Tunneling in Móberg

VIRKIR

Orkustofnun
National Energy Authority

Tunneling in Móberg Formations

Study

Summary and Conclusions

EWI

Electro-Watt, Engineering Services Ltd.
Zurich, Switzerland

VIRKIR

Associated Engineering Consultants Ltd.
Reykjavik, Iceland

March 1972

S U M M A R Y   A N D   C O N C L U S I O N S

L I S T   O F   C O N T E N T S

INTRODUCTION

Several proposed hydro-electric projects in Iceland are situated in areas where the rock consists mainly of so-called moberg, but to date only one tunnel, the headrace tunnel of the Efra Sog power plant on the River Sog in southern Iceland, has been constructed in this characteristic volcanic formation. The diversion of the River Skafta into the Tungnaa, which would permit the installation of a fourth group at the proposed Sigalda station, would however 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 Skafta, 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.

The main report on the study consists of two volumes: the text and the Appendices consisting of photographs, drawings and documentation. In this summary volume the geological and engineering characteristics of the moberg formations are only briefly described, the methods and special procedures to be adopted when tunneling in moberg are outlined and the results of the cost estimates are given. In addition, the main conclusions of the report are restated.

SUMMARY

Geology and Engineering Characteristics of Moberg

Iceland is entirely built up of volcanic formations. The extensive glaciations which occured during late Tertiary and Quaternary times greatly affected 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 moberg formations in the areas of the Sigalda and Thorisvatn projects are fully described in the geological reports on these projects, and their extent can be clearly seen on the 1:250 000 geological map of South-central Iceland.

The moberg at Sigalda, which can be considered as typical of these formations, is of late Pleistocene age - most of it dating from the last glaciation - and it 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 with veins,dykes or minor intrusions of basalt. 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.

In the Sigalda area there are no outstanding tectonic features, the landscape following the general tectonic pattern of 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. Earthquakes are common in this part of the country but within the volcanic belt they are usually shallow and weak, beyond this belt however, although they are less common, they are often deeper and of larger magnitude.

The engineering properties of the various moberg rock types have been investigated at both Sigalda and Thorisvatn and the results are fully detailed in the respective reports. Seismic surveys and ripping trials indicated that most rock can be excavated by ripping with the exception of the basaltic intrusions and certain pillar lavas, doubt existing about the latter because of the possibly unrepresentative seismic velocities recorded in the honeycomb formation.

The drilling and grouting tests carried out at Sigalda demonstrated that drilling in moberg formations may often be difficult and slow, this being a result both of the heterogenity of the formations as well

as of the softness and friability of certain of the rocks. Drilling for cores was only successful in about 30 % of the cases and elsewhere 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; the tests indicated that the formations could be grouted but that for effective sealing pressures of at least 40 kg/cm2 would be necessary.

The various rock types have extremely variable permeabilities but, without extensive tests, it would be difficult to determine representative values. It is clear however that moberg formations in general are highly permeable, and that k-values of 1 or even 2 x 10^{-3}cm/s are to be expected. The formations are good aquifers in which large groundwater flow systems exist, and above the permanent groundwater table, perched groundwater tables are found in many places. For any tunnel project, it is imperative that the groundwater conditions be fully investigated before work is started in order that the dry zones, satchurated zones and perched water tables can be located and examined.

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. This classification is shown in the following diagram; it can only at present constitute a first estimate, but is considered necessary to more closely define the properties of the moberg formations.

Technical classification of Moberg formations

Moberg Formation	Appearance and structure	Hardness (Basalt = V.hard, Granite = hard)	Seismic velocity in m/s.	Ripping performance with Cat.D-9	Drillability	Excavation methods	Supporting system (lining)
Pillow lava (photos 1,2, 3+4)	Rounded or cubic cobbles of basalt, 10-30 cm, and strongly jointed. Massive texture with some basalt intrusions	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)	Fractured, heterogeneous formation with basalt cobbles, sandy or clay filling and some basalt veins	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)	Well consolidated and cemented with pillow lava and basalt fragments in f.g. matrix. outcrops in vertical walls and caves	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)	Sandy gravel with lava fragments, cohesive but porous and weak	weak	300-600	easily ripped or can be simply dozed	drilling not possible	blasting often un-necessary	steel sheets and concrete with poling-plate advance
Tuff (photos 6+12)	Dense and stable rock mass, sometimes affected by tectonics, faulted and cracked	medium hard	1000-2000	rippable	drilling possible	blasting sometimes necessary	shotcrete and mesh or steel sheets and concrete

Tunnel sections

For the purpose of this study and for the estimation of tunneling costs, three tunnel cross-sections were specified (A, B and C), with areas of about 25, 50 and 75 m2 respectively. In addition four different lining types were 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. Only free-flow tunnels have been considered 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. The minimum lining thickness considered consists of steel mesh and shotcrete and for the other lining types it was assumed that use would be made of perforated steel sheets erected to provide immediate protection of the tunnel section and also to form the reinforcement of the concrete lining. The Bernold system, a Swiss tunnel supporting method described in the enclosed documentation, makes use of such sheets and offers both static and cost advantages over conventional systems employing heavy steel arches and beams, and it is on the use of this system that the study and cost estimates have been based.

Tunneling procedures

Tunneling will proceed in principle in four phases as shown in Appendix 2 - 18. Excavation will normally require drilling and blasting but in friable and unconsolidated formations it will be possible to use mechanical excavators although continuous support of such sections with poling-plates may be required. Mucking-out will be done by tracked loaders serving trucks or dumpers. The perforated steel supporting sheets would then be erected against light steel arches and finally backfilling with concrete using pumps or spraying machines would be carried out. The tunnel invert would be concreted normally several weeks after excavation. Partial advance by the Belgian method has been assumed for the two larger sections considered.

For dealing with concentrated inflows of groundwater, allowance has been made for the application of the so-called Oberhasli method using plastic guttering fixed to the rock face with mortar. In the case where groundwater appears as a strong rain, plastic sheeting reinforced with steel mesh will be erected with the supporting sheets and thus will be concreted into the lining proper. Allowance has been made for the use of such sheeting with all lining types.

Drilling in moberg will pose problems and in certain cases will only be possible by using special methods. Progress will be slow and wide variations in the stability of boreholes must be expected. Pure percussion drilling will be impractical and rotary percussion drillings, or at times such procedures as overburden drilling, will have to be employed, often in conjunction with special drill bits. To prevent jamming of the drill steels, high pressure water flsuhing must be allowed for. It will be necessary to carry out extensive drilling tests in order to select the correct equipment for as particular job.

In addition to the problems with drilling, complications must also be expected with the blasting procedure itself. Because of irregularity and partial blockage of the boreholes, it could prove difficult to introduce the charges and also, because of the voids and cracks within the formations, contact between explosive and rock will be poor and the effeciency of the blasting thereby reduced. These problems could both be overcome by the use of semi-liquid "slurry" explosive. Consumption of explosive will almost certainly not exceed that in basalt and will often be lower.

Rock anchors and bolts will certainly be required when tunneling in moberg but again, because of the rock characteristics, special methods may be required. In fissured or friable moberg, the use of normal cone anchors will be out of the question, but "Perfo" bolts would be very suitable as the hole is filled for its whole length with mortar and good contact between bolt and rock is assured, whilst at the same time the mortar tends not to flow into voids and fissures. 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, and systems which utilise the drill steel as anchor must therefore be considered. The need to use pre-stressed anchors, in particular to tie-back the vault lining during partial advance of the larger tunnel sections, may be assumed.

Mechanical tunneling

Recent notable improvements in the design of tunneling machines, which have resulted in lower costs and improved performance, make it necessary to consider their utilisation in moberg formations as alternatives to conventional tunneling methods.

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, particularly in moberg formations, is however the avoidance of blasting, not least because of the consequent reduction in the drilling requirements.

There exist of course limitations to the employment of these machines. Design improvements should ensure a continuing reduction of the technical problems, 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.

The factors which can influence this economic 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.

Two main types of machine were considered, the digger shield machines which advance by ripping the tunnel face and the true tunnel borers with cutters mounted on a rotating head. The performance of the former machines in various formations can be deduced from the results of the normal ripping tests, i.e. they could remove all moberg formations except the basaltice intrusions and pillow lavas. The suitability and running costs of the tunnel borers are above all dependant on the performance of the cutters, and hence the hardness of the rock. On the basis of a correlation between available hardness classifications and the estimated compressive strengths of the various moberg rocks, it was concluded that the boring of pillow lavas would proceed at satisfactory speed whilst progress through short sections of intruded basalt, although slow and probably supplemented by limited blasting, would be possible.

It was therefore concluded that on the basis of presently available information, there appears to be no technical reason for not using mechanical tunneling methods in moberg, but that 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.

Tunneling costs

Cost estimates have been prepared for tunneling both with and without protective measures against ground-water inflow. The unit prices include the costs of labour, materials, equipment and plant operation, and in their calculation a rate of exchange of 88 Iceland Kroners per U.S. Dollar has been assumed. The rates of advance for each tunnel section and lining type have been estimated and hence, for a given distribution of lining types, the average monthly advance rates and average linear construction costs have been deduced for each of the three cross-sections considered (see the following curves). Allowances have been made in these costs of 10 % for contingencies and 15 % for overheads and miscellaneous costs. Total construction costs have been calculated for tunnels of 1, 3 and 7 km length of each section considered, an additional 15 % having been added for the cost of site installations; these costs are summarised in the table below:

Tunnel costs in thousands of U.S. Dollars	Without provisions for dealing with water			With provisions for dealing with water		
Tunnel lengths	1 km	3 km	7 km	1 km	3 km	7 km
Section A: ϕ = 5.8 m						
Construction costs	1'110	3'330	7'770	1'606	4'818	11'242
Installations, 15 %	170	500	1'170	244	722	1'688
Total costs	1'280	3'830	8'940	1'850	5'540	12'930
Section B: ϕ = 8.6 m						
Construction costs	2'190	6'570	15'330	2'991	8'973	20'937
Installations, 15 %	330	990	2'300	449	1'347	3'143
Total costs	2'520	7'560	17'630	3'440	10'320	24'080
Section C: ϕ = 10.4 m						
Construction costs	3'620	10'860	25'340	4'677	14'031	32'739
Installations, 15 %	550	1'630	3'800	703	2'109	4'911
Total costs	4'170	12'490	29'140	5'380	16'140	37'650

CONCLUSIONS

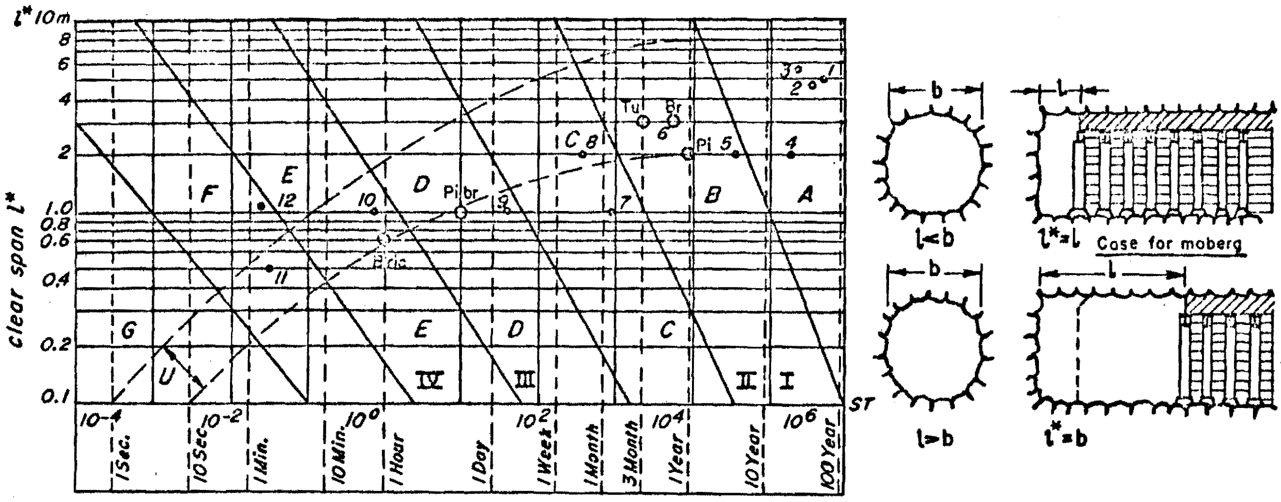
Moberg is a structurally heterogeneous formation of varying hardness, but the study has shown that it consists in general of relatively stable formations in which, using the construction methods described, it will be possible to construct tunnels having the cross-sections detailed and fulfilling the 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 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 m^2 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.

Despite the widely varying physical and structural characteristics of the moberg formations, it can be concluded that 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.

LAUFFER'S DIAGRAM



ROCK MASS

A MASSIVE
B JOINTED
C VERY JOINTED
D BRITTLE
E VERY BRITTLE
F SQUEEZING
G VERY SQUEEZING

U = RANGE OF MOST APPLICATION
ST=STABILITY TIME

EXAMPLES**

1 MASSIVE GRANITE
2,3 GRAYWACKE WITH THICK LAYERS
4, JOINTED QUARTZITE
5,6 WEATHERED GNEISS
7,8 CLAYEY SHALES (ORDOVICIAN)
9 DITTO, WEATHERED
10 ELUVIUM OF GNEISS
11,12 DENSE SAND (WET)

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

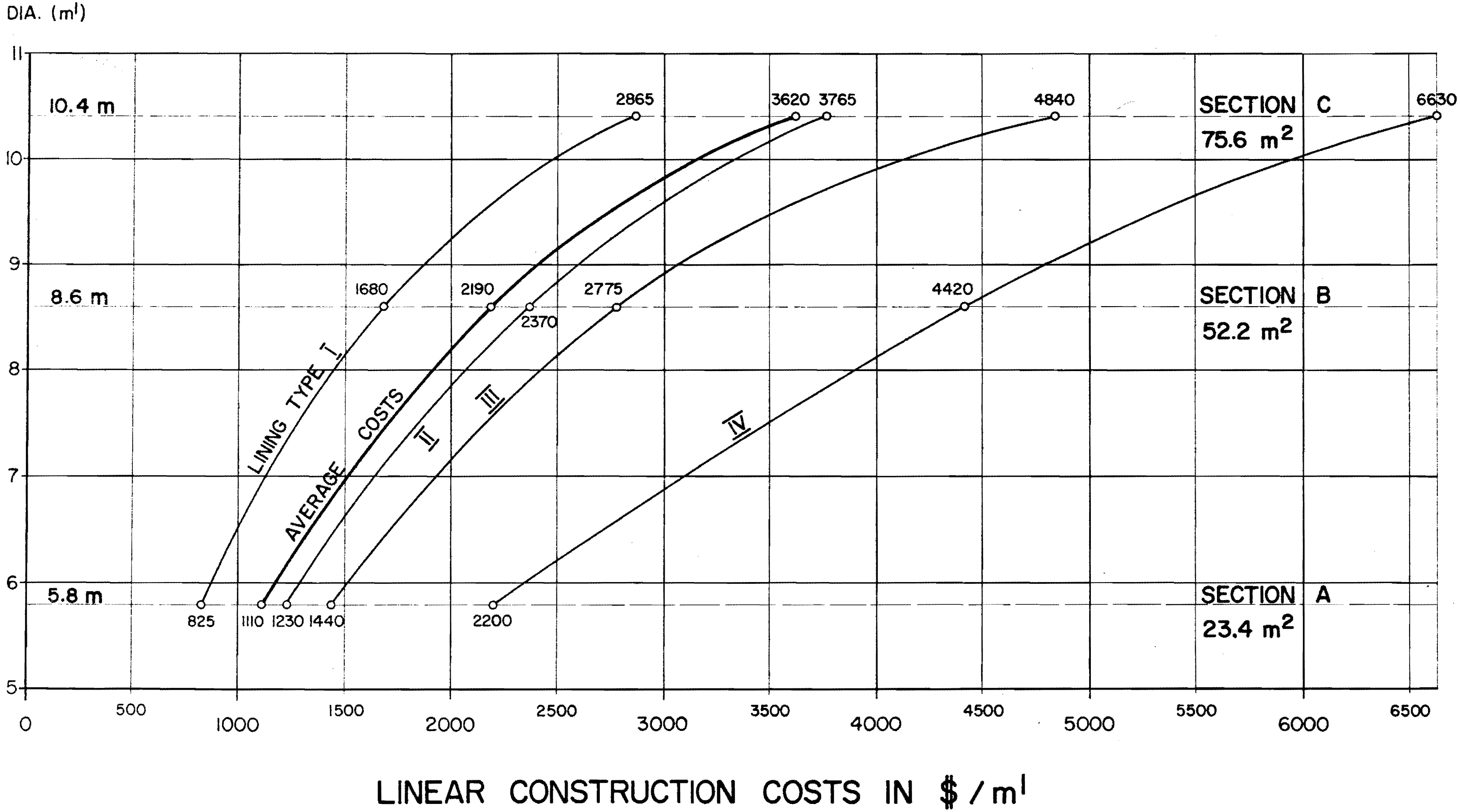
ESTIMATE OF CLASSIFICATION FOR MOBERG

CLASS	MOBERG FORMATION	TYPE OF LINING
B JOINTED	Pi = PILLOW LAVA Br = BRECCIA Tu = TUFF	I or II
D BRITTLE	Pibr = PILLOW LAVA BRECCIATED	III
E VERY BRITTLE	Brlc = BRECCIA LOOSELY CEMENTED TYPE "MORAINE"	IV

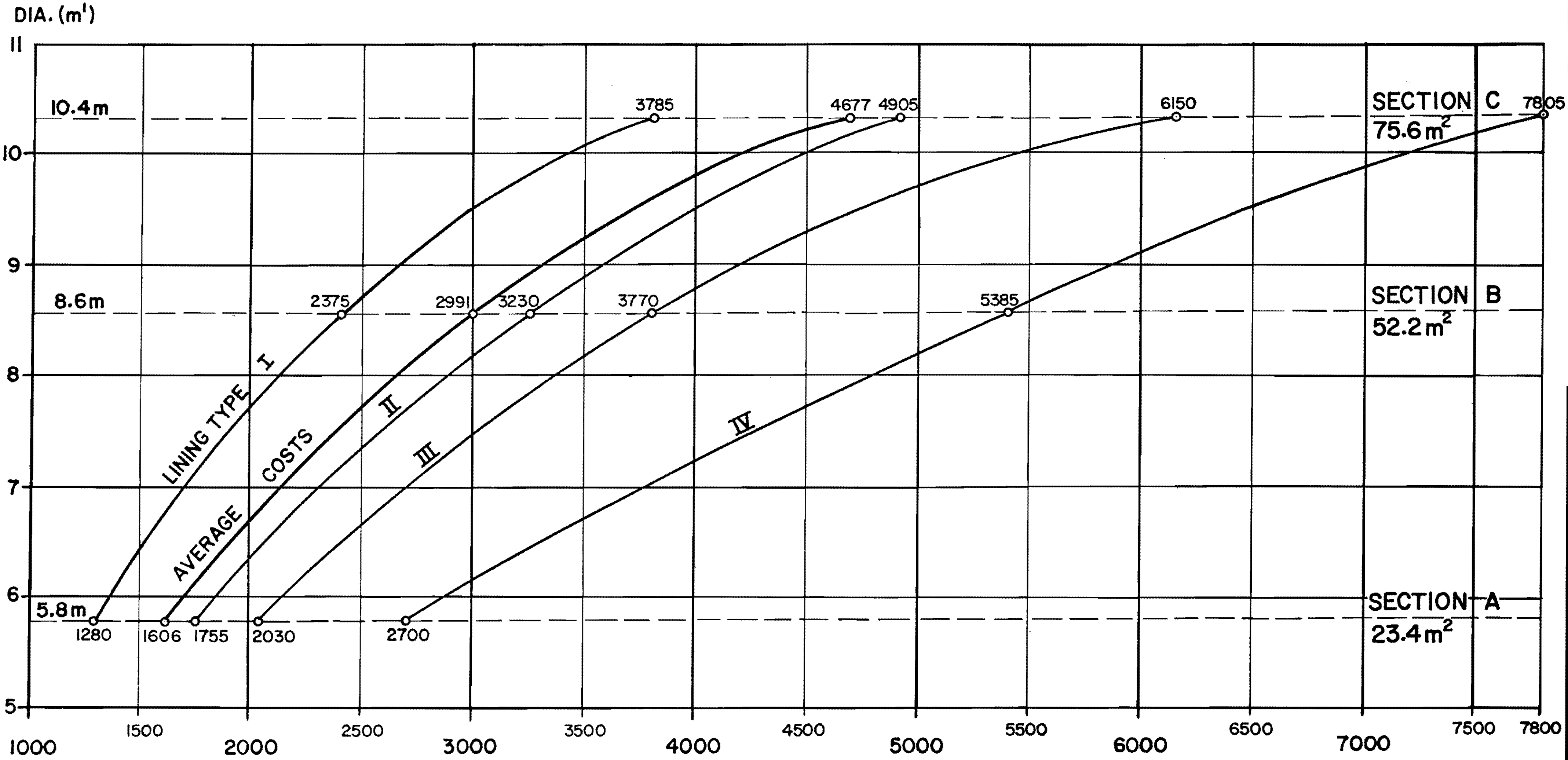
TUNNELING IN MOBERG

LAUFFER'S DIAGRAM OF STABILITY TIME

TUNNELING IN MOBERG



TUNNELING IN MOBERG



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

EWI/VIRKIR 15.3.72

1301-112.888 APPX 2-38

PHOTOGRAPHS

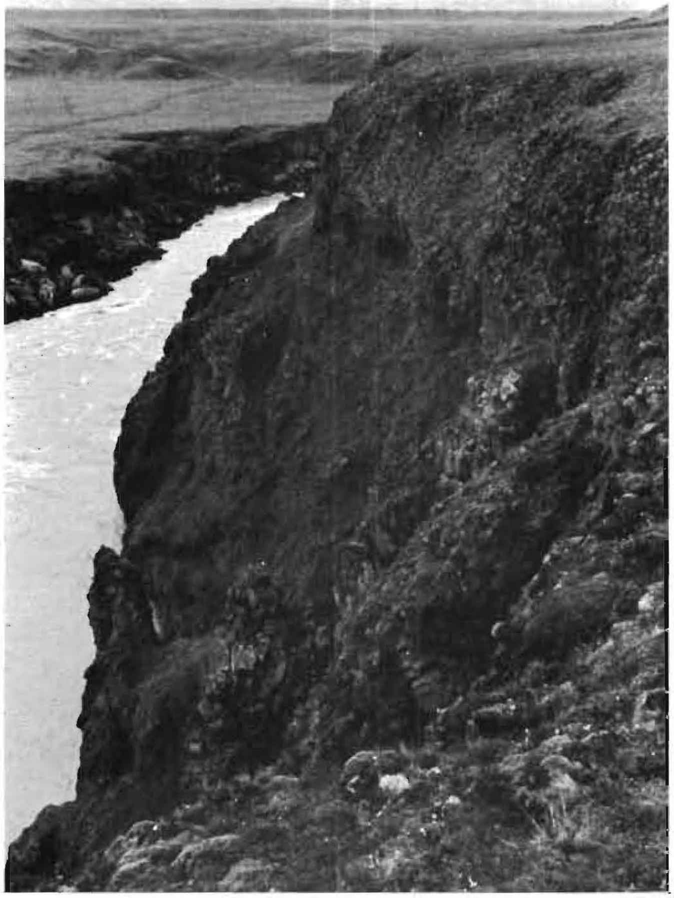
APPENDIX 1

LIST OF PHOTOGRAPHS

Photo No.	Film No.	Description	Date
1	8-17A	Right bank of the Tungnaa river at Sigalda showing the extension of the grouting test. This photograph shows the heterogenity of moberg formations: pillow lava, breccia and tuff. In the right-hand upper corner the site of drilling hole E-11 can be seen.	21.7.71
2	2-6A	Pillow lava in the left river bank, just downstream of the Tungnaa waterfalls at Sigalda. Practically vertical walls of solid and hard rock but including many cracks and voids between the pillows.	13.7.71
3	8-5A	Detail of the pillow lava at Sigalda shown on photograph No.4. The rock is strongly jointed but not loose, and is hard and solid. The individual particles are very hard and of size varying from about 3 to 40 cm thickness. The jointing surfaces tend in all directions and are both flat and curved. Lower right-hand corner, pillow lava can be seen grading into breccia with a sandy (yellow) matrix. In the upper half of the photograph, a basaltic vein (dyke) can be seen. Such veins and intrusions will only be rippable when thin.	21.7.71
4	8-19A	Mass of rounded pillows at Sigalda, between which voids can be seen.	21.7.71
5	9-10A	Tuff breccia at Sigalda, a compact and desne rock with basalt fragments. This formation can be compared with a well consolidated conglomerate or conglomeritic sandstone.	21.7.71
6	9-9A	A solid and nearly vertical wall of tuff and tuffbreccia at Sigalda. This face contains the rock shown in photograph No.5 and is found above the site of the grouting tests.	21.7.71

