Seismic Design and Performance of Green-Faced Geotextile Reinforced Retaining Structures

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ABSTRACT: The concepts of environmental sustainability and green infrastructure are resulting in an increase in the use of green-faced retaining walls for applications in land development and in the management of grade changes. Volumetric changes and instability in the biodegradable organic materials that are used as growing media at a wall face (face units in some wall configurations) can influence the design and performance of these walls. Using geotextiles as a primary and/or secondary reinforcement in the mechanically stabilized portion of the wall adds complexity and uncertainty to the design due to the stress-strain characteristics of the materials and potential durability issues, relative to the more traditionally used geogrid reinforcement elements. These aspects of wall design are discussed in this study and a new configuration is introduced to mitigate these compounding variables. A review of pertinent literature and guidelines for seismic design of retaining structures in high seismicity zones is introduced to assist with the selection of design approach and level of analysis. The results of a parametric study are summarized to illustrate the affects of surcharge and backfill geometry on the calculated pseudo-static stability of geotextile reinforced walls.

Keywords: Retaining walls, geotextiles, sustainability, seismic earth pressures, deformations.

Introduction

Geotextiles are widely used in various earthwork applications including roads, slopes, embankments, and permanent and temporary retaining walls. Case histories support the use of geotextile fabrics in wall construction up to about ten metres high. Typically, these geotextile-reinforced walls can tolerate the high deformations associated with soil movement and/or settlement. A number of design approaches have been advanced to predict the behaviour and performance of geotextile-reinforced walls, as is described below. However, uncertainty in performance remains, particularly in the domain of seismic response. Experimental studies using the shake table indicate that the obtained results vary from those of full scale walls (Bathurst and Alfaro, 1997, and Bathurst and Hatami, 1998). Researchers continue efforts to better understand the performance of these structures.

The use of geotextiles to reinforce engineered earth fills has a number of potential advantages in comparison to the more traditionally used geogrids. Considerations include reduced cost, ease of handling and installation and improved interaction (planer over grid) due to higher apparent covering ratio. Fig. 1 presents (a) stress-strain curves for three geosynthetic products and (b) the coefficient of interaction for two products to illustrate these characteristics. As noted, the strength and stiffness of the geotextile product are higher than those for the geogrid.

Geotextile products provide opportunity for good soil confinement, which in turn results in greater load distribution. Finer grained soils can be retained by geotextiles, providing opportunities for the use of on site soils that might otherwise be marginalized. Geotextiles, if embedded in a cohesive soil, can enhance lateral drainage as well as serving to provide reinforcement. The soil interaction coefficients of geotextiles are frequently greater than those obtained through geogrid interlock. Another advantage of geotextiles relates the frequent ability to utilize the length of the roll parallel to the alignment of the wall. Such considerations have resulted in an increased desire from contractors and developers to consider geotextiles in preference of geogrids for reinforced earth applications.

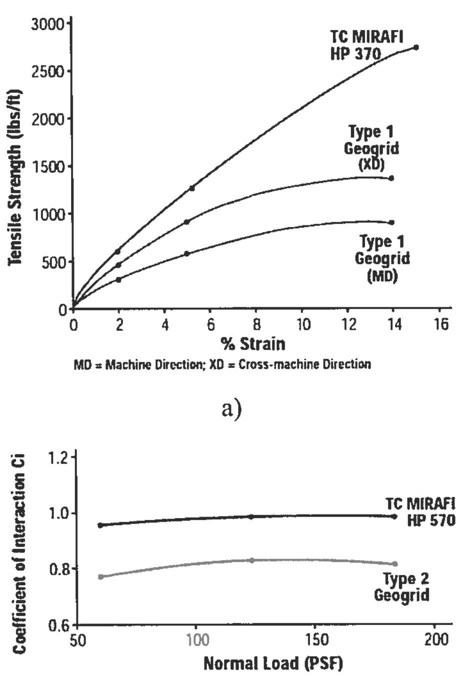
The focus of this paper is the design and performance of green faced geotextile-reinforced walls under seismic loads.

