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Performance of a geosynthetic segmental block wall structure to support bridge abutments

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ABSTRACT: Geogrid reinforced soil structures with segmental block facing have been constructed to directly support end spans for a major bridge in NSW, Australia. Abutments are up to 10m high constructed in a terraced arrangement. The results of large scale pull-out tests of the geogrid have been incorporated into a numerical study of the structure using finite difference methods. These results are compared with actual monitored field performance.

1 INTRODUCTION

This paper describes the first large scale use of masonry faced geogrid reinforced soil structures to directly support end spans for a major bridge on the Pacific Highway 104km south of Brisbane, Australia. The bridge consists of a nine span superstructure over the Tweed River at Barney's Pt. Construction was project managed by the Roads and Traffic Authority of New South Wales during 1994. The 'Keystone' segmental block wall system was adopted as the front facing of the bridge abutments with 'Tensar' HDPE geogrids manufactured by Netlon (UK) used as soil reinforcements.

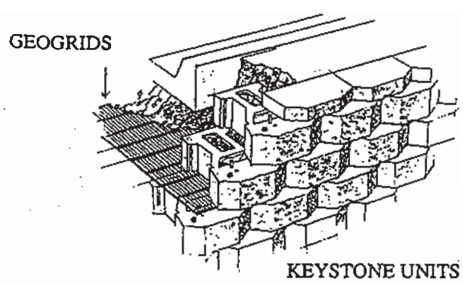

Figure 1 Internal view of segmental block wall

Foundation conditions at the site for the northern bridge abutment (referred to as Abutment B) consists of a 1 to 3m thick layer of loose silty sands containing thin discontinous silty clay layers overlying a medium dense silty sand layer varying in thickness from 7 to 10 metres. Sandstone bedrock is present at 13m depth.

2 SELECTION OF REINFORCED SOIL ABUTMENTS

At the concept design stage the use of a reinforced soil structure to form the bridge abutments was considered more cost effective than piling options. It was anticipated that the select fill would be a dredged sand containing a high salt content. Furthermore during flood periods it was estimated that the lower portion of both abutments would be inundated by salt water as the Tweed River is estuarine. Two schemes using non metallic 'Tensar' HDPE SR110 and SR80 geogrid reinforcements were considered. The first option was to use gabion rock filled baskets as a facing for the geogrid reinforced abutments designed at 1:1 (H:V) slope. The second option was to use a segmental block 'Keystone' masonry front facing in a terraced arrangement whilst keeping to the same slope profile. The segmental block system as a facing was considered more durable than gabions, offered construction advantages and was aesthetically pleasing.

3 SEGMENTAL BLOCK FACING DETAILS

The arrangement of the 'Keystone' block facing is shown in Figure 1. Individual blocks are 200mm high, 455mm wide and 315mm deep made from unreinforced concrete with nominal 20 MPa strength. Blocks are stacked and interlocked by high strength fibreglass dowels (12mm diameter) with two dowels per block. The connection strength between the geogrid and the facing is generated by friction with the block and shear resistance with the

