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1. Abstract

Twin 8km long, 7.7m internal diameter (ID) tunnels carry the railway between Zealand and the island of Sprogø. These tunnels were driven through glacial tills and the underlying Upper Palaeocene Marl by earth pressure balance TBMs and generally lined with segmental rings of precast reinforced concrete. Twenty nine 4.5m ID cross passages were bored and lined between the twin tunnels to act as equipment rooms and as escape passages.

The paper describes the geotechnical conditions anticipated and encountered, and the development of the design from the tender stage onwards. It was realised that the ground conditions might be difficult so provision was made for geotechnical investigations and ground treatment through the main tunnel linings at each passage location.

The construction method and design details were strongly influenced by the need to ensure safety during construction, and a range of measures were made available and adopted as necessary, including ground treatment and freezing, local and overall dewatering, use of a pilot tunnel in the tills, piecemeal excavation and primary support of the large collars at the junctions with the main tunnels, safety doors and temporary props in the main tunnels.

Details are given upon the rates of progress achieved in the successful and safe construction of the cross passages.

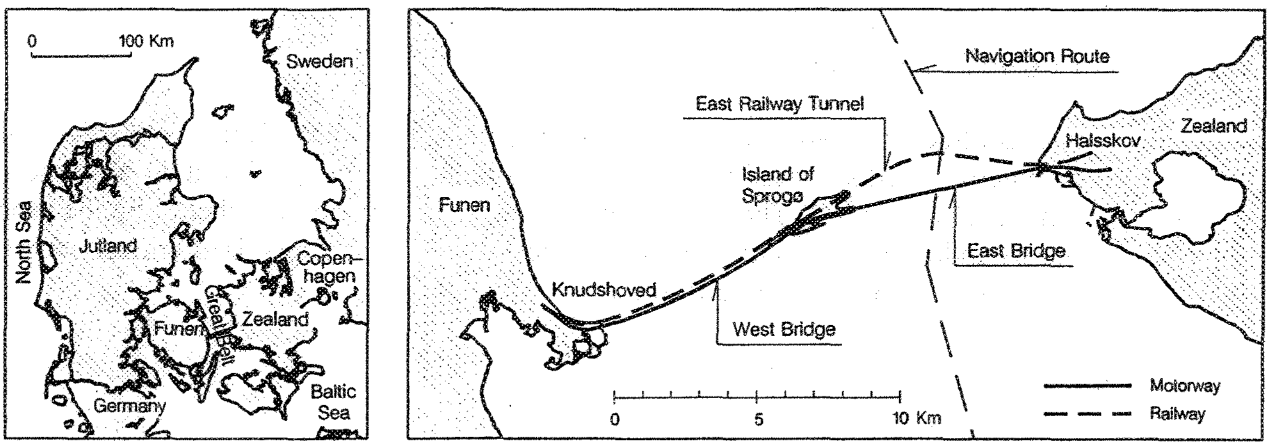

Figure 1 - Project Layout

2. Introduction

The 18km wide Danish Great Belt channel, or Storebælt, separates the islands of Funen and Zealand, where the capital Copenhagen is located. The small island of Sprogø is positioned centrally in the Storebælt. Sprogø is connected to Funen to the west by a low level combined rail/road bridge, and to Zealand by the eastern railway tunnel and one of the longest single span suspension bridges in the world, currently under construction (Fig. 1). The overall cost of the Storebælt project is £2.4 billion, of which the tunnel project represents £0.6 billion. The tunnel is predicted to carry 26,000 train passengers/day, with a total rail crossing time of seven minutes.

The 8km long rail tunnel comprises two 7.7m ID bored tunnels and a total of 500m of cut and cover construction. The bored tunnels were constructed using four earth pressure balance TBMs supplied by James Howden and Company. The spacing between the tunnels is generally 17m (25m between tunnel axes). Cross passages connect the two tunnels at approximately 250m intervals providing emergency escape routes for passenger evacuation, access routes for rescue and maintenance personnel, and housing for electrical and mechanical equipment.

There are thirty one cross passages in total: twenty nine were bored and two were constructed at each end of the tunnel as part of the cut and cover works. This paper describes the design and construction of the bored cross passages.

3. Organisation

After more than 125 years of debate, the Danish State finally agreed to the fixed link. The client company A/S Storebæltforbindelsen (SBF) was established in January 1987 and registered as a limited company with the Danish State as sole shareholder. The purpose of the company is to plan, design, implement and operate the fixed link.

The design of the tunnel was carried out by a joint venture between COWI of Denmark and Mott MacDonald of the UK.

The tunnel project was undertaken as a remeasurement contract awarded to MT Group in November 1988. This was an international joint venture comprising: Monberg & Thorsen of Denmark, Dyckerhoff & Widmann of Germany, Campenon Bernard and Sogea both of France, and Kiewit Construction of the USA.

3.1 Quality Assurance

The contractor established a Quality Programme in accordance with ISO 9001. Method statements, works procedures and inspection plans were produced in advance of each main construction activity. The contractor carried out quality control checks by the production teams and by independent quality control engineers. SBF also supervised critical activities especially the excavation of the cross passages.

4. Geology

The Storebælt is a glacial erosion channel. The water depth increases gradually from both sides and more rapidly in the middle to a depth of 55m (Fig. 2). The water depth in the Central Channel prevented a direct alignment for the tunnel crossing. The northern diversion resulted in a shallower depth of water at the nadir and satisfied the tunnel alignment and cover criteria.

