1 Introduction

The Norwegian tunnelling industry has built tunnels and underground facilities for more than 100 years. A rugged geography and sometimes harsh climate have demanded solutions to tough problems to improve daily life. Our recipe consists of thorough pre-investigations, full use of hard-won acquired experience and cutting-edge technologies and procedures. This has resulted in some 'first, longest, largest, deepest' references, at least sometimes. However, always with a focus on safe solutions and high cost efficiency.

The "Norwegian Way of Working" (NWW) plays an important role in efficient and successful tunnelling. Well-proven technology, short and clear lines of communication to reach decisions and experts available on site. This allows even the most complex of situations to be handled efficiently and effectively. What stands out as unique compared with many other countries, is the very important decision-making direct involvement of the shift crew at the tunnel face. These guys are highly paid, they have bonus systems for efficiency and based on their experience it turns out that if allowed, they are often more inventive than most engineers will give them credit for. This simple fact can have a tremendous effect on keeping construction time short and thereby lowering cost.

The main benefit of NWW is fast and safe tunnel excavation at affordable cost; or put in other words; time and cost-efficient tunnelling while maintaining excellent work safety, high final quality and without compromising required operational standard and design lifetime.

Another element is adaptation to actual ground conditions using a well-established system for dealing with encountered rock mass in order to install best-suited rock reinforcement. In most cases and whenever possible, support will be based on state-of-the-art technologies for sprayed concrete and rock bolts, both for immediate and permanent support. The advantages of this solution are hard to catch with many traditional design methods for rock support. However, application of the Observational Method (OM) as authorized in Eurocode 7, offers substantial advantages for reduction of construction time and cost, while even leading to simpler design development without reduction of safety and durability. Pre-Excavation Grouting (PEG) is the main method used for control and restriction of ground water ingress as required by the project, which also works very well with the above support solution.

The overall approach and its execution do focus on quality, cooperation, experience and innovation. These are key words in describing tunnelling and working underground in 2018. A lot has changed since the pioneers and entrepreneurs of the 60'ies, 70'ies and so on when muscles counted more than computer aided design, health and safety was less of a topic and hand-held pneumatic drill units were used. The most important elements of NWW have developed over time and the current state of this approach is outlined below.

2 Pre-investigations

2.1 Introduction

For any type of underground project, pre-construction investigations of high quality, well adapted to the geological conditions and the project characteristics are crucial. If the investigations are insufficient or inadequate, unexpected and in worst case uncontrollable ground conditions may be encountered, and poor quality and high cost will often be the result for the completed project.

Pre-construction investigation, often simply called pre-investigation, is therefore very important for evaluating the feasibility of the project and for planning and design. Among many other good reasons to focus on pre-investigation, the following effects are particularly important:

- Gives basis for analysing stability and estimating rock support requirement.
- Provides input for evaluating alternative tunnelling methods and selecting equipment/tools for excavation and rock support.
- Provides a basis for predicting performance and capacities.
- Provides a basis for estimating time schedule and cost.
- Is important for assessing potential environmental impacts.
- Gives a basis for preparing tender documents.

If the pre-investigations are insufficient or of poor quality, reports and tender documents will not reflect a correct picture of the actual geological conditions. Conflict between contractor and owner due to "unforeseen geological conditions" will very easily be the result and in worst case the project may end up in court with more time lost and extra cost. Proper investigation is therefore very important for all aspects of the project.

The rock mass as material is in many ways complex and quite different from other construction materials such as steel and concrete. The rock mass is inhomogeneous and mostly anisotropic, it contains complex structures such as folding and faults, and other factors such as rock stresses and groundwater are also strongly influencing the conditions. In addition, the planned project is located underground, while the pre-investigations mainly must be carried out from the surface. This means that interpretation is required for estimating the conditions at the level of the planned underground project. Estimation of rock mass conditions based at the pre-construction stage is therefore often a difficult task, and experience is very important for a good result.

