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Lessons Learned in Applying the Norwegian Method of Tunnelling (NMT) in Soft Ground Conditions

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ABSTRACT

The Bergåsen road tunnel in northern Norway was originally designed as a conventional hard rock tunnel through marble and granite formations. During excavation, an unexpected 75 m long mixed-face zone consisting of over-consolidated clay moraine and highly weathered rock was encountered, requiring major modifications to the excavation and support strategy. The clay moraine exhibited shear strengths ranging from approximately 250 to 1,200 kPa under triaxial loading, while the weathered rock showed highly variable strength characteristics. Additional investigations, laboratory testing, and numerical analyses were undertaken to characterize the weak ground conditions and redesign the tunnel support system.

The revised tunnelling approach incorporated short excavation rounds, pipe umbrella support, reinforced ribs of sprayed concrete (RRS2), grouted face bolts, and continuous deformation monitoring. The paper presents the investigation program, the design of temporary and permanent support, the excavation methodology, and the construction performance under challenging soft-ground conditions. The performance of spiling bolts, pipe umbrella systems, and reinforced sprayed concrete arches is discussed in relation to construction efficiency, deformation control, and risk mitigation.

The case study demonstrates that the Norwegian Method of Tunnelling (NMT), traditionally applied in hard rock conditions, can be successfully adapted to weak ground conditions when supported by systematic investigations, observational design approaches, and robust support systems. The lessons learned from Bergåsen provide valuable guidance for future tunnelling projects involving unexpected soft-ground zones in predominantly hard rock environments.

Keywords: Soft ground tunnelling; Sprayed concrete; NMT; RRS; Tunnel support system; Construction engineering

1. INTRODUCTION

This paper is an extended and revised version of a conference paper presented at INDOROCK-2025, 5th – 7th November 2025, New Delhi. The authors gratefully acknowledge the valuable feedback received during the conference, which contributed to the development of this journal article.

Tunnelling projects in Norway are generally designed with the expectation of excavating through competent, jointed hard rock formations (NGI, 2022). However, the Bergåsen road tunnel near Mosjøen presented a rare and challenging exception. During excavation in December 2022, the contractor Leonhard Nilsen & Sønner (LNS) encountered a tunnel face dominated by soil-like material rather than solid rock at around Chainage 580 m. Figure 1 shows the location of the tunnel in northern Norway. While preliminary investigations had suggested the presence of a weakness zone of limited extent, approximately 5–10 m, the actual conditions proved far more complex. A continuous zone of roughly 75 m was revealed, consisting of clay moraine interspersed with highly weathered and disintegrated rock (Figs. 2 & 3). In Norway, the expected or predicted weakness zones in hard rock tunnel projects are pre-investigated by total sounding drilling, which measures the resistance of drilling in the geomaterial. The method is not sensitive enough to distinguish between soft rock and friction soils, especially when blocks or boulders in soil are encountered, as in this case. The behaviour of the clay moraine at the tunnel depth was assessed to be soft rock with uncertainty, and the total sounding did not get as deep as in the weathered rock material below the clay moraine. If this had been properly predicted beforehand the use of seismic could have been used before the tunnel excavation.

Figure 1 - Map of Norway, where the red pin shows the location of the tunnel project

The length of the tunnel was 2.0 km, and the hard rock section was designed to be a 9.5 m wide tunnel according to Norwegian regulations in the tunnel design handbook (NPRA, 2022).

These unexpected geological conditions required significant modifications to the excavation strategy. A supplementary investigation program was initiated in January 2023, accompanied by a comprehensive design review to reassess excavation techniques and temporary support requirements. The revised tunnelling strategy integrated soft-ground excavation principles with established Norwegian tunnelling practices, incorporating measures such as short excavation rounds (1.5–2.0 m), spiling and pipe umbrella bolts, drainage holes to reduce pore pressure, grouted face bolts for stability, and reinforced sprayed concrete layers for crack detection and worker safety. Continuous monitoring of ground movements using inclinom and the installation of reinforced sprayed concrete ribs further ensured structural integrity during advance. The inclinom showed deformation over the last excavation in the range of 5-15 mm during excavation. In case of total deformation exceeding 20 mm or 10 mm displacement per day, a discussion of possible countermeasures was carried out between the client, contractor, and consultant.