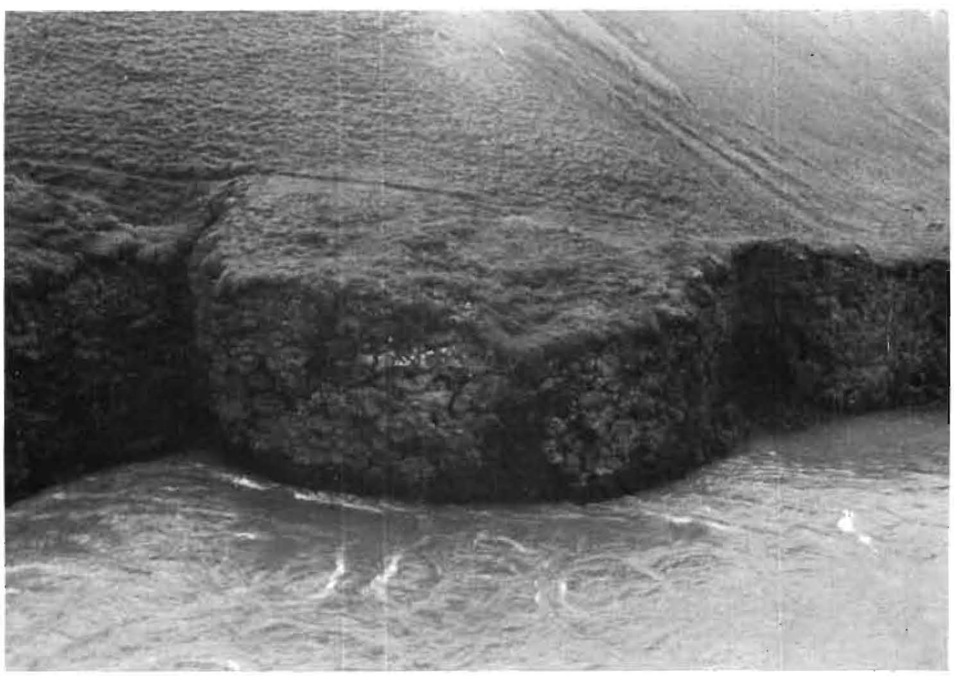
Photo No.	Film No.	Description	Date
7	9-3A	Breccia with pillow structures, the matrix being hard tuff with small pillow fragments. The Sigalda formation is solid and hard and the edges cannot be broken by kicking. Only in the zones where the pillows are not well embedded in the matrix is the rock loose.	21.7.71
8	9-6A	Tuff and tuffbreccia with eroded caves in vertical walls on the right river bank at the beginning of the Tungnaa Canyon at Sigalda. This formation seems to be rather resistant against erosion.	21.7.71
9	1-25A	Southern side of the trench of Sigalda ripping test No.2, showing breccia with fragments of pillow lava. The material has only little cementation and is easily broken out with a hammer. This breccia, which is similar to moraine, is cohesive and was stable with slopes of up to 60^{o} or 70^{o}. It can be easily ripped.	13.7.71
10	9-15A	Northern side of the ripped trench No.1 at Sigalda, showing details of the remaining wall of the brecciated pillow lava with a vein of basalt. This basalt intrusion is rippable as long as it is well fissured.	21.7.71
11	9-20A	Vatnsfell diversion canal, a view from control structure under construction. The canal cuts into the rather compact tuff of the crater wall formations, and the lower part of the control structure is situated in pillow breccia of the lava flow formations.	21.7.71
12	1-29A	Canal wall near the outlet structure, showing stony tuff of the well-consolidated crater wall formation. Overbreak occured due to cracks and sandy lenses at the base.	13.7.71
13	2-33A	Moberg landscape near Grettir looking north north-east towards the Skafta river near the Sveinstindur,1090 m.a.s.l. In this moberg topography it is proposed to drive tunnels for the Skafta diversion. To the left of the photograph is Graenifjallgardur.	14.7.71

Photo No.	Film No.	Description	Date
14	3-14A	Aerial view of the Skafta river near the future diversion dam south of Sveinstindur, looking southwest. The high peak on the right hand side is the Sveinstindur, and diversion tunnel would pass beneath the lower mountains on the left hand side. The Skafta flows from right to left.	15.7.71
15	K-22	Sigalda ripping test No.2 CATERPILLAR D-8 bulldozer with single shank ripper.	3.7.70
16	ZSCH 18-79	Drilling platform under a heavy rain of percolating water in very permeable and strongly jointed but stable dolomite. Diameter of the tunnel 5.70m, water temperature was 4°. This photograph, which was taken in the Val S-charl headrace tunnel of the Engadine hydro-electric power plant in Switzerland, shows tunneling conditions under heavy water infiltrations such as may be expected in moberg below ground water level.	26.4.65



Terrace of grouting test, borehole E-11

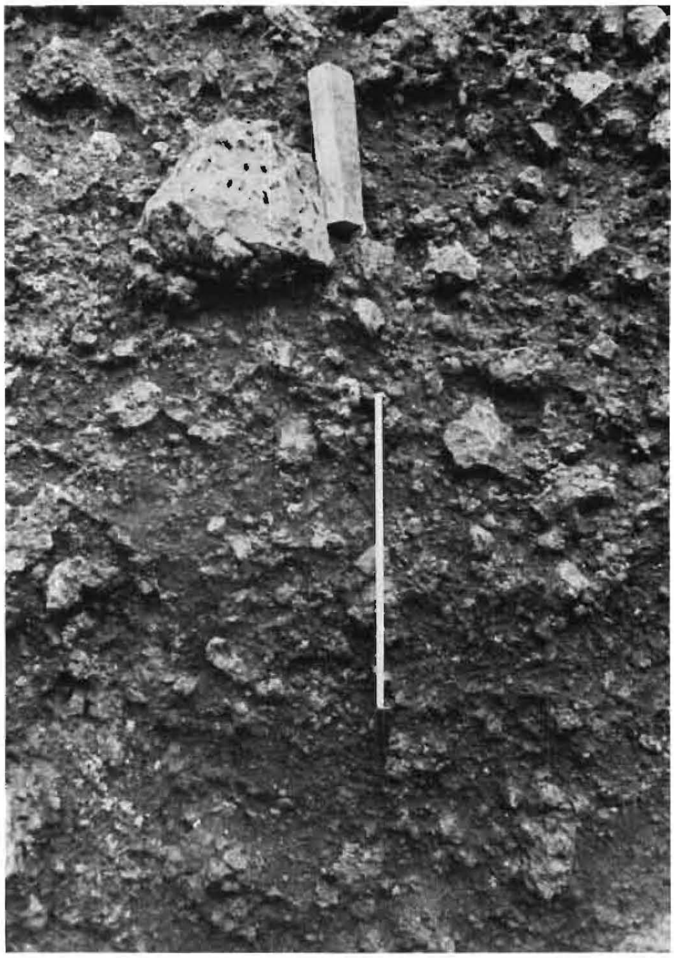
① Right bank of the Tungnaa river with pillow lava, breccia and tuff outcropping



② Pillow lava in Tungnaa left bank, downstream of the waterfalls

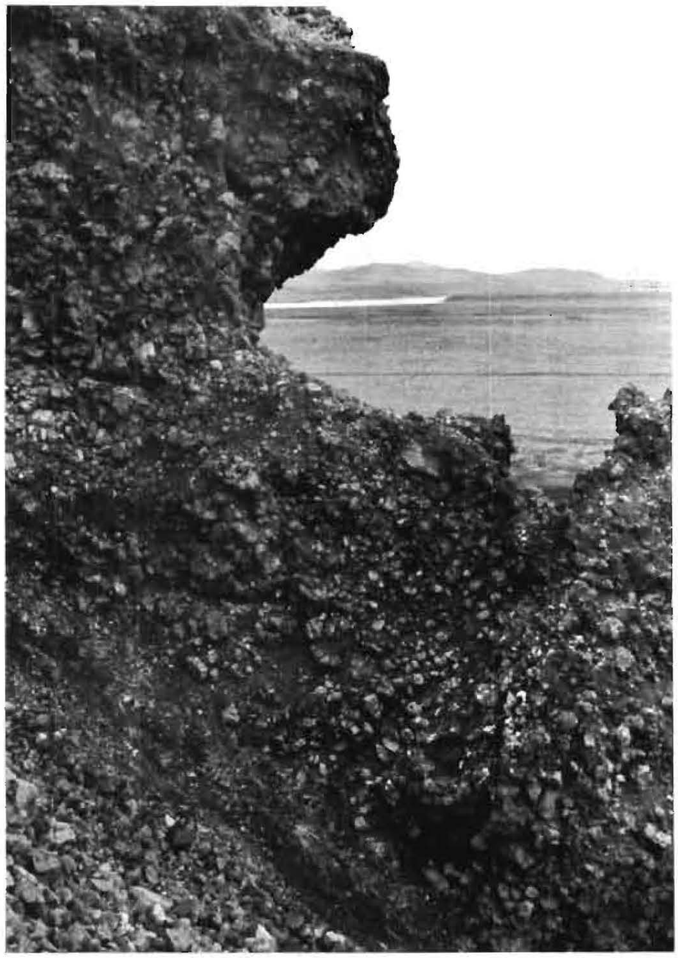
3 Strongly jointed, hard and solid pillow lava formation

4 Pillow lava



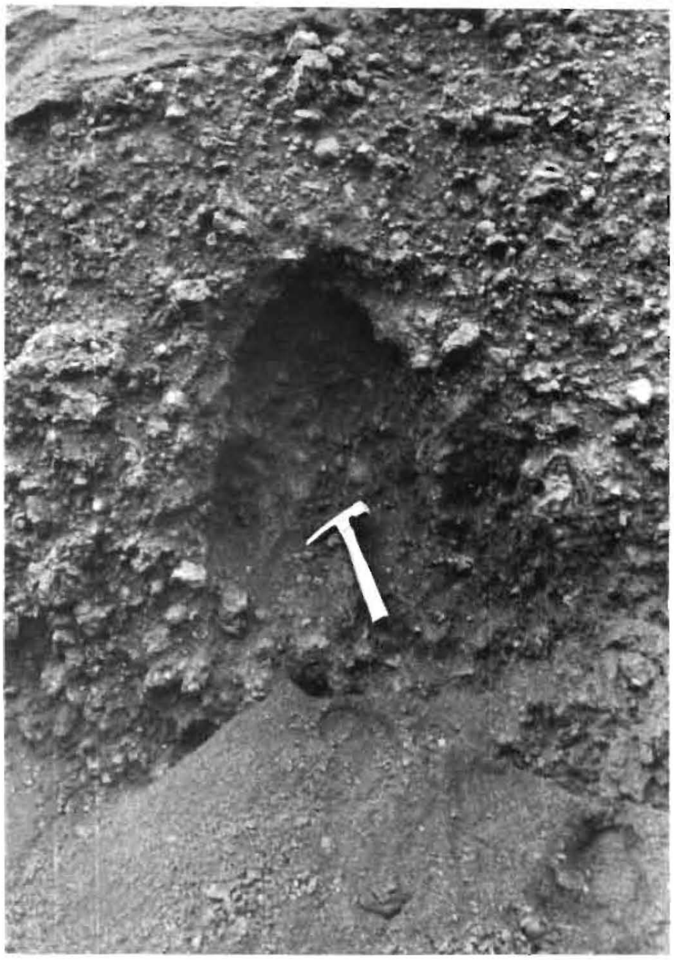
5 Tuff breccia. The upper piece of wood is 20 cm long

6 Tuff and tuff breccia



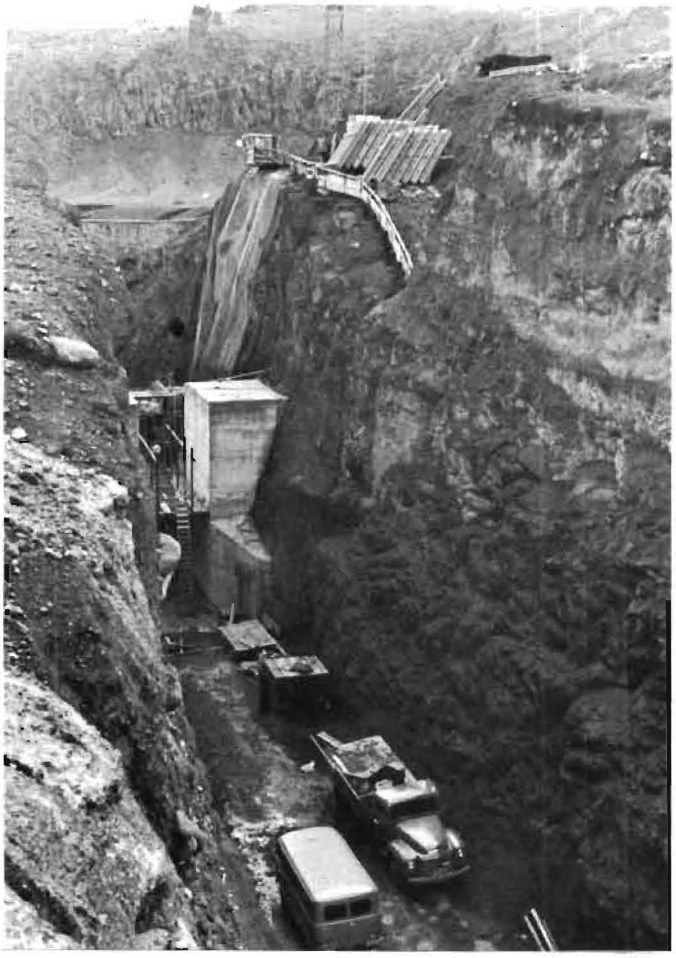
7 Breccia with pillow structure

8 Caves in the Tungnaa right bank, tuff and tuff breccia



⑨ Ripping Test II, breccia with fragments of pillow lava. The tillite cover is visible at the top of the photograph

⑩ Ripping Test I, brecciated pillow lava with a basalt vein



Vatnsfell tuff

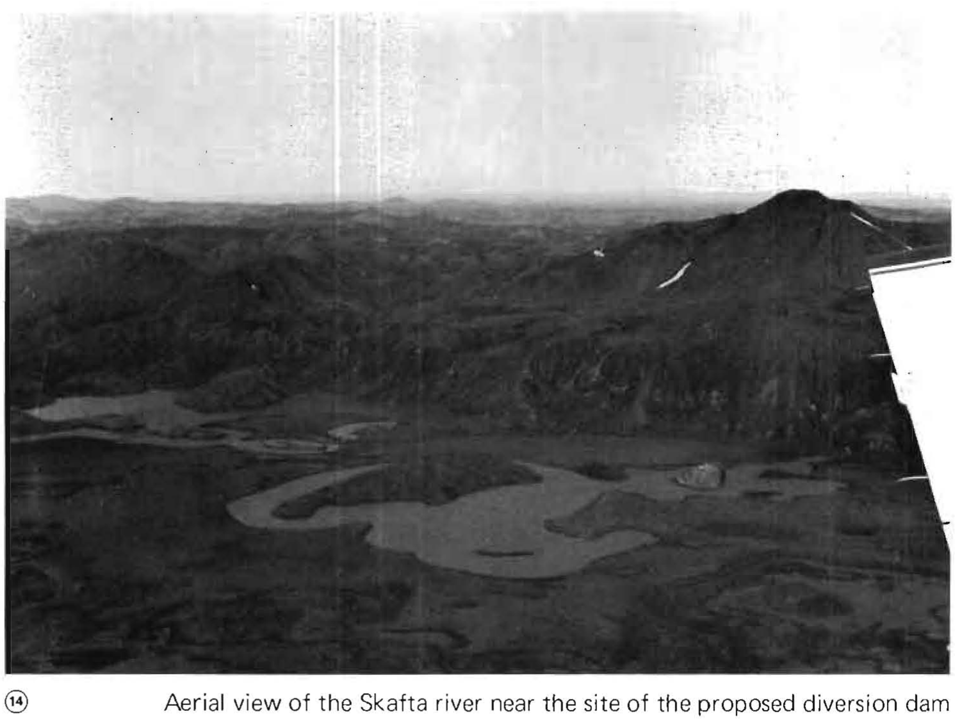
Pillow breccia

(11)
View upstream towards Vatnsfell control structure

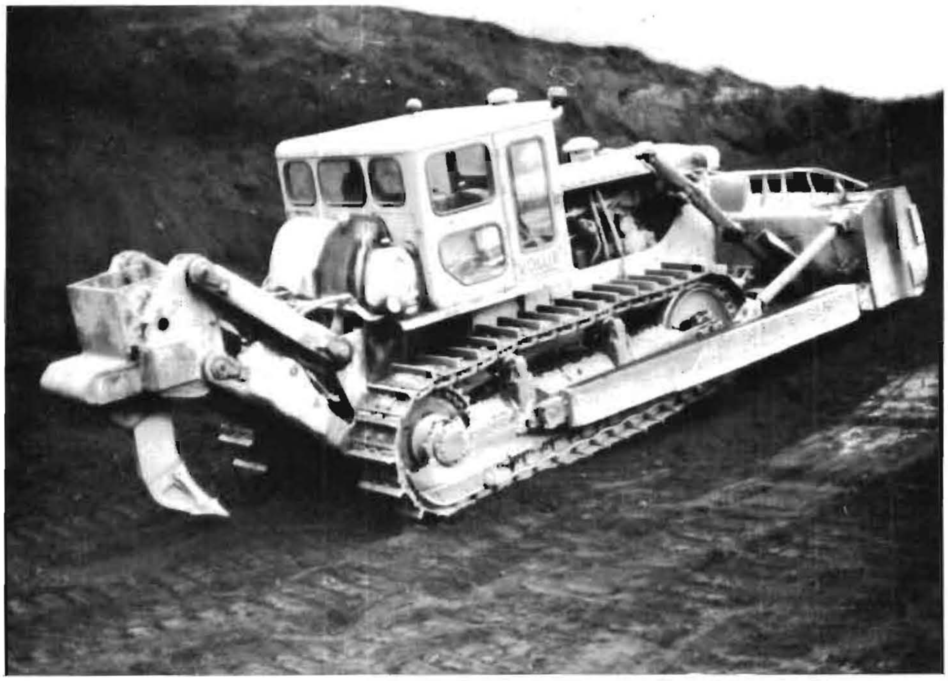
Compact tuff

(12)
The canal wall near the outlet structure

⑬ Moberg landscape near Grettin, near the proposed Skafta diversion. Aerial view NNE towards the Skafta



⑭ Aerial view of the Skafta river near the site of the proposed diversion dam



⑮ Ripping Test II, Cat. D-8 with single shank ripper

⑯ Val S-charl headrace tunnel Engadine hydro plant, Switzerland. Heavy rain from very permeable dolomite, tunnel diameter is 5.70 m

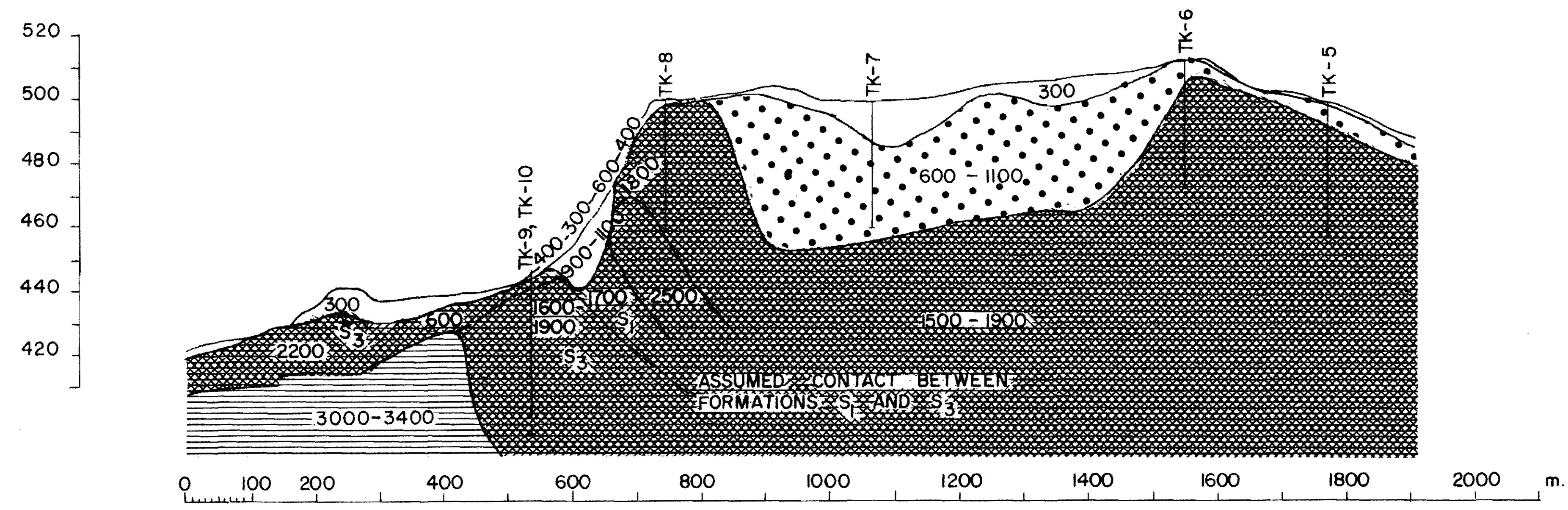
DRAWINGS

A P P E N D I X   2

D R A W I N G S

2-02	Sigalda Project, geological section
2-08	Ripping performance related to seismic velocity
2-09	Typical tunnel cross-sections A,B and C
2-10	Section A I, S = 23.4 m2, H = 5.0 m
2-11	Section A II, S = 23.4 m2, H = 5.0 m
2-12	Section A III, S = 23.4 m2, H = 5.0 m
2-13	Section A IV, S = 23.4 m2, H = 5.0 m
2-14	Section B III, S = 52.2 m2, H = 7.5 m
2-15	Section C III, S = 75.6 m2, H = 9.0 m
2-18	Tunnel construction method
2-33	Circular cross sections A,B and C

SECTION A - A ALONG ℄ OF WATERWAYS (APPROACH CANAL)



MOBERG AT SIGALDA

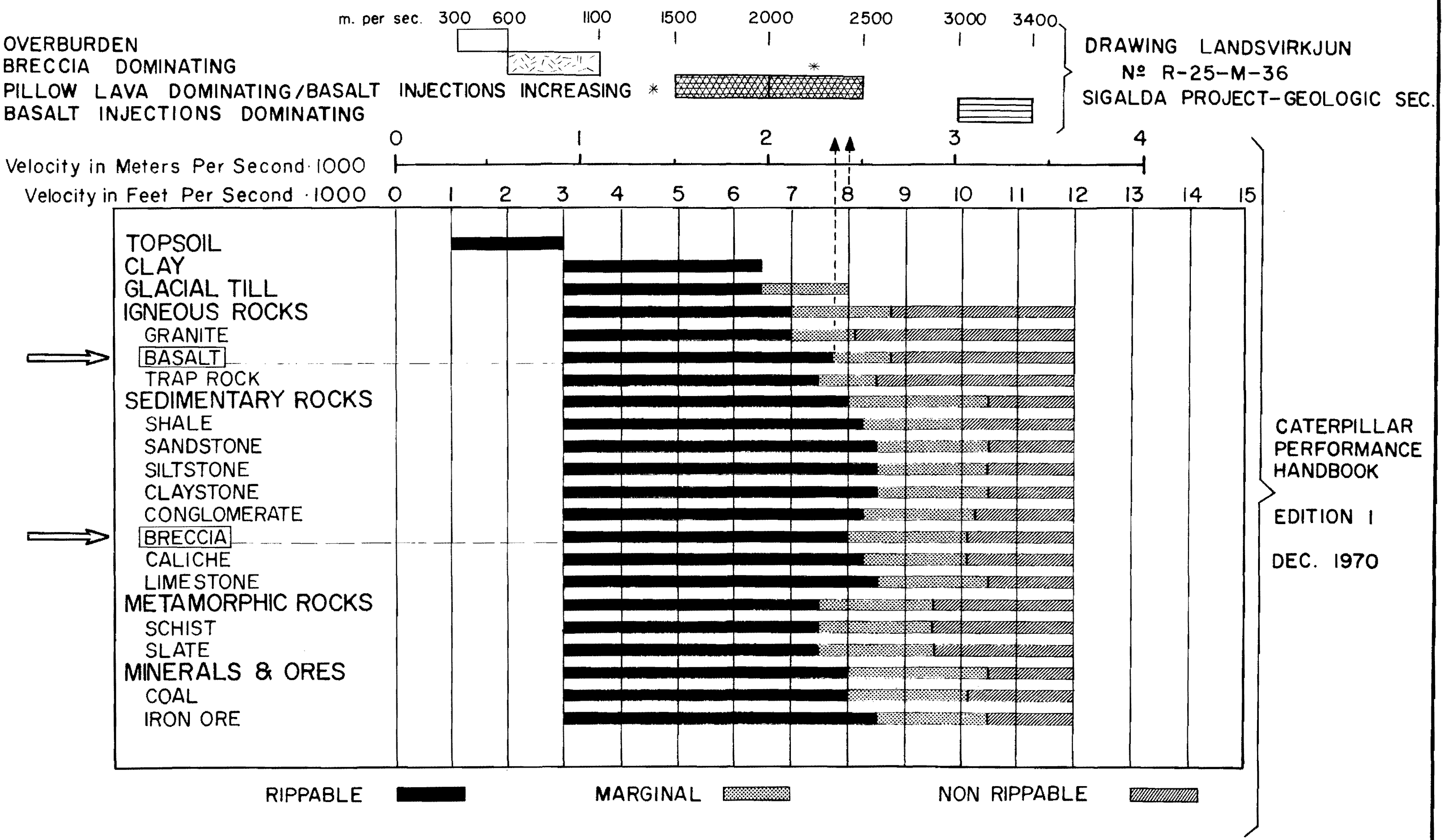
SEISMIC VELOCITY m per sec.		
300-600		OVERBURDEN
600-1100		BRECCIA DOMINATING. ALSO MOBERG IN GENERAL
1500-2000 2000-2500		PILLOW LAVA DOMINATING. BASALT INJECTIONS INCREASING WITH INCREASING SEISMIC VELOCITY.

SEISMIC VELOCITY m per sec		
3000-3400		BASALT INJECTIONS DOMINATING.
		DRILL HOLE LOCATED ALONG SECTION.