Geotextiles for retaining walls

Geotextiles are typically composed of three synthetic polymers; polyester, polyethylene and polypropylene. Certain products can undergo large creep deformations under static loading conditions with the highest values typically being associated with polypropylene products. The tensile strength of geotextiles varies considerably depending on various factors and can approach 1000 kN/m for polyester multifilament yarns. Commonly used geotextiles, however, have a strength range of 10 to 70 kN/m with costs that are nominally less than half those of geogrid products. Typically, geotextile products require greater strain than geogrids to develop the required or specified strength for stability. For instance, geotextiles may exhibit strains up to about 25% or more before failure. However, there is a wide range of available products and some high strength geotextiles provide over twice the ultimate strength, and twice the strength at 2% and 5% strain, as compared to more commonly used geosynthetics including some geogrids.

Mechanically stabilized earth (MSE) walls are normally designed with 2% to 5% lateral strain depending on the application under consideration. Therefore, in considering geotextiles for application in wall design, the geotechnical engineer must be aware of and consider the deformation characteristics of the particular geotextile. The consideration of strain characteristics in wall design is more critical for geotextile reinforcement than for the more commonly used geogrids. Limit equilibrium methods for stability analysis of slopes and global wall stability do not address this design aspect. These potential deformations could be critical for high or sensitive structures.

Fig. 1. Coefficient of interaction and tensile strength of geotextiles compared to geogrids.



b)

Green-faced walls

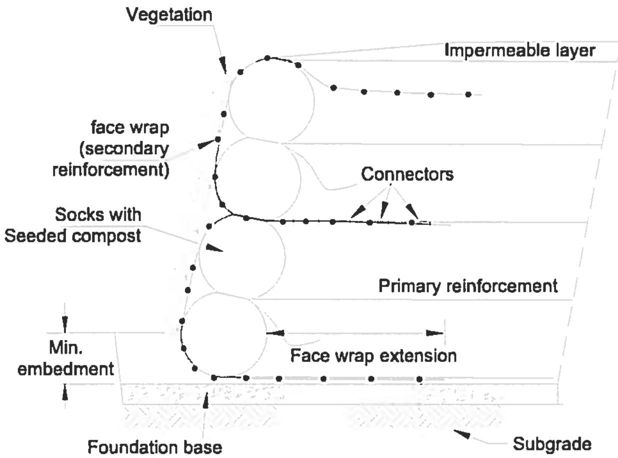
Land development in general is incorporating increased attention to the concepts of environmental sustainability and the use of green infrastructure. As a result, there has been an increased demand to consider green-faced retaining walls. These walls incorporate a growing medium within the face to allow vegetation to grow. Beneficial characteristics, aside from aesthetics, include sound reduction and improved stormwater management. Economically, the green-facing is typically less costly than concrete type segmental alternatives. The wrapped-face wall is a green-faced alternative which recent projects have shown to be less costly and quicker to install than more traditional systems such as wire-mesh and segmental systems.

Green walls have specific face elements for growing vegetation; a particular prototype design incorporates flexible (deformable) geotextile tubes that are infilled with seed-impregnated growing medium such as compost or bark mulch, as shown on Fig. 2. In this wall configuration, the encapsulated tubes of growing media are retained within the face of primary reinforcement elements. These walls can accommodate large total and differential settlement without collapse and are well suited to strain tolerant landscape areas. In less strain tolerant environments, however, the potential volumetric changes in the unstable biodegradable materials used as growing media at the wall face require careful consideration in terms of the design and long term performance of wrapped-face retaining walls.

Degradation of the organic material may cause volumetric shrinkage leading to soil migration and a loss of tension in the geotextile reinforcement. Alternatively, plant growth might result in an increase in the volume of the face element putting more tension in the geotextiles. Utilizing a configuration such as that shown in Fig 2 introduces greater uncertainty in wall performance due to a reliance on the facing material by the primary reinforcement. The way the wall will perform (shrinkage or expansion of face elements) is difficult to predict and is dependant on actual environmental condition and maintenance. For small height landscaping walls, the effects of shrinkage or expansion may be tolerable. However, for tall and/or more critical walls, such as those required to support infrastructure or buildings, the uncertainty in long term performance of the organic component typically results in the Fig 2 design configuration being unacceptable.

In order to overcome the limitations described above, the authors developed an alternative approach to the green-faced wall design. This approach is able to accommodate volumetric changes without hindering the performance of the structural component of the mechanically stabilized wall.

