The tensile capacity of steel pipe piles drilled into the bedrock

R. M. Sirén
Aalto University School of Engineering, Finland, rosa.siren@vr.fi

L. K. Korkiala-Tanttu
Aalto University School of Engineering, Finland, leena.korkiala-tanttu@aalto.fi

ABSTRACT
The tensile forces affecting pile foundations are usually transferred to the bedrock by rock anchors. If drilled piles could transfer some of these tension forces, foundations could be lighter, execution works easier and the whole system could be more cost-efficient. The aim of the paper is to present the loading tests made for the drilled, grooved pipe piles to define their tensile capacity and to assess their ability to work as tension force transferring structure. The study includes literature review of existing research on tension piles and testing. The review covers the different methods for transferring tensile forces, the geotechnical design of tension piles and the main problems in the design. The testing part focuses on a pull-out test on pipe piles with a grooved surface.

The literature review found several reasons to update current geotechnical design practices on tension piles. It showed that the design methods are conservative and not at an optimal level of precision. In addition, several recent studies have examined the tensile resistance of piles, revealing three main factors affecting bond strength: the relation between the diameter of the drill hole and that of the pile, the roughness of the steel surface and the quality of the grouting. Furthermore, the stresses are not distributed uniformly along the length of the pile, but are highest on the top of the pile and lowest at the tip of the pile. Hence, the bond strength cannot be increased by increasing the bond length. The results of the pull-out test proved that the bond strength of the pile is significantly increased by the grooved surface. However, over half of the steel piles could not be pulled out of the bedrock as loading had to be discontinued at the yield capacity of the steel piles for safety reasons. Thus, the actual tensile capacity of the piles remained undetermined. Anyway the grooved pipe piles proved to have a great potential to be used as tensile force transferring structures.

Keywords: Steel pipe piles, drilled pile, tension pile, bond strength, grooving

1 INTRODUCTION

Large, heavy counterweight structures often transfer tensile loads. If drilled piles could transfer some of the tensile loads formed to the structure, the cost savings could be significant. For this reason it is important to investigate what options are available to improve tensile capacity and how much these methods increase the tensile capacity of drilled piles in practice?
A preliminary study about drilled pipe piles under tensile forces was done by Ahomies (2015) as his Master's thesis. The title of the study was: "The grouted and anchored drilled pile in the bedrock". The study was done in cooperation with Finnish Transport Agency and SSAB. The study contained a pull-out test for 15 steel pipe piles which were drilled and grouted into the bedrock. The results showed that the most common failure mechanism of the tension piles was the failure of the bond capacity between the steel and grouting. Based on this, the purpose of the following study (Sirén, 2015) was to investigate methods to improve the tensile capacity and especially the bond strength between grouting and steel. These were

investigated based on literature and a pull-out test. Literature was studied to identify methods to improve the tensile capacity and a pull-out test was conducted to investigate the bond capacity. To be more precise, the pull-out test examined the bond strength of drilled and grouted pipe piles that had cut grooves on the surface.

The pull-out test was arranged in Masku, Finland in the summer of 2015. In the test, 13 drilled and grouted pipe piles were tested. All piles were drilled two and a half meters into the bedrock. The purpose was to test the effect of a grooved pile surface to the bond strength and at the same time to test the total pull-out capacity. Piles were also monitored during the load test. (Sirén, 2015)

2 TENSION PILES

2.1 Literature review

Literature review studied the basic failure mechanisms of tension piles and factors that impact on the bond capacity. The failure mechanisms were bond strength between grouting and steel, bond strength between grouting and rock and the tensile strength of the bedrock. In addition, stress distribution, cracking of grouting and corrosion protection was covered.

The bond strength between grouting and steel pile depends on friction force on pile surface. Because the friction force depends on the normal force and friction factor, the friction can be increased by shaping steel surface and changing the properties of steel and grouting, which effect on normal force on pile surface when pile is under tensile stress.

Jesús Comés et al (2005) tested the bond strength for four different pile types that were grouted in footings. They observed several things from the results. First, by comparing the behavior of smooth surface casing and casing with ribs welded onto the surface they noticed that the casing with welded ribs offered greater bond capacity. Second, the behavior was more plastic with welded casing. Third, they did not find any substantial correlation between bond length and mobilized bond strength. Although bond length did not affect maximum bond strength, it seemed to decrease displacement caused by small loads. The diameter of the drilled hole had a remarkable effect on bond strength. They observed the same effect on the test made for a casing that was cast in a concrete footing. When the diameter of the drill hole was decreased, the bond strength increased. (Jesús Comés et al., 2005)

Brown (2014) addressed the very conservative approach to the failure of rock mass, from the point of view of rock mechanics. Several methods can be used to evaluate the possibility of rock cone failure, but the minority of these are satisfactory. The main problems in this rock cone assumption are the theoretical stress distribution, the failure mechanism of the rock mass uplift, ignorance of the real structure of the bedrock and the constant value for the tensile and shear strengths (Brown, 2014).