FOR LOCATION OF GEOLOGIC SECTION SEE APPENDIX 2-01

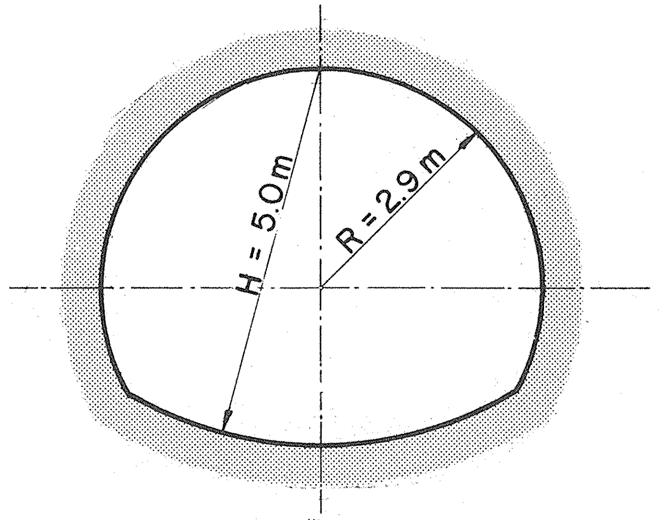
SIGALDA PROJECT
GEOLOGIC SECTION
LANDSVIRKJUN
Nr. R-25-M-36

A	B	C	D	E	F	DATE	15.3.72	APPX 2-02
ELECTRO-WATT	CONSULTING	ENGINEERS			VIRKIR	DESIGN	RAG	DRAWING NUMBER
						APPROVED	Met	1301112852



CATERPILLAR D9G (385 H.P.)–Nº 9 SERIES D
MULTI AND SINGLE SHANK RIPPER
RIPPER PERFORMANCE AS RELATED TO
SEISMIC WAVE VELOCITIES

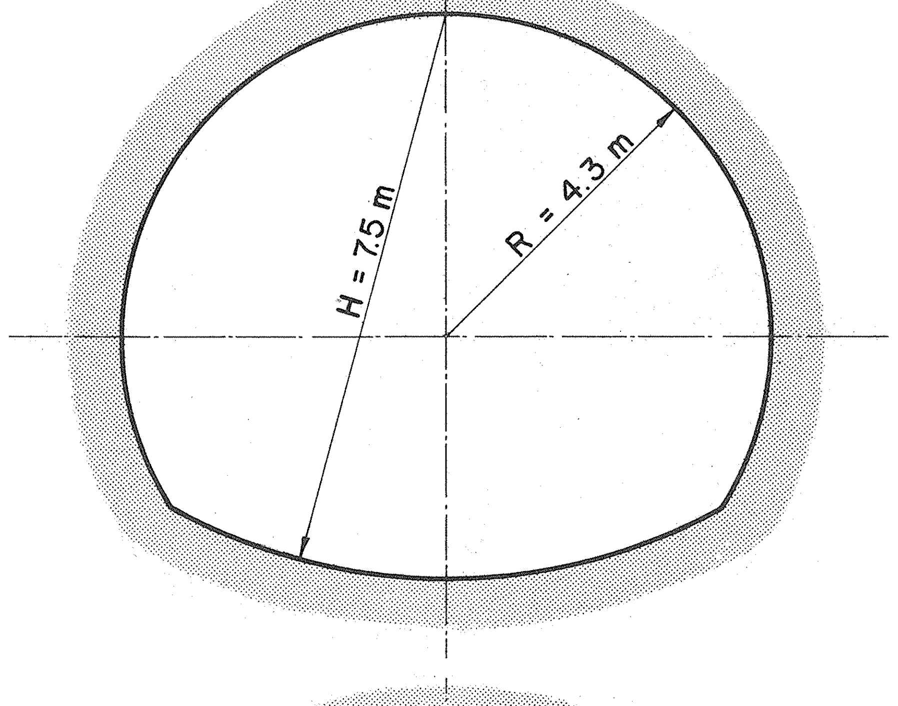
A	B	C	D	E	F	DATE	15.3.72	APPX 2–08
ELECTRO-WATT	CONSULTING ENGINEERS				VIRKIR	DESIGN	RAG	DRAWING NUMBER
						APPROVED	Met	1301 1 1 2858



SECTION A
23.4 m^2

H = 5.00 m

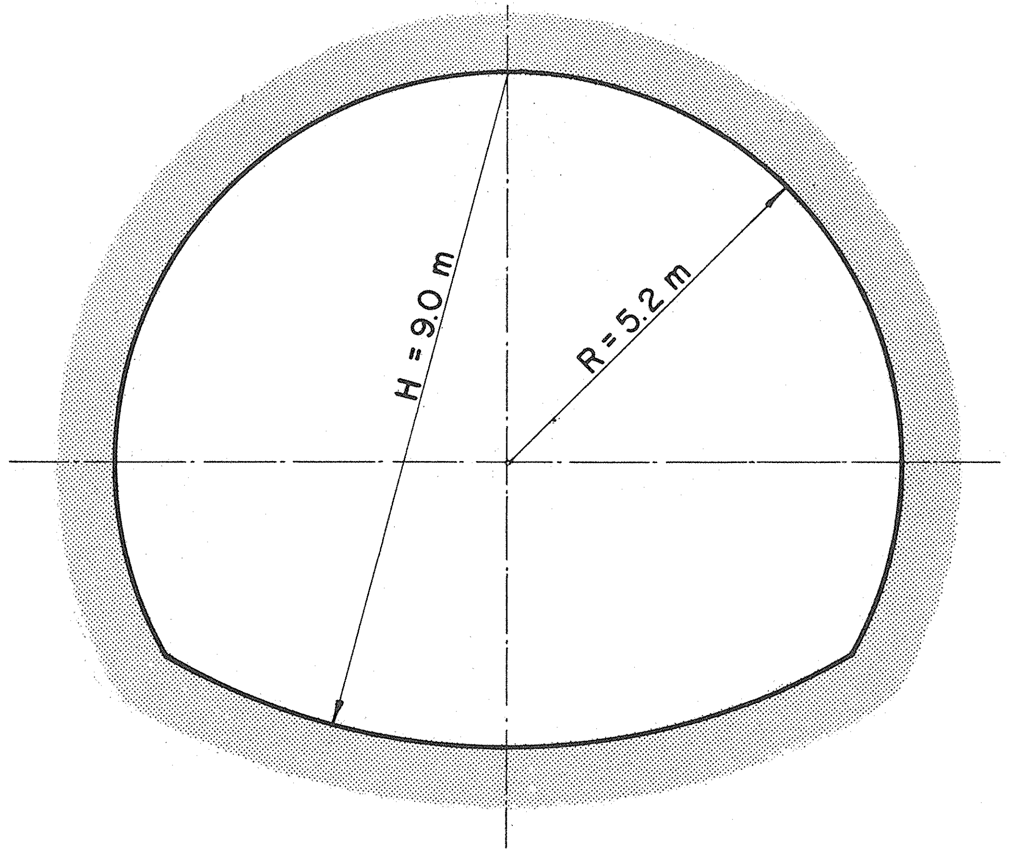
		I	II	III	IV
EXCAVATION TO PAY LINE INCL. DRAINAGE	m^3	28.2	30.3	31.5	29.7
CONCRETE P 300 TO PAY LINE	m^3	1.7	6.1	7.3	4.0
SHOTCRETE TO PAY LINE	m^3	2.9	–	–	–
GUNITE	m^3	–	0.6	0.6	0.6
ROCK SECURITY MATS	m^2	12.8	–	–	–
ANCHORS IF REQUIRED	m	–	–	–	–
SHEETS „BERNOLD" OR EQUIVALENT	m^2	–	12.7	12.7	12.7
STEEL REINFORCEMENT FOR INVERT	kg	34	40	46	53
MORTAR GROUTING	m^3	–	–	–	1.5



SECTION B
52.2 m^2

H = 7.50 m

		I	II	III	IV
EXCAVATION TO PAY LINE INCL. DRAINAGE	m^3	59.7	64.0	66.7	64.7
CONCRETE P 300 TO PAY LINE	m^3	2.3	10.6	13.3	9.1
SHOTCRETE TO PAY LINE	m^3	4.9	–	–	–
GUNITE	m^3	–	0.9	0.9	0.9
ROCK SECURITY MATS	m^2	18.9	–	–	–
ANCHORS IF REQUIRED	m	–	–	–	–
SHEETS „BERNOLD" OR EQUIVALENT	m^2	–	18.8	18.8	18.8
STEEL REINFORCEMENT FOR INVERT	kg	49	66	75	86
MORTAR GROUTING	m^3	–	–	–	2.2



SECTION C
75.6 m^2

H = 9.00 m

		I	II	III	IV
EXCAVATION TO PAY LINE INCL. DRAINAGE	m^3	84.6	90.8	99.9	91.7
CONCRETE P 300 TO PAY LINE	m^3	2.7	13.8	22.9	12.0
SHOTCRETE TO PAY LINE	m^3	6.0	–	–	–
GUNITE	m^3	–	1.1	1.1	1.1
ROCK SECURITY MATS	m^2	22.5	–	–	–
ANCHORS IF REQUIRED	m	–	–	–	–
SHEETS „BERNOLD" OR EQUIVALENT	m^2	–	22.4	22.4	22.4
STEEL REINFORCEMENT FOR INVERT	kg	60	92	106	136
MORTAR GROUTING	m^3	–	–	–	2.7

ORKUSTOFNUN
NATIONAL ENERGY AUTHORITY

TUNNELLING IN MOBERG

TUNNEL
TYPICAL CROSS-SECTIONS
SECTIONS A,B AND C

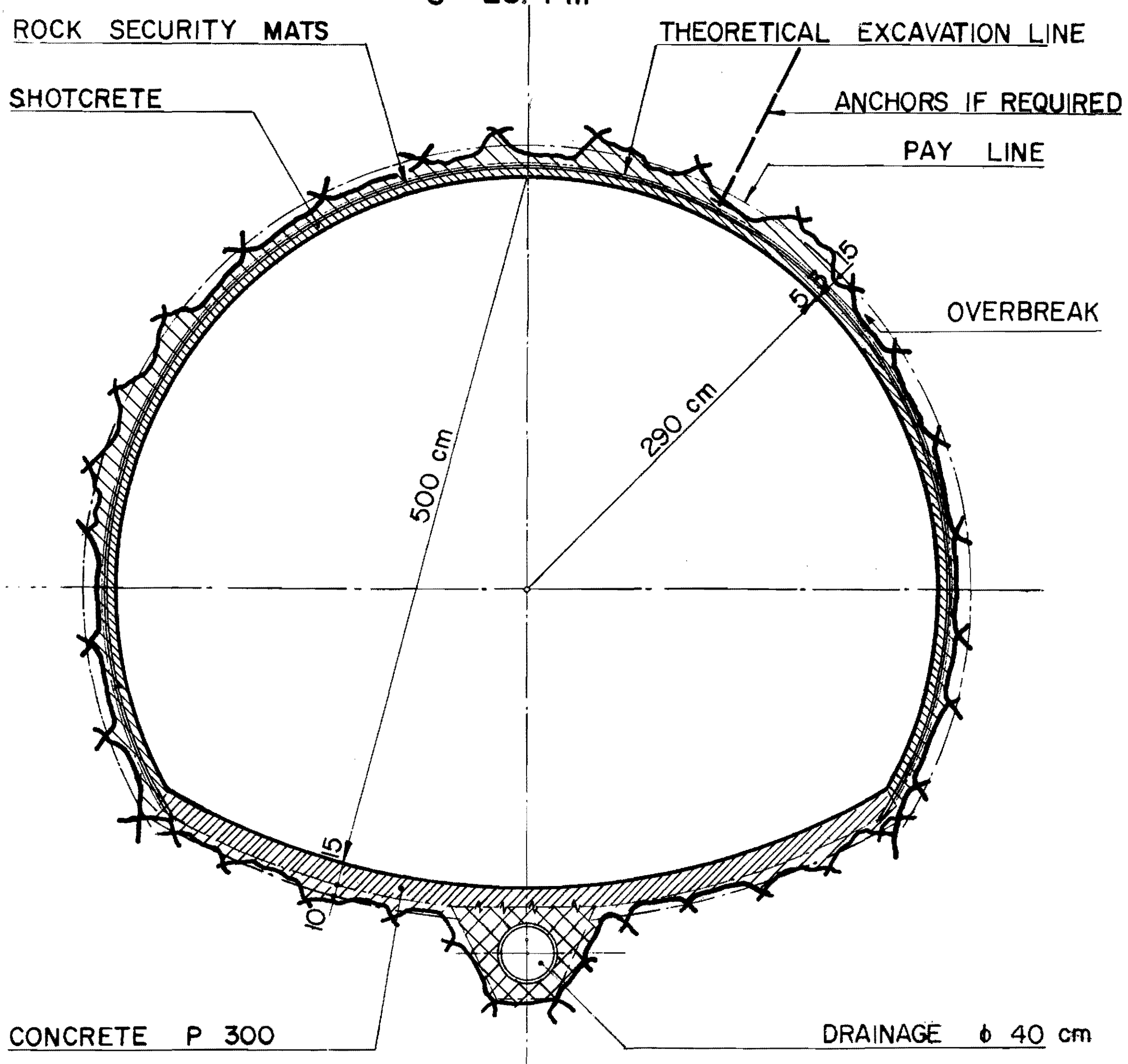
ELECTRO-WATT VIRKIR
CONSULTING ENGINEERS
ZURICH REYKJAVIK

DRA. RAG
APP. Mat

SCALE	DATE	DRAWING NUMBER	APPENDIX
1:100	15.3.72	1301 1 12859	2-09

H = 5 m SECTION A I 1:50

$S = 23.4\ m^2$

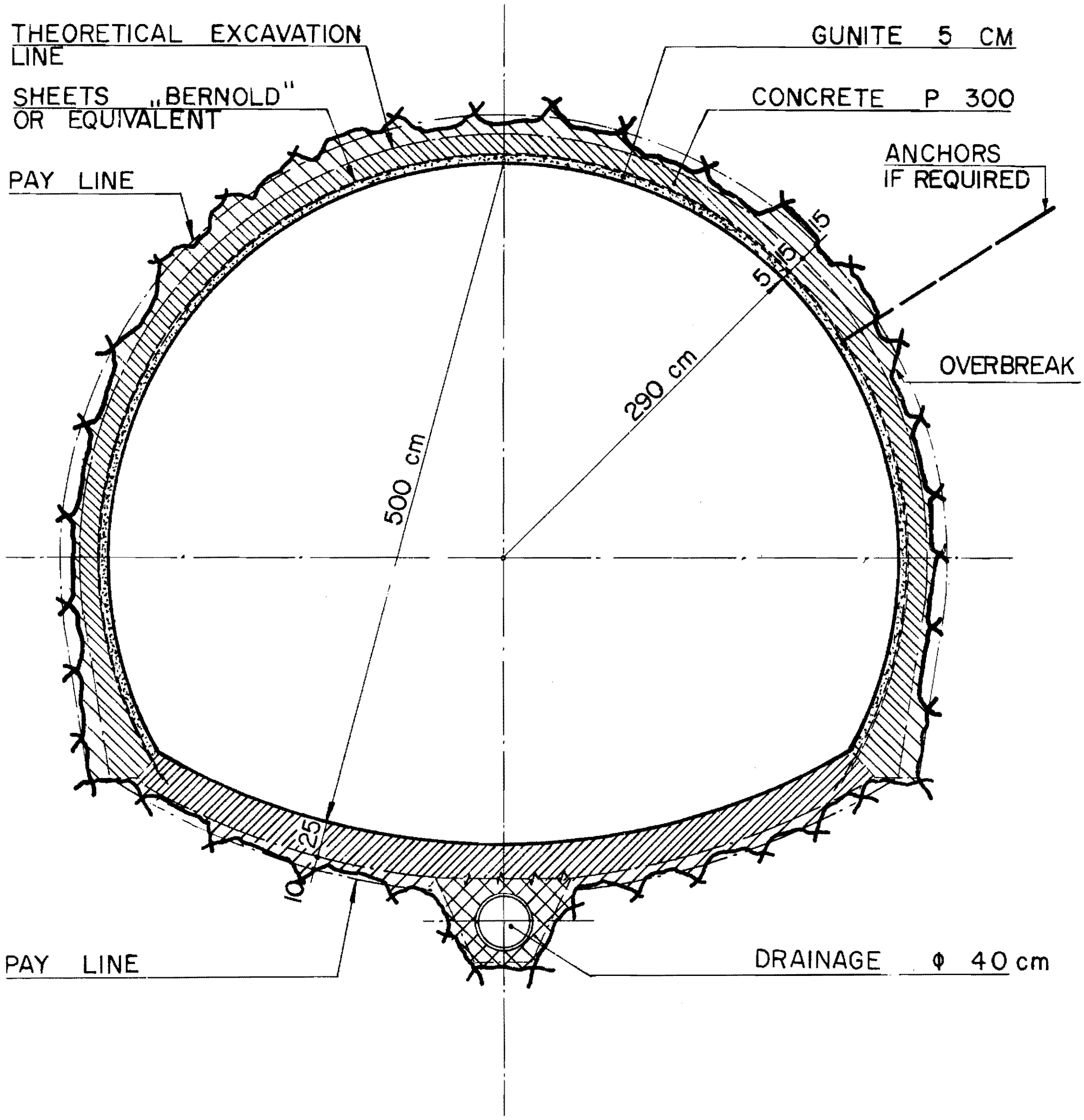


EXCAVATION TO PAY LINE	$28.2\ m^3$
CONCRETE P 300 TO PAY LINE	$1.7\ m^3$
SHOTCRETE TO PAY LINE	$2.9\ m^3$
GUNITE	$-\ m^3$
ROCK SECURITY MATS	$12.8\ m^2$
ANCHORS IF REQUIRED	$-\ m^1$
SHEETS „BERNOLD" OR EQUIVALENT	$-\ m^2$
STEEL REINFORCEMENT FOR INVERT	34 kg
MORTAR GROUTING	$-\ m^3$

A	B	C	D	E	F	DATE	15.3.72	APPX 2-10
ELECTRO-WATT	CONSULTING ENGINEERS			VIRKIR		DESIGN	RAG	DRAWING NUMBER
						APPROVED	Met	1301 1 12860

H = 5 m SECTION A II 1:50

S = 23.4 m²

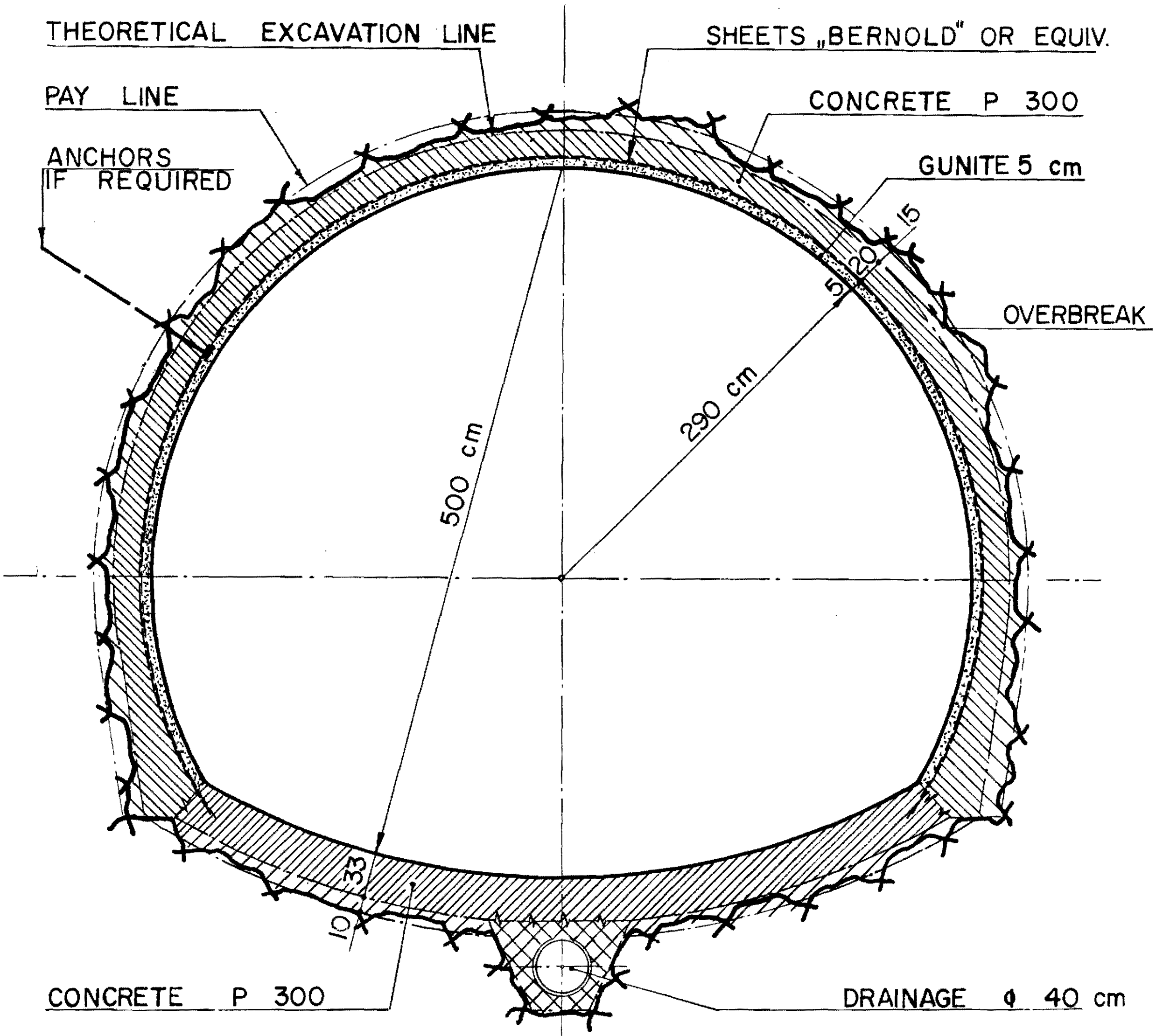


EXCAVATION TO PAY LINE	30.3 m^3
CONCRETE P 300 TO PAY LINE	6.1 m^3
SHOTCRETE TO PAY LINE	- m^3
GUNITE	0.6 m^3
ROCK SECURITY MATS	- m^2
ANCHORS IF REQUIRED	- m^1
SHEETS „BERNOLD" OR EQUIVALENT	12.7 m^2
STEEL REINFORCEMENT FOR INVERT	40 kg
MORTAR GROUTING	- m^3

A	B	C	D	E	F	DATE	15.3.72	APPX 2-11
ELECTRO-WATT CONSULTING ENGINEERS VIRKIR						DESIGN	RAG	DRAWING NUMBER
						APPROVED	Met	1301 1 1286 1

H = 5 m SECTION A III 1:50

S = 23.4 m^2



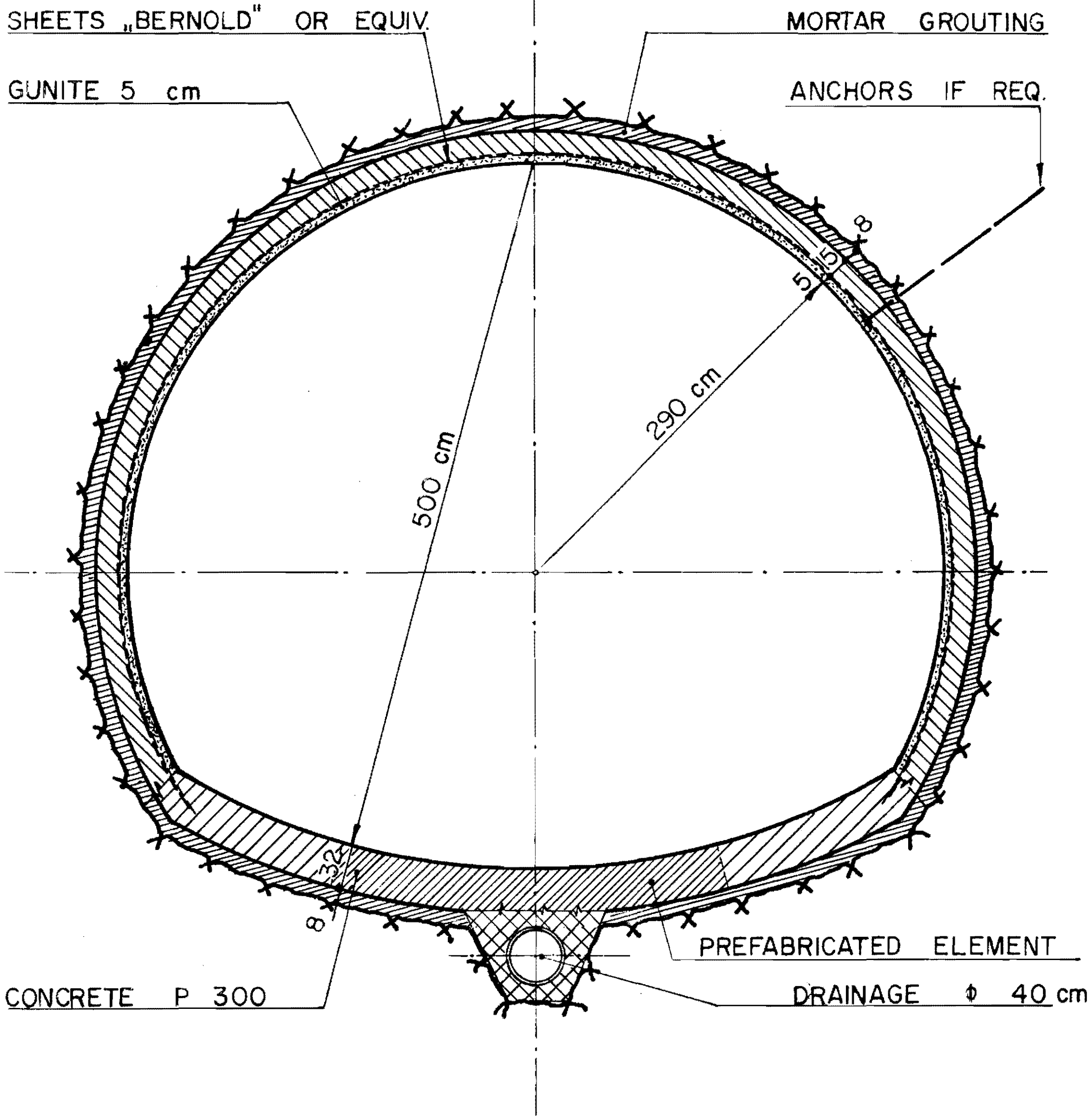
EXCAVATION TO PAY LINE	31.5 m^3
CONCRETE P 300 TO PAY LINE	7.3 m^3
SHOTCRETE TO PAY LINE	- m^3
GUNITE	0.6 m^3
ROCK SECURITY MATS	- m^2
ANCHORS IF REQUIRED	- m^l
SHEETS „BERNOLD" OR EQUIVALENT	12.7 m^2
STEEL REINFORCEMENT FOR INVERT	46 kg
MORTAR GROUTING	- m^3

