Proceedings of the 8th World Congress on Civil, Structural, and Environmental Engineering (CSEE'23)
Lisbon, Portugal – March 29 – 31, 2023
Paper No. ICGRE 103
DOI: 10.11159/icgre23.103

Performance of Three-Tiered Geogrid Reinforced Soil Wall with Modular Concrete Facing Block: A Case Study

Siamak Yoosefi Sigari[1], Hadi Barghemadi[2], Amir Akbari Garakani[3], Christopher L. Meehan[4]
[1] Graduate Student at Dept. of Civil & Environmental Engineering, University of Delaware
19716, Newark, USA,
siamakys@udel.edu; aakbari@nri.ac.ir; cmeehan@udel.edu
[2] MSc Degree of Geotechnical Engineering, Tehran University of Technology, Tehran, Iran
[3]Assistant Professor, Structural Research Dept., Niroo Research Institute
1468617151, Tehran, Iran
[4]Professor at Dept. of Civil & Environmental Engineering, University of Delaware
19716, Newark, USA,

Abstract **-** This paper describes a case study of the construction of three-tiered geogrid reinforced soil wall with modular concrete facing block, backfilled with soil-rock mixture, with total height of 20 meter, which is constructed to meet demanding site grade requirements. The construction issues, materials and project components are well documented and described. The results of limited monitoring program by surveying 18 specific points on facing of the embankment during and after the completion of construction are presented. Vertical deformations in sections with maximum height have higher values in the upper portion of each tiered wall ranging from 9 (mm) to 29 (mm). Horizontal movements of wall system are about 0.8% to 2.4% of the tiered wall height, which is in the order of anticipated movements from different guidelines. Finite element analysis using GeoStudio Sigma/w were performed under working stress condition. The soil modelled using elastic-plastic model with Mohr-coulomb failure criterion, and a linear elastic model considered for the geogrid reinforcement, which are modelled with beam elements. The interfaces between the adjacent concrete blocks with shear key were modelled using the thin layer element, and connection between the reinforcements and the surrounding soil is assumed with perfect interface adherence. Besides the challenges of predicting the facing displacements of mechanically stabilized wall with numerical modelling, the results of the numerical analysis are in acceptable agreement with measured field displacement.

Keywords**:** Reinforced Multi-tiered wall, Geogrid, FEM analyses, Field monitoring

1. Introduction

Construction limitations may require a steeper slope than is stable in compacted embankment soils without reinforcement. Geosynthetic reinforcement are widely used to improve the stability of the compacted fill embankments by providing tensile reinforcement. In the case of steep or vertical facing, the use of rigid facing is accommodating to enhance the static stability and reducing seismic settlements [1]. On the other hand, using stiff facing instead of flexible systems, results in significant reductions in reinforcement loads due to the increase in soil confinement caused by the facing system [1],[2]. The use of modular concrete block walls (SRWs) has the largest growth due to construction easiness like ease of placement using manual labour and ease of geosynthetic connection to the facing [3]. When there is obligation to use high height MSE walls, multitiered walls are often used, in which an offset between adjacent tiers is used. Empirical data available for multitiered geosynthetic walls is inadequate and generation of an extensive database for tiered walls is a major challenge [4]. The backfill in reinforced soil structures is the key element in satisfactory performance, any accessible soil at the site can be used as backfill materials, with considering good drainage, careful evaluation of soil and reinforcement interaction characteristics, field construction control, and performance monitoring [5]. However, there are few studies regarding the usage of soil-rock mixtures as the reinforced backfill. Some issues regarding geogrid installation and layer compaction may arise during construction due to existence of rock materials [6], so special care will be required during design phase and construction. Performance monitoring is recommended for reinforced fill soils that fall outside of the requirements listed by berg et al. (2009) [5], to ensure the proper function of the structure. The reinforced embankment components and results of limited monitoring program are presented for this case study.

2. Project Description

The site is located in west of Iran and is part of a water conveyance project. The 41600-m3 embankment is part of the route that is constructed to facilitate the crossing of a valley. The topographic condition of the project was the main obligation for construction the embankment to meet demanding site grade requirements. The embankment foundation after maximum 2-meter alluvial deposits in riverbed, and its abutments consist of claystone and siltstone with layers of sandstone, the top alluvial foundation has been completely excavated and replaced by dental concrete. A cast-in-place concrete box culvert with the dimension of 6.2×7.2 meter (width × height) was first constructed to carry the seasonal water under the embankment.

As simply described by Berg et al (2009), the construction sequence of MSE (Mechanically Stabilized Earth) structures consist mainly of preparing the subgrade, placing, and compacting backfill, laying reinforcing layer into position and installing the facing elements [5]. Figure 1 shows the progress of reinforced embankment construction. Construction ended after about 300 days, but a surcharge of 3-meter height soil on the crest of the embankment have been applied to lessen any possible settlements under working stress condition after the construction of concrete box culvert for water conveyance on the embankment. The embankment is 20-meter height consists of geogrid-reinforced soil with the height of 8-meter span followed by a 2.5-meter length berm and the facing system of dry cast modular block, and 1H: 6V side slopes. The total length of the embankment is about 134 meters. The reinforcement are placed horizontally from upstream face continuously to downstream face and spaced from 0.40 to 0.20 meter vertically (depending on the elevation it is being placed). Drain material with 0.5-meter width is placed and compacted with special care between backfill materials and facing elements. The base of the retaining wall was embedded into the bedrock to ensure adequate toe restraint. Figure 2 shows the maximum cross section of the reinforced embankment with the details of geogrid layout, which were determined to satisfy the requirements of FHWA and NCMA [7] design codes. Figure 3 shows the reinforced embankment at the end of construction.

