# SEISMIC DESIGN AND PERFORMANCE OF GEOSYNTHETIC REINFORCED SEGMENTAL RETAINING WALLS

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**ABSTRACT:** The use of masonry concrete block (segmental) retaining walls (SRWs) reinforced with geosynthetics has increased dramatically in recent years due to their ease of construction and low cost. A unique feature of SRWs is the use of mortarless dry-stacked modular concrete blocks to form the facing column. The interlocking blocks provide transverse rigidity and shear capacity over the height of the wall. The stability of these structures under static loading has been adequately addressed by using conventional limit equilibrium methods. However, concern has been addressed regarding the stability of these systems under earthquake loading. At the Royal Military College of Canada a research program is underway that is focussed on design, analysis and performance features of these systems under seismic loading conditions. The paper reviews recent research activities at RMC and the implications of some research results to the seismic design and performance of geosynthetic reinforced segmental retaining walls.

## INTRODUCTION

The term *segmental retaining wall* (SRW) has been recently adopted by the National Concrete Masonry Association (NCMA) in North America to identify soil retaining wall structures built with a hard facing comprising a column of dry-stacked modular concrete units (Simac et al. 1993). These walls are often reinforced with geosynthetic layers in order to achieve greater heights. An example structure is illustrated in **Figure 1**.

The modular concrete facing units are produced using machine molded (masonry) or wet-casting methods and are available in a wide range of shapes, sizes and finishes. The dry-stacked (mortarless) concrete blocks are discrete units that transmit shear through interface friction, concrete keys, mechanical connectors or a combination of these methods. Examples of some commercially available segmental units are illustrated in **Figure 2**. Most proprietary units are 80 to 600 mm in height, 150 to 800 mm in width (toe to heel), and 150 to 1800 mm in length. The modular units typically vary from 14 to 48 kg each. The modular concrete units may be solid, hollow, or hollow and soil infilled. Segmental retaining walls are usually constructed with a stepped face that results in a facing batter that ranges from 3 to 15 degrees from the vertical.

The modular facing column introduces performance issues not associated with other types of reinforced retaining wall systems. The interfaces between the dry-stacked facing units provide potential failure planes through the facing column and this requires that stability calculations be carried out to estimate interface shear forces and to compare these forces with available shear capacity. In addition, the connection between the reinforcement layers and the facing column is typically formed by extending the reinforcing layers along the interface between facing units to the front of the wall. The connection detail must also be evaluated for satisfactory design capacity. A review of the state of practice in North America

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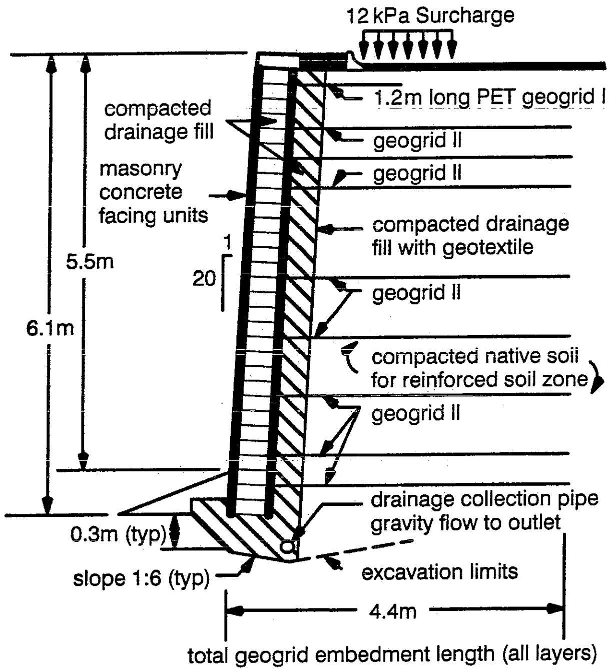



**Fig 1.** Typical geosynthetic reinforced soil segmental retaining wall cross-section (after Simac et al. 1991).

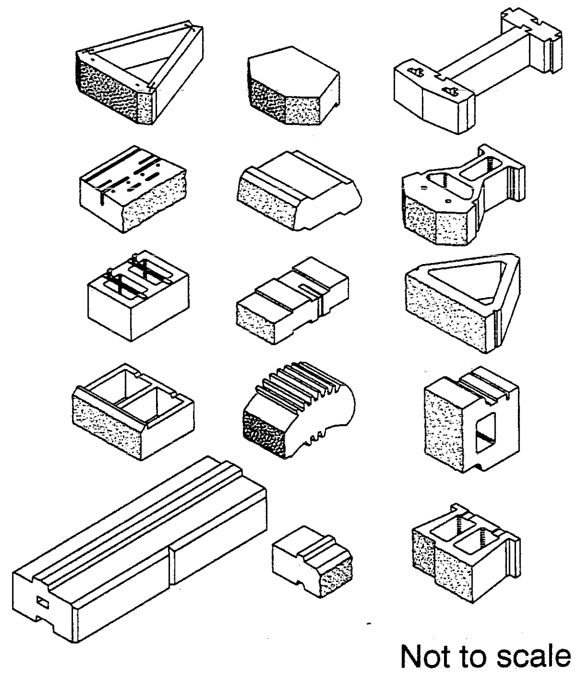



**Fig 2.** Examples of segmental retaining wall units.

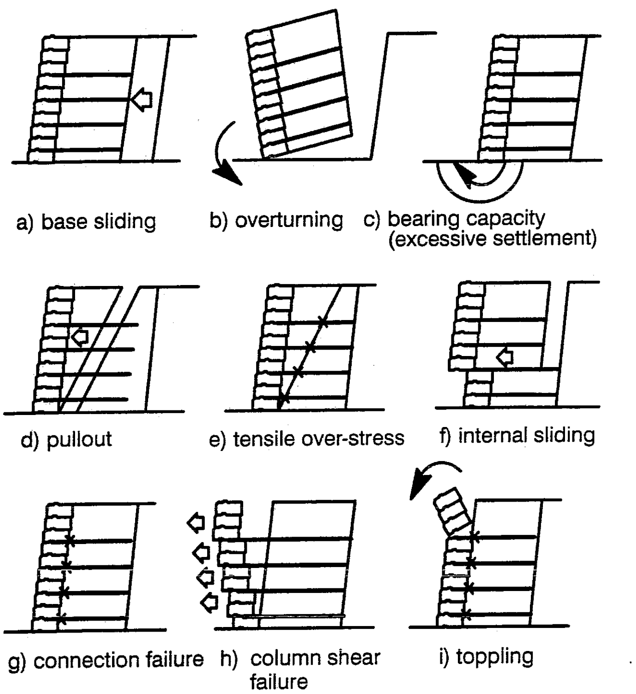



**Fig 3.** Modes of failure: external (top row); internal (middle row); facing (bottom row) (adapted from Simac et al. 1993).

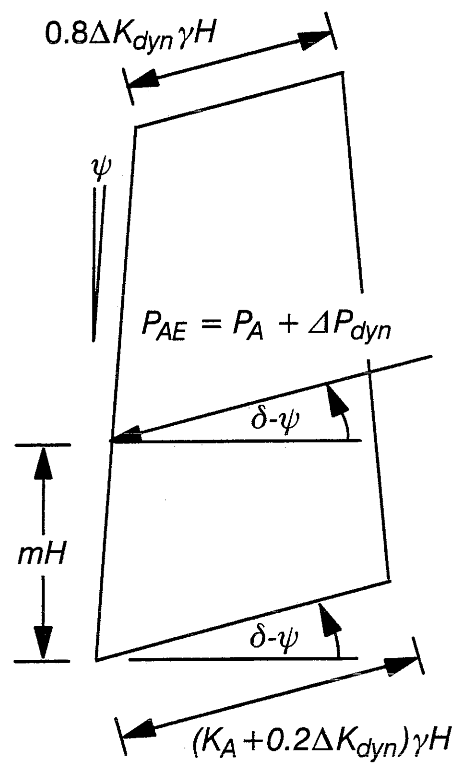



**Fig 4.** Calculation of dynamic (total) earth pressure distribution under seismic loading (after Bathurst and Cai 1995).

with respect to geosynthetic reinforced soil segmental retaining walls under *static* loading conditions has been reported by Bathurst and Simac (1994).

A large number of geosynthetic reinforced segmental retaining wall structures have been built in seismically active areas on the west coast of North America. Despite concerns regarding the stability of the dry-stacked facing column during a seismic event (e.g. Allen 1993) these structures have performed well during earthquakes in recent years (Eliahu and Watt 1991; Collin et al. 1992; Sandri 1994; Bathurst and Cai 1995). Nevertheless, the guidelines published by the NCMA are restricted to routine structures in which seismic loading is not addressed (Bathurst et al. 1993a; Bathurst and Simac 1995). Current AASHTO (1992) and FHWA (Christopher et al. 1989) guidelines for geosynthetic reinforced soil walls provide recommendations for seismic design and analysis but do not specifically address segmental retaining walls. The current edition of the Canadian Foundation Engineering Manual (CFEM 1992) does not address seismic design issues for geosynthetic reinforced soil walls of any type.

The Geotechnical Research Group at the Royal Military College (RMC) has been engaged in a research program over the last 5 years related to the development of methods of analysis and design for segmental retaining walls and test methods for the component materials. This paper gives an overview of recently completed work that is focused on the design, analysis and performance of reinforced segmental retaining walls under *seismic* loading conditions.

## POTENTIAL FAILURE MODES

Potential failure modes for reinforced segmental retaining wall structures are illustrated in **Figure 3**. External failure mechanisms consider the stability of an equivalent gravity structure comprising the facing units, geosynthetic reinforcement and reinforced soil fill. Not included in **Figure 3** is global instability which involves failure mechanisms passing through or beyond the reinforced soil mass. Modes of failure that require special attention in reinforced segmental retaining wall design and analysis are illustrated in the last four diagrams of **Figure 3**.

The NCMA design guidelines that were developed by the first writer and co-workers adopt a limit-equilibrium approach that uses Coulomb earth pressure theory for the stability analysis of conventional (statically loaded) structures. The advantages of the Coulomb approach are that it is familiar to geotechnical engineers and can explicitly accommodate the contribution to lateral earth pressure of wall inclination angle, backslope angle and shear mobilized at the interfaces between the reinforced soil and retained soil zones, and the facing column and reinforced soil zone.

## SEISMIC ANALYSIS

Analytical approaches for the seismic analysis of segmental retaining walls can be divided into three categories: 1) pseudo-static methods; 2) displacement methods; and 3) finite element methods. In the current paper, global stability modes of failure are not addressed and the structures are assumed to be seated on firm foundations for which settlement and collapse of the foundation materials are not a concern. In the discussions to the follow the reinforced and retained soils are assumed to be homogeneous, unsaturated and cohesionless.

## Pseudo-static method

Pseudo-static rigid body approaches that use the Mononobe-Okabe (M-O) method to calculate dynamic earth forces (Okabe 1926) acting on earth retaining structures are well established in geotechnical engineering practice (e.g. Seed and Whitman 1970; Richards and Elms 1979). The M-O method can be recognized as an extension of the classical Coulomb wedge analysis and hence provides the designer with a consistent approach to that recommended by the NCMA for statically loaded structures.

