Numerical Analysis of Geosynthetic Reinforced Soil Wall as Bridge Abutment

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ABSTRACT

This paper presents the finite element analysis of a geosynthetic reinforced soil wall as a bridge abutment built in Tehran, and the predictions are compared with the available field measurements. This abutment is analysed using both Limit Equilibrium Method (LEM) for stability analysis and Finite Element Method (FEM) for deformation analysis. Two dimensional plane strain finite element model is adopted for the simulation. Polyvinyl Alcohol (PVA) geogrid with high tensile moduli and low creep characteristics has been adopted in this project to limit the deformation of the bridge abutment. In this model, the backfill soil and geogrids simulated adopting Mohr-Coulomb model, and the elasto-plastic material model that only works in tension, respectively. Bridge abutments can be stabilised by including geosynthetic layers with high tensile moduli satisfying both stability and deformation criteria reducing the construction cost and time, post construction deformations, and future maintenance cost.

Keywords: Bridge Abutment, Geosynthetic, Reinforced Soil, Finite Element

1 INTRODUCTION

Reinforced soil wall is a cost effective option for retaining structures, and is being increasingly used in recent years around the world. In comparison to other retaining structures, geosynthetic reinforced soil has received the highest attention due to low material cost, short construction period, ease of construction and aesthetic appearance. Geosynthetics may be also used to stabilise platforms for heavy roads and rail tracks both during construction and as a maintenance procedure (Fatahi and Khabbaz, 2011). Instead of a conventional bridge deck supported on a pile-cap or concrete wall abutments, geosynthetic reinforced soil walls (GRS walls) composed of alternating layers of compacted fill material and geosynthetic reinforcement such as geogrids or geotextiles to provide support for the superstructure may be employed. GRS bridge abutments with a flexible facing have been the subject of several studies (e.g., Gotteland et al., 1997; Adams, 1997; Ketchart and Wu, 1997; Miyata and Kawasaki, 1994; Werner and Resl, 1986; and Benigni et al., 1996). There has been major projects adopting this system for the bridge abutments such as Vienna railroad embankment in Austria (Mannsbart and Kropik, 1996), the New South Wales GRS bridge abutments (Won et al., 1996), the Black Hawk bridge abutments in Colorado, (Wu et al., 2001), the Founders/Meadows bridge abutments in Colorado (Abu-Hejleh et al., 2000) and Ilsenburg bridge abutment in Germany (Herold, 2000). This paper presents the numerical analysis and performance study of Milad GRS bridge abutment in Tehran which provides an access to Milad Tower, tallest tower in Iran and sixth tallest tower in the world.

2 MILAD GRS BRIDGE ABUTMENT

Milad Tower is located in Tehran, Iran and is the sixth tallest tower in the world at the moment and stands 435 m high from base to tip of the antenna. Milad GRS bridge abutment provides access to Milad Tower. This bridge abutment carries the load of 20m single span of a 114m long cable-stayed bridge on west side of Milad Tower. The length of the sill is 23.7m and the bridge consists of 4 lanes. The maximum height of the GRS abutment to below the sill (lower wall) is 3.5m and total height of the abutment to top of the pavement is 7.5 meters. The geogrid length in lower wall is 9.5 meters. In addition, southern wing wall has 8 meters height with geogrid length of 7 meters in perpendicular direction to the GRS abutment section. The location, view and a cross section from the bridge and abutment are shown in Figures 1 to 4.

Figure 1. The location of the Milad GRS bridge abutment in Tehran (Courtesy of Google Map)

Figure 2. Completion of the bridge deck in 2011 (Courtesy of Google Map)

Figure 3. The view of the Milad GRS bridge abutment with gabion facing

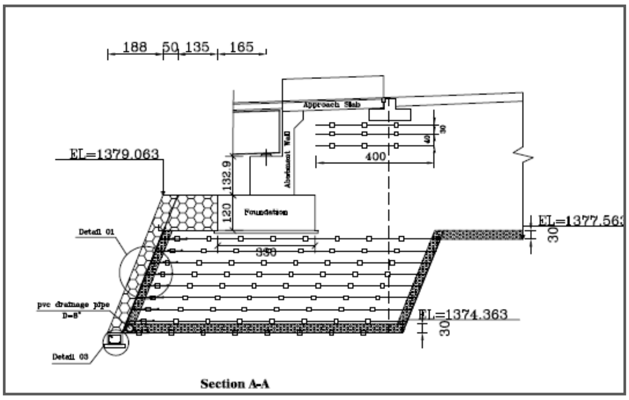

Figure 4. A cross section from Milad GRS bridge abutment (lower wall=3.5m, upper wall=4.0m, total height=7.5m)

