# Tunneling in Móberg

VIRKIR

**Orkustofnun**
**National Energy Authority**

# Tunneling in Móberg Formations

**Study**

**Part I**
**Text**



**EWI**
Electro-Watt, Engineering Services Ltd.
Zurich, Switzerland

**VIRKIR**
Associated Engineering Consultants Ltd.
Reykjavik, Iceland

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# LIST OF CONTENTS

# 1 - INTRODUCTION

Several proposed hydro-electric projects in Iceland are situated in areas where the rock consists mainly of so-called moberg. This formation is described in the geological report on the Sigalda Project and, whilst moberg does very considerably from one location to another, that found at Sigalda can be considered as typical.

Only one tunnel has to date been constructed in moberg, the headrace tunnel of the Efra Sog (Steingrimsstöd) power plant on the River Sog in southern Iceland. The difficulties encountered in this work were due mainly to the inflow of ground water and to overbreak. The diversion of the River Skafta into the Tungnaa, which would permit the installation of a fourth group at Sigalda station, would involve tunneling through extensive moberg formations.

Orkustofnun, The National Energy Authority, requested Electro-Watt Engineering Services Ltd of Zurich, and Virkir Associated Engineering Consultants Ltd of Reykjavik, to report on the probable methods and costs of constructing in moberg formations, tunnels with cross-sections of 25, 50 and 75 m2, lengths of one to seven kilometres, and situated both above and below the ground water table.

Between the 12th and 19th of July 1971, a trip was made to Iceland for the purpose of studying the moberg formations at the sites of the Sigalda and Thorisvatn schemes, as well as in the areas of the proposed East Iceland and Dettifoss hydro-electric schemes. Those taking part in the visit were Mr Thomasson of Orkustofnun, Messrs Mettler, Schaer and Dr.Pircher of Electro-Watt and Messrs Kristjansson and Hallgrimsson of Virkir.

This report consists of a description of the geological and engineering characteristics of the moberg formations, details of the methods to be adopted when tunneling in moberg and of special construction procedures which may have to be employed, and an estimate of the cost of constructing tunnels of the three specified cross-sectional areas.

In various chapters of this report, reference is made to specific products and equipment, this being unavoidable in any discussion of up-to-date tunneling methods. It must be stated, however, that such references imply no particular recommendation of the products concerned, they are intended solely to describe certain typical modern techniques which could be made use of when tunneling in moberg formations.

# 2 - SUMMARY AND CONCLUSIONS

Moberg is the name given to volcanic rocks of late Tertiary and Quaternary times which were erupted beneath glaciers over extensive areas of Iceland. The formation of lava flows was thereby prevented and the heterogeneous primary moberg, which consists of tuff, breccia, pillow lava and basaltic intrusions was formed. Reworked moberg is that which has been eroded and transported by the overlying glaciers and mixed with morainic material. The moberg formations are generally very permeable and contain large ground water systems as well as perched water tables. Site tests have indicated that with the exception of certain pillow lavas and the basalt intrusions, the formations can be removed by ripping, but that drilling in moberg may be often slow and difficult.

For the purposes of this study and for the estimation of tunneling costs, three tunnel cross-sections were specified with areas of about 25, 50 and 75 m2. In addition four different lining types have been designed for each section to allow for variation in rock conditions. Thin concrete linings in continuous contact with the rock have been assumed, these are able to deform during the process of redistribution of stresses and the full supporting effect of the lining only develops when the tunnel ring is closed by concreting the invert. Free-flow tunnels have been assumed and therefore no internal water pressure has been allowed for, and the build-up of external water pressure on the lining will be prevented by the provision of drainage holes. Construction methods for each tunnel section and lining type are fully described and costed. The minimum lining thickness considered consists of steel mesh and shotcrete and for the other lining types use would be made of Bernold sheets which provide immediate protection of the tunnel section and also form the reinforcement of the concrete lining.

Particular construction methods to be employed in ground water zones are described, and several methods of drilling, blasting and rock-bolting are detailed which could be used to overcome certain difficulties posed by the structural characteristics of the moberg formations. The employment of mechanical tunneling methods - shield diggers and tunnel borers - is examined and the technical and economical limitations on the use of such equipment are discussed.

Detailed cost estimates for each tunnel section and lining type are given and additional estimates are made for work in ground water zones. For an assumed distribution of lining types total costs are hence developed for the construction of tunnel of each section having lengths of 1,3 and 7 kilometres. It must be stressed due to the still limited geological data and the shortage of information on tunneling experience and costs in Iceland, these estimates can at present only be considered as approximate and much work remains to be done in connection with particular projects if accurate estimation of construction costs is to be made possible.

Moberg is a structurally heterogeneous formation of varying hardness, but this study has shown that it consists in general of relatively stable formations in which, using the construction methods described, it will be possible to construct tunnels having the cross-sections detailed and fulfilling the requirements of hydraulic free-flow conditions. In all sections, steel supports are provided which will themselves form a part of the final lining and the use of continuous supports such as Bernold sheets which can be erected immediately and backfilled with pumped concrete would be very adaptable to the changing rock conditions, would ensure safe working conditions and would prevent rockfall and excessive overbreak. It must be made clear that tunnels of 50, and particularly of 75 m2 cross section are large structures in which difficult rock conditions would pose correspondingly greater construction problems than need be expected with small size tunnels.

Before embarking on the construction of any tunnel several exploratory adits must be constructed at the site in order to closely examine the stability and condition of the rock formations, and in particular the strength and deformation characteristics of the various rock types must be measured by means of both in situ and laboratory tests. Extensive drilling and blasting trials will also be required in order to assess the drillability of the various rock types, to closely study the efficiency and effects of blasting, and to enable the most suitable construction method to be selected. An extensive programme of bore hole drilling will be required in order to determine the position of the ground water table and, by means of piezometer measurements, the flow behaviour therein.

In connection with the possible employment of tunneling machines, which from a technical point of view would appear quite feasible, it will be necessary, in addition to certain of the exploratory studies mentioned above, to also measure the hardness and compressive and structural strengths of the formations to be excavated in order that cutter costs and power consumption can be calculated.

A final conclusion is that, despite the widely varying physical and structural characteristics of the moberg formations, tunnel construction in them is most certainly possible, providing that the principles and construction methods described in this report are applied. The only qualification to this conclusion is that the alignment of any proposed tunnel be closely studied by means of exploratory adits and trial borings and that the physical properties of the various rock types be determined by extensive measurements and tests. It is these studies which must constitute the next stage of any project for tunneling in moberg.

# 3 - GEOLOGY

## 3.1. GENERAL

Iceland is entirely built up of volcanic formations. The extensive glaciations which occured during late Tertiary and Quaternary times greatly effected the volcanic activity which continued beneath the ice; the formation of lava flows was prevented and so-called moberg, a mixture of pillow lavas, breccias and tuffs, was produced.

The extent of the moberg formations in the Tungnaa and Skafta regions can be clearly seen from sheet 6 (south-central Iceland) of the 1:250,000 Geological Map of Iceland.

In preparing this report on tunneling in moberg formations, extensive use has been made of the following geological reports in which the characteristics of moberg are described:

Sigalda Hydro-electric project.
Feasibility Report of 1971, Elektro-Watt
and Virkir.
Chapters 2 and 3.

Thorisvatn Geological Report,
Thoroddsen and Partners
Volumes I and III - February 1970
Supplement to Volume II, The Vatnsfell
Diversion, September 1970.

## 3.2. THE MOBERG SERIES AT SIGALDA

The moberg at Sigalda is of late Pleistocene age, most of it dating from the last glaciation, and it is found mainly in ridges which trend from north-east to south-west. Moberg is structurally an extremely heterogeneous formation, and in the Sigalda area massive moberg occurs in two main forms: primary moberg and reworked (or pseudo-) moberg.

Primary moberg consists of several rock types which often merge into one another, and vary from moberg tuff and breccia to pillow lava and veins,dykes or minor intrusions of basalt. The tuff can appear either as a dense massive formation tens or even hundreds of metres thick, or as a matrix in the breccia or between the pillows of pillow lava.

Moberg breccia mainly consists of fragments of basalt in a matrix of moberg tuff, the proportion of fragments varying considerably. Pillow lava is made up of piles of rounded, elongated masses or pillows of lava, with glassy or very fine crystalline outer surfaces caused by rapid cooling. The interiors of the pillows are more coarsely crystalline although also finely grained and often vesicular. Each pillow has its own jointing system, the cracks being generally formed perpendicular to and along the rounded surface and growing thinner towards the centre. As mentioned above, moberg tuff is often found in the interstices between pillows. Basaltic veins,dykes and intrusions are usually closely jointed, and form a reinforcing grid within the moberg mass.

Reworked moberg is moberg which has been eroded and transported by the overlying glacier and often also mixed with other glacier transported material. It varies between primary moberg on one hand and consolidated moraine (tillite) on the other, this variation, as well as the colour range from light brown to light grey, depends on the extent of the reworking and the degree of mixing with other material. Reworked moberg is finer grained and more compact than primary moberg and, although thin in comparison, is the tightest part of the moberg group.

## 3.3. THE MOBERG SERIES IN THE VATNSFELL AREA

The moberg of the Vatnsfell area is very fully described in the Thorisvatn Geological Report of Thoroddsen and Partners, which details the geological studies carried out in 1969 and 1970 in connection with the development of Lake Thorisvatn as a storage reservoir for the Burfell and proposed Sigalda power plants. The diversion canal which will connect the Lake with the River Tungnaa upstream of Sigalda crosses extensive and varied moberg formations which were reported on in great detail by Thoroddsen; this chapter contains therefore only a short summary of these findings.

The Vatnsfell moberg formations resulted from a late glacial eruption and are composed of three main constituents, crater wall or clastic material, lava flows and crater fillings.

The crater wall formations consist of well consolidated breccias, coarse tuffs and thin-bedded tuffs which usually show some stratification. Drilling tests gave good core recovery except in the thin-bedded tuffs and average drilling speeds with tricone bits of 9m/hour were recorded; low permeabilities were indicated. Seismic velocities varied from about 1100 m/s in the tuffs to over 3000 m/s in the breccias.

The lava flow formations in fact consist of pillow lava with regular structure, mixtures of pillow lava with compacted sand and basaltic veins, and fractured and friable tuffaceous sand with basalt fragments; these formations having been formed by rapid cooling of the magma in contact with melting ice. Drilling speeds varied from one 1-2 m/hour in the pillow lava to 15 m/hour in the sand, and high permeabilities were measured which varied from 10 to nearly 200 Lugeons - it should be noted that the pillow lavas are much more permeable than the sands. Seismic velocities were on average about

1000 m/s - this indicates rocks which can be easily ripped by a CAT.D-9 - and rose to 1500 m/s below ground water level. Tillite appears in many places on the surface of these formations; it is a hard consolidated moraine material formed by the reworking effect of the glacier.

The crater filling material is reworked moberg in the form of sands which are found as deposits in lakes, depressions and ancient craters. The drilling speed was high (approx. 20 m/hour) but no core recovery was possible, and on average permeability of 58 Lugeons was measured. The seismic velocity is low at 300-1300 m/s.

## 3.4. TECTONICS

In the Sigalda area there are no outstanding tectonic features, the landscape following the general tectonic pattern in this region which is dominated by ridges of moberg and pillow lava formed by eruptions along fissures in the north-east to south-west direction.

Some graben tectonics probably occured in connection with this volcanism, which was very active during the postglacial period but which did not reach the Sigalda area and, smaller and more isolated fractures and small faults are seen at some places in the moberg areas.

Earthquakes are common in this part of the country. Within the volcanic belt they are usually shallow and weak, but beyond this belt, they are less common but often deeper and of larger magnitude.

In the Thorisvatn area the most common tectonic systems are normal faults in a north-east - south-west direction. These faults are often graben walls and are most prominent in the volcanic belt east of the lake. The displacement of the main postglacial graben systems varies between 2 and 20 m.

# 4 - ENGINEERING GEOLOGY

## 4.1. GENERAL

In addition to the general geological description given in the previous chapter, it is important to consider the engineering properties of the various formations and to assess their probable behaviour during construction. Reference is again made to the Sigalda and Thorisvatn reports, from which the data for this chapter is mostly drawn.

A tunnel was studied as a possible alternative to the inlet channel at Sigalda; this would have crossed relatively hard but fissured pillow lava and consolidated breccia, and steel support during construction would have been required. A concrete lining would have been necessary both to prevent water loss and to stabilise the walls and vault. This tunnel solution was eliminated in favour of a cut-and-cover channel and inlet canal to be excavated in the main by ripping.

Four alternative routes for the diversion from Lake Thorisvatn to the Tungnaa were investigated of which that consisting of a canal passing to the west of Vatnsfell was selected in preference to tunnel routes farther east. On the basis of his studies, Thoroddsen drew the following conclusions concerning the suitability of the Vatnsfell moberg for tunneling:

- Of the crater wall formations, the breccia would be good tunneling rock in which blasting would probably be necessary, but lining could be avoided. The coarser tuffs are more heterogeneous and lining to prevent erosion would be necessary, these were, however, still classed as good tunneling rock. The thin-bedded tuff was considered much less suitable.

- The lava flow formations would certainly be poor rock for tunneling; in particular the loosly consolidated tuffaceous sand would be difficult. Avoidance of blasting would reduce excavation problems in these formations, but a structural concrete lining would always have to be allowed for.

- The crater filling material probably has similar engineering properties to the tuffaceous sand of the lava flow formations.

It should be recorded that the tunnel for the Sog River project in southern Iceland was excavated in moberg which gave negligable core recovery and was thus probably similar to the Vatnsfell moberg.

The difficulties here, which were overcome however without prohibitive cost increase, resulted probably from the ground water and conventional blasting technique.

## 4.2. RIPPING TESTS AND SEISMICITY

Seismic investigations and ripping tests have been carried out in connection with both the Sigalda and Thorisvatn projects and their results are fully detailed in the respective reports. In connection with tunneling, the ripping characteristics of the formation are particularly important for the consideration of the use of certain types of modern tunnel boring machines as described in chapter 11.

The seismic velocities measured at Sigalda can be summarised as follows:

| Formation | Seismic velocity, m/s |
|---|---|
| Overburden | 300 - 600 |
| Predominantly breccia | 600 - 1100 |
| Predominantly pillow lava | 1500 - 2000 |
| Predominantly basalt intrusions + dykes | 3000 - 3400 |

The values recorded increased from 2000 to 3400 with increasing proportions of basalt. According to the Caterpillar Performance Handbook, formations have seismic velocities of up to 2500 m/s which are rippable with a CAT D-9.

The ripping tests at Sigalda were made using a CAT D-8 which, again according to the makers, should be able to rip about 140 m3/hour of in situ material with a seismic velocity of 2000 m/s. In one test in brecciated pillow lava, an output of 60-90 m3/hour was obtained which suggests that from both technical and economic points of view the rock is effectively almost unrippable. In loosly cemented breccia on the other hand, the material was very easily removed with relatively little ripping.

For the Thorisvatn tests, a CAT D-7E was employed. In pillow lava having a seismic velocity of 1000 m/s, it was possible to remove 135 m3/hour. In another lava flow formation which consisted of pillow lava, sand and basaltic veins, a vein was only penetrated with great difficulty whereas the coarse tuffaceous sand and broken-up pillow lava could be excavated without ripping at all.

The results of these tests, and the conclusion that moberg formations are generally rippable and that blasting will only be necessary for basaltic intrusions and veins, were confirmed during the excavation of the Thorisvatn diversion during 1970. The application of ripping techniques during tunnel construction is discussed in detail in chapter 11.

## 4.3. DRILLING IN MOBERG

The drilling and grouting tests carried out at Sigalda, and the prospection drillings made at Vatnsfell, demonstrated that drilling in moberg formations may often be difficult and slow. This is a result both of the heterogenity of the formations as well as of the softness and friability of certain of the rocks.

At Sigalda, drilling for cores was only successful in about 30% of the cases. Elsewhere, and particularly in formations of pillow lava with a sandy matrix, it was necessary to resort to tricone drilling whereby the rock types are identified partly on the basis of the measured drilling speed. For the grouting tests, tricone bits were also used but it was necessary to grout up the hole and re-drill about every three metres, in order to prevent the drill steel and grouting packer from getting stuck by collapse of the borehole wall.

The average drilling time for the Sigalda grouting tests was 16 minutes per metre, equivalent to 3.75 m/hour, (see Appendix 2-07).At Vatnsfell the speed varied between 1 and 20 m/hour.

Methods of overcoming the problems of drilling in moberg are discussed in detail in section 1 of chapter 10.

## 4.4. PERMEABILITY

The various rock types found in moberg formations have extremely variable permeabilities but,without extensive tests, it would be difficult to determine a representative value of the permeability coefficient (k) for the heterogeneous formation. Some values of permeability have, however, been deduced from the Lugeon values measured during the pressure tests carried out at Sigalda; on the assumption that one Lugeon is approximately equal to a permeability of $10^{-5}$cm/s, average values of k for the two deepest boreholes at Sigalda (E15 and E11) of $5 \times 10^{-4}$ and $1 \times 10^{-3}$ cm/s respectively were estimated. It is clear, however, that moberg formations in general are highly permeable, and k- values of 1 or even $2 \times 10^{-3}$cm/s are to be expected. The variations in permeability between one rock type and another can be clearly seen from the borehole logs given in Appendix 2-03, on which the measured Lugeon values are plotted. A similar wide variation in values was measured at Thorisvatn.

