

TECHNICAL REPORT

THE DU TOITSKLOOF TUNNEL PROJECT

THE CONSTRUCTION OF THE (SOUTH) SOFT GROUND TUNNEL

WESTERN CAPE, SOUTH AFRICA

PREPARED FOR:

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GLOSSARY

- Arch : curved roof of an underground opening.
- Bulkhead : A barrier placed at a tunnel heading to prevent inflow of water or shattered rock into the tunnel.
- Centre-line : The line at the centre of the finished concrete lining of circular cross-section.
- Drift : A small tunnel driven ahead of the main tunnel bore.
- Grout : Mixture of cement, water and for special purposes, various additives, pumped into fractured rock to fill voids, seal off water flows, increase strength, etc.
- Mole : A tunnelling machine which cuts and abrades a tunnel of circular cross-section.
- Permanent Support : A concrete or shotcrete lining of the tunnel.
- Pilot bore (tunnel) : A small tunnel opening driven in advance of the main, larger bore.
- Portal : The entrance to a tunnel, commonly situated in decomposed, weathered rock areas, as provisions for protection and support of the tunnel entrance and approach.
- Rock bolt : Steel bolt split at one end or with an expansion head inserted into drilled hole in rock to support rock.
- Support : Any fabricated structure, steel, wood or concrete, placed to prevent failure of rocks or ground around underground opening.

- Temporary support : Support placed to support tunnel rock or ground until permanent support can be installed.
- Tunnel face : The tunnel heading.
- Tunnel section : Outline of tunnel as measured at right angles to the centre-line.
- Tunnel support : Wood, steel or concrete structures placed to prevent collapse or failure of the tunnel.
- Tunnel machine : A device rotating on a horizontal axis with cutting edges which enables excavation of a tunnel of circular cross-section.

INTRODUCTION

The four kilometre twin-bore Du Toitskloof Tunnel is situated in the Klein Drakenstein mountain range, approximately 60 kilometres from Cape Town, near the town of Paarl. The tunnel will form part of the new National road system, upgraded to a freeway standard, linking Johannesburg and Cape Town.

The subject of this report is the construction of the Du Toitskloof tunnel, with emphasis on the excavation and construction of the soft ground tunnel, by means of artificial ground freezing for ground support and stabilisation. The soft ground section of the tunnel being situated at the Western portal, and forming approximately 168 metres of the four kilometre main tunnel.

The objectives of the report are:

1. to investigate the general history of tunnelling through the ages.
2. to investigate the general aspects and conditions surrounding the Du Toitskloof Tunnel project.
3. to investigate the pertinent geological investigations and conditions which prevail within the tunnel project.
4. to investigate the formation of the main tunnel within the Soft Ground tunnel contract.
5. to investigate the application of the system of artificial ground freezing, inserting ground stabilisation, within the decomposed-weathered granite zone of the main soft ground tunnel project.

6. to investigate the methods of testing and monitoring the freezing process being applied to the soft ground for implementing temporary support.
7. to investigate the application of a reinforced Shotcrete lining against the frozen ground of the Du Toitskloof Soft Ground tunnel.
8. to investigate the possible alternatives for the excavation of the Soft Ground section of the Du Toitskloof tunnel.

The information on which this report is based was gathered, by means of conducting structured and unstructured interviews with various members of the Du Toitskloof Tunnel projects professional team. Further information was gained from consulting various magazine articles, reports etc., these either being published or unpublished, and were produced locally and/or abroad. Furthermore, information gained from various sources was supported by readings made from books concerning tunnelling operations and techniques.

The (south) Main Soft Ground tunnel contract was completed in 1983, due to this, many of the original project members have moved to various locations in South Africa, causing difficulty in conducting constructive interviews. But fortunately, some members of the contract, have settled in Johannesburg and Cape Town making interviews possible. Furthermore, most of the original projects consultants are still located in Cape Town and at the Du Toitskloof tunnel site itself, this allowed for visits to be made to the tunnel site itself, which proved to be very informative and benefited the write-up of this report.

CHAPTER 1

INTRODUCTION AND HISTORY OF TUNNELLING

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CHAPTER 1

INTRODUCTION AND HISTORY OF TUNNELLING

1.1 PURPOSE OF TUNNELS

Demand on passenger and goods transportation have increased with social development. Rivers, mountains, seas etc. have been barriers which have interrupted the flow of transportation of passengers and goods.

To overcome these barriers, man has developed methods to overcome these obstacles, some of these methods may be the construction of bridges spanning rivers and by tunnels underpassing mountains and even tunnels underpassing seas, e.g. proposed tunnel underpassing the North Sea.

These methods play an important part in developing the social unity of mankind.

The purpose of tunnels is to achieve the direct transportation of passengers and goods through barriers. Depending on the type of barriers to overcome and the type of transportation to be achieved, tunnels can be classified into various types.

These various types of tunnels are grouped according to the purpose fulfilled by the tunnels.

The types of tunnels can be grouped into two main groups, namely:

1) CONVEYANCE TUNNELS

Examples of these are: tunnels from industrial supply (water tunnels), hydroelectric power station tunnels and tunnels for the

transportation of
public services.

ii) TRAFFIC TUNNELS

Examples of these are: railway tunnels, road
tunnels, pedestrian tunnels,
subway tunnels under cities.

1.2 DEVELOPMENT OF TUNNELLING

From prehistoric times men have made use of underground space, both natural and manmade, for a multitude of needs. The beginning of man's efforts to go underground by means of subterranean excavations have been lost in the haze of antiquity. Because of limited tools, much of the work was carried out in very soft rock and soft ground which has subsequently collapsed or been eroded away, so that no evidence is now available.

Some of the earliest manmade subterranean excavations are the catacombs of Malta, Rome, Paris and Naples. The catacombs were formed by the excavation of limestone areas forming tunnels and niches which were generally used as small chapels, meeting places and for burials.

The Bible makes reference to the Sinai copper mines operated since the Bronze Age 3000 BC, with further evidence of the stone age flint mines 13000 BC, formed in soft chalk areas in Europe.

The ancient Egyptians obtained their gold from mines excavated by slaves, further the Hallstatt salt mine positioned in the Austrian Alps, and which is still in operation, dates back to 2500 BC.

In the 4th century BC, in Asia Minor, Gorema, there are remarkable remains of subterranean excavations forming villages with many tunnels and caverns, these being formed, by excavations being carried out by hand and with

the use of picks, in the soft tufa of the region. These underground villages were connected by many long tunnels.

During the 4th century BC, in the Middle East, especially in Egypt, aqueducts were excavated, to carry water for many kilometres, beneath the desert, to cities, some being as long as 12 kilometres, and are evident by the typical shafts being excavated at irregular intervals along such aqueducts.

Many other tunnels were constructed in ancient times in the search for precious metals, by the Aztecs, Incas and by people of India, Persia and Egypt.

Tunnelling methods developed slowly over the ages, the first method of tunnelling was by hand, and with the aid of some simple tools, such as picks and scrapers, evidence of these were found in flint mines of the Stone Age (Drawing No. 1; a Deer antler pick, simple tool for excavation purposes).

At the beginning of the Bronze Age a new technique of tunnelling was developed, using the wedging technique, this involves the use of wooden wedges soaked in water and then driven into cracks of the rock structure, this method was still in use up till the 18th century AD.

The early Egyptian tunnellers used the wedge and hammer technique, drilling holes were made by means of bow drills and tubular copper and bronze drill bits with abrasive cutting ends. This technique was supplemented by the use of large dolerite balls mounted on suspended rams, which were used to spall off the rock face by striking the rock face.

Another technique of tunnelling was developed at the end of the Bronze Age, this was the technique of fire-setting. The technique of fire-setting occurred where a fire-wall

was built against the rock face to be excavated, when the rock face became red hot, then the heated rock face was wetted, by spraying it wet with water or sometimes even vinegar. (See drawing 2, typical application of the technique of fire-setting). This caused the rock to spall off due to the rapid cooling effect of the water (vinegar) on the rock face.

During the dark ages there were no significant developments in tunnelling and few tunnels were constructed.

Most probably the first of the modern civil engineering tunnels, was the construction of the Malpas tunnel on the Lonquedoc canal, in Southwestern France, between 1679 and 1681. It was 157 metres long, 6,7 metres wide and 8,2 metres high, furthermore it was most probably one of the first tunnels excavated with the use of gunpowder.

1.3 USE OF EXPLOSIVES IN TUNNELLING

The first signs of the use of explosives (gunpowder) in the excavation of tunnels, occurred at the Malpas tunnel on the Lonquedoc canal, in 1679.

In the following century many tunnels were excavated with the use of gunpowder, this was the start of a new era in tunnelling, the use of explosives as a means of excavating tunnels.

In the middle of the 19th century many of the great Alpine tunnels were constructed using gunpowder for excavating hard rock faces. One of these tunnels being the Fréjus tunnel, which was started in 1857 and completed in 1870, the tunnel was approximately 12 kilometres long and with a rock coverage overhead of 1220 metres.

Another well known tunnel constructed through the Hoosac Mountains during the 19th century was the 8 kilometre long Hoosac railway tunnel, in Western Massachusetts, started

in 1858 and completed in 1874. The Hoosactunnel construction introduced the use of the mechanical air compressor, and was also responsible for the introduction of electric firing of the powder charges, which before was carried out with the use of slow fuses, which often caused fatalities.

The invention of Nitroglycerine in 1847, was used for the first time in the United States in the Hoosac tunnel in 1866. Nitroglycerine is a hazardous substance, as it is very susceptible to shock, which causes unsafe working conditions for its users.

In 1867, dynamite was introduced by Alfred Nobel, which proved to be more handable by its users, and was used for the first time in the St Gothard Tunnel through the Alps in 1872, and has been used in many tunnelling contracts ever since.

But dynamite has its disadvantages, in that it produces lethal fumes after being used and it is costly, therefore further research was carried out to produce a more suitable substance. Since 1950 a more acceptable substance has been used, this is ammonium nitrate mixed with fuel oil and detonated with a dynamite detonator, this substance is safe to handle and place.

1.4 DEVELOPMENT OF DRILL HOLES (BLAST HOLES)

The methods in forming better more accurate and more rapid, "blast holes" has developed greatly since the Egyptians first used copper drill bits. Later the Romans used iron drill bits which lasted until the introduction of steel products, in the 17th century. The use of steel drill bits and which were driven by hand, achieved remarkable rates of drilling, but the drill bits required frequent resharping.

With the introduction of the mechanical compressed air drill the rates of drilling increased, but at the cost

of the steel drill bits, as the steel drill bits required more frequent sharpening than before. The problem now was to develop a drill bit from some type of material that had properties of extreme durability and hardness, and that required minimal sharpening.

The discovery of tungsten carbide in Germany in the 1920's and which was kept a secret until the end of the Second World War, was the solution to the problem, and tungsten is now used in virtually all percussion drill bits.

1.5 DEVELOPMENT OF TUNNEL SUPPORT

The development of support to the walls and roofs of tunnels, has advanced greatly over the centuries.

The support requirements generally depend on the types of ground-rock that is to be excavated, as well as the required dimensions of the tunnel.

The early tunnellers tried to minimise the span of tunnels, so that minimal or no support would be required for these tunnels.

The first known support of semi-subterranean structure, was the use of whale bones and jaws to reinforce sod and stone walls of houses of the eskimo Denbigh culture in 4000 BC.

Where minimal support was required, the use of wooden posts and beams were used, and is still in limited use today. But as requirements for larger spans in tunnels were required, the tunnellers had to develop more advanced permanent and temporary support systems. Linings to tunnels, i.e. supports, consisted of stone in Roman times, and subsequently developed to the use of bricks, concrete and precast elements.

Complex timber support systems are still being used today, especially for temporary support systems, as timber offers great flexibility in installation to changing conditions in rock structure and support requirements. The introduction of cast iron and steel supports systems for tunnel linings, gave tunnelling an advantage in that larger spans could be constructed, as cast iron and steel members could support greater loads. But the use of cast iron and steel members removed the aspect of flexibility of adapting the changing requirements within the rock structure and support requirements.

In the 1940's, there was a major development with the introduction of rock bolts, rock bolts can be used to replace the traditional support systems and they can also be used as temporary and permanent support systems. A rock bolt is a bolt that is placed into a predrilled hole in the rock structure, it can be embedded in the rock structure by the use of grout or with the use of adhesives. Once the rock bolt is held tight in its desired position then a washer (metal plate) is applied held against the rock structure with a bolt, the bolt can then tensioned so as to hold the rock faults, cracks together. Typical rock bolts and application thereof (see diagram 1, a typical rock bolt).

Rock bolts assist the rock structure to provide its own arching and beam effect within the tunnel profile. Rock bolts can be rapidly installed, so as to allow the rock structure to maintain its integrity and not lose its inherent shear and compressive strengths, which may be caused by the opening of fractures and joints within the rock structure.

Rock bolts can be installed as unstressed components, or more commonly as prestressed components, which fulfil the requirements of varying rock structures, shapes and sizes of tunnels.

Another method used to support rock faces, is the use of wire mesh pinned to the sides and roof of the tunnels and then the mesh is covered with shotcrete (sprayed concrete), this was developed in the 20th century, and is in common use in most tunnelling projects of today. The method prevents the rock and/or ground face from relaxing and therefore allows the rock structure to maintain its inherent strength, so that the rock structure can support itself (see diagram 2, application of rock bolts for tunnel support).

1.6 TUNNELLING MACHINES

Tunnel bore machines, also known as moles or TBM are machines that have existed for many years, but have been generally unsuccessful until recently. Tunnel bore machines have been successful in general soft rock and/or soft ground excavations, but are becoming more and more successful in very hard rock excavations.

The major problems lie where the mole has to operate in various types of ground within a single project. A mole is a machine that excavates a rock face with the use of rotary cutters, as it advances along the proposed tunnel profile.

The use of tunnel bore machines is very costly and is used in very few projects, they have mainly been used in projects in Europe.

1.7 ARTIFICIAL GROUND FREEZING

The art of tunnelling has progressed over approximately 5000 years, from the arts of wedge and hammer, fire-setting etc. to the modern computerised tunnel bore machines.

But every project develops its own problems and difficulties, as the method of tunnelling depends greatly on the ground conditions to be excavated.

In some cases, especially where the ground is decomposed and water-logged, a freezing process may be used, this enables the ground to achieve certain suitable standards of stability.

But this method is generally used as the last resort in the excavation of tunnels, as it is very complex and timeous method.

Natural forms of freezing which aids the excavation of tunnels, basements etc. have been utilised. For example in Siberia, open cast gold mines are excavated during the winter months so that the walls of the mine are stable due to the natural freezing of the ground.

Another example was in France in 1852, where a mine shaft was sunk during the presence of severe winter frosts which aided ground stability, without such conditions it would have been impossible to excavate such a shaft.

The first known use of artificial ground freezing to stabilise the unstable waterlogged ground, occurred in South Wales coalfield, in 1862, and was carried out by Siebe Gorman Co. Further developments were carried out in 1880 in the Ruhr coalfield by the German engineer Potsch.

Due to extensive research and development of artificial ground freezing, and with the aid of computerised testing of the freezing process, specialised drilling methods, etc., the freezing process can now compete favourably with other improvement methods of ground stabilising, such as extensive dewatering of site.

But one must remember the artificial ground freezing is a very costly and timeous process, and should only be utilised as the last resort.

CHAPTER 2

THE DU TOITSKLOOF TUNNEL PROJECT: AN OVERVIEW

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CHAPTER 2

THE DU TOITSKLOOF TUNNEL PROJECT: AN OVERVIEW

2.1 INTRODUCTION

The Du Toitskloof tunnel is situated in the Klein Drakenstein mountain range in the Western Cape. The tunnel will form part of the national road link (N1), between Johannesburg and Cape Town, and is approximately 64 kilometres from Cape Town and a few kilometres from the town of Paarl (see Map 1, location plan of the Du Toitskloof tunnel). The first plans for the proposed tunnel were drawn up as early as 1938, by the National Road Board, but due to the lack of funds and the outbreak of the Second World War, the present road pass was accepted for construction instead of the tunnel.

By 1964, there were increasing traffic volumes over the present Du Toitskloof pass, and there was an expected economic growth within the Cape Peninsula in the years ahead, therefore the spotlight was again focused on a possible road tunnel through the Klein Drakenstein mountain range.

By 1970, the National Transport Commission, appointed the consultant engineers of Van Niekerk, Kleyn and Edwards to investigate the possibilities of a road tunnel through the area, so as to upgrade the present road system, to modern freeway standards.

The feasibility report, by the consultant engineers, was completed in 1974, the report highlighted that there were no other feasible routes, in producing a 'modern' national freeway over the surrounding mountains, other than of 4 kilometre tunnel through the mountain range. After considering the recommendations of the report, the National Transport Commission advised the consultant engineers to prepare a contract for the construction of a pilot tunnel.

2.2 GEOLOGICAL CONDITIONS WITHIN THE DU TOITSKOOOF TUNNEL AREA

The Du Toitskloof tunnel travels through three major ground/rock types. At the eastern portal the tunnel travels through down-faulted quartzitic sandstone for approximately 600 metres, after which it enters the main Du Toitskloof fault zone, thereafter it enters approximately 3300 metres of fresh-solid granite which forms the major part of the tunnelling works. The last 168 metres of the tunnel, from the western portal travels through a decomposed-weathered granite zone, which is water-logged, and very unstable ground conditions prevail in this zone.

This report covers the construction of the main Du Toitskloof through the decomposed-weathered granite zone, i.e. the soft ground contract.

2.3 THE DU TOITSKLOOF PILOT TUNNEL

In July 1975, the tender for the construction of the pilot tunnel was advertised, and the contract was awarded in October 1975, to the consortium of Bomar-LTA. Bomar was mainly involved in the underground works while LTA was responsible for most of the surface works. The contract started on the 5th of November, 1975 and was completed in March 1979. The final contract value was R7,4 million.

The main purpose for the construction of the pilot tunnel was to investigate the geology along the line of the tunnel and also to identify any major problem areas that might affect the construction of the main tunnel. The pilot tunnel is 4 kilometres in length and has a cross-sectional area of 10 m^2 and the pilot tunnel is off-set from the main tunnel's centre-line by 36 metres.

After fourteen months of tunnelling within the pilot tunnel, through the saturated decomposed granite zone, approximately 62 metres from the western portal, severe problems in the conditions of tunnelling began to occur. Up to this stage of the works, tunnelling had been carried out by traditional

soft ground tunnelling techniques.

In November 1976, a serious mudflow occurred at 62 metres from the western portal, the contractors tried to overcome the problems by applying techniques such as, cement grouting, bentonite-cement grouting, etc., so as to try and seal off the water flows. In February 1977, tunnelling began again, and after a further 3 metres, a large mudflow occurred forming a sinkhole from the surface 35 metres above (see diagram 3, longitudinal section through the decomposed granite zone, illustrating the massive collapse), and (see photograph 1, formation of the sinkhole at the surface, above the pilot tunnel and 2, the mudflow within the pilot tunnel).

Subsequently, it was decided to complete the last 50 metres or so, by means of artificial ground freezing.

Excavation started again, in May 1977 using the technique of ground freezing and was completed in September 1977, where the tunnel had reached the hard granite, and conventional drilling and blasting methods were used in the excavation of the fresh-hard granite. The contract was completed, some 8 months after the official completion date.

2.4 THE WESTERN PORTAL

Excavations during 1973 and 1974, revealed problems in the surface talus and decomposed granite as a result of the saturated conditions. This therefore called for a stabilising of the area in the western portal area by the construction temporary retaining walls, tied back with ground anchors, this was carried out before the excavation of the pilot tunnel was commenced (see photograph 3, temporary tied-back retaining wall - western portal) and (see diagrams 4 and 5: western portal: temporary tied-back retaining wall).

The western portal contract was awarded to Savage and Lovemore, which involved construction work to form a stable portal area and will provide for the future control buildings to the tunnel. The construction involved the erection of a piled tied back retaining walls, which would form a stable vertical face.

The height of the face is 21,5 metres and 57 metres in length with two wing walls of approximately 34 metres in length. The piles were installed by Frankpile with Ground Engineering and Piling carrying out the anchorage of the piles.

The western portal retaining wall contract was started in November 1979 and was completed in July 1981, with the final contract value being R1,8 million (see photographs 4 and 5, western portal: piled tied-back retaining wall).

2.5 THE MAIN DU TOITSKLOOF TUNNEL

The main Du Toitskloof tunnel consists of the excavation and construction of a tunnel through the decomposed granite zone, the fresh-hard granite zone and the quartzitic sandstone zone at the eastern portal. The client and the consultant engineers however decided to divide the main Du Toitskloof tunnel project into two contracts, one being the Soft Ground contract, the first to be started and the other being the Hard Rock Contract.

The main tunnel contract was divided into two parts due to the very specialised work required in the Soft Ground tunnel construction and also that the Soft Ground contract has many unknown factors attached to it, i.e. it is considered a pioneering operation.

Furthermore, it was thought that one tendered, the lowest bidder, might have been very capable in undertaking the excavation and construction of the Hard Rock tunnel, but

may not have been adequately capable in undertaking the specialised Soft Ground contract.

Therefore to promote the well-being of the project and to protect the client, the main tunnel contract was divided into two contracts. The report only covers the Soft Ground contract.

2.6 THE MAIN DU TOITSKLOOF SOFT GROUND CONTRACT

Due to the instability of the saturated decomposed granite encountered in the pilot tunnel contract, it was decided by the client (NTC) and the consultants (VKE), to utilise the technique of artificial ground freezing, as the technique had been successfully used in the construction of the pilot tunnel.

The soft ground contract involved the construction of a lined tunnel, approximately 168 metres in length.

The tendering system for the Soft Ground contract was split into two parts, one part being a prequalification phase and the other part being a normal pricing of the contract phase.

The contract was open for tender in February 1980, and by September 1980, the consortia had been selected on the basis of the prequalification tenders. The selected consortia were then permitted to submit priced tenders for the Soft Ground contract which was completed in December 1980.

In March 1981, the consortium of LTA Construction Ltd - Shaft Sinkers (Pty) Ltd, in association with Foraky Ltd, as the ground freezing specialists, were awarded the contract.

The Soft Ground tunnel contract commenced on the 1st of June, 1981.

2.6.1 Scope of the Soft Ground Contract

The contract consisted of the excavation and construction of a 168 metre tunnel, with a cross-sectional area of 122 m^2 within the standard tunnel profile, while the cross-sectional area increased to 184 m^2 in the enlarged drilling areas.

The tunnel was divided up into five stages, stages being 32 metres in length, except in the last stage (stage 5) where the length increased to approximately 40 metres. The last 8 metres of each of the stages, one to four, was enlarged for drilling operations, to the following 32 metre stage.

The last stage, of the five stages, was longer than 32 metres as it incorporated a drainage gallery, in the transition zone between the decomposed granite and hard granite (see diagram 6, longitudinal section through the soft ground tunnel, showing the five freezing stages).

2.7 IMPLEMENTATION OF THE FREEZING PROCESS

The artificial ground freezing involved the formation of a frozen arch around the tunnel profile with a frozen wall thickness of 2 metres minimum and the formation of vertical bulkheads at the end of each 32 metre stage, with a wall thickness of 3 metres.

The frozen arches were formed with the insertion of horizontal freeze tubes, at an inclined angle, from the start of each section to the end of each section, allowing for the enlarged drilling areas, located at the end of each 32 metre section. There were approximately 50 - 60 freeze tubes inserted, for the formation of each 32 metre frozen arch (see diagram 7, cross-section through 'frozen' soft ground tunnel) and (see diagram 8, plan of first 32 metre stage, showing vertical bulkhead and position of freeze lances).

The frozen vertical bulkheads were formed with the insertion of vertical freeze tubes, inserted from the surface, and each bulkhead becoming deeper and deeper, as the work moved from stage to stage, i.e. the vertical bulkheads were formed at greater depth within the mountain slope, after each 32 metre stage. There were approximately 22 - 25 freeze tubes inserted, for the formation of each vertical bulkhead.

The vertical and horizontal freeze tubes were inserted into predrilled holes, which required accurate alignment so as to promote adequate formation of the frozen arch and frozen vertical bulkheads, i.e. so as to provide structural stability within the tunnelling operations.

The freezing process was monitored by the installation of strategically placed "observation" holes these holes contained temperature recording devices - thermocouples, which gave an indication of the ice growth. Other temperature measuring techniques were also used to determine the formation of the frozen ground (see diagram 8) and (see drawing 3, typical freeze tube circuit connections).

2.8 THE FREEZING PLANT

The freezing plant was imported from the United Kingdom, and was operated by the freezing specialists, Foraky Ltd. The freezing plant circulated a refrigerant through the freeze tubes, namely calcium chloride brine solution, at a temperature of approximately -40°C . The freezing plant had a capability of producing 200 tonnes of ice per day.

2.9 THE PERMANENT LINING TO THE MAIN SOFT GROUND TUNNEL

After the tunnel had been mechanically excavated, a shotcrete lining was placed, the excavation and placing of the shotcrete lining (support) was carried out in 2 metre intervals, along the length of each 32 metre stage.

The permanent support, is formed of steel arches, mesh and layers of shotcrete and has a final thickness of 450 mm, and was placed and completed before the freezing process of each 32 metre stage could be switched-off.

The main Soft Ground tunnel has become South Africa's most expensive "rands per metre" tunnel. But its contribution to the completion of the main Du Toitskloof Tunnel has made it justifiable, as the tunnel project has become one of South Africa's most prestigious civil engineering projects ever undertaken. Furthermore, the Du Toitskloof Tunnel has become South Africa's longest road tunnel and probably the longest in Africa, and has offered great experience to South Africa's Construction Industry, especially in the art of artificial ground freezing.

CHAPTER 3

GEOLOGICAL ASPECTS OF THE DU TOITSKLOOF TUNNEL

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CHAPTER 3

GEOLOGICAL ASPECTS OF THE DU TOITSKLOOF TUNNEL

3.1 INTRODUCTION AND DEVELOPMENT

The chain of mountains that stretches from Clanwilliam in the north to Cape Agulhas in the south, and particularly the Klein Drakensteinberge, Western Cape, are the mountains that are in the spotlight for the construction of the Du Toitskloof road tunnel.

At present, the national road link (N1) between Cape Town and the areas of the north, passes over the Klein Drakensteinberge.

Due to rapid economic development of the Western Cape over the past years, the Department of Transport decided to upgrade the present national road system. The present N1 is to be upgraded to a modern freeway standard, so that the expected increase in transportation between the north and south could be catered for.

The Klein Drakensteinberge forms a barrier for the proposed uplifting of the N1 to a freeway standard, therefore investigations were started to overcome this barrier.

The consultants to the Du Toitskloof project were VKE (Van Niekerk, Kleyn and Edwards) who were appointed by the National Transport Commission (client), to produce solutions to the upgrading of the present N1 road system over the Klein Drakensteinberge or produce alternative solution to the problem.

After years of investigations, especially during 1971 and 1972, VKE consultants presented to the National Transport Commission (NTC) the feasibility reports regarding this matter. The reports stated two major aspects; that the present N1 pass over the Klein Drakensteinberge was uneconomical to upgrade to a modern freeway standard. Furthermore, that all the possible

routes around the mountain range were impractical and uneconomic to achieve a modern freeway standard.

Therefore the only alternative solution was to construct a twin-bore road tunnel, this would upgrade the present N1 pass over Du Toitskloof to a modern freeway standard. After feasibility studies had been carried out and presented to the NTC (1972), the method of the transportation link between the north and south by means of a 4 kilometre road tunnel through the Du Toitskloof Area, was accepted by the client.

Detailed planning of the proposed tunnel was carried out from 1972 to 1974, after which it was submitted to the client for approval.

After approval, it was decided by the consultants to construct a pilot tunnel, in line with the proposed main tunnels. The pilot tunnel would be used to obtain information that would enable VKE consultants, in association with the overseas consultants, appointed by the client, Electrowatt Engineering Services Ltd, Zurich, Switzerland, to design the method of construction of the main Du Toitskloof road tunnel.

One of the most important aspects of the preliminary works is the exploration of the geological conditions. The geological environment decisively affects both the loads acting on the proposed tunnel and the choice of the preferable tunnelling method(s) to be employed.

Before the pilot tunnel could be constructed, the pilot tunnel location had to be determined by using the position of the existing N1 mountain pass location but the final position was determined by the prevailing geological conditions in the areas.

The first method of obtaining the geological conditions of the proposed tunnel location was carried out by the drilling

of exploratory boreholes in the region, approximately 60 in number (see diagram 9, typical core sample results obtained from an exploratory borehole). From the borehole results and past geological surveys, the position of the pilot tunnel and therefore the main tunnel could be plotted, the results obtained from the boreholes, were the first indications of the geological conditions present in the tunnel 'area' (see map 2, geological survey result showing granite outcrops within the tunnel location). The pilot tunnel, when excavated and constructed would justify previous geological results, and further information will be obtained regarding factors which would affect the design and construction of the main tunnel, e.g. rock pressures, rock types, etc.

Investigations carried out in the pilot tunnel will highlight the possible problem areas in the construction of the main tunnel.

The pilot tunnel was constructed by LTA/Bomar, and was started in October 1974, and has a cross-sectional area of 10 m^2 , furthermore it was constructed down the centre-line of the proposed north main tunnel.

From all the geological investigations carried out, the main geological features along the proposed main tunnel route could be shown (see Diagram 10, longitudinal section along the centre-line of the pilot tunnel) and (see diagram 11, geological sectional plan at the crown of the tunnel).

Most of the 4 kilometre tunnel occurs in intact granite i.e. fresh granite. The tunnel also includes an area which incorporates the main Du Toitskloof fault, this fault is caused by the down-thrusting of the Table Mountain series (TMS) sandstones against the fresh granite zone. The tunnel also includes four zones of fractured granite, which is also very unstable when excavated and requires extra support during the construction stage. These zones of fractured granite exist at the western portal end and the eastern

portal end.

At the western portal (Paarl area), there is a zone of decomposed-weathered granite, the decomposed-weathered granite is overlain by transported granite and sandstone talus, i.e. material that has originated from the main rock zones, but due to time has eroded away - broken off, and come to rest on top of the main rock zones.

The decomposed-weathered granite area, is the area that has caused a considerable number of problems in the excavation of both pilot and main tunnels. This is due to the ground being very unstable and water-logged. To overcome this problem, the technique of artificial ground freezing was applied. The technique stabilised the ground conditions, enabling the excavation of part of the pilot tunnel and 168 metres of the main south tunnel.

Furthermore it should also be noted, that the pilot tunnel was also constructed to allow for the haulage of materials and personnel from the eastern portal area, as this would eliminate large storage requirements at the eastern portal, where space was limited. The pilot tunnel also allowed for the haulage of tunnel spoil to areas in the western portal, where fill is required for the construction of the approach roads to the western portal. The pilot tunnel allows for the drainage of the main tunnel, by cross connections between the pilot and main tunnel, the water was drained under gravity from the eastern portal to the western portal. The pilot tunnel also allowed for possible delays in the excavation of the main tunnel, as headings could be opened from the pilot tunnel to overcome delays; i.e. to allow for continual excavation of the main tunnel. Finally, the pilot tunnel also allowed for improvements to be made to the excavation and construction of the main tunnel, e.g. dewatering applied from the pilot tunnel to areas in front of the main tunnel's working face.

3.2 GEOLOGICAL CONDITIONS WITHIN THE DU TOITSKLOOF AREA

In the Du Toitskloof tunnel area, there is Wellington granite pluton, i.e. a body of rock that has crystallised deep in the earth's crust and has been exposed by erosion; and the Wellington granite pluton is covered by the quartzites of the Peninsula Sandstone Formation.

The main tunnel cuts through approximately 3,4 kilometres of the granite formation and approximately 0,7 kilometres of the down-faulted quartzitic sandstones.

The eastern portal is formed in rock of the Table Mountain group, while the western portal is located in residual soil, or otherwise known as soft decomposed granite which forms 168 metres of the tunnel excavation (see diagram 12, section through decomposed granite zone of the tunnel: western portal) and (see photograph 6: eastern portal: quartzitic sandstone).

The design and feasibility of the tunnel was determined mainly from the values obtained by the geological investigations.

3.2.1 RESIDUAL (DECOMPOSED) GRANITE OF THE DU TOITSKLOOF TUNNEL

3.2.1.1 The nature of the material

The granite is decomposed into a residual soil to a depth of up to 50 metres, at the western portal (Paarl side). The mineral types that exist within a sample of the residual granite, were kaolinite the dominant product, with other minerals such as quartz orthoclase, minor plagioclase and mica.

According to the Irfan and Dearman's (1978) scale of mass weathering grades in granite, the granite encountered in this section of the tunnel, is a granite that is completely to highly weathered granite.

The residual soil showed remarkable strengths, in the undisturbed state, but when the soil was disturbed the soil lost its inherent strength easily.

Generally residual soil (decomposed granite) has the tendency to erode easily, but the major problem occurs where the soil is accompanied by excess water, then the residual soil deteriorates into a mud. Due to this, a ground stabilisation technique was used in the excavation of the main soft ground tunnel.

3.2.1.2 Hydrogeological conditions

Def: The science that deals with subsurface waters and related geologic aspects of surface waters.

The residual granite is affected more by the hydrogeological conditions compared to say the fresh granite zones, found along the route of the tunnel.

During the construction of the pilot tunnel, it was noticed that water-flows and excessive seepages were present along most of the residual granite section of the tunnel. The water-flows however increased towards the zones of the weathered granite areas. The water-flows became so extensive during the construction of the pilot tunnel, that grouting was applied to try and reduce the water-flows, but eventually the use of artificial ground freezing was applied to stop the water-flows, etc.

