Geotechnical Challenges in Design and Construction of Bridge Foundations and Approaches in Hilly Granite Formation

Q. J. Yang and J. J. Dane

Abstract This paper presents a case study of geotechnical design and construction challenges of bridge foundations and approaches in a hilly granite formation in northern New South Wales, Australia. Firstly, the geological formation and existing cut slope conditions which have high risks of rock fall will be described. The original design was based on the available geotechnical information and assumed construction methodology. Reinforced concrete cantilever retaining walls founded on mass concrete were adopted for the bridge southern approach to resolve constructability issues over hilly terrain. The design considered retaining wall block sliding stability while overturning and internal stabilities were satisfied. Slope treatments using a rock fall fence together with individual boulder stabilisation or removal were also considered. It was found during construction that the actual ground conditions were different to that originally inferred and modifications to pad footing designs were deemed necessary. Additional investigations were undertaken, and the subsurface ground models updated to inform the revised design. For the northern bridge abutment foundation, a piled foundation was introduced to optimise the design with due consideration of temporary piling platform and access along a new geotextile reinforced approach embankment. The revised design was developed in close collaboration with the Contractor and the Principal. The foundation design of Pier 2 was revised using micro-piles to address the presence of a weak rock layer intrusion. In the end, key lessons learnt from this challenging project have been summarised for future project references.

Keyword Hilly granite terrain · Pad footing · Piled foundations · Retaining walls · Micropiles · Reinforced slope

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1 Introduction

The New England Highway Upgrade at Bolivia Hill is situated between Glen Innes and Tenterfield, New South Wales, Australia. The project is about 2.1 km in length and includes a bridge of approximately 320 m length crossing a steep valley. The bridge structure is an in-situ concrete box girder by balanced cantilever construction, with the main span being 150 m and the two end spans being 80 and 86 m. Arcadis was engaged by Transport for New South Wales (TfNSW, Principal) to undertake the concept design and Review of Environmental Factors (REF), and then the detailed design, in two stages.

This paper presents a review of the geological settings and the site history, the concept design developments of key elements, the geotechnical model and design parameters, the main bridge foundation design, the options considered for the approaches and the upslope rock fall risk management strategy. The geotechnical challenges encountered during the design development process and construction phase are further discussed. Key lessons learnt from this project such as the importance of constructability issue through hilly site formation are also summarised.

2 Project Background

Transport for New South Wales (formerly Roads and Maritime Services) proposed an upgrade of the New England Highway at Bolivia Hill, with primary project objectives to improve local traffic efficiency, road safety, road transport productivity, efficiency and reliability of travel. The New England Highway is commonly used by heavy vehicles, which make up approximately 25% of the total traffic volume in this area.

The Bolivia Range forms part of the Great Dividing Range in Australia and includes two main hills: Bolivia Hill at a Reduced level of 1225 m Australian Height Datum (AHD) and Little Bolivia Hill at a reduced level of 1100 m AHD. Surface elevations of the highway range from 935 m AHD at the south-eastern end to 817 m AHD at the north-eastern end of the project.

The existing road was constructed by cut and fill method in the 1950s, following the landform with curves requiring 75 km/h advisory speed. The road geometry, combined with the unforgiving steep terrain, resulted in a poor crash history compared with other sections of the highway network. Cuts were primarily formed into the upslope on the eastern side of the road with embankment on the western side of the road. The existing cuttings were typically 2–8 m high and battered at 30°–70° to the horizontal. The existing embankments ranged between 10 and 30 m in height at a batter gradient of 30°–55°. The exposed cut surface comprised granite bedrock of varying strength with little soil cover over bedrock. At the crest of the hill there were either isolated boulders or piles of boulders from construction of the adjacent disused railway some 100 years ago. The existing slopes were known for frequent rock fall and assessed to be medium to high risk.

3 Detailed Design Outcome of Key Elements

The design development process went through environmental assessment, concept design development and option engineering stages prior to detail design development. The approved detail design adopted a balanced cantilever bridge with a central span of 150 m to provide a straight alignment across a side valley.

3.1 Main Bridge and Abutments

An in-situ concrete box girder bridge by balanced cantilever construction was selected after due consideration of environmental constraints and constructability issues. The bridge is 316 m long and 12 m wide, as shown on Fig. 1, based on Arcadis Australia Pacific (AAP) reports [2].

The span arrangement and foundation solutions were heavily influenced by the steep terrain and granite landform, and the presence of Bolivia Wattle which created a construction exclusion zone. Access to the piers required significant temporary works, including large scale temporary retaining walls and rock bolt retention systems for upslope cuttings at the piers. Due to the access constraints for piling rigs and the very hard granite material expected, the originally proposed piled foundations were changed to pad foundations. This method of construction allowed the access tracks to be reduced in scale (although sections of the temporary walls were still up to 9 m high) as the working areas at the piers for cranes and concrete pumps could be set several metres below the underside of pad, whereas piling rigs would need access tracks to extend up to underside of pile cap level and be constructed at a lower gradient. Indicative access track designs including temporary retaining walls were

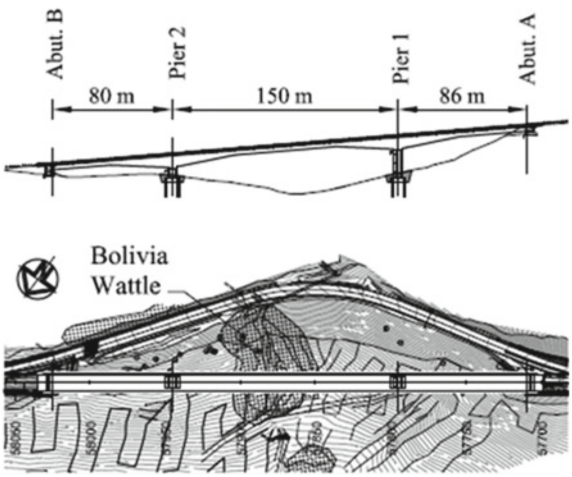


Fig. 1 Layout plan of proposed bridge and abutments

provided to the tenderers to ensure this significant element of the construction was adequately considered in their tender.

Figure 2 shows the 3D view of the proposed bridge close to completion of construction with respect to the existing road. The redundant section of highway is being retained for use as an access road for bridge maintenance.

The pad size for Pier 1 and Pier 2 presented in the final design, as shown in [19], is 10 m along the traffic direction by 8 m in the transverse direction, as shown on Fig. 3. Four rows of five temporary ground anchors were arranged at both ends of the pad to deal with the large overturning moments from out-of-balance load cases during the cantilever deck construction.

