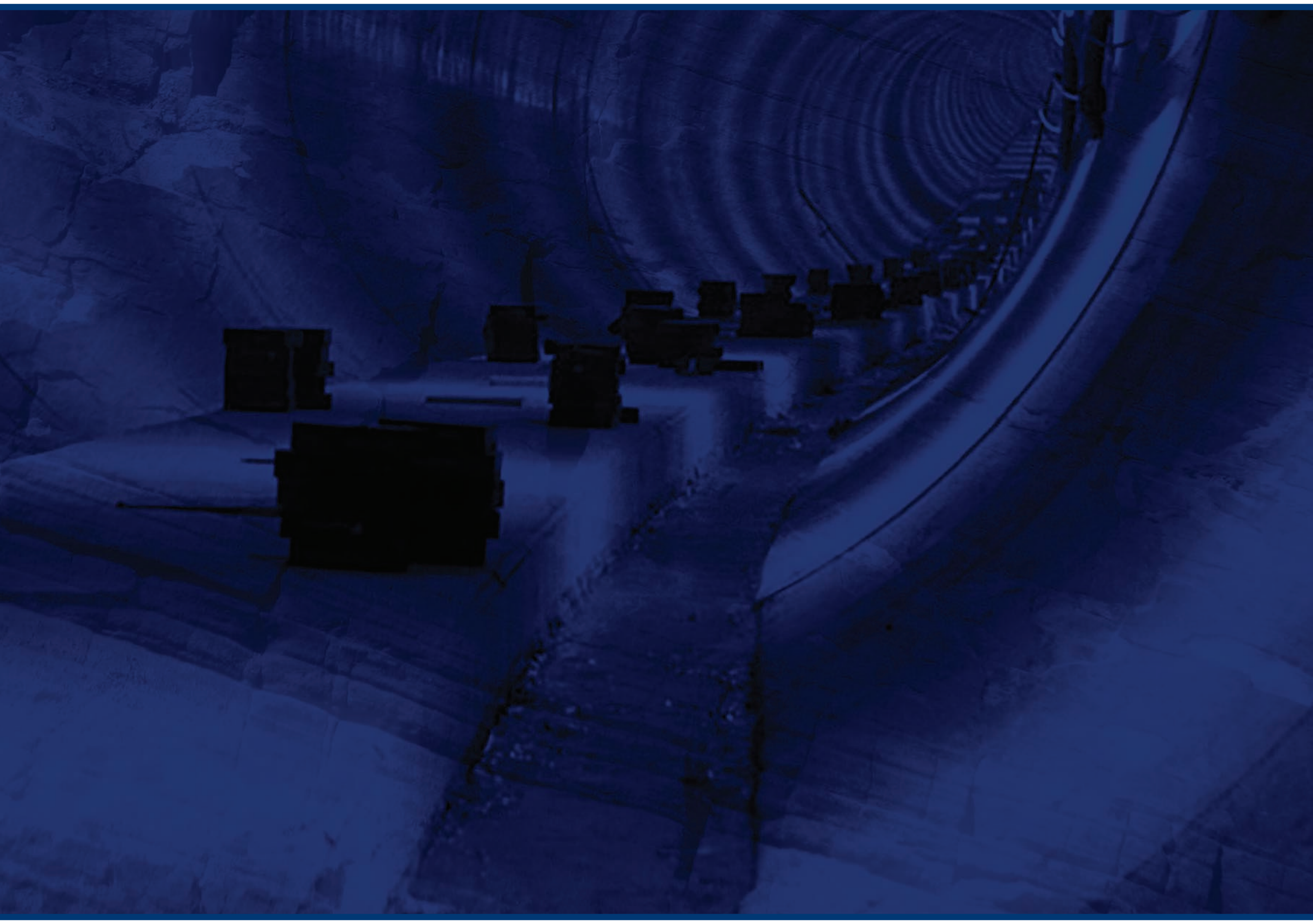
PROCEEDINGS

2018 AUSTRALIAN GEOMECHANICS SOCIETY VICTORIAN SYMPOSIUM

Geotechnics and transport infrastructure

Wednesday, 24 October 2018, 8:00am – 6:00pm
Rydges Hotel, 186 Exhibition Street, Melbourne



AUSTRALIAN GEOMECHANICS SOCIETY
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PREFACE

The Victorian chapter of the Australian Geomechanics Society invited academics and practitioners in the field of geotechnical and ground engineering to attend the 2018 Australian Geomechanics Society Victorian Symposium on ‘Geotechnics and transport infrastructure’ held on 24 October 2018.

