Full-Scale Pullout Tests of Rock Anchors in Limestone Testing the Interfacial Bond Strengths

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ABSTRACT: Rock anchors are used to stabilise large scale infrastructures. The literature describes four failure modes of rock anchors: (1) steel tensile failure; (2) anchor-grout interface failure; (3) grout-rock interface failure; and (4) rock mass uplift. In this study full scale field tests of rock anchors were performed in a limestone quarry. These tests were designed to test failure mode 2 and 3 with bar anchors, with and without an endplate. The tests of failure mode 2 showed that the shear stress on the anchor-grout interface is highest at the proximal and attenuates towards the distal end at small loads and it becomes approximately uniform at 50% of the ultimate pullout load. The anchors designed to test failure mode 3 had an endplate, they showed that the shear stress on the grout-rock interface was highest at the distal end above the endplate and attenuated upward before slip starts on the interface.

Keywords: Rock anchor, load transfer, shear stress distribution, bond shear strength, field tests.

1 INTRODUCTION

A commonly used safety measure with high capacity to stabilise large-scale infrastructures are rock anchors. They provide a stabilizing force in the direction of the anchor, as well as a confining stress on the ground, which consolidates, strengthens, and improve the mechanical properties of the ground (Hobst and Zajíc, 1983).

The appropriate application of rock anchors demands knowledge and understanding of the behaviour of the anchors, this includes their failure modes, strengths, load, displacement, and steel relaxation characteristics (Brown, 2015). In principle, rock anchors can fail in four ways: (1) tensile failure of the anchor steel or anchor head; (2) anchor-grout interface failure; (3) grout-rock interface failure; and (4) rock mass uplift (Littlejohn and Bruce, 1977). The capacity of an anchoring system is as strong as the weakest mode, therefore the capacity of all failure types must be ensured (Kim and Cho, 2012; Brown, 2015).

There are three main quantities that are of interest in rock anchors, which are the applied load, the anchor head displacement, and the shear stress distribution along the interfaces (Benmokrane et al., 1995). It is common to assume a uniform shear stress distribution along the bonded length in design for ease of calculation, even though it has been shown in many studies that this is incorrect. The shear stress is concentrated at the proximal or top end of the bonded length and decrease exponentially towards the distal end or bottom of the bonded length, as shown in Figure 1, which is derived from the current theoretical and analytical models of shear stress distribution along a grouted anchor (Li and Stillborg, 1999; Liu et al., 2017).

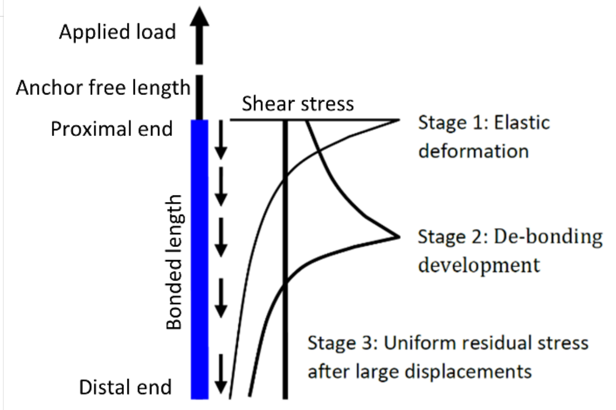


Figure 1. Shear stress distribution along the bonded length of a rock anchor, which can be divided into three stages depending on the applied load and displacement: (1) elastic deformation, (2) debonding development, and (3) uniform residual shear stress after large displacements.

Brown (2015) listed in a review several deficiencies with the current design methods against failure mode 3. The most profound deficiencies are mentioned here. The shear stress is assumed to be uniform, which may lead to wastefully long bonded lengths. The presumptive shear strength values used are often decades-old empirically based. The design does not recognise the progressive failure of the rock grout bond. Several factors that greatly affect the shear resistance, such as the type of grout, borehole roughness and diameter, are not included in the design.

This paper investigates the stress distribution along the interfaces of rock anchors from full-scale field pullout tests of threaded bars and bar anchors. The test objective is to verify the current theoretical models and to show the stress distribution and failure mechanism along the grout-rock interface. The test setups are designed to ensure that bond failure occurs at one of the two interfaces, anchor-grout or grout-rock. The tests were performed in a medium hard rock mass in a limestone quarry. The dimensions of the tested bar anchors were the same as those commonly used for foundation reinforcement in Norway. The load distribution along the anchor and in the grout were monitored by fibre optic cables. It is anticipated that the test results will address some of the deficiencies listed by Brown (2015) and improve our knowledge on rock anchoring, which may lead to improved rock anchor design.

