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Post-Cyclone Jasper remediation of landslide-impacted linear transport infrastructure in Far North Queensland

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ABSTRACT: In response to the damage to infrastructure sustained during intense rainfall associated with Tropical Cyclone Jasper between 13 and 18 December 2023, this paper presents the approach adopted to restore integral linear transport infrastructure, particularly focusing on interim risk mitigation and long-term slope remediation at a critically affected site. In collaboration with the Client, emergency works, interim risk mitigation measures and detailed design were completed to address slope instability at the Site. An initial aerial inspection identified a compromised fill embankment and an exposed relic retaining structure. An ensuing site inspection and slope risk assessment revealed stability risks at the downslope embankment, presenting unacceptable risk to persons and property resulting in the temporary cessation of traffic operations. In consultation with stakeholders, a catalogue of control measures was developed to manage the geotechnical risks and allow restricted operations to resume while the detailed design was prepared. Concurrently a geotechnical investigation consisting of intrusive methods and photogrammetry was conducted to inform detailed design. The design involved (i) remediation of a 30 m section of the downslope embankment and (ii) stabilisation of a newly constructed 145 m long upslope cutting, graded at 45° to 80° with maximum vertical height of 12 m, created by excavating into existing cutting and natural hillslope to widen the formation to allow for the realignment of traffic operations away from the failed downslope embankment. Due to time constraints, the cutting was excavated using an observational approach under full-time supervision. The cutting was steeply profiled to minimise ground disturbance within the Wet Tropics World Heritage Area and avoid the nearby property boundary. The long-term design comprised a combination of passive dowels, reinforced shotcrete, rockfall mesh and subsurface drainage enhancements. The remedial designs considered sensitive enviro-cultural setting, site constraints, climate, material availability, contractor capabilities, durability and identified slope instability mechanisms.

KEYWORDS: Cyclone Jasper, World Heritage, Wet Tropics, disaster recovery, slope risk assessment, slope remediation.

1 INTRODUCTION

Between 13 and 18 December 2023, intense and prolonged rainfall associated with Tropical Cyclone Jasper (TC Jasper) (Sugawara et al., 2024) resulted in the accumulation of approximately 1,891 mm of rainfall recorded at a weather station located 2 km north of the site. This rainfall event is estimated to be equivalent to a 1-in-500-year Average Recurrence Interval (ARI) storm event. Following an initial aerial inspection, twenty-six (26) sites along the transport corridor were identified as requiring remediation. These included areas of severe riverbank erosion and instability of cut and fill slopes above and below the transport corridor. The site assessed as having the highest associated geotechnical risk at the time was selected as the primary example for this paper.

The project site is composed of a linear transport corridor constructed adjacent to a major river. It is located within the Djabugay Nation Native Title Claim and the environmentally sensitive Wet Tropics World Heritage Area (WTWHA). Elevated water levels in an adjacent major river resulted in extensive erosion to a section of downslope embankment supporting a culturally significant transport corridor in Far North Queensland (FNQ) and exposing an unknown relic concrete retaining wall. To minimise disruption to the community and economic impact due to prolonged suspension of the transport route, a 145 m long section of the corridor was widened by excavating into an adjacent upslope cutting and native hillslope above, enabling the alignment to be deviated away from the affected downslope embankment and allowing operations (with restrictions) to resume. Due to the environmental and cultural significance of the site and proximity to the neighbouring property boundary, the cutting was steeply profiled with marginal slope stability in the long term.

This paper examines the emergency works, interim risk mitigation measures, site investigation techniques, slope risk assessment (SRA) and design approach of long-term remedial works at the Site.

2 PROJECT EVOLUTION

Following TC Jasper, the Client and Consultant conducted an aerial inspection on 20 December 2023 to assess damage to a culturally significant transport corridor as part of emergency works. The site had previously been identified as having a risk of slope instability due to elevated river levels in March 2023, a condition exacerbated by the cyclone. Operations were suspended, prompting immediate interim measures to minimise economic losses and risks to people and property.

An initial site walkover on 3 January 2024 assessed site conditions, evaluated risks, and identified interim mitigation measures including (i) installation of a monitoring regime on the downslope embankment and (ii) excavating the upslope cutting under full-time geotechnical supervision. These works realigned the corridor to restore operations while allowing time for detailed design of long-term downslope remediation.