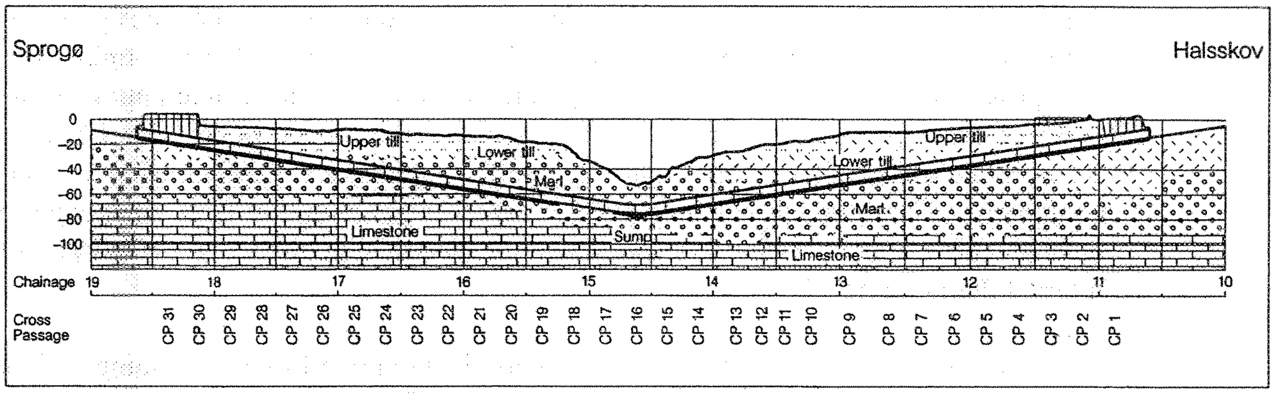

Figure 2 - Tunnel Longitudinal Section

The geology can be divided into three main types: glacial tills, Upper Palaeocene Marl and Tertiary Lower Palaeocene Danian Limestone.

Glacial tills are mainly clay tills with layers of silt and sand tills interbedded with meltwater deposits. The tills contain boulders of granite and gneiss up to 3m diameter. Glacial deposits are compact and dense in character having been preconsolidated by the overlying glaciers. The tills can be subdivided into two main strata having been deposited during successive ice advances; the Upper and Lower Till.

The Upper Till is comparatively uniform with a few isolated sand deposits (less than 1% of total mass). The undrained vane shear strengths vary from less than 100kPa to 700kPa. The Lower Tills are less homogeneous having been affected by later glaciations. Sand and gravel deposits are more frequent (up to 20% of total mass). The undrained vane shear strengths vary from 200kPa to more than 700kPa. The tills are typically 40% sand, 40% silt and 20% clay. The permeability ranges between 10^{-5} and 10^{-7} m/s.

The Upper Palaeocene Marl, with a compressive strength greater than 2MPa, is a weak to moderately weak rock according to B.S. Classification. The marl is fissured and jointed. The permeability ranges between 10^{-4} and 10^{-6} m/s depending on the extent of fissuring. It can be subdivided on the basis of variations in $CaCO_3$ content, colour and the amount of silification.

The Tertiary Lower Palaeocene Danian Limestone is a weak sandy and silty limestone containing occasional flints. The tunnel does not enter this layer.

The expected complex nature of the glacial tills was revealed by the face logging of the TBMs and confirmed by the investigations made at each cross passage, thus justifying the provisions for ground investigations made in the Contract and in the design of the main tunnel linings at each cross passage. Difficulties experienced in the driving of the main tunnel caused unexpected disturbance to the tills immediately surrounding them. The investigation and treatment methods were able to cope with this situation. The cross passages in the marl encountered ground conditions which were not as unfavourable as might have been expected.

5. Design

The passenger evacuation time was based on the American standard NFPA130, which requires that the last escapee from a train progresses from one tunnel through a cross passage and into the other tunnel within 6 minutes. Calculations based on a fully loaded IC3-train, 300m long with 720 passengers, confirmed that this requirement was met with a maximum distance of 250m between cross passages. Minimum dimensions for the escape route were 1.85m in width and 2.1m in height. These dimensions, as well as the space requirements to house electrical equipment for the tunnel operation such as signalling, ventilation fans, emergency lighting, and mechanical equipment mainly to cool down the electrical equipment, resulted in a cross passage internal diameter of 4.5m.

The contract was signed based on using Spheroidal Graphite Iron (SGI) lining and mass concrete for the junctions with the main tunnels (Fig. 3). The contractor later proposed to change the support to a sprayed concrete primary lining followed by a secondary lining of mass concrete (the NATM scheme) which was an option in the tender documents. This was accepted and a detailed design was produced.

