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DESIGN OF REINFORCED SOIL STRUCTURES UNDER SEISMIC LOADING USING A TWO-PART WEDGE METHOD

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ABSTRACT

The performance of polymer geogrid reinforced soil structures in recent strong earthquakes has generally been very good, although there have also been failures, some of which have received a lot of attention and have highlighted weaknesses both in the construction techniques, as well as in the calculation procedures used in design. One specific case consisted of three modular block walls which were shaken by the Chi Chi earthquake in 1999 resulting in collapse of two of the walls, whereas the third remained intact. This case raises a number of questions, in particular how a design method takes into account connection strength between the reinforcement and the facing when it is significantly lower than the strength of the reinforcement itself. The majority of routine design methods used for reinforced soil structures are based on limiting equilibrium, and for the seismic design case, the additional inertia loads due to the earthquake are included as equivalent static loads. These methods are referred to as pseudo-static design methods.

This paper describes the development of a simple pseudo-static calculation procedure using a two-part wedge, in which a large number of possible failure mechanisms are examined. This two-part wedge method has the advantage that few assumptions are required, and it may take into account earthquake loading. The important development of the method is the technique used to take into account the contribution to stability provided by the wall facing in the case of reinforced soil retaining walls which use the popular modular block facing system. The method is applied to the modular block walls which were shaken by the Chi Chi earthquake using a computer program called **TensarSoil®** which demonstrates that the observed behaviour could have been predicted.

INTRODUCTION

The performance of polymer geogrid reinforced soil structures in recent strong earthquakes has generally been very good, for example Nishimura et al (1996) describe the performance of ten structures reinforced with high density polyethylene (HDPE) geogrids which were shaken by the 1995 Hyogo-ken Nanbu (Kobe) Earthquake, and suffered either no damage or minor superficial cracks. Tatsuoka et al (1996) give a comprehensive account of the performance of a wide range of retaining structure types during the same earthquake, and come to the conclusion that those built using reinforced soil techniques performed the best, particularly those incorporating geosynthetic reinforcement. In fact the performance of reinforced soil structures was so good that many damaged retaining structures were reconstructed using the same techniques after the earthquake.

Just four years later another strong earthquake on the same tectonic boundary caused extensive damage in central Taiwan, the Chi Chi earthquake in 1999. There were many retaining structures in the epicentral area of this earthquake which were affected by the shaking, including reinforced soil

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structures, many of which performed very well. However there were also some failures, two in particular, which became the subject of intense investigation and analysis, and many papers and reports were published regarding these structures. One was a very high reinforced soil slope which had been constructed to create a cutting as part of the access road to the Chi-Nan University. A description of this structure as well as some of the investigations and analyses carried out are reported by Chou & Fan (2001) and Holtz et al (2001) amongst others. The second structure which created a lot of interest was actually a series of three structures, consisting of low height retaining walls built a short time before the earthquake as part of a road widening scheme. The retaining walls were built using the modular block system, which consists of small (40kg) concrete blocks which are dry stacked, with geogrid reinforcement used to stabilise the soil mass behind the facing. In the case of these walls, the woven coated polyester reinforcement was joined to the facing by laying the reinforcement between the concrete blocks, creating what is normally referred to as a "frictional connection". Detailed information about these walls is provided by Huang & Tatsuoka (2001), and several other papers published at that time.

The author of this paper visited the site of the retaining walls a few months after the earthquake had taken place. Figure 1 shows two of many photographs taken, and similar photographs may be seen in the many published papers referred to above. The first important point is that two of the walls had failed, and one had remained stable. The mode of failure was as shown in the right-hand photo of Figure 1, namely a bursting failure along a horizontal line approximately half up the wall face. It is important to note that the facing blocks along the top and base of each wall remained roughly in position, and there was no general failure of the reinforced soil mass behind the wall face. The failure was essentially the facing bursting forwards due to the inertia forces created by the earthquake.

Figure 1. Two modular block retaining walls located close to the epicentre of the 1999 Chi Chi earthquake in Taiwan, showing the results of strong shaking (Site 3 on left, Site 1 on right)

A further interesting feature of the wall which did not fail may be seen in the left-hand photo in Figure 1, namely the gap below the top row of retaining wall blocks. Modular block walls are normally built with a layer of drainage gravel behind the facing, and on closer inspection, it can be seen that the gap contains large gravel particles. This means that during the earthquake shaking, a gap was created between the blocks, however briefly, and the gravel particles entered the gap. This was possible presumably due to a very high upward vertical component of force, combined with the horizontal force created by the earthquake (the vertical accelerations recorded during the Chi Chi earthquake were generally in the order of 0.5× to 1.0× the horizontal peak accelerations).

Based on the observations outlined above, it is clear that design methods used for this form of structure, namely reinforced soil retaining walls with modular block facing, should be capable of predicting or indicating the mechanism of failure which took place, which was dominated by the performance of the facing and the connection between the facing and the reinforcement. This paper describes such a method, developed over the years since the Chi Chi earthquake, and now incorporated into a computer program developed by the Author's company.