In recent years Victoria has seen significant investment in transport infrastructure as part of a plan to manage the demands of a growing population and expanding urban fringe. The construction of Melbourne Metro, a second crossing of the Yarra River, rail and freeway upgrades as well as numerous level crossing removal projects are just some of the major transport projects currently underway in Melbourne and regional Victoria. Many of these projects carry numerous complex geotechnical challenges.

The 2018 Australian Geomechanics Society Victorian Symposium covers a variety of geotechnical challenges associated with transport geotechnics and present overviews of current infrastructure challenges, state of the-art practices, innovation, new research results and case studies demonstrating applications of advanced techniques and cost effective solutions in the construction and design of local transport infrastructure. The Symposium brought together professional engineers, researchers, specialist contractors, regulators, educators and students to share and discuss their experiences on the topic of transport infrastructure and associated geotechnical challenges and applications.

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Soil nails and rock bolts: construction challenges

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ABSTRACT

Soil nails and rock bolts are a cost effective method for long term stabilisation of slopes or near vertical excavations where staged, top-down construction is advantageous compared to other traditional methods of slope retention. This method of ground support can provide near-instant active or passive support against ground instability. This paper presents the challenges encountered on a recent Melbourne infrastructure project during the installation of soil nails and rock bolts, and the solutions adopted to overcome them. In these case studies, passive soil nails and post-tensioned rock bolts were installed in residual clay soils, weathered siltstone rock, and basalt rock across two sites in Melbourne as part of a transport infrastructure project. The challenges encountered throughout the construction process, in particular the variability of ground conditions and construction timeframes shows the need for supervising engineers to be flexible in the development and implementation of the designs. Where designs have been modified additional checks should be completed to ensure the design intent is achieved. Sacrificial testing required as part of the design was not undertaken to failure of the grout to soil bond so the reported results only provide an indication of the mobilised bond stress. Occasionally this was because the test load required to cause failure would exceed the tendon yield stress, however typically the nominated test loads were not high enough for pull out failure. Whilst the testing confirms adequate bond stress for project-specific requirements, the results may not provide information on the ultimate geotechnical performance of the materials being tested.

Keywords: soil nails, bond stress, siltstone, basalt

1 INTRODUCTION

The Hurstbridge Line rail upgrade project involved the removal of two level rail crossings in the suburbs of Alphington and Rosanna, and duplication of an existing section of single track rail in Heidelberg, north-east of Melbourne. The project was delivered by the North East Program Alliance, which comprised the Level Crossing Removal Authority (LXRA), Metro Trains Melbourne (MTM), Laing O'Rourke, Fulton Hogan, and Jacobs. The project involved:

- Lowering of rail into an 800 m long trench under Grange Road, Alphington.
- Widening of existing embankments and cuttings, and duplication of an existing rail bridge and underpass in Heidelberg.
- Construction of a rail viaduct and new train station over Lower Plenty Road, Rosanna.

A number of soil nails and rock bolts were required for both temporary and permanent works at the sites in Alphington and Heidelberg. Retention systems utilised on the project were designed and constructed based on principles outlined in VicRoads Section 683 (VicRoads, 2015).

This paper presents the results from sacrificial testing of soil nails and rock bolts, as well as the challenges encountered during installation.

2 SETTING & GEOLOGY

2.1 Heidelberg Site

The Heidelberg site is located approximately 10 km north-east of the Melbourne CBD between Heidelberg and Rosanna railway stations. The extent of the site is approximately 1100 m and runs in an approximate south to north direction. The site is generally undulating, with the existing rail corridor situated on steeply sloping fill embankments in the south transitioning into a deep cutting to the north.

