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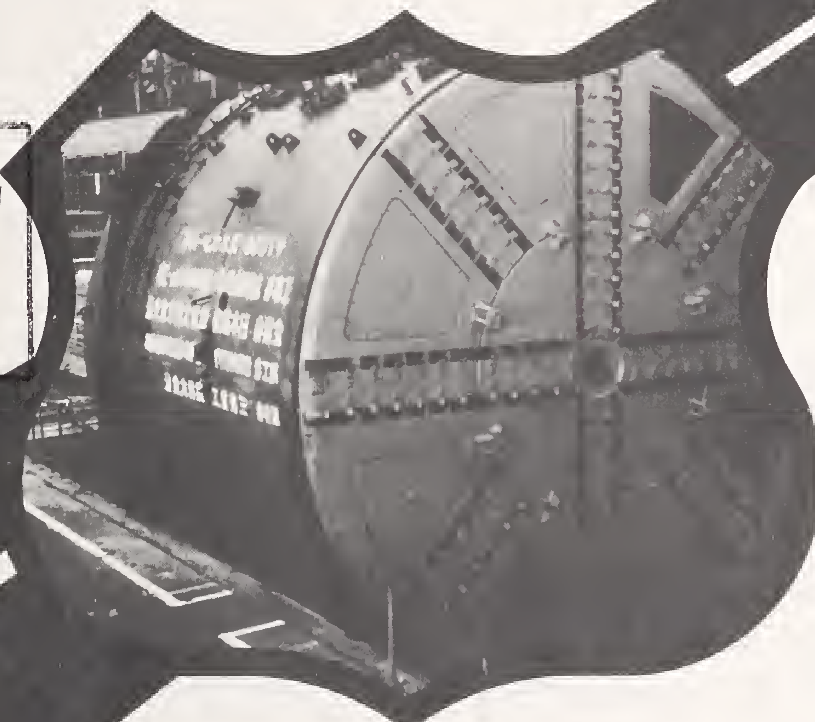
ROUNDWATER CONTROL IN TUNNELING

Vol. 2. Preventing Groundwater Intrusion into Completed Transportation Tunnels

April 1982
Final Report



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FOREWORD

This three volume report summarizes best available practices in groundwater control both during and after tunnel construction. This volume is concerned with the permanent structure. The report describes typical groundwater problems that can occur during the life of the tunnel structure. It points out good construction practices to minimize or eliminate those problems and describes effective remedial and maintenance practices.

Sufficient copies of the report are being distributed to provide two copies to each regional office, one copy to each division office, and two copies to each State highway agency. Direct distribution is being made to the division offices.

Charles F. Scheffey

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

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16. Abstract

This is the second volume of a three-volume report. This Volume 2 describes various groundwater control methods for keeping tunnels dry during the life of the structure. The methods discussed include means employed to provide impervious structural concrete, types of waterproofing membranes, methods used to seal segmented tunnel linings, grouting of soils and rock, and sealing sunken tube tunnels. Problems resulting from inadequate methods or failure of control measures are discussed as well as maintenance programs to maintain the integrity of various groundwater control systems.

Volume 1 of this report describes the state-of-the-art of groundwater control methods employed during the construction of transportation tunnels. It also describes problems associated with inadequate groundwater control during the construction stage, and methods employed in geohydrological investigations and evaluation. Legal and contractual conditions are also discussed. Volume 1 is Report No. FHWA/RD - 81/073.

Volume 3 contains guidelines for implementing groundwater control methods under varying site conditions, including compatibility of temporary and permanent measures. Recommendations for improvement of traditional and innovative control measures are also contained herein. Volume 3 is Report No. FHWA/RD - 81/075.

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PREFACE

Volume 1 of this report describes groundwater control methods employed during the construction of transportation tunnels. It also describes problems associated with inadequate groundwater control during the construction stage, methods employed in geohydrological investigations and evaluation procedures for determining proper control methods. Groundwater control method selection criteria, and legal and contractual conditions are also discussed.

This Volume 2 describes various groundwater control methods for keeping tunnels dry during the life of the structure. The methods discussed include means employed to provide impervious structural concrete, types of waterproofing membranes, methods used to seal segmented tunnel linings, grouting of soils and rock, and sealing sunken tube tunnels. Problems resulting from inadequate methods or failure of control measures are discussed as well as maintenance programs to stop leaks and maintain the integrity of various groundwater control systems.

Volume 3 contains guidelines for implementing groundwater control methods under varying site conditions, including compatibility of temporary and permanent measures. Recommendations for improvement of traditional and innovative control measures are also contained in this volume.

