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Seismic design of mechanically stabilized wall structures

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ABSTRACT: New Zealand limiting equilibrium (NZLE), Terre Armee Internationale, Australian Code and Federal Highway Administration earthquake design methods for mechanically stabilized walls are compared by applying them to typical wall geometries. These design procedures gave significantly different results for sliding stability and soil reinforcement density. It was concluded that the NZLE method produced designs that would not undergo significant permanent displacements in strong ground shaking, or alternatively, the method could be used in conjunction with Newmark sliding block theory to predict displacements. Walls designed in accordance with the other methods will move outwards an amount that cannot be readily estimated.

1 INTRODUCTION

Many retaining walls mechanically stabilized with steel strips (Reinforced Earth) have been subjected to strong ground shaking in major earthquakes in Japan, Turkey and USA (Kobayashi et al, 1996, Freyssinet, 2000, TAI, 1994). There have been reports of Reinforced Earth (RE) walls moving outwards or settling but there have been no reports of wall failures. Over 100 RE walls were subjected to strong shaking in the 1995 Hyogoken-Nanbu earthquake. Approximately 21 of these were subjected to peak horizontal ground accelerations (PGA's) greater than 0.4 g but none sustained significant damage. The facings of three of the walls moved outwards between 20 to 110 mm at the top of the walls (Wood and Asbey-Palmer, 1999).

Experimental research (Fairless, 1989) and full scale testing (Richardson and Lee, 1975) has shown that providing RE walls are designed to have ductile failure modes with either material yield in the strips or pull-out of the strips, they can undergo appreciable outward movement without loss of integrity. This characteristic is of considerable advantage and has been a factor leading to good performance of RE walls in shaking with peak ground accelerations considerably greater than design acceleration levels.

In New Zealand, and in other countries in regions of high seismicity, RE walls are specifically designed for earthquake loads. Two different design procedures have evolved. Terre Armee Internationale (TAI, 1989) was first to develop a method based on the active wedge theory used for static loads. A limitation of this method is that it is not possible to determine the critical acceleration at which the Reinforced Block (RB) commences to move outwards; or the extent of outward movement if the critical acceleration is exceeded. The New Zealand limiting equilibrium method (NZLE) was developed (Fairless, 1989, Wood and Elms, 1990) to allow the earthquake design of RE walls to be based on limiting the outward movement in strong ground shaking. Because testing and theory used as the basis of the design methods was unable to clearly identify the influence of soil amplification and the degree of coherence in accelerations over the long length of soil influencing the pressures on the facings, there remains uncertainty about the magnitude and point of application of the inertia forces acting on the RB and the retained soil behind the block.

In this paper, the two different design methods are compared by summarising results obtained for typical design parameters and wall configurations using four different design guideline or code documents. The NZLE design procedure specified in the Transit New Zealand Bridge Manual, 2003, and three different versions of the TAI active wedge procedure; TAI Design Guide, 1991, US Department of Transportation, Federal Highway Administration Guidelines (FHWA-NHI-00-043, 2001) and the Standards

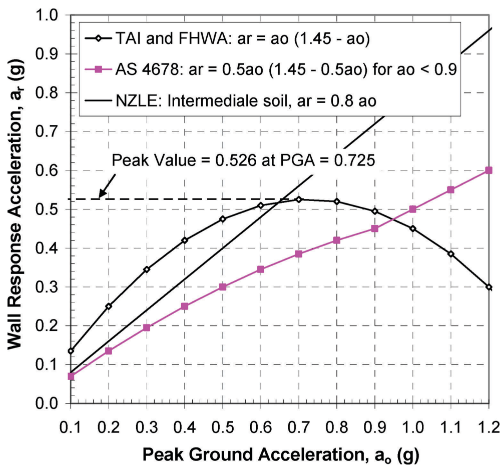

Figure 1. Response acceleration calculation.