A	B	C	D	E	F	DATE	15.3.72	APPX 2-12
ELECTRO-WATT CONSULTING ENGINEERS VIRKIR						DESIGN	RAG	DRAWING NUMBER
						APPROVED	Met	1301 112862

ADVANCE WITH STEEL POLING PLATES

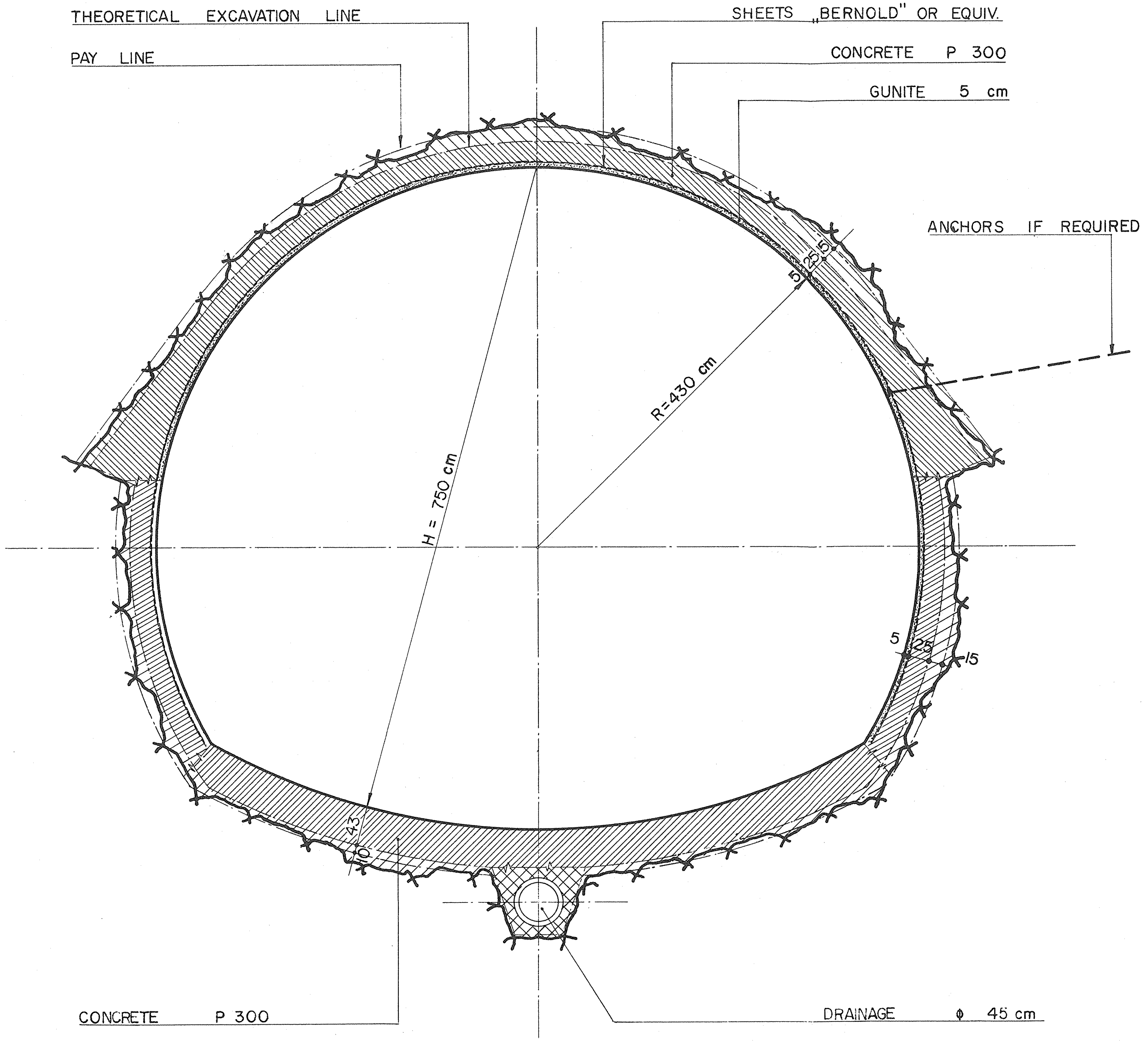
H = 5 m SECTION A IV 1:50

S = 23.4 m^2



EXCAVATION TO PAY LINE	29.7 m^3
CONCRETE P 300 TO PAY LINE	4.0 m^3
SHOTCRETE TO PAY LINE	– m^3
GUNITE	0.6 m^3
ROCK SECURITY MATS	– m^2
ANCHORS IF REQUIRED	– m^l
SHEETS „BERNOLD" OR EQUIVALENT	12.7 m^2
STEEL REINFORCEMENT FOR INVERT	53 kg
MORTAR GROUTING	1.5 m^3

A	B	C	D	E	F	DATE	15.3.72	APPX 2-13
ELECTRO-WATT	CONSULTING ENGINEERS	VIRKIR				DESIGN	RAG	DRAWING NUMBER
						APPROVED	Met	1301 112863



SECTION B III $S = 52.2\ m^2$ $H = 7.5\ m$ 1 : 50

EXCAVATION TO PAY LINE INCL. DRAINAGE	66.7 m^3
CONCRETE P 300 TO PAY LINE	13.3 m^3
SHOTCRETE TO PAY LINE	– m^3
GUNITE	0.9 m^3
ROCK SECURITY MATS	– m^2
ANCHORS IF REQUIRED	– m^1
SHEETS „BERNOLD" OR EQUIVALENT	18.8 m^2
STEEL REINFORCEMENT FOR INVERT	75 kg
MORTAR GROUTING	– m^3

ORKUSTOFNUN
NATIONAL ENERGY AUTHORITY

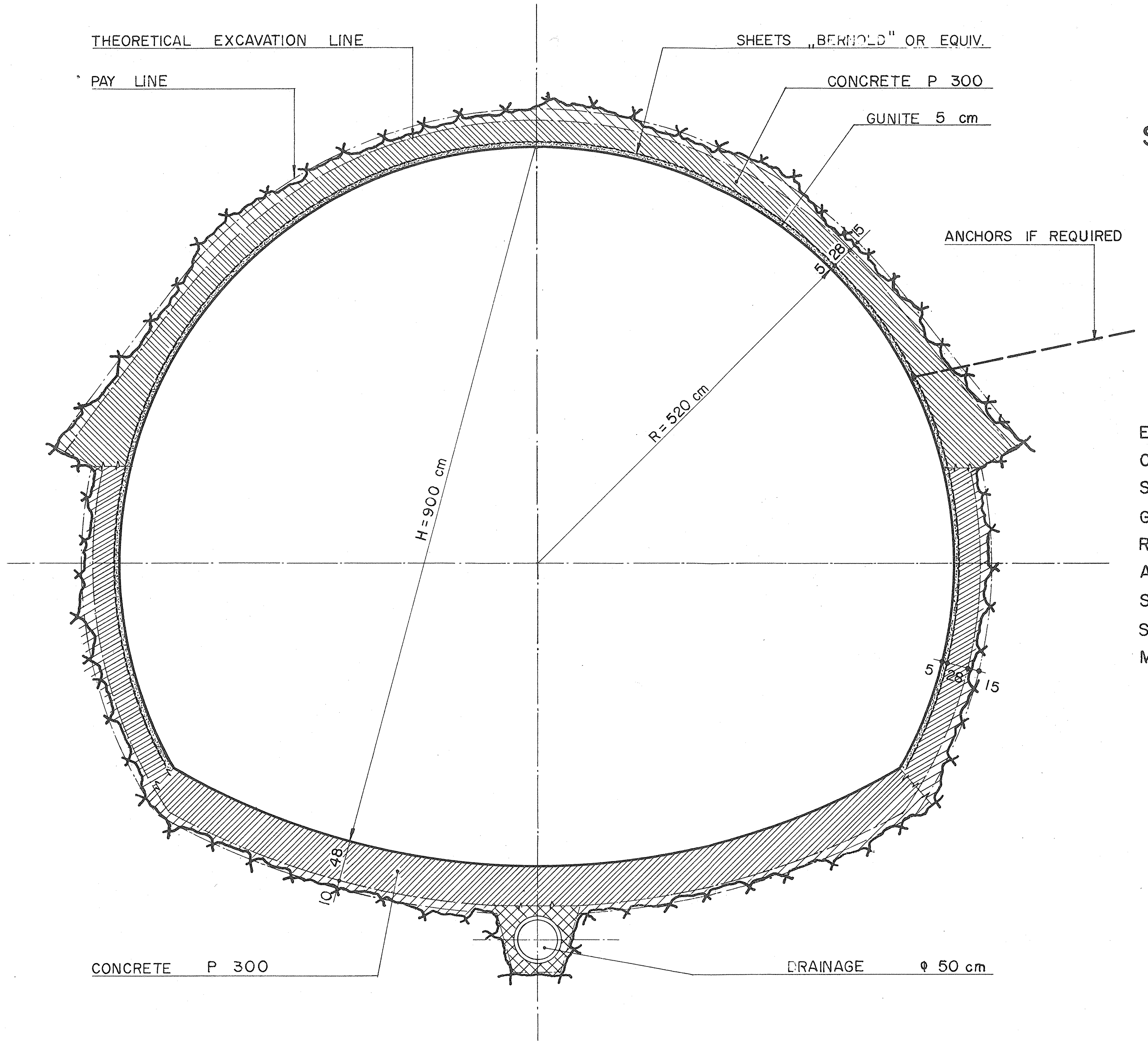
TUNNELLING IN MOBERG

SECTION B III
$S = 52.2\ m^2$
$H = 7.5\ m$

ELECTRO-WATT CONSULTING ENGINEERS ZURICH — VIRKIR REYKJAVIK

IND.	DATE	CHE.
J		
H		
G		
F		
E		
D		
C		
B		
A		
DRA.	RAG	
CHE.		
APP.	Mef	

SCALE	DATE	DRAWING NUMBER	APPENDIX
1:50	15.3.72	1301112864	2–14



SECTION C III S = 75.6 m² 1 : 50
H = 9.0 m

EXCAVATION TO PAY LINE INCL. DRAINAGE	99.9 m³
CONCRETE P 300 TO PAY LINE	22.9 m³
SHOTCRETE TO PAY LINE	– m³
GUNITE	1.1 m³
ROCK SECURITY MATS	– m²
ANCHORS IF REQUIRED	– m
SHEETS „BERNOLD" OR EQUIVALENT	22.4 m²
STEEL REINFORCEMENT FOR INVERT	106 kg
MORTAR GROUTING	– m³

ORKUSTOFNUN
NATIONAL ENERGY AUTHORITY

TUNNELLING IN MOBERG

SECTION C III
S = 75.6 m²
H = 9.0 m

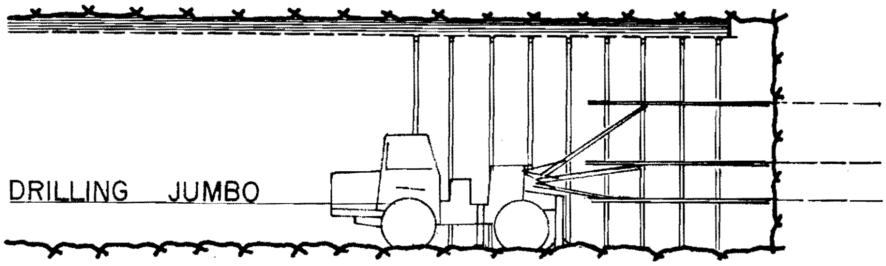
ELECTRO-WATT VIRKIR
CONSULTING ENGINEERS
ZURICH REYKJAVIK

SCALE	DATE	DRAWING NUMBER	APPENDIX
1:50	15.3.72	1301112865	2–15

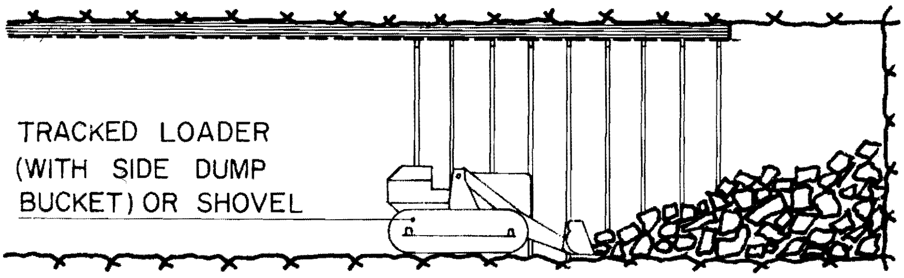
IND.	DATE	CHE.
J		
H		
G		
F		
E		
D		
C		
B		
A		
DRA.	RAG	
CHE.		
APP.	MGf	

TUNNEL CONSTRUCTION METHOD

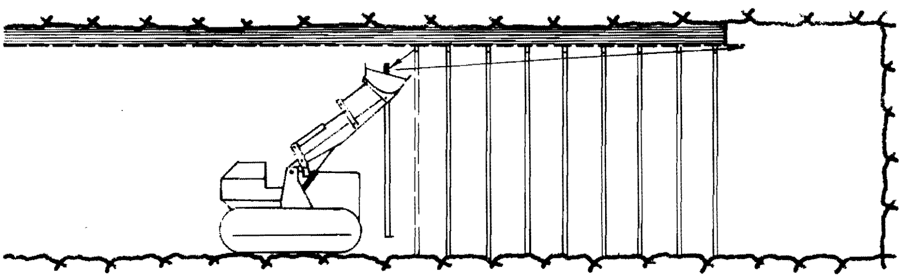
(1) DRILLING AND BLASTING



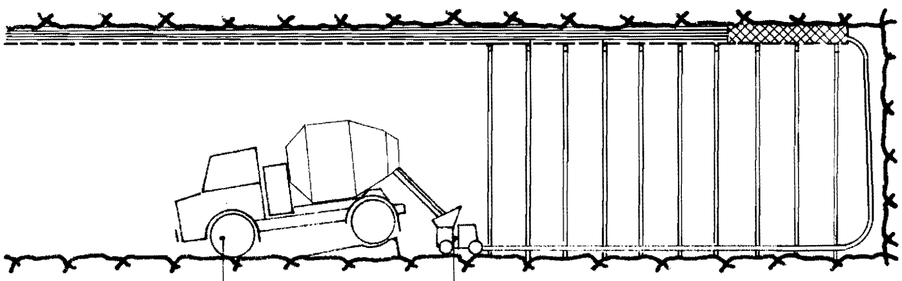
(2) MUCKING-OUT



(3) ERECTION OF STEEL FRAMES AND BERNOLD SHEETS OR EQUIVALENT



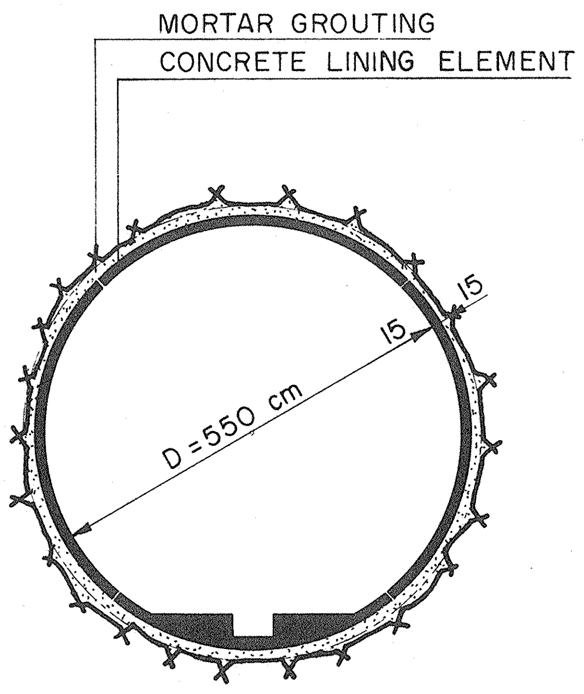
(4) CONCRETING



A	B	C	D	E	F	DATE	15.3.72	APPX 2-18
ELECTRO-WATT	CONSULTING ENGINEERS			VIRKIR		DESIGN	RAG	DRAWING NUMBER
						APPROVED	Met	1301 11 2868

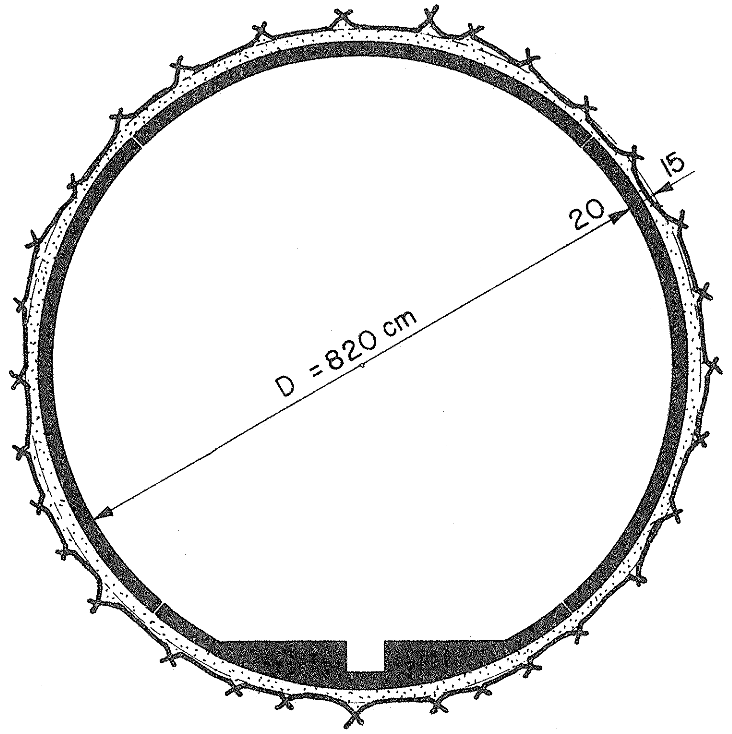
SECTION (A)

S = 23.2 m²



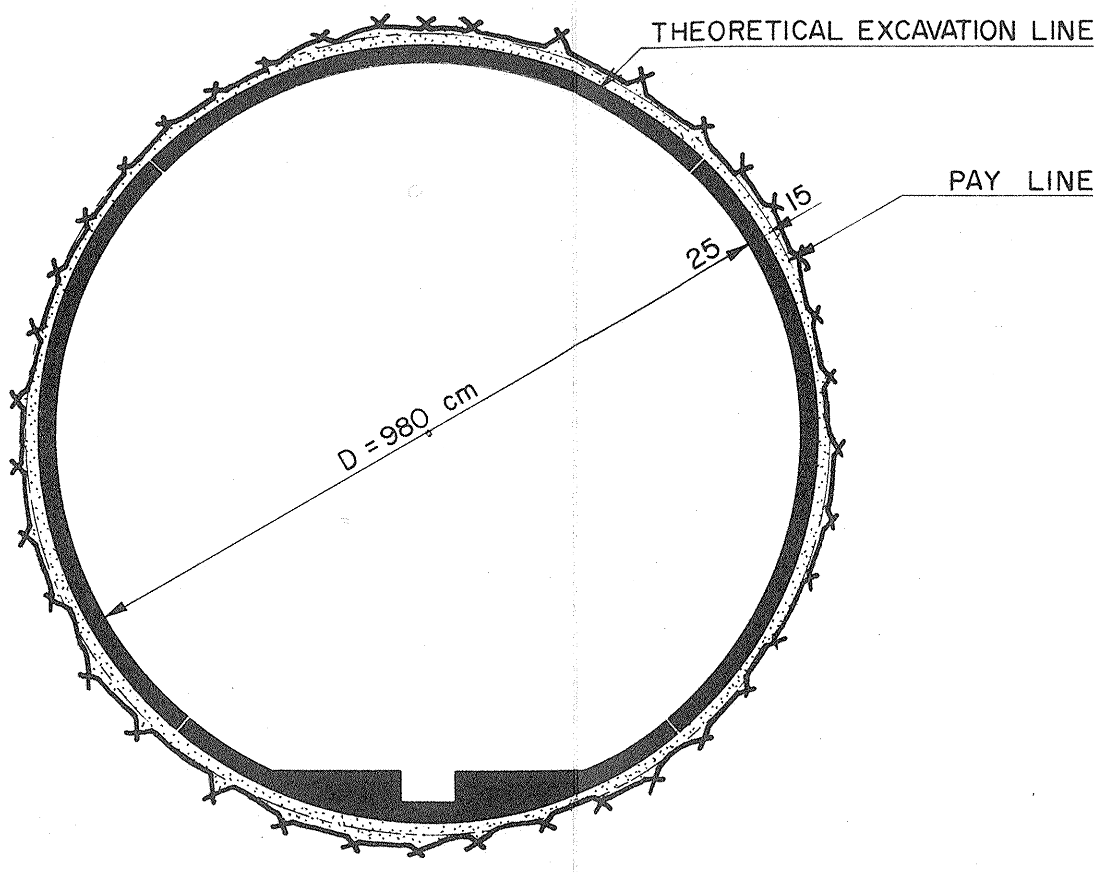
SECTION (B)

S = 51.9 m²

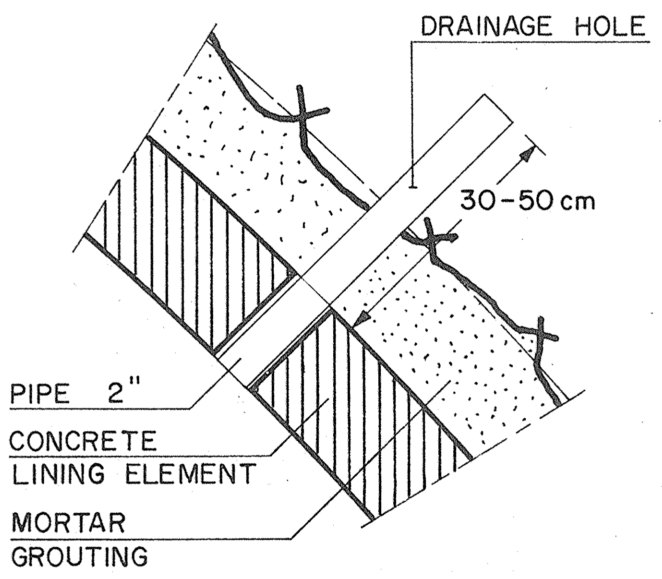


SECTION (C)

S = 74.4 m²



DETAIL OF A DRAINAGE HOLE



NOTE:

- DRILLING OF DRAINAGE HOLE APPROX. ONE DAY AFTER MORTAR GROUTING
- WHEN DRILLING OF DRAINAGE HOLE IN THE ROCK IS NOT POSSIBLE, THE METHOD SHOWN ON APPENDIX 2-27 MUST BE EMPLOYED.

	Section A	Section B	Section C
EXCAVATION TO PAY LINE	29.2 m³	62.2 m³	88.2 m³
CONCRETE ELEMENT	2.7 m³	5.3 m³	7.9 m³
MORTAR GROUTING	2.8 m³	4.1 m³	4.9 m³

IF IT IS NOT POSSIBLE TO LOWER THE GROUNDWATER LEVEL, A SECOND LINING WILL BE NECESSARY FOR A WATER HEAD OF 50 m, IN THIS CASE THE FOLLOWING ADDITIONAL QUANTITIES WILL BE NECESSARY:

	Section A	Section B	Section C
CONCRETE P 300	4,1 m³	7.4 m³	10.4 m³
STEEL REINFORCEMENT	130 kg / m'	195 kg / m'	265 kg / m'

ORKUSTOFNUN
NATIONAL ENERGY AUTHORITY

TUNNELLING IN MOBERG

CIRCULAR TUNNEL SECTIONS
SECTIONS A,B and C

ELECTRO-WATT CONSULTING ENGINEERS ZURICH — VIRKIR REYKJAVIK

IND.	DATE	CHE.
J		
H		
G		
F		
E		
D		
C		
B		
A		

DRA. RAG
CHE.
APP. Met

SCALE	DATE	DRAWING NUMBER	APPENDIX
1:100/10	15.3.72	1301 1 1 2883	2-33

DOCUMENTATION

AUTOROUTE DU LÉMAN

Société générale pour l'Industrie Lausanne

C. Zschokke — H.R. Schmalz SA

TUNNEL-TUNNELLING

SYSTEM BERNOLD

Contents

Bibliography

(1) Rabcewicz, L. v.: Aus der Praxis des Tunnelbaus. Einige Erfahrungen über echten Gebirgsdruck (Geol. und Bauw. 27, H. 3–4, 1961);

(2) Sattler, K.: Neuartige Tunnelmodellversuche – Ergebnisse und Folgerungen (Felsmechanik und Ing. Geologie Suppl. IV, 1968);

(3) Thürlimann, B.: Plastische Berechnungsmethoden;

(4) Schulze, H. und Duddeck, H.: Statische Berechnung schildvorgetriebener Tunnel. (Festschrift Beton- und Monierbau AG 1889–1964);

(5) Kastner, H.: Statik des Tunnel- und Stollenbaus (Springer-Verlag 1962);

(6) Felsmechanik & Ing. Geologie Suppl. I 1964, Beiträge: F. Pacher, L. v. Rabcewicz;

(7) Wöhlbier, H. und Natau: Der Ausbau unterirdischer Hohlräume mit S und A Bleche System Bernold, Bergbauwissenschaften 16 (1969), S. 117–126.

1970 Edition

Concrete Lining in Tunnel Constructions according to the Bernold System

Jean Bernold, Civil Engineer

I. Introduction

An ideal instance of tunnel construction would be to simultanecusly excavate the profile and put up the final concrete lining. This can be realized in modern gallery- and tunnel construction by means of new technical equipment.

Although excavating and concreting cannot be carried out in one single process, the time gap between those two operations has been so thoroughly reduced that they can be finished while rock pressures still equal zero or at least are very low.