The reference list following the text in this Volume 2 is based mainly on topics included in this volume on permanent groundwater control methods. Not all these sources have been referenced directly in the report, but all have been reviewed in whole or in part and contribute to the report. A list of companies and agencies named in the report follows the list of references. While the naming of a product or supplier does not constitute an endorsement, the authors gratefully acknowledge the contributions of referenced works and that of the companies and agencies in these lists.

VOLUME 2

TABLE OF CONTENTS

Table of Contents (Continued)

Table of Contents (Continued)

Table of Contents (Continued)

LIST OF FIGURES

List of Figures (continued)

List of Figures (continued)

LIST OF TABLES

APPLICABLE SI UNIT CONVERSIONS

AREA

1 acre	= 4047 sq. m.
1 sq. in.	= 6.45 sq. cm.
1 sq. ft.	= 0.0929 sq. m.
1 sq. mi.	= 2.59 sq. km.

DENSITY

1 lb. mass/cu. ft.	= 16.018 kg/cu. m.

FLOW

1 gallon/min.	= 0.063 l./sec.
1 gallon/min.	= 0.00379 cu. m./min.

FORCE

1 lb. - force	= 4.448 Newtons
1 kg. - force	= 9.807 Newtons

LENGTH

1 in.	= 25.4 mm.
1 ft.	= 0.3048 m.
1 yd.	= 0.9144 m.
1 mi.	= 1.609 km.
1 mil	= 0.0254 mm.

PRESSURE AND STRESS

1 lb. per sq. in.	= 6.895×10^{3} Pa.
1 atm.	= 1.013×10^{5} Pa.
1 kg-force/sq. cm.	= 9.807×10^{4} Pa.
1 bar	= 1×10^{5} Pa.
1 kip/sq. in.	= 6.895×10^{6} Pa.
1 lb-force/sq. ft.	= 47.88 Pa.

VOLUME

1 cu. in.	= 16.4 cu. cm.
1 cu. ft.	= 0.0283 cu. m.
1 cu. yd.	= 0.765 cu. m.
1 gal.	= 0.00379 cu. m.

1.00 INTRODUCTION

1.10 PURPOSE AND SCOPE

The handling and control of groundwater inflow presents one of the most frequent, hazardous and troublesome problems encountered in tunnel and cut-and-cover construction. Although these problems are usually more severe during actual construction, if not properly resolved they can be a source of continued concern and expense throughout the life of the structure.

Volume 1 of this report deals with methods of controlling groundwater during construction. This Volume 2 discusses various methods of permanently waterproofing an underground structure to prevent water inflow during its useful life. In some cases, the construction and permanent groundwater control methods are the same. Corrective measures for leakage after construction are also dicussed.

While a certain amount of water inflow may be tolerated during construction, depending on the type of structure and construction methods employed, such a condition may not be acceptable for long term usage of the structure. The expected life span of the structure may be one hundred years or more, so temporary dewatering or other methods described in Volume 1 are often used during construction until the more stringent waterproofing methods needed for the long tunnel life are installed.

To view this work in its proper perspective it must be noted that tunnels are used for a variety of purposes:

1) for transportation of vehicular, rapid transit, railroad or pedestrian traffic;
2) for conveyance of water, sewage or almost any other liquid or liquid/solid slurry;
3) to contain utility pipelines, gaslines, steamlines, communication or power ducts either singly or in common (utilidors);
4) underground storage of fuel or water;
5) powerhouses and other underground facilities;
6) for protection in time of natural or man-made crises.

Tunnels are constructed in ground varying from soft ground to hard rock by tunneling or cut-and-cover methods. The ground can vary from compact and impermeable, to porous and wet, with an infinite recharge of water from a nearby body of water. Under the varying circumstances of use and site conditions, it is as impractical to insist that each tunnel must be perfectly waterproof and "bone-dry" as to specify that every above ground structure be built to with-stand an earthquake of 8.0 on the Richter Scale.

There is a point of diminishing returns in efforts to reduce water-resistance. It is important to consider all factors of use, construction and ground conditions when specifying the amount of leakage permitted. Transportation tunnels, for which this study is intended, require a high standard of water-resistance compared to tunnels where personal safety of public patrons is not a factor. This report is intended to aid the tunnel planner design a tunnel of maximum water-tightness possible for the type of tunnel and ground conditions under consideration. An example of a properly sealed rapid transit tunnel below the groundwater table is shown in Figure 1.

The following sections of this report will discuss, in addition to the need for waterproofing, different types of waterproofing and other groundwater control methods, their compatibility with various construction methods, the options and range of materials and methods applicable, and the maintenance required during the life of the tunnel or structure.