DESIGN OF REINFORCED SOIL STRUCTURES: AN OUTLINE

The reinforced soil retaining wall design method discussed in this paper is based on limiting equilibrium principles, which are used for the majority of published methods. The design is carried out in two stages. Firstly an external stability analysis is carried out, which is used to determine the overall dimensions of the reinforced fill block, namely dimension B as shown in Figure 2. The external stability check is essentially a gravity retaining wall calculation, and is much the same in all codes and guidelines. This part of the design procedure fixes the length of the reinforcement. It should be noted that in many methods there is a limit on the ratio B/H as shown in Figure 2, and that this limit will often determine the reinforcement length rather than any other calculation.

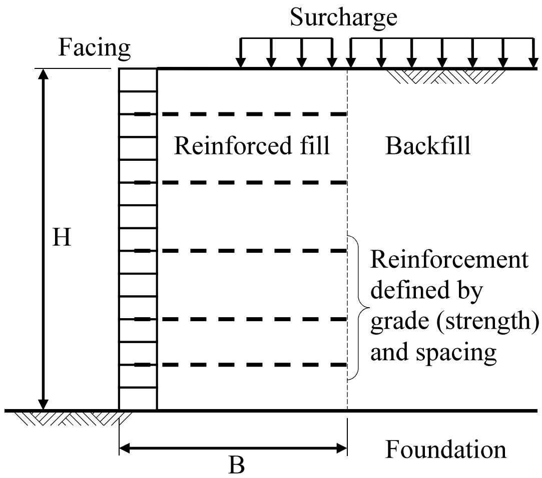

Figure 2. Reinforced soil structure main elements

The second stage of the calculation is to examine internal stability, to ensure that the layout of reinforcement (grade/strength and vertical spacing) is sufficient to meet the design requirements. The internal stability calculation should also take into account design features such as the connection strength between the reinforcement and the facing, and earthquake loading if required.

For both external and internal stability calculations, any "method" of design may be divided into three main elements: calculation procedure, material parameters and factors. These are outlined in more detail in Table 1, and the remainder of this paper concentrates on the calculation procedure for the internal stability check, as well as definition of the relevant material properties.

Table 1. The main elements of a reinforced soil design method

Element of design method	**Details**	**Comments**
Calculation procedure	Method of calculating forces and stresses in order to make a design, covering both external and internal stability	For external design, most methods are the same, but for internal design there are significant differences
Material parameters	- Soil parameters - Reinforcement parameters - Interaction between soil and reinforcement	Material parameters should be measured using appropriate test methods and assessed as suitable for design
Factors	- Safety factors - Partial load and material factors - Wall friction angle on back of reinforced soil block - Soil strength definition	Factors ensure the margin against failure of the structure, and define some important design parameters

INTERNAL STABILITY: TWO-PART WEDGE DEFINITION

The calculation procedure used to check internal stability as described and developed in this paper is referred to as the "two-part wedge" method. The basis of the two-part wedge method of analysis for internal stability is shown on Figure 3. The chosen geometry is typical of reinforced soil structures, but the method of analysis can incorporate all features shown without the need for any simplifying assumptions. The method of analysis is that of limiting equilibrium, but with the important requirement that any mechanism used should be admissible (ie. can actually happen) and that all forces associated with that mechanism should be taken into account.

There is a second main method used to assess the internal stability of reinforced soil retaining walls, called the "tie-back wedge" method. This method is used in many published guides and codes, and relies on identifying a single failure mechanism (normally the Rankine wedge), and then basing all calculations, including the seismic design case, on this one mechanism. There are considerable issues with this method, in particular that many assumptions are required, and in the case of a low strength connection, then this strength is applied over the entire length of the reinforcement. For a more detailed discussion of the tie-back wedge method, see Dobie (2011).

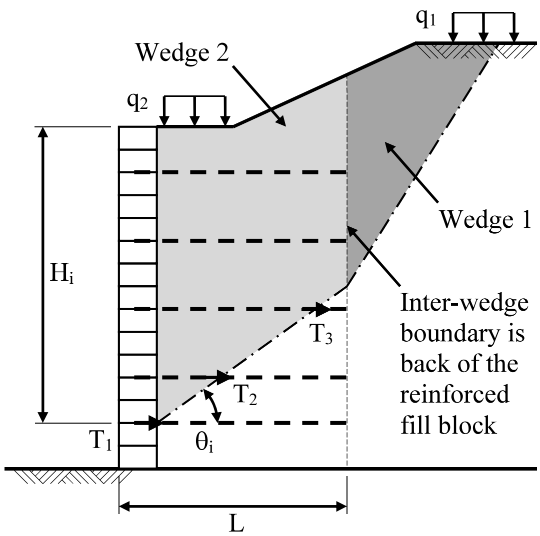

Figure 3. Basis of the two-part wedge method

The two-part wedge is defined as follows with reference to Figure 3:

(1) Fix a distance H_i below the top of the wall
(2) Draw a line at an angle θ_i across the reinforced soil block, defining Wedge 2
(3) Starting at the point where Wedge 2 intersects the back of the reinforced soil block, define a second wedge, Wedge 1 as shown
(4) The inter-wedge boundary is defined as the back of the reinforced soil block

The assumption that the inter-wedge boundary coincides with the back of the reinforced soil block is clearly very convenient, but might not result in the worst case. This would be a problem for lesser facing angles, but for walls where the inclination of the back of the reinforced soil block is generally > 70°, this does not introduce significant errors.

