H1}}{p} + L_a \quad Z_{\text{vzero}} = 3.656$$

$$\text{Must be less than} \quad Z_{\text{max}} := H_{\text{wall}} - L_c \quad Z_{\text{max}} = 3.667$$

$$M_{\text{max}} := \frac{p \cdot L_a}{2} \cdot \left(Z_{\text{vzero}} - 2 \cdot \frac{L_a}{3} \right) + \frac{p \cdot (Z_{\text{vzero}} - L_a)^2}{2} - T_{\text{anchor}} \cdot (Z_{\text{vzero}} - H_1)$$

$$M_{\text{max}} = -44.698$$

$$M_{\text{star}} := M_{\text{max}} \cdot 1.6 \quad M_{\text{star}} = -71.517$$

M^* = ULS design bending moment

6. Selection of wall structural element

A typical spacing for a soldier pile wall is 2 m crs. Therefore ULS design moment $M^* = 143 \text{ KNm}$ per each. For example, a 200UC52.2 steel column section would suffice.

For the purpose of this study, it was assumed that either, such a steel column was set into a concrete filled 450 mm diameter hole, or, a reinforced concrete soldier pile of the same diameter was used.

7. Calculation of embedment depth for soldier piles

Simple Broms theory is used, with K_p calculated using Coulomb theory. Coulomb theory is used to be compatible with M-O theory for later earthquake design case. Interface friction is limited to $\phi/2$ because Coulomb theory is unconservative at higher levels of interface friction, the resulting value for K_p in this case is quite close to the value given by the NAVFAC charts which are based on log-spiral theory.

$B := 0.45$	Pile diameter	
$D := 2.1$	Depth of embedment of pile	
$\gamma := 8$	Soil unit weight (buoyant)	
$\phi := 35\text{-deg}$		
$k_h := 0$	horizontal acceleration in g	$\theta := \text{atan}(k_h) \quad \theta = 0\text{deg}$
$\beta := 0\text{-deg}$	slope of the back of the wall	
$i := 0\text{-deg}$	slope of the backfill	
$\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$	angle of interface friction (passive)	

Passive K_p

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_p = 0.089$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p} \quad K_{PE} = 8.032$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 7.581$$

Ultimate Horizontal Resistance

$$H_u := \frac{3}{2} \cdot \gamma \cdot B \cdot K_{PEH} D^2 \quad H_u = 180.527$$

A strength reduction factor of 3 is recommended to be applied to the ultimate horizontal resistance calculated using Broms' theory because of the large plastic

strains required to mobilise full passive resistance. Therefore, for piles spaced at 2 m centres:

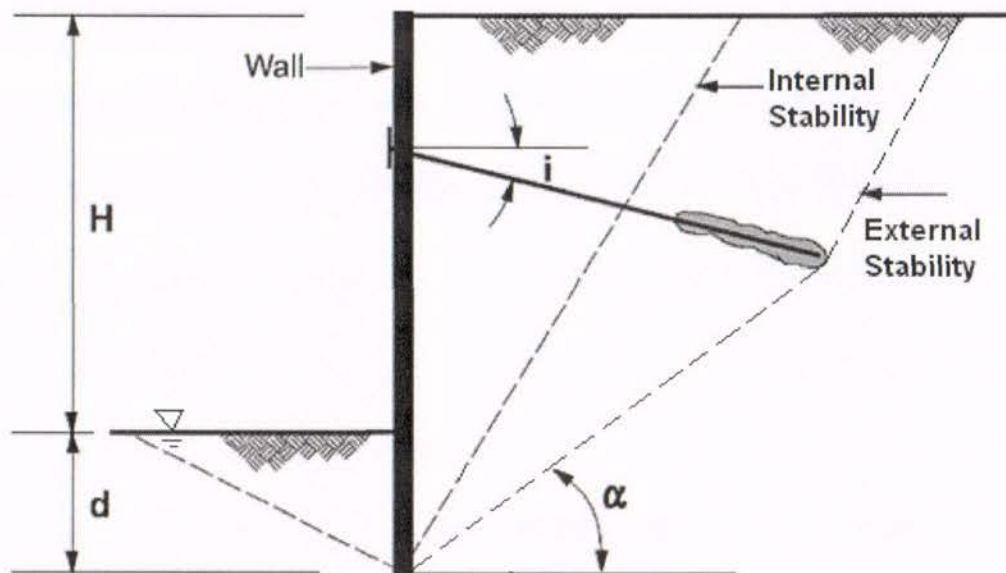
$$\text{Design Resistance} = 180/3 = 60 \text{ kN each pile or } 30 \text{ kN/m run}$$

$$\text{Design Demand } (R_{\text{base}}) = 30 \text{ kN/m}$$

Therefore, embedment depth of 2.1 m is optimum.

8. Check for internal stability

(True FS determined by successive reduction of ϕ)



$$\begin{aligned}
 k_h &:= 0 && \text{horizontal acceleration in g} && \theta := \text{atan}(k_h) && \theta = 0 \text{ deg} \\
 \beta &:= 0 \text{ deg} && \text{slope of the back of the wall} \\
 i &:= 0 \text{ deg} && \text{slope of the backfill} \\
 \phi &:= 26.78 \text{ deg} && \text{angle of internal friction} \\
 \delta_i &:= \text{atan}(0.5 \tan(\phi)) && \text{angle of interface friction (passive)} && \delta_a := \delta_i
 \end{aligned}$$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.408$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.341$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.331$$

Failure surface inclination

$$\begin{aligned}
 aa &:= \phi - i - \theta && bb := \phi - \beta - \theta && cc := \delta_a + \beta + \theta \\
 aa &= 26.78 \text{ deg} && bb &= 26.78 \text{ deg} && cc = 14.163 \text{ deg}
 \end{aligned}$$

$$\rho_A := \phi - \theta + \text{atan} \left[\frac{\sqrt{\tan(aa) \cdot (\tan(aa) + \cot(bb)) \cdot (1 + \tan(cc) \cdot \cot(bb))} - \tan(aa)}{1 + \tan(cc) \cdot (\tan(aa) + \cot(bb))} \right]$$

$$\rho_A = 54.832 \text{ deg}$$

Passive Kp

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_p = 0.201$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p} \quad K_{PE} = 4.092$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 3.968$$

Wedge Calculation

Single anchor, water at base of excavation

$$H_{exc} := 7 \quad \text{Depth of excavation}$$

$$H_{embed} := 2.1 \quad \text{Embedment of piles}$$

$$F_H := 108.133 \quad \text{Anchor horizontal force (ultimate)} \quad F_H = 143.64$$

$$\gamma_{above} := 16$$

$$\gamma_{below} := 8$$

$$P_A := K_{AEH} \left(0.5 \gamma_{above} \cdot H_{exc}^2 + \gamma_{above} \cdot H_{exc} \cdot H_{embed} + 0.5 \gamma_{below} \cdot H_{embed}^2 \right) \quad P_A = 213.411$$

$$P_P := K_{PEH} 0.5 \gamma_{below} \cdot H_{embed}^2 \quad P_P = 69.994$$

Stability calculation

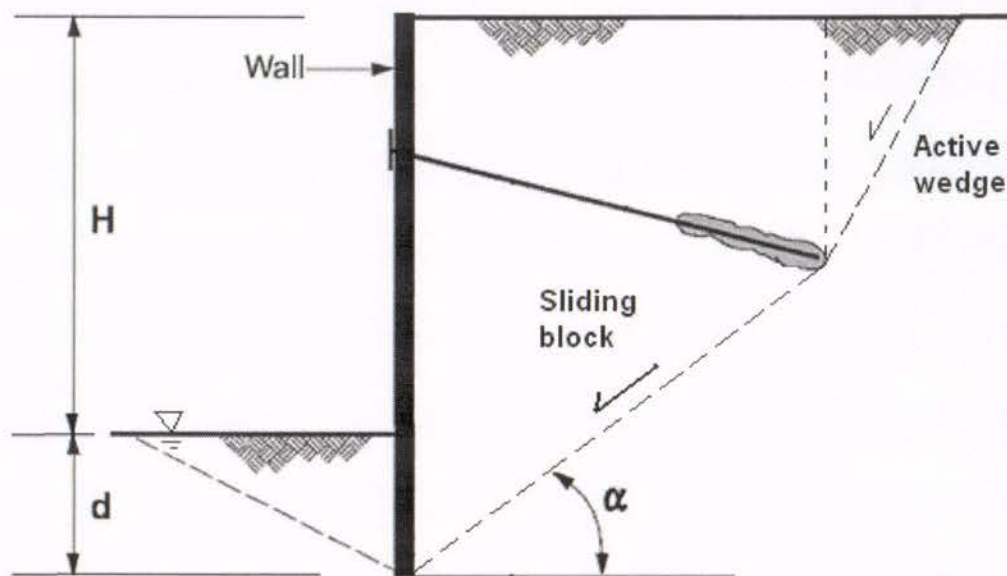
$$H_{net} := P_A - P_P - F_H \quad H_{net} = -0.223 < 0 \text{ for stability}$$

FS Calculation (by trial and error to set $H = 0$)

$$\phi_{design} := 35 \text{ deg} \quad FS := \frac{\tan(\phi_{design})}{\tan(\phi)} \quad FS = 1.387$$

(Note: For comparison, a simple FS, calculated as $(P_P + F_H)/P_A$, = 1.7)

9. Check for external stability



$k_h := 0$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 0 \text{ deg}$

$\beta := 0 \text{ deg}$ slope of the back of the wall

$i := 0 \text{ deg}$ slope of the backfill

$\phi := 21.1 \text{ deg}$ angle of internal friction

$\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := \delta_i$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.076$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.427$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.419 \quad \text{Equivalent Horizontal Component}$$

Sliding block details

$B := 9$ Breadth of block

$\alpha := 16.5 \text{ deg}$ Failure surface

$H_{exc} := 7$ Depth of excavation

$H_{embed} := 2.1$ Embedment of piles

$\gamma_{above} := 16$

$\gamma_{below} := 8$

$$W_{\text{block}} := \gamma_{\text{above}} \cdot \left[B \cdot (H_{\text{exc}} + H_{\text{embed}}) - 0.5 \tan(\alpha) \cdot B^2 \right] \quad W_{\text{block}} = 1.118 \times 10^3$$

$$W_{\text{buoy}} := W_{\text{block}} - (\gamma_{\text{above}} - \gamma_{\text{below}}) \cdot 0.5 \frac{H_{\text{embed}}^2}{\tan(\alpha)} \quad W_{\text{buoy}} = 1.059 \times 10^3$$

correction for water
assumes active
wedge is dry

$$H_{\text{block}} := W_{\text{block}} \cdot \left(\frac{\sin(\alpha) - \tan(\phi) \cdot \cos(\alpha)}{\cos(\alpha) + \tan(\phi) \cdot \sin(\alpha)} + k_h \right) \quad H_{\text{block}} = -89.989$$

this is the net contribution to horizontal movement - should be negative unless $\alpha = \phi$

Active pressure wedge (zero interface friction)

$$H_{\text{active}} := H_{\text{exc}} + H_{\text{embed}} - B \cdot \tan(\alpha)$$

$$P_{\text{ah}} := 0.5 \cdot \gamma_{\text{above}} \cdot K_{\text{AEH}} \cdot H_{\text{active}}^2$$

Passive Resistance

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{\text{PE}} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p}$$

Equivalent Horizontal Component

$$K_{\text{PEH}} := \cos(\delta_i + \beta) K_{\text{PE}} \quad K_{\text{PEH}} = 2.785$$

$$P_{\text{ph}} := 0.5 \cdot \gamma_{\text{below}} \cdot K_{\text{PEH}} \cdot H_{\text{embed}}^2 \quad P_{\text{ph}} = 49.119$$

Wedge-block Stability Calculation

$$H_{\text{net}} := P_{\text{ah}} + H_{\text{block}} - P_{\text{ph}} \quad H_{\text{net}} = -0.268 \quad < 0 \text{ for stability}$$

FS Calculation (set H = 0 by trial and error)

$$\phi_{\text{design}} := 35 \cdot \text{deg} \quad \text{FS} := \frac{\tan(\phi_{\text{design}})}{\tan(\phi)} \quad \text{FS} = 1.815$$

A.2 Design calculations for case study Sand 1b – M-O based design

0.1 g

1. Calculation of K_a

$$\begin{aligned}
 k_h &:= 0.1 && \text{horizontal acceleration in g} && \theta := \text{atan}(k_h) && \theta = 0.1 \\
 \beta &:= 0\text{-deg} && \text{slope of the back of the wall} \\
 i &:= 0\text{-deg} && \text{slope of the backfill} \\
 \phi &:= 35\text{-deg} && \text{angle of internal friction} \\
 \delta_i &:= 0 && \text{angle of interface friction} && \delta_a := \delta_i
 \end{aligned}$$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi - \theta - i)}{\cos(\delta_i + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.344$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_i + \theta)} \cdot D \quad K_{AE} = 0.328$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_i) K_{AE} \quad K_{AEH} = 0.328$$

2. Calculation of apparent earth pressure

(Refer to Figure 2.2.2 (a). Units = KN/m²)

$$\begin{aligned}
 H_{\text{wall}} &:= 7 && \text{Height of wall} \\
 H_1 &:= 2 && \text{Distance to anchor} \\
 K_A &:= 0.328 \\
 \gamma &:= 16 \\
 \text{Load} &:= 0.65 K_A \cdot \gamma \cdot H_{\text{wall}}^2 && \text{Load} = 167.149 \\
 p &:= \frac{\text{Load}}{0.667 H_{\text{wall}}} && p = 35.8
 \end{aligned}$$

3. Calculation of anchor design load and reaction force required at base of wall.

$$\begin{aligned}
 L_a &:= 2 \cdot \frac{H_1}{3} && L_a = 1.333 \\
 L_b &:= \frac{H_{\text{wall}}}{3} && L_b = 2.333 \\
 L_c &:= 2 \cdot \frac{H_{\text{wall}} - H_1}{3} && L_c = 3.333 && L_{\text{check}} := L_a + L_b + L_c && L_{\text{check}} = 7
 \end{aligned}$$

$$M_R := p \cdot \left[\frac{L_c^2}{3} + L_b \cdot \left(\frac{L_b}{2} + L_c \right) + \frac{L_a}{2} \cdot \left(\frac{L_a}{3} + L_b + L_c \right) \right] \quad M_R = 720.172$$

$$T_{\text{anchor}} := \frac{M_R}{H_{\text{wall}} - H_1} \quad T_{\text{anchor}} = 144.034$$

$$R_{\text{base}} := \text{Load} - T_{\text{anchor}} \quad R_{\text{base}} = 39.931$$

4. Calculation of cantilever moment in wall element above anchor

$$L_{a2} := \frac{H_1}{3} \quad L_{a2} = 0.667$$

$$M_c := p \cdot \left[\frac{L_a}{2} \cdot \left(\frac{L_a}{3} + L_{a2} \right) + \frac{L_{a2}^2}{2} \right] \quad M_c = 34.474$$

5. Calculation of maximum bending moment in wall element below anchor

$$V_{La} := p \cdot \frac{L_a}{2} \quad V_{La} = 23.866$$

$$V_{H1} := V_{La} + p \cdot (H_1 - L_a) \quad V_{H1} = 47.733$$

$$Z_{\text{vzero}} := \frac{T_{\text{anchor}} - V_{H1}}{p} + L_a \quad Z_{\text{vzero}} = 3.656$$

$$\text{Must be less than} \quad Z_{\text{max}} := H_{\text{wall}} - L_c \quad Z_{\text{max}} = 3.667$$

$$M_{\text{max}} := \frac{p \cdot L_a}{2} \cdot \left(Z_{\text{vzero}} - 2 \frac{L_a}{3} \right) + \frac{p \cdot (Z_{\text{vzero}} - L_a)^2}{2} - T_{\text{anchor}} \cdot (Z_{\text{vzero}} - H_1)$$

$$M_{\text{max}} = -54.1$$

$$M_{\text{star}} := M_{\text{max}} \cdot 1.6 \quad M_{\text{star}} = -86.559$$

M^* = ULS design bending moment

6. Selection of wall structural element

Keeping the same pile spacing as the gravity design of 2 m, the ULS design moment $M^* = 173$ kNm per each. For example, a 200UC59.5 steel column section would suffice.

