PARAMETRIC ANALYSIS OF REINFORCED SOIL WALLS WITH DIFFERENT BACKFILL MATERIAL PROPERTIES

Kianoosh Hatami
School of Civil Engineering and Environmental Science
University of Oklahoma, Norman, OK, USA

Richard J. Bathurst
GeoEngineering Centre at Queen’s-RMC
Royal Military College of Canada, Kingston, Ontario, Canada

ABSTRACT

The influence of backfill type and material properties on the performance of reinforced soil segmental retaining walls under working stress conditions (end of construction) is investigated using a numerical model. The numerical model has been validated against the measured data from several 3.6 m-high test walls in a previous investigation and is now used in this study to investigate the response of idealized 6 m-high wall models at the end of construction. Four different backfill types representing a select granular fill, a backfill with low friction angle, and backfill types with significant fines content (i.e. a cohesive strength component) were used in the numerical simulations. The numerical results demonstrate that for the case of a granular backfill, a small amount of soil cohesion can significantly reduce wall lateral displacements provided that the relative displacement between reinforcement and backfill (i.e. backfill-reinforcement interface compliance) is negligibly small.

BACKGROUND

The qualitative effect of backfill strength properties on the performance of reinforced soil walls has been addressed in a number of numerical simulation studies in the recent years. Rowe and Ho (1997) found that the total force required for internal equilibrium of a reinforced soil wall is essentially independent of backfill material properties other than its internal friction angle value. Rowe and Ho (1998) using an idealized 6 m-high propped-panel wall showed that the magnitude of wall lateral displacement can be greatly influenced by the magnitude of the backfill friction angle. Helwany et al. (1999) concluded that backfill type is the most important parameter influencing the performance of reinforced soil wall systems. Ling and Leshchinsky (2003) confirmed that the lateral displacements and reinforcement strains are greater in walls built with weaker granular backfills (i.e. with smaller friction angle value).

A total of 11 full-scale instrumented reinforced soil walls were built over the past few years at the Royal Military College of Canada (RMC). The walls were 3.6 m in height and constructed with a column of solid modular concrete units, wrapped-face or incremental facing panels (e.g. Bathurst et al. 2002). The same clean uniform sand backfill was used in all test cases. The wall structures were built over a rigid foundation. The soil reinforcement was comprised of different arrangements of a weak biaxial polypropylene geogrid, woven polyester geogrid or
welded wire mesh materials. The matrix of test results has allowed a database of high quality experimental results to be collected that can be used to isolate the contribution of wall facing type, reinforcement type and spacing, to wall performance under working stress conditions (i.e. end of construction) and at conditions approaching collapse under uniform surcharge loading. Each of the structures was heavily instrumented to record horizontal and vertical toe boundary reactions, vertical earth pressures at the foundation, facing horizontal displacements, connection loads, and reinforcement strains.

The measured data from the full-scale reinforced soil retaining walls was used to verify a numerical model (Hatami and Bathurst 2005a,b) using the program FLAC (Itasca 2005). A novel feature of the model is that it simulates backfill compaction and the moving datum of the segmental facing units and backfill layers during construction. These are two very important modeling features that have been demonstrated by the writers to have an important effect on the predicted construction response of the RMC walls.

In this paper, the same numerical model is used to investigate the influence of backfill strength properties on the behavior of a series of idealized wall structures similar to the 3.6-m high RMC walls with a modular block facing, but extended to a height of 6 m. Different material models representing an idealized dense, high-strength granular soil, a relatively less dense granular soil, and two granular soil types with a large fines content (e.g. silty sand) are included in the parametric analysis. To simplify the interpretation of numerical results, only the strength properties of the materials were varied. The analyses were carried out assuming that the backfill was in a drained condition. The influence of soil-reinforcement interface stiffness on computed wall lateral displacements and reinforcement loads is also investigated.

NUMERICAL MODEL

Backfill model and material properties

Figure 1 shows the (FLAC) numerical model of the 6 m-high segmental walls used in this study. The facing column of the model walls is comprised of 40 rows of solid masonry concrete blocks. The concrete modular blocks were modeled as linear elastic continuum zones separated by nulled zones of zero thickness that contained interfaces. The compacted backfill soil was modeled as a homogenous, isotropic, nonlinear elastic-plastic material with Mohr-Coulomb failure criterion and dilation angle (non-associated flow rule). The cohesion and plane-strain friction angle values for the backfill were varied as shown in Table 1. The backfill dilation angle and bulk unit weight were $\psi = 11^\circ$ and $\gamma = 16.8 \text{kN/m}^3$, respectively.