The total dynamic active earth force, $P_{AE}$, imparted by the backfill soil is calculated as (Seed and Whitman 1970):

(1) $$P_{AE} = \frac{1}{2}(1 \pm k_v) K_{AE} \gamma H^2$$

where: $\gamma$ = unit weight of the soil; and $H$ = height of the wall. The dynamic earth pressure coefficient, $K_{AE}$, can be calculated as follows:

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

where: $\phi$ = peak soil friction angle; $\psi$ = total wall inclination (positive in a clockwise direction from the vertical); $\delta$ = mobilized interface friction angle at the back of the wall (or back of the reinforced soil zone); $\beta$ = backslope angle (from horizontal); and $\theta$ = seismic inertia angle given by:

(3) $$\theta = \tan^{-1}\left(\frac{k_h}{1 \pm k_v}\right)$$

Quantities $k_h$ and $k_v$ are horizontal and vertical seismic coefficients, respectively, expressed as fractions of the gravitational constant, g. In the discussions to follow, it is convenient to decompose the total dynamic (active) earth force, $P_{AE}$, calculated according to Equations 1 and 2 into two components representing the static earth force component, $P_A$, and the incremental dynamic earth force due to seismic effects, $\Delta P_{dyn}$ (Seed and Whitman 1970). Hence:

(4) $$P_{AE} = P_A + \Delta P_{dyn}$$

or

$$(1 \pm k_v) K_{AE} = K_A + \Delta K_{dyn} \tag{5}$$

where: $K_A$ = static active earth pressure coefficient; and $\Delta K_{dyn}$ = incremental dynamic active earth pressure coefficient. Bathurst and Cai (1995) have proposed the total active dynamic earth pressure distribution illustrated in **Figure 4** for external, internal and facing stability analyses of reinforced segmental retaining walls. The normalized point of application of the resultant total earth force varies over the range $1/3 \leq m \leq 0.6$ depending on the magnitude of $\Delta K_{dyn}$. The assumed pressure distribution is based on a review of the literature for conventional gravity retaining wall structures and is identical to that recommended for the design of flexible anchored sheet pile walls under seismic loads (Ebling and Morrison 1993). The point of force application under seismic loading is also consistent with the work of Steedman and Zeng (1990) who used a pseudo-dynamic model to show that the resultant total earth pressure against cantilever retaining walls with a fixed base may act above $H/3$ during seismic loading. In the absence of ground acceleration the distribution reduces to the triangular active earth pressure distribution due to soil self-weight.

In order to address specific concerns raised by Allen (1993) related to facing stability of geosynthetic-reinforced segmental retaining walls during a seismic event that includes time coincident vertical ground accelerations, parametric analyses were carried out using the range $k_v = -2k_h/3$ to $+2k_h/3$. The upper limit on the ratio $k_v$ to $k_h$ is equal to the calculated ratio of peak vertical ground acceleration to peak horizontal ground acceleration from seismic data recorded in the Los Angeles area (UCB/EERC 1994).

It should be noted that possible amplification of horizontal accelerations through the height of the structure is not directly considered in the approach used here. The results of finite element modelling of reinforced soil walls by Segrestrin and Bastick (1988), Cai and Bathurst (1995) and some limited 1/2 scale experimental work (Chida et al. 1982) have shown that the average acceleration of the composite soil mass may be equal to or greater than the peak (site) horizontal acceleration depending on a number of factors such as: magnitude of peak ground acceleration; predominant modal frequency of ground motion; duration of motion; height of wall; and stiffness of the composite mass. Based on their pseudo-dynamic model, Steedman and Zeng (1990) have shown that the effect of horizontal amplification with height above the base of conventional gravity wall structures results in an increase in total earth pressure that is qualitatively similar to an increase in $k_h$ applied uniformly through the depth of retained soil. It can also be argued that the dynamic earth pressure increment distribution illustrated in **Figure 4** may indirectly account for amplification of horizontal ground accelerations since the center of gravity of this distribution is above the 1/3 wall height as described earlier.

A detailed discussion on strategies to select an appropriate value of peak ground acceleration based on ground acceleration records is beyond the scope of this paper. A review of the literature suggests that there is no consensus view on how to select a design value for $k_h$ in pseudo-static earth pressure calculations. For example, Whitman (1990) reports that values of $k_h$ from 0.05 to 0.15 are typical for the design of conventional gravity wall structures and these values correspond to 1/3 to 1/2 of the peak acceleration ($a_m$) of the design earthquake. Bonaparte et al. (1986) used $k_h = 0.85 a_m/g$ to generate design charts for geosynthetic reinforced slopes under seismic loading using the M-O method of analysis. Current FHWA guidelines use an equation proposed by Segrestin and Bastick

(1988) that relates $k_h$ to $a_m$ according to $k_h = (1.45 - a_m/g) \times a_m/g$ and results in $k_h > a_m/g$ for $a_m < 0.45g$. It appears that in practice, the selection of $k_h$ for design is based on engineering judgement, experience and, in some instances, local regulations.

The reader is referred to the paper by Bathurst and Cai (1995) for a detailed discussion of the selection of appropriate parameter values and the influence of these parameters on the calculated earth pressures for analysis and design.

## External stability

External stability calculations for factors of safety against base sliding and overturning of a geosynthetic reinforced segmental retaining wall are similar to those carried out for conventional gravity structures. For reinforced structures, the gravity mass is taken as the composite mass formed by the reinforced soil zone and the facing column. The earth pressure distribution shown in **Figure 4** is used to calculate the destabilizing forces in otherwise conventional expressions for factor of safety against sliding along the foundation surface and overturning about the toe of the structure. The simplified geometry and body forces assumed in these calculations are illustrated in **Figure 5**. The term $W_H$ in **Figure 5** is the weight of the reinforced zone plus the weight of the facing column. The quantity $k_h \lambda W_H$ represents the horizontal inertial force of the composite mass. A value of $\lambda = 0.6$ has been used for this empirical constant in the analysis of both geosynthetic-reinforced soil walls (Christopher et al. 1989) and for Reinforced Earth Company (RECO) walls that use steel reinforcement strips (Segrestin and Bastick 1988). Parameter $\lambda$ is assumed to be less than unity to account for the transient nature of the peak accelerations in the gravity mass and retained soils and the expectation that the inertial forces induced in the gravity mass and the retained soil zone will not reach peak values at the same time during a seismic event.

## Reinforcement over-stressing and connection loads

The additional dynamic earth pressures that develop at the back of the facing column will require that the reinforcement layers carry larger loads than those computed assuming a static loading condition. The conventional approach to assign static tensile loads is based on a contributory area method and is easily adapted for the seismic case as illustrated in

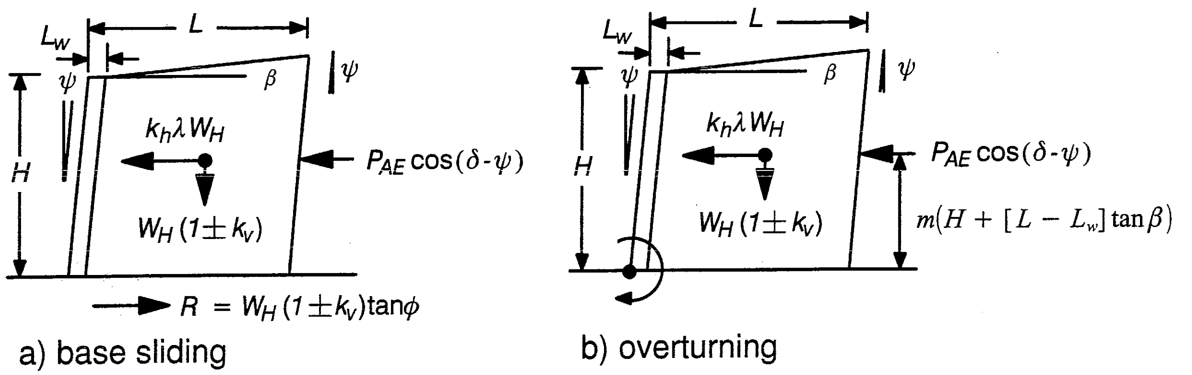



**Fig 5.** Forces and geometry for external stability calculations.

**Figure 6**. An example of the influence of seismic loading on reinforcement forces is shown in **Figure 7**. The figure shows that load amplification (with respect to the static case) is greatest at the wall crest. This result is not unexpected due to the change in the active earth pressure distribution that results from dynamic loading. The plotted data reveals that the reinforcement amplification factor, $r_F$, is sensibly independent of the magnitude of $k_v$ for $k_h \leq 0.35$, and hence solutions using $k_v = 0$ are sufficiently accurate for design over this range and even slightly conservative at the shallowest depth investigated ($z/H$=0.2). An implication of these results to design is that the number of reinforcement layers may have to be increased at the top of the wall in order to keep tensile loads within allowable limits. However, it should be noted that the allowable design tensile load ($T_{allow}$) under seismic loading is routinely taken as a greater percentage of the index strength of the reinforcement than the percentage used for static loading design because of the short duration peak tensile loading during a seismic event. AASHTO (1992) guidelines can be interpreted to permit the value of $T_{allow}$ used for static loading designs to be increased by 33% for the seismic loading condition.

Rapid in-isolation wide-width strip tensile loading of a typical high density polyethylene (HDPE) geogrid reinforcement reported by Bathurst and Cai (1994) has demonstrated a potentially large increase in reinforcement stiffness of the material when compared to conventional rates of loading (**Figure 8**). This observation suggests that HDPE geogrids may be designed for much greater working strengths under seismic loading than those values that result from the interpretation of current AASHTO recommendations. Alternatively, the results can be interpreted to show that an additional margin of safety against tensile over-stressing is present, but not accounted for, in current seismic design methods. The same study showed that the rate of monotonic loading for PET geogrids did not have a significant influence on the load-strain response of these materials.

The loads carried by the reinforcement are assumed to be transmitted to the connection with the facing column. Consequently, the load amplification factor illustrated in **Figure 7** has implications to the design of the connections. Peak connection load capacities under static loading conditions have been described using bi-linear failure envelopes based on the results of full-scale connection tests carried out at rates of loading matching the 10% strain/min rate used in the ASTM D 4595 method of test (Bathurst and Simac 1993). A Coulomb-type law with a maximum connection load cut-off has been used by the first author and co-workers to characterize a large number of (static) test results. The results of connection tests carried out at different rates of loading have demonstrated that peak connection capacities may be sensitive to rate of loading (Bathurst and Simac 1993). At the time of writing there is no data available that can be used to quantify changes in connection capacity that may develop as a result of repeated application of load and the rapid loading rates anticipated during a seismic event.

