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Engineering geology and rock engineering applied in the design of Norwegian tunnels

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SUMMARY

Most Norwegian tunnels have been excavated by the drill and blast method generally at an average tunnelling progress of 40 - 80 m/week. Capacities of more than 100 m/week have been experienced working 100 h/week. The cost for drill & blast and mucking out of the blasted material in a 50 m^2 tunnel amounts to 1,000 to 1,500 USD per metre. A key to the high capacities and low costs has been the continuous exchange of experience and the close co-operation between the engineering geologists, the designers and the contractors.
Prediction of the ground conditions along the tunnel is mainly based on geological mapping, geophysical investigations and engineering geological survey. The cost for these investigations is 0.25 - 7 % of the construction cost
The rock supporting works in Norwegian tunnels are designed for the actual rock mass conditions encountered during construction. They consist mainly of fibre reinforced shotcrete and rock bolts.
An example is shown how systematic application of engineering geological data and construction aspects can be combined to assess the amount of rock support and tunnelling progress for a tunnel.

1. INTRODUCTION

Norway is a mountainous country. Therefore, tunnel solutions are frequently used for various purposes. The annual length of tunnels constructed in Norway is around 130 km. We do not know exactly how many kilometres of tunnel have been constructed over the years. A rough estimate indicates 3000 km, mining excluded.

The many hundreds of kilometres of tunnel driven in connection with hydropower development, underground storage and transportation have resulted in an accumulation of experience in tunnelling that is equalled in few other countries in the world. An important feature for this success has been the willingness of the tunnelling society to accept new methods that could result in safer techniques and in cost savings. Thus, over the past 40 years Norwegian contractors have become among the most skilled in hard rock tunnelling.

Most people involved in tunnelling outside Scandinavia believe that the rock conditions for tunnelling in Norway are good. This is true in many occasions as the rocks found are 'hard rocks', i.e. old igneous and metamorphic rocks with compressive strength > 50 MPa. These old rocks have, however, suffered several periods of earth movements resulting in numerous faults, shear zones and thrust zones [1]. Such features often cause great challenges and problems for the tunnel excavation in Norway.

Figure 1 A simplified geological map of Norway. Most of the rocks are older than 250 mill. years. Rocks originally formed as sediments and also many of the plutonic rocks have been metamorphosed to the hard rocks we find today. The most important geological features for underground excavations are joints and faults, which form a complex pattern of discontinuities in the ground.

Another important feature of the Scandinavian ground is the glacier erosion, which took place during the Ice Age. This process 'cleaned' the rock surface by removing the weathered rocks, leaving a fresh surface where the in situ state of the ground can easily be observed today. Where the rocks are exposed at the surface, it is possible to predict the underground conditions from simple surface observations [1].

Because of this, the rock mass conditions can often be evaluated only from geological surface observations as is further dealt with in the next chapter.

2. PRE-INVESTIGATIONS FOR TUNNELS

The amount of investigations performed for a tunnel project is adjusted to the geology (how complicated it is), the availability to observe exposed rock at the terrain surface, and the amount of possible problems expected. The investigations are generally carried out at two main stages:

- the pre-investigations made as *field investigations* before tunnelling starts, and
- the *exploratory drilling* done during tunnel excavation.

2.1 Field investigations

Most of Norway has been geologically mapped in scale 1:50 000 or 1:100 000. Based on these maps and studies of aerial photos in scale 1:15 000 to 1:30 000 the main trends in the ground are found as is shown in Figure 2.

This information is used in an engineering geological survey which concentrates on the properties of the rocks and the occurrence and properties of the joints. The ground conditions evaluated from all available information.

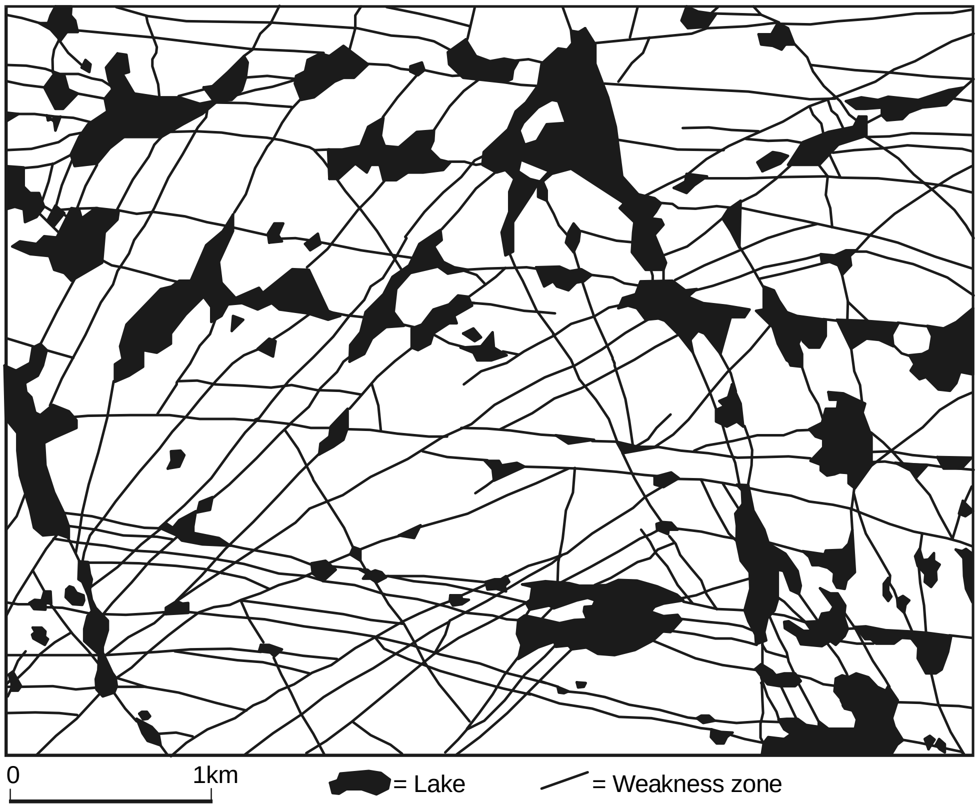


Figure 2 It is often possible to study the geological conditions from plain surface observations, since well exposed and unweathered (fresh) rocks occur in large parts of the Norwegian land surface. The pattern of weakness zones shown on the figure is found from aerial photographs. With the geology revealed like this it is generally a simple job to find the best location for the tunnel.