Grouting tests indicated that moberg formations can be sealed but that for this sealing to be effective, pressures of at least 40 kg/cm2 will have to be employed. The spread of cement grout in moberg is about 2 metres.

## 4.5. GROUNDWATER

The very permeable moberg formations form good aquifers in which large groundwater flow systems exist, these having been investigated by means of explorarory borings at both Sigalda and Thorisvatn.

At Sigalda the groundwater flow system lies in a W to NNW direction. The pattern has been somewhat disturbed where the river had eroded a deep channel into the bedrock, thus intercepting the shallow groundwater flow; in such cases, numerous springs indicate the approximate level of the ground water table. In Borehole E-11 (see Appendix 2-03) this level was located 50 m below ground surface.

In the Thorisvatn area, the groundwater potential lines have been mapped. A large groundwater flow system exists in the moberg to the east of Thorisvatn Lake, this feeds the lake with an unknown quantity of water as well as springs upstream of Sigalda where 8 l/s have been measured. In Borehole O-2, near the Vatnsfell control structure, the water table was 40 m deep.

Above the permanent groundwater table, perched groundwater tables exist in many places, these do not carry water over long distances and gradually seep down into the main system (see Appendix 2-06). In many cases these perched tables are fed only by local precipitation and will therefore not yield much water when encountered in e excavations, except during a short period immediately following exposure. When, however, they are fed by lakes (e.g. Thorisvatn) unlimited inflow could result.

For every tunnel project, the groundwater conditions must be fully investigated before work is started in order that the dry zones, satchurated zones and perched water tables can be located and examined. Piezometres should be installed in all boreholes to enable variations in ground water conditions to be measured

# 5 - STUDY TOUR OBSERVATIONS

It was possible to supplement the information available in existing reports by observations of moberg formations made during the visits to the Sigalda and Thorisvatn areas in July 1971. The variation of moberg formations between one site and another,from soft earth material to hard rock, is pronounced, and great heterogeneity of structure, from fine grained tuff to coarse breccia and strongly jointed pillow lava, with pillows up to 30 cm in diameter, is evident.

Moberg appears at first sight to be very loose, brittle and disintegrated but on closer investigation of the rock faces can in fact be seen to be quite stable and firm. These natural rock walls are very steep and often even vertical or overhanging, and their surfaces are relatively hard and solid. Along the right bank of the River Tungnaa, numerous natural caves have been eroded in the tuff breccia; these are of considerable size - up to 5 m in diameter - and their abutments are resistant to erosion by the river flow (see photograph 8). Natural caves of up to 3 m diameter have even been observed in pillow lava formations. The existance and stability of these caves imply characteristics which would favour tunnel construction in moberg. The loosely-cemented breccia which was exposed in the trench excavated during Ripping Test No.II seemed to be the weakest moberg formation encountered at Sigalda, but despite this appeared quite cohesive (see photographs 9 and 15).

The crater wall tuff seen at the site of the outlet structure on the Vatnsfell diversion canal (photographs 11 and 12) is one of the best moberg rocks and would appear to be good tunneling rock.

The photographs taken during the visit are reproduced in Appendix 1 together with a detailed description of each view, and give a good idea of the nature of the moberg formations which were observed.

# 6 - TECHNICAL CLASSIFICATION

## 6.1. TYPES OF ROCK MASS

On the basis of the data at present available which is summarised in chapters 3 and 4 and of the observations made during the visit to Iceland in 1971 the moberg formations can technically be classified into the five following types:

- pillow lava
- brecciated pillow lava
- breccia
- loosely cemented moraine-type breccia
- tuff

The technical characteristics of these formations are described in the following section.

## 6.2. PILLOW LAVA

### 6.2.1. Appearance, structure, etc.

Rounded or cubic cobbles of hard basaltic rock, 10-30 cm in size, the rock-mass being strongly jointed in radial and surface directions. The texture is massive and solid against loosening and the structure spherical or sometimes bedded. Basaltic veins, dykes and interstices exist.

### 6.2.2. Probable construction characteristics.

Steep, vertical and even overhanging walls would be stable and self-supporting vaults can be assumed providing the local joint configuration is favourable. Drilling is possible but may prove difficult; the maximum drilling depth will be 2-3 metres although in some cases only one metre may be possible. Ripping will either be impossible or very difficult and uneconomic, and blasting is therefore essential for excavation. Tunnel linings of mesh and shotcrete or locally of steel sheeting and concrete will be necessary.

## 6.3. BRECCIATED PILLOW LAVA

### 6.3.1. Appearance, structure, etc.

A very fractured, heterogeneous, weathered and friable formation. Rounded or cubic hard basaltic cobbles of 5-20 cm size, similar to a coarse stone mixture. Sandy or clay filling in interstices and cracks with some basaltic veins. No cementation, maximum natural slope 60$^{o}$.

#### 6.3.2. Probable construction characteristics

Steep but not vertical walls will be stable but self-supporting vaults will rarely be possible. Percussion drilling will be very difficult and, due to the irregular fragmentation, only possible to 1-2 metres depth. Ripping is possible but will not be easy, although this will depend on the extent of the development of fissures. Steel sheet lining will probably be essential for tunnel excavation, and in places even poling-plate advance may be needed (see chapter 8).

### 6.4. BRECCIA

#### 6.4.1. Appearance, structure, etc.

Well consolidated and cemented rock-mass with pillow lava and basalt fragments in a resistant, medium hard, dense and fine-grained matrix. It is possible that the firm, and what seems to be well consolidated surface is only an effect of weathering conditions, and that within the rock-mass the breccia may be less stable. Firm and stable outcrops exist in vertical walls and natural caves, and the sharp edges and hard surface borders of these can rarely be broken.

#### 6.4.2. Probable construction characteristics.

Steep, vertical or overhanging faces would be stable and self-supporting vaults of 3-5 m span would be possible. Drilling is possible to a depth of 2,3 or even 4 metres, but due to rock-mass configuration holes of only 1 m depth may prove difficult locally. On the surface, ripping is possible and may not be difficult, but for tunnel excavation, drilling and blasting will probably be necessary. It must be assumed that lining with mesh and shotcrete or with steel sheets and concrete will be necessary.

### 6.5. LOOSELY CEMENTED MORAINE-TYPE BRECCIA

#### 6.5.1. Appearance, structure, etc.

A sandy gravel with numerous pillow fragments, the whole being cohesive but porous and weak, more akin to soil than to rock. Natural slopes can be as steep as $60^{o}$ or even slightly more.

#### 6.5.2. Probable construction characteristics

This formation can easily be excavated to give stable slopes of up to $60^{o}$, but some danger of slides cannot be excluded. Normal tunnel driving may not be possible and excavation without blasting but by poling-plate advance must be allowed for. Surface excavation will be very easy by ripping or even by simple dozing.

## 6.6. TUFF

### 6.6.1. Appearance, structure, etc.

Fine grained or coarse, dense, stable and massive rock-mass of average to low hardness, sometimes strongly affected by tectonics. Faults exist and cracks are filled with clay sediments.

### 6.6.2. Probable construction characteristics

It will be possible to excavate near vertical or overhanging walls and self-supporting vaults of 3-5 metres span, or even more depending on quality of the rock, should be stable. Normal percussion drilling should not prove difficult to depths of 2-4 metres and blasting will be necessary. Mesh and shotcrete lining or a thin concrete lining with steel sheets is recommended. Ripping is possible.

## 6.7. CLASSIFICATION ACCORDING TO LAUFFERS DIAGRAM

Lauffers diagram is used to classify the rock by considering the relationship between the clear span (1 *) and the estimated maximum allowable time between excavation and the placing of the tunnel supports (the stability time, $S_t$)

The degrees of stability for the various moberg formations have been estimated, and for each one its probable position in Lauffers diagram has thus been fixed. For this purpose, the diameter of the tunnel has been assumed to be greater than 5 metres and therefore the limiting clear span is equal to the distance between the last support and the tunnel face.

The relationships are shown on the following diagram, on which they can be compared with the values for other rock formations which were published in 'Rock Mechanics', Vol.2, No.4, of December 1970.

This classification can only at present constitute of first estimate, but is considered necessary to more closely define the properties of the moberg formations. Additional investigations will be required to give a more exact classification.

## 6.8. THE ESTIMATED TECHNICAL CHARACTERISTICS OF THE MOBERG FORMATIONS

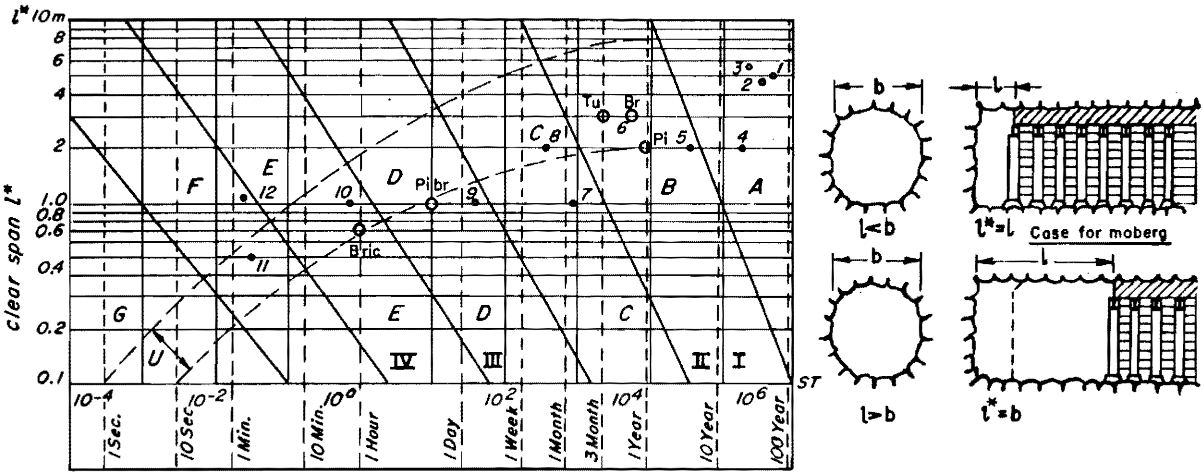
The analysis and classification of the moberg formations are summarised in the following table:

Estimated technical characteristics of Moberg formations

| Moberg Formation | Class according to Lauffers Diagram | Quality of Rock-mass | Hardness * | Seismic velocity | Ripping performance with CAT.D-9 | Drillability | Excavation | Supporting system (lining) |
|---|---|---|---|---|---|---|---|---|
| Pillow lava (photos 1,2, 3+4) | B jointed | Stable to slightly friable | very hard | at least 1500-2500 | probably not rippable | drilling possible but only with difficulty | blasting required | shotcrete and mesh or steel sheets and concrete |
| Brecciated pillow lava (photo 10) | D brittle | friable | hard | 1000-2500 | can be ripped with difficulty | drilling very difficult | blasting required | steel sheets and concrete with locally poling-plates |
| Breccia (photos 1,5, 6,7,8+11) | B jointed | stable to slightly friable | medium hard | 600-1100 | rippable | drilling possible but only with difficulty | blasting required | shotcrete and mesh or steel sheets and concrete |
| Loosely cemented moraine-type breccia (photo 9) | E very brittle | very friable | weak | 300- 600 | easily ripped or can be simply dozed | drilling not possible | blasting often unnecessary | steel sheets and concrete with poling-plate advance |
| Tuff (photos 6+12) | B jointed | stable to slightly friable | medium hard | 1000-2000 | rippable | drilling possible | blasting sometimes necessary | shotcrete and mesh or steel sheets and concrete |

* Basalt = very hard, Granite = hard

## LAUFFER'S DIAGRAM




| ROCK MASS | EXAMPLES** |
|---|---|
| A MASSIVE | 1 MASSIVE GRANITE |
| B JOINTED | 2,3 GRAYWACKE WITH THICK LAYERS |
| C VERY JOINTED | 4, JOINTED QUARTZITE |
| D BRITTLE | 5,6 WEATHERED GNEISS |
| E VERY BRITTLE | 7,8 CLAYEY SHALES (ORDOVICIAN) |
| F SQUEEZING | 9 DITTO, WEATHERED |
| G VERY SQUEEZING | 10 ELUVIUM OF GNEISS |
| | 11,12 DENSE SAND (WET) |

U = RANGE OF MOST APPLICATION
ST = STABILITY TIME

**) FROM ROCK MECHANICS DEC. 1970 VOL. 2 Nr. 4 SPRINGER VERLAG

## ESTIMATE OF CLASSIFICATION FOR MOBERG

| CLASS | MOBERG FORMATION | TYPE OF LINING |
|---|---|---|
| B<br>JOINTED | Pi = PILLOW LAVA<br>Br = BRECCIA<br>Tu = TUFF | I or II |
| D<br>BRITTLE | PIbr = PILLOW LAVA BRECCIATED | III |
| E<br>VERY BRITTLE | BrIc = BRECCIA LOOSELY CEMENTED TYPE "MORAINE" | IV |

# TUNNELING IN MOBERG
# LAUFFER'S DIAGRAM OF STABILITY TIME

# 7 - TUNNEL SECTIONS AND STATIC CONSIDERATIONS

## 7.1. GENERAL

The three cross-sections studied, A,B and C, have areas of about 25,50 and 75 m2 respectively. Their main dimensions are given below and the sections are detailed in Appendix 2 - 09.

| Section | Radius R(m) | Internal Area(m2) | Max.internal height (m) | Max.internal width (m) |
|---|---|---|---|---|
| A | 2.9 | 23.4 | 5.00 | 5.80 |
| B | 4.3 | 52.2 | 7.50 | 8.60 |
| C | 5.2 | 75.6 | 9.00 | 10.40 |

To allow for the considerable differences in the constructional properties of the moberg formations, four lining types were designed for each of the three tunnel sections studied; these are designated I to IV. The differences in rock properties are related to the standing time, i.e. the time which can elapse between excavation and erection of the tunnel supports. For all sections, relatively thin linings have been considered.

## 7.2. THE NEW AUSTRIAN METHOD

The tunnel linings have been defined on the basis of the knowledge and experience gained during the application of the new Austrian construction method. This method uses, instead of the previously employed strong lining of thickness $t \approx R/5$, a relatively thin elastic shell lining placed in continuous contact with the rock and having a thickness of R/15 - R/25. The following dimensions have thus been selected:

| Section | R | R/5 | R/25 - R/15 | Selected values Lining II | Selected values Lining III |
|---|---|---|---|---|---|
| A | 2.9 m | 58 cm | 12 - 19 cm | 20 cm | 25 cm |
| B | 4.3 m | 86 cm | 17 - 29 cm | 25 cm | 30 cm |
| C | 5.2 m | 104 cm | 21 - 35 cm | 28 cm | 33 cm |

The thin lining offers only little resistance to the rock face during the process of redistribution of stresses; this process is in consequence accompanied by relatively large deformations and a considerable reduction in the final loading on the lining when the ultimate condition of stress (the formation of protective zones) is attained (see Appendix 2 - 16, relationship between lining resistance and deformation). The lining furthermore deforms in such a way that its neutral axis conforms extensively to the thrust line of the loading on the arch, and thus bending moments in the lining are almost avoided.

When redistribution of stresses has taken place, usually after a period of a few months, the gunite sealing layer or, if necessary, a secondary structural lining can be applied. For adduction tunnels, a secondary lining is normally not necessary, except when required to resist external ground water pressure (see section 5.2 of this chapter)

It is necessary from both structural and construction points of view to allow for concreting of the invert as soon as possible in order to close the tunnel ring and develop the full supporting effect of the lining; in moberg formations it is estimated that this should be carried out within one month of excavation.

## 7.3. CROSS - SECTIONS AND DETAILS OF THE LININGS

For many obvious reasons in tunneling a circular section is desirable, but such a section has the disadvantage from the constructional point of view of a strongly curved invert. The pressures to be expected in the cases here discussed are not very high (the maximum overburden depth will be about 100 metres) and for this reason it is thus possible to assume horseshoe sections with considerably flatter invert curvatures for the tunnels. For each section, the internal dimensions are kept constant irrespective of the rock type and lining dimensions.

The thicknesses of the four thin linings proposed have been determined to correspond with the rock qualities encountered. Continuous lining is proposed for static reasons, in order to improve hydraulic flow conditions by giving favourable values of the Strickler friction coefficient, and also to avoid any danger of erosion of rock surfaces. Lining type I will be formed of shotcrete with steel mesh reinforcement, and types II - IV will consist of varying thicknesses of reinforced concrete with normally a gunite inner facing.

For all sections, a longitudinal drainage duct will be constructed beneath the lightly reinforced concrete invert lining; continuous drainage and hence clean working conditions will thereby be ensured.

Lining type IV will be applied in very loose rock when excavation using steel poling plates is necessary, the lining is therefore slightly different from types II and III, as pumped mortar will be necessary to fill the void between the concrete lining and the rock which is occupied during driving by the poling plates themselves (see chapter 8).

The lining types are detailed in Appendices 2 - 10 to 2 - 13 for section A (23.4 m2) but they apply equally for the larger sections B and C (Appendices 2 - 14 and 2 - 15).