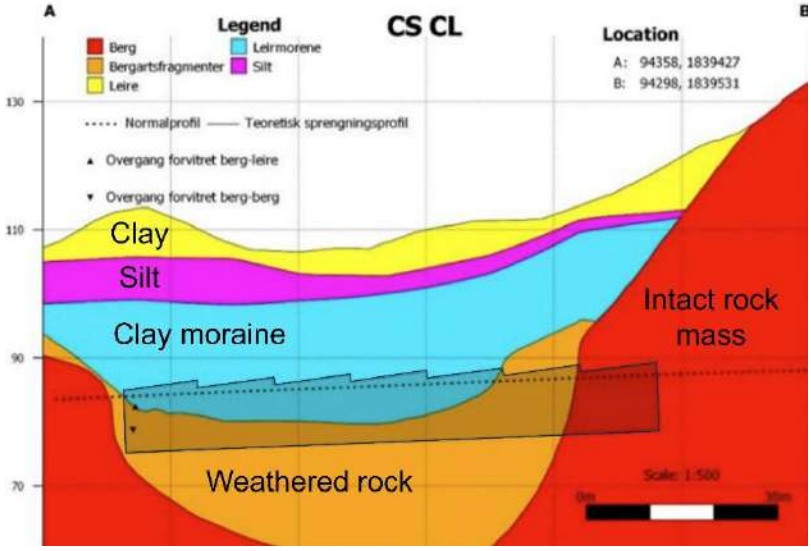

Figure 2 - Profile showing the distribution of rock mass, clay moraine and weathered rock along the 75 m long soft ground zone (Nilsen et al., 2023)

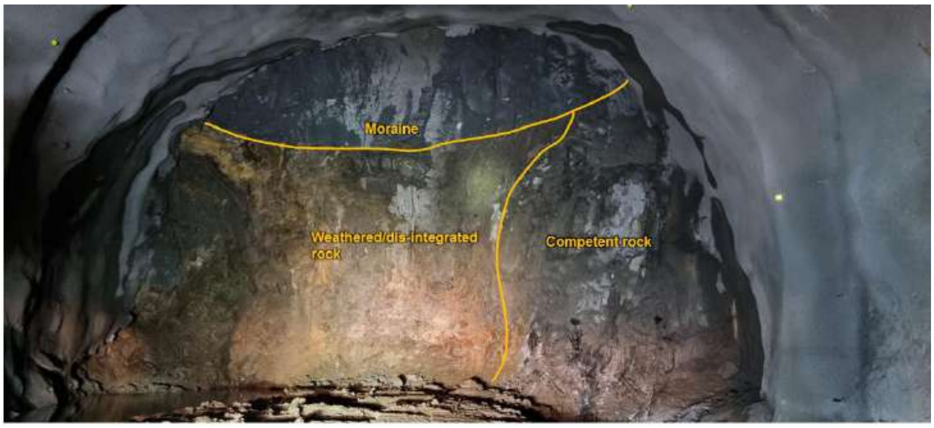

Figure 3 - Photo showing the first face in soft ground consisting of moraine, weathered rock and competent hard rock

Although Norwegian tunnelling methods are well-suited for jointed hard rock conditions, similar challenges have been documented in previous projects, notably the Joberg tunnel (2015–2016), where lessons learned proved invaluable for Bergåsen (Aagaard et al., 2017). Before finalizing the approach, alternative methodologies such as NATM (Stipek et al., 2012) and ADECO-RS (Lunardi, 2000) were evaluated. Ultimately, the Norwegian Method of Tunnelling (NMT) was selected as the most appropriate solution. For a detailed discussion of the pre-investigation program and the analytical and empirical design considerations for the soil zone (Terron-Almenarra et al., 2024). The ultimate reason for this was done due to the contractor's experience and the availability of support materials in the Norwegian market. By designing and tunnelling with the NATM method, there would be an additional time delay of several months for designing, producing and transporting lattice girders.

The Q-values ranged from less than 0.1 to more than 10 for the majority of the tunnel in hard rock, which comprised marble. In the 75 m long soft ground zone, the observation of the material was presumably out of the Q-system's case studies.

2. GROUND INVESTIGATIONS AND PLANNING OF TUNNELLING

Effective ground characterisation is a prerequisite for safe and efficient tunnelling, particularly when unexpected soil conditions are encountered. At Bergåsen, the discovery of a 75 m long zone of clay moraine and highly weathered rock required a rapid and systematic investigation program to reduce uncertainty and guide decisions.

Between mid-January and mid-February 2022, an investigation program was carried out to characterise the unexpected soil zone encountered at the tunnel face. Five core boreholes were drilled directly from the tunnel heading to delineate the extent and properties of the weak ground. Two of these were executed using conventional rock core drilling equipment, a practical choice given its availability in Norway. Although this method yielded a relatively low core recovery rate, it was selected due to its widespread availability in Norway and its ability to provide a preliminary estimate of the soil zone's extent. The bore holes, which were 68-76 m deep, were drilled from the face of the tunnel where the loose soil conditions were encountered and ended when the intact rock mass was met. Measurements while drilling (MWD) indicated the loose soil conditions, which consisted of clay and silt.

To improve sample quality and obtain more representative data, the remaining three boreholes employed a triple-tube coring system combined with drilling fluids and a core catcher setup. This approach significantly enhanced core integrity and recovery, enabling a more reliable assessment of the heterogeneous soil and weathered rock conditions.