2.1. Backfill Materials and Construction Challenges

The local or on-site soil-rock mixtures have been used as reinforced backfill in this project, which have rock content (particle size > 76.2 mm) of approximately 8% by weight. Apart from the rock content, the soil can be classified as poorly graded gravel with silt and sand according to United Soil Classification System. For controlling the compaction quality, the dry unit weight determined with water replacement method in a test pit based on ASTM D5030 method B and the average dry unit weight 2.20 (gr/cm3) were achieved. The shear strength of reinforced backfill were determined by direct shear test under consolidated drained conditions based on ASTM D3080 and the internal friction angle of 35 degree and Cohesion of 0.01 (kg/cm2) have achieved. Considering that the rock content could increase the friction angle of soil-rock mixtures (Lindquist. 1994; Xu et al. 2011) the results of the direct shear test are on the safety side. A plate load test (PLT) was carried out based on ASTM D1194 on a compacted layer during construction to govern the elastic modulus of the reinforced backfill by in-situ soil test and a value of 500 (kg/cm2) have attained. The materials have a sodium sulphate soundness loss of 3% for fraction of coarser than No. 4 sieve and 9% for fraction finer than No. 4 sieve after four cycles based on ASTM C88 test procedure, which is less than 15 percent that is required for backfill materials [5]. Table 1 shows the properties of reinforced backfill materials obtained from the laboratory and in-situ tests based on ASTM standards [8].

Figure 4 shows the maximum, minimum and average gradation curve of backfill soil materials, and the results are compared by the recommendations of three different references. Reinforced backfill materials are poorly graded gravel that contain rock content lower than 10 percent. It is evident that according to different guidelines our reinforced backfill are out of allowable range, and have maximum particles exceed 19 mm, which will definitely lead to installation damage and compaction issues that needed great care and proper inspection during construction. Based on previous studies [9] and current project observations, this additional damage due to existence of oversize materials would reduce the long-term strength and stiffness of the backfill materials and have raised the attention in the design and construction phase.

The only factor, which consider the damage caused by the mechanical actions during design process, is installation damage factor (RF_{id}), which only accounts for the damaging effects of placement and compaction of soil or aggregate over the geosynthetic during installation and does not consider misplacing that might be inevitable during construction due to existence of oversize materials. Construction obligations outlined by berg et al. (2009) states “the reinforcement should be secured with retaining pins to prevent movement during reinforced fill placement" [5], but slippage of geogrid sheets during fill placement, even by using retaining pins, is inevitable and might happen to some extent and it is the defect, which had not been considered in evaluation of allowable tensile strength. Especially in the areas where we use overlap connection, the steadiness and eliminating the slippage or movement of two overlapped geogrids is of great importance. Minimizing this defect have been achieved with considering two main tasks below:

- Use retaining pins with closer spacing that needs more labour;
- Use Nylon Cable Ties in overlap area of adjacent geogrid rolls;

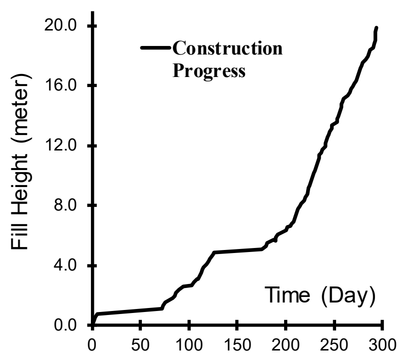

Fig 1: Progress of embankment construction.

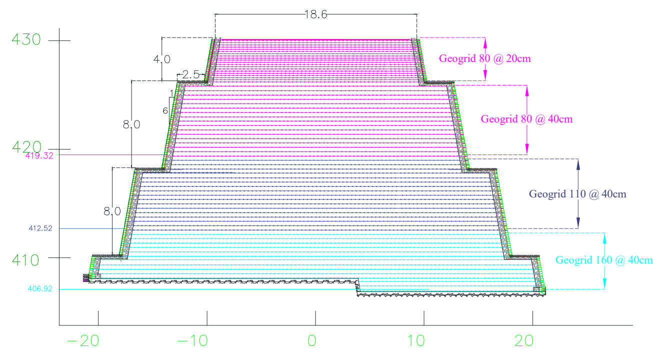

Fig 2: The maximum cross section of the reinforced embankment.

Fig 3: Reinforced embankment at the end of construction (Downstream View).

Dowel with the diameter of 10 or 12 mm was shaped like U for using as retaining pins, which were used with spacing of 5 to 10 meters to pin the geogrid roll to the subsoil layer. It has been observed that using metal pins with the spacing of 10 meter was so susceptible to geogrid slippage but reducing the spacing to 5 meters considerably reduced this defect. Besides, using Nylon Cable Ties in the zones where the two adjacent sheets of geogrid were overlapped was very effective in order to avoid the movement or deformation of the reinforcement during fill placement. It has been observed during the construction, mentioned tasks would reduce the movement of the reinforcements, but sometimes the slippage of geogrid was inevitable due to the existence of oversize materials (>76.2 mm).

