World Academy of Science, Engineering and Technology
International Journal of Geotechnical and Geological Engineering
Vol:11, No:9, 2017

Geotechnical Design of Bridge Foundations and Approaches in Hilly Granite Formation

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Abstract**—**This paper presents a case study of geotechnical design of bridge foundations and approaches in hilly granite formation in northern New South Wales of Australia. Firstly, the geological formation and existing cut slope conditions which have high risks of rock fall will be described. The bridge has three spans to be constructed using balanced cantilever method with a middle span of 150 m. After concept design option engineering, it was decided to change from pile foundation to pad footing with ground anchor system to optimize the bridge foundation design. The geotechnical design parameters were derived after two staged site investigations. The foundation design was carried out to satisfy both serviceability limit state and ultimate limit state during construction and in operation. It was found that the pad footing design was governed by serviceability limit state design loading cases. The design of bridge foundation also considered presence of weak rock layer intrusion and a layer of “no core” to ensure foundation stability. The precast mass concrete block system was considered for the retaining walls for the bridge approaches to resolve the constructability issue over hilly terrain. The design considered the retaining wall block sliding stability, while the overturning and internal stabilities are satisfied.

Keywords**—**Pad footing, hilly formation, stability, block works.

I. Introduction

THE existing New England Highway situated between Glen Innes and Tenterfield, New South Wales, Australia, consists of a two-lane sealed road that traverses the north western side of the Bolivia Range. It comprises a two-lane road with narrow shoulders on hilly terrain, and is commonly used by heavy vehicles of which make up approximately 25% of the total traffic volume.

The Roads and Maritime Services (Roads and Maritime), New South Wales, has proposed an upgrade of this section of New England Highway at Bolivia Hill, which is to include a realignment of the highway to improve poor horizontal alignment, steep grades, and poor crash history. The primary project objectives are to improve the alignment of the New England Highway to enhance local traffic efficiency, road safety, road transport productivity, efficiency and reliability of travel. Arcadis was engaged to undertake the concept design and Environmental Impaction Assessment (EIS) and then detailed design in two stages. The section is about 2.1 km in length, with a bridge of approximately 320 m length crossing steep valley. The final design is in-situ concrete box girder by balanced cantilever construction, with the main span being 150 m and the other two spans being 80 m and 86 m.

This paper presents 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 are further discussed. Some of lessons learnt from this project such as the importance of constructability issue through hilly site formation are also summarized.

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II. Site Topography and History

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.

The section of road subject to the upgrading starts at about Ch 56800 m and continues to Ch 58900 m (i.e. overall distance of 2100 m). Surface elevations of the existing road grade from 935 m AHD at the south-eastern end down to about 817 m AHD at the north-eastern end of the road.

The existing road was constructed by cut and fill method in the 1950s. There is a sharp bent at this section of alignment which caused a few fatalities in recent years. Cuts were primarily formed into the uphill on the eastern side of the road whereas embankment on the western side of the road. The existing cuttings were up to about 2 m to 8 m high and battered at 30° to 70° to the horizontal. The existing embankments are of varying heights ranging between 10 m to 30 m at a batter gradient of 30° to 55°. The exposed cut surface comprises granite bedrock of varying strengths with little soil cover over bedrock. At the crest of the hill there are either isolated boulders or piles of boulders formed by construction of the disused railway some 100 years ago. The existing slopes are known for frequent rock fall and assessed to be medium to high risk.

Site vegetation comprised numerous mature and semi mature trees, predominately eucalypts.

III. Geological Setting and Investigations

Reference to the 1:250,000 scale geological series map for Grafton (SH 56/6) indicates that the site is underlain by Bolivia Range Leucoadamellite (renamed as Leucomonzogranite). Previous studies [1], [2] suggest the Dundee Adamellite Porphyrite (renamed as Dundee Rhyodacite) is located to the north of the project site and potentially at Ch 58600 m (about 150 m north-east of the project site).

