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CASE STUDY OF GEOGRID REINFORCED SEGMENTAL RETAINING STRUCTURE

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ABSTRACT

A geogrid reinforced Keystone® segmental retaining wall structure was designed for a large industrial project in Redbank, southeast Queensland, Australia. The 230m long segmental retaining structure with a maximum height of 7.2m has recently been completed. Insufficient ground bearing capacity was identified due to the presence of soft to firm clay and ground improvement was required to address this issue. The worst ground condition appeared to be underneath the highest section of the structure, as it is located within the vicinity of the creek crossing. Considering the size of the Keystone® blocks, the significant height of the structure and the unsatisfactory ground condition, excessive wall deflection and differential settlement were the key issues for the design. In addition to the conventional stress-based limit equilibrium design approach, the strain-based approach was adopted to address these design challenges. This paper describes the design and analysis methodology adopted to mitigate the long term performance risk. The results of the finite element and limit equilibrium assessments will be presented and discussed.

Keywords: Geogrid reinforced segmental wall, Ground improvement, Stress-based approach, Strain-based approach, Long term serviceability

INTRODUCTION

A new industrial development at Monash Road, Redbank, Queensland required an in total 230m long geosynthetics reinforced segmental retaining structure. The facing comprised the Keystone® modular masonry concrete units with the nominal unit dimensions of 203mm (H) x 457mm (W) x 315mm (D). The structure was reinforced by high-tenacity woven polyester geogrids at 600mm vertical spacing. The most critical section of the structure is in excess of 7.2m high and founded on soft to firm clay layer.

Geosynthetic reinforced soil structures were introduced in the 1970’s. The use of dry-stacked columns of interlocking modular concrete units as the facing for geosynthetic reinforced retaining wall structures were first appeared in the mid-1980’s, and has since increased rapidly [1], [2], [3]. McGown [4] pointed out that the geosynthetic reinforced soil structures were adopted in practice successfully first and researched and standardised later. Therefore, technically efficient and reliable design and analysis approaches have been of great concern for maintaining the confidence of the industry practitioners.

Stress-based limit equilibrium approach has been employed dominantly to date. The limitation of this approach is that it deals with all loads as pseudo-static loads and does not take the strains into account. To address this shortcoming, strain-based finite element design approach has been adopted lately. It studies the stress-strain behaviour of the structure and provides more comprehensive solutions for structures sensitive to deflection or settlement.

This paper presents the design approach and analytical results of the most critical section of a geogrid reinforced segmental retaining wall. The design considers the stability and long term performance of the structure. Stability assessment was conducted by using limit equilibrium commercial software Slope/W. Displacement of the structure were simulated by finite element commercial software Phase2. The outcomes concluded from this case study could provide a helpful reference for design and analysis of geogrid reinforced Keystone® segmental retaining structure in south-east Queensland.

DESIGN METHODOLOGY

Current Design Practices Review

The most common design and analysis methodologies of segmental retaining walls can be found in guidelines published by Federal Highway Administration (FHWA) [5]; the National Concrete Masonry Association (NCMA) [6]; and the American Association of State Highway and Transportation Officials (AASHTO) [7], [8]. All of these guidelines are based on stress-based limit equilibrium approach [9]. This approach generally treats all types of loads as pseudo-static loads and with no regard to the stress-strain behavior of various structures under different loading combinations. This can lead to insufficient design considerations on the long term performance of the structure induced by displacement.

Apart from the conventional limit equilibrium approach, FHWA guideline [5] also suggests that both total and differential settlements should be considered, however, there is no standard method to evaluate the overall displacement of reinforced soil wall.

Berg et al. [10] and Greenway et al. [11] carried out systematic studies of the design methods and assessment techniques adopted in North America for Geosynthetic Reinforced Soil Structures since the 1970's. They revealed that slight modification has been made on design codes and guidelines to date. The design of the reinforced segmental retaining wall requires analytical models and performance data unique to this system [9]. More specific and comprehensive design approach should be adopted.

