EFFECT OF REINFORCEMENT CHARACTERISTICS ON SEISMIC RESPONSE OF GEOSYNTHETIC-REINFORCED SOIL BRIDGE ABUTMENTS

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ABSTRACT
This paper presents the results of finite difference analysis on a "segmental bridge abutment", subjected to seismic loading, with special emphasis on reinforcement characteristics. FLAC2D with the FISH programming option of it is used for implementing the desired model for the numerical analysis. The Masing nonlinear hysteretic loading-unloading rule is used for the nonlinear elastic part under seismic condition, after which a Mohr-Coulomb softening model is used for plastic behaviour. The effect of changing in geogrid length, spacing and stiffness is studied using numerical parametric analyses in seismic conditions and the results are illustrated on facing deformation, displacement and rotation of the bridge footing, and the geogrid load. The results show that increase in stiffness and length of reinforcement cause decrease in deformation and little effect on the reinforcement load. The parametric study results show significant effect of the geogrid spacing such that decrease in spacing causes linear decrease in deformation and non-linear decrease in geogrid load.

1. INTRODUCTION
Reinforcement type and its quantity used in a reinforced soil system play an important role in response as well as economy of the system. The advantages such as easy construction, flexibility, cost-effectiveness, and aesthetically pleasing appearance in one hand and producing various type of reinforcing materials in other hand, make geosynthetic-reinforced soil systems more popular in recent years. Geogrids have been used in particular, because of their interactive behaviour with the granular soil which is normally used in reinforced soil structures. Responses of geosynthetic-reinforced walls, such as segmental reinforced walls, have been numerically studied in both static and seismic conditions in recent years (eg. Cai and Bathurst, 1995; Karpurapu and Bathurst, 1995). Bridge abutments also have been constructed using reinforced soil systems, namely reinforced bridge abutments. A special type using segments as the facing of the system was constructed in Denver, Colorado, named Founders/Meadows bridge abutment (Abu-Hejleh et al., 2000)

An elaborated study on numerical modeling of such a system for static and seismic behavior was conducted lately by Fakharian and Mojtehedi (2002), Mojtabahidi and Fakharian (2002), and Fakharian and Attar (2005, 2006, 2006a).

The main objective of this paper is to investigate the effect of reinforcement geometry and stiffness on deformation and load distribution of the system. The parameters include reinforcement length, reinforcement spacings, and reinforcement stiffness. The horizontal displacement of the abutment facing, vertical deformation of the footing supporting the bridge near the abutment top, and the load distribution of reinforcements are to be investigated as the system response.
2. DYNAMIC NUMERICAL MODELLING

The Founders/Meadows bridge abutment constructed and completely instrumented in Denver is used for numerical modelling (Fig. 1). The model generated by FLAC is shown in Fig. 2 with slight changes compared to the prototype, such as changing the facing height from 5.9m to 6.0m, geogrid element lengths from 8 to 7.2m at the base (equivalent to 0.9H) and removing the soil beside the facing wall. As appropriate drainage system has been installed, no water is considered in soil. The bridge deck load is substituted by a 200 kN/m vertical load on the abutment wall.

The initial condition for the seismic analysis is the static stability of the system, which is maintaining the initial stresses while resetting all the deformations. The grid is the same throughout static and seismic analysis, but the boundary conditions and stress-strain relations are different. The soil model is nonlinear elastic with M-C (Mohr-Coulomb) failure criterion under static condition, similar to that used by Hatami & Bathurst (2001), but softening effects after failure are also considered in this study. The hyperbolic parameters for soil and the step construction and loading conditions reported by Abu-Hejleh et al. (2000) are used in the numerical modeling.

In the dynamic analysis, a hysteretic nonlinear behavior applying Masing rule in unload/reload process is used. More information on model verification, grid, boundary condition, loading, soil model and other details are presented in the subsequent sections.

2.1 Model verification

FLAC has been formerly used for reinforced soil walls successfully both in static and seismic conditions (e.g. Hatami & Bathurst, 2001; El. Emam et al. 2004), but the reinforced soil walls used as bridge abutment are relatively new and no seismic numerical results are available in literature, and there is also no instrumented physical modelling test result for comparison purposes. But it is assumed that with the comparison of the static analysis of bridge abutments with the instrumented prototype and also the dynamic analysis with the reinforced soil wall physical tests under dynamic conditions, the numerical modelling of the dynamic response of the bridge abutment is thus verified.