The adopted design solution involved widening of the existing embankment and cutting which required the use of soil nails and rock bolts to stabilise the slopes from global and localised failures.

The geology of the site generally comprises Silurian-age siltstone and sandstone rock with minor areas of low lying Quaternary-age alluvium (Geological Survey of Victoria, 1959). Geotechnical investigations undertaken for the project encountered fill, alluvium, and residual soil of varying thickness overlying the siltstone and sandstone basement rock. Geotechnical design parameters have been derived based on the geotechnical investigation, soil and rock laboratory testing, and results of back analysis and are specific to particular areas of the Heidelberg site.

Geotechnical parameters for each unit are summarised in Table 1.

2.2 Alphington Site

The Alphington site is approximately 6 km north-east of the Melbourne CBD between the Fairfield and Alphington railway stations. The site is approximately 800 m in length from east to west and is bordered by residential housing to the south and a residential street to the north.

The adopted design solution required a trench extending to a maximum depth of 7.5 m below existing ground level, with temporary excavations extending to approximately 10 m below existing ground level. A major constraint for the site is the narrow rail reserve limits meaning the use of near vertical slopes was required as space was limited. Structural retention systems, comprising soil nails, rock bolts and shotcrete, were required on both sides of the cutting, to fit within the property boundaries.

The geology of the site comprises Quaternary-age Newer Volcanic Group described as basalt rock, with minor scoria and tuff deposits (Geological Survey of Victoria, 1959). Geotechnical investigations undertaken for the project confirm the underlying ground conditions to comprise shallow soils overlying highly weathered to slightly weathered basalt rock, with fresh basalt rock described below the proposed design depth. Rock Quality Designation (RQD) varies across the site, with areas of highly jointed rock and lower RQD (0% to 60%) and other areas of more competent and less jointed rock and RQD tending to 100%. Rock strength is generally high to very high strength as defined in AS 1726 (Australian Standards, 2017) with Unconfined Compressive Strength (UCS) test values ranging between 23 and 139 MPa for rock within the cut.

The ground profile was characterised into three distinct units summarised in Table 2.

Design parameters for the soil materials were estimated from in-situ testing during the geotechnical investigation. For the rock mass, shear strength parameters were derived using RocLab software and the Generalised Hoek-Brown Criterion.

3 DESIGN ULTIMATE BOND STRESS ESTIMATES

Design ultimate bond stresses (TBSult) were determined during design based on published methods and data. For soils encountered at both sites, the ultimate bond stress in clay was determined using an empirical relationship α.Su, where α is a co-efficient between 0.4 and 0.6 (Byrne et al., 1998). For siltstone rock at the Heidelberg site, the ultimate bond stress was determined using the relationship 0.1.UCS (PTI, 1996). For basalt rock at Alphington, an ultimate bond stress of 0.95 MPa for Moderately Weathered basalt was estimated based on published data for the Melbourne region (J. H. MacLeod,1992).

The design ultimate loads that may be applied to the soil nails and rock bolts are much lower than the design ultimate bond stresses presented in Table 1 and 2, which ensures an adequate Factor of Safety is achieved.

4 INSTALLATION AND TESTING METHOD

Retention systems utilised on the project were designed and constructed based on principles outlined in VicRoads Section 683 – Soil Nail Walls (VicRoads, 2015). Soil anchors and rock bolts are not specifically included within the scope of VicRoads Section 683 so design was undertaken in accordance with AS 5100.3 (Australian Standards, 2007).

Testing was undertaken using a hydraulic jack and metric dial gauges to measure displacement. The measuring equipment used during testing was calibrated by a NATA accredited laboratory to measure elongation with a maximum uncertainty no greater than 2%. A typical testing set up is presented in Figure 1.

While the bar and grout properties are generally well defined and reliable, the interaction between the grout and soil or rock is variable depending on ground conditions and the bar installation technique. VicRoads Section 683 and AS 5100.3 requires verification testing to be undertaken on nails and bolts to verify the ultimate bond stress at failure or to confirm that a suitable adhesion factor of safety is achieved.