Table 1: Properties of reinforced backfill materials.

Parameter		Test description	Standard	Value	unit
Unified Soil Classification System		-	ASTM D2487	GP-GM GP-GC	-
Fractions	Cobbles		Particle-Size Analysis of Soils	ASTM D 422	0-8
	Gravel				56-69
	Sand				19-30
	Fine Materials				4-14
LL, PL, PI		Atterberg Limits of Soils	ASTM D4318	NP 24, 16 ,8	-
W_{opt}		Laboratory Compaction Using Standard Effort	ASTM D698	6	%
γ_{dmax}				2.22	gr/cm^3
Shear strength parameters		Direct Shear test ($30^{cm}\times30^{cm}$)	ASTM D3080	Ø=35	degree
				C=0.01	Kg/cm^2
Loss by abrasion and impact		Resistance to Degradation in the Los Angeles Machine	ASTM C131	28.2	%
Weighted percentage loss	Coarser than No. 4 sieve		Soundness of Aggregates by Use of Sodium Sulfate	ASTM C88	3
	Finer than No. 4 sieve				9

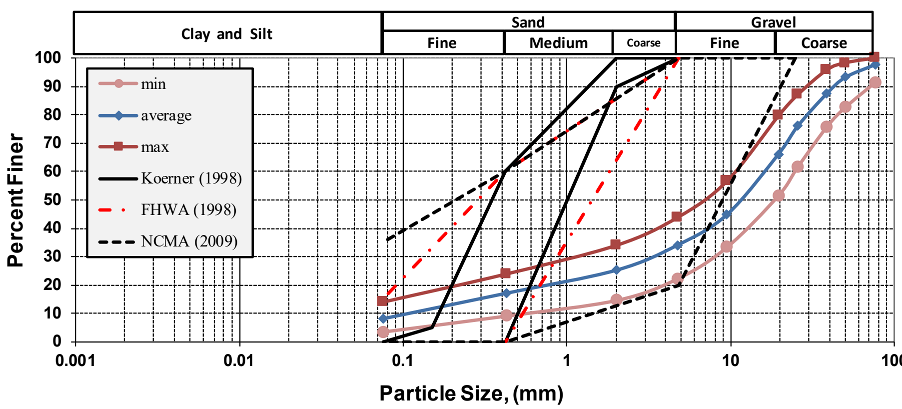

Fig 4: Gradation curve of reinforced soil materials

2.2. Reinforcement and facing system materials

Reinforcements are geogrids made from high-tenacity PET yarns that provides high tensile strength at low elongation, which are covered with PVC coating. Three types of geogrids with different mechanical properties were used, table 2 shows the mechanical properties of the geogrid reinforcements. Figure 5 presents the results of performed multi-rib tensile test of geogrid reinforcement on GP 110 and GP 160 according to ASTM D6637. The facing elements have a dimension of 457×300×200 mm (Length × Width × Height) with 28-day Compressive Strength of 210 (Kg/cm2). The pre-casted concrete facing block units have a shear key that aid in maintaining side slope of approximately 80 degree (wall batter close to 10 degree).

Table 2: Properties of geogrid reinforcement.

Parameters	unit	GP 80	GP 110	GP 160
Tensile Strength (Longitudinal)	kN/m	80	110	160
Tensile Strength (Transverse)	kN/m	20	20	20
Strain at F_{max}	%	11	11	11
Mesh Size	mm	33×25	31×25	30×25

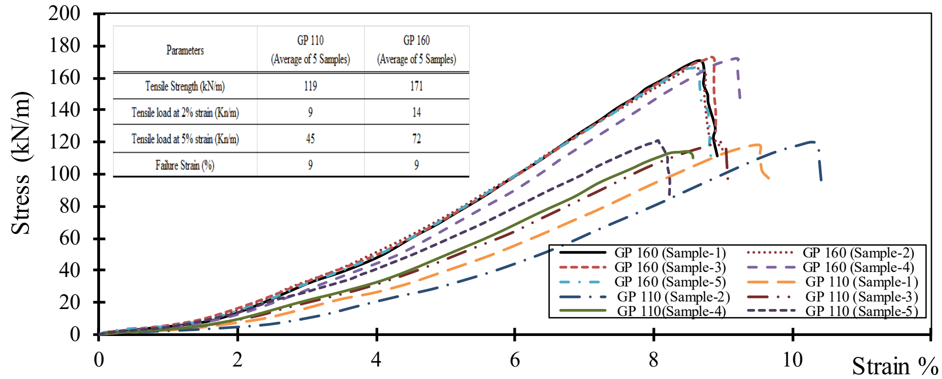

Fig. 5: Stress-strain curves from multi-rib tensile test for GP 110 and GP 160 in longitudinal direction

3. Limited monitoring program

The limited monitoring program based on the instruction outlined by Berg et al. (2009) [5] was carried out to monitor the behaviour of embankment and record movements of facing elements of embankment by surveying method. Vertical arrays of reflector points were set up at nine different wall sections, and each array consisted of from two to four points fixed into the block face. Figure 6 shows survey reflector point's layout on the facing of the embankment. A monthly reading for both horizontal and vertical movements of specified points was carried out during construction and after the embankment completion. Local movements or deterioration of the facing elements have been monitored by visual observations, and it can be reported that after two heavy rainfalls after the completion of the construction, there were no evidence of deterioration of modular concrete blocks or any kind of longitudinal or transverse cracks on the crest of the embankment.