### Anchorage length

A consequence of a strict application of the M-O method to internal design is the reorientation of the internal failure plane (angle $\alpha_{AE}$ from the horizontal) with increasing horizontal acceleration. This analytical result is not typically a concern for conventional gravity structures. However, for geosynthetic reinforced wall systems the internal failure plane is used to delineate the active failure wedge from the anchorage zone in pullout capacity calculations (tie-back wedge method). The failure plane is assumed to propagate

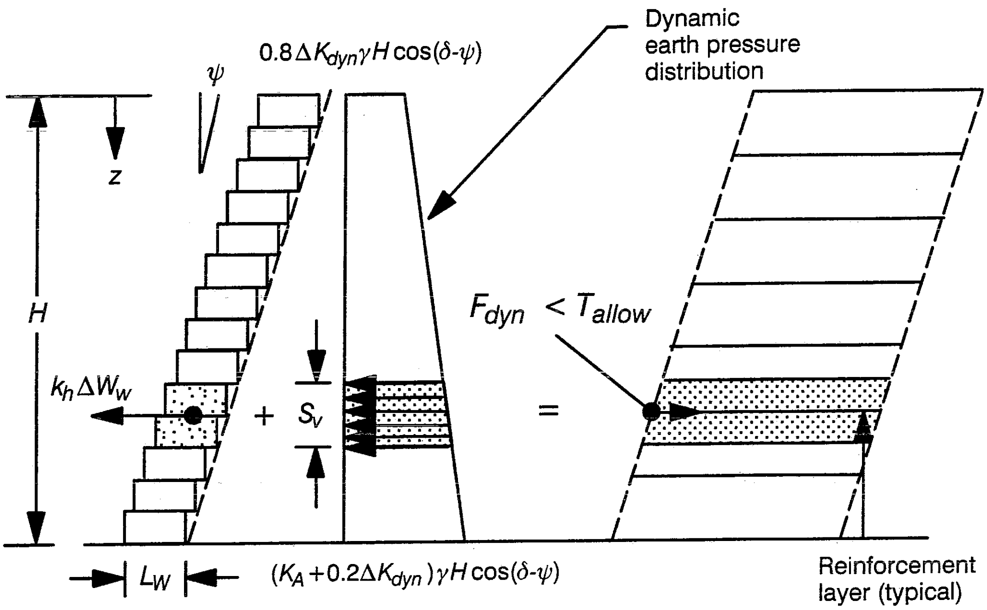



**Fig 6.** Calculation of tensile load, $F_{dyn}$, in a reinforcement layer due to dynamic earth pressure and wall inertia. (Note: $S_v$ = contributory area of reinforcement layer.)

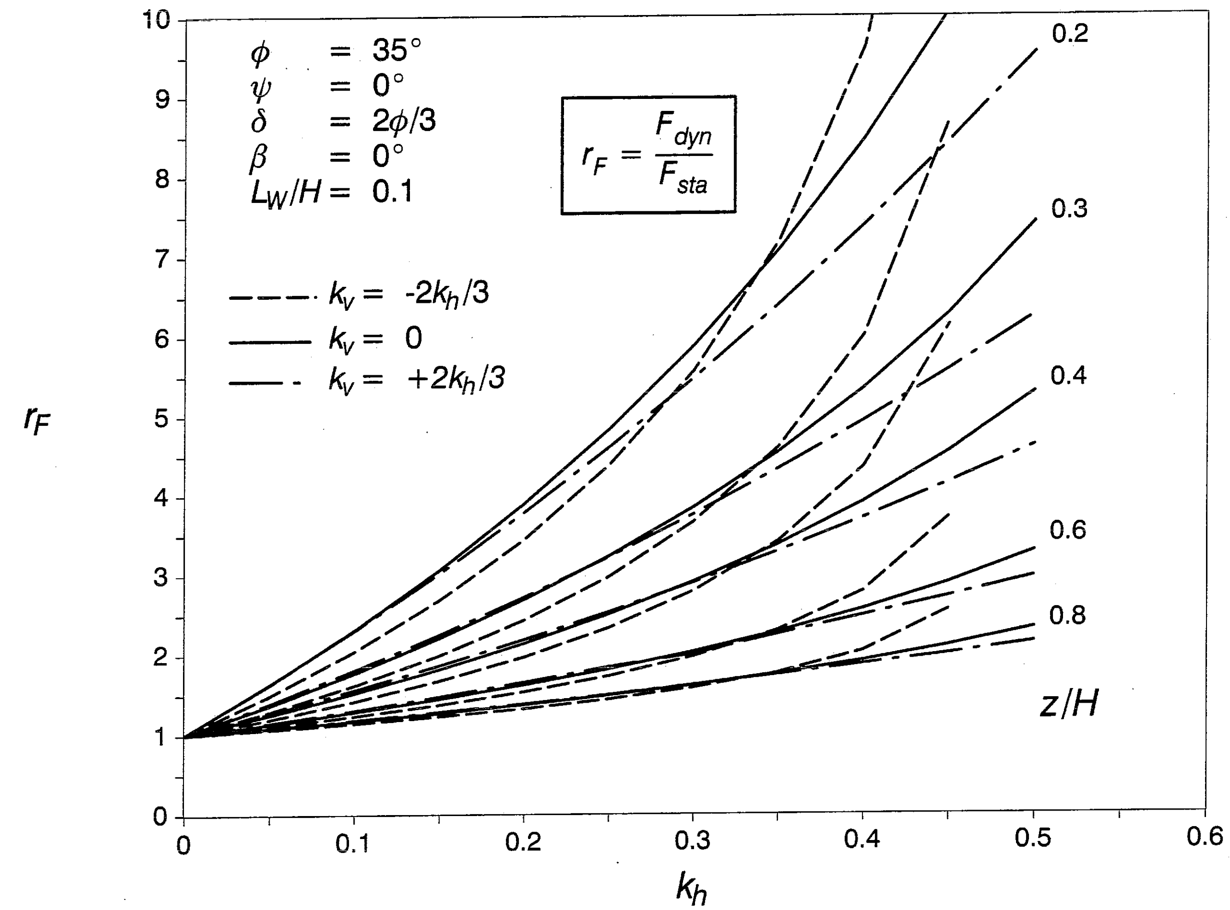



**Fig 7.** Influence of seismic coefficients, $k_h$ and $k_v$, and normalized depth below crest of wall, $z/H$, on dynamic reinforcement force amplification factor, $r_F$. (Notes: $F_{dyn}$ = factor of safety under seismic loading; $F_{sta}$ = factor of safety under static loading.)

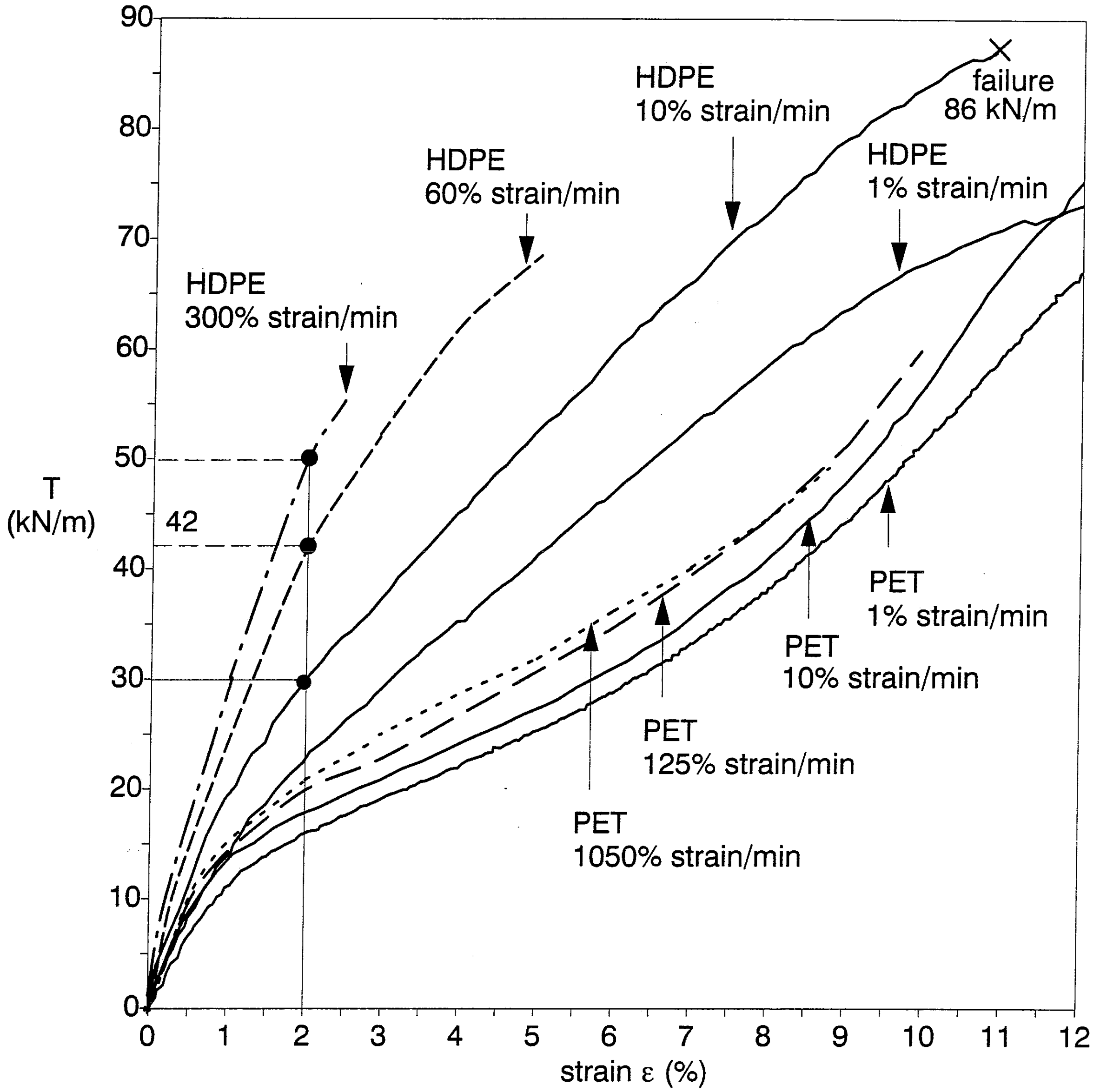



**Fig 8.** Influence of strain rate on monotonic load-extension behaviour of typical geogrid reinforcement products (after Bathurst and Cai 1994).

from the heel of the lowermost segmental unit into the reinforced soil zone. The influence of the magnitude of seismic coefficient on failure plane orientation is illustrated in **Figure 9**. For example, this figure shows that for a horizontal seismic coefficient, $k_h$ = 0.25, reinforcement lengths would have to be increased by 60 to 100% of the lengths required under static loading conditions. The requirement that the lengths of the uppermost reinforcement layers may need to be increased for reinforced slopes subject to seismic loading has been noted by Bonaparte et al. (1986) and is based on similar arguments. Tension cracks observed in the reinforced soil zone of a segmental retaining wall as a result of the Northridge earthquake in California have been attributed to inadequate reinforcement lengths close to the wall crest (Bathurst and Cai 1995). Tilting of a geogrid-reinforced soil wall that was observed after the Great Hanshin Earthquake of 17

January 1995 in Japan has also been attributed to inadequate reinforcement lengths at the top of the structure (Tatsuoka et al. 1995).

