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Practical experience during rock stress measurements using HF and HTPF methods at Røldal HPP

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Abstract

The power plant "Røldal Kraftverk" is located close to the village of Røldal in the Vestland county, Norway. Røldal power plant operates with a water head of 382 m. The installed capacity is 172 MW (two Francis turbines), with an average annual production of about 919 GWh. The project has been designed and in operation since 1966. This was the time when rock stress measurements were not common, so the steel lining of the project was designed based on guidance from an empirical rule of thumb. During 60 years of development, since Røldal HPP was built, the rock stress measurement technology has developed and capable to provide a far more accurate and precise description of the in situ stress conditions. So, for the new in situ stress evaluation at Røldal HPP, the NoRSTRESS research project carried out a campaign of rock stress measurement at this site in 2023, using hydraulic test methods. For the measurement, 9 drillholes were drilled in three mutually perpendicular orientations. This is to test different layouts of measurement and to test different method for estimating in-situ stress. Attempt to obtain orientation of pre-existing and induced fractures was also made by using impression packer and optical televiewer (OTV).

This paper presents practical experiences and challenges during the measurement campaign together with the measurement results.

Keywords

Hydraulic fracturing of intact rock (HF), Hydraulic tests on pre-existing fractures (HTPF), Stress measurement, Hydropower.

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1 Introduction

The power plant "Røldal Kraftverk" is located close to the village of Røldal in the Vestland county, Norway. Røldal power plant has been in operation since 1966 and operates with a water head of 382 m. The installed capacity is 172 MW (two Francis turbines), with an average annual production of about 919 GWh.

The waterway system of the project consists of two headrace tunnels, taking water from two catchments to the powerhouse. These two tunnels are merged into one tunnel before arrival to the underground powerhouse. The layout of the project is as shown in Fig. 1. The headrace and tailrace tunnels were designed according to the Norwegian practice of unlined pressure tunnels, where the tunnel utilises the capacity of the rock mass to support itself. The tunnels were supported by rock bolts and sprayed concrete. Cast in place concrete lining was used only in extremely poor rock mass conditions such as faults or weakness zones. Steel lining was used with a certain length upstream of the underground powerhouse, and this length was intentionally kept at an absolutely minimum to save the cost and construction time of the project.

