The influence of reinforcement spacing on the performance of three geosynthetic reinforced soil retaining walls



Axel Nernheim GeoEngineering Centre at Queen's-RMC, Civil Engineering Department – Royal Military College of Canada, Kingston, ON, Canada Richard J. Bathurst GeoEngineering Centre at Queen's-RMC, Civil Engineering Department – Royal Military College of Canada, Kingston, ON, Canada

ABSTRACT

A series of eleven full-scale geosynthetic reinforced soil wall tests has been recently completed at RMC. This paper reports the results of three walls that were constructed with a dry-stacked column of solid masonry block units. The walls were each 3.6 m in height and nominally identical except for the reinforcement spacing. The walls were constructed in the full-scale wall test facility at RMC using a target batter of 8 degrees to the vertical and a high quality sand backfill. Following construction each wall was uniform surcharge loaded in stages to load levels well in excess of working stress conditions. The tests allow the influence of reinforcement spacing on wall displacements, footing loads, reinforcement strains and connection loads between the facing column and the reinforcement layers increase as well. The results are compared to predictions using the current AASHTO Simplified Method (tie-back wedge method). The paper also compares measured loads with predictions using the K-stiffness method. These comparisons demonstrate that the K-stiffness Method gives better agreement with measured loads than the current AASHTO method.

RÉSUMÉ

Une série de onze essais sur murs renforcés de géosynthétiques à pleine-échelle a récemment été complétée au CMR. Cet article rapporte les résultats de trois murs qui ont été construits avec un parement de blocs de maçonnerie pleins empilés sans mortier. Les murs avaient chacun 3.6 m de hauteur et étaient voulus identiques en tout sauf pour l'espacement des renforcements. Les murs ont été construits avec des espacements des renforcements de 0.30, 0.60 et 0.90m, qui donnent 11, 6 et 4 couches de renforcement. Les murs ont été construits dans l'installation d'essais de murs à pleine échelle au CMR en utilisant une inclinaison de parement de 8 degrés et un remblai de sable de haute qualité. Suite à la construction, chaque mur a été sujet à l'application en palier d'une surcharge uniforme allant bien au delà des conditions de charges de service. Les essais permettent d'isoler l'influence de l'espacement des renforcements sur les déplacements des murs, les charges sur les semelles, les déformations du renforcement et les efforts aux connexions entre le renforcement et le parement. Les comparaisons montrent qu'avec l'augmentation de l'espacement, l'effort dans les couches de renforcement augmente aussi. Les résultats sont comparés aux prédictions utilisant la méthode simplifiée AASHTO actuelle (méthode du coin ancré). L'article compare aussi les efforts mesurés aux prédictions basées sur la « méthode de rigidité K ». Ces comparaisons démontrent que la « méthode de rigidité K » est en meilleur accord avec les efforts mesurés, que la méthode AASHTO courante.

1 INTRODUCTION

Current methods for internal stability design of geosynthetic reinforced soil walls are based on limitequilibrium tie back wedge methods of analysis (CFEM 2006, AASHTO 2002, NCMA 1997, BS8006 1995). These methods assume that reinforcement loads are proportional to reinforcement spacing. Linear increase in reinforcement load with reinforcement spacing is also predicted using the K-stiffness Method which is an empirical-based workings stress design method with parameters back-fitted to data from instrumented field walls (Allen at al. 2003, WSDOT 2006, Bathurst et al. 2005, 2008). The expected trend of increasing reinforcement load with increasing spacing has also been demonstrated by the results of parametric analyses of reinforced soil walls using numerical methods (Ho and Rowe 1996, Ling and Leshchinsky 2003). However, a systematic study of the isolated influence of reinforcement spacing on wall performance using fullscale instrumented walls has not been reported in literature to the best of the writers' knowledge.

Eleven full-scale reinforced soil walls 3.6 m high have been recently completed at the Royal Military College of Canada (RMC). Details of some of these walls are reported by Bathurst et al. (2000, 2006) and Nernheim and Bathurst (2007). In this paper the results of three walls in the larger RMC research program are presented. The wall configurations varied only with respect to reinforcement spacing. Two of the walls, Wall 1 (control)



Figure 1. Control wall with six layers of reinforcement (Wall 1)

and Wall 3, were constructed with light compaction equipment and Wall 7 with relatively heavier compaction equipment. Construction-induced wall deformations and horizontal toe loads are adjusted upward for these walls using the method reported by Nernheim and Bathurst (2007) to account for the lighter compaction equipment. Reinforcement loads deduced from strain gauge and extensometer readings are compared to values predicted using the current Simplified Method (AASHTO 2002) and the K-stiffness Method (Bathurst et al. 2008). The predicted magnitude and distribution of reinforcement loads are shown to be more accurate using the Kstiffness Method.