The static analysis results have shown good agreement with instrumentation results of the Founders/Meadows bridge (Fig. 3) reported by Abu-Hejleh et al. (2001). The results of the 1/6 scale shaking table tests on a reinforced wall presented by Bathurst et al. (2002) are used for seismic verification of the numerical model. El-Emam et al. (2004) had used FLAC 2D for comparison with the above shaking table test results. In this study, however, FLAC 2D 4.0 is used applying a nonlinear hysteretic model for soil instead of linear, considering slip between geogrid elements and grid (representing soil mass), and using interface elements instead of a thin layer of soil. Some sample verification results are shown in Fig. 4, details of which are available in Attar (2004) and Fakharian & Attar (2006), Good agreement is observed between measured and predicted results approving the predictions.

2.2 Grid, Boundary condition and loading

Geometry, boundary conditions, loading, reinforcement distribution, interface element and the finite difference grid are all shown in Fig. 2. The backfill width, B, is extended equivalent to 60m behind the face wall segments to minimize the effect of cut-off boundary on the response. Bathurst & Hatami (1998) conducted a parametric study and realized that by extending the backfill width more than five times of the wall height, the effect of boundary will be negligible. Free-field boundary condition is applied to the left and right vertical edges close to which a non-yielding zone is used for soil. A 10-m deep soil layer is placed in one stage as bed-soil; the stage construction is used in the model for placing each soil layer, block segment, and geogrid. Interface elements are placed at the joints between bed-soil and reinforced soil mass and bottom of block segments, enabling the sliding of the reinforced soil as well as the base of the front face.

After establishing static stability and resetting of deformations in all components, the nodes at the base are subjected to a horizontal variable - amplitude harmonic
Non-yielding region

Base acceleration (Time history as shown in the inset)

Right edge of numerical grid and free-field transmitting boundary

\[ \text{Base acceleration (m/s}^2) \]

\[ \alpha_g = 0.5g \]

Interface Elements
1: Face/Soil
2: Block/Block
3: Face/Bed-Soil
4: Reinforced Soil/Bed-Soil
5: Bridge Footing/Soil
6: Abutment Wall/Bridge Footing
7: Polystyrene/Soil
8: Abutment Wall/Polyurethane

Fig. 2. Numerical grid, interface elements, and boundary conditions for the seismic analysis of segmental bridge abutment

Figure 3. Measured and predicted outward Disp. of abutment face in elevation of No. 10 geogrid (Fig. 1) against the construction stages.

Figure 4. Measured and predicted connection loads and horizontal toe loads at difference input base acceleration amplitudes for shaking table test.
ground motion record illustrated in Fig. 2 (inset). This is a simplification of real earthquake records. Bathurst & Hatami (1998) used this record for parametric seismic analysis of reinforced walls with geogrid, and is expressed as:

\[ u(t) = \frac{k}{2} \times \beta \times e^{-\alpha t} \times \sin(2\pi f t) \]

where: \( \alpha = 5.5, \beta = 55, \zeta = 12 \) are constant coefficients; \( f \) = frequency; and \( t \) = time; \( k \) = Peak amplitude of the input acceleration assumed as 0.5 \( g \), and the frequency, \( f = 3 \) Hz, was selected to represent a typical predominant frequency of medium- to high-frequency content earthquakes (inset of Fig. 2). \( t \) is time and varies between 0 and 6 seconds. Equation 1 was implemented in model using the FISH programming of FLAC and was input as points at time intervals of less than 0.0008 sec.

2.3 Seismic Soil model

Considering the dense granular soil type in reinforced zone, the M-C failure criterion with post-peak softening capability is used. A constant shear modulus throughout loading may not be appropriate due to dynamic and nonlinear behavior. To clarify the effect of soil model type on results, both linear and nonlinear models are considered in the analysis.

The M-C elasto-plastic softening model is readily available in FLAC. The nonlinear model was implemented by FISH programming in which the dynamic shear modulus is determined in softening M-C model in tangent from \( G_t \), applying the hysteretic nonlinear Masing rule in reload/unload process. Therefore, tangent shear modulus of each zone varies according to the cyclic stress-strain states within the zone during the loading. The tangent shear modulus if the first cycle is determined by the following relation:

\[ G_t = \frac{G_{\text{max}}}{\left[1 + \left(\frac{G_{\text{max}}}{\tau_{\text{max}}}\right)\gamma_{\varepsilon} - \gamma_{\tau}\right]^2} \]

where \( G_{\text{max}} \) is the initial shear modulus, \( \tau_{\text{max}} \) is the maximum shear stress and \( \gamma \) is the shear strain. The Masing stress-strain equation for the unloading or reloading stage is given by:

\[ G_{\text{max}} = 21.7 \times K_{2\max} \times P_a \times \left(\frac{\sigma_m}{P_a}\right)^{1/2} \]

in which \( \sigma_m \) is the average normal stress, \( P_a \) is the atmospheric pressure, and \( K_{2\max} \) is the shear modulus constant.