There is a general presence of water throughout this section, but the major flows occur along the weathered granite zones, i.e. mainly between the fresh granite and the weathered granite. Examples of flow rates that were recorded in the excavation

of the pilot tunnel in 1978, indicates how extensive the flow rate is, namely 8,1 litres/second was recorded in 1978, while at present the flow rate has dropped to 6 litres/second, in the decomposed granite region.

The excess water entering the tunnel profile is drained out through the western portal, as there is a 0,8% gradient within the tunnel from west to east.

The chemical analysis of samples of ground water, obtained from weepholes in the pilot tunnel, indicates that the ground water encountered has a mild aggressive property, towards steel and concrete products (Appendix 1, table results of chemical analysis of water sample, in the western portal zone).

Mineralogical and chemical tests carried out on samples obtained in the decomposed and weathered granite areas of the tunnel, indicate the types of minerals and chemicals present within such samples (Appendix 2: results of mineralogical and chemical tests on samples of decomposed and weathered granite zones).

3.2.1.3 The behaviour of the Residual Granite during the construction of the pilot tunnel

The tunnelling process through the residual granite proved to be a difficult and complex operation, as the ground encountered was unstable and even more so when water-flows were encountered.

During the first 40 metres of the pilot tunnel excavation, few water-flows were encountered and the over-head coverage of soil, i.e. overburden,

was only 30 metres, this all contributed to relatively problem free excavation of the first 40 metres of the pilot tunnel from the western portal. As the tunnelling operations started to reach the weathered granite zone, difficulties occurred as the overburden had increased considerably and water-flows became more frequent.

A major problem occurred in the excavation of the pilot tunnel, an uncontrollable mud flow occurred approximately 65 metres from the western portal. The mud flow removed approximately 2000 m³ of soil in the formation of a sinkhole, and the sinkhole also coincided with a previous exploratory geological borehole, this may have aided the formation of the sinkhole. The borehole may have caused disturbance of the soil, and the hole could have eroded away to form a sinkhole (see photographs 1 and 2, pilot tunnel mudflow and sinkhole formation).

It was also said that increases in water flows and consequent erosion of the soil was accelerated when the residual soil was disturbed, for example by excavation, drilling of boreholes, grouting operations, drilling to provide drainage, etc.

After studying the collapse of the pilot tunnel, the researchers concluded that the collapse was most probably due to an increase in stress and water pressure within the soil with increasing depth, a change from primary stress to secondary stress conditions due to excavations and the low permeability and high erodibility of the in situ material. Strong flows of water along the joints in the soil caused rapid erosion, which increased further as the flow paths were enlarged, the final

effect was a mudflow and the formation of a sinkhole.

Various techniques were attempted to improve the conditions for tunnelling, such as extensive drainage, grouting of fissures, well-points etc.; but all these techniques only improved the situation slightly. The only possible solution at this stage was to drive the existing 50 metres of the pilot tunnel by means of artificial ground freezing, with the tunnel being supported by steel ribs - arches. As the residual granite and weathered granite was extremely saturated and very unstable when disturbed, and the best possible solution at the time was the use of ground freezing which would stabilise the ground.

The technique of ground freezing would seal off all minor seepage areas and all zones of water inflows, also the technique would provide a frozen arch around the working area. The ground freezing technique would provide strength within the ground structure, approximately 10 mPa at -20°C , which is approximately 80 times stronger, before ground freezing was applied.

3.2.1.4 Weathered granite zones

The weathered granite zone is situated between the decomposed granite and the fresh granite zones. The rock at this stage of weathering is classified in "Irfan and Dearman's grades in granite", as moderately weathered and grades rapidly into slightly weathered rock, i.e. towards the fresh granite zones.

The colour of the weathered granite moves from zones of brown and olive colour to zones of blueish and grey for less weathered granite.

In the weathered granite zone, the method of excavation, was by means of blasting, which was carried out in stages of 1 metre lengths along the weathered granite zone. Support was applied with the use of rock bolts in less weathered granite zones.

In areas where strong water occurred and the granite was more weathered, additional support was required, such as steel arches, shotcrete-mesh, etc.

The degree of weathering of granite material decreased from the beginning of the weathered granite zone (from the western portal), to the junction between the weathered granite and fresh granite.

3.2.2 SOLID (FRESH) GRANITE OF THE DU TOITSKLOOF TUNNEL

The excavation (blasting operation) of the fresh granite, constituted approximately 83% of the tunnel excavation, and was generally excavated with few problems. Problems that did occur were in the fault zones.

The granite types found at the Du Toitskloof project, are, Porphyritic granite, fine-grained granite, and gneissic granite.

3.2.2.1 Porphyritic granite

Porphyritic granite was encountered for approximately 2 kilometres of the pilot tunnel, which

is approximately 50% of the excavated material of the pilot tunnel, the rock is medium hard to hard, and softer areas were encountered where weathering had taken place. The colour of the granite varies from grey, to grey with red and pink shades, plus spottings of black, white and green. Types of minerals present were quartz, orthoclase, microperthitic, microcline, biotite and chlorite.

This type of granite requires little or no support.

3.2.2.2 Fine-grained granite

Fine-grained granite was encountered for approximately 235 metres of the tunnel i.e. 6% of the excavated material of the pilot tunnel.

Fine-grained granite is very hard rock and varies in colour from reddish brown to greenish grey. Minerals encountered in the granite classification were quartz, orthoclase, muscovite, chlorite and biotite. The fine-grained granite required little or no support in the excavation of the pilot tunnel, and fine-grained granite has a compressive strength of approximately 207 MPa, which provides adequate support if not fractured badly, then the rock face may require rock bolts for permanent support.

3.2.2.3 Gneissic granite

Gneissic granite was encountered over 730 metres of the pilot tunnel i.e. approximately 20% of the excavated material of the tunnel.

The Gneissic granite is "very hard" to "hard",

and where this rock occurred in faults zones, support was required. Support was in the form of steel arches and shotcrete. The colour of the Gneissic granite varies from grey, to grey-red, and including green, white and black spottings.

3.2.3 FAULTS WITHIN THE DU TOITSKLOOF TUNNEL

There are two sets of faults, the one being older than the other, the older one being confined to the granite zone, the other fault being represented in both the sandstone and granite sections of the tunnel (see cross-section through tunnel). The older faults developed within the granite zone before the deposition of the rocks of the Table Mountain group, the younger set of faults developed during the post-Cape phase of folding.

3.2.3.1 The Du Toitskloof Fault

The Du Toitskloof fault has resulted due to the downward thrust of the sandstone zone, which is side by side to the granite zone.

The width of the Du Toitskloof fault is approximately 150 metres, and it is composed of fractured sandstone and altered granite, the altered granite forming a band of 50 metres within the fault. The altered granite consists of soft rock, highly weathered and sheared material, and consists of the following materials: quartz, orthoclase, calcite, clay, sericite and chlorite. The fault is very susceptible to erosion when in contact with water, but fortunately there was not much water encountered within this fault region.

Shotcrete was applied as a support, but due to the formations of cracks in the shotcrete, due to movement, steel arches were placed, and a concrete floor was cast over the altered granite zone, this provided adequate temporary support, until the final concrete in situ lining is placed.

3.2.3.2 Faults in the granite zones

Approximately 4% of the granite encountered in the tunnel was fractured material, and about 70% of this fractured material required permanent support, in the form of shotcrete and/or steel arches. Three characteristic zones of alteration were present in the pilot tunnel (alteration occurs where chemical and physical changes affect the rocks and minerals, during their lifetime).

- 1) Zones of sheared granite with clay along the fault contact, less than 300 mm wide;
- 2) Soft friable (easily crumbled) zones, approximately 0,5 metres to 2 metres wide;
- 3) Fractured zones, approximately 0,2 to 4 metres wide.

Investigations within the pilot tunnel showed that fault widths were irregular and the degree of deterioration of the materials within the faults were also irregular.

Finally support requirements to the fault zones are in the form of shotcrete and mesh applications and included steel arches where the fault zones were more unstable and required additional support. Some of the fault zones were in the presence of water penetration, which caused additional problems,

and which were overcome with the placing of plastic membranes, drainage facilities etc.

In conclusion, all the geological investigations involving inspection, testing, monitoring, analysis of core samples, results, solutions, alternative solutions, etc., which were carried out within the pilot tunnel, was mainly for the benefit of the construction of the main Du Toitskloof tunnels, so as to be able to attempt to produce a "perfect" solution to all tunnelling operations and construction.

The information obtained from these investigations, testing, etc., were used by the consultants etc. to enable them to design solutions for all areas within the main tunnels, and to establish all possible problem areas to which appropriate solutions could be applied.

The pilot tunnel is in fact an aspect of the planning stage of the main Du Toitskloof tunnel contract.

CHAPTER 4

THE DU TOITSKLOOF SOFT GROUND TUNNEL FREEZING PROCESS

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CHAPTER 4: THE DU TOITSKLOOF SOFT GROUND TUNNEL FREEZING PROCESS

4.1 INTRODUCTION TO THE FREEZING PROCESS

The use of natural ground freezing has aided man for centuries in the exploration of areas below ground level, and was mainly experienced in countries in sub-zero temperatures.

The first recorded use of the technique of artificial ground freezing was performed in the South Wales coalfield in 1862, by the Siebe Gorman Company. This technique was further developed by Potsch, a German engineer at the Ruhr coalfield in 1880. Generally artificial ground freezing has developed slowly over the years, but major developments have occurred over the past 25 years, especially developments in the fields of refrigerants and refrigeration plants, which make the freezing process more economical and of a higher success rate.

The art of ground freezing is becoming more and more advanced, especially in developments in the monitoring and testing of the artificial ground freezing process, in which highly sophisticated electronic equipment is being utilised.

The ground freezing system has been used in many countries, especially in European regions, where the environmental conditions and dense building coverage makes this technique highly favourable.

The use of artificial ground freezing had never been performed in Africa, until the technique was utilised in South Africa at the Du Toitskloof road tunnel project (Cape).

The first application of the technique occurred in the construction of the pilot tunnel which has a cross-sectional area of 10 m^2 and a length of 3925 metres, ground freezing was applied to the last 50 metres of the soft ground section of the pilot tunnel. The technique

was brought about due to a collapse (mudflow) earlier on in the construction of the pilot tunnel. The pilot tunnel was completed in 1979, and the knowledge gained of ground conditions, construction problems, and general constructional experience, aided the project consultants and their associates overseas, in designing the main Du Toitskloof tunnel with the highest possible success rate, considering the relatively poor ground conditions present.

The major problems in design, was in the saturated decomposed-weathered granite zone, situated at the western portal (Paarl side) which involved the excavation of unstable decomposed-weathered granite.

Due to the decomposed-weathered granite being water-logged and very unstable when disturbed, and as the freezing method was successfully carried out in the construction of the pilot tunnel, the technique of ground freezing was adopted for the construction of the main soft ground tunnel. The freezing process involved the freezing of approximately 145 metres of decomposed granite and approximately 11 metres of weathered granite which underlies the decomposed granite zone and overlies the fresh granite zone. The difficulty in ground freezing increased as operations moved closer towards the weathered granite zone, as water flow increased along water paths and joint systems, making ground conditions very unstable.

The freezing process was undertaken by Foraky Ltd, a British/Belgium company who can be considered as the leading practitioners in artificial ground freezing in the world today.

The contractors to the soft ground tunnel project were the consortium of LTA (Construction) and Shaft Sinkers, and the consultants engineers for the project were Van Niekerk, Kleyn and Edwards in association with

Electrowatt Engineering Services, Zurich, Switzerland and the client being the National Transport Commission of South Africa.

The general construction procedure involved in the soft ground tunnel contract was the drilling and installation of freeze pipes, the actual freezing process, the excavation process and the application of a shotcrete permanent lining to the tunnel profile.

The main soft ground tunnel had a proposed duration of 18 months and was originally planned to start on the 1st March 1981, but was delayed as the tunnel design had to be modified to accommodate the installation of a main water pipeline within the tunnel. The contract eventually started on the 1st June, 1981 and was completed on the 2nd June, 1983.

The main soft ground tunnel contract is the most costly (Rands/metre) tunnelling contract ever undertaken in South Africa and probably in Africa.

4.2 THE TENDERING PROCESS (SOFT GROUND TUNNEL CONTRACT)

After the construction of the unlined pilot tunnel had been completed, the next stage of construction was the construction of the main Du Toitskloof Tunnel. Due to complexities experienced in the construction of the pilot tunnel, it was decided by the National Transport Commission and suggested by the consultants, that the main tunnel project (contract) should be divided into two parts, i.e. two contracts.

The first contract being the excavation and construction of the soft ground section of the main tunnel by means of artificial ground freezing process. The second contract being the excavation and construction of the hard rock tunnel by means of blasting operations and the placing of an in situ concrete lining.

Due to the complexities involved in the soft ground contract which required specialised methods of tunnelling, it was decided to adopt a two phase tendering system. This was done so that the client (NTC) would be assured that the chosen contractor possessed adequate knowledge and skills that were required in order to carry out such a complex project.

The contract for the soft ground tunnel was open to tender in February 1980.

The first phase of the two phase tendering system invited all interested contractors (consortia) to submit an unpriced prequalification application of the works. This prequalification application by the consortia was assessed by the consultants and the client, they evaluated the methods of construction, the proposed equipment to be utilised and the general abilities, experience and knowledge held by the contractors of previous contracts. The proposed working programme in carrying out the works was also analysed.

After the prequalification applications had been submitted, the consultants and the client had to analyse many proposed freezing methods for the soft ground tunnel. At the end of the day, two alternative methods were selected and other methods were discarded due to reasons of impracticability and methods being uneconomical. The two alternative methods that were accepted were, the first a full 360° frozen arch support to ensure invert stability of the tunnel profile, compared to the basic horse-shoe shape of freezing. The second method was a vertical drilling approach which would result in the whole tunnel cross-section being frozen.

After judgement of the prequalification applications, the consultants and the client, nominated five consortia to tender for the second phase of the two phase tendering

system.

The second phase of the two phase tendering system involved the submission of priced bills by the nominated contractors. The priced bills had to be based on the contractor's prequalification applications, i.e. no major changes were allowed to their initial prequalification applications.

The tender documents for the soft ground tunnel contract allowed for the construction of the 168 metre section to be constructed in five-32 metre stages, work being commenced from the western portal retaining wall. Furthermore, the 32 metre long stages of the tunnel required the formation of a peripheral in situ frozen arch for the full 32 metre length. Furthermore the arch was to be at least 2 metres thick, and existing outside the 113 m² tunnel excavation profile.

The creation of the required frozen arch could be achieved by the drilling of horizontal holes which would take the freezing tubes (freezing lances), and these pipes would circulate a chilled brine solution which would freeze the surrounding ground.

A vertical in situ bulkhead of frozen ground of 3 metres thick was to be achieved at the end of each 32 metre stage. This would provide stability within the tunnel face and would allow for the drilling operation of the horizontal holes for the next 32 metre stage, under more stable conditions. The vertical in situ bulkhead of frozen ground would be achieved by the insertion of vertical freeze tubes drilled in from the surface.

Within each 32 metre stage the last 8 metres of the 32 metres had to be enlarged to allow for the drilling operations of the next 32 metre stage. The enlargement zone was a critical area, as the span of the tunnel increased rapidly requiring additional in situ support.

The primary support to the 32 metre section of the tunnel, had to be provided after 2 metres or less had been excavated i.e. support to the tunnel profile had to be applied after not more than 2 metres of excavation had taken place. The primary support consisted of reinforced shotcrete, steel arches and the thickness of the completed primary support arch is 450 mm.

Also required within the contract was the implementation of an extensive drainage system to the weathered granite contact face which was achieved by the drilling of a cross-drift excavated from within the pilot tunnel.

After the five consortia's priced bills had been evaluated, the winning tender was achieved by the consortium of LTA Construction, Ltd and Shaft Sinkers (Pty) Ltd with the ground freezing specialists (sub-contractors) - Foraky Ltd (British/Belgium company), in March 1981.

Finally, the contract was awarded on the basis that the construction of stage 1, was to be achieved by the use of a horseshoe-shaped frozen arch, and the further 32 metre stages were to be constructed with the implementation of a full 360° frozen arch as temporary support.

4.3 PROPERTIES OF THE FROZEN SOIL

The properties of the frozen soil must be established by elaborate laboratory tests to determine such aspects as the strength, thermal conductivity, creep behaviour of the soil, moisture content etc. of the frozen soil.

The data gained from the above tests will provide for the establishment of an effective design solution for the final process of freezing the ground.

From tests, carried out in other artificial ground freezing projects of the past, it can be stated that the strength of the frozen ground increases with the increase in particle size, with the increase in the water content of the soil, with the decrease of the temperature of the soil and with the increase in the rate of freezing.

From tests carried out on samples of sandy soil and clayey soil, it was shown that sandy soil with a moisture content of 16% at -15°C has a strength of 15 MPa, while clayey soil of the same temperature and moisture content has a strength of 8 MPa.

Furthermore, to obtain the correct solution for the design of the ice wall, one must consider the economics of the temperature vs strength vs creep behaviour vs wall thickness, and which must be related to the method(s) of excavation and the cycles (time) of the excavation process.

THE TYPICAL MECHANICS OF ARTIFICIAL GROUND FREEZING

The application of the ground freezing process is carried out in certain steps which will be discussed below, but the major application of the process at the Du Toitskloof tunnel project will be highlighted in later sections of the chapter.

The ground is frozen by the circulation of a refrigerant (a cooled fluid) at very low temperatures, through a number of freeze tubes, which are drilled into the required freezing location and are placed closely together i.e. pipes placed at minimal cc's. The extremely low temperature of the refrigerant (freezing fluid) causes the ground in the immediate vicinity of the pipes to begin to freeze. After a certain duration, depending on the circumstances, e.g. the type of soil, water flows etc., the amount of frozen ground area increases in a radial movement, i.e. from the pipe outwards. At a certain stage, the frozen area around the pipe will link up with the frozen areas around the

other pipes, this is in effect the start of the formation of the ice wall within the soil. After time the frozen areas around all the pipes, will overlap with each other, i.e. neighbouring pipes frozen areas overlap totally forming the completion stage of the formation of the ice wall (see diagram 13, typical formation of an ice wall using freezing lances).

The size of the ice wall is determined by the number of pipes circulating the cooled fluid and the spacings between the pipes.

Where for example a thick ice wall is required such as the vertical bulkhead ice wall at the Du Toitskloof soft ground project, the pipes are placed in parallel and staggered. Where a narrow ice wall is required, such as the ice arch at the Du Toitskloof project, then the freeze pipes are aligned in single file (see diagram 14, positioning of freeze tubes within a vertical bulkhead).

The temperature of the refrigerant circulation through the freeze tubes must be determined and must be co-ordinated with the required thickness of the ice wall and the time available for the freezing process. This specified temperature of the refrigerant will allow for the uniform formation of the ice wall over the entire freezing area.

Furthermore the area of freezing is usually limited only to the areas outside the proposed excavation area, as the freezing of proposed excavation material is wasteful, uneconomical and serves no purpose.

There are generally two ways-methods of inserting the freezing tubes, so as to provide for adequate clearance in the process of excavating the tunnel area. The first method relates to tunnels lying close to the ground surface. Here the freeze tubes are installed from the surface, to form a "tent-like" frozen area over the perimeter of the tunnel. This method has not been used

at the Du Toitskloof project. The main advantage of this method is that the installation of the freeze tubes is done very quickly and with ease, compared to horizontal or underground drilling processes. But this is all dependant on the type of soil encountered, e.g. drilling operation made difficult if ground contains boulders in line of the drilling operations. It is also dependant on the restrictions that may exist on the surface e.g. sloping ground, buildings etc., this may prevent drilling operations (see diagram 15, tent-like frozen areas for tunnel construction close to the ground surface).

The second method is adopted where the proposed tunnel is located beneath the ground surface or where it is impossible to implement method one, i.e. surface installation of freezing tubes. The method involves the insertion of freeze tubes drilled horizontally into the ground around the proposed tunnel perimeter. This method is more difficult to implement as the drilling of horizontal holes requires great accuracy, so as to remain within the required proposed freezing area (see diagram 15). At the Du Toitskloof soft ground project, two methods of installing freeze tubes were utilised, firstly horizontal drill holes were undertaken, within the proposed frozen arch area, then length of the holes were approximately 32 metres. The second method was the drilling of vertical holes to take the freeze pipes for the vertical bulkhead. The drilling operation of the horizontal holes was carried out within the tunnel itself, while the drilling of the holes for the vertical bulkhead was carried out from the surface.

4.5 REQUIREMENTS FOR FREEZING PROCESS TO SUCCEED

There are certain requirements that need to be fulfilled before an artificial ground freezing process can be chosen and implemented.

Firstly, the particles contained within the soil have to be within a certain size range, namely 5 μ to 15 mm, which relates to a physical range from silts to medium gravels, this particle range will enhance good ground freezing possibilities. Where larger than 15 mm particle sizes are encountered, then a more suitable method of ground stabilising would be the use of grouting techniques.

A more important requirement is that the soil must contain sufficient water-free water, so that the application of the ground freezing process will allow for the formation of ice within the ground. This is required as bonds need to be formed around and between the particles, so that the frozen area of ground becomes a "uniform whole", that enables more stable ground conditions.

But it must be remembered that excessive flowing water within the soil will cause problems in the freezing process. As the water being cooled around the freezing tubes is replaced constantly, by uncooled water, this therefore defies the process of freezing, as water movement must be stabilised to a certain extent within the proposed freezing area. It has generally been accepted that it is better to have more water than too little water, as excess water can be reduced by certain techniques e.g. dewatering, and the problem can also be overcome by using a high capacity freezing plant.

Where the soil possesses a water content below the desired value for the application of freezing, then this problem can be overcome by the application of injecting water and chemicals into surround soil. The technique of injecting water and chemicals into the ground would only be applied if the water content of the soil is just below the required value, otherwise the application of the freezing process would be uneconomical.

It is generally accepted that the heat extraction process introduced by the freezing process on the soil, has to achieve a desirable rate of heat extraction for the freezing process to be successfully applied.

Before the freezing process is selected, it is vitally important that samples of the soil are tested in laboratories, such as the freezing of an undisturbed core sample of soil in an ordinary domestic refrigerator and to analyse the freezing effect on the soil sample.

Other typical tests that should be carried out on the samples are the strength of the frozen soil at various temperatures, frost heave, time taken to freeze soil to certain temperatures, etc.

The above tests will determine whether or not the application of the freezing process will be successful and economical, for the project concerned. The tests will also enable the artificial ground freezing specialists to select the appropriate refrigeration plant set-up for the project.

4.6 INTRODUCTION TO REFRIGERANTS AND REFRIGERATION SYSTEMS

In today's world of artificial ground freezing there are generally two main methods of ground freezing.

The first method involves the storage of the substance in a liquid state under pressure, at normal daily temperatures, the liquid is then allowed to "boil" within the freezing tubes, this is done by releasing the confining pressure of the storage vessel (container).

In order to convert the state of the substance from a liquid to a gas, energy is required to split the molecules of substance, i.e. liquid to gas phase. When the temperature of the "boiling" process at atmospheric

pressure is lower than the temperature of the surrounding material, then heat (energy) will be supplied by the surrounding material. The surrounding material being the freeze tubes and the surrounding ground, will then cool down due to the heat (energy) conversion process, from the surrounding material to "substance".

The ideal substance for this method is the use of liquid nitrogen as liquid nitrogen "boils" at atmospheric pressure and reaches a temperature of approximately -200°C .

The capital cost of establishing such a liquid nitrogen freezing plant is low, but this is generally cancelled out by the relatively high running costs, and therefore the system becomes generally unattractive.

The use of this method is generally utilised in small projects, where amount of ground freezing required is minimal and the duration of such projects is short. The major disadvantage of using liquid nitrogen is that the substance is very dangerous and can produce unsafe working conditions. While its major advantage is that it is capable of achieving very low temperatures which aids projects which encounter strong water flows within the soil i.e. it freezes the water present in the soil at a relatively fast rate.

The second method entails the utilisation of a refrigeration plant system which allows for the continuous recycling of the refrigerants, this method is used generally in larger ground freezing contracts.

The system usually consists of a compression refrigeration plant, which operates between -10°C and -60°C , refrigerants are used in this system, as the freezing agent. In contracts, the refrigeration plant makes use of only a primary refrigerant which is pumped from the plant through

the freeze tubes and then reprocessed (cooled) at the refrigeration ^{plant} again.

A more common procedure is the use of a secondary refrigerant in conjunction with the primary refrigerant. This is done so as to reduce costs, as the cost of the primary refrigerant is very high. It also provides for the identification of leaks in the system, as the secondary refrigerant is usually easily detectable.

Generally, the setting up costs of a compression refrigeration plant is expensive, but this is largely off-set by the very inexpensive running costs, compared to the first method i.e. the nitrogen freezing plant.

The primary refrigerant in the second method of ground freezing, should possess the following properties. It should be readily available and not too expensive, it must not be toxic, inflammable and corrosive. It must not be explosive when mixed with air, the pressure of the refrigerant during the refrigeration process should be approximately equal to atmospheric pressure so as to minimise leakage in the circulation network. Finally, the primary refrigerant should be easily identifiable and detectable, e.g. should have a distinct smell or colour.

A type of primary refrigerant still used widely today is ammonia, even though it might not satisfy all the above requirements. A more recent type of refrigerant is the use of halocarbon refrigerants, which is derived from methane, ethane and includes freon.

Secondary refrigerants act as a heat exchanger between the soil and the freezing plant. An example of a good secondary refrigerant is calcium chloride brine solution, which is cheap, provides for very safe working conditions and can be used for temperatures to about -30°C .

4.7 ARTIFICIAL GROUND FREEZING COMPANY

The soft ground tunnel contract involves the complex operation of artificial ground freezing, due to the lack of ground freezing specialists in South Africa, expertise was sought abroad. The artificial ground freezing company Foraky Ltd, were the sub-contractors to the main contractors, and were responsible for carrying out the freezing process. Foraky is a British-Belgium based company, and has vast amounts of knowledge in the art of artificial ground freezing, they have been involved in many well known ground freezing, in Europe and America. In fact, Foraky Ltd is known as the leading company in today's world of artificial ground freezing processes.

Foraky had to ensure and convince the consultants of the proposed project that their proposed methods of freezing would achieve the minimum requirement i.e. the 2 metre thick ice arch and the 3 metre thick vertical bulkhead ice wall.

Many calculations were carried out in the Foraky laboratories overseas, which clarified the consultants requirements for both the ice arch and wall structural thicknesses. A thermal analysis was also carried out which indicated that the heat extraction required to form the 2 metre frozen arch was $45\ 920\ \text{K cal/m}^3$.

The required spacing of the freeze lances (tubes) was also determined, this was very important, as the spacing of the freeze tubes had to promote the adequate formation of the ice arch and ice wall. The spacing of the freezing tubes within the ice arch had to be determined over the full 32 metre length of each stage. The spacing of the pipes were determined at the entry point of the freeze tubes and at the end of the 32 metre section, so as to fulfil the structural requirements of the frozen arch. The difference in spacing at the

beginning and end of the 32 metre section is due to the fact that the holes to take the freeze lances had to be drilled at an angle so as to accommodate for the last 8 metres of the 32 metre section, which is a drilling enlargement area. That is, larger spacings were required at the end or in the enlargement area of the 32 metre section of the proposed frozen arch. After the above was considered, the spacing suggested by Foraky were 0,92 metres centres at the start of the 32 metre stage and 1,2 metres at the end of the 32 metre stage.

After the proposals such as, number of freeze holes, position of holes, pattern of holes, diameter of holes etc., had been submitted by the freezing specialists Foraky, the consultants had to check the proposals, so as to see if they fulfilled their requirements. After this stage the freezing specialists were given the go-ahead to make the necessary arrangements for sending over the equipment.

4.7.1 The Freezing Plant

The freezing plant would provide the required refrigeration process required to freeze the soft ground sections. The establishment of the freezing plant was the responsibility of Foraky, who had to provide all the required machinery etc. so as to make the plant functional.

The size of the freezing plant that would be required to satisfy the requirements in forming the ice arch and wall timeously, had to be chosen, from the following range which was offered by Foraky. The range being 150 000, 300 000 and 900 000 K cal/hour capacity, packaged freezing units.

After evaluation of the range offered by Foraky Ltd, the consultants (VKE consultants) decided to utilise

two 300 000 K cal/hour units, which would operate in parallel, so as to enable the adequate formation of the frozen areas within a reasonable time interval. With the above plant capabilities and an average spacing of 1,10 metres between the freeze tubes, it was calculated by the consultants that the frost wall between the freeze tubes would close in approximately 13 days. While the formation of the 2 metre thick ice arch was estimated to form within a time period of 46 days, for the full 32 metre section.

The entire freezing plant set-up and its components i.e. pipes, freezing lances etc. was operated and controlled by Foraky Ltd over the entire duration of the soft ground contract.

4.8 ESTABLISHMENT OF THE FREEZING PLANT AND ITS COMPONENTS

The consortium of LTA-Shaft Sinkers was awarded the contract on the 13th March, 1981.

Foraky Ltd planned to start work as early as possible on site and by mid-May 1981, all the required equipment for the freezing process had arrived by ship from Britain and had been transported to the Du Toitskloof site.

The foundations to the freezing plant building were started on the 26th May, 1981, after which the establishment of the freezing plant began (see diagram 16, freeze plant layout).

The freezing plant consisted of two packaged freezing units, each comprising a two-stage twelve-cylinder reciprocating compressor driven by a 180 K Watt electric motor.

The primary refrigerant that was used was anhydrous

ammonia, which has an advantage due to its identifiable smell, which could be used to locate leaks and was also chosen for economical reasons.

The secondary refrigerant that was used was a calcium chloride brine solution and had to be checked regularly to prevent the solution from becoming corrosive due to its properties as the corrosive action would affect the freezing components workability.

Due to the high technical aspects of the functioning parts of the freezing plant itself, this report will not discuss the components contained within the two packaged freezing units.

4.8.1 Freeze Tubes (Lances)

Once the soil pressures, the required thickness of the ice wall, the properties of the soil, the thermal energy required to produce the ice wall in a given time at a certain temperature, etc. had all been determined, then it was possible to determine such aspects as the diameter of freeze holes, the diameter of the tubes, the spacing of the freeze tubes etc. The final solution being the most economical and acceptable solution considering the circumstances.

The freeze tubes used in the contract to freeze both the vertical-bulkhead and the structural arch, were manufactured locally.

The most common sizes of freeze tubes (lances) used in today's artificial ground freezing processes, are between 50 mm to 120 mm in diameter and the size of the drilled holes are between 75 mm and 150 mm in diameter. Spacings

between the freezing tubes can vary between 0,5 to 2,5 metres, depending on the area of freeze, size of freeze plant etc.

The freeze tubes themselves, when used in a circulation of refrigerant system, i.e. the recycling of the refrigerants from the freeze plant through the freeze tubes and back to the freeze plant are usually constructed in the form of concentric tubes.

The outer casing of the freeze tubes (lances) must be watertight, this is tested by means of pressure tests. If the tubes are not airtight, then this will cause the secondary refrigerant to leak into the surrounding soil. When these leakages occur, then the efficiency of the freezing process decreases and may ultimately prevent the ground from freezing adequately.

The freeze tubes consist usually of a seamless outer tube made of steel and a steel inner tube. The freeze tubes used in the soft ground contract consisted of 1,6 mm thick seamless - steel tubes, where the outer casing of the tube has an internal diameter of 90 mm while the inner steel tube has an internal diameter of 50 mm. The outer and inner tubes were hydraulically tested to twice their required working pressure, which was required in the operation of the freezing process. Final pressure tests were undertaken on the tubes after they had been installed into pre-drilled holes within the soil and before they were cement grouted into the holes.

The secondary refrigerant used in the freezing

process was calcium chloride brine, which was circulated from the freeze plant through the tubes and back to the freeze plant. The direction of the secondary refrigerant within the freeze tubes themselves can either be pumped through the inner tube or through the outer tube, but usually it is pumped in the outer tube and then returned to the freeze plant via the inner tube.

The hosing used to transport the secondary refrigerant from the freezing plant to the tubes and back to the freezing plant consisted of high-pressure rubber flexible hosing. The hoses were connected to the steel freezing tubes (lances), by means of a specially designed steel coupling i.e. the coupling connected both the outer tube (casing) and the inner tube (lance) to the delivery and return flexible rubber hosing (see photograph 7, freeze tube connections).

The delivery and return of the brine solution was pressure tested constantly at each freeze tube, and was monitored at the freezing plant (data-logger), this was done so as to be able to check for any malfunctions, such as pressure leaks within the tubes or around its couplings.

The average number of freezing tubes used in a structured frozen arch was 55, while in a vertical bulkhead, approximately 23 tubes were used.

The drilling of the holes, to take the freeze tubes has to be carried out with the utmost accuracy so that the maximum spacing of the freeze tubes is not exceeded, as inaccurate placement of the holes would cause possible

structural failure to the ice wall or arch. Alternatively, inaccurate placement may cause the formation of the ice wall or arch to exceed the allowed time limit and thereby making the freezing operation a very costly one.

Past experience has proved that the contractors profit in artificial ground freezing projects can be greatly reduced if the freezing holes-tubes have to be redrilled due to inaccurate placement of the holes. The specialised drilling of the freeze holes will be further discussed in chapters to follow.

4.9 THE IMPLEMENTATION OF THE ARTIFICIAL GROUND FREEZING PROCESS

The following procedure was followed for each of the five 32 metre stages of artificial ground freezing, in order to allow for the excavation of the main Du Toitskloof tunnel soft ground section.

