Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 78 (1998)

Rubrik: Session 5: Construction

SESSION 5

Construction

Chairmen:
A. Haack, Germany, and L. Skogsberg, Sweden

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Construction of elements for the Øresund Immersed Tunnel

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Summary

The western part of the Øresund Fixed Link between Copenhagen, Denmark and Malmö,Sweden consists of a 3.5 km long immersed concrete tunnel. The tunnel is divided into 20 elements each 176 m long, made up of 8 segments each 22m long. The outer crossectional dimensions are 8.5 m by 41.7 m enclosing two railway tubes, two motorway tubes and an escape gallery.
This paper describes the incremental construction and launch of the elements for the tunnel, the development of the various components of the cast and launch facility and relates the contractor's experience with the use of the facility and the quality assurance system developed and used for the design and construct contract.

1. Introduction

1.1 Øresund Link and Øresund Tunnel Contractors I/S

The Øresund Tunnel, currently under construction in Copenhagen, will be the largest immersed tunnel in the world. The 3.5 km long immersed tunnel under the Drogden Channel is part of the 16,4 km long Øresund Link between Kastrup at Copenhagen, Denmark and Lernacken at Malmö, Sweden. The tunnel with ramps and portal buildings is currently being constructed on behalf of the Øresundskonsortiet (a client company set up and owned by the Danish and Swedish governments) by Øresund Tunnel Contractors I/S (ØTC). ØTC is a joint venture comprising NCC AB of Sweden (leader), John Laing Construction Ltd of England, Dumez-GTM SA of France, Boskalis Westminster Dredging bv of The Netherlands and E.Phil & Søn A/S of Denmark.

1.2 Design and Construct Contract

The contract signed by Øresundskonsortiet and ØTC is a design and construct contract including the design, construction, inspection, testing and handing over of the completed immersed tunnel with ramps and portals and all related mechanical , electrical and local control and communication systems. This type of contract made it possible for the conceptual design of the tunnel to be developed concurrently with the construction methods for the tunnel.
Early in the tendering period ØTC recognised that the 3.5 km length of the Øresund Tunnel would justify the development of a high production precast facility tailored specifically to the tight time schedule and quality demands presented in the tender documents. It was further recognised that while such a facility would be expensive to design and build, it could be made economical if

recognised that while such a facility would be expensive to design and build, it could be made economical if the large casting basin normally associated with conventional tunnel casting operations could be eliminated and if the high cost of concrete cooling normally associated with tunnel casting could be significantly reduced.
During the tender period ØTC's engineering staff worked in close co-operation with the tunnel structural designer Symonds Travers Morgan (STM) of England and the tunnel M&E design and construct sub-contractor SSB-Øresund, a joint venture of SEMCO of Denmark and Spie-Enertrans of France. Through this close co-operation ØTC and its design team were able to design a higher quality tunnel while reducing the construction cost of the tunnel by:

- standardising the precast segments to allow for a highly repetitive casting system
- eliminating all cooling of tunnel concrete
- substantially reducing the required size of the casting basin

1.3 Construction Schedule

The Contract was signed in July 1995 and Commissioning is scheduled to summer year 2000. See Figure 1.

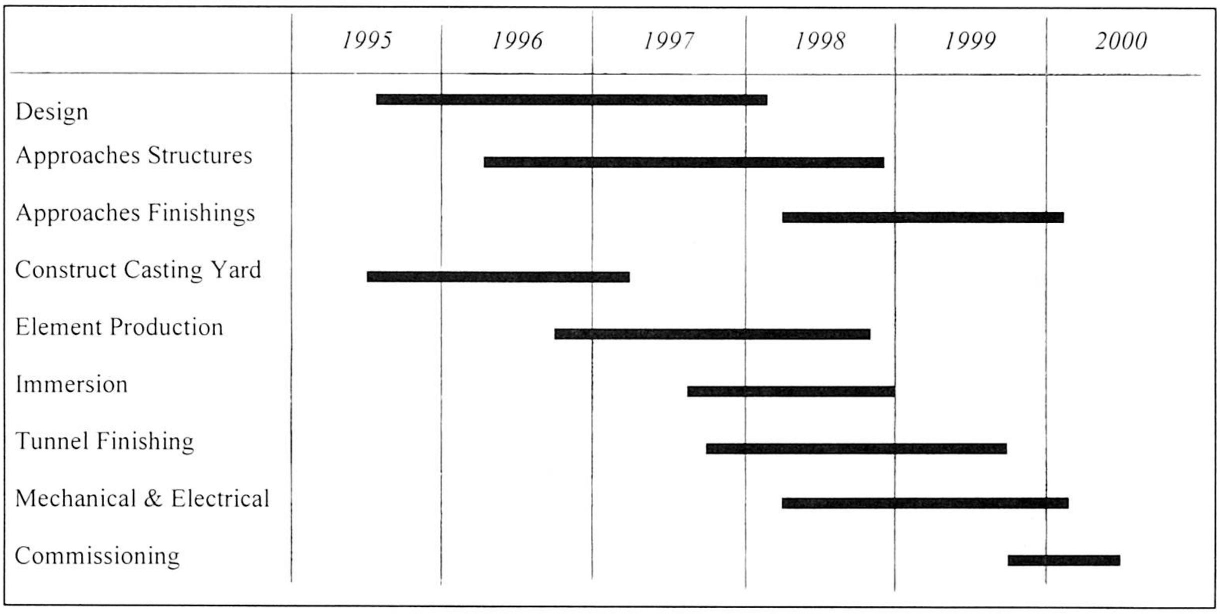

Fig. 1 ØTC time schedule for the tunnel

2. General description of construction method

2.1 Construction method

The immense size of the Øresund Tunnel made it economically feasible to invest in large facilities in order to achieve an industrialised construction sequence. Thus all construction activities for production of the twenty tunnel elements are performed in a rebar and precast building erected only for this purpose.
Different construction locations were studied including existing docks and shipyards and their impact on transportation techniques and cost. The final choice was The North Harbour of Copenhagen. Thus the tunnel elements are fabricated at the ground level on

two parallel production lines.
The factory is situated in the western part of the casting yard. At the eastern end is the launch basin, an area bounded by earth dykes to allow the impounding of water. The construction facility is shown on Figure 2.
The 3.5 km immersed tunnel comprises 20 reinforced concrete elements, each 175.4 m long. Each tunnel element is divided into 8 segments of 21.9 m weighing approximately 6500 tonnes. The outer crossectional dimensions are 8.5 m by 41.7 m enclosing two railway tubes, two motorway tubes and an escape gallery, see figure 3. The production facility will work on the principle of incremental launch method. The method involves the casting of each discrete segment on a fixed casting bed and then after a minimum curing period the segment is pushed clear of the casting bed for match casting of the next element in the same bed. Once a complete tunnel element of eight segments has been cast the entire element is pushed approximately 100 m into the shallow part of the launch basin for outfitting.
After outfitting flooding can be done allowing the tunnel element to be moved to the deep part of the basin. The basin is surrounded by earth dykes to the level 10 m above sea level. The entrance or the sliding gate forms the closure towards the fabrication building. The exit or floating gate forms the closure at the seaward end of the basin. Water level is lowered to sea level when the tunnel element has reached its position in the deeper part of the basin. The tunnel element will then be towed to the tunnel alignment in the Drogden channel between the peninsula near Copenhagen Airport and the artificial island.
Before flooding can be performed the tunnel element will be trimmed with ballast tanks and temporary bulkheads at the end to allow for the towing. Temporary prestressing will also be installed to act during towing and immersion.

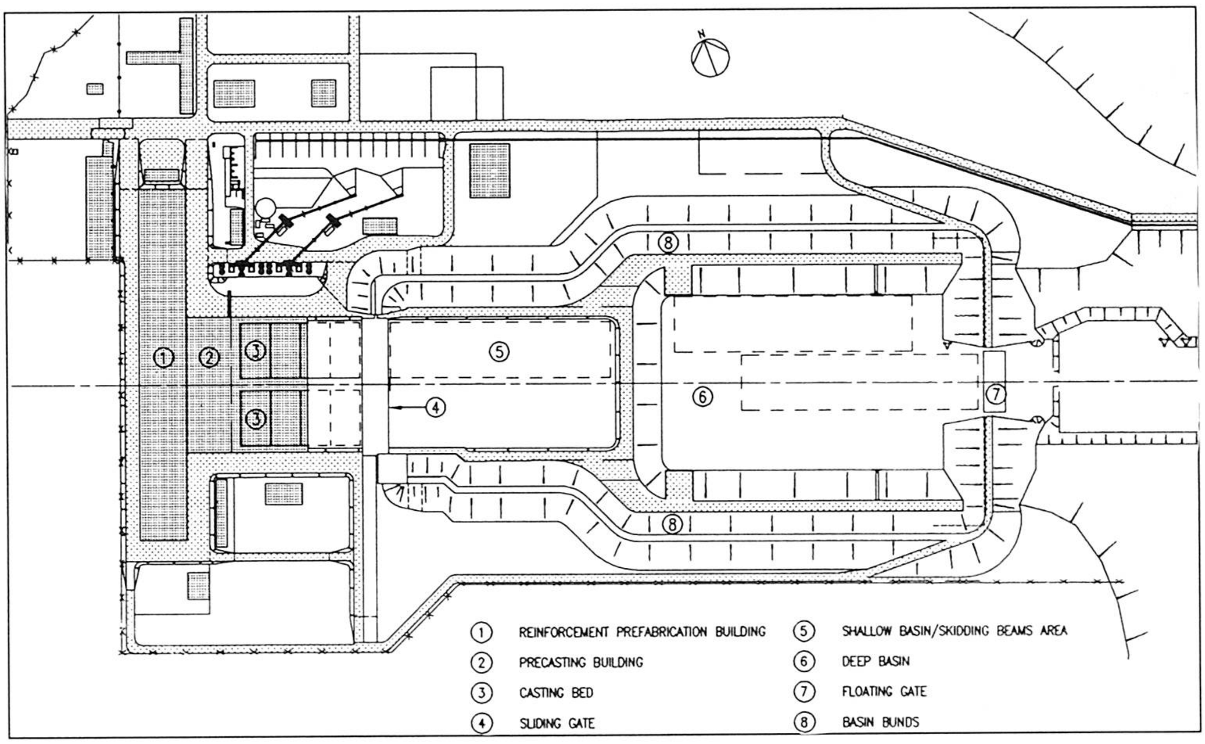

Figure 2. Plan of construction facility.

2.2 Concrete technology with respect to construction method

The contract documents stipulated a 100-year design life for the tunnel, implying high requirements on concrete durability and permeability. Moreover no early-age cracking were permitted. To achieve a high quality and construction efficiency a concrete with good workability was another requirement.

One characteristic feature of the construction method is that each tunnel segment is cast in one single poor.. By this method excessive measures for early-age crack control could be avoided, for instance installation of cooling pipes.

The incremental launch method also give rise to stresses to the young concrete which had to be considered in the process of selection of concrete mix, especially type of cement.

In the evaluation process two types of cement were tested: Std Portland Cement (OPC) and Blast Furnace Slag Cement (BFSC).

Six different concrete mixes were tested with respect to early age properties, three with BFSC three with OPC. Different coarse aggregates and content of additives were considered. The heat development and risk for early age cracking were analysed for the different mixes. The analyses were performed by the Dutch consulting company Intron SME. In the matemathical model used by Intron the influence of deformations in the skidding beams could be taken into account. The result of the study showed that the risk for early age cracking were similar and acceptable for some of the mixes with BFSC and OPC. The type of coarse aggregate had an impact on early age properties of the concrete implying that aggregates which gave a lower elastic modulus at early age were more favourable in that respect. Use of OPC mix enabled higher early strengths which meant the segments could be jacked forward earlier. As a resultant of this fact and of the analyses of risk for early age cracking a concrete mix with OPC were selected.

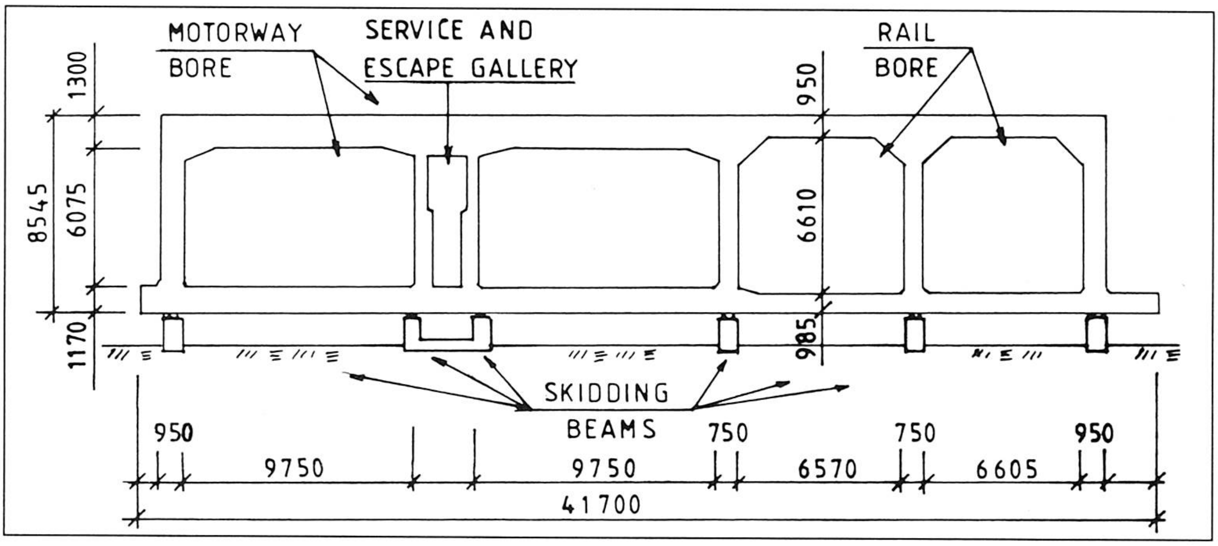

Figure 3. Cross section through tunnel and skidding beams.

3. Incremental construction and launch

3.1 Concept Selection and Development

During the tender period ØTC developed the following criteria for the casting method:

- **Schedule** – The first tunnel element should be launched within 18 months of contract award, and the entire 3.5 km of immersed tunnel should be cast and launched within an additional 24 months.
- **Standardization** – The casting method should take advantage of the relatively constant cross section of the tunnel to standardize the casting operation.
- **Weather Affect on Operations** – The casting method and associated facility should allow for year-round operation without shutdown by cold weather, rain, snow or ice.
- **Continuity of Work** – The method should allow for a continuous flow of work with a relatively balanced amount of work for the various crews working on fabrication of reinforcement, forming, casting, curing and outfitting of the tunnel elements.
- **Reinforcing Fabrication Economy** – The casting method should allow for an independent prefabrication of reinforcing .
- **Forming Economy** – The method should allow for use of steel forms casting on a fixed bed.
- **Concrete Quality and Workability** – The time between batching of the concrete and placement should be kept to absolute minimum by batching and mixing the concrete adjacent to the casting bed.
- **Concrete Cooling** – The casting method should allow the use of continuous full section casting and thereby eliminate the need for concrete cooling.
- **Match Casting** – The casting method should incorporate the use of match casting and the use of the water stops specified in the tender documents.

ØTC considered a number of different casting options including vertical match casting, casting on floating platforms, and casting with traveling forms in a conventional casting basin. The casting method which best met the above criteria however is a method which has been successfully used in bridge construction for over 30 years, incremental cast and launch. The method involves the match casting of identical segments on a fixed casting bed followed by incremental launching of the cast segments on a regular cycle.
A review of the various casting methods indicated that the segmental casting and incremental launch technique would only need a relatively small casting basin and that by casting the tunnel segments in one continuous pour it would be possible to totally eliminate the need for concrete cooling. This elimination of the cooling was later confirmed after a thermal analysis showed that the thermal stresses in the concrete could be maintained within acceptable limits by full section casting of tunnel segments.

The final conclusion was that a segmental concrete tunnel design combined with the incremental cast and launch techniques would be the most cost competitive solution. The key cost reduction factors were deletion of the concrete cooling for 440,000 m^3 of concrete in the immersed tunnel and reduction of the required capacity of the deep casting basin from 8 elements to 2 elements. An important feature of the cycle described in chapter 2.1 is that the casting of the tunnel segments is allowed to proceed on a continuous basis uninterrupted by the outfitting and launch of the completed tunnel elements. This is in sharp contrast to a conventional tunnel casting basin where all casting operations are halted during the typical outfitting and launch phase.
By allowing the casting to run concurrently with the outfitting and launch it was possible to cut over 6 months from the construction schedule.

3.2 Construction Sequence

The typical construction cycle for a tunnel segment takes 7 calendar days and approximately 2 months to cast, outfit and launch two complete tunnel elements.

The complete construction concept is summarised in the following sequence:

- Deliver cut and bent reinforcing to site and fabricate into complete cages
- Slide the reinforcing cage onto a pre-set casting bed positioned over 6 skid beams
- Insert internal steel forms and position external steel forms
- Cast the complete full section 21.9 m segment in one pour
- Cure the segment for 52 hours and release bottom forms and side forms
- Skid the completed segment 21.9 m, allowing the next segment to be cast
- After casting 8 segments push the entire element into the shallow basin for outfitting
- While outfitting the element continue with casting of segments as above
- Outfit the two tunnel elements for launch
- Close sliding gate and floating gate
- Flood basin to elevation + 9.9 m
- Ballast and trim tunnel element
- Winch tunnel element into the deep end of the basin
- Drain the basin back to sea level and lower tunnel element to sea level
- Open floating gate and tow completed tunnel element for additional marine outfitting and then to immersion site
- Open sliding gate and allow next completed tunnel element to be pushed in to the shallow basin for outfitting.

The various systems of the incremental cast and launch method are presented below.

3.3 Reinforcing Fabrication

The pre-fabrication of complete reinforcing cages for each segment is carried out in a 260 m long by 35 m wide building serviced by three 20-ton capacity gantry cranes. The total floor area allows pre-fabrication of two bottom mats 22 m by 42 m to be pre-assembled on a skid system built in line with the casting beds. At either end of the reinforcing building are wall templates equipped with self elevating work platforms which are set up for the pre-fabrication and storage of completed wall panels. Following completion of the bottom mat and erection of stabilizing templates, the overhead gantry cranes are used to set the wall panels. The reinforcing cage is after completion with the roof mat braced internally and rolled into a buffer area until the form system is ready to accept the completed cage. The pulling of the cage is performed with two winches.

3.4 Forming

The tunnel segments are formed on two parallel casting beds. The forming system for the tunnel consists of four main components: the vertically retractable bottom form, the retractable exterior wall forms, the telescoping interior forms and the stop-end forms. All of the forms are positioned and adjusted hydraulically. The bottom forms are mounted on pile supported concrete foundations and fitted to the six skid beams that pass through the centre of the casting bed. The interior tunnel forms are designed to roll on travelling steel trusses which telescope into each of the five tunnel bores. These trusses are used to position each interior form into the centre of each bore and are designed to carry only the dead weight of the side and roof forms. The vertical dead load of the fresh roof concrete is transferred by internal bracing and the side forms into the fresh concrete of the bottom slab. Two rows of standard wall ties are used in the interior walls and two rows of special waterstop ties are used in the exterior walls.

Prior to pushing the cured tunnel segment off the casting bed, the exterior side forms are retracted and the bottom forms are lowered, transferring the weight of the tunnel segment onto tunnel support bearings positioned on the skid beams. The tunnel segment is then pushed 21.9 m with the interior forms still in place. After resetting of the bottom form and installation of a new reinforcing cage, the support beams of the interior forms are telescoped back into the new reinforcing cage and the forms are then rolled on the support beams into position for casting the next segment.

3.5 Concrete Batching, Mixing and Placement

Concrete is produced in twin Sermec CF 150 batch plants with a rated capacity of 130 m^3 per hour each. A single plant has the required capacity for casting a typical segment. The second plant is provided as assurance of completing a segment should one of the plants break down. Both plants are located within 100 m of the two casting beds. All materials are delivered to the plant by truck except for the fine and coarse aggregate which are delivered by ship. Following mixing, the concrete is discharged directly into one of four concrete pumps and pumped direct to the casting bed via articulating booms. The pumps have a rated capacity of 70 m^3 per hour through the 125mm diameter booms. Each casting bed is serviced by 4 booms.
The typical placing sequence starts in the bottom slab, directly under each of the 6 walls and finishes at mid points in the roof slabs over each of the 4 tunnel bores. The 2700 m^3 of concrete in each segment is typically placed and compacted in a continuous 30 hour pour.
A system of external form vibrators and internal poker vibrators is used to compact the concrete.

3.6 Skid Beams

The skid beams support approximately 70 % of the tunnel segment dead weight during casting and initial curing. The beams provide an additional side benefit during the curing of the tunnel segments, by allowing the under side of the tunnel to cool off at the same rate as the other three sides. This helps to minimise the thermal stresses in the young concrete and minimises the risk of thermal cracking. The primary purpose of the skidding beams however is to provide support for the completed tunnel segments as they are pushed clear of the casting bed after 52 hours of curing.
There are six concrete skidding beams aligned with each casting bed. Each beam is centred under one of the six walls of a typical tunnel section. (See Figure 3) The reinforced concrete skid beams are 1500 mm deep by 700 mm wide supported at approximately 3 m centres by 400 mm square reinforced concrete piles driven into the underlying limestone. The number and size of the piles was determined by settlement limits for the support beams. Each beam has a low friction epoxy surface on its entire 285 m length. The beams are stabilised in the transverse direction by 1000 mm deep by 750 mm wide reinforced concrete ground beams located on about 10 m centres. In order to minimise deflection stresses in the young precast tunnel segments, the skid beams over the casting bed area were cast to a level tolerance of +/- 2mm and the remaining 260 m of the skid beams were cast to a tolerance of +/- 3mm. The anticipated variation in the elastic shortening of the support points was +/- 1mm.

3.7 Bearing System

The primary purpose of the bearing system is to ensure even load distribution of support reactions into the young concrete of the tunnel segments as the tunnel segments are skidded over the uneven surface of the skid beams. (Even with the tight level tolerances stated above, the theoretical vertical difference between any two support points could be as high as 12 mm.) Hydraulic bearings consisting of low profile piston jacks are provided under each tunnel segment at 3.65-m centres along the skidding beam. Each segment is supported on 36 bearings (6 bearings per skid beam) of 300 to 400-ton capacity for a total of 288 bearings for a complete tunnel element. The 36 jacks under each segment are interconnected into three separate hydraulic circuits. Each of the three circuits is connected to an accumulator to provide a spring like effect on a three point support. A PTFE pad is bonded to the bottom of each jack for sliding on the epoxy surface of the skid beam. Each jack is fitted with a flow control valve to lock off the jack if a break or excess leakage occurs in the hydraulic circuitry.

3.8 Pushing & Guiding System

The dead weight of a complete 175.4 m tunnel element is approximately 52,000 tons. The maximum friction coefficient measured is about 4% at break out and 1% moving. The pushing system consists of 6 pushing assemblies, one assembly mounted on each of the six skidding beams. Each of the six pushing assemblies is equipped with two 250-ton jacks for a total pushing capacity of 3000 tons. The stroke on each jack is 1200 mm. During a typical pushing cycle, the jacking assemblies advance in an inchworm progression. Each pushing assembly anchors itself to the skid beam by engaging four 370-ton capacity gripping jacks mounted on the side of the pushing assembly. Following completion of each push cycle, the gripping side jacks are released and the ram on each pushing jack is retracted which pulls the jacking assembly ahead to the next push point.

Two guiding devices are mounted on the underside of the tunnel element, one on the leading edge of the tunnel segments and one on the trailing edge. The devices consist of roller assemblies attached to 30-ton jacks which begin applying force if the tunnel strays more than 25 mm from true alignment. The 6 pushing assemblies and 2 guiding devices are controlled and monitored from one central station.

4. Launch Basin and Gates

To allow for the selected construction method a launch basin is created east of the tunnel factory. The launch basin is comprised of a shallow basin at 2 m above sea level and a deep basin with bottom at -10. The basin is surrounded by bunds to the level +10.2. The entrance or the sliding gate forms the closure towards the factory. The exit or floating gate forms the closure at the seaward end of the basin.

4.1 Bunds

The bunds or earth dykes which surround the basin are made up mainly of existing material from the excavation for the deeper part of the basin.The ground conditions generally consist of 7-8 m of fill, 1 m of sea deposits, 5-6 m of glacial deposits mainly clay till overlying the Copenhagen Limestone.The limestone is heterogeneous and sometimes has high permeability. Faces of the bunds are lined with 5m blanket of compacted clay to minimise seepage out of the basin and porepressure build-up in the dykes. Compaction of dykes and blankets are made in layers at optimum water content to a specified compaction degree.

4.2 Sliding gate

The sliding gate, approximately 100 m long, consists of reinforced concrete frames at 5.0 m nominal centres with a steel plate as watertight skin.
The task for the sliding gate is to form part of the closure of the basin during filling to + 9.9 without any leakage. After lowering of the water level the gate is moved to its parking area on an extended foundation giving place for the tunnel elements to be launched on the skidding beams.
The foundation of the sliding gate consists of reinforced concrete slab on vertical and inclined piles driven to refusal in the limestone. Steel piles were chosen with respect to the risk of excessive pilebreakage during driving. Dynamic measurements of pile performance and integrity were done. Deformations of the foundation should be small due to connection to skidding beam and due to loading/unloading effects.
Below the foundation a vertical sheetpile wall is installed in order to decrease waterpressure uplift forces as well as seepage below the foundation.Drainage layer and relief wells at 10m centers are provided to draw off any seepage before pressure can build up. Moreover, water pressure in the limestone under the sliding gate will be monitored to detect any risk of decreased safety.
The sliding gate with a weight of approximately 2000 tonnes are moved by the same pushing jacks as the tunnel elements.
Watertightness between the gate and the foundation is provided by a rubber membrane seal
The gate is connected to sheetpile wall abutments. Sheetpile wall is extended into the earth dykes to prevent seepage. Section of the sliding gate is shown in Figure 4.