The engineering geological factors that need to be investigated for a planned underground project are mainly:

- Soil cover, particularly for portal areas and sections of potentially insufficient rock cover.
- Bedrock, with emphasis on rock type boundaries.
- Fracturing of the various rock types.
- Faults/weakness zones.
- Groundwater conditions.
- Rock stress conditions.
- Mechanical properties of rocks and potential gouge materials.

2.2 Investigation stages

Normally, the investigations are carried out in a stepwise procedure and linked with the progress of engineering design. The general ground investigation procedure for tunnels and underground excavations in Norway is illustrated in Table 1.

Table 1. Main steps used for ground investigations for tunnels and underground openings.

Pre-construction			During construction	During operation
Project conception	Feasibility study	Detail investigation		
- Basic knowledge of ground conditions	-Desk study of maps, aerial photos, reports -Field investigation of key points -Visit to nearby excavations	-Eng.geol. mapping -Geophysical investi-gations -Drilling -Sampling -Lab. testing	-Tunnel mapping Probe drilling -Monitoring (rock stress, convergence etc. -Sampling -Lab. testing	-Monitoring (extensometer etc.) -Quality control
=> Recognition major challenges	=> Preliminary design	=> "Final design"	=> Modification of design	=> Maintenance

A complete discussion of the subject of pre-investigations can be found in Publication No. 26, "The Principles of Norwegian Tunnelling" by Norwegian Tunnelling Society, 2017.

2.3 Some general remarks

Pre-investigation tools are available for practically any kind of ground condition and any kind of site characterization. However, it is important to realize that even when great effort has been made in the pre-investigation stage, some uncertainty will still remain regarding the ground conditions. Pre-construction site investigations therefore always must be followed up by continuous engineering geological investigation during tunnelling.

In many cases in Norwegian tunnelling practice some of the detailed design is postponed so that results from construction stage investigations can also be included in the final evaluations (i.e. rock stress measurements, particularly for hydropower projects).

It is important to realize that the ground conditions may vary within wide ranges, and there is therefore no "standard investigation procedure" that will fit all types of conditions and all types of underground projects. The investigations for tunnels and underground excavations must be planned according to the characteristics of each individual project, and should always be adjusted to:

1) The difficulty and complexity of the geological conditions.
2) The complexity and special requirements of the project.

The investigation should always be carried out in stages, with a willingness to modify design and execution for an optimum result of the final project. The flexible solutions for immediate and permanent support as well as the ability to very quickly take overreaching decisions when needed is a key aspect of NWW. Within the established total range and distribution of ground conditions, the detailed adaptations and tailoring of support to actual conditions will typically save substantial money and time for the project. It may even reduce the necessary need for detailed pre-investigations and can prevent general use of over-conservative support solutions that in reality are only necessary to cover worst case locations.

3 Rock mass classification

3.1 Introduction

There are many different systems for rock mass classification and the preferences regarding which system to use will vary with geographic regions and personal views on what system that is most suitable for a given purpose.

For natural reasons the Q-system that was developed by NGI between 1971 and 1974 (Barton et al. 1974) is the one mostly used in Norway. It is also a widely accepted system in world-wide tunnelling and mining.

The Q-system has been revised and re-published several times, like the update based on 1050 project examples in 1993 and another 900 examples added in 2002. Some of the revisions also reflect developments in the use of fibre reinforced sprayed concrete and the increasing use of reinforced ribs of sprayed concrete (RRS).

The Q-system is applicable for two different primary purposes:

1. Classification of rock mass quality in relation to stability of underground construction, used either as part of surface site investigations and geological mapping, or as part of mapping of ground conditions during excavation. Note that in the last case, the Q-value will depend on the rock cover of the underground opening and may therefore be different to the Q-value recorded on surface (for the same type of rock).