## Interface shear and toppling

The discrete nature of the facing column requires that the shear load transmitted across each facing column interface not exceed the shear capacity of that interface. The approach adopted in current NCMA guidelines for statically loaded structures is to consider the facing column as a continuously supported beam with the reinforcement layers providing the reactions. The reaction spacings are related to the contributory areas introduced earlier to partition distributed loads (earth pressures). The shear load to be transmitted at an interface can be referenced to **Figure 10**.

The performance of a particular system can only be established using large-scale testing as described by Bathurst and Simac (1994). A large number of tests on more that 40 different combinations of segmental retaining wall systems have been carried out by the writers. The database shows that interface shear capacity for block to block contact may be influenced by the presence of the geosynthetic inclusion that is used to create the connection. This effect is illustrated by the data in **Figure 11**. The performance of the interface at peak load and after a prescribed amount of relative displacement (serviceability criterion) can be described using a Mohr-Coulomb type failure criterion as illustrated in the figure. The

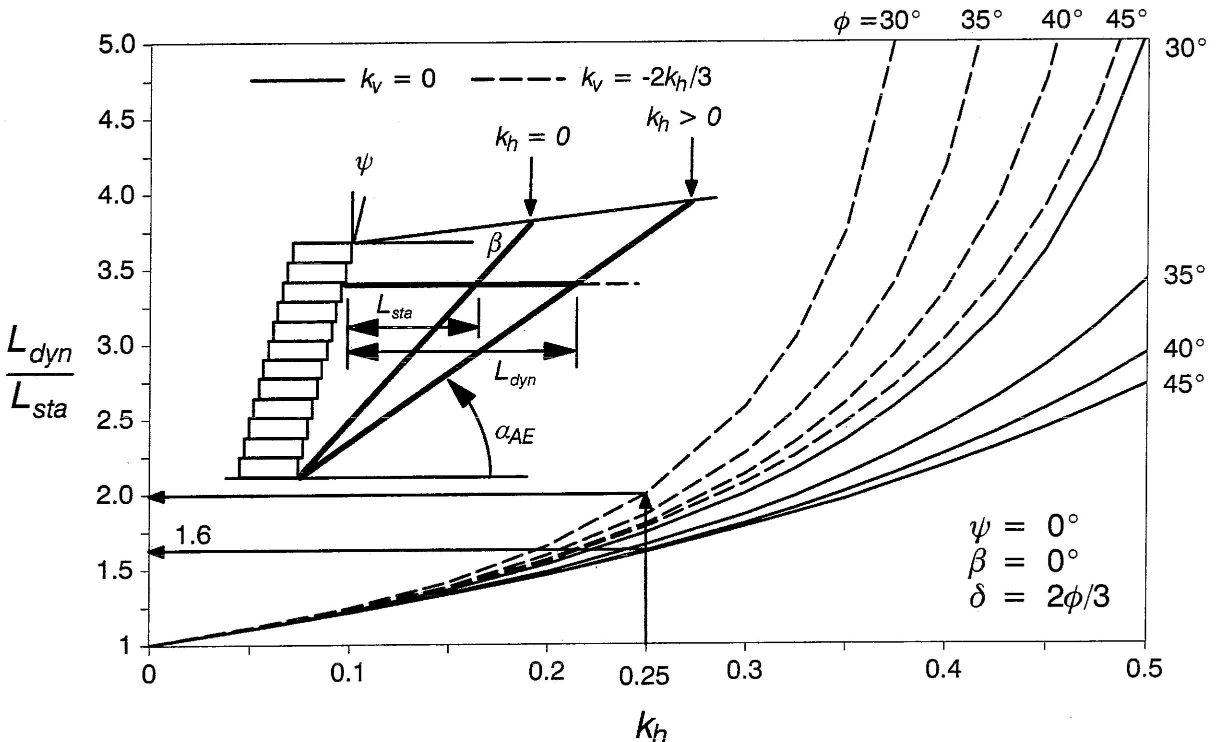



**Fig 9.** Influence of seismic coefficients, $k_h$ and $k_v$, and soil friction angle, $\phi$, on ratio of minimum reinforcement lengths, $L_{dyn}/L_{sta}$, to capture the internal failure wedge. (Notes: $L_{dyn}$ = minimum reinforcement length under seismic loading; $L_{sta}$ = minimum reinforcement length under static loading.)

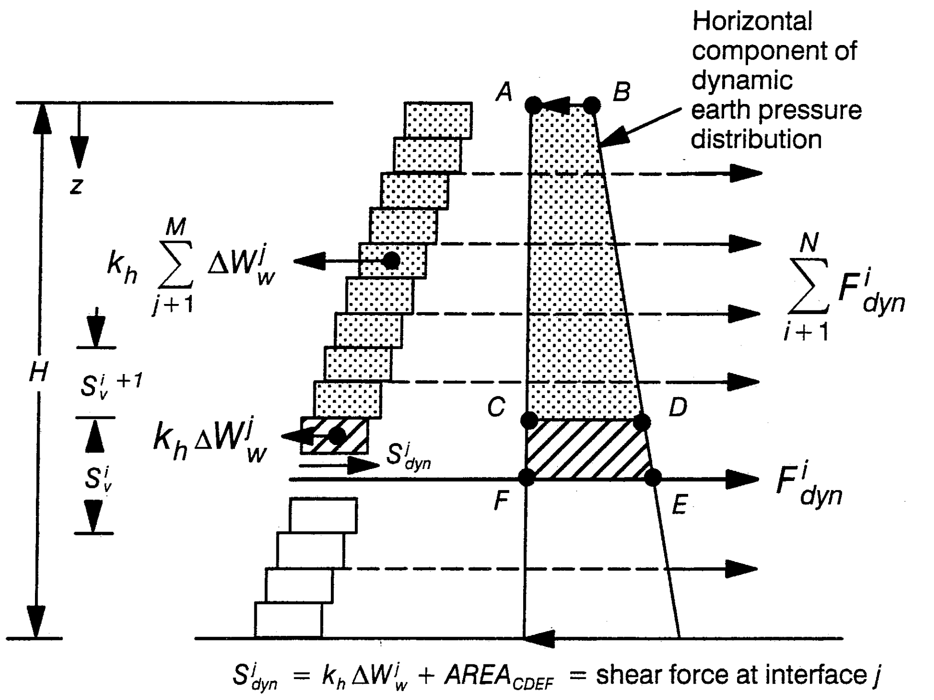



**Fig 10.** Calculation of dynamic interface shear force acting at a reinforcement elevation. (Notes: $N$ = total number of reinforcement layers; and $M$ = total number of facing units.)

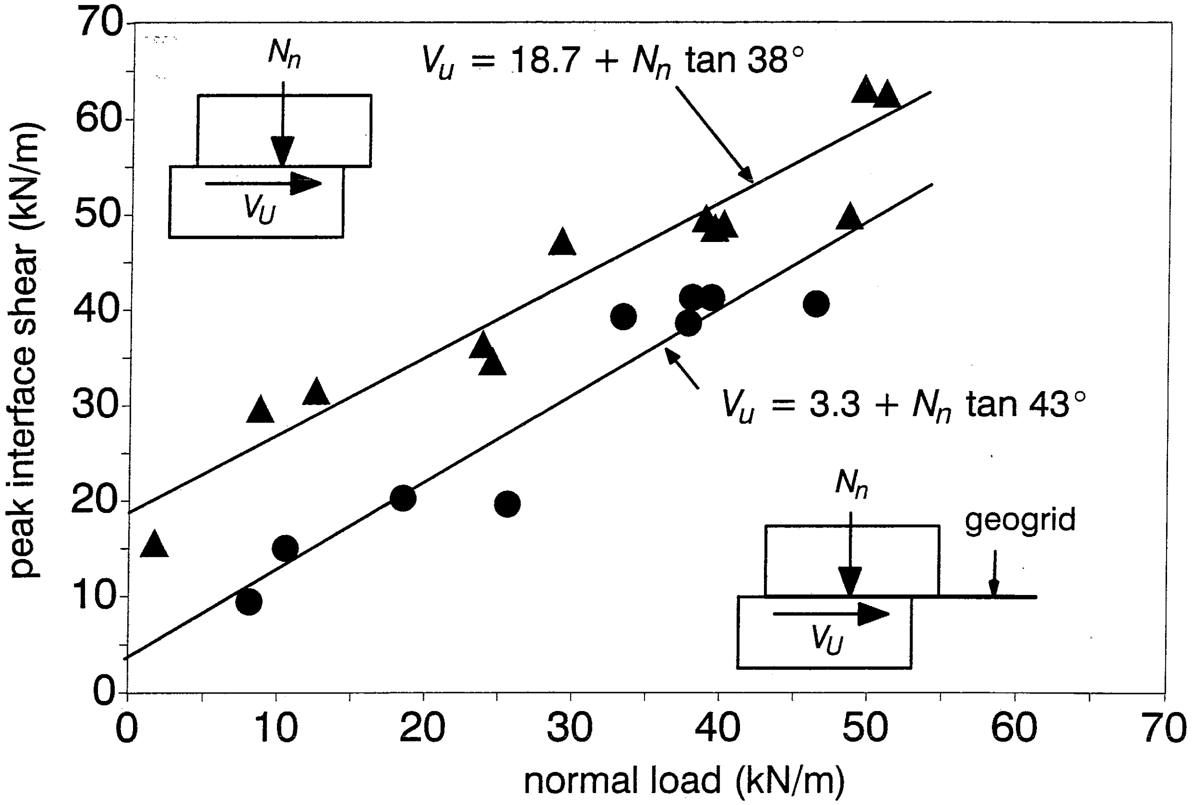



**Fig 11.** Influence of geosynthetic layer inclusion on peak interface shear capacity (gravel infilled unit with concrete shear key).

displacement criterion is typically 2% of the height of the facing unit. In some systems the peak and serviceability criterion envelopes are best described through bi-linear curves using a maximum shear capacity cut-off at some threshold magnitude of shear capacity $V_u$.

The influence of the magnitude of acceleration coefficients on interface shear stability is illustrated in **Figure 12**. The data shows that the potential for interface shear failure under seismic loading increases with proximity of the interface to the crest of the wall. However, the effect of vertical acceleration on calculated dynamic factors of safety diminishes with height of interface above the base of the wall for the worst case condition of vertical seismic force components acting upward. The curves for $z/H$=0.1 correspond to local stability of the top unreinforced portion of the wall facing. These curves appear to support the argument that narrow unreinforced heights of segmental facing units are susceptible to local shear failure. However, in practice, large values of interface shear capacity are possible with many modular block systems that are constructed with shear keys, or other forms of positive interlock. These systems, as opposed to systems that rely solely on frictional sliding resistance, are the preferred choice in order to achieve an adequate margin of safety against interface shear.