Two sets of geotechnical investigations were undertaken: One for the concept design development and the other for the detail design development. The results of both investigations confirmed the presence of pink medium grained leucogranite and coarse-grained granite together with microgranite. Intrusions of rhyolite and basalt were also identified at the site together with rhyodacite in the northern part of the site at borehole BH12 (Ch 58470 m), which confirm the geological mapping for the area.

IV. Geotechnical Model and Design Parameters

The geotechnical profiles for the main bridge and approaches are summarized below:

Unit 1a) Fill, comprised of variable mixtures of pavement materials (asphalt), silt, and sand with some high to very high strength granite cobbles and boulders.

Unit 1b) Colluvial Sand / Gravel, variable mixtures of sand and gravel with the lower parts grading into extremely weathered granite.

Unit 1c) Residual, derived from the complete weathering of the underlying bedrock. Typically comprised of brown sandy clay, silty clay, or clay.

Unit 2a) Granite, slightly weathered to fresh, fragmented to fractured, high to very high strength granite. It was generally medium - coarse grained, and orange pink, pink and grey, and brown and grey in color.

Unit 1b) and Unit 1c) have been grouped into one unit, namely Unit 1b), due to relatively thin layer for Unit 1c).

The rock classifications for the granite were based on the unconfined compression strength (UCS) and defect spacing as shown in Table I.

The geotechnical design parameters for pile design and embankment stability analysis are presented in Table II. Note that the allowable bearing capacity for the pile design was based on the geotechnical interpretative report [3], [4] and other published data [5], which was derived from the associated deformation limits of the order of 20 mm to 30 mm. Consideration was also given the proposed concrete strength of piles and pad footing which are lower than the rock strength for class I and Class II granite.

TABLE I
Classification System for Granite

Rock Class	Point Load Strength, $Is_{(50)}$, (MPa)	Unconfined Compression Strength, (MPa)*	Defect Spacing
V	< 0.3	< 6	< 60 mm
IV	0.3 to 1	6 to 20	< 60 mm
III	1 to 3	20 to 60	> 60 mm
II	3 to 6	60 to 120	> 200 mm
I	> 6	> 120	> 600 mm

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

The ultimate bond stresses for five classes of rock for passive ground anchors were taken to the same as the corresponding shaft adhesion for bored pile. The allowable bond stresses for passive anchors were taken to be 50% of the corresponding ultimate values.

The stiffness of rock mass was formulated in such a way that the defects, seams, and fracturing within the rock mass will be reflected.

TABLE II
Geotechnical Design Parameters Adopted for Each Unit

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

V. Main Bridge Foundation Design

A. General

During the concept design development stage, four shortlisted bridge options were studied, including:

1) Super-T girder bridge;
2) In-situ concrete box girder bridge by balanced cantilever construction;
3) Incrementally launched steel girder bridge; and
4) Arch bridge.

The in-situ concrete box girder bridge by balanced cantilever construction was selected with due consideration of environmental constraints including the constructability issues and the Bolivia Wattle restricted zone at the middle of the bridge as shown on Fig. 1 (a) [6].

The bridge is 316 m long and 12m wide with a middle span of 150 m as shown on Fig. 1 (a). The Stations for Abutments A and B are 57705 And 58029 respectively, with Pier 1 and Pier 2 being at Stations 57795 and 57945 respectively.

Fig. 1 (b) shows 3D view of the proposed bridge after construction with respect to the existing road. The existing sharply bent road will be used as an access road for bridge maintenance.

