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Anchor investigation in weak, soft, mudstone to assess the impacts of flush type and potential of underream methods

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ABSTRACT

When installing anchors in weak argillaceous rocks selecting appropriate ultimate grout to ground bond strength parameters can be challenging, yet critical to ensure safe and rational anchor design. Literature notes groundwater and/or drill flush type can influence the ultimate bond strength due to water softening effects. One tool which can be deployed to increase the capacity of an anchor in weak rock is underreaming, locally increasing the diameter of the anchor fixed length. This paper summarises anchor investigation tests undertaken in the Mount Messenger Formation in North Taranaki. It compares the ultimate capacities of straight shafted anchors drilled with air and water flush, in addition to an underream anchor. The data presented may support anchor practitioners working in similar 'papa' lithologies, or equivalent Late-Miocene soft rocks in New Zealand and internationally.

Keywords: anchor, investigation, mudstone, underream, water-softening.

1 INTRODUCTION

1.1 Project requirements

Mount Messenger is a remote area 50 km north-east of New Plymouth in Taranaki, NZ. The existing section of State Highway (SH3) in Mount Messenger is steep, narrow and winding. The Te Ara o Te Ata: Mt Messenger Bypass project provides a new 6 km upgrade improving safety and resilience. A temporary cableway is required to provide early access for critical path construction activities. This is a significant engineering feat with heli-access constraints and anchor demands of approximately 4.5 MN.

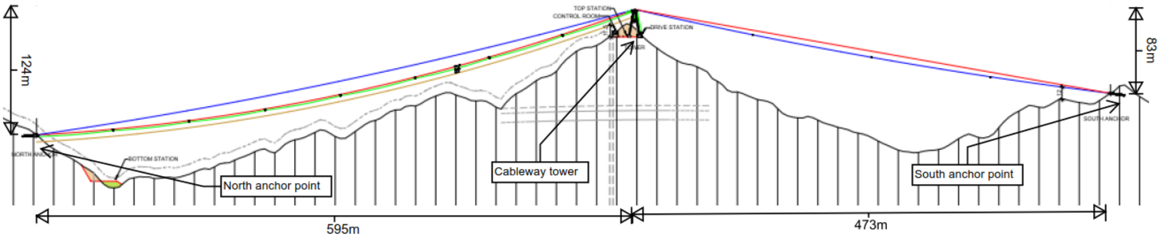


Figure 1: Cableway general arrangement with the north anchor block, the south anchor block and the tower.

An investigation was carried out to confirm both the ultimate bond strength parameters appropriate for design and the constructability of the anchors with the plant and equipment proposed.

1.2 Ground conditions

The host rock in the area is known as the Mount Messenger Formation (11-9 Ma). These typically soft rocks include a continuum of silty, fine grained sandstones to silty mudstones. Bedding is typically thick to massive dipping gently (around 2 to 4°) towards the west.

Table 1: Summary of typical engineering parameters of the Mount Messenger Formation (MMF).

Bulk density (t/m^3)	**UCS (MPa)**	**Moisture content (%)**	**RQD %**	**GSI**	**RMR**	**E (GPa)**	**SPT (N value)**
2.1 to 2.25	1 to 5	13 to 22	75 to 90	70 to 80	41 to 60 (fair rock)	0.1 to 1.0	>50 for 15 to 40 mm

There is little published data for anchor testing in similar 'papa' lithologies in the Taranaki area, or equivalent Late-Miocene soft rocks in New Zealand, and internationally.

2 LITERATURE REVIEW

A literature review was undertaken to assist in designing the anchor investigation in the MMF. Part of this is presented below.

The simple design equation for calculating the ultimate capacity of an anchor in rock is:

$$T_{ult} = \pi \cdot d \cdot L_{fix} \cdot \tau_{ult} \quad [Eqn.\ 1]$$

where: d = anchor diameter, L_{fix} = anchor fixed length and τ_{ult} = ultimate bond stress.

Equation 1 above assumes a uniform load distribution at the grout-ground interface and ignores the progressive debonding phenomena which occurs on longer anchor fixed lengths.