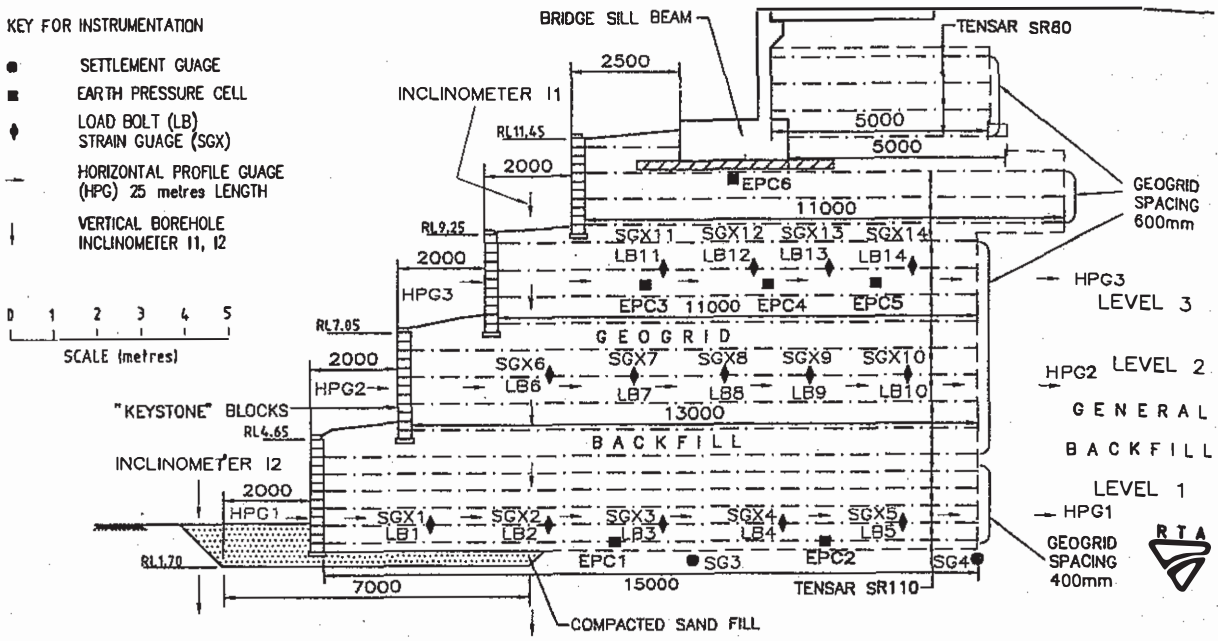

Figure 2 Geogrid reinforced soil structure showing instrumentation location

pins. The blocks are partly voided internally and 20mm aggregate is used to fill the blocks during construction.

4 GEOGRID REINFORCEMENT DETAILS.

The typical geogrid layout is shown in Figure 2. Abutment A consisted of three terraced segmental block walls with 12 layers of SR110 geogrid beneath the sill beam. Total tiered height is 6.5m. Abutment B comprised four terraced walls with 17 layers of SR110 geogrid beneath the sill beam as shown in Figure 2. Total tiered height is 9.5m. Vertical spacing of geogrid layers were 400mm or 500mm. Maximum grid lengths were 15 metres. Behind the sill beam, additional layers of geogrid 5m in length in a wrap around structure were used to reduce active earth pressures behind the sill beam. A polystyrene layer was installed at the back of the sill beam.

The sill beam was set back 2.5m from the edge of the top wall to reduce the effects of horizontal pressure due to sill beam load dispersal through the reinforced soil. The width of the sill beam was 2.5m placed on a 200mm thick transfer slab of unreinforced concrete.

In view of the loose nature of the foundation soil, the top 1m was excavated and compacted in the vicinity of the lowest wall as indicated in Figure 2.

5 DESIGN DETAILS

The 'PC SLOPE' (1992) limit equilibrium slip circle method of analysis was used initially with tensile reinforcements simulated as line loads acting at each level of reinforcement. For subsequent analysis the 'STARES' program developed by Balaam (1993) which incorporates Bishop's simplified method was used. This program could analyse a series of reinforced soil blocks to account for the terraced wall layout.

The reinforced fill zone is a fine sand with a design friction angle of $\varnothing' = 32^0$ determined from direct shear testing. Allowable long term design strengths for 'Tensar' SR80 and SR110 geogrids were 17kN/m and 27kN/m respectively to account for creep, temperature and construction damage.

Analysis assumed that the factor of safety against soil shear along the potential failure surface, at pullout and at maximum allowable geogrid strength is identical. The frictional coefficient f* (ie frictional efficiency) between the geogrid and the soil was assumed to be independent of effective vertical stress. A conservative value of f*= 0.7 was adopted based on the research work of Ochiai (1992) where pull out resistance data of geogrids in sand has been reported.