The design requires that a minimum of three verification tests are to be undertaken for each ground type encountered and for each nail/bolt configuration. Additional proof testing was undertaken on a minimum of 5% of the installed nails to establish the adequacy of the soil nail installation. Sacrificial testing is undertaken at 1.5x the ultimate bond stress, and no more than 90% of the tendon yield stress, and held for a minimum of 4 hours at the maximum nominated test load.

Table 1: Heidelberg site – adopted geotechnical parameters

Unit	**Unit Weight, γ (kN/m3)**	**Effective Cohesion, C' (kPa)**	**Effective Friction Angle φ' (°)**	**Estimated Ultimate Bond Stress (kPa)**
Existing Fill	18	0	35	N/A
Embankment Fill	19	5-9	30	50
Residual Soil	19	5-10	28-30	50-75
XW Rock	20-22	10-26	33-34	75-250
HW-MW Rock	22	50-60	32-37	150-300

RS = Residual Soil, XW = Extremely Weathered, HW = Highly Weathered, MW = Moderately Weathered, SW = Slightly Weathered

Table 2: Alphington site – adopted geotechnical parameters

Unit	**Description**	**Weathering**	**GSI**	**UCS**	**Effective Cohesion, C' (kPa)**	**Effective Friction Angle φ' (°)**	**Estimated Ultimate Bond Stress (kPa)**
Soil	Fill/Residual Soil	RS	N/A	N/A	5	28	75
Ground Type 1	Intact Rock	MW-SW	50	36.7	100	45	950
Ground Type 2	Fractured Rock	HW-MW	35	36.6			950

RS = Residual Soil, HW = Highly Weathered, MW = Moderately Weathered, SW = Slightly Weathered

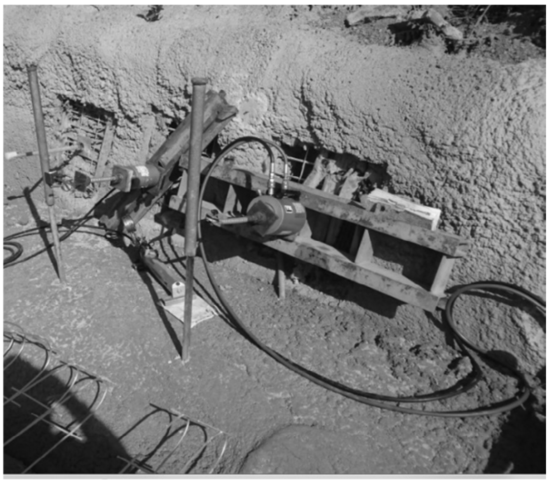

Figure 1. Typical nail testing set up

5 NAIL AND BOLT TESTING

Results from sacrificial testing at both sites are presented in Table 3 using the following calculation for the bond stress:

$$T_{(BSUlt)} = T_{Ult}/\pi dL \qquad (1)$$

Where:
$T_{(BSUlt)}$ = ultimate bond stress
T_{Ult} = test load
d = drill hole diameter
L = bonded length of nail

The calculated mobilised bond stress is likely to be a lower bound estimate as the full area of the nail was considered. In reality this is unlikely as the tensile forces decrease towards the end of the nail, particularly for longer bonded lengths

For a number of the tests, the test load could not be increased due to equipment or structural limitations, however of the remaining tests, the mobilised bond stresses were within the estimated range.

6 INSTALLATION CHALLENGES

The installation of nails, bolts and anchors was met with challenges throughout the construction process. As a result, adjustments to the design were required in order to ensure the design intent and performance was maintained.

6.1 Unexpected Ground Conditions

Given that the project sites are situated within active rail corridors, works were primarily undertaken during infrequent rail shutdowns. The majority of intrusive investigations could only be undertaken outside of the rail corridor. However, as geological conditions can vary both vertically and laterally, this introduces a degree of uncertainty during the design phase.

At the Heidelberg site it was not practical to undertake intrusive investigations in the existing rail embankment prior to the design phase. Back analyses were undertaken to derive suitable soil parameters for the embankment, which were later confirmed during construction.