Littlejohn (1980) found that the bond mobilised at the grout-ground interface is unlikely to be uniform unless the rock is soft and/or the modular ratio (E_{Grout}/E_{Rock}) is > 10.

Progressive debonding occurs due to differing elastic properties of the tendon, grout and ground. Barley (1995) notes this phenomenon to be negligeable in short anchors and only relevant to longer fixed lengths. The mechanism of progressive debonding is depicted in Figure 2 below.

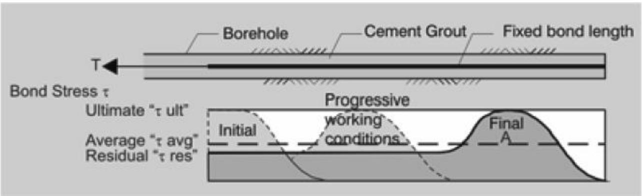


Figure 2: Development of bond stress distribution along a fully bonded fixed anchor length (Barley, 1995).

Numerous authors have undertaken research and proposed efficiency factors to account for progressive debonding on anchors with longer fixed lengths. However, efficiency factor studies are generally limited to investigations of anchor performance in sands and clays.

$$T_{ult} = \pi \cdot d \cdot L_{fix} \cdot \tau_{avg} \qquad [Eqn.\ 2]$$

Minor modification to Equation 1: $\tau_{avg} = \tau_{ult} \times f_{eff}$ = average bond stress mobilised along fixed length at the ultimate load.

Water softening can occur in fine grained argillaceous rocks resulting in reduced grout to rock bond strength. Barley (1988) and Weersinghe (1993) noted water to cause softening of borehole walls in mudrocks. The standard BS 8081 (2018) notes the time between drilling and grouting to be of vital importance in materials susceptible to water softening.

Weersinghe (1993) in his study on behaviour of anchorages in weak mudstone notes there are two ways to enhance the capacity of weak rock anchorage systems:

1. Distribute load in fixed length uniformly e.g. SBMA or multi-stage anchors.
2. Increase mechanical interlock at grout-ground interface e.g. underreams.

Weersinghe & Littlejohn (1997) found a single underream in weak mudstone increased anchor capacity by 145% in comparison to that of a straight shafted anchor. BS 8081 (2018) provides the following equation to calculate the ultimate grout-ground resistance of underream anchors:

Ultimate geotechnical capacity = side shear underream + end bearing + side shear (no underream).

$$R_{GG;calc} = \pi D L_{fixed(U)} c_s + \frac{\pi}{4}(D^2 - d^2) N_c c_b + \pi d L_{fixed(NU)} c_a \qquad [Eqn.\ 3]$$

Where; D and d, are diameter of straight shaft and enlarged lengths, $L_{fixed(u)}$ and $L_{fixed(Nu)}$ are fixed lengths of underream and non-underream lengths, c_s c_b and c_a are average bond strength(s) and Nc is the bearing capacity factor.

3 ANCHOR INVESTIGATION

3.1 Aim

The aims of the anchor investigation were:

1. Confirm the ultimate grout-ground bond strength of a 200 mm diameter bore in the MMF.
2. Drill anchors with air flush and water flush to investigate impact on bond strength.
3. Install an underream to assess suitability of this method for anchoring in the MMF.
4. Drill anchors with a helicopter portable rig to confirm constructability.

3.2 Construction and testing

Three test anchors were installed in April 2022 using a rubber tracked mini skid steer (Vermeer S925TX) with a drill mast attachment (combined weight of 1.5t). Two Atlas Copco (XAS400) compressors provided air flow (800 CFM) to the drill string. Where water flush was used it was delivered at 60 to 80 L/M (Litres per Minute). All three anchors were drilled 15° below the horizontal with down the hole camera inspections completed prior to installing Titan Ishbeck (103/51) bar tendons. All anchors were grouted using a hydraulic powered twin bowl shear mixer grout plant within 3 hours of the fixed length being drilled and flushed clean. Investigation testing was undertaken in accordance with BS EN 1537 (2013) using Test Method 1 as outlined in ISO 22477-5 (2018).