In the paper by Bathurst and Cai (1995) a factor of safety expression for local overturning about the toe of any facing unit is presented. Parametric analyses show that it is the top unreinforced portion of the facing column that is most susceptible to toppling. Clearly, this portion of the wall can be treated as a separate gravity structure. Similar to the comments made with respect to column shear stability, the superimposed effect of a vertical acceleration component was also shown to be insignificant for $k_h < 0.3$.

The only strategy to minimize the potential for local shear and toppling failure mechanisms is to introduce reinforcement layers close to the wall crest and to ensure that these layers have adequate facing connection capacity and anchorage length. Although the results of analyses suggest that toppling of the facing column is a potential problem, the results of the analyses reported by Bathurst and Cai (1995) for two walls that survived the Northridge earthquake in 1994 show that this problem did not develop in practice because reinforcement layers were placed close to the top of the structure and initial static factors of safety against local overturning were very high.

## Displacement methods

As with all limit-equilibrium methods of analyses, the pseudo-static approach cannot explicitly include wall deformations. This is an important shortcoming since failure of segmental retaining wall systems may be manifested as unacceptable movement without structural collapse.

The permanent displacement of a geosynthetic reinforced segmental soil retaining wall due to the sliding or column shear mechanisms introduced earlier can be estimated using one of two general approaches (Cai and Bathurst 1996a). For a given input acceleration time history, Newmark's double integration method for a sliding block can be used to calculate the permanent displacement (Newmark 1965). However, if the input acceleration data is specified only by characteristic parameters such as the peak ground acceleration and the peak ground velocity, then empirical methods that correlate the expected permanent displacement to the characteristic parameters of the earthquake and a critical acceleration ratio for the structure are required. Alternatively, if the tolerable permanent displacement of

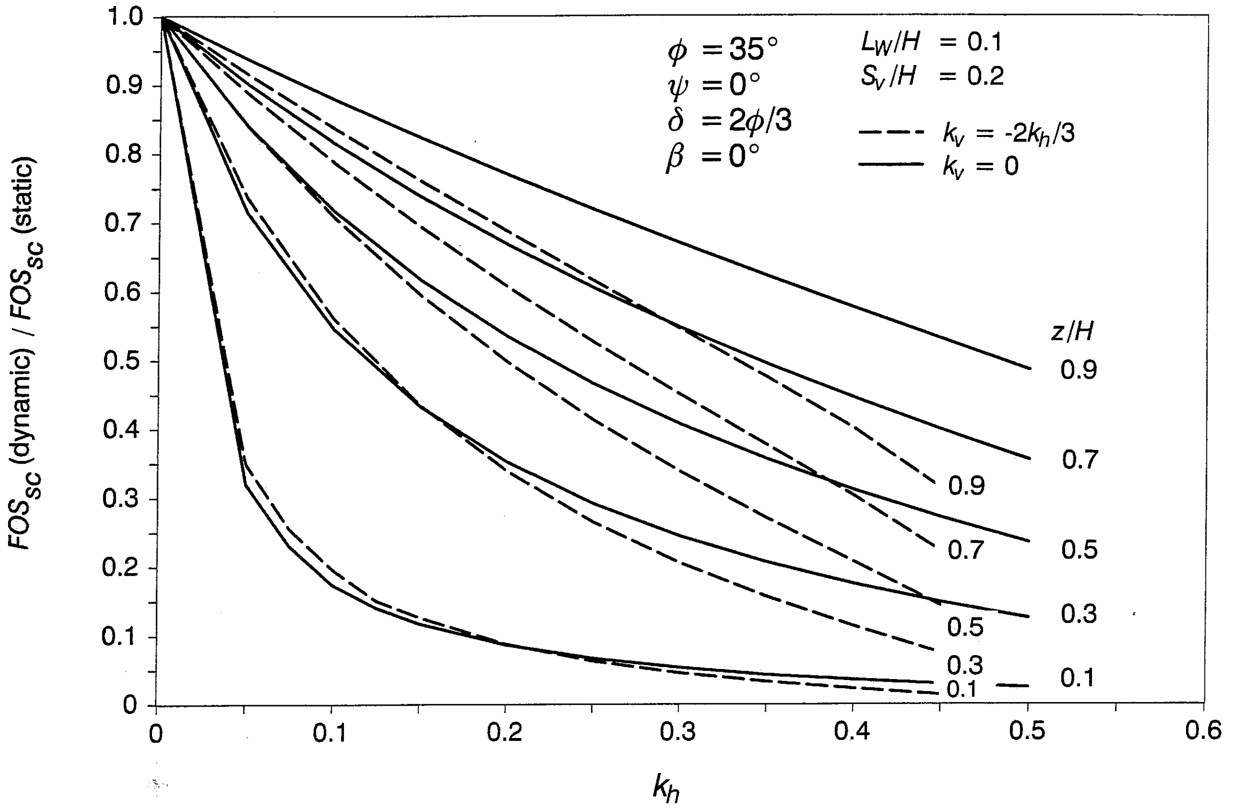



**Fig 12.** Influence of seismic coefficients, $k_h$ and $k_v$, and normalized depth below crest of wall, $z/H$, on the ratio of dynamic to static interface shear factor of safety.

the structure is specified, based on serviceability criteria, the wall can then be designed using an empirical method so that expected permanent displacements do not exceed specified values.

Geosynthetic reinforced soil *segmental* retaining walls can be considered to be a special class of gravity wall structures. However, adaptations of rigorous sliding block methods or empirical methods are restricted to sliding or shear mechanisms: i.e. 1) external sliding along the base of the total structure which includes the reinforced soil mass and the facing column (**Figure 3a**); 2) internal sliding along a reinforcement layer and through the facing (**Figure 3f**) and; 3) interface shear between facing units with or without the presence of a geosynthetic inclusion (**Figure 3h**).

## Newmark's method

According to Newmark theory, a potential sliding body is treated as a rigid-plastic monolithic mass under the action of seismic forces. Permanent displacement of the mass takes place whenever the seismic force induced on the body (plus the existing static force) overcomes the available resistance along the potential sliding/shear surface. Newmark's method adapted to the segmental retaining wall problem requires that the critical acceleration $k_c$ to initiate sliding or shear failure be determined for each translation failure mechanism. The value of $k_c$ can be determined by searching for values of $k_h$ that give a factor of safety of unity in pseudo-static factor of safety expressions. The critical acceleration is then applied to the horizontal ground acceleration record at the site and double integration is performed to calculate cumulative displacements as illustrated in **Figure 13** where: g is the gravitational constant; $a(t)$ is the horizontal ground acceleration function with time $t$; $a_m = k_m g$ is the peak value of $a(t)$; and, $a_c = k_c g$ is the critical horizontal acceleration of the sliding block. For a given ground acceleration time history and a known critical acceleration of the sliding mass, the earthquake induced displacement is calculated by integrating those portions of the acceleration history that are above the critical acceleration and those portions that are below until the relative velocity between the sliding mass and the sliding base reduces to zero.

## Empirical approaches

Newmark's sliding block theory has been widely used to establish empirical relationships between the expected permanent displacement and characteristic seismic parameters of the input earthquake by integrating existing acceleration records. The critical acceleration ratio, which is the ratio of the critical acceleration ($k_c g$) of the sliding block to the peak horizontal acceleration ($k_m g$) of the earthquake, has been shown to be an important parameter that affects the magnitude of the permanent displacement. Thus, the seismic displacement of a potential sliding soil mass computed using Newmark's theory has been traditionally correlated with the critical acceleration ratio ($k_c/k_m$) and other representative characteristic seismic parameters such as the peak ground acceleration ($k_m g$), the peak ground velocity ($v_m$), and the predominant period ($T$) of the acceleration spectrum (e.g. Newmark 1965; Sarma 1975; Franklin and Chang 1977).

The writers have reformulated a number of existing displacement methods based on non-dimensionalized displacement terms that are common to the methods, and divided them into two separate categories based on the characteristic seismic parameters referenced in each method (Cai and Bathurst 1996b). An example relationship between dimensionless displacement term $d/(v_m^2/k_m g)$, where $d$ is the actual expected permanent displacement, and the critical acceleration ratio is shown in **Figure 14**. Other similar curves are available in the literature but it should be noted that any empirical curve will be influenced by the earthquake data that is used to establish the curve and the interpretation of the original data. Predicted displacements must be viewed as order-of-magnitude estimates rather than accurate predictions. Engineering judgement plays an important role in the interpretation of results using any empirical approach.

## Example

A summary of calculation results for the geosynthetic reinforced soil wall structure shown in **Figure 15** is given in **Table 1**. The material properties for the facing units have been taken from large-scale laboratory tests carried out at RMC. The block-geosynthetic interface shear properties represent a system with relatively low interface shear capacity in order to generate a worst case set of displacement predictions. The E-W (90°) horizontal ground acceleration component recorded at Newhall Station (California Strong Motion Instrumentation Program) during the 17 January 1994 Northridge earthquake (M=6.7) was