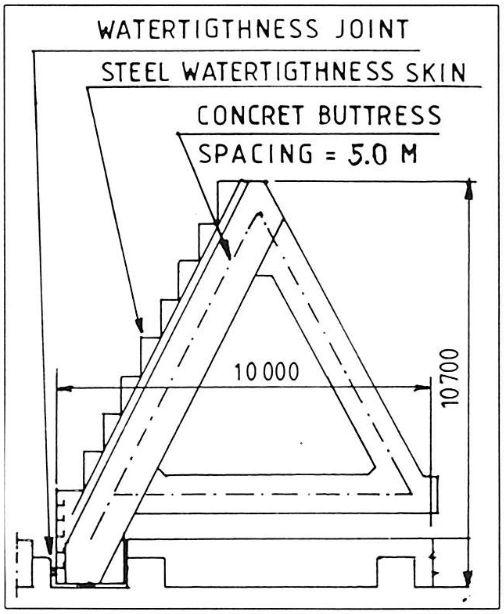

Figure 4. Sliding gate cross section

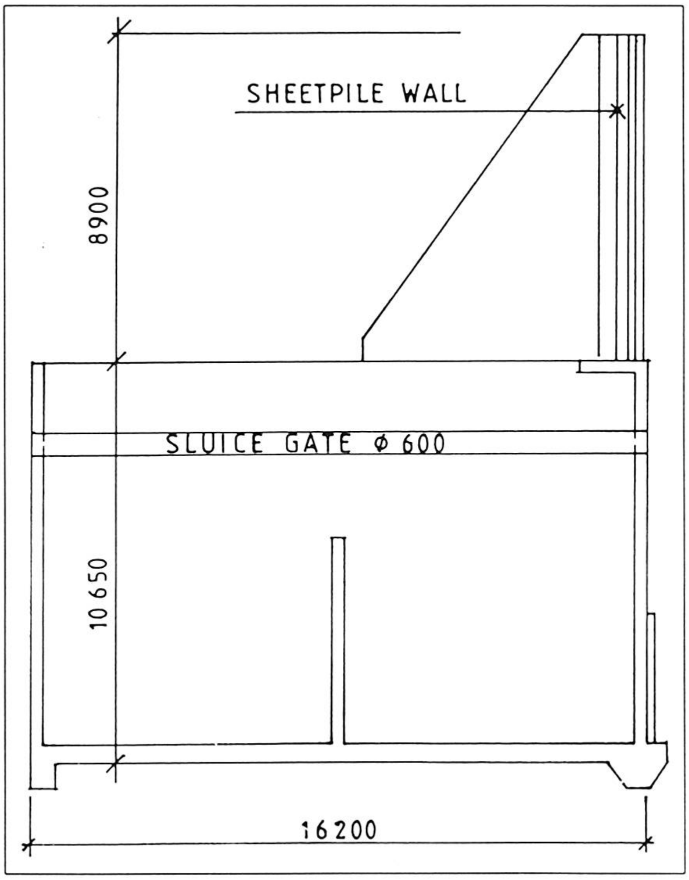

Figure 5. Floating gate cross section

4.3 Floating gate

The floating gate approximately 45 m long consists of concrete cells from bottom level at - 9 to level + 1. A steel sheetpile wall forms the part up to elevation + 10.
The floating gate is used to close off the seaward opening of the launch basin to enable the waterlevel in the basin to be raised for transportation of tunnel elements to the deeper part. After this operation and after lowering of the water level the floating gate is opened again and taken away to its parking position.
The gate is put on a RCC (Roller Compacted Concrete) foundation down to the Copenhagen Limestone. A grout curtain is performed to control seepage and uplift forces below the foundation.
Closing operation starts with transportation of the floating gate from its parking position with two tugs. The gate is positioned in its immersion position with winches and a Multicat and then lowered on its foundation by filling four ballast tanks. After having reached final position the gate is completely filled with water. Opening of floating gate is executed by emptying the ballast tanks. Section of the gate is shown in Figure 5.

5. Element outfitting and launch basin operations

5.1 Outfitting

After all the eight segments forming a tunnel element has been cast the whole element is pushed 100 m along the skid beams to the shallow part of the launch basin.
The element outfitting operations then take place in this part of the basin before floating and include installation of :

- Temporary ballast tanks
- GINA immersion seal
- Temporary bulkheads
- Temporary post-tensioning
- Some towing and immersion equipment
- Some ballast concrete
- Some mechanical and electrical items.

The main ballast tanks consist of five partitions (or bulkheads) dividing the southern motorway bore into four tanks. The bulkheads are sized to facilitate dismantling and transport by truck.
A homogeneous rubber softnose GINA seal is installed surrounding the primary end immersion joint. The GINA seal is bolted to the steel end frame of the immersion joint. In order to protect the seal during handling in the basin and towing a wooden cover is fixed.
Steel bulkheads are mounted at the conclusion of the outfitting operations in both ends of each tunnel bore before floatation. The bulkheads are re-useable and equipped with watertight doors for access to the immersion joint area between two bulkheads.
Temporary post-tensioning cables are installed to act during towing and immersion. The posttensioning consists of 24 tendons placed in the bottom slab and the roof slab. After backfilling of the tunnel elements the tendons are cut.
For mooring, towing and immersion the tunnel elements are equipped in the dock basin with among other things 600 kN bollards placed on the tunnel roof and 4500 kN immersion lugs for connecting the wires from the lowering system on the immersion pontoons.

Ballast concrete is calculated and cast to a quantity which guarantees level flotation. With empty ballast tanks the elements will have a minimum freeboard of approximately 30 cm in fresh water and some 50 cm in salt water.

Marine outfitting are done at the dolphin moorings just outside the basin such as:

- Immersion pontoons with associated winching system
- Guiding system on the roof of the tunnel element comprising a guiding nose, male part on the primary end and a receptor, female part on the secondary side of the element
- A laser tower and a survey tower carrying the survey installation
- A command tower carrying the main operators cabin or command unit with monitors for position and winch forces

5.2 Launch basin operations

Following the outfitting operation the floating and the sliding gates are closed and the basin is flooded to an elevation of 9,9 m above sea level by pumping sea water over the bund into the basin. Four diesel driven water pumps with a total aggregate capacity of 17.000 m^3/hr are used. The maximum allowable filling rate , from geotechnical boundary conditions, is 0.3 m/hr. The filling operation takes around 50 hours.
Before flooding of the basin the tunnel elements have been equipped with a system of fenders, wires and struts. The system keeps the elements at a constant distance and prevents relative movement during float up.
As soon as the tunnel elements starts floating the trim of the element is monitored by survey. Once full floatation is reached trim is adjusted till even trim .Acceptable trim deviations at this stage are less than 5 cm over full width as over full length.
After floatation both elements are moved together to the deep section of the basin. Six winches located on the bunds are required to achieve full travel. Four winches of 25 tons are employed in longitudinal direction, two pulling and two holding, plus two 10 tons winches in transversal direction.
After the tunnel elements are completely shifted to the deep part of the basin the waterlevel lowering commence. The sluice gates at the floating gate are partially opened so that basin water runs into the sea, resulting in a lowering speed of the basin waterlevel not exceeding 0.3 m/hr. When the basin waterlevel reach the sea level the floating gate is opened and towed to a mooring facility in Copenhagen North Harbour. The tunnel elements are towed out of the basin . During this operation the immersion pontoons are installed over the elements. The tunnel element is finally moored against dolphins with fenders at the south side of the channel to the basin. The marine outfitting is here completed before the element is towed to the tunnel alignment in the Drogden channel between the artificial peninsula outside Copenhagen Airport and the artificial island.

6. Quality assurance system

6.1 Quality system

Øresund Tunnel Contractors I/S (ØTC) has established and implemented a Project Quality Programme for the Øresund Tunnel Project. The Programme is based on the Quality System Requirements included in the Contract for the Tunnel Project, which in turn are based on the EN 29001 Standard.

The Programme covers ØTC quality management of the three areas :

- Permanent Works and Temporary Works (those affecting the quality of the Permanent Works)
- External Environment
- Working Environment (Health and Safety)

The Programme is documented by a Quality Manual including General Procedures and a number of Quality Plans. The Quality Plans cover planning, implementation and performance of the quality assurance and quality control activities specially adapted to the relevant activity.
The detailed planning of the work has been documented in a great number of Work Procedures describing "How to do" and "Who will do". The procedures are prepared generally by the site engineer and reviewed by the designer and quality assurance staff before approval by the appropriate manager.
The Work Procedure generally includes an Inspection & Test Plan (ITP), which clearly specify inspection points, inspection methods, extent of inspection, criteria for acceptance and schedule for documentation. The ITP also identify the specific stages of the construction activities - witness points and hold points - where the Client and/or ØTC Quality Control staff shall be called for witnessing and/or approval of the works prior to the execution of the following activities.
The quality system is subject to regular quality audits, both internal and by the Client, to ensure its continued adequacy and implementation.

6.2 Quality assurance/control organisation

A Quality Department is set up headed by the Project Quality Manager. The department consists of three sections: Quality Assurance, Quality Control and Document Control.
In the Quality Assurance Section the duties of three Quality Engineers include to prepare and review quality documents and to perform quality audits both internal and external. A major part of the departments duties is in the selection of suppliers and sub-contractors by review of the existing quality systems and visits to manufacturing plants to ensure consistent supplies.
The Quality Control Section headed by the Works Quality Manager includes three Quality Control Engineers working on respective sites. They are assisted by seven Quality Inspectors who perform the daily independent quality control on a random basis and also control and file all the quality records from site and suppliers/sub-contractors. The primary responsibility for quality control falls on the site management.
The Document Control Section headed by the Document Controller includes four Document Clerks using a computerised document management system and performing copy services.

7. Conclusion

The Øresund Tunnel project has shown that the teaming of design services to concurrently develop the design and construction methods for a structure is an effective way of improving the structure design while reducing construction cost.
This teaming effort on the Öresund Tunnel has successfully demonstrated that the incremental cast and launch technique is a viable construction option for long immersed tunnels. It has also shown that the application of full-section casting on an elevated form surface is an effective way of eliminating the high cost of concrete cooling in tunnels.

Deep Soil-Cement Mixing for CA/T Project

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Summary

In order to construct one segment of a complex cut & cover tunnel network deep into soft clays, large scale insitu ground improvement installed via deep soil-cement mixing (Deep-Mix Method or DMM) has been initiated. DMM emerged ahead of conventional cofferdam methods, based on design, construction, and economic considerations.

This project synthesized the resources of past practice and conventional wisdom with new technologies, to push the USA state of practice into accepting another viable design & construction expedient: the Deep-Mix Method. Both excavation support systems and permanent foundations for structures can be facilitated by this technology. The analytical and analogous assumptions which were crucial in advancing this application are described.

1.0 Introduction

The Central Artery/Tunnel (CA/T) Project in Boston, MA is the largest and most complex urban highway project undertaken by the United States of America (USA). The multi-billion project will replace a forty+ year old elevated Central Artery (I-93) viaduct with a modern underground expressway and extend the Massachusetts Turnpike (I-90) to Logan International Airport, all through an arrangement of tunnels which include cut & cover, immersed tube, and jacked tunnel elements.

One important aspect of the multi-billion dollar CA/T project has been the I-90 extension. A section of this extension must cross through a channel (Fort Point Channel), which is underlain by deep, soft blue clay (refer to Figure 1). The site is surrounded on three sides by 1) an active rail facility owned and operated by AMTRAK and the Massachusetts Bay Transit Authority, 2) the largest private employer in Massachusetts; Gillette, Inc., and 3) the northeast distribution center for the United States Postal Service. Disruptions in service caused by construction to any of these abutters would be troublesome. Hence, the impact of construction-induced ground movements had to be maintained within allowable limits.

This section was complex due to the alignment of the tunnels cutting deep and wide into the soft BBC, all the while bordering the land/marine interface. This siting of the tunnels precipitated major concerns on global stability and heave during excavation; net lateral-load tunnel foundation capacity; and lateral ground movements, both during and post-construction. Several construction methods received attention during conceptual design, essentially exploring conventional wisdom: 1) full and/or partial channel bypassing & backfilling, with conventional cut & cover construction; 2) full and/or partial marine cofferdam excavation. These types of systems failed to instill confidence that the major concerns cited above could be manageable.

Recognizing that delays would cause the entire project to experience significant delays, a team of experts was mobilized to study the problem area. During those discussions, the idea of utilizing DMM was conceived. B/PB and the SDC developed a preliminary design and construction strategy, including construction cost and schedules. Global stability, heave, and ground movements all appeared manageable, and confidence levels were reinvigorated. Successfully balancing all of the concerns, B/PB recommended that DMM be pursued as the solution to the myriad problems of this site. It was further recommended to divide the area into two construction contracts; the first would stabilize the ground with DMM; the second would build the cut & cover tunnels in the stabilized ground - this would make the Project's opening-day schedule once again viable.

This paper discusses aspects of design that were crucial in providing the support conditions and constructibility necessary to facilitate follow-on cut & cover tunnel construction. There are many facets to this project that pushed the state of practice beyond the limits of timid imagination. This case study represents the first-ever DMM use in the USA to serve as both an excavation support system and a permanent foundation for cut & cover tunnels.

The owner of the project is the Massachusetts Highway Department (MHD); the joint venture of Bechtel/Parsons Brinckerhoff (B/PB) serves as Management Consultant and performed preliminary design; the joint venture of Maguire/Harris was the Section Design Consultant (SDC), and prepared plans and specifications for the construction contract.

2.0 Site Conditions & History

Soil stratigraphy for the site, from bedrock to ground surface, is composed of glacial till, marine clay, organic silts, and man-made fills. The bedrock typically consists of moderately to severely weathered and kaolinized argillite, weathered to depths of 7.6m or more. The dense glacial till deposit varies from 1.5m-6.1m thickness. The most influential stratum is the deep deposit of BBC. This deposit is approximately 22.9m thick, and varies in shear strength from 38kPa to 48kPa. The organic silt deposit has a shear strength of approximately 5kPa, and varies in thickness between 3.0m to 6.0m. Significant areas of these naturally occuring sediments were backfilled during colonial times to create land suitable for human habitation and development. These fills are approximately 7.6m thick.

Groundwater conditions of the site include a lower, confined aquifer in the glacial till, and an upper, unconfined aquifer in the man-made fills. There is no apparent connection between these two water sources - piezometric levels in the deep glacial till deposits are lower than the upper backfills. The Fort Point Channel (FPC) traverses these fill areas.

3.0 Soil-Cement Design

3.1 Background

Once DMM was selected to be pursued, B/PB and the SDC met frequently to partner through design criteria issues, since DMM has no established US code of practice. B/PB and the SDC advanced the analogy of utilizing DMM technology to create a series of "weak-concrete" shearwalls, to serve as buttresses capable of withstanding insitu lateral soil pressures with little or no movement, while relieving the tunnels from long-term differential ground movements. The "aggregate" of the "weak-concrete" would be the insitu soils. An unconfined compression strength increase of two-orders of magnitude relative to the unconfined compression strength of BBC would be facilitated by this process. The layout was chosen based on control of external (global) and internal (stress) limit states. This application of DMM was analyzed, designed and detailed to function as though it was a structural element, although discussion continues on whether it is a ground-improvement technique or a man-made underground structure.

Typical equipment necessary to install DMM is shown in Figure 2. The rig is outfitted with a system of inter-related shafts, each possessing discontinuous, alternating auger flights and mixing paddles. Soil-cement installations initiate from the ground surface. The penetration stroke is generally utilized to fluidize the soil-column in preparation for cement-mixing during the withdrawal stroke. The withdrawal stroke introduces the majority of a proportioned cement grout at the cutting head. The cutting head has the capacity to reduce the soils to less than 25.4mm particle sizes when RPM and advancement/withdrawal rates are optimized for the soils to be encountered. The withdrawal stroke creates the contiguous soil-cement element, mixing the fluidized soil-column with grout. Succesively advancing this multiple-shaft system along a line of progression that repenetrates the outside bores of adjacent soil-cement elements creates a continuous soil-cement shearwall. Figure 3 describes the advancement sequence necessary to create a shearwall. Excavation support may be expedited by installing steel wide flange shapes into the fluid soil-cement boreholes, which can be designed as soldier-piles. This aspect of DMM use is not covered as part of this paper.

A laboratory-study of BBC mixed homogeneously with varying amounts of grout found that an unconfined compressive strength (q_u) of 2.1MPa was reasonably achievable given the site conditions, available equipment, and preliminary treatment pattern geometric requirements. As for cement materials, Portland Cement Type II was specified, since the insitu soils possessed a moderate risk for sulfate attack. The design then proceeded based on the analytic assumption that soil-cement is an elastic, brittle material. For internal stability, limit states and their allowable stresses were then chosen to keep total material stresses within the elastic range.

The limit states and their allowable stresses received much attention during development of the project. Compression, tension, direct shear and diagonal tension were considered. A factor of safety of 2.0 for the long-term compression allowable stress, with a one-third allowable stress increase for short term (construction) cases was selected as the basic parameter, since compression-strength could be used as a primary quality assurance parameter.

Evaluation of the available literature regarding Japanese experience concluded that soil-cement installed via DMM has been designed similar to unreinforced concrete design practice in the

USA. The basic "long-term" allowable stresses for which the project was designed were as follows:

$$q_{ac} = 0.5q_u$$
$$q_{at} = q_{adt} = -0.05q_u$$
$$t_{ad} = 0.25q_u$$

where q_u = specified minimum compressive strength of insitu soil-cement
q_{ac} = allowable compressive stress
q_{at} = allowable tensile stress
q_{adt} = allowable diagonal tension stress
t_{ad} = allowable direct shear stress

3.2 Model Development

A number of representative cross-sections of the tunnels were studied via a series of manual computations. Construction sequences for the tunnels were assumed to be similar to cut & cover construction, with the exception that the ground had been stabilized with soil-cement shearwalls. These shearwalls were studied to provide the requisite lateral and vertical stiffness to serve as permanent foundations for the tunnels.

Transverse to the tunnel alignment, a unit-width design strip was assumed. Loadings on an averaged soil-cement buttress were proportioned according to the layout geometry and influence width. The initial layout reflected the geometry of the future tunnels. Soil-cement shearwalls were then located to traverse the width of the tunnels, similar to timber ties beneath a railroad track. This layout geometry was then optimized based on evaluation of the external and internal stability analyses outlined below.

3.3 External Stability Analysis

The first phase of design focused on external stability. The limit states here were two-dimensional analyses of sliding and overturning. This process attempted to close-in on the treatment pattern utilizing a series of traditional "retaining-wall" calculations to estimate the length of the buttress required to create a reaction along the soil-cement/till interface which was within the middle-third. This length was then compared with the future tunnel network width requirements. It was discovered that the width required was generally well in excess of the tunnel geometry, since shearwalls need to be wider than they are tall for efficiency. As it turned out, the length of the buttress which was required provided another construction expedient - a 100% (see section 4.6) soil-mix cofferdam to install struts against during tunnel excavation sequences, as shown in Figure 4.

Conventional Japanese practice of utilizing active/passive conditions for external stability analysis and at-rest conditions for internal stability were generally followed, with the exception that external stability analysis assumed active/at-rest rather than active/passive conditions. This was prudent considering that large ground movements to mobilize passive resistance were contrary to mitigating construction and permanent lateral ground movements.

These stability analyses were performed assuming geostatic earth pressures proportioned to the shearwall geometry. The lateral earth pressure coefficients used for analysis appear below:

Fill:	$K_{ACTIVE} = 0.33$	
Organic Silt:	$K_{ACTIVE} = 0.40$	
Marine Clay:	$K_{ACTIVE} = 0.46$(drained)	Driving side
	$K_{AT\text{-}REST} = 0.75$(oc)	Resisting side

where: oc = over consolidated

Note that some relief over the $K_{AT\text{-}REST}$ condition was recognized for the short-term (construction) condition on the buttress-driving load. It was recognized that the construction-method would partially relieve the insitu $K_{AT\text{-}REST}$ pressures. These pressures were assumed to climb back to $K_{AT\text{-}REST}$ for the long-term condition. Stability against overturning and sliding were generally governed by a construction case (open excavation - minimum weight, minimum available base shear resistance).

3.4 Internal Stability Analysis

The next phase of design development then focused on internal stability, or stress limit states. Analysis of the soil-cement shearwalls was performed using GT-STRUDL, a microcomputer-based software program. The analysis required the nonlinear and Finite Element (FE) capabilities of GT-STRUDL; the till reaction was assumed to be a nonlinear, compression-only response. Two-dimensional elastic plane-stress elements were used as the basis of evaluating the behavior of the soil-cement shearwalls.

For output, GT-STRUDL allowed the visual presentation of graphical images to generate colored plots of direct and principal stresses. The analysis was then iterated-on each time the stress plots were interpreted. The design process included both "thickening" the FE mesh where stresses were excessive, and "thinning" where stresses were low.

These stability analyses were performed assuming geostatic earth pressures proportioned to the shearwall geometry. The lateral earth pressure coefficients used for analysis appear below:

Fill:	$K_{AT\text{-}REST} = 0.50$	
Organic Silt:	$K_{AT\text{-}REST} = 0.60$	
Marine Clay:	$K_{AT\text{-}REST} = 0.60$(nc), 0.75(oc)	Driving side
	$K_{AT\text{-}REST} = 0.75$(oc)	Resisting side

where: oc = over consolidated
nc = normally consolidated

Elastic properties for the FE analyses:

Unconfined compression strength, $q_u = 2.1$MPa
Shear strength, $S_u = 1.03$MPa
Poisson's Ratio, $v = 0.25$

An important parameter necessary to evaluate the elastic behavior of the buttress designs is the material property of Elastic Modulus, E. During design, insufficient information was available to determine the actual value as soil-mixing experience with BBC was limited. Borrowing from information obtained from a DMM research-related trip to Japan, it was found that typically the Elastic Modulus is in the range of $350q_u$-$1000q_u$. An assumed value of $300q_u$ was used, with the understanding that a lower bound would provide conservative estimates of displacements. There would be no effect on stresses by this presumption, since the elastic plane-stress FE analyses would preserve the relationship between strain and displacement.

3.5 Boundary Conditions

The glacial till layer was modeled in two directions: a) Vertically, bilinear-stiffness compression only springs based on subgrade modulus values proportioned to a load/displacement relationship were utilized. This bilinear stiffness approach allowed the soil-cement vertical base reactions to be redistributed as the till gets "softer" with increasing bearing pressure; b) Horizontally, the reaction of the soil-cement buttresses engaging the till was proportioned with elastic springs, at locations where the vertical springs demonstrated compression reactions. Since the overall loadings on the soil-cement buttresses have a calculated net overturning force, the buttresses analytically rotate, theoretically disengaging "tensile" springs, and activating "compression" springs, as in the design case of a rigid footing losing contact with soil due to overturning loads. This principal was extended to similarly release the horizontal reaction, given the lack of "confinement" by a bearing pressure. This is shown conceptually in the "base reaction" in Figure 4.

3.6 Interpretation and Optimization

In order to optimize the DMM treatment pattern, both an "open-cut" construction model and a "final-grade" FE model was necessary to study the distribution of material stresses. As patterns were changed to optimize stress distributions, external stability analyses were updated to assure that global stability and heave between shearwalls remained acceptable.

For construction, a unit buttress was assumed to consist of three (3) soil-cement shearwalls, or 3 rows of interlocked, ground treatment. This initial assumption was then called "38% DMM" (percent DMM is the amount of treated ground strips in proportion to the total area treated and untreated). The actual physical spacing of the buttresses (shearwalls) are governed by the auger-diameter of the DMM equipment. For the tunnel geometry and presumed lateral loadings, a shearwall layout of 38% (3 rows treated out of 8 rows possible) was necessary for "equivalent timber ties" vertically under the tunnels. Due to the analytical "rotation" of the buttresses, the shearwall base reaction resembles a classic retaining wall footing contact-pressure diagram. The pattern of ground treatment responds by increasing the coverage at the analogous "toe" to 100% (8 rows out of 8 rows), via a 63% (5 rows treated out of 8 rows possible) "reinforcing" between 38% and 100% coverage areas. The 100% zone serves both to dissipate stresses and provides a "heave" cut-off barrier.

4.0 Quality Assurance

Coupled with design, A Quality Assurance/Quality Control program was necessary to assure compliance with the intent of the design. Three important checks on construction are measured:

1) Unconfined compressive strength (q_u): A minimum average compressive strength was specified, and fluid samples of the soil-cement elements are taken at various depths within the element. A 56-day strength was specified rather than the typical 28-days, in order to reduce the cement dosages required to achieve the minimum strength. Similar to concrete practice, there is a 10% failure criterion assuming strengths lie in a normal distribution. Also, the cylinder strengths are tested on a stress-controlled device, on 152.4mmx304.8mm soil-cement cylinders, which have been moist-cured at 38 degrees Celsius. These curing conditions were assumed to be typical of insitu conditions, and are mirrored in the laboratory.

2) Verticality: The specifications require a 2.0% tolerance on verticality for each soil-cement element. Inclinometer readings are taken in one of the outside bores of the auger-shaft equipment at regular intervals. Verticality is used as a convenient measure of continuity in the shearwall. The real concern is loss-of-contact with depth in the ground of the shearwall, as boreholes may drift apart with depth. The auger-systems are very-stiff in the in-plane direction, but weaker in the out-of-plane direction. Repenetrating the outside auger-boreholes effectively eliminates in-plane drift from becoming a problem, but out-of-plane drift can lead to loss of continuity with depth.

3) Engagement of Till: In order to develop the lateral-load capacity of the shearwalls, they must engage the firm till stratum. The depth to the till layer was compiled into contour maps from the multitude of available boring logs. The plans call for a minimum depth drilled & mixed into the till of 0.3m, determined by drilling an additional 0.3m after a significant increase in penetration resistance is noticed by the rig operator on his instruments. Knowing the anticipated depth to the stratum was a necessary protocol since an obstruction encountered during the penetration stroke might falsely suggest encountering the firm stratum.