3 MATERIAL PROPERTIES AND SITE CONDITIONS

According to the geological classification provided by Rieben (1966), the city of Tehran is founded on Quaternary alluvium. The city is located at the foot of the Alborz Mountain Range, part of Alpine-Himalayan belt with high earthquake potential. The ground is mainly composed of Eocene pyroclastic deposits. From engineering point of view, the natural soil of the bridge location consists of very dense (N_{spt}>50) mixture of gravel and sand with clay (GC/SC-clayey sand with gravel or clayey gravel with sand) with the low to medium cementation. The clayey soil contains clay with Plasticity Index (PI) of lower than 15% and fine content of less than 20%. According to the laboratory testing results, the in-situ soil of the area has been a suitable material for the soil reinforcement purposes and meets the design criteria. Therefore, the natural soil has been re-used as the backfill material.

Geogrid material is used as the reinforcing elements in this project. Some of the advantages of the applied geogrid are: high tensile strength at small strains, high resistance to chemicals in soils, high level of microbiological resistance, high resistance to damage during installation, optimal interlocking with coarse-grained soils, high pull-out resistance, and low values of creep ensuring the long-term stability. According to the reinforcement manufacturer's datasheet, the combined reduction factor to evaluate the design strength (T_a) of this product with consideration of creep (A1), installation damage (A2), connections (A3), and acid and alkaline effects (A4), is 1.6 for design life of 120 years ($T_a=T_{ult}/[A1\times A2\times A3\times A4\times 1/\gamma]$). The characteristics of Milad GRS bridge abutment are summarised in Table1. The adopted engineering properties of the backfill soil are based on the previous geotechnical site investigations in the area including several direct shear and triaxial tests on disturbed samples.

Table 1: Characteristics of Milad GRS bridge abutment

Height	Back fill	Reinforcement type	Reinforcement Spacing/ length	Facing type/ connection	Sill
3.5m (lower wall), 7.5m total	Sand and gravel with clay (GC/SC) c = 10 kPa $\phi=35^{\circ}$ $\gamma = 20$ kN/m^3 (density >98% of T-99)	Kordarna, Armatex M(80/30) for the lower wall T_{ult} = 80 kN/m@$\varepsilon_{\alpha} =$ 6% & M(55/30) for the upper wall	9.5m @ 0.4m for the lower wall, 4m@ 0.4 for the upper wall	Geogrid with Gabion facing, with angle of 68° (2H:5V)	1.2m×3.3m with 1.35m clearance from the face

4 ANALYSIS AND DESIGN CRITERIA

Milad GRS Bridge abutment has been analysed and designed according to Federal Highway Administration Manual (FHWA-NHI-00-43, 2001) and National Cooperative Highway Research Program (NCHRP REPORT 556, 2006) as a guideline, considering Allowable Stress Design (ASD) method. Table 2 summarises the minimum required factors of safety for GRS abutments in ASD method based on FHWA (2001). Considering the field studies of actual structures, AASHTO (1996) suggests that tolerable angular distortions (i.e., limiting differential settlements) between abutments or between piers and abutments be limited to 0.005, and 0.004 for simple and continuous spans, respectively. This means that, for instance, for a 20 m span with no ensuing overstress and damage to superstructural elements, differential settlements of 80 mm for a continuous span or 100 mm for a simple span, would be acceptable. On an individual project basis, differential settlements of smaller amounts may be required considering the performance criterion.

Table 2: Milad GRS bridge abutment design criteria based on FHWA (2001) in static (1) condition

External stability						Internal stability		
Overall slope stability(2)	Bearing capacity			Overturning	Sliding	Pull out	Rupture	connection
External/ Internal/ Compound	deep seated	lateral squeeze -soft soils	eccentricity					Wrapped
1.5	2.5	(NA)	e>L/6	2	1.5	1.5	1.5	Return length(3)>1 m

1. The above factors of safety will be reduced to 75% in seismic condition. The PGA of the site in this study is 0.35g for 475 years earthquake.
2. The factor of safety is 1.6 in RTA standard (R57)
3. The return lengths of geogrids are 1.6m and the only upper geogrid layer beneath the sill is 3.5m in this project with wrapped connection because of seismic considerations.