For tunnels, where the majority of the rock are covered, for example subsea tunnels for which the rock surface is hidden by water, or where complicated geology occurs, additional field investigations are performed to collect information on the rock mass conditions. This may consist of:

> *Refraction seismic* measurements are mainly carried out to obtain more information on the ground conditions along the tunnel alignment. In addition to a more accurate location of the rock surface and thickness of possible loose deposits, these measurements give qualitative information of the rock mass conditions, see Figure 3. Especially important is the possibility from interpretation of the measurements to locate weaker zones or faults below rock covered by loose deposits. Together with other information, i.e. geology and the stratigraphy can fairly well be indicated together with assumed ground conditions. [2]

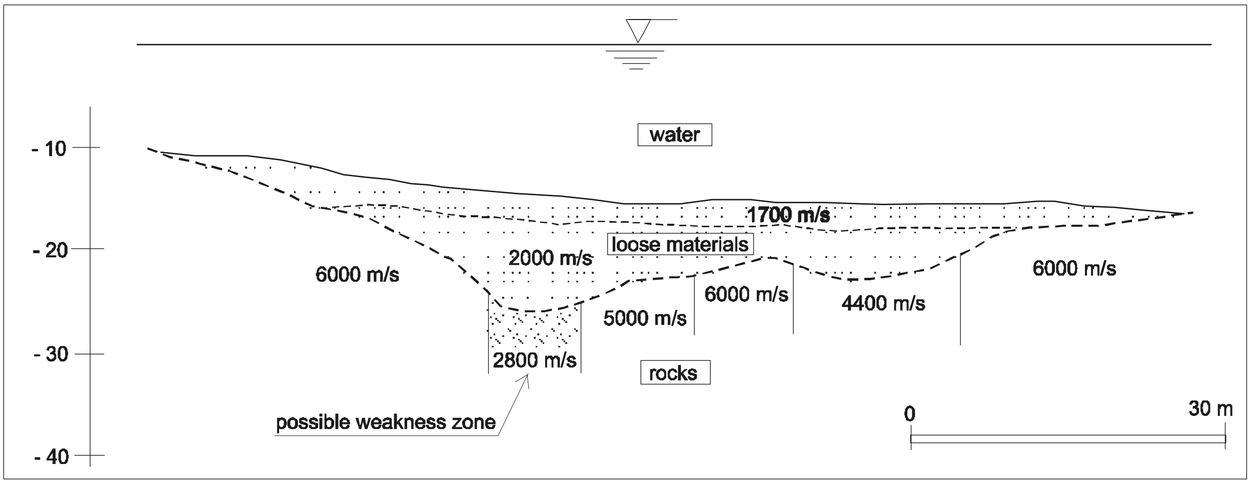


Figure 3 Refraction seismic measurement is the most frequently used field investigation method in addition to engineering geological surface mapping. This geophysical method reveals both the location of the rock surface where it is covered by soils, and the various sonic velocities in the rock masses. Low velocities 2500 - 3500 m/s for example, indicate weakness zones or weathered rocks where stability problems probably will occur. (The example is from measurements on the sea bottom for a subsea tunnel)

Core drillings are mainly performed to collect information of the rock mass conditions and the composition of possible weakness zones. In addition to core recovery it is also usual to perform water pressure tests to obtain information on ground water and possible leakage problems.

Based on the results from the field investigations the expected ground conditions are evaluated and the major weakness zones localised. This is applied in the selection of the final tunnel alignment [2]. Further, the expected types, methods and amounts of rock support and the system for exploratory investigations are worked out. The example in Section 6.1 shows how the rock support and tunnel progress can be assessed. This information is applied in the tender documents.

2.2 Exploratory drillings from tunnel face during tunnel excavation

A part of the investigation for a tunnel can be carried out as exploratory or probe drilling during the tunnel excavation. This is mainly performed for subsea tunnels where there are greater uncertainties connected to the geological prognosis. The aim of this work is to detect any water leakage zones or adverse ground conditions before they are encountered in the tunnel. In this way it is possible to carry out measures ahead of the tunnel face and thus avoid or reduce possible tunnelling problems.

One of these measures is pre-grouting ahead of the tunnel face. Where probeholes, or some of the blast holes, reveal water bearing rocks in front of the face, the pre-grouting is performed as described in Section 3.1.1.

Experience has shown that the effect of water sealing is significantly improved when high grouting pressure (50 - 100 bars) is applied. Thus pre-grouting is also applied in tunnels where limited permanent water leakage is required.

2.3 Costs for pre-investigations

The experience over the last 15 years is that costs for pre-investigations for Norwegian tunnels are generally small, mostly between 0.1 % and 3 % of the construction costs. For subsea tunnels it is generally higher, up to 7 %; in addition to 1.5 - 2 % for probe drillings.

The main reason for this low figure is, as mentioned earlier, the possibility to assess the geology from simple suface observations. Another important factor in reducing investigation costs, is the use of experience gained from nearby tunnels combined with the general confidence to underground construction among the tunnelling people.

3. TUNNEL EXCAVATION

Norwegian tunnels are excavated by the drill and blast method and by tunnel boring machines (TBM). Successful driving of a tunnel depends heavily on personal craftsmanship and the flexible design adopted to the local ground conditions encountered.

3.1 Excavation by drill and blast

Foreign visitors to Norwegian tunnel construction sites are often surprised to realise that only three men are doing the drilling, blasting and mucking out, as well as the rock support works. Most commonly there are two shifts per day working 5½ days per week. The shifts can be 7.5 hours or 10 hours.

The principles for the drill and blast method have not changed very much during the last decades, but improvements in regard to equipment and explosives have resulted in increased capacities. The excavation operations are generally performed in the following order, see Figure 4:

1. The plug or the cut is blasted in order to clear the way for the following dislodgement.
2. The size of the cavity is increased by successive detonations of the explosives in the boreholes around.
3. The holes in the contour are fired last, with reduced charges to obtain smoother tunnel walls.

Special care is made to improve the tunnel contour to minimise the need for rock support and overbreak in the blasted tunnel. Requirement as to the accuracy in positioning of the holes is usually ± 10 cm with a direction deviation better than 5%. To meet this the drilling rig is often equipped with automatic positioning- and directioning instruments so that it automatically follows a preset drilling pattern.

