HØGSKOLEN I SØR-TRØNDELAG

AVDELING FOR TEKNOLOGI

Studieprogram bygg og miljø

06-2011

Proposition for new Railway line between Ranheim and Midtsanden

Forslag til ny jernbanetrasé mellom Ranheim og Midtsanden


Photo: Svein E. Sando, 02.07.2003 ©

Jernbaneverket

Authors
Jørn Fosen Simonsen
Eivind Pagander Tysnes

HØGSKOLEN I SØR – TRØNDELAG

AVDELING FOR TEKNOLOGI
Program for bygg og miljø
7004 Trondheim

Besøksadresse : Arkitekt Christies gt 2

REPORT
BACHELOR THESIS

Title
Proposition for new Railway line between Ranheim and Midtsanden

Forslag til ny jernbanetrasé mellom Ranheim og Midtsanden

Project no.
06 - 2011

Authors
Jørn Fosen Simonsen
Eivind Pagander Tysnes

Client

Jernbaneverket

Date delivered	Number of reports	Total Number of Pages	School counselor
25.05.2011	2	128	Nils Kobberstad

Short summary
In order to reduce the travel time between Trondheim and Steinkjer, the Norwegian National Rail Administration is looking into upgrading different section of the line to comply with today's standard. In this thesis two different alternatives were proposed that would reduce the travel time between Ranheim and Midtsanden. This thesis proposes broadening curves, allowing higher velocities, and running the line through more tunnels than the existing line. Ground conditions may cause problems locally.

Project keywords
Location of the line, soil mechanics, tunnels, geology

The Thesis

The Norwegian National Rail Administration would like a new railway line between Ranheim and Midtsanden on Nordlandsbanen (from switch no.2 at Ranheim to switch no.1 at Midtsanden), in conjunction with plans for electrification of the Trønderline. Jernbaneverket would like to dimension for velocities above 250 km/h, which today's line does not allow. If a line dimensioned for 250 km/h makes the project disproportionately more expensive, and/or if the trains are unable to utilize the full potential in velocity, a location of the line allowing a lower velocity can be investigated.

We shall develop two new alternatives for locations of the line between Ranheim and Midtsanden. Vikhammer station is the only place where passenger trains stop today. Because of this, it will be of interest to keep the station as a stop for passenger trains. We will look into one alternative where the Vikhammer station is situated where it is today (or in close vicinity), and an alternative where the station is situated in a tunnel under Vikhammer. The last alternative will cause a relocation of the station. If we consider that a station in a tunnel is complicated because of the long distance from the station to the surface, then we might consider not keeping a station at Vikhammer. An investigation of the use of tunnels and bridges to get the largest achievable curve radius will also be of interest.

The capacity on today's line is small and with a new intermodal yard placed on the line between Hommelvik, Muruvik and Hell it will be of interest to investigate a double tracked main line to get a higher capacity. Currently the Gjevingåsen-tunnel is being built; this will reduce the travelling time between Trondheim and Stjørdal with almost five minutes. The Gjevingåsen-tunnel is prepared for a double tracked line. With higher velocities the capacity will rise and the travelling time will be reduced even more on the same distance. We will dimension the location of the line with the consideration that conventional rolling stock shall be able to travel at a velocity of 250 km/h, this will again in terms give severe requirement when it comes to the dimensioning of the superstructure.

Novapoint, with its modules, will be used in this thesis. Mainly this will be Novapoint Railway, but it can be of interest to use Novapoint Bridge, Novapoint Tunnel and Novapoint Virtual Map.

We will evaluate the soil mechanics and engineering geology in the area using present reports. These evaluations will, among others, be decisive for the location of the line. A relocation of surplus gobbing from tunnels and cuts will be suggested. On sections where the location of the line will pass over water or surface area, we will look at bridge solutions.

Preface

We chose to write our thesis for the Norwegian National Rail Administration (Jernbaneverket). This was because of our interest in roads and railways, and the possibility to write a somewhat interdisciplinary thesis since we are studying different fields of civil engineering. This thesis is to be used as a foundation for the preparation of a new master plan for the region.

When we started working on our thesis, we started out in our separate fields of specialization. As the thesis progressed, we found that we had to interact more with each other's work than we initially thought. After a time, we came to the conclusion that the thesis itself might have been a bit extensive for only two students. This resulted in lack of detail in a few areas. All in all, we think we have reached a satisfying result for this proposition.

We want to thank our counselor Alf Helge Løhren at Jernbaneverket for the help he has given us. We want also to thank John Våge and Ingeborg Tulluan at Jernbaneverket, Jan Erik Hoel at Vianova and Stig Gunnar Lillevik at Statens Vegvesen for the help they have given us. Thanks will also be given to Svein E. Sando for allowing us to use some of his pictures.

Trondheim

Jørn Fosen Simonsen

Eivind Pagander Tysnes

Summary

The project group consists of two students from the civil engineering department of Sør-Trøndelag University College working on behalf of the Norwegian National Rail Administration (Jernbaneverket). The students have studied two different fields of specialization; heavy construction and infrastructural planning. Through working on the thesis, the students have gained more knowledge in railway planning, and the work required in seeing a project through.

The goal of this thesis has been to upgrade a section of Nordlandsbanen to today's standards. This resulted in two planned lines, and evaluations of ground conditions between Ranheim and Midtsanden. These two alternatives differ in that one of the alternatives maintains Vikhammer station where it is today. Keeping the station also allows for trains to keep stopping in the area. Moving the line completely may result in a distance without any stops on the way. There is a possibility that the particular area around Vikhammer is not suited for an underground station.

Two different velocities were used during the positioning of the line. This was because the alternative keeping Vikhammer station did not allow for large enough curves to be dimensioned for 250 km/h. Instead 200 km/h were used on that alternative.

Both alternatives have to cross Være bay. This bay presently is zoned as a recreational area, and a bridge would have a great visual impact.

The evaluation of the ground conditions is based on old reports obtained from The Norwegian National Rail Administration and The Norwegian Public Roads Administration (Statens vegvesen).

The location of the line has taken precedent, which resulted in relying heavily upon stabilizing measures instead of moving the line itself. In a soil mechanical perspective, Hundhammeren has been recognized as the most problematic area, relying on cement lime piling to stabilize the cuts.

There is expected much poor rock in the tunnels (in both alternatives), which have to be countered with stabilizing measures. The main problem in the tunneling is how to achieve sufficient overburden.

Index

1. Introduction

The Norwegian National Rail Administration gave the students a part of Nordlandsbanen to upgrade to today's standard. From that point on, the students themselves defined the goal for this thesis.

The existing railway on this section of Nordlandsbanen is comprised of many small and large curves which act as limiting factors on the velocity of trains. Today's trains run at up to 300 km/h, which is more or less impossible on this section of Nordlansbanen with the curves being as they are. In order to achieve higher velocities, the curves have to be straightened out. In doing so, the line has to be moved from its existing course.

This thesis proposes two alternative lines which will allow for higher velocities. The ground conditions along the lines are evaluated to see if the lines are plausible. It was decided that the location of the line was to take precedence, and poor ground conditions would not automatically cause relocation unless it simply could not be done otherwise. As a basis for the evaluation of the ground conditions old reports were used. No new geological surveys were made. As a result the data applies for conditions in proximity of the line, but it still gives a good idea of what to expect. It also gives grounds for comparison.

The blueprints are built upon SOSI-files (maps) obtained from the Norwegian National Rail Administration and are presumed to be accurate.

The main structure of this report is compartmentalized in profile intervals containing current information deemed of interest to the construction of the lines. The different blueprints produced for the thesis are gathered in a blueprint booklet separated from this report.

2. Location of the line

2.1 Background

The Railway line today between Ranheim and Midtsanden is for the most part following the shoreline. The line was built in the late 1800s and put into service in 1881. At that time the cost of the railway was considered more critical than the velocity and travelling time. This again meant for the railway to follow the terrain and make use of short distances to move the surplus gobbing from both rock and soil cuts to where embankments would be. Since the line follows the terrain and for the most part of it is going along the coast, it has many curves with smaller radii. This results in an average velocity of approximately 90 km/h. This is considered slow for modern trains.

The line is almost alone in Norway in having the line running down by the waterfront. The elevation of the line varies from 4 – 5 meters above sea level to approximately 12 meters above sea level and has provided a beautiful view of the fjord and the hills on the other side of the fjord. This also means that many of the beaches and other areas of recreation have a restricted access for the public. A new line will free up many of these areas for the public.

Travelling time between Trondheim and Hell is today 33 minutes. With the addition of the new Gevingåsen Tunnel the travelling time will decrease with approximately 5 minutes. With an upgrade of the line between Ranheim and Midtsanden the travelling time can further be reduced with 2-5 minutes.

In light of the increased amount of traffic on rails in the region the capacity of today's line are starting to exceed its maximum capacity. Pending on where a new freight terminal for Trøndelag will be situated the line might see a lot more traffic than today. Because of this a new line with upgraded infrastructure is in much need. A new line will have to be electrified due to environmental concerns. Double tracking the line will also increase the capacity and will improve the travelling time between Trondheim and Hell.

Today the railway lines north of Trondheim Central Station are dieselized. Because of this a vast amount of carbon dioxide is emitted into the atmosphere every year. Most of the passenger trains travelling on this line have a lower maximum velocity than the equivalent electric passenger trains. A new line will be planned for electrification. This will in turn reduce the emission of carbon dioxide into the atmosphere to almost zero. With the electrification of the line new electric trains can be put in with a higher maximum velocity.

For an overview of the existing railway line and the propositions for new railway lines see blueprints B0 – B7.

2.2 Alternative 1

In this alternative we will look into a new line which keeps the existing Vikhammer station as it is today with the required improvements. This alternative will be dimensioned for 200 km/h. Horizontal minimum radius is 2400 meter with a super elevation of 208 mm as given by Teknisk Regelverk JD 530 (Jernbaneverket, 2011, Appendix a). In this alternative the horizontal minimum radius is 2500 meter. Minimum vertical radius is calculated by the formula (1) and yields a minimum radius of 15385 meter. Minimum vertical radius in this alternative is set to 15500 meter.

$$R_{min} = \frac{V^2}{2{,}6} [m] \quad (1)$$

Switches used at the station are with a diverging route of 1:14 and a radius in the diverging route at 760 m. Velocity through the diverging route is 80 km/h. Radius and velocity are given by Teknisk Regelverk JD 530 (Jernbaneverket, 2011, chapter 5).

Based on Teknisk Regelverk JD 520 (Jernbaneverket, 2011, chapter 5) standard section for cuts, embankments and tunnels are drawn. See blueprints F1 and F2 for these profiles.

Vertical and horizontal alignment can be viewed at blueprints C1 – C14.

Trains passing Ranheim station at 100 km/h will need minimum 7,235 m to reach 200 km/h. Trains stopping at Ranheim station will need minimum 8,588 m to reach 200km/h. Trains need at least 998 m to decelerate from 200 km/h to 100 km/h and 1,331 m to decelerate from 200 km/h to 0 km/h. Appendix 7 shows the calculations of velocities.

In this alternative there are three tunnels. These are from profile 9825 to profile 11230, from profile 12600 to profile 13025 and from profile 13370 to profile 13700.

2.2.1 Ranheim – Være: Profile 7780 – Profile 9825

From Ranheim station the new line follows the existing line for approximately 100 meters before it continues straight while the existing line curves of to the south. The line enters a large curve just after it has broken off from the existing line. From profile 7980 to profile 8060 there will be a large embankment on the left hand side of the tracks, on the right hand side the embankment is smaller due to the slope of the terrain. The embankment will go into the fjord. Today there is a road going through the area where the new line is placed from profile 7910 to profile 8075. This road will have to be moved. By placing a retaining wall on the left hand side of the tracks the road can be moved to the north side of the retaining wall. This will also free up the waterfront and make it as accessible as it is today.

The west abutment for the bridge across the Være bay will be at profile 8075. From there the bridge will cross the bay at a track elevation of 10.370 meters above sea level. At profile 9565 the east abutment will stand. The east end abutment is placed just east of the existing line. This makes for building of the abutment without interrupting the traffic on the railway today. The traffic will only have to be shut down during the construction of the last part of the bridge.

The bay has been graded by Trondheim municipality as a recreational area as shown in figure 1. Because of this the bridge has to have an aesthetic look to it. The bridge is sitting high above the water and will have a huge visual impact on the view from the waterfront. The bridge will not come closer than approximately 30 meters to the headland northeast of the area.

Figure 1: Development plan Være bay

In this area excavated soil and blasted rock may be used to improve the recreation area and make it more accessible than today. Removing the old line through this area will free up more areas for recreation.

From the east abutment the line will pass through farmland and make a soil cut through it before heading into the tunnel opening at profile 9825. From profile 9730 to profile 9790 the old E6 will pass over. At this intersection a bridge for the road to cross the railway will be among the things to consider. The other option is to start the tunnel at this point with a concrete culvert for the railway.

2.2.2 Vikhammer Station: Profile 11230 – Profile 12600

The new line will exit the tunnel through Hundhammeren at profile 11230. From here the line will go through farmlands in a soil cut. The sidings for Vikhammer station will start from profile 11350 and end at profile 12500. This will allow freight trains to stop at the sidings for passing trains. The passenger platforms will be placed in the same area as on the existing line. Through the station the two right hand tracks will be placed on the same place as the existing track. The two left most tracks will therefore come closer to the sea than the existing tracks.

Just before the line starts to follow the old line there is a rock cliff with a couple of houses and an outhouse that has to be moved for the tracks. On the left hand side there is a boat house that has to be moved or demolished. To keep the area taken up by the railroad through the station the use of retaining wall will have to be used instead of cuts and embankments. To reduce the noise for the people living close to the railway, sound barriers will have to be installed. On the east side of the station there are seven houses to the north of the tracks that has to be moved to make room for the line. On the South side 4 to 5 houses need to be moved.

When the line curves to the east it will occupy a lot of farmlands. The municipality of Malvik can seize this opportunity to develop some of the occupied farmland into residential areas.

2.2.3 Naustan: Profile 13025 – Profile 13370

From the tunnel opening the line will cut through farmlands. Because of the small area of farmland around the cut there is a possibility to develop this area into residential areas.

In this area the line runs alongside the old line for approximately 100 meters. The buildings that are situated south of the track here will be able to stay at it is today because the right track is following the same path as the old line.

2.2.4 The Malvik Bay: Profile 13700 – Profile 14500

There are three houses on the west side of the bay that are situated in the middle of the line that has to be demolished or moved to make way for the new line. Just after these houses the line crosses the old line and at profile 13815 the abutment for a new railway bridge will be. Here an approximately 600 meter long bridge will cross the bay at an elevation of 8.120 meters above sea level. On the east side of the bay the abutment will be at profile 14430. From the abutment there will only be small areas of farmland occupied by the causeway and embankment. A cut will start from profile 14460. This cut will take up small areas of farmland in this area.

2.2.5 Haugan – Torp: 14500 – 15900

From here on the line will go in a cut through farmlands with no buildings until the line starts to follow the old line from profile 14800. A road crosses the railroad at profile 14610 and has to cross the railway on a bridge. From profile 14800 the line is laying a little lower in the terrain than the old line, but are on a steeper grade than the old line and will almost come up to the same elevation as the old line. There are a couple of road underpasses that has to be extended and lowered to allow the new line to pass through the area. From where the new line starts to follow the old line there will be a large cut in the beginning for approximately 150 meter. At profile 14920 there is a house that the cut has to move around by the use of retaining walls. From profile 15000 the cut will be small and around buildings the best solution is to put up retaining walls instead of demolishing.

2.2.6 Torp – Midtsanden: Profile 15900 – Profile 17990

From profile 15900 the new line breakes off to the south from the old line and heads in a straighter line towards Midtsanden than the old line. As the line breaks of from the old line a cut is needed before the line crosses a small chasm in the terrain at profile 16060. A small stream is passing through the chasm at profile 16120. After the line has crossed the chasm the line pass through a small village. The line is in this area at an elevation of 8 to 10 meters below the existing terrain. This means that all the houses have to be either demolished or moved. If a concrete culvert is the chosen solution the houses can be temporarily moved and placed back after the construction is done.

The culvert should run to profile 16900 due to the farm the line will pass through from profile 16480 to profile 16600, and a couple of roads that cross the line. From profile 16630 to profile 16900 Fv 950 will pass over the line. In this area the road might have to be raised to make room for the railway underneath. Here the only solution is a concrete culvert.

On the last part of the new railway the line is mostly passing through a cut until it reaches the connecting point to the old line. Where required retaining wall should be built to reduce the number of houses needed to be moved or demolished. At profile 16970 a stream crosses the railway. This stream is already in pipes since it also crosses the old line. A road going alongside the old line has to be moved to make room for the new line.

2.3 Alternative 2

In this alternative we will look into a new line where Vikhammer station will be moved into a mountain hall if possible. Due to the depth from the surface to the height of the rail in the area that an underground station is best placed due to the population a station might be eliminated completely and passenger taking the train will have to travel to either Leangen in Trondheim municipality or Hommelvik in Malvik municipality to take the train.

This alternative will be dimensioned for 250 km/h. Horizontal minimum radius is 4000 meter with a super elevation of 90 mm as given by Teknisk Regelverk JD 530 (Jernbaneverket, 2011, Appendix a).

Minimum vertical radius is calculated by the formula (1) and yields a minimum radius of 24039 meter. Minimum vertical radius in this alternative is set to 24500 meter.

Switches used at the station are with a diverging route of 1:14 and a radius in the diverging route at 760 m. Velocity through the diverging route is 80 km/h. Radius and velocity are given by Teknisk Regelverk JD 530 (Jernbaneverket, 2011, chapter 5).

Based on Teknisk Regelverk JD 520 (Jernbaneverket, 2011, chapter 5) standard section for cuts, embankments and tunnels are drawn. See blueprints F1 and F2 for these profiles.

Vertical and horizontal alignment can be viewed at blueprints C15 – C27.

Trains passing Ranheim station at 100 km/h will need minimum 12,657 m to reach 250 km/h. Trains stopping at Ranheim station will need minimum 14,010 m to reach 250km/h. Trains need at least 1,746 m to decelerate from 250 km/h to 100 km/h and 2,079 m to decelerate from 250 km/h to 0 km/h. Appendix 7 shows the calculations of velocities.

There is one long tunnel in this alternative starting at profile 9825 and ending at profile 16600.

2.3.1 Ranheim – Være: Profile 7780 – Profile 9825

In this profile interval alternative 2 is almost identical with alternative 1. The line is placed a little further to the south mainly because of a larger curve radius on the bridge. Because of this, the large embankment on the west side of the bay will not come in conflict with the water and can be placed as it is. The road passing under the embankment has to be moved and most likely end up being placed on the north side of the railway as in alternative 1. The bridge will intersect with the headland at approximately profile 9040 to profile 9110 but at a higher elevation than the terrain.

2.3.2 New Vikhammer Station: Profile 10600 – Profile 11700

The new station will be an underground station. The platforms will be placed from profile 11150 to profile 11400 due to the lower terrain elevation in this area of Vikhammer. Using the road 100 meters to the north of the railway at profile 11300 as a public entrance might be the best solution.

Given the lower terrain elevation by the road this will result in a more horizontal entrance. A layout of the station similar to the new Holmestrand station in Vestfold County should be considered. In conjunction with the entrance a possibility for an underground parking garage may also be considered.

2.3.3 Midtsanden: Profile 16600 – Profile 17007

From the tunnel opening at profile 16600 the line will pass through a large cut. Because of the cut some houses need to be demolished and Fv 950 has to be raised to allow the trains to pass under. The line will come in to the old line and continue on the old line at profile 17007.

3. Ground conditions

3.1 Soil mechanics

The ground along the new railway lines is mainly dominated by marine deposits (clay,) often of great thickness. Clay as foundation soil is usually unproblematic, but in steep terrain and/or if it is very sensitive clay i.e. quick clay, it can cause great problems. If not handled properly it can cause large and extremely dangerous landslides.

3.1.1 Quick clay

Quick clay is clay deposited underneath the marine limit. This clay had an original salt content of 35‰ in the clay's pore water. When the land rose after the deglaciation and the fresh water streamed to the "new" ocean level, the fresh water washed out the salt content in the pore water. The salt content thereby diminished gradually. When the salt content becomes lower than 5 ‰ quick clay may develop. A small tremor is all it takes for the quick clay to slide.
What most characterizes quick clay is the disturbed clay's shear strength, S_r. For quick clay this value is: $S_r < 0.5$ kN/m^2.
The sensitivity is also very high: $S_t > 50$
In situ, the clay has a very high water content which is higher than the liquid limit (yield point): $w > w_L$. Aarhaug (1984, s. 50 - 51)

3.1.2 Stabilizing measures

Adding salt

Adding salt increases the strength of the clay, but due to large dimensions In situ and the impermeability of the clay, the incorporation takes a very long time. Even if added salt solutions in concentrated places (holes), the incorporation through the pores will take a very long time. This can be done by electrolysis, but this is a very expensive method. Aarhaug (1984, s. 52)

Lime stabilization

Lime stabilization has proven to be a very utilizable method of stabilizing sensitive clay. If added a minimum of 3mass-% charred lime, the clay changes its ductile properties totally. Having liquefied clay ($w > w_L$) and gradually increase the incorporation of lime, the water content in the clay will be reduced. When the lime incorporation reaches about 3 % we get firm clay ($w < w_P$). Aarhaug (1984, s. 52 - 53)

A commonly used newer method is cement lime piling where an approximately 50 centimeters in diameter bore is drilled down through the clay until it reaches firm ground. When the bore is pulled back up charred lime is fed through a hole in the bore with compressed air. The charred lime is then milled into the clay with the help of the bore's rotation. Aarhaug (1984, s. 53)

3.1.3 Alternative 1

Ranheim – Profile 7780 – 8075

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations reports UB.100577-000 (1959), UB.101763-000 (1964) and UB.101800-000 (1984).