The thicknesses of the various linings for the three sections A, B and C are given below in cm.

| Section | Shotcrete | Concrete | Gunite | Pumped mortar | Total | Over-break* | Invert lining |
|---|---|---|---|---|---|---|---|
| AI | 8 | - | - | - | 8 | 15 | 15 |
| AII | - | 15 | 5 | - | 20 | 15 | 25 |
| AIII | - | 20 | 5 | - | 25 | 15 | 33 |
| AIV | - | 15 | 5 | 8 | 28 | - | 40 |
| BI | 8 | - | - | - | 8 | 15 | 15 |
| BII | - | 18 | 5 | - | 23 | 15 | 30 |
| BIII | - | 25 | 5 | - | 30 | 15 | 43 |
| BIV | - | 22 | 5 | 8 | 35 | - | 52 |
| CI | 8 | - | - | - | 8 | 15 | 15 |
| CII | - | 20 | 5 | - | 25 | 15 | 35 |
| CIII | - | 28 | 5 | - | 33 | 15 | 48 |
| CIV | - | 25 | 5 | 8 | 38 | - | 56 |

* The actual depth of overbreak is difficult to estimate in advance. Experience shows that this depth depends on such factors as geological conditions, excavation method, care taken during excavation and the dimensions of the section, etc. In order to avoid unnecessary complications in the calculations for this stage of the study, the overbreak depth for all three sections has been assumed to be 15 cm in the vault and 10 cm at the invert. For lining type IV, the use of poling plates will prevent overbreak.

For the large cross-sections B (52.2 m2) and C (75.6 m2), it must be assumed that full-face excavation will not be possible. For this reason, it is proposed that the lining of the upper half of the vault, i.e. that part excavated first, be formed with enlarged abutments able to support the vault during the remaining excavation. These abutments can furthermore be tied-back if necessary with rock anchors. The excavation method, which is known as the Belgian method, is illustrated in Appendices 2 - 21 and 2 - 22.

For lining type IV, that designed for tunnel driving using steel poling plates, it is proposed to use pre-cast concrete elements to form the centre section of the invert. This will allow immediate passage of construction traffic and will also provide convenient support for the steel erection arches. The remaining invert sections will be later formed with in situ concrete. Use of pre-cast elements has the further advantage of greatly simplifying shuttering, and could therefore also be considered for linings types I - III.

## 7.4. BEHAVIOUR OF THE ELASTIC LINING UNDER LOAD

In order to be able to correctly dimension the tunnel linings, it is necessary to consider their behaviour under load and in particular their mode of failure. According to Rabcewicz (Refs. 1,2 and 7), the mode of failure of a circular elastic tunnel lining can be described as follows and as illustrated in Appendix 2-17.

Phase 0
Under the effect of the loading in the direction of the principal compressive stresses, the lining bulges slightly such that the length of the diameter in this direction is slightly reduced.

Phase I
Shear failure of the lining occurs and the sections of lining parallel to the loading direction are forced inwards whilst at the same time shear wedges are squeezed out perpendicular to the loading.

Phase II
With the effective structural span so increased a further reduction in the length of the diameter in the loading direction results.

Phase III
The lining finally buckles at the limits of this diameter under the effect of the continuing stresses, and collapse of the section occurs.

The above failure process has been observed numerous times in practice (Refs. 1,2,6 and 9) and it has been shown that the cause of rupture in the case of linings in continuous direct contact with the rock is the high lateral load, the resulting collapse being a shear failure.

Sattler (Ref.6) has stated that, on the basis of his tests, it is essential for the stability of the lining that it be in perfect tension-free contact with the rock, and that therefore the modulus of elasticity of the rock mass is of relatively little importance.

The tunnel sections proposed in this study have been developed and dimensioned on the basis of this understanding of the mode of failure.

## 7.5. THE LOADS ON THE LINING

### 7.5.1 Redistribution pressure

The redistributed pressure in moberg can at the present time be only very roughly evaluated, as no material characteristics are yet known. To obtain however a general idea of the magnitude of the pressures to be expected, use is made of the simplified diagram given in Appendix 2 - 16. From this diagram it can be seen that the load which a very elastic lining will have to bear after redistribution of stresses, is between 10 and 20 % of the total rock mass pressure corresponding to the overburden depth. Similar results are given by the studies of Lombardi (see Ref.11 and also Refs. 2 and 3).

The following methods can be used to study the behaviour of the rock mass and to dimension the tunnel lining:

a) The deformation characteristics of the rock mass can be measured in a specially excavated investigation tunnel and the results obtained used to deduce the values for the proposed tunnel.
b) In situ measurements can be made in the tunnel itself but, because of the limited stability time of moberg, these cannot be carried out before placing of the lining. The following should be measured: deformation of the vault, radial stresses in the vault, tangential stresses in the lining and, in places, the development of tangential stresses in the rock mass as a function of time. Measurements should be made at suitable intervals and in geologically characteristic locations, and accurate descriptions of the rock conditions should be made at each measurement site. Observations should be continued in general for several months, but experience has shown that it is usually possible after a matter of only a few weeks to get a good picture of the tendancy of the behaviour of the rock mass. By this method it is possible to apply the results of tests made in the actual tunnel to the dimensioning, and if necessary the recalculation, of the lining of succeeding sections.

### 7.5.2. Water pressure

It has been assumed, as is usual for adduction tunnels, that the tunnels will be designed for free-flow and that therefore no internal water pressure will exist.

Of much greater importance therefore is the case of external ground water pressure. If the ground water table is crossed by a tunnel, its level may be lowered during construction. The build-up of external water pressure can thus be prevented by providing drainage holes in the tunnel lining, providing of course that the rock mass is such as to allow continuous flow of ground water into the tunnel (see Appendix 2-25, figure 4).

If, however, such drainage is not possible, a second inner-structural lining will be necessary, able to withstand the external pressure. Secondary linings could be constructed in all the tunnel sections here discussed, although they would involve a slight decrease in the internal cross-sectional area. This, however, could be partly compensated for by the increased Strickler friction coefficient resulting from the use of good quality shuttering for construction of the second lining.

In the case of perched ground water tables, it should be possible to avoid the second vault, providing that the tunnel crosses only the zone of infiltration below the water table (see Appendix 2-06).

## 7.6. PERMISSABLE LOADING ON THE LINING

The permissable loads on the various tunnel linings due to overburden pressure have been calculated using the theory of semi-stiff shells, as per Sattler (Ref.5). A value of $\beta_w$ = 200 kg/cm2 has been assumed for the concrete cube crushing strength, this corresponding to normal concrete with a cement content of 300 kg/m3 concrete, and 5 and 10cm2 of reinforcement per linear metre of tunnel have been assumed, with $\sigma_{0.2}$ = 2700 kg/cm2

The following permissable loads, in tons/m2, were calculated, these include a safety factor of 2.3

| Lining | Section A | | | Section B | | | Section C | | |
|---|---|---|---|---|---|---|---|---|---|
| type | Concrete | Steel | Total | Concrete | Steel | Total | Concrete | Steel | Total |
| I | 16 | 4 | 20 | 11 | 3 | 14 | 9 | 2 | 11 |
| II | 30 | 9 | 39 | 24 | 6 | 30 | 23 | 5 | 28 |
| III | 40 | 9 | 49 | 33 | 6 | 39 | 32 | 5 | 37 |
| IV | 46 | 9 | 55 | 40 | 6 | 46 | 38 | 5 | 43 |

The variation in permissable values results from the different concrete thicknesses, reinforcement quantities and radii of curvature. Overbreak concrete was not taken into account in the calculations.

## 7.7. REFERENCES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# 8 - CONSTRUCTION METHODS

## 8.1. GENERAL

Moberg is a very heterogeneous rock mass which varies from hard rock to loose material, and for this reason widely varying formations must be expected when tunneling. This variation is clearly illustrated by the Geological sections for Sigalda and Vatnsfell (Appendices 2-02 and 2-05) where it can be seen that depending on level, any tunnels would encounter widely differing types of moberg.

The method selected for constructing tunnels in moberg must therefore be planned in such a way that it is applicable without any basic modification in all formations encountered. In the first place it is therefore necessary to fulfill the requirements both of a sufficiently flexible excavation method and of the continuous support. In order that this support may be as effective as possible, steel supports will be required which can be backfilled with shotcrete or pumped concrete to form an uninterrupted contact with the rock mass.

A very suitable such support method would be a new Swiss system known as the "Bernold tunnel construction system" which makes use of thin expanded steel sheets supported on steel arches; these so-called Bernold sheets provide temporary protection against rock fall and later form the reinforcement of the concrete lining. The hitherto normal method employing steel supporting arches and steel beams could of course also be used, but this is very expensive and has statical disadvantages. The Bernold system is fully described in the Appendix-Documentation.

## 8.2. WORKING PHASES

The proposed method of tunnel construction in moberg employing the Bernold support system, can be in general described as follows:

Phase 1: Excavation.
Due to the structure and general hardness of moberg formations, drilling and blasting will usually be necessary. In certain formations however, drilling will be complicated and difficult, and special measures will have to be adopted; these are described in detail in section 1 of chapter 10. Under the Bernold system, drilling and blasting will normally be carried out under the protection of supports extending right to the tunnel face.

For drilling, a self-propelled, rubber-tyred Jumbo will be most suitable (see Appendix Documentation). For the reasons of safety and stability, the drilling depth will depend on the rock quality and the standing-time during which the newly-excavated section can be left unsupported; rounds of 2, 2 1/2 or 3 metres should be normal but in very friable rock only 1 metre rounds may be possible. Blasting will also pose certain problems, both because of the voids which exist in places in moberg, as well as because of shocks which could endanger the stability of any tunnel sections which might be temporarily unsupported for any reason.

If the moberg formation is very soft and relatively unconsolidated, excavation will be possible without recourse to blasting. In this case, the bore-Jumbo would be replaced by a conventional excavator able to scrape and break in a purely mechanical way the rock material from the tunnel face. Use could also be made of compressed air equipment, hydraulic shovels and similar plant. It will very probably be necessary in such conditions to continuously extend the tunnel supports, possibly by using steel poling-plates as described in section 4 of this chapter.

Phase 2 : Mucking-out.
After blasting, the spoil will be loaded with traxcavators or shovels into trucks or dumpers, this work is usually carried out under the protection of steel supports erected up to the face prior to blasting. Only the newly blasted section will therefore be unsupported during loading.

Phase 3 : Erection of supports.
On completion of mucking-out the steel arch supports will be erected up to the newly exposed face and the perforated Bernold steel plates placed as immediate protection against falling rock. A great advantage of the Bernold system is that the steel arches are not embedded in the lining concrete and only remain in place until the Bernold sheets have been backfilled with concrete or shotcrete; the arches can thus be used over and over again and are simply advanced after each mucking-out phase.

In the better moberg formations, a light steel mesh covered with shotcrete will suffice in place of the Bernold sheets and concrete.

Phase 4 : Concrete backfilling.
For fully effective support, it is essential that the space between the Bernold sheets and rock be backfilled as soon as the sheets are in position. This can conveniently be carried out by either pumping concrete behind the sheets or by spraying freshly-mixed concrete through the perforations in the sheets (in both these cases, the water is added and the concrete mixed at the mixer, and not at the nozzle of the gun as for shotcrete). The concrete must be vibrated until it starts to ooze through between the ribs of the Bernold sheets

The normal daily cycle would comprise these four work phases. In order, however, to make the lining fully effective, the ring must be closed by concreting the invert which, in the case of moberg, it is estimated should not be delayed more than four weeks after excavation. The drainage duct will be concreted together with the invert.

## 8.3. CONSTRUCTION OF THE TUNNEL SECTIONS

### 8.3.1. Tunnel Section A (see Appendices 2-19 and 2-20)

It is expected that in moberg tunnel section A (23.4 m2, R = 2.9 m) can be driven full-face. In good conditions, the 8 cm thick mesh/shotcrete lining should be sufficient, this being the absolute minimum lining thickness proposed in moberg formations.

In poorer rock conditions, linings type II and III will be constructed using Bernold plates, and in loose rock conditions, poling-plate advance (lining type IV) will be employed.

As previously mentioned, the horseshoe sections have been selected for structural reasons with a slightly curved, rather than a flat invert. In order, however, to facilitate the movement of vehicles, excavation can initially be taken only to the level of the vault abutments, the curved sections then being excavated prior to concreting of the invert.

If in particular rock condition full-face advance proves difficult, partial heading could be substituted, with the section being excavated initially above the horizontal diameter, a height of about 3 metres.

### 8.3.2. Tunnel Section B (see Appendices 2-14 and 2-21)

Tunnel section B (52.2 m2, R = 4.30 m) is already of a size which in moberg will probably necessitate partial heading, such as the Belgian construction method detailed in Appendix 2-21. The stages in this case would be as follows for sections BII and BIII in stable rock conditions:

- I Excavation of the upper section
- II Erection of the Bernold sheets and concreting of the vault, if necessary with enlarged abutments.
- III Excavation of the lower centre section.
- IV Excavation of the side benches by smooth blasting.
- V Concreting of the drainage duct and invert.
- VI Erection of the Bernold plates and concreting of the side walls.
- VII Guniting of the vault.

In friable, less resistant rock, the procedure would have to be modified as follows:

- I Excavation of the upper section.
- II Erection of the Bernold sheets, concreting of the vault and if necessary tying-back of the enlarged vault abutments with rock anchors.
- III Excavation of the side walls, lower centre section.
- IV Excavation of the side benches step by step by smooth blasting.
- V Concreting of intermediate supporting pillars and of the side walls.
- VI Concreting of the drainage duct and invert.
- VII Guniting of the vault.

In these cases, the upper section will be excavated as a pilot tunnel and its length must be decided on the basis of the rock conditions encountered. Excavation of the upper section should not, however, be allowed to proceed too far in advance of the remaining excavation, as it is important that the effect of the complete tunnel lining, including the invert, be obtained as soon as possible; only in this way can wholly stable and safe conditions be assured.

The lining type can of course be varied depending on the rock encountered, but for a section of this size, the type I lining of shotcrete and mesh will only be allowed by extremely good rock conditions. It can therefore be assumed that in moberg only sections BII and BIII need be considered, with the sole reservation that under certain conditions poling-plate driving (section BIV) might be possible; this is discussed in detail in the following section.

The stability of the upper vault abutments during excavation of the lower side walls must be thoroughly investigated and under certain conditions tying-back of the abutments with rock anchors may be necessary. In less resistant rock the side benches can only be excavated by the intermediate construction of short lengths of the lower lining which are formed as retaining walls and which support the vault lining whilst the remainder of the side benches is excavated.

In order to accurately define the construction procedure for tunnel section B, extensive prospection and study of the moberg formations will be necessary.

### 8.3.3. Tunnel Section C (see Appendices 2-15 and 2-22)

This is a very large tunnel section (75.6 m2, max. height 9.0m, max.width 10.4m) and excavation by partial advance, e.g. by the Belgian method, will be necessary.

As the excavation procedures, working phases and other considerations would, depending on rock conditions, be the same as already described for Section B, it is not necessary to repeat them here. However, the construction of a tunnel of this size by any method would only be feasible in very good rock conditions, and, therefore, it would be necessary to make extensive investigations and studies before deciding on such a tunnel instead of an alternative solution such as, for instance, two smaller parallel tunnels.

## 8.4. TUNNEL DRIVING WITH STEEL POLING PLATES
(see Appendix 2-23)

For certain moberg formations, for example the loosly cemented breccia of the moraine type, it is no longer possible to talk of rock tunneling, but rather of earth tunneling. In such very loose formations, which can be compared perhaps with gravely sand or moraine, it will probably be necessary to drive the tunnel using steel poling-plates. This method, which can be used in conjunction with the Bernold system, as described in the Appendix Documentation, makes use of poling-plates, or lances, which are of box-section (60-150 mm x 250-500 mm) and from 4 to 7 metres long. Steel guide arches are erected in the tunnel, behind which the poling plates are placed around the circumference of the vault; the plates interlock in such a way that they can take up the curve of the vault but the connection between adjacent plates is such as to allow one plate to be driven forward at a time,whilst at the same time forming a sufficiently tight closure to prevent material passing between the plates. The plates which have pointed tips, are driven forward one by one by hydraulic rams acting against the guide arches, and mucking-out at the fact is carried out by hand or using conventional diggers. The procedure is clearly illustrated in the Appendix Documentation.

The Bernold plates are erected as previously described against supporting arches (not the poling-plate guide arches), and in front of the rear sections (tails) of the poling plates; the concrete backfilling being therefore immediately placed between the Bernold sheets and the poling plates. With continuing advance of the latter, a void, the width of the poling plate (about 8 cm) is created between the concrete backfill and the rock which must be continuously filled with pumped mortar.

Depending on the behaviour of the rock material and therefore only after adequate investigations, poling plates can be used to drive the invert of the tunnel section. Use of Bernold plates in the invert should, however, be avoided, but, as previously mentioned in section 3 of chapter 7, pre-cast concrete elements can be placed to form the centre invert section with the remaining area being concreted normally.