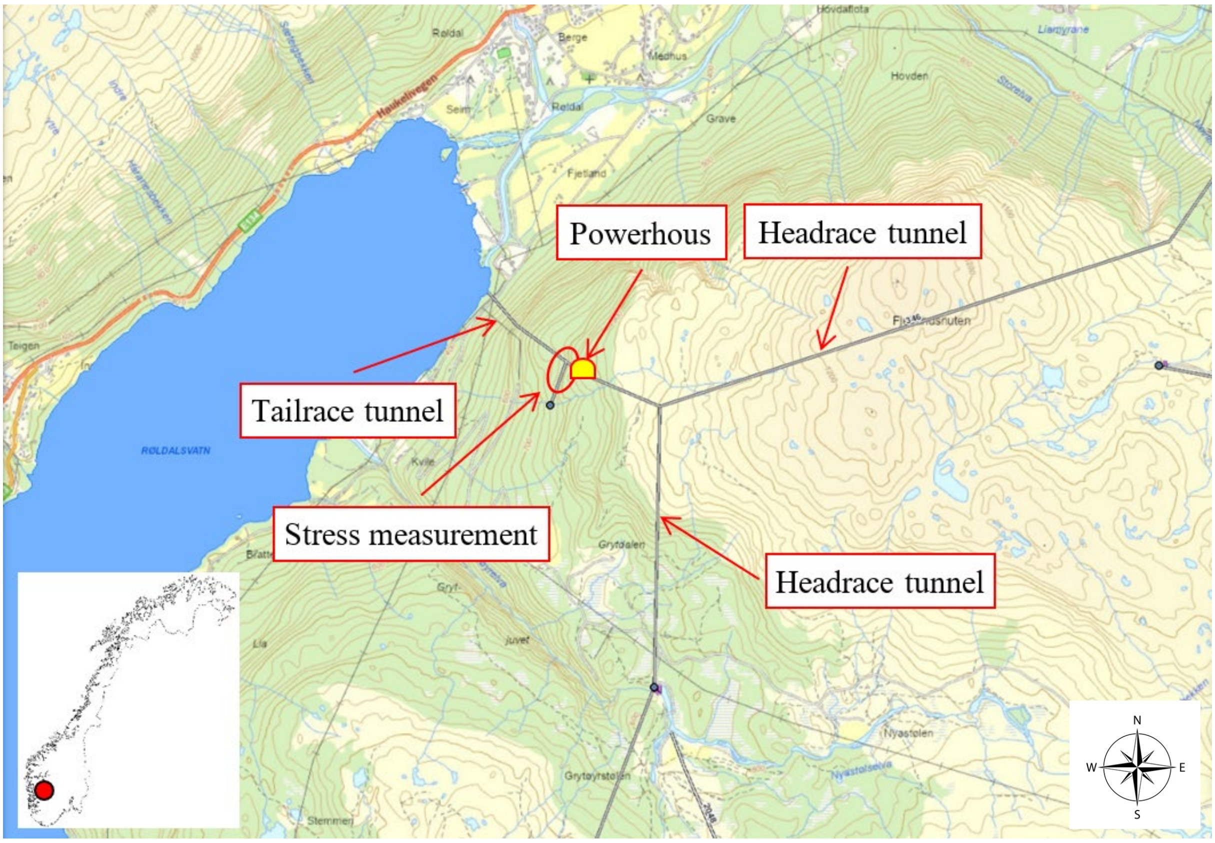

Fig. 1. Location and layout of the Røldal HPP.

According to Broch (1984), the steel lining design in Norway in the years before 1968, was based on guidance from an empirical rule of thumb. For construction reasons, the inclination of the unlined shafts varied between 31 to 47 degrees, with 45 degrees as the most common. The rule of thumb was applied for every point along the tunnel according to the following formula:

$$h > \mathrm{c} \times \mathrm{H} \tag{1}$$

Where *h* is the vertical depth of the point studied (in m) – required rock overburden.
c is static water head (in m) at the point studied.
H is a constant, which was 0.6 for valley side with inclinations up to 35 degrees and increased to 1.0 for valley sides inclinations of 60 degrees.

Steel lining will be used wherever the above formular is not satisfied. From the calculation, it seems sufficient to apply approximately 210 m steel lining in the inclined pressure shaft, however a length of approximately 380 m steel lining was installed, which is the whole length of the shaft. This was because rock stress measurements were not common at the time, and using empirical formula may contain lots of uncertainties which required higher factors of safety. During 60 years of development, since Røldal HPP was built, the rock stress measurement technology has developed and found capable to provide a far more accurate and precise description of the in situ stress conditions than in the 1960s. So, for the new in situ stress evaluation at Røldal HPP, the NoRSTRESS project carried out a campaign of rock stress measurement at this site in 2023, using hydraulic test methods. This paper presents practical experiences and challenges during the measurement campaign together with the measurement results.

2 Brief geological conditions

The Røldal area lies within the Caledonian Nappe System, a geological feature formed during the Silurian-Devonian collision of continental plates (Andresen et al. 1974). The regional geology is dominated by allochthonous rock units thrust over the Precambrian basement during the Caledonian orogeny. These units include a mix of metamorphic and sedimentary rocks that were subjected to varying degrees of deformation and metamorphism during the tectonic events. The region's structural framework is defined by significant folding and thrust faulting, which have influenced the stress regimes in the area. The folding and thrust faulting are shown in Fig. 2 together with the approximate location of the rock stress measurement site. The rock type at the site consists of gneisses.

Engineering geological mapping was performed in the access tunnel over a 100-meter stretch covering the borehole locations. Several joint sets were observed in the mapped section, all with a spacing varying between several tens of centimetres to meter. This is also reflected in the mapped Rock Quality Designation which varies between 75 and 90. Some water seepage into the tunnel was observed. Fig. 3 gives an impression of the onsite conditions. The access tunnel is supported with sporadic rock bolting.