Fig. 2 Aerial view of the bridge near completion

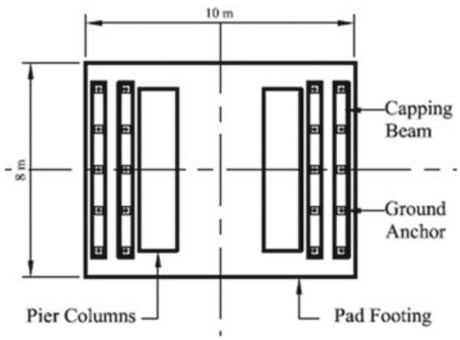


Fig. 3 Layout plan of pad footing for Pier 1 and Pier 2

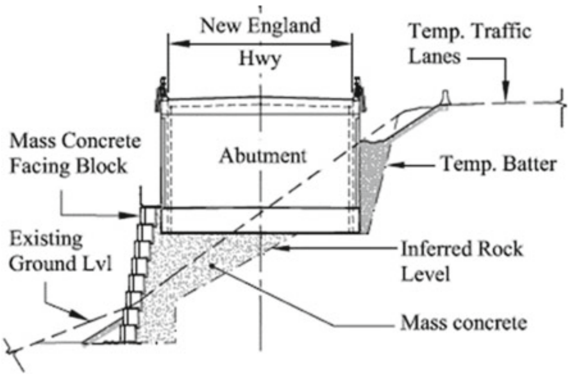


Fig. 4 Typical elevation of abutment retaining wall

The dimensions of Abutment A and Abutment B in plan in final design as presented by [19] are 7.26 m long and 11.8 m wide, with approach slab of 6 m length connecting to the abutments at both ends.

3.2 Northern and Southern Approaches

The permanent works design included two retaining structures, as shown on Figs. 4 and 5 [19], for the provision of the required road widening approaching Abutment A (southern end of bridge) and Abutment B (northern end of bridge). These were to be constructed adjacent to the existing road to allow for sufficient access to construct the bridge abutment structure, and to support the approach embankment fill (the road widening works) respectively. Part of the walls were to be formed in cuttings and the remaining are to be built over the existing ground by filling. A simple and flexible construction methodology was proposed for the wall and abutment foundations, using mass concrete with precast blocks as permanent formwork, to allow for some variance in foundation material level along the length of the structure.

3.3 Slope Stabilisation Works

The southern section of the alignment has been subject to “rock fall” risks for a long time; this site has been used for training for TfNSW (formerly RMS) slope risk assessment field works. TfNSW had previously undertaken slope assessment of the existing cuts and upslope, with some sections having an Assessed Risk Level (ARL) of 2 as reported by Roads and Maritime Services [16]. Subsequently TfNSW decided to carry out emergency remedial work prior to this upgrade project. The scope of works primarily included removal of unstable isolated boulders, stabilisation of rock

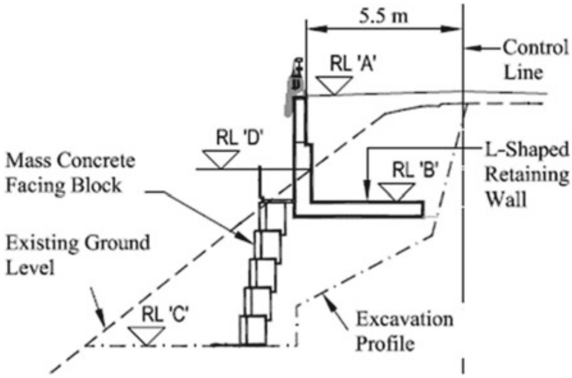


Fig. 5 Typical elevation of approach retaining wall

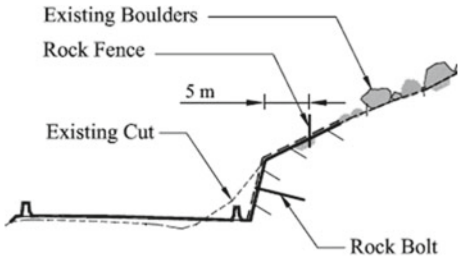


Fig. 6 Rock fence with new cut and upslope

boulders, and the "rock stockpile" formed during construction of the adjacent disused railway over 100 years ago.

A rock fence near the crest of the new cutting for the realigned southern approach was provided as part of the detailed design development. Rock fall modelling analysis showed that the proposed 35 kJ rock fall barrier located 5 m above the crest of the batter would be effective in preventing individual boulders less than 0.8 m diameter dislodged mid-backslope from reaching the carriageway. Furthermore, the design provided for field-mapped potentially unstable boulders to be stabilised using rock pins and grout. Figure 6 shows a typical cross-section of the rock fence in relation to the new cutting and the upslope profile and boulders.

3.4 New Cut Batter Design

There were several new cuttings proposed along the route of the scheme, primarily formed in rock materials of varying strength and defect spacing. Cuttings in soils were designed to have a batter gradient of 2 horizontal to 1 vertical for long-term stability and 1.5 horizontal to 1 vertical for short-term stability. The new cuttings in competent granite were to be trimmed to 1 horizontal to 4 vertical gradients.

The excavatability of the rock was assessed using the method by Pettifer and Fooker [15] and the results indicated that blasting would potentially be required for some portions of cutting through strong and massive granite at southern abutment and its approach retaining wall.

Assessment of the new cut batter slopes based on the defect information in the geotechnical investigation reports by Douglas [8, 9] indicated there were three possible failure mechanisms during the excavation phase:

- Localized slumping and shallow rotational movement: This would likely be induced by excavation and de-vegetation undermining within the highly weathered zones along the cutting.
- Boulder rolls: This would likely be caused by dislodgement of isolated rock block within the exposed cut face due to stress relief during and post excavation works.
- Block sliding and/or rotation of rock blocks: This would likely be due to the presence of unfavourable joints within the rock mass, with the block size dependent upon the joint set pattern where the cut face daylighted.

The contract documents provided for a qualified geotechnical engineer or engineering geologist to map the exposed rock cut face during excavation and confirm the need of any stabilisation measures on site. It was anticipated that minor scaling, rock bolting and shotcrete application may be required for highly weathered zones to stabilise the localised unstable rock mass where encountered.

As part of the design, the cuttings in soil were proposed to be grassed so that surface erosion can be controlled in the short and long term. The rock cuttings would be exposed and subject to long-term weathering, and therefore it was considered necessary to allow for clearing of the loose material from the slope and at the toe of the rock cutting. To ensure minimal maintenance in the long term a soft facing solution, such as mesh facing, was included in the suite of proposed treatments. This will minimize the potential risk of any rock falling onto the carriageway and affords the opportunity to carry out remedial works if deemed necessary.

4 Construction Stage Challenges

The project construction contract was awarded to Georgiou and SRG JV (GeorgiouSRG). Geoinventions was engaged by GeorgiouSRG to provide site investigation, geological assessment, and temporary work design services. Construction works started in May 2018 and completed in September 2021. Arcadis was retained to provide the construction stage services to TfNSW during the construction period.

4.1 Slope Stabilisation Works

The existing rock cut slope has experienced deterioration since construction in the 1950s and rock fall has occurred. It was considered necessary to carry out additional rock stabilisation works prior to major construction activities for the proposed new project works to ensure safety to the road user and construction crew. The design included a rock fall fence together with a schedule of potential unstable rock blocks that had to be stabilised. The contract documentation was set out in such a way that the contractor would be fully responsible for the safety of the road users and construction crew. Noting the general "precision" of the RMS Slope Risk Analysis procedure is such that in 80% of assessments two trained reviewers can independently generate ARL levels within plus/minus one ARL level, as reported by Baynes et al. [12], the design documentation allowed for varying quantities of a range of stabilization solutions.

