THE INTERNAL STABILITY OF REINFORCED SOIL RETAINING WALLS, INCLUDING CONNECTION STRENGTH FOR MODULAR BLOCK SYSTEMS AND SEISMIC LOADING

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ABSTRACT

Polymer geogrid reinforced structures have performed very well in strong earthquakes, although some failures occurred during the Chi-Chi earthquake, and in the case of modular block walls these were partly attributed to low connection strength between the reinforcement and the facing. Calculation procedures for both internal stability under seismic conditions and facing connection strength as given in the NCMA and FHWA manuals are discussed, and inconsistencies are highlighted. An alternative design procedure is described based on a two-part wedge technique, in which a large number of potential failure mechanisms are checked, thereby reducing the number of assumptions required. This method has been adapted to take account of seismic forces and facing connection strength. A simple worked example illustrates how this method would predict instability near the top of a reinforced soil retaining wall when moderate earthquake forces are present, and connection strength is low. Two case histories further illustrate the application of the method to actual situations.

1.0 INTRODUCTION

Observations of the condition of polymer geogrid reinforced soil structures after strong earthquakes have demonstrated that they have very good seismic resistance, and have survived very high seismic forces with minimal damage (for example Tatsuoka et al, 1996 and Nishimura at al, 1996). The Chi-Chi earthquake, which took place in the early hours of 21st September 1999, caused extensive damage in central Taiwan. Retaining structures and slopes were severely affected and many failures occurred. A large number of geosynthetic reinforced soil structures were located within the epicentral area. The majority of these structures survived intact or with minimal damage, although failures occurred, some of which have been the subject of detailed investigation (for example Huang, 2000 and Huang & Tatsuoka, 2001). Some modular block retaining walls collapsed and a major contribution to the observed mode of failure appeared to be low connection strength between the facing and the geosynthetic reinforcement.

This paper examines the design of reinforced soil retaining structures (face angle steeper than 70º), looking principally at internal stability analysis, and in particular at the modeling of seismic forces and connection strength in the case of modular block walls. A brief summary is given of current practice, outlining some of the assumptions made to take into account earthquake forces and connection strength. Alternative proposals are then presented based on a two-part wedge method of analysis. The main aim of this approach is to reduce the number of simplifying assumptions required by analysing a large number of potential failure mechanisms.

This paper is a further development from a paper published at the International Geosynthetics Engineering Forum (IGEF) held in Taipei in November 2001 (Dobie M J D, 2001).

2.0 CURRENT DESIGN PRACTICE

Methods of designing reinforced soil retaining structures have been published for more than 20 years. There are many governmental and national design manuals or standards published today, which all have a common basic approach. Design of structures is divided into two main steps:

- **External stability analysis**: the reinforced soil mass is treated as a gravity retaining wall, and well established design methods are applied. Differences exist in some of the assumptions made (for example choice of the method of earth pressure calculation), but generally these are not of major significance. The calculation results in the dimensions required for the reinforced soil block, which defines the length of the soil reinforcement.
- **Internal stability analysis**: the internal stability of the reinforced soil mass is examined in order to find the required grade (strength) and spacing of the soil reinforcement. Further specific stability checks may be required, for example for facing stability.

In most published design methods, internal stability calculations are carried out using the "tie-back wedge" method of analysis. For the following discussion, two published design manuals which use the tie-back wedge are considered:

- **NCMA** manual (Simac et al, 1993 as well as later editions, and seismic supplement, Bathurst, 1997)
- **FHWA** manual (Elias & Christopher, 1997)

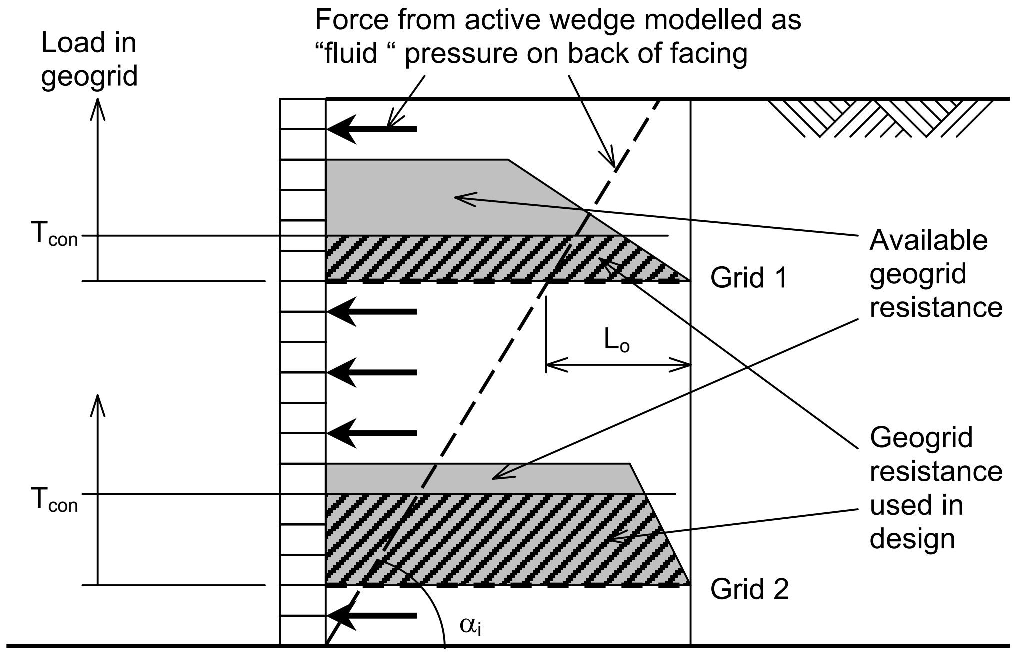

Figure 1 Assumptions made in tie-back wedge method of internal stability analysis

Both methods include seismic design as well as modular block facings, so are directly relevant to this topic. Figure 1 outlines the main assumptions used in the tie-back wedge internal stability calculation, which are summarised and discussed below. It should be noted that Figure 1 only shows two layers of soil reinforcement for clarity.

2.1 Internal static failure mechanism and resulting forces

In order to calculate forces applied to the reinforcement, only one internal failure mechanism is considered, namely active earth pressure. This is represented by a wedge of soil inclined at α_i to the horizontal. FHWA assumes Rankine conditions, so that $\alpha_i = 45 + \phi/2$ (for a vertical wall with horizontal upper surface) where ϕ is the internal friction angle of the soil. NCMA assumes Coulomb conditions, including interface friction between the facing and the soil mass, in which case α_i is given by a far more complex expression, but is less than the Rankine value. The resulting horizontal force is assumed to be applied to the back of the facing as a fluid pressure. This permits calculation of the tensile force in the reinforcement. Pullout is also checked using this failure mechanism. For example for Grid 1, the anchorage length L_o is checked to ensure that the available pullout resistance is greater than the tensile load in the grid.

The assumptions described above are fundamentally incorrect. Both the Rankine and Coulomb calculations of earth pressure assume a homogenous isotropic soil mass, for example, as might exist in sand fill behind a cantilever wall. Layers of reinforcement change this condition, and alter the forces applied to the failing wedge. This is not a serious problem for calculating maximum tensile load in the reinforcement, provided it is arranged such that its distribution of resistance is similar to the fluid pressure distribution. However this is difficult to arrange in practice. Furthermore the presence of the reinforcement layers means that the active wedge can no longer be assumed to represent the critical wedge [NB: in some versions of the tie-back wedge method a large number of wedges are used to check the pullout failure condition].

