PII:S0886-7798(97)00005-9

# Chapter 1

# INTRODUCTION

by

**AHMET GURSOY**
**Parsons Brinckerhoff International, Inc.**
**U.S.A.**

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.83-86, 1997

Printed in Great Britain
0886-7798/97 $17.00+0.00

Pergamon

PII:S0886-7798(97)00005-9

# Chapter 1

# INTRODUCTION

by

**AHMET GURSOY**
**Parsons Brinckerhoff International, Inc.** **U.S.A.**

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.83-86, 1997

Printed in Great Britain
0886-7798/97 $17.00+0.00

Pergamon

# Chapter 1: Introduction

1. SECOND EDITION: STATE-OF-THE-ART REPORT

2. HISTORY OF THE WORKING GROUP

3. CONTENTS OF THE REPORT

4. ACKNOWLEDGMENTS

## 1. Second Edition: State-of-the-Art Report

The first edition of the state-of-the-art report on immersed and floating tunnel technology was published in 1993. The 170-page report received immediate worldwide interest and demand for reprinting. Recognizing this global demand, in 1994 the International Tunnelling Association (ITA) Working Group on Immersed and Floating Tunnels started preparations for the second edition, to include four new chapters and two appendices, covering environmental issues, hazard analyses, transportation of tunnel elements and seismic analyses.

In 1995, the working group decided to renew and update four of the major topics covered in the 1993 edition: structural design, watertightness, submerged floating tunnels, and the catalogue of tunnels. In the author credits for these chapters, only the major authors and those who contributed comments for the second edition are listed.

In updating the tunnel catalogue, the group also decided to provide valuable information on utility tunnels constructed globally. Appendix A, presents a condensed version of a report prepared by the European National Highway Research Laboratories (ENHRL) on Submerged Floating Tunnels (SFT). In Appendix B, a general description of Instrumentation, Documentation and Verification (IDV) system is presented for future SFT projects.

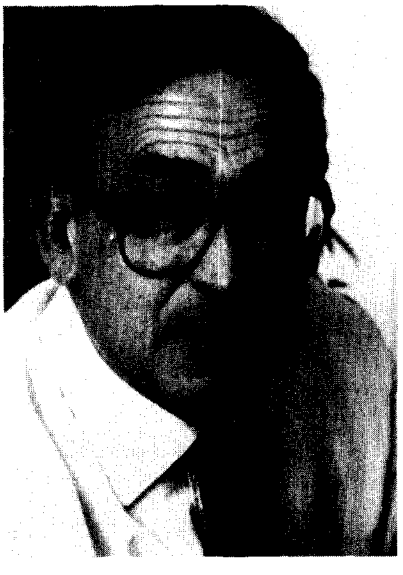
The second edition reflects various aspects of immersed tunneling techniques based on input of tunnel experts from the United States, Norway, Sweden, Denmark, United Kingdom, the Netherlands, Germany, Belgium, Italy, Japan, and China. We trust that this edition will contribute to a much better understanding of the conditions under which immersed tunnels are designed and constructed, based on the experience gained in publishing the first edition.

## 2. History of the Working Group

The ITA Working Group on Immersed and Floating Tunnels was established in 1988, at the ITA conference in Madrid. Mr. V. L. Molenaar served as the group's Animateur for one year, after which Mr. P. C. Van Milligen assumed this position until 1993. In 1994, Mr. A. Gursoy became the Animateur and Mr. Jan Saveur the Vice Animateur. The group's tutor from 1988 to 1992 was Mr. B. Pigorini, who was succeeded by Prof. A. Glerum until 1993. Mr. W. DeLathauwer became the tutor for the group in 1994.

The first meeting of the Working Group was held in Manchester, United Kingdom, in 1989. At that meeting five topics for publication were selected and five subcommittees were established. The first edition of this report was the result of the work produced by these sub-working groups.

From the beginning, the working group members displayed great interest in reviewing the state-of-the-art characteristics of immersed and floating tunnels. For immersed tunnels, the examination of the two major construction methods—one practiced in Europe, the other in the United States—was the focal point in setting the group's early agenda.



*Ahmet Gursoy, Working Group Animateur*

## 3. Contents of the Report

This report includes discussions on the state-of-the-art in immersed and submerged floating tunnels, with special emphasis on basic features of global construction practices for steel and concrete tunnels.

Chapter 2 is a glossary of terms used in immersed and submerged floating tunnels.

Chapter 3 examines structural design features of steel and concrete immersed tunnels including construction stages, accident loads, load factors for combinations of loadings, and material specifications.

Chapter 4 deals with specific details of the design for steel and concrete tunnels to obtain watertightness, concentrating mainly on the various joints used in steel tunnels and on membranes, coating techniques and joints for concrete tunnels.

Chapter 5 addresses the environmental issues associated with immersed tunnels. It covers effects on the watercourse and the groundwater regime, disposal of excavated material and land use issues. Case studies are used extensively to illustrate typical problems and their solutions. Recommendations for good practice are made.

Chapter 6 discusses the important accidental loads that can affect an immersed tunnel. These include internal flooding, sunken ship loading, falling anchors, dragging anchors, life safety, fire, and explosion. Design approaches are described for each loading.

Chapter 7 covers transportation of tunnel elements from their construction site to the tunnel site, with particular emphasis on the problems of sea transport.

Chapter 8 deals with the topic of earthquake analysis for immersed tunnels.

Chapter 9 comprises a catalogue of immersed tunnels, starting with the first Detroit River Tunnel in 1910, and continuing through to very recent tunnels that are still under construction. The second edition of the tunnel catalogue includes:

i. A completely new catalogue of utility tunnels.
ii. Information on additional transportation tunnels.
iii. Revised and updated information on the tunnels included in the first edition.

Chapter 10 includes a review of the state-of-the-art in submerged floating tunnels. For this edition, the current global status of SFT projects is described and the technical aspects of the 1993 state-of-the-art report are updated.

Appendix A is an edited and condensed version of the Submerged Floating Tunnels Analysis Project report by the European National Research Laboratories on Submerged Floating Tunnels.

A brief, edited version of an IDV (Instrumentation, Documentation and Verification) system indicating the future R&D incentives to be used for SFT applications is presented in Appendix B.

## 4. Acknowledgments

The work presented in this publication presents the results of the working group's tireless efforts, conducted on a truly global scale. Credit is due to the principal authors and collaborating working group members who performed the work with dedication and care. The active membership (as of May 1996) is listed in Appendix C.

PII:S0886-7798(97)00006-0

# Chapter 2

# GLOSSARY OF IMMERSED TUNNEL TERMS

**by**

**JAN SAVEUR** — **The Netherlands**
**Volker Stevin Construction Europe bv**

**CHRISTIAN INGERSLEV** — **U.S.A.**
**Parsons Brinckerhoff Quade & Douglas, Inc.**

***Contributions and comments by:***

**Ahmet Gursoy** — **U.S.A.**

**Walter Grantz** — **U.S.A.**

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.87-92, 1997

Printed in Great Britain
0886-7798/97 $17.00+0.00

Pergamon

# Chapter 2: Glossary of Immersed Tunnel Terms

**This glossary is intended to facilitate international communication on the subject of immersed tunnels in the English language. The terms defined relate to typical design and construction practice for steel and concrete immersed tunnels.**

***Access Shaft:***
Temporary access shafts are commonly provided to allow entry of personnel and occasionally equipment to the interior of an immersed tunnel while floating or submerged. The shafts are usually removed when alternate access is available, such as along the tunnel. The access shaft may be attached to the temporary end bulkhead or may be attached over a temporary hole in the structure which will later have to be made watertight.

***Ballast:***
(1) *Permanent Ballast:* Non-structural solid material placed inside or outside an immersed tunnel to increase its effective weight permanently. Material placed outside should either be attached to the tunnel or retained, thereby preventing accidental falling off or loss of the material. Backfill that may be scoured or accidentally dredged away is not ballast.
(2) *Temporary Ballast:* Material used to temporarily increase the effective weight of the tunnel or a tunnel element during the fabrication and installation phases until replaced by backfill or permanent ballast. The material may be solid or liquid.

***Backfill:***
Material placed around the sides and over the top of the tunnel within the excavated trench after the tunnel is installed in the trench. The material is usually granular, rock, or excavated material.

***Binocular Section:***
A term used to describe an element consisting of two adjacent circular steel tunnels, usually each of two lanes, combined into a common structure.

***Bore:***
A term borrowed from mined tunneling to describe a cell.

***Box (Shape):***
An indication that the overall cross-section of the tunnel is approximately rectangular.

***Bulkhead:***
An upright watertight partition used to generate compartments, usually totally closing off the inside of a cell. Temporary bulkheads are provided at the ends of tunnel elements to keep water out (make them watertight) during the floating and installation stages.

***Buoyancy:***
(1) The resultant upward force on a body partially or fully immersed in a liquid, caused by the pressure of the liquid acting on the body. The magnitude of the force is equal to the weight of liquid displaced.
(2) *Positive buoyancy or Negative Buoyancy:* Jargon expressions for the amount by which buoyancy exceeds the weight of a body when totally immersed in a liquid. Positive buoyancy indicates that the body tends to float (buoyancy > weight); negative, that it tends to sink (buoyancy < weight).

***Casting Basin:***
A place where elements for immersed tunnels can be fabricated in the dry, and which can be flooded to allow the elements to be floated out and taken away. Generally used for concrete tunnels.

***Cell:***
Continuous space within the cross-section of an element, bounded by walls, floor and ceiling. A cross-section may contain many cells, hence multiple-cell box, where for example separate cells may be used for each traffic direction, emergency egress, utilities, supply air and exhaust air.

***Chamfer:***
Corners of box section tunnels are often chamfered (bevelled, with the corners missing) to remove unnecessary space where it serves no useful purpose, or thickened to reduce moments and shears (haunches) and to allow dragging anchors to pass more easily over the tunnel.

***Cill:***
Usually the highest point of the floor on which or against which the gate rests, and over which elements must pass during removal from the fabrication facility.

***Closure Joint:***
See ***Joint, closure.***

***Concrete Tunnel:***
Term applied to a tunnel not designed to leave the fabrication facility until the external concrete structure is essentially complete. Steel plate, if used, is usually limited to acting as a waterproofing membrane. (See also ***Steel Tunnel***)

***Draft:***
The depth below the still-water surface of the deepest part of a floating body.

***Dam Plate:***
Term used in the United States for the temporary end bulkhead.

***Dredging:***
The operation of excavating the trench. It is usually carried out in two stages, first bulk dredging, then trimming the excavation shortly before placing an element. Compensation dredging refers to additional dredging of a waterway to make up for loss of water depth elsewhere in the cross-section.

***Dry Dock:***
Usually a man-made area that can be dewatered for the repair of ships. A dry dock may also be a semi-submersible floating structure. Immersed tunnel elements are sometimes fabricated or repaired in dry docks. The term is also sometimes applied to a graving dock or casting basin.

***Duct:***
A term used to describe a cell, particularly for supply or exhaust ventilation, or for utilities.

***Element:***
A length of tunnel that is floated and immersed as a single rigid unit. The rigidity may be temporary and later released.

***End of Tunnel Element:***
(1) *Primary or Inboard End:* The end of the tunnel element that is to be connected first. This end will face either the previously immersed and adjoining element, or the terminal structure. This end is usually the end equipped with the immersion gasket.
(2) *Secondary or Outboard End:* The other end of the tunnel element.

***Fabrication:***
The stage of construction of a tunnel element before it can float. The fabrication facility may be a casting basin, graving dock, dry dock, ship yard or a green field site. The construction of a tunnel element may need to be completed at an outfitting dock.

***Factor of Safety*** (with regard to uplift):
The ratio of the weight of a tunnel, or a portion thereof, to the buoyancy. Different required factors of safety may be specified depending upon whether backfill is included or removable items are excluded, and depending upon the stage of construction. Water density must be specified, since buoyances will vary with changes in water density.

***Fitting Out:***
Also known as outfitting, this term refers to work that is carried out while the element is afloat. It may consist, for example, of completing any remaining necessary construction of the element prior to immersion, the addition of ballast, the installation or removal of temporary equipment such as navigation lights, survey beacons, and access shafts, and adjusting the trim of the floating element. Some of the work may be necessary before transportation (towing), but the remainder must be completed after towing. Additional construction applies mainly to steel tunnels where much of the internal structural concrete may not be completed until the element is close to its final destination. Some of the work may not be carried out until the element is supported by the immersion equipment.

***Freeboard:***
The height above the still-water surface of the highest part of a floating body.

***Gasket:***
(1) A device that acts as a seal between two contacting surfaces.

(2) *Gina Gasket:* A proprietary form of gasket used to seal immersion joints, particularly on concrete tunnels. It consists of a full-bodied rubber section able to transfer large compression forces, and a soft nose able to provide an initial seal under low compression. For binocular sections, each circular tunnel usually has its own gasket around the perimeter, whereas most other forms of tunnel use a single gasket around the external perimeter. The gasket provides a temporary seal and compression contact face during immersion installation, remains in place, and may provide a permanent seal at flexible joints.

(3) *Omega Gasket or Seal:* This seal, shaped like the Greek letter Omega (Ω) is installed across flexible immersion joints from within a tunnel after immersion and joining. It may form a secondary permanent seal or it may become the primary seal. It is bolted to the internal faces each side of the joint. It may be replaced in a similar manner on an as-needed basis. Because of its shape, it can sustain fairly large longitudinal and transverse movements at the joint.

(4) *Soft-nosed Gasket:* See *Gina Gasket*, above.

(5) *Temporary Immersion Gasket:* This is usually an extruded rubber section that acts as a seal when it is compressed. After completion of the permanent joint, the seal is no longer needed. This type of gasket is commonly used in the United States.

***Gate:***
Usually either hinged to a wall or floating, this structure is used to close off the fabrication facility from the adjacent water to allow dewatering of the facility.

***Gravel (Bed) Foundation:***
See ***Screeded Foundation.***

***Graving Dock:***
An area that can be dewatered to form a casting basin.

***Green Field Site:***
An area above water level converted to enable construction of tunnel elements, usually steel shell tunnels. The elements may be side or end launched into the water when capable of floating, or may be incrementally launched.

***Grouted Foundation Bed:***
A foundation formed by filling the space between the underside of an element and the pre-excavated trench bottom with grout. Until this operation is complete, elements require temporary support.

***Haunch:***
A thickening of a wall or slab to increase locally the bending strength and shear capacity of the section.

***Immersion:***
The phase of construction covering the period between the element floating on the surface and installed on its foundation or temporary supports at bed level.

***Incremental Construction:***
A method of construction whereby a short section of an element is constructed, then jacked along to enable the adjacent section to be cast against the previous section.

***Installation:***
This phase of construction covers preparation for immersion, the immersion, foundation preparation, backfilling, and completion of the interior works.

***Jet Fan:***
A ducted propeller, usually mounted adjacent to or above the traffic, that helps to maintain air velocity within that cell.

***Joint:***
(1) *Closure or Final Joint:* Where the last element has to be inserted rather than appended to the end of the previous element, a marginal gap will exist at the secondary end. This short length of tunnel will need to be cast-in-place and is known as the closure or final joint.

(2) *Construction Joint:* A horizontal or vertical connection between monolithic parts of a structure, used to facilitate construction. A waterstop is commonly placed in such a joint.

(3) *Earthquake Joint:* An immersion joint of special design to accommodate large differential movements in any direction due to a seismic event. It is also applied to a semi-rigid or flexible joint strengthened to carry seismic loads and across which stressed or unstressed prestressing components may be installed.

(4) *Expansion Joint:* A special moveable watertight joint between segments of a tunnel element.

(5) *Immersion Joint:* The watertight joint that is dewatered when an element is installed at the seabed. It may remain flexible or can be made rigid, as is usual with steel tunnels. A temporary immersion gasket or soft nosed gasket is usually used, and an omega seal may also be installed later.

***Keel Clearance:***
The least vertical distance between the deepest part of a floating body and the bed beneath.

***Keel Concrete:***
Concrete, often ballast, placed in the lowest portion of an element.

***Lifting Lugs:***
Temporary lifting points from which an element is suspended during immersion, usually removed after an element is set on its foundation.

***Locking Fill:***
Backfill, usually granular, placed carefully around the lower part of a tunnel to hold it in position.

***Outfitting:***
See ***Fitting Out.***

***Portal:***
The structure or the end face of the structure at the two ends of the tunnel at the interface of the covered and open sections.

***Prestress, Temporary:***
Used mainly in concrete tunnels to temporarily lock a flexible joint, to modify stresses until immersion, or to provide additional strength during transportation and installation.

***Pumped Sand Foundation:***
See ***Sand Bedding.***

***Rigging:***
A system of lines, winches and hoists used to control the position of an element, both horizontally and vertically, especially during immersion. Lines may be attached indirectly to the shore, anchors, pontoons, derrick barges or other lowering equipment.

***Rock Protection or Armour:***
The provision of larger stone or rock to prevent erosion or dredging of required backfill or bed. The term is also applied to systems for protecting a tunnel against potential collisions and dragging anchors.

***Roof Protection:***
Protection provided to the waterproofing membrane on the roof against accidental damage. Also applied to combinations of backfill and rock protection placed above the roof to protect against sinking or grounding vessels, etc.

***Sand Bedding:***
A foundation formed by filling the space between the underside of an element and the pre-excavated trench bottom with sand. The sand is placed hydraulically with the sand-flow or sand-jetting method. Until this operation is complete, elements require temporary support. A small gap may exist at the underside of the element after this operation, so that the temporary supports must be released or deactivated to lower the element onto the foundation.

***Sand Flow:***
A method of sand bedding whereby the sand-water mix is transported through a pipe system with fixed outlets in the soffit of the element. The mix is usually discharged through one outlet at a time. As the velocity of the mix decreases after leaving the outlet, sand is deposited by gravity to form a firm pancake-shaped mound almost touching the underside of the tunnel, with a small depression beneath the outlet. While pancake dimensions vary, an area of 100 square metres would not be unusual. The sand-water mix may be supplied either externally though inlets in the roof or walls, or from inside through non-return valves.

***Sand Jetting:***
A method of sand bedding whereby the sand-water mix is transported through a jet pipe which can be moved anywhere in the void between the underside of the tunnel and the trench bottom. As the velocity of the mix decreases after leaving the jet, sand is deposited by gravity. The resulting density of the sand is less than by the sand flow method. The work can only be done from the outside.

***Screeded Foundation:***
Following trench excavation and before immersing an element, a gravel foundation is prepared by screeding to close tolerances and onto which elements are placed directly without further adjustment. Temporary supports at bed level are not required.

***Segment:***
A monolithic section of a tunnel element only separated from other segments by vertical joints. For concrete tunnels, a segment is typically the length of a single concrete placing operation. Some tunnel elements, particularly in the Netherlands, consist of a number of discrete segments held rigidly together during installation by temporary prestress and joined by expansion joints.

***Shear Dowel:***
A device to transfer shear across a joint. Shear dowels are sometimes used in concrete tunnel elements across immersion, closure or expansion joints to provide continuity of alignment. Such shear dowels must permit relative longitudinal movement. For immersion and closure joints, the dowels would be embedded in cast-in-place concrete at the joint face after immersion.

***Shear Key:***
A device to transfer shear across a joint, usually a moveable immersion joint. In concrete tunnels, the shear key components may form integral parts of the structure of each element. The keys are usually placed in the space between the end bulkheads adjacent to the immersion gaskets so that they can be inspected and repaired if necessary.

***Sill:***
Spelling in United States for ***Cill.***

***Snorkel:***
See ***Access Shaft.***

***Squat:***
The additional draft of a floating body moving relative to the water in which it floats, as compared with the draft when stationary. It is caused by a reduction in water pressure below the body because of directional changes in flow around the body. When keel clearances are marginal, squat may cause elements under tow to touch the bottom. Similarly, an element below a passing vessel may experience uplift due to squat or propeller wash. This may need to be taken into account in selecting safety factors against uplift during installation.

***Steel Shell Tunnel:***
(1) *Single Shell:* Term applied to a tunnel consisting of elements where an outer structural steel membrane (the shell) is constructed first, very much in the manner of a ship. The steel plate also acts as a waterproofing membrane. Elements are usually designed to leave a green field site before the structural concrete is placed, though this may not be the case when other types of fabrication facility are used. Depending upon floating stability requirements, keel concrete may or may not be placed prior to launching. In this condition, draft is usually less than 3 m, making long tows relatively easy while afloat. Nevertheless, transport on barges is not uncommon. The shell plate acts as the exterior form plate for the structural reinforced concrete with which it is designed to act compositely. While stability and strength requirements may require some of the structural concrete to be placed before transportation, it is usual for this concrete to be completed during outfitting, close to the final location. Ballast may be located inside, but more usually outside on top.

(2) *Double Shell:* An outer steel plate, usually octagonal in shape, is added to a single shell tunnel element to act as an external form plate for tremie concrete placed as permanent ballast. The tremie concrete protects the inner shell plate from corrosion, while the outer form plate is left as sacrificial. Behaviour of the inner shell plate, and the compositely acting reinforced concrete within it, is similar to a single shell tunnel element except that the stiffening elements are usually placed outside the inner shell plate.

***Submersion:***
The part of the installation activity that takes an element from being afloat to sitting on the bed.

***Submerged Floating Tunnel (SFT):***
A tunnel through water that is not in direct contact with the bed. It may be either positively or negatively buoyant, and may be suspended from the surface, or supported from or tied down to the bed.

***Sump:***
Sumps (reservoirs) are provided at the portals and at low points (nadirs) to contain quantities of run-off and leakage water compatible with storage requirements of the pumps provided. Oil-water separators are usually required, and sumps within a tunnel most often discharge to portal sumps.

***Suspended Slab:***
Slab provided to span across a cell above a space, such as ceiling and roadway slabs when ventilation ducts above or below the roadway are used.

***Terminal Structure:***
The non-immersed structure abutting the first and last immersed tunnel elements.

***Towing or Transportation:***
Several phases of construction may involve towing. The first tow is only a short distance from the fabrication facility to the location where outfitting for the main tow is to be carried out, if needed. The main tow is to the location where outfitting for immersion is to be done, usually close to the immersion point. The final tow is to the immersion point.

***Trench:***
The space left below bed level after excavation for the immersed tunnel and for its foundation is complete.

***Tube:***
Roadway, track and service cells are each often referred to as tubes. Also used in the jargon expression immersed tube tunnel, meaning immersed tunnel (both circular and box shaped), perhaps originally intended to imply an immersed tunnel with a circular cross-section.

***Unit:***
Term sometimes used to refer to an element.

***Ventilation:***
(1) *Longitudinal:* A system in which fresh air is supplied at one end of the traffic tunnel and the polluted air is expelled at the other.

(2) *Semi-transverse:* A system in which a separate ventilation duct is used for the supply of fresh air through many supply vents along the tunnel. The polluted air is discharged through the end of the traffic tunnel. Also used to describe a system where fresh air is supplied from the end of the tunnel and polluted air is drawn out over the length of the tunnel by exhaust fans.

(3) *Transverse:* A system in which separate supply and exhaust duct systems are used, so that fresh air is distributed and polluted air is collected over the length of the tunnel by supply and exhaust fans.

***Waterproofing Membrane:***
A skin provided external to the tunnel to improve the watertightness of concrete. The membrane may be of steel or other more flexible materials. It may either be sprayed on, or applied to the exterior surface, or the concrete may be placed onto or against it. Most types of flexible membrane require protection against damage by backfill.

***Waterstop:***
Special components embedded in concrete construction joints to reduce the permeability of the joint. Waterstops may be flexible for expansion joints.

***Watertightness:***
A measure of the capability of a tunnel to resist the penetration of water (leakage).

***Warp:***
Cables used to move elements. Warping is the act of moving elements using warps, usually out of a narrow channel or dock.

PII:S0886-7798(97)00007-2

# Chapter 3

# STRUCTURAL DESIGN OF IMMERSED TUNNELS

by

JAN SAVEUR — The Netherlands
Volker Stevin Construction Europe bv

WALTER GRANTZ — U.S.A.
Chesapeake Bay Bridge and Tunnel District

*Contributions and comments for the 1997 edition by:*

Christian Ingerslev — U.S.A.

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.93–109, 1997

Printed in Great Britain
0886–7798/97 $17.00+0.00

Pergamon

# Chapter 3: Structural Design of Immersed Tunnels


**1. INTRODUCTION**

**2. OVERVIEW OF IMMERSED TUNNEL PRACTICES**
2.1 Immersion Installation
2.2 Fabrication
2.3 Joints
2.4 Watertightness
2.5 Cross-section Types

**3. WATERTIGHTNESS**
3.1 General
3.2 Steel Shell Tunnels
3.3 Concrete Tunnels
3.4 The Expansion Joint Concept for Concrete Tunnels
3.5 The Waterproofing Membrane Concept
3.6 Protection of Concrete Against Chemical Attack
3.7 Longitudinal Prestressing

**4. DESIGN OF TYPICAL TUNNEL SECTION**
4.1 Interior Geometry
4.2 Typical Composite Steel Shell Cross-Section
4.3 Typical Concrete Box Structure
4.4 Weight and Balance

**5. LONGITUDINAL ARTICULATION AND JOINTS**
5.1 Steel Shell Tunnels
5.2 Concrete Tunnels
5.3 Shear Transfer in Intermediate Joints
5.4 Intermediate Flexible Rubber Joint Design
5.5 Expansion Joint Design
5.6 Final Joint of Concrete Tunnels

**6. STRUCTURAL ANALYSIS FOR CONCRETE TUNNELS**
6.1 Transverse Analysis
6.2 Longitudinal Analysis

**7. STRUCTURAL ANALYSIS FOR COMPOSITE STEEL SHELL TUNNELS**
7.1 Stage 1: Fabrication and Launching
7.2 Stage 2: Internal Outfitting with Concrete
7.3 Stage 3: Final Condition after Backfilling in Place

**8. LOADINGS**
8.1 Loading Combinations and Allowable Stress Increments
8.2 Accidental Loads

**9. TYPICAL MATERIAL SPECIFICATIONS**

The purpose of this report is to develop an understanding of the state-of-the-art structural design for immersed road and railway tunnels. The report covers common and specific aspects of steel and concrete tunnels. It is the first time that engineers from Europe, the U.S.A., and Japan have collaborated on a report of this kind for immersed tunnels.

This report is very timely in that, in recent years, a long-standing tradition has begun to fade. No longer are steel shell tunnels the *only* types of tunnels constructed in the United States, and concrete box tunnels the *only* type of tunnels used in Europe. A steel shell tunnel was a serious contender in the competition for the Great Belt Tunnel in Denmark, while the concrete box will be used for the Fort Point Channel Tunnel in Boston (Massachusetts, U.S.A.), currently under design.

The history of immersed tunnel practice began in 1910, with the construction of a two-track railroad tunnel across the Detroit River between the United States and Canada. For the next thirty years or so, virtually all immersed tunnels were constructed in the United States. During this time a rather specific steel shell technology emerged—a technology that has continued largely unchanged up to the present. In 1941, construction of the Maas Tunnel in Rotterdam (the Netherlands) began, marking the beginning of the use of immersed tunnels in Europe.

Shallow river crossings with multiple-lane requirements led naturally to the concrete box scheme, a very different method of design and construction from the steel shell. A considerable number of tunnels were constructed in Europe with this method; and, as a result, engineers in the Netherlands, Germany, Sweden, France, Belgium, and Denmark adopted the concrete box method exclusively. This tradition persists today. Meanwhile, Japan was building tunnels of both types. The authors believe that Europe and the United States, as well as other countries, would do well to follow Japan's example.

It is hoped that this ITA Report will encourage engineers to make equal use of both of these basic technologies. Each has unique advantages and limitations, depending on the site. The selection of method should be made by engineers on the basis of job site conditions—not because of a lack of understanding of either method.

## 1. Introduction

Even though immersed tunnels are designed and constructed worldwide, special codes for immersed tunnels do not exist. Standard codes for highway structures are often used, although these codes relate to structures designed for a different structural performance and generally more severe environmental exposure than immersed tunnels. The layout and design of an immersed tunnel is very much related to construction opportunities and site conditions. The state-of-the-art described in this report covers different practices that serve the same goal: namely, to produce watertight and durable immersed tunnels.

Important issues for concrete tunnels are the differences in practices for providing watertightness and durability against aggressive groundwater. These practices cannot be evaluated properly without an understanding of the influence of the longitudinal performance of the tunnel structure. These topics are discussed in some depth in this chapter, using numerical examples.

Both steel and concrete tunnels require that the towing, immersing, and final conditions be analysed. For the structural design of steel shell tunnels, two additional construction stages require separate and different design analyses. These are described and the assumptions are detailed for each case.

## 2. Tunnel Practices

### *2.1 Immersed Tunnel—Definition*

An immersed tunnel consists of one or more prefabricated tunnel elements that are floated to the site, installed one by one, and connected to one another under water. An immersed tunnel is generally installed in a trench that has been dredged previously in the bottom of a waterway between terminal structures constructed in the dry.

The space between the trench bottom and the soffit of the tunnel can be a previously prepared gravel bed; or it can be sand bedding, either pumped or jetted underneath the tunnel. Piled foundations are sometimes used, where soil conditions require them. As construction proceeds, the tunnel is backfilled. The completed tunnel is usually covered with a protective layer over the roof.

### *2.2 Fabrication*

Most concrete tunnel elements are prefabricated inside drydocks or specially constructed casting basins. Sometimes the cofferdam for the approach ramp structure is first used as a basin for the fabrication of the tunnel elements. Occasionally elements have been fabricated on semi-submersibles or launched using a marine lift.

Steel shell tunnel elements are usually fabricated in a shipyard. After the element is launched, most of the interior concrete is installed while the element is floating. The element is then placed in the trench. Steel shell tunnels have also been fabricated and partly concreted in drydocks.

### *2.3 Joints*

All joints are gasketted and tightly closed. The immersion joints between the tunnel elements can be permanently flexible rubber compression gaskets, as is often the case for concrete tunnels. These gaskets are pre-installed at one end of each tunnel element.

Intermediate joints can also be made rigid. This is done at the inside of the temporary immersion seal by welding lap plates to the shells of steel tunnels or by placing concrete for concrete tunnels. The final joint must always be made *in situ*.

### *2.4 Watertightness*

Immersed tunnels have few *in-situ* joints. With regard to watertightness, this is quite an advantage over most bored tunnels. Immersed tunnels are designed to be watertight. Standards for acceptable leakage rates that are state-of-the-art for bored tunnels have no meaning for immersed tunnels.

Steel shell tunnels are watertight by virtue of the quality of the many welds of the shell made in the fabrication yard, by virtue of the quality of the *in-situ* joints, and on the quality of the flexible joints (if they are used). The watertightness of concrete tunnels depends on the quality of the joints, on the absence of full-depth cracks in the concrete, and on the quality of the waterproofing (if it is used).

Many concrete tunnels are provided with watertight enveloping membranes. In addition to providing watertightness, these membranes are sometimes needed to shield the structural concrete against aggressive chemical agents. There are distinctly different views among design engineers about the necessity of such membranes.

### *2.5 Tunnel Cross-section*

In most cases, the selection of the typical cross-section is determined by preferences based on successful previous experience in the specific region or country, as well as local site constraints (as witnessed by the practice in the U.S.A. of selecting steel shell tunnels; in northwest Europe, of selecting concrete tunnels; and in Japan, where both concepts are applied). Recent international immersed tenders in Europe have included alternative options for steel tunnels.

The structure of a *steel shell tunnel* consists of relatively thin-walled composite steel and concrete rings. The steel shell provides the water barrier. The ballast concrete is placed outside the shell in pockets formed between the structural diaphragms. *Concrete tunnels* are monolithic structures in which most of the final weight is incorporated in the structural components.

There is a wide range of cross-sectional configurations, depending on the intended use of the tunnels. In determining the ultimate shape and size of the tunnel cross-section, the designers must consider, for example, whether the tunnel is to be used for railway or motor traffic; how many tracks or lanes altogether are required; whether it will be a single tube, double tube, or multiple tube; what the ventilation requirements will be; and what construction practices will be applied.

#### 2.5.1 Steel (shell) tunnels

For steel shell tunnels, a circular-shaped section for a single tube or a binocular shape for a double-tube cross-section are most economical for external pressure loading, as most sections of the structural ring or rings are in compression at all times. An additional benefit lies in the fact that the space between the roadway slab and the invert and the space above the suspended ceiling, if applied, can be used for air supply and exhaust for transverse ventilation. These spaces can also be used for services.

For larger vehicular tunnels, the usual configuration involves one or two tubes, each having two roadway lanes. The structure consists of a circular steel shell stiffened with steel diaphragms. A reinforced concrete ring installed inside the shell is tied to the shell and acts composite with the shell and the diaphragms. This is the main structure, designed to resist applied hydrostatic and soil loadings. Welded to the exterior flange plates of the diaphragms is a second "shell", the form plate, which acts as a container for the ballast concrete, partly placed as tremie. This ballast weight provides the required negative buoyancy. This type of tunnel is known as a "double-steel-shell tunnel".

The single-steel-shell concept is used for tunnels with one or two relatively narrow tubes, such as tunnels for metro rail transportation. The steel shell is on the outside and acts compositely with the internal ring concrete. The ballast concrete, which is proportionately less than for a larger double-steel-shell highway tunnel, is placed on top of the element to keep the shell as small in section as possible. For single tubes, a circular shape is preferable; sharp corners are avoided. Single-steel-shell tunnels have little spare allowance for internal ducting.

Examples of steel shell tunnel cross-sections are shown in Figure 3-1.

#### 2.5.2 Concrete Tunnels

For concrete tunnels, circular shapes have also been used for single tubes (in combination with transverse ventilation) and for relatively narrow service tunnels. For railway tunnels with two single-track tubes, the near binocular shape is often used because of the obvious advantage for transverse load transfer.

However, the shape most often used for double- and multiple-tube concrete traffic tunnels is the rectangular box, which may have to be widened with extra cells for ventilation air supply and services. The box shape best approaches the rectangular internal clearance required for motor traffic, with good conformity between resistance and weight. The box shape also permits practical concrete construction practice. When longitudinal ventilation would be sufficient, all of the services can be kept within the traffic tubes—that is, along the roof and inside the ballast concrete underneath the roadway or walkway. Often, however, a special services gallery is preferred or may be required by the fire department for emergency escape.

Examples of cross-sections of concrete immersed tunnels are shown in Figure 3-2.

#### 2.5.3 Miscellaneous

Low-point drainage sumps have to be provided within the confines of the structure. In binocular double-steel-shell tunnels, the sump can be placed between the tubes. In single-steel-shell and concrete tunnels, the sumps have to be placed underneath the roadway. The presence of service galleries is helpful in positioning the pumps.

Generally, for given cross-sectional requirements for vehicular space and ventilation, the concrete box section can be made shallower than the steel section. However, for the same conditions, the steel section is generally a narrower section. For a given internal air volume, the final weight will be the same in both cases

## 3. Watertightness

### *3.1 General*

Although watertightness is one of the primary objectives of any immersed tunnel design, the design would be deficient if the consequences of incidental small leakage were ignored. On the large perimeter surface of a tunnel element, the possibility of an undetected pinhole in a steel weld or an undetected construction imperfection of the concrete or waterproofing membrane cannot be ruled out completely.

Suitable repair methods exist and must be specified in the design. The buildup of water pressure within the tunnel wall system must be avoided by provision of proper drainage into the tunnel drainage system. Seepages of this kind are very small and do not require extra drainage and sump capacity.

### *3.2 Steel Shell Tunnels*

For steel shell tunnel elements, watertightness is provided purely by the steel shell itself. The watertightness relies on the quality of the large number of welds. The concrete inside the shell is transversely under compression.

### *3.3 Concrete Tunnels*

For concrete tunnels, leakage is related to the quality of the waterproofing membrane, if used, and to the development of cracks. Therefore, an understanding of the different structural behaviours in the transverse and longitudinal directions is important.

In the transverse direction, box-shaped reinforced concrete tunnels always have zones in the roof and base slabs that experience bending tension, notwithstanding the transverse compression. The tunnel section is designed so that the resulting cracks can only partially penetrate, leaving the concrete in the compression zone sufficiently thick to avoid leakage through cracks.

In the longitudinal direction, the stresses are of a much lower magnitude than in the transverse direction. The basic stress is marginal compression. Secondary effects that can cause partial tension should not lead to full depth cracks. Thermal shrinkage cracks are a typical secondary effect.

In thick concrete members, the heat of hydration causes substantial heating of the member. After some time the member will cool off to the ambient temperature. The resulting contraction of the now-hardened concrete may be subjected to restraint. For example, this occurs when casting walls onto a base slab that was cast at an earlier stage. The result of the cooling contraction of the wall connected to the rigid base slab is compression in the base slab and longitudinal tensile strain in the bottom part of the walls.

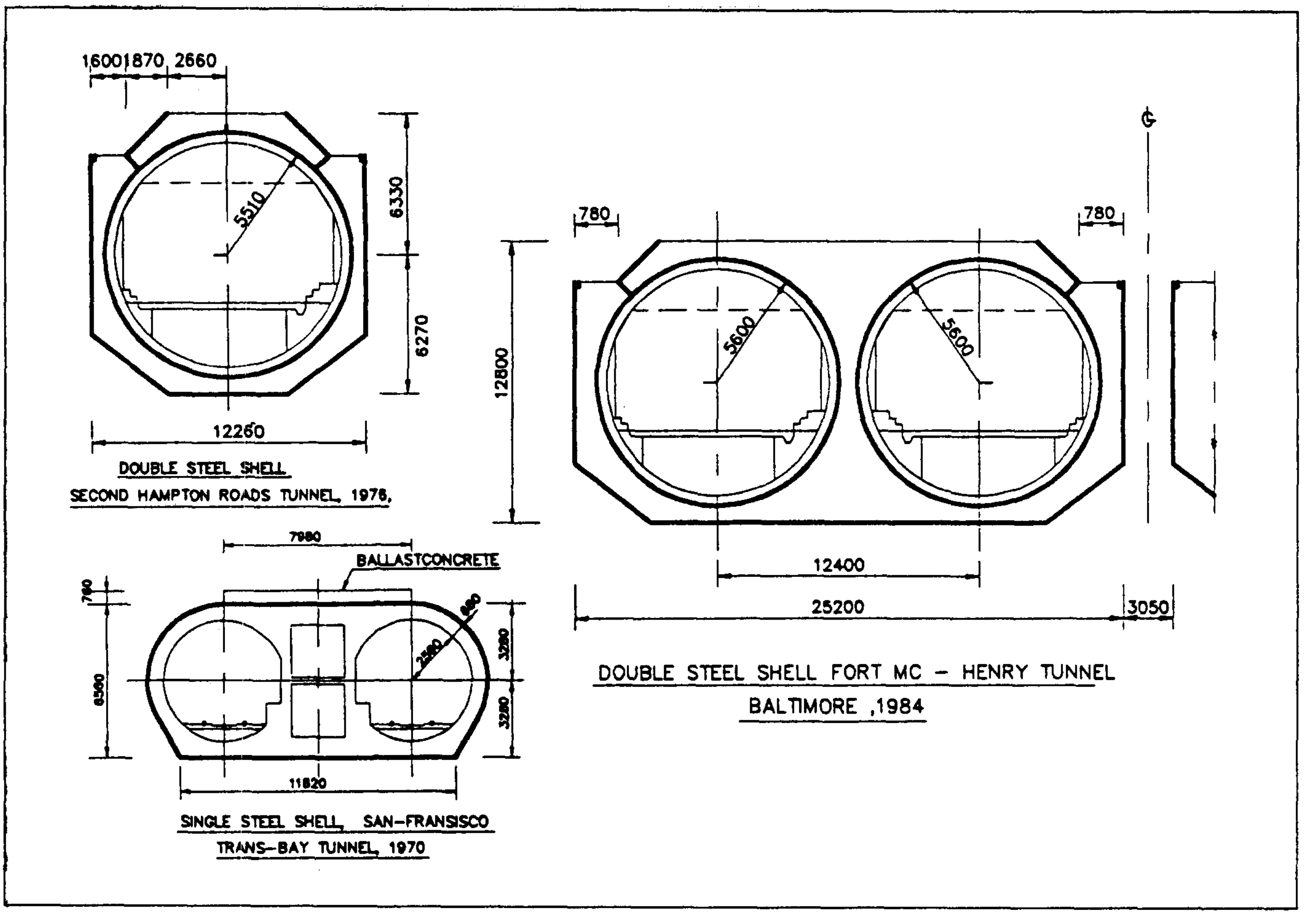



*Figure 3-1. Examples of cross-sections of steel-shell immersed tunnels.*

Unless proper measures are taken, vertical full-depth cracks can occur at about 5-m intervals. Satisfactory processes have been developed to avoid these construction cracks, namely by reducing heat of hydration by using concrete with relatively low cement content and forced cooling in the lower part of the walls (see Fig. 3-3). Sometimes this is done in combination with insulation and heating of the base slabs. If cracks nevertheless are found, remedial grouting appears to be effective.

Effective control of differential heat development largely depends on the heat of hydration of the concrete. This process, in turn, is a direct function of the amount of cement in the mix. Therefore, the use of typical concrete mixes with high cement factors (taken from highway structures codes) can be counterproductive in this respect.

There are two basic concepts regarding control of leakage for concrete tunnels:

1. The *expansion joint concept* involves avoiding longitudinal stresses that can cause cracks, thereby relying on the watertightness of the uncracked concrete.

2. The *waterproofing membrane concept* involves enveloping the concrete tunnel element in a waterproofing membrane.

Each of these concepts is discussed in more detail below.

### 3.4 Expansion Joint Concept for Concrete Tunnel Waterproofing

The expansion joint concept involves achieving watertightness by avoiding transverse cracking of the concrete. A tunnel element 100 m long or more is designed with expansion joints. The length between the expansion joints is determined by the practical length of a complete concrete pour, which is in the range of 20 m.

The vertical joint between two segments is basically an unreinforced cold joint provided with a cast-in flexible waterstop. In this way, the tunnel element can be subjected to flexural deformations without developing longitudinal tensile strain between the expansion joints, which can cause cracking of the concrete. It is common practice to provide an external sealant or waterproofing membrane across the immediate vicinity of the joint (see Fig. 3-9).

Great care must be taken to assure that shrinkage of thermal cracking does not occur in the segments and that any cracks which do occur are sealed prior to floating the element out of the casting basin.

During transportation and installation, the segments of such a tunnel element must be structurally coupled in the longitudinal direction. This can be done by temporary coupling rods, with or without prestress; or by temporary longitudinal prestressing tendons over the full length of the tunnel element. The temporary prestressing tendons are usually cut after installation. However, they are sometimes maintained permanently—for example, when flexural bending of the tunnel is not expected, as may be the case with a piled foundation.

### 3.5 Waterproofing Membrane Concept

For a number of reasons, such as seismic loading or control of displacements, it may be desirable to make tunnel elements monolithic in, say, 100-m lengths. Full-depth transverse concrete cracks can develop, especially in the vertical construction joints, unless action is taken to reduce that possibility. With such a concept, the membrane waterproofing that envelopes the element is a prerequisite and plays an important part. There may be other reasons for using a waterproofing membrane, such as to protect the concrete against aggressive qualities of the surrounding environment,

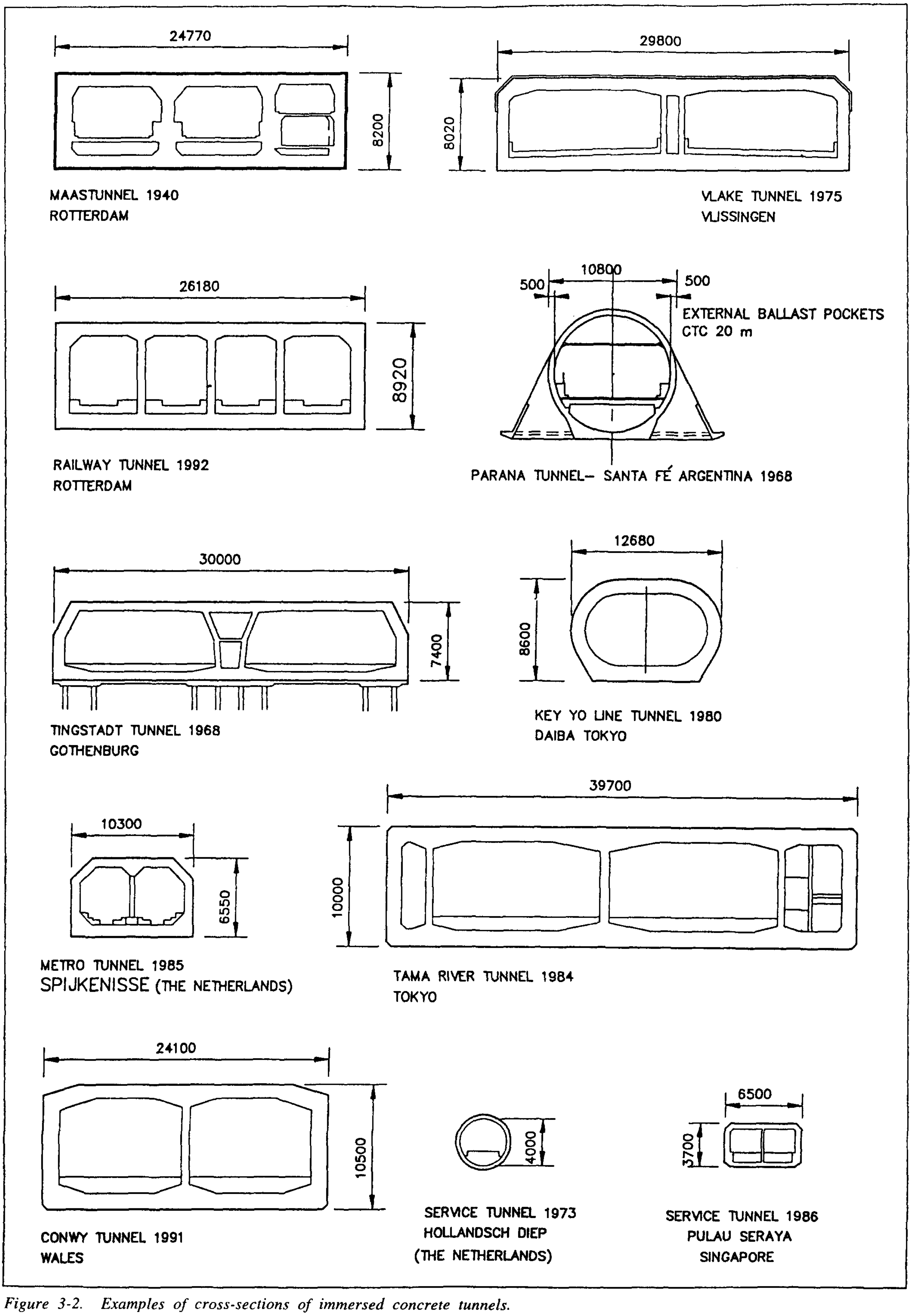



*Figure 3-2. Examples of cross-sections of immersed concrete tunnels.*

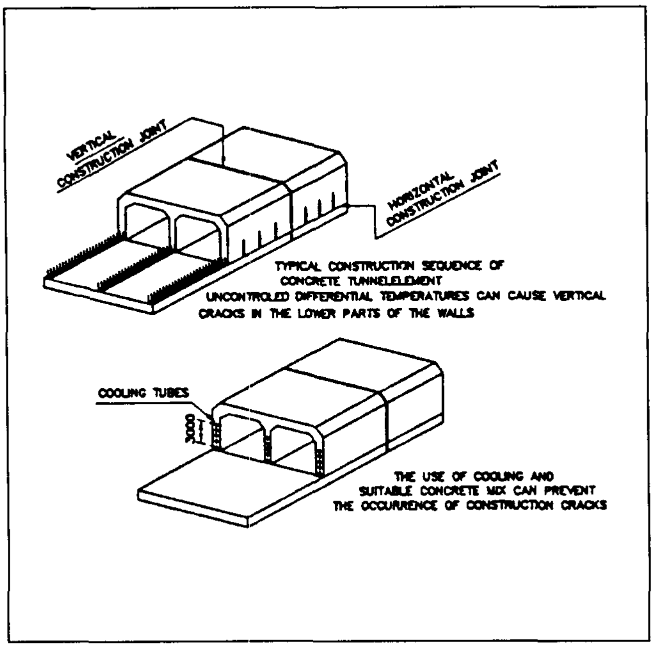



*Figure 3-3. Thermal shrinkage problem and solution.*

or to cover the cracks that may be inevitable with the selected method of construction. When a waterproofing membrane is used, the concrete beneath the membrane will be monolithic, using regular vertical construction joints with continuous longitudinal reinforcement. The waterproofing membrane is made continuous over the full length of the tunnel element.

The following options are used for waterproofing membranes:

- Steel plate below the base, with bituminous or plastic membranes on the walls and roof.
- Steel plate below the base and on the sides, with bituminous or plastic membrane on the roof only.
- Steel plate all around.
- Plastic membranes all around.

All bituminous and most plastic membranes must be protected against mechanical damage. This protection is usually provided by a concrete layer, particularly on the roof. Protection must be permanently held against the waterproofing layer. On the sides, some form of protection board may be used rather than concrete. Unfortunately, concrete protection requires anchors that penetrate the membrane. Other necessary penetrations are for bollard and fender connections, access shafts, and certain immersion equipment.

Steel plate skins applied on the sides for waterproofing are also used as stay-in-place formwork for the walls of the element. For steel membranes across the roof area, holes can be left in the plates for placing and vibrating the concrete. The holes are closed up later. This method is more complicated for a flat roof than for an arched roof. An alternative method is to apply the steel roof plates after concreting the roof. Steel rails are partly embedded in the top of the concrete. Steel plates will be welded to these rails and any voids will be grouted afterwards.

Steel plate membranes (typically 6 mm thick) are anchored into the concrete by welded studs, usually four studs per square meter. Because the thin plates are rather sensitive to temperature deformations, much care is required to keep the plates flat. The bond between the steel plate skin and the concrete should not be relied upon, notwithstanding the use of the studs, because the steel plate membrane does not provide a structural contribution to the concrete structure.

To protect the steel plate membranes against corrosion, impressed current cathodic protection can be installed. Sometimes coating of the steel plate is also specified, but it is practically impossible to provide it without some local deficiencies (e.g., at splices). Nevertheless, the coating, although imperfect, will reduce the direct current demand.

Obviously, a waterproofing membrane must be fully continuous around the whole of a tunnel element. The emphasis for steel plates is on the quality of the welds. For bituminous and plastic membranes, the emphasis must be on providing the proper application; observing the proper limitations for weather conditions; and providing good overlaps over the upstand of the steel membrane.

Sometimes the transition area is also clamped. A small leak in a non-adhering membrane can allow water to flow underneath the membrane to weak spots in the concrete at other locations, thereby making the source of leakage difficult to trace. Such problems may be reduced by using membranes divided into compartments, the edges of which are attached or embedded in the concrete.

A combination of these two methods—i.e., using both waterproofing membranes and the expansion joint method—is not considered cost-effective or advisable.

### 3.6 Protection of Structural Concrete against Chemical Attack

The direct exposure of the structural concrete of immersed tunnels to their external ambient environment is much less severe than the exposure of highway structures to the atmosphere. Observations in the course of durability investigations have attested to the long-term reliability of reinforced concrete structures permanently immersed in sea water, particularly in temperate climates.

In sea water, chloride ions will diffuse into the concrete, thereby depassivating the concrete. Corrosion of the steel reinforcement can begin when the depassivation front has reached the reinforcement. However, the corrosion of the reinforcement at the outside face of the concrete will not be effective, because of the very scarce supply of oxygen. The type of cement used and the density of the concrete are important factors for the penetration rate of chloride ions and oxygen. The chloride diffusion front may never penetrate to the inside face of the concrete.

Sulphate attack in sea water is not accompanied by expansion of the concrete because of the presence of chlorides. Gypsum and sulpho-aluminates, which are soluble in chloride-rich environments, are leached out. Therefore, special sulphate-resisting cement is not needed; and, moreover, would increase the sensitivity of the concrete to chloride penetration.

The inside face of the concrete, especially in road tunnels, is generally exposed to chloride attack in an oxygen-rich environment where the air may be aggressive. Furthermore, in many tunnels, passing vehicles bring in deicing salts, which add to the depassivation of the concrete by carbonation. Similar results can occur as a result of ventilation using salt-laden air. The steel reinforcement also could be subject to corrosion. Again, the best practical protection is good concrete density and sufficient concrete cover. Even though the steel reinforcement is more sensitive to corrosion at the inside face of the concrete than at the outside face, protective membrane waterproofing methods are not used on the interior faces.

### 3.7 Longitudinal Prestressing

To avoid uncontrollable full-depth cracks, the longitudinal working tensile stress should not exceed the tensile strength of the concrete. Normally this requirement can be satisfied in the design of concrete tunnels. If needed, a fairly small rate of longitudinal prestress can substantially increase the allowable range of loading that causes tensile stresses, a procedure that has been adopted for a number of tunnels. For example, for concrete with a tensile strength of 2 MPa and 1 MPa longitudinal prestress, the range of loading causing longitudinal tensile stress could be 50% higher.

## 4. Design of Typical Tunnel Section

### *4.1 Interior Geometry*

The interior geometry of immersed tunnels depends largely on local, state or national highway or railway design standards applicable to the type and volume of traffic for which the tunnel is designed. Interior clearance envelopes for ceiling height and width of lanes or tracks are set by these agencies. Other requirements may involve ventilation equipment; overhead sign clearances; safety and access walkways; placing of services; and use of suspended ceilings, curb details, etc.

Roadway drainage, superelevation, and sight distance for horizontal, and vertical curvature may also play a part in the interior geometrical design. The vertical clearance should be extended to compensate for expected (unequal) settlements and dimensional inaccuracies. The horizontal clearance may have to be extended for horizontal inaccuracies.

### *4.2 Description of Typical Composite Steel Shell Cross-section*

Figure 3-4 shows a typical cross-section of a double-steel-shell tunnel. The main structural element consists of an interior steel shell plate made composite with the reinforced concrete ring within it. The exterior steel, called the "form plate," envelopes the interior shell in an octagonal shape up to the elevation of the crown of the interior shells. The shell and the form plate are interconnected by steel plate diaphragms at 4- to 5-m centres. Exterior concrete fills the space between the shell and the formplate and completely covers the shell plate.

The bottom part and the roof part of the exterior concrete are poured in the dry and are part of the composite structure, along with the concrete ring inside the inner shell. The exterior concrete on the sides, which is poured under water, serves to restrain buckling of the diaphragm flanges and acts as ballast and protection to the steel shell. The interior steel shell is provided on the inside with J-shaped steel hooks that are stud-welded to the shell plate. These hooks tie the ring concrete and its reinforcing steel to the shell to provide the composite action.

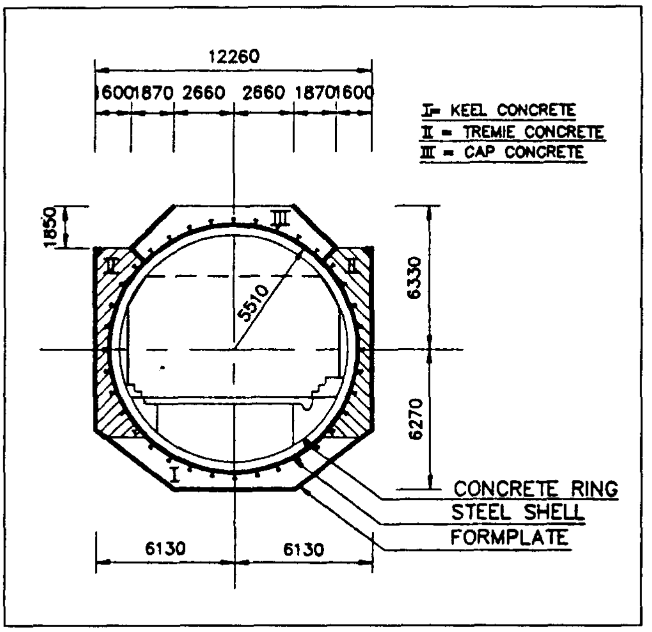



*Figure 3-4. Typical cross-section of a double-steel-shell tunnel.*

The steel assembly is usually built on land, provided with end bulkheads, and then launched sideways or longitudinally. It is usual for the bottom part of the exterior concrete to be in place. In most cases, this "keel" concrete provides the necessary additional weight to ensure stability, and perhaps also sufficient strength, during towing. The ring concrete inside and remaining outside concrete ballast are placed while the assembly is afloat.

The structural loadings during the subsequent construction stages are quite complex. The stresses in the shell are more severe during launching, towing and outfitting with interior concrete than occur when the element is finally immersed. The interior shell is about 8 mm thick and is stiffened with external longitudinal stiffeners. The form plate is usually 6 mm thick.

For double-shell design, the composite steel concrete tunnel is completed by covering the steel shell all around with a thick concrete cover, which provides mechanical and corrosion protection for the steel shell. The double-steel-shell element end detail shown in Figure 3-5 illustrates the typical structural steel arrangement.

The single-steel-shell type of tunnel is simpler in concept, although also subject to critical steel shell stresses in the launching stage, especially when the interior concrete is not yet installed. To stiffen and stabilise the element in the launching stage, the base of the interior concrete ring is installed prior to launching. The steel shell is stiffened with stiffening plates and temporary transverse spiderweb frames. For double-tube single-shell elements, a vertical longitudinal steel truss is constructed in the center between the shells. This truss will later be absorbed in the interior concrete and partly removed where cross-passages are required. Figure 3-1 shows an example of a single steel-shell element.

Cathodic protection is sometimes required for single-steel-shell tunnels if the tunnel passes through a zone of stray currents, such as a subway system or an industrial facility. In such cases, provisions are made to measure the currents at test locations. These tests will determine the required level of impressed current needed to prevent loss of steel section.

### *4.3 Description of Typical Concrete Tunnel*

Concrete tunnels are usually rectangular and can be considered as a monolithic frame comprising base, walls and roof. Most tunnels have a horizontal construction joint between the base and the walls. Waterstops are sometimes provided in these construction joints (they are not used in The Netherlands). It should be noted that the application of a waterstop may obstruct the easy placing and compaction of the concrete, which is not the case on a clean flat surface. Generally, the weight and downward forces in the whole of the wall provide enough pressure to make the joint watertight without special provisions being made.

The walls and slabs for a traffic tunnel are usually at least 1 m thick. To increase the resistance for shear and hogging moments, haunches are provided at junctions between walls and slabs. When designed without haunches, the base slab is up to 1,5 times thicker than the roof, with the top following the cross-fall of the roadway. This arrangement is not only attractive for construction, but also provides mass stability to the tunnel element.

Sometimes the floating condition may require a thinner base with haunches in the corners. Base haunches require special attention for concreting. The ballast concrete is placed on top of the base.

***Allowance for services in concrete tunnels.*** The main electrical feeders are usually placed in conduits in the ballast layer and/or are suspended from the roof. The services inside the tubes can be reduced to local distribu-

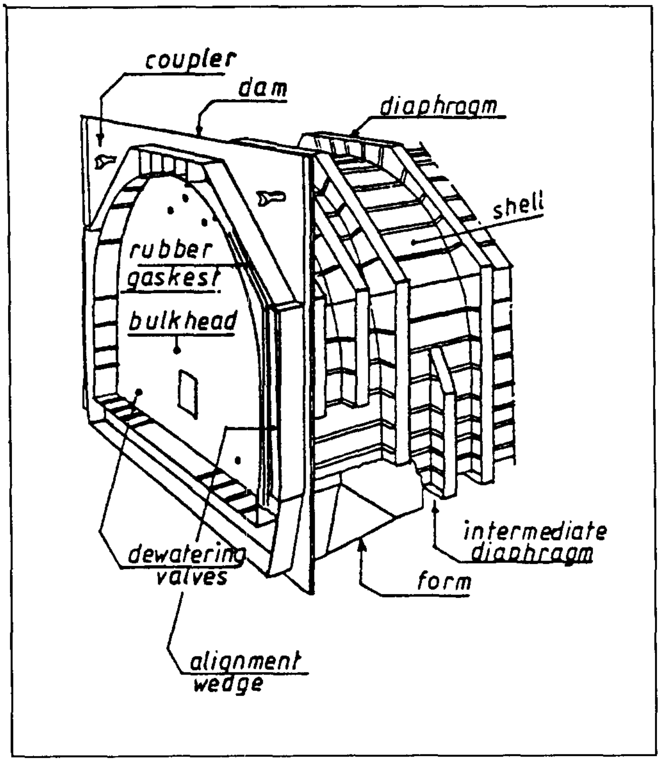



*Figure 3-5. Double-steel-shell element and detail, and typical structural steel arrangement. Formplate is only partially indicated.*

tion if a special services gallery is used. The exterior walls in the latter case would not require large panel recesses, which can act as crack initiators. The same could apply if the size of the low point drainage sump would require deep recessing into the base slab.

### 4.4 Weight Balance

The design of the cross-sectional geometry is very sensitive to variations in the density of the water and the construction materials; dimensional inaccuracies; and the weights of temporary equipment needed for transportation, temporary installation and the permanent condition.

A concrete tunnel element must be able to float with all temporary immersion equipment on board. The freeboard should be minimal to reduce the amount of permanent and temporary ballasting. The temporary on-bottom weight, with the water ballast tanks filled, must be sufficient. For the permanent condition, it must be guaranteed safe against uplift with the fixed ballast in place.

The roof is usually covered with a protective concrete layer that is also used for trimming purposes. When bituminous waterproofing membranes are applied to the walls, protective wood cover or concrete cover is also applied to the walls, either as cast-in-place or precast panels. The roof edges are usually bevelled, to reduce the risk of hooking ship's anchors.

The minimum factor of safety for the permanent condition of immersed tunnels is often specified as 1.10, based on the following conditions:

1. *Uplift forces:*
   - Buoyancy by the water at the maximum expected density and according to the theoretical displacement.
   - Hydraulic lag, if applicable in tidal waterways.
2. *Stabilizing loads:*
   - The theoretical weight of the structural steel, concrete and reinforcement steel, assuming a realistic density for the concrete that will not exceed the actual density.
   - The fixed permanent ballast concrete, inside or outside.
   - The weight of protective membranes and cover concrete.
   - The roadway pavement, suspended roadway slabs, or fixed-track support concrete.
3. *Factors not considered as stabilizing are*::
   - Backfill surcharge and downward friction.
   - The weight of mechanical equipment and suspended ceilings.

These factors are used to determine the required geometry. The actual safety factor may be slightly higher or lower, depending on the actual as-built dimensions. For example, a part of the stable type of roof protection, such as rip rap, may be allowed to be included in the safety factor of 1.10. Sometimes a minimum safety factor of 1.06 is applied for concrete tunnels with only the structural concrete and ballast concrete as stabilising factors. For steel shell tunnels, 1.07 is applied, excluding the contribution of the side walk concrete.

The minimum temporary safety factor during installation is usually 1.03 after release of the immersion equipment.

Sometimes allowance has to be made for uplift caused by hydraulic gradient. A hydraulic gradient can be caused by tidal lag of piezometric height underneath the tunnel, which can occur with silty or clayey backfill in tidal waterways. Another cause of hydraulic gradient is the suction caused by the squatting of ships passing over; however, this factor is only considered in the installation stage.

*For steel shell tunnels*, the total amount of concrete needed for the weight balance amply exceeds that required for strength. The external ballast concrete is the variable factor for the weight balance. *For concrete tunnels*, the thickness of the structural concrete is generally sufficient for the strength. The determination of the final geometry is more complicated because the ballast concrete is on the inside. Variation of the internal ballast volume affects the internal geometry.

## 5. Longitudinal Articulation and Joints

Immersed tunnels are rigid structures in the longitudinal direction. The stresses with which the structure would respond to axial tensile strain (temperature) and longitudinal bending strain (unequal settlement or large surcharge discontinuities) depend on the material properties and the longitudinal articulation.

### 5.1 Steel Shell Tunnels

Steel shell tunnels have approximately the same longitudinal flexural rigidity as concrete tunnels. However, by virtue of the inherent ductility of the steel shell, they have a larger longitudinal strain capacity, and are therefore less sensitive to foundation discontinuities and temperature deformations than concrete tunnels. The concrete part of the composite structure inside the steel shell is not controlled in the design for transverse cracking under longitudinal tensile strain. It will not affect the hoop resistance of the composite ring.

Generally the steel shell is made fully continuous within the length of the immersed tunnel and with special joints at the terminal structures (e.g., as for the BART railway tunnel in San Francisco); or with sleeved joints, as recently developed for Boston's Ted Williams Tunnel.

The continuity joint is often made by a bayonet-type fitting of the abutting ends of the interior shells, which are lapped by plates welded to them on the inside. Temporary watertightness is achieved by tremie concrete between the shell and a cofferdam formed by the "element end bulkheads" and side closure plates. This type of joint is called a "tremie concrete joint" (see Fig. 3-6).

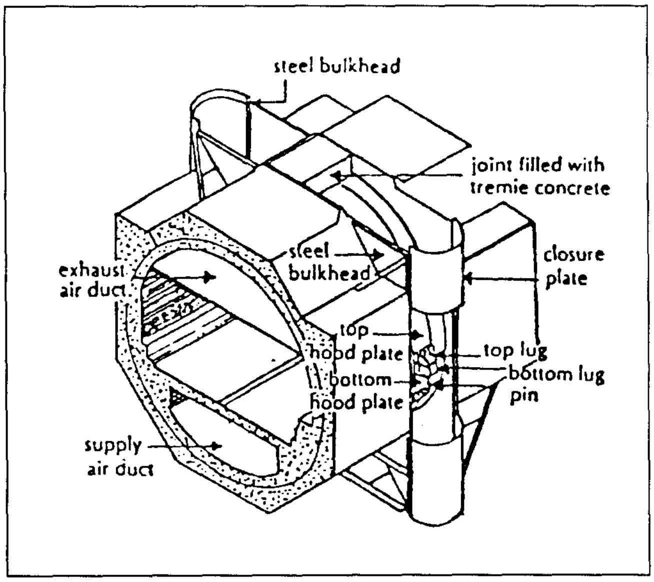



*Figure 3-6. Typical tremie concrete joint for a double-shell steel tunnel.*

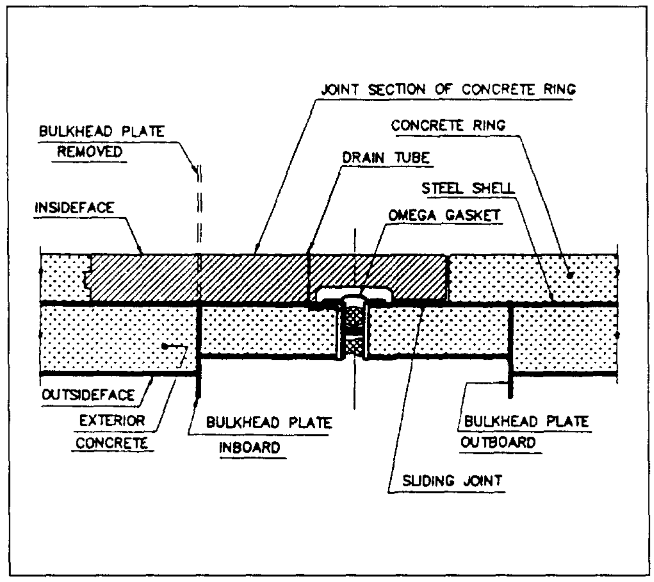



*Figure 3-7. Intermediate joint with sliding arrangement for a double-steel-shell tunnel .*

### 5.2 Concrete Tunnels

Concrete tunnels are usually provided with permanent flexible joints to reduce restraint to temperature contraction and to reduce flexural bending. However, there is an example of a concrete tunnel made monolithic over 400 m, but with flexible terminal joints.

The type of solid rubber gaskets generally used between concrete immersion units normally can provide enough flexibility for this purpose without losing their sealing capacity. These rubber gaskets only transfer compression effectively. To prevent shear deformation in an intermediate joint, which is desirable for the alignment and for sealing performance, shear keys are required. An example of an intermediate joint for a double steel-shell tunnel is shown in Figure 3-7.

### 5.3 Shear Transfer in Intermediate Joints

Transfer of large shear forces in intermediate joints can best be achieved by shear keys in the walls that are made *in situ* in front of the inner face of the permanent watertight gasket. They can also be installed prior to placement with provisions for *in-situ* adjustment. Shear transfer for small shear forces can be accomplished using shear keys or longitudinally movable dowels in the base slab area of the joint.

Settlement discontinuities often occur at the terminal joint. Shear fixity is desirable, but the shear force and resulting bending moment can become very large. To reduce the shear, the following solutions, or a combination thereof, can be applied:

- Ground improvement, in the case of soft upper layers.
- Delaying the shear connection until a part of the settlement has taken place.
- Readjustment of the tunnel elevation with reinjection of bedding sand.
- Preloading by internal flooding.

### 5.4 Intermediate Flexible Rubber Joint Design

A type of solid rubber gasket, sometimes referred to as a "Gina" gasket, is used for practically all concrete tunnels. It is used as a temporary seal at the installation stage and remains as a flexible compression seal for the permanent stage. The facing tunnel element ends are lined with steel plates that are matched as parallel planes within ±5 mm tolerance. The gasket is clamped at its backside.

The specifications for material characteristics and geometry are usually based on the permanent sealing requirement under expected long-term decompression and relaxation of the gasket. Nevertheless, a second flexible rubber water barrier is installed at the dewatered joint by bolting it to the inside faces of the two tunnel elements. The shape of this curved, slab-type rubber seal or gasket, often referred to as the "Omega" seal because of its shape, is sometimes considered to be the main seal. The space between the Gina gasket and the Omega seal is usually drained off to the inside of the tunnel, providing a direct indication of the performance of the outer gasket. An example of an intermediate flexible joint design for concrete tunnels is shown in Figure 3-8. This type of joint can also be used for steel tunnels, when flexible joints are required.

The Gina-type gasket acts as a flexible joint under compression, and can practically be considered as a hinge in longitudinal moment transfer. Shear resistance can be ignored, because it has a tendency to slip along its base under shear deformation. For this reason, shear deformation across a joint is not limited by properties of the Gina gasket, but rather by the allowable shear strain of the Omega gasket, especially with regard to its corner sections.

The exterior rubber gasket should be placed as much as possible to the outside of the structure, in order to keep the recess between the side and the outside of the gasket shallow. This arrangement prevents backfill material from accumulating gradually and obstructing the proper movement of the joint.

### 5.5 Expansion Joints

These joints are used in some concrete tunnel elements at the location of the vertical construction joints. They are basically cold joints without steel reinforcement, provided with steel-rubber flexible waterstops. Special provisions are made to enable pressure grouting access around the waterstop after the main structural concrete is hardened. A typical Dutch expansion joint is shown in Figure 3-9. Sealant is provided at the external face to prevent entry of backfill.

When the tunnel in its permanent condition is subjected to longitudinal curvature, these joints will open at the "tension" side. However, tension cannot be transferred through these joints. This relieves the concrete between the expansion joints of longitudinal tension transfer.

### 5.6 Final Joint of Concrete Tunnels

The final joint cannot be made with the intermediate rubber compression joint that is used in the regular intermediate joints. Wedges or spacers are placed by diver between the opposing tunnel faces to maintain separation during dewatering. They are positioned to lie inside the future temporary watertight enclosure and will maintain the longitudinal compression.

The joint can be made monolithic, or with shear transfer provision to the concrete on one side of the joint and with a flexible waterstop on the other side. In this way, the joint will resemble the action of the expansion joint described in Section 5.5, above. An example of a final joint for concrete tunnels is shown in Figure 3-10.

## 6. Structural Analysis for Concrete Tunnels

### *6.1 Transverse Analysis*

For the transverse analysis, a rectangular concrete tunnel can be considered as a series of plane frames. When the loads and soil reactions are constant in the longitudinal direction, or vary only gradually in that direction, the frames can be analysed with balanced loads.

However, in areas of heavy surcharge, such as embankments, and, especially, near discontinuities of surcharge, as well as in areas of expected redistribution of soil reactions, the external vertical loads acting on the plane frame are not balanced. The shear forces between the adjacent frames need to be analysed for these conditions. An elastic beam analysis can be conducted to determine the longitudinal distribution of the vertical subsoil reaction, and the longitudinal shear distribution. The shear forces can then be entered as vertical loads acting along the vertical members of the frame.

The application of hydrostatic pressure and surcharge loads is straightforward. The magnitude of the lateral soil pressure and possible wall friction caused by the backfill cannot easily be determined. It is best to assume lower and upper bond values to find extreme bending in different members of the frame.

In numerical analysis, it is practical to model an elastic foundation for the base with a given spring constant. For the soft soil case, the effect on transverse moment distribution is practically the same as a uniform ground pressure distribution. In the case of hard subsoil, it is advisable to investigate the sensitivity to the spring constant, because the spring constant cannot precisely be determined and may vary with time.

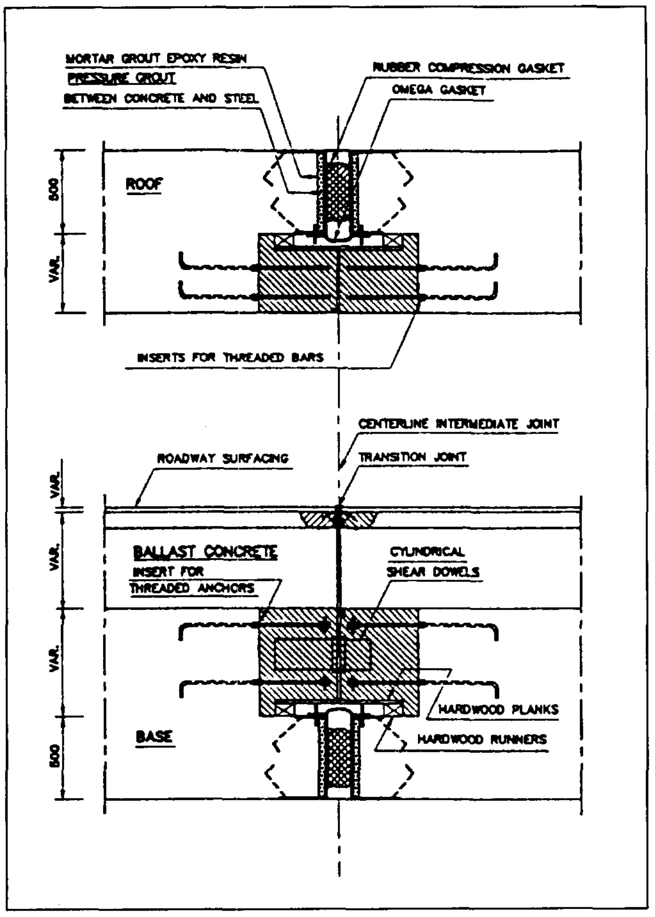



*Figure 3-8. Intermediate flexible joint for concrete tunnels (Dutch solution).*

### 6.1.1 Structural resistance

Under some national codes, only the Ultimate Limit State (U.L.S.) is checked, using the appropriate load and material factors. In U.L.S., the reinforcement is yielding. Other codes also require the control of crack width at the Serviceability Limit State (S.L.S.). At this state, most of the loads are factored by unity, and the reinforcement is not yielding. The maximum allowable crack width varies in the different specifications, often because of differing assumptions inherent in its calculation in each case; an effective width of 0.2 mm is usually specified.

A distinction should be made between longitudinal cracks caused by transverse bending, and transverse cracks caused by longitudinal action. The transverse cracks will cause leakage. An important contribution to the transverse bending moments in S.L.S. are the restraint moments caused by a temperature gradient across the thickness of the slabs and walls. The temperature gradient effect is not factored for the U.L.S. condition. Its contribution to the reinforcement steel stresses should be taken into account.

Special cases of concentrated loads must be examined. Examples include temporary jacking pin supports, the effects of shear dowels, and discontinuities of cross-section that occur at the joints.

### *6.2 Longitudinal Analysis for Concrete Tunnels*

The understanding of the longitudinal performance of concrete tunnels is important in view of the relatively low tensile strength capacity of the concrete and the desire to avoid transverse cracks. It has been shown that these cracks can indeed be avoided.

The effects of hydrostatic compression, temperature stresses and longitudinal bending on the longitudinal concrete stresses are explained below with numerical examples.

### 6.2.1 Longitudinal hydrostatic compression

The longitudinal hydrostatic compression force in any vertical section of the tunnel is equal to the water pressure that would act at the centroid depth, D (avg.), of that section times the gross area of that section. With the simplified assumption that the ratio between the gross area and the

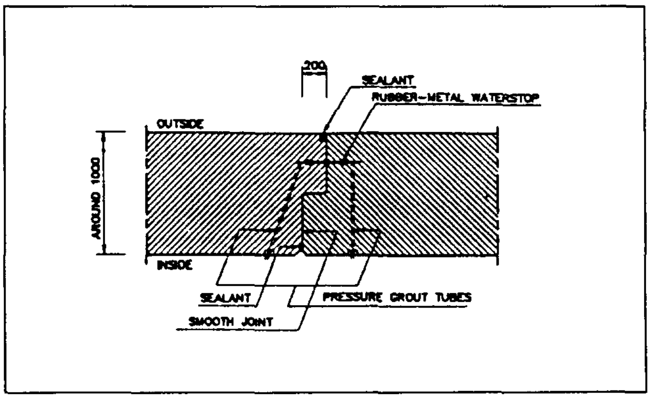



*Figure 3-9. Typical Dutch expansion joint.*

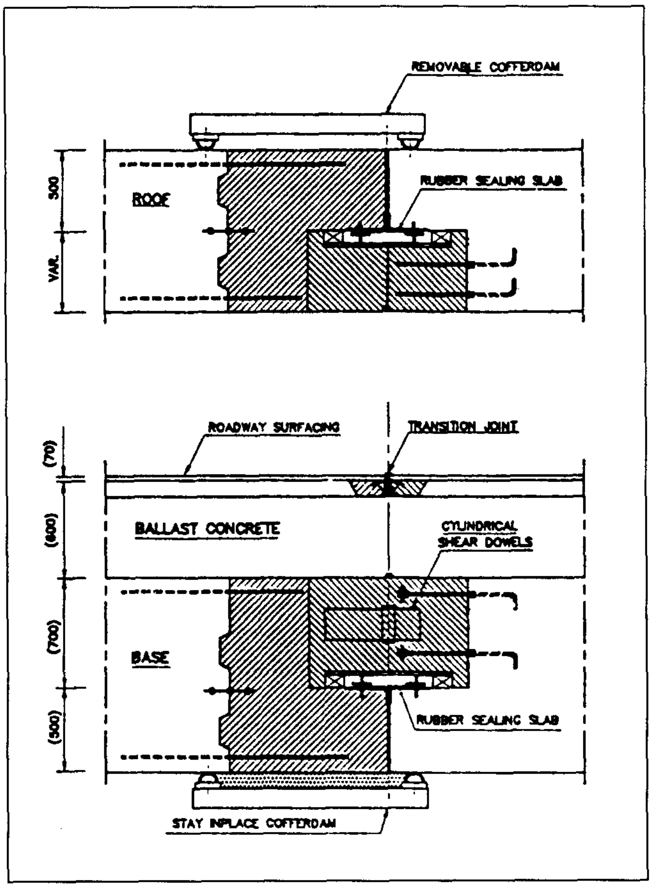



*Figure 3-10. Example of the final joint for concrete tunnels.*

concrete area is equal to the ratio of the densities of concrete and water, a simple expression for the average concrete stress is:

$$f_c \text{ (avg.)} = D_c * 0{,}024 \text{ MPa}, (D_c \text{ in m})$$

The longitudinal hydrostatic compression force will follow the changes in water depth (e.g., by the tide), as long as the immersed part of the tunnel has a free end and friction between the free end and the section under consideration can be ignored.

The hydrostatic compression will be fixed as soon as the tunnel is closed between the terminal structures. The compression is assumed to act at the centroide of the structure when flexible intermediate compression joints are used.

*Losses of the fixed hydrostatic compression.* Losses of the fixed hydrostatic compression are caused by time-dependent relaxation of rubber compression joints and shortening of concrete, causing decompression of the joint.

The degree of relaxation of the rubber compression joints varies with the characteristics of the selected type and the range of compression. Generally the loss will be about 50% of the initial compression. Typical performance rates for flexible rubber compression gaskets used as intermediate joints are shown in Figure 3-11.

Shortening of the concrete can be caused by shrinkage and temperature deformation. Creep of the concrete can be practically ignored. The concrete codes do not provide proper parameters to determine the shrinkage strain for thick-walled concrete members of immersed concrete tunnels. If calculated according to the FIP-CEB model code, a very small strain is found. The shrinkage strain is not restrained when flexible intermediate joints are utilised, but the resulting decompression of these joints will cause a minor loss of the locked-in compression force. In the Netherlands, the shrinkage strain is ignored for tunnels in which the concrete is directly exposed at the outside.

Temperature deformation caused by a temperature decrease of the tunnel by 10°C would cause a decompression of about 10 mm of joints positioned at intervals of 100 m. A numerical example of concrete compression, based on the use of a rubber compression joint, is shown below:

**for $D_c$ = 20 m:**
initially $f_c$ (avg.) = 20 x 0.024 = 0.48 MPa
long-term $f_c$ (avg.) = 0.5 x 0.48 = 0.24 MPa*
**for D (avg.) = 6 m (near shallow end of a tunnel):**
long-term fc (avg.) = 0.5 x 6 x 0.024
= 0.072 Mpa*

* *the factor 0.5 is estimated*

It is noted that the compressive stress in the concrete resulting from the hydrostatic compression, although very small, is sufficient to keep the joints sealed. Because the rubber joints are flexible, the restraint to longitudinal strain of the tunnel elements is practically eliminated.

### 6.2.2 Concrete stresses by temperature deformations

The relevant temperature changes of the concrete are of a seasonal nature. At the time of installation, the whole of the structure can be assumed to be the same temperature as the surrounding water.

The temperature changes of the concrete are usually specified in two parts:

1. An overall increase or decrease, relative to the ambient ground water, with a range from ± 15°C to ± 10°C.
2. A temperature "gradient" over the thickness of a wall or slab (e.g., a linear temperature variation from 0°C on the outside to ± 10°C on the inside: this gradient can then be split into an overall increase or decrease of 5°C, to be added to (1), above, and a linear variation over the thickness from -5°C to +5°C, or vice-versa).

The overall increase or decrease of the temperature of the structure will lead to longitudinal deformation. For tunnels with monolithic intermediate joints between the terminals, this could lead to concrete stresses in the range of the tensile strength or yielding of the vertical joints. Tunnels with intermediate flexible compression joints are not restrained to longitudinal deformation of the concrete, but the flexible joints will react with a minor increase or decrease of compression.

The linear temperature variation over the thickness of a wall or slab of +5°C to -5°C will cause bending stresses varying from -1.0 MPa to +1.0 MPa over the thickness of a concrete wall or slab. Although small, these stresses are a magnitude larger than the hydrostatic precompression. On the outside, there will be tension in the summer and compression in the winter. The stress diagrams of Figure 3-12 clearly show that the temperature variations will not cause transverse cracks.

### 6.2.3 Longitudinal bending

Discontinuity of surcharge and discontinuity of settlement have the same effect. They cause longitudinal bending of the tunnel, which can be analysed as a beam on elastic foundation. The large beam stiffness of the tunnel leads to high shear force and bending moment response to longitudinal bending.

This is illustrated by the following theoretical example for a monolithic tunnel element (not segmented) that wants to settle 50 mm, but has shear restraint at one end (which could be the terminal joint). The joint is flexible for rotation. The main variable is the spring constant of the subgrade reaction. For this example, $K_o$ = 2 MN/m³ is used, corresponding to a deflection of 0.05 m under a pressure of 0.1 MPa (or 10 t/m², as could be caused by heavy surcharge).

Figure 3-13 shows an example of longitudinal analysis for the response of concrete tunnels to unequal settlement.

*Example calculation:*

| | | |
|---|---|---|
| External cross-section: | $h$ | $= 8$ m |
| | $b$ | $= 30$ m |
| Modulus of Inertia: | $I$ | $= 1.200\ m^4$ |
| Section modulus: | $S$ | $= 300\ m^3$ |
| Elastic modulus: | $E$ | $= 2 \cdot 10^4$ MPa |
| Spring constant: | $K$ | $= k_o\, b$ |
| | | $= 2 \cdot 30$ |
| | | $= 60\ MN/m^2$ |
| Characteristic length: | $l_o$ | $= 35.5$ m |
| Undisturbed settlement: | $Y_o$ | $= 0.05$ m |
| Maximum shear force (at fixed end): | $V_{max}$ | $= 2\ EI \cdot Y_o/l_o^3$ |
| | | $= 53$ MN |
| Maximum bending moment (at 3/4 $l_o$ from fixed end): | $M_{max}$ | $= 0.322\ l_o\ V_{max}$ |
| | | $= 612$ MN/m |
| Maximum bending stress: | $f_c$ (max) | $= 2.04$ MPa |

This simple calculation shows that a range of 50 mm of such differential settlement can cause monolithic tunnel elements to crack transversely. These cracks would be full-depth over the thickness of the base slab and the walls.

While it is preferable to prevent forced deformations of such magnitude, sometimes it cannot be avoided. Controlling the width of such cracks by longitudinal reinforcement will not provide ductility of the tunnel element as a whole; furthermore, it is not economical. However, transverse cracks in the concrete can be avoided by providing ductility at the construction joints.

Because the vertical construction joint cannot transfer concrete tensile stresses, it will crack at an early stage. The longitudinal reinforcement through the joint should not yield within the expected bending range, which should be safely below the bending capacity of the uncracked concrete. Beyond that range, the joint should yield, and thereby provide sufficient rotation capacity to avoid an increase in tensile stresses in the uncracked concrete. The resistance of the longitudinal reinforcement will not be mobilised, except in the joints. The concrete will not crack.

If the spring constant in the above example were four times higher, or if it were the same but the undisturbed settlement was limited to 25 mm, the maximum bending moment would only be 300 MN/m and the maximum concrete tensile stress would be 1.0 Mpa. The reinforcement through the construction joint could then be 0.25 % of the concrete area, without yielding of the joint.

Monolithic concrete tunnel elements can be designed with a relatively low amount of longitudinal reinforcement (similar to concrete tunnel elements with expansion joints), without the risk of concrete cracks resulting from the *in-situ* performance. However, special attention should be paid to the watertightness of the construction joints of monolithic tunnel elements, as is also the case for expansion joints. In addition, watertight enveloping membranes may require special attention at the construction joints to keep them watertight in case of yielding of the construction joint.

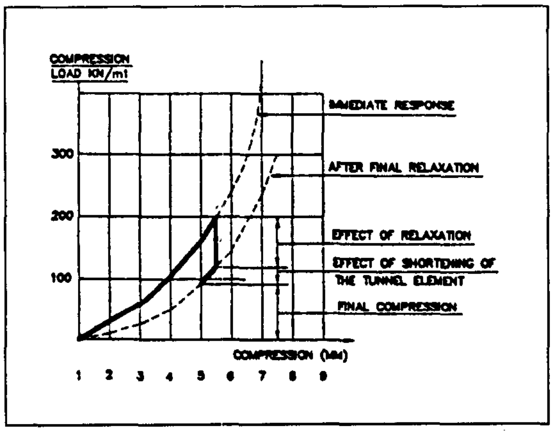



*Figure 3-11. Typical performance of flexible rubber compression gaskets used for intermediate joints.*

### 6.2.4 Temporary construction loads

The effects of temporary construction loads are of a temporary nature only. The tunnel elements are not uniformly loaded during the transportation and immersion stages. The temporary bending moments for a tunnel element 120 m long for the given example are in the range of 50 to 100 MN/m, causing concrete stresses in the range of 0.15 MPa to 0.30 MPa. The longitudinal bending would increase with increased tunnel element length and due to the effects of long waves during sea transportation.

For monolithic tunnel elements, steel reinforcement stresses must be safely in the elastic range. For tunnel elements with expansion joints, temporary longitudinal prestressing is usually applied to meet the condition of no concrete tensile stresses.

### 6.2.5 Longitudinal reinforcement

The longitudinal concrete stresses are very small. The necessary ductility should be provided by the intermediate, expansion, or construction joints. If, instead, the longitudinal reinforcement of the homogenous sections is used for longitudinal ductility, the concrete of these sections could be subjected to full-depth cracks.

The longitudinal reinforcement is designed to act as secondary reinforcement to the main transverse reinforcement or as minimum reinforcement for two-way slabs and walls, in compliance with the applicable concrete codes. This usually results in a longitudinal reinforcement of about 0.2 percent of the total concrete area, for a reinforcement yield stress of 500 N/mm$^2$. This same percentage, applied to the construction joint, would limit the maximum tensile stress in the concrete between these joints to about half the ultimate tensile strength.

### 6.2.6 Permanent longitudinal prestress

Longitudinal prestressing is not an effective way to economise in the design of the cross-section of concrete immersed tunnels, as it is for many other structures. It must be applied uniformly over the depth of a tunnel.

As can be seen from the previous example of longitudinal bending analysis, the prestress levels required would be very moderate. The longitudinal prestress can be used to allow higher longitudinal bending loads, causing tensile stresses. If concrete tensile stresses are not allowed to occur, as is often specified in the case of prestressing application, more prestress would be needed to compensate for the tensile strength of the concrete in the operating condition.

Longitudinal prestress should be applied to increase the tension and bending capacity of the vertical construction joints, in particular, to a level where these joints will not open under certain extreme conditions such as earthquakes. The homogeneous concrete sections are always stronger than the construction joints because of their inherent tensile strength.

A typical example of longitudinal prestressing is Tokyo's Tama River Tunnel, which is located in an active earthquake zone. The tunnel elements were longitudinally prestressed in addition to the longitudinal reinforcement. Special prestressing tendons were installed and stressed *in situ* to act as flexible ties across the intermediate joints with rubber compression gaskets. The Tama River Tunnel is also provided with a watertight enveloping membrane.

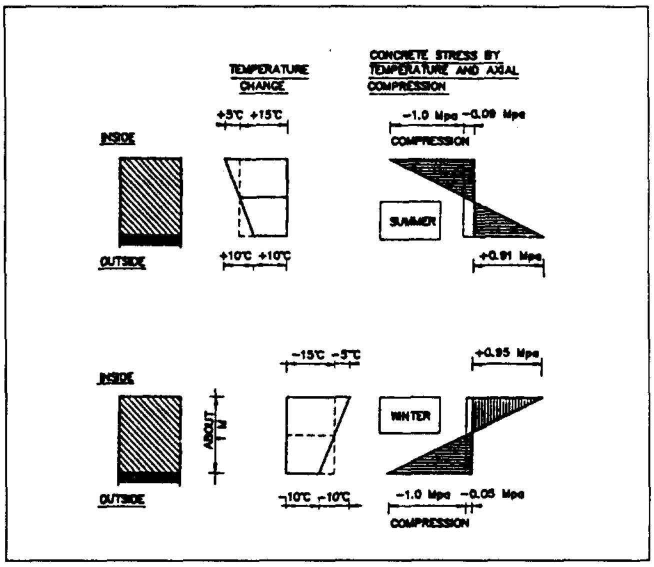



*Figure 3-12. Diagrams of concrete temperature and stresses for summer and winter, valid for tunnels with flexible intermediate joints.*

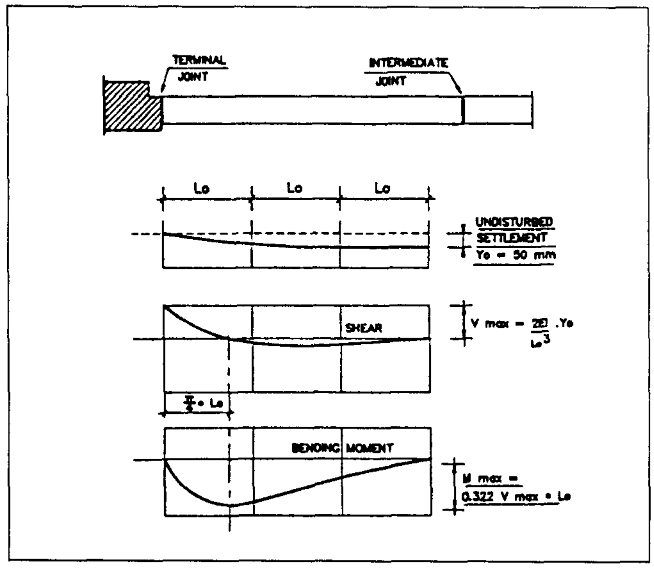



*Figure 3-13. Examples of longitudinal analysis of the response of concrete tunnels to unequal settlement.*

## 7. Structural Analysis for Composite Steel Shell Tunnels

The design of steel shell immersed tunnels differs from that of concrete tunnels because of the manner of their construction. Concrete tunnels are cast in a basin, floated, and then placed and backfilled without much change in the basic structural section. In contrast, the fabrication of steel shell elements involves a structure that undergoes a series of stages, each involving a basically different structure.

The description below applies to double steel-shell structures. The structural concept is simpler for the single-shell type because there is no exterior concrete, other than ballast concrete on the roof. In fact, the single steel-shell tunnel has the same stages as basically different structures. Because this type of shell has no external diaphagms, temporary internal spider frames are utilised. In the case of double-tube shells, temporary longitudinal trusses also are utilised in the middle.

The structural stages are:

*Stage 1*: Fabrication and launching.

*Stage 2*: Internal outfitting with concrete.

*Stage 3*: Final condition after backfilling in place.

Each of these stages is described in detail below.

### 7.1 Stage 1: Fabrication and Launching

During Stage 1, the loads on the structure are largely a function of the method of support and launching, if the element is to be constructed on shipways. It may be launched sideways or end-launched, in either a controlled or an uncontrolled launching.

As an example of a different process, the elements of the (I-664) Monitor-Merrimac Tunnel in Hampton Roads, Virginia (U.S.A.) were constructed in a drydock and floated out, with all of the interior concrete installed prior to installation of the end bulkheads. However, for this tunnel the design had been based on being a launched element because the expectation had been that the elements would be constructed in the conventional way with fitting out being done while afloat. Boston's Ted Williams Tunnel was constructed in the same drydock, although very little of the interior concrete was placed prior to floating out because of the long distance of travel to Boston.

During fabrication, each steel binocular or monocular module (with about eight modules per element) must be supported adequately to avoid distortion or local buckling. In some cases, interior "spiders" may be required to maintain the roundness of the shell plate. Accurate roundness must be maintained to assure a uniform thickness of the interior ring concrete. This is critical to the eventual structural integrity of the elements when it is backfilled.

A portion of the exterior concrete about 1.5 m thick and called the "keel," is usually placed in the bottom of the formplate prior to launch. This provides protection for the bottom of the element during launch, as well as stability during towing. After this concrete is placed, the loads of the element are transferred to launching sleds or, in the case of end-launching, fore and aft poppets prior to launching. The shell and diaphragms may require additional reinforcements in either case. The keel concrete is considered to act as a longitudinal beam acting compositely with the stiffened shell plate and diaphragms. Both static and dynamic water pressures that will occur during launch must also be determined and allowed for in the design.

While the basic design of the elements takes into account some of the expected launching loads, the contractor must be made responsible for checking the details for his proposed method of supporting and launching, and must provide whatever additional reinforcement that may be required.

Towing may impose longitudinal stresses on the steel element. Often the practice has been to tow the element under its own flotation, because the draft of an element that has not been outfitted with interior concrete and the remainder of the exterior concrete is only about 2 to 3 m. If the elements are to be towed in ocean waters, the height of long periodic swells becomes important, because the swells impose significant longitudinal bending moments. Unless they are accounted for, these bending moments could cause buckling of the top shell. More recent practice, when ocean towing is required, has been to tow elements on large offshore construction barges in order to reduce this effect.

Chapter 7 of this edition discusses the impact of tunnel element transportation on immersed tunnel design for both steel and concrete options.

### 7.2 Stage 2: Internal Outfitting with Concrete

During outfitting, the loading conditions on the shell change constantly during concrete placement. At the beginning of outfitting, the element is supported uniformly as it floats with a constant draft. The end bulkheads are quite heavy and impose a hogging moment on the element. As sections of the interior concrete are placed, additional moments are imposed on the shell. The sequence of concrete placement is designed to counteract the hogging effect and reduce its deflection so that at the end, the element is as straight as possible.

As concrete weight is added, the element settles lower into the water, thereby imposing transverse moments on the shell plate and diaphragm. The formplate does not take any of this load, as water is allowed into the pockets between the shell and the form plates. The only exception occurs at the end pockets between the dam plate (extension of the bulkhead) and the first diaphragms. These pockets are filled with structural concrete, placed in the dry, to help resist the large forces that act on the joint during dewatering. Typically, the shell plate is designed to take the water pressure as the outfitting progresses.

The placing sequence is determined by the engineer as a series of reasonable volumes that can be placed while limiting the moments and shears in the exposed shell plate. The concrete placement should be symmetrical about the transverse and longitudinal center-lines of the element. As the head of water increases on the shell plate, the circumferential moments must be resisted by the diaphragms and shells embedded and, therefore, fixed in the keel concrete. This condition must be checked for each stage of the placement sequence; it generally becomes most critical during the placement of the heavy haunch sections adjacent to the bulkheads.

Longitudinal loads are resisted by the shell plate and the radial stiffeners. Stress analysis for critical compressive stress for buckling of curved panels under uniform compression are used to determine moment capacity after applying a suitable safety factor.

The local buckling of the shell is investigated for loading in torsion and transverse shear. Buckling of the arch between stiffeners is investigated. The shell plate and longitudinal stiffeners are considered as a cylindrical shell spanning between diaphragms. The shell plate alone can resist water pressure as a ring structure and a cylindrical beam. Assumptions are made regarding how these loads are distributed between the shell and the stiffeners.

The diaphragm is designed as a ring structure, with its ends fixed in the keel concrete. The effective width of the shell plate that acts with each diaphragm, is determined as a function of the centerline radius of the shell and its thickness.

### *7.3 Stage 3: Final Condition after Backfilling in Place*

This final transverse analysis stage considers that the tunnel is completed and fully backfilled. It is subjected to a series of loading combinations, listed below.

Bending moments and direct stresses are computed on the basis of six superimposed loading conditions:

1. Uniformly distributed loading— top.
2. Uniformly distributed loading— side.
3. Triangular side loading.
4. Loading from exterior upper quadrants.
5. Buoyancy forces.
6. Water surcharge.

Generally, the effect of these loading combinations results in a moment diagram with negative values (tension on the outside) near the center wall on the top and bottom and in the central portion of the exterior walls; and positive values (tension on the inside) in the regions in between. This is somewhat similar to the moment diagram for a two-cell rectangular box loaded top and bottom, as might be expected.