Following implementation of the interim measures, a subsequent SRA for the new upslope cutting was conducted on 6 February 2024 to inform the long-term detailed design.

This project was driven by political will to fast-track disaster recovery, implement interim measures, collaborate closely with the Client, and streamline traditional design processes to meet stringent deadlines. This ensured swift interim and long-term solutions, minimising socio-economic impacts, and restoring connectivity with rural communities.

3 GEOLOGICAL AND GEOMORPHOLOGICAL SETTING

The site is located 8 km from the coastline and is composed of the geological unit Hodgkinson Formation – mudstone (Dh/m) as per 1:100k Detailed Surface Geology (DNRME, 2018). The Dh/m is a low-grade metamorphic rock derived from mainly sedimentary rock deposited into a marine basin; the Hodgkinson Basin, between late Silurian (420 Ma) to early Carboniferous (360 Ma) (Willmott & Stephenson, 1989). This basin was about 10 km thick, 160 km wide and stretched

340 km from Cooktown to Tully in FNQ (Martin, 2000). The Dh/m consists of fine-grained, dark grey, thinly bedded mudstone (locally phyllitic) with subordinate thin to thick bedded quartzo-feldspathic arenite beds, minor chert, and basalt (Bain & Draper, 1997) that have been metamorphosed to micaceous phyllite. The 1:500k Regional Surface Geology (DNRME, 1997) indicates two prominent structures: (i) sheared zones running from north-west to south-east parallel to the river system, as illustrated in Figure 1; (ii) complex gorge system formed through erosion of weaker planes within the Hodgkinson Formation resulting in high defects due to stress relief (Fell et al, 2015). Locally the formation is steeply dipping, highly folded, foliated, and often overturned with prominent cleavage, resulting in high discontinuities (Denaro et al, 2007).

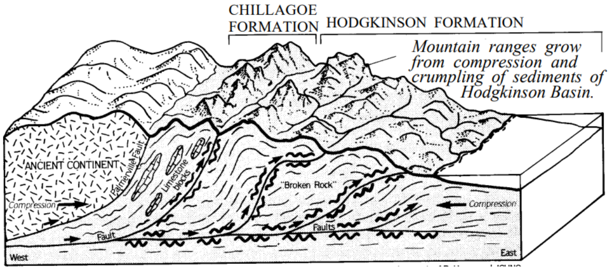


Figure 1. Geological block diagram depicting creation of Hodgkinson Formation (Willmott & Stephenson, 1989).

The outcrops of the unit are generally deeply weathered and consist of varying depths of extremely weathered material, residual soils, and colluvium soils along the mountain slopes. Fresh rock exposure is found mostly in the rejuvenated sections of the river system (Keyser & Lucas, 1968).

4 INVESTIGATION

A combination of intrusive and non-intrusive investigations were conducted at the Site to assess subsurface conditions in conjunction with topographical surveys to assist in the development of a geotechnical ground model for use in the detailed design of the long-term remedial works.

4.1 *Initial aerial inspection*

An initial geotechnical inspection of the Site was undertaken by helicopter on 20 December 2023, shortly after TC Jasper, to assess damage along the entire infrastructure alignment. Figure 2 provides an aerial overview and closeup of the affected section of the riverbank and embankment. Elevated river water levels resulted in catastrophic damage to an adjacent weir structure and extensive scouring of the riverbank and embankment resulting in the exposure of a relic concrete retaining structure understood to be constructed in the 1950s.

Figure 2. Aerial photo of the affected site (taken from a helicopter).

4.2 *Site inspection*

An initial walkover inspection was undertaken on 3 January 2024 to assess the extent of damage to the downslope embankment, document site and slope geological and geomorphological features, identify potential slope instability mechanisms and the associated risks and provide advice on interim risk mitigation measures.

4.3 *Intrusive investigation*

A targeted intrusive site investigation was subsequently completed to inform detailed design. The investigation consisted of three (3) geotechnical boreholes within the corridor drilled to depths between 3.0 m to 15.0 m below ground level (BGL) and twelve (12) dynamic cone penetration (DCP) tests to depths between 0.5 m to 2.9 m BGL, as illustrated in Figure 3. Standard Penetration Tests (SPT) were conducted within the soil strata to obtain *in-situ* soil strength parameters of subsoil materials. Table 1 outlines the typical material description of the major sub-surface material units encountered.