The reinforcement layers carry the internal soil loads by building up friction along their lengths. Hence the pressure applied to the back of the facing will be considerably less than active pressure. However the assumptions described above require that the soil flows past the reinforcement like a fluid, so that the full active pressure is applied at the back of the facing. NCMA justifies the use of interface friction on the back of the facing as a method of reducing the applied pressures so that they become closer to observed behaviour. However this would imply relative vertical movement between the fill and the facing, which would not be desirable.

2.2 Internal failure mechanism and resulting forces under seismic conditions

For internal stability analysis under seismic conditions, an additional horizontal load is calculated to take into account the inertia force which occurs during an earthquake. This is called the dynamic increment in NCMA. The methods used to calculate the dynamic increment, and vertical distribution of that force, are quite different in the FHWA and NCMA methods.

In FHWA, under seismic conditions, the critical active wedge is still assumed to be that given by Rankine, as shown on Figure 1, with $\alpha_i = 45 + \phi/2$ (for a vertical wall with horizontal upper surface). The dynamic increment is taken as the weight of the active wedge multiplied by the horizontal earthquake coefficient. This force is distributed in proportion to the anchorage length of the geogrid at each level. As can be seen in Figure 1, the anchorage length of Grid 2 is much longer than that of Grid 1, so that Grid 2 will carry much more load than Grid 1. Therefore, for walls with uniform reinforcement length, this will result in a distribution of the dynamic increment which increases from the top of the wall downwards.

NCMA calculates the dynamic increment as:

[Dynamic increment] = [force from Mononobe-Okabe] – [force from Coulomb]

The Mononobe-Okabe formula is similar to Coulomb, but incorporates the effects of seismic inertia forces. The dynamic increment is assumed to be applied 0.6H above the base of the wall. The additional force in each layer of reinforcement is calculated by the "contributory area approach", so depends on vertical spacing. For uniform reinforcement spacing, this will result in a distribution of the dynamic increment which increases from the base of the wall upwards. This is opposite to FHWA.

In both methods, the pullout calculation is carried out using L_o based on the active wedge derived from static conditions, as shown on Figure 1. However under seismic loading, wedge angle α_i of the critical wedge will reduce as seismic forces increase, implying that L_o as shown on Figure 1 would decrease as seismic forces increase.

This brief summary demonstrates that there is confusion about the assumptions required to calculate both the magnitude and the distribution of the dynamic increment of force under seismic conditions. Furthermore there may be concern that the critical wedge for static conditions is also being used as the critical wedge under seismic conditions, and that no attempt is made to check other failure mechanisms. For the same reason, critical anchorage length may be much shorter than assumed under static conditions.

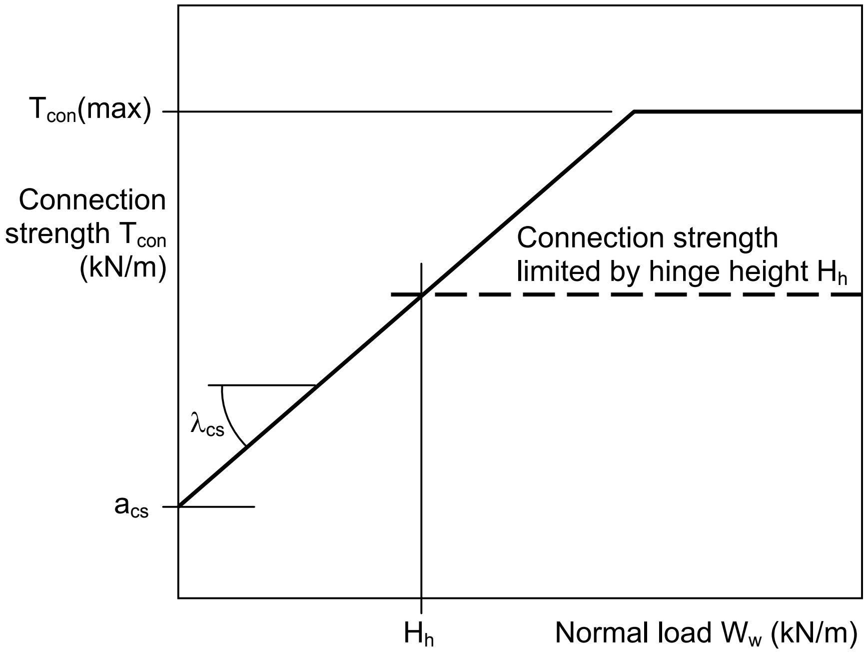

Figure 2 Relationship of connection strength to normal load in modular block walls

2.3 Facing connection strength in modular block walls

Connection strength between modular block facings and geosynthetic reinforcement is determined by carrying out connection strength tests. The method of test normally used is that given in Appendix A of the NCMA Manual. The test results are interpreted to give an equivalent Mohr-Coulomb relationship, as shown on Figure 2:

$$T_{con} = a_{cs} + W_w \tan\lambda_{cs} \leq T_{con}(\max)$$

where
- T_{con} = connection strength (kN/m)
- a_{cs} = connection strength at zero normal load (kN/m)
- λ_{cs} = slope of T_{con} versus W_w (deg)
- W_w = normal load from wall (kN/m)
- T_{con}(max) = Maximum connection strength (kN/m)

T_{con}(max) is the upper limit, or maximum connection strength, generally governed by rupturing of the reinforcement, rather than pull-out. T_{con}(max) is normally less than the tensile strength of the grid measured by wide width testing (eg: test method ISO 10319) due to the inevitable imperfections which will occur due to clamping the reinforcement between concrete blocks, rather than in high quality tensile testing clamps. In addition, the time to failure in the connection test may well be slightly longer than in the wide width tensile test.

The NCMA connection test is a short term test, which results in a short term strength. The NCMA manual requires that both peak load and load at 20mm deformation are reported, so that the Mohr-Coulomb relation can be derived for both conditions. The parameters for 20mm deformation are used as a serviceability check with a lower safety factor. FHWA divides connection behaviour into two modes: reinforcement pullout and reinforcement rupture. In the case of rupture, a creep reduction factor is applied to T_{con} to derive the design connection strength.

A further restriction on connection strength is given by the “hinge height”, H_h. The hinge height is the maximum height a stack of unsupported facing blocks may reach before toppling. The formula for hinge height is derived by taking moments about the lower front corner of the stack of facing blocks. For vertical walls, hinge height is infinite, but most modular block facings incorporate a set-back on each course, resulting in a finite hinge height. The effect of hinge height on calculated connection strength is shown on Figure 2. In NCMA and FHWA hinge height for seismic design is the same as the value used for static design.

On Figure 2, the normal load axis is equivalent to wall height. Therefore connection strength will be minimum at the top of the wall, increasing with depth below the top. The effect of this distribution of connection strength is included on Figure 1. For the two grids shown, the shaded areas represent the distribution of available load in each reinforcement layer, assuming that connection strength is equal to the long term design strength of the grid (ie: 100% efficient connection). Immediately behind the facing, and moving to the right, the load is constant (failure by rupture of the grid). It then decreases towards the right end (failure by pullout), reaching zero at the free right hand end. If connection strength is less than long term design strength, then connection strength will define the available load in the reinforcement immediately behind the facing. However, due to the simplistic method of analysis used in the tie-back wedge method (ie: use of a single failure mechanism), the connection strength effectively becomes the long term design strength along the entire length of the reinforcement up to the point where pullout near the right hand end takes over. Therefore, the envelope of available load in the reinforcement is given by the hatched areas for the two geogrid layers shown. Near the top of the wall, this could result in very inefficient use of the reinforcement (as indicated on Figure 1).