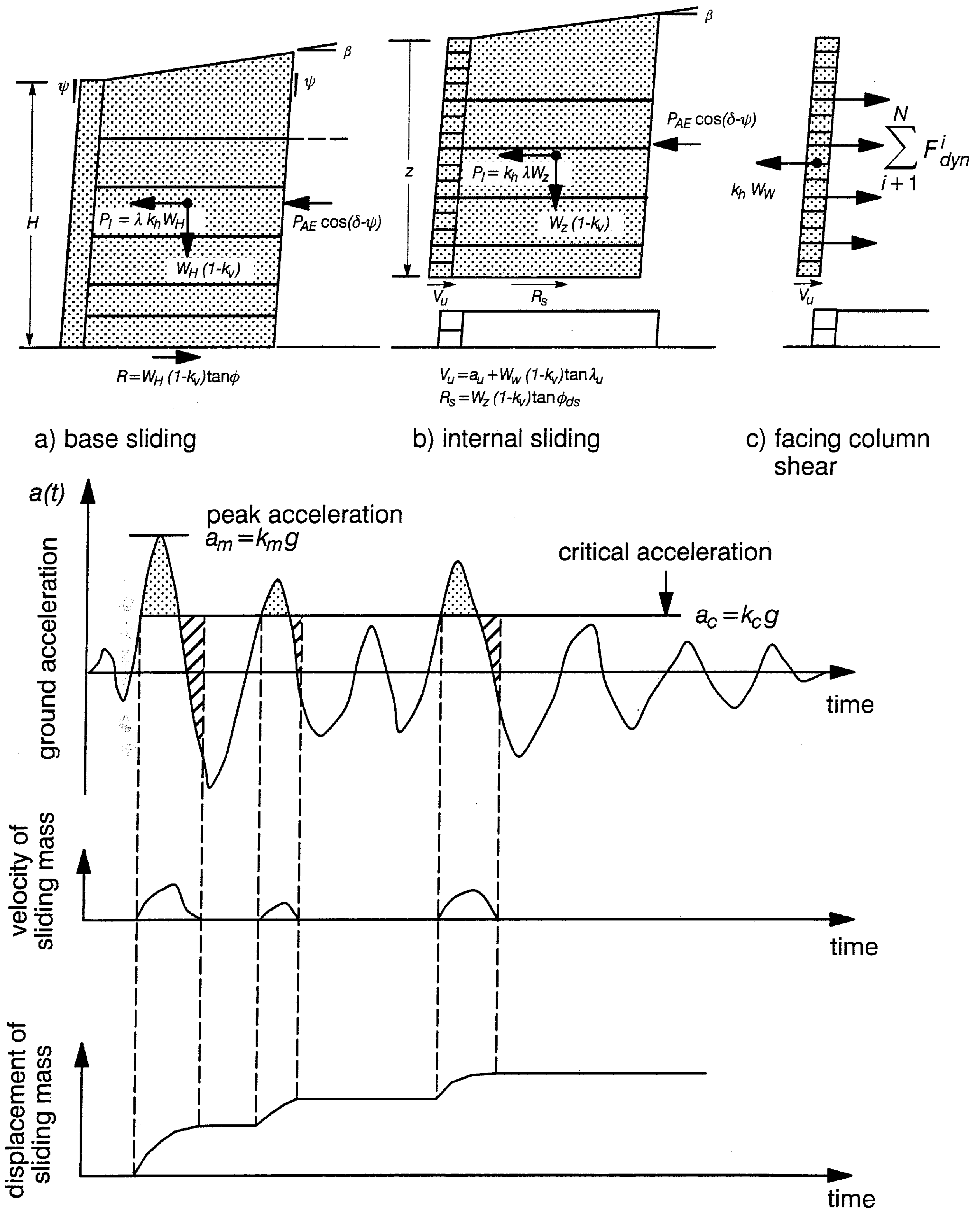



**Fig 13.** Newmark's sliding block method applied to geosynthetic reinforced segmental retaining wall structures.

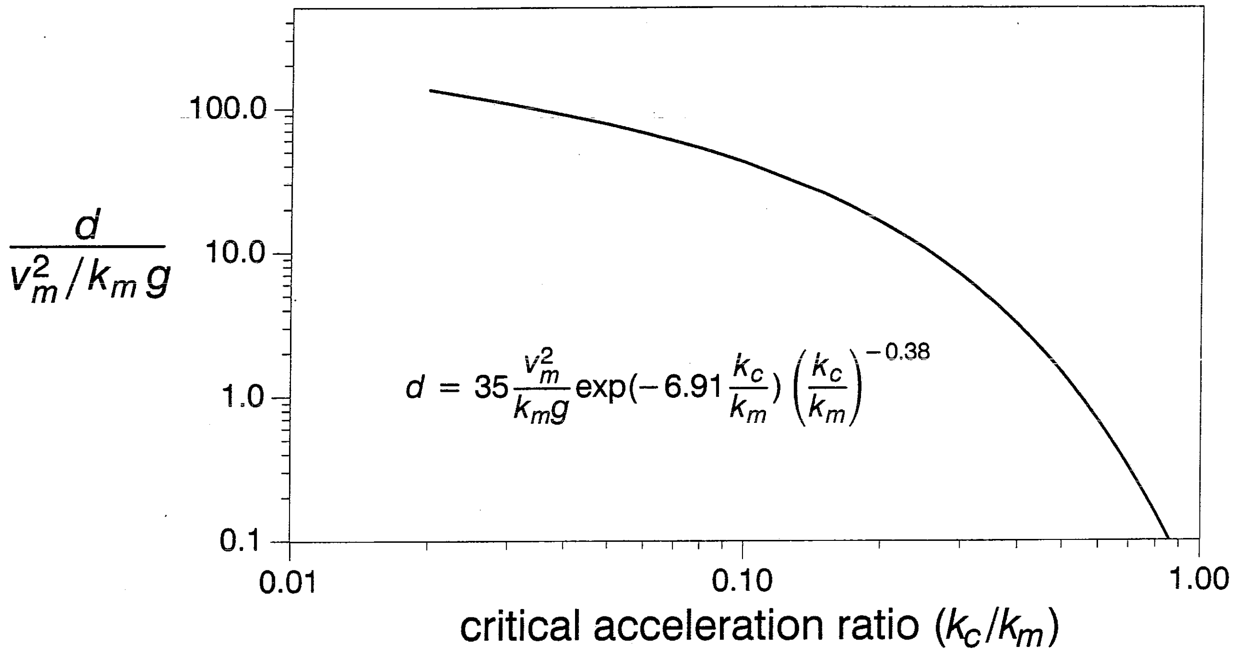
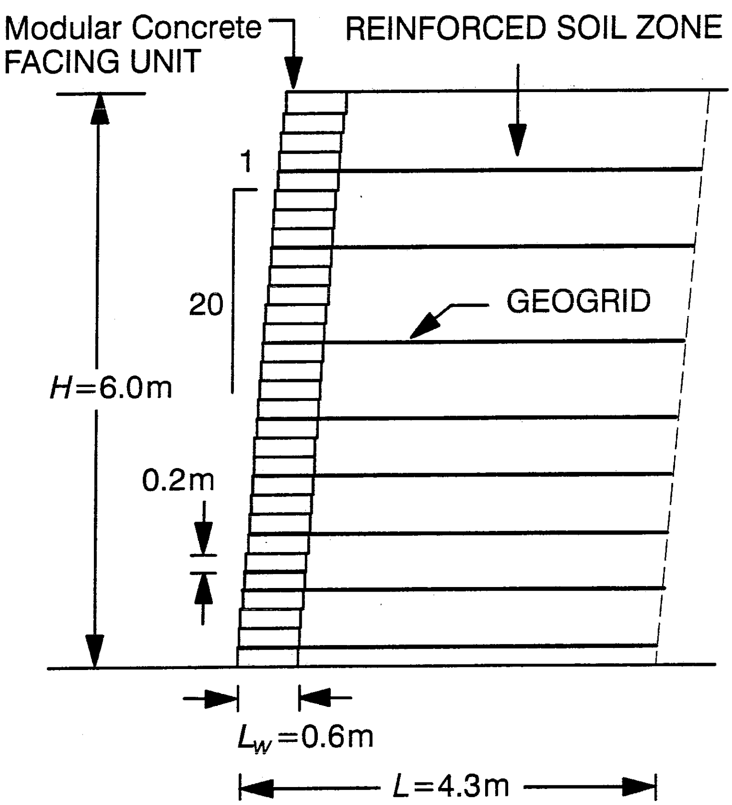



**Fig 14.** Example relationship between non-dimensionalized displacement term and critical acceleration ratio (after Cai and Bathurst 1996b).




| Layer Number | $z/H$ | $S_v/H$ |
|---|---|---|
| 8 | .133 | .2 |
| 7 | .267 | .15 |
| 6 | .433 | .15 |
| 5 | .567 | .117 |
| 4 | .667 | .1 |
| 3 | .767 | .1 |
| 2 | .867 | .1 |
| 1 | .967 | .083 |

**Fig 15.** Example geosynthetic-reinforced segmental soil retaining wall (after Bathurst et al. 1993b).

used as the input earthquake data. The record shows a peak horizontal ground acceleration of $k_m$=0.60. Substituting $k_h$=$k_m$=0.6 in the pseudo-static seismic stability analyses represents a very extreme loading but is introduced here for illustration purposes only. For example, on the west coast of British Columbia (the most seismically active area of Canada), the typical maximum design horizontal ground acceleration on rock, based on a 10% probability of exceedance in 50 years, is $a_h$ = 0.32g (CFEM 1992).

The total permanent displacement at the wall face at each elevation from the initial static position is estimated by adding the layer displacement to the cumulative displacement below that layer. The layer displacement is taken as the larger of the column shear displacement or internal sliding at that layer.

The data in **Table 1** shows that large displacements are possible at the top of the wall. Nevertheless, with a high quality fill (i.e. $\phi$=40°) the sliding of the top four units is greatly reduced. In fact, analyses with better block-geosynthetic properties resulted in insignificant or no displacements at elevations above the base of the example structure for the same extreme loading condition. This last result is consistent with observations made at the site of two segmental retaining walls after the Northridge earthquake that showed no detectable shear movement of the facing column units despite significant horizontal ground accelerations estimated to be as high as 0.5g (Bathurst and Cai 1995).

Newmark's method and the empirical approach give similar results for the three displacement mechanisms examined when large displacement values are calculated (i.e. backfill soils with $\phi$=30° and 35°). The order of magnitude accuracy of the empirical method (compared to Newmark's method) is satisfied for all large displacement results. The discrepancies for displacements less than (say) 12 mm in the analyses with $\phi$=40° are larger but can be judged to be of little practical consequence because of their small magnitudes.

## Finite element methods

The writers have carried out dynamic finite element modelling of geosynthetic reinforced segmental retaining walls in order to investigate the entire load-deformation response of an example system under simulated earthquake loads (Cai and Bathurst 1995). The attraction of properly formulated finite element methods is that they can implement complex models for the component materials such as non-linear and hysteretic properties of the soil and reinforcement materials.

A 2-D dynamic finite element code, TARA-3, originally developed at the University of British Columbia (Finn et al. 1986), has been modified and extended by the writers to account for the effects of extensible reinforcing elements subjected to cyclic loading. The shear behavior of granular soils under cyclic loading is modelled using a non-linear and hysteretic constitutive relation that follows the Masing rule during unloading and reloading. The load-strain cap/hysteretic unload-reload model adopted for the soil materials (Yogendrakumar and Bathurst 1992) has been modified for polymeric reinforcement materials and is illustrated in **Figure 16**. In order to determine cyclic load parameters for the reinforcement model, in-isolation cyclic load tests were carried out on typical polymeric