5.0 Design Implementation

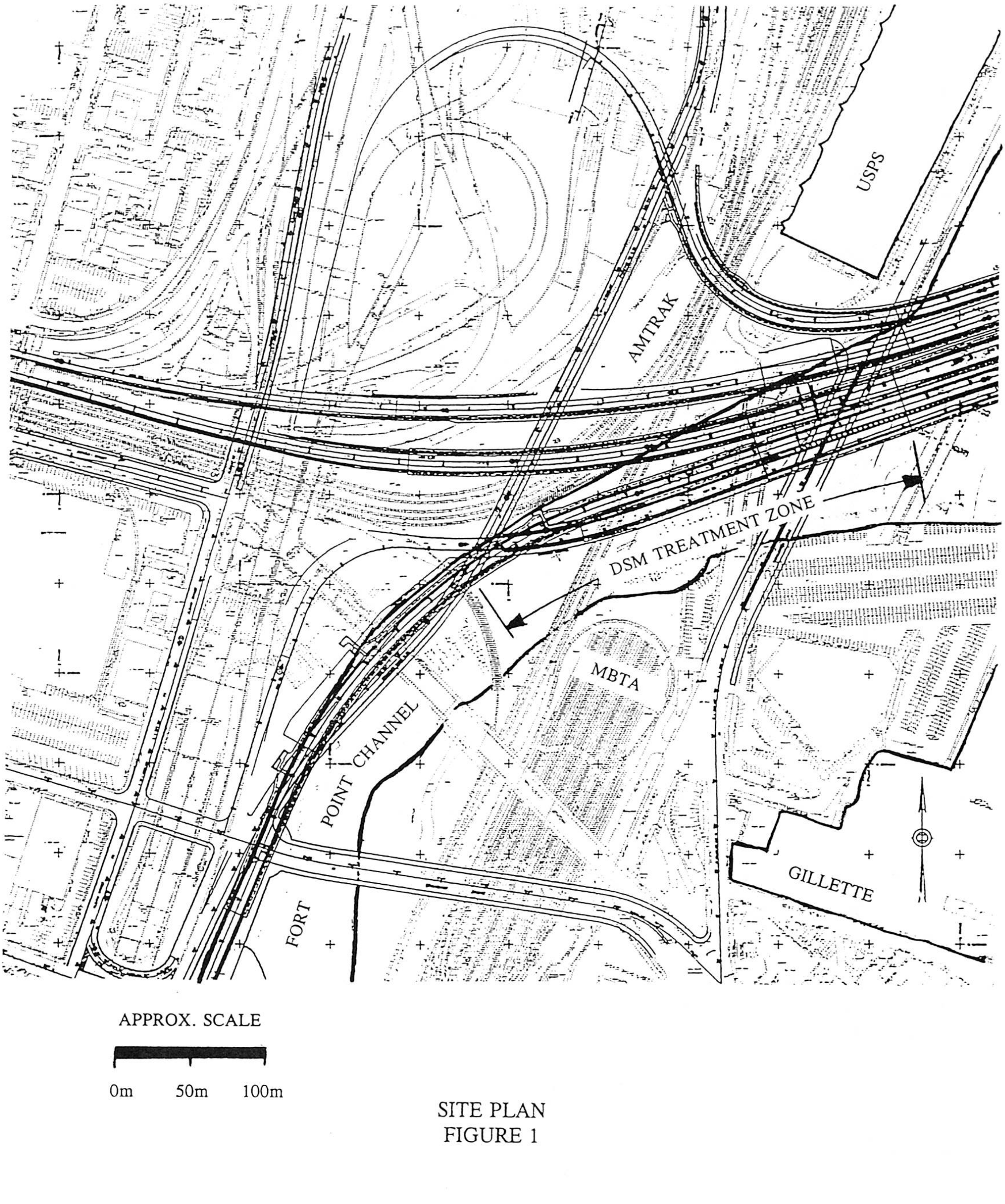
Auger diameters were given a range on the drawings, so that a proprietary system would not be specified. This allowed maximum competition between bidders. The construction contract was estimated to involve the installation of 17,000 cubic meters of soil-cement, with land-based and water-based operations. The Engineers' Estimate was $152 million, and the contract was awarded to the low-bidder at $132 million. The second-lowest bid arrived at $165 million. Thus, the results of competition were readily apparent. Construction has been scheduled to span a total of 42 months. Installation of the DMM accounts for approximately 26 months of the contract work.

6.0 Conclusion

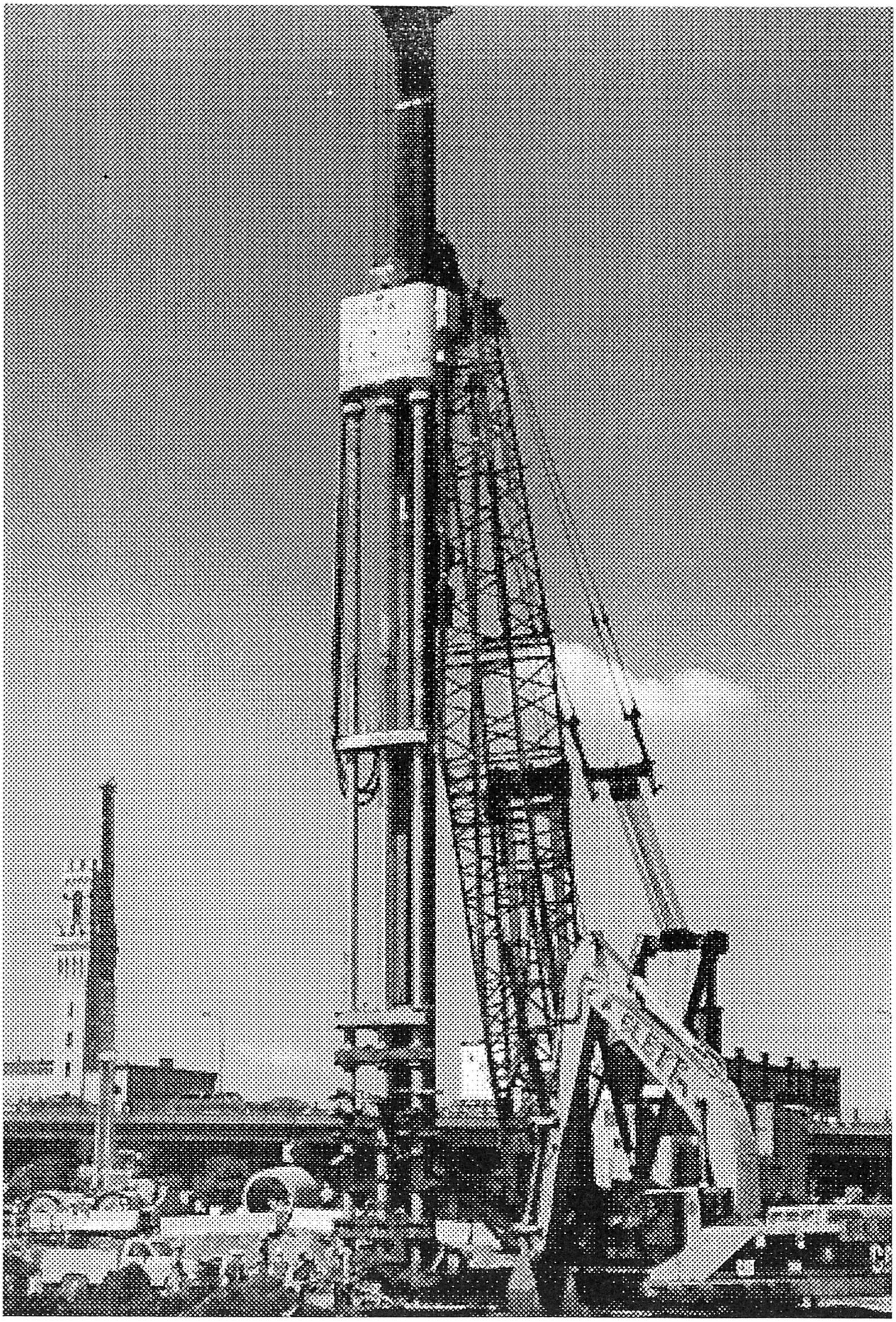
This project synthesized the resources of past practice and conventional wisdom with new technologies, to push the USA state of practice into accepting another viable design & construction expedient: Deep Soil-Cement Mixing. This technology can serve as both an excavation support system and as a permanent foundation for structures. The analytical and analogous assumptions which were crucial in advancing this application have been described.

Acknowledgements

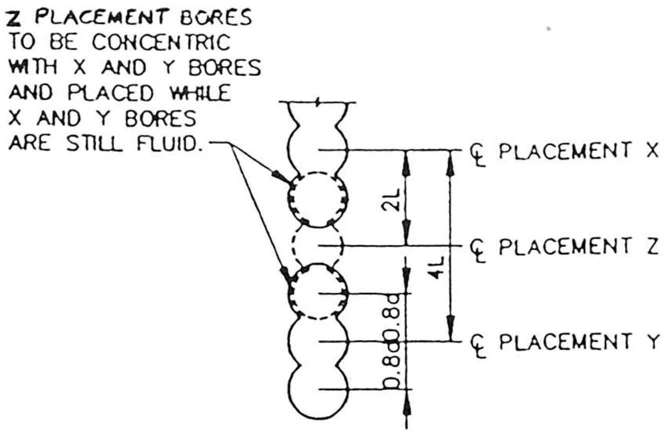
The authors wish to thank the Massachusetts Highway Department, the Federal Highway Administration, and Bechtel/Parsons Brinckerhoff for their support in advancing DMM technology, and for sharing its development with the engineering profession.



SITE PLAN
FIGURE 1



TYPICAL DSM EQUIPMENT
FIGURE 2



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LONGITUDINAL PROGRESSION
MULTIPLE AUGER
(3 AUGER SYSTEM SHOWN)

REPENETRATION SEQUENCE

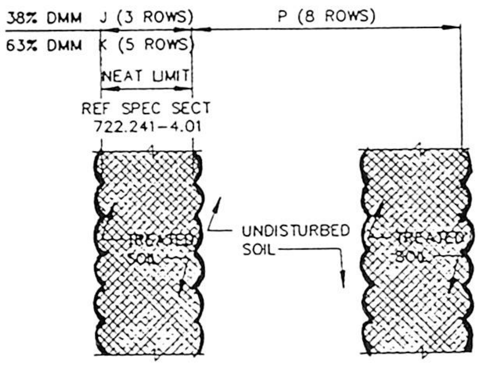


TABLE 1								
AUGER CONFIGURATION	DIAMETER RANGE (FT–IN)	J	K	P	T	L	W1	W2
SINGLE	4'–0" ≤ d ≤ 6'–0"	2.3d	3.7d	5.8d	0.7d	0.5d	3.2d	2.8d
SINGLE	6'–0" < d ≤ 8'–0"	2.3d	3.9d	6.4d	0.8d	0.7d	3.8d	0.0d
DOUBLE	4'–0" ≤ d ≤ 6'–0"	2.3d	3.7d	5.8d	0.7d	0.5d	3.2d	2.8d
DOUBLE	6'–0" < d ≤ 8'–0"	2.3d	3.9d	6.4d	0.8d	0.7d	3.8d	0.0d
MULTIPLE •	2'–4" ≤ d ≤ 3'–6"	2.2d	3.8d	6.4d	0.8d	0.8d	2.8d	24'–2.8d

LEGEND:

d = DIAMETER OF THE SINGLE AUGER/MIXING PADDLE ASSEMBLY.

J = EFFECTIVE WIDTH FOR 38% COVERAGE AREAS.

K = EFFECTIVE WIDTH FOR 63% COVERAGE AREAS.

P = SPACING OF TREATED SOIL ROWS.

T = TRANSVERSE SPACING BETWEEN PRIMARY AND SECONDARY ROWS.

L = LONGITUDINAL SPACING BETWEEN ADJACENT AUGERS IN A ROW.

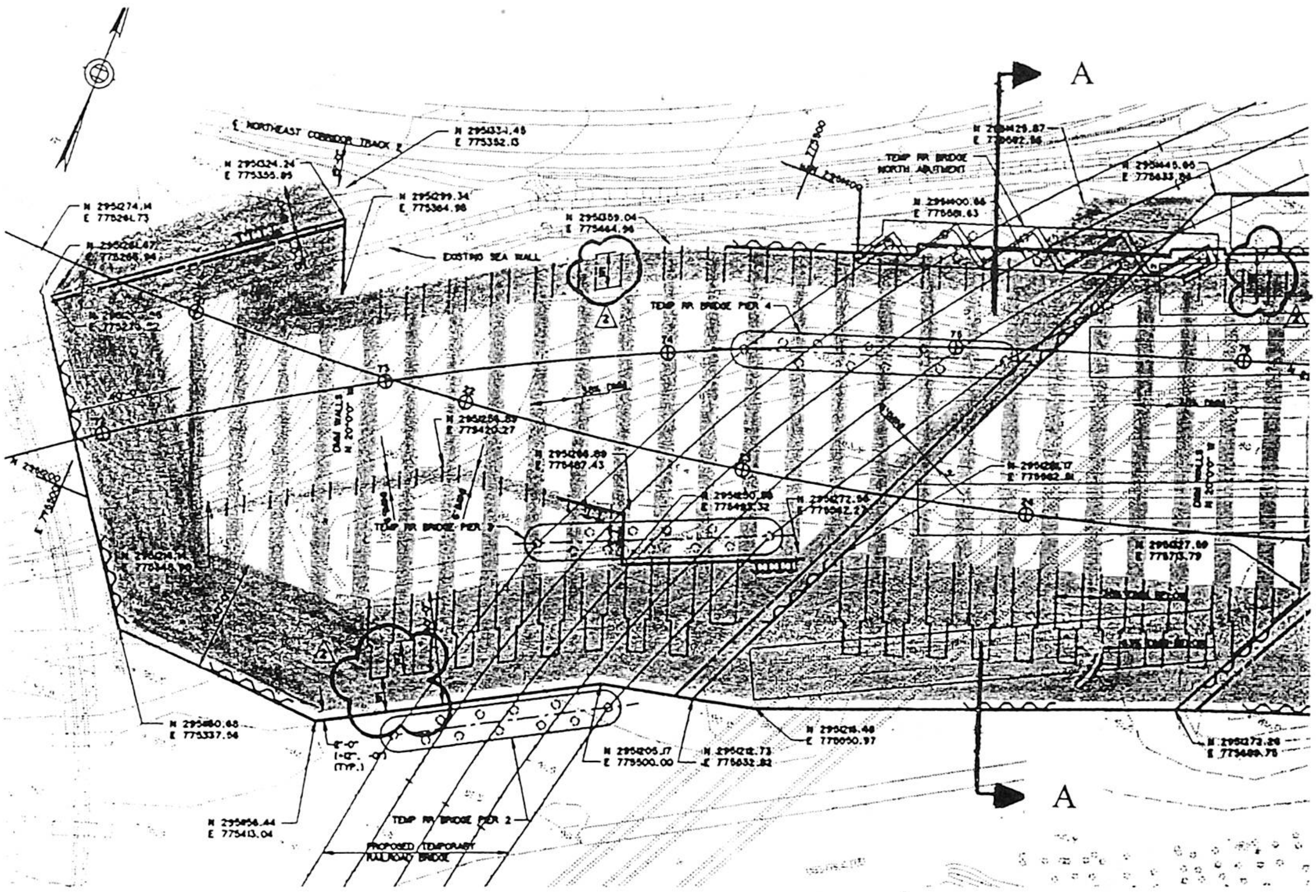
W1 = WIDTH OF 100% DMM TREATMENT AREA.

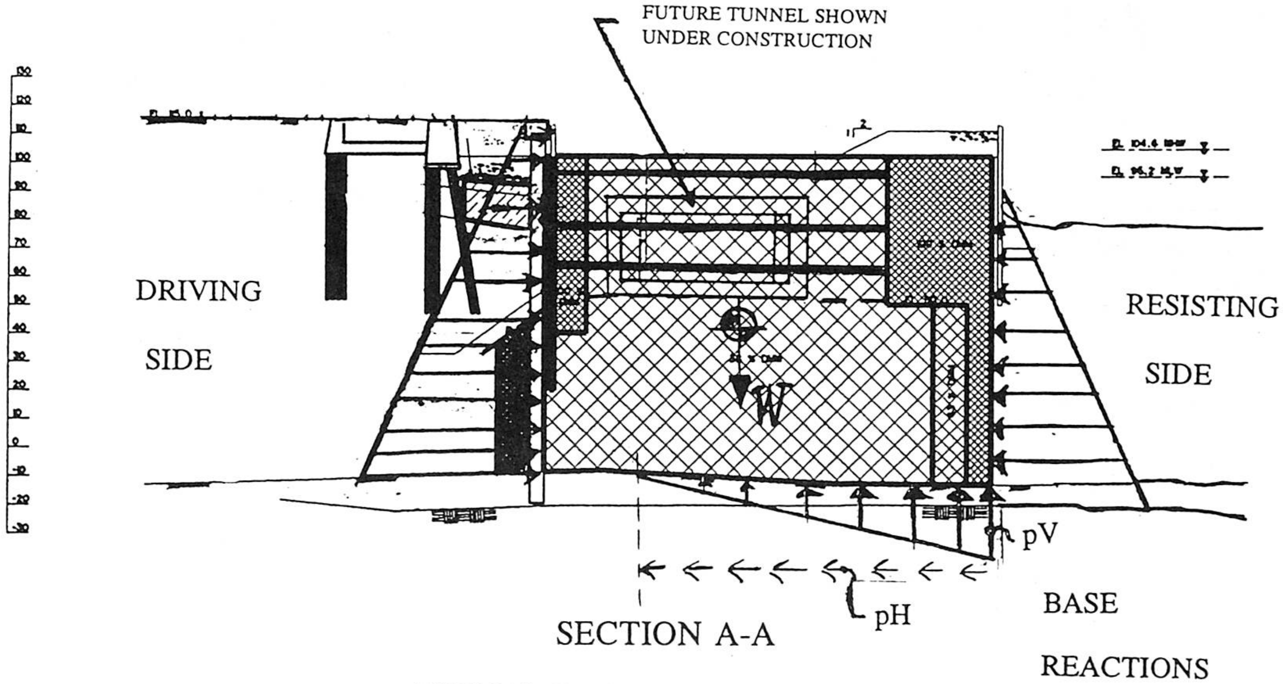
W2 = WIDTH OF 63% DMM TREATMENT AREA.

DETAIL 4
TRANSVERSE TREATMENT PROGRESSION FOR
38% AND 63% DMM COVERAGE AREAS

SHEARWALL LAYOUT DETAILS

DSM INSTALLATION DETAILS
FIGURE 3





TYPICAL PLAN & SECTION OF DSM
FIGURE 4

DISCRETE SUPPORTS IN SUBMERGED TUNNELS.

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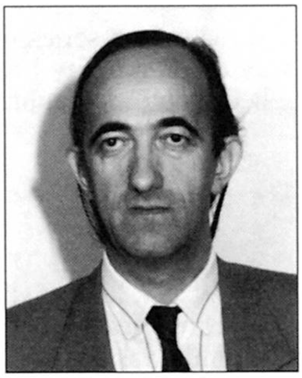
Technical Manager
IMEBISA
Bilbao, Spain



Joseramón Madinaveitia, born 1943, obtained his civil engineering degree in 1968 and his history degree in 1985, in charge of the Bilbao Metro project since 1991.

Summary

The Bilbao metro's underriver crossing at Olabeaga was executed using the immersed tunnel technique. The elements of the tunnel were prefabricated in a provisional dry dock which formed part of the future land tunnel. The presence of competent rock at bed level enabled sand injections to be replaced by discrete concrete supports, acting as keelblocks. Microconcrete-injected sacking bags were used to make the required bed adjustments. River traffic immediately above the tunnel, including vessels up to 10,000 DWT, remained uninterrupted throughout the operations.

1. Background

When the Basque Government decided, in 1989, to begin work on the Bilbao Subway, one of the major technical challenges involved was how to overcome the barrier of the river at Olabeaga, used by vessels of more than 10,000 DWT.

The engineering company which produced the project analysed a number of possibilities, including modifying the route to pass under the bed and boring the entire tunnel in rock. This solution was discarded because it meant that the nearest stations would have needed to be much deeper underground, which was felt to be a negative move as far as attracting passengers was concerned.

The submerged tunnel solution was retained and to develop this method, new in Spain, the engineering company sought advice and support from an English consultant, working on the Conwy project at the time. The design included the habitual technique of sand injections under the structures once they had been put into place in their final position.

When the winning construction firm started work, it sought the advice of a Danish firm with long experience in sand injection processes under submerged tunnels.

2. The Concept of Discrete Supports

However, the geotechnical campaigns carried out prior to the design phase detected the presence of the rock ceiling at levels similar to those planned for the submerged tunnel supports.

With the quality of the rock confirmed, after dredging the fragmented, decomposed metre at the top, the need to rethink the support system became clear.

The idea was to transmit work loads to the rock via a series of discrete supports in concrete to replace the layer of injected sand. This way the continuity between the rigidity of the concrete structure of the tunnel units and the rock bed would be maintained without the need to insert a looser element like injected sand.

3. Construction Area.

The river Nervión, which runs through Bilbao, has a long tradition of shipbuilding, and a major effort was made to make the section of the submerged tunnel units compatible with one of the existing dry docks. However, analysis showed this was not a viable solution. (Fig. 1)

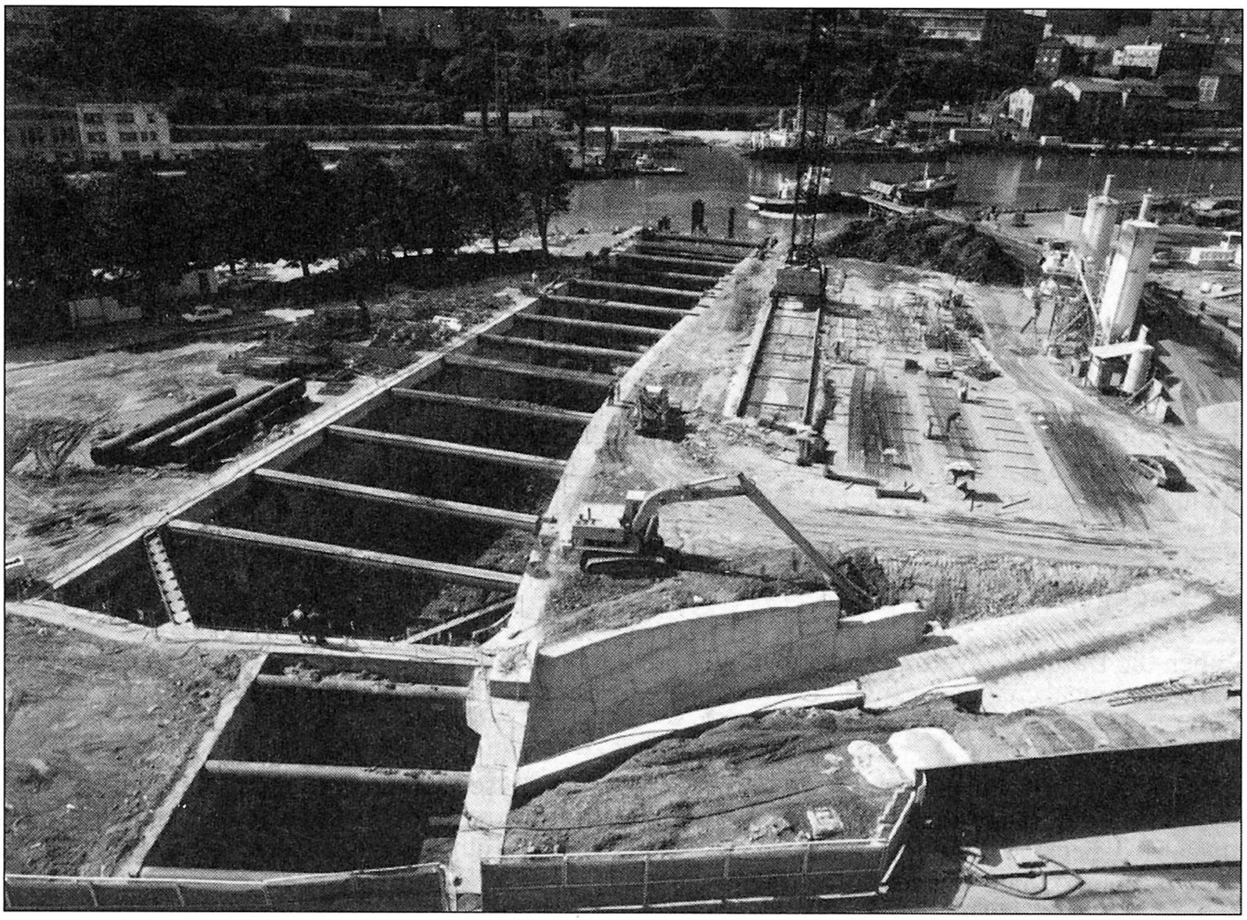


Fig. 1 Construction Site

Instead, the decision was taken to prepare a provisional dry dock where the different modules of the submerged tunnels could be built. The area chosen corresponded exactly to the route of the tunnel on the right bank. That way, a large part of the structure required for the provisional dry dock could be used to shape the definitive section of the subway tunnel.

The floor of the dry dock would be the roof of the tunnel and the dock walls, in cocrete diaphragm walls, would be the side walls of the definitive tunnel.

The diaphragn walls were embedded in rock at lower levels using trench cutter equipment. As the between-diaphragmwalls excavation progressed to shape the dock, head support struts were installed.

Highly liquid mud in the surrounding earth put the diaphragm walls under great pressure, so a jet-grouted slab at tunnel crown level was performed before excavation.

The slab acted as a strut from the beginning and prevented screen wall deformation and negative effects on nearby buildings while excavation went ahead.

The construction area was 100 x 15 metres and 12 metres deep. It was separated from the river by a metal gate removable by crane. The gate was prepared to withstand differential thrusts in both directions. On one side and with the dry dock, the thrust from the current and the tidal range; on the other side, during the test period on the structures in the dock, the water could reach anything up to 5 metres higher than the river level.

4. Submerged Tunnels

The submerged tunnels measured 11.60 x 7.30 metres and were each 85.35 metres long. (Fig. 2).