2 EXPERIMENTAL PROGRAM

2.1 Wall configurations

The walls were constructed in the RMC Retaining Wall Test Facility. The facility allows full-scale test walls to be constructed and tested under conditions approaching an idealized plane strain condition.

Figure 1 shows a cross-section of the very stiff-face segmental (modular block) wall that was constructed with six layers of polypropylene (PP) geogrid at a spacing of 0.6 m and a facing batter of 8°. This is the control structure (Wall 1) in the larger RMC testing program that includes the three walls that are the focus of the current paper. The wall was designed to satisfy current National Concrete Masonry Association guidelines (NCMA 1997). An additional design constraint was that the reinforcement layer spacing should not exceed a distance

equal to twice the modular block toe to heel dimension (AASHTO 2002). The wall was constructed from the bottom up with no external support for the facing column. The toe of the wall was restrained horizontally but free to rotate. A course of modular block facing units was placed first, followed by a 150 mm-high lift of compacted soil. The wall facing was built with three discontinuous vertical sections with separate reinforcement layers in plan view (i.e. each reinforcement layer was discontinuous in the cross-plane strain direction). The width of the middle wall section was 1 m which was located between two 1.15 mwide outer sections. The friction between the backfill soil and sides of the test facility was minimised by placing a composite arrangement of plywood, Plexiglas and lubricated polyethylene sheets over the side walls. The discontinuous wall facing and reinforcement layers, and side wall treatment were used to minimise the frictional effects of the lateral boundaries of the test facility and to allow the instrumented middle section of the wall structure to approach a plane strain test condition as far as practical.

Wall 3 was constructed with four layers of the same reinforcement at spacing $S_v = 0.9$ m starting at 0.45 m above the base. Wall 7 was constructed with 11 layers of reinforcement starting at 0.30 m above the base (spacing of 0.3 m). All other parameters were kept the same between the three walls.

More than 300 instruments were monitored during construction and staged uniform surcharging, including displacement-type potentiometers, vertical settlement plates, soil strain inductance coils, load rings and load cells, earth pressure cells, and wire-line extensometers and strain gauges attached directly to the reinforcement layers. Further details of the instrumentation program are reported by Bathurst et al. (2000, 2006).

2.2 Soil

The backfill soil was a uniformly graded, naturally deposited rounded beach sand (SP according to the Unified Soil Classification System) with $D_{50} = 0.34$ mm, coefficient of curvature $C_c = 1.09$ and coefficient of uniformity $C_u = 2.25$. The fines content (particle sizes < 0.075 mm) was less than 1%. Hatami and Bathurst (2005, 2006) reported the results of direct shear tests on this material under confining pressures representative of vertical stress levels in the three walls. They reported the peak direct shear friction angle as $\phi_{ds} = 41^{\circ}$ and constant volume friction angle as $\phi_{cv} = 35^{\circ}$. The (secant) peak plane strain (bi-axial) compression tests carried out under similar load levels and reported as $\phi_{ps} = 44^{\circ}$ (called the peak plane strain friction angle in this paper).

For the control wall the sand was compacted using a walk-behind gasoline-powered vibrating plate compactor in 150 mm lifts to a bulk unit weight of 16.7 kN/m³ at a moisture content of 3 to 5%. A heavier electrical powered vibrating rammer was used for the other two walls. Similar densities and moisture contents were achieved as a result of the flat compaction curve for the sand material. Furthermore, the first 0.5 m distance directly behind the wall facing was hand tamped to the same density using a rigid steel plate. This precautionary measure was taken to minimise construction-induced outward deformation and to reduce compaction-induced lateral stresses against the back of the facing. Nevertheless, detectable differences in end-of-construction wall deformations due to method of compaction were detected. Hence, some measurements for Wall 1 and 3 are corrected for compaction level using the method reported by Nernheim and Bathurst (2007).