The distribution of the stresses was observed to have a high impact on the tensile capacity of the drilled pipe piles. The stresses are at highest on the top of the bond length of the pile and lowest near pile tip. Hence, if the tensile capacity of the pile is increased by increasing the pile length, the stress in the uppermost part of the bond length will increase unpredictably high and it may break the beginning of the bond length. (Sirén, 2015)

This distribution of stresses was observed in several studies. First of all, the study of Jesús Comés et al (2005) found the same behavior for tested piles and the study of Littlejohn (1997) revealed the same distribution for fully grouted rock anchor. Also a study of Charlie C. Li et al (2014) observed the same behavior for rock bolts.

2.2 Design of tension piles

The basic failure mechanisms to determine in geotechnical design of tension piles according to Eurocode are the bond strength between grouting and steel and that of between rock and grouting. Also the rock cone failure needs to be calculated. (RIL 254-2011)

The bond strength between grouting and steel is based on the compression strength of grouting and on the shape of the steel surface.

The bond strength between rock and grouting is based on compression strength of grouting (EN 1997). Hence, the design does not take into account majority of the factors that effect on the bond strength.

3 PULL-OUT TEST

3.1 Test configuration and conditions

Altogether 13 steel pipe piles, which were drilled and grouted 2.5 m into the bedrock, were load tested. The main purpose was to study the bond capacity between grouting and steel, which was improved by cutting grooves on the surface of the piles. For additional information on the test, refer to Sirén (2015).

Loading was done using hydraulic jacks through a load transfer structure. Monitoring was carried out using strain and displacement gauges, and the applied force was monitored using stress gauges. The displacements were monitored from two points on the pile surface and from two points on the rock surface. The strains were monitored from three points on the pile surface. The pile testing was made approximately twelve days after grouting. The structure of the pile is illustrated in Figure 1.

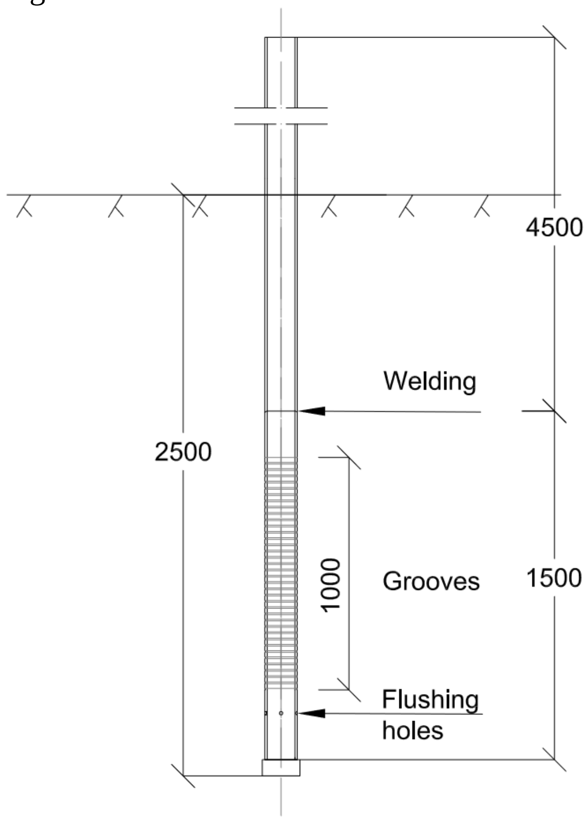

Figure 1 Pile structure

Flushing holes were made at the lower end of the pile for cleaning and grouting the pile. The purpose of the flushing holes was to ensure proper cleaning of the drill hole and even spreading of the grouting. The grouting had to be able to fill the space between the pile and rock.

The test included one pile which was monitored differently to the others. Hereafter, this pile is referred to as the instrumented pile. The purpose of the instrumented pile was to monitor how the tensile force was transmitted along the pile length.

All drilling was done in an exposed bedrock surface. The bedrock surface was close to the ground surface and the clearance of the construction site included a maximum excavation of 0.5 m. The main rock type at the construction site was mica gneiss and the rock was observed to have good quality and only few fractures.