The design static and live loads induced from the bridge structure acting on the sill beam were 410 kN/m vertically with an outward horizontal force of 98kN/m. A traffic surcharge load of 20 kPa was assumed to act on the approach embankments.

Overall minimum factors of safety against rotational failure for the reinforced soil abutments were 1.50 for normal tidal conditions and 1.3 in flood conditions.

A numerical modelling study using the 'FLAC' Finite Difference program developed by Cundall (1994) was used where geogrids could be modelled as cable elements. This analysis was carried out to determine the distribution of maximum tensile forces in reinforcements and near the connection with the segmental block facing. Preliminary modelling which was subsequently refined (refer to Section 9) indicated that the maximum generated axial forces in the grid were of the order of 18kN/m in the lower layers and that tension forces on the top wall near the connection were less than 8kN/m which was within values of connection strength criteria for segmental block 'Keystone' walls proposed by Collin (1993).

6 CONSTRUCTION DETAILS

Following the installation of a course of 'Keystone' units, sand fill was spread, levelled and compacted in 200mm lifts. The maximum vibrated dry density for the sand was 1.6 t/m^3. Plate vibrators were used within the 1.5m zone behind the wall. For the rest of the reinforced fill zone the sand was flooded with water to achieve compaction to at least 95% Standard Relative Density. Vibratory rollers were used to assist compaction in the top most walls. Work commenced on Abutment A in early May 1993 and construction time was 3 months. Abutment B was completed to sill beam level in November 1993. Bridge girders were placed progressively on the sill beam at Abutment B in December 1993 to January 1994.

7 MONITORING PROGRAMME

A comprehensive monitoring program was implemented to evaluate the performance of Abutment B.

7.1 Load and strain monitoring of geogrids

Load and strain monitoring of geogrid reinforcement at three levels in Abutment B was carried out as detailed in Figure 2. For the lower and mid geogrid layers five sets of load bolts (LB 1 to 5) and strain extensometers (SGX 1 to 5) were installed. At each location, the grid was cut and joined with two 325mm wide steel clamps at the geogrid transverse ribs. The clamps were connected via a universal joint by vibrating wire load bolts to monitor tensile geogrid force. Adjacent to each load bolt location a vibrating wire strain transducer was attached to the geogrid to measure strain development over a 250mm gauge length (ie 2 ribs of geogid). The range of the strain gauges is 10%.

7.2 Vertical borehole inclinometers and horizontal profile gauges

Inclinometer I1 was placed at the toe of the lowermost 'Keystone' wall (see Figure 2) to a depth of 18m to monitor lateral ground movements under the abutment. Inclinometer I2 was installed in the reinforced soil zone emerging through in the third terrace. This inclinometer was progressively 'built up' during wall construction to monitor lateral movements in the reinforced soil block. The SINCO horizontal profile gauging system was installed to measure the internal vertical settlement of the reinforced soil zone at three levels over a 25m length.

7.3 Vertical stress and temperature monitoring

Six total earth pressure cells (EPC1 to 6) were installed to measure the distribution of vertical stress and the effect of the sill beam loading through the structure. The cells are the oil filled diaphragm type with pressure transducer readout. Thermocouples were installed to monitor temperature conditions up to 4m behind the wall face.

8 MONITORING RESULTS

The development of measured tension force in the geogrid layers at three levels is given in Figure 3 up to when the abutment was completed in mid December 1993, at June 1994 and at September 1995. It is noted that sill beam loading occurred during January 1994. Maximum monitored tensions at level 1 as indicated approach 33kN/m towards the back of the reinforced soil block. A zone of 'detensioning' is evident in level 1 reinforcement about half way along the layer where indicated geogrid tensions are 14kN/m. Maximum indicated tensions at level 2 are 21kN/m towards the back of the reinforced soil block. At level 3 reinforcement the effect of sill beam loading is evident with maximum indicated tensions of 22kN/m occuring under the sill beam region.