In any kind of rock (with the exception of boggy rock in which water pressure is decisive) a certain space can remain open for some time without requiring propping, i. e. rock pressure does not occur and the fresh lining is not stressed. The slackening zones around the cavity are forming gradually and advancing towards the inner part of the mountain. Quick lining with closely fitting concrete prevents or interrupts those movements. It may therefore be said that rock pressure increases from zero on with the progress of the excavation (Wiedermann 1948).

Kastner (1962) also writes about the subject: It takes some time for the pressure to develop, i. e. as a rule the plastic deformations of the rock only start some time after the slackening caused by the excavation. If a concrete lining of adequate thickness could be produced within this time range, we should be able to work faster than with any other known construction method.

Experience has taught us that Wiedermann's and Kastner's opinion on rock pressure is correct and that we can rely upon a minimum temporary stability in any kind of rock, except boggy rock.

Although in the course of the last 15 years steel linings have successfully supplanted the traditional timber linings, new experiments have been made during the last years, trying to introduce a more economical method of tunnel lining, the ring-construction-method. This method which uses embedded steel arches or lattice girders, wire-mats and sprayed mortar, has afforded good practical results (Schwaikheimer tunnel and Autostrada tunnels). However, it did not prove economical enough to supplant steel linings.

It is the Bernold System, concrete lining in tunnel constructions with boarding and reinforcing metal sheets, all patents for wich are in the author's possession, which has first succeeded in replacing conventional building methods as described above.

II. The Boarding and Reinforcing Metal Sheet according to the Bernold System

1. Technical Data

a) Sizes

Standard size 1200 x 1080 mm
Sheet thickness 1, 2, 3, to 5 mm

The sheets are bent to fit the tunnel radius exactly and are locked by means of connection-rods, which guarantee a quick assembly.

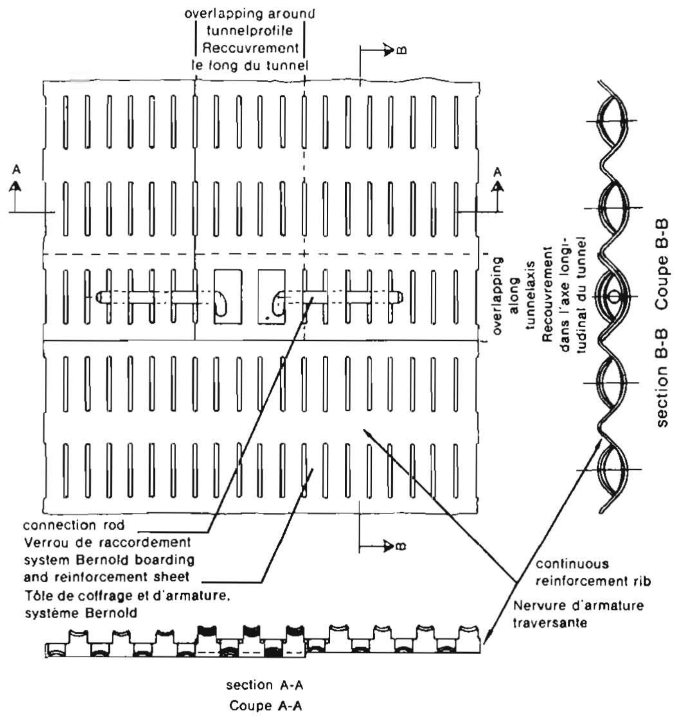


Fig. 1
Bernold System boarding and reinforcement sheet
Tôle de coffrage et d'armature, système Bernold

b) Weights

Table 1

Sheet thickness	1 mm	2 mm	3 mm	5 mm
1 standard sheet (1.08 x 1.20 = 1.296 m2)	10.5 kg	21.0 kg	31.4 kg	52.4 kg
Total weight incl. connection rod per m2 app.	11.0 kg	21.0 kg	31.0 kg	52.0 kg

The exact weight per running meter and per m² can be determined as soon as the cross section of the tunnel is known.

c) Cross sections of reinforcements

The following (Fe-) steel cross sections may be considered as concrete-reinforcement.

Table 2

Sheet thickness	1 mm	2 mm	3 mm	5 mm
Steel cross section of one reinforcement rib	0.57 cm²	1.09 cm²	1.62 cm²	2.7 cm²
Diameter of a steel rod with the same cross section	8.5 mm	11.8 mm	14.4 mm	18.5 mm
Distance between ribs	12 cm	12 cm	12 cm	12 cm
Fe (iron) per meter with an overlap of 12 cm	5.3 cm²	10.2 cm²	15.2 cm²	25.2 cm²

2. Use of the Sheet as Boarding

Their special shape makes the Bernold sheets suitable for boarding. On the one hand the perforated and deformed sheets are rigid enough to resist the boarding pressure, on the other hand tight enough to prevent leakage of a concrete of stiff-plastic consistency. For the concrete filling in method, which has been most frequently used up to now, the concrete should have a grain of 0—30 mm, 250 — 300 kg of cement and a water/cement ratio of 0.4 to 0.5. After filling, the concrete is vibrated with a dip-vibrator in order to obtain a good bond of rock, concrete and sheet. By means of appropriate additives it is possible to produce a concrete that is absolutely waterproof.

In the spraying-mortar-through-method the sheet also acts as perforated boarding.

Besides the sheet has proved an excellent support for sprayed concrete. This is especially advantageous when the Bernold sheets are being used for mechanically excavated tunnels.

3. Use of the Sheet as Reinforcement

The cross section of the longitudinal, V-shaped ribs in the sheet may be considered as reinforcement (compare with table 2). The EMPA (Eidgenössische Material- und Versuchsanstalt) in Zurich has made several tests on this matter.
The EMPA test-reports nrs. 59 392, 60 410 and 67 839 have given full confirmation regarding the suitability of the Bernold sheets as reinforcement. Owing to the larger surface and the good locking, the bond between concrete and sheet is much better than with rod-irons.
In spite of maximum strain occurring in rock that is very friable or under stress, there are no visible cracks or other damages of any kind to be found in any of the jobs done with the concrete lining according to the Bernold System. Measurements show a maximum crown deflection of 1/300 of the span. These deformations are small and even desirable for the convenient rearrangement of loads.

III. Rock Pressure

Load is at the base of the calculation of rock pressure. The pressures which arise in a mountain cannot be accurately calculated and one must therefore rely upon estimates, as the vibrations caused by the excavation and blastings destroy the balance of the rock and it is no longer homogeneous.
In order to determine the method to be used we have classified rocks as follows:

I.	Slightly friable rock	(Shearing strength larger than tangential tensions in the range of the excavation zone).
II.	Friable rock	
III.	Very friable rock	Shearing strength smaller than the tangential tensions in the range of the excavation zone).
IV.	Rock under stress	

Experience and measurements show that the following values for temporary stabilty can be safely reckoned with:

I.	Slightly friable rock	24—48 hours
II.	Friable rock	8—18 hours
III.	Very friable rock	4—12 hours
IV.	Rock under stress	0— 2 hours

As the length of excavation is determined by the quality of the rock, the span from the last concrete vault to the drilling front will be such that the above temporary stabilities may be observed.

IV. The Bernold System and it's Application

1. The general principle

The concrete lining construction according to the Bernold System consists of a concrete lining which fits the rock and being well vibrated clings to the excavation profile **without hollows.** It thus prevents further destruction of the rock through alien influences such as air and water and it diverts the loads from the less resisting rocks to the stronger rock formations. It secures the excavation zone in a very little time and in accordance with rock pressure and construction method.
The boarding and reinforcement sheets, which are supported by the fitting arches prevent the concrete lining from cracking during blasting processes as often happens with sprayed-concrete-securing for instance.
This method of putting up a concrete lining in one single working process on the job site itself with absorption of rock pressures by the homogeneous structure of rock/concrete boarding, reinforcement and fitting arches and with-

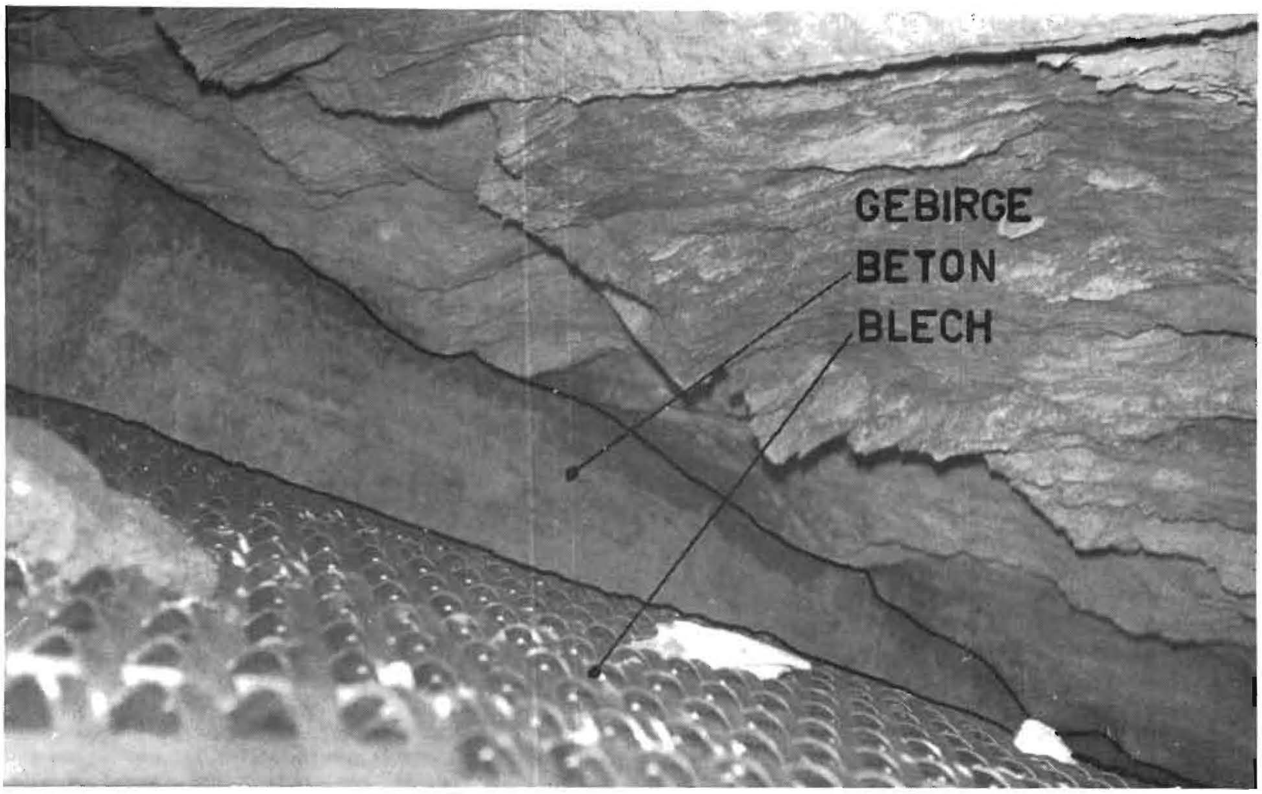


Fig. 2 ROCK - CONCRETE - SHEET / ROCHE - BETON - TOLE

out causing the slightest interruption in the excavation process, has made a successful entry in tunneling in the course of two years only.
During this period the method has been tested as to technical, statical and economical qualities on 42 buildings sites in Switzerland, Germany, France, Italy, Norway, Austria and Algeria.

vantages and possibilities of this new tunneling system, building sites may be visited all over Europe.
In Austria, Germany, Switzerland and France the first highway tunnels and army buildings have already been tendered for submission and inquiries from various countries have come in for large projects in which millions could be saved with the Bernold System.

The method has been used for the construction of:

— Road- and highway-tunnels
— Large underground shelters and fortresses
— Galleries for power plants
— Sewage galleries for clarifying plants
— Vertical and inclined shafts
— Galleries for river diverting (Energie Atomique Française)
— Tunnels for pipe-lines (Algeria)
— Stream cuts and subways
and more recently in coal and oremining for:
— Shaft constructions
— Galleries in direction of the rock
— Pit-eyes and cross-cuts

The Bernold System has proved satisfactory in every respect.

Tunneling experts are especially taken with the simple way it can be operated and the large scale of possibilities for application as regards handling techniques and statics.
In technical negotiations it always appears that there is a tendency to rate the method as a complicated one, owing to the special shape of the boarding and reinforcement sheets. We therefore ask potential clients to visit a building site and see for themselves. This makes negotiations much more to the point.
Owing to the fact that engineers working for contractors and as building supervisors have recognized the essential ad-

2. Tunnel- and Shaft-Constructions

For securing friable and very friable rock in the excavation zone we generally use

the concrete filling method

on which a well known tunneling engineer writes:
"With this sheet-concrete-rock-bond we for the first time get a lining, which can be set up immediately after excavation and the putting up of which does not take more time than that of the traditional steel lining. A final and hollowless lining can therefore be built in immediately without causing any loss of time or changes in the course of operations. Besides, owing to the fitting arches, the immediate bearing capacity afforded by the Bernold System is at least as good as that of the steel lining covered with sprayed concrete."

The concrete filling method implies the use of

Fitting arches

the number of which is determined by the daily excavation capacity, since the fitting arches should remain on their place for approx. 36 hours. Then they can be moved on to the drilling front. In blasting excavation the average amounts to 12—18 arches. These are already bent to profile when delivered and provided with hinged joints. Owing to these joints it is possible to move the arches without taking them apart on a caterpillar or tyre vehicle. This takes about 20 to 30 minutes. The fitting arches have been statically conceived in such a way as to absorb the rock pressure until the concrete lining has reached it's full bearing capacity without getting deformed or damaged by the constant blasting to which the fitting arch approx. 0.5—1.0 m behind the drilling front is exposed.

The boarding and reinforcement sheets are delivered bent to profile as well. They are linked by means of overlaps and locked together by 2 connection-rods per m^2. The sheets are shaped according to a unit-composed system. They can therefore overlap in tangential and axial direction of the tunnel at will and be reinforced, should rock pressure make it necessary, without any alterations. An assembly drawing will be set up if the tunnel profile has different radii and the sheets identified accordingly. They allow for compact stacking and are easy to transport.

3. Building Process

As soon as the blasting and loading work have been finished 1—3 fitting arches will be moved depending on the rocks pressure. As a rule the distance between the arches is either 0.96 or 1.92 m. The fitting arches are joined by spacing iron rods which absorb tension and pressure.

The sheets are put up one by one on both sides of the shaft, operations starting from the floor, installed on the fitting arches, overlapped and locked. Simultaneously the concrete is filled between rock and sheet by means of a concrete pump.

The filled in concrete will then be vibrated until it flows fully through the ribs. We thus obtain a rigid bond of rock/concrete and sheet with a good bearing capacity.

We use a normal filling concrete of stiff-plastic consistency with a grain of 0—30 mm, 250—300 kg of cement and a water/cement ratio of 0.4—0.5.

The concrete's resistance to pressure is without exception very high in this process.

Up to an angle of 100—120° approx. the concrete is filled in from above and vibrated. The filling in between the top sheets and the crown takes place from the front.

In this part the whole space is filled with pumped concrete and then carefully vibrated. However it can only be fully vibrated after a second filling. If there should still be hollows in the top, the concrete must be shot in under pressure.

In order to prevent the concrete from flowing out between the rock and the sheet in the direction of the drilling front, a front boarding of expanded metal will have to be put up. The front boarding remains in it's place until the next concrete ring is started. When putting up the front boarding, one should be careful to leave the last rib of the sheet free to be locked with the following sheet. If the work is done with reasonable care, the locking should be granted.

The distance to be kept between the drilling front and the building in of the lining depends upon the rock stability and must, for safety's sake, be determined by the engineer in charge. When there is a danger of rock fall, the rock should be secured by means of a thin layer of sprayed concrete or by means of hydraulic props, if it is very strongly fissured.

When the loading can take place very briefly after the excavation and a front protection has to be put up, fore rails shall be used. An additional roof part of the fitting arches (as a rule 60—70° of the tunnel development) will be put on these and brought to a distance of approx. 0.50 m from the tunnel front.

The boarding and reinforcement sheets will then be placed downwards from the crown, on both sides and locked with the steel ring that had been built in during a prior concreting stage.

Under the protection of this safety device the hollow between the sheet and the rock will then be filled up from both sides with pumped or sprayed concrete.

When the loading is finished with, the side parts of the arch will be put up and the sheets placed from above, whereafter the concrete filling method may be used.

As soon as the concreting work has been finished, the next section may be cleared by drilling, blasting and loading and the last fitting arch can be taken down after having been in place for 24—36 hours and put up for the building in of the next concrete ring.

4. Concrete Filling Method:

These two pictures show in practice the ideal case of the building in of a concrete lining directly behind the excavation or the drilling front.

In one instance the building site is located in Rhine slate with strong rock fall in the other instance in friable Grison slate with temporary stabilities of 6—12 hours. The concrete lining is at only 0.5 m of the drilling front.

On these building sites conventional methods were used until the summer 1968, embedded arches, wire-matting and sprayed concrete in one case and steel arches, lining plates and filling up with concrete of the surplus excavation in the other case.

Fig. 3

Fig. 4

Fig. 5

Fig. 5 shows the transition to the Bernold System with 3 mm metal sheets. It is perfectly visible that with the Bernold System the sheets alone remain in the construction, contrarily to the traditional steel construction, where steel arches and lining plates remain in the construction. During the last years the hollows behind lining plates had to be filled up with concrete as well, because of the strong distortions in the steel arches resulting from missing bond. These steel arches had to be replaced.

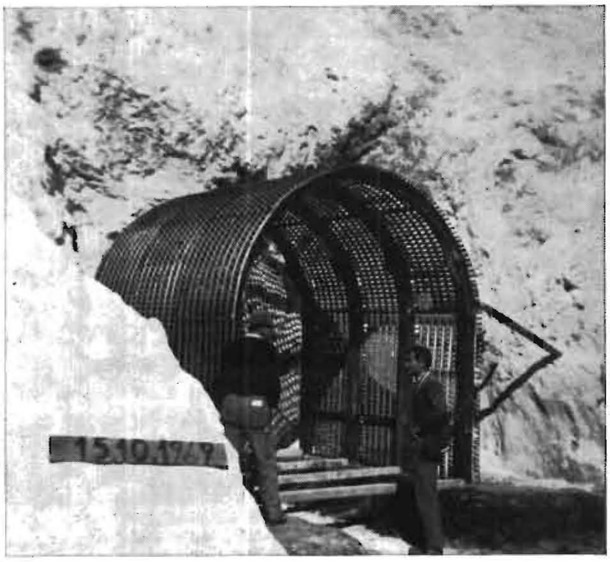


Fig. 6

Fig. 7

For the construction of the flight-shaft for the Gotthard *road tunnel* Bernold Sheets were used even for (6) the preliminary cut.

In this contact zone in the trias with partial water irruptions enormous difficulties had to be overcome. (7)

The sheets are placed on the fitting arches from the floor upwards and locked (fig. 8).

In the case of the Chauderon highway tunnel, excavations of 2—3 m are made in the very friable molass with 12—15 m layers of marl and the concrete lining is brought in immediately (fig. 9).

Concrete of a stiff-plastic consistency (water/cement ratio 0.4—0.5) is pumped into the hollows between rock and sheet after the putting up of 1—2 sheets (fig. 10 and 11).

Fig. 8

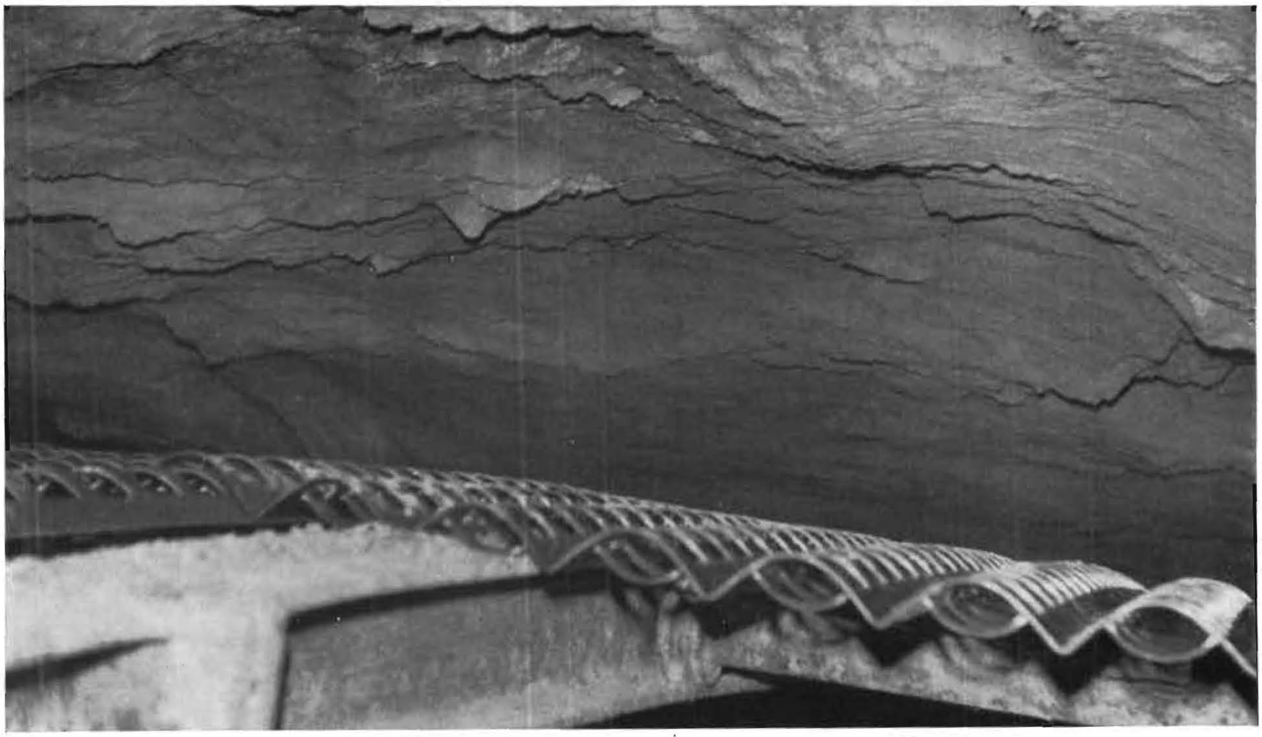


Fig. 9

Fig. 10

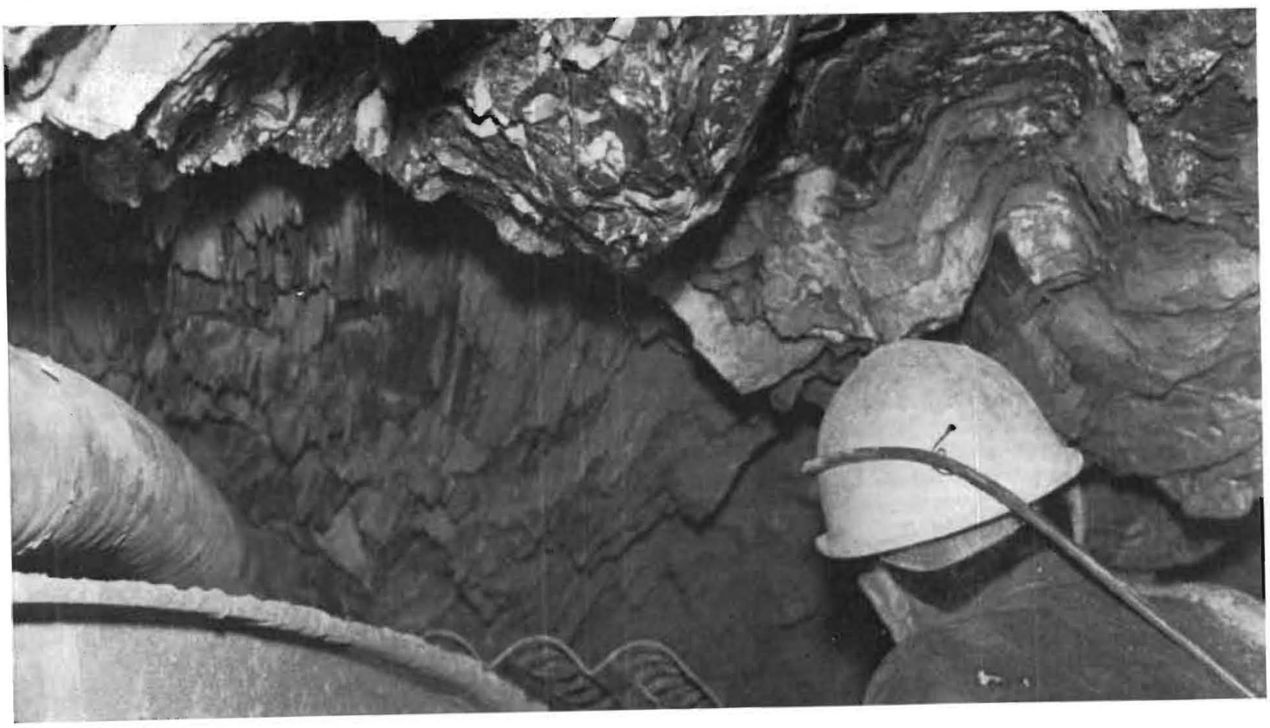


Fig. 11

Fig. 18

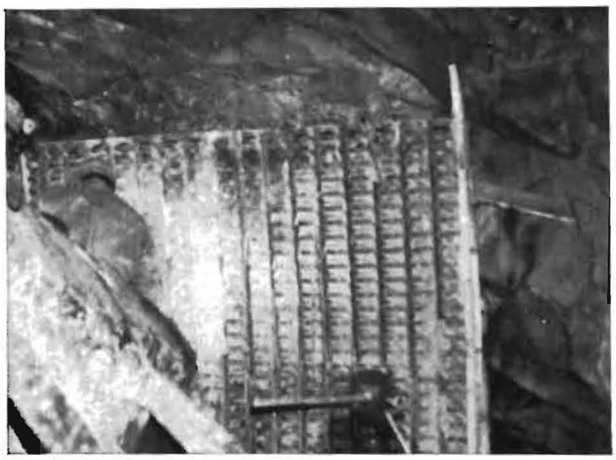


Fig. 20

The sheets are put up and the system is applied with rock anchors (in this case prestressed VSL anchors).