After awarding the contract the contractor engaged an independent geologist to carry out an assessment of the rock fall risks to both road users and the construction crew down slope. It was apparent that the attitude to the uncertainties in risk rating to the road user and construction workers were different. The Contractor's responsibilities under WHS legislation drove a robust approach to worker safety (considering what is reasonably practicable to achieve) whereas the basis of the RMS slope risk assessment guidelines is safety of road users who are exposed to risk for merely seconds as they pass through the hazardous zone. Therefore, a greater emphasis was placed on safety for construction crew down slope, leading to increased upslope treatment measures including using rock fall mesh. Figure 7 shows the removal process of a potentially unstable rock block above the road as part of upslope stabilisation works.

4.2 Pier 1 Subgrade

The originally inferred founding material at Pier 1 was Class III granite based on the rock strength, defect spacing as proposed by Irfan and Powell [14] at the anticipated level of RL 849 m. There were two boreholes (BH04 and BH05) drilled close to Pier 1 footprint during the design phase, with borehole BH04 close to the northern end of the footing, as shown in Fig. 8, and the other (BH05) is approximately 10 m to the north. Therefore, as specified in the contract, two additional boreholes were required as a minimum but five were drilled by coring technique when access to Pier 1 was gained during construction. It was inferred from boreholes BH04, P1-B1, P1-B3, P1-B5, P1-B7 and P1-B9 that most of the rock mass beneath the pad footing satisfied the original design intent except for the northwest corner, as shown in Fig. 8.

The exposed founding material indicated that a localised shallow "remove and replace" by lean mix concrete up to 1.5 m deep was required to ensure uniformity of

Fig. 7 Removal process of a potentially unstable rock block (after TfNSW website)

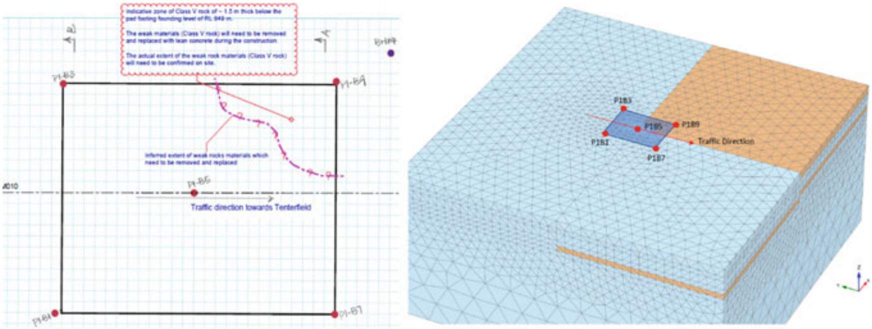


Fig. 8 Identified weak rock zone and additional borehole locations at Pier 1

the founding subgrade for the pad footing. Plaxis 3D modelling, as shown in Fig. 8, indicated differential settlement remained within acceptable limits.

4.3 Pier 2 Redesign

4.3.1 Ground Conditions

Two cored boreholes, BH06 and BH07 as shown in Fig. 9, were available for the detail design of the 10 m × 8 m pad footing. Three additional boreholes were drilled at Pier 2 by the contractor when access was available, as per the specification. The additional investigations indicated the presence of very low strength rhyodacite beneath part of the pad footing towards the upslope (south-east). Ten additional cored boreholes were subsequently undertaken by Geoinventions Consulting Services (GCS) [11, 12] to delineate the boundary of granite and rhyodacite, which is intruded in-between the granite formation as shown in Fig. 9. Figure 10 presents an inferred geological cross-section A-A of Fig. 9, showing the varying rock types and quality. Rhyodacite was interpreted to be very low strength with depths varying from 5 to 9 m. Figure 11, based on GCS report [11, 12], shows the cored log photographs of two distinctly different rock mass qualities (P2-B11 for granite and P2-B4 for rhyodacite) beneath the pad footing, with the redline mark showing the pad footing level of RL 849 m.

The concern for the performance of the bridge structure was serviceability, particularly during construction: the elastic response of the foundation would not be uniform across the pad potentially affecting the alignment of the 80 m cantilevers.

4.3.2 Option Development

Three initial options were investigated, including (1) Excavate and replace with concrete; (2) Trenching and replacement by concrete; and (3) Micropile ground improvement. Option 1 was not pursued due to the required depth of excavation in hard rock (as well as weaker material) and additional work/rework of temporary retaining system on the upslope cut. Option 2 had constructability difficulties due to limited working area for heavy excavators. Option 3 was preferred based on the site access and constructability. However, the micropile application was not typical for such a significant bridge structure in NSW. Therefore, there was a process to provide technical justification to TfNSW together with proof engineering of the alternative design foundation system before approval was given to proceed.

This option required excavating down to the design foundation level of RL 849 m and installing micropiles to form a partly piled raft footing with a more uniform foundation response. The proposed micropiles were to be approximately 200–300 mm in diameter, with the diameter adjusted to suit the equipment that the contractor has planned for the project. The micropiles would be grout piles and would typically

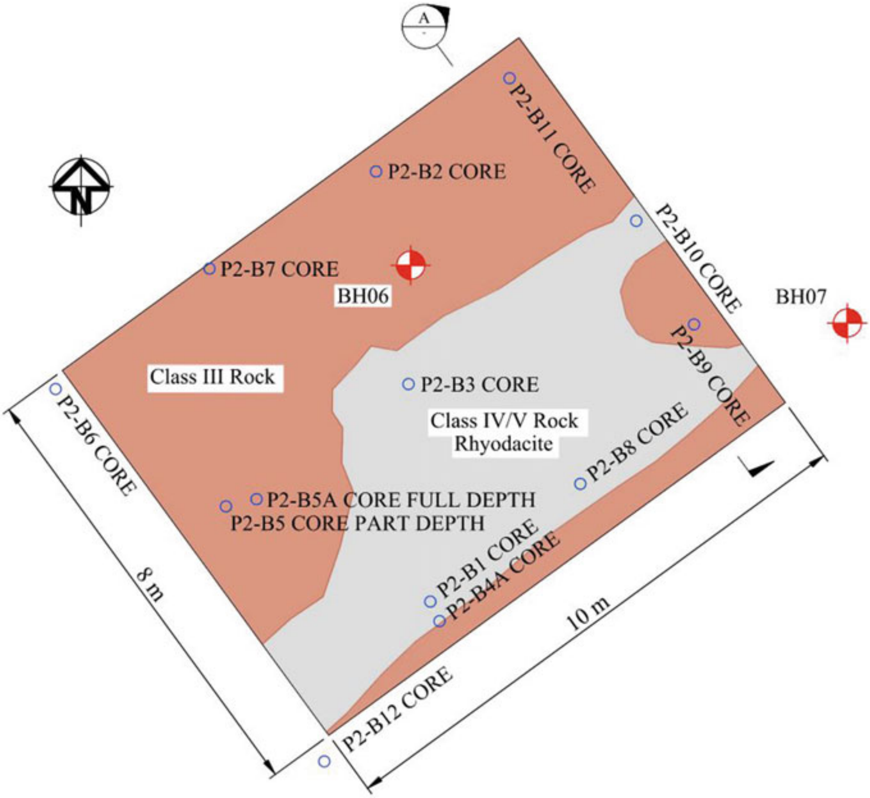


Fig. 9 The extent of rhyodacite and locations of drilled boreholes (after GCS [11, 12] information)

require at least one reinforcement bar through the middle of the pile. The adoption of micropiles required further analysis of the bridge structure to confirm its performance.