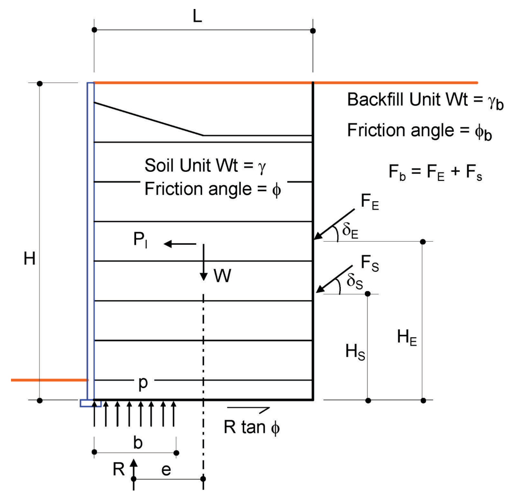

Figure 2. External stability analysis.

Australian Code for earth-retaining structures (AS 4678 – 2002) were investigated. In order to simplify the comparison, the walls were assumed to have rectangular reinforced blocks, horizontal ground surface behind the wall, zero live loading and to be free standing without other structures (such as bridge abutments and building foundations) loading them. The walls were assumed to be acted on by only horizontal earthquake accelerations without simultaneous vertical motions.

2 COMPARISON OF DESIGN METHODS

2.1 *Response acceleration*

The response acceleration acting on the RB and retained soil masses is calculated by modifying (reducing or increasing) the PGA. A comparison of the response acceleration calculation methods is shown in Figure 1. In the NZLE procedure the response acceleration is obtained by reducing the PGA by a constant factor related to the stiffness of the foundation soils. In the other design methods the response acceleration is assumed to vary as a function of the PGA. It both the TAI and FHWA methods the response acceleration is greater than the PGA up to a PGA of 0.45 g and less than the PGA at higher PGA's. In the AS 4678 method the response acceleration is less than the PGA for all values of PGA.

2.2 *External stability*

External stability is investigated by a similar procedure in both the NZLE and TAI active wedge methods although the assumptions regarding the magnitude and application of the earthquake forces on the RB, strength reduction factors and factors of safety differ considerably. External stability design involves checking the resistance to sliding on a plane through the base of the wall using horizontal equilibrium equations, and checking the base pressures (assumed to be uniform) using moment equilibrium equations. Gravity and earthquake forces acting on the RB are assumed to be those shown in Figure 2. The magnitude of the earthquake forces and their point of application are listed in Table 1.

2.3 *Internal stability*

In the NZLE method, a bilinear failure surface is assumed to develop at the toe of the wall and propagate up through the RB and the retained soil behind the RB. An upper-bound failure criterion is applied to find the critical failure surface inclination angles and the acceleration at which sliding develops. The disturbing forces acting on the sliding block are the soil block weight, inertia force, and the Mononobe-Okabe (M-O) pressure (Wood and Elms, 1990) on the back of the soil block. These are resisted by soil friction and cohesion (usually zero) on the failure plane and the forces in the reinforcing strips that cross the failure surface. Forces acting on the failure wedge are shown in Figure 3.

In the TAI active wedge method empirical rules are used to define the active wedges shown in Figure 4. The FHWA wedge is 18% larger in area than the TAI wedge. (Wedge details are not specified in the AS 4678 code.) The active wedge surface running through the block defines the line of maximum tension in the reinforcement strips as determined by experimental results. Strip lengths in the resistive zone behind the active zone are designed to resist the sum of the static

Table 1. External stability comparison.