The blasting rounds are usually drilled by 4.8 m or 5.4 m (16 - 18 feet) long drill rods resulting in a round length of 4.4 or 5.0 m. The pull (i.e. the tunnel advance) is mainly between 85% and 95% of the round length. The diameter of the holes is 43 mm; in the cut the 3 - 5 unloaded holes are reamed to 75 - 100 mm. 90 - 100 charge holes are made for blasting of a two-lane road tunnel with 50 m^2 theoretical cross section, see Figure 4.

Explosives used are usually Anfo, which is a mixture of ammonium nitrate and diesel oil. In addition to its low price - close to one fourth of ordinary dynamite - Anfo has great safety benefits regarding storage. Production of the required amount of Anfo is carried out on each shift.

The time required for the various operations included in the drill and blast excavation using 4.8 m long drilling rods (4.4 m round length) in a 2-lane road tunnel is generally:

Scaling of loose blocks and fragments	½ hour
Drilling (60 m^2 cross section)	2 ½ hours
Installation of ventilation duct	¼ hour
Mucking-out of blasted material	2 hours
Total time per blasted round of 4.4 m	5 ¼ hours

This means that 1.4 blast rounds are made during a shift of 7.5 hours. Working 11 shifts per week, a tunnelling progress of 61 m is achieved with pull of 90%. However, the time required to perform rock support will reduce this figure. For example, tunnelling in poor rock mass conditions, where continuous heavy rock support by rock bolts, shotcrete and concrete lining are necessary, may reduce the tunnelling progress to less than 20 m/week.

Included rock supporting works the average tunnelling progress today in medium quality rock masses is between 40 m and 60 m per week. Thus, from 1.6 to 2.5 km tunnel can be excavated annually.

In good rock conditions where only a small amount of rock support is required, more than 100 m per week has been recorded. Working 100 hours per week, this gives a progress of more than 10 m or 2 rounds (of 5 m) for each shift of 10 hours.

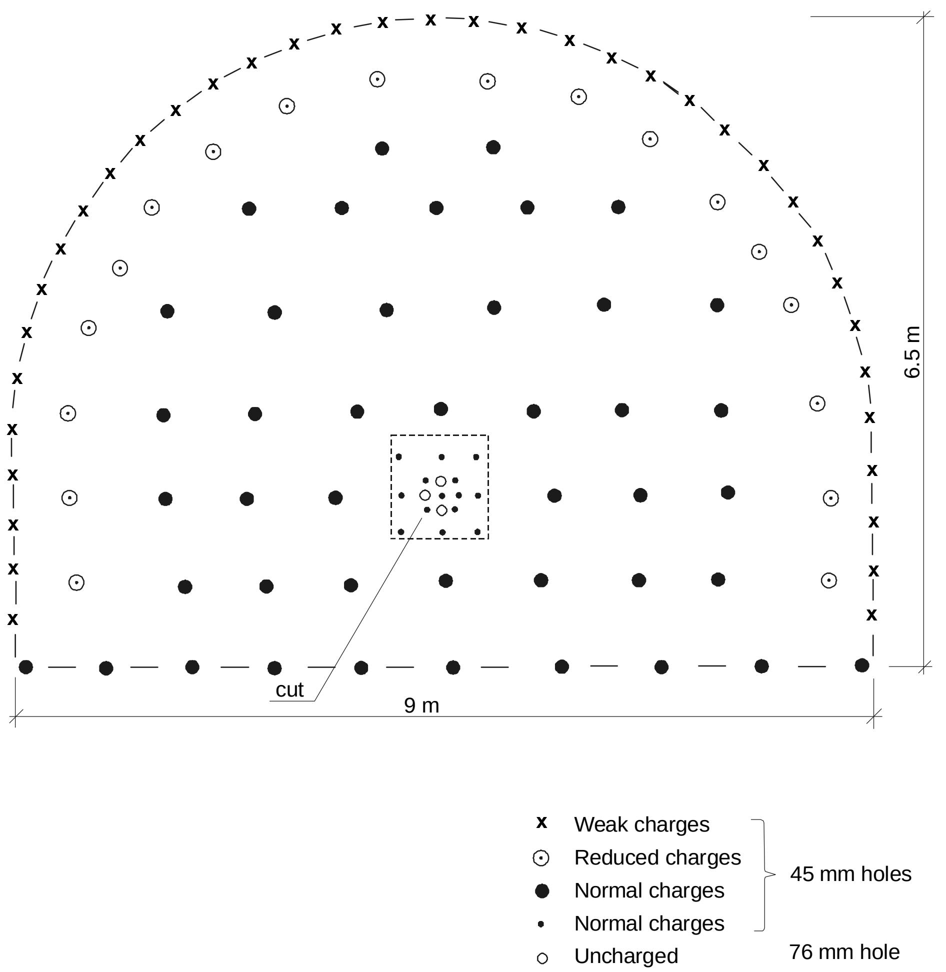


Figure 4 Principle of a drilling pattern for a 50 m^2 tunnel.

The drill & blast method has several advantages of which should be mentioned:

* Almost any type and cross sectional shapes can be made.
* It can be applied to nearly any type of rock.
* It gives great flexibility in the performance of the excavation.
* The rock support can be installed easily and quickly.

Some of the disadvantages are:

* Production of unpleasant gases and smoke from the explosives. which often leads to poor working conditions for the crew.
* Vibrations on nearby structures from the blasting.
* Rough surface gives head loss for water tunnels.
* The blasting creates new cracks in the rocks, which leads to increased rock support.

3.1.1 Probe drilling and pre-grouting as part of the drill and blast excavation

As mentioned in Section 2.2 it is often important that possible water inflows are discovered by probe drilling far enough ahead of the face so that these can be sealed by pre-grouting [2]. The practical implementation of this drilling is that the ordinary excavation works are stopped and the jumbo (drilling rig) is used to perform the necessary amount of probe drillings.

The principles of probe drilling and pre-grouting is shown in Figure 5. In addition to its water sealing effect the grouting also often improves the stability of poor rock masses. To shorten the curing time of the grout so that for the next blasting can start earlier, the grouting is often completed by use of a special fast hardening cement (Cemsil) at a cut-off-pressure of 60 bars. By this, the time for curing is reduced from about 24 hours to 2 hours.