The ground is dominated by shifting layers of sensitive clay/quick clay, silt and silty sand. Underneath a 2 to 4 meter thick layer of eroded soil, the undrained shear strength increases linearly with the depth. At profile 7950 where the new line breaks off from the old line and continues towards the bridge the depth to the rock surface is about 5 meters. Near the shoreline where the abutment for the new railway bridge will be placed, there will be need for more specific examination of the ground.

In the proximity of the old Ranheim train station (east side) the ground is dominated by firm clay. The new line will be built on or alongside the existing line and the earth materials are not considered problematic. Expansion of existing causeways should be feasible.

Between profile 7980 and profile 8060 the line runs so close to the shoreline that part of the causeway/embankment on the left hand side will have to be built in water unless one chooses a retaining wall solution. When building the embankment one has to be aware of the pore pressure and pressure gauges should be used during construction to ensure the continued stability of the embankment. Blasted rock could be used.

Væresbukta – Profile 8075 – 9565

The new railway line will cross the bay on a 1490 meter long bridge. Foundations for this bridge will not be assessed in this thesis. The condition of the sea floor has not been investigated.

Være – Profile 9565 - 9650

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations reports UB.100761-000 (1964), UB.101762-000 (1964), UB.101772-000 (1966) and UB101807-000 (1995).

When it comes to the placement of the abutment for the railway bridge more specific examination of the ground will have to be made here in the same way as on the Ranheim side of the bay.

The land in the immediate proximity of the new railway line is used for agriculture, and the ground is dominated by sensitive clay and quick clay underneath a thin layer of eroded soil (dry-crust). The clay is firmer near the shoreline and the depth to solid rock along the old line ranges from 5 meters before Væresholmen to 15 meters after Væresholmen. The terrain inland has a relatively steep ascent. After the intersection, the depth to solid rock ranges from approximately 20 meters and then decreases in closer proximity to Hundhammeren.

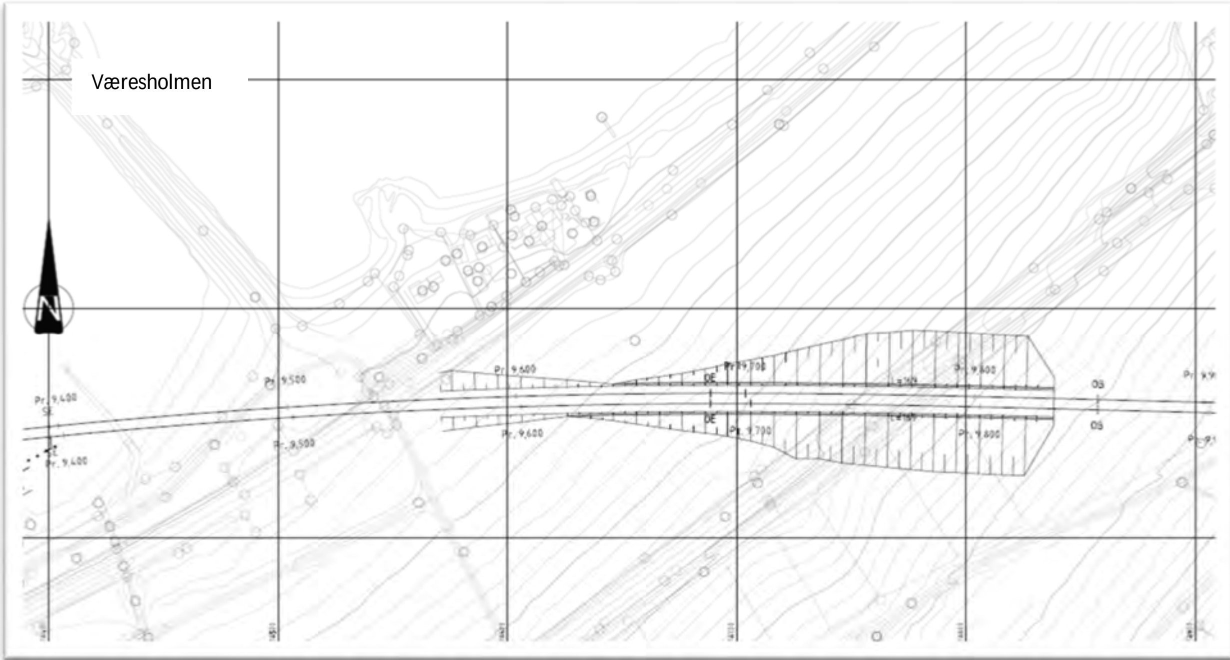


Figure 2: Intersection at Være

Hundhammeren west – Tunnel cut – Profile 9650 - 9825

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations report UB.101807-000 (1995).

At the western tunnel opening, east of profile 10040 on the old railway line, we have sparse rock at terrain level and/or at shallow depth. West, alongside the old line to Ranheim, the depth to solid rock increases to a maximum of 20 meters. In the tunnel cut the ground consists mainly of firm clay in the top layers, but its strength decreases by depth and we have sensitive clay and/or quick clay in the lower layers.

Due to the condition of the soil and the new grade elevation of the new line, relatively large excavations will have to be made during construction in this area. In order to gain satisfactory slope stabilization, the angle of pitch must probably be in the range 1:3 – 1:4. This alternative however, requires so much surface area that a huge amount of the surrounding farm land will be permanently consumed by the railway and may therefore be a less feasible solution?

Another solution would be to use pile dikes, perhaps all the way down to solid rock. This requires less space and can be used in a permanent construction. Still, there will have to be made extensive stabilizing measures, such as cement lime piling due to the sensitivity of the clay, which at some depths almost reaches the incredible 100%. Much of the cement lime piling will have to be carried out before the excavations begin.

The quick clay pockets in the vicinity, which is the main concern in this area, have been registered at depths of 5 to 6 meters below terrain level in the area. The worst case scenario is that these pockets will be punctured in order to make room for the causeway, see figure 3. More thorough geological surveys will have to be made.

The capacity of the soil in the layers underneath the causeway, is not considered to be a problem due to the large amount of masses being excavated in order to reach the given grade elevation. In that regard we more or less use the principal of compensated foundations.

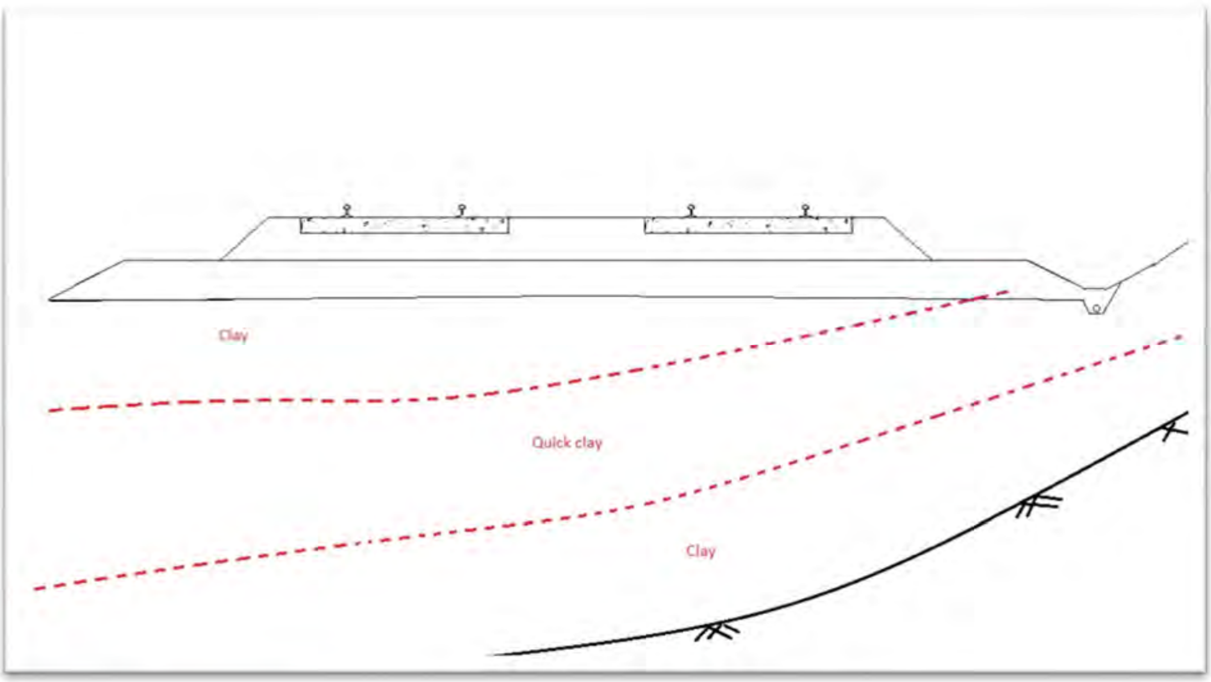


Figure 3: Schematic diagram: Quick clay pockets

Between profile 9750 and where the tunnel portal will be at approximately profile 9825, a distance of 75 meters, the line crosses 10 meters below the old E6 – highway. Due to the road and the thick layer of overlying masses, the railway line will have to go through a culvert until it reaches the tunnel itself.

Hundhammeren East – Profile 11230 – 11600

The ground conditions in this paragraph are gathered from the Norwegian National Rail

Administrations reports UB.101234-000 (1966) and UB.101807-000 (1995).

At the eastern tunnel opening there is sparse rock at terrain level and/or at shallow depth. The ground in this area is much similar to the west side of Hundhammeren. It consists mainly of clay. Underneath a 2 – 4 meter thick layer of dry-crust clay we have sensitive clay and quick clay. First after 10 meters down, the soil changes to stronger clay again. In vicinity of the new line the depth to solid rock varies from 2.9 meters to 17.5 meters.
Like on the west side, there will also be need for extensive stabilization measures. In itself, the capacity of the soil should not be a problem due to the vast amount of masses that have to be removed to make room for the causeway. Otherwise it is the same premises for both the west side and the east side of Hundhammeren.

Vikhammer (Saksvikbukta) – Profile 11600 – 12300

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations report UB.101807-000 (1995).

Between profile 11525 and profile 11650 the old line passes through a rock cut on the right hand side. Since this is a intersect point between the old line and the new line this rock cut will have to be widened/expanded in order to make room for the new double track main line.
This is a vegetated area and due to the already existing rock cut, an expansion here is considered to be unproblematic, although the rock cut will be higher. The degree of rock scaling and other safety measures i.e. rock bolting will have to be determined at a later stage.

Between profile 11625 and profile 12100, a distance of 475 meters, the new line will partially go on and alongside the old line. On the left hand side, along the shoreline the ground is dominated by sensitive clay and quick clay underneath an approximately 2 meter thick layer of dry-crust clay. There are also sections with layers of sand, silt and less sensitive clay. The expansion of the causeway requires more specific geotechnical surveys in this area. Due to the close proximity to the shoreline and the surrounding ground condition, there may be a need for a berm on the left hand side of the causeway. This will in addition to be a stabilizing measure, also function as a filter against the sea. Rock from tunnel blasting can be used here. The causeway will be 2 to 5 meters high.

Vikhammer west – Profile 12300 – 13300

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations reports UB.101236-000 (1966), UB.101807-000 (1995) and NSBs Hovedplan Ranheim – Malvik (1996).

The land surrounding the line is used for agriculture. The ground consists mainly of clay with medium strength underneath a top layer of gravel and sand. The clay reaches from 2 to 7 meters in depth. Just above the rock surface there is a thin layer of drift clay (moraine). Alongside the old railway line, the depth to solid rock in area varies from 4 to 8 meters, but in the tunnel cut this depth increases to a maximum of 12 meters. Due to the condition of the soil, the construction of the causeway is considered to be relatively unproblematic. According to NSBs Master plan Ranheim – Malvik (1996), the angle of pitch in the cut was assessed to be stable at a 1:2 relation as long as the cut was no higher than 6 meters. Although the new line varies somewhat from the alternative in the Master plan 1996, the same conditions are expected here. Up to profile 12500, a distance of 200 meters, no extra ordinary stabilizing measures are expected to be needed.

Between profile 12500 and profile 12700 we have from 6 to 12 meters of clay and gravel above the given grade elevation. In order to gain satisfactory stability of the cut on the right hand side the angle of pitch needed to be in the relation 1:3 – 1:4 for the case in NSBs Master plan Ranheim – Malvik (1996) and we assume similar conditions here. However, on the east side of the camping site there have been suggested a concrete culvert through the earth materials. In order to reduce the amount of farm land consumed by the railway a concrete culvert could also be an alternative on the west side. A culvert between approximately profile 12575 and the portal of the tunnel at profile 12700 with backfill of excavated soil would free about 125 meters of the railway to farm land.

Vikhammer east – Profile 12860 – 13200

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations report UB.101807-000 (1995).

The area between Vikhammer camping and east to profile 13200 is mainly farmland, and the ground in this area is dominated by drift clay (moraine clay). The depth to solid rock in this end of the camping site has been measured to 8.8 meters near profile 13065 and 17 meters near profile 13015 (the tunnel cut). The suggested concrete culvert will in this case be placed between approximately profile 12860 and profile 13025. With backfill over the culvert, the camping site can eventually be up and running again after the construction period. From the end of the culvert and east to profile 13200 the line will go through a cut. Due to the soil condition the angle of pitch can probably be in the relation 1:2 with no extra stabilizing measures needed. At profile 13200 the new line will intersect the old line and will run on/alongside the old line for approximately 100 meters, up to profile 13300.

Naustan west – Profile 13300 – 13500

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations reports UB.101807-000 (1995) and NSBs Hovedplan Ranheim – Malvik (1996).

At the tunnel cut where the line crosses the old E6 highway there is bare rock at terrain level or/and at shallow depth (2 to 5 meters) and the hill has a steep ascent. West of this point the depth to solid rock increases to a maximum of 14.5 meters. The ground is dominated by sand with some pockets of clay.

Up to approximately profile 13500 the new line proposal does not vary much from the alternative in NSBs Master plan Ranheim – Malvik (1996). The angle of pitch for cuts in this area can probably be in the standard relation 1:2 and still have sufficient stability. The new line will run a little closer to the shoreline than alternative 3 and 4 in NSBs Master plan Ranheim – Malvik (1996), but the tunnel cut and opening will still be underneath the Rv 950 (approximately profile 13440), which makes this area somewhat problematic.

The Rv 950 crosses in approximately profile 13435 and it is somewhat uncertain whether or not there will be sufficient overburden for the tunnel at this location. Between profile 13350 and profile 13400 we have solid rock 3 to 5 meters below terrain level, which yields approximately 8.5 meters of solid rock above the top edge of the rails. The quality of the rock is unknown.

In order to keep the Rv 950 open a support construction will have to be put in place during the construction period of the tunnel. The most feasible support would be sheet piling which is bolted to solid rock. A permanent solution is most likely though, i.e. an extension of the tunnel in form of a concrete culvert.

Naustan east – Profile 13700 – 13850

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations reports UB.101238-000 (1966) and UB.101807-000 (1995).

The depth to solid rock has near profile 13910 been measured to 9.8 meters and the maximum depth in the area to 17.6 meters. The ground consists mainly of firm clay in the top layers, but its strength decreases by depth. In the deeper layers we have sensitive clay and quick clay.
The tunnel itself will run from approximately profile 13400 to profile 13700, a distance of 300 meters. In the tunnel cut on the east side there is some uncertainty around the ground conditions since the geotechnical surveys have been done in the field just south of proposed line. The exact depth to solid rock along the line is also unknown. The new line will cross the old line in approximately profile 13800 at 1.02 m below the old line elevation. Based on surveys done further north in Naustan we assume solid rock at a depth of approximately 5 meters below sea level. More geotechnical surveys will have to be done in order to map the extent of the quick clay pockets in the area although we expect more stable conditions in the downhill just north of the old E6. The abutment for the railway bridge across the Malvik bay will be place in approximately profile 13815.

Haugan – Profile 14400 – 15900

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations reports UB.100579-000 (1959) and UB.101240-000 (1966).

The abutment for the railway bridge will be placed in approximately profile 14430.
After the bridge the line will run gradually 3 to 9 meters below terrain level, between profile 14500 and profile 14800. The exact depth to solid rock on this distance is somewhat uncertain, but is assumed to be in the range of 3 – to 10 meters. As UB.100579-000 (1959) states that the rock surface in the areas around the overpass in profile 14610 is quite uneven with varying depth below terrain level.
The ground is dominated by firm clay and silt, and the conditions are said to be good. The area is dominated by farm land and the line will split one of the fields in half. Given the good ground conditions in the area the cut is presumed to be stable with an angle of pitch in the standard relation

1:2. Still, surveys will have to be performed in order to confirm this presumption.
At profile 14800 the new line intersects the old line and will run on/alongside it until profile 15900. The proposed line will run approximately 3 meters lower in the terrain than the old line, here too are the ground conditions expected to be good, so an expansion of the causeway on this distance is considered to be unproblematic from a geotechnical point of view.

Torp – Midsanden – Profile 15900 – 16600

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations reports UB.100395-000 (1953) and UB.101241-000 (1966).

At profile 15900 the new line breaks off from the old line and continues through farm land to profile 16060. The ground conditions are expected to be similar to the conditions in the previous profile interval.
At profile 16080 the line crosses a small chasm in the terrain. The stream running through the chasm has to be put in a culvert and an embankment constructed on top in order to cross the chasm.
At profile 16150 the terrain rises and we have between 6 to 10 meters of soil masses above the grade elevation of the line for a distance of 750 meters (to profile 16900). The exact depth to solid rock in this general area is unknown.
The area between profile 16200 and profile 16330 is somewhat populated and some houses needs to be either moved or demolished to make room for the line. With the given grade elevation of the line and the terrain it could be possible to run the line through a concrete culvert and fill back excavated masses. This way the houses could be moved back to their original position although this is a rather expensive alternative.
The ground in this area is dominated by dry-crust clay with clay of medium strength underneath. Given the high grade elevation, the angle of pitch in a cut here would probably have to be in a 1:2 - 1:3 relation to gain satisfactory stability. A cut will therefore consume much of the surface area.

Between profile 16400 and profile 16500 a new overpass would have to be built unless one chooses a culvert solution. The same goes for the small village between profile 16500 and profile 16600. The houses and cottages here would be in the way of a cut solution. However, the grade elevation of the rails and the terrain suggests the possibility of running the line through a concrete culvert here also.

Based on our blueprints and aerial photos it seems possible to squeeze a culvert in between the houses and leave them untouched, but this has to be investigated more closely.

Midtsanden – Profile 16600 – 17100

The ground conditions in this paragraph are gathered from the Norwegian National Rail Administrations report UB.101241-000 (1966).

For a small distance the line runs directly underneath the Fv 950 road. During construction parts of this road and a couple of other small local roads will have to be demolished. When it comes to the Fv 950, it has either to be moved or the railway has to run through a culvert until it intersects with the old railway line. An overpass (bridge) for the Fv 950 could also be a solution, but it is likely that the grade elevation of the road would change if it is to follow its original course after the construction period.
The ground conditions in this area, especially near the old line, are expected to be good. The ground is dominated by silt with sand and gravel underneath. The layer of silt is approximately 5 meters thick. The depth to solid rock was not determined by a geotechnical survey here, but is expected to be at 10 meters or deeper.

3.1.4 Alternative 2

Ranheim – Hundhammeren - Profile 7780 – 9825

This profile interval is similar to alternative 1 both in regard to the ground conditions and the course of the line. See alternative 1 for details.

Midtsanden - Profile 16600 - 16800
The new line intersects the old line in approximately profile 16800. Near the old line the ground conditions are said to be good (dominated by silt with sand and gravel underneath) and similar conditions are expected closer to the tunnel portal although surveys will have to be performed to confirm this. In profile 16650 the line crosses Sandvikbekken (stream) which will have to be put

through a culvert underneath the causeway. The depth to solid rock is unknown.

3.2 Geology

The Trondheim field is one of the main provinces within the Caledonian mountain range in Norway. It includes most of Sør-Trøndelag County and the inland parts of Nord-Trøndelag County.
The rock is mostly of sedimentary or volcanic origin, and it has a varying degree of metamorphosis. There are cases of almost non-metamorphic schist and limestone containing fossils, and there are cases of highly metamorphic garnetiferous mica schists and gneiss.
The rock in the Trondheim field is folded in several phases and along with over thrust, also in several phases, which makes the geotectonic difficult.