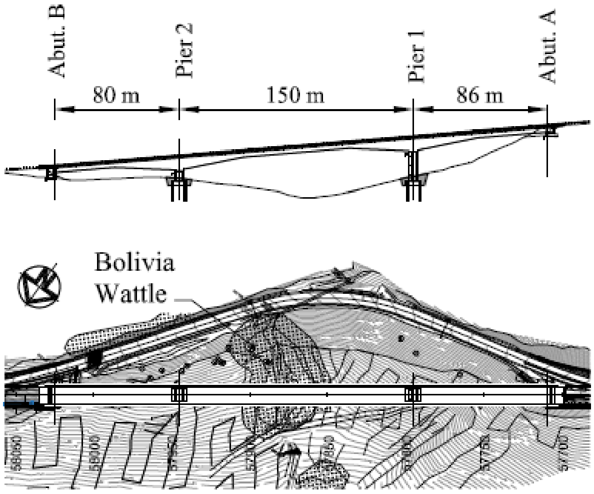

Fig. 1 (a) Layout plan and elevation of main bridge

Fig. 1 (b) 3D view of the completed bridge from bottom of slope

B. Foundation Option Consideration

The concept design of main bridge foundations was based on piles due to the large unbalanced bending moment for the most onerous load case. This was also driven by the fact that there is "no core" of 280 mm at borehole BH04 near Pier 1. Preliminary investigation indicated that a very large pad footing would be required to overcome the large bending moment induced by the accidental load case during construction. After review of the stage II geotechnical investigation data and careful consideration of constructability of the pier foundation, it was decided to use pad footing together with tie-down passive anchors for the Pier 1 and Pier 2 foundations. The abutment piles were also replaced by pad footing with infill mass concrete for the stepped excavation. Sacrificial precast mass concrete block works were considered to accommodate the steep terrain.

The pad size for Pier 1 and Pier 2 presented in the final design [6] is 10 m along the traffic direction by 8 m in the transverse direction, as shown in Fig. 2. Four rows of ground anchors, each having five evenly distributed, were arranged at both end of the pad to deal with the large bending moment for the accidental load case in the event of a segment fall off during construction.

The dimensions of Abutment A and Abutment B in plan in final design [7] are 7.26 m long and 11.8 m wide, with approach slab of 6 m length connecting to the abutments at both ends.

C. Foundation Design and Analysis

For the main bridge foundations, three types of analyses have been undertaken:

1) The first one was to determine the dimension of pad-footing for Pier 1 and Pier 2 based on the principles of rock mechanics. The analyses included bearing, sliding, and overturning stability of the pad footing under all design load cases. These checks were performed using in-house excel spreadsheet developed by Arcadis.
2) The second one was the serviceability limit state analysis, which was carried out using finite element program Plaxis 2D. Table III summarizes the calculated ground settlements at the piers for the worst loading cases. It is noted that the settlement at abutments is less than that at Piers where larger loads occur. It can be readily seen that the calculated ground settlements are less than 25 mm at both piers.
3) The last was a global stability analysis using Slope/W to assess the minimum factor of safety (FoS) against potential slip planes beneath the pad footing.

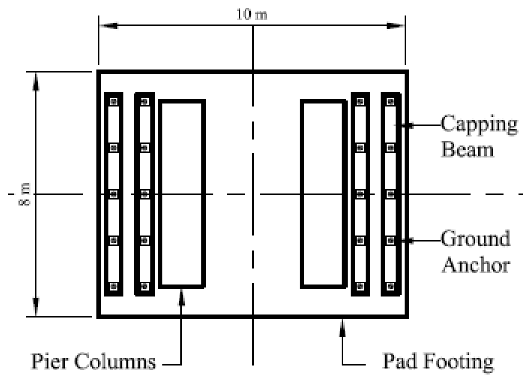

Fig. 2 Layout plan of pad footing for Pier 1 and Pier 2

Three broad load cases were considered in the global slope stability analyses for pier foundations:

1) long term loading;
2) traffic impact loading where applicable; and
3) earthquake condition with a ground acceleration coefficient of 0.07.

TABLE III
CALCULATED SETTLEMENTS AT BRIDGE ABUTMENTS AND PIERS

Location	Foundation Material	Calculated Settlement (mm)
Abutment A	Class III Granite	2 to 5
Pier 1	Class III Granite	9 to 25
Pier 2	Class III Granite	8 to 20
Abutment A	Class III Granite	5 to 10

The required and calculated minimum factors of safety for the slopes at Pier 1 and Pier 2 are summarized in Table IV. The calculated lower FoS value at Pier 2 was due to the upslope instability whereas the calculated FoS at Pier 1 was due to presence of a "no core" of 280 mm at a depth of approximately 8 m below the pad footing. The results of slope stability analyses for abutments will be presented in the following section.