3.2 Tunnel excavation by TBM (tunnel boring machine).

This method has mainly been developed after World War II, first for the weaker types of rocks having good stability. Later the method has been greatly improved to be used also in harder rocks. In addition to the strength of the rock the degree of jointing and other weakness planes are important factors for the possible tunnelling progress.

The tunnel diameter has been between 3 m and 10 m. The capacity will vary with the type of rock and the presence of weakness planes. In Norwegian hard rock more than 400 m tunnel has been bored per week.

The large capacity obtained by this method makes it possible to construct very long tunnels without adits. In rock suitable for TBM the excavation cost compete with those for drill & blast with the same cross-sectional area. In hard rocks with less favourable conditions TBM is still more expensive than drill & blast. TBM requires, however, less rock support, which tends to reduce the cost of this method.

A method to evaluate the TBM tunnelling progress and the wear of the cutters has been developed at the Technical University of Norway. It is based on the so-called drilling rate index (DRI) and bit wear index (BWI) of the rock, in addition the joints intersecting the rock mass and the specifications of the TBM are applied [3].

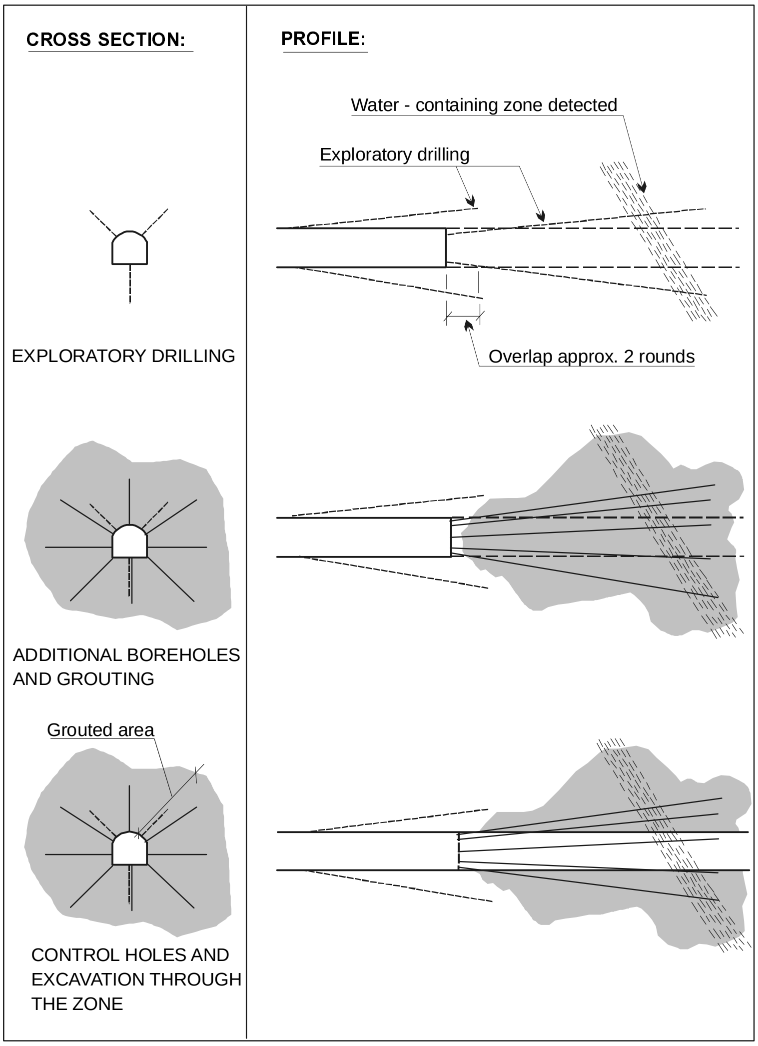


Figure 5 Principles for probe drilling and pregrouting (from [2]).
TOP: As a water-containing zone has been detected, it is decided to perform pregrouting.
MIDDLE: Several long holes for grouting are drilled and cement mixture is injected.
BOTTOM: The tunnel excavation is continued after curing of the injected grout.

Advantages using TBM excavation are:

- It requires less rock support.
- It gives smoother tunnel walls and reduced head loss in water tunnels.
- Longer tunnel sections can be excavated between adits.
- It has higher tunnelling capacity.
- It gives better working conditions for the crew.

The disadvantages of TBM are:

- More (better) geological information from the pre-investigation stage is required.
- It is more sensitive to tunnelling problems in poor rock mass conditions.
- It is a less flexible method than drill & blast method.
- Only longer tunnel sections can be bored more economically (because of larger investment and rigging costs) than drill and blast.
- The TBM may get stuck under squeezing rock conditions.
- It is difficult to perform / install rock support at the tunnel face.

4. ROCK SUPPORT

Rock support is carried out to improve the stability in an underground opening. The main principle in Norway is to design the rock support for the actual ground conditions encountered in the tunnel. This requires flexible support methods, which can be quickly adjusted to meet the continuously changing quality of the rock masses. Such flexibility is achieved by the use of rock bolts, shotcrete and cast-in-place concrete lining, either alone or as an integral element of the support.

The rock supporting works are carried out in two main stages:

1. *The initial support*. It is installed to secure safe working conditions for the tunnelling crew. The main types of support to be used are decided in the agreement between the owner and the contractor. The contractor is responsible for the initial support. In practice the working crew decides the amount of rock support necessary for their own safety.

2. *The permanent support*. This is carried out to meet the requirements for a satisfactory function of the tunnel during its life. The owner determines the final rock support. Normally he and his consultant decide after excavation both the methods and amounts of the support.

The types of support used for initial and permanent support are generally selected so that they may be combined. In this way the initial support can be included as a part of the permanent support. The latter is therefore installed only where it is necessary to strengthen the initial support.

The total cost of rock support in hundreds of kilometres of water tunnels built in Norway has varied from less than 10% to more than 200% of the cost for drilling, blasting and mucking out. In traffic tunnels and underground public halls the rock supporting cost is often higher due to stricter safety requirements.

The active use of engineering geologists in tunnelling is important, not only to solve rock stability problems during construction and to recommend rock support, but also to predict the probable rock conditions to be encountered.

4.1 Scaling

Scaling of loose rock is carried out after each blast round, and in many cases also periodically at later stages. Manual scaling (with a crowbar) is still the most common method. Mechanised scaling makes this work safer and more efficient, especially at bigger cross sections.