The above discussion appears to be highly critical of the tie-back wedge method. However its strong point is that it has been in use for more than 20 years and huge numbers of structures have been successfully designed and built using it. For static design of walls with high strength connections, the simplifying assumptions are not significant provided the layout of reinforcement results in a similar distribution of resistance to the fluid active pressure distribution. However the increasing use of modular block walls, coupled with some instances of poor performance in earthquakes, justifies a reassessment of the internal stability calculation procedure.

3.0 TWO-PART WEDGE METHOD OF INTERNAL STABILITY ANALYSIS

The Deutsches Institut für Bautechnik Certificate Z 20.1-102 (referred to here as DIBt) includes a procedure for static design of reinforced soil retaining walls and steep slopes based on a two-part wedge method of calculation (see Figure 3).

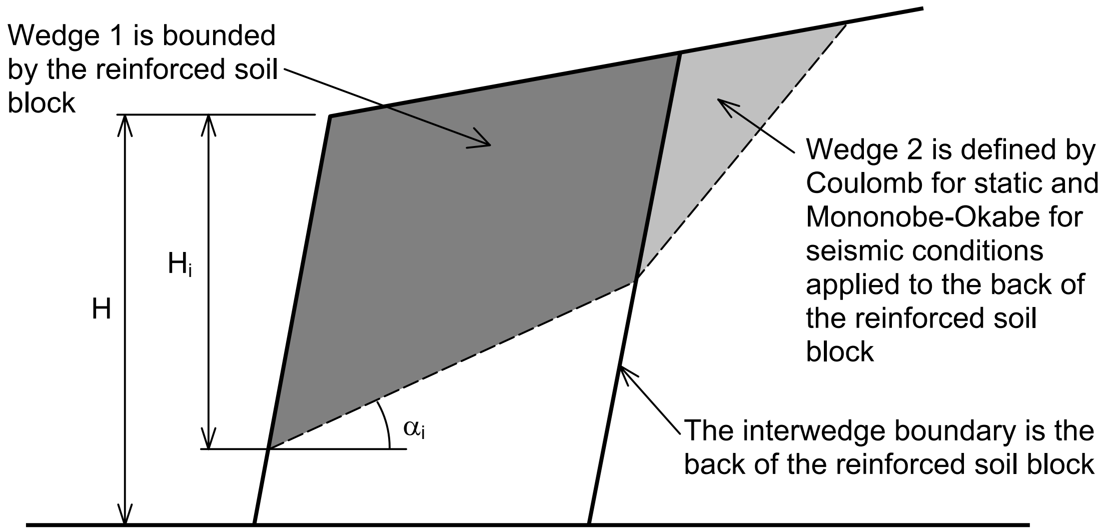

Figure 3 Basis of the DIBt method of calculation – the two-part wedge

Wedge 1 is bounded by the front and the back of the reinforced soil block, and Wedge 2 is the earth pressure force applied to the back of the block. For static conditions earth pressure forces are calculated according to the Coulomb method of calculation. The interwedge boundary is defined by the back of the reinforced soil block. All modes of failure, both internal and external, can be modelled by the two part wedge described above, by adjusting H_i and α_i. External stability is checked by making H_i = H, and α_I =0, so that sliding, overturning and bearing capacity may be checked in the same way as most other design methods.

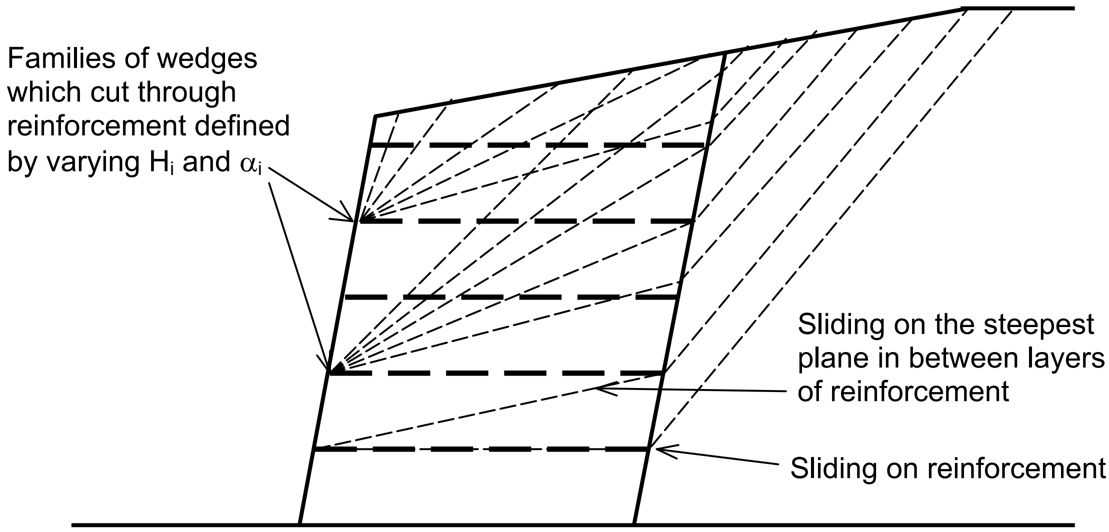

Figure 4 Examples of internal failure mechanisms checked in DIBt method

The major difference between DIBt and many of the tie-back wedge methods comes in the internal stability check. In DIBt all internal stability checks are made with the reinforcement (and the implied forces) present. No assumptions are made about the critical wedge, but instead, a large number of potential failure mechanisms are checked. Examples of internal failure mechanisms are shown on Figure 4, and fall into three categories:

- Sliding along layers of reinforcement (also checked in NCMA),
- Sliding on the steepest inclined plane in between layers of reinforcement,
- Families of wedges which cut through reinforcement (carried out at the base of the wall and various points higher up, with α_i at 3° intervals).

It is beyond the scope of this paper to present a detailed description of the DIBt method of design, although this is available (Tensar International, 2001). DIBt includes neither seismic design nor modular block connection strength. However the two-part wedge method is easily adapted to look at both these design aspects. These adaptations are described in the following sections, principally in terms of ultimate limit state design.

3.1 DIBt method adapted to include seismic design forces

Assumptions concerning seismic coefficients and forces have been taken mainly from NCMA. This is referred to as pseudo-static design, because the transient forces created during an earthquake are modelled as static forces, by applying horizontal and vertical accelerations expressed as a proportion of gravity. The main assumptions made are summarised below.

Notation	A_h	=	horizontal ground acceleration
	A_v	=	vertical ground acceleration
	$k_h(ext)$	=	horizontal seismic coefficient for mechanisms which do not cut reinforcement
	$k_v(ext)$	=	vertical seismic coefficient for mechanisms which do not cut reinforcement
	$k_h(int)$	=	horizontal seismic coefficient for mechanisms which cut reinforcement
	$k_v(int)$	=	vertical seismic coefficient for mechanisms which cut reinforcement
	ϕ'_{cv}	=	soil shear strength under constant volume conditions

- Both horizontal and vertical ground accelerations are included (A_h and A_v).
- For mechanisms which do not cut through reinforcement, a reduced value of the peak ground acceleration is used ($k_h(ext) = 0.5A_h$). This means that the design maximum force may be exceeded for brief periods during the seismic event, resulting in some displacement. Because

reinforcement is not intersected by these mechanisms, the resulting displacements cannot result in rupture or excessive loading of the reinforcement. Wood & Elms (1990) give guidance on the assessment of likely displacement, and make the important point that the critical acceleration, k_h, should be calculated using the maintainable shearing resistance of the soil at large strain (or ϕ'_{cv}). This is consistent with the DIBt method which uses the constant volume definition of soil shear strength in stability calculations. In Figure 4, the internal mechanisms of sliding on reinforcement and sliding on an inclined plane in between reinforcement would both fall into this category, as would external stability checks.