2. Selection of rock support for an underground opening based on combination of the Q-classification of the local rock quality and the rock support diagram of the Q-system. This gives recommended support derived from the recorded support-example database of previous successfully executed solutions for similar rock conditions. The recommendations cover both immediate-, temporary- and permanent support.

The Q-values generated based on geological mapping and rock classification from within underground excavations will give the most precise expression of rock quality. When the Q-system is used for on surface site mapping, classification of core samples or recordings inside boreholes, it becomes more difficult to establish accurately some of the parameters used to calculate the Q-value.

Below can be found excerpts from the NGI handbook “Rock mass classification and support design” from 2015 for a first impression and for reader convenience. However, for full access note that complete documentation is accessible in PDF-format from www.ngi.no and also on this website. It is strongly recommended to download the complete document for any practical use of the Q-system. There is also an app available for use on a smartphone or tablet device.

3.2 Stability of rock masses

During underground excavation it is very important to carry out close visual observation of the rock surface in the whole tunnel periphery before the rock is covered by sprayed concrete. In addition to the visual observation, hammering with a scaling rod or a hammer will give important indications of any unstable wedges through the generated sound. Also, small cracks, invisible from the invert, will be detected with such a close look. Altered rock may show the same geological structures as the original fresh and un-weathered rock and may not be noticed if only observed from a distance. In order to have a close observation it is of outmost importance to have access to the face and crown by use of lifting equipment especially designed for this purpose. Rock mass stability is influenced by several parameters, but the following three factors are the most important:

- Degree of jointing (block size)
- Joint friction and alterations
- Stress situation

The degree of jointing, or block size, is determined by the joint patterns, i.e., joint orientation and joint spacing. At a certain location in the rock mass, there will, in most cases, be a joint pattern which could be well or not so well defined. Often joint directions exist systematically in rock masses, and most of the joints will be parallel with one of these directions. Near parallel joints form joint sets and the joint spacing within each set will usually show characteristic distributions. The joint spacing may be reduced considerably along some zones in the surrounding rock. Such zones are called fracture zones. Stability will generally decrease when joint spacing decreases and the number of joint sets increases. In soft rocks where deformation can occur independently of joints, the degree of jointing has less importance than it has in hard rocks.

In hard rocks, deformations will occur as shear displacements along joints. The friction along the joints will therefore be significant for the rock mass stability. Joint friction is dependent on joint roughness and thickness and type of any mineral fillings. Very rough joints, joints with no filling or joints with only a thin, hard mineral filling will be favourable for stability. On the other hand, smooth surface and/or a thick filling of a soft material will result in low friction and poor stability. In soft rocks where deformation is less dependent of joints, the joint friction factor is less significant.

The vertical stress in a rock mass commonly depends on the depth below the surface. However, tectonic stresses and anisotropic stresses due to topography can be more influential in some areas. Stability of the underground excavation will generally depend on the stress magnitude in relation to the rock strength. Moderate stresses are usually favourable for stability. Low stresses are often unfavourable for the stability. In rock masses intersected by zones of weak mineral fillings such as clay or crushed rock, the stress situation may vary considerably within short distance. Experience from tunnel projects in Norway has shown that if the magnitude of the major principal stress approaches about 1/5 of the compressive strength of the rock, spalling (strain bursting) may occur. When tangential stresses exceed the magnitude of the rock compressive strength, squeezing may occur. In other words; the anisotropy of the rock mass plays an important role when designing rock support.