For the purpose of this study, it was assumed that either, such a steel column was set into a concrete filled 450 mm diameter hole, or, a reinforced concrete soldier pile of the same diameter was used.

7. Calculation of embedment depth for soldier piles

Simple Broms theory is used, with K_p calculated using Coulomb theory. Coulomb theory is used to be compatible with M-O theory for later earthquake design case. Interface friction is limited to $\phi/2$ because Coulomb theory is unconservative at higher levels of interface friction, the resulting value for K_p in this case is quite close to the value given by the NAVFAC charts which are based on log-spiral theory.

$B := 0.45$	Pile diameter	
$D := 2.4$	Depth of embedment of pile	
$\gamma := 8$	Soil unit weight (buoyant)	
$\phi := 35\text{-deg}$		
$k_h := 0.1$	horizontal acceleration in g	$\theta := \text{atan}(k_h) \quad \theta = 5.711\text{deg}$
$\beta := 0\text{-deg}$	slope of the back of the wall	
$i := 0\text{-deg}$	slope of the backfill	
$\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$	angle of interface friction (passive)	

Passive K_p

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_p = 0.114$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p} \quad K_{PE} = 7.387$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 6.972$$

Ultimate Horizontal Resistance

$$H_u := \frac{3}{2} \cdot \gamma \cdot B \cdot K_{PEH} D^2 \quad H_u = 216.865$$

A strength reduction factor of 3 is recommended to be applied to the ultimate horizontal resistance calculated using Broms' theory because of the large plastic strains required to mobilise full passive resistance. Therefore, for piles spaced at 2 m centres:

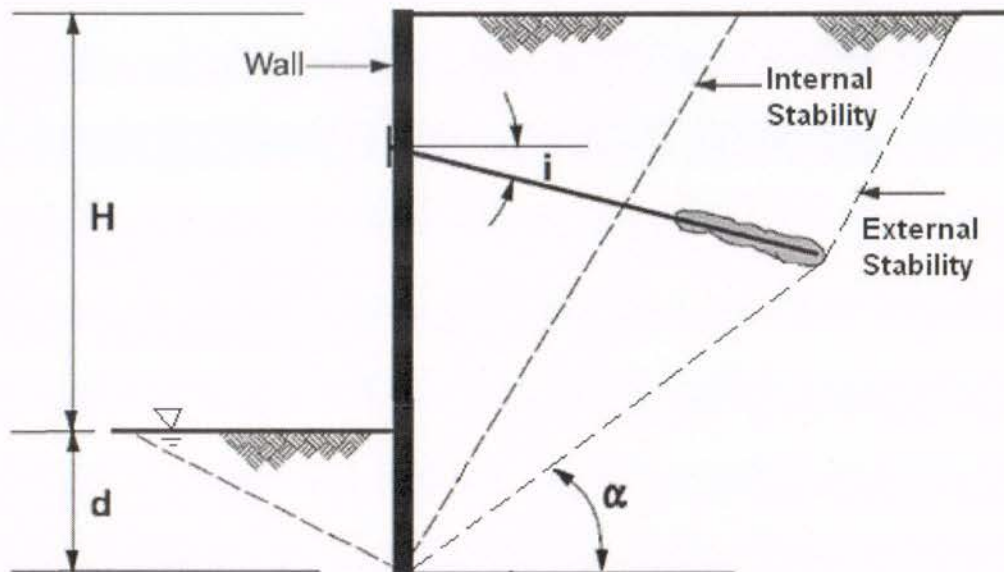
$$\text{Design Resistance} = 217/3 = 72 \text{ KN each pile or } 36 \text{ KN/m run}$$

$$\text{Design Demand } (R_{\text{base}}) = 36 \text{ KN/m}$$

Therefore, embedment depth of 2.4 m is optimum.

8. Check for internal stability

(True FS determined by successive reduction of ϕ)



$k_h := 0.1$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 5.711 \text{ deg}$
 $\beta := 0 \text{ deg}$ slope of the back of the wall
 $i := 0 \text{ deg}$ slope of the backfill
 $\phi := 29.6 \text{ deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.098$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.403$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.403$$

Failure surface inclination

$$aa := \phi - i - \theta \quad bb := \phi - \beta - \theta \quad cc := \delta_a + \beta + \theta$$

$$aa = 23.889\text{deg} \quad bb = 23.889\text{deg} \quad cc = 5.711\text{deg}$$

$$\rho_A := \phi - \theta + \text{atan} \left[\frac{\sqrt{\tan(aa) \cdot (\tan(aa) + \cot(bb)) \cdot (1 + \tan(cc) \cdot \cot(bb))} - \tan(aa)}{1 + \tan(cc) \cdot (\tan(aa) + \cot(bb))} \right]$$

$$\rho_A = 55.049\text{deg}$$

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_P = 0.196$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P} \quad K_{PE} = 4.606$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 4.43$$

Wedge Calculation

Single anchor, water at base of excavation

$$H_{exc} := 7 \quad \text{Depth of excavation}$$

$$H_{embed} := 2.4 \quad \text{Embedment of piles}$$

$$F_H := 131.133 \quad \text{Anchor horizontal force (ultimate)} \quad F_H = 174.23$$

$$\gamma_{above} := 16$$

$$\gamma_{below} := 8$$

$$P_A := K_{AEH} \left(0.5 \cdot \gamma_{above} \cdot H_{exc}^2 + \gamma_{above} \cdot H_{exc} \cdot H_{embed} + 0.5 \gamma_{below} \cdot H_{embed}^2 \right) \quad P_A = 275.249$$

$$P_P := K_{PEH} 0.5 \cdot \gamma_{below} \cdot H_{embed}^2 \quad P_P = 102.074$$

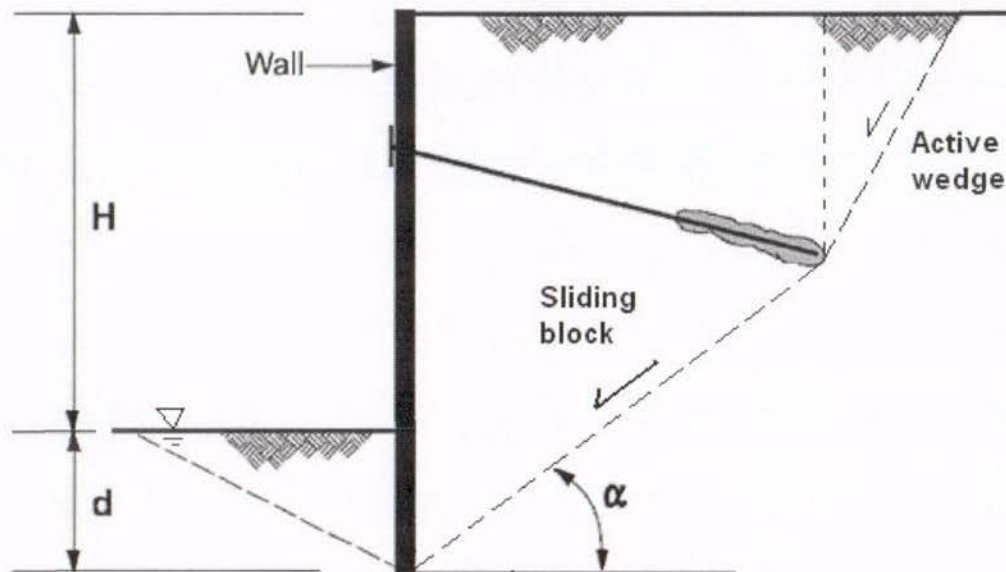
Stability calculation

$$H_{net} := P_A - P_P - F_H \quad H_{net} = -1.055 \quad < 0 \text{ for stability}$$

FS Calculation (by trial and error to set $H = 0$)

$$\phi_{design} := 35\text{-deg} \quad FS := \frac{\tan(\phi_{design})}{\tan(\phi)} \quad FS = 1.233$$

9. Check for external stability



$k_h := 0.1$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 5.711\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 27.4\text{deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 1.998$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.437$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.437 \quad \text{Equivalent Horizontal Component}$$

Sliding block details

$B := 11.1$ Breadth of block
 $\alpha := 21.7\text{-deg}$ Failure surface
 $H_{exc} := 7$ Depth of excavation
 $H_{embed} := 2.4$ Embedment of piles
 $\gamma_{above} := 16$
 $\gamma_{below} := 8$

$$W_{\text{block}} := \gamma_{\text{above}} \cdot \left[B \cdot (H_{\text{exc}} + H_{\text{embed}}) - 0.5 \tan(\alpha) \cdot B^2 \right] \quad W_{\text{block}} = 1.277 \times 10^3$$

$$W_{\text{buoy}} := W_{\text{block}} - (\gamma_{\text{above}} - \gamma_{\text{below}}) \cdot 0.5 \frac{H_{\text{embed}}^2}{\tan(\alpha)} \quad W_{\text{buoy}} = 1.219 \times 10^3$$

correction for water
assumes active
wedge is dry

$$H_{\text{block}} := W_{\text{block}} \cdot \left(\frac{\sin(\alpha) - \tan(\phi) \cdot \cos(\alpha)}{\cos(\alpha) + \tan(\phi) \cdot \sin(\alpha)} + k_h \right) \quad H_{\text{block}} = 0.238$$

This is the net contribution to horizontal movement

Active pressure wedge (zero interface friction)

$$H_{\text{active}} := H_{\text{exc}} + H_{\text{embed}} - B \cdot \tan(\alpha)$$

$$P_{\text{ah}} := 0.5 \cdot \gamma_{\text{above}} \cdot K_{\text{AEH}} \cdot H_{\text{active}}^2$$

Passive Resistance

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{\text{PE}} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p}$$

Equivalent Horizontal Component

$$K_{\text{PEH}} := \cos(\delta_i + \beta) \cdot K_{\text{PE}} \quad K_{\text{PEH}} = 3.775$$

$$P_{\text{ph}} := 0.5 \cdot \gamma_{\text{below}} \cdot K_{\text{PEH}} \cdot H_{\text{embed}}^2 \quad P_{\text{ph}} = 86.985$$

Wedge-block Stability Calculation

$$H_{\text{net}} := P_{\text{ah}} + H_{\text{block}} - P_{\text{ph}} \quad H_{\text{net}} = -0.046 \quad < 0 \text{ for stability}$$

FS Calculation (set H = 0 by trial and error)

$$\phi_{\text{design}} := 35\text{-deg} \quad \text{FS} := \frac{\tan(\phi_{\text{design}})}{\tan(\phi)} \quad \text{FS} = 1.351$$

A.3 Design calculations for case study Sand 1c – M-O based design

0.2 g

1. Calculation of K_a

Mononobe-Okabe Theory

$k_h := 0.2$	horizontal acceleration in g	$\theta := \text{atan}(k_h)$	$\theta = 0.197$
$\beta := 0\text{-deg}$	slope of the back of the wall		
$i := 0\text{-deg}$	slope of the backfill		
$\phi := 35\text{-deg}$	angle of internal friction		
$\delta_i := 0$	angle of interface friction		

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi - \theta - i)}{\cos(\delta_i + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.205$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_i + \theta) \cdot D} \quad K_{AE} = 0.396$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_i) K_{AE} \quad K_{AEH} = 0.396$$

2. Calculation of apparent earth pressure (Refer to Figure 2.2.2 (a). Units = KN/m^2)

$H_{\text{wall}} := 7$	Height of wall
$H_1 := 2$	Distance to anchor
$K_A := 0.396$	
$\gamma := 16$	
$\text{Load} := 0.65 K_A \cdot \gamma \cdot H_{\text{wall}}^2$	Load = 201.802
$p := \frac{\text{Load}}{0.667 H_{\text{wall}}}$	$p = 43.222$

3. Calculation of anchor design load and reaction force required at base of wall.

$$L_a := 2 \cdot \frac{H_1}{3} \quad L_a = 1.333$$

$$L_b := \frac{H_{\text{wall}}}{3} \quad L_b = 2.333$$

$$L_c := 2 \cdot \frac{H_{\text{wall}} - H_1}{3} \quad L_c = 3.333 \quad L_{\text{check}} := L_a + L_b + L_c \quad L_{\text{check}} = 7$$

$$M_R := p \cdot \left[\frac{L_c^2}{3} + L_b \cdot \left(\frac{L_b}{2} + L_c \right) + \frac{L_a}{2} \cdot \left(\frac{L_a}{3} + L_b + L_c \right) \right] \quad M_R = 789.995$$

$$T_{\text{anchor}} := \frac{M_R}{H_{\text{wall}} - H_1} \quad T_{\text{anchor}} = 157.999$$

$$R_{\text{base}} := \text{Load} - T_{\text{anchor}} \quad R_{\text{base}} = 43.803$$

4. Calculation of cantilever moment in wall element above anchor

Cantilever moment

$$L_{a2} := \frac{H_1}{3} \quad L_{a2} = 0.667$$

$$M_c := p \cdot \left[\frac{L_a}{2} \cdot \left(\frac{L_a}{3} + L_{a2} \right) + \frac{L_{a2}^2}{2} \right] \quad M_c = 41.621$$

5. Calculation of maximum bending moment in wall element below anchor

$$V_{La} := p \cdot \frac{L_a}{2} \quad V_{La} = 28.814$$

$$V_{H1} := V_{La} + p \cdot (H_1 - L_a) \quad V_{H1} = 57.629$$

$$Z_{\text{vzero}} := \frac{T_{\text{anchor}} - V_{H1}}{p} + L_a \quad Z_{\text{vzero}} = 3.656$$

$$\text{Must be less than} \quad Z_{\text{max}} := H_{\text{wall}} - L_c \quad Z_{\text{max}} = 3.667$$

$$M_{\text{max}} := \frac{p \cdot L_a}{2} \cdot \left(Z_{\text{vzero}} - 2 \cdot \frac{L_a}{3} \right) + \frac{p \cdot (Z_{\text{vzero}} - L_a)^2}{2} - T_{\text{anchor}} \cdot (Z_{\text{vzero}} - H_1)$$

$$M_{\text{max}} = -65.315$$

$$M_{\text{star}} := M_{\text{max}} \cdot 1.6 \quad M_{\text{star}} = -104.504$$

M^* = ULS design bending moment

6. Selection of wall structural element

Keeping the same pile spacing as the gravity design of 2 m, the ULS design moment $M^* = 209$ KNm per each. For example, a 250UC72.9 steel column section would suffice.

For the purpose of this study, it was assumed that either, such a steel column was set into a concrete filled 450 mm diameter hole, or, a reinforced concrete soldier pile of the same diameter was used.

7. Calculation of embedment depth for soldier piles

Simple Broms theory is used, with K_p calculated using Coulomb theory. Coulomb theory is used to be compatible with M-O theory for later earthquake design case. Interface friction is limited to $\phi/2$ because Coulomb theory is unconservative at higher levels of interface friction, the resulting value for K_p in this case is quite close to the value given by the NAVFAC charts which are based on log-spiral theory.