The pile types tested were RD140/10, RDs140/10, RD220/10 and RDs220/10. The small letter “s” refers to the steel grade of S550J2H. Basic steel grade is S440J2H. The RD140/10 pile has a diameter of 139.7 mm and the RD220/10 pile a diameter of 219.1 mm. Two types of grooves were used in the test, shallow and deep. From these variables, the tested pile types are shown in Table 1.

Table 1 Tested pile types

Number of piles	**Pile type**	**Steel grade**	**groove type**
4	RD140/10	S440J2H	shallow
3	RD140/10	S440J2H	deep
3	RDs220/10	S550J2H	shallow
3	RD220/10	S440J2H	deep

Drilling was done using the centralized drilling method and down-the-hole hammer. In both pile sizes, the ring bit diameter was approximately 20 mm larger than the diameter of the pile. Hence, the RD140/10 piles had a relatively larger empty space around the pipe pile. This also meant cleaning and grouting was relatively easier for RD140/10 piles.

The piles were flushed with compressed air immediately after drilling and later with water. Water was conducted to the bottom of the pile pipe through a tube-á-manchette and through the flushing holes outside the pipe. The pile was flushed with water until the water was clean or the pile had been flushed with at least 1000 liters of water.

The grouting was mixed at the construction site. The grouting used in the test was Nonset 50, a cement based dry grout made by Mapei AS. Nonset 50 consists of cement, selected sand and additives, which cause expansion, stabilizing and plasticizing of the grouting. The grouting expands 1-3% before hardening. The maximum grain size of Nonset 50 is 0.2 mm.

Grouting work was started after cleaning the piles. The tube-á-manchette was assembled near the bottom of the hole to ensure the grouting would not fall to the bottom and separate with the water in the hole.

The quality of the grouting was ensured using test samples, which were taken from the grout mass. This was done to verify the intended compression strength of the grouting. The prism samples were taken on each grouting day at the beginning of grouting and when finishing the work.

3.2 Loading and monitoring

Figure 2 shows the load transfer system. The applied force for the piles was induced using four hydraulic jacks. On the pile surface was affixed a console part by longitudinal welds and the force was transmitted from the jacks, through the console to the pile. The reaction force was directed to the bedrock.

The force was monitored using separate stress sensors, load cells. The load cells were assembled between the jacks and pile consoles. Monitoring the load between the jacks and the pile console ensured that all the jacks transferred the same size load to the pile console. Hence, the force that was transmitted through the console to the pile was axial.

Figure 2 Load transfer structure

For each pile, the maximum applied load was based on the maximum axial resistance, which was based on the yield strength of the pile material. The axial resistance was calculated based on the actual yield strength of the steel from the material certificates and the actual cross-section area. If the yield strength of the steel material had been reached, the pile could have failed suddenly. If this had been allowed to happen, the failure of the load transfer structure would have been uncontrollable. In addition, the analyzing of the results would be harder once the yield behavior of the pile is non-linear.

Pile displacements were monitored through a steel band placed around the pile surface. When pile moved in a vertical direction, the steel band also moved. The displacement gauge was attached to the stationary platform and measured the movement of the steel band, from two sides of the pile.

Displacement of the rock surface was monitored to ensure the rock did not move and affect the results. Bedrock displacement was measured from two points around each pile. The first point was at a distance of 0.5 m and second point was 1 m away from the pile.

The strains were monitored for each pile during the pull-out test. Each pile had three strain gauges above the bedrock surface. All three strain gauges were affixed at the same level at regular intervals around the pile surface. The gauges were affixed with glue on grind and cleaned the steel surface.

The strains were investigated more closely from the instrumented pile that had nine strain gauges. The strain gauges were set at three different levels, which each had three gauges. The first level was above the bedrock surface and others were below the bedrock surface. Figure 3 shows the positions of the two lowest levels of the strain gauges. As shown in the figure, the measuring level in the middle was located between the grooved sections.

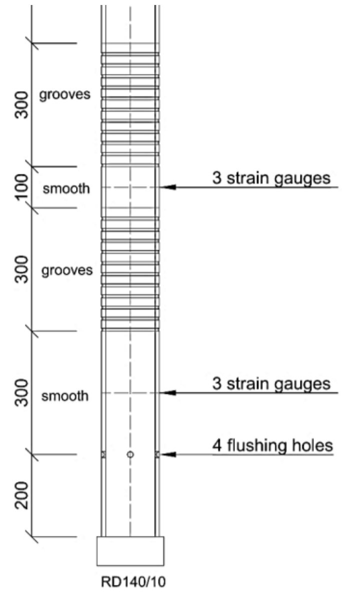

Figure 3 Instrumented pile and the positions of the strain gauges

4 RESULTS

4.1 Bond strength

Only four of test piles exhibited bond strength failure between the grouting and steel. These piles were all RD220/10 piles. With the other piles, the maximum applied load was too low to cause failure. Thus the bond strengths are calculated using the maximum applied load. Therefore also the bond strength is lower for piles with deeper grooves because the maximum applied load was smaller due to smaller cross-section area.