2 TEST ARRANGEMENT AND PROCEDURE

2.1 Test setups

The full-scale field tests planned to investigate the bond failure between the interfaces anchor-grout and grout-rock. Three test setups were designed to test the interfaces through 14 pullout tests. The tests were performed with 64-mm diameter bar anchors in 1.5 m deep boreholes with 140-mm diameter. The anchor steel had Young's modulus of 200 GPa with a nominal tensile strength of 1000 MPa, for a 64-mm bar the ultimate load capacity was 3217 kN. In test setup A, a threaded bar was used to test the interfaces anchor-grout and grout-rock. Test setups B and C used bar anchors with an endplate to test bond failure between grout-rock. In test setup B, a debonding sleeve was used to transfer all the load to the endplate, while for test setup C there were a threaded section above the endplate, which were used to check the effect the thread had on the load transfer to the endplate.

The pullout tests were performed in a medium hard limestone rock mass in the open pit quarry of Verdalskalk AS in Tromsdalen, Norway. The test location was in a corner with strong unweathered limestone away from the active area of the quarry. The limestone is homogeneous with an average uniaxial compressive strength (UCS) of 75 MPa, ranging from 36.6 to 87.8 MPa. The rock mass of the test site was formerly mapped with the Q-system by Pedersen (2014) and it got a rating 32, which is in the "good quality" range of the system. There were found four joint sets in the rock mass, where all of them were planar and rough. It was evident that the rock mass close to the bench crest was blast damaged since the quarry was active.

The boreholes were pneumatically drilled with a 140-mm diameter button bit drill to depth of 1.6 m with a minimum spacing of 1.5 m, and a minimum distance of 3 m to the bench crest.

Optical fibres with fibre Bragg grating (FBG) were used to measure the strains on seven anchors and in the grout for three anchors. FBG is a quasi-distributed fibre-optic sensing technique. There were 20 measuring points on each fibre which were continuously logged at 10 Hz to a computer during the tests.

Concrete platforms sized 50 × 50 cm were casted around all the boreholes to level the surface. The bottom of the boreholes was filled with gravel until the length from the platform surface to the bottom of the borehole became 1.6 m. Then the anchors were installed in the boreholes. Two water-cement ratios (W/C) were used in the tests, 0.42 and 0.55. The 7-days strength of the grout was 45.4 MPa for W/C 0.42. The 28-days strength of the grout was 60.5 MPa for W/C 0.42 and 37.8 MPa for W/C 0.55.

2.2 Test procedure

After the anchors had been installed and the grout had hardened for a week the tests started. First a 5-cm thick 50 × 50 cm steel plate was lifted onto the platform to distribute the load from the hydraulic jack evenly on the concrete platform. Then the jack was lifted onto the anchor. The jack had a capacity of 3500-kN with a stroke of 300 mm.

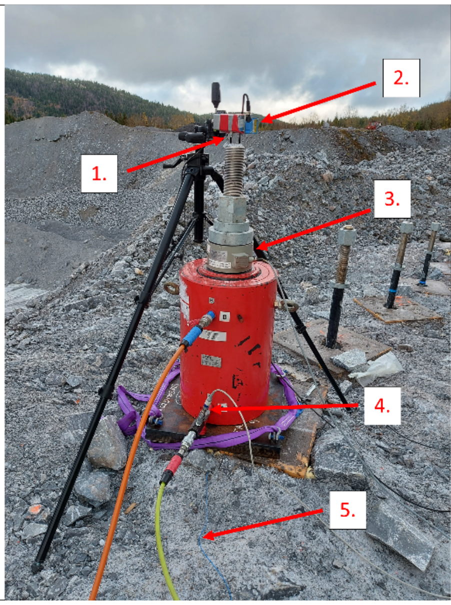
The anchors were instrumented during the tests. A 1500-kN load cell was placed at the head of the rock anchor to measure the pull load applied. Next to the platform on the solid ground was a tripod placed which had a thread extensometer (LVDT) and laser displacement meter attached. These were used to measure the anchor head displacement and settling of the jack. When all instrumentation had been installed, the testing commenced as described in Figure 2. The testing procedure of the first two anchors was not used for the rest of the anchors since the hydraulic jack could not hold a constant pressure and the load started to drop when the pump was stopped.

Procedure first two anchors:

1. **100 kN load steps. Each load step held for 3 to 5 minutes.**
2. **The load steps were reduced to 50 kN when the load-deformation curve started to bend.**
3. **Loading-unloading cycles were done at some loads to study the hysteresis behaviour.**
4. **After failure, the anchors were pulled until the remaining load capacity was low enough to be pulled out by an excavator.**

Procedure rest:

1. **Loaded monotonically at a load rate of 5-10 kN/s until the ultimate load capacity was reached.**
2. **After failure, the anchors were pulled until the remaining load capacity was low enough to be pulled out by an excavator.**



Legend:

1. **LVDT**
2. **Laser displacement meter**
3. **Load cell**
4. **Pressure gauge**
5. **FBG-cable**

Figure 2. Test setup of an anchor with the testing procedure for all pullout tests.