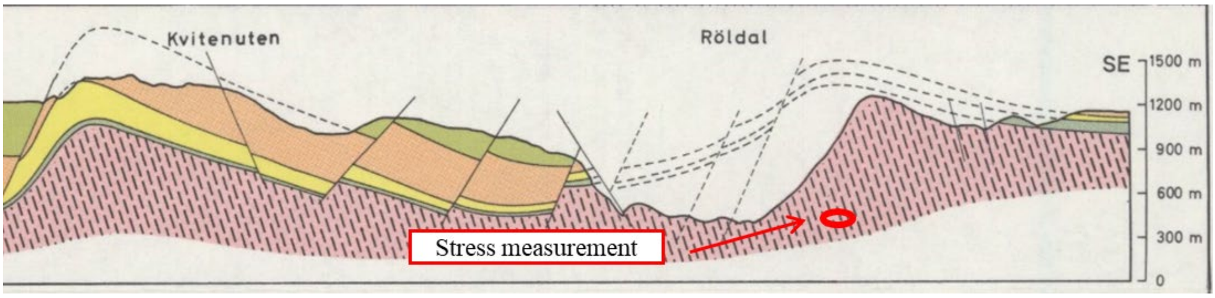

Fig. 2. Map of bedrock by (Andresen et al. 1974). The horizontal axis is approximately 15 km long. The rock type is described as gneisses, mostly granitic.

3 Layout of the measurements

The physical testing at Røldal includes nine drillholes, which were hammer drilled at three locations at the end of the access tunnel. At each location, three drillholes were drilled perpendicular to each other. The layout of the drillholes is as shown in Fig. 4.

- Length of each drillhole is approximately 30 m.
- H – "Horizontal": Horizontal drillhole, sub-parallel to topographical contour lines. Orientation dip/dip-direction: 0/45.
- S ("Skrå" means inclined in Norwegian language): Oblique upwards (sub-perpendicular to rock slope surface). Orientation dip/dip-direction: 40/315.
- V "Vertical"/ dipping down: Sub-vertical drillhole. Orientation dip/dip-direction: 50/135.
- Location "1" is furthest in tunnel, location "2" is the middle location, and location "3" is closest to tunnel entrance. Distance between locations is approximately 25 m. Thus, the involved rock volume is approximately 80x30x30 m or less than 100 thousand cubic meters.
- Name code for each drillhole consists of its orientation and sequence of its location: H1, S1, V1, H2, S2, V2, H3, S3, and V3.

The layout was intentionally made to test different alternatives of in situ rock stress measurements (HF and HTPF) and calculation methods (classical and Integrated Stress Determination Method – ISDM calculations).