Axial thrusts act in addition to the moments and are caused by overburden and side pressures. These have the effect of reducing the tensile stresses caused by bending, while increasing the compressive stresses.

Before computers became readily available, the tunnel configuration assumed for analysis was simplified to circular or elliptical shapes in order to facilitate otherwise very difficult, or perhaps even impossible, calculations. Thanks to the availability of relatively inexpensive, accurate, and easy-to-use computer software such as STAAD III, STRUDL and others, the structure may be modelled to the exact shape of the proposed tunnel. Because the location of the frame line, the cross-section area, and the moment of inertia are all interrelated and affect the results (moments, shears, axial loads and deformations), an iterative design approach is necessary. Using such an approach, a series of computer runs are made, each using information from the previous run, to modify frame line location and member properties, thereby converging on an accurate and economical solution.

For the preliminary design, the total thickness of tunnel walls may be based on previous experience or very rough hand-calculations. The tunnel is drawn accurately and the frame line is laid out by "eye" or judgment, more or less along the mid-thickness of tunnel walls, discounting exterior tremie concrete.

Coordinates of selected "joints" along the frame line can be very conveniently obtained if the work is done by CADD drafting. Using the results of this first run, the design engineer can proportion steel and concrete depths and thicknesses based on stress analysis, and obtain new section properties. These data are used in turn as input for a second, more refined computer analysis, wherein the frame line is located at the center of gravity of the gross composite section and member properties are based on the same.

This process is repeated until the designer is satisfied that changes resulting from further refinement are negligible. Theoretically, the final computer run should be based on a frame line corresponding to the center of gravity of the cracked section, and member properties, area, and moment of inertia are based on the same.

It should be kept in mind that the composite three-dimensional tunnel structure with variations in loadings is extremely complex. The goal of the analysis should be to determine an envelope of maximum moments, shears and axial loads so that all possibilities of overstress are eliminated. For example, gross section properties may result from maximum effect at one location and minimum effect at another, whereas the cracked section may show the reverse. This approach, inconceivable in the past, is not at all difficult today with the help of clever computer programming.

Future analysis methods may make use of three-dimensional finite element modeling. This will be particularly interesting in the analysis of loadings such as ship collision, anchor dragging, and internal explosion. The longitudinal distribution of these loads is otherwise very difficult to determine.

### *7.4 Field Measurements*

The final assessment of any design as complex as that for immersed elements can only be made on the basis of field measurements. To our knowledge, this has never been done for steel shell tunnels in any sophisticated way.

It was proposed that strain gauges be attached to the diaphragms and interior reinforcing steel of the Ted Williams Tunnel in Boston, Massachusetts (U.S.A.) at various locations, representative of the wide range of imposed loading conditions that exist for this tunnel. If this program goes forward, the results obtained will increase our understanding of the structural action of steel shell tunnels and could lead to better, more economical designs in the future.

In the future, where unique conditions exist or, in general, where value can be gained from similar measurements of the action of immersed tunnel elements, there may be merit in including test programs of this type in the construction bid packages.

*Table 3-1. Indication of allowable stress increments or load factors for loading combintaions.*

| Stress Increments (S) or Load Factor (F) | Type of Structure | | |
|---|---|---|---|
| | **I (S): Steel shell traffic tunnel (U.S.A.)** | **II (S): Reinforced concrete tunnel with waterproofing membrane (Japan)** | **III (F): Reinforced concrete traffic tunnel (Netherlands)** |
| A. BASE LOADING<br>Unfavorable combination of:<br>• Dead load<br>• Backfill<br>• Surcharge and live load<br>• Lateral earth pressure<br>• Water pressure at mean high or low water | 1.00 | 1.00 | 1.5 * |
| B. TOTAL STRESS INCREMENT FOR COMBINATION OF BASE LOADING WITH ANY OF THE FOLLOWING: | | | |
| B1. Extreme high water | 1.25 | | 1.5 *** |
| B2. Anchor dragging or dropping | 1.25 | | |
| B3. Sunken ship load | 1.25 | | |
| B4 Temperature restraints | — | | |
| B5. Unequal settlements | — | 1.00<br>1.30 ** | — |
| B6. Temperature restraints and unequal settlements | — | 1.15 | — |
| B7. Internal explosion | — | | 1.0 |
| B8. Earthquake, unequal settlement | — | 1.50 | — |
| B9. Earthquake, temperature restraints, unequal settlements | — | 1.65 | — |
| B10. Erection condition | — | 1.30 | |

NOTE: A dash indicates that this aspect is known not to be reviewed, or is not critical.
* Refers to Dutch practice: the load factor used for the ultimate limit state is 1.7, reduced for the material factor incorporated.
** The factor 1.30 also includes extremely high water.
*** 1.4 * A + 1.15 * B1.

## 8. Loadings

### *8.1 Loading Combinations and Allowable Stress Increments*

An indication of the types of loads and their combinations used for the design of immersed elements is given in Table 3-1. The table, based on data provided from the U.S.A., Japan, and the Netherlands, is not exhaustive. It should be noted that the factors given in the table are based entirely on the specific project conditions and requirements.

The reference projects in the table relate to tunnels of widely differing structural nature. They are intended to provide an understanding of the range of values for loading conditions that are encountered in immersed tunnel design(s).

The reference projects used in the table are:

I: A steel shell traffic tunnel (Ted Williams Tunnel, Boston, Massachusetts, U.S.A.)

IIa: A longitudinally prestressed reinforced concrete traffic tunnel with waterproofing membrane (Tama River Tunnel, Japan).

IIb: A reinforced concrete railway tunnel with waterproofing membrane (Keyo-Line Daiba Tunnel, Japan).

III: A typical Dutch reinforced concrete traffic tunnel (Tunnel De Noord).

For reference II, only the ultimate-limit state factors are given. However, service-limit state verification is also done in view of watertightness requirements; for example, temperature restraint has to be considered.

### *8.2 Accidental Loads*

While Chapter 6 discusses in detail accidental loads associated with tunnel operations, this chapter deals only with the relevant loads that affect the structural design. Loads to be considered may include:

1. Sunken ship loads.
2. Dropping or dragging ships anchors.
3. Flooding of the tunnel.
4. Internal explosion loads.

These loads are subject to different probability for each project. They are usually specified as deterministic loads, corresponding to an excedance probability of $10^{-4}$ per year. Increases in working stresses are usually allowed.

A discussion of earthquake loading is included in Chapter 6. Immersed tunnels are known to have withstood major earthquakes successfully. Ample experience with earthquake design for steel shell and concrete tunnels exists in the U.S.A. and Japan.

#### 8.2.1 Sunken ship loads

For immersed tunnels in soft ground, the tunnel may respond more rigidly than the adjacent backfill. This can be accounted for in the load to be specified, usually as a uniform load over a minimum area. For the design of an immersed tunnel across the Great Belt in Denmark, which is an international waterway used by very large vessels, the specification was 100 kN/m$^2$ over 250 m$^2$.

#### 8.2.2 Dropping and dragging anchors

Immersed tunnels are generally covered with a protective concrete layer on the roof. For navigation conditions, protective stone cover is also applied.

The energy of an object free falling in water has to be absorbed by the stone cover and partly by the crushing of the concrete cover layer. The structural roof load is related to the impact pattern. This factor can usually be accommodated without additional reinforcement.

The lateral load of a dragging anchor hooking behind the edge of the tunnel roof can be derived from the effective anchor-rope-breaking loads. For large vessels, the load can be in the range of 3,000 kN, acting as low as 4 m below the rooftop, depending on the type of bottom material and anchor. The upper part of the walls may need to be provided with cover concrete to avoid mechanical damage to the structural concrete. Dragging anchor loads of this magnitude normally can be resisted by the available friction of the foundation and passive pressure behind the tunnel.

#### 8.2.3 Flooding of tunnels

The probability of internal flooding is very small during the operational life of a tunnel, although several such incidents have already occurred. It makes sense to investigate this type of incident in the light of possible undesirable settlements. In many cases, it is only one tube that has flooded, and then usually not higher than roof level at the lowest point.

#### 8.2.4 Internal explosion loads

The probability and extent of internal explosion loads depends very much on how the tunnel is to be used. An example is the rectangular Liefkenshoek Belgium, located near an industrial area, which has been designed for an internal explosion pressure of 4 bar. This requirement substantially increased the amount of transverse reinforcement needed.

## 9. Typical Material Specifications

### *9.1 Structural Concrete for Concrete Tunnels*

Although specifications vary considerably worldwide, a distinction can be made between two main groups of specifications. One group of contractors tends to use highway structure codes, which usually specify high-grade concretes, with characteristic strength in the range of 40 Mpa, and with durability requirements associated with sulphate attack. The latter requirements generally lead to the use of sulphate-resisting cement or Portland cement to which pulverised fly ash (p.f.a.) has been added.

The use of silica fumes with p.f.a. and Portland cement concrete is being considered in some countries.

The other group, of which the Netherlands is typically representative, uses lower-grade concrete, with the emphasis on construction crack avoidance, low permeability, and chloride penetration resistance. Watertight membranes are not used.

A typical concrete specification for Dutch immersed tunnels is:

| | |
|---|---|
| *Characteristic strength*: | 22.5 Mpa |
| *Cement types*: | Dutch blast furnace cement (more than 65% slag) |
| *Max. cement content*: | 275 kg/m$^3$ |
| *Max. water/cement ratio:* | 0.5 |
| *Permeability*: | Less than 20 mm in penetration test, according to DIN 1048 |

### *9.2 Materials for Steel Shell Tunnels*

Typical material specifications for structural concrete and structural steel, as presently used in the U.S.A., are:

**Structural concrete**:

| | |
|---|---|
| *Strength*: | 4,000 psi (also for tremie concrete) |
| *Cement*: | Portland Cement: AASHTO M85, Type I or II |
| *Cement content*: | 565–610 lb/yd$^3$ |
| *Water/cement ratio*: | 0.48–0.50, depending on size of aggregate |
| *Slump:* | 2 in.–5 in. |
| *Permeability*: | 2,000 coulombs per 6 hours, where tested per AASHTO T-277 |
| *Fly ash*: | will be substituted for 5% of the cement for all concrete. |

| | |
|---|---|
| **Structural steel**: | ASTM Grade A36 (mild steel with 36,000 psi yp) |
| **Reinforcing steel:** | AASHTO M31 Grade 60 (60,000 psi yp) |

PII:S0886-7798(97)00008-4

# Chapter 4

# WATERPROOFING AND MAINTENANCE

by

WALTER GRANTZ — U.S.A.
Chesapeake Bay Bridge and Tunnel District

LIONG TAN — The Netherlands
Bouwdienst Rijkswaterstaat

EGON SØRENSEN — Denmark
COWIconsult A/S

HANS BURGER — The Netherlands
DHV

*Contributions and comments for the 1997 edition by:*

Ahmet Gursoy — U.S.A.

Christian Ingerslev — U.S.A.

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.111–124, 1997

Printed in Great Britain
0886–7798/97 $17.00+0.00

Pergamon

# Chapter 4: Waterproofing and Maintenance


**1. INTRODUCTION AND BACKGROUND**

**2. STEEL TUNNELS**
2.1 Recent Joint Evolution
2.2 Waterproof Joints for Immersed Tunnel Elements
2.3 Closure Joints
2.4 Terminal Joints
2.5 Underwater Connections to Terminal Elements Constructed in Rock
2.6 Seismic Joints
2.7 Concrete Requirements
2.8 Cathodic Protection of Steel Elements

**3. CONCRETE TUNNELS**
3.1 Water Leakage through the Concrete Structure
3.2 Joints

**4. MAINTENANCE**
4.1 Experience with Leakage in Steel Shell Tunnels
4.2 Leakage in Concrete Tunnels

**5. REFERENCES**

## 1. Introduction and Background

This chapter deals with ensuring watertightness of immersed tunnels; and, specifically, with the methods used to ensure watertight integrity of immersed tunnels. Material in the chapter covers both steel shell tunnels and concrete box tunnels.

For concrete tunnels, the two basic watertightness design philosophies are described. The first makes use of applied exterior steel and/or a waterproofing membrane. The second uses no exterior waterproofing layer, but rather accomplishes waterproofing by dividing the element into separate segments where concrete shrinkage-cracking can be prevented. Because steel tube tunnels are completely enclosed by a steel shell, the issue of waterproofing largely concerns the design of the joint between elements and corrosion of the steel.

Development of methods for providing watertightness in joints in steel tunnels over the past decades has been largely empirical and mainly the result of incorporating the experience gained on one tunnel in the design of the next. This process is ongoing in the United States. In the European countries, however, development has centered on concrete tunnels, constructed with or without membranes.

This chapter provides information on current design practice for types of joints, membranes and watertight concrete. It seems appropriate to begin this section on watertightness with a brief history of its evolution in immersed tunnelling.

The history of subaqueous immersed tunnels has just attained its first centennial. In 1894, the first immersed tunnel large enough for a person to walk through was constructed. The Boston Metropolitan sewer, beneath Shirley Gut in Boston Harbor, used 15-m-long, 2.7-m-diameter elements consisting of brick and concrete, with 10-cm-thick exterior wood sheathing. Temporary wooden bulkheads were installed at both ends of each element and external flanges with rubber gaskets were provided to permit the elements to be bolted together. Thus, the first immersed tunnel used rubber-gasketted joints.

In 1910, the Detroit River Railway Tunnel—considered to be the first full-scale immersed tunnel—used this same method. This tunnel was unusual by present-day standards in that it was of double-shell construction, placed on the bottom, with no concrete inside or out. The exterior concrete was then placed by tremie methods. After all the elements were in place, the tubes were accessed and the interior concrete lining was installed continuously, as a mined tunnel would be lined. The Detroit River Tunnel used rubber double-gasketted joints around each of the two tubes. The joint was made watertight by bolting it tight and then grouting the space between the two gaskets, in a manner not too unlike the Boston sewer built some 16 years previously. The shell was 9.5-mm riveted steel, lapped and ship-caulked.

The next immersed tunnel was only one element long. This was the La Salle Street railroad tunnel in Chicago, constructed in 1912. The ends of the element tied directly into cofferdams. Again, the shell consisted of riveted ship-caulked and lapped construction.

With the third immersed tunnel, the Harlem River Tunnel in New York, constructed in 1914, the design of the joints between tubes took a significant turn. The design no longer used a rubber gasket (perhaps because of the difficulty anticipated in mating the four tubes incorporated in these elements). Instead, it used a riveted steel liner plate closure across a square-butted joint after the exterior had been concreted with a tremie enclosure between elements. The space behind the liner plate was grouted.

This method was quite successful and set the pattern for all of the immersed tunnels constructed subsequently in the U.S., until the Bay Area Rapid Transit (BART) Transbay Tunnel in San Francisco was constructed in 1970. The BART Transbay Tunnel was the first to use the double rubber gasketted joint now commonly used in the U.S. The grouted steel liner plate detail (welded) was used on the BART Tunnel and continues to be used on almost all steel shell tunnels to this day.

The Posey Tunnel, constructed in 1928 in Oakland, California, and the Webster St. Tunnel—its near-twin, constructed in 1962—are the only concrete tunnels without a steel shell ever constructed in the United States, although the Fort Point Channel Tunnel Boston will soon be a third. Both tunnels were waterproofed with an external bituminous membrane protected with wood lagging, and both used tremie concrete joints. The Posey Tunnel was the first concrete immersed tunnel constructed as we know them today, preceded only by the Friedrichshafen pedestrian tunnel, which was built as two pneumatic caissons with the joint made in a cofferdam.

The first immersed tunnel to use welding for its steel lining was the Detroit-Windsor vehicular tunnel, constructed in 1930. Only the longitudinal seams were welded; the circumferential seams were riveted, as in previous tunnels. The first all-welded steel shell was used for the elements of the Bankhead Tunnel, constructed in Mobile, Alabama, in 1940.

By the time the Transbay Tunnel for San Francisco's Bay Area Rapid Transit (BART) system was designed, concrete immersed tunnels had long been constructed in Europe and Canada using single main gaskets and mobilizing the water pressures to compress the gasket. The first concrete tunnel in Europe was the Maas Tunnel of 1941, and a single-element steel-covered concrete tunnel with rigid joints was constructed in 1944 in Japan. The first such gasketted tunnel in North America was the Deas Island tunnel, constructed in 1959 in Vancouver, British Columbia (Canada), which used an inflatable rubber gasket for the initial seal and water pressure to effect the final seal—but it was not until the Rendsburg Tunnel in 1961 that European immersed tunnels really took off.

Thereafter, the Gina and Omega profiles came into prevalent use in European countries, as well as in Japan, Hong Kong, Taiwan and, recently, the People's Republic of China.

## 2. Steel Tunnels

### 2.1 Recent Joint Evolution

Apart from certain special considerations concerning corrosion, which are covered later in this chapter, the outer steel shell constitutes the basic waterproof enclosure of all steel tunnels. It is therefore mostly in the joint area that refinements in design details can be made.

From 1930 through 1960, the design of the joints in steel tunnels in the United States did not change very much. The method of operation during placement remained basically the same as it had been for the previous fifty years.

The joint was mated and aligned and then pinned with two heavy steel pins. A form was installed on both sides of the joint and a massive tremie concrete pour was made, which completely enclosed the joint and, hopefully, sealed it sufficiently to allow the internal liner plates to be welded in place without great difficulty. This was not always the case, however, because tremie concrete is often imperfect. The joints would leak or, worse yet, the concrete would penetrate into the interior of the joint and harden, requiring a time-consuming "mining" operation to remove it. On the other hand, the tremie joint had one major benefit: it formed a rather strong structural connection between the elements.

Although the gasketted joint is very effective in providing a more reliable working environment than the tremie joint for the installation of the steel liner plate, a disadvantage is that the structure of the joint became inherently weaker. The former thick tremie concrete encasement, which helped to tie the elements together, was no longer used. In addition, in recent years the thickness of the structural interior concrete ring has been reduced, as higher strength concrete and more sophisticated methods of analysis have come to be used.

These changes have resulted in an increasing problem with the effects of thermal expansion and contraction, causing longitudinal movements in the joints between elements and cracking and spalling of the interior finishes of the tunnel.

No leakages have yet been traced to these movements in any of the tunnels where they have been observed (two tunnels in Hampton Roads, Va., U.S.A., and the Fort McHenry Tunnel in Baltimore, Md., U.S.A.). However, there is concern that the liner plate—the major line of defense against eventual deterioration of the main gaskets—might begin to leak as a result of the seasonal joint contraction and compression; and, certainly, the effects on the wall tile are a serious maintenance consideration.

In the case of the Ted Williams Tunnel in Boston, the design of the joint was revised to accomplish two objectives:

- To make the joint flexible and thus prevent damage to the seal resulting from motions observed on similar tunnels; and
- To provide a controlled location for the movement in the joint to appear in the surface of the wall.

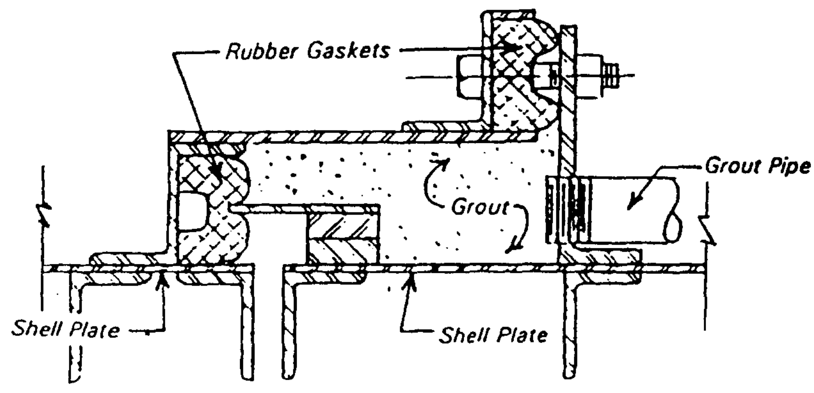



*Figure 4-1a. Joint detail for the Detroit River Tunnel (1910).*

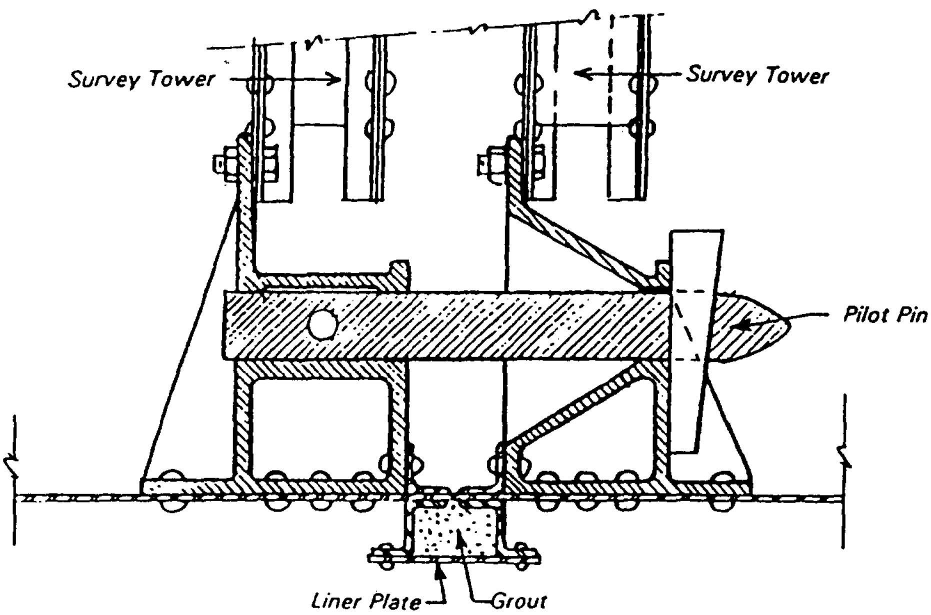



*Figure 4-1b. Joint detail for the Harlem River Tunnel (1914).*

This new joint detail may be said to constitute the state of the art, insofar as it has been recommended for the above project by the two American consulting firms that have designed the large majority of steel tunnels in the U.S.A.

Therefore, this discussion of waterproofing as it relates to steel tunnels will use this most recent development as its main example for joint design. European designers will realize that this joint detail is very close to that which is commonly used in Europe for concrete tunnels. In fact, the Gina type of profile was specified as an acceptable alternative type of main gasket for the Ted Williams Tunnel.

## *2.2 Waterproof Joints for Immersed Tunnel Elements*

### 2.2.1 Typical joints

For the purposes of this discussion, a "typical" joint is considered to be the immersion joint used between almost all of the elements in an immersed tunnel . Typical joints are distinguished from certain other special joints designed for specific purposes, i.e.:

- Closure joints, which are used at the secondary end when the last element has to be inserted rather than appended to the end of the previous element.
- Terminal joints, which are used between the shore ends of the immersed tunnel and the land portion.
- Underwater joints into rock, between immersed tunnel elements and rock tunnels.
- Seismic joints, which are designed to absorb triaxial seismic displacements without structural failure or failure in watertightness.

Figures 4-1a and 4-1b show early examples of the "typical joint" for the Detroit River Tunnel and the Harlem River Tunnel. Figures 4-2a and 4-2b show an isometric and a plan section of a typical tremie joint (a closure joint can be very similar to this type of joint). Figure 4-3 shows the original design of the typical joint for steel tunnel that was proposed for use on the most recent Ted Williams Tunnel in Boston.

This joint detail permits longitudinal movement generated by temperature variations during seasonal cycles. It is also designed to transfer shear forces caused by seismic events and differential settlements, as these might cause small rotational movements at joints.

The Omega gasket is an internally replaceable seal against water that might escape past the main exterior gaskets, and conducts it around and down to a drainage valve in the invert of the element. The valve relieves the pressure and drains the Omega gasket into the lower air supply duct. Although the Omega is rated for the ambient exterior water pressure, the main gaskets do most of the work; only a slight seepage or trickle might have to be intercepted by the Omega gasket. In the event of a major failure of the other seals, the valve could be closed and the Omega would take over.

The interior structural ring in the joint area is mainly the typical structure, composed of the steel shell tied compositely to the concrete. However, 60 cm of the joint ring concrete overlaps from one tube to the other, to provide a full structural shear key from element to element. This overlap permits relative longitudinal motion between elements, by virtue of the fact that the concrete is entirely isolated from the steel shell in this zone by a section of joint filler that prevents bonding or jamming of the shear ring as it gradually moves in response to expansion and contraction.

### 2.2.2 Contingency method for sealing typical joints

All immersion joints are equipped externally with vertical slots provided in the edges of the bulkheads to permit the installation of curved form plates. Such an arrangement allows the joint to be sealed using tremie concrete in the event that a seal cannot be obtained in the usual manner with the gaskets. This curved form plate is identical to the one shown in Figure 4-2a for the typical tremie joint. Although this provision is always made, this second line of defense has rarely, if ever, been used.

## *2.3 Closure Joints*

These joints are used at one end of the element that will close the gap in an immersed tunnel being constructed from both shores. The other end of this element is provided with a typical rubber-gasketted joint that would be joined first.

The closure joint provides roughly half a meter of joint adjustment for the accumulated variation in the length of the tunnels as they approach each other. It is sealed initially by tremie concrete contained within sheet pile enclosures and is kept out of the interior of the joint using enclosures with overlapping top and bottom hoods, designed to provide the range of joint separation noted above. In contrast, the typical tremie joint does not provide for a range of joint separation.

## *2.4 Terminal Joints*

Terminal joints are the interfaces between the portion of the tunnel constructed by means other than immersed elements and the immersed element portion of the tunnel. Terminal joints vary widely in design, depending on certain basic factors such as placement sequence, seismic effects, thermal movements, and differential settlement:

- If the elements are placed first, the joint will usually be constructed in the dry. The detail is relatively simple in this case. If the terminal structures are constructed first, then these structures must be provided with a built-in element joint adaptor to which the first and/or last element can be attached, using normal placement procedures.
- Seismic motions may control the design of the joint and its waterproofing detailing. The terminal structure to which the end element must connect (e.g., a ventilation building) will usually have a very different natural period of vibration, which will cause large relative movements of the terminal structure with respect to the immersed tunnel. This condition may require a full seismic joint with triaxial motion capability. Such a joint was used for the Transbay Tunnel in San Francisco (see Section 2.6 below).
- Allowances must be made for anticipated expansion and contraction between terminal elements and the terminal structures.
- Differential settlement between the immersed tunnel and the terminal structure may be an important aspect in the joint design if ground conditions are rather poor. Some designs (e.g., Fort McHenry Tunnel in Baltimore, Maryland, U.S.A) have attempted to avoid high stresses in the immersed elements by permitting differential settlements at the joints. Other designs (e.g., Second Hampton Roads Tunnel, Hampton Roads, Virginia, U.S.A.) have been designed to allow for expansion and contraction but no vertical movement.

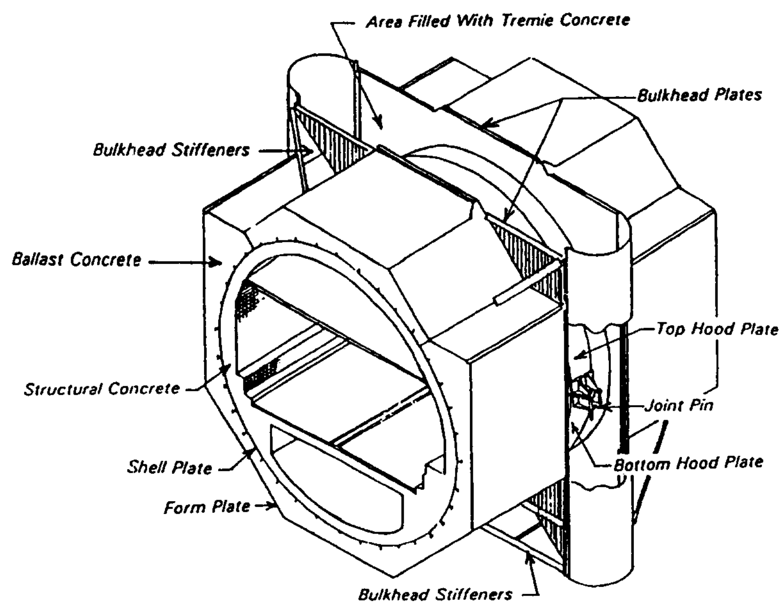



*Figure 4-2a. Typical tremie concrete joint for a typical steel double-shell structure.*

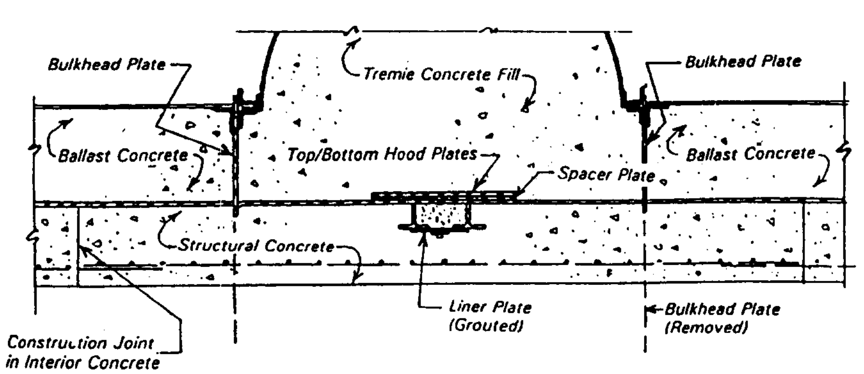



*Figure 4-2b. section through a typical tremie joint.*

## *2.5 Underwater Connections to Terminal Elements Constructed in Rock*

For the 63rd Street Tunnel (New York City, U.S.A.), two two-element tunnels were placed: one between the New York shore and Roosevelt Island, in the East River, and the other between the island and the Brooklyn shore. These two immersed tunnels were joined at the island by a rock tunnel.

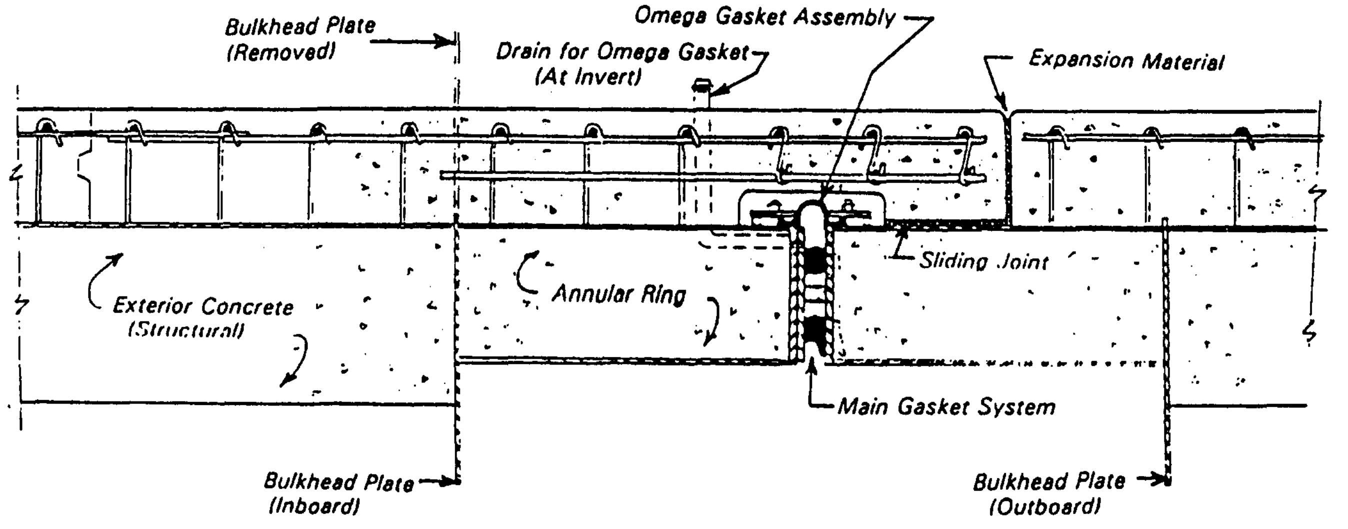



*Figure 4-3. Typical joint detail for Boston's Ted Williams Tunnel.*

## 2.6 Seismic Joints

The only seismic joint design used to date in the United States was developed for the BART Transbay Tunnel in San Francisco. The seismic joint was designed to permit triaxial displacements of ±8 cm in the longitudinal direction and ±15 cm in any direction in a vertical plane, while maintaining watertight integrity.

The vertical and transverse horizontal motions are permitted by precompressed rubber gaskets sliding on radial Teflon bearing surfaces. The longitudinal motion is taken by similar gaskets sliding on a circumferential Teflon bearing surface. Both sets of bearings are compressed by tensioned cables that allow the motions by rotating on Teflon-coated spherical bearings.

The assembly is protected from mud and the marine environment by exterior rubber boot enclosures. These assemblies were prefabricated, assembled and tested as units before being attached to the elements. Four of these seismic joints were required for the Transbay Tunnel. The joints performed well during the 1989 earthquake in San Francisco. Figure 4-4 shows the arrangement of this seismic joint.

## 2.7 Concrete Requirements

One of the basic advantages of steel tunnels lies in the fact that the watertightness does not rely heavily on the ability of the concrete to withstand cracking, as it does for concrete tunnels. The concrete strength is important from the structural standpoint, and random temperature and shrinkage cracking is a concern in this regard, however, from a leakage standpoint, steel tunnel elements can tolerate a great deal more cracking in their concrete linings than can concrete elements.

In addition, the concrete tunnel is not as accommodating to variable ground settlements as is a steel tunnel. As a result, intermediate immersion joints similar to those commonly used to introduce flexibility in concrete tunnels are not required. It should be noted that this situation can lead to designer complacency with regard to temperature and shrinkage control in the steel elements; and this can have the undesirable effect of introducing excessive reflected cracking in finished tile surfaces.

## 2.8 Cathodic Protection of Steel Elements

Except for concrete protection, it has not been the practice in the United States to provide special protection to the steel shell of immersed tunnels, unless stray direct currents are determined to be present. Where stray currents do exist or are expected, such as in an electrified rapid transit tunnel, provisions may be made for coating the steel shell with materials such as coal tar epoxy and/or the provision of impressed current cathodic protection systems.

Alternatively, stray current test stations may be specified to permit periodic measurements at several locations along the steel shell of the immersed tunnel. This arrangement permits flexibility in adjusting the extent of the cathodic protection to the actual conditions encountered in the completed project. In this regard, the internal reinforcing steel and the steel shell must be carefully bonded

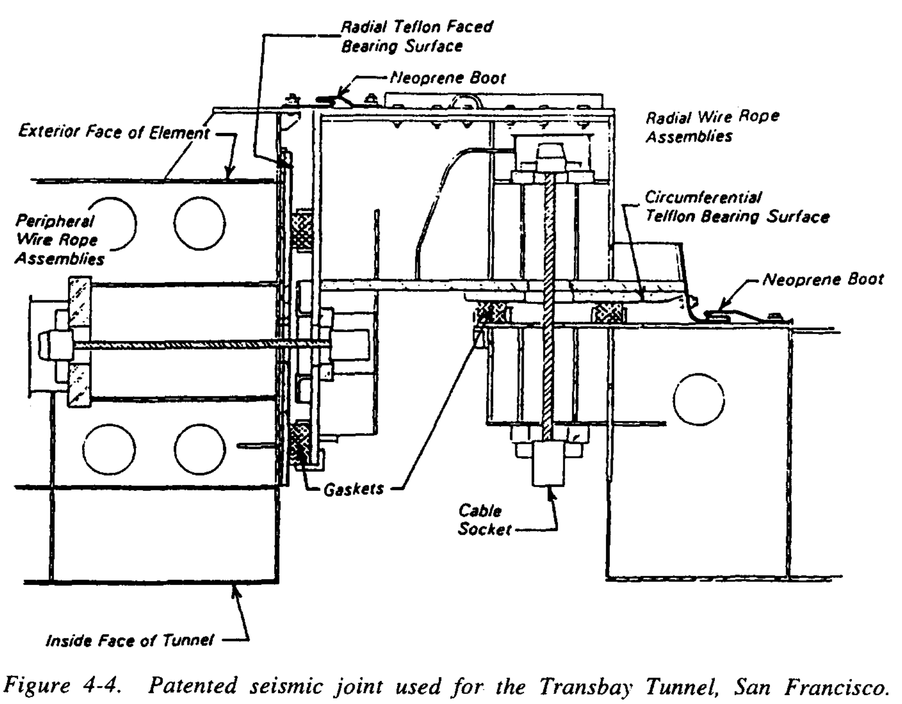



*Figure 4-4. Patented seismic joint used for the Transbay Tunnel, San Francisco.*

electrically throughout the tunnel to prevent deterioration, or even accelerated localized corrosion, in the presence of stray currents.

## 3. Concrete Tunnels

Because steel is expensive on the European mainland, alternatives to the steel shell have been developed. In the past, problems have occurred because the concrete was not watertight, as a result of both cracks and the lack of density of the concrete.

Leakage water can enter the tunnel in two ways:

- Through the concrete structure.
- Through the joints.

These types of leakage are discussed below.

### *3.1 Water Leakage through the Concrete Structure*

It is very important that the concrete be impermeable. Possible causes of cracks during construction are climatic temperature fluctuations and hydration heat following concrete placement. If the concrete structure cracks, water can leak into the tunnel. Penetration of water into the concrete can cause corrosion of the reinforcement.

To prevent water from leaking into the tunnel through the concrete, two different measures are applied:

- Membranes.
- Cooling the concrete to avoid cracking.

It should be noted that no concrete is 100% watertight, since some water seepage will occur even through a dense concrete, though this could take years. Such water is not visible because it evaporates at the inner face of the tunnel structures. If the water is saline, the salt will remain in the concrete and, after some years, may have contaminated it to such a level that it may cause corrosion of the reinforcement at the inner side of the structure. This has been the case in the Limfjord tunnel, where a combination of an imperfect membrane, saline water, and a concrete with many fine cracks has already caused chloride contamination above the critical level in some parts of the tunnel.

### 3.1.1 Membranes

Membranes are provided on many immersed tunnels to help prevent outside water from entering the tunnel space. To achieve this goal, the membrane must have a life expectancy similar to that of concrete, and it must be chemically and biologically resistant to the environment. It also must be able to resist forces from outside during construction and in the permanent condition, and it must be sufficiently elastic to accommodate any cracks that may be expected in the structure. It is advisable that the membrane adhere to the entire surface, thereby eliminating the possibility of water flow between the membrane and the concrete, in the event that the membrane is breached.

Several types of waterproofing membranes have been used in the construction of immersed concrete tunnels:

- Steel membranes.
- Bituminous membranes.
- Polymeric sheet membranes.
- Liquid-applied membranes.

Each of these types of membrane is discussed below.

#### *3.1.1.1 Steel membranes*

This is the oldest method used for preventing water from penetrating into the tunnel. The steel lining encases the tunnel completely and has no structural function. Generally, the steel and concrete are not permitted to be considered as a composite structure.

Normally steel membranes consist of 6- to 8-mm-thick steel plates welded together. The connection between the membrane and the concrete structure is provided by shear studs welded to the membrane. In addition to normal control of the welds, all welds between plates are further tested for watertightness. This testing may be done rather simply by using a portable vacuum box.

Because steel membranes are subject to corrosion, protective measures may be needed. If the tunnel is well below the water surface and is surrounded by backfill, corrosion will occur very slowly.

The following minimal measures must be taken in order to avoid local galvanic corrosion:

- Light sandblasting should be used to remove mill scale from the steel plates.
- Weld material should be cathodic, compared with the steel membrane.

For the Guldborgsund Tunnel in Denmark, the aforementioned measures have been considered sufficient to ensure a lifetime of 100 years of the steel membrane, which is placed at the bottom and the sides of the tunnel. If the planned inspections show that further measures are required, cathodic protection similar to that used on steel tunnels may be installed.

On the Conwy Tunnel in Wales and the Tingstad Tunnel in Sweden, the steel membrane has been protected with both painting and cathodic protection.

The shear studs connecting the membrane with the concrete structure should be designed so that they can transmit the shear forces from the soil. It should further be ensured that the number of studs is sufficient to limit differential movements between membrane and structure, so that individual studs are not overloaded—a situation that could create cracks through the membrane.

Although the concrete is placed against it, the steel membrane will not adhere to the concrete. The separation may take place in the casting basin because the temperatures of the membrane and concrete are different. The separation of steel and concrete will allow water from a leak in the membrane to find any leak in the concrete. In order to limit possible leakages, consideration should be given to dividing the membrane into minor areas of perhaps 10 x 10 m, using ribs welded onto the inner side of the membrane.

Steel membranes have mainly been used at the bottom or the bottom and sides of immersed tunnels. The remaining part of the tunnel surface has then been covered with another type of membrane, usually a bituminous membrane. The transition between steel and bituminous membrane requires complex clamping in order to ensure permanent watertightness.

A few concrete tunnels, including the Tingstad Tunnel in Sweden and the Deas Island Tunnel in Canada, have been provided with steel membrane all around.

Steel membranes have the following advantages:

- Good experience has been gained with such membranes in the past.
- It is a well-known technology.
- It yields savings in formwork.
- The membranes are robust.

The disadvantages of steel membranes are:

- There is a risk of corrosion and they require protection.
- A leakage in the membrane may cause water penetration into the tunnel far from the leak in the membrane.
- The membranes are quite costly.
- A leakage in the membrane cannot be repaired.

#### *3.1.1.2 Bituminous membranes*

Bituminous membranes generally are prefabricated mats reinforced by polyester or glass-fibre fabric. They may be glued on with hot bitumen or by warming a bitumen layer applied to the mat during fabrication. Either hot-blown or polymer-modified bitumen may be used. The latter has

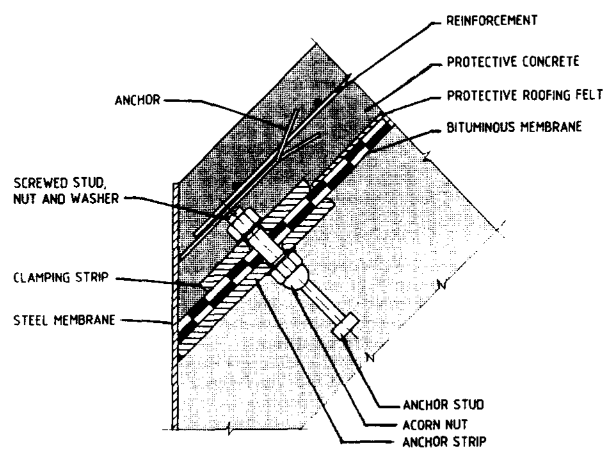



*Figure 4-5. Connection between steel and bituminous membrane.*

better elastic properties than the former, a factor that is important, for example, in clamping strips at the edges.

For tunnel waterproofing, the polymer-modified bituminous membrane is preferable to the normal bituminous membrane because it reduces plastic deformations and loss of pressure under the clamping plates. The membranes must fully adhere to the primed concrete surfaces in order to prevent water ingress through leaks in the membrane to flow in the space between the membrane and the concrete structure.

A bituminous membrane is normally composed of two layers of bitumen-impregnated mats. The mats are placed with at least 100 mm overlap, and the joints in the two layers are staggered.

In order to avoid water seeping from behind the membrane, all free edges of the membrane must be clamped, e.g., by clamping strips of steel (see Fig. 4-5). A concrete slab 100 mm to 150 mm thick initially protects the bituminous membrane when it is placed on the tunnel roof. Because the bituminous membrane cannot transmit shear forces of long duration, it is necessary to anchor the protection slab (see Fig. 4-6).

If the membrane is used on the tunnel sides, a sliding layer of soft bitumen is applied to reduce shear transfer from the backfill (as was done at the Rupel Tunnel in Belgium), or protected by 100-mm-thick concrete plates bolted to the walls (as at the Drecht Tunnel in The Netherlands). At the tunnel bottom, where it is difficult and complicated to secure the membrane to the concrete, steel plate is still used.

Bituminous membranes at the tunnel roof have often been combined with steel membranes at the sides (and bottom) of the tunnels. This method has been applied to the Elbe Tunnel in Germany, the Conwy Tunnel in Wales, and the Guldborgsund Tunnel in Denmark.

Bituminous membranes have the following advantages:

- They are cheaper than steel.
- Their performance has been demonstrated to be successful.
- The membranes are relatively robust, if placed in two layers.
- The membranes can bridge normal fissures.
- The membranes can be applied on roofs, where steel membranes are difficult to place.

The following disadvantages are associated with bituminous membranes:

- The success of the membrane depends on an experienced labour force to ensure watertightness at the edges and at anchor bolts for protection of the concrete.
- Because the application of the membrane can be started only after the structural concrete is properly cured and is dry, this operation is on the critical path. Therefore, the timing between concreting and flooding of the casting basin requires close coordination with a specialist subcontractor.
- Bituminous membranes do not transfer shear in the membrane plane.
- Special arrangements at expansion joints (steel Omega or similar) may be required.
- A durable and complete adhesion to the concrete surface is difficult to obtain.
- A leakage in membrane may cause water to penetrate into the tunnel far from the place where the leak in the membrane occurred.

### *3.1.1.3 Polymeric sheet membranes*

Polymeric sheet membranes may be thermoplastic materials such as polyvinyl chloride (PVC), polyethylene (PE), chlorinated polyethylene (PEC), and polyisobuthylene (PIB), or elastomeric materials as chlorosulphanated polyethylene (CSM), polychloroprene (Neoprene) (CR) and isoprene-isobuthylene (butyl rubber) (IIR).

Thermoplastic materials may be joined by hot air or plate welding, which are rather simple operations. Elastomeric materials are joined by adhesion, while the rubber types (neoprene, butyl, etc.) may be vulcanized. Although fairly expensive, vulcanization provides a strong connection.

These types of membranes are very thin and therefore are very vulnerable to mechanical damage during construction.

The butyl rubber membrane used on Denmark's Limfjord Tunnel was glued on by a polymer-modified cement slurry.

The advantages of polymeric sheet membranes are:

- They are low-cost, in comparison to membranes.
- Fewer discontinuities in details, thereby reducing the chance of errors.
- They are less subject to damage during and after construction.

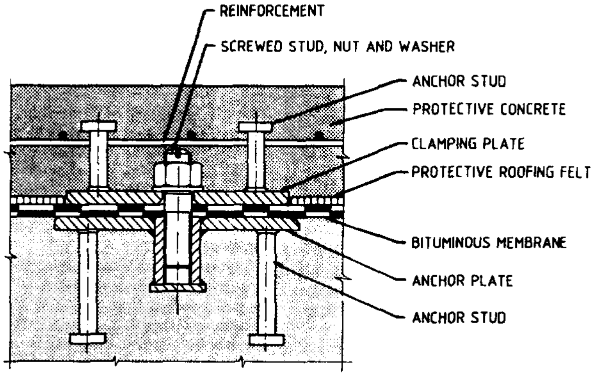



*Figure 4-6. Connection between structural and protective concrete.*

- Better concrete quality can be maintained.
- They permit reduction in construction time.
- They facilitate a clean and straightforward design.

The disadvantages of butyl rubber membranes are:

- Workmanship problems with the joining of the sheets.
- If applied in only one layer, they are very vulnerable, especially immediately after application.
- Degradation of seams occurs over time.
- Embrittlement and degradation of the sheets occurs over time.
- Tensile and puncture strength may be inadequate.
- A durable and complete adhesion to the concrete surface is difficult to obtain.
- A leakage in membrane may cause water to penetrate into the tunnel far from the place where the leak in the membrane occurred.
- A leakage in the membrane cannot be repaired.

#### *3.1.1.4 Liquid-applied membranes*

Liquid-applied membranes, which may be epoxy-based or polyurethane-based, can be applied by spraying or by rolling. The concrete surface must be clean and dry in order to achieve good adhesion. If the membrane does not separate from the concrete around fissures, the elongation of the membrane over a fissure formed after spraying will be infinite.

Sprayed membranes have not often been used. A sprayed membrane used in the construction of an earlier tunnel in Hong Kong consisted of a 2.0-mm-thick layer of epoxy tar. Unfortunately, the first tunnel elements leaked because the coating was not able to absorb the strain when cracking of the concrete occurred after the waterproofing had been applied (*Engineering News Record*, July 13, 1989). Later on in the project, elements sprayed with an improved membrane showed a better performance. Considerable advances have since been made in spray-on membranes used on subsequent tunnels, especially in bond, elasticity, and toughness.

Advantages of liquid-applied membranes are:

- They are low in cost.
- They are continuous, i.e., without joints.
- They may transfer shear.

The following disadvantages are associated with these membranes:

- Difficulties in bridging fissures may occur after the membranes have been applied.
- There has been little proof of their performance.
- A leakage in the membrane cannot be repaired.

### 3.1.2 Cooling the concrete to prevent cracking

During placement of concrete in walls that are on previously placed floors, the temperature in the walls increases because of hydration heat. When the walls are cooling down, the shrinkage is obstructed by the cool floor, creating tensile stresses in the concrete. When the tensile stresses exceed the ultimate strength of the concrete, cracks will occur.

To prevent the extreme rise in temperature that can lead to the above scenario, reduced cement contents (275 kg/m$^3$) or a partial cement replacement with pulverized fuel ash (PFA) can be applied, and a cement that develops relatively little hydration heat (blast furnace cement with more than 65% slag) may be used. Moreover, the lower parts of the outer walls can be cooled during the first days after placement. This is done by pumping water through cast-in metal tubes.

The controlled process results in a considerably reduced maximum concrete temperature just above the floor slab, while a gradual temperature increase is obtained from this spot to the uncooled concrete in the upper part of the walls and the roof. In addition, the length of each pour may be limited to approximately 25 m.

This method of preventing cracks in the walls has been successfully applied in the Netherlands since the late 1970s. Belgium and Germany have also adopted this method (e.g., on the Liefkenshoek and Ems Tunnels, respectively), and it was used for the Western Harbour Crossing in Hong Kong.

Compared with membranes, the cost of cooling is low and it reduces construction time. Even though cracks cannot be avoided totally, those cracks that still appear can be injected prior to flooding the casting basin. Injection must be done in the absence of water flow; otherwise, the crack will stay open and water will continue to flow into the tunnel.

## *3.2 Water Leakage through the Joints*

The following subsections discuss requirements for joint design and provide examples of joint layout. The following types of joints are normally used in immersed concrete tunnels:

- Immersion joint: used between tunnel elements.
- Terminal joint: the joint between the shore end of the immersed tunnel and the land structures .
- Closure joint: the joint made in situ after the last element has been placed.
- Expansion joint: used within each individual tunnel element, without continuous reinforcement.
- Construction joint: used within the individual tunnel element with continuous reinforcement.
- Joints between precast units for prestressed tunnel elements.

Many tunnels have been designed without expansion joints. In such cases, the immersion joints, terminal joints and/or closure joints are normally maintained as permanent expansion joints. When used, permanent expansion joints are located approximately every 25 m. Because these joints are generally difficult to execute, there may be a risk of leakage occurring at the joints.

A few tunnels have been designed as monolithic tunnels. The following descriptions deal mainly with immersed concrete tunnels provided with waterproofing membranes.

### 3.2.1 Assumptions for joint design

The purpose of a joint is:

- To connect individual castings, segments, or prefabricated units.
- To allow for movements caused by differential settlements, temperature, creep and shrinkage.
- To facilitate construction.

A joint is a discontinuity in the normal homogeneous concrete structure. Therefore, one of the main tasks in joint design is to ensure watertightness within the expansion limits for which the joint should act.

In a tunnel with a membrane, two waterproof barriers exist: (1) the reinforced concrete, and (2) the membrane.

For the design of watertightness in the joints, it is important that joint movements, alignment and tolerance limits have been investigated and specified. The maximum water pressure for which the joint should be watertight should also be specified.

It is essential to know the properties of the joint materials, not only when new but also when aged, as well as their design life. Joints can only be replaced if this possibility is foreseen in the design stage.

### 3.2.2 Intermediate joints

Normally the immersion joints are provided with a Gina gasket and an Omega seal (see Fig. 4-7).

The Gina gasket is mounted on a steel frame at the primary end of the tunnel element. After the tunnel element is placed, this end is drawn against the secondary end of the

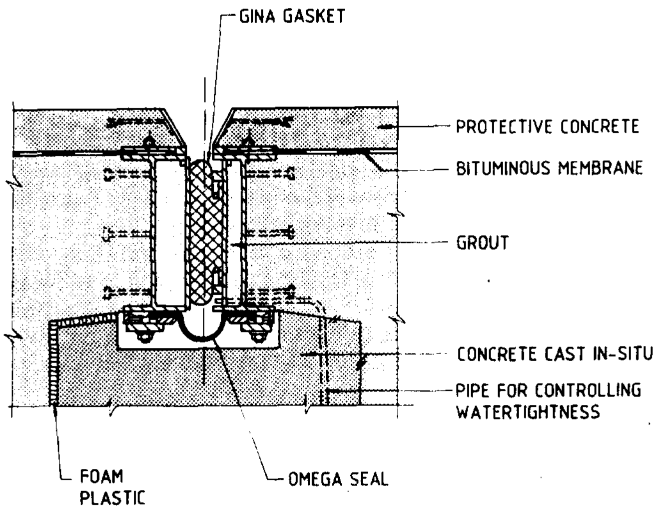



*Figure 4-7. Intermediate joint.*

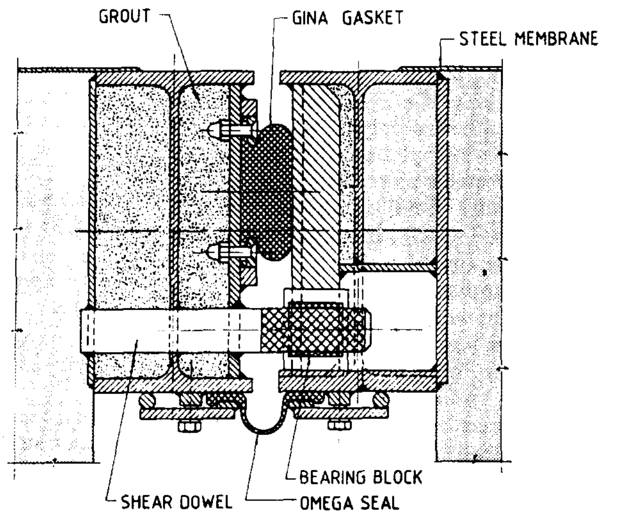



*Figure 4-8. Shear key of steel at intermediate joint.*

previous tunnel element, which has been provided with a matching steel frame.

In some early immersed tunnels (e.g., the Limfjord Tunnel), the frames were made of U-profiles. Although cast-in separately, the frames could not be adjusted, after the concrete had been cast, to ensure sufficiently accurate alignment. The Dutch tunnels and other recent tunnels (e.g., the Elbe, Conwy and Guldborgsund Tunnels), have used an H profile, and the end plates of the frames were welded after the concrete had been cast. The space behind the plates was injected with grout. This method permits very small tolerances to be met.

After the Omega seal has been installed, the two tunnel elements can be connected by casting reinforced concrete in the remaining space between them. This procedure was used on the Limfjord and the Guldborgsund Tunnels.

Alternatively, a permanent expansion joint with shear keys can be made. The shear keys may be constructed of reinforced concrete or of steel. This method was used on the Conwy Tunnel, the Singapore Cable Tunnel, and the new Bilbao (Spain) Metro Tunnel (see Fig. 4-8).

If the shear keys are of reinforced concrete, which has been common practice until recently, access to the Omega seal is hindered, whereas access is retained if shear keys of steel are used. The shear keys of Dutch tunnels are installed only in the floor and are made of steel tubes filled with concrete (see Fig. 4-9).

Even when the joint is used as a permanent expansion joint, the movements are usually not large and the Gina gasket stays watertight. This means that the Gina and the Omega constitute a watertight barrier in the joint. No serious leakages of this type of joint have been reported.

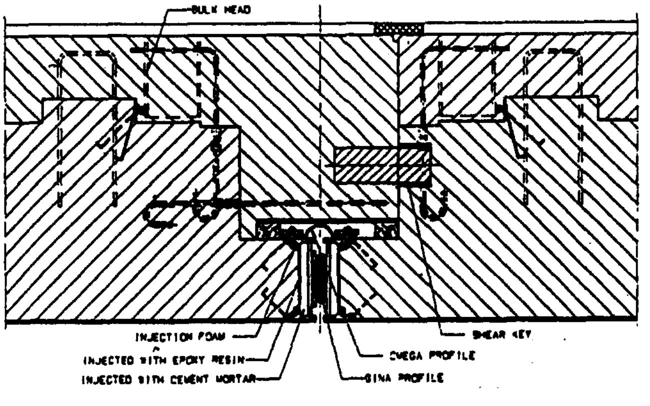



*Figure 4-9. Shear key in floor.*

### 3.2.3 Terminal joints

This type of joint is often identical to the immersion joints when the portal has been built before the element is placed. In this case, the sea or river end of the portal building is provided with a bulkhead and a cast-in steel frame, like the secondary ends of the tunnel elements.

If the elements are built in the entrance, as was done for the Guldborgsund Tunnel and the Prinses Margriet Tunnel, the joints will be sealed temporarily between the tunnel element and the entrance structure. The permanent water-tightening Omega seal could be constructed as shown in Figure 4-10.

The possibility of differential settlements between the *in-situ* constructed part and the immersed tunnel part must always be considered. With a sandjetted or sand-flow foundation, settlement will occur when the load on the temporary foundations is transferred to the sand foundation.

In Dutch tunnels, the shear keys in the floor are placed as late as possible in the construction process in order to let the sand settle. The shear keys prevent unequal settlement between the immersed element and portal while

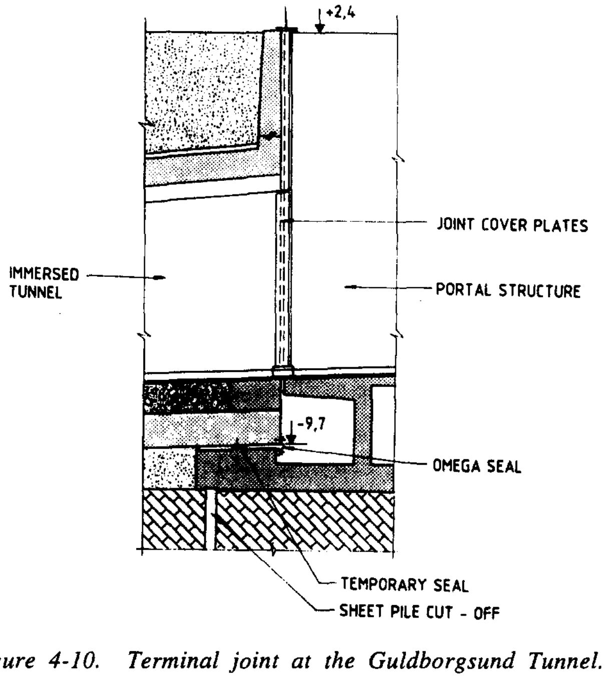



*Figure 4-10. Terminal joint at the Guldborgsund Tunnel.*

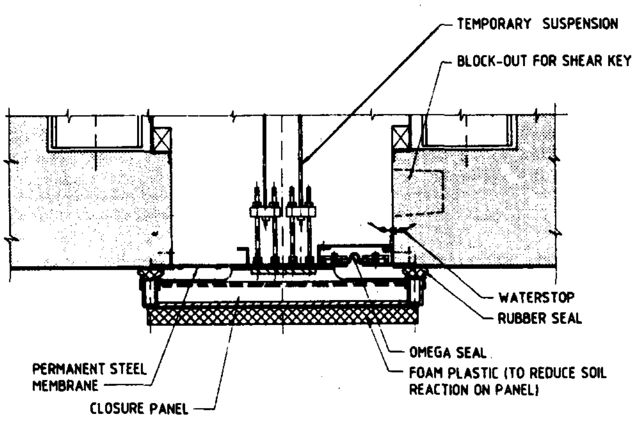



*Figure 4-11. Closure joint at bottom slab.*

allowing for rotation when the opposite end of the immersed segment settles.

### 3.2.4 Closure joints

In most cases, after the last tunnel element has been placed, a gap approximately 1–2 m wide remains to be closed. This closure joint is located between the last tunnel element placed and either a previously placed tunnel element (or an *in-situ* cast tunnel) or a portal structure. After external closure panels have been installed around the tunnel, the gap can be dewatered and closed from the inside of the tunnel by casting reinforced concrete.

In order to facilitate placing and making the panels watertight, the end of the last tunnel element placed and the structure to which it shall be connected are normally given a rectangular outer shape. The two parts can be monolithically connected, or a permanent expansion joint can be installed. Figure 4-11 shows an example of a closure joint for a tunnel provided with a waterproofing membrane. As an alternative, a permanent intermediate joint may be affixed to the end of the previous element while the adjacent final joint is made monolithic. After completion of the final joint, the intermediate joint can be released.

In the older Dutch tunnels, the closure joints were provided with a double-Omega profile (see Fig. 4-12). This type of joint can be very vulnerable. In construction practice today, a segment of a normal cross-section of the tunnel is built into the closure joint (see Fig. 4-13), while normal rubber-metal waterstops provide the watertightness.

### 3.2.5 Expansion joints

Depending on local conditions, individual tunnel elements may be provided with a number of expansion joints. In order to facilitate handling of the tunnel elements, these joints are fixed in the construction stage, normally by prestressing cables or rods (see Fig. 4-14). After the element is placed, these connections are released or cut in order to allow free expansion.

The watertightness of the expansion joints is ensured by cast-in waterstops. The tunnel membrane can be continued across the joint in the form of a steel Omega profile, as was done at the Elbe and the Conwy Tunnels (see Fig. 4-15).

The design of the steel Omega profile, where the corners are especially critical, has been verified by full-scale testing.

Since the mid-1970s, all of the immersed tunnels in the Netherlands have been constructed in segments, with the lower portions of the walls cooled and the membranes omitted. The only waterstop in the expansion joint is a rubber-metal waterstop. Because the rubber is subject to deformation by the water pressure, the metal strip in the concrete is used to guarantee watertightness. This single waterstop must be of high quality. To ensure that the waterstop works properly, injection tubes at the end of the metal strap are provided, as shown in Figure 4-16.

Temperature fluctuations cause the joint to open and close. When the joint opens because of shrinkage, sand or soil can enter the joint. Then, when the tunnel section expands, the joint cannot close up again because of the sand or soil in the joint. To keep the joint clean, it is provided with a rubber gasket, as shown in Figure 4-17. Because it is difficult to place the gasket in the bottom slab, the gasket is replaced by a steel strip, as shown in Figure 4-18.

### 3.2.6 Construction joints

Construction joints may be designed as expansion joints; generally, however, at least some of the construction joints are made with continuous reinforcement.

Even though good adhesion between the concrete at each side of the joint is achieved by sandblasting or waterjetting of the primary concrete face, there is always a risk that a crack will arise at the joint. Joints are therefore provided with waterstops that act as a secondary barrier for tunnels provided with a membrane that is carried continuously across the joint (see Fig. 4-19).

Another type of construction joint is the joint between

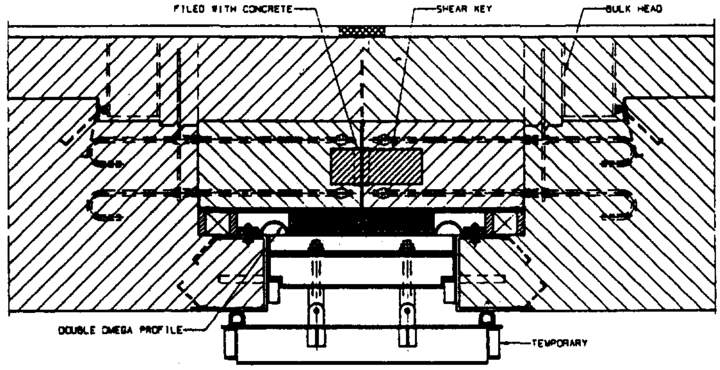



*Figure 4-12. Closure joint formerly used in Dutch tunnels.*

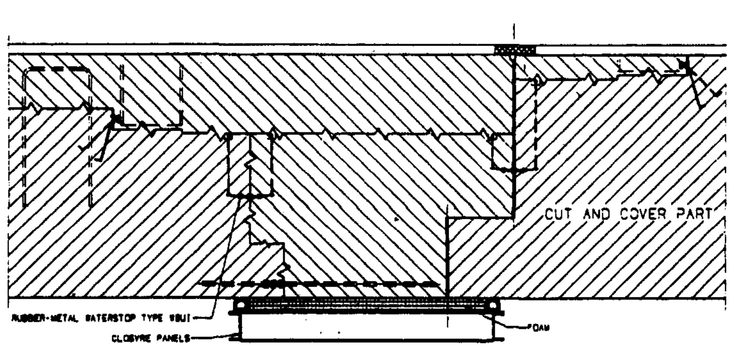



*Figure 4-13. Closure joint used on modern Dutch tunnels.*

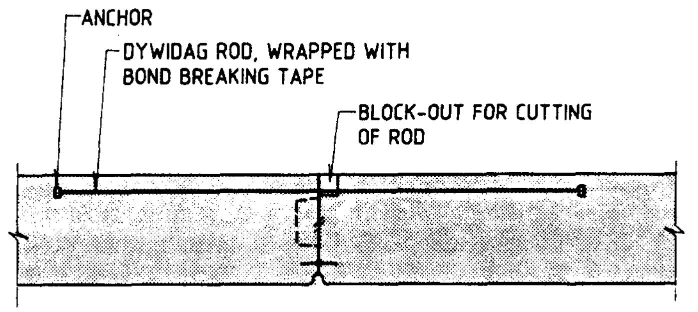



*Figure 4-14. Temporary reinforcement at expansion joint.*

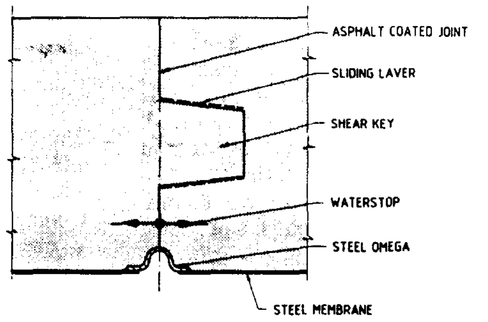



*Figure 4-15. Expansion joint with steel Omega profile.*

precast segments of prestressed tunnels. An example from a cable tunnel in Singapore is shown in Figure 4-20.

## 4. Maintenance

### 4.1 Experience with Leakage in Steel Tunnels

Leakage in the immersed portion of steel tunnels generally is minimal. Watertightness depends to the greatest extent on the care with which the integrity of the steel shell is maintained through design and execution. Tunnel specifications must therefore require suitable welder qualification, as well as radiographic, ultrasonic and dye-penetration methods of weld inspection and tests for watertightness during fabrication.

Permanent penetrations of the shell are to be avoided whenever possible in the design. Where openings are provided for access or concrete placement, great care must be taken to inspect and test welds of the closure plates for watertightness.

The soap bubble and vacuum box test are good watertightness tests for smaller openings in the shell, after the concrete has been placed. Temporary piping used in placing operations must be cut off and sealed with a cover plate seal-welded over the opening. Special details such as cross-passages between tubes must permit good welding accessibility to assure quality welding.

Although the experience with watertightness of steel tunnels has been very good, given proper design and fabrication procedures, if a leakage problem arises, it most often occurs at the terminal joints with the land section. The problem usually occurs at the transition from a totally enclosing steel shell to a conventional externally applied structural waterproofing system. Furthermore, there may be problems in keeping the excavation area dry where the waterproofing is being installed. Hence, proper detailing at this interface is critical.

### 4.2 Leakage in Concrete Tunnels

#### 4.2.1 Leakage through the tunnel structure

Reports of leakages in some Dutch concrete tunnels, both with and without membranes, have mainly concerned minor leakages through the floor, walls and roof.

In tunnels with membranes, leakage through cracks in the walls or floor is difficult to repair because it is almost impossible to find where the corresponding leak in the membrane is located. In such cases, the leakage is stopped by injecting all cracks. When leakage through one crack is stopped, water will appear through another crack. It is easier to stop leakage in tunnels without membranes, because the leakage point can be detected. In both cases, the injection has to be done in the absence of a water flow.

There have been some leakage problems at the Limfjord Tunnel in Denmark. The leaks have mainly occurred through transverse construction joints, which were provided with waterstops. The total amount of water leakage in the tunnel is rather small—on the order of 50–100 l/hour (based upon measurements of a minor area). However, even small amounts of leakage through the concrete structure are unacceptable, as the water in the fjord is saline and therefore has caused corrosion of the reinforcement in the tunnel structure.

The tunnel was provided with a membrane of butyl rubber. Obviously, this membrane is not watertight; and, because the adhesion to the concrete is probably insufficient, the water may have spread between the membrane and the structure. The leakages had already begun during the construction stage. Since then, further attempts have been made to stop the water seepage. Trial injections have been made with epoxy, polyurethane and acrylic gel. The latter has produced the best results, but in all cases the leaks started again after some time had elapsed since injection. It is assumed that the reason for this lies in

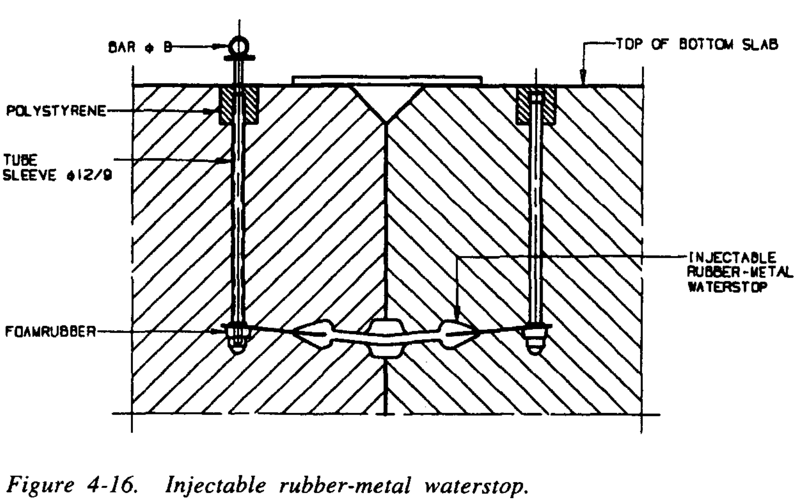



*Figure 4-16. Injectable rubber-metal waterstop.*

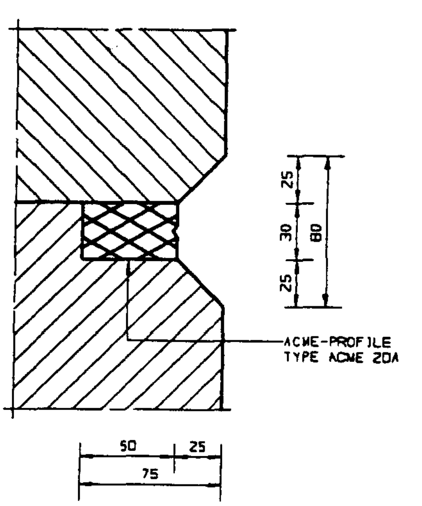



*Figure 4-17. Expansion joint in walls.*

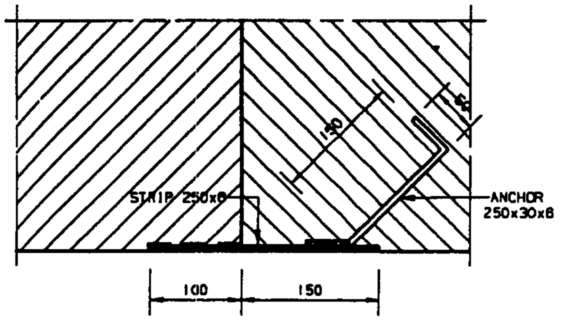



*Figure 4-18. Expansion joint in floor.*

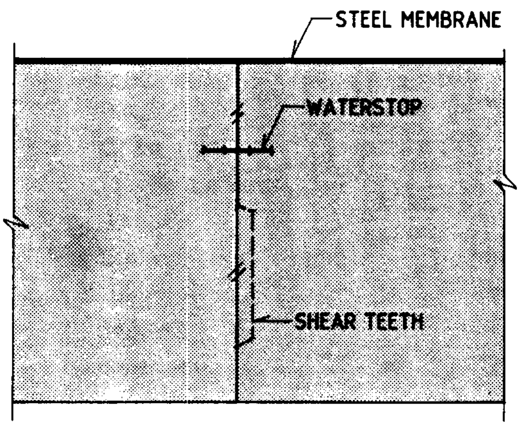



*Figure 4-19. Construction joint.*

seasonal temperature variations, since the long monolithic tunnel is alternately exposed to tension and compression, which cause movements of the cracks.

Other measures, such as internal membranes (polyurethane and fiber texture-reinforced cement-paste membrane) have been tested. The adhesion of the polyurethane membrane to the concrete was not sufficient, and the cement paste membrane was very expensive. Therefore, attempts to find a suitable repair method are continuing.

### 4.2.2 Leakage through joints

Leakages through expansion joints are usually caused by gravel pockets that formed under the rubber-metal waterstop during concrete placement. Leakage problems are usually handled by injection; however, if injection is not successful the water must be directed to the drains within the tunnel.

In Dutch tunnels, a control system is available to detect leakage through the Gina gasket. During construction, a 1/2-in. pipe is embedded in the concrete, connecting the space between the Gina and Omega to the central gallery.

A leak through the Gina was detected at the Zeeburger Tunnel. However, the water did not enter the tunnel, which means that the Omega seal is watertight.

This type of leakage occurs because of the high positioning of the terminal joint, which lies just below the surface of the water. During winter, when the river freezes, the Gina loses its elasticity and the tunnel shrinks, causing the joints to open. The stiff Gina cannot swell to keep the gap closed.

A survey of leakage in existing tunnels was performed within the Working Group. The responses indicated no serious leakages, and only some small drippings, for both steel and concrete immersed tunnels.

### 4.2.3 Summary

Inherently, immersed tunnels offer great opportunities for engineers and contractors to build bone-dry tunnels. For this reason, the watertightness that can be achieved within an immersed tunnel is a major advantage in favour of selecting this form of tunnel, particularly for deep sea crossings.

For both concrete and steel tunnels, the watertightness and the continuity of the joints between the tunnel elements are critical. The joints should allow some movement without leakage or any other detrimental effect on the functioning of the tunnel. In the Ted Williams Tunnel in Boston, a flexible joint has been used for the first time in a steel tunnel in the United States.

As the awareness of seismic exposure increases, so the joints in a tunnel must increasingly be designed to carry high seismic shears and must be restrained positively against excessive opening. Axial motions can be restrained by using stressed or unstressed post-tensioning across the joints, while vertical shears must often be carried by steel shear keys stressed onto the concrete and fitted with bearings.

For watertightness, joints within and at the ends of tunnel elements are some of the most vulnerable zones in an immersed tunnel crossing. The goal is to achieve watertight intermediate and immersion joints that can be successful for all time and are maintenance free. Improvements to these design details continue to be made, both for temporary and permanent seals. For joints within elements, considerable advances have been made in this respect with the use of hydrophillic sealants and groutable waterstops.

As stated above for concrete tunnels, care taken in concrete mix design, cooling, and reductions in the heat of hydration can help considerably in reducing or preventing the formation of cracks. Recent tunnels have also used pulverized fuel ash (p.f.a.) to replace some cement content, further reducing the heat generated. Maintaining a low water/cement ratio through additives also aids in creating a high quality concrete.

For membrane application, the ability to bridge fissures that develop after application of the membrane has been a drawback to earlier membranes that were comparatively brittle, but the most recent membranes have improved this significantly. With complete adhesion, the source of a leak can be traced. Repairs to a spray-on membrane are difficult, though improvements in this field are expected. As the quality of these membranes improves, so the emphasis on preventing leaks must switch to the joints used in both steel and concrete tunnels. To improve the puncture resistance of areas between immersion joints, advances are being made in spray-on membranes.

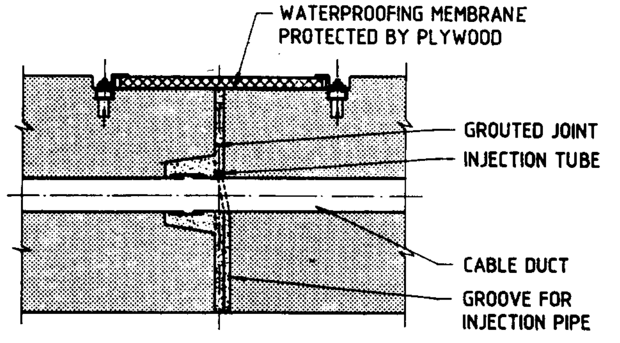



*Figure 4-20. Joint between precast segments.*

For concrete tunnels, a cost comparison must be made between producing a dense, crack-free concrete and providing external barriers to leakage—i.e., barriers compatible with existing site conditions.

## 5. References

Reina, Peter and Naoaki, Usui. 1989. Sunken tube tunnels proliferate. *Engineering News Record*, July 13, 1989, 30–37.

PII:S0886-7798(97)00009-6

# Chapter 5

# ENVIRONMENTAL ISSUES

by

**CHRIS MARSHALL** U.K.
**Symonds Travers Morgan**

***Input received for case studies from:***

| | | |
|---|---|---|
| Walter Grantz | U.S.A. | Ted Williams Tunnel, Fort Point Channel |
| Ahmet Gursoy | U.S.A. | Fort McHenry |
| Chris Hakkaart | The Netherlands | Medway |
| Kjell Hestner | Sweden | Saltsjön |
| Christian Ingerslev | U.S.A. | Western Harbour Crossing |
| Chris Marshall | U.K. | Conwy, Lee, Drogden |
| Jan Saveur | The Netherlands | Noord, Piet Hein |
| Cor Th. van Doorn | The Netherlands | Wijker |

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.125–134, 1997

Printed in Great Britain
0886–7798/97 $17.00+0.00

Pergamon

# Chapter 5: Environmental Issues


1. INTRODUCTION

2. EFFECTS ON THE WATERCOURSE
2.1 Changes in Flow Patterns
2.2 Changes in Scour and Siltation Patterns
2.3 Pollution of the Watercourse
2.4 Effects on Wildlife
2.5 Recommendations for Good Practice

3. EFFECTS ON THE GROUNDWATER REGIME
3.1 Depletion and Pollution of Aquifers
3.2 Recommendations for Good Practice

4. DISPOSAL OF EXCAVATED MATERIAL
4.1 Disposal of Uncontaminated Material
4.2 Disposal of Contaminated Material
4.3 Recommendations for Good Practice

5. LAND USE
5.1 Land Reclamation
5.2 Graving Docks
5.3 Recommendations for Good Practice

## 1. Introduction

Concern for the environment has never been more acute than it is now. Construction professionals are rightly recognising that the environmental impact of their projects must be taken into account at every stage, from inception to operation. In recent years, increasingly formal procedures have been developed for measuring the environmental impact of a proposal so that appropriate ameliorative measures can be incorporated during execution. Such assessments have resulted in some projects being abandoned.

The aim of this chapter is to identify the specific aspects of immersed tunnel design and construction that commonly cause environmental concern. Such issues are described, and examples are included of how particular problems have been solved in practice. Recommendations are given for future good practice. The text does not attempt to comment on procedural matters, nor does it address the broader debate regarding environmentally sustainable transport policy. The chapter deals with four main areas of environmental impact, briefly described below. The bulk of the material presented is drawn from thirteen case studies relating to particular tunnels. Table 5-1 summarises how the topics and the case studies are related.

**Effects on the Watercourse** (Section 2). All immersed tunnels cross a watercourse, typically a river estuary or narrow sea channel, though examples exist across canals, harbours and inland rivers. Such watercourses are usually characterised by complex physical characteristics, such as currents, tides, bathymetry, plan shape, sediment transport, scour and deposition characteristics. In addition, they usually support patterns of plant and animal life which, besides being intrinsically complex, are sensitive to any changes in the physical characteristics of the watercourse.

This section reviews effects on the watercourse under the following four headings: changes in flow patterns; changes in patterns of scour and siltation; pollution of the watercourse and effects on aquatic life.

**Effects on the Groundwater Regime** (Section 3). Immersed tunnel construction commonly involves large scale ground water lowering for two purposes: to construct approach cuttings and to create a tunnel element graving dock. Although there may be little choice in selecting the location of the approaches, there is usually more freedom in siting a graving dock. As an alternative, existing dry docks or steel fabrication yards have also been used for tunnel element construction.

Where ground water lowering is necessary, the environmental consequences can be significant. The two main issues discussed are ground water used for drinking (problems include depletion, contamination and saline intrusion), and contaminated ground water (problems include migration of polluted ground water and disposal of extracted ground water).

**Disposal of Excavated Material** (Section 4). Immersed tunnel construction invariably involves a major dredging operation, and normally requires large dry excavations for approach structures. While reuse of dredged and excavated material within the works is always considered, disposal of large volumes of material is commonly necessary. Examples are given of how disposal has been effected in an environmentally sensitive manner for several completed tunnels, including some involving contaminated material.

**Land Use Consequences** (Section 5). Immersed tunnelling is sometimes viewed adversely in comparison with bored tunnelling because it can involve more (or at least more sensitive) landtake. Whilst construction can be environmentally damaging (for example, loss of shore habitats), immersed tunnelling can also afford opportunities for land use improvements. The section describes problems that typically arise in this area and how they are solved.

*Table 5-1. Key to case studies discussed in Chapter 5.*

| | Watercourse | | | | Groundwater | Disposal | | Land Use | |
|---|---|---|---|---|---|---|---|---|---|
| **Case Study** | **2.1** | **2.2** | **2.3** | **2.4** | **3.1** | **4.1** | **4.2** | **5.1** | **5.2** |
| Conwy | ♦ | | | | | ♦ | | | ♦ |
| Drogden | ♦ | | | | | | | | ♦ |
| Fort McHenry | | | ♦ | | | | ♦ | | |
| Fort Point Channel | | | | | | | | | ♦ |
| Lee | ♦ | ♦ | | | ♦ | | | ♦ | |
| Medway | | | | | ♦ | ♦ | | | ♦ |
| Noord | | | | | | | ♦ | | |
| Piet Hein | | | | | | | ♦ | | |
| Saltsjön | ♦ | | | | | | | | |
| Sydney | | | | | | | | | ♦ |
| Ted Williams, Boston | | | ♦ | ♦ | | | ♦ | | |
| Western Harbour Crossing | | | | | | | ♦ | | |
| Wijker | | | | | ♦ | | | | |

Key to sections:

| | | | |
|---|---|---|---|
| 2.1 | Changes in flow patterns | 3.1 | Depletion and pollution of aquifers |
| 2.2 | Changes in scour and siltation patterns | 4.1 | Disposal of uncontaminated material |
| 2.3 | Pollution of the watercourse | 4.2 | Disposal of contaminated material |
| 2.4 | Effects on wildlife | 5.1 | Land reclamation |
| | | 5.2 | Graving docks |

## 2. Effects on the Watercourse

### 2.1 Changes in Flow Patterns

Watercourses traversed by immersed tunnels include sea channels, harbours, rivers, river estuaries, and canals. This variety leads to a correspondingly diverse range of flow regimes. Some watercourses, such as canals and some rivers, are highly managed, while others will be in their natural state—quasi-stable, changing only with the slow processes of natural erosion and deposition.

During the feasibility study stage of an immersed tunnel project, it is usual practice to study the hydraulic regime of the watercourse in some detail to evaluate whether the proposed works will change the flow regime, and, if so, whether any such change will have detrimental effects.

#### 2.1.1 Lee Tunnel, Ireland

The Lee Tunnel carries a new ring road beneath the River Lee, some 4 km downstream of Cork City. Preliminary studies for the scheme included extensive computer modelling of the river hydrology. The analysis was calibrated using river monitoring data collected over several years. One purpose of the study was to determine whether the construction of the tunnel, combined with some associated land reclamation, would increase the risk of flooding in Cork City.

Flooding is rare in Cork, but severe events can occur in which high rainfall, leading to large freshwater flows, coincides with onshore winds and a surge tide. This can occasionally result in flooding of the city centre.

The proposed land reclamation covered some 15 ha of tidal mud flats, resulting in a 20% reduction in the river cross section at high tide (but no change at low tide). The modelling confirmed that the proposed works did not increase the risk of city centre flooding.

#### 2.1.2 Conwy Tunnel, UK

A hydraulic study was performed for the Conwy Tunnel. It began with a historical review, identifying long term trends in estuary topology. There followed (i) a hydrographic and hydraulic survey of the existing estuary, to provide a detailed benchmark for design; (ii) mathematical modelling of the estuary, to predict the consequences of tunnel construction; and (iii) an assessment of probable siltation patterns in the tunnel trench during construction.

The surveys established data on tidal ranges, salinity, current velocities, volume flows, bed material grading, and sediment transport behaviour. These data were used to calibrate the numerical models, which in turn provided detailed estimates of how the tunnel construction would influence currents, erosion, and sedimentation in the estuary.

#### 2.1.3 Saltsjön Tunnel, Sweden

For many immersed tunnels, it is possible to design a vertical alignment that places the entire tunnel below the original bed level. Any adverse effects on the watercourse are thus confined to the construction period. However, situations sometimes arise where raising the existing seabed permanently to accommodate the new crossing is desirable.

An example of this occurred with the feasibility study for the Saltsjön Crossing in Stockholm. The tunnel will form part of a new ring road for the city, most of which is to be constructed in rock tunnels. Saltsjön is the innermost inlet of the Baltic Sea, and presents a special obstacle to the ring road, in that an otherwise continuous rock stratum contains a geological fault at this location. As a result, the rockhead plunges locally to 80 m below sea level. The associated sea channel is approximately 40 m deep.

To cross the sea inlet, two immersed tunnel solutions were studied. One placed the tunnel conventionally, beneath the sea bed. The second raised the tunnel on stilts, to slacken the approach gradients in the adjacent rock tunnels.

A study was performed of the existing current regime in Saltsjön. The results revealed four currents flowing at different depths in the channel. They were shown to be interrelated. For example, the 'first' current is primarily a superficial freshwater discharge from Lake Mälar, upstream of Stockholm; but it also incorporates saline water from greater depths as it flows seaward. This causes a compensating upstream current (the 'second' current) at greater depth. A further complication is introduced by two major sewage outfalls which discharge into the Saltsjön upstream of the tunnel site. The present flow regime, which is in part driven by the outfalls, satisfactorily dilutes the sewage, and passes it out to sea.

The study showed that the raised tunnel solution would interfere directly with the deepest of the four currents, and that this in turn created a risk of disturbing the existing pattern of sewage dilution and removal from the Stockholm area. This consideration formed one of the main arguments in favour of the technically more difficult solution, placing the tunnel in a trench on the sea bed.

#### 2.1.4 Drogden Tunnel, Denmark

The Drogden Tunnel, currently under construction, forms part of a larger sea crossing between Copenhagen in Denmark and Malmö in Sweden. The link crosses Øresund, a 15-km wide sea channel that is one of only two openings from the almost landlocked Baltic into the North Sea.

The Baltic Sea is a unique environment, possessing a salinity lower than that of the oceans due not only to the quantity of fresh water flowing into it but also to the relatively constrained connections to the North Sea. Early studies for the crossing were inconclusive about whether modest changes in the overall cross section of the Øresund would affect the delicate balance in the Baltic. After much debate and further study, a decision was taken to adopt what became known as "the zero solution".

The zero solution permitted modest changes in the shape of the sea channel due to the construction of the link, but eliminated the reduction in cross sectional area that had previously been envisaged. To achieve this, it was necessary to:

- modify the outline design of the artificial island that connects the bridge portion of the link to the tunnel;
- modify the tunnel approach land reclamation on the Danish coast;
- further constrain the tunnel alignment; and
- introduce "compensation dredging".

Compensation dredging consists of artificially deepening the sea bed at selected locations to compensate for loss of channel cross section elsewhere. This proposition remains a source of debate today, since it is impossible to guarantee that the sea bed will not reestablish its original level.

### 2.2 Changes in Scour and Siltation Patterns

In some cases, such as at the Saltsjön and Drogden tunnels described above, the behaviour of the water currents themselves is of concern to the potential tunneller. In others, the indirect consequences of scour and siltation are of more interest.

#### 2.2.1 Lee Tunnel, Ireland

The Lee Tunnel, described in Section 2.1 above, crosses an important commercial waterway, and has to be dredged on a regular basis to maintain the channel depth. River features close to the tunnel site include a fast flowing tributary entering the main stream, a bend in the river and the entrance to Lough Mahon—a sea lough just downstream of the site.

Since the proposed works were expected to change the river cross section, the pre-contract hydraulic studies included a detailed consideration of the changes in scour and siltation patterns that would occur as a result of construction. The study showed that the works could be built without adversely affecting the siltation patterns if an appropriate design was selected. The results were used to determine the constraints to be imposed on the contractor regarding graving dock location and extent, land reclamation extent, and finished bed profiles.

### *2.3 Pollution of the Watercourse*

Immersed tunnel construction necessarily involves some interference with the watercourse being crossed, since a trench must be dredged, tunnel elements installed and backfill placed. Such activity is often perceived as a particular threat to the environment, occasionally leading to rejection of immersed tunnel solutions on these grounds. Experience, however, tells a different story. Techniques have been developed to permit immersed tunnel construction in widely varying conditions of watercourse and subsoil with minimal environmental impact.

#### 2.3.1 Ted Williams Tunnel, U.S.A.

Construction of the Ted Williams Tunnel, a 1200-m-long road tunnel across Boston Harbour, produced approximately 1 million $m^3$ of excavated material for disposal. About a third of this total consisted of uncontaminated dredged material that could be disposed of at sea. The chosen site was the Massachusetts Bay Disposal Site (MBDS), an established location for the disposal of dredged material some 30 km offshore from Boston.

Although the MBDS is a designated site for disposal of dredged material, it is nevertheless subject to stringent regulations regarding disposal method to be used and the quantity of suspended solids released into the surrounding waters. For the Ted Williams Tunnel project, compliance with these regulations was achieved through a state-of-the-art exercise in tracking, measuring and monitoring the plume arising from the disposal site. The data gathered were then analysed using a specialised comparison computer model that had been established for the site.

Ocean plume tracking was performed using two marine research vessels. One vessel tracked the plume using an Acoustic Doppler Current Profiler (ADCP) while the other performed in situ measurements and collected water samples. Both vessels were equipped with precision microwave navigation systems. The ADCP uses four acoustic beams and the Doppler effect to establish current velocity by measuring reflections from water-borne particles. Results give the vertical distribution of suspended sediments in the water column. The sampling vessel was equipped with conductivity-temperature-depth probes, a rosette sampler and a transmissometer system. Samples were collected at the densest part of the plume.

Tracking was performed by the two vessels working together behind the disposal barge. Following discharge, the two vessels made repeated transects normal to the plume. The tracking vessel identified the plume position using the ADCP equipment, thus permitting the sampling vessel to be correctly positioned. The process was repeated over a larger and larger area to measure the extension of the plume with time.

The method was highly successful. The ADCP identified the vertical distribution of the plume at any chosen transect, allowing accurate sampling to be performed. Four discharges were monitored over a period of four days, showing that the disposal operation would not violate marine water quality standards.

#### 2.3.2 Fort McHenry Tunnel, U.S.A.

Fort McHenry Tunnel traverses Baltimore Harbour, a harbour with a 200-year history of industrial activity that has resulted in significant contamination of the bay bottom. Consequently, the full 2.7 million $m^3$ of dredged material arising from the tunnel trench had to be specially treated for disposal.

The greater part of the solid contaminated material was ultimately disposed of in purpose built containment structures, but the treatment process necessarily created a liquid effluent that was discharged back into the harbour. Environmental restrictions required that this effluent be limited to a solid content not exceeding 400 parts per million, a standard achieved by a complex process utilising weirs, chemical flocculation and settlement basins. A fuller description of the treatment process is given in Section 4 below.

### *2.4 Effects on Wildlife*

All the phenomena so far described in this section can have adverse effects on water-borne plants and animals. This is one reason for taking measures to control effects on currents, pollution, etc. However, some construction activities can affect aquatic life more directly, and these have to be considered on a case by case basis. The starting point for such studies is a knowledge of the particular species that inhabit the waterway in question, and of their behaviour. Such data gathering is a routine part of feasibility study work for immersed tunnel schemes.

#### 2.4.1 Ted Williams Tunnel, U.S.A.

Certain fish species that live in the sea habitually migrate into freshwater to spawn. Because Boston Harbour is a migration route for several such species, the harbour authorities have imposed a moratorium on blasting in the harbour during May and June each year.

The Ted Williams Tunnel project required extensive underwater rock blasting to form the tunnel trench. Since the construction programme required blasting to take place through the spring and summer, stopping work during May and June would have been very undesirable. To avoid the need for such a stoppage, the contractor worked closely with the various environmental regulators to devise an alternative solution.

The basic proposal was to use state-of-the-art acoustic technology to locate schools of migrating fish and persuade them to leave the blast area. Location was achieved using sonar, and an electronic fish-startle system diverted the fish away from the dredging site.

To prove the efficacy of the system, a programme of testing and calibration was undertaken. This involved three phases:

1. Test blasting to establish how the pressure wave magnitude varied with distance from the blast;
2. A literature search, to estimate fish sensitivity to pressure waves;
3. A further test blast, using caged fish, to confirm their ability to survive at the calculated 'safe distance' from the blast.

The test programme, which also showed the reliability of the fish location and startle systems, provided sufficient evidence to satisfy the environmental authorities, and blasting was permitted to continue throughout the migration season.

### *2.5 Recommendations for Good Practice*

As soon as the possibility of an immersed tunnel crossing is established, tunnel planners should begin collecting data on the behaviour of the watercourse. The quality of future numerical modelling, and the confidence that can be placed in such work, is highly dependent on the quality of raw data available for calibration purposes.

In studying watercourse behaviour, problems associated with currents (usually complex, generally varying with depth), tides, variations in salinity, sediment transport, scour, and siltation should be considered. Some hydraulic studies should be performed early in the project; numerical modelling is usually more appropriate than physical modelling. Appropriate modelling can often make it possible to demonstrate that apparently significant changes in watercourse geometry have little adverse effect in practice.

Dredging, if properly managed, is less environmentally damaging than is commonly feared. Some disturbance is inevitable during dredging, though with proper control, the impact on the environment is seldom serious.

Disposal of dredged material can be a source of watercourse pollution, though techniques now exist to dispose of spoil cleanly and to measure the consequences for the watercourse accurately. Contracts should be drafted to strike a balance between control and cost.

Exaggerating the risk to wildlife from changes in the watercourse is quite possible. While some changes can be devastating, the more common experience is that aquatic wildlife populations will adapt to modest changes without adverse effect. This is particularly true of temporary habitat disturbance. The key to identifying the exceptions, i.e. the cases where populations would be genuinely threatened, is local knowledge. No two sites are comparable in this regard, and nothing can substitute for advice from specialists who know the local conditions.

## 3. Effects on the Groundwater Regime

One of the most attractive features of immersed tunnels, when compared with bored tunnels, is that they are generally completely or very nearly watertight on completion, with obvious direct benefits to the tunnel owner and users. In contrast, bored tunnels often leak sufficiently to require additional drainage, sometimes enough to cause permanent changes to the surrounding groundwater regime—a phenomenon unknown in immersed tunnels.

Temporary groundwater lowering, on the other hand, is a common feature of immersed tunnels. Occasionally, the extent is modest, though dry excavations are commonly required below ground water level over large areas, especially for concrete tunnels. These are usually necessary for approach structure construction, and sometimes for a tunnel element casting site. Ground water lowering produces many engineering problems, but it can also be detrimental to the environment, particularly if the ground water in question is especially valuable (e.g. an aquifer), or is contaminated.

### *3.1 Depletion and Pollution of Aquifers*

The consequences of dewatering a large excavation depend on several variables, the most important of which are the permeability of the ground, the size of groundwater reservoirs nearby, and the engineering techniques used to lower the water table. Selection of the best engineering solution is a question of finding a balance between the costs of pumping and the costs of physical barriers to water movement.

If the preferred solution is to pump large volumes of water without the use of cut-off structures, the question of ground water depletion becomes important. The effects of a major dewatering exercise can often be measured at considerable distances from the site. A further problem can arise in coastal sites due to fresh water aquifers becoming contaminated by saline intrusion because of temporary ground water lowering.

#### 3.1.1 Medway Tunnel, U.K.

The Medway Tunnel was constructed using an on-site casting basin located within one tunnel approach. The construction sequence thus required the basin to be dewatered twice - once for the construction of tunnel units, and once for construction of the approach works. Dewatering was necessary for an extended period, and to depths of 10 m below the natural water table. Ground conditions consisted of made ground, clay, and gravel overlying the chalk bedrock. The chalk stratum is an aquifer, and provides part of the local water supply.

Dewatering was achieved by pumping from deep wells around the perimeter of the basin. Pumping rates reached 40 Megalitres per day for a total of some 21 months. Drawdown was monitored extensively in the surrounding area, using both existing wells and newly installed observation wells. Even 2 km from the site, drawdown approaching half a metre could be observed. Despite this apparently large impact on the water table, the contract was completed without any adverse effects on the public abstraction wells. This was achieved by a combination of appropriate engineering, comprehensive monitoring, and careful liaison with the water authority.

Water quality was also monitored, to establish whether any salt water was entering the aquifer as a result of the dewatering. Some saline intrusion was measured, but the effect was sufficiently modest to be accepted by the National Rivers Authority (NRA). When the time came to decommission the dewatering system, detailed discussions were held with the NRA to arrive at a method that would not result in any permanent saline intrusion.

#### 3.1.2 Lee Tunnel, Ireland

The Lee Tunnel lies over an area of karstified limestone containing an aquifer used for drinking water. Pumping tests performed prior to construction showed that the limestone possessed high transmissivity combined with low storage capacity, thus creating a risk of saline intrusion in the event of major dewatering. While fresh water in the limestone appeared to be effectively separated from the saline river water by the impermeable river bed materials, guaranteeing that saline intrusion would not occur was not possible.

In preparing the contract, two concerns had to be addressed. First, should an on-site graving dock be proposed with the risk of saline intrusion into the aquifer needing to be controlled? This was dealt with by requiring extensive monitoring for saline intrusion using screened wells around the site perimeter.

The second problem was that the dredged trench for the tunnel would breach the natural barrier between the salt water of the river and the fresh water in the limestone. While this could not be avoided in the short term, the contractor was required to reinstate the bed such as to achieve an equivalent barrier after the tunnel was complete.

#### 3.1.3 Wijker Tunnel, The Netherlands

The Wijker Tunnel is a road tunnel of approximately 1 km length. The tunnel crosses the North Sea Canal to the west of Amsterdam. Major dewatered excavations were required on the tunnel site for the tunnel approaches and at a remote location for a graving dock. The excavations were maintained in a dry condition by using deep wells. This technique is capable of dewatering very large excavations to significant depths, but can sometimes result in unnecessarily deep ground water lowering over parts of the site.