3.3 The Q-System

The Q-value provides a numerical expression of general rock mass quality with the aim of being useful for evaluation of underground excavation and the stability of excavated openings. High Q-values indicate good stability and low values mean poor stability. Based on 6 parameters the Q-value is calculated using the following equation:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

The six parameters included are:

1. RQD = Degree of jointing (Rock Quality Designation)
2. Jn = Joint set number
3. Jr = Joint roughness number
4. Ja = Joint alteration number
5. Jw = Joint water reduction factor
6. SRF = Stress Reduction Factor

The individual parameters are determined during geological mapping using tables that specify numerical values to be assigned to a described situation. Paired, the six parameters express the three main factors which influence the stability in underground openings:

RQD/Jn = Degree of jointing (or block size)
Jr/Ja = Joint friction (inter-block shear strength)
Jw/SRF = Active stress and influence of ground water

3.4 Rock support design based on the Q-System

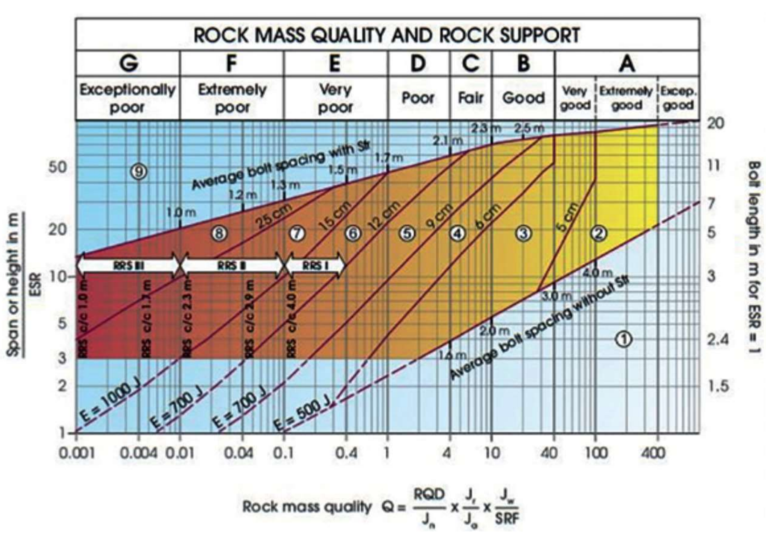
Q-value and the six appurtenant parameter values give a description of the rock mass. Based on a large number of documented case histories a relation between the Q-value and the permanent support is deducted, and this can be used as a guide for the design of support in new underground projects.

In addition to the rock mass quality (the Q-value) two other factors are decisive for the support design in underground openings and caverns. These factors are the safety requirements and the dimensions, i.e., the span or height of the underground opening. Generally there will be an increasing need for support with increasing span and increasing wall height. Safety requirements will depend on the use (purpose) of the excavation. A road tunnel or an underground power house will need a higher level of safety than a water tunnel or a temporary excavation in a mine. To express safety requirements, a factor called ESR (Excavation Support Ratio) is used.

The recommended ESR-values can be found in the table below:

7	Type of excavation	ESR
A	Temporary mine openings, etc.	ca. 3-5
B	Vertical shafts*: i) circular sections ii) rectangular/square section * Dependant of purpose. May be lower than given values.	ca. 2.5 ca. 2.0
C	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks), water supply tunnels, pilot tunnels, drifts and headings for large openings.	1.6
D	Minor road and railway tunnels, surge chambers, access tunnels, sewage tunnels, etc.	1.3
E	Power houses, storage rooms, water treatment plants, major road and railway tunnels, civil defence chambers, portals, intersections, etc.	1.0
F	Underground nuclear power stations, railways stations, sports and public facilitates, factories, etc.	0.8
G	Very important caverns and underground openings with a long lifetime, ≈ 100 years, or without access for maintenance.	0.5

When the Q- and the ESR-values are known, the empirical rock support recommendation chart can be used to establish a support solution. The chart is given below for the reader's convenience without further explanation, but the whole referred documentation needs to be reviewed before use in an actual case.



3.5 Some general comments on rock support

As mentioned in the introduction about rock mass classification, the Q-system is one of many different ways of performing classification of rock conditions. However, the basic purpose of it all is to communicate in as specific as possible terms the quality of the rock in a tunnel or a cavern in hard rock. Bottom line is that immediate and permanent support must be installed and decisions about what to install must be taken on short notice.