The maximum target load for RD140/10 piles with shallow grooves was 1.6 MN and for piles with deep grooves it was 1.3 MN. The calculated average bond strengths were 1.47 MPa for RD140/10 piles with shallow grooves and 1.20 MPa for piles with deep grooves.

Maximum target load for RD220/10 piles with shallow grooves was 3.0 MN and for piles with deep grooves 2.0 MN. The average bond strength for RDs220/10 with shallow grooves was 1.85 MPa and for RD220/10 with deep grooves it was 1.23 MPa. Three of the piles that had bond strength failure were RD220/10 piles with shallow grooves and one was a pile with deep grooves.

The observed failure point of the bond capacity in RD220/10 piles was sudden and displacement grew varying between 50 mm and 80 mm before re-attachment. Failure point was determined to be at the point where displacement grew at constant tensile force. Re-attachment was assumed to be caused by the ring bit, which was greater than the pile diameter.

Figure 4 shows the bond strengths that were calculated from the test results. Each pile has a small letter after the number which tells whether the groove type was shallow (s) or deep (d). RD140/10 piles are in blue columns and RD220/10 piles in red columns. The piles (7, 8, 9, 13) that had bond capacity failure are shown with a pattern. Pile 13, which was an RD140/10 pile, was the instrumented pile.

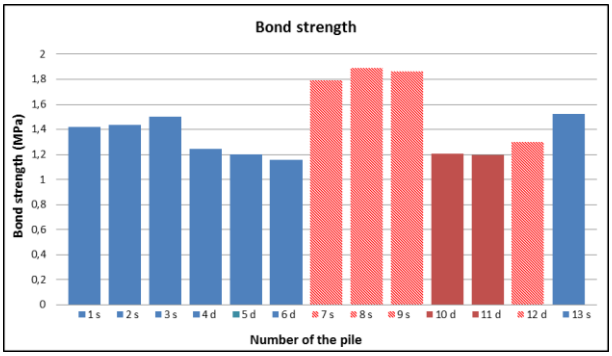

Figure 4 Bond strength between grouting and steel

4.2 Grouting

Average value for compression strength of the grouting was 41.3 MPa. This is calculated according to EN 196-1 (8). The grouting samples can be divided into four batches. Each batch comprised three samples from the mixer and three samples of the grouting that rose from the drill hole. Based on results, the quality of the uplift grout is practically the same as the quality from the mixer and that the required quality was reached.

4.3 Longitudinal strains

The average strains for RD140/10 piles are shown in Figure 5 and that of RD220/10 piles are in Figure 6. The average strains are calculated from the three gauges on the same pile and at the same cross-section. From these figures it is easy to see elastic and plastic behavior of the piles.

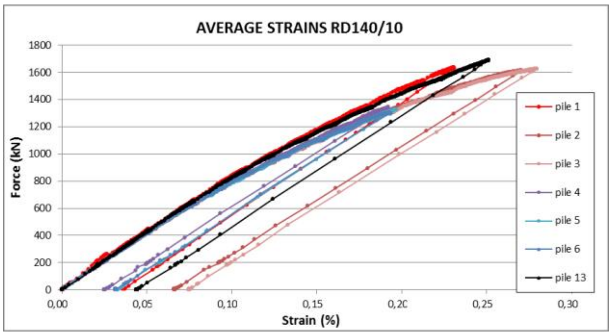

Figure 5 Average strains in RD140/10 piles

The strains are the relatively same for each RD140/10 pile. The piles are colored based on groove type; red curves are piles (1, 2 and 3) which had shallow grooves and blue curves are piles (4, 5 and 6) which had deep grooves. Pile number 13, the instrumented pile, is in black, because it had fewer grooves.