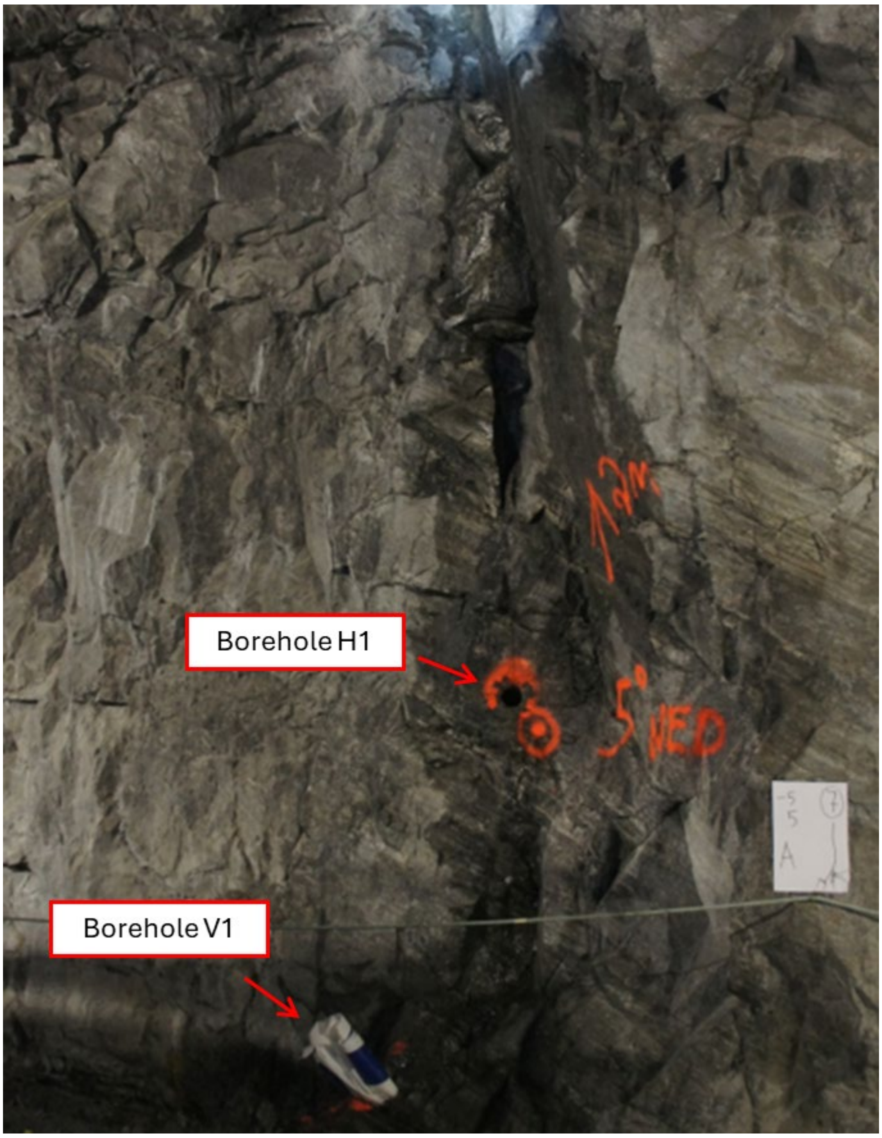

Fig. 3. Photograph of the rock mass with A4 paper for scale and borehole H1 marked with spray paint. Borehole V1 is jammed and stuffed with material visible lower down in the photograph.

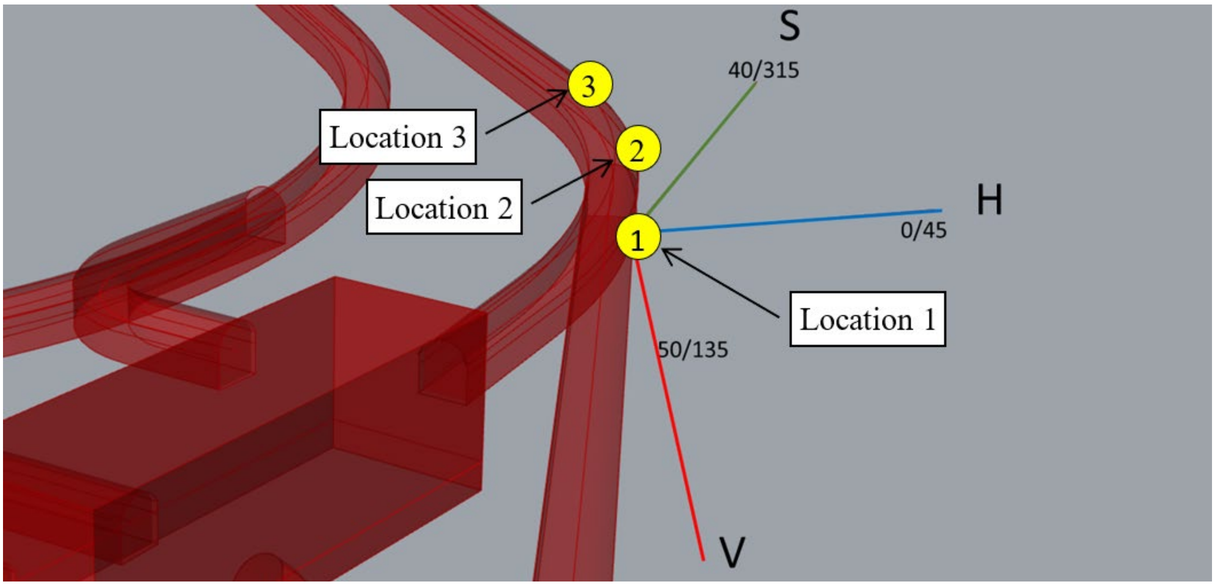

Fig. 4 Layout of the drillholes at the end of the access tunnel.

4 Results of the measurements

Hole V1 was abandoned due to a blockage situation, whilst a total number of 54 tests were carried out at selected intervals along the boreholes in the remaining 8 qualified drillholes. Each hole has 6 to 7 test sections. Before and after carrying out the tests, the drillholes were scanned with an optical televiewer (OTV). Imprint of the boreholes were made for 11 test sections to obtain the orientation of the induced fractures. The imprints were obtained using a standard impression packer. Due to time constraints, the number of imprints was limited to only 11 test sections.

During the hydraulic test, the flow and pressure were logged, and curves with pressure versus time were used to estimate shut in pressure, using methods as presented in Trinh et al. (2023).