3 RESULTS AND ANALYSIS

Figure 3 shows load-displacement curves of four representative anchors, with at least one anchor from each of the three setups. The figure presents the load as load per metre bond for ease of comparison since the bond length of the anchors varied.

The load-displacement pattern of the A-anchors were similar. The peak load was reached at a small displacement, around 5 mm. Then the load oscillated periodically, with around 4 mm displacement between each peak, for 5 of the 7 anchors. The 4 mm displacement between the peaks are equal to the tooth spacing on the anchor thread. The oscillating behaviour seems to be caused by slip of the anchor thread in the grout, which indicates that failure occurred at the anchor-grout interface. The last two A-anchors had no oscillations post peak, which can be interpreted that slip was along the borehole wall and therefore failure occurred at the grout-rock interface.

The B- and C-anchors had similar load-displacement patterns. All the anchors with an endplate have a nonlinear behaviour in the pre-peak stage. Two anchors had very short bond length, which resulted in fragile and unreliable post-peak behaviour, these were excluded from the analysis. The other B- and C-anchors had similar post-behaviour. The load remained at a level slightly lower than the peak load for large displacement, which can be described as a high toughness in these anchors.

The average bond strength of the anchors was calculated based upon the interpreted failure mode. These calculations are not based upon slip only, as there also were some crushing of the grout in the tests. The anchor diameter of 64 mm and borehole diameter of 140 mm were used in the calculations of the average bond strength for anchor-grout and grout-rock failure, respectively. The test results and the average bond strengths are summarised in Table 1.

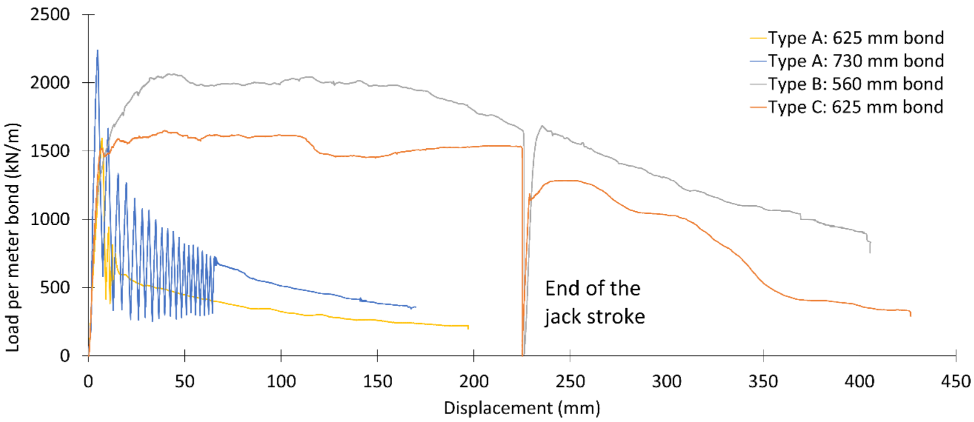


Figure 3. Load per metre bond and displacement of four representative anchors of the three anchor types with similar bond length. The load drops around 225 mm for the B- and C-anchors is the end of the jack stoke.

Table 1. The ultimate loads and average bond strengths of all anchors.

Anchor no.	W/C	Bond length (mm)	Maximum load (kN)	Mean bond strength Anchor-grout (MPa)	Mean bond strength Grout-rock (MPa)	Interpreted failure mode
1A	0.42	485	1404	14.40	-	Anchor-grout
2A	0.42	523	1240	11.79	-	Anchor-grout
3A	0.42	625	995	(7.92)	3.62	Anchor-grout-rock
4A	0.42	730	1634	11.13	-	Anchor-grout
5A	0.42	1080	1418	(6.53)	2.99	Anchor-grout-rock
6A	0.42	1100	1484	-	3.07	Grout-rock
7A	0.55	415	377	-	2.07	Grout-rock
1B	0.42	315	1084	-	(7.83)	Grout-rock
2B	0.42	560	1156	-	4.69	Grout-rock

3B	0.42	770	1257	-	3.71	Grout-rock
4B	0.55	435	720	-	3.76	Grout-rock
1C	0.42	280	826	-	(6.71)	Grout-rock
2C	0.42	625	1031	-	3.75	Grout-rock
3C	0.42	840	1376	-	3.72	Grout-rock

The axial load along the anchor and in the grout were measured with FBGs in eight of the tests, two fibres broke before the tests. The load distribution along the A- and B-anchors are presented in Figure 4 (a) and (c), while Figure 4 (b) shows the strains in the grout along a B-anchor. These figures are representative for the different anchor types.