The groundwater control methods to be discussed in depth include:

1) design, placing, curing and joint treatment of structural concrete to insure maximum water-resistance of the basic structure;
2) various materials and application methods of providing a waterproofing envelope around the structure;
3) materials and methods for sealing various types of segmented tunnel linings;
4) other methods and aids for resisting water inflow into completed tunnels including chemically grouted envelopes around soft ground tunnels, consolidation grouting of rock tunnels, contact grouting of cast-in-place concrete linings, diaphragm cutoff walls, and permanent lowering of the groundwater table;
5) construction methods, waterproofing and sealing of sunken tube tunnels.

The section of the report dealing with selection of the most applicable water control methods for a given tunnel situation includes tables of typical current waterproofing specifications for a variety of tunnels and stations built by cut-and-cover and by tunneling in soft ground or rock. Compatibility of the various methods with respect to design, construction and temprorary water control methods and relative costs are also discussed.

The maintenance requirements section reviews design procedures to reduce maintenance. Periodic checks for leakage and the efficiency of water control methods under a normal maintenance program are discussed as well as procedures for remedial action when these checks indicate structural cracks, a failure of the waterproofing, or other defects of the water control system.

Figure 1 - EXAMPLE OF WELL SEALED RAPID TRANSIT TUNNEL (BARTD System - Courtesy of MacLean-Grove & Co.)

1.20 PROBLEMS CAUSED BY INADEQUATE CONTROL OF GROUNDWATER INTRUSION

Large quantities of water cannot be permitted to drain uncontrolledly through a tunnel, and in particular through a transportation tunnel, intended for either highway or rail vehicles. What about smaller amounts, a leak, or a drip? This section will discuss possible sources of water inflow, the problems associated with varying amounts of inflow, and current permissible leakage requirements.

1.21 Sources of Water

The most likely common source of water requiring control measures is groundwater. There are many factors that control the amount of groundwater flow into a tunnel in the absence, or failure, of waterproofing methods. The height of the water table above the tunnel determines the water pressure, and the permeability of the ground affects the possible flow quantity. The extent in depth and size of the aquifer in which the tunnel is located and its distance from a possible source of recharge (river, lake, etc.) also affect possible flow. Aquifers are typically heterogenous containing layers and pockets of material with varying degrees of permeability. Furthermore, they are usually anisotropic with horizontal permeability likely to be several times that of vertical permeability. An aquifer trapped between layers of relatively impervious materials may be an artesian body having a greater pressure than the upper water surface elevation would indicate. In the case of a tunnel driven through rock that is itself relatively impervious, groundwater will usually be able to reach the tunnel through joints, cracks, shear zones, or faults.

Perched water can exist in pockets of soft ground or in rock such as limestone or sandstone. Perched water is more likely to be a problem during tunnel construction, than in the completed structure.

In addition to natural groundwater, it is possible that water from other sources can percolate through the ground above the tunnel. For this reason even tunnels and stations above the water table will often have the roof or arch waterproofed. There are several possible sources for such water. Rainfall is one source that can be predicted with fair reliability, but other sources may be quite unexpected. A broken water or sewer line can surround the tunnel suddenly with a large quantity of water depending on the line size and location with respect to the tunnel.

Construction of nearby building foundations or other tunnels could also contribute water unexpected at the time of tunnel construction. Another possible water source is known or abandoned wells or other underground structures. In urban areas it is not uncommon to encounter industrial waste dumps or abandoned industrial facilities which add a variety of liquid or gaseous wastes, some of them hazardous, to the ground as well as water. A short section of the Massachusetts Bay Transportation Authorities' Red Line Extension through Cambridge will be constructed by cut-and-cover methods near a major chemical manufacturing facility which includes surface dumping of waste products. The groundwater in the area has a pH value of approximately 2 which is extremely corrosive and aggressive towards concrete. At the time of this writing a design solution had not been decided upon, however, unusual and costly measures will be required for the tunnel to have a resonably long service life under these harsh conditions.

While the engineer cannot quantitatively evaluate all water sources, groundwater is still the main concern. If the geotechnical information is sufficient to determine the effects of groundwater on possible leakage, the water control measures will take care of the other water sources as well, in most cases. If the water table is below the tunnel however, the engineer must be aware that these other water sources may cause tunnel leakage.

1.22 Possible Causes of Water Inflow

Two conditions must be present for leakage into a tunnel to occur. There must be a source of water, such as those discussed above, and a means of access for the water to enter the tunnel. Most tunnels are lined with cast-in-place concrete or segmented cast iron, steel or concrete linings. If proper precautions are taken in designing, mixing, placing and curing of concrete, it is relatively impervious in the thicknesses usually used for tunnel linings. Special joint treatment and waterproof envelopes can increase the water resistance of the concrete. In a like manner, segmented linings, if properly fabricated are also highly impervious and only the joints require effective sealing measures. These topics are treated in more detail in subsequent sections of this report. For now it is sufficient to note that if these linings are installed properly and any defects (honeycombing, shrinkage cracks, improperly placed caulking, etc.) are corrected, a change must occur if excessive leakage is to take place.