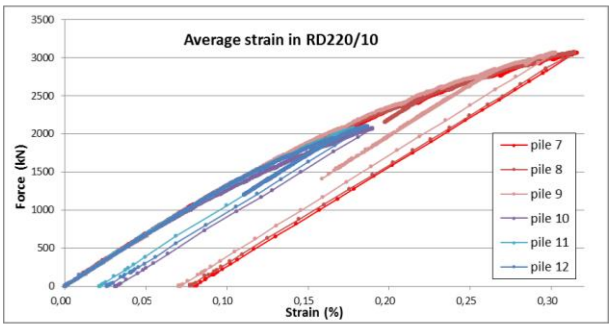

Figure 6 Average strains in RD220/10 piles

Figure 6 shows that the force-strain relation was similar in all RD220/10 piles, although a small difference can be seen between piles of a different groove depth. Piles with shallow grooves are shown in red and piles with deep grooves are in blue. The strains appear to grow with smaller force in piles that had the deep grooves. Groove depth should not affect this because strain gauges were on the smooth part of the pile. The biggest strains were measured in piles 7 and 8. Both had the maximum strain of 0.32%. These were also the piles that had the maximum loads. The behavior in load removal was the same with each pile.

In the instrumented pile, the strain gauges that were at the uppermost level on the pile are marked with U1, U2 and U3, strain gauges that were below the bedrock surface and at the lowest level are marked with L1, L2 and L3. The strain gauges M1, M2 and M3 were below the bedrock surface in between the afore-mentioned strain gauge levels. The maximum strain at the lowest strain gauge level was 0.01%, at the middle level it was 0.12% and at the uppermost level it was 0.27%. Thus, strains did not develop evenly along the pile length. Distance between the uppermost strain gauges and the middle strain gauges was about 1.7 m, and the distance between middle strain gauges and lowest strain gauges was about 0.5 m. Even so, the strains in the uppermost gauges were 2.3 times greater than the strains in middle gauges and the strains in the middle gauges were 8.7 times greater than the strains in the lowest strain gauges. Totally, the strains in the uppermost gauge were 20 times greater than strains in the lowest gauges.

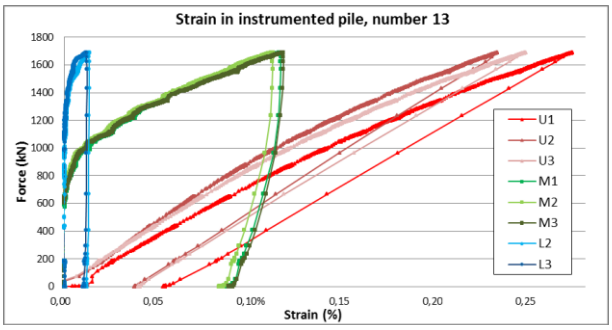

Figure 7 Strain development in instrumented pile, strain gauges at the uppermost level were U1 – U2, at the middle level M1 – M2 and at the lowest level L2-L3

4.4 *Displacements*

The displacement sensors of the pile were about 0.5 m above the rock surface, both at the same level and in the same cross-section. Figure 8 shows displacements for RD140/10 piles and Figure 9 displacements for RD220/10 piles. The maximum displacement of RD140/10 piles was 6.27 mm. The displacements had a greater variation than the strains between pile because the development of displacement is in relation to many factors, like grouting, voids, steel, grooves, etc. The strains above the bedrock surface mainly depend on pile size and steel properties.

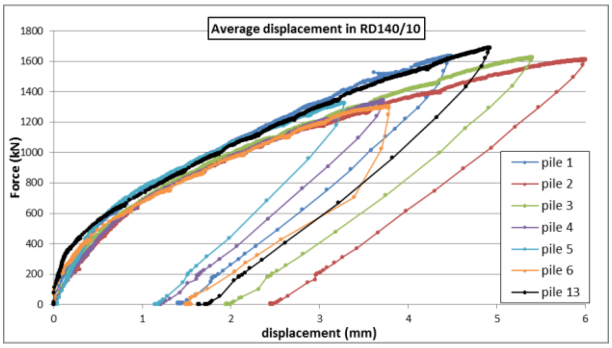

Figure 8 displacements in RD140/10 piles

For RD220/10 piles that were loaded at the failure point of bond capacity, the displacements were larger. Piles 10 and 11, which did not reach the failure load, do not show in the figure due to small displacement. Maximum displacement was 86.1 mm. It occurred at a load of about 2.1 MN. The displacement for piles 10 and 11 were less than 5 mm.

Displacements on the bedrock surface were monitored to ensure that they did not occur. Some displacements were observed near the piles, but displacement was mostly small scale.