4.9.1 Vertical bulkhead

The first vertical bulkhead was located at approximately 33,5 metres from the western portal, the positions of the various freeze holes that were drilled to take the freeze tubes are shown (see diagram 8, general positions of freeze holes within a typical vertical bulkhead layout) and (see Drawing 4, vertical bulkhead freeze tube location).

The positions of the freeze holes within the vertical bulkhead were designed so as to form a 3 m thick frozen wall and also had to be aligned, so as to accommodate for the drilling

operation of the horizontal holes-tubes with the arch, for the next 32 metre section. That is, the holes within vertical bulkhead had to be placed in an irregular pattern, in between the vertical bulkhead holes-freeze tubes.

The holes located on the uphill side were spaced close together so as to prevent water movement penetrating the ice face i.e. the holes-tubes are placed close together to form a barrier against water flows from the uphill face of the vertical bulkhead.

Furthermore, the vertical bulkhead was monitored during the freezing process by the insertion of thermocouples into predrilled observation holes. There were normally 3 observation holes per vertical bulkhead, two of these holes were located at each end of the vertical ice wall, while one was located usually near the middle and uphill side of the vertical bulkhead. The freeze holes and the observation had the same diameter and were drilled to the same depth (see diagram 8 and drawing 4).

After the freeze holes had been drilled, the holes were then filled with bentonite mud which provided an insulation - contact zone between the freeze tubes and the surrounding soil. The outer freeze tube was then placed into drilled hole, which displaced the unnecessary bentonite mud, thereafter the tubes were grouted in with cement.

The above process was followed for the formation of the other vertical bulkhead operations.

4.9.2 Formation of Frozen Arch (Horizontal Drilling)

The drilling of the horizontal holes had to be drilled for the full 32 metre length of each stage, of the five 32 metre stages of the soft ground tunnel.

The horizontal drilling operation was carried out with the use of a drilling gantry which had two mounted drilling booms coupled to it, the gantry itself was mounted on railway tracks for easy mobility.

The setting positions of the two drilling booms were premarked on the drilling gantry, so as to accommodate for the precise location of the holes to be drilled, this would also reduce the time in setting and aligning the drilling rigs. The permitted deviation for the drilling of the horizontal holes was 1%, which calls for precision drilling, considering that the holes had to be drilled over 32 metres.

Problems in maintaining this 1% deviation occurred as the drill bits were often deflected by possible in situ boulders, misaligned concrete ground anchors of the western portal retaining wall, etc., this caused drilled holes to become misdrilled and inaccurate. The remedy for this problem of inaccurately drilled holes, was the drilling of extra holes, so that there was a proper-adequate 'fanning' of freeze tubes so as to promote the formation of the 2 metre thick by 32 metres long ice arch.

During the first 32 metre stage, an extra seven freeze holes had to be drilled to overcome the above problem. The total length of freeze tubes

used in the first 32 metre stage, was approximately 1584 metres, distributed over 50 freeze holes. The drilling of the horizontal holes also included the provision for 6 observation holes which were drilled parallel to the freeze holes. As in the vertical bulkhead observation holes, thermocouples were also placed in the horizontal ice arch observation holes, so as to monitor the temperature within the ground.

The position of the observation holes was determined by analysing the results obtained from the difficulties encountered in the drilling of the horizontal freeze holes, where there were large spacings between the freeze tubes or where there was an expected water movement to occur, observation holes were drilled in these areas (see drawing 3, typical freeze tube circuit connection).

Due to the required accuracy in drilling, the freeze holes and the nature of the ground, etc., problems were encountered in the drilling of the uncased horizontal holes. The holes often collapsed before the freeze tubes could be installed. Therefore attempts were made with the use of an encased hole system, especially where the ground was very water-logged and unstable, which proved to be successful in some circumstances.

After the freeze tubes had been properly installed, then the freeze tubes were connected to the freezing plant process, with the use of couplings connected to rubber hoses. Each tube being connected to two hoses, one on inlet and one on outlet, i.e. recycling of the secondary refrigerant, from the freezing plant to the freeze tubes and vice versa.

4.9.3 Pre-freezing operations of the vertical bulkhead and the tunnel arch

The freezing process started operating on the 3rd September 1981, for the formation of the first 3 metre thick vertical bulkhead ice wall, within the first 32 metre stage of freezing. On the 21st September, the freezing process for the structural ice arch was started, covering 32 metres.

The brine solution was circulated through the pipes (hoses), arch tubes, at a temperature of -27°C and less, after approximately two weeks of continuous freezing, sufficient support had been accomplished by the freezing process, thereby allowing the first stage of excavation to begin.

The first stage of excavation was the excavation and demolition of part of the western portal retaining wall i.e. to allow for the formation of the tunnel profile within the western portal retaining wall.

After three weeks of continuous freezing, the ice formation of the ice arch was sufficient enough for the excavation process to begin within the tunnel profile itself. Support was installed in the form of shotcrete-mesh-steel arches, so as to ensure safe working conditions. The support was installed in 2 metre intervals, i.e. 2 metres of ground was excavated, along and within the tunnel profile, after which the 2 metres of tunnel profile was adequately supported with the required shotcrete etc.

The rate of freezing of the vertical bulkhead was undertaken at a slower rate than that of the horizontal-arch holes, as it was not necessary

to achieve the formation of the three metre thick ice wall, so early in the excavation of the tunnel.

It was noted during the freezing of stage 1, that water flows influenced the rate of freezing and the temperature around the freezing tubes, causing a longer and more intense freezing process to commence. This was caused for example by an increase in rainfall in the area, which caused greater waterflows within the hillside; these waterflows caused an increase in the temperature around the freezing tubes as monitored by the thermocouples, and therefore caused a loss in energy.

On the other hand, where well-pointing was carried out from the pilot tunnel, the temperature decreased significantly around the freezing area, i.e. promoted a saving in energy.

The freezing process was implemented in the same manner to the other 32 metre stages, with the exception of sections 4 and 5, which were frozen and excavated at the same time. This was done, as a drainage area had been established within the fifth stage against the weathered-fresh granite zone, for the purpose of draining the large amounts of water encountered in this area. The drainage area in stage 5 was enlarged so as to accommodate for the drilling gantry, which drilled holes from within the drainage areas towards the horizontal holes of the fourth stage, thereby forming a total frozen arch of approximately 50 metres in length. Sections 4 and 5 were excavated together so as to promote a saving in time, as the project had fallen behind the proposed programme somewhat, due to delays in the freezing process.

After each 32 metre stage had been totally excavated and supported, the thawing process began.

4.9.4 Thawing Procedures

The thawing process of the ice arch was commenced after the total 32 metre of each stage had been excavated and properly supported. A natural thawing process was implemented where the freezing plant was switched off, and the circulation of brine solution was cut-off from the horizontal freeze tubes of the frozen 32 metre arch.

The thawing process was monitored by the observation holes, by both horizontal and radial probes, housing thermocouples.

After the ground contact zone between the shotcrete support-lining and ground had reached 0°C or greater, a grouting process was commenced, where cement grout was injected between the ground and the shotcrete lining, so as to fill all voids that may have formed. Voids were formed due to the expansion of the soil during freezing - ice expands and the contraction of the soil during thawing. The voids were filled with cement grout so as to provide an equal distribution of loads on the support lining, i.e. to promote a uniform load distribution on the arch lining.

The thawing process of the first vertical bulkhead was only commenced after a sufficient depth had been excavated within the second stage. This was done as the frozen vertical bulkhead provided support for the horizontal drilling operations of the second 32 metre stage i.e. provided work face support at the end of the first stage of the tunnel.

The freeze tubes in both the vertical bulkhead and structural arch were not removed ~~for~~ reuse in the next stages of freezing i.e. in every stage new freeze tubes were utilised.

The success of the freezing process depended heavily on the proper and accurate monitoring of the freeze process during the freeze. The next section - Instrumentation and Monitoring of the freezing process, will deal with these aspects.

4.10 INSTRUMENTATION AND MONITORING OF THE FREEZING PROCESS

4.10.1 Temperature recording

The structural strength of the soft ground tunnel profile could have been endangered if there was an inadequate formation of ice, or also known as the formation of an 'ice window'; this could have brought about a collapse in the structural ice arch.

Therefore it was necessary to establish a comprehensive monitoring programme in line with the freezing plant operations, so as to check the progress of the formation of the ice arch and vertical bulkhead ice wall.

Aspects that could affect the formation of the ice arch, etc. were water flows along the periphery of the tunnel profile, which would continually increase the temperature of the water which was being 'cooled' by the freezing process, i.e. water being cooled so as to promote the formation of ice, was being replaced by 'warm' water, which causes problems in the freezing process. The devices

that monitored these areas were carried out by the use of thermocouples which were placed within the observation holes. The observation holes contained thermocouples at 2 metre centres along the entire 32-metre stage of freezing.

Monitoring the ice wall temperature and the thickness of the ice wall against time was achieved by correlating temperatures obtained from the thermocouples.

The monitoring of the thermocouples (sensors) was aided by a data-logger which scanned the sensors automatically every 10 minutes; if one of the sensors was below the minimal allowed temperature, the monitoring system would alert the plant supervisor, who would check and make the necessary corrections, so as to maintain a steady flow of freezing.

The data-logger was connected to a computer system which recorded all data, such as soil temperatures, water temperatures, refrigerant temperatures etc., and which could be recalled and analysed at any particular time. Furthermore, the computer and the data-logger provided a daily data sheet on the temperatures of each observation hole related to time, and also plotted graphical data of each observation hole (see diagram 17, typical print-out by the freezing plant computer), (see graph 1, temperature graph for diverging freeze pipes - hole 11) and (see graph 2).

The longitudinal observation holes could only give a generalised 'picture' of the temperatures within the ice arch and vertical bulkhead, therefore it was necessary, after sections of the tunnel had been excavated, to insert (drilled

in) into the frozen tunnel profile, extra thermocouples, so as to verify the ice formation and the thickness of the arch. These radial probes were also utilised to monitor the thawing process, on completion of each 32-metre section of the soft ground tunnel (see diagram 18, the placement of radial probes containing thermocouples), (see drawing 5 and 6, temperature monitoring probes radial and temperature monitoring probes for vertical bulkhead).

Generally, the required temperatures that had to be achieved within the structured-ice arch and the vertical bulkhead, as determined during the design stage of the project, had to be certified by the actual temperature readings given by the above utilisation of thermocouples within the observation holes etc. These actual readings compared to the planned temperature readings, would establish if the artificial ground freezing process was operating according to plan and within an acceptable time performance.

Furthermore, the temperatures obtained from the observation holes enabled engineers to determine the moisture content of the soil, ground waterflows and air pockets that may have existed in the zones where the freeze pipes were situated. The different temperatures reflect good and bad freezing areas, along the ice arch and vertical bulkhead, which could be the direct result of the above aspects.

When all the temperatures registered by the thermocouples are equal to or below the required minimum temperature, the formation of the frozen soil is complete and structurally sound. When this stage is reached the freezing plants supply of energy (heat extraction) is lowered so as to maintain the desired ice wall thickness, i.e. lowered so as not to promote further ice growth.

The desired thickness of the ice arch and vertical bulkhead wall was determined by measuring the distance between any two points, where the two points must have a temperature of not less than or equal to -3°C . The average temperature within the ice arch and frozen bulkhead must not exceed -10°C .

Finally, the consultant engineers (VKE) carried out further tests on the formation of the frozen ground with the use of hand-held temperature reading equipment. The equipment consists of a hand-held probe which is inserted into the face of the ice wall during excavation and which gave instantaneous temperature readings of the ice wall.

This instrument was mainly used to check the results analysed from the observation hole readings containing the thermocouples.

In conclusion thermocouples with the observation holes were analysed daily, so as to determine the ice growth (see graph 3, typical temperature readings obtained within the first 32 metre stage of freezing - freezing isotherm). As from the graph, point A, shows that 0°C isotherm was reached after 62 days of actual freezing compared to a predicted 50 days of freezing.

The radial probes drilled into the ice arch gave an indication of the ice wall arch thickness in respect to time (see diagram 18, the placement of radial probes containing thermocouples). The diagram, for example, at probe number 5 at 16,5 metres from the western portal gave a reading of 3,5 metre ice thickness of the wall after 70 days of freezing. These methods of temperature recorded were applied to all the 32 metre stages of freezing, so as to reassure proper ice growth.

4.10.2 Freeze holes testing

The drilling of the freeze holes were tested, to determine their position and accuracy in respect to the proposed areas of freezing. The tests were undertaken with the use of a portable borehole deflectometer, which is an instrument that is placed within the freeze tube and is moved down the tube taking readings every metre, after which the readings are plotted and compared to the theoretical values (see diagram 19, a typical portable borehole deflectometer) and (see photographs 8 and 9).

The portable borehole deflectometer, as shown in diagram 19, is applied in the following manner. The instrument is placed within the freeze hole, and lined up in a known straight line direction. The instrument is then pushed or lowered in the case of the vertical bulkhead freeze holes, into the freeze tubes. Readings are recorded at every metre along the freeze tube and readings are analysed by an electronic device built into the deflectometer.

Any deviations from the straight line causes the front part of the deflectometer to deflect relative to the rear end, this is registered into a control panel which records the information. The procedure of 1 metre interval readings is continued until the end of the freeze hole is reached (see graph 4, displacement determined by a portable borehole deflectometer).

After all readings have been taken, it is processed to produce co-ordinates x, y and z in 'space', which is plotted to illustrate the position of the freeze tubes within the tunnel profile (see diagram 20, computer print-out of P.B.D. co-ordinates).

This will illustrate whether the freeze tubes are at sufficient spacings and whether adequate coverage of the proposed freezing area has been achieved. Where the spacings between two freeze tubes was greater than allowed, an extra tube was inserted so as to give proper coverage of the freeze tubes over the proposed freezing area (see diagram 21, position of freeze tubes within the tunnel profile, and determined with the aid of a portable borehole deflectometer). The positioning of freeze tubes caused problems during the construction of the soft ground tunnel, especially the horizontal freeze tubes, as they had to be drilled-in over large distances - approximately 32 metres.

4.10.3 Ground heave measurements

Further testing was carried out in the form of ground heave measurements, these were recorded at the surface. The ground heave recordings were carried out merely for interest sake, along the length of the soft ground tunnel, as any "ground heave" would not affect any obstacles on the surface, such as buildings, roads etc. Ground heave measurements due to the formation - expansion of the ice within the frozen ground, were measured by the installation of 'pins' levelled before freezing began along the centre-line of the tunnel, on the surface. After 10 weeks of freezing the pins reflected approximately 45 mm of ground heave.

4.10.4 Convergence of tunnel lining measurements

Measurements were carried out to measure convergence of the tunnel lining and creep of the ice arch, this was carried out at certain stages of the work, so as to detect any movement which may have affected the structural strength of the tunnel. The measurements were carried out at certain time intervals, so as to allow for remedial action, if problems had been encountered. The maximum displacement of the tunnel lining-support that was experienced was approximately 12 mm (see graph 5, convergence of tunnel support-lining at 3,5 metres from the western portal, with respect to duration).

In conclusion, many other tests were carried out by the contractor, sub-contractors and consultant engineers, e.g. (see drawing 7, test for freeze tube coverage to provide adequate ice arch support), to determine how

successfully the tunnel construction was performing, I have mentioned the main monitoring and testing methods above, which were undertaken during the construction of the main Du Toitskloof soft ground tunnel.

CHAPTER 5

THE DRILLING OF THE FREEZE HOLES AND THE EXCAVATION OF THE MAIN SOFT GROUND TUNNEL

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CHAPTER 5: THE DRILLING OF THE FREEZE HOLES AND THE
EXCAVATION OF THE MAIN SOFT GROUND TUNNEL

5.1 THE DRILLING OF THE HORIZONTAL AND VERTICAL
FREEZE HOLES

The ground conditions encountered in the western portal area of the Du Toitskloof tunnel project caused problems in the excavation and construction of the soft ground tunnel. The material encountered in this area was decomposed-weathered granite which was saturated and due to these aspects was very unstable and even more so when disturbed. To overcome these difficulties, it was decided by the project consultants to utilise a ground freezing technique, which would insert stability within the decomposed-weathered granite zones.

The artificial ground freezing technique called for the holes to be drilled within the decomposed granite zone, and which would contain the freeze tubes - lances, in order to provide for the formation of frozen ground.

These freeze holes had to be drilled to great depths in a vertical plane and horizontal plane for the structural ice arch and vertical bulkhead respectively. Due to the unstable conditions of the soil and the great depths required by the freeze holes, special drilling techniques had to be applied, so as to provide for the most reliable and economical drilling of the freeze holes. The accuracy of the drilled freeze holes was monitored by the use of a portable borehole deflector as described in the previous chapter.

Two methods of drilling the freeze holes were applied, firstly, the horizontal holes within the structural 'frozen' arch, were achieved by utilising a drilling gantry onto which two drilling booms were attached. Secondly, the vertical freeze holes within the vertical

bulkhead, were achieved by utilising a truck-mounted drilling rig. Both these methods of drilling had to achieve an accuracy of not greater than 1% deviation from the theoretical position of the freeze tubes, in respect to the depth of the hole (see graph 4, displacement determined by a portable borehole deflectometer). One of the major problems that caused displacement of freeze holes during drilling was the presence of in situ material, such as granite boulders, sandstone boulders etc. This type of in situ material caused the drilling bits to reflect out of the intended line of drilling and therefore causing the displacements of the freeze holes.

The diameter of the horizontal freeze holes were 150 mm and were drilled to an average depth of approximately 32 metres for all the 32 metre stages, with the exception of stages 4 and 5. Here the depths of the drilled holes increased somewhat, so as to accommodate for the overlap between the horizontal freeze tubes of stages 4 and 5 which were excavated simultaneously.

The diameter of the vertical freeze holes were also 150 mm and were drilled to a depth of approximately 30 metres in the first 32-metre stage of freezing. However the depths of the holes increased as each new vertical bulkhead was constructed, i.e. as each new vertical bulkhead was constructed higher up the hillside, the depths of the vertical freeze holes increased.

The vertical bulkhead freeze holes were drilled by means of an Ingersoll Rand TH55 truck-mounted rig which was adapted so as to enable holes to be drilled without a temporary casing being placed to keep the holes 'open', so as to allow for the insertion of the freeze tubes - lances - technique, known as a rotary mud flush technique.

The horizontal freeze holes of the structural ice arch were drilled by means of an air-operated rotary drill rig, with a 150 mm drag bit including rod stabilisers and foam flush, this enabled the drilling of uncased holes.

The horizontal holes within the first stage of the five 32 metre stages were drilled with relative ease, and were drilled without casing, using the 150 mm drag bit, this was possible due to there being a relatively low ground water table. Foam was used as the flushing medium and the freeze tubes were inserted with relative ease before the drilled holes could collapse. The sequence of drilling the holes into the proposed ice arch was started from the crown of the tunnel, and proceeded down either side of the arch.

Problems occurred in the second stage of the five 32 metre stages, when the drilled holes collapsed with relative ease, here casing had to be used, so as to overcome the problem. Investigation into this problem showed that the sequence of drilling the holes 'played' a major role in the collapse of the drilled holes. This was due to the fact that, as the water content within the soil had increased, and the drilling operation disturbed the ground and made the water concentrate below the hole being drilled. Due to this, the hole to be drilled next below the previous drilled hole, caused the sides of the hole being drilled to collapse very easily, and thereby disallowing the freeze tube to be inserted. This was overcome by the alteration of the drilling sequence, the sequence of drilling was commenced, from both 'bases' of the arch instead of starting the drilling operations from the crown of the tunnel. That is, the drilling operations now started from the 'bases' of the arch and moved

upwards, instead of from the crown of the tunnel downwards. This procedure was maintained for the rest of the stages of the five 32 metre stages, except for the introduction of a third drilling rig, mounted on the drilling gantry and the use of 'cop 62 hammers' for penetration through the frozen vertical bulkhead of stages 2, 3 etc.

Finally, most of the problems encountered in the drilling operations of the vertical bulkhead and horizontal holes-structured arch, were resolved in the first and second stages of the five 32 metre stages, after which the efficiency and production of drilling holes increased considerably. But the production of inserting holes depended greatly on the accuracy at which the holes were drilled. Here, the direction of the holes depended greatly on the accuracy at which the operator aligned his drilling rig, as well as his control of the rate of feed and rotation of the advancing drill bit.

After the holes for each stage had been drilled, the freeze tubes inserted and the freezing plant circulation commenced, the freezing operation took an average of 4,5 weeks to freeze the surrounding ground. The next operation after this average duration of freezing, was the excavation of the soft ground tunnel.

5.2 EXCAVATION OF THE SOFT GROUND TUNNEL

After the tunnel arch had been frozen to a degree that would provide structural support-stability to the saturated decomposed granite i.e. approximately 4,5 weeks, the excavation operations could commence.

The first stage of the excavation process of the

project involved the demolition of part of the western portal retaining wall (see photographs 10 and 11), so as to allow excavation to begin within the decomposed granite itself, i.e. within the tunnel profile itself. The demolition of part of the retaining wall commenced on the 25th September 1981 and had a duration of approximately two weeks. Then excavation of the 'frozen' decomposed granite within the first stage of the tunnel programme, was started on the 8th October 1981. Furthermore, while the excavation was taking place in the first section, the drilling crew concentrated their work on the drilling of the second and third vertical bulkheads at the ends of stages 2 and 3 of the five 32 metre stages.

The excavation of the frozen ground within the frozen structural arch was achieved with the use of an electrical driven roadheader, (Anderson Mavor rotary cutting head), which was mounted on the boom of a Poclain HC300 excavator. The roadheader excavated most of the frozen ground, but concentrated its works on the upper regions of the tunnel profile, while the lower sections of the tunnel profile was excavated when possible, with the use of a RH6 backactor (backhoe excavator) (see photographs 10, 11 and 12).

A major problem with the use of the poclain excavator-roadheader cutter, was that the excavator was just too short to reach the crown of the tunnel. This problem was overcome with the use of a portable ramp, onto which the excavator moved so as to be able to excavate the crown of the tunnel. Time delays occurred here, as the ramp had to be moved after each 2 metres of excavation had been completed.

The excavated material within the tunnel was loaded into dump trucks (twin-engined Dux dumpers), with the use of a front-end loader, this material was used as fill material for the surround approach roads to the

Du Toitskloof tunnel.

The finer details of the excavation works were carried out with the use of the drilling gantry from which excavation works were undertaken by hand.

The size of the excavated profile of the tunnel was the same for the first 24 metres of the 32 metre stage, thereafter the tunnel profile was enlarged so as to accommodate for the drilling operation of the second stage of the five 32 metre stages. This enlargement area allowed for the drilling and placing of the horizontal freeze tubes for the next 32 metre stage, the enlargement area was contained within the last 8 metres of each 32 metre stage. All five of the 32 metre stages were excavated in the same manner as above, with the exception that stages 4 and 5 were undertaken simultaneously. The average duration for the completion of each 32-metre stage; the excavation and application of the shotcrete lining; took approximately 9 weeks. The work being done in three 8 hour shifts per day, starting on the Sunday at 11 p.m. and ending on the Saturday at 11 p.m. (i.e. weekly cycle).

5.2.1 The excavation of stages 4 and 5 of the soft ground tunnel

The 168 metre soft ground tunnel was divided up into five stages, each stage being 32 metres in length. Stages 1, 2 and 3 were excavated and constructed in the manner as mentioned above, while stages 4 and 5 were excavated and constructed simultaneously, but still adopting the manner in which 1, 2 and 3 stages were excavated and constructed.

The contractor was required to 'drive' a cross-drift from the pilot tunnel to a position on the centre-line of the future main soft ground tunnel. At the end of the cross-drift, on the centre-line of the main tunnel, a drainage gallery was established. The drainage gallery was situated within the hard granite zone just ahead of the soft ground section of stage five, of the five 32-metre stages.

Due to the formation of this drainage gallery in stage 5, the contractor, who was at the time 'running' behind schedule in the contract, decided to investigate the possibility of freezing, excavating and constructing the main tunnel from stage 5 toward the western portal. After the investigation and consultation with the consultant engineers, it was decided that the above procedure would be adopted.

The drainage gallery had to be enlarged to a radius of approximately 8,5 metres, so as to allow for the positioning of the drilling gantry, which would enable holes to be drilled towards the freeze tubes of stage 4. The freeze tubes of stages 4 and 5 overlapped, so as to form a continuous frozen arch from the drainage gallery to the beginning of stage 4 on the western portal side.

The method adopted in stages 4 and 5 brought about the turning point in the soft ground tunnel contract, as it provided a saving in "contract time", which therefore allowed the contract to finish on time, instead of finishing after the completion date.

Furthermore, the 'very deep' vertical bulkhead at the end of stage 4 was eliminated.

Generally there was a saving in cost, due to the methods adopted in stages 4 and 5, but was offset slightly as a larger drainage gallery was required.

Finally, the excavation of the soft ground tunnel was excavated in 2 metre intervals, after each 2 metre interval a reinforced shotcrete lining-support was applied. The next chapter will investigate the application of the reinforced shotcrete lining.

CHAPTER 6

THE APPLICATION OF A SHOTCRETE LINING (SUPPORT) WITHIN THE FROZEN SOFT GROUND TUNNEL

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CHAPTER 6: THE APPLICATION OF A SHOTCRETE LINING (SUPPORT)
WITHIN THE FROZEN SOFT GROUND TUNNEL

6.1 GENERAL

The application of a shotcrete lining against a frozen tunnel wall was undertaken for the first time in South Africa, in the Du Toitskloof soft ground tunnel project. The consortium of LTA/ Shaft Sinkers played a major role; during the pricing phase of the two phase tendering system; in the investigation of applying a shotcrete lining onto a frozen surface. The consortium carried out tests on the application of a shotcrete lining, on a test model (test tunnel). All these tests were carried out in Johannesburg, and the tests were carried out in close consultation with the consultant engineers of the Du Toitskloof project.

6.2 SPECIFICATIONS OF THE TUNNEL LINING (SHOTCRETE)

The specification¹ of the tunnel lining called for the following:

- at day 3, to have a strength of 11,25 MPa, i.e. 45% of the 28 day strength
- at day 7, to have a strength of 17,50 MPa, i.e. 70% of the 28 day strength
- at day 28, to have a strength of 25,00 MPa tested on a 50 mm diameter core sample
- test cubes when crushed in both parallel and perpendicular to the direction of application, required a compressive strength 20% higher than that of the core samples.

- the required minimum cement content must be 350 kg/m^3 for aggregate size greater than 6 mm and 400 kg/m^3 for aggregate size less than 6 mm.
- the water/cement ratio must be within the range of 0,35 - 0,50, with the maximum moisture content of the material to be 4%.

6.3 THE OBJECTIVES OF THE SHOTCRETE APPLICATION TESTS

The objectives of the shotcrete application tests² by the consortium of LTA/Shaft Sinkers in Johannesburg, were to determine the effects of sub-zero temperatures on the setting time of the shotcrete, the rebound and adherence characteristics of the shotcrete, the strengths achievable by the shotcrete on a frozen surface, the rate of energy extraction between the frozen wall and different thickness of shotcrete, and the effects of an insulation layer between the shotcrete and the frozen wall.

6.4 THE DESCRIPTION OF THE TEST TUNNEL FOR SHOTCRETE APPLICATION TESTS

The test tunnel³ was formed by the following components, it consisted of steel arches, mesh, hessian, shuttering, saturated decomposed granite, freeze tubes, observation tubes, thermocouples and a freezing agent of liquid nitrogen at -50°C .

A 3 metre shutter was erected which was coated with polyurethane for ease of stripping, behind the shutter a saturated decomposed granite was hand packed. Into the decomposed granite, freeze tubes were located, at 400 mm centre, and also 20 mm diameter tubes were installed which contained thermocouples for temperature recording purposes. These thermocouple-tubes were placed between 50 - 200 mm from the freeze tubes and covered the full length of the freezing zone. There

were 18 freeze tubes installed and 8 (thermocouple) tubes, the former being activated by means of liquid nitrogen as the freezing agent, so as to maintain a ground temperature of -20°C . Mesh was then placed on the frozen surface with a 25 mm spacing before the shotcrete was applied.

6.5 RESULTS OF SHOTCRETE TESTS

The test results were generally disappointing, as the results did not fulfil the required specification of the concrete lining as mentioned earlier.

Shotcrete of both high and low water/cement ratios were applied, the former producing a very low strength development within the shotcrete over time. This was due to the great 'energy extraction' from the shotcrete by the frozen wall, which caused the water molecules to freeze and thereby preventing the shotcrete ~~from gaining~~ strength.

More rewarding results were achieved with the application of a 'dry mix' i.e. with a low water/cement ratio, this was due to a slower rate in the temperature drop of the shotcrete and thereby allowing more strength to develop.

The shotcrete application generally gave good results with reference to the placement of sound and dense layer of shotcrete. Furthermore, it was found that the shotcrete adhered well to the frozen wall, i.e. had good bonding characteristics.

The use of an insulation layer between the frozen wall and the shotcrete also produced poor results in strength achievements, the insulator being formed of a layer of aerolite glass fibre placed between 2 layers of 0,25 mm hyperlastic plastic sheeting.

Due to the above test results being generally negative,

it became very apparent that the required specifications for the lining (shotcrete) were unattainable.

The potential contractors for the contract shared their disappointing results with the consultant engineers (VKE consultants). The consultant engineers³ were not too bothered by the results, but they were prepared to take test results from control panels made at ambient temperatures as the criteria for acceptance of the shotcrete, due to this, the required specifications became acceptable to the tenderers in phase two, of the two phase tendering system.

6.6 THE MATERIALS AND EQUIPMENT UTILISED IN THE FORMATION OF THE MAIN TUNNEL SUPPORT (REINFORCED SHOTCRETE)

The shotcrete was made up of cement aggregate, sand, water and a cement 'accelerator'.

The cement used was ordinary Portland cement, supplied locally, and stored in two 150 tonne silos. An accelerator was added, so as to increase the reaction time of the mix i.e. setting time of the shotcrete is reduced (a shotcrete accelerator used).

The type of aggregate used was granite, as the cement has a very high Na_2O content, and the use of granite would reduce the risk of alkali-aggregate reaction. The aggregate was kept in covered bays, this was done so as to maintain strict control over the moisture content of the aggregate, especially as the works are situated in a very wet climate area.

The sand used is a very good quality pit sand with a high quartz content and the sand had a very low water demand. The sand was also stored in covered bays, so as to control its moisture content.

The mix design chosen for the production of the shotcrete was as follows:³

- * Ordinary portland cement = 450 kg/m³
- * Aggregate (6,7 mm stone) = 608 kg/m³
- * Sand = 1184 kg/m³
- * Water/cement ratio = 0,45
- * Accelerator = 3% (by weight of cement for first 50 mm layer of shotcrete)
= 2% (for subsequent layers of shotcrete)

A batch plant was used to produce the shotcrete; this was done under very strict quality controls; the shotcrete was mixed in a 0,5 m³ pan mixer, with an output of 20 m³/hour. After being mixed, it was then conveyed from the batch plant to within the tunnel by 4 m³ truck mixers. From the trucks it was fed into a 150 litre pan mixer, where the accelerator was mixed in according to the weight of the cement within the mix. From the pan mixer it was conveyed by a small conveyor belt to the 'gun' (standard rotary drum type using a 9 round hole rotor discharging into a 50 mm hose). The 50 mm hose was connected to a nozzle which applies the shotcrete, water to the mixture was added 2,5 metres back from the nozzle at a pressure of 500 kPa. The above system produced shotcrete at a rate of 3 m³/hour, due to the large amounts of shotcrete required, two of the above systems were used so as to produce a maximum output of 6 m³/hour.

The nozzle applied the shotcrete onto the frozen walls of the tunnel, which was applied in layers, the layers incorporating mesh reinforcement, steel arches, etc.

6.7 THE APPLICATION OF THE PERMANENT SUPPORT (REINFORCED SHOTCRETE) ONTO THE FROZEN MAIN TUNNEL ARCH

The shotcrete was applied to the walls of the tunnel with the above mentioned 'shotcrete plant', due to the height and large area of the tunnel arch, the application of shotcrete was aided with the use of the drilling gantry. The drilling gantry provided the appliers of the shotcrete, a working platform from which work could be undertaken, in the application of the shotcrete layers. The gantry also aided the workforce in the erection of the large steel arches of the permanent support.

The following procedure was followed in the application of the permanent support-lining (shotcrete), to the walls of the frozen tunnel arch:

* The first step began with the fixing of a light weight mesh (2 kg/m^2) to the frozen arch. The mesh was fixed with the aid of 'cruciform' dowels, made of 10 mm high tensile bars of lengths between 300 mm to 600 mm. The dowel had one end sharpened and a 'cross piece' was welded 50 mm from the top of the exposed dowel. The dowel was hammered into position, into the frozen arch, the dowel being rigidly held in position as the ice reformed around the dowel. The mesh was then fixed onto the 'cross piece' with a spacing of 25 mm between the frozen wall and the mesh, spacer blocks were also used to maintain an average spacing of 25 mm. To the first layer of mesh, 'stools' were attached, this provided for the fixing and spacing of the second layer of mesh, within the second layer of shotcrete. After the 'stools' were in place, the first layer of 50 mm of shotcrete was applied. The first layer was allowed to strengthen and harden before the application of the second layer.