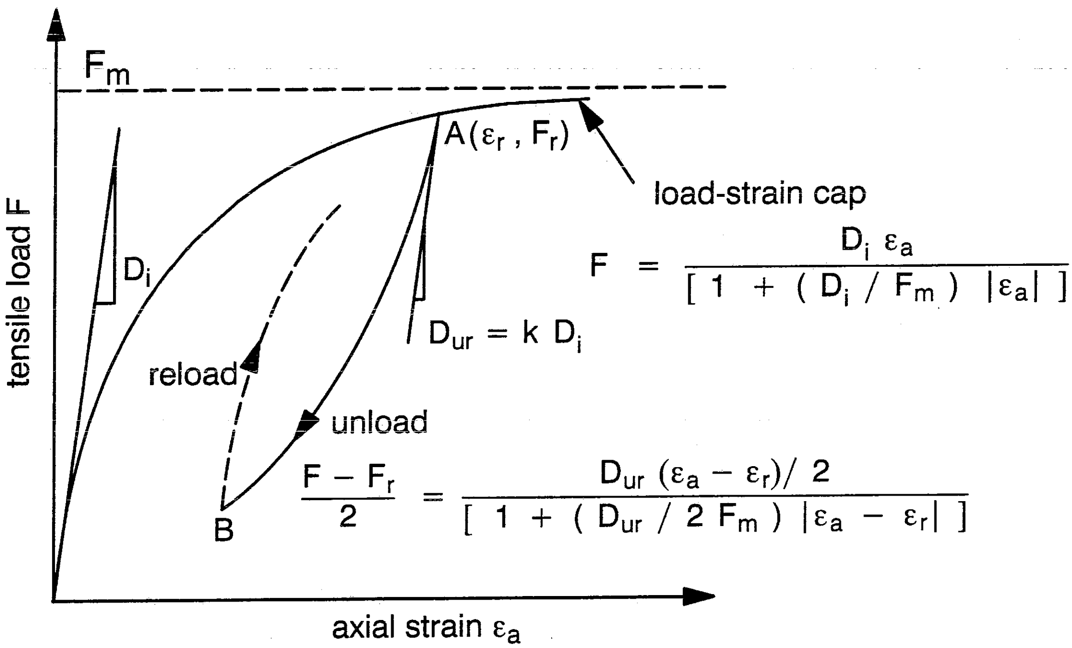



**Fig 16.** Cyclic unload-reload model for polymeric reinforcement.

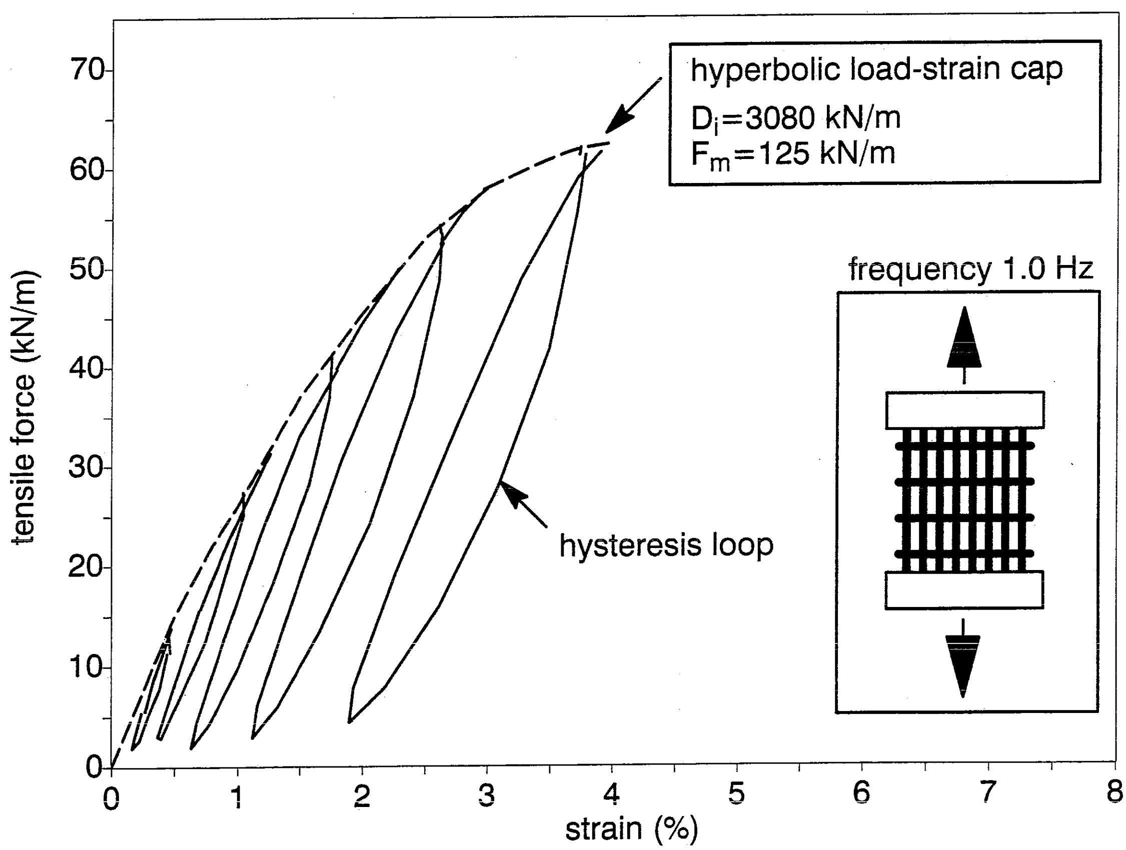



**Fig 17.** In-isolation cyclic load test on a HDPE geogrid (Cai and Bathurst 1994).

geogrid reinforcement materials (Bathurst and Cai 1994). Example results are illustrated in **Figure 17**.

**Table 1. Total permanent displacement considering all displacement mechanisms.**

| Layer number | Soil friction angle | | | | | |
|---|---|---|---|---|---|---|
| | $\phi=30^{o}$ displacement (mm) | | $\phi=35^{o}$ displacement (mm) | | $\phi=40^{o}$ displacement (mm) | |
| | Newmark's method | Empirical method | Newmark's method | Empirical method | Newmark's method | Empirical method |
| 8 | 286* | 370* | 154* | 206* | 86* | 126* |
| 7 | 158* | 198* | 47* | 70* | 12* | 26* |
| 6 | 126* | 160* | 29* | 49* | 1.8 | 12 |
| 5 | 116 | 146 | 25 | 41 | 1.2 | 8 |
| 4 | 116 | 140 | 25 | 41 | 1.2 | 8 |
| 3 | 113 | 133 | 25 | 41 | 1.2 | 8 |
| 2 | 103 | 119 | 24 | 36 | 1.2 | 8 |
| 1 | 85.4 | 97 | 21 | 29 | 1.2 | 8 |
| Base sliding | 48 | 54 | 11 | 15 | 0.6 | 4 |

Note: * controlling mechanism is facing shear, otherwise internal sliding controls.

The results of large scale interface shear and connection tests were used to provide input parameters for the modelling of the facing column. The interface shear capacities that were used are considered to be relatively poor for segmental retaining wall systems, based on a large amount of testing at RMC.

The base reference acceleration-time history used is a scaled El-Centro 1940 earthquake record. Spectrum analysis of the input acceleration record gives a dominant frequency range of 0.5 to 2 Hz. Details of the finite element model, constitutive models and selected parameters can be found in the paper by Cai and Bathurst (1995).

Predicted cumulative lateral deformations through the height of the facing column at the end of two scaled base input records are illustrated in **Figure 18**. The relative displacements are largest at reinforcement elevations where locally greatest interface shear loadings occurred. While the potential for interface shear leading to collapse of these structures is clear it is worth noting that the vertical out-of-alignment is less than 1% of the height of the wall. In practice this amount of relative displacement is within the limits usually achieved during construction (Bathurst et al. 1995) and, hence, from practical considerations may be judged to be insignificant. The results suggest that for the range of peak accelerations and duration of excitation applied to this low height wall, the structure performed well despite relatively poor interface shear characteristics.

An important observation made by the writers was that reinforcement forces predicted by the FE model were consistently lower than those computed using the pseudo-static M-O approach described earlier. This result is consistent with the opinion of many practitioners that M-O earth pressure is conservative for routine soil retaining wall structures.

## SHAKING TABLE TESTS

In order to investigate further the influence of interface shear properties on facing column stability, a set of 1/6 scale model shaking table tests was carried out at RMC. The shaking table has a rated payload capacity of 4500 kg and is driven by a 100 kN actuator with a $\pm 75$ mm horizontal stroke. The model walls were constructed inside a plexiglas box and are 2400 mm long by 1400 mm wide by 1020 mm high. A typical test configuration is illustrated in **Figure 19.**

The models were constructed with concrete blocks 100 mm wide (toe to heel) x 160 mm wide x 34 mm high. Five layers of a weak geogrid (HDPE bird fencing) were used to model the reinforcement. The backfill was a standard laboratory silica 40 sand prepared at a relative density of 67%.

Four test configurations are discussed here and are summarized in **Table 2**. The differences between tests are related to interface shear capacity and wall batter. Interfaces identified as frictional in the table derive shear capacity solely from sliding resistance at the interface. These interfaces represent a very poor facing column detail with respect to shear capacity. In two of the tests the interfaces were fixed at some locations in order to simulate systems with high shear capacity at all or selected facing column interfaces (i.e. positive interlock due to effective shear keys, pins or other types of connectors).

Each test was subjected to a staged increase in base input motion resulting in the acceleration-time record shown in **Figure 20**. The base input frequency was kept constant