It can be readily seen that the imposed pad foundation loading would not cause any slope instability problems at Pier 1 and Pier 2 for all loading cases considered.

TABLE IV
CALCULATED MINIMUM FACTORS OF SAFETY AT PIER1 AND PIER 2

Load case	Load case description	Required FoS	FoS at Pier1	FoS at Pier 2
1*	Long Term	1.5	1.7	1.5
2	Vehicle Impacting Barrier	1.2	N/A	N/A
3	Earthquake Condition	1.2	1.8	1.9

*Note that both circular and non-circular cases were considered for the long-term stability. N/A = Not Applicable.

VI. GEOTECHNICAL DESIGN OF BRIDGE APPROACHES

A. General

There are two proposed retaining structures, as shown in Figs. 3 (a) and (b), for the provision of the required road widening along Abutment A (southern end of bridge) and Abutment B (northern end of bridge). These will be constructed adjacent to the existing road to allow for sufficient access to construct the bridge abutment structure as part of temporary works, and to support the approach embankment fill (the road widening works), respectively. Part of walls is to be formed in cuttings, and the rest is to be built over the existing ground by filling.

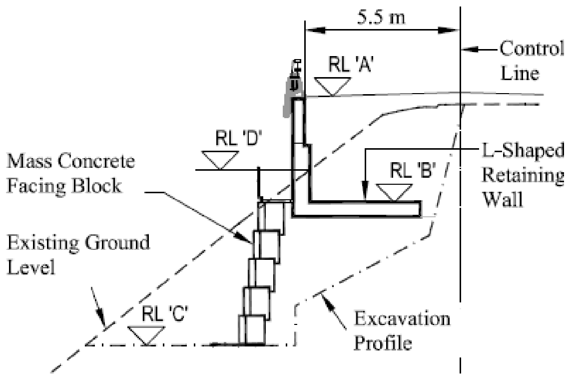

Fig. 3 (a) Typical elevation of approach retaining wall

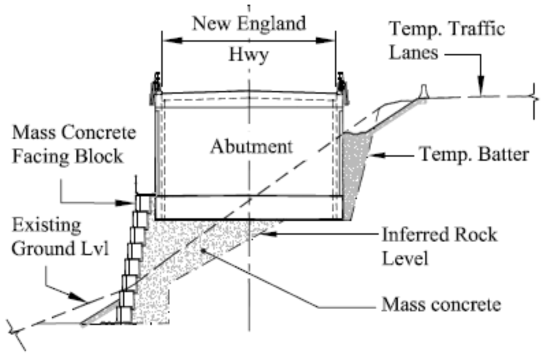

Fig. 3 (b) Typical elevation of abutment retaining wall

B. Retaining Wall Options

Retaining wall options that were considered for the bridge approaches are described below:

1) Soldier piled wall or contiguous piled wall with a reinforced capping beam was considered as either a cantilever or tie-back structure. Cantilever retaining structure was considered feasible where adequate socket into the Granite bedrock below the proposed excavation level could be achieved. When a pile would terminate above the bulk excavation, the toe of pile should be anchored back into the stable ground to achieve the lateral wall stability.
2) Composite retaining wall comprising a conventional L-shaped retaining wall or a modular precast concrete gravity retaining wall founded on the bedrock could be considered.
3) For other structures or low retaining walls not considered in the above, consideration could be given to the use of strip footings.

Inclusion of suitable drainage behind the retaining walls was considered to minimize water pressure build-up behind the proposed retaining walls. Notwithstanding the provision of drainage behind the walls, a nominal water pressure at 1/3 of the retaining height was considered in the retaining wall designs.

The biggest challenge of the bridge approach design was the steep terrain. To maintain the live traffic, it was fundamental to keep at least one lane open so that the disruption to the commuters could be minimized. One of the options considered at the detail development was to have two-lane open during construction. It was not pursued further primarily due to high cost on the temporary extra lane and other requirements.