- * The second step began with the fixing of the second layer of mesh, the mesh applied was a 'heavy-pre bent' mesh, varying from 10,28 kg/m² in the sidewalls to 6,58 kg/m² in the crown of the tunnel. This heavy pre bent mesh was fixed to the steel stools as applied in step 1, of the shotcrete application. Stools were also connected to the second layer of mesh and protruded into the third layer of shotcrete and positioned at centres of 500 mm - 1000 mm. The heavy mesh was then covered with the second layer of shotcrete, 100 mm thick.

- * The third step was the erection and installation of heavy steel arches (29 kg/m) placed at 1 metre centre, between the stools attached to the heavy mesh, applied in the second step of the application of the shotcrete support. The areas between the heavy steel arches were then filled with 200 mm thick layer of shotcrete, these areas were attached to the second layer of shotcrete by the installed stools.

- * The fourth step and final step was the application of another layer of heavy mesh, which was attached to stools, the stools being welded to the steel arches. Thereafter a final layer of shotcrete, 100 mm thick was applied (see photograph 13 and 14, application of the shotcrete lining).

The final thickness of the reinforced shotcrete was 450 mm thick which formed the initial permanent lining of the tunnel (see diagram 22, typical section through the tunnel lining applied within the soft ground tunnel).

6.8 TESTING OF APPLIED SHOTCRETE: SECOND 32-METRE STAGE

The testing of the in situ shotcrete material was carried out by the drilling of 100 mm diameter test cores within the tunnel lining. While the results

obtained from the 'crushing test' applied to these 'in situ' cores, was compared to 'crushing tests' applied to test cores from specially prepared test panels (e.g. similar to 'test cube').

The average results of the tests on both the in situ and panel cores were very similar in the '28 day strength tests', the average strength being 50 MPa. At 3 days the strength of the in situ shotcrete and the test panel shotcrete was 28 MPa, while at 7 days, the strength of the in situ shotcrete was 46 MPa and the test panel shotcrete was 43 MPa (see diagram 23, typical 100 mm diameter test core).

The strengths achieved in the actual application of shotcrete to the frozen tunnel arch compared to the strengths achieved in the 'test tunnel' before the start of the project, differed greatly. What were the possible reasons for this.

6.9 REASONS FOR HIGHER STRENGTH RESULTS DURING THE ACTUAL APPLICATION OF SHOTCRETE TO THE DU TOITSKLOOF TUNNEL

The reasons for higher strength results in the actual application of shotcrete to the frozen soft ground tunnel compared to the shotcrete applied to the 'test tunnel', can be summarised as follows:³

- * the saturated-disturbed decomposed granite within the test tunnel contained a higher water content than the decomposed granite within the actual tunnel, which resulted in a higher cold transfer to the applied shotcrete layer.
- * the shotcrete was applied to a 'relatively warm' surface in the main tunnel, as the surface tended to 'heat up' after excavation, and thereby allowing

for better strength achievements in the shotcrete.

- * There was a steeper temperature gradient in the test tunnel as the freezing agent used was nitrogen compared to the brine solution in the actual tunnel (i.e. nitrogen freezes the ground at a faster rate).
- * The insulating ability of the first 25 mm of shotcrete allowed subsequent layers of shotcrete to be generally unaffected by cold transfer induced by the freezing plant (i.e. the cold transfer induced by the freezing plant was generally not great enough to penetrate the 25 mm of shotcrete effectively).
- * The in situ cores drilled from the main tunnel had to be trimmed so as to form a planar crushing edge i.e. to allow for a proper crushing test to be applied due to this approximately 25 mm was removed, that is the layer affected by the frozen surface of the ice tunnel arch. Therefore the shotcrete within the in situ cores was generally applied close to ambient temperatures, and thereby giving good strength results.
- * The cement content used in the shotcrete mix of the main tunnel (450 kg/m^3) was higher than the content used in the test tunnel (400 kg/m^3), plus the tunnel shotcrete had an accelerator added to it (3% of cement weight), this therefore generated more heat (i.e. from the mix reaction) which increased the temperature of the surrounding ground, thereby allowing higher strength to develop within the main tunnel shotcrete.

Finally, the application of the permanent tunnel support (reinforced shotcrete) to the main tunnel was carried out very successfully, even though past trials had caused some concern over the strengths of the shotcrete on a frozen surface. Furthermore, due to the high strengths achieved, it was decided that the aggregate content be reduced by 50%, and thereby reducing the cost of the new mix design. The application of the new mix design also produced good strength results in stages 3, 4 and 5 of the five 32-metre stages.

CHAPTER 7

ALTERNATIVE METHODS FOR THE EXCAVATION OF THE MAIN SOFT GROUND TUNNEL

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CHAPTER 7: ALTERNATIVE METHODS FOR THE EXCAVATION OF THE
MAIN SOFT GROUND TUNNEL

The methods available in the excavation of the soft ground tunnel depends largely on the properties of the material present within the decomposed granite. Certain properties within the decomposed granite may restrict or prevent the use of various methods of excavation, but the decomposed granite may be made artificially acceptable to certain methods of excavation, by the implementation of ground condition improvement methods. Such ground condition improvement methods may include such aspects as extensive drainage of the proposed excavation area(s), the freezing of excavation area, the implementation of grouting techniques etc.

The following methods may be considered as alternative methods for the excavation of the main soft ground tunnel, as opposed to the artificial ground freezing method adopted in both the pilot tunnel and the main (south) Du Toitskloof soft ground tunnel.

The alternative methods that could have been considered for the excavation of the main soft ground tunnel are:

1. The shield method: full face excavation.
2. The use of a tunnelling machine (mole).
3. Multiple heading attack method.

The utilisation of the above alternative methods of excavation would depend heavily on the method's acceptability, considering the conditions which surround the project. Factors, such as the economics (costs) of the method, the availability of the method locally, the skills required by the method, the success rate of the method etc. all play a vital role in the selection criteria of the method, to be adopted in the excavation of the main soft ground tunnel.

The above mentioned alternative methods considering the Du Toitskloof project background, will be analysed below, for the method's acceptability in the Du Toitskloof soft ground tunnel.

7.1 THE SHIELD METHOD: FULL FACE EXCAVATION

7.1.1 General description of the method

Shield tunnelling consists of a moving metal casing (shield), which has a 'sharpened end' at the face of the tunnel. The shield is driven forward by the use of hydraulic jacks. The shield is driven in advance of the permanent tunnel lining, which supports the ground of the tunnel around the 'construction' area. The shield or metal casing is open at both ends, so as to allow for the removal of the material at the tunnel face and also to allow for the erection of the permanent lining at the rear of the shield.

The shield method allows for the full face of the tunnel to be excavated, it offers a temporary moving support within the excavation and construction areas, i.e. it eliminates all other forms of temporary support requirements and it allows for speedy tunnel construction within a soft ground zone. The shield method does not apply hard rock excavations (see diagram 24, principles of shield system).

7.1.2 Application to the Du Toitskloof Tunnel Project

The use of the shield method in the Du Toitskloof project would be totally uneconomical, even though it may have been practically acceptable. The cost of manufacturing a shield to the diameter of the main tunnel would be a very costly exercise, and would be unacceptable for a mere 168 metres of tunnel, as the shield method cannot be adopted to excavate the hard rock section of the tunnel,

which constitutes 95% of the tunnel excavation works.

However, the use of the shield method in the excavation of the pilot tunnel soft ground section, especially after the collapse (sinkhole), could have been feasible, as the manufacture of the shield would have been very economical.

Finally, another problem that may have prevented the use of a shield method, is that local experience in this method is very limited and which would therefore require the importation of expertise.

Due to the above reasons, the shield method was not acceptable for the excavation and construction of the Du Toitskloof tunnel project.

7.2 THE USE OF A TUNNELLING MACHINE (MOLE)

7.2.1 General description of the method

A mole is a very complex and sophisticated machine which consists of a rotating "cutting face", which loosens the earth or breaks the rock at the tunnel face. The rotating cutting machinery is encased by a shield, which provides for temporary support around the tunnelling machine. Moles are generally more successful in soft rock and soft ground conditions, but are generally unacceptable in very hard rock conditions, where conventional methods of excavation are more acceptable and feasible. Finally, a mole advances forward by the cutting of the face of the tunnel, forming a tunnel of circular cross-section (see diagram 25,

26, and 27: typical tunnelling machines - moles).

7.2.2 Advantages and disadvantages of tunnelling machines which influences its acceptability in the Du Toitskloof tunnel project

7.2.2.1 The advantages of tunnelling machines

- * a smooth circular cross-section of the tunnel is achieved with less support required compared to where the tunnel has been created by drilling and blasting, i.e. there is no blast damage and the rock structure is generally more stable.
- * There is minimal over-break to the rock surface, which thereby promotes savings in cost.
- * The rate of tunnelling can be higher than conventional methods of tunnelling, but the rate is highly dependant on the suitability of the ground or rock e.g. hard rock produces slower rates of tunnelling, mixture of soft ground and hard rock also provides slower rates of tunnelling.
- * Less manpower is required with the use of tunnelling machines, but this could be classed as a disadvantage in an industry that requires labour intensive methods.

7.2.2.2 The disadvantages of tunnelling machines

- * Tunnelling machines are not suitable for tunnels of short lengths or for tunnels with very large diameters i.e. the cost factor here does not verify the use of such moles as compared to conventional tunnelling methods.

- * The initial cost of tunnelling machines is high, also considering that the machine has to be manufactured abroad and imported, as the technical knowledge to produce such machines locally is very limited.
- * The operation of such machines would require the importation of foreign experts.
- * Tunnelling machines cannot be adapted quickly to changing ground and rock conditions, e.g. from soft ground to hard rock requires replacement of different cutting teeth, etc.
- * Costs in tunnelling in hard rock are very expensive and do not compare with the relatively cheap 'drilling and blasting' techniques in hard rock tunnelling.

7.2.3 Application to the Du Toitskloof tunnel project

From the above mentioned, advantages and disadvantages of the use of a tunnelling machine (mole), an assessment can be made to whether or not a tunnelling machine is acceptable for the excavation of the Du Toitskloof tunnel. Due to the large diameter of the main tunnel and the costs involved in the establishment of such tunnelling machine i.e. large mole, initial costs, cost of expertise required etc., the use of such a tunnelling machine is totally unacceptable.

Furthermore, due to the changing rock-ground conditions expected within the main tunnel, the requirement for the project to be labour intensive so as to reduce unemployment in the area and limited experience in the industry of such methods, the utilisation of a tunnelling machine is unacceptable for such a 'one-off' tunnel project.

It was established that more conventional methods of tunnelling would fulfil the requirements of the Du Toitskloof project, especially for the approximate 3,8 kilometres of hard rock tunnel construction.

7.3 MULTIPLE HEADING ATTACK METHOD

7.3.1 General description of the method

This method has many forms of carrying out the multiple heading attack, and they are identified by various names, such as the Belgian method, the German method, Italian method., etc.

The multiple heading attack is generally applied to soft ground tunnelling and the type of method chosen i.e. German method etc., to carry out the multiple heading attack, is determined by the prevailing ground stability conditions within the proposed tunnel location.

Multiple heading attack can be used successfully where ground conditions are unstable as the multiple heading attack provides for the excavation of a number of small tunnels (drifts) within the main tunnel cross-section, which are more easily contained and managed in poor ground conditions (see drawing 8, typical multiple heading attack, which was applied to the second main (north) soft ground tunnel at the Du Toitskloof tunnel project).

Ground stabilisation techniques, such as cement grouting, extensive drainage etc., are usually applied to the proposed tunnel zone, before the actual multiple heading attack is applied for the excavation of the main tunnel.

After the ground has been sufficiently stabilised the excavation of the 'small' tunnels (drifts) commences within the main tunnel profile, and the sequence in which the drifts are excavated depend on the type of form chosen for the multiple heading attack.

7.3.2 Application to the Du Toitskloof tunnel project

The multiple heading attack and the artificial ground freezing methods were the two most acceptable methods for the excavation of the 168 metre soft ground section of the main Du Toitskloof tunnel.

Both methods required ground improvement methods to be applied, such as extensive drainage, cement grouting. The freezing method was adopted rather than the multiple heading attack, as the decomposed granite was very saturated, the application of the multiple heading attack would have imposed a greater risk factor in the excavation of the soft ground tunnel. Furthermore, the technique of ground freezing had proved to be very successful in the excavation of the pilot tunnel and this made it more acceptable for the excavation of the main soft ground tunnel.

7.3.2.1 The second main soft ground tunnel -
(north soft ground tunnel)

During the construction of the main hard rock tunnel (south tunnel) undertaken by the consortium of Concor-Hochtief (West Germany), there was a proposal made by the consortium to the client (NTC), for the excavation of the (north), main tunnel, which previously was not to be constructed, as financial funds were insufficient. But due to a very reasonable quote for the excavation of the second main tunnel, the client decided to award the contract for the second main tunnel to the consortium. This brought about the need for the excavation of the second main soft ground tunnel i.e. the north soft ground tunnel section.

The consortium had to decide on the type of method that was going to be utilised for the excavation of north soft ground tunnel. After analysing the possibilities, a decision was made to utilise a multiple heading attack method instead of an artificial ground freezing method. The former method was chosen as the decomposed granite had become relatively 'stable' compared to the initial conditions experienced at the beginning of the tunnel project. The decomposed granite had become 'more stable' due to the lowering of the water content within the soil i.e. the lowering of the water table. The water table had been lowered due to the continuous drainage carried out within the other excavated areas surrounding the proposed new main north tunnel i.e. drainage within the main soft tunnel, the pilot tunnel etc. This therefore provided a more 'dry' decomposed granite zone.

Furthermore, the ground freezing method was not chosen as it implied the re-importation of the freezing equipment, which was costly and time consuming. A brief explanation will now be given of the multiple heading attack method applied to the second soft ground tunnel section (north main soft ground tunnel).

7.3.2.1.1 Multiple heading attack applied to the north main soft ground tunnel

The multiple heading attack started in the beginning of 1986 on the north main soft ground tunnel, the excavation involves the sequential excavation of small tunnels (drifts) within the main tunnel profile which will finally form the excavated main tunnel (see photographs 15, 16 and 17: position of drifts within main tunnel profile).

Before and during this method of excavation, extensive dewatering (drainage) was carried out, reducing the water content within the area. The extensive drainage consisted of the installation of "vacuum lances" within the area and ahead of each drift, usually in a 'fan' pattern (see drawing 8, typical multiple heading attack undertaken at the north main soft ground tunnel - position of vacuum drainage tubes) and (see photograph 18, typical vacuum lances).

After each drift had been excavated approximately 1 - 2 metres, 'temporary-part permanent' support was applied in the form of mesh, steel arches, shotcrete etc. The 'part permanent' support refers to part of the steel arches placed within each drift would form part of the final arch of the main tunnel (see drawing 8: 'temporary-part permanent support' applied to the drifts forming part of the

main soft ground tunnel support) and (photograph 19: application of drift support).

The sequence of tunnelling operations within the main tunnel's profile, in the form of drifts (small independent tunnels) is as follows:

(Refer to drawing 8, typical multiple heading attack undertaken at the north main soft ground tunnel) and (see photographs 15 to 18).

Stage 1

As per drawing, parts 1 are excavated and supported first, excavated and support applied in 1 metre intervals, the 'outer' sides of the support arches forming part of the main tunnel's arch support.

Stage 2

As per drawing, part 2 is excavated and supported, in the same manner as in Stage 1, but here the crowns of the steel arches installed as support in the drift become part of the main tunnel's arch support.

The excavation of the other parts i.e. 3, 4, 5, 6 and 7 are undertaken in the same manner, until the final main tunnel's profile is achieved.

Finally, I personally think that the application of the above method is very risky as the nature of the decomposed granite does not have a high load bearing capacity, which may cause certain sections of the tunnel works to collapse,

especially in drifts 2 and 4.

Comparing it to the first (south) main soft ground tunnel, where ground freezing was used, the ground freezing method provided ample support before excavation within the main tunnel commenced.

CHAPTER 8

CONCLUSION

The art of tunnelling has developed greatly over the years from tunnels being dug by hand, using fire setting, and other primitive methods, to tunnels formed by the most sophisticated 'tunnelling machines' of today.

Tunnelling operations in South Africa have reached another milestone with the construction of the Du Toitskloof tunnel. The tunnel will be the longest road tunnel in South Africa, and is one of the most complex civil engineering projects ever undertaken in the Republic.

The tunnel was a multi-contract project, employing many South African and foreign contractors. Though much of the works incorporated traditional tunnelling methods, the soft ground tunnel contract utilised a very interesting and complex system of ground freezing for the stabilisation of the surrounding saturated decomposed granite.

Personally, having been on site on many occasions, I have observed the complexities that exist within such a tunnel venture.

When considering the large cross-sectional area of the tunnel, and the very poor quality of the decomposed granite experienced on site, it is difficult to believe that a full face excavation cycle was possible, under ground freezing applications. The design requirements and precise implementation of such a freezing method in forming the frozen arch and vertical bulkheads, providing temporary tunnel support was achieved with great success, although many problems did exist.

After having understood the problems that existed and the limitations surrounding possible alternative solutions, the following approaches could have been utilised in resolving the significant problems:

- * The drilling of horizontal freeze holes within the proposed frozen arch caused great difficulties, particularly in achieving the required alignment and accuracy of the holes along the 32-metre section.

The problem could have been prevented by either early participation by nominated sub-contractor who was to undertake tests with various drilling machinery and accessories prior to the commencement of the contract. Or, "shorter lengths" of holes should have been drilled, reducing the tendency of the holes to deflect out of alignment, e.g. 16-metre stages, giving 16 metre hole lengths per stage. This alternative would increase the number of vertical bulkhead drillings, but as the drilling of vertical holes were undertaken relatively successfully, the alternative solution would have been feasible.

- * Surveying freeze holes with the aid of a portable borehole deflectometer, proved to be very inaccurate at times, due to the sensitivity of the equipment.

It was made known to me that an alternative instrument, capable of surveying such holes more accurately was not available.

I thought that as the drilling of the horizontal and vertical freeze holes was a major part and of great importance to the success of the temporary frozen support, greater effort should be made in research and development, to develop an alternative instrument for future operations.

- * Leaks formed around joints/connections of the freezing circuit.

Due to the freezing lances being manufactured locally, and the lack of technology in being able to manufacture such 'high standard' pressure coupling, experts should have realised the implications of such poor quality coupling under great pressure. Delays may have been prevented, if the

pressure couplings had undergone proper pressure testing prior to installation, then it would have been realised at an earlier date, that alternative high quality coupling were required from abroad. All in all, in my opinion an example of poor planning.

- * Portable ramp required by roadhead, due to inadequate 'reach' capabilities, in excavating the crown of the frozen tunnel. The problem would have been prevented with proper planning of plant capabilities i.e. extension to the roadheader could have been designed and assembled prior to commencement of the excavation stage.

Generally, the above problems could probably have been prevented by adequate planning with proper insight into the requirements of the soft ground tunnel contract.

Taking a look at the overall tunnel project, much knowledge has been gained by South African civil engineering companies on the complexities of tunnelling operations under traditional and advanced tunnelling methods, this alone has made the project a worthwhile exercise. Furthermore, due to the complexities of the tunnel construction, much interest has developed abroad in the outcome of the final project, which makes overseas construction companies aware of South Africa's construction industry's capabilities, and may produce work for South African companies abroad.

The tunnel will boost the standard of the national road system, which will allow traffic to flow freely between Cape Town and Worcester, and in the long term may encourage development in Worcester and the surrounding areas. Furthermore, road users will benefit in savings of petrol costs, wear and tear and travelling time, due to the elimination of the mountain pass/route.

The tunnel will become another 'great' landmark in South Africa, and will provide the country with a more efficient national road system for the future, which is important for a developing

country.

Personally, even though the great cost of such a project may not justify its need, considering the state of the country at present where such capital could have been put to better use, benefitting the individual more directly. However as unemployment is increasing rapidly, such a project has provided many with work, thereby providing some direct benefit for the 'man in the street'.

The civil engineering industry of South Africa has accomplished the construction of a tunnel that could be ranked as one of the world's top 20 tunnelling accomplishments, considering the complexities of such a tunnel project.

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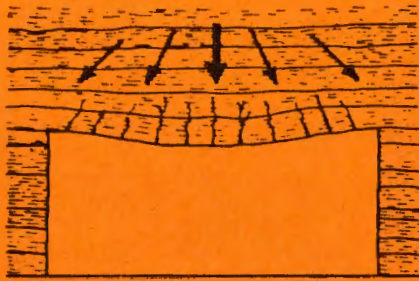
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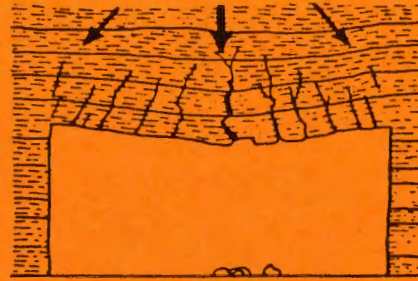
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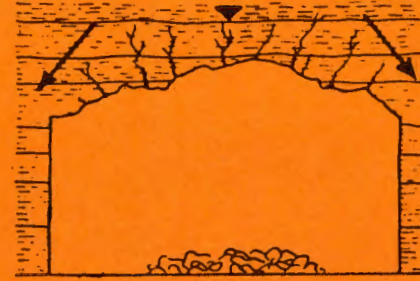
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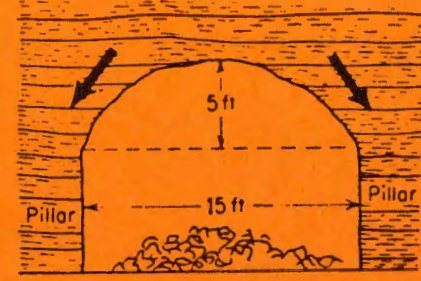
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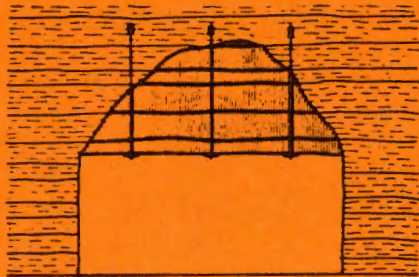
cause it to crack, followed by ...



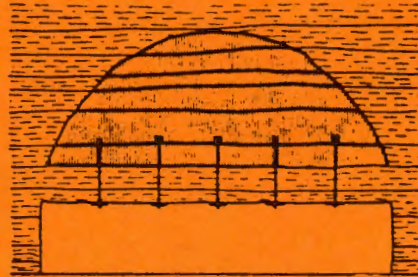
rockfalls that enlarge excavation until ...



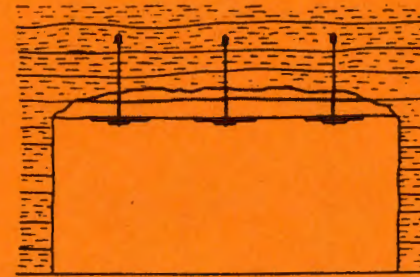
natural arch forms and falls stop.



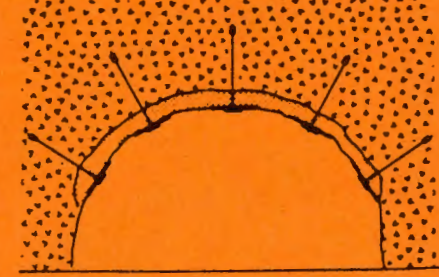
Remedy: Rock bolts can suspend roof;



pin strata to form a rock beam;

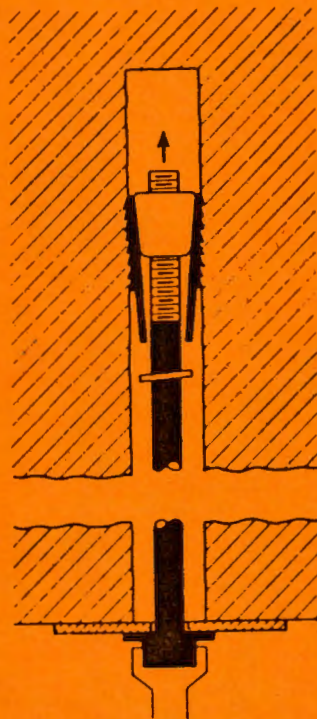
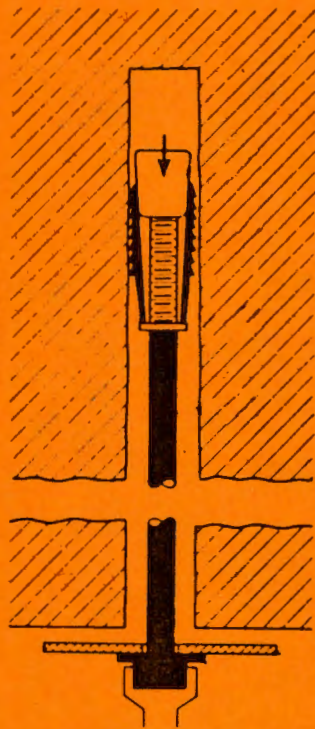


stabilize rock that tends to spall;

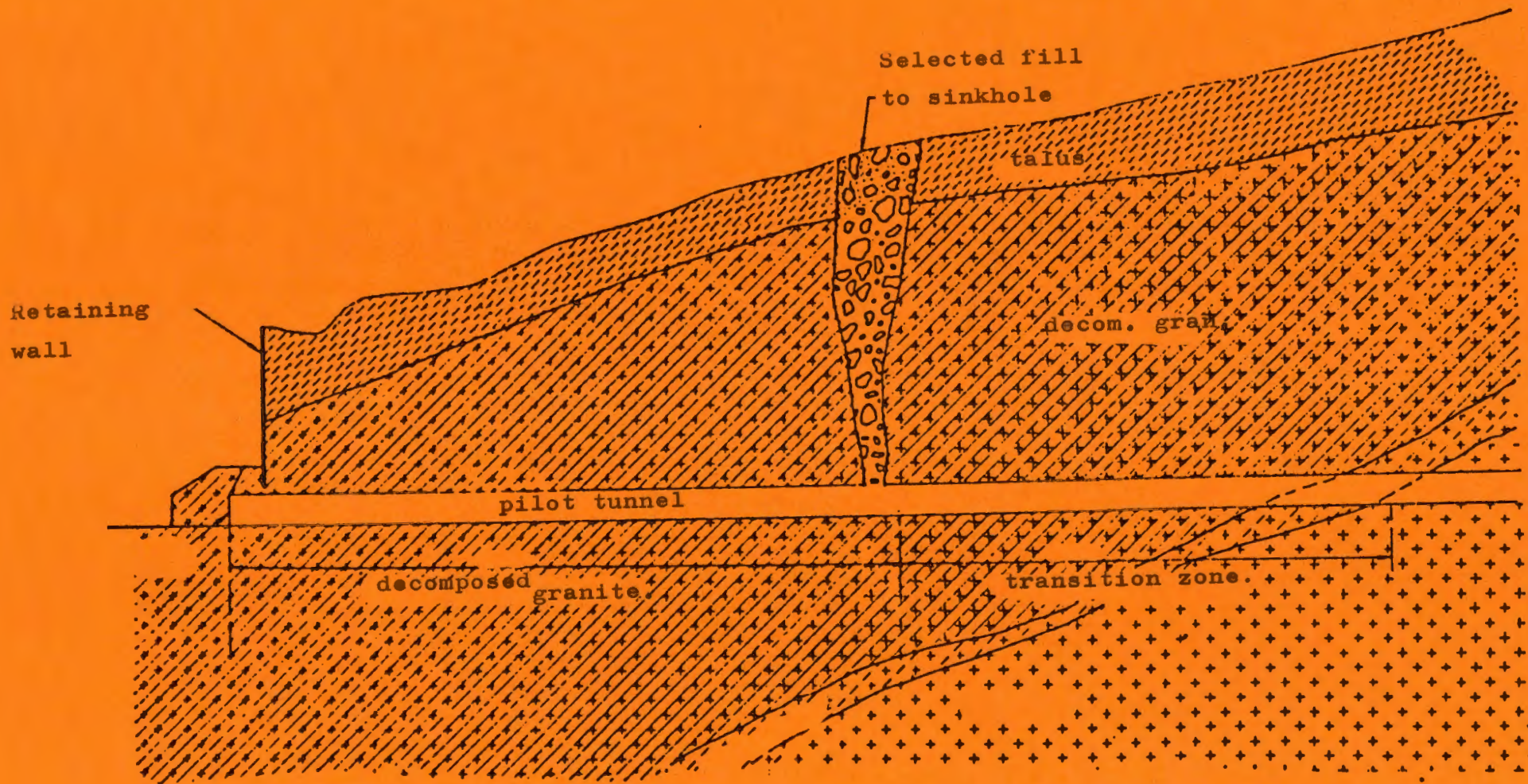


prevent rock burst in rock under pressure.

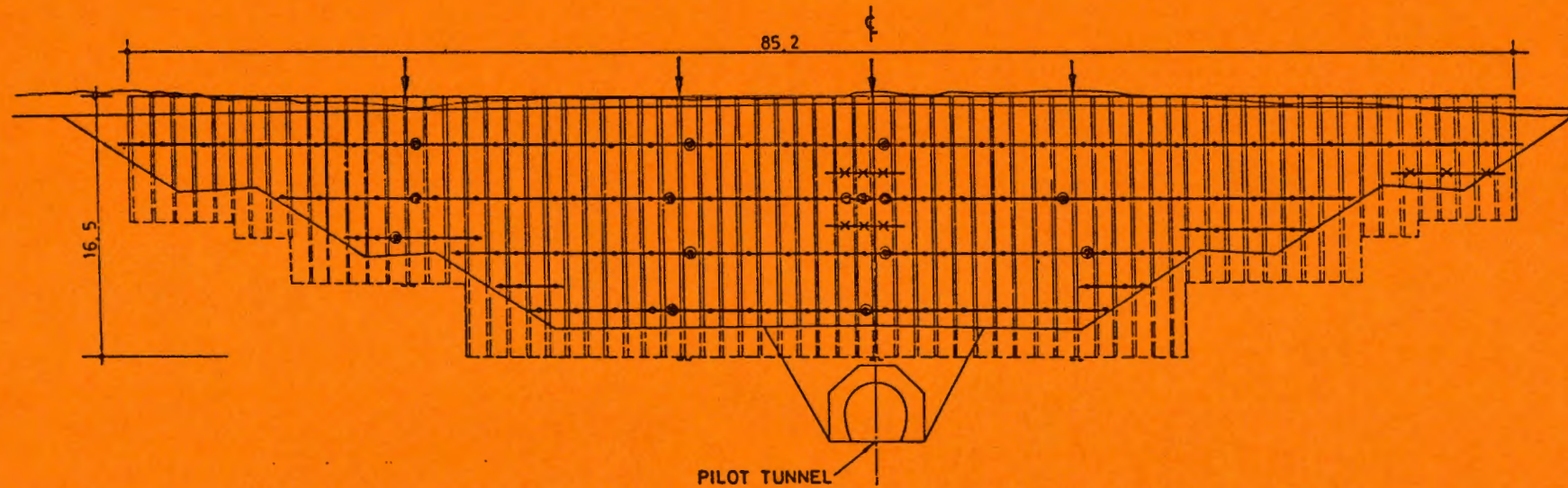
D1. ROCK BOLT APPLICATION. (refer bib. no 2i).