Item	Symbol	NZLE (Transit NZ BM)	TAI	FHWA-NHI-00-043	AS 4678
RE block inertia force	P_I	$P_I = a_r L H \gamma$ (a_r = response accn)	$P_I = 0.5 a_r L H \gamma$	$P_I = 0.5 a_r H2 \gamma$	$P_I = 0.5 a_r L H \gamma$
Back face G + E horizontal force component	F_b	$F_b = 1.35 \times 0.5 \Delta K_{AE} \gamma_b H^2 \cos\delta + F_s$ $F_s = 1.35 \times 0.5 K_A \gamma_b H^2 \cos\delta$ K_A = Coulomb active coefficient 1.35 is load factor on earth pressures for G + E load	$F_b = 0.5 \Delta K_{AE} \gamma_b H^2 + F_s$ $F_s = 0.5 K_A \gamma_b H^2 \cos\delta$ K_A = Coulomb active coefficient $\Delta K_{AE} = K_{AE} - K_{AO}$ K_{AE} based on $0.5 a_r$ (a_r = resp accn)	$F_b = 0.25 \Delta K_{AE} \gamma_b H^2 + F_s$ $F_s = 0.5 K_A \gamma_b H^2$	$F_b = 1.25 \times 0.5 K_{AE} \gamma_b H^2 \cos\delta$ 1.25 is load factor on driving forces (see below).
Height of back face force	H_b	$H_b = 0.333$ H For total active force.	$H_s = 0.333$H $H_E = 0.60$ H	$H_s = 0.333$H $H_E = 0.60$ H	$H_b = 0.4$H to 0.55H For total active force.
Back face friction angle	δ	Not specified but usually taken as: $\delta = 0.67\phi_b$ for both G & E comps.	$\delta = 0.8(1 - 0.7\,L/H)\,\phi_b$ Static component only.	Taken as 0 for both static and earthquake components.	Not specified but code indicates it is taken as non-zero.
Strength reduction		None specified.	None specified.	None specified.	0.95 or 0.9 factor on $\tan\phi$.
FOS for sliding under E, or load Factors	S_s	$S_s = 1.0$	$S_s = 1.2$	$S_s = 1.5 \times 0.75 = 1.125$	Driving forces from gravity are increased by 1.25 and resisting forces reduced by 0.8.

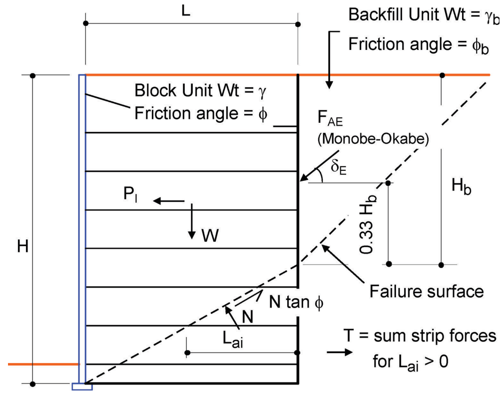

Figure 3. NZLE analysis.

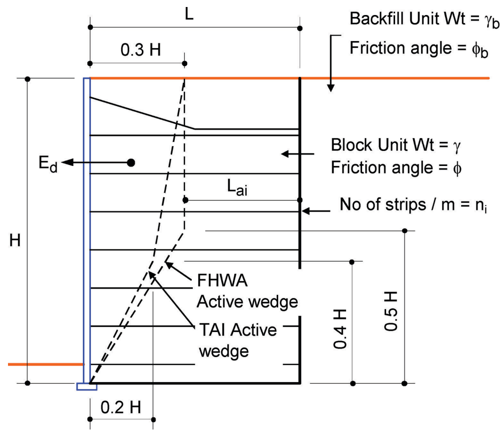

Figure 4. TAI and FHWA internal stability analyses.

and earthquake pressures generated on the wall facing. The total earthquake pressure on the facing is taken as the inertia force acting on the active wedge and is distributed to the strips in proportion to the product of their section area per unit length of wall and their resistive length.

The applied forces, pressure distribution assumptions, strength reduction factors, and factors of safety used in the internal stability design methods are summarised in Table 2.

3 ANALYSIS RESULTS

3.1 *External stability – sliding on base*

The PGA to produce the limiting factors of safety against base sliding are plotted against length/height (L/H) ratio of the RB in Figure 5. Similar results for response acceleration versus L/H are shown in Figure 6. To make these comparisons it was necessary to

Table 2. Internal stability comparison.