This option would not require any additional excavation compared to the other options. The micropiles were only required in the areas where very low strength Rhyodacite was found, and therefore the number required was confirmed by inspection when excavation reached the design founding level.

4.3.3 Rock Mass Classification and Geotechnical Design Parameters

The rock classification and the geotechnical design parameters were based on the original geotechnical interpretative Report. The rock mass classification was generally based on the method by Irfan and Powell [14] and the specific UCS values and defect spacing for each class of rock is presented in Table 1.

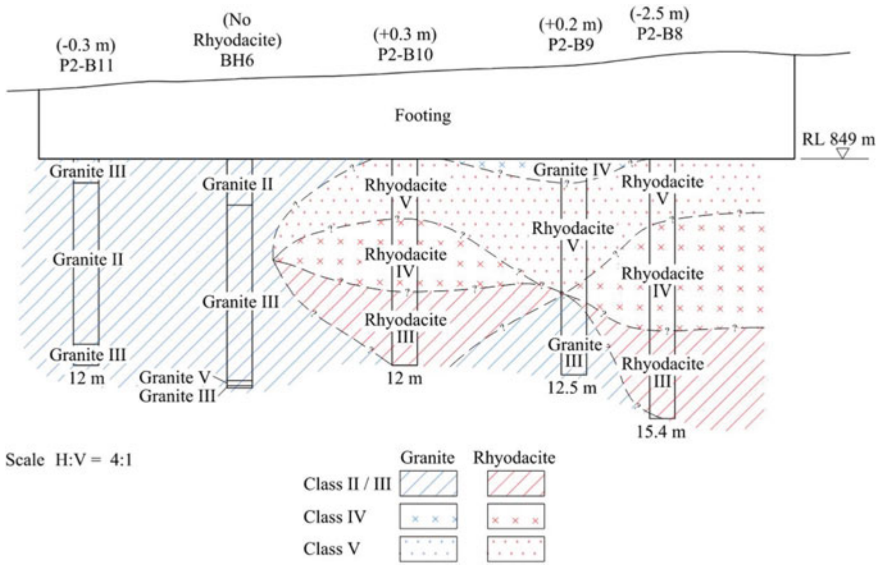


Fig. 10 Inferred geological cross-section A-A (after Geo-inventions Consulting Services [11, 12])

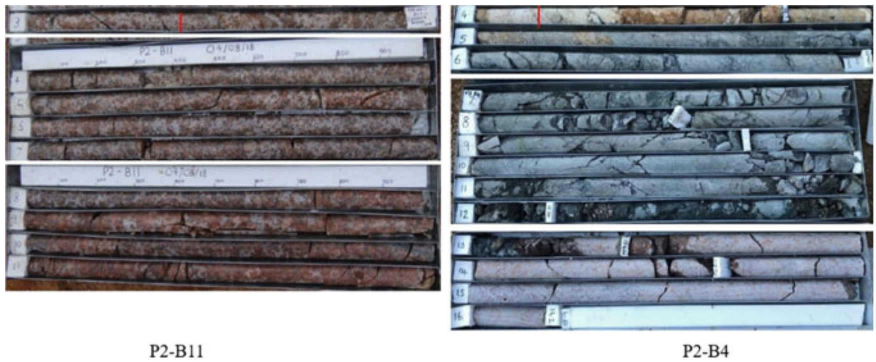


Fig. 11 Cored borehole photos of granite (P2-B11) and rhyodacite (P2-B4)

Table 1 Classification of granite and other rocks

Rock class	Point load strength, Is(50), (MPa)	Unconfined compression strength (MPa)*	Defect spacing (mm)
V	<0.3	<6	<60
IV	0.3–1	6–20	<60
III	1–3	20–60	>60
II	3–6	60–120	>200
I	>6	>120	>200

*UCS was taken to be 20 times $Is_{(50)}$

Table 2 presents the geotechnical design parameters adopted for the detail design development and the re-design for the pad footing and micropiles.

Table 2 Geotechnical design parameters

Unit	ϒ	c'	Φ'	k_0	f_{su}	f_{br}	E'	f_{all}	μ
1(a)- Fill	18	0	30	0.5	N/A	N/A	28	_	0.3
1(b)- Alluvium	20	0	33	0.45	0.03	N/A	40	_	0.3
Class V Rock	23	0	38	0.8	0.2	3	150	1	0.25
Class IV Rock	24	5	38	1	0.9	9	500	3	0.25
Class III Rock	25	140	38	1	2.0	24	900	6	0.2
Class II Rock	26	540	38	1	3.0	32	1600	12	0.2
Class I Rock	27	4000	38	1	4.0	50	2190	24	0.2

Definitions ϒ (kN/m^3) = bulk density; c' (kPa) = effective cohesion, Φ' (Deg) = effective friction angle, k_0 = in-situ stress ratio, f_{su} (MPa) = ultimate shaft adhesion for bored pile and passive ground anchors, f_{bu} (MPa) = ultimate end bearing for bored pile, E' (MPa) = Young's modulus, f_{all} (MPa) = allowable end bearing pressure, μ = poisons ratio

4.3.4 Micropile Detail Design Considerations

The Limit State structural capacities of the micropile for both the short term (Construction) and long term (Operational) were in accordance with Australian Standard AS5100 [5].

Construction and detailing of the micropiles was in accordance with the requirements of Project Specification B114 and applicable recommendations of the FHWA NHI-05-039 December 2005 Micropile Design and Construction—Reference Manual [10]. Static Acceptance Load testing of the micropiles was to be undertaken in accordance with FHWA NHI-05-039 with reference to AS2159 [4] where necessary.

The particular aspects considered in the micro-pile design included the following:

- Durability—use of a steel CHS 168 OD (outer diameter) casing placed over the length of the micro pile between the underside of the pad foundation and the Class III rock. An additional 0.02 mm/year corrosion allowance was accounted for in the assessment of structural capacity in the operational load cases;
- Settlements and rotations—The maximum rotation and settlement of the foundation was kept within the limit used in the conforming design of the bridge including substructure and superstructure;
- Slope stability—The global stability of the slope is not reduced by the improvement works;
- Structural section capacity—The axial and flexural capacities of the micropile in both the construction and operational phases have been provided;

- Casing availability—A 168 × 11 mm thick casing was procured to ASTM A53M [3] specification and this was adopted in the structural analysis;
- Construction tolerances—The micropiles were designed to resist induced bending moments resulting from verticality tolerances of 1% in accordance with AS2159. The addition of the steel casing has been considered in the structural capacity of the pile in both bending and axial compression. Verticality tolerance of up to 4% was also assessed as a sensitivity case; and
- Testing of piles for tension and compression.