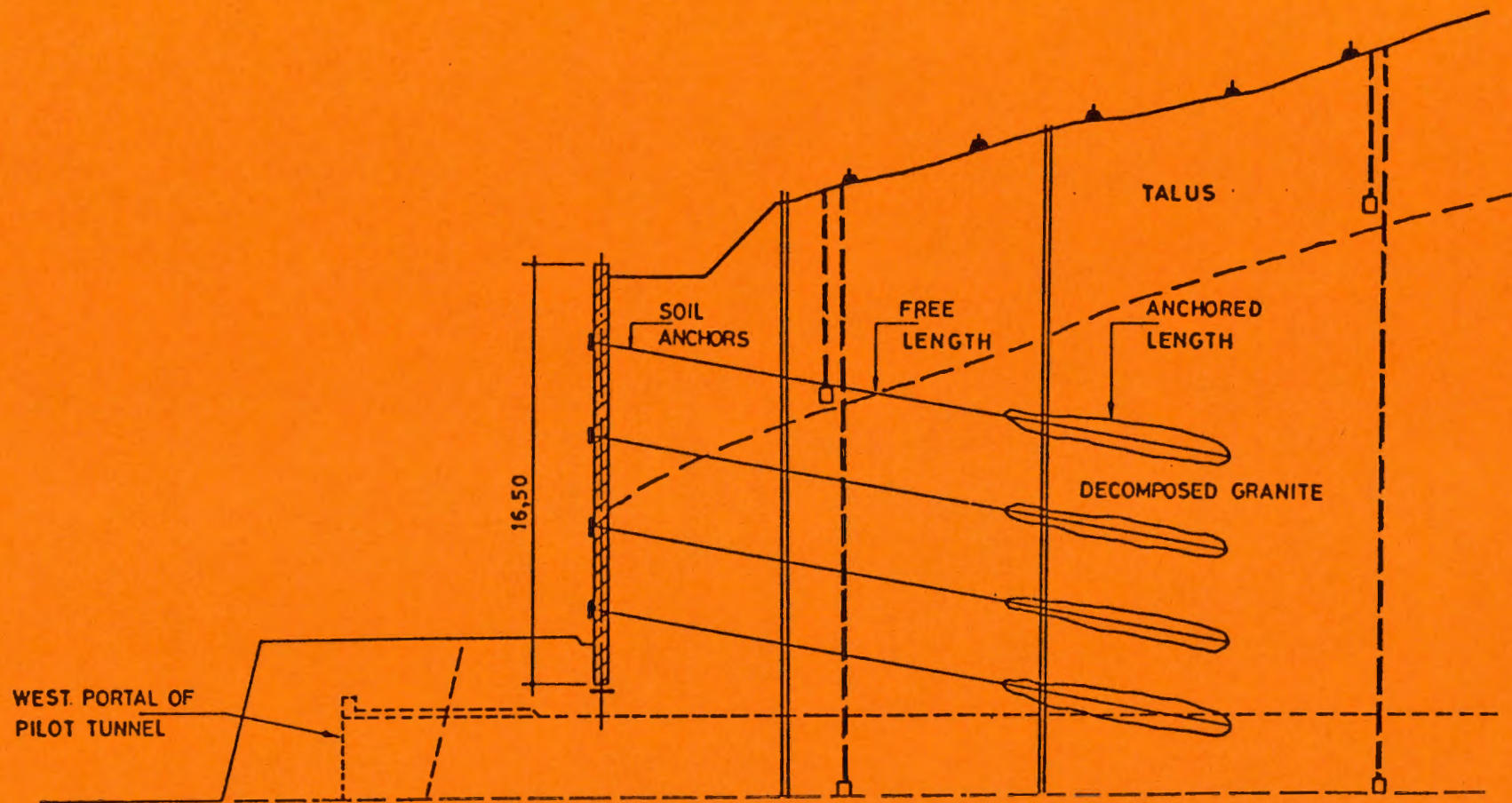
D2. ROCK BOLT APPLICATION. (refer bib. no 21)



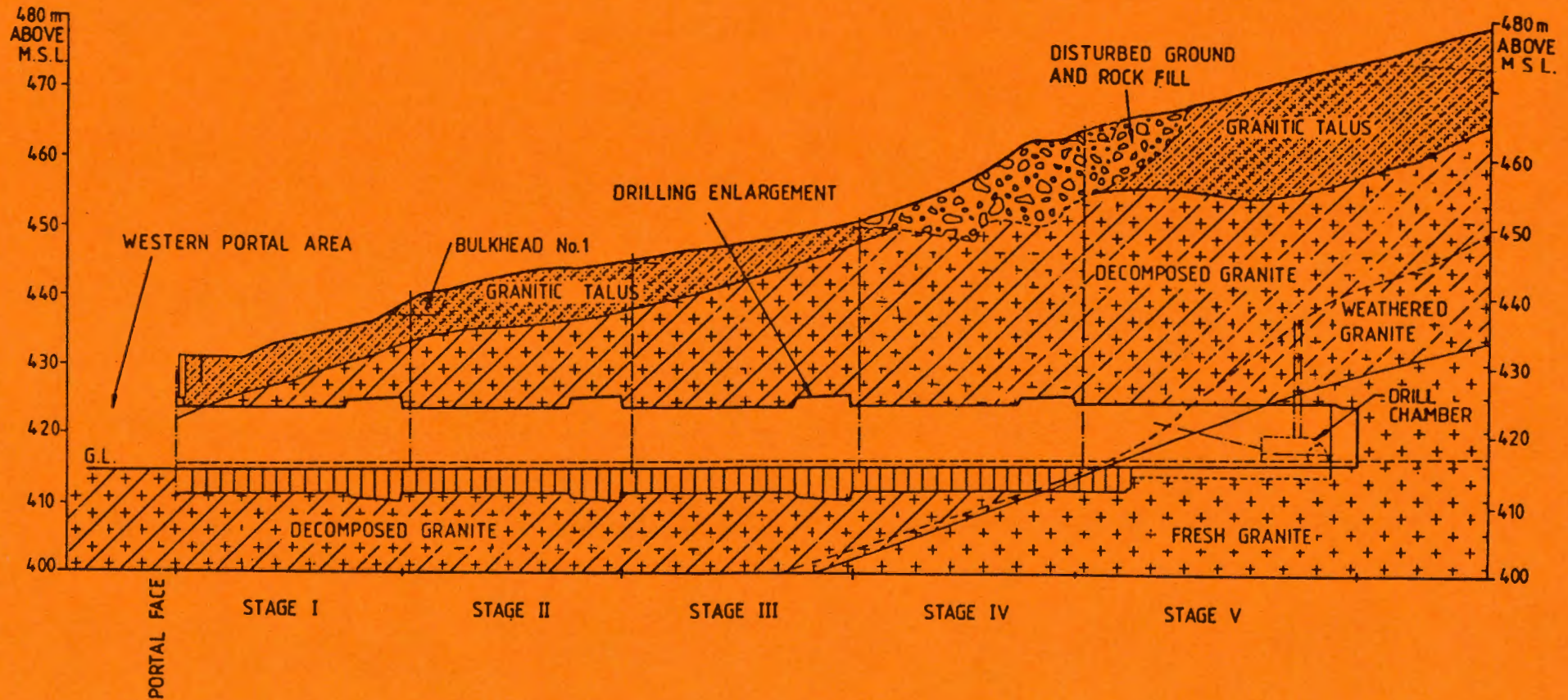
D3. Longitudinal section through the decomposed granite zone illustrating the massive collapse.
(reference no.1).



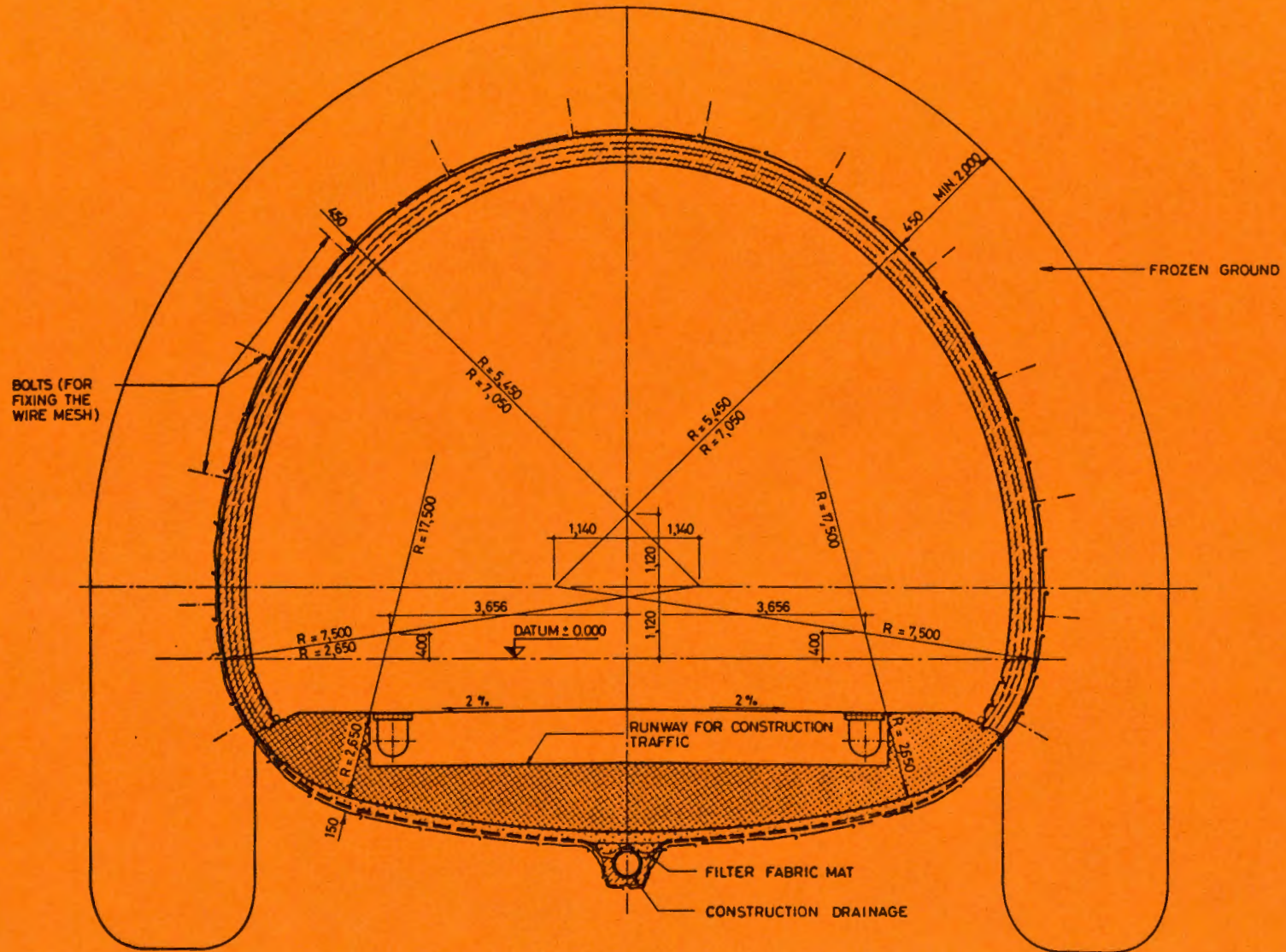
D4. Western portal temporary tied-back retaining wall.
(refer bib. no. 12).



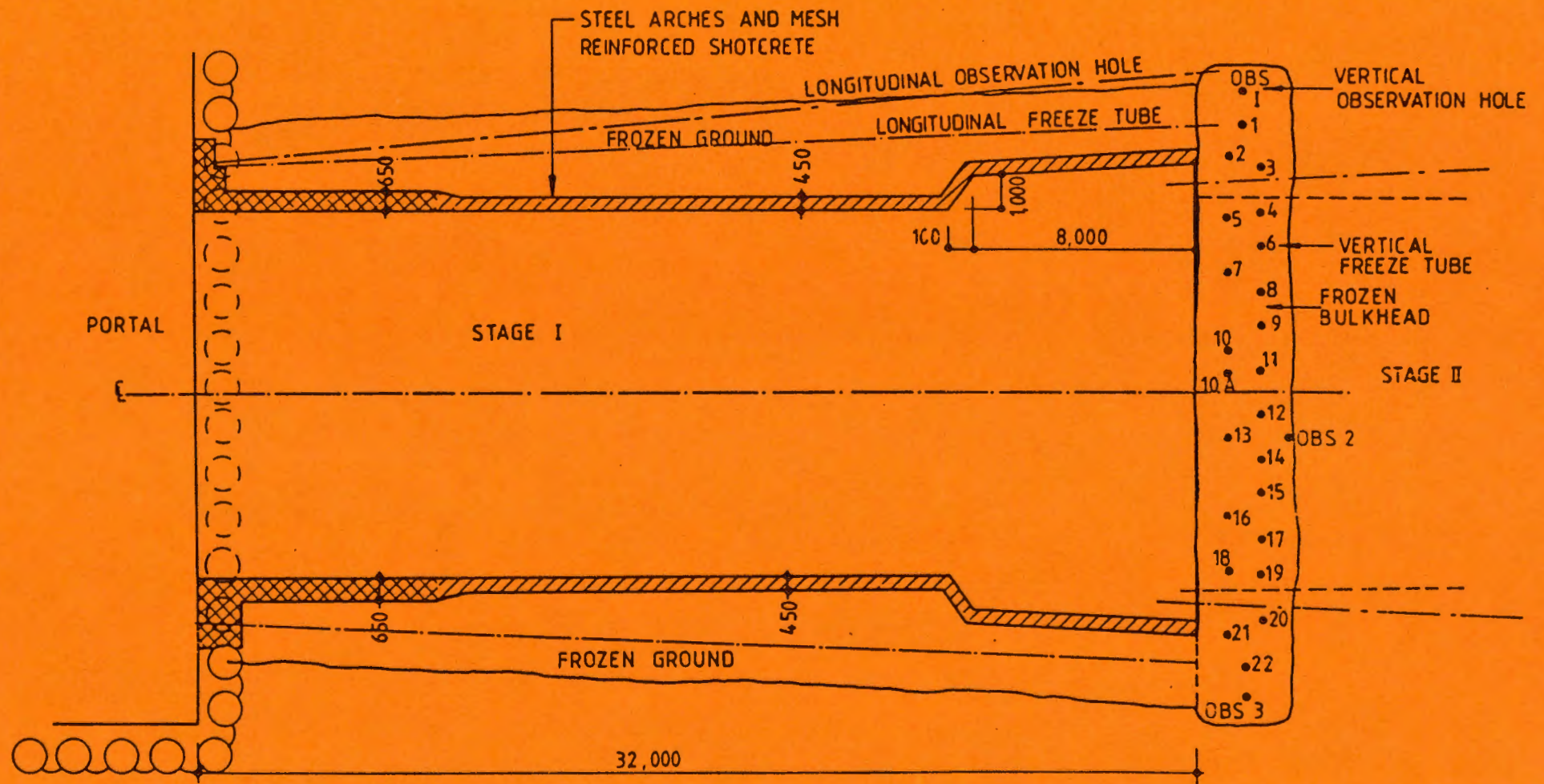
D5. Western portal temporary tied-back retaining wall.
 (refer b.b. Na 12).



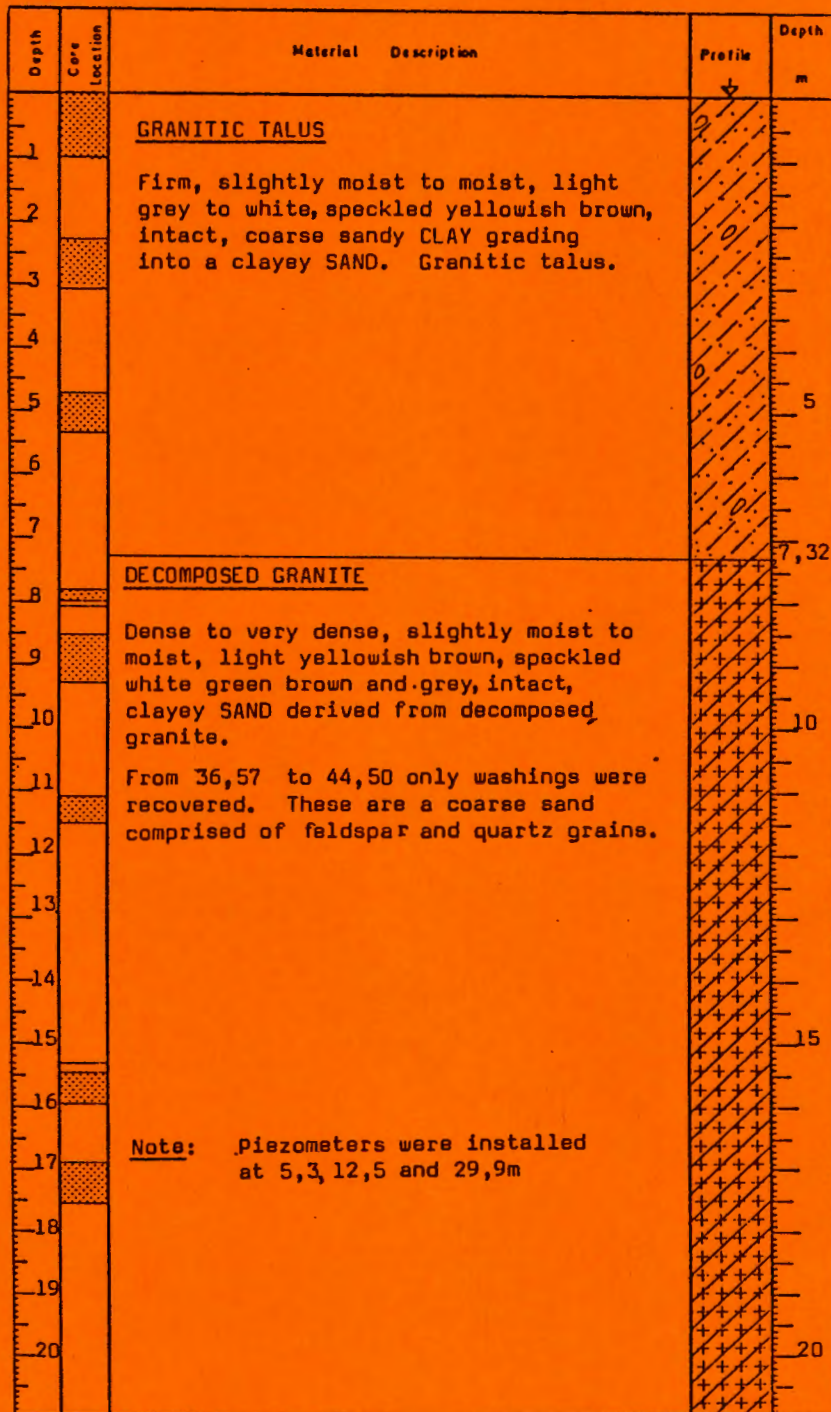
D6. Longitudinal section through the soft ground tunnel, showing five freezing stages. (refer bib no. 9).



D7. Cross-section through frozen soft ground tunnel.
 (reference NO. 1.).



D8. Plan of first 32-metre stage, showing the vertical bulkhead and position of freeze lances.



D9. Typical core sample results obtained from an exploratory borehole. (reference NO. 1).

Depth	Core Location	Material Description	Profile	Depth m
20		<u>DECOMPOSED GRANITE</u> (Continued)		20
21				21
22				22
23				23
24				24
25				25
26				26
27				27
28				28
29				29
30				30
31				31
32				32
33				33
34				34
35				35
36				36
37				37
38				38
39				39
40				40

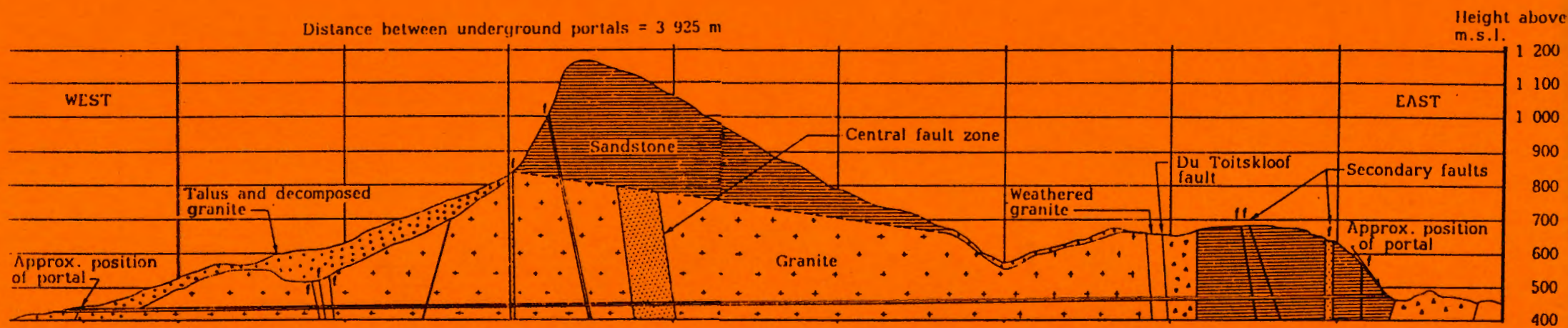
D9. (Cont.)

PTO.

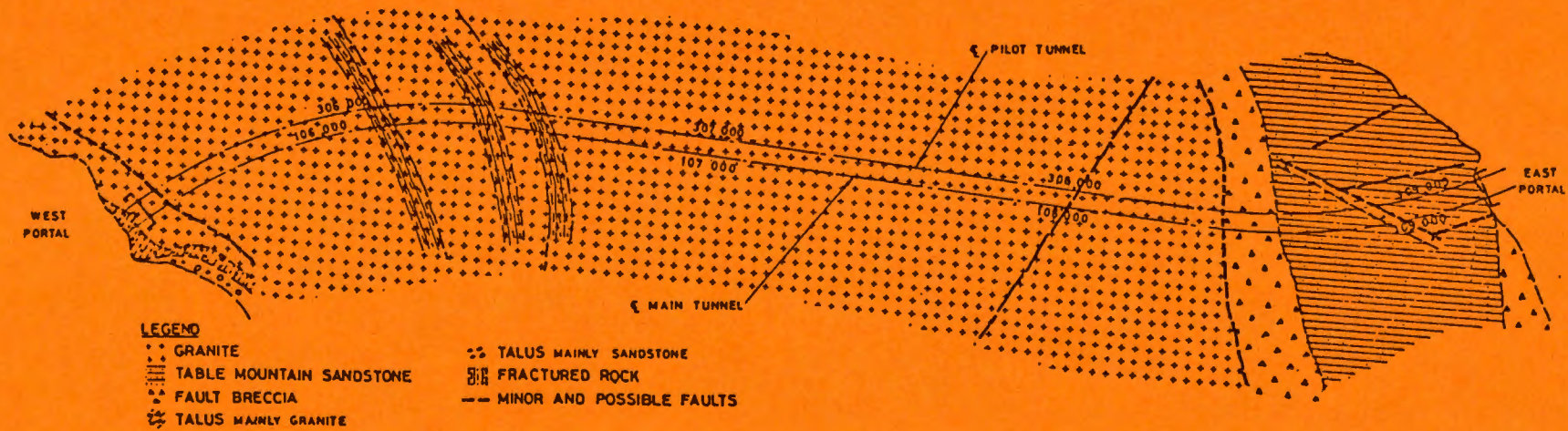
Depth	Core Location	Material Description	Profile	Depth m
40		<u>DECOMPOSED GRANITE</u> (Continued)	+++++	40
41			+++++	
42			+++++	
43			+++++	
44			+++++	44,52
45		<u>WEATHERED GRANITE</u>	+++++	45
46		Very soft rock varying with soft rock, green, orange, light grey and occasionally pink, shattered to fractured and occasionally microshattered, highly weathered granite. Prominent smooth vertically aligned joints with white clay gouge and chloritization on the joints, found at 50,94 and 51,60.	+++++	
47			+++++	
48			+++++	
49			+++++	
50			+++++	50
51			+++++	
52		<u>GRANITE</u>	+++++	51,60
53		Hard rock, light grey, green and black, jointed but shattered in places, slightly weathered coarse grained granite.	+++++	53,21

D9. (Cont.)

Distance between underground portals = 3 925 m



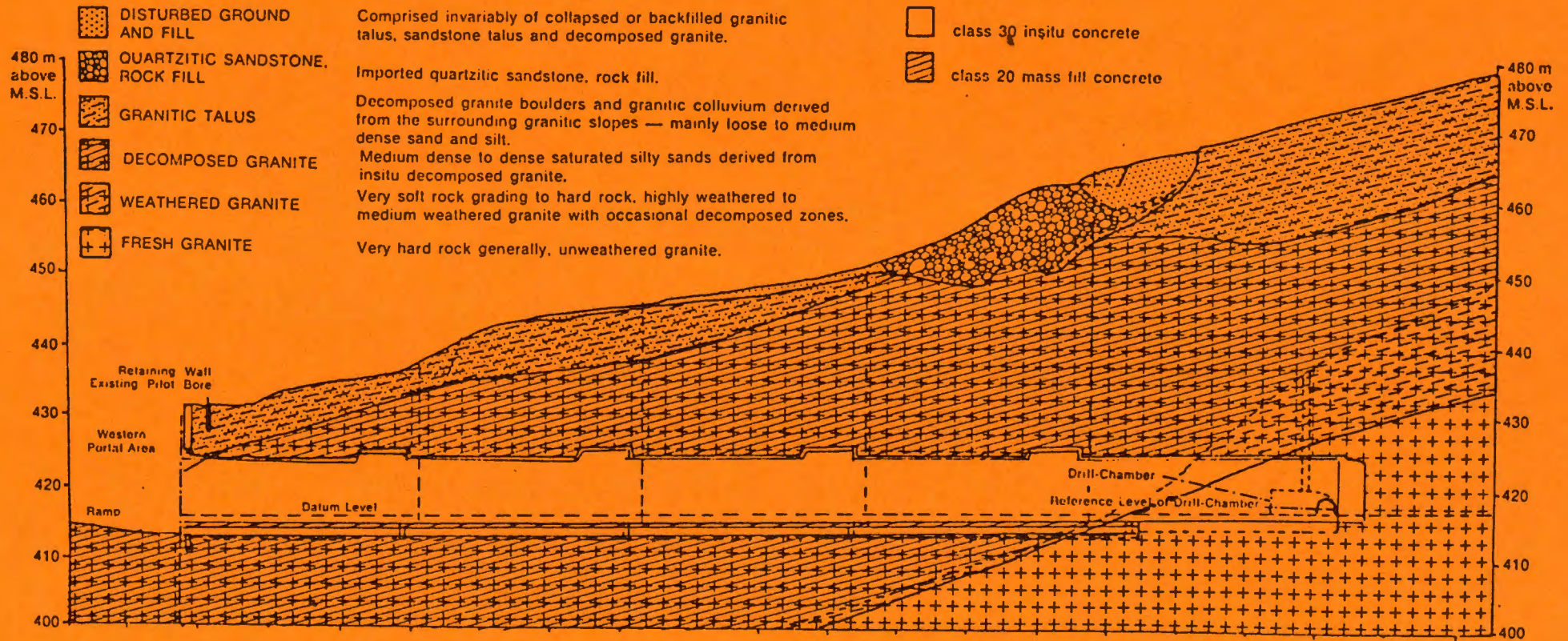
D10. Longitudinal section along the centre-line of the pilot tunnel.



D11. Geological sectional plan of the crown of the tunnel.

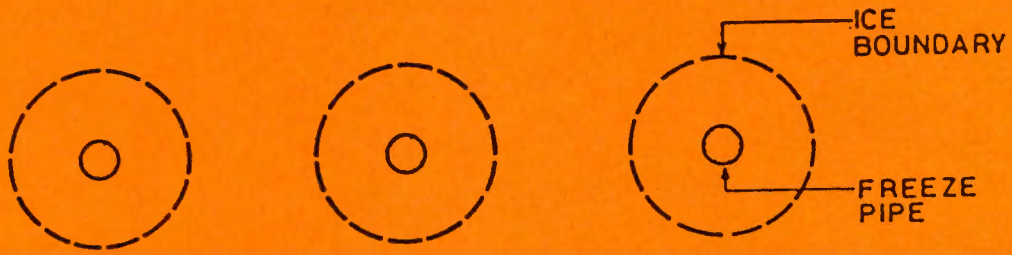
GEOLOGICAL LEGEND

**CONSTRUCTION MATERIAL
LEGEND**

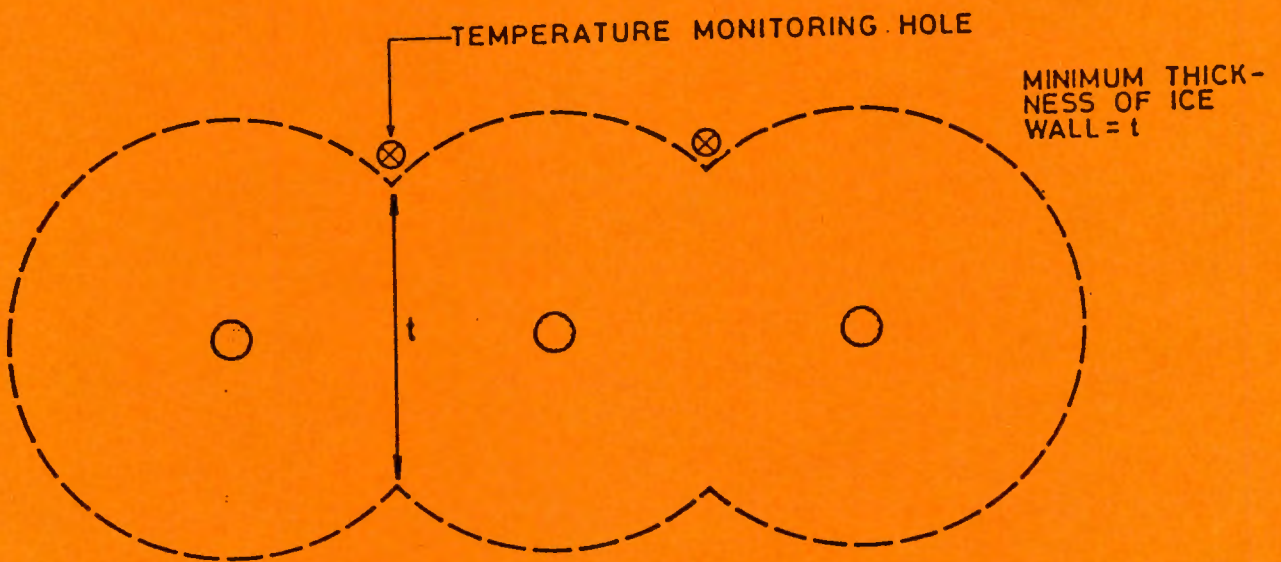
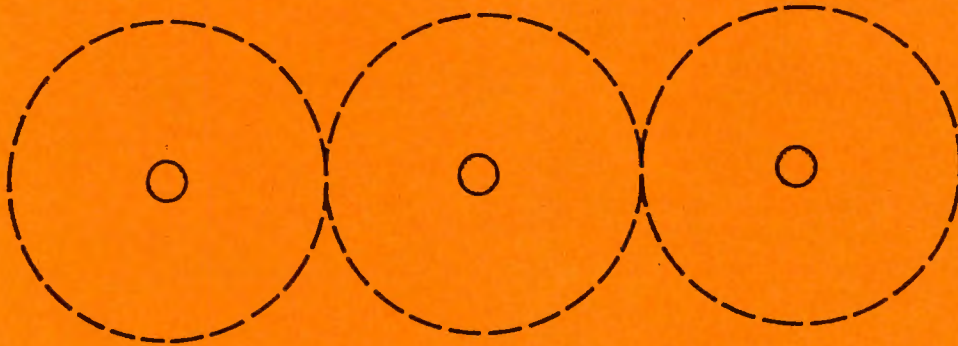


D12. Section through the decomposed granite zone of the tunnel : Western portal.