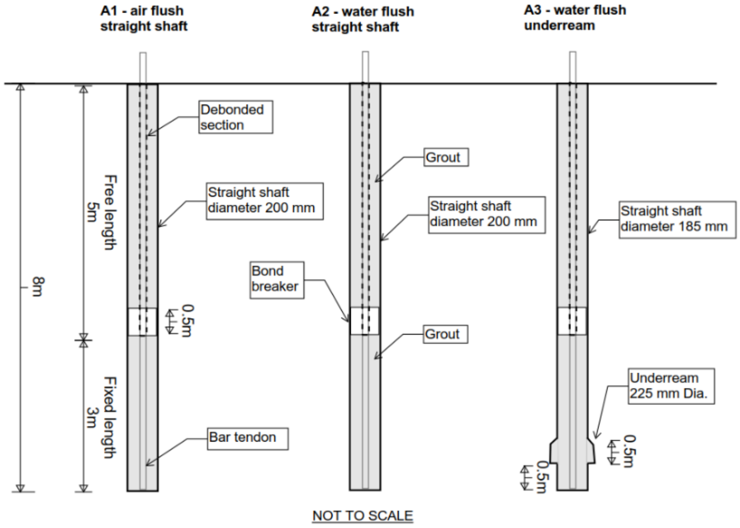


Figure 3: Sketch of the three investigation anchors installed in the Mount Messenger Formation.

Figure 4 below presents photos of the drilling equipment and demonstrates investigation activities.

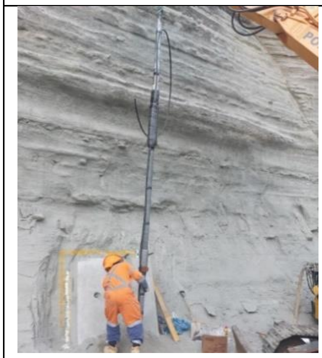
 *a. The 200 mm diameter Poly Crystalline Drill (PCD) bit.*	 *b. The 185 mm diameter drag bit.*	 *c. The 225 mm diameter underreamer with blades 'flared' behind the drag bit.*
 *d. Installing a 7 m long bar tendon section.*	 *e. Grout flowing out of proximal end of the anchor hole.*	 *f. Typical anchor investigation test set up.*

Figure 4: Construction and testing photos from investigation trial.

3.3 Results

Investigation tests were designed to fail at the grout-ground interface over nine incremental load cycles to a proof load (2,928 kN).

3.3.1 Test data

The investigation test data is presented below for the three anchors.

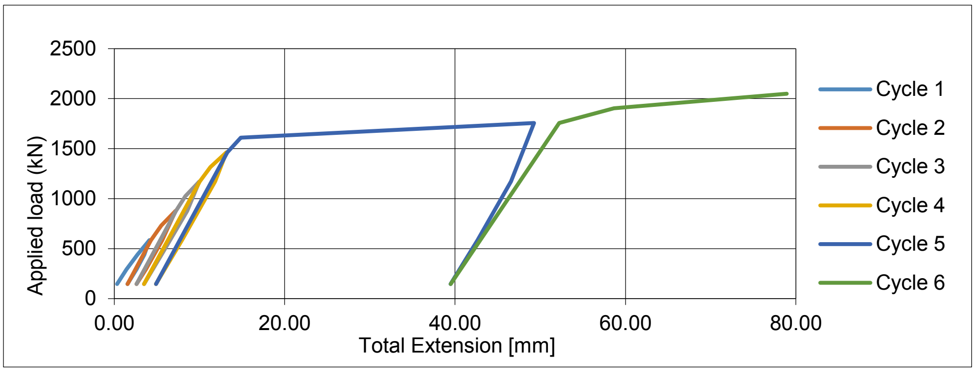


Figure 5: Anchor A1 (air flush) investigation test failed on creep on the sixth load cycle.

Upon building to the peak load increment of cycle 5 with anchor A1, approximately 30 mm displacement occurred before stabilising at the peak load. A release of friction between the bar and whaler is suspected to have caused the excessive displacement as damage to the underside of bar was identified post-test.