A total of 29 micropiles, as shown in Fig. 12, were considered for the identified "weak" zone of Class V or Class IV rock inferred from the available cored borehole data.

The micro piles were taken to form an overall ground improvement and stiffening of the Class IV/V layer allowing the majority of loads to be transferred to the more competent Class III layers below. Structurally the micropiles were assumed to transfer these bearing loads in axial compression. An allowance for vertical tolerance of 1/100 has been included in the design in accordance with AS2159, however, up to 4/100 has been assessed to demonstrate the impact of the surrounding rock confinement on limiting induced moments.

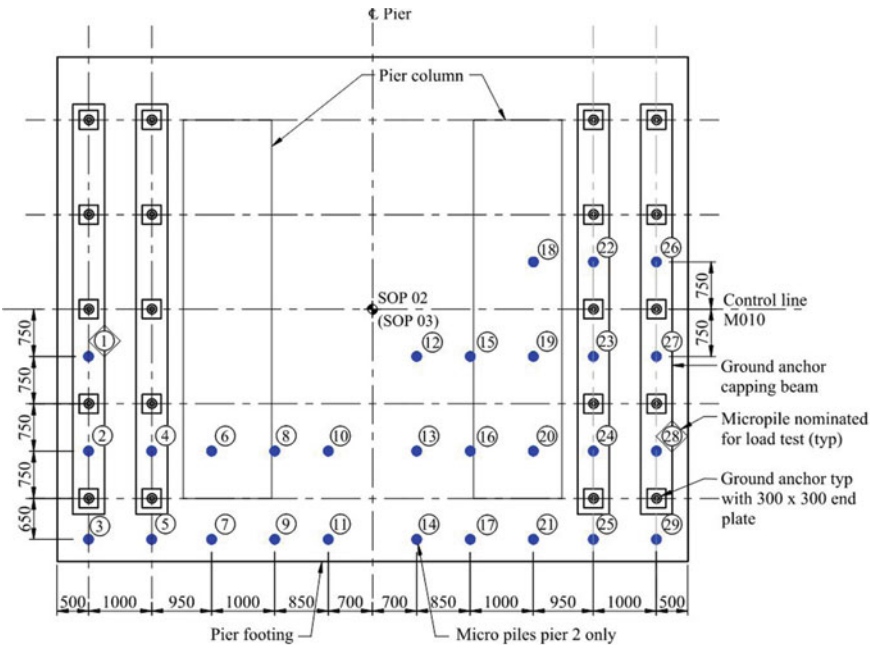


Fig. 12 Layout plan of 29 micropiles of varying lengths at Pier 2

4.3.5 Plaxis 3D Modelling and Results

Three-dimensional finite element analysis software, Plaxis 3D version 2017, was used to simulate the foundation behaviour to assess the performance of Pier 2 for both SLS and ULS loading conditions. Soil and rock strata were modelled as elastic–plastic materials obeying the Mohr Coulomb failure criterion. Micropile and footing were modelled as linear-elastic material, with the grout Young's modulus of 35 GPa and 30 GPa, respectively. The adopted material parameters for soil and rock strata were based on those shown in Table 2.

The model developed using Plaxis 3D is shown in Fig. 13. Three most critical cases were analysed using Plaxis 3D: (i) Case A—Critical construction-SLS, (ii) Case B—Critical construction-ULS, and (iii) Case C—Service earthquake.

The vertical, horizontal and equivalent bending moment in both longitudinal and transverse directions at the centre of the pad footing are presented in Table 3.

The results of the Plaxis 3D modelling for these three cases are summarised in Table 4, with the axial forces for Case B presented in Fig. 14. Note that the values shown at the bottom of each pile are loads at the tip of pile.

The calculated settlements and rotations induced on the foundation have been assessed and can be accommodated by the bridge structure both during the temporary construction and permanent operational states.

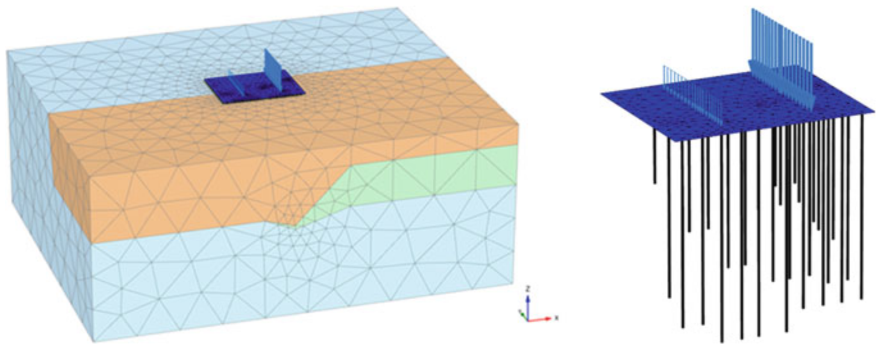


Fig. 13 Plaxis 3D model at Pier 2 with 29 micropiles

Table 3 Summary of loads acting on Pier 2

Cases	P (MN)	ML (MNm)	MT (MNm)	VT (MN)	VL (MN)
A: Critical construction-SLS	86.8	140.0	1.5	0.5	0.5
B: Critical construction-ULS	91.0	277.0	2.0	1.0	1.0
C: Service EQ	72.0	65.0	55.0	5.0	2.0

Note P = Vertical load at the centre of pad; ML = Bending moment in longitudinal direction; MT = Bending moment in transverse direction, VT = shear in transverse direction; VL = shear in longitudinal direction

Table 4 Summary of Plaxis 3D modelling results

Cases	N (kN)	Q (kN)	BM (kN)	S (mm)*	RT-L	RT-T
A	1650	166.1	19.4	2.5 to 16.5	1 in 715	1 in 2285
B	2103	215.1	28.3	–4.5–28	1 in 310	1 in 1330
C	1700	180.5	26.9	–1.5–16.5	1 in 650	1 in 840

Note *N = Maximum axial load of a single micropile; Q = Maximum resultant shear force of a single micropile; BM = Maximum resultant bending moment of a single micropile S = Calculated settlement range, *–ve means upwards; +ve downwards; RT-L = Rotation in traffic direction; and RT-T = Rotation in transvers direction