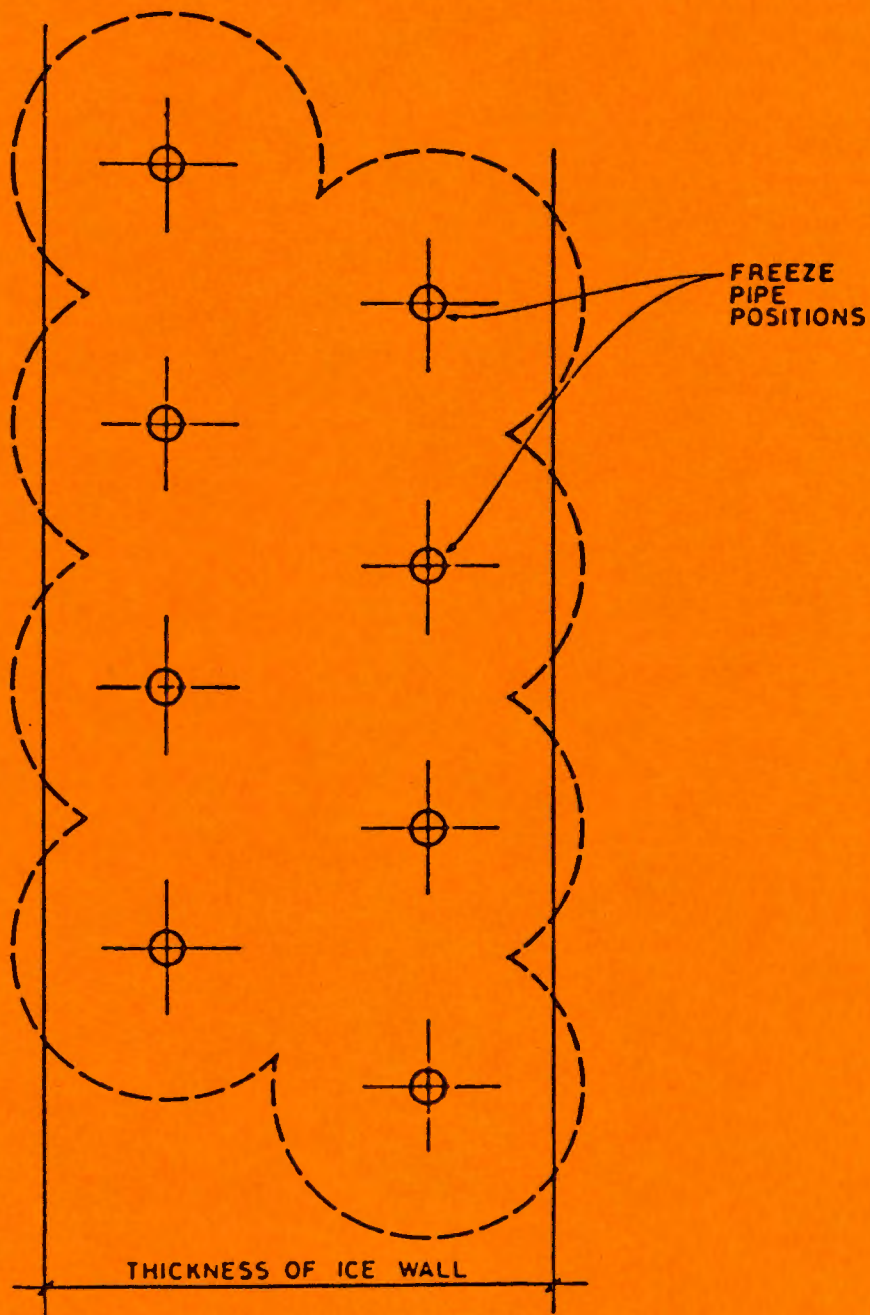
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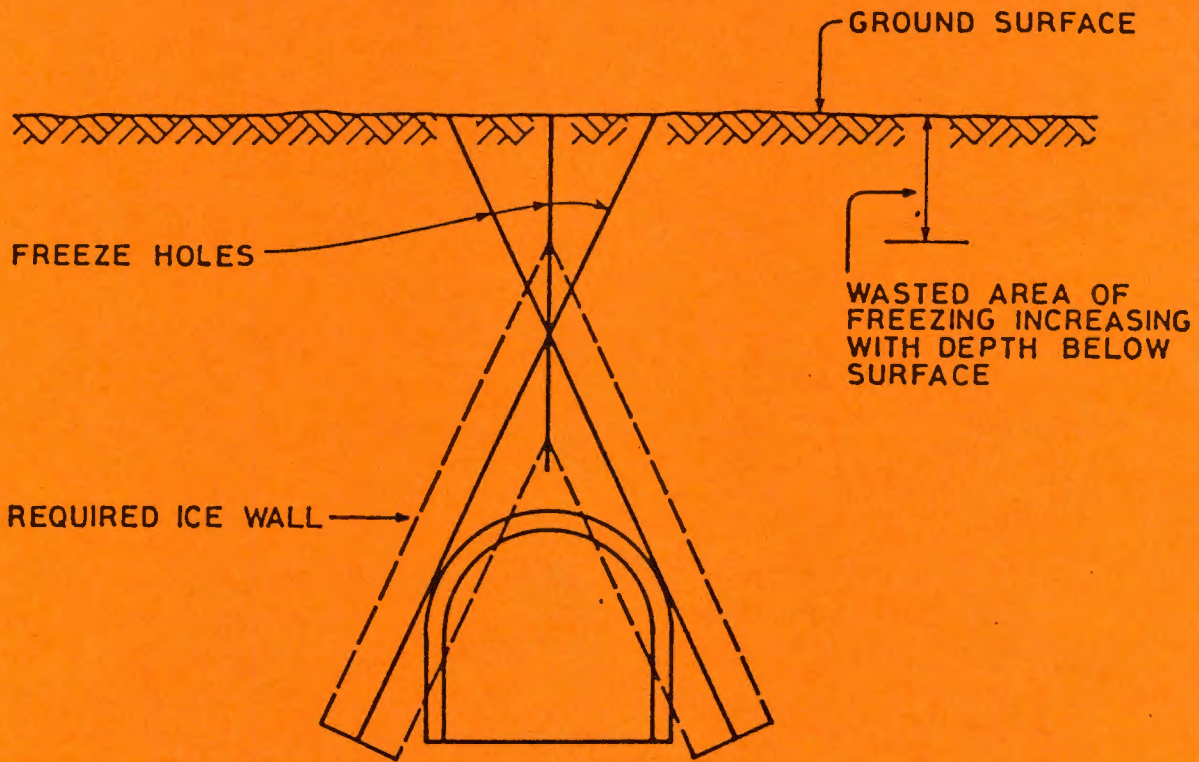
AFTER FREEZING FOR TIME T₁



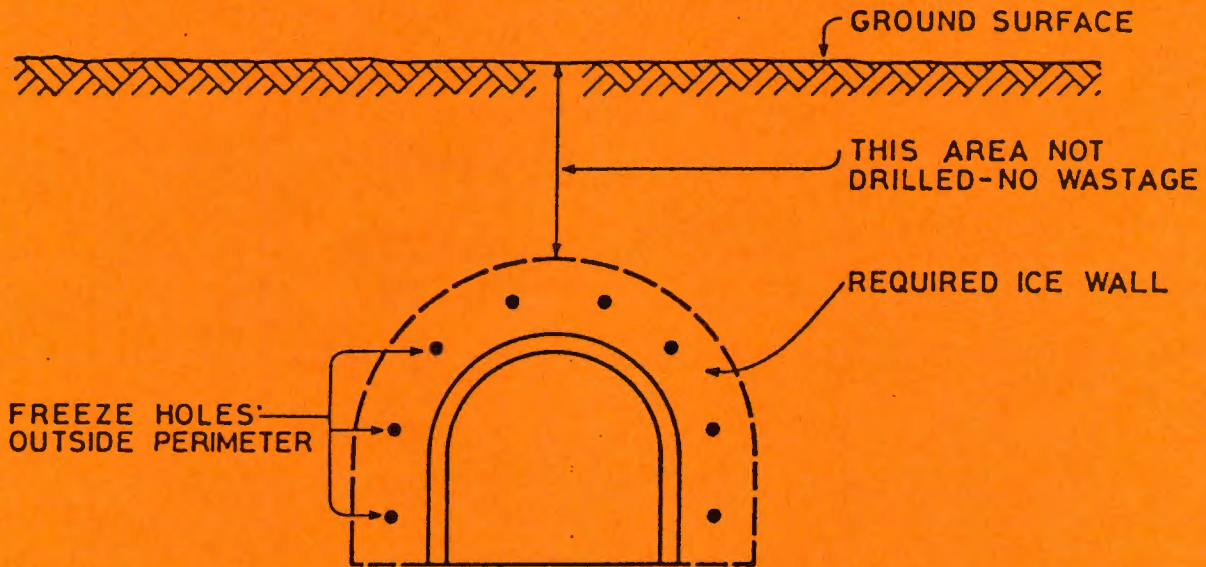
D13. Typical formation of a ice wall using freezing lances.
 (refer bib. no. 14).



D14. Positioning of freeze tubes within a vertical bulkhead.
(refer bib. no. 14).

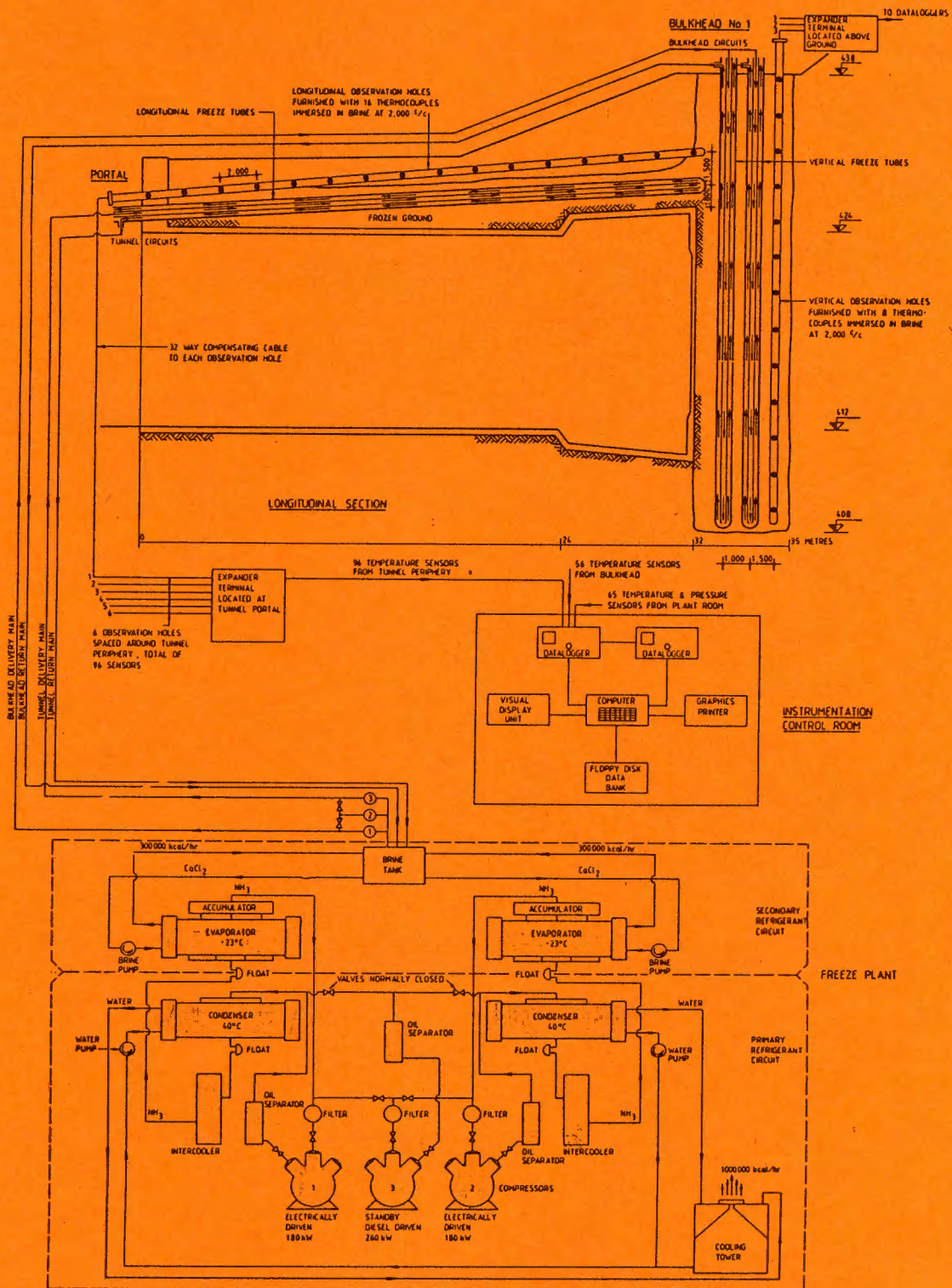


DRILLING FROM GROUND SURFACE



PARALLEL DRILLING

D15. Tent-like frozen areas for tunnel construction close to the surface and Parallel drilling forming ice arch.
 (refer bib.no. 14)



D16. Freeze plant layout.

 * DUTOITSKLOOF TUNNEL PROJECT *

K/CALORIE CALCULATIONS FOR DAY 364
 =====

TUNNEL
 STAGE 3

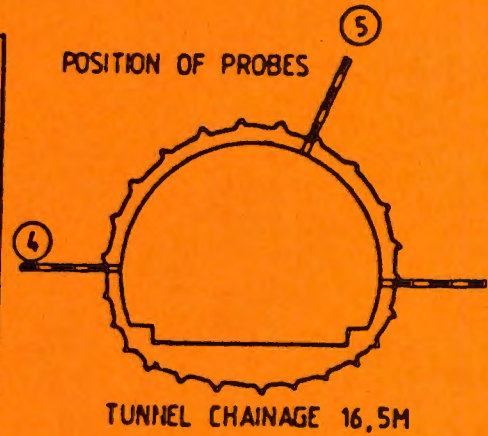
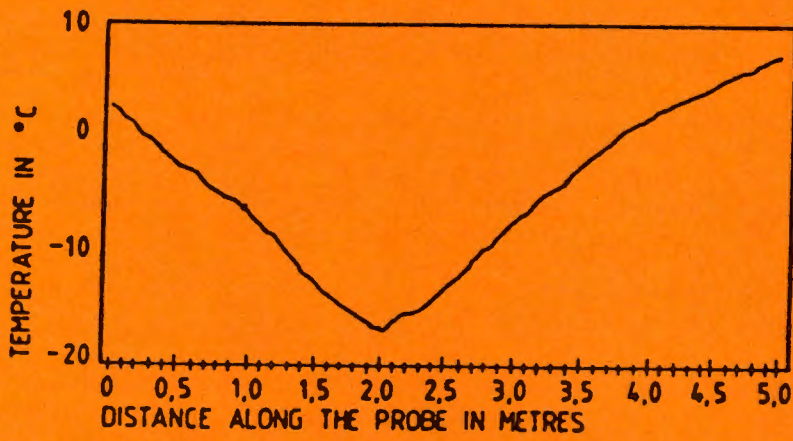
START OF 24 HOUR FLOW READING	34369000
END OF 24 HOUR FLOW READING	35180700
GALLONS IN 24 HOURS	811700
LITRES IN 24 HOURS	3701352
AVERAGE TEMPERATURE DIFFERENCE	1.81
SPECIFIC GRAVITY OF BRINE	1.29
K/CALORIES IN 24 HOURS	8642286.78
PREVIOUS TOTAL K/CALORIES	132773974
TOTAL TO DATE	141416261

 BULKHEAD

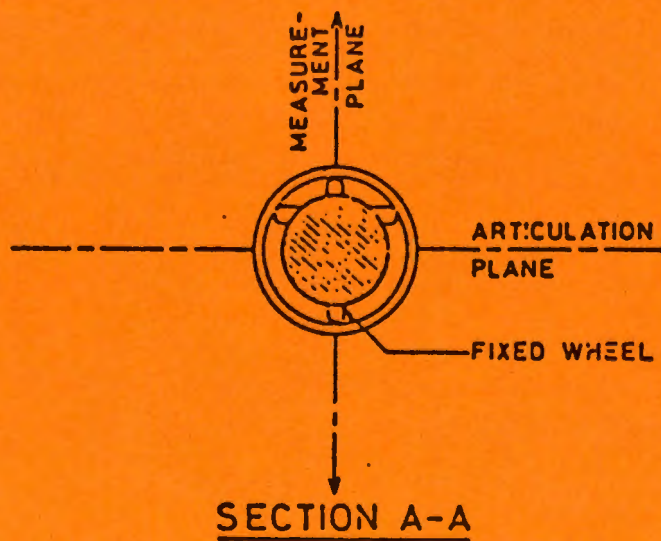
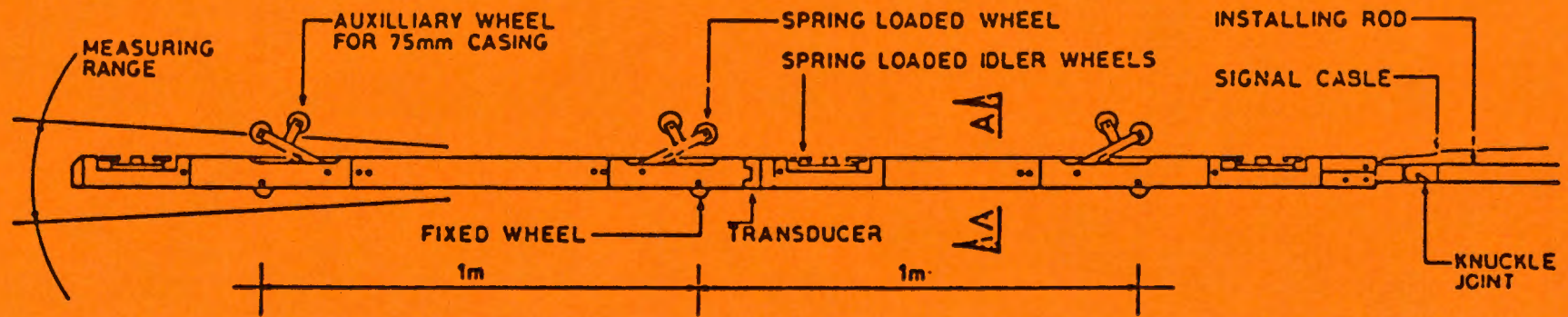
START OF 24 HOUR FLOW READING	825929900
END OF 24 HOUR FLOW READING	826052000
GALLONS IN 24 HOURS	122100
LITRES IN 24 HOURS	556776
AVERAGE TEMPERATURE DIFFERENCE	1.91
SPECIFIC GRAVITY OF BRINE	1.29
K/CALORIES IN 24 HOURS	1371840.39
PREVIOUS TOTAL K/CALORIES	955290972
TOTAL TO DATE	956662812

TOTAL K/CALORIES/24 HOURS	10014127.2
K/CALORIES PER HOUR	417255.299

RADIAL TEMPERATURE PROBE No. 5
TUNNEL CHAINAGE - 16,5 METRES



D18, The placement of radial probes containing thermocouples.

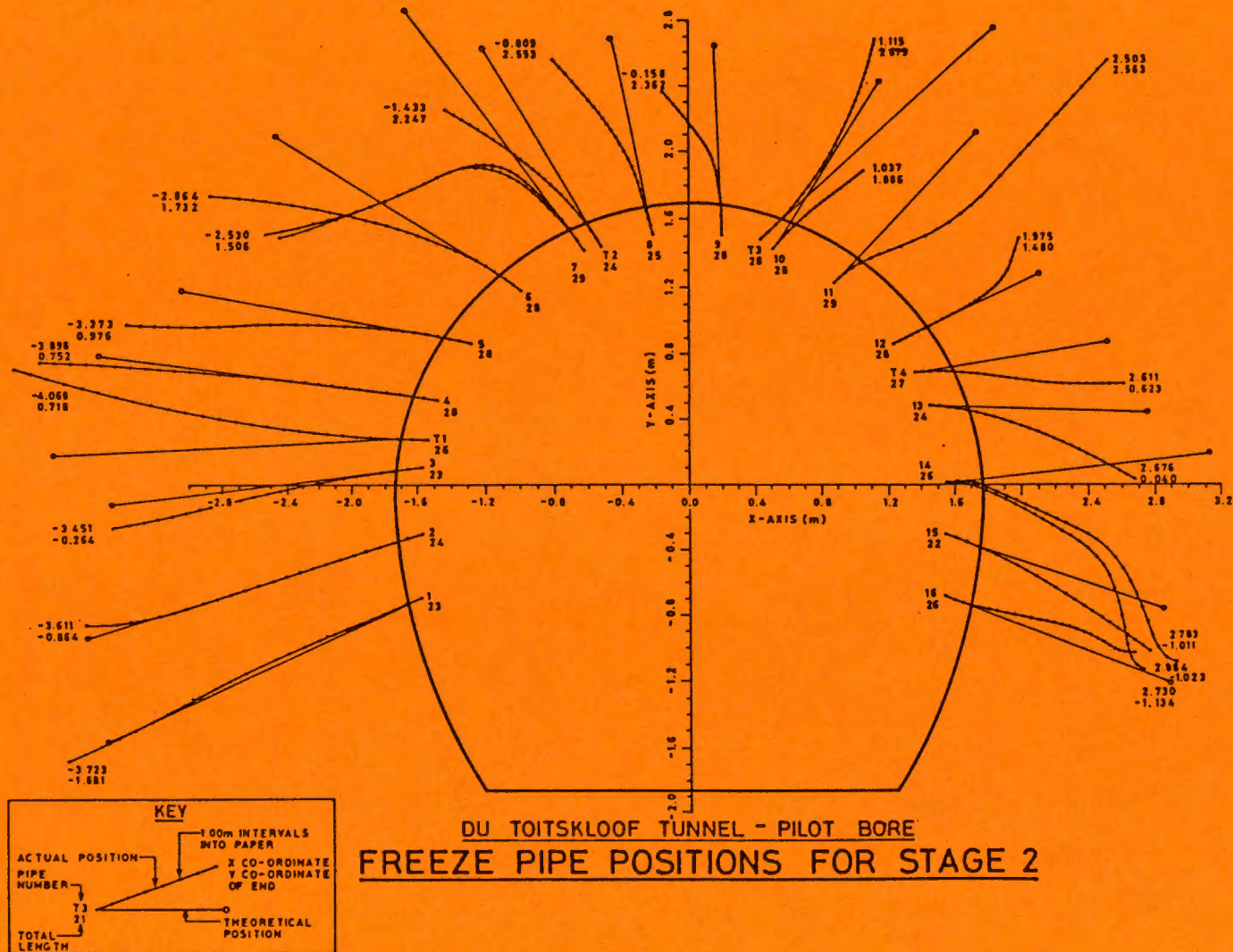


D19. A typical portable borehole deflectometer.
 (refer bib. no 11).

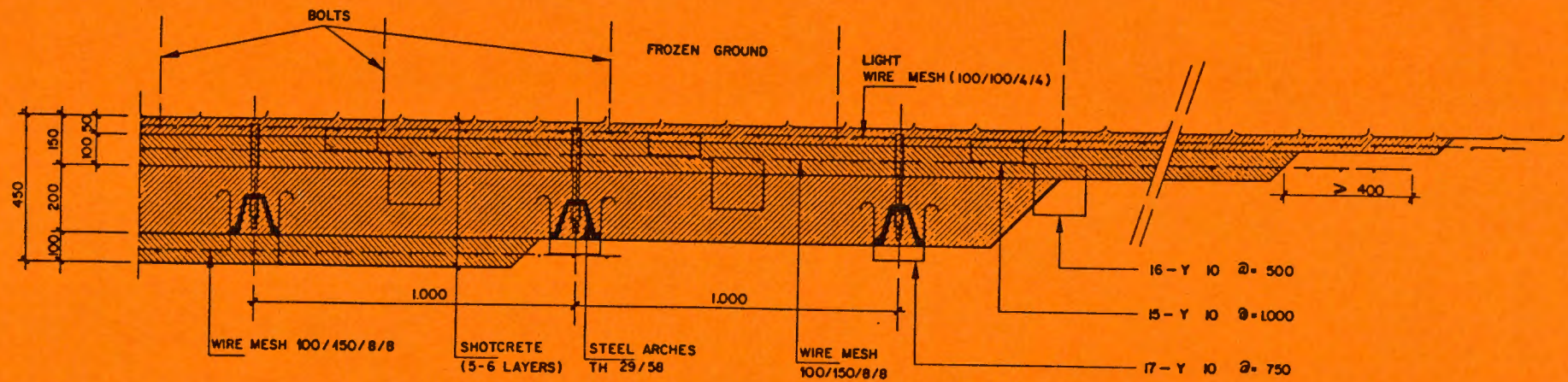
UDZ : 22222
 CAL : 3.667E-03
 PX : 4.636
 PY : 422.11
 QX : 5.015
 QY : 422.267
 D : 5
 A : .25

	DISTANCE CAL. (M)		CO-ORDS. (M)		CAL CO-ORDS. (M)		TH. CO-ORDS. (M)		ACT. CO-ORDS. (M)	
	X		X	Y	X	Y	X	Y	X	Y
0	5.034		5.034	422.275	422.275	5.034	422.275	5.034	422.275	
1	5.094		5.131	422.285	422.323	5.110	422.306	5.034	422.275	
2	5.136		5.251	422.274	422.387	5.186	422.338	5.112	422.304	
3	5.158		5.389	422.242	422.469	5.261	422.369	5.193	422.331	
4	5.157		5.544	422.190	422.570	5.337	422.400	5.274	422.356	
5	5.132		5.715	422.119	422.690	5.413	422.432	5.350	422.380	
6	5.086		5.904	422.028	422.830	5.489	422.463	5.424	422.404	
7	5.020		6.114	421.919	422.990	5.565	422.495	5.495	422.429	
8	4.933		6.344	421.790	423.170	5.640	422.526	5.567	422.454	
9	4.826		6.593	421.643	423.369	5.716	422.557	5.639	422.480	
10	4.696		6.859	421.479	423.591	5.792	422.589	5.709	422.506	
11	4.539		7.139	421.298	423.834	5.868	422.620	5.778	422.535	
12	4.362		7.440	421.096	424.095	5.944	422.652	5.839	422.566	
13	4.163		7.759	420.876	424.377	6.019	422.683	5.901	422.596	
14	3.941		8.098	420.634	424.677	6.095	422.714	5.961	422.626	
15	3.700		8.456	420.370	424.996	6.171	422.746	6.019	422.656	
16	3.436		8.833	420.087	425.336	6.247	422.777	6.078	422.687	
17	3.152		9.230	419.788	425.699	6.323	422.809	6.135	422.712	
18	2.846		9.647	419.468	426.081	6.398	422.840	6.191	422.743	
19	2.521	10.084	10.084	419.126	426.481	6.474	422.871	6.247	422.774	
20	2.175	10.541	10.541	418.767	426.902	6.550	422.903	6.303	422.804	
21	1.809	11.018	11.018	418.389	427.345	6.626	422.934	6.358	422.835	
22	1.421	11.512	11.512	417.985	427.803	6.702	422.966	6.413	422.867	
23	1.010	12.022	12.022	417.555	428.273	6.777	422.997	6.467	422.894	
24	0.577	12.551	12.551	417.103	428.761	6.853	423.028	6.516	422.914	
25	0.124	13.101	13.101	416.634	429.271	6.929	423.060	6.564	422.932	
26	-0.347	13.673	13.673	416.146	429.803	7.005	423.091	6.613	422.957	
27	-0.841	14.262	14.262	415.636	430.352	7.081	423.123	6.663	422.975	
28	-1.355	14.871	14.871	415.104	430.917	7.156	423.154	6.711	422.994	
29	-1.889	15.501	15.501	414.552	431.502	7.232	423.185	6.758	423.011	
30	-2.444	16.151	16.151	413.979	432.104	7.308	423.217	6.806	423.027	
31	-3.021	16.820	16.820	413.387	432.727	7.384	423.248	6.853	423.041	
								6.900	423.057	

D20. Computer print-out of portable borehole deflectometer co-ordinates.



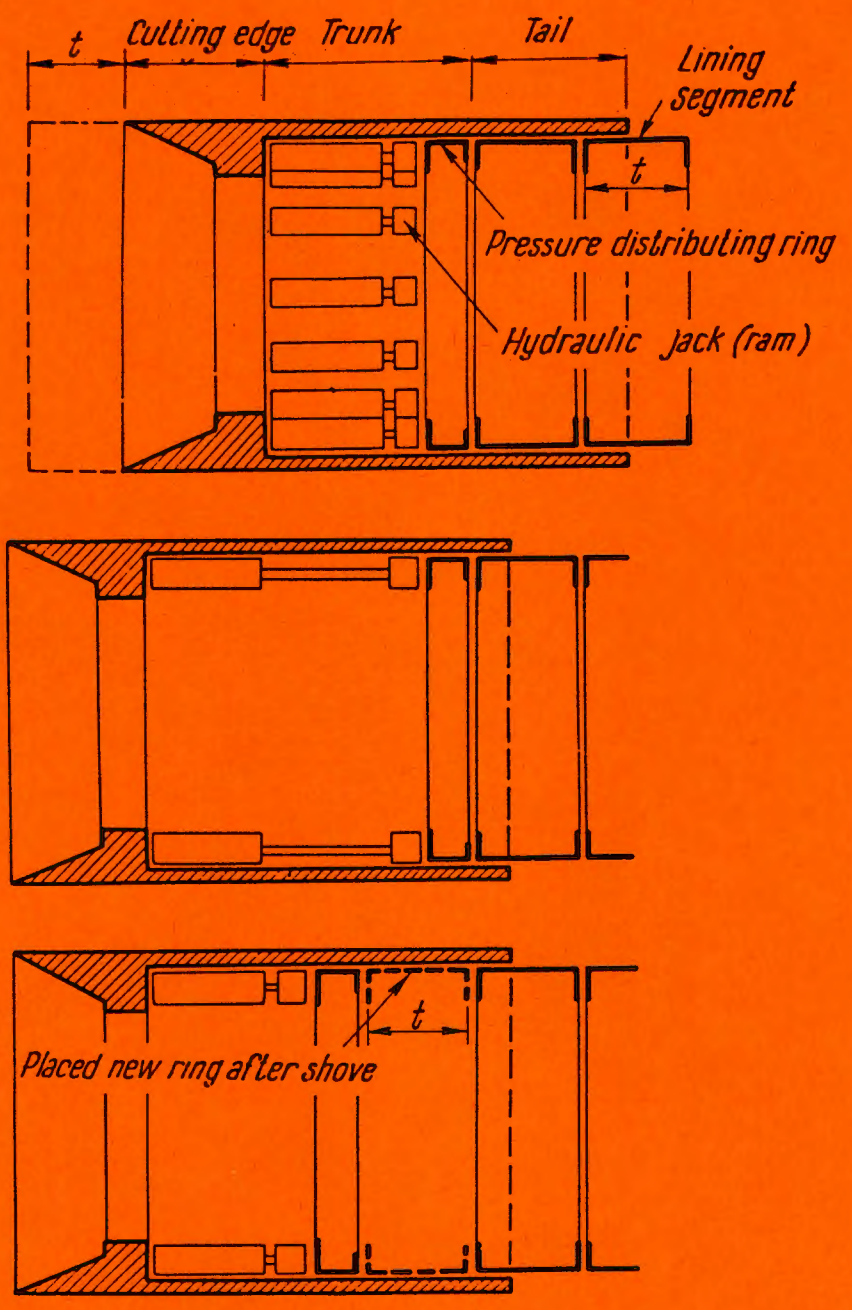
D21. Position of freeze tubes within the tunnel profile and determined with the aid of a portable borehole deflectometer.
(refer bib. no. 13)



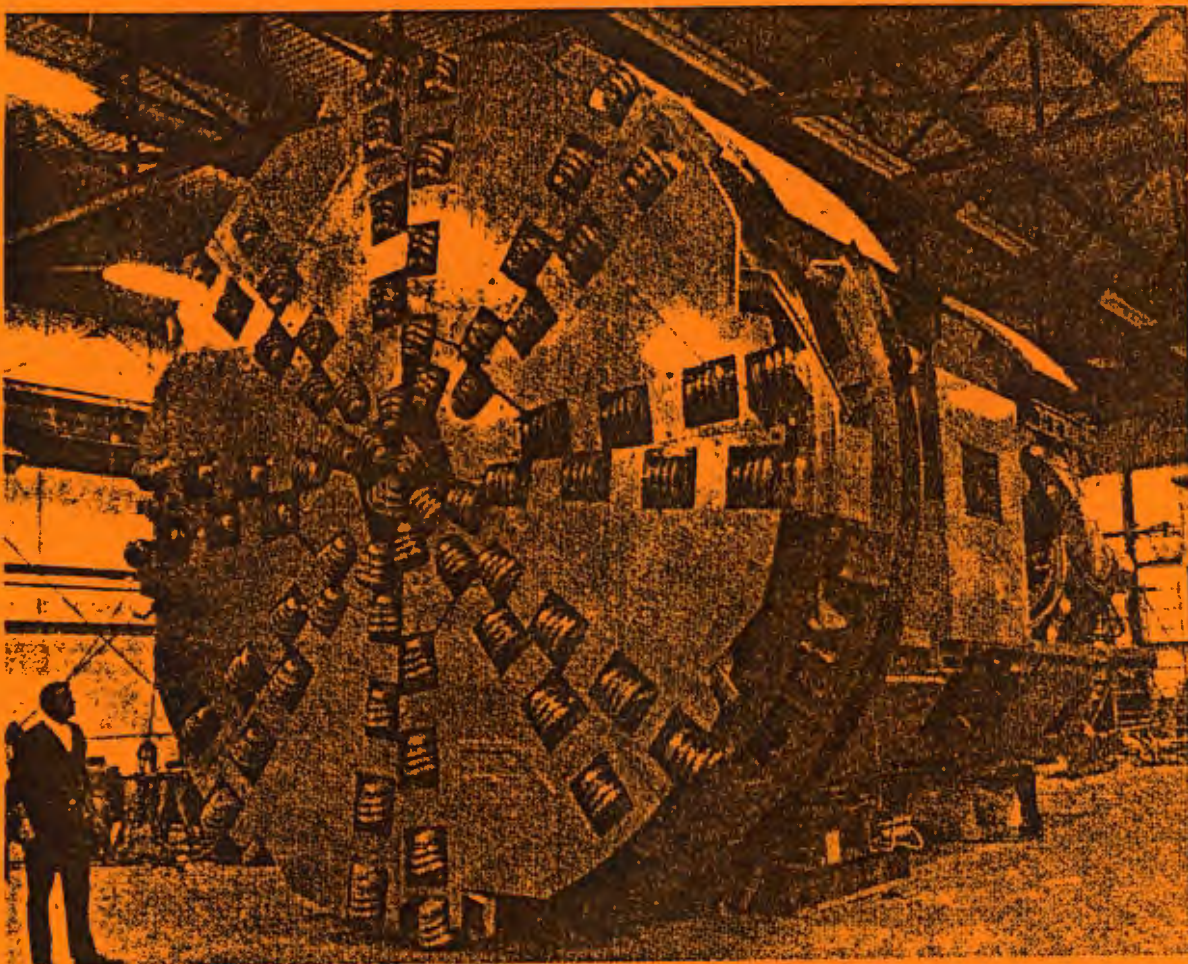
D22. Stages of shotcrete lining application.
 (reference NO. 1).



D23. Typical 100 mm diameter test core.
(refer bib. no. 19).



D24. Principle of a shield system.
 (refer bib. no. 24)



D25. Typical tunnelling machine - mole.
(refer bib no.24).

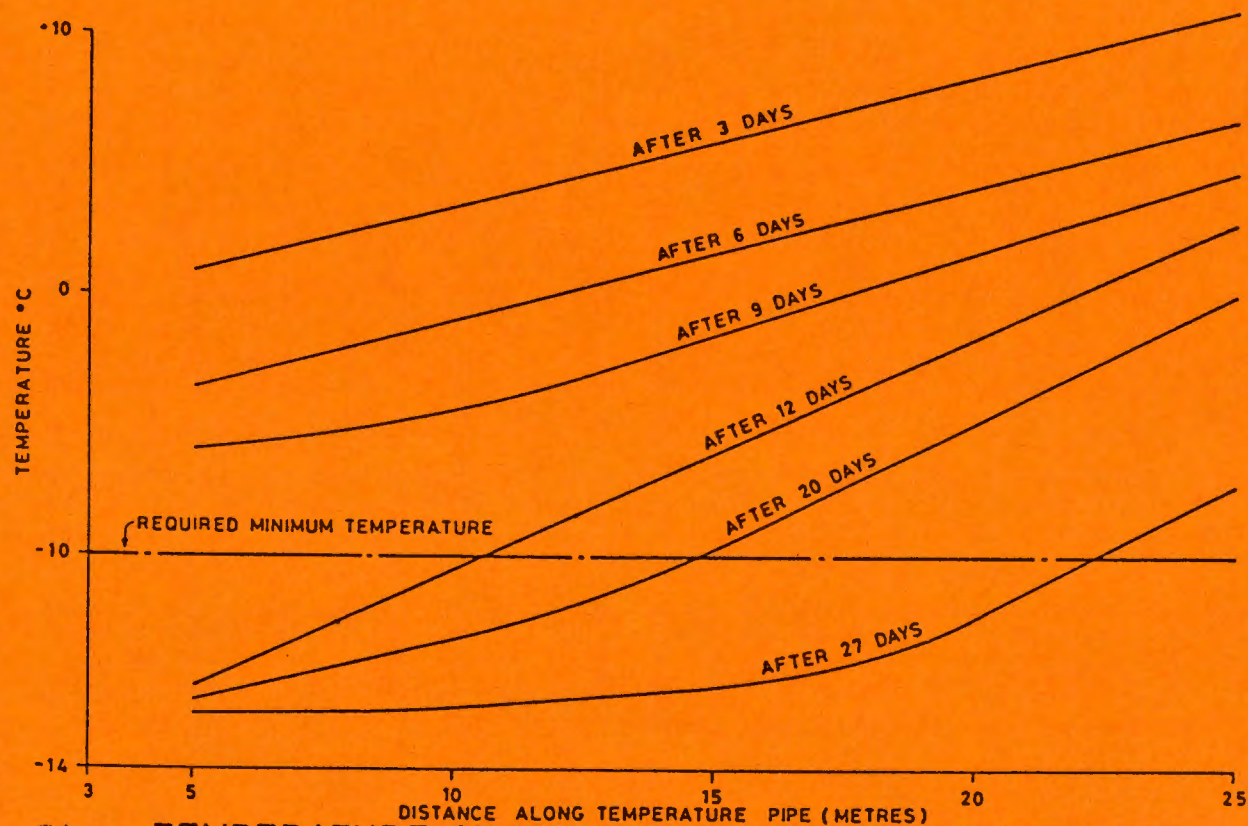


D26. Typical tunnelling machine - mole.
(refer bib no. 24).