The backfill nonlinear elastic response was first determined using the stress-dependent hyperbolic model proposed by Duncan et al. (1980) and using the soil bulk modulus from triaxial testing. The tangent elastic modulus was adjusted to fit the results of plane strain laboratory tests carried out on the same reference RMC sand material (Hatami and Bathurst 2005a,b). The value for soil bulk modulus was adjusted during the computations to ensure that the soil Poisson’s ratio value for each soil zone remained in the range $0 \leq \nu \leq 0.49$ throughout the simulations. The backfill stiffness properties given in Table 2 were assumed the same for all four backfill types.
Figure 1 – Numerical model of 6 m-high segmental (modular block) wall examined to isolate the influence of backfill strength properties and plastic behavior on reinforced soil wall response.

The backfill compaction during construction of the wall models was simulated by applying a uniform vertical stress equal to \( q = 8 \) kPa to the entire surface of each new soil layer before solving the model to equilibrium. This vertical stress increment was applied only to the top of the soil backfill zone at each lift as the wall was constructed from the bottom-up and was removed before the placement of the next soil layer. The value \( q = 8 \) kPa was used to simulate compaction effects due to the vibratory plate compactor that was used in the full-scale RMC walls (Hatami and Bathurst 2005a,b).

A fixed boundary condition in the horizontal direction was assumed at the numerical grid points on the backfill far-end boundary allowing for free settlement of soil along that boundary. A fixed boundary condition in both horizontal and vertical directions was used at the bottom boundary matching the foundation condition in the physical tests at RMC.

**Reinforcement model and material properties**

The reinforcement type in this study was a biaxial polypropylene (PP) geogrid with the material properties given in Table 3. The stiffness properties correspond to a hyperbolic model for the reinforcement defined as (Hatami and Bathurst 2005b):
Table 1 – Backfill Types and Strength Properties Examined in the Study

<table>
<thead>
<tr>
<th>Analysis Case</th>
<th>Backfill Type</th>
<th>Friction Angle, $\phi_{ps}$ ($^\circ$)</th>
<th>Cohesion, $c$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dense Granular Soil</td>
<td>40</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>Loose Granular Soil</td>
<td>30</td>
<td>1</td>
</tr>
<tr>
<td>3 &amp; 4a, 4b(1)</td>
<td>Granular Soils with large fines content</td>
<td>30</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>20</td>
<td>10</td>
</tr>
</tbody>
</table>

(1) See Table 4 for description of soil-reinforcement interface models used for each analysis case and parameter values.

Table 2 – Backfill Stiffness Properties (all models)

<table>
<thead>
<tr>
<th>Stiffness Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_e$ (elastic modulus number)</td>
<td>1150</td>
</tr>
<tr>
<td>$K_b$ (bulk modulus number)</td>
<td>575</td>
</tr>
<tr>
<td>$K_{ur}$ (unloading-reloading modulus number)</td>
<td>1380</td>
</tr>
<tr>
<td>$n$ (elastic modulus exponent)</td>
<td>0.5</td>
</tr>
<tr>
<td>$m$ (bulk modulus exponent)</td>
<td>0.5</td>
</tr>
<tr>
<td>$R_f$ (failure ratio)</td>
<td>0.86</td>
</tr>
<tr>
<td>$\nu_t$ (tangent Poisson's ratio)</td>
<td>0 - 0.49</td>
</tr>
</tbody>
</table>

\[
J_t (\varepsilon, t) = \frac{1}{J_o (t) \left[ \frac{1}{J_o (t)} + \frac{\eta(t)}{T_f (t)} \varepsilon \right]^2}
\]  

where: $J_o (t)$ is the initial tangent stiffness, $\eta(t)$ is a scaling function, $T_f (t)$ is the stress-rupture function for the reinforcement, and $t$ is time. The stiffness parameter values in Table 3 represent a reinforcement product that is three times as stiff as the weak PP reinforcement used in the 3.6 m-high RMC test walls. This stiffness value is within the range of typical PP products used in the field (GFR 2005).
Table 3 – Reinforcement Material Properties (all models)

<table>
<thead>
<tr>
<th>Reinforcement type</th>
<th>$J_0(t)$, $\eta(t)$, $T_f(t)$ (kN/m) (Equation 1 and $t = 1000$ hours)</th>
<th>Ultimate (index) strength $T_y^{(1)}$ (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PP geogrid</td>
<td>339, 0.85, 23.1</td>
<td>42</td>
</tr>
</tbody>
</table>