In parallel, three vertical boreholes were drilled from the surface to investigate the stratigraphy above the tunnel alignment. These surface holes also served as monitoring stations for pore pressure measurements during excavation, supplying hydrogeological data throughout the tunnelling process. The depth was approximately 20-24 m, to obtain material from all lithologies above and at the tunnel level. These were distributed along the length of the soil zone at various locations.

The combined results from these investigations, supplemented by laboratory testing of recovered samples, formed the basis for the geotechnical model of the soil zone. Laboratory analyses included:

- Determination of natural water content,
- Grain size distribution through sieve analysis,
- Mineralogical composition of soil and rock fragments,
- Measurement of swelling pressure and free-swelling behaviour, and
- Triaxial shear tests to evaluate cohesion and internal friction angle.

The updated investigations yielded average geotechnical param for the soil materials encountered, which proved essential for refining both the tunnel design and the temporary support strategy. A key finding was that the ground ahead of the tunnel face remained relatively dry, alleviating concerns about excessive pore water pressures during excavation.

It is worth noting that during the tender phase of the Bergåsen project, seven sounding boreholes had been drilled from the surface near the anticipated soil zone. These early investigations were interpreted as indicating a poorly weathered rock mass. However, because all boreholes terminated above the actual tunnel level, the interpretation was misleading, resulting in the incorrect assumption that intact rock existed along the alignment. This discrepancy highlights a critical limitation of surface-based investigations when tunnel depth and stratigraphic variability are not adequately considered.

Following the updated investigations, the geotechnical model – supported by laboratory test results – was integrated into a Plaxis numerical simulation. This model was employed to analyse face stability during excavation, predict long-term surface settlements, and evaluate loading conditions on temporary support structures throughout construction. Numerical modelling provided a robust framework for anticipating ground behaviour and optimizing support measures under complex geological conditions.

As excavation progressed, the Hoek–Brown failure criterion (Marinos et al., 2000) was adopted to characterize the mechanical behaviour of the weathered rock encountered. Due to discrepancies between observed in-situ conditions and earlier interpretations, previously derived param for weathered rock were discarded in favour of updated values calibrated against actual excavation performance. This iterative refinement of geotechnical param underscores the dynamic nature of underground construction, where design assumptions must be continuously validated and adjusted based on real-time observations.

Ultimately, the Bergåsen tunnel case shows the importance of combining multiple investigation techniques – face drilling, triple-tube sampling, surface boreholes, laboratory testing, and numerical modelling – to develop a reliable geotechnical model. It also emphasizes the necessity of adaptive design strategies in tunnelling projects, where unexpected ground conditions can significantly influence both construction methodology and support system requirements.

2.1 Excavation and Temporary Design

The excavation and temporary support system for the Bergåsen tunnel was developed to address the challenges posed by weak ground conditions and the need for rapid progress. Figures 4 and 5 show, respectively, the design of the tunnel with pipe umbrellas and the implementation of RRS2. The cross-sectional area at the start of each pipe bolt is 87.9 $m^{2,}$ increasing to 112 m^2 at the end of one screen. The length of the arch (the length from the left pipe-umbrella to the right pipe in Figure 4) is between 22.4 and 26.6 m. The initial excavation length was 1.5 m, which was extended to 2.0 m and finally to 2.5 m after 30 m of excavation length. The reason for extending the length was to improve the efficiency by using displacement measurements as an indication of stability. The space between RRS2 was similar to the excavation length, i.e., 1.5 to 2.5 m.

The excavation and temporary support strategy was implemented through cyclic rounds, each comprising the following activities:

- Removal of previously applied sprayed concrete from the tunnel face or selected areas.
- Excavation using a scaling hammer or rotating drum, followed by surveying.
- Application of 8–10 cm of fibre-reinforced sprayed concrete (classified as E1000 according to N500, NPRA, 2022) on the face, roof, and walls, followed by surveying.
- Installation of grouted radial bolts and construction of reinforced sprayed concrete ribs.
- Placement of reinforcement within the sprayed concrete ribs and subsequent surveying.
- Spraying of the RRS2 layer and surveying.
- Commencement of the next excavation round.

The decision to adopt RRS2 instead of lattice girders was primarily driven by time constraints. Procuring lattice girders in Norway typically requires several months, which was incompatible with the project's schedule. Rapid excavation was necessary to access the northern area of the Bergåsen tunnel for bridge construction. Additionally, RRS2 was favoured because it is a well-established technique in Norwegian tunnelling practice, and crews are highly familiar and competent in its implementation. The design of the temporary support was performed with an analytical method based on Terzaghi's silo theory on load, supplemented by 2D and 3D numerical analyses in Plaxis. During construction, the design was validated by displacement measurements on the support, as well as displacement above the tunnel heading measured by inclinom.