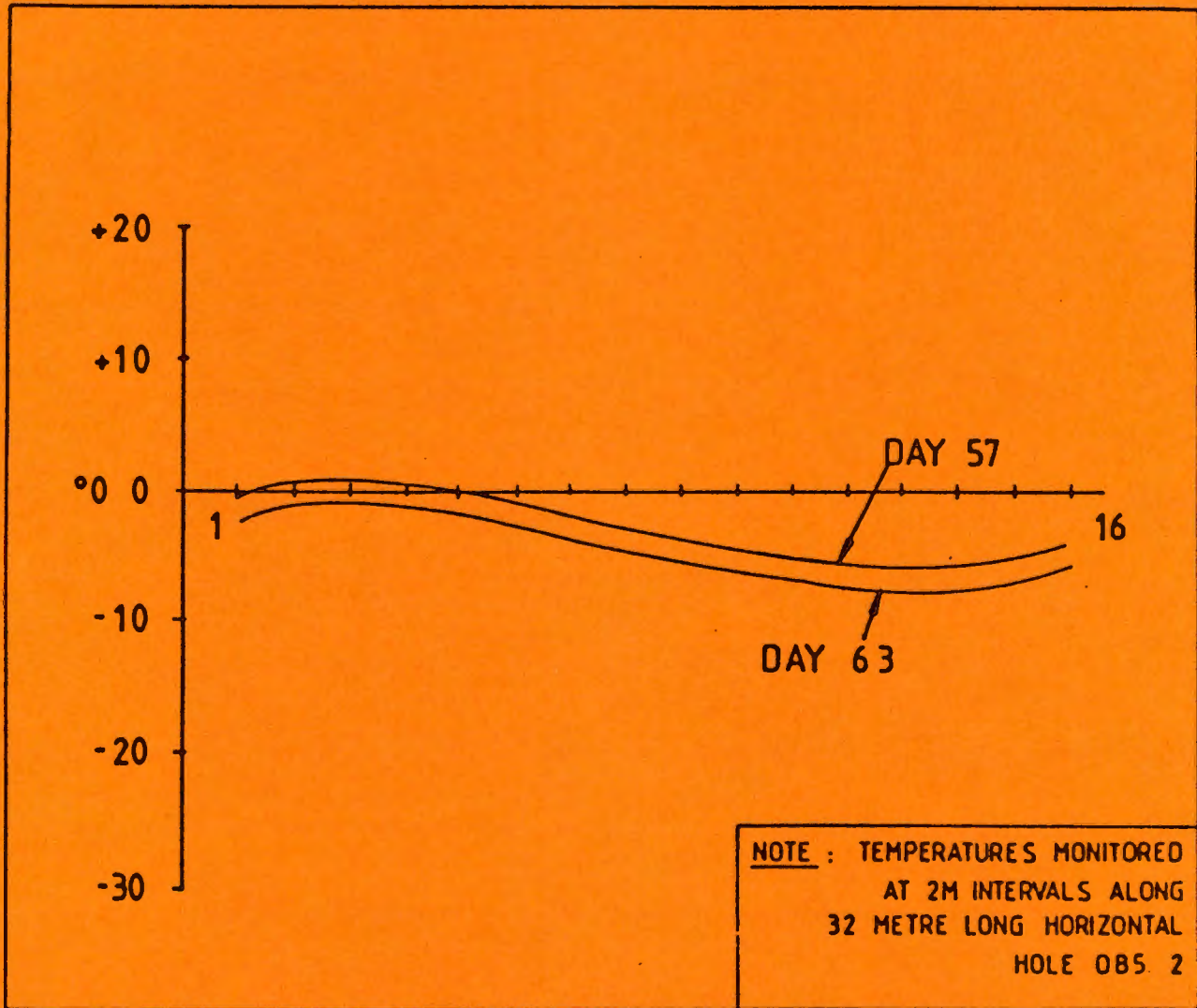


D27. Typical tunnelling machine - mole.
(refer bib no. 24)

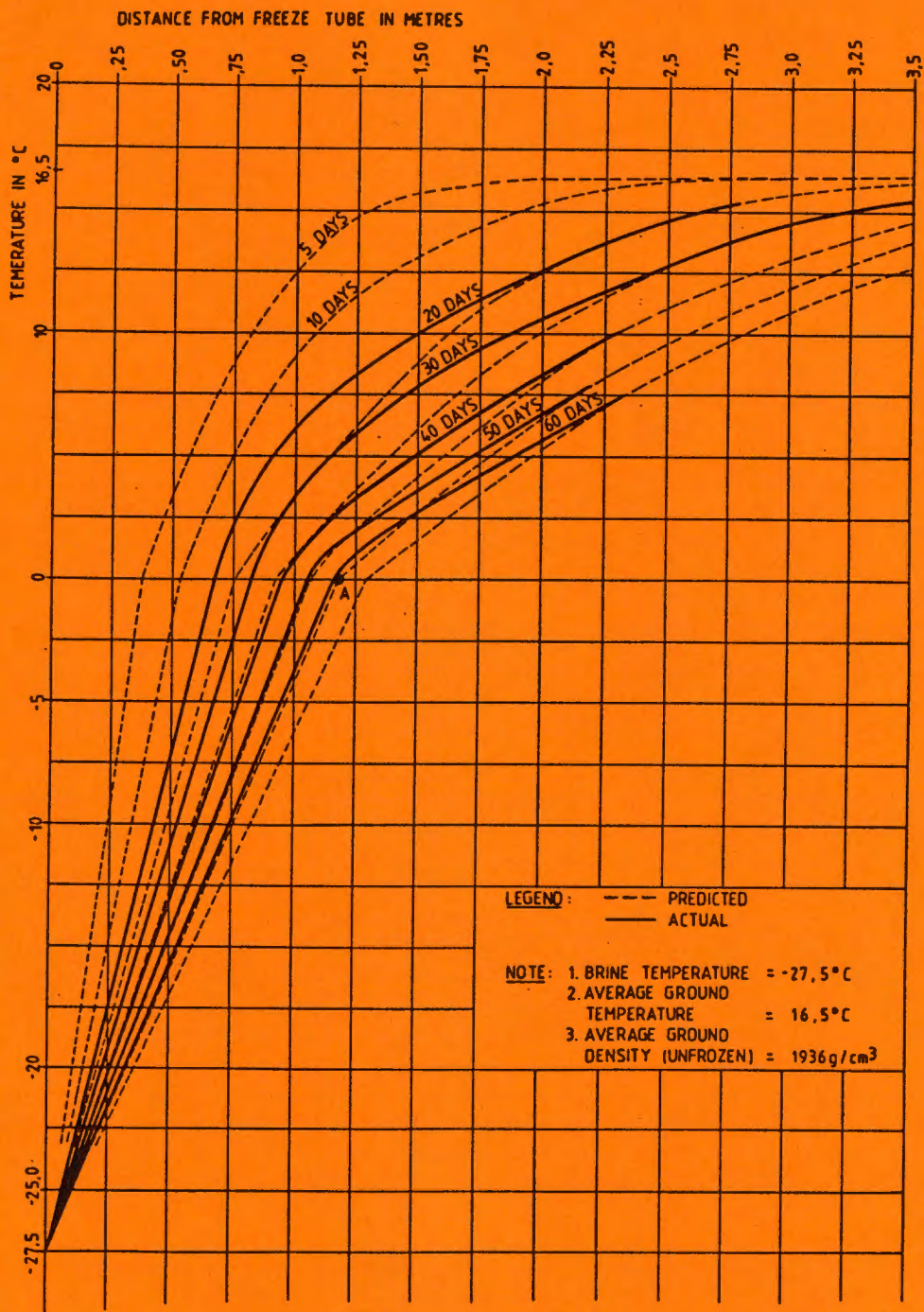
G R A P H S



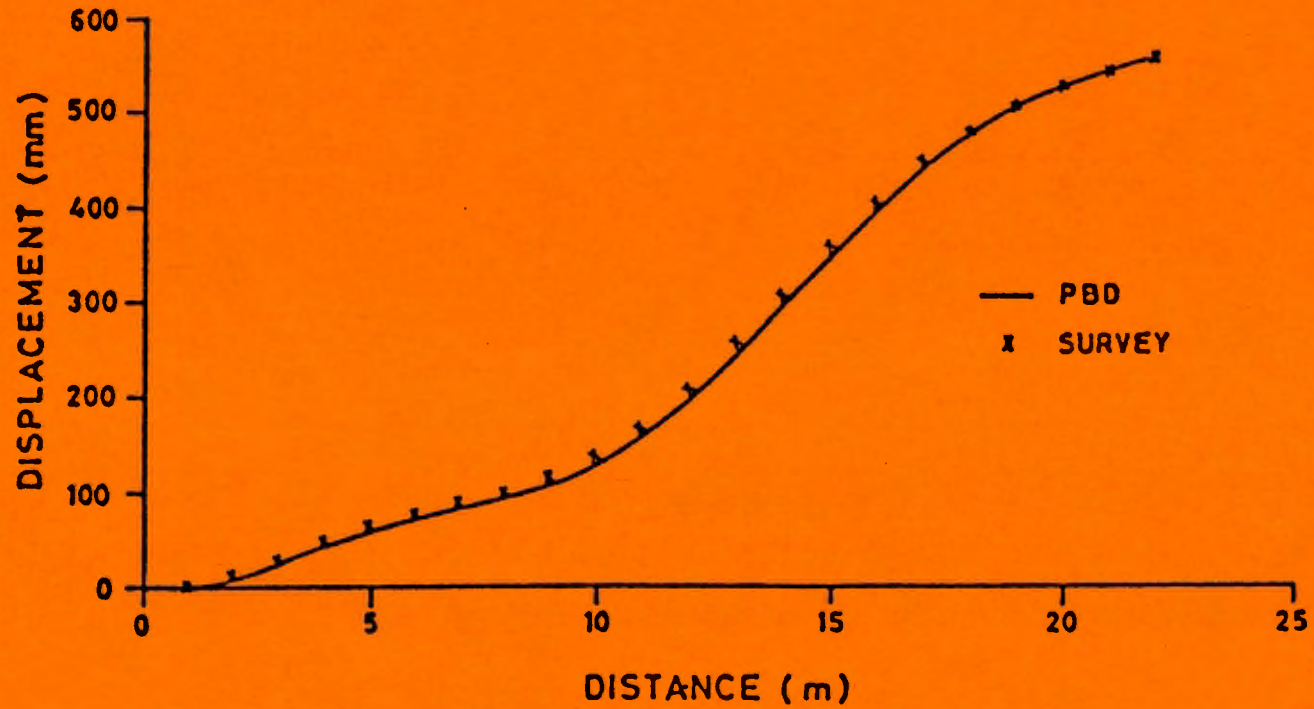
G1. TEMPERATURE GRAPH FOR DIVERGING FREEZE PIPES
(DU TOITSKLOOF TUNNEL HOLE No 11)



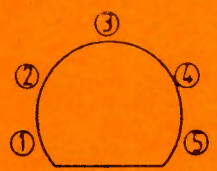
G2. Temperature readings for observation hole no. 2, within the frozen arch.



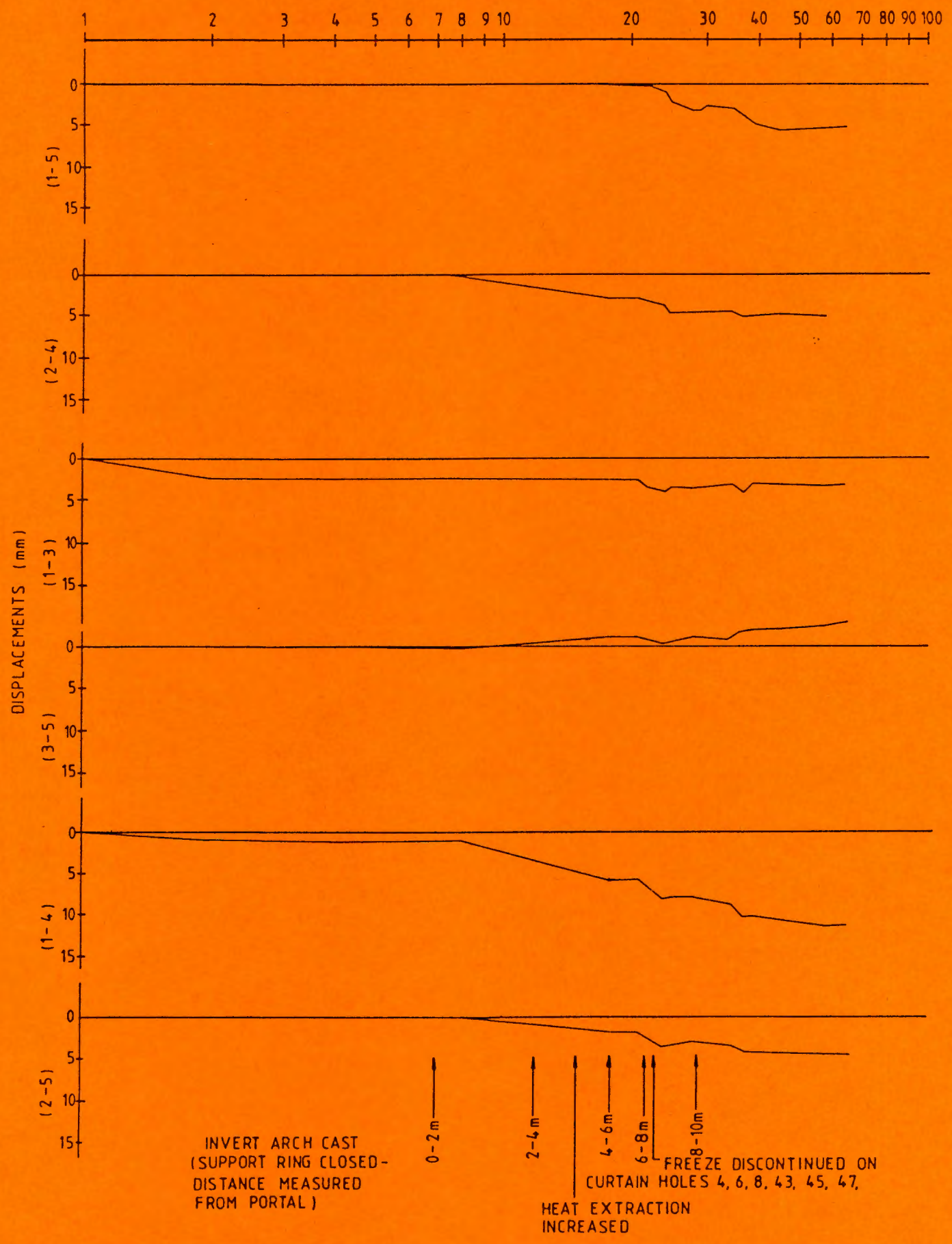
G3. Typical temperature reading obtained within the first 32-metre stage of freezing - freezing isotherm.



G4. Displacement determined by a portable borehole deflectometer.
(refer bib no 11.)

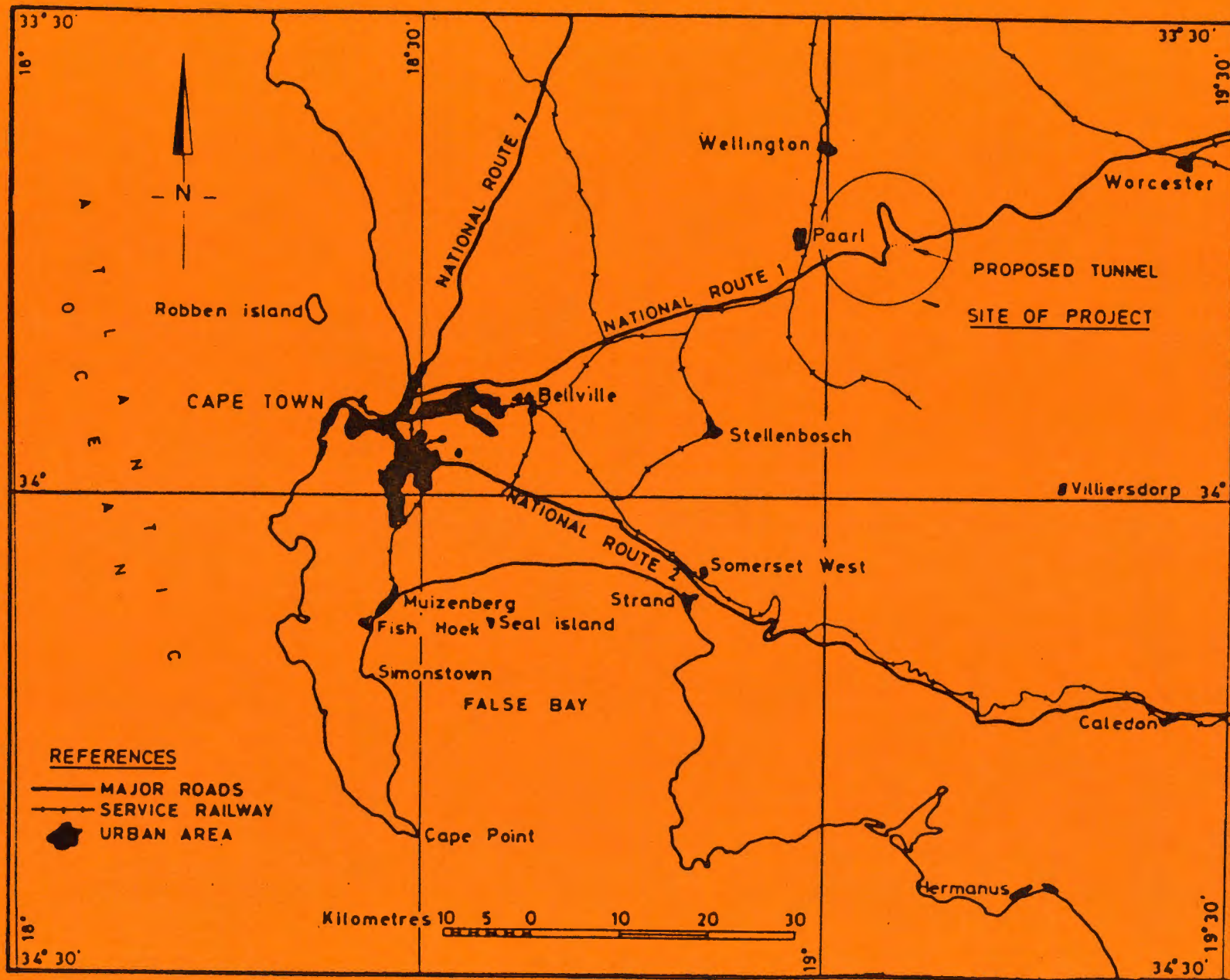


TIME (days)

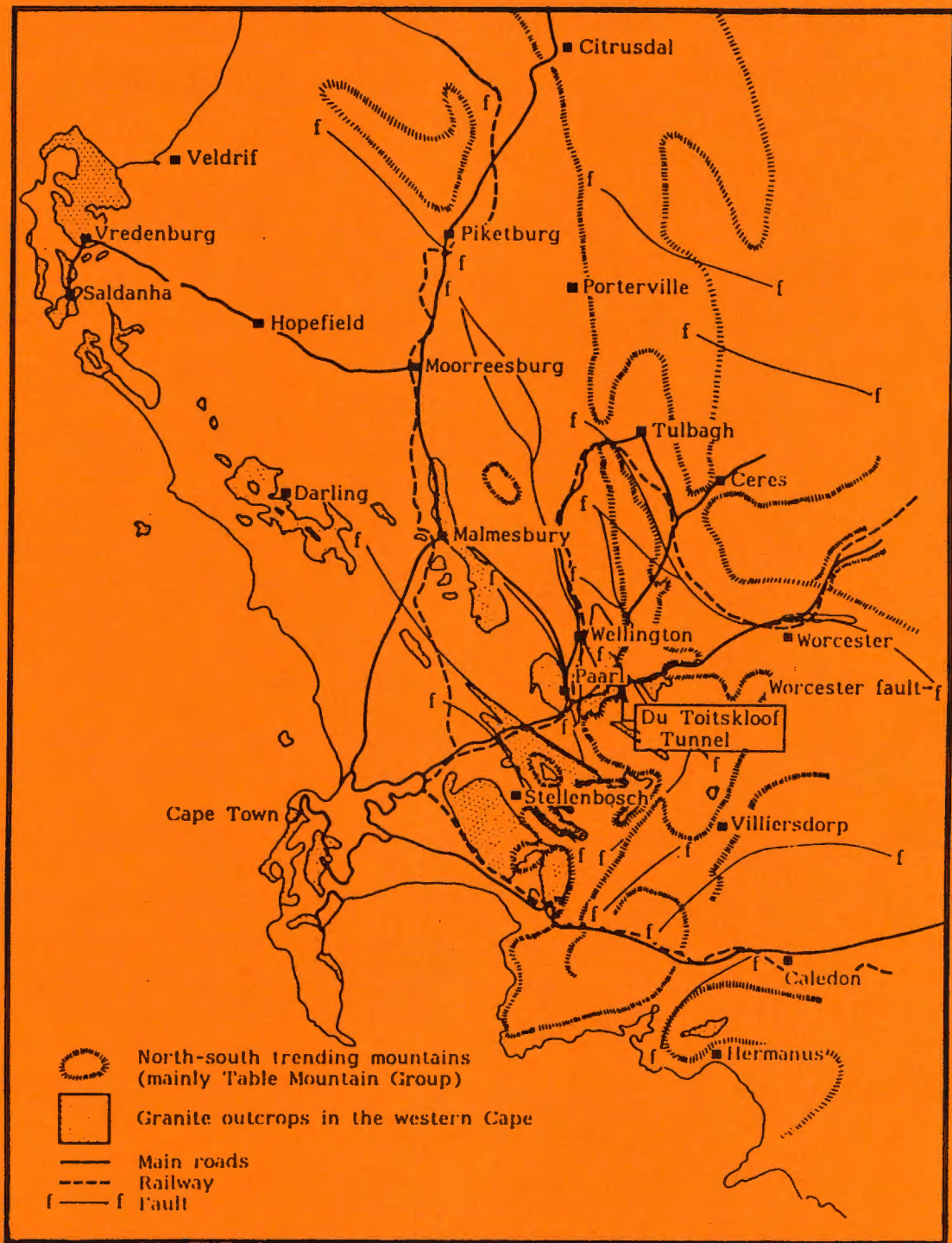


G5. Convergence of tunnel support - lining at 3,5 metres from the western portal, with respect to duration.

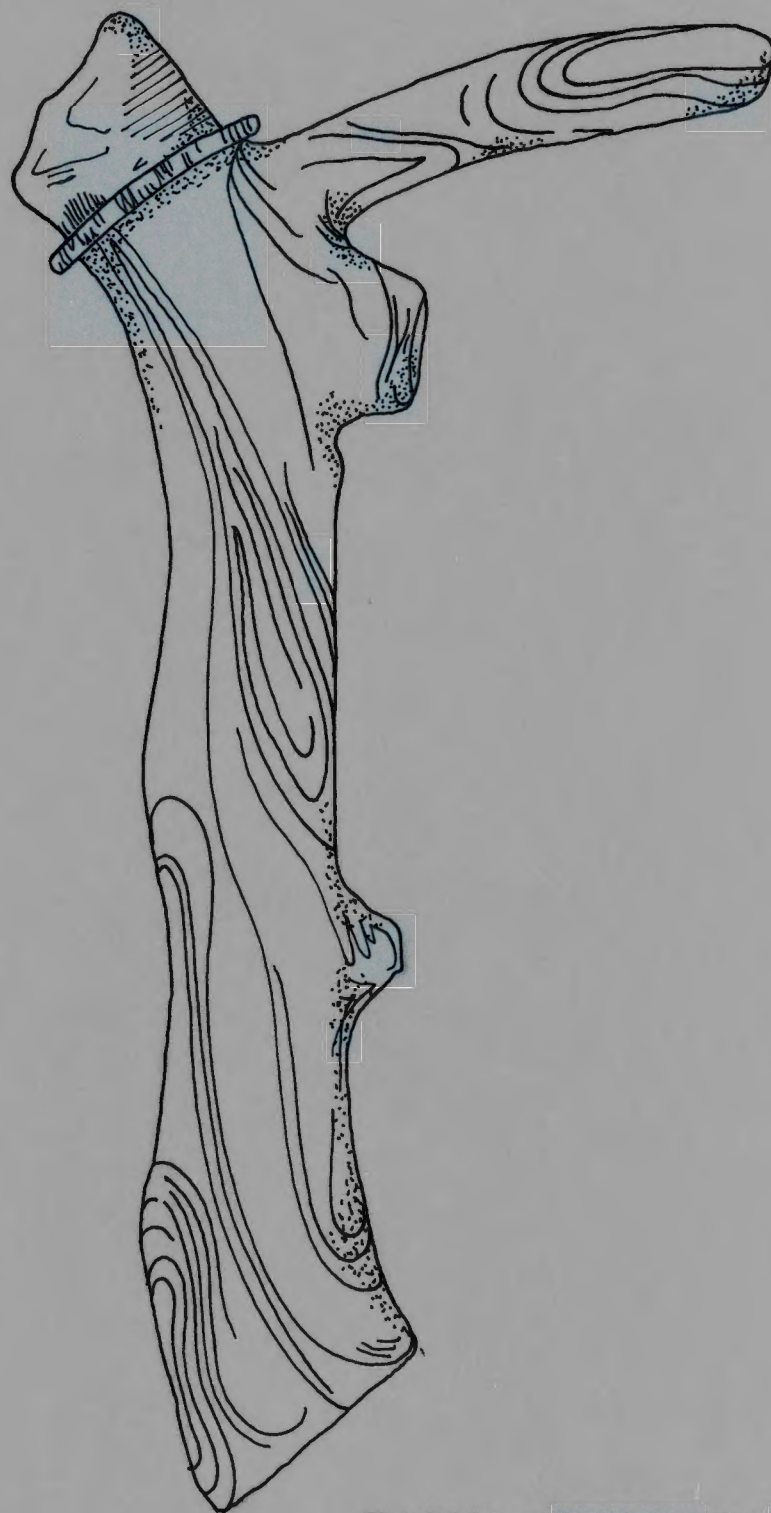
M A P S



M1. Location plan of the Du Toitskloof tunnel.
 (reference No. 1).

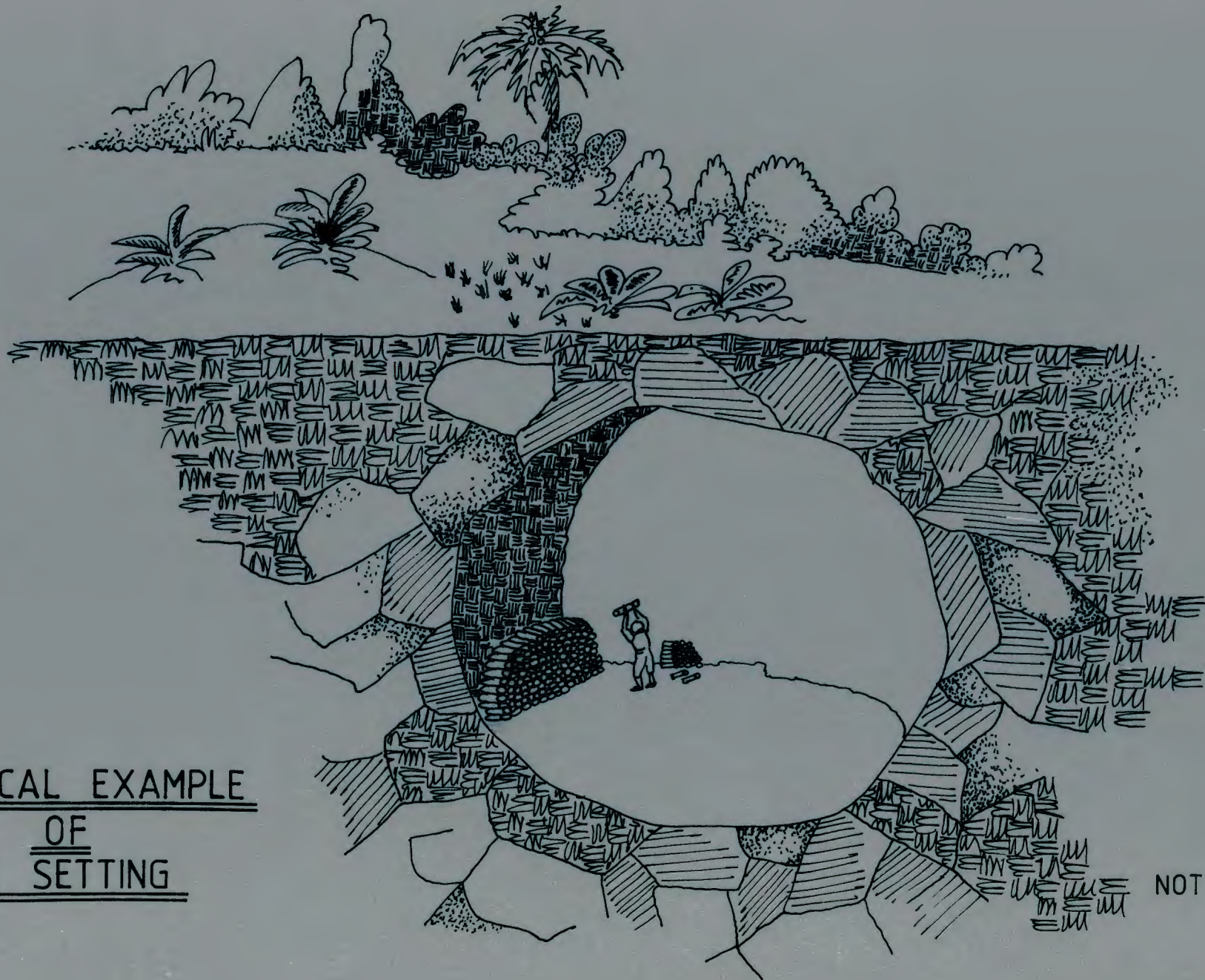


M2. Geological survey results showing granite outcrops within the tunnel location.
(reference no.1).