Within the Trondheim field there is a large amount of schistous and relatively weak rock. This means that the direction of the tunnel in relation to the foliated structure may be of great importance to the stability of the rock. In general, weak rock, i.e. Green schist, and thrust zones may cause stability problems locally.

The rock we come in contact with on the distance between Ranheim and Midtsanden consists mainly of green stone and green schist. Green stone is in most cases relatively massive, but may have developed a foliation. In general, green stone do not cause problems in tunneling. Green schist however is generally of far lesser quality and it may contain chlorite and also clay in some cases. This means that we may get poor stability in areas consisting of green schist. The degree of metamorphosis may still influence the stability of the rock. A higher degree of metamorphosis results in improved stability. Løset (2006)

Because of the areas vegetated state, along with it being a suburban area can make the location and direction (strike) of failure planes and thrust zones somewhat difficult to interpret from aerial photos, but as mentioned in NSBs Master Plan Ranheim – Malvik (1996), the main direction (strike) of the biggest failure planes in the area follow the structure of the topographical formations. This direction is North-north-east – South-south-west (NNE-SSW) which also is the main direction of joints in big parts of the area although systems of failure planes and joints also run more or less perpendicular to this direction, NW–SE and E-W. Other systems may dominate locally and there have been registered horizontal joints all over the area. The dip varies from north to a more eastern direction.

3.2.1 Alternative 1

The tunnels

All the tunnels in this thesis run at very shallow depths. The maximum overburden in alternative 1 is 90 meters in the tunnel through Hundhammeren. We do therefore not expect to encounter tension related stability problems such as “crackle-rock”. Instead the main problem with these tunnels is to achieve sufficient overburden. Several places there will be need for jet grouting before blasting.

As we have performed no surveys on our own during the making of this thesis, the rock and tunnels, in regard to engineering geology, will only be compared to existing data from other tunnel projects in the region. Based on reports from these projects we will make an assumption on what to expect during construction of the new tunnels.
As mentioned the minimum requirement for overburden is set to 3 meters in this thesis. If the rock quality allows this is however a different matter which will not be devoted further attention other than mentioning other tunnel projects in the region where it has been done, i.e. the new Strindheim tunnel. In the new Strindheim tunnel where they use tunnel profile such as T 9.5 and T 12.5 they have run with as little overburden as 2 to 3 meters. This has however demanded that they secure the tunnel ceiling with fiber reinforced concrete arches as mentioned in Mapping of rock support and control of ground vibrations in Strindheimtunnel – part of the project E6 Trondheim – Stjørdal (Guddal with others, 2011).

Hundhammeren – Profile 9825 – 11230

The tunnel length will be about 1,405 meters. At the portals we have little or no overburden. The exact depth to solid rock from the surface in this profile is unknown. From this point on the overburden increases to a maximum of about 90 meters in profile 10575 and then decreases towards the portal on the east side of Hundhammeren. The quality of the rock along the tunnel axis is unknown. Over solid rock we have varying amounts of soil masses and there have been registered soil filled chasms in the rock in this area according to UB.101808-000 (1996). It is a populated area with a large variety of houses. Otherwise the area is dominated by fields and patches of forest. On the west side of Hundhammeren the tunnel will run underneath houses with as little as 18 meters of rock and soil in between them and the tunnel roof. On the east side it is somewhat better with about 36 meters in between. Still, this requires light charges and easy blasting of almost the entire length of the tunnel in order to reduce tremors that could cause damage to buildings and infrastructure.

Another important issue to be aware of is the leakage of groundwater into the tunnel. As the tunnel more or less passes in between two hill tops; Hundhammeren and Aunvåttan, the risk of this happening is most definitely present although steep terrain could reduce the effect. Aunvåttan and Brannåsen are covered with sparse marshes with small streams running from them. If groundwater in the area were to leak into the tunnel, buildings with foundations on loose masses could be damaged due to consolidation of the soil.

Figure 4: Hundhammeren

This tunnel runs almost parallel to the Være tunnel which runs approximately 1.3 kilometers to the south.
In regard to failure planes, joints and rock type one can expect much the same conditions as for the Være tunnel. The rock types will mainly consist out of greenstone and green schist with possible appearances of phyllite, chlorite and clay filled joints.

Vikhammer – Profile 12700 – 12860

The tunnel length will be about 160 meters. Also here the portals must be in a profile with little or no overburden. Maximum thickness of overburden we get between profile 12750 and profile 12800 were it is approximately 9 meters. Most of the surface area above the tunnel belongs to the

Vikhammer camping site. This means that most of the caravans and such can be moved during the construction period. There are however a couple of buildings left that have to be considered. The buildings are placed over the section of the tunnel with the most overburden. The rock surface throughout the hill has not been thoroughly mapped, but based on the data at hand it seems possible to build the tunnel underneath these houses as long as light charges are used and otherwise go to great lengths to achieve careful blasting. The rock quality must be measured though, either through core samples or other methods.

There is varying depth to the rock surface and varying amount of soil masses on top. The foundations of the few buildings left will have to be mapped in order to decide whether or not extra measures have to be made to reduce consolidation of the soil due to water leakage although the risk is considered to be low.

Figure 5: Vikhammerløkka

This is not a very large hill and the terrain is not very steep. Without any bare rock and/or distinct natural chasms or cleavages in the terrain it is difficult to say anything about weak planes and failure planes. As to the rock types in this area, it is expected to encounter the same types here as in other parts of the region. That means mainly greenstone and green schist with transitional forms. One can also expect to find phyllite and clay-filled joints.

East from profile 12860 we no longer have any overburden with the given grade elevation of the line. The suggested concrete culvert from this point can most likely have most of its foundations built on solid rock.

Naustan – Profile 13400 – 13700

The tunnel length will be about 300 meters. Between profile 13450 and profile 13550 we have a maximum overburden of approximately 20 meters while between profile 13550 and 13675 the overburden varies from 3 to 6 meters. The surface area is partially vegetated and partially developed housing area.

The same conditions apply here as for the other tunnels. Due to the limited overburden light charges and careful blasting is advised. The exact depth to the rock surface will have to be determined and the type of foundations for the houses in the area mapped. As before, this is to decide what measures that have to be made in regard to damage caused by consolidation of the soil due to water leakage, but here also the risk is considered to be low.

Figure 6: Naustan

Otherwise when it comes to water leakage in the tunnel there are no streams, marshes or small lakes in the immediate area so the ground water is presumably the only source.

Also here the rock types are believed to be mostly greenstone and green schist with their transitional forms. Phyllite and/or chlorite may occur locally.

3.2.2 Other tunnels in the region

The Være tunnel

This paragraph is based on a partial tunnel report obtained from Statens vegvesen 27.04.2011. Contact: Stig Gunnar Lillevik, Statens vegvesen.

The Være tunnel runs parallel 1.3 kilometers south of Hundhammeren.

The rock types that were encountered in the Være tunnel are of the Undre Hovin group of the Trondheim fields overlain rock types of Cambro-Silurian age. The rock material is in great extent of volcanic origin in form of lava or water transported sediments. It has a medium degree of metamorphosis. The characters of the schists vary from clay schist via sandstone to green schist with all its transitional forms, which makes the rock boundaries somewhat diffuse.
When it comes to failure planes and joints the area is under heavy tectonic strain and has apparently a very small degree of recrystallization after the last folding and thrust movements. To some extent, entire packets of rock have been exposed to shear fracture and internal movement which makes it difficult to get a clear picture of continuous planar failure planes.
For the most part in the Være tunnel, the rock types appear as schists converted from greenstone. In addition to the foliated structure there is an occurrence of joints which vary both in system and dispersion. It is typical that most these joints are filled with chlorite and/or clay. Due to the high degree of jointing they have experienced water leakage in most part of the Være tunnel.
Stabilization measures that were used in the Være tunnel were mainly plastered pipe bolts in combination with steel fiber reinforced shotcrete.

Gevingåsen 5 kilometers east of Midtsanden

Greywacke and conglomerate in interaction with sandstone can in some parts have great mechanical strength. By systematic segregation of the blasting one can assume that some of the blasted rock can be used for road purposes. UB101700-000 (1994)

The rock in the area around Gevingåsen has in general a medium to high degree of fracturing. The failure planes/joints are however impermeable and without fill or coating. They are in most cases plain, but often have an irregular course. In total, joints and failure planes made up 24 percent of the total E6-tunnel length, but only one of the failure planes was characterized as difficult. The smaller failure planes and joints were almost arduous. UB101700-000 (1994)

In the western part (the part which relates most to the tunnels proposed in this thesis) of the railway tunnel there were registered 3 failure planes by seismic surveys during planning. Also aerial photos and rock charts were used, and one could conclude that the western part of the tunnel would come across major failure planes. The seismic surveys showed that they could expect severely fractured/crushed rock.

The rock types along the E6-tunnel axis and the railway tunnel for that matter are of the Trondheim fields' Caledonian complex. The rock consists of sediments with a low degree of metamorphosis. The sediments vary from conglomerates, greywacke and sandstone containing lime to clay schists and phyllite. The rock types have been exposed to several generations of folding and they have an undulant structure on scale from centimeters to meters. The failure planes are complicated, but are mainly dominated by arduous joints and traverse joints. In Gevingåsen they were aware that they could find flat thrust zones, which is a typical feature in the Caledonian mountain range. These flat thrust zones /planes can reach all the way down to a few tens of meters and may therefore be hard to discover from surface observations. These zones can often be very slaty, have a high content of mica minerals, especially chlorite, and smooth undulant slickensides. UB101699-000 (1993)

Due to the limited overburden (less than 200 meters in this case) one does not expect to encounter tension related stability problems such as "crackle rock". Schist with a low E-module also helps in this regard. The schist sooner develops deformations of the type squeezing over time rather than an immediate tension release. UB101700-000 (1994)

3.2.3 Alternative 2

This is the more inland alternative. Up to profile 9700 and the tunnel portal at Hundhammeren west the railway line more or less follows alternative 1. From here it runs in an approximately 6.9 kilometer long tunnel up to profile 16625 at Midtsanden. The depth to the rock surface along the line has not been determined and the thickness of the overburden is unknown. When talking about overburden in this chapter both the actual overburden and the overlying soil masses are meant and the values they are given are close, but not precise. Due to the small overburden along most of the tunnel axis tremors can be a serious problem while blasting. This means that the blasting has to be planned carefully in regard to the amount of explosives used in each charge. Blast monitors should be installed in several locations in order to reduce the chance of causing damage to buildings and infrastructure.

Hundhammeren - Profile 9700 – 11150

This part of the tunnel follows alternative 1 almost to the point. That means that it too runs parallel with the Være tunnel and the rock conditions can be expected to be similar. On this distance the tunnel has a maximum overburden 96 meters so one has to take care during the tunnel blasting period. For further details see alternative 1.

Profile 10600 – 11700

This is the location of the proposed Vikhammer station. An underground station requires a bigger span than the regular standard section of the tunnel. Whether or not the rock quality and overburden allows for an expansion of this magnitude is uncertain and will not be assessed to any extent in this thesis.

In this profile interval we cross underneath the foot of a summit with as little as 27 meters to the surface. It is of great importance to establish the depth to the rock surface here as it is not guaranteed that there will be sufficient overburden. On the surface we have a developed housing area where the building foundations most likely are built on soil rather than solid rock. The risk of damage to buildings due to leakage of ground water into the tunnel is therefore high and there will probably be a great need for jet grouting in order to seal the tunnel. This is a likely location of failure planes. Otherwise one can here also expect conditions similar to the Være tunnel as it still runs parallel.

Profile 11700 – 12050

Up to profile 11700 there is a suburban housing area, but from here and up to profile 12050 the line runs underneath a farm with its fields and a vegetated area thereafter. There will have to be taken precautions not to cause damage to the buildings as a result of consolidation or blast tremors. The maximum overburden on this distance is 84 meters.

Profile 12050 – 12200

On this distance the tunnel crosses underneath the Vikhammer valley. This area could very likely be characterized by failure planes and weak rock types which continues deep into the rock. The volume of the soil and the depth to the rock surface on the bottom of the valley is unknown, but it is likely that Statens vegvesen (The Norwegian Public Roads Administration) has reports describing this since the Rv 941 runs alongside the Vikhammer river through the valley. The overburden at the bottom of the valley is approximately 30 meters. When crossing underneath the valley one should expect to rely heavily upon jet grouting to seal the tunnel since a possible failure plane and/or joints could in worst case "lead the river" into the tunnel. Given a failure plane here there will most likely already be water pressure in the rock so when masses are removed in the tunnel it is possible that water could start flowing into the tunnel.

Profile 12200 – 13900

From the Vikhammer valley the terrain rises until it yields a maximum overburden of approximately 147 meters in profile 13350. The surface area is mostly populated except for a distance of 350 meters between profile 12450 and profile 12800 where the tunnel passes underneath fields and vegetated area. The overburden on this distance is considered to be unproblematic, but as always blast tremors and consolidation of the soil due to water leakage could cause damage to buildings.

Profile 13900 – 14400

The tunnel passes underneath the foot of a summit with very little overburden. At the lowest point between profile 14050 and profile 14175 there is approximately 6 to 9 meters from the tunnel ceiling to the surface. A geological survey has yet to be performed here and depending on how much of the overburden that is soil it is not certain that one can go through with an ordinary tunnel here. It could be an alternative to "blow the roof" here and let the line see daylight, perhaps in order to make an emergency escape route? A concrete culvert could possibly also be an alternative. In either case one must be careful not to lower the ground water table whereas this could cause consolidation of the soil and thereby damage to buildings just to the west. The surface area at this point is in the middle of a field about 450 meters due north of the E6 highway. There are also smaller roads connecting the various farms in the immediate area.

Between profile 13900 and profile 14050 there is a cluster of farm houses. Due to the small overburden one must be careful not to cause to powerful tremors during blasting. Water leakage due to consolidation of the soil could cause problems.

Profile 14400 – 15650

From profile 14400 the terrain rises a little and reaches a maximum overburden of 57 meters. The tunnel crosses underneath Rv 873 in profile 14800 otherwise the surface area above the tunnel consists mainly out of fields and vegetation until it reaches the small river chasm in profile 15650 in which the Vulubekken stream runs through. The lowest point in this chasm lies approximately 18 meters above the tunnel ceiling, but the depth to the rock surface is unknown. The situation here is much the same as in the Vikhammer valley. With possible failure planes and joints water pressure in the rock could cause a major problem. Also here one should expect to rely heavily on jet grouting when crossing underneath.

Profile 15650 – 16625

In approximately profile 15850 crosses under the E6 highway digression towards Midtsanden. The average overburden in this profile interval is about 27 meters. The overburden is presumed to be sufficient up to profile 16600. In profile 16600 there is a small local road and it is uncertain whether or not there will be sufficient overburden to support it or if a support construction or an overpass bridge solution has to be made. Either way the road will most likely have to close during construction.

4. Surplus gobbing

These calculations do not include causeway inside tunnels neither is the ballast. The numbers are merely approximate whereas the exact amount of what is what could not be determined, i.e. some of the excavated soil volume will in reality be blasted rock. The numbers are extracted from Novapoint with the preset that there are cuts where there most likely will be culverts hence a bigger amount of gobbing. See appendix 4 and 5 for printouts from Novapoint.

Alternative 1

Conversion factor BCM to LCM:	1.10	
Blasted rock:	221,100 BCM	
Excavated soil:	1,103,069 BCM	
Embankment (fill):	22,099 BCM	24,308 LCM
Causeway (ballast excluded):	83,875 BCM	

Alternative 2

Blasted rock:	817,700 BCM	
Excavated soil:	148,208 BCM	
Embankment (fill):	5281 BCM	5810 LCM
Causeway (ballast excluded):	15,432 BCM	

4.1 What to do with the surplus masses

Given the quality of the blasted rock and the strict regulations regarding the materials used in the causeway of a railway, only a small amount (mainly greenstone) of the blasted rock is applicable for use in the embankments of the railway which in turn should result in regulations regarding the fragmentation of the rock after blasting in different parts of the tunnels. Given that the greenstone is applied to embankments whenever possible there is still a huge amount of surplus gobbing left. The other rock materials; green schist, phyllite and chlorite are not really suited for construction purposes, but some of it can be used for berms which have somewhat lighter restrictions/regulations in regard to the materials used in them.

A possible solution to getting rid of the surplus masses is to use them in an expansion of the shoreline in the bays and/or other places along the line. The proposed bridges crossing the bays will probably meet resistance from the local population and an effort to give the area the best possible aesthetic view must be made. If this is to be the solution it will in most cases dictate from which end the tunneling is done in order to reduce the travel distance. Transportation is often an expensive affair.

5. Conclusion

In reality neither alternative is better than the other since both alternatives have its advantages and disadvantages. The cost of each line would normally be the deciding factor, but since we have neglected the cost in this thesis the lines have to be assessed in other terms.

Keeping Vikhammer station where it is today speaks for choosing alternative 1 whereas choosing alternative 2 might result in no station at all because the rock might not allow the spans needed to have an underground station. If further examinations conclude that an underground station is possible a lot has to be done to upgrade the infrastructure of roads, walkways and bicycle paths to the site of the station. Choosing alternative 1 yields a maximum speed of 200 km/h compared to the 250 km/h maximum speed allowed by alternative 2. Because the connecting line at Ranheim and at Midtsanden is dimensioned for lower velocities trains cannot take advantage of the maximum velocity on the proposed lines. Due to the small differences in elevation the trains can be expected to keep constant velocity throughout the proposed lines.

Some 20 to 30 houses have to be demolished or moved in alternative 1 to make way for the railway. This may be problematic due to the fact that the inhabitants of these buildings might not want to move. Alternative 2 does not have this problem.

Alternative 2 yields the possibility of going directly to Hommelvik instead of going to Midtsanden reducing the travel time even more.

The bridge across Være is in conflict with the regulation of the surrounding areas. If the bridge is eliminated the railway has to follow the existing line from Ranheim station to Hundhammeren and from there break away from the existing line and starts to follow one of the proposed lines. This will result in that trains will have travelled a longer distance to achieve the same velocity as with the bridge in place. The shoreline can be developed by use of excess masses and a bridge that is designed to be aesthetic is required.

This thesis is somewhat limited in terms of calculations of soil mechanics since no extra surveys where made and the reports used merely provides information on conditions in the general vicinity. Any calculations performed would have to be redone at a later stage with the exact material properties from surveys along the line.

However, the most problematic area is at Hundhammeren where the risk of puncturing the quick clay pockets is high. For alternative 1 where there is one short tunnel and it has openings on both sides of Hundhammeren there will be need for a larger quantity of stabilizing measures than for alternative 2 which only has one opening at Hundhammeren. The best form of stabilizing measure to be used here is believed to be cement lime piling which increases the characteristic shear strength before failure in the material. The exact extent of the cement lime piling could not be decided, but is believed to be great in order to stabilize the cut properly.

For alternative 2 and the tunnels in alternative 1 there is expected to be much rock of poor quality, but the trend today is to make do with the rock you have. The location of the line takes precedence and leaves little room for maneuvering around failure planes and areas with particularly weak rock. This also applies for soil mechanical aspects along the lines. In this regard the main problem is to achieve sufficient overburden. With the information at hand it is believed that the tunnel solutions in this thesis have accomplished just that although new surveys could partially disprove it. For alternative 2, water leakage could prove to cause damage to infrastructure. For alternative 1 the same risk is believed to be low.

The rock types encountered during tunneling is expected to be greenstone, green schist, phyllite and chlorite, where only greenstone is suitable for road and railway construction purposes.

A third option must also be considered when deciding the alternative to use. This is to look at the line all the way from Trondheim Central Station to Hommelvik.

6. References

Aarhaug, O. R. (1984) *Geoteknikk og fundamenteringslære*. Rud: NKI-forlaget

Guddal, M. R., Hagen, C. B. and Karlsen, C. S. (2011) *Mapping of rock support and control of ground vibrations in Strindheimtunnel – part of the project E6 Trondheim – Stjørdal*. Program for bygg og miljø, Høgskolen i Sør-Trøndelag, Trondheim

Jernbaneverket (1996) *UB.101808-000*. Trondheim.

Jernbaneverket (1995) *UB.101807-000*. Trondheim.

Jernbaneverket (1994) *UB.101700-000*. Trondheim.

Jernbaneverket (1993) *UB.101699-000*. Trondheim.

Jernbaneverket (1984) *UB.101800-000*. Trondheim.

Jernbaneverket (1966) *UB.101234-000*. Trondheim.

Jernbaneverket (1966) *UB.101236-000*. Trondheim.

Jernbaneverket (1966) *UB.101238-000*. Trondheim.

Jernbaneverket (1966) *UB.101240-000*. Trondheim.

Jernbaneverket (1966) *UB.101241-000*. Trondheim.

Jernbaneverket (1966) *UB.101772-000*. Trondheim.

Jernbaneverket (1964) *UB.100761-000*. Trondheim.

Jernbaneverket (1964) *UB.101762-000*. Trondheim.

Jernbaneverket (1964) *UB.101763-000*. Trondheim.

Jernbaneverket (1959) *UB.100577-000.* Trondheim.

Jernbaneverket (1959) *UB.100579-000.* Trondheim.