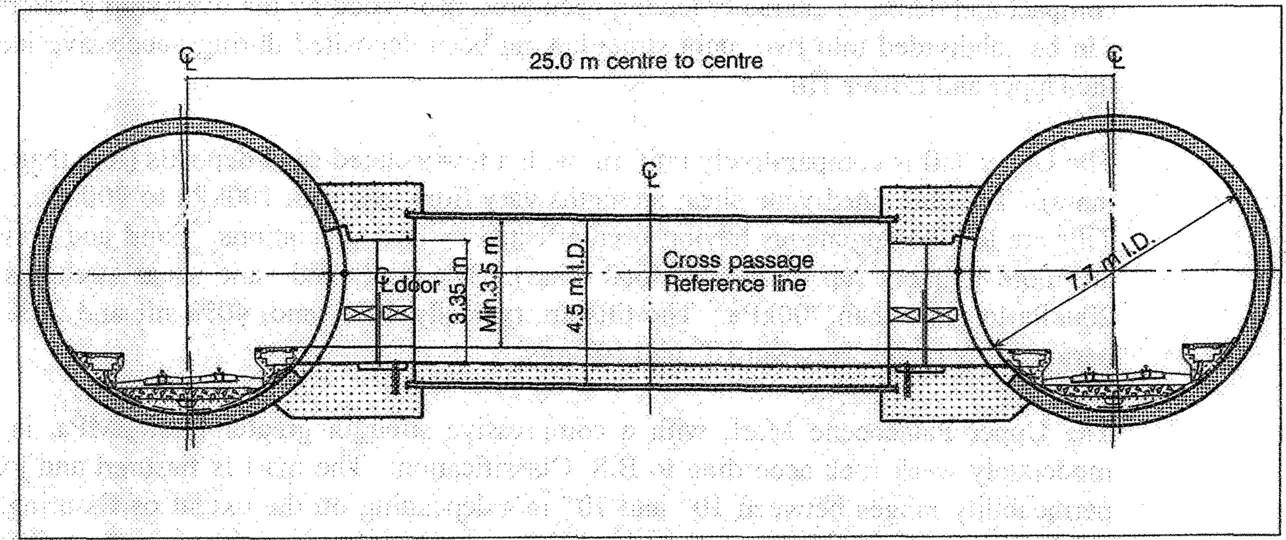

Figure 3 - Section through Main Tunnels and Cross Passages

Following the flooding of the two Sprogø tunnels in October 1991, which highlighted the risk of a collapse in the tills and sudden water inflow into the cross passages, the design was changed back to SGI lining. In addition, a 1.8m diameter SGI lined pilot tunnel was required along with temporary safety doors at all cross passage openings.

Based on experience gained in the tills, it was decided to continue using the SGI lining for the cross passages in the marls, however, the pilot tunnel was not required.

The SGI lining was designed for full overburden. The original design comprised nine segments plus the key. The contractor's alternative of 17 segments plus the key was accepted. The smaller segments were easier to cast and handle but this made the ring more flexible and possibly more prone to leaks. The ring width was 600mm and the skin thickness 14mm. The lining was fully bolted, and gasketted to resist 8 bar pressure. Caulking grooves were provided at the joints to enable water direction to drainage channels in case of leaks. Corrosion protection of the lining was provided by a bitumen coating. Fire protection of the lining was not considered necessary due to the fire capacity of the doors.

The 6.2m external diameter junction collar between the cross passage and the main tunnel was designed as an unreinforced insitu 35MPa concrete structure. Reinforcement was avoided for durability, and any cracks were to be injected.

All materials, components and systems specified were designed to ensure that generally the underground works were sealed against the ingress of groundwater. The acceptable tunnel leakage was specified as 0.1 litre/m^2/day over the length of the tunnel and cross passages. Leakage into the tunnel and cross passages was channelled to sumps at the lowest point of each tunnel.

A permanent emergency door, with a 60 minute fire rating, was included at each end of the cross passage. The doors were designed to resist the air pressures generated by the passing trains.

6. Project Moses

In 1992, 'Project Moses', a major dewatering scheme, was introduced. It had two main objectives: 1) To reduce the pore water pressure at tunnel level to 3 bar or less to allow conventional compressed air man entry into the TBM working chamber. 2) To improve soil stability and reduce the need for local dewatering at the cross passages during construction.

The marl, due to its fissures, is waterbearing. The glacial till, overlying the marl, has generally a low permeability, and can act as a cover against infiltration of water from the Storebælt to the marl aquifer. Deep wells, 405mm diameter, were drilled from jack up rigs 10m into the limestone underlying the marl. A total of 43 deep wells were split into 6 groups and powered from barges housing diesel generators. The wells were generally located every 125m along the alignment and offset by 35m on alternating sides. The total installed nominal pumping capacity was 3100m^3/hour. Over the area of influence, the pressure reduction achieved was approximately 3.5 bar in the marl and 1-2 bar in the tills. In case of well failure, the recovery rate was sufficiently slow to enable adequate precautions to be taken.

7. Site Investigation

7.1 General

In addition to the main site investigation which included 58 boreholes down to tunnel level, extensive ground investigations were required at all cross passage positions prior to any ground treatment.

The purpose of the site investigation in the glacial tills was to determine the presence of any water charged meltwater sands or sand tills within 10m of the excavation. The recharge rate of such features and the presence of any unstable or weak material within the excavation were also determined. This enabled the design of a cut-off grouting and dewatering pattern specific to the geology at each cross passage as well as ground treatment to improve the strength characteristics. Probes in the marl were used to detect the extent of water charged fissures.

Methods of investigation included face inspections by geotechnical engineers during tunnelling of the main drives. This was dependent on the mode of driving and restrictions imposed by procedures for man entry into the working chamber. Soil samples were taken for testing along with shear vane tests of the exposed strata.

Some ground radar investigations were carried out for the first few cross passages to provide a general picture of the soil structure. However, the method was limited by the interpretation techniques and was not pursued further.

A joint venture of Soletanche-Rodio was employed as the specialist ground treatment subcontractor. All local site investigation and ground treatment processes were performed by a purpose built drill train consisting of a drill rig mounted on a rail car.

7.2 Drilling Preparation

Two types of opening rings were installed in the tunnels at the cross passages: precast concrete and SGI. The concrete opening segments were provided with a number of drilling 'windows' which were clearly marked by paint. The epoxy coated reinforcement in the segments was displaced at the windows and a stainless steel cage was inserted at each face to mark the zone boundaries.

A dedicated pedestrian walkway to the TBM was suspended on the cross passage side of the tunnels for access. This walkway was diverted to the opposite side of the tunnel by two high level bridges over the tracks during cross passage drilling and construction.

The first stage of the drilling operation was to install a Blow Out Preventer (B.O.P) for all holes for ground investigation and treatment to prevent water outflow and ground loss. Holes were cored in the concrete to a depth of 320mm, and a diameter of 125mm. A threaded stainless steel stand pipe was fixed in the hole using epoxy resin and the B.O.P. with a guillotine valve was later inserted. The remaining 80mm of the concrete segment was drilled using a 89mm diameter tricone button bit.

After cross passage construction, the holes in the segments were sealed and repaired with a cement-based mortar except for the holes in the crown which were covered with a stainless steel plate.

The SGI opening segments, installed at cross passages 9-20, were provided with bolted plates which were temporarily replaced with a B.O.P. during drilling.