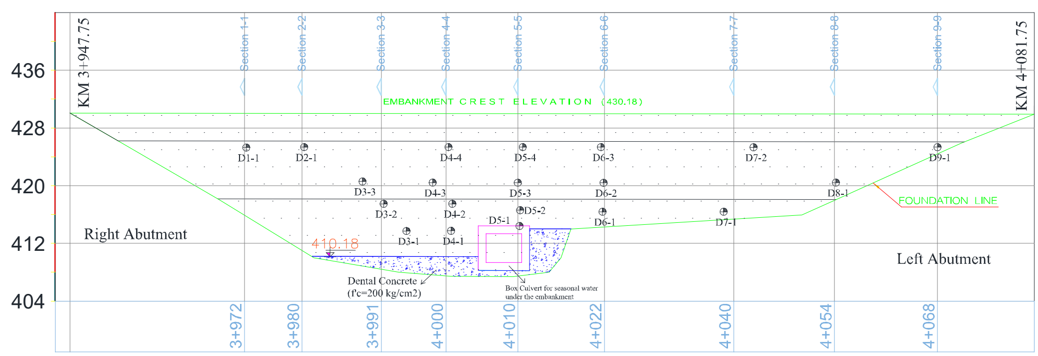

Fig. 6: Location of monitored point on longitudinal section of downstream facing

3.1. Vertical deformations

Figure 7 presents the results of the measured vertical displacements of points in two sections with the maximum embankment heights, with the progress of embankment construction. Generally, the vertical deformation in all points in all sections increased from start until the end of the construction (including the surcharge loading), after that, the monitoring continued for about 9 months and even after the rainy season in the last two month the amount of vertical settlement did not change significantly and remained constant. The vertical deformation has the minimum and maximum values of 9 (mm) at point D5-3 at station 4+010 (section 4-4) and 29 (mm) at point D3-2 at station 3+991 (section 3-3) respectively.

By comparing the final vertical deformations in figure 8, it can be concluded that the vertical deformation for points near the right abutment and the centre of the valley are higher to some extent than the values of the points on the left abutment, but no specific trend can be gained from scrutinizing the results. For better comparison of the results, the variation of final vertical deformation of points located in sections with maximum height versus the normalized heights are compared in figure 9. The normalized height of each point is based on the position of the point on each tiered reinforced wall and the total height of the tiered wall. The variation of the results shows that vertical deformations have higher values in the upper portion of each tiered wall, except for section 4-4 that all the four points have the same value of about 20 (mm).

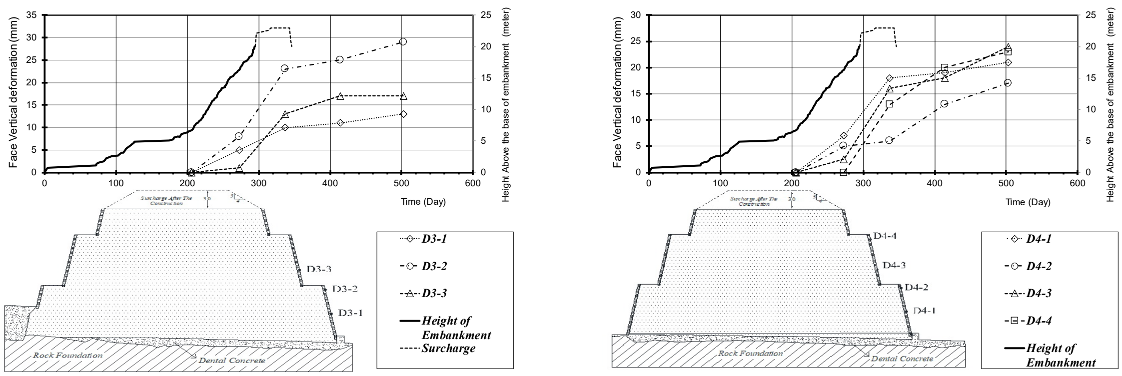

Fig. 7: Vertical Deformation of points on downstream facing in (a) section 3-3, (b) section 4-4.

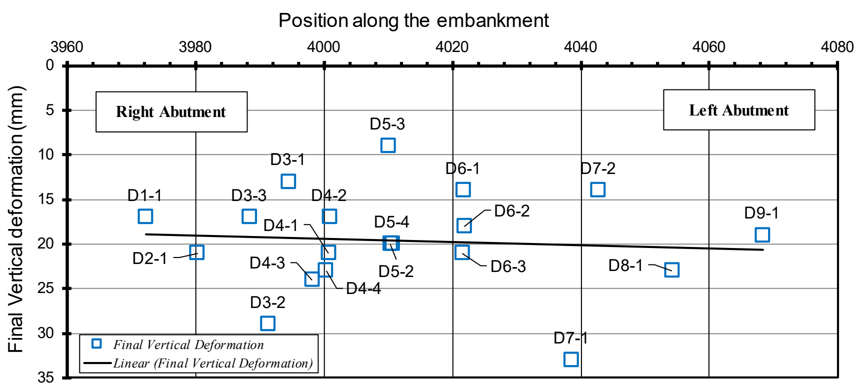

Fig. 8: Final vertical deformations of the monitored points on the facing.