Similar issues were faced at the Alphington site. In order to undertake verification pullout tests, a trial excavation was undertaken on a nearby parcel of land leased to the project. This allowed the project to obtain design ultimate bond values for the materials expected to be encountered during the bulk excavation works.

During the bulk excavation localised areas of residual soil and extremely weathered rock was encountered to deeper depths than originally anticipated. Whilst generally this would be remedied through the installation

Table 3: Verification test results on nails, anchors, and bolts used on the NEPA project

Site	Material	Nail No.	Hole Diameter (mm)	Length (m)	Bonded Length (m)	Maximum Test Load (kN)	Mobilised Bond Stress (kPa)
Heidelberg	Fill/Residual Soil	SN1	150	15	13.5	230*	36
		SN2		12	10.5	230*	46
		SN4		12	11	230*	44
		SN5					
		SN6					
	Residual Soil	T-1	102	4	1.2	45	117
		T-2		2.7	1.2	80	208
		T-3		4	1.2	45	117
	HW Rock	SA1	150	18	6	650*	230
		SA2		16			
		SA3		12			
	HW Rock	T-6	102	4	2.5	180	225
		T-7					
		T-8					
Alphington	Overburden Soil	SN1	125	4	2.5	74	75
		SN2					
		SN3					
	Basalt Rock	RB1	45	3	3	236^	556
		RB2				220^	519
		RB3				250^	589

* Maximum test load limited to 90% of tendon yield stress
^ Maximum test load limited by maximum jack capacity

of additional soil nails, deep soils were encountered adjacent to a sensitive residential property boundary, preventing the use of design length soil nails. The likely failure mechanism in this area was determined to be localised face instability rather than global instability.

To maintain face stability, shorter rock bolts were installed at a reduced spacing in larger diameter drill holes. By increasing the drill hole diameter, and therefore the grout to soil contact area, the maximum pullout resistance was increased; compensating for the reduced nail length. Stability analyses were undertaken as construction proceeded with the actual soil conditions and the as-built nail pattern to confirm an adequate Factor of Safety.

6.2 Construction Timelines

VicRoads Section 683 requires sacrificial nails to be exhumed to verify the adequacy of the grout cover in the context of corrosion protection. The majority of the construction works for the project were undertaken during a 51-day shutdown, including commissioning and testing. Due to such tight time constraints it was difficult to adhere to this design standard requirement.

Exhumation of sacrificial nails installed in the rail embankment was not feasible given that the embankment carried operational rail traffic. Additional corrosion protection was included in the design to compensate for not exhuming nails. Elsewhere at Heidelberg, glass fiber reinforced polymer (GRP) soil nails were installed rather than galvanised steel nails. GRP nails are not subject to the same corrosion protection requirements and therefore it was an acceptable risk to the project alliance to not exhume nails. Exhumation of rock bolts installed at Alphington was also not feasible, however non-destructive testing (NDT) was undertaken to assess the adequacy of the grout coverage.

6.3 Rock Overbreak

During the trial excavation at Alphington, it became apparent that excavation in the basalt was generally controlled by defect spacing of the rock mass. It was anticipated that significant overbreak would be encountered during the main works. Line drilling, comprising 50 mm diameter holes were drilled to a maximum of 10 m depth at 0.5 m horizontal spacing in an effort to control the extent of overbreak. At the same time, designs were developed to provide construction engineers with solutions to build out areas of significant overbreak with additional mesh and shotcrete.

Despite the efforts of line drilling, significant overbreak of rock occurred during bulk excavation under tight timeframes at Alphington and resulted in caving up to 1000 mm beyond the design face of the shotcrete wall. The largest voids were generally confined to the upper portion of the excavation where the rock mass was blocky. It was quickly apparent that it was not feasible to build out overbreak of that extent due to the weight of the additional shotcrete.