- For mechanisms which cut through reinforcement layers it would not be acceptable to permit large displacements because this would result in either rupture or extensive distortion of the reinforcement. Therefore for these mechanisms an allowance is made for some amplification of the peak ground acceleration ($k_h(\text{int}) = (1.45 – A_h)A_h$). In Figure 4, the families of wedges which cut through reinforcement would fall into this category.
- Temporary surcharges (live loads or traffic) are assumed to be absent during a seismic event.
- Pseudo-static analysis is limited to $A_h = 0.29$. For higher accelerations, dynamic analysis is required. The two assumptions which follow are both intended to reduce the conservatism inherent in pseudo-static design, in which it is assumed that all forces are applied at the same time at all points in the structure despite the fact that seismic loading is transient, and of very short duration.
- Earth pressure force applied to the back of the reinforced soil block is calculated using Mononobe-Okabe (Wedge 2). The force calculated using Coulomb for the static condition is subtracted from this to find the dynamic increment. In stability calculations the dynamic increment is reduced by 50%.
- Inertia forces acting on the reinforced soil block due to seismic accelerations are assumed to be applied only to the front part of the block, given by a width of 0.5H.
- Safety factors for sliding, overturning and bearing capacity under seismic conditions are taken as 75% of the values required under normal static conditions. For rupture and pullout of reinforcement, the full static values are retained for seismic conditions.

3.2 DIBt method adapted to include modular block connection strength

Figure 1 shows one of the many internal failure mechanisms which would be checked by the DIBt method. The horizontal force required to prevent the wedge from failing is compared to the combined resistance of all the reinforcement layers at the points where they intersect the failure plane. As shown there are only two layers, and the shaded diagram above each shows the distribution of available resistance provided by the reinforcement. The horizontal part of each diagram is the factored design strength of the reinforcement. For static conditions this is given by:

$$P_{des} = \frac{P_c}{f_m f_d f_e FS_r}$$

The sloping part of each diagram is the pullout resistance. For static conditions this is given by:

$$P_{des} = \frac{2x_r \sigma'_v \alpha_p \tan\phi'}{FS_{po}}$$

For seismic design both expressions are adapted as follows:

$$P_{des} = \frac{T_{ult}}{f_m f_d f_e FS_r} \quad \text{and} \quad P_{des} = \frac{2x_r \sigma'_v (1 \pm k_v(\text{int}))\alpha_p \tan\phi'}{FS_{po}}$$

where	P_{des}	=	design strength
	P_c	=	rupture strength at required design life derived from creep tests
	f_m	=	partial safety factor for material properties
	f_d	=	partial safety factor to allow for the effects of construction activities
	f_e	=	partial safety factor to allow for the effects of environmental conditions
	x_r	=	distance along the reinforcement layer measured from the right free end
	σ'_v	=	mean vertical effective stress along the anchored length
	α_p	=	coefficient of interaction for pullout
	ϕ'	=	angle of shearing resistance of the fill material
	FS_r	=	overall factor of safety against rupture
	FS_{po}	=	overall factor of safety against pullout
	T_{ult}	=	short term tensile strength

However if connection strength is less than the design strength of the reinforcement, then a reduction is required in the resistance available from the layer of reinforcement, especially close to the back of the wall, as per Grid 2. One limitation of the tie-back wedge method discussed above was that the assumption of a single mode of failure requires that connection strength must be assumed as the long term design strength along the full length of each layer of reinforcement (ie: the hatched areas on Figure 1). This is potentially very conservative.

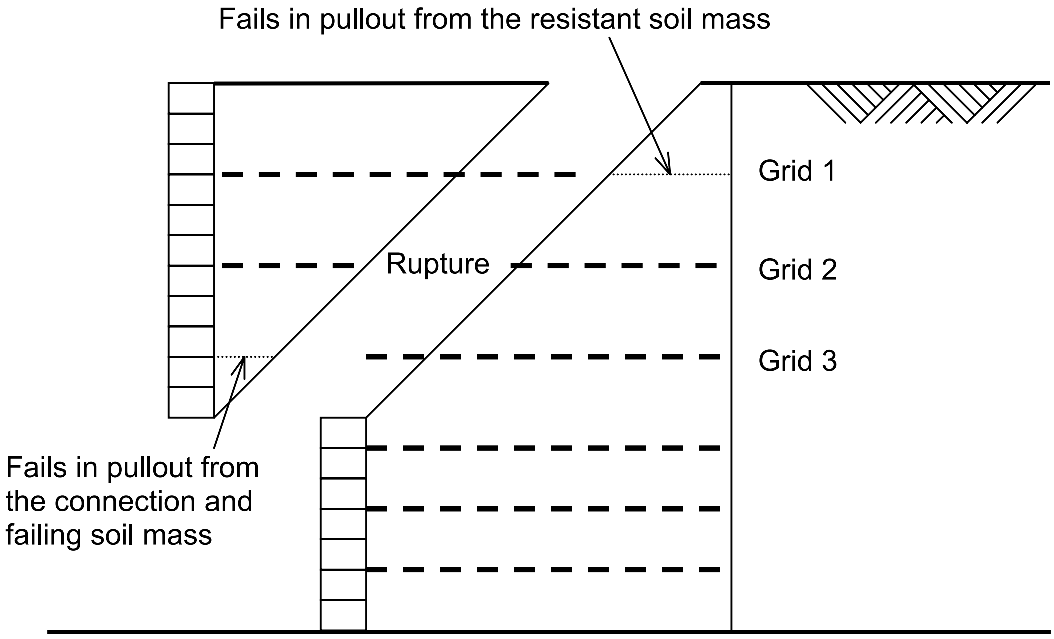

Figure 5 Typical failure for wedge with low connection strength at facing

Using the DIBt two-part wedge method with its multiple checks of many failure mechanisms, it is possible to use a more realistic distribution of resistance along each layer of reinforcement. This can be visualised on Figure 5, which shows a typical failure wedge intersecting three layers of reinforcement in the case where facing connection strength is relatively low. Grid 1 pulls out of the resistant soil mass and Grid 2 ruptures somewhere near the middle. However Grid 3 fails by pulling out of the connection and the failing soil mass. These three modes of failure can all be modelled by making a small adjustment to the distributions of available resistance in each grid layer shown on Figure 1.

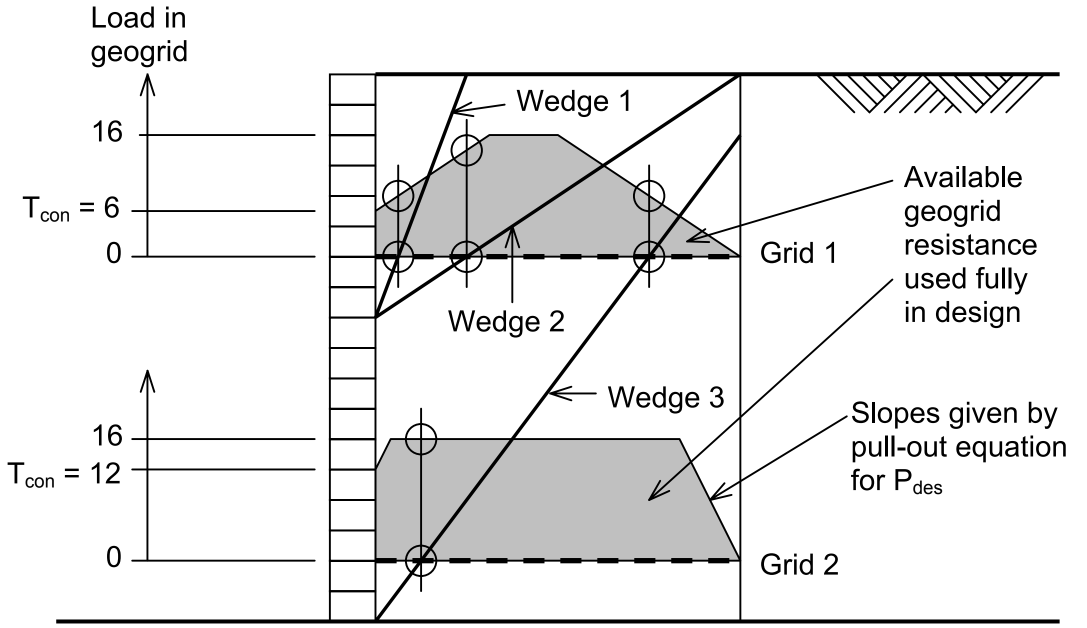

Figure 6 Realistic modelling of distribution of resistance along reinforcement layers