*(1) Three times the peak strength measured during 10% strain/min constant-rate-of-strain (CRS) test on PP geogrid used in RMC test walls (ASTM D4595).*

Table 4 – Interface Properties

<table>
<thead>
<tr>
<th>Interface</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil-Block</td>
<td></td>
</tr>
<tr>
<td>$\delta_{sb}$ (friction angle) (degrees)</td>
<td>44</td>
</tr>
<tr>
<td>$\psi_{sb}$ (dilation angle) (degrees)</td>
<td>11</td>
</tr>
<tr>
<td>$k_{nsb}$ (normal stiffness) (MN/m/m)</td>
<td>100</td>
</tr>
<tr>
<td>$k_{ssb}$ (shear stiffness) (MN/m/m)</td>
<td>1</td>
</tr>
<tr>
<td>Block-Block</td>
<td></td>
</tr>
<tr>
<td>$\delta_{bb}$ (friction angle) (degrees)</td>
<td>57</td>
</tr>
<tr>
<td>$c_{bb}$ (cohesion) (kPa)</td>
<td>46</td>
</tr>
<tr>
<td>$k_{nbb}$ (normal stiffness) (MN/m/m)</td>
<td>1000</td>
</tr>
<tr>
<td>$k_{sbb}$ (shear stiffness) (MN/m/m)</td>
<td>50</td>
</tr>
<tr>
<td>Backfill-Reinforcement</td>
<td></td>
</tr>
<tr>
<td>$k_b$ (stiffness) (kN/m/m) $^{(1)}$</td>
<td>10, $10^2$, $10^3$, $10^4$</td>
</tr>
<tr>
<td>$s_b$ (adhesion strength) (kPa)</td>
<td>$10^3$</td>
</tr>
<tr>
<td>$\phi_b$ (friction angle) (degrees) $^{(2)}$</td>
<td>20, 30, 40</td>
</tr>
</tbody>
</table>

*(1) Analysis Cases 1, 2 and 3 carried out using with the FLAC “grout” model with equivalent reinforcement-soil interface stiffness value of $k_b = 10^3$ kN/m/m (control case for influence of reinforcement-soil interface stiffness). Analysis Case (4b) was carried out with $k_b = 10$, $10^2$, $10^3$ and $10^4$, and $\phi_{ps} = 20^\circ$ and $c = 10$ kPa (Table 1). Analysis Case 4a was carried out with “pinned” (i.e. fully-fixed) reinforcement-soil interface.

*(2) $\phi_b = \phi_{ps}$ in all simulations.*
Hatami and Bathurst (2005b) reported that it took approximately 1000 hours to build each of the RMC full-scale test wall models. The in-situ loading rate of the reinforcement during construction of the full-scale test walls was determined to be in the order of $10^{-5}$ % strain/min. This loading rate can be regarded as representative of construction times in the field (Walters et al. 2002) and is substantially lower than the 10% strain/min loading rate applied during the standard ASTM D4595 test protocol. Therefore, the reinforcement load-strain-time material properties for the numerical model in Table 3 were obtained from 1000-hour isochronous data using creep tests carried out on multiple-strand specimens of the PP geogrid.

The reinforcement layers were modeled in the simulations using two-noded elastic-plastic cable elements with strain-dependent tangent tensile stiffness, $J(\varepsilon)$ (Equation 1), tensile yield strength, $T_y = T_r(t)$ and no compressive strength. The stiffness of each cable reinforcement element in the model was continuously updated based on the calculated axial strain during construction.

**Modular blocks**

The modular concrete blocks were modeled as a homogenous isotropic linear elastic medium with $E = 23$ GPa and $\nu = 0.15$. However, it should be noted that the facing performance was influenced by the interface model and parameter values assigned to the interface between modular block layers rather than the properties assigned to the block zones (see next section).

**Interfaces**

The interfaces between dissimilar materials were modeled as linear spring-slider systems with interface shear strength defined by the Mohr-Coulomb failure criterion. Details of the interface and boundary models employed in numerical simulations and the rationale for the selection of parameter values (Table 4) are reported by Hatami and Bathurst (2005a,b).