Changes in lining stresses due to ground loads can cause cracks in a concrete lining or joint displacement in a segmented lining. If the cracks are too wide, or joints loosened sufficiently, leakage will occur. The change in loading may be brought on by seismic forces, pile driving activities nearby, excavation of an adjacent structure or tunnel, or a change in the depth of cover over the tunnel.

Changes in groundwater conditions may increase leakage into the tunnel. Minor defects that may not have admitted much water in a tunnel with the roof above the water table may cause problems if some change such as a leaking or broken water main alters the groundwater conditions. Heavy water leakage into a tunnel often aids in deteriorating the leakage area making small cracks larger, admitting more water. If fine soil comes into the tunnel, voids may be created in the surrounding ground, or loosening may occur which increases the permeability of the surrounding soil and the possibility of surface subsidence.

Deterioration of waterproofing or joint seals over a period of time is another possible source of leakage. This could be due to poor aging qualities of the material itself, stress fatigue of brittle caulking, or the deleterious effect of chemical components of the groundwater.

Possible causes of leakage of major proportions in a transportation tunnel are, an accident caused by a derailment or collision, or a fire in a vehicle or equipment housed in the tunnel. Either of these may cause cracking, spalling or structural damage of the tunnel lining. The repair of all causes of leakage described above will be discussed in the section on maintenance.

1.23 Problems Caused by Water Inflow

Water inflows into a transportation tunnel even in small amounts can cause stains, discoloration and deterioration of architectural surfaces. Often the water will contain calcium or mineral impurities causing unsightly buildup of deposits on exposed surfaces. Loosening of tiles and corrosion of finish surface hangers or supports may also result. For highway tunnels or public areas of stations this may be highly unacceptable, while it might be a relatively minor problem in transit line section tunnels.

As water leakage increases in volume, other problems arise. Corrosion of reinforcing steel or embedded metal in concrete can occur. Leakage can interfere with a cathodic protection system and stray currents can increase corrosion further. In highway tunnels wet road surfaces or leaking water dripping on windshields can be hazardous, particularly in below freezing weather. Underground stations often house electrical substations, electrical and mechanical equipment and ventilation equipment that may be highly susceptible to water damage. Inflowing water can bring with it noxious or flammable liquids or gases if they are present in the ground.

All transportation tunnels contain drainage channels, piping and pumps to handle water even though they are constructed highly resistant to groundwater inflow. Washing of surfaces, and rainwater from station entrances and sidewalk vent gratings are some of the sources of water that must be handled. If leakage into the tunnel is too great it may overload the drainage system. This problem may be aggravated by an accumulation of dirt and debris in the drainage channels, clogging strainers, pipes or pump. If the water rises high enough in a rapid transit facility it could cause a short circuit of the third rail and become a hazard to the patrons.

Several years ago while a twin tube express tunnel was being driven beneath The Avenue of the Americas in New York City, a water main in the street above broke, sending hundreds of thousands of gallons of water into the ground below. The surge of water broke into the tunnel carrying soil with it. When the flow was finally stemmed it was found that a cavernous void of approximately 600 cubic yards (460 m^3) was left above the tunnel. Above the void four existing rapid transit tubes sat precariously, side by side, across the width of the street. Pumping cement grout into the void was begun immediately on a twenty-four hour per day basis. The rapid transit lines above were shut down for almost three weeks till the void was completely filled and surveys showed that they could resume operation. Although this happened during construction, the effect could be similar with such a flow into an existing structure. This illustration also shows how much the existing adjacent tunnel structures were affected by an unexpected flow of water.

Large inflows of water, even without carrying soil, can lower the water table in the tunnel area and affect nearby structures. Wood piles supporting above ground structures, will last 100 years or more if they are below the permanent water table, but they will rot quickly if alternately wet and dry. Lowered groundwater in or above compressible soils such as peat or organic silt, inorganic silt or clay can cause settlement due to increased effective stress with subsequent damage to utilities and structures.

1.24 Permissible Leakage

There is at present no common standard measure of permissible leakage for tunnels. This is generally determined by the engineer in conjunction with the owner on individual projects or standarized for a complete system. The two most important considerations should be how the tunnel will be used and how much it will cost to achieve the desired degree of water tightness. The latter is not always easy to determine. This consideration must take into account the size, configuration and probable construction method of the tunnel. As discussed previously the conditions of the ground and groundwater help to determine applicable control methods. When permissible leakage is specified, it is usually given in two parts, a maximum flow for a given tunnel length or inside surface area, and a maximum leakage at any given point. The latter is usually a verbal description such as, "there shall be no visable leaks", or "so dry that dust does not cling".