$B := 0.45$	Pile diameter	
$D := 2.8$	Depth of embedment of pile	
$\gamma := 8$	Soil unit weight (buoyant)	
$\phi := 35 \cdot \text{deg}$		
$k_h := 0.2$	horizontal acceleration in g	$\theta := \text{atan}(k_h) \quad \theta = 11.31 \text{deg}$
$\beta := 0 \cdot \text{deg}$	slope of the back of the wall	
$i := 0 \cdot \text{deg}$	slope of the backfill	
$\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$	angle of interface friction (passive)	

Passive K_p

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_p = 0.148$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p} \quad K_{PE} = 6.727$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 6.349$$

Ultimate Horizontal Resistance

$$H_u := \frac{3}{2} \cdot \gamma \cdot B \cdot K_{PEH} D^2 \quad H_u = 268.809$$

A strength reduction factor of 3 is recommended to be applied to the ultimate horizontal resistance calculated using Broms' theory because of the large plastic strains required to mobilise full passive resistance. Therefore, for piles spaced at 2 m centres:

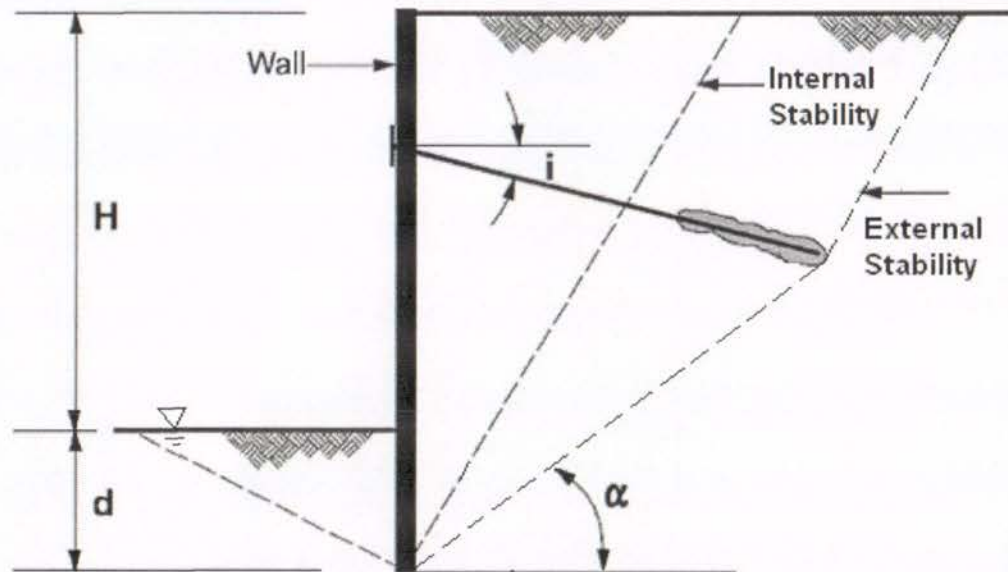
$$\text{Design Resistance} = 269/3 = 90 \text{ KN each pile or } 45 \text{ KN/m run}$$

$$\text{Design Demand } (R_{\text{base}}) = 44 \text{ KN/m}$$

Therefore, embedment depth of 2.8 m is optimum.

8. Check for internal stability

(True FS determined by successive reduction of ϕ)



$k_h := 0.2$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 11.31\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 29.2\text{-deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := \delta_i$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.228$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.465$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.448$$

Failure surface inclination

$$\begin{aligned} aa &:= \phi - i - \theta & bb &:= \phi - \beta - \theta & cc &:= \delta_a + \beta + \theta \\ aa &= 17.89\text{deg} & bb &= 17.89\text{deg} & cc &= 26.922\text{deg} \end{aligned}$$

$$\rho_A := \phi - \theta + \text{atan} \left[\frac{\sqrt{\tan(aa) \cdot (\tan(aa) + \cot(bb)) \cdot (1 + \tan(cc) \cdot \cot(bb))} - \tan(aa)}{1 + \tan(cc) \cdot (\tan(aa) + \cot(bb))} \right]$$

$$\rho_A = 44.359\text{deg}$$

Passive Kp

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_P = 0.257$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P} \quad K_{PE} = 4.026$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 3.878$$

Wedge Calculation

Single anchor, water at base of excavation

$$\begin{aligned} H_{exc} &:= 7 && \text{Depth of excavation} \\ H_{embed} &:= 2.8 && \text{Embedment of piles} \\ F_H &:= 158.133 && \text{Anchor horizontal force (ultimate)} \quad F_H = 210.14 \\ \gamma_{above} &:= 16 \\ \gamma_{below} &:= 8 \end{aligned}$$

$$P_A := K_{AEH} \left(0.5 \gamma_{above} \cdot H_{exc}^2 + \gamma_{above} \cdot H_{exc} \cdot H_{embed} + 0.5 \gamma_{below} \cdot H_{embed}^2 \right) \quad P_A = 329.929$$

$$P_P := K_{PEH} 0.5 \gamma_{below} \cdot H_{embed}^2 \quad P_P = 121.599$$

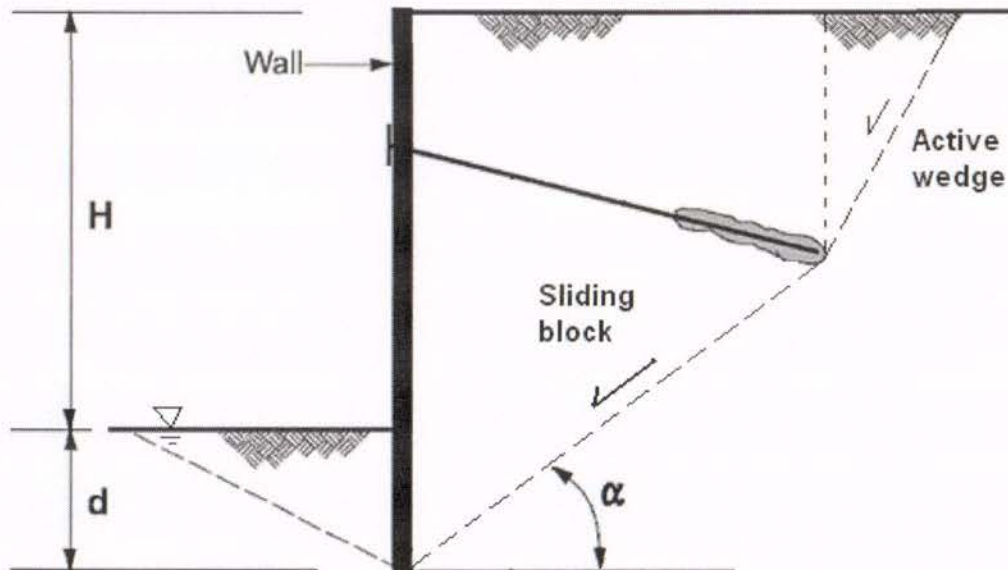
Stability calculation

$$H_{net} := P_A - P_P - F_H \quad H_{net} = -1.81 \quad < 0 \text{ for stability}$$

FS Calculation (by trial and error to set H = 0)

$$\phi_{design} := 35\text{-deg} \quad FS := \frac{\tan(\phi_{design})}{\tan(\phi)} \quad FS = 1.253$$

9. Check for external stability



$k_h := 0.2$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 11.31\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 31.1\text{-deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := \delta_i$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.352$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta)} \cdot D \quad K_{AE} = 0.435$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.417 \quad \text{Equivalent Horizontal Component}$$

Sliding block details

$B := 12.3$ Breadth of block
 $\alpha := 20\text{-deg}$ Failure surface
 $H_{exc} := 7$ Depth of excavation
 $H_{embed} := 2.1$ Embedment of piles
 $\gamma_{above} := 16$
 $\gamma_{below} := 8$

$$W_{\text{block}} := \gamma_{\text{above}} \left[B \cdot (H_{\text{exc}} + H_{\text{embed}}) - 0.5 \tan(\alpha) \cdot B^2 \right] \quad W_{\text{block}} = 1.35 \times 10^3$$

$$W_{\text{buoy}} := W_{\text{block}} - (\gamma_{\text{above}} - \gamma_{\text{below}}) \cdot 0.5 \frac{H_{\text{embed}}^2}{\tan(\alpha)} \quad W_{\text{buoy}} = 1.302 \times 10^3$$

correction for water
assumes active
wedge is dry

$$H_{\text{block}} := W_{\text{block}} \cdot \left(\frac{\sin(\alpha) - \tan(\phi) \cdot \cos(\alpha)}{\cos(\alpha) + \tan(\phi) \cdot \sin(\alpha)} + k_h \right) \quad H_{\text{block}} = 5.142$$

This is the net contribution to horizontal movement

Active pressure wedge (zero interface friction)

$$H_{\text{active}} := H_{\text{exc}} + H_{\text{embed}} - B \cdot \tan(\alpha)$$

$$P_{\text{ah}} := 0.5 \cdot \gamma_{\text{above}} \cdot K_{\text{AEH}} \cdot H_{\text{active}}^2$$

Passive Resistance

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{\text{PE}} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta)} \cdot D_P$$

Equivalent Horizontal Component

$$K_{\text{PEH}} := \cos(\delta_i + \beta) \cdot K_{\text{PE}} \quad K_{\text{PEH}} = 4.504$$

$$P_{\text{ph}} := 0.5 \cdot \gamma_{\text{below}} \cdot K_{\text{PEH}} \cdot H_{\text{embed}}^2 \quad P_{\text{ph}} = 79.452$$

Wedge-block Stability Calculation

$$H_{\text{net}} := P_{\text{ah}} + H_{\text{block}} - P_{\text{ph}} \quad H_{\text{net}} = -3.067 \quad < 0 \text{ for stability}$$

FS Calculation (set H = 0 by trial and error)

$$\phi_{\text{design}} := 35 \cdot \text{deg} \quad \text{FS} := \frac{\tan(\phi_{\text{design}})}{\tan(\phi)} \quad \text{FS} = 1.161$$

A.4 Design calculations for case study Sand 1d – M-O based design

0.3 g

1. Calculation of K_a

Mononobe-Okabe Theory

$k_h := 0.3$	horizontal acceleration in g	$\theta := \text{atan}(k_h)$	$\theta = 0.291$
$\beta := 0\text{-deg}$	slope of the back of the wall		
$i := 0\text{-deg}$	slope of the backfill		
$\phi := 35\text{-deg}$	angle of internal friction		
$\delta_i := 0$	angle of interface friction		

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi - \theta - i)}{\cos(\delta_i + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.055$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_i + \theta) \cdot D} \quad K_{AE} = 0.478$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_i) K_{AE} \quad K_{AEH} = 0.478$$

2. Calculation of apparent earth pressure (Refer to Figure 2.2.2 (a). Units = KN/m²)

$H_{\text{wall}} := 7$	Height of wall
$H_1 := 2$	Distance to anchor
$K_A := 0.478$	
$\gamma := 16$	
$\text{Load} := 0.65 K_A \cdot \gamma \cdot H_{\text{wall}}^2$	Load = 243.589
$p := \frac{\text{Load}}{0.667 H_{\text{wall}}}$	$p = 52.172$

3. Calculation of anchor design load and reaction force required at base of wall.

$$L_a := 2 \cdot \frac{H_1}{3} \quad L_a = 1.333$$

$$L_b := \frac{H_{\text{wall}}}{3} \quad L_b = 2.333$$

$$L_c := 2 \cdot \frac{H_{\text{wall}} - H_1}{3} \quad L_c = 3.333 \quad L_{\text{check}} := L_a + L_b + L_c \quad L_{\text{check}} = 7$$

$$M_R := p \cdot \left[\frac{L_c^2}{3} + L_b \cdot \left(\frac{L_b}{2} + L_c \right) + \frac{L_a}{2} \cdot \left(\frac{L_a}{3} + L_b + L_c \right) \right] \quad M_R = 953.579$$

$$T_{\text{anchor}} := \frac{M_R}{H_{\text{wall}} - H_1} \quad T_{\text{anchor}} = 190.716$$

$$R_{\text{base}} := \text{Load} - T_{\text{anchor}} \quad R_{\text{base}} = 52.873$$

4. Calculation of cantilever moment in wall element above anchor

$$L_{a2} := \frac{H_1}{3} \quad L_{a2} = 0.667$$

$$M_c := p \cdot \left[\frac{L_a}{2} \cdot \left(\frac{L_a}{3} + L_{a2} \right) + \frac{L_{a2}^2}{2} \right] \quad M_c = 50.239$$

5. Calculation of maximum bending moment in wall element below anchor

$$V_{La} := p \cdot \frac{L_a}{2} \quad V_{La} = 34.781$$

$$V_{H1} := V_{La} + p \cdot (H_1 - L_a) \quad V_{H1} = 69.562$$

$$Z_{\text{vzero}} := \frac{T_{\text{anchor}} - V_{H1}}{p} + L_a \quad Z_{\text{vzero}} = 3.656$$

$$\text{Must be less than} \quad Z_{\text{max}} := H_{\text{wall}} - L_c \quad Z_{\text{max}} = 3.667$$

$$M_{\text{max}} := \frac{p \cdot L_a}{2} \cdot \left(Z_{\text{vzero}} - 2 \cdot \frac{L_a}{3} \right) + \frac{p \cdot (Z_{\text{vzero}} - L_a)^2}{2} - T_{\text{anchor}} \cdot (Z_{\text{vzero}} - H_1)$$

$$M_{\text{max}} = -78.84$$

$$M_{\text{star}} := M_{\text{max}} \cdot 1.6 \quad M_{\text{star}} = -126.144$$

M^* = ULS design bending moment

6. Selection of wall structural element

Keeping the same pile spacing as the gravity design of 2 m, the ULS design moment $M^* = 252$ kNm per each. For example, a 250UC72.9 steel column section would suffice.

For the purpose of this study, it was assumed that either, such a steel column was set into a concrete filled 450 mm diameter hole, or, a reinforced concrete soldier pile of the same diameter was used.

7. Calculation of embedment depth for soldier piles

Simple Broms theory is used, with K_p calculated using Coulomb theory. Coulomb theory is used to be compatible with M-O theory for later earthquake design case. Interface friction is limited to $\phi/2$ because Coulomb theory is unconservative at higher levels of interface friction, the resulting value for K_p in this case is quite close to the value given by the NAVFAC charts which are based on log-spiral theory.