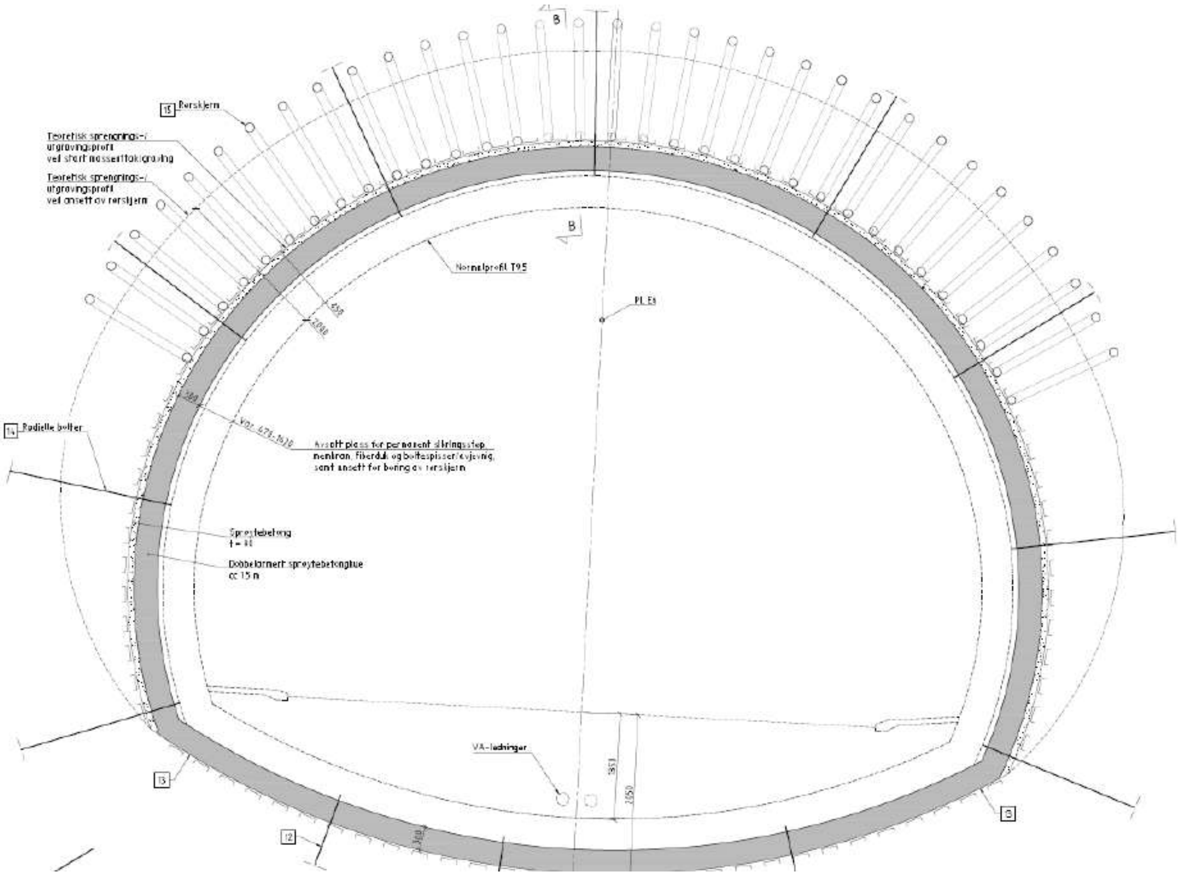

Figure 4 - Cross-section of the excavation showing the pipe umbrella, rock bolts, and cast-in invert (text written in Norwegian)

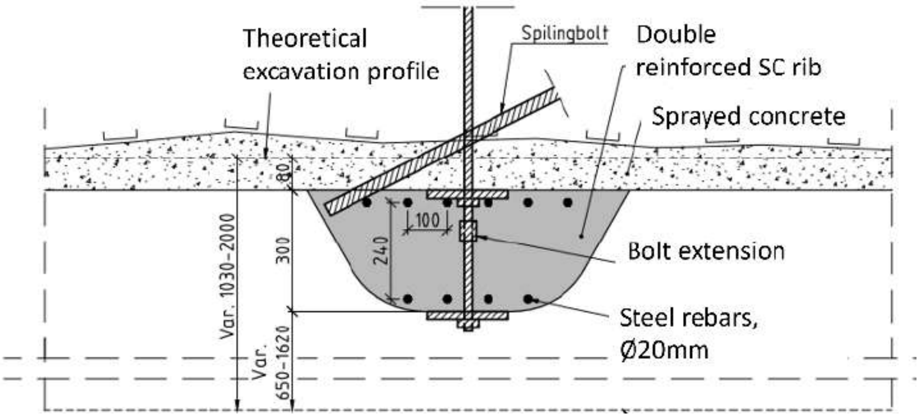

Figure 5 - Reinforced ribs of sprayed concrete (RRS2) combined with pipe umbrella support used in the weak ground section