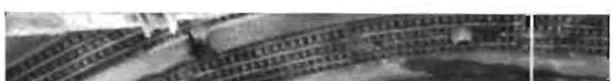
6. Various possibilities of application in tunneling

The sheets can even be put up without fitting arches as shown in the following examples.

Fig. 19

Application of 1 or 2 mm sheets as boarding between steel arches during the construction of Glion Highway Tunnel.

Fig. 21



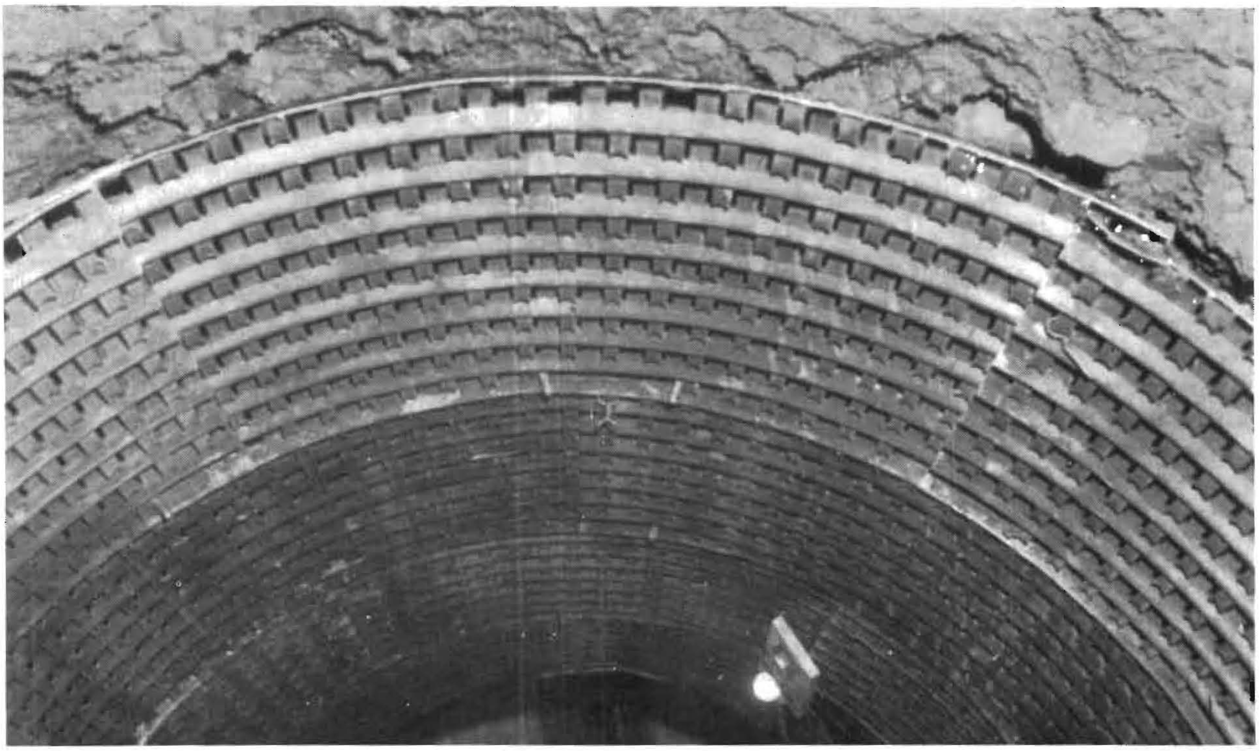


Fig. 22

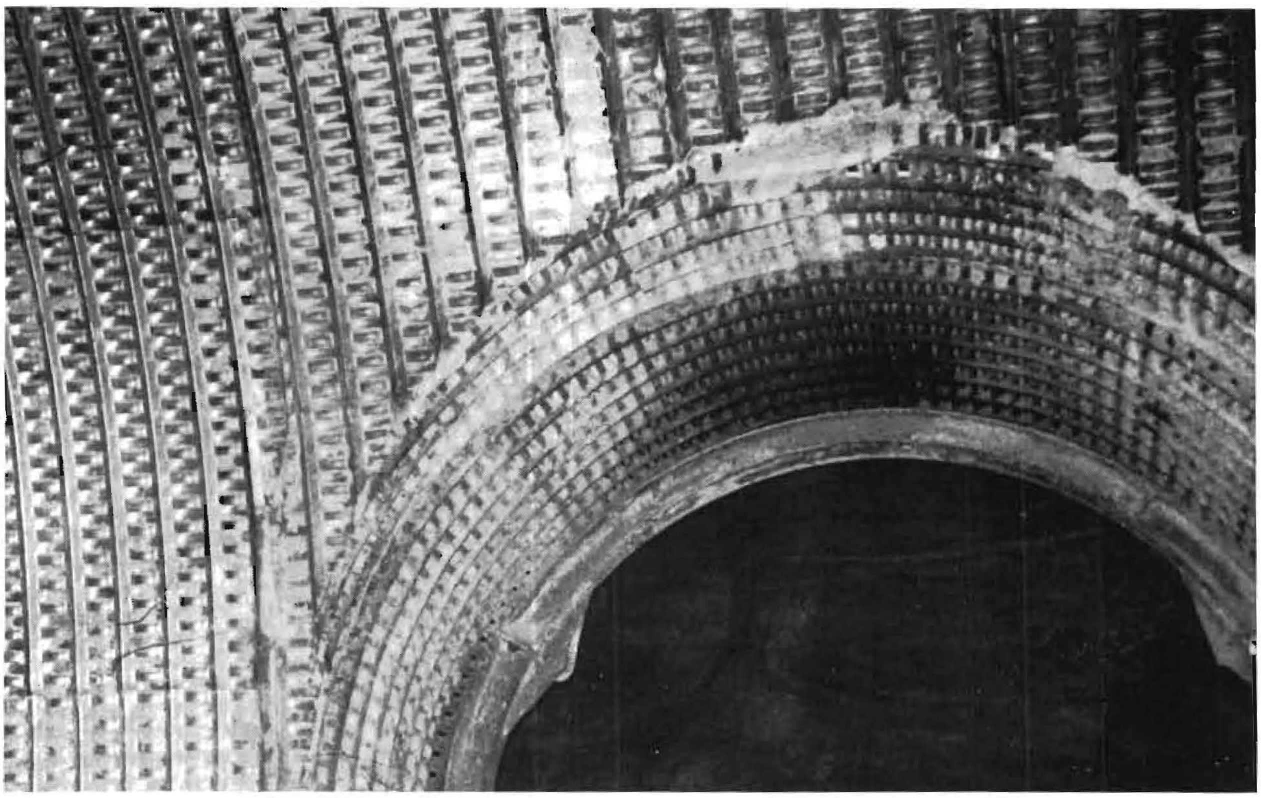


Fig. 23

Ring construction in the sewage gallery Bietigheim, by Baresel, Stuttgart, without fitting arches and without concrete or gunite in sand and clay. Diameter of gallery 2.00 m.

With some practice it is possible to build clean branchings and crossings as is done by Wagener, Essen, who builds underground galleries for the army somewhere in Germany.

7. Securing of the rock behind the tunnel excavation machine

The Bernold System holds a special position in tunnel excavation. During the Stini Kolloquium 1969 in Salzburg, Austria, Dr. ing. Naber, manager of the Bodensee-Water Supply, Stuttgart, Germany, held an extensive talk on mechanical tunnel-excavation.
During the discussion Dr. ing. Nathau of the «Technische Universität» Clausthal-Zellerfeld said among other things:

"In addition to the already known securing methods with steel rings, anchors and sprayed concrete, we should like to introduce a new method that has been applied for the first time in the Oker-Grane-Gallery in the Harz. This gallery is being constructed by the associated Wix & Liesenhoff Gebr. Abt. KG and Deilmann-Haniel with Demag excavating-machine and the zones in the Wissenbacher slate and Kahlenberg sandstone in which there is danger of rock fall are secured by means of Bernolds sheets (fig. 1). These specially shaped and perforated norm sheets with an approximate size of 1m² have been developed for the simultaneous boarding and reinforcement of steel-concrete-structures (1). In this case they have proved, even without the use of concrete, to be a securing that can be put up quickly, is reliable and economical. They have further advantages such as the slight sheet thickness of 4 cm and the possibility of creating a safety vault of homogeneous aspect that fits the rock closely, is sufficiently stable (fig. 2) and can absorb tension and pressure in all directions. It has not yet been decided which type of final lining should be used for this gallery. We should like to point out, that this provisional lining can be transformed into a thin-walled steel/concrete bond lining by means of the wet concrete spraying method (2). In this way the zones requiring a lining of greater bearing capacity can be provided by degrees with a lining that excludes the loss of building elements and can be integrated organically in the other excavation processes from the very first securing of the rock to the final lining."

This article is concerned with the use of sheets in mechanical excavation with or without the use of sprayed concrete. When the excavation is done with tunneling machines in cross sections of 30 m² and more, the sheets cannot be put up without auxiliary tools. For such large spans the light fitting arches must not be moved until the concrete is sprayed on.
There is another way of building a statical steel/concrete lining (fig. 28).
A movable boarding structure is erected on a working vehicle. The length may vary from 2—6 m depending on the excavation capacity and the rock stability. This steel structure may be lifted and lowered mechanically and matches a tunnel development of 160°. The sheets which are bent exactly to profile are put up on the boarding structure in low position on that side of the vehicle, which is away from the drilling front and then brought to the drilling front and to their proper place.
Then the boarding structure is mechanically lifted and pressed against the excavated rock.

Fig. 24 Locking of the sheets

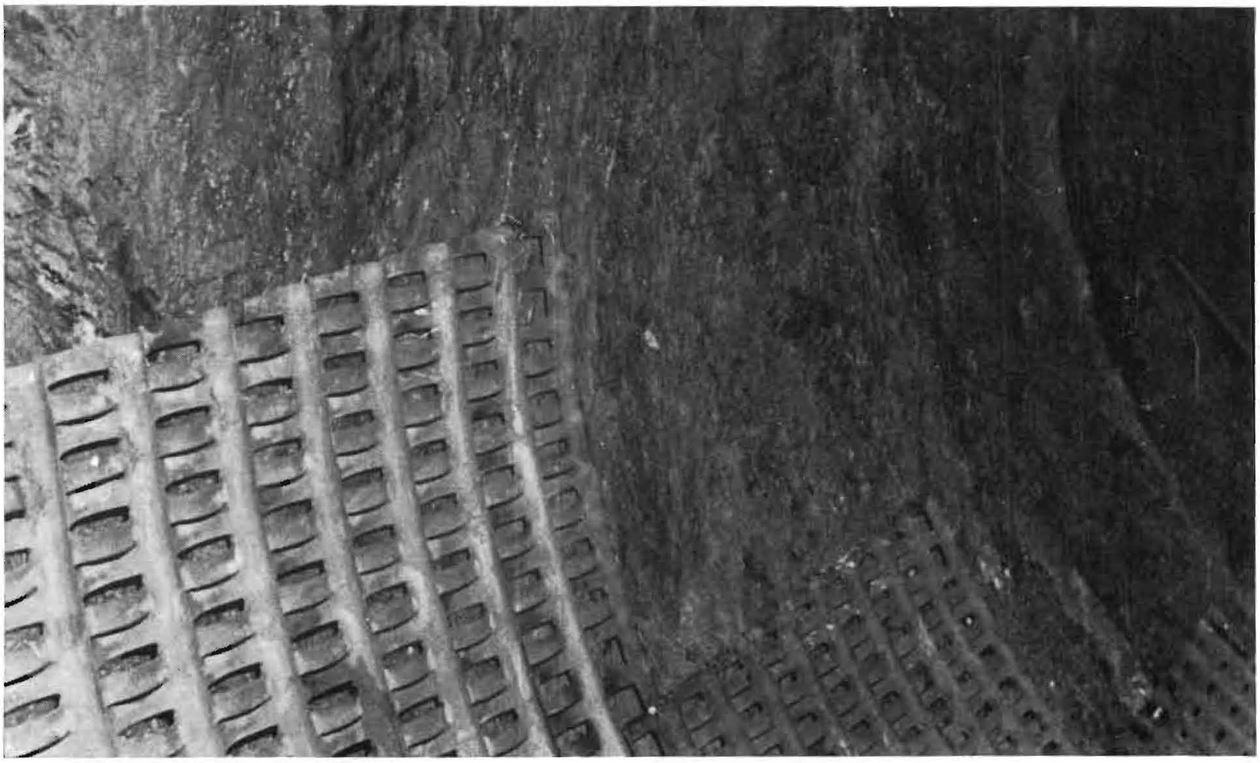


Fig. 25 The sheets are staggered

We now have two possibilities for fastening the sheets. Either by means of rock anchors and bolts. When this is done the boarding structure may be **lowered** and driven back and concrete is sprayed on. Or, the other possibility is to leave the boarding structure until sprayed concrete or gunite with a thickness of 4—6 cm has been sprayed on and bond is achieved. The boarding structure may then be lowered and driven back. After a few hours another 4—6 cm of concrete are sprayed on and the hollows caused by the boarding structure filled up.

Fig. 26 Securing of the roof section by means of the Bernold sheets

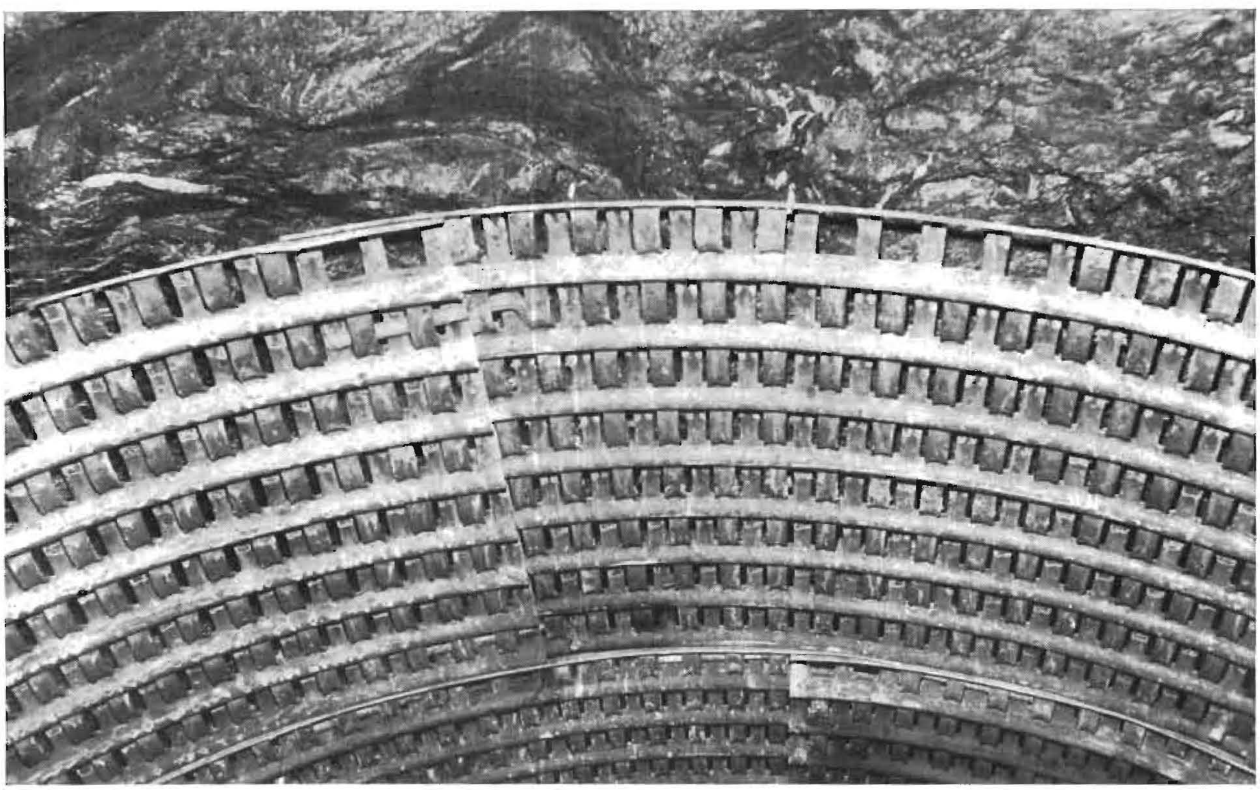
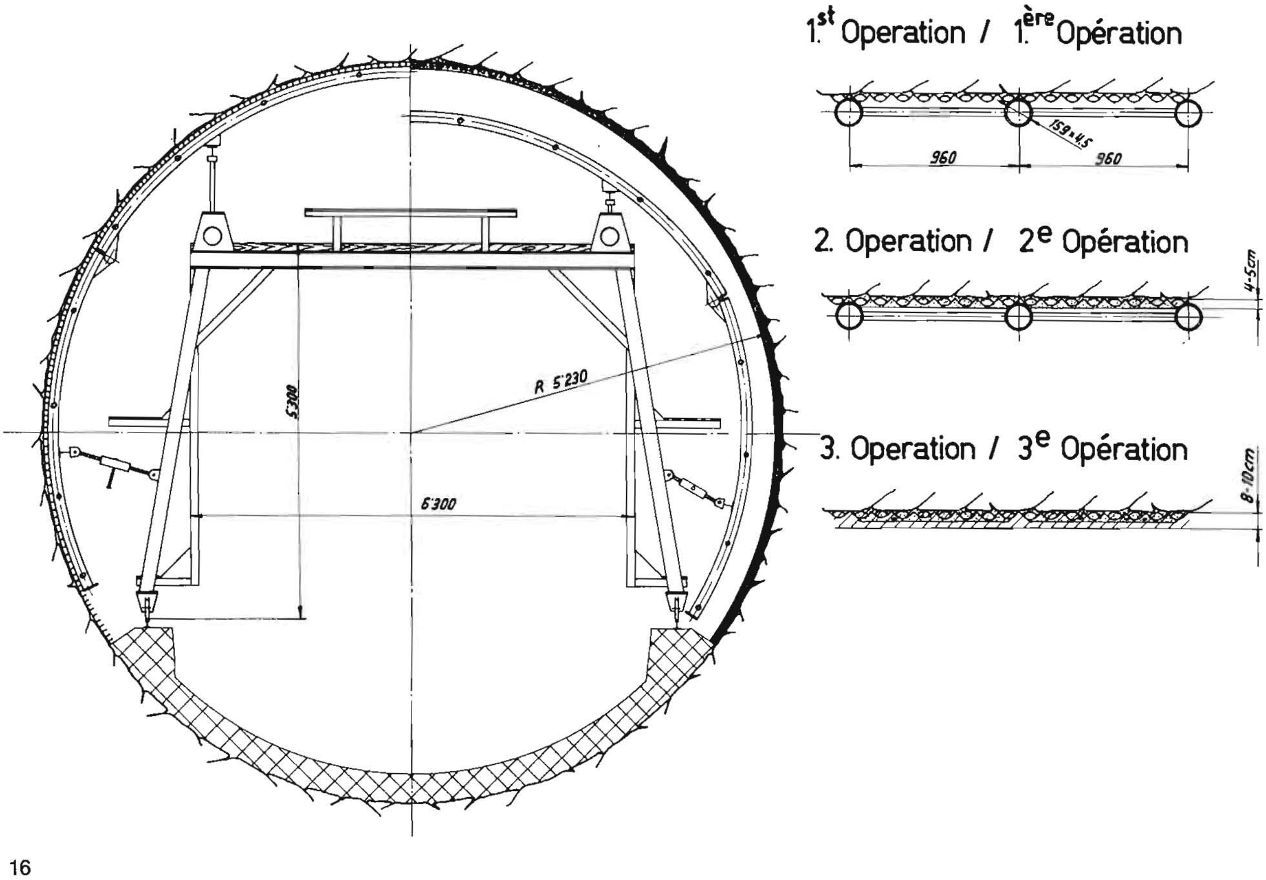


Fig. 27 Securing of the ring in the Oker-Grane-Gallery

With this method we also have the possibility of fastening an appropriate isolating foil directly on the rock surface and to integrate this auxiliary vault in the statics. There we have another opportunity for making tunneling with mechanical excavation even more economical.

Fig. 28



8. Shaft-Construction with the Bernold System

For the driving of inclined or vertical shafts, either by blasting or with the excavating machine, the sheets are used with or without concrete.
The method was applied for the first time in autumn 1968 in Rhine slate for driving a 12 m shaft underground (fig. 29).
The shaft was excavated first and then the sheets were put up from the bottom to the top and concrete was filled in.

Fig. 29

Some weeks later work started on the 35 m Gose-shaft on the Oker-Grane building site. This shaft of a ϕ 2.10 m was driven from the top. As soon as an excavating stretch had been completed, the Bernold sheets were put up and concreted with the concrete filling method (fig. 30).

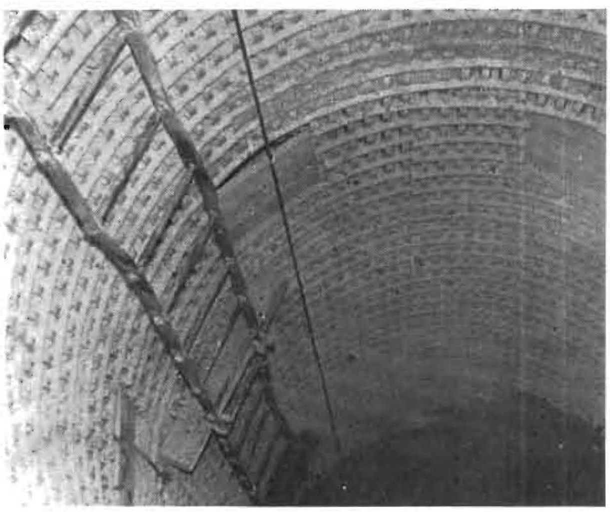


Fig. 30

The first use of Bernold lining in **mining**

Mining firms began to show interest in the new method. In summer 1969 Preussag in Bad Grund started with two shafts 50 and 58 m deep and with a ϕ of 1.4 m and 2.00 m. These shafts are being mechanically excavated and the Bernold lining is being put up directly behind the excavating machine (fig. 31).

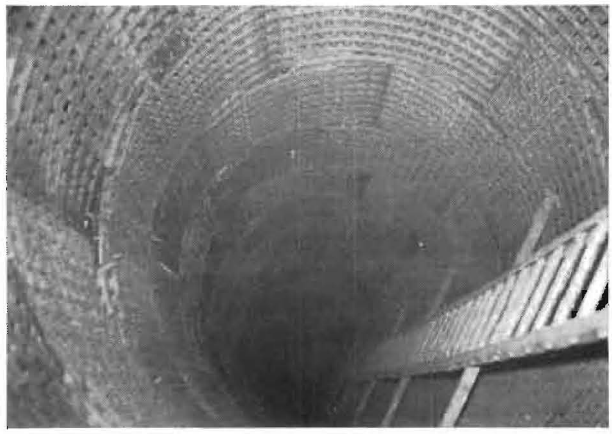


Fig. 31

Shaft construction with excavating machine, Preussag, Bad Grund.

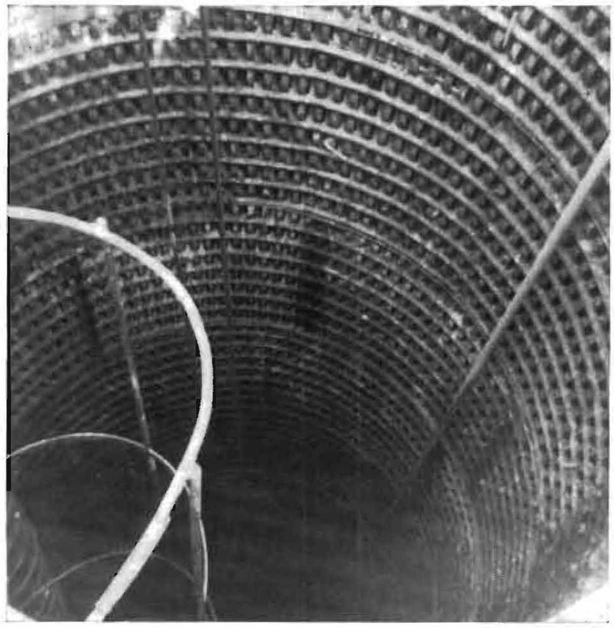


Fig. 32

Some time ago a vertical shaft with a diameter of 3.0 m has been lined with Bernold sheets in Lucerne.

In Germany the construction of a large number of shafts with various diameters is under way.

Why are Bernold sheets used for shaft-construction?

An engineer for shaft-construction answers this question as follows:

"The Bernold lining is especially suited for shaft-construction because it can be adapted to the rock stability. I need not conform to certain fixed sizes because the sheets can overlap in axial and tangential directions at will. All sheets whether their thickness be 2 or 5 mm have the same shape and identical connection-parts may be used for them."

Another engineer for shaft-construction declares:

"So far the Bernold lining is the most economical method for securing excavated shafts.
The costs for excavating with an excavating machine are approximately the same as with the traditional methods, while the lining according to the Bernold System affords definitive advantages."

9. Repair work in existing galleries

A large number of galleries, which had been constructed in solid rock some decades ago, must now be lined, since slackening zones have formed in the course of time, which are a potential danger for all users. In this respect the Bernold boarding and reinforcement sheets have stood the test as a lining true to profile and of good bearing capacity, that is most economical.

10. Application of the Bernold System in Rock under Stress

The question frequently arises whether the system can be used in rock under stress with no temporary stability, in sand, clay or moraines.

As was already stated in the account of 1967 this question can be answered by yes, with the one difference that we are now in possession of experimental data and have been able to develop essential technical improvements.

In mining the question of safety and of the securing methods to be used always comes first before cutting through rock under stress. The method of driving under lances has been developed to make work under protection of a steel lining possible. This construction has been developed and improved for more than 20 years and can cope with any kind of difficulty occurring in rocks (except boggy rocks), provided that geological conditions, rock pressure and unhomogeneous conditions be known to the planners.