The bulk modulus is obtained using a constant Poisson ratio \( (\nu = 0.35) \) and the shear modulus. The main advantage of this model for soil is that there is no need for artificial application of damping to the soil mass and the hysteretic behavior of soil yields to dynamic energy dissipation and the damping is automatically applied on the basis of soil stress-strain state.

The granular materials usually exhibit hardening-softening response in dense condition, whereas in loose condition they show hardening response only. For dense state, the angle of internal friction, \( \Phi \), is considered a variable which reduces from a maximum at zero plastic strain to a residual value of 30° at a large plastic strain. The dilatancy angle, \( \psi \), of the soil was related to the friction angle, \( \Phi \), using the relation proposed by Bolton (1986) with a constant critical state friction angle \( \Phi_{cr}=30 \), i.e.

\[ \phi = \phi_{cr} + 0.8\psi \]

similar to that used by Rowe & Ho (1997).

2.4 Reinforcement model

The reinforcing elements are modeled by elasto-perfectly plastic cable elements with no compressive strength, available in FLAC. The injection layer option around cable elements was used as the interface to simulate the frictional behavior of soil-geogrid. The thickness of this layer was assumed zero and friction angle and cohesion were considered 0.75 \( \Phi \) and zero, respectively. Considering the assigned perimeter around the cable element (in our case equivalent to 2m which is the unit thickness of the wall in plane strain condition and doubled for above and below the geogrid effect) and the confining
stress (determined by program), the slip limit or failure criterion is established.

Bathurst and Cai (1994) studies on two geogrids (PET & HDPE) showed that the geogrid modulus does not vary with loading rate for practical purposes. Therefore, an elasto-perfectly plastic assumption in seismic loading has sufficient accuracy for geogrids.

2.5 Structural components

The structural components of the system include the facing blocks, the abutment wall and the abutment foundation (Figs. 1 & 2). The elastic behavior is used for all the structural components. Interface elements are used at the contact surfaces between concrete structural elements and soil, block/block of facing segments and abutment wall to the abutment foundation.

The segmental blocks are having width and height of 0.28m and 0.20m, respectively. The hollow blocks are having a metal connector to prevent their relative slip and the reinforcing geogrids are attached to them. Cai & Bathurst (1995) reported that relative slip of segmental blocks may cause instability of the segment. To account for the connector in the model, interface elements with high friction are used to prevent the relative slip, but with zero tensile strength to allow their rotation.

Due to insufficient vertical stress between the top few blocks, reinforcing bar and concrete was placed inside the hollow space of the top 1m to increase the stability. The numerical results also indicated such local instabilities by increasing the earthquake magnitude. In the numerical model, considering tensile strength between the top blocks of the unstable zone will contribute to stabilization of the system in that zone.

2.6 Interface element model

The interface element of FLAC was used to model the friction between difference contact surfaces of soil-soil, soil-concrete and concrete-concrete, as stated in the former sections and demonstration in Fig. 2. The shear and normal stiffness of the interface elements were assigned considering the stiffness of the adjacent materials as directed by FLAC, and then the desired friction angle was input. To verify its performance, a concrete block was modeled on a slope and it was observed that the block slides when the slope angle exceeds the friction angle of the interface.

3. ANALYSIS RESULTS

Effect of reinforcement characteristics on horizontal displacement of abutment facing, vertical displacement of the deck footing, and the reinforcement forces are presented in the following sections.

3.1 Effect of reinforcement stiffness (J)

Increase in reinforcement to some extend contributes to reduction of deformations, as eventually slip between soil and reinforcement prevails. It is important however, to determine the influencing magnitude of stiffness on limiting the undesired deformations. A parametric study is thus performed with stiffness magnitudes of 500 (weak geogrid), 1000, 2000 (strong geogrid), 9000, and 96000 kN/m (metal). Some values were formerly used by Bathurst and Hatami (1998) for reinforced soil walls with continuous facing, concerning a wide range of stiffness from low-grade geogrids to metal reinforcements.

Figure 5 presents the maximum normalized horizontal displacement of the abutment facing with respect to the reinforcement stiffness, it is observed that horizontal displacement reduces substantially with increase in reinforcement before reaching 2000 kN/m after which the horizontal displacement variations reduces. No variations are observed after a stiffness equivalent to 9000 kN/m. The normalized vertical displacement variation of the bridge footing with respect to reinforcement is presented in Fig. 6. The variations are similar to those related to horizontal displacement. The analysis results showed that the reinforcement stiffness variations have little impact on the reinforcement forces.