Figure 6 is the same as Figure 1, except that the envelopes of available geogrid resistance have been adjusted to take into account the connection strength with the facing. The left hand end of each envelope is given by T_{con}, and the slope to the right of this point is given by the pullout equation. The three wedges drawn demonstrate various modes of failure. Wedges 1 and 2 both intersect Grid 1, and both result in a connection/pullout failure at the left end. Wedge 3 intersects both reinforcement layers, with Grid 2 failing in rupture while Grid 1 pulls out from the right.

3.3 Assessment of hinge height

The concept of "hinge height" is discussed in Section 2.3 above. It is defined as the maximum height which the unsupported stack of facing blocks may reach before toppling, and in the static case it is found by taking moments about the front of the lowest block. In NCMA and FHWA, the static hinge height is also applied for the seismic design case. However, the method of calculation can be adjusted to take into account seismic forces. This is shown on Figure 7.

Notation
- H_{hs} = hinge height under seismic loading (m)
- G_m = mass of facing blocks within hinge height
- k_h = horizontal seismic coefficient
- k_v = vertical seismic coefficient
- α_w = inclination of wall facing measured from vertical
- d_m = distance from front of block to its centre of gravity (including any infill)
- w_m = width of block from back to front

Taking moments about "O", and rearranging the terms gives the following expression for hinge height, including the effect of seismic forces:

$$H_{hs} = \frac{2(w_m - d_m)}{\tan\alpha_w - \dfrac{k_h}{1 \pm k_v}}$$

If $k_h = k_v = 0$, then this reduces to the expression given in NCMA and FHWA for H_h. It should be noted that this expression is only applicable for the case of $k_h < 0$, that is when the facing is pushed backwards towards the fill. For positive k_h it is necessary to take moments about the opposite corner of the lowest block to look at the case of the facing being pushed away from the fill. In this case it is considered necessary to ensure that tension is not permitted under the toe of the block,

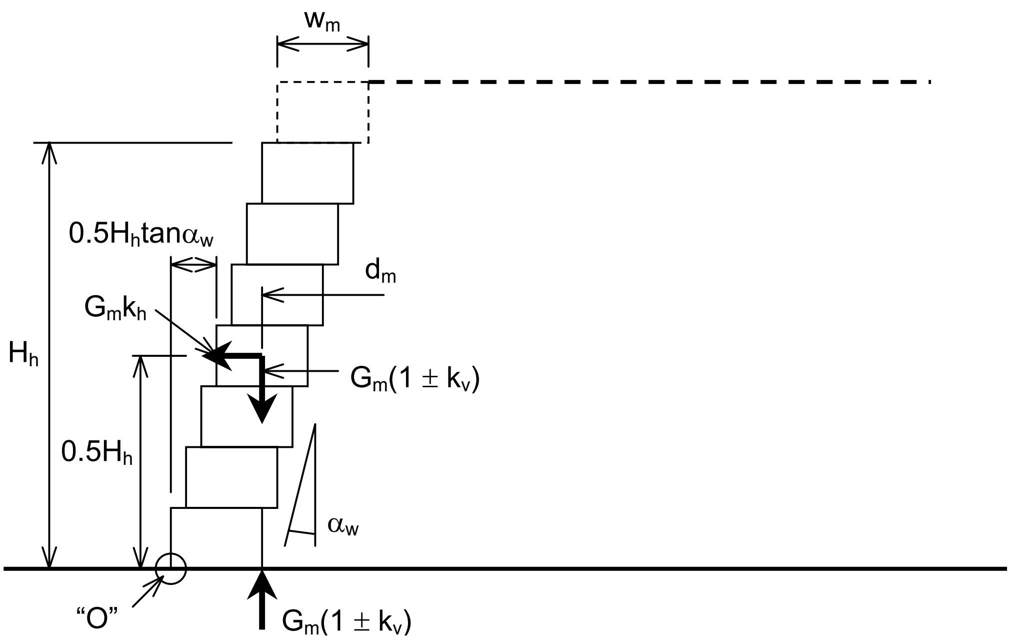

Figure 7 Derivation of hinge height

so the position of the reaction force is taken to be $2w_m/3$ from the back of the lowest block. In this case hinge height is given by:

$$H_{hs} = \frac{2(d_m - w_m/3)}{\frac{k_h}{1 \pm k_v} - \tan\alpha_w}$$

These expressions are examined graphically on Figure 8 for a typical modular block with w_m = 0.3m, d_m = 0.15m, α_w = 7°.

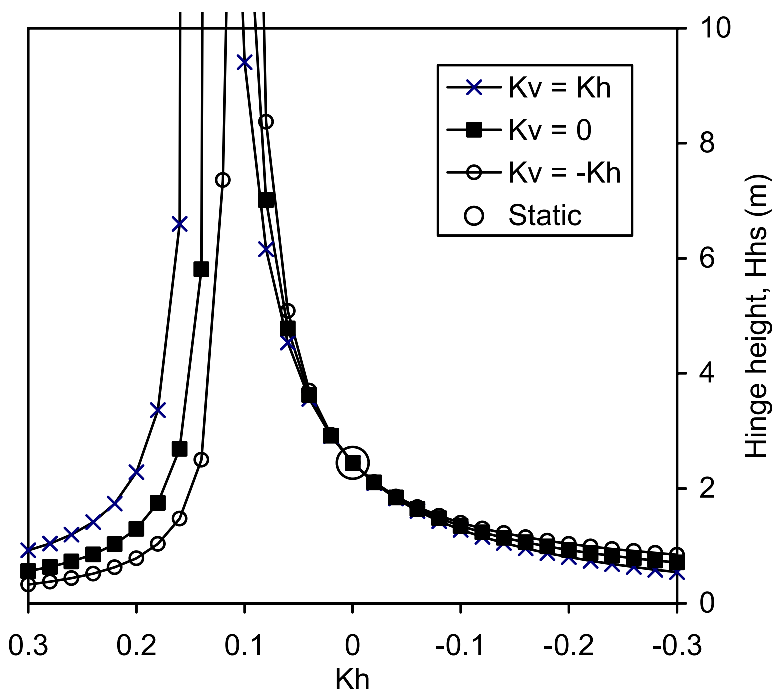

Figure 8 Relationship between hinge height and horizontal acceleration

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 vertical acceleration has only a small influence. k_h = 0.29 is the maximum value recommended for pseudo-static design, so Figure 8 represents the full range of design conditions likely to be encountered. The dimensions chosen are typical of many modular blocks, and it is clear from Figure 8 that hinge height under seismic conditions can be very small, certainly less than 1.0m. The situation is worse for vertical or near vertical walls for positive k_h.
For design, H_{hs} is calculated for $+k_h$ and $-k_h$, and the lower value is used. To reduce the likelihood of large deformations at the wall facing, it is proposed that the higher k_h(int) definition of seismic design coefficient is used.