2.3 Reinforcement

The geosynthetic reinforcement product used in the construction of the walls was a relatively weak, biaxiallydrawn polypropylene (PP) geogrid that was oriented in the weak direction. Each layer of geogrid had a total length of 2.52 m measured from the front of the facing column. The aperture size for the PP reinforcement was 25 mm between longitudinal members and 33 mm between transverse members. The load-strain-time properties of this geogrid material have been determined from in-isolation constant load (creep) and constant rate of strain tests (Bathurst et al. 2006).

2.4 Segmental (modular) blocks

The modular facing units for the segmental walls were a solid masonry block with continuous concrete shear keys. All blocks were 300 mm long (toe to heel), 150 mm high, 200 mm wide and had a mass of 20 kg. The wall facing units were built with a staggered (running joint) pattern matching the construction technique used in the field.

2.5 Construction and surcharge loading

Following construction, each wall was stage uniformsurcharged using a system of airbags placed over the entire surface of the backfill soil. Each constant surcharge load increment was applied for 24 to 500 hours to monitor time-dependent deformations in the wall. The surcharging was continued until excessive outward deformations of the wall face and large strains in the reinforcement layers were recorded, and (or) the surcharge capacity of the test facility was reached. The combined duration of construction and surcharging ranged from about 3200 to 3500 hours.

3 TEST RESULTS

measured relative Figure 2 shows horizontal displacements at monitored points on the wall facing column shortly after the end of construction. Actual measurements for Wall 1 and 3 deformations were multiplied by a factor of 1.78 and 1.65, respectively, to account for the lighter compaction effort used in these walls compared to Wall 7 (see Nernheim and Bathurst 2007). The deformations at each elevation have a local datum corresponding to the time each row of displacement potentiometers was installed. Hence the data are moving datum profiles rather than total out-ofalignment wall profiles. The horizontal bars show maximum displacement ranges over the (instrumented) centre one metre of running length of wall. Hence, the relative displacements between Walls 1 and 3 are within measurement variability. Some variability can be expected since the discrete blocks do not move uniformly outward at each elevation.

Post-construction wall deformation profiles at two different surcharge levels for Walls 1, 3 and 7 are plotted in Figures 3a and 3b. The figures show that post-construction wall deformations are small (< 12 mm) and similar in magnitude for all three walls at 30 kPa surcharge. At larger surcharge levels, differences in wall profiles become more visible.

The facing column of each wall was seated on a steel footing supported on rollers and load cells to record decoupled horizontal and vertical toe loads (Figure 1). Figure 4 shows plots of vertical toe load at end of construction and during subsequent surcharge loading. The vertical toe loads for Walls 1, 3 and 7 vary by less than 15% at end of construction. This variation is likely due to small unquantifiable effects of initial seating and alignment of the modular block facing units and placement and attachment of the reinforcement to the facing column. Nevertheless, the total toe force is greater than the self-weight of the facing column. Down-drag forces are developed at the connections between the wall facing and the reinforcement layers as the soil behind the facing column settles with respect to the hard facing. This settlement occurs as the soil is compacted, compresses under increasing vertical stress and as outward wall deformations accumulate.

During subsequent surcharging, the rate of increase in toe load with surcharge level is least for Wall 7 with the greatest number of reinforcement layers. This is



Figure 2. Relative horizontal displacement of wall facing recorded at end of construction. Note: Range bars show maximum variation in displacements recorded across centre metre of wall face

consistent with this wall developing the least outward deformation of the three walls during surcharging.

Horizontal toe load histories with surcharge pressure are presented in Figure 5. The end-of construction horizontal toe loads have been increased by a factor of 1.41 and 1.48 for Wall 1 and 3, respectively. This correction gives toe load values for Walls 1 and 3 had the backfill in these walls been compacted in the same manner as Wall 7 (Nernheim and Bathurst 2007). The change in horizontal toe load due to surcharging alone is least for Wall 7 that was constructed with the largest number of reinforcement layers. Another observation from the data for Wall 7 is that the plot is smoother than for the less heavily reinforced soil walls. This may be attributable to the greater load capacity redundancy in the system that occurs with more closely spaced reinforcement layers.

Strain gauges bonded directly to the surface of the reinforcement and extensometer points were used to record strains in the reinforcement layers.