Rock securing with hydraulic driving under lances

(Drawing 35)

3 guiding arches are statically adapted to the rock pressure to be expected. For loads of more than 25 tons/m² a circular profile is generally chosen.

The guiding arches are put up true to profile, locked with each other and anchored.

Then the top lance which is provided with guiding rails on both sides is fastened and the other lances are put up left and right.

To support the lance ends a concrete ring with a length of 2 m and exactly fitted to the profile of the tunnel can be preconcreted.

Every lance is fitted with a guiding rail (A) which acts as hinge joint to assure the polygonal adaptation to the radius of the tunnel. This guiding rail prevents deviations of more than 5 cm from the axis of the tunnel. Deviations of ± 5 cm can easily be corrected. It is essential that the guiding arches should be installed with accurateness.

Lances (B) have sharpened points in direction of the drilling front and are fitted with advancing catches on a length of 2.0—2.5 m, which are necessary for advancing the lances with the lifting cylinder.

The end of the lance which is generally 2—4 m long, is polished as it acts both as support of the rock between guiding arch 3 and fitting arch 1 and as outer boarding for the concrete ring. In accordance with the thickness of the concrete vault, the radius of fitting arches (1—3) is smaller, however their size is chosen in such a way as to enable them to absorb rock pressures which might arise. As a rule concrete rings with a length of 1.0—3.0 m are concreted according to the working rhythm of the lances.

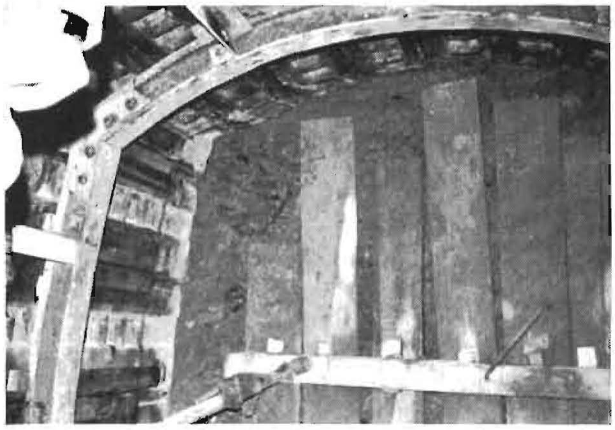


Fig. 33

The hydraulic driving lances have been installed, after the difficulties at the beginning, which stemmed from the strong rock pressure, had been overcome. The normal working rhythm had been reached after an introductory 4 days.

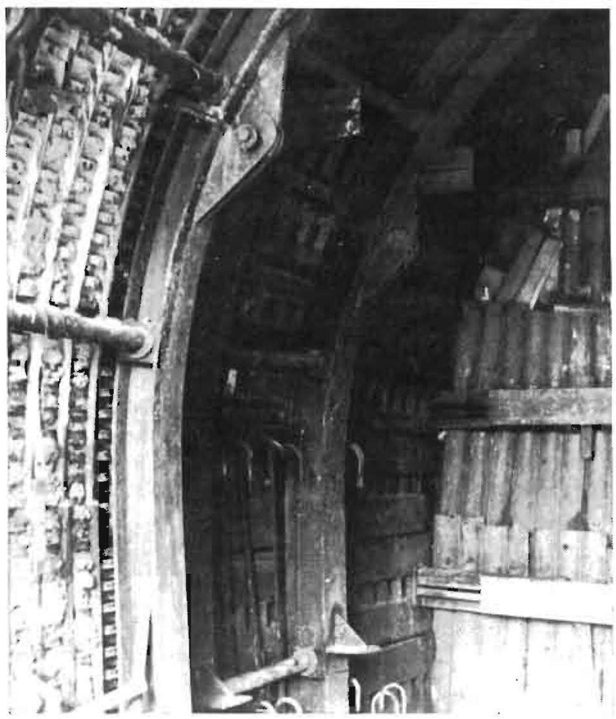


Fig. 34

For the front securing a device was created that consists of cross pieces of adjustable width (A), which can be fastened to the lances by a catch and remain there.

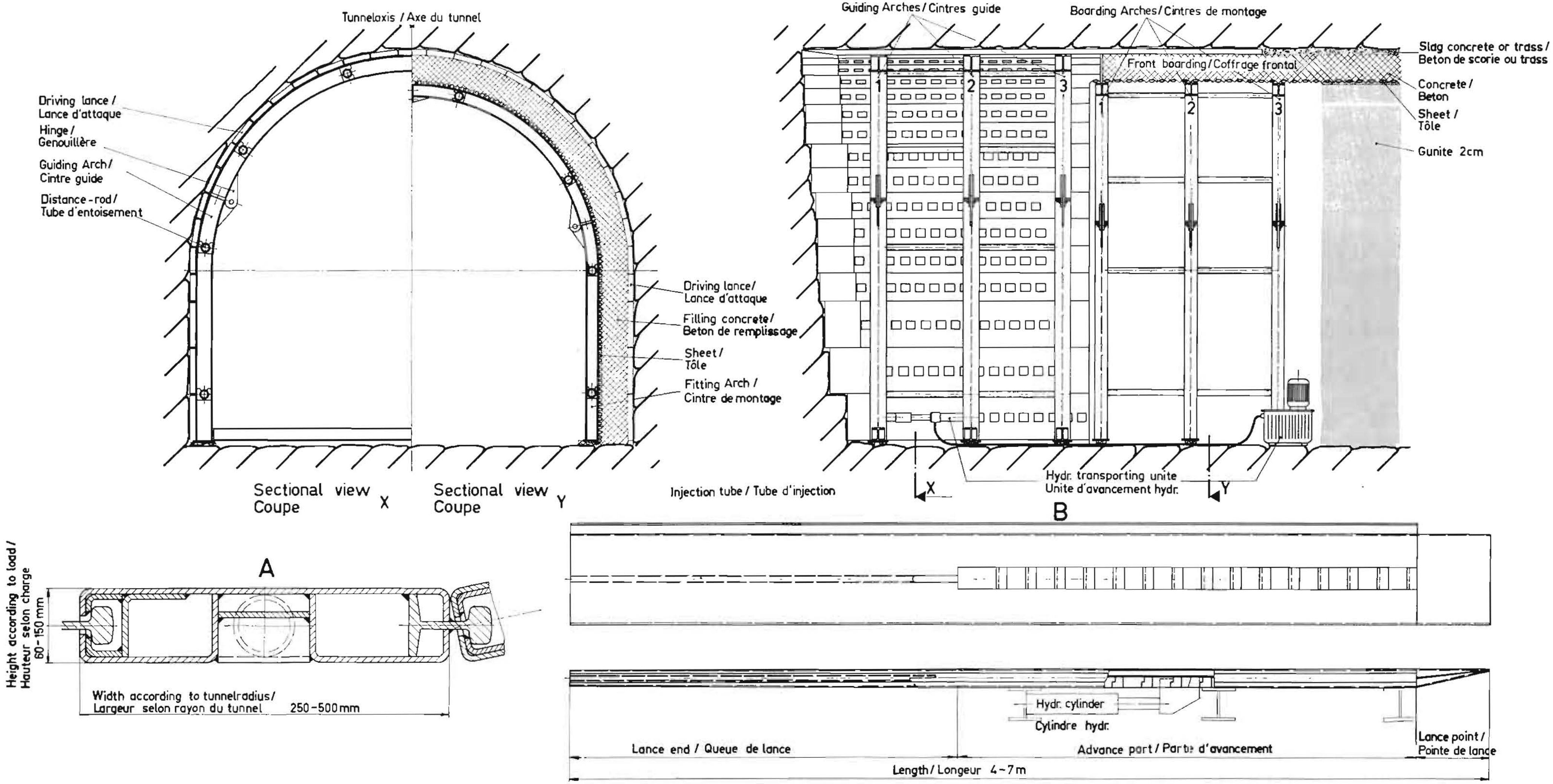
As the excavation is done by steps from the crown to the bottom, the cross piece can be driven ahead with the lances on both sides and then the liner plates (B) be put in to secure the rock.

The following description of the working process will help to elucidate further points:

When the guiding arches and lances have been set up, the lifting cylinders are put into place and the driving starts. As the material is being taken down at the points of the lances, these must be pushed on together.

When the lances have passed guiding arch 1 by 1—1.20 m, the Bernold sheets can be put up and locked on the fitting arches 1 and 2. The front boarding will then be fastened and the concrete pumped in between lance end and sheet and vibrated.

The lances may not be driven forward during concreting. Only when the whole concrete ring has been finished, may the driving operation start in the crown. Simultaneously with the excavation, the hollow space which develops between the rock and the lance end, is pressed out with lag concrete or trass by means of a special device, in order to avoid settlings of the soil.



Tunnelaxis / Axe du tunnel
Driving lance / Lance d'attaque
Hinge / Genouillère
Guiding Arch / Cintre guide
Distance-rod / Tube d'entoisement
Driving lance / Lance d'attaque
Filling concrete / Beton de remplissage
Sheet / Tôle
Fitting Arch / Cintre de montage
Sectional view Coupe X
Sectional view Coupe Y
Guiding Arches / Cintres guide
Boarding Arches / Cintres de montage
Front boarding / Coffrage frontal
Slag concrete or trass / Beton de scorie ou trass
Concrete / Beton
Sheet / Tôle
Gunite 2cm
Injection tube / Tube d'injection
Hydr. transporting unite
Unite d'avancement hydr.
X
Y
A
Height according to load / Hauteur selon charge 60-150 mm
Width according to tunnelradius / Largeur selon rayon du tunnel 250-500mm
B
Hydr. cylinder
Cylindre hydr.
Lance end / Queue de lance
Advance part / Partie d'avancement
Lance point / Pointe de lance
Length / Longeur 4-7m

Drawing 35

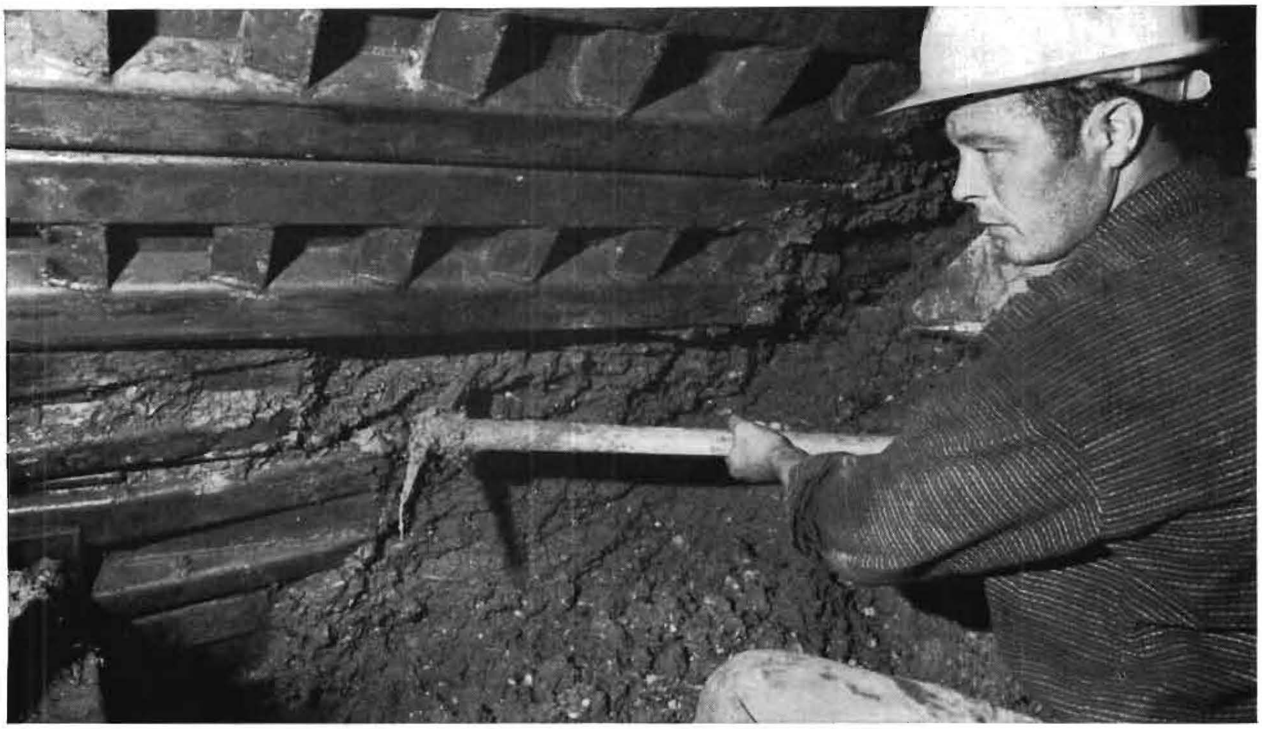


Fig. 36

While the lances are driven forward, the material should be taken down as shown on the picture. If the work is to be interrupted for some time no more than the points of the lances should be pushed into the ground. The lances should never be driven more than 1.20 m ahead of guiding arch 1.

Fig. 37

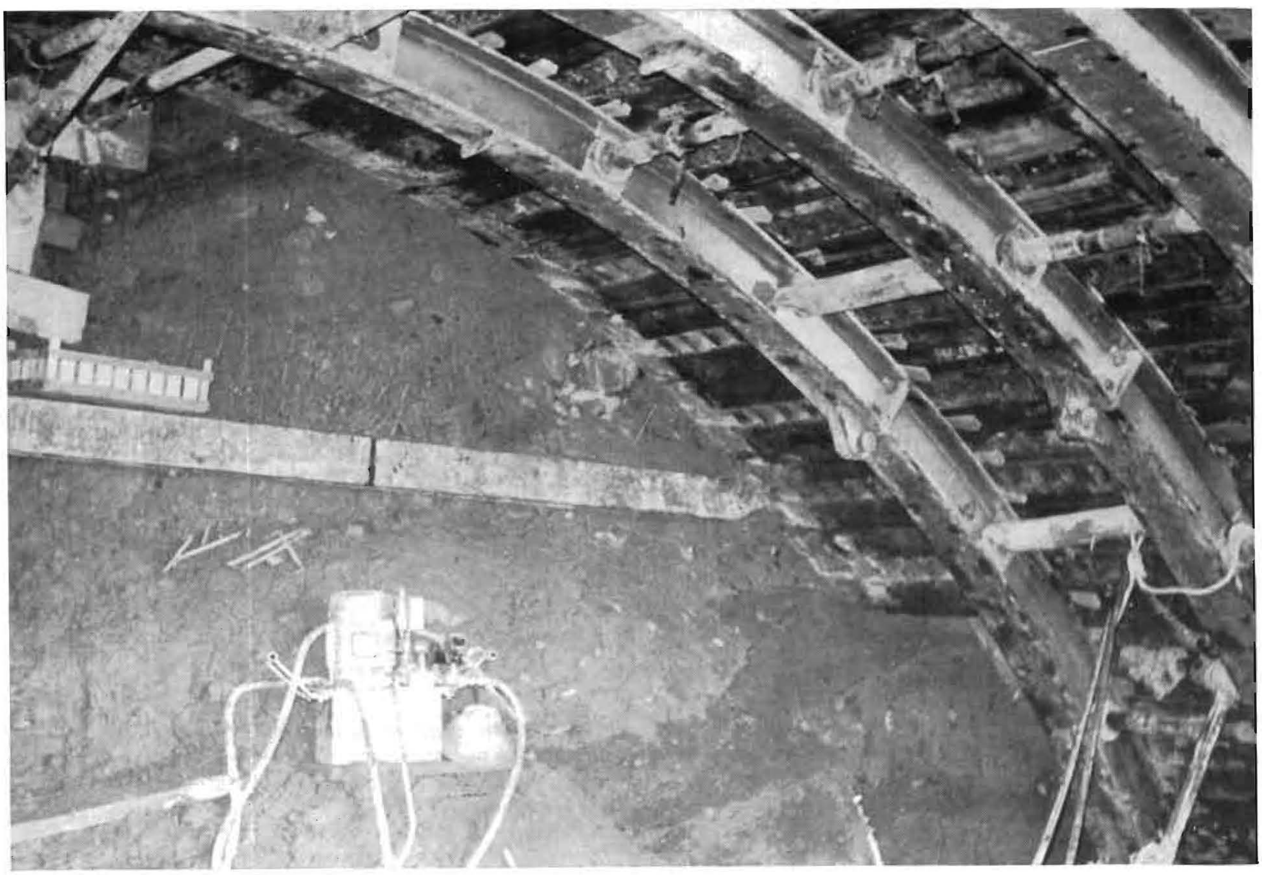
Driving ahead of the lances by means of a hydraulic cylinder with a pressure of 20—30 atmospheres between guiding arches 1 and 2. As for any systematical work it takes some practice to carry out this operation properly. 2—3 driving cylinders are needed depending on the size of the tunnel cross section.

In the crown, the lances have already been driven so far ahead, that no more than the polished lance ends appear near guiding arch 2, while the lances on the sides still must be driven ahead.

Fig. 38

In the foreground you may see guiding arch 3 (A) and lance end (B) resting on the concrete ring. In the background appear fitting arches (1—4) with the Bernold sheets (C) which have been put up and concreted. Follows the moving of guiding arch 3 to the drilling front and the advancing of fitting arch 4. Then the putting up of the sheets and the front boarding may start as well as the concreting of the next ring. In hydrous rock the longitudinal locks of the lances should be made waterproof before concreting. Depending on tunnel cross section and material, the daily excavation capacity of this system ranges from 2.0 to 3.0 m. This capacity can be doubled when there is a cover of 30 m and more, but it may also be reduced to 1.0 m and less in the case of a permanent front securing as shown in pictures 34 and 35.

Fig. 39 Highway Tunnel Flonzaley Conrad Zschokke S.A. — H. R. Schmalz AG.



V. Mining

Special features of mining

For many years a fundamental improvement of lining operations by mechanization was striven for in mining. Since a mechanization of the construction of curved sections seems hardly possible, the only successful attempts were achieved with concrete lining so far.

First trials have shown that the lining of drifts and mining tunnels with the concrete lining system and Bernold boarding and reinforcement sheets has a great future ahead. These lining operations can be fully mechanized. The fitting arches which can be reused at any time, can be moved simply by hydraulic devices. The putting up of Bernold sheets is as simple as could be. The concrete is brought in by means of the concrete filling method. This process can easily by mechanized and it is merely a question of organization to facilitate the transport of concrete. The high strength which is achieved by this method is especially significative in mining. Thin walled concrete linings, which are true to profile, have a bearing capacity three times as big as that of the traditional drift lining, which costs as much. For these reasons they will have conquered mining very soon.

In those cases however, in which mining tunnels are to be but short-lived and a destruction of the drifts is put up with, conventional lining techniques will not be dislodged so quickly.

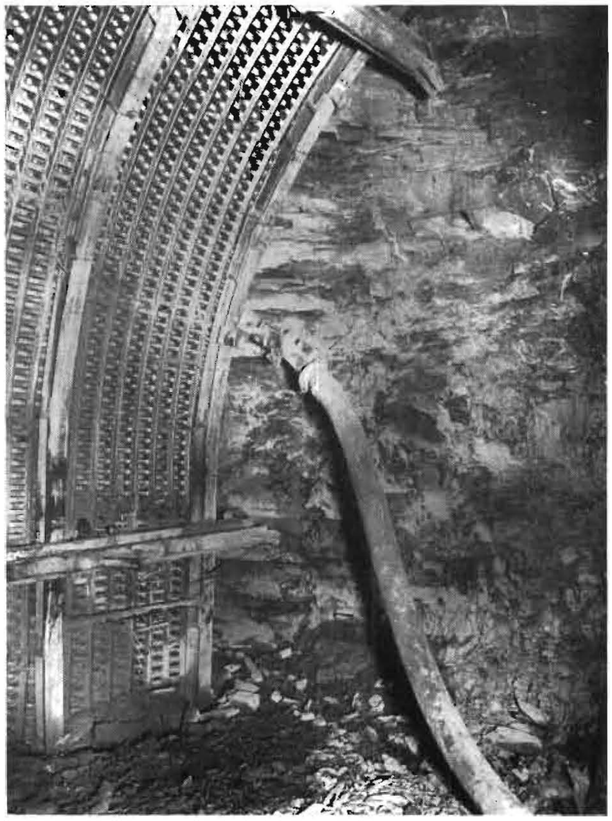


Fig. 40

Fig. 41 Eschweiler Bergbau-Verein, Grube Emil Mayrisch

VI. The Mortar Spraying Through Method

For the mortar spraying through method the arches are installed in the same way as for the concrete filling method, but the boarding and reinforcement sheets are put up and locked on the whole development of the tunnel.
The nozzle of the wet concrete spraying machine is designed in such a way as to horizontally fit the outer rib of the sheet (fig. 42 and 43), whereafter the concrete penetrates through the sheet with a pressure of 6 atmospheres into the empty space between the rock and the sheet.
State of the sheet directly after the spraying through, before the levelling coat has been applied.
The filling of the hollows between the rock and the sheet depends to a large exent on the routine of the man who holds the spraying nozzle. Experience has shown, that one can count on all hollows being filled but that the hollows in the crown should be filled from the front as well if they are more than 20 cm thick.
The spraying through method should only be used for concrete linings, which have no big statical strain to resist and the theoretical concrete thickness of which does not exceed 10—25 cm. It can also be used for local securing, the sheet being fastened with rock anchors and the hollows filled in with the spraying through method.
One instance of application which has already proved quite successful is the spraying through method in connection with mechanical excavation. The Bernold sheet acting as front protection is anchored to the rock directly behind the excavating machine or provided with distance blocks, put up and gunited, in order to obtain a direct bond with the rock and a statical vault.
With this latter method the rock is secured directly behind the dust-shield and there are no steel profiles to obstruct the way back. Contrarily to liner plates no pressing out with cement/mortar is required as the sprayed concrete or gunite fills possible hollows.

Fig. 42

Fig. 43

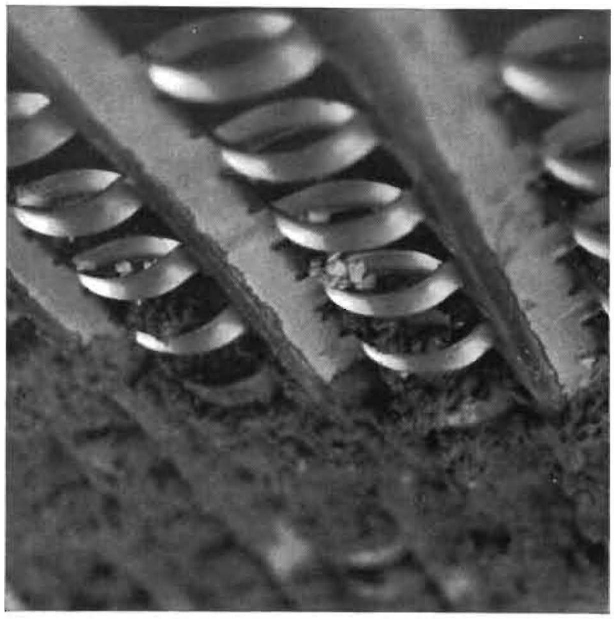
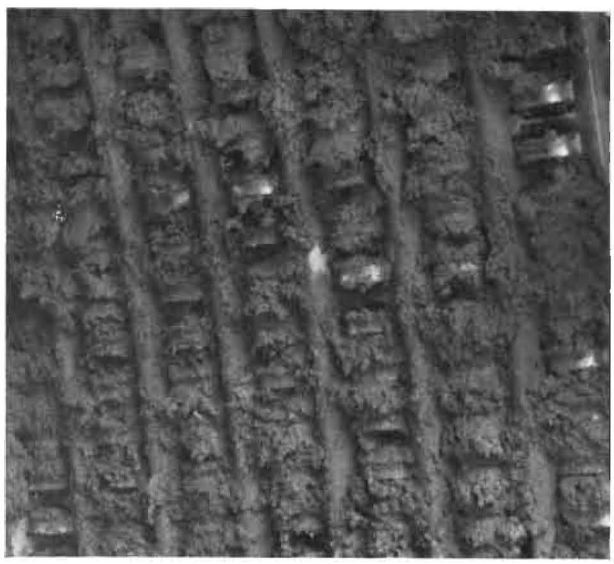


Fig. 44

Fig. 45
Installation Process
View from within before the spraying



VII. The Bernold System in the Construction of Underground stream passages and Subways

As a rule the wet-spraying system is used for these constructions after a sole of poor concrete has been put in to match the slope.
Then the boarding and reinforcement sheets are put up (fig. 46 and 47), which operation requires only very little time.

The economy of this procedure for the construction of underground stream passages depends on the diameter, which ranges from 1.50 to 4.0 m for a thickness of the concrete wall of 8—20 cm.

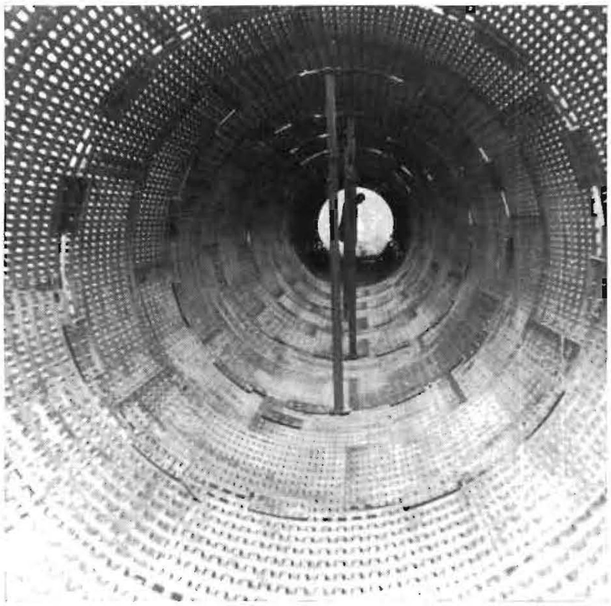


Fig. 46 View on the inside before spraying

Fig. 47 Assembling process

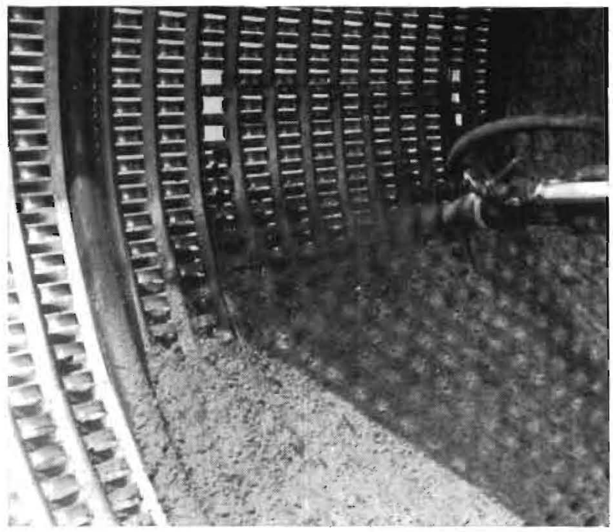


Fig. 48

Construction of the concrete lining with the wet-spraying method and the use of spraying concrete PC 350—400 with a grain of 0—8 or 0—15 mm.