Jernbaneverket (1953) *UB.100395-000.* Trondheim.

Løset, F. (2006) *Norges tunnelgeologi*. Oslo: Norges geotekniske institutt

NSB (1996) *Hovedplan Ranheim – Malvik.* Trondheim.

Teknisk Regelverk (2011) *Underbygging: JD520: kapittel 5: Minste Tverrsnitt.* Oslo: Jernbaneverket

Teknisk Regelverk (2011) *Overbygning: Prosjektering: JD 530: kapittel 5: Sporets trasé.* Oslo: Jernbaneverket

Teknisk Regelverk (2011) *Overbygning: Prosjektering: JD 530: Kapittel 5: Vedlegg A.* Oslo: Jernbaneverket.

7. Figures

Front cover photo: Svein E. Sando (02.03.2007): *Være vest: Fra bro ved tidl. Være hp og vestover*

Figure 1: Development plan Være bay (http://webhotel2.gisline.no/GISLINEWebInnsyn_Trondheim/, 16.05.2011

Figure 2: Intersection at Være

Figure 3: Schematic diagram: Quick clay pockets

Figure 4: Hundhammeren

Figure 5: Vikhammerløkka

Figure 6: Naustan

Front cover photo Blueprint Booklet: Svein E. Sando (04.08.2006): *Være midten: Allé v. Minde småbruk*

Appendixes

Appendix 1

Appendix 2

Appendix 3

Appendix 4

Appendix 5

Appendix 6

Appendix 7

Appendix 8

Appendix 9

Appendix 10

Appendix 1

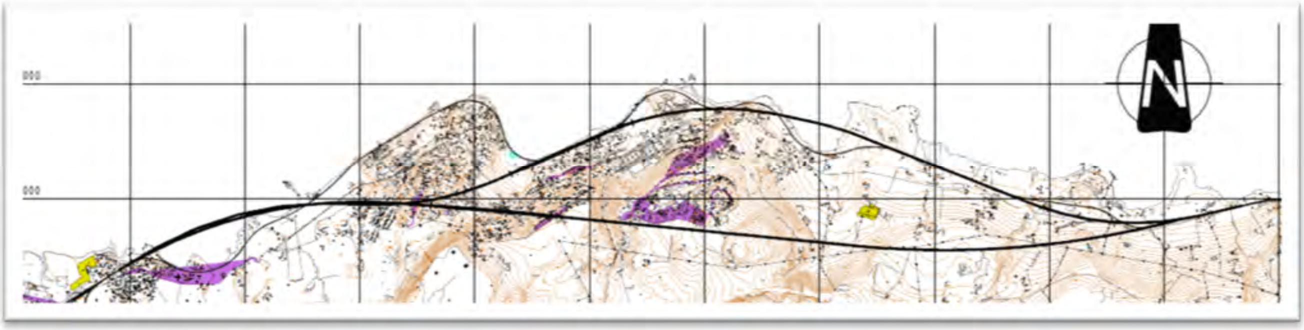
Article

From 100 km/h to 250 km/h?

Two students from Sør-Trøndelag University College are finishing their thesis concerning the reduction of travel time on the Nordlandsbanen between Trondheim and Steinkjer.

Two lines

In this thesis the students are proposing two alternative lines between Ranheim and Midtsanden that could reduce the travel time by 2-5 minutes. Their plans involve moving and straightening the line to allow trains to run at higher speeds.



Why two alternatives?

The main reason for proposing two alternatives was that the Norwegian National Rail Administration wanted to keep Vikhammer station. However, keeping Vikhammer station set restrictions on the achievable velocity within that distance. A second alternative would be adding an underground station at Vikhammer, which would allow trains to reach a velocity of 250 km/h.

Local challenges

One of the main challenges in getting one of the new lines approved is the bridge(s) across Være bay (and Malvik bay). The bridge(s) could have a great visual impact on the local area and might meet heavy resistance from the local community. The students think the bridge(s) are an unfortunate necessity to gain the needed velocity for the distance. But with careful design of the bridge(s) and the shoreline, they think the area can acquire quite an aesthetic view and perhaps become landmark in the region.

Another challenge along the lines is poor ground conditions. Large areas have sensitive and quick clay, so there will be a lot of "digging in the dirt". There will have to be a lot of stabilization of the ground in order for this to work.

Seizure of land areas

Large quantities of land, especially farmland, will have to be seized for the course of the new railway. Some of these areas can be redeveloped for residential or commercial purposes after construction has ended.

Political Decision

In the end, the approval of a new line will be a political decision. This proposition is given to the Norwegian National Rail Administration, who in conjunction with national and local governments makes the final decision. Changes in our proposition can, and most likely will be made before the final decision is reached.

Appendix 2

Poster

Studieprogram bygg og miljø

Prosjektnr
2011-06

Bacheloroppgave 2011
Proposition for new railway line between Ranheim and Midtsanden
Forslag til ny jerbanetrasé mellom Ranheim og Midtsanden

Jørn Fosen Simonsen
Eivind Pagander Tysnes

Ekstern kontakt: Jerbaneverket

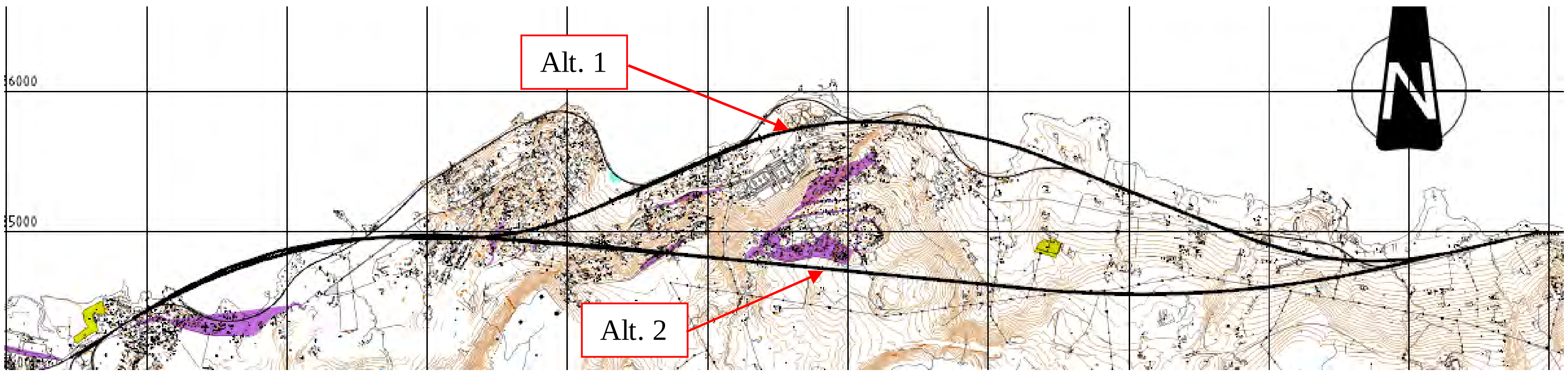
Intern veileder: Nils Kobberstad

The project:

In order to reduce the travel time by train between Trondheim and Steinkjer, two new railway lines are evaluated based on achievable train velocity and ground conditions

The lines:

Alt.1:
- Planned velocity: 200 km/h
- Track length: 10210m
- 3 tunnels
  - Hundhammeren: L = 1405 m
  - Vikhammer: L = 425 m
  - Naustan: L = 330 m



- Min radius: R = 2500 m
- 2 bridges
  - Være: L = 1490 m
  - Malvik: L = 615 m

Problem areas:

Hundhammeren: The ground is dominated by pockets of quick clay at shallow depths which may cause problems locally

The tunnels: Weak rock may cause alterations to the planned line

Jernbaneverket

Litteraturgrunnlag:

- Jerbaneverket: Geotekniske rapporter
- Jerbaneverket: Teknisk Regelverk
- Kart: Norge digitalt
- Løset F. (2006) *Norges tunnelgeologi.* NGI
- Aarhaug, O. R. (1984) *Geoteknikk og fundametingslære 1 og 2*. NKI Forlaget

Alt. 2
- Planned velocity: 250 km/h
- Track length: 9227 m
- 1 tunnel: L = 6900 m
- Min radius: R = 4000 m
- Bridge: Være: L = 1490 m


Foto: Svein E. Sando, 02.07.2003

Conclusion:

Both alternatives have advantages and disadvantages. In the end, it would be decided between the governmental authorities and the Norwegian National Rail Administration.

Appendix 3

Dictionary

English	Norwegian
Abutment	Landkar
Bank cubic meter (BCM)	prosjekterte faste masser (pfm3)
berm	motfylling, stabiliserende masse
Causeway	banelegeme, fylling
Consolidation	setning (geo)
Culvert	Kulvert
cut	skjæring(fjell, løsmasse)
Digression	Avstikker
diverging route	Avikende spor
drift clay, moraine	moreneleire
embankment	fylling
filter cloth	filterduk
Foot of the summit	Lavbrekk (topografisk)
formation	traubunn
gobbing	løsmasser
jet grouting	Injisering/jet injisering
Loose cubic meter (LCM)	utført anbrakte masser (plm3)
marine deposit	marin-/sjø-/fjordavstening
Overburden	Overdekning
pile dike	spuntvegg
Quick clay, marine clay	Kvikkleire (høy sensitivitet)
Shotcrete	Sprøytebetong
Slickenside	glidestripe
standard section	normalprofil
sub drain, culvert	stikkrenne
super elevation	ovehøyde
Switch	Sporveksel
undulant	bølgende, bølget

Appendix 4

Gobbing Alternative 1

Novapoint

Mengder sammendrag

Sammendrag

Modell: J:\RH - MS\Terrengmodell\Ranheim - Midtsanden

Start profil:	7780,00
Slutt profil:	8075,00
Dato sist endret:	16.05.2011

Mengde	Prosjekterte masser	Masseomreg-ningsfaktorer	Utførte anbrakte masser	Areal og lengde
Planering	m3		m3	
Jord	1967	1,00	1967	
Fjell	0	-	0	
Dypsprenging	0	-	0	
Fylling	9773	1,10	10750	
Tilskuddsmasser	m3		m3	
Jord	0	-	0	
Fjell	0	-	0	
Fylling	0	-	0	
Diverse mengder	m3			
Utskiftingsmasser:	0			
Matjord	0			
Vegetasjon	0			
Utlagte masser	1519			
Bakkeplanering, skjæring	0			
Bakkeplanering, fylling	0			
Justeringsmasser	0			
Avrunding, skjæring	0			
Avrunding, fylling	0			
Inngår i planering	m3			
Lukket grøft, jordskjæring	0			
Lukket grøft, fjellskjæring	0			
Lukket grøft, fylling	0			
Tilleggsflater, jordskjæring (+/- 3.x)	0			
Tilleggsflater, fjellskjæring (+/- 3.x)	0			
Tilleggsflater, fylling (+/- 3.x)	0			
Overbygning	m3			m2
Slitelag	0			0
Bindlag 1	0			0
Bindlag 2	0			0
Bærelag 1	345			3393
Bærelag 2	2296			3509
Forsterkningslag 1	1654			4058
Forsterkningslag 2	0			0
Filter- / Frostsikringslag	4			4161

Novapoint

Mengder sammendrag

Sammendrag

Modell: J:\RH - MS\Terrengmodell\Ranheim - Midtsanden

Start profil:	9565,00
Slutt profil:	9825,00
Dato sist endret:	16.05.2011

Mengde	Prosjekterte masser	Masseomreg-ningsfaktorer	Utførte anbrakte masser	Areal og lengde
Planering	**m3**		**m3**	
Jord	51394	1,00	51394	
Fjell	0	-	0	
Dypsprenging	0	-	0	
Fylling	1455	1,10	1601	
Tilskuddsmasser	**m3**		**m3**	
Jord	0	-	0	
Fjell	0	-	0	
Fylling	0	-	0	
Diverse mengder	**m3**			
Utskiftingsmasser:	0			
Matjord	0			
Vegetasjon	0			
Utlagte masser	1400			
Bakkeplanering, skjæring	0			
Bakkeplanering, fylling	0			
Justeringsmasser	0			
Avrunding, skjæring	0			
Avrunding, fylling	0			
Inngår i planering	**m3**			
Lukket grøft, jordskjæring	0			
Lukket grøft, fjellskjæring	0			
Lukket grøft, fylling	0			
Tilleggsflater, jordskjæring (+/- 3.x)	0			
Tilleggsflater, fjellskjæring (+/- 3.x)	0			
Tilleggsflater, fylling (+/- 3.x)	0			
Overbygning	**m3**			**m2**
Slitelag	0			0
Bindlag 1	0			0
Bindlag 2	0			0
Bærelag 1	304			2992
Bærelag 2	1963			3094
Forsterkningslag 1	1240			3212
Forsterkningslag 2	0			0
Filter- / Frostsikringslag	3			2945

Novapoint

Mengder sammendrag

Sammendrag

Modell: J:\RH - MS\Terrengmodell\Ranheim - Midtsanden

Start profil:	11230,00
Slutt profil:	12700,00
Dato sist endret:	16.05.2011

Mengde	Prosjekterte masser	Masseomreg-ningsfaktorer	Utførte anbrakte masser	Areal og lengde
Planering	m3		m3	
Jord	227528	1,00	227528	
Fjell	0	-	0	
Dypsprenging	0	-	0	
Fylling	1512	1,10	1663	
Tilskuddsmasser	m3		m3	
Jord	0	-	0	
Fjell	0	-	0	
Fylling	0	-	0	
Diverse mengder	m3			
Utskiftingsmasser:	0			
Matjord	0			
Vegetasjon	0			
Utlagte masser	7771			
Bakkeplanering, skjæring	0			
Bakkeplanering, fylling	0			
Justeringsmasser	0			
Avrunding, skjæring	0			
Avrunding, fylling	0			
Inngår i planering	m3			
Lukket grøft, jordskjæring	0			
Lukket grøft, fjellskjæring	0			
Lukket grøft, fylling	0			
Tilleggsflater, jordskjæring (+/- 3.x)	0			
Tilleggsflater, fjellskjæring (+/- 3.x)	0			
Tilleggsflater, fylling (+/- 3.x)	0			
Overbygning	m3			m2
Slitelag	0			0
Bindlag 1	0			0
Bindlag 2	0			0
Bærelag 1	1730			17003
Bærelag 2	11076			17589
Forsterkningslag 1	6755			17779
Forsterkningslag 2	0			0
Filter- / Frostsikringslag	16			15960

Mengder sammendrag

Sammendrag

Modell: J:\RH - MS\Terrengmodell\Ranheim - Midtsanden

Start profil:	12860,00
Slutt profil:	13400,00
Dato sist endret:	16.05.2011

Mengde	Prosjekterte masser	Masseomreg-ningsfaktorer	Utførte anbrakte masser	Areal og lengde
Planering	**m3**		**m3**	
Jord	158760	1,00	158760	
Fjell	0	-	0	
Dypsprenging	0	-	0	
Fylling	616	1,10	678	
Tilskuddsmasser	**m3**		**m3**	
Jord	0	-	0	
Fjell	0	-	0	
Fylling	0	-	0	
Diverse mengder	**m3**			
Utskiftingsmasser:	0			
Matjord	0			
Vegetasjon	0			
Utlagte masser	3047			
Bakkeplanering, skjæring	0			
Bakkeplanering, fylling	0			
Justeringsmasser	0			
Avrunding, skjæring	0			
Avrunding, fylling	0			
Inngår i planering	**m3**			
Lukket grøft, jordskjæring	0			
Lukket grøft, fjellskjæring	0			
Lukket grøft, fylling	0			
Tilleggsflater, jordskjæring (+/- 3.x)	0			
Tilleggsflater, fjellskjæring (+/- 3.x)	0			
Tilleggsflater, fylling (+/- 3.x)	0			
Overbygning	**m3**			**m2**
Slitelag	0			0
Bindlag 1	0			0
Bindlag 2	0			0
Bærelag 1	644			6334
Bærelag 2	4115			6552
Forsterkningslag 1	2477			6559
Forsterkningslag 2	0			0
Filter- / Frostsikringslag	6			5823

Novapoint

Mengder sammendrag

Sammendrag

Modell: J:\RH - MS\Terrengmodell\Ranheim - Midtsanden

Start profil:	13700,00
Slutt profil:	13850,00
Dato sist endret:	16.05.2011

Mengde	Prosjekterte masser	Masseomregningsfaktorer	Utførte anbrakte masser	Areal og lengde
Planering	m3		m3	
Jord	14060	1,00	14060	
Fjell	0	-	0	
Dypsprenging	0	-	0	
Fylling	4825	1,10	5307	
Tilskuddsmasser	m3		m3	
Jord	0	-	0	
Fjell	0	-	0	
Fylling	0	-	0	
Diverse mengder	m3			
Utskiftingsmasser:	0			
Matjord	0			
Vegetasjon	0			
Utlagte masser	862			
Bakkeplanering, skjæring	0			
Bakkeplanering, fylling	0			
Justeringsmasser	0			
Avrunding, skjæring	0			
Avrunding, fylling	0			
Inngår i planering	m3			
Lukket grøft, jordskjæring	0			
Lukket grøft, fjellskjæring	0			
Lukket grøft, fylling	0			
Tilleggsflater, jordskjæring (+/- 3.x)	0			
Tilleggsflater, fjellskjæring (+/- 3.x)	0			
Tilleggsflater, fylling (+/- 3.x)	0			
Overbygning	m3			m2
Slitelag	0			0
Bindlag 1	0			0
Bindlag 2	0			0
Bærelag 1	187			1841
Bærelag 2	1214			1904
Forsterkningslag 1	787			2016
Forsterkningslag 2	0			0
Filter- / Frostsikringslag	2			1914

Novapoint

Mengder sammendrag

Sammendrag

Modell: J:\RH - MS\Terrengmodell\Ranheim - Midtsanden

Start profil:	14400,00
Slutt profil:	17990,00
Dato sist endret:	16.05.2011

Mengde	Prosjekterte masser	Masseomreg-ningsfaktorer	Utførte anbrakte masser	Areal og lengde
Planering	m3		m3	
Jord	649360	1,00	649360	
Fjell	0	-	0	
Dypsprenging	0	-	0	
Fylling	3918	1,10	4309	
Tilskuddsmasser	m3		m3	
Jord	0	-	0	
Fjell	0	-	0	
Fylling	0	-	0	
Diverse mengder	m3			
Utskiftingsmasser:	0			
Matjord	0			
Vegetasjon	0			
Utlagte masser	19133			
Bakkeplanering, skjæring	0			
Bakkeplanering, fylling	0			
Justeringsmasser	0			
Avrunding, skjæring	0			
Avrunding, fylling	0			
Inngår i planering	m3			
Lukket grøft, jordskjæring	0			
Lukket grøft, fjellskjæring	0			
Lukket grøft, fylling	0			
Tilleggsflater, jordskjæring (+/- 3.x)	0			
Tilleggsflater, fjellskjæring (+/- 3.x)	0			
Tilleggsflater, fylling (+/- 3.x)	0			
Overbygning	m3			m2
Slitelag	0			0
Bindlag 1	0			0
Bindlag 2	0			0
Bærelag 1	4214			41420
Bærelag 2	26850			42844
Forsterkningslag 1	15956			42525
Forsterkningslag 2	0			0
Filter- / Frostsikringslag	37			37198

Appendix 5

Gobbing Alternative 2

Novapoint

Mengder sammendrag

Sammendrag

Modell: J:\RH - MS\Banemodeller\alternativ 2

Start profil:	7780,00
Slutt profil:	8075,00
Dato sist endret:	16.05.2011

Mengde	Prosjekterte masser	Masseomreg-ningsfaktorer	Utførte anbrakte masser	Areal og lengde
Planering	m3		m3	
Jord	3240	1,00	3240	
Fjell	0	-	0	
Dypsprenging	0	-	0	
Fylling	5054	1,10	5560	
Tilskuddsmasser	m3		m3	
Jord	0	-	0	
Fjell	0	-	0	
Fylling	0	-	0	
Diverse mengder	m3			
Utskiftingsmasser:	0			
Matjord	0			
Vegetasjon	0			
Utlagte masser	1552			
Bakkeplanering, skjæring	0			
Bakkeplanering, fylling	0			
Justeringsmasser	0			
Avrunding, skjæring	0			
Avrunding, fylling	0			
Inngår i planering	m3			
Lukket grøft, jordskjæring	0			
Lukket grøft, fjellskjæring	0			
Lukket grøft, fylling	0			
Tilleggsflater, jordskjæring (+/- 3.x)	0			
Tilleggsflater, fjellskjæring (+/- 3.x)	0			
Tilleggsflater, fylling (+/- 3.x)	0			
Overbygning	m3			m2
Slitelag	0			0
Bindlag 1	0			0
Bindlag 2	0			0
Bærelag 1	345			3393
Bærelag 2	2284			3498
Forsterkningslag 1	2447			4021
Forsterkningslag 2	0			0
Filter- / Frostsikringslag	4			4018