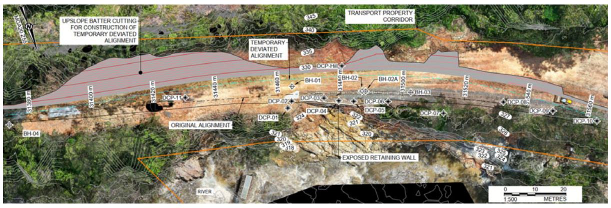


Figure 3. Intrusive site investigation plan for the Site.

Table 1. Summary of sub-surface conditions within the boreholes.

Material units	Depth (BGL)	Description
Unit 1 – Fill	0.0 m – 1.0 m (up to 2.5 m behind the wall)	Variable FILL, consisting typically of SAND and GRAVEL.
Unit 2 – Residual Soil	0.5 m – 3.0 m	Low to high plasticity Sandy CLAY, part of residual soils with stiff to very stiff consistency correlated with SPT results.
Unit 3 – XW to HW Phyllite	1.2 m – 9.6 m	Extremely to highly weathered (XW/HW) PHYLLITE, recovered as low to medium plasticity CLAY to Sandy Gravelly CLAY.
Unit 4 – MW to SW Phyllite	7.0 m – 15.0m (till target depth)	Moderately to slightly weathered (MW/SW) PHYLLITE, medium strength, and indistinctive bedding dipping at 60º to 80º with chert-enriched localised zones with intersecting, iron-stained, sub-horizontal and sub-vertical defects dipping within the range of 5º to 30º and 60º to 90º, typically spaced at 30 mm to 300 mm, with persistence ≤ 5 m, forming a blocky structure.

The reported Rock Quality Designation (RQD) values for Unit 3 ranged from 0% to 38% and for Unit 4 from 40% to 89%, with the average RQD for moderately or less weathered phyllite being over 60%. The strength of phyllite varies depending upon the weathering condition. Point Load Index (PLI) tests showed high variability in strength, with Unit 3 having an assessed axial PLI value of 0.17 MPa and Unit 4 ranging from 0.28 to 1.75 MPa. Diametral PLI results for Unit 3 ranged from 0.06 to 0.99 MPa, and Unit 4 from 0.81 to 4.75 MPa. The ratio of lateral to compressive strength (~3) suggests high horizontal stress environment especially near the valley floor (Fell et al, 2015).

5 HAZARD ASSESSMENT

5.1 *Methodology*

A slope risk assessment (SRA) for the affected embankment and upslope cutting was carried out in February 2024 in accordance with the TfNSW Geotechnical Risk Assessment and Hazard Management (2019) and modified risk matrix per ongoing Sydney Trains Western Line slope risk assessments (2023 – 2026) following the excavation of the upslope cutting to widen the formation in order to provide temporary deviation of the alignment further away from the affected downslope embankment to resume operations.

5.2 *Geotechnical risk assessment – downslope embankment*

The affected section of the embankment was assessed to be ~30 m in length and partly supported by a relic concrete retaining wall measuring ~16 m (l) x 3.5 m (h) x 0.4 m (t) constructed at ~80 degrees. As-constructed details of the wall were unknown; however, a review of available historical imagery indicates the wall was constructed in the 1950s. The wall was observed to be in reasonable condition with no apparent signs of distress except for a local undercutting (void). The embankment slope on either side of the retaining wall was 8 m to 11 m high and grading at ~45 ° to 60 °, locally steepening to ~70 ° to 80 ° near the crest. The lower riverbank consists of highly undulating and stepped bedrock outcrops. Materials exposed in the upper proportion of the embankment consist of a localised wedge fill, underlain by native subsoil units (colluvium and residual soil) and extremely weathered rock (phyllitic), transitioning to low to medium strength or better phyllite. A ground-based photograph of the affected downslope embankment is shown in Figure 4.

Figure 4. Ground-based photograph of the affected section of the downslope embankment and adjacent linear alignment.

The assessment indicated two main hazards: (Hz1) quasi rotation-translational slide failure mechanism and (Hz2) local failure of the existing retaining wall. The risk rating for Hz1 was assessed to be an undesirable high risk (B-, P1), with the risk to loss of life and damage to infrastructure to be Major/Critical. Similarly, Hz2 was assessed to be a tolerable medium risk (C+, P1) with a risk to loss of life and damage to infrastructure to be Minor/Critical.