Test results at H2-4, H3-4, S2-4, V2-2, V2-3, and V3-6 were rejected (marked with red circles) due to large pre-existing joints observed in the test section. The shut in pressure at V2-3 was rejected as it might be compromised by the previous test section. Shut in pressures obtained from the tests in Røldal HPP are presented in Fig. 5.

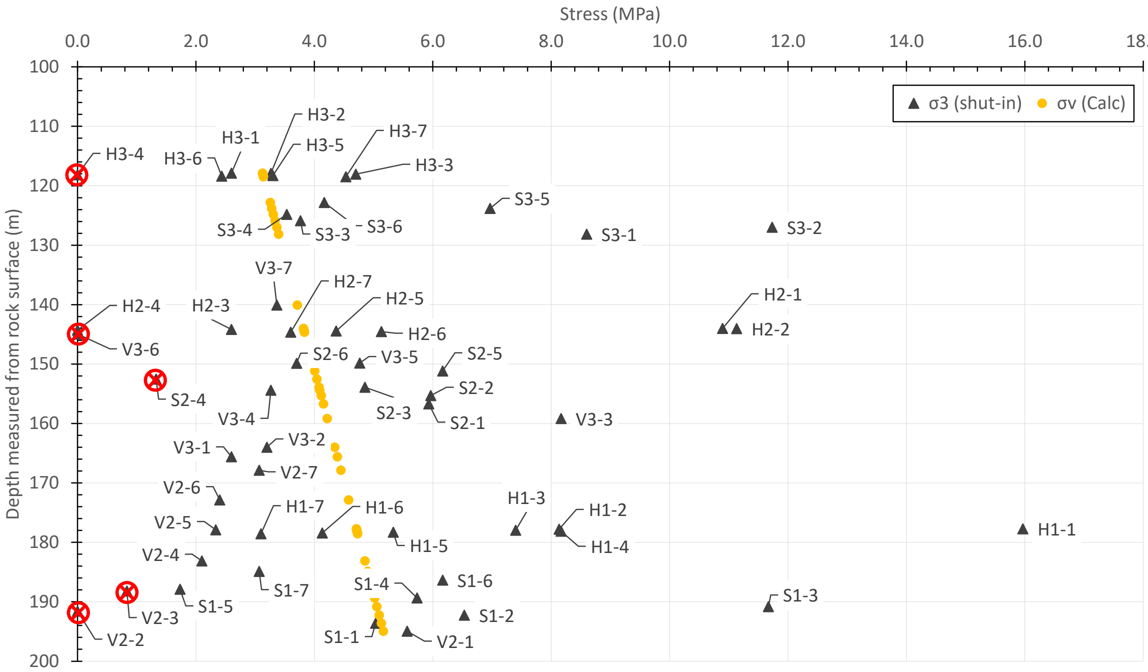

Fig. 5 Estimated shut in pressures based on pressure curve obtained inside test sections, plotted versus the depth from surface. The vertical stresses were calculated based on unit weight of the rock and overburden. A sequential number is added to the drillhole name to indicate the sequence of the test.

5 Practical experiences from the measurements

With 54 tests, the field campaign covered both test sections with HF and HTPF. For some test sections it was not possible to classify whether it was HF or HTPF. More detailed comments from the tests are as follows:

- Many tests clearly showed significantly higher peak pressure in the first test cycles compared to the subsequence tests cycles. An example is as shown in Fig. 6. It means that these first tests cycles were able to generate new induced fracture. Imprint work (for 11 test sections) by using impression packer confirm the new fracture.
- Inhomogeneity and/or pre-existing fractures were observed in all test section. However, induced fractures could still be generated in many test sections. This could be due to (*a*) the pre-existing fractures have low water conductivity, and/or (*b*) a relatively large discharge was used during the test (Qmax = 35 l/min).
- Some tests showed a similar peak pressure between the first and the subsequent test cycles. Conclusion from these tests is that one or multiple pre-existing fractures were opened rather than creating new induced fractures. These tests can be classified as HTPF tests. An example of this test is as shown in Fig. 7.