The Construction Industry Research and Information Association (CIRIA) of London has suggested a permissible leakage classification system in a recent report (Ref. 25). It is reproduced here as Table 1. To the original table there is an added column of units (gpm/10,000 ft.^{2}) for those readers not yet used to thinking in metric units.

Table 1 CIRIA-Permissible Leakage Classification

CIRIA Classification	MAXIMUM PERMISSIBLE LEAKAGE Litre/day/metre^2	(gpm/10,000 ft^2)
O	Nothing Visible	Nothing Visible
A	1	0.17
B	3	0.51
C	10	1.70
D	30	5.11
E	100	17.04
U	Unlimited	Unlimited

Notes:

1. A stated class is applied to define the upper limit for overall leakage flow arising in a given tunnel.

2. A stated class is applied to define the upper limit of local leaks measured over one of two standard 'square' areas on the internal surface of the tunnel, having either 1 m or 100 mm sides.

3. Examples of notation:
   A/all: B/1 = Class A overall, Class B over 1 m square
   A/all: B/100A = Class A overall, Class B over 100 mm square.

For comparison with this table, some specifications of rapid transit agencies, summarized in Section 7 of this report give a permissible leakage range of 0.2-0.4 gpm/10,000 square feet (1-2 litre/d/m^2) which would rate a "B" classification on the CIRIA scale. The CIRIA table of backup data on permissible leakage of various types of tunnels is drawn only on experience in the United Kingdom. No comparable work done in the United States has been found. This would be a worthwhile study in the near future, to justify or modify the CIRIA classification system and recommend guidelines for its use in specifying waterproofing for tunnels under varying use conditions. These could then be either accepted or modified for use on individual projects.

The CIRIA report further recommended the following definitions for descriptive terms in the event they were used in specificatons:

Damp Patch:	Discoloration of part of the surface of a lining, moist to touch.
Seep:	Visible movement of a film of water across a surface.
Standing Drop:	A drop of water which does not fall within a period of 1 minute.
Drip:	Drops of water which fall at a rate of at least 1/min. (Note: 1 litre/day is 3 to 4 drips/min.)
Continuous Leak:	A trickle or jet of water. (Note: Drips become a continuous trickle when they fall at a rate of about 300/min.)

It is recommended that these or similar measurable, defined quantities be used as they are considerably preferable to descriptive terms subject to varying interpretation by individual contractors and inspectors.

1.30 EVALUATION OF SITE CONDITIONS

The urban environment in which most underground transportation tunnels and stations are constructed, influences in many ways, the location, design and construction of such structures. To the natural soil conditions that existed in such areas, man has disturbed and altered such conditions with excavations, backfills, pile driving, rerouting water courses, and often inadvertently contaminating soil and groundwater. He has, in a more or less haphazard manner, placed piles, deep foundations, and a maze of assorted utilities in the ground. Where there are no structures, he has usually placed a mantle of concrete and asphalt which effectively hide all traces of the original soil and rock outcrops. Although man has added to the complications of evaluating site conditions, these very same activities have added to his knowledge of the urban underground environment. More stored knowledge based on previous investigations made for existing structures is available for most urban areas than for more rural areas.

1.31 Preliminary Studies

Accumulated knowledge of ground conditions available in the form of maps of original ground surface conditions, previous geophysical investigations, foundation excavation experience, and other tunnel excavation experience can be of considerable help in augmenting preliminary studies. The dense urban business districts and outlying residential areas that create the need for transportation tunnels also limit alternate route alignments to relatively narrow corridors. Unless tunnnel construction coincides with an extensive urban renewal program, economy dictates that alignment be confined primarily to existing street right-of-ways, with minimum purchase of private property. Even vertical alignment has built-in constraints. Cut-and-cover construction is most cost effective when constructed as close to the surface as possible. (Ref. 144) For tunneling, depth considerations are much less important than the type of ground. Other conditions being equal, tunneling is most cost effective in the following order: 1) All in rock, 2) All in earth, 3) In mixed face conditions. The ground surface or the top of bedrock may be steeper than allowable vehicle road or transit rail grades, limiting the alternatives even further. Added to these restraints, and of prime importance in this report, are those relating to groundwater conditions. Generally, in uniform soil conditions, the deeper the tunnel alignment below the water table, regardless of whether it be constructed by tunneling or cut-and-cover, the more costly the methods of groundwater control become. Relatively impervious soil or rock, however, limits the amount of potential water flow that must be controlled.