Various interim measures were implemented to manage the associated risk while the long-term remedial design was being progressed. These included; (i) installation of tiltmeters on the wall, (ii) ongoing implementation of an existing risk management plan and ground-level inspections based on reduced rainfall trigger levels, (iii) adoption of vehicle speed restrictions through the Site and (iv) use of a "spotter" within 30 minutes of planned traffic movements through the Site.

5.3 *Geotechnical slope risk assessment – upslope cutting*

The adjacent upslope cutting and native hillslope directly above was excavated to widen the formation and enable the alignment to be moved further away from the damaged downslope embankment and reduce the risk in the interim and improve access for subsequent treatment works. The new cutting was required to be steeply profiled at between 45 ° to 80 ° degrees, up to a maximum vertical height of 12 m to account for the nearby property boundary and highly sensitive environmental setting. The natural slope above is densely vegetated and inclined within the range of 35 ° to 45 ° to an overall vertical height of about 25 m above the corridor. Given the significant time constraints, excavation of the cutting was carried out based on an observational approach under full time supervision by Geotechnical Engineer.

An SRA was undertaken on the newly excavated cutting using a method similar to the downslope embankment. The assessment was supplemented by surface mapping (derived from photogrammetry completed by ADAM Technology using 3DM Analyst software) to aid in the characterisation of the cutting into geological/geotechnical zones, potential slope instability mechanisms observable in the cutting, the magnitude of failure, associated likelihood and consequence in the event of failure and the associated risk rating. The risk rating was used to develop a catalogue of recommended short-term risk mitigation measures to manage the residual risk at the Site. The excavated face was characterised into four (4) distinct zones, as shown in Figure 5.

Figure 5. Key zones identified across the excavated upslope cutting.

The results of the SRA for each zone are summarised in Table 2 and illustrated in Figure 6. In addition to the risk mitigation measures implemented for the downslope embankment, the following interim measures were applied to the upslope cutting; (i) scaling of loose/ partly embedded rock blocks within the cutting (ii) installation of crack monitoring gauges spanning key rock mass discontinuities; (iii) a camera to record activity on the excavated face; (iv) an automated continuous laser scanner system capturing critical zones, Zone 1 and Zone 3 with 30 minutes alert response time.

Table 2. Summary of SRA of upslope cutting.

No.	Slope description	Potential failure mechanism	Risk rating
Zone 1	Up to 12 m high at 80 ° formed in a blocky weathered rock mass with sub-vertical joints and prominent basal plane dipping at 35° to 40°, abundant soil infilled and iron-stained defects and tree roots.	"Structurally controlled" "wedge" failure sliding along a basal plane, ~10 m W, 10 m H, 1 – 5 m D, estimated failure volume of 100 to 150 m^3 with individual blocks up to 1 m.	(B+) Undesirable (High) **Critical**
Zone 2	Up to ~10 m high at 70 ° formed in residual soil consisting of pale brown clayey sand/ sandy clay.	Localised surficial instabilities (erosion/ slumping) estimated failure volume <10 m^3 mainly soil with isolated blocks up to 0.4 m in size.	(B-, P1) Undesirable (High) **Critical**
Zone 3	Up to 12 m high at ~80 °, locally flattening to ~50 ° at the upper slope, formed in variable weathered rock with closely spaced sub-vertical joints.	Localised block detachments up to 0.4 m in size and potential surficial instabilities within the overlying soil mantle.	(B-, P1) Undesirable (High) **Critical**
Zone 4	Up to 7 m high at ~45 ° formed mostly in soil with XW material across the lower slope.	Erosion and potential rotational slides within the range of 20 to 30 m^3	(B-, P1) Undesirable (High) **Critical**