Item	Symbol	NZLE (Transit NZ BM)	TAI	FHWA-NHI-00-043
Earthquake load in soil reinforcement	T_{m2}	Inertia load assumed uniformly distributed over height of wedge with loads in strips determined by pull-out resistance or rupture strength.	Earthquake load from wedge, E_d, is distributed to strips using: $T_{m2} = E_d \frac{L_a}{\sum_{i=1}^{i=N} n_i L_{ai}}$ L_a = resisting length of strip and n_i = number of strips with uniform surface area per unit length of wall in layer i.	Earthquake load from wedge, E_d, is distributed to strips using: $T_{m2} = E_d \frac{L_a}{\sum_{i=1}^{i=N} L_{ai}}$ L_a = resisting length of strip. Text states that E_d is distributed in proportion to "resistant area" (equation incorrect).
Friction factor for strips	f*	For ribbed type strips f* varies from $f_o^* = 1.5$ at surface to $f^* = \tan\phi_b$ at depth of 6 m.	For ribbed type strips f* varies linearly from $f_o^* = 1.2 + \log C_u$ at surface to $\tan\phi_b$ at depth of 6 m. C_u is soil uniformity and ϕ_b frict angle. Default, $f_o^* = 1.5$	For ribbed type strips f* varies linearly from $f_o^* = 1.2 + \log C_u$ at surface to $\tan\phi_b$ at depth of 6 m. C_u is soil uniformity and ϕ_b frict angle. Default, $f_o^* = 1.8$
Backface force	P_z	Mononobe-Okabe pressure on the backface of sliding wedge section.	Gravity pressure force from backfill soil behind the block is used to calculate the vertical pressures at the wall face.	No backface soil pressure is used in the internal stability assessment.
Vertical pressure at facing	$\sigma_v(z)$	Not required in this analysis.	Calculated from weight of block and pressure on backface.	Calculated from weight of block.
Horizontal pressure coefficient	K	Not required in this analysis.	Horizontal pressure on facing calculated from vertical pressures using: $K = K_o$ at surface & K_a at 6 m depth. K_o is at rest coefficient and K_a is active coefficient.	Horizontal pressure on facing calculated from vertical pressures using: $K = 1.7\,K_a$ at surface & $1.2\,K_a$ at 6 m depth. K_a is the active pressure coefficient.
Strength reduction factors		0.8 for strip pull-out resistance. 0.9 on strip yield strength.	0.8 for strip pull-out resistance under E loads.	0.8 for strip pull-out resistance under E loads.
Factors of safety		1.0 to calculate critical acceleration for sliding failure.	For strip rupture = 1.5 on UTS generally or 1.65 on UTS for important walls. For strip pull-out = 1.0.	Strip rupture = 1.36 on strip yield stress (1/ 0.55 × 0.75). Strip pull-out = 1.125 (0.75 × 1.5).

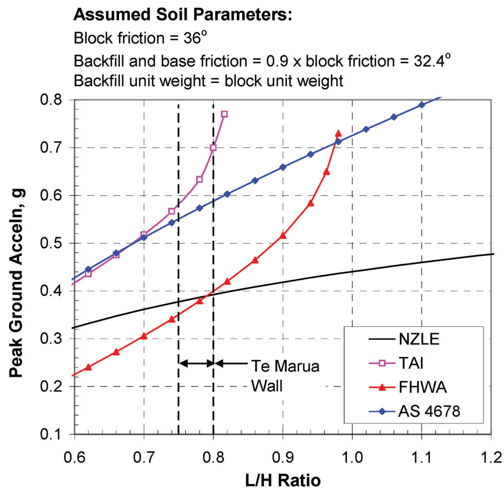

Figure 5. Sliding stability in terms of PGA.

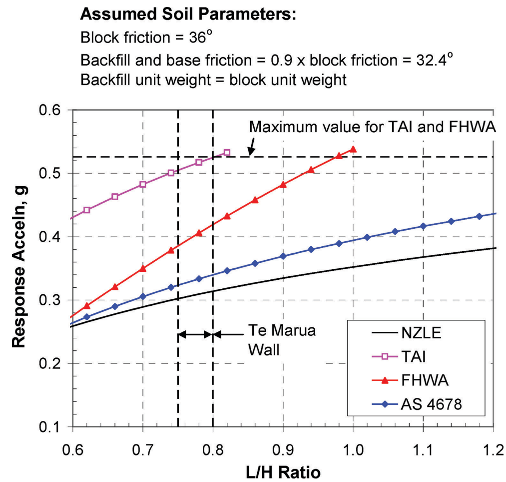

Figure 6. Sliding stability in terms of response acceleration.

define the ratios between the RB, backfill and base soil friction angles, and the ratio of the soil unit weights in the RB and backfill. For the comparisons shown in Figures 5 and 6 these ratios were taken as 1.0: 0.9: 0.9 and 1.0: 1.0 respectively. A PGA reduction factor of 0.8 was used in the NZLE method.