Scaling is carried out directly from the muck pile, in larger tunnels also from the wheel loader.
For small and medium scale tunnels, the average scaling time is normally 15 - 30 minutes.
Thorough scaling is important for the safety of the crew during the drill and blast process.

Scaling is also regarded as part of the rock support as blocks can be scaled down instead of being supported by rock bolts or shotcrete. This is the case when ’extra scaling’ (more than for example 1.5 hours scaling) is required in highly jointed and crushed rock and where rock bursting occurs.

4.2 Rock bolts

Rock bolting is the most common method for rock support. Approximately 500,000 bolts are installed annually in Norway. Rock bolting is a flexible method. It is frequently used as initial support at the tunnel face to obtain safe working conditions for the crew and it also forms part of the final support. As a measure for capacity, 50 - 100 bolts can be installed per shift (7.5 hours) by a crew of 3 men.

Figure 6 The two main applications of rock bolting. LEFT: Local rock bolts are installed individually to stabilise single, loose blocks. RIGHT: Systematic rock bolts are installed in a certain pattern as a more general support in unstable areas. (In the figure instability is caused by joints shown as lines.)

4.3 Shotcrete

This type of rock support is obtained by spraying concrete on the rock surfaces. Shotcrete has been used in Norway since 1950. The method is being used increasingly because of its good properties together with high capacity and flexibility. In Norway the so-called "wet" method - i.e. with water added before the concrete is being pumped through the nozzle - is being used.

Today three different shotcrete methods are in use:

1. Ordinary shotcrete sprayed in layers up to about 10 cm thickness.
2. Net reinforced shotcrete. This is produced by first spraying a layer of concrete before installing the net. Then a second and sometimes more layers are applied to cover the net entirely.
3. Fibrecrete. This is a type of shotcrete where thin needles or fibres, 3 - 5 cm long, of steel or other materials are mixed into the (wet) concrete. In Norway today this method is the mostly used [4]. It has almost completely replaced the net reinforced shotcrete.

The main advantages of shotcrete are:

* Short time is required to mobilise the equipment, ready for use in the excavation.
* No formwork is needed.
* It is independent of the shape of the excavation.
* It has high placing capacity.
* It can be combined with other supporting methods.

The disadvantages are:

* Collapses of shotcrete have occurred where it has been applied on swelling rocks.
* Shotcrete may have limited effect in clay containing rocks or joints with lack of adhesion.

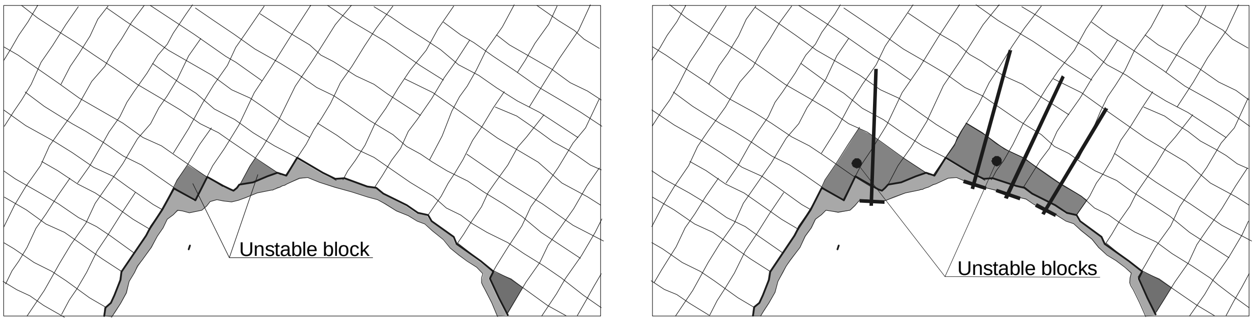


Figure 7 LEFT: Rock support by shotcrete or fibrecrete applied for reinforcement of unstable fragments and small blocks. RIGHT: The combination of shotcrete/fibrecrete and rock bolts offers attractive solutions for many rock mass conditions. This method has in many occasions replaced the in situ concrete lining, because it is quicker to install and can more easily be adapted to the actual rock mass conditions.

Spraying capacities of 30 - 40 m^3 shotcrete per shift (7.5 hours) have often been achieved in underground excavations. The Norwegian contractors have been in the forefront during the development of shotcrete and fibrecrete.

4.4 Concrete lining

Since this method can take large loads due to its arching effect, the cast-in-place concrete lining is preferably applied where large areas or volumes of poor rock mass conditions occur. It is however, the most costly and time-consuming rock supporting method.

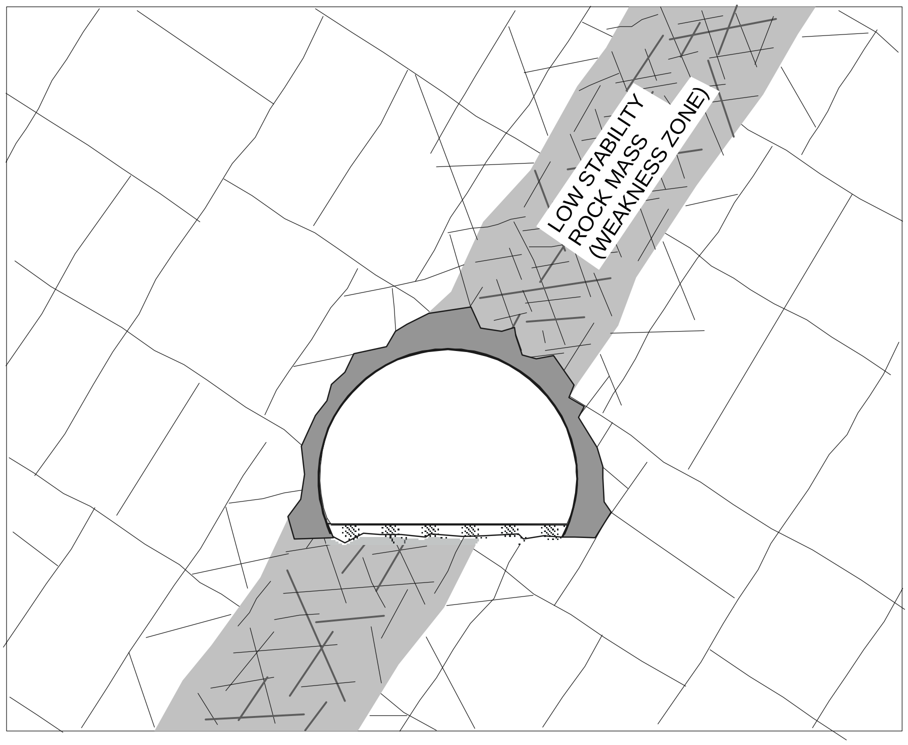


Figure 8 Rock support by cast-in-place concrete lining.