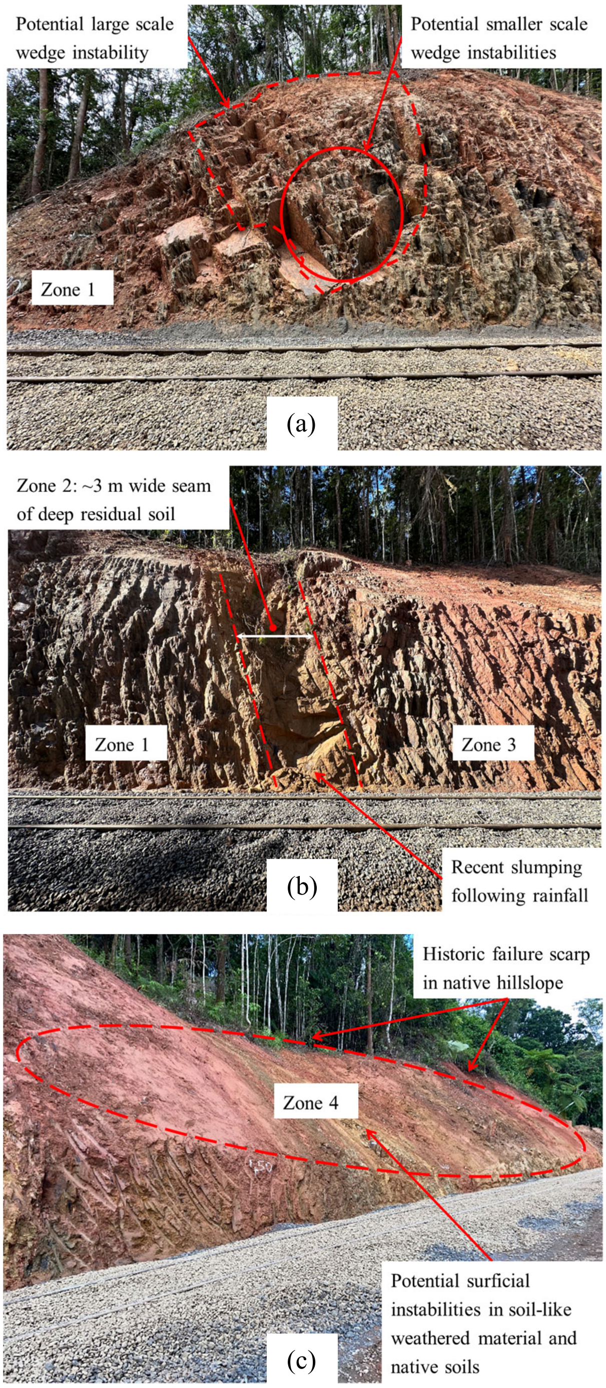


Figure 6. Key zones identified across the excavated upslope cutting: (a) Zone 1 shows potential large-scale wedge instability, (b) Zone 2 shows deep-soil seam, and Zone 3 shows potential isolated block detachment, (c) Zone 4 shows potential surficial instability.