Figure 6a presents strain profiles at the end of construction for reinforcement layers at similar elevations in the three walls. The strain values are very low at end of construction but (as expected) generally increase with decreasing number of layers. The data also show a trend of increasing strain values closer to the connections. This observation is consistent with relative settlement of the soil at the reinforcement connections described earlier. The locally larger strains toward the free end of the reinforcement layer in Wall 7 are likely the effect of unavoidable small variations in laying out the reinforcement, fill placement and compaction.

Figure 6b shows strain profiles at surcharge load level of 70 kPa. In general, strains are much higher and increase with decreasing number of reinforcement layers. However, the largest strains in each plot are located well within the reinforced soil zone and become more pronounced with decreasing number of reinforcement layers. These local high strains are consistent with the



Figure 3. Post-construction facing profiles



Figure 4. Vertical toe load versus surcharge pressure



Figure 5. Horizontal toe load versus surcharge pressure

onset of internal soil failure mechanisms. Bathurst et al. (2000) showed that the points connecting peak internal strains on the individual reinforcement layers in Wall 1 traced a log-spiral failure surface, or (almost as accurate) a Coulomb failure surface propagating from the heel of the facing column consistent with tie back wedge methods.

4 MEASURED VERSUS PREDICTED LOADS

The results of the physical tests described here can be used to compare measured to predicted reinforcement loads using current and recently proposed design methods. However, reinforcement loads were not measured directly in these experiments except at the connections. For reinforcement loads at interior locations, the "measured" maximum reinforcement load, T_{max} , must be computed using a suitably selected isochronous stiffness value, $J(\epsilon, t)$, which is a function of strain (ϵ) and duration of loading (t). Hence, $T_{max} = J \times \epsilon$. The method



a) end of construction



b) at 70 kPa surcharge level



adopted here to compute loads from strains is described in detail by Walters et al. (2002) and Bathurst et al. (2006). The duration of loading was taken as the elapsed time since the layer was placed in the wall.

Measured loads for each of the three walls at the end of construction are plotted in Figure 7. Range bars representing ± 1 standard deviation on the measured values are plotted at each measurement point based on estimated variability in component parameters used to make the T_{max} computation (i.e. variability in strain measurement and stiffness values). In all cases, the loads are small. The reinforcement loads for Wall 3 with the fewest reinforcement layers are detectably greater than for the two walls with 6 and 11 layers of reinforcement.

Plotted in the figures are the predicted values using the current AASHTO (2002) Simplified Method, also know as the "tie back wedge method". The reinforcement loads using this method are computed using the tributary area approach expressed as:

$$T_{max} = S_v K_a (\phi, \omega, \delta) \gamma z$$
(1)

where, S_v is the layer spacing, K_a is the active earth pressure coefficient computed as a function of the soil friction angle (ϕ), interface friction angle ($\delta = 0$) and the facing angle (ω). Parameter γ is the bulk unit weight of the soil and z is the depth of the reinforcement layer from the crest of the wall. In order to make fair comparisons between different design methods, the plane strain friction angle ($\phi = 44^{\circ}$) was used in all calculations. When the Simplified Method is applied to the three walls in this investigation, the reinforcement loads plot as linear distributions with depth below the crest of the wall. Nevertheless, the predicted values are typically well above measured values. The discrepancy increases with depth. If lower friction angle values are used (i.e. constant volume values, or peak values from direct shear or triaxial compression tests that are not corrected to plane strain values), the discrepancy would be even greater.

The maximum reinforcement load using the Kstiffness Method is:

$$T_{max} = 0.5 \text{ K} \gamma (\text{H+S}) S_v D_{tmax} \Phi_g \Phi_{local} \Phi_{fs} \Phi_{fb} \Phi_c$$
(2)

Here: S = equivalent height of uniform surcharge pressure q (i.e. S = q/ γ); D_{tmax} = load distribution factor that modifies the reinforcement load based on layer location. The remaining terms, Φ_g , Φ_{local} , Φ_{fs} , Φ_{fb} and Φ_c are influence factors that account for the effects of global and local reinforcement stiffness, facing stiffness, face batter and soil cohesion, respectively. The coefficient of lateral earth pressure is calculated as K = 1 - sin ϕ with $\phi = \phi_{\text{ps}}$ = secant peak plane strain friction angle of the soil. Details to calculate these values for the RMC walls can be found in the paper by Bathurst et al. (2008). To remove the choice of friction angle as a variable between calculation methods, the same value of peak friction angle is used in both sets of calculations (i.e. $\phi = \phi_{\text{ps}}$ = secant peak plane strain friction angle of the soil).