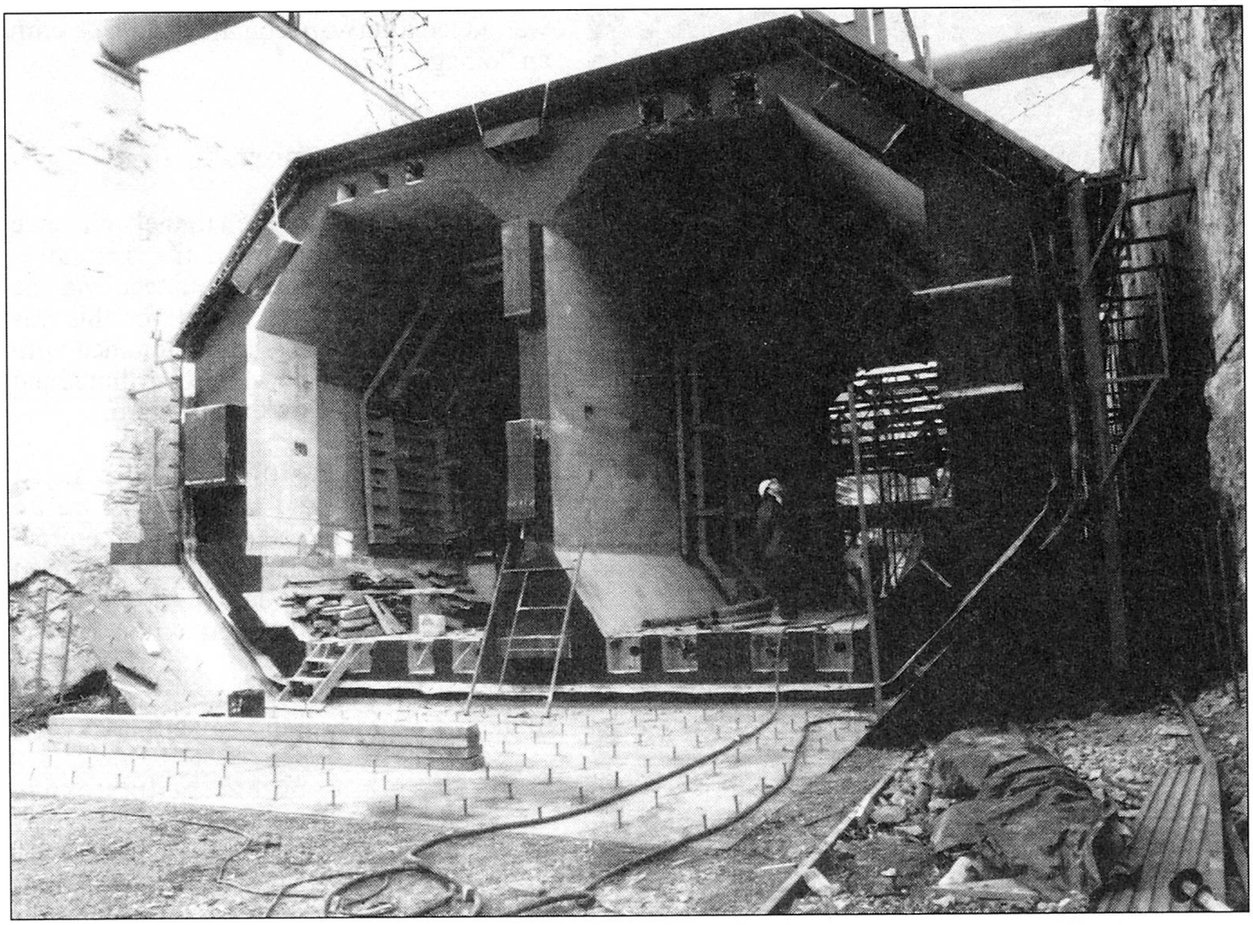


Fig. 2 Tunnel units. Cross Section.

Each unit was divided into two compartments, one for each direction, by a continuous 0.40 metre thick concrete wall. The outer walls were 0.80 m. thick at the base and sides and 0.70 m. thick at the top with chamfered corners.

They were built in modules to be integrated via post-tensioning, a technique used only temporarily to guarantee performance in flotation and initial support worst-case hypotheses at adjustment points.

Re-usable metal gates closed off the end compartments during floating and controlled sinking operations. A metal chimney tube enabled access during floating and sinking and also facilitated ventilation and energy supply.

5. Positioning and Adjustment

Vertical and horizontal jacks (8 Ud. in all) were fitted at either end of each unit for adjustments. The jacks moved the unit vertically and horizontally until correctly positioned, with errors of less than 10 mm.

However, in one of the operations, vertical mobility was limited by unusual deposits in the support zone. The structure was lifted slightly with the aid of the positioning catamaran and the area hosed out with water at high-pressure to shift the deposits brought by a recent high tide.

One important function of positioning is to give the joint between the structure and the receiving work on land continuous perimetral support, thereby ensuring initial watertightness and enabling the submerged structure's connection with land to be kept open. (Fig. 3)

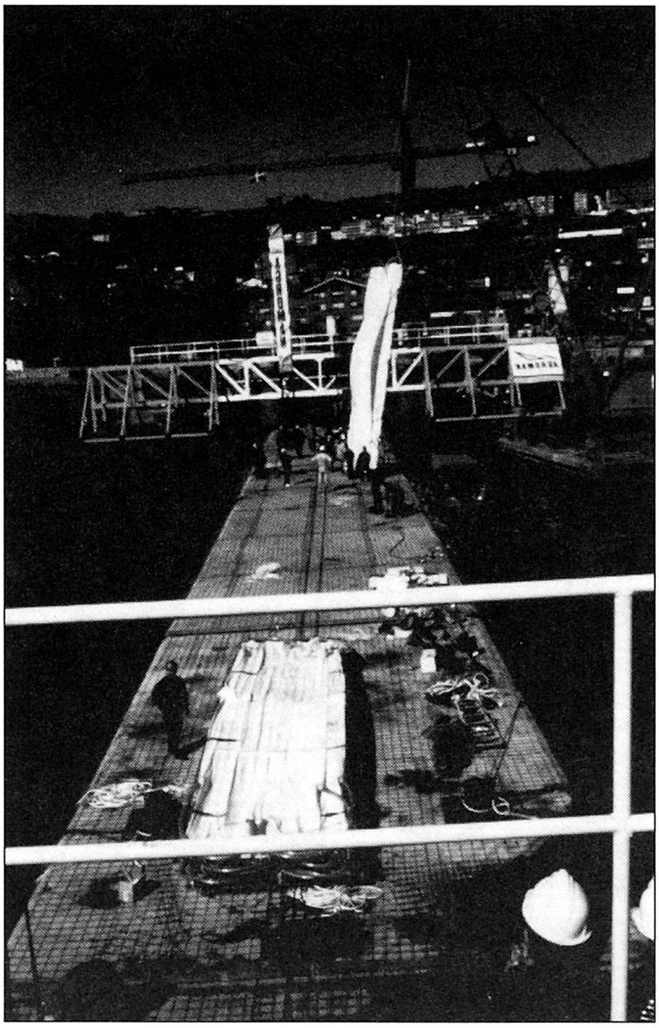


Fig. 3 Tunnel Positioning

A "gina" joint was used and a second seal was added afterwards on the inner side with an "omega" joint.

6. Discrete Supports

The initial support for each tunnel unit, once in place at the bottom of the specially-excavated trench, was guaranteed via the jacks at the four corners. However, this was only a temporary situation maintained with the aid of water ballast which distributed and minimised stresses on the structure.

In the definitive situation, with fixed concrete ballasts and variable rail traffic loads, almost continual support is required. Besides, the post-tensioning technique used on the modules was temporary and better support was also needed when it was released.

In the Bilbao Metro, the novelty involved:

a) preparing concrete bedplates as keelblock on the river bed, the top part of which were to be a metre below the definitive bed level for the structure. These bedplates were built using an external metal mould subsequently filled with submerged concrete. The plates would be adjusted to the right levels in a second phase. (Fig. 4)

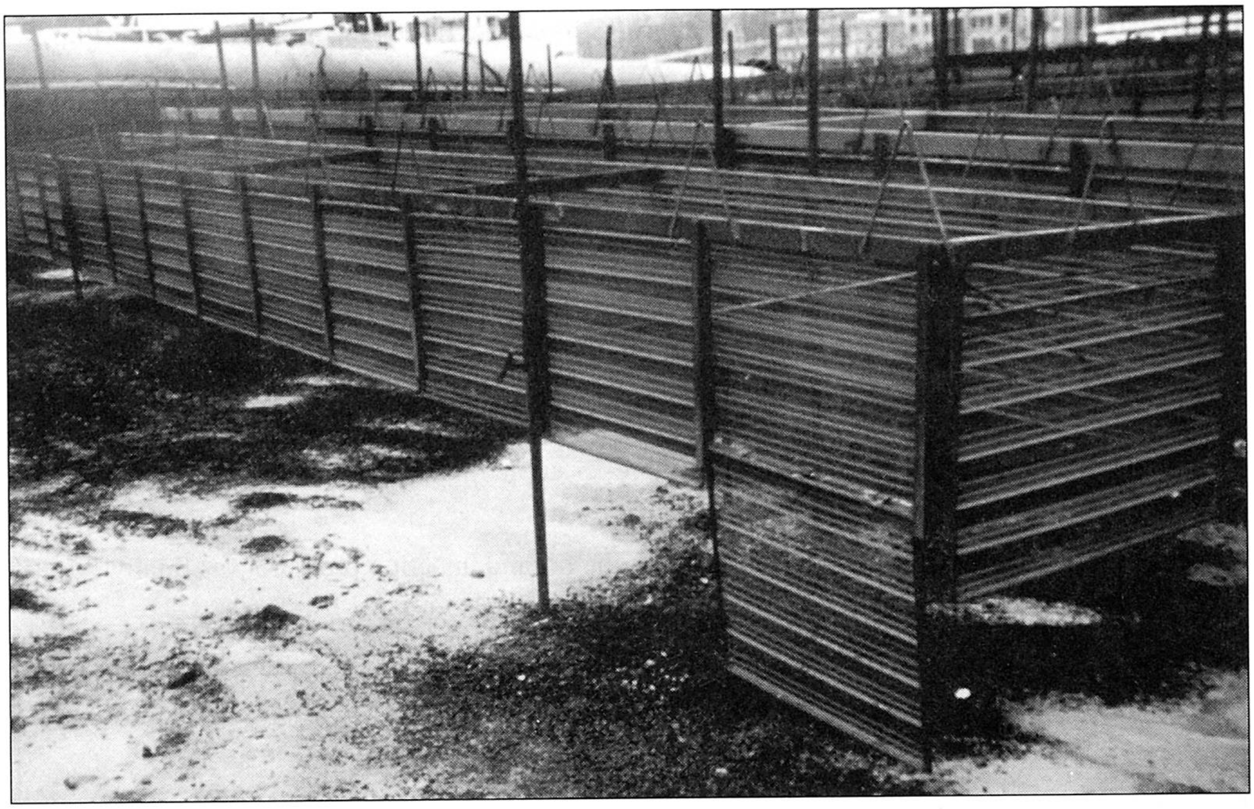


Fig. 4 Metal mould for keelblocks

b) placing empty sacks—9.40 m long and 0.80 diameter when filled—on these plates. The first time, divers were used to place the sacks in position after the structure had been positioned, but in later manoeuvres the sacks—three per plate—were placed before controlled sinking was performed. (Fig. 5)

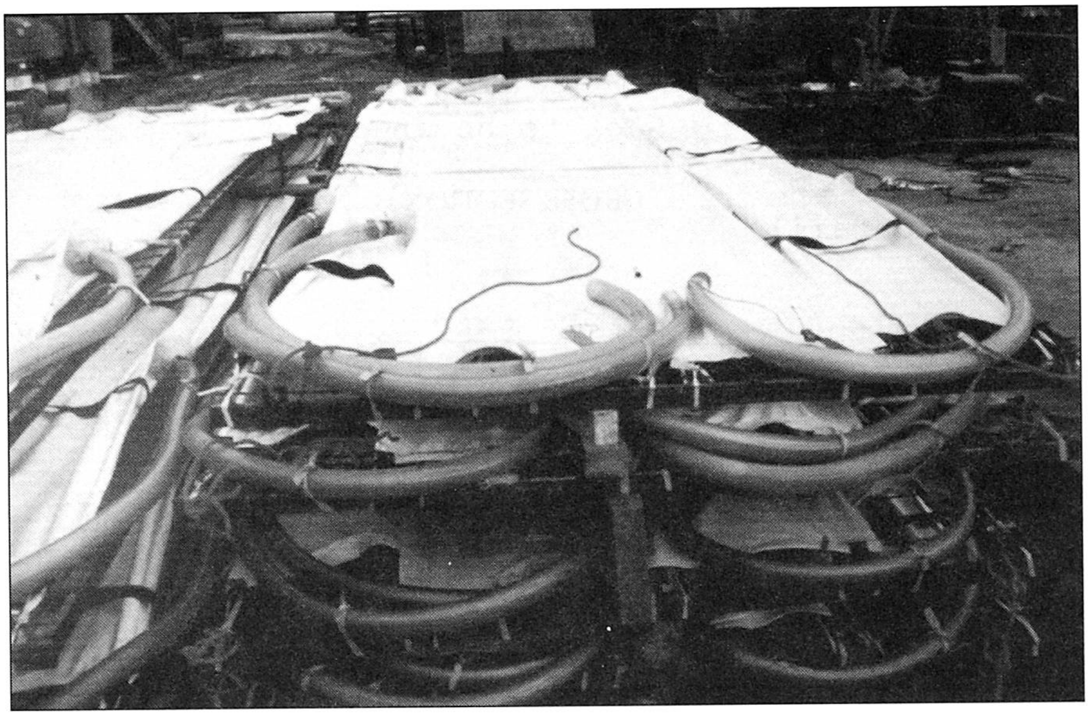


Fig. 5 Sacks before injection

The sacks were made of a synthetic, Bullflex-type material which enabled microconcrete to be injected as the water and air was pushed out through the material, thus ensuring a perfect fill.

Supports were separated by 3 or 5 metres depending on the areas. Given the relative lack of experience with this method, and the fact that the work had to be done blind at a depth of more than 20 metres, a number of test runs were carried out.

The test protocol gave us the following parameters for the best results:

- sack diameter between 0.60 m and 2.50 m.
- injection pressure from 1 to 5 kg/cm².
- fill time less than an hour, variable according to diameter
- microconcrete with 500 to 1000 kg of cement per cubic metre, with maximum aggregate size limited to 6 mm.

Each sack was fitted with 50 mm valves to check the fill and injection pressure was finally 1.5 kg/cm². (Fig. 6)

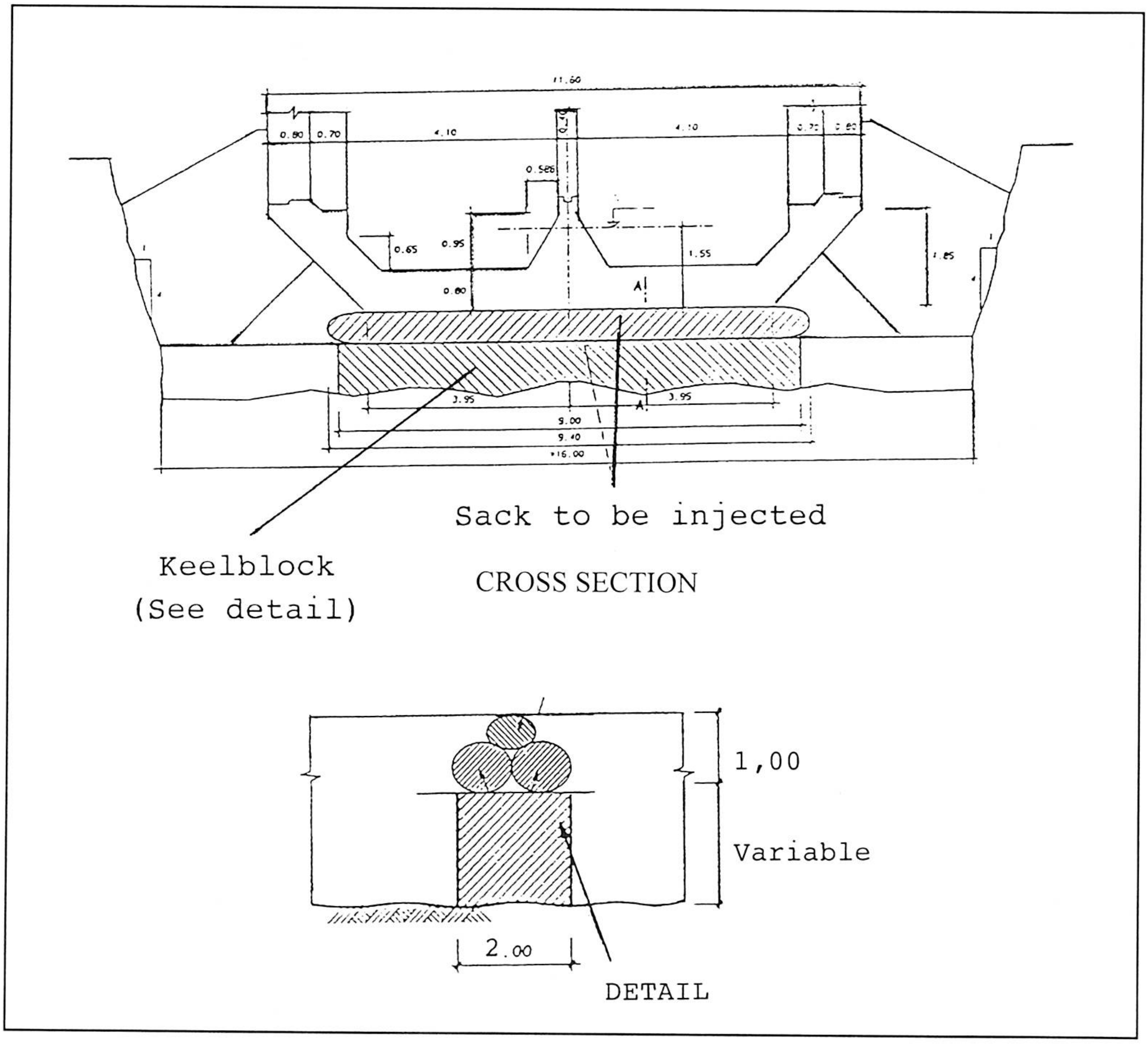


Fig. 6 Discrete supports

Safe backfilling due to Finite Element precalculations

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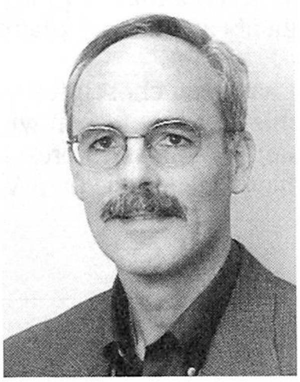


Juerg Matter, born 1952, received his civil engineering degree at the Swiss Federal Institute of Technology in 1976. After earthquake and risk engineering, General Site Manager in Saudi-Arabia he has been working now for over 13 years as a General Project Manager of several highway and railway tunnels.

Summary

The Habsburg tunnel is part of the lately completed National highway A3 from Zurich to Basel. At the southern end of the 1500 m long tunnel, 400 m of the two tubes with two lanes each were built in a foundation trench of up to 25 m depth, using the cut and cover method. The Finite Element calculations have not only been used to solve the statical problems but also to evaluate the backfilling procedure to prevent the arches from asymmetric deformations. To control the deformations during the backfilling phase, an extensive measuring concept with 8 cross sections per tube was developped. In the areas, where the backfilling work complied with the instructions, the measurements showed a very close conformity with the calculated values. In one area with asymmetric backfilling, the measurements showed much higher values.

1. The Project

On october 17, 1996, the last 13 km of the Swiss National Highway A3, from Zurich to Basel, including 3 tunnels, were inaugurated after a total construction time of 10 years.

The Habsburg-Tunnel with two tunnel tubes, each with two lanes, was constructed using five different construction methods. At the southern end, 400 m were built in a foundation trench of up to 25 m depth, using the cut and cover method. The overburden varies between 5 m and 12 m. To reduce transportation of excavation material, the excavated soil was directly used to backfill the already constructed tubes.

The tunnels have an inner diameter of 10.2 m and a reinforced concrete wall thickness of 45 cm. Every week two tunnel elements of 10 m length have been completed. In parallel with at least one month delay, after sealing the concrete surface of the tubes, the elements were continuosly backfilled.

After the second tube's breakthrough with the open shield at the northern end of the foundation trench, the gap between cut and cover and the underground section was closed, the foundation trench was fully backfilled and the surface could be reused as agriculture soil again.

2. The Finite Element Precalculations

2.1 The geometric model

To consider the influence of the backfilling procedure on the behaviour of the concrete arches, a two-dimensional Finite Element Model was established. This model allowed the simulation of a stepwise compression of the backfill layers. By simulating only one of the tubes because of the problem's symmetry, the number of elements and the calculation time was cut in half .

The following figure shows the area that has been chosen for the model. It consists of 611 knots, 1117 triangles and 200 beams. It represents an area of 42 m width and 34 m hight. The earth load above the Finite Elements is simulated as a uniform pressure. The backfilling layers have a thickness of 1 m each. The calculations have been made on a Vax computer using the software Rheo Staub.

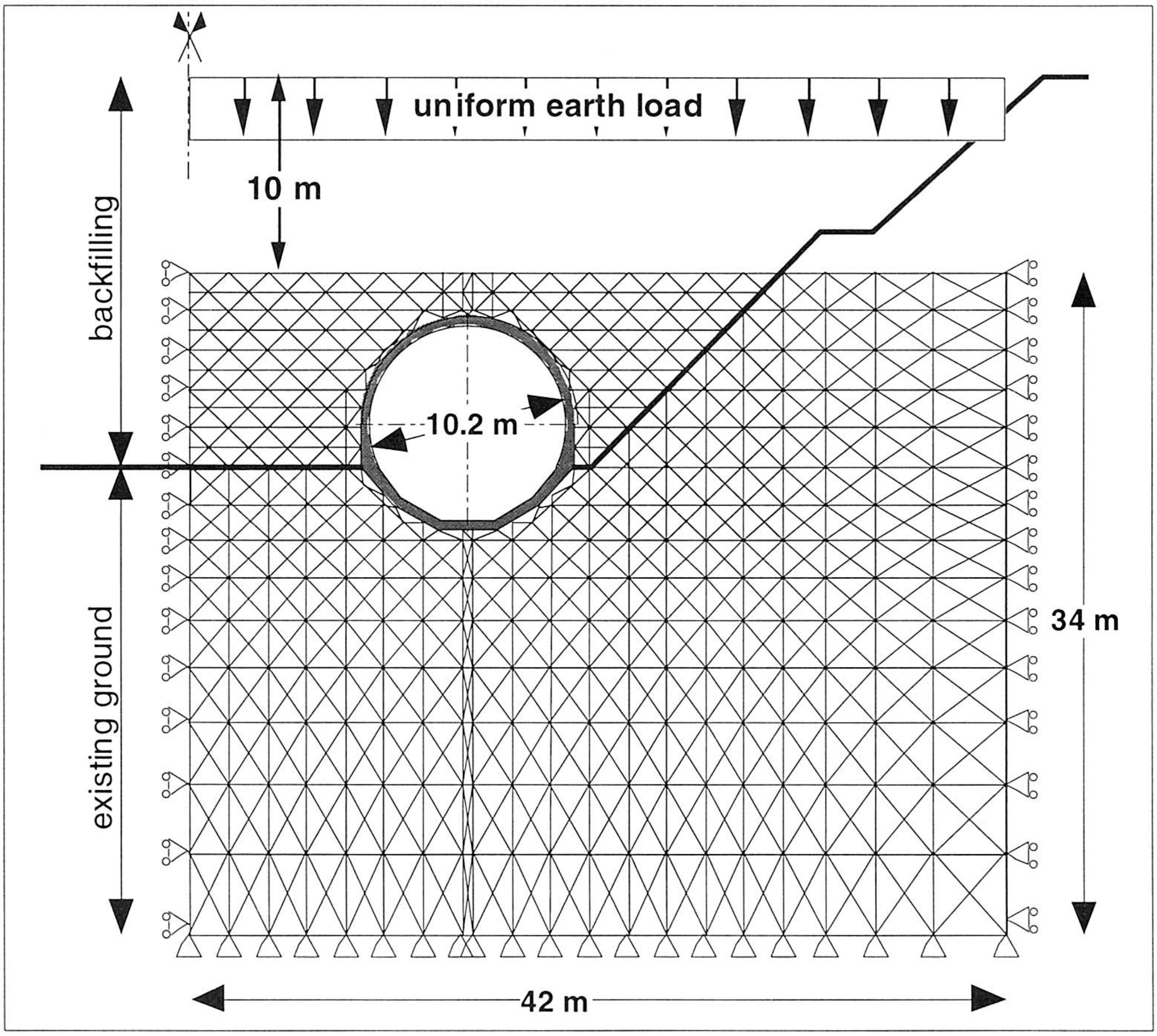


Fig. 1 Cross section through half of the foundation trench with the area of the Finite Element model

2.2 The Assumptions

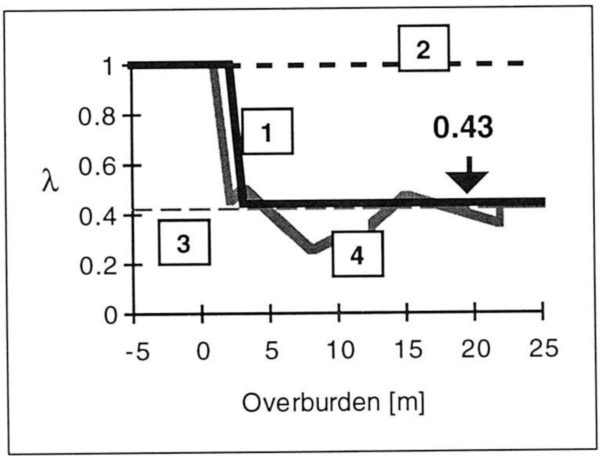
The following values have been used for the calculations

Soil	young's module of existing soil	300'000 kN/m^2
	unit weight of existing soil	22 kN/m^3
	young's module of the backfill material	50'000 kN/m^2
	unit weight of the backfill material	21 kN/m^3
Reinforced concrete	young's module	33'000 N/mm^2
	compression strength after 28 days	30 N/mm^2
	unit weight	25 kN/m^3

Determinative for the stresses and deformations in the concrete construction is the assumption of the lateral earth pressure. In the calculations the ratio between vertical and lateral earth pressure is defined as λ. The influence of λ on the tunnels stresses is much higher than that of the soils young's module.