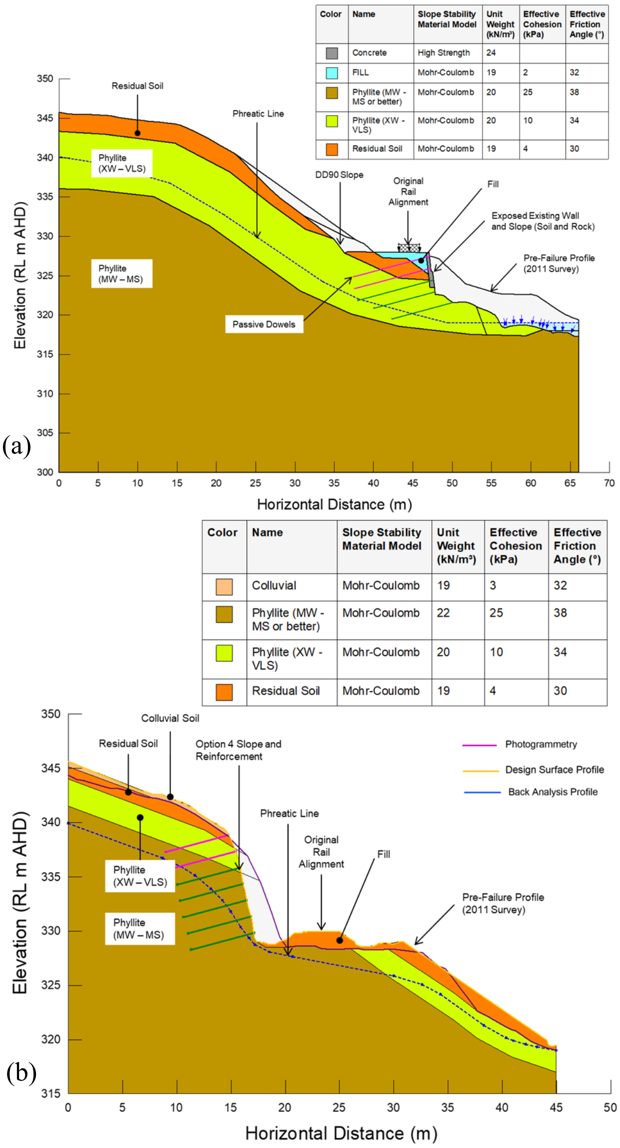
6 REMEDIATION DESIGN

6.1 *Design criteria*

The design of long-term remedial measures was developed to demonstrate compliance with The Department of Transport and Main Roads, Queensland, Australia (TMR) Geotechnical Design Standards Minimum Requirements (GDSMR, 2020). Target factors of safety (FoS) with respect to local and global slope instability were assigned based on a consequence rating "C2" to persons and property, as defined per the procedures outlined in the NSW Roads and Maritime (RMS) Guide to Slope Risk Analysis (2014). A FoS of 1.40 was considered for long-term conditions with a best-estimated groundwater level, while a FoS of 1.25 was considered for short-term transient conditions with a moderately adverse groundwater level.

6.2 *Geotechnical models and parameters*

Geotechnical model and design parameters were developed based on a literature review using Burt Look's Handbook of Geotechnical Investigation and Design Tables (2014), intrusive site investigation, surface mapping, site observation, and back analyses of the pre-existing failure slips based on limit-equilibrium principles. Mohr-Coulomb (MC) equivalent strength parameters for the rock mass were estimated using the Rocscience program RocLab based on the Hoek-Brown (HB) failure criterion (Bertuzzi, 2019). The slope stability assessment was carried out using the Geostudio program SlopeW. A FoS less than or equal to 1 under wet conditions was targeted for back analyses to validate the ground model. A porewater pressure coefficient (r_u) of 0.1 was assigned to all material units to simulate wet conditions. The assumed geotechnical models in section through the downslope embankment and newly excavated cutting with the proposed reinforcement elements is illustrated in Figure 7.



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	Concrete	High Strength	24		
■	FILL	Mohr-Coulomb	19	2	32
■	Phyllite (MW - MS or better)	Mohr-Coulomb	20	25	38
■	Phyllite (XW - VLS)	Mohr-Coulomb	20	10	34
■	Residual Soil	Mohr-Coulomb	19	4	30

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	Colluvial	Mohr-Coulomb	19	3	32
■	Phyllite (MW - MS or better)	Mohr-Coulomb	22	25	38
■	Phyllite (XW - VLS)	Mohr-Coulomb	20	10	34
■	Residual Soil	Mohr-Coulomb	19	4	30

Figure 7. Geotechnical ground model adopted for remedial design of (a) downslope embankment and (b) upslope cutting.

6.3 *Downslope remediation*

6.3.1 *Global stability*

After validating the geotechnical model and material parameters, the stability of remedial design (passive dowels with reinforced shotcrete) was analysed. As per *BS8006: Code of practice for strengthened/ reinforced soils*, the pull-out resistance is considered the lesser of; (i) tendon tensile strength, (ii) bond resistance at grout to bar interface, and (iii) bond resistance at grout to ground interface. The bond strength between the grout and ground interface is considered critical and was determined from the material strength parameters and experience on similar past projects. The design considered two (2) passive dowel types to reflect expected geotechnical conditions over the embedment (bonded length). The corresponding average ultimate grout to ground resistance over bonded length was estimated to be 100 kPa (top two rows) and 200 kPa (bottom three rows). These ultimate pull-out strengths were of similar magnitude to 10% of the average UCS of the rock observed on-site, as recommended by *FHWA-SA-96-069R: Manual for design and construction monitoring of soil nail wall*. A FoS of 2.0 was adopted for ultimate pull-out resistance and tendon tensile strength. The factored tensile strength included an allowance for potential corrosion of the tendon over the design life in the event that protective treatments are damaged during installation.