Monitored field geogrid strain data were less consistent than the tensile load data due to the disturbing influence of fill movement during compaction. This affected initial readings and made interpretation of strain data difficult for some gauges. Nonetheless a summary of monitored axial strains is given in Table 1.

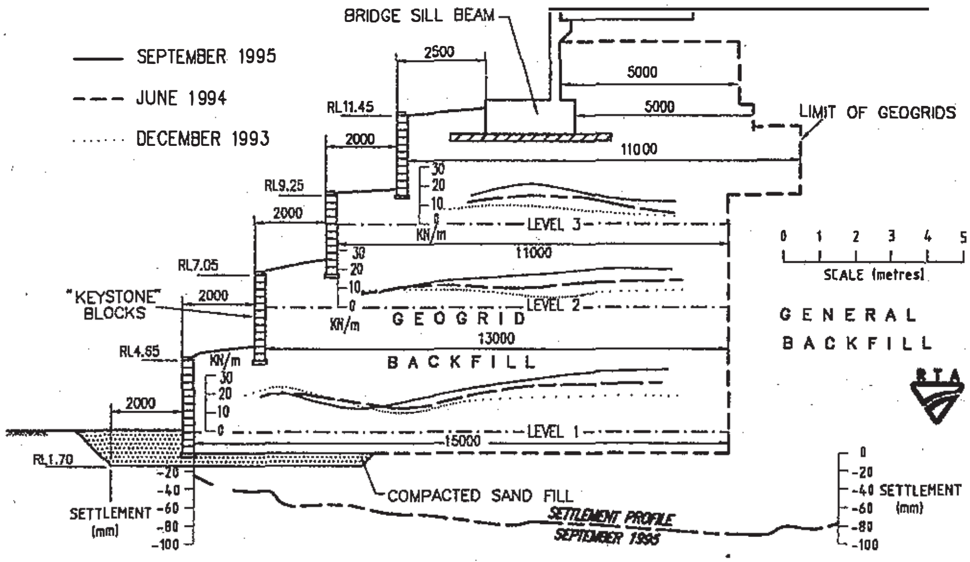

Figure 3 Distribution of tensile forces in geogrids at Abutment B

Table 1. Monitored axial strains in geogrids (as % strain)

Geogrid Level	December 1993	September 1995	Inferred Maximum Geogrid Tensions (kN/m)
Level 1	0.4 - 0.9 (SGX5) (SGX4)	0.6 - 1.6 (SGX5) (SG[illegible])	11
Level 2	0.1 - 0.4 (SGX7) (SGX9)	0.4 - 1.4 (SGX10) (SGX9)	10
Level 3	0.1 - 0.3 (SGX13)(SGX12)	0.4 - 0.5 (SGX14)(SGX13)	4

NB Strain gauge locations shown in brackets

Maximum strains of 1.6 % occur at level 1. Based on available isochronous creep curve data for 'Tensar' SR110 (at 20^0 C) supplied by the Netlon Corporation (UK) maximum inferred geogrid loads from the strain data is approximately 11kN/m which is lower than field loadbolt measurements. This difference could be due to the relative stiffness of the load clamp mechanism on the grids which may have a tendency to attract load and act as passive soil anchors.

Results of vertical settlements are plotted at the base of the structure in Figure 3 with maximum settlements of 80mm recorded. Lateral movements of the reinforced soil structure deduced from wall survey and inclinometers I1 and I2 are 10mm up to the completion of the abutment (December 1993) and 26mm post construction movements for the lowest wall. The vertical profile shape of inclinometer data suggest a rigid body lateral displacement of the abutment is occurring. Subsequent site investigations of the loose upper silty sand layer indicate the presence of thin discontinous seams of medium stiff silty clay which probably could have contributed to the deformational response at the base of the structure.