During backfilling of another cut and cover tunnel near the international airport of Zurich in 1970 we made pressure measurements around the tunnels surface. We found out that the value of λ varies with the hight of the backfilling. From the bottom up to 2 m over the tunnel roof λ was 1.0 and decreased rapidly to about 0.45 for the rest of the backfilling. Figure 2 shows the shape of the measured curve (4) in relation to the overburden. Curve (2) with a very stiff bedding has been determined from a two lane road tunnel near Zurzach. Instead of a v-shaped foundation trench like the one at the Habsburg tunnel and at the airport, the foundation trench in Zurzach was built with anchored bored piles. In addition the tunnel wall thickness is 30 cm compared to the 40 cm and 45 cm of the other two tunnels. Curve (3) with a constant λ value of 0.43 represents a soft bedding and is therefore very conservative. Because the geometrical and the geological conditions for the Habsburg tunnel were quite similar to the airport tunnel we used curve (1) with a variable λ value for our calculations.

The diagram on the right side in figure 2 shows the influence of the λ value on the bending moment at the tunnel roof.



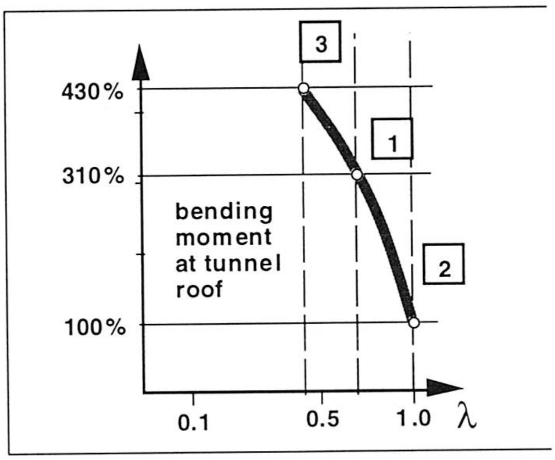


Fig. 2 Different shapes of the lλvalue and its influence on the bending moment at the tunnel roof

3. The backfilling instructions

The calculations showed, that the influence of an asymmetric pressure on the tunnel construction is much higer in the upper half than in the lower half. So we determined strict rules for the contracter's backfilling procedure. The difference permitted between the backfilling surface on the left and the right side of the tunnel decreases from 3 m at the bottom to 50 cm at the top. The maximum thickness of one backfill layer was limited to 50 cm and 50 N/mm^2 were required for the minimum elasticity in the backfill material.

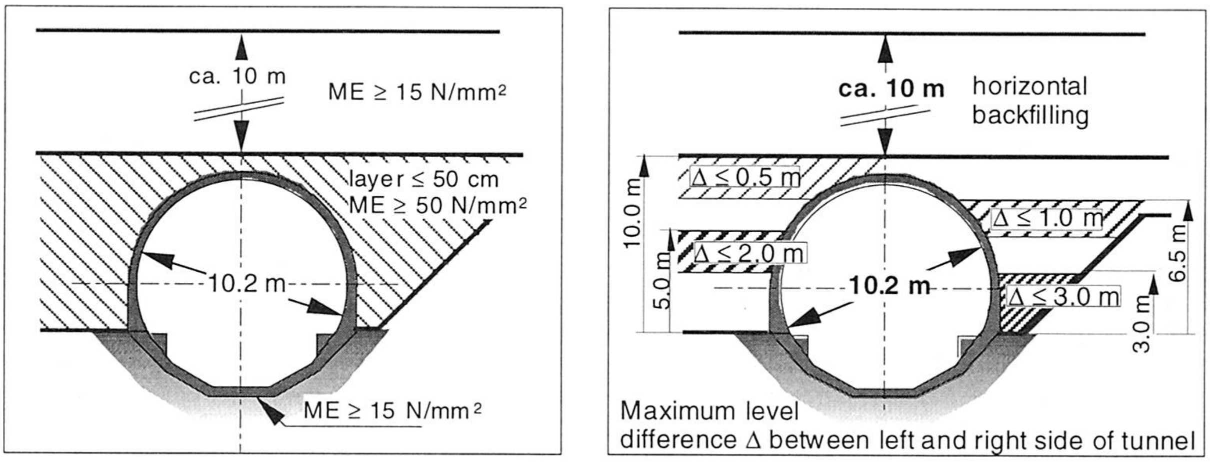


Fig. 3 The backfill instructions for the contractor

4. The measuring concept

To superwise the actual deformations during construction and to verify the calculations, an extensive measuring concept with eight cross sections per tube was developped and realized. The following figure shows the cut and cover section of the Habsburg tunnel with the measuring cross sections.

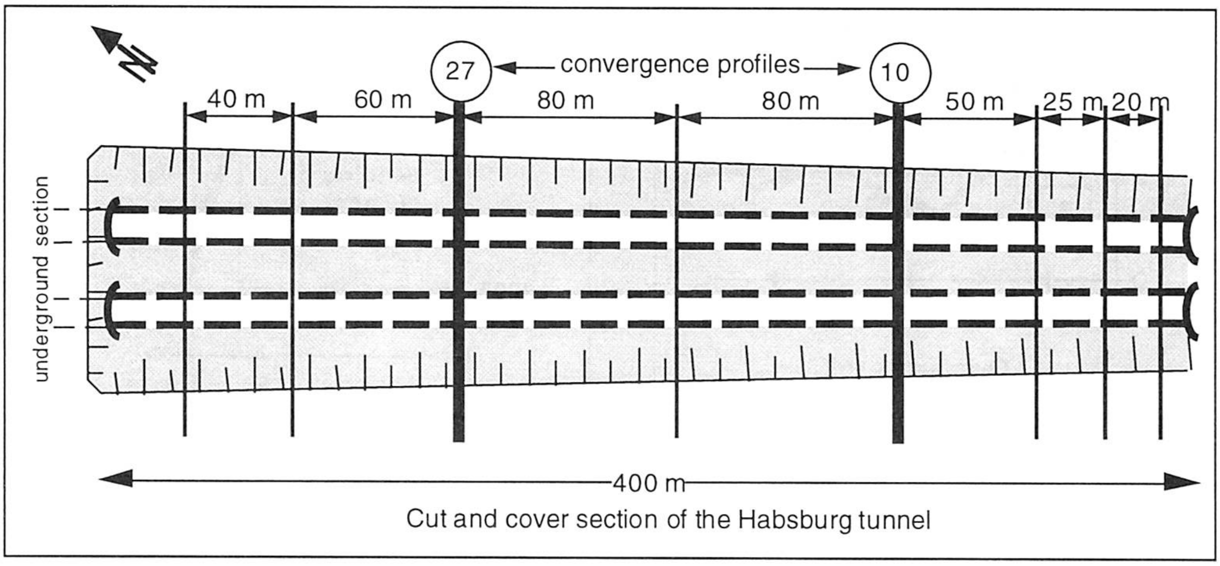


Fig. 4 Situation of the cut and cover section with the measuring cross sections

In six of the eight traces, only vertical deformations at three points have been measured. The two convergence profiles at the tunnel elements 10 and 27 have been monitored in cooperation with the Swiss Federal Instiitute of Technology. The following figure shows the arrangement of the two types of measuring points within the tunnel profile. For the discussion of the results the present paper focuses especially on the two convergence profiles.

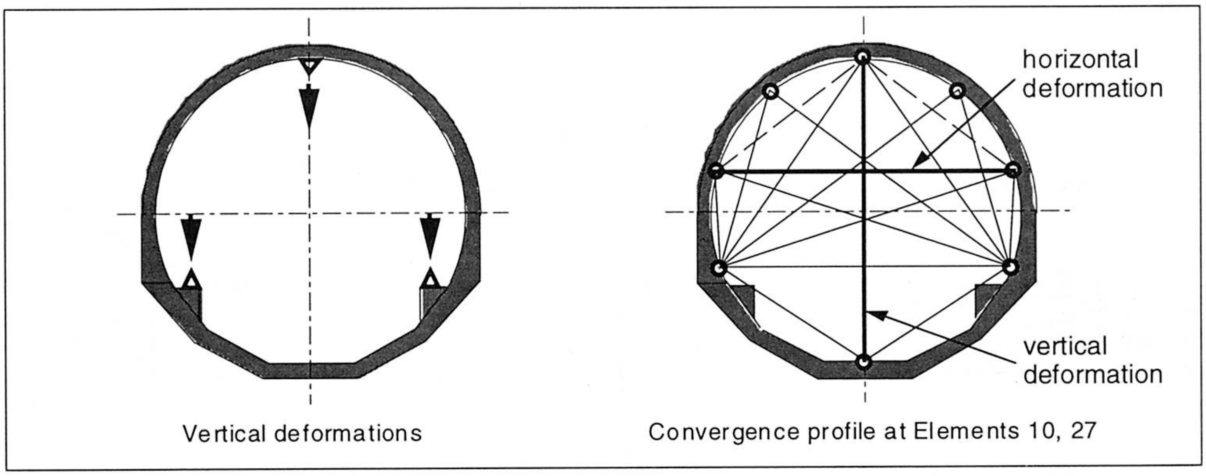


Fig. 5 Cross sections through the tunnel tubes with the measuring points for vertical deformation measurement and convergence profile

5. The results

In the beginning the convergence measurements showed, that the effect of the horizontal pressure of the compressed backfill material on the tunnel construction is less than calculated. This phenomenon corresponds with the expectations. In reality, the horizontal stiffness of the backfillmaterial does not remain constant as assumed in the Finite Element calculations. Up to two meters over the tunnel top the instructions for the backfiiling procedure were followed quite well. Then, in the southern part of the foundation trench, the contractor used the surface above the tunnel tubes as a site to store excavation material. To have easier access to the northern area he left an opening in the middle of the trench and caused by that an asymmetric load (see Fig. 6)

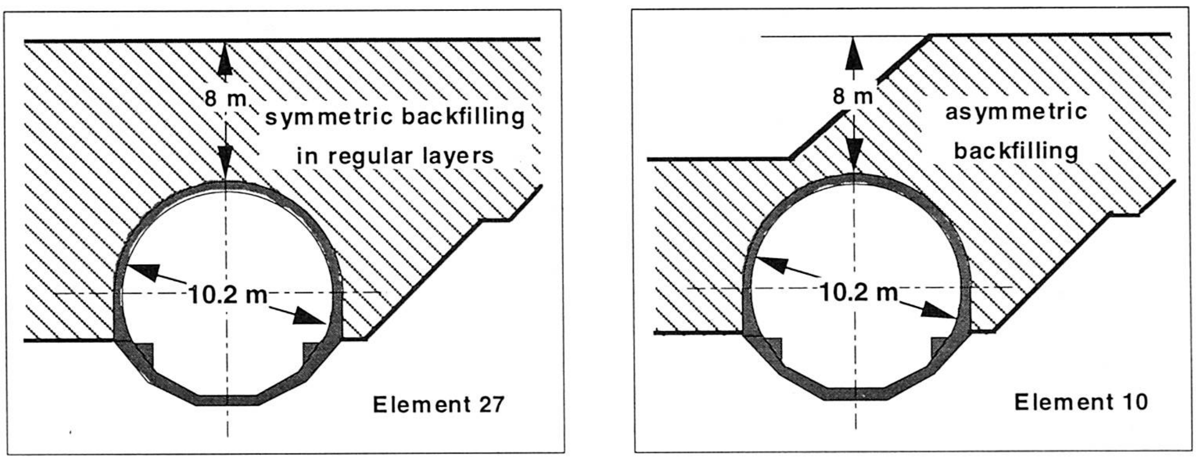


Fig. 6 Correct symmetric backfilling (Element 27) and incorrect backfilling (Element 10)

The following figures 7 and 8 show the difference between the calculated and measured vertical and horizontal deformations at the tunnel elements 10 and 27. Element 10 was backfilled with an asymmetric load, element 27 was correctly backfilled.

At element 27 the comparison between the calculated and the measured deformations shows a close conformity. For the vertical deformation 12 mm instead of 14.4 mm (minus 17 %) and for the horizontal deformation 15 mm instead of 12.6 mm (plus 19 %) were measured.

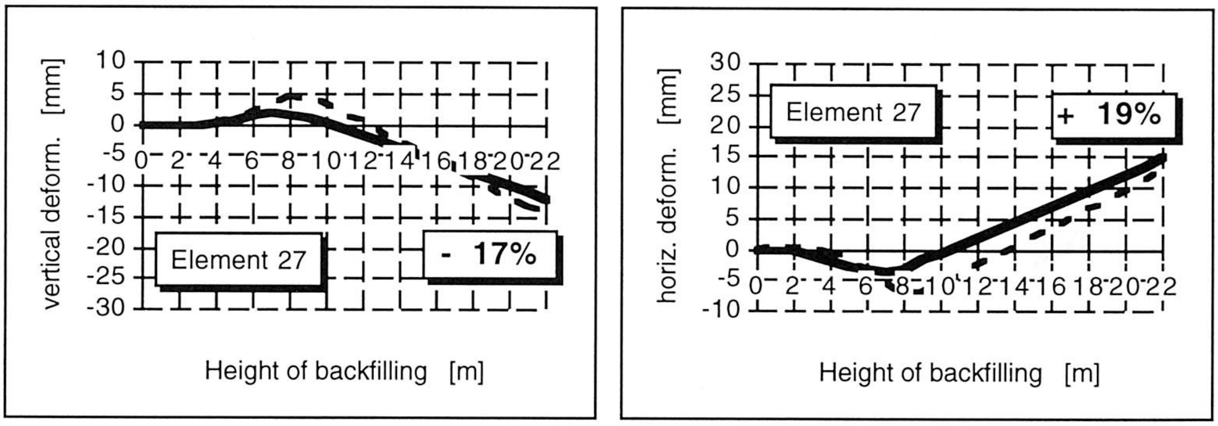


Fig. 7 Difference between calculation (dotted line) and measurement of the vertical and horizontal deformations at element 27

At element 10 the comparison between the calculated and the measured deformations shows a dramatically higher difference. For the vertical deformation 29.5 mm instead of 13.2 mm (plus 125 %) and for the horizontal deformation 30.5 mm instead of 10.2 mm (plus 200 %) were measured !

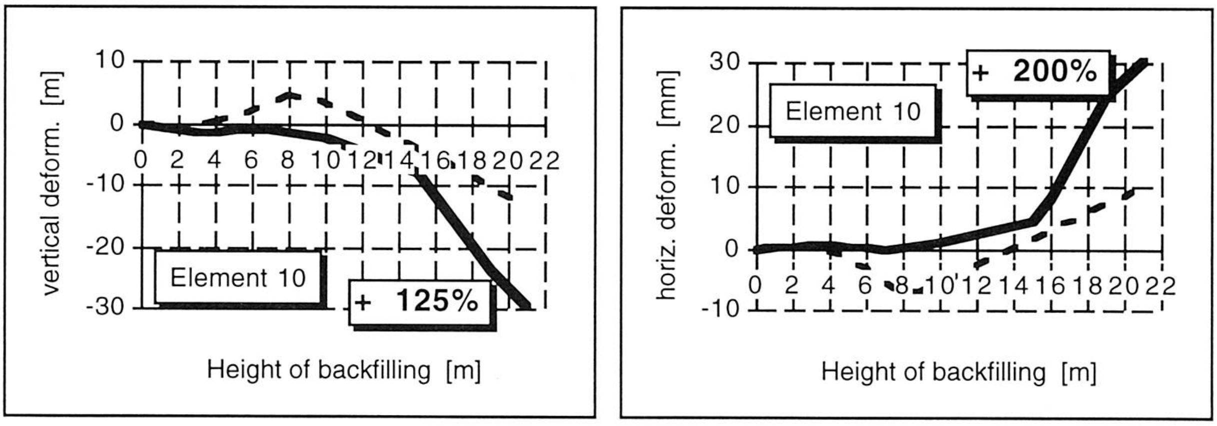


Fig. 8 Difference between calculation (dotted line) and measurement of the vertical and horizontal deformations at element 10

The close conformity of the results of the Finite Element calculations with the measurements in tunnel element 10 demonstrate, that the assumptions concerning the Finite Element model and the λ value for the lateral earth pressure were correct.

On the other hand, the mesurements on element 27 indicate that a strict compliance with the backfilling instructions is indispensable to prevent the tunnel construction from inadmissible stresses and deformations.

Railway Tunnel under Sydhavn Station

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Summary

As a part of the Danish Landworks for the coming fixed link between Denmark and Sweden a tunnel underpassing the freight tracks near Sydhavn Station has been constructed. This paper describes the design and the construction of Tunnel Sydhavn Station, which is situated inside the Copenhagen railway freight yard. The chosen construction methods for the construction pit bracing and the ground water lowering are described along with the observed settlements of the adjacent tracks for passenger and freight trains.

1. Introduction

In March 1991 the Swedish and the Danish governments agreed to establish a fixed link across the Øresund between Copenhagen in Denmark and Malmö in Sweden. The Link will consist of a four-lane motorway, and a dual-track railway. The Danish landworks include a railway from the Central Station of Copenhagen to the coast of the Øresund near the Copenhagen Airport and an extension of the existing motorways to the coast of the Øresund. The Danish Landworks are constructed by A/S Øresundsforbindelsen (ASØ), which is a state-owned limited liability company. The total length of the railway is approximately 18 km and the total length of the new motorway is approx. 9 km. The Danish Landworks are expected to open in 1998 and the entire link in the year 2000.

2. Tunnel Sydhavn Station

In the following, a specific Contract - "Tunnel Sydhavn Station" - of the Danish Landworks is described.

2.1 Alignment

The new ASØ tracks are connected to the existing tracks at Copenhagen Central Station and pass the Central Freight Station. In the freight depot area at Enghavevej, ASØ's track will descend into the "Tunnel Sydhavn Station" and will remain in a tunnel (Tunnel Sydhavnsgade) until the main street Sjællandsbroen is crossed near the strait Kalvebod, where the railway will ascend from the tunnel, and cross the strait on a low bridge.

The design speed for the line is 120 km/h in the curve at Sydhavn Station. The tracks have a maximum gradient of 1.7% in the tunnel at Sydhavn Station and 2.44% in the ramp. The gradient is acceptable due to the fact that only passenger trains use the tracks. The steep logitudinal profile was necessary to ensure that the tunnel could underpass the street Gl. Vasbygade and still only affect those tracks which nevertheless had to be closed to make room for the ramp.

2.2 Geology

The geology of the area comprises soil infill and boulder clay in various thicknesses overlying the Copenhagen Limestone. The fill layers vary in thickness from 1 m to 8 m. The limestone is found at depths between 4 and 12 m below ground surface. The limestone is a primary aquifer which contains the ground water resource of the Copenhagen area. The undisturbed ground water level is found at approximately level 0 i.e. at sea level.

2.3 Tunnel Structure

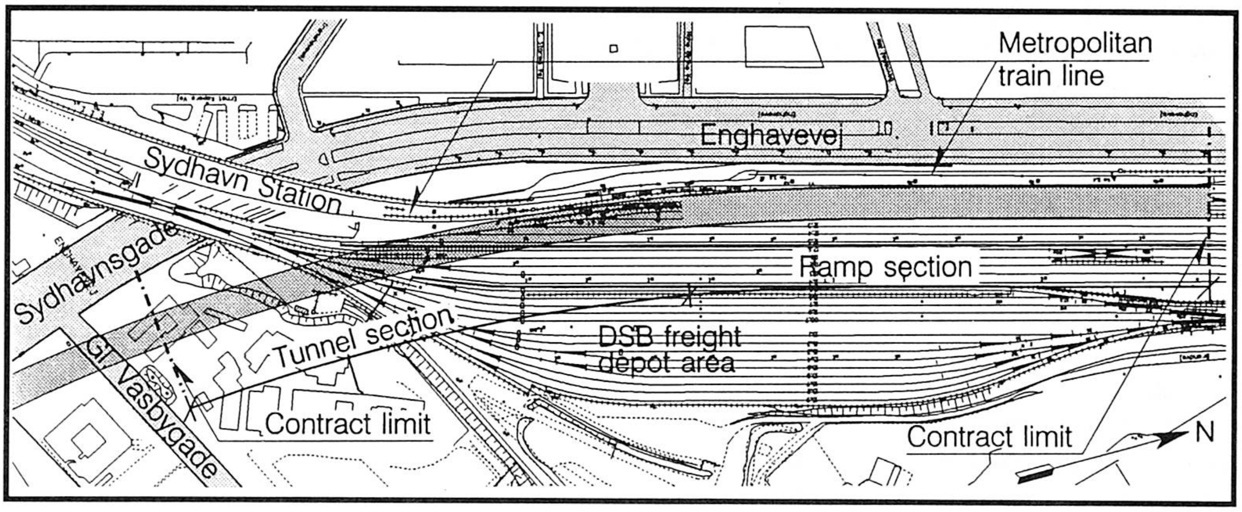

Figure 1 "Tunnel Sydhavn Station" -plan

The Contract "Tunnel Sydhavn Station" comprises 275 m of cut & cover concrete tunnel along with an 300 m anchored sheet pile ramp. The ramp section is situated inside the Danish State Railway's (DSB's) existing freight yard, while the tunnel section underpass the entrance to the freight yard very close to the Metropolitan Line Station "Sydhavn Station", cf. Fig. 1. The tunnel extend into an approximately 1,6 km concrete cut & cover tunnel running along the east side of the street Sydhavnsgade. This tunnel was constructed under a separate Contract named "Tunnel Sydhavnsgade". "Tunnel Sydhavn Station" was constructed by a Danish consortium HPA (a joint venture of the companies Højgaard & Schultz A/S, Pihl & Søn A/S and Per Aarsleff A/S) on the basis of a main project prepared by R/N (a joint venture of the consulting engineering companies RAMBØLL and Nellemann, Nielsen & Rauschenberger A/S). The contract sum was approximately USD 15 million, and the work took place from September 1994 to December 1996.

In the initial planning phases of the railway in the Sydhavnsgade area, the tunnel was assumed to be carried out as an open dewatered structure. But out of concern for the groundwater level in Copenhagen and the existing winnings in the area the initial decision was altered after the finalization of the preliminary geotechnical investigations and the tunnel was instead constructed as a cut & cover tunnel made of watertight concrete with drained sheet pile ramp.

Several solutions for the passing of the tracks at Sydhavn station were considered in the initial planning phase. Tunneling was not assumed possible due to the lack of sufficient overburden. Likewise tunnel jacking had not been tried in Denmark for a tunnel with this large cross section, a length of almost 300 m and an overburden between 0.5 m and 4 m. As DSB accepted that the tracks could be closed for shorter periods if the closings were planned and coordinated with DSB, it was decided to construct the tunnel as an cut & cover tunnel.

The internal height of the tunnel is approximately 6.5 m. The internal width is approximately 11 m. The walls have a thickness of 0.7 m, the roof is 0.75 m to 0.9 m thick and the bottom is 0.75 m to 1.0 m thick. The width of the tunnel base slab exceeds the tunnel width. This safeguards the tunnel against uplift. The tunnel walls and -roof are covered by a bentonite membrane of the type DUAL SEAL.

In both sides of the tunnel there is a combined cable canal and pedestrian walkway, which serves as an escape route in case of an accident in the tunnel.

3. Construction Requirements

3.1 Planning requirements

The entire work area for the Contract was situated within the existing railway yard. Thus, there was very little space available and all works had to be planned with due consideration to the safety regulations for tracks and catenary systems in operation. The train traffic on all railway tracks should maintain a full schedule during the construction period. In the tender documents, it was stated that the tracks could be closed in weekends agreed upon. The time periods were dependent on the tracks in question.

3.2 Design of the construction pit

Functional requirements for design and execution of the construction pit bracing were given in the tender documents, as temporary structures in Denmark are normally designed and constructed by the contractor no matter if the permanent structures are constructed in accordance with the owner's design or according to the "design and build" principle.

The construction pit for the “Tunnel Sydhavn Station” was situated very close to existing track areas. The construction pit bracing was thus constructed to meet the highest safety requirements. Normally, 20 kN/m^2 are estimated as the surface load when designing retaining walls adjacent to trafficked areas. For "Tunnel Sydhavn Station", the pit walls were designed for train loads on the tracks crossing or running along the bracing giving loads of more than 50 kN/m^2.

To reduce the settlements in the area behind the retaining wall, the construction had to be planned and executed in a such a way that no voids occurred behind the wall. Furthermore, it had to be documented by calculations that no point on the wall would have a horizontal deflection exceeding 10 mm for the upper 3 m of the wall and 50 mm for the lower parts of the wall. The contractor still being responsible for settlements in the area behind the retaining wall.

4. Construction Experience

4.1 Construction Progress

In connection with the execution of the construction pit for the tunnel, a cover was to be provided in the areas, in which the tracks should overpass the construction area. In his design, the Contractor, preferred a simple connection between the interim cover and the pit walls. The cover consisted of steel plates on HE1000B profiles every 1.2 m. and the walls were constructed as anchored Berlin walls with profiles per 1.2 m. As filling between the profiles, steel plates were used for the upper 3 m of the wall. They were pressed down in connection with the driving of the profiles, i.e. before placing the cover. For the remaining part of the wall, the filling was constructed by shotcreted concrete arches with a minimum thickness of 5 cm cast concurrently with the excavation.

Driving the steel profiles was mainly done during 6 weekends in Autumn/ Winter of 1994/95. The train traffic was diverted to other tracks in these weekends, so that the tracks could be removed to establish room for the pile drivers. During 3 weekends in the Winter of 1995, the tracks were again removed, so that the interim cover could be established.

After finishing the interim cover, the Contractor excavated along the whole tunnel length to a level corresponding to the lower edge of the steel plates. Hereafter, the contractor was allowed to excavate up to a height of 1.5 m (depending on the ground conditions) immediately after which the shotcreting between the profiles should be performed. No uncovered soil surface was allowed after working hours and furthermore the excavation was inspected every night around 1.30 a.m. during the entire excavation period.

After finishing the concreting and the membrane works, the Contractor backfilled the excavation to a level just below the interim cover.

Finally, during 2 weekends in September and October 1996, the tracks were again removed and the steel profiles cut minimum 1 m below the lower edge of the sleepers, after which the remaining backfilling was performed and the tracks relayed.