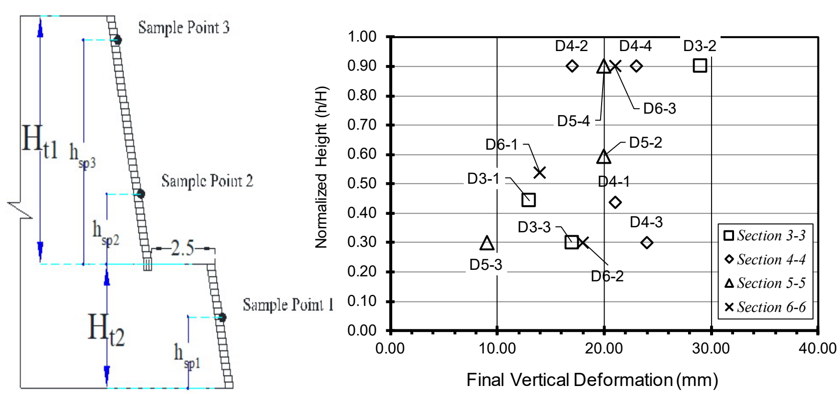

Fig. 9: Vertical deformations of the monitored points on the facing versus normalized height of each tiered wall.

3.2. Horizontal deformations

By comparing the final horizontal deformations of all surveying points in figure 10, it is obvious that the amount of horizontal deformation increases from right to left abutment. The higher values of horizontal deformation in left abutment is due to machinery work in the downstream right abutment for excavation tasks. The value of horizontal deformation varies from minimum value of 61 (mm) to maximum of 190 (mm) in left abutment. Table 3 shows the recommendations of different guidelines for prediction of horizontal facing deformations, in which the results are in proper agreement with the anticipated values from guidelines. The variation of final horizontal deformation of points located in sections with maximum height versus the normalized heights are compared in figure 11. The results show approximately same horizontal deformation in each section along the height of the wall.

Table 3: Anticipated horizontal deformations from different guidelines.

Source	Wall type	$\Delta x/H$ (Horizontal deformation to height of the wall)	Anticipated horizontal deformation for 8-meter height of each tier (mm)
FHWA (2008)	All Walls	4% to 0.9%	(4-0.9) % * 8 (m) = 72 to 320 (mm)
Bathurst et al. (1995) [11]	Segmental	1% for H = 8 m 1.5% for H > 8 m	1.5% * 8 (m) = 120 (mm)
BS8006 (1995) [10]	All Walls	0.5%	0.5% * 8 (m) = 40 (mm)
NCMA (2009)	Segmental	3.5%	3.5% * 8 (m) = 280 (mm)

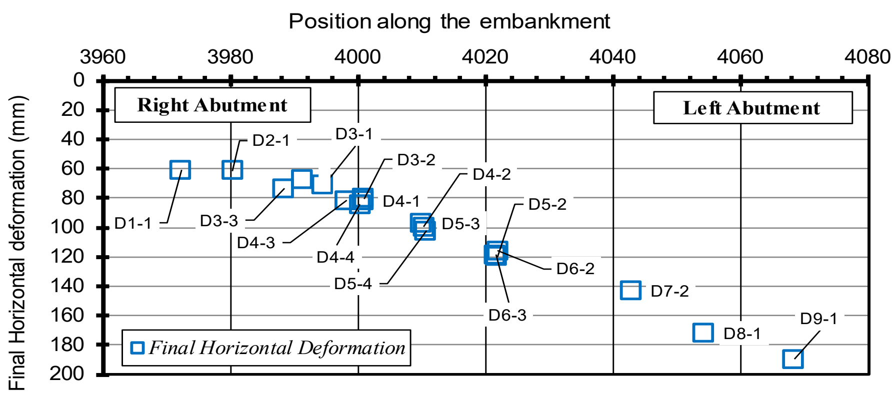

Fig. 10: Final horizontal deformations of the monitored points on the facing

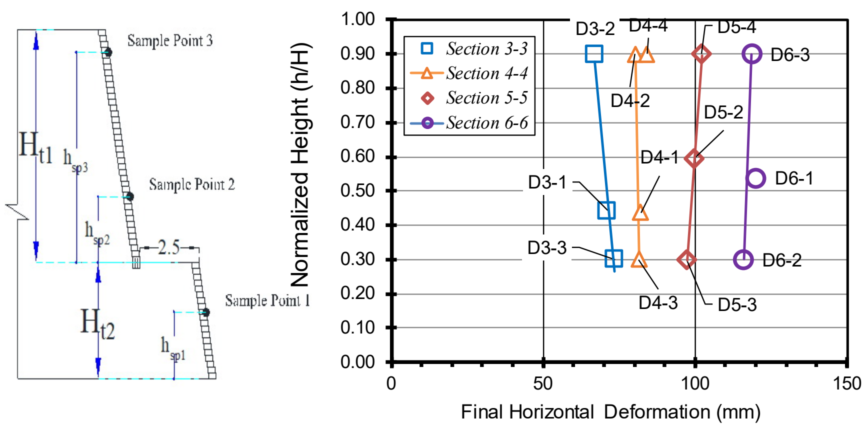

Fig. 11: Horizontal deformations of the monitored points on the facing versus normalized height of each tiered wall