- In many cases of HTPF tests in an in situ rock mass, the fracture opening pressure is slightly higher than the fracture reopening pressure. One possible explanation for this is that the existing fracture has a certain strength/bond, which requires an extra pressure to break/open it in the first cycle. Once it is opened, the mentioned extra pressure is not required in the subsequent cycles.
- Some tests did not show clear evidence for either HF or HTPF. Example of such a test is shown in Fig. 8. For these cases, it is not possible to classify the test as HF or HTPF without further investigation of borehole wall condition before and after the test.
- In test sections with multiple pre-existing fractures, it could be a situation where not only one, but several pre-existing fractures may respond to the applied hydraulic pressure. With current development of hydraulic test, it is not possible to identify shut in pressure individually for each fracture in this situation.
- Five tests were not successful due to presence of large pre-existing joint, as shown in the example in Fig. 9. These pre-existing joints caused excessive water loss and made it impossible to build up sufficient pressure in the test sections. It was also found that one test may have been affected by the previous test section nearby, resulting in low built-up pressure. Thus, the total number of successful hydraulic tests is 48.

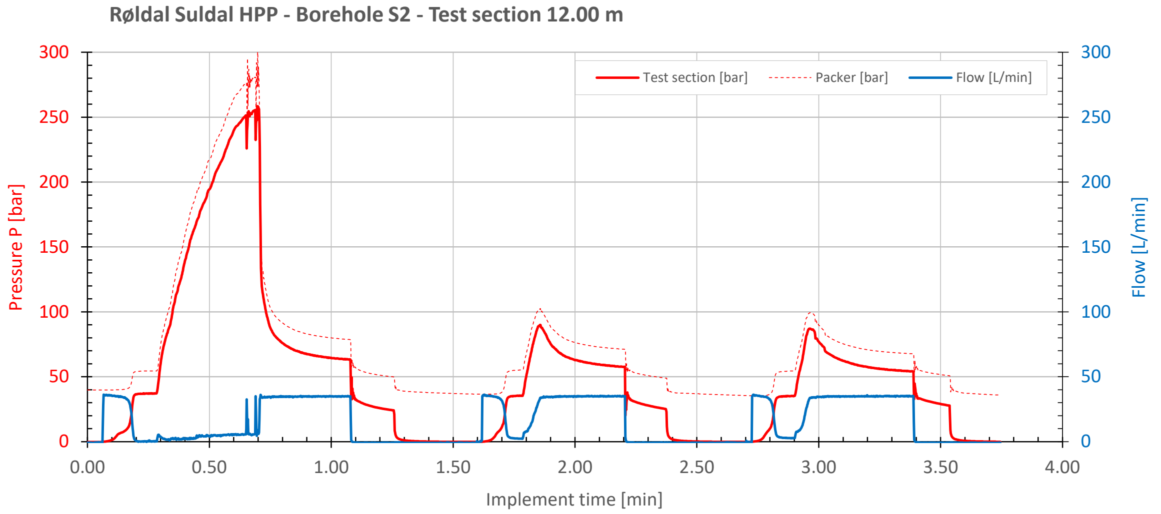

Fig. 6. A typical hydraulic test that can considered to be a HF test, which is characterised by significantly higher Pmax in the first test cycle than Pmax in the subsequence test cycles.