Application of the poling plate method to section A (23.4 m2, referred to as section AIV) would be convenient. For section BIV, however, (52.2 m2) the procedure would be more complicated and if required over a long length of tunnel would probably not be as suitable as the use of a shield. It is thus clear that the success of poling-plate advance for section C (75.6 m2) is very questionable and, in the absence of more accurate geological data this can not be accurately assessed. A further difficulty in such loose ground is maintaining the stability of the face itself, this obviously becomes more difficult as the tunnel section increases. Even with partial advance using poling plates (e.g. by the Belgian method) breasting to support the face would almost certainly be necessary for sections greater than AIV (see Appendix Documentation).

It can thus be seen that there exist definite technical and economic limitations to the use of the poling-plate method of tunneling.

# 9 - C O N S T R U C T I O N  M E T H O D S IN G R O U N D  W A T E R

## 9.1. GENERAL

As explained in chapter 4, ground water must be expected when tunneling in moberg which, depending on the rock conditions and the type of source, can appear in different ways and have various effects on the tunneling procedure. In certain locations it is possible that work will have to proceed about 50 metres below the ground water table. This chapter consists therefore of a description of the various methods available for coping with ground water when tunneling by the procedures described in chapter 8; these methods being summarised in Appendix 2-24. In these cases it is particularly important that the ground water inflow be diverted or stopped as soon as possible in order to prevent the freshly placed shotcrete lining or concrete backfill behind the steel supports from being washed out. In any areas where ground water is expected it is recommended that prospection drillings be made and installed with piezameters before excavation is commenced, in this way flow conditions can be observed, necessary precautions taken, and unpleasant surprises avoided. During tunneling itself the execution of pilot drillings is recommended at the face as shown in Appendix 2-06, to give warning of changes in ground water flow.

## 9.2. THE OBERHASLI METHOD (METHOD I)

This method of dealing with local, concentrated inflows of water, such as for instance those issuing from cracks when tunneling beneath a suspended water table, is carried out immediately following excavation and prior to the erection of the supporting sheets. The individual inflows are diverted directly into the drainage duct in the invert by means of plastic gutters of semi-circular section which are fixed to the rock face with a covering of rapid-hardening cement mortar, containing for instance Sika 4a binder or similar additive. This mortar can either be applied by hand or sprayed, and the method is detailed in the Appendix Documentation. The steel supports or shotcrete/mesh lining can be erected immediately the inflows have been diverted, but the Oberhasli method cannot of course be used with poling-plate advance.

## 9.3. WATERPROOFING METHODS USING PLASTIC SHEETING

### 9.3.1. Method II, Appendix 2-25

When ground water appears as a strong rain pouring from the whole tunnel section (see Photograph No.16), a waterproof plastic membrane,reinforced with mesh, must be employed, such as 'Guniplast' or similar (see Appendix Documentation). In these conditions, construction proceeds in the following stages:

1. Excavation as normal.
2. Supported on steel fitting arches steel,mesh rock security mats are erected to which the plastic membrane is attached with light steel L-angle sections and Sika mortar. The security mats protect the membrane against falling rock.
3. Erection of the Bernold sheets behind separate arches and backfilling with concrete of the space between these sheets and the plastic membrane. The steel security mats prevent the membrane from being damaged or pushed against the rock face during backfilling. Small diameter plastic tubes, temporarily closed, are embedded in this concrete.
4. After hardening of the concrete backfill, the void between the membrane and rock is filled by pumping mortar through the plastic tubes. Finally, these tubes are extended by drilling through the hard mortar and into the rock mass to form drainage ducts which prevent the build-up of water pressure behind the lining.

### 9.3.2. Method III, Appendix 2-26

In poorer rock quality corresponding to lining type III, the rock security mats will be replaced by a second layer of Bernold sheets which will protect the plastic membrane against falling rock. Otherwise, this method is identical with method II.

### 9.3.3. Method IV, Appendix 2-27

This is a variation of method II which can be used when poling-plate advance is necessary. The plastic membrane is laid directly against the poling-plates and concreting carried out as normal behind Bernold sheets. After pumping mortar into the space left by the poling plates, the drainage holes will be bored and filled with a filter material to prevent the loose rock material from being washed out. This method could be used for examble in water-bearing looslycemented breccias.

The dimensions of the three cross-sections with the modified linings necessary to deal with ground water are given in Appendix 2-28.

In order to give an idea of how tunnel section A could appear in water-bearing moberg, reference is again made to photograph No.16 which shows a tunnel constructed in the Swiss Alps through very pervious dolomite. The rock itself was very solid and required no support, but when below the water table extensive water-proofing work was necessary, following immediately behind the tunnel excavation. In this case, for reasons both of cost and programming, 'Guniplast' sheeting was used.

## 9.4. INNER CONCRETE LINING

It is conceivable that driving a tunnel through water-bearing moberg could produce a lowering of the ground water table. However, if a completely water-tight lining must be constructed, or even if subsequent grouting is necessary behind the lining, it is possible that the water level around the bore could again rise with a consequent build-up of pressure on the lining. In such circumstances, an inner concrete lining would have to be constructed to withstand this pressure, as shown in Appendix 2-29.

# 10 - SPECIAL CONSTRUCTION PROBLEMS

The study of the moberg formations has indicated that certain construction problems could be encountered which are not normally expected when tunneling in more solid and homogeneous rock; these problems are briefly discussed in this chapter.

## 10.1. DRILLING

### 10.1.1. General

Drilling in moberg will pose difficult problems. The heterogeneity of the formations can be clearly seen from photographs Nos.1-10, and the consequent variations in hardness between pillow lava on one hand and loosly consolidated breccia on the other, with always the possibility of intervening basalt veins, are hardly conducive to trouble-free drilling. Because of these widely differing strata through which it will be necessary to drill, wide variations in the stability of the boreholes must be expected and allowed for; this was seen during the drillings carried out for grouting trials at Sigalda where for instance with Tricone bits it was necessary to grout up the holes every three metres. These variable strata also lead to wide differences in drilling speeds as is well illustrated in Appendix 2-07, but with, however a low average speed using Tricone bits of only 3.75 metres per hour. For these reasons, it is necessary to carefully consider drilling procedures in moberg and to always use the correct, and in some cases special equipment. In the table 'Technical characteristics of moberg' in chapter 6, section 8, a preliminary general forecast is given of the drillability of moberg formations.

### 10.1.2. Percussion drilling

For pure percussion drilling, it is essential that the rock be sufficiently hard to ensure adequate rebound of the drill bit, otherwise the drill steel will become progressively embedded and stuck in the hole and will be extremely difficult to extract. It is therefore clear that this method can only be used for drilling in moberg formations in the rare cases when particularly hard and massive rock is encountered. Even with pillow lava however, although the rock itself is of hardness which should be conducive to satisfactory percussion drilling, difficulties will occur when the bit passes through the voids between pillows and impringes on the surface of the following pillow. When this surface is inclined to the drilling direction the bit will have difficulty in biting and there will be a tendancy for the drill steel to be deflected.

For these reasons, pure percussion drilling will not be practical in most moberg formations and can be excluded from further discussion.

### 10.1.3. Rotary percussion drilling

Rotary percussion drilling (RP drilling) provides a method much better able to cope with variable rock conditions. The drill tool can be rotated independantly of the percussion effect and at speeds which can be adjusted as required. Depending on rock conditions, drilling can be carried out by either rotation without percussion, percussion without rotation, or by similtaneous rotation and percussion. Two examples of RPD drills, the Atlas Copco 90 ED and the Joy drill are described in the Appendix Documentation.

The selection of the correct drilling methods and equipment will be important when working in moberg if adequate progress at minimum cost is to be maintained. In particular, the drill steels and bits must be carefully chosen and this choice should be made only on the basis of prior tests and trials.

Plain chisel-edged drill bits will not be suitable for moberg and four-point bits, or even at times special soft rock bits, must be used. When excavating in heavily fissured moberg there may be a tendancy, even with RP drilling, for the drill steel to become jammed due to the partial collapse of the hole behind the bit;for this reason the use of so-called 'Retrac' bits should be considered. These bits have backward facing cutters behind the main bit and are therefore able, when rotated in the reverse direction, to drill themselves free.

It must also be decided whether integral drill steels or extension steels are to be used, and this will depend mainly on the required depth and diameter. The following table gives some characteristics dimensions of integral and extension steels with four-point bits.

| Drill steel diameter | Max.effective length in mm | Bit edge diameter in mm | Flushing duct diameter in mm |
|---|---|---|---|
| Integral steels: | | | |
| 7/8" (22.2mm) | 4600 | 38 | 6.7 |
| 1 " (25.4mm) | 4600 | 38 | 7.6 |
| Extension steels: | | | |
| 7/8" (22.2mm) | optional | 35 - 38 | 6.7 |
| 1 " (25.4mm) | optional | 38 - 45 | 8.8 |
| 1 1/4" (31.8mm) | optional | 48 - 51 | 9.4 |
| | | 51 - 64 * | |
| 1 1/2" (38.1mm) | optional | 64 - 102 | 12.5 |
| | | 64 - 89 * | |
| 1 3/4"(44.5mm) | optional | 76 - 115 | 14.0 |
| | | 76 - 115 * | |

* Bit edge dimensions for 'Retrac' bits which are only available for steel diameters of 1 1/4 " and greater.

It is expected that because of the generally relatively low consumption of explosive anticipated for moberg, that 7/8" to 1 1/4" diameter steels will be adequate. However, where 'Retrac' bits must be used, the steel size will probably have to be increased, as will probably also be the case when drilling in pillow lava to reduce drill deflection between pillows.

It is known from experience gained at Burfell that when drilling in volcanic formations, a fine clayish drilling mud has a tendancy to form which can plug the flushing holes, thus preventing the flushing out of the drilling and causing the drill steel to stick. This could also happen in moberg and it is therefore necessary to use the highest possible water pressure when drilling together with steels having the maximum possible flushing duct diameter.

As previously mentioned, difficulties may be encountered with percussion drilling in pillow lava due to the voids and fissures, and this problem would only be partially overcome by RP drilling. In these conditions, therefore, large diameter, stiffer steels should be used, or alternatively guide rods or special coupling sleeves could be employed, although these latter do have the disadvantage of limiting the drill-hole diameter. Drilling tests in such conditions are thus absolutely necessary.

10.1.4. Overburden Drilling Method

If, due to collapse of the drillhole walls, neither of the drilling methods described above prove successful, cased drilling will be necessary. This could well be the case with breccious pillow lava and loosly cemented breccias which may probably turn out to be practically undrillable by convential methods.

The Overburden Drilling method (OD-Method)which was developed by Atlas Copco and which was used to drill the lava formations during channel excavation at Burfell, could in such circumstances be employed. Basically a extension of the RP- method, the main drilling steels are sunk within drilling cases and continuously flushed with high-pressure water. The method is described in Appendix Documentation. Such a method will have to be used under certain circumstances when tunneling in moberg, particularly when wholly unexpected geological conditions are encountered in which drilling cannot be avoided. The method will not be cheap, and does have the further disadvantage that even the smallest diameter which can be so bored (88-92mm) is almost too large for normal tunnel driving, but it can be assumed that when tunneling it will only be rarely necessary to employ the OD-Method.

In loose rock, for instance loosly cemented breccia,where blasting is not necessary, drilling will of course not normally be required. If, however, isolated veins exist, for instance of basalt, which can only be removed with blasting, then overburden drilling would provide a possible, albeit expensive, method of drilling for the placing of the charges.

### 10.1.5. Advance per Round and Drilling Patterns.

Although also influenced by the capacity of the drilling and mucking-out equipment, the advance per round fired depends mainly on the tunnel section, rock characteristics and driving method.

As a general rule, Langefors and Kihlström*give the advance per round as 40 to 70% of the maximum tunnel width (B) which, taking an average value for moberg of 50%, gives the following results:

| Section | Tunnel section m2 | Max.width (B) m | Advance per round, m |
|---|---|---|---|
| A | 23.4. | 5.8 | 2.9 |
| B | 52.2 | 8.6 | 4.3 |
| C | 75.6 | 10.4 | 5.2 |

It is clear, however, that in moberg formations the round advance will be limited either by the maximum depth which can be drilled or by the maximum unsupported clear span for each rock type. This can be seen from the following figures taken from chapter 6:

| Rock type | Max.drilling depth (m) | Clear unsupported span (m) ** |
|---|---|---|
| Pillow lava | 2 - 3 | 2 |
| Brecciated pillow lava | 1 - 2 | 1 |
| Breccia | 2 - 4 | 3 |
| Loosly cemented breccia | *** | 0.6 |
| Tuff | 2 - 4 | 3 |

** According to the principle of Lauffers' method
*** Probably undrillable by normal methods, max.depth by OD-Method cannot be estimated.

This shows that in fact for moberg formations it is the structural properties dictating the unsupported clear span which will generally govern the selection of the advance per round.

Accurate drilling in moberg formations will be difficult or very expensive,in particular because of the voids which can exist in for example pillow lava formations, and therefore it will be necessary to arrange the drillings in a wedge pattern in the tunnel face, as it will certainly not be possible to drill parallel holes. The

* The Modern Technique of Rock Blasting - Upsala, Sweden, 1967

number of drillholes necessary for each round depends obviously on the area of cross-section of the tunnel as well as on the loosening characteristics of the rock. According to the diagram prepared by Dr. Maidl (Schweizerische Bauzeitung, Issue No 7, 1972) on the basis of his study of numerous tunnels, the following values are obtained, for example for section AIII ($S = 31.5$ m2):

| Rock classification group | Number of holes (n) | Number of holes per m2 of face |
|---|---|---|
| 1. Easy to loosen | $n = 25.1 + 0.77S = 49$ | 1.55 |
| 2. Moderately difficult to loosen | $n = 30.9 + 1.00S = 62$ | 1.97 |
| 3. Difficult to loosen | $n = 37.6 + 1.40S = 82$ | 2.60 |

Moberg formations with low seismic values and which are generally rippable can be considered to belong to group 1., but pillow lava itself should probably be classified in group 2.

These values, which were actually recorded on site, are in general higher than the calculated values (see Diagram 4 in article referred to).

### 10.1.6 Drilling Equipment

A large range of drilling equipment is available today for dealing with difficult problems and conditions and it is therefore important that extensive, comparative drilling tests be carried out in the various moberg formations in order that the most suitable and economical equipment can be selected for a particular tunneling project.

A self-propelled rubber-tyred Jumbo would be very suitable for tunneling in moberg, as it could carry both conventional percussion and rotary percussion drills as well as, when necessary, overburden drilling equipment.

In rock where drilling proves to be impossible or very expensive, use could be made of a hydraulic excavator with a jack-hammer fitted in place of its bucket to break up the rock at the face; this rock would then be loaded into trucks by a separate shovel. In certain tunnels a jumbo fitted with several such hammers could be used. An example of such a machine, the Secoma Type 225, is illustrated in the Appendix Documentation. This was used in 1967 for construction of a tunnel beneath the centre of Paris where blasting was not allowed.

A newly-developed tunneling machine known as the "Roadheader" could also be considered for working in moberg. This has a circular cutting head, mounted on a telescopic boom, which traverses the tunnel face, the rock material so removed falling onto a conveyor belt which carries it to a waiting truck standing behind the machine. Such a machine manufactured by Anderson Mavor Ltd. of Motherwell, Scotland, is also illustrated in the Appendix Documentation.(see also ch.11.5.)

## 10.2. BLASTING TECHNIQUES

### 10.2.1. Conventional techniques

In addition to the problems of drilling in moberg, complications must also be expected with the blasting procedure itself. Normal methods of blasting must be modified and special precautions taken, to deal with the following two main difficulties.

Firstly, the charging of the drill holes will be impeded should the holes be partially blocked either by protruding fragments of, for instance, basalt or by rock fragments which have fallen into the hole. In such cases the normal rigid gelignite cartridges may get jammed in the hole and it may be impossible to push them to the full depth.

The second difficulty arises from the voids and cracks which often exist in moberg formations, in particular for example in pillow lava. These voids can prevent good contact between gelignite and rock and in addition, the air pockets can further reduce the effect of the explosion.

### 10.2.2. The use of slurry explosive

Many of the problems of blasting in moberg could be avoided by the use of semi-liquid explosive packed in soft polythene cartridges. The cartridges are slit open before charging so that the jelly-like explosive can flow into close contact with the walls of the hole and can also fill any cracks and cavities in the rock mass, thereby increasing the effectiveness of the explosion.

Ireco slurry is such a semi-liquid explosive. Originally manufactured in the U.S.A. several years ago, it has been developed in Europe for underground blasting by Haniel Ltd. who market it under the name "Gotthardit slurry". The jellified slurry can be supplied in varying strength with densities of between 0.7 and 1.4 and therefore adapts well to variable rock conditions, and its consistency is such that it can spread to fill a drill hole or void without flowing away. The cartridges can be supplied in varying lengths and with diameters from 25 mm and the slurry itself is completely water impervious. In terms of both explosive power and cost Gotthardit is the equal of normal gelignite.

### 10.2.3. Consumption of explosive.

It is at present difficult to estimate the consumption of explosive when blasting in moberg due to the shortage of geological data and information on previous experience. Some conclusions can, however, be drawn from the seismic velocity values which are available, from the limited records of projects already constructed in Iceland, as well as from personal evaluation following examination of moberg outcrops in the various project areas.