Important, and often irreversible, decisions of alignment, tunnel configuration and locations of stations, vent structures, etc., are made on the basis of preliminary studies. Although later detailed studies may show problems that were not previously known, special interest groups or institutional restraints may oppose changes in alignment and station locations. The general nature of the preliminary studies, and the need to review a number of alternate sites, limits the amount of field investigations performed. It is necessary therefore to locate and utilize previous studies to the extent they are available. A number of studies have been made on the types and availability of geotechnical data needed for tunneling (Ref. 8, 39, 112, 113). The U.S. Geological Survey study (Ref. 39) gives a summary of available data for selected urban areas including groundwater data. A summary of major sources of available hydrological data is given in Section 3 of Volume 1 of this report.

As a thorough discussion on the geohydrological data required for determining groundwater controls is given in Section 3 of Volume 1, it need not be repeated here except to summarize those portions applicable to the permanent structure. Acquisition of hydrological data has unfortunately not been sufficiently stressed in most preliminary studies unless unusually severe flow conditions are expected. An early preliminary evaluation of hydrologic qualities of tranmissibility, storage capacity, boundary conditions and water quality is necessary to determine the potential problems. In general the same basic data are required for determining groundwater control for the permanent structure as required for construction except that additional testing of water quality for natural or man made waste contaminants is necessary to determine corrosion potential and for the selection of waterproofing materials. On most projects the information needed to properly evaluate the need for groundwater control during construction exceeds that required for waterproofing the permanent structure.

In general, preliminary geotechnical investigations include widely spaced test borings along one or more proposed tunnel alignments. The location of borings at this stage is often determined by ease of access at approximately equal distances along the proposed tunnel routes. In addition to detailed logging of the holes, several routine types of testing may be performed. These include borehole permeability testing of soil units, pressure testing of rock units, groundwater quality testing, and the installation of observation wells. Any observation of artesian or perched water conditions should be noted.

1.32 Detailed Studies

A discussion on detailed geohydrological studies is also given in Section 3 of Volume 1, and will be summarized here. After determination of final tunnel alignment, detailed geotechnical and geohydrological studies are conducted on this alignment to augment the preliminary studies, define any suspected problems areas, to aid in final structure design, and to aid in construction. These detailed studies generally include additional routine field studies, laboratory tests, and special testing.

Routine field studies consist of additional wash borings and core borings to supplement the preliminary borings to produce a geologic profile along the proposed alignment. The number of borings will vary with the complexity of the system and subsurface conditions. In addition to supplying soil and rock samples the boreholes can be used for groundwater observations. While borehole permeability tests are not sufficiently accurate to define aquifer transmissibility they can indicate changes in permeability and aid in planning large scale special investigations. Observation wells and piezometers can be installed in completed boreholes for establishing piezometric levels in confined and unconfined aquifers, and for monitoring water levels during special testing and construction. Water samples can be obtained from the observation wells for water quality testing.

Laboratory testing for hydrological properties are performed on both soil and water samples. Soil samples can be used to establish gradation of soil units to aid in evaluating the validity of the borehole permeability tests. Extreme care must be taken in obtaining undisturbed soil samples to insure a close correlation between laboratory test results and actual field conditions.

Careful visual photographic examination of soil and rock samples can be useful in evaluating the potential variation between vertical and horizontal permeabilities of soil units and the size and condition of joints in rock units. Water quality tests should be aimed at assessing possible effects on construction methods and the completed structure. Chemicals or contaminants in the water can have adverse effects on the use of some grouts, bentonite slurries or waterproofing material. Some chemicals will speed corrosion of embedded metal in the event of leakage into the completed tunnel. Water quality tests should include conductivity, temperature and pH tests.

During the course of the primary investigations, development of the general geohydrologic profile may indicate that additional special field investigations are warranted. Where the predrainage of an aquifer may be required, pumping tests of sufficient size and duration to provide data for proper planning and design of the system may be called for. If grouting, freezing, dewatering by electro-osmosis, or difficult cut-off wall construction are being considered, a test section may be warranted. Each situation must be considered individually. While the costs of such special investigations are comparatively high they must be weighed against the alternative of not having the information to be gained. If the designer is faced with insufficient data his waterproofing design will necessarily be conservative. If the contractor is faced with insufficient data to plan his construction operation it will most likely add to the contingency costs in his bid. The planner must consider these factors when such special investigations are being assessed.

Based on the results of the preliminary and detailed geotechnical investigations and other available data, the planner must design the tunnel or underground structure to be structurally safe, watertight, and as maintenance free as possible during its life span. Depending on the tunnel use and general construction methods, the designer will specify one or more groundwater control measures as detailed in the next five sections of this report. The criteria used in selection of the most appropriate control methods for individual projects is reviewed in Section 7 following the descriptions of methods.