9 NUMERICAL MODELLING USING BACK ANALYSED PULL-OUT TESTS

In order to understand the actual soil structure behaviour a series of pull-out tests using the sand fill from the site was conducted at the University of Technology, Sydney. The equipment details are given elsewhere by Hausmann (1994). Geogrids 1m long were horizontally placed and compacted in a 500mm wide box with a target moisture content of 4% to achieve a dry density of 1.58 t/m^3. Pull-out displacement curves were obtained at three normal stresses of 41kPa, 78kPa and 114kPa.

Table 2 : Summary of geogrid pull-out tests

Overburden Stress σ_v	Average Shear Stress Inferred from Tests	Adopted Average Shear Stress at Failure ($\tau_{shear} = 0.56\sigma_v$)
kPa	kPa	kPa
41	22 - 24	23
78	39 - 44*	44
114	44* - 45*	64

NB * These values limited by rupture of the grid during testing

Internal displacements of the grid during pull-out were measured at 200, 400, 600 and 800mm along the embedded length. The geogrid displacements for selected load levels as a function of position along the grid for an overburden stress of 78kPa is given in Figure 4. The range of average shear stress values inferred from pull-out tests is summarised in Table 2.

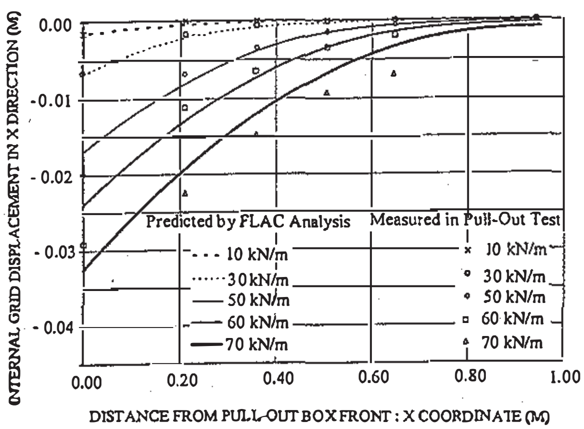

Figure 4 Analysis of pull-out tests showing measured and predicted geogrid displacements for 78 kPa overburden stress

The geogrid to soil interface stiffness(K_b) was found to vary with overburden stress from 12,500 to 50,000 kN/m per metre of deformation. This compares with the results of Ochiai(1992). For modelling, the stiffness was assumed to be proportional to overburden stress with a value of 480 kPa increase in stiffness per kPa overburden stress adopted. The 'FLAC' program was used to model the geogrid pull-out tests using the following parameters :

$E_{sand} = 50$ MPa $\quad \O_{sand} = 34^0$

$\nu_{sand} = 0.33 \quad E_{grid} = 1600$ MPa

$\gamma_{sand} = 16$ kN/m$^3 \quad$ Area$_{grid} = 6.1 * 10^{-4}$ m^2

These analyses of pull-out curves favorably matched the shape and magnitude of the displacement variation for lower load levels (10 to 40 kN/m). This range brackets the expected design loads.

A finite difference grid comprising 11,600 zones was used to model the terraced structure. A Mohr Coulomb material model was used. Grid zones included a layered stiffness of the soil strata under the geogrid reinforced abutment. Soil parameters are summarised in Table 3. The analysis was modelled in two stages : (a) Up to completion of abutment without bridge sill beam loading. (b) Abutment plus bridge sill beam and live loads.

Table 3 : Parameters for soil strata under abutment

Soil Layer	Thickness (m)	E (MPa)
Loose Sand	1.2	10
Medium Dense Sand	3.2	20
Dense Sand	6.2	30

Geogrid layers were modelled as cable elements using grid/soil (K_b) bond stiffness parameters derived from pullout testing. The shear interface parameter (S_b) for the cable element was $2\tau_{shear}$ where τ_{shear} is $0.56\sigma_v$ An initial analysis was used to determine the variation in vertical stress distribution (σ_v) to recalculate interface bond and shear stiffness parameters. The segmental block facing was modelled as beam elements with the following properties (on a metre width basis).