A block wall option together with L-shaped wall of approximately 4 m height was preferred since it is relatively easy to construct the wall from bottom up in a step by step manner. The driving factor was to ensure that the block works would be used as “form” work to construct the retaining wall approximately 15 m high. The main requirement for the block retaining wall was to ensure that the loose colluvium material near the existing surface would be removed and the founding material will be on class IV granite or better. One of the design considerations was to place non-fine concrete behind the block works to ensure that there would be no pore water pressure built-up behind the block works. The other measure considered was to have benches formed to ensure that there would be no shear plane along the potential weak interface between newly placed fill and the original steep terrain.

C. Design and Analysis of Retaining Walls

For the approach retaining walls there are three portions: 1) Abutment walls; 2) concrete block retaining wall; and 3) transition to tie in the existing wall. This section is focused on the design and analysis of block retaining walls.

Two typical cross-sections were selected for global slope stability analyses for the southern and northern approaches respectively. Sections 1 and 2 are for the southern approaches, with Section 2 being at Abutment A, whereas Section 3 and 4, with Section 2 being at Abutment B, for norther approaches.

The results of slope stability analyses are summarized in Tables V and VI, respectively.

It can be readily seen that the norther approach condition is worse than for the southern ones. This is due to that the retaining wall is relatively higher, and the ground condition is worse.

TABLE V
CALCULATED MINIMUM FACTORS OF SAFETY (FOS) SECTIONS 1 AND 2

Load case	Load case description	Required FoS	FoS for Sect.1	FoS for Sect. 2**
1*	Long Term	1.5	3.6	2.6
2	Vehicle Impacting Barrier	1.2	4.1	2.7
3	Earthquake Condition	1.2	3.7	3.1

*Note that both circular and non-circular cases were considered for the long-term stability. **The presented FoS values are for the transverse section as they are worse than the longitudinal ones.

TABLE VI
CALCULATED MINIMUM FACTORS OF SAFETY (FOS) SECTIONS 3 AND 4

Load case	Load case description	Required FoS	FoS for Sect.3	FoS for Sect. 4**
1*	Long Term	1.5	1.5	1.7
2	Vehicle Impacting Barrier	1.2	1.6	1.9
3	Earthquake Condition	1.2	1.5	1.9

*Note that both circular and non-circular cases were considered for the long-term stability. **The presented FoS values are for the transverse section as they are worse than the longitudinal ones.

TABLE VII
GEOTEXTILE REQUIREMENTS FOR TRANSITION ZONES

Embankment height (m)	No. of layers	Maximum vertical spacing (m)	Minimum length (m)
1.2 to 1.5	1	N/A*	1.9H
1.5 to 2.0	2	0.5	1.9H
2.0 to 3.5	3	0.7	1.9H
3.5 to 6.5	4	0.9	1.9H

* N/A=Not Applicable.

The transition section of about 10 m length was developed as reinforced soil slope to achieve the slope stability requirements. Each layer of geotextile was designed to have a minimum working load of 200 kN/m at 5% long term strain based on British Standard BS8006-1995 [8]. The internal stability design was carried out using commercially available software GEO5. A summary of the required geotextile strength for different wall heights is presented in Table VII.

The factor of safety against global slope instability for both transition zones was checked using Slope/W and was found to be acceptable.

VII. SLOPE RISK MANAGEMENT STRATEGY

A. Existing Upslope Risk Assessment

The southern section of the alignment has been subject to "rock fall" risks for a long time. Roads and Maritime has undertaken slope assessment of the existing cuts and upslope, with some sections having an assessed risk level (ARL) of 2. Subsequently, Roads and Maritime decided to carry out the emergency remedial work prior to this upgrade project. The scope of works primarily includes removal of unstable isolated boulders, stabilization of rock boulders, and the "rock avalanche" formed during the disused railway over 100 years ago. Construction of the rock fence near the crest of the new cutting for the new southern approach was provided as part of the detailed design development.