7.3 Drilling

In the till cross passages, typically 16 to 18 investigation holes, 89mm diameter, were drilled using tricone bits from both main tunnels. In unstable ground, the lost bit drilling method was used successfully. The probes were performed with destructive drilling techniques and logged with Enpasol equipment. The Enpasol was an electronic monitoring device developed by Soletanche which automatically recorded a number of drilling parameters every 5mm depth, including drilling torque, penetration speed and water inflow/outflow. For the marl cross passages, 5 to 6 holes were drilled using drag bits from one side only. The overall length of holes drilled was approximately 5000m.

Some investigation holes were drilled with continuous core extraction. Double core barrels, 76mm diameter, were used in both the tills and marls. The total cored length was approximately 500m. The Enpasol drilling parameters were calibrated with the results of adjacent cored boreholes.

8. Dewatering and Ground Treatment

Ground treatment was not considered as part of the permanent tunnel structure, and was classed as Temporary Works. The design of the ground treatment was the responsibility of the contractor and the contract included provisional items for the measurement of the ground treatment. Table 1 indicates the ground treatment methods used at each cross passage.

8.1 Contact Grouting

The first stage of ground treatment was contact grouting. The objective of this treatment was to improve the zone of disturbed ground behind the main tunnel lining for a thickness of 0.5m to 1.5m especially around the cross passage opening. This treatment also reduced ground losses in loose ground during the drilling of subsequent ground treatment holes.

For the first cross passages, ordinary cement-bentonite grout was injected through the drill string. For the deeper cross passages in the tills, grouting was through PVC tubes-a-manchettes. Ultrafine cement grout without bentonite was used to improve the permeation in fine granular cohesionless ground.

8.2 Dewatering

The dewatering criteria were originally based on seepage analyses of various scenarios of geometry, permeability and geological conditions. The objectives of these studies were to establish whether a ring of well points around the perimeter of the excavation could establish and maintain suitable conditions for the safe excavation in free air. The target maximum acceptable

Table 1 - Summary of Cross Passage Ground Treatment

CP No.	Soil type	Contact grout	T.A.M. grout	Spile bars	Electrodes	Well points	Freezing	CP No.	Soil type	Contact grout	T.A.M. grout	Spile bars	Electrodes	Well points	Freezing
2	Upper-lower till	X	X	X	X	X		17	Marl		X			X	
3	Upper-lower till	X	X	X		X		18	Marl		X			X	
4	Lower till	X		X		X		19	Marl		X			X	
5	Lower till	X		X		X		20	Marl					X	
6	Lower till	X	X	X		X		21	Marl			X		X	
7	Lower till	X	X	X		X	X	22	Marl			X		X	
8	Lower till	X	X	X		X	X	23	Marl			X		X	
9	Till-Marl	X	X	X		X	X	24	Marl			X		X	
10	Marl		X			X		25	Marl			X		X	
11	Marl		X			X		26	Upper-lower till	X		X	X	X	
12	Marl		X			X		27	Lower till	X	X			X	X
13	Marl		X			X		28	Upper-lower till	X	X	X		X	
14	Marl		X			X		29	Upper-lower till	X	X			X	
15	Marl		X			X		30	Upper till	X		X		X	
16	Marl		X			X									

Key
CP cross passage
T.A.M. tube-a-manchette

pore water pressure within a sand or sand till strata was 0.05 bar pressure. It was considered that seepage resulting from this pressure could be safely controlled by close timbering. The maximum acceptable hydraulic gradient into the excavation was 0.2.

A typical well point layout in the tills is shown in Fig. 4. A similar layout of well points was drilled from both tunnels to the cross passage mid point. The well points were generally spaced at 2.5m centres approximately 3m from the excavation. At the crown and invert at the openings the well point spacing and distances were shorter due to the geometrical constraints.

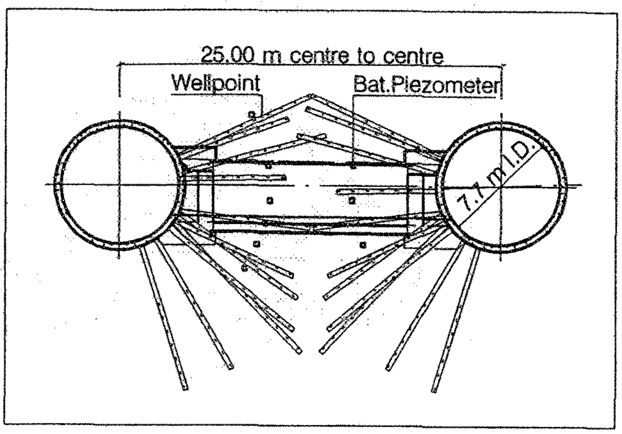

Figure 4 - Cross Passage Dewatering

The well points were 8-12m in length and contained a filter tip with slot widths of 1mm. On completion of drilling in the tills, a biodegradable polymer suspension containing approximately 50% by weight of sand was introduced through the drill bit. Once the drill string was withdrawn, the well point was introduced.

The well points were connected to a pumping system capable of discharging 60m^3/hour against 20m head with 9.5m of water vacuum head. Discharge was usually of the order of 10m^3/hour. The pumps, with one available on standby, were connected to the emergency power supply. The system was generally installed, run and monitored for at least 10 days prior to mining. Some 'tuning' of well points was required in order to prevent excessive amounts of air from being drawn and reducing the effectiveness of the system. This was carried out by partially closing valves.

Vibrating wire and BAT fast response piezometers were installed to monitor the pore pressure on a daily basis and more frequently during excavation.

Electro-osmosis was used at cross passages 2 and 26 to improve the efficiency of the well point system due to the higher clay content in the tills. A pattern of 32mm diameter steel spiles were installed from the main tunnels above the crown of the excavation. These were connected to the anode of a DC welding set (50-80 volts and 30 amps) with steel well points connected to the cathode.