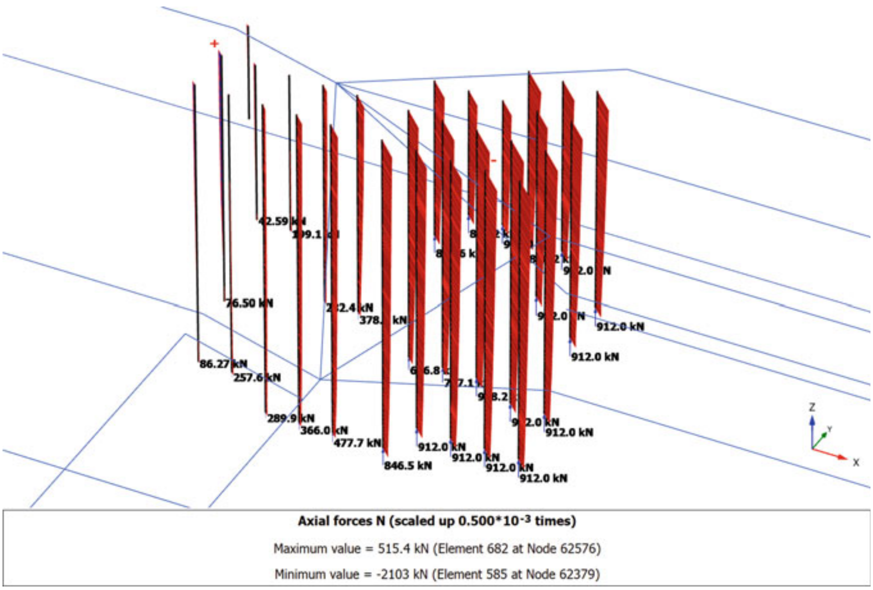


Fig. 14 Axial loads in micropiles from Plaxis 3D model—Case B

Pile geotechnical capacity assessment in accordance with [1] LRFD indicated that approximately 4 m of socket into Class III rock was required (when ignoring the casing to rock bond over the plunge length). For each location/area the pile length was adjusted to achieve the required geotechnical and structural capacity. In general, the length of the piles will be varying between approximately 5 and 11.5 m.

4.3.6 Structural Design of Micropiles

A typical micropile detail is presented in Fig. 15. An axial compression load of 2435 kN was designed for each micropile of 225 mm diameter, reinforced with a 168.3 ×

11 steel CHS section to ASTM A53M and centrally placed 75 mm diameter high-tensile steel bar. The micropile was infilled with grout of 65 MPa. As requested by RMS Bridge Branch each micropile was anchored within the footing and the CHS section was extended a minimum 1000 mm socket into Class III rock. The high-tensile bar was extended from the toe of the socket into the pad foundation and was terminated within the pad foundation with a cast-in anchor plate detail to provide full development of the bar.

The combination of CHS and high-tensile bar reinforcement contributed to the micropile capacity in axial compression. The high-tensile bar alone was considered in shear and tension capacity. The CHS section enhanced the capacity to cater for induced bending moments resulting from the applied loads and vertical tolerances in accordance with AS2159.

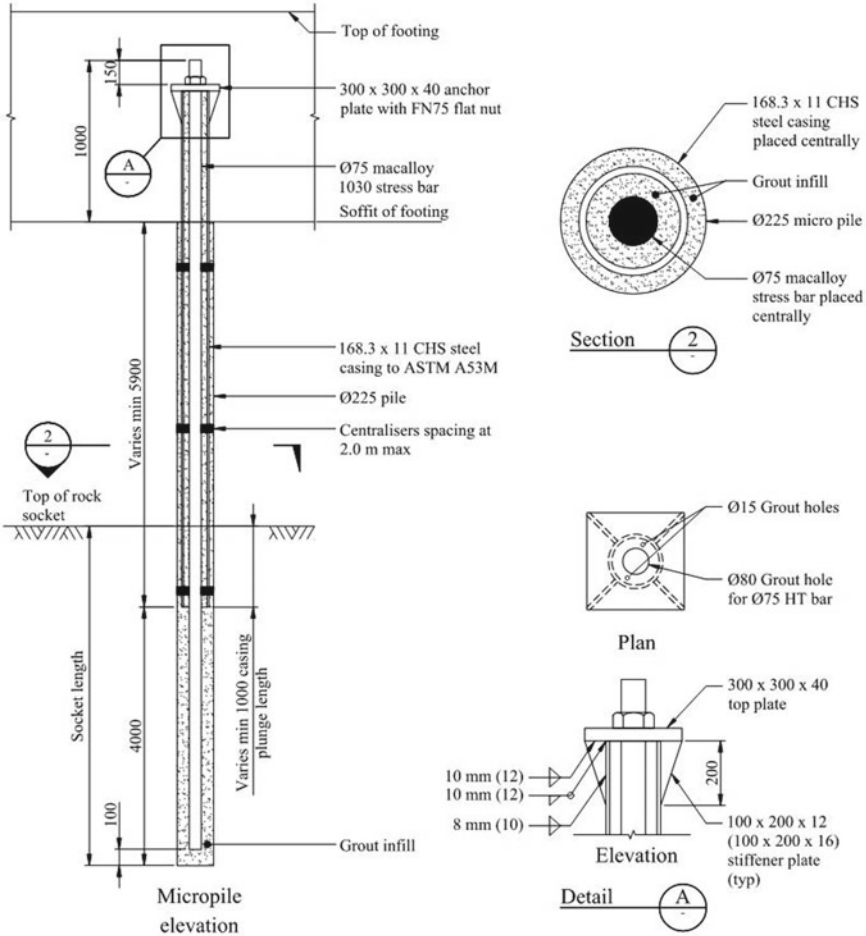


Fig. 15 Typical details of a micropile

The full CHS section was utilised for the critical short-term construction load cases. A corrosion allowance of 0.02 mm/year for the CHS as per AS2159 Table 6.5.3 (mild exposure) was considered in determining the capacity for longer term loading though limited information for the site indicating non-aggressive conditions. The high-tensile bar is adequately protected by the steel CHS and surrounding grout/concrete cover.

4.3.7 Construction Stage Verification

When the excavation of pad footing was carried out down to RL 849 m the exposed rock surface was assessed by an engineering geologist prior to pour of concrete. Figure 16 shows the approximate extent of mapped classes of rock mass at the founding level beneath the pad footing at Pier 2.

Fig. 16 Exposed rock at elevation of RL 849 m at Pier 2

4.4 Abutment B and Approach Retaining Wall

Abutment A and approach retaining wall were constructed as the original design, with some expected variation of the ground profile and support for the retaining wall.

Abutment B and approaches were originally designed using the same precast blocks and concrete infill for the lower part of retaining wall. However, the substrata profile for was found to vary more than was anticipated from the design investigations, and so after an optioneering and constructability exercise with the Principal and Contractor, the design was changed to a reinforced embankment and piled abutment foundation with the following features:

- Reduction in excavation of the existing road embankment and around abutment B; generally limited to removal of loose material and boulder removal on the existing face;
- Reduction in temporary works—soil nails and shotcrete—however some permanent soil nails required for slope stability;
- Reduction in concrete volume by removing the mass concrete foundation to the retaining walls and abutment, although somewhat offset by the mass concrete keyed toe for the new reinforced slope embankment;
- Reducing the overall height of the in-situ concrete retaining wall supporting the traffic barrier; and
- Reduction in retained height at Abutment B.