The generally low seismic velocities in moberg formations lead one to expect low consumption of explosive as compared with more solid rock masses such as granite, gneiss or limestone. Pillow lava is certainly a hard rock which might require more explosive, but because of its characteristic formation, the blasting might only be necessary to loosen the bond between the pillows. Breccia and tuff on the other hand, although more homogeneous, are less resistant than pillow lava, and, therefore, lower explosive consumption can be expected; this is also suggested by the 'rippability' of these formations. The relatively large tunnel sections to be driven will also result in low specific consumption values (kg/m3); the values will decrease as the cross-section excavated increases, although this reduction will be limited by partial advance .

The data tabulated below has been collected from projects already constructed in Iceland:

| Tunnel | Rock type | Excavated cross-section,m2 | Number of drill holes | Specific drilling length m/m3 ex-cavated | Depth of a round | Specific consumption of explosive kg/m3 |
|---|---|---|---|---|---|---|
| Burfell: Pressure tunnel | Basalt | 89 | - | - | - | 0.8-1.0 |
| Sog River: Tailrace tunnel | Basalt | 64 | 80 | 1.36 | 3.6 | 0.82 * |
| Headrace tunnel | Moberg | 27 | 54 | 2.05 | 2.0 | Not known |
| Laxa III: Tailrace tunnel | Basalt | 40 ** | - | - | 3.5 | 1.3 *** |

* Taken from a report on the Sog River scheme
** Partial advance
*** Obtained verbally

For full face excavation in solid homogeneous rock, Wallin * indicates the following explosive requirements:

| Excavated cross section, m2 | 10 | 20 | 40 | 60 | 80 | 100 |
|---|---|---|---|---|---|---|
| Specific consumption of explosive kg/m3 | 2.5 | 1.8 | 1.1 | 1.0 | 0.8 | 0.7 |
| Specific drilling length required m/m2 excavation | 4.0 | 2.8 | 1.8 | 1.4 | 1.2 | 1.0 |
| Number of drillings | 32 | 42 | 60 | 75 | 84 | 100 |

Consumption values for moberg will almost certainly not exceed those for basalt and will very probably be smaller. The following values have been deduced from those quoted for the Sog River Project, allowance having been made for the cross-sectional area excavated.

| Tunnel section | | A | B | C |
|---|---|---|---|---|
| Approximate excavated area | m2 | 30 | 60 | 80 |
| Explosive consumption acc.to Wallin | kg/m3 | 1.3 | 1.0 | 0.8 |
| Relative consumption acc. to Wallin | % | 130 | 100 | 80 |
| Explosive consumption at Sog River | kg/m3 | - | 0.82 | - |
| Deduced values | kg/m3 | 1.04 | - | 0.66 |
| Estimated consumption in moberg | kg/m3 | 1.05 | 0.80 | 0.65 |

These are rough guiding values for specific consumption in moberg for full-face tunneling. In poorer quality rock the values would be reduced and for partial advance, corresponding values can be deduced.

* "Tunnelbau" Szechy; Springer Verlag, Vienna, 1969.

## 10.3. MUCKING-OUT EQUIPMENT

Definite proposals concerning mucking-out equipment cannot at this stage be made but it is possible to discuss the question in general terms in the light of the excavation methods proposed.

The basic decision which has to be made is whether rail-trucks or tracked and rubber-tyred vehicles are to be used. This choice must be made by taking into account technical factors such as tunnel size, slope and length, geological conditions, construction method, ventilation facilities, etc, but is ultimately an economic decision. In this case, and in particular because of the uncertain and variable nature of the rock, tracked and wheeled shovel loaders working in conjunction with heavy-duty trucks and dumpers, will offer the most convenient and flexible method of mucking-out. The only exception to this would be in connection with the use of tunneling machines. In this case the continuous excavation process which is possible can be more adequately adapted to by mucking-out trains. This is more fully discussed in chapter 11.

Because of the restricted space, both when driving full-face for section A as well as by partial-advance for sections B and C, the movement of the loaders will be limited; in particular side dump shovels must be fitted as the machines will not be able to turn. For the same reason, overhead (rocker) shovel loaders could be employed. Conventional shovel loaders have the advantage that they can also be used for moving forward the supporting arches as these sit easily on the raised bucket, although it is possible that special mobile lifting equipment might be used for moving these arches. Overhead loaders require less space than the conventional shovel loaders but are consequently of smaller capacity. Dimensions of several well-known models are given below for comparative purposes, although of course very many different makes of machines are now available.

| Make and Model | Bucket capacity m3 | Necessary overhead clearance, m | Horse power | Capacity * m3/hour |
|---|---|---|---|---|
| Overhead (rocker) loaders: | | | | |
| Eimco 105 | 1.15 | 4.1 | 130 | 100 |
| Atlas Copco Cavo 520 ** | 0.60 | 3.4 | 25 | 85 |
| Shovel loaders: | | | | |
| Caterpillar 955 Traxcavator | 1.34 | 4.8 | 115 | 100 |
| 930 Tyre loader | 1.34 | 4.5 | 130 | 90 |
| Salzgitter HL 583 ** | 1.00 | 4.0 | 48 | 80 |

* General values for loose material which are dependant on working conditions

** Compressed air machines

An example of efficient modern mucking-out technique which could be applied when tunneling in moberg is described in the March/April 1972 issue of the English magazine 'Tunnels and Tunneling'. For the construction of a 30 m2 tunnel in hard rock for the Skjomen hydro-electric project in northern Norway, mucking-out was performed by a Cat-955 with side tipping shovel loading Kiruna K-162 trucks of 17 m3 heaped capacity.

## 10.4. CONCRETE PUMPS

The three proposed tunnel linings require the placing of the following types of concrete:

| | |
|---|---|
| Type I | Shotcrete, in situ invert concrete |
| Types II and III | Concrete backfill,in situ invert concrete and gunite |
| Type IV | Concrete backfill,pumped mortar,in situ concrete at sides of invert, and gunite |

Because of the variable nature of the rock, and consequent frequent changes in construction method, suitable concrete pumps will be necessary which can be quickly set up wherever required and which require no special installations. The concrete can be pumped through portable flexible hoses to the points of application.

For backfilling with moist, relatively stiff concrete, a pump such as the Spirocret S-2000 could be used (see Appendix Documentation). This machine is suitable for pumping the mix behind the Bernold sheets as well as through the perforations in the sheets when normal backfilling is not possible because of protruding rock or rock falls behind the sheets.

For applying gunite and shotcrete, spraying machines of the type represented by the Aliva series in the Appendix Documentation could be employed. The Aliva-300 model gave excellent results during tunnel construction for the El Toro project in Chile with concrete containing an extremely abrasive sand of lava origin. For pumping mortar behind the poling plates for lining type IV, the Spribag SG-15 machine could be used; this is similar to but smaller than the Aliva 300.

It is possible to use sprayed mortar with additive to fix the plastic Oberhasli ducts to the rock face during waterproofing work (see chapter 9), thus avoiding much hand work. Small machines such as the Alivamat-50 can be used for this purpose.

## 10.5. ROCK BOLTING SYSTEMS

### 10.5.1. General

In modern tunnel construction, the use of rock anchors is now accepted whenever the work is menaced by the danger of imminent rock fall or wherever it is necessary to tie-back concrete or other structures to the rock mass. Rock anchoring must therefore be allowed for when tunneling in moberg but, because of the pronounced and somewhat unusual characteristics of the rock, not all systems will be suitable. In this section, the available types of anchor are described and their suitability in moberg discussed; it must, however, be stressed that extensive trials are still necessary to confirm the adequacy or otherwise of these methods and to select the most suitable.

### 10.5.2. Cone bolts

These very frequently used and simple bolts depend for their grip with the rock on an expandable wedge or cone, and therefore are most effective in hard and massive rock. In fissured, broken up and relatively soft moberg, the suitability of this system is questionable.

### 10.5.3. 'Perfo' bolts

This well-tried, adaptable and relatively inexpensive system makes use of a split perforated tube which is filled with mortar and introduced into the drill hole. A ribbed bolt is then pushed into the hole and the mortar is thereby squeezed out through the perforations and into close contact with the rock. Good bond is thus assured and the bolt is furthermore protected against corrosion. The system is particularly useful in rising drillings where it would otherwise be difficult to introduce the mortar. Further details of these bolts are given in the Appendix Documentation.

The Perfo-method would be very suitable in moberg as the hole is filled for its whole length with mortar and good contact between bolt and rock is assured. Equally, however, the mortar tends not to flow into voids and fissures and this would be an advantage in pillow lava. When it is necessary to reduce the hardening time, the mortar can be replaced with a synthetic resign.

### 10.5.4. In situ anchors

In rock where drilling is only completed with difficulty it is doubtful whether it will be possible to remove the drill steel and insert an anchor into the hole; a system which utilises the drill steel as anchor must therefore be considered. In the Sanvik system shown in Appendix 2-32, the drilling bit itself forms the anchor and is fixed by means of grouting through the flushing duct of the drill steel. Normal drilling equipment can be used and various lengths of steel are available. This system was developed and first used in Sweden and was fully described in the Swedish magazine 'Byggmästaren' (No.11, 1964).

A variation of this method makes use of a tube of about 40 mm diameter with a drilling bit screwed into it, this is drilled and then grouted as shown in Appendix 2-30. This method is useful for rising holes and, although somewhat complicated and expensive, is effective in unstable drillholes.

### 10.5.5. Cemented bolts and anchors (see Appendices 2-30 and 2-31)

When stable holes can be drilled without difficulty in moberg, cemented anchors can be used and provide a relatively simple alternative to Perfo-bolts. Holes which are inclined downwards are simply filled with cement grout into which the ribbed anchor is introduced - after hardening of the grout the anchor plate is fitted and the anchor force applied by screwing a nut on to the anchor. In rising holes, a sleeve is fitted to prevent the grout, which in this case must be pumped into the hole, from flowing out. An advantage of this system is that the anchor is continuously threaded and that variations in length can be easily coped with when screwing up the nut. Accelerators can be added to the cement to reduce the delay in tensioning the anchor.

10.5.6. Prestressed rock anchors

Prestressed anchors are much used today in underground work and would in the present case be ideal for the tying-back of the abutments of the vault which will probably be necessary during partial advance of tunnel sections B and C. The anchor cable is about 15 mm in diameter and 4-6 metres long, and can be tensioned to 12-15 tons. The cable is introduced into the drillhole within a polythene tube of about 30mm diameter which is also partly filled with a synthetic resin mortar such as 'Colmasin'. When the tube is withdrawn, an anchorage of not less than about 70 cm length is formed at the extremity of the hole and after about 24 hours the anchor can be initially tensioned to full load. For the construction of such anchors, a stable drillhole is necessary, and if casing must be used to ensure this it is important that this be partially withdrawn with the polythene tube if a perfect bond with the rock is to be obtained.

## 10.6. GROUTING

In the EWI/Virkir report on the Sigalda grouting tests (November 1971), initial assessments were given of the effectiveness of grouting in moberg formations. The tests confirmed that the formations could be sealed without difficulty but that pressures of up to 40 kg/cm2 would have to be used. Since however these tests were carried out for the purpose of investigating the feasibility of the grout curtain of Sigalda Dam, only general indications rather than accurate conclusions can be drawn form them concerning the effectiveness of grouting during tunnel construction.

In the descriptions of the various tunnel construction methods given in Chapter 8, no mention is made of grouting for the very good reason that these methods should allow excavation to be carried out satisfactorily, even in the most difficult formations, without recourse to previous consolidation grouting.

When forming concrete linings of types II and III, the placing and vibration of the concrete in the crown of the vault could prove troublesome, although with correct use of the concrete pumps at sufficient pressure, closure of the vault should be possible. Any cavities and voids which however appear could be filled by systematic mortar grouting although, due to the thinness of the lining, the grouting pressure should never exceed 2 kg/cm2 (see section 6, chapter 7). The same method could also be used for the mortar backfilling during construction of lining type A IV.

Equally high pressure grouting need not be expected as consolidation of the moberg formations should not be necessary to resist the loading of the free-flow tunnel. If despite all, however, it should prove necessary to grout either to additionally seal or consolidate the mountain mass, this should only proceed after completion of adequate static calculations and exhaustive tests.

# 11 - TUNNELING MACHINES

## 11.1. GENERAL

The development of tunneling machines has progressed rapidly in recent years and the number of mechanically bored tunnels throughout the world continues to grow. Improvements in design have resulted in a lowering of costs together with an increase in performance and this means more and more often that the use of a machine provides a competitive alternative to conventional excavation methods. It is therefore necessary in this study that the various types of tunneling machines be discussed with respect to the moberg formations in Iceland, and that their advantages and disadvantages as compared with normal procedures be considered.

The advantages of these machines are clear. Tunneling proceeds more rapidly and in safer conditions, overbreak and temporary supporting works are reduced, and pre-cast concrete lining elements can be erected quickly and easily immediately behind the face. The most valuable advantage however is the avoidance of blasting, a procedure which is still basically dangerous and often unpredictable and which can submit the rock mass to heavy stresses and shocks as well as causing excessive loosening and cracking. This avoidance of the use of explosive would be particularly valuable in moberg, not least because of the consequent reduction in the drilling requirements.

There exist however limitations to the employment of these machines which must also be taking in account. Design improvements should ensure a continuing reduction in the technical problems of mechanical tunnel boring, but one basic economic consideration will remain, that is that a tunneling machine is a very expensive piece of equipment and that its use, despite the smaller labour force that this permits, will only be economic for the excavation of a considerable length of tunnel. If a single tunnel of sufficient length is planned all well and good, but if it is proposed to bore a number of tunnels it must be remembered at the planning stage that it is only rarely possible to change the diameter of the machine between one job and another, and that therefore a series of adduction tunnels for instance must all be designed with the same diameter. It must also be pointed out that the use of a machine to bore several tunnels for a single scheme may well not in the long run be quicker than excavation by conventional methods, since the driving of all tunnels concurrently is ruled out, and even for a single long tunnel, the time saving may be illusory as simultaneous advance from intermediate adits is not possible.

Tunneling machines are less adaptable when widely varying rock conditions are encountered since, although cutting tools can be exchanged, there is no question of changing the machine's basic excavation method. In very wet conditions, it is important that the insulation of the electric circuits be perfect as failures can be both dangerous and expensive. A problem with certain machines in relatively weak rock is posed by the pressure pads which act against the tunnel wall to hold the body of the machine in place; if the rock is too weak and unless it is possible to advance the lining sufficiently to jack against that, it may not be possible to adequately support the machine or to obtain the required forward thrust.

An important technical factor, however, which up to now has limited the use of machines for tunneling has been the hardness of the rock, but recent advances in the design of cutters and tools mean that today tunnels can be mechanically bored in all but the very hardest strata. This question of rock hardness is discussed in more detail in section 4 of this chapter.

Development of tunneling machines has reached such a stage that, despite the limitations referred to, their employment in Iceland for the driving of tunnels in certain moberg formations can by no means be ruled out. In the following section therefore, the three circular tunnel sections which have been studied are described. Following this the two main types of machines, the soft rock shield digging machine and the pure boring machine with a rotating head are described, and their application in moberg is discussed.

## 11.2. TUNNEL SECTIONS

For mechanical tunnel driving, the following three circular cross-sections have been designed; these are detailed in Appendix 2-33,

| Section | Internal section | Internal diameter | Thickness of lining segments |
|---|---|---|---|
| A | 23.2 m2 | 5.50 m | 15 cm |
| B | 51.9 m2 | 8.20 m | 20 cm |
| C | 74.4 m2 | 9.80 m | 25 cm |

These sections were designed to the same principles as the horse-shoe sections given in Chapter 7. The ring will be formed by four pre-cast lining segments each about one metre wide and fitted as shown in the drawing; the invert segment being cast with a partly horizontal surface to facilitate movement of plant, as well as a drainage duct. The segments are erected after each advance of the driving face and immediately behind the machine, after which the space between the ring and the rock face is grouted with mortar.

The permissable loadings on the lining were calculated on the basis of the shear effect in semi-rigid shells for P300 concrete (cube crushing strength, $\beta_{W28}$ = 350 kg/cm2) without steel reinforcement. The following values were obtained, these incorporate a safety factor of 2.3 and no allowance was made for the grouted mortar layer between the lining segments and the rock.

| Section | Internal diameter | Permissable load, tons/m2 |
|---|---|---|
| A | 5.50 m | 42 |
| B | 8.20 m | 37 |
| C | 9.80 m | 38 |

If the drainage holes drilled through the lining into the rock do not serve to lower the ground water table and hence external water pressure, then a second, inner structural lining will be required.

## 11.3. DIGGER SHIELD MACHINES

As has been explained in Chapters 4 and 6, it is now certain that many moberg formations can be excavated by ripping. The limit for ripping with a CAT D-9 appears to be pillow lava formations which are not dominated by basaltic intrusions and, in the absence of actual ripping tests in pillow lava, this limit has been assumed to correspond to formations having a seismic velocity of about 2500 m/s. It must be pointed out however that such an assumption may give an over-optimistic view of the ripping characteristics of pillow lava, since, because of the voids and cracks in these formations, the seismic velocity actually measured could well be less than that in the pillows themselves. It can be added that field observations of outcrops tend to confirm these doubts as to whether pillow lava can be removed in this way. The definition of this upper limit must therefore be the object of further investigations and trials. The estimates of the suitability of the various moberg for ripping are summarised in the table of rock characteristics which follows Chapter 6, from which it can be seen that it is only in connection with pillow lavas that doubts remain.