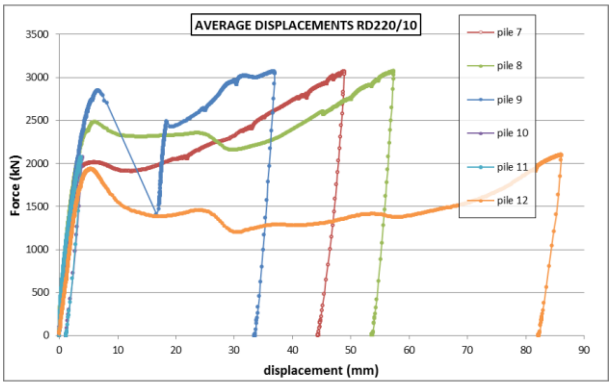

Figure 9 displacements in RD220/10 piles

5 ANALYSE OF THE RESULTS

5.1 *Factors effecting on the results*

The main difference was that piles were drilled straight into the rock surface. Piles are normally drilled into the surface of the ground and through a soil layer. Now the absence of the soil layer simplified several construction phases.

First of all cleaning of the drill hole was easier and more reliable because there was no soil collapsing into the drill hole. Also it was possible to see the mouth of the drill hole. This, in turn assisted flushing of the hole because it was possible to see when the hole was definitely cleaned. The drill cutting was observed to block the drill hole easily, so if there had been a soil layer, a more effective cleaning method might have been required.

The absence of the soil layers helped also grouting work. The quality of uplifted grout could be ensured because it was not mixed with the soil. On the other hand, while the soil layer may complicate the observation of quality, now piles were grouted against a purely open drill hole. Normally, soil would close the upper part of the hole and cause a little counter pressure at the beginning of grouting. This could induce better penetration of grouting to surround of the pile.

Bedrock quality was observed to be good. There were no remarkable fractures or weakness zones that could have had seriously affected to the work. The strength of the bedrock was considered high.

Piles that ended up failing in bond capacity were re-attached to the drill hole after large displacement and the load started to increase again. This re-attachment can be explained by the mechanical bonding of the ring bit. The ring bit is larger than the pipe diameter and after bond failure the ring bit no longer fits through the hole, but remains stuck in the grouting. The ring bit will start to transfer larger loads than the failure load of the bond capacity.

Flushing holes can also affect the results and especially the magnitude of the displacement after bond capacity was exceeded. The flushing holes were made 200 mm above the ring bit. It is possible that the grout did not spread downwards from the flushing holes. Or the grout may have gone downwards, but only for a short distance. This might have caused a gap between the flushing holes and the ring bit. If there was a gap, the pile would have had a chance to move upwards until the grouting was reached. This gap is shown more closely in Figure 10.

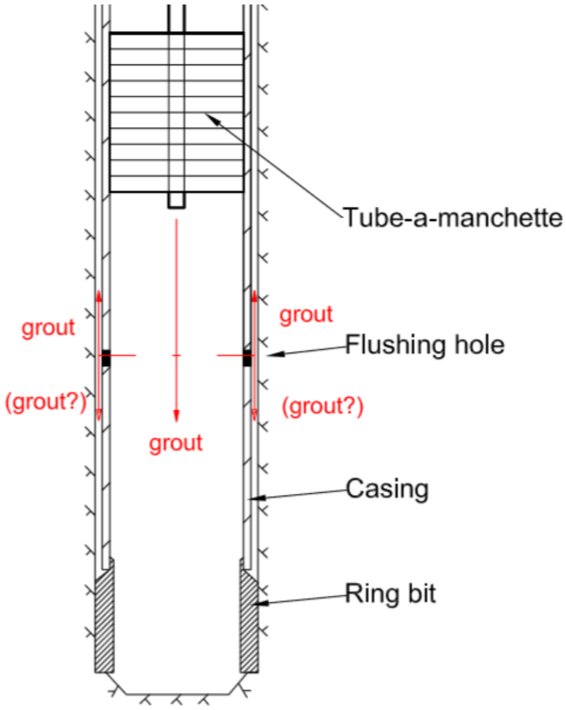

Figure 10. The spread of grouting and the gap between ring bit and flushing holes

One of the problems was that the failure mechanism could not be proved. The failure of the bond strength between the grouting and steel was assumed to be the failure mechanism based on the previous study and the fact that the failure was a brittle failure. In addition, if the failure had been between the rock and grouting, the grouting would have moved upwards with the pipe. This was not observed in the test. Furthermore, if the failure had occurred between the grouting and rock, the failure would have happened more slowly, displacement would have grown more steadily and the re-attachment would not have taken place.