The alternative design introduced the requirements for an additional volume of fill material and geotextile reinforcement and relied upon the feasibility of constructing bored piles to support the abutment, which was confirmed by the Contractor. The piling rig was able to access the abutment location from the existing road using the partially completed reinforced embankment. Steel casings were required to retain the newly placed fill and existing road embankment (uncontrolled) fill and were pre-bored then driven to a stable rock layer.

4.4.1 Site Investigation

There were limited geotechnical investigation data available at the detailed design development stage due to access issues. As such additional boreholes were mandated as part of the construction stage requirement. Figure 17 shows borehole and test pits completed at concept design, detail design and construction stages, including boreholes along the edge of the existing highway, borehole near Abutment B, boreholes along the retaining wall and test pits bear the toe of the embankment. The available information confirmed the high level of variability of the inferred top of Class V and Class III rock levels. It is worth noting that the required boreholes for the piled abutment could not be acquired due to access issues until the reinforced earth embankment was constructed to around RL845 m.

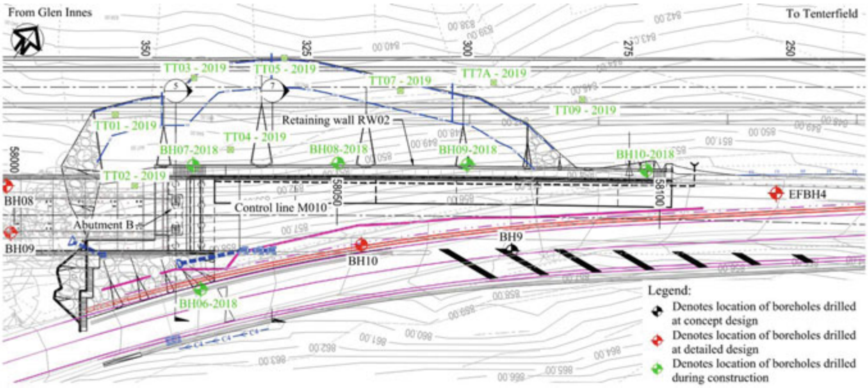


Fig. 17 Layout plan of boreholes drilled at various stages (without showing boreholes at abutment)

4.4.2 Abutment B and Slope Design

The modified design of Abutment B and approach retaining wall avoided significant additional temporary soil nail works and mass concrete foundations, due to deeper than expected rock levels.

It was proposed that the reinforced slope would be constructed bottom up to achieve a level at the abutment of RL 854.5 m. Access for piling works would then be from the north along the partially complete reinforced slope. The design of the reinforced slope needed to consider the temporary loads associated with the installation of the large diameter cast-in-place (CIP) piles.

4.4.3 Geotechnical Design and Analysis

The design of bored pile foundations followed the design assumptions and methodologies as described below:

- The serviceability assessment of the pile load capacity and displacement was carried out based on the SLS and the relevant parameters in Table 4.
- Geotechnical strength reduction factor of 0.4 was applied to the ultimate geotechnical capacity based on AS2159-2009.
- The ultimate geotechnical strength to resist the applied compression loading was calculated in accordance with AS5100.3 and AS2159 (2009) based on the design parameters derived at the original design.
- Shaft adhesion and end bearing parameters assumed a clean rock shaft socket and that the base of the pile will be cleaned using mechanical and/or air lift techniques prior to pour of concrete.
- For tension checking, shaft adhesion resistance together with rock cone/wedge was considered.

- Adequacy of piles was checked against lateral loads (i.e. ULS shear force and bending moment) at the top of rock socket based on using the methodology described in Hong Kong GEOGUIDE 1 [13].

4.4.4 Abut B Piled Foundation Design Summary

Eight (8) 1050 mm piles were designed for the Abutment B with each pile socket in Class III granite or better to achieve the pile SLS and ULS capacity. These piles varied in length from 8 to 13 m to accommodate the interpolated sloping rock profile. A pile socket of 3 m into Class III rock was required.

4.4.5 Reinforced Slope Design Outcome

The reinforced slope design was to provide piling plant access and platform to the top of pile level and subsequently to the final subgrade level after piling works. A typical cross-section of the slope is shown in Fig. 18, with a cross-section through the abutment B being presented in Fig. 19. A minimum width of 4 m was nominated for the high strength geotextile reinforcement to not only provide sufficient length for embankment stability but also for the trafficability of the proposed plant by the contractor.

The bearing capacity of existing fill was assessed inadequate for the new reinforced embankment of higher than 20 m. Therefore, removal of existing fill and replacement by rock fill keyed into Class IV/V or better rock near the toe of the reinforced embankment was proposed in this design to form a berm key. A layer of separation

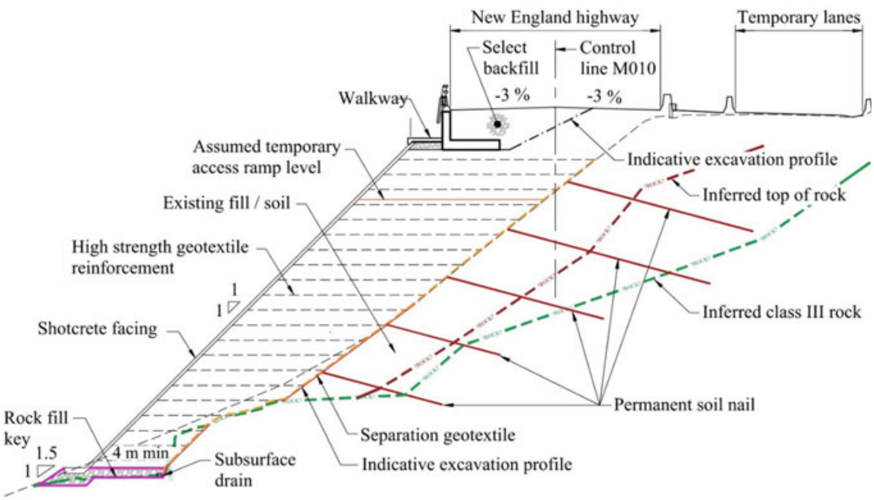


Fig. 18 Typical cross-section of reinforced embankment with permanent soil nails

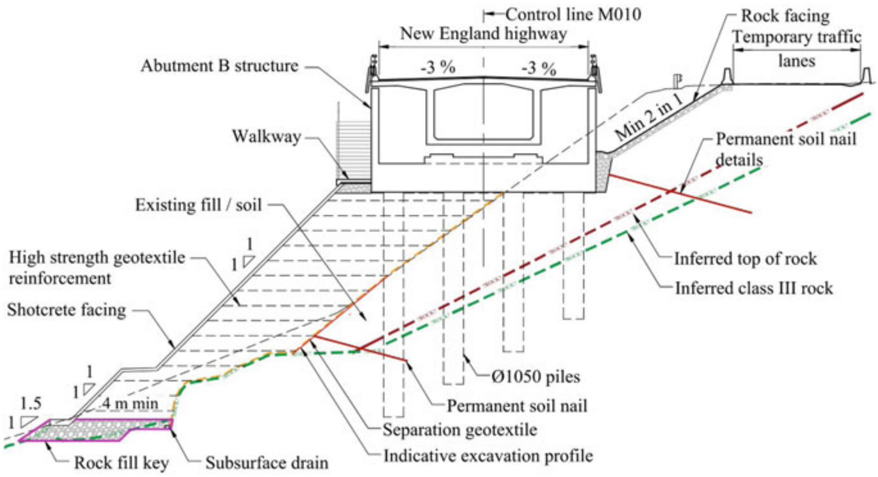


Fig. 19 Typical cross-section of reinforced embankment through piled Abutment B

geotextile was used at the interfaces with the existing soils and the new fills. This was to provide the stability of the embankment and avoidance of water pressure built-up.