Novapoint

Mengder sammendrag

Sammendrag

Modell: J:\RH - MS\Banemodeller\alternativ 2

Start profil:	9565,00
Slutt profil:	9825,00
Dato sist endret:	16.05.2011

Mengde	Prosjekterte masser	Masseomregningsfaktorer	Utførte anbrakte masser	Areal og lengde
Planering	m3		m3	
Jord	60233	1,00	60233	
Fjell	0	-	0	
Dypsprenging	0	-	0	
Fylling	227	1,10	250	
Tilskuddsmasser	m3		m3	
Jord	0	-	0	
Fjell	0	-	0	
Fylling	0	-	0	
Diverse mengder	m3			
Utskiftingsmasser:	0			
Matjord	0			
Vegetasjon	0			
Utlagte masser	1427			
Bakkeplanering, skjæring	0			
Bakkeplanering, fylling	0			
Justeringsmasser	0			
Avrunding, skjæring	0			
Avrunding, fylling	0			
Inngår i planering	m3			
Lukket grøft, jordskjæring	0			
Lukket grøft, fjellskjæring	0			
Lukket grøft, fylling	0			
Tilleggsflater, jordskjæring (+/- 3.x)	0			
Tilleggsflater, fjellskjæring (+/- 3.x)	0			
Tilleggsflater, fylling (+/- 3.x)	0			
Overbygning	m3			m2
Slitelag	0			0
Bindlag 1	0			0
Bindlag 2	0			0
Bærelag 1	303			2992
Bærelag 2	1961			3076
Forsterkningslag 1	1802			3252
Forsterkningslag 2	0			0
Filter- / Frostsikringslag	3			2716

Novapoint

Mengder sammendrag

Sammendrag

Modell: J:\RH - MS\Banemodeller\alternativ 2

Start profil:	16600,00
Slutt profil:	17007,00
Dato sist endret:	16.05.2011

Mengde	Prosjekterte masser	Masseomreg-ningsfaktorer	Utførte anbrakte masser	Areal og lengde
Planering	m3		m3	
Jord	84735	1,00	84735	
Fjell	0	-	0	
Dypsprenging	0	-	0	
Fylling	0	-	0	
Tilskuddsmasser	m3		m3	
Jord	0	-	0	
Fjell	0	-	0	
Fylling	0	-	0	
Diverse mengder	m3			
Utskiftingsmasser:	0			
Matjord	0			
Vegetasjon	0			
Utlagte masser	2153			
Bakkeplanering, skjæring	0			
Bakkeplanering, fylling	0			
Justeringsmasser	0			
Avrunding, skjæring	0			
Avrunding, fylling	0			
Inngår i planering	m3			
Lukket grøft, jordskjæring	0			
Lukket grøft, fjellskjæring	0			
Lukket grøft, fylling	0			
Tilleggsflater, jordskjæring (+/- 3.x)	0			
Tilleggsflater, fjellskjæring (+/- 3.x)	0			
Tilleggsflater, fylling (+/- 3.x)	0			
Overbygning	m3			m2
Slitelag	0			0
Bindlag 1	0			0
Bindlag 2	0			0
Bærelag 1	480			4735
Bærelag 2	3082			4859
Forsterkningslag 1	2720			5026
Forsterkningslag 2	0			0
Filter- / Frostsikringslag	4			4040

Appendix 6

Partial Tunnel Report: Være Tunnel

4.0 GEOLOGISKE FORHOLD

4.1 Bergarter

Bergartene som Væretunnelen går igjennom, tilhører Undre Hovingruppen av Trondheimsfeltets overskjøvne bergarter av kambrosilurisk alder. Det bergartsdannende materialet har i stor utstrekning vulkansk opprinnelse i form av lava eller vanntransporterte sedimenter, og er senere middels metamorfosert.

Skifrenes karakter varierer fra leirskifer via sandstein til grønnskifer med overgangsformer. Bergartsgrensene er derfor ofte diffuse.

4.2 Svakhetssoner og oppsprekking

Området er sterkt tektonisk påkjent, tilsynelatende med lite rekrystallisasjon etter de siste folde- og skyvebevegelsene.

Hele bergartspakker har vært utsatt for skjærbrudd og interne bevegelser, slik at det i liten grad fremstår noe entydig bilde med klart avgrensede, gjennomgående og plane knusningssoner.

I tunnelen opptrer bergartene i stor grad som skifre og omdannet grønnstein. Foruten skifrigheten opptrer flere sett av sprekker som varierer sterkt både i system og utbredelse.

Det er et typisk trekk at man på de fleste sprekker og stikk finner kloritt og/eller leire.

Den sterke oppsprekkingen fører til at det opptrer lekkasjer i form av jevnt fordelte drypp i størstedelen av tunnelen.

Primæroppsprekking og svake bergarter har medført at bergmassen utenfor konturen lett ble ytterligere skadet ved dårlig tilpasset sprengningsopplegg.

5.0 STABILITETSSIKRING

5.1 Anvendte metoder

Stabilitetssikringen er i hovedsak utført med ettergyste rørbolter i kombinasjon med stålfiberarmert sprøytebetong og stigebånd. Full betongsutstøping er utført på begrensede partier.

På grunn av stort behov for sprøytebetongsikring, ble driften tidlig lagt opp slik at mest mulig av sikringen ble fullført på stuff. Dette gjenspeiler seg i de relativt små mengdene med ettersikring som er utført.

5.2 Bolter

Det er i hovedsak benyttet endeforankrede rørbolter som sikring på stuff. Disse er senere gyst med ekspanderende mørtel. Valget av denne boltetypen framfor limforankrede bolter som er beskrevet i anbudet, skyldes dels problemer hos entreprenøren med å få innarbeidet god teknikk med limbolter, og dels kombinasjon av kloritt og vann i boltehullene. Til sikring bak stuff er det i hovedsak benyttet gyste kamstålbolter.

Tabell I gir en samlet oppstilling over anvendte boltetyper. Bolter med 2,4 m lengde er i hovedsak brukt til festing og stramming av fjellbånd.

Alle bolter er utstyrt med sfærisk underlagsplate, halvkuleformet brikke og mutter. Alle boltedeler er varmforsinket.

Gysing av rørbolter er stikkprøvekontrollert med resonansmålinger.

Boltetype	Lengde	Antall	Sum
Bolter på stuff			

Rør og limbolter	2,4 m	263 stk	
	3 "	4112 "	
	4 "	5132 "	
	6 "	16 "	9523
Innstøpte bolter	3 m	43 stk	
	4 m	298 "	341
Bolter på stuff			9864
Ettersikring			

Innstøpte bolter	3 m	582 stk	
Rør og limbolter	3 m	203 "	785
Bolter totalt			10649
			=====
Herav gyste rørbolter			7557

Tabell I: Oversikt over bolteforbruket i Væretunnelen

5.3 Fjellbånd

Det er i tunnelen montert tilsammen 7314 m varmforsinkete stigebånd. Disse er i hovedsak brukt som sikring i oppsprukne partier med vann og leire, kombinert med sprøytebetong. For en stor del har dette gjort det mulig å benytte sprøytebetong der det var dårlig heft, og full utstøpning ellers ville vært alternativet. I noen tilfeller var imidlertid bruken av stigebånd sterkt overdrevet i forhold til den nøkternt vurderte nytte.

5.4 Sprøytebetong

Som stabilitetssikring er det i tunnelen benyttet ca 2900 m^3 sprøytebetong. Det meste av betongen er armert med 75 kg EE fiber pr. m^3. Se oversikt i tabell II. Det er ikke benyttet membranherder på sprøytede flater.

I tillegg til den sprøytebetongen som er brukt til stabilitetssikring, er det brukt stålfiberarmert sprøytebetong som brannbeskyttelse utenpå PE-skumplater. Dette er rapportert separat, se kap. 6.5.

Sted	Type	Volum	Sum
Sprøytebetong på stuff			
	Uten fiber	25,5 m^3	
	Med stålfiber	2212 m^3	2237,5 m^3
Sprøytebetong bak stuff			
	Med stålfiber	704,5 m^3	704,5 m^3
Totalt sprøytebetong som stabilitetssikring			2942,0 m^3

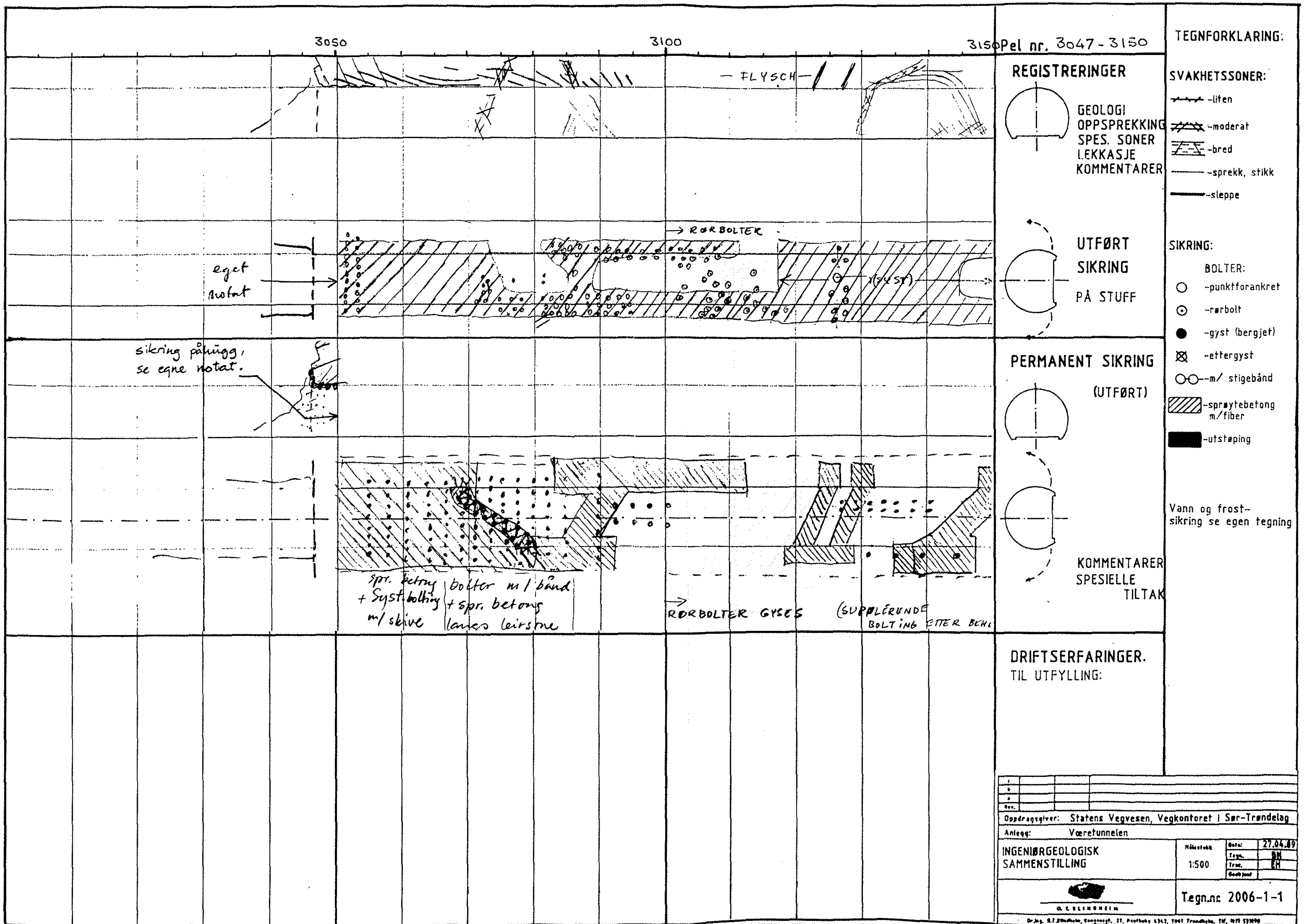
Tabell II: Forbruk av sprøytebetong ved stabilitetssikring

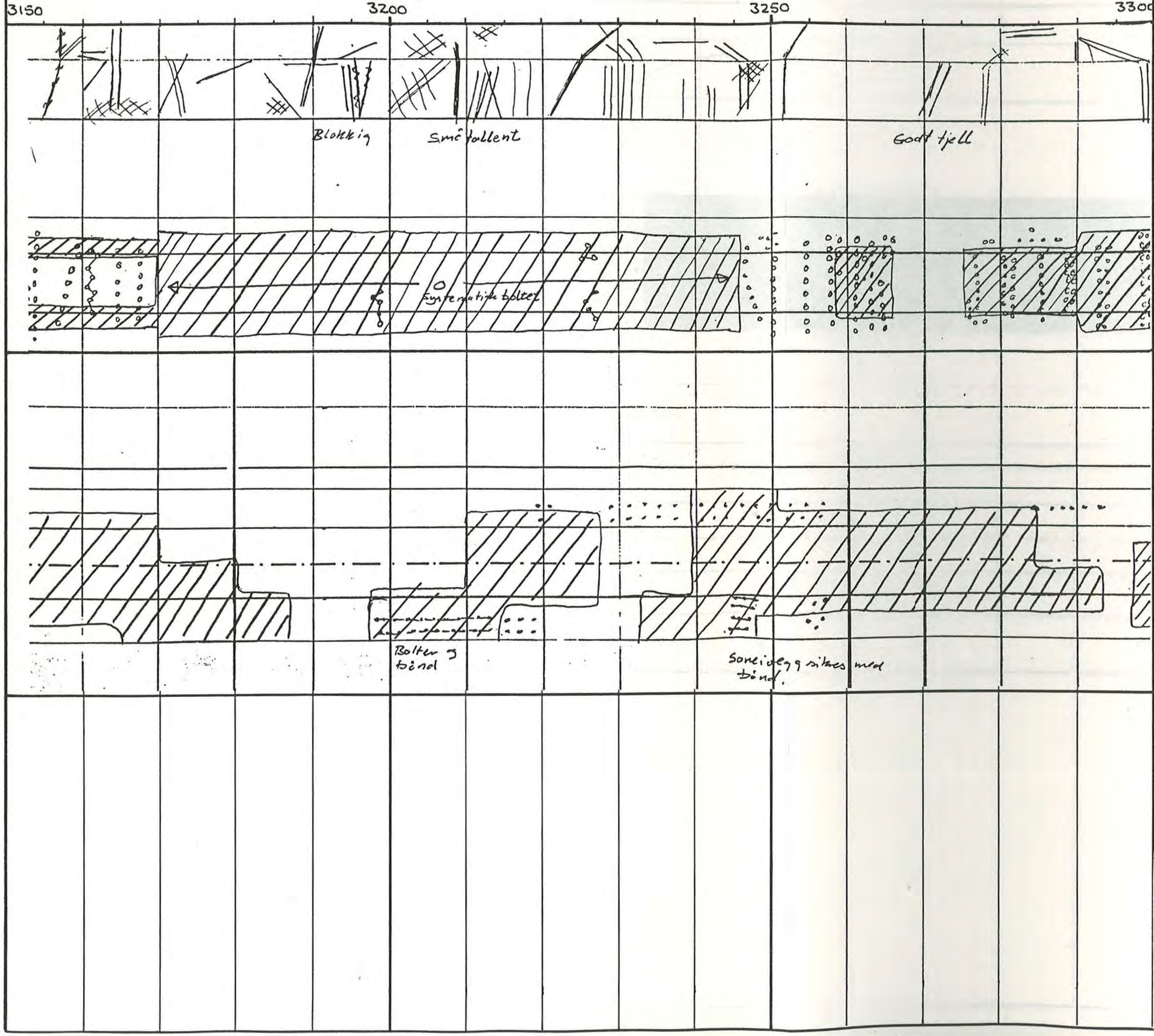
5.5 Betongutstøping

Det er i tunnelen tilsammen utført 212 m betongutstøping på stuff. Dette er fordelt på 7 partier langs hele tunnelen. Utstøping er valgt der kraftige leirsoner eller sterkt leirinfisert, oppsprukket og lekkasjeførende fjell har gjort lettere sikring utilstrekkelig.

Anbudet var forberedt for å tette støpeskjøter med injeksjon. Dette er ikke gjennomført, da en likevel vanskelig kunne unngå lekkasjer som ville kreve vann- og frostsikring.

En mindre del av de utstøpte partiene måtte sikres med bolter, bånd og sprøytebetong som arbeidssikring.


3050
3100
3150
Pel nr. 3047 - 3150
TEGNFORKLARING:
REGISTRERINGER
GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER
SVAKHETSSONER:
-liten
-moderat
-bred
-sprekk, stikk
-sleppe
FLYSCH
→ RØRBOLTER
eget notat
T(SYST)
UTFØRT SIKRING PÅ STUFF
SIKRING:
BOLTER:
-punktforankret
-rørbolt
-gyst (bergjet)
-ettergyst
-m/ stigebånd
-sprøytebetong m/fiber
-utstøping
sikring påhugg, se egne notat.
PERMANENT SIKRING
(UTFØRT)
Vann og frost-sikring se egen tegning
Spr. betong + Syst. bolting m/ skive
bolter m/ bånd + spr. betong
lanes leirstne
RØRBOLTER GYSES
(SUPPLERENDE BOLTING ETTER BEHL
KOMMENTARER
SPESIELLE TILTAK
DRIFTSERFARINGER.
TIL UTFYLLING:
Oppdragsgiver: Statens Vegvesen, Vegkontoret i Sør-Trøndelag
Anlegg: Væretunnelen
INGENIØRGEOLOGISK SAMMENSTILLING
Målestokk 1:500
Dato: 27.04.89
Tegn. BN
Trac. EH
Tegn.nr. 2006-1-1



Pel nr. 3150 - 3300

REGISTRERINGER

GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER

UTFØRT
SIKRING
PÅ STUFF

PERMANENT SIKRING

(UTFØRT)

KOMMENTARER
SPESIELLE
TILTAK

DRIFTSERFARINGER.

TIL UTFYLLING:

TEGNFORKLARING:

SVAKHETSSONER:

- -liten
- -moderat
- -bred
- -sprekk, stikk
- -sleppe

SIKRING:

BOLTER:

- -punktforankret
- -rørbolt
- -gyst (bergjet)
- -ettergyst
- -m/ stigebånd
- -sprøytebetong m/fiber
- -utstøping

Vann og frost-sikring se egen tegning

Rev.			

Oppdragsgiver: Statens Vegvesen, Vegkontoret i Sør-Trøndelag

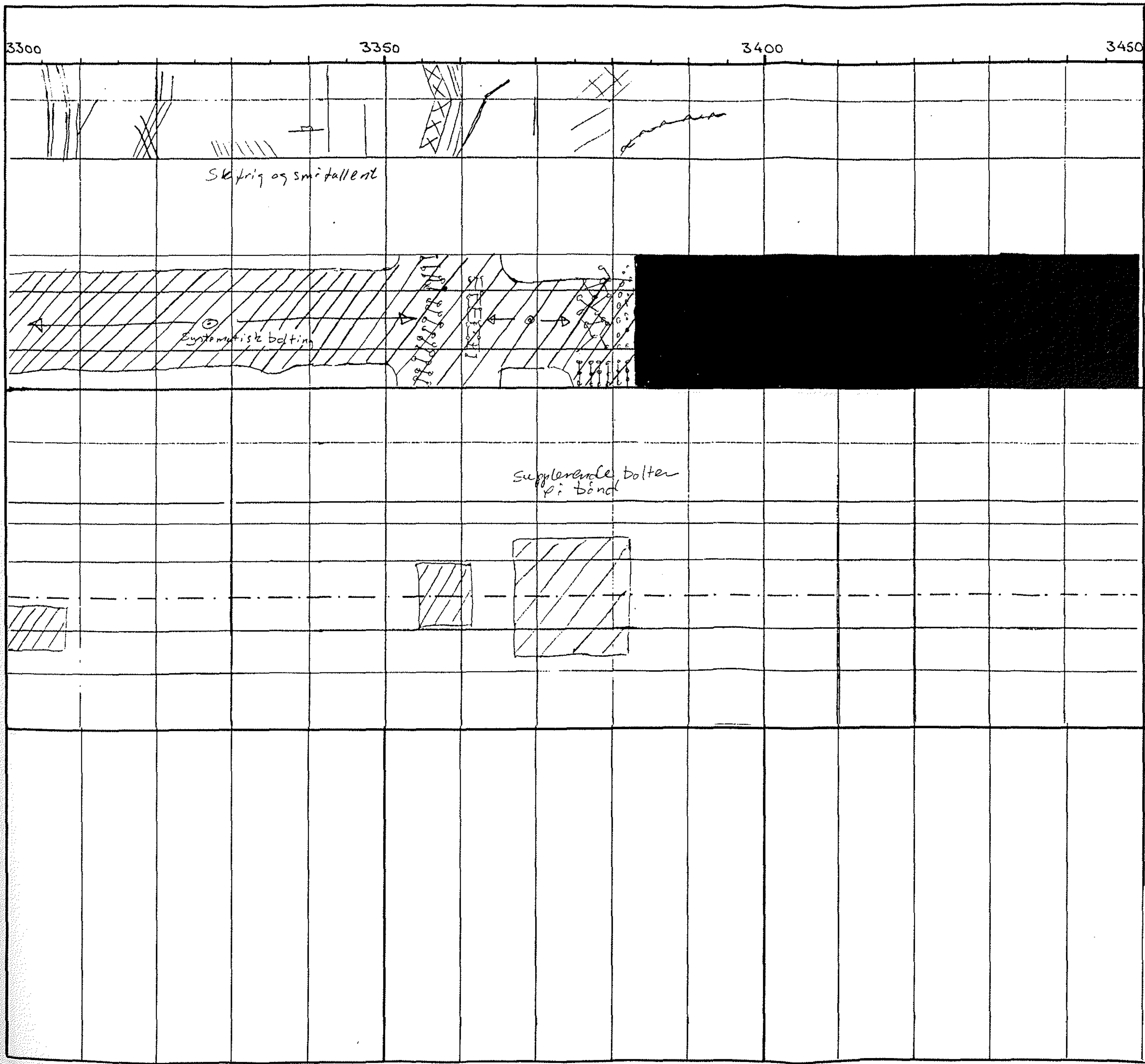
Anlegg: Væretunnelen

INGENIØRGEOLOGISK SAMMENSTILLING	Målestokk 1:500	Dato: 27.04.89
		Tegn. BN
		Trac. EH
		Godkjent

O. T. BLINDHEIM

Tegn.nr. 2006-1-2

Dr.ing. O.T.Blindheim, Kongensgt. 27, Postboks 4347, 7001 Trondheim, Tlf. (07) 531900



Pel nr. 3300 - 3450

REGISTRERINGER

GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER

UTFØRT
SIKRING
PÅ STUFF

PERMANENT SIKRING

(UTFØRT)

KOMMENTARER
SPESIELLE
TILTAK

DRIFTSERFARINGER.