Wedge 1 is used to calculate the earth pressure forces applied to the back of the reinforced soil block, and for simple geometry and conditions, this may be replaced by the Coulomb formula (or Mononobe Okabe for the seismic design case). However for the geometry and isolated surcharge as shown on Figure 3, it is not possible to use the Coulomb formula without making some simplifying assumptions. In this situation, to obtain the maximum lateral forces applied by Wedge 1 rigorously, it is necessary to use a trial wedge method in which the angle of Wedge 1 is varied until the maximum lateral thrust is obtained. This is known as the Culmann method or Coulomb sweeping wedge, and is necessary to avoid introducing simplifying assumptions to this part of the internal stability calculation.

The aim of the calculation is to make sure that the resistance provided by the facing and reinforcement which is intersected by Wedge 2 ($T_1 + T_2 + T_3$ as shown on Figure 3) is sufficient to avoid instability of the two wedges. Once this has been established for the wedges shown on Figure 3, another pair of wedges is selected (by adjusting Hi and θi) and the process is repeated. It cannot be judged in advance which pair of wedges will be critical, so it is normal to set up a search routine, which is described below. Details of the calculation procedure are also described in a later section of this paper.

INTERNAL STABILITY: SEARCH PROCEDURE

In order to find the critical two-part wedge, it is necessary to search through a large number of combinations of wedges. This is normally done as shown on Figure 4 (left). For a specific value of H_i, various values of θ_i are used so that a "fan" of wedges is checked. H_i is then adjusted and the fan of wedges repeated. Normally H_i is chosen starting at the base of the wall ($H_i = H$), then at each elevation where reinforcement intersects the facing. In software developed by the author's company, θ_i is chosen at 3° intervals. In this way a large number of two-part wedges are checked.

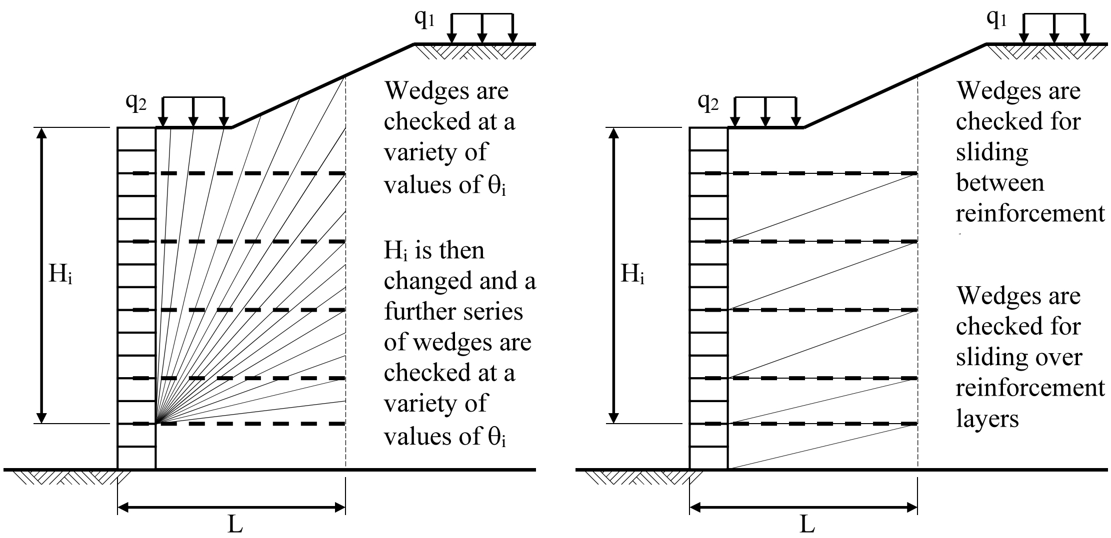

Figure 4. Search procedures used in the two-part wedge method

There are some special cases of two-part wedges which should be checked, as shown on Figure 4 (right). Wedges defined by the maximum possible values of θ_i which do not intersect reinforcement may well be critical, especially if vertical spacing is large. This check is normally carried out between all pairs of reinforcement layers, as shown on Figure 4 (right). In the case of uniform spacing and surcharge, the critical case is the lowest wedge. However at higher levels where vertical reinforcement spacing is increased, this check may be critical again. It may also be critical if large isolated surcharges are present just behind the reinforced soil block. This check also has the benefit of ensuring that vertical spacing does not become too large.

The second check is sliding over the reinforcement, which is also required by some tie-back wedge methods. This check may be critical in cases where the fill/reinforcement combination has a low sliding interaction factor, and is generally critical for the lowest layer of reinforcement.

INTERNAL STABILITY: FORCE REQUIRED TO STABILISE WEDGE

The method of calculation is divided into two stages. Firstly the force required to stabilise any wedge is calculated as shown on Figure 5 for both the static and seismic load cases. The various forces applied to Wedge 2 are calculated as outlined in Table 2.