5 NUMERICAL MODELLING

A series of 2D finite element analysis has been undertaken using PLAXIS 2D V9 in plain strain condition during and after the construction of the GRS abutment. The construction analysis of the wall was conducted layer-by-layer following the sequence exactly as in the field. Compaction stresses induced during construction were not considered in the analysis. The wall section is modelled using 15 node plane-strain triangular elements. The stiffness matrix for quadrilateral interface elements is obtained by means of Gaussian integration using the integration points. Although significant effort was put into the modelling of interface element, it was observed that this modelling aspect only has a minor effect on the analysis outcomes in spite of difficulties in mesh generation around interface elements. The construction stages in the modelling are summarised in Table 3. The mesh is also depicted in Fig. 5 illustrating that more elements are available for analysis in the geogrid interface. Standard fixities are selected to create the boundary conditions, where the roller boundary conditions are generated at the vertical sides and the pin fixities at the base. The water table is assumed well below the surface of the ground.

Table 3: summary of the construction stages in the numerical modelling

Construction and analysis stages	Description
1	Excavation of natural ground
2	Lower wall construction layer by layer
3	Sill construction
4	Upper wall construction
5	Bridge construction and traffic loading

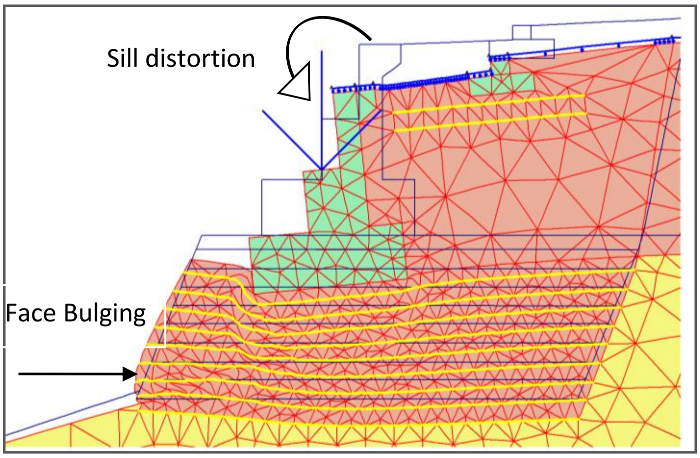

Figure 5. Mesh deformations after construction of the bridge deck and traffic loading in stage 5 of Milad GRS bridge abutment with PLAXIS 2D.

The natural in-situ soil re-used as reinforced soil material has been compacted in 150mm thick layers with compaction level of more than 98% of AASHTO T-99, and ±2% of optimum moisture, w_{opt}. Backfill and natural soils have been modelled considering Mohr-coulomb model with the parameters summarised in Table 4.

Table 4: Soil and concrete properties used in the analysis

Unit	c'(kPa)	ϕ' (degree)	ψ (degree)	E(MPa)	ν	γ (kN/m^3)
Backfill	10	35	2	80	0.3	20
Natural	40	36	4	150	0.3	20
Concrete (Sill)	Linear Elastic model			30000	0.25	24

c': *effective cohesion*; ϕ' *effective internal friction angle*; ψ: *dilation angle;* E*: Young's modulus;* ν: *Poisson's ratio*

Numerical modelling performed by Rowe and Ho (1998) have concluded that the reinforcement tensile stiffness have a significant effect on the deformation of GRS walls. Thus, Polyvinyl Alcohol (PVA) geogrid with high tensile moduli and low creep characteristics has been adopted in this project. In comparison to the other commonly used geogrids, due to the high moduli under extension of the adopted PVA geogrid, lower deformations of the wall are expected. Table 5, summarises the properties of the geosynthetic adopted in the numerical modelling.

A surcharge of 20 kPa has been considered for traffic loading and a line load of 500 kN/m has been considered for ultimate induced load from the bridge deck to the sill. For simplicity, gabion facing elements giving extra stability have not been considered in the numerical simulations.