B. Rock Fall Fence Design and Slope Residual Risk

Rock fall modelling was completed for potential natural boulder rolls from the backslope above cutting ID 14039 to review effective rock fall fence capacity.

Modelling was undertaken using the Colorado Rock Fall Simulation program Ver 4.0 (CRSP). Slope profile was selected at approximate Station 57700. Slope parameters were based on the latest survey data, and boulder properties are presented in Table VIII. Other parameters were based on the default values in the program. The proposed rock fall barrier was to be installed approximately 5 m from the crest of the batter. This is at analysis point 1 (AP1).

TABLE VIII
KEY PARAMETERS FOR ROCK FALL SIMULATION

Density (kg/m^3)	Diameter (m)	Analysis point	Release zone	No. of boulders
2700	0.5, 0.8	AP1: X=51 (5m from crest), AP2: X=56 (Crest of batter), AP3: X=70 (Fog line)	Y= 40 to 25	1000

Based on model outputs, as shown in Figs. 4 and 5, the proposed 35-kJ rock fall barrier located 5 m above the crest of the batter is effective in preventing individual boulders less than 0.8 m diameter dislodged mid backslope from reaching the carriageway. Fig. 6 shows a typical cross-section of rock fence in relation to the cut profile and the upslope profile and boulders.

C. New Cut Design Considerations

There are several new cuttings proposed along the route of the scheme. These are located between Stations 56,823 and 57,614. The cuttings are likely to be formed in predominantly rock materials. Cuttings in soils (Unit 1) may be designed at 2 horizontal to 1 vertical for long term stability and 1.5 horizontal to 1 vertical for short term stability. The new cuttings in the Granite may be trimmed to 1 horizontal to 4 vertical.

The excavatability of rock cutting was assessed using the method by Pettifer and Fooker [9], and the results indicate that blasting may be required for some portion of cutting and bridge foundation excavation.

Our assessment of the newly cut batter slopes based on the defect information in the geotechnical investigation reports [1], [2] indicates that three possible failure mechanisms may be present during excavation phase:

1) Localized slumping and shallow rotational movement: This is likely induced by excavation and de-vegetation/tree undermining within the highly weathered zones along the cutting.
2) Boulder rolls: This is likely to be caused by dislodgement of isolated rock block within the exposed cut face due to stress relief during and post excavation works.
3) Block sliding and/or rotation of rock blocks: This is likely due to the presence of unfavorable joints within rock mass whose size is dependent upon the joint set pattern when the cut face day lighted during and after excavation.

It is anticipated that minor scaling, rock bolting, and shotcrete application may be required for highly weathered zones to stabilize the localized unstable rock mass where encountered.

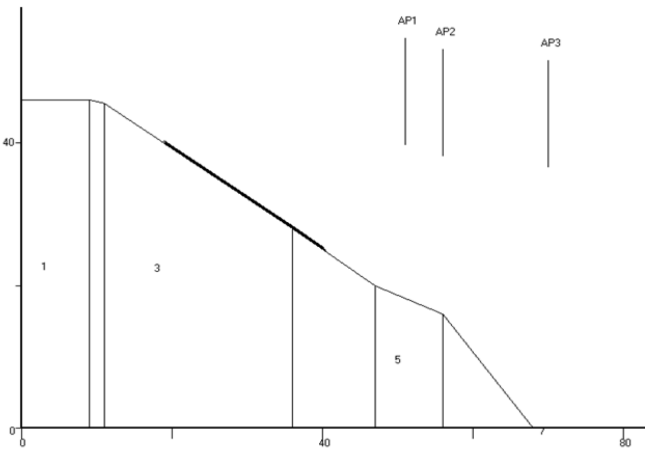

Fig. 4 Cutting 14039: Rock fall model output for 0.8 m diameter boulder with starting velocity H = 0.8 m/s, V = - 0.8 m/s