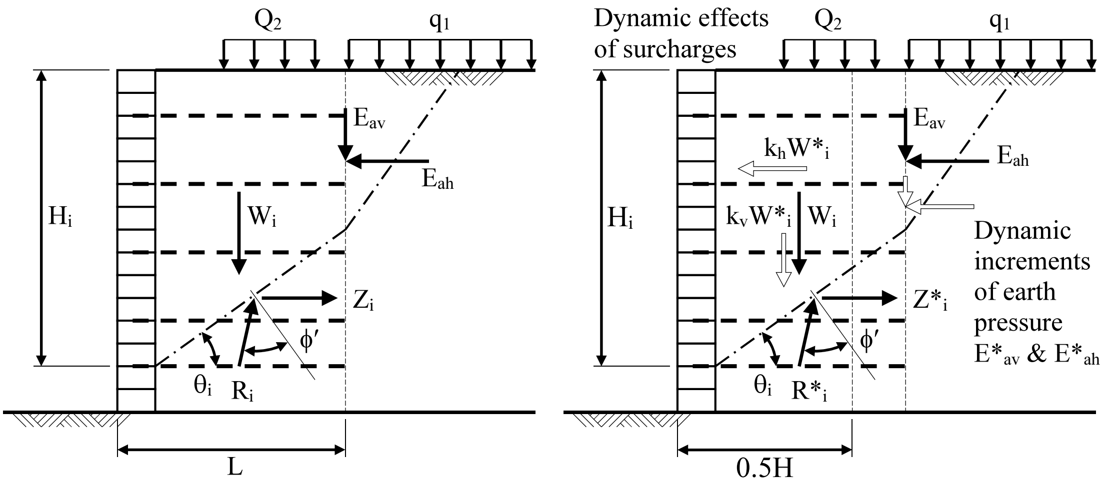

Figure 5. Calculating force required in the static case (left) and seismic case (right)

The search procedure for seismic design is the same as for static design. The main difference comes in the method of calculation of forces applied to Wedge 2. Additional forces are defined to represent the inertia caused by earthquake shaking, as shown on Figure 5 (right), with comments given in Table 2. Forces due to earthquake loading are denoted with an asterisk (*) to distinguish them from static forces. The basic approach is to assess the additional forces due to the earthquake, and add these to the underlying static forces.

Table 2. Forces applied to Wedge 2

Forces for the static case		**Additional forces for the seismic case**	
E_{ah}	Horizontal earth pressure force applied by retained backfill and any superimposed surcharges behind the reinforced soil block	E^*_{ah}	Additional horizontal earth pressure force applied by retained backfill and any superimposed surcharges due to earthquake (dynamic increment)
E_{av}	Vertical component of E_{ah}	E^*_{av}	Vertical component of E^*_{ah}
W_i	Weight of Wedge 2	$k_hW^*_i$	Horizontal inertia of Wedge 2* defined by a width of 0.5H from front of the facing
		$k_vW^*_i$	Vertical inertia of Wedge 2* which can act either up or down. It is not certain which will be critical, so it is normal to check both and use the critical case
Q_2	Any surcharge applied to the top of the reinforced soil block. If Q_2 is a live load, then it is not immediately obvious whether it should be included or not, so it is normal to check both with and without live load and use the critical case	Q^*_2	Surcharges applied to the top of the reinforced soil block have both horizontal and vertical inertia
R_i	Resistance on the base of Wedge 2	R^*_i	Resistance on the base of Wedge 2

A simple calculation is carried out to find Z_i which is the horizontal force required to stabilise the two wedges shown. Z_i is found by resolving the forces applied to Wedge 2, as given in Equation (1) for the static case. For the seismic case the procedure is the same, but with the additional forces.

$$Z_i = \Sigma H_i - \Sigma V_i \tan(\phi' - \theta_i) \qquad (1)$$

where ΣH_i = Sum of all the horizontal forces = E_{ah} in this case

ΣV_i = Sum of all the vertical forces = $W_i + Q_2 + E_{av}$ in this case

INTERNAL STABILITY: FORCE AVAILABLE TO STABILISE WEDGE

For a satisfactory design, the force Zi or Z*i calculated in the previous section must be resisted by the reinforcement or the facing or a combination of both, for all wedges investigated. This section describes how these forces may be assessed in order to find a satisfactory design. In particular, with regards to the reinforcement, the concept of the "distribution of available resistance" is introduced, which provides the basis for determining the stabilising force. To help visualise what might happen when a pair of wedges fail, the mode of failure is sketched on Figure 6.