8.3 Tube-a-Manchette Grouting

The objective of the tube-a-manchette grouting in the tills was to penetrate any fissures present around the cross passage excavation and hence prevent any unacceptable water inflows. Some local hydro-fracturing of the weaker tills was necessary to ensure interception of the existing fissure system. PVC tube-a-manchettes (OD 48mm) were installed at a spacing of 1.5m to 2.0m at the end of the holes (Fig. 5). The injection was carried out by a double inflatable packer which was positioned at the valve to be grouted. The valve spacing was 0.33m along the tube-a-manchette. The cement mix was injected using a piston pump at a controlled flow rate of 500 - 700l/hour. The injection continued up to predetermined pressure and/or volume limits. During the grouting the pressure and flow rates were continuously recorded.

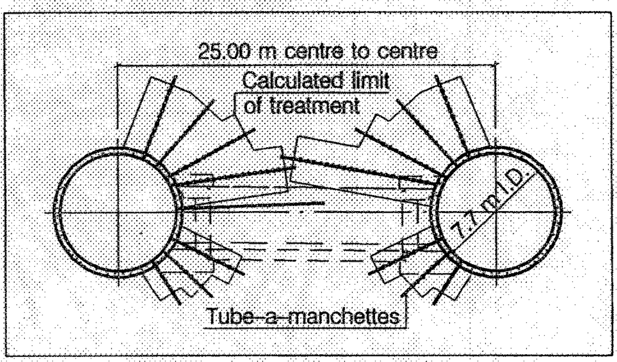

Figure 5 - Cross Passage Tube-a-Manchette Grouting

The grouting was carried out in a number of passes. The first four valves behind the tunnel lining were injected first. This was followed by four passes of injecting one valve in four starting at the end of the hole. The maximum volume of grout was predetermined, and equivalent to approximately 40% of the volume of soil to be treated for the meltwater sands. The pressure directly behind the lining was limited to 4 bars above ambient ground water pressure. This was increased to 10 bars at distances exceeding 5m away from the lining, and depended on the results of previous injections. In addition, where sands and gravels were detected, the aim was to lower the permeability of the ground to 10^{-6} m/s by reducing the porosity. The spacing between the tube-a-manchettes was reduced to 1.0m to 1.5m at the end of the holes.

In the marls, the objective of the ground treatment was to penetrate fissure systems present around the excavation and hence reduce unacceptable water inflows. The grouting was carried out using PVC tube-a-manchettes (OD 48mm) spaced at 2.0m to 3.0m at the ends of the holes. Ultrafine cement grouts were used.

The total quantity of grout varied at each cross passage. At CP27 for example, located on the edge of a buried valley structure, approximately 150m^3 of grout was injected, whereas at CP16, the deepest cross passage located in the marl, 28m^3 of grout was used.

The combination of grouting and dewatering proved to be a safe and economical method of treating the tills and marls.

8.4 Ground Freezing

At several cross passage locations, disturbance to watercharged sand bodies, during excavation by the TBMs, resulted in the development of seabed depressions and hydraulic connections between the sands and the Storebælt. At these locations, a frozen hood of ground over the excavation was required in order to consolidate and waterproof the soil. The treatment consisted of circulating brine (a solution of calcium chloride $CaCl_2$ in water) cooled by means of a freon freezing plant, through two arrays of sub-horizontal pipes installed from the main tunnels. Before freezing, some cement grouting was necessary to fill local discontinuities.

The freezing plant was electrically powered and suspended on the side of the main tunnel between two cross passages. The flow of brine into the pipes was continuous during the freezing phase and sometimes intermittent during the maintenance phase depending on temperature readings taken in the ground. Thermocouples were installed with tube-a-manchettes and connected to data loggers to constantly monitor the temperature.

Freezing pipes were generally located to ensure a maximum spacing of 1.1m around the excavated profile (Fig. 6). The pipes were installed through the drilling windows in the lining taking into account the geometrical constraints, including the location of the emergency door and propping frame. Some pipes had to be located through the collar. Once exposed during excavation these pipes were isolated and removed.

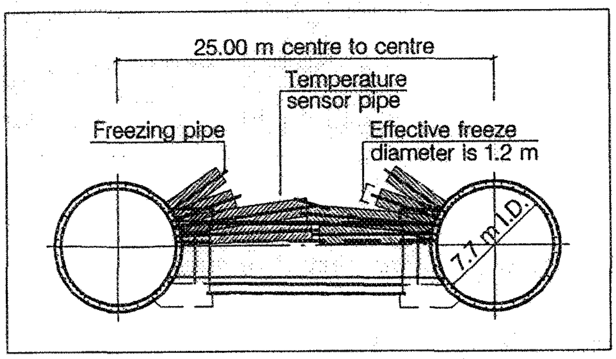

Figure 6 - Cross Passage Ground Freezing

Each freezing plant had a capacity of 100,000kcal/hour and supplied brine at an initial temperature of approximately -27°C. Once the target temperatures of approximately -5°C at 600mm from the freeze pipe were achieved, it was considered safe to commence excavation, and this generally took 4 weeks.

9. Temporary Propping

Temporary propping of the main tunnel lining was required to support the hoop load in the lining and maintain the shape of the rings. Four steel ring beams were installed in both main tunnels at every cross passage. The ring beams were supported by props and distribution beams arranged so that they did not interfere with the construction trains (Fig. 7).

In the opening side, the propping supported a safety door which was constructed to effectively seal the opening against unacceptable water ingress. The safety door was operated by a steel chain and air operated hoist, and secured against the frame with hydraulic jacks. An emergency supply of bottled compressed air was automatically available to close the safety door in case the main air supply was interrupted. In the event of an emergency, the door could be closed and sealed within 1.5 minutes. The door operation was regularly tested but fortunately never required in an emergency.

In the deeper marls, the propping was modified to carry the increased loading by incorporating additional raking props across the tunnel. These props were located above the passing rail traffic.