4. Numerical modelling

Two-dimensional finite element analysis using GeoStudio Sigma/w software were performed to compare the results of vertical facing deformation with actual values gained in the field, and to investigate the effects of facing stiffness value on the reinforced embankment's response under working stress condition. The model was discretized with 4-noded quadrilateral and 3-noded triangle elements with 4 and 3 integration points respectively that had global element size of 1 meter with total of 5290 elements and 5540 nodes. The bottom of the foundation was fixed in both horizontal and vertical direction (X=0, Y=0) and two other sides were only fixed in horizontal direction (X=0), but the facing have free surface displacements along both x and y directions. Initial stresses of the foundation before construction, established by "Insitu" analysis and the staged construction process simulated by Load-Deformation analysis type in 60 steps, including the placement of the fill, concrete facing block and geogrid reinforcement layer. At the end of construction, the addition 3-meter height soil construction and excavation also considered in the modelling steps to simulate the surcharge of 3-meter height soil on the crest of the embankment. Compaction stresses induced during construction were not accounted for the analysis. In the final step, the surcharge loading due to weight of the concrete box is modelled with the value of disturbed load of 100 (kPa). Figure 12 shows the numerical model and mesh details.

4.1. Soil and Reinforcement Properties

Elastic-plastic model with Mohr-coulomb failure criterion was adopted for both foundation and reinforced backfill, properties of the materials are presented in Table 4. Specifying the soil stiffness modulus E as a function of the effective overburden stress for soil materials in Load-Deformation analysis in Sigma/w software is not possible, so for comparing the results for effect of the constant value of elastic modulus in linear elastic-elastic model, two values are considered for this parameter. Using elastic-plastic mohr-coulomb constitutive model for soil material to predict reinforced walls behavior have shown satisfactory results in the literature [13].

The modular concrete blocks, which is filled with drain materials are modelled as linear elastic units and the interface between the adjacent concrete blocks with shear key, were modelled using the thin layer element and the properties presented in table 4 based on the recommendations for the same blocks in the literature [14]. The block's elastic modulus was considered based on other researcher's assumed value for this type of facing system [15], [14]. The geogrid reinforcements are modelled using beam elements, which is formulated using the conventional (Bernoulli) beam theory (Hinton and Owen, 1979), but the moment of inertia value of zero and working in tension only is considered for these elements. The properties of beam element as geogrid reinforcement, with linear elastic model, are presented in table 5. Connection between the reinforcements and the surrounding soil is assumed with perfect interface adherence. Some studies showed that the assumption of perfect adherence results in an agreement between calculated values and measured results under working stress condition [14],[16]. It should be mentioned that the cross-sectional area of the structural beam elements is calculated for unit width of the wall.

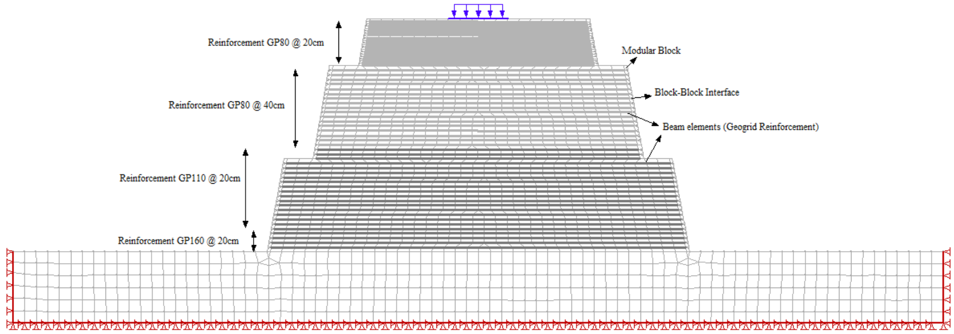

Fig. 12: The discretization, boundary condition and different parts of the numerical model

4.2. Comparison of predictions and field measurement

The vertical and horizontal facing displacements from numerical analysis and field measured values at different targets located at the same elevation on the wall face are compared in figure 13. For comparing the numerical modelling horizontal deformation with field measurements, only the survey points on the wall facing near the right abutment (Figure 6) are considered, since the results in the left abutment are affected by the previously mentioned excavation tasks after the completion of the embankment construction. Figure 13 shows the contours of horizontal and vertical deformation in model 1.

Table 4: Parameter values of backfill, block and foundation materials.