Fig. 5. Effect of reinforcement stiffness on horizontal displacement of abutment facing under seismic condition.

Fig. 6. Effect of reinforcement stiffness on vertical displacement of abutment footing under seismic condition.
3.2 Effect of reinforcement length \((L_0/H)\)

In parametric studies, the reinforcement length is increased with a 1:1 slope from bottom to top, similar to the recent abutment in Denver, Colorado (Fig. 1). The increase in length is appropriate for bridge abutments in which a concentrated load from the bridge deck and vertical pressure from the approach embankment exist, in addition to the retained soil pressure. The reinforcements length is denoted by the ratio of the lower row length \((L_0)\) to the abutment height \((H = 8\, \text{m in this case})\). To investigate the effect of reinforcement length on the seismic response of the system, the \(L_0/H\) ratio has been varied between 0.5 to 1.1.

Figure 7 presents the abutment facing profiles (or in fact horizontal displacements) with reinforcement length variations. The facing profile before seismic loading is at position zero (Fig. 7). The facing profile shifts to the left with cycles of the load, such that higher horizontal displacement ratio is obtained for lower \(L_0/H\) values. The effect of reinforcement length on displacement control, however, reduces with increase in \(L_0/H\) such that no significant changes are obtained for after 0.9 ratios. Figure 8 presents the bridge footing profiles with reinforcement length variations. The figure presents both the footing vertical displacement and rotation.

The results show that increase in reinforcement length has little effect on the vertical displacement as long as the external stability is satisfied. But it should be noted that insufficient reinforcement length results in substantial rotation in the bridge footing, as observed for \(L_0/H = 0.5\).

3.3 Effect of reinforcement spacing \((S_v)\)

The reinforcement spacing is also affecting the internal stability of reinforced walls and abutments. Increase in spacing or in fact reduction of No. of reinforcements causes increase in reinforcement forces. The increase in reinforcement loads increases the slip and pullout potential as well as higher creep type of deformations with time. Increase in reinforcement spacing also violates the continuity of the system.

Figure 9 shows the maximum normalized horizontal deformation of the abutment facing against the vertical reinforcement spacings. The maximum vertical displacement of the bridge footing with increase in spacings is presented in Fig. 10. It is observed that both displacements increase linearly with increase in reinforcement spacings. Figure 11 shows the maximum reinforcement load variations with change in spacings, showing significant nonlinear variations.
4. SUMMARY AND CONCLUSIONS

Parametric analysis results for a segmental bridge abutment using the 2-D finite difference program FLAC4.00 was presented. The numerical model was verified by two well-instrumented reinforced systems, one of which was the Founders/Meadows segmental bridge abutment under static loads, and the second was the shaking table 1/6 scale physical model of a reinforced soil wall.

The FISH programming option of FLAC is used for implementing the desired model for the numerical analysis. An elastic nonlinear model is used up to the failure (peak), after which a Mohr-Coulomb softening model is used for plastic behaviour for both static and seismic conditions. The Duncan Hyperbolic model is used for the nonlinear elastic part under static condition, while the Masing nonlinear hysteretic loading-unloading rule is used for the nonlinear elastic part under seismic condition. The reinforced geogrids are modelled by elasto-perfectly plastic cable elements. The slip limit of geogrid reinforcements are determined by some factors such as the confining stresses, perimeter, and friction angle around the geogrid.

The main objective of this study was to propose some guidelines for design engineers on optimized selection of reinforcement geometry and properties. Parametric studies were performed to study the effect of reinforcement stiffness, length, and spacing in segmental bridge abutments. The following conclusions can be made out of the presented results:

- Overall, increase in reinforcement stiffness, despite its large span, has a limit in contribution to reduction in deformations and load distribution.
- Utilizing steel reinforcement ($J = 69000$ kN/m) compared to stiff geogrids ($J = 9000$ kN/m) does not help to reduce the deformation under seismic excitations. However, the designer can choose geogrids with stiffness between 2000 to 9000 kN/m depending on the design requirements.
- Weak geogrids with stiffness below 2000 kN/m may not be appropriate for bridge abutments.
- To satisfy the internal stability of the system, the reinforcement has to be long enough to exceed the failure wedge. The critical length for reinforcement ($L_0/H$) is obtained 0.7, below which system instability may occur.
- An initial length ratio ($L_0/H$) of more than 0.9 has no effect on seismic response improvement of the system.
- The reinforcement spacing has a linear contribution to vertical and horizontal deformations, but it has a nonlinear relationship with reinforcement load distributions.
- Further studies with different abutment heights and bridge deck load levels are required to generalize the aforementioned conclusions.

5. REFERENCES


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