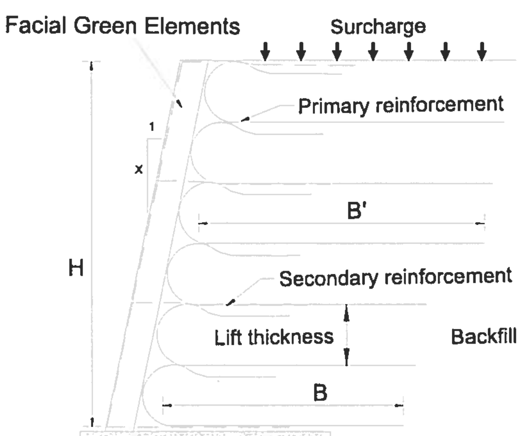
Fig. 2. Flexible Vegetated-Face MSE Wall.



Proposed wall configuration

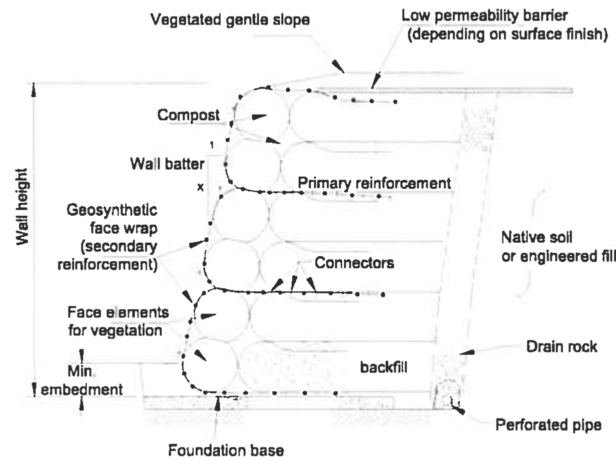
The authors concluded that volume changes in the biodegradable (compost or bark mulch) units at the face are inevitable. Reliable long term control of volumetric changes is difficult if not impossible. In order to reliably predict the performance of the wall, the structural components of the wall should be dependant upon non-organic materials with engineering properties that are reliable over the life time of the structure. The alternative configuration introduced by the authors introduces an approach to overcome the need for the primary reinforcement to be dependent upon the growing medium (Fig. 3). In this alternative approach, the geotextile bound vegetation elements are attached to every second primary reinforcement element using a system of secondary reinforcement. Typically, the primary reinforcement in this design is vertically spaced at nominal 300 mm intervals.

Fig. 3. Green-Faced Wrap-around MSE wall.



This alternative configuration provides opportunity for the use of different types of growing media at the wall face including growing media/compost packed in polyester fabric (socks), lockdown netting or a vegetation blanket as the covering elements (secondary facing) to achieve the goal of green-facing. The secondary reinforcement connection between the main wall and the green facing elements is designed to withstand potential static and, if applicable, seismic loads taking into consideration drainage, exposure to UV rays and construction damage. An example of a green-faced wall using compost socks as a growing media is shown on Fig. 4.

Fig. 4. Proposed Geometry of Vegetated Wrapped-Face MSE Wall.



Design criteria

The seismic stability of non-critical earth-retaining structures usually commences with the pseudo-static approach. For a MSE wall, external and internal stability need to be considered. The use of the pseudo-static approach may not be sufficiently complete for tall walls or walls with low strain tolerance due to the inability to address time-domain variations.

The pseudo-static approach is heavily weighted on amplitude of the seismic event (e.g., the Peak Ground Acceleration, PGA). However, this is not the only parameter which influences the seismic response of the structure. The dynamic effects of earthquake shaking are complex, transient and interrelated. Representing these effects by a single constant unidirectional pseudo-static acceleration is a crude approximation, the limitations of which should be understood by the designer.

The authors recommend deformation-based analysis be performed in seismically active regions and for important or high structures. Designing a structure to accommodate small tolerable displacements rather than no permanent displacement is typically advantageous from a cost viewpoint, provided that it can be demonstrated to meet serviceability limit state deformation criteria. Certain building codes (e.g., New Zealand Transit Bridges Manual) stipulate that deformations up to 100mm for rigid walls and 150mm for flexible walls can be acceptable for retaining walls that do not support structures (Wood, 2008). For a performance-based analysis, a suitable serviceability lateral deformation may be specified and the corresponding design acceleration calculated. In Euro Code 8, the permissible displacement under seismic conditions is calculated using Richard and Elms (1979) model.