To maintain the design bonded length of the bolts, each bolt was installed and anchored onto the rock face as per standard practises. However, this led to difficulty for the installation of SL81 shotcrete mesh as the rigid mesh could not be contoured to suit areas of overbreak. Modified lacer bars were used to tie the bolt handle plate to the SL81 shotcrete mesh, providing a rigid connection between the anchor plate and the shotcrete mesh. The use of lacer bars to maintain rigidity ensured that the design bond length could be utilised to retain the basalt rock.

In areas where overbreak was limited to approximately 500 mm, the voids were built out in multiple layers of shotcrete and mesh before applying the main shotcrete face. The potential for the shotcrete to slump was reduced by allowing the layers of shotcrete to cure before continuing. Where overbreak was more significant, the protruding rock was scaled back to contour the rock face and provide more gentle transitions. An example of the overbreak encountered, and modified bolt plate is presented in Figures 2 and 3.

7 DISCUSSION OF TESTING

The results from sacrificial testing shows that pull out failure was not reached in any of the tests undertaken. The reported mobilised bond stress therefore only represents the value achieved in testing rather than the full available bond stress.
In some of the tests, in particular those undertaken in competent siltstone and basalt rock, the nails were not loaded to failure as the maximum test load applied was limited at 90% of the tendon yield stress. Exceeding 90% of the tendon yield stress may result in rupture of the tendon rather than failure of the soil to grout bond. Not only does this present a safety risk to those undertaking the test, but the test results for ultimate geotechnical strength would not be obtained.

Figure 2. Example of overbreak in basalt rock

Figure 3. Example of modified lacer bar and mesh

Although the results do not provide suitable data to determine the ultimate bond stress of the ground at failure, all the tests undertaken on the project exceeded the design ultimate loads and therefore meet their intended purpose.

This brings into question the relevance of undertaking sacrificial tests at 1.5x the ultimate bond stress when working loads would never exceed the design ultimate bond stress under limit state design.

What is not clear from the design standard is whether testing is to be conducted at the ultimate bond stress with the purpose of failing the soil to grout bond, or to test at 1.5x the design ultimate bond stress. An update to Section 683 was released in July 2018 to clarify the test load requirement. Under the latest update, design verification testing is to be undertaken at the design ultimate bond stress (VicRoads, 2018).

8 CONCLUSION

The challenges and ground variations encountered during construction required further modifications to the design, irrespective of a rigorous design and planning process involving both design and construction engineers. This is because it is usually not practical to capture all possible varying scenarios that may be encountered during construction. As such, it is important to maintain design flexibility and adapt for actual construction conditions.

The trial excavation undertaken at Alphington gave designers insight into the behaviour of the rock mass and allowed for modifications to the design to be developed to accommodate for potential overbreak. It was anticipated that line drilling would prevent excessive overbreak. However, our experiences during construction show that line drilling did little to control it when work was required under significant time constraint. Line drilling may be more effective at closer spacing; however, this may become uneconomical.

Both design and construction engineers need to be able to react to the conditions on site and make changes to the design in order to meet the actual conditions, but it is equally important to verify the adopted solution is fit for purpose (preferably with alternative design scenarios before construction commences).

Site access was a major constraint in undertaking geotechnical investigations during the design stage of the project. Project teams should liaise with key stakeholders and landowners to obtain early access. For example, overnight rail corridor shutdowns, colloquially known as “after last [train] before first [train]” (ALBFs) can provide brief access to the rail corridor to undertake geotechnical investigations, rather than waiting for infrequent weekend occupations.

Designers should also consider undertaking sacrificial testing using a larger diameter tendon with a higher yield stress, as well as a shorter length (approximately 3 to 4 m for soil nails). This would allow for tests to be undertaken to failure without risking elongation of the tendon or exceeding jacking capacity.

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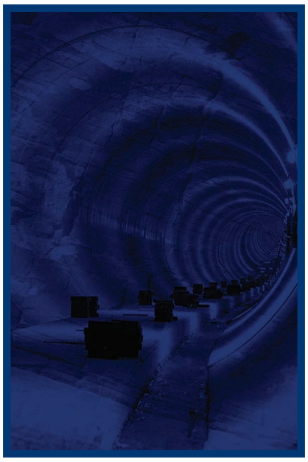

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