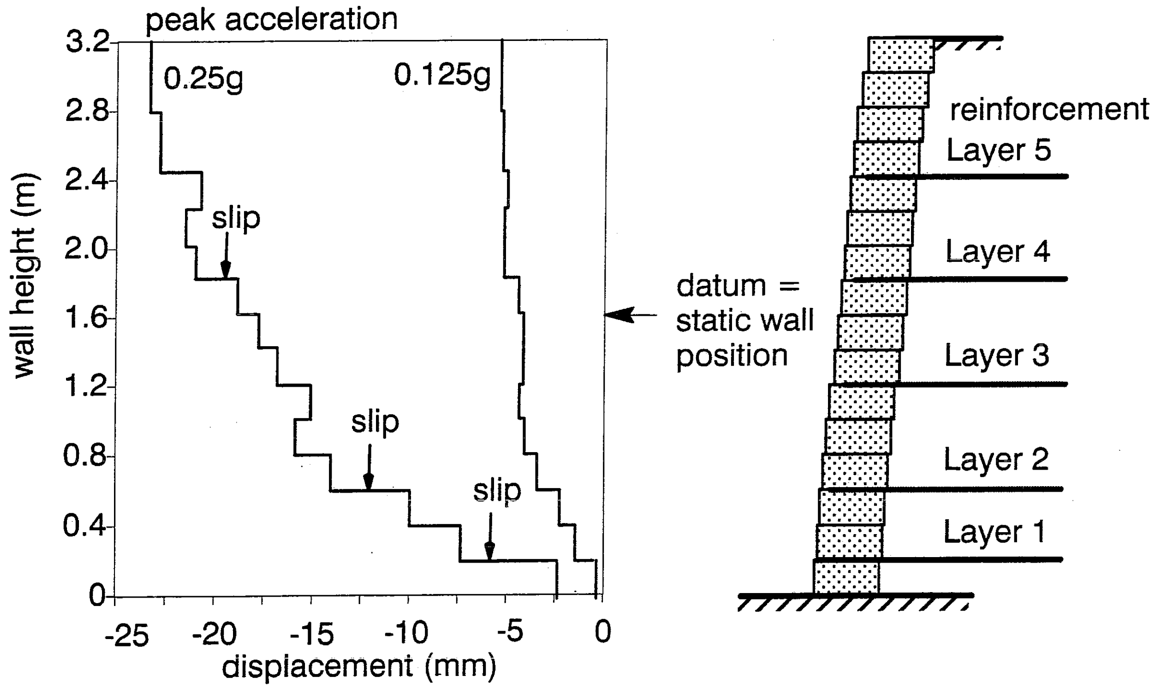



**Fig 18.** Lateral displacement profiles along wall facing at end of excitation history.

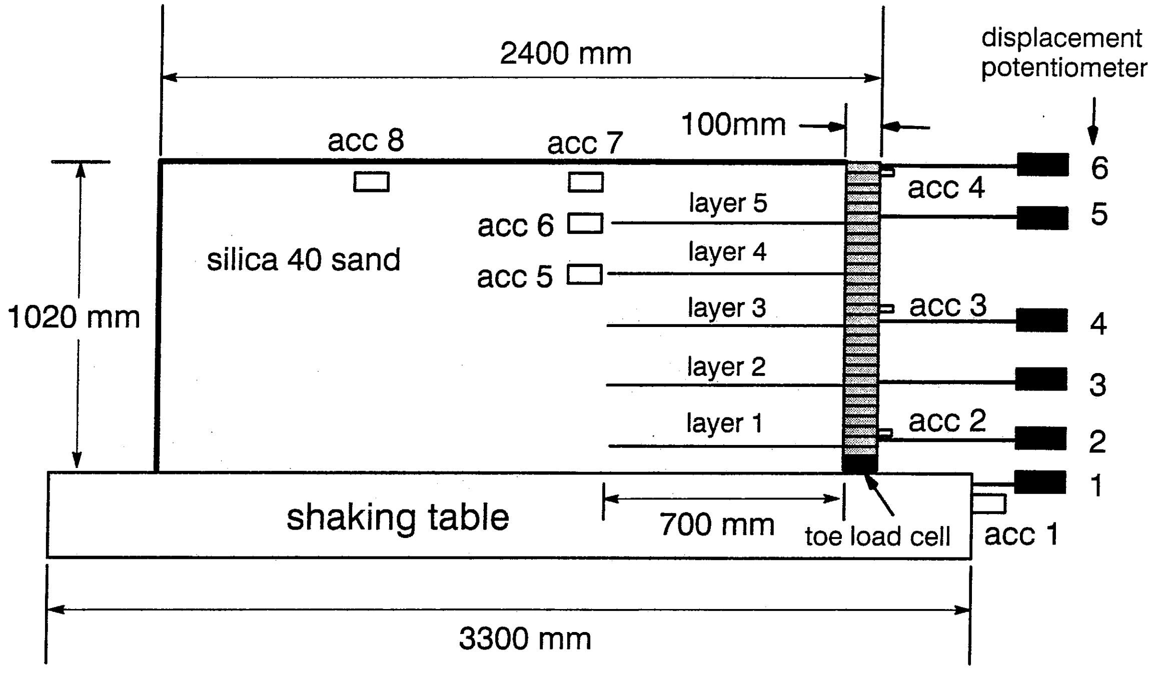



**Fig 19.** Example shaking table test model of reinforced segmental retaining wall.

at 5Hz. At the prototype scale this frequency corresponds to 2 Hz.

**Table 2. Model test configurations.**

| Test number | Facing batter | Block-block interface | Block-geosynthetic interface |
|---|---|---|---|
| 1 | vertical | frictional | frictional |
| 2 | vertical | fixed | frictional |
| 3 | 8 degrees* | frictional | frictional |
| 4 | vertical | fixed | fixed |

* from vertical

The influence of interface shear capacity and facing batter can be seen in **Figure 21**. The vertical wall with fixed interface construction (high shear capacity at each interface) required the greatest input acceleration to generate large wall displacements during staged shaking (Test 4). The vertical wall with poor interface shear at all facing unit elevations performed worst (Test 1). However, the resistance to wall displacement was improved greatly for the uniformly weakest interface condition by simply increasing the wall batter (Test 3). The vertical wall with poor interface properties only at the geosynthetic layer elevations (Test 2) gave a displacement response that fell between the results of walls constructed with uniformly poor interface shear properties (Test 1) and the nominally identical structure with uniformly good interface shear properties (Test 4).

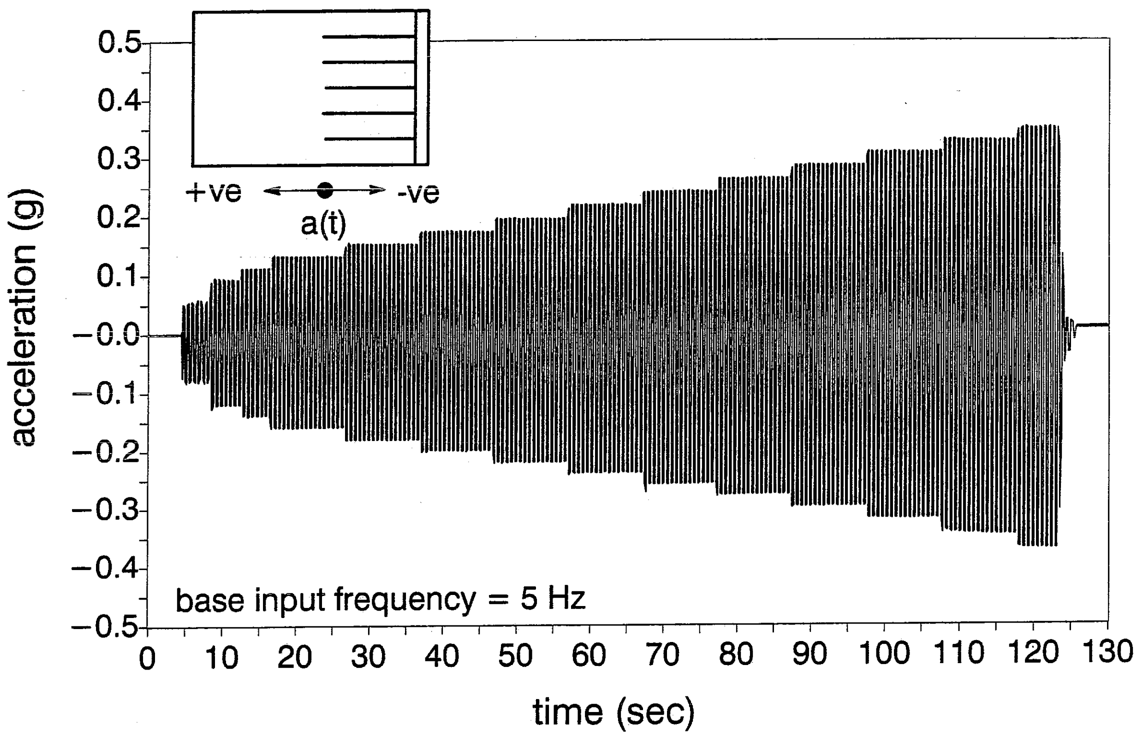



**Fig 20.** Base input acceleration record for shaking table tests.

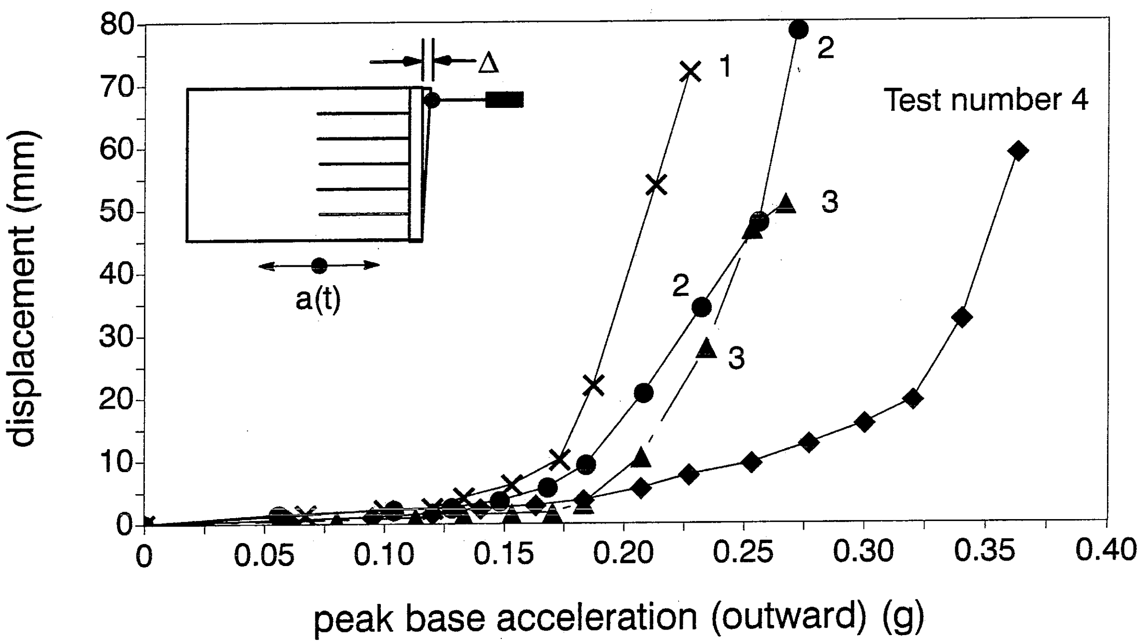



**Fig 21.** Displacement close to top of wall versus peak base input acceleration.

## CONCLUSIONS

The results of research on the analysis, design and performance of geosynthetic reinforced segmental retaining walls has led to a number of important observations and conclusions by the writers.

- Segmental retaining walls are discrete systems with potential failure mechanisms not found in conventional gravity wall structures or geosynthetic reinforced soil wall systems with continuous hard facings. Acceptable factors of safety against these unique failure mechanisms are required to ensure satisfactory performance in both static and seismic loading environments.

- Conventional Coulomb wedge methods of analysis can be easily extended to account for the additional destabilizing forces that will act on a segmental retaining wall during a seismic event. These methods are based on Mononobe-Okabe earth pressure theory and are familiar to geotechnical engineers since the general approach has been used for many years for the design and analysis of conventional gravity wall structures.

- Block-geosynthetic connection properties and interface shear properties are important parameters in design and analysis. These properties vary widely between facing systems available on the market today. Consequently, large-scale laboratory testing is required to quantify these properties for design. The influence of a geosynthetic inclusion at an essentially frictional shear interface is an important detail in the analysis, design and performance of reinforced SRWs. However, there are a number of modular block systems available on the market today that incorporate efficient shear transfer devices and provide connections that will ensure local stability of the facing system under both static and (moderate) seismic loading conditions.

- The most critical part of a geosynthetic reinforced segmental retaining wall under seismic loading is the unreinforced facing column at the top of the wall structure. The number of reinforcement layers at the top of a SRW may have to be increased in order to ensure an adequate margin of safety against column shear failure or toppling.

- The length of reinforcement layers close to the crest of a SRW may have to be increased beyond lengths required for static design in order to ensure adequate anchorage length for these layers under seismic loading conditions.

- Displacement methods hold promise as a technique to evaluate segmental retaining wall performance based on deformation (serviceability) criteria. However, the methods as currently developed by the writers, are restricted to translational modes of failure. In addition, empirical displacement methods can at best provide order-of-magnitude estimates of wall displacements. Predicted values can be expected to vary greatly depending on the empirical method selected. In practice, these methods are best suited to making relative performance comparisons between nominally identical wall structures constructed with different candidate block systems.

- Finite element methods are a useful research tool to examine the entire load-deformation response of segmental retaining walls. Nevertheless, these methods require properties for the component materials that are typically not readily available and the interpretation of numerical results requires experienced modellers.

- Small scale shaking table tests have confirmed the importance of interface shear properties on the facing column stability in simulated SRW structures.

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