The application of ripping to tunnel excavation is incorporated in the so-called digger shield machines, manufactured by Messrs.Robbins of Seattle. According to the representative of this firm in Switzerland these machines can work in any formation which can be ripped by a D-9, such a machine is shown in Appendix 2-34 from which it can be seen that it is fitted with a hydraulically powered arm which can carry various excavating tools and can move in the following four ways:

- backwards and forwards with a reach of about 3.30 metres
- upwards and downwards
- laterally
- rotationally at about 20 r.p.m.

This freedom of movement enables the operator to reach any point on the face and rip it in several ways, the axial movement serving mainly to pull back the spoil onto a conveyor belt beneath the machine. The upper half of the shield is fitted with hydraulically adjustable segments which serve to support an unstable tunnel breast. In addition to the ripping claw, in changing conditions the arm can carry any of the following tools; these can be exchanged in only 10-15 minutes:

- a toothed scraper plate (as shown in Appendix 2-34)
- a hydraulic grab
- a rotating scraper head
- a jack-hammer
- disc or cone bits

The shield extends back about 5 metres from the face and immediately behind this the lining elements are erected using special equipment fitted to the tail of the machine.

Larger diameter machines can be fitted with one or more intermediate working platforms each with their own supporting segments and fitted with at least one ripping arm; such a machine could certainly be considered for sections B and C (8.20 m and 9.80 m diameter respectively). Tunnels almost as big as this have in fact already been excavated by a MEMCO * digger shield for the Metropolitan Water District of Southern California. This 7.9 m diameter machine was successfully used to dig a total length of 8 km of tunnels and in the 4 km tunnel maintained an average advance of 34.4 m/day.

In the following table, details are given of the Robbins machine which was used to excavate the San Fernando Water Tunnel in California, this being the machine illustrated in Appendix 2-34.

| | |
|---|---|
| Model: | Robbins 221 S |
| Geology: | Sand, gravel, sandstone and large boulders |
| Project date: | Work started 6th January 1970 |
| Tunnel length: | 8840 metres |
| Progress up to 17.9.1970: | 4570 metres (approx.570 m/month) |
| Shield diameter: | 6.65 m (S = 34.6 m2) |
| Machine dimensions: | Shield approx. 5m long, tail 2.7 m |
| Horsepower: | 650 |
| Total weight: | 225 tons |
| Cost of machine: | $ 880,000 (in 1969) |
| Design capacity: | 500 tons/hour(equivalent to 4.57 m/hour) |
| Lining: | 4 concrete segments per ring, each 20 cm thick, 122 cm wide and 506 cm long, and weighing 3 tons. |
| Mucking-out method: | By train |

* Mining Equipment and Manufacturing Co., Racine, Wisconsin, U.S.A.

The following maximum advance rates were recorded with this machine:

| | |
|---|---|
| Advance per 7-hour shift | 31.7 m (4.53 m/hour) |
| Advance per 3-shift day | 84 m (28 m/shift, 4m/hour) |
| Advance per 5-day week | 298 m (19.9 m/shift, 2.84 m/hour) |

## 11.4 TUNNEL BORING MACHINES

### 11.4.1. General.

The second group of machines to be considered are the true tunnel borers or moles. These machines have an electrically-driven, rotating head the diameter of the tunnel which carries the cutters. The main frame of the machine is supported by hydraulic pads acting against the tunnel wall and the cutter head is thrust against the tunnel face by hydraulic rams acting against the frame. In comparison with the digger shields, these borers are essentially rock machines - and the continuing development of cutters is making rock hardness less and less of a limitation on their use - but models are now available which are able to cope with varying rock conditions and even faulted and fissured formations. The excavated material is passed by conveyor belt to the rear of the machine and, as previously mentioned, mucking-out trains are almost always used as they are able to cope more easily with the uninterrupted excavation process which tunneling machines make possible.

In discussing the use of tunnel borers in moberg formations, three main technical factors must be considered; these are the very changeable formations which may be encountered, the heavy ground water inflow which must be expected, and the hardness of certain of the formations, particularly the pillow lavas and basaltic intrusions.

For tunneling in soft or unstable rock conditions, borers are available which are fitted with a short shield behind the cutting head. This protects the machine from rock fall as the precast lining rings can be erected, by using lifting equipment on the machine frame, up to the rear of the shield. An important development of this shield principle is the incorporation of a compressed air bulkhead which enables excavation at the face to proceed under pressure to reduce water inflow whilst allowing the machine crew to work at normal pressure. Machines have also been built on which it was possible in soft ground to retract the cutter head into the shield.

For tunneling in ground water, it is imperative that the electrical circuits of the machine, which could be consuming several hundred kilowatts of power, be perfect. Pumping may well be necessary but is perfectly feasible with mechanical boring; for instance during the construction of the road tunnels under the River Mersey at Liverpool, England, using an 11 metre diameter Robbins mole, it was necessary in places to pump nearly 200 l/s of salt water. A difficulty which can arise in very wet conditions is the clogging up of the holes in the cutting head through which the excavated material passes; rock which when dry breaks into small, even particles can, when mixed with water, form large lumps of muck which are very difficult to convey back to the mucking-out trains. This problem can be reduced if allowance is made for it when designing the machine for a particular rock, otherwise it is a question of breaking up offending lumps with shovels and picks.

### 11.4.2. Rock hardness and cutter performance

The most important parts of any tunnel borer are the cutters. On their performance will depend the success or otherwise of the machine in a particular rock and the extent of wear of cutters will largely determine the day-to-day running costs. It is therefore worthwhile discussing the question of rock hardness and cutter performance, and above all considering whether limitations on mechanical tunneling are posed by the hardness of certain moberg formations.

Let it be said straight away, that there is no doubt that many moberg rocks would be removed without much trouble at all by cutters now available, and that any restriction on the use of borers in such formations would be the result of the other technical factors which have been discussed or, most likely, of simple economic considerations. Cutters have now been developed which are suitable for widely varying rock hardnesses, and it is a quick and easy procedure to change the cutters when new rock conditions are encountered.

It is however important to consider the maximum rock hardness which,all other things being equal, it is economical to bore, i.e. what minimum rate of progress and maximum cutter wear can be accepted. According to Robbins,the American manufacturer, the use of boring machines can be profitable in rock whose hardness corresponds to a compressive strength of at least 2100 kg/cm2, and this forecast is confirmed by the successful boring by a German Wirth machine of an inclined pressure shaft for the Emosson power project in Switzerland through gneiss having a compressive strength of 2200-2400 kg/cm2.

Robbins have defined degrees of hardness on the basis of compressive strength as shown below:

| Compressive strength psi | kg/cm2 | Hardness designation |
|---|---|---|
| ≤ 10,000 | < 700 | soft |
| 10-20,000 | 700-1400 | moderately hard |
| 20-30,000 | 1400-2100 | medium hard |
| 30-40,000 | 2100-2800 | hard |
| 40-50,000 | 2800-3500 | very hard |
| > 50,000 | > 3500 | extremely hard |

This classification compares favourably with the following which is quoted by Wagner*, and which gives a general designation of hardness of particular rock types.

| Compressive strength kg/cm2 | Hardness designation | Typical rock types |
|---|---|---|
| < 800 | soft | Trachytic, phenolithic and basaltic tuffs, less compact limestones. |
| 800 - 1800 | medium hard | Basaltic lava, limestone, dolomite |
| > 1800 | hard | Granite, basalt, gneiss, quartzite diorite, andesite, conglomerate |

In the absence of any compressive strength values for moberg rocks it is obviously not possible to know for certain where pillow lava and moberg basalt belong in the above classification, but it can certainly be assumed that they can be designated as medium hard and hard respectively.

F. de Quervain** has grouped various rock according to their compressive strengths in the table given below, and in which also the supposed positions of the moberg rock types are indicated.

* Verkehrs - Tunnelbau, Part I, Verlag von Wilhelm Ernst & Sohn, Berlin - München.

** Technische Gesteinkunde, Birkhäuser Verlag, Basel, 1967.

| Compressive strength* in kg/cm2 | Rock types |
|---|---|
| < 400, very low | <u>Tuffs,</u> chalky limestone, very porous sandstone |
| 400- 800, low | Sandstone, porous limestone, <u>breccia</u>, <u>tillite</u> |
| 800-1800, average | Limestone, sandstone, gneiss, medium to coarse grained granite, <u>consolidated tuffs</u>, <u>pillow lava</u>. |
| 1800-2800, high | Fine grained granites, diorites, quartz porphyries, <u>basalt</u>, compact limestone and sandstone. |
| > 2800, very high | Diabase, occasionally sandstone, <u>sound basalt</u> |

* These values are measured by crushing 4-8 cm cubes or cylinders of 15 cm length.

The clear implication of this assessment of compressive strength is that pillow lava could be bored at satisfactory speed using normally available cutters. It must of course also not be forgotten that other rock characteristics besides hardness can influence cutter performance, particular among these are such indefinable factors as the toughness and resiliance of the rock mass, as well as more easily assessed properties like tensile strength and joint spacing. It is probably fair to say that such characteristics would contribute to an improvement of cutter performance in pillow lava, certainly the intensive cracking and fissures will facilitate the breaking up of the pillows under the action of the cutters.

It is the basaltic intrusions which exist within moberg formations which could pose problems for mechanical borers. Since the compressive strength of certain basalts can reach over 3000 kg/cm2, it is questionable whether such rock can be mechanically excavated. It must however be pointed out that the maximum values given by Robbins refer to the economic limit of tunnel boring rather than to the actual maximum hardness which can be bored. Cutters are certainly available for specialised work, such as raise drilling or oil well boring, which can penetrate much harder rock (for instance in Australia, a vertical 1.80 metre diameter shaft was drilled for 120 metres through rock having a compressive strength of 7000 kg/cm2 - in this case some form of mechanical drilling was imperative and therefore the economic considerations were different). Basalt intrusions would certainly slow up boring but, providing that their thickness were not excessive, it cannot be said that their existance would rule out the use of machines. If necessary, it is in any case possible to employ blasting to break up particularly hard rock over short distances when tunneling mechanically.

The American company, G.W. Murphy Industries, has developed a series of cutters for tunnel borers and, as can be seen from the Appendix Documentation, these can be selected directly on the basis of the compressive strengths of the rock to be removed. For comparative purposes, the hardness designations given by Murphys are given below with the corresponding compressive strength in metric units:

| | |
|---|---|
| Soft | < 420 kg/cm2 |
| Medium | 420- 850 kg/cm2 |
| Medium hard | 850-1750 kg/cm2 |
| Hard | >1750 kg/cm2 |

The usual speed of rotation of the cutter head is 5-10 r.p.m. and rates of advance of 1.50-3.00 metres/hour are normal in average conditions, this corresponding to an effective drilling time of 60-65%. The following information on cutter costs is quoted by Murphys on the basis of experience with their Jarva machines between 1966 and 1970:

Minimum \$2.50/m3 For a 6km tunnel in rock of from 70 to 1600 kg/cm2

Maximum cost $5.54/m3. For a 5.4km tunnel in rock of from 1050-2800kg/cm2

Average cost $3.94/m3 This corresponded to an average rate of 1.73 m/hour

On the basis of data obtained from Ingersoll-Rand, the following table of power consumption has been prepared:

| Tunnel dia(m) | Rock type | Hardness range (kg/cm2) | Power consumption kWh/m3 |
|---|---|---|---|
| 5.60 | Clayish slate | 250-1050 | 6.63 |
| 4.16 | Dolomitic limestone | 1100-2600 | 19.50 |
| 3.80 | Argillite, anesite + rhyolite | 1700-3800 | 23.30 |
| 3.80 | Quartzite,limestone,conglomerate | 850-3400 | 21.40 |
| 5.00 | Sandstone, siltstone and marl | 1700-3300 | 17.60 |

During excavation of the Emosson pressure shaft which is referred to earlier in this chapter, power consumption was about 40 kWh/m3 for 2200-2400 kg/cm2 rock, and an Atlas Copco machine working in Switzerland in rock of up to 1500 kg/cm2, required about 22 kWh/m3.

These figures can only be very approximate but they are quoted to give an idea of cutter and power costs for mechanical tunneling since these contribute by far the largest part of the daily running costs of a boring machine.

## 11.5. CONCLUSIONS

On the basis of presently available information, there would appear to be no technical reason for not using mechanical tunneling methods in moberg, but this can only be confirmed by exhaustive field investigations and tests. Much more needs to be known about the physical properties of moberg rocks, in particular for instance crushing strengths, extent of cracking and etc, before accurate forecasts of cutter performance and costs can be made. Ground water conditions must also be studied in detail, and certainly it will be necessary to make frequent borings along the line of any proposed tunnel to determine exactly the position of the water table and the flow conditions therein. But even after all such tests have been completed, there remains the all-important economic decision to be made. The factors which can influence this decision are many and extend beyond the single question of the cost of the machine itself. Transport costs to Iceland and maintenance costs once there will be considerable and will be independent of whether it is possible to find a suitable second-hand machine elsewhere in the world. The availability and cost of skilled tunneling labour will also be important as will be the cost of bringing the electric power to the site. All such factors must be taken into account in the light of the length of tunnel to be excavated and the time available for completion of the project.

# 12 - CONSTRUCTION COSTS

## 12.1. BASIC CONSIDERATIONS

### 12.1.1. General

Having defined the tunnel sections and construction methods for the moberg formations, it is now possible to proceed with the cost estimations. This estimate of tunneling costs can at this stage only be very approximate since much necessary data is not at the present time available.

The unit prices are made up of:

Labour costs
Materials costs
Equipment and plant operating costs

The cost estimates are detailed below, the price being given in U.S. Dollars. An exchange rate of 88 Icelandic Kroners per Dollar has been assumed.

The following additional costs have been included in the calculations:

| | |
|---|---|
| Social costs: | approx. 40% |
| Site costs: | " 25% |
| Total | approx. 65% (these make up the work costs) |

Final addition to work costs for contingencies, risk, etc. 15%

### 12.1.2. Labour costs

The following agreed basic hourly labour rates have been assumed:

| | | | |
|---|---|---|---|
| Rate | 9 | (after 2 years) | Kr.102.35/hour |
| " | 8A | ( " 2 " ) | Kr. 98.05/ " |
| " | 4 | ( " 2 " ) | Kr. 86.40/ " |

The following three eight-hour shifts have been assumed:

| | |
|---|---|
| Morning shift | 0500 - 1300 |
| Afternoon shift | 1300 - 2100 |
| Night shift | 2100 - 0500 |

Time actually paid = 8 hours + 30% extra = 10.4 hours/shift

Two hours overtime per shift have been allowed.

The hourly rates of pay have thus been calculated as:

For day shifts, basic rate + 40%
For night shifts, basic rate + 80%

The labour force will be made up of the following men:

| | |
|---|---|
| Tarif 9 + 30% | Foremen |
| Tarif 9 | Specialists: loader driver, jumbo driver, electrician |
| Tarif 8A | Skilled men: miner, mechanic, fitter, etc. |
| Tarif 4 | Labourers and unscilled drivers |

The following average hourly wages have been therefore calculated by taking account of the above considerations:

| | Average hourly wages | work cost incl. 65% supplement |
|---|---|---|
| Foreman | Kr.240,$2.75 | Kr.396, $ 4.5 |
| Loader driver,jumbo driver,electrician | Kr.180,$2.05 | Kr.297, $ 3.4 |
| Miner, mechanic, fitter | Kr.170,$1.95 | Kr.280, $ 3.2 |
| Labourers | Kr.150,$1.70 | Kr.247, $ 2.8 |

### 12.1.3. Material costs

The following material costs have been assumed:

Basic prices (without extras)

| | | |
|---|---|---|
| Explosive | Kr. 44/kg | approx.$00.50/kg |
| Electric fuses | Kr. 18/piece | " $00.20/piece |
| Fuse wire | SFr. 7.20/100 metres | " $01.85/100 metres |
| Connectors | SFr. 13.--/100 pieces | " $03.35/100 pieces |
| Drilling steels | SFr. 1.60/m'drilling hole | " $00.40/m'drilling hole |
| Reinforcement | SFr. 1.50/kg | approx.$00.40/kg |
| Cement | Kr.2660/ton | " $30.23/ton |
| Aggregates | | $17.--/m3 |

Bernold rock security mats
BF-5 Ø5mm 100x100 mm 9.36 kg/m2
SFr. 2.27/kg approx.$00.60/kg

Bernold perforated sheets
2 mm; 15.43 kg/m2

| | | |
|---|---|---|
| 2 mm FOB Bremen | SFr.1298/ton | |
| shipping costs | SFr. 164/ " | |
| transport within Iceland | SFr. 100/ " | |
| | SFr.1562/ton | approx.$00.40/kg |

| | |
|---|---|
| Concrete from batching plant | $ 35.0/m3 |
| Mortar from batching plant | $ 42.0/m3 |
| Shotcrete from batching plant | $ 42.4/m3 |

| | | | |
|---|---|---|---|
| Plastic sheets | SFr. 40.--/m2 | approx. | $10.--/m2 |
| Diesel oil | SFr. 00.82/kg | " | $00.21/kg |
| | SFr. 00.68/1 | " | $00.17/1 |
| Cost of electricity | Kr. 4 /kWh | " | $00.05/kWh |

## 12.2. TUNNEL SECTION A

### 12.2.1. Calculation of advance rates

| Lining type | Total | AI | AII | AIII | AIV |
|---|---|---|---|---|---|
| Advance/day in three shifts | - | 8.00m | 4.80m | 3.84m | 1.92m |
| Estimation of lining types | - | 50% | 30% | 15% | 5% |
| Advance in 4weeks(18days tunneling+6days f.invert concreting) | - | 144.00m | 86.40m | 69.12m | 34.56m |
| Advance in 4 months | 334.08m | - | - | - | - |
| Average monthly advance | 83.52m | - | - | - | - |
| Average daily advance incl. invert concreting | - | 6.00m | 3.60m | 2.88m | 1.44m |

### 12.2.2. Construction periods

For a 1 km tunnel: Time = 1000/83.52 = 12.6 months
For a 3 km tunnel: Time = 3000/83.52 = 37.8 months

This time could be reduced to about 19 months by tunneling at two faces.