2.00 WATER-RESISTANT CAST-IN-PLACE CONCRETE LININGS

2.10 GENERAL

2.11 Purpose

Cast-in-place concrete tunnel linings often serve as the major permanent support of the ground load. In tunnels through sound rock capable of maintaining a stable opening, rock bolts with or without shotcrete may be sufficient for reinforcing the rock and for preventing local spalling. A concrete lining may still be specified if a smooth finish is required for hydraulic properties in water tunnels, or for aesthetic purposes in public areas of transportation tunnels. If groundwater is present and a dry tunnel is required, the lining must also resist hydrostatic pressure. The increase in the quantity of concrete required to withstand this pressure may be great and some designs provide for pressure relief by drainage outside the permanent lining. The long term effectiveness of such drainage systems is subject to proper design and construction procedures for the tunnel, ground conditions, and groundwater conditions of a particular project. Railroad tunnels usually have high vertical walls which offer poor resistance to high water pressure. As standard practice the AREA Manual for Railway Engineering recommends (Ref. 4):

"6. Drains

> "Wherever groundwater is encountered, vertical and diagonal openings, trench drains, tile or iron pipe drains shall be installed between the concrete lining and rock. Provide adequate outlets through sidewalls with the outer end of the outlets not less than 12 in. above the bottom of the gutter. Provide subdrains under the concrete floor wherever groundwater is found. Provide drains through curb to drain ballast section.
>
> "Wherever groundwater drains are installed, they shall be sealed to the rock so as to prevent being clogged when conrete is poured."

The invert slab must either be sufficiently strong and heavy to resist the uplift pressure of the groundwater or contain weepholes to permit the flow of water through the slab and then into the tunnel drainage system as has been done in some sections of the New York City subway when a six-inch slab is used on rock inverts (Ref. 93).

2.12 Lining Thickness

For tunnels in either soil or rock that have no groundwater infiltration problems there are various minimum concrete thicknesses suggested. An old rule of thumb for rock tunnels is 1 inch (25 mm) of concrete for every foot of tunnel diameter but not less than 8 inches (200 mm) (Ref. 88). Concrete thinner than 8 in. (200 mm) is difficult to place. If drill and blast overbreak averages about six inches (150 mm) the actual average minimum thickness of concrete in a drill-and- blast hard rock tunnel would be 14 inches (350 mm).

For railway tunnels with a drainage system outside the lining to reduce water pressure, AREA recommends thicknesses of 14 to 15 inches (360 to 380 mm) for tunnels 16 to 29 feet (5 to 9 m) wide by 25 to 30 feet (7.5 to 9 m) high if placed against timber or steel primary supports with lagging or liner plates. If placed against rock, the recommendation is 18 inches (500 mm) for the narrower tunnels and 24 inches (600 mm) for the wider ones (Ref. 4).

Modern tunnel designers recognize the overdesign of the rule of thumb determinations. With better site investigations and new approaches to design, considerable savings can be made. The designer using the predesign site investigations produces a conservative original design based on the worst anticipated conditions. Using field determinations of the actual ground conditions surrounding the now existing opening, a more economical design can be prepared and utilized if appropriate (Ref. 10).

In keeping with these new approaches the United States Water and Power Resources Service (formerly the Bureau of Reclamation), uses lining thicknesses from 8 inches (200 mm) to more than 3 feet (900 mm) for tunnels 17 to 21 feet (5 to 6.5 meters) in diameter (Ref. 10). California's Department of Water Resources varies the thicknesses of concrete for a given diameter tunnel also, but does not use so wide a range.

In Ontario, Canada, the thickness of the secondary lining for tunnels in soft ground requiring an initial support system is generally 1 to 1-1/2 inches (25 to 37.5 mm) per foot (300 mm) of finished tunnel diameter and is unreinforced (Ref. 56).

When the tunnel must be kept dry the concrete lining must be made strong enough to withstand the hydrostatic pressure as well as ground loads. R. B. Peck (Ref. 102) demonstrates the magnitude of this change. The illustration assumes a 20-foot tunnel built in fair to good quality rock at a depth of 2500 feet (760 m). Steel sets are used to support the rock load and 12 inches (300 mm) of concrete, compressive strength 4,000 psi (30 MPa), would be placed to protect the steel from corrosion, the water draining around the tunnel into a drainage system. When this is changed to an impervious lining the required thickness becomes 32-1/2 inches (830 mm) an increase of 170%. He further states, that were the lining shotcrete instead, the corrosion protection coating could be 4 to 6 inches (100 to 150 mm) thick, but if the shotcrete lining were to support the hydrostatic load, at a thickness of 32-1/2 inches (830 mm), the increase would be 550%. Although most transportation tunnels are relatively shallow, they may pass under hills or through mountains where high water pressures must be considered.