Typical L/H ratios for walls designed for seismic resistance vary from 0.75 to 1.0. Over this range the four methods investigated show a considerable variation in PGA or response acceleration to produce the specified limiting factors of safety with the NZLE method generally giving the most conservative PGA and response acceleration values.

3.2 *External stability – base pressures*

Base pressures calculated for a PGA of 0.3 g are compared in Figure 7. (Pressures were divided by the RB soil unit weight γ and the wall height H to give a dimensionless parameter.) The ratios of the soil friction angles and units weights were the same as used for the sliding stability comparison.

With the exception of the FHWA results, there is reasonable agreement between the base pressures. FHWA pressures are significantly higher than the others, especially for low L/H ratios.

3.3 *Internal stability*

To compare the strip distributions required to satisfy the internal stability requirements of the NZLE, TAI Guidelines and the FHWA methods, typical 10 m and 12 m high wall sections designed to support a highway at Te Marua, Wellington, New Zealand were used. (AS 4678 was not considered because it does not define an active wedge.) The geometry of the wall sections and

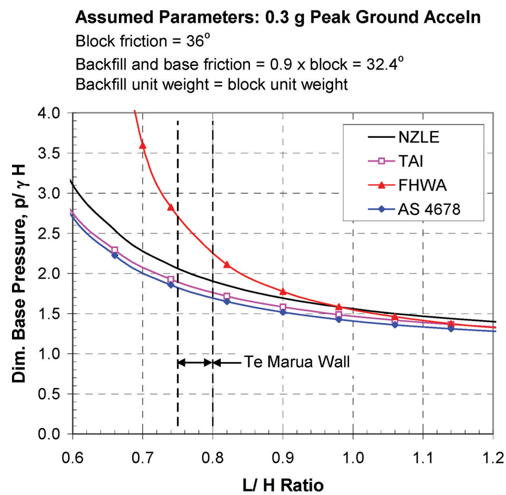

Figure 7. Dimensionless base pressures.

the assumed soil properties are given in Figures 8 and 9. The wall was designed using the NZLE method for a PGA of 0.4 g and a response acceleration of 0.32 g.

The TAI and FHWA active wedge geometries are compared with the NZLE sliding wedge in Figures 8 and 9. Also shown for comparison with the NZLE failure surface are the slip circles obtained by analysis of the wall sections with the STARES slope stability software (Balaam, 1999) which is based on the Bishop Simplified Method. The STARES input parameters for the soil and reinforcing strips were the same as used in the NZLE method. The circles shown in Figures 8 and 9 are the most critical circles passing through the wall facing footings. Slightly more critical circles passing through points on the wall facing were obtained but

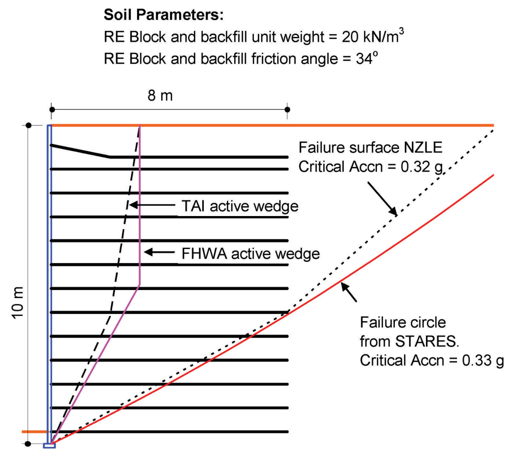

Figure 8. Te Marua 10 m high wall details.

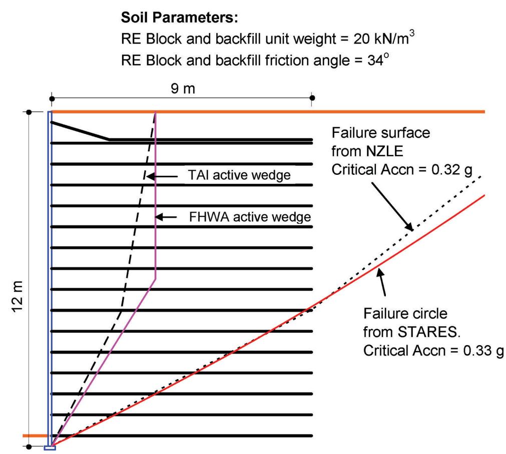

Figure 9. Te Marua 12 m high wall details.