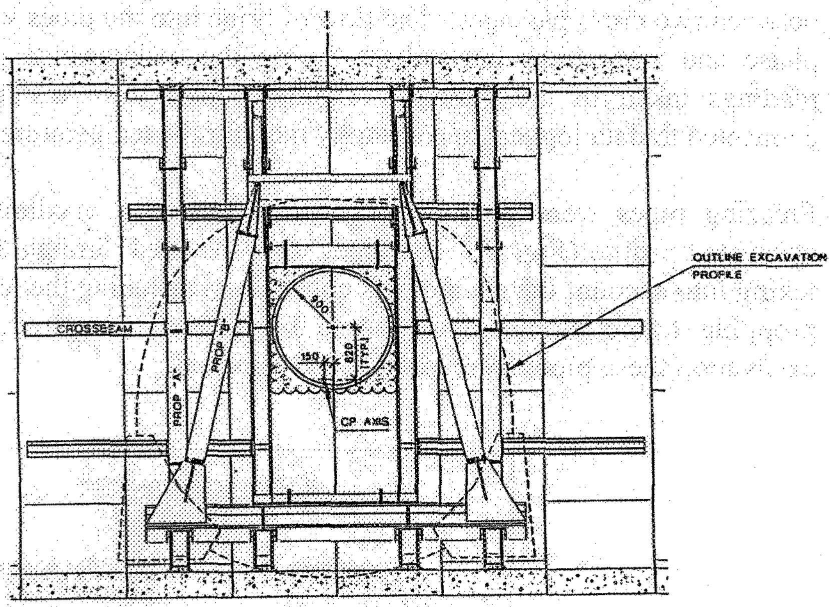

Figure 7 - Cross Passage Temporary Propping

10. Excavation and Ring Build

The cross passages were excavated concurrently with the main tunnel drives. Twelve of the cross passages were located in the tills, sixteen in the marl and one in the transition zone between the till and the marl.

A labour only subcontractor was employed to excavate and build the cross passage rings which included the excavation and shotcrete support of the collars connecting the main tunnels. Traditional hand tools were used to excavate the ground, with a system of conveyors used to transport the muck into waiting muck cars in the main tunnel. The cross passage rings were built from timber staging using a system of air winches and pulleys. Traditional timber face support was used in the tills and sometimes sprayed with shotcrete to assist in the preservation of the dewatering vacuum.

The main tunnel concrete segments on the opening side of the cross passage were removed in two stages by a combination of saw cutting and coring. Initially a small opening was formed in the segments located in the upper section of the cross passage. A pilot, 1.8m diameter and