To mitigate this effect for the Wijker Tunnel, a technique of ground water recycling was adopted. Most of the wells at each excavation site were used to draw water from the ground, but at each location, a proportion were used to return water to the ground. Careful positioning of the extraction and return wells allows greater control of the lowered water table, with reduced total abstraction. At the Wijker Tunnel site, this technique eliminated the risk of dewatering causing unacceptable foundation movements in nearby buildings.

The rates of water abstraction and return at this site were 1200 $m^3$/hour and 200 $m^3$/hour respectively for the graving dock and 3600 $m^3$/hour and 1200 $m^3$/hour respectively for the approach works.

### *3.2 Recommendations for Good Practice*

Groundwater lowering can have effects over a wide area, though typically this causes no particular problems, and nothing more than careful monitoring is needed. If effects are troublesome (problems include depletion or pollution of water supplies and settlement of nearby buildings), engineering can be modified to mitigate the effects. Solutions include use of cut-offs (walls, slurry walls, grout curtains etc.), selective recirculation of groundwater and reducing the need to dewater (e.g. by dewatering in stages or by modifying the permanent works).

Care must be taken not just during dewatering, but also when the dewatering system is decommissioned. If not done in a controlled manner, decommissioning could lead (for example) to saline intrusion.

## 4. Disposal of Excavated Material

Disposal of surplus or unsuitable material arising from dredging probably constitutes the biggest single environmental challenge facing the promoter of an immersed tunnel scheme. Many proposed tunnels cross sites where the existing sea or river bottom has become contaminated, most commonly by a build up over time of industrial pollutants. Such sites present the most difficult challenges. However, even uncontaminated dredging spoil presents significant environmental questions, particularly where large volumes of surplus material arise but disposal sites are scarce.

### *4.1 Disposal of Uncontaminated Material*

A typical immersed tunnel—say, 1 km in length, and carrying 2 x 2 lanes of traffic—will create about 1 to 2 million $m^3$ of dredging spoil. Some of the material can sometimes be reused as backfill, though the nature of the material or difficulties over temporary storage can preclude this. Usually a considerable volume of dredged material has to be disposed of off-site.

The final choice of disposal site is usually a decision for the contractor, but it is common practice for the scheme promoter to restrict the choice of disposal location for environmental reasons. Such restrictions will inevitably have a cost impact for the project, so careful study is needed during the pre-contract stage to find the optimum balance between environmental impact and cost.

#### 4.1.1 Medway Tunnel, U.K.

The site of the Medway Tunnel is in the upper reaches of the Medway estuary, some 15 km from its confluence with the Thames estuary. A large part of the estuary downstream of the tunnel site is a Site of Special Scientific Interest (SSSI) due to its importance to local bird populations. While the tunnel contract was being prepared, English Nature, the authority responsible for the SSSI, were applying to extend its boundaries and to have it recognised both as a Special Protection Area under European Union rules, and as a Wetland of International Importance under the Ramsar Convention.

Due to the importance of the estuary to wildlife, the area contained within the SSSI was precluded as a dredging disposal site. Further negotiations were necessary between the tunnel promoter, English Nature and the Royal Society for the Protection of Birds to resolve the status of two sites adjacent to the SSSI but outside its original boundary.

Consideration was given by the promoter to defining a particular disposal site in the contract, and obtaining all the necessary permissions prior to contract award. However, this was viewed as unnecessarily restrictive. Finally, a contract was drafted leaving the contractor free to choose his own disposal site subject to the particular exclusions discussed above. Disposal at sea was technically feasible but expensive, and the successful contractor ultimately opted for land disposal some distance downstream from the site.

#### 4.1.2 Conwy Tunnel, U.K.

Planning for the Conwy Tunnel indicated that about 1.5 million $m^3$ of dredged material would have to be disposed of. Off-site disposal options were perceived to be difficult and costly, so a decision was taken to provide the main works contractor with a disposal area close to the site. No immediately suitable site was available, so an advance works contract was let to construct a containment bund within the estuarine mudflats, a little upstream of the tunnel site. As the bund separated the mudflats from the main estuary, a suitable site was formed for hydraulic disposal of material excavated by cutter-suction dredger.

The site provided was used successfully by the main works contractor for both disposal of unsuitable material, and for storage of reusable materials. After the tunnel was completed, the backfilled area was converted into a new nature reserve to support local populations of wading birds.

### *4.2 Disposal of Contaminated Material*

Many potential sites for immersed tunnels have a history of commercial or industrial activity, often resulting in the upper layers of the watercourse bed being polluted, sometimes quite severely. This presents the immersed tunnel engineer with an additional problem, since the materials that must be removed as part of the dredging process are then usually subject to very stringent disposal criteria.

Disposal criteria vary from country to country, and from site to site. Typically, samples will be taken from the tunnel site and subjected to testing to establish the degree of contamination and its nature. These results will be used to establish precisely how the material can be disposed of. For example, lightly contaminated material is normally acceptable for backfill and reclamation, but only in designated areas, whereas more heavily contaminated material must be treated before disposal.

Disposal of contaminated material invariably presents a problem for the immersed tunnel promoter. Nevertheless, experience shows that viable technical solutions can usually be found, and that the politics of pollution can often be more of a problem than the engineering.

#### 4.2.1 Piet Hein Tunnel, The Netherlands

The Piet Hein Tunnel forms part of a new road link in Amsterdam, traversing the old Eastern Docks area of the city. The immersed tunnel is 1260 m long and crosses the Amsterdam-Rijn Canal in an area known as the "Railway Dock."

Before construction, the Railway Dock was between 7 m and 12 m deep. The upper layer of the dock bed was lightly contaminated with hydrocarbons associated with the unloading of coal, and with heavy metals. Most of the material was suitable for restricted reuse (not in residential or most commercial areas).

Local policy when dredging contaminated material was to minimise transportation and if possible, deposit the material locally in an area already similarly contaminated but not needing to be excavated. This led to a preferred solution of local deposition, in two discrete but adjacent sites.

The subsequent scheme history shows how technical, environmental and legal considerations can become intertwined. Separate permit applications were prepared for the two sites and submitted to the relevant authorities. The first stumbling block was a new regulation that automatically required an Environmental Impact Study (EIS) for all depositions of contaminated material exceeding 500,000 $m^3$ in volume. While neither of the two sites alone exceeded this figure, the two taken together did so, and the application was set aside pending an EIS.

Further technical studies were carried out, and a second proposal was put forward. This reduced the deposition volume to 475,000 $m^3$ by excluding any temporary storage at the deposition site. Thus, the EIS was avoided even though the final volume of material permanently deposited at the site remained unchanged. This second proposal had significant technical disadvantages however, since temporary storage had to be achieved on site, which was only possible using an elaborate staged construction scheme.

This proposal was accepted by the permitting authorities, subject to several technical constraints. The existing surface mud layer in the deposition area had to be sealed prior to deposition, the completed deposition also had to be sealed, and the whole area had to be confined within underwater berms. In summary, a technically feasible but much more expensive solution had been devised in order to deal with the very onerous consequences of a new planning regulation.

Finally, a contract was awarded based on the new expensive deposition scheme. However, time had passed since the original planning application. A second application was submitted by the contractor along the lines of the original withdrawn scheme. On this occasion, the permit was granted without an EIS being required.

### 4.2.2 Fort McHenry Tunnel, U.S.A.

Dealing with contaminated dredging spoil presented one of the central technical problems in the construction of the Fort McHenry Tunnel. The tunnel, completed in 1985, crosses Baltimore Harbour. Extensive contaminated sediments in the bay bottom have resulted from 200 years of industrial activity in the harbour area.

Initial studies for the tunnel showed the presence of the contaminated material. Removal to a controlled landfill site was the obvious solution, but a more imaginative, and ultimately more environmentally satisfactory proposal was developed. This used the contaminated dredging spoil for the construction of a new port facility within Baltimore Harbour.

Substantial environmental impact studies were carried out prior to construction, and it was established that treatment of the dredged material would be necessary before its incorporation in the port facility. The chosen solution required the construction of a permanent containment structure to hold the treated spoil, together with a treatment plant to separate the contaminated solid material from the liquid effluent that was to be returned to the harbour.

Some 2.7 million $m^3$ of material was to be dredged from the harbour. To allow for bulking, the containment structure had to have a capacity approaching 4 million $m^3$. The cellular cofferdams used provided both the required barrier between the disposal area and the harbour, and the necessary strength to contain the completed landfill and a design surcharge load of 50 kPa. The cofferdam enclosed a water area of some 60 ha.

Cutter-suction dredging was used, and dredged material was transported to the disposal site by pipeline. The site did not provide sufficient area for sedimentation to take place solely in settlement ponds. This problem was overcome by a four-stage process. Dredged material was first passed over weirs, which removed most of the solid material. The remaining liquid effluent was then chemically treated and subjected to flocculation. Finally, it passed through a conventional settling basin before discharge back into the harbour. The effluent was limited to a solid concentration not exceeding 400 parts per million.

This was the first project to adopt this treatment process for dredged material, so that design had to depend on theoretical considerations and extrapolations from previous experience.

### 4.2.3 Tunnel de Noord, The Netherlands

The 500-m-long Tunnel de Noord carries a major highway under the River de Noord, a tidal river near Rotterdam. Early studies indicated that the tunnel excavation would encounter some contaminated material, although this was not expected to be extensive. The decision was taken not to perform any pre-contract contamination surveys. In recognition of this, the contract placed responsibility for the extra cost of handling contaminated material with the client.

Early in the contract, the client performed a survey that identified contamination in the surface layer over most of the site. It also revealed a local area of deeper contamination at the site of an old harbour entrance. At that location, polyaromatic hydrocarbons were found to depths of 5 m.

A total of about 100,000 $m^3$ of contaminated material had to be removed. This was achieved using a closed grab loading into spoil barges. The barges were not allowed to be filled within 0.5 m of the top of the bin. The material was then transported some 50 km to a registered disposal site.

### 4.2.4 Ted Williams Tunnel, U.S.A.

Most dredged material from the Ted Williams Tunnel was clean soil and rock, and was disposed of at sea (this process is described elsewhere in this chapter). However, the harbour in Boston also contained a typical, thin layer of contaminated material on the sea bed. This material required special treatment.

The contaminated material was deposited in a purpose-built disposal pond. The pond was contained within dykes and sealed with two plastic liners separated by a gravel layer. Controlled drainage of the gravel layer was thereby possible, trapping any seepage through the first liner. Seepage thus collected was trucked to suitable disposal facilities.

### 4.2.5 Western Harbour Crossing, Hong Kong

Western Harbour Crossing in Hong Kong is a highway tunnel, the immersed portion of which is about 1350 m long. The harbour is polluted, with heavy metals present to some extent near the surface. Contaminated material was removed by grab dredger and transported by barge to dumping areas allocated by the Hong Kong government.

## *4.3 Recommendations for Good Practice*

Where dredging spoil is expected to be uncontaminated, every effort should be made at contract preparation stage to allow the contractor reasonable freedom regarding disposal. Excessive controls can lead to unnecessary costs for the scheme promoter with little or no corresponding environmental benefit. Occasionally, obtaining permits for particular disposal sites may be worthwhile for the promoter (for example where the time necessary to obtain them would delay the contract). In other cases, allowing the contractor to choose a site himself may be better. Above all, blanket constraints should be avoided unless they are very soundly based.

If dredging is expected to encounter contaminated material, sufficient site investigations at the pre-contract stage must be performed to find out the type and extent of the material. An understanding of local regulations is essential, as is early investigation of the options for disposal. It is necessary to distinguish between different classes of contaminated material. Because this is often as much a political as an engineering matter, planners must aware of possible scheme delays in dealing with permits, etc.

## 5. Land Use

The construction of tunnel approaches invariably involves significant landtake, whatever the primary method of tunnel construction. For immersed tunnels, this landtake often includes environmentally sensitive coastal or riverside habitats, calling for extra care at the preliminary design stage. However, as well as presenting challenges, immersed tunnels can offer land use opportunities. Sometimes, an immersed tunnel scheme can incorporate some beneficial land reclamation, which without tunnel construction would have been either uneconomic or technically infeasible. The immersed tunneller's art therefore includes consideration of the optimum use of land in the tunnel approaches.

### *5.1 Land Reclamation*

It is common for an immersed tunnel scheme to include some reclamation of land from the river or sea channel in the tunnel approaches. Sometimes this will be for reasons only of cost, whereas in others, the effect on cost might be neutral, with other factors determining the nature and extent of land reclamation. Reclamation of land always has environmental effects that must be taken into account in scheme design and construction.

#### 5.1.1 Lee Tunnel, Ireland

The Lee Tunnel crosses the River Lee some 4 km downstream of Cork City and about 15 km from the sea. The river is tidal at this location. Hydraulic studies during the pre-contract phase showed that an area of about 15 ha of mudflats on the south shore could be reclaimed without an adverse impact on flow patterns, scour or siltation. This was possible because the effect of the reclamation was to smooth out a kink in the shoreline, rather than intrude into the main river channel. (At low tide, the river channel was unaffected by the reclamation, whereas at high tide, the cross section of the channel was reduced by some 20%.)

Besides determining the effects of the proposal on the river hydraulics, pre-contract studies considered the importance of the mud flats to local bird populations, and the potential benefits of land reclamation to local residents. These latter studies included the realisation of additional amenity land and the removal of the tunnel portal to a location more remote from residential properties.

Overall, the environmental effects of reclaiming this land were considered broadly neutral (benefits to residents being balanced by loss of wildlife habitats) so economic considerations were allowed to dominate. The contract set a maximum limit on land reclamation, but left the contractor free to decide on the extent. In practice, some land was reclaimed, but the extent was less than the maximum permitted by the contract.

### *5.2 Graving Docks*

One problem with immersed tunnelling is the need for a large temporary site for tunnel element construction. For concrete tunnels, this normally requires a major excavation below water level, typically for one to two years. This consideration can add significantly to the environmental impact of an immersed tunnel scheme, especially if possible graving dock locations near the tunnel site are environmentally sensitive.

The best solution to the graving dock question is a site-specific matter. In a few cases, a trouble-free site is available at reasonable cost near to the tunnel site. But often the decision is more difficult, leading to more imaginative solutions. These have included remote construction, use of existing dry docks, reuse of an established graving dock and establishing a secondary use for a graving dock once tunnel construction is complete.

#### 5.2.1 Sydney Harbour Tunnel, Australia

The Sydney Harbour Tunnel carries a major highway beneath the spectacular harbour scenery, in the shadow of the world famous Sydney Opera House. The road alignment and tunnel site were selected following an exhaustive study of options, each with different consequences in terms of cost, benefit to motorists, and environmental impact. One major problem with the chosen site was the absence of a suitable location in the vicinity in which to construct a graving dock. The adopted solution was to construct the tunnel elements some 100 km from Sydney. This was more demanding than local construction, since the elements had to be designed to withstand a long sea tow in the large swells of the Pacific. However, it did remove the difficulty of finding an appropriate graving dock site in the sensitive Sydney Harbour area.

#### 5.2.2 Medway Tunnel, U.K.

The Medway Tunnel is located on a very constrained site, close to some historically important disused dock buildings. As a result, very limited scope existed for construction of an on-site graving dock. Tenders were invited for design and construction of the tunnel. The tender documents made clear the limited land availability to the tenderers, but left them free to find their own solutions.

Most of the tenderers opted for remote construction, with one proposal requiring transport of tunnel elements across the North Sea. However, remote construction here would have required significant additional dredging of the River Medway to allow the tunnel elements access to the construction site. Although not decisive in selecting the contractor, this was a consideration at tender stage. The successful contractor solved the problem by constructing his graving dock on the line of the tunnel approach. Having constructed the tunnel elements in a dewatered excavation, it was then necessary to flood the dock to permit immersion, then dewater again for the purposes of approach works construction.

#### 5.2.3 Conwy Tunnel, U.K.

The Conwy Tunnel utilised a traditional graving dock, immediately beside the tunnel alignment, so minimising construction difficulties. The original scheme required that the graving dock be reinstated after construction, since this was perceived to minimise the overall environmental impact. However, during the contract, a proposal was developed to convert the graving dock into a marina after construction. This proposal was eventually adopted, resulting in considerable benefits to users of the estuary, and an attractive addition to the surroundings of the historic town of Conwy.

#### 5.2.4 Drogden Tunnel, Denmark

The Drogden Tunnel is exceptionally large, consisting of 20 tunnel elements each 175 m long and 40 m wide. During contract preparation, it was considered likely that a very large graving dock would be needed near the tunnel site (probably large enough to construct one third of the tunnel at a time). Preliminary studies were performed to identify possible locations for such a facility. When the contract was put out to tender, tenderers were made aware of these studies but left free to choose their own construction site.

The successful contractor devised a scheme that combined local construction within a disused dock area with minimum environmental impact.' The proposal involves constructing tunnel elements above sea level in pairs and jacking them from the construction location to an adjacent area within which water can be impounded to a depth of 10 m. The completed elements can then be floated off the ground and into a deeper part of the dock. Finally, the impounded water is drained out, leaving the element floating in water at sea level.

The benefits of this scheme include: fast construction (construction does not stop whilst elements are floated); minimum landtake; no major long term dewatering and relatively little excavation. In addition, once the tunnel is completed, the site will be suitable for conversion into a new dock facility.

### 5.2.5 Fort Point Channel Tunnel, U.S.A.

Also in Boston, close to the Ted Williams Tunnel and along the same highway, the Fort Point Channel Tunnel is being constructed in an extremely constrained site, above an existing transit tunnel, close to bridges and between highly industrialised shorelines. Lack of space for a graving dock will be overcome by using one tunnel approach in which to fabricate elements in two cycles of three tunnel elements per cycle.

### 5.2.6 The Netherlands

Nowhere in the world are immersed tunnels more numerous than in the Netherlands. Because of the major waterways that traverse the country and the high population densities, pressure is maintained for more tunnels. However, the construction of new graving docks has become increasingly unpopular. This is particularly true in urban sites, where contamination of the sea or river bed means that dredging and excavation both need to be kept to an absolute minimum.

Dutch engineers have addressed this problem by establishing some semi-permanent graving docks, which are available for repeated use. Sometimes a single dock has been in use for the construction of two tunnels simultaneously. This approach has created some considerable engineering challenges for tunnel transport in restricted waterways, and is successfully limiting the environmental impact of new immersed tunnel schemes.

### 5.2.7 Steel tunnels in the U.S.A.

American immersed tunnels have traditionally been of steel construction, which permits remote fabrication of the steel shell, with concrete being added on site while the tunnel element is floating. This precludes the need for a graving dock. Examples include the Ted Williams Tunnel in Boston, for which the elements were fabricated 600 km away at Baltimore, and the Second Elizabeth River Tunnel in Virginia, for which the fabrication site was in Texas, some 4,000 km away.

## *5.3 Recommendations for Good Practice*

Local knowledge is paramount. Planners of immersed tunnels must know about all current and planned land uses. They must understand in particular specialist interests—for example, relating to wildlife. Recognition that modest realignment, or use of retaining structures to control landtake, can often overcome otherwise unsolvable problems.

It is essential to recognise the land use opportunity that an immersed tunnel presents. During construction of an immersed tunnel, land can often be reclaimed for no additional cost. In some cases, the inclusion of land reclamation in a scheme can overcome problems with balancing excavation quantities with fill. Numerical modelling of the waterway will generally be necessary to predict the effects of any land reclamation.

Imaginative solutions to the graving dock problem offer some of the best opportunities to limit environmental impact. It is important to consider all the options and to allow contractors the freedom to build remotely. River or sea transport of elements need not be feared: while costly, transport does not present insurmountable technical difficulties in most cases.

Finally, planners should be aware that local politics can create pressure for local construction, even if it is not in the best interests of the environment.

PII:S0886-7798(97)00010-2

# Chapter 6

# HAZARD ANALYSIS

by

JAN SAVEUR | The Netherlands
Volker Stevin Construction Europe bv

*Additional text by:*

Silas Li | U.S.A.
Parsons Brinckerhoff Quade & Douglas, Inc.

*Contributions and comments by:*

Christian Ingerslev | U.S.A.
Parsons Brinckerhoff Quade & Douglas, Inc.

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.135-144, 1997

Printed in Great Britain
0886-7798/97 $17.00+0.00

Pergamon

# Chapter 6: Hazard Analysis


**1. INTRODUCTION**

**2. INTERNAL FLOODING**

**3. SUNKEN SHIP LOADING**
3.1 Introduction
3.2 Properties of Immersed Tunnel Structures
3.3 Properties of Sunken or Stranded Ships
3.4 Calculation of Ground Loads of Sunken or Stranded Ships
3.5 Calculation of Ground Load for Partly Sunken Ships Grounded at One End
3.6 Calculation of Bed Pressures from Stranded Ships
3.7 Loading Effects on the Tunnel Structure
3.8 Longitudinal Performance of the Tunnel Structure
3.9 Conclusion

**4. DROPPING ANCHORS**
4.1 Impact Loads Directly on the Concrete
4.2 Impact Load with Granular Roof Protection Layer

**5. DRAGGING ANCHORS**

**6. LIFE SAFETY, FIRE AND EXPLOSION**

**7. HAZARD ANALYSIS**

## 1. Introduction

This chapter describes the aftereffects of selected major accidents and hazard scenarios on immersed tunnel structures. These events include accidental loadings such as internal flooding, sunken ships, dropping and dragging anchors; and hazard scenarios such as fire and explosion, which are the major hazards that can occur. Effects of fire and explosion are not unique to immersed tunnels and are widely described in literature and national codes. The section on fire and explosion (Section 6) in this review is therefore limited to immersed tunnels. Other items are less important from the point of view of probability, although available literature about these hazards is scarce.

## 2. Internal Flooding

It is recommended that immersed tunnels be designed to maintain integrity for accidental internal flooding. There have been several known occurrences of flooding of one tube of a multiple tube tunnel.

In the transverse direction after flooding, external walls or slabs would lose pressure due to external hydrostatic loading. The reverse would be true for an internal wall or slab. It is recommended that the internal components be designed to resist the resulting loads at ultimate strength.

Another consequence of internal flooding is that the increased weight of the tunnel because of the water it contains may cause settlement and damage to the joints between the tunnel elements, especially at the terminal joints.

It is recommended that the likely effects of the aftermath of an internal flooding incident be considered and the possibility for repair be allowed for in the design of an immersed tunnel.

## 3. Sunken Ship Loading

### *3.1 Introduction*

Most immersed tunnels cross beneath waterways used for shipping. Although records show no instances of any major ship sinking onto an immersed tunnel, many smaller vessels have either sunk or stranded on immersed tunnels. Nonetheless, the possibility of a major ship sinking or stranding on an immersed tunnel cannot be ignored and should therefore be considered in the design as an accidental load. This implies that the tunnel structure should resist the load with a load factor of unity, just meeting the ultimate structural resistance.

The purpose of this review is to provide an understanding of critical accidental loading effects on immersed tunnel structures caused by sunken or stranded ships. This is not a guideline. However, by combining a proper understanding of the mechanics explained here with a knowledge of specific project conditions, safe design criteria for these events may be derived. For many projects, equivalent loads can be derived directly from a worst-case event. Occasionally a probability analysis may have to be carried out based on survival criteria.

### *3.2 Properties of Immersed Tunnel Structures*

Immersed tunnels that cross beneath waterways are usually embedded in the ground, with only a metre or two of ground cover, making them essentially directly exposed to grounded ship loads. The underkeel clearance of immersed tunnels in waterways used for shipping is usually just sufficient to allow for the cover and the minimum clearance for ships with the expected deepest draft. Because ships likely to sink on a tunnel would usually be able to sink only a few metres, the probability of big ships stranding should also be considered. It therefore may be more logical to consider ships grounding or stranding, rather than sunken ships.

Structurally, the external *shell* of an immersed tunnels is strong in the transverse direction. Because the shell is designed to resist permanent load and water pressures at moderate stresses, it has the capacity to resist additional accidental loads at a higher stress level.

Under permanent loading, immersed tunnels have relatively low foundation pressures, around 10 kN/m$^2$. Sunken ship loads could result in local foundation pressures that are much higher, and this would also effect the longitudinal performance of the structure.

### *3.3 Properties of Sunken or Stranded Ships*

When a ship is afloat, its weight is supported by its buoyancy, equal to the product of the volume of water displaced by the ship and the density of the water. If the weight of a ship changes, e.g., by water flooding into the hull, the ship will need additional displacement and therefore a deeper draft. After a ship is submerged, the displacement remains constant; the ship will continue to sink as long as the buoyancy is less than the weight. When a vessel rests on the bottom, the support provided by the ground is equal to the weight of the ship less its buoyancy at that time.

Compared to immersed tunnel structures, ships have thin shells that are sensitive to concentrated loads. The hulls of cargo ships are divided into compartments by intermediate bulkheads. The ship's cargo typically is distributed over the length of the ship, in order to keep the keel even and to minimize bending of the hull. Although the flooding of one compartment should not lead directly to sinking of the ship, it could make the ship heel. The combined action of other factors such as heavy seas and shifting of cargo could cause more flooding, which in turn could result either in heavy listing and capsizing, or in more heel and partial submersion. When the ship is partially submerged, the higher water pressures may cause the collapse of bulkheads. Unfavourable weight distribution over the hull may need to rupture the hull before the vessel will sink.

However, at the location of an immersed tunnel, a heeling or sinking ship would soon touch the ground and be stabilized by the ground. Capsizing could not occur, except for very small ships and river craft. Factors likely to aggravate the condition of a damaged ship in the open sea are less likely to be present. The probability is therefore low that a ship above a tunnel would flood sufficiently to need support from the ground beneath; and the probability is high that the magnitude of such support would be small, if needed at all.

For immersed tunnels in deep sea straits, a sunken ship event could be worse than such an event in a port area or in a shallow sea strait, because the probability of complete flooding is higher.

Another possibility is that instead of sinking, a ship might strand. This could be the case for a vessel with excessive draft, for a ship that loses control and strays beyond the fairway into shallower water, or for a ship that is drifting with increasing draft because of internal flooding. Although the initial ground support in such cases is theoretically nil, if the tide or water levels were to fall after stranding, the subsequent loss of buoyancy would cause ground pressures that exceed those for sunken ships that are fully submerged.

### *3.4 Calculation of Ground Loads of Sunken or Stranded Ships*

The load exerted on the bed of the waterway by a sunken or stranded ship is the difference between the weight of the ship and its buoyancy, sometimes called the *negative buoyancy*. A ship settles lower in the water when its weight is increased by water flooding into the hull. When a ship is both fully submerged and fully flooded without air pockets,

Table 6-1. *Characteristics of the two reference ships.*

| Reference Ship | Large Bulk Carrier | Large Freighter |
|---|---|---|
| Deadweight (DWT) in tons | 70,000 | 15,000 |
| Length b.p. (l) in m | 215 | 150 |
| Beam (b) in m | 32.2 | 20.0 |
| Hull depth (h) in m | 19.0 | 13.5 |
| Design draft (d) in m | 13.0 | 9.5 |
| Block coefficient | 0.9 | 0.7 |
| Self weight ($W_{ship}$) in tons | 11,500 | 5,000 |
| Displacement ($B_0$) in tons | 81,500 | 20,000 |
| Number of compartments | 8 | 6 |
| Flat keel area (A) in $m^2$ | 5,000 | 1,800 |

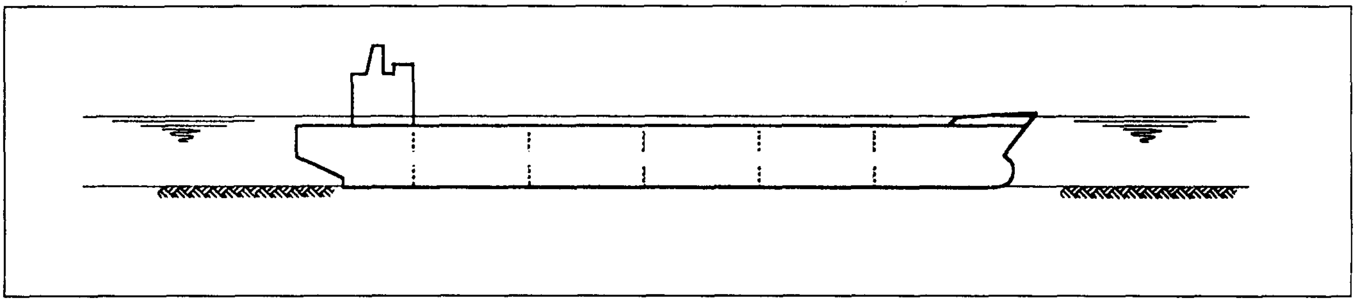


*Figure 6-1. Fully submerged and grounded ship.*

the buoyancy is equal to the weight of water displaced by the hull and other materials that make up the ship plus the weight of water displaced by the cargo and supplies. In other words, the load applied to the bed is equal to the submerged weight of the materials composing the ship and cargo.

Supplies such as fuel and water are certainly not heavier than water and are therefore usually ignored. Most types of cargo have a density below that of water and would reduce the ground load were they to remain in the ship after sinking. This applies to coal, crude oil and fuel, most liquid products, all agricultural products, and wood; to crated and containerized goods, because of the air locked within them; and to passenger ships and car carriers. Thus, the maximum bed loading of these ships, when fully submerged and completely flooded, is rarely more than the buoyant weight of the material composing these ships.

The only types of cargo with material densities greater than water are mineral ores and stones. These are usually transported in large bulk carriers. Steel products, despite their much greater density, are transported in mixed cargo freighters that are substantially smaller than bulk carriers.

Large crude oil tankers are not considered because they do not easily sink. Bed loadings are compared below for a large bulk carrier of 70,000 tons DWT and a large mixed cargo freighter of 15,000 tons DWT. Both normal cargo and highest-density cargo are considered. The characteristics of the two reference ships are given in Table 6-1.

The theoretical ground load calculations below are based on the theoretical assumptions given below. The total ground load is calculated and the average ground pressure $q$ is then derived as follows:

- The ship is fully submerged, fully flooded and sitting on the ground over the whole of the flat part of the keel (Fig. 6-1).
- For the sake of simplicity, water density is 10 kN/m$^3$.
- Material density of the ship is 70 kN/m$^3$.
- Material density of "light" cargo and supplies is 10 kN/m$^3$.
- Material density of iron ore is 30 kN/m$^3$.
- Material density of steel products 70 kN/m$^3$.

Table 6-2. *Theoretical ground load calculations.*

| | Large bulk carrier | Large freighter |
|---|---|---|
| $V_{empty\ ship} = W_{ship} \times (70-10)/70$ | 100,000 kN | 43,000 kN |
| $q_{e,\ empty\ ship} = V/A$ | 20 kN/m$^2$ | 24 kN/m$^2$ |
| $V_{c,\ \text{"normal cargo"}}$ | – | – |
| $V_{c,\ iron\ ore}$ :$0.9 \times 700{,}000\ (30-10)/30$ | 420,000 kN | n.a. |
| $V_{c,\ steel}$ : $0.9 \times 15{,}000\ (70-10)/70$ | n.a. | 115,700 kN |
| $q_c = V_c/A$ | 83 kN/m$^2$ | 64 kN/m$^2$ |
| Total ground pressure ($q_e$+$q_c$) "normal cargo" | 20 kN/m$^2$ | 24 kN/m$^2$ |
| Total ground pressure "heavy cargo" | 103 kN/m$^2$ | 88 kN/m$^2$ |

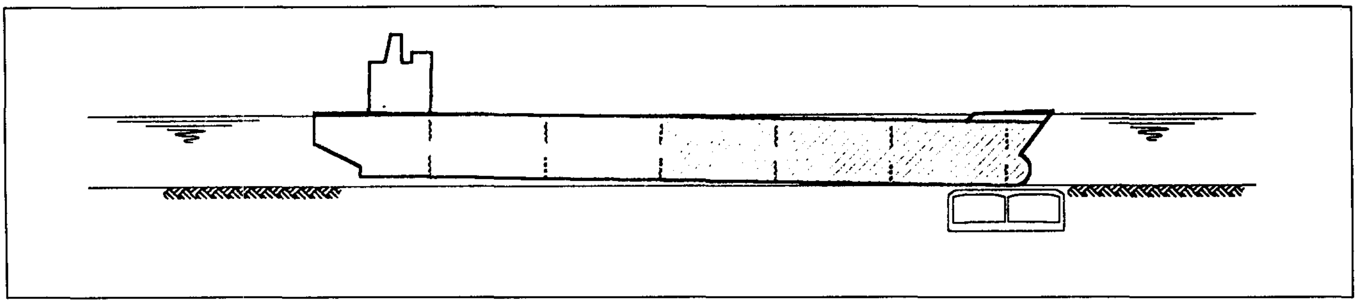


*Figure 6-2. Partially submerged ship, grounded at one end.*

- Cargo weight is 90% of DWT. 1 long ton as used for DWT is assumed to be 10 kN.

The calculations for a large bulk carrier and a large freighter are shown in Table 6-2. This calculation shows that smaller ships, notwithstanding the higher density of their cargo, have smaller maximum ground pressures. The calculated pressures are given by way of example only. The situation assumed above is highly unlikely to occur at the location of an immersed tunnel because of the limited water depth usually present at such locations.

### 3.5 Calculation of Ground Load for Partly Sunken Ships Grounded at One End

With sufficient water depth available, a ship could theoretically be stable with one-sixth of the length of the hull fully flooded and the flooded end just submerged. Were it to touch the bed (Fig. 6-2), additional flooding would be supported partly by increases in bed reaction at the low end, and partly by positive buoyancy provided by those parts of the ship not yet flooded. The magnitude of the bed reaction will be at maximum when flooding has progressed sufficiently for the far end to be on the point of submerging. Once that point has passed, the whole of the ship would settle onto the bed, providing that the hull does not break before then.

To simplify the calculation, the buoyancy of the superstructure may be ignored. In this example, the hull is assumed to be rectangular with constant horizontal cross-section $A_0 = 0.9 \text{ b} \times \text{l}$. Hence, the surplus buoyancy is equal to that cross-section times the original freeboard: $B = f \times A_0$. The flooded compartments are assumed to be fully flooded with a weight of water (W), and the flooding progresses from the compartment at the low end to the neighbouring compartment and so on over a distance (X). The maximum value of the end ground support (R) is the difference between the upward resultant of the submerged displacement and the weight of the ship including its cargo and supplies. This difference is equal to (B). The moment of (B) around the end support must be equal and opposite to the moment of (W) around that point.

The weight of water that can flood into the ship per metre length (w = W/l) is equal to the total displacement of that section minus the displacement of the material of the ship and the cargo. For the bulk carrier above, with iron ore:

$$w = 0.9 \times b \times h \times 10 - (W_{ship} \times 10/70 - W_{supplies} - W_{ore} \times 10/30) \times 10/l$$
$$= 5506 - (76 + 325 + 976)$$
$$= 4128 \text{ kN/m.}$$

Moment around end support:

$$X^2/2 \times w = B \times {}^1/_2\, l$$
$$= 373{,}842 \times 215/2$$

Hence, $X = 134.4$ m (5 compartments flooded)

and
$$R = w \times X - B$$
$$= 554{,}803 - 373{,}842$$
$$= 180{,}961 \text{ kN}$$

In this case, however, the bending of the hull would be beyond its breaking strength. If only four compartments were fully flooded, the hull could just resist breaking. A part of the high end of the ship would then not be submerged. For that case, $R_{max} = 146{,}280$ kN.

For the above calculations, the end support was assumed to be a line load at the extreme end of the ship. If, on the other hand, this load is distributed over an area, and if the center of this end support area is assumed to lie 10 m from the end, $R_{max} = 126{,}890$ kN.

For the same bulk carrier loaded with "normal cargo," $R_{max} = 26{,}700$ kN.

For the mixed cargo freighter above, $R_{max} = 31{,}500$ kN when loaded with steel products, and $R_{max} = 17{,}500$ kN with "normal" cargo.

### 3.5 Calculation of Bed Pressures from Stranded Ships

Were the tide or water levels to fall following a stranding, increased bed pressures would compensate for the consequential loss of buoyancy. This phenomenon is independent of the type of cargo carried by the stranded ship. For the bulk carrier above, assuming that it is resting on the whole of the keel area, the ground pressure after a tidal drop (t) would be:

$$Q_s = 10 \times t \times A_0/A$$
$$= 12.5 \times t \text{ kN/m}^2$$

Similar results would apply to a large crude oil tanker.

If a heeling ship were to strand, the free high end of the hull would move up and down with the tide. The ground support reaction at the stranded end would become equal to 1/4 of the loss of buoyancy by the tidal drop taken as $10 \times A_0 \times t$. The high end of the ship would go down 1½ times the tidal drop, reducing the buoyancy of the hull by $0.75 \times 10 \times A_0 \times t$. Assuming t = 3 m, then $R_s = 46{,}720$ kN and the free end would descend 4.5 m. Were it to touch the ground and were the tide to continue dropping, ground pressures would start to increase, whereas $R_s$ would not increase further.

If a stranded ship were also to flood, bed pressures would increase, depending on the type of cargo: the higher the density of the cargo, the smaller its volume and the greater the remaining volume available for flooding. If the level of the flooded water within the ship follows the external water level, bed pressures similar to fully or partly submerged ships can develop even though a freeboard still exists.

Consider a stranded vessel that has one end supported on the bed and is flooded over a part of its length. The maximum pressure at the supported end is reached when the other end is just free from grounding. This result is the same as that for the nearly fully submerged vessel discussed earlier. Therefore, a stranded ship that is unevenly flooded can also break.

### 3.7 Loading Effects on the Tunnel Structure

The orientation of a ship grounded at an immersed tunnel location is likely in most cases to be more or less perpendicular to the axis of the tunnel, assuming that the tunnel is normal to the waterway.

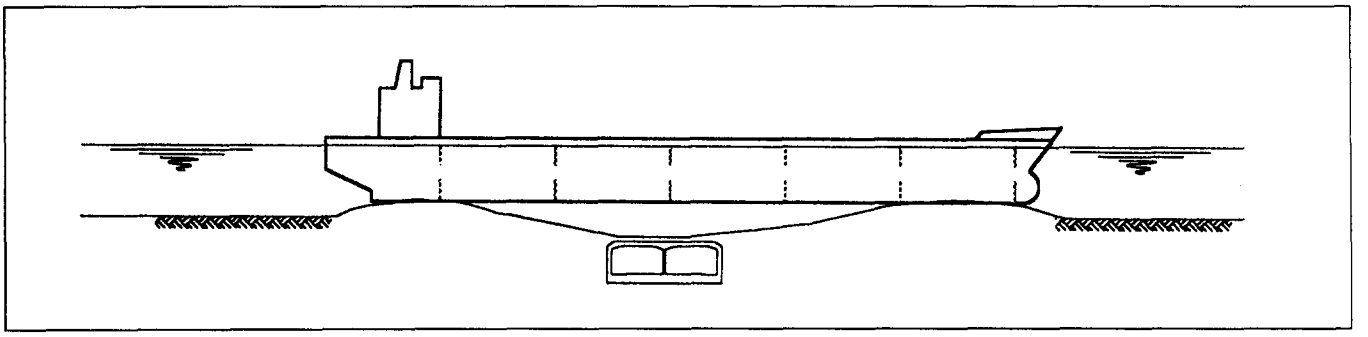


*Figure 6-3. Ship grounded over a tunnel in a depression.*

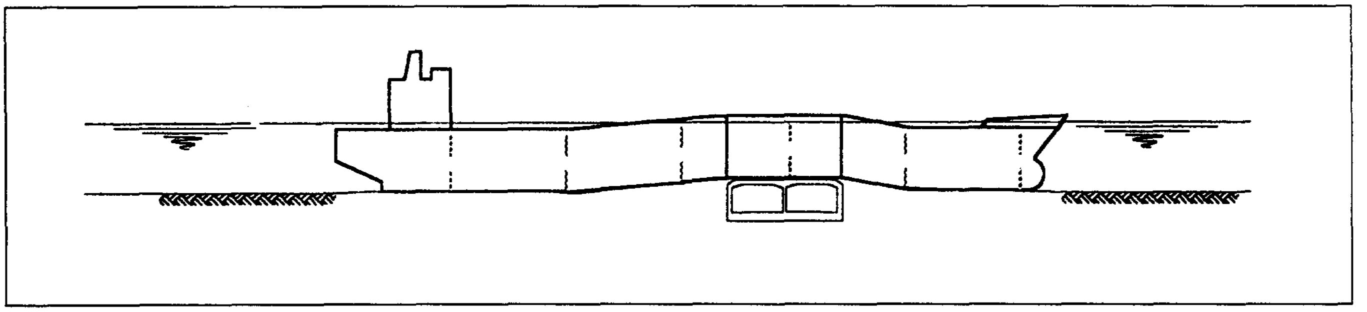


*Figure 6-4. Ship grounded over a tunnel in embankment.*

For ships resting on the bottom over their whole length, either fully submerged or stranded with a falling tide, the maximum average ground pressure is likely to be about 30 $KN/m^2$. This figure would be exceeded by ships fully loaded with iron ore or steel products. The pressure applied to the top of a tunnel from a ship straddling over it would depend much more on bed levels of the waterway beneath the vessel than on soil properties of the ground adjacent to the tunnel. Analysing the load distribution between the tunnel roof and the ground next to it on the basis of a level bed would be of academic interest only and is unlikely to match reality. The top of the cover to the tunnel will either be below the majority of the bed or protruding above it. Furthermore, immersed tunnels are sometimes partly in embankment.

Where the top of the cover to the tunnel lies in a depression, the concave shape of the bed would eliminate practically all vertical loading from fully grounded ships straddling the tunnel (Fig. 6-3).

Conversely, where the top of the cover to the tunnel protrudes above general bed level, load would be attracted to the tunnel by cantilevering of those parts of the ship adjacent to the tunnel structure (Fig. 6-4). Assuming a negative buoyancy of 30 $KN/m^2$ and the location of the tunnel roof under the middle of the grounded ship, the cantilever load at each edge of the tunnel structure could be as high as 70,000 kN for the bulk carrier discussed above, and 30,000 kN for the freighter. These loads exceed by far the direct loading to the roof of about 18,000 kN. Immersed tunnels cannot be reasonably designed for this type of loading.

The maximum value of the end bed-support load for any partly sunken ship not carrying iron ore or steel is calculated to be about 25,000 kN. Because of yielding of the keel structure, this load may be assumed to be spread over the width of the keel. In this case, the load would be spread over an area of 25 m by 10 m, with an average contact pressure of 100 $kN/m^2$.

A ship grounding on top of a tunnel roof parallel to the tunnel axis would be a very special case. If it were to ground at one end only, the effect would be practically the same as if it were perpendicular. If it were to ground over its full length on a level section of the bed, it would be less severe than if it were to ground perpendicular to the tunnel. If it were to ground over its full length on a concave section of the bed, the ship would be supported at the ends only. Each end support load would then be equal to half the total negative buoyancy, which could well be beyond the capacity of the tunnel structure.

### 3.8 Longitudinal Performance of the Tunnel Structure

The load received via the roof has to be transferred to the foundation (Fig. 6-5). Immersed tunnels often have a relatively soft foundation bed, which will cause the load to spread over a relatively large length of the structure, depending on its articulation. The critical factors, especially for concrete tunnels, are high longitudinal bending stresses and the performance of movable joints under such load transfer.

Quite apart from considering the effects of a ship grounding on the tunnel, the ship may be resting on the soil immediately adjacent to the tunnel (Fig. 6-6). This surcharged soil will transmit an increased lateral loading to the tunnel structure due to active earth pressure. The magnitude of these lateral loads is much less than that of the vertical loads, but relatively high to be absorbed by the friction between the structure and its foundation bed. Fortunately, immersed tunnel structures tend to be very rigid and wide in the lateral direction and will therefore spread the load over a large length.

### 3.9 Conclusion

Immersed tunnels are capable of surviving accidental loading effects from nearly all ship grounding events without requiring much adaptation. The exceptions are:

- A ship grounded perpendicular to the tunnel and straddling over it when the top of the tunnel cover is protruding above the bed of the waterway.
- A large bulk carrier grounded parallel to and on top of a tunnel over a long concave section of the bed.
- Nearly all modes of grounding with maximum internal flooding of large bulk carriers fully loaded with iron ore.

It is difficult to design for these exceptions as accidental loads. A probability analysis might help, not only for the probability of grounding and the probable extent of aggravating conditions such as flooding, but also for the likelihood of timely salvaging options to prevent aggravation of the condition. In most cases of grounding in the vicinity of an immersed tunnel, the extent of flooding would be much less than assumed in the sample calculations given in this review.

## 4. Dropping Anchors

In order to protect the tunnel against dropping anchors, roof protection is required. Depending upon the amount of roof protection provided, some additional roof reinforcement may be required to prevent structural damage. Appropriate equivalent static design loads are derived below for variations in the nature and thickness of roof protection and the impact of the dropping object.

An anchor of mass M, free falling in air, will accelerate until it hits the water, at which time the slamming effect at the surface will provide an initial deceleration. Thereafter the velocity will still increase due to gravity, but at a reduced rate because of the flow resistance, which is proportional to the square of the velocity. The terminal velocity for an anchor can be calculated and has been demonstrated by tests to be about 7 m/s. Hence, the anchor terminal impact energy $E = 1/2\ Mv^2$, i.e. 24.5 M kN.m, where M is in tons.

### *4.1 Impact Loads Directly on the Concrete*

The consequences of an anchor dropping on the tunnel will be greatest if it hits the roof directly. Some crushing of the concrete will be inevitable. Tests have shown that the depth of crushing will not exceed 0.15 m, which therefore represents the minimum thickness appropriate to protection concrete. The depth of penetration *X*, which differs from the crushing depth, may be calculated by the method given in *CEB Bulletin* no. 187:

$$X = 10 \times N \times D_e,$$

where
$$N = V_i \sqrt{(M_w/E_c \times D_e^3)}$$
$$D_e = \sqrt{(4A/\pi)}$$
$$A = 0.6 + 0.2\ M_a/1000$$

and
- $X$ penetration depth
- $N$ penetration parameter
- $D_e$ equivalent diameter of striking area of anchor (m)
- $M_w$ mass of anchor reduced by the mass of the displaced water (kg)
- $E_c$ modulus of elasticity in the longitudinal direction of the concrete (N/m²)
- $V_i$ impact velocity of anchor (m/s)
- $A$ striking area of anchor

The dynamic equivalent load can be derived from the Impulse (I). A maximum value is used based on a minimum penetration $X_{min} = 1/2\ X_{calculated}$.

$$F_{max,dyn} = 2I/T_d = 2 \times M_w \times V_{av}/T_d \text{ (N)}$$
$$I = M_w\ V_{av}, \text{ with } V_{av} = 0.5\ V_i$$
$$T_d = X_{min}/V_{av} = X/V_i$$

The static equivalent load $F_{stat} = DLF \cdot F_{max,\ dyn}$, where DLF is the natural period of oscillation of the tunnel roof structure and depends on the ratio of $T_d$ over $T_e$ as given below. The theoretical maximum value for DLF is 2.0. For typical immersed concrete tunnels for traffic $T_e$ is about 0.02 s.

| $T_d/T_e$ | max | 1.0 | 0.4 | 0.2 | 0.1 |
|---|---|---|---|---|---|
| DLF | 2.0 | 1.6 | 1.0 | 0.7 | 0.3 |

An example is given below for an anchor with a mass in air of 10,000 kg:

$E_c$ = $3 \times 10^{10}$ N/m²
$V_i$ = 7 m/s:
$A$ = 2.6 m²
$D_e$ = 1.82 m
$N$ = 0.00147
$X$ = 0.0268 m
$X_{min}$ = 0.0134 m
$I$ = 28,000 Ns
$F_{max,dyn}$ = 14,700 kN
$T_d$ = 0.0038 s
$T_d/T_e$ = 0.2
DLF = 0.7
$F_{stat}$ = $0.7\ F_{max,\ dyn}$ = 10,000 kN

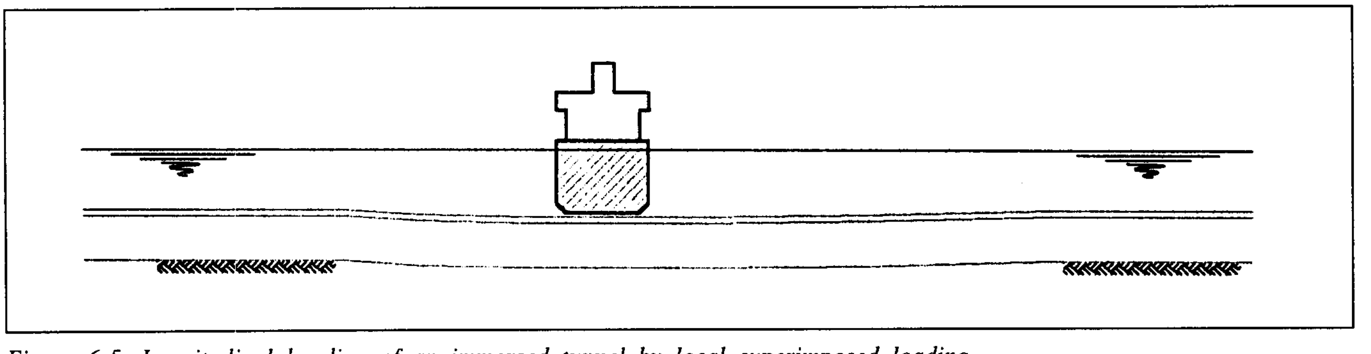


*Figure 6-5. Longitudinal bending of an immersed tunnel by local superimposed loading.*

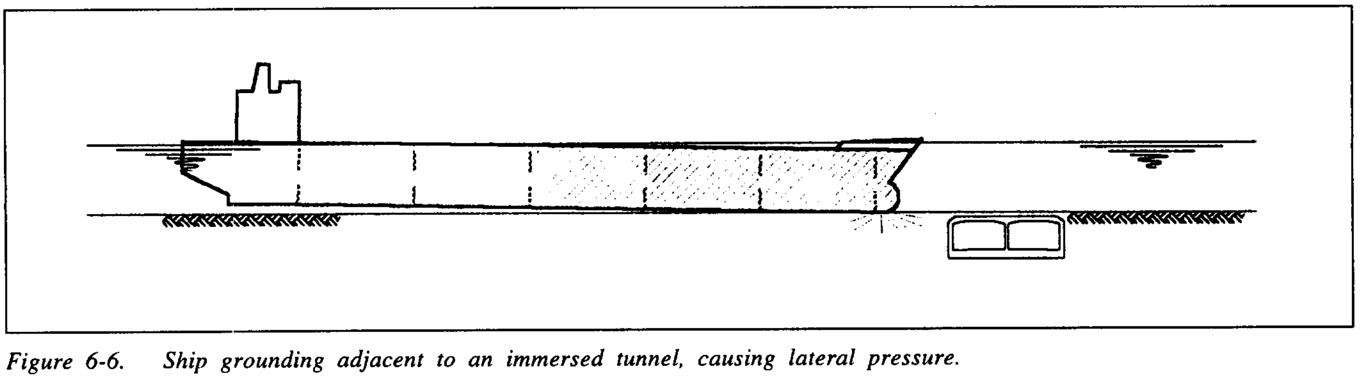


*Figure 6-6. Ship grounding adjacent to an immersed tunnel, causing lateral pressure.*

### 4.2 Impact Load with Granular Roof Protection Layer

For the anchor impact from the previous example, the theoretical static penetration into a sand layer would be 0.4 to 0.6 m. However, the dynamic resistance under water is higher than the static resistance, since pore water pressures also act. An actual penetration depth of about 0.3 m is therefore expected for almost any anchor, provided that the cover layer is thick enough to avoid direct response from the concrete underneath.

The dynamic impact load is equal to twice the kinetic energy divided by the penetration:

$$F_{dyn} = 2 \times M_a \times 0.5 \times V_i^2/0.3 \ \ (N)$$

Because the penetration time is relatively long, $F_{stat} = 2 \times F_{dyn}$, for the same anchor as before:

$$\begin{aligned} F_{stat} &= 2 \times 8{,}000 \times 7^2/0.3 \\ &= 2{,}613{,}333 \text{ N} \\ &= 2{,}613 \text{ kN} \end{aligned}$$

This load may be considered to be spread over an area with effective radius $D_e$ plus half the thickness of the roof slab.

The granular protection layer must be thick enough to reduce the imposed hazard pressures to acceptable values on the roof below. In general, thicknesses of 0.9 m and greater are applied. It may be necessary to make up the thickness using more than one grading. Rock protection may be required to ensure hydraulic stability in strong currents and with exposure to jetting by ship propellers. Where a rock layer is used over a granular layer, it is essential to provide sufficient thickness of granular material beneath the rock to prevent point loads of the rock beneath an anchor from reaching the roof below. This depends on the size of the rocks used. The grading of the granular layer should be in accordance with the hydraulic stability requirements beneath the rock layer.

Where a single-sized stone grading is used, at least five layers of stone would be required. For a stone grading of 2 to 30 kg, a minimum a total thickness of 1 m would be required. Alternatively, three layers of these stones on top of 0.5 m of coarse sand could be used. Should hydraulic stability require heavier cover stones, the total thickness of the roof protection would need to be increased accordingly.

## 5. Dragging Anchors

Anchors continue to drag through bed material until there is sufficient resistance to meet the pull of the anchor chain. When anchors are used to brake the velocity of a ship, the drag distance may exceed a kilometre. Under gale, storm, typhoon or hurricane conditions, anchors may continue to drag indefinitely and even big ships may be tossed ashore. Such conditions are not exceptional; they exist, for example, in Hong Kong Harbour, where soon there will be five immersed tunnel crossings, and where historical records document ships dragging anchors and photographs show beached ships.

An anchor mobilizes resistance in the bed because of its weight and shape while being pulled horizontally by its chain. The chain must be sufficiently long and heavy to make it lie on the bed near the anchor. The holding capacity of an anchor can easily be ten to twenty times the weight of the anchor itself. If the chain lifts off the bed to pull at an angle, this reduces the gape of the anchor and causes it to move towards the surface.

Without appropriate provisions of cover to the roof of an immersed tunnel, itself not normally more than one or two metres thick, a dragging anchor might engage the side of the tunnel structure. The kinetic energy of the ship would cause the chain to break, resulting in loss of the ship's braking power, and the anchor load would damage the tunnel structure. Making appropriate provisions to prevent such an occurrence could release the anchor to the surface before it reaches the tunnel.

Rock berms provided along each side of the tunnel roof can lift the anchor chain and release the anchor to the surface by choking the gape of the anchor. Current practice in Hong Kong is the use of 5-m-wide and 3-m-deep berms of large size rock outside and close to the line of the tunnel. After release, the anchor will not reengage within the width of the berms.

Another practice is the use of a rock layer on top of the roof and extending beyond the sides of the tunnel over a distance of 10 to 15 m; the top of the layer is level with the bed of the waterway. Tests have shown that dragging anchors will break out within that distance. The thickness of the rock layer should be at least equal to the dig-in depth of those anchors expected to be used in the area. For large ships, a thickness of 2.5 to 3 m is recommended. The cover to the top of the roof itself is only determined by the protection of the roof against dropping objects. In the Netherlands, a rock grading of 10 to 60 kg is used.

Instead of a relatively thick rock layer, stone asphalt mats, with a thickness in the range of 0.6 to 0.8 m, are sometimes used. Adjacent to the roof, the mats are laid on a slope so that at 10 m from the roof, they are 3 m below it. This slope is designed to lift the anchor.

Additional precautions are often used to minimise damage, such as providing large chamfered edges to the roof to assist anchors in riding up. For concrete tunnels, a non-structural protective concrete layer some 100 to 150 mm thick is often provided to minimise damage to the top of the roof by dragging anchors.

Should an anchor nevertheless engage on a tunnel, the lateral load exerted on the tunnel structure would not exceed the breaking strength of the cable or chain. The tunnel structure, because of its lateral rigidity, would spread this load over a large distance and can therefore easily mobilize sufficient foundation friction and passive resistance to balance the load.

## 6. Life Safety, Fire and Explosion

The design requirements of a tunnel should also include a prevention plan and a management plan with regard to life safety in case of fire and explosion. Consideration should be given to whether all classes of vehicles should be allowed to use a tunnel, or whether some vehicles should be restricted to traveling in convoy at certain times, or prohibited altogether. The owner should base this decision on the availability of alternative routes and on the advisability of having vehicles that carry explosive, potentially explosive, or hazardous materials (which would include fuel tankers) traveling together with passenger vehicles in a confined space such as a tunnel. There have been incidences where high-risk vehicles have caught fire and resulted in passenger deaths.

The prime directive in designing a tunnel for fire and explosion hazards is life safety. Issues involved in life safety include, but are not limited to, the items on the following list. Effective life safety depends on the coordinated interaction of these.

- The tunnel must maintain its structural integrity and preferably be capable of repair.
- Ventilation systems should be designed with procedures to be implemented in case of fire and in case of noxious and dangerous fumes, e.g., from vehicle exhausts. The spread of hot gases and smoke must be controlled.
- Any cabling likely to be exposed to hot gases in case of fire should be rated. This applies especially to cabling in the ventilation ducts that is not otherwise protected, and to all emergency circuits.
- Fans and ducts likely to carry hot fumes must be designed to operate under the expected ambient temperatures. A minimum of 250° C for at least one hour is recommended for fan design.

- Tunnel finishes should neither be capable of spreading fire or toxic fumes, nor of bursting.
- Emergency egress for the occupants of stopped vehicles should be planned. This may require escape galleries or cross passages to the adjacent traffic tube.
- Fire detection equipment, fire alarms, fire extinguishers, and fire-fighting equipment should be considered. Where the possibility exists for a wet standpipe system to freeze in winter or rupture in earthquake, a dry system should be used. Alternatively, heating of the wet system can be used if there is no earthquake danger.
- Closed-circuit TV, traffic control and detection systems, and two-way communication systems for emergency services should be considered. Loop detectors could provide an alarm when traffic is stationary, indicating an incident which could include fire.
- Emergency lighting and power is recommended. Emergency power should operate for a minimum of one hour and a half, and emergency lighting should be capable of operating at 500° C for one hour.

The heat release from a burning fuel tanker fire is generally taken as 100 MW. For other fires, a heat of 50 MW is usually considered. The heat release from a single passenger car is about 5 MW. The 1995 edition of the proceedings of the Permanent International Association of Road Congresses (PIARC) describes procedures and temperatures to be used in design. PIARC also provides design methods for control of fumes from sources other than fire. The standard ISO-temperature curve is unrealistic for tunnels. The German hydrocarbon curve, the so-called MOBIL curve, is often used. This curves gives a temperature rise to 900° C in 3 minutes, to 1100° C in 20 minutes, and increasing to 1200° C after a total of 90 minutes. In the Netherlands, the so-called RWS curve is used, which would apply for a 300-MW fire. This curve shows a rise to 1100°C in 4 minutes, increasing to 1350° C after a total of 60 minutes, and going down to 1200° C after a total of 2 hours.

PIARC statistics indicate that a fire occurs about once per 10 million vehicle kilometres. Most of these are small fires that can be brought under control by using fire extinguishers when they first start. For that reason, it has now become standard practice to provide easily accessible portable fire extinguishers in many vehicular tunnels.

The National Fire Protection Association (NFPA) Standard 502 of the United States, 1996 edition, is intended primarily for guidance in the design, construction, operation, maintenance, and fire protection of tunnels and other structures. The purpose of NFPA Standard 502 is to establish minimum criteria that provide a reasonable degree of protection from fire and related hazards.

Generally, several ventilation systems, including longitudinal, semi-transverse and fully transverse, would be evaluated in order to select the most appropriate ventilation system for a particular tunnel. Based on the length of tunnel, emission rates, pollutant levels, airflow requirements, safety, feasibility, cost-effectiveness, and space constraints, the optimum ventilation scheme would be selected. [The second edition of the *Tunnel Engineering Handbook* (Bechell et al. 1996), PIARC guidelines, and other publications address the tunnel ventilation issue in great detail.] Following an actual fire test in a disused tunnel, the Federal Highway Administration of the United States Department of Transportation issued a memorandum that stated the agency's willingness to accept the use of jet fans in the design of short highway tunnels, less than 700 m in length.

Where extraction of polluted air at ceiling level is not part of the ventilation system, such as for longitudinal ventilation, the ceiling or roof of the a tunnel will prevent dispersal upwards of smoke and heat, creating a serious hazard to occupants. Smoke and heat that rises will move longitudinally at the ceiling or roof of the tunnel and will move rapidly upgrade. This stratification, if maintained, will enhance the short-term safety of occupants at road level.

Sprinkler systems are not recommended in tunnels because they delaminate the stratified smoke and heat layer and cause turbulent mixing of air and smoke. Fires are usually inside or underneath a vehicle, so sprinklers would only cool down the exterior surfaces. There is also a risk of explosion from the vapours generated by the use of sprinklers after a fire has been extinguished.

In the case of longitudinal ventilation of short tunnels, the most effective method to control the longitudinal spread of smoke and heat for the safety of the occupants is the use of reversible fans. The fans should blow in the traffic direction so that occupants can escape and rescue teams have access to the vicinity of the fire. The capacity of the longitudinal ventilation system should be such that sufficient air velocity can be provided to prevent the return flow of smoke and heat along the ceiling or roof of the tunnel. On the downstream side, the smoke and heat will be spread over a large section of the remaining length of the tunnel.

With semi-transverse or fully transverse ventilation systems, the smoke and heat can usually be extracted through the available ventilation ducts by suitable use of emergency dampers and fans set to exhaust the tunnel. These more expensive ventilation systems therefore present fewer hazards than longitudinal ventilation systems.

In case of fire, the tunnel structure will be heated from the inside, reducing the structural strength. Steel heated to 400° C will have a factor of safety of just 1.0. As reinforcement and residual moisture in the concrete are heated, spalling of the concrete is likely to occur, aggravating the situation. The heating can be both delayed and reduced by the application of fire-proofing material to the roof and upper parts of the walls. The material used should maintain its integrity and remain in position in case of fire. Special protection is required at joints to maintain their integrity and protect any gaskets present.

An internal explosion exerts a high internal pressure on the tunnel structure. An explosion of the deflagration type could exert up to 8 bar (0.8 MPa) pressure to the structure, whereas a detonation-type explosion could exert pressures up to 20 bar (2.0 Mpa). Unfortunately, it is not impossible that a deflagration-type explosion could result in a detonation.

An immersed tunnel structure might be capable of resisting moderate deflagration pressures. The hydrostatic pressures in the deeper parts of the tunnel will help offset the internal pressures on the exterior components of the structure, so that these components are probably least at risk. However, the internal walls and slabs do not have that advantage and are therefore the critical elements. If these elements are designed to resist internal flooding of one tube of a twin-tube tunnel, they are also more resistant to deflagration pressures. Near the portals of the tunnel, the favourable hydrostatic pressure is minimal. These parts of the tunnel are therefore more critical than the deeper sections.

The limiting internal pressure to which an immersed concrete tunnel can be designed is in the range of 4 to 5 bar, with any internal structural members being critical to the subsequent survival of the structure. A steel double-shell tunnel with cylindrical interior shells will resist internal pressure by hoop forces of the interior shell. For a 10-m internal-diameter shell with a 10-mm plate thickness and a yield stress of 240 Mpa, the ultimate net internal pressure would be about 5 bar. Consequently, it is clear that moderate deflagration pressures can be designed for, but higher pressures cannot be resisted. In this respect, bored tunnels in soft ground would have similar problems. These physical design limits should be taken into account when permitting the passage of hazardous goods through immersed tunnels.

Typical accidental overpressures for which immersed tunnels have been designed are:

- 1 bar for those in the Netherlands;
- 4 bar for the Liefkenshoek tunnel in Belgium, because it forms the access to an industrial area in the port of Antwerp;
- 5 bar in Sweden, as required by Swedish codes.

## References


Bechell, J.; Kuesel, T.; and King, E. 1996. *Tunnel Engineeirng Handbook, 2nd ed.* London: Chapman & Hall.

*CEB (Commitée Européan Béton) Bulletin* No. 187.

*National Fire Protection Association (NFPA) Standards.* 1996. *NFPA Standard 502.* NFPA.

*Proceedings of the Permanent International Association of Road Congresses (PIARC).* 1995. PIARC.

Federal Highway Administration of the United States Department of Transportation. Memorandum.

PII:S0886-7798(97)00011-4

# Chapter 7

# TRANSPORTATION

by

CHRIS HAKKAART — The Netherlands
Hollandsche beton en Waterbouw / DMC

*Contributions and comments by:*

Walter Grantz — U.S.A.

Christian Ingerslev — U.S.A.

Frits Brink — The Netherlands

Stoffel de Ronde — The Netherlands

Cor Th. van Doorn — The Netherlands

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.145–156, 1997

Printed in Great Britain
0886–7798/97 $17.00+0.00

Pergamon

# Chapter 7: Transportation


**1. INTRODUCTION**

**2. TRANSPORTATION ROUTE**
2.1 Transporting Concrete Elements
2.2 Transporting Steel Elements
2.3 Route Evaluation

**3. PREPARATION FOR TRANSPORT**

**4. INTERNAL FORCES DURING TRANSPORT**

**5. TOWING FORCES**

**6. NAUTICAL ASPECTS**
6.1 Element Stability
6.2 Motions of Elements
6.3 Course Stability

**7. HYDRAULIC MODEL TESTS**

**8. TRANSPORTATION BY BARGE**

**9. SAFETY/RISK**

**10. INSURANCE**

**11. EXAMPLES OF INLAND TRANSPORTATION**
11.1 Tunnels in The Netherlands
11.2 Second Hampton Roads Bridge Tunnel (U.S.A.)
11.3 Fort McHenry Tunnel (U.S.A.)
11.4 Monitor-Merrimac Tunnel (U.S.A.)
11.5 Western Harbor Crossing (Hong Kong)

**12. EXAMPLES OF OFFSHORE TRANSPORTATION**
12.1 63rd Street Tunnel (U.S.A.)
12.2 Second Downtown Tunnel (U.S.A.)
12.3 Sydney Harbour Tunnel (Australia)
12.4 Ted Williams Tunnel (U.S.A.)
12.5 Wijker Tunnel (The Netherlands)
12.6 Piet Hein Tunnel (The Netherlands)

**13. CONCLUSIONS**

## 1. Introduction

Given suitable site conditions, the immersed tunnel method has several important advantages over other tunnelling methods. One of these is the fact that the tunnel is prefabricated in sections in a controlled shipyard or casting basin environment. This fabrication facility can be hundreds of kilometers away from the actual tunnel construction site. Consequently, requirements for transporting tunnel elements over bodies of water, including the open ocean, are an integral part of the design and construction of immersed tunnels.

Transporting a tunnel element is different from towing a ship or vessel with a streamlined hull. While tunnel elements occasionally have been equipped with a temporary, removable bow, most tunnel elements are blunt-ended, resulting in slow towing speeds and difficult maneuvering behavior.

The transportation of tunnel elements has an inherent hazard not common to ordinary vessels or barges—there is no redundancy in flotation to protect the element from sinking if the hull is breached or cracks. The element can rely only on the single buoyancy chamber provided by its tube(s). Furthermore, the freeboard is as low as 100 mm for some concrete elements, leaving them little spare floating capacity.

Immersed tunnels are designed to be placed on prepared foundations without having significant permanent differential loads (except the differential settlements of the soil). The only large, permanent, external load is the circumferential water/soil pressure. During transportation, however, more severe loading cases can result from factors such as: the weight of the end bulkheads; the equipment mounted on the element for placing; the temporary mounting or support of the element during transport; the offshore wave height and period; and the structural staging of the element at the time of transport. All of these factors must be taken into account in the design of the element and the provisions for the method of transport.

Although the transport of an element is a special activity that occurs only once in the life of the tunnel, it takes special planning to avoid affecting the more important long-term requirements of the design. In the following sections, basic issues related to the transportation of tunnel elements (steel or concrete) are explained in more detail and illustrated using various completed projects as examples.

In immersed tunnels, steel and concrete are used as the main materials in various structural configurations. Steel tunnels generally fall into a few basic configuration categories—monocular and binocular tube arrangements and single and double shell structural configurations—although there have been departures from these basic styles: the 63rd Street Tunnel, for example, with two-over-two tubes; or the Harlem River Subway Tunnel, with four tubes side-by-side in a single element. Concrete elements are generally rectangular and have one or more tubes. Examples of the structural characteristics of these types of tunnels can be found in Chapter 3 of this report.

In steel tunnels, the fabrication of the steel shell is the first stage of construction. The tunnel is eventually outfitted with interior and exterior concrete in a series of subsequent stages of construction. Because this type of construction offers many options during which towing can take place, there is a large range of potential drafts for towing. Tows at very shallow draft are therefore possible. Concrete immersed tunnels do not have this flexibility of draft, and they generally must be towed with only a small freeboard. Occasionally, the roof slab is left off to reduce the depth of water required for the tow.

While elements are usually transported using their inherent buoyancy, in recent years some steel-shell elements have been transported on barges. Those elements have been approximately 100 m long and have ranged in width from 12 to 24 m, with a height of approximately 12 m.

## 2. Transportation Route

The risks involved in the transport of immersed elements are largely determined by the route that will be taken.

### 2.1 Transporting Concrete Elements

The location of the casting yard for concrete elements is generally selected from available sites as close to the tunnel project location as possible. Concrete elements for highway tunnels, when completed for transport, often have a draft between 8 and 9 m.

Occasionally the tunnel elements are constructed with decreased draft requirements by eliminating the roof slab until the elements are at the tunnel site. Such a large draft requires a route that follows existing main navigation channels. The towing of concrete elements involves the trip from the casting yard to the site where the element is placed in the tunnel trench. The very small freeboard reduces the margins for error and therefore makes offshore towing more risky. This means that there are usually very few or no alternative routes available for towing concrete elements if risks are to be minimised.

### 2.2 Transporting Steel Elements

For steel elements, the transportation route can be more flexible since the route is normally divided into two phases. The first phase is the trip between the shipyard and the outfitting yard. The steel shell is fabricated at the shipyard where perhaps keel concrete may also be placed. During this first phase, the element may weigh only a few thousand tons and typically draws only 2 to 3 m of water. Because of this, it can be carried on a barge or, as previously stated, can be towed very long distances under its self-buoyancy. At the end of the first phase, the element is completed at the outfitting yard with the installation of all the interior structural concrete and most of the exterior ballast concrete.

The second phase of towing involves the trip from the outfitting yard to the site where the element will be placed. The location of the outfitting yard is generally a very short distance from the tunnel site.

With a draft now similar to concrete elements, this route must provide much deeper water to allow for at least all the interior structural concrete having been placed. For double-shell elements, placing of the exterior ballast concrete can be delayed until the element is right above the tunnel trench, thereby reducing draft requirements by perhaps 2 m. Fully ballasted and carried in the placement barge, a double-shell steel element often draws about 12 m of water.

Largely because of this two-phase transport available for steel tunnels, a much wider range of shipyards is eligible to bid competitively for the fabrication of the elements. This can be advantageous in reducing costs to the owner.

### 2.3 Route Evaluation

For both concrete and steel, the intended route must be carefully evaluated for hazards. Some important factors that must be considered are:

- **Type of Waterway:** Rivers, canals, lakes, inland waterways, offshore
- **Physical Obstructions:** Water depth, bridges, locks, quay walls, channel alignment
- **Navigation Aspects:** Ship traffic, temporary relocation or closing of shipping channels
- **Hydraulic Aspects:** Currents, tides, waves, water densities
- **Meteorological Aspects:** Wind, fog, storms, hurricanes, typhoons
- **Refuge Areas:** Shelter areas and moorings during severe weather

- **Towing Aspects:** Towing speed, maneuverability, width of towing array of tugs, escort by maritime authorities
- **Harbor Facilities:** Departure and arrival facilities in case a transportation barge is to be used. Sufficient sheltered water depth is needed to float the element on and off the barge.
- **Structural Capability:** The element must withstand the anticipated loads to which it will be subjected during transport.
- **Environmental Aspects:** Avoid the need to dredge waterways to provide passage for elements.
- **Other issues:** The skill of the people planning and participating in the transport operations, provision for contingencies such as tug breakdowns, broken cables, leakage in the element, and so on.

## 3. Preparation for Transport

By definition, an immersed tunnel must be floated into position. Watertight bulkheads must be provided. These are constructed of a steel plate supported by steel beams or, for some concrete immersed tunnels, by a concrete wall also supported on steel beams. In the earliest days, timber bulkheads, made watertight with a bituminous membrane, were used.

To provide immediate safe access into the joint area after joining the elements together, a hatch at roadway level is generally provided (see Fig. 7-1). This consists of a marine bulkhead door sealed by a heavy gasket against which the door is pushed by the exterior ambient water pressure. The door must therefore open outward. Even so, because this door seal is a source of potential leakage, it has been the practice in recent years to secure it against leakage by welding a temporary, watertight steel plate over the opening at the inside face of the bulkhead. This is very important when the bulkhead doors are to remain submerged for a long period of weeks or months and if no access is available to the interior of the tunnel. This plate is removed at the time the element is connected as the joint area must then be entered.

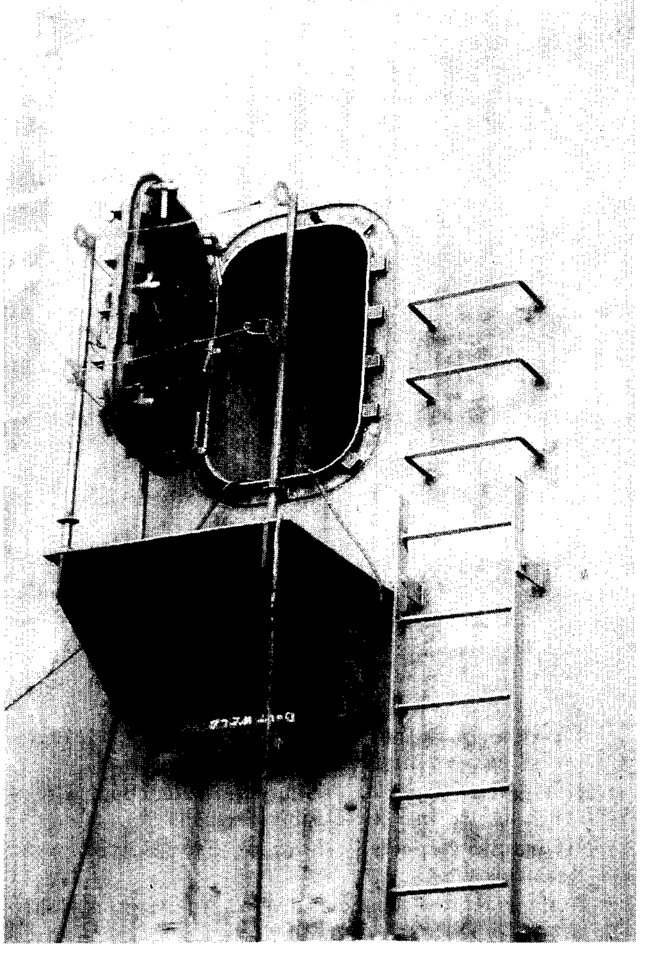


*Figure 7-1. Typical hatch in bulkhead.*

Hatches are generally needed in the roof slabs during the interior outfitting of elements and to provide temporary access at the ends of the tunnel before permanent access is available. Because of the extent of outfitting, many larger hatches are used in steel tunnels, whereas fewer smaller hatches are needed in concrete tunnels. Hatches will also permit access during storage [mostly for concrete elements], or access during placing.

During transport, these access hatches are treated in different ways for different conditions. For concrete elements, long access towers are often installed immediately before placement to allow internal access during placement, and then removed. During tow or storage in sheltered waters, a short access shaft is used. When towing in open ocean, the access shaft must be designed to resist anticipated wave forces or omitted altogether. Some elements are sealed without hatches for placement.

Hatches in the top of steel elements are usually sealed for towing, but not provided with access towers until they are ready to place. Since steel elements generally ride high in the water during long tows and are sealed without hatches for placement, hatches do not present the same problems of access as they do for the concrete tunnels. An exception to this is at the two ends of the tunnel where access must always be provided.

Towing lines must be connected to bollards or pad-eyes at hard points on the sides and at the corners of the element. These must be designed to prevent damage to the towing hawsers and be strong enough not to break, since breakage could result in loss of control over the element.

Attention must be given to the stiffness of the towing system. When long lines cannot be used because of the geometry of the space or the water depth, shorter lines can be used if special polypropylene shock absorbers, or the equivalent, can be incorporated in the tow line.

Experience has shown that a tunnel element behaves differently under tow than a vessel does. This is largely because of the deep draft and the large flat-plate areas presented by the end bulkheads of a typical element. For a concrete element with only a few centimeters of freeboard, pushing the element with the bow of a tug is next to impossible because the bow rides so much higher than the concrete element. Steel tunnels without ballast, on the other hand, have plenty of freeboard to push against, but only some areas strong enough to take the forces. These include the end ballast pockets on each side when filled with concrete, and the sides where the four heavy lowering connections are encased in concrete. Other areas are normally not designed to resist large side loads.

Sometimes special pushing panels are provided (see Fig. 7-2). Pushing against an element to resist current during maneuvering can only be considered for sheltered waters. This is impossible in a heavy swell at sea because of the relative motion between the tug and the element. Navigation lights required by the controlling maritime agencies are normally mounted on a small mast installed on the element and removed before placement.

A spare towing line is often stored on top of the element, attached at the bow, and provided with a floating extension that trails behind the element. If the main towing line breaks, this spare line can be easily picked up from the water surface. In severe weather, the alternative of landing a man on top of the element to install a new line may be too dangerous.

For concrete elements being transported with little freeboard, emergency pumps are an important requirement.

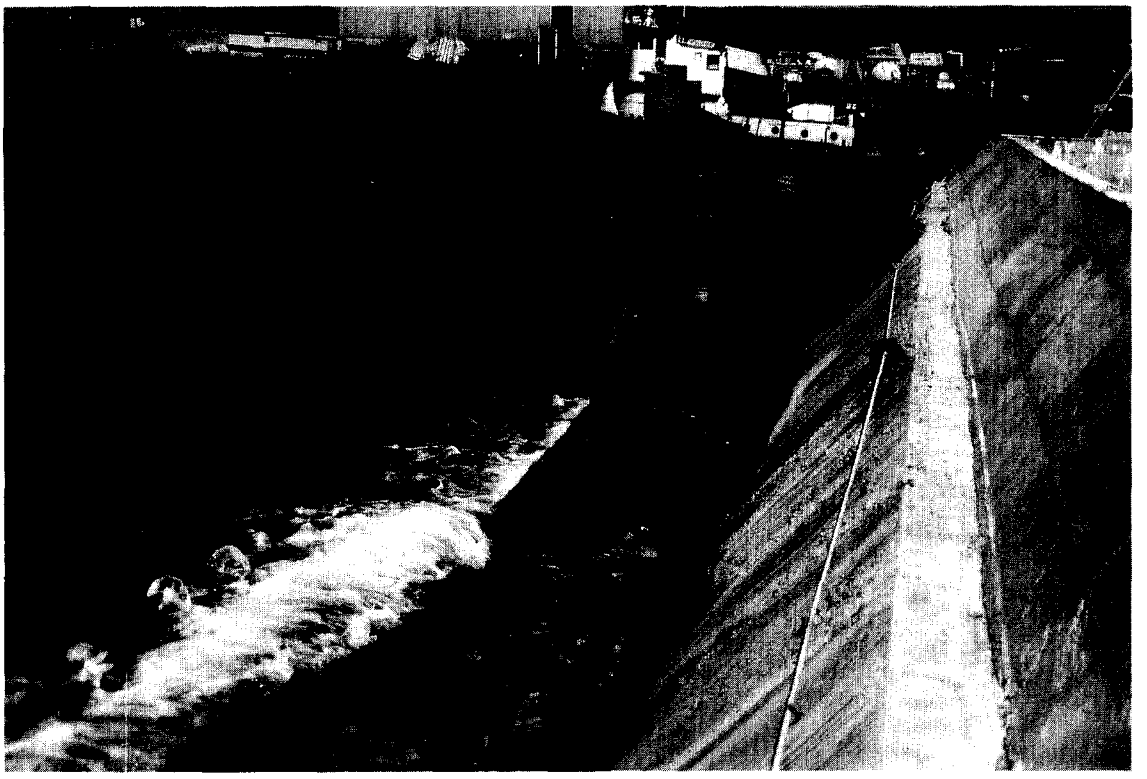


*Figure 7-2. Pushing panel.*

These pumps must be able to begin operating very quickly and have sufficient capacity to override any potential inflow since the spare freeboard will not maintain flotation and stability very long. Measures should be taken to equip the element with intermediate temporary low dams to assure stability under such conditions.

## 4. Internal Forces During Transport

Tunnel elements under tow must withstand loading conditions comparable to ordinary ships. These include wave-induced longitudinal, transverse and/or torsional moments; static and dynamic forces on the various individual items such as the bulkheads, the outer pockets, etc.; and potential impacts of collisions with floating objects. The magnitude of these forces depends largely on the expected sea conditions along the route. This in turn depends on the length of time of exposure during the trip. All of these conditions must be carefully investigated.

Once these conditions have been determined, the more complicated part of the evaluation starts. The elements must be checked to ensure that overstresses do not occur. Hogging, sagging, transverse, and torsional moments must be established. An envelope of moments for all load cases must be developed. Based on that, the suitability of the method and configuration of the towing system can be evaluated.

If the tow is through sheltered waters, the maximum moments are likely to occur during placement of the element. When the elements must be towed in open waters, wave-induced moments may be a critical design case. The moment envelope determines the required structural strength of the structure. For steel shell tunnels or non-segmented, non-prestressed concrete tunnels, flexing of the element may cause unacceptable cracking of the concrete. The anticipated amount of cracking must be checked against requirements for structural integrity and water tightness.

Empirical rules are used to design ships. These rules are based on a vessel with sufficient freeboard. A tunnel element however may have very little freeboard, so that overtopping by waves is probable. Mathematical and hydraulic modeling may be required to investigate potential moments.

A segmented concrete tunnel needs prestressing during its float up off the floor of the casting basin, during the tow, and during placement.

Taking care of moments induced by wave action through additional prestressing is relatively easy. Both the structural integrity and watertightness are improved by this prestress. The prestressing cables are usually cut at the segment joints at some stage after the element has been placed.

## 5. Towing Forces

To establish the required tug capacity, the drag of the element under different towing speeds needs to be investigated. This investigation must be done not only for calm conditions but also for heavy seas.

After the drag at several speeds is known, the following factors should be optimized: the preferred average speed during the voyage, the related towing capacity, the capacity required for maneuverability; the available capacity of tugs in the area; the cost of tugs; and the optimum layout of these tugs. If the tow must traverse an area with large currents, tidal or otherwise, the capacity of the tugs needs to be at least sufficient to keep the element in position during the strongest adverse currents; otherwise, the tugs and element will need to anchor. On the other hand, provisions must also be made to keep the element under control when sailing in the same direction as the current. Sufficient speed through the water is then an absolute requirement. Sometimes extra tugs are needed when the tow must pass through such locations.

## 6. Nautical Aspects

### 6.1 Element Stability

Concrete elements have a low freeboard. Steel tunnel elements may have a larger freeboard, depending on their intermediate ballast situation. In either case, the elements are usually symmetrical in cross-section.

Generally, the steel elements have a sufficient metacentric height; if not, it can be adjusted by adding keel concrete to increase the righting moment. The extra width of binocular steel tunnel elements makes them inherently more stable than monocular elements.

Typically, steel elements have been transported at high freeboard with only keel concrete for stability during the long journey between the shipyard and the outfitting yard.

### *6.2 Motions of Elements*

As with any vessel, a tunnel element will move in all three directions of freedom (x, y, and z) and rotate around three axes through its center of gravity (roll—longitudinal axis; pitch—transverse axis; and yaw—vertical axis). While the amplitudes of these motions can be calculated, the best assessment of the element's behavior can be established only by means of hydraulic model tests.

In sheltered waters, motions have always been very small and have caused no problems as long as water depth was sufficient. During tows at sea, motions are slow and usually well within acceptable limits. While the roll movement may attract the most attention, the pitch movement produced by ocean swells may cause the element to dive under the water surface and to surface again only after several minutes. The least significant situation involves short waves crossing over the element.

### *6.3 Course Stability*

The absence of any hydraulic streamlining elements causes yaw under tow. This is most noticeable when towing with a single bow tug. The yaw is usually limited to a few degrees—acceptable for this once-in-a-lifetime tow. When a standard spread of five tugs is employed during a tow through narrow channels, the yawing behavior can be controlled completely.

## 7. Hydraulic Model Tests

Hydraulic model tests have often been used to investigate the motions and resulting moments of an element under tow on its own keel in the open sea (see Fig. 7-3). The advantage of these model tests is that the effects of extreme wave conditions can be studied and provided for in the design.

For such tests, the physical model of the element at a workable scale, typically 1 to 3 m long, is made from wood or steel. Applicable physical characteristics such as mass, moments of inertia, and geometry are all scaled down to equivalent model values.

For studying the motions, the model must be equipped with special devices for recording the motions during test runs in a long basin.

When moments are to be studied, the model is cut in half and provided with strain gauges to measure the forces at the top and bottom. By combining the output of these gauges, the longitudinal and transverse moments can be established. In one case where very bad weather conditions were anticipated, the model was outfitted with pressure gauges in the bulkhead to register the wave impact on the bulkhead.

In addition to constructing the physical model, wave parameters and spectra must be established; these include the fetch, wave height and period. Published wave spectra, available for all oceans, are used for this purpose. For example, the Jonswap Spectrum would be used for the North Sea. This spectrum defines the energy distribution in a large wave field. The advantage of working with spectra is that all types of wave combinations, both height and period, are represented. These irregular waves are the best representation of a general sea state.

During the preparations for the hydraulic model tests, the incoming direction of waves in relation to the towing direction needs to be considered.

When the tow is to be made along a shoreline where the water depth is limited and where swell may be present, the depth of water must be considered. The height of the swell will increase because of the reduced depth, also changing the wave heights and periods to be used in design.

Once the scope of the tests has been fixed, the actual tests can be done in a long basin. If possible, radio-controlled model tugs should tow the element through different modeled sea states.

Test results in the form of many response functions will be assembled and tabulated, showing the response of a certain type of motion or moment to a certain wave spectrum. These data will be used to assess critical situations that must be provided for in the design of the element and the method of towing. The results can be presented as a probability-of-exceedance graph, showing the probability of occurrence of the peak values of the specified motion or moment for each spectrum (see Fig. 7-4). Such graphs can be used in establishing the required strength of the element in relation to the probability of occurrence of particular waves.

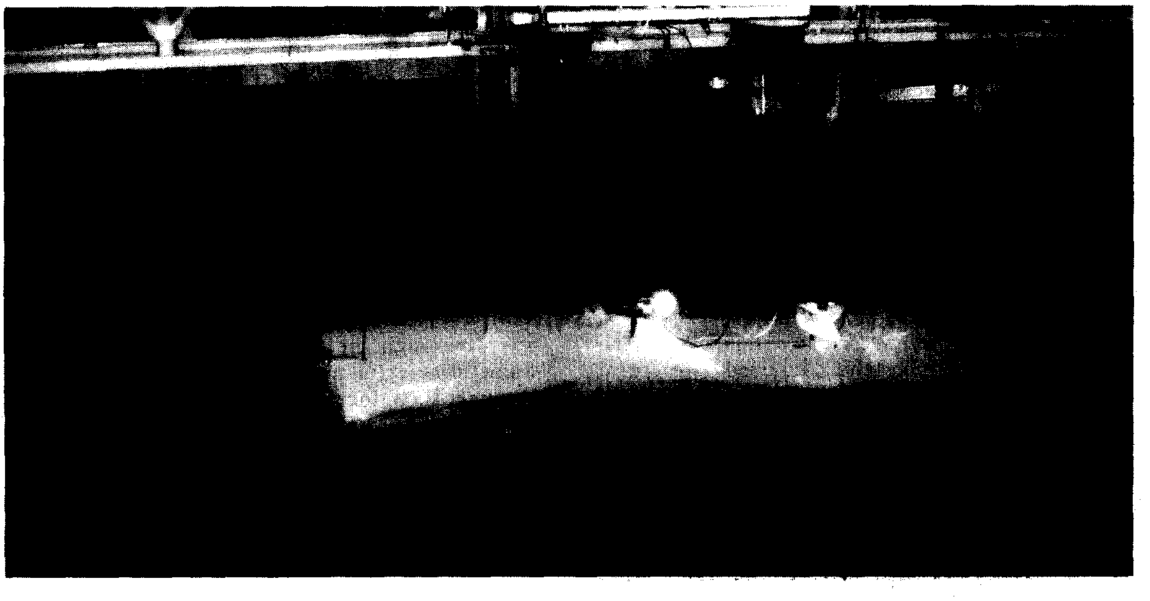


*Figure 7-3. Model test.*

Based on the results of the hydraulic model tests, those conditions to which the element needs to be designed must be selected. Larger waves can be tolerated if the strength of the element is increased by providing temporary prestressing. This can easily be realized for concrete elements by providing additional prestressing. Doing so will increase costs, but may reduce towing downtime. For steel tunnel elements, the original structural strength has to be checked against the maximum anticipated sea state stresses.

After the structural design strength has been established, the model test results can provide the basis for a "go" or "no-go" decision for the transport of an element. Weather forecasting is an inexact science at best, and the reliability of predictions diminishes rapidly after 24 to 48 hours. This applies to the accuracy of the forecast wave height and period as well. Therefore, there is often a risk of encountering more severe wave spectra during an offshore voyage than anticipated during the design. This reduction in the level of safety during potentially more severe conditions needs to be considered. Data developed in a model test can be of great help in evaluating the degree of exposure and risk. An example of this is the graph shown in Figure 7-5, developed for the voyages of the Wijker Tunnel. For tunnel projects to date that have required offshore towing, very few difficulties have been encountered during the actual tows because of the extensive studies, preparation and balanced decision-making during the design process.

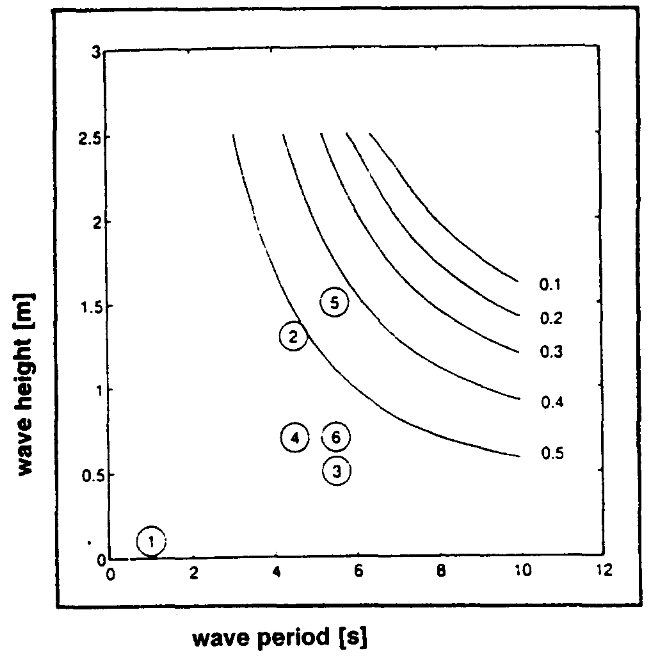



*Figure 7-5. Wave height vs. Wave period graph for the Wijker Tunnel.*

## 8. Transportation by Barge

When a tunnel element is transported by a submersible barge, special attention must be given to the process of floating the element on and off the barge. During these operations, stability of the barge and the element must be assured. This is achieved by ensuring that the barge and element are sufficiently inclined in the longitudinal direction to engage an adequate water plane. Alternatively, the end of the barge/element combination may be supported on the bottom to prevent an uncontrollable roll.

In an inclined position of the barge, the element will be lifted at one end by the barge during a part of the operation, causing extra longitudinal bending moments in the element. The moment capacity of the element must be checked for this phase of the operation. During transportation, the wave-induced moments will be shared by both the barge and the element. The distribution of these moments depends on the stiffness of each of these bodies. Although the moments on the element are reduced by the stiffness of the barge, calculations have to show whether they are acceptable.

Concrete elements for highways are usually too heavy (mostly in the order of 30,000 to 40,000 tons) to transport as a load on a barge, even on an offshore barge. Either the deadweight tonnage of the barge is insufficient or modification of the barge is required to distribute the weight over the frames. An aspect that should not be overlooked is the fact that ordinary barges are not designed as a submersible to go to a depth of about 10 m. These comments would not necessarily apply to elements used for rail or transit systems.

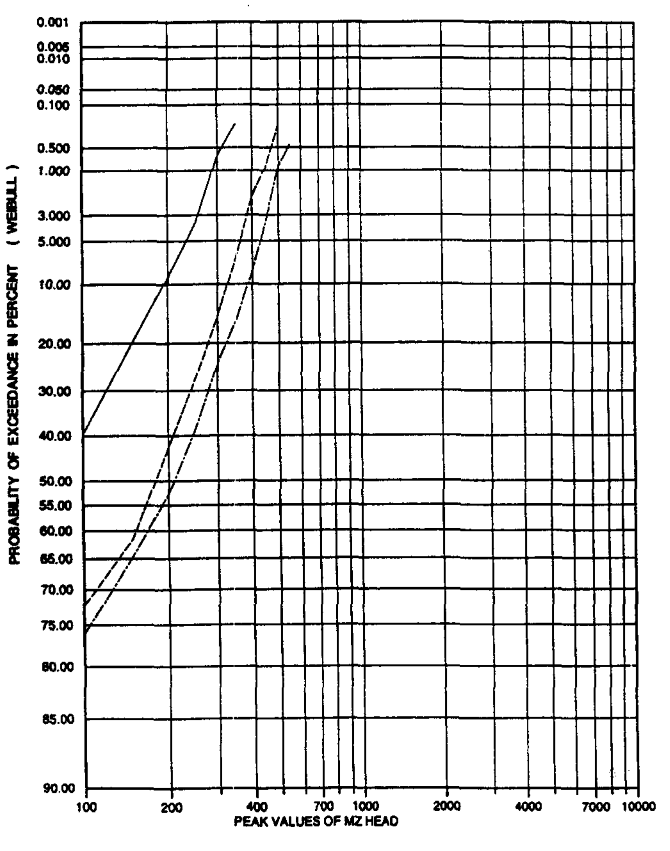



*Figure 7-4. Distribution graph showing probability of exceedance of certain peak values.*

## 9. Safety/Risk

Analysis for the transport of tunnel elements may use either of two approaches:

- *The conventional deterministic method:* The risks are covered by the safety factor and are described clearly in a design report. This is a straightforward approach that may result in over-design.
- *The probabilistic method:* Complex mathematical calculations determine the probability of occurrences. The calculations are undertaken during the design phase and provide a good understanding of the capability of the element to resist combinations of the severe loading cases. It is essential that a practical translation of the probabilistic assessment be examined and evaluated by construction people with hands-on experience.

## 10. Insurance

The cost of transporting an immersed tunnel is not entirely dependent on the engineering aspects described above, nor on the cost of the tow itself and the equipment involved. Much of the cost is dictated by the insurance underwriters and how they view such an unconventional sea voyage for such costly items as immersed tunnel elements. In recent years, insurance companies have taken a harder look at the risks they are exposed to during the transport of steel and concrete tunnel elements. This has resulted in a wide range of conditions and requirements, based on their understanding, experience and knowledge of civil engineering projects of this kind.

For steel tunnels, two methods of transportation are available: the elements can be towed using their inherent buoyancy or they can be loaded on a barge. Although the cost of insuring the element is lower in the second case than in the first case, additional insurance is needed for the barge itself. As a result, the total cost of insurance may be similar for the two options. The final choice must be made after all of the other factors are known.

Large concrete elements have always been transported using their own buoyancy. The insurance costs must be based on this premise, as it is impractical to consider towing the elements by barge. These costs must be negotiated, and when additional measures are required by the insurer, the costs escalate rapidly. In one case, a double bulkhead was required by the underwriter for each end of each element. The perceived risks and related costs of insurance are often based on an inadequate understanding of the processes involved and who is carrying the financial risks, rather than on a realistic technical evaluation.

## 11. Examples of Inland Transportation

### *11.1 Tunnels in the Netherlands*

The Netherlands has the highest density of immersed tunnels in the world. All Dutch tunnels are of the concrete rectangular type and have included both segmented and non-segmented tunnels. Most of these tunnels are in the Amsterdam and Rotterdam areas; all were fabricated in graving docks. In the Rotterdam area, the Barendrecht dock has been used for the construction of the elements of more than eight tunnel projects. In two cases, the elements of two different tunnel projects were built in this dock at the same time.

Usually the towing route has followed shallow rivers and tidal waters. At times, the clearance under the keel was not more than 1 m. Because the Barendrecht dock is on a river, the towing of an element is governed by river currents and periods of slack tidal waters. Waves are normally not a concern, since they are small and do not exceed 0.5 m. In addition to the currents and waves, there are many bridges to pass and many turns that must be made at river confluences.

Because of the above conditions, the five-tug "marine spread" is the standard layout of the tow. This involves two tugs at the bow and two tugs at the stern, all pulling in the forward direction, plus a pushing, steering tug at the stern. This layout allows sufficient maneuverability in the confined space of the Dutch rivers (see Fig. 7-6). Tugs of 3,000 to 4,000 horsepower are generally used.

Towing distances in the Netherlands have ranged from about 20 to 60 km. These distances allow the tow to be completed in a single day. More than 120 elements have been towed successfully through the Netherlands.

### *11.2 Second Hampton Roads Bridge Tunnel*

The Second Hampton Roads Bridge Tunnel is in the southern part of Chesapeake Bay, between Hampton and Norfolk, Virginia, U.S.A.

The elements of this tunnel were steel, double-shell, single-tube elements approximately 100 m long. The elements were launched with keel concrete to give them stability. They were towed, floating under their own buoyancy, from the Wiley Shipyard at Port Deposit, Maryland, through Chesapeake Bay to a pier at Norfolk, Virginia for outfitting.

The elements were then completed by casting all of the internal concrete (including the roadway slab but excluding the sidewalks, which are added after the elements are in final position on the bottom). Final ballast to load the element to placement weight was added at the tunnel site. In this way, the draft for the 300-km trip was only approximately 2.5 m. After outfitting, the draft increased to approximately 10 m, and with the element at final placement weight hanging in the placement barges, the draft was close to 12 m. Two tugboats in front of the element, arranged in tandem on a long tow line, were used for the tow.