The most common method of evaluating permanent deformations involves the use of the Newmark-sliding block method. In this method, the ratio of yield acceleration to peak ground acceleration is related to permanent displacements. Various charts and equations have been developed to show this relationship (e.g., Makdisi and Seed, 1978; Bray and Rathje, 1998; ATC/MCEER, 2003). These methods were developed using a combination of simplified Newmark and more rigorous techniques. They often differ in terms of the databases that were used to develop the correlation between peak ground acceleration, yield acceleration and ground displacement.

Nakamura (2006) test results indicate that "rigid" block theory is not correct and that part of the backfill actually deforms plastically while the wall slides. As such, the authors propose using the Bray and Travasarou (2007) method for estimating lateral wall deformations. With some adjustments to account for soil reinforcement effects and natural period of a potential sliding mass, the Bray and Travasarou (2007) equations could be used as a more accurate method to calculate outward displacements. The Bray and Travasarou (2007) method includes an updated database, regression analysis and equations based on Newmark's sliding block theory with consideration to the change in the natural period of the sliding mass. The analysis may be run at different intensities of earthquake shaking in accordance with applicable code requirements and sensitivity studies. Calculations undertaken by the authors indicated that the potential deformations obtained using the Bray and Travasarou (2007) method was heavily influenced by the earthquake moment magnitude (M_w) used in the calculation.

The Canadian Foundation Engineering Manual (CFEM), 4th edition, recommends using the tie-back wedge method for calculating the length of reinforcement for internal stability of MSE walls. This approach can result in long reinforcement elements relative to other approaches. Recent research (e.g., Bthrust et al. 2000) indicates that the tie-back wedge approach is overly conservative and uneconomical. Deformation-based designs are being developed as the next level of analysis to the pseudo-static approach.

Earth pressures

An accurate estimation of dynamic earth pressures is important for seismic design of retaining structures in high magnitude earthquake zones. Several methods are available to evaluate seismic forces, some of which are complicated and beyond the scope of this paper.

The effect of ground motion on retaining walls was recognized by Okabe (1924) and Mononobe (1929) after reviewing damages to retaining structures resulting from earthquake loads. The dynamic earth pressure forces are dependant upon the relative displacement between the soil and the structure (soil-structure interaction). Greater wall movements typically result in reduced earth pressures. Based on shaking table model tests, the active earth pressure state was found to occur at a similar strain level (wall rotation) as that required under static conditions (Ebeling and Morrison, Jr., 1992). The earth pressures under seismic conditions for mechanical stabilized earth walls are usually computed using the Mononobe–Okabe method.

An important but sometimes overlooked factor with respect to earth pressures is the degree of saturation of backfill soil. Infiltration of surface runoff particularly in the landscaping environment can cause saturation of the reinforced soil fill that will significantly increase earth pressures, reduce soil strength and ultimately reduce the stability of the retaining wall. Earth pressures due to saturated backfill conditions can be calculated using Matsuzawa et al. (1984) and Ishibashi et al. (1985). From a practical viewpoint, it is preferable to install a drainage path (trench) at the back of the reinforced earth zone, rather than behind the face units, to mitigate these pressures.

Recent studies indicate that seismic soil pressure is affected by the low-frequency component of ground motions and soil nonlinearity. There are time and phase changes in shear and compression waves propagating in the backfill behind the retaining wall. These factors control the seismic earth pressures (Ostadan, 2006). The earth pressures could be amplified near the resonant frequency of the backfill. It is necessary to undertake complex time-domain analysis to address these factors. Researchers as well as practitioners recommend this type of analysis for large, important and critical earth structures, especially if PGA is high, as is discussed in the next sections.

For the wall configuration on Fig. 2, Coulomb earth pressure wedge analysis is complicated by the flexible and compressible nature of the facing units. The Coulomb wedge analysis assumes that the failure wedge moves along the back of the wall developing friction forces along the backface and, as such, the authors believe that Coulomb theory for static earth pressures are not suitable for the analysis of wrapped-face walls. Rankin earth pressure theory is recommended for the analysis of the Fig 2 wall configuration as it assumes neither friction nor cohesion (interaction) between the soil and face elements. However, the designer may choose to consider Coulomb theory for his analysis which would give higher earth pressures.