Regional and Local Geology

The structure is located within close proximity to the Brisbane River, in the suburb of Redbank midway between Brisbane and Ipswich. The regional geology of the Ipswich Basin comprises sedimentary and igneous rocks of the Bundamba Group and Ipswich Coal Measures. According to previous geotechnical investigation of the area, the site was believed to be underlain by alluvial soils which in turn overlies sedimentary rocks of Ipswich Basin [12].

Additional site inspections were carried out prior to this project and Dynamic Cone Penetrometer (DCP) tests were conducted to verify the local geotechnical condition. The results indicated that the subsurface soils consist of soft to stiff alluvial clay up to 3.0m overlying stiffer alluvial soils. The worst ground condition appears underneath the highest section of the structure which is within the vicinity of a creek crossing.

Design Requirements and Material Properties

The design life of the Keystone® segmental retaining structure is 120 years in accordance with Australian Standard (AS) 4678-2002 [13], categorized as major development zone by local council. The structure was designed to support the main access road into the industrial park with high vehicle volume. The critical section for design is 7.2m high and founded on the soft to firm clay layer up to 1.0m depth overlying 3.0m depth of firm to stiff clay.

The soil properties adopted in the retaining structure were determined based on the site investigation results and authors' previous project experiences in southeast Queensland, Australia. For limit equilibrium stability assessment, the adopted soil parameters are listed in Table 1.

Table 1 Properties of the structure materials

Material Properties	Friction angle, ϕ'_d (Deg)	Unit weight, γ (kN/m^3)	Cohesion, c' (kPa)
Reinforced Fill	36	20	0
Controlled Fill	28	18	5
Crushed Rock	45	20	0
Soft to Firm Clay	26	16	2
Firm to Stiff Clay	28	18	3
Stiff to Very Stiff Clay	30	19	5

Minimum embedment depth was computed as total wall height/20 in accordance with Roads and Maritime Services (RMS) Specification R57. To prevent potential bearing failure due to the excessive vertical stress and insufficient bearing capacity of the soft clay layer, a 750mm thick layer of crushed rock fill was adopted as the ground improvement option to replace the existing soft clay layer. In addition, a 300mm concrete footing was casted beneath the Keystone® blocks as raft foundation.

Uniaxial polyester geogrids Miragrid® 8XT were adopted as reinforcement in the design. The long term design strength of the reinforcement was determined by applying partial factors for durability, installation damage and creep for a 120 year design life. The long term design strength was adopted as 49.7kN/m.

A 20kPa uniformly distributed load was adopted to simulate the traffic loading on top of the structure. Groundwater was defined in accordance with the Q100 flood level recorded. The overview of the completed wall is shown in Fig. 1.

Fig. 1 Overview of the completed wall.

Modelling and Analysis

The analysis conducted includes stress-based limit equilibrium modelling to assess the global stability of the structure, and strain-based finite element modelling to assess the displacement of the structure.

Stress-based approach – limit equilibrium analysis

Commercial software Slope/W 2012 was employed to assess the global slope stability of the critical cross sections of the structure to achieve a minimum factor of safety (FOS) ≥ 1.50. In this study, the proposed geometries with and without ground improvement layer were analyzed using the Morgenstern-Price method. The grid and radius method was adopted to define circular trail failure surfaces between specified coordinate limits and the critical failure surface with the minimum FOS was obtained [14].

Soil properties adopted in the stability assessments are listed in Table 1. Instead of adopting the conventional monolithic model for the Keystone® wall, a new constitutive model was developed based on the large scale direct pull-out test results undertaken by Bathurst, Clarabut Geotechnical Testing, Inc. in Canada. The normal stress versus shear stress relationship was modelled to simulate the shear capacity and the operational behavior of reinforcement at different wall heights. In this design, pore-water pressure was defined by piezometric line function in Slope/W based on the local Q100 flood level provided. It indicated a water table of approximately 3.0m above the ground surface at the critical wall section as shown in Fig. 2.