ASØ had in the Contract stipulated compliance with ASØ´s general environmental management system prepared on the basis of BS 7750. Based on this general system, the Contractor should prepare a detailed environmental action plan, which comprises the special procedures concerning the environmental issues of the Contract. For details about the environmental requirements and the monitoring see (Hess et al., 1996).

4.3 Monitoring of Settlements

The execution of the deep construction pit was expected to cause settlements in the areas close to the retaining walls. This would be caused by the driving of HEB-profiles and by the deflections of the retaining walls during excavation of the pit. Furthermore the extensive groundwater lowering was expected to cause settlements in the fill layers.

Within the railway yard adjacent to the excavation, daily monitoring of the settlements of the nearby tracks for the Metropolitan Train Line was required by DSB, in order to secure that the train traffic was not adversely affected by differential settlements of the rails. The rails for the freight trains were monitored less frequently, about once a week. Settlements of max. 30 mm due to the deflection of the retaining wall were foreseen in the design.

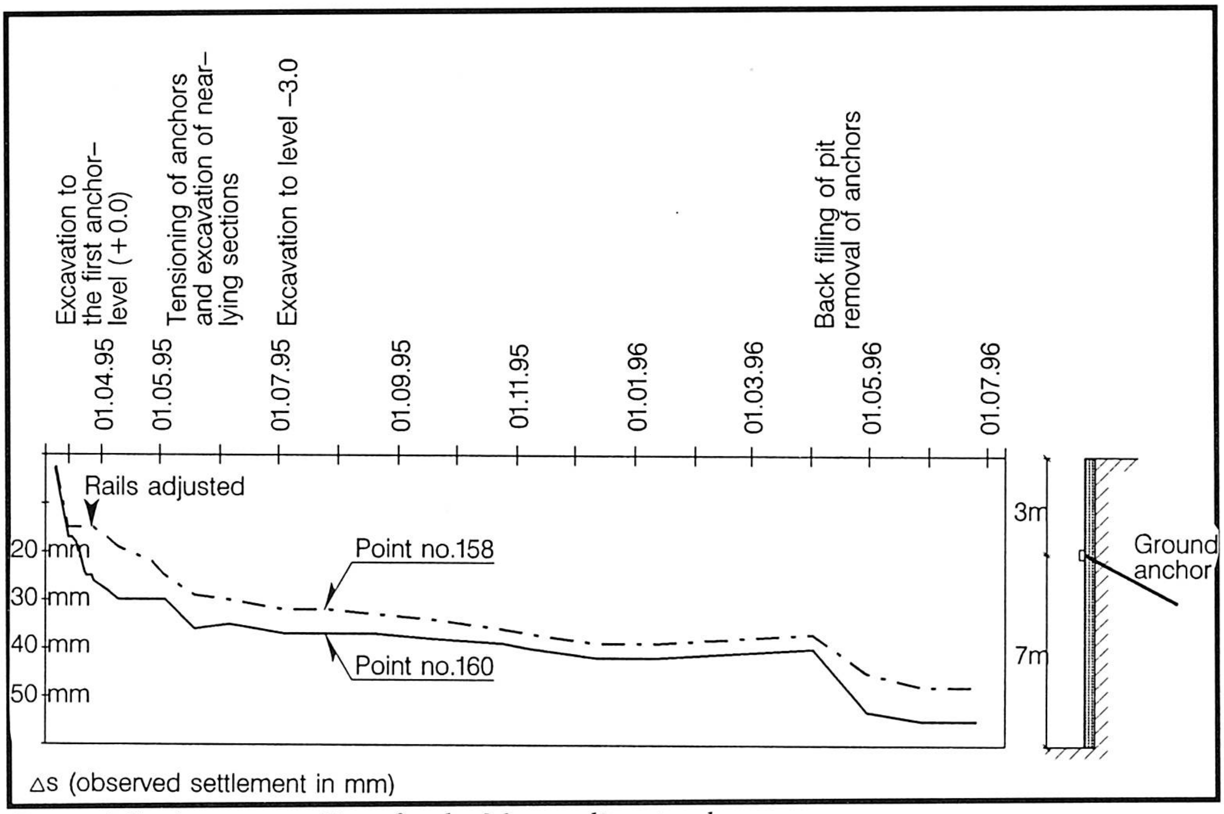

Figure 2 Settlements vs. Time for the Metropolitan track

Daily settlement monitoring on approximately 30 measuring points was performed during excavation of the construction pit and furthermore, the tracks for the Metropolitan train were surveyed by DSB personnel during all operating hours. In Fig. 2, the development of the settlements during the excavation and backfilling process is shown for two selected measuring points placed directly on the rails very close to the construction pit (less than 2 m from the edge). The maximum settlements of the track amounted to 55 mm. More than half of the settlements developed during excavation to the first anchor level. After the first quite dramatic settlements, it was necessary to adjust the rails, but after tensioning of the anchors, the progress of the settlements slowed down considerably. The settlements increased again when backfilling of the tunnel and removal of the anchors were performed.

The settlements of the freight tracks were between 2 and 30 mm dependent on the distance of the measuring point from the excavation.

5. Groundwater Lowering and Monitoring

At Sydhavn Station the dewatering was performed through 10 12 inch filter wells, all drilled to approx. 22 m below ground level (10 - 12 m in the limestone) outside the Berlin walls. The site was situated in an area close to an existing groundwater abstraction and therefor the dewatering was carried out with a high degree of adjustment to the actual excavation level. In the tender documents it was stated that the ground water level only was allowed to be lowered to 2 m below the deepest excavation level.

The groundwater lowering system operated in 77 weeks starting in November 1994 and stopping in July 1996. In this period approximately 44 m^3/h was pumped from the primary aquifer (the limestone), approximately 72 m^3/h at the beginning of the period and approximately 25 m^3/h at the end. In total approximately 600000 m^3 of groundwater was pumped up.

To evaluate the effects on the surroundings in the construction and the permanent phase a 3-D ground water model had been established by use of MIKE-SHE (Systéme Hydrologie Européne). On the basis of the MIKE-SHE model a monitoring programme was planned with the following objectives:

- to register the amounts of water pumped up.
- to monitor changes in the ground water level around the site.
- to monitor possible chances in the natural ground water chemistry.
- to register chances in the dispersion of conterminants in the ground water reservoir.

For details about the MIKE-SHE model see (Hess et al., 1996).

In the permanent phase it is necessary to relieve the ground water pressure in the deep end of the sheet pile ramp. Thus 3 permanent 10 inch bleederwells approximately 10 m deep (5 m in the limestone) have been established. Only the well in the deepest end of the ramp yields water, the amount is not systematically registered.

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Hess, U., Tantholdt, S., Bonde, C., Pedersen, S., Ladefoged, L., 1996, Environmental considerations regarding the Danish landworks for the fixed link across the Øresund. *Proc. 6th Int. Conf. & Exhibition on Piling and Deep Foundation,* Bombay 15.-17. Jan. 1996.

Long Distance Underpass Construction Beneath Railway

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Summary

In Japan in recent years it has become increasingly common for crossings with operating railways to be made into underpasses, in line with the ongoing maintenance of infrastructure. One construction method that has been used in the past in these cases was to secure space under the railway line by constructing temporary supports under the line or by redirecting it temporarily and then using open excavation. The other method was closed excavation from the side of the line while the line itself remains in place. Of these alternatives the open excavation method has the advantage that it can produce the structure provided there is superior management, but it still has a considerable impact on the existing lines, forcing trains to pass slowly. The areas and lines where it can be applied are particularly limited in cases where traffic on the line is heavy. On the other hand with the closed excavation method the construction method must be determined with close reference to the site conditions and the advantages and disadvantages of each technique. If however the task demands "a method that can be applied to construction of a long structure which crosses the rail track and has little impact on the line" there is, as yet, no method which is ready for practical application and meets these conditions.

The writers have proposed a new method (the pre-cast block lateral tightening method) for constructing a box culvert under a railway line under the restrictions inherent in the closed excavation method. This method has actually been applied to the construction of a crossing under a railway line and this report details the design approach and the results of the construction.

1. Introduction

One method of constructing a box culvert underpass under a railway line is to drive rectangular-section concrete PRC beams, which are fabricated in a factory, into drilled holes as the lateral girders and integrate these to concrete main piles driven in to form the structure. (Figure 1) However, the maximum length of crossing which can be planned under this method is limited by the scale of

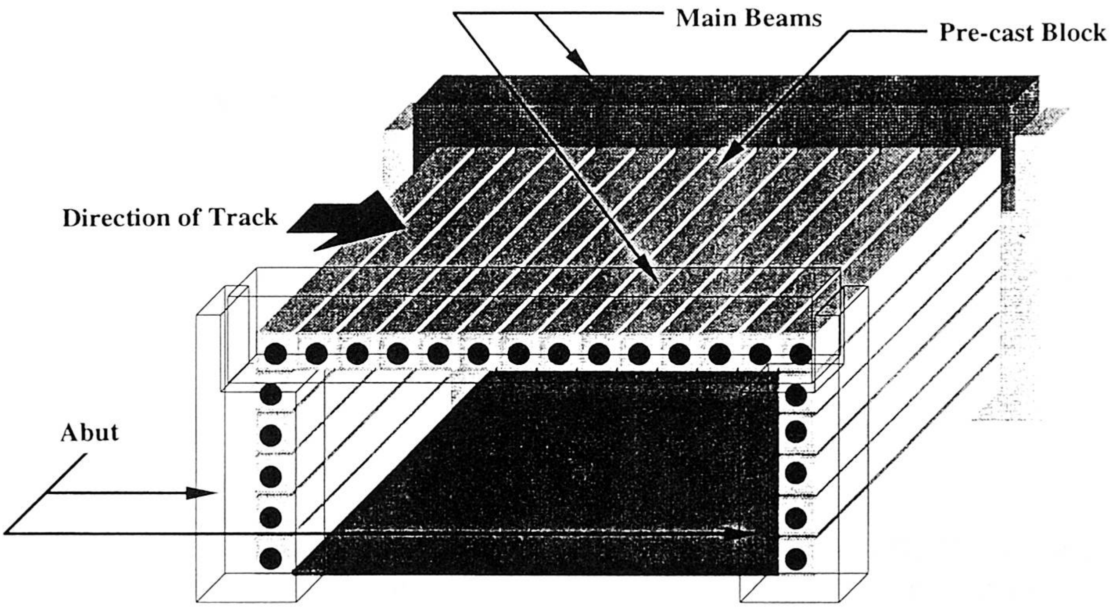


Fig. 1 Conceptional Diagram of Pre-cast Block Method

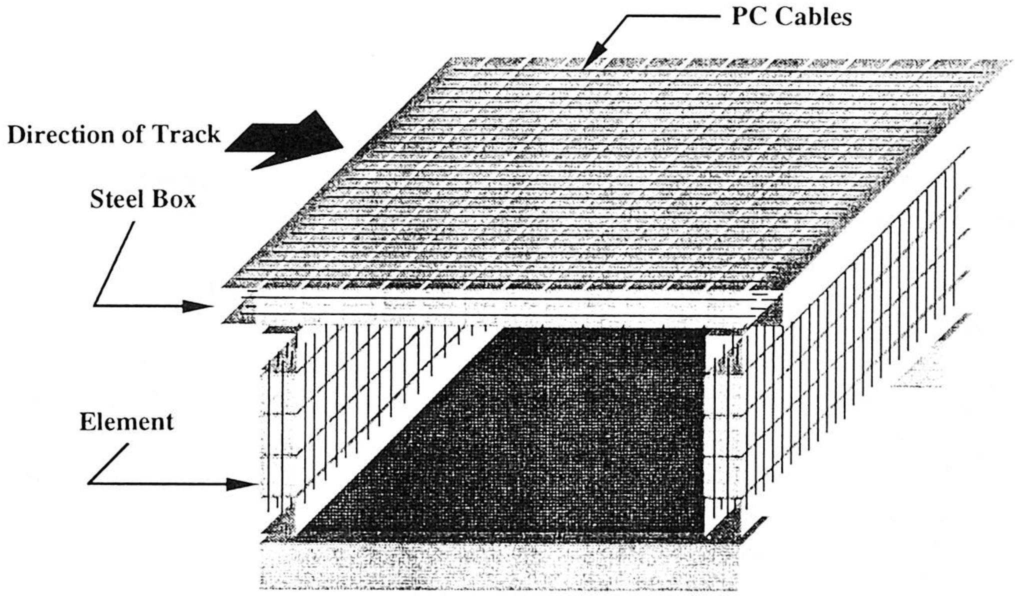


Fig. 2 Conceptional Diagram of Pre-cast Block Lateral Tightening Method

the PRC beams. The method proposed by the writers is the pre-cast block lateral tightening method, and that is to drive in the PRC beams perpendicular to the direction of the track and then introduce a pre-stress in the direction of the track to form a slab beam which can be used as a structural element in the direction of the track (Figure 2). The work involved in inserting and tightening the PC cables in the direction of the track is a drawback of this method, but the only things propelling under the track are small-section concrete elements. There is little impact on the track when the elements are put in place. The structural form on completion is boxes or beams spanning in the direction of the track. There is no restriction on the distance the structure extends across the track.

The fact that the structure is box-shaped means that the main beams running in the direction of the track and large-scale abuts have been required so far, but not necessary any more. These are the merits of this construction method.

East Japan Railway Company has used this method to build an underpass under the JR Sobu line near Kinshicho station where the route of an urban planning road passes under the railway. The structure is a box culvert which spans three spans of approx. 18m in the direction of the track and approx. 32m across the track. The design and construction methods for this structure are reported below.

2. Summary of Construction

The construction project was to build a new box culvert of 60m for a road to pass under a total of eight tracks belonging to our Sobu Honsen and maintenance lines. The section of track concerned is a major trunk route linking the center of the capital with neighboring Chiba prefecture. The traffic is extremely heavy at 880 services per day, particularly in the morning and evening rush hours when the interval between trains falls to two minutes and 25 seconds (Figure 3). There are also some 70 limited express services per day which cannot be delayed by driving slowly over the construction area. Overall the impact on services must be kept to the absolute minimum.

With a view to choosing a construction method with the minimum impact on trains, the pre-cast block lateral tightening method (a closed excavation method) was adopted.
The length of construction across the tracks using the pre-cast block lateral tightening method was 32m (Figure 4) with earth depth being 0.4m from the formation level of track. The soil conditions were, from the surface down, banking, silt and fine sand. All layers are soft and weak with N values of 5 or less. The groundwater level lay 5m below the formation level of track.

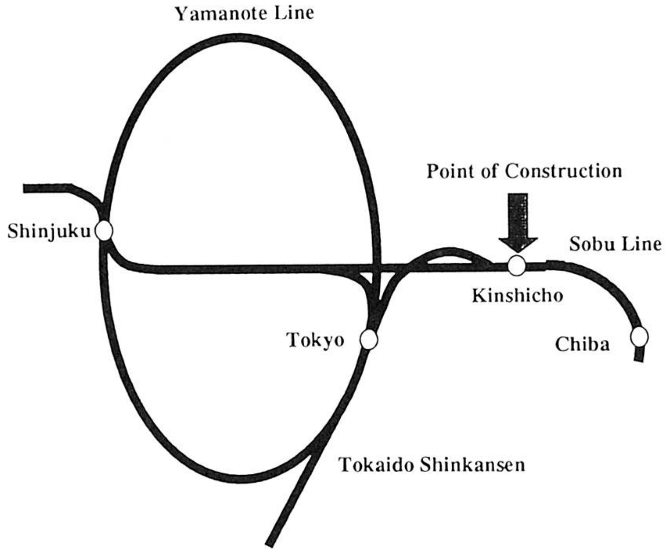


Fig. 3 Location of Construction

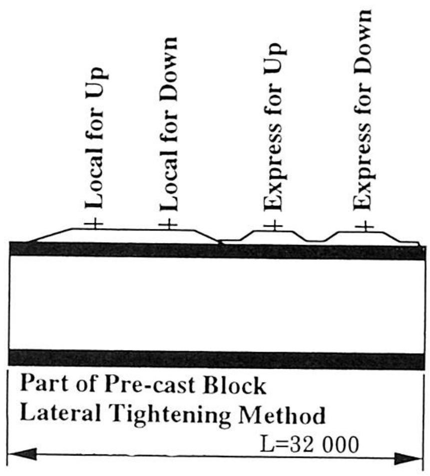


Fig. 4 Crossing View of Railway Line

Figure 5 shows the structural form of the box culvert. The top slab consists of 16 horizontal PRC beams (1,050mm x 1,050mm) and the walls are five PRC columns (950 x 950) driven in vertically. After working steel boxes are pushed into both ends, they are to be bound together in a direction perpendicular to their insertion and create PC slabs. As for the ground under the bottom slab and middle wall, the ground improvement was done as a countermeasure against liquefaction of the fine sand layer. The ease of construction was taken into consideration and concrete piles on site was taken.

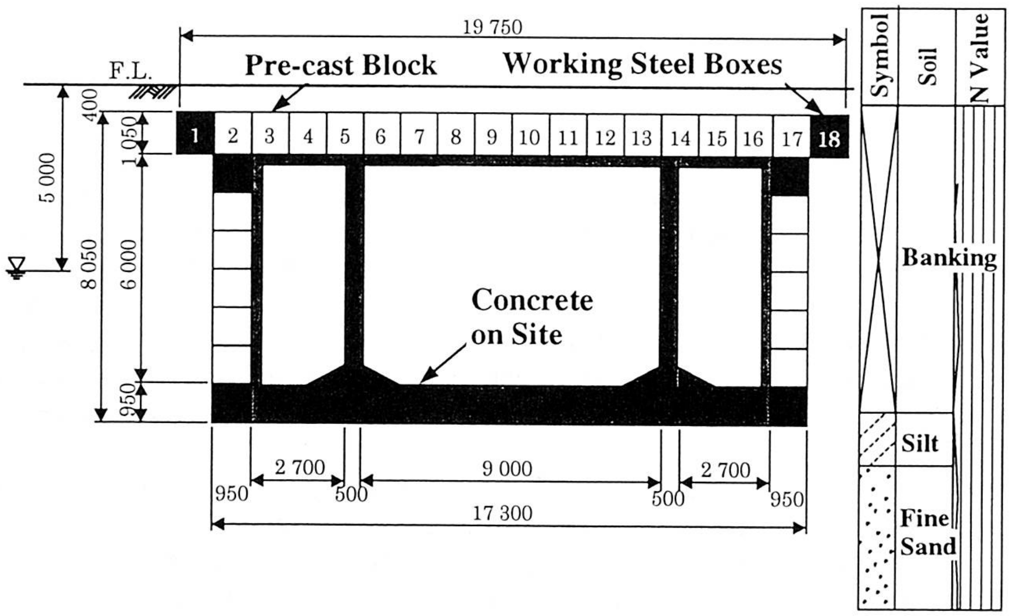


Fig. 5 Sectional View of Slab Beam Type

3. Design

One structural form of box culvert for railway underpasses is the through beam type. This is a method as follows. After PRC beams being pushed into slabs under the track to bear the overburden and side walls to hold back soil pressure, the RC pillars which support the side walls and the PC or RC beams, which hold the slabs are driven into place on site. However the maximum cross-track scale of this form of structure is 20m, dictated by the scale of the PRC beams.
In the method adopted for this project all the beams are bound together by PC cables to form a PC slab, a structural form which is not limited in cross-track length.

3.1 Analytical Model

The concepts behind the analytical model used in the structural calculations are illustrated in Figure 6. The joints between the top slab and the side walls, and the joints between the top slab and central wall were taken as pin joints while the joints between the side walls and the bottom slab were taken as rigid joints. Thus the structural analysis was carried out. The joints assumed to be rigid were those between PC and RC structural elements, the detailing was confirmed by an experiment to exactly evaluate the rigidity of the corners surrounded by the working steel boxes.

3.2 Joints Between the Two Side Walls and the Bottom Slab

The structural form shown in Figure 7 was adopted as a method able to provide the joint rigidity assumed in the design model. The PC steel cables of the side walls and the reinforcing steel of the lower slab are fastened to the working steel boxes and each of the fastened reinforcing bars is lapped to achieve rigidity in the corner close to that of a lap joint.

To evaluate the rigidity of the corner joints and the optimum arrangement of reinforcement, test samples of various forms of joints were constructed at nearly full scale (scale 0.75 ~ 0.85) and subjected to bending load tests. These tests confirmed that the structural performance of the corner joint satisfied capabilities of moment transmission and strength. They also showed that the rigidity of the corner joint can be evaluated by ignoring the steel plate on the compression side and taking 1/5 of the cross-sectional area of the steel plate on the tension side as reinforcement cross section for the RC elements.

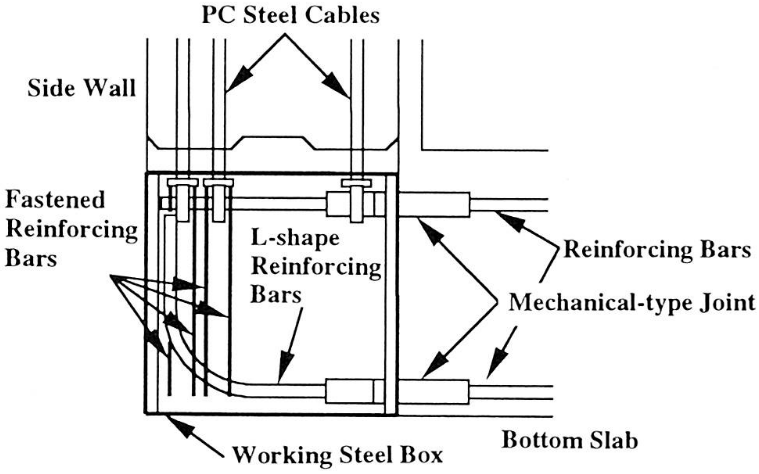


Fig. 6 Layout for Strengthening Reinforcement Bars of Corner Joint

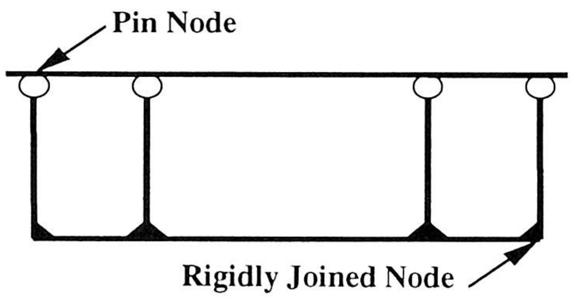


Fig. 7 Conceptional Diagram of Frame

4. Construction

4.1 Insertion of Pre-cast Element (PRC element)

This construction method uses PRC beams as the main structural elements with cables passed through them to apply lateral pre-stress. Increased precision is required to achieve this method. It can be assumed that the soil will include stones and other obstructions. Therefore the substitution method illustrated in Figure 8 was adopted. In this method three temporary steel boxes were driven in parallel. The temporary steel boxes were being pushed behind by PRC beams in the middle part, forcing them ahead. Both the horizontal and the vertical sections were constructed using this method. When the PRC beams are pushed in, their length (32m) is considerable. It was decided to push in 8m sections at a time, linking four together to produce the full length. Each line of four sub-beams was joined together by PC bar to form a single 32m beam.

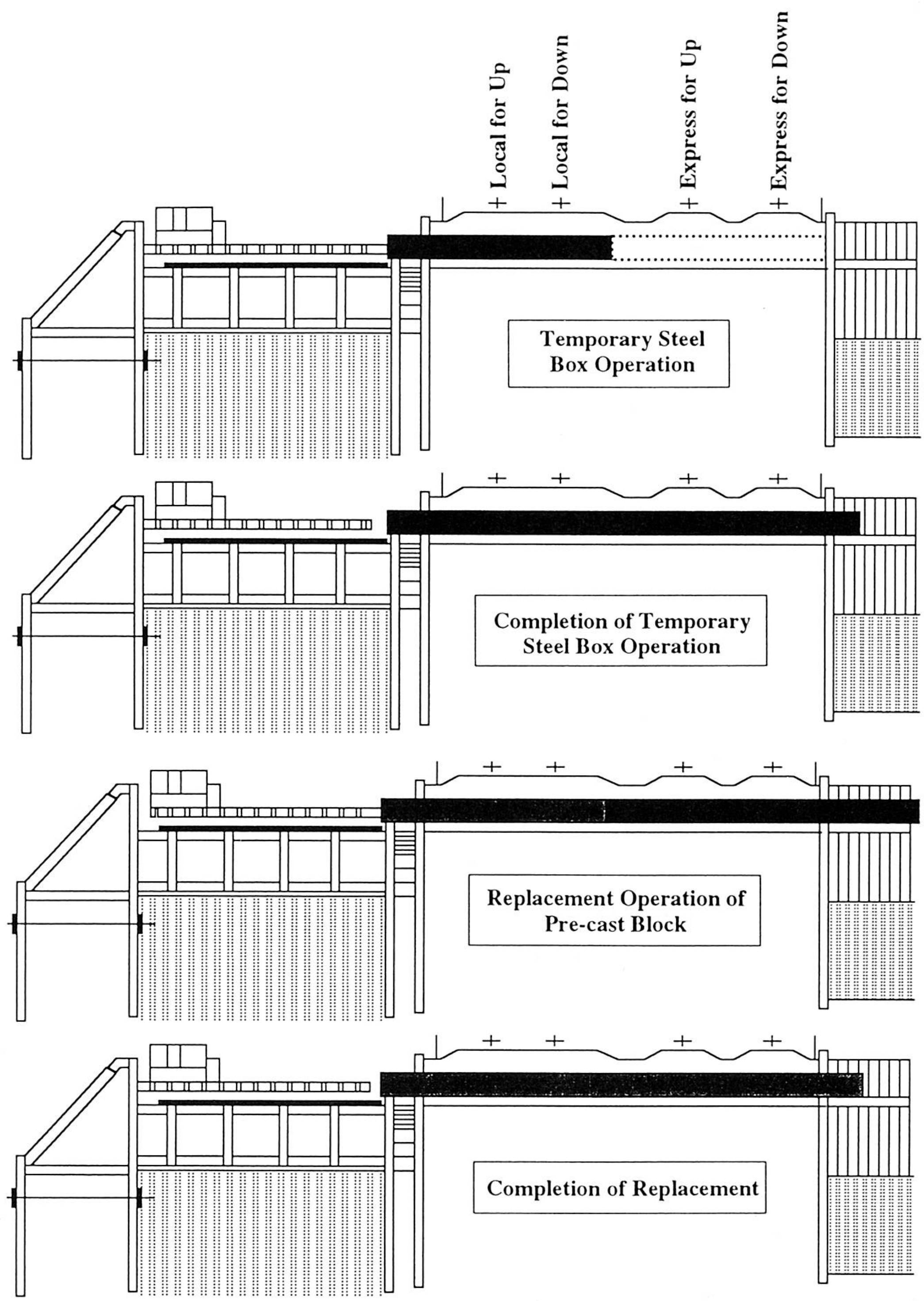
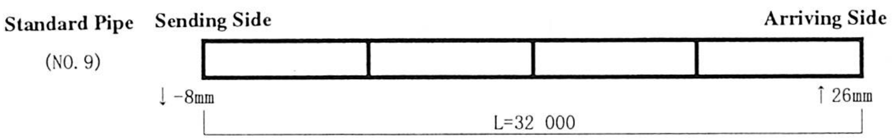


Fig. 8 Sequence of Replacement Construction

Adequate space was required for the operation of passing PC cables through each end of the horizontal and vertical elements and tightening them. Box beams were inserted to provide space. These were filled with concrete after the pre-stress had been applied to the PC cables.
The process of inserting the beams for the horizontal section takes the beams in the center of the horizontal section as the standard box and, as the shallow soil coverage demands high precision, excavation for this pipe proceeds by hand. Excavation for other temporary steel boxes are excavated by machinery. The precision of the base pipe's placement is, at the insertion end, 8mm to the left and 8mm down while at the far end it is 26mm to the right and 26mm up. Therefore the degree of precision both vertically and laterally was 1/940. The precision results for the top slab beams are shown in Table 1. Despite the great length of the construction at 32m the target accuracy (1/500) was achieved in even the worst case. This seems to indicate the validity of the substitution method.