In this study, the objective was to find failure mechanisms of drilled piles. A reasonable accuracy for displacement of a pile could be about 0.1 mm. The scale of displacement is comparable with a strain accuracy of 0.0025% and a force accuracy of 20 kN. The measurements were uniaxial and done in the direction assumed to have greatest deformations. The measuring devices were more accurate that was practically needed.

5.2 Discussion of the load test results

The main problem on the utilization of the results is the fact that the work was executed from a revealed rock surface, which does not correspond to an actual execution of drilled piles. This absence of a soil layer may have affected the results. The soil layer might affect the cleaning of the drill hole, which in turn may weaken the quality of grouting. The soil may remain on the pile surface or get stuck in the grooves on the pile surface. Either way, the bond strength will decrease.

While the grooved surface was detected to have a great impact on the bond strength, the groove depth was not found to affect the results. This may be due to the fact that the piles could not be pulled up with sufficient force. The results gave no reason to use deeper grooves. If the steel properties already limit the design before the bond capacity, deep grooves are not needed. When grooved piles are designed in practice, it is important to realize the influence of corrosion. While the grooves decrease the yield capacity of the steel, the sacrificial steel for corrosion protection will decrease the capacity even more.

The utility of bond length was quite obvious from the results of the instrumented pile. With loads of this size, there are no reasons to increase the bond length, because loads are transferred to the bedrock at the beginning of the bond length. Figure 11 shows the difference between applied maximum force and developed stress. The presented stresses are calculated based on measured strains so they are not exact values of stress but are exact in relation to each other and suitable for illustrating purpose.

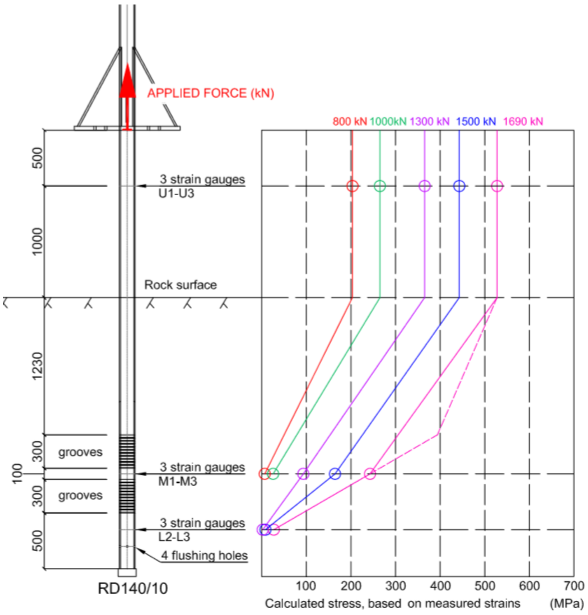

Figure 11 Stress distribution in instrumented pile

In Figure 11, each color represents a different applied force and the magnitudes of the forces are shown above the curves. Hence, the applied force is shown in upperpart of the figure and the induced stress is on the horizontal axis. In the left side of the figure is an illustration of the strain gauge locations on the pile length. The first two applied forces did not cause any stresses at the lowest measuring points, so the curves end at the middle gauge. The stress is assumed to be constant in the length of the pile, which is above the bedrock. Below the bedrock, the bond length will start to transfer loads. The stress clearly decreases faster between the lowest two gauge levels than between the highest gauge levels. The impact of the grooves cannot be separated from the figure, but it is possible, that the behavior is the same in both grooved lengths. This possibility is illustrated for the applied force of 1,690 kN, with purple dashed line.

If these piles are designed according to Eurocode, one problem to be considered is the effect of the grooves on bond capacity. Eurocode's instructs to use a different friction factor for piles that have a smooth surface and for piles that have a threaded surface. For smooth piles, the friction factor is 0.7 and for threaded piles, the friction factor is 2.0. So, the problem is to decide whether the grooved pile is smooth, threaded or something between these two.

Bond strengths for these pile types were calculated according to Eurocode and these characteristic values are shown in Table 2. The bond strength values were calculated both for smooth and threaded piles. Hence, the calculation for smooth piles was made for a bond length of 2.5 m and the calculation for threaded piles was made as a combination of a threaded and smooth pile which included 1.5 m of smooth pile and 1 m of threaded pile. When this is compared with the actual results obtained from this study, it can be observed that the actual results are closer to the smooth surface steel than the threaded steel. The bond strengths from the pull-out test are given in the same table on the right side. The "note" box contains a reference to whether the bond strength is real or based on the maximum load.