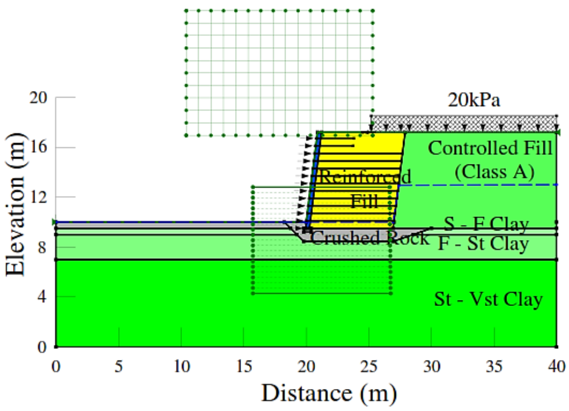

Fig. 2 Slope/W model of geometry with ground improvement.

Strain-based approach – finite element analysis

Part of the design risks of this project were the potential wall deflection and excessive settlement due to the soft clay foundation layer and the significant height of structure. Strain-based design approach was adopted to justify the design for long term performance. The commercial finite element software Phase2 was employed to assess the total, horizontal and vertical displacements.

The initial finite element analysis focused on the regulated maximum allowable settlement and wall face deflection of 50mm in accordance with AS4678-2002. However, this regulated allowance applies to all structures and does not consider the magnitude of structure height. This could lead to potential long term serviceability issue for structures with significant height. In this design, the potential cracking on the wall face induced by excessive tensile strain was a significant long term performance indicator. Bathurst and Simac [9] reported that modular concrete blocks have been observed to crack in reinforced segmental retaining wall structures. The appearance of cracking is not desirable from either aesthetic or structural point of view, especially for high structures consist of smaller concrete blocks. The differential settlement was summarized by Anderson [15] as one of the major sources causing cracking. In this design, the critical section of the structure was initially located on top of a 3.0m (W) x 1.5m (H) dimension concrete culvert which founded on soft clay area as shown in Fig. 3. This layer was then replaced by a layer of crushed rock fill as ground improvement option. The potential differential settlement between the combination of wall and culvert section versus the full height wall section was of primary concern.

Fig. 3 Overview of concrete culvert section.

Due to the lack of standard method to evaluate the serviceability problem caused by displacement, a methodology introduced by Mair et al. [16] for classifying the damage of masonry structures with prediction of ground movement and calculation of strains induced within the structure was adopted for this study. It established an important link between the estimated tensile strain and the potential damage category, which thus provided a principle for verifying the segmental block wall design from the serviceability assessment perspective.

Six categories of damage was defined by Burland et al. [17], numbered 0 to 5 in increasing severity.

Extensive case histories were analyzed by Boscardin and Cording [18], and the damage categories related to the magnitude of the tensile strain induced in the structure were summarized in Table 2. Based on this relationship, structures with induced tensile strain greater than 0.3% which classified category 4 to 5 represent severe to very severe degree of damage and require extensive or major repair. Therefore, the suggestion of 0.3% limiting tensile strain was adopted to compare with the computed differential strain from the finite element analysis to justify the serviceability of the structure.

Table 2 Relationship between category of damage and limiting tensile strain [18]

Category of damage	Normal degree of severity	Limiting tensile strain (%)
0	Negligible	0-0.05
1	Very slight	0.05-0.075
2	Slight	0.075-0.15
3	Moderate	0.15-0.3
4 to 5	Severe to very severe	>0.3

The finite element geometry was modelled with and without crushed rock fill ground improvement. The mesh was defined as a uniform pattern of approximately 1500 triangular elements with 6 nodes as shown in Fig. 4. The analysis will compute the variation of stress and strain throughout the mesh. The computed displacement outcomes were used to verify the long term serviceability of the structure.

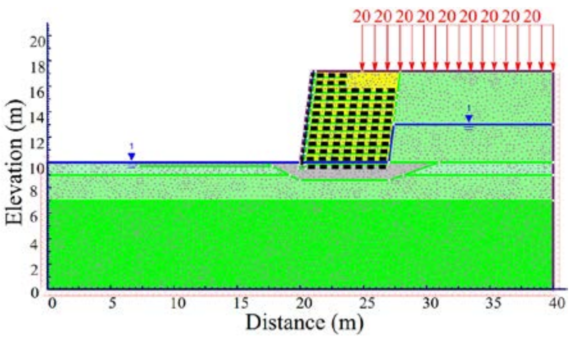

Fig. 4 Phase2 model with ground improvement.