EARLY EXCAVATION TOOL -
DEER ANTLER PICK

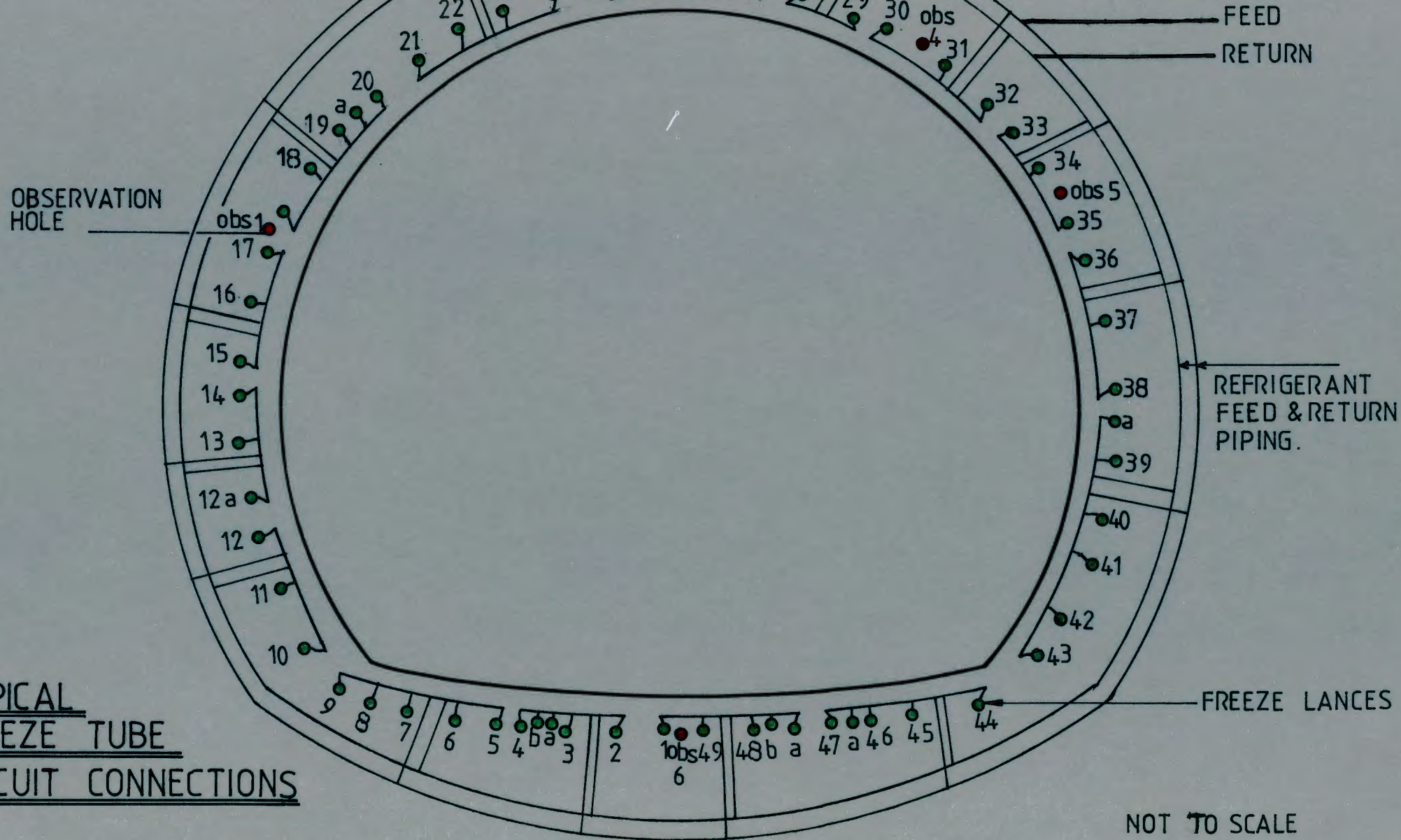
NOT TO SCALE



TYPICAL EXAMPLE
OF
FIRE SETTING

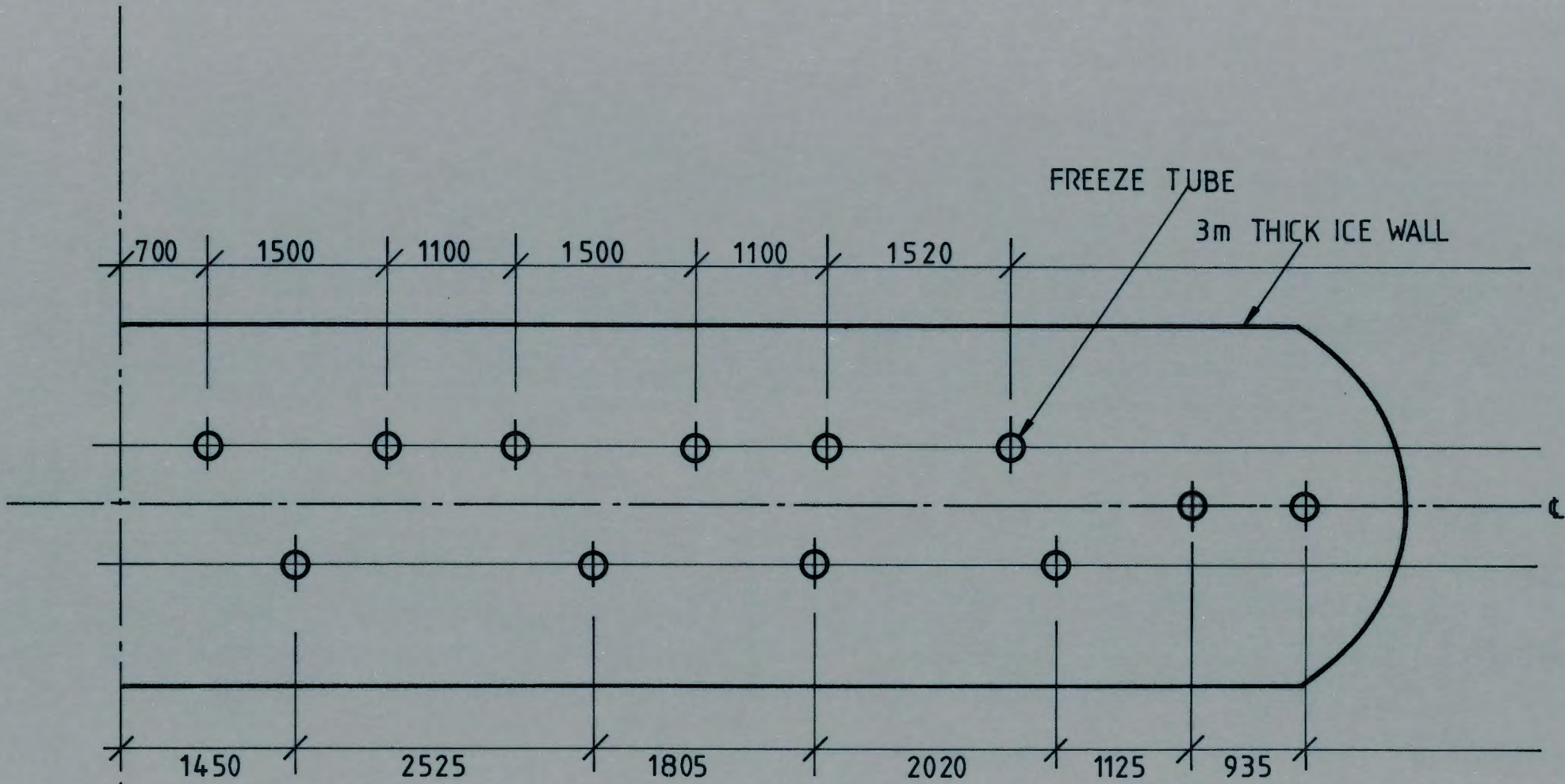
NOT TO SCALE

3



TYPICAL
FREEZE TUBE
CIRCUIT CONNECTIONS

NOT TO SCALE



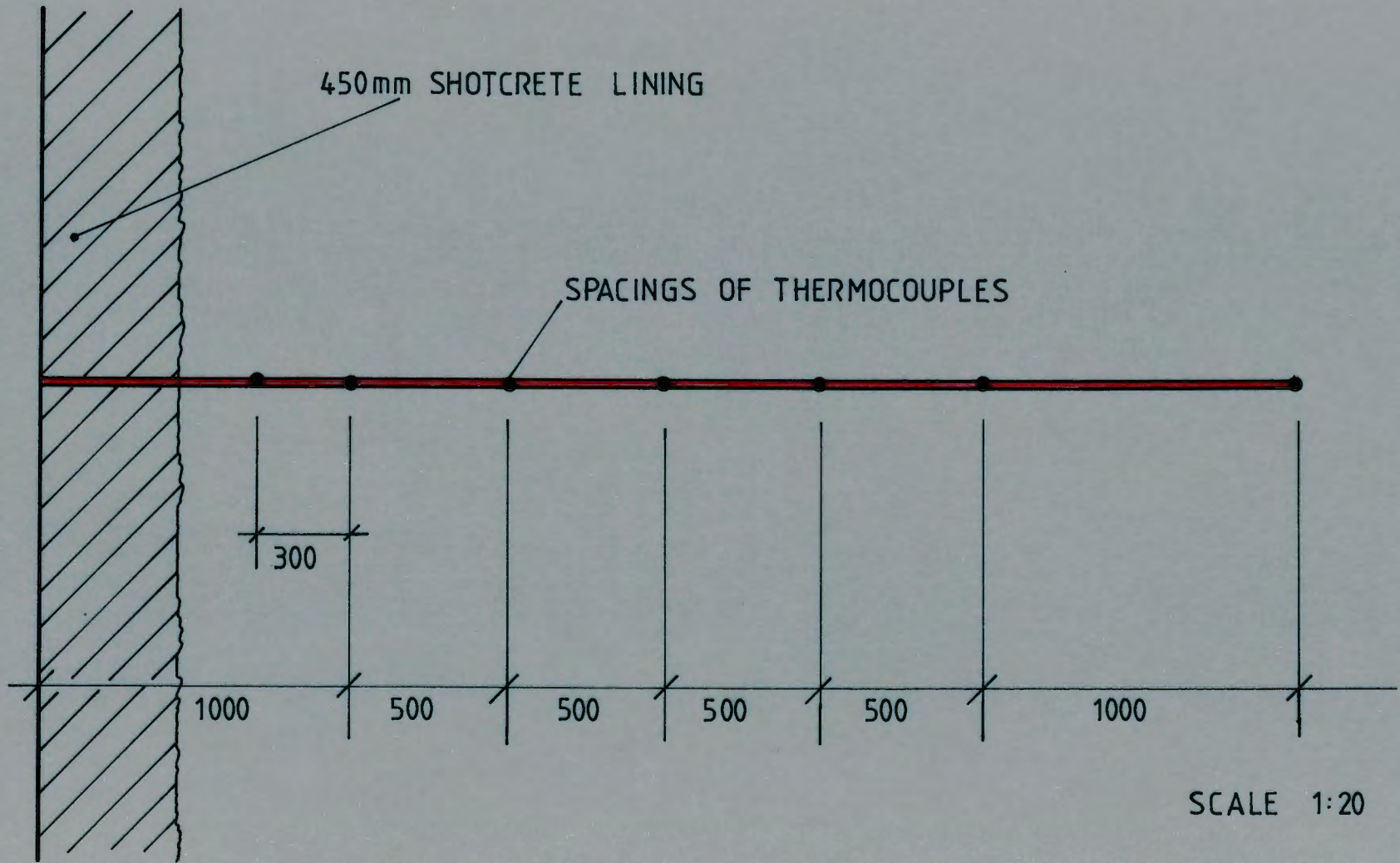
TUNNEL C

SCALE 1:50

VERTICAL BULKHEAD FREEZE TUBE LOCATION

WESTERN PORTAL SIDE
↓

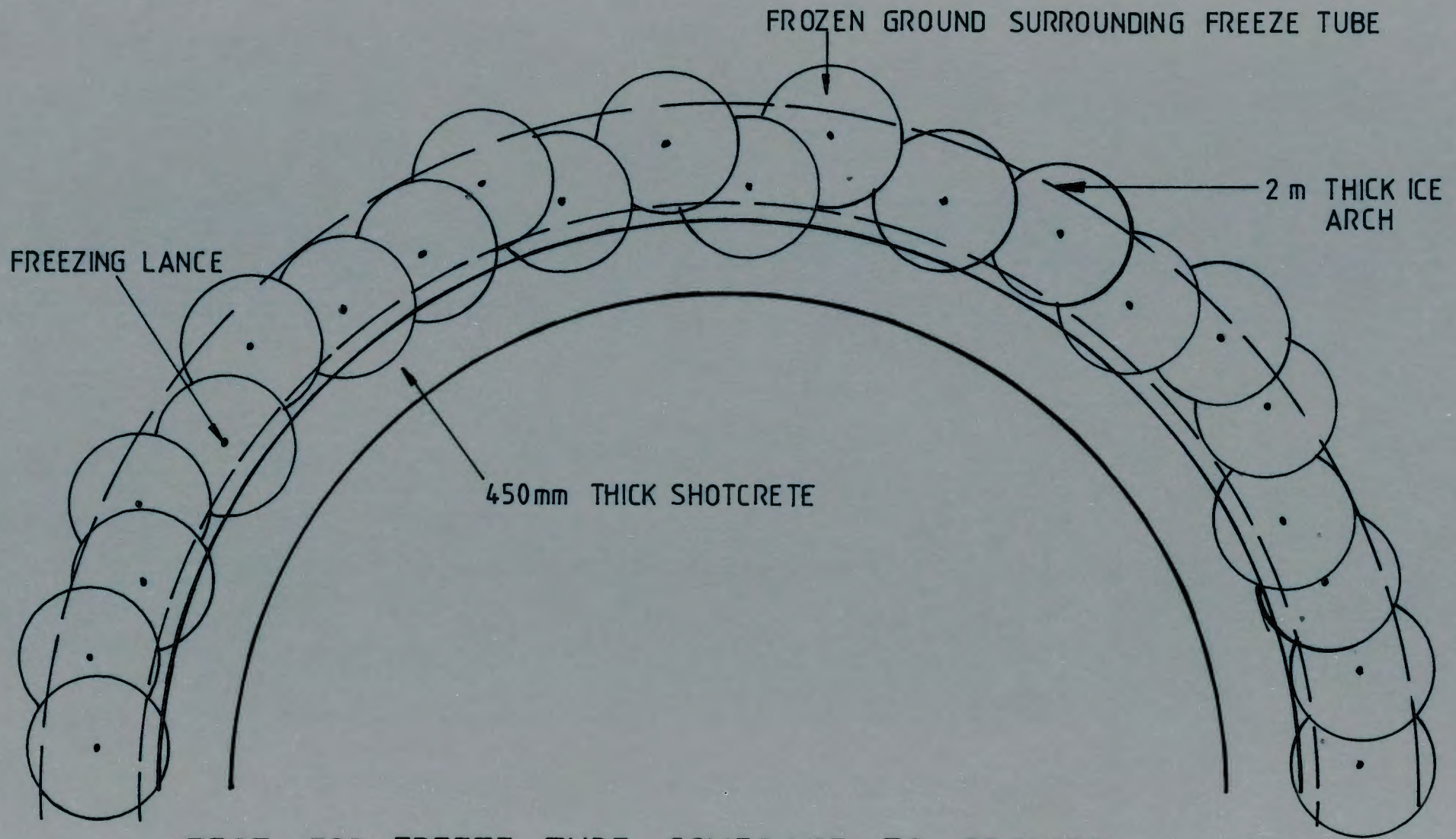
DR 4



TEMPERATURE MONITORING PROBES RADIAL



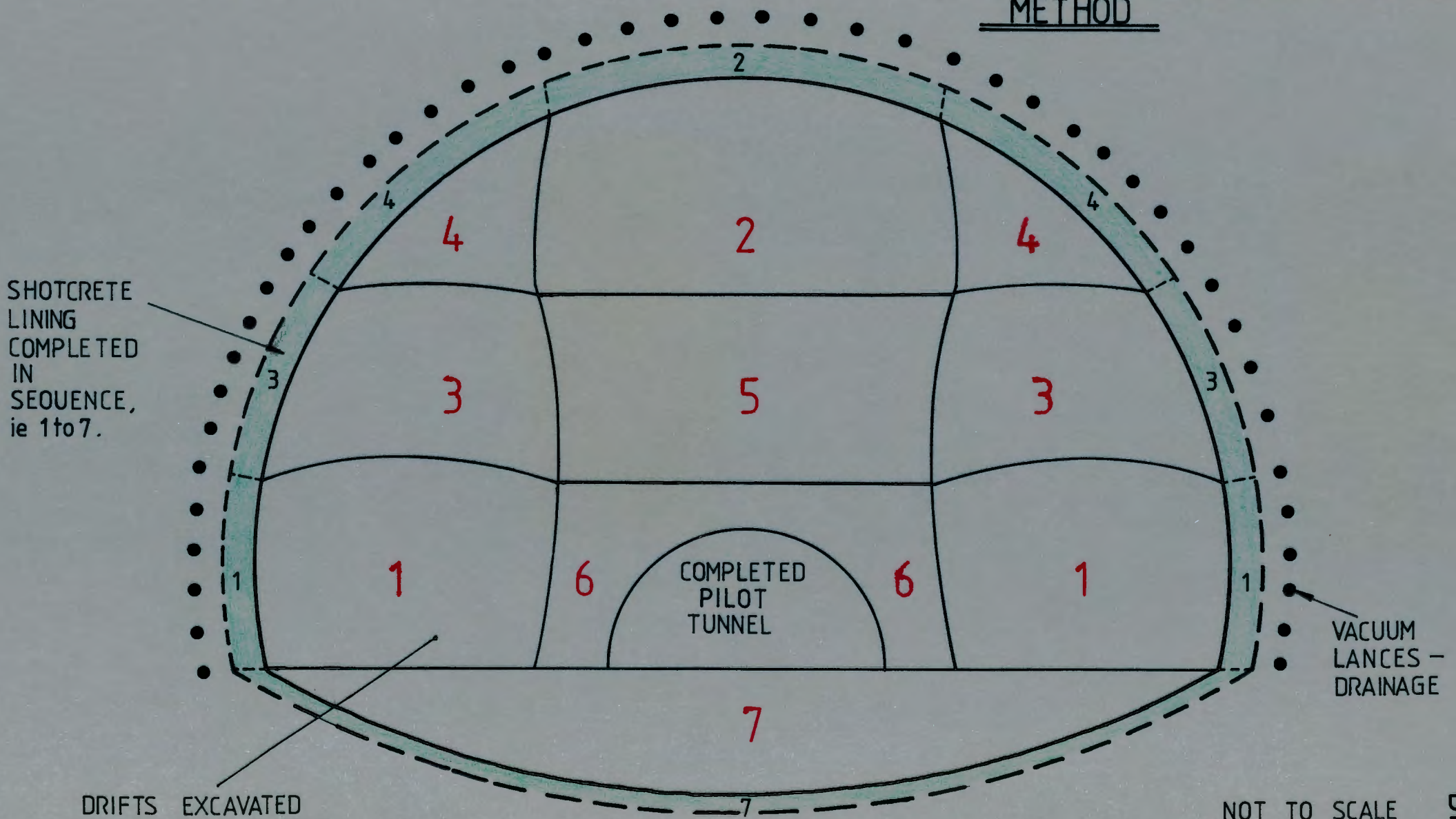
TEMPERATURE MONITORING PROBE FOR VERTICAL BULKHEAD



TEST FOR FREEZE TUBE COVERAGE TO PROVIDE
ADEQUATE ICE ARCH SUPPORT

NOT TO SCALE

MULTIPLE HEADING ATTACK
METHOD



SHOTCRETE LINING COMPLETED IN SEQUENCE, ie 1 to 7.

VACUUM LANCES - DRAINAGE

DRIFTS EXCAVATED IN SEQUENCE, ie 1 to 7.

NOT TO SCALE

DR8

P H O T O G R A P H S

Formation of sinkhole at the surface above pilot tunnel, sinkhole later to be filled with cement grout and selected fill - rock fill.

P1.



P2.

Mudflow within pilot tunnel, caused by the formation of a sinkhole, due to this situation of poor ground conditions, the technique of artificial ground freezing was implemented for the excavation of the pilot tunnel.



Western portal : showing temporary tied - back retaining wall, taken September 1977, retaining wall formed of railway sleepers, 'I' beams and tied back with ground anchors.

P3.



P4.

The near completion of the tied - back piled retaining wall including wing walls to portal, provides protection to western portal working area, and will be the site for the tunnel's terminal buildings.



Construction at the western portal, of the piled tied-back retaining wall, including the construction of the retaining wall's wing walls. Temporary retaining wall in background.

P5.



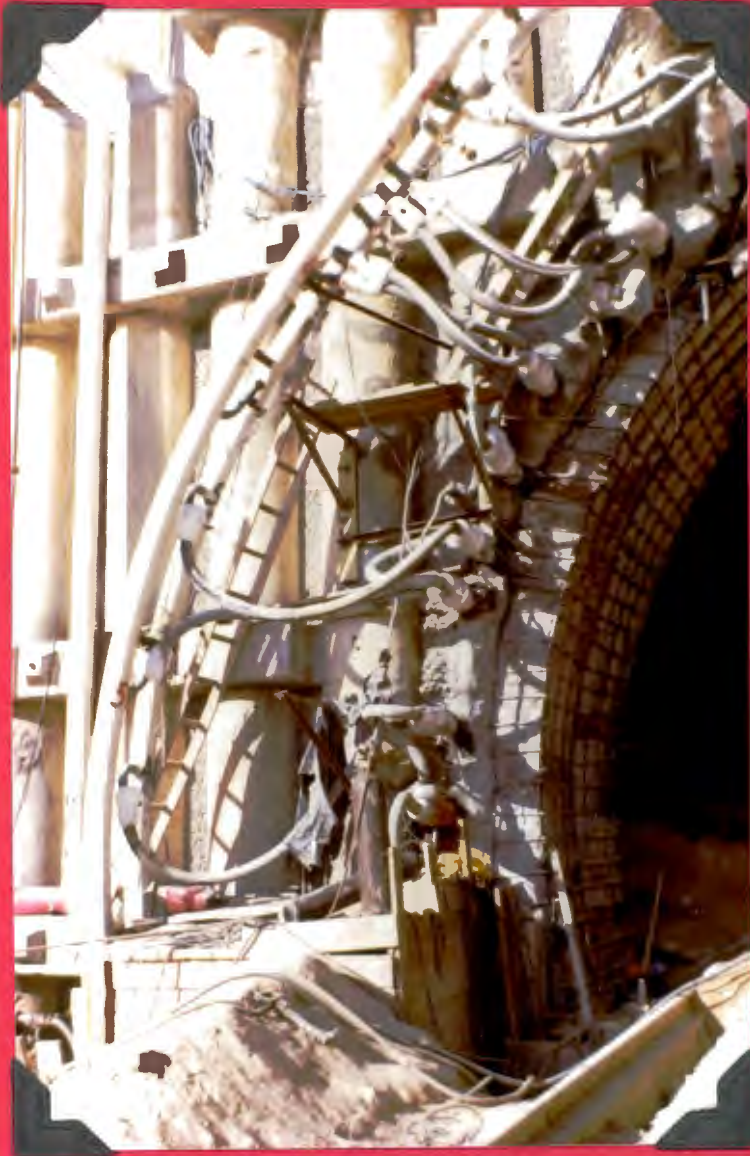
P6.

Eastern portal : formation of portal, pilot tunnel situated in background of portal, this is the formation of the second main tunnel at the eastern 'portal' - Worcester side.



Typical placement of horizontal freeze tubes at western portal, tubes connected to 'feed and return' main circulation pipes from freezing plant, these freeze tubes forming the 2m thick frozen arch, rubber hosing connecting freeze tubes to cir. pipe

P7.



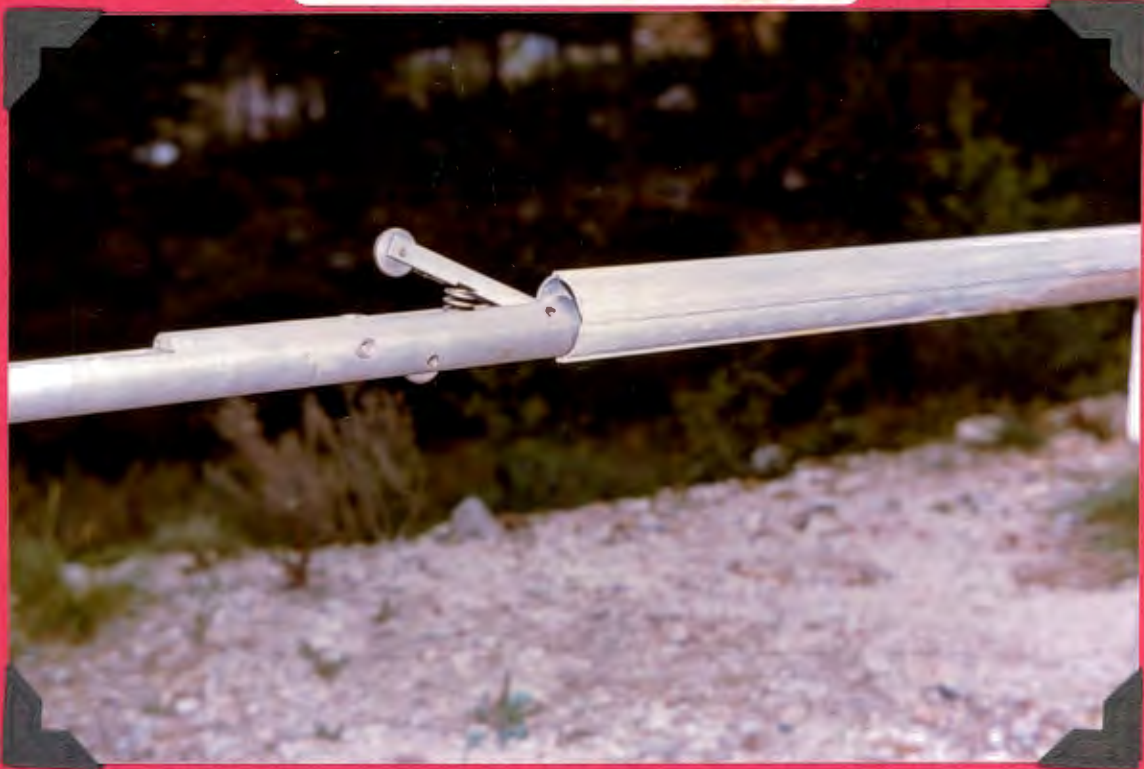
P8.

Portable borehole deflectometer, used to measure the accuracy of the drilled freeze holes, so as to provide for adequate formation of frozen tunnel arch and vertical bulkheads.



Application of portable borehole deflectometer into a test tube to check for the instrument's accuracy before applying to the actual drilled freeze holes within the tunnel.

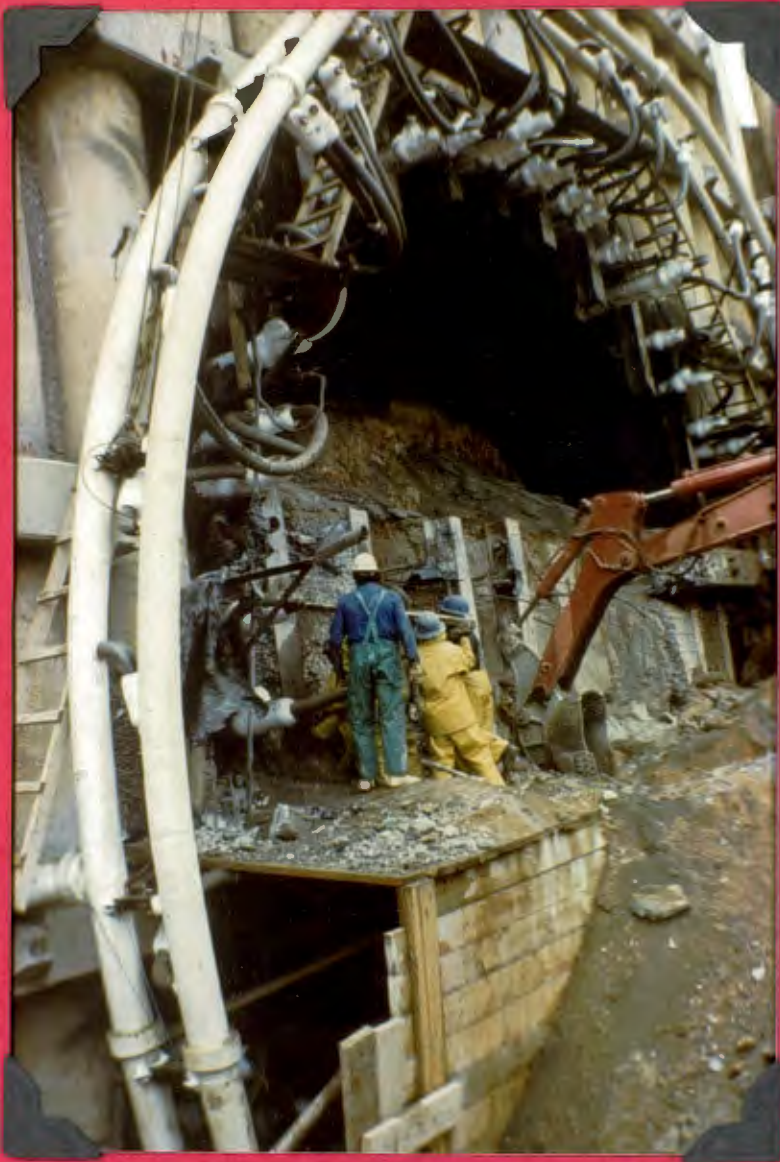
P9.



P10.

Freeze lances within tunnel arch form frozen arch which provides adequate support, this allows for the demolition of the western portal retaining wall within the tunnel cross-section, and excavation of decomposed granite begins.





P11.

Horizontal freeze tubes are shown, placed within tunnel profile forming frozen 2m thick arch, also showing excavation and demolition of part of the western portal retaining wall with the use of a back-actor.

P12.

'Roadheader' entering tunnel to excavate decomposed granite within frozen, good quality finish to reinforced shotcrete support lining as seen on tunnel walls and crown.



Application of support to frozen ground ; applying first layer of mesh with the use of stools - drilled into frozen decomposed granite.

P13.



P14.

Application of reinforced shotcrete support - lining formed of mesh, steel arches etc, and frozen vertical bulkhead in background.



Formation of drifts 1 and 2 of the multiple heading attack method within the main tunnel cross-section, final main tunnel's profile is marked in black dotted line.

P15.



P16.

View of a typical drift within the multiple heading attack method, steel arches are also seen providing support, and overhead ventilation - extraction pipe are also present.



Entrance to a drift (no. 1)
within the second main Soft
Ground tunnel (north tunnel)

P17.



P18.

Drainage of the decomposed
granite area with the use of
vacuum lances, undertaken
within the drifts of the
multiple heading attack
method; drainage by suction
of water from surrounding
ground.



Application of reinforced shotcrete support to perimeter of drifts within multiple heading attack method. Support consists of mesh, steel arches and shotcrete.

P19.



P20.

Formation of portal to the soft ground tunnel section, portal surrounded by piled tied - back retaining wall. Extraction fan - pipes placed on either side of tunnel 'floor'.



Erection of mobile shutter, which will form final tunnel lining (support), and moves within tunnel on tracks. Shutter premanufactured off site - Concor LTD.

P21.



P22.

Alignment of mobile tunnel shutter, mechanics of shutter is operated on hydraulic jacks, after the tunnel shutter has been aligned concrete is pumped in between steel shutter and plastic lined tunnel walls.



Application of final tunnel lining - support, formed with the use of the mobile shutter, after which a pre-cast suspended ceiling is attached, which conceals certain services, ceiling's main function = ventilation.

P23.



APPENDICES

TABLE OF CONTENTS

APPENDIX 1 : TABLE RESULTS OF CHEMICAL ANALYSIS OF
WATER SAMPLES IN THE WESTERN PORTAL
ZONE

APPENDIX 2 : RESULTS OF MINERALOGICAL AND CHEMICAL
TESTS IN SAMPLES OF DECOMPOSED AND
WEATHERED GRANITE ZONES

**CHEMICAL ANALYSES ON WATER SAMPLES IN THE
WESTERN ZONE OF THE PILOT BORE**

Stake Value	3052669,0	305295,5	305314,0
Sample No.	6	10	11
Source of Water	Weep hole No. 4A	Freeze holes 1st Frozen Section	Freeze hole 11 2nd Frozen Section
pH	6,5	6,2	6,2
Dionic conductivity at 25°C (mS/m)	5,28	4,5	5,0
Total dissolved solids at 180°C (mg/l)	32	27,0	10,6
Chloride (Cl) (mg/l)	10,6	11,5	
Total Hardness (CaCO ₃) (mg/l)	20	7,0	1,0
Alkalinity (CaCO ₃) (mg/l)	10,	Nil	1,0
Calcium (Ca) (mg/l)	4	Nil	
Magnesium (Mg) (mg/l)	2,4	1,7	
Sulphate (SO ₄) (mg/l)	1	1	Nil
Calcium Hardness (mg/l)			1,0
Langlier Index	-3,32	-5,5	-5,6

T1. Appendix 1 : Results of chemical analysis of water samples in the western portal.

X RAY DIFFRACTION AND PETROGRAPHIC ANALYSES OF THE DECOMPOSED
AND WEATHERED GRANITE AT THE WESTERN PORTAL.

DESCRIPTION	DECOMP. GRANITE (HOMOGEN)	DECOMP. BIOTITIC	DECOMP. GRANITE	FAULT GOUGE IN THE DECOMP. GRANITE	DECOMP FINE GRAINED GRANITE	BLACK CLAY GOUGE ON JOINT PLANES	DECOMP. GRANITE	WEATHERED GRANITE	GREY PORPHYRITIC UN- WEATHERED GRANITE (G 2-2)
S.V.	305 249	305 270	305 298	305 324	305 324	305 326	305 327	305 344	307 895
MINERALOGICAL COMPOSITION (%.)									
QUARTZ	33	47	43	35	35	30	30	30	30
MICROCLINE	28	10	20	15		20	23		
ORTHOCLASE	22	10	20	20	35	20	27	20	39
MICROPERTHITE								45	20
PLAGIOCLASE								1	5
MICA	5	3	5	10	7	5	5	1,5	3
MONTMORILLONITE		30							
KAOLINITE	12		12	20	23	25	15		
CHLORITE								2	2
						HORNBLLENDE SPIENE		0,5	1
CHEMICAL COMPOSITION (%.)									
S ₂ O ₂	64,87	70,63	70,11					74,60	71,77
T ₁ O ₂	0,35	0,32	0,34					0,34	0,32
Al ₂ O ₃	15,81	12,08	14,71					12,84	14,07
Fe ₂ O ₃	1,48	3,24	1,74					1,26	0,29
FeO	0,87	< 0,1	0,59					1,10	1,88
MnO	0,02	0,58	0,03					0,06	0,002
HgO	0,34	1,18	0,07					0,79	0,47
CaO	0,24	0,48	0,03					0,49	1,17
H ₂ O	0,07	0,07	0,01					3,11	2,77
K ₂ O	3,96	4,92	4,99					4,55	4,77
P ₂ O ₅	0,08	0,04	0,09						0,11
H ₂ O [*]	6,06	4,83	3,56					1,12	0,92
H ₂ O ⁻	4,06	1,18	0,65					0,36	0,29
Ba	0,043	0,069	0,048						0,036

T2. Appendix 2 : Mineralogical and chemical tests on samples of decomposed and weathered granite zones.