The design traffic loading considered a 96-tonne diesel locomotive and wagon as per consultation with the Client. The combined design traffic load action was estimated in general accordance with the *AS5100.2: Bridge design – design load*, inclusive of dynamic load effects.

The analyses considered instability mechanisms in the form of surficial instabilities within the pre-existing fill and native subsoils and deep-seated quasi-rotational/translational failures extending into the underlying extremely weathered and low strength phyllite.

A sensitivity analysis was undertaken to assess the resilience of the remedial measures to an array of credible adverse conditions that may eventuate over the design life including; (i) elevated phreatic surfaces within the embankment and site slopes and erosion and (ii) undercutting of the riverbank directly downgradient of the treatment works.

6.3.2 *Remedial design*

The design included 8 m and 10 m long passive dowels (28 mm diameter) encased in an HDPE sheath, installed within a 150 mm drill hole at a nominal staggered 1.5 m grid spacings and fitted with terminations consisting of a steel bearing plate and locking nut. The hard facing design consisted of 200 mm thick shotcrete, with 40 MPa concrete, including SL81 steel mesh and 4 m long 2D500N16 reinforcing bars, orientated horizontally and vertically at each dowel head based on outcomes of the structural assessment. Enhancements to subsurface drainage were incorporated into the design to alleviate excess porewater generation behind the hard facing. Subsurface drainage included 10 m long inclined seepage drains at 3 m spacings, 170 mm wide strip drains placed diagonally at 3 m horizontal spacing and 50 mm diameter weep holes at 3 m grid spacings.

6.4 *Upslope cutting*

6.4.1 *Kinematic stability assessment*

A kinematic stability assessment for potential structurally controlled failure mechanisms within the rock mass (planar, wedge, toppling) was conducted using DIPS 8.0 based on the defect orientation data obtained from the photogrammetric mapping including measurements of the orientation and persistence of key rock mass discontinuities. The computerized discontinuity data was compared with field measurements for validation. The overall dip and dip direction of the cutting was measured to be 80/015 and was assessed to comprise four prominent intersecting joint sets (81/243, 82/163, 26/028, and 75/306), as presented graphical in Figure 8.

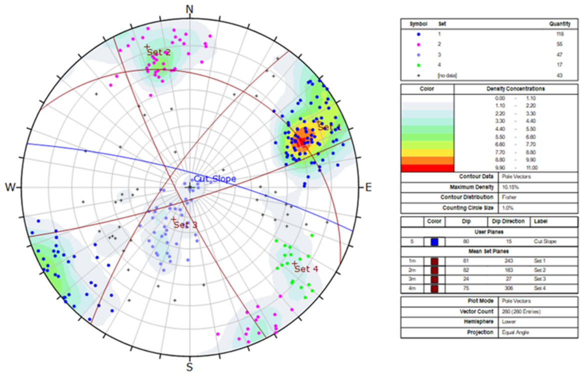


Figure 8. Stereograph of the cutting slope (blue line) and four prominent joint sets (dark red lines).

A limit-equilibrium wedge stability assessment was undertaken using the Rocscience software Unwedge. The analysis used a probabilistic approach that considered the mean joint set orientations and ranges assessed based on photogrammetric mapping data. The analysis aimed to conservatively estimate the largest feasible wedge between the four intersecting joint sets and cutting and assess its stability following installation of the proposed remedial measures.

6.4.2 *Global stability*

Three (3) geotechnical models were developed to represent varying geometric and geotechnical conditions along the 145 m section of cutting. The global stability analysis was undertaken adopting a similar approach to the downslope embankment.

6.4.3 *Local stability*

Local stability mechanisms were evaluated using Ruvolum software developed by Geobrugg, the manufacturer of the proposed pinned rockfall mesh system. Analyses considered local instability mechanisms in the form of simple wedge-shaped failures and minimum 1 m thick slumps (disturbed rock mass or loose soil) parallel to the slope face between the individual passive dowels. The assessment considered dry and wet conditions, reflecting near-surface slope conditions immediately following rainfall. A sensitivity analysis was completed for range of cut slope gradients and credible thicknesses for the layer of material subject to local instability to demonstrate design resilience. Based on the outcomes of the analysis, the proprietary Geobrugg TECCO ® G65/3 pinned high-tensile steel rockfall mesh system with an erosion protection underlay (TECMAT®) was adopted.