Design parameters

The long-term design strength (LTDS) of a geosynthetic reinforcement is determined per AASHTO, FHWA, and NCMA guidelines where;

[1] $LTDS = T_{ult} / (RF_{cr}. RF_{id}. RF_{d})$

where T_{ult} is the ultimate tensile strength of reinforcement, RF_{cr}, RF_{id}, RF_{d} are reduction factors for creep, installation damage and durability, respectively. The strength reduction factors of geotextiles to account for creep, installation damage and durability are typically higher than those for geogrids and need to be selected with careful consideration. The wide-width tensile strength of geotextiles is the parameter used in retaining structures design. The total elongation of most commonly used geotextiles is large at ultimate strength. Therefore, it may not be appropriate for the designer to use the ultimate strength in stability analysis if governed by low deformation serviceability criterion. The strength commensurate with the serviceability deformation level for the structure (e.g., 2% or 5%) should be used.

Deformation considerations become increasingly important for critical structures. The creep-deformation reduction factor (RF_{cr}) for geotextiles is considerably higher than that of geogrids. This factor typically varies from about 1.5 for polyester to 4.0 for polypropylene products. The installation damage reduction factor (RF_{id}) depends on backfill type and can reach 2.0 for geotextile products. Similarly, the durability reduction factor could reach 3.0. In the absence of product specific testing, the higher reduction factors should be adopted for internal and external analyses per AASHTO specification (2002).

Seismic Loads

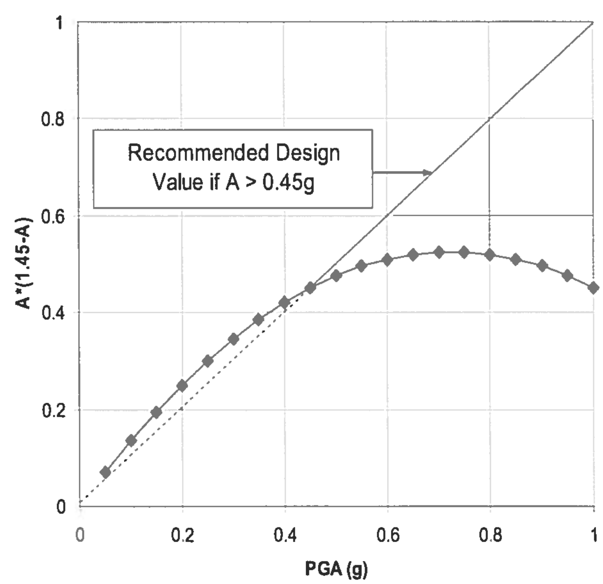
The CFEM and AASHTO guidelines for reinforced earth structures subjected to seismic forces recommend using Equation 3 for calculating the maximum horizontal acceleration for a retaining structure.

[3] $A_m = (1.45 - A) A$

where A is the Peak Ground Acceleration (PGA) for the project site and A_m is the maximum acceleration at the centre of the retaining wall. A representation of Equation 3 is shown on Figure 5. The PGA may be calculated from a site-specific deaggregation analysis or local codes. However, the acceleration at ground surface, not at bedrock or firm ground, should be used in the analysis, if the acceleration is expected to differ as a result of the soil column.

The AASHTO guidelines indicates that a value of one half (0.5) of PGA may be used for design provided that the serviceability criterion allows the retaining structure to deform/displace by a minimum amount equal to 250×PGA (in mm). The design value for internal stability may be chosen equal to PGA in this scenario. However, the AASHTO guidelines recommend setting A_m to A if PGA is higher than 0.45g, as shown on Fig. 5.

Fig. 5. AASHTO recommended maximum acceleration for a retaining wall.



This stipulation applies to high seismicity zones such as the west coast and Vancouver Island regions in BC. Given the cost implications associated with large design PGA values, the authors recommend that a site-specific response analysis be completed to determine the acceleration at ground surface for large and/or critical structures. Sensitivity analysis and case studies on Vancouver Island by the authors indicate that high PGA motions are frequently attenuated during wave propagation through the soil column, if resonance is not present. It is noted that a deformation analysis may be warranted if PGA exceeds 0.3g depending on the acceptable deformation level of adjacent structures (AASHTO, 2002).