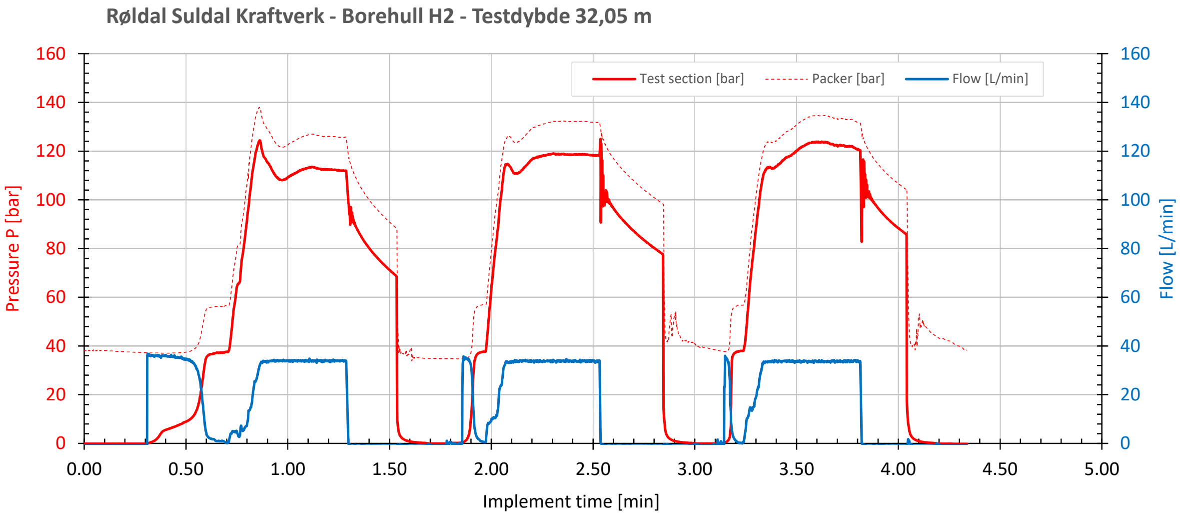

Fig. 7 A typical hydraulic test that can considered to be a HTPF test, which is characterised by a similar Pmax in all test cycles.

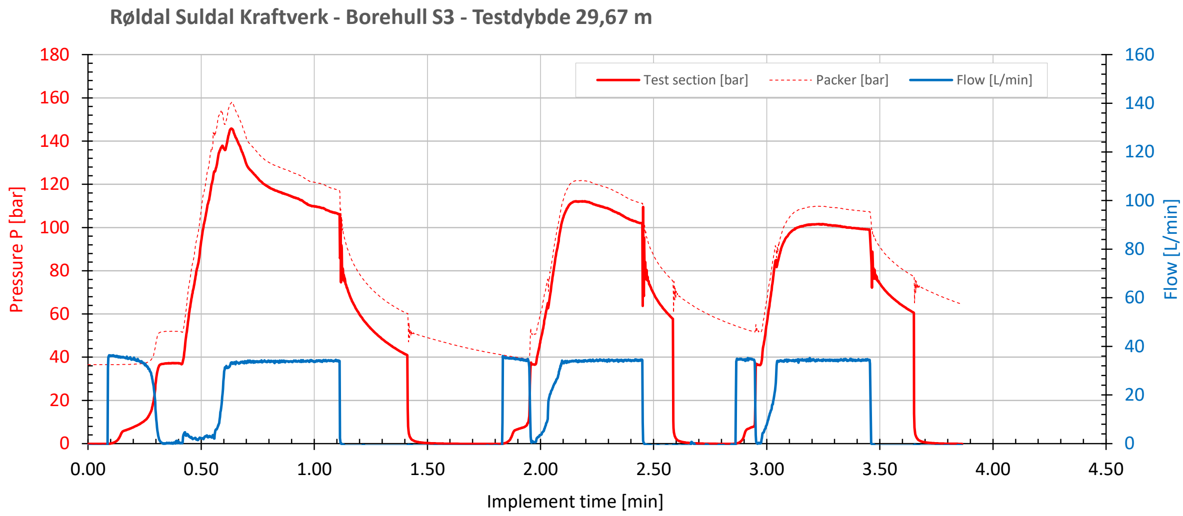

Fig. 8 A typical hydraulic test that cannot be clearly classified as a HF or HTPF test.

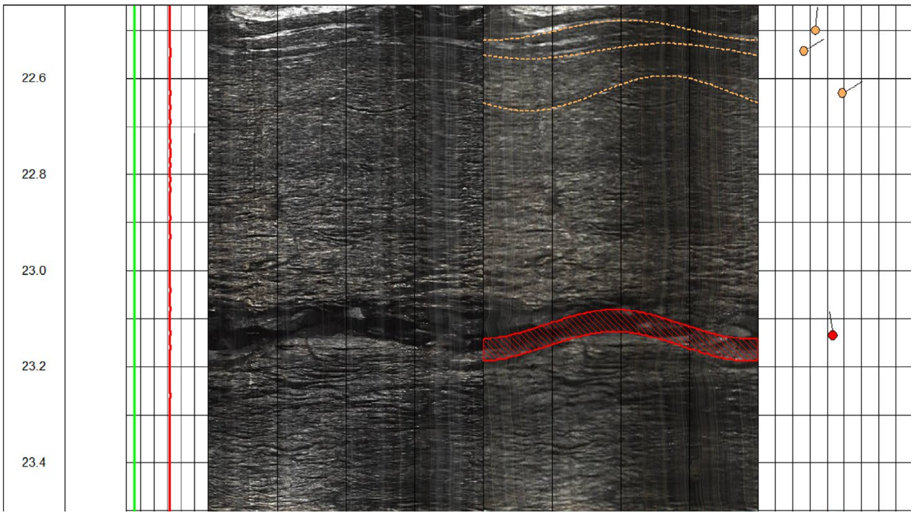

Fig. 9. A condition with large open joint making it impossible to build up pressure. Scanning before test (left) and after test (right).