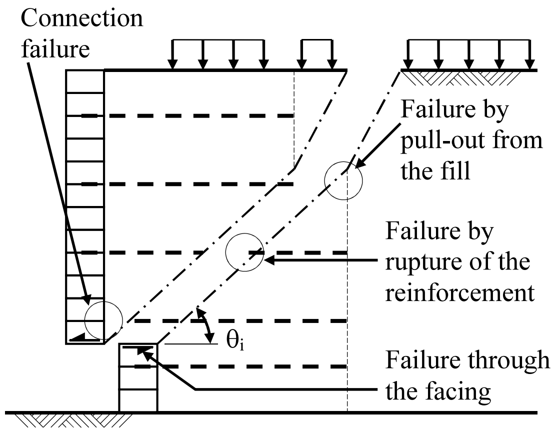

Figure 6. Resisting forces which contribute to stability of the two-part wedge

As shown on Figure 6, as the wedges slide outwards, three layers of reinforcement are involved, each with a different failure mode:

Upper	Fails due to reinforcement pulling out of the fill
Middle	Fails by rupture of the reinforcement
Lower	Fails by pulling away from the facing combined with pull-out through the fill behind the facing

In addition to the three layers of reinforcement there is also failure through the facing, in this case by sliding between two of the facing blocks, which also provides resistance. However from the point of view of the reinforcement, it is necessary to assess the available resistance at three different locations, with three different failure mechanisms. This can be done by establishing a distribution of available resistance along each layer of reinforcement as outlined below.

The envelope of available resistance is developed as shown in Figure 7 (left). This is best described as a series of steps as follows below, where the vertical axis on Figure 7 (left) is the available tensile resistance, T* (for the seismic case):

Step 1	Starting at right end of reinforcement and moving to the left, T increases according to the pull-out equation
Step 2	A maximum value is reached given by the reinforcement tensile strength
Step 3	The resistance at the facing is limited to the connection strength
Step 4	Moving to the right from the facing resistance increases according to the pull-out equation

This process results in an envelope shown by the shaded area. The shape of this envelope is quite complex, however it is readily combined with the two-part wedge method of analysis as described previously.

The parameters used on Figure 7 are defined as follows:

T^*	=	Available resistance (seismic case)
T^*_{al}	=	Tensile strength (seismic case)
T^*_{con}	=	Connection strength (seismic case)
x	=	Distance along reinforcement measured from buried end
x'	=	Distance along reinforcement measured from facing
σ_v'	=	Vertical effective stress on reinforcement
F^*	=	Pull-out resistance (seismic case) = $\alpha_p \tan\phi'$
α_p	=	Pull-out interaction coefficient
ϕ'	=	Frictional strength of fill
FS	=	Appropriate factor of safety or partial factor depending on design method

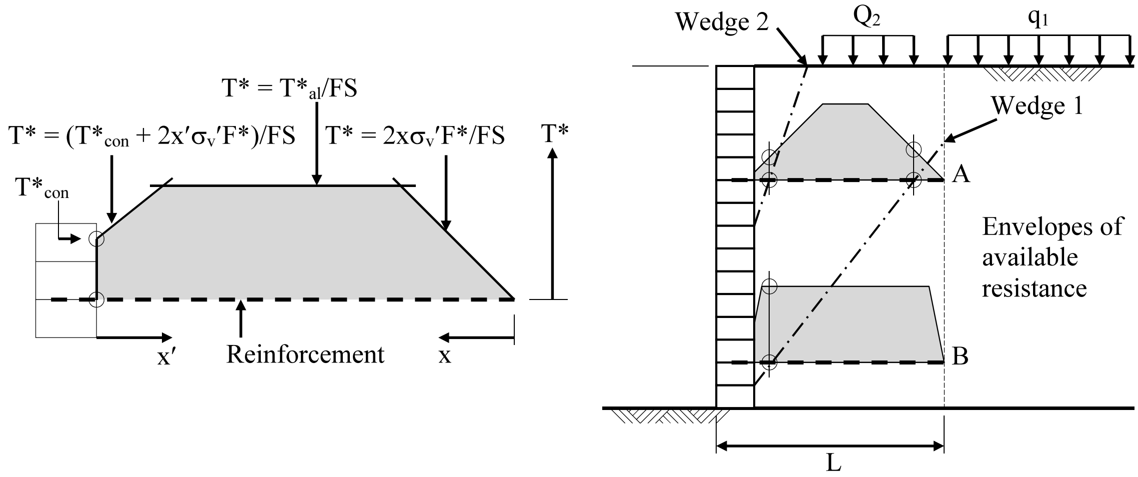

Figure 7. Definition of envelope of available resistance (seismic) and inclusion in two-part wedge analysis

An envelope of available resistance may be developed for each layer of reinforcement in a structure. Figure 7 (right) shows how these envelopes might appear. For clarity only two layers of reinforcement are shown. The sloping sections of each envelope are steeper for the lower layer of reinforcement because this slope is controlled by the vertical effective stress at the elevation of the reinforcement. This is much higher for the deeper layer.

Two wedges have been added to Figure 7 (right), and the contribution to resistance for each wedge is described as follows:

Wedge 1	Cuts Layer B near the facing, but reading up to the envelope, full tensile strength is developed. Cuts Layer A close to the buried end so that resistance comes from pull-out, and is quite low.
Wedge 2	Cuts Layer A at the same distance from the facing as Wedge 2 cutting layer B, but resistance is much smaller due to the lower connection strength and less pull-out resistance through the fill behind the facing

In the case that connection strength is relatively low near the top of the wall (as is the case with frictional connections), this analysis will result in fans of steep failing wedges near the top. When seismic forces are added, then failures of this type generally become more severe.