Typically, all projects of underground excavation will describe rock quality classes and corresponding rock support classes that have been pre-designed for use at the project. The rock quality class, established e.g. by the Q-system, can be used to decide on the rock support as shown above, but there are also other ways, like analytic calculations and numeric analysis. In really complicated and high-risk cases, all useful tools will normally be employed to reach a conclusion.

Still, there will often be an element of uncertainty, depending on conservatism employed and often project specific political decisions of various types. The support recommendations presented by the Q-system may be taken as the final decision, but in Norwegian practice, it is quite normal to take it as one of many possible “recommendations” and to verify the final support choice installed by observation of performance. This is what is termed the Observational Method (OM), which is covered in more detail below. The OM is part of Eurocode 7 and it is required in Norway to employ this Code to underground construction support design. The details of Eurocode 7 usage are currently under revision, but verification of sufficiency of support by observation (mostly meaning some level of instrumented monitoring) is central if OM has been selected as the design method under Eurocode 7.

4 The Observational Method (OM)

4.1 Introduction

The previous chapters have covered in some detail the preparations that are necessary before start of construction of a tunnel or cavern and it is all done to provide basis for, among other things, the selection of rock support design principles and the construction and support methods. The rock mass classification may start already at the pre-investigation stage on surface and can be used also for systematic recording of rock condition data as excavation takes place. Furthermore, it is important to decide if the rock surrounding the excavation will be part of the load bearing structure, or if the rock is simply seen as a source of load onto an installed support.

The normally applied principle in Norwegian tunnelling is based on rock support being adapted to the local rock conditions, both regarding immediate support and for the permanent support. Also, the surrounding rock is taken into consideration as part of the overall structure that creates stability for the underground opening. Contrary to the approach in many other regions of the world, even the temporary support is required to satisfy quality and durability as specified for the permanent support. This way, all installed support can be integrated into the permanent support. This approach offers the advantage of saved time and materials and the use of one single over-conservative permanent support solution for the whole tunnel can be avoided. A single support solution throughout would have to cover the worst condition encountered along the tunnel.

For civil construction tunnels, many different methods are used to decide on rock support solutions for given projects. Design typically must cover both the immediate or temporary support case as well as the permanent support, and the latter may involve a time horizon of 100 years. Ground conditions for tunnelling range from shallow tunnels through soil to deep seated tunnels through solid and massive hard rock. Some tunnels get lined with one single support solution that must cover all conditions for its full length, while the lining in most Norwegian tunnels gets adapted to the local ground conditions as they are encountered. When combining all the variables involved, like excavation method, immediate and permanent support, worst case single support solution or adapted solution and including many different design methods, the number of possible combinations becomes very large.

To a varying degree, all tunnel support design methods end up with an unknown factor of safety, as demonstrated by the fact that there are sometimes failures and collapses. Also, there are cases where the installed lining gets no load whatsoever. The ground conditions and stress situation along a tunnel alignment is typically so variable that it becomes impossible to accurately determine all the parametric input needed in support calculations. Also, empirical methods will suffer from mapping and classification mistakes and subjectivity. Numerical methods are not any different and cannot possibly be allowed to hold up the face progress while input parameters are measured and calculations executed for results to be used for initial support decision and execution.