3.4 Assessment of connection strength

Section 2.3 presents the method of determining connection strength given in NCMA and FHWA. The approaches are similar, although FHWA requires that, for connections which fail in rupture, a creep reduction factor should be applied to the measured connection strength. No distinction is made between static and seismic design conditions. The NCMA test is a short term test, so it measures a short term strength. The result are expressed in terms of a Mohr Coulomb relationship:

$$T_{con} = a_{cs} + W_w \tan\lambda_{cs} \leq T_{con}(max)$$

This is reproduced on Figure 9.

Notation			
	T_{con}	=	connection strength (kN/m)
	a_{cs}	=	connection strength at zero load [mechanical contribution] (kN/m) used for seismic conditions
	a_c	=	connection strength at zero load [mechanical contribution] (kN/m) used for static conditions
	λ_{cs}	=	slope of S_c versus W_w [frictional contribution] (deg) used for static and seismic conditions
	W_w	=	normal load from wall (kN/m)
	$T_{con}(max)$	=	maximum connection strength (kN/m) used for seismic conditions
	T_{cs}	=	maximum connection strength (kN/m) used for static conditions
	RF_{cr}	=	creep reduction factor
	H_h	=	hinge height under static conditions (m)
	H_{hs}	=	hinge height under seismic conditions(m)

It is proposed that connection test data should be interpreted as follows:

- a_{cs} is the mechanical contribution and λ_{cs} is the frictional contribution to connection strength under short term loading
- For static design it is assumed that the mechanical contribution and maximum connection strength are reduced by the creep reduction factor, but the frictional contribution is unaffected. This gives: $a_c = a_{cs}/RF_{cr}$ and $T_{cs} = T_{con}(max)/RF_{cr}$ for static design
- For seismic design the parameters measured from testing are used directly and normal load is given by $W_w(1 \pm k_v(int))$
- The resulting design envelopes are modified by the appropriate value of hinge height

The resulting envelopes of design connection strength for both static and seismic conditions are shown on Figure 9. The appropriate value of T_{con} is taken from these relationships to complete the envelopes of available reinforcement resistance shown on Figure 6.

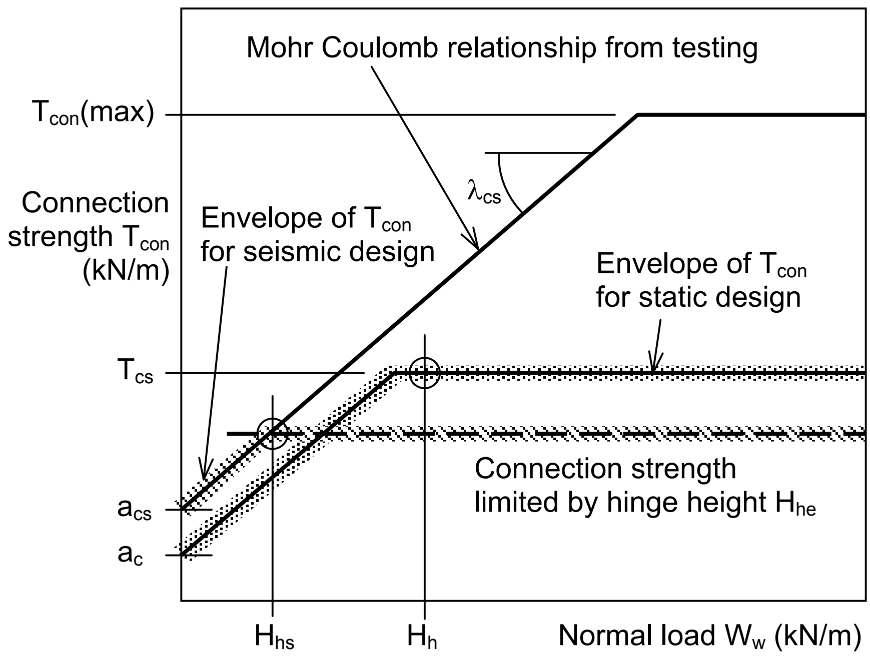

Figure 9 Definition of connection strength for static and seismic conditions

4.0 WORKED EXAMPLE

The design method described above is illustrated below using a simple worked example. The wall has been designed for the following conditions:

Wall height, H	=	5m	ϕ'_{cv} (fill)	=	30°
Reinforcement length, L	=	3m	γ (fill)	=	19 kN/m^3
Facing inclination, α_w	=	0°	α_p	=	1.0
Slope above wall, β	=	0°	λ_{cs}	=	26.6°
Block dimensions, $W_u \times H_u$	=	0.3m × 0.2m	k_h	=	0.2
P_{des} (seismic)	=	28.0 kN/m	k_v	=	0.1

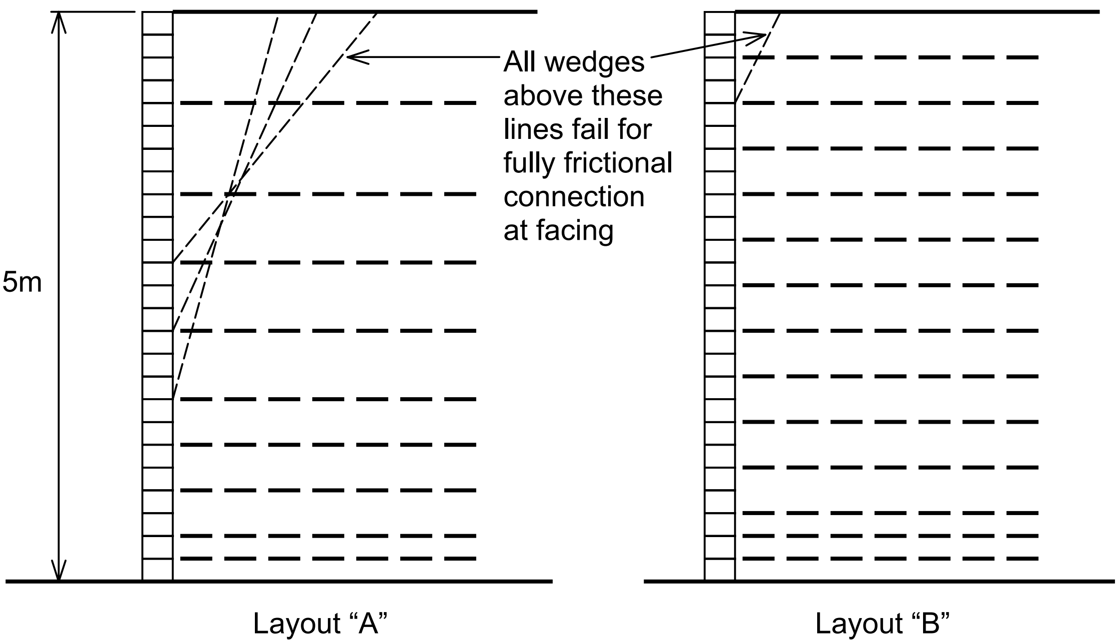

Figure 10 Reinforcement layouts for worked example

Figure 10 shows the resulting reinforcement layout. Layout "A" is satisfactory assuming that the wall connection is full strength (T_{con} = P_{des} from grid rupture). If the connection is fully frictional, then all wedges steeper than the lines shown on Layout "A" will fail due to connection pullout. The only way to improve the design is to increase the number of layers. Layout "B" uses half the reinforcement spacing for the upper part of the wall, yet there are still some minor stability problems [NB: FHWA recommends that fully frictional connections should not be used for A_h > 0.19g].