At small loads, only the upper parts of the A-anchor are activated. When the load reached around 50% of the ultimate load, the whole anchor length is activated with the highest loads at the proximal end. Close to the ultimate load and in the post-peak stage, the FBG measurements became hard to interpret because of large fluctuations in the measurements. It is likely that the FBG became damaged or disturbed close to the ultimate load.

For the B-anchors, the load was transferred directly to the endplate as the thread was covered with a debonding sleeve. The strains were measured in the grout close to the borehole wall, these were compressive above the endplate, shown in Figure 4 (b). The strains in the grout were highest close to the endplate at attenuated upward. The maximum strain in the grout moved upward with the increasing anchor loading. The measurements are presented as strains since the grout in the borehole got fractured, and therefore were not an elastic material anymore. Qualitatively, a high strain is equivalent to a high stress.

The C-anchors had a similar load distribution as the A-anchors at small loads, see Figure 4 (c). At small loads, the load was highest at the proximal end, and it attenuated towards the distal end. When loads were above 50% of the ultimate load, all the load was transferred to the endplate.

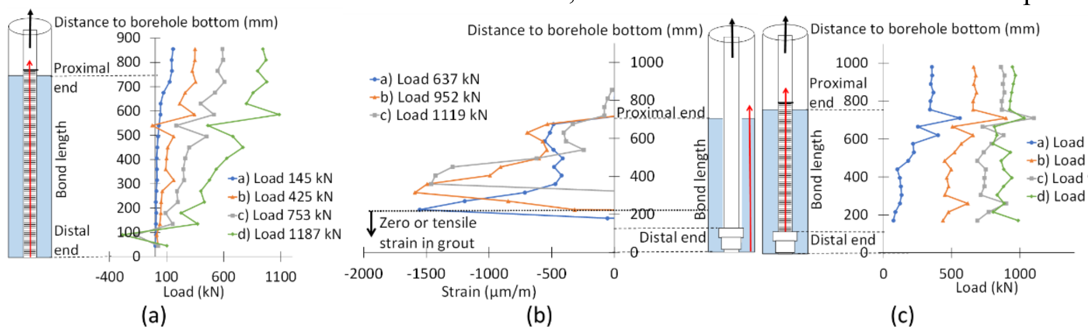


Figure 4. Fibre measurements from three tests: (a) load distribution along anchor 4A; (b) strain distribution in the grout along anchor 2B; and (c) load distribution along anchor 2C.

4 DISCUSSION AND CONCLUSIONS

Pullout tests of three types of rock anchors were conducted in limestone to investigate the bond strength of the interfaces anchor-grout and grout-rock.

The load and strain distribution along the anchor types are shown in Figure 4. The fibre sensors were very sensitive, and the measurements varied, likely due to dilation in the grout as the anchors were pulled out, so the general trends are more emphasised than the exact measured values. For the A-anchors, which are anchored by threads, the axial load was always greatest at the proximal end and attenuated towards the distal end, as shown in Figure 4 (a). The load bearing section of the anchor increased with increasing applied load. The load distribution became approximately linear when the applied load was around 50% of the ultimate pullout load and it remained linear until the ultimate load was reached. The shear stress along the anchor had a similar distribution in the early loading stage, it became approximately constant at 50% of the ultimate load with increasing magnitude with increasing applied load.

The load distribution along the grout-rock interface for the B-anchors, which has an endplate in the distal end of the anchor, are shown in Figure 4 (b). The axial load in the grout was always highest slightly above the endplate and attenuated upward. When the grout column started to slip along the borehole wall, the load distribution became linear. The shear stresses along the interface were highest at the distal end before slip. After slip, the shear stress became approximately constant with the highest magnitude at the ultimate pullout load.

The C-anchors, which had an endplate at the distal end and threads, had a similar load distribution as the A-anchors at small loads, shown in Figure 4 (c). The load attenuated from the proximal end towards the distal end. When the entire threaded section became mobilised, the load in the anchor became approximately constant and all load was transferred to the endplate. The shear stress was highest at the proximal end and attenuated downwards at small loads, it became constant when the whole threaded section was mobilised.

The load capacity of the different anchor types was similar as can be seen Figure 3. The ultimate loads are approximately the same. The A-anchors reached the peak load at a smaller displacement and the load capacity dropped fast after reaching the ultimate load. The B- and C-anchors reached the ultimate load at greater displacement, and they maintained the load at a higher-level post-peak. Therefore, one could say that the endplate increased the toughness of the anchors.

The bond shear strengths of the interfaces are presented in Table 1. The shear strengths on the anchor-grout interface were approximately 25% of the UCS of the grout in the tests, while for the grout-rock interface the bond strength was slightly above 5% of the UCS of the grout in the tests. The bond strength between grout and rock can reasonably be estimated as 5% of the UCS of the weakest of the grout or the rock.

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