2.2 Observations and Risk Mitigation

Continuous monitoring and risk management were essential to ensure safe excavation under highly variable ground conditions. To facilitate this, daily coordination meetings were organized by the designer (NGI) with representatives from the contractor and the tunnel owner. These meetings focused on evaluating critical aspects of tunnel stability and safety, including:

- Water leakages and potential erosion of the tunnel face,
- Instabilities at the face during excavation,
- Instabilities between the spiling bolts / pipe-umbrella bolts,
- Instabilities under the spiling bolts/pipe,
- Deformation/displacements in the RRS2,
- Visual observation of cracks in the sprayed concrete,
- Visual observations of the tunnel floor with special focus on the transition from wall to floor, and
- Instabilities behind the tunnel face.

Based on these aspects together with pore pressure measurements, daily conditions were classified using a three-level “caution scale” (0, 1, or 2). This assessment framework supported the implementation of a set of 13 predefined countermeasures, available on-site to address emerging risks. Table 1 summarises the measures available on-site, distinguishing between those applied or used during construction (1 to 7, Table 1) and those that remained unused (8 to 13, Table 1).

Table 1 - Risk mitigation measures available on-site

Measures available and used	Measures available, but not used
1. Grouting leakage zones in or ahead of the tunnel face	8. Reducing the excavation area
2. Drilling drainage holes in and around the tunnel face	9. Shortening the excavation length
3. Diverting water away from the tunnel floor	10. Increasing bolt lengths
4. Applying sprayed concrete to the tunnel floor	11. Adjusting bolt spacing

5. Placing coarse rock material to stabilize the floor	12. Installing spiling bolts in the lower tunnel walls
6. Conducting pre-investigations (probe drilling with MWD or core drilling)	13. Adding bolts between reinforced sprayed concrete ribs
7. Installing additional RRS2 ribs between existing ones	-----

As can be seen in Table 1, six risk mitigation measures were not required during construction. The daily coordination meetings enabled systematic evaluation of observations, risk conditions, and mitigation measures, discussing potential interventions, and documenting ground behaviour, ensuring that excavation proceeded safely and adaptively under challenging conditions.

The displacement between the last RRS2 and the face during construction was measured by inclinom. The displacement in this area was generally 15 mm, with some areas exceeding 25 mm. The walls and roof were measured with prisms. The displacement in these areas was in the range of around 5 mm.

2.3 Spiling Bolts and Pipe Umbrella

The initial 12 m of the soil zone were stabilized using spiling bolts with a diam of 32 mm installed ahead of the tunnel face. The spacing of the spiling bolts was 0.3 m, and the inclination toward the tunnel was 5.5°. This approach was selected due to two key constraints: the long lead time required for pipe umbrella bolt delivery and the limited tunnel cross-section, which made the installation of pipe umbrellas impractical at the onset of excavation in unforeseen soil conditions.

Once this initial section was completed, Ø114 mm top-hammer pipe umbrella bolts with an 8 mm wall thickness were introduced, as shown in Figure 4. A total of five pipe umbrella screens, every 15 m in length and comprising 34 pipe bolts, were installed. To ensure continuity and stability, each screen overlapped the previous by 3-4 m.

Installation precision improved significantly over time. Screens 2 through 5 exhibited deviations of only a few centims across the full 12 m span, whereas the first screen showed deviations on the decim scale. These inaccuracies required the cutting and removal of five out of the 34 pipe bolts during excavation.

The installation of the first pipe umbrella screen required 115 h, while screens 2 through 5 averaged 60 h each. This reduction in time reflects the contractor's learning curve and improved operational efficiency throughout the process.

3. TIME CONSUMPTION

Excavation progress varied significantly between hard rock and soft ground conditions. The initial 600 m of hard rock tunnelling, preceding the unforeseen soil zone, were completed in 14 weeks. This phase included the full drill-and-blast cycle and installation of permanent rock support in accordance with Norwegian regulations (NPRA,2022).

In contrast, the 75-m section of soft ground required 13 weeks to excavate, despite its considerably shorter length. The extended duration reflects the complexity of working in weak ground, which demanded short excavation rounds, intensive support installation, and continuous monitoring. Following this zone, the remaining 1,150 m of hard rock were excavated in 24 weeks. The Q-values on each side of the soft ground zone were from approximately 1 to 10, and the main rock support classes were B and C, which included systematic bolting and 8 cm sprayed concrete.

Figure 6 illustrates the time consumption for installing spiling bolts during the first excavation round and for the five successive pipe umbrella screens. Installation times decreased markedly after the first screen, highlighting the contractor's learning curve and improved efficiency. Actual excavation advance per round ranged from 1.5 to 2 m, but cumulative progress varied between 3 and 15 m depending on ground excavability and operational constraints.