5.0 CASE HISTORIES

To illustrate and evaluate the technique further, two case histories are considered below. In the first case the static performance of a series of full-scale test walls is examined. In the second case modular block retaining walls affected by the Chi Chi earthquake in 1999 are reassessed.

5.1 Static case: Full-scale test walls reported by Bathurst et al, 2001

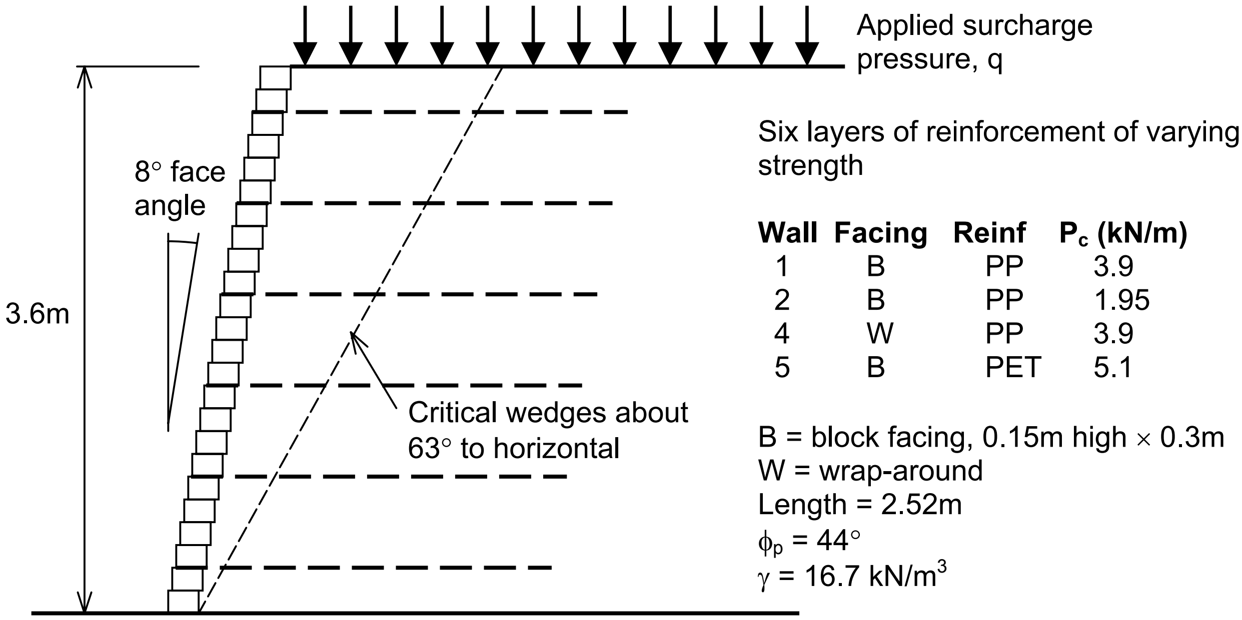

Figure 11 Full scale test walls reported by Bathurst et al, 2001

In his Special Lecture to the 2001 Kyushu International Symposium on Earth Reinforcement, Bathurst reported the performance of a number of full-scale test walls built at the Royal Military College of Canada (Bathurst et al, 2001). Figure 11 summarises the layout of three of the walls (1, 2 and 5), each with 6 layers of polymer reinforcement and a modular block facing. Wall 4 also had 6 layers of reinforcement, but the facing consisted of a wrap-around. Following construction and filling (duration 500 to 2000 hours), a surcharge was applied in stages (120 kPa reached after about 2000 hours) and the behaviour of each wall was observed. Some features of each of the tests on Walls 1, 2, 4 and 5 are summarised in Table 1.

Wall	Facing	Reinf type and strength (kN/m)			Surcharge (kPa)		2-part wedge result for q_{max}	
		Polymer	P_{ult}	P_c	q_{max}	q_{yield}	P_{des} (kN/m)	α_I (°)
1	B	PP	13.8	3.9	92	58	9.4	63
2	B	PP	6.9	1.95	50	37	6.1	63
4	W	PP	13.8	3.9	30		4.6	63
5	B	PET	16.0	5.1	100		9.9	63

Table 1 Summary of some features of the RMC test walls

Facing	B	denotes modular block
	W	denotes wrap-around
Polymer	PP	denotes polypropylene
	PET	denotes polyester

where P_{ult} = reinforcement tensile strength
P_c = reinforcement creep limited strength
q_{max} = surcharge when P_c exceeded in most heavily loaded reinforcement layer
q_{yield} = surcharge when marked increase in outward wall movement occurred
P_{des} = required design strength according to two-part wedge analysis to resist q_{max}
α_i = inclination of critical failure plane to horizontal

The reinforcement strengths are relatively low, so that conditions close to failure could be reached. P_c is the long term strength based on limiting strain according to creep tests. q_{max} is the surcharge reached during the test when it was assessed that P_c had been exceeded in the most heavily stressed layer of reinforcement. P_{des} are the values of reinforcement strength required according to the two-part wedge analysis for each wall to resist q_{max}, and α_I is the inclination of the critical failure plane.

Some observations from these analyses:

- Comparing Wall 1 and Wall 3 (the only difference is the type of facing) demonstrates that the modular block facing provides additional restraint which is not taken into account by the 2 part wedge method described above,
- The two-part wedge analysis method substantially over-predicts the reinforcement force required compared to P_c measured from creep testing for all three modular block cases. However the interpretation of P_c results in values which are rather low for these types of material and durations of loading,
- For the wrap-around facing the two-part wedge method is far closer at predicting P_c,
- q_{yield} (which is more representative of the overall behaviour of the critical wedge) is the load at which sudden increases in outward wall movement were first observed as the surcharge was increased. For Walls 1 and 2, q_{yield} was significantly less than q_{max}, and would result in closer agreement between between observed behaviour and the results from the two-part wedge analysis,
- The predicted critical failure plane was 63° from the horizontal in all cases – similar to the behaviour observed in the tests.

The results of this comparison are promising, and indicate that the restraining effect of the modular block facing has a beneficial effect, which is not taken into account in the two-part wedge method as described above.

5.2 Seismic case: Back-analysis of modular block walls affected by Chi Chi earthquake

A major earthquake occurred near the town of Chi-Chi in Nantou County, central Taiwan on 21st September 1999 at 1.47 am. Magnitude was 7.6 (M_w) and focal depth was about 10 km, resulting in extensive loss of life and damage to property. This was one of the strongest earthquakes to occur in Taiwan during the 20th century. A large number of retaining structures were situated within the epicentral area, and many were damaged. As well as traditional construction there were a significant number of reinforced soil structures, most of which performed well, although there were some failures. Three modular block faced walls were located almost due east of Taichung City, and a short way east of the fault break. These walls were of similar height, yet two collapsed one remained intact. The author visited this site in early 2000, and inspection of the damage indicated that low connection strength must have been a contributory factor to the failures.

A considerable number of papers have been written about these walls, some reporting extensive numerical analysis (eg: Huang, 2000, Huang & Tatsuoka, 2001 and Lee et al, 2001), and the data used here are based on these papers. The aim of this back-analysis is to investigate whether the simple two-part wedge method described in this paper could have predicted that all three walls would have been stable under static conditions, but that two would have failed under the earthquake loading which occurred whereas the intact wall would have remained intact.