Parameters	Reinforced backfill	Foundation	Modular Blocks	Block-Block Interface
Material Category	Total stress	Total stress	Total stress	-
Material Model	Elastic-Plastic	Linear Elastic	Linear Elastic	
Young modulus, E (kPa)	50,000 (Model 1) 70,000 (Model 2)	500,000	100,000	
Poisson's ratio, ν	0.35	0.25	0.15	
Unit weight, γ (kN/m^3)	23.1	24	23	
Friction angle, φ (degree)	33	-	-	57
Dilation angle, ψ (degree)	3	-	-	0
Cohesion, c (kPa)	1	-	-	46

Table 5: Properties of reinforcement layer for linear elastic model.

Parameters	Geogrid [b]		
	GP80	GP100	GP160
Element type	Beam		
Young modulus, E (kPa)	333,000	611,000	890,000
Section Area, A (m^2) [a]	0.002	0.002	0.002
Moment of Inertia, I (m^4)	0	0	0

Notes:
[a] Geogrid thickness assumed 2 (mm)
[b] Stiffness of geogrid elements are calculated based on the assumed 2 mm thickness, 1 meter width and the stress-strain curves of tensile tests in figure 5.

The results in figure 14 shows that the measured facing horizontal displacements in the lower portion of the wall are in the order of the values from numerical modelling results. However, the computed horizontal displacements are lower than measured values in the upper portion. Data for the upper 4-meter wall have not been surveyed and the comparison with numerical results cannot be performed. The vertical deformations for lower height of second eight-meter tiered wall are in good agreement with numerical results. The vertical deformations in the second 8-meter wall are over-estimated in numerical modelling compared to the measured data, and it can be related to not considering the effect of overburden stress on soil stiffness. Beside the challenges exist in numerical modelling of geo-structures which include interaction with reinforcement elements, the results are in acceptable agreements with the measured data.

5. Conclusion

Three-tiered geogrid reinforced soil wall with modular concrete facing block were built to meet demanding project grade requirements. A complete description of the construction materials and project components is documented and explained in this paper. Based on the experiences gained during construction, using soil-rock mixtures even with low amount of oversize materials need special care and attention to ensure proper fill compaction. In addition, reducing damage to geogrid during fill material spreading and compaction is of great importance, which should be considered in design stage, and special care should be given during construction stage. Other than mentioned matter about installation damage, the installation issues about misplacing, which is inevitable during construction due to existence of oversize materials, needs special attention during construction. The steadiness and eliminating the slippage or movement of geogrids have been achieved with considering two main tasks (1) Use retaining pins with closer spacing than usual that needs more labour and (2) Use Nylon Cable Ties in overlap area of adjacent geogrid rolls.

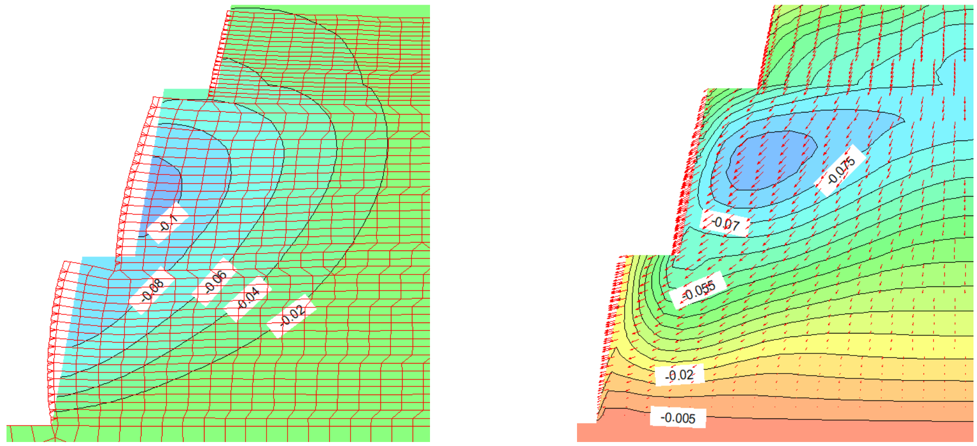

Fig. 13: Contours of (a) horizontal deformation with deformed mesh and (b) vertical deformation (cm) with displacement vectors in model 1. Deformed mesh is shown with 10 times magnification.