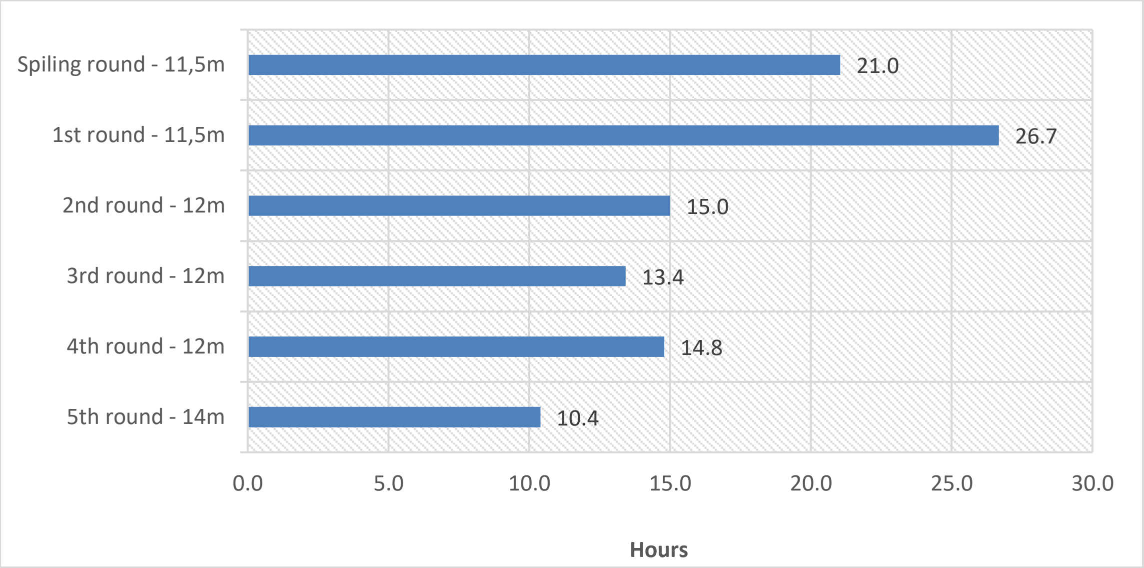

Figure 6 - Hours used for installation of one spiling round and 5 rounds of pipe-umbrellas (Jakobsen et al., 2024)

Details of the excavation activities and their corresponding net average time consumption per round are summarised in Table 2. Excavation averaged 7 h per round but varied significantly (3–15 h) depending on ground excavability. Support installation activities, particularly bolting and face bolt installation, accounted for the largest share of time, averaging 10 h combined. Sprayed concrete application and rebar installation each required approximately 3 h per round. Minor tasks such as drainage drilling and bolt adjustments were less time-intensive but occurred intermittently. The breakdown underlines that support installation, rather than excavation, was the primary driver of cycle duration in the soft ground zone. The variability in bolting and excavation times reflects the influence of ground conditions and opportunities for efficiency improvements through installation techniques and planning.

Table 2 - Time taken in various activities during the excavation and initial support (Jakobsen et al., 2024)

Activity	Time (h)	Remarks
Excavation	7	Varied from 3 – 15 h dependent on excavability
Initial spraying of concrete	3	Varied from 2-4 h
Bolting	5	Varied from 2 – 10 h dependent on bolt-hole stability

Installation of rebars in RRS2	3	Varied from 2.5 – 10 h
Spraying of RRS2	3	Varied from 2-4 h
Installation of face bolts	5	Only installed every 8 to 12 m of the tunnel. Approximately 0.5-1.0 h per round
Adjustment of face bolts	0.5	Cutting and installation of new bolt plate after each exc. round
Drilling of drainage holes	0.5	Only occurred 7 times during the project
Unforeseen issues		Cutting of pipe bolts

4. PERMANENT SUPPORTS

During the design phase for excavation and temporary support (Fig. 7) at the Bergåsen tunnel, additional clearance was intentionally reserved around the tunnel contour to allow for the installation of a cast-in-place concrete lining if required. According to the Norwegian Road Tunnel Design Code (N500), guidelines for permanent support are based on rock mass quality expressed through Q-values (Barton et al., 2014; NGI, 2022), typically greater than 0.01. However, the ground conditions encountered at Bergåsen – comprising clay moraine and highly weathered rock with soil-like behaviour – fell outside the scope of these provisions.

Figure 7 - Photo of the temporary support of Bergåsen tunnel, before the permanent RRS support

In such exceptional cases, the design code stipulates that "excavation and permanent support shall be dimensioned as a special case." At Bergåsen, engineering geologists and tunnel engineers were responsible for the excavation and temporary support design, while structural engineers developed the permanent support system.