Predicted loads using the K-stiffness Method generally capture the more uniform load distribution with depth and are closer in magnitude to measured values. It should be recalled that the K-stiffness Method is an empirical-based working stress method with coefficient terms determined by back-fitting to measurements recorded in a database of instrumented field walls. The calibration was restricted to walls with surcharge levels less than S = 1 m. Nevertheless, the method does very well at an equivalent surcharge height of about S = 3 m for the RMC walls.

5 CONCLUSIONS AND DISCUSSION

The paper reports the results of three full-scale geosynthetic reinforced soil walls with geometry varying only with respect to reinforcement spacing. The wall facings were constructed with the same dry-stacked column of modular blocks and a compacted sand backfill. The walls were seated on a rigid foundation together with a stiff horizontal toe support to simplify construction and interpretation of test results.

Data in a companion paper showed that the effect of compaction method was as important as global reinforcement stiffness on initial (end-of-construction) outward wall deformations and the development of horizontal earth load at the toe (Nernheim and Bathurst 2007). In the current paper, adjustments to these measured values based on compaction method were made in order to compare the three walls. A practical implication to similar hard-faced walls in the field is that compaction method can mask the relative trend in wall deformations due to reinforcement stiffness and spacing that would be expected for otherwise nominally identical structures. However, as the walls were surcharged, the influence of compaction effort was attenuated due to greater vertical stresses that likely overcame locked-in soil stresses.

Post-construction deformations increased with larger spacing (or decreasing global reinforcement stiffness) as expected. However, there was also a trend towards increasing fraction of total earth force carried by the reinforcement layers as the number of layers (or global stiffness) decreased.

Comparison of measured maximum reinforcement loads with predicted values using the AASHTO Simplified Method and the K-stiffness Method show that the latter was more accurate and able to capture the uniform reinforcement load distributions observed in the RMC walls.

The data from the three walls described in this investigation provide valuable qualitative insight into the contribution of reinforcement spacing to the performance of reinforced soil walls. Nevertheless, it is important to note that the toe of the walls in this study were likely stiffer than many comparable walls in the field. As toe restraint is relaxed and (or) walls with a structural facing become taller, the relative contribution of the toe to wall capacity can be expected to reduce (Huang et al. 2007, 2008).

Finally, the type and structural stiffness of the facing column will quantitatively affect the performance of geosynthetic reinforced soil walls as demonstrated by the K-stiffness Method (e.g. Bathurst et al. 2005, 2008) and other RMC test walls (Bathurst et al. 2006, 2007). The large number of parameters and the mechanical complexity of modern geosynthetic reinforced soil walls have led the writers and co-workers to abandon the use of classical deterministic models to predict loads in walls (i.e. limit equilibrium-based approaches such as the Simplified Method and variants). The AASHTO development of the empirical-based K-stiffness Method that is based on calibration against results from monitored wall structures offers promise as a new approach to improve load prediction accuracy for geosynthetic reinforced soil walls under operational conditions. The improvement in load predictions using the K-stiffness Method compared to the AASHTO Simplified Method is demonstrated for the carefully instrumented laboratory walls in the current study.



a) end of construction

b) surcharge q = 50 kPa

Figure 7. Predicted versus measured maximum reinforcement load in instrumented layers. Note: Range bars represent ±1 standard deviation on measured reinforcement load

ACKNOWLEDGEMENTS

The first author is grateful for a fellowship awarded by the Postdoc-Programme of the German Academic Exchange Service (DAAD). Financial support for this study was also provided by the Natural Sciences and Engineering Research Council (NSERC) of Canada, grants from the Department of National Defence (Canada) and the following US State Departments of Transportation: Alaska, Arizona, California, Colorado, Idaho, Minnesota, New York, North Dakota, Oregon, Utah, Washington and Wyoming.