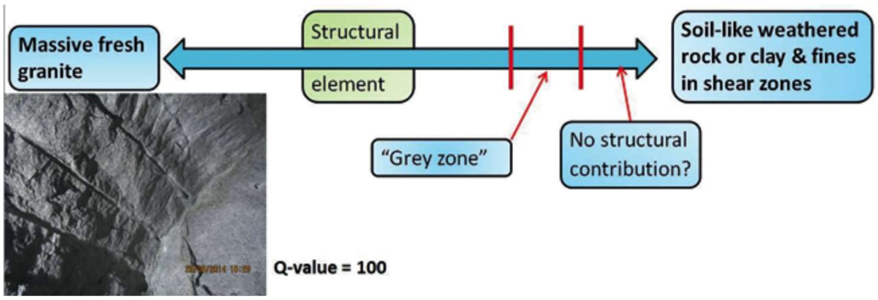
In principle, all decisions about rock support solutions in tunnels carry uncertainty and anything pre-planned, but not yet excavated and installed, can only be considered a support *prognosis*. This will be the case irrespective of design methods and tools used to reach decision about what to install for different ground conditions. The rational approach is to face this reality and apply the Observational Method (OM) for verification of sufficiency of whatever has been installed for rock support. Rock 'support' in Norway is in most cases in reality rock reinforcement, e.g. installed inside the rock
as rock bolts, or as surface reinforcement (e.g. sprayed concrete mostly with fibres). Actual (functionally) rock *support* is normally either an in-situ cast concrete lining or backfilled concrete segments in a TBM tunnel. In any case, the tunnel stability depends on an interaction between the surrounding rock and the installed reinforcement and support and we will basically never know all the relevant parameters and mechanisms that play a role in this composite action. To circumvent this problem and use reality as a full-scale test laboratory, installed instrumented monitoring sections or other means of observation, can prove (or disprove) whether the tunnel is stable or not. If the real-life observations are not satisfactory, this will allow mitigation by installation of added support. Without monitoring, unsatisfactory performance could go unnoticed and will sometimes develop into a collapse.

It should be noted that the practical details of an OM approach must be adapted to the case at hand. As mentioned, the use of analysis tools for design of support solutions for expected ground quality classes will depend on the range of ground conditions identified, but also how complicated and possibly risky the project may be. It should be obvious that a 15 m^2 tunnel in good granite will be much simpler to design than a cavern system with large dimensions in poor rock and may be under high stresses. Clearly, the same considerations apply when deciding what methods of observation will be necessary for verification of performance of installed support. For the mentioned small tunnel in good granite, observation can be limited to performing visual inspection, while the cavern system would typically require quite an elaborate system of monitoring devices. It may be claimed that there is in principle no distinction between visual observation and instrumented monitoring in this respect. In both cases the implemented steps are just different ways of executing 'observation' to satisfy the need for verification, while adapted to the requirements of the case at hand.

4.2 Some basic considerations

Even tunnels in generally good rock qualities will typically have to cross shear zones (faults) of sometimes extremely low quality. The situation can be illustrated as in the figure below. In the situation to the left, no measures are needed since the rock is globally stable at almost any free span of underground excavation. All the way to the right, heavy support structures will be necessary and swelling clay content may add significantly to the loads that must be supported.

The dominating paradigm at the base of rock support design in many countries is approaches needed for cases from the right side of the Figure below. At this end of the ground quality scale, a heavy support solution may be warranted, but it should not be extended too far to the left. In most of hard rock cases, even with the normal jointing patterns and quality "flaws" of rock, the rock can be utilized as a structural element when designing and creating stability. This can be done by focusing on and installing rock reinforcement rather than rock support. Rock reinforcement will typically be some selection of different rock bolts or cable anchors installed in boreholes combined with surface reinforcement by fibre reinforced sprayed concrete. Even polymer-based spray-on surface reinforcement is today available.



That satisfactory stability in the majority of rock qualities can be established using rock reinforcement only, as outlined above, without heavy concrete lining with or without structural reinforcement, cannot be disputed. This approach is routinely being used for all kinds of temporary- or initial support in drill and blast tunnelling. Even if defined as temporary, it often takes months and sometimes years before the final lining, or permanent lining gets installed and lack of stability is very seldom an issue. Frequently, the heavy cast concrete lining is not required other than for reasons of durability, ground water control or aesthetic requirements. An increasing volume of tunnel and cavern excavation is furthermore successfully lined by rock reinforcement methods with strict requirements on quality and durability for both initial and final support. The end-result is often just 200 mm average thickness of reinforced sprayed concrete combined with permanent quality rock bolts working as permanent lining, thus replacing bolts and sprayed concrete defined as temporary, followed by additionally 5X as much in-situ concrete lining, sometimes with double reinforcement.