From a practical perspective, the installed RRS2 (double-reinforced sprayed concrete arches) performed well during excavation, showing no signs of degradation or displacement (Fig. 7). However, the sprayed concrete cover over the steel reinforcement was insufficient to guarantee long-term corrosion protection. While applying additional sprayed concrete to the temporary arches might have been a simple solution, this approach was considered risky in terms of long-term durability. Instead, a more robust strategy was adopted: installing permanent load-bearing RRS2 arches between the existing temporary ones (Fig. 9).

Figure 8 - Photo of a part of the temporary RRS2 that needed to be scaled down to fit the permanent support

Although technically feasible and of high quality, this solution presented several challenges:

- **Complex reinforcement interface**: The junction between the cast-in-place concrete invert and the permanent RRS2 arches contained excessive reinforcement, complicating sprayed concrete application. This section was ultimately supported with cast-in-place concrete (Fig. 7).

- **Space constraints**: In some areas, temporary RRS2 arches had to be scaled back by 1–15 cm to create sufficient clearance for permanent arches (Fig. 8). From a time, cost, and environmental perspective (CO_2 emissions), the removal and reapplication of sprayed concrete is debatable.

- **Extended construction time**: The permanent support phase required approximately eight months-twice the duration of excavation and temporary support. However, this activity was not on the critical path of the overall highway project schedule.

Figures 8 illustrates the adjustments made to temporary arches, and Figure 9 shows the sequential construction process for the permanent tunnel lining, which combined reinforcement and sprayed concrete in multiple stages:

- Installation of primary reinforcement: The first layer of reinforcement bars (rebars) was positioned along the excavation surface and securely tied in place.
- Application of initial sprayed concrete: A preliminary layer of sprayed concrete was applied over the first row of rebars, forming the initial structural lining.
- Placement of secondary reinforcement: An additional row of rebars was installed to enhance structural integrity and load-bearing capacity.
- Final sprayed concrete application: The permanent lining was completed by applying the final layer of sprayed concrete over the second row of rebars and the temporary arches.