$B := 0.45$	Pile diameter	
$D := 3.3$	Depth of embedment of pile	
$\gamma := 8$	Soil unit weight (buoyant)	
$\phi := 35\text{-deg}$		
$k_h := 0.3$	horizontal acceleration in g	$\theta := \text{atan}(k_h) \quad \theta = 16.699\text{deg}$
$\beta := 0\text{-deg}$	slope of the back of the wall	
$i := 0\text{-deg}$	slope of the backfill	
$\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$	angle of interface friction (passive)	

Passive K_p

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_p = 0.192$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p} \quad K_{PE} = 6.046$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 5.707$$

Ultimate Horizontal Resistance

$$H_u := \frac{3}{2} \cdot \gamma \cdot B \cdot K_{PEH} D^2 \quad H_u = 335.579$$

A strength reduction factor of 3 is recommended to be applied to the ultimate horizontal resistance calculated using Broms' theory because of the large plastic strains required to mobilise full passive resistance. Therefore, for piles spaced at 2 m centres:

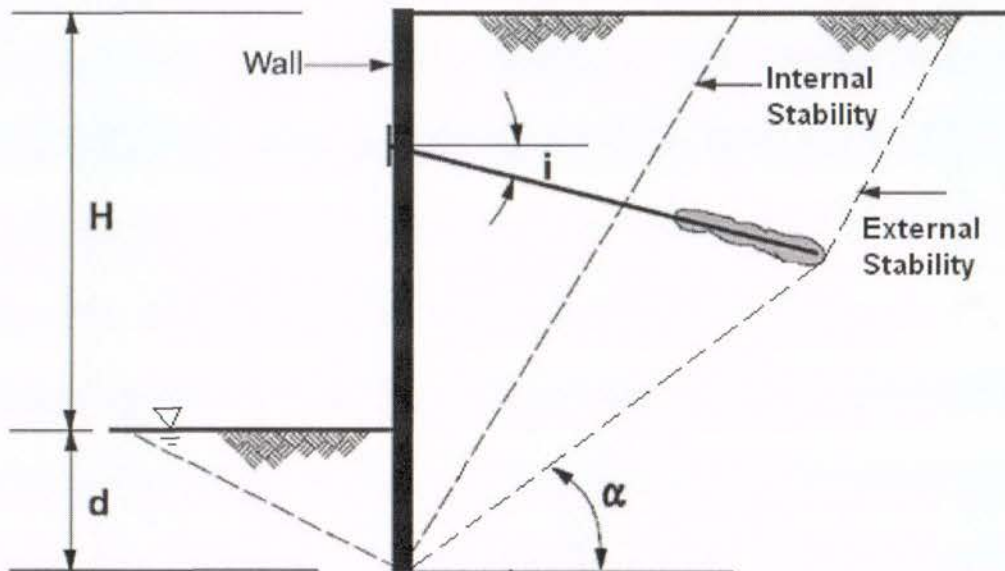
$$\text{Design Resistance} = 336/3 = 112 \text{ KN each pile or } 56 \text{ KN/m run}$$

$$\text{Design Demand } (R_{\text{base}}) = 53 \text{ KN/m}$$

Therefore, embedment depth of 3.3 m is close to optimum.

8. Check for internal stability

(True FS determined by successive reduction of ϕ)



$k_h := 0.3$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 16.699\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 30.8\text{deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := \delta_i$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.141$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.549$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.526$$

$$\begin{aligned} aa &:= \phi - i - \theta & bb &:= \phi - \beta - \theta & cc &:= \delta_a + \beta + \theta \\ aa &= 14.101\text{deg} & bb &= 14.101\text{deg} & cc &= 33.296\text{deg} \end{aligned}$$

$$\rho_A := \phi - \theta + \text{atan} \left[\frac{\sqrt{\tan(aa) \cdot (\tan(aa) + \cot(bb)) \cdot (1 + \tan(cc) \cdot \cot(bb))} - \tan(aa)}{1 + \tan(cc) \cdot (\tan(aa) + \cot(bb))} \right]$$

$$\rho_A = 38.433\text{deg}$$

Passive Kp

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_P = 0.288$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P} \quad K_{PE} = 4.077$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 3.907$$

Wedge Calculation

Single anchor, water at base of excavation

$$\begin{aligned} H_{exc} &:= 7 && \text{Depth of excavation} \\ H_{embed} &:= 3.3 && \text{Embedment of piles} \\ F_H &:= 191.133 && \text{Anchor horizontal force (ultimate)} \quad F_H = 254.03 \\ \gamma_{above} &:= 16 \\ \gamma_{below} &:= 8 \end{aligned}$$

$$P_A := K_{AEH} \left(0.5 \gamma_{above} \cdot H_{exc}^2 + \gamma_{above} \cdot H_{exc} \cdot H_{embed} + 0.5 \gamma_{below} \cdot H_{embed}^2 \right) \quad P_A = 423.462$$

$$P_P := K_{PEH} 0.5 \gamma_{below} \cdot H_{embed}^2 \quad P_P = 170.208$$

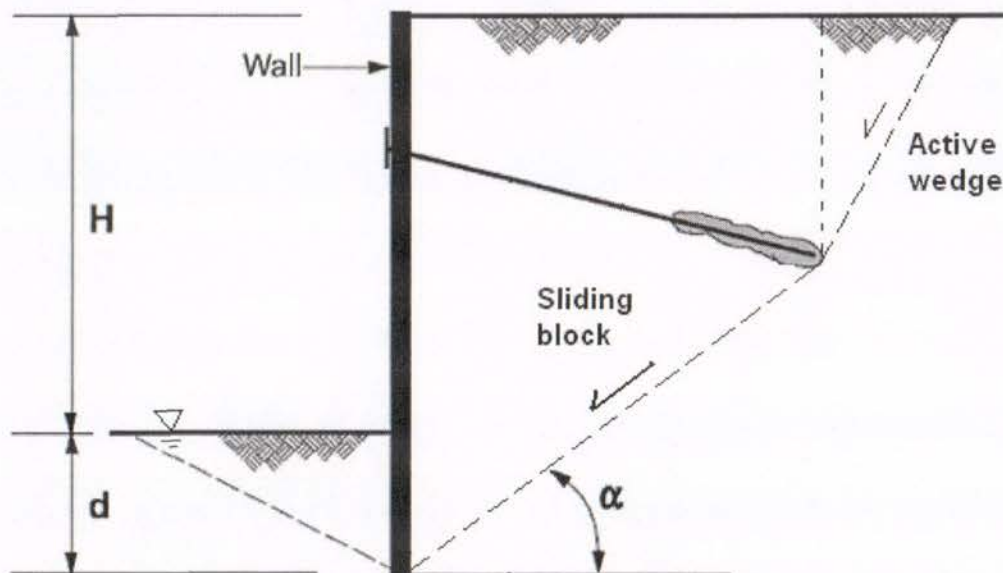
Stability calculation

$$H_{net} := P_A - P_P - F_H \quad H_{net} = -0.776 < 0 \text{ for stability}$$

FS Calculation (by trial and error to set H = 0)

$$\phi_{design} := 35\text{-deg} \quad FS := \frac{\tan(\phi_{design})}{\tan(\phi)} \quad FS = 1.175$$

9. Check for external stability



$k_h := 0.3$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 16.699\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 32.1\text{-deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := \delta_i$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.232$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.525$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.501 \quad \text{Equivalent Horizontal Component}$$

$B := 14.6$ Breadth of block
 $\alpha := 16.7\text{-deg}$ Failure surface
 $H_{exc} := 7$ Depth of excavation
 $H_{embed} := 3.3$ Embedment of piles
 $\gamma_{above} := 16$
 $\gamma_{below} := 8$

$$W_{\text{block}} := \gamma_{\text{above}} \cdot \left[B \cdot (H_{\text{exc}} + H_{\text{embed}}) - 0.5 \tan(\alpha) \cdot B^2 \right] \quad W_{\text{block}} = 1.894 \times 10^3$$

$$W_{\text{buoy}} := W_{\text{block}} - (\gamma_{\text{above}} - \gamma_{\text{below}}) \cdot 0.5 \frac{H_{\text{embed}}^2}{\tan(\alpha)} \quad W_{\text{buoy}} = 1.749 \times 10^3$$

correction for water
assumes active
wedge is dry

$$H_{\text{block}} := W_{\text{block}} \cdot \left(\frac{\sin(\alpha) - \tan(\phi) \cdot \cos(\alpha)}{\cos(\alpha) + \tan(\phi) \cdot \sin(\alpha)} + k_h \right) \quad H_{\text{block}} = 46.517$$

This is the net contribution to horizontal movement

Active pressure wedge (zero interface friction)

$$H_{\text{active}} := H_{\text{exc}} + H_{\text{embed}} - B \cdot \tan(\alpha)$$

$$P_{\text{ah}} := 0.5 \gamma_{\text{above}} \cdot K_{\text{AEH}} \cdot H_{\text{active}}^2$$

Passive Resistance

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{\text{PE}} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p}$$

Equivalent Horizontal Component

$$K_{\text{PEH}} := \cos(\delta_i + \beta) K_{\text{PE}} \quad K_{\text{PEH}} = 4.367$$

$$P_{\text{ph}} := 0.5 \gamma_{\text{below}} \cdot K_{\text{PEH}} \cdot H_{\text{embed}}^2 \quad P_{\text{ph}} = 190.211$$

Wedge-block Stability Calculation

$$H_{\text{net}} := P_{\text{ah}} + H_{\text{block}} - P_{\text{ph}} \quad H_{\text{net}} = -3.21 \quad < 0 \text{ for stability}$$

FS Calculation (set $H = 0$ by trial and error)

$$\phi_{\text{design}} := 35 \text{ deg} \quad \text{FS} := \frac{\tan(\phi_{\text{design}})}{\tan(\phi)} \quad \text{FS} = 1.116$$

A.5 Design calculations for case study Sand 1e – M-O based design

0.4 g

1. Calculation of K_a

Mononobe-Okabe Theory

$$\begin{aligned}
 k_h &:= 0.4 && \text{horizontal acceleration in g} && \theta := \text{atan}(k_h) && \theta = 0.381 \\
 \beta &:= 0 \cdot \text{deg} && \text{slope of the back of the wall} \\
 i &:= 0 \cdot \text{deg} && \text{slope of the backfill} \\
 \phi &:= 35 \cdot \text{deg} && \text{angle of internal friction} \\
 \delta_i &:= 0 && \text{angle of interface friction} && \delta_a := \delta_i
 \end{aligned}$$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi - \theta - i)}{\cos(\delta_i + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 1.892$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_i + \theta) \cdot D} \quad K_{AE} = 0.581$$

2. Calculation of apparent earth pressure (Refer to Figure 2.2.2 (a). Units = KN/m^2)

$$\begin{aligned}
 H_{\text{wall}} &:= 7 && \text{Height of wall} \\
 H_1 &:= 2 && \text{Distance to anchor} \\
 K_A &:= 0.581 \\
 \gamma &:= 16 \\
 \text{Load} &:= 0.65 K_A \cdot \gamma \cdot H_{\text{wall}}^2 && \text{Load} = 296.078 \\
 p &:= \frac{\text{Load}}{0.667 H_{\text{wall}}} && p = 63.413
 \end{aligned}$$

3. Calculation of anchor design load and reaction force required at base of wall.

Moments about base

$$\begin{aligned}
 L_a &:= 2 \cdot \frac{H_1}{3} && L_a = 1.333 \\
 L_b &:= \frac{H_{\text{wall}}}{3} && L_b = 2.333 \\
 L_c &:= 2 \cdot \frac{H_{\text{wall}} - H_1}{3} && L_c = 3.333 && L_{\text{check}} := L_a + L_b + L_c && L_{\text{check}} = 7
 \end{aligned}$$

$$M_R := p \cdot \left[\frac{L_c^2}{3} + L_b \cdot \left(\frac{L_b}{2} + L_c \right) + \frac{L_a}{2} \cdot \left(\frac{L_a}{3} + L_b + L_c \right) \right] \quad M_R = 1.159 \times 10^3$$

$$T_{\text{anchor}} := \frac{M_R}{H_{\text{wall}} - H_1} \quad T_{\text{anchor}} = 231.812$$

$$R_{\text{base}} := \text{Load} - T_{\text{anchor}} \quad R_{\text{base}} = 64.266$$

4. Calculation of cantilever moment in wall element above anchor

$$L_{a2} := \frac{H_1}{3} \quad L_{a2} = 0.667$$

$$M_c := p \cdot \left[\frac{L_a}{2} \cdot \left(\frac{L_a}{3} + L_{a2} \right) + \frac{L_{a2}^2}{2} \right] \quad M_c = 61.065$$

5. Calculation of maximum bending moment in wall element below anchor

Maximum moment at zero shear

$$V_{La} := p \cdot \frac{L_a}{2} \quad V_{La} = 42.276$$

$$V_{H1} := V_{La} + p \cdot (H_1 - L_a) \quad V_{H1} = 84.551$$

$$Z_{\text{vzero}} := \frac{T_{\text{anchor}} - V_{H1}}{p} + L_a \quad Z_{\text{vzero}} = 3.656$$

$$\text{Must be less than} \quad Z_{\text{max}} := H_{\text{wall}} - L_c \quad Z_{\text{max}} = 3.667$$

$$M_{\text{max}} := \frac{p \cdot L_a}{2} \cdot \left(Z_{\text{vzero}} - 2 \cdot \frac{L_a}{3} \right) + \frac{p \cdot (Z_{\text{vzero}} - L_a)^2}{2} - T_{\text{anchor}} \cdot (Z_{\text{vzero}} - H_1)$$

$$M_{\text{max}} = -95.829$$

$$M_{\text{star}} := M_{\text{max}} \cdot 1.6 \quad M_{\text{star}} = -153.326$$

M^* = ULS design bending moment

6. Selection of wall structural element

Keeping the same pile spacing as the gravity design of 2 m, the ULS design moment $M^* = 306$ KNm per each. For example, a 250UC89.5 steel column section would suffice.

For the purpose of this study, it was assumed that either, such a steel column was set into a concrete filled 450 mm diameter hole, or, a reinforced concrete soldier pile of the same diameter was used.

7. Calculation of embedment depth for soldier piles

Simple Broms theory is used, with K_p calculated using Coulomb theory. Coulomb theory is used to be compatible with M-O theory for later earthquake design case. Interface friction is limited to $\phi/2$ because Coulomb theory is unconservative at higher levels of interface friction, the resulting value for K_p in this case is quite close to the value given by the NAVFAC charts which are based on log-spiral theory.

$B := 0.45$	Pile diameter	
$D := 3.8$	Depth of embedment of pile	
$\gamma := 8$	Soil unit weight (buoyant)	
$\phi := 35\text{-deg}$		
$k_h := 0.4$	horizontal acceleration in g	$\theta := \text{atan}(k_h) \quad \theta = 21.801\text{deg}$
$\beta := 0\text{-deg}$	slope of the back of the wall	
$i := 0\text{-deg}$	slope of the backfill	
$\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$	angle of interface friction (passive)	

Passive K_p

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_p = 0.254$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p} \quad K_{PE} = 5.333$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 5.034$$

Ultimate Horizontal Resistance

$$H_u := \frac{3}{2} \cdot \gamma \cdot B \cdot K_{PEH} D^2 \quad H_u = 392.514$$

A strength reduction factor of 3 is recommended to be applied to the ultimate horizontal resistance calculated using Broms' theory because of the large plastic strains required to mobilise full passive resistance. Therefore, for piles spaced at 2 m centres:

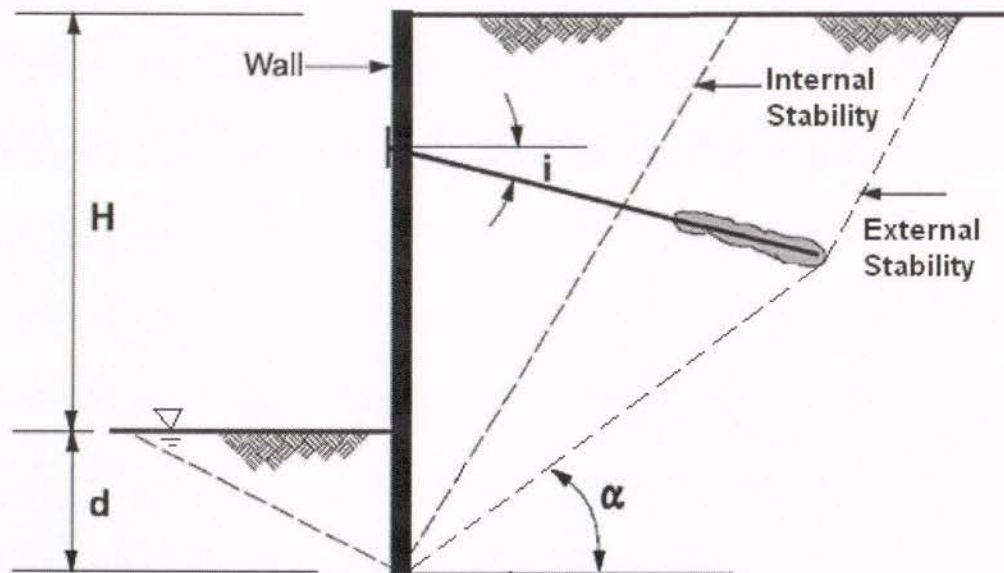
$$\text{Design Resistance} = 392/3 = 130 \text{ KN each pile or } 65 \text{ KN/m run}$$

$$\text{Design Demand } (R_{\text{base}}) = 64 \text{ KN/m}$$

Therefore, embedment depth of 3.8 m is close to optimum.