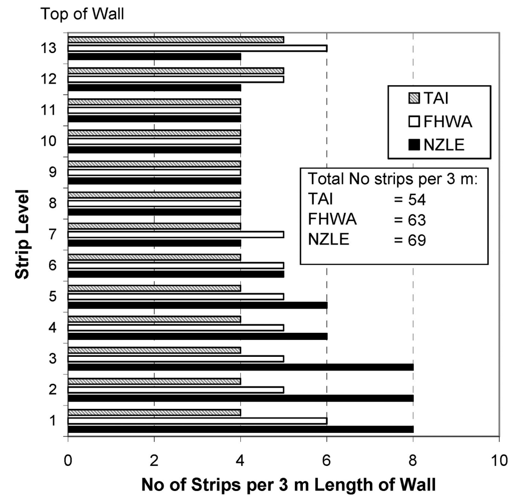

Figure 10. Strip density and distributions on 10 m high section to satisfy internal stability requirements.

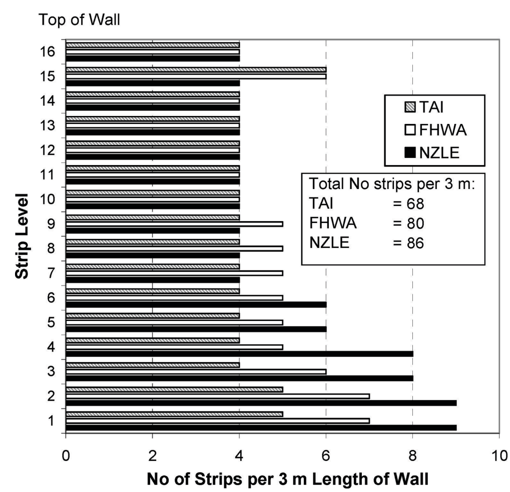

Figure 11. Strip density and distributions on 12 m high section to satisfy internal stability requirements.

the strength of the facing prevents these from developing at lower accelerations than the circle through the wall footing. The STARES critical circles have similar locations in the RB's to the NZLE failure surfaces and the critical acceleration of 0.33 g from STARES for both wall sections is in good agreement with the 0.32 g from the NZLE method.

Strip densities and the distribution over the wall height calculated to satisfy the internal stability requirements of the three design methods are shown in Figures 10 and 11 for the 10 and 12 m high sections respectively. For the 10 m high section the total number of strips per unit length of wall required by the FHWA and NZLE methods are 17 and 28% higher respectively than required by the TAI Guidelines. The TAI and FHWA strip densities were also analysed using the NZLE method which gave critical accelerations for the 10 m high section of 0.17 and 0.24 g respectively. Newmark sliding block theory (Newmark, 1965) indicates that sections with the TAI and FHWA strip densities would displace outwards about 80 and 20 mm respectively in the design level earthquake (0.32 g response acceleration by NZLE).

4 CONCLUSIONS

The currently used seismic design procedures for RE walls give significantly different results for sliding stability and soil reinforcement density.

The NZLE method is based on comprehensive theoretical and experimental research. Analyses for this paper, and back analyses of structures which exhibited permanent displacement as a result of the 1995 Hyogoken-Nanbu earthquake (Wood and Asbey-Palmer 1999) shows that this method gives failure predictions similar to those obtained using conventional slope stability analyses and also back analysis of actual structures subjected to strong ground shaking. Walls designed by the NZLE method are expected to remain elastic under the expected loading without significant permanent displacement. Alternatively, if permanent displacements are acceptable, structures can be designed to reduced acceleration levels and a specified outward displacement using the NZLE method in conjunction with the Newmark sliding block theory.

The active wedge method, in which internal stability is assessed with the inertia load on an active wedge resisted by the soil reinforcement in proportion to the product of the resistive lengths behind the wedge and section area per unit length of wall, and the external stability assessed using an assumed lack of coherency in soil mass accelerations is likely to result in significant permanent outward displacements in strong ground shaking.

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