Although Chesapeake Bay is an inland waterway, a tow can be rather exposed. During a storm or frontal passage, winds can become quite strong and wave height can reach 2 to 3 m. In total, 21 elements were towed without incident or damage.

### *11.3 Fort McHenry Tunnel*

The Fort McHenry Tunnel is located in Baltimore, Maryland, U.S.A. This tunnel was constructed by placing two steel, four-lane, double-shell binocular tunnels consisting of 16 elements each, side by side in the same trench, approximately 3 m apart.

The elements were fabricated at the Wiley Shipyard at Port Deposit, Maryland. After being side-launched from two slipways into the Susquehanna River, they were towed from Port Deposit to Baltimore. For this tow, they were stabilized with keel concrete and used their own buoyancy.

The tow was an easy, short trip of only 60 km in the upper, more sheltered reaches of Chesapeake Bay. The only difficulties encountered involved squeezing the wide, curved elements between the piers of several bridges at the mouth of the river (see the cover photo of this issue). In one particularly bad winter, the upper bay froze solid and impeded the transport of elements for a few weeks.

During the tow, the draft was approximately 2.5 m to the outfitting pier adjacent to the site. For the western end of the project, draft was temporarily restricted because of the presence underwater of an existing water main to be later relocated as part of the project. Consequently, the early elements were outfitted internally only, limiting their draft to approximately 9 m until they were over the trench, after which they could be ballasted to their full 12-m draft.

### *11.4 Monitor-Merrimac Tunnel*

The Monitor-Merrimac Tunnel is located between Portsmouth and Newport News, Virginia, at the point where the James River enters Hampton Roads, an estuary off Chesapeake Bay. This tunnel consists of 15 steel, double-shell, binocular-type elements.

This marked the first time that steel shell elements were constructed with all the internal concrete inside a dry dock. The elements were fabricated in groups of four inside the VLCC (Very Large Crude Carrier) drydock at the Bethlehem Steel Shipyard (Bethship), Sparrows Point, Maryland.

Because the dock avoided the stresses of launching, all of the interior concrete (except the sidewalks) could be placed while the elements were still in the dock. After being floated out with a draft of about 7.5 m, the elements were towed through Chesapeake Bay to the outfitting pier at Newport News, Virginia, where the ballast concrete was added just before placement. No difficulties were encountered at this draft during this inland tow down the bay. For the tow, a main 5,500 horsepower tug at the bow and a second 3,000 horsepower steering/standby tug at the stern were used. The tow took about three days.

*Figure 7-6. Typical marine spread.*

### *11.5 Western Harbor Crossing*

The tunnel elements are rectangular, concrete, box-type, non-segmented and non-prestressed. They were constructed in a purpose-built casting basin at Shek O Quarry. After being floated out, they were towed a distance of 15 km around Victoria Island to the outfitting location at Junk Bay. From there, a 12-km tow was required through inland waters to the tunnel site. Because this area is known for its typhoons, the tow and mooring facilities were designed to cater to these conditions. The requirements established are shown in Table 7-1.

The towing arrangement used the five-tug marine spread: two tugs at the bow and two at the stern, all pulling forward, and one at the stern, standing by to stop the forward motion of the element. The tug capacity ranged between 3,000 and 4,000 horsepower.

## 12. Examples of Offshore Transportation

While unnecessary offshore transportation is to be avoided, the examples given below show that both steel shell and concrete box tunnel elements have been transported over large distances by sea. More steel shell elements than concrete elements have been moved long distances in the open sea. In several cases, towing has used the elements' buoyancy, while in other cases submersible barges have been used to transport the elements.

Table 7-2 lists the main characteristics of various offshore tows. The remainder of this section describes transport aspects of each tunnel in more detail. Other details regarding these tunnel projects can be found in Chapter 9, "Catalogue of Immersed Tunnels."

### *12.1 63rd Street Tunnel*

The 63rd Street Tunnel crosses the East River between Manhattan and Brooklyn in New York City. This immersed tunnel was quite unusual. Single-shell steel elements were used that were almost square in cross-section (11.2 m x 11.7 m). Each element contained four tubes: two for the New York Central Railroad tracks, located above two tubes for Long Island Railroad Tracks. The elements were fabricated at the Wiley Shipyard at Port Deposit, Maryland, and side-launched into the Susquehanna River. They were then towed approximately 300 km to Norfolk,

*Table 7-1. Requirements for towing of tunnel elements for the Western Harbour Crossing.*

| Location | Distance | Condition | Wave Height |
|---|---|---|---|
| Around the island | 15 km | Ordinary | 1.6 m |
| | | Seek Shelter | 2.65 m |
| Sheltered tow | 12 km | Ordinary | 0.85 m |
| | | Seek Shelter | 1.3 m |
| Temporary mooring | | Typhoon | 3.5 m |

*Table 7-2. Main characteristics of various offshore tows.*

| Aspects | Tunnel Projects | | | | | |
|---|---|---|---|---|---|---|
| | 63rd St. | SDT | Sydney | Ted Williams | Piet Hein | Wijker |
| **Year** | 1973 | 1988 | 1990 | 1994 | 1995 | 1995 |
| **Country** | U.S.A. | U.S.A. | Australia | U.S.A. | Netherlands | Netherlands |
| **No. of Elements** | 4 | 8 | 8 | 12 | 8 | 6 |
| **Trips** | 4 | 4 | 8 | 12 | 8 | 6 |
| **Type** | Steel S/S | Steel D/S | Concrete | Steel D/S | Concrete | Concrete |
| **Shape** | Square | Single Tube | Rectangular | Double Tube | Rectangular | Rectangular |
| **Tow Configuration** | Keel | On Barge | Keel | On Barge | Keel | Keel |
| **Distance** | 500 km | 3,000 km | 85 km | 900 km | 175 km | 70 km |
| **Draft** | ± 9 m | ± 3 m | ± 7 m | ± 3 m | ± 7.8 m | ± 7.8 m |

Virginia where they were completed internally with concrete (walls, roof, and floor slabs), after which they were towed at almost full draft to the tunnel site. This second tow was approximately 500 km long along the east coastline of the U.S.A. The tow was made more hazardous by the deep draft of the fully outfitted elements; and, in fact, one element was accidentally grounded in the shipping channel and had to be dry-docked for repairs.

### *12.2 Second Downtown Tunnel (SDT)*

The Second Downtown Tunnel crosses the Elizabeth River between Norfolk and Portsmouth, Virginia. The shape of the elements was unusual for a steel double-shell tunnel. It was configured in an inverted horseshoe shape with a flat bottom. The elements were fabricated in Corpus Christi, Texas, on the Gulf of Mexico. Although they were designed so that they could be towed under their own buoyancy, the contractor chose to use a submersible barge to carry these elements. Eight single-tube elements were transported two at a time for the long 3,000-km voyage through the Gulf, across Florida and up to Norfolk.

### *12.3 Sydney Harbor Tunnel*

The Sydney Harbor Tunnel, in Sydney, Australia, was designed with non-segmented, non-prestressed elements. These were built in a special graving dock at Port Kembla. After float-out, the elements were towed on their own buoyancy along the coastline in the Pacific Ocean north to Sydney—a distance of about 85 km.

The design provided reinforcement sufficient to resist the stresses imposed by a maximum wave height of 3.5 m. Each of the eight elements was towed from Port Kembla to Sydney in a single day. The maximum allowable wave height was based on the assumption that a reliable forecast could be obtained up to 24 to 48 hours ahead of a scheduled tow. Two 3,000-horsepower bow tugs were used for the tow.

### *12.4 Ted Williams Tunnel*

The Ted Williams Tunnel, in Boston, Massachusetts, U.S.A., crosses the harbour to the city's airport. The 12 elements were fabricated in groups of two at Bethship Sparrows Point shipyard facility in Baltimore, in the same VLCC drydock that was used for the Monitor-Merrimac Tunnel described above. For this project, the contractor opted to transport the elements one at a time on a submersible barge. This decision was made because the voyage was approximately 1,100 km long. It involved, first, a 300-km tow through Chesapeake Bay, and then some 900 km out in the open Atlantic Ocean. Before towing, only the concrete in the invert plus the exterior keel concrete was placed at the shipyard.

The main reasons for transporting the elements by barge were to avoid:

- possible damage to internal concrete due to the flexing of the 105-m-long barge and the element on long period swells; and
- possible damage to the outer steel shell from wave impact.

Another advantage of the transport by barge was that it took much less time than the alternative. In good weather, the tow only took four days, whereas towing the elements on their own keels could have taken two or three times as long. In one instance, the tow had to take shelter in New York Harbor because of stormy conditions. In winter weather, the tow hugged the New Jersey coastline and passed through the Cape Cod Canal. In the summer, in order avoid the large number of small pleasure craft that use the canal in that season, the tow took the shortest route across the Atlantic Ocean from Hampton Roads Channel at Norfolk on a direct line to Boston, and bypassed the canal.

One large ocean-going tug was generally used (see Fig. 7-7), except where the tow was maneuvered through the canal. Then a second tug was used, lashed to the side of the barge. No incidents or problems occurred during the towing of the 12 elements. The tows often carried miscellaneous construction materials that Bethship was required by contract to supply. These materials, which were stowed

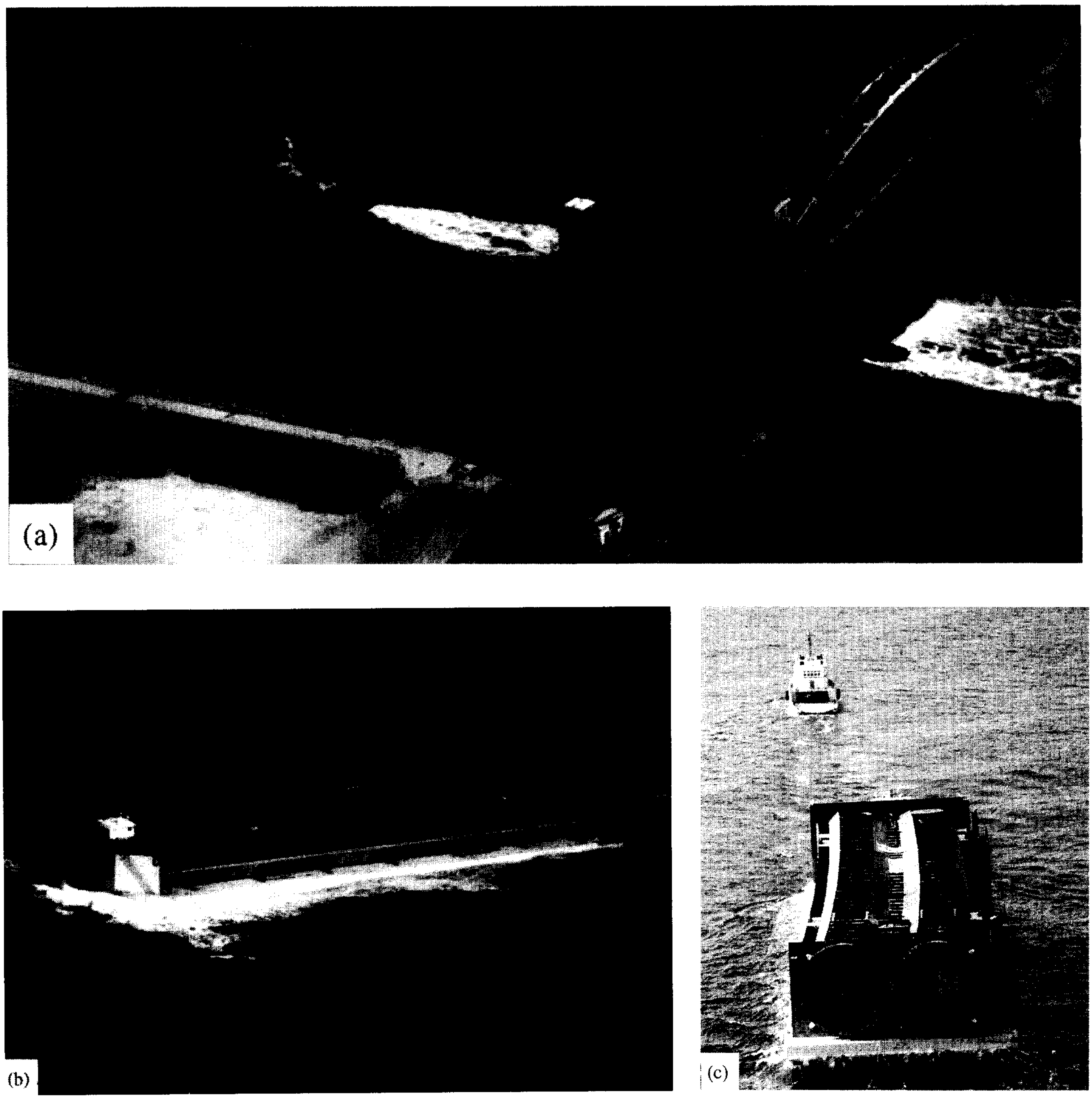


*Figure 7-7. Transport of Ted Williams Tunnel on barge.*

on top of the unit, included epoxy-coated reinforcing steel for the joint areas and the equipment and personnel snorkels used at Element No. 1 and Element No. 12 for temporary access.

## 12.5 Wijker Tunnel

The Wijker Tunnel, near Ijmuiden, Velsen, the Netherlands, is a segmented, reinforced concrete box section that was temporarily prestressed during transport. It was towed on its own buoyancy. The elements were built in the previously used Barendrecht graving dock, south of Rotterdam. The first part of the tow was through river estuaries within the tidal influence of the sea. The second part was over the North Sea, from Rotterdam to Ijmuiden—a distance of about 70 km. Arriving at Ijmuiden, the elements had to pass through a large lock at the entrance of the canal to Amsterdam.

The average duration of the tow was about 38 hours, 18 hours of which were at sea. Extensive studies had been made regarding the potential sea states that might occur, and especially the probable wave heights during the towing period. The elements were towed during the months of May, June, and July, and were designed for a maximum wave height of 2.0 m.

## 12.6 Piet Hein Tunnel

The Piet Hein Tunnel is in the eastern part of Amsterdam, the Netherlands. The tunnel, a permanently prestressed concrete box section, was towed on its own buoyancy. The elements were built in an existing graving dock in Antwerp,

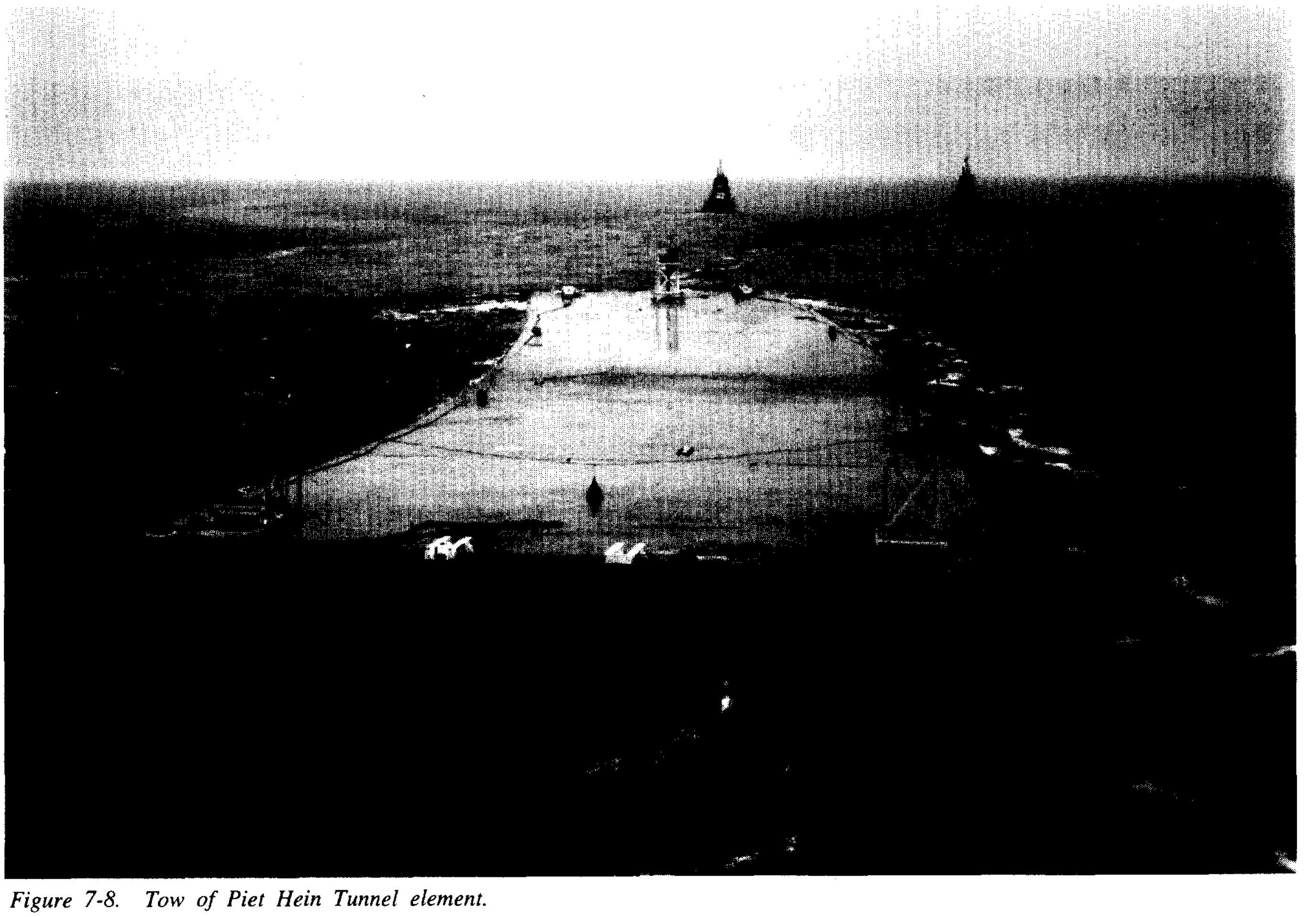


*Figure 7-8. Tow of Piet Hein Tunnel element.*

Belgium, making this the first tunnel to cross the international border between Belgium and the Netherlands.

The first part of the tow was through the basins of Antwerp. After passing a lock, the elements were towed down the Westerschelde, where large tidal influences govern. The second part of the tow was through the North Sea, from Vlissingen to Ijmuiden—a distance of about 175 km (see Fig. 7-8). The total distance was approximately 300 km. Arriving at Ijmuiden, the elements had to pass through the large lock at the entrance of the canal to Amsterdam.

The duration of the total voyage was about 60 hours. Extensive studies were made of sea states that might occur and of the probable wave heights during the towing period. The elements were towed during the months of May, June, and July; they were designed for a maximum wave height of 2.0 m for normal conditions and 3.5 m for maximum conditions.

During the passage of narrow inland rivers and the canal, the elements were towed by two bow tugs of 60 tons bollard pull. Each tug was assisted by two stern tugs, each pulling 30 tons.

## 13. Conclusions

Approximately 150 immersed traffic and service tunnels have been designed and constructed in this century (see Chapter 9, "Catalogue of Immersed Tunnels," for details of individual tunnels). The safe transport of the elements for these tunnels during the construction period played a paramount role in the successful completion of these projects. In Holland, where early elements were mostly transported over inland waters, transport by sea has been successfully accomplished in recent projects. For most of the early history of steel shell immersed tunnels, the elements were transported long distances on their concrete-filled keels.

It should be noted that this situation is gradually changing over to a safer, faster method using large submersible barges. There are recent examples of concrete box tunnels being transported offshore as well, requiring that anticipated sea states are properly appraised and that adequate structural reinforcement is provided. Continuing advancements in design and construction techniques have made this possible. With no limits to the distance immersed elements can be transported, immersed tunnels can be built anywhere in the world and moved anywhere else. Errata of Chapter 7 is included at the end of the issue.

*(Please note that the Corrigenda for this chapter are printed on p. 355 of this issue)*

PII:S0886-7798(97)00012-6

# Chapter 8

# EARTHQUAKE ANALYSIS

by

CHRISTIAN INGERSLEV U.S.A.
Parsons Brinckerhoff Quade & Douglas, Inc.

OSAMU KIYOMIYA Japan
Port and Harbour Research Institute,
Ministry of Transport

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.157–162, 1997

Printed in Great Britain
0886–7798/97 $17.00+0.00

Pergamon

# Chapter 8: Earthquake Analysis


1. INTRODUCTION

2. SEISMIC EVENT MAGNITUDE

3. GROUND FAILURE DURING EARTHQUAKE

4. LOADS TO BE COMBINED WITH EARTHQUAKES

5. SEISMIC COEFFICIENTS

6. "FREE FIELD" GROUND DEFORMATION

7. ANALYSIS OF SOIL-STRUCTURE INTERACTION

8. HANNSHIN EARTHQUAKE OF 1995 AND DAMAGE TO OSAKA IMMERSED TUNNEL UNDER CONSTRUCTION

9. LOMA PRIETE EARTHQUAKE OF 1989 AND THE BAY AREA RAPID TRANSIT (BART) TUNNEL, CALIFORNIA

10. REFERENCES

## 1. Introduction

Earthquake loads, or more correctly seismic loads, are included among the loads on a structure that are required to be considered by most current design codes. Guidance is given in this section on the magnitudes to be selected for the seismic loads and how to apply them to an immersed tunnel. Many immersed tunnels have been constructed in areas where seismic effects are significant, such as in Japan, Taiwan and California. These tunnels have survived major earthquakes with little damage. All immersed tunnels should be designed for seismic effects appropriate to their location. Seismic effects can occur during the construction phase and should therefore also be considered; an appropriate level of risk should be agreed with the owner. For example, the Osaka tunnel in Japan was struck during construction by the Hannshin earthquake of 1995; this major earthquake caused little damage to the tunnel or the construction works..

The seismic hazard at a tunnel site can be quantified by a project-specific seismic hazard assessment. An estimate of the movements of the bedrock at the tunnel site can be made by considering:

- Historical records of seismicity
- Relevant seismic data
- Ground motion characteristics for the area
- Other recent seismic hazard assessments for nearby structures
- Geology at the site
- A study of active faults likely to produce a seismic event affecting the tunnel site and vicinity.

Using computer modeling, the effects of ground attenuation or amplification based on topography and geology at the site can be calculated by iteration. The effective movements at the tunnel due to the seismic event can then be determined. Care must be taken in translating the displacements into stresses in the tunnel. Because the shape of the tunnel as a whole under seismic loads is controlled by the surrounding soils, and because the tunnel is not free to move independently, its behavior is displacement controlled. The strains resulting from the imposed curvature can be determined and by using stress-strain relationships, the corresponding stresses can be deduced.

Within the tunnel, there may be walls and slabs that do not form part of the external surface in contact with soil. These elements, and any equipment installed in the tunnel, may not be constrained by the external soil in the same way. They may be free to move according to the effective imposed accelerations resulting from the imposed tunnel external displacement. The most critical acceleration will be that corresponding to the resonant frequency of such an element.

## 2. Seismic Event Magnitude

For the design of a tunnel, three magnitudes of earthquake loading are considered:

- The safety evaluation earthquake (SEE).
- The functional evaluation earthquake (FEE), also known as the design basis earthquake (DBE).
- A smaller Serviceability Limit State (SLS) earthquake, corresponding to a five- to ten-year return period, may also be included as an ordinary static live load to be combined with other live loads.

To use the loads caused by earthquakes with these magnitudes, the seismic hazard levels to which the tunnel is exposed must be characterized and quantified. The structural response to or performance with these loads must be defined and agreed upon with the owner. The appropriate design of the tunnel can be achieved only if the extent of cracking, movement, damage, formation of plastic hinges, etc., are defined, enabling corresponding allowable design stresses and displacements to be determined. Considering three levels of structural response is recommended, as given below. Neither collapse nor catastrophic inundation would be acceptable. Typical criteria are given for each, but these may vary from owner to owner, from project to project, and according to the relative importance of the tunnel in the region's transportation network:

- **Minimum performance level:** Significant damage, sometimes but not always reparable; damage may require full closure or replacement; flooding on roadway limited to that passable by emergency vehicles and slow-moving traffic, limited lighting and ventilation.
- **Limited performance level:** Intermediate damage, repairable over 12 months for example; limited emergency and public traffic allowed within hours; limited leakage; full lighting and ventilation.
- **Full performance level:** Light damage; minor repairs; public traffic allowed immediately; minor leaks; effectively no significant loss of service.

The SEE is concerned not only with life safety, but also with the survivability of the structure under the most severe seismic event considered at the location, through the use of the Progressive Collapse Limit State (PLS). The level of probability of this event is considered to be comparable to that for collapses of dams and bridges, flooding resulting from breach of dikes, etc. Commonly accepted values for the return period of such an event lie in the range of 1 to 2000 years. A longer return period may be appropriate where large earthquakes occur less often. Instead of this return period, it may sometimes be appropriate to select the maximum credible earthquake (MCE), i.e., the maximum foreseeable earthquake,

A performance level such as those suggested above must be defined, to which the tunnel will be designed when subjected to the SEE. Typically, an FEE would be used first to design the structure for either the limited or the full performance level, following which the SEE would be checked to ensure compliance with minimum performance. If necessary, the strength of some parts of the structure may have to be enhanced to comply. The return period or magnitude selected for SEE and the corresponding performance level should be defined in the design parameters for each project. For SLS and ULS, the structural components are checked against conventional elastic limits of the materials while for PLS the resources of ductility of individual members or the structure as a whole are exploited.

As stated above, the FEE is concerned with the design of the structure, especially for the Ultimate Limit State (ULS). The magnitude of the FEE should correspond to that typically required by structural design codes, such as an earthquake occurring once in a period equal to one to three times the design life (a 100- to 300-year return period). A performance level such as those suggested above must be defined, to which the tunnel will be designed when subjected to the FEE.

Depending upon the performance level selected, it may also be appropriate in some cases to consider the Serviceability Limit State (SLS) with the FEE. Not all combinations of earthquake and performance level may be useful to consider. The strategic importance of the tunnel route, the availability of alternative routes, the risks that the owner is prepared to carry, and cost may decide which combinations are used for design.

Occasionally, checking the SEE against the limited performance level may be worthwhile. The additional costs may be outweighed by other more pressing factors such as public confidence and the use of vital emergency routes. The return period or magnitude selected for FEE and the corresponding performance level should be defined in the design parameters for each project.

## 3. Ground Failure During Earthquakes

Immersed tunnels are generally embedded close to the surface in ground that is usually soft. Dynamic characteristics of immersed tunnels differ from those of surface structures. The displacements of a tunnel resulting from a seismic event depend largely on the behavior of the surrounding ground and its stiffness. Some movements may be large enough for limiting friction at the surface of the tunnel to be exceeded, causing slippage between the tunnel surface and the soil. Displacements of the soft surface layers may be amplified by the shear vibration (liquefaction) caused by the seismic event.

Records of local ground failures caused by historical seismic events can be very important to understanding potential ground behavior at the location. Evidence of fault dislocations, sliding of slopes, and liquefaction can be vital. For an immersed tunnel, ground failure can take place if complete or significant loss of soil strength occurs. Very important to the behavior of a tunnel are changes in the underlying topography and geology, such as part of a tunnel founded directly on rock, sudden changes in overburden, fault lines, or shear zones.

Liquefaction of soils around an immersed tunnel should be avoided. The liquefaction potential should be examined, taking into account any records of liquefaction having occurred. If a risk of liquefaction exists, replacement of these soils by others that will not liquefy should be considered at the design phase, especially for backfill. A less desirable alternative would be to implement measures to improve the existing ground. Appropriate methods would include densification, sand or stone columns which provide both densification and relief of excess pore water pressure, and jet or fracture grouting.

Loose sandy soils are particularly vulnerable to liquefaction, the consequences of which may include:

- Loss of lateral or vertical support;
- Movements or rotations of the tunnel, which may be large (6 to 8 m movement horizontally was recorded at Niigata, Japan, in 1964).
- The tunnel floating up in the soil, which is behaving like a liquid with an effective density of around 20 kN/m$^3$, much greater than that of the tunnel, which is usually approximately 10.5 kN/m$^3$ when backfill is ignored.

## 4. Loads to be Combined with Earthquakes

The magnitude and combinations of loads should generally be based upon local codes. Load combinations should be selected with regard to simultaneous probability. Where structural codes do not consider the particular conditions that apply to immersed tunnels or their approaches, the following classifications for applied loadings could be considered:

(1) **Dead loads**, to include all long term loads, earth loads and mean water level (static loads).

(2) **Live loads**, to include creep, shrinkage, prestress, temperature, backfill (the effects of which can vary with temperature, compaction, etc.), erosion of the seabed, siltation, traffic, and variations (considering appropriate static and dynamic effects), each with an annual probability of being exceeded of 0.2 or greater, of water level (including waves), current, storm loads and smaller earthquakes.

(3) **Exceptional loads**, including loss of support (subsidence) below the tunnel or to one side, occurring as a result of a major seismic event.

(4) **Extreme loads**, including, where appropriate, the design basis earthquake (DBE or FEE) predicted for the location, and the resulting movement of soils. The design basis earthquake should have a probability of being exceeded not more than once during the design life. [For SEE, see (e) below.]

(5) **Construction loads**, including temporary structures (e.g., sheet piles) and loads due to handling, transporting, and placing, combined with environmental loads appropriate to the season, duration of use, and location. Abnormal and extreme conditions may be included, but usually with an appropriately lower magnitude (a greater probability of occurrence), for example, not more than once in 10 years.

Application of the above load classifications should be considered with due regard to simultaneous probability. For example, extreme live loads would be inappropriate with an extreme earthquake. At a minimum, the following conditions suggesting how to combine the above loads should be considered during seismic analysis and design:

(a) *Normal operating conditions*, with the tunnel operation unaffected by environmental and smaller seismic loads.

(b) *Abnormal conditions,* showing that either a tunnel with live loads can remain operational under the aftereffects of extreme or exceptional loads (but not flooded) or settlement, or that with operations ceased and closed to traffic, the tunnel can survive some loss of support beneath or to one side.

(c) *Extreme actions,* including the abnormal conditions above combined with an extreme load. Tunnel operations may have ceased and the tunnel may be closed to traffic.

(d) *Construction conditions.*

(e) *Life safety*, an ultimate check using the safety evaluation earthquake (SEE or MCE).

## 5. Seismic Coefficients

The effects of a seismic event on a tunnel as a whole can be integrated to give an effective acceleration at the tunnel location, expressed as a seismic coefficient times the acceleration due to gravity. Three seismic coefficients are usually obtained, for longitudinal, lateral and vertical effects. Where internal structural members or installations have natural frequencies in the same range as the earthquake, such items may be subject to much greater seismic coefficients than the tunnel as a whole. These coefficients might even exceed unity. Typical items for which seismic coefficients would be used for design include flexible ceiling, floor and wall slabs, jet fans and suspended pipes, ground stability (sliding), ventilation and other buildings, open approaches, and mechanical systems.

## 6. "Free-field" Ground Deformation

The effects of a seismic event at a fault will be attenuated over the distance between the fault and the bedrock below the tunnel. The attenuation can be determined using published data and by performing deterministic maximum magnitude earthquake and source-to-site distance estimates. By considering wave propagation through the soil overburden between the bedrock and the tunnel location, using programs such as SHAKE (Schnabel et al. 1972, Idriss and Sun 1992), spatially varying ground motion response spectra for input to the tunnel seismic analyses may be determined.

The overall behavior of a tunnel below ground is more sensitive to the distortions of the surrounding ground than to inertial effects (Wang 1991). Ground movements surrounding a tunnel may be determined without considering the tunnel. Ignoring the presence of the tunnel, these distortions of the ground, known as "free-field" ground deformations, are caused by the traveling seismic waves. Simplified sinusoidal wave propagation techniques are generally used in computing, assuming homogeneous, isotropic, elastic media. A conservative assumption using the most critical angle of incidence is generally used to determine the maximum strain. Determining the maximum strain based on the actual angle is unreliable if not impossible. The equations given below may be used to estimate the maximum ground deformation values. For a shear wave with an angle of incidence of $\theta$, a longitudinal strain $\varepsilon$ and curvature $\kappa$ can be expressed (Schnabel et al. 1972) as:

$$\varepsilon = \frac{V_s}{C_s} \sin\theta \cos\theta$$

$$\kappa = \frac{A_s}{C_s^2} \cos^3\theta$$

where $A_s$ = peak particle acceleration for the shear wave,
$C_s$ = effective propagation velocity for the shear wave, and
$V_s$ = peak particle velocity for the shear wave.

Equations for the Rayleigh or surface wave using similar nomenclature (subscript R) can be expressed as:

$$\varepsilon = \frac{V_R}{C_R} \cos^2\theta$$

$$\kappa = \frac{A_R}{C_R^2} \sin\theta \cos\theta$$

The maximum longitudinal strains will be induced in the tunnel when the seismic wave interacts with the tunnel at 45°. Peak longitudinal strains, assuming no slippage between the tunnel and the surrounding ground, may be estimated by adding the axial strains to the bending strains arising from the snaking of the tunnel. If the overburden pressure is such that friction limits the seismic effects that can be transferred axially to the tunnel, the axial strain limit may be easily calculated from the limiting frictional force.

Conversion of the applied strains in each direction into forces requires an understanding of the structural behavior of the tunnel. Since both steel and concrete immersed tunnels with similar external dimensions contain similar quantities of concrete, it is the behavior of the concrete under tension that will greatly influence overall behavior. Unless it is highly compressed or prestressed, the concrete will crack under large seismic loads. (By definition, a reinforced concrete section is designed assuming that the concrete is cracked.)

The stress and strain distribution across a section is significantly different for cracked and uncracked concrete, and the position of the neutral axis will change as the curvature of the tunnel reverses. Therefore, direct consideration of stresses in an uncracked purely elastic model may not yield particularly useful results.

The space in the ground occupied by the tunnel is small compared with the volume of the ground as a whole. Nevertheless, depending upon the rigidity of the ground, the wavelength of the earthquake, and other factors, the tunnel may be stiff or very stiff relative to the soil. The anticipated deformations of the tunnel could therefore be much less severe than deformation of the surrounding soils.

## 7. Analysis of Soil-structure Interaction

The response of the tunnel to the free-field soil displacements must consider the stiffness of both the tunnel and the soil. For a tunnel in rock, tunnel deformations will match those of the rock; however, in softer soils the tunnel will resist soil pressures. While this complex analysis may be solved numerically using computers, some simplified procedures have been published; the analysis method of St. John and Zahrah (1987) is one of the more commonly used. Both their method (ITA 1993) and that of the Japanese (JSCE 1988) may be very conservative if the free-field ground displacement response amplitude decreases with wavelength rather than being independent of wavelength, as they have assumed. The tunnel ground system is represented as an elastic beam on an elastic foundation, with traveling waves in the ground. Axial and curvature deformations imposed on the ground will generate smaller deformations in the tunnel, from which axial loads, moments and shears on the tunnel can be calculated.

When the tunnel is treated as a beam connected to the free field by springs, the free-field ground displacements may be input as imposed displacements to the springs. The force that can be transmitted to the tunnel may be limited by the frictional capacity of the ground, although this may be difficult to model. The resulting strains imposed on the tunnel structure can then be used to determine stress by taking into account appropriate section properties of the tunnel. These section properties may change with applied strain, for example if concrete sections crack, or with varying axial loads. The movements of the tunnel caused by the soil-structure interaction will be smaller than the free-field soil motions input as boundary conditions to the model. The tunnel motion time-histories so developed can be used to assess the relative motion expected at interfaces between tunnel structures.

Spring stiffnesses used with a beam model must be chosen with care to ensure that the model behavior best reflects the actual soil behavior expected. This behavior might be nonlinear and require the secant stiffness. Because of the combination of free field and soil-tunnel interaction, values of stiffness would be expected to lie below the in-place low-strain soil properties.

The spacing of supports longitudinally in a beam model should be selected by taking into account the location of the structural joints (ends of units), the distance between them, and the overall depth of the section. Excessive numbers of supports contribute little to the analysis. Effects of flexible joints and of ventilation structures can also be modeled. Analysis results should be reviewed to check that behavior is as expected, or else rerun after correction of data, since assessing appropriate values of stiffness is difficult.

Having determined overall behavior of the tunnel, the calculated pressures and strains may be applied to a slice of the cross-section to determine local behavior, such as asymmetric distortion (ovalling or racking) of the section.

Another critical mode of the section response is the racking or ovalling effects caused by vertically propagating shear waves. For shallow tunnels, typical of immersed and cut-and-cover tunnels, these effects should be determined using ground response analyses such as SHAKE to estimate the free-field ground strains.

Quasi-static loading may be used for the analysis of the tunnel itself if the dynamic amplification of stresses associated with the stress waves is small. This will be the case if the rise time for a pulse is more than about twice the transit time for a pulse to cross the tunnel, i.e., if the dominant wavelengths exceed about eight times the width of the tunnel. Approximate wave lengths may be estimated using:

$$\lambda = \frac{c_s}{2\pi f}$$

where $\lambda$ = wave length
f = frequency of vibration of peak ground motions (two cycles/sec for El Centro)

## 8. Hannshin Earthquake of 1995 and Damage to Osaka Immersed Tunnel under Construction

The rectangular, concrete, three-cell Osaka Immersed Tunnel was still under construction when the Hannshin Earthquake hit on January 17, 1995. Six of the ten elements and two ventilation buildings had been constructed. The magnitude of the earthquake was 7.2 on the Richter scale, and the distance from the tunnel to the epicenter was about 15 km. The maximum ground acceleration recorded in the Osaka area was 0.266 g, almost equal to the design value;

the tunnel was undamaged and no residual cracking was evident. After the earthquake, displacements of 20–30 mm were observed at every flexible joint, and the Omega type rubber seals at every joint were twisted, symptomatic of some residual displacements. These were considered minor defects, and construction continued without any repairs being effected. A mass-spring model (beam model) was used in the tunnel seismic design.

## 9. Loma Prieta Earthquake 1989 and the Bay Area Rapid Transit (BART) Tunnel, California

The Loma Prieta Earthquake of October 17, 1989 registered 7.1 on the Richter scale and was centered about 70 miles south of the BART tunnel, in the Santa Cruz mountains of California. The earthquake caused extensive damage in both San Francisco and Oakland, including the collapse of a section of the San Francisco to Oakland Bay bridge.

BART facilities sustained very little damage and continued to operate on a normal schedule after a 24-hour shutdown for inspection. During the earthquake, a train was traveling southbound in the tunnel, and it was reported that the passengers and operator were not aware of the earthquake.

Following the Loma Prieta earthquake, the special seismic joints provided at each end of the tunnel were inspected. Not only had the joints moved in accordance with design during the earthquake, but, it was concluded, they would likely survive another earthquake equal to a maximum credible earthquake on the Hayward Fault or the San Andreas Fault. The tunnel itself sustained negligible damage in comparison to surrounding structures. The original 1960 analyses of the tunnel section for earthquake have been updated since the Loma Prieta Earthquake; their basic conclusion is that the tunnel is safe.

## References


Idriss and Sun. SHAKE analysis. 1992.

International Tunnelling Association Working Group on Immersed and Floating Tunnels. 1993. *Tunnelling and Underground Space Technology* **8**(2): Special Issue on Immersed and Floating Tunnels.

Japan Society of Civil Engineers. 1988. *Specification for Earthquake-Resistant Design of Immersed Tunnels.* Tokyo: JSCE.

Schnabel, Lysmer, and Seed. 1972. SHAKE—A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites. EERC Report No. 72-12, University of California at Berkeley.

St. John, C. and Zahrah, A. 1987. Aseismic design of underground structures. *Tunnelling and Underground Space Technology* **2**(2):165–197.

Wang, Jaw-Nan (Joe). 1991. "Seismic Design of Tunnels, a Simple State-of-the-Art Approach." 1991 William Barclay Parsons Fellowship, Parsons Brinckerhoff Monograph 7, June 1993.

PII:S0886-7798(97)00013-8

# Chapter 9

# CATALOGUE OF IMMERSED TUNNELS

by

**NESTOR RASMUSSEN** — Denmark
**Consultant**

**WALTER GRANTZ** — U. S. A.
**Chesapeake Bay Bridge and Tunnel District**

***Graphics by:***

**Arvall K. Wroten** — U.S.A.
**Chesapeake Bay Bridge and Tunnel District**

***Contributions and comments for 1997 edition by:***

| | |
|---|---|
| **Chris Hakkaart** | **The Netherlands** |
| **Liong Tan** | **The Netherlands** |
| **Goichi Fukuchi** | **Japan** |
| **Christian Ingerslev** | **U.S.A.** |


*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.163–316, 1997

Printed in Great Britain
0886-7798/97 $17.00+0.00

Pergamon

# Chapter 9: Catalogue of Immersed Tunnels

**1. ITA CATALOGUE OF IMMERSED SERVICE TUNNELS**

1.1 Reference list of immersed service tunnels

Individual immersed service tunnel listings

**2. ITA CATALOGUE OF IMMERSED TRANSPORTATION TUNNELS**

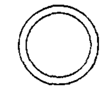
2.1 Reference list of immersed transportation tunnels

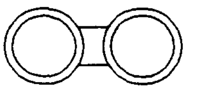
Individual immersed transportation tunnel listings

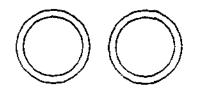
## 1. ITA Catalogue of Immersed Service Tunnels

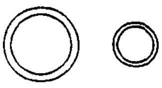
### *1.1 Reference List of Service Tunnels*

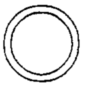
| Ref. No. | Tunnel / Conduit / Culvert | Country | Year Completed |
|---|---|---|---|
| S.1 | Sewer Culvert under Boston Harbour | U.S.A. | 1893 |
| S.2 | Sewer Culvert under the Seine,Paris | France | 1893 |
| S.3 | Culvert under the Harbour at Nyhavn, Copenhagen | Denmark | 1900 |
| S.4 | Culvert under the Harbour at Langebro, Copenhagen | Denmark | 1900 |
| S.5 | Culvert under the Seine, Paris | France | 19— |
| S.6 | Culvert under Canal, Karlsruhe | Germany | 19— |
| S.7 | Double Culvert under the Rhine, Cologne | Germany | 1929 |
| S.8 | Sewer Outfall, Buenos Aires | Argentina | 1930 |
| S.9 | Sewer Outlet, Gentofte | Denmark | 1939 |
| S.10 | Sewer Outlet, Scheveningen | Netherlands | 19— |
| S.11 | Sewer Tunnel, Durban | South Africa | 1960 |
| S.12 | Syphon under the Nile, Cairo | Egypt | 1964 |
| S.13 | Multi L.T. Cable Tunnel, Stockholm | Sweden | 1965 |
| S.14 | Siphon de la Mutatiere, Lyon | France | 196- |
| S.15 | Water Intake Tunnel, Marsden | New Zealand | 1967 |
| S.16 | Atsumi Karyoku Tunnel, Atsumi | Japan | 1970 |
| S.17 | Dokai Bay Conveyor Tunnel | Japan | 1972 |
| S.18 | Dokaiwan Conveyor Tunnel | Japan | 1972 |
| S.19 | Calvert Cliffs Outfall, Maryland | U.S.A. | 1972 |
| S.20 | Amsterdam-Rhine Canal Tunnel | Netherlands | 1973 |
| S.21 | Hollandsch Diep Service Tunnel | Netherlands | 1973 |
| S.22 | District Heating Tunnel, Odense | Denmark | 1974 |
| S.23 | Oude Maas Service Tunnel, Heinenoord | Netherlands | 1975 |
| S.24 | Cooling Water Outfall, Unterweser | Germany | 1975 |
| S.25 | LNG Terminal Tunnel, Cove Point | U.S.A. | 1976 |
| S.26 | Dokai Bay Gas Pipeline Tunnel | Japan | 1977 |
| S.27 | Cooling Water Outfall, Blayais Power Station | France | 1978 |
| S.28 | Cooling Water Outfall, Kilroot | United Kingdom | 1978 |
| S.29 | Conveyor Belt Tunnel, Bakar Golf | Yugoslavia | 1978 |
| S.30 | Syphon Middellandskanal Bramsche | Germany | 1978 |
| S.31 | Underwater Outlet, Lake Piru, California | U.S.A. | 1978 |
| S.32 | Cooling Water Outfall, Halileh Persian Gulf | Iran | 1979 |
| S.33 | Sewer Outfall, Portsmouth Harbour | United Kingdom | 1979 |
| S.34 | Cooling Water Intake, Dora Creek, Lake McQuarle | Australia | 1981 |
| S.35 | Keihin Conveyor Tunnel | Japan | 1981 |
| S.36 | Landing Tunnel, Karmoey | Norway | 1983 |
| S.37 | Hase Tunnel at Bramsche Canal | Germany | 1984 |
| S.38 | Cooling Water Intake, Brokdorf | Germany | 1984 |
| S.39 | Jurong Strait Utility Tunnel | Singapore | 1986 |
| S.40 | Sizewell Water Intake and Outfall | United Kingdom | 1992 |
| S.41 | Cooling Water Culverts, South Humber Bank | United Kingdom | 1996 |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Sewer Culvert under Boston Harbour; Boston, U.S.A.; 1893** | | | **S.1 - Boston Sewer**<br> |
| TUNNEL TYPE AND USE:<br>Sewer Culvert | | TUBES:<br>One tube | |
| NO OF ELEMENTS: | LENGTH: | HEIGHT: 2.75 m OD circular | WIDTH: |
| TOTAL IMMERSED LENGTH: | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD: | | JOINT TYPE: | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | | | |
| FOUNDATION METHOD: | | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | | | |

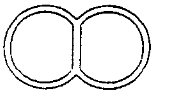
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Sewer Culvert under the Seine; Paris,France; 1893** | | | **S.2 - Paris Sewer**<br> |
| TUNNEL TYPE AND USE:<br>Steel plate; sewer culvert | | TUBES:<br>Two tubes | |
| NO OF ELEMENTS: | LENGTH: | HEIGHT: dia 1.0 m | WIDTH: circular |
| TOTAL IMMERSED LENGTH: 202 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>The culvert was built on a bascule slip which was horizontal during the construction and could be slanted by means of jacks when launching the culvert. | | JOINT TYPE: | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | | | |
| FOUNDATION METHOD: | | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | | | |

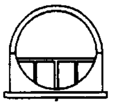
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Culvert under the Harbour at Nyhavn; Copenhagen, Denmark; 1900** | | | **S.3 - Nyhaven Sewer**<br> |
| TUNNEL TYPE AND USE:<br>Steel shell, syphon for waste water | | TUBES:<br>Two unconnected tubes | |
| NO OF ELEMENTS:<br>One for each pipe | LENGTH:<br>43 m approximately | HEIGHT: 0.75 m ID | WIDTH: (circular) |
| TOTAL IMMERSED LENGTH: 43 m | | DEPTH AT BOTTOM OF STRUCTURE: 10.6 m | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | Moderate tidal current | | |
| FABRICATION METHOD:<br>Entire element built of two concentric riveted wrought iron shells on shore. After culvert had been submerged the space between shells was filled with cement grout. Element launched by means of two floating cranes (fixed A-frame type) | | | |
| WATERPROOFING METHOD: | Not applicable | | |
| PLACEMENT METHOD: | By two barges with A-frame type hoisting gear. Buoyancy provided by oil drums. Water ballast was charged into central pipe and into space between inner and outer shell. | | |
| FOUNDATION METHOD: | Sand fill | | |
| DREDGING METHOD: | By dredge. Part of trench required blasting of sand stone with flint layers. | | |
| COVER AND TYPE: | 0.3 m thick layer of concrete filled bags. | | |
| ADDITIONAL INFORMATION: | OWNER: Municipality of Copenhagen<br>DESIGNER: Owner<br>CONTRACTOR: N C Monberg | | |

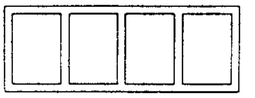
<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Culvert under the Harbour at Langebro; Copenhagen, Denmark; 1900</td><td>S.4 - Langebro Sewer<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Steel shell, syphon for waste water</td><td colspan="2">TUBES:<br>Two unconnected tubes</td></tr>
<tr><td>NO OF ELEMENTS:<br>One for each pipe</td><td>LENGTH:<br>185 m approximately</td><td>HEIGHT: One 1.20 m ID<br>One 0.75 m ID</td><td>WIDTH: (circular)</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 185 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 11.2 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3"></td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Moderate tidal current</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>Inclined and horizontal part built of two concentric riveted wrought iron shells on slipway. Inclined end sections were joined with horizontal part in a dry dock. After culvert had been submerged the space between shells was filled with cement grout.</td><td colspan="2">JOINT TYPE:<br>Riveted</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Not applicable</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">By two barges with A-frame type hoisting gear. Buoyancy provided by oil drums. Water ballast was charged into central pipe and into space between inner and outer shell.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Sand fill</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">By dredge. Part of trench required blasting of sand stone with flint layers.</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">0.3 m thick layer of concrete filled bags.</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: Municipality of Copenhagen<br>DESIGNER: Owner<br>CONTRACTOR: N C Monberg</td></tr>
</table>

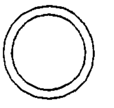
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Culvert under the Seine; Paris, France; 19xx** | | | **S.5 - Paris Sewer**<br> |
| TUNNEL TYPE AND USE:<br>Steel shell, culvert | | TUBES:<br>One tube | |
| NO OF ELEMENTS: 1 | LENGTH: 80 m | HEIGHT: 0.40 m ID | WIDTH: (circular) |
| TOTAL IMMERSED LENGTH: 80 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>The 80 m long element was made up of 11 segments, each 7 m long. Segment was fabricated from steel plate 10 mm thick. Segments were riveted together on a transverse slipway and launched as a whole. | | JOINT TYPE: | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | | | |
| FOUNDATION METHOD: | | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | | | |

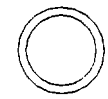
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Culvert under Canal; Karlsruhe, Germany; 19xx** | | | **S.6 - Karlsruhe Sewer**<br> |
| TUNNEL TYPE AND USE:<br>Culvert | | TUBES:<br>Two tubes, each 1.80 m ID | |
| NO OF ELEMENTS: 2 | LENGTH: 44 m | HEIGHT: 2.65 m | WIDTH: 6.30 m |
| TOTAL IMMERSED LENGTH: 88 m | | DEPTH AT BOTTOM OF STRUCTURE: 8.75 m | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>Elements built in dock | | JOINT TYPE:<br>Tremie concrete | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | Element was lowered from two trestles with cross-beams, one at each end | | |
| FOUNDATION METHOD: | | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | 1 m sand covered by 1 m rock fill | | |
| ADDITIONAL INFORMATION: | | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Double Culvert under the Rhine; Cologne, Germany; 1929** | | | **S.7 - Cologne Sewer**<br> |
| TUNNEL TYPE AND USE:<br>Culvert | | TUBES:<br>Two tubes | |
| NO OF ELEMENTS: 11 | LENGTH: | HEIGHT: 1.85 m dia<br>1.25 m dia | WIDTH: |
| TOTAL IMMERSED LENGTH: 341.50 m | | DEPTH AT BOTTOM OF STRUCTURE: 16 m | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>Elements were constructed on temporary fitting-out jetty | | JOINT TYPE: | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | Gravel ballast was used to bring pipe-assembly into a horizontal position. Element ends were placed on ten foundation blocks, 4.35 m long, 1.30 m wide and 2.0 m high. | | |
| FOUNDATION METHOD: | See above | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Sewer Outfall; Buenos Aires, Argentina; 1930** | | | **S.8 - Buenos Aires Sewer**<br> |
| TUNNEL TYPE AND USE:<br>Sewer outfall | | TUBES:<br>One tube | |
| NO OF ELEMENTS: 61 | LENGTH: 15 m | HEIGHT: Internal 2.25 m | WIDTH: 6 m |
| TOTAL IMMERSED LENGTH: 915 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD: | | JOINT TYPE: | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | **Side launching, towing to site and placement under temporary jetty from which element was lowered by means of gantry cranes.** | | |
| FOUNDATION METHOD: | | | |
| DREDGING METHOD: | Floating dredge | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Sewer Outlet; Gentofte, Denmark; 1939** | | | **S.9 - Gentofte Sewer**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box; Sewer outfall | | TUBES:<br>Four tubes 0.90 x 0.65 m | |
| NO OF ELEMENTS: 100 | LENGTH: 15 m | HEIGHT: 1.10 m | WIDTH: 3.00 m |
| TOTAL IMMERSED LENGTH: 1,500 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>Elements were constructed on a slipway situated in neighboring harbour. | | JOINT TYPE:<br>Hinges and packings at ends of elements. | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | Element was towed to site with 15 cm freeboard and lowered by use of two gantry cranes installed on boats. | | |
| FOUNDATION METHOD: | On piles in soft soil, on grillages/gravel beds in hard ground. | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Sewer Outlet; Scheveningen, The Netherlands; 19xx** | | | **S.10 - Scheveningen Sewer**<br> |
| TUNNEL TYPE AND USE:<br>Steel shell; Sewer drain | | TUBES:<br>One tube | |
| NO OF ELEMENTS: | LENGTH: | HEIGHT: 2.20 m dia | WIDTH: (circular) |
| TOTAL IMMERSED LENGTH: | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | Trench was protected against filling with sand by means of sheet pile retaining walls. | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>Elements were welded together using 15 mm thick steel plate, protected against corrosion on outside by means of asphalt, asbestos and coco-fibre, on the inside by asphalt and asbestos only. | | JOINT TYPE: | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | By two gantry cranes running on a temporary fitting-out jetty. | | |
| FOUNDATION METHOD: | | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | 2 m cover | | |
| ADDITIONAL INFORMATION: | | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | S.11 - Durban Sewer |
|---|---|
| **Sewer Tunnel; Durban, South Africa; 1960** |  |

| TUNNEL TYPE AND USE: | TUBES: |
|---|---|
| Segmental concrete tunnel, sewer | One tube |

| NO OF ELEMENTS: 5 | LENGTH: 2 - 52.1 m<br>2 - 44.8 m<br>1 - 43.4 m | HEIGHT: 4.6 m OD | WIDTH: (circular) |
|---|---|---|---|

| TOTAL IMMERSED LENGTH: 237.3 m | DEPTH AT BOTTOM OF STRUCTURE: 20.3 |
|---|---|

| | |
|---|---|
| UNUSUAL FEATURES: | Three tunnel elements were placed horizontally across the channel and joined to the two terminal chambers by the remaining elements which were placed at the unusually steep slope of 40% (2.5 Hor. to 1 Vert.). |
| ENVIRONMENTAL CONDITIONS: | Constructed across entrance to Durban Harbour |

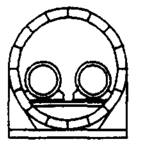
| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| The five reinforced concrete elements were constructed in a drydock. Each element was built from three sections of about 15.25 m in length cast in a vertical position and connnected by post-tensioning cables. | Not applicable | Double rubber seal, one soft and one hard. |

| | |
|---|---|
| WATERPROOFING METHOD: | Not required |
| PLACEMENT METHOD: | Light pontoon rig |
| FOUNDATION METHOD: | Temporary support on concrete rafts on gravel beds. Permanent support on sand fill. |
| DREDGING METHOD: | The greater part by suction dredgers; the sloping trenches and trimming of horizontal excavation by bucket dredger |
| COVER AND TYPE: | Sand backfill |
| ADDITIONAL INFORMATION: | OWNER: Durban Corporation<br>ENGINEERS/CONTRACTORS: Christiani & Nielsen (S.A.) (Pty) Ltd |

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Syphon under the Nile; Cairo, Egypt; 1964**

**S.12 - Cairo Syphon**



TUNNEL TYPE AND USE:
Segment concrete tunnel, syphon

TUBES:
One tube; carries two 1.20 m dia sewer force mains.

| NO OF ELEMENTS: 9 | LENGTH: 40.5 - 56.8 m | HEIGHT: 4.35 m | WIDTH: 3.75 m |
|---|---|---|---|

TOTAL IMMERSED LENGTH: 464 m

DEPTH AT BOTTOM OF STRUCTURE: 20.9 m at flood

UNUSUAL FEATURES:
Vertically cast segmental construction. Interior pipes installed prior to installing end bulkheads. 5 m segments weighed about 40 t. Up to 11 segments were posttensioned into a tunnel element.

ENVIRONMENTAL CONDITIONS:
Schedule had to fit low water during January and flood conditions during August through October. Launching could not be done during low water. Dredging and placing could not be accomplished during flood conditions. Slipways were constructed at site.

FABRICATION METHOD:
Elements were vertically cast, then turned, lined up, and posttensioned. Special runners picked them up. These runners were used to launch the element sideways into the water.

OUTFITTING:
Internal force mains were installed prior to installation of the dam plates.

JOINT TYPE:
Tremie concrete joint. Interior welded steel liner plates welded in place after elements were accessed from inside.

WATERPROOFING METHOD:
Vertically cast using high density concrete. Steel joint closures.

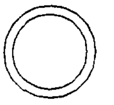
PLACEMENT METHOD:
Catamaran pontoon assembly at the one end and a jack-up platform at the other, both equipped with lowering winches were used. Ballasting was done by partly filling the sewer lines installed in the elements. Alignment was done by using survey columns.

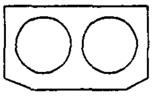
FOUNDATION METHOD:
Saddle blocks were cast in place against the bottom of the trench using tremie concrete. Forms were attached to the elements prior to placement and sand bags were used to seal around them. Used floating batch plant.

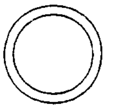
DREDGING METHOD:
The trench was a nominal 7.4 m deep in the river bed and was dredged using a suction dredge. Fine sand and mud were dredged with a 250 mm suction intake with no cutterhead. Side slopes ranged between 1:3 and 1:4. Elements were laid immediately after the related dredging was completed.

COVER AND TYPE:
2 m of quarry stone over one meter of sand backfill above tunnel.

ADDITIONAL INFORMATION:
OWNER: Municipality of Cairo
ENGINEERS: Hochtief, AG
CONTRACTOR: Hochtief, AG

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Multi Low Tension Cable Tunnel; Stockholm, Sweden; 1965** | | | **S.13 - Stockholm Cable Tunnel**<br> |
| TUNNEL TYPE AND USE:<br>Cylindrical concrete tunnel for telecommunication cables | | TUBES:<br>One tube | |
| NO OF ELEMENTS: 4 | LENGTH: 71 m | HEIGHT: 4.45 m OD<br>3.55 m ID | WIDTH: (circular) |
| TOTAL IMMERSED LENGTH: 288 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>Tunnel elements were cast in existing naval drydock, two at a time. | | JOINT TYPE:<br>Provisional rubber seal supplemented by annular fabric-reinforced neoprene rubber strip bolted from inside | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | Two catamaran rigs | | |
| FOUNDATION METHOD: | Pile-supported grillages at element joints | | |
| DREDGING METHOD: | No dredging | | |
| COVER AND TYPE: | No cover | | |
| ADDITIONAL INFORMATION: | OWNER: Telephone & Telegraph Department of Stockholm City<br>ENGINEERS: Christiani & Nielsen A/S<br>CONTRACTORS: Svenska AB Christiani & Nielsen | | |

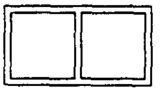
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Siphon de la Mutatiere; Lyon, France; 19..** | | | **S.14 - Lyon Syphon**<br> |
| TUNNEL TYPE AND USE:<br>Prestressed concrete. Siphon in sewer system | | TUBES:<br>Two tubes, each I.D. 2.40 m | |
| NO OF ELEMENTS: 4 | LENGTH: 2 - 52.8 m<br>1 - 35.0 m<br>1 - 32.0 m | HEIGHT: 3.31 m | WIDTH: 6.36 |
| TOTAL IMMERSED LENGTH: 172.6 m | | DEPTH AT BOTTOM OF STRUCTURE:<br>10 m at MWL, 15 m at HHW | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>In casting basin. | | JOINT TYPE:<br>Tremie concrete | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | | | |
| FOUNDATION METHOD: | | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | | | |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>**Water Intake Tunnel; Marsden Power Station, New Zealand; 1967**</td><td>**S.15 - Marsden Water Intake**<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Segmental construction</td><td colspan="2">TUBES:<br>One tube</td></tr>
<tr><td>NO OF ELEMENTS:</td><td>LENGTH: 36.5 m (max)</td><td>HEIGHT: 1.98 m ID</td><td>WIDTH: (circular)</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH:</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE:</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3"></td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3"></td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>2.44 m long segments were manufactured at Mechanics Bay and stressed together with epoxy glued joints to maximum lengths of 36.5 m.</td><td colspan="2">JOINT TYPE:<br>Tremie grouted.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3"></td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">The tunnel elements were (assembled as rafts) towed to site (approx 24 hours) and sunk into excavated trench by use of 80 ton capacity floating crane.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Concrete sills.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">By 18 inch cutter suction dredge</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3"></td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: New Zealand Government<br>ENGINEER: Bechtel Pacific Corp Ltd<br>DESIGNER: Detailed design by contractor<br>CONTRACTOR: McConnell Dowell Ltd Auckland and Hume Industries (N.Z.) Ltd in consortium.</td></tr>
</table>

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Atsumi Karyoku Cooling Water Tunnel; Atsumi Peninsula, Aichi Pref., Japan; 1970**

**S.16 - Atsumi Cooling Water Tunnel**



| | |
|---|---|
| TUNNEL TYPE AND USE: Reinforced concrete box tunnel; Cooling water intake tunnel for Atsumi Power Plant (500 MW x 2) | TUBES: Two tubes |

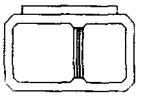
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 1 | LENGTH: 36.5 m | HEIGHT: 4.0 m | WIDTH: 8.4 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 36.5 m | DEPTH AT BOTTOM OF STRUCTURE: 18.0 m |

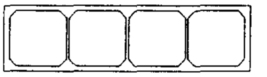
| | |
|---|---|
| UNUSUAL FEATURES: | Tunnel consisting of three caissons and one immersed element. The immersed element inclines with the steep gradient of 1:3.4 down to the water intake caisson. The total length of the intake is 119.4 m |
| ENVIRONMENTAL CONDITIONS: | Open seashore. |

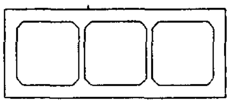
| | |
|---|---|
| FABRICATION AND OUTFITTING: The horizontal elements having lengths of 18.7 m, 30.5 m and 30.4 m respectively were sunk by pneumatic caisson method. The immersed element was fabricated of reinforced concrete in a temporary drydock over two of the three horizontal caissons. The lower half of the water intake caissons were fabricated off site 36 km away at the Isle Ohminato dockyard. The upper part was finished at the site. The immersed tube was fitted with 4 hydraulic jacks, alignment towers and access manholes. | JOINT TYPE: Rubber gasketted flexible joint from immersed tube to intake caisson to provide for difference in natural seismic frequencies. The caisson end was joined rigidly to the land structures. |

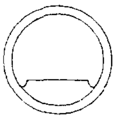
| | |
|---|---|
| WATERPROOFING METHOD: | Waterproofing was not required for water intake. |
| PLACEMENT METHOD: | After floating test in the drydock, the immersed element was moved into position and lowered using two cranes at the outboard end and one crane at the inboard end. |
| FOUNDATION METHOD: | Screeded bedding was used for the 1:3.4 gradient after which the voids were grouted. |
| DREDGING METHOD: | Grab bucket (4-8 $m^3$) |
| COVER AND TYPE: | 1.5 m of sand backfill at the top of the tunnel with stone along the sides to protect against tidal currents, wave action, and earthquake action. |
| ADDITIONAL INFORMATION: | OWNER: Chubu Electric Power Company, Inc<br>CONTRACTOR: Hazama Corporation |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Dokai Bay Conveyor Tunnel; Nakahara Jisaki, Tobata-ku, Kitakyushu City, Japan; 1972** | | | **S.17 - Dokai Bay Conveyor**<br> |
| TUNNEL TYPE AND USE:<br>Concrete box elements; Conveyor belts | | TUBES:<br>Two tubes each carrying a conveyor belt for transporting coke. | |
| NO OF ELEMENTS: 17 | LENGTH: 80 m | HEIGHT: 4.55 m | WIDTH: 8.218 m |
| TOTAL IMMERSED LENGTH: 1,363 m | | DEPTH AT BOTTOM OF STRUCTURE: 20.9 m | |
| UNUSUAL FEATURES: | Used jack-up platform for placement of elements. Elements cast in a specially constructed sheet pile cofferdam drydock, two at a time. The total length of tunnel included 17 elements plus a 55.5 m connection section (9.2 m wide) | | |
| ENVIRONMENTAL CONDITIONS: | There was a requirement not to interfere with marine traffic in the busy Wakamatsu Fairway, a busy international sea lane. For this reason new construction methods were devised including the jack-up platform. | | |
| FABRICATION METHOD:<br>Manufactured in new drydock, Nippon Steel Corporation's Tobata Works, (24 m x 95 m. Outlet 13 m x 10 m). Elements were made in pairs, towed in turn and temporarily moored and immersion works carried out. | | OUTFITTING:<br>Underwater retention bollard. Automatic joining mechanism (coupler) | JOINT TYPE:<br>Single cantilever gasketted joint. Omega gasket used across joint. |
| WATERPROOFING METHOD: | 6 mm thick circumferential steel plate attached with steel studs. | | |
| PLACEMENT METHOD: | The drydock was flooded and the box element towed by tugboat. The element was then secured to the underwater bollards situated in the Sakai River. Ballast was placed into pockets on top of element. Immersion work was carried out by shipboard crane where there was a lack of depth, and by self-elevating platform elsewhere. | | |
| FOUNDATION METHOD: | Screeded foundation placed from jack-up barge. | | |
| DREDGING METHOD: | Grab bucket and dipper dredges. | | |
| COVER AND TYPE: | 3.0 m of 40 mm crushed stone. | | |
| ADDITIONAL INFORMATION: | OWNER: Mitsui Mining Coke Industries Ltd<br>CONTRACTOR: Mitsui, Kajima Joint Venture | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: Dokaiwan Conveyor Tunnel; Kita-Kyushu, Japan; 1972 | | | S.18 - Dokaiwan Conveyor |
|---|---|---|---|
| TUNNEL TYPE AND USE: Steel-plate covered reinforced concrete; Transportation of ore by belt conveyor | | TUBES: | |
| NO OF ELEMENTS: 16 | LENGTH: 13 - 80 m<br>3 - 81.04 m | HEIGHT: 4.55 m | WIDTH: 8.2 m |
| TOTAL IMMERSED LENGTH: 1,283 m | | DEPTH AT BOTTOM OF STRUCTURE: 15.9 m | |
| UNUSUAL FEATURES: | Rock bed was exposed at the depth of -10 m along about 300 m in southern approach. To lessen amount of rock excavation at this part, the slope of approach sections have a steep gradient. | | |
| ENVIRONMENTAL CONDITIONS: | Dense navigation. Very rough seas in winter season. The platform was designed to be stable in typhoon winds of 150 KPH. | | |
| FABRICATION METHOD: Two elements were fabricated in one cycle in a drydock. Heavy end-struts were installed in the casting beds to limit growth due to thermal expansion during casting to less than 5 mm. | | OUTFITTING: At mooring berth 9 km from the site. Concrete curing took place at the outfitting dock. | JOINT TYPE: Rigid connection with rubber gasket. |
| WATERPROOFING METHOD: | Elements were covered with 6 mm thick steel plate. | | |
| PLACEMENT METHOD: | By a semi -submersible jack-up platform, especially fabricated for this project at dock yard. Crushed stone and slag were used for placement ballast. | | |
| FOUNDATION METHOD: | A crushed stone bed (stone < 40 mm) was screeded to have the thickness of 0.7 m and a width of 9.3 m. Screeding was operated from the jack-up platform. | | |
| DREDGING METHOD: | Dipper dredge for rock bed and grab bucket boat for soil bed. | | |
| COVER AND TYPE: | Riprapping at both sides; backfilled by sand. Protective crusher-run layer was placed over tunnel. | | |
| ADDITIONAL INFORMATION: | | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: **Calvert Cliffs Outfall Conduit, Maryland, USA; 1972** | | | **S.19 - Calvert Cliffs Outfall**  |
|---|---|---|---|
| TUNNEL TYPE AND USE: Outfall conduit for nuclear powerplant | | TUBES: Four tubes | |
| NO OF ELEMENTS: 5 | LENGTH: 61 m | HEIGHT: 4.41 m | WIDTH: 16.76 m |
| TOTAL IMMERSED LENGTH: 305 m | | DEPTH AT BOTTOM OF STRUCTURE: 6.6 m | |
| UNUSUAL FEATURES: | Tunnel was constructed from elements which had three removable bulkheads. The inboard end bulkheads for each separate tube was divided into four sections which were light enough to be hooked onto and dragged by cable. The outboard bulkhead on each tube could be hoisted away as a single piece. The elements were floated into position and ballasted internally to a negative buoyancy of 100 tons. These were then lowered to the previously dredged and screeded bottom and two jacks were used to tighten the new element against the previous one with a simple dumbbell gasket in between. The new element was then flooded and divers progressively removed the outboard bulkhead, the middle bulkhead, and finally the four pieces of the inboard bulkhead. Model tests of the handling characteristics of the element were undertaken at the U. S. Corps of Engineers laboratory in Vicksburg, Mississippi. These tests indicated that wave action caused a significant increase on the loading of the lowering falls during placement. | | |
| ENVIRONMENTAL CONDITIONS: | Heavy marine traffic throughout tow route. Shallow depth exposure to wave action. | | |
| FABRICATION METHOD: Cast in an excavated basin in Chesapeake, Virginia and towed 200 km to the site. | | OUTFITTING: In casting basin. | JOINT TYPE: Dumbbell shaped neoprene gasket. Compressed by jacking only. Ultimate watertightness not critical because of nature of tunnel. |
| WATERPROOFING METHOD: | Not applicable. | | |
| PLACEMENT METHOD: | Temporary guide beams. Water ballast to 100 ton negative buoyancy. Sections were jacked together. Element was then completely flooded and bulkheads were removed by divers as described above. | | |
| FOUNDATION METHOD: | Screeded bed using temporary pile supported frame. | | |
| DREDGING METHOD: | Hydraulic dredge was used for casting basin and to level bottom under tunnel. | | |
| COVER AND TYPE: | There was no protective cover over the top of the element. The element was not placed in a trench but just sat on the screeded bed. | | |
| ADDITIONAL INFORMATION: | OWNER: Baltimore Gas & Electric Co.<br>ENGINEERS: Bechtel Associates<br>CONTRACTOR: Tidewater Construction Corporation | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br>**Amsterdam-Rhine Canal Water Transport Tunnel; The Netherlands; 1973** | | | **S.20 - Amsterdam Canal Tunnel**<br> |
| TUNNEL TYPE AND USE:<br>Segmental concrete box tunnel; Water transport | | TUBES:<br>Three tubes interconnecting the Amsterdam Rhine Canal with the Vaartse Rijn which crosses it. | |
| NO OF ELEMENTS: 1 | LENGTH: 132 m | HEIGHT: 2.85 m | WIDTH: 8.89 m |
| TOTAL IMMERSED LENGTH: 132 m | | DEPTH AT BOTTOM OF STRUCTURE: 10 m approx. | |
| UNUSUAL FEATURES: | Entire tunnel fabricated as a single immersed element from 44, 3 m prefabricated segments post-tensioned together. | | |
| ENVIRONMENTAL CONDITIONS: | Constructed across an active canal. | | |
| FABRICATION METHOD:<br>Match cast segments. A special measuring post was used to measure the length of the next segment to be cast. The segments were joined with epoxy resin and temporarily prestressed. Finally 32, 55 mm cables and 6 BBRV cables were used for final post-tensioning. Element was assembled in a vacant canal lock. | | OUTFITTING:<br>Fitted with bollards, hooks for winching, watertight bulkheads and ballast tanks. | JOINT TYPE:<br>None required. |
| WATERPROOFING METHOD: | None required. | | |
| PLACEMENT METHOD: | Placed using two floating cranes. Ballasting with concrete slabs on roof of tunnel. | | |
| FOUNDATION METHOD: | Screeded gravel. | | |
| DREDGING METHOD: | After concrete structures on both banks completed. | | |
| COVER AND TYPE: | Sand fill to bottom of canal | | |
| ADDITIONAL INFORMATION: | OWNER: Rijkswaterstaat<br>ENGINEERS/CONTRACTORS: N.V. Nestum II (Consortium) consisting of: N.V. Nederlandsche Beton Maatschappij, Christiani & Nielsen NV, Van Hattum & Blankevoort, Hollandsche Beton Maatschappij NV, NV Internationale Gewapend Beton Bouw, NV Nederlandsche Aannemingsmaatschappij, NV Amsterdamsche Ballast Maatschappij | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | S.21 - Hollandsch Diep Service Tunnel |
|---|---|
| **Hollandsch Diep Service Tunnel; Tunnel under the Hollandsch Diep near Strijen, The Netherlands; 1973** |  |

| TUNNEL TYPE AND USE: | TUBES: |
|---|---|
| Segmental cylindrical concrete tunnel; Utility tunnel | One tube. Will carry up to 40 utility pipes. |

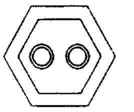
| NO OF ELEMENTS: 27 | LENGTH: 60 m | HEIGHT: 4.65 m OD (4.0 m ID) | WIDTH: (circular) |
|---|---|---|---|

| TOTAL IMMERSED LENGTH: 1,627 m | DEPTH AT BOTTOM OF STRUCTURE: 15.7 m |
|---|---|

| | |
|---|---|
| UNUSUAL FEATURES: | 10 vertically cast 6 m long cylindrical segments form 60 m long elements. Tunnel terminates at both ends in pier structures 8 x 30 m and 8 x 45 m to provide room for expansion and contraction and to serve as transitions to the land connections. Pipelines were installed after the tunnel was in place. Gradient was limited to 7.5% for installation of pipelines. Pipe size could be as much as 0.60 m (24"). |
| ENVIRONMENTAL CONDITIONS: | Busy shipping channel |

| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| Vertical casting. Assembly of all elements in a single dewatered basin. Match casting was not used. Instead a joint of 1-5 cm was made using interior and exterior forms and rubber packers in the cable ducts. These joints were injected with epoxy resin. The elements were then post-tensioned. Ballast concrete was added in bottom. | Dished steel bulkheads with watertight manholes were installed. 0.5 Bar pressure test was made. Leaks were sealed by injection. | Gina and Omega joint protected with flexible internal steel closure plate. |

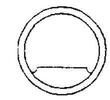
| | |
|---|---|
| WATERPROOFING METHOD: | High density concrete 0.325 m thick was adequate. No external waterproofing membrane was used. Pressure test prior to flotation also added to security against leakage. |
| PLACEMENT METHOD: | Transport to site was by catamaran barge. Placement was made from two accurately positioned spudded barges. Inboard end rested on cantilever beams and outboard end on adjustable jacks. |
| FOUNDATION METHOD: | A jetted sand foundation was installed. The first stage was up to about a meter above the bottom of the tunnel. Then the jacks were removed. Sandjetting continued to the top of the tunnel after which hopper barges completed the cover. |
| DREDGING METHOD: | Initially two cutterhead dredges were used. The trench was completed using a bucket dredge. |
| COVER AND TYPE: | Approximately 2 m of ordinary backfill |
| ADDITIONAL INFORMATION: | OWNER/DESIGNER: Rijkswaterstaat<br>CONTRACTORS: Koninlijke Nederhorst Bouw B.V., Dirk Verstoep Nederland B.V. |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**District Heating Tunnel; Odense, Denmark; 1974** | | | **S.22 - Odense Heating Tunnel**<br> |
| TUNNEL TYPE AND USE:<br>Segmental construction for two 470 mm heating pipes | | TUBES:<br>One tube | |
| NO OF ELEMENTS: 1 | LENGTH: 90 m | HEIGHT: 2.67 m | WIDTH: 3.08 m |
| TOTAL IMMERSED LENGTH: 90 m | | DEPTH AT BOTTOM OF STRUCTURE: 9.1 m | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | Tunnel site on protected Odense Canal | | |
| FABRICATION METHOD:<br>Precasting of 5 m long segments in vertical position. Assembling of segments and post-tensioning in nearby purpose-built drydock | | OUTFITTING:<br>At tunnel site | JOINT TYPE:<br>Injected cement-mortar paste with additives in segments joints |
| WATERPROOFING METHOD: | No waterproofing membrane | | |
| PLACEMENT METHOD: | Two pontoon rigs were used to sink the tunnel element. Heating mains were utilized as ballast chambers. | | |
| FOUNDATION METHOD: | Jetted sand foundation was installed using patented Christiani & Nielsen method | | |
| DREDGING METHOD: | Partly by drag-line excavator and below MSL by grab dredging | | |
| COVER AND TYPE: | 1 m of sand fill | | |
| ADDITIONAL INFORMATION: | OWNER: Odense Municipality Corporation<br>ENGINEERS: Christiani & Nielsen A/S<br>CONTRACTORS: Dansk Entreprenørselskab Christiani & Nielsen A/S | | |

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Oude Maas Service Tunnel; under the Oude Maas Near Heinenoord, The Netherlands; 1975**

**S.23 - Oude Maas Service Tunnel**



TUNNEL TYPE AND USE:
Segmental cylindrical concrete tunnel; Utility tunnel

TUBES:
One tube. Will carry up to 40 utility pipes.

| NO OF ELEMENTS: 8 | LENGTH: 60 m | HEIGHT: 4.65 m OD (4.0 m ID) | WIDTH: (circular) |
|---|---|---|---|

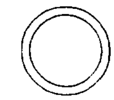
TOTAL IMMERSED LENGTH: 481 m

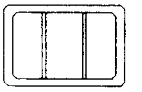
DEPTH AT BOTTOM OF STRUCTURE: 3 m below the existing canal bottom.

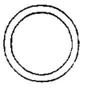
| | |
|---|---|
| UNUSUAL FEATURES: | 10 vertically cast 6 m long cylindrical segments form the 60 m long elements. The first section was immersed against the abutment on the south bank. The abutment on the north bank consists in a vertical circular shaft with a hexagonal box on top of it. A 4 m hole is provided at the bottom to accommodate the end of the terminal tube. Similar in design and construction to the service tunnel under the Hollandsch Diep. |
| ENVIRONMENTAL CONDITIONS: | Busy shipping channel. |

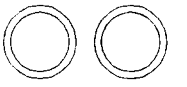
| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| Vertical casting. Assembly of all elements in a single dewatered basin. Match casting was not used. Instead a joint of 1-5 cm was made using interior and exterior forms and rubber packers in the cable ducts. These joints were injected with epoxy resin. The elements were then post-tensioned. Ballast concrete was added in bottom. | Dished steel bulkheads with watertight manholes were installed. 0.5 Bar pressure test was made. Leaks were sealed by injection. | Gina and Omega joint protected with flexible internal steel closure plate. |

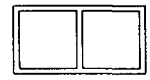
| | |
|---|---|
| WATERPROOFING METHOD: | High density concrete 0.325 m thick was adequate. No external waterproofing membrane was used. Pressure test prior to flotation also added to security against leakage. |
| PLACEMENT METHOD: | Transport to site was by catamaran barge. Placement was made from two accurately positioned spudded barges. Inboard end rested on cantilever beams and outboard end on adjustable jacks. |
| FOUNDATION METHOD: | A jetted sand foundation was installed. The first stage was up to about a meter above the bottom of the tunnel. Then the jacks were removed. Sandjetting was then continued to the top of the tunnel after which hopper barges completed the cover. |
| DREDGING METHOD: | |
| COVER AND TYPE: | Approximately three meters of ordinary backfill. |
| ADDITIONAL INFORMATION: | OWNER/DESIGNER: Rijkswaterstaat<br>CONTRACTORS: Koninklijke Nederhorst Bouw B.V., Dirk Verstoep Nederland B.V. |

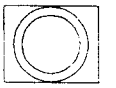
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Cooling Water Outfall; Nuclear Power Station, Unterweser, Germany; 1975** | | | **S.24 - Unterweser Outfall**<br> |
| TUNNEL TYPE AND USE:<br>R.C. pipe; Cooling water outfall | | TUBES:<br>One tube. | |
| NO OF ELEMENTS: | LENGTH: 50 m | HEIGHT: 4.00 m ID | WIDTH: (circular) |
| TOTAL IMMERSED LENGTH: 425 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | Current velocity 1.5 m/sec. Waves. | | |
| FABRICATION METHOD:<br>**Elements constructed in 12.50 m wide casting basin provided with dock gate. Basin could accommodate 2 x 3 elements.** | | JOINT TYPE: | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | | | |
| FOUNDATION METHOD: | Jetted sand | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | | | |

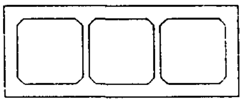
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**LNG Terminal Tunnel; Cove Point, Maryland, U.S.A.; 1976** | | | **S.25 - Cove Point LNG**<br> |
| TUNNEL TYPE AND USE:<br>Steel shell box shaped; Utilities | | TUBES:<br>Three tubes; each outer tube contains a 0.81 m dia cryogenic LNG unloading line and a 0.36 m dia cryogenic vapor return line. Center tube carries miscellaneous piping. | |
| NO OF ELEMENTS: 20 | LENGTH: 80.16 m | HEIGHT: 4.88 m | WIDTH: 8.23 m |
| TOTAL IMMERSED LENGTH: 1,610 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | Tunnel to end of LNG off-loading pier. Used to make facility visually acceptable by eliminating long pipeline structure out to pier. | | |
| ENVIRONMENTAL CONDITIONS: | Stringent environmental protective measures were maintained throughout all phases of the construction. | | |
| FABRICATION METHOD:<br>The subaqueous tunnel consists of 20 elements, each having an outer steel plate shell reinforced internally with transverse ribs and two longitudinal trusses. The open ends of each section were temporarily sealed by end bulkheads during the early stages of construction.<br>After fabrication the steel shells were launched sideways and towed to an outfitting pier where the internal structural concrete was installed. | | JOINT TYPE:<br>After each of the 20 sections had been placed in position, the joints were sealed externally with tremie concrete and internally with steel liner plates. | |
| WATERPROOFING METHOD: | Continuous outer steel shell. | | |
| PLACEMENT METHOD: | A specially-built catamaran type lay-barge was used to place each tunnel element. The elements were first ballasted by placing concrete on the top surface, then lowered onto a prepared gravel bed in the dredged trench on the bay bottom. | | |
| FOUNDATION METHOD: | | | |
| DREDGING METHOD: | Gravel bed screeded to a tolerance of 3 cm by a special screed barge designed for the purpose. | | |
| COVER AND TYPE: | The trench was backfilled and tubes completely covered. | | |
| ADDITIONAL INFORMATION: | CLIENT: Colombia LNG Corporation<br>DESIGNER/CONTRACTOR: Raymond International (leading partner) | | |

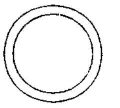
| | |
|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:** | **S.26 - Dokai Bay Gas Pipeline** |
| **Dokai Bay Seibu Gas Pipeline; Kita-Kyushu City, Wakamatsu Fairway, Japan; 1977** |  |
| TUNNEL TYPE AND USE: | Prestressed concrete block (for gas pipe) |
| TUBES: | One tube; 2 gas pipes |
| NO OF ELEMENTS: | 10 |
| LENGTH: | 45 m |
| HEIGHT: | 3.2 m |
| WIDTH: | (circular) |
| TOTAL IMMERSED LENGTH: | 434 m |
| DEPTH AT BOTTOM OF STRUCTURE: | 18.2 m |
| UNUSUAL FEATURES: | The immersed elements were manufactured using the match casting fabrication method. Watertightness was greatly enhanced through prestressing so that no waterproofing was provided. Because of ship traffic, immersion work was carried out at night. Construction time was 14 months. |
| ENVIRONMENTAL CONDITIONS: | Busy shipping channel. |
| FABRICATION METHOD: | 4 m long cylindrical concrete sections were joined together with epoxy resin and post-tensioned to form 45 m immersed element. |
| OUTFITTING: | Two 25 ton jacks. Mooring bollards. |
| JOINT TYPE: | Gina type rubber gasket immersion joints. |
| WATERPROOFING METHOD: | Waterproofing was provided only around the epoxy resin joints using urethane resin. |
| PLACEMENT METHOD: | The immersed elements were launched by 700 t floating crane and towed by tugboat to a mooring point where an element was placed on a self-elevating platform. The self-elevating platform loaded with an element was towed to the construction site by tugboat. After securing the platform, the element was lifted and then lowered with a gantry crane. |
| FOUNDATION METHOD: | After backfilling, mortar grouting was carried out in the spaces between the elements and crushed stone foundation. |
| DREDGING METHOD: | Grab bucket dredger. |
| COVER AND TYPE: | Section up to the top of the immersed elements was filled with crushed stone. 3 m of marine aggregate cover was provided over the tunnel. |
| ADDITIONAL INFORMATION: | OWNER: Seibu Gas Co<br>CONTRACTOR: Mitsui Construction Company |

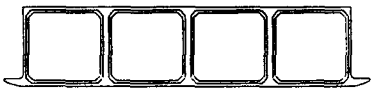
<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>**Cooling Water Outfall; Blayais Power Station, France; 1978**</td><td>**S.27 - Blayais Outfall**<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Reinforced concrete; Cooling water outfall</td><td colspan="2">TUBES:<br>Two separate tubes</td></tr>
<tr><td>NO OF ELEMENTS: 145</td><td>LENGTH: 25 m</td><td>HEIGHT: 4.30 ID</td><td>WIDTH: (circular)</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 1,935 m x 2</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE:</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Elements lowered after removal of end bulkheads.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Current velocity in tidal Gironde up to 3 m/sec.</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>5 m long segments were cast in vertical position, and subsequently 25 m elements were assembled in horizontal position, posttensioned and provided with temporary bulkheads.</td><td colspan="2">JOINT TYPE:<br>Bell and spigot.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3"></td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Elements were towed to catamaran type lay-barge "Duc de Saint-Simon". Temporary bulkheads were moved and elements lowered onto temporary supports.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Sand placed via "elephant trunk".</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3"></td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">Sand, dumped</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">CLIENT: Electricité de France, Région d'Equipement Paris<br>CONTRACTOR: Campenon Bernard Cetra-Dodin<br>SUBCONTRACTOR:<br>Dredging : Atlantique Dragage<br>Steel : Welbond<br>Prestressing: STUP Freyssinet</td></tr>
</table>

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Cooling Water Outfall; Kilroot, Northern Ireland, United Kingdom; 1978** | | | **S.28 - Kilroot Outfall**<br> |
| TUNNEL TYPE AND USE:<br>Segmental construction. Cooling water outfalls for 1200 MW power station | | TUBES:<br>Two tubes. Capacity: 33 m³ per second | |
| NO OF ELEMENTS: 5 | LENGTH: 4 - 94.0 m<br>1 - 85.8 m | HEIGHT: 3.53 m | WIDTH: 6.91 m |
| TOTAL IMMERSED LENGTH: 584 m | | DEPTH AT BOTTOM OF STRUCTURE: 8-7 m | |
| UNUSUAL FEATURES: | Immersed tunnel placed in the Irish Sea, subject to wave loadings. Lay-out and design of temporary supports and warping/sinking equipment was based on results of hydraulic model tests. | | |
| ENVIRONMENTAL CONDITIONS: | Exposed tunnel site facing the Irish Sea | | |
| FABRICATION METHOD:<br>Tunnel elements were cast in one batch in temporarily closed water intake basin | | OUTFITTING:<br>Done in flooded casting basin at low water | JOINT TYPE:<br>Trelleborg rubber gasket in element joints |
| WATERPROOFING METHOD: | No waterproofing membrane | | |
| PLACEMENT METHOD: | Two catamaran rigs with 200 tonne lifting capacity each lifted tunnel element out of the casting basin. The elements were not in a buoyant state. The rigs were equipped with hydraulic winches and were used for lowering the elements to their final position | | |
| FOUNDATION METHOD: | Jetted sand foundation installed using patented Christiani & Nielsen method. Temporary support on U-shaped sills | | |
| DREDGING METHOD: | Bucket dredger with heavy duty buckets for excavation of rock marl | | |
| COVER AND TYPE: | 1 m of armour stone | | |
| ADDITIONAL INFORMATION: | OWNER: Northern Ireland Electricity Service<br>ENGINEERS: Allott & Lomax, Sale - Christiani & Nielsen A/S<br>CONTRACTORS: Christiani & Nielsen Ltd | | |

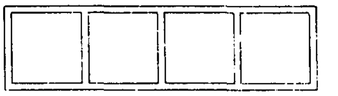
| TUNNEL NAME/LOCATION/DATE COMPLETED: **Conveyor Belt Tunnel; Bakar Golf, Yugoslavia; 1978** | | | **S.29 - Bakar Golf Conveyor**  |
|---|---|---|---|
| TUNNEL TYPE AND USE: Segmental construction; Reinforced concrete; Coal transport | | TUBES: One tube; capacity 4.5 m/sec | |
| NO OF ELEMENTS: 9 | LENGTH: 40 m | HEIGHT: 3.50 m ID | WIDTH: (circular) |
| TOTAL IMMERSED LENGTH: 360 m | | DEPTH AT BOTTOM OF STRUCTURE: 11.0 m | |
| UNUSUAL FEATURES: | Designed for earthquake loading. | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD: Elements consisted of 8 prefabricated, 4.55 m long segments and 0.60 m long 5.00 x 5.00 m end plates. Segments were constructed on shore and assembled on flat deck barge, 3 elements at a time. Elements were side-launched during grounding of barge at shallow water. | | JOINT TYPE: Rubber seal, type Gina plus Omega-shaped seal | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | By means of floating crane | | |
| FOUNDATION METHOD: | Pile foundation (at tunnel joints), pile length 20-30 m | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | OWNER:<br>ENGINEERS: Hochtief - Codema Essen, Germany<br>CONTRACTOR: Pomgrad of Split, Yugoslavia | | |

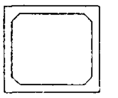
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Syphon Mittellandskanal; Bramsche, Germany; 1978** | | | **S.30 - Bramsche Syphon**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box; Syphon | | TUBES:<br>Three tubes | |
| NO OF ELEMENTS: 1 | LENGTH: 81.78 m | HEIGHT: 5.30 m | WIDTH: 20.80 |
| TOTAL IMMERSED LENGTH: 81.78 m | | DEPTH AT BOTTOM OF STRUCTURE: 10 m | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>Tunnel element was built in a purpose-built casting basin located next to canal. | | JOINT TYPE:<br>No joints, monolithic structure. | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | Permanent vertical structures at ends of tunnel element provided stability during sinking which took place without sinking pontoons.<br>Landing on temporary foundations. | | |
| FOUNDATION METHOD: | Cement grout. | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | OWNER:<br>DESIGNER:<br>CONTRACTOR: Philipp Holzmann AG | | |

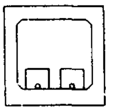
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Underwater Outlet; Lake Piru, California, U.S.A.; 1978** | | | **S.31 - Lake Piru Outfall**<br> |
| TUNNEL TYPE AND USE:<br>Segmental cylindrical concrete tunnel. Reservoir outlet | | TUBES:<br>One tube | |
| NO OF ELEMENTS: 1 | LENGTH: 12.80 m | HEIGHT: 1.91 m ID<br>2.29 m OD | WIDTH: (Circular) |
| TOTAL IMMERSED LENGTH: 12.80 m | | DEPTH AT BOTTOM OF STRUCTURE:<br>At installation stage: 19.2 m<br>At maximum water level in reservoir: 49.7 m | |
| UNUSUAL FEATURES: | Prefabricated concrete tunnel element was installed in vertical position. | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>Element composed of seven segments each 1.83 m long, constructed at work site 56 km from the reservoir. Corrugated metal pipe was used for inner and outer casting form. Element was prestressed with 10 steel rods. Tunnel element was launched from a ramp, 18.3 m wide with 12% slope. | | JOINT TYPE:<br>Underwater concrete. | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | Weight controlled by ballast water / compressed air. Position controlled by floating platform consisting of two rafts each 7.3 m square. | | |
| FOUNDATION METHOD: | Founded on upper end of underground outlet section. | | |
| DREDGING METHOD: | Not applicable | | |
| COVER AND TYPE: | Not applicable | | |
| ADDITIONAL INFORMATION: | OWNER / ENGINEER: United Water Conservation District, Santa Paula<br>CONTRACTOR: Owner | | |

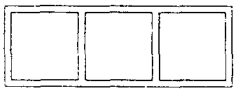
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Cooling Water Outfall; Halileh, Persian Gulf, Iran; 1979** | | | **S.32 - Halileh Outfall**<br> |
| TUNNEL TYPE AND USE:<br>Concrete box section; Cooling water outfall | | TUBES:<br>Four tubes | |
| NO OF ELEMENTS:<br>10 + nozzle section | LENGTH: 105 m | HEIGHT: 6.07 m | WIDTH: 24.15 m |
| TOTAL IMMERSED LENGTH: 1,200 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | The elements were placed at sea. Extra ballasting was required to prevent uplift by waves. | | |
| ENVIRONMENTAL CONDITIONS: | Earthquake zone. Tube placement in ocean waves. Stiff clay near shore and weak layers at seaside. | | |
| FABRICATION METHOD:<br>Tunnel elements were built in purpose-built casting basin next to outfall. | | JOINT TYPE:<br>Gina and Omega rubber seal. | |
| WATERPROOFING METHOD: | Not applicable. | | |
| PLACEMENT METHOD: | Catamaran type sinking rigs which were kept in fixed vertical position by means of cables attached to stay piles. Lowering of element onto temporary foundation blocks. | | |
| FOUNDATION METHOD: | Pile foundation for six tunnel elements. Jetted sand foundation for four tunnel elements closest to shore. | | |
| DREDGING METHOD: | Bucket dredge. | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | OWNER: Iranian Atomic Energy Commission<br>ENGINEERS:<br>CONTRACTORS: J.V. with Hochtief AG Essen as leading partner | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: **Sewer Outfall; Portsmouth Harbour, United Kingdom; 1979** | | | **S.33 - Portsmouth Outfall** |
|---|---|---|---|
| TUNNEL TYPE AND USE: Reinforced concrete box; Sewer outfall | | TUBES: Two tubes | |
| NO OF ELEMENTS: | LENGTH: | HEIGHT: 2.6 m | WIDTH: |
| TOTAL IMMERSED LENGTH: | | DEPTH AT BOTTOM OF STRUCTURE: up to 27 m | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD: | | JOINT TYPE: Rubber gaskets 115 mm thick. | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | Tunnel elements were towed to sinking place provided with air bags for support and lowered by purpose-built catamaran rig. | | |
| FOUNDATION METHOD: | Sand bedding by injection of sand (300 mm thick) | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | OWNER:<br>DESIGNER:<br>CONTRACTOR: John Laing | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Cooling Water Intake; Dora Creek, Lake McQuarie, N.S.W., Australia; 1981** | | **S.34 - Dora Creek Intake**<br> | |
| TUNNEL TYPE AND USE:<br>Concrete tunnel; Cooling water intake capacity 125 $m^3$ per second. | | TUBES:<br>Four tubes. | |
| NO OF ELEMENTS: 1 | LENGTH: 260 m | HEIGHT: 5.0 m | WIDTH: 21.0 m |
| TOTAL IMMERSED LENGTH: 260 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | **Because of the low height to length ratio, the element was sensitive to longitudinal bending moments during float-out and lowering into final position.** | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>Tunnel element was cast in sections 20 m long with 2 m gaps between them. These gaps were not concreted until much later. Element was supported on concrete piles driven in basin bottom. | | OUTFITTING: | JOINT TYPE:<br>**No element joints. Section joints were plane faced with no shear keys or water bars.** |
| WATERPROOFING METHOD: | **Not applicable.** | | |
| PLACEMENT METHOD: | | | |
| FOUNDATION METHOD: | **Sand/fly ash pumped in underneath tunnel. Method was tested beforehand.** | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | OWNER: **Electricity Commission of N.S.W.**<br>ENGINEER: **Evans Eddie Connell, a JV of Connell Eddie & Associates, John Connell & Associates, and Maldwyn Evans & Associates**<br>CONTRACTOR: **John Holland (Constructions) Pty Ltd** | | |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Keihin Conveyor Tunnel, Japan; 1981</td><td>S.35 - Keihin Conveyor Tunnel<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Concrete box; conveyor belt</td><td colspan="2">TUBES:<br>One tube.</td></tr>
<tr><td>NO OF ELEMENTS: 3</td><td>LENGTH: 40 m</td><td>HEIGHT: 4.8 m</td><td>WIDTH: 4.1 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 120</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE:</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3"></td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3"></td></tr>
<tr><td colspan="2">FABRICATION METHOD:</td><td>OUTFITTING:</td><td>JOINT TYPE:</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3"></td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3"></td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3"></td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3"></td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3"></td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">CONTRACTOR: Kumagai Gumi</td></tr>
</table>

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Landing Tunnel; Karmoey, Norway; 1983</td><td>S.36 - Karmoey Tunnel<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; protecting oil pipes through surf zone</td><td colspan="2">TUBES:<br>One tube</td></tr>
<tr><td>NO OF ELEMENTS: 5</td><td>LENGTH: 118.0 m</td><td>HEIGHT: 8.5 m - 7.5 m</td><td>WIDTH: 7.0 m - 6.6 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 590 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE:</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3"></td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Design wave (100 year) height : 18.5 m<br>period : 14 sec<br>length : 230 m</td></tr>
<tr><td colspan="2">FABRICATION METHOD:</td><td colspan="2">JOINT TYPE:</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3"></td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">After ballasting elements were pulled down to the foundations. Installation criteria:<br>Significant wave height 1.0-1.5 m/sec<br>period 6-8 sec<br>Wind speed: 5-8 m/sec</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Prefabricated R.C. sills at element joints placed on "in situ" built concrete foundation.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3"></td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3"></td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: Statoil, Norway<br>ENGINEER: F. Selmer A/S with consultants Dr. Techn. Olav Olsen, Noteby and Nybro-Hansen A/S<br>CONTRACTOR: F. Selmer A/S</td></tr>
</table>

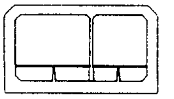
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Hase Tunnel; Bramsche Canal, Germany; 1984** | | | **S.37 - Bramsche Canal Syphon**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box element; Syphon | | TUBES:<br>Three tubes | |
| NO OF ELEMENTS: 1 | LENGTH: 81.78 m | HEIGHT: 5.30 m | WIDTH: 21.20 m |
| TOTAL IMMERSED LENGTH: 81.78 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>Element was constructed in purpose-built casting basin at river shore in tunnel line. | | JOINT TYPE: | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | | | |
| FOUNDATION METHOD: | Underwater concrete. | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | OWNER:<br>DESIGNER:<br>CONTRACTOR: Philipp Holzmann AG | | |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Cooling Water Intake Tunnel; Nuclear Power Station, Brokdorf, Germany; 1984</td><td>S.38 - Brokdorf Intake</td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>R.C. box; Cooling water intake</td><td colspan="2">TUBES:<br>Two tubes</td></tr>
<tr><td>NO OF ELEMENTS: 26</td><td>LENGTH: 51.00 m</td><td>HEIGHT: 5.0 m</td><td>WIDTH: 12.50 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 1,326 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE:</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3"></td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3"></td></tr>
<tr><td colspan="2">FABRICATION METHOD:</td><td colspan="2">JOINT TYPE:<br>Phoenix rubber seal and Omega seal.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3"></td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">By use of floating cranes. Element was lowered onto temporary foundation blocks.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Jetted sand foundation</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3"></td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3"></td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3"></td></tr>
</table>

TUNNEL NAME/LOCATION/DATE COMPLETED:

**Jurong Strait Utility Tunnel; between Pulau Seraya and Mainland Singapore; 1986**

**S.39 - Jurong Utility Tunnel**



TUNNEL TYPE AND USE:
Segmental construction. Electrical cables; water supply

TUBES:
Two tubes. Seven oil filled 230 kV, 500 MVA power lines with cooling pipes. Two potable water lines

| NO OF ELEMENTS: 26 | LENGTH: 100 m | HEIGHT: 3.68 m | WIDTH: 6.50 m |
|---|---|---|---|

TOTAL IMMERSED LENGTH: 2,600 m

DEPTH AT BOTTOM OF STRUCTURE: 25 m

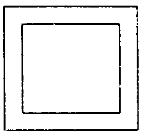
| | |
|---|---|
| UNUSUAL FEATURES: | Very long utility tunnel: 2,600 m with 1,800 m below 23 m sea level. The extent to which assembly line methods were used for fabrication of the tunnel elements and the high quality 100% watertight concrete which was achieved. The extent to which permanent civil, mechanical and electrical installations were completed within the elements prior to launching. Construction speed: 26 elements installed and backfilled in 27 weeks. |
| SPECIAL CONDITIONS: | Very tight schedule required installation of power cables only 16.5 months after award of contract |

FABRICATION METHOD:
Precasting five 3.6 m segments per day. Assembling 29 segments on a marine lift. Launching from the marine lift, sinking, joining, founding and backfilling one element per week. Segments were cast in vertical position and moved by two 50 t gantry cranes to marine lifts, aligned, joints grouted and post-tensioned

OUTFITTING:
Most of the electrical and mechanical systems were mounted in the elements before closing and launching. This minimized the work to be done after place-ment.