RESULTS AND DISCUSSIONS

Effects of Soft Ground Improvement on the Slope Stability

To assess the stability of the structure in terms of FOS, the analysis introduced in previous chapter was conducted for different ground conditions. Prior to design, it was reported that the in-situ ground bearing capacity was sufficient and no ground treatment would be required. However, the ground condition underneath the highest section of the structure was later found to be on a soft clay layer within a creek vicinity and ground improvement option was required. A 750mm thick layer of crushed rock fill was adopted to replace the soft clay layer. The crushed rock fill was designed to distribute the vertical stress uniformly into stiffer soil strata below and to prevent excessive wall displacement.

The Slope/W result shown in Fig. 5 shows that the FOS of structure founded on soft ground without treatment is 1.36 thus does not meet the minimum long term requirement of FOS 1.50 by local authority [19]. The Slope/W result illustrated in Fig. 6 indicates that the crushed rock fill effectively improved the ground condition and the FOS increased to 1.51 which satisfied the long term stability FOS requirement.

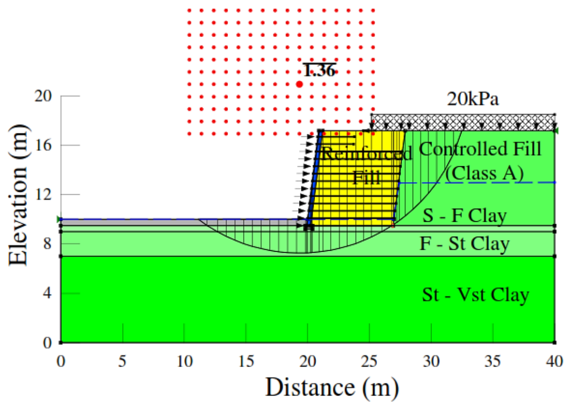

Fig. 5 FOS of geometry without ground improvement.

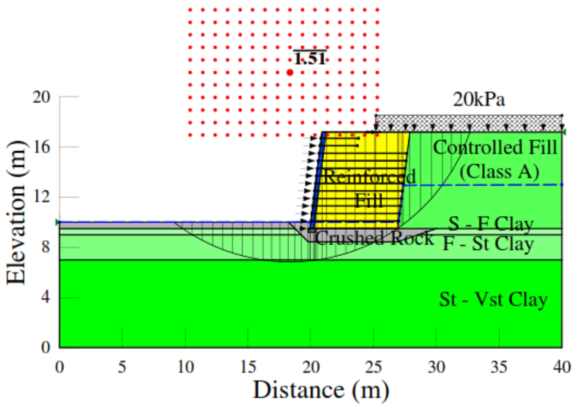

Fig. 6 FOS of geometry with ground improvement.

In order to further justify the effectiveness of the ground improvement method, the slope stability analysis of improved ground scenario were undertaken with different friction angle of the crushed rock ranging from 30 degree to 60 degree. The sensitivity check results illustrated in Fig. 7 presents the trend of the FOS increasing linearly with the friction angle of the crushed rock fill. It was noted that in order to satisfy a FOS of 1.50, the crushed rock material must achieve a minimum friction angle of 45 degree. In this project, the crushed rock material was tested and verified in the laboratory before construction commenced.

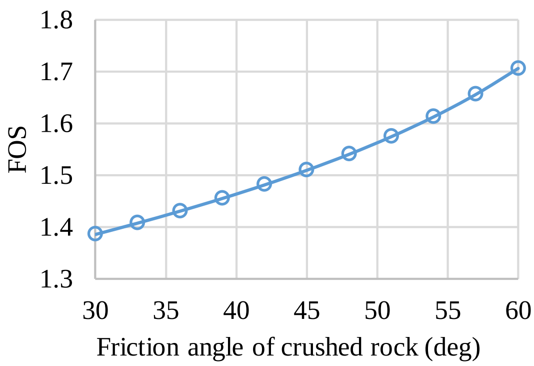

Fig. 7 FOS sensitivity check with different friction angle of crushed rock.