INTERNAL STABILITY: CONNECTION STRENGTH FOR MODULAR BLOCK WALLS

Facing systems using small pre-cast concrete blocks (typically 30 to 50kg each) have become widely used over the last 10 to 15 years, and are now one of the most popular techniques for forming the facing of reinforced soil retaining walls and are referred to as modular block facing systems. The blocks are stacked with mortar-less joints and the connection between the reinforcement and the facing is formed by laying the reinforcement between the blocks as they are installed. The strength of this connection is an important component of the wall design.

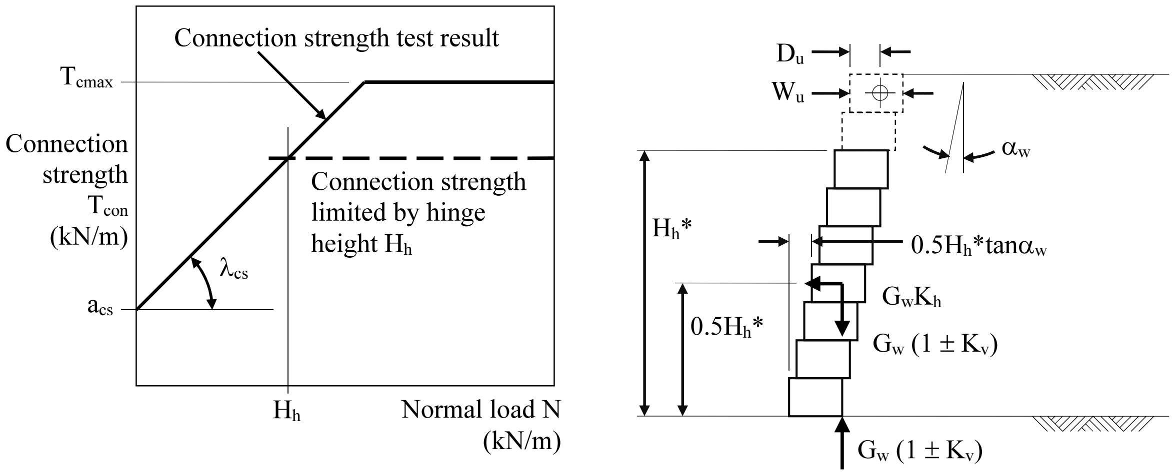

Figure 8. Results from connection testing and defining (H_h*) hinge height for the seismic case

For any particular combination of reinforcement and modular block, it is necessary to carry out testing to measure the connection strength. The normal test standard used is ASTM D6638-07, and a typical result is shown on Figure 8 (left), in terms of connection strength versus the normal load applied to the block at the level of the connection. There are two main types of connection technique:

Frictional — The reinforcement is clamped between the surfaces of the blocks above and below, and relies on friction generated by the normal load from the blocks above.

Mechanical — Some form of connector is incorporated at the point of connection, and the resulting strength is independent of the normal load above the point of connection

More commonly, the actual behaviour of a connection is a combination of both frictional and mechanical elements, and Figure 8 (left) shows such a result. In the explanation and discussion which follows, the following nomenclature is used:

G_w	=	weight of blocks within hinge height	W_u	=	width of block back-to-front
D_u	=	distance from front of block to its centre of gravity	K_h	=	horizontal seismic coefficient
α_w	=	facing angle with respect to the vertical	K_v	=	vertical seismic coefficient

The nature of the relationship shown on Figure 8 is similar to the Mohr Coulomb soil strength model, but with an upper limit. This may be written as given in Equation (2):

$$T_{con} = a_{cs} + N\tan\lambda_{cs} < T_{cmax} \tag{2}$$

Without any further restriction this formula describes the solid line shown on Figure 8 (left). However in order to interpret this information as design strength, it is necessary to introduce the concepts of "hinge height". Hinge height is the maximum height that a stack of unsupported blocks may reach before toppling, and is used to define the maximum possible normal load (N) which may be applied at the connection level. The formula for hinge height is derived by taking moments about the front lower corner of the stack of blocks (assuming that they lean backwards, towards the fill) and is given in

Equation (3). For vertical walls the hinge height is infinite (so that N would be defined by the actual height of blocks), but most modular block systems incorporate a set-back at each course so that the facing leans backwards and hinge height is finite.

$$H_h = \frac{2(W_u - D_u)}{\tan \alpha_w} \qquad (3)$$

The hinge height is equivalent to a normal load which can be plotted on Figure 8, thereby restricting the available connection strength as shown. Most published design guides also use the hinge height as given in Equation (3) for the seismic design case, although it is derived for static conditions only. However it is possible to include seismic forces in the derivation of hinge height, as shown on Figure 8 (right), with results as follows:

$$H_h{}^* = \frac{2(W_u - D_u)}{\tan \alpha_w - \dfrac{K_h}{1 \pm K_v}} \text{ for -ve } K_h \quad \& \quad H_h{}^* = \frac{2(D_u - W_u/3)}{\dfrac{K_h}{1 \pm K_v} - \tan \alpha_w} \text{ for +ve } K_h \qquad (4)$$

These expressions are examined graphically on Figure 14, for a typical modular block. The static hinge height for this block is 2.4m. However during an earthquake, as the accelerations cause the facing to rock backwards and forwards, the hinge height varies dramatically. It can be seen that the vertical acceleration has only a small influence on the calculated hinge height. It is clear that the hinge height under seismic conditions can become much less than the static value. For design purposes $H_h{}^*$ is calculated for both $-K_h$ and $+K_h$, and the lower value is used.