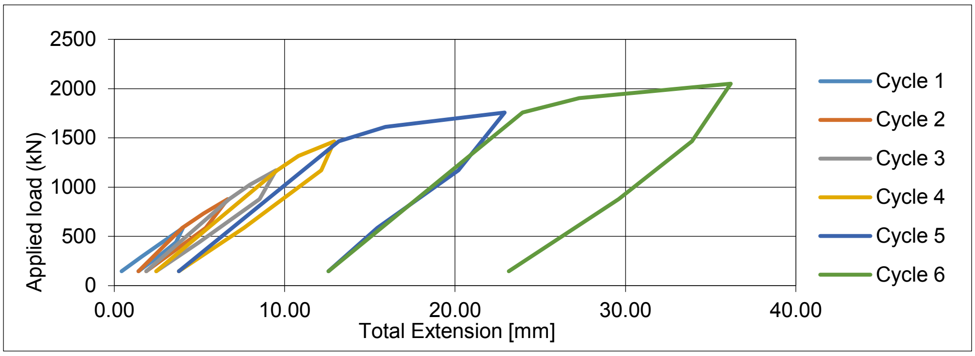


Figure 6: Anchor A2 (water flush) investigation test failed on creep on the sixth load cycle.

After creep failure was noted and cycle 6 was completed, the anchorage was overloaded in a 7th cycle to investigate the peak load and residual capacity. The peak load sustained by the anchor was 2,685 kN with a gradual pull-out noted (no sudden release). After approximately 50 mm of additional displacement, the residual load recorded was 2,600 kN. The load-displacement readings from the overload test are not presented in Figure 6.

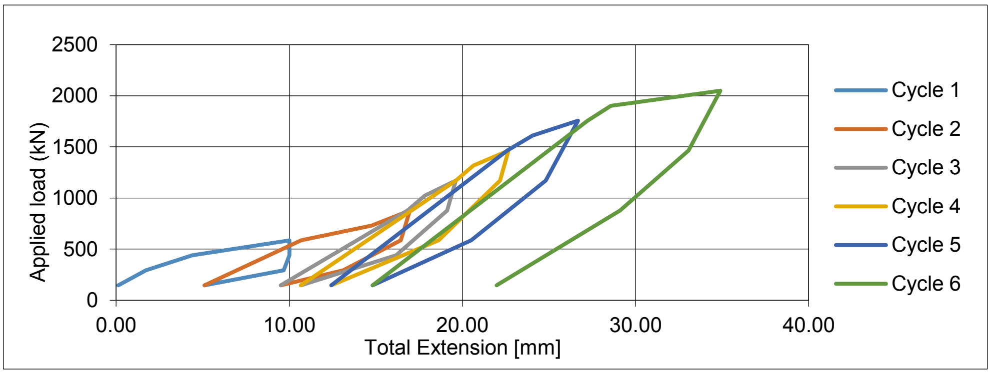


Figure 7: Anchor A3 (underream) investigation test failed on creep on the sixth load cycle.

Issues with the test set-up on Anchor A3 resulted in irregular displacement readings in cycle 1 to 2. The waler did seat properly until higher loading, as evident in Figure 8 (cycle 3 onward). After creep failure was noted in cycle 6, anchor A3 was overloaded in a 7th cycle with a peak load of 2,685 kN held. This was followed by a sudden release and drop in load to 2,300 kN. This residual load of 2,300 kN was held by the anchor after an additional 50 mm of displacement. The load-displacement readings from the overload test are not presented in Figure 7.

3.3.2 Data interpretation

Apparent tendon free lengths were found to be generally in accordance with ISO 22477-5 (2018) indicating the anchors had well-formed free lengths. The geotechnical resistance Ultimate Limit State (ULS) criteria for Test Method 1 was used in the creep assessment. A creep (α1) limit of 2 mm was adopted as specified in EC7 (2013). Ultimate loads are determined graphically in Figure 8 below.