8. Check for internal stability

(True FS determined by successive reduction of ϕ)



$k_h := 0.4$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 21.801\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 32.5\text{-deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 1.763$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.635$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.635$$

Failure surface inclination

$$aa := \phi - i - \theta \quad bb := \phi - \beta - \theta \quad cc := \delta_a + \beta + \theta$$

$$aa = 10.699\text{deg} \quad bb = 10.699\text{deg} \quad cc = 21.801\text{deg}$$

$$\rho_A := \phi - \theta + \text{atan} \left[\frac{\sqrt{\tan(aa) \cdot (\tan(aa) + \cot(bb)) \cdot (1 + \tan(cc) \cdot \cot(bb))} - \tan(aa)}{1 + \tan(cc) \cdot (\tan(aa) + \cot(bb))} \right]$$

$$\rho_A = 37.428\text{deg}$$

Passive Kp

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_P = 0.325$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P} \quad K_{PE} = 4.142$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 3.947$$

Wedge Calculation

Single anchor, water at base of excavation

$$H_{exc} := 7 \quad \text{Depth of excavation}$$

$$H_{embed} := 4.34 \quad \text{Embedment of piles}$$

$$F_H := 232.133 \quad \text{Anchor horizontal force (ultimate)} \quad F_H = 308.56$$

$$\gamma_{above} := 16$$

$$\gamma_{below} := 8$$

$$P_A := K_{AEH} \left(0.5 \cdot \gamma_{above} \cdot H_{exc}^2 + \gamma_{above} \cdot H_{exc} \cdot H_{embed} + 0.5 \gamma_{below} \cdot H_{embed}^2 \right) \quad P_A = 605.718$$

$$P_P := K_{PEH} 0.5 \gamma_{below} \cdot H_{embed}^2 \quad P_P = 297.377$$

Stability calculation

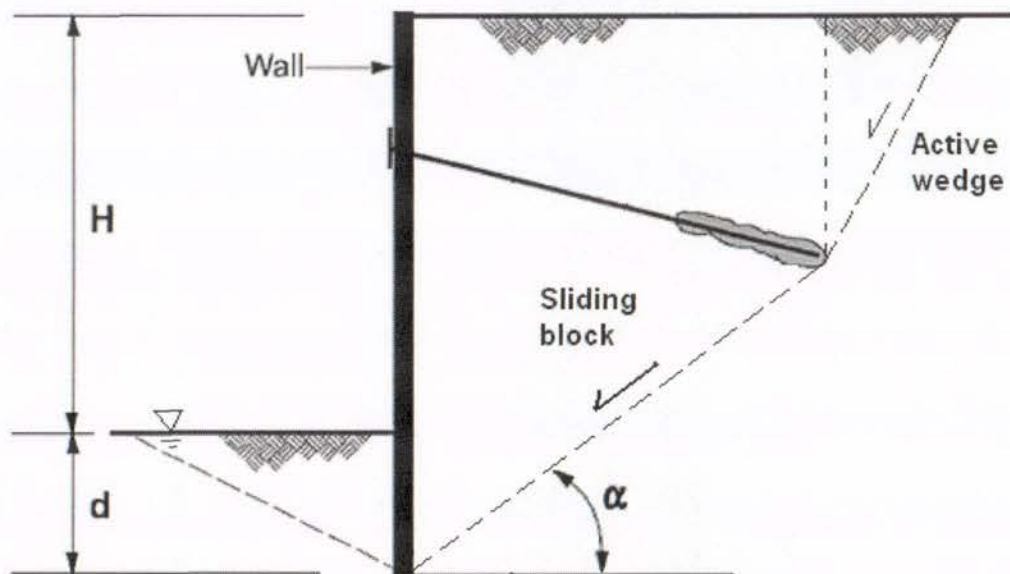
$$H_{net} := P_A - P_P - F_H \quad H_{net} = -0.22 \quad < 0 \text{ for stability}$$

FS Calculation (by trial and error to set $H = 0$)

$$\phi_{design} := 35\text{-deg} \quad FS := \frac{\tan(\phi_{design})}{\tan(\phi)} \quad FS = 1.099$$

In this case the initial calculation gave a low factor of safety, less than 1.1. Therefore, the depth of embedment of the wall was increased by trial and error to 4.3 m to improve the factor of safety.

9. Check for external stability



$k_h := 0.4$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 21.801\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 35.1\text{-deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 1.897$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.579$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.579 \quad \text{Equivalent Horizontal Component}$$

Sliding block details

$B := 15.63$ Breadth of block
 $\alpha := 17.7\text{-deg}$ Failure surface
 $H_{exc} := 12$ Depth of excavation
 $H_{embed} := 4.3$ Embedment of piles
 $\gamma_{above} := 16$
 $\gamma_{below} := 8$

$$W_{\text{block}} := \gamma_{\text{above}} \cdot \left[B \cdot (H_{\text{exc}} + H_{\text{embed}}) - 0.5 \tan(\alpha) \cdot B^2 \right] \quad W_{\text{block}} = 3.453 \times 10^3$$

$$W_{\text{buoy}} := W_{\text{block}} - (\gamma_{\text{above}} - \gamma_{\text{below}}) \cdot 0.5 \frac{H_{\text{embed}}^2}{\tan(\alpha)} \quad W_{\text{buoy}} = 3.221 \times 10^3$$

correction for water
assumes active
wedge is dry

$$H_{\text{block}} := W_{\text{block}} \cdot \left(\frac{\sin(\alpha) - \tan(\phi) \cdot \cos(\alpha)}{\cos(\alpha) + \tan(\phi) \cdot \sin(\alpha)} + k_h \right) \quad H_{\text{block}} = 299.059$$

This is the net contribution to horizontal movement

Active pressure wedge (zero interface friction)

$$H_{\text{active}} := H_{\text{exc}} + H_{\text{embed}} - B \cdot \tan(\alpha)$$

$$P_{\text{ah}} := 0.5 \cdot \gamma_{\text{above}} \cdot K_{\text{AEH}} \cdot H_{\text{active}}^2$$

Passive Resistance

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{\text{PE}} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P}$$

Equivalent Horizontal Component

$$K_{\text{PEH}} := \cos(\delta_i + \beta) \cdot K_{\text{PE}} \quad K_{\text{PEH}} = 5.085$$

$$P_{\text{ph}} := 0.5 \cdot \gamma_{\text{below}} \cdot K_{\text{PEH}} \cdot H_{\text{embed}}^2 \quad P_{\text{ph}} = 376.107$$

Wedge-block Stability Calculation

$$H_{\text{net}} := P_{\text{ah}} + H_{\text{block}} - P_{\text{ph}} \quad H_{\text{net}} = 515.692 \quad < 0 \text{ for stability}$$

FS Calculation (set H = 0 by trial and error)

$$\phi_{\text{design}} := 35 \text{ deg} \quad \text{FS} := \frac{\tan(\phi_{\text{design}})}{\tan(\phi)} \quad \text{FS} = 0.996$$

Appendix B

B.1 Design calculations for case study Sand 2a – Gravity based design

1. Calculation of K_a

$k_h := 0$	horizontal acceleration in g	$\theta := \text{atan}(k_h)$	$\theta = 0$
$\beta := 0\text{-deg}$	slope of the back of the wall		
$i := 0\text{-deg}$	slope of the backfill		
$\phi := 35\text{-deg}$	angle of internal friction		
$\delta_i := 0$	angle of interface friction		

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi - \theta - i)}{\cos(\delta_i + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.476$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_i + \theta) \cdot D} \quad K_{AE} = 0.271$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_i) K_{AE} \quad K_{AEH} = 0.271$$

2. Calculation of apparent earth pressure (Refer to Figure 2.2.2 (a). Units = KN/m^2)

$H_{\text{wall}} := 12$	Height of wall	
$H_1 := 3$	$H_2 := 5$	Distance to anchors
$K_A := 0.27$		
$\gamma := 16$		
$\text{Load} := 0.65 K_A \cdot \gamma \cdot H_{\text{wall}}^2$	Load = 404.352	
$H_3 := H_{\text{wall}} - H_1 - H_2$	$H_3 = 4$	
$p := \frac{\text{Load}}{H_{\text{wall}} - \frac{H_1}{3} - \frac{H_3}{3}}$	$p = 41.83$	Ref Fig 2.2.2 (a)

3. Calculation of anchor design loads and reaction force required at base of wall.

Anchor forces

$$T_1 := p \cdot \left(\frac{H_1}{3} + \frac{H_1}{3} + \frac{H_2}{2} \right) \quad T_1 = 188.233$$

$$R_{base} := \frac{1}{2} \cdot \frac{3}{4} \cdot p \cdot \frac{H_3}{2} \quad R_{base} = 31.372$$

$$T_2 := Load - T_1 - R_{base} \quad T_2 = 184.747$$

Cantilever moment

$$M_c := \frac{13}{54} \cdot p \cdot H_1^2 \quad M_c = 90.631$$

Maximum moment between 2 anchors

$$Z_{12} := \frac{T_1}{p} - \frac{2}{3} \cdot H_1 \quad Z_{12} = 2.5$$

$$M_{12} := p \cdot \left[\frac{1}{3} \cdot H_1 \cdot \left(\frac{13}{18} \cdot H_1 + 2Z_{12} \right) + \frac{1}{2} Z_{12}^2 \right] - T_1 \cdot Z_{12} \quad M_{12} = -40.087$$

Moment at anchor 2

$$M_2 := p \cdot \left[\frac{1}{3} \cdot H_1 \cdot \left(\frac{13}{18} \cdot H_1 + 2H_2 \right) + \frac{1}{2} H_2^2 \right] - T_1 \cdot H_2 \quad M_2 = 90.631$$

Other cases non critical - but need checking in PLAXIS

$$M_{star} := M_2 \cdot 1.6 \quad M_{star} = 145.009$$

M^* = ULS design bending moment

6. Selection of wall structural element

A typical spacing for a soldier pile wall is 2 m crs. Therefore ULS design moment $M^* = 290$ KNm per each. For example, 94 percent of a 250UC89.5 steel column section would suffice.

For the purpose of this study, it was assumed that either, such a steel column was set into a concrete filled 450 mm diameter hole, or, a reinforced concrete soldier pile of the same diameter was used.

7. Calculation of embedment depth for soldier piles

Simple Broms theory is used, with K_p calculated using Coulomb theory. Coulomb theory is used to be compatible with M-O theory for later earthquake design case. Interface friction is limited to $\phi/2$ because Coulomb theory is unconservative at higher levels of interface friction, the resulting value for K_p in this case is quite close to the value given by the NAVFAC charts which are based on log-spiral theory.

$B := 0.45$	Pile diameter	
$D := 2.2$	Depth of embedment of pile	
$\gamma := 8$	Soil unit weight (buoyant)	
$\phi := 35\text{-deg}$		
$k_h := 0.0$	horizontal acceleration in g	$\theta := \text{atan}(k_h) \quad \theta = 0\text{ deg}$
$\beta := 0\text{-deg}$	slope of the back of the wall	
$i := 0\text{-deg}$	slope of the backfill	
$\delta_i := \text{atan}(0.5 \tan(\phi))$	angle of interface friction (passive)	

Passive K_p

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_p = 0.089$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p} \quad K_{PE} = 8.032$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 7.581$$

Ultimate Horizontal Resistance

$$H_u := \frac{3}{2} \cdot \gamma \cdot B \cdot K_{PEH} D^2 \quad H_u = 198.13$$

A strength reduction factor of 3 is recommended to be applied to the ultimate horizontal resistance calculated using Broms' theory because of the large plastic strains required to mobilise full passive resistance. Therefore, for piles spaced at 2 m centres:

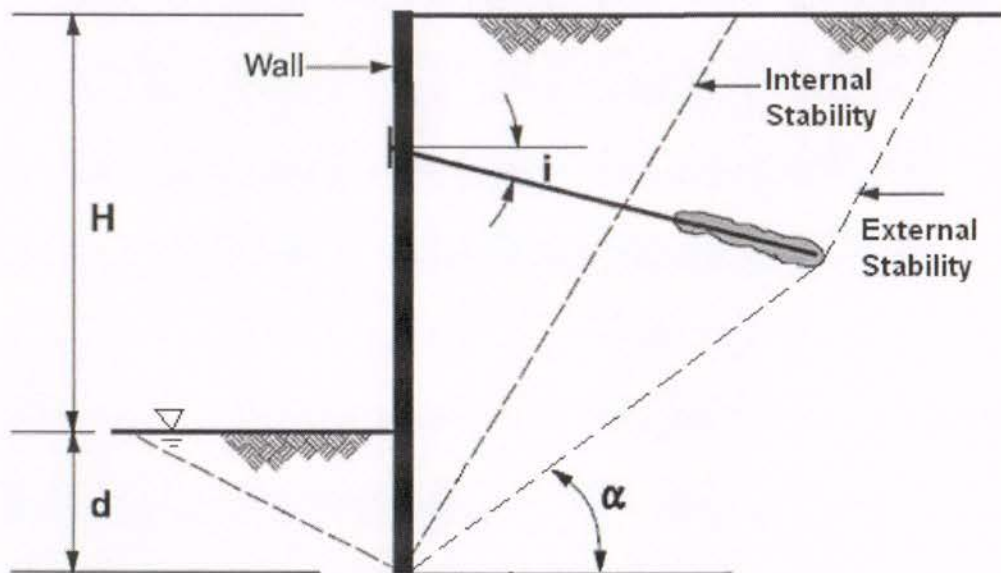
$$\text{Design Resistance} = 198/3 = 66 \text{ KN each pile or } 33 \text{ KN/m run}$$

$$\text{Design Demand } (R_{\text{base}}) = 31 \text{ KN/m}$$

Therefore, embedment depth of 2.2 m is optimum.