No. of Beams	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Sending Side	-35	-33	-23	-18	-16	-12	-10	-7	-8	-9	-12	-11	-8	-7	-11	-10	-14	-14
Arriving Side	28	26	26	26	18	22	24	32	26	22	20	22	22	19	14	17	17	21
Precision (1/○○)	510	540	650	730	940	940	940	820	940	1030	1000	970	1070	1230	1280	1190	1030	910

Unit (mm), (+) is on the right, (-) is on the left



Result for Precision of Perpendicular Direction (Standard Pipe only)

Table 1 Results for Precision of Horizontal Direction (Top Slab Beams)

For the vertical sections temporary steel pipe sections linking with the horizontal sections were drilled into place and connected in place of the working box beams. Then the temporary steel pipes were sequentially pushed in and replaced by the PRC beams so that the pipe sections replaced the working box beams at the bottom as well.

The pushing in of temporary steel pipe sections in both the horizontal and vertical sections, and their later replacement by PRC beams, was conducted at night after the cessation of train services (a period of approximately 3.5 hours/day). Therefore the construction period for one PRC beam was approximately seven days for the insertion of the temporary steel pipe sections and three days for substitution with the PRC beam, a total of ten days per beam.

4.2 Lateral Pre-stressing

After the PRC beams have been pressured into place, PC cables are inserted and a pre-stress applied to enable use of the PRC beams as a structural element in the perpendicular direction against insertion. The PC cables are arranged as shown in Figure 9. The cables themselves are Afterbond PC steel cables (1T21.8) inner grouted into sheathes and an outer grout is applied between the PRC beam and the cable after the time of insertion. Tension was applied from one side only, from the right in the case of the top slab beams and from the top in the case of the side wall beams. A tension of 43tf was applied to each cable. The introduction of tension forces was based on control value of ±5% of extention volume against planned load of PC cables, and pre-stressing work was carried out to keep

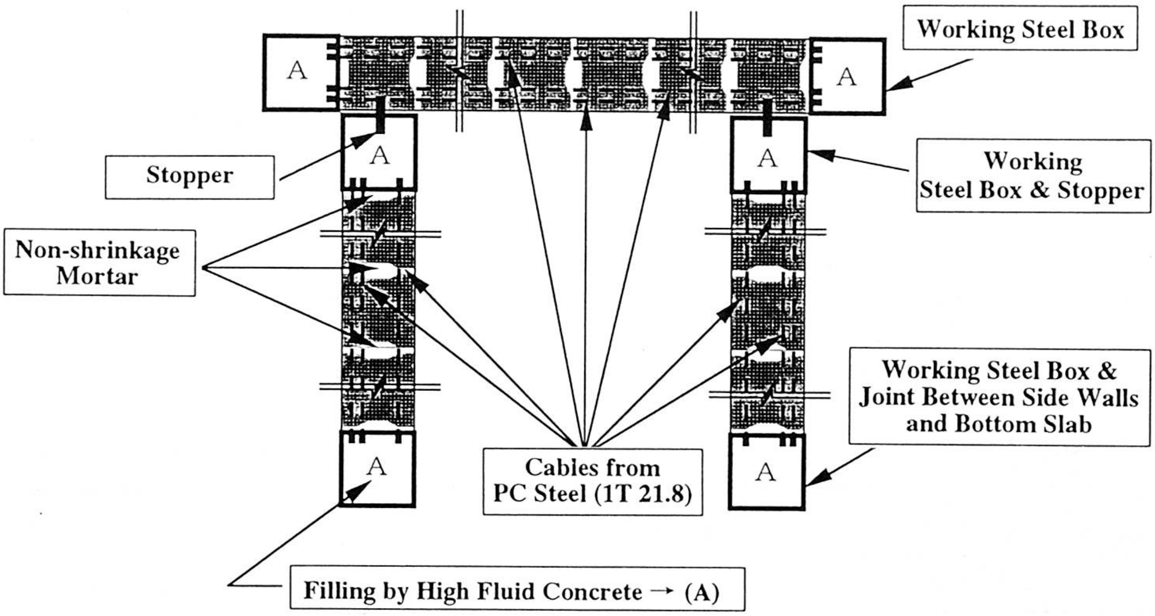


Fig. 9 Layout of Cables

extention volume within the control value. The planned tension forces and the results of construction are shown in Table 2.

The number of cables inserted in this process was 1,196. Of these 428 were in the top slab beams and all tightened mechanically. The process of inserting the cables by machine was rehearsed on a full-scale model to confirm the practicality of the insertion method. The 768 cables in the walls were inserted and tightened by hand. The record of construction results for both the walls and the top slabs is shown in Table 3. The daily number of cables processed in the top slab was approximately twice that for the side walls, which indicates the efficacy of mechanization.

For the other elements to be integrated by the PC cables, all the joints between elements must be filled with mortar. In this case it was anticipated that filling the entire 32m length of the joints would not be possible. The mortar insertion method and mix proportion were confirmed in advance on a scale model.

	Setting Value	Measured Value	Remark
Maximum Load	42.6tf	42.6tf	
Growth Volume	Error is within ±5% for 37mm (Approx. 35mm~39mm)	Maximum Value 39mm Minimum Value 35mm Average Value 37.1mm	Maximum load is fixed for top slab and side walls by equipment.

Table 2 Result of Pre-stressing

Parts of Construction	Cable extention (m/pce.)	No.	No. of days for construction	Work per day: Cable extention	Work per day: No. of the inserted	Work per day: No. of pre-stressing	Remark
Top Slab Beams (automation)	17.65	428	18	420m	24	25	Side walls were constructed by two formation.
Side Walls (human power work)	6.05	768	20	230m	50~80	80	

* (Total of both sides)

Table 3 Comparison of Construction

5. Conclusion

In the future the number of underpass constructions under busy railway lines can be expected to increase. In order to meet the demands of a wide variety of construction conditions East Japan Railway Company is examining a number of other construction methods in addition to the box culvert method described above. These include design and construction methods for ring-type culverts, methods for constructing the whole body by inserting elements on the part of bottom slab, and methods of using cheaper steel elements in place of concrete elements.

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Advantages of British Steel Bi-Steel in Immersed Tunnel Construction

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Hugh Bowerman, born 1960, has a degree in civil engineering from Manchester University. Working initially in the design of offshore structures, he then focussed on subsea production systems which introduced him to mechanical and controls technology. The past 4 years he has been developing Bi-Steel, his work covering the production technology, product R&D program and the writing of the Bi-Steel design manual.

John Pryer, born 1962, has a degree in civil engineering from Nottingham Polytechnic. He worked for over 10 years for a civil engineering contractor gaining extensive experience both on site and in the design office. He joined British steel in 1994 as a piling advisory service engineer, moving to the Bi-Steel business in 1996 to lead the sales and marketing activities.

Summary

There has been interest in the use of steel-concrete-steel sandwich composites in immersed tube tunnel construction for over 10 years. Research has shown this form of construction to have good structural performance. Despite this, its use has been limited owing to difficulties in construction. British Steel Bi-Steel is a new, unique and patented composite construction system comprising two parallel steel plates held together by transverse bars welded at each end to the plates. The void is filled with concrete. Bi-Steel overcomes the construction difficulties, offering rapid construction times. It also offers opportunities to design and construct immersed tunnels in ways which are impractical with conventional methods. If these benefits can be realised in a project, then the use of Bi-Steel will result reduced costs. This paper explains what Bi-Steel is, and suggests how the material may be advantageously used.

1 Introduction

Immersed tube tunnels are usually constructed in a dry dock or casting basin in a number of segments. These are floated to the tunnel location where careful ballasting lowers them onto a prepared foundation. Adjacent elements are sealed together by welding, grouting or rubber seals. Permanent ballast is applied to ensure on-bottom stability. Most immersed tunnels are located in dredged trenches and are subsequently covered up.

For applications where water depth makes the construction of conventional immersed tube tunnels impractical, the concept of the immersed floating tunnel has been developed. The tunnel section is designed to have slight positive buoyancy, thus supporting its weight by floating. Segments are joined together and sealed as previously. However, the tunnel is held underwater and at the required depth by being tethered to the sea or river bed.

A number of different construction methods are used for immersed tube tunnels. These fall broadly into two categories: concrete or steel.

In concrete construction the tunnel tubes are usually rectangular cross section. The flat sides are heavily reinforced with steel to withstand the bending forces. Water is excluded by an external coating of bitumen, although some recent tunnels have deemed this unnecessary. Occasionally an outer layer of steel is used to exclude water. Tunnels of this type have been in use for many years - the Maas tunnel in The Netherlands was first opened in 1942 [1].

Steel immersed tunnels have been in use even longer, the first example being under the Detroit River in 1910 [2]. Whilst there are many variations on the general theme, all steel tunnels use a stiffened steel shell construction. Mass and additional strength is added by casting concrete around the shell, either on the inside or outside. Such tunnels are usually round in cross-section and the strength issue is not so severe as in the rectangular concrete tunnels. Thus, whilst some composite action between the steel and the concrete is assumed, the technique has never needed to develop the full strength potential available by utilising an inner and outer steel skin.

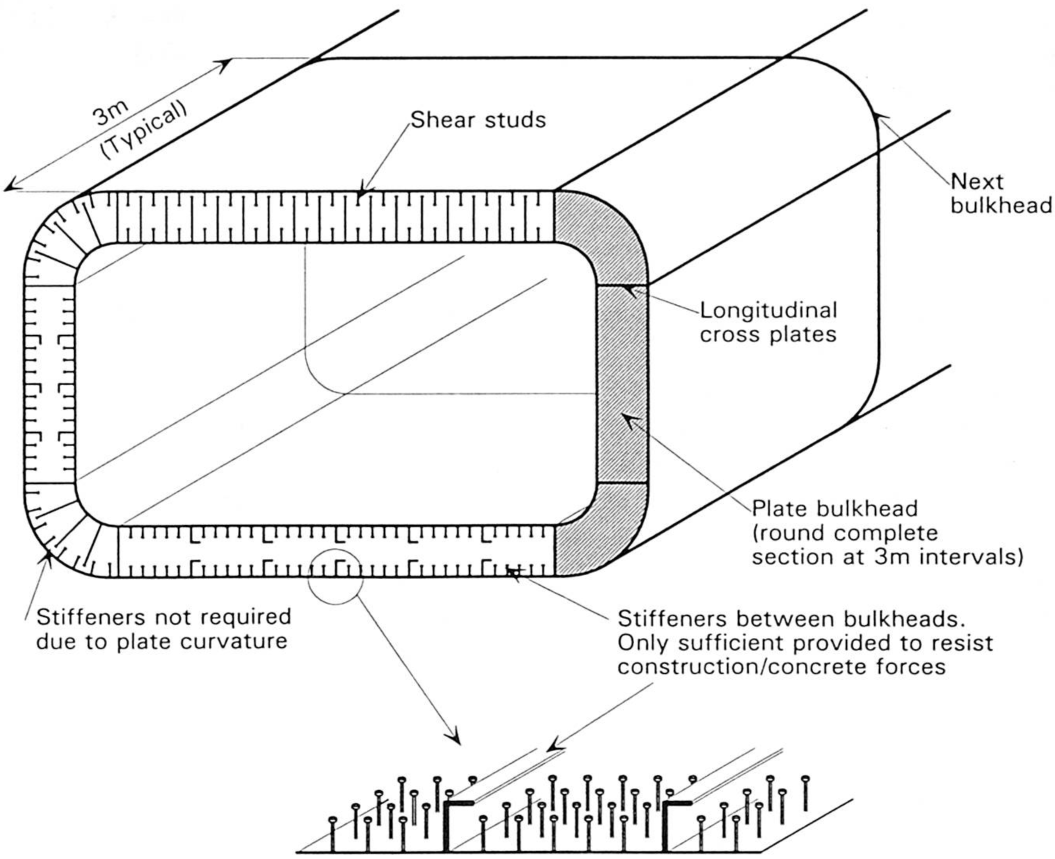

Fig.1 Steel-Concrete-Steel (SCS) construction using shear studs.

The interest in the use of double skin composite where both an inner and outer skin are fully utilised is more recent. As for steel construction, there are a variety of forms. That of concern here is referred to as SCS. This comprises of two steel plates with shear studs attached to each, see figure 1. The idea was first postulated for tunnels by the Tomlinson Partnership in response to preliminary work being carried out for the Conwy river crossing project in North Wales. Although essentially a concrete tube, the proposal was to enclose the concrete with an external steel skin. Why not make the skin work for you by keying it to the concrete using shear connectors? And why not provide a similar inner skin and dispense with most of the concrete reinforcement? Preliminary work indicated the scheme to be economically viable, and a program of research was instigated to provide the necessary design data.

Research continued for about 10 years, resulting in the publication of two design documents [3,4]. However, SCS has yet to be used on a major project. There are many reasons for this, a major one being the apparent mixing of steel and concreting trades on one project. But a second reason is undoubtably because it is not as easy to construct in SCS as had been anticipated. In particular, the relatively thin face plates need to be rigidly held in position during concreting. This substantially negates one of the major advantages of SCS which is that the steel plates act as permanent formwork. Either the plates must be stiffened, as in figure 1, or extensive temporary works are required. British Steel Bi-Steel, which is the subject of this paper, resulted from attempts to solve these construction problems. It is a patented construction system and the objective of this paper is to introduce Bi-Steel and thereby stimulate the designers of immersed tube tunnels to consider how it can be applied to these types of structures.

2 What is Bi-Steel?

2.1 Product description

Bi-Steel comprises of two parallel steel plates. Transverse bars are welded between the plates to hold them apart. The resulting panels can be considered as semi-rigid - they are stiff and strong enough to hold their shape during handling, but not so rigid as to prevent panels being fitted together.

The initial range of panel sizes available are summarised in figure 2. Note that it is possible to manufacture both flat and curved panels. The availability of curved panels opens up a number of interesting possibilities for immersed tunnel construction.

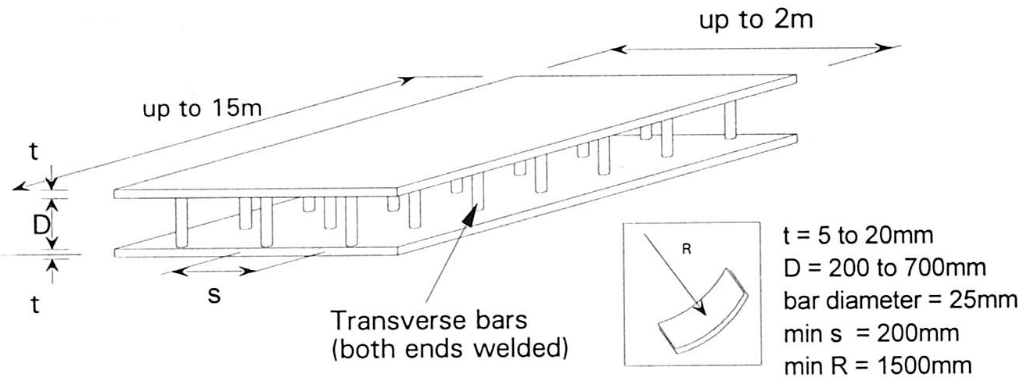

Fig. 2 Initial production range of Bi-Steel panels

In use, Bi-Steel panels would be welded together to form the shell of a structure. The panels would then be filled with concrete. It is the composite action of the steel and concrete which gives Bi-Steel its strength. Unlike concrete or steel construction, this composite action is fully utilised.

2.2 Basics of designing in Bi-Steel

The most useful way of understanding how Bi-Steel works is to regard it as a truss type structure. Figure 3 shows a section through a panel which is subject to simple bending. Using the truss analogy, the top steel plate and concrete act as a compressive top chord, the transverse bars act as tension ties, the concrete acts as diagonal compressive struts and the bottom steel plate acts as the bottom tensile chord. For the system to work requires that the relevant forces can be transferred at the 'joints', i.e., where the transverse bars are welded to the plates.

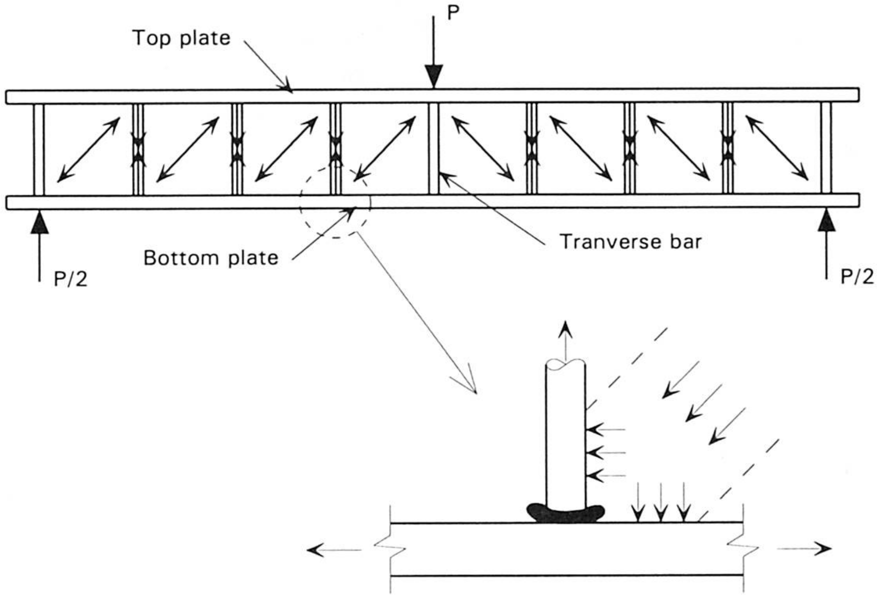

Fig.3 Diagram showing how Bi-Steel may be visualised as carrying load

The possible failure modes of Bi-Steel can be related to each of the elements in the truss analogy:

- *Compressive failure of top chord.* The top steel plate can carry a significant proportion of the compressive load. If this buckles, then there is a sudden loss of load carrying capacity. In practice the transverse bars are sufficiently close together that the compressive plate can reach yield stress before the onset of buckling.
- *Failure of transverse bar in tension.* This may occur by the bar yielding or by the weld connection to the plate failing. Yielding of the bar is a ductile failure mode. Usually the yielding leads to another failure mode being triggered (e.g., compression plate buckling). Weld failure leads to rapid load shedding.
- *Concrete compressive strut failure.* This tends to be gradual. Note that the concrete is confined and has no where to fail to unless a plate buckles.
- *Yielding of bottom plate.* This is a ductile type failure. As the plate yields, large amounts of energy can be absorbed by the structure.
- *Bar to plate joint failure.* This joint is under complex loading. The concrete is both pushing sideways onto the bar and down on the plate causing local bending. The bar is also in strong tension. The plate may be close to compressive or tensile yield. If joint

failure is by fracture, then a sudden failure occurs. It is therefore essential to achieve adequate joint toughness to ensure failure is ductile.

The Bi-Steel design guide teaches a comprehensive understanding of the above. The design method presented is based on providing adequate bar connectors in order to ensure that first failure is yielding of the steel plates. Special rules are presented to ensure that in compressive elements the steel plate does not buckle before yield stress is reached.

3 Benefits of Bi-Steel in immersed tunnel construction

3.1 Water exclusion

A primary objective of any immersed tunnel is to keep out water. Bi-Steel has two continuous steel barriers. Depending on the method of installation and element sealing technique selected, this double barrier may be maintained along the entire length of the tunnel.

3.2 Ductility

Bi-Steel has been tested in out-of-plane bending and has exhibited considerable ductility. Similar ductility may be anticipated for in-plane behaviour. An immersed or floating tunnel made from Bi-Steel will thus be highly tolerant to ground movement and accidental impact. This tolerance is enhanced by the orthotropic behaviour of a Bi-Steel panel - loads can be distributed in all directions rather than preferentially in one. A further observation arising from tests is that, when Bi-Steel is subject to large rotations, the steel plates deform rather than rupture. Thus water tightness is maintained.

3.3 Tunnel cross-section

Steel tunnels are generally round or octagonal in cross-section. Concrete tunnels tend to be rectangular in section. Because Bi-Steel is available as semi-rigid curved panels, there is much greater flexibility in the cross-section of tunnel that may be economically constructed. Figure 4 shows a number of options.

Figure 4(a) shows a rectangular cross-section. Bi-Steel is used in the walls and roof. However, the relatively thin Bi-Steel panels do not have enough mass to give the tunnel on bottom stability. This is therefore achieved using a thick concrete base. Extra benefit can be obtained from the backfill by adding small Bi-Steel wings.

Figure 4(b) shows a variant on (a) but using curved Bi-Steel panels. This reduces the thickness of panels. The voids around the traffic envelope may be used for fans, services, signals etc. Again, due to the low mass of the Bi-Steel, a massive concrete base could be used.

Figure 4(c) shows a tunnel cross section that may be used for a floating tunnel. The shape derives from two different diameter cylindrical elements. In Bi-Steel such a cross-section may be constructed as easily as a round section. Advantages relative to a round section are:

- an ability to better fit the traffic envelope
- less resistance to loads resulting from water currents. This is on account of the lower vertical height and more streamlined cross section
- greater vertical damping due to increased width
- less enclosed volume, and hence less need for internal ballast

The variations by using curved and flat Bi-Steel panels is quite extensive. What is significant is that there are options which are not readily available with the traditional materials.

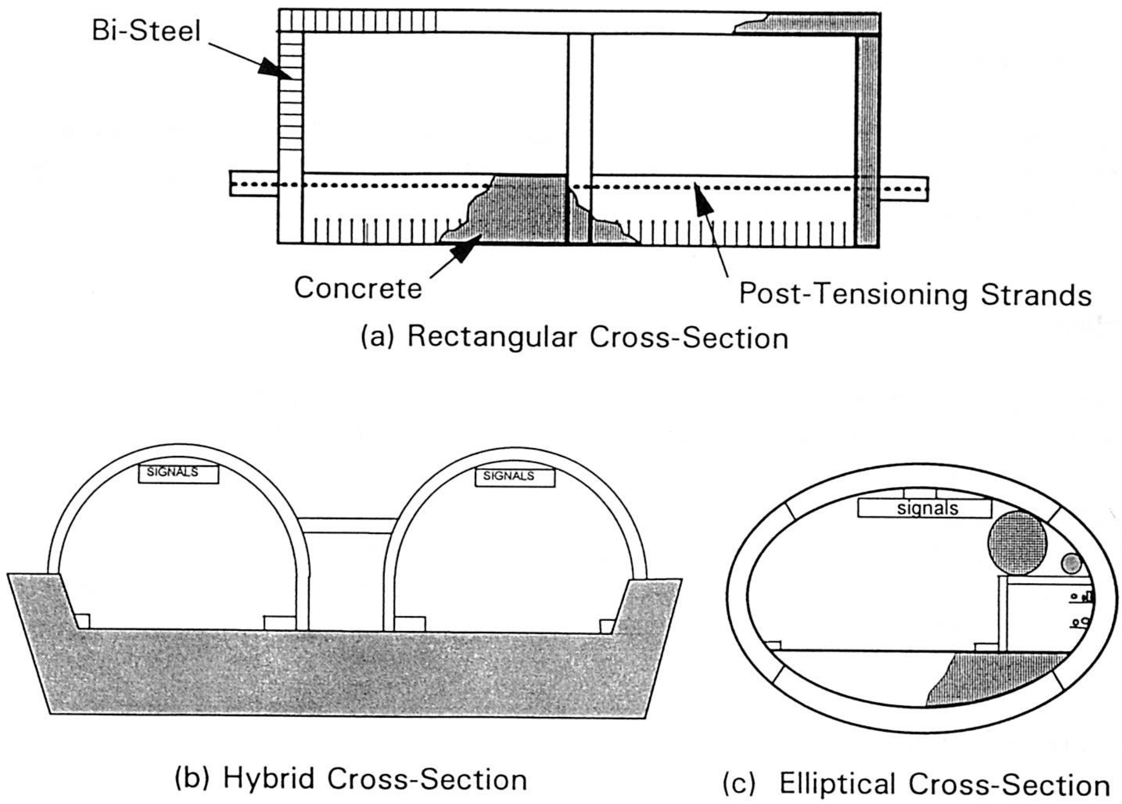

Fig. 4 Immersed tunnel cross-sections possible using Bi-Steel

3.4 Construction speed

Bi-Steel panels are factory produced to accurate tolerance. They may be delivered with surface coatings already applied. Where required, weld backing strips are fitted in the factory. In addition to assisting in the making of a good quality site weld, these also help the assembly of panels. A typical construction sequence is as follows:

- receive panels from factory. Store in a suitable location.
- using pre-fitted lifting lugs, crane panel into position. Note that unfilled panels are relatively lightweight - a 3.6 x 10 x 0.7 metre flat panel suitable for the roof of a two lane tunnel at 20m depth would weigh 8.5 tonnes empty, 69 tonnes when concrete filled.