Large faults or weakness zones with highly unstable rock masses are often supported using cast-in-place concrete lining. This is performed using a full lining shield designed to fit the actual cross section of the tunnel, see Figure 8. Normally it takes only 2 - 4 hours from decision has been taken to use this type of rock support till the concrete casting can start.

No incidents of collapse of concrete lining have been reported from Norwegian tunnels where a minimum thickness of 0.3 m is used. The efficiency in installing concrete lining has increased considerably during the last 15 years after improved prefabricated steel formwork had been introduced. Capacities of 1 - 2 m lining per shift during excavation and 50% more after break-through of the tunnel are normal.

In adverse rock mass conditions showing short stand-up times it is normal to first apply shotcrete shortly after blasting and then, before installing the concrete lining. This intial support is mostly also sufficient as permanent support.

6. EVALUATION OF STABILITY AND ROCK SUPPORT

Stability in this connection is connected to the potential for rock falls, rock slides, or slow inward movement (squeezing) of the surrounding rocks. The stability depends on the rock mass quality and the location, size and geometry of the excavation. Stability in Norwegian tunnels is mainly influenced by:

* A combination of unfavourable joints (unstable blocks),
* Faults with crushed zones with or without clay, showing especially low strength and stability. Possible swelling properties of clay materials in these zones may result in additional reduction of the stand-up time and thus tend to increase the tunnelling problems.
* Rock bursting in massive, brittle rocks subjected to high rock stresses.

Though yielding of the tunnel surface has been recorded in some clay zones, instability from squeezing is not a problem encountered in Norwegian tunnels.

The rock support evaluations are partly based on experience of the people involved and from tunnels constructed in the same region. This has been quantified in the Q-system - a numerical support system developed in Norway [5]. The main input of the ground conditions into the Q system is:

- The degree of jointing expressed as the rock quality designation (RQD) and the number of joint sets (Jr)
- The joint characteristics given as the joint wall roughness number (Jr) and the joint alteration number (Ja)
- The stress conditions or the occurrence of weakness zones is represented in the stress relief factor (SRF), and
- The ground water conditions given as the joint water reduction factor (Jw)

As the underground conditions are never known until they are revealed during excavation, the decision of the amount and methods for rock support to be applied is not taken before they can be observed in the tunnel. The Norwegian philosophy is that rock support is only installed where necessary, i.e. the method(s) and amount are determined when the rock conditions can be studied on the actual tunnel after excavation.

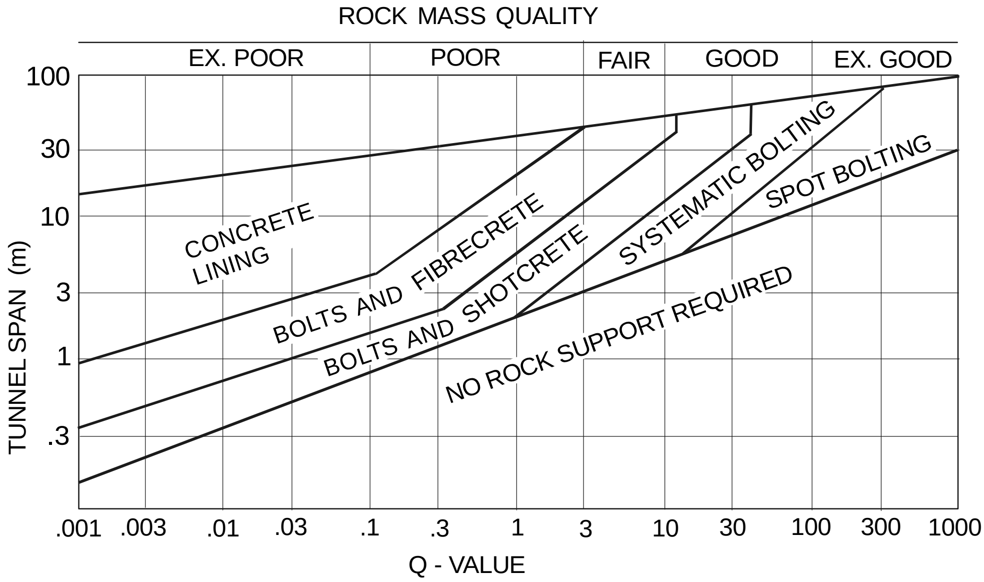


Figure 9 The Q system developed by the Norwegian Geotechnical Institute (NGI), is used world-wide for classification and evaluation of the rock support.

6.1 Example: Calculation of the amount of rock support and the tunnelling progress

The ground conditions along a tunnel have been assessed from geological maps and an engineering geological survey. A simplified section along 350 m of the tunnel is shown in Figure 10. Based on this information a classification of the ground has been worked out for the tunnel. It consists of the following 9 classes as shown in Table I:

- The ground conditions between weakness zones and faults, consisting of:
  - rock masses without rock stress problems with 4 classes
  - rock masses with rock bursting problems with 2 classes
- Weakness zones or faults with 3 classes