$I = 1.8*10^{-4}$ m^4 $\quad E = 3*10^4$ MPa $\quad$ Area = 0.6m^2

9.1 Modelling results of abutment completion without bridge sill beam loading

This modelling stage corresponds to field monitoring results in December 1993 which have been taken in relation to the start of construction in June 1993. For inclinometer I1 actual lateral movements at the toe of the abutment vary from 4 to 11mm in the loose sand layer. 'FLAC' predictions are 3mm. Maximum predicted vertical settlements of 50mm match actual settlement data as at December 1993 from horizontal profile gauges.

The predicted distribution of geogrid forces is shown in Figure 5. Maximum predicted force is 9kN/m for the lower geogrid layers occuring towards the front of the lower wall. Grid forces average 5kN for the remainder of reinforcements. These values are significantly lower than the measured field values which were 18kN/m for lower geogrids and 10kN/m on average for other layers (see Figure 3). Predicted vertical stresses at locations EPC 3 to 5 are 15-20 % greater than field values.

9.2 Modelling results of abutment with design bridge sill beam and traffic loading

This modelling stage (using ultimate design loads) nearly corresponds to field monitoring results in September 1995 except that actual traffic loading had not occurred on the structure. Additional lateral movements at inclinometer I1 at the toe of the structure are 15mm. Surveyed movements for the lower wall are of the order of 25mm laterally. This compares with modelling prediction of only 2mm

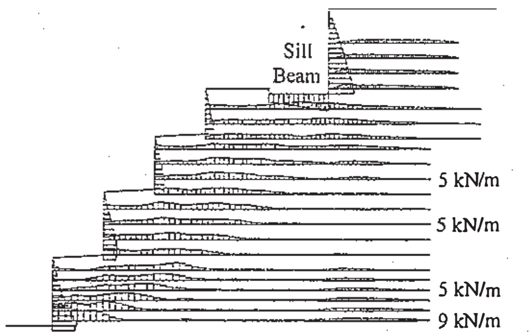

Figure 5 Predicted geogrid tensile forces for abutment gravity load case only

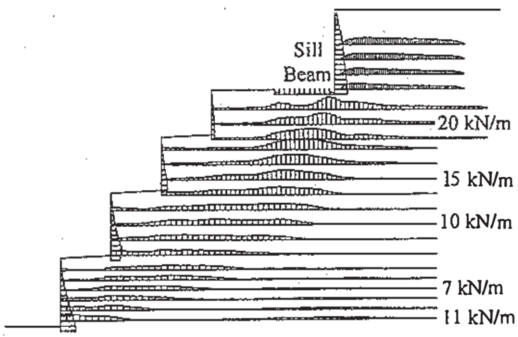

Figure 6 Predicted geogrid tensile forces for abutment gravity, sill beam and live load case

near I1. Predicted total and actual field settlements of 70 to 80mm show good agreement.

The predicted distribution of geogrid forces is shown in Figure 6. Maximum predicted forces of the order of 20 kN/m occur under the sill beam region and this compares favourably with field results. Predicted forces in lower layers are between 8 to 12 kN/m which are significantly lower than measured average forces of 20 to 25 kN/m. It is most likely that the load bolt readings have been influenced by the mismatch in stiffness between the geogrid/load bolt arrangement and the interaction of the grips with the fill especially during compaction, thus giving higher geogrid loads. The strain gauge data (see Table 1), results of 'FLAC' analysis and inferred strain from lateral movements would suggest lower geogrid forces.

10 CONCLUSIONS

The paper has summarised the application of geogrid reinforced soil structures with segmental block facing to form bridge abutments to directly support bridge sill beam loadings. Results of field monitoring are compared with numerical modelling using pull-out parameters derived from laboratory tests. The relative stiffness of load bolt clamping systems to measure developed geogrid forces require special consideration in the interpretation of field data. Geogrid strain data and numerical modelling have suggested that load bolts may overestimate geogrid tensions.

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