Optical televiewer (OTV) was done to scan the borehole wall of all tested boreholes, before and after the test. Based on the scanning work, the following comments are made:

- From the OTV results, only pre-existing fractures could be observed. Induced fractures were not visible, also not by comparing the impression from the impression packer. A typical picture of a test section obtained by the OTV is shown in Fig. 10. The OTV contractor stated that the induced fractures are too tight, and it is beyond the resolution of the camera. It is also possible that the hammer drilling results in rough borehole wall making it more difficult to detect the new induced fractures. To be able to obtain the trace of induced factures, other tools should have been applied such as impression packer (as used in 11 test sections in this campaign), Mosnier's azimuthal laterolog (Mosnier and Cornet 1989), Schlumberger's Formation Micro Scanner (Rummel et al. 1990) or an acoustic borehole televiewer (Joong-Ho Synn et al. 2015).
- From the OTV scan, pre-existing fractures can be detected in almost all test sections. In many cases, several pre-existing fractures can be observed within a test section.
- Pre-existing fractures have also been observed in all the test sections with induced fractures. The rock is an inhomogeneous material and with pre-existing fractures with varying apertures (from tightly healed to 55 mm). Thus, it was not possible to find any location meeting the idealised conditions as required in classical HF.

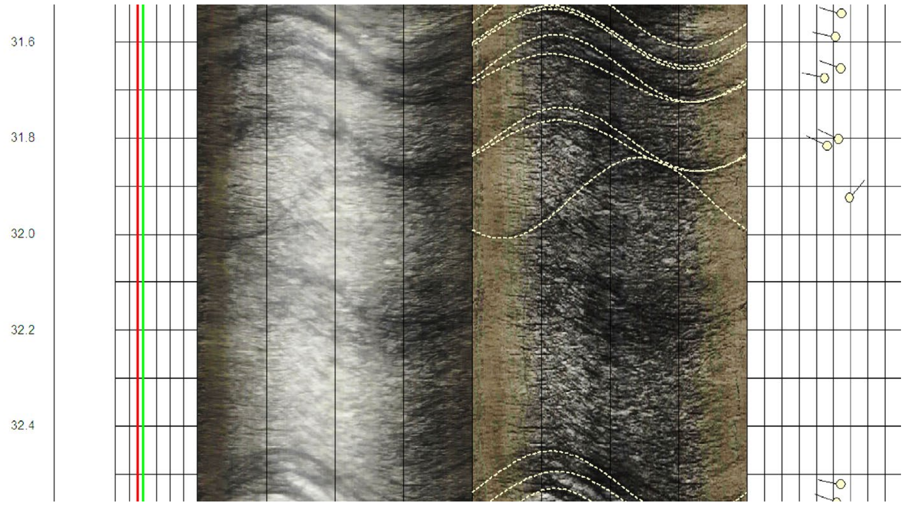

Fig. 10. A typical test section with multiple pre-existing fractures. Scanning before test (left) and after test (right).

6 Concluding remarks

The rock stress measurement campaign at Røldal HPP has provided valuable insights into both the in situ stress conditions and the practical challenges associated with applying hydraulic fracturing (HF) and hydraulic testing on pre-existing fractures (HTPF) methods. A total of 54 tests were conducted across eight boreholes, yielding 48 successful measurements. Several test sections showed clear signs of new induced fractures, marked by significantly higher peak pressures during the initial test cycles, confirmed through imprint analysis. Other test sections indicated the reactivation of existing fractures, reflected in consistent peak pressures across cycles. In contrast, there were also test sections where larger open fractures, the presence of multiple pre-existing fractures, or possible fracture cohesion making results difficult to interpret or utilize for further analysis. The attempt to use OTV for faster fracture orientation determination did not give the anticipated results. The OTV scans revealed only pre-existing fractures, with induced fractures remaining undetectable, likely due to their tight nature, the limitations of the camera resolution, and borehole roughness from hammer drilling.

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