TIL UTFYLLING:

TEGNFORKLARING:

SVAKHETSSONER:

- -liten
- -moderat
- -bred
- -sprekk, stikk
- -sleppe

SIKRING:

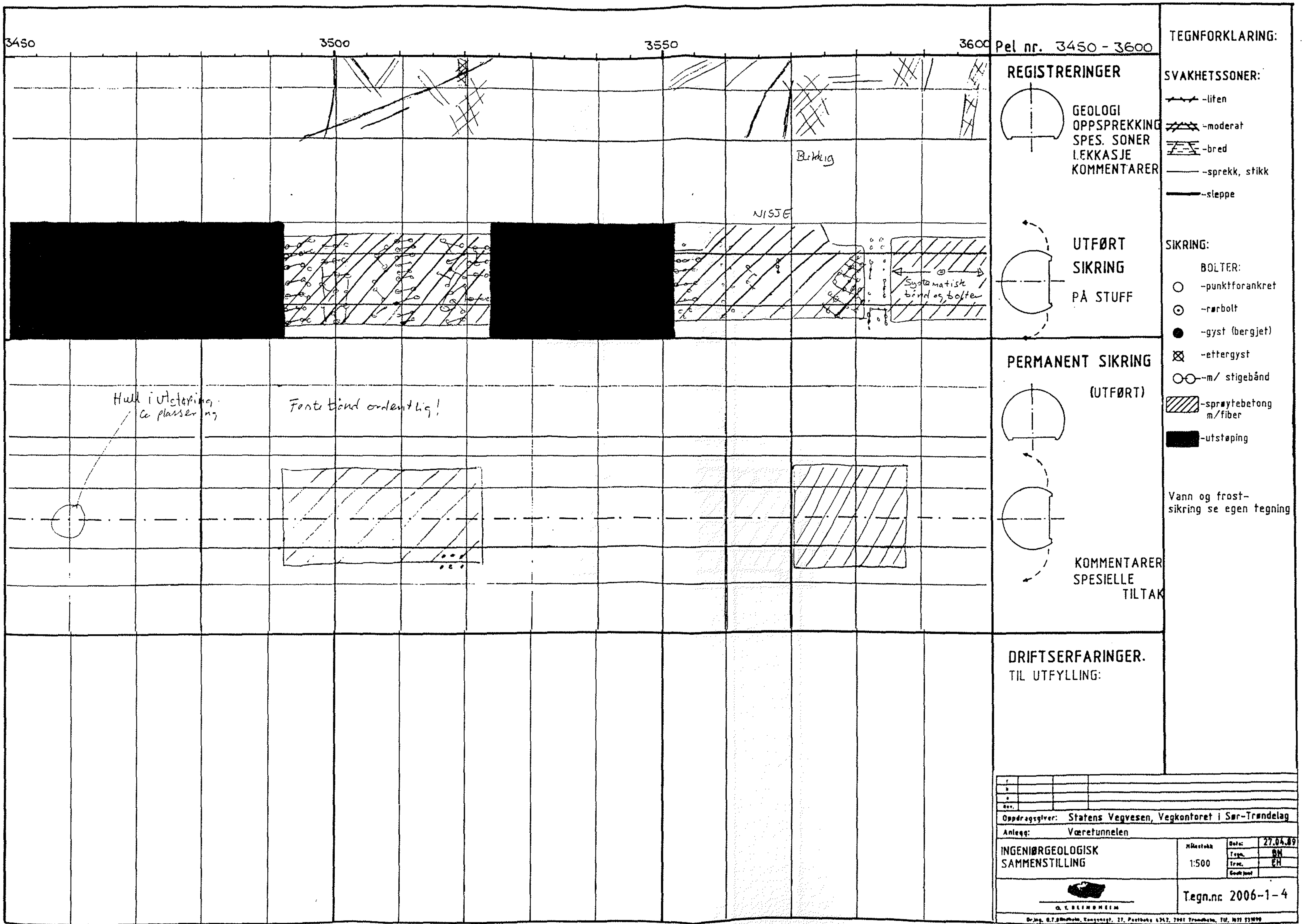
BOLTER:

- -punktforankret
- -rørbolt
- -gyst (bergjet)
- -ettergyst
- -m/ stigebånd
- -sprøytebetong m/fiber
- -utstøping

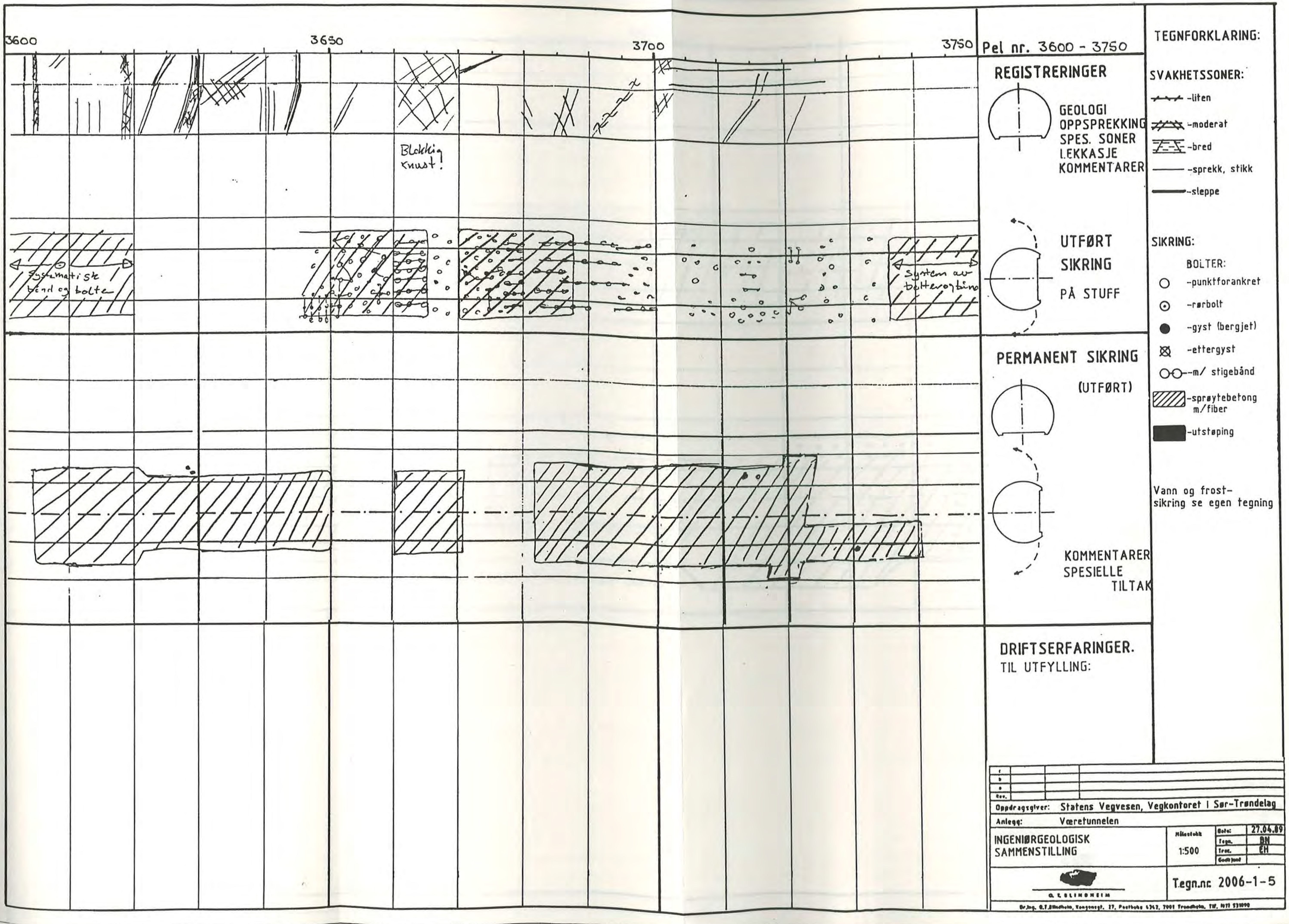
Vann og frost-sikring se egen tegning

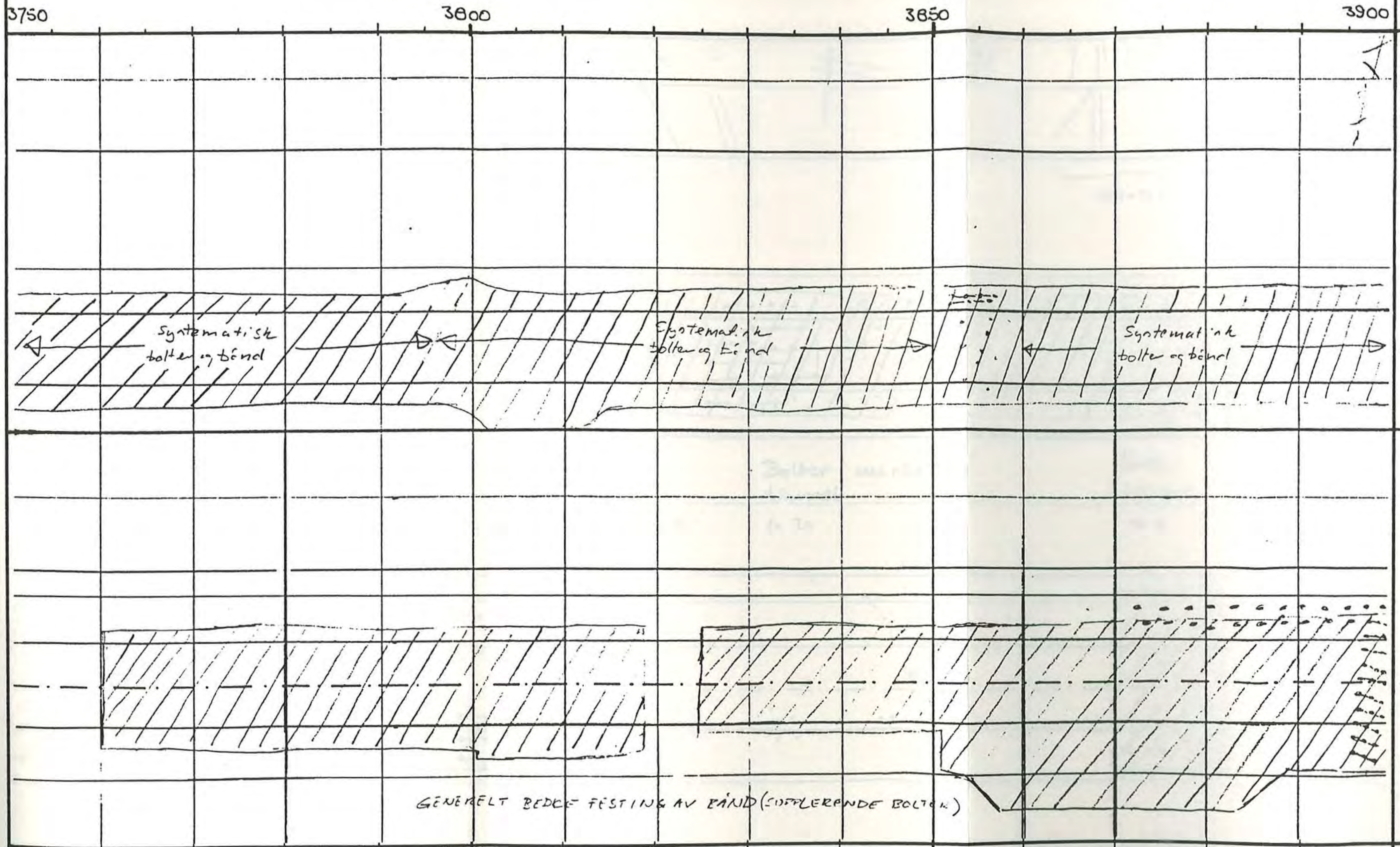
Rev.			
Oppdragsgiver:	Statens Vegvesen, Vegkontoret i Sør-Trøndelag		
Anlegg:	Væretunnelen		
INGENIØRGEOLOGISK SAMMENSTILLING	Målestokk 1:500	Dato:	27.04.89
		Tegn.	BN
		Trac.	EH
		Godkjent	
O. T. BLINDHEIM	Tegn.nr. 2006-1-3		

Dr.ing. O.T.Blindheim, Tungasletta 17, Postboks 6342, 7001 Trondheim, Tlf. (07) 531000



3450
3500
3550
3600
Bukkig
NISJE
Systematisk bånd og bolter
Hull i utstøping
Ca plassering
Feste bånd ordentlig!
Pel nr. 3450 - 3600
REGISTRERINGER
GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER
UTFØRT
SIKRING
PÅ STUFF
PERMANENT SIKRING
(UTFØRT)
KOMMENTARER
SPESIELLE
TILTAK
DRIFTSERFARINGER.
TIL UTFYLLING:
TEGNFORKLARING:
SVAKHETSSONER:
-liten
-moderat
-bred
-sprekk, stikk
-sleppe
SIKRING:
BOLTER:
-punktforankret
-rørbolt
-gyst (bergjet)
-ettergyst
-m/ stigebånd
-sprøytebetong m/fiber
-utstøping
Vann og frost-sikring se egen tegning
Rev.
Oppdragsgiver: Statens Vegvesen, Vegkontoret i Sør-Trøndelag
Anlegg: Væretunnelen
INGENIØRGEOLOGISK
SAMMENSTILLING
Målestokk
1:500
Dato: 27.04.89
Tegn. BN
Trac. EH
Godkjent
Tegn.nr. 2006-1-4


3600
3650
3700
3750
Pel nr. 3600 - 3750
REGISTRERINGER
GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER
Blokkig knust!
UTFØRT
SIKRING
PÅ STUFF
Systematiske bånd og bolter
System av bolter og bånd
PERMANENT SIKRING
(UTFØRT)
KOMMENTARER
SPESIELLE
TILTAK
DRIFTSERFARINGER.
TIL UTFYLLING:
TEGNFORKLARING:
SVAKHETSSONER:
-liten
-moderat
-bred
-sprekk, stikk
-sleppe
SIKRING:
BOLTER:
-punktforankret
-rørbolt
-gyst (bergjet)
-ettergyst
-m/ stigebånd
-sprøytebetong m/fiber
-utstøping
Vann og frost-sikring se egen tegning
Rev.
Oppdragsgiver: Statens Vegvesen, Vegkontoret i Sør-Trøndelag
Anlegg: Væretunnelen
INGENIØRGEOLOGISK
SAMMENSTILLING
Målestokk 1:500
Dato: 27.04.89
Tegn. BN
Trac. EH
Godkjent
Tegn.nr. 2006-1-5
O. T. BLINDHEIM
Dr.Ing. O.T.Blindheim, Kongensgt. 27, Postboks 4342, 7001 Trondheim, Tlf. (07) 531090



Pel nr. 3750 - 3900

REGISTRERINGER

GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER

UTFØRT SIKRING PÅ STUFF

PERMANENT SIKRING

(UTFØRT)

KOMMENTARER
SPESIELLE
TILTAK

DRIFTSERFARINGER.

TIL UTFYLLING:

TEGNFORKLARING:

SVAKHETSSONER:

- -liten
- -moderat
- -bred
- -sprekk, stikk
- -sleppe

SIKRING:

BOLTER:

- -punktforankret
- -rørbolt
- -gyst (bergjet)
- -ettergyst
- -m/ stigebånd
- -sprøytebetong m/fiber
- -utstøping

Vann og frost-sikring se egen tegning

c			
b			
a			
Rev.			

Oppdragsgiver: Statens Vegvesen, Vegkontoret i Sør-Trøndelag

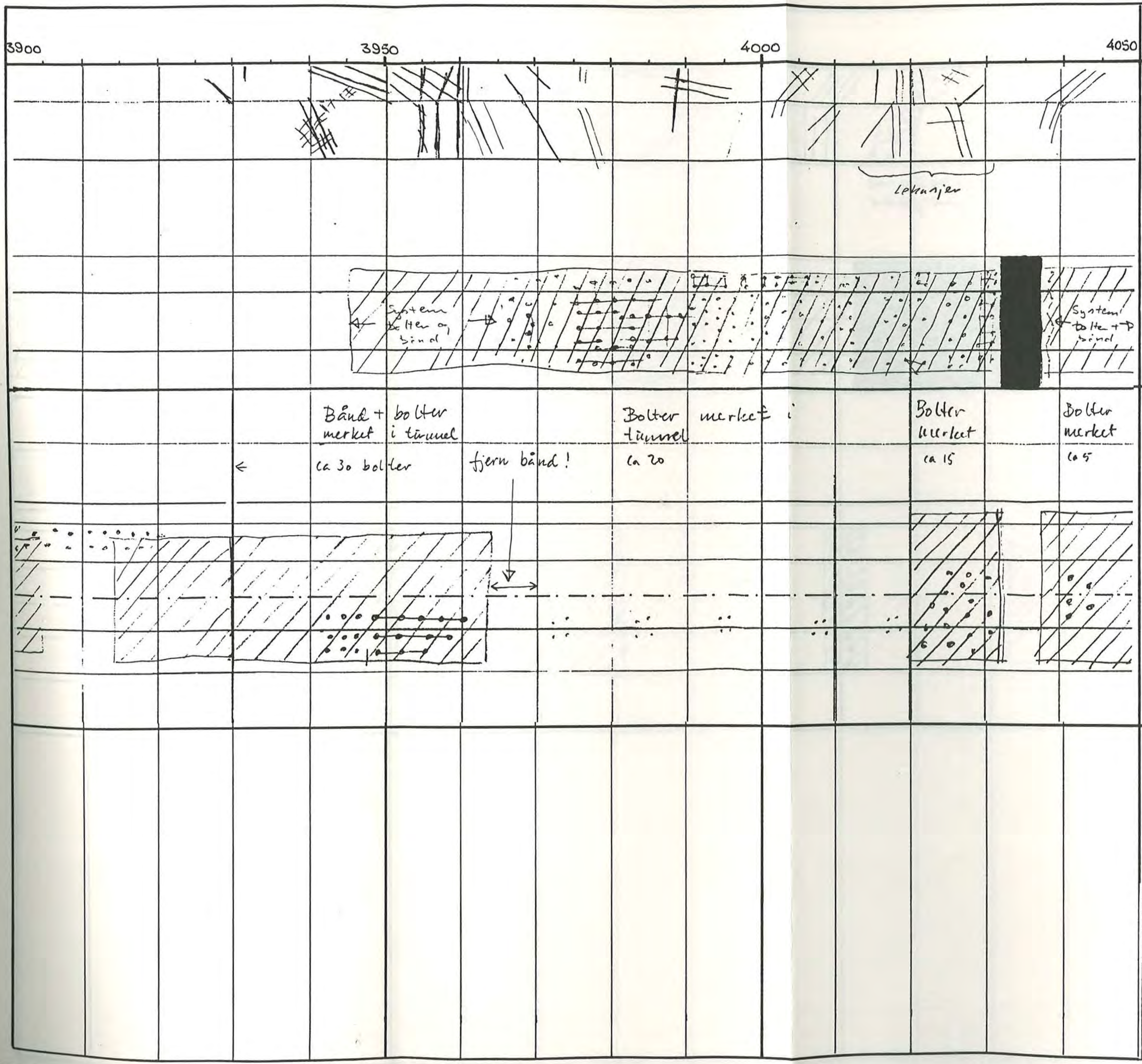
Anlegg: Væretunnelen

INGENIØRGEOLOGISK SAMMENSTILLING	Målestokk 1:500	Dato:	27.04.89
		Tegn.	BM
		Trac.	EH
		Godkjent	

O. T. BLINDHEIM

Tegn.nr. 2006-1-6

Dr.ing. O.T.Blindheim, Kongensgt. 27, Postboks 4347, 7001 Trondheim



Pel nr. 3900 - 4050

REGISTRERINGER

GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER

UTFØRT SIKRING PÅ STUFF

PERMANENT SIKRING (UTFØRT)

KOMMENTARER
SPESIELLE TILTAK

DRIFTSERFARINGER.
TIL UTFYLLING:

TEGNFORKLARING:

SVAKHETSSONER:
- -liten
- -moderat
- -bred
- -sprekk, stikk
- -sleppe

SIKRING:

BOLTER:
- -punktforankret
- -rørbolt
- -gyst (bergjet)
- -ettergyst
- -m/ stigebånd
- -sprøytebetong m./fiber
- -utstøping

Vann og frost-sikring se egen tegning

f			
b			
a			
Rev.			