One important reason for this conservatism in design of permanent rock support is that analysing the stability case of sprayed concrete and rock bolts in a drill and blast tunnel is extremely complicated and can hardly be done accurately. For some designers there will be a feeling of lack of confidence about the reliability of calculated design. Rock bolts and a relatively thin skin of sprayed concrete on the undefined and variable geometry of the blasted rock surface is just part of the problem. Another important element is the frequent variation in rock quality along the tunnel, as well as variable rock stresses and ground water conditions.

4.3 The resulting, unavoidable practical approach

For the purpose of designing the initial support, all available design methods have serious and well-known practical limitations. Analytic and numeric calculation methods will suffer from:

- Inaccurate and missing input values.
- The validity of the geological model may often be questioned.

- Approximations, simplifications and assumptions are used to be able to execute the calculations and the validity of mathematical models will because of this suffer significantly.
- Tunnel advance is anyway far too fast to allow any per blast-round analysis for support selection identified by calculation results.

The empirical methods offer a simpler and faster approach, which therefore is a quite practical alternative, but:

- They are no better or worse than the cases included in the recorded data-base that is the foundation of suggested support solutions.
- Geological mapping mistakes and classification subjectivity are normal deviations that are hard to completely avoid.

For practically all drill and blast tunnels and other open face excavation methods, selecting initial rock support therefore ends up being managed the same way:

1. First, identify the total range of identified rock conditions along the tunnel alignment.
2. Sub-divide this total range into Ground Condition Classes (typically anywhere from 4 to 10).
3. Pre-design a “support” solution for each Ground Class.

To decide on practical support solution for each individual Ground Class, any and all available design methods may be used based on preference and necessity and this work will naturally be much influenced by the complexity presented by the findings under above item 1.

Once tunnelling has started, the process continues by:

4. Mapping of the rock conditions in the tunnel, typically on a per blasting-round basis to identify which Ground Class that applies for determination of the “support” solution to install as pre-designed according to above item 3.

There are good reasons to ask why this approach may not be used also to design and decide on the permanent part of the support solution. After all, the temporary support typically works very well, even for extended periods of time, but formalities of verification and estimation of factor of safety often presents a problem in this regard.

In short, it can be summed up as a problem of overall structural analysis of the interaction between the rock material and the installed reinforcement under the normal geometric conditions found in a drill and blast tunnel, especially if used for the permanent solution. Verification of sufficiency can and often will be challenged, often leading to over-conservative solutions.

On the other hand, the standard case of a full concrete lining sometimes with double reinforcement will provide a known load carrying capacity resulting from standard reinforced concrete analysis methods. As a side remark, this normal final lining approach of showing load-carrying capacity, is often disregarding that the *actual* load (if any) is still not really known. The only real difference to the temporary rock reinforcement approach is the typically resulting very conservative load-carrying capacity of the installed support structure.

4.4 Combination of immediate- and later installed complementary support

When using the OM as the permanent lining design method, primarily based on sprayed concrete application and rock bolts, what ends up being the final and permanent lining may be constructed in more than one step. Even if the materials and processes involved placing the immediate support do satisfy the quality and durability requirements of the permanent lining, the fact that part of the final solution has been installed in more than one step may cause objections that the final product is not one unit.

It may certainly happen that the immediate support gets subjected to high rock stress and some deformations before stability is reached, regardless if reached on its own or after additional measures have been placed. In extreme cases, it may be claimed that the immediate support has been damaged by elongation of bolts and probably some cracking of the initial sprayed concrete layer and that the immediate support therefore must be disregarded when considering the permanent lining. However, modern combination bolts are not suffering durability issues as long as they are not snapping from overload and deformation. Fibre reinforced sprayed concrete used under such severe conditions should implement synthetic structural fibres and cracking will not cause fibre corrosion and loss of reinforcing effect.