The dynamic horizontal thrust, P_{ae}, and the structure's inertia load, P_{ir}, should be calculated in the design of a retaining structure to resist seismic loads. As previously mentioned, earth pressure is usually calculated using Mononobe–Okabe formula, which applies Coulomb's earth pressure computed from the equilibrium of seismic and static forces in a pseudo-static state. Inertia forces are not considered in Mononobe–Okabe theory. The effective inertia force, P_{ir}, is a horizontal load acting at the centre of gravity of the effective mass. The Canadian Foundation Engineering Manual (CFEM) considers the effective mass as half of the total mass of a retaining structure with the dynamic earth pressure acting on a plane at the middle of the reinforced earth zone. Some geotechnical guidelines and codes stipulate that the inertia forces should be applied to all parts of the structure (the sliding mass used to calculate earth pressure, the retaining structure (face elements), as well as to the ground behind the structure). The following equation may be used to calculate P_{ir}:

[4] $$P_{ir} = K_h W = (A_m / g) .\gamma.H.L$$

Where W is the total weight of the effective mass, K_h is the horizontal acceleration ratio (= A_m / g), A_m is the maximum acceleration (as defined in Equation 3) and g is the gravitational acceleration.

Inertia forces can be small and of limited significance for small walls at low values of PGA. However, inertia forces can be significant for large face-blocks, especially in areas of high PGA. It is noted that inertia forces can be critical for gravity walls as a result of their mass.

A modification to the Mononobe–Okabe method was developed by Richard and Elms (1979) by incorporating the inertia force of the retaining wall into the analysis. They provided an equation to calculate the minimum weight of a gravity wall to achieve stability and limit permanent displacements to permissible values. Richard and Elms (1979) also provided an equation to estimate the permanent displacement of retaining walls (d_{perm}) as follows,

[5] $$d_{perm} = 0.087(V^2_{max}\, a^3_{max}) / a^4_y$$

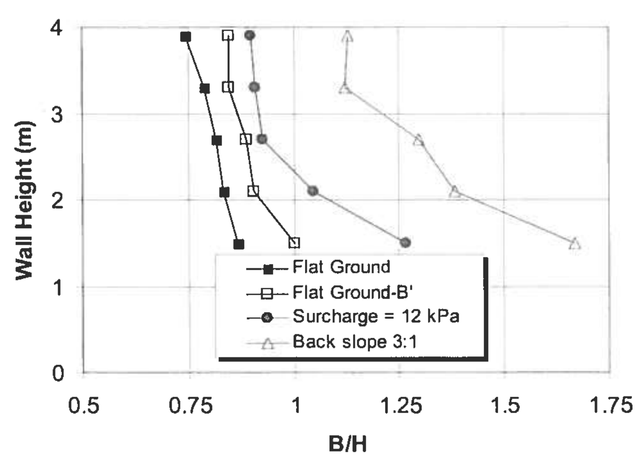
where v_{max} is the peak ground velocity, a_{max} is the peak ground acceleration and a_y is the yield acceleration. Whitman and Liao (1985) introduced a similar equation.

Based on centrifuge model tests, Nakamura (2006) showed that a phase difference exists between the maximum dynamic force and the peak of the inertia loads. For large walls where significant inertia loads exist, it may be necessary to refine loading diagrams using time domain analysis.

Parametric study

The National Concrete Masonary Association (NCMA) software (SRWall 4.0) and guidelines were used by the authors to complete a parametric study on the wall configuration as generally shown in Fig 2. The CFEM reference the NCMA guidelines, analysis and test methods for retaining walls. A parametric study was carried out for three wall configurations (horizontal back slope, 12 kPa surcharge and 3H:1V sloping ground behind the wall). The wall batter was maintained at 1:6, and the wall height varied from 1.5m to 3.9m. An angle of internal friction of 30° was used for the retained soil and backfill. The site PGA was 0.6g and design maximum acceleration (A_m) was chosen equal to 0.3g. Mirafi HP370 and HP570 geotextile reinforcement (see Fig. 1) were used in the model at a vertical spacing of 0.3m. The results of this study are shown on Fig. 6 in terms of the relationship between wall height (H) and the ratio of reinforcement length to wall height (B/H). The analyses showed that the minimum B/H ratio was 0.75 for walls designed to resist the seismic forces. This finding compared favourably to the AASHTO specifications that recommend a minimum B/H of 0.7. The AASHTO specifications recommend extending the reinforcement elements at least 1.0 metre beyond the predicted Rankin slip surface, with a minimum reinforcement length of 2.0 metres. Published data indicates that the B/H ratio can reach 1.0 or more [e.g., Holtz and Lee (2002) and Michalowski (1998)], as seen on Fig. 6, dependent upon variables such as seismic loading, wall configuration and soil properties.