Oppdragsgiver: Statens Vegvesen, Vegkontoret i Sør-Trøndelag

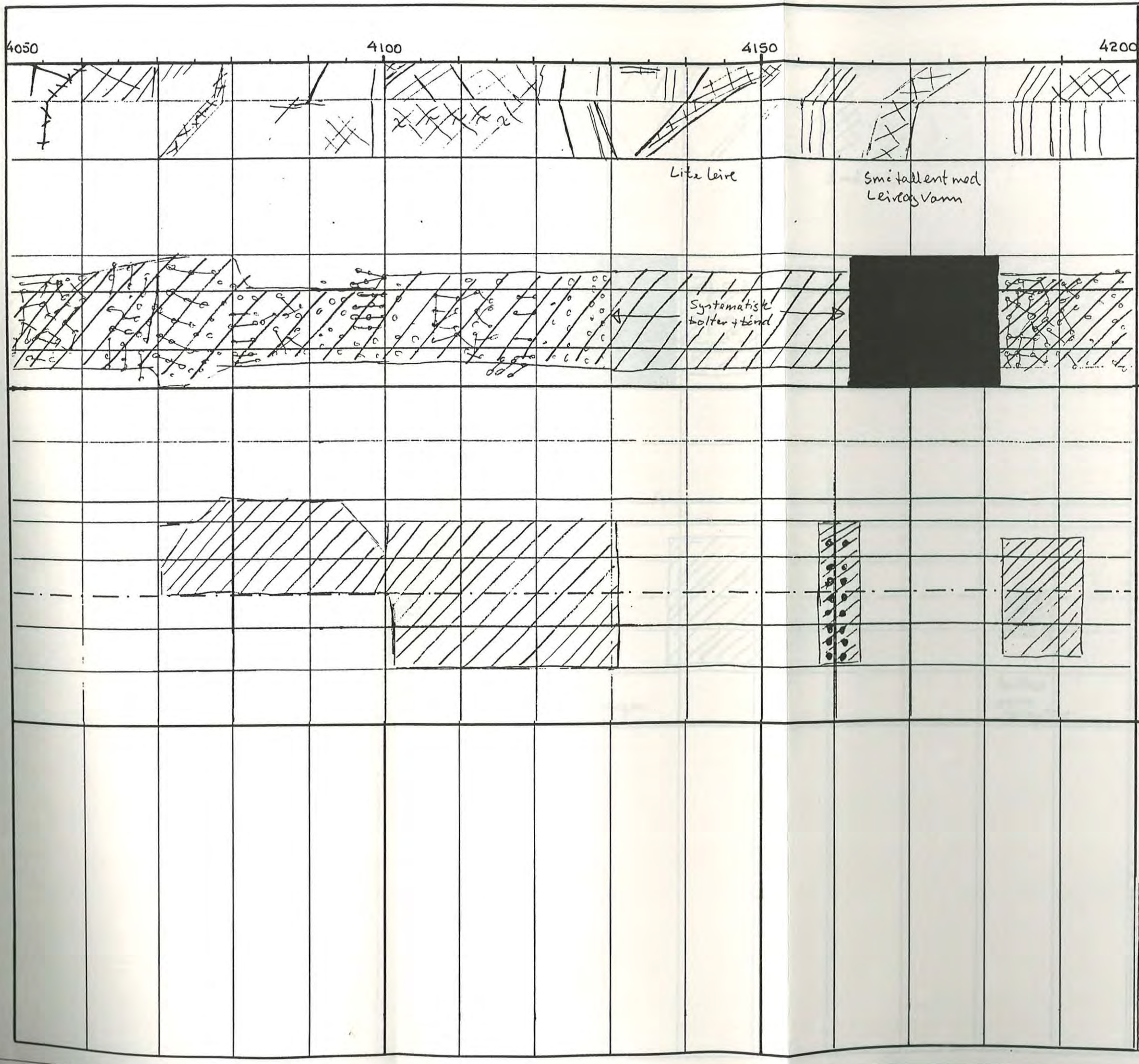
Anlegg: Væretunnelen

INGENIØRGEOLOGISK SAMMENSTILLING	Målestokk 1:500	Dato:	27.04.89
		Tegn.	BN
		Trac.	EH
		Godkjent	

O. T. BLINDHEIM

Tegn.nr. 2006-1-7

Dr.ing. O.T.Blindheim, Kongensgt. 17, Postboks 4747, 7001 Trondheim. Tlf. (07) 531070



Pel nr. 4050 - 4200

REGISTRERINGER

GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER

UTFØRT SIKRING

PÅ STUFF

PERMANENT SIKRING

(UTFØRT)

KOMMENTARER
SPESIELLE
TILTAK

DRIFTSERFARINGER.

TIL UTFYLLING:

TEGNFORKLARING:

SVAKHETSSONER:

- -liten
- -moderat
- -bred
- -sprekk, stikk
- -sleppe

SIKRING:

BOLTER:

- -punktforankret
- -rørbolt
- -gyst (bergjet)
- -ettergyst
- --m/ stigebånd
- -sprøytebetong m/fiber
- -utstøping

Vann og frost-sikring se egen tegning

c			
b			
a			
Rev.			

Oppdragsgiver: Statens Vegvesen, Vegkontoret i Sør-Trøndelag

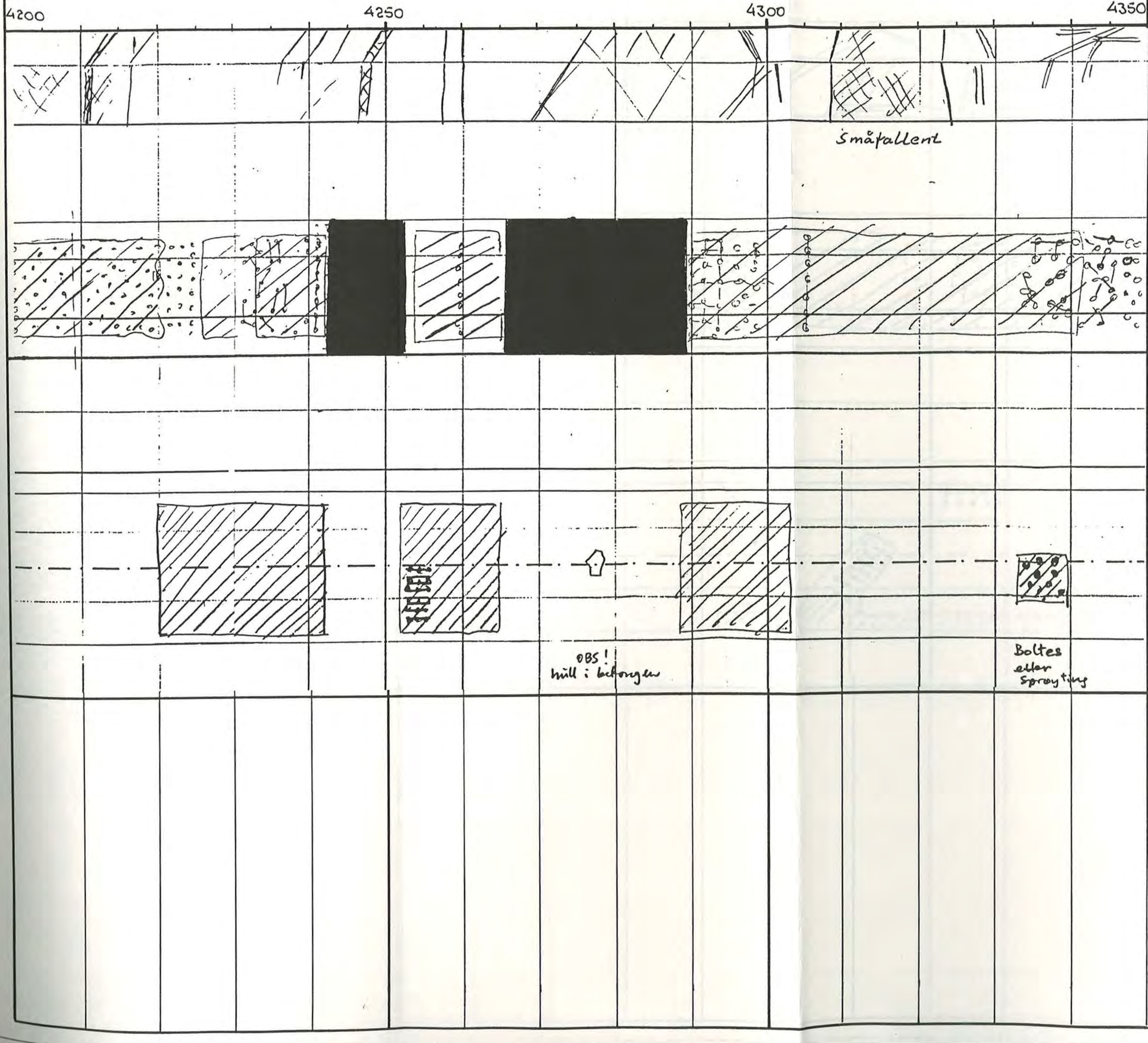
Anlegg: Væretunnelen

INGENIØRGEOLOGISK SAMMENSTILLING	Målestokk 1:500	Dato:	27.04.89
		Tegn.	BN
		Trac.	EH
		Godkjent	

O. T. BLINDHEIM

Tegn.nr. 2006-1-8

Dr.ing. O.T.Blindheim, Kongensgt. 27, Postboks 4347, 7001 Trondheim, Tlf. (07) 531090



Pel nr. 4200 - 4350

REGISTRERINGER

GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER

UTFØRT SIKRING PÅ STUFF

PERMANENT SIKRING

(UTFØRT)

KOMMENTARER
SPESIELLE TILTAK

DRIFTSERFARINGER.

TIL UTFYLLING:

TEGNFORKLARING:

SVAKHETSSONER:

- -liten
- -moderat
- -bred
- -sprekk, stikk
- -sleppe

SIKRING:

BOLTER:

- -punktforankret
- -rørbolt
- -gyst (bergjet)
- -ettergyst
- -m/ stigebånd
- -sprøytebetong m/fiber
- -utstøping

Vann og frost-sikring se egen tegning

c			
b			
a			
Rev.			

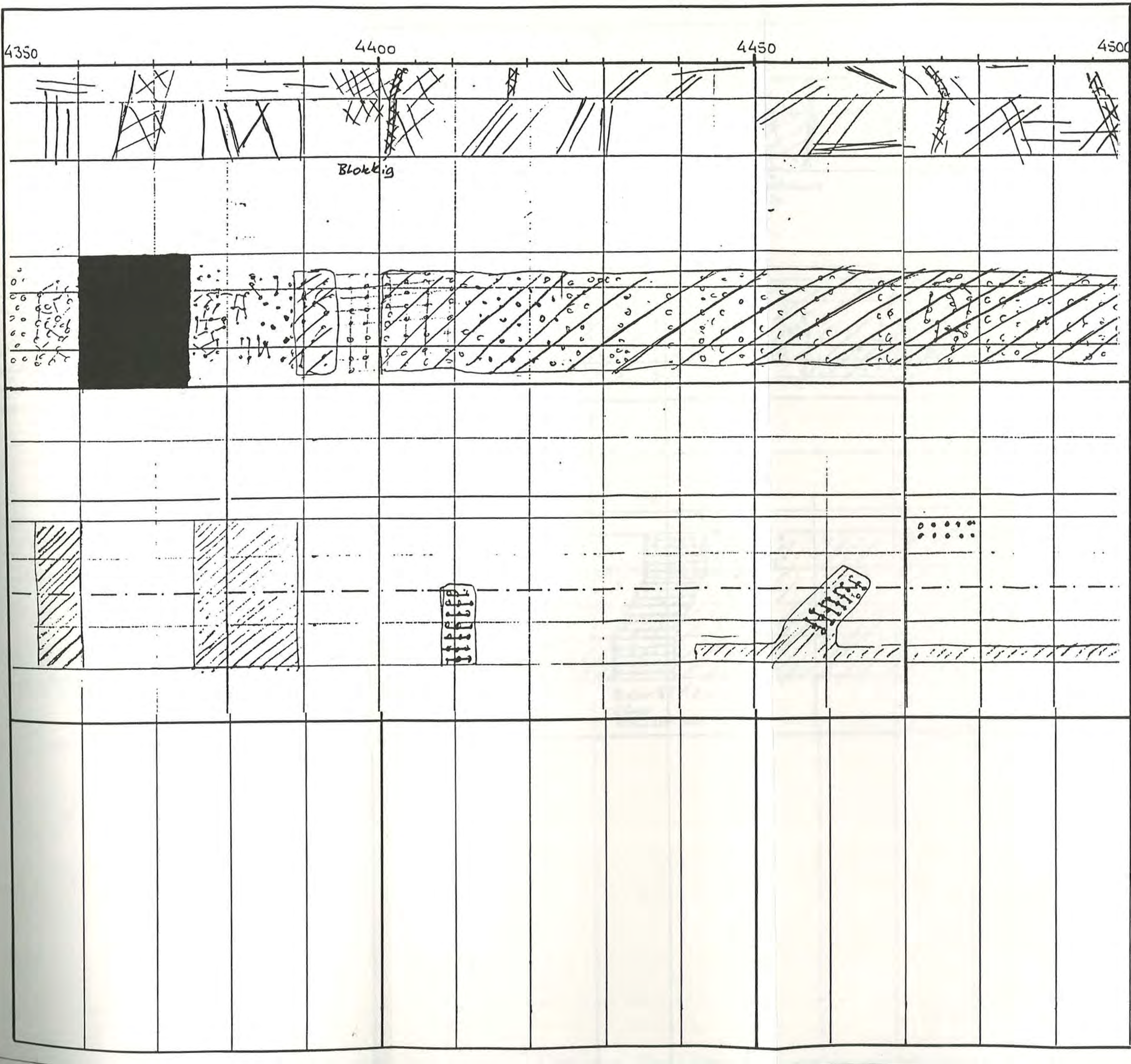
Oppdragsgiver: Statens Vegvesen, Vegkontoret i Sør-Trøndelag

Anlegg: Væretunnelen

INGENIØRGEOLOGISK SAMMENSTILLING	Målestokk 1:500	Dato: 27.04.89
		Tegn. BN
		Trac. EH
		Godkjent

O. T. BLINDHEIM

Tegn.nr. 2006-1-9



Pel nr. 4350 - 4500

REGISTRERINGER

GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER

UTFØRT
SIKRING
PÅ STUFF

PERMANENT SIKRING

(UTFØRT)

KOMMENTARER
SPESIELLE
TILTAK

DRIFTSERFARINGER.

TIL UTFYLLING:

TEGNFORKLARING:

SVAKHETSSONER:

- -liten
- -moderat
- -bred
- -sprekk, stikk
- -sleppe

SIKRING:

BOLTER:

- -punktforankret
- -rørbolt
- -gyst (bergjet)
- -ettergyst
- -m/ stigebånd
- -sprøytebetong m/fiber
- -utstøping

Vann og frost-sikring se egen tegning

Rev.			
c			
b			
a			

Oppdragsgiver: Statens Vegvesen, Vegkontoret i Sør-Trøndelag

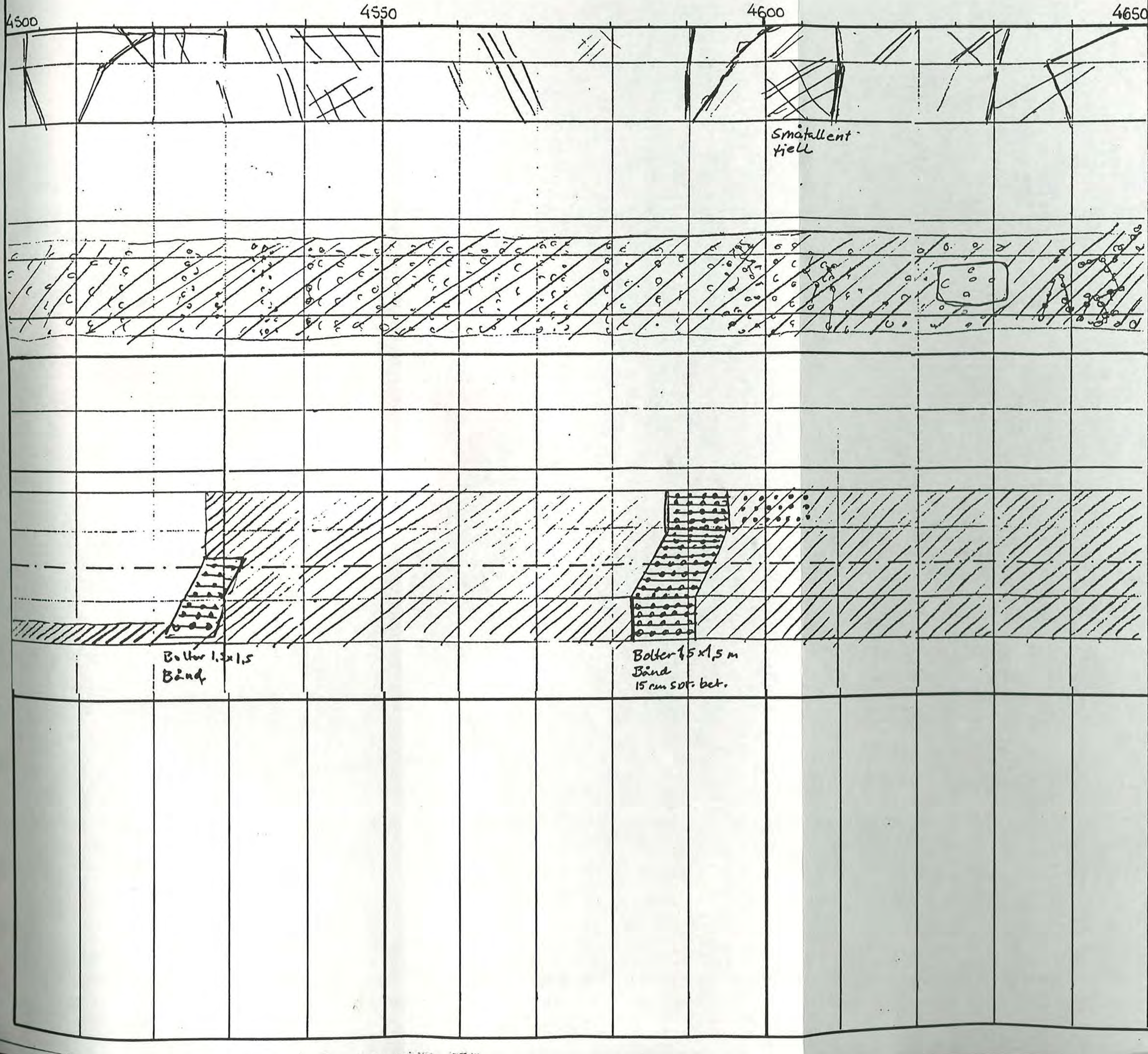
Anlegg: Væretunnelen

INGENIØRGEOLOGISK SAMMENSTILLING	Målestokk 1:500	Dato:	27.04.89
		Tegn.	BN
		Trac.	EH
		Godkjent	

O. T. BLINDHEIM

Tegn.nr. 2006-1-10

Dr.Ing. O.T.Blindheim, Kongensgt. 27, Postboks 4347, 7001 Trondheim, Tlf. (07) 531090



Pel nr. 4500 - 4650

REGISTRERINGER

GEOLOGI
OPPSPREKKING
SPES. SONER
LEKKASJE
KOMMENTARER

UTFØRT
SIKRING
PÅ STUFF

PERMANENT SIKRING

(UTFØRT)

KOMMENTARER
SPESIELLE
TILTAK

DRIFTSERFARINGER.
TIL UTFYLLING:

TEGNFORKLARING:

SVAKHETSSONER:
- -liten
- -moderat
- -bred
- -sprekk, stikk
- -sleppe

SIKRING:

BOLTER:
- -punktforankret
- -rørbolt
- -gyst (bergjet)
- -ettergyst
- -m/ stigebånd
- -sprøytebetong m/fiber
- -utstøping

Vann og frost-sikring se egen tegning

c			
b			
a			
Rev.			