Table 5: Geogrid properties used in the analysis

Geogrid Type	Location	Description	T_{ult} (kN/m)@ $\varepsilon_{\alpha} = 6\%$ (MD)/(CD)[1]	Axial stiffness EA (kN/m)
1	Lower Wall	Armatex M, woven geogrid, high tenacity PVA (Polyvinyl Alcohol) yarns with PVC coating	80/30	1333
2	Upper Wall		55/30	915

[1]*MD*: *Machine Direction*; *CD*: Cross-*Machine Direction*.

6 RESULTS AND DISCUSSION

The analysis results are well consistent with the displacement monitoring results during various stages of the construction. The maximum horizontal and vertical displacements of the GRS abutment are reasonably below the design criteria as shown in Table 6. Numerical predictions indicate 0.5mm, 1mm, 3mm, and 7mm of the maximum accumulative horizontal displacement on the facing at the end of Stages 2, 3, 4, and 5, respectively. The maximum accumulative settlements of the facing are predicted to be 5mm, 6mm, 10mm, and 15mm at the end of Stages 2, 3, 4, and 5, respectively. Predictions are in a good agreement with the available field measurements indicating less than 2mm horizontal and vertical deformation in Stages 3-4. Finite element analysis results show 2.6mm, 9.6mm, and 22mm of accumulative maximum settlement for the sill at the end of Stages 3, 4, and 5, respectively. The results are in a good agreement with the 3mm maximum observed settlement of the sill in Stage 4. Both predicted and observed horizontal displacement of the sill in Stages 3 and 4 are less than 2mm. It is observed that the available displacement monitoring results are less than the predicted values. It should be noted that the geogrids have been modelled by linear elastic-plastic elements. The assumption of the linear elasticity of geogrids is warranted because the observed maximum geogrid strain has been well below 1% in which the geogrid behaviour is linear. The distribution and maximum axial forces in geogrids are presented in Figure 6. It is observed that the axial forces are well below the maximum design strength (T_a) of the geogrids. In addition, the distortion of the sill in Stage 5 is anticipated to be less than 0.05° (forward tilt). Furthermore, Figure 7 illustrates the distribution of plastic points after construction of the bridge deck and traffic loading in Stage 5.

It is noted that although monitoring and instrumentation of GRS walls are costly and complicated tasks, installation of inclinometers as well as extensometers on geogrid layers during construction is recommended to better understand the behaviour of GRS walls. This can contribute to further develop and improve the current design and analysis procedures.

Table 6: Predicted accumulative displacement values in comparison with monitoring results up to July 2011

Construction stages	Facing maximum displacements				Sill maximum displacements			
	Horizontal (mm)		Vertical (mm)		Horizontal (mm)		Vertical (mm)	
	Calc.	Monitored	Calc.	Monitored	Calc.	Monitored	Calc.	Monitored
Stage 2	<1	-	5	-	-	-	-	-
Stage 3	<1	<1	6	<1	<1	-	2.6	-
Stage 4	3	<1	10	<1	1	<1	9.6	2-3
Stage 5	7	[*]	15	[*]	3	[*]	22	[*]
[*] The bridge has not been under traffic loading yet.								

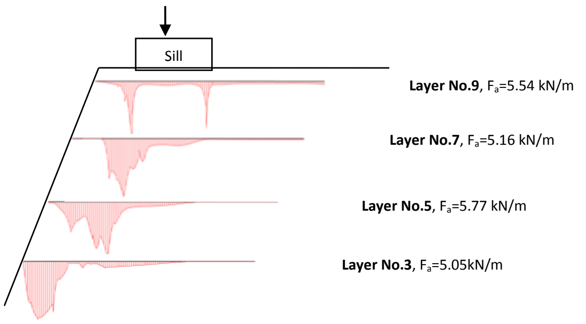

Figure 6. The distribution of axial force in geogrids predicted due to the loadings in Stage 5

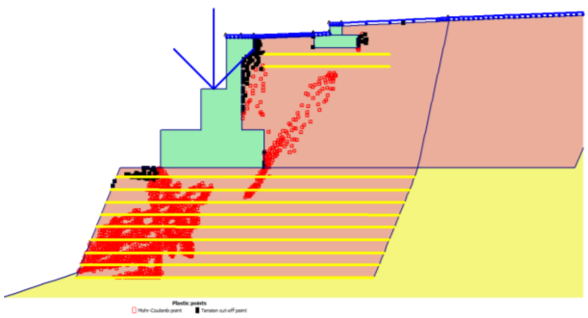

Figure 7. Plastic points after construction of the bridge deck and traffic loading in Stage 5.