Fig. 6. Variation of B/H with wall height.



The B/H ratio increases with the decrease of wall height because of the decrease in effective stresses. It can be realized that, by increasing wall height, the

B/H ratio converges to a constant value for each loading condition. For the flat ground condition, the B/H ratio near ground surface (B'/H; see Fig. 3) is higher as seen on Fig. 6. In this condition, the internal stability was found to govern the minimum required reinforcement length. The sloping ground condition was found to require the longest reinforcement length (highest B/H ratio) with a minimum B/H ratio of 1.13. For the horizontal ground condition with a surcharge load of 12 kPa, the minimum B/H ratio was 0.9. Internal stability analyses indicated no need to change the type of reinforcement between cases (loading condition) to satisfy tensile forces in reinforcement.

Simplified equations for estimating lateral movements were used to assess the performance of the walls used in the study. Evaluation based on AASHTO guidelines indicated that these walls would experience a lateral displacement of 150mm (250x0.6) under a seismic event having a PGA of 0.6g; such as the 2% in 50 year (A2475) design event for Victoria, BC. These walls would be able to resist the seismic forces from an earthquake of a return period of 10% in 50 years (A475) with minor permanent deformations. The strains in the geotextile elements were estimated to be less than 5%.

Permanent and Temporary Walls

There are applications for both geotextiles and geogrids in MSE wall design. For the green-faced wall applications discussed herein, geotextiles are considered to be a worthy option to the more traditionally used geogrids.

Table 1: Structural System of Proposed Permanent Walls

Wall/Slope Component	Function
Uniaxial (UX) geogrids	Primary reinforcement mechanically reinforces the fill material.
Biaxial (BX) geogrids	Secondary reinforcement that ensures surficial stability of the slope.
Facing wrap around element	Primary reinforcement to wrap around the overlying layer of backfill at the wall-face, and re-embedded into the backfill.
Site-specific green covering system (bioengineered vegetated face)	Provides aesthetic value by offering multiple facing options (socks, lockdown netting or ecoblanket) with the use of a growing medium.

This is particularly the case for grade changes in landscaping areas where strain tolerance is acceptable. In cases where durability (life time), high construction damage and stringent deformation criteria occur, the use of geogrids as primary reinforcement is expected to prevail. Table 1 introduces proposed wall configurations for a permanent wrapped-face green wall where deformation is to be controlled.

Conclusions

A new green-faced wall configuration was introduced in this paper to overcome the potential impacts of volume changes in deformable face-units of a green wall. Performance specifications for geotextiles should be chosen with consideration to stress-strain characteristics, construction issues and durability requirements relative to performance criteria. Inertia forces as well as dynamic earth pressures must be considered in seismic design of retaining structures. While Mononobe–Okabe earth pressures do not include inertia forces, time domain analysis may indicate a phase difference between maximum earth and inertia pressures. Site response analysis may be warranted in the design of large or critical retaining structures in high seismicity zones. Calculating site-specific accelerations and velocities at ground surface could result in more economical design in certain soil columns due to de-amplification of ground motions.

Displacement-based analysis is recommended for design of retaining structures in high seismicity regions, if deformations are critical for the proposed structure. This approach should consider the amplification/de-amplification of earthquake motions in the backfill, dynamic soil properties and nonlinear effects of soil, especially for sensitive and large structures. For many routine non-critical applications, the authors propose using pseudo-dynamic approach for the analysis by incorporating either Bray and Travasarou (2007), Whitman and Liao (1985) or Richard and Elms (1979) method for estimating deformations of retaining walls. More sophisticated numerical modelling would be warranted for critical applications.

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