Oppdragsgiver: Statens Vegvesen, Vegkontoret i Sør-Trøndelag

Anlegg: Væretunnelen

INGENIØRGEOLOGISK SAMMENSTILLING	Målestokk 1:500	Dato:	27.04.89
		Tegn.	BN
		Trac.	EH
		Godkjent	

Tegn.nr. 2006-1-11

G. T. BLINDHEIM

Dr.ing. G.T.Blindheim, Kongensgt. 17, Postboks 4342, 7001 Trondheim, Tlf. (07) 531090

Appendix 7

Calculations of velocity and acceleration

We will need to look at the distances travelled by trains to reach the proposed velocity for the two lines to see if they will reach that velocity before they reach the end of the new proposed lines and entering the existing line.

The following abbreviations will be used in the calculations of velocity and acceleration:

s – distance

v – velocity

v_0 – start velocity

a - acceleration

R – radius

h – super elevation

I_{maks} – lacking of super elevation

The following equations will be used in the calculations:

The relation between difference in velocity and distance and acceleration can be put as following

$$v^2 - v_0^2 = 2as \qquad (1)$$

Velocity in curves as per *Teknisk regelverk* JD530 chapter5.4.1 equation (5)

$$v = 0.291 \cdot \sqrt{R(h + I_{maks})} \qquad (2)$$

The following values will be used for acceleration.

$a_1 = 0.51$ m/s^2 for velocity below 80 km/h

$a_2 = 0.16$ m/s^2 for velocity between 80 km/h and 210 km/h

Since there aren't any data available for acceleration above 210 km/h we will assume the same value as for between 80 km/h and 210 km/h.

$a_3 = a_2 = 0.16$ m/s^2 for velocity above 210 km/h

For deceleration the value r_{max} = - 1.16 m/s^2 will be used.

All the above values are for NSB Type 73, except for a_3 which we assume from a_2.

In all calculations we assume that the track is horizontal and level. The line is a combination of horizontal rail, rail on an incline and rail on a decline.

Alternative one is dimensioned for 200 km/h and alternative two is dimensioned for 250 km/h. We will therefore calculate the distance of travel during acceleration and deceleration for both alternatives, the velocity reach by a passing train at Vikhammer station is also of interest for which type of switch will be used. At Ranheim Station the trains will pass by in 100 km/h unless stopped by a red signal.

From equation (1) we can get the following equation:

$$s = \frac{v^2 - v_0^2}{2a} \qquad (3)$$

We will be using equation (8) for the calculations of distance travelled during acceleration and deceleration.

Alternative one

This alternative will look at the proposed line where Vikhammer station will be where it is today. Because of geometry the highest velocity that will be allowed is 200 km/h.

The distance travelled by trains accelerating from 100 km/h to 200 km/h.

v_0 = 100 km/h = 27.78 m/s

v = 200 km/h = 55.56 m/s

$a = a_2 = 0.16 m/s^2$

(3) $s = \frac{(55.56\ m/s)^2 - (27.78\ m/s)^2}{2 \cdot 0.16\ m/s^2} \approx 7\ 235 m$

The distance travelled by trains accelerating from 0 km/h to 200 km/h.

v_0 = 0 km/h = 0 m/s

v_1 = 80 km/h = 22.22 m/s

v_2 = 200 km/h = 55.56 m/s

$a_1 = 0.51\ m/s^2$, $\quad$ 0 m/s < v < 80 km/h

$a_2 = 0.16\ m/s^2$, 80 m/s < v <200 km/h

(3) $s = \frac{v_2^2 - v_1^2}{2a_2} + \frac{v_1^2 - v_0^2}{2a_1} = \frac{(55.56 m/s)^2 - (22.22 m/s)^2}{2 \cdot 0.16\ m/s^2} + \frac{(22.22 m/s)^2 - (0 m/s)^2}{2 \cdot 0.51\ m/s^2} \approx 8\,588\ m$

The distance travelled by trains decelerating from 200 km/h to 100 km/h.

v_0 = 200 km/h = 55.56 m/s

v = 100 km/h =27.78 m/s

a = r_{max} = - 1.16 m/s^2

(3) $s = \frac{(22.78\ m/s)^2 - (55.56\ m/s)^2}{2 \cdot (-1.16\ m/s^2)} \approx 998\ m$

The distance travelled by trains decelerating from 200 km/h to 0 km/h.

v_0 = 200 km/h = 55.56 m/s

v = 0 km/h =0 m/s

a = r_{max} = - 1.16 m/s^2

(3) $s = \frac{(0 m/s)^2 - (55.56\ m/s)^2}{2 \cdot (-1.16\ m/s^2)} = 1\,331\ m$

The velocity for a passing trains at switch no. 1 at Vikhammer station for a train passing Ranheim station at 100 km/h.

From equation (1) we get:

$$v = \sqrt{2as + v_0^2} \qquad (4)$$

v_0 = 100 km/h = 27.78 m/s

a = 0.16 m/s^2

s = 11 350 m – 7 780 m = 3 570 m

(4) $v = \sqrt{2 \cdot 0.16\ m/s^2 \cdot 3\,570\ m + (27.78\ m/s)^2} \approx 43.75 m/s \approx 158\ km/h$

The velocity of trains passing switch no. 1 at Vikhammer station, for trains stopping for a red signal at Ranheim station.

v_0 = 0 km/h

v_1 = 80 km/h = 22.22 m/s

a_1 = 0.51 m/s^2, 0 m/s < v < 80 km/h

a_2 = 0.16 m/s^2, 80 m/s < v <200 km/h

s = 11 350 m – 7 780 m = 3 570 m

Distance travelled to reach 80 km/h

(3) $s_1 = \frac{(22.22 m/s)^2 - (0 m/s)^2}{2 \cdot 0.51 m/s^2} = 485\ m$

$\Delta s = s - s_1 = 3\,570\ m - 485\ m = 3\,085\ m$

(4) $v = \sqrt{2a \cdot \Delta s + v_1^2} = \sqrt{2 \cdot 0.16 m/s^2 \cdot 3\,085\ m + (22.22 m/s)^2} = 38.48 m/s = 139 km/h$

The maximum velocity allowed in curves is defined by equation (6). This equation gives us faster velocity for larger radius. In this alternative the smallest radius is 2 500 m. We will use this to calculate the highest velocity we can use on this alternative.

R = 2 500 m

h = 100 mm

I_{max} = 100 mm

(2) $V = 0.291 \cdot \sqrt{2\,500 \cdot (100 + 100)} = 205.77\ km/h$

The value of V is higher than the velocity on the planned line which is at 200 km/h.

Alternative two

This alternative will look at the proposed line where Vikhammer station will be in tunnel. This yields a better geometry for the tracks which again will yield higher velocity. The highest velocity allowed is 250 km/h.

The distance travelled by trains accelerating from 100 km/h to 250 km/h.

v_0 = 100 km/h = 27.78 m/s

v = 250 km/h = 69.44 m/s

a = a_2 = 0.16m/s^2

(3) $s = \frac{(69.44\ m/s)^2 - (27.78\ m/s)^2}{2 \cdot 0.16\ m/s^2} \approx 12\ 657 m$

The distance travelled by trains accelerating from 0 km/h to 200 km/h.

v_0 = 0 km/h = 0 m/s

v_1 = 80 km/h = 22.22 m/s

v_2 = 250 km/h = 69.44 m/s

a_1 = 0.51 m/s^2, 0 m/s < v < 80 km/h

a_2 = 0.16 m/s^2, 80 m/s < v <250 km/h

(3) $s = \frac{v_2^2 - v_1^2}{2a_2} + \frac{v_1^2 - v_0^2}{2a_1} = \frac{(69.44m/s)^2 - (22.22m/s)^2}{2 \cdot 0.16m/s^2} + \frac{(22.22m/s)^2 - (0m/s)^2}{2 \cdot 0.51m/s^2} \approx 14\ 010\ m$

The distance travelled by trains decelerating from 250 km/h to 100 km/h.

v_0 = 250 km/h = 69.44 m/s

v = 100 km/h =27.78 m/s

a = r_{max} = - 1.16 m/s^2

(3) $s = \frac{(27.78m/s)^2 - (69.44m/s)^2}{2 \cdot (-1.16\ m/s^2)} \approx 1\ 746\ m$

The distance travelled by trains decelerating from 250 km/h to 0 km/h.

v_0 = 250 km/h = 69.44 m/s

v = 0 km/h =0 m/s

a = r_{max} = - 1.16 m/s^2

(3) $s = \frac{(0m/s)^2 - (69.44\ m/s)^2}{2 \cdot (-1.16m/s)} = 2\ 079\ m$

The velocity for a passing trains at switch no. 1 at Vikhammer station for a train passing Ranheim station at 100 km/h.

v_0 = 100 km/h = 27.78 m/s

a = 0.16 m/s^2

s = 10 600 m – 7 780 m = 2 820 m

(4) $v = \sqrt{2 \cdot 0.16\, m/s^2 \cdot 2\,820\, m + (27.78\, m/s)^2} \approx 40.92 m/s \approx 148\, km/h$

The velocity of trains passing switch no. 1 at Vikhammer station, for trains stopping for a red signal at Ranheim station.

v_0 = 0 km/h

v_1 = 80 km/h = 22.22 m/s

a_1 = 0.51 m/s^2,   0 m/s < v < 80 km/h

a_2 = 0.16 m/s^2,   80 m/s < v <200 km/h

s = 10 600 m – 7 780 m = 2 820 m

Distance travelled to reach 80 km/h

(3) $s_1 = \frac{(22.22 m/s)^2 - (0 m/s)^2}{2 \cdot 0.51 m/s^2} = 485\, m$

$\Delta s = s - s_1 = 2\,820\, m - 485\, m = 2\,335\, m$

(4) $v = \sqrt{2a \cdot \Delta s + v_1^2} = \sqrt{2 \cdot 0.16 m/s^2 \cdot 2\,335\, m + (22.22 m/s)^2} = 35.23 m/s = 127 km/h$

The maximum velocity allowed in curves is defined by equation (6). This equation gives us faster velocity for larger radius. In this alternative the smallest radius is 2 500 m. We will use this to calculate the highest velocity we can use on this alternative.

R = 4 000 m

h = 90 mm

I_{max} = 100 mm

(2) $V = 0.291 \cdot \sqrt{4\,000 \cdot (90 + 100)} = 253.69\, km/h$

The value of V is higher than the velocity on the planned line which is 250 km/h.

Appendix 8

Dimensioning of frost blanket course and subbase

Teknisk regelverk JD 520 (Jernbaneverket, 2011, Chapter 9)

Dimensioning of frost blanket course and subbase (Z)

For main tracks we use F_{100} – values for the frost index in the different municipalities.

In these calculations we neglect the reduction factor for distance from the municipality center.

Since frost penetrates rock faster than gobbing, the gobbing has to be exchanged with non- frost susceptible materials to a certain depth D, and of a certain length L from the rock cut.

Z is the total sum of the frost blanket course and the subbase.

Trondheim municipality

F_{100} = 16,000 h°C (amount of frost in the air)

Material: gravel, sand Z = 0.9 meters

Material: 700 mm blasted rock subbase Z = 0.9 meters

Z (near rock cut) = 1.3 meters D = 0.5 x Z = 0.65 meters L = 10 x Z = 13 meters

Malvik municipality

F_{100} = 15,000 h°C (amount of frost in the air)

Material: gravel, sand Z = 0.82 meters

Material: 700 mm blasted rock subbase Z = 0.82 meters

Z (near rock cut) = 1.3 meters D = 0.5 x Z = 0.65 meters L = 10 x Z = 13 meters

Standard section and drainage course

Due to the large portion along the line where the earth materials have a relatively high sensitivity it is not enough with a filter cloth at the bottom of the causeways formation. The optimal solution may be filter cloth in combination with a 200 mm thick gravel drainage course.

For the standard section we now get a total thickness of:

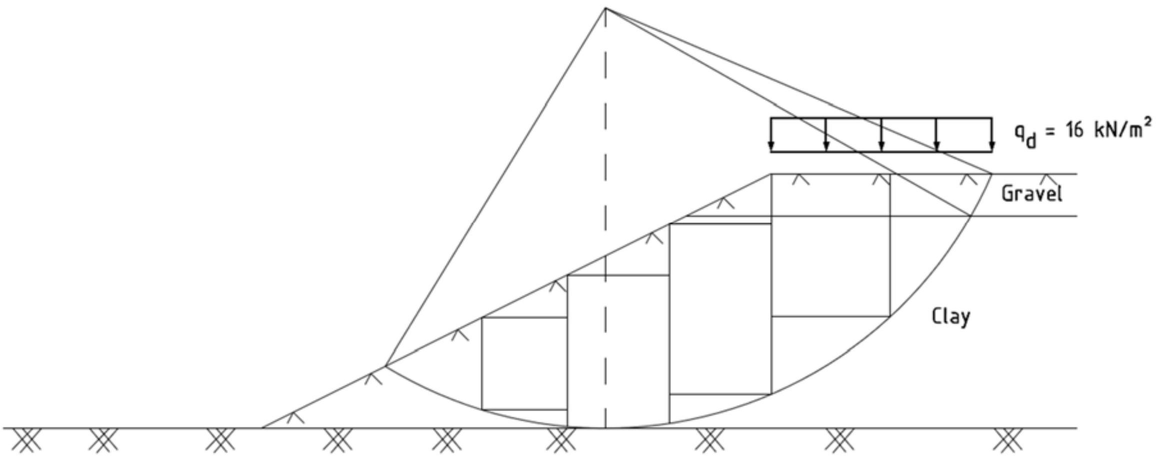
Trondheim municipality: $Z_{tot} = 0.9 + 0.2 = 1.1$ meters

Malvik municipality: $Z_{tot} = 0.82 + 0.2 = 1.02$ meters

For the sake of simplicity we have set $Z_{tot} = 1.0$ meters for the whole distance in this thesis.

Appendix 9

A simplified calculation of the stability of a cut at Vikhammer



Circular cylindrical shear section

(Direct method)

Height of cut:	H = 12 m
Depth to rock surface from the foot of the cut:	D = 0 m
Angle of pitch:	1:2, b =2, $\beta = 26.6^o$
Terrain load:	$q_d = 16\ kN/m^2$
Undrained shear strength:	$Su = 35\ kN/m^2$
Density:	$\rho = 2.0\ g/cm^3$, $\gamma = 20$
Correction factor (terrain load):	$\mu = 0.98$

Reference strength: $\sigma_d = \frac{\gamma H + q_d - \gamma_w H_w}{\mu_q \mu_w \mu_t} = 261\ kN/m^2$

Stability number: $N_c = 8.4$

Stability: $\gamma_m = N_c * \frac{S_u}{\sigma_d} = 1.13$

Location of the shear section: X_0 = 16.2 m

Y_0 = 19.8 m

R = 19.8 m

(Lamella method)

i	Δx_i [m]	x_i [m]	ΔG_i [kN]	$\Delta G_i' x_i$ [kNm]
1	4.6	-7.3	202.4	-1477.5
2	4.0	-3.1	76.0	-235.6
3	4.0	-3.8	176.0	-668.8
4	4.8	1.4	110.4	154.6
5	4.8	0.6	681.6	409.0
6	4.8	6.1	115.2	702.7
7	4.8	5.1	758.4	3867.8
8	4.0	-3.1	36.0	-111.6
9	4.8	4.6	67.2	309.1
10	5.6	10.6	761.6	8073.0
11	5.6	9.6	212.8	2042.9
12	4.8	15.0	364.8	5472.0
			ΣM_{drive} + Terrain load (= 2163.2) =	20700.0

$$\Delta l_i = \frac{2*pi*19.8*91^o}{360^o} = 31.45m$$

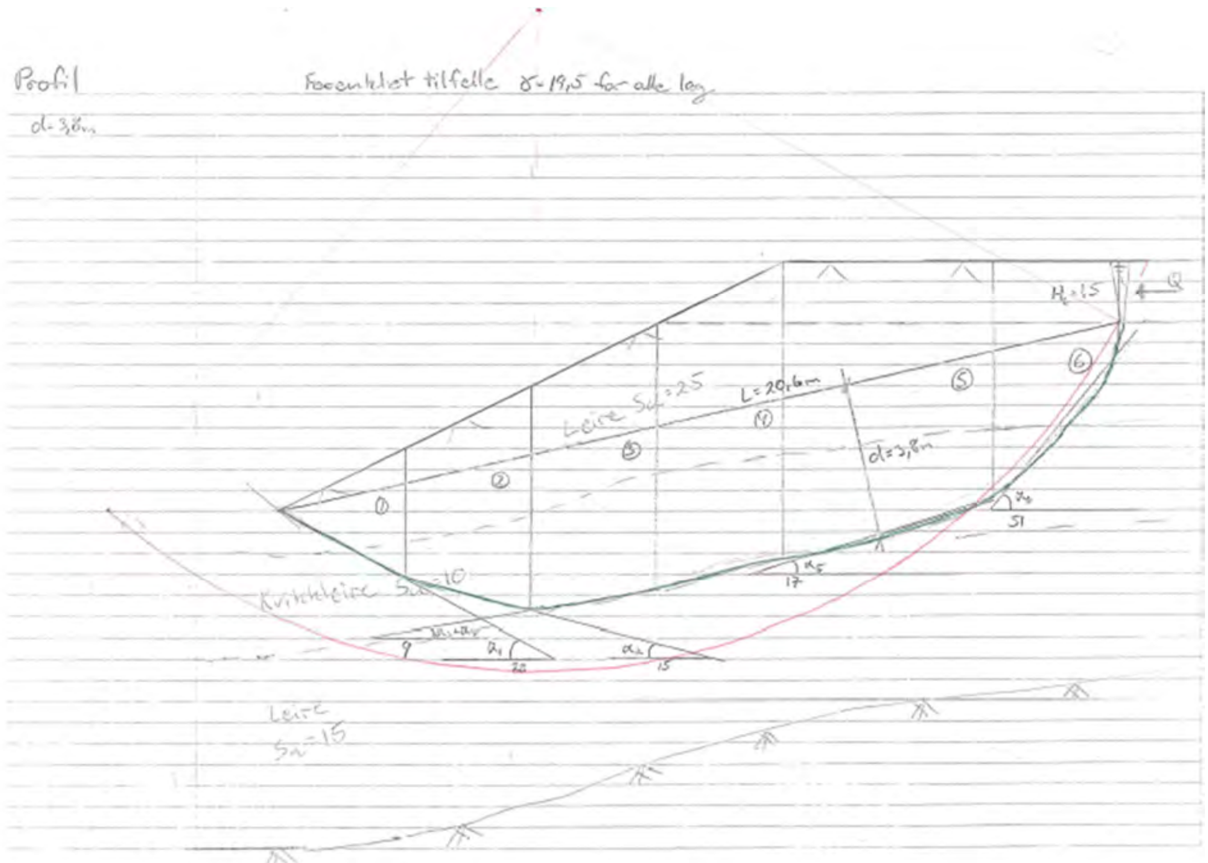
$$M_{stab} = R * S_u * \Delta l_i = 21795.0\ kNm$$

Stability: $\gamma_m = \frac{21795}{20700} = 1.05$

The results from both methods are in close proximity of each other, but the values are too small. For S_u – analysis the value of γ_m should be > 1.2. This means that the angle of pitch in the cut should be somewhat slighter.

Appendix 10

A simplified calculation of the stability of a cut at Hundhammeren



Complex shear section, circular cylindrical shear section

γ = 19.5

L = 20.6 m

D = 3.8 m

d/L = 0.18 => f_0 = 1.085

γ_f = 1.2 (E.5.1.4, NS3479)

Angle of pitch: 1:2

Water pressure from a crack in the dry-crust:

$$Q_d = 0.5 * H_t^2 * \gamma_w * \gamma_f = 13.5\ kN/m$$

Values from the profile							Calculations	
i	**tan α_i**	**$\cos^2 \alpha_i$**	**σ_{zi}**	**Δx_i**	**S_{ui}**		**$S_{ui}\, \Delta x_i / \cos^2 \alpha_i$**	**$S_{ui}\, \Delta x_i \tan \alpha_i$**
1	-0.53	0.28	41.0	3.0	10		107.1	-65.2
2	-0.27	0.93	89.7	3.0	10		32.3	-72.7
3	0.16	0.98	120.9	3.0	10		30.6	58.0
4	0.16	0.98	138.5	3.0	10		30.6	66.5
5	0.31	0.91	132.6	5.0	10		54.9	205.5
6	1.23	0.40	93.6	3.0	10		75.0	345.4
						Σ =	330.5	537.5

Stability: $\gamma_m = \frac{330.5}{13.5+537.5} = 0.6$

The cut is <u>not</u> stable with an angle of pitch: 1:2.

(Direct method)

H = 6 m

D = 8 m

B = 2, $\beta = 26.6^{o}$

$H_t = 1.5$ m

d/H = 1.3

$H_t/H = 0.25$

$\mu_t = 0.96$

$\gamma = 19.5$

$X_0 = 6$ m

$S_u = 17$ kN/m^2 (average value)

$Y_0 = 12$ m

$N_c = 5.65$

$$\sigma_d = \frac{\gamma * H}{\mu_t} = 122\ kN/m^2$$

Stability: $\gamma_m = \frac{N_c * S_u}{\sigma_d} = 0.8$

The cut is <u>not</u> stable with an angle of pitch: 1:2.

Both methods conclude that the cut needs a slighter angle of pitch in order to gain satisfactory stability.