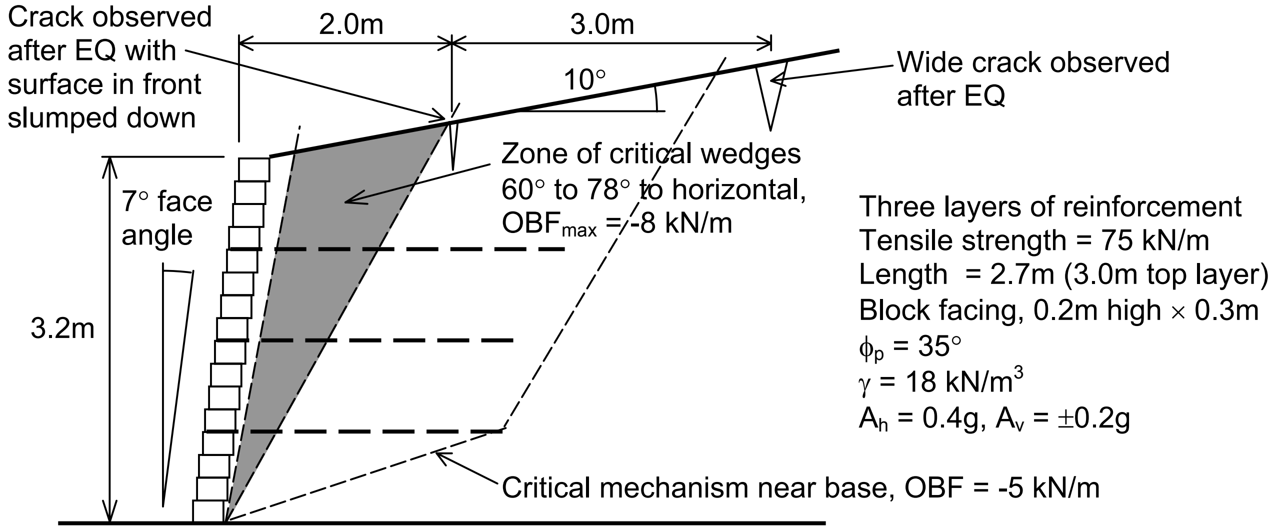

Figure 12 Modular block wall at Site 1 which failed during Chi Chi earthquake

Following the nomenclature used by Huang & Tatsuoka (2001), Figure 12 shows a simplified section of the wall at Site 1, which failed. The second wall at Site 1 was very close by, almost identical and also failed. The various parameters observed or assumed for the analyses are summarised on Figure 12. The soil reinforcement consisted of a polyester geogrid, and the connection could be classified as being fully frictional. Based on hinge height assessment under seismic loading as described in Section 3.3 above, together with likely values of λ_{cs}, maximum connection strength would be about 2 kN/m. Using this data in the two-part wedge analysis results in quite a large number of potential failure mechanisms, as shown on Figure 12.

OBF, or out-of-balance force, is the difference between the force available from the reinforcement and the force required to stabilise Wedge 1. When OBF is negative, then the wedge is unstable. In this case there is a fan of unstable wedges from 60° to 78° to the horizontal with OBF$_{max}$ = -8 kN/m, which is quite high for a small wall. There is also a low angle unstable wedge with OBF = -5 kN/m. The implied failure mechanisms coincide very closely with the cracks and displacements reported.

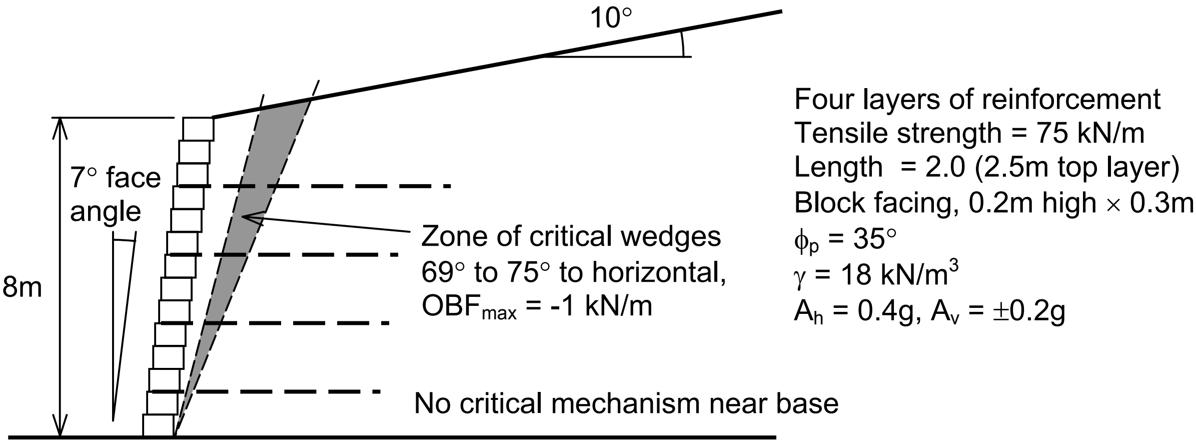

Figure 13 Modular block wall at Site 3 which remained intact after Chi Chi earthquake

A simplified section for the third wall, at Site 3, is shown on Figure 13. Following the same procedure, and using the same parameters as Site 1, critical wedges are also predicted, but only in a narrow band, and with OBF$_{max}$ = -1 kN/m, which is negligible. There is no critical mechanism near the wall base in this case.

This simple analysis indicates that, for seismic conditions, the two walls at Site 1 could well have been expected to collapse whereas the wall at Site 3 was borderline. The principal mode of failure is one of steep wedges of soil, together with the facing blocks, sliding off the reinforcement, which is left embedded in the resistant soil mass. This happens due to the low connection strength between the reinforcement and the facing. Even a moderate amount of mechanical connection strength would greatly increase the OBF for these wedges. It should be noted that the walls at Site 1 are slightly higher (3.2m) than at Site 3 (2.8m), but only have three layers of reinforcement at 0.8m spacing, compared to four layers of reinforcement at 0.6m spacing. There is also quite a large space between the wall base and the first reinforcement layer.

For static conditions all three walls are stable, although the application of normal safety factors indicates that the designs are not satisfactory.

One final point is worth noting concerning these retaining walls. The wall at Site 3, although intact suffered a little distress. After the earthquake, drainage material was observed by the author between the upper layer of blocks and the second layer. This indicated that the blocks had jumped upwards to allow the gravel particles to run under the blocks. This emphasises, only too graphically, that vertical acceleration can have a major part to play in seismic design of retaining walls, of all types.

CONCLUSIONS

- Geosynthetic reinforced soil structures have been observed to perform very well in strong earthquakes, although failures have occurred, some of which appear to be related to low connection strength between the soil reinforcement and modular block facings.
- The tie-back wedge method of designing reinforced soil structures has been in use for many years and forms the basis of several published design methods and manuals. However it includes assumptions and simplifications, particularly when the active wedge alone is used to check pull-out, which give some concerns in regards to the internal stability check, especially when modelling seismic loading and facing connection strength.
- This paper outlines a two-part wedge method of design given in Deutsches Institut für Bautechnik Certificate Z 20.1-102 (referred to here as DIBt). This method differs from the tie-back wedge by searching a large number of possible failure mechanisms which include the effect of any layers of reinforcement which are intersected by the failure plane.
- An adaptation of the two-part wedge method of calculation used in DIBt is presented which takes into account seismic loads and facing connection strength. As part of the design method, proposals are included for interpretation of connection test results for both static and seismic design conditions, including the effects of hinge height.
- A simple worked example shows how low connection strength can result in instability near the top of a wall, especially under seismic loading.
- Back-analysis of full-scale test walls indicates that the modular block facing provides additional restraint, which should be taken into account in the design method.
- Reassessment of three modular block walls, affected by the Chi Chi earthquake, demonstrates that instability could have been anticipated using the two-part wedge method. The analysis also shows that a moderate amount of mechanical connection strength would greatly improve stability of these walls.

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