- weld panels together. Semi-automatic welders running on pre-fitted tracks enable high quality welds to be made quickly and economically. Once one panel is positioned and part welded, adjacent panels may be fitted.
- on completion of welding, paint the weld zone (if required).
- fill panels with concrete. Mass concrete may be pumped in. Magnetic vibrators attached to the outside of the panel may be used to compact the plasticised concrete. Note that prior to placing the concrete the steel shell is already a substantial structure. Thus steel construction can proceed well ahead of concrete placement.

The above sequence can be carried out with surprising rapidity. It is estimated that a 90m x 10m tunnel element roof comprising 25 no. 3.6m x 10m panels could be assembled, welded, concreted and paint systems made good in 10 shifts. The labour on site required to complete this would be considerably less than for an all steel or concrete equivalent.

3.5 Construction flexibility

Concrete structures require large graving docks in order to be able to float the deep draft tunnel elements. US practice with steel tunnels has shown that shallower drafts enables tunnel elements to be built in conventional shipyards. Bi-Steel tunnels can be similarly built in shipyards, and then floated to deeper water prior to concrete filling.

Within a shipyard, Bi-Steel tunnel elements can be built vertically. This simplifies assembly and welding (no overhead welding). Due to their relatively light weight, the elements can be lowered down to horizontal and moved to position as required.

Building much larger elements in a vertical orientation is possible in deep water locations such as fjords. The general concept is shown in figure 5. Elements would be welded together and concreted off the side of a suitably fitted out flat barge. The completed section would be lowered into the water as construction continued. By suitable ballast control and mooring, tunnel elements of considerable length could be constructed. Once an element is completed, it would be upended and towed to site.

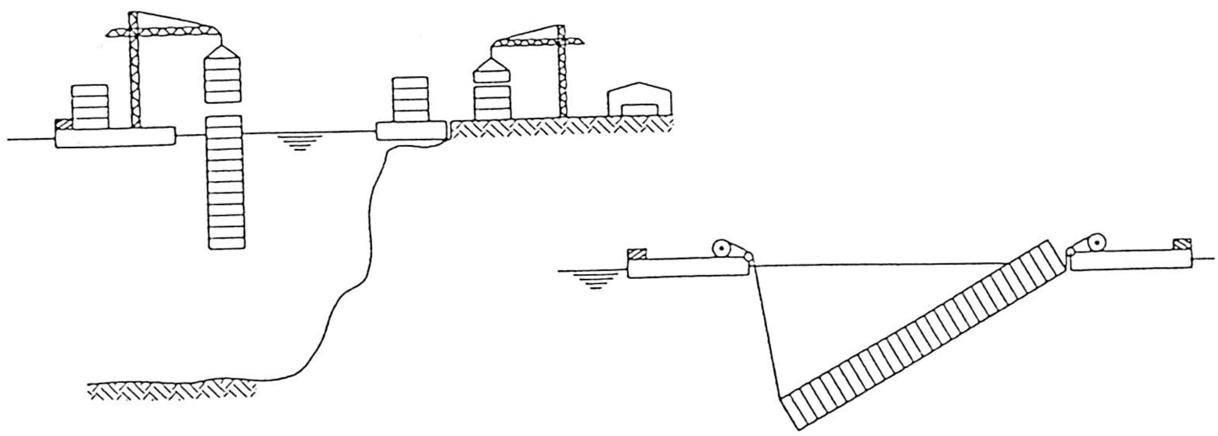

Fig. 5 Construction of tunnel elements in deep water

Where water depth is inadequate, a variation on constructing in a vertical orientation would be to construct on a slip way. As the tunnel elements are completed, so the tunnel is lowered into the water. In order to minimise loading on the slip way, the tunnel sections may be concreted as they enter the water. The nature of the construction sequence with Bi-Steel makes such a 'launched' element possible, saving both on graving dock and upending costs.

The ultimate extension of the above concept is to build the immersed tunnel as a continuous element. This is illustrated in figure 6. In this particular example the tunnel is part of a circular arc enabling all joints to be rigid. First step is to construct the approach roads which double as slip ways. Tunnels elements are then assembled on the slip way. A winch on the opposite bank pulls the tunnel across. In order to avoid stiction to the soil, provision for water jetting would be provided. Once the tunnel is completely across the channel, the external of the tunnel would be sealed off at the water bond. Advantages of this method are a lack of underwater joins (the tunnel is a continuous, double skin construction), reduced marine operations (dredging would still be required), speed (3m a day is considered attainable) and an ability to carry out a level of outfitting whilst the tunnel is being constructed.

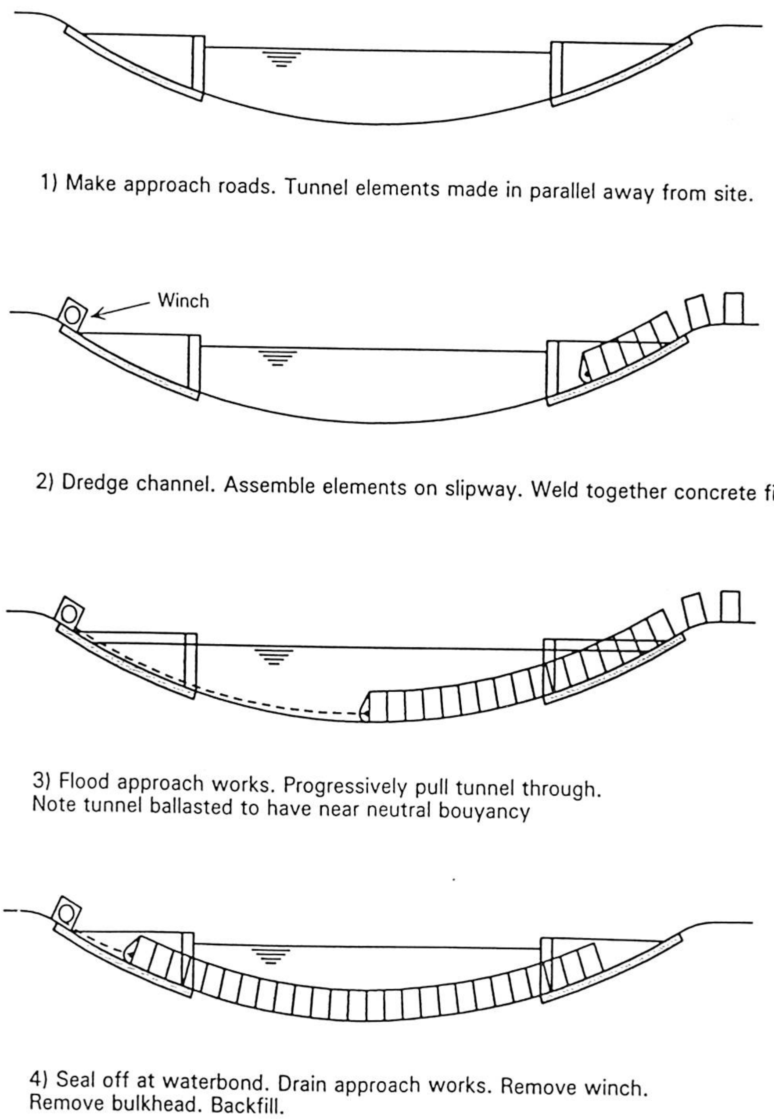

Fig. 6 Example of a launched tunnel using slipway construction

4 CONCLUSION

Immersed tube tunnels are not new, having been around for nearly 90 years. During this time nearly all tunnels have been built from concrete or steel. Steel solutions have made use of the concrete associated with them, but have not needed to mobilise the full potential of the composite action. Whilst some composite tunnels have been considered and tried, they have not become widely accepted. A reason for this is issues associated with construction.

British Steel Bi-Steel is a new, unique and patented construction system. It has been developed specifically with the objective of enabling large, composite structures to be easily constructed. However, for structures such as immersed tube tunnels, the material has a number of other benefits. The objective of this paper has been to introduce engineers and contractors to Bi-Steel. It is hoped that the ideas it contains will have stimulated some to consider ways in which Bi-Steel may be advantageously applied - it is the view of British Steel that Bi-Steel offers opportunities for considerable cost reductions if appropriately applied.

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Soldier Pile Tremie Concrete Walls As Part of the Final Tunnel Structure

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Summary

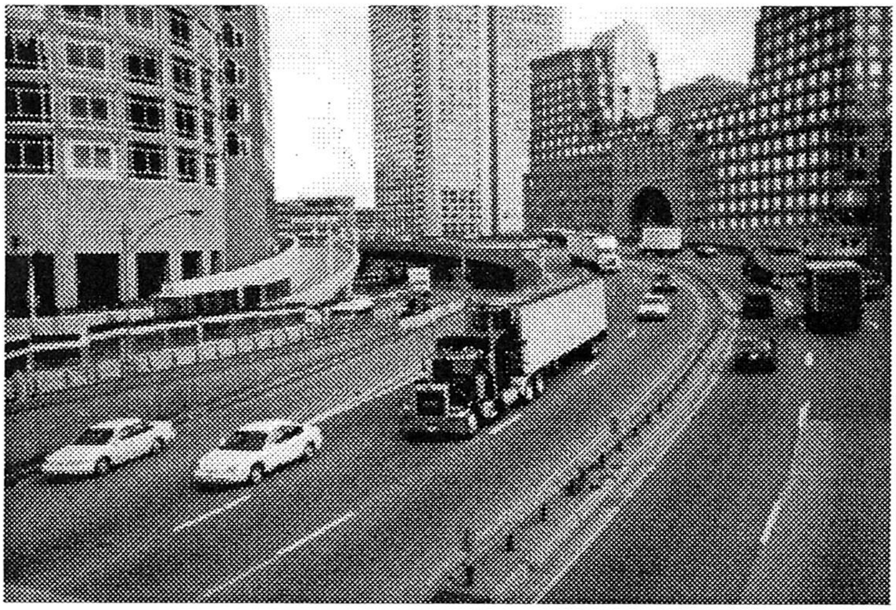
This paper discusses the use of Soldier Pile Tremie Concrete (SPTC) Slurry Walls as permanent tunnel walls on the Massachusetts Highway Department (MHD) Central Artery (I-93)/Tunnel (I-90) (CA/T) project in Boston, U.S.A.. It describes design considerations for choosing the SPTC walls as part of permanent tunnel structure as well as methods of analysis, design and evaluation of construction staging.

1.0 Introduction

The I-93 portion of the Central Artery (I-93)/Tunnel (I-90) project includes over 6.1 kilometers (20,000 feet) of slurry walls. Most of the walls are to be constructed as Soldier Pile Tremie Concrete (SPTC) walls. Steel wide flange soldier piles are installed at 1.2 meter to 1.8 meter (4 to 6 feet) spacing in a slurry trench and tremie concrete is placed to form the concrete wall. The steel wide flange piles form the primary reinforcement. In most areas, the concrete is designed to act as "lagging" spanning between structural steel piles. During construction, the walls are used as an excavation support system. After excavation is completed, the tunnel concrete base slab is moment-connected to the SPTC walls. Steel composite roof beams are connected to the walls in a pined connection to form the tunnel.

Many SPTC wall panels need to be built under the low headroom constraint imposed by the existing Central Artery viaduct. This six lane expressway viaduct must remain in service; thus using the SPTC walls as the foundation to underpin the existing viaduct during excavation and tunnel construction.

Figure 1 shows the existing Central Artery looking North. The tunnel will include three or four SPTC walls along its cross section. The west wall of the proposed tunnel and the most of the middle wall will be built below the existing viaduct. Special equipment is necessary for construction under the low head restriction of the existing viaduct, in some places as tight as 4.3 meters (14 feet). Since the soldier piles are 24 meters (80 feet) to 30 meters (100 feet) long, the piles need to be spliced using bolted splice connection.



Figire 1. Existing Central Artery Viaduct Looking North

For low head room construction, SPTC walls have an advantage over a heavily-reinforced conventional concrete slurry wall. The piles are easier to assemble and splice than multi-layer reinforcement bar cages. This ease of construction was an important consideration in downtown Boston, considering difficult soil conditions and tight construction work zones. Another benefit was that the SPTC walls are stiffer than a conventional concrete slurry walls of the same thickness, leading to improved performance for protection of existing buildings adjacent to the site.

2.0 Design Consideration

Since the SPTC walls are used both as part of support of excavation and as permanent final structure, it is necessary to consider the construction staging load along with the permanent load condition for the stress and deformation analysis. Construction stages include sequential excavation and strutting with cross lot braces, installation of the base slab, sequential removal of struts, installation of the roof, and placement of backfill. The load on the SPTC wall during the construction stage, which is temporary, is different from that in the final stage. Although the magnitude of total load on the SPTC wall in the temporary construction stage is less than that in permanent condition, it is very important to realize that most lateral deformation of the SPTC wall is associated with the excavation process. The construction induced ground movement, both horizontal and vertical, is of most concern to the project.

A unique aspect of the SPTC wall design is that the maximum lateral deformation occurs at a stage when the total lateral load is relatively small compared with its maximum magnitude. The excavation induced lateral movement will reduce the lateral soil pressure to some magnitude equal to or greater the active soil pressure depending on deflection of wall. Typical tunnel load diagram used for design is shown on Figure 2.

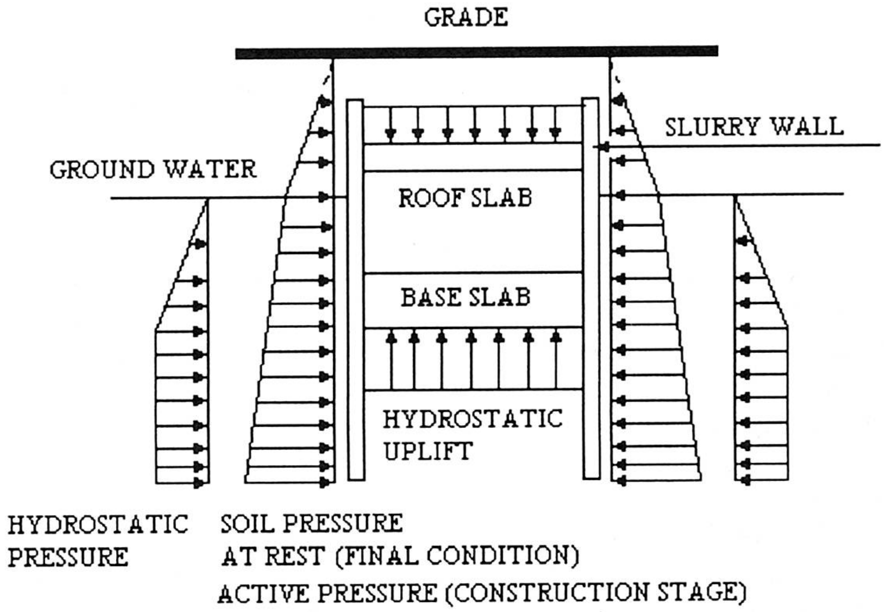


Figure 2. Typical Tunnel Load Diagram

In the final condition, the soil pressure will increase over time and will reach at rest value. This requires the structural design of the SPTC wall to first consider deformation control at the construction stage when the load is relatively small, and then the strength of the wall for the permanent condition. For this purpose, the CA/T project adopted a design approach to use an allowable stress design method together with additional stringent deformation control requirements for design of the SPTC wall during temporary construction stage and Load Factor Design for the final condition.

3.0 Analysis of the SPTC Wall

Two general methods of analysis have been used for analysis and design of SPTC walls. The "structural model" uses frame analysis for structural design and selection of wall members. The material properties of the structural elements in the SPTC wall are assumed to be homogeneous, isotropic, and elastic. Nonlinearity associated with the staged excavation is significant to the stress and strain in the SPTC wall and, therefore, needs to be modeled accordingly. Both linear and nonlinear finite element programs have been used for SPTC wall analysis to achieve acceptable results. The "geotechnical model" considers the overall stiffness of the wall support and soil system, and has been used for evaluation of potential construction-induced effects to buildings along the tunnel right-of-way. This method utilizes the Finite Element Analysis to model soil and use the vertical wall as a boundary condition.

3.1. Structural Model

The wall structural model for the construction phase is nonlinear and requires stress-history-dependent soil structural interaction analysis. Prior to tunnel excavation, the newly-installed wall is in a state of equilibrium with the surrounding soil. The first stage excavation on one side of the slurry wall creates different ground elevations and mobilizes soil pressure on the retained side to push laterally against the wall. The soil below the bottom of excavation acts as support to maintain the stability of the wall. To understand the nonlinear behavior of the system, it is important to note that the wall deflects into the excavation some distance below the strut support level, before the strut can be installed. What this implies is that the wall will deflect more into the excavation than would be predicted by a linear model of loads applied to the wall and full-excavated depth strutting levels.

In order to minimize the ground movement, the strut is preloaded between walls. In other words the strut acts as a prestressed spring support to the wall with spring constant equal to the axial stiffness of the strut. Preloading of the strut pushes the wall against the retained soil and generates additional soil reaction on the wall. Each stage of excavation is about 3.66 meters (12 feet). For each subsequent excavation stage, the excavation support structure experiences following changes sequentially:

- Excavation removes the support of subgrade soil to the slurry wall and lowers the wall's soil support down to the bottom of the excavation stage.
- Excavation releases the subgrade soil reaction of the previous stage and causes stress redistribution between the wall and soil below the bottom of the excavation.
- Excavation results in change of lateral soil pressure outside walls.
- Installation of a support strut adds a support point in the model, which is treated as a linear spring in the analysis.
- Preloading of the strut may generate additional soil reactions on the wall by pushing the wall back into the soil.

The final stresses in the wall are accumulated in increments from one excavation stage to the next. As the excavation proceeds, the load on the wall increases while the structural configuration of the wall/support system as well as its boundary conditions change. The final stresses in the wall are not linearly related to the displacements of the structure, requiring an incremental nonlinear analysis.

To use a linear elastic structural software program for the SPTC wall analysis, the structural model assumes that all loads mobilized by excavation will be applied to the wall at once for each excavation stage. The inward deformation of the wall at a strut level prior to the strut's installation is accounted for by introducing a fictitious concentrated load at the position where the strut is to be placed. The fictitious load forces the wall to move inward by a deflection amount equal to the deflected shape prior to strut placement. The subsequent excavation stage analysis can then be run with the new level of strutting in place, and the wall correctly deflected inward. For the computer software program with nonlinear capacity, each stage of excavation may be modeled sequentially from top to bottom. The inward movements and stress in the previous stages of excavation will be accounted accordingly.

The thickness of the SPTC slurry wall varies between 0.9 to 1.2 meter (3 to 4 feet) thick. The soldier piles are spaced between 1.21 meter to 1.83 meter (4 to 6 feet). The structural steel soldier piles are designed as primary load carrying members and the concrete between the soldier piles are considered as lagging between the piles. The flexural stress of the concrete between soldier piles is very small, so no steel bar reinforcement is needed. However, in some locations where the surcharge loading from adjacent building foundations is high, reinforcement may be needed. In most cases, W36 x 393 wide flange sections with 3.45×10^5 kpa (50 ksi) yield strength are used as soldier piles. The minimum 28 days strength of the tremie concrete is 2.76×10^4 kpa (4000 psi) with a slump between 17.78 to 25.4 centimeters (7 to 10 inches).

The concrete base slab thickness varies between 2.44 to 4.57 meters (8 to 15 feet). The base slab is rigidly connected with the steel soldier piles as shown in Figure 3. The roof structures are simply supported from the soldier piles. The roof structures are composite structural steel plate girders with a concrete roof top slab. Waterproofing material is provided under the base slab and on the top of the roof slab.

3.2. Ground Movement Model

While the structural analysis was performed to size the structural elements in the wall, the geotechnical model was developed as a tool to compare different systems of wall stiffness for predicted performance during the excavation. As excavation proceeds, the wall tends to deform into the excavation, leading to soil movement behind the wall. This movement, along with other factors such as consolidation due to dewatering and construction vibration, is of great concern for protection of existing structures along the right-of-way.

For this analysis, the soil and structure are modeled as a mass. The mass is broken up into discrete elements and assigned elastic or inelastic properties. Typically, the analysis assumes an elastic model for structural elements and an inelastic, hyperbolic stress-strain model for the soil. The model predicts wall and soil movement by simulating the states of stress caused by the construction process. This analysis includes the same stages of construction used in the structural analysis. Unlike the structural model, the geotechnical model includes some stiffness of the unreinforced concrete between piles as part of the overall stiffness of the system. To ignore the stiffness of the concrete, which partially encases the soldier piles, would be over-conservative when trying to estimate movement of the walls during construction.

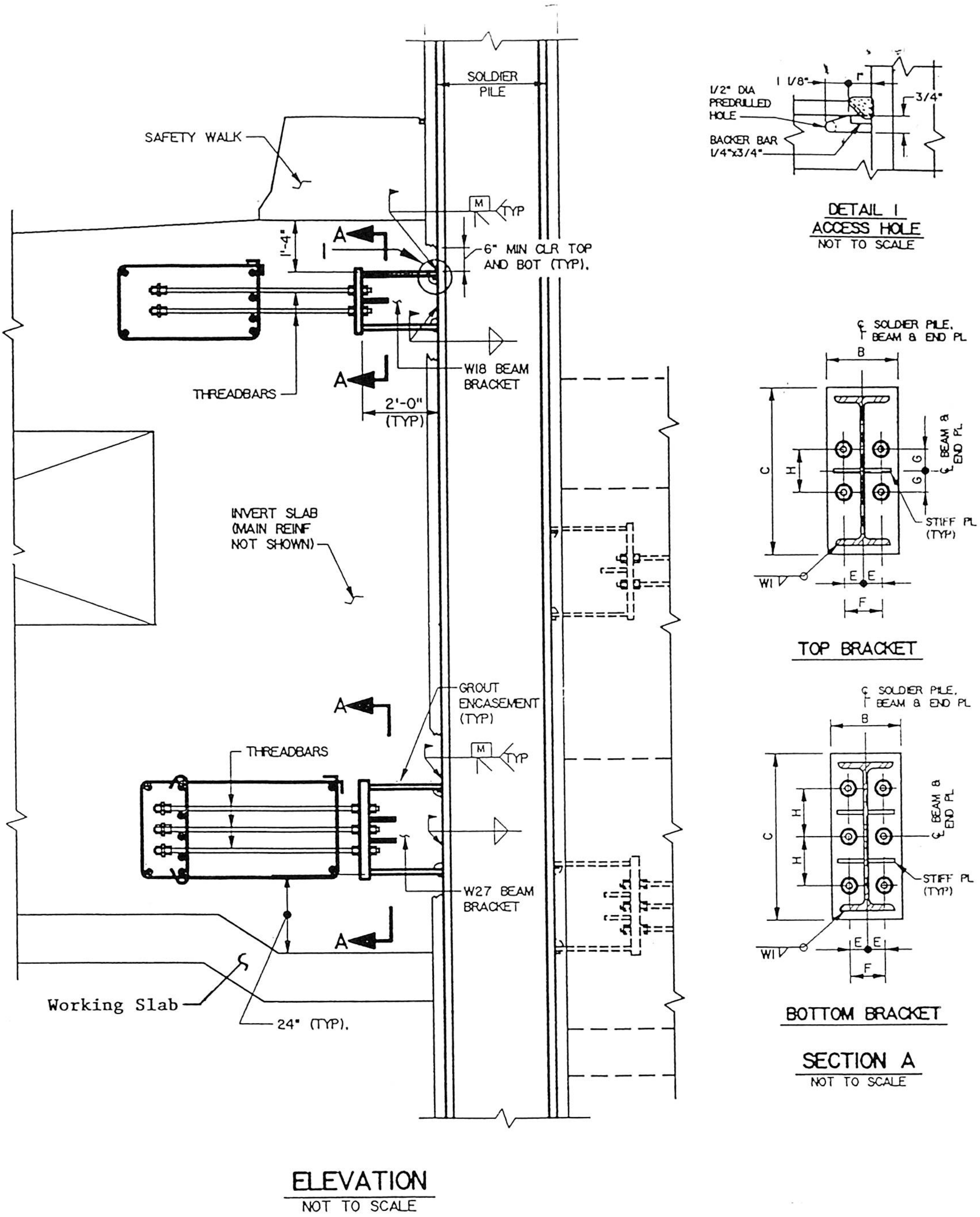
The geotechnical analysis includes elastic modeling for wall and bracing elements, hyperbolic stress-strain relationships to model the behavior of the soil mass, and a capability of simulating staged construction. For this last capability, the engineer makes assumptions about the sequence of the excavation (similar to the structural analysis), including depths of each cut and installation of bracing. The model sequentially deactivates blocks of soil and adds bracing pre-loads. The soil mass behind the wall reacts to the modeled construction behavior based on the constitutive material properties input into the program. The geotechnical model is analytically more refined than the structural model. It does not require any assumptions regarding lateral load on the wall, but requires an estimation of soil parameters that is difficult to verify.

Therefore, the geotechnical model is good for wall performance comparison, but it has limited ability to precisely predict soil movement behind the excavation support wall.

The movement of the SPTC slurry wall and the adjacent buildings are monitored by extensive instrumentation system to confirm the design assumption and to activate the contingency plan, if the deflection exceeds the threshold values. Threshold Values have been established to provide advance warning of potential problems or detrimental effects related to construction and are typically one half of the Limiting Values. Both Threshold Values and Limiting Values are based on the allowable ground movements the adjacent facilities and utilities can take with acceptable impact. Due to fact that the different facilities and utilities have different sensitivities to the ground movements, the Threshold Values and Limiting Values vary from location to location.

Acknowledgments

The authors acknowledge the Massachusetts Highway Department, the Federal Highway Administration and the Department's Management Consultant, Bechtel/Parsons Brinckerhoff, for the organizations' support in preparation of this paper.



INVERT SLAB TO SPTC WALL CONNECTION

FIGURE 3

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