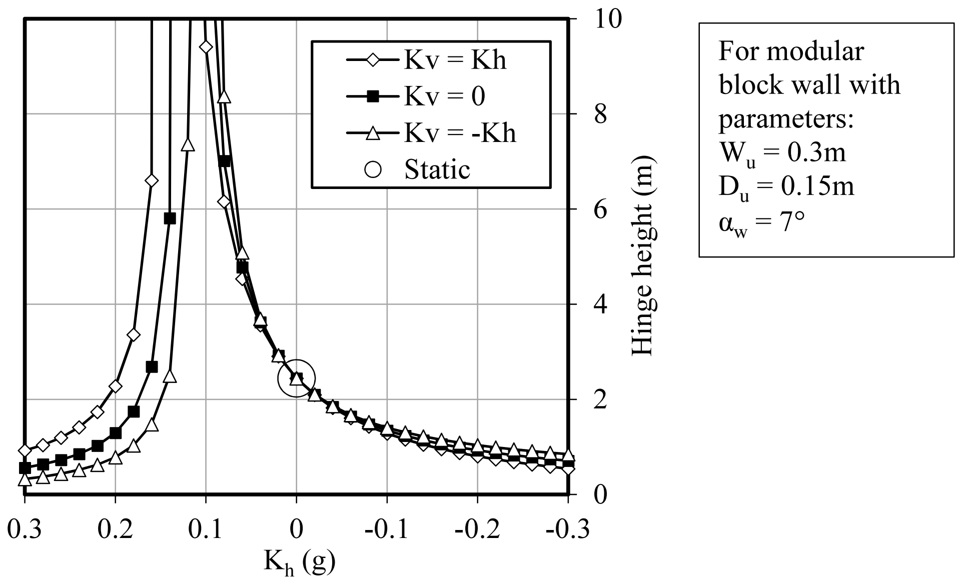

Figure 9. Hinge height versus acceleration K_h

As noted on Figure 6 and the explanatory text, resistance to wedge failure is also provided by sliding failure through the facing, termed the interface resistance, T_{if}. Testing is also required to establish the interface shear resistance parameters (a_u and λ_u), with similar behaviour as shown on Figure 8 (left), but without the upper limit. This is defined in Equation (5) as:

$$T_{if} = a_u + N\tan\lambda_u \qquad (5)$$

The value of the normal force N in the expression for T_{if} is also limited by the hinge height, and under seismic loading will also be subject to the relationship illustrated in Figure 9.

BACK-ANALYSIS OF THE CHI CHI MODULAR BLOCK WALLS

The calculation procedure described above is not suitable for manual calculations, although it may be set up in a spreadsheet. Over the last 12 years the authors company has developed a computer program called **TensarSoil®** for the design of reinforced soil retaining walls and steep slopes. Many design methods are included in the program, several of which make use of the two-part wedge calculation procedure as outlined in this paper. One such method is the two-part wedge calculation procedure combined with the partial factors and recommendations in AASHTO/LRFD.

This method has been used to back-analyse the modular block walls shown in Figure 1. Full details for these walls are given by Huang & Tatsuoka (2001), and are summarised in Table 3.

Table 3. Design parameters for the modular block walls shown in Figure 1

Wall facing	Height (exposed) 3.2m for Site 1 and 2.65m for Site 3, face angle 11°
Fill properties	$c' = 0$ kPa, $\phi' = 34.4°$, $\gamma = 18$ kN/m^3 (foundation assumed competent)
Modular blocks	$W_u = 0.3$m, $L_u = 0.46$m, $H_u = 0.2$m, $D_u = 0.15$m, $G_u = 38$ kg $a_{cs} = 0$ kN/m, $\lambda_{cs} = 30°$, $a_u = 1$ kN/m, $\lambda_u = 30°$
Reinforcement	$T_{ult} = 75$ kN/m, L = 2.7m @ 0.8m spacing Site 1, L = 2.0m @ 0.6m spacing Site 3
Earthquake	Peak ground acceleration $A_h = 0.44$g, $A_v = \pm 0.19$g

Analysis shows that both walls were stable under static conditions, and external stability was satisfactory for both static and seismic loading. However differences are seen in the internal stability check in the seismic design case, as shown in Figure 10. Figure 10 shows part of the **TensarSoil®** interface, which is a cross-section of each wall, where red lines indicate wedges which are potentially unstable. For the wall at Site 1 (on the left) there are a large number of steep red wedges, suggesting that a collapse of the facing is likely to take place, although the majority of the soil mass is stable. For the wall at Site 3 (on the right) no such wedges are seen. In both cases, the seismic loading is with vertical acceleration upwards (there are no red wedges for vertical acceleration downwards).