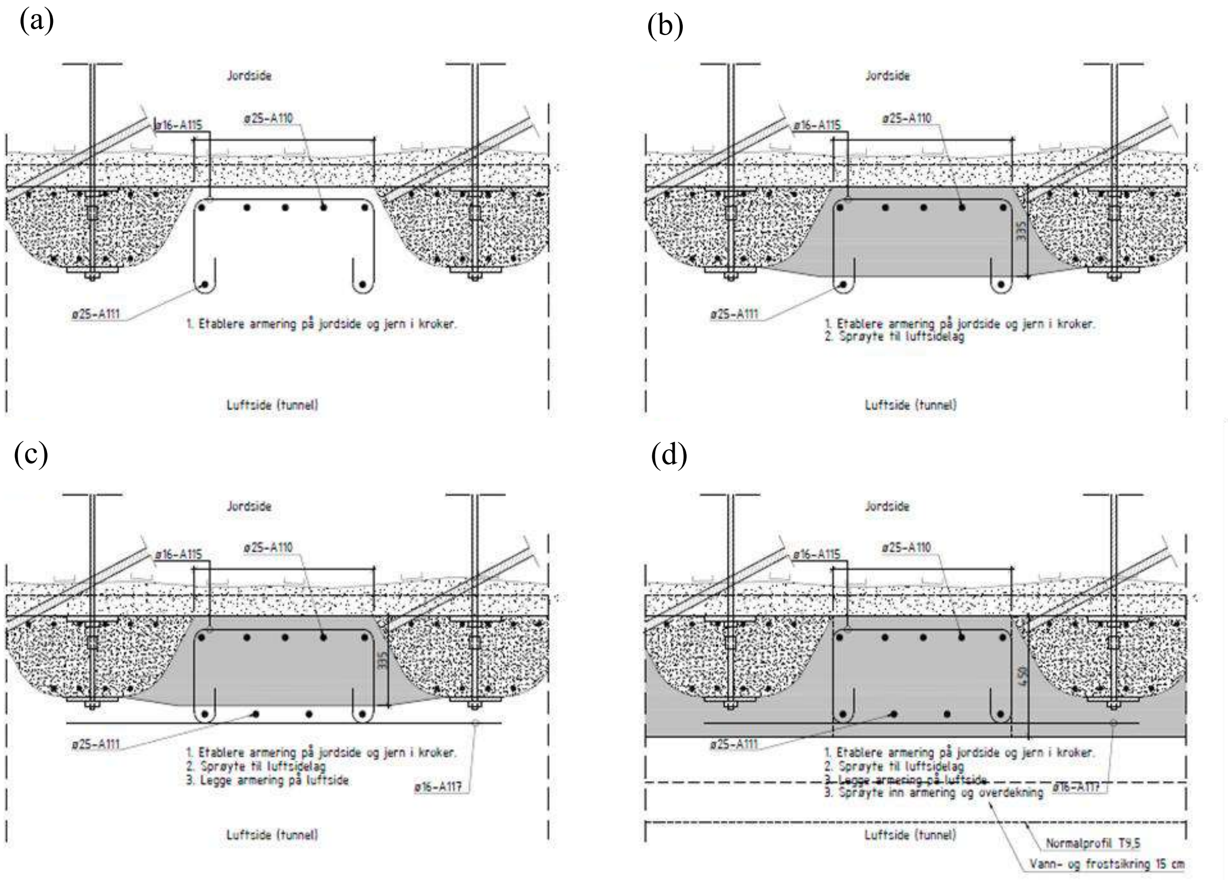

Figure 9 - Sequential construction of permanent tunnel arches: (a) initial reinforcement installation, (b) first sprayed concrete layer, (c) additional reinforcement placement, and (d) final sprayed concrete application over reinforcement and temporary arches

This staged approach ensured adequate bonding between layers, improved durability, and provided long-term corrosion protection for the reinforcement system.

The support approach implemented at Bergåsen, consisting of reinforced sprayed concrete arches installed between temporary RRS2 supports, shows a strategy for ensuring long-term durability in tunnels excavated through weak ground. This approach provided enhanced structural integrity, improved corrosion protection for reinforcement, and minimised the risk associated with simply thickening temporary sprayed concrete layers. Although the method required additional time and posed challenges related to space constraints and reinforcement congestion, it offers a practical alternative to cast-in-place linings in these types of geological conditions. Thus, the experience shows the importance of designing adaptable support systems during the early phases of tunnelling projects and provides valuable guidance for future cases where conventional design codes do not apply.

5. KEY FEATURES OF THE NMT METHOD

As shown in the Bergåsen case, one of the defining characteristics of the Norwegian Method of Tunnelling (NMT) is the use of Reinforced Ribs of Shotcrete (RRS) as a primary support element. In general, the selection of tunnel and cavern support in Norway is guided by rock mass classification using the Q-system. This system assists in feasibility studies, site characterization during mapping, and systematic design throughout excavation (Barton & Grimstad, 2014). Effective application of the Q-system requires engineering geologists with appropriate training and experience.

Based on Q-system principles and Norwegian tunnelling practice, a single-shell support concept is advocated, in contrast to the more expensive double-shell NATM-style (New Austrian Tunnelling Method) approach. Unlike NATM, NMT does not distinguish between temporary and permanent support; the installed support is intended to serve as part of the permanent lining. This philosophy emphasises careful selection and quality of support components, including systematic bolting, fibre-reinforced shotcrete, and, where necessary, reinforced ribs of shotcrete (RRS) for poor to exceptionally poor rock masses, as illustrated in the Bergåsen tunnel.

The use of steel sets, common in other tunnelling methods, has largely been replaced by RRS in Norwegian practice. Empirical design based on the Q-system is often supplemented by displacement measurements, instrumentation data, and numerical modelling to verify load-bearing capacity and optimise support performance. NGI has instrumented several RRS sections and conducted numerical analyses to calibrate design assumptions (Bhasin et al., 1999; Bhasin & Shabanimashcool, 2024).

With more than 6,000 km of tunnels constructed in Norway, contractors have accumulated extensive experience in varying geological conditions, from very good to exceptionally poor rock masses. The Bergåsen project contributes valuable empirical data for extending the applicability of NMT to longer sections of soft ground, reinforcing its adaptability and robustness in challenging conditions.

6. CONCLUDING REMARKS

The Bergåsen tunnel project shows that the Norwegian Method of Tunnelling (NMT) can be successfully adapted to weak ground and mixed-face conditions when combined with systematic investigations, numerical modelling, observational construction practices, and robust temporary support systems.

The unexpected 75 m long zone of clay moraine and highly weathered rock required significant modifications to the original hard rock tunnelling strategy. Additional site investigations, laboratory testing, numerical analyses, and continuous monitoring were essential for reducing uncertainty and ensuring safe excavation.

The combined use of pipe umbrella support, double reinforced ribs of shotcrete (RRS2), short excavation rounds, and deformation monitoring proved effective in controlling instability and maintaining excavation safety. Construction efficiency improved progressively as installation accuracy and operational experience increased during the project.

One of the key lessons learned is that permanent support systems should ideally be integrated with excavation and temporary support activities at an early stage. Such an approach could reduce rework, sprayed concrete consumption, and overall construction time. The project also highlights the importance of flexible design approaches in situations where conventional tunnel design codes are not directly applicable.

The Bergåsen case provides valuable guidance for future tunnelling projects encountering unexpected soft-ground conditions within predominantly hard rock environments and contributes to extending the applicability of NMT principles beyond traditional hard rock tunnelling.

Future research should focus on strengthening the applicability of Norwegian tunnelling principles in complex geological conditions and supporting the evolution of best practices for soft ground tunnelling.

DISCLOSURES

The authors declare that they have no conflicts of interest related to this work.

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