A conventional slope of engineered fill at a gradient of 1 vertical to 1 horizontal would have been too steep to achieve the required factor of safety. Therefore, geosynthetic reinforcement straps were incorporated to enhance the internal stability of the engineered fill block based on BS8006-1995 [7]. The reinforcement straps were high strength geosynthetic reinforcement (minimum 400 kN/m at 5% long term strain) across the 1:1 slope fill block, which varied in length from 4 to 10 m.

Permanent soil nails were required to provide resistance against the potential slope instability planes through the existing uncontrolled fill. The slope stability assessment results indicated that 3–4 rows of permanent soil nails would be required to achieve the minimum factor of safety of 1.5 for the long-term condition. Some of these permanent soil nails doubled as temporary support for the excavation near the toe of the embankment. The proposed permanent soil nails were to be Ø40 Grade 500N reinforcement bar in accordance with AS 4671 in 150 mm diameter drill hole with a typical embedded length of approximately 4.5–6.5 m into Class IV/V rock or 3 m into Class III rock. The total soil nail length was expected to range from 8 to 15 m, subject to site validation testing.

As these soil nails were to be drilled through the uncontrolled fill, including boulders and cobbles, the Contractor was required to consider temporary/permanent casing to prevent drill hole collapse.

Instead of reinforced shotcrete face for the soil nail, it was proposed to have individual reinforced concrete (either Grade 40 concrete or sprayed concrete) soil nail head with the size of 400 mm × 400 mm and minimum 250 mm thick to be embedded in the slope.

To prevent potential weak slip plane between the new engineered fill and the existing uncontrolled fill, the interface treatment included removal of existing floating boulders and any loose material on the existing slope of the order of 0.6–1.2 m diameter. In addition, localized benching was formed as per RMS R44 [17] to avoid formation of a weak interface between the new and existing fills.

A subsurface drainage layer wrapped in geotextile layer as per R44 was provided between the existing and new batter interface to ensure minimal water pressure built up behind the wall. Furthermore, a plastic drainage pipe in accordance with RMS specification 3552 [18] was also included within the drainage layer to ensure adequate discharge of seepage water from the upper slope fill to the dedicated outlet point.

Comprehensive monitoring during construction was undertaken to validate design assumptions, determine additional drainage if seepage observed during the excavation of the existing slope, and implement contingency plan if movements beyond predicted values were encountered.

For a typical section of the RW02 wall, as shown in Fig. 17, there was enough space for the proposed retaining wall founding on the existing slope without the reinforced embankment. The global stability against the additional loading from the retaining wall and traffic surcharge was enhanced by inclusion of the permanent soil nails after removal of loose boulder on the existing slope face. The bearing capacity of the existing fill was improved by over-excavation of existing fill replaced by a reinforced compacted new fill. Erosion control measures was considered for the final slope surface, such as native grass mix plus compost blanket.

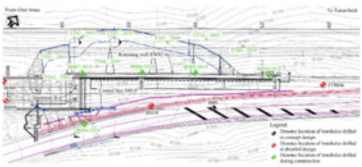
5 Construction Stage Photos and Lessons Learnt

Some of construction stage photos and lessons learnt from the process of design and construction of this project may be summarised in the Table 5.

6 Conclusions

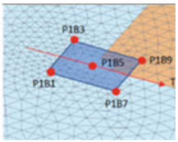
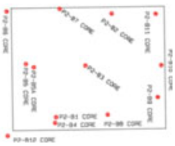
The paper has described a case study of the design of an in-situ concrete box girder bridge of approximately 320 m by balanced cantilever construction, with the middle span of 150 m, in hilly granite formation. Several technical challenges including re-design of the Pier 2 footing using micro-piles and Abutment B using bored piles together with a geotextile reinforced embankment as piling access track and platform. Lessons learnt from this project have also been presented in this paper, which could be useful for future projects. Out of these lessons learnt the wholistic design and construction approach was the key to project success, which requires input from all the parties involved. The performance monitoring to date indicated that the bridge performance is satisfactory, and it has been open to traffic in September 2021.

Table 5 Construction stage photos and lessons learnt

ID	Photo	Activity description	Lessons learnt
1		Steep terrain, the environmental constraints (Bolivia Wattle), access, and constructability drove the adopted bridge option and construction method	An integrated approach to design and construction is critical given the uncertainties of temporary works and method of construction
2		Rock fall risk assessment and stabilisation measures for assessed high ARL slopes using RMS slope risk assessment guidelines	Assessment guideline is subjective and can result in quite different recommendations depending upon the assessor. Safety of workers during construction requires a higher level of mitigation
3		Steep terrain and difficulty access constraints for site investigation data collection and variability of ground conditions	Appreciation of the potential high risks of variable ground condition may lead to change of design concept
4		Use of rock fall fence for the unstable rock blocks, the shotcrete for highly weathered rock and drape mesh for “blocky rock” in the new cuts	Variation of the rock quality in a 3D space is inevitable. Consideration of exposed rock cut face should be given in design
5		Use of precast block works as temporary form together with concrete infill to construct retaining wall in steep terrain	Adequate geotechnical site investigation data must be obtained to confirm the rock levels and depth of excavation in steep terrain

(continued)

Table 5 (continued)

ID	Photo	Activity description	Lessons learnt
6	 	Piling rig for 1050 mm diameter pile through fill and granite of highly variable strength and levels of Class III rock	Selection of appropriate type of piling rig, cutting heads and cleaning bucket, and provision of casing through fill to ensure production
7	 	Temporary support for excavation at pier locations for footings with temporary retaining wall works for the access road	Access difficulties -key to constructability and temporary works design for optimisation
8	 	Pier 1 interpretation and actual ground inspection and assessment	The weathering and rock quality change with depth was not highly variable within Pier 1 with only localized removal and replacement by concrete
9	 	2 boreholes available at detail design but the contact interface between granite and rhyodacite unable to be delineated until excavation to pad footing level	Granite and rhyodacite interface delineated with 10 additional boreholes and the very low strength of rhyodacite up to 9 m at Pier 2 unexpected, leading to ground improvement
10	 	Geosynthetics reinforced soil slope with shotcrete surface protection achieved 1 in 1 embankment batter without removal of existing fill and made piling platform feasible	Uncertainties of the existing fill quality and presence of weak rock required permanent soil nails and rock fill toe key and drainage to achieve the long-term stability of the embankment

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