The interaction and relative movement of the reinforcement layer in the backfill was modeled using two different approaches. In the first approach, the possibility of reinforcement “pullout” (i.e. relative displacement of the reinforcement with respect to the backfill) was included in the wall model (Cases 1, 2, 3 and 4b). In these cases, the backfill-reinforcement interaction was modeled using the FLAC cable “grout” utility (Itasca 2005). The relative movement of the reinforcement with respect to the backfill was governed by the bond stiffness, $k_b$ and the Mohr-Coulomb type failure criterion characterized by the bond adhesion strength, $s_b$ and the friction angle, $\phi_b$. Analysis Case 4b was carried out with different values of bond stiffness as $k_b = 10, 10^2, 10^3$ and $10^4$ kN/m/m and $\phi_b = 20^\circ$ and $c = 10$ kPa. In the second approach, the structural nodes of the reinforcement cable elements were rigidly attached to the grid points of the backfill numerical mesh (Case 4a). The FLAC “grout” interface was not activated and pullout of the reinforcement from the backfill was prevented.
RESULTS

Facing lateral displacement

Figure 2a shows the predicted facing lateral displacement of model walls with different backfill strength properties. As may be expected, the plots show that facing deflections diminish in magnitude as soil strength increases due to an increase in friction angle or increase in soil cohesion or both. The pattern of deflected shape is also influenced by the addition of soil cohesion. An increase in soil cohesion moves the location of maximum wall deflection lower down the wall and is particularly effective in reducing deflections at the wall crest. This observation is consistent with classical earth pressure theory which predicts a zone of tensile (or zero) lateral stress at the top of a wall that retains soil with a cohesive strength component.

A similar qualitative effect has been reported by Lo (2001) who showed using numerical modeling, that lateral facing deformations at the top of a 8 m-high full-height panel reinforced soil wall increased by a factor of five when the backfill cohesion value was increased from 10 to 30 kPa. Esfahani and Bathurst (2002) carried out a series of 1.2 m-high reduced-scale model reinforced soil wall tests in which the fines content of the backfill was systematically increased. The magnitude of facing deflections under uniform surcharging was shown to decrease by more than one half when the fines content was increased from 0% to 50%.

Figure 2b shows the influence of the magnitude of backfill-reinforcement interface stiffness, $k_b$, on the predicted wall deformation. The deformation response of the model wall with a pinned reinforcement condition (i.e. when no relative displacement of reinforcement with respect to the backfill is allowed in the model) is very close to that of the model with interface stiffness values $k_b \geq 1000$ kN/m/m. Larger interface stiffness values will lead to slightly smaller deformation predictions for the wall but at the expense of significantly greater computation time. For values of $k_b \leq 1000$ kN/m/m, lower backfill-reinforcement stiffness values result in greater wall deformation for otherwise identical wall models. For example, Figure 2b illustrates that the wall deformation magnitude increased by a factor of two when the value of $k_b$ was reduced by two orders of magnitude from $k_b = 10^3$ kN/m/m to $k_b = 10$ kN/m/m. In current limit-equilibrium analysis and design of reinforced soil walls, the backfill-reinforcement interlocking efficiency is expressed only in terms of an interface coefficient, $C_d$, which is a function of interface friction angle and cohesion according to a Mohr-Coulomb type model (e.g. Collin (NCMA) 1997). Results shown in Figure 2b clearly indicate that the magnitude of soil-reinforcement interface stiffness plays a very important role in wall deflection response. Nevertheless, quantifying this parameter from the results of independent interface shear or pullout laboratory tests is problematic.

Reinforcement loads

Figure 3 shows the magnitude and distribution of maximum reinforcement load in each layer for the model walls examined. The results are presented for maximum connection loads and maximum reinforcement loads at locations along the length of the reinforcement layers within the reinforced soil zone. The simulation results show that connection loads are often higher than the loads developed in the reinforcement within the reinforced soil zone. A similar
Figure 2 – Facing displacement response of wall models with different backfill material properties and reinforcement-soil interface stiffness values at end of construction.

Not unexpectedly, reinforcement loads are greater for the walls with weaker backfills. It is also observed that the distribution of maximum load along the wall height varies between a parabolic shape, for the cases with a relatively cohesionless granular backfill, and a more linear shape when the backfill is more cohesive. A parabolic shape has been noted from the results of instrumented walls in the field (Allen et al. 2003) and in the full-scale reinforced soil wall tests carried out at RMC (Bathurst et al. 2002) constructed with low fines-content granular soils. The
observation that more cohesive granular backfills result in reinforcement load distributions that increase linearly with depth is qualitatively consistent with current design methods based on the Simplified Method (AASHTO 2002, Elias et al. (FHWA) 2001). The parabolic shapes observed for the relatively cohesionless backfill soil cases in this investigation are consistent with the general distribution of reinforcement loads proposed in the recent empirically-based K-stiffness Method (Allen et al. 2003, Bathurst et al. 2005). Importantly, the results of this numerical investigation suggest that the reinforcement load distribution function used in the K-stiffness Method may have to be modified for the case of non-select fills with a cohesive strength component.