7 CONCLUSIONS

The Geosynthetic Reinforced Soil (GRS) is one of the most appropriate solutions for bridge abutments considering the abutment performance, construction cost, time and safety in comparison to other conventional methods and it is observed that the numerical modelling can contribute to more innovative and effective GRS walls and bridge abutment design. The numerical and available field measurement results show that the displacements of Milad GRS bridge abutment is well within the design criteria, and the reinforced abutment mass, employing Polyvinyl Alcohol (PVA) geogrid with high tensile moduli and low creep characteristics, has high stiffness and capacity for heavy loads as anticipated. The analyses show that the maximum calculated axial forces of the geogrids are well below the design strength (T_{allow}) of the geogrids and the maximum tensile strain of the geogrids is less than 1%. The most effective factors influencing the performance of the GRS walls are the quality of the compacted soil, type of the reinforcement materials as well as the construction details. It should be noted that the abutment is not yet open to traffic, thus further field monitoring is being conducted which will be disseminated in the follow up papers.

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Foreword

The first Australia New Zealand Conference on Geomechanics was held in Melbourne, Australia, on 9-13 August, 1971. It was sponsored by the then just formed Australian Geomechanics Society and the New Zealand National Society of Soil Mechanics and Foundation Engineering, both technical units of the Australian and New Zealand Institutions of Engineers, respectively. There were 85 listed authors; Bill Bamford, Ted Davis, Charles Gerrard, Peter James, Geoff Just, Marcis Kurzeme, Peter Moore, Harry Poulos and Len Walker are among the notable ones who presented more than one paper at that time.

Bill Bamford and John Styles, witnesses of the 1st ANZ, today remember that the meeting was the first in the region to use the title "Geomechanics", embracing rock mechanics and engineering geology alongside soil mechanics, and welcomed major participation from the mining industry as well as the traditional civil engineering supporters. For many young engineers who attended, it was the first time to meet people they knew only as authors of technical papers. To discuss issues and gain insights as to thinking beyond problem solving was of particular benefit.

The Australia New Zealand Conference on Geomechanics is the regional conference of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) and is held approximately every 4 years. The 11th ANZ conference has returned to Melbourne, where it all began, more than 40 years later. It focused on "Ground Engineering in a Changing World". The spirit of the theme of the 1st ANZ, "Geomechanics – A Tool in National Development", was still embraced in the recent 11th ANZ. However, the world has changed, and the conference encompassed not just geomechanics, but a more comprehensive ground engineering. Changes around the world seem to have accelerated with time, and ground engineers are learning to react to those: climate change, financial systems change, legislative change, clients' sophistication change, not to mention the tremendous technological changes. This conference aimed to explore and better understand these changes and the risks and opportunities they present to the profession.

The main themes of the 11th ANZ conference included:

1. Supporting our Structures
2. Evolving Geotechnics & Site Characterisation
3. Mining and Underground Geotechnics
4. Sustainable Geotechnics and Geo-Environmental Engineering (in a Resource Hungry World)
5. Near-shore and Off-shore Geotechnics
6. Geo-Hazards and Risk

During the conference, 5 invited speakers, from both Industry and Academia, local and global, provided a review of topics and innovations that are pivotal to ground engineering in our changing world. In addition, a total of 270 peer-reviewed technical papers were presented and discussed, in oral and poster presentations, throughout the conference. This allowed exchange of advanced knowledge and ideas, cross fertilisation, and promotion of a true community of colleagues in ground engineering. Authors of the 15 highest ranked papers have been invited to resubmit an extended version to the International Journal of Geotechnical Engineering, for publication in a special edition. In addition, papers of particular relevance to Australasia will be included in the AGS Australian Geomechanics Journal.

We would like to thank all members of the Organising Committee, the Senior Advisory Committee, professional conference organisers and helpers, invited speakers, our reviewers, our sponsors, and more importantly, all authors and delegates for their sincere efforts and collegial collaboration. Without them Melbourne ANZ 2012 would have not been the success it was.

Guillermo A. Narsilio
Arul Arulrajah
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