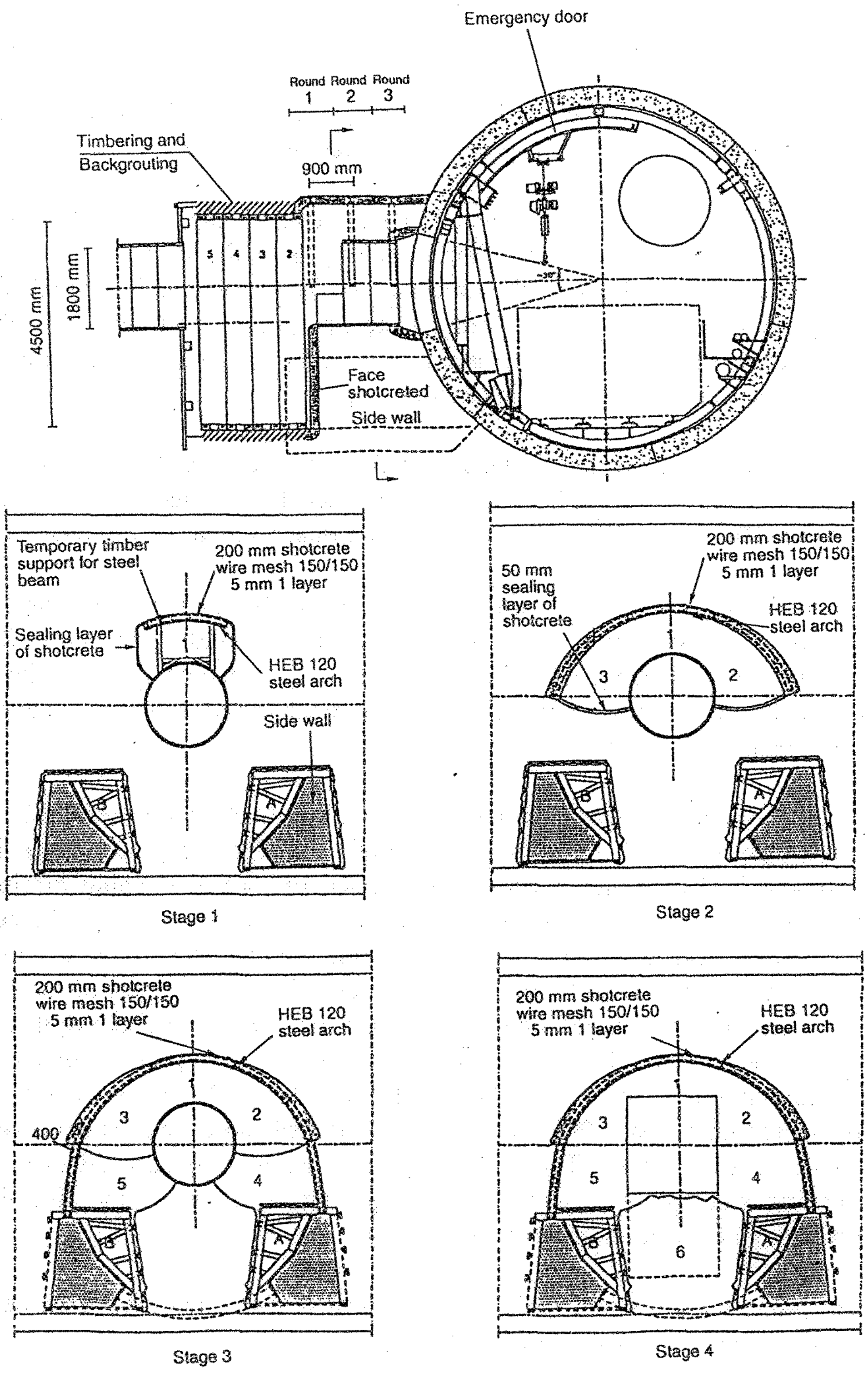

Figure 8 - Cross Passage Excavation in the Glacial Tills

600mm ring width, was then driven to the back of the other main tunnel, grouting each ring progressively and using timber face support. Timber packings were inserted in the circle joints of rings 5 - 8 to assist the break up for the first two 4.5m diameter SGI rings. The break up location was a compromise between the following: 1) the desire to have the enlarged opening at the main tunnel as early as possible to assist access for further excavation, 2) the requirement to provide adequate structural resistance by building a number of rings to prevent excessive deflection of the main tunnel lining into the opening, and 3) to form the break up under the protection of the spile bar hood. Upon completion of building and grouting rings 5 and 4, rings 3 and 2 were built in sequence, progressively removing the pilot rings.

The opening collar was constructed in a number of stages (Fig. 8). Initially two 'filler walls' were constructed at knee position from ring 3 to the back of the main tunnel using timber headings, and later backfilled with concrete. These filler walls provided stability to the back of the main tunnel in conjunction with the steel propping, and provided a suitable foundation for the collar crown construction. The main tunnel propping was adjusted immediately after the filler wall construction by exchanging raking props.

The collar was excavated in stages installing steel arches, shotcrete and wire mesh as temporary support. Initially the top heading was constructed in 3 rounds from ring 2 to the back of the main tunnel. This was followed by the bench and finally the invert between the filler walls.

Upon completion of the collar, the opening in the main tunnel was enlarged. This enabled improved access to mine in the opposite direction, building rings 6 to 21. The finishing collar was constructed using the same sequence as the opening collar. The finishing side opening segments were removed providing access through the cross passage, and this was followed by the building of the end rings 1 and 22.

A small roadheader was procured for the marls but the miners decided to continue excavating the cross passages with traditional hand tools. The cross passage opening rings in the marls comprised SGI segments with an integrated preformed opening originally developed for the Channel Tunnel. A pilot tunnel was not required in the marls and the sequence of construction was less complicated. The opening was formed in two stages. The top segments were unbolted and removed, followed by the excavation and shotcrete support of the collar top heading in 1m advances (Fig. 9). Steel arches, wire mesh reinforcement and Swellex rock bolts also formed part of the support. The bench and invert followed in stages to complete the opening collar prior to removing the lower opening segments. The 4.5m ID rings were built and grouted towards the opposite tunnel, using 200mm of mesh-reinforced shotcrete as support and a 50-100mm layer of shotcrete at the face. The finishing side collar was constructed in a similar sequence to the opening side, prior to removing the opening segments in the finishing tunnel and thereby providing access through the cross passage.

For programme reasons, CP9, which is located in the transition zone between the till and marl, was mined in two halves from each side.

Continuous gas monitoring was carried out during cross passage excavation especially as the risk of methane contamination was highlighted in the contract. Self rescue masks were provided for emergency cases.

The wet mix shotcrete was transported to the cross passage opening using rail mounted remixers, where it was transferred to a pump as required. The mix was designed to allow placing up to 12 hours after batching as it was necessary to always have shotcrete available at the cross passage as

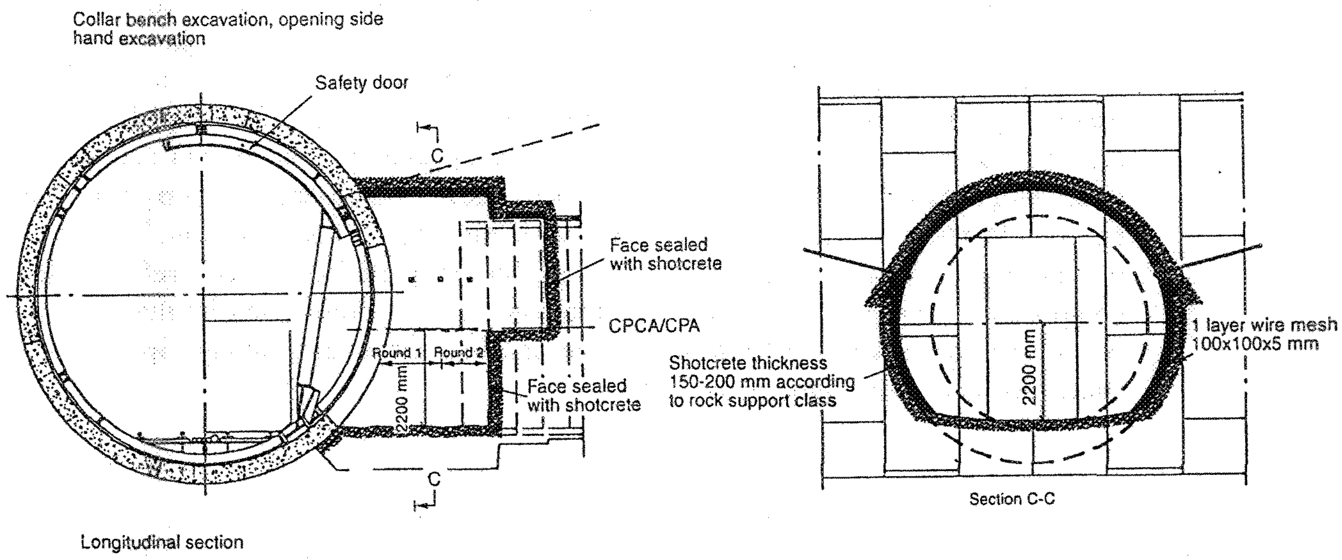

Figure 9 - Cross Passage Excavation in the Marl

a contingency during excavation. Cores were regularly taken and tested for quality control purposes.