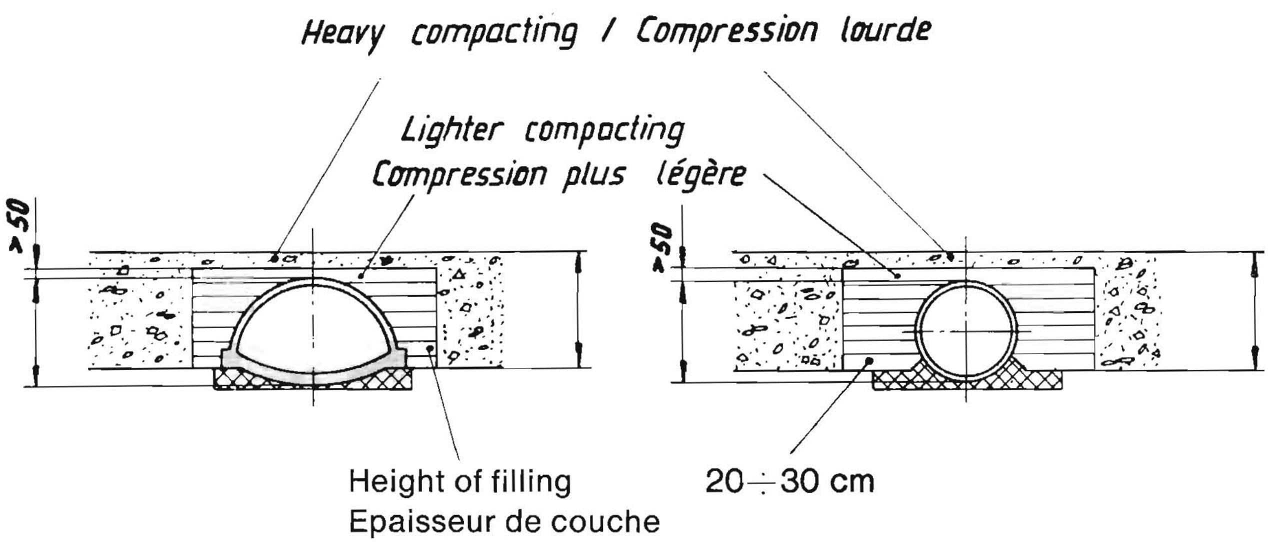


Fig. 49 The following drawings inform about sections, shapes of profile and capacity of bearing.

Fig. 50 Underground stream passage under Swiss Highway N 3

VIII. Statics

(Modern knowledge of tunneling statics
and the Bernold System)

It is not in the scope of this article to give directions about the calculation and dimensioning of tunnel linings. But we should like to point out the close relationship between modern tunneling statics and the Bernold System.

1. Essential points for the static calculation of underground buildings

A static calculation ought to enable the economical and safe dimensioning of supporting structures with an intended safety degree. Owing to the knowledge of the laws of equilibrium and of the deformation qualities of building materials, techniques of calculation have been developed during the past decades, which allow the building of daring structures. Thus bridges with a span of up to 1300 m and skyscrapers and towers of a height of 500 m have been planned and built.
To make such calculations possible the dimensions of the supporting structure, the loads and their repartition, the quality and stability of the building materials and the disposition of the supporting parts must be known. The result naturally depends on the accuracy of these values.
This now is the special difficulty in tunneling. The above mentioned values are but estimates and affect the reliability of the results so much, that the economy and therefore the aim of the whole calculation become doubtful.

The close correlations between the thickness of the concrete vault, the deformation qualities of the rock and the size and direction of the load make it impossible even with the use of computers to set up a reliable calculation according to the theory of elasticity. For tunneling other ways had to be sought and found, to make economical building possible.

The experience made with tunnel buildings, that had not been correctly dimensioned and the corresponding load tests, were the reason for developing new calculation methods, which result in the following conclusions, diverging in part from the elasticity theory:

- Thin walled tunnel linings (thickness of wall approx. 1/15 to 1/25 of the radius) are better suited for the safe absorption of rock pressures.
- The determinating cause of rupture manifestly is shearing rupture.
- Bending ruptures are possible only if hollows have remained unfilled behind the lining.
- Thick walled tunnel linings (thickness of wall approx. 1/5 to 1/8 of the radius) can only absorb comparatively small loads without suffering damage: bending ruptures and other results of destruction soon show.

We therefore know, that for the construction and calculation of tunnel linings:

- Building methods must be used, by which a slim final concrete lining may be built in as quickly as possible.
- It is decisive, that the lining should fit the rock without leaving hollows, adequate care is essential.
- It is very important, that a profile should be chosen that matches the rock quality. A circle is the ideal profile. The sole may be slightly flatter, straight parts ought to be avoided or limited to stable rock-conditions.
- The statical calculation can be limited to a shearing rupture test with triple security, according to the theory of semi-stiff construction.
- Nowadays differing methods are appropriate only for uneconomical thick-walled tunnel linings.
- Bending ruptures are hardly to be expected in thin walled tunnel linings, since large normal forces reduce possible bending-tensile stresses.
- An additional inner reinforcement has the task of absorbing bending-tensile stresses resulting from local disorder in the rock.

These claims which are uncontested by modern tunnel engineers, are all fulfilled by the Bernold System. The final concrete vault is erected immediately after excavation with a minimum thickness of 15 cm. The gradual filling in of concrete behind the perforated sheets can be optically controlled at any time. The fitting arches, which are reusable, guarantee a lining true to profile and an undisturbed working process.
Owing to it's compression the concrete is of a very good and homogeneous quality. The inner reinforcement is achieved by the simultaneous use of boarding and reinforcement sheets in one element and one single working process.

2. The bearing capacity of thin concrete linings

Some figures to illustrate what was said in the above chapter:

Example: Circular tunnel with a radius of 5 m, wall thickness 25 cm, compressive strength of concrete cubes larger than 300 kp/cm^2. In such linings bending ruptures occur only at a load of 95 to/m^2. A triple safety therefore makes an approx. 32 to/m^2 admissible.
When checking the ring pressure, we get a concrete compressive strain of 63 kp/cm^2 under a load of 32 to/m^2. As compared with the cylinder compressive strength of concrete, the safety is larger almost by four times. Many tests for such calculations have been carried out or are still under way (lit. 2).
The question of the slenderness limit, i. e. the moment when shearing rupture is determinating or in other words when the concrete lining does not absorb load movements any more without being damaged, can be answered as follows:
It is commonly known, that rock pressure does not occur suddenly, develops slowly. The first loads arise in vertical direction and occasion a slight crown deflection and as a result a movement of the side walls towards the mountain. Thus side forces are roused so that finally the line of pressure lies in the range of the lining. The slenderness depends on the size of the movement necessary to cause these side forces. For elastic rocks such as slate, clay, marl, sand and moraine the ratio wall thickness/radius should be approx. 1/25. For more stable rocks the limit is 1/15.
Other interesting methods of calculation are the well known bearing stress procedures. With the hypothesis of so called bending mechanisms, loads for thin walled tunnel linings can be calculated, which lie far higher than shear breaking loads and confirm the calculation method for semi-stiff constructions.
There is a constant development of new calculation methods: for known rock conditions they are being refined as much as is economically interesting and technically practicable. At the end however the tunneling engineer will always have to consider new and unknown inhomogeneities which make an exaggerated calculatory work for underground operations questionable.

3. The adequate shearing reinforcement in connection with Bernold sheets

Since dimensioning in tunnel construction should be determinated as said before by the causes of breaking to be expected, and we know that shear breaking is responsible for it, a strengthening of the concrete vault with a shearing reinforcement may be required in various cases. The shear stirrups increase the dowelling action of the Bernold sheets, prevent detaching caused by radial deflection forces under high tensile stresses and absorb the tensile forces under shearing stress that leads to shearing cracks.

4. Fundamental principles for dimensioning

The fundamental principles for dimensioning are based upon the following test reports of the EMPA:

1. EMPA-report nr. 69 953/3	Tension tests on single ribs
2. EMPA-report nr. 59 392	Plate-bending tests with single Bernold sheets
3. EMPA-report nr. 67 839	Plate-bending tests with overlapped Bernold sheets
4. EMPA-report nr. 60 410/2	Breaking tests with simply and doubly reinforced semi-circular arches.

Besides the expertise of the Institute for Statics of Prof. Dr. Ing. H. Duddeck we also call upon our own calculations and experience with finished constructions.

Evaluation of the tests

a) Tensile strength of the sheets and measuring of bendings

The basic material has a tensile strength of 3500—4200 kg/cm², the yield point at 2700—3200 kg/cm² (EMPA-report nr. 60 410/2, annex 21).

By the perforating and cold deformating of the basic material into Bernold sheets we naturally get a compacting of the material with a loss of plasticizing capacity. The resulting reinforcement rib has a variable jointly carrying breadth.

I. e. under tensile stress the working line of the rib features a slightly higher E-module of approx. 2.3—2.6 10^6 kg/cm² if the tension is related to the minimum cross section.

In the minimum cross section the tension is higher than in the range of the rib, the flow range will therefore be reached sooner there. This means, that the flow range of the rib as a whole is higher and the elongations accordingly smaller.

The nominal yield point was between

3680 and 3810 kg/cm²

The tensile strength amounted to

3820—4070 kg/²

The ductile yield amounted to 8.6—9.6‰ for 3 mm-sheets and 12.5—14.3‰ for 2 mm-sheets. In concreted ribs higher elongations at rupture of approximately 30% were measured. An exact study is under way in order to allow a better judgement on the rotational capacity of plastic moments.

b) Necessary lengths of overlapping

EMPA-test nr. 67 839 deals with the sheet-overlappings.

Under bending stress alone the breaking load was only 10% less with an overlap of 9 cm (= 3 ribs) than at an overlap of 15 cm (= 5 ribs).

At breaking point steel tension amounted to

= 3000 kg/cm² (in the range of the overlap).

It depends on the condition of the building site, whether this value should be fixed somewhat lower.

It is to be expected that 2 mm-sheets will have stronger steel tensions in the overlap. As long as no further tests on this point exist, the above mentioned value may be used.

c) Shear dimensioning

As a rule shearing forces in tunnel linings are small. The bond between Bernold sheet and concrete is very good. A test has been made in which the shearing tensile stress was of a good 20 kg/cm² although the quality of concrete was low: 170 kg/cm². Since we know of no further tests and we want to give a value that is on the safe side,

$$\tau_{Br} \left(kp/cm^2\right) = 0{,}75 \sqrt{\beta_w} \quad \left(kp/cm^2\right)$$

should be the basic formula for dimensioning (as for steel concrete plates).

If shearing stirrups are ordered, this value may be doubled, providing that the shearing forces are absorbed by stirrups.

d) Radial forces with tensile stress

Although up to now no damages have shown in finished constructions, an evidence of radial forces should be kept for the dimensioning of bent steel concrete parts. If there are no stirrups, detaching may occur through radial forces from.

= 45 Mp/m² on.

Radial forces of such strength are rare and should be absorbed by stirrups, especially in the range of overlaps.

e) Widths of cracks

Concrete, that has been reinforced with Bernold sheets, is well secured against cracks. Tests have shown, that the ratio between the load under which the admissible width of cracks of 0.2 mm occurs and the breaking load ranges from 0.70 to 0.84. I. e., that under service loads no visible cracks need be expected.

A special calculation of crack-safety is therefore superfluous.

Summary

The tests, that have been made so far, are quite sufficient to justify the full use of boarding and reinforcement sheets according to the Bernold System, with due regard to the fundamental principles of dimensioning as described above. All the experiences made in finished constructions confirm this fact. Until today no damages of any kind could be found. However the favourable experiences gathered, are not simply a result of the fact that Bernold sheets act as reinforcers but are also based on the immediate, hollowfree concreting of the final lining after excavation, whereby an extremely favourable effect on the developement of rock pressure and the consolidation of the rock is achieved.

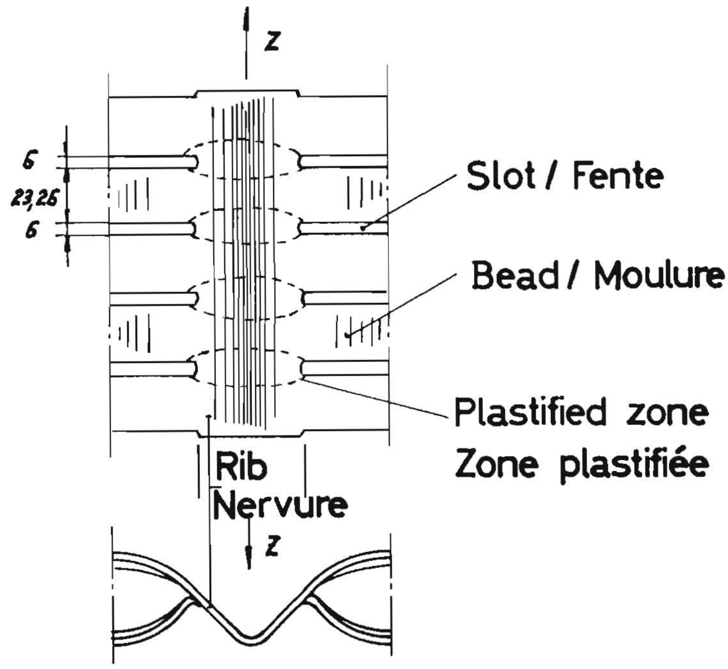
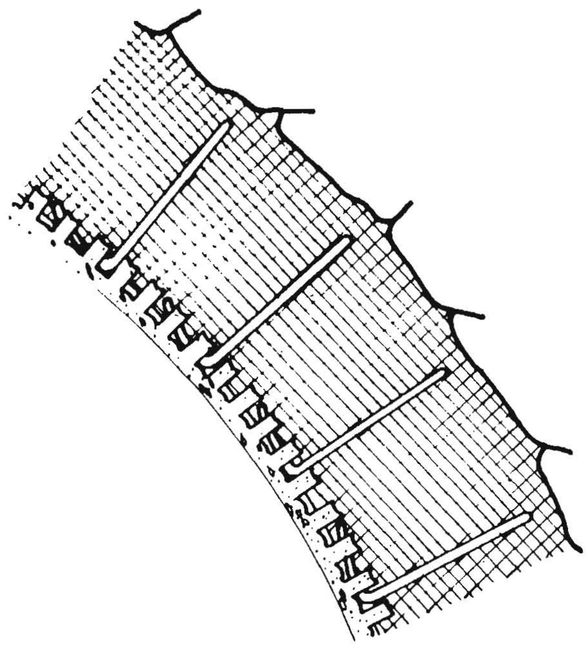
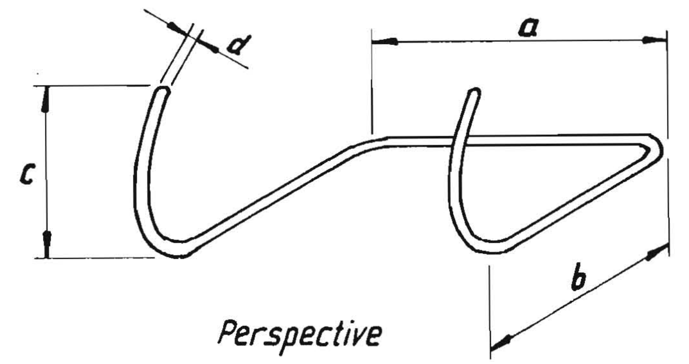


Fig. 51





a = 24 or/ou 36 cm
b = according to vault thickness selon épaisseur de la voûte
c = approx. / env. 12 cm
d = 6 ÷ 14 mm ⌀

Shearing stirrups to transfer shearing forces and absorb radial forces in zones under very high stress.

Etrier pour transmettre le cisaillement et absorber les forces radiales dans des zones fortement sollicitées.

Fig. 52

IX. Economical Advantages of the Bernold System

To make a success, a new building system must not only be advantageous from a technical point of view but also from an economical one.
Questions to this effect were asked in the article of 1967. In the meantime the necessary figures have been collected and evaluated. To sum up we can say, that the indications, which had then been given on possible cost reductions have proved true and to some extent even exceeded all expectations.

We would like to specially point out that:

Because they use the system, a large amount of tunneling sites abandon sprayed concrete, the compressive strength of which, as has been proved, lies far under that of filling concrete, which is filled in between sheet and concrete and vibrated. Post-calculations have shown, that the costs for sprayed concrete are 50—80% higher than those for filling-concrete PC 300.
In other cases the contractor renounces to steel or to be more exact to centering arcs. This means a reduction in weight of 50%.
If one calculates the actual weight of the built in centering arcs, the weight is of 150% as compared to the 100% of the Bernold sheets. These figures only comprise the actual delivery, without higher installations costs.

Special attention was given to the **installation costs,** since an identical base of calculation cannot be adopted on every building site or in every case.
The following indications are average values for a modern, well equipped and well organized building site, which have been compiled from the figures and experience gained on several building sites, on which the Bernold System was used for the construction of galleries and tunnels.

Work per m² of lining

Average tunnel developement 15.0 m	Worker minutes
1. Transport tunnel entry — place of use	2.1
2. Moving and placing of a fitting arch	12.4
3. Putting up and locking of sheets of 1, 2, 3 mm	22.0
4. Putting up of the front boarding	14.3
5. Installation and operation of the concrete conveying machine and equipment Filling in and vibrating of the concrete	21.6

Average time: 72.4 worker minutes per m²

The total building costs including all supplements amount to a total of sfr. 80.— to 120.— per m², when the concrete lining has a thickness of 20 cm.

Comparison of offers

A graphic chart of the comparison of offers shows how economical the Bernold System is, when several methods are genuinely competing. It is interesting that everyone of the 5 contractors has calculated an average reduction of 1.1 million with the Bernold lining as compared to the traditional steel linings, that means approximately 11% of the total building costs — but approx. 38% of the lining costs. Experience has shown that the economical advantages of the Bernold System even increase with the degree of difficulty in the excavation.

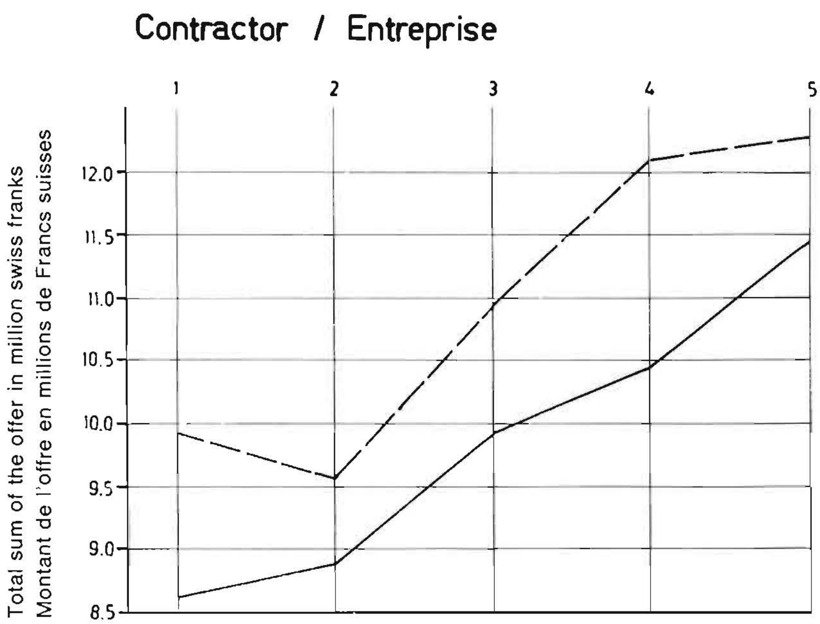


Fig. 53

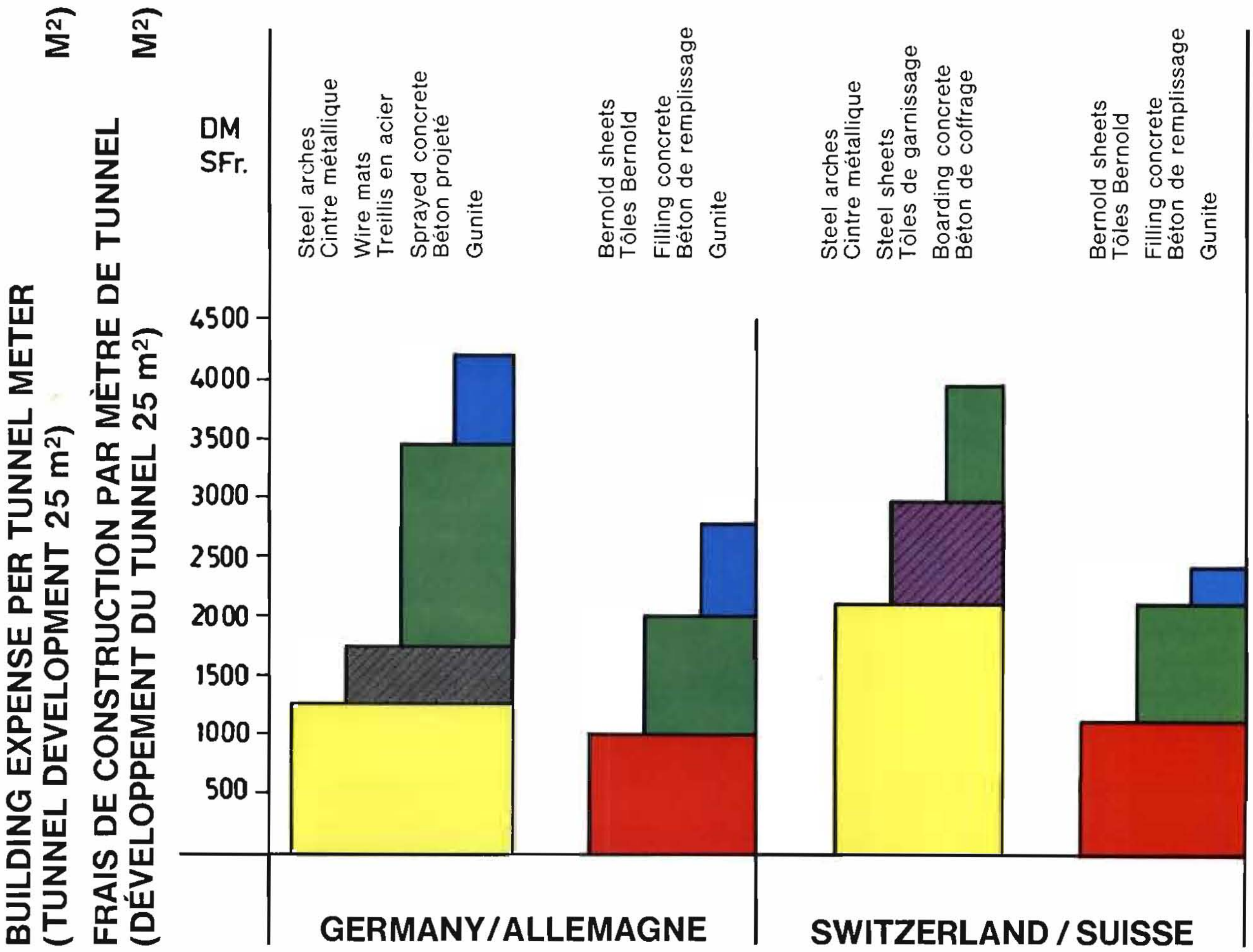


Fig. 54

A graph of the actual and total building expenses for two underground buildings in Switzerland and in Germany, which were built under similar rock pressure and working conditions, clearly show the economical advantages of the Bernold System for modern tunnel- and gallery-construction.

We should like to close with a few lines from the expertise by Prof. Dr. Ing. H. Duddeck, Institut für Statik, Technische Universität Braunschweig:

"THE BERNOLD SHEETS ARE WELL SUITED FOR THE REINFORCEMENT OF CONSTRUCTIONS, WHICH ARE MAINLY UNDER COMPRESSIVE STRESS.

THIS IS TRUE FOR THIN WALLED LININGS OF TUNNELS AND GALLERIES.

AS TENSILE REINFORCEMENT FOR BENDING STRESS ONLY, THE SHEETS MAY BE EFFICIENTLY USED WITH LARGER OVERLAPS OR REINFORCING IRONS TO STRENGTHEN THE JOINTS.

THE BERNOLD SHEETS ARE NOT ONLY ADVANTAGEOUS FOR THE WORK AT THE DRILLING FRONT (PROTECTION FROM ROCK FALL, SIMPLICITY OF INSTALLATION, COMBINATION OF BOARDING AND REINFORCEMENT IN ONE STRUCTURAL ELEMENT), BUT THEY ALSO MAKE POSSIBLE THE USE OF CONSOLIDATED LOCAL CONCRETE, (BETTER QUALITY AND SMALLER DIFFERENCES IN SOLIDITY THAN WITH SPRAYED CONCRETE).

TESTS AND CALCULATIONS HAVE PROVED THAT CONCRETE LININGS, WHICH ARE REINFORCED WITH BERNOLD SHEETS, ARE PERFECTLY SUITED FROM A STATICAL POINT OF VIEW FOR TUNNELS AND GALLERIES UNDER ALMOST ANY KIND OF STRESS."

Buchdruck + Offset AG Bad Ragaz