For a 7 km tunnel: Time = 7000/83.52 = 88.4 months

This is a quite unrealistic time and would have to be reduced by tunneling at at least two faces, in such a case it would be almost certainly economic to construct an intermediate adit or shaft to allow four face advance.

### 12.2.3. Calculation of linear construction costs

| Lining type | AI | AII | AIII | AIV |
|---|---|---|---|---|
| Cost per linear metre in $ * | 825.00 | 1230.00 | 1440.00 | 2200.00 |
| Distribution of lining types | 50% | 30% | 15% | 5% |
| Cost distribution in $ | 412.50 | 369.00 | 216.00 | 110.00 |

Average price per metre = $ 1110

* see following table

| Lining type | AI | AII | AIII | AIV |
|---|---|---|---|---|
| Advance per day in 3 shifts | 8.00m | 4.80m | 3.84 | 1.92m |
| Average daily advance incl. concreting of invert | 6,00m | 3,60m | 2.88m | 1.44m |
| Labour force per shift | 16 men | 16 men | 16 men | 16 men |
| Volume of excavation | 169.20m3 | 109.08m3 | 90.72m3 | 42.77m3 |

| Costs (in $) | | | | |
|---|---|---|---|---|
| Labour | 1212.- | 1212.- | 1212.- | 1212.- |
| Explosive | 135.4 | 87.3 | 72.6 | 10.3 |
| Fuses | 42.- | 28.- | 28.- | 6.- |
| Fuse wire | 5.6 | 3.7 | 3.7 | 1.9 |
| Connectors | 7.- | 4.7 | 4.7 | 1.0 |
| Drilling steels | 184.8 | 123.2 | 95.2 | 18.- |
| Transport costs per day * | 400.- | 400.- | 400.- | 400.- |
| Concrete | - | 768.6 | 735.8 | 201.6 |
| Bernold sheets | - | 354.8 | 284.- | 141.9 |
| Shotcrete | 737.8 | - | - | - |
| Rock security mats | 431.3 | - | - | - |
| Invert concrete | 357.- | - | - | - |
| Reinforcement | 81.6 | 57.6 | 53.- | 30.5 |
| Electricity | 169.2 | 163.7 | 136.1 | 64.2 |
| Mortar | - | - | - | 90.7 |
| Concrete and mortar pumps | 122.5 | 105.- | 105.- | 70.- |
| Supporting arches | 18.- | 10.8 | 8.6 | 4.3 |
| Poling plates ** | - | - | - | 172.8 |
| | 3904.2 | 3319.4 | 3138.7 | 2425.2 |
| Misc.costs and spares (10%) | 390.8 | 331.6 | 311.3 | 242.8 |
| | 4295.- | 3651.- | 3450.- | 2668.- |
| Contingencies (15%) | 645.- | 547.- | 510.- | 400.- |
| Total cost | 4925.- | 4198.- | 3960.- | 3068.- |
| Cost/linear m,excl.gunite | 825.- | 1165.- | 1375.- | 2130.- |
| Placing of final gunite inner lining | - | 65.- | 65.- | 65.- |
| Total cost per metre | 825.- | 1230.- | 1440.- | 2220.- |

* 1 No. CAT.955 + 3 No. 15m3 capacity trucks
** Estimated for 300 m tunnel

## 12.3. TUNNEL SECTION B

### 12.3.1. Calculation of advance rates

| Lining type | BI | BII | BIII | BIV |
|---|---|---|---|---|
| Daily advance, upper section | 8.00m | 5.76m | 4.80m | 1.92m |
| Daily advance, lower section | 16.00m | 11.52m | 9.60m | 3.84m |
| Upper section advance in 3weeks (18 days) | 144.00m | 103.70m | 86.40m | 34.60m |
| Lower section excavation of this length | 9 days | 9 days | 9 days | 9 days |
| Concreting invert over this length | 12 days | 10 days | 10 days | 6 days |
| Concreting side walls over this " | 6 days | 5 days | 5 days | 3 days |
| Total time, inc.excavation upper section | 45 days | 42 days | 42 days | 36 days |
| Average daily advance | 3.20m | 2.46m | 2.05m | 0.96m |
| Advance in four weeks (24 days) | 76.8m | 59.0m | 49.2m | 23.0m |
| Distribution of lining types | 50% | 30% | 15% | 5% |
| | 38.4m | 17.7m | 7.4m | 1.2m |

Average monthly advance = 64.7 metres

### 12.3.2. Construction periods and labour requirements

For a 1 km tunnel: Time = 15.5 months
Single face advance from one site

For a 3 km tunnel: Time = 46.4 months
Tunneling at two faces required which necessitates two site installations

For a 7 km tunnel: Time = 108 months
Four face advance would be required,this would necessitate an intermediate adit or shaft and three sets of site installations

Three shifts, each of 22 men, have been assumed, this gives a daily labour cost of $1694.4.

### 12.3.3. Calculation of linear construction cost

| Lining type | BI | BII | BIII | BIV |
|---|---|---|---|---|
| Cost p.linear metre in $ * | 1680.00 | 2370.00 | 2775.00 | 4420.00 |
| Estimation of lining types | 50% | 30% | 15% | 5% |
| Cost distribution in $ | 840.00 | 711.00 | 416.00 | 221.00 |

* see following table

Average price per metre = $2190.-

| Lining type | BI | BII | BIII | BIV |
|---|---|---|---|---|
| Average daily advance incl. concreting of invert | 3.20m | 2.46m | 2.05m | 0.96m |
| Labour force per shift | 22 men | 22 men | 22 men | 22 men |
| Volume of excavation | 191.04m3 | 157.44m3 | 136.74m3 | 62.11m3 |
| Costs in $ | | | | |
| Labour | 1695.00 | 1695.00 | 1695.00 | 1695.00 |
| Explosive | 122.20 | 100.80 | 87.50 | 14.90 |
| Fuses | 32.00 | 16.00 | 16.00 | 8.00 |
| Fuse wire | 7.40 | 3.70 | 3.70 | 1.90 |
| Connectors | 5.40 | 2.70 | 2.70 | 1.30 |
| Drilling steels | 115.20 | 83.20 | 70.40 | 19.20 |
| Transport costs per day * | 600.00 | 600.00 | 600.00 | 600.00 |
| Concrete | - | 912.70 | 954.50 | 305.90 |
| Bernold sheets | - | 358.90 | 299.10 | 140.00 |
| Shotcrete | 664.80 | - | - | - |
| Rock security-mats | 339.70 | - | - | - |
| Invert concrete | 257.60 | - | - | - |
| Reinforcement | 62.70 | 64.90 | 61.50 | 33.00 |
| Rock bolts | - | 176.00 | 176.00 | - |
| Electricity | 191.00 | 236.20 | 205.10 | 93.20 |
| Mortar | - | - | - | 88.70 |
| Concrete and mortar pumps | 140.00 | 157.50 | 157.50 | 105.00 |
| Supporting arches | 19.20 | 14.80 | 12.30 | 5.80 |
| Poling-plates ** | - | - | - | 169.00 |
| | 4252.20 | 4422.40 | 4341.30 | 3280.90 |
| 10% for spares, etc. | 425.80 | 442.60 | 434.70 | 328.10 |
| | 4678.00 | 4865.00 | 4776.00 | 3609.00 |
| 15% Contingencies | 702.00 | 730.00 | 716.00 | 541.00 |
| Total cost | 5380.00 | 5595.00 | 5492.00 | 4150.00 |
| Cost linear metre,excl.gunite | 1680.00 | 2275.00 | 2679.00 | 4323.00 |
| Placing of final gunite inner lining | - | 95.00 | 96.00 | 97.00 |
| Total cost per metre | 1680.00 | 2370.00 | 2775.00 | 4420.00 |

* 1 No. CAT 755 + 3 No.15m3 capacity trucks
** Estimated for 300 m tunnel

## 12.4. TUNNEL SECTION C

### 12.4.1. Calculation of advance rates

| Lining type | CI | CII | CIII | CIV |
|---|---|---|---|---|
| Daily advance, upper section(48m2) | 6.00m | 4.80m | 3.84m | 1.92m |
| Daily advance,lower section | 9.00m | 7.20m | 5.76m | 2.88m |
| Upper section advance in 3weeks (18 days) | 108.00m | 86.40m | 69.10m | 34.60m |
| Lower section excavation of this length | 12 days | 12 days | 12 days | 12 days |
| Concreting invert over this length | 12 days | 10 days | 10 days | 6 days |
| Concreting sidewalls over " " | 6 days | 4 days | 4 days | 2 days |
| Total time,incl.excavation of upper section | 48 days | 44 days | 44 days | 38 days |
| Average daily advance | 2.25m | 1.96m | 1.57m | 0.91m |
| Advance in four weeks(24 days) | 54.00m | 47.04m | 37.68m | 21.84m |
| Distribution of lining types | 50% | 30% | 15% | 5% |
| | 27.00m | 14.10m | 5.70m | 1.10m |

Average monthly advance = 47.90 metres

### 12.4.2. Construction periods and labour requirements

For a 1 km tunnel: Time = 20.8 months
Single face advance from one site

For a 3 km tunnel: Time = 62.6 months
Tunneling at two faces with two site installations

For a 7 km tunnel: Time = 146 months
Advance at four faces would be required with inconsequence three site installations and an intermediate adit or shaft.

Three shifts, each of 30 men, have been assumed, this gives a daily labour cost of $2402.4.

### 12.4.3. Calculation of linear construction cost

| Lining type | CI | CII | CIII | CIV |
|---|---|---|---|---|
| Cost per linear metre in $ * | 2865.00 | 3765.00 | 4840.00 | 6630.00 |
| Distribution of lining types | 50% | 30% | 15% | 5% |
| Cost distribution in $ | 1433.00 | 1130.00 | 726.00 | 331.00 |

*see table on page 12-9

Average price per metre = $ 3620.

## 12.5. SUMMARY OF LINEAR TUNNEL COSTS

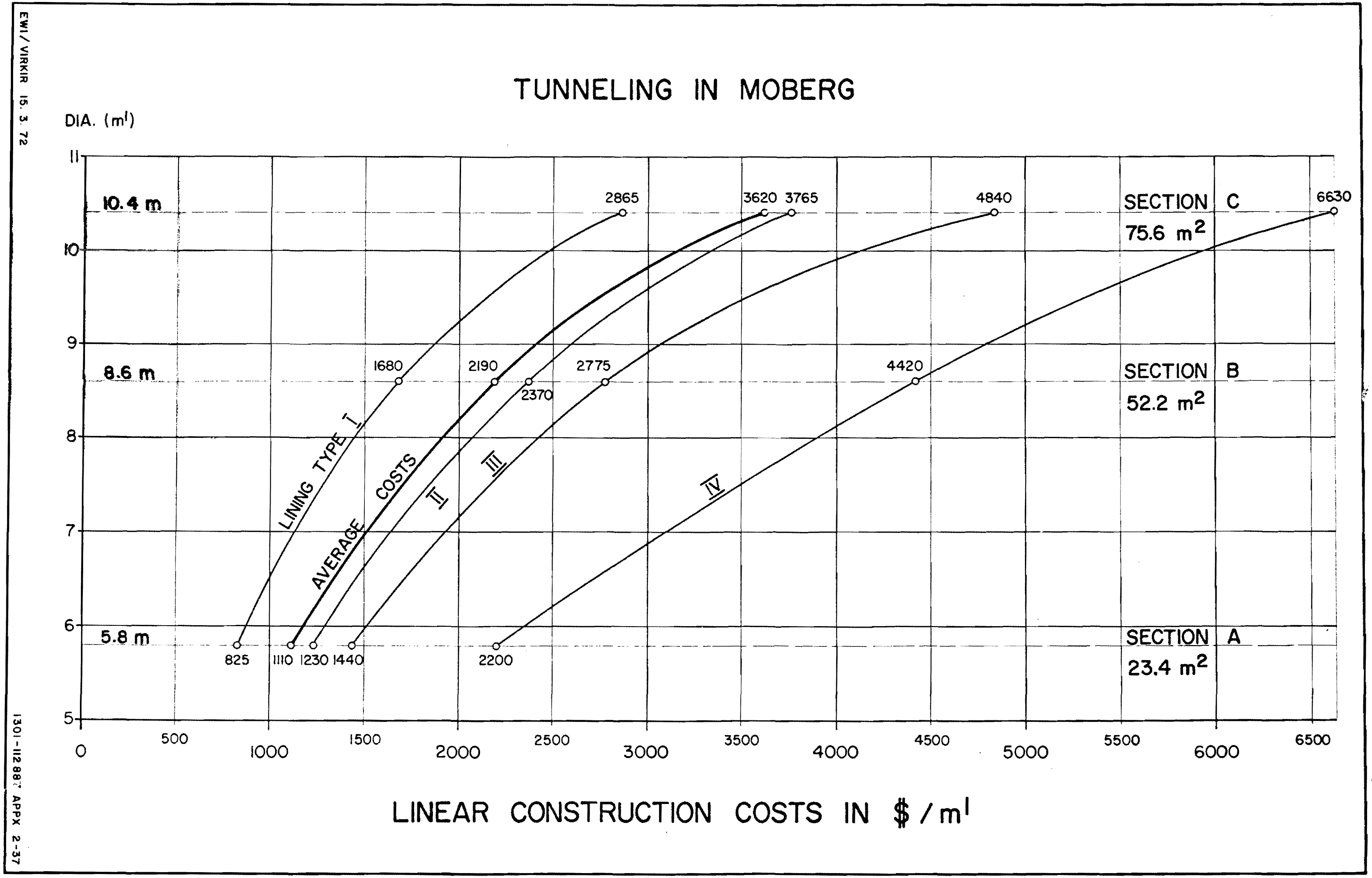
| Tunnel Section | Lining type I | II | III | IV | Average price |
|---|---|---|---|---|---|
| A:23.4m2 | 825.- | 1230.- | 1440.- | 2200.- | 1110.- |
| B:52.2m2 | 1680.- | 2370.- | 2775.- | 4420.- | 2190.- |
| C:75.6m2 | 2865.- | 3765.- | 4840.- | 6630.- | 3620.- |

These costs are plotted graphically in the following drawing

| Lining type | CI | CII | CIII | CIV |
|---|---|---|---|---|
| Average daily advance incl. concreting of invert | 2.25m | 1.96m | 1.57m | 0.91m |
| Labour force per shift | 30 men | 30 men | 30 men | 30 men |
| Volume of excavation in m3 | 190.35 | 177.97 | 156.84 | 83.45 |

Costs in $

| | | | | |
|---|---|---|---|---|
| Labour | 2402.4 | 2402.4 | 2402.4 | 2402.4 |
| Explosive | 99.- | 92.5 | 81.6 | 20.- |
| Fuses | 20.- | 20.- | 20.- | 10.- |
| Fuse wire | 3.7 | 3.7 | 3.7 | 1.4 |
| Connectors | 3.4 | 3.4 | 3.4 | 1.7 |
| Drilling steels | 100.- | 84.- | 72.- | 20.- |
| Transport costs per day * | 900.- | 900.- | 900.- | 900.- |
| Concrete | - | 945.- | 1260.- | 385.- |
| Bernold sheets | - | 340.7 | 272.9 | 158.2 |
| Shotcrete | 572.4 | - | - | - |
| Rock security mats | 284.3 | - | - | - |
| Invert concrete | 210.- | - | - | - |
| Reinforcement | 54.- | 72.1 | 66.6 | 49.5 |
| Rock bolts | - | 264.- | 264.- | - |
| Electricity | 285.5 | 355.9 | 313.7 | 125.2 |
| Mortar | - | - | - | 252.1 |
| Concrete and mortar pumps | 140.- | 157.5 | 192.5 | 140.- |
| Supporting arches | 20.3 | 17.6 | 14.1 | 8.2 |
| Poling plates | - | - | - | 216.- |
| | 5095.- | 5658.8 | 5866.9 | 4689.7 |
| Misc.costs and spares (10%) | 505.- | 561.2 | 583.1 | 468.3 |
| | 5600.- | 6220.- | 6450.- | 5158.- |
| Contingencies (15%) | 840.- | 933.- | 970.- | 772.- |
| Total cost | 6440.- | 7153.- | 7420.- | 5930.- |
| Cost/linear metre, excl.gunite | 2865.- | 3650.- | 4725.- | 6515.- |
| Placing of final gunite inner lining | - | 115.- | 115.- | 115.- |
| Total cost per metre | 2865.- | 3765.- | 4840.- | 6630.- |