REFERENCES

- Allen, T.M., Bathurst, R.J., Holtz, R.D., Walters, D.L. and Lee, W.F. 2003. A new working stress method for prediction of reinforcement loads in geosynthetic walls, *Canadian Geotechnical Journal*, Vol. 40, No. 5, pp. 976-994.
- AASHTO. 2002. *Standard Specifications for Highway Bridges.* American Association of State Highway and Transportation Officials (AASHTO), 17th ed., Washington, D.C.
- Bathurst, R.J, Allen, T.M. and Walters, D.L. 2005. Reinforcement loads in geosynthetic walls and the case for a new working stress design method, *Geotextiles and Geomembranes*, Vol. 23, No. 4, pp. 287-322.
- Bathurst, R.J., Miyata, Y., Nernheim, A. and Allen, T.M. 2008. Refinement of K-stiffness method for geosynthetic reinforced soil walls, *Geosynthetics International*, Vol. 15, No. 4, (in press).
- Bathurst, R.J., Vlachopoulos, N., Walters, D.L., Burgess, P.G. and Allen, T.M. 2007. Reply to the discussions on "The influence of facing rigidity on the performance of two geosynthetic reinforced soil retaining walls", *Canadian Geotechnical Journal*, Vol. 44, No. 12, pp. 1484-1490.
- Bathurst, R.J., Vlachopoulos, N., Walters, D.L., Burgess, P.G. and Allen, T.M. 2006. The influence of facing rigidity on the performance of two geosynthetic reinforced soil retaining walls, *Canadian Geotechnical Journal*, Vol. 43, No. 12, pp. 1225-1237.
- Bathurst, R.J., Walters, D., Vlachopoulos, N., Burgess, P. and Allen, T.M. 2000. Full scale testing of geosynthetic reinforced walls, ASCE Special Publication No. 103, Advances in Transportation and Geoenvironmental Systems using Geosynthetics, Proceedings of Geo-Denver 2000, 5-8 August 2000, Denver, Colorado, pp. 201-217.
- BS8006. 1995. Code of practice for strengthened/reinforced soil and other fills, BRITISH STANDARDS INSTITUTION, Milton Keynes, United Kingdom.

- Chapter 27. Reinforced Soil Walls. Canadian Foundation Engineering Manual (CFEM) 2006. *Canadian Geotechnical Society*, 4th edition, BiTech Publishers, Richmond, BC, Canada.
- Hatami, K. and Bathurst, R.J. 2006. A numerical model for reinforced soil segmental walls under surcharge loading, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 132, No. 6, pp. 673-684.
- Hatami, K. and Bathurst, R.J. 2005. Development and verification of a numerical model for the analysis of geosynthetic reinforced soil segmental walls under working stress conditions, *Canadian Geotechnical Journal*, Vol. 42, No. 4, pp. 1066-1085.
- Ho, S.K. and Rowe, R.K. 1996. Effect of wall geometry on the behaviour of reinforced soil walls, *Geotextiles and Geomembranes*, Vol. 14, No. 10, pp. 521-541.
- Huang, B., Bathurst, R.J. and Hatami, K. 2007. Numerical study of the influence of block interface stiffness on reinforced soil segmental walls of variable height, In CD proceedings of the *60th Canadian Geotechnical Conference*, Ottawa, Ontario, October 2007, 8 p.
- Huang, B., Bathurst, R.J. and Hatami, K. 2008. Numerical study of the influence of reinforcement length and spacing on reinforced soil segmental walls of variable height, In CD proceedings of the *First Pan American Geosynthetics Conference & Exhibition*, 2-5 March 2008, Cancun, Mexico (eds Bathurst & Palmeira), IFAI, pp. 1256-1264.
- Ling, H.I. and Leshchinsky, D. 2003. Finite element parametric study of the behavior of segmental block reinforced-soil retaining walls, *Geosynthetics International*, Vol. 10, No. 3, pp. 77–94.
- NCMA. 1997. Design manual for segmental retaining walls (2nd edition), *National Concrete Masonry Association*, (ed. J. Collin), Herndon, VA, USA.
- Nernheim, A. and Bathurst, R.J. 2007. Influence of facing batter on the performance of three geosynthetic reinforced soil walls, In CD proceedings of the *60th Canadian Geotechnical Conference*, Ottawa, Ontario, October 2007, 9 p.
- Walters, D.L., Allen, T.M. and Bathurst, R.J. 2002. Conversion of geosynthetic strain to load using reinforcement stiffness, *Geosynthetics International*, Vol. 9, Nos. 5-6, pp. 483-523.
- WSDOT 2006. *Geotechnical Design Manual M46-03*, Washington State Department of Transportation, Tumwater, WA, USA.