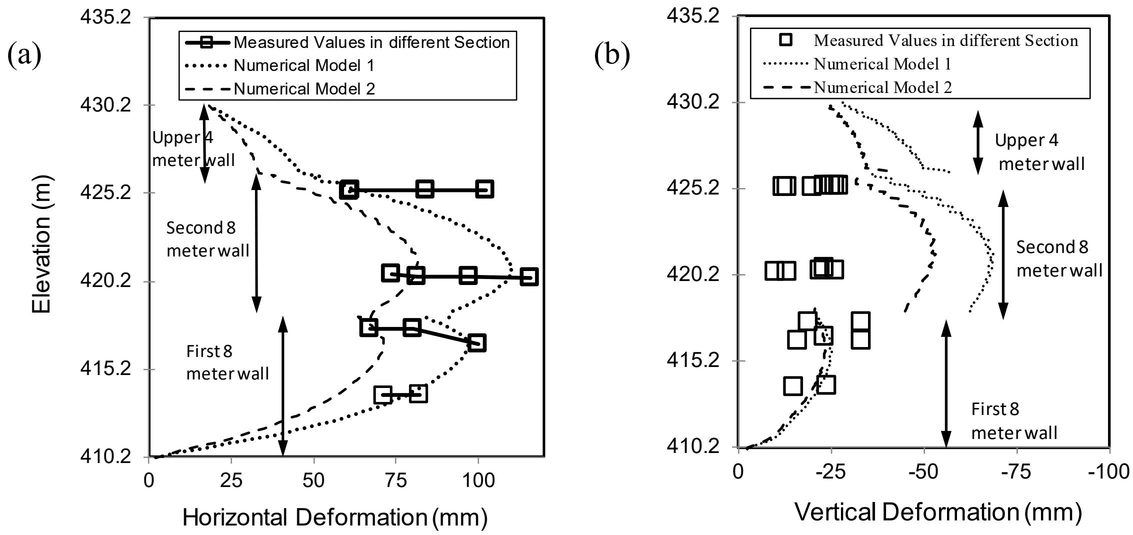

Fig. 14: Measured and calculated facing (a) horizontal deformation and (b) vertical deformation. Range of measured horizontal deformations are for sections with same height as numerical model.

A limited monitoring program have been performed and the results show that vertical deformations in sections with maximum height have higher values in the upper portion of each tiered wall ranging from 9 (mm) to 29 (mm). Total post construction horizontal movements of wall system are about 0.8% to 2.4% of the tiered wall height, which is in the order of anticipated movements from different guidelines. The results show approximately same horizontal deformation in each section along the height of the wall but increase significantly from right to left abutment. In the writer's opinion, the higher values of horizontal deformations in left side of the embankment are due to the post construction machinery works in that area. Numerical modelling technique and assumed elements for different parts of the model using Sigma/w were explained, and although predicting the displacements of mechanically stabilized wall with numerical modelling is a challenging task, but the results are in acceptable agreements with the measured data.

Acknowledgements

The presented results in this paper are part of a project conducted for Iran Water and Power Resources Development Co. as client, which is greatly appreciated.

References

[1] Tatsuoka F. (1993), Roles of Facing Rigidity in Soil Reinforcing keynote lecture in Proc. Int. Symp. on Earth Reinforcement Practice (Fukuoka Japan) vol 2 pp 831–870.

[2] Bathurst R J, Vlachopoulos N, Walters D L, Burgess P G and Allen T M. (2006), The Influence of Facing Stiffness on the Performance of Two Geosynthetic Reinforced Soil Retaining Walls Canadian Geotechnical Journal 43 1225–37.

[3] Koerner R M, Te-Yang Soong. (2001), Geosynthetic Reinforced Segmental Retaining Walls. Geotextiles and Geomembranes 19 359–386.

[4] Leshchinsky D, Jie Han. (2004), Geosynthetic Reinforced Multitiered Walls J. Geotechnical and Geoenvironmental Engineering 130 No. 12 1225–35.

[5] Berg R, Christopher B R and Samtani N C. (2009), Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes (U. S. Department of Transportation Publication Federal Highway Administration No. FHWA-NHI-10-025).

[6] Yang G Q, Liu H, Zhou Y T and Xiong B L. (2014), Post-construction Performance of a Two-Tiered Geogrid Reinforced Soil Wall Backfilled with Soil-Rock Mixture Geotextiles and Geomembranes 42 91–97.

[7] NCMA (2009), Design Manual for Segmental Retaining Walls (3rd ed.) (Herndon VA USA) National Concrete Masonry Association (ed. M. Bernardi).

[8] ASTM 2011, CD ROM annual book of ASTM standards Vol 4. American Society for Testing and Materials, West Conshohocken, Penn.

[9] Bathurst R J, Huang B and Allen, T M. (2011), Analysis of Installation Damage Tests for LRFD Calibration of Reinforced Soil Structures Geotextiles and Geomembranes 29 323–334.

[10] BS8006 1995 Code of Practice for Strengthened/Reinforced Soil and Other Fills. British Standards Institution, Milton Keynes, U.K.

[11] Bathurst R J, Simac M R and Sandri D. (1995), Lessons learned from the construction performance of a 14 m high segmental retaining wall. Proc. Geosynthetics: Lessons Learned from Failures. Nashville, Tennessee, February, 15 p.

[12] GEO-SLOPE International Ltd. (2012), Stress-Deformation Modelling with SIGMA/W, Engineering Methodology, Third Edition, GEO-SLOPE International, Ltd Calgary, Alberta, Canada.

[13] Huang B, Bathurst R J and Hatami K. (2009), Numerical Study of Reinforced Soil Segmental Walls Using Three Different Constitutive Soil Models J. Geotechnical and Geoenvironmental Engineering 135(10) 1486–98.

[14] 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 Can. Geotech. J. 42(4) 1066–85.

[15] Guler E, Hamderi M and Demirkan M M. (2007), Numerical Analysis of Reinforced Soil-Retaining Wall Structures with Cohesive and Granular Backfills Geosynthetics International 14(6) 330–345.

[16] Mirmoradi S H and Ehrlich M. (2015), Numerical Evaluation of the Behavior of GRS Walls with Segmental Block Facing under Working Stress Conditions J. Geotechnical and Geoenvironmental Engineering 1090-0241/04014109(8).