Design Verification Using Strain-based Approach

As the long term excessive wall deflection and differential settlement were the key issues of this design, it was justified by using the strain-based approach described in previous chapter.

The finite element analytical results computed using Phase2 were adopted to justify the requirements stated in AS4678-2002 [20] which allows a maximum 50mm displacement. The comparison of the computed maximum horizontal and total displacement on the wall face of geometries with and without crushed rock fill are demonstrated in Fig. 8 and Fig. 9. The maximum horizontal displacement decreased from 52mm to 37mm, while the maximum total displacement decreased from 56mm to 38mm. Both results indicate that the crushed rock fill layer was effective on reducing the wall face displacements.

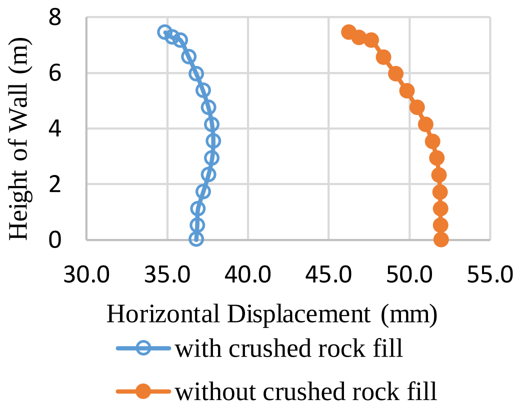

Fig. 8 Computed horizontal displacement along wall face with & without crushed rock fill.

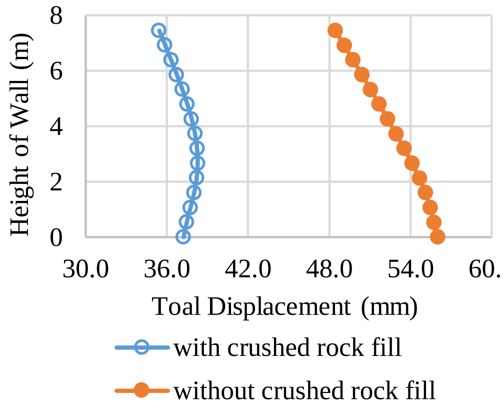

Fig. 9 Computed total displacement along wall face with & without crushed rock fill.

The strains percentage of wall displacement was then computed from the Phase2 results and checked for the serviceability requirement. The strains along wall face were calculated and the maximum value was adopted to compare with the 0.3% of limiting tensile strain described in previous chapter. The estimated maximum strain on the wall face was calculated as 0.26% based on total displacement outcome shown in Fig. 8.

The result indicates that the potential strains induced cracking on the face of the critical wall section could be classified as category 3. It corresponds to a moderate level of severity damage according to Table 2. Implication of typical damage and ease of repair and maintenance were suggested by Burland et al. [17]. This classification of damage could be conservative and the original paper suggested that there was no evidence of severe damage resulted from the tensile strains up to 0.3% by then. However, it provides a practical methodology for prediction of serviceability issue. Thus enables the design to be evaluated from the serviceability perspective which is unable to be addressed by stress-based analysis approach.

CONCLUSION

The paper has focused on the design and analysis of the critical section of a geogrid reinforced segmental retaining wall that employs the Keystone® modular concrete units as the wall facing system. The stability of the structure was investigated by the conventional stress-based limit equilibrium approach. The long term performance of the wall face subject to displacements were analysed by strain-based finite element approach coupled with a serviceability evaluation methodology for masonry structures. The following conclusions were drawn from this case study:

- The long term performance of the wall under displacement is one of the key issues for the design of Keystone® segmental wall with significant height. In order to fully address the design risks, the strain-based finite element approach for displacement analysis should be adopted in addition to the traditional stress-based limit equilibrium approach for stability assessment.

- The wall face cracking is a major performance indicator of reinforced masonry segmental retaining structure. The strain-based finite element approach could be extended by adopting a classification of damage methodology to examine the potential damage category based on the estimated tensile strains. Thus the long term serviceability could be evaluated in a more specific and comprehensive way with prediction of potential cracking damage in addition to the general displacement outcomes.

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