8. Check for internal stability

(True FS determined by successive reduction of ϕ)



$k_h := 0$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 0 \text{ deg}$
 $\beta := 0 \text{ deg}$ slope of the back of the wall
 $i := 0 \text{ deg}$ slope of the backfill
 $\phi := 27.9 \text{ deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.155$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.362$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.362$$

Failure surface inclination

$aa := \phi - i - \theta$ $bb := \phi - \beta - \theta$ $cc := \delta_a + \beta + \theta$
 $aa = 27.9 \text{ deg}$ $bb = 27.9 \text{ deg}$ $cc = 0 \text{ deg}$

$$\rho_A := \phi - \theta + \text{atan} \left[\frac{\sqrt{\tan(aa) \cdot (\tan(aa) + \cot(bb)) \cdot (1 + \tan(cc) \cdot \cot(bb))} - \tan(aa)}{1 + \tan(cc) \cdot (\tan(aa) + \cot(bb))} \right]$$

$$\rho_A = 58.95 \text{ deg}$$

Passive Kp

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_p = 0.182$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p} \quad K_{PE} = 4.433$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 4.286$$

Wedge Calculation

Single anchor, water at base of excavation

$$H_{exc} := 12 \quad \text{Depth of excavation}$$

$$H_{embed} := 2.2 \quad \text{Embedment of piles}$$

$$F_H := 373.133 \quad \text{Anchor horizontal force (ultimate)} \quad F_H = 496.09$$

$$\gamma_{above} := 16$$

$$\gamma_{below} := 8$$

$$P_A := K_{AEH} \left(0.5 \gamma_{above} \cdot H_{exc}^2 + \gamma_{above} \cdot H_{exc} \cdot H_{embed} + 0.5 \gamma_{below} \cdot H_{embed}^2 \right) \quad P_A = 577.679$$

$$P_P := K_{PEH} 0.5 \gamma_{below} \cdot H_{embed}^2 \quad P_P = 82.969$$

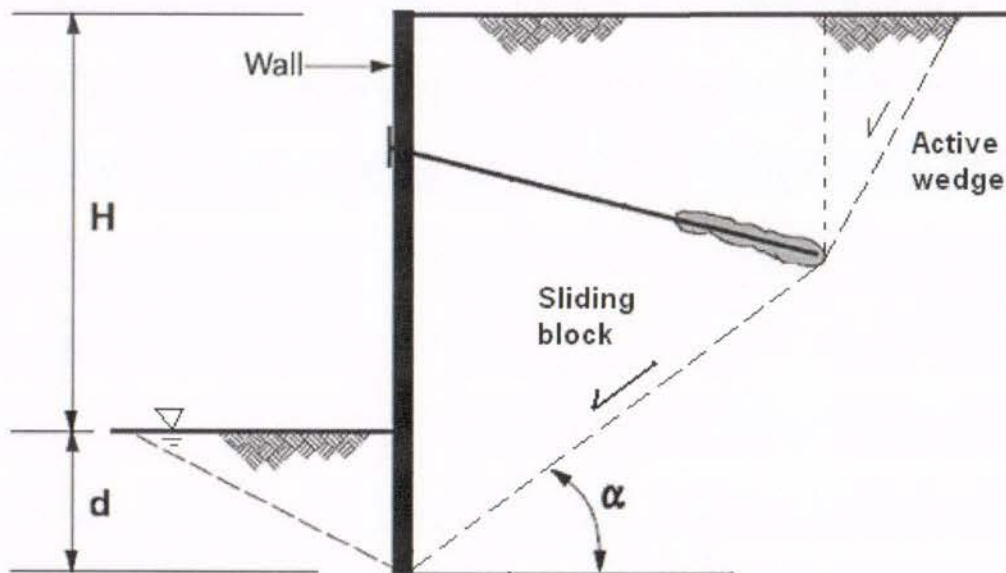
Stability calculation

$$H_{net} := P_A - P_P - F_H \quad H_{net} = -1.38 \quad < 0 \text{ for stability}$$

FS Calculation (by trial and error to set H = 0)

$$\phi_{design} := 35 \text{ deg} \quad FS := \frac{\tan(\phi_{design})}{\tan(\phi)} \quad FS = 1.322$$

9. Check for external stability



$k_h := 0$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 0 \text{ deg}$
 $\beta := 0 \text{ deg}$ slope of the back of the wall
 $i := 0 \text{ deg}$ slope of the backfill
 $\phi := 28 \text{ deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.159$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.361$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.361 \quad \text{Equivalent Horizontal Component}$$

Sliding block details

$B := 9.6$ Breadth of block
 $\alpha := 20.8 \text{ deg}$ Failure surface
 $H_{exc} := 12$ Depth of excavation
 $H_{embed} := 2.2$ Embedment of piles
 $\gamma_{above} := 16$
 $\gamma_{below} := 8$

$$W_{\text{block}} := \gamma_{\text{above}} \cdot \left[B \cdot (H_{\text{exc}} + H_{\text{embed}}) - 0.5 \tan(\alpha) \cdot B^2 \right] \quad W_{\text{block}} = 1.901 \times 10^3$$

$$W_{\text{buoy}} := W_{\text{block}} - (\gamma_{\text{above}} - \gamma_{\text{below}}) \cdot 0.5 \frac{H_{\text{embed}}^2}{\tan(\alpha)} \quad W_{\text{buoy}} = 1.85 \times 10^3$$

correction for water
assumes active
wedge is dry

$$H_{\text{block}} := W_{\text{block}} \cdot \left(\frac{\sin(\alpha) - \tan(\phi) \cdot \cos(\alpha)}{\cos(\alpha) + \tan(\phi) \cdot \sin(\alpha)} + k_h \right) \quad H_{\text{block}} = -240.159$$

This is the net contribution to horizontal movement

Active pressure wedge (zero interface friction)

$$H_{\text{active}} := H_{\text{exc}} + H_{\text{embed}} - B \cdot \tan(\alpha)$$

$$P_{\text{ah}} := 0.5 \cdot \gamma_{\text{above}} \cdot K_{\text{AEH}} \cdot H_{\text{active}}^2$$

Passive Resistance

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{\text{PE}} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P}$$

Equivalent Horizontal Component

$$K_{\text{PEH}} := \cos(\delta_i + \beta) \cdot K_{\text{PE}} \quad K_{\text{PEH}} = 4.316$$

$$P_{\text{ph}} := 0.5 \cdot \gamma_{\text{below}} \cdot K_{\text{PEH}} \cdot H_{\text{embed}}^2 \quad P_{\text{ph}} = 83.553$$

Wedge-block Stability Calculation

$$H_{\text{net}} := P_{\text{ah}} + H_{\text{block}} - P_{\text{ph}} \quad H_{\text{net}} = -2.039 \quad < 0 \text{ for stability}$$

FS Calculation (set H = 0 by trial and error)

$$\phi_{\text{design}} := 35 \cdot \text{deg} \quad \text{FS} := \frac{\tan(\phi_{\text{design}})}{\tan(\phi)} \quad \text{FS} = 1.317$$

B.2 Design calculations for case study Sand 2b – M-O based design

0.1 g

1. Calculation of K_a

$$\begin{aligned}
 k_h &:= 0.1 && \text{horizontal acceleration in g} && \theta := \text{atan}(k_h) && \theta = 0.1 \\
 \beta &:= 0\text{-deg} && \text{slope of the back of the wall} \\
 i &:= 0\text{-deg} && \text{slope of the backfill} \\
 \phi &:= 35\text{-deg} && \text{angle of internal friction} \\
 \delta_i &:= 0 && \text{angle of interface friction} && \delta_a := \delta_i
 \end{aligned}$$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi - \theta - i)}{\cos(\delta_i + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.344$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_i + \theta) \cdot D} \quad K_{AE} = 0.328$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_i) K_{AE} \quad K_{AEH} = 0.328$$

2. Calculation of apparent earth pressure (Refer to Figure 2.2.2 (a). Units = KN/m²)

$$\begin{aligned}
 H_{\text{wall}} &:= 12 && \text{Height of wall} \\
 H_1 &:= 3 && H_2 := 5 && \text{Distance to anchors} \\
 K_A &:= 0.328 \\
 \gamma &:= 16 \\
 \text{Load} &:= 0.65 K_A \cdot \gamma \cdot H_{\text{wall}}^2 && \text{Load} = 491.213 \\
 H_3 &:= H_{\text{wall}} - H_1 - H_2 && H_3 = 4 \\
 p &:= \frac{\text{Load}}{H_{\text{wall}} - \frac{H_1}{3} - \frac{H_3}{3}} && p = 50.815 && \text{Ref Fig 2.2.2 (a)}
 \end{aligned}$$

3. Calculation of anchor design loads and reaction force required at base of wall.

Anchor forces

$$T_1 := p \cdot \left(\frac{H_1}{3} + \frac{H_1}{3} + \frac{H_2}{2} \right) \quad T_1 = 228.668$$

$$R_{base} := \frac{1}{2} \cdot \frac{3}{4} \cdot p \cdot \frac{H_3}{2} \quad R_{base} = 38.111$$

$$T_2 := Load - T_1 - R_{base} \quad T_2 = 224.433$$

Cantilever moment

$$M_c := \frac{13}{54} \cdot p \cdot H_1^2 \quad M_c = 110.099$$

Maximum moment between 2 anchors

$$Z_{12} := \frac{T_1}{p} - \frac{2}{3} \cdot H_1 \quad Z_{12} = 2.5$$

$$M_{12} := p \cdot \left[\frac{1}{3} \cdot H_1 \cdot \left(\frac{13}{18} \cdot H_1 + 2Z_{12} \right) + \frac{1}{2} Z_{12}^2 \right] - T_1 \cdot Z_{12} \quad M_{12} = -48.698$$

Moment at anchor 2

$$M_2 := p \cdot \left[\frac{1}{3} \cdot H_1 \cdot \left(\frac{13}{18} \cdot H_1 + 2H_2 \right) + \frac{1}{2} H_2^2 \right] - T_1 \cdot H_2 \quad M_2 = 110.099$$

Other cases non critical - but need checking in PLAXIS

$$M_{star} := M_2 \cdot 1.6 \quad M_{star} = 176.159$$

M^* = ULS design bending moment

6. Selection of wall structural element

A typical spacing for a soldier pile wall is 2 m crs. Therefore ULS design moment $M^* = 352$ KNm per each. For example, 83 percent of a 310UC96.8 steel column section would suffice.

For the purpose of this study, it was assumed that either, such a steel column was set into a concrete filled 450 mm diameter hole, or, a reinforced concrete soldier pile of the same diameter was used.

7. Calculation of embedment depth for soldier piles

Simple Broms theory is used, with K_p calculated using Coulomb theory. Coulomb theory is used to be compatible with M-O theory for later earthquake design case. Interface friction is limited to $\phi/2$ because Coulomb theory is unconservative at higher levels of interface friction, the resulting value for K_p in this case is quite close to the value given by the NAVFAC charts which are based on log-spiral theory.

$B := 0.45$	Pile diameter	
$D := 2.5$	Depth of embedment of pile	
$\gamma := 8$	Soil unit weight (buoyant)	
$\phi := 35\text{-deg}$		
$k_h := 0.1$	horizontal acceleration in g	$\theta := \text{atan}(k_h) \quad \theta = 5.711\text{deg}$
$\beta := 0\text{-deg}$	slope of the back of the wall	
$i := 0\text{-deg}$	slope of the backfill	
$\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$	angle of interface friction (passive)	

Passive K_p

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_p = 0.114$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p} \quad K_{PE} = 7.387$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 6.972$$

Ultimate Horizontal Resistance

$$H_u := \frac{3}{2} \cdot \gamma \cdot B \cdot K_{PEH} D^2 \quad H_u = 235.313$$

A strength reduction factor of 3 is recommended to be applied to the ultimate horizontal resistance calculated using Broms' theory because of the large plastic strains required to mobilise full passive resistance. Therefore, for piles spaced at 2 m centres:

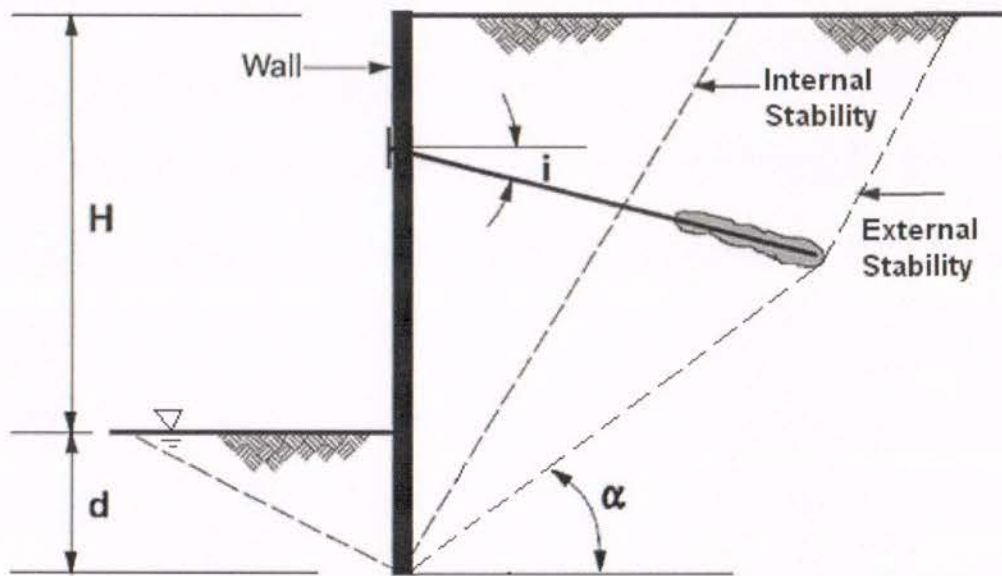
$$\text{Design Resistance} = 235/3 = 78 \text{ KN each pile or } 39 \text{ KN/m run}$$

$$\text{Design Demand } (R_{\text{base}}) = 38 \text{ KN/m}$$

Therefore, embedment depth of 2.5 m is optimum.

8. Check for internal stability

(True FS determined by successive reduction of ϕ)



$k_h := 0.1$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 5.711\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 28.2\text{-deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.034$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.424$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.424$$

Failure surface inclination

$$\begin{aligned}
 aa &:= \phi - i - \theta & bb &:= \phi - \beta - \theta & cc &:= \delta_a + \beta + \theta \\
 aa &= 22.489\text{deg} & bb &= 22.489\text{deg} & cc &= 5.711\text{deg}
 \end{aligned}$$

$$\rho_A := \phi - \theta + \text{atan} \left[\frac{\sqrt{\tan(aa) \cdot (\tan(aa) + \cot(bb)) \cdot (1 + \tan(cc) \cdot \cot(bb))} - \tan(aa)}{1 + \tan(cc) \cdot (\tan(aa) + \cot(bb))} \right]$$

$$\rho_A = 54.176\text{deg}$$

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_P = 0.222$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P} \quad K_{PE} = 4.137$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 3.996$$

Wedge Calculation

Single anchor, water at base of excavation

$$H_{exc} := 12 \quad \text{Depth of excavation}$$

$$H_{embed} := 2.5 \quad \text{Embedment of piles}$$

$$F_H := 453.133 \quad \text{Anchor horizontal force (ultimate)} \quad F_H = 602.49$$

$$\gamma_{above} := 16$$

$$\gamma_{below} := 8$$

$$P_A := K_{AEH} \left(0.5 \gamma_{above} \cdot H_{exc}^2 + \gamma_{above} \cdot H_{exc} \cdot H_{embed} + 0.5 \gamma_{below} \cdot H_{embed}^2 \right) \quad P_A = 702.38$$

$$P_P := K_{PEH} 0.5 \gamma_{below} \cdot H_{embed}^2 \quad P_P = 99.905$$

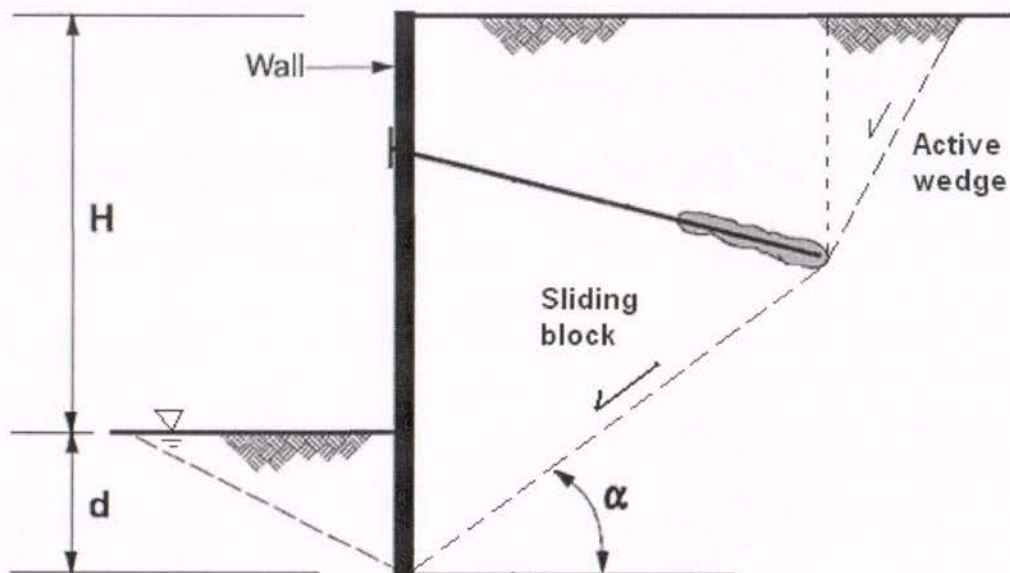
Stability calculation

$$H_{net} := P_A - P_P - F_H \quad H_{net} = -0.015 < 0 \text{ for stability}$$

FS Calculation (by trial and error to set $H = 0$)

$$\phi_{design} := 35 \text{ deg} \quad FS := \frac{\tan(\phi_{design})}{\tan(\phi)} \quad FS = 1.306$$