During cross passage construction, an intensive monitoring regime was undertaken. This included tape extensometer readings and levelling of the shotcrete collars, 3-D optical monitoring of the main tunnel lining including the propping and strain gauge readings of the propping.

In the tills, the pilot was constructed in 3-6 days. It took a further 18-24 days from the breakup to completion. The total time was on average 25 days for continuous seven day/week excavation. However, in the marls, the total excavation time was on average 13 days. CP12 was completed in only nine days.

11. Collar Concreting

Upon completion of the excavation, the concreting works commenced with the installation of an elaborate system of waterproofing tubes and hydrophillic strips on the contact area between the collar and the main tunnel. The tubes consisted of injection hoses and neoprene elastomeric foam strips. The tubes were later injected with polyurethane in a planned sequence through connection tubes with outlets located in the door handle recesses. A water channelling membrane was fixed to the shotcrete collar and later grouted after the collars were cast.

In order to reduce shrinkage cracking, the specification for the concrete imposed strict limitations on the mix design, including a maximum water-cement ratio of 0.45, a maximum water content of 140kg/m^3 and maximum temperature differentials between adjacent pours. The contractor developed a mix which met these requirements, and his own requirements of pumpability over a life of up to six hours from batching. For the deeper cross passages which required the longest travel time in the rail mounted concrete mixers, post-dosing of super plasticizers was required at the cross passages.

The initial concrete pour was in the invert of the SGI segments between the collars. The following invert pours to both collars produced a construction joint at passage floor level (Fig. 10). The walls and soffit of each collar were cast in one operation using a timber shutter which was designed to ensure that the safety door operation was not restricted during casting. The crown stop end was partly omitted during the wall pours to enable man access and the use of poker vibrators for the walls. Once the concrete level had reached the top of the walls, the stop end was installed, including a gate valve for continuing the concrete pour. The use of vibrators was not viable at this stage, and the self-levelling properties of the concrete enabled acceptable compaction. The pour continued until the concrete was pressurised by the pump and concrete was observed discharging from three large diameter air relief/grouting pipes through the soffit shutter located at the high points.

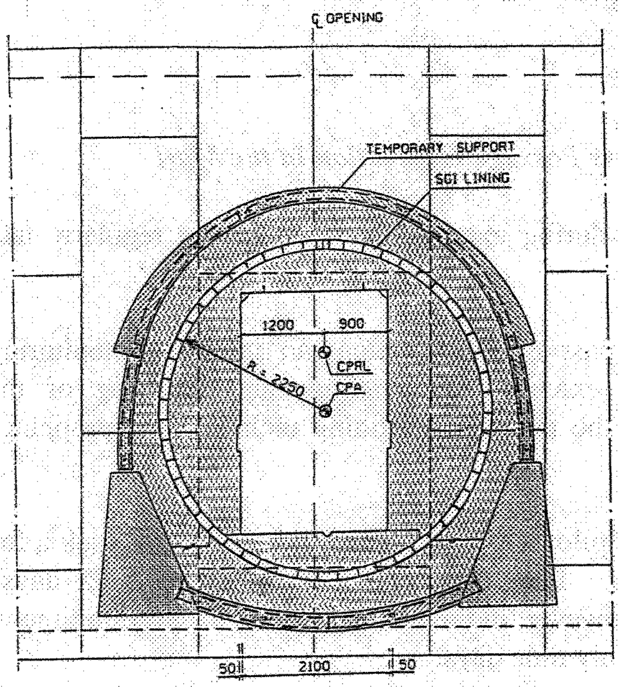

Figure 10 - Cross Passage Concrete Collar

The main concreting works, including cleaning, fixing of waterproofing items and casting the SGI invert and two collars, generally took three weeks to complete.

12. Finishing Works

Concrete plinths were cast along the passage to support a suspended steel floor which enabled cabling to be routed underneath.

A dust sealing coat was applied to all concrete surfaces within the air tight cross passage doors to reduce the amount of dust potentially affecting the electrical and signalling equipment housed in the cross passages.

The collars were grouted and the concrete was allowed to set for four days prior to removing the safety door and part of the ring beam props directly adjacent to the opening. In the concrete opening sets epoxy coated steel jambs were installed. The jambs were encased with stainless steel reinforcement for fire resistance, and concreted through prefabricated steel formwork. After a minimum of three days the entire propping in the main tunnel was removed. The SGI opening sets did not require separate jamb frames.

Once the collars were grouted, the local dewatering was decommisioned. The well points were then backgrouted and the standpipes in the cored holes of the main tunnel concrete segments were repaired.

Some water leakage occurred through joints in the SGI lining and through cracks in the concrete collars as well as from the contact area between the collar and the main tunnel. These were sealed by injecting polyurethanes through steel packers, and sometimes acrylic resins were used when a less viscous material was required. To mitigate tunnelling delays, early access in the cross passages of the fixed equipment contractor was provided. The cross passages were tested for watertightness once the full water pressure was obtained. During the operation of Moses dewatering, artificial recharging of the well points around the cross passage was necessary to locally increase the water pressure.

Following the fire in the northern tunnel in June 1994, emergency measures were rapidly taken to prevent a potential collapse of the fire damaged tunnel lining leading to the flooding of both tunnels. These measures included temporary steel drawbridge-type flood doors in cross passages 2-7. Until a bulkhead was constructed in the tunnel between the fire damaged TBM and CP7 six months later, no work was possible in these cross passages.

13. Summary

The success of the cross passage construction in such difficult ground conditions can be attributed to a combination of factors including the many different ground treatment measures which ensured a largely 'dry' excavation, the many safety measures employed resulting in no serious injuries, and the skill and dedication often in very adverse conditions of the drilling crews, miners, Danes and expatriates, to whom the authors offer their sincere thanks.

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