Figure 4 shows a comparison of predicted maximum reinforcement loads for model walls with different backfill-reinforcement interface stiffness values. It can be observed that as the reinforcement-soil interface stiffness value increases, the connection loads and interior
reinforcement loads generally increase. For the case of maximum reinforcement loads within the soil mass (Figure 4b), there is no practical difference between computed results using \( k_b = 10^4 \) kN/m/m and the pinned reinforcement model. For the connection load results (Figure 4a), there is a trend towards the largest connection loads moving higher up the wall while the load distribution pattern for the maximum reinforcement loads is essentially independent of interface stiffness values. The explanation is that as the anchorage stiffness of the top layers improves, the reinforcement close to the back of the facing column mobilizes additional load due to relative settlement of the soil behind the facing column. The magnitude of settlement behind the facing as it rotates outward increases with elevation above the toe. Clearly, mechanisms that generate load in the reinforcement close to the connections with the wall face are different from those that mobilize load in the reinforcement at greater distances. This distinction is not accounted for in the current Simplified Method and its variants to estimate reinforcement loads (e.g. AASHTO 2002, Elias et al. (2001), Collin (NCMA) 1997).

CONCLUSIONS

The influence of backfill strength properties and reinforcement-soil interface stiffness on the performance of reinforced soil segmental retaining walls under working stress conditions is investigated using a numerical model. The numerical model has been validated against measured data from several full-scale 3.6 m-high modular block reinforced soil walls at the end of construction (Hatami and Bathurst 2005a).

The numerical results demonstrate that for the case of a granular backfill, a cohesive strength component as low as 10 kPa can significantly reduce wall lateral displacements provided that the relative displacement between reinforcement and backfill is negligible (i.e. provided that the stiffness of the reinforcement layer is not reduced by a low-stiffness backfill-reinforcement interface). Furthermore, the addition of cohesion to the backfill soil had a significant influence on the shape of the deformed wall face and the distribution of reinforcement loads at the connections and within the reinforced soil zone.

The data reported in this paper suggests that assumptions regarding the distribution of reinforcement loads in the recently proposed K-stiffness Method may need to be refined for cases in which a non-select fill with a cohesive strength component is used.

Finally, it is important to note that while the numerical results reported here show that reductions in reinforcement loads are possible with non-select cohesive granular soils, there are important practical caveats to their use. For example, soils with a high fines content may be more difficult to place and compact in the field, may exhibit time-dependent deformations, may have mechanical properties that are sensitive to moisture content, and require special attention with respect to control of groundwater and surface drainage. Nevertheless, if appropriate care is paid to construction technique and drainage, further cost efficiencies for geosynthetic reinforced soil walls may be possible by using less expensive non-select fills.
a) Connection loads

b) Maximum loads within the backfill

Figure 4 – Influence of backfill-reinforcement interface stiffness values on the predicted maximum reinforcement loads at end of construction
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REFERENCES


Association of State Highway and Transportation Officials, Washington, DC, USA.


Stress Method for Prediction of Reinforcement Loads in Geosynthetic Walls,” Canadian Geo-

Strip Method.” 1996 Annual book of ASTM standards, American Society for Testing and
Materials (ASTM), West Conshohocken, Pa., USA, Vol. 4.09, pp. 698-708.


Walls and the Case for a New Working Stress Design Method,” Geotextiles and Geomembranes,
Vol. 23, pp. 287-322.

Bathurst, R.J, Walters, D.L., Hatami, K., Saunders, D.D., Vlachopoulos, N., Burgess, G.P. and
217-220.

National Concrete Masonry Association, Herndon, VA, USA.

Modulus Parameters for Finite-Element Analysis of Stresses and Movements in Soil Masses,”
Report No. UCB/GT/80-01, Department of Civil Engineering, University of California,
Berkeley, CA, USA.

Elias, V., Christopher, B.R. and Berg, R.R. (2001), Mechanically Stabilized Earth Walls and
Reinforced Soil Slopes - Design and Construction Guidelines, FHWA-NHI-00-043, Federal
Highway Administration, Washington, DC, USA.

Retaining Wall Behavior,” Proc. 55th Canadian Geotechnical Conference, Niagara Falls, Ontario,
8 p.


Itasca Consulting Group (2005), FLAC - Fast Lagrangian Analysis of Continua, v 5.00, Itasca Consulting Group Inc., Minneapolis, MN, USA.