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TUNNELING IN MOBERG
DIA. (m')
10.4 m
8.6 m
5.8 m
SECTION C
75.6 m2
SECTION B
52.2 m2
SECTION A
23.4 m2
LINING TYPE I
AVERAGE COSTS
II
III
IV
2865
3620
3765
4840
6630
1680
2190
2370
2775
4420
825
1110
1230
1440
2200
LINEAR CONSTRUCTION COSTS IN $ / m'
1301-112887 APPX 2-37

## 12.6. TUNNELING COSTS IN GROUND WATER ZONES

### 12.6.1. General

It has been assumed for the purpose of this estimation that when tunneling in zones where the groundwater inflow appears as a heavy rain, the methods of dealing with water which are detailed in Appendix 2-24 will be employed, i.e. for section AI, the Oberhasli Method and for sections II-IV, the erection of plastic sheeting (see Appendix-Documentation). Furthermore, allowance has been made for treating the whole of the vault by either method and for the need to carry out this work at the time of excavation. The following additional costs must therefore be estimated:

- Labour costs
- Costs of materials
- Pumping costs

### 12.6.2. Materials and equipment

The following costs and quantities of materials and equipment have been assumed, the quantities being based on experience on projects in Switzerland.

| | |
|---|---|
| Sika 4a, rapid hardening additive | \$ 0.9/kg |
| Spribag plastic drainage guttering | \$ 0.5/m |
| "Guniplast" plastic sheeting | \$ 6.0/m2 |

| | Oberhasli method | Plastic sheeting |
|---|---|---|
| Steelwork | 1.1 kg/m2 treated | 1.1 kg/m2 treated |
| Cement | 110 kg/m2 " | 57 kg/m2 " |
| Sika 4a | 5 kg/m2 " | 3.2 kg/m2 " |
| Drainage guttering | 5 m /m2 " | 0.3 m /m2 " |
| Plastic sheeting | - | 1 m2/m2 " |

6.5 HP water pump \$1.3/hour = \$ 31.2/24 hour day
Alivainat mortar pump \$ 17.5/hour

### 12.6.3. Cost estimates for dealing with water

The linear costs for sections AI-AIV, BI-BIV and CI-CIV are calculated in the tables on the following pages. In the tables given below, the total costs are compared with those calculated for tunneling in the absence of heavy groundwater inflow.

| Section | AI | AII | AIII | AIV | Average |
|---|---|---|---|---|---|
| Estimated distribution | 50% | 30% | 15% | 5% | - |
| Without groundwater $ | 825.- | 1230.- | 1440.- | 2200.- | 1110.- |
| With groundwater $ | 1280.-* | 1755.- | 2030.- | 2700.-** | 1606.- |
| Percentage increase | 55%* | 43% | 41% | 23%** | 45% |

| Section | BI | BII | BIII | BIV | Average |
|---|---|---|---|---|---|
| Estimated distribution | 50% | 30% | 15% | 5% | - |
| Without groundwater $ | 1680.- | 2370.- | 2775.- | 4420.- | - |
| With groundwater $ | 2375.-* | 3230.- | 3770.- | 5285.-** | 2991.- |
| Percentage increase | 41%* | 36% | 35% | 22%** | 37% |

| Section | CI | CII | CIII | CVI | Average |
|---|---|---|---|---|---|
| Estimated distribution | 50% | 30% | 15% | 5% | - |
| Without groundwater $ | 2865.- | 3765.- | 4840.- | 6630.- | 3620.- |
| With groundwater $ | 3785.-* | 4905.- | 6150.- | 7805.-** | 4677.- |
| Percentage increase | 32%* | 30% | 27% | 18%** | 29% |

* The increase in price is greatest for lining type I since plastic sheeting cannot be erected and therefore the relatively expensive Oberhasli method must be employed

** The price increase for lining type IV is smallest since with poling plate advance conditions are most favourable for erection of the plastic sheeting.

### 12.6.4. Summary of linear tunneling costs in ground water

| Tunnel section | Lining type I | II | III | IV | Average price |
|---|---|---|---|---|---|
| A: 23.4m2 | 1280.- | 1755.- | 2030.- | 2700.- | 1606.- |
| B: 52.2m2 | 2375.- | 3230.- | 3770.- | 5385.- | 2991.- |
| C: 75.6m2 | 3785.- | 4905.- | 6150.- | 7805.- | 4677.- |

These costs are plotted graphically in the following drawing

| | AI | AII | AIII | AIV |
|---|---|---|---|---|
| Average daily advance without groundwater | 6.00m | 3.60m | 2.88m | 1.44m |
| Labour per shift | 16 men | 16 men | 16 men | 16 men |
| Extra labour for dealing with water | 8 men | 8 men | 8 men | 8 men |
| Rate of waterproofing (hours/m2) | 1.75 | 1.75 | 1.75 | 1.75 |
| Treated area (m2/metre) | 12.80 | 13.30 | 13.50 | 12.90 |

Costs in $

| | AI | AII | AIII | AIV |
|---|---|---|---|---|
| Daily advance (not incl.inner gunite) in absence of groundwater | 4950.- | 4198.- | 3960.- | 3068.- |
| Labour | 848.4 | 530.3 | 429.3 | 207.1 |
| Steel | 33.8 | 21.1 | 17.1 | 8.2 |
| Cement | 255.4 | 82.5 | 67.- | 32.- |
| Sika 4a | 345.6 | 137.9 | 112.- | 53.5 |
| Guttering | 192.- | 7.2 | 6.- | 2.8 |
| Guniplast | - | 287.3 | 233.3 | 111.5 |
| Rock security mats | - | 268.9 | - | - |
| Bernold sheets | - | - | 301.7 | - |
| Alivamat mortar pump | 294.- | - | - | - |
| Pumping of water | 53.- | 89.7 | 126.8 | 146.1 |
| Electricity | 118.4 | 71.6 | 48.2 | 11.- |
| Total, additional costs | 2140.6 | 1496.5 | 1341.4 | 572.2 |
| Misc.costs and spares (10%) | 214.4 | 149.5 | 134.6 | 57.8 |
| | 2355.- | 1646.- | 1476.- | 630.- |
| Contingencies (15%) | 355.- | 247.- | 221.- | 95.- |
| Total cost dealing with water | 2710.- | 1893.- | 1697.- | 725.- |
| Cost of one day's advance | 7660.- | 6091.- | 5657.- | 3793.- |
| Cost per linear metre | 1280.- | 1690.- | 1965.- | 2635.- |
| Placing of final gunite lining | - | 65.- | 65.- | 65.- |
| Total cost per linear metre | 1280.- | 1755.- | 2030.- | 2700.- |

| | BI | BII | BIII | BIV |
|---|---|---|---|---|
| Average daily advance without groundwater | 3.20m | 2.46m | 2.05m | 0.96m |
| Labour per shift | 22 men | 22 men | 22 men | 22 men |
| Extra labour for dealing with water | 10 men | 10 men | 10 men | 10 men |
| Rate of waterproofing (hours/m2) | 1.75 | 1.75 | 1.75 | 1.75 |
| Treated area | 60.48 | 48.22 | 41.41 | 18.62 |

Costs in $

| | | | | |
|---|---|---|---|---|
| Daily advanc e (not incl.inner gunite) in absence of groundwater | 5380.- | 5595.- | 5492.- | 4150.- |
| Labour | 741.6 | 600.3 | 512.- | 229.5 |
| Steel | 26.6 | 21.2 | 18.2 | 8.2 |
| Cement | 201.1 | 83.1 | 71.4 | 32.1 |
| Sika 4a | 272.2 | 138.9 | 119.3 | 53.6 |
| Guttering | 156.2 | 15.- | 12.4 | 5.6 |
| Guniplast | - | 289.3 | 248.5 | 111.7 |
| Rock security mats | - | 270.8 | - | - |
| Bernold sheets | - | - | 321.3 | - |
| Alivamat mortar pump | 183.8 | - | - | - |
| Pumping of water | 89.7 | 169.- | 243.8 | 283.4 |
| Electricity | 83.6 | 83.6 | 62.- | 12.6 |
| Total, additional costs | 1754.8 | 1671.2 | 1608.9 | 736.7 |
| Misc.costs and spares (10%) | 175.2 | 167.8 | 161.1 | 73.3 |
| | 1930.- | 1839.- | 1770.- | 810.- |
| Contingencies (15%) | 290.- | 276.- | 265.- | 120.- |
| Total cost of dealing with water | 2220.- | 2115.- | 2035.- | 930.- |
| Cost of one day's advance | 7600.- | 7710.- | 7527.- | 5080.- |
| Cost per linear metre | 2375.- | 3135.- | 3675.- | 5290.- |
| Placing of final gunite lining | - | 95.- | 95.- | 95.- |
| Total cost per linear metre | 2375.- | 3230.- | 3770.- | 5385.- |

| | CI | CII | CIII | CIV |
|---|---|---|---|---|
| Average daily advance without groundwater | 2.25m | 1.96m | 1.57m | 0.91m |
| Labour per shift | 30 men | 30 men | 30 men | 30 men |
| Extra labour for dealing with water | 12 men | 12 men | 12 men | 12 men |
| Rate of waterproofing (hours/m2) | 1.75 | 1.75 | 1.75 | 1.75 |
| Treated area | 50.63 | 45.47 | 37.05 | 18.75 |

Costs in $

| | CI | CII | CIII | CIV |
|---|---|---|---|---|
| Daily advance (not incl.inner gunite) in absence of groundwater | 6440.- | 7153.- | 7420.- | 5930.- |
| Labour | 750.8 | 660.7 | 540.5 | 270.3 |
| Steel | 22.3 | 20.- | 16.3 | 8.3 |
| Cement | 168.4 | 78.3 | 63.8 | 32.3 |
| Sika 4a | 227.8 | 131.- | 106.7 | 54.- |
| Guttering | 126.6 | 13.6 | 11.1 | 5.6 |
| Guniplast | - | 272.8 | 222.3 | 112.5 |
| Rock security mats | - | 255.4 | - | - |
| Bernold sheets | - | - | 287.5 | - |
| Alivamat mortar pump | 131.3 | - | - | - |
| Pumping of water | 122.9 | 238.7 | 305.8 | 347.1 |
| Electricity | 89.2 | 97.9 | 70.6 | 14.1 |
| Total additional costs | 1639.3 | 1768.4 | 1624.6 | 844.2 |
| Misc.costs and spares (10%) | 163.7 | 176.6 | 162.4 | 84.8 |
| | 1803.- | 1945.- | 1787.- | 929.- |
| Contingencies (15%) | 270.- | 292.- | 268.- | 139.- |
| Total cost of dealing with water | 2073.- | 2237.- | 2055.- | 1068.- |
| Cost | | | | |
| Cost of one day's advance | 8513.- | 9390.- | 9475.- | 6998.- |
| Cost per linear metre | 3785.- | 4790.- | 6035.- | 7690.- |
| Placing of final gunite lining | - | 115.- | 115.- | 115.- |
| Total cost per linear metre | 3785.- | 4905.- | 6150.- | 7805.- |

## 12.7. TOTAL CONSTRUCTION COSTS

For any tunnel construction project, particular site installations will be required. These include camps, offices, workshops, magazines, compressed air plant, concrete mixers, transport facilities, boring equipment, electrical installations, water supply, site roads, etc. The costs for these facilities must be added to the pure tunnel costs already calculated, but it is at the present time very difficult to exactly estimate these fixed site costs since neither the allocation for the site nor the length and size of the tunnels are known. To make, however, a rough allowance for these costs, the tunneling costs will be increased by 15%, this figure being estimated on the basis of experience. This allowance can only be approximate and could vary both with the site and the contractor.

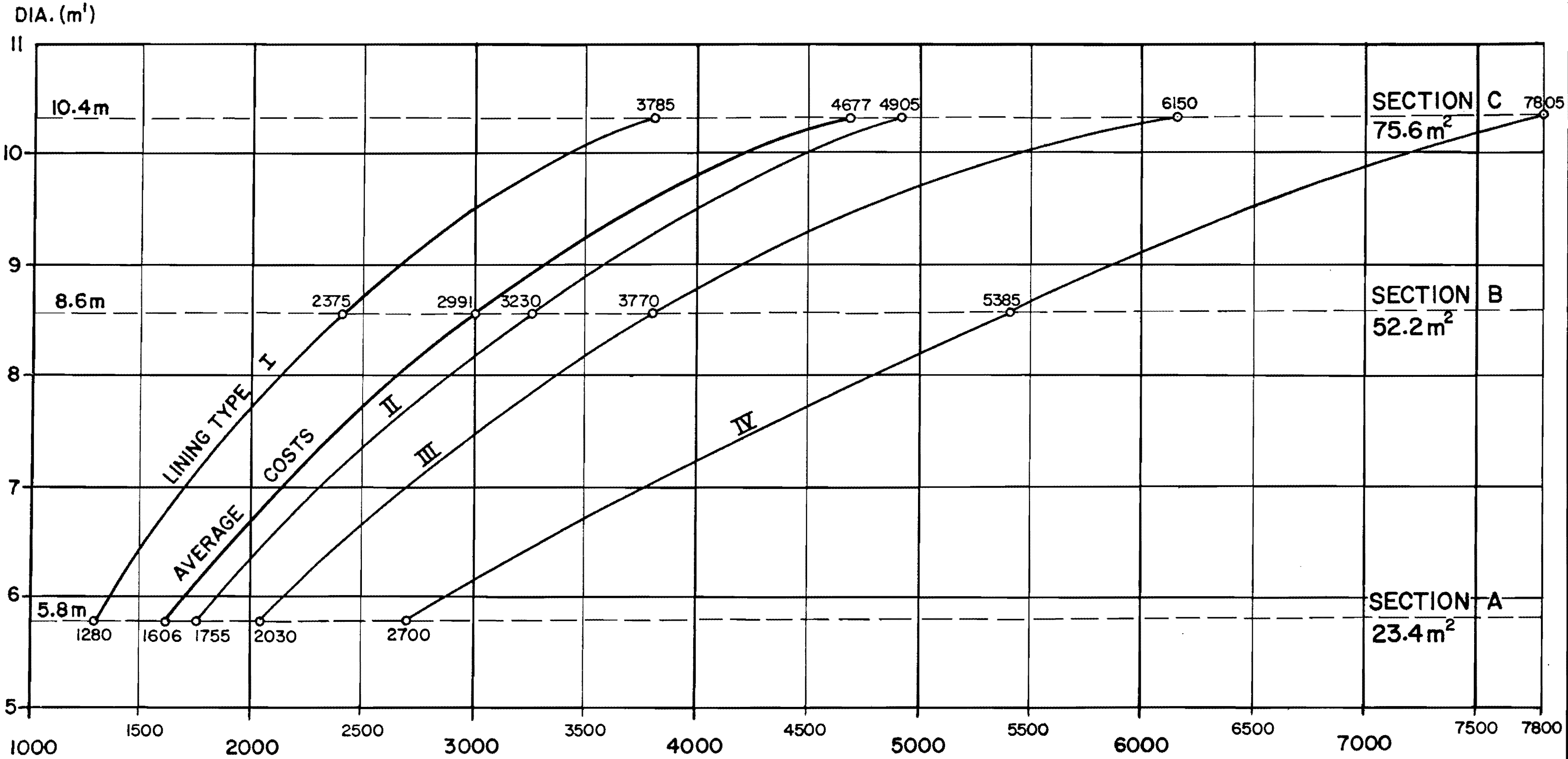
Total tunnel costs, including the 15% for site installations, are given in the table below and these can be considered sufficient to cover also the cost of design and site supervision.

| Tunnel Construction Costs (without cost of dealing with water) | Tunnel length 1 km $ 1000 | 3 km $ 1000 | 7 km $ 1000 |
|---|---|---|---|
| Section A: ∅ = 5.8m; S = 23.4m2 | | | |
| Construction costs | 1'110 | 3'330 | 7'770 |
| Installations 15% approx. | 170 | 500 | 1'170 |
| Total | 1'280 | 3'830 | 8'940 |
| Section B: ∅ = 8.6m; S = 52.2m2 | | | |
| Construction costs | 2'190 | 6'570 | 15'330 |
| Installations 15% approx. | 330 | 990 | 2'300 |
| Total | 2'520 | 7'560 | 17'630 |
| Section C: ∅ =10.4m; S = 75.6m2 | | | |
| Construction costs | 3'620 | 10'860 | 25'340 |
| Installations 15% approx. | 550 | 1'630 | 3'800 |
| Total | 4'170 | 12'490 | 29'140 |

A further reason why these cost estimates can at present only be very approximate is the fact that it is not possible to specify exactly the distribution of the lining types I-IV over the whole tunnel length. This distribution has been estimated at 50%, 30%, 15% and 5% for types I - IV respectively but could - depending on the geological conditions - vary considerably. It is however possible with the prices given in this chapter to calculate the costs for a particular case even when the tunnel concerned must be constructed in ground water.

The measures necessary for dealing with ground water can result in an increase of 20-50% in tunnel construction costs, and therefore, ground water zones should obviously whenever possible be avoided.

# TUNNELING IN MOBERG




LINEAR COSTS OF TUNNELING IN GROUNDWATER, IN $/m'

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1301-112888 APPX 2-38