Another concern may be that later layers of sprayed concrete may prevent a monolithic structure when looking at the overall sprayed concrete thickness. Provided proper surface preparation when applying later placements of sprayed concrete, the interlayer bond strength will be 1.0 MPa or more and actually the same as when building large thickness in several passes during the same shift. This is the normal way of building the required final thickness of any sprayed concrete structure and it is not known to have caused any problems of practical nature or from testing of core samples from executed projects.

When using the OM, the very strong main advantage of this methodology is exactly that verification by Observation (mostly monitoring) will be made on the interaction of the support structure and the rock conditions as they actually are, so concerns like the ones described above are taken care of. Still, if conditions are really extreme in terms of large deformations ongoing for extended period of time, special considerations can always lead to adaptations that are normally not necessary. The integration of immediate into permanent support is still a recommended working principle for reduced time of construction and lowered overall cost, without sacrificing quality and durability.

4.5 Outline of Observational Method (OM), practical steps

1. Design rock support solutions for expected rock conditions (Ground Classes).
    - Use all necessary methods (analytical, numerical and empirical).
    - The case complexity dictates which methods to use.
    - The designed support solutions must be considered support *prognosis* at this stage and should not be taken as a design end-result.
2. Observe the support performance, or rather the performance of the surrounding rock and installed reinforcement interaction over time, while excavation is being continued.
    - The observation has the purpose of verifying the support prognosis.
    - Verification means that the observations get checked against the support prognosis. The design must contain estimated radial deformation against time with acceptance limits and levels of warning limits. Also, other parameters may additionally be checked, also against specific criteria of acceptance or alarm.
    - Observation methods may range from simple visual checking in excellent hard rock conditions and small tunnels, to very complex instrumented monitoring and convergence readings in large caverns and complex and poor rock cases.
    - If the acceptance values from design (support prognosis) are not verified, then additional support measures need to be installed and the above steps must be repeated until verification has been recorded.
3. In case of local unsatisfactory performance and need for additional support, especially if a recurring phenomenon, then the available information must be fed back to the relevant part of the design to upgrade and adjust it to avoid further non-conformance under the same conditions.
4. Selection of Final Permanent lining Option must be decided.

- If all elements of the installed reinforcement have satisfactory durability to be defined as permanent and the installed solution has been verified stable by observations, then it is acceptable as the final lining solution. However, it is of course possible to add another step after this, installing an additional and extra support for increase of the factor of safety. In most cases it will of course not be necessary to go to the extreme standard approach of adding an in-situ concrete lining. Some additional bolt pattern and possibly another layer of sprayed concrete would normally be enough.

By use of the OM adapted for the case at hand, many advantages can be listed:

- All factors influencing stability are covered, whether known or not known, since the “mountain” is being used as a full scale ‘laboratory’.
- Extensive rock sampling and parameter testing can be minimized.
- No scale-effect errors since the actual case is being observed and monitored.
- No errors from approximations and assumptions. Changes over time, like the effects of ground water flow and rock stresses are covered.
- The installed rock reinforcement and support gets adapted to the actual rock conditions, no more and no less.
- No expensive worst-case support installed along the whole tunnel.

The Observational Method is an accepted approach according to Eurocode 7 and offers final lining solutions adapted to ground conditions and therefore typically at lower overall cost. It may also be claimed that the advantages of OM will be enhanced in case of very complicated objects of underground construction. The more complicated (poor ground, system of caverns and other openings, sensitive neighbours etc.), the more unreliable will design methods depending on any kind of calculations be. In comparison, results from a proper OM-approach cannot to the same extent be questioned due to its built-in self-adjusting properties.

Norwegian Tunnelling Network

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