Further insight into this behaviour may be seen in Figure 11, which shows the distribution of available resistance along the lowest layer of reinforcement at Site 1, for both static loading and seismic with vertical acceleration upwards. These diagrams are as per Figure 7, and appear quite different. In the static case (right-hand diagram) the permitted tensile strength is based on the long term reinforcement strength, and the connection strength is small, due to the inclination of the wall, at about 3 kN/m. However in the seismic case on the right, the connection strength is negligible because the connection is purely frictional, and the seismic hinge height is very small (actually about 0.3m). There is no horizontal section to this diagram is because for the seismic case reinforcement strength is based on short term tensile strength, so that with the short length of the reinforcement, pull-out from the fill and from the facing meet in the middle at a permissible load less than the tensile strength.

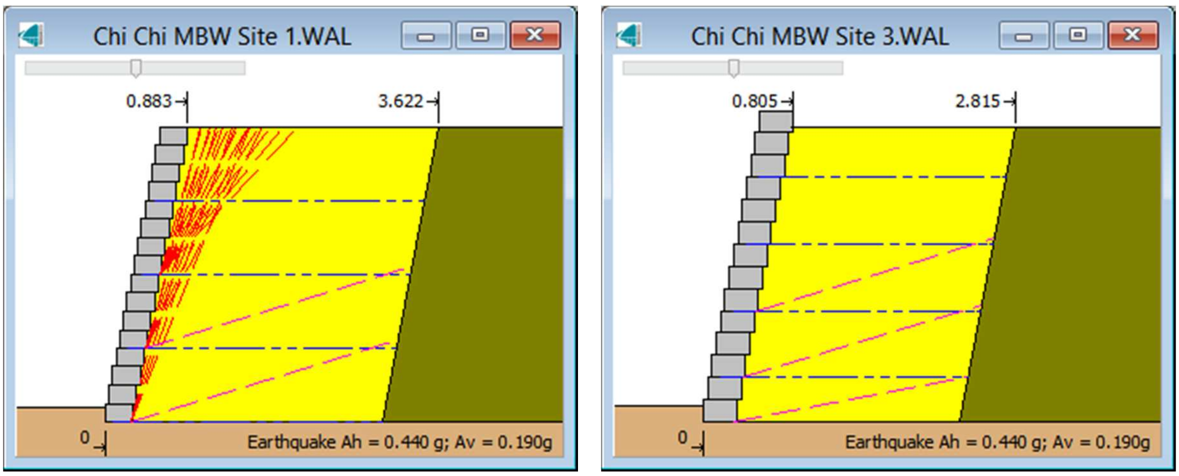

Figure 10. Images from the internal stability analysis of the walls at Sites 1 and 3 using **TensarSoil®**

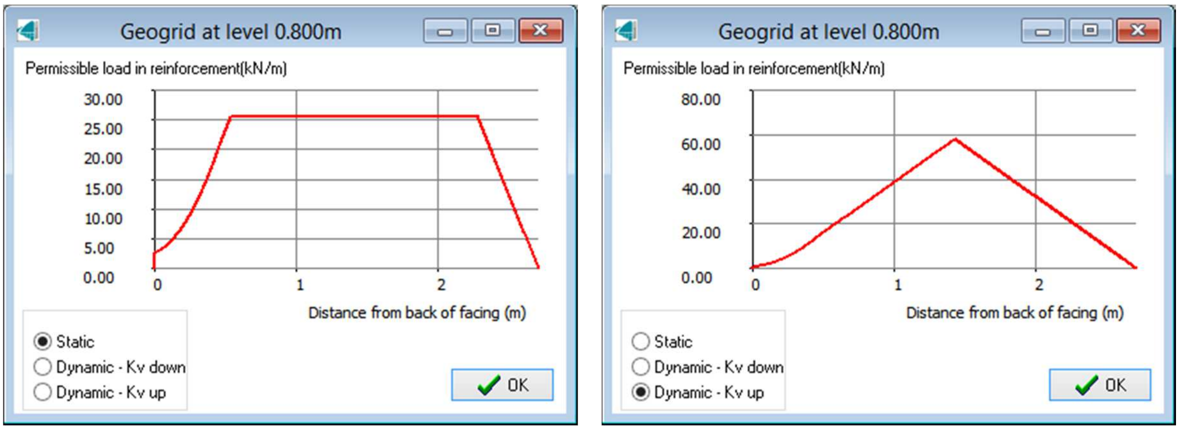

Figure 11. Distribution of available resistance for the lowest reinforcement layer at Site 1

CONCLUSIONS

This paper describes a two-part wedge method of analysis of the internal stability of reinforced soil retaining walls, which is based on complete mechanisms in which all forces are included (also seismic forces when required), and which uses a search procedure to establish the critical case. The contribution of reinforcement is defined in terms on an envelope of available resistance, which takes into account the connection strength between the reinforcement and the facing. The resistance from sliding between facing blocks is also included in the analysis in the case of walls with modular block facings. Using a computer program **TensarSoil®** two retaining walls shaken by the Chi Chi earthquake in 1999 were analysed, and the obsevred behaviour (one wall collapsed, whilst the second remained stable) could be anticipcated based on the two-part wedge method.

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