JOINT TYPE:
Rubber sealing gasket. Steel shroud and temporary bulkhead.

| | |
|---|---|
| WATERPROOFING METHOD: | Segmental construction without longitudinal joints combined with grouted transverse joints and post-tensioning provided watertightness without additional exterior waterproofing membrane. |
| PLACEMENT METHOD: | Buoyancy tanks were installed on top of the element by lowering it with the marine lift. The element and buoyancy tank assemblies were towed to the lay barge, the tanks ballasted and lowered with two cranes |
| FOUNDATION METHOD: | The tunnel elements were landed on temporary steel supports, joined to the previously placed element, jacked to line and grade, and a pumped sand foundation was installed by the Christiani & Nielsen method |
| DREDGING METHOD: | Some 850 m of the trench passed through rock which had to be removed by drilling and blasting. Siltation of the trench required clean-up. |
| COVER AND TYPE: | 0.5 m topping fill, 1.0 m armour rock |
| ADDITIONAL INFORMATION: | OWNER: Public Utilities Board of Singapore<br>ENGINEERS: Christiani & Nielsen A/S; Ove Arup & Partners<br>CONTRACTORS: Christiani & Nielsen A/S |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Cooling Water Tunnels; Sizewell B Power Station, United Kingdom; 1992** | | | **S.40 - Sizewell Intake/Outfall**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box elements. Water intake, 500 m and outfall 300 m | | TUBES:<br>One tube | |
| NO OF ELEMENTS: 8 | LENGTH: 100 m | HEIGHT: 6 m | WIDTH: 6 m |
| TOTAL IMMERSED LENGTH: 800 m | | DEPTH AT BOTTOM OF STRUCTURE: 6.5 to 12.5 m | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | 10 year design wave height 6 m | | |
| FABRICATION METHOD:<br>Tunnel elements cast in idle drydock in Teeside. | | JOINT TYPE: | |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | Lowering of tunnel elements by means of two 400 t floating cranes. | | |
| FOUNDATION METHOD: | Screeded stone bed by use of purpose-built equipment. | | |
| DREDGING METHOD: | | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | OWNER: Nuclear Electric<br>ENGINEERS: Nuclear Design Associates<br>CONTRACTOR: Kier Construction | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: **Cooling Water Culverts; South Humber Bank Power Station, United Kingdom; 1996** | | | **S.41 - Humber Intake** |
|---|---|---|---|
| TUNNEL TYPE AND USE: Reinforced concrete box; Cooling water culverts | | TUBES: | |
| NO OF ELEMENTS: 74 | LENGTH: 45 m | HEIGHT: | WIDTH: 6 m |
| TOTAL IMMERSED LENGTH: 1,665 m x 2 | | DEPTH AT BOTTOM OF STRUCTURE: | |
| UNUSUAL FEATURES: | | | |
| ENVIRONMENTAL CONDITIONS: | Current velocity up to 2 m/sec | | |
| FABRICATION METHOD: 74 elements were fabricated within Immingham docks 2 km upriver. Elements were towed barge mounted to open end of trench cofferdam and off-loaded by means of two 400 t shearleg floating cranes. | | OUTFITTING: | JOINT TYPE: Rubber seal 230 mm thick |
| WATERPROOFING METHOD: | | | |
| PLACEMENT METHOD: | Elements placed by cranes, winches and guide ropes from top of sheetpiled cofferdam along trench. | | |
| FOUNDATION METHOD: | Gravel bed | | |
| DREDGING METHOD: | Within 15 m wide open-ended sheetpiled cofferdam, 8 m deep trench | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | OWNER: Humber Power<br>ENGINEER/DESIGNER: Allott & Lomax<br>CONTRACTOR: Kier | | |

## 2. ITA Catalogue of Immersed Transportation Tunnels

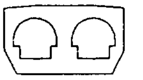
*2.1 Reference List of Immersed Transportation Tunnels*

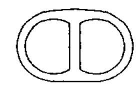
| Ref. No. | Tunnel (s) | Country | Year Completed |
|---|---|---|---|
| T.1 | Detroit River | U.S.A. | 1910 |
| T.2 | La Salle St. | U.S.A. | 1912 |
| T.3 | Harlem River | U.S.A. | 1914 |
| T.4 | Friedrichshagen | Germany | 1927 |
| T.5 | Oakland-Alameda (Posey) | U.S.A. | 1928 |
| T.6 | Detroit Windsor | U.S.A. | 1930 |
| T.7 | Bankhead | U.S.A. | 1940 |
| T.8 | State Street | U.S.A. | 1942 |
| T.9 | Maas | Netherlands | 1943 |
| T.10 | Aji River | Japan | 1944 |
| T.11 | Washburn | U.S.A. | 1950 |
| T.12 | Elizabeth River No. 1 | U.S.A. | 1952 |
| T.13 | Baytown | U.S.A. | 1953 |
| T.14 | Baltimore Harbor | U.S.A. | 1957 |
| T.15 | Hampton Roads No. 1 | U.S.A. | 1957 |
| T.16 | Havana | Cuba | 1958 |
| T.17 | Deas Island | Canada | 1959 |
| T.18 | Rendsburg | Germany | 1961 |
| T.19 | Webster Street | U.S.A. | 1962 |
| T.20 | Elizabeth River No. 2 | U.S.A. | 1962 |
| T.21 | Chesapeake Bay | U.S.A. | 1964 |
| T.22 | Liljeholmsviken | Sweden | 1964 |
| T.23 | Haneda (vehicular) | Japan | 1964 |
| T.24 | Haneda (monorail) | Japan | 1964 |
| T.25 | Coen | Netherlands | 1966 |
| T.26 | Wolfburg Pedestrian | Germany | 1966 |
| T.27 | Rotterdam Metro | Netherlands | 1966 |
| T.28 | Benelux | Netherlands | 1967 |
| T.29 | Lafontaine | Canada | 1967 |
| T.30 | Vieux-Port | France | 1967 |
| T.31 | Tingstad | Sweden | 1968 |
| T.32 | Ij | Netherlands | 1968 |
| T.33 | J.F. Kennedy (Scheldt E3) | Belgium | 1969 |
| T.34 | Heinenoord | Netherlands | 1969 |
| T.35 | Limfjord | Denmark | 1969 |
| T.36 | Parana (Hernandias) | Argentina | 1969 |
| T.37 | Dojima River | Japan | 1969 |
| T.38 | Dohtonbori River | Japan | 1969 |
| T.39 | Haneda (Tama River) | Japan | 1970 |
| T.40 | Haneda (Keihin Channel) | Japan | 1970 |
| T.41 | Bay Area Rapid Transit | U.S.A. | 1970 |
| T.42 | Charles River | U.S.A. | 1971 |
| T.43 | Cross Harbour | Hong Kong | 1972 |
| T.44 | 63rd Street | U.S.A. | 1973 |
| T.45 | Interstate Route 10 (Mobile) | U.S.A. | 1973 |
| T.46 | Kinuura Harbour | Japan | 1973 |
| T.47 | Ohgishima | Japan | 1974 |
| T.48 | Elbe | Germany | 1975 |
| T.49 | Vlake | Netherlands | 1975 |
| T.50 | Kanonerski | Russia | 1975 |
| T.51 | Sumida | Japan | 1976 |
| T.52 | Hampton Roads No. 2 | U.S.A. | 1976 |
| T.53 | Paris Metro | France | 1976 |

*continued on following page*

| Ref. No. | Tunnel (s) | Country | Year Completed |
|---|---|---|---|
| T.54 | Tokyo Port | Japan | 1976 |
| T.55 | Drecht | Netherlands | 1977 |
| T.56 | Prinsess Margriet | Netherlands | 1978 |
| T.57 | Kil | Netherlands | 1978 |
| T.58 | WMATA Washington (DC) | U.S.A. | 1979 |
| T.59 | Hong Kong Mass Transit | Hong Kong | 1979 |
| T.60 | Hemspoor | Netherlands | 1980 |
| T.61 | Botlek | Netherlands | 1980 |
| T.62 | Daiba | Japan | 1980 |
| T.63 | Tokyo Port Dainikoro | Japan | 1980 |
| T.64 | Kawasaki | Japan | 1981 |
| T.65 | Rupel | Belgium | 1982 |
| T.66 | Metropolitan Rail Main | Germany | 1983 |
| T.67 | Bastia Old Harbour | France | 1983 |
| T.68 | S-Bahn Rein-Main | Germany | 1983 |
| T.69 | Coolhaven | Netherlands | 1984 |
| T.70 | Kaohsiung Harbour | Taiwan | 1984 |
| T.71 | Spijkenisse Metro | Netherlands | 1985 |
| T.72 | Fort McHenry | U.S.A. | 1985 |
| T.73 | Second Downtown | U.S.A. | 1988 |
| T.74 | Guldborgsund | Denmark | 1988 |
| T.75 | Ems | Germany | 1989 |
| T.76 | Marne River | France | 1989 |
| T.77 | Zeeburger | Netherlands | 1989 |
| T.78 | Eastern Harbour Crossing | Hong Kong | 1989 |
| T.79 | Conwy | United Kingdom | 1991 |
| T.80 | Liefkenshoek | Belgium | 1991 |
| T.81 | Monitor-Merrimac | U.S.A. | 1992 |
| T.82 | Sydney Harbour | Australia | 1992 |
| T.83 | Grouw | Netherlands | 1992 |
| T.84 | Noord | Netherlands | 1992 |
| T.85 | Pearl River | P.R. China | 1993 |
| T.86 | Météor | France | 1994 |
| T.87 | Ted Williams | U.S.A. | 1994 |
| T.88 | Willemspoor | Netherlands | 1994 |
| T.89 | MTRC | Hong Kong | 1994 |
| T.90 | Bilbao Metro | Spain | 1994 |
| T.91 | Tama River | Japan | 1994 |
| T.92 | Schiphol | Netherlands | 1995 |
| T.93 | Medway | United Kingdom | 1996 |
| T.94 | Wijker | Netherlands | 1996 |
| T.95 | Yong River | P. R. China | 1996 |
| T.96 | Piet Hein | Netherlands | 1997 |
| T.97 | MTRC | Hong Kong | 1997 |
| T.98 | Western Harbour Crossing | Hong Kong | 1997 |
| T.99 | Aquaduct | Netherlands | 1997 |
| T.100 | Niigata Port Road | Japan | ¤ |
| T.101 | Kawasaki Fairway | Japan | ¤ |
| T.102 | Osaka South Port | Japan | ¤ |
| T.103 | Aktion-Preveza | Greece | ¤ |
| T.104 | River Lee | Eire | ¤ |
| T.105 | Fort Point Channel | U.S.A. | ¤ |
| T.106 | Drogden | Denmark | ¤ |
| T.107 | Tokyo Port Seaside Road | Tokyo, Japan | ¤ |
| T.108 | Kobe Port Minatojima | Japan | ¤ |

¤ indicates tunnel planned or under construction

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Detroit River Tunnel; International tunnel between Detroit, Michigan, U.S.A. and Windsor, Canada; 1910</td><td>T.1 - Detroit River<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Steel shell lined with concrete. Exterior concrete ballast. Railroad tunnel</td><td colspan="2">LANES/TRACKS:<br>Two tubes, one track each way</td></tr>
<tr><td>NO OF ELEMENTS: 10</td><td>LENGTH: 78.2 m</td><td>HEIGHT: 9.4 m</td><td>WIDTH: 17 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 782 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 24.4 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Method of construction involved sinking elements by flooding them with water and controlling escaping air. Elements were lowered to steel grillages on the bottom, where all exterior concrete was then placed. Later, the interior of the element was dewatered and the concrete lining was then installed under water.</td></tr>
<tr><td>ENVIRONMETAL CONDITIONS</td><td colspan="3">River currents</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>Shipyard</td><td>OUTFITTING:<br>Underwater after placement</td><td>JOINT TYPE:<br>Unique rubber</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Continuous steel shell plate.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Pontoons were used to control sinking.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Steel grillages were placed to exact grade and tubes were lowered to them. Tremie concrete, used to make the interface contact with the dredged trench, became the foundation for the tunnel elements.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">Dipper dredge for first 14 m; remainder was done by clamshell dredge.</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">Riprap covering over side backfills to protect them from scouring. Tunnel roof at grade of river bottom.</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">If this tunnel were to be constructed today, it would be regarded as incorporating many innovative ideas because it is so different from the way we currently do immersed tube tunnels. Some of the ideas could be very useful someday in a particular situation because they apparently were successful.<br><br>OWNER: Michigan Central Railroad Company<br>DESIGNER: Detroit River Tunnel Company<br>CONTRACTOR: Butler Brothers Construction Co</td></tr>
</table>

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.2 - La Salle St. |
|---|---|
| **La Salle St. Tunnel; Chicago, Illinois, U.S.A.; 1912** |  |

| | |
|---|---|
| TUNNEL TYPE AND USE: Single steel shell (riveted); railway | LANES/TRACKS: Two tubes; each with one track |

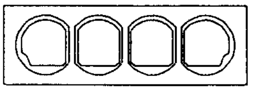
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 1 | LENGTH: 84.8 m | HEIGHT: 7.3 m | WIDTH: 12.5 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 84.8 m | DEPTH AT BOTTOM OF STRUCTURE: 15.5 m |

| | |
|---|---|
| UNUSUAL FEATURES: | Was replacement for masonry tunnel on the same alignment. The masonry tunnel was demolished and removed by dredging. Crossing was made with a single element. Interior lining increased in thickness from top to bottom. Underside of tube had "V" shape to make backfilling easier. Structure had longitudinal central truss encased in concrete. |

| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| Constructed in a drydock in Chicago using a riveted steel shell (to boiler specifications) lined with concrete. Timbered bulkheads were provided. Internal ballast tanks were installed in each tube at each end, operated by remotely controlled valves and pumps. | Interior concrete for the center wall and lower keel area were placed first in drydock. The element was then floated to the outfitting dock (3,000 tons), where the rest of the interior concrete was placed, bringing the total to 8,000 tons. Steel shell roundness was maintained with struts and tierods. | Special diaphragms were provided at the two ends of the single element, to which cofferdams could be engaged to permit land-side construction to proceed. Two manhole shafts were provided at each end of the element for access during and after placement. Cofferdam detail provided for later bridge pier to straddle the tunnel, if required. |

| | |
|---|---|
| WATERPROOFING METHOD: | Double butt strap caulked longitudinal joints and alternate inside and outside lap circumferential caulked joints. Steel shell provided watertightness. Coated with red-lead paint. |
| PLACEMENT METHOD: | Internal water ballast was used for placement. Pile driver barges and/or hoisting engines on adjacent piers were used for control of alignment and grade during placement. |
| FOUNDATION METHOD: | Two-pile supported piers were provided for temporary support. Piles were installed by statically loading them to a known load, rather than driving, in the expectation that adjustments to grade could be made by overloading by a known amount. As tube was placed on supports and adjusted to grade, sand fill was placed around it. |
| DREDGING METHOD: | A dipper dredge and a clamshell dredge were used. A great deal of difficulty was involved in removing the old tunnel, the roof of which had been demolished by blasting. |
| VENTILATION TYPE: | Piston action of trains. |
| ADDITIONAL INFORMATION: | OWNER: Chicago Railways Company.<br>DESIGNERS: E.C. & R.M. Shankland and J.W. Pearl<br>CONTRACTOR:M.H. McGovern / J.A. Green / Charles Green and R.H. Green. |

| | |
|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Harlem River Tunnel; New York City, NY, U.S.A.; 1914** | **T.3 - Harlem River**<br> |

| | |
|---|---|
| TUNNEL TYPE AND USE:<br>Steel shell; Railroad tunnel | LANES/TRACKS:<br>Four tubes; one track each |

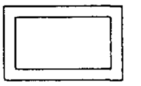
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 5 | LENGTH: 67 m | HEIGHT: 7.5 m | WIDTH: 23.2 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 329 m | DEPTH AT BOTTOM OF STRUCTURE: 15.2 m |

| | |
|---|---|
| UNUSUAL FEATURES: | Four tubes in a single element section. Element placement closely followed Detroit River Tunnel method where element was lowered to grade filled with water and exterior concrete jacket was placed by tremie. Later interior was pumped out and lined with concrete. |

| | | |
|---|---|---|
| FABRICATION METHOD:<br>A yard one mile from tunnel site. Launched by floating off shipways on nine canal boats, then sinking the canal boats out from under the element. | OUTFITTING:<br>Interior concrete linings were installed after the five elements were placed and tremied and the tunnel had been pumped out between bulkheads placed at the ends. Connections to the land sections were made later through cofferdams. Leakage was minimal. | JOINT TYPE:<br>Rivetted liner plate over steel butt joint; grouted |

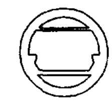
| | |
|---|---|
| WATERPROOFING METHOD: | Continuous steel shell. |
| PLACEMENT METHOD: | 12 inch valves were opened in the bottom of the bulkheads in the two outside tubes, allowing them to fill gradually. The two inner tubes were open. The rate of sinking after the tubes were half full was controlled by air valves. Trim was accomplished by half bulkheads at two central locations to allow adjustment of filled portions. As the tubes became completely filled, the flotation was carried by four pontoons, which were partially filled and went down with the element. The rest of the weight was carried by derrick boats moored on either side. Masts were used to measure line and grade. Once in place, the space below the bottom of the element caused by overdredging was filled with lean tremie concrete. Then each pocket formed by the diaphragms and wood side lagging was filled with structural concrete by the same method. |
| FOUNDATION METHOD: | Tremie concrete placed after element was in position. |
| VENTILATION TYPE: | Piston action of trains |
| ADDITIONAL INFORMATION: | OWNER: New York City Rapid Transit System.<br>CONTRACTOR: Arthur McMullen and Hoff Company. |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Friedrichshagen Tunnel; Berlin, Germany; 1927** | | | **T.4 - Friedrichshagen**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete; Pedestrian | | LANES/TRACKS:<br>One tube; Footway | |
| NO OF ELEMENTS: 2 | LENGTH: 52.9 m | HEIGHT: 6.67 m | WIDTH: 7.65 m |
| TOTAL IMMERSED LENGTH: 105.8 m | | DEPTH AT BOTTOM OF STRUCTURE: 10.8 m | |
| UNUSUAL FEATURES: | Constructed as two pneumatic caissons. Element structure was constructed on fill placed halfway to center of river. Excavation under this structure, which was provided with cutting edges, allowed it to be lowered to grade below the river bottom. One side was done at a time. The two elements were sealed at the middle joint after the second element was at grade. | | |
| FABRICATION METHOD:<br>In place. | | JOINT TYPE:<br>Concrete joint formed in cofferdam. | |
| WATERPROOFING METHOD: | Concrete-protected membrane all around. | | |
| PLACEMENT METHOD: | Caisson method. | | |
| FOUNDATION METHOD: | Concrete filled caisson under tunnel tube. | | |
| VENTILATION TYPE: | Natural ventilation | | |
| COVER AND TYPE: | 1.5 m backfill | | |

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Oakland-Alameda (Posey) Tunnel; between Oakland and Alameda, California, U.S.A.; 1928**

**T.5 - Posey**



| | |
|---|---|
| TUNNEL TYPE AND USE:<br>Reinforced concrete cylindrical section; Vehicular | LANES/TRACKS:<br>One tube; two lanes; one each way |

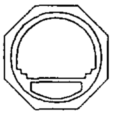
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 12 | LENGTH: 61.9 m | HEIGHT: 11.3 m (dia) | WIDTH: (circular) |

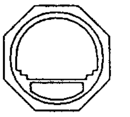
| | |
|---|---|
| TOTAL IMMERSED LENGTH: 742 m | DEPTH AT BOTTOM OF STRUCTURE: 25.5 m |

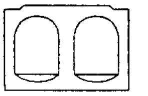
| | |
|---|---|
| UNUSUAL FEATURES: | Circular concrete section (without steel shell) was cast in a drydock. Waterproofed with external membrane protected with timber lagging. Invert waterproofing was laid on timber lagging and ended by metal flashing at end brackets. Roadway slab and ceiling tierods were used for structural members. Placement and foundation methods were very unusual. 122 m at south end were founded on pile-supported tremie concrete cradle. Tunnel ends (vent buildings) are National Historic structures. This tunnel is a significant early example of the Art-Deco Style by Architect Henry H. Meyer. |

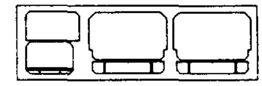
| | | |
|---|---|---|
| FABRICATION METHOD:<br>In drydock with five elements per cycle, chuted concrete, steel forms. | OUTFITTING:<br>At time of fabrication | JOINT TYPE:<br>Tremie concrete joints. |

| | |
|---|---|
| WATERPROOFING METHOD: | As described above. |
| PLACEMENT METHOD: | Ballasting involved placing dry sand on the floor slab before and after the element was positioned over the trench. The lower duct was filled with sea water and an additional load of sand was placed. The final load adjustment was made by wetting the sand on the roadway slab. For the first five joints, sand jacks mounted on pile supported piers were used to land the elements. This method was simplified by using timber grillages lowered to bear on piles driven 15 cm high, to allow for settlement. This grillage was intended to crush under settlement. If the element was not to suitable grade, some of the sand bed had to be removed - a very difficult and time-consuming operation. The elements were supported by a derrick barge at one end and leads from a pile driver at the other. Lateral movements were controlled by lines from winches mounted on dolphins. Immediately after being set in position, more water ballast was added. The exterior sand bed was placed by pumping a sand slurry under the element as directed by divers. On completion of the sand bed, the element was filled with water. The tremie joint was made and completed from inside later. Before dewatering or removing ballast, at least 3.4 m of backfill had to be placed to overcome buoyancy (the stability of the tunnel depends on this cover being maintained throughout its life!). |
| VENTILATION TYPE: | Fully transverse, using lower tube for supply and upper for exhaust. |
| COVER AND TYPE: | See above |
| ADDITIONAL INFORMATION: | Major repairs due to leaching of chlorides were performed on the tube invert in the late 1980's.<br><br>OWNER: Alameda County bond holders<br>DESIGNER: George A Posey Chief Engineer; W.H. Burr, Ole Singstad and Charles Derleth, Jr.<br>CONTRACTOR: California Bridge and Tunnel Co |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Detroit Windsor Tunnel; International tunnel between Detroit, Michigan, U.S.A. and Windsor, Canada; 1930** | | | **T.6 - Detroit Windsor**<br> |
| TUNNEL TYPE AND USE:<br>Double steel shell elements; Vehicular | | LANES/TRACKS:<br>One tube; two lanes; one each way | |
| NO OF ELEMENTS: 9 | LENGTH: 74.3 m | HEIGHT: 10.6 m | WIDTH: 10.6 m |
| TOTAL IMMERSED LENGTH: 669 m | | DEPTH AT BOTTOM OF STRUCTURE: 18.5 m | |
| UNUSUAL FEATURES: | The tunnel project was a combination of open approach structures, cut-and-cover structures, shield-driven tunnelling, and immersed tunnel construction. | | |
| FABRICATION METHOD:<br>Welded steel construction. Steel fabricator; controlled side launching into river 6 miles from site. | | OUTFITTING:<br>At dock near tunnel site. Exterior formwork was erected at outfitting pier. | JOINT TYPE:<br>Tremie concrete. |
| WATERPROOFING METHOD: | Continuous steel shell. | | |
| PLACEMENT METHOD: | Ballasting was done to bring the element to near neutral buoyancy. Concrete blocks were placed at one end to sink it some five feet. A barge was placed transversely to the end and lowering cables were attached. After this was also done at the opposite end, the element could be raised or lowered using these barges. | | |
| FOUNDATION METHOD: | Screeded bedding. | | |
| VENTILATION TYPE: | Fully transverse ventilation system | | |
| COVER AND TYPE: | No cover | | |
| ADDITIONAL INFORMATION: | OWNER: Detroit & Canada Tunnel Corporation.<br>DESIGNER: Parsons Klapp Brinckerhoff & Douglas<br>CONTRACTOR: Porter Brothers and Robert Porter of Spokane, Washington. | | |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Bankhead Tunnel; Mobile, Alabama, U.S.A.; 1940</td><td>T.7 - Bankhead<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Double steel shell tunnel; Vehicular</td><td colspan="2">LANES/TRACKS:<br>One tube; two lanes; one each way</td></tr>
<tr><td>NO OF ELEMENTS: 7</td><td>LENGTH:<br>5 - 90.8 m<br>2 - 78.0 m</td><td>HEIGHT: 10.4</td><td>WIDTH: 10.4</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 610 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 25 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Placed from pile-supported frames. Foundation material was placed with element held in position from frames. Elements were placed a few inches high to accommodate settlement.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS</td><td colspan="3">Close to mouth of river at Mobile Bay, with fresh water wedge sometimes extending beyond the site. The variation from upper salt water into fresh water had to be taken into account in ballasting the elements. Hurricane tide gates were provided at the portals.</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>In shipyard 1 km away from site. The elements were side-launched into river, then placed in shipyard's drydock for placement of keel concrete (for stability). The element was then towed to the outfitting yard near the site.</td><td>OUTFITTING:<br>Concrete was placed from floating batch plant.</td><td>JOINT TYPE:<br>Tremie concrete.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Continuous steel shell.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Gallows frames supported from H-pile clusters.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Element was held at grade while sand was placed along the sides with tremie pipes.</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Longitudinal; fresh air enters at portals, exhausted by air duct (roadway flues near tunnel midpoint).</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">A minimum of 3 m of backfill</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: City of Mobile revenue bond issue<br>DESIGNER: Messrs Wilberding and Palmer Inc (of Washington D.C)<br>CONTRACTOR: Arundel Corporation of Baltimore and the Alabama Drydock and Shipbuilding Company.</td></tr>
</table>

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>State Street Tunnel; Chicago, Illinois, U.S.A.; 1942</td><td>T.8 - State Street<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Double steel shell elements; Railway</td><td colspan="2">LANES/TRACKS:<br>Two tubes; one track each</td></tr>
<tr><td>NO OF ELEMENTS: 1</td><td>LENGTH: 61 m</td><td>HEIGHT: 6.9 m</td><td>WIDTH: 12.0</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 61 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 15.8 m</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS</td><td colspan="3">Mild currents.</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>In a drydock. Welded construction. Pressurized to 0.5 psi for soap bubble test. 1,000 m³ placed in drydock.</td><td>OUTFITTING:<br>At a dock with 6 m of draft available.</td><td>JOINT TYPE:<br>Tremie concrete.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Continuous steel shell.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Element hung from under scows by cables fed through wells. Element had to be sunk below scows to attach them.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Element was landed on tremie concrete pads formed to exact grade; screw jacks on the corners were used to level the element. A screeded bed was provided as well.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">2.5 m³ clamshell ring</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Piston action of trains.</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: City of Chicago<br>ENGINEER: Department ot Subways and Super Highways<br>CONTRACTOR: Merrit-Chapman & Scott Corporation</td></tr>
</table>

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Maas Tunnel; Rotterdam, The Netherlands, under Nieuwe Maas-River; 1943</td><td>T.9 - Maas<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Concrete box elements; Vehicles, cyclists, pedestrians</td><td colspan="2">LANES/TRACKS:<br>Four lanes in two tubes for vehicles. One tube for cyclists and one tube for pedestrians</td></tr>
<tr><td>NO OF ELEMENTS: 9</td><td>LENGTH: 61.35 m</td><td>HEIGHT: 8.39 m</td><td>WIDTH: 24.77 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 584 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 22.5 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Immersed tunnel terminated in ventilation buildings constructed as pneumatic caissons. Construction spanned the occupation of Holland during Second World War</td></tr>
<tr><td>FABRICATION METHOD:</td><td colspan="3">Existing dock in Heyse Harbor. Roof slab was left off until the element was floating in Waal Harbor. Three cycles of three elements were used.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">6 mm steel membrane covered with coating of concrete to inhibit rusting.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Lowering by means of floating cranes. Pontoons along the sides of the elements were used to provide positive buoyancy</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Sandjetted foundation. Very first application of Christiani & Nielsen method</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Transverse ventilation system</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">CLIENT/OWNER: Gemeente Rotterdam (Municipality of Rotterdam)<br>DESIGNER: Christiani & Nielsen A/S, Copenhagen<br>CONTRACTOR: N.V. Maastunnel, a joint venture consisting of:<br>Christiani & Nielsen N.V. of the Hague, N.V. Internationale Gewapend Beton Bouw (now member of Royal Bos Kalis Westminster Group), N.V. Nederlandsche Aanneming Mij (now member of Ballast-Nedam Group), M.J. Hattum Havenwerken NV, and W. Blankevoort Czm's Aanneming Mil, NV (now member of Stevin Group)</td></tr>
</table>

| | |
|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Aji River Tunnel; Osaka, Japan; 1944** | **T.10 - Aji River**<br> |

| | |
|---|---|
| TUNNEL TYPE AND USE:<br>Vehicular and pedestrian;<br>Single shell steel box section | LANES/TRACKS:<br>Two tubes; one lane each with walkway duct |

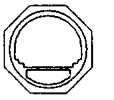
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 1 | LENGTH: 49.2 m | HEIGHT: 7.0 m | WIDTH: 14.0 m |

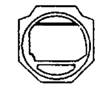
| | |
|---|---|
| TOTAL IMMERSED LENGTH: 49.2 m | DEPTH AT BOTTOM OF STRUCTURE: 14.9 m |

| | |
|---|---|
| UNUSUAL FEATURES: | Structure acts like a bridge. Supported at both ends by abutments constructed by pneumatic caisson method. Single concrete element reinforced with reinforcing steel and rolled steel sections. |
| ENVIRONMENTAL CONDITIONS | The depth of water was only 5.5-6.3 m. All navigation had to be stopped for nearly 15 hours during placement of the element. |

| | | |
|---|---|---|
| FABRICATION METHOD:<br>The outside 9 mm shell was fabricated at a shipyard. The element was then outfitted with reinforcing steel and concrete at a dock. | OUTFITTING:<br>In flotation at dockside 14 km from tunnel site in Osaka Bay. | JOINT TYPE:<br>Rigid concrete filled joint to abutment structures. |

| | |
|---|---|
| WATERPROOFING METHOD: | Continuous steel shell. |
| PLACEMENT METHOD: | The element was towed supported by two barges and placed using floating cranes. Exact position between elements was controlled by divers. |
| FOUNDATION METHOD: | Two L-shaped abutment caissons were sunk in place at the river shorelines. The river bed between the abutments was excavated and the element was supported on the abutments, rather than on a prepared foundation, as might normally be the case. |
| VENTILATION TYPE: | A semi-transverse ventilation system was first adopted; however, the tunnel ventilation was later converted to a longitudinal system. |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Washburn Tunnel; between Houston and Galveston, Texas, U.S.A.; 1950** | | | **T.11 - Washburn**<br> |
| TUNNEL TYPE AND USE:<br>Double steel shell elements; Vehicular tunnel | | LANES/TRACKS:<br>One tube; two lanes, one each way | |
| NO OF ELEMENTS: 4 | LENGTH: 114.37 m | HEIGHT: 9.75 I.D. | WIDTH: 10.97 |
| TOTAL IMMERSED LENGTH: 457 m | | DEPTH AT BOTTOM OF STRUCTURE: 24 m | |
| FABRICATION METHOD:<br>At shipyard in Pascagoula, Miss. end launched and towed 640 km to Pasadena, Texas. Special launching reinforcing was removed and ballast concrete was placed at an outfitting dock before the tow to Texas. | | JOINT TYPE:<br>Tremie concrete. | |
| WATERPROOFING METHOD: | Continuous steel shell. | | |
| VENTILATION TYPE: | Semi-transverse | | |
| COVER AND TYPE: | Sand fill | | |
| ADDITIONAL INFORMATION: | OWNER: Local associations, The Harbour of Houston<br>DESIGNER: Palmer & Baker, Mobile, Alabama<br>CONTRACTOR: Merritt, Chapman and Scott, New York | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T. 12 - Elizabeth River No. 1 |
|---|---|
| **Elizabeth River Tunnel No 1; between Norfolk and Portsmouth, Virginia, U.S.A.; 1952** |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Double steel shell elements; Vehicular | One tube; two lanes, one each way |

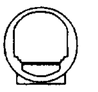
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 7 | LENGTH: 91.5 m | HEIGHT: 10.9 m | WIDTH: 10.4 m |

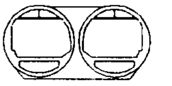
| | |
|---|---|
| TOTAL IMMERSED LENGTH: 638 m | DEPTH AT BOTTOM OF STRUCTURE: 29 m |

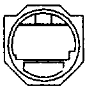
| | |
|---|---|
| UNUSUAL FEATURES: | Sequence of construction started with Element A being placed prior to construction of west cut-and-cover section. Elements B, C, etc. followed, until the final element was placed and connected under water to east cut-and-cover section already in place.<br><br>Water depth required that the last element be temporarily "parked" some 0.5 m beyond the next to-the-last section until that section was connected to the preceding one. The final section was then moved back and connected to both the preceeding element and the cut-and-cover section. This was done within a sheeted trench, using land cranes.<br><br>Ventilation building was independently supported on piles off to one side of the tunnel. Ventilation tubes had flexible connections to the tunnel structure. to allow for differential settlements. |

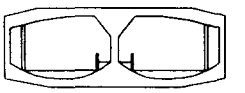
| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| Fabricated on slipways, provided with 1,000 tons of concrete for keel, fitted with a launching bow and end-launched. | At dock near tunnel site. | Tremie concrete joint. |

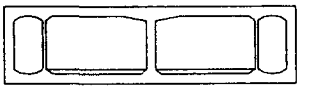
| | |
|---|---|
| WATERPROOFING METHOD: | Continuous steel shell. |
| PLACEMENT METHOD: | Two floating derricks; 100-ton negative buoyancy |
| FOUNDATION METHOD: | Screeded bedding. |
| VENTILATION TYPE: | Semi-transverse system, whereby vitiated air is withdrawn at top and bottom air ducts and fresh air enters through portals. |
| COVER AND TYPE: | 1.5 m minimum backfill |
| ADDITIONAL INFORMATION: | OWNER: Elizabeth River Tunnel Commission, Commonwealth of Virginia<br>DESIGNER: Parsons, Brinckerhoff, Hall and McDonald, New York<br>CONTRACTOR: Merrit Chapman and Scott Corporation |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Baytown Tunnel; Baytown, Texas, U.S.A.; 1953** | | | **T.13 - Baytown**<br> |
| TUNNEL TYPE AND USE:<br>Single steel shell elements; Vehicular | | LANES/TRACKS:<br>One tube; two lanes, one each way | |
| NO OF ELEMENTS: 9 | LENGTH: 91.4 and 76.2 m | HEIGHT: 10.62 OD<br>8.94 ID | WIDTH: Circular |
| TOTAL IMMERSED LENGTH: 778.5 m | | DEPTH AT BOTTOM OF STRUCTURE: 33.5 m | |
| UNUSUAL FEATURES: | Circular element with 25 mm exterior steel shell. Interior concrete ring 91 cm thick. Temporarily supported on fabricated steel chair until backfilled. Two alternates were bid; the other was a double steel shell design. | | |
| OUTFITTING:<br>At work pier constructed near site. | | JOINT TYPE:<br>Tremie concrete joint. | |
| WATERPROOFING METHOD: | Continuous steel shell plate, 25 mm. | | |
| PLACEMENT METHOD: | Supported by two straddling barges attached with the element sitting on bottom. | | |
| FOUNDATION METHOD: | Screeded bedding. | | |
| VENTILATION TYPE: | Semi-transverse with fresh air supplied through lower, under-roadway air duct and exhausted at portals. | | |
| COVER AND TYPE: | 1.5 m minimum backfill | | |
| ADDITIONAL INFORMATION: | OWNER: State of Texas Highway Department and U.S. Bureau of Public Roads<br>DESIGNER: Parsons, Brinckerhoff, Hall and McDonald<br>CONTRACTOR: Brown & Root, Inc (of Houston), Texas | | |

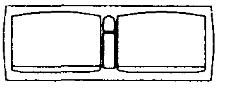
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br>**Baltimore Harbor Tunnel; Baltimore, Maryland, U.S.A.; 1957** | | | **T.14 - Baltimore Harbor**<br> |
| TUNNEL TYPE AND USE:<br>Double-steel shell elements; Vehicular tunnel | | LANES/TRACKS:<br>Two tubes, four lanes; two each way | |
| NO OF ELEMENTS: 21 | LENGTH: 91.4 m | HEIGHT: 10.7 m | WIDTH: 21.3 m |
| TOTAL IMMERSED LENGTH: 1920 m | | DEPTH AT BOTTOM OF STRUCTURE: 30 m | |
| UNUSUAL FEATURES: | Shell plate protected with 6.4 cm gunnite coating over exterior 120 degree section of both tubes. | | |
| ENVIRONMENTAL CONDITIONS: | Mild currents and tides. | | |
| FABRICATION METHOD:<br>Shipyard in Baltimore and end-launched from shipways | | OUTFITTING:<br>At dock near site | JOINT TYPE:<br>Tremie concrete joints |
| WATERPROOFING METHOD: | Continuous steel shell plate. | | |
| PLACEMENT METHOD: | Two to four floating derricks were used. | | |
| FOUNDATION METHOD: | Screeded bedding. | | |
| DREDGING METHOD: | Cutterhead suction dredge. Spoil used for development of port site. | | |
| VENTILATION TYPE: | Full transverse ventilation system | | |
| ADDITIONAL INFORMATION: | OWNER: Maryland State Roads Commission<br>DESIGNER: Singstad & Baillie, and J E Greiner Company<br>CONTRACTOR: Merritt-Chapman & Scott, Inc, C J Langenfelder and Son Inc, Leo Butler and Co, in joint venture | | |

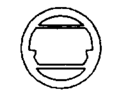
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Hampton Roads Bridge Tunnel No 1; Hampton Roads, Virginia, U.S.A.; 1957** | | | **T.15 - Hampton Roads No. 1**<br> |
| TUNNEL TYPE AND USE:<br>Double steel shell elements; Vehicular tunnel | | LANES/TRACKS:<br>One tube, two lanes; one each way (later became north-bound lanes when Second HRT was completed in 1976) | |
| NO OF ELEMENTS: 23 | LENGTH: 91.5 m | HEIGHT: 11.25 m | WIDTH: 11.25 |
| TOTAL IMMERSED LENGTH: 2091 m | | DEPTH AT BOTTOM OF STRUCTURE: 37 m | |
| UNUSUAL FEATURES: | First immersed tunnel to be constructed from two man-made islands. From the islands, this crossing reached the shores by way of two causeways. Later, four other tunnels using this same arrangement were constructed in the Hampton Roads and Chesapeake Bay area. | | |
| ENVIRONMENTAL CONDITIONS: | Strong currents and wave action made island building difficult until rock dikes were constructed. Seaward face of islands had to be protected by cyclopean-sized armor stone to prevent erosion. | | |
| FABRICATION METHOD:<br>Shipyard in Baltimore | | OUTFITTING:<br>At dock near site of tunnel | JOINT TYPE:<br>Tremie concrete joints. |
| WATERPROOFING METHOD: | Continuous steel shell plate. | | |
| PLACEMENT METHOD: | Catamaran barges. | | |
| FOUNDATION METHOD: | Screeded bedding. | | |
| DREDGING METHOD: | Cutterhead suction dredge for shallower portions of trench. Sand was used to build the islands. Deeper portions were done with a clamshell bucket dredge. | | |
| VENTILATION TYPE: | Full transverse system. | | |
| COVER AND TYPE: | 1.5 m of sand, in some cases protected with riprap against scour. Backfill came from channel dredging being done concurrently by U.S: Corps of Engineers. | | |
| ADDITIONAL INFORMATION: | OWNER: Virginia Highway Department<br>DESIGNER: Parsons, Brinckerhoff, Hall and McDonald<br>CONTRACTOR: Merritt-Chapman & Scott, Inc | | |

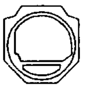
<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Havana Harbour Tunnel; Havana, Cuba; 1958</td><td>T.16 - Havana<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Prestressed concrete box elements; Vehicular tunnel</td><td colspan="2">LANES/TRACKS:<br>Two tubes; two lanes each</td></tr>
<tr><td>NO OF ELEMENTS: 5</td><td>LENGTH:<br>4 - 107.5 m<br>1 - 90.0 m</td><td>HEIGHT: 7.10 m</td><td>WIDTH: 21.85 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 520 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 23 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">First fully prestressed immersed tunnel. Both longitudinal and transverse prestressing was utilized.</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>All by timber formwork in a casting basin in groups of two</td><td colspan="2">JOINT TYPE:<br>Tremie concrete joints.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Placed from catamaran placing barges. Pontoons were used to assist flotation of element prior to sinking.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Final support for each element consisted of two rows of concrete chairs, each about 9 m long, poured under water in special formwork with jute sacking at the bottom and closed wire mesh on the sides.</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Partly longitudinal and partly semi-transverse</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: Government of Cuba<br>DESIGNER: Société des Grands Travaux de Marseille<br>CONTRACTOR: Société des Grands Travaux de Marseille</td></tr>
</table>

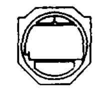
| | |
|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Deas Island Tunnel; Vancouver, British Columbia, Canada; 1959** | **T.17 - Deas Island**<br> |

| | | | |
|---|---|---|---|
| TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Vehicular tunnel | | LANES/TRACKS:<br>Two tubes; four lanes (two in each tube) | |
| NO OF ELEMENTS: 6 | LENGTH: 104.9 m | HEIGHT: 7.16 m | WIDTH: 23.8 m |
| TOTAL IMMERSED LENGTH: 629 m | | DEPTH AT BOTTOM OF STRUCTURE: 22 m | |
| ENVIRONMENTAL CONDITIONS: | Extensive river, tunnel placement, and cover stability laboratory modelling was conducted. Designed for earthquake loadings (Zone 3). | | |
| FABRICATION METHOD:<br>Casting basin next to tunnel site for all six elements | OUTFITTING:<br>At outfitting jetty next to tunnel site | JOINT TYPE: Inflatable rubber-gasketted joint used for initial seal. Final seal made in conventional way, by dewatering joint and mobilizing hydrostatic pressure. Monolithic permanent joint. | |
| WATERPROOFING METHOD: | 5 mm steel plate on bottom lapping with bituminous membrane up the sides and over the roof slab. The waterproofing was protected with a 10 cm layer of reinforced concrete under the bottom and on the top and by 10 cm of wood planking on the walls. | | |
| PLACEMENT METHOD: | Four barges, two on each side of the element, were arrayed in a catamaran arrangement. Manuevering lines included vertical lifting lines, transverse and longitudinal tag lines and rigging from fairleads on the element, acting horizontally to main anchors. Control and survey towers were used to access the inside of the elements and control the positioning of the element | | |
| FOUNDATION METHOD:<br>Sandjetted foundation | | DREDGING METHOD:<br>Cutterhead suction dredging; also used for casting basin. | |
| VENTILATION TYPE: | Semi-transverse from two ventilation buildings. Fresh air is drawn into the tunnel through the portal. In the second half, air is introduced into the tube and leaves the tunnel at the exit portal. | | |
| COVER AND TYPE: | Double layers of 1,500 lb. stone on top of the structure with additional protection of the sides consisting of 500 lb. stone extending out 50 ft on either side of the tunnel box | | |
| ADDITIONAL INFORMATION: | OWNER: Department of Highways of British Columbia<br>DESIGNERS: Foundation of Canada Engineering Corporation Ltd and Christiani & Nielsen Canada assisted by Christiani & Nielsen Copenhagen<br>CONTRACTOR: Peter Kiewit & Sons Co of Canada Ltd and B.C. Bridge and Dredging Co Ltd Joint Venture; Narod Construction Ltd and Dawson and Hall | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Rendsburg Tunnel; Kiel Canal, Rendsburg, West Germany; 1961** | | | **T.18 - Rendsburg**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Vehicular tunnel | | LANES/TRACKS:<br>Two tubes; two lanes each | |
| NO OF ELEMENTS: 1 | LENGTH: 140 m | HEIGHT: 7.3 m | WIDTH: 20.2 m |
| TOTAL IMMERSED LENGTH: 140 m | | DEPTH AT BOTTOM OF STRUCTURE: 22 m | |
| FABRICATION METHOD:<br>Cast in approach section. Composed of seven 20 m sections tied together with reinforcing steel. | | JOINT TYPE:<br>Gasketted joint. | |
| WATERPROOFING METHOD: | Steel shell on sides and bottom; membrane over top. | | |
| PLACEMENT METHOD: | Hung from pile-supported jacking devices. | | |
| FOUNDATION METHOD: | Bottom screeded from special screed system attached to rails on side of element, which planed the bottom before lowering the element to final position. | | |
| VENTILATION TYPE: | Longitudinal | | |
| ADDITIONAL INFORMATION: | OWNER: Wasser- und Schiffahrtsdirektion, Kiel<br>DESIGNER: Wasser- und Schiffahrtsdirektion, Kiel<br>CONTRACTOR: Joint Venture of Philipp Holzmann, Dyckerhoff & Widmann, Grün & Bilfinger, Siemens Bauunion GmbH and Wayss & Freytag.<br><br>An alternative design prepared by contractor was actually used. | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Webster Street Tunnel; between Oakland and Alameda, California, U.S.A; 1962** | | | **T.19 - Webster Street**<br> |
| TUNNEL TYPE AND USE:<br>Cylindrical reinforced concrete elements; Vehicular tunnel | | LANES/TRACKS:<br>One tube, two lanes | |
| NO OF ELEMENTS: 12 | LENGTH: 61 m | HEIGHT:11.27 m (O.D.) | WIDTH: (circular) |
| TOTAL IMMERSED LENGTH: 732 m | | DEPTH AT BOTTOM OF STRUCTURE: 25 m | |
| UNUSUAL FEATURES: | See below | | |
| FABRICATION METHOD:<br>Constructed in a graving dock excavated 2 miles from the project site. Two elements were constructed per cycle. A 70 ton stop-log gate, which could be removed by a floating crane, was used to permit the elements to exit the graving dock. | | OUTFITTING:<br>During fabrication in graving dock | JOINT TYPE:<br>Tremie concrete joints |
| WATERPROOFING METHOD: | Membrane with timber protection | | |
| PLACEMENT METHOD: | Element first brought to 100 tons positive buoyancy with combination of dry ballast spread on roadway and water ballast in tanks under roadway. The 100 tons were offset and a 50 tons negative buoyancy was developed using water sprinkled on the dry ballast. Derricks on each end supported this modest load and lowered the element in place. Wood chips were used to extend the dry ballast volume. | | |
| FOUNDATION METHOD: | Placed on temporary supports with capacity to take section ballasted to 300 tons. Sand placed with tremie pipe bent to direct ballast under element. Additional turbulence at the end of the pipe was produced using a water jet. Each element was bedded into the original deposit of sand by ballasting to more than 600 tons. This weight was sufficient to guarantee failure of the temporary support after a minimum settlement of 10 cm. Actual settlements were only 2.5 cm, assuring that the load was picked up by the bedding. In the channel 13,000 tons of iron ore were used to provide additional ballast because sand ballast alone could not provide the required weight in the available vertical distance. | | |
| VENTILATION TYPE: | Full transverse ventilation | | |
| ADDITIONAL INFORMATION: | Relatively minor damage resulting from the Loma Prieta Earthquake was repaired in 1990.<br><br>OWNER / ENGINEER: California Division of Highways<br>CONTRACTOR: Pomeroy-Bates and Rogers-Gerwick | | |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Elizabeth River Tunnel No 2; Between Norfolk and Portsmouth, Virginia, U.S.A.; 1962</td><td>T.20 - Elizabeth River No. 2<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Double steel shell tunnel; Vehicular tunnel</td><td colspan="2">LANES/TRACKS:<br>One tube; two lanes, one each way</td></tr>
<tr><td>NO OF ELEMENTS: 12</td><td>LENGTH: 84.2 m</td><td>HEIGHT: 11.0 m</td><td>WIDTH: 10.8 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 1010 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 30 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Due to very poor soil conditions, Element No 1 was supported on timber compaction piles.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Mild currents and tides</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>Side-launched (uncontrolled) from shipyard in Port Deposit, Maryland, about 300 km away. Keel concrete placed for towing stability.</td><td>OUTFITTING:<br>At dock near tunnel site.</td><td>JOINT TYPE:<br>Tremie concrete</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Continuous steel shell plate</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Placement with floating cranes</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Screeded bedding.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">Clamshell dredging</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Semi-transverse ventilation system</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: Elizabeth River Tunnel Commission<br>DESIGNER: Parsons Brinckerhoff, Hall and McDonald<br>CONTRACTOR: Merritt-Chapman & Scott Corporation, Diamond Construction Company, Savannah</td></tr>
</table>

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.21 - Chesapeake Bay |
|---|---|
| **Chesapeake Bay Bridge Tunnel; between Virginia Beach and Cape Charles, Virginia, U.S.A.; 1964** |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Double steel shell elements; Vehicular tunnel | One tube; two lanes, one each way |

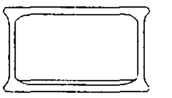
| NO OF ELEMENTS: | LENGTH: 91.4 m | HEIGHT: 11.25 m | WIDTH: 11.25 m |
|---|---|---|---|
| Thimble Shoal Channel: 19<br>Chesapeake Channel: 18 | | | |

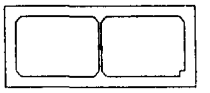
| TOTAL IMMERSED LENGTH: | DEPTH AT BOTTOM OF STRUCTURE(s): |
|---|---|
| Thimble Shoal Channel: 1750 m<br>Baltimore Channel: 1661 m | Thimble Shoal Tunnel: 31.4 m<br>Chesapeake Tunnel: 32.1 m |

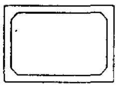
| | |
|---|---|
| UNUSUAL FEATURES: | Part of the 29 km crossing of Chesapeake Bay. The immersed tunnel consisted of two separate tunnels under the two shipping channels: (1) the Thimble Shoal Channel and (2) the Chesapeake Channel. Construction was accomplished in virtually open ocean conditions. Tunnels terminated in man-made islands, similar to those constructed for the first Hampton Roads Tunnel. |
| ENVIRONMENTAL CONDITIONS: | Severe storm exposure. Hurricane destroyed the "Big D" jackleg platform. A storm caused one element to slide down trench some 4 m. |

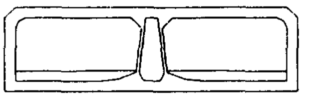
| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| Shipyard in Orange, Texas; end-launched. Trimmed with extra weight of concrete at "stern" 4 ft low for better ocean towing. | At dock near tunnel site. | Tremie concrete joints |

| | |
|---|---|
| WATERPROOFING METHOD: | Continuous steel shell |
| PLACEMENT METHOD: | Catamaran barges |
| FOUNDATION METHOD: | Screeded bedding. |
| VENTILATION TYPE: | Fully transverse ventilation system |
| COVER AND TYPE: | 3 m of selected backfill |
| ADDITIONAL INFORMATION: | OWNER: The Chesapeake Bay Bridge and Tunnel District.<br>DESIGNER: Sverdrup & Parcel and Associates, St Louis<br>CONTRACTOR: Joint venture of Tidewater Construction Corp., Merritt-Chapman & Scott Corp, Raymond International Inc, and Peter Kiewit Sons Co |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Liljeholmsviken Tunnel; Stockholm, Sweden; 1964** | | | **T.22 - Liljeholmsviken**<br> |
| TUNNEL TYPE AND USE:<br>Prestressed concrete box element; Railroad tunnel | | LANES/TRACKS:<br>One tube; two tracks | |
| NO OF ELEMENTS: 1 | LENGTH: 124 m | HEIGHT: 6.13 m | WIDTH: 8.82 m |
| TOTAL IMMERSED LENGTH: 124 m | | DEPTH AT BOTTOM OF STRUCTURE: 13 m | |
| UNUSUAL FEATURES: | This tunnel was constructed in two separate sections, which were post-tensioned together in flotation and placed as a single element. This element was designed to act as a continuous structure to span 53 m between two supports in rock. In effect, this was an underwater bridge between two rock slopes.<br><br>A year after completion, a compressive force of 5000 tons was introduced into a temporary joint between a non-prestressed section constructed in a cofferdam and the abutting immersed tunnel. Jacks were used. This arrangement put the whole tunnel into a longitudinal prestressed condition. No leakage has been reported. | | |
| ENVIRONMENTAL CONDITIONS: | Crossing between two rock faces underwater. | | |
| FABRICATION METHOD:<br>In a drydock | | JOINT TYPE:<br>Constructed in dry cofferdams. | |
| WATERPROOFING METHOD: | Temperature matching between the top and bottom sections of the walls was used to prevent cracking. The lower part of the structure was warmed before casting the upper part to avoid tensile cracking in the walls. Waterproof concrete and the full prestressing in all three axes made the section virtually watertight. No membrane was employed. | | |
| PLACEMENT METHOD: | Lowered by means of hydraulic jacks supported by temporary portal frame at each end. Ballasting by partial water fill. | | |
| FOUNDATION METHOD: | Lowered onto fixed supports | | |
| DREDGING METHOD: | None required for immersed tunnel section. | | |
| VENTILATION TYPE: | Piston action of trains | | |
| COVER AND TYPE: | Tunnel was not covered | | |
| ADDITIONAL INFORMATION: | The tunnel has been designed to withstand the load of a 1,500 ton ship resting on it or the tunnel completely filling with water.<br><br>OWNER: Stockholm Stads Gatukontor | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Haneda Highway Tunnel (Ebitori River Tunnel); Tokyo, Japan; 1964** | | **T.23 - Haneda (vehicular)**<br> | |
| TUNNEL TYPE AND USE:<br>Steel shell rectangular section; Vehicular | | LANES/TRACKS:<br>Two tubes; two lanes in each tube | |
| NO OF ELEMENTS: 1 | LENGTH: 56.0 m | HEIGHT: 7.4 m | WIDTH: 20.1 m |
| TOTAL IMMERSED LENGTH: 56.0 m | | DEPTH AT BOTTOM OF STRUCTURE: 12 m | |
| UNUSUAL FEATURES: | Element is supported on caisson foundations at both ends (ie creating a sort of bridge under water). The design provided for composite action between steel shell and concrete structure. See also "Fabrication Method", described below.<br><br>First full-scale immersed tunnel in Japan | | |
| ENVIRONMENTAL CONDITIONS: | Haneda International Airport is located near the site. | | |
| FABRICATION METHOD:<br>Steel shell was constructed in shipyard. Concrete for bottom slab was placed in flotation. The sidewalls and roof slab were pumped with concrete after the element was placed. | | JOINT TYPE:<br>U-shaped steel plate over rubber joint. | |
| WATERPROOFING METHOD: | Steel skin on sides and bottom. Concrete protected membrane on top of element. | | |
| PLACEMENT METHOD: | Three heavy lift cranes were used to control placement. | | |
| FOUNDATION METHOD: | Caisson foundations | | |
| VENTILATION TYPE: | Longitudinal ventilation | | |
| ADDITIONAL INFORMATION: | OWNER: Metropolitan Expressway Public Corporation<br>DESIGNER: Pacific Consultants K.K., Ishihawajima Harima Heavy Industries Co Ltd<br>CONTRACTOR: Kajima Construction Co Ltd | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Haneda Highway Monorail Tunnel (Ebitori River); Tokyo, Japan; 1964** | | | **T.24 - Haneda (monorail)**<br> |
| TUNNEL TYPE AND USE:<br>Single-shell steel box section; Monorail system | | LANES/TRACKS:<br>One tube; two tracks | |
| NO OF ELEMENTS: 1 | LENGTH: 56 m | HEIGHT: 7.4 m | WIDTH: 10.95 m |
| TOTAL IMMERSED LENGTH: 56 m | | DEPTH AT BOTTOM OF STRUCTURE: 11.7 m | |
| UNUSUAL FEATURES: | Monorail tunnel. The tunnel is supported at both ends on caisson abutments in common with the Haneda Vehicular tunnel adjacent to it (see T.23) | | |
| ENVIRONMENTAL CONDITIONS: | Height of crane booms was limited because of the tunnel's proximity to Haneda International Airport. | | |
| FABRICATION METHOD:<br>Rectangular steel box section fabricated at the IHI dockyard | | OUTFITTING:<br>After launching, the element was towed to a pier near the tunnel site. The floor slab concrete and protective roof concrete were placed in flotation. The side-wall concrete and ceiling concrete were placed by pumping after the element was immersed. | JOINT TYPE:<br>Rubber gasketted joint. |
| WATERPROOFING METHOD: | Continuous steel shell | | |
| PLACEMENT METHOD: | Heavy lift cranes | | |
| FOUNDATION METHOD: | Element spans between abutments. No special foundation preparation. | | |
| DREDGING METHOD: | Excavation by grab bucket to depth of 11.9 m | | |
| VENTILATION TYPE: | Piston action of trains | | |
| COVER AND TYPE: | Backfill with sand and counterweight of slag on top of tunnel | | |
| ADDITIONAL INFORMATION: | Cathodic protection was applied to steel shell plate.<br><br>OWNER: Hitachi Transport, Tokyo Monorail Corporation. | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.25 - Coen |
|---|---|
| **Coen Tunnel; Amsterdam, Netherlands; 1966** |  |

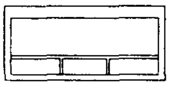
| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Reinforced concrete box section; Vehicular | Two tubes; two lanes in each tube |

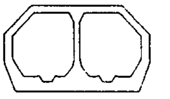
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 6 | LENGTH: 90 m | HEIGHT: 7.74 m | WIDTH: 23.33 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 540 m | DEPTH AT BOTTOM OF STRUCTURE: 24 m |

| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| In casting basin dredged for the purpose. All six sections were cast in one cycle | Outfitting was done partly at the time of fabrication in the basin and partly at the outfitting jetty in the flooded basin. | Gina-type gasket system |

| | |
|---|---|
| WATERPROOFING METHOD: | Asphalt bitumen covered with fiberglass was used to provide a waterproof covering about 1 cm thick. This material was in turn covered with concrete, which was attached to the main concrete box using ramset studs, each with special waterproof steel collars |
| PLACEMENT METHOD: | Four pontoons were used to lower the sections. Towers for survey control and for operations control and access were provided (one tower at each end of the elements) |
| FOUNDATION METHOD: | Placed on four temporary supports resting on pads of gravel and adjusted to grade, using jacks. Sand was later jetted under the elements. |
| DREDGING METHOD: | Hydraulic dredge |
| VENTILATION TYPE: | Longitudinal ventilation supplemented under extreme conditions by semi-transverse ventilation. |
| ADDITIONAL INFORMATION: | OWNER: Ministry of Public Works, Rijkswaterstaat<br>DESIGNER: Locks and Weirs Division of Rijkswaterstaat with Christiani & Nielsen as consultants<br>CONTRACTOR:<br>Tunnel element & approaches: Aannemingsmaatschappij Hillen & Roosen B.V.<br>Sinking & founding: Christiani & Nielsen N.V. |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | | | **T.26 - Wolfburg Pedestrian** |
|---|---|---|---|
| **Wolfburg Pedestrian Tunnels; Wolfburg, Germany - two tunnels under Mittelland Canal, opposite the VW plant; 1966** | | |  |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box section; Footways | | LANES/TRACKS:<br>One tube | |
| NO OF ELEMENTS:<br>1 for each tunnel | LENGTH: 58.0 m | HEIGHT: 5.30 m | WIDTH: 11.0 m (12.52 m at the ventilation shafts) |
| TOTAL IMMERSED LENGTH: 58.0 m per tunnel | | DEPTH AT BOTTOM OF STRUCTURE:<br>10 m below mean water level | |
| UNUSUAL FEATURES: | Because of the narrow shipping canal and the high groundwater level in the adjacent excavations, the ends of the floating elements had to be provided with an upward bend to allow for connections to the landside tunnels. | | |
| ENVIRONMENTAL CONDITIONS: | Virtually no current | | |
| FABRICATION METHOD:<br>Cast in approach excavation | | OUTFITTING:<br>In casting basin during fabrication | JOINT TYPE:<br>Construction in the dry. Joint made watertight with bitumen seal coat |
| WATERPROOFING METHOD: | All around, multiple-layer bitumen seal protected by pearl gravel. Roof slab membrane protected with reinforced concrete slab. | | |
| PLACEMENT METHOD: | Floating and lowering using block and tackle from transverse steel girders supported on the outboard ends of the support of excavation for the approach tunnels | | |
| FOUNDATION METHOD: | Bottom screeded by a plow (girder) attached to steel girders, which ran on slide rails set to grade | | |
| DREDGING METHOD: | Clamshell and hydraulic dredges | | |
| VENTILATION TYPE: | Natural ventilation. Fresh air supply at entrances and exhaust at ventilation shafts also was provided for. | | |
| COVER AND TYPE: | Sandfill | | |
| ADDITIONAL INFORMATION: | OWNER: Volkswagen AG, Wolfsburg<br>OPERATOR: Volkswagen AG, Wolfsburg<br>DESIGNER: Philipp Holzmann AG, Frankfurt<br>CONTRACTOR: Philipp Holzmann AG, Frankfurt | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.27 - Rotterdam Metro |
|---|---|
| **Rotterdam Metro Tunnels; Rotterdam, The Netherlands; 1966** |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Reinforced concrete sections; Metro rail system | Two tubes; one track in each tube |

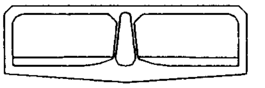
| NO OF ELEMENTS: 36 | LENGTH: 8 - 90 m<br>28 - 75 m | HEIGHT: 6.21 m | WIDTH: 10 m |
|---|---|---|---|

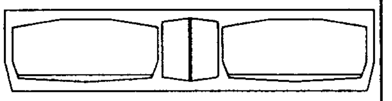
| TOTAL IMMERSED LENGTH: |
|---|
| 1,040 m under the Nieuwe Maas and 1,815 m between Central Station and Leuvehaven |

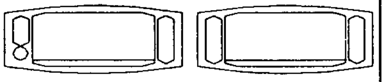
| | |
|---|---|
| UNUSUAL FEATURES: | Because of high ground water and favorable alignment conditions, 24 elements were floated in along the land portion of the Metro system. This included partial station sections as well. Inflatable adjustable pile caps were used to support the tunnel elements |
| ENVIRONMENTAL CONDITIONS: | Construction in high ground water table. Construction through densely populated urban area |

| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| Made of 15 m sections temporarily post-tensioned together into 90 m elements, in a casting basin excavated near the site. | Part of fabrication operation | Gina rubber joints |

| | |
|---|---|
| WATERPROOFING METHOD: | Bituminized felt-covered glass covered with fiberglass fabric. Top and bottom portions were protected with concrete attached with dowels. |
| PLACEMENT METHOD: | Catamaran barges were used for the water crossing. Monorails and hoists were suspended from the upper bracing for the elements placed inside the trenches on land. Water ballast tanks were used for both areas to lower the elements. |
| FOUNDATION METHOD: | Pile foundations were used. Adjustable pile caps inflated with grout were used to support the elements |
| VENTILATION TYPE: | Piston action of trains. |
| ADDITIONAL INFORMATION: | OWNER: Municipality of Rotterdam<br>DESIGNER: The client, Christiani & Nielsen and Hollandsche Beton<br>CONTRACTORS: Christiani & Nielsen N.V. and Hollandsche Beton Maatschappi; sinking was done by the client's organization |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Benelux Tunnel; Rotterdam, The Netherlands; 1967</td><td colspan="2">T.28 - Benelux<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Reinforced concrete box sections; Vehicular tunnel</td><td colspan="3">LANES/TRACKS:<br>Two tubes; two lanes of traffic in each tube</td></tr>
<tr><td>NO OF ELEMENTS: 8</td><td>LENGTH: 93 m</td><td>HEIGHT: 7.84 m</td><td colspan="2">WIDTH: 23.9 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 744 m</td><td colspan="3">DEPTH AT BOTTOM OF STRUCTURE: 24 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="4">Tunnel is completely curved, with all tubes cast as identical curved sections</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="4">Swift river currents created sedimentation problems in the dredged trench</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>At casting basin constructed for the tunnel. All eight sections were cast at the same time.</td><td>OUTFITTING:<br>Took place in casting basin.</td><td colspan="2">JOINT TYPE:<br>Pulled together using a hook, the Gina-type gasket was engaged; conventional dewatering followed to make up the joint</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="4">Bituminous membrane, protected by reinforced concrete</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="4">Used four pontoons for lowering. A tower for survey was provided at the outboard end of the tube; a tower for access and control was provided on the inboard end.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="4">Used foundation plates supported on layer of gravel for temporary grade adjustment. Sand was then jetted under tunnel elements, after acceptable conditions were obtained. Silt skirts were used to protect the area under the elements from being filled with soft silt scour off the bottom of the river</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="4">Longitudinal, using jet fans, with auxiliary semi-transverse system.</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="4">OWNER: Ministry of Public Works; Rijkswaterstaat<br>DESIGNER: Locks and Weirs Department of the Rijkswaterstaat<br>CONTRACTORS: N.V. Amsterdamsche Ballast Mij; Christiani & Nielsen N.V.; Internationale Gewapendbeton-Bouw N.S.; Nederlandsche Aanneming Mij, v/h firma H.F. Boersma and N.V. Nederlandsche Beton Maatschappij (N.V. Nestum II)</td></tr>
</table>

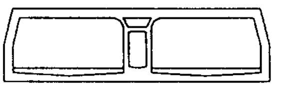
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Lafontaine Tunnel; Montreal, Canada; 1967** | | | **T.29 - Lafontaine**<br> |
| TUNNEL TYPE AND USE:<br>Prestressed concrete box sections; Vehicular tunnel | | LANES/TRACKS:<br>Two tubes; three lanes in each tube | |
| NO OF ELEMENTS: 7 | LENGTH: 109.7 m | HEIGHT: 7.84 m | WIDTH: 36.75 m |
| TOTAL IMMERSED LENGTH: 768 m | | DEPTH AT BOTTOM OF STRUCTURE: 27.5 m | |
| UNUSUAL FEATURES: | Because of difficulty in applying high prestressing forces after the element was placed, combined with the fact that the same forces could not be applied before placing, temporary vertical midspan prestressing was applied to prevent upward buckling of the roof slab. This permitted the transverse prestressing to be fully applied prior to placement. | | |
| FABRICATION METHOD:<br>In casting basin construction in conjunction with approach area | | OUTFITTING:<br>As part of fabrication in casting basin | JOINT TYPE:<br>Gina-type rubber gasketted joint |
| WATERPROOFING METHOD: | Bituminous membrane on walls and roof. Steel membrane on bottom | | |
| PLACEMENT METHOD: | Catamaran barges. Lowering using linear winches. Temporarily supported on four piles | | |
| FOUNDATION METHOD: | Sandjetted foundation | | |
| VENTILATION TYPE: | Semi-transverse | | |
| ADDITIONAL INFORMATION: | OWNER: Department of Highways, Province of Quebec, Canada<br>DESIGNERS: Brett & Ouellette, Lalonde & Valois and Per Hall & Associates. Christiani & Nielsen designed the sandjetting operation<br>CONTRACTOR: Atlas-Winston, Janin (Joint Venture) | | |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Vieux-Port Tunnel; Marseilles, France; 1967</td><td>T.30 - Vieux-Port<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Two side-by-side concrete box sections; Vehicular tunnel</td><td colspan="2">LANES/TRACKS:<br>Two single tube sections, each with two lanes</td></tr>
<tr><td>NO OF ELEMENTS: 6</td><td>LENGTH: 45.4 m</td><td>HEIGHT: 7.16 m</td><td>WIDTH: 14.60 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 273 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 15 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Two tunnels placed beside each other.</td></tr>
<tr><td colspan="2">ENVIRONMENTAL CONDITIONS:<br>Trench dredged in chalk.</td><td>OUTFITTING:</td><td>JOINT TYPE:<br>Gina/Omega</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">4 mm steel membrane on bottom, bituminous membrane on sides and roof.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Stabilization and control of element during sinking by means of vertical cylinders, element being pulled down by tackle connected to anchor blocks installed after dredging.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">After placing the elements were surrounded with an enclosure and fixed by a tremie concrete pour which extended under the element.</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3"></td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Semi-transverse</td></tr>
</table>

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Tingstad Tunnel; Gothenburg, Sweden; 1968**

**T.31 - Tingstad**



TUNNEL TYPE AND USE:
Concrete box tunnel; Vehicular tunnel

LANES/TRACKS:
Two tubes; three lanes each way

| NO OF ELEMENTS: 5 | LENGTH: 4 - 93.5 m<br>1 - 80.0 m | HEIGHT: 7.3 m | WIDTH: 29.9 m |
|---|---|---|---|

TOTAL IMMERSED LENGTH: 454 m

DEPTH AT BOTTOM OF STRUCTURE: 16 m

ENVIRONMENTAL CONDITIONS: Situated in a valley of postglacial clay. Siltation of bottom during placement of elements

FABRICATION METHOD:
Steel shell (bottom and walls) constructed in shallow dock. Concreting at jetty in floating condition

OUTFITTING:

JOINT TYPE:
Gasketted joints

WATERPROOFING METHOD: 6 mm steel membrane all around. Roof and slanting parts coated with 10 cm of concrete; remainder protected by anti-corrosive paint. Cathodic protection is also provided.

FOUNDATION METHOD: Elements were placed on large nylon sacks that could be pumped full of grout. These sacks formed the adjustment between the element and piled foundations along the element. After the bottom of the trench was levelled with gravel, timber piles approximately 22 m long were driven into the clay, which is 80-100 m deep. The space under the elements at the pile groups had to be protected against the incursion of mud. After the sacks had been grouted first, to assure good contact, the other areas were also grouted.

VENTILATION TYPE: Semi-transverse ventilation with ventilation buildings at each end of the tunnel

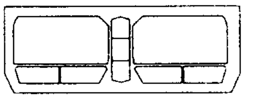
BUOYANCY SF: 1.1

CONCRETE CUBE STRESS: 35 MN/m²

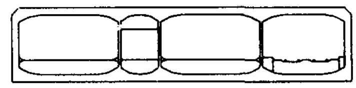
COVER AND TYPE: Protection stone layer 0.5 m thick was placed over the tunnel

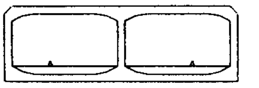
ADDITIONAL INFORMATION:
No damage to the concrete has been observed. The lower parts of the walls are covered by tiles. No unexpected settlements have occurred. Maximum settlements, most of it occurring early, ranged between 40 and 50 mm. Minor cracks occurred in the wall and roof. Cathodic protection has been renewed once. Settlements directly after immersion were approximately 10 mm. After backfilling, the settlements increased to 20 mm. Backfill surcharge was approximately 50 kN/m². The differences in settlements were caused by variation in ground conditions and depth of surcharge. Areas of the facilities that were not pile-supported have settled more than 0.5 m

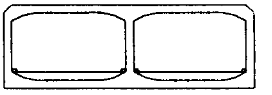
OWNER: Swedish National Road Administration
DESIGNER:
Preliminary: Christiani & Nielsen A/S
Final: Skanska
CONTRACTOR: Skanska

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.32 - Ij |
|---|---|
| **Ij Tunnel; Amsterdam, The Netherlands; 1968** |  |

| | | | |
|---|---|---|---|
| TUNNEL TYPE AND USE:<br>Reinforced concrete box section; Vehicular tunnel | | LANES/TRACKS:<br>Two tubes; two lanes in each tube | |
| NO OF ELEMENTS: 9 | LENGTH: 90 m (up to) | HEIGHT: 8.55 m | WIDTH: 23.9 m |
| TOTAL IMMERSED LENGTH: 786 m | | DEPTH AT BOTTOM OF STRUCTURE: 23.63 | |
| UNUSUAL FEATURES: | Two sections on precast piles were made in open excavation between sheet pile walls; one section was made by means of four pneumatic caissons. Each of the immersed sections was subdivided by two temporary joints and was supported on four pile caps constructed in the dry at the bottom of the river under a pressurized air chamber. | | |
| FABRICATION METHOD:<br>Started one element at a time then enlarged casting basin to permit two at a time | | OUTFITTING:<br>Part of fabrication operation in the casting basin | JOINT TYPE:<br>Gina-type gasket |
| PLACEMENT METHOD: | Tunnel elements were pulled downward onto the two central pile caps, using multipart lines. Water ballast was added to provide a negative buoyancy of 200 tons. The cables were disconnected and jacking devices were installed by divers to pull the element in and make initial gasket contract. The water pressure in the joint space was then released to compress the gasket. The end sections were closed using exterior plates with rubber gaskets, which permitted the joint concrete to be formed in a dry environment. | | |
| FOUNDATION METHOD: | Elements were placed on cast-in-place caps described above. | | |
| VENTILATION TYPE: | Fully transverse, with air supply and exhaust ducts under roadway. Fresh air enters on the right side and crosses to the left side, exiting through large louvres. | | |
| ADDITIONAL INFORMATION: | OWNER: Municipality of Amsterdam<br>DESIGNER: Department of Public Works Service in Amsterdam<br>CONTRACTOR: Combinatie Bouw Ij-Tunnel (COBIJT) Joint Venture:<br>Bataafse Aannemings Mij<br>Nederlandsche Aannemings Mij<br>Aannemersbedrijf v/h J.P. Broekhoven<br>NV Tot Aanneming van Werken v/h H.J. Nederhorst<br>Hollandsche Aannemersbedrijf Zanen en Verstoep<br>Ways & Freytag AG, Frankfurt am Main<br>Philipp Holzmann AG, Düsseldorf | | |

| | |
|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:** J.F. Kennedy Tunnel (Scheldt E3); Antwerp, Belgium; 1969 | **T.33 - Scheldt E3**  |
| TUNNEL TYPE AND USE: Prestressed concrete box elements; Vehicular and railroad | LANES/TRACKS: Two roadway tubes with three lanes in each tube; one railroad tube with two tracks |
| NO OF ELEMENTS: 5 | LENGTH: 4 - 99 m<br>1 - 115 m |
| HEIGHT: 10.1 m | WIDTH: 47.85 m |
| TOTAL IMMERSED LENGTH: 510 m | DEPTH AT BOTTOM OF STRUCTURE: 25 m |
| UNUSUAL FEATURES: | Extremely wide tunnel with combined rail and road usage. The project used the largest tunnel elements ever constructed at the time (47,000 tons). A quay wall, which was replaced over and supported on the tunnel, was constructed as hollow concrete caissons |
| ENVIRONMENTAL CONDITIONS: | River currents of up to 3.0 m/s. Tides with a mean range of 4.8 m and a maximum range of 8.86 m |
| FABRICATION METHOD: All five elements were cast in one cycle in a casting basin near the tunnel site. | JOINT TYPE: Gina gasket for temporary joining. Tunnel monolithic with ventilation buildings |
| WATERPROOFING METHOD: | 5 mm steel sheet on bottom painted with tar epoxy. Joints between the single sheets were sealed with bituminous strips. Walls and roofs were provided with a three ply membrane. On the walls, membrane was protected by timber sheeting supported on steel beams fixed to the concrete. The 3 cm space between the timber and the membrane was filled with mortar. Roof protection consisted of a 10 cm reinforced concrete slab. Bulkheads were constructed of 14 mm steel plate supported on horizontal and vertical beams (to save weight). The joints between the sheets were covered with bitumen mastic and the whole bulkhead was covered with a 2 mm butyl membrane. |
| PLACEMENT METHOD: | Ten tugs (1200 HP) were required to place the first element. Four custom-built square pontoons on the element were used with control and survey towers in customary fashion. |
| FOUNDATION METHOD: | Because of high water velocities, siltation was a serious problem. Skirts around the bottom of the elements were tried, but silt penetrated beneath them. A special patented sand jetting system, devised by Christiani & Nielsen, removed the silt and replaced it with sand. |
| DREDGING METHOD: | River sediments, mainly loamy sand, were removed by cutterhead suction dredges. Stiff clay was removed by a chain bucket dredge which could dredge to 30 m. Because of current velocities at mid-stream, the dredge had to excavate the upstream slope and the downstream slope alternately. In addition, the trench was dredged wide enough to accommodate the elements moored in the direction of the current (floating in the event of an emergency) |
| VENTILATION TYPE: | Longitudinal system with ventilators at entrance only |
| COVER AND TYPE: | No cover over elements, other than 10 cm concrete |
| CONCRETE CUBE STRESS: 450 kg/cm$^2$ | POST-TENSIONING: Partial: 235 ton transverse tendons at 0.5 m, roof and bottom |
| ADDITIONAL INFORMATION: | OWNER: Intercommunale E3<br>DESIGNER/CONTRACTORS: Enterprises Ackermans & van Haaren, Compagnie d'Entreprises C.F.E., Compagnie International des Pieux Armes Frankignol, Société Belge des Betons and Christiani & Nielsen A/S |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Heinenoord Tunnel; Barendrecht, The Netherlands; 1969** | | | **T.34 - Heinenoord**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Vehicular tunnel | | LANES/TRACKS:<br>Two tubes; three lanes each | |
| NO OF ELEMENTS: 5 | LENGTH:<br>4 - 115 m<br>1 - (centre) 111 m | HEIGHT: 8.8 m | WIDTH: 30.7 m |
| TOTAL IMMERSED LENGTH: 574 m | | DEPTH AT BOTTOM OF STRUCTURE: 28 m | |
| UNUSUAL FEATURES: | The third lane in each direction is provided for slow-moving traffic such as cyclists, tractors, and mopeds. Elevators at both ends of the tunnel permit cyclists and pedestrians to avoid the long, steep grades. Placement of elements used a three-point bearing method, two outboard foundation blocks, and a single point at the tunnel in place. | | |
| FABRICATION METHOD:<br>All five elements were fabricated in one cycle in a casting basin | | JOINT TYPE:<br>Gina joint | |
| WATERPROOFING METHOD: | Bottom covered with 6 mm steel plate. Walls and roof waterproofed with three layers of asphalt-impregnated membrane. Roof protected with 15 cm of reinforced concrete. | | |
| PLACEMENT METHOD: | Two pontoons were arranged transversely over the element at each end with control and survey towers in conventional fashion. Elements were placed on plates; vertical jacks were used for vertical alignment adjustment. Three-point temporary foundation with pivot arrangement to allow good register between the tunnel and the element being placed. | | |
| FOUNDATION METHOD: | Jetted sand used for foundation | | |
| VENTILATION TYPE: | Longitudinal, using jet fans | | |
| ADDITIONAL INFORMATION: | OWNER: Ministry of Public Works, Rijkswaterstaad<br>DESIGNER: Locks and Weirs Division of Rijkswaterstaad<br>CONTRACTOR: Nestum II Group, a joint venture consisting of:<br>Christiani & Nielsen N.V., N.V. Amsterdamsche Ballast Mij (now member of Ballast Nedam Group), Van Hattum & Blankevoort (now member of Stevin Group), Hollandsche Beton Mij N.V. (member of Hollandsche Beton Group), N.V. Nederlandsche Aanneming Mij (now member of Ballast Nedam Group), Bato Jansen N.V. (now member of Royal Bos Kalis Westminster Group | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.35 - Limfjord |
|---|---|
| Limfjord Tunnel; Aalborg, Denmark; 1969 |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Reinforced concrete box elements; Vehicular tunnel | Two tubes; three lanes each |

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 5 | LENGTH: 102 m | HEIGHT: 8.54 m | WIDTH: 27.4 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 510 m | DEPTH AT BOTTOM OF STRUCTURE: 20.8 m |

UNUSUAL FEATURES:
Surcharged sandfill replacing existing soil was used to support immersed tunnel. Tension piles were used to support and hold down open approach and cut-and-cover sections of tunnel

FABRICATION METHOD:
Cast in a casting basin excavated for the purpose, about 10 km from the tunnel site. Elements were cast in 12.8 m long sections - first the bottom; then the walls; and, last, the roof. The sections were separated by a 1.8 m gap into which rebars protruded. These gaps were filled after concrete shrinkage had taken place

OUTFITTING:
Elements were outfitted at a pier after floating with temporary steel foundation blocks at the free end, two alignment towers and two sets of sinking rigs, each consisting of two 150 ton pontoons connected by steel girders

JOINT TYPE:
Gina type. Special gasketted end closure plates allowed connection to submerged face of cast-in-place northern tunnel. Immersed tunnel is monolithic, with a contraction joint between it and the cast-in-place northern tunnel and a combined expansion contraction joint between it and the north portal building. The joints are made watertight with rubber gaskets.

WATERPROOFING METHOD:
The elements are waterproofed with a 2 mm butyl membrane, protected on the bottom by a 9 cm layer of reinforced concrete and on the top by a 20 cm layer of reinforced concrete, and glued to the bottom protection and to the walls and roof of the tunnel by a PVC cement slurry.

PLACEMENT METHOD:
Four straddling placement barges were used. Horizontal alignment was accomplished by lines connected at the top of the element

FOUNDATION METHOD:
Sandjetting

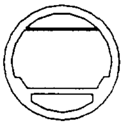
DREDGING METHOD:
Dredging was taken down to -31 m and backfilled to grade with clean sand because of soft mud layers in the river bed. The sand was surcharged with additional sand to a load equal to the tunnel weight to cause early settlement to take place. The sand was removed shortly before placing the elements.

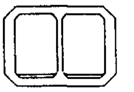
VENTILATION TYPE:
Longitudinal; jet fans

COVER AND TYPE:
No cover provided. 20 cm of reinforced concrete is the only protection to the waterproofing layer.

ADDITIONAL INFORMATION:
The tunnel is designed to be unmanned, with automatic ventilation and lighting controls, and automatic and remote control used for traffic control. Aluminium sunscreens are used at the approaches. An acoustic suspended ceiling is provided.