9. Check for external stability



$k_h := 0.1$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 5.711 \text{ deg}$
 $\beta := 0 \text{ deg}$ slope of the back of the wall
 $i := 0 \text{ deg}$ slope of the backfill
 $\phi := 31.8 \text{ deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.198$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.371$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.371 \quad \text{Equivalent Horizontal Component}$$

Sliding block details

$B := 10.2$ Breadth of block
 $\alpha := 20.3 \text{ deg}$ Failure surface
 $H_{exc} := 12$ Depth of excavation
 $H_{embed} := 2.5$ Embedment of piles
 $\gamma_{above} := 16$
 $\gamma_{below} := 8$

$$W_{\text{block}} := \gamma_{\text{above}} \cdot \left[B \cdot (H_{\text{exc}} + H_{\text{embed}}) - 0.5 \tan(\alpha) \cdot B^2 \right] \quad W_{\text{block}} = 2.059 \times 10^3$$

$$W_{\text{buoy}} := W_{\text{block}} - (\gamma_{\text{above}} - \gamma_{\text{below}}) \cdot 0.5 \frac{H_{\text{embed}}^2}{\tan(\alpha)} \quad W_{\text{buoy}} = 1.991 \times 10^3$$

correction for water
assumes active
wedge is dry

$$H_{\text{block}} := W_{\text{block}} \cdot \left(\frac{\sin(\alpha) - \tan(\phi) \cdot \cos(\alpha)}{\cos(\alpha) + \tan(\phi) \cdot \sin(\alpha)} + k_h \right) \quad H_{\text{block}} = -212.958$$

This is the net contribution to horizontal movement

Active pressure wedge (zero interface friction)

$$H_{\text{active}} := H_{\text{exc}} + H_{\text{embed}} - B \cdot \tan(\alpha)$$

$$P_{\text{ah}} := 0.5 \gamma_{\text{above}} \cdot K_{\text{AEH}} \cdot H_{\text{active}}^2$$

Passive Resistance

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{\text{PE}} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P}$$

Equivalent Horizontal Component

$$K_{\text{PEH}} := \cos(\delta_i + \beta) K_{\text{PE}} \quad K_{\text{PEH}} = 5.266$$

$$P_{\text{ph}} := 0.5 \gamma_{\text{below}} \cdot K_{\text{PEH}} \cdot H_{\text{embed}}^2 \quad P_{\text{ph}} = 131.65$$

Wedge-block Stability Calculation

$$H_{\text{net}} := P_{\text{ah}} + H_{\text{block}} - P_{\text{ph}} \quad H_{\text{net}} = -3.436 \quad < 0 \text{ for stability}$$

FS Calculation (set H = 0 by trial and error)

$$\phi_{\text{design}} := 35 \text{ deg} \quad FS := \frac{\tan(\phi_{\text{design}})}{\tan(\phi)} \quad FS = 1.129$$

B.3 Design calculations for case study Sand 2c – M-O based design

0.2 g

1. Calculation of K_a

$$\begin{aligned}
 k_h &:= 0.2 && \text{horizontal acceleration in g} && \theta := \text{atan}(k_h) && \theta = 0.197 \\
 \beta &:= 0\text{-deg} && \text{slope of the back of the wall} \\
 i &:= 0\text{-deg} && \text{slope of the backfill} \\
 \phi &:= 35\text{-deg} && \text{angle of internal friction} \\
 \delta_i &:= 0 && \text{angle of interface friction} && \delta_a := \delta_i
 \end{aligned}$$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi - \theta - i)}{\cos(\delta_i + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.205$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_i + \theta)} \cdot D \quad K_{AE} = 0.396$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_i) K_{AE} \quad K_{AEH} = 0.396$$

2. Calculation of apparent earth pressure (Refer to Figure 2.2.2 (a). Units = KN/m²)

$$\begin{aligned}
 H_{\text{wall}} &:= 12 && \text{Height of wall} \\
 H_1 &:= 3 && H_2 := 5 && \text{Distance to anchors} \\
 K_A &:= 0.396 \\
 \gamma &:= 16
 \end{aligned}$$

$$\text{Load} := 0.65 K_A \cdot \gamma \cdot H_{\text{wall}}^2 \quad \text{Load} = 593.05$$

$$H_3 := H_{\text{wall}} - H_1 - H_2 \quad H_3 = 4$$

$$p := \frac{\text{Load}}{H_{\text{wall}} - \frac{H_1}{3} - \frac{H_3}{3}} \quad p = 61.35 \quad \text{Ref Fig 2.2.2 (a)}$$

3. Calculation of anchor design loads and reaction force required at base of wall.

Anchor forces

$$T_1 := p \cdot \left(\frac{H_1}{3} + \frac{H_1}{3} + \frac{H_2}{2} \right) \quad T_1 = 276.075$$

$$R_{base} := \frac{1}{2} \cdot \frac{3}{4} \cdot p \cdot \frac{H_3}{2} \quad R_{base} = 46.012$$

$$T_2 := Load - T_1 - R_{base} \quad T_2 = 270.962$$

Cantilever moment

$$M_c := \frac{13}{54} \cdot p \cdot H_1^2 \quad M_c = 132.925$$

Maximum moment between 2 anchors

$$Z_{12} := \frac{T_1}{p} - \frac{2}{3} \cdot H_1 \quad Z_{12} = 2.5$$

$$M_{12} := p \cdot \left[\frac{1}{3} H_1 \cdot \left(\frac{13}{18} H_1 + 2Z_{12} \right) + \frac{1}{2} Z_{12}^2 \right] - T_1 \cdot Z_{12} \quad M_{12} = -58.794$$

Moment at anchor 2

$$M_2 := p \cdot \left[\frac{1}{3} H_1 \cdot \left(\frac{13}{18} H_1 + 2H_2 \right) + \frac{1}{2} H_2^2 \right] - T_1 \cdot H_2 \quad M_2 = 132.925$$

Other cases non critical - but need checking in PLAXIS

$$M_{star} := M_2 \cdot 1.6 \quad M_{star} = 212.68$$

M^* = ULS design bending moment

6. Selection of wall structural element

A typical spacing for a soldier pile wall is 2 m crs. Therefore ULS design moment $M^* = 425$ KNm per each. For example, 101 percent of a 310UC96.8 steel column section would suffice.

For the purpose of this study, it was assumed that either, such a steel column was set into a concrete filled 450 mm diameter hole, or, a reinforced concrete soldier pile of the same diameter was used.

7. Calculation of embedment depth for soldier piles

Simple Broms theory is used, with K_p calculated using Coulomb theory. Coulomb theory is used to be compatible with M-O theory for later earthquake design case. Interface friction is limited to $\phi/2$ because Coulomb theory is unconservative at higher levels of interface friction, the resulting value for K_p in this case is quite close to the value given by the NAVFAC charts which are based on log-spiral theory.

$B := 0.45$	Pile diameter	
$D := 2.9$	Depth of embedment of pile	
$\gamma := 8$	Soil unit weight (buoyant)	
$\phi := 35 \cdot \text{deg}$		
$k_h := 0.2$	horizontal acceleration in g	$\theta := \text{atan}(k_h) \quad \theta = 11.31 \text{deg}$
$\beta := 0 \cdot \text{deg}$	slope of the back of the wall	
$i := 0 \cdot \text{deg}$	slope of the backfill	
$\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$	angle of interface friction (passive)	

Passive K_p

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_p = 0.148$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p} \quad K_{PE} = 6.727$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 6.349$$

Ultimate Horizontal Resistance

$$H_u := \frac{3}{2} \cdot \gamma \cdot B \cdot K_{PEH} D^2 \quad H_u = 288.352$$

A strength reduction factor of 3 is recommended to be applied to the ultimate horizontal resistance calculated using Broms' theory because of the large plastic strains required to mobilise full passive resistance. Therefore, for piles spaced at 2 m centres:

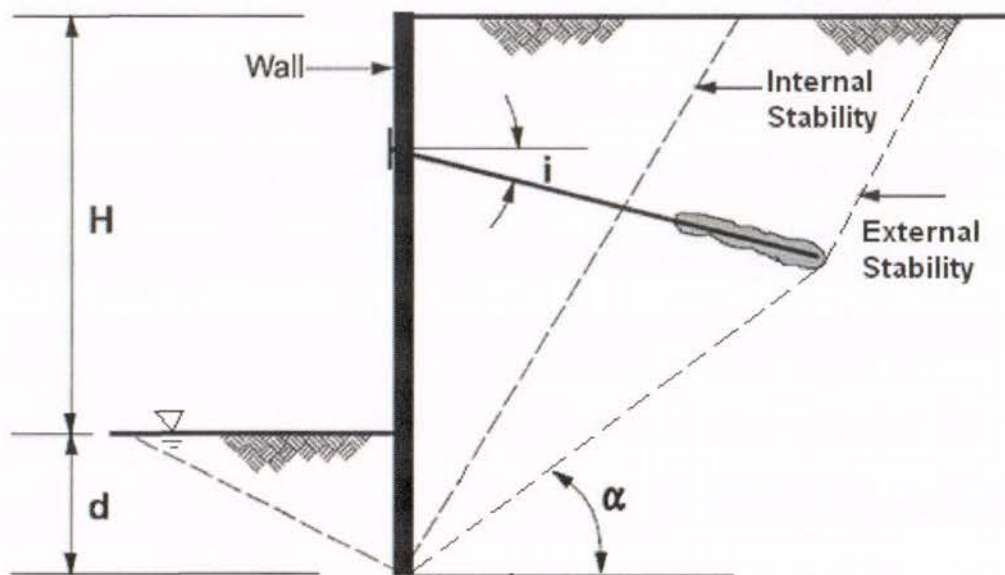
$$\text{Design Resistance} = 288/3 = 96 \text{ KN each pile or } 48 \text{ KN/m run}$$

$$\text{Design Demand } (R_{\text{base}}) = 46 \text{ KN/m}$$

Therefore, embedment depth of 2.9 m is optimum.

8. Check for internal stability

(True FS determined by successive reduction of ϕ)



$k_h := 0.2$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 11.31\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 29\text{-deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 1.925$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta)} \cdot D \quad K_{AE} = 0.49$$

Equivalent Horizontal Component

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.49$$

Failure surface inclination

$aa := \phi - i - \theta$ $bb := \phi - \beta - \theta$ $cc := \delta_a + \beta + \theta$
 $aa = 17.69\text{deg}$ $bb = 17.69\text{deg}$ $cc = 11.31\text{deg}$

$$\rho_A := \phi - \theta + \text{atan} \left[\frac{\sqrt{\tan(aa) \cdot (\tan(aa) + \cot(bb)) \cdot (1 + \tan(cc) \cdot \cot(bb))} - \tan(aa)}{1 + \tan(cc) \cdot (\tan(aa) + \cot(bb))} \right]$$

$$\rho_A = 48.789\text{deg}$$

Passive Kp

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D_P = 0.262$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P} \quad K_{PE} = 3.963$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 3.819$$

Wedge Calculation

Single anchor, water at base of excavation

$$H_{exc} := 12 \quad \text{Depth of excavation}$$

$$H_{embed} := 2.9 \quad \text{Embedment of piles}$$

$$F_H := 547.1.33 \quad \text{Anchor horizontal force (ultimate)} \quad F_H = 727.51$$

$$\gamma_{above} := 16$$

$$\gamma_{below} := 8$$

$$P_A := K_{AEH} \left(0.5 \gamma_{above} \cdot H_{exc}^2 + \gamma_{above} \cdot H_{exc} \cdot H_{embed} + 0.5 \gamma_{below} \cdot H_{embed}^2 \right) \quad P_A = 854.253$$

$$P_P := K_{PEH} 0.5 \gamma_{below} \cdot H_{embed}^2 \quad P_P = 128.468$$

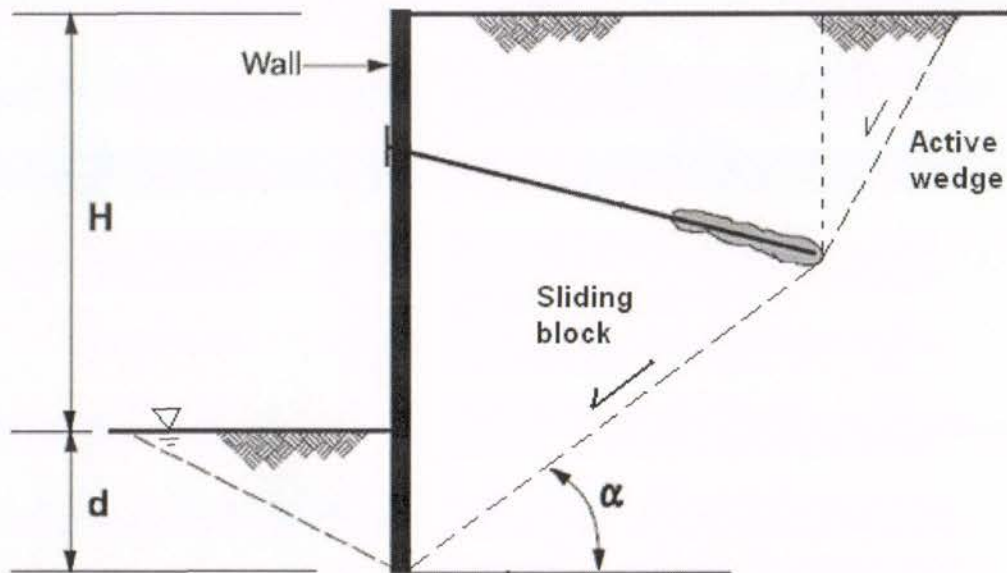
Stability calculation

$$H_{net} := P_A - P_P - F_H \quad H_{net} = -1.725 \quad < 0 \text{ for stability}$$

FS Calculation (by trial and error to set $H = 0$)

$$\phi_{design} := 35\text{-deg} \quad FS := \frac{\tan(\phi_{design})}{\tan(\phi)} \quad FS = 1.263$$

9. Check for external stability



$k_h := 0.2$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 11.31\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 35\text{-deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.205$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.396$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.396 \quad \text{Equivalent Horizontal Component}$$