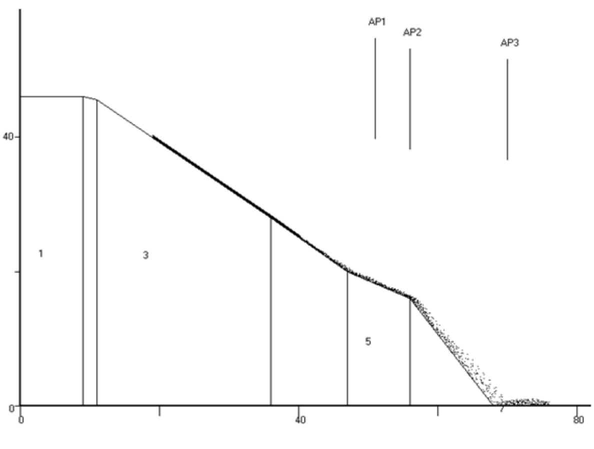

Fig. 5 Cutting 14039: Rock fall model output for 0.8 m diameter boulder with starting velocity H = 0.95 m/s, V = - 0.95 m/s

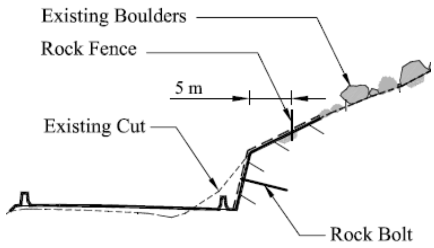

Fig. 6 Rock fence with new cut and upslope

D.Long Term Cut Slope Protection Measures

As part of the design, the cuttings in soil should be grassed so that surface erosion can be controlled in the short and long term. The rock cuttings will be exposed and subject to long term weathering and therefore it is 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, could be considered. This will minimize the potential risk of any rock falling onto the carriageway and afford the opportunity to carry out remedial works if deemed necessary.

The exposed rock cut face shall be mapped by a qualified geotechnical engineer or engineering geologist during excavation and shall confirm the need of any stabilization measures on site.

VIII.Construction Validation

The pier foundation would need to be inspected and mapped by a qualified geotechnical engineer or engineering geologist on site to confirm that the required ultimate geotechnical capacity can be achieved. In addition, two boreholes of 10 m depth will need to be drilled to ensure that there would be any potential “voids” below the pad footing founding level. The exposed cutting batters shall be mapped to ensure that any potential unstable wedges will be stabilized.

Similarly, the footing of the abutments shall be inspected and mapped by a qualified geotechnical engineer or engineering geologist on site to confirm that the required ultimate geotechnical capacity can be achieved. Attention shall be paid to the sloping ground in front of the abutment to ensure that any potential unstable rock wedges will be stabilized prior to abutment construction.

The exposed rock for the retaining wall construction shall be inspected and assessed by a qualified geotechnical engineer or engineering geologist on site to ensure that a “rough” surface as shown on the drawings will be achieved to avoid formation of any weak plane at the interface between mass concrete and the parent rock. Fig. 7 shows the completed bridge flying over the steep valley in relation to the exiting road when completed.

Fig. 7 Completed bridge in relation to existing road

IX. Safety-In-Design

Safety-in-design is a mandatory requirement for design works in Australia. A safety-in-design workshop for the entire project was carried out and the geotechnical related potential risks were identified in a risk register. The geotechnical design process was carried out to either eliminate or minimize those identified risks where possible. Where the potential risks could not be eliminated, mitigation measures were appropriately considered to ensure that the safety requirements would be met.

X. Conclusion

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 150m, in hilly granite formation. The mass concrete block was proposed to deal with the steep terrain to make the retaining wall structure. The upper L-shaped retaining wall and U-structure at the abutments have achieved the most economic design for the approaches to the main bridge. The constructability consideration and the further geotechnical data eliminated the pile foundation by introduction of temporary passive ground anchors. The design has been approved and accepted by the client, and a contract will be awarded by the time of paper presentation.

Acknowledgment

This paper was developed through an actual project which was fully supported by Arcadis staff and the project managers and reviewers from Roads and Maritime Services, New South Wales, Australia. Without them this paper would not be possible. The author would like to express his gratitude to all who contributed to the success of this project.

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