OWNER: Ministry of Public Works
DESIGNER: Christiani & Nielsen A/S
CONTRACTOR: The tunnel elements by NYBYG; approaches by Monberg & Thorsen A/S; Christiani & Nielsen A/S carried out the installation of the elements, including sandjetting.

| | |
|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Parana (Hernandias) Tunnel; Between Santa Fe and Parana, Argentina; 1969** | **T.36 - Parana**<br> |
| TUNNEL TYPE AND USE:<br>Circular reinforced concrete tube; Vehicular | LANES/TRACKS:<br>One tube; two lanes |
| NO OF ELEMENTS: 37 | LENGTH: 36 - 65.4 m<br>(one 10.75 m element was used on the Parana side) |
| HEIGHT: 10.8 m | WIDTH: 10.8 m |
| TOTAL IMMERSED LENGTH: 2,367 m | DEPTH AT BOTTOM OF STRUCTURE: 32 m |
| UNUSUAL FEATURES: | Light elements, filled with water to induce final settlement, were held down with six outrigger pockets each. Backfill was vibrocompacted around the element. A jack-up barge was used for element and locking fill placement. Partial ballasting was accomplished by filling between light end bulkheads and interior bulkheads (the latter capable of taking submerged water pressure), spaced at 13-m intervals. The exterior bulkheads could be detached and floated to the dock for reuse. |
| ENVIRONMENTAL CONDITIONS: | Design current: 1.35 m/s |
| FABRICATION METHOD:<br>A casting basin, designed for nine cycles, was excavated. Four elements were produced on a three-month cycle. The basin was 156 m long, 46 m wide, and 13 m high. The dock was closed by a 15 m high, 23 m dia floating cylindrical caisson acting against two fixed cylindrical tanks with 50 cm wall thickness. | JOINT TYPE:<br>A tremie concrete joint using inflatable gaskets was used for all elements. A bottom slab was used to form the tremie. Concrete hoods and collars were used. |
| WATERPROOFING METHOD: | 4 mm preformed three-layer glass-fibre reinforced polyester resin waterproofing all around the cylinder. No protection was used. |
| PLACEMENT METHOD: | Model tests were made. The element was first brought to the site parallel to the current to the centerline of the tunnel, using six 465-HP pusher rigs mounted on flexifloats. Two other similar elements were used to guide the element parallel to the current. At the jack-up rig, the element was turned transverse to the current using winches mounted on pontoons. A trolley running under the platform of the jack-up rig moved the element to its final position. It was then handled by four vertical and horizontal winches of the jack-up rig. Two ballast chambers under the roadway on both sides of the interior bulkheads were filled with water to produce 150 tons of negative buoyancy. |
| FOUNDATION METHOD: | Sand fill was compacted around each element, using deep compactors. |
| DREDGING METHOD: | A cutterhead suction dredge was used to dredge the trench and place backfill |
| VENTILATION TYPE: | Fully transverse. |
| COVER AND TYPE: | 4.0 m of compacted sand fill |
| ADDITIONAL INFORMATION: | OWNER: Argentine government<br>DESIGNER/CONTRACTORS: Consortium of Hochtief AG, Vianini SpA and Sailav SA |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.37 - Dojima |
|---|---|
| **Dojima River Tunnel (Osaka Subway Tunnel); Osaka, Japan; 1969** |  |

| | |
|---|---|
| TUNNEL TYPE AND USE:<br>Reinforced concrete box section; Railway | LANES/TRACKS:<br>Two tubes; one track each |

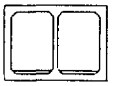
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 2 | LENGTH: 36 / 34.5 m | HEIGHT: 8.38 m including a layer of concrete protection 0.50 m thick on the top. | WIDTH: 10.43 m - 11.04 m |

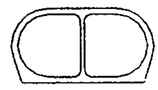
| | |
|---|---|
| TOTAL IMMERSED LENGTH: 70.5 m | DEPTH AT BOTTOM OF STRUCTURE: 14.3 m |

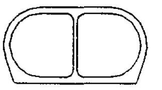
| | |
|---|---|
| UNUSUAL FEATURES: | Constructed in very tight quarters. Tunnel crossing is askew the Dojima River. Two elements were immersed between caissons at both banks. Immersed elements were fabricated inside the excavation for cut-and-cover approach tunnels. |
| ENVIRONMENTAL CONDITIONS: | Tidal river. |

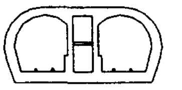
| | |
|---|---|
| FABRICATION METHOD:<br>Reinforced concrete elements were fabricated one by one in a coffered dock provided near two Nakanoshima caissons, which later became part of the tunnel. The elements were enclosed in 6 mm thick steel plate. These were fabricated in six sections and welded together inside the cofferdam. | JOINT TYPE:<br>Single cantilever gasket in combination with heavy Omega gasket. Space between gaskets was filled with tremie concrete. |

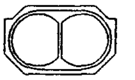
| | |
|---|---|
| WATERPROOFING METHOD: | 6 mm steel shell all around element. |
| PLACEMENT METHOD: | Catamaran barges. Water ballast was used for each element. |
| FOUNDATION METHOD: | Screed rails were installed at grade, attached to steel piles. Spreader box was fed by four hopper pipes. Spreader system was supported on floats and guided by lines from placement barges. Crushed stone 20 mm to 4 mm in size was placed to form a foundation course, approximately 0.60 m thick, which was grouted after the elements were in place. |
| DREDGING METHOD: | Grab bucket |
| VENTILATION TYPE: | Piston effect of trains |
| COVER AND TYPE: | 3.0 m of backfill. |

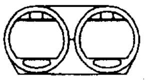
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Dohtonbori River Tunnel; Osaka, Japan; 1969** | | | **T.38 - Dohtonbori**<br> |
| TUNNEL TYPE AND USE:<br>Single-shell steel box; Railway | | LANES/TRACKS:<br>Two tubes; one track each | |
| NO OF ELEMENTS: 1 | LENGTH: 24.9 m | HEIGHT: 6.96 m | WIDTH: 9.65 m |
| TOTAL IMMERSED LENGTH: 25 m | | DEPTH AT BOTTOM OF STRUCTURE: 10 m | |
| FABRICATION METHOD:<br>A temporary working platform was set up at the site and the steel shell element was fabricated on this platform. | | JOINT TYPE:<br>One end is a rigid joint; the other is an expansion joint. | |
| WATERPROOFING METHOD: | Continuous steel shell treated with a cathodic protection system. | | |
| PLACEMENT METHOD: | A gantry crane was used for placement. | | |
| FOUNDATION METHOD: | The steel shell element was supported at both ends by caissons in the manner of a submerged bridge. | | |
| VENTILATION TYPE: | Piston effect of trains | | |

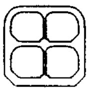
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Keiyo Line Haneda Tunnel (under Tama River); Tokyo, Japan; 1970** | | | **T.39 - Haneda (Tama)**<br> |
| TUNNEL TYPE AND USE:<br>Single-shell steel binocular section; Railway | | LANES/TRACKS:<br>Two tubes; one track each | |
| NO OF ELEMENTS: 6 | LENGTH: 80 m | HEIGHT: 7.95 m (Typ.) | WIDTH: 13.0 m (Typ.) |
| TOTAL IMMERSED LENGTH: 480 m | | DEPTH AT BOTTOM OF STRUCTURE:<br>17 m below datum | |
| UNUSUAL FEATURES: | Binocular shaped steel-shell section for railway tunnels. Immersed elements tie into pneumatic caissons at both ends. Three pick-up points were used for placing. Ventilation tower was constructed by pneumatic caisson method. Last element was specially designed to transition to bored tunnels. | | |
| ENVIRONMENTAL CONDITIONS: | Height restriction (10 m. From sea level) due to adjacent Haneda International Airport. | | |
| FABRICATION METHOD:<br>Steel shell fabricated at shipyard | OUTFITTING:<br>At jetty in back of trench excavation. | JOINT TYPE:<br>Double gasket and steel closure plate. | |
| WATERPROOFING METHOD: | Continuous steel shell provided with cathodic protection | | |
| PLACEMENT METHOD: | Placed by catamaran barges. | | |
| FOUNDATION METHOD: | 70 cm of 20 to 40 mm crushed stone was placed and screeded. Five telescopic gravel pipes equipped with hoppers fed material to screed box. Spreader ran on rails installed underwater, supported on piles. | | |
| DREDGING METHOD: | Cutterhead section dredge. | | |
| VENTILATION TYPE: | Piston effect of trains | | |
| COVER AND TYPE: | 3.0 m of protection using riprapped slopes of crushed stone and sand backfill. | | |
| ADDITIONAL INFORMATION: | Special pressure tests were made on the double gasket.<br><br>OWNER: Japan Railway Constructon Public Corporation<br>CONTRACTOR: Kajima Corporation and Ohbayashi Corporation | | |

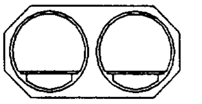
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Keiyo Line Haneda Tunnel (under Keihin Channel); Tokyo, Japan; 1970** | | | **T.40 - Haneda (Keihin)**<br> |
| TUNNEL TYPE AND USE:<br>Single-shell steel binocular section; Railway | | LANES/TRACKS:<br>Two tubes; one track each | |
| NO OF ELEMENTS: 6 | LENGTH: 80.00 m | HEIGHT: 7.97 m | WIDTH: 13.00 m |
| TOTAL IMMERSED LENGTH: 480.00 m | | DEPTH AT BOTTOM OF STRUCTURE: 17.70 m | |
| UNUSUAL FEATURES: | Binocular shaped steel shell section for railway tunnels. Immersed tunnel elements tie into pneumatic caissons at both ends of tunnel. | | |
| ENVIRONMENTAL CONDITIONS: | The height of crane booms was limited by the proximity to Haneda International Airport. | | |
| FABRICATION METHOD:<br>Steel shell fabricated at shipyard. | OUTFITTING:<br>At jetty near site. | JOINT TYPE:<br>Double rubber gaskets. Joint connecting sealing area is provided outside of main tunnel structural section, permitting rigid connection. | |
| WATERPROOFING METHOD: | Continuous steel shell provided with cathodic protection system. | | |
| PLACEMENT METHOD: | Placed by catamaran barges. | | |
| FOUNDATION METHOD: | 70 cm of 20 mm - 40 mm crushed stone was placed and screeded using screed box and mechanical screeding device. Five telescopic gravel pipes equipped with hoppers fed material to the screed box. This spreader ran on rails installed under water and supported on piles. | | |
| DREDGING METHOD: | Cutterhead suction dredge. | | |
| VENTILATION TYPE: | Piston effect of trains | | |
| COVER AND TYPE: | 3 m of sand backfill. | | |
| ADDITIONAL INFORMATION: | OWNER: Japan Railway Construction Corporation<br>CONTRACTOR: Kumagai Gami, Co., Ltd. | | |

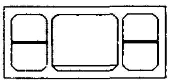
<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Bay Area Rapid Transit Tunnel; San Francisco, California, U.S.A.; 1970</td><td>T.41 - BART<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Single-shell steel binocular section; Railway</td><td colspan="2">LANES/TRACKS:<br>Two tubes; one track each</td></tr>
<tr><td>NO OF ELEMENTS: 58</td><td>LENGTH: 83.2 to 111.6 m</td><td>HEIGHT: 6.5 m</td><td>WIDTH: 14.6 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 5,825 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 40.5 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Longest immersed tube tunnel. Triaxial earthquake joints. Ventilation building constructed as caisson. Connections to shield-driven soft ground tunnels.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Constructed across the San Francisco Bay</td></tr>
<tr><td>FABRICATION METHOD:<br>On shipways and end-launched.</td><td>OUTFITTING:<br>At dockside near site.</td><td colspan="2">JOINT TYPE:<br>Double rubber gasketted joints with steel closure plates. Special triaxial earthquake joints.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Continuous steel shell; cathodic protection system.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Catamaran barges straddling elements</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Screeded foundation. Sand fed into screed box.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">Cutterhead suction (shallow); 10 m$^3$ clamshell (deep)</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Piston effect of trains; exhaust system for smoke during fire emergency using central duct.</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">0.6 m fill plus 1.6 m stone blanket.</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: Bay Area Rapid Transit System.<br>DESIGNER: Joint Venture: Parsons-Brinckerhoff-Tudor-Bechtel<br>CONTRACTOR: Trans-Bay Constructors and joint venture of Peter Kiewit & Sons Co., Raymond International, Tidewater Construction Col, and Healy Tibbets Construction Co.</td></tr>
</table>

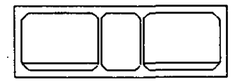
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Charles River Tunnel; Boston, Massachusetts, U.S.A.; 1971** | | | **T.42 - Charles River**<br> |
| TUNNEL TYPE AND USE:<br>Double-shell steel binocular section; Railway | | LANES/TRACKS:<br>Two tubes; one track each | |
| NO OF ELEMENTS: 2 | LENGTH: 73 m | HEIGHT: 6.86 m | WIDTH: 11.4 m |
| TOTAL IMMERSED LENGTH: 146 m | | DEPTH AT BOTTOM OF STRUCTURE:<br>12.3 m below MLW | |
| UNUSUAL FEATURES: | Selected as option to cofferdam method of crossing river. Transition sections in 29 m dia compression ring cofferdam shafts. | | |
| FABRICATION METHOD:<br>Shipyard. Side launched. | | OUTFITTING:<br>At dockside near tunnel site | JOINT TYPE:<br>Tremie joint |
| WATERPROOFING METHOD: | Continuous steel shell. | | |
| PLACEMENT METHOD: | Lowered from pile-supported structures. | | |
| FOUNDATION METHOD: | Screeded foundation course. | | |
| VENTILATION TYPE: | Piston effect of trains | | |
| ADDITIONAL INFORMATION: | OWNER: Massachusetts Bay Rapid Transit Authority.<br>DESIGNER: Praeger Kavanagh and Waterbury<br>CONTRACTOR: Steel shells by Wiley Manufacturing Co. | | |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Cross Harbour Tunnel; Hong Kong; 1972</td><td>T.43 - Cross Harbour<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Single-shell steel binocular section; Vehicular</td><td colspan="2">LANES/TRACKS:<br>Two tubes; two lanes each way</td></tr>
<tr><td>NO OF ELEMENTS: 15</td><td>LENGTH: 114 m</td><td>HEIGHT: 11.0 m</td><td>WIDTH: 22.16 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 1,600 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 28 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Ballast concrete in midsection between tubes. No side ballast pockets. Steel shell protected with concrete covering. Fast-track design in New York; rolling plate in England and fabrication of elements in Hong Kong. Ventilation building was held down using 100 ton rock anchors to develop a 1.25 safety factor against buoyancy.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Potential for typhoon conditions dictated heavy cover protection. Possibility of out-of-control ships dragging anchor over tunnel or even sinking. Provision made for 1000 PSF allowed for this loading of sunken ship.</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>Shipyard in Hong Kong. Plates curved by rolls were fabricated into cans measuring 10.36 m dia by 3.5 m long. Five cans were placed on a manipulator to allow them to be welded together automatically. The cans were, in turn, welded together to form each element. Controlled side-launch was used.</td><td>OUTFITTING:<br>At dock near the tunnel site</td><td>JOINT TYPE:<br>Tremie concrete joints</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Continuous steel shell protected with concrete coating.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Catamaran straddling barges.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Screeded bedding.</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Semi-transverse exhaust at portals.</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">1.0 m stone cover over 1.3 m crushed stone filter layer.</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: Hong Kong Cross-Harbour Tunnel Co<br>CONSULTANTS TO OWNER: Scott Wilson Kirkpatrick & Partners and Freeman Fox and Partners<br>DESIGNER: Parsons Brinckerhoff Quade & Douglas, Inc for contractor<br>CONTRACTOR: Consortium headed by Costain International Ltd; other members were Raymond International, Inc and Paul Y Construction Co Ltd</td></tr>
</table>

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**63rd Street Tunnel; New York City, New York, U.S.A.; 1973** | | | **T.44 - 63rd Street**<br> |
| TUNNEL TYPE AND USE:<br>Single-shell steel elements; Railway and subway | | LANES/TRACKS:<br>Four tubes; one track in each. Two NYC transit tracks over two Long Island Railroad tracks. | |
| NO OF ELEMENTS: 4 | LENGTH: 114.3 m | HEIGHT: 11.2 m | WIDTH: 11.7 m |
| TOTAL IMMERSED LENGTH:<br>Two tunnels, each 229 m long | | DEPTH AT BOTTOM OF STRUCTURE: 30 m | |
| UNUSUAL FEATURES: | Only two-over-two tunnel ever constructed. Actually two separate tunnels: one section between New York City and Welfare Island, and the other between Welfare Island and Brooklyn. The ends of each tunnel section were tremied into slots blasted into the rock shores. Once in place, the four ends of the two tunnels were accessed by mining to them. Tunnel was constructed in very strong current conditions. | | |
| ENVIRONMENTAL CONDITIONS: | Very swift current of 2.7 m/s (5.2 knots) | | |
| FABRICATION METHOD:<br>RIverside shipyard at Port Deposit, Maryland; uncontrolled side-launch. | | OUTFITTING:<br>At dockside in Norfolk, Virginia, and towed to New York with full draft. | JOINT TYPE:<br>Tremie concrete joints |
| WATERPROOFING METHOD: | Continuous steel shell; cathodic protection | | |
| PLACEMENT METHOD: | Placed from straddling catamaran barges. | | |
| FOUNDATION METHOD: | Screeded foundation consisting of large (15 cm) stone was used due to high current velocities in the East River. | | |
| DREDGING METHOD: | Barge-mounted excavators | | |
| VENTILATION TYPE: | Train piston action. | | |
| COVER AND TYPE: | 1.3 m riprap over 9 m of crushed stone | | |
| ADDITIONAL INFORMATION: | OWNER: Metropolitan Transportation Authority and the New York City Transit Authority<br>DESIGNER: Parsons Brinckerhoff Quade & Douglas Inc<br>CONTRACTOR: Joint Venture: Kiewit-Slattery-Morrison Knudsen | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Interstate Route 10 (Mobile) Tunnel; Mobile, Alabama, U.S.A.; 1973** | | | **T.45 - Mobile**<br> |
| TUNNEL TYPE AND USE:<br>Double steel shell binocular elements; Vehicular | | LANES/TRACKS:<br>Two tubes; two lanes each | |
| NO OF ELEMENTS: 7 | LENGTH: 106 m | HEIGHT: 12.2 m | WIDTH: 24.5 m |
| TOTAL IMMERSED LENGTH: 747 m | | DEPTH AT BOTTOM OF STRUCTURE: 30 m | |
| UNUSUAL FEATURES: | Portal ventilation buildings equipped with steel joint transition structures to which elements are attached. First project in U.S.A. to use modern sandjetting method for foundation. | | |
| FABRICATION METHOD:<br>Fabrication yard set up on site. Uncontrolled side launch. | | OUTFITTING:<br>At pier near site. | JOINT TYPE:<br>Tremie concrete joints |
| WATERPROOFING METHOD: | Continuous steel shell plate. | | |
| PLACEMENT METHOD: | Four barges straddling the element. Element placed on temporary footings and adjusted to line and grade. | | |
| FOUNDATION METHOD: | Jetted sand foundation. | | |
| VENTILATION TYPE: | Semi-transverse ventilation. Air supplied through lower air duct travels longitudinally to portal ventilation buildings. | | |
| ADDITIONAL INFORMATION: | OWNER: Alabama Highway Department<br>DESIGNER: Palmer and Baker Engineers Inc, Mobile<br>CONTRACTOR: Mobile Tunnel Contractors (joint venture):<br>Winston Brothers Co, Minneapolis<br>Atlas Construction Co Ltd, Montreal<br>Foundation Co of Canada Ltd, Toronto | | |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Kinuura Harbour Tunnel; Aichi near Nagoya, Japan; 1973</td><td colspan="2">T.46 - Kinuura Harbour<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Single shell steel box elements; Vehicular</td><td colspan="3">LANES/TRACKS:<br>One tube with two lanes</td></tr>
<tr><td>NO OF ELEMENTS: 6</td><td>LENGTH: 80 m</td><td colspan="2">HEIGHT: 7.10 m</td><td>WIDTH: 15.6 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 480 m</td><td colspan="3">DEPTH AT BOTTOM OF STRUCTURE: 21.7 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="4">Ventilation structures were constructed by pneumatic caisson method. Unusually wide box section for steel shell construction. Structural requirements were mainly determined by earthquake design loads.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="4">Severe earthquake loading requirements.</td></tr>
<tr><td>FABRICATION METHOD:<br>Steel shell was constructed in a shipyard.</td><td colspan="3">OUTFITTING:<br>The two survey alignment towers, and various equipment needed for placing the elements, were installed at the outfitting pier.</td><td>JOINT TYPE:<br>Rigid joint with single rubber gasket.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="4">Continuous steel shell around reinforced concrete box structure.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="4">Element was supported from catamaran barges and ballasted with gravel loaded on the element.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="4">Screeded gravel bed and cement mortar grout into vinyl sacks.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="4">Suction dredge was used to remove soft mud from trench bottom; grab dredge was used for sand.</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="4">Semi-transverse ventilation system.</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="4">1.0 m of riprap was used as protection against ships' anchors.</td></tr>
<tr><td colspan="2">BOUYANCY SF: 1.10 for earthquake;<br>1.20 for normal conditions</td><td colspan="3">CONCRETE WORKING STRESS: 80 kgf/cm²</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="4">Primary external forces due to earthquake.</td></tr>
</table>

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.47 - Ohgishima |
|---|---|
| **Ohgishima Tunnel; Kanagawa, Japan; 1974** |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Single steel shell; Vehicular and utility<br>Steel box elements | Three tubes; two tubes each with two lanes, one tube for conveyor belt. |

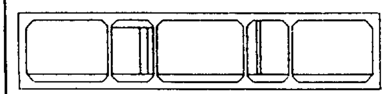
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 6 | LENGTH: 110 m | HEIGHT: 7.05 m | WIDTH: 21.6 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 660.4 m | DEPTH AT BOTTOM OF STRUCTURE: 21 m |

| | |
|---|---|
| UNUSUAL FEATURES: | Jetted sand foundation mixed with aluminous cement clinker binder to help achieve compaction and preclude liquefaction. Steel shell used to provide flexibility for better earthquake resistance. Final joint between the last element and cut and cover tunnel was sealed with gasketted steel panels. |
| ENVIRONMENTAL CONDITIONS: | 0.4 m/s current |

| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| Steel shell assembled in shallow drydock 3.3 km from tunnel site. Fully closed box section was constructed including bulkheads and interior reinforcing steel. | Outfitted at a wharf 1.6 km from the tunnel site; concrete placed from barges | Gina-type joints with steel Omega; steel rods coupled across joints; steel panel joints at cut-and-cover structures. |

| | |
|---|---|
| WATERPROOFING METHOD: | Continuous steel shell |
| PLACEMENT METHOD: | Transverse pontoons and two alignment/control towers with interior access shaft. Joint closure and alignment jacks actuated from within the element. |
| FOUNDATION METHOD: | Jetted sand foundation treated to prevent liquefaction (see above) |
| DREDGING METHOD: | Grab bucket dredger |
| VENTILATION TYPE: | Longitudinal ventilation |
| COVER AND TYPE: | Sand backfill and rubble stone protection |
| ADDITIONAL INFORMATION: | OWNER: NKK Corporation<br>DESIGNER: Christiani & Nielsen, Copenhagen for sinking and founding<br>CONTRACTOR: Taisei Corporation |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.48 - Elbe |
|---|---|
| Elbe Tunnel; Hamburg, Germany; 1975 |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Reinforced concrete box section; Vehicular | Three tubes; two lanes plus lane for slow traffic each |

| NO OF ELEMENTS: 8 | LENGTH: 132 m | HEIGHT: 8.4 m | WIDTH: 41.7 m |
|---|---|---|---|

| TOTAL IMMERSED LENGTH: 1,056 m | DEPTH AT BOTTOM OF STRUCTURE: 29 m |
|---|---|

| | |
|---|---|
| UNUSUAL FEATURES: | Very wide tunnel, with six lanes providing reverse traffic flow in middle two lanes. Prestressed concrete design was investigated for both transverse and longitudinal loads; conclusion was that it was not economical. Joints were provided at 27 m intervals for both temperature and settlement effects. Initially it was decided that these intermediate joints were unnecessary; however, because of uncertainty in predicting settlements, they were eventually included. Special exterior Omega steel plates were used at these joints. Joint was held together by prestress bars until tunnel element was in place on jetted foundation, bars were then cut to allow flexibility. The river channel was switched from one side of the river to the other during construction |
| ENVIRONMENTAL CONDITIONS: | River currents of 1.3 m/s (flood) |

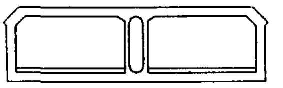
| FABRICATION METHOD: | JOINT TYPE: |
|---|---|
| All eight elements were constructed at the same time in a casting basin. 190,000 m³ were first removed by dredging. Then, after closing the end with a 250 x 11 m cofferdam, 380,000 m³ were excavated in the dry. | Gina gasket joint with rubber Omega interior gasket. Joints allows for longitudinal expansion and contraction. |

| | |
|---|---|
| WATERPROOFING METHOD: | Steel membrane on bottom and side. Bituminous membrane on top protection with 15 cm of concrete. Waterstops and steel Omega (re-entrant) joints at intermediate joints (corners formed, not mitered) |
| PLACEMENT METHOD: | Deck-mounted square pontoons with control and alignment towers. Each tunnel element had 12 ballast tanks with a total capacity of about 5,000 m³. Buoyancy depended almost entirely on the weight of the concrete. The density, as well as the concrete dimensions, were continuously controlled and the findings treated statistically, in order to maintain the total weight within 0.5% of theoretical. Because of the river currents, piled anchorages to pull against were established in the river bottom. Between seven and nine 1,200-Hp tugs were used to tow and position elements. |

| | |
|---|---|
| FOUNDATION METHOD: Sandjetting. Element No 8 ran into a problem with excess mud deposits. This material was removed before placing the sand. | DREDGING METHOD: The trench was 45 m wide, with side slopes of 1:3. A cutterhead suction dredge was used to -15 m; thereafter, a bucket dredge was used, down to -30 m. |

| | |
|---|---|
| VENTILATION TYPE: | Fully transverse |

ADDITIONAL INFORMATION:
OWNER: Freie und Hansestadt Hamburg
DESIGNER: Joint Venture "E3 Elbtunnel", see below, except for submerged tunnel: Christiani & Nielsen, Copenhagen
CONTRACTOR: Arbeitsgemeinshaft "E3 Elbtunnel" consisting of Dyckerhoff & Widmann A.G., Hochtief A.G., Ways & Freytag A.G., Christiani & Nielsen Ingenieurbau A.G. (Hamburg)

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Vlake Tunnel; Zeeland, The Netherlands; 1975** | | | **T.49 - Vlake**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box section; Vehicular | | LANES/TRACKS:<br>Two tubes; three lanes each (two active, one breakdown) | |
| NO OF ELEMENTS: 2 | LENGTH: 125 m | HEIGHT: 8.02 m | WIDTH: 29.80 m |
| TOTAL IMMERSED LENGTH: 250 m | | DEPTH AT BOTTOM OF STRUCTURE: 17 m | |
| UNUSUAL FEATURES: | Used cooling water in wall pours to reduce shrinkage cracking so that exterior water-proofing membrane could be eliminated. First use of the sand-flow method, whereby sand/water slurry is pumped under the element from internal ports. | | |
| FABRICATION METHOD:<br>Casting basin next to tunnel site, later to become part of projected canal widening. | | | |
| WATERPROOFING METHOD: | Shrinkage control and division of elements into shorter lengths of 21 m, post-tensioned together. 6 mm steel plate provided in bottom of element. | | |
| PLACEMENT METHOD: | Transverse pontoons with two control and alignment towers. Three-point bearing method used for alignment adjustment. | | |
| FOUNDATION METHOD: | Sand-flow method (used for the first time on this project). | | |
| DREDGING METHOD: | Cutterhead suction dredge. | | |
| VENTILATION TYPE: | Longitudinal | | |
| ADDITIONAL INFORMATION: | OWNER: Ministry of Public Works, Rijkswaterstaat<br>DESIGNER: Locks and Weirs Division of Rijkswaterstaat<br>CONTRACTOR: Kombinatie Vlake joint venture of Hollandsche Beton Maatschappij B.V., de Amsterdamse Ballast-Beton en Waterbouw N.V. | | |

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Kanonerski Tunnel; St. Petersburg, Russia; 1975**

**T.50 - Kanonerski**

| | |
|---|---|
| TUNNEL TYPE AND USE: Reinforced concrete box type; Vehicular | LANES/TRACKS: |

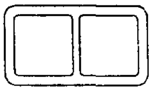
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: | LENGTH: 75 m | HEIGHT: 8.05 m | WIDTH: 13.3 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: | DEPTH AT BOTTOM OF STRUCTURE: 20 m |

| | |
|---|---|
| UNUSUAL FEATURES: | |
| ENVIRONMENTAL CONDITIONS: | |

| | | |
|---|---|---|
| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |

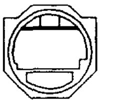
| | |
|---|---|
| WATERPROOFING METHOD: | |
| PLACEMENT METHOD: | |
| FOUNDATION METHOD: | Sandjetting |
| DREDGING METHOD: | Width of trench at bottom 18 m. Slope inclination 1:4. |
| VENTILATION TYPE: | |
| COVER AND TYPE: | |
| ADDITIONAL INFORMATION: | OWNER:<br>DESIGNER:<br>CONSULTANTS on foundation method and manufacturer of sandjetting apparatus: Christiani & Nielsen A/S |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | **T.51 - Sumida** |
|---|---|
| **Sumida River Tunnel; Tokyo, Japan; 1976** |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Railway; single steel shell box section | Two tubes; one track each |

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 3 | LENGTH: 2 x 67.0 m<br>1 x 67.5 m | HEIGHT: 7.8 m | WIDTH: 10.30 m |
| TOTAL IMMERSED LENGTH: 201.5 m | | DEPTH AT BOTTOM OF STRUCTURE: | |

| | | |
|---|---|---|
| UNUSUAL FEATURES: | **Steel shell was fabricated at the shipyard in the Nagoya Port, 360 Km southwest of tunnel site and towed open sea for 38 hours.** | |
| ENVIRONMENTAL CONDITIONS: | **The freeboard of the steel shell was decreased to pass under the five low level bridges over the Sumida River during towing.** | |
| WATERPROOFING METHOD: | **Continuous steel shell.** | |
| FABRICATION METHOD:<br>**Steel shell was fabricated at shipyard.** | OUTFITTING:<br>**At jetty near the tunnel site.** | JOINT TYPE:<br>**Single rubber gasket and closure plate.** |
| PLACEMENT METHOD: | **Placed by catamaran barge.** | |
| FOUNDATION METHOD: | **Concrete was pumped from inside the element and then mortar was grouted to fill the gap.** | |
| VENTILATION TYPE: | **Piston action of trains.** | |
| COVER AND TYPE: | **1.5 m of gravel** | |
| ADDITIONAL INFORMATION: | **OWNER: Tokyo Metropolitan Government**<br>**CONTRACTOR: Shiraishi Corporation** | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.52 - Hampton Roads No. 2 |
|---|---|
| **Hampton Roads Bridge Tunnel No 2; Hampton Roads, Virginia, U.S.A.; 1976** |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Double steel shell elements; Vehicular | One tube; two lanes |

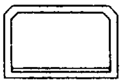
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 21 | LENGTH: 105 m | HEIGHT: 12.3 m | WIDTH: 12.0 m |

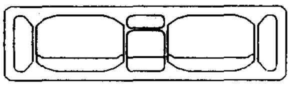
| | |
|---|---|
| TOTAL IMMERSED LENGTH: 2,229 m | DEPTH AT BOTTOM OF STRUCTURE: 37 m |

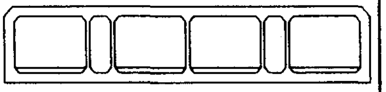
| | |
|---|---|
| UNUSUAL FEATURES: | This tunnel was constructed adjacent to the existing tunnel about 75 m away without damaging it. Ground near the south half length of the tunnels was very poor. Sand drains and surcharge were used to stabilize the expanded South Island before the elements could be placed and before the ventilation building and open approach structures could be constructed. |
| ENVIRONMENTAL CONDITIONS: | Fairly severe winds, waves and currents during storm conditions. Currents about 1 m/s (2knots) |

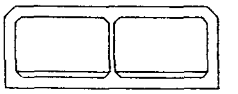
| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| At a shipyard at Port Deposit, Maryland about 300 km by water from the site. Elements were side-launched (uncontrolled). Draft slightly more than 2 m. | At a pier about 5 km from tunnel site. Up to six elements were outfitted with concrete at one time along one side of a pier (elements were moored in twos). | Double rubber gasket system with interior liner plate to joint steel shells. |

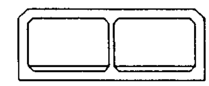
| | |
|---|---|
| WATERPROOFING METHOD: | Continuous steel shell joined by welding liner plates at immersion joints which were tested for watertightness. |
| PLACEMENT METHOD: | Straddling laybarge, consisting of two railroad car barges with support beams between them in catamaran form. |
| FOUNDATION METHOD: | Screeded foundation. Screed rig was designed to be unaffected by tidal variations by being taut moored with flotation tanks held underwater. Three elements had to be removed from trench and reset after rescreeding due to siltation of screeded foundation. |
| DREDGING METHOD: | Cutterhead suction dredge used for depths up to 15 m; then a 12 $m^3$ clamshell bucket dredge was used for deeper work. Surcharge sand on South Island was pumped to construct North Island using a booster pump to mix sand and water under a truck grizzly. |
| VENTILATION TYPE: | Fully transverse ventilation, divided between two ventilation buildings. |
| COVER AND TYPE: | 1.5 m sand cover, with some areas of riprap protection against scour |
| ADDITIONAL INFORMATION: | OWNER: Virginia Department of Highways and Transportation.<br>DESIGNER: Parsons Brinckerhoff Quade & Douglas Inc<br>CONTRACTOR: Tidewater Construction Corp., Raymond International and Peter Kiewit and Sons Inc |

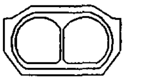
<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Paris Metro Tunnel; Paris, France; 1976</td><td>T.53 - Paris Metro<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Reinforced concrete box; Railway</td><td colspan="2">LANES/TRACKS:<br>One tube; two tracks</td></tr>
<tr><td>NO OF ELEMENTS: 4</td><td>LENGTH: 19.2 - 43.2 m</td><td>HEIGHT: 9.80 m including 2.0 m caisson edge</td><td>WIDTH: 10.40 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 129.20 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 14 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">The elements were built on an island and lowered into the water by cranes. They were then towed to location, lowered to the bottom and final lowering was done by the caisson excavation method.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3"></td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>Pretension cables, transversely and longitudinally</td><td>OUTFITTING:</td><td>JOINT TYPE:<br>Tremie concrete</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">No membrane</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Using four buoyancy tanks (rectangular cross section), one at each corner of tunnel element deck</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Caisson space filled with injected concrete</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Not applicable</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">Concrete poured under water</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3"></td></tr>
</table>

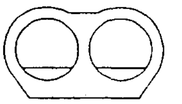
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Tokyo Port Tunnel, Wangan-Sen; Tokyo, Japan; 1976** | | | **T.54 - Tokyo Port**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box section; Vehicular | | LANES/TRACKS:<br>Two tubes; three lanes each | |
| NO OF ELEMENTS: 9 | LENGTH: 115.8 m | HEIGHT: 8.80 m | WIDTH: 37.4 m |
| TOTAL IMMERSED LENGTH: 1,035 m | | DEPTH AT BOTTOM OF STRUCTURE: 23 m | |
| UNUSUAL FEATURES: | 1. Flexible seismic joints were provided.<br>2. Steel pipe pile foundation was adopted for elements near northern ventilation shaft to limit force due to consolidation.<br>3. New foundation method was developed whereby bentonite mortar was pumped from inside the elements. Very wide, heavy elements. | | |
| ENVIRONMENTAL CONDITIONS: | Site is located in busy fairway. | | |
| FABRICATION METHOD:<br>Casting basin close to tunnel site for all nine elements. | | JOINT TYPE:<br>Flexible joint using Gina gasket type joint. A steel Omega-shaped plate was used as a second seal. | |
| WATERPROOFING METHOD: | 6 mm steel skin on sides and bottom. Concrete protected rubber membrane on top of element. | | |
| PLACEMENT METHOD: | Placed from catamaran barges. | | |
| FOUNDATION METHOD: | A bentonite mortar layer 0.5 m thick was pumped under the elements over a 0.7 m thick crushed sandstone layer. Edges of elements were sealed with sand/gravel filter to prevent mortar from escaping. Steel pile foundation was used for three elements. | | |
| DREDGING METHOD: | Cutterhead suction dredging. | | |
| VENTILATION TYPE: | Semi-transverse, using side and center ducts. | | |
| COVER AND TYPE: | 1.5 m sandstone riprap for protective cover. Sandstone rock spoil used for locking fill. | | |
| BUOYANCY SF: 1.10 | | CONCRETE WORKING STRESS: Approx 90 kgf/m² | |
| ADDITIONAL INFORMATION: | OWNER: The Tokyo Expressway Public Corp. (TE PC)<br>CONTRACTOR: Joint venture of:<br>Taisei Corporation, Kajima Corporation, Ohbayashi-Gumi and Maeda Construction Co | | |

| | |
|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br>**Drecht Tunnel; Dordrecht, The Netherlands; 1977** | **T.55 - Drecht**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Vehicular | LANES/TRACKS:<br>Four tubes; two lanes in each tube |
| NO OF ELEMENTS: 3 | LENGTH: 115 m |
| HEIGHT: 8.08 m | WIDTH: 49.04 m |
| TOTAL IMMERSED LENGTH: 347 m | DEPTH AT BOTTOM OF STRUCTURE: 15 m |
| UNUSUAL FEATURES: | Widest element ever constructed. Displacement 45,000 MT. Approach structures are of a very unusual construction, utilizing drained slabs and top-down construction, with slurry walls carried to clay. Tunnel elements were attached to end structures already in place - on one end of tunnel, using a Gina joint; and on the other, using a gasketted plate joint system. Two-lane spans were used to reduce depth of elements. Sand-flow method was used for second time. |
| ENVIRONMENTAL CONDITIONS: | River currents. |
| FABRICATION METHOD:<br>At casting yard at Barendrecht (used previously for the Heinenoord and some pipeline tunnels), 12 km from site. | JOINT TYPE:<br>Gina type joints |
| WATERPROOFING METHOD: | Membrane waterproofing was used because of the great width of the tunnel. The elements were divided into six subsections with temporary prestress, which was cut after backfilling. |
| PLACEMENT METHOD: | Model tests were used to determine the number of tugs required to handle the segment. The tests indicated 5,000 to 6,000 HP; however, 11,000 HP eventually were provided for safety. Clearances for towing were very restricted, and an elaborate electronic horizontal control method was implemented to keep elements from running aground. Element blocked 40% of river at times during towing.<br>Transverse pontoons with two control/alignment towers were used for placement. Four-point support was used at placement because of the width of the section. Vertical and horizontal jacks were used for adjustment of alignment. |
| FOUNDATION METHOD: | The sand-flow method used on the Vlake tunnel was repeated for this tunnel. |
| DREDGING METHOD: | Because of the tunnel width, rough dredging was done and the variations were compensated for with the placement of the sand foundation. |
| VENTILATION TYPE: | Longitudinal; jet fans |
| ADDITIONAL INFORMATION: | DESIGNER: Rijkswaterstaat, Directive Sluizen en Stuwen<br>CONTRACTOR: Kombinatie Tunnelbouw: Dirk Verstoep BV, Van Hattem en Blankevoort and Koninklijke Nederhorst Bouw BV |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Prinsess Margriet Tunnel; Sneek, The Netherlands; 1978** | | | **T.56 - Prinsess Margriet**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box element; Vehicular | | LANES/TRACKS:<br>Two tubes; three lanes each | |
| NO OF ELEMENTS: 1 | LENGTH: 77 m | HEIGHT: 8.0 m | WIDTH: 28.5 m |
| TOTAL IMMERSED LENGTH: 77 m | | DEPTH AT BOTTOM OF STRUCTURE: - 14.35 | |
| UNUSUAL FEATURES: | Single element designed to engage the walls of both open-approach structures. Element was constructed in one of the approaches with transverse wingwalls attached. The element was floated into position and sealed using inflatable gaskets. | | |
| FABRICATION METHOD:<br>Fabricated in open approach section. Element was cast in four articulated sections to limit shrinkage cracking. | | JOINT TYPE:<br>Inflatable gaskets at both ends of element in wingwalls and along bottom. This arrangement was used as a temporary seal so that permanent closure could be cast. | |
| WATERPROOFING METHOD: | 6 mm steel membrane on bottom. Two-ply bituminous membrane on sides and top. Top protected with layer of concrete. | | |
| PLACEMENT METHOD: | Floated into position and lowered by winching. Clearance of wingwalls only 4.5 cm. Ballast tanks were used. | | |
| FOUNDATION METHOD: | Sand Flow. | | |
| VENTILATION TYPE: | None required; natural ventilation | | |
| ADDITIONAL INFORMATION: | OWNER: Ministry of Public Works; Rijkswaterstaat<br>DESIGNER: Locks and Weirs Division of Rijkswaterstaat<br>CONTRACTOR: Aannemersskombinatie Prinses Magriettunnel (APM):<br>B.V. Aannemingsmij v/h H&P Voormolen (member of OGEM Building Division)<br>DURA Aannemingsmij BV | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Kil Tunnel; Dordrecht, The Netherlands; 1978** | | | **T.57 - Kil**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Vehicular | | LANES/TRACKS:<br>Two tubes; three lanes each. Two lanes for fast-moving traffic and one truck lane. | |
| NO OF ELEMENTS: 3 | LENGTH: 111.5 m | HEIGHT: 8.75 m | WIDTH: 31.0 m |
| TOTAL IMMERSED LENGTH: 330 m | | DEPTH AT BOTTOM OF STRUCTURE: - 19.19 m | |
| UNUSUAL FEATURES: | The third lane in each direction is provided for slow-moving traffic. Escalators at both ends of the tunnel permit cyclists and pedestrians to avoid the long, steep grades. | | |
| ENVIRONMENTAL CONDITIONS: | High water velocity to 2.1 m/sec. caused difficulties during immersion operation | | |
| FABRICATION METHOD:<br>At the Barendrecht basin alongside the elements for the Drecht Tunnel. | | JOINT TYPE:<br>Gina gasketted joints on the elements. Gasketted plates at the end joint. | |
| WATERPROOFING METHOD: | Bottom provided with 6 mm steel plate. Shrinkage control by cooling wall pours and division of elements into shorter lengths of 22.0 m. Posttensioned together during transport and immersion. | | |
| PLACEMENT METHOD: | Transverse pontoons with two control/survey towers. | | |
| FOUNDATION METHOD: | Sand-flow method, similar to Vlake and Drecht Tunnels. | | |
| VENTILATION TYPE: | Longitudinal | | |
| ADDITIONAL INFORMATION: | OWNER: Province of South - Holland<br>DESIGNER: Locks and Weirs Division of Rijkswaterstaat<br>CONTRACTOR: Kombinatie Tunnelbouw: Dirk Verstoep B.V., Van Hattem en Blankevoort and Koninklijke Nederhorst Bouw B.V. | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br>**WMATA Washington Channel Tunnel; Washington D.C., U.S.A.; 1979** | | | **T.58 - WMATA**<br> |
| TUNNEL TYPE AND USE:<br>Double-shell steel elements; Railway | | LANES/TRACKS:<br>Two tubes; one track each | |
| NO OF ELEMENTS: 3 | LENGTH: 103.6 m | HEIGHT: 6.7 m | WIDTH: 11.3 m |
| TOTAL IMMERSED LENGTH: 311 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| FABRICATION METHOD:<br>At shipyard at Port Deposit, Maryland, about 120 km from the site. Uncontrolled side launching was used. | | OUTFITTING:<br>At dockside at fabrication yard in Port Deposit. | |
| WATERPROOFING METHOD: | Continuous steel shell. | | |
| PLACEMENT METHOD: | Winches from pile-supported guide beams. | | |
| FOUNDATION METHOD: | Screeded bedding. | | |
| VENTILATION TYPE: | Piston action from trains. | | |
| ADDITIONAL INFORMATION: | OWNER: Washington Metropolitan Transit Authority (WMATA)<br>DESIGNER: Parsons Brinckerhoff Quade & Douglas Inc<br>CONTRACTOR: Joint venture of Perrini, Horn and Morrison-Knudsen | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: Hong Kong Mass Transit Tunnel; Victoria Harbour, between Kowloon and Hong Kong Island; 1979 | **T.59 - H. K. Mass Transit**  |
|---|---|

| TUNNEL TYPE AND USE: Reinforced concrete double binocular elements, longitudinally prestressed; Railway | LANES/TRACKS: Two tubes; one track each |
|---|---|

| NO OF ELEMENTS: 14 | LENGTH: 100 m | HEIGHT: 6.5 m | WIDTH: 13.1 m |
|---|---|---|---|

| TOTAL IMMERSED LENGTH: 1,400 m | DEPTH AT BOTTOM OF STRUCTURE: -24.24 m |
|---|---|

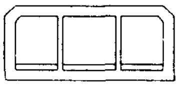
| | |
|---|---|
| UNUSUAL FEATURES: | Semi-submersible; screeding and placing of elements using same jack-up rig; jacked-up when screeding and semi-submersible during placement of element. Caisson ventilation buildings set on end elements. All of the tunnel elements were constructed with a 2800 m. radius curve. |
| ENVIRONMENTAL CONDITIONS: | Currents of 1.5 m/s (2.9 knots). Several typhoons had to be contended with during construction. Tunnel was designed for 5 m tides. |

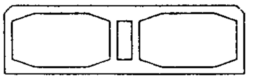
| FABRICATION METHOD: Casting basin cast in four batches. | JOINT TYPE: Gina/Omega gasketted joints. |
|---|---|

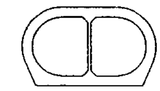
| | |
|---|---|
| WATERPROOFING METHOD: | From the bottom to the first construction joint, 6 mm steel plate treated with corrosion protection. The remainder was waterproofed with two-ply bituminous membrane. |
| PLACEMENT METHOD: | Used jack-up special marine platform. |
| FOUNDATION METHOD: | Screeded bedding, using jack-up rig. |
| DREDGING METHOD: | To -19 m, with bucket dredge; finished with grab dredge using 4 $m^2$ to 12 $m^2$ buckets. |

| VENTILATION TYPE: Piston action from trains. | BUOYANCY SF: 1.20 |
|---|---|

| | |
|---|---|
| ADDITIONAL INFORMATION: | Ventilation buildings were floating caissons set on top of the end elements. The end elements had a special rotation joint within to allow settlement after the placing of the ventilation buildings; they were grouted solid after expected settlement/time elapsed. The zone of all ends were grouted and TBM drilled into the circular ends of the immersed tube to complete the tunnel. Tremie concrete final joint with a precast soft panel lowered into the tunnel zone.<br><br>CLIENT/OWNER: Mass Transit Railway Corporation<br>DESIGNER:<br>Main Consultant: Freeman Fox and Partners (Far East)<br>Subconsultant: Per Hall Consultants Ltd<br>M&E: Kennedy & Donkin (Far East)<br>Architectural: Design Research Unit<br>CONTRACTOR & OPERATOR (22 years): Kumagai Gumi Co. Ltd. Tokyo, Japan |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Hemspoor Tunnel; Under North Sea Canal, The Netherlands; 1980</td><td>T.60 - Hemspoor<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Railway</td><td colspan="2">LANES/TRACKS:<br>Three tubes; one track each.</td></tr>
<tr><td>NO OF ELEMENTS: 7</td><td>LENGTH: 4 - 268 m<br>3 - 134 m</td><td>HEIGHT: 8.70 m</td><td>WIDTH: 21.43 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 1,475 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 26 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Three-track tunnel; middle track is used for wide goods trains. Four straight elements were constructed twice as long (268 m) as the remaining three curved elements (134 m). These long elements displaced a record 50,000 tonnes. This was possible because there is no current in the North Sea Canal and there was sufficient room to manuever. The sections of each element were temporarily tied together with plain reinforcing steel instead of prestressing rods. As in other tunnels, these ties were cut after the element was in its final condition.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Virtually no current in the canal.</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>Tunnel elements built in drydock formed by extending the existing docks used for the Coen and Ij tunnels. Tow 6 km. long.</td><td colspan="2">JOINT TYPE:<br>Gina joints. Last joint was made using gasketted plates to permit joint dewatering.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Intermediate joints at 22.35 m intervals were provided to reduce shrinkage cracking and accommodate settlements. The roof was protected with a concrete slab placed just prior to sinking the tube. This reduced the freeboard to only 5 cm.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Transverse placement barges on top of element. Survey towers were not used, but an access shaft was provided on each element. The center between immersion load lines was used for positioning. Proper register between elements was obtained with a sensor operated between the bulkheads. Levelling was done internally.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">The sand-flow method was different, in that sand slurry was pumped into the individual elements through hoses attached by divers.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">Cutterhead suction dredges, placing spoil in adjacent land areas.</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Piston action of trains.</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: Dutch Railways.<br>DESIGNER: Locks and Weirs Division of the National Public Works.<br>CONTRACTOR: Hemspoor joint venture: Royal Bos Kalis Westminster Group N.V. and Stevin Groep N.V.</td></tr>
</table>

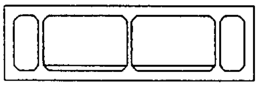
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Botlek Tunnel; Under Oude Maas River, The Netherlands; 1980** | | | **T.61 - Botlek**<br> |
| TUNNEL TYPE AND USE:<br>Prestressed concrete box elements; Vehicular | | LANES/TRACKS:<br>Two tubes; three lanes each<br>(two traffic lanes plus a breakdown lane) | |
| NO OF ELEMENTS: 5 | LENGTH: 4 - 105 m<br>1 - 87.5 m | HEIGHT: 8.8 m | WIDTH: 30.9 m |
| TOTAL IMMERSED LENGTH: 508 m | | DEPTH AT BOTTOM OF STRUCTURE: 23.3 m | |
| UNUSUAL FEATURES: | Heavy floating shear leg cranes were used for placement. | | |
| FABRICATION METHOD:<br>All five elements were cast in an existing casting basin used for the Benelux Tunnel elements, having been refurbished. The sections were left submerged for about three months. They were refloated without problems by pumping out ballast water. | | JOINT TYPE:<br>Gina joints were used, except at the end tube, where gasketted plates were employed. | |
| WATERPROOFING METHOD: | The elements were constructed of six 17.5 m sections post-tensioned together. The post-tensioning was bonded. The wall pours were cooled using water pipes to eliminate cracking due to heat of hydration. | | |
| PLACEMENT METHOD: | Elements were placed using shear leg cranes. Survey and control towers were used for alignment control and access. | | |
| FOUNDATION METHOD: | Sand-flow method. As for the Hemspoor Tunnel, sand slurry was pumped into each individual element through hoses attached by divers. | | |
| DREDGING METHOD: | Cuttersuction dredger. | | |
| VENTILATION TYPE: | Longitudinal, using jet fans. | | |
| ADDITIONAL INFORMATION: | OWNER: Ministry of Public Works; Rijkiswaterstaat<br>DESIGNER: Public Works of Rotterdam<br>CONTRACTOR: Joint venture of Voormolen and Ways & Freytag | | |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Keiyo Line Daiba Tunnel; around Tokyo Port, Japan; 1980</td><td>T.62 - Daiba<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Single-shell steel binocular; Railway</td><td colspan="2">LANES/TRACKS:<br>Two tubes; one track each</td></tr>
<tr><td>NO OF ELEMENTS: 7</td><td>LENGTH: 96.6 m</td><td>HEIGHT: 8.05 m to 8.60 m</td><td>WIDTH: 12.20 m to 17.53 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 672 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 23.9 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Tunnel was designed to be sufficiently flexible to accommodate up to 1.2 m of settlement and survive a severe earthquake. Joints are tied together with cables, but allow some flexibility. These flexible joints are provided between all of the elements and at the terminal joints. Immersed elements tie into pneumatic caissons at both ends of tunnel. To meet bored tubes, tunnel width gradually increases 5 m over elements Nos 5-7.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Site is situated in fairway of a busy port area.</td></tr>
<tr><td>FABRICATION METHOD:<br>Steel shells were fabricated at shipyard. Bulkheads and rubber gaskets were installed and the elements were towed to pier at outfitting yard.</td><td>OUTFITTING:<br>Reinforcement, and concrete was placed at outfitting jetty near tunnel site. Survey towers were installed after the element was positioned into a jack-up platform.</td><td colspan="2">JOINT TYPE:<br>Joints between elements have a rubber gasket on the outside and a small gap on the inside for flexibility. Cables through the joints tie the elements together. H-shaped steel members form a shear key embedded in the ends of the elements at each joint to prevent excessive vertical and horizontal displacement under settlement and earthquake loadings. Caissons were used for the end structures to protect existing port structures.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Continuous steel shell. Each element was protected with 200 aluminum anodes, giving an estimated life of 60 years.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Placement from a jack-up platform because of space restrictions in Port area precluded use of anchor lines. Crushed stone was used for placement ballast.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">A 70 cm thick, 12.4 m wide layer of 30 mm to 40 mm crushed stone foundation course, screeded from jack-up platform.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">Grab dredge was used for the initial cut; a cutterhead dredge was used for the final cut and soft mud removal.</td></tr>
<tr><td colspan="2">VENTILATION TYPE:<br>Piston action of trains</td><td>BUOYANCY SF:<br>1.1</td><td>COVER AND TYPE:<br>1.1 m of crushed stone</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER AND DESIGNER: Japan Railway Constructin Public Corp.<br>CONTRACTOR: Joint venture of Kajima Corp., Sato Kokyo Co. and Tekken Construction Co.</td></tr>
</table>

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Tokyo Port Dainikoro Tunnel; Tokyo, Japan; 1980**

**T.63 - Tokyo Port Dainikoro**



| | |
|---|---|
| TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Vehicular | LANES/TRACKS:<br>Two tubes; two lanes each |

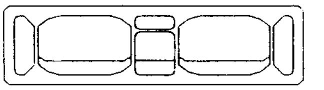
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 6 | LENGTH: 124 m | HEIGHT: 8.8 m | WIDTH: 28.4 m |

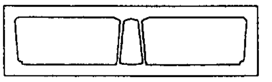
| | |
|---|---|
| TOTAL IMMERSED LENGTH: 744 m | DEPTH AT BOTTOM OF STRUCTURE: 23 m |

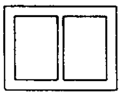
| | |
|---|---|
| UNUSUAL FEATURES: | 1. Bentonite mortar mixture was pumped for the foundation of the tunnel elements.<br>2. Flexible joints for earthquake displacements.<br>3. Moderate longitudinal prestress used to control cracking and resist earthquake stress. |

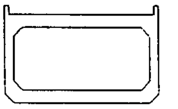
| | |
|---|---|
| FABRICATION METHOD:<br>At a casting basin near the tunnel site. All six elements were cast at once. Approximately two years were required to establish the basin and produce the elements. | JOINT TYPE:<br>Gina/Omega gasket between elements; tension ties were provided across the joints for earthquake loads. |

| | |
|---|---|
| WATERPROOFING METHOD: | 8 mm steel plate on bottom and sides; 2.5 mm membrane on top of element, protected with 15 cm of reinforced concrete. |
| PLACEMENT METHOD: | Catamaran barges. Two temporary foundations at free end and steel bracket at primary end. |
| FOUNDATION METHOD: | Bentonite mortar was pumped over stone bedding. |
| DREDGING METHOD: | Grab bucket dredger for the initial dredging; a cutterhead suction dredge for final dredging and clean-up of soft mud. |
| VENTILATION TYPE: | Semi-transverse ventilation, using side ducts with a ventilation building at each end of the tunnel. |
| COVER AND TYPE: | Fine crushed stone on both sides, with rock armor on top. |
| ADDITIONAL INFORMATION: | OWNER AND DESIGNER: Bureau of Port & Harbour, Tokyo Metropolitan Government<br>CONTRACTOR: Joint venture of Taisei Corp., Kajima Corp., Ohbayashi-Gumi Ltd, Kumagai Gumi Co Ltd, and Maeda Construction Co Ltd |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Kawasaki Tunnel; Kanagawa, near Tokyo, Japan; 1981** | | | **T.64 - Kawasaki**<br> |
| TUNNEL TYPE AND USE:<br>Single-shell steel box elements; Vehicular | | LANES/TRACKS:<br>Two tubes; two lanes each | |
| NO OF ELEMENTS: 8 | LENGTH: 100-110 m | HEIGHT: 8.8 m | WIDTH: 31.0 m |
| TOTAL IMMERSED LENGTH: 840 m | | DEPTH AT BOTTOM OF STRUCTURE: 22 m | |
| UNUSUAL FEATURES: | Large width of box section. Land tunnel support on steel pipe piles. Sectional dimensions mainly based on earthquake loadings. Earthquake observations have been carried out since 1980 on this tunnel. | | |
| ENVIRONMENTAL CONDITIONS: | Severe earthquake loading requirements. | | |
| FABRICATION METHOD:<br>Steel shells were fabricated at shipyard next to outfitting pier. | | OUTFITTING:<br>Reinforcing steel and concrete were placed at outfitting pier with element in flotation. Alignment towers and placement equipment also were installed at the outfitting pier. | JOINT TYPE:<br>Rigid joint with rubber gaskets. Flexible joint was used between terminal element and ventilation tower. |
| WATERPROOFING METHOD: | Continuous steel shell. | | |
| PLACEMENT METHOD: | Element supported by pontoons. Ballasting with water into tanks in element. | | |
| FOUNDATION METHOD: | Mortar bed pumped in place from inside elements. | | |
| DREDGING METHOD: | A grab dredge was used for the initial dredging. A dredge with a sealed grab was used to do the final dredging and removal of soft mud that drifted into the trench. | | |
| VENTILATION TYPE: | Semi-transverse ventilation system. | | |
| COVER AND TYPE: | 1.5 m of gravel was used over the tubes as a protective layer. | | |
| BUOYANCY SF: 1.1 for earthquake and<br>1.2 for normal conditions. | | CONCRETE WORKING STRESS:<br>100 $kgf/m^2$ | |
| ADDITIONAL INFORMATION: | Deformed bars of 51 mm dia were used for longitudinal reinforcement to resist axial earthquake forces. | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Rupel Tunnel; Boom, Belgium; 1982** | | | **T.65 - Rupel**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Vehicular | | LANES/TRACKS:<br>Two tubes; three lanes each<br>Service tube with emergency exit | |
| NO OF ELEMENTS: 3 | LENGTH:<br>137.9, 99.8 and 98.3 m | HEIGHT: 9.35 m | WIDTH: 35.10 m |
| TOTAL IMMERSED LENGTH: 336 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| FABRICATION METHOD:<br>Elements were cast in the open approaches to the tunnel | | OUTFITTING:<br>In casting basin | JOINT TYPE:<br>Gina/Omega joints |
| WATERPROOFING METHOD: | Steel plate on bottom of elements and bituminous membrane on the sides and top | | |
| PLACEMENT METHOD: | Deck-mounted pontoons, control and alignment towers | | |
| FOUNDATION METHOD: | Jetted-sand method | | |
| VENTILATION TYPE: | Longitudinal, using booster fans | | |
| DREDGING METHOD: | Bucket dredge | | |
| ADDITIONAL INFORMATION: | CLIENT/OWNER: Ministerie van Openbare Werken / Bestuur der Wegen<br>DESIGNER: Tijdelijke Vereniging Rupeltunnel (see below)<br>CONTRACTOR: Tijdelijke Vereniging Rupeltunnel consisting of Compagnie d'Entreprises CFE S.A., Compagnie Internationale des Pieux Armés Frankignoul, Société Belge de Betons S.A., Ackermans en Van Haaren N.V. (later renamed Dredging International N.V.), Christiani & Nielsen A/S | | |

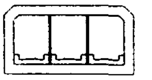
<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Metropolitan Railway Tunnel under the Main; Frankfurt am Main, Germany; 1983</td><td>T.66 - Metropolitan Rail Main<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Railway</td><td colspan="2">LANES/TRACKS:<br>Two tubes; one track each tube</td></tr>
<tr><td>NO OF ELEMENTS: 2</td><td>LENGTH:<br>1 - 61.5 m<br>1 - 62 m</td><td>HEIGHT:<br>8.55 m<br>8.55 to 10.28 m</td><td>WIDTH:<br>12.10-13.10 m<br>12.10-12.70 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 123.5 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE:<br>17 m below water level</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Varying cross sections</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Mild currents; flow of bottom sediments during periods of high water</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>Casting basin in the approaches on each side. Intermediate joints provided at 13.70 m spacing to reduce temperature and shrinkage cracking.</td><td>OUTFITTING:<br>In casting basin as part of fabrication operation</td><td>JOINT TYPE:<br>Gina/Omega joints</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Impermeable reinforced concrete.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Lowering from bridging frame over the excavation and from a pontoon bridge in the river.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Sand-flow method, using openings in bottom slab.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">Upper portion of sloped trench above 7 m was dredged using regular dredge; lower portion was cut between sheet pile walls using a large floating backhoe dredge capable of 19 m depth.</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Piston action from trains.</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">Fill plus stone blanket.</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: Deutsche Bundesbahn (German Railway)<br>OPERATOR: Deutsche Bundesbahn, Frankfurter Verkehrsverbund (FVV)<br>DESIGNER: Deutsche Bundesbahn<br>CONTRACTOR: Joint Venture: Dyckerhoff + Widmann AG, Frankfurt; Bilfinger + Berger, Frankfurt; Josef Riepl AG, Strabag AG</td></tr>
</table>

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Bastia Old Harbour Tunnel; under Old Harbour in Bastia, Corsica, France; 1983** | | | **T.67 - Bastia Old Harbour**<br> |
| TUNNEL TYPE AND USE:<br>Concrete box elements; Vehicular | | LANES/TRACKS:<br>One tube; two lanes, plus shoulders | |
| NO OF ELEMENTS: 4 | LENGTH: 62.33 m | HEIGHT: 7.58 + 0.7 m | WIDTH: 14.10 m |
| TOTAL IMMERSED LENGTH: 249.72 | | DEPTH AT BOTTOM OF STRUCTURE: 13.5 m | |
| UNUSUAL FEATURES: | The graving dock used a floating door. The final joint was made between the last element and a fixed cut-and-cover section in the space formerly used as a dock. | | |
| ENVIRONMENTAL CONDITIONS: | Tunnel crosses a harbor. Half the tunnel trench was in rock, the other in sand and gravel. | | |
| FABRICATION METHOD:<br>Tunnel elements built in approach ramps. Permanent longitudinal prestressing. | | JOINT TYPE:<br>Gina and Omega rubber seals | |
| WATERPROOFING METHOD: | On first tunnel element a thin PVC film; on remaining three tunnel elements no water-proofing. | | |
| PLACEMENT METHOD: | By means of floating cranes and four vertical cylindrical tanks mounted on tunnel roof. | | |
| FOUNDATION METHOD: | The elements were enclosed on the sides and tremie concrete was placed for the permanent foundation. | | |
| DREDGING METHOD: | Rock blasting and dredging with clamshell dredge. | | |
| VENTILATION TYPE: | Transverse ventilation through side ducts on each side of roadway. | | |
| COVER AND TYPE: | | | |
| ADDITIONAL INFORMATION: | OWNER: Direction départementale de l'équipement de Haute-Corse<br>DESIGN: GTMBTP, Marseille<br>CONTRACTORS: GTM France with Corse Traveaux y Ph. Agostini | | |

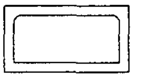
**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**S-Bahn Rhein-Main Tunnel; Hessen, Germany; 1983**

**T.68 - S-Bahn Rein-Main**



| | |
|---|---|
| TUNNEL TYPE AND USE:<br>Reinforced concrete box section; Railroad | LANES/TRACKS:<br>Two tubes |
| NO OF ELEMENTS: 2 | LENGTH:<br>61.50 m and<br>62.00 m |
| HEIGHT: | WIDTH: |
| TOTAL IMMERSED LENGTH: 123.50 m | DEPTH AT BOTTOM OF STRUCTURE: |
| UNUSUAL FEATURES: | **Tunnel roof only 4 m below existing Main river bottom.** |
| ENVIRONMENTAL CONDITIONS: | **Subsurface at tunnel site comprised recent fill, quaternary aggradation deposit, Young Tertiary marine deposits and Old Tertiary fresh and briny water deposits (mica sands).** |
| FABRICATION METHOD:<br>**Tunnel elements were constructed in docks in tunnel line, one on northern, another on southern bank.** | JOINT TYPE: |
| WATERPROOFING METHOD: | |
| PLACEMENT METHOD: | **At river end catamaran rig, at river bank fixed cross beam.** |
| FOUNDATION METHOD: | **Tunnel elements were landed on temporary foundation blocks.** |
| DREDGING METHOD: | |
| VENTILATION TYPE: | |
| COVER AND TYPE: | |
| ADDITIONAL INFORMATION: | OWNER: Deutsche Bundesbahn (German Railway) |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.69 - Coolhaven |
|---|---|
| **Coolhaven Tunnel; Rotterdam, The Netherlands; 1984** |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Reinforced concrete box elements; Metro railway | One tube; two tracks |

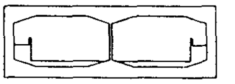
| NO OF ELEMENTS: 7 | LENGTH: | HEIGHT: 6.35 m | WIDTH: 9.64 m |
|---|---|---|---|
| | 3 - 45.59 m<br>3 - 74.98 m<br>1 - 49.00 m | | |

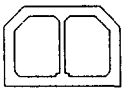
| TOTAL IMMERSED LENGTH: 411 m | DEPTH AT BOTTOM OF STRUCTURE:<br>- 11.15 m |
|---|---|

| | |
|---|---|
| UNUSUAL FEATURES: | Elements were floated into place between cross-braced sheet pile walls. They were then hooked up at both ends to carriers, which ran on rails supported on the sheet piling. The elements were then ballasted internally and drawn together with jacks on the top slab. The water pressure was then released in the joint area in the conventional way. The foundation was unique, as it utilized special piles provided with tops that could be adjusted to the underside of the tunnel element by inflation with grout. This was accomplished with a precast upper section connected to the lower section with a guiding dowel. The expandable section was enclosed within a folded nylon bag. |
| ENVIRONMENTAL CONDITIONS: | Construction through residential urban area and existing canal. |

| | |
|---|---|
| FABRICATION METHOD: Elements constructed in two cycles using open approaches as casting basin. | JOINT TYPE: Gina-type joint. |

| | |
|---|---|
| PLACEMENT METHOD: | Lowered from carriers on rails (see above). |
| FOUNDATION METHOD: | Installed on adjustable piling (see above). |
| DREDGING METHOD: | Clamshell bucket excavation. |
| VENTILATION TYPE: | Piston action from trains. |
| ADDITIONAL INFORMATION: | OWNER: Municipality of Rotterdam<br>DESIGNER: Public Works of Rotterdam<br>CONTRACTOR: Hoofdaannemer BBM, Bredase Beton-en Aanneming Maatschappij B.V. |

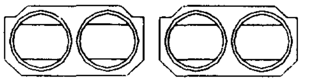
| | |
|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Kaohsiung Cross Harbour Tunnel; Kaohsiung, Taiwan; 1984** | **T.70 - Kaohsiung Harbour**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box section; Vehicular | LANES/TRACKS:<br>Two tubes; two lanes each. Each tube is also provided with 2.6 m wide motorcycle way |
| NO OF ELEMENTS: 6 \| LENGTH: 120 m | HEIGHT: 9.35 m \| WIDTH: 24.4 m |
| TOTAL IMMERSED LENGTH: 720 m | DEPTH AT BOTTOM OF STRUCTURE: 23 m |
| UNUSUAL FEATURES: | Longitudinal post-tensioning was used for the immersed elements. The roof of two elements had vertical curve. |
| ENVIRONMENTAL CONDITIONS: | Designed to take loads of a sunken ship or earthquake, including consideration of liquefaction of the silty sands beneath the tunnel, differential settlements, and frictional restraint of movement caused by temperature, shrinkage, and creep |
| FABRICATION METHOD:<br>Casting basin constructed near the site. All elements cast in a single cycle. Element cast in six sections of 20 m each | JOINT TYPE:<br>Gina/Omega type. Tension ties were provided across the joints for earthquake loads. |
| WATERPROOFING METHOD: | Two-ply bituminous membrane system was applied for the sides at top. Steel plate was used for bottom and collars. Waterproofing was protected with concrete. Element was cast in six segments of 20 m. each. |
| PLACEMENT METHOD: | Transverse barges with alignment and control towers were used. |
| FOUNDATION METHOD: | Sand-flow method, using pipes previously cast in the bottom of the tunnel elements; accessible from the outside. 5% of cement clinker was mixed with sand to prevent liquefaction of foundation during earthquake. |
| VENTILATION TYPE: | Longitudinal system |
| ADDITIONAL INFORMATION: | OWNER: Kaohsiung Harbor Bureau<br>DESIGNER/CONTRACTOR (Turnkey): Retired Serviceman Engineering Agency (RSEA) and Kumagai Gumi for constructions, Kampsax - Per Hall Ltd acted as specialist consultants |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br>**Spijkenisse Metro Tunnel; Rotterdam, The Netherlands; 1985** | | | **T.71 - Spijkenisse Metro**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Railway | | LANES/TRACKS:<br>Two tubes; one track each. | |
| NO OF ELEMENTS: 6 | LENGTH: 90 m | HEIGHT: 6.55 m | WIDTH: 10.3 m |
| TOTAL IMMERSED LENGTH: 530 m | | DEPTH AT BOTTOM OF STRUCTURE: -22.46 | |
| ENVIRONMENTAL CONDITIONS: | River currents. | | |
| FABRICATION METHOD:<br>Elements were cast in casting basin used previously for the elements for the Rotterdam Metro. Three elements were cast at a time, in two cycles. | | JOINT TYPE:<br>Gina/Omega rubber joints between element. Final joint between last two elements was made using gasketted steel plates installed by divers. Provision was made in joint to the land section for settlement. | |
| PLACEMENT METHOD: | Lowered by cranes. Survey tower on outboard end. | | |
| FOUNDATION METHOD: | Jetted-sand method was used. Silt was first removed by air-jetting for agitation by new method developed by contractor, and the action of the river current. At one end, unsuitable material was removed by over-excavation and replaced with sand, which was then vibro-compacted. | | |
| DREDGING METHOD: | Bucket dredge and cutterhead suction dredge. | | |
| VENTILATION TYPE: | Piston action from trains. | | |
| ADDITIONAL INFORMATION: | OWNER: Municipality of Rotterdam<br>DESIGNER: Gemeentewerken Rotterdam (G.W.R.)<br>CONTRACTOR: Van Hattum & Blankevoort, Netherlands | | |

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Fort McHenry Tunnel; Baltimore, Maryland, U.S.A.; 1987**

**T.72 - Fort McHenry**



TUNNEL TYPE AND USE:
Double-shell steel binocular tunnels; Vehicular

LANES/TRACKS:
Two parallel tunnels, each with two tubes and two lanes in each tube, for a total of eight traffic lanes.

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 32 (16 elements for each tunnel) | LENGTH: 104.8 m | HEIGHT: 12.7 m | WIDTH: 25.1 m |

TOTAL IMMERSED LENGTH: 1,646 m

DEPTH AT BOTTOM OF STRUCTURE: 31.7 m

| | |
|---|---|
| UNUSUAL FEATURES: | Two parallel immersed tunnels, only 3 m apart, mostly on a curve. Overall largest vehicular immersed tunnel project in the world. Elements were placed alternately from the northbound tunnel to the southbound tunnel. An 11 m deep ship wharf was removed during construction and later replaced as a float-in caisson on top of the twin tunnel. Scheduling of element placement depended on the relocation of a 122 cm critical city watermain, which crossed the alignment. This relocation involved a deep cofferdam in the middle of the river, to make the changeover. First use of remotely controlled wedges for horizontal alignment adjustment. |
| ENVIRONMENTAL CONDITIONS: | Very mild currents and tidal conditions, except for storm effect. Flow of bottom sediment was a problem affecting several elements. Three elements required rescreeding. |

| | | |
|---|---|---|
| FABRICATION METHOD: Shipyard at Port Deposit, Maryland about 65 km from tunnel site. Uncontrolled side launch. | OUTFITTING: At dockside in Baltimore, 70 km. from the site. Up to six elements were being outfitted at one time. | JOINT TYPE: Double rubber gasket. Welded, grouted liner plate. |

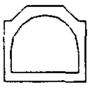
WATERPROOFING METHOD: Continuous steel shell

PLACEMENT METHOD: Catamaran barge system.

| | |
|---|---|
| FOUNDATION METHOD: | Option was provided for either jetted sand or screeded bedding. The contractor opted for the screeded method. A special taut moored rig was used, with the flotation tanks fully submerged to prevent tidal effects. |
| DREDGING METHOD: | Virtually all dredging was done by a large cutterhead suction dredge (69 cm dia pipe) pumping via a submerged pipeline to a disposal site 4 km away. A booster pump was mounted on a barge. |
| VENTILATION TYPE: | Fully transverse ventilation, shared by a ventilation building located at each end of the pair of immersed tunnels. |
| COVER AND TYPE: | 1.5 m of ordinary backfill |

ADDITIONAL INFORMATION:
Disposal site for dredged material was segregated into two areas. One area contained the highly contaminated near-bottom materials behind a sealed cofferdam and clay-lined dikes. The other area was used to contain the better, more granular materials; the latter has since been developed into a major container port facility.

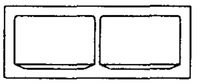
OWNER: Interstate Division Baltimore City (IDBC)
DESIGNER: Joint Venture: Sverdrup Corporation and Parsons Brinckerhoff Quade and Douglas Inc
CONTRACTOR: Joint Venture: Peter Kiewit & Sons Inc, Raymond International Inc and Tidewater Construction Corp

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Second Downtown Tunnel; Norfolk to Portsmouth, Virginia, U.S.A.; 1988** | | | **T.73 - Second Downtown**<br> |
| TUNNEL TYPE AND USE:<br>Double steel shell horseshoe element; Vehicular | | LANES/TRACKS:<br>One tube; two lanes | |
| NO OF ELEMENTS: 8 | LENGTH:<br>85.3 m to 101.2 m | HEIGHT: 10.5 m | WIDTH: 12.2 m |
| TOTAL IMMERSED LENGTH: 765 m | | DEPTH AT BOTTOM OF STRUCTURE: 13.7 m | |
| UNUSUAL FEATURES: | Horseshoe shape was used for economy because of the ventilation adopted. Elements were fabricated in Texas and shipped in pairs on a semi-submersible ocean-going barge. | | |
| ENVIRONMENTAL CONDITIONS: | Current 1.0 m/s (2 knots) max. Tide 1.2 m | | |
| FABRICATION METHOD:<br>In shipyard in Corpus Christi, Texas, 3,500 km from tunnel site. | | OUTFITTING:<br>At dockside near site. | JOINT TYPE:<br>Double gasket system. |
| WATERPROOFING METHOD: | Continuous steel shell. | | |
| PLACEMENT METHOD: | From catamaran barge system. Tube was brought out almost fully ballasted to reduce the time in the river channel.<br>Remotely controlled wedges used for horizontal alignment. | | |
| FOUNDATION METHOD: | Screeded gravel bedding. | | |
| VENTILATION TYPE: | Semi-transverse, using ceiling exhaust air duct. One ventilation building. | | |
| COVER AND TYPE: | 1.5 m of ordinary backfill. | | |
| ADDITIONAL INFORMATION: | OWNER: Virginia Department of Highways and Transportation.<br>DESIGNER: Parsons Brinckerhoff Quade & Douglas Inc<br>CONTRACTOR: J A Jones Construction Co and Schiavone Construction Co | | |

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Guldborgsund Tunnel; crossing the sound between the islands of Lolland and Falster, Denmark; 1988**

**T.74 - Guldborgsund**



TUNNEL TYPE AND USE:
Reinforced concrete box sections; Vehicular

LANES/TRACKS:
Two tubes; each with two lanes

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 2 | LENGTH: 230 m | HEIGHT: 7.6 m | WIDTH: 20.6 m |

TOTAL IMMERSED LENGTH: 460 m

DEPTH AT BOTTOM OF STRUCTURE: 13.8 m

UNUSUAL FEATURES: 230 m long, vertically curved elements are some of the longest ever produced. Computer-controlled dynamic ballasting was used.

FABRICATION METHOD:
The two tunnel elements were cast one at a time in a drydock located in the future tunnel ramp at Lolland. The elements were cast in 15 m to 15.3 m segments to prevent cracks. When cracks occurred in the first segment, cooling was used in the second element, and no cracks were found.

JOINT TYPE:
Between tunnel elements; Gina gasket supplemented with flat rubber interior gasket. At shore ends of tunnel: Temporary tightening by rubber lip gaskets, allowing ramps to be emptied of water, whereupon the permanent rubber Omega gaskets were installed between the tunnel and the portal structures. The tunnel was made monolithic by casting reinforced concrete in the temporary gap between the two tunnel elements.

WATERPROOFING METHOD: The elements were waterproofed with a 6 mm steel membrane on the bottom and sides. On the top, a bituminous membrane was installed and protected with 15 cm of reinforced concrete.

PLACEMENT METHOD: Each of the tunnel elements was provided with bulkheads at the ends and equipped with six ballast tanks with a total capacity of 4,000 m³ of water. After flotation, the elements were moved to their final position and sunk onto temporary supports approximately 1 m above the bottom of the excavated trench.

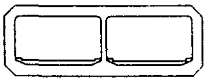
FOUNDATION METHOD:
Sandjetting

DREDGING METHOD:
A cutterhead suction dredge was used

VENTILATION TYPE: Longitudinal ventilation by means of jet fans

COVER AND TYPE: The eastern part, where the tunnel is located above the seabed, is protected with 1 m of gravel. No cover is provided for the remaining part of the tunnel.

ADDITIONAL INFORMATION:
The tunnel ramps are located in low water areas in the sound, and surrounded by dikes. Within the dikes, dewatering is performed by relief wells; the collected water is pumped out into the sound at the tunnel portals. The tunnel is designed to be unmanned, with automatic and remote controls
OWNER: Road Directorate
DESIGNER / ENGINEER: Christiani & Nielsen, Copenhagen
CONTRACTOR:Guldborg Sound Consortium: Armton A/S and Pihl & Son A/S

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.75 - Ems |
|---|---|
| Emstunnel; Leer, Germany; 1989 |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Concrete box elements; Vehicular | Two tubes; two lanes plus one emergency lane per tube. |

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 5 | LENGTH: 127.5 m | HEIGHT: 8.40 m | WIDTH: 27.50 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 639.5 m | DEPTH AT BOTTOM OF STRUCTURE:19 m below MWL |

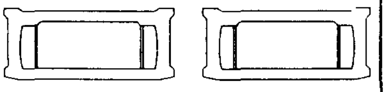
| | |
|---|---|
| UNUSUAL FEATURES: | An unexpectedly high rate of sedimentation (approx 15 cm per tide change) filled the trench immediately after each element was placed. Special equipment was developed to remove the siltation under the element a few meters in front of the sandjetting operation. |
| ENVIRONMENTAL CONDITIONS: | Mean tide 2.61 m. Mean current 1 m/s. Siltation was a problem |

| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| The elements were built together in a graving dock adjacent to the tunnel trench. Each element was cast in five 25.5 m segments. To prevent cracks, a cooling system was installed. The segments were post-tensioned for placing, after which the post-tensioning bars were cut. | After all of the elements were finished, the dock was flooded and the dike was removed using a dredge. Before floating each element, two pontoons equipped with lowering and mooring winches were floated over the elements and connected to them. After deballasting, the element floated with the pontoons on top. | Gina/Omega system. Elements divided into 25 segments. |

| | |
|---|---|
| WATERPROOFING METHOD: | Impermeable reinforced concrete with a bituminous layer over the roof, protected with 0.2 m of protection concrete. |
| PLACEMENT METHOD: | After winching the element to its location, it was placed by ballasting and lowering. It was temporarily supported by primary and secondary supports. |
| FOUNDATION METHOD: | Jetted-sand foundation. Special equipment was developed to clean the area under the tunnel just in front of the sandjetting operation |
| DREDGING METHOD: | Cutterhead suction dredge and dustpan dredger |
| VENTILATION TYPE: | Longitudinal |
| COVER AND TYPE: | Approximately 1 m of stone cover on reinforced concrete slab on roof the elements |

| | |
|---|---|
| BUOYANCY SF: 1.02 for placement | CONCRETE WORKING STRESS: DIN B35 |

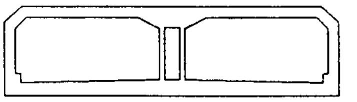
| | |
|---|---|
| ADDITIONAL INFORMATION: | CLIENT/OWNER: Niedersachsisches Landesamt für Strassenbau through agent: Autobahn Neubau Amt of Oldenburg<br>DESIGNER: IMS Ingenieurgesellschaft mbH, Hamburg<br>CONTRACTOR: Joint venture consisting of: Hollandsche Beton- en Waterbouw (HBW), and the local firms Brewaba (German company owned by HBW), Beton- und Tiefbau Mast Hermann Hein, Martin Oetken, Bauunternehmen Hein |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Marne Tunnel; under the River Marne, Paris, France; 1989** | | | **T.76 - Marne River**<br> |
| TUNNEL TYPE AND USE:<br>Prestressed concrete box sections; Vehicular | | LANES/TRACKS:<br>Two separate tunnels, each one tube, with three lanes of traffic in each tunnel. | |
| NO OF ELEMENTS: 7 | LENGTH: 45-55 m | HEIGHT: 9 m | WIDTH: 17.5 m |
| TOTAL IMMERSED LENGTH:<br>West Tunnel: 210 m East Tunnel: 140 m | | DEPTH AT BOTTOM OF STRUCTURE: | |
| ENVIRONMENTAL CONDITIONS: | Restricted channel area | | |
| FABRICATION METHOD:<br>In a casting basin at northern ramp site. | | JOINT TYPE:<br>Phoenix primary gasket and Omega secondary gasket. | |
| WATERPROOFING METHOD: | No waterproofing membrane, concrete prestressed in three directions. | | |
| PLACEMENT METHOD: | Using mooring lines and ballasting. Level control maintained by four corner tanks. Two access shafts, one at each end of element. | | |
| FOUNDATION METHOD: | Jacks and jetted or sand-flow foundation. | | |
| DREDGING METHOD: | Backhoe-type dredge. | | |
| VENTILATION TYPE: | Transverse, using side ventilation ducts. | | |
| ADDITIONAL INFORMATION: | OWNER: Departement de Val de Marne, Direction de l'Equipement<br>ENGINEER: Owner<br>CONTRACTOR: Bouygues Offshore | | |

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Zeeburger Tunnel; Amsterdam, The Netherlands; 1989**

**T.77 - Zeeburger**



TUNNEL TYPE AND USE:
Reinforced concrete box section; Vehicular

LANES/TRACKS:
Two tubes; three lanes each

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 3 | LENGTH: 112 m | HEIGHT: 7.82 m | WIDTH: 29.80 m |

TOTAL IMMERSED LENGTH: 336 m

DEPTH AT BOTTOM OF STRUCTURE: 14.60 m

UNUSUAL FEATURES:
Used telescoping pile to support element. Constructed inside cofferdam for cut-and-cover tunnel. End of last element protrudes into dock, and has a collar around it to permit dewatering and construction in the dry.

ENVIRONMENTAL CONDITIONS:
Bad soil conditions to - 28 m - then clay to - 38.0 m.

FABRICATION METHOD:
Each of the three elements was constructed in sequence in the open construction dock at one end of the tunnel, which was equipped with a specially designed gate. Each element was cast in five 22.4 m long sections, using a cooling system to prevent cracking.

JOINT TYPE:
Gina and Omega gaskets.

WATERPROOFING METHOD:
To prevent cracking, the elements were constructed of 22.4 m long sections, post-tensioned together. No exterior waterproofing system was used.

PLACEMENT METHOD:
A conventional system was used. The element was temporarily supported at the out-board end on two piles while the permanent pile connections were made.

FOUNDATION METHOD:
Supported on piles driven to a depth of 46 m. Eight piles were used along the outer walls of the tunnel for each 22 m section. The piles are equipped with telescoping inflatable forms, which can be post-grouted to complete the closure between the top of the pile and the underside of the tunnel. 120 steel tubular injection piles 0.5 m in diameter were used to support the elements.

DREDGING METHOD:
Barge-mounted crane and bucket.

COVER AND TYPE:
Gravel was placed on top of the tunnel. The trench was filled to -10 m, with the rest to be filled by natural siltation.

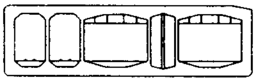
CONCRETE QUALITY:
B25

ADDITIONAL INFORMATION:
OWNER: Ministry of Public Works; Rigkswaterstaat
DESIGNER: Locks and Weirs Division of Rigkswaterstaat
CONTRACTOR: Van Hattum en Blankevoort b.v. Beverwijk

TUNNEL NAME/LOCATION/DATE COMPLETED:

**Hong Kong Eastern Harbour Crossing; Victoria Harbour; 1989**

**T.78 - Eastern Harbour Crossing**



TUNNEL TYPE AND USE:
Reinforced concrete box; Vehicular and railway

LANES/TRACKS:
Two rail tubes; two two-lane highway tubes; one duct for ventilation.

NO OF ELEMENTS: 15

LENGTH:
10 - 122.0 m
4 - 128.0 m
1 - 126.5 m

HEIGHT: 9.5 m

WIDTH: 35 m

TOTAL IMMERSED LENGTH: 1,859 m

DEPTH AT BOTTOM OF STRUCTURE: 27 m

UNUSUAL FEATURES: Very large elements with 40,000-42,000 ton displacement. Use of waterfilled inflatable rubber bag "jacks" to hold element at grade until sand could be jetted in place under it. The Cha Kwo Ling Ventilation Building was built as a floating structure with the last five elements. Double gates were used for the casting basin.

ENVIRONMENTAL CONDITIONS: Very busy harbour. Typhoon weather. Strong current (3 knots).

FABRICATION METHOD:
Elements were constructed in a rock quarry on the alignment. Gates were constructed in place before first flooding. Five elements were constructed at a time. Cycle times were 27 weeks, 25 weeks and 22 weeks, respectively.

OUTFITTING:
Installation of alignment survey towers and pontoons equipped with winches. Special telescoping towers were used due to height restriction of airport approach.

JOINT TYPE:
Gina and Omega gaskets.

WATERPROOFING METHOD: Concrete was poured in three stages - base, walls and roof - in successive bays up to 18 m long. Reinforcement was designed to limit crack width to prevent seepage. Heat of hydration was limited by replacement of 20% of cement with flyash. This also limited sulphate attack. The sides and roof were externally coated with sprayed-on epoxy rubber membrane. Visible cracks were grouted with epoxy resin prior to float-out.

PLACEMENT METHOD: Pontoons, mounted transversely over the element and controlled from the tower, lowered the element in place. Waterfilled inflatable bags were used as temporary support of element. Sand was jetted under element using slurry nozzles on each side of the element, fed from a barge.

FOUNDATION METHOD: Jetted sand.

VENTILATION TYPE: Fully transverse for vehicular tubes. Piston action for trains.

COVER AND TYPE: 1.5 m of backfill

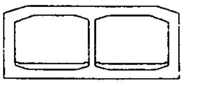
BUOYANCY SF: 1.02 with no backfill; 1.15 with backfill.

ADDITIONAL INFORMATION:
OWNER: New Hong Kong Tunnel Company
DESIGNER: Freeman Fox
CONTRACTOR: Kumagai Gumi Co Ltd, Marubeni Corp, Lilley Construction, Paul Y Construction, and China Int Trust and Investment Corp in Joint Venture

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Conwy Tunnel; North Wales, United Kingdom; 1991**

**T.79 - Conwy**



TUNNEL TYPE AND USE:
Concrete box elements; Vehicular

LANES/TRACKS:
Two tubes; two lanes each

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 6 | LENGTH: 118 m | HEIGHT: 10.4 m | WIDTH: 24.1 m |
| TOTAL IMMERSED LENGTH: 710 m | | DISPLACEMENT: 30,000 MT | DEPTH AT BOTTOM OF STRUCTURE: 17 m |

UNUSUAL FEATURES: The first immersed tunnel in the U.K. Due to the traffic requirements in the U.K., the tunnel is approximately 2 m higher than usual. The tunnel is skewed to the river, with an angle of less than 45 degrees. Two elements curved in plan.

ENVIRONMENTAL CONDITIONS: Sensitive historic and scenic area. River crossing. Required relocation of small boat anchorages

FABRICATION METHOD:
Elements were constructed together in a graving dock located next to the trench. Before casting, exterior waterproofing plates were placed on the bed. Each element was cast in segments, although continuous reinforcing was used. No cooling system was installed for the wall pours; but special coarse cement and pulverized fuel ash was used. Aggregates were cooled.

OUTFITTING:
After all of the elements were completed, the dock was flooded and the dike was removed by dredging. Before float-up of each element, two pontoons with lowering and mooring winches were floated over each element to be placed and hooked up to it. After deballasting, the element floated with the pontoons on top.

JOINT TYPE:
Gina/Omega joint with six separate shear keys. Continuous shear keys of concrete not provided at joint. Joint covered with stainless steel joint covers on inside of tunnel.

WATERPROOFING METHOD: The underside and walls are waterproofed with steel plate protected by a paint system and cathodic protection. A concrete protected bituminous waterproofing membrane is provided on the roof.

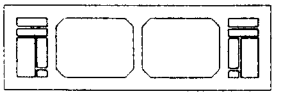
PLACEMENT METHOD: After winching the element to its location, it was placed by ballasting and lowering onto temporary footings.

FOUNDATION METHOD: Jetted-sand foundation. Delay in the start of sandjetting allowed sedimentation to occur under the tube. This was removed by airlifting.

DREDGING METHOD: Cutterhead suction dredge. Spoil used for project embankment construction

VENTILATION TYPE: Longitudinal, using booster fans

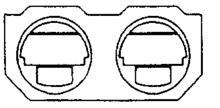
ADDITIONAL INFORMATION:
OWNER: Welsh Office
DESIGNER: Travers Morgan & Partners in association with Christiani & Nielsen A/S
CONTRACTOR: The principal contractor was a joint venture of Costain and Tarmac. Hollandsche Beton en Waterbouw were subcontractor for tunnel immersion and sand jetting.

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Liefkenshoek Tunnel; under River Schelde, Antwerp, Belgium; 1991</td><td>T.80 - Liefkenshoek<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Vehicular</td><td colspan="2">LANES/TRACKS:<br>Two tubes; two lanes each, with shoulders</td></tr>
<tr><td>NO OF ELEMENTS: 8</td><td>LENGTH: 142 m</td><td>HEIGHT: 9.6 m</td><td>WIDTH: 31.25 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 1,136 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE:</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Specifically designed to carry vehicles with hazardous cargos. Designed to be fire and explosion resistant. Funded under a concession contract. Separate escape passageways provided for each tube. A simulation was done of an anchor dropping on the immersed elements, by impact tests on the in-situ tunnel. Dynamic responses were measured.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Strong currents</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>All eight elements were built at the same time in a casting basin located in the harbour area. Each element was cast in six 23.65 m long segments. A cooling system was used to prevent cracks.</td><td>OUTFITTING:<br>After all elements were finished, the dock was flooded and the dike was removed by a dredge. Transverse floats were used. A temporary post-tensioning system was used. The elements had to pass through a lock before entering the river.</td><td>JOINT TYPE:<br>Gina and Omega gaskets</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">The elements were divided into six 23.7 m long segments. Longitudinal prestressing was provided in the floor and roof. Carefully designed concrete was used: 1140 kg 4/28 gravel; 730 kg sand 0/5; 270 kg blast-furnace cement HL30; 80 kg flyash; 130 l water; and 3 kg superpasticiser.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">A complete river and placement modelling study was carried out. The study showed that placement should be restricted to periods of neap to average tides to limit holding forces. A layer of trimming concrete was placed to reduce the freeboard without water ballast to 50 mm. Alignment towers 30 m high and other placement equipment was installed. The elements were towed using four 3,000 HP tugs. Other tugs were on standby. The element was supported on two support points on the tunnel in place, and on two jacks at the outboard end. Support pads 6 x 6 x 1.2 m were used.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Sand-water mixture was pumped through sandfill valves. To avoid siltation, this operation was started within one hour after the sinking operation was completed.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">A cutterhead suction dredger</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Full transverse ventilation was used because of the hazardous cargo criteria</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">Special asphalt mattresses were incorporated in the tunnel cover design to protect the tunnel</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">The interior of the tunnel was protected by 50 mm of insulation as a result of testing for a four-hour fire exposure. The tunnel design required some 50-60 kg of additional reinforcing to cope with a 5-bar explosion overpressure.<br>OWNER: Ministry of Public Works and Transport<br>DESIGNER: Haecon NV, TKB and TFD (Belgium)<br>CONTRACTOR: Joint venture of De Meyer-Van Laere-Betonac</td></tr>
</table>

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Monitor-Merrimac Tunnel; Hampton Roads, between Newport News and Chesapeake, Va, U.S.A.; 1992**

**T.81 - Monitor-Merrimac**



| | |
|---|---|
| TUNNEL TYPE AND USE:<br>Double-steel shell binocular section; Vehicular | LANES/TRACKS:<br>Two tubes; two lanes in each tube |

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 15 | LENGTH: 95 m | HEIGHT: 12 m | WIDTH: 24 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 1,425 m | DEPTH AT BOTTOM OF STRUCTURE: 36 m |

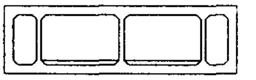
| | |
|---|---|
| UNUSUAL FEATURES: | Extends between man-made peninsula and a man-made island, where it joins a trestle. At 6 miles long, it is one of the longest subsea crossings. This is one of the few immersed tunnel projects in the U.S. for which a drydock was used. The navigation channel was so narrow that it had to be closed during placement of the elements. |
| ENVIRONMENTAL CONDITIONS: | Very strong tidal currents around the point where the James River meets the open water of Hampton Roads required heavy anchor lines and special precautions during maneuvering and placement of the elements. Very active ship channel, with much pleasure-boating. |

| | | |
|---|---|---|
| FABRICATION METHOD:<br>Elements were fabricated in Baltimore at the Sparrows Point Shipyard of Bethlehem Steel Co. The elements were assembled in a drydock, and all interior concrete (except walkways) was installed before closing of the ends with bulkheads. The elements were then floated out with approximately 7.5 m draft. An additional 2,700 $m^3$ was added at a pier prior to towing the element to Newport News (draft was about 10 m). | OUTFITTING:<br>Interior concrete was placed at shipyard. Final concrete was placed at jetty in Newport News, Va. | JOINT TYPE:<br>Standard double gasket and closure plate detail used in the U.S. for many years. |

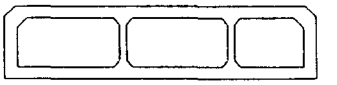
| | |
|---|---|
| WATERPROOFING METHOD: | Steel shell and liner plate. |
| PLACEMENT METHOD: | Remotely controlled wedges used for horizontal alignment. Conventional catamaran barge system was used, with a laybarge consisting of two railroad car barges with support beams between them. |
| FOUNDATION METHOD: | Screeded foundation. Screed rig designed to be unaffected by tidal variations. |
| DREDGING METHOD: | Cutterhead suction dredging and pumping behind dikes to form man-made island and peninsula. Clean-up dredging was done with a clamshell dredge. |
| VENTILATION TYPE: | Fully transverse ventilation system split between two ventilation buildings. |

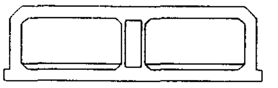
| | | |
|---|---|---|
| COVER AND TYPE:<br>A 0.90 m wall was constructed on each element to contain rock ballast. In addition, rock and armour stone were placed on top of the element as protection. | BUOYANCY SF: 1.03.<br>After dewatering, joints, 1.07 without backfill. | CONCRETE WORKING STRESS:<br>4,000 psi |

| | |
|---|---|
| ADDITIONAL INFORMATION: | OWNER: Virginia Department of Transportation.<br>DESIGNER: Sverdrup Corporation.<br>CONTRACTOR: Morrison Knudsen Inc, and Interbeton Inc in Joint Venture. |

| | |
|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br>**Sydney Harbour Tunnel; Sydney, Australia; 1992** | **T.82 - Sydney Harbour**<br> |
| TUNNEL TYPE AND USE: Concrete box elements; Vehicular | LANES/TRACKS: Two tubes; two lanes each |
| NO OF ELEMENTS: 8 · LENGTH: 120 m | HEIGHT: 7.80 m · WIDTH: 29.4 m |
| TOTAL IMMERSED LENGTH: 960 m | DEPTH AT BOTTOM OF STRUCTURE: 25 m |

| | | |
|---|---|---|
| UNUSUAL FEATURES: | Most northern element was laid against the ventilation building. A short stub section of tunnel built into the ventilation building was provided with a sill beam to receive the first element. A cofferdam was built over and around the end of the element; a tremie seal to the sandstone rock was made at the front. To allow for differential settlement between element Nos 7 and 8, a special flexible joint was attached to the end of element No 7. A permeable form was used to improve durability of concrete for the elements. | |
| ENVIRONMENTAL CONDITIONS: | Very environmental sensitive area at scenic Sydney Harbour, with the Sydney Opera House almost in the alignment. These conditions led to the unique use of an adjacent bridge pylon for vent stack. | |
| FABRICATION METHOD:<br>The elements were built in two groups of four in a graving dock. The elements have continuous reinforcement, but were cast in sections. | TOWING / OUTFITTING:<br>Elements were cast in Port Kembla, 100 km away. Extensive model studies were undertaken to assure feasibility of tow. A freeboard of 0.5 m was used during tow. At the outfitting pier in Sydney, further ballast and placing equipment was installed, bringing elements to 10 cm of freeboard. | JOINT TYPE:<br>Gina and Omega type of joint. A special prefabricated settlement joint was provided on element No 7, which was founded on sand. Element No 8 was connected into a sandstone wall and a tunnel was mined to it. |
| WATERPROOFING METHOD: | A PVC membrane was used on the bottom. The sides and top were covered with an epoxy resin coating. Low heat of hydration with good impermeability to chloride ion penetration was achieved using a high replacement blend of Type A cement and ground granulated blast furnace slag. Suphate resistance was also good. | |
| PLACEMENT METHOD: | Transverse pontoons. | |
| FOUNDATION METHOD: | Sand-flow method utilizing pipes installed in the walls from the roof slab to the base slab. Element No 8 was supported on a foundation of cement-based grout. The other elements were founded on sand. | |
| DREDGING METHOD: | Alluvial deposits were dredged using a grab dredge. Sandstone deposits were dredged using a cutter-head suction dredge. Provisions made for blasting were not required. | |
| VENTILATION TYPE: | Semi-transverse, using side ducts. Pylons of the Sydney Harbour bridge were used as exhaust stacks at one end of tunnel | |
| COVER AND TYPE: | A 2 m cover of rock fill with rock armour flanks was provided. The rock fill was designed to absorb the impact of a falling anchor; the rock armour was designed to deflect an anchor dragged across the tunnel. | |
| ADDITIONAL INFORMATION: | CLIENT/OWNER: N.S.W. Ministry of Public Works and Roads<br>DESIGNER:<br>Main consultants: McDonald Wagner, Freeman Fox (HK)<br>Rock tunnels and approaches: Mott, Hay & Anderson, Maunsell & Partners<br>M&E: Parson Brinckerhoff, Gutterridge, Haskins and Davey<br>Soil survey: Coffey and Partners<br>CONTRACTOR: Sydney Harbour Tunnel Group: Transfield Pty Ltd 50%<br>Kumagai Gumi Co Ltd 50% | |

| | |
|---|---|
| TUNNEL NAME/LOCATION/DATE COMPLETED: **Grouw Tunnel; Grouw, The Netherlands; 1992** | **T.83 - Grouw**  |
| TUNNEL TYPE AND USE: Concrete box tunnel; Vehicular | LANES/TRACKS: Two tubes with two lanes each for the highway and one tube for bidirectional local road. |
| NO OF ELEMENTS: 1 | LENGTH: 68 m |
| HEIGHT: 7.05 m | WIDTH: 31.75 m |
| TOTAL IMMERSED LENGTH: 68 m | DEPTH AT BOTTOM OF STRUCTURE: 11.5 m |
| UNUSUAL FEATURES: | Supported on the abutments. |
| FABRICATION METHOD: | The element was constructed in the approach. It was cast in four sections without flexible joints. Reinforcement was continuous and the tunnel was permanently prestressed. |
| OUTFITTING: | After the approach structure was flooded, the element was pulled through it across the channel. Water ballast tanks were in river retaining structures on top of the element. |
| JOINT TYPE: | An inflatable rubber gasket and an Omega gasket. |
| WATERPROOFING METHOD: | There was no separate exterior waterproofing membrane. |
| PLACEMENT METHOD: | The river retaining structures were designed to eliminate the need for additional equipment. Water ballasting was sufficient. |
| FOUNDATION METHOD: | The element was supported on both abutments like a bridge. No sand was provided under the tunnel. |
| DREDGING METHOD: | Cutterhead suction dredger. |
| VENTILATION TYPE: | No ventilation system |
| COVER AND TYPE: 10 cm of concrete. | CONCRETE QUALITY: B35 |
| ADDITIONAL INFORMATION: | OWNER: Ministry of Transport, Public Works and Water Management; Rijkswaterstaat<br>DESIGNER: Civil Engineering Division (former Locks and Weirs Division) of Rijkswaterstaat<br>CONTRACTOR: Ballast Nedam Beton en Waterbouw b.v. |

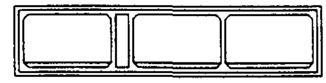
| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.84 - Noord |
|---|---|
| **Noord Tunnel; Alblasserdam, The Netherlands (crossing De Noord River); 1992** |  |

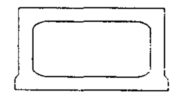
| TUNNEL TYPE AND USE: | | LANES/TRACKS: | |
|---|---|---|---|
| Concrete box elements; Vehicular | | Two tubes; three lanes each tube | |
| NO OF ELEMENTS: 4 | LENGTH:<br>3 - 130 m<br>1 - 100 m | HEIGHT: 8.03 m | WIDTH: 29.95 m |
| TOTAL IMMERSED LENGTH: 492 m | | DEPTH AT BOTTOM OF STRUCTURE: 16 m | |

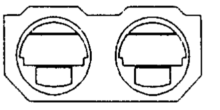
| | |
|---|---|
| UNUSUAL FEATURES: | To avoid dredging of the river to allow transport of the elements, the roofs of the elements were only partially cast. This reduced the draft of the elements enough to tow them to the site. The roofs were then completed. |

| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| The elements were constructed together in a graving dock, at the same time that the elements for the Willemspoor Tunnel were under construction. Three elements were cast in five 26.15 m segments; one element was cast in four 24.85 m segments. A cooling system was installed. | The elements were deballasted, floated, and prepared for a two-day tow over tidal waters and rivers. After the elements arrived at the site, the remaining portions of the roof were cast, as well as a portion of the ballast concrete on the bottom slab. This was used as added weight for placing | Gina/Omega |

| | |
|---|---|
| WATERPROOFING METHOD: | No exterior waterproofing was used. Concrete segments were sized and designed to avoid cracking. Concrete was designed to reduce thermal cracking and produce high-density impermeable segments. |
| PLACEMENT METHOD: | Four small pontoons with hoisting and mooring winches were hoisted on top of the tunnel. Conventional placing methods were used. The closure joint was made in the river between the last two elements. |
| FOUNDATION METHOD: | Sand-flow method |
| DREDGING METHOD: | Cutterhead suction dredge. |
| VENTILATION TYPE: | Longitudinal |
| COVER AND TYPE: | 1.0 m of sand protection |
| ADDITIONAL INFORMATION: | OWNER: Ministry of Transport, Public Works, and Water Management, Rijkswaterstaat<br>DESIGNER: Civil Engineering Division of Rijkswaterstaat<br>ENGINEER: Consortium - DHV, Haskoning and Wittenveen & Bos forming Tunnel Engineering Consultants (TEC) worked in a form of Joint Venture with engineers from Rijkswaterstaat.<br>CONTRACTORS: Kombinatie Tunnelbouw (KTB), a consortium of four firms: Ballast Nedam, Van Hattum & Blankevoort, Hollandsche Beton & Waterbouw, and Dirk Verstoep. |

| | | | |
|---|---|---|---|
| TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>**Pearl River Tunnel; Guangzhou, P.R. China; 1993** | | | **T.85 - Pearl River**<br> |
| TUNNEL TYPE AND USE:<br>Concrete box elements; Vehicles and metro | | LANES/TRACKS:<br>Two tubes for two lane traffic, one tube for two tracks (metro) | |
| NO OF ELEMENTS: 5 | LENGTH: 105 m; 120 m; 120m; 90 m, 22 m | HEIGHT: 7.95 m | WIDTH: 33.0 m |
| TOTAL IMMERSED LENGTH: 457 m | | DEPTH AT BOTTOM OF STRUCTURE: - 13.16 m | |
| UNUSUAL FEATURES: | Heavily weathered rock on river bottom. | | |
| ENVIRONMENTAL CONDITIONS: | High water velocity to more than 2.0 m/sec. Elements stabilized by added water ballast. Earthquake zone. | | |
| FABRICATION METHOD: In dry dock at site of cut and cover tunnel, one at a time for elements over 100 m. long. Dry dock provided with a steel gate. Cooling water was used in wall pours. | | JOINT TYPE: Gina and Omega joint. | |
| WATERPROOFING METHOD: | Shrinkage control by cooling during concrete walls pours and division of elements into shorter lengths, continuous reinforcement. Steel plate attached to bottom slab. | | |
| PLACEMENT METHOD: | Lowering by means of a 500 ton crane barge. Two alignment towers were used on the elements. Three point temporary foundation, in board supports on roof of previous element and two jacks on temporary support on outboard end. | | |
| FOUNDATION METHOD: | Sand-flow. | | |
| DREDGING METHOD: | Blasting and grab dredge. | | |
| VENTILATION TYPE: | Longitudinal with jet fans | | |
| COVER AND TYPE: | 1 m backfill of crushed stone | | |
| ADDITIONAL INFORMATION: | OWNER: Guangzhou Municipal Government<br>DESIGNER: Guangzhou Metro Design Institute<br>CONTRACTOR: Local construction companies | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Météor Tunnel; under la Seine at Pont de Tolbiac, Paris, France; 1994** | | | **T.86 - Météor**<br> |
| TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Railroad | | LANES/TRACKS:<br>One tube; two tracks | |
| NO OF ELEMENTS: 4 | LENGTH: 34 m | HEIGHT: 6.30 m | WIDTH: 9.18 m |
| TOTAL IMMERSED LENGTH: 136 m | | DEPTH AT BOTTOM OF STRUCTURE: 14 m | |
| UNUSUAL FEATURES: | Trench excavated in limestone. Elements constructed on a hydraulically operated platform next to trench, then lowered on platform and moved into trench. | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD: | | JOINT TYPE: | |
| WATERPROOFING METHOD: | Permanently prestressed. | | |
| PLACEMENT METHOD: | Four buoyancy towers on the roof were used to sink load element. Element was temporarily supported on concrete pads. | | |
| FOUNDATION METHOD: | After element was in position on the temporary concrete pads, it was permanently fixed by the placement of tremie concrete underneath. | | |
| DREDGING METHOD: | Clamshell dredging | | |
| VENTILATION TYPE: | Piston action of trains | | |
| COVER AND TYPE: | Approximately two meters of sand and stone | | |
| ADDITIONAL INFORMATION: | OWNER: RATP<br>CONTRACTOR: JV of 9 firms, Spie-Batignolles being leading partner | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.87 - Ted Williams |
|---|---|
| **Ted Williams Tunnel; Boston Harbour, Boston, Massachusetts, U.S.A.; 1994** |  |

| | |
|---|---|
| TUNNEL TYPE AND USE: Double steel shell binocular section; Vehicular | LANES/TRACKS: Two tubes; two lanes each tube |

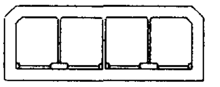
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 12 | LENGTH: 98.30 m | HEIGHT: 12.29 m | WIDTH: 24.43 m |

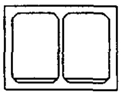
| | |
|---|---|
| TOTAL IMMERSED LENGTH: 1,172.9 m | DEPTH AT BOTTOM OF STRUCTURE: 30 m |

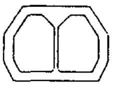
| | |
|---|---|
| UNUSUAL FEATURES: | Much of tunnel trench excavated through hard argellite rock. Immersed tunnel ends incorporated into very deep cofferdams. Depth at cofferdams at bottom of end elements more than 20 m below sea level. First use of Omega gasket in a U.S. tunnel |
| ENVIRONMENTAL CONDITIONS: | Many environmental constraints regarding filling in harbour, spawning of fish, blasting, siltation, work around and airside at Logan International Airport, and impacts from harbour celebrations and activities such as "Sail Boston '92." |

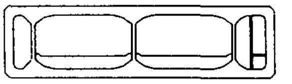
| | | |
|---|---|---|
| FABRICATION METHOD: At shipyard, elements assembled in drydock two at a time and floated out. Each tube floated onto large ocean-going submersible barge (106 x 30 m) for tow to Boston. Tow made using a single 9,000 HP ocean tug direct route between Norfolk, VA., and Cape Cod, Mass. | OUTFITTING: At pier near tunnel site. | JOINT TYPE: Double gasket system commonly used in U.S., but with Omega secondary gasket. Joint detail provides for thermal expansion of joints. |

| | |
|---|---|
| WATERPROOFING METHOD: | Steel shell and Omega gasket closures. Closure plate detail not used on this project. |
| PLACEMENT METHOD: | From catamaran barges. Remotely controlled wedges used for horizontal alignment. |
| FOUNDATION METHOD: | Screeded foundation. |
| DREDGING METHOD: | Drill and blast. Used the "Super Scoop", owned by Dutra Corp., with up to 17.5 $m^3$ buckets for removal of silts, sands, till and rock. |
| VENTILATION TYPE: | Fully transverse ventilation system. Centrifugal fans. |
| COVER AND TYPE: | 0.6 m of protective rock over 0.9 m of gravel |
| ADDITIONAL INFORMATION: | OWNER: Massachusetts Highway Department.<br>DESIGNER: Preliminary Design: Bechtel/Parsons Brinckerhoff. Final Design: Sverdrup Corporation.<br>CONTRACTOR: Morrison Knudsen/Interbeton/J.F. White joint venture |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Willemspoortunnel; Rotterdam, The Netherlands; 1994</td><td>T.88 - Willemspoor<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Concrete box elements; Railway</td><td colspan="2">LANES/TRACKS:<br>Four tubes; one track each tube</td></tr>
<tr><td>NO OF ELEMENTS: 8</td><td>LENGTH: 115-138 m</td><td>HEIGHT: 8.62 m</td><td>WIDTH: 28.82 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 1,014 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: - 17.5</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Unusually difficult, restrictive urban and marine site. Elements placed between coffer-dam walls in some areas.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Very difficult urban and marine constraints on construction. Heavily trafficed waterway.. Close to many sensitive existing structures and rail facilities.</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>Elements were fabricated in five or six 23 m segments post-tensioned together. A cooling system was used. Segments have large triangular shear keys shaped in the vertical walls. Casting was done in an existing basin at Barendrecht 30 km from tunnel site where several other immersed tunnels have been built.</td><td>OUTFITTING:<br>Outfitting for placement was done in the casting basin. Elements were ballasted to keep them on the bottom prior to placement. Each was then floated individually and towed to the site with 20 cm of free-board. The trip took one day. A catamaran barge system was used for placing.</td><td>JOINT TYPE:<br>Gina/Omega</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">No waterproofing layers were used. Concrete segments were sized and designed to prevent cracking, and careful concrete mix and temperature control were used for each segment.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">On the left (northwest) side of the Maas, the elements were lowered from girders. On the right side, across the Nieuwe Maas, floating pontoons were used. The closure joint was made between the last two elements.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Sand-flow foundation.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">Backhoe dredging and a cutterhead suction dredge</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Piston action of trains</td></tr>
<tr><td colspan="2">COVER AND TYPE: 1.0 m protection</td><td colspan="2">CONCRETE QUALITY: B30</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: Dutch Railway<br>DESIGNER: Civil Engineering Division of Rijkswaterstaat<br>CONTRACTOR: KWT Joint Venture: Dirk Verstoep Rotterdam; Ballast Nedam; Hollandsche Beton en Water bouw; Strukton Beton bouw</td></tr>
</table>