Sliding block details

$B := 11.07$ Breadth of block
 $\alpha := 19.6\text{deg}$ Failure surface
 $H_{exc} := 12$ Depth of excavation
 $H_{embed} := 2.9$ Embedment of piles
 $\gamma_{above} := 16$
 $\gamma_{below} := 8$

$$W_{\text{block}} := \gamma_{\text{above}} \cdot \left[B \cdot (H_{\text{exc}} + H_{\text{embed}}) - 0.5 \tan(\alpha) \cdot B^2 \right] \quad W_{\text{block}} = 2.29 \times 10^3$$

$$W_{\text{buoy}} := W_{\text{block}} - (\gamma_{\text{above}} - \gamma_{\text{below}}) \cdot 0.5 \frac{H_{\text{embed}}^2}{\tan(\alpha)} \quad W_{\text{buoy}} = 2.196 \times 10^3$$

correction for water
assumes active
wedge is dry

$$H_{\text{block}} := W_{\text{block}} \cdot \left(\frac{\sin(\alpha) - \tan(\phi) \cdot \cos(\alpha)}{\cos(\alpha) + \tan(\phi) \cdot \sin(\alpha)} + k_h \right) \quad H_{\text{block}} = -172.771$$

This is the net contribution to horizontal movement

Active pressure wedge (zero interface friction)

$$H_{\text{active}} := H_{\text{exc}} + H_{\text{embed}} - B \cdot \tan(\alpha)$$

$$P_{\text{ah}} := 0.5 \gamma_{\text{above}} \cdot K_{\text{AEH}} \cdot H_{\text{active}}^2$$

Passive Resistance

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{\text{PE}} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p}$$

Equivalent Horizontal Component

$$K_{\text{PEH}} := \cos(\delta_i + \beta) K_{\text{PE}} \quad K_{\text{PEH}} = 6.349$$

$$P_{\text{ph}} := 0.5 \gamma_{\text{below}} \cdot K_{\text{PEH}} \cdot H_{\text{embed}}^2 \quad P_{\text{ph}} = 213.594$$

Wedge-block Stability Calculation

$$H_{\text{net}} := P_{\text{ah}} + H_{\text{block}} - P_{\text{ph}} \quad H_{\text{net}} = -6.346 \quad < 0 \text{ for stability}$$

FS Calculation (set H = 0 by trial and error)

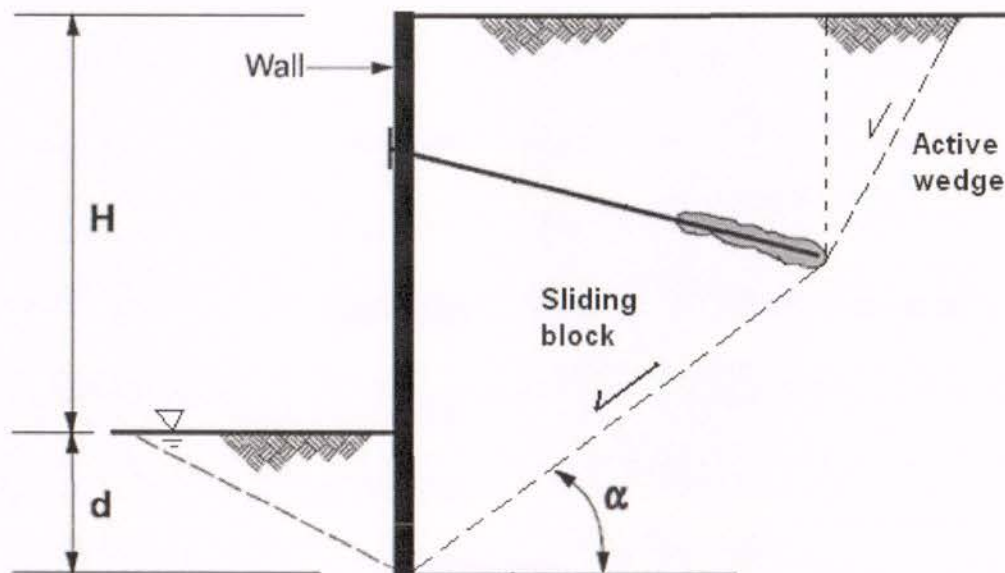
$$\phi_{\text{design}} := 35\text{-deg} \quad \text{FS} := \frac{\tan(\phi_{\text{design}})}{\tan(\phi)} \quad \text{FS} = 1$$

Appendix C

C.1 Design calculations for case study Sand 3a – Gravity based design

Steps 1 to 8 same as for Sand 2a.

9. Check for external stability



$k_h := 0$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 0 \text{ deg}$
 $\beta := 0 \text{ deg}$ slope of the back of the wall
 $i := 0 \text{ deg}$ slope of the backfill
 $\phi := 26.7 \text{ deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.101$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.38$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.38 \quad \text{Equivalent Horizontal Component}$$

Sliding block details

$B := 10.14$ Breadth of block
 $\alpha := 19\text{-deg}$ Failure surface
 $H_{exc} := 12$ Depth of excavation
 $H_{embed} := 2.2$ Embedment of piles
 $\gamma_{above} := 16$
 $\gamma_{below} := 8$

$$W_{block} := \gamma_{above} \cdot \left[B \cdot (H_{exc} + H_{embed}) - 0.5 \cdot \tan(\alpha) \cdot B^2 \right] \quad W_{block} = 2.021 \times 10^3$$

$$W_{buoy} := W_{block} - (\gamma_{above} - \gamma_{below}) \cdot 0.5 \cdot \frac{H_{embed}^2}{\tan(\alpha)} \quad W_{buoy} = 1.964 \times 10^3$$

correction for water
assumes active
wedge is dry

$$H_{block} := W_{block} \cdot \left(\frac{\sin(\alpha) - \tan(\phi) \cdot \cos(\alpha)}{\cos(\alpha) + \tan(\phi) \cdot \sin(\alpha)} + k_h \right) \quad H_{block} = -273.193$$

This is the net contribution to horizontal movement

Active pressure wedge (zero interface friction)

$$H_{active} := H_{exc} + H_{embed} - B \cdot \tan(\alpha)$$

$$P_{ah} := 0.5 \cdot \gamma_{above} \cdot K_{AEH} \cdot H_{active}^2$$

Passive Resistance

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P}$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) \cdot K_{PE} \quad K_{PEH} = 3.947$$

$$P_{ph} := 0.5 \cdot \gamma_{below} \cdot K_{PEH} \cdot H_{embed}^2 \quad P_{ph} = 76.406$$

Wedge-block Stability Calculation

$$H_{net} := P_{ah} + H_{block} - P_{ph} \quad H_{net} = -1.033 \quad < 0 \text{ for stability}$$

FS Calculation (set $H = 0$ by trial and error)

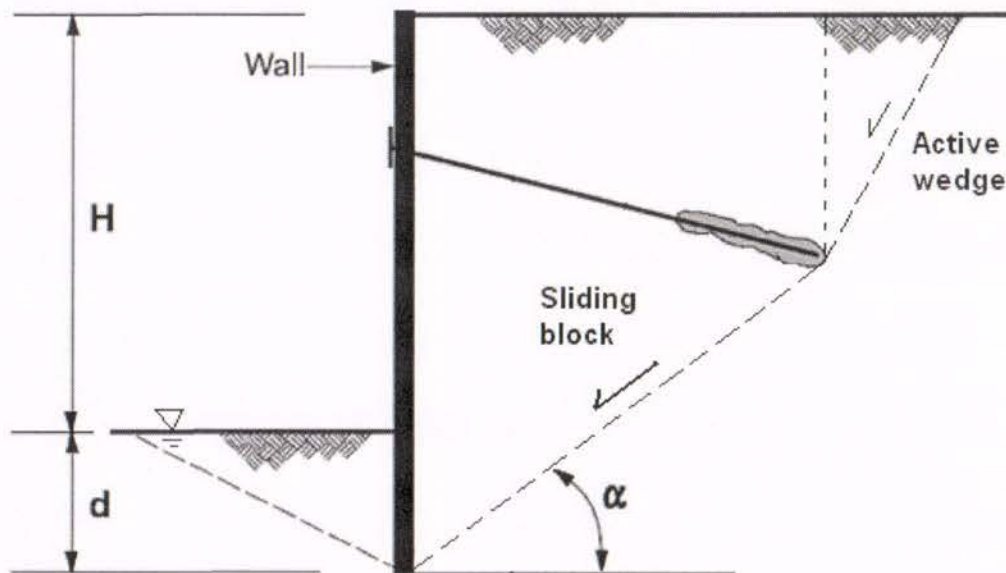
$$\phi_{design} := 35\text{-deg} \quad FS := \frac{\tan(\phi_{design})}{\tan(\phi)} \quad FS = 1.392$$

C.2 Design calculations for case study Sand 3b – M-O based design

0.1 g

Steps 1 to 8 same as for Sand 2b.

9. Check for external stability



$k_h := 0.1$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 5.711\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 30.6\text{deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.143$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.388$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.388 \quad \text{Equivalent Horizontal Component}$$

$$k_h := 0.1 \quad \text{horizontal acceleration in g} \quad \theta := \text{atan}(k_h) \quad \theta = 5.711 \text{deg}$$

$$\beta := 0 \text{-deg} \quad \text{slope of the back of the wall}$$

$$i := 0 \text{-deg} \quad \text{slope of the backfill}$$

$$\phi := 30.6 \text{deg} \quad \text{angle of internal friction}$$

$$\delta_i := \text{atan}(0.5 \cdot \tan(\phi)) \quad \text{angle of interface friction (passive)} \quad \delta_a := 0$$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.143$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.388$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.388 \quad \text{Equivalent Horizontal Component}$$

Active pressure wedge (zero interface friction)

$$H_{\text{active}} := H_{\text{exc}} + H_{\text{embed}} - B \cdot \tan(\alpha)$$

$$P_{ah} := 0.5 \cdot \gamma_{\text{above}} \cdot K_{AEH} \cdot H_{\text{active}}^2$$

Passive Resistance

$$D_P := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_P}$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) K_{PE} \quad K_{PEH} = 4.784$$

$$P_{ph} := 0.5 \cdot \gamma_{\text{below}} \cdot K_{PEH} \cdot H_{\text{embed}}^2 \quad P_{ph} = 119.598$$

Wedge-block Stability Calculation

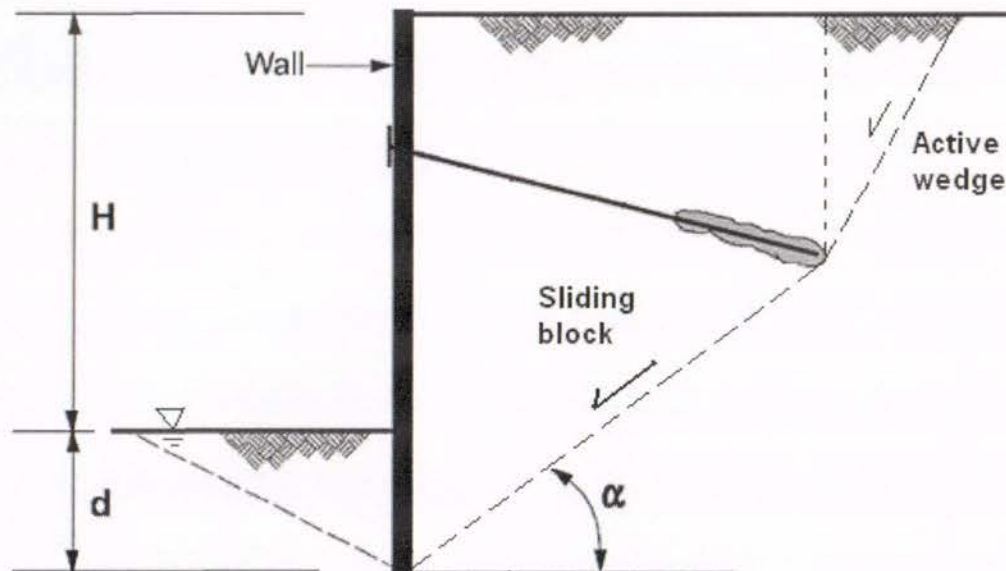
$$H_{\text{net}} := P_{ah} + H_{\text{block}} - P_{ph} \quad H_{\text{net}} = -5.643 \quad < 0 \text{ for stability}$$

FS Calculation (set H = 0 by trial and error)

$$\phi_{\text{design}} := 35 \text{-deg} \quad FS := \frac{\tan(\phi_{\text{design}})}{\tan(\phi)} \quad FS = 1.184$$

C3 Design calculations for case study Sand 3c – M-O based design 0.2 g

Steps 1 to 8 same as for Sand 2c.



$k_h := 0.2$ horizontal acceleration in g $\theta := \text{atan}(k_h)$ $\theta = 11.31\text{deg}$
 $\beta := 0\text{-deg}$ slope of the back of the wall
 $i := 0\text{-deg}$ slope of the backfill
 $\phi := 34.1\text{-deg}$ angle of internal friction
 $\delta_i := \text{atan}(0.5 \cdot \tan(\phi))$ angle of interface friction (passive) $\delta_a := 0$

Calculation

$$D := \left[1 + \left(\frac{\sin(\phi + \delta_a) \cdot \sin(\phi - \theta - i)}{\cos(\delta_a + \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2 \quad D = 2.163$$

$$K_{AE} := \frac{\cos(\phi - \theta - \beta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\beta + \delta_a + \theta) \cdot D} \quad K_{AE} = 0.409$$

$$K_{AEH} := \cos(\delta_a + \beta) K_{AE} \quad K_{AEH} = 0.409 \quad \text{Equivalent Horizontal Component}$$

Sliding block details

$B := 11.66$	Breadth of block
$\alpha := 18\text{-deg}$	Failure surface
$H_{exc} := 12$	Depth of excavation
$H_{embed} := 2.9$	Embedment of piles
$\gamma_{above} := 16$	
$\gamma_{below} := 8$	

$$W_{block} := \gamma_{above} \cdot \left[B \cdot (H_{exc} + H_{embed}) - 0.5 \tan(\alpha) \cdot B^2 \right] \quad W_{block} = 2.426 \times 10^3$$

$$W_{buoy} := W_{block} - (\gamma_{above} - \gamma_{below}) \cdot 0.5 \frac{H_{embed}^2}{\tan(\alpha)} \quad W_{buoy} = 2.323 \times 10^3$$

correction for water
assumes active
wedge is dry

$$H_{block} := W_{block} \cdot \left(\frac{\sin(\alpha) - \tan(\phi) \cdot \cos(\alpha)}{\cos(\alpha) + \tan(\phi) \cdot \sin(\alpha)} + k_h \right) \quad H_{block} = -215.06$$

This is the net contribution to horizontal movement

Active pressure wedge (zero interface friction)

$$H_{active} := H_{exc} + H_{embed} - B \cdot \tan(\alpha)$$

$$P_{ah} := 0.5 \cdot \gamma_{above} \cdot K_{AEH} \cdot H_{active}^2$$

Passive Resistance

$$D_p := \left[1 - \left(\frac{\sin(\phi + \delta_i) \cdot \sin(\phi + i - \theta)}{\cos(\delta_i - \beta + \theta) \cos(i - \beta)} \right)^{0.5} \right]^2$$

$$K_{PE} := \frac{\cos(\phi + \beta - \theta)^2}{\cos(\theta) \cos(\beta)^2 \cos(\delta_i - \beta + \theta) \cdot D_p}$$

Equivalent Horizontal Component

$$K_{PEH} := \cos(\delta_i + \beta) \cdot K_{PE} \quad K_{PEH} = 5.836$$

$$P_{ph} := 0.5 \cdot \gamma_{below} \cdot K_{PEH} \cdot H_{embed}^2 \quad P_{ph} = 196.324$$

Wedge-block Stability Calculation

$$H_{net} := P_{ah} + H_{block} - P_{ph} \quad H_{net} = -7.674 \quad < 0 \text{ for stability}$$

FS Calculation (set H = 0 by trial and error)

$$\phi_{design} := 35\text{-deg} \quad FS := \frac{\tan(\phi_{design})}{\tan(\phi)} \quad FS = 1.034$$