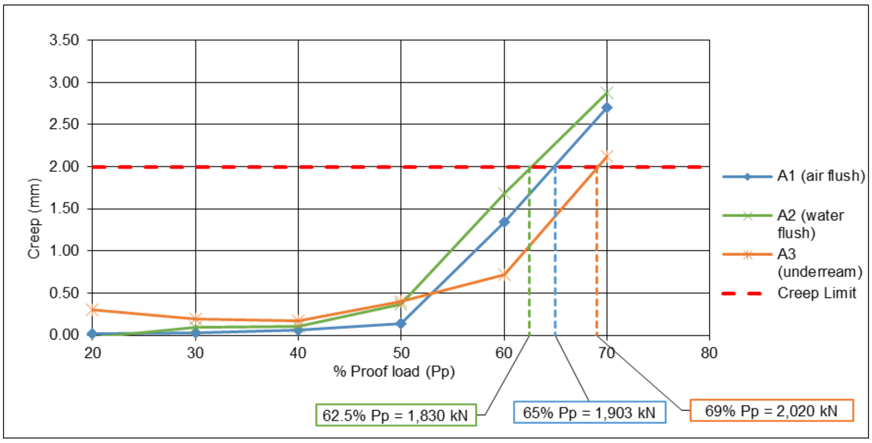


Figure 8: Graph of proof load vs creep showing the geotechnical ultimate capacities derived for each anchor.

A creep displacement vs log time plot is presented below for test cycles 4 (1,464 kN) and 5 (1,757 kN) to compare creep rates at the load cycles prior to failure (cycle 6). The creep rate at 1,464 kN has been extrapolated to estimate creep displacements over a one-year period.

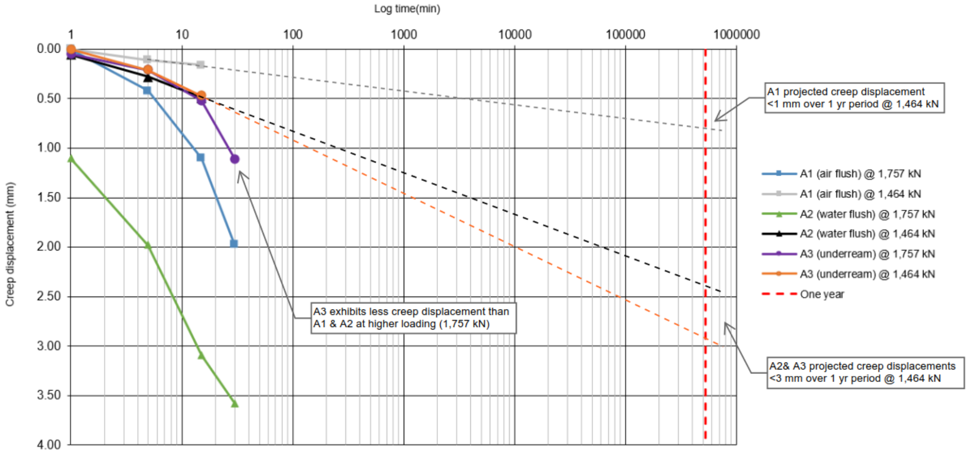


Figure 9: creep displacement vs log-time plot.

The anchor test results are summarised in Table 2 below.

Table 2: Summary of anchorage investigation test results.

Anchor ID	**Bond length (m)**	**Bond area (m^2)**	**Failure mechanism**	**Failure load (kN)**	**Ultimate bond capacity (kPa)**
A1 (air flush)	3.0	1.885	Creep	1903	1010
A2 (water flush)	3.0	1.885	Creep	1830	971
A3 (water flush, underream)	[1]3.0	[2]1.806	Creep	2020	1,118

Table notes:

1. The bond length of anchor A3 comprised a 185 mm diameter, 2.5 m long straight shaft section and a 0.5 m underream length approximately 7.0 to 7.5 m into the anchor.
2. The down the hole camera footage indicates the underream may have had a smaller area due to tapered edge at the proximal end. This would result in a conservative ultimate bond capacity being calculated for A3.