*Table 2. Designed and measured bond strengths (*no failure, **all failed, ***one failed)*

Pile no.	**Designed bond strength between grouting and … (MPa)**			**Measured avg. bond strength (MPa)**	
	Smooth steel	Threaded steel	Rock	Groo-ved	Note
1-3	1.54	2.68	1.13	1.47	*
4-6	1.54	2.68	1.13	1.20	*
7-9	1.26	2.20	0.91	1.85	**
10-12	1.40	2.44	0.99	1.23	***

If measured bond strength between grouting and rock are compared with the results of the Eurocode calculation, the difference is also quite big. The pull-out test clearly showed that the bond strength between grouting and rock was over 1.20 MPa.

6 CONCLUSIONS

The grooved surface of the pile improved the bond strength between the pile and grouting. In an earlier study by Ahomies (2015), the average bond strength was 0.30 MPa and in this test the average bond strength was 1.38 MPa. Hence, the average bond strength improved 460% by grooved surface. The conditions and the methods were not the same in tests, but these results still give a reliable direction for further investigations.

The only real limiting factor of the use of the study was the absence of the soil layer and the cleaning of the drill hole can be considered as the greatest risk in the work. Drill cutting was observed to block the drill hole easily and if there was also a soil layer above, there might be a need for more effective cleaning methods. The main problem would be that the grooves would become clogged during drilling. Cleaning the grooves would be challenging, but it might be even more difficult to verify the cleanliness of the grooves. The study did not found any effect of the groove depth on the results, but cleaning of the deep grooves may be easier. The best option is to do a trial pile and test the functionality of the grooves.

The uneven stress distribution affects also pile design. In this study, the most limiting factor when using steel pipe piles for transferring tensile forces was the yield capacity of the steel. Due to the uneven stress distribution, this yield load could have been increased based on the results. The main stresses were along the smooth length of the steel pile and the strains decreased rapidly along the bond length. Thus, the stresses on the grooved part were only about 50% of the yield resistance of the steel.

The literature review revealed some weaknesses in pile design practices and concerns about the inaccuracies in design calculations. The currently used methods have not been developed in years, although the methods and knowledge have grown. These design methods should be questioned and compared with the actual conditions and properties of the construction site. Design should be based on proper site investigations and on test piles. With target specific design the designer can pay attention to the main risks in the construction and avoid overdimensioning.

7 ACKNOWLEDGEMENTS

The idea and organization for the study was by SSAB. They provided this study and did it possible and hence the greatest complement goes to them. The pull-out test was organized on the yard of Suomen Teräspaalutus Oy and it was also the used contractor. Tampere University of Technology was responsible for the monitoring during the loading. Special thanks for all participants.

8 REFERENCES

Ahomies, Marko. (2015). The grouted and anchored drilled pile in the bedrock. Master of Science Thesis. Tampere University of Technology. Civil Engineering, Tampere.

Brown, E.T. Rock engineering design of post-tensioned anchors for dams - a review. [Online]. September 18, 2014. [Accessed 11 September 2015]. Available at: http://www.sciencedirect.com/science/article/pii/S1674775514000730.

Charlie C. Li, Gisle Stjern, Arne Myrvang. A review on the performance of conventional and energy-absorbing rockbolts. [Online]. March 19, 2014. [Accessed 11 September 2015]. Available at: http://ac.els-cdn.com/S1674775514000225/1-s2.0-S1674775514000225-main.pdf?_tid=4e0c1fa6-5869-11e5-8ec9-00000aacb360&acdnat=1441964680_ec9b6b42c3aff4ea10019f8dde7abc88.

Jesús Gómez, Allan W. Cadden, Robert P. Traylor, Donald A. Bruce (2005). Connection Capacity between Micropiles and Concrete footings: Interpretation of Test Results and Design Recommendations. [Online]. 2005. [Accessed 11 September 2015]. Available at: http://www.geosystemsbruce.com/v20/biblio/215%20Connection%20Capacity%20between%20Micropiles%20and%20Concrete%20Foot.pdf

Littlejohn, G. S (1997). Uplift capacity of shallow anchorages in weak mudstone. Ground anchores and anchored structures. London : Thomas Telford, , ss. 23-33. ISBN: 0-7277-2607-2 .

Sirén, Rosa. (2015). The tensile capacity of steel pipe piles drilled into the bedrock, Master’s thesis. Aalto University School of Engineering, Espoo.

Suomen Rakennusinsinöörien Liitto RIL ry, Suomen Geotekninen Yhdistys SGY ry. RIL 254-2011 Paalutusohje 2011, PO-2011. 1. Helsinki : Suomen Rakennusinsinöörien Liitto RIL ry, 2011. ISBN 978-951-758-528-6