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>MTRC Advanced Unit; Hong Kong; 1994</td><td>T.89 - MTRC<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Concrete box, railway</td><td colspan="2">LANES/TRACKS:<br>Two tubes, one track each</td></tr>
<tr><td>NO OF ELEMENTS: 1</td><td>LENGTH: 80 m</td><td>HEIGHT: 7.57 m</td><td>WIDTH: 12.42 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 80 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 16 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Part of a land reclamation contract and was needed early for that reason. Provides stub-end for remainder of immersed tunnel crossing (see T.97). Ventilation and station building attached to the landward end as part of a separate contract via a cast-in-place reinforced concrete connection. Tracks diverge. Temporary access via two snorkels on roof.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">Placed in sheltered area, partially buried in land reclamation.</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>Constructed on semisubmersible barge, harbor tow.</td><td colspan="2">JOINT TYPE: Gina gasket, Omega seal and shear key joint built into the end for later release after cast-in-place connection to building.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Steel plate on underside. Spray on membrane to sides and roof</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Floated, placed by crane</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Screed</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">Bucket grab dredger</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Longitudinal</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">Reclamation</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">OWNER: Mass Transit Railway Corporation, Hong Kong<br>DESIGNER: Acer Consultants (Far East) Ltd.<br>CONTRACTOR: Dragages - Penta Ocean Joint Venture</td></tr>
</table>

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Bilbao Metro; rail tunnel under Bilbao Estuary for Deusto-Olavega section of Bilbao Metro Line 1, Bilbao, Spain; 1994</td><td>T.90 - Bilbao Metro<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Reinforced concrete box elements; Metro railway</td><td colspan="2">LANES/TRACKS:<br>Two tubes; one track each tube</td></tr>
<tr><td>NO OF ELEMENTS: 2</td><td>LENGTH: 85.35 m</td><td>HEIGHT: 7.2 m</td><td>WIDTH: 11.4 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 172.2 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: 17 m</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>In casting basin within excavation for cut-and-cover tunnel on Deusto side of river.</td><td>OUTFITTING:<br>Partly in casting basin, partly at tunnel site.</td><td>JOINT TYPE:<br>Gina/Omega</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Steel membrane on bottom and sides. Bituminous membrane on roof.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Catamaran type sinking rig at river end, fixed cross beam at opposite end</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Grout bags</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Piston action</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3">CLIENT/OWNER: Gobierno Vasco, Departamento de Transportes y Obras Publicas<br>DESIGNER:<br>Tender Project: SENER assisted by Mott McDonald<br>Final Project : Agroman Empresa Constructora S.A. assisted by<br>Christiani & Nielsen Ltd</td></tr>
</table>

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.91 - Tama River |
|---|---|
| **Tama River Tunnel; Tokyo, Japan; 1994** |  |

| | |
|---|---|
| TUNNEL TYPE AND USE: Reinforced concrete box prestressed longitudinally for watertightness; Vehicular | LANES/TRACKS: Two tubes; three lanes each. Escape passageways and utility space for gas, electric and drainage lines in the side tubes. |

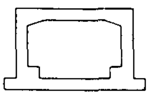
| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 12 | LENGTH: 128.6 m | HEIGHT: 10.0 m | WIDTH: 39.9 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 1,549.5 | DEPTH AT BOTTOM OF STRUCTURE: 30 m |
| UNUSUAL FEATURES: | Half of the tunnel was constructed on a curve. Flexible earthquake joints were provided between all the tunnel elements. Piles were driven in casting basin bottom after it was stabilized with soil cement. Liquid nitrogen was used to cool sand during summer concreting. Blast furnace cement was used to reduce thermal cracking. |
| ENVIRONMENTAL CONDITIONS: | At the mouth of the Tama River. |
| FABRICATION METHOD: Fabricated in a graving basin used jointly with the Kawasaki Fairway Tunnels. The basin accommodated 11 elements at a time and had a floor area of 107,000 m² (592 m x 190 m). | JOINT TYPE: Gina/Omega type joint. Flexibility is provided by the rubber joint, in combination with the use of post-tensioned rods tightened across the joint. Three vertical shear keys provided in the walls and one horizontal key in the floor provide stability against displacements due to earthquakes and/or settlements. |
| WATERPROOFING METHOD: | 8 mm steel plate on sides and bottom of elements; concrete protected by 2.5 mm thick rubberized membrane on top slab. Steel plates are cathodically protected. |
| PLACEMENT METHOD: | Catamaran barges. 10 cm freeboard. 500 MT during lowering (1%) and 1500 MT in place (3%) prior to placing final ballast (1.1 SF against flotation, not counting backfill). |
| FOUNDATION METHOD: | Mortar pumped over gravel foundation bed. |
| VENTILATION TYPE: | Longitudinal ventilation. |
| COVER AND TYPE: | 1.5 of stone cover |
| ADDITIONAL INFORMATION: | Mix proportions for tunnel element concrete: w/c 0.515; water 155 kg/m³; cement 301 kg/m³; sand 834 kg/m³ and gravel 1034 kg/m³.<br><br>OWNER: Metropolitan Expressway Public Corporation<br>CONTRACTORS: Tamagawa Tunnel Joint Venture: Kajima Corporation; Kumagai Gumi Co Ltd; Ohbayashi Corporation; Shimizu Corporation; Nishimatsu Construction Co Ltd; Okumura Corporation; Fujita Corporation, Toa Corporation. |

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Schiphol Railway Tunnel; Amsterdam, The Netherlands; 1995**

**T.92 - Schiphol**



| | |
|---|---|
| TUNNEL TYPE AND USE:<br>Concrete box tunnel; Railway | LANES/TRACKS:<br>One tube; two tracks |

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 4 | LENGTH: 125 m | HEIGHT: 8.05 m | WIDTH: 13.60 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 500 m | DEPTH AT BOTTOM OF STRUCTURE: 9 m |

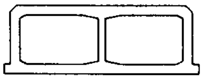
| | |
|---|---|
| UNUSUAL FEATURES: | The railway tracks under the airport were doubled (from two to four). The new tunnel was constructed directly adjacent to the existing tunnel. Most of the 5.7 km of the railroad tunnels were constructed by cut-and-cover; however, a section 500 m long passing under a runway was constructed as an immersed tunnel. This was done so that the closure of the runway could be reduced to the minimum time possible. |

| | | |
|---|---|---|
| FABRICATION METHOD:<br>The casting basin made use of the adjacent cut-and-cover area. Each element consisted of six segments with flexible joints. | OUTFITTING:<br>The casting basin consisted of a cross-braced sheetpile cofferdam. The elements were fully outfitted in the basin. | JOINT TYPE:<br>Gina and Omega gaskets were used. |

| | |
|---|---|
| WATERPROOFING METHOD: | No exterior waterproofing membrane was used. |
| PLACEMENT METHOD: | After the dock was filled with groundwater, the elements were floated to a freeboard of 15 cm and pulled by winches under the bracing to their position. The elements were lowered from hoists on beams which spanned across the top of the cofferdam as they were ballasted with water. |
| FOUNDATION METHOD: | Sand-flow method |
| DREDGING METHOD: | Land excavation |
| VENTILATION TYPE: | No ventilation system. |

| | |
|---|---|
| COVER AND TYPE:<br>1 m of compacted layers under the runway pavement. | CONCRETE WORKING STRESS:<br>B35 |

| | |
|---|---|
| ADDITIONAL INFORMATION: | CLIENT/OWNER: Dutch Railway<br>DESIGNER: Holland Rail Consult<br>CONTRACTOR: KSS, Consortium of Hollandsche Beton en Waterbouw and Structon |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.93 - Medway |
|---|---|
| Medway Tunnel; Rochester, United Kingdom; 1996 |  |

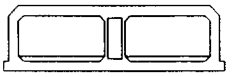
| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Concrete box sections; Vehicular | Two tubes with two lanes each. |

| NO OF ELEMENTS: 3 | LENGTH: 2 - 126 m<br>1 - 118 m | HEIGHT: 9.15 m | WIDTH: 25.1 m |
|---|---|---|---|

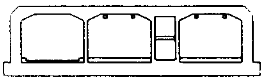
| TOTAL IMMERSED LENGTH: 370 m | DEPTH AT BOTTOM OF STRUCTURE: 18.65 m |
|---|---|

| FABRICATION METHOD: | OUTFITTING: | JOINT TYPE: |
|---|---|---|
| A graving dock for the elements was provided at the eastern portal. On completion of the elements, the graving dock was opened, and the elements were removed and placed. The approach structure was then constructed in the closed dock. The elements were each cast in six 21 m segments. | The elements were deballasted, floated and towed out of the dock into the trench and placed. A small catamaran barge system was used. | Gina/Omega gasket system |

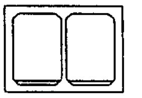
| | |
|---|---|
| UNUSUAL FEATURES: | First design/construct tunnel in United Kingdom. The tunnel was designed to withstand possible uplift from water pressures in a chalk layer |
| WATERPROOFING METHOD: | No waterproofing membrane. Concrete cooling was used to make structure watertight. |
| PLACEMENT METHOD: | The elements were lowered by two small catamarans while the elements were positioned with winches. The closure joint was made in the dry between the last element and the approach structure. |
| DREDGING METHOD: | Backhoe |
| FOUNDATION METHOD: | Sand-flow method |
| VENTILATION TYPE: | Longitudinal. Fans are located in recesses in the roof of both portal cut-and-cover sections. This arrangement reduced the height of the elements considerably. |
| COVER AND TYPE: | 2 m of stone protection and a seal in the river on top of the tunnel, to prevent river water from entering the chalk |
| ADDITIONAL INFORMATION: | CLIENT/OWNER: Kent County Council with the support of Rochester Bridge Trust, English Partnerships, Rochestor upon Medway City Council and Gilligan Borough Council<br>DESIGN ENGINEER; OWNER'S CONSULTANT: Travers Morgan in association with Rendel Palmer & Tritton. Detailed design on behalf of contractor was performed by Mott MacDonald<br>CONTRACTOR: Joint Venture of Tarmac and Hollandsche Beton Maatschappij (Design - Construct) |

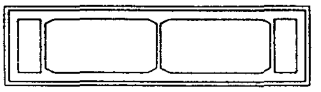
| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:** **Wijker Tunnel; Wijkermeer near Ijmuiden, Velsen, The Netherlands; 1996** | | | **T.94 - Wijker**  |
| TUNNEL TYPE AND USE: Concrete box section; Vehicular | | LANES/TRACKS: Two tubes with three lanes each | |
| NO OF ELEMENTS: 6 | LENGTH: 95.67 m | HEIGHT: 8.05 m | WIDTH: 31.5 m |
| TOTAL IMMERSED LENGTH: 574 m | | DEPTH AT BOTTOM OF STRUCTURE: 24.5 m | |
| UNUSUAL FEATURES: | Tunnel elements built at Barendrecht at Oude Maas River, towed through Nieuwe Waterweg to the North Sea, through the sluice at Ijmuiden and the North Sea Canal to tunnel site. | | |
| ENVIRONMENTAL CONDITIONS: | Canal, no current | | |
| FABRICATION METHOD: Elements constructed in existing casting basin in Barendrecht near Rotterdam. Elements were prestressed for loading conditions due to transport. Concrete working stress B35. | | OUTFITTING: On site after arrival. | JOINT TYPE: Rubber seal type Gina and Omega seal |
| WATERPROOFING METHOD: | No waterproofing membrane. Cooling of concrete was used to assure that no cracking occurred during fabrication of elements. | | |
| PLACEMENT METHOD: | Element mounted pontoons at primary end; single floating crane at secondary end of tunnel element. | | |
| FOUNDATION METHOD: | Sand-flow method | | |
| DREDGING METHOD: | | | |
| VENTILATION TYPE: | Longitudinal. | | |
| COVER AND TYPE: | 1.0 m of sand protection | | |
| ADDITIONAL INFORMATION: | OWNER: Ministry of Transport Public Works and Water Management; Rijswaterstaat<br>DESIGNER, Basic: Civil Engineering Division of Rijkswaterstaat<br>Detailed: Tunnel Engineering Consultants (TEC)<br>CONTRACTOR: Tunnel Combinatie Nederland, a joint venture of Strukton Betonbouw, Haverkort Infrabouw, Bam Bredero Bouw, Van Oord ACZ and Dywidag | | |

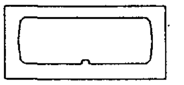
| TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>**Yong River Tunnel, Ningbo, P. R. Of China; 1996** | | | **T.95 - Yong River** |
|---|---|---|---|
| TUNNEL TYPE AND USE:<br>Concrete box elements; traffic tunnel | | LANES/TRACKS:<br>One tube for two lanes | |
| NO OF ELEMENTS: | LENGTH: 4 - 85 m<br>1 - 80 m | HEIGHT: 7.65 m | WIDTH: 11.9 m |
| TOTAL IMMERSED LENGTH: 420 m | | DEPTH AT BOTTOM OF STRUCTURE: - 15.55 m | |
| UNUSUAL FEATURES: | On soft soil (clay) | | |
| ENVIRONMENTAL CONDITIONS: | Near river mouth. Rapid rate of siltation. | | |
| FABRICATION METHOD:<br>Cast in dry dock in one batch. | | JOINT TYPE: Gina and Omega joint | |
| WATERPROOFING METHOD: | Steel plate on bottom slab. | | |
| PLACEMENT METHOD: | Lowered using steel pontoon of 46 m long | | |
| FOUNDATION METHOD: | 60 cm crushed stone and 40 cm cement mortar | | |
| DREDGING METHOD: | Grab dredger | | |
| VENTILATION TYPE: | Longitudinal with jet fans | | |
| COVER AND TYPE: | Layer of soil | | |
| ADDITIONAL INFORMATION: | OWNER: Ningbo Municipal Government<br>DESIGNER: Zhejiang Communication Design Institute and Shanghai Tunnel Engineering Design Institute<br>CONTRACTOR: Local Construction Companies | | |

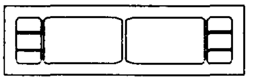
| TUNNEL NAME/LOCATION/DATE COMPLETED: | **T.96 - Piet Hein** |
|---|---|
| **Piet Hein Tunnel; Amsterdam, The Netherlands; 1997** |  |

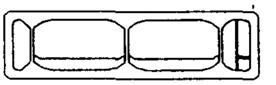
| | |
|---|---|
| TUNNEL TYPE AND USE:<br>Reinforced concrete box section; Vehicular and light train traffic. | LANES/TRACKS:<br>Three tubes; two with two lanes for vehicles<br>one with two tracks for trains |
| NO OF ELEMENTS: 8 / LENGTH: 160 m approx | HEIGHT: 8 m / WIDTH: 32 m |
| TOTAL IMMERSED LENGTH: 1,265 m | DEPTH AT BOTTOM OF STRUCTURE: -16.97 |
| UNUSUAL FEATURES: | Tunnel elements built in existing casting basin, Verreroek, 16 km from Antwerp, Belgium, and towed through Western Scheldt, through locks at Ijmuiden to Amsterdam. |
| ENVIRONMENTAL CONDITIONS: | Sand, clay, and soft soil conditions |
| FABRICATION METHOD:<br>Elements cast in segments, prestressed to take transport loadings. | JOINT TYPE:<br>Gina/Omega gaskets |
| WATERPROOFING METHOD: | Elements cast in segments, concrete was cooled to prevent cracking and prestressing was provided during transport. |
| PLACEMENT METHOD: | Pontoons on top of the element at the inboard end and a floating crane on the outboard end. |
| FOUNDATION METHOD: | Sand flow |
| DREDGING METHOD: | Bucket dredge |
| VENTILATION TYPE: | Longitudinal ventilation with jet fans |
| COVER AND TYPE: | Sand at shipping canal; other part not covered. |
| ADDITIONAL INFORMATION: | OWNER: Municipality of Amsterdam<br>ENGINEERS: Consortium of Grabowsky & Poort, D3BN and Jongen forming SAT Engineering v.o.f.<br>CONTRACTOR: Combinatie Piet Hein Tunnel, consortium of C.F.E., Stukton de Meijer, Dredging International, Besix, van Laere Aannemingen. De Werger consultant as subcontractor for the engineering<br>(Design - Construct) |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**MTRC Completed Tunnels; Hong Kong, 1997** | | | **T.97 - MTRC**<br> |
| TUNNEL TYPE AND USE:<br>Concrete Box, railway | | LANES/TRACKS:<br>Two tubes, one track each | |
| NO OF ELEMENTS: 10 | LENGTH: 126 m | HEIGHT: 7.57 m | WIDTH: 12.42 m |
| TOTAL IMMERSED LENGTH: 1260 m | | DEPTH AT BOTTOM OF STRUCTURE: 28 m | |
| UNUSUAL FEATURES: | Curved to radius of 850 m; longitudinally post-tensioned | | |
| ENVIRONMENTAL CONDITIONS: | Sheltered channel 1 m/s tidal current. | | |
| FABRICATION METHOD:<br>Precast in dry dock casting basin. One element fabricated with batch 3 of Western Harbour Crossing, remainder in adjacent basin. | | JOINT TYPE: Gina gasket hydrostatic joints with secondary Omega seal. Offshore closure made with final element. | |
| WATERPROOFING METHOD: | Steel plate on underside, spray-on membrane on sides and roof | | |
| PLACEMENT METHOD: | Floated, immersed from sinking pontoons | | |
| FOUNDATION METHOD: | Sand bedding injected via pipes cast into central wall | | |
| DREDGING METHOD: | Bucket grab dredger, blasting in completely decomposed granite | | |
| VENTILATION TYPE: | Longitudinal | | |
| COVER AND TYPE: | Reclamation at northern landfall, rock armouring generally | | |
| ADDITIONAL INFORMATION: | CLIENT: Mass Transit Railway Corporation, Hong Kong<br>DESIGNER: Acer Consultants F. E. Ltd.<br>CONTRACTOR: Kumagai Tarmac Joint Venture | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Western Harbour Crossing; Hong Kong; 1997** | | | **T.98 - Western Crossing**<br> |
| TUNNEL TYPE AND USE:<br>Concrete box tunnel, vehicular | | LANES/TRACKS:<br>Two tubes; three lanes each tube | |
| NO OF ELEMENTS: 12 | LENGTH: 113.5 m | HEIGHT: 8.57 m | WIDTH: 33.4 m |
| TOTAL IMMERSED LENGTH: 1363.5 m | | DEPTH AT BOTTOM OF STRUCTURE: 25.3 m | |
| UNUSUAL FEATURES: | Element 1 is tapered for 19.25 m; the first segment of Element 12 is placed with Element 11 so that the final joint lies inside Element 12; final joint dewatered and cast in the dry; one element of MTRC tunnel included in Batch 3. PFA replaced 35% of cement. Very low water/cement ratio. Cooling pipes used. | | |
| ENVIRONMENTAL CONDITIONS: | Sheltered passage, 1 m/s tidal current. | | |
| FABRICATION METHOD:<br>Graving basin in unused part of Shek-O Quarry, ocean tow to mooring. Sheltered harbour tow to final location. | | JOINT TYPE: Flexible Gina/Omega-type joints, steel shear keys between elements. | |
| WATERPROOFING METHOD: | Bottom plate is ribbed plastic, sides and top have tough spray-on membrane, without protection to sides. | | |
| PLACEMENT METHOD: | Lowered from two transverse barges initially resting on top surface of tunnel element. | | |
| FOUNDATION METHOD: | Initial support on previous section of tunnel, free end supported on two hydraulically-operated temporary supports while sand-flow foundation was prepared utilizing pipes installed in the walls from the roof slab to the base slab. | | |
| DREDGING METHOD: | Clamshell, with material taken to offshore disposal sites selected by HK Government Fill Management Committee. | | |
| VENTILATION TYPE: | Semi-transverse, using air ducts at outside wall. | | |
| COVER AND TYPE: | Concrete protection to roof waterproofing; general backfill covered with rock protection, total 1.5 m thick; rock dikes for anchor release close to tunnel. | | |
| ADDITIONAL INFORMATION: | CLIENT: (Franchisee) Western Harbour Tunnel Co. Ltd., ultimate owner HK Government<br>DESIGNER: Joint venture of Acer Consultants and Parsons Brinckerhoff on behalf of the Maunsell Acer Parsons Brinckerhoff Joint Venture<br>DESIGN CHECKER: Scott Wilson Kirkpatrick & Partners<br>CONTRACTOR: Kumagai Gumi on behalf of the Nishimatsu-Kumagai Joint Venture | | |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Aquaduct, Alphen aan de Rijn; The Netherlands; 1997** | | | **T.99 - Aquaduct**<br> |
| TUNNEL TYPE AND USE:<br>Vehicular | | LANES/TRACKS:<br>One tube with four lanes (two each way) | |
| NO OF ELEMENTS: 1 | LENGTH: 41 m | HEIGHT: 8.5 m | WIDTH: 21.35 m |
| TOTAL IMMERSED LENGTH: 41 m | | DEPTH AT BOTTOM OF STRUCTURE: 12.55 m | |
| UNUSUAL FEATURES: | Permanently prestressed in longitudinal direction | | |
| ENVIRONMENTAL CONDITIONS: | | | |
| FABRICATION METHOD:<br>The element is built in a dock in one of the open approaches. The total project, including the open approaches, is 1100 m long. The expected construction period of the total project is 80 weeks. | | JOINT TYPE: Inflatable rubber as first seal and Omega as second at both sides. | |
| WATERPROOFING METHOD: | Cooled concrete. | | |
| PLACEMENT METHOD: | Only ballast tanks. No laybarge because stability during immersion is guaranteed by both portal connections which cross the waterline. | | |
| FOUNDATION METHOD: | The element is placed as a bridge on both entrances. | | |
| DREDGING METHOD: | Excavation between underwater sheetpile wall. | | |
| VENTILATION TYPE: | No special ventilation system. | | |
| COVER AND TYPE: | None | | |
| ADDITIONAL INFORMATION: | OWNER: Rijkswaterstaat<br>DESIGNER: Rijkswaterstaat<br>CONTRACTOR: Hollandsche Beton en Waterbouw | | |

<table>
<tr><td colspan="3">TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>Niigata Port Road Tunnel; Niigata, Japan; U/C</td><td>T.100 - Niigata Port Road<br></td></tr>
<tr><td colspan="2">TUNNEL TYPE AND USE:<br>Prestressed concrete box elements; Vehicular</td><td colspan="2">LANES/TRACKS:<br>Two tubes; two lanes each tube</td></tr>
<tr><td>NO OF ELEMENTS: 8</td><td>LENGTH: 105-107.5 m</td><td>HEIGHT: 7.64 m</td><td>WIDTH: 28.6 m</td></tr>
<tr><td colspan="2">TOTAL IMMERSED LENGTH: 850 m</td><td colspan="2">DEPTH AT BOTTOM OF STRUCTURE: -23 m</td></tr>
<tr><td>UNUSUAL FEATURES:</td><td colspan="3">Constructed in earthquake zone. Ambient ground is soft sand with potential for liquefaction. Measures were taken to prevent liquefaction.</td></tr>
<tr><td>ENVIRONMENTAL CONDITIONS:</td><td colspan="3">At the mouth of a river with high velocity currents. Heavy snows during the winter affected land operations.</td></tr>
<tr><td colspan="2">FABRICATION METHOD:<br>Fabricated in dry dock.</td><td>OUTFITTING:</td><td>JOINT TYPE:<br>Flexible joint used Gina-type gasket and post-tensioned cables across joint.</td></tr>
<tr><td>WATERPROOFING METHOD:</td><td colspan="3">Steel plate around the element with upper slab covered with rubberized waterproofing layer.</td></tr>
<tr><td>PLACEMENT METHOD:</td><td colspan="3">Element supported by pontoons. Ballasting by internal water tanks.</td></tr>
<tr><td>FOUNDATION METHOD:</td><td colspan="3">Non-segregating cement grout was pumped under the elements.</td></tr>
<tr><td>DREDGING METHOD:</td><td colspan="3">Grab bucket and cutterhead suction dredging.</td></tr>
<tr><td>VENTILATION TYPE:</td><td colspan="3">Longitudinal ventilation.</td></tr>
<tr><td>COVER AND TYPE:</td><td colspan="3">1.5 m of backfill with concrete layer on top of element.</td></tr>
<tr><td>ADDITIONAL INFORMATION:</td><td colspan="3"></td></tr>
</table>

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.101 - Kawasaki Fairway |
|---|---|
| Kawasaki Fairway Tunnel; Tokyo, Japan; U/C |  |

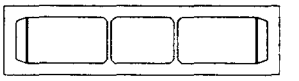
| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Reinforced concrete box prestressed longitudinally for watertightness; Vehicular | Two tubes; three lanes each. Escape passageways and utility space for gas, electric and drainage lines in the side tubes. |

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 9 | LENGTH: 131.2 m | HEIGHT: 10.0 m | WIDTH: 39.7 m |
| TOTAL IMMERSED LENGTH: 1,180.9 m | ELEMENT DISPLACEMENT WEIGHT: 52,000 MT | | DEPTH AT BOTTOM OF STRUCTURE: 26 m |

| | |
|---|---|
| UNUSUAL FEATURES: | Flexible earthquake joints were provided between all the tunnel elements. Piles were driven in casting basin bottom after it was stabilized with soil cement. Liquid nitrogen was used to cool sand during summer concreting. Blast furnace cement was used to reduce thermal cracking. Part of same roadway as Tama River Tunnel |
| ENVIRONMENTAL CONDITIONS: | At the mouth of Daishi Canal and entrance to Kawasaki Port. |

| FABRICATION METHOD: | JOINT TYPE: |
|---|---|
| The basin accommodated 11 elements at a time and had a floor area of 107,000 $m^2$ (592 m x 190 m). Basin was used jointly with Tama River Tunnel. | Gina/Omega type joint. Flexibility is provided by the rubber joint, in combination with the use of post-tensioned rods tightened across the joint. Three vertical shear keys provided in the walls and one horizontal key in the floor provide stability against displacements. |

| | |
|---|---|
| WATERPROOFING METHOD: | 8 mm steel plate on sides and bottom of elements; concrete protected by 2.5 mm thick rubberized membrane on top slab. Steel plates are cathodically protected. |
| PLACEMENT METHOD: | Catamaran barges. 500 MT during lowering (1%) and 1500 MT in place (3%) prior to placing final ballast (1.1 SF against flotation, not counting backfill). |
| FOUNDATION METHOD: | Mortar pumped over gravel foundation bed. |
| VENTILATION TYPE: | Longitudinal ventilation. |
| COVER AND TYPE: | 1.5 m of stone cover |
| ADDITIONAL INFORMATION: | Mix proportions for tunnel element concrete: w/c 0.515; water 155 kg/$m^3$; cement 301 kg/$m^3$; sand 834 kg/$m^3$ and gravel 1034 kg/$m^3$.<br><br>OWNER: Metropolitan Expressway Public Corporation<br>DESIGNER: Oriental Consultants Co, Ltd; Pacific Corporation Co Ltd<br>CONTRACTOR: Maeda Construction Co Ltd; Hazama Gumi Ltd; Tobishima Corporation; Penta-Ocean Construction Co Ltd; Sato Koyo Co Ltd; Taisei Construction Co Ltd. |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.102 - Osaka South Port |
|---|---|
| **Osaka South Port Tunnel; Osaka, Japan; U/C** |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Rectangular steel and concrete composite box section; Vehicular and railway | Two tubes each with two lanes for vehicles. One tube with two tracks for railway. |

| | | | |
|---|---|---|---|
| NO OF ELEMENTS: 10 | LENGTH: 100 - 105 m | HEIGHT: 8.60 m | WIDTH: 35.20 m |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: 1,025 m | DEPTH AT BOTTOM OF STRUCTURE: 27 m approx. |

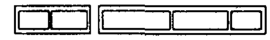
| | |
|---|---|
| UNUSUAL FEATURES: | First immersed tube tunnel in Japan for both roadway and railway use. Composite structure employed in the design of rectangular box section. Section was modelled and load tested. Flexible joint design for earthquake loads. Steel caisson used for ventilation towers to which element make underwater connection. Connection to man-made island at Osaka port. A special "V-block" type of final joint was used. This final closure consists of a special prefabricated wedge which slips between the opposing elements (with sloping faces). This wedge contains a flexible joint with sheer keys. The wedge faces are sealed by hydrostatic pressure, then welded to the elements and finally concreted. |
| ENVIRONMENTAL CONDITIONS: | Busy port environment. Sea bottom is soft clay and settlements are anticipated. |

| FABRICATION METHOD: | JOINT TYPE: |
|---|---|
| Steel shell and concrete composite constructed in ship-building dock. | GINA gasketted joint. Post-tensioned cables across joint. Vertical and horizontal shear keys provided at all walls and slabs. Designed for seismic loads, ground settlement and thermal stresses. |

| | |
|---|---|
| WATERPROOFING METHOD: | Steel plate all around |
| PLACEMENT METHOD: | Supported by pontoons. Ballasting with water into interior ballast tanks. |
| FOUNDATION METHOD: | Non-segregating cement grout 0.5 m thick pumped under element over crushed stone. |
| DREDGING METHOD: | Grab bucket and cutterhead suction dredging. |
| VENTILATION TYPE: | Piston action for rail lines. Longitudinal for roadway spaces. |
| COVER AND TYPE: | 1.5 m of backfill over 0.1 m of concrete protection. |
| ADDITIONAL INFORMATION: | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: Aktion-Preveza Tunnel; Preveza, Greece; U/C | | | T.103 - Aktion-Preveza |
|---|---|---|---|
| TUNNEL TYPE AND USE: Reinforced concrete box sections; Vehicular | | LANES/TRACKS: One tube with two lanes | |
| NO OF ELEMENTS: 8 | LENGTH: 4 - 134.5 m<br>2 - 119 m<br>1 - 75 m<br>1 - 49 m | HEIGHT: 8.55 m | WIDTH: 12.60 m |
| TOTAL IMMERSED LENGTH: 900 m | | DEPTH AT BOTTOM OF STRUCTURE: 26.5 m | |
| UNUSUAL FEATURES: | Earthquake zone | | |
| ENVIRONMENTAL CONDITIONS: | Earthquake loadings required strengthening of subsoil by means of stone columns. | | |
| FABRICATION METHOD: The elements were constructed in a casting basin placed in the alignment of the southern approach. | | JOINT TYPE: Gina/Omega type joint in combination with post-tensioning rods across the joint. | |
| WATERPROOFING METHOD: | HDPE sheet membrane to bottom and sprayed on modified Polyurethane membrane to roof and sides. | | |
| PLACEMENT METHOD: | Catamaran type sinking rig and use of waterballast | | |
| FOUNDATION METHOD: | Mortar pumped over gravel foundation bed. | | |
| DREDGING METHOD: | Grab and suction dredging | | |
| VENTILATION TYPE: | Longitudinal ventilation | | |
| COVER AND TYPE: | 0.75 m rock protection | | |
| ADDITIONAL INFORMATION: | OWNER: Ministry of Environmental Planning and Public Works<br>ENGINEER:<br>CONTRACTOR: Christiani & Nielsen Ltd and Technical Company of General Construction S.A. | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED:<br><br>**River Lee Tunnel; Cork, Ireland; U/C** | | | **T.104 - River Lee** |
|---|---|---|---|
| TUNNEL TYPE AND USE:<br>Reinforced concrete box sections; Vehicular | | LANES/TRACKS:<br>Two tubes with two lanes each | |
| NO OF ELEMENTS: 5 and one boat element | LENGTH: 122 m | HEIGHT: 8.4 m | WIDTH: 23.8 m |
| TOTAL IMMERSED LENGTH: 610 m of tunnel<br>120 m boat element | | DEPTH AT BOTTOM OF STRUCTURE: 20 m | |
| UNUSUAL FEATURES: | Cast in south open approach structure. One approach ramp (north end) constructed in casting basin and floated into position. | | |
| ENVIRONMENTAL CONDITIONS: | Sensitive scenic area downstream of Port of Cork. Internationally important site for some species of wading birds. | | |
| FABRICATION METHOD: Cast in segments and temporarily prestressed | | JOINT TYPE: Gina and Omega | |
| WATERPROOFING METHOD: | No separate membrane. Cooling of concrete adopted to control early age stresses and ensure watertightness. | | |
| PLACEMENT METHOD: | Elements winched to position from casting basin ballasted and lowered onto temporary footings. | | |
| FOUNDATION METHOD: | Sandflow | | |
| DREDGING METHOD: | Backhoe generally. Cutterhead suction for underlying sands and gravels. | | |
| VENTILATION TYPE: | Longitudinal | | |
| COVER AND TYPE: | 1 m thick layer of rock protection | | |
| ADDITIONAL INFORMATION: | OWNER: Cork Corporation<br>DESIGNER: Owner's engineer was Ewbank Preece O hEocha in association with Symonds Travers Morgan. Contract is design and build. Contractor's designer is Mott MacDonald.<br>CONTRACTOR: J.V. of Tarmac Construction and (Irish) P J Walls | | |

| TUNNEL NAME/LOCATION/DATE COMPLETED: | T.105 - Fort Point Channel |
|---|---|
| Fort Point Channel Tunnel; Boston, Massachusetts, U.S.A.; U/C |  |

| TUNNEL TYPE AND USE: | LANES/TRACKS: |
|---|---|
| Two rectangular reinforced concrete box tunnels side by side; Vehicular | East Bd. Tunnel: 3 Tubes: 5 Lanes + Transitions<br>West Bd. Tunnel: 2 Tubes: 4 Lanes + Transitions |

| NO OF ELEMENTS: 6 | LENGTH: | HEIGHT: | WIDTH: |
|---|---|---|---|
| | 99 m minimum<br>127 m maximum | 7.9 m | 21.34 m minimum<br>47.25 m maximum |

| | |
|---|---|
| TOTAL IMMERSED LENGTH: Two separate tunnels approximately 330 m long | DEPTH AT BOTTOM OF STRUCTURE: 17.7 m |

| | |
|---|---|
| UNUSUAL FEATURES: | Two parallel immersed tunnels with only 1.2 m separation between them. The tunnels are both to cross over two existing operating subway tunnels. The space in between the I-90 Tunnels and the Red Line Subway tunnels will be only 1.5 meters. The elements will be placed in alternating sequence in pairs starting on the west end and ending on the east end. The west end of both tunnels will be provided with large, 7.3 m tall "snorkles" which will form the foundations for the vent building which will straddle the tunnels. |
| ENVIRONMENTAL CONDITIONS: | Crosses historic channel no longer used for navigation. Mild currents produced by tidal action and storm water runoffs. Tide range quite large: 4 m. Both shores heavily congested with industrial activities. Existing bridges restrict access to site in width and height. |

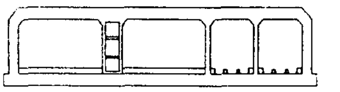
| | | |
|---|---|---|
| FABRICATION METHOD: The six elements will be constructed in the east approach excavation in three cycles of two elements each. | OUTFITTING: Installation of temporary cofferdam walls will be made on top of last two elements to permit final dewatering for connection of cut-and-cover tunnels on east end. | JOINT TYPE: Gina and Omega gaskets except on east end where cut-and-cover connection will be made to element in the dry. |

| | |
|---|---|
| WATERPROOFING METHOD: | Spray membrane using acrylic or polyurethane resins on sides and top. Ribbed polyethylene sheet on bottom (current design). |
| PLACEMENT METHOD: | Conventional pontoon method of placement is precluded due to insufficient water depth. Placement will most likely be done using a system of cables and winches operated from the shore areas surrounding the tunnels on three sides. Temporary lowering towers constructed out in the channels may be also used for lowering and positioning each element. |
| FOUNDATION METHOD: | Options will be provided in the contract for either using a screeded bed or a pumped sand foundation. |
| DREDGING METHOD: | Clamshell dredge into barges most likely method. |
| VENTILATION TYPE: | Fully transverse ventilation from side ducts incorporated into each tube. |
| COVER AND TYPE: | There is no vertical room available for cover except for a concrete protective layer on top and locking fill on the sides of each tunnel. |
| ADDITIONAL INFORMATION: | OWNER: Massachusetts Highway Department<br>MANAGEMENT CONSULTANTS: Bechtel/Parsons Brinckerhoff<br>SECTION DESIGN CONSULTANTS: Gannett Fleming, Inc. ACER Engineers & Consultants, Inc.<br>CONTRACTOR: Immersed tunnel contract not out for tender as yet. |

**TUNNEL NAME/LOCATION/DATE COMPLETED:**

**Drogden Tunnel; Oresund, Denmark; U/C**

**T.106 - Drogden**



TUNNEL TYPE AND USE:
Reinforced concrete box sections; Vehicular and rail

LANES/TRACKS:
Two tubes with two lanes each
Two tubes with one track each
Central services/escape gallery between road tubes.

| NO OF ELEMENTS: 20 | LENGTH: 175.2 m | HEIGHT: 8.50 m | WIDTH: 42 m |
|---|---|---|---|

TOTAL IMMERSED LENGTH: 3,510 m

DEPTH AT BOTTOM OF STRUCTURE: 22 m approx.

| | |
|---|---|
| UNUSUAL FEATURES: | The 22 meter tunnel segments will be cast in a temperature controlled factory in a single 24 hour long pour to prevent differential cracking. After casting sements will be jacked forward similar to incrementally launched bridges. After eight segments are completed and given 10 days of cure under cover, the element will be pushed into a lowering basin for completion and launched. The temporary lowering basin works similar to a canal lock system. |
| ENVIRONMENTAL CONDITIONS: | The flow through the Drogden Channel must not be reduced. Consequently compensatory dredging will be adjusted after the tunnel is in place and backfilled and the flow is measured. |

FABRICATION METHOD:
The 178 m long elements are cast in segments of 22 m in a temporary production yard in the Copenhagen North Harbour by use of incremental launch method.

JOINT TYPE:
Gina and Omega gaskets.

OUTFITTING:
Outfitting with pontoons took place in the canal lock.

| | |
|---|---|
| WATERPROOFING METHOD: | No separate membrane is provided as concrete is designed to be watertight. Early age stresses are controlled by choice of casting sequence and environment, hence obviating the need to artificially cool the concrete. |
| PLACEMENT METHOD: | Pontoons are used to lower the elements onto the screeded gravel bed. |
| FOUNDATION METHOD: | Gravel bed placed to level without large scale screeding. |
| DREDGING METHOD: | Separate contract from tunnel construction. |
| VENTILATION TYPE: | Longitudinal jet fans. |
| COVER AND TYPE: | |
| ADDITIONAL INFORMATION: | Owner: Øresundkonsortiet, a company set up to build and operate the link, and owned 50/50 by the Danish and Swedish governments.<br>Owner's Consultant: Øresund Link Consultants, a joint venture of TEC (Holland), Halcrow (UK) and others.<br>Contractor: Øresund Tunnel Contractors (OTC), a joint venture of NCC (Sweden), Boskalis (Holland), Dumez-GTM (France), John Laing (UK) and Pihl and Son (Denmark). Design and build contract.<br>Contractor's designer: Symonds Travers Morgan. |

| | |
|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:** **Tokyo Port Seaside Road Tunnel (provisional name); Tokyo, Japan; U/C (1999)** | **T.107 - Tokyo Port Road** |
| TUNNEL TYPE AND USE: Reinforced concrete box prestressed longitudinally for watertightness; Vehicular | LANES/TRACKS: Two tubes; two lanes each. Escape passageways and utility space for telecommunication cables. Power cables and water supply in the side ducts. |
| NO OF ELEMENTS: 11 | LENGTH: 10 - 120.0 m<br>1 - 125.2 m |
| HEIGHT: 10.0 m | WIDTH: 32.2 m |
| TOTAL IMMERSED LENGTH: 1328.8 m | DEPTH AT BOTTOM OF STRUCTURE: 29.2 m |
| UNUSUAL FEATURES: | Flexible joints (earthquake) are to be provided. A steel-shell pneumatic caisson will be used for the vertical shaft. The tunnel is curved in horizontal alignment with R=300 m, L=420 m. The terminal block method is to be used on the final joint. |
| ENVIRONMENTAL CONDITIONS: | Crossing under the Tokyo Port No. 1 West Channel with heavy marine traffic. |
| FABRICATION METHOD: | 11 elements are cast in one cycle at the Oi graving dock. This graving dock is 604 m long and 190 m wide. |
| OUTFITTING: | Two alignment towers, bollards, an access hole and ballast tanks are to be provided. |
| JOINT TYPE: | Considering earthquake forces, all elements will be joined with rubber gaskets, and posttensioning rods will be tightened across each joint. Three shear keys in the vertical walls and a shear key in the base slab will be provided. |
| WATERPROOFING METHOD: | 8-mm steel plate on sides and bottom of elements: a 2.5 mm thick rubber membrane on the top slab is provided and protected with concrete. |
| PLACEMENT METHOD: | Elements to be towed from Oi dry dock by placing barge. The elements are to be immersed, drawn together and joined by jacks after adjusting the horizontal height using vertical jacks at vertical shear keys on the inboard end and on jacking pads at the outboard end. |
| FOUNDATION METHOD: | Bentonite mortar is grouted between the gravel foundation bed and the base slab. Each grout pipe will feed 120 locations under the element. 1 m of foundation gravel was used. |
| DREDGING METHOD: | A grab bucket dredger was employed. |
| VENTILATION TYPE: | Longitudinal ventilation. |
| COVER AND TYPE: | 1.5 m of crushed hard sandstone cover. |
| ADDITIONAL INFORMATION: | OWNER: Bureau of Port and Harbor. Tokyo Metropolitan Government |

| | | | |
|---|---|---|---|
| **TUNNEL NAME/LOCATION/DATE COMPLETED:**<br><br>**Kobe Port Minatojima Tunnel, Hyogo, Japan; U/C** | | | **T.108 - Kobe Port Minatojima**<br> |
| TUNNEL TYPE AND USE:<br>Rectangular, steel-concrete composite; Vehicular | | LANES/TRACKS:<br>Two tubes, 3 lanes each tube. | |
| NO OF ELEMENTS: 6 | LENGTH: 88 m | HEIGHT: 9.1 m | WIDTH: 34.4 m |
| TOTAL IMMERSED LENGTH: 520 m | | DEPTH AT BOTTOM OF STRUCTURE: 22.6 m | |
| UNUSUAL FEATURES: | Five of the six sections will use steel-concrete-steel (SCS) designs. This will eliminate the need for traveller type formwork. Bids had just been opened on these first sections when this was written. | | |
| ENVIRONMENTAL CONDITIONS: | Sea bottom is very soft clay. Seismically active area (Hannshinn Earthquake 1995). "V-block" type of final joing was used (see description for T.102). | | |
| FABRICATION METHOD:<br>Framing fabricated in a shop and assembled in a dry dock. Concrete placed in the dry dock. High fluidity concrete was pumped into SCS sections. | | JOINT TYPE: Flexible joint with Gina gaskets and post-tension cables across joints. | |
| WATERPROOFING METHOD: | Steel shell. | | |
| PLACEMENT METHOD: | Supported by pontoons. Ballasting with water into interior ballast tanks. | | |
| FOUNDATION METHOD: | Non-segregating cement grout pumped under element. | | |
| DREDGING METHOD: | Grab bucket and suction. | | |
| VENTILATION TYPE: | Longitudinal ventilation using jet fans. | | |
| COVER AND TYPE: | 1.5 m of backfill over 0.1 m concrete protection. | | |
| ADDITIONAL INFORMATION: | | | |

PII:S0886-7798(97)00014-X

# Chapter 10

# SUBMERGED FLOATING TUNNELS—
## A CONCEPT WHOSE TIME HAS ARRIVED

by

**Donna Ahrens** **U.S.A.**
***Tunnelling and Underground Space Technology***

***Technical editing:***

**Ahmet Gursoy** **U.S.A.**
**Parsons Brinckerhoff International**

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.317–336, 1997

Printed in Great Britain
0886–7798/97 $17.00+0.00

Pergamon

# Chapter 10: Submerged Floating Tunnels—
## A Concept Whose Time Has Arrived


**1. INTRODUCTION**
1.1 The ITA and Submerged Floating Tunnels
1.2 Scope of the Chapter

**2. CURRENT STATUS OF SFT PROJECTS: Designs Currently Under Study or Consideration**
2.1 Høgsfjord Crossing (Norway)
2.2 Strait of Messina Crossing (Italy)
2.3 Proposed SFTs for Japan—Funka Bay, Uchiura Bay, and Osaka Bay Crossings
2.4 Lake Lugano Crossing (Italy)

**3. SUBMERGED FLOATING TUNNELS: TECHNICAL STATE-OF-THE-ART**
3.1 Concept Description and Conditions Favorable to SFT Construction
3.2 Feasibility of Submerged Floating Tunnels
3.3 Costs
3.4 Environment and Possible Locations
3.5 Structural Principles
3.6 Static and Dynamic Analyses
3.7 Risk Analysis
3.8 Instrumentation
3.9 Durability and Maintenance
3.10 Construction Methods
3.11 Construction Time
3.12 Market Expectation

**4. REFERENCES**

## 1. Introduction

Although the concept of a submerged floating tunnel (SFT) has been in existence for a quarter of a century and offers the possibility of creating crossings never before thought possible, no country has yet approved construction of this type of structure. But if enterprising tunnel engineers have their way, SFT theory may be translated into practice by the turn of the century.

The SFT represents a new concept for crossing deep waters: unlike conventional immersed tube tunnels, an SFT is not an embedded structure, but instead is suspended above the sea floor, anchored by a support system such as pontoons on the surface or by anchoring to the water bed.

Submerged floating tunnels can provide a more economical means of crossing a body of water than a suspension bridge or undersea tunnel, depending on the local sea and hydrographic conditions.

Interest in this relatively new field of tunnelling has grown dramatically over the past decade, as witnessed by attendance at the first international conference devoted solely to the topic of submerged floating tunnels. The conference, sponsored by the Norwegian Public Roads Administration, was held in Sandnes, Norway, in June 1996. Initially intended as a small seminar of perhaps 40 invited experts, the conference attendance swelled to nearly 100, as engineers worldwide caught wind of the opportunity to hear about the latest research and design work being done on submerged floating tunnels in several countries.

In fact, studies of SFTs are being promoted aggressively in a number of countries, where considerable research over the past five years has been devoted to developing and refining the SFT concept.

### 1.1 The ITA and Submerged Floating Tunnels

In 1989, the International Tunnelling Association (ITA) established a Working Group on Immersed and Floating Tunnels in order to give more worldwide attention to this type of tunnelling. The initial task of the Working Group was to present the state of the art for two types of tunnels: (1) *Immersed or Submersed Tunnels* and (2) *Submerged Floating Tunnels* (SFTs).

Because both types of tunnels involve many different aspects, a number of sub-working groups were established. Each subgroup, including one on submerged floating tunnels, was composed of specialists in the particular field. The six chapters in the 1993 edition of this report reflected the different subgroups represented at that time.

As work on the second edition of the report progressed, different subgroups again were established to deal with other aspects of immersed tunnelling.

Since publication of the 1993 state-of-the-art report, in a number of countries (especially Norway, Italy, Japan, and the Netherlands), extensive research was being performed on specific tunnel schemes for crossing deep and, usually, narrow waterways. The solution—a suspended tunnel structure—has passed the preliminary and conceptual design stage, supported by detailed research and hydraulic model tests. It is expected that an actual project employing this new type of tunnel will be realized in the near future.

Indeed, a research programme for the conceptual development of Submerged Floating Tunnels (SFTs) was recently approved by the European Commission. With EU funding, a study group representing the Forum of European National Highway Research Laboratories is currently assessing prospects for an EU-sponsored SFT project. A condensed version of this group's July 1996 report on its ongoing study of SFTs is presented in Appendix A.

In response to these continuing developments in the study of submerged floating tunnels, the Norwegian delegation to the Working Group requested that the ITA Executive Council to establish a permanent subgroup devoted to the study of submerged floating tunnels. This request was granted in June 1996. With the completion of this edition of the ITA state-of-the-art report, the SFT subgroup can now concentrate its efforts on technical advances and development of the SFT concept.

### 1.2 Scope of the Chapter

This chapter defines the current state-of-the-art for submerged floating tunnels. Section 2 describes the status of the following SFT projects, which are the subject of ongoing research:

- Norway's Høgsfjord crossing.
- A crossing for the Messina Strait between mainland Italy and the island of Sicily.
- Three Japanese SFT proposals: Funka Bay crossing, Uchiura Bay crossing, and Osaka Bay crossing.
- An SFT crossing of Lake Lugano, on the Swiss/Italian border.

Section 3 combines an edited version of the 1993 state-of-the-art report, published in *T&UST* 8:2 (ITA 1993) with portions of a recent report on immersed tunnels, published in *T&UST* 11:4 (Ahrens 1996). This section describes the SFT concept, conditions favorable to SFT construction, technical aspects of SFT technology, and opportunities that this technology can provide.

## 2. Current Status of SFT Projects

### Designs Currently Under Study or Consideration

The development of the principle of an SFT from conception to a feasible concept has been achieved in less than two decades. In the context of the development of comparable complex structures in history, this is a short period. However, the brevity of the development period is offset by the fact that those involved have gained experience in comparable technologies in the civil, marine and offshore industries. Conventional design methods and existing technologies have been used and found sufficient to enable feasible SFT schemes to be developed.

Extensive studies have been carried out to assess the feasibility of an SFT under the Høgsfjord in Norway and at the Strait of Messina in Italy. Conceptual work has been done for other locations in Norway and for Lakes Como and Lecco in Italy, and general desk studies have been done in other parts of the world. The experience gained through these studies has provided increased confidence in the feasibility of a submerged floating tunnel.

This section briefly describes six proposed SFTs—one in Norway, two in Italy, and three in Japan—that have been the subject of considerable research in recent years. Although the studies are at different stages, the projects reflect the considerable gains in SFT research and development that have been made over the past decade.

### 2.1 Høgsfjord Crossing (Norway)

The first patents for SFTs were granted in Norway (in 1923 and 1947), but study of a possible fjord crossing by immersed tunnel did not begin in earnest until the mid-1980s. The Norwegian Public Roads Administration has chosen the Høgsfjord on the west coast of Norway as a pilot project for an SFT. The proposed tunnel will be in a remote fjord area of Rogaland County, where a fixed crossing between the villages of Lauvvik and Oanes is needed. The Høgsfjord SFT, which will be part of the Norwegian coastal road system (see Fig. 10-1), will make it possible for 13,000 people to travel to Stavanger and rest of the county without using ferries. An estimated 2,500 vehicles will use the tunnel daily.

The site for the proposed Norwegian SFT presents no serious problems with regard to geology, seabed morphology, or sea traffic.

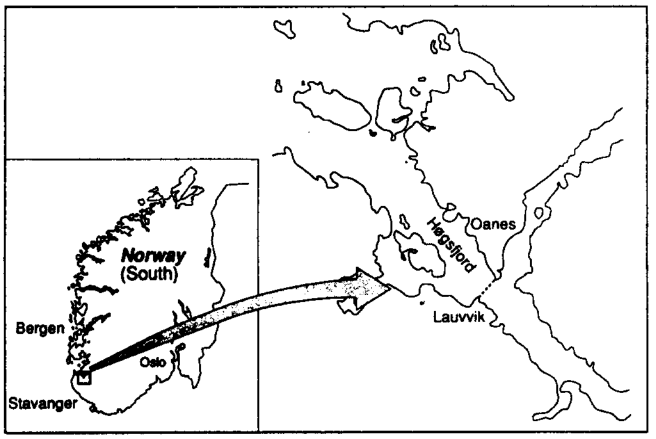



*Figure 10-1. Location of Norway's Høgsfjord, for which a submerged floating tunnel solution has been proposed.*

The Høgsfjord is 1400 m wide and 155 m deep, with an average current of 0.6 m/s and significant wave height of 1.5 m; water depth above the tunnel is approximately 25 m.

In 1987–88, four Norwegian construction firms that had participated in studies of the project submitted design concepts to the Norwegian Public Roads Administration, the State authority responsible for the project. From 1988–91, the Roads Authority developed specifications for tender documents.

After a hiatus of several years, work on the project continued in 1994, with the four pre-qualified construction firms (EEG-Henriksen Anlegg as, Kværner Rosenberg as, Selmer, and Aker Norwegian Contractors) providing new estimated construction costs. As a result, in the fall of 1995 the Ministry of Transport and Communications granted 14 million NOK (approx. $US2 million) to the development work deemed necessary in the following two years, so that tender documents can be prepared and invitations to tender sent out in 1997. Tender bids will be submitted in 1998, construction may begin in 1999 and the SFT open to traffic in 2001 or 2002.

Over the past decade, some of the most important areas of research for the Høgsfjord project have been:

- Site-specific measurement of environmental loads.
- The development of programmes for calculating dynamic forces and the behaviour of the structure.
- Hydraulic model testing of the dynamic behaviour of several concepts.
- Development of special instrumentation for monitoring dynamic behaviour.
- Material investigations for tethers.
- Preparation of specifications.

The four different tunnel concepts that have been proposed for the Høgsfjord crossing are based on concrete, steel, or a combination of these (see Fig. 12-2). The different solutions for vertical support range from tension legs to pontoons floating on the surface to towers standing on the sea bottom. The total length of the tube structure ranges from about 1500 to 1600 m, with a cross-section of about 10 m. All of the proposals incorporate fixed ends in rock.

The estimated cost of the SFT across Høgsfjord is 900 million NOK ($US128.5 million) (1996 prices); construction is expected to take about three years. It is possible that a toll payment scheme could help pay for the project; financial support from Rogaland County and the Norwegian State government are expected.

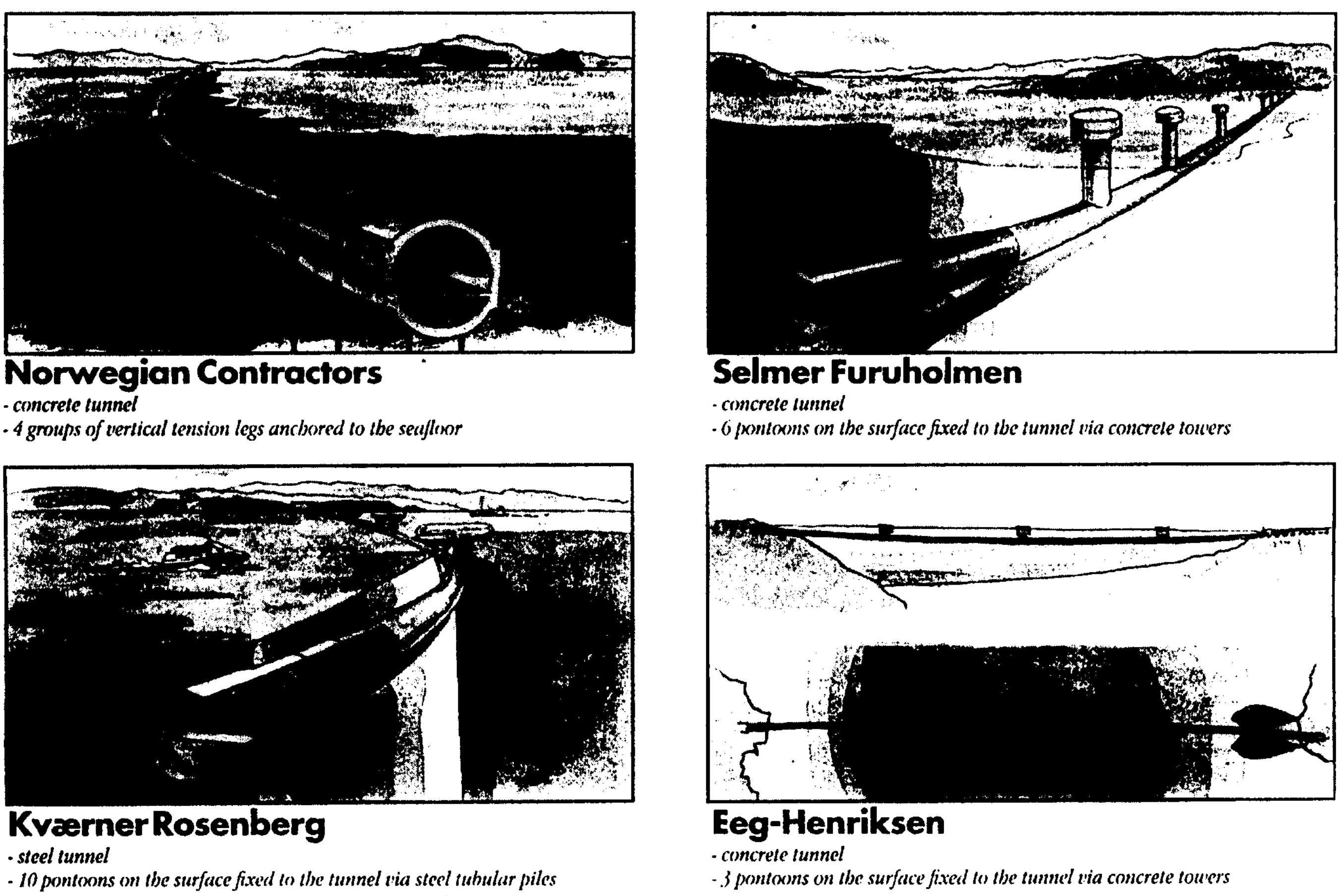



*Figure 10-2. SFT design concepts for the Høgsfjord crossing. Designs were submitted by four Norwegian contracts to the Norwegian Public Roads Administration.*

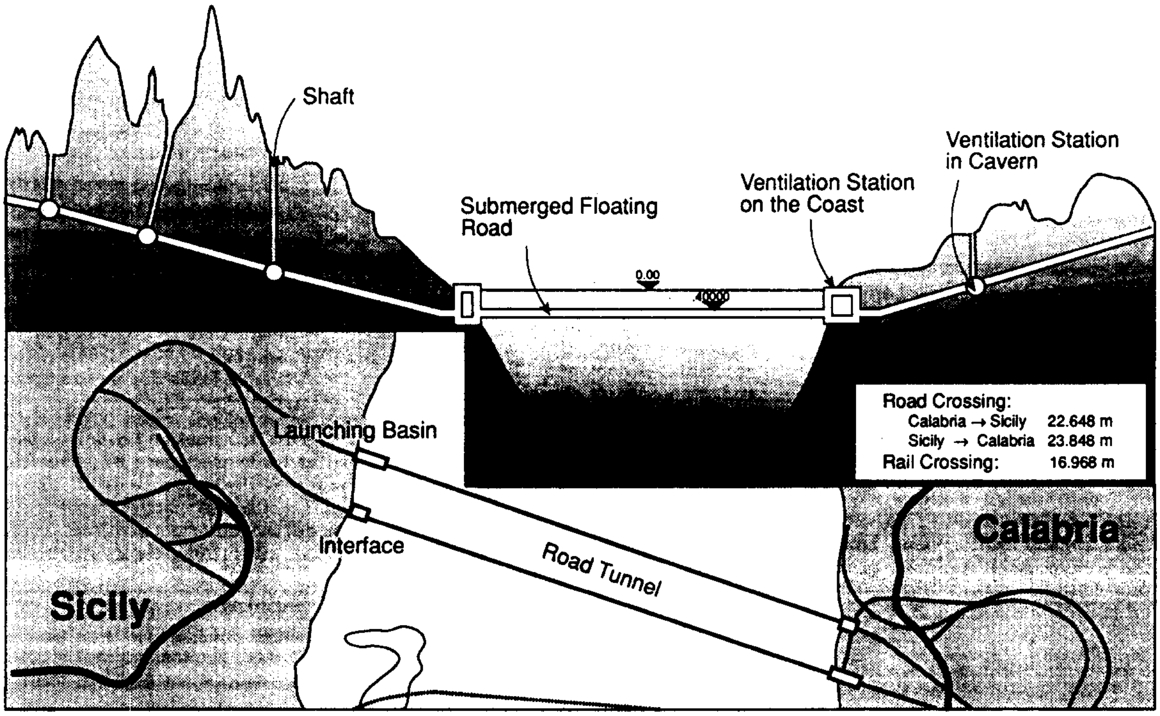



*Figure 10-3. Location of the Strait of Messina, showing the alignment of a proposed SFT.*

## 2.2 Strait of Messina Crossing (Italy)

Italy has had a long involvement in designing a fixed link crossing for the Messina Strait between the mainland and the island of Sicily. Studies have continued for 25 years, but realization of a connection has been hampered by two major obstacles:

- Severely limiting environmental conditions, and
- Congested traffic conditions.

Environmental conditions that design engineers must deal with include high winds that create strong currents and waves; the location of the crossing in an area of high seismicity; and highly faulted geology. Traffic in the waterway includes ferries, cargo ships, and fishing vessels that make 60,000 axial crossings per year.

Some relevant site conditions for the project are given below:

| | |
|---|---|
| Width of waterway: | 3000 m |
| Water depth: | 350 m |
| Currents: | 1 to 2 m/s, depending on load case. |
| Water depth above tunnel: | 35 m |
| Significant wave height: | 9 to 16 m, depending on load case |
| Amount of shipping traffic: | high |

Seismic loading is expected.

As a result of these limiting factors, there are only two possibilities for a permanent crossing: a suspended one-span bridge, or an SFT. [Final selection of the type of crossing has not been announced by the Italian government.] Extensive studies to explore the possibility of an SFT connection have been undertaken by Stretto di Messina S.p.A, the concessionaire company; Figure 10-3 shows a proposed route alignment.

An arrangement of three separate tunnel alignments (one for rail) placed 500 m apart is contemplated. The studies have shown that an SFT would be a viable solution, notwithstanding the harsh environmental and seismic conditions existing in the area.

The anchoring system comprises anchoring stations located along the tunnel at 72-m spacing. Each anchoring station is composed of a horseshoe connection with the tunnel, a piled anchor block and a cable in between, running 45 degrees to the vertical axis to provide horizontal and vertical stiffness. The use of both steel and kevlar as cable material is being investigated. This system will counterbalance the net buoyancy of the tunnel. The degree of buoyancy selected will keep the cables from receiving any slack when the tunnel experiences extreme environmental or seismic events.

In 1994, a project involving three tunnels—one for a railway connection and two as motorways (three lanes in each direction)—was presented to the Italian authorities by the consortium ENI Group (a joint venture of the Italian firms Saipem, Snamprogetti and Tecnomare). In addition to considering the very severe design conditions for environmental conditions and seabed morphology, the proposal evaluated and analyzed various potential attack scenarios and collisions. The proposed scheme was estimated to cost about US$2.5 billion (1991 prices).

In anticipation of government approval of an SFT project for the Messina Strait crossing, the Italian shipping industry has developed "Design Guidelines for the Certification of SFTs" as a reference tool for SFT design. The guidelines deal with structures, foundations, equipment (for both normal operations and emergency conditions), construction and installation, inspection, and repair and maintenance of the SFT structure. The guidelines are intended (1) to provide designers with all of the criteria that must be taken into account in creating a proposal for an SFT, and (2) to establish criteria that can be applied in recommending or rejecting a proposed SFT.

## 2.3 Proposed SFTs for Japan—Funka Bay, Uchiura Bay, and Osaka Bay Crossings

The Japanese also have shown considerable interest in SFTs. Several potential SFTs are under study:

- A Funka Bay crossing, connecting Hokkaido with Honshu islands;
- A crossing of Uchiura Bay at the Muroran fault; and
- A crossing of Osaka Bay, connecting Kobe Airport with Kansai International Airport.

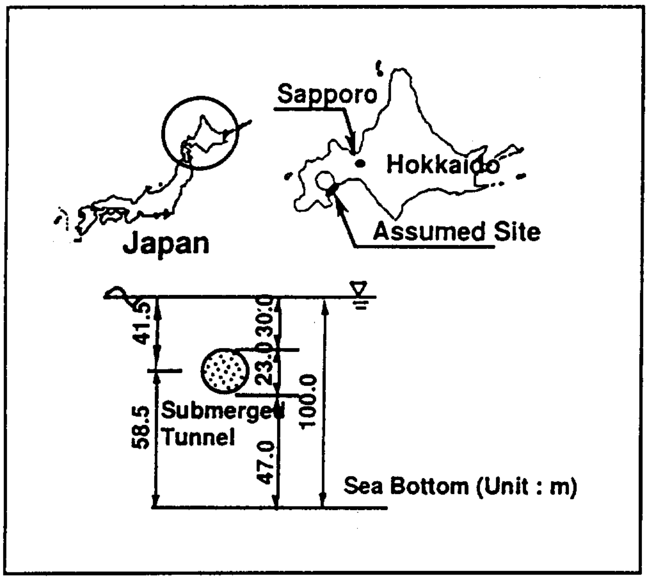



*Figure 10-4. Location of proposed SFT crossing for Uchiura Bay, Japan.*

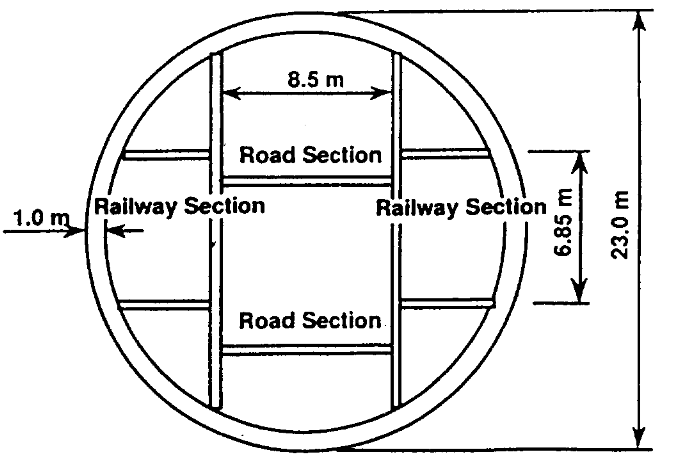



*Figure 10-5. Dimensions of the cross-section for proposed Uchiura Bay SFT crossing.*

Research for the first two crossings has been performed under the auspices of the Society of Submerged Floating Tunnel Technology Research in Hokkaido, which was inaugurated in 1990 by Hokkaido's industrial, government, and academic sectors as a public interest corporation. Since then, seven research subcommittees have been studying the establishment of comprehensive techniques for the planning, design, construction, and other aspects of SFTs.

Phase 1 of the research, a study of the overall concept of the SFT, has been completed; the results were published in 1995. The report summarized the results of five years of study for two possible SFT projects: one crossing Funka Bay, connecting Hokkaido with Honshu islands (two subcases were studied); and the other across Uchiura Bay (crossing the Muroran fault). Various mooring systems, foundations, and construction methods were analyzed for the projects.

Now that the Japanese study has entered Phase 2, researchers are conducting full-scale verification tests under sea conditions similar to those expected, in order to test watertightness, corrosion resistance, stability, and various potential construction methods. In addition, simulations are being conducted to measure drivers' attitudes toward and reactions to the experience of driving through an SFT.

### 2.3.1 Funka Bay crossing

This project involves a 30-km crossing of Funka Bay, which is 50–90 m deep, in southern Hokkaido, via a double-track railway. Such a crossing would strengthen access to Hokkaido from Honshu, and thus is seen as a favorable economic link. Hazard conditions, including design wave condition and design seismic acceleration, were studied for two types of anchoring systems. Displacement, acceleration, tension, and foundation issues were also examined in order to establish the best design system.

The construction costs are estimated at ¥2,000 billion (approximately ¥62 million per m) [$US17.7 billion and $US.55 million per m, respectively] (1996 prices), and the project would take about 14 years to complete.

### 2.3.2 Uchiura Bay crossing

For the proposed Uchiura Bay road/rail crossing on Hokkaido island (see Figs. 10-4 and 10-5), researchers at Hokkaido University in Sapporo have developed two modelling techniques to predict likely environmental impacts of an SFT. The first simulated changes in tidal patterns resulting from an SFT, and the second was a dynamic model of ecological growth (e.g., sea population densities for scallops and plankton).

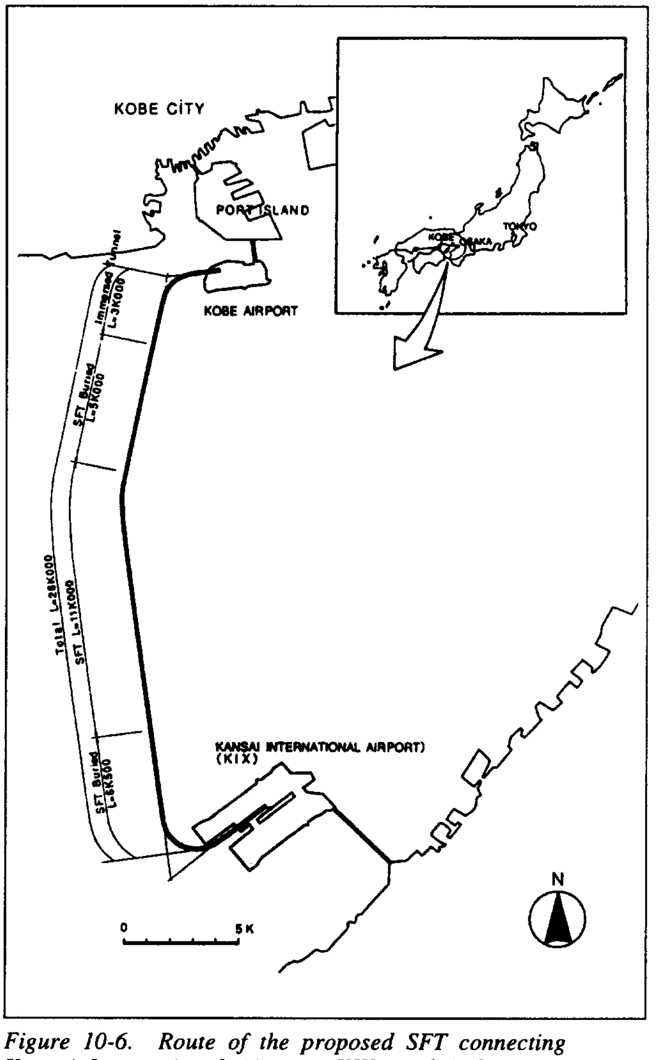



*Figure 10-6. Route of the proposed SFT connecting Kansai International Airport (KIX) and Kobe Airport.*

The main conclusions were that the influence of SFT construction on the environment would be slight because of the large-scale bay area. However, a large impact on the immediate surroundings of the tunnel was predicted. Research to determine the best way to control vibration caused by tidal waves is continuing.

### 2.3.3 Osaka Bay crossing

Another proposed project, connecting Kansai International Airport with Kobe Airport across Osaka Bay, is also under study. In the Kansai area, an SFT is being proposed that will connect the Kansai International Airport (KIX) with the Kobe Airport, construction of which is now being planned. (KIX, which is located on a newly reclaimed man-made island, started its operations as a 24-hour operating airport in September 1994.) The two airports are located on opposite sides of the Osaka Bay (see Figs. 10-6 and 10-7). The distance between the two airports is approximately 30 km, and the sea depth ranges from 20–40 m.

Although access to KIX is possible by train directly from Osaka and Kyoto, direct access from Kobe is possible only by bus or by ferry. Bus service depends entirely on traffic conditions, while ferry operation depends on the weather. Thus, residents of Kobe are anticipating railway access to KIX with great interest.

It would be technically possible to build a conventional bridge crossing Osaka Bay. Practically, however, bridge construction is difficult because of heavy shipping traffic, as well as high costs. Shield tunnels and submerged tunnels are alternative means to cross the bay, but they also are costly. In addition, shield tunnels in particular would require the construction of a man-made island in between, because the center blades of a shield machine should be changed after it has been driven a certain distance in the ground. Moreover, a man-made island built in the bay obviously could cause serious disturbances to ship navigation.

Studies for the proposed projects have been carried out since 1993 under the auspices of a Research Cooperative Working Group of the Japan Society of Civil Engineers, (Kansai Branch). The Working Group discussed fundamental problems encountered in the design, and carried out numerical analyses in order to obtain an approximate idea of the dimensions of the structure of an SFT that could be safely built in this area.

In the numerical analyses, the most important subjects are the external loads and how to model the structures. The fundamental study took into account, as external loads, the dead load of the structure; live loads from the train; environmental loads, including currents, waves, and tides; and seismic loads.

During the course of the study, the Kobe earthquake of January 17, 1995, occurred. The Working Group therefore carefully investigated the dynamic behavior of the SFT under earthquake excitation, and concluded that the design criteria for earthquakes should be changed: the acceleration of the ground vibration was changed from 350 gal to 750 gal, on the basis of seismic data recorded during the Kobe earthquake.

After the three-year feasibility study was completed, the Working Group concluded that the SFT is technically feasible, and recommended that the two airports should be connected by the SFT. The route of the SFT has been decided (see Fig. 10-7) based on considerations such as the depth of the water, geological condition of the seabed, ship navigation routes, and the location of aquaculture farms.

The SFT is designed for a double-track railway. The main structure of the SFT is made of concrete covered with steel plates on the outer surface; the cross-section is oval. The central space of the cross-section is to be used for ventilation, maintenance, and fire safety.

The unit weight of the SFT is a little heavier than that of sea water so that it sinks. Therefore, the SFT would be supported by supporting structures that consist of steel piles, spaced at 100-m intervals. Each steel pile is 2 m in diameter, approximately 60 m long, and sits on a diluvial formation located 60 m below sea level.

The estimated total cost of the project is approximately $US3 billion. This SFT project is now being studied in more detail by the Sub-Committee of the structural engineering round-table study group Kohzo-Konwakai. In order for the project come to fruition, various technical, economic, and social impact aspects must also be investigated.

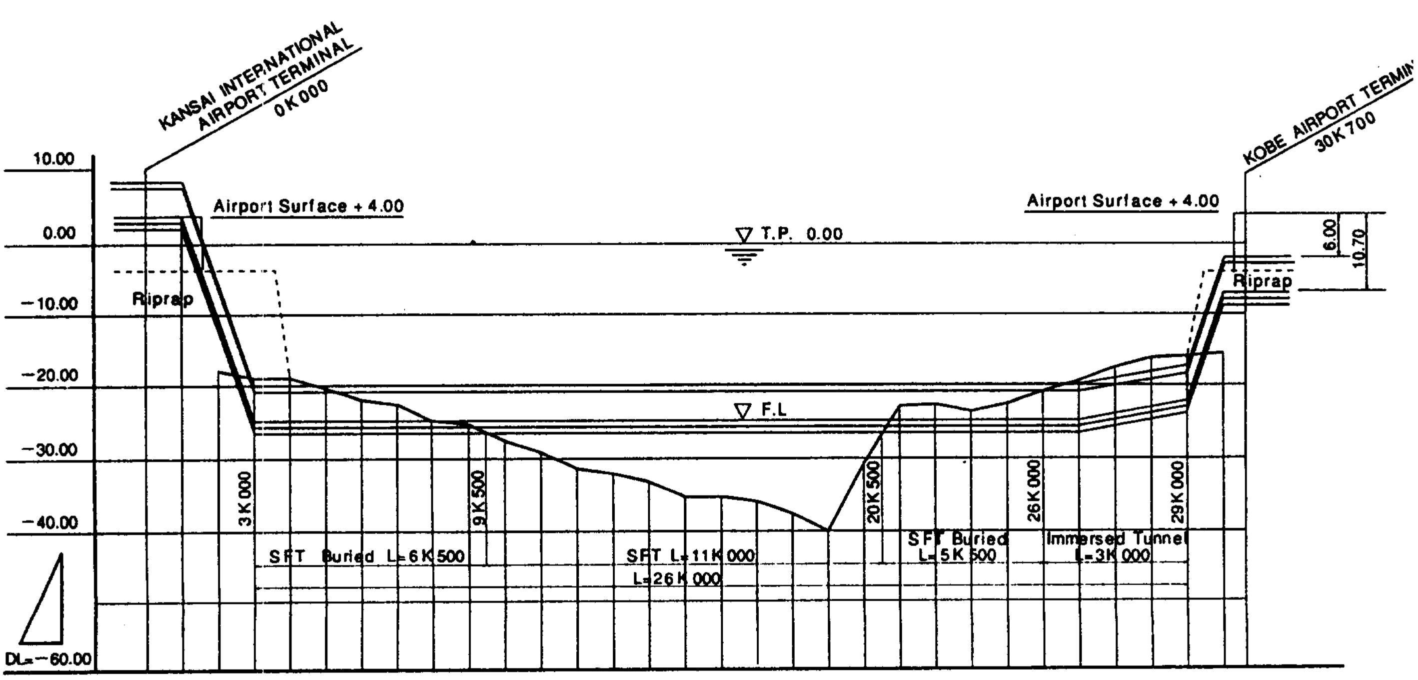



*Figure 10-7. Vertical section of the route for a proposed SFT across Osaka Bay, connecting Kansai International Airport and Kobe Airport.*

### 2.4 Lake Lugano Crossing

The Swiss transit authority wants to realize a high-speed north-south train connection as part of the Alp Transit system, but without obscuring the scenic views around Lake Lugano. An SFT has been proposed as an alternative to routing traffic around the lake, which has a maximum depth of 74 m.

Preliminary data has been collected on environmental conditions and estimated tunnel weights, and researchers are investigating possibilities for the prefabrication area (near a railway) and installation procedures for an SFT structure. The proposed SFT would carry 350 trains a day, travelling at 230 km per hour. The double-track cross-section would include three compartments underneath: two for ballasting and one for a service tunnel. The total length of the crossing would be 1360 m, 930 m of which would be submerged. The cost, including land acquisition, is projected at 185 million Swiss francs ($US138 million) (1996 prices).

## 3. Submerged Floating Tunnels (SFT): Technical State-of-the Art

This section is an edited version of 1993 state-of-the-art report (ITA 1993, *T&UST* 8:2) combined with updated information from a report published in *T&UST* 11:4 (Ahrens 1996).

### 3.1 Concept Description and Conditions Favorable to SFT Construction

In essence, a submerged floating tunnel (SFT) is intended to serve all types of traffic between two shores separated by deep water. The unique feature of this type of tunnel is that it provides an underwater connection that bridges the two shores and is not buried in or placed on top of the bed of the waterway. From the user's point of view, the structure has all the usual characteristics of a tunnel. Because it is enclosed, it is considered a "tunnel" rather than a "bridge."

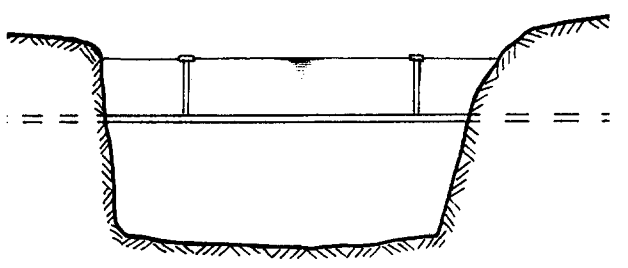


*Figure 10-8. Pontoons on the surface.*

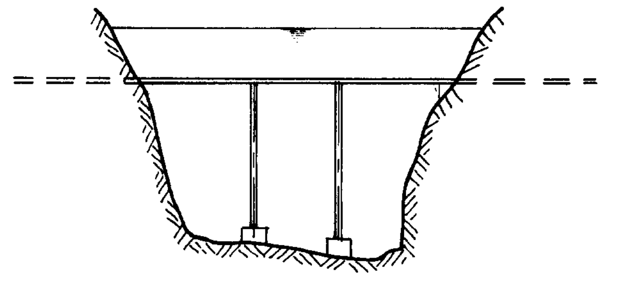


*Figure 10-9. Fixed supports.*

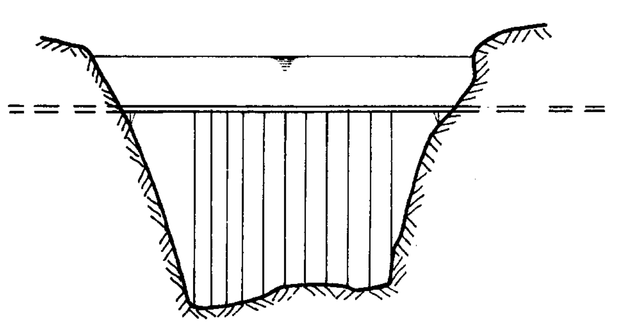


*Figure 10-10. Tension anchoring to the bed.*

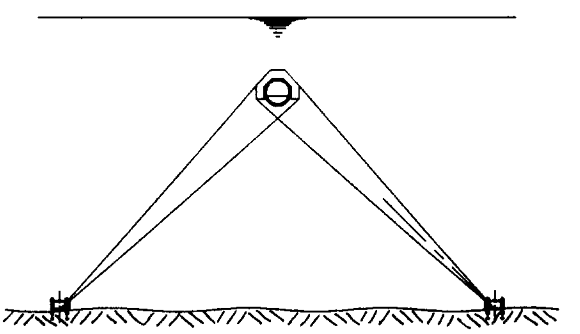


*Figure 10-11. Combined horizontal and vertical support.*

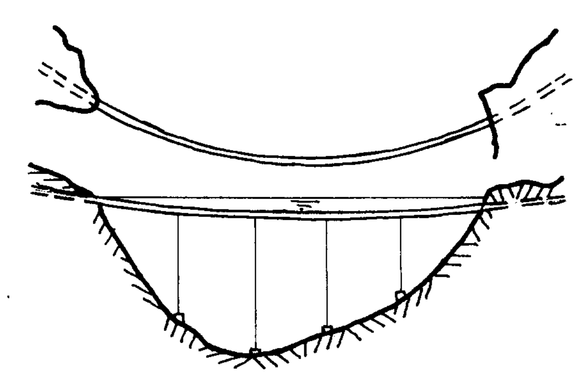


*Figure 10-12. Horizontal anchoring.*

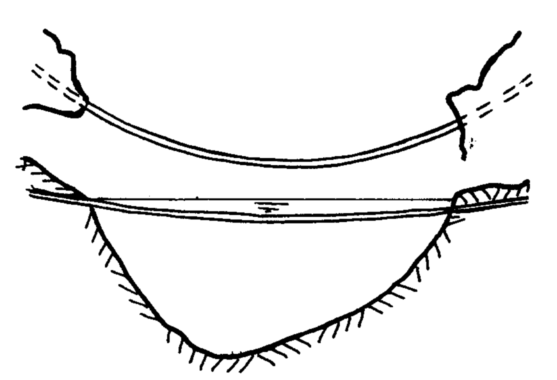


*Figure 10-13. Abutment anchoring.*

*Figures 10-8 through 10-13. Different types of submerged floating tunnels, characterized by type of support system.*

The difference between this type of tunnel and a conventional immersed or bored tunnel is that the floating tunnel structure is surrounded by water; that is, the tunnel is neither placed within nor bored through the ground. Instead, the tunnel structure is kept in position by virtue of its own structural capability, though it may be augmented by a support system. The support system may take several forms, such as:

- Pontoons on the surface.
- Tension anchoring to the bed.
- Columns or a support system from the bed.
- Horizontal anchoring.
- Combinations of the above methods.
- Anchoring at the abutments only.

The tunnel and its support system are positioned sufficiently deep below the surface that shipping can pass freely above them, ast least within the shipping channel. When pontoons are used, they can assist in identifying the shipping channel. Different types of submerged floating tunnels are illustrated in Figures 10-8 through 10-13.

The SFT concepts examined in this chapter use existing technology. The materials, the design methods, and the construction methods are known and conventional, but they are combined in a new and specific way. *It is important to stress that these structures do not include any totally new features that have not been encountered in tunnelling.*

Listed below are the main conditions under which an SFT could be considered a viable and competitive option for a waterway crossing:

- Where the water depth is greater than 50 m;
- Where the crossing length is greater than 1 km; and
- Where preservation of a scenic view or a natural habitat is considered highly important.

Designs being studied at present are considered feasible for conditions where tidal and stream currents are reasonably limited. Areas where submarine operations are conducted are considered unsuitable for an SFT, and in conditions where the water is reasonably shallow, an SFT will not be competitive. In addition, local factors such as wave action may be unsuitable for an SFT.

## *3.2 Feasibility of Submerged Floating Tunnels*

### Opportunities for Submerged Floating Tunnels

A submerged floating tunnel can span different types of waterways—estuaries, rivers, fjords, sounds, lakes, etc.—without interfering with shipping.

In some cases, an SFT may be the *only* possible, affordable and acceptable form of fixed crossing because of constraints such as deep water, distance between the shores, or environmental considerations (see Section 3.4 below).

The SFT technique offers the opportunity to plan crossings where they have never before been thought possible. In other cases, an SFT may be an alternative to another type of fixed crossing, such as a bridge, a pontoon bridge, a floating bridge, a bored tunnel, or an immersed tunnel. Investigations have shown that the SFT may be highly competitive with these solutions in some situations.

Another advantage of the SFT is that the crossing may be invisible from the surface, making it attractive from an environmental standpoint. In addition, the gradients are very small and surface waterway traffic is, in principle, not obstructed. Submerged floating tunnels have applications not only for road and rail traffic (e.g., as with the crossings described below), but for use as pedestrian tunnels and service tunnels (e.g., to guide pipelines and cables) as well.

### Safety

Research has shown SFTs to be technically and economically feasible. But are they also safe? The answer is yes, because *the level of safety required for an SFT will be the same as that required for comparable structures* such as bridges. The probability of failure of an SFT is of the same order as for other forms of crossings. This is not surprising, considering that the technologies used for the SFT, notably those concerning materials and design methods, are based on already existing technologies.

There are two particular aspects related to safety of submerged floating tunnels:

1. *Technological safety.* This will be assured, during the design of the total structure and its components, by adapting design rules and knowledge derived from experience.

2. *Psychological safety.* Although the expected movements are structurally acceptable, the human interpretation of such movements can be of major importance. It is therefore critical that the movements of the structure be kept below the limits of human observation. Research has shown that this is the case for SFT projects currently being prepared. In general, it can be said that the psychological interpretation limit of the movements is more critical than the technological limits.

Because the SFT concept is new, extra attention must be given to the safety aspects. This means that the first SFT will be based on rather conservative criteria and design assumptions. Research on the Høgsfjord and Messina Strait crossings, discussed in Section 4, has shown that all safety regulations can be met.

Measurements can be taken to minimize the risks of ship collision and to bring these risks into a range that is lower than that for comparable risks (see the discussion below and in Chapter 6).

## *3.3 Costs*

### Construction costs

At present, the cost of building an SFT can be only an order of magnitude estimate. Obviously the construction costs will vary from project to project because they are highly dependent on the local site conditions.

Based on cost experience on projects of similar magnitude and on the preliminary cost estimates of SFT projects studied in recent years, a comparison with the cost of relevant alternatives, such as a suspension bridge, can be made (see Fig. 10-14). The construction costs of an SFT, per meter length, do not increase significantly with the length of the crossing or with the depth of the waterway, making an SFT competitive with suspension bridges having spans greater than 800 to 900 m.

Even in cases where the SFT would be considered the only feasible alternative, the costs are not unreasonably high.

For an SFT with more than two lanes, the large uplift forces may increase the costs.

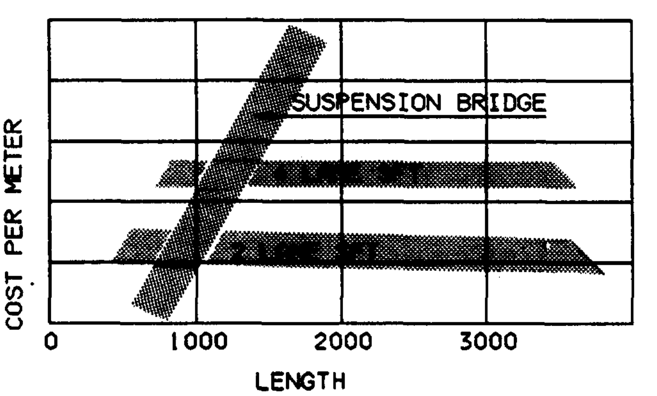



*Figure 10-14. Cost comparison for SFT solution vs. suspension bridge.*

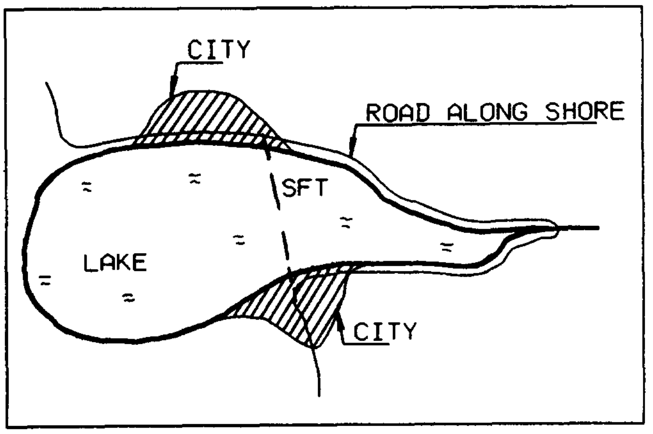



*Figure 10-15. SFT used for crossing a lake.*

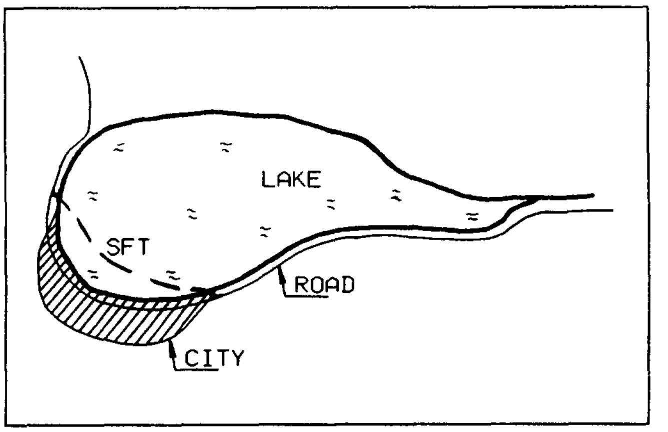



*Figure 10-16. SFT used for by-passing a city.*

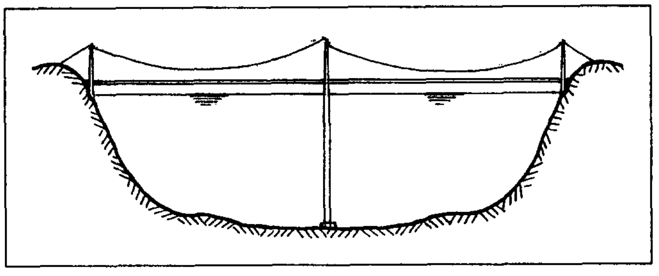


*Figure 10-17. SFT used for a very long crossing.*

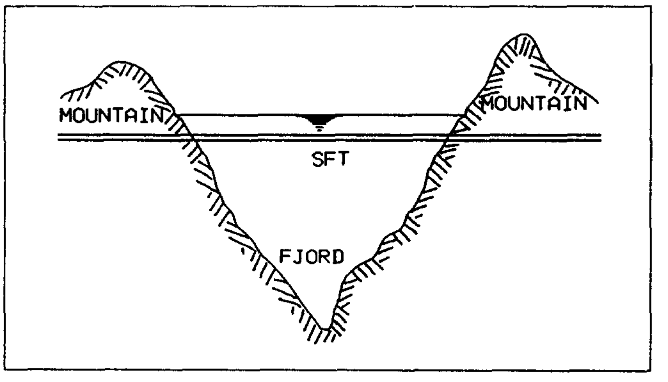



*Figure 10-18. SFT used to cross deep water, with mountains adjacent.*

**Operational costs**

In general terms, the expected operational cost of an SFT will comprise:

- The usual operating and maintenance costs related to tunnels in general, such as lighting, ventilation, interior cleaning, drainage, traffic control, etc.; and
- Costs particular to an SFT structure. The first SFT constructed will carry additional costs, at least during the early years, for instrumentation, monitoring, inspection, reporting and, subsequently, verification of the design. [For a detailed description of Instrumentation, Documentation and Verification (IDV) systems, see Appendix B.]

Maintenance costs for the external surface are expected to be very low. Because marine growth is one of the design criteria, the design may make removal of marine growth unnecessary. For all tunnels, external inspection and maintenance may be routinely necessary and will continue throughout the life of the tunnel. This maintenance may require replacing components, although the total cost of such inspection and maintenance is not expected to be high.

## *3.4 Environment and Possible Locations for SFTs*

**Environment**

SFTs have a number of special features, many of which offer good solutions to environmental constraints. As noted above, the use of an SFT may make a crossing possible where planners in the past thought it was impossible.

In many parts of the world, it is necessary that certain crossings be made invisible in order to avoid interfering with the natural beauty of the surroundings. Structures such as suspension bridges, cable-stayed bridges or floating bridges, no matter how elegant and high-tech, cannot avoid interfering in some way with the surface environment.

Some examples of the SFT solution to environmental problems are given below.

*Case #1: An inland lake having severe traffic problems around the perimeter, especially during tourist seasons.* The car and heavy truck traffic has become such a problem that the area is becoming less attractive every year. However, because of the natural beauty of the lake, a suspension bridge or floating bridge is not desirable. In such a case, only three "invisible" alternatives are possible:

1. A tunnel underneath the bed of the lake.
2. An immersed tunnel.
3. An SFT.

If the lake is very deep, a tunnel underneath the bed might be very long and, consequently, expensive to construct, maintain and use. If the water depth is moderate, an immersed tunnel may be contemplated; however, if the depth is considerable, this solution must be ruled out.

For such a situation, an SFT could be the only answer. It could be located invisibly at a level fairly near the surface of the water, because the sea state of the lakes is much more than moderate, and the tunnel could take the shortest route across the lake (see Fig. 10-15). Furthermore, an SFT would require only a limited construction area at the side of the lake, thereby minimizing interference with the existing traffic. Additionally, the SFT solution would have the merit of limited environmental impact.

*Case #2: A city or an area of environmental interest alongside a fjord or lake has busy traffic passing through it, and no options to reroute the traffic on land available because of geographical and/or environmental reasons.* The solution could be to reroute the traffic through an underwater tunnel parallel with the shore (see Fig. 10-16). An SFT would be invisible and, in comparison with an immersed tunnel, would make fewer demands on the underwater bathymetry.

*Case #3: A strait of considerable width to be crossed.* Using an SFT anchored to the sea bottom would avoid creating a visible impact on the environment. There are no construction limitations to the length of such a structure. In contrast, a bridge has limitations in its span length; and the piers and anchorages that might be required for a multi-span bridge would pose obvious difficulties in deep water (see Fig. 10-17).

*Case #4: A deep fjord or waterway with high mountains directly adjacent to it, and the need for a fixed crossing as part of the national transportation system.* In the past, planners would have considered such a crossing impossible. However, an SFT could be built to connect directly with tunnels at both sides through the mountains, as shown in Figure 10-18. The SFT would make it possible to create a crossing in which no part of the structure is visible—a characteristic that could yield great environmental advantages.

These four examples illustrate possible ways that an SFT may offer a unique solution to important and difficult environmental problems.

### Possible Locations

As discussed above, a submerged floating tunnel may be applicable in a number of locations where alternative structures are also feasible. In such cases, the SFT will need to be a competitive alternative if it is to be adopted. In contrast, at some locations an SFT may be the only feasible alternative.

Possible locations for SFTs fall into two main categories: (1) coastal regions and (2) inland regions. The technicalities related to each of these locations are discussed below.

#### *Coastal regions*

Coastal regions, which include fjords, sounds, and connections between islands, often pose the greatest difficulties for fixed crossings.

Although there are possible locations for coastal SFTs on all continents, certain areas present particularly worthwhile opportunities. Some of these are listed in Table 10-1.

#### *Inland regions*

Large lakes exist in virtually every country. In the past, many of these lakes have been considered impossible to cross, for a number of reasons. In some cases, the lakes have been considered too wide for conventional bridges, and environmental concerns have prohibited a visible structure. The obvious choice would be to construct a tunnel underneath the bed of the lake. However, such tunnels often become very long and expensive, or the depths are excessive.

An SFT offers the possibility of crossing at very gentle gradients and without visible environmental impact. SFTs could reduce some of the increasing road traffic around a number of lakes by reducing the travel distance, in addition to hiding it from view.

Table 10-2 shows a number of lakes throughout the world that are of particular interest for application of the SFT concept. This list, although incomplete, reflects current opportunities perceived by the authors. There are undoubtedly further opportunities in other parts of the world. Local authorities and developers will have to determine whether they might be suitable for this form of crossing.

### *3.5 Structural Principles*

To understand the structural principles of SFTs, we may compare them with other fixed crossings. SFTs may have a variety of forms, depending on parameters such as distance between the shores, water depth and wave conditions. Table 10-3 shows whether or not the different structures are

*Table 10-1. Possible coastal locations for SFTs worldwide.*

| Country | Possible Locations for Coastal SFTs |
|---|---|
| Norway | Many fjords |
| Italy | Strait of Messina |
| Greece | Mainland to islands |
| Turkey | In-between continents; between mainland and islands |
| Spain/Morocco | Strait of Gibraltar |
| France | Gironde |
| U.S.A. | Fjords on west coast |
| Alaska | Bering Strait |
| Canada | Fjords on west coast |
| South China Sea | Between islands |
| Coast of Southeast Asia | Mainland to islands |
| Japan | Mainland to islands; between islands |

*Table 10-2. Potential inland SFT locations, by continent.*

| Country | Location of Lake |
|---|---|
| **EUROPE:** | |
| Italy | Como/Lecco |
| Italy | Maggiore |
| Italy | Lugano |
| Italy | Iseo |
| Italy | Garda |
| Switzerland | Neuchatel |
| Switzerland | Vierwaldstettersee Lake Lucerne |
| Switzerland | Zürichsee |
| France, Switzerland | Geneve/Leman |
| Germany, Austria, Switzerland | Bodensee/ Constance |
| Sweden | Vättern |
| Portugal | Rio Tejo |
| **THE AMERICAS:** | |
| Canada, U.S.A. | Superior |
| Canada, U.S.A. | Huron |
| Canada, U.S.A. | Erie |
| Canada, U.S.A. | Ontario |
| U.S.A. | Michigan |
| Nicaragua | Managua |
| Peru, Bolivia | Titicaca |
| **ASIA:** | |
| Israel, Jordan | Dead Sea |
| Japan | Biwa Ko |
| Ukraine | Azov |
| **OCEANIA:** | |
| New Zealand | Taupo |
| New Zealand | Wakatipu |

affected by these parameters. The comparison shows that one of the most critical advantages of the SFT is that it opens up possibilities where formerly no fixed connection was considered possible.

### Design Methods

Like every other structure, an SFT must be designed on the basis of expected and possible combinations of loading cases. The allowable design methods and general criteria are mainly determined by the national codes. Otherwise, the client/owner, together with the designer/contractor, must establish them.

Today's design methods are often related to those used in the offshore industry, and are commonly based on the semi-probabilistic *limit state* approach, using partial safety coefficients on both *loads* and strength of *materials*.

#### Limit states

The semi-probabilistic approach divides the design into the following design limit states:

*SLS* (Serviceability Limit State): The SLS conditions are set to ensure that the structure meets practical criteria with regard to deflections, crack widths, factors of safety, accelerations, etc.