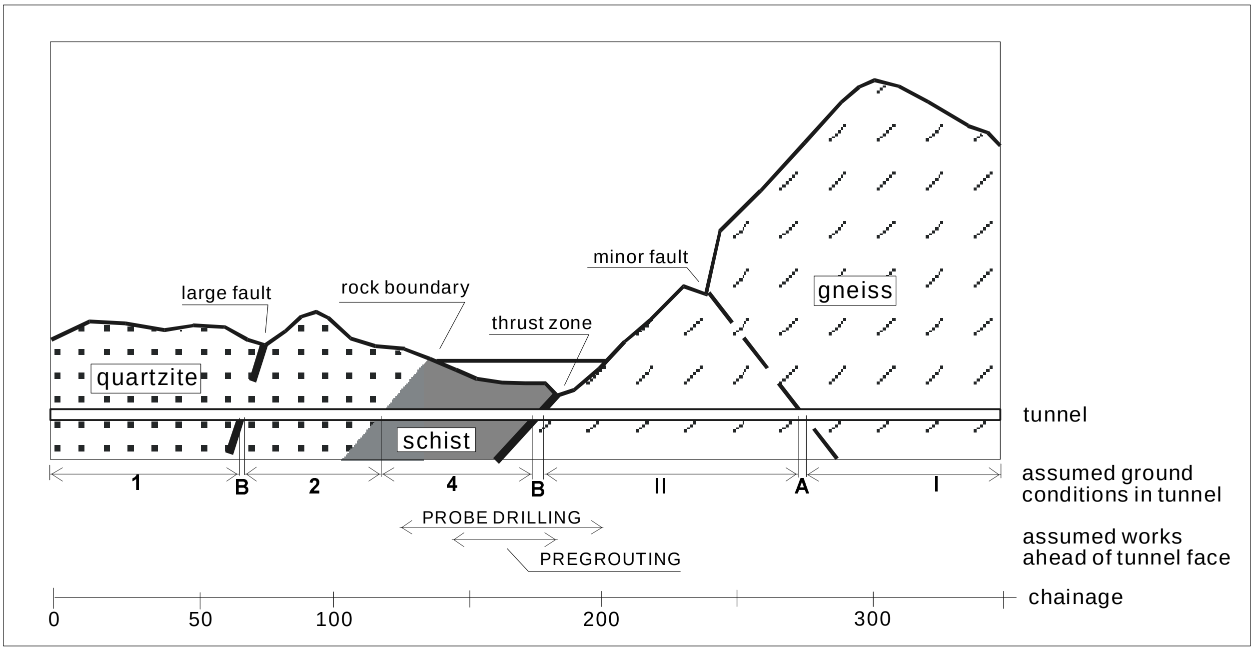


Figure 10 The assumed geology and ground classes along a tunnel

The support in each class is determined from the ground condition (Q-value) within the class, the size of the tunnel and the required stability of the tunnel.

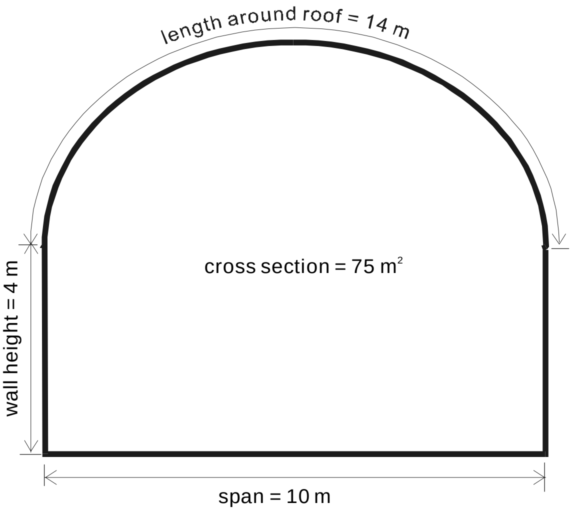
The assumed distribution of these classes along the tunnel shown in Figure 10 has been applied to calculate the distribution, amount and types of rock support in Table III. Also the tunnel data in Table II have been applied in the calculations.

TABLE I THE CLASSIFICATION OF THE EXPECTED GROUND CONDITIONS INTO 9 CLASSES WITH APPROPRIATE ROCK SUPPORT

CLASS	ROCK MASS QUALITY	Approx. Q-value	EXTRA SCALING (h / round)	ROCK BOLTS (nos / 10 m2) wall		roof		SHOTCRETE (mm thickness and % per m tunnel) wall			roof			CONCR. LINING % per m tunnel	
				at face	later	at face	later	at face	later	% / m	at face	later	% / m	at face	later
		1. ROCK MASSES WITHOUT ROCK STRESS PROBLEMS													
1	Good	> 4	0	0	0,1	0,2	0,1	0	0	0	0	50	5	0	0
2	Fair	1 - 4	0	0	0,2	0,5	0,2	0	40	5	0	50	10	0	0
3	Poor	0,1 - 1	0,2	0,2	0,3	1	0,5	50	30	20	50	30	40	0	0
4	Very poor	< 0,1	1	1	0,5	2	1	60	40	40	60	40	80	5	15
	DEGREE OF ROCK	**2. ROCK MASSES WITH ROCK BURSTING OR SPALLING PROBLEMS**													
I	Moderate	0,1 - 1	0,5	1	0,5	1	1	0	0	0	0	50	25	0	0
II	Heavy	< 0,1	1	1	0,5	2	1	40	30	20	50	30	75	0	0
	SIZE OF ZONE	**3. WEAKNESS ZONES AND FAULTS**													
3	Small	0,1 - 1	Small weakness zone (thickness < 1 m) comparable to rock support in 'poor rock mass quality'												
A	Moderate *)	< 0,1	1	2	1	2	1	50	30	25	50	40	40	0	0
B	Large *)	< 0,1	2	3	1	3	2	60	40	75	80	60	75	20	50

**) Moderate weakness zone: thickness 1 - 10 m; Large weakness zone: thickness > 10 m*

TABLE II THE APPLIED INPUT DATA FOR THE EXCAVATION AND ROCK SUPPORTING WORKS



Working time:	2 shifts/day	10 hours/shift	10 shifts/week
Tunnelling data:	4.5 m drilled round length	90% pull/round	1.5 rounds/shift
Support capacities:			
- extra scaling:	2 men/shift doing the work		
- fibrecrete:	5 m^3 /shift placed at face	5 m^3/hour placed later	
- rock bolts:	10 bolts/hour installed at face	15 bolts/hour installed later	
- concrete lining:	0.15 m/hour at face	0.2 m/hour installed later	

TABLE III THE CALCULATED AMOUNT OF ROCK SUPPORT AND WORKS AHEAD OF TUNNEL FACE. FOUND FROM THE ASSUMED GROUND CONDITIONS ALONG THE TUNNEL SHOWN IN FIGURE. 12 AND THE CLASSIFICATION IN TABLE I.