6.4.4 *Remedial design*

The design was optimized along the length of the cutting to reflect site conditions. Zone 1, 3 and 4 of the upslope area consisted of 6 m long passive dowels (28 mm diameter) encased in an HDPE sheath, installed within a 150 mm drill hole at nominal staggered grid spacings between 1.5 m to 2 m and fitted with terminations consisting of a spike plate with domed locking nut, rock mesh (Proprietary Geobrugg TECCO G65/3) and erosion control matting (Proprietary Geobrugg TECMAT). The installation of rock mesh consisted of 14 mm diameter steel perimeter ropes, 4 m long perimeter rope anchors dowels, and 2 m long profiling dowels installed perpendicular to the local slope face. A key selection criterion of the pinned rockfall mesh was the ability of the slope to naturally revegetate in the medium to long term, thereby enhancing visual aesthetics.

Zone 2 of upslope cutting consisted of similar 6 m passive dowels (28 mm diameter) at 2 m spacing integrated with 185 mm thick reinforced shotcrete to address the presence of low strength and highly erodible subsoil material exposed in the cut face.

Subsurface drainage was enhanced by proposing the installation of two rows of inclined seepage drains, 8 m long, at 4 m horizontal spacing.

6.5 *Design for durability*

Durability requirements for in-ground elements of the long-term works were developed based on a 100-year design life in consultation with the Client. Durability design was based on a climatic category "C3/Tropical", corresponding to a near-coastal and tropical environment site setting less than 50 km of the coastline with high humidity and rainfall all year round, in general in accordance with AS 4312 Atmospheric Corrosivity Zones in Australia. The corrosion protection requirements consisted of cement grout annulus around dowels, HDPE sheath, epoxy powder coating, hot-dipped galvanised (HDG) and a sacrificial steel annulus of threadbar. The durability of the exposed elements (i.e. pinned rockfall mesh and terminations) was less than 100 years and required periodic inspection and maintenance. The performance of the long-term works is dependent on the actual exposure conditions (atmospheric and corrosivity of subsurface materials) on corrodible components and the performance of protective treatments over the design life. To this effect, the design relied on the implementation of an inspection and maintenance regime over the service life of the long-term works to regularly inspect and undertake repairs when damage occurs.

6.6 *Construction quality control*

The long-term remedial design included construction quality control requirements to validate design assumptions and observe construction processes and methodology. Key features of quality assurance include; (i) ultimate load testing on two (2) sacrificial passive dowels installed within target geotechnical units to verify the ultimate bond resistance between grout-to-ground interface, (ii) acceptance load testing on min. 10% of the total passive dowels, (iii) compressive strength testing of cementitious construction materials, and (iv) full-time surveillance of key construction activities such as material handling, logging dowel drilling, and formwork for shotcrete.

7 PROJECT LEARNINGS

The as-constructed information for the relic concrete retaining wall was not available. Orientated borehole drilling through the wall during site investigation was avoided to prevent triggering further slope instability which may comprise the integrity of the adjacent infrastructure. The boreholes were positioned beyond the influence zone of the retaining wall to manage the risk to the field personnel and plant during the site investigation.

In the absence of site-specific hydrogeological data, a sensitivity analysis was considered with an array of credible phreatic surfaces, elevated water levels within the river system and potential undercutting of the treatment area due to erosion of native materials in the lower riverbank over the design life.

Due to limited safe access to undertake borehole drilling below the embankment and along the crest of the upslope cutting, the design was developed on the basis the aforementioned quality control requirements are undertaken during construction.

8 CONCLUSION

This remediation project demonstrated a highly adaptive, risk-informed, and context-sensitive approach to restoring a geotechnically complex and culturally significant transport corridor within the Wet Tropics World Heritage Area. By integrating rapid emergency works, interim risk mitigation measures and optimised detailed design procedures, the project team was able to develop and implement durable long-term solutions that balanced engineering best practice, under different pressures. The project reinforced the importance of geotechnical presence onsite during high-risk earthworks, proactive stakeholder collaboration, and the application of observational risk mitigation principles for accelerated delivery timelines. These strategies not only restored critical transport connectivity but also delivered a resilient slope remediation solution that is expected to perform reliably under the region's challenging geological, topographic and climatic conditions.

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