*ULS* (Ultimate Limit State): The ULS conditions are set to confirm that the structure has the necessary margin of strength to survive factored loads and load combinations, with the factors being set to provide an acceptable risk of failure. The factors must also be sufficient to ensure that the structure is capable of continuing to operate satisfactorily after an unfactored event. The accepted risk level differs among countries.

*PLS* (Progressive Collapse Limit State): The PLS conditions are designed to preserve human lives in the event of certain loads or load combinations at a very low probability of occurrence. In this case, even if the structure may be severely damaged, loss of lives is still not acceptable. These conditions are often defined at a probability level of approximately $10^{-4}$ per annum.

*FLS* (Fatigue Limit State): The FLS is required to account for the fact that some materials lose strength due to repeated loading. By computing the accumulated damage in the material and consequently checking the computed life of the structure against the operational life, the sensitivity of certain components of the structure can be established. A safety factor of 3 to 10 between computed life and required operational life is often adopted, depending on the consequences of failure and the opportunities for repair of the components.

*Materials:* The safety factors to be used for each type of material are often determined in the national design codes, or they may be specified explicitly by the client.

#### Loads

The individual loads are combined to give the design loads by using partial load factors. These load factors are generally established in the national design codes, or they may be specified explicitly by the client.

Five different types of loads usually must be considered:

1. *PL* (Permanent Loads): Permanent loads are classified as loads that will be permanently present during the lifetime of the structure. The structure will first be exposed to those loads during the construction period. The most common permanent loads are:

- Structural dead weight.
- Hydrostatic pressure.
- Buoyancy.

2. *FL* (Functional Loads): Functional loads are those loads that will be caused by the usage of the structure. For an SFT, these loads are:

*Table 10-3. How the parameter of distance, water depth, and sea state affect different crossing structures.*

| Type of Crossing | Distance | Water Depth | Sea State |
|---|---|---|---|
| Bridge | Yes | Yes | No |
| Pontoon bridge | No | No | Yes |
| Floating bridge | No | No | Yes |
| Bored tunnel | No | Yes | No |
| Immersed tunnel | No | Yes | No |
| Submerged floating tunnel | No | No | Limited |

- Loads caused by traffic.
- Loads resulting from changes in ballast conditions.
- Variable loads during construction.

3. *DL* (Deformation Loads): Deformation loads are caused by geometric changes in the structure itself. These loads are often associated with the properties of the materials involved. Typical loads within this category are caused by:

- Shrinkage.
- Creep and relaxation.
- Post- or pre-tensioning.
- Differential settlements.
- Temperature variations.
- Remaining internal loads resulting from the construction method.

4. *EL* (Environmental Loads): Environmental loads are caused by the (local) site conditions. Special investigation and study are often required to determine the magnitude of these loads. To assess the effects of such loads on the structure, mathematical and hydraulic models may be needed. For an SFT, the most important environmental loads are:

- Loads resulting from wave action.
- Static loads caused by current.
- Dynamic loads caused by vortex of current.
- Loads resulting from tidal variation.
- Loads caused by floating ice on the water surface.
- Loads resulting from changes in water density.
- Response caused by earthquake.

5. *AL (Accidental Loads):* Accidental loads are, by their nature, not supposed to happen. However, the fact that they *do* happen from time to time necessitates their specification and a rational way of dealing with these loads. Effects to be considered in this load category are:

- Explosions inside or outside the tube.
- Fire from burning cars or fluids.
- Loss of buoyancy.
- Failure within the support system.

#### Load application

The magnitudes of applied loads and the associated acceptable structural behaviour are defined in terms of their probability of occurrence. As an example, for the Strait of Messina crossing, which has a design life of 200 years, the following levels were adopted:

1. *Return period equal to 50 years:* Serviceability behaviour of both the main structure and the secondary

components, without any damage or need for inspection after the event.

2. *Return period equal to 400 years:* Serviceability behaviour of the main structures and local damage (plastic behaviour) for the secondary components. Limited damage, if any, must be reparable without any interruption in the use of the tunnel.

3. *Return period equal to 2000 years:* This was defined as plastic behaviour with the complete exploitation of the ductility resources of all the structural components. This would be analysed today as PLS. Damage, but not collapse, of the main structure is accepted. Damage and collapse are accepted in the secondary components.

In addition to load cases similar to those described above, one special load case associated with the Strait of Messina project must be taken into account. This load occurs by means of:

- A slow tectonic slip between Sicily and Italy, occurring in the longitudinal direction of the tunnel, the value of which could reach approximately 0.20 m during the expected life span of the structure.
- A slow relative movement of the several faults present in the area, which could cause a differential settlement of the foundation blocks.

### Ship collision

All of the feasibility studies for SFTs must focus attention on the accidental loading caused by the collision of a ship or submarine. The safety of an SFT is based on avoiding collision with vessels large enough to damage the structure seriously, as has happened many times to various bridges. Collision of surface vessels can be easily avoided, as the SFT can be positioned at virtually any depth beneath the water surface.

In cases where there is heavy surface traffic, the probability of a sinking ship at that particular location, and the subsequent consequences, must be considered. The energy associated with the impact of a sinking ship (or other object) against the structure must be (partially) absorbed by local and/or global deformation. The magnitude of absorption will depend on the type of ship.

For example, for the SFT proposed for the Strait of Messina, the impact of a 5000-dwt sinking ship must be within level 1, while the impact of a 250,000-dwt sinking ship must be within level 3. The type and size of ship and the appropriate levels will be different for each SFT.

Another form of accidental loading, which is probably more frequent but less sensitive, is the impact of fishing equipment. This type of impact can largely be avoided by adherence to marine regulations.

A traffic regulation system has to be provided for submarine traffic. If necessary, measures must be taken to ensure that both the tunnel and its supports system—especially cables—can be monitored by the submarine navigational equipment. A warning system may be used to ensure the safety of the traffic in the SFT itself.

## *Dimensions*

Site conditions have an impact on the dimension of the structure in terms of criteria for both the design of a crossing and the construction methods.

Conditions to be taken into account include:

- Water depth.
- Length.
- Marine environment: currents, water densities.
- Sea state: long and short waves.
- Geology at the entrances.
- Geology at the foundations of the support system, if applicable.
- Environment.
- Seismic activity in the area.
- Access for bringing marine equipment to the site.
- Surface and subsurface water traffic.

The dimensions of the main body of an SFT are determined by the internal, external and structural requirements. The dimensions of the secondary components of an SFT are mainly determined by structural requirements and installation methods.

### Internal dimensions

The internal dimensions depend on the purpose of the tunnel and, therefore, must be defined by the user. Space also is required for supporting facilities such as ventilation, lighting, signs, ballast, emergency equipment, etc.

### External dimensions

The external dimensions are mainly determined by:

- The required internal dimensions .
- The overall structural concept.
- The local structural requirements.
- The influence of external loadings (ship collision).
- The construction methods.

It has been found that the minimum internal dimensions, which are required to accommodate the required number of traffic lanes and the necessary equipment, result in almost the optimum design. The consequent external dimensions have a significant influence on the behaviour of the tunnel because the most important design loads are related to its volume. Examples of such loads are the circumferential compression load, the buoyancy and the added mass, and the forces of inertia associated with wave motion and seismic excitation.

Recently developed concepts have featured circular-shaped or hexagonal-shaped tunnels. To reduce these loads for the SFT proposed for the Strait of Messina Crossing, three parallel and independent tunnels are planned.

## *Buoyancy*

The vertical stability of an SFT is a very important consideration. The concept can best be explained by comparing the SFT with other types of tunnels.

In the case of a *bored tunnel,* the vertical stability is sufficiently ensured by the weight of the soil above the tunnel. This weight is always greater than the buoyancy (in ground water), both during construction and in the final situation.

In the case of an *immersed tunnel,* the vertical stability is ensured by additional ballast concrete, which is added after the element is placed in the trench. There will always be a resultant downward vertical force, equal to the difference between the final structural weight and the buoyancy of the element. The weight of the backfill in the trench will increase the vertical stability. Only during the transport and, in some cases, during placing of the element, when no ballast concrete is present, will the buoyancy exceed the weight of the element.

In the case of a *submerged floating tunnel,* the relationship between buoyancy and self-weight is very important. In general, the weight of the SFT and its buoyancy are almost in equilibrium, even with the large magnitude of each of these loads. Because an SFT will never be covered by soil, the vertical stability has to be ensured by the structure and its support system. There are two possibilities for accomplishing this:

1. A **tether system,** in which the structure delivers a resulting upward force (more buoyancy than weight), which is taken by the downward force in the support system.

2. A **pontoon system,** in which the structure delivers a resulting downward force (more weight than buoyancy), which is taken by the upward force in the support system.

Because the resulting load in the tether or pontoon system is the difference between two large forces, accuracy in determining these forces is of the utmost importance. The following uncertainties in regard to these forces may arise:

**Tolerances in geometry and dimensions.** The acceptable tolerances in the geometry and dimensions must be established during the design stage, depending on the choice of construction method. During construction, an experienced contractor with an appropriate quality assurance system is the client's best insurance for keeping the tolerances under control. After construction, this parameter is no longer a variable but, rather, is exactly known. Given sufficient flexibility in the amount of ballast concrete, it is possible to adjust the weight afterwards.

**The specific weight of concrete.** Although the specific weight of concrete will vary during construction, it can easily be measured. Nevertheless, the acceptable range has to be established beforehand, during the design stage. After construction, this parameter is known exactly; by using ballast concrete, the final weight of the structure can be adjusted. The "weight" of the structure may change slightly over time, as the concrete absorbs water. However, this alteration is minimal in comparison to the capacity of the support system.

**The specific gravity of water.** The range in the specific gravity of the water will be particular to the site, and should be obtained at an early stage in the design process. The variation in buoyancy resulting from the change in the specific gravity of water is a permanent variable, and may have special importance in coastal areas, where the amount of river runoff or melting ice or snow can change the value rapidly. The design has to cope with these variations.

**The amount and stability of marine growth.** Marine growth is known to concentrate at the sea floor and at the surface (see Fig. 10-19). If the SFT is not located in the critical surface layer, the effects of marine growth will be minor. However, where such growth does occur, it will increase the weight and the current resistance. Therefore, accurate predictions of the amount and weight of marine growth are required. The amount of marine growth may differ among projects, depending on the seawater temperature and the depth that the tunnel lies below the water surface.

The range in weight/buoyancy ratio of the tunnel and the support components has to be investigated carefully in order to avoid a net reverse in the resultant forces. During installation, other conditions may occur, depending on the design and the construction method.

### *Supports*

The SFT is supported by the abutments at the shores and, if necessary, by further supports along the tunnel. When the SFT reaches a certain length, which will depend on factors such as diameter, shape, material choice, etc., it will be necessary to support the tunnel in both the horizontal and vertical directions.

#### Horizontal supports

Usually the static horizontal loads are relatively small in relation to the allowable span length. Currents are the main cause of these horizontal loads. This means that large spans may be used with a minimum of horizontal support.

Some proposed schemes span the total waterway, using the principle of an arch to carry the horizontal loads (see Fig. 10-13). At a certain span width, the arch shape will not be sufficient to cope with the horizontal loading. A horizontal mooring system is then required.

#### Vertical bottom supports

The requirement of safe vertical stability under all loading cases results in an upward force that is generally larger

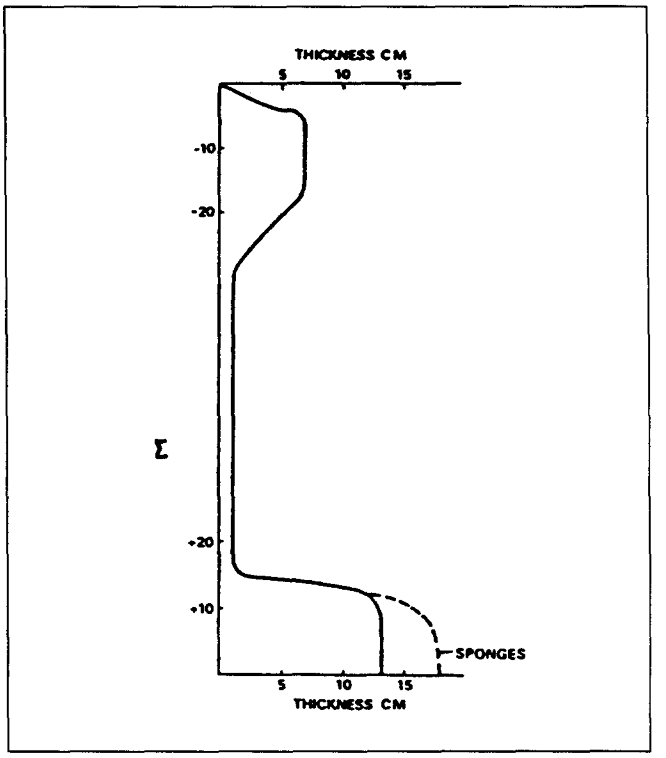



*Figure 10-19. Marine growth profile.*

in magnitude than the forces in the horizontal direction. A support system is therefore required at shorter intervals. The structural principle in the vertical direction involves a continuous beam with intermediate supports.

The principle of the bottom mooring is that the SFT should have sufficient positive (upward) buoyancy to keep the tethers to the bottom under tension at all times, even during seismic events.

A tethered solution is attractive because it can be used in very great water depths. The tethers or tether groups spacing depends on the weight/buoyancy ratio and the cross-sectional strength of the tube (see Fig. 10-10). The intervals between the tethers will vary. For example, the intervals in the studies for the Strait of Messina crossing varied between 50 m and 70 m, compared to 400-m intervals for the Høgsfjord crossing.

The tethers can be fabricated of steel wire or steel tubes, similar to those used in the offshore industry for the mooring of tension leg platforms (TLPs). Because the tethers of an SFT are subject to smaller motions and loads than those of a TLP, the tethers and connections for SFTs can be less complex than those used in the offshore industry. Experience has been gained with this type of tether mooring in water as deep as 550 m.

The use of aramid cables has also been studied. These cables have advantages related to corrosion resistance, stiffness/resistance ratio and low weight in the water, which facilitates installation and avoids catenary effects. A disadvantage of aramid cables is the lack of catenary effect during large motions of the tunnel.

The tethers transfer the upward tension force to the bottom anchor points. The type of *anchor point* used will depend on the soil conditions and installation methods. Both piled anchors and gravity anchor blocks have been proposed for projects. When rock exists in the bottom, drilling and grouting of the anchors may be a solution.

An alternative is the use of *fixed bottom* supports. The height, which depends on the water depth, can range from some tens of meters to about 100 m. The base of the support generally will be a gravity structure, which will require good soil conditions. The structural system formed by the tunnel and its supports will correspond to a continuous beam with fixed supports (see Fig. 10-9). Normally the analysis will be fairly straightforward. The response under load of such an arrangement may be quite different from the response when a cable arrangement is used.

#### Vertical pontoon supports

In the case of a pontoon support, the SFT will be carried by the pontoons at the surface. This is a more elastic system than the bottom support system (see Fig. 10-8). Although this solution has the advantage of being independent of water depth, it has to cope with the interaction with ships, waves and ice.

#### Combined horizontal and vertical supports

If the SFT is long and sufficient horizontal support cannot be obtained from the shore, a combined horizontal and vertical bottom support can be achieved by using an anchor cable configuration such as that shown in Figure 10-11. The angle of inclination of the cables may be chosen so as to yield an appropriate ratio between the horizontal and vertical forces and stiffness for the particular solution.

#### Abutments

At each shore a special form of abutments is needed that will contain an axial expansion/structural hinge device. This device will limit the longitudinal and transversal forces induced, for example, by the seismic effects. It is preferable that the abutments be below water level; otherwise, the SFT may be subject to wave loading. Where such an arrangement is not possible, a protective barrier such as an earth-filled structure is desirable.

The approach structures on each shore may be composed of ramps, bored or cut-and-cover tunnels, or a combination of these. Depending on the construction method, one of the ramp structures may be used as a construction site.

### *3.6 Static and Dynamic Analysis*

As part of the design process, several static and dynamic analyses must be performed, both for the structure as a whole and for certain components. The SFT must be considered as a long continuous beam that is unsupported or is on fixed or flexible supports. At the ends, however, the road has to be continuous; and, therefore, no significant rotations can be accepted there.

#### Global Static Analysis

Global static analyses are necessary to investigate the interactions between the components of the structure and to reach a balanced design following an optimization process. The global model can begin with a straightforward beam-column model. However, a (three-dimensional) finite element model that takes the elastic foundations into account will be needed soon afterwards, in order to achieve a sufficiently accurate analysis. Displacements of the structure depend on the structural system used. They must be correctly taken into account in order to investigate the consequent geometric changes.

Special software may be required to handle the large number of load cases and the considerable quantity of data. The load cases and the limit states are discussed above. The critical load cases will be combined with the results from the dynamic response analyses before the dimensions and stresses are checked. At this point in the design, the preferred pre-tension in the support system has to be established. This pre-tensioning will act together with the loadings, in both the tether system and the pontoon system.

Experience gained in conceptual studies for proposed crossings has shown that the weight/buoyancy ratio is the determining factor in the dimensioning of the structure. All other load conditions are of minor importance.

#### Local static analysis

Local static analyses are normally performed as detailed finite element models, both two-dimensional and three-dimensional, for areas where high stress concentrations are expected. These areas must be studied in detail. Some typical areas requiring study are:

- The joint between the SFT and the abutment.
- The tether connections.
- The pontoon connections.

#### Dynamic analysis

A characteristic feature of an SFT is its dynamic behaviour. This behaviour can be analyzed using the same or more specialized finite element software programs that are used for static analysis, and by executing hydraulic model tests in a laboratory.

In the dynamic analysis, considerable care must be taken in selecting the data used and the interpretation of the results. It takes experience to ensure analyses of acceptable quality. Items of importance for these analyses are:

- The mass and expected added mass of the structure and its individual components in relation to each other.
- The stiffness of the structure and its individual components in relation to each other.
- The damping of the structure and its individual components in relation to each other.

Because of the length of the SFT, the highest resonance periods of the tube will be well above the wave period range. However, continuous beams such as an SFT have a large number of eigen periods, some of which inevitably will be in the wave range. The dynamic amplification of a resonant system is very sensitive and requires careful investigation.

After an SFT has passed the conceptual stage, further analysis can be done with a hydraulic model, simulating certain loading cases. Furthermore, the hydraulic model can be used to calibrate the results from the mathematical models. However, it must be kept in mind that both are only models.

##### *Dynamic response caused by marine loading*

The resonance frequencies in the system cause the structure to be sensitive both to wave frequencies and to structural damping. Therefore, a design wave analyses approach is not sufficient; rather, the analyses should be based on a stochastic approach. Short-term analyses may be acceptable if uncertainty in the wave spectrum period is properly accounted for. The preferable approach for the wave dynamic analyses involves long-term analyses. Analysis has shown that for both the proposed Høgsfjord and Strait of Messina SFT projects, the response on sea states is well under the acceptable design values.

Fatigue in the different components (e.g., the tunnel, cables and connections), caused by sea states of different magnitudes, is expected to be very low. Even though the SFT is a slender structure, the possibility of vortex shedding by current needs to be considered. However, the studies mentioned in this chapter have not indicated that this would pose difficulties. For example, at Høgsfjord, where currents are below 1 m/s, no movements are expected.

##### *Dynamic response resulting from seismic loading*

Under seismic disturbance, the response can occur in three directions: transverse, vertical and longitudinal. A seismic disturbance results in motions with frequencies in the higher and lower ranges. The low frequencies are typical of ground displacements. The SFT has a large number of eigen frequencies in the high and low ranges.

Analysis indicates that the system filters the high frequencies, but is sensitive to the low frequencies. This may differ from project to project and from design to design.

The *vertical* excitation will cause the maximum accelerations of the SFT. This type of excitation is mainly caused by the seismic pressure waves in the water, generated by the earthquake.

The *transversal* excitation will cause the maximum displacement and, thus, the maximum bending moment in the tunnel section. The maximum axial forces will occur in the anchoring cables, if they are used. Here again, the excitation is mainly caused by the seismic pressure waves in the water, generated by the earthquake.

The *longitudinal* excitation requires expansion devices at the connections to the shore in order to limit the effects of the earthquake. Otherwise, assuming perfect linear elastic behaviour, very high axial forces will occur. This behaviour could be critical for the connection of the tunnel to the shore. The longitudinal excitation may be caused by the seismic input at each end of the tunnel.

#### Dynamic response caused by passing trains

The effects of a passing train on the global SFT are negligible, compared with the effects of environmental and seismic loads, in the case of the Strait of Messina. Analyses from the same project have shown that for the case in which a train separates in the tunnel and the individual parts hit behind each other, the dynamic lateral forces caused by this event can be taken by an SFT, assuming the rail support system has been designed for it.

### 3.7 Longevity and Risk Analysis

During the design, a probabilistic risk analysis needs to be performed. The possibility of failure of an SFT will be determined by studying the probability of failure of the individual structural components. An IDV (Instrumentation Documentation Verification) system application will be ideally suited to assessing the longevity of an SFT structure. Because longevity and durability are important inputs for the owner's commitment to any SFT undertaking, a brief edited version of an IDV proposal highlighting future R&D incentives for SFTs is presented in Appendix B.

The distribution of the possibility of failure of the components can be indicated by fault trees (see Fig. 10-20). If necessary, action can be taken to reduce the local risk.

### 3.8 Instrumentation

To monitor the SFT for safety and to validate the design, the first SFT will be outfitted with extensive instrumentation. The instrumentation will be specially designed to monitor both the environmental forces and the response of the overall structure and the individual elements. Material behavior, as well as displacements, will be monitored. The instrumentation will be used during construction and will be operational after completion. If an IDV is adopted, the instrumentation should be part of that system application.

### 3.9 Durability and Maintenance

In a marine environment, the durability of the materials used requires special attention. The tunnel itself should have a design life of perhaps 100 years, although elements such as pontoons, cables, connections and electrical and mechanical plant may have a shorter design life.

A comprehensive maintenance programme should be established to include periodic inspections and the scheduled replacement, as necessary, of individual components. If an IDV is adopted, maintenance aspects should be part of the verification part of that programme.

Special provisions and loading cases arising from replacement operations can be taken into account during the design stage. It also may be useful to develop special equipment to inspect and maintain the structure and its parts. Of particular importance is the resistance of the concrete in a chloride environment and the anti-corrosion provisions required for a steel shell, if used. In the case of a cable support system, the durability of the cables needs to be investigated with regard to both corrosion and fatigue.

Sufficient experience with the maintenance of structures in sea water and in rough environmental conditions has been gained from offshore projects. This experience includes both the structural repair and maintenance procedures and the working methods to fulfill the operations. Special procedures and working methods for both repair and maintenance of the various parts of the tunnel must be prepared as contingency measures.

### 3.10 Construction Methods

Experience with similar complex projects, such as those offshore, has proven that design and construction cannot be isolated from one another. Therefore, feasibility studies for SFT projects must consider both design and construction—including, with regard to the latter, the possible methods of construction and the availability of suitable construction sites.

Site-specific conditions, such as marine environment and soil conditions, may affect both the design of the preferred structure and the construction method. It is important that structural analyses of the construction stages be performed early, in order to avoid surprises later on in the project. Additional loading cases must be kept to a minimum to reduce extra costs.

The successful construction of a project of this degree of complexity relies largely on the experience of and equipment available to the contractor. It is therefore advisable to obtain advice from experienced contractors during the feasibility study phase.

Two basic construction methods have been developed:

1. *Construction in elements.* The tunnel is fabricated in elements in a construction dock and coupled to the previous installed elements. The supports can be cables, pontoons or fixed piers.

2. *Incremental launching.* The tunnel is fabricated in sections and pushed out in successive steps. Launching is repeated until the opposite side of the fjord is reached. During launching, a cable-stay system supports the tunnel.

Each of these methods is described in detail below. It should be noted that the suggested methods are generic and must be modified to suit the given site requirements.

#### Construction in elements

The SFT elements are constructed in a dock. After all or a number of the elements have been completed, the dock is flooded and the elements are towed to the site. Bulkheads, used to seal the ends and to maintain positive buoyancy during construction, are joined at the site. The length of each element is determined partly by the structural capabilities of the SFT (the designed distance between the supports) and partly by the available length of existing ship docks, slipways or construction docks. The construction of the elements is similar to construction for immersed tunnels.

Because SFTs and immersed tunnels are designed for different permanent and temporary loading cases, the length of the elements for an SFT is not limited to the 100 to 150 m typical of immersed tunnels. Particularly when permanent prestressing is used, the length of the elements can be increased.

After reaching the site, the installation barge supports the element during assembly and lowering to the intended depth. At each joint location, a set of tethers is pre-installed and coupled in a horseshoe-shaped support. The element is lowered under the support while temporarily pulling the support aside.

After the element has been fitted into the predetermined tether support system, it is de-ballasted, causing the load to

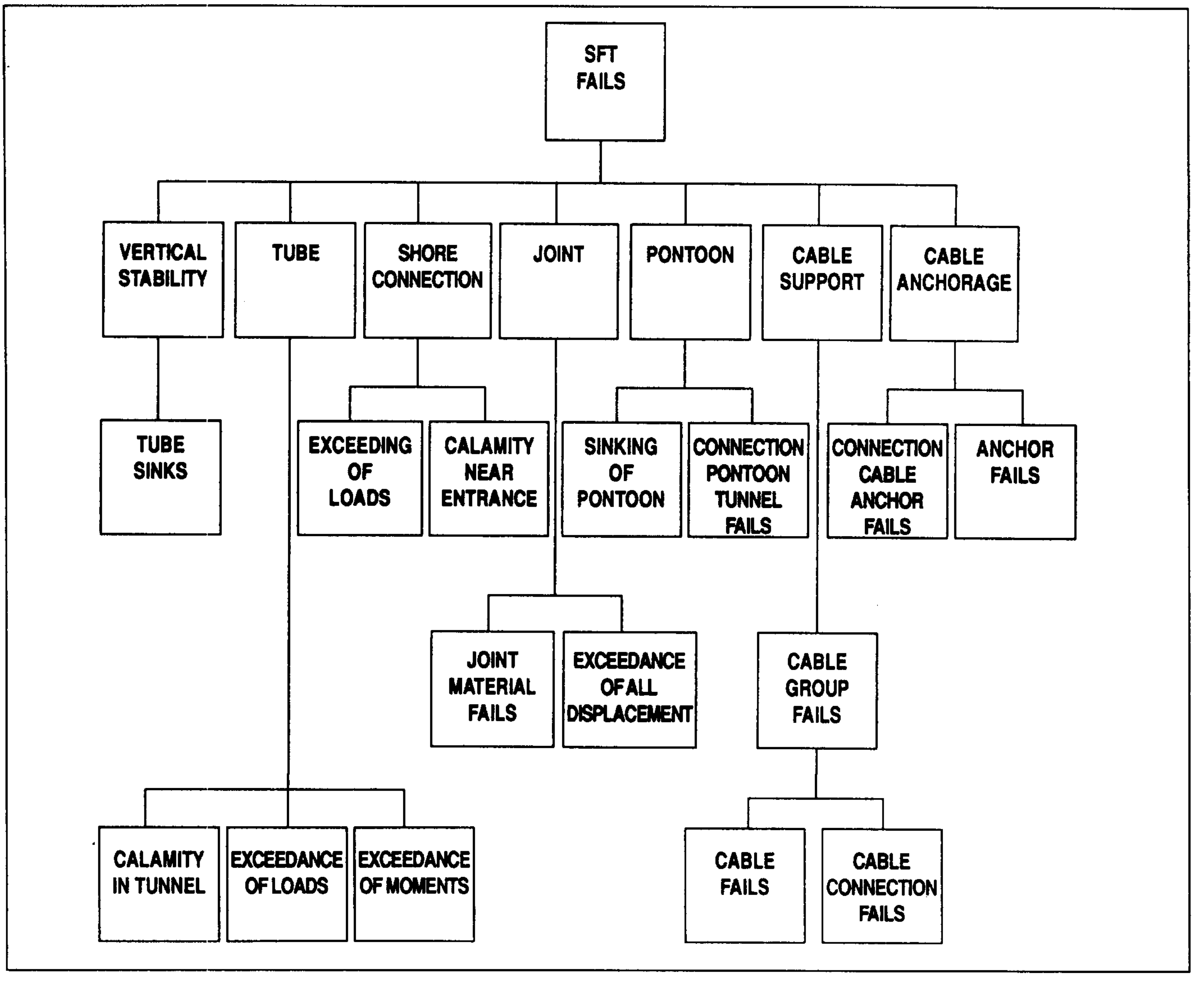



*Figure 10-20. Fault tree showing the failure possibilities for SFT components.*

transfer from the installation barge to the tether system. During this process, the length of the tethers is adjusted at the support shoe to prevent unacceptable deflections of both tethers, the position of new element and of previous elements. The adjustments can be made by remote control or by diver or ROV (Remotely Operated Vehicle) assistance. Both forces as displacements are monitored during this process. Hydraulic couplers located on the previously installed element then pull the new element into place.

An initial watertight seal is provided by a rubber gasket. After de-watering the area between the bulkheads, the permanent joint between the two elements is made and post-tensioned to obtain full structural strength. The rubber gasket has only a temporary function during construction. This operation is similar in many respects to the placing of immersed tunnel elements. A notable difference, however, is that the support from the cable system is much weaker than that founded by the ground in the case of an immersed tunnel. This must be taken into account in planning the joining operations.

Several solutions may be used for the final connection between the tunnel and the tethers. These connections depend on the location of the connection, i.e., whether they are external or within special chambers added to the tunnel. It also depends on whether the preference is to connect the tethers first to the anchor points on the bed or to the elements before it is lowered. An example of one possible method is shown in Figure 10-21.

Remotely operated vehicles (ROVs) will be used for inspection and for installation assistance.

### Incremental construction and launching

In this method, the construction takes place at one of the abutments, which are modified and increased in size to accommodate the construction site and the associated plant, equipment and materials.

The tube is constructed in consecutive sections on an inclined skidway in the abutment. After each section is completed, the tube is moved forward into the water, over the length of one section (see Fig. 10-22) through a gate in the abutment. For this purpose, the segment is constructed on saddles and pushed forward by hydraulic jacks, similar to those used in "No-Dig" technology. Before it is pushed through the gate, the tube is coupled to the previous segments with pretension cables. The part of the tube pushed out into the water must be kept under control. A temporary cable system and/or a pontoon support system is a likely method for this purpose.

If pontoons are used for the vertical support during push-out, a shore-based cable system may be needed to keep the SFT under control in the horizontal direction. The stiffness of the cable system in relation to the stiffness at the push-out gate will require careful consideration. New pontoons are connected to the tube as it moves forward. The pontoons, which may be temporary or permanent, follow the structure across the waterway.

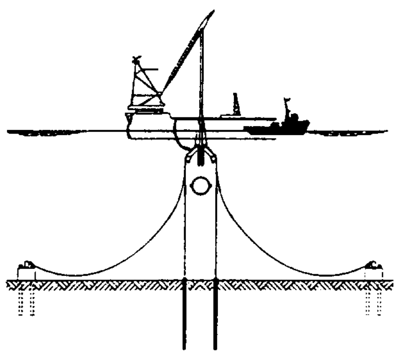


Figure 10-21. Type of installation at site.

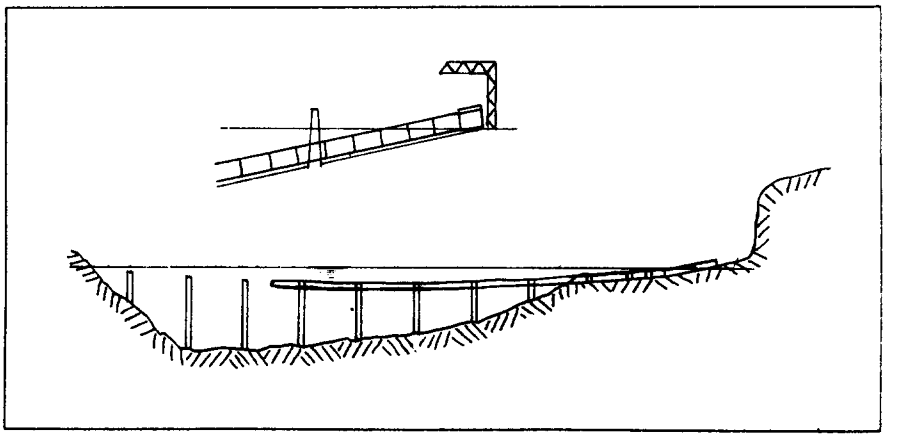


Figure 10-22. Incremental construction and launching of SFT elements.

An alternative would be to use final tethers, installed in advance and supported by temporary pontoons. To allow the tube to move, a special guidance and support system is needed at the top end of the tethers. Such an arrangement calls for ingenuity. It may be possible to install rollers, guidance plates or hydrojet bearings in a saddle, which may be temporary or which may become a part of the final construction. As the tube is pushed forward, it meets the guidance structure. The tether tension is then provided by the buoyancy of the tube.

### Element construction and incremental launching

Another construction method involves fabricating tunnel elements in a basin, and then towing them to a specially constructed push-out dock at one of the abutments (see Fig. 10-23). The element is floated into the dock and lowered on a temporary foundation. After the dock is de-watered, the element is pushed forward, coupled to the previous element, and pushed through a launching gate into the water. Support of the tunnel is similar to the arrangement described above.

## *Abutments*

The construction of the abutments depends on the chosen SFT concept, i e., prefabricated elements or incremental launching. In the case of rock abutments, either of two classic methods—the "collection chamber" method or the "concrete plug" method—can be used to connect the tunnel with the abutment. These methods are described below.

**Collection chamber method.** In this method, the tunnel entrance is made in advance and the joint is prepared to receive the tunnel. In front of the abutment, a collection chamber is made in the rock. After the preparations have been completed and the abutment has been temporarily sealed with a bulkhead, the rock between the abutment and the sea is blasted and is allowed to fall into the collection chamber (see Fig. 10-24). The area in front of the abutment is now clear and the tunnel can be placed and joined to the abutment. This joint is finished in a manner similar to joints between elements. All special facilities can be installed.

**Concrete plug method.** This method also calls for the tunnel entrance to be made in advance. At the seaside, enough rock is removed under water to allow the tunnel to be placed in the invert. After a tremie concrete plug has been placed over the front end of the tunnel, the remaining rock between abutment and tunnel can be removed from the inside. If special facilities are required, another separate joint has to be made at some distance from the abutment.

For other soil conditions, the ramps may be constructed as a concrete structure in a cofferdam. After the ramp is finished, the cofferdam is breached and the first element is joined to the concrete ramp. Finishing can be done similarly to the finishing of the joints between the elements.

For **incremental launching,** one of the ramps is used as a temporary construction site. After the full length of the tunnel has been pushed out, the ramp structure can be completed.

## *Anchor Points*

Several possible anchorage systems may be used to secure an SFT to the bed of the waterway. These include gravity anchors, driven piles and drilled piles. Selection of the appropriate system will depend on the site geology. In the case of a cable support system, anchor points are required at the bed, which may be at considerable depth.

As noted above, two concepts are available from the offshore industry: (1) gravity foundations and (2) piled templates. These two types of foundations are described in more detail below.

### Gravity anchors

Foundations based on gravity anchors may be used if the soil conditions are favorable. This type of foundation may take the form of caissons (in concrete or steel), prefabricated in a dock on land or on a floating barge, transported to the site and lowered on the sea bed using a floating crane. Ballast is then placed into the caisson to obtain the necessary weight.

### Piled foundations

Different arrangements may be adopted for piled foundations and the associated tethers. For example, a tether may be anchored by a single pile or by a group of piles; similarly, a group of piles may serve a number of tethers. The latter situation would require a prefabricated pile cap, which can also serve as a template for the piles. Given the technology and experience available in the offshore industry, no specific problems are foreseen during the installation of the templates and piles.

The template can be fabricated in steel or concrete, and incorporates pile sleeves and connection points for the tendons. Additional connection points should be provided to facilitate tendon replacement. The piles are driven from a surface vessel using an underwater pile hammer (see Fig. 10-25). This technique has been used in both the North Sea and Gulf of Mexico, in water depths of up to several hundred meters.

Attention must be paid to the tolerances that may be adopted in installation of the template. These tolerances depend on the adjustment possibilities of the tethers and the allowable tolerances in the tunnel alignment.

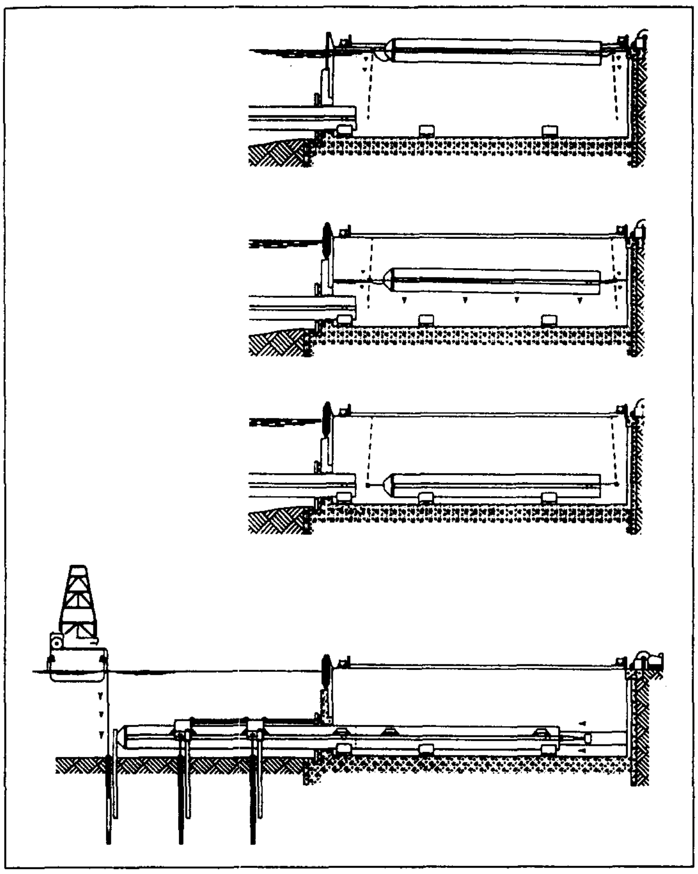


*Figure 10-23. Incremental launching.*

### Fixed supports

When fixed supports are used for the foundation, the construction method for the piers may be similar to that used for offshore platforms. If the height of the supports is in the range of 50 m to 250 m, the supports could be fabricated afloat in a deep fjord. After completion, the piers must be outfitted with a temporary extension to reach the water surface, in order to facilitate placing them in position accurately.

## *3.11 Construction Time*

The construction time will depend mainly on the following factors:

- The availability of suitable existing facilities for fabricating the units.
- Suitable space at the abutments with good access, to allow *in- situ* fabrication.
- An adequate supply of raw materials during production.
- The availability of suitable plant, equipment and manpower during both construction and installation.
- Adequate periods of workable weather, especially during the installation stages.
- The client's requirements regarding the final outfitting.

Obviously, construction time will vary from project to project, as noted below for the proposed Høgsfjord and the Strait of Messina SFTs:

- **Høgsfjord crossing:** It is estimated that the Høgsfjord SFT could be constructed in about three years—comparable to the construction period for suspension bridges of similar length.
- **Strait of Messina crossing:** Two studies of existing resources, in naval and in offshore yards, have shown that it would be possible to find sufficient materials, equipment and manpower in Italy to carry out the prefabrication of all of the required components. It is estimated that construction of the 120 elements would require approximately 1800 person-years over a period of about seven years. The corresponding estimate for the construction of the templates, piles, collars and cables is 1,600 person-years, also over a period of seven years.

## 3.12 Market expectation

Although there is undoubtedly a market for Submerged Floating Tunnels, it will be necessary to demonstrate the structure in practice. After this has been done, and when the public and politicians have become familiar with this type of structure, many sites around the world could benefit from it.

At present, the market for SFTs is limited to areas where roads have to cross deep waterways. However, SFTs can assist the environment in other locations by enabling traffic to be handled "invisibly"—i.e., out of sight of the surface.

The results of the conceptual studies for several of the SFTs described in Section 2 have been sufficiently promising to give a "green light" to the technical considerations. It is likely that construction of the first SFT will be underway by the end of the twentieth century.

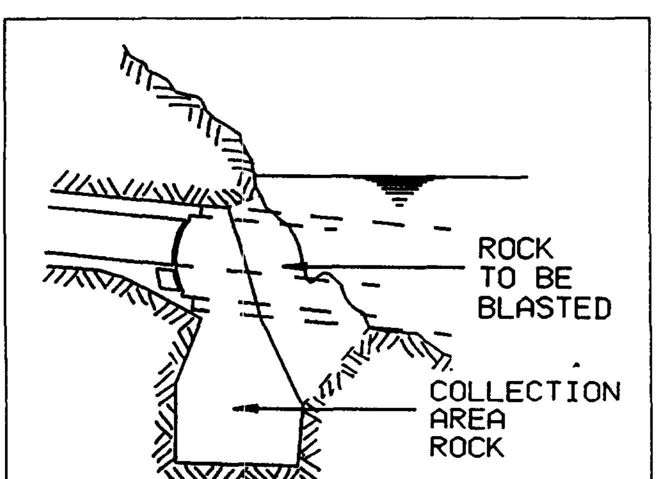



*Figure 10-24. Abutment in rock.*

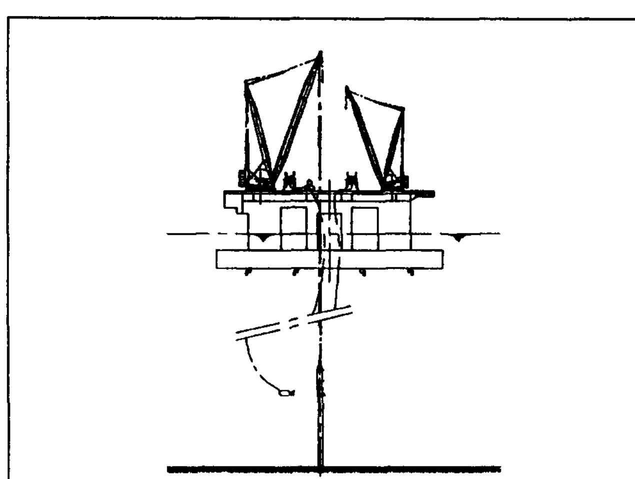


*Figure 10-25. Pile driving at great water depth.*

## 4. References

Ahrens, Donna. Submerged Floating Tunnels—a concept whose time has arrived. *Tunnelling and Underground Space Technology* **11**(4), 505–510.

Hakkart, Ch. J. A.; Lancelotti, A.; Østlid, H.; Marazza, R.; Nyhus, K. A.; and Culverwell, D. R. "Chapter 6: Submerged Floating Tunnels." *Tunnelling and Underground Space Technology* **8**(2), Special Issue on Immersed and Floating Tunnels, 265–283.

Norwegian Public Roads Administration. 1996. "International Conference on Submerged Floating Tunnels, May 29–30, 1996, in Sandnes, Norway." Oslo: Norwegian Public Roads Administration.

*Strait Crossings: Proc. of the First International Symposium, 1986, Stavanger, Norway.* Trondheim: Tapir Publishers.

*Strait Crossings: Proc. of the Second International Symposium, 1990, Trondheim, Norway.* Rotterdam: A.A. Balkema.

"Submerged floating tunnel proposed to connect Kansai, Kobe Airports." *Tunnelling and Underground Space Technology* **11**(4), 511–513.

PII:S0886-7798(97)00015-1

# Appendix A

# "SUBMERGED FLOATING TUNNELS ANALYSIS PROJECT"

## (edited and condensed version)

**Prepared by the Forum of European National Highway Research Laboratories**


*Original text prepared by:*

**GIOVANNI FERRO (Sections 1 and 8)** — **Italy**
**Consulting Engineer**

**HÅVARD ØSTLID (Sections 2 and 3)** — **Norway**
**Statens Vegvesen, Vegdirektoratet**

**FREDERICO MAZZOLANI (Section 4)** — **Italy**
**University of Napoli**

**TORE SØREIDE (Section 5)** — **Norway**
**Reinertsen Engineering**

**KJELL NYHUS (Section 6)** — **Norway**
**Aker Marine, Aker Norwegian Contractors**

**ANTONIO FIORENTINO (Section 7)** — **Italy**
**University of Napoli**

*Edited and condensed by:*

**Ahmet Gursoy**
**Parsons Brinckerhoff International** — **U.S.A.**

**Christian Ingerslev** — **U.S.A.**
**Parsons Brinckerhoff Quade & Douglas, Inc.**


*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.337–346, 1997

Printed in Great Britain
0886-7798/97 $17.00+0.00

Pergamon

# Appendix A: "Submerged Floating Tunnels Analysis Project" (edited and condensed version)


1. INTRODUCTION

2. SFT CONCEPTS
2.1 Applications
2.2 Structural Principles and Components
2.3 Design and Loads
2.4 Site Characterization
2.5 Instrumentation
2.6 Durability and maintenance
2.7 Construction methods

3. SFT BENEFITS AND COST
3.1 Environmental Features of SFTs
3.2 Competitive Features
3.3 SFT Siting
3.4 Possible sites for SFT in EU and worldwide

4. REGULATIONS

5. DYNAMICS
5.1 Dynamic Response Issues

6. CONSTRUCTION
6.1 Tethering and Anchoring
6.2 Foundations

7. SAFETY
7.1 Critical Structural Components
7.2 External Events

8. SUGGESTED RESEARCH AND DEVELOPMENT

9. REFERENCES

## Introduction

A research programme for the conceptual development of Submerged Floating Tunnels (SFTs) has been approved by the European Commission. All research and technological development activities of the European Commission are being developed according to the Fourth Framework Programme, adopted by the European Parliament and Council for the period 1994–1998. This programme is targeted to support the development and implementation of a common transport policy, with the general aim of achieving "efficient and cost-effective transport networks for goods and passengers under the best possible environmental, social, and energy consumption conditions." Included among the more specific aims of the programme are:

- *"to develop more efficient, safer and environmentally friendly transport systems for passengers and goods"; and*
- *"to promote the design and management of infrastructures with a view to reducing the damage to the environment and improving the quality/price ratio."*

Under such premises, the Danish Road Institute, acting on behalf of the Forum of European National Highway Research Laboratories (FEHRL), submitted to the European Commission an application for a grant to support an analysis on the concept of Submerged Floating Tunnels, to be developed jointly with the Norwegian Road Research Laboratory and the Italian Company Ponte di Archimede S.p.a., with the following goals:

- To produce reliable documentation about Submerged Floating Tunnels as a concept, with special emphasis on safety, economy and environmental impact;
- To identify and describe procedures involved in planning, design, construction and operation of SFTs;
- To identify and describe future research projects related to SFTs, including cost estimates and time schedules.

The project was approved by the Commission and a grant was awarded to cover half of the project budget; the remainder was covered by the three project partners. The contract was signed in March 1996 with an October 1996 completion date.

Although many studies and specific projects have been developed for SFTs in recent years, this project is the first attempt to place SFT technology in a strategic perspective in the development of European Union (EU) transportation policy. According to the project manager, "Additionally, this is the first effort based on a real multinational cooperation, which has already proven to be extremely fruitful, allowing a deep exchange of significantly different experiences."

The results of this effort, reflecting progress to June 1996, are summarized in this condensed version of the draft report by permission of the Project Manager EU, Mr. Giovanni Ferro of Italy. It is based on the June 1996 version of the draft report.

## 2. SFT Concepts

### 2.1 Applications

Submerged Floating Tunnels (SFTs) are a new transportation concept for providing shallow-depth water crossings in deeper waters. SFTs are suitable for use in crossing coastal straits, sounds, fjords, inland lakes or waterways in general, and between islands. They are essentially buoyant tunnels constrained to lie beneath the water surface within the water column, deep enough to permit expected marine traffic and ice floes to pass above (see Figures 10-8 through 10-13 in Chapter 10). As with any structure in water, they must be moored, tethered or otherwise constrained against excessive movements. Within the EU-Community, SFTs will find their most important applications in the following areas:

- Crossing of inland lakes to minimize traffic congestion along the lake shores. This is particularly important in areas of heavy traffic congestion.
- Crossings of straits, sounds and fjords along the coastlines. This will increase the efficiency of transport by replacing ferry connections and providing crossings with gentle gradients.

While much of the required technology is available, some areas still need investigation. Areas identified as needing special attention are:

- Design and production of SFTs.
- Dynamic behaviour of SFTs.
- Safety aspects.
- Operation and maintenance.
- Specific problem areas such as tension anchoring, shore connections and joints, and earthquakes and tsunamis.

The bulk of the research work in these areas has already been carried out in Japan, Italy, and Norway. Immersed tunnels, also tunnels in water, are supported either on a uniform foundation or on supports at regular intervals. This technology carries with it experience gained over nearly 100 years on more than 140 immersed tunnels. There are two basic types of immersed tunnels: concrete and steel. Either type would be a suitable for SFT application, or the two types could perhaps be merged to form a new composite type.

Combining SFT research with long experience with immersed tunnel technology, the actual construction of an SFT could start before the turn of the century. The aim of this report is to convince decision makers, i.e., politicians, road and rail authorities and their financiers, that it is possible to build an SFT.

### 2.2 Structural Principles and Components

The structural principles of SFTs are in many ways similar to those of immersed tunnels, but there are some great differences. A major difference is that the SFTs are located within the water column and may move in the water. Dynamic behaviour and fatigue therefore play an important part in SFT design that does not apply to immersed tunnels. Forces causing movement such as tide, current, wave action, traffic, and seismic events must therefore be considered in detail. Shore connections, joints, and anchoring arrangements will need to take account the dynamic nature of SFT and changes in water level.

The greatest care is needed to ensure that all potential applied loads have been considered in the design. Although unlikely, potential impact with the tunnel by a submarine or anchor may need to be considered. It is expected that the majority of SFTs will be held in position with, say, 30 m of water above the roof to allow ships to pass. Support could be provided by pontoons on the surface if negatively buoyant, or by tension legs to the bottom if positively buoyant. The tension leg can be fixed to dead weights on the bottom or to tension piles. Only in the narrowest of waterways could all supports be dispensed with between the shore connections. If supported on columns, SFTs could be either buoyant or negatively buoyant. However, attachment to columns would remove most of the dynamic aspects.

Water depths suitable for a column support are limited by the practical aspects of constructing the columns and are unlikely to exceed 100 m. Water depths are otherwise virtually unlimited. Only when suspended from pontoons could the tunnel location be directly identified from the surface, which could be important where shipping lanes must be constrained. A pontoon may also serve other useful purposes, for instance as a restaurant.

The actual structural behaviour of an SFT can be established only by monitoring a full-scale SFT. Until then, model testing and theoretical approaches must be used.

While national codes can provide the basis for the design of a structure, design methods for SFTs are not yet fully developed.

Some structural components of SFTs are listed and briefly commented on below:

**Shore Connections:** These joints will either be fixed, and have to take the forces from, e.g., tidal movements; or be free to move vertically with the tide. This applies only to sea crossings or large lakes, where movements may be important.

**Joints within Span:** All joints will have to be constructed with the dynamic behaviour of SFTs in mind.

**Anchoring to the Bottom:** This may take the form of tension legs to a dead weight on the bottom. An alternative is to install a tension pile with direct connection to the tension leg.

**Anchoring to the Surface:** Here, pontoons can be used. These may be of steel or concrete, and must have sufficient compartments to ensure buoyancy and stability in the event that some compartments are flooded, e.g., as a result of ship impact. The fixing of the pontoons to the SFT should be made through a "weak link" joint, which would save the SFT in the event of a ship collision, i.e., the pontoon would be sheared off and the tunnel tube would remain intact.

## 2.3 Design and Loads

SFT dimensions will vary according to national standards. The design of SFTs does not differ from the design of more or less similar structures. The basis for design will be found in the national codes. The design methods for SFTs are not yet fully developed and the owners/clients will have to work with consultants/designers/constructors to establish these methods. The main reference will be offshore structures and immersed tunnels. Collection of the experience gained from these structures will be important.

Because an SFT has never been built before, this is a very important topic. Design is based on presumed worst case loading, so that errors and omissions here could have disastrous consequences. In addition to the loading associated with bridges and tunnels, loading specific to this structure must be considered. Hydrodynamic forces due to waves and currents will be of primary concern, not to mention accidental loads such as falling anchors or submarines.

A circular two-lane SFT would have a diameter of about 10 m. In order to obtain durability in the face of potential fatigue problems arising from dynamic behaviour, conservative design is currently preferred.

As with similar complex projects, design, methods of construction, and the availability of suitable construction sites must be considered together. Advice from experienced contractors should be considered early in the design. Two basic construction methods have been developed so far: (1) the prefabrication of elements, and (2) incremental launching. For the latter, a cable-stay system may be needed to support the tunnel.

Safety of the structure should be covered by design codes and national approval procedures. However, because many SFT principles are new, an instrumentation, documentation and verification (IDV) procedure should be included (see Appendix B). The design of this procedure should specifically include IDV to ensure complete safety in the operational state. Monitoring programmes and reports would be expected at fairly short intervals during the first two years of operation. Operating and maintenance costs would be expected to match those of immersed tunnels.

## 2.4 Site Characterization

Site investigation data similar to data required for bridges and immersed tunnels will be required. Site-specific data for SFTs include:

- Plans, alignments, and geometrical characteristics.
- Geological and geotechnical investigations and data on seismic activity.
- Sea state, wind, waves, currents, layered currents, standing waves, and water density variations (also variations with depth).
- Ship traffic and submarine activity.

The plans will position the SFT, but geological and geotechnical information may have a decisive influence on the exact position of the SFT, both in the vertical and horizontal plane. The sea state investigations must include records from several years of wind, waves and currents.

## 2.5 Instrumentation

The instrumentation of SFTs may be divided into the following categories:

- **Control of production**: Quantities, qualities and stresses.
- **Normal operation**: SFT acting as a traffic tunnel.
- **Structural monitoring**: Stresses, strains, static and dynamic response.
- **Material behaviour**: Cracking, corrosion and marine growth.
- **Environmental monitoring**: Currents, waves and temperature.

## 2.6 Durability and Maintenance

The only valid reference, as far as durability and maintenance are concerned, stems from immersed tunnels. The aim for SFTs is to attain a similar level of durability and maintenance; however, the dynamic nature of the structure may make fatigue problems more prominent. At present, combining well-known technology and materials based on conservative design is preferred. See Appendix B for details on the proposed Instrumentation, Documentation and Verification (IDV) system.

## 2.7 Construction Methods

Experience with similar complex projects, such as those offshore, has proven that design and construction cannot be isolated from each other. Therefore, feasibility studies for SFT projects must consider both design and construction and the availability of suitable construction sites.

Site-specific conditions, such as marine environment and soil conditions, may affect both the design of the preferred structure and the construction method. It is important that structural analyses of the construction stages be performed early in order to avoid surprises later on in the project. Additional loading cases must be kept to a minimum to reduce extra costs.

The successful construction of a project with this degree of complexity relies largely on the experience of and the equipment available to the contractor. It is therefore advisable to obtain advice from experienced contractors during the feasibility study phase.

Two basic construction methods have been developed for SFTs:

**Construction in Elements**: The tunnel is fabricated in elements in a construction dock; after construction, the elements are floated out to the site and then coupled to the previously installed elements. The supports can be cables, pontoons or fixed piers.

**Incremental Launching:** The tunnel is fabricated in sections and pushed out in successive steps. Launching is repeated until the opposite shore is reached. During launching, a cable-stay system supports the tunnel.

It should be noted that the suggested methods are generic and must be modified to suit the given site requirements. Undoubtedly, the single most critical difficulty to overcome—and thereby, to obtain acceptance for SFT construction—is that this is a new type of structure. Future owners will have to be convinced that this is a safe and economical structure. Provided that the site characteristics are favourable, there are no serious technological problems to confront in constructing the first SFT before the end of the twentieth century.

## 3. SFT Benefits and Costs

An SFT is completely surrounded by water, and is located somewhere between the surface and the bottom. This arrangment enables ships to pass without any hindrance and may also provide a completely "invisible" crossing for road or rail traffic.

The main advantage of the SFT method over other tunneling methods lies in the possibility of providing a comparatively shallow depth crossing in locations where a deep tunnel would otherwise be required. Gentle gradients and much shorter lengths of tunnel could be used in locations which might otherwise be considered difficult at best, or even impossible. Gentle gradients would be attractive to rail traffic as well as cars. As an alternative for water crossings, an SFT must clearly show advantages and offer attractive environmental or economic features. The actual adoption of SFT will depend on several factors, especially:

- Presenting an SFT as an actual alternative for a proposed crossing.
- Basing the SFT presentation on known and accepted technology.
- Presenting the new concept clearly and simply to decision makers who might not be engineers.
- Paying special attention to the safety of a proposed SFT crossing, especially since the notion of having water all around when driving through the tunnel may be disturbing to many people.
- Finding a reputable owner willing to promote SFT as a desirable alternative.

In order to obtain information about possible future sites for SFTs in Europe, a questionnaire has been distributed to all possible future owners within the EU.

### *3.1 Environmental Features of SFTs*

In many areas around the world, roads have been built along sea or lake shores. As traffic increases, these roads may limit or destroy areas of natural beauty. Waters in mountainous areas are frequently deep and often represent valuable tourist interests. Roads in these areas often run along the shores, creating frequent traffic jams, much to the chagrin of the local inhabitants.

Some such waters may be deep and wide enough for an SFT to provide a viable alternate route across the water, relieving perimeter roads. Such a subaqueous crossing need not interfere with surface activities, and could link directly into subterranean tunnels, thus avoiding urban areas. Links to underground parking could also be very attractive in many instances.

Because the space occupied by an SFT usually will be small in comparison with the waterway being traversed, an SFT will not normally upset the water regime. However, areas with stratified flows may be affected, perhaps changing the mixing process between the layers and affecting fish habitats to the detriment of fish farming.

Marine growth can be expected on the surface of an SFT. This growth, its thickness, and its composition will influence structural safety, because of both the added weight and the increased hydrodynamic loading.

The impact of the SFT on the environment during construction largely depends on the construction method, and will often be limited to the impact from the construction yard.

### *3.2 Competitive Features*

Options for wider and deeper crossings include suspension bridges, cable-stayed bridges, balanced cantilever bridges, and floating bridges over the waterway, as well as immersed and other tunnels beneath the waterway bed. All of these options are characterized by sharp increases in cost as the depth and width of the crossing increases. Tunnel lengths, costs, and transportation costs increase rapidly with the depth and length of gradients, thereby making SFT competitive in these cases. This is the case for the proposed Høgsfjord crossing in Norway (see also Chapter 10).

Using tension anchor foundations, the length of an SFT crossing is limited only by cost. Using barges to support an SFT, practical aspects may limit crossings to less than 1500 m. Site characteristics especially favourable to SFT are:

- Water depth exceeds 50 m.
- Crossing lengths exceed 1000 m.
- Currents are below 2 m/sec.
- Ships passing are small.
- Submarine activity is low.
- Low seismic activity present.
- Invisible crossing is required at low gradients.

### *3.3 SFT Siting*

As discussed above, SFTs may be applicable in a number of locations where alternative structures are also feasible. In such cases, SFTs will need to be competitive alternatives if they are to be adopted. In contrast, an SFT may be the only feasible alternative at other locations.

### *3.4 Possible Sites for SFTs in the EU and Worldwide*

Possible locations for SFTs can be divided into two main groups: coastal regions and inland regions. The technicalities related to each of these locations are discussed below.

#### 3.4.1 Coastal regions

Coastal regions, which include fjords, sounds, and connections between islands, often pose the greatest difficulties for fixed crossings. Although there are possible locations for SFTs on all continents, some areas present particularly worthwhile opportunities. Examples of these are listed in Table A-1.

#### 3.4.2 Inland regions

Large lakes exist in virtually every country. In the past, for a number of reasons, many of these lakes have been considered impossible to cross. In many cases where only conventional bridges have been considered, environmental concerns have prohibited a visible structure and thereby have prevented bridge construction. The logical alternative would have been to construct a tunnel beneath the bed of the lake. However, these tunnels often become very long and expensive, or the depths are excessive.

SFTs offer the possibility of providing crossings at very gentle gradients and without visible environmental impact. In addition, they could eliminate some of the increasing road traffic around a number of lakes by providing shorter alternate routes, as well as the benefit of removing the traffic from view.

Table A-2 lists a number of lakes throughout the world that are particularly suitable for the application of the SFT concept. This list, although incomplete, reflects current opportunities identified by the authors. There are undoubtedly other opportunities in other parts of the world that remain for local authorities and developers to decide.

## 4. Regulations

The basic criteria for the design and execution of structures are governed by national and international stan-

*Table A-1. Potential inland SFT locations.*

| Country | Location of Lake |
|---|---|
| **EUROPE:** | |
| Italy | Como/Lecco |
| Italy | Maggiore |
| Italy | Lugano |
| Italy | Iseo |
| Italy | Garda |
| Switzerland | Neuchatel |
| Switzerland | Vierwaldstettersee Lake Lucerne |
| Switzerland | Zürichsee |
| France, Switzerland | Geneve/Leman |
| Germany, Austria, Switzerland | Bodensee/ Constance |
| Sweden | Vättern |
| Portugal | Rio Tejo |
| **THE AMERICAS:** | |
| Canada, U.S.A. | Superior |
| Canada, U.S.A. | Huron |
| Canada, U.S.A. | Erie |
| Canada, U.S.A. | Ontario |
| U.S.A. | Michigan |
| Nicaragua | Managua |
| Peru, Bolivia | Titicaca |
| **ASIA:** | |
| Israel, Jordan | Dead Sea |
| Japan | Biwa Ko |
| Ukraine | Azov |
| **OCEANIA:** | |
| New Zealand | Taupo |
| New Zealand | Wakatipu |

*Table A-2. Potential inland SFT locations.*

| Country | Location |
|---|---|
| Alaska | Bering Strait |
| Canada, U.S.A. | Fjords on west coast |
| Canada, U.S.A. | Superior |
| Canada, U.S.A. | Huron |
| Canada, U.S.A. | Erie |
| Canada, U.S.A. | Ontario |
| Nicaragua | Managua |
| Peru, Bolivia | Titicaca |
| U.S.A. | Michigan |
| Coast of Southeast Asia | Mainland to islands |
| Israel, Jordan | Dead Sea |
| Japan | Between islands |
| Japan | Biwa Ko |
| New Zealand | Taupo |
| New Zealand | Wakatipu |
| South China Sea | Between islands |
| Ukraine | Azov |

dards, codes, and recommendations. Changes in safety awareness and design philosophy are making these tools increasingly specialized and detailed. Such criteria emanate from Standards Institutions at different levels, e.g.:

- State or national levels:

| | |
|---|---|
| AFNORM | France |
| BSI | U.K. |
| DIN | Germany |
| NBN | Belgium |
| NBR | Norway |
| NEN | Netherlands |
| SVR | Sweden |
| UNI | Italy |
| VSM | Switzerland |
| Individual U.S.A. state standards | |

- Federal or European level:

| | |
|---|---|
| ACI | U.S.A. |
| AISC | U.S.A. |
| ANSI | U.S.A. |
| CEN - TCs | EU |

- International level:

| | |
|---|---|
| ISO - TCs | International |

Some of the most recent codes for the design of structures are European. While the current codes refer mainly to buildings, new European codes are being formulated for other structures. However, no current or proposed codes cover SFTs. There is therefore a pressing need for a similar set of rules to cover SFT topics such as:

- Choice of structural scheme.
- Choice of materials.
- Identification of mechanical models.
- Evaluation of the external actions.
- Definition of calculation methods.
- Check of materials and connections.
- Analysis of construction phases.
- Identification of the execution process.
- Choice of the erection system.

Initially, such rules can be derived directly from or refer directly to those for comparable structures, especially maritime and offshore structures, and shipping. The behaviour of materials, analysis of structures, and design of structures may also be obtained from European codes. A challenge therefore exists to develop a similar document specifically related to SFTs. If tied directly to European codes, such a document might more easily become one itself.

## 5. Dynamics

The dynamics of the submerged tunnel at Høgsfjord have been investigated. Through the initiative of the Norwegian Road Authority, dynamic effects have been evaluated by experts in load effects and in structural behaviour. The scope of these investigations was to:

- Identify the major dynamic load effects.
- Consider the quality of previous studies, including both model testing and numerical analyses.
- Propose any further research needed to cover the range of dynamic load effects.

In order to draw up a set of guidelines and requirements for the designer, it was found essential that dynamic load effects be integrated with the design specifications. It was found that participation in this process by both design engineers and experts eased the incorporation of new ideas into the specifications.

Previous detail design work on the two floating bridges in Norway has resulted in valuable experience being gained in dynamic effects on long flexible structures. A floating bridge has several modes of response similar to the response of an SFT, including the fact that behaviour during installation is quite different from that after completion.

### *5.1 Dynamic Response Issues*

The type of loading depends on both the nature of the action and the structural characteristics. Inertia effects differentiate dynamic from static responses to applied loading. Loads considered are:

- Waves.
- Current.
- Wind.
- Internal waves.
- Tide.
- Ice impact.
- Earthquake

While load effects were evaluated individually, attention was also paid to load combinations, especially waves and current. Further studies on the slowly varying response from waves and current will be carried out first by computer parametric studies, then by model tests. For extreme loads, current caused by the related effects of wind and wave must be separated current related to tidal effects. Available documents on internal waves show that these may be neglected in design. Because of the flexibility of the tunnel, analyses for earthquake should especially consider longer period excitations. Time series should be applied, to include effects of different intensities at the foundations and allowing for variations in geotechnical conditions.

Minor modifications are needed to the design guidelines to include the above phenomena. Design specifications are also to be given for the temporary phases of fabrication and installation.

## 6. Construction

### *6.1 Tethering and Anchoring*

Tethering systems are based on sea bed moorings used for tension leg platforms in the oil industry (*Proc. Offshore Technology Conference, Houston Texas, U.S.A.* 1985, 1987, 1990, 1991, 1992, 1996), five of which are currently installed.

Unlike the SFTs, such moorings are designed for a dynamic load ten times greater than the static load. Cost benefits may be realized by simplification and adaptation of the successful oil field applications, although no research programmes would be required if existing tethering systems were to be used.

#### 6.1.1 Design loads

Tethering systems are designed to remain in tension throughout their operational life. For an SFT, the tension will be static with a small dynamic component. There may also be a small angular offset (tilt) caused by current-induced displacements. It is proposed that such an offset be resisted by using a curved horizontal alignment between abutments, although the moorings could be designed to resist the offset. A typical design load for an attachment point is 5000 kN, which represents 10–50% of the design load used on tension leg platforms.

Loads during installation are dictated by the installation method and are expected to govern the design. If the installation/construction method is based on pushing the constructed parts from one side to the other (incremental launching), the mooring may have to accommodate angular displacements of up to 10° to combat friction at the sliding supports.

#### 6.1.2 Functional requirements

The design of a tether is driven by the installation method. The lower end of the component is connected to a prepared foundation structure on the sea bed through a bottom connector. The top connector has to provide for length adjustment to meet construction and installation tolerances as well as allowing installation into the tunnel connection point. The tether string itself may consist of one or more sections with intermediate connectors (Fig. A-1).

#### 6.1.3 Top connector

For tension leg platforms, the top connector is based on a threaded section at the top of the string, locked off by a nut which transfers the load into the structure. The nut entry and engagement through spinning can be rather complicated; for subsea or subsurface engagement, a split nut has been developed. With the nut segments retracted, the nut passes freely over the string. When the nut segments are engaged, they move radially to match the threads of the string. Tension is adjusted hydraulically using a device lowered onto the top of the string and engaged with the threads or a top groove. The threaded section is hydraulically pushed down to release the primary nut so that adjustment can then be made (Fig. A-2).

As on floating platforms, attachment to the tunnel may be made through forged stub cantilevers equipped with vertical holes through which the tether string passes and onto which the nut rests. To ease installation, the holes

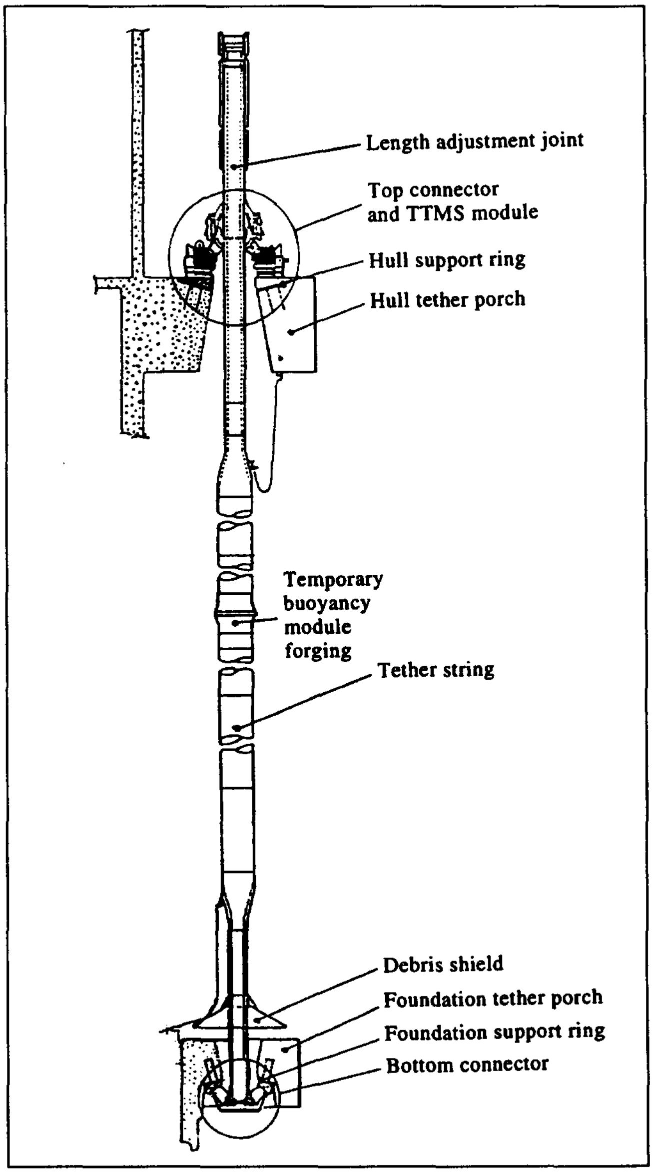



*Figure A-1. Tether system for a tension leg platform.*

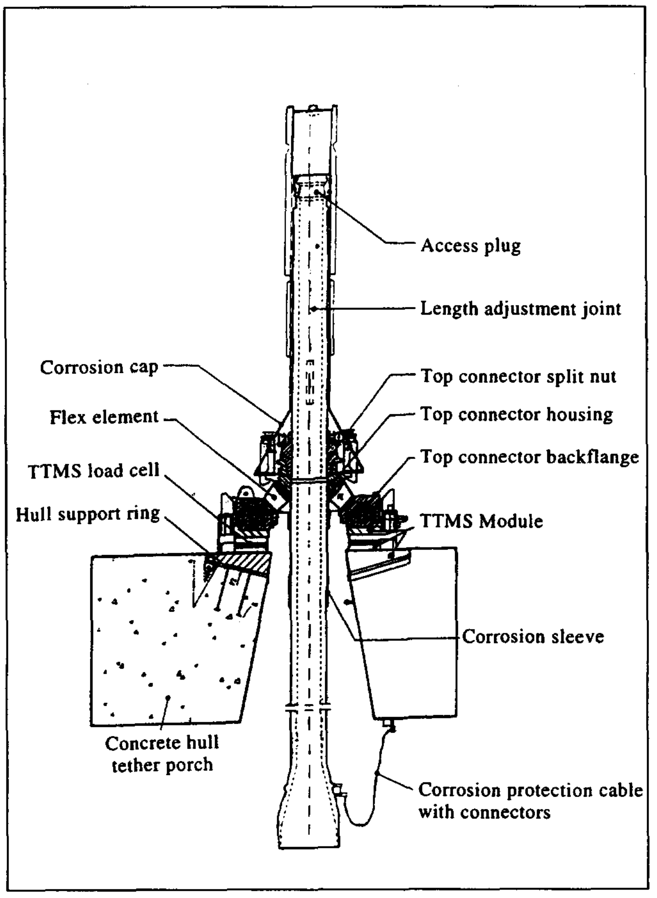



*Figure A-2. Top connector.*

may be open to one side, to allow for side entry. While forged material would provide the ductility needed to resist fatigue, castings or plate may be considered for the predominantly static SFT loads thereby reducing costs considerably.

The top connector for an SFT may be similar in function but significantly than the top connection for offshore platforms. Furthermore, if construction tolerances are accurately measured, the length adjustment feature may be replaced by a fixed length tether, shimmed to length, based on as-built dimensions.

### 6.1.4 Bottom connector

Design of the bottom connector varies with the installation method and the required angular flexibility. Two methods are used, a vertical stab-in and engagement by rotation or actuation, and side entry through an open slot in the foundation. As for top connectors, cost savings may result from using alternatives to forgings (Fig. A-3).

### 6.1.5 Flexelement

Angular flexibility at the top and bottom of the string is provided by flexelements, typically built into the end connectors. Flexelements for platforms are designed for dynamic movements throughout their life; for SFTs, requirements would be substantially less with flexing only required during installation. They consist of laminated rubber and steel, similar to bridge bearings. The steel ring load faces are spherically shaped. The working principle is that the rubber carries the load in bulk loading. This is achieved by a high shape factor, the ratio between the bonded and the free edges.

The angular flexibility is provided by shearing the rubber layers. For mooring the tunnel, flexing is only required during the installation. Flexelements may therefore be replaced by a sliding bearing, with spherical or cylindrical mating surfaces (Fig. A-4).

### 6.1.6 Tether string

Tether strings are made of pipe for platforms (Fig. A-6). Considerable research has been made into materials other than steel, such as wire rope, chain, carbon or aramide fibres. However, there is an order of magnitude increase in cost, although wire rope may offer advantages to SFTs during installation.

Threaded connectors using couplings or welds are used to form longer lengths. Welds are double-sided and ground flush so as to obtain high fatigue strength. The pipes are filled with air to avoid requirements for internal corrosion protection, to simplify inspection, and to give maximum effective tension at the top. Where tether strings are welded into one piece, installation is simplified if the string has neutral buoyancy, corresponding to a diameter to thickness ratio of 30. Stock pipe with longitudinal seam welds will provide better tolerances than spiral seams. Details of the transverse welding must provide the necessary ductility for design.

## *6.2 Foundations*

Foundations may either be gravity- or tension-piled. Gravity bases are usually ballasted to reach their required weight following installation. Three of the tension leg platforms have a pile template to provide redundancy with respect to pile capacity (Fig. A-5). The latest, Mars, has one independent pile per tether; the tether locks directly onto a receptacle on top of the pile.

Design of a gravity foundation is dependent upon the method of installation. The foundation may be lifted directly into position by a crane, or may be lowered neutrally buoyant. Gravity foundations to date have been concrete, but similar concepts would apply to steel structures. Tension leg platform foundations have skirts to carry both the

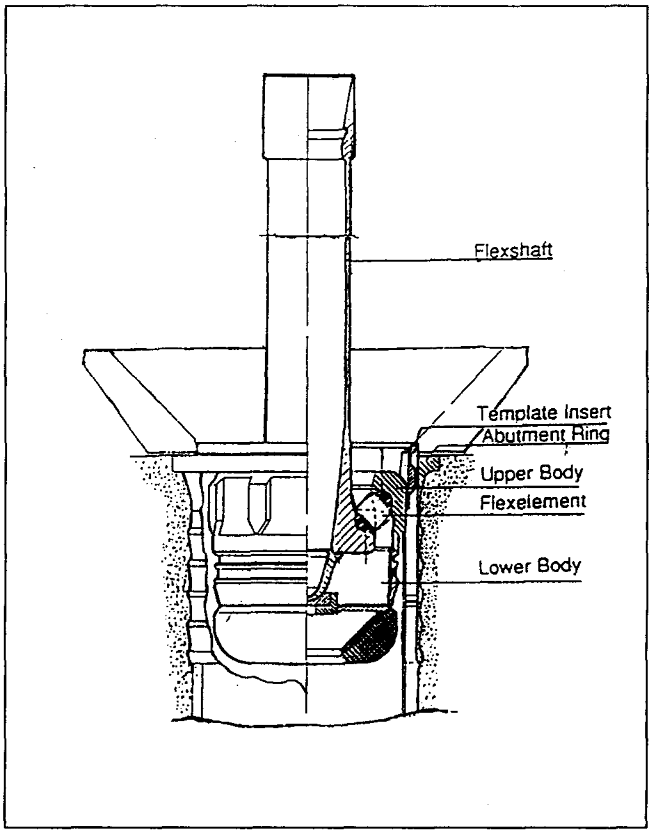



*Figure A-3. Bottom connector and anchor latch assembly.*

horizontal reactions and the high-frequency vertical variations. Field specific data at an SFT site would be used to develop and select the most cost-efficient design.

## 7. Safety

Since the concept of an SFT is still innovative, acceptable risk levels may initially need to be higher than for comparable projects because new safety issues are raised. Safety requirements should therefore be quantified to determine acceptable risk levels compared with the expected benefits during the working life of an SFT. Rational methodologies need to be established to achieve a suitable level of safety in the realization of an SFT.

The safety objective is to achieve a probability of structural failure below a prescribed allowable limit. Because SFTs are unconventional, this must be achieved not only by meeting the requirements of applicable standards, but also through specific reliability analyses, strength assessments, and simulation of appropriate risk scenarios. In this way, not only structural design requirements, but also the safety objectives may be met for the tunnel system and equipment as a whole. SFT safety criteria may be viewed in two ways:

- As a normal rail or road tunnel, emphasizing structural safety.
- As a comprehensive transportation system, emphasizing operational safety.

### *7.1 Critical Structural Components*

In order to make a safety assessment, those components of a tunnel that are critical to life safety or the environment in the event of collapse, need to be identified, i.e.:

- Tendons or tether system.
- Terminal structures.
- Connecting system between the tunnel modules.
- Anchorages and foundations.
- Outer and inner shells.

Critical design conditions should include:

(a) Extreme design environmental conditions (e.g. wave, current or seismic load with a return period comparable with or greater than the life of the structure).

(b) Operational accidental conditions.

(c) Damaged conditions due to deterioration or collapse of some of the above-mentioned primary components.

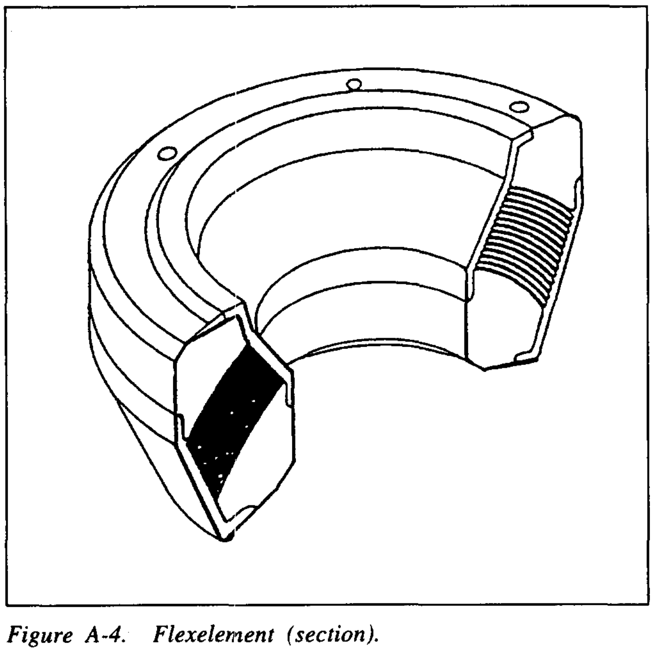


*Figure A-4. Flexelement (section).*

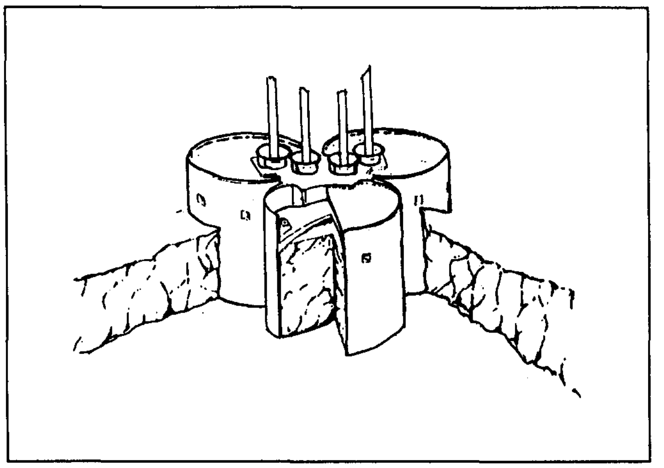


*Figure A-5. Concrete foundation template.*

### *7.2 External Events*

The probability of a ship sinking on an SFT must be considered. On the basis of expected traffic, characteristics of the ship, including displacement, impact parameters, and associated probabilities of occurrence, should be determined. For example, ships with an annual probability of sinking on the tunnel equal to or exceeding 10-4 should be selected. The analysis of a ship sinking is a well-developed state-of-the-art for stochastic models, both on traffic and on the influence of separating schemes (e.g., the work done in Denmark for the Storebælt). Such an event must be prevented in practice by using active measures such as radar, guard ships, sound and light signals, etc. Further research is required to verify the effects of a ship sinking and its impact against tunnel structures.

A sinking ship may impact either the external shell of the tunnel or a tether. The capacity of the external shell to absorb such an event by plastic deformation must be checked so that the inner structure of the tunnel remains intact. Since it is unlikely that a tether can resist ship impact, analyses must demonstrate sufficient capacity to absorb dynamic effects due to the loss of one or more tethers.

Because features of a submarine differ from those of a ship, a collision with a submarine must be considered separately. Both the velocity of impact and the probability of occurrence will differ from those of a surface ship. Although such an event would be rare, shell structures in particular are unlikely to be able to resist a submarine collision. It may therefore be necessary to preclude such an event by requiring submarines to surface before crossing the alignment, and by using radar control.

The dropping of objects (e.g., anchors or containers) happens relatively frequently. Compared to ship collision, effects on the tunnel should be negligible. SFT design should allow such events to occur without causing damage to the submerged tunnel of such severity as to compromise structural integrity and thereby lead to collapse.

## 8. Suggested Research and Development

While progress is being achieved on the Høgsfjord project, and while there is no doubt that construction of an SFT is possible, further research and development is still required before SFT technology can be used extensively. Most research needed for SFT at present is applied research. Issues of particular concern include:

- Development along the lines of European codes of regulations for the design, construction, operation and maintenance of SFTs.
- Improvement of technology for use in environments more adverse than Høgsfjord.

- Improvement of design and construction procedures and thereby optimize SFT technology, reduce costs, and standardize SFT schemes.
- Dissemination of SFT technology to both the private and the public sectors.
- Assessment of uniform and appropriate standards and criteria for safety and vulnerability.

Attaining such research and development targets requires multinational efforts coordinated by the European Commission. Interested European industrial companies are needed to participate in the project group and thereby improve their ability to compete in this market. In parallel with these efforts, demonstration of applicable technology can start. The first such application that can be used is the Høgsfjord project, since this is the most advanced in the European Union. Other alternatives, such as small-scale pilot projects, are also possible.

The five tension leg platforms installed thus, three of which are in the North Sea, have well proven the concept of tethering structures. However, conceptual work is required to adapt the selected installation method to SFT use. Simplification of tether leg system components is also required in order to reduce costs.

In considering the structural safety assessment, a critical condition would exist if a tether were to slacken, for example, as the result of a seismic event. Research is required to determine whether temporary dynamic instability conditions would be acceptable under pulsating compressive actions due to seismic loads, and what would the safety margins be. Such studies for tension leg platforms infer that if the principle is adopted, then the tether can withstand loads that are much higher than the static or quasi-static critical load. However, confirmation is still required that such mathematical models reflect actual physical behaviour.

## References


ITA Working Group on Immersed and Floating Tunnels. 1993. *Tunnelling and Underground Space Technology* **8**(2): Special Issue on Immersed and Floating Tunnels.

*Proceedings of Offshore Technology Conference, Houston, Texas, U.S.A.*: 1985, 1987, 1990, 1991, 1992, 1996.

*Strait Crossing Proceedings*, 1986, 1990 and 1994.

PII:S0886-7798(97)00016-3

# Appendix B

# INSTRUMENTATION, DOCUMENTATION AND VERIFICATION (IDV)

by

**HÅVARD ØSTLID** — **Norway**
**Statens Vegvesen, Vegdirektoratet**

***Edited and condensed by:***

**Ahmet Gursoy** — **U.S.A.**
**Parsons Brinckerhoff International, Inc.**

**Christian Ingerslev** — **U.S.A.**
**Parsons Brinckerhoff Quade & Douglas, Inc.**

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.347–350, 1997

Printed in Great Britain
0886–7798/97 $17.00+0.00

Pergamon

# Appendix B: Instrumentation, Documentation and Verification (IDV)


**1. INTRODUCTION**
1.1 Owner's Coordinating Responsibility

**2. IDV APPLICATION**
2.1 Design Stage
2.2 Production Stage
2.3 Finished Structure Stage

**3. PITFALLS OF IDV SYSTEMS**

**4. RECOMMENDATIONS**

## 1. Introduction

The topic of durability in assessing the longevity of structures has recently become a major issue, addressed primarily by owners and operators of private or national infrastructure systems. For example, the Public Road Administration of Norway has performed considerable research and development work in establishing Instrumentation, Documentation and Verification (IDV) systems to evaluate the durability of its bridges.

The purpose of IDV systems is to assess the durability of a given structure, based on obtaining reliable results from monitoring, the effectiveness of instrumentation, accuracy of documentation, and the reliability of the verifications. The IDV principles are ideally suited for Submerged Floating Tunnel (SFT) structures, and the information obtained can be of enormous value for future SFT projects.

The basic philosophy of IDV systems is the production of reliable documentation on the performance of the tunnel and associated elements, as compared to the performance anticipated in the design. This goal can only be accomplished by monitoring actual behaviour over several years. The results of this monitoring should be presented in an understandable summary for the benefit of the owner, consultant and contractor.

Where monitoring is in place, reliable information will be accumulated. It will form the basis for improved quality of ownership and will provide valuable input for future usage. The information thus established will change the rules and regulations and will show where research and development work should be concentrated. This appendix presents general recommendations for future studies, research and development incentives, in order to bring IDV application closer to the design and construction of a Submerged Floating Tunnel.

### 1.1 Owner's Coordinating Responsibility

This section presents an example in which the owner specifies the various parameters (documented and verified) after thorough discussions with the consultant and the contractor, who then jointly decide on the instrumentation. The owner has the overall responsibility for specifying the requirements, as well as the ultimate responsibility for using the collected data. However, the owner and consultant should have had full discussions with the contractor and the contractor's consultant before the final requirements are specified and included in the contract documents. The real difficulty lies in selecting the parameters to measure, picking the right time and place to perform the instrumentation, and preparing a report and documentation.

Large amounts of data may be gathered, containing not only accurate values, but also wrong values and perhaps intermediate values. Therefore, the need for documentation, to interpret the results and to select which data reflect reality, is of paramount importance. The data collection process may be divided into three basic categories: the properties of materials, the construction response and behaviour, and the environmental factors. For each of these categories, a set of tables indicating the measures to be taken and the locations will be identified, and specific requirements will be developed.

After it has been decided what to measure and where to measure, the next steps are data collection, processing, reliability control, and presentation. These steps are perhaps the most critical of all these complex processes. A list of major descriptors for each of the above-mentioned categories must be identified.

The instrumentation of an SFT may be divided into the following categories:

**Properties of materials:** Composition, and properties of material through specific stages of construction and design life; materials include concrete, steel, rubber, plastics, and corrosion of reinforcement.

**Construction behaviour:** Durability indicators such as settlement, movement, acceleration, stress and strain, forces, dynamic and static behaviour, temperature influence, etc.

**Environmental factors:** Useful data will consist of both data collected previously and that registered during and after construction. Such data may include: Soil movements, settlement (ground), wind, waves, currents, salinity and densities of water, marine growth, corrosive soils, pollution during and after construction (both inside and outside the tunnel), etc. Some of these factors are also durability indicators.

The information thus obtained will form the basis for the future development and improvement of SFT structures. For example, for measuring the development of cracks, the following properties could be a suitable topic for research:

- *Physical properties*
  - Delamination
  - Permeability (air, oxygen, carbon dioxide and water)
  - Impact / Ultrasonics
- *Visual properties*
  - Visual inspections
  - Petrography
- *Chemical properties*
  - Phenolphthalein
  - pH meter
  - Chloride content
  - Relative humidity
- *Electrical/Electrochemical properties*
  - Half cell potential
  - Polarization resistance
  - Electrochemical noise
  - Galvanostatic pulse
  - A.C. impedance
  - Macrocell probes
  - Resistance probe

## 2. IDV Application

The simplest IDV application for a new structure can be accomplished in three stages: (1) the design stage; (2) the production stage; and (3) the finished structure stage.

### 2.1 Design Stage

There are several ways of dealing with the placement of an IDV system on a structure, perhaps the simplest of which is to follow the time sequence of a new structure to be produced. IDV requirements will be developed jointly by the owner and the consultant, depending on the resources and project-critical items.

### 2.2 Production Stage

It is preferable that the consultant develop an IDV system with the selected contractor early in the contract. Additions and modifications may be required during construction. The contractor will have to meet certain specifications during both production and installation, and also must implement the jointly developed IDV system. As an example, the following can be listed for concrete production:

- **Control production**—of quantities, qualities and stresses.
- **Record temperatures of concrete** at various stages, both at the batch plant and from the time of placing onwards, at selected locations, perhaps also on a long-term basis.
- **Keep ongoing records of the formation of cracks,** from the plastic stage until the concrete has cooled after hardening. Crack detection and measurement should continue until some time after installation of the tube elements.

- **Keep records of stresses and strains** at selected locations that may be important throughout the production and operation stage, and inspect any tendons.
- **Determine the efficiency of corrosion protection.**
- **Establish initial values** for future important measured values, such as:
  - Chloride level and penetration depth.
  - Moisture content and the relative humidity of concrete.
  - Diffusion coefficients.
  - Temperature levels, both within and outside the concrete.
  - Permeability.
  - Overall position, settlement, joint movement, etc.
  - Local stresses and strains in concrete and reinforcement bars.
  - Crack measurements.
  - Concrete reinforcing bar cover.

### *2.3 Finished Structure Stage*

The use of consultants to prepare regular reports on the IDV for the five years following completion of the structure should be considered. The owner should make provisions for this in the original design contract to ensure continuity of the IDV process. These reports should also be copied to the contractor. Such reports will be invaluable to interested parties. Suitable topics would include annual reports on:

- Chloride levels.
- Corrosion levels.
- Selected crack measurements.
- Alignments and settlements.
- Joint movements.
- Vibration characteristics and dynamic response of Submerged Floating Tunnels and their critical components.

These and other measurements could be reported every year for the first five years, and thereafter perhaps only every fourth or fifth year. The whole IDV system may be incorporated into the inspection schedules for the structure. These schedules will vary somewhat from country to country, but it is suggested that an overall inspection should be performed at least every fifth year for major structures.

## 3. Pitfalls of IDV Systems

Results of the instrumentation may give rise to long and complicated discussions. A long list of "good intentions" can eliminate this problem and improve the usefulness of IDV systems. These difficulties would be eliminated were the owner to specify exact requirements for IDV reports at various stages, e.g.:

*Design stage:* A complete description of IDV proposed for the design stage, IDV for new designs, and suggestions for IDV appropriate to research.

*Production stage:* IDV for the contractor as jointly agreed, and requirements for intermediate and final IDV.

*Finished structure stage:* Clear directives as to who is doing what and when, and who is to receive reports.

In addition to directives for the stages themselves, pitfalls such as those listed below lie in the execution of the IDV itself:

*Sensors:* Selection of reliable sensors can be difficult; in general, the simplest sensors with the best reputation offer advantages. The positioning of sensors is a difficult yet very critical item, requiring optimization of available choices. Reliability of sensor output is very important to increasing confidence in measured values. Some overlapping of sensors can be beneficial.

*Electrical Fields and Sensor Output:* Most submerged tunnels carry high-voltage electrical cables and some are intended for trains. Electrical fields generated by both could seriously affect sensor output and, thus, the accuracy of measurements taken.

*Acquisition and Processing of Data:* Data flow must be kept low, with processing done as simply as possible. Simple graphical presentation of raw data will be necessary.

*Presentation of Measured Values:* Data must be presented in a simple and understandable way. Comments should be made directly on graphically presented data.

*Summary Reports:* Summary reports should be prepared for the end user, and should contain simple and clear conclusions.

## 4. Recommendations

1. An IDV system should be used for Submerged Floating Tunnels. The effect of a successful IDV system is expected to lead to overall progress in design and construction, produce high-quality structures, and improve service life considerably.

2. The ultimate aim of the IDV system should be to steadily increase knowledge through documented and verified experience in owning, designing, and constructing submerged floating tunnels. It is hoped that the instrumentation through IDV will result in savings on the designs of future submerged floating tunnels.

3. The first Submerged Floating Tunnelshould be outfitted with extensive instrumentation to monitor the SFT for safety and to verify the design. The instrumentation should be specially designed to monitor the environmental forces and the response of both the overall structure and the individual elements. Material behaviour, as well as displacements, should be monitored. Instrumentation should be used during construction and should remain operational after completion.

4. The overall responsibility of the IDV system should rest with the owner. Long-term contracts for operating and reporting the IDV results would be very useful for the entire industry.

## References

Østlid, Håvard. 1994. Instrumentation, Documentation and Verification, the IDV-System: a system for planning and using data from instrumentation to ensure factual experience to be understood and used for future projects. *Strait Crossings '94* (Jon Krokeborg, ed.), 197–200. Rotterdam: A.A. Balkema.

PII:S0886-7798(97)00017-5

# Appendix C

# "IMMERSED AND FLOATING TUNNELS"
## MEMBERSHIP OF ITA WORKING GROUP NO. 11

*Tunnelling and Underground Space Technology*, Vol.12, No.2, pp.351-354, 1997

Printed in Great Britain
0886-7798/97 $17.00+0.00

Pergamon

# Appendix C: "Immersed and Floating Tunnels"

## Membership of ITA Working Group No. 11

## Appendix C. Active Membership of ITA Working Group Immersed and Floating Tunnels*

| Name | Company/Address | Country |
|---|---|---|
| A. Gursoy (Animateur) | Parsons Brinckerhoff International Inc.<br>One Penn Plaza<br>New York, NY 10119 | United States |
| J. Saveur (Vice-Animateur) | Volker Stevin Construction Europe<br>P.O. Box 525<br>3440 AM Woerden | The Netherlands |
| W. De Lathauwer (Tuteur) | Ministry of Communications and Infrastructure<br>Résidence Palace<br>Rue de la Loi 155-b1<br>1040 Brussels | Belgium |
| S. De Ronde | SAT Engineering v.o.f.<br>P.O. Box 84319<br>2508 AH Den Haag | The Netherlands |
| H. Duddeck | Technical University of Braunschweig<br>Institut für Statik<br>Beethovenstrasse 51<br>38106 Braunsweig | Germany |
| A. Glerum | Technical University of Delft (retired)<br>Prof. Molenaarlaan 2<br>2241 RC Wassenaar | The Netherlands |
| W. C. Grantz | 4684 Crossborough Road<br>Virginia Beach, VA 23455 | United States |
| Ch. J. A. Hakkaart | Delta Marine Consultants<br>P.O. Box 63<br>2800 AB Gouda | The Netherlands |
| K. Hestner | Skanska Teknik AB<br>S-18225 Danderyd | Sweden |
| L. C. F. Ingerslev | Parsons Brinckerhoff Quade & Douglas Inc.<br>One Penn Plaza<br>New York, NY 10119 | United States |
| O. Kiyomiya | Ministry of Transport<br>Port and Harbour Research Institute, Structures Division<br>Kanagawa Yokosuka<br>Nagase 3-1-1 | Japan |
| C. Marshall | Symonds Travers Morgan<br>Symonds House, Wood Street<br>East Grinstead<br>West Sussex | United Kingdom |
| V. Molenaar | Burg. Kerstenslaan 16<br>4837 BM Breda | The Netherlands |
| H. Østlid | Norwegian Road Research Laboratory<br>Postbox 1842 DEP<br>N-0033 Oslo | Norway |

| | | |
|---|---|---|
| N. Rasmussen | Consulting Engineer<br>10 Oscar Ellingersvej<br>DK-2000 Frederiksberg | Denmark |
| G. L. Tan | Ministry of Transport<br>Public Works and Water Management<br>P.O. Box 20 000<br>3502 LA Utrecht | The Netherlands |
| C. Th. Van Doorn | TEC<br>Projectbureau Noord/Zuidlijn<br>Entrada 231<br>1096 EG Amsterdam | The Netherlands |
| J. Wang | Southwest Research Institute of China<br>Academy of Railway Sciences<br>118 Xiyuecheng St.<br>Chengdu | People's Republic of China |
| H. Wind | Philipp Holzmann AG<br>An der Gehespitz 50<br>63256 Neu Isenburg | Germany |

**membership as of May 1996*

PII:S0886-7798(97)00018-7

# CORRIGENDA

***Please note the following corrections to Chapter 7:***

| *Page* | *Section* | *Paragraph* | *Correction* |
|---|---|---|---|
| 65 | 1. | 8 | After " . . . 12 m.", add: "Concrete elements that have been transported on their own buoyancy overseas varied in length from 100 to 160 m, with widths up to 32 m and a height of approximately 9 m." |
| 65 | 2.1 | 2 | Delete "occasionally . . . navigation channels" and replace it with: "In one case, the roof of a concrete tunnel was cast partially, to reduce the draft during towing through a shallow river." Delete "The very small freeboard . . . if risks are to be minimised." |
| 66 | 3. | 5 | Delete "Since steel elements . . . always be provided", and replace it with: "On concrete tunnels, personnel hatches provide, without problems, access during towing and lowering of the element." |
| 67 | 3. | 3 | Delete "For concrete elements being transported with little freeboard", and replace it with: "Emergency pumps are an important requirement for both steel and concrete tunnel elements during towing." |
| 69 | 7. | 2 | After "anticipated sea state stresses.", add: "Furthermore, the many thin steel plate sections need to be checked for local strength and wave impact. Local strengthening will increase costs." |
| 72 | Table 7-2 | | Replace "±" with "approx." |
| 74 | 13. | 1 | After "concrete-filled keels.", add: "through inland waters and on barges overseas (except in one nonstandard case)." |

*Tunnelling and Underground Space Technology*, Vol. 12, No. 2, p. 355, 1997

Printed in Great Britain
0886-7798/97 $17.00+0.00

Pergamon