GROUND CONDITION			PROBE DRILLING & PREGROUTING				ROCK SUPPORT						
		ROCK	PROBE	PREGROUTING			INSTALLED AT TUNNEL FACE				INSTALLED LATER		
CHAINAGE	LENGTH	CLASS	DRILLING	grout matr.	drilling	packer inst.	scaling	bolts	shotcrete	c. lining	bolts	shotcrete	c. lining
	m	type	m hole	tons	m hole	nos	manhours	nos	m^3	m	nos	m^3	m
0	0												
60	60	1					0	17	0,0	0	13	2,1	0
65	5	B					4	33	6,0	1	18	4,4	2,5
120	55	2					0	39	0,0	0	24	4,7	0
140	20	4	50				8	72	17,3	1	36	11,5	3
170	30	4	75	15	600	50	12	108	25,9	1,5	54	17,3	4,5
180	10	B	25	5	240	20	8	66	12,0	2	36	8,7	5
190	10	II	25	5	240	20	4	36	5,9	0	18	3,6	0
200	10	II	25				4	36	5,9	0	18	3,6	0
270	70	II					28	252	41,2	0	126	25,4	0
275	5	A					2	22	1,9	0	11	1,4	0
350	75	I					15	165	0,0	0	135	0,5	0
TOTAL AMOUNT =			200	25	1080	90	85	845	116	6	489	83	15

TABLE IV CALCULATED TIME CONSUMPTION FOR THE VARIOUS TUNNELLING WORKS AND CALCULATED TUNNELLING PROGRESS

GROUND CONDITION			TIME CONSUMPTION				CURRENT TIME							
		GROUND	EXCA-	PROBE	PRE-	ROCK	TUNNEL		PROBE DRILLING		ROCK		TUNNELLING	
Chainage	Length	CLASS	VATION	DRILLING	GROUTING	SUPPORT	EXCAVATION		+ PREGROUTING		SUPPORT		PROGRESS	
	m		hours	hours	hours	hours	hours	weeks	hours	weeks	hours	weeks	hours	weeks
0	0		0	0	0	0	0	0	0	0	0	0	0	0
60	60	1	99	0	0	3	99	1,0	0	0,0	3	0,0	102	1,0
65	5	B	8	0	0	28	107	1,1	0	0,0	31	0,3	138	1,4
120	55	2	91	0	0	6	198	2,0	0	0,0	37	0,4	235	2,3
140	20	4	33	0	0	41	230	2,3	0	0,0	78	0,8	309	3,1
170	30	4	49	1	105	62	280	2,8	106	1,1	140	1,4	526	5,3
180	10	B	16	0	42	55	296	3,0	148	1,5	195	2,0	640	6,4
190	10	II	16	0	42	9	313	3,1	190	1,9	204	2,0	707	7,1
200	10	II	16	0	0	9	329	3,3	191	1,9	213	2,1	732	7,3
270	70	II	115	0	0	61	444	4,4	191	1,9	274	2,7	909	9,1
275	5	A	8	0	0	5	453	4,5	191	1,9	278	2,8	921	9,2
350	75	I	123	0	0	33	576	5,8	191	1,9	311	3,1	1078	10,8

The tunnelling progress has been found using the workin time and capacities for the tunnelling works, shown in Table II. The given rate of excavation (drill & blast and mucking out) expressed as the amount rounds per shift has a major influence on the tunnelling progress. It depends on several factors such as the tunnel size, the equipment to be used and the experience of the contractor. A thorough analysis of the features involved is important.

The tunnelling progress shown in Table IV has been calculated from the data given in Tables II and III. The data in this table are presented graphically in Figure 11. For the 350 m long section of the tunnel the construction time is 10.8 weeks including 3.1 weeks for the rock support and 1.9 weeks for the works ahead of the tunnel face. This gives an average progress of 32.4 m/week. (Without rock support and works ahead of the tunnel face 60.8 m/week could be achieved.)

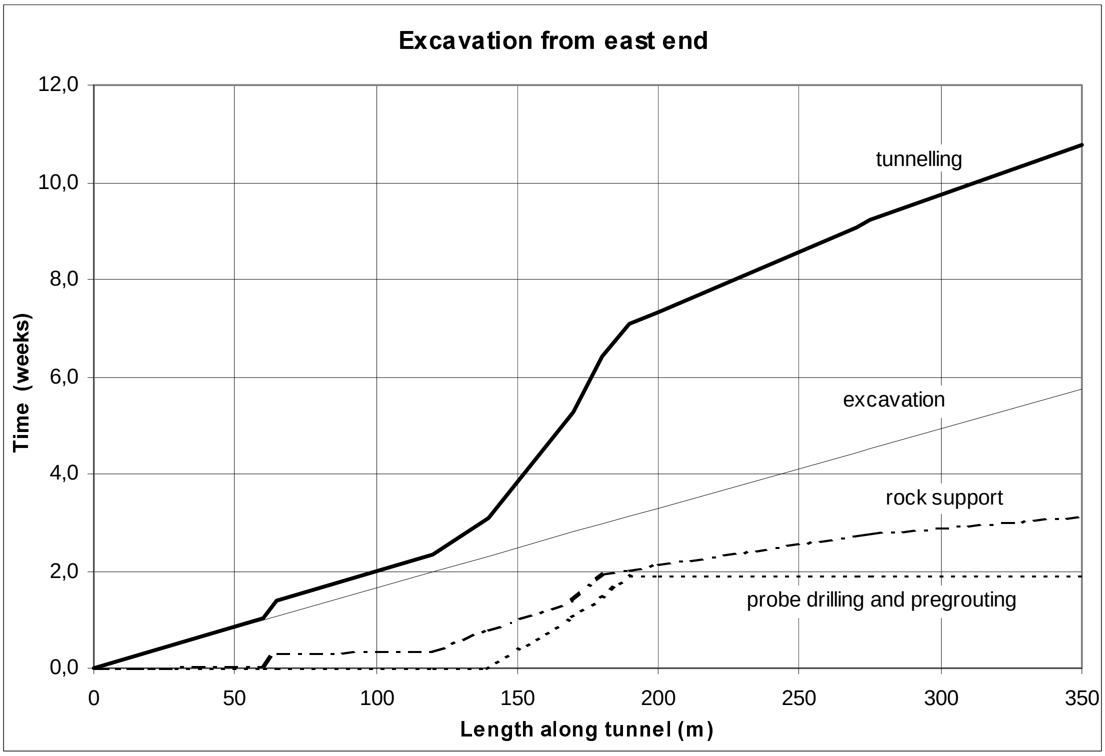


Figure 11 The tunnel progress and the time required for the main tunnelling works.

As computer spreadsheets have been used in all the calculations, it is easy to adjust for different capacities, working time or changes in the ground conditions.

7. REFERENCES

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