4 DISCUSSION

The investigation tests indicate the principal mode of failure in the MMF is creep. This mechanism may also be dominant in other similar 'papa' lithologies or equivalent Late-Miocene soft rocks.

The underream anchor (A3) demonstrated a higher ultimate capacity than the two straight shafted anchors. Due to a reduced straight shaft diameter the capacity is not readily comparable to anchors A1 and A2. Installing a single 225 mm diameter underream in a 185 mm straight shaft anchor increased the load capacity by approximately 120% (π x 0.185 m x 3.0 m x 971 kPa = 1,693 kN).

Weersinghe & Littlejohn (1997) underream investigation in weak mudstone proved a 145% increase in capacity however it is noteworthy their underream to straight shaft (D/d) diameter ratio was >2.0 (280/130mm). The D/d ratio in this trial was ~ 1.22. A higher D/d ratio is expected to translate to increased capacity through additional end bearing.

The creep log-time plot (Figure 9) was used to compare creep rates at the peak load increment of cycle 4 (1,464 kN) and cycle 5 (1,757 kN). During cycle 4 it is noteworthy that the creep rate when extrapolated to one year, indicates water flush anchors (A2 & A3) may creep 2-3 times more than the anchor drilled with air flush. A longer hold period would have been required to confirm this trend.

The creep log-time plot shows at higher loads (cycle 5) the underream anchor (A3) creeped the least. This indicates the benefit of underreams are realised at higher loads after load-displacement of the proximal fixed length above. Anchor overload investigations (post creep failure) indicate residual loads in MMF to be high.

The straight shafted anchors (A1 & A2) pulled out slowly, whereas the underream anchor (A3) exhibited a sudden release. Using Equation 3 (Section 2) to estimate the ultimate design resistance of the underream anchor the estimated capacity is 2,279 kPa. This is 10% higher than the failure load determined (2,020 kN). There are two inaccuracies which may have contributed to this over-prediction:

1. The underream has a tapered edge rather than the cylinder assumed.
2. Grout-ground capacities (c_a, c_b and c_s) were assumed to be equal (971 kPa).

Based on the more sudden failure mode witnessed with A3 during the trial, the author suspects the load was concentrated in the underream and the 0.5 m long, 185 mm diameter straight shafted section beneath the underream was sheltered from applied loads. Assuming this 0.5 m section provided negligeable contribution to the design resistance prior to failure, the estimated capacity is:

$$R_{GG;calc} = [\pi \text{ x } 0.225 \text{ m x } 0.5 \text{ m x } 971 \text{ kPa}] + [\tfrac{\pi}{4}(0.225^2 - 0.185^2) \text{ x } 42 \text{ x } 971 \text{ kPa}] + [\pi \text{ x } 0.185 \text{ m x } 2.0 \text{ m x } 971 \text{ kPa}]$$
$$= 1{,}997 \text{ kN. (Estimated capacity 2)}$$

Underream anchor (A3) estimated capacity 2 is within 1% of the value proven in the trial.

5 CONCLUSION

The findings of this investigation are strictly only applicable to the MMF. Geological engineering parameters are presented (Section 1.2) for the host rock to enable other anchor practitioners to assess the relevance of this investigation to their projects. Despite the limited number of investigation tests undertaken, the author hopes this case study will help the industry optimise anchor performance in similar very weak, soft, mudstones.

Conclusions are presented as follows:

- Ultimate grout to ground bond strengths in the MMF were found to be similar (within 5%) when using air or water flush drilling techniques [τ_{ult} = 1,010 kPa (dry), τ_{ult} = 970 kPa (wet)].
- Potential water softening effects from drilling with water flush can be controlled in the MMF if bores are flushed with clean water (multiple times) at 70 litres/minute and anchors are grouted within 3 hours of drilling.
- Anchors drilled with water flush may exhibit more creep displacement over time in comparison to those drilled with air flush.
- A single underream installed at the distal end of a 3 m anchor fixed length in the MMF increased the ultimate load capacity by approximately 20%.

6 ACKNOWLEDGEMENTS

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