2.20 STRUCTURAL DESIGN FOR WATERTIGHTNESS

If a cast-in-place concrete tunnel lining must have the strength to resist the hydrostatic pressure, it should, preferably, be impervious thus serving as a major deterent to the infiltration of the groundwater. This is possible with today's concrete technology, manufacturing techniques, and equipment. To place such concrete, however, requires rigid control of all materials and processes involved. This can add greatly to the cost and prove difficult in an adverse tunnel environment.

2.21 Where Leaks Occur

Water can pass into a concrete structure through joints, cracks, honeycomb, and the pores within the concrete itself. To produce impervious concrete all joints must be carefully and permanently made watertight, design must be such that cracks do not develop, honeycomb must be prevented by careful placement, and all pores capable of transmitting water must be filled or otherwise closed off from the groundwater.

2.22 Structural Design

To accomplish these things the subgrade and foundations of cut-and-cover tunnels must be designed and prepared to prevent settlement cracks. If this cannot be done an elastic membrane must be used to bridge the settlement cracks when they occur. The structure itself must be designed to support, without cracking, all loads to which it may be subjected from the start of construction through the period of its expected usefulness.

For tunnels in competent rock ground support requirements may be minimal and the concrete lining may carry little or no rock load. For tunnels in soft ground the support is usually rigid and the strength requirements can be calculated with adequate accuracy (Ref. 88). Design of tunnel linings in poor quality rock is much more difficult. During and after excavation, the rock mass will loosen and may collapse if not supported quickly. If relatively rigid steel rib supports are placed they must be sufficient to carry the entire rock load. A more flexible support such as shotcrete, with rock bolts if necessary,will permit the redistribution of rock loads and aid in stabilizing the tunnel opening. This must be considered during the design of the permanent lining of such a structure. With the recent research efforts toward continuous tunnel construction, with concreting of the lining following immediately after the excavation, these higher loads must be provided for to avoid failures. Use of a preliminary lining of more flexible shotcrete that allows redistribution of loads is precluded by this type of construction. In any type of ground, if the concrete lining is to prevent groundwater infiltration, the permanent lining is more likely to be free from cracks if it is placed after the ground movement has become minimal (Ref. 14).

Since concrete is weak in tension and reinforcing steel must carry the tensile loads, the use of at least twice the minimum longitudinal reinforcing steel required by ACI for shrinkage is recommended (Ref. 15). This decreases the likelihood of shrinkage cracks and decreases the stress in the steel.

2.30 DESIGN OF CONCRETE MIX

The design of watertight concrete should follow the best practices used in the design of high quality structural concrete. The aggregate should be clean, sound, not reactive with the cement, free from organic and other deleterious materials and sized appropriately for the section to be placed. The aggregate should be well graded so that the voids in the combined aggregate are at a minimum. The portland cement should be of the type best suited to the ground conditions, type of structure, and conditions of placing. The water should be free from delterious amounts of akalies, acids, chlorides, oils and organic materials. Any admixtures used should be compatible with the other constituents of the mix. The water-cement ratio should be as low as possible consistent with good workability.

All conditions which must be considered in designing for durable concrete in the situation in which the watertight concrete is to be placed must be considered in the design. If, as might be true near the entrance to a tunnel, the concrete will be subject to freeze-thaw cycles the mix must be designed to withstand these. If loads will have to be carried soon after placement, a high-early-strength design will be needed.

Besides these requirements for high quality structural concrete a few other requirements must be considered.

2.31 Porosity

For water to flow through hardened concrete there must be interconnected pores large enough for it to pass through. Concrete consists of the cement paste, which may include some admixtures, and the aggregates. If the hardened cement paste is porous, obviously the hardened concrete will be porous. But also, if the aggregate is of porous rock and not completely sealed by the paste from the groundwater the hardened concrete will be porous. This porosity does not necessarily affect the strength of the concrete.

Aggregates

Ideally the aggregates should come from rock of very low porosity. This is especially important if the cement paste is somewhat porous since it reduces the effective area through which the water can flow and increases the length of the path thus reducing the rate of flow (Ref. 89).

Cement Paste

The cement used in the mix should be a fine-ground cement since studies have shown that the porosity of fully hardened cement paste, for a given water-cement ratio, is greater for coarse-ground cement than for fine-ground cement (Ref. 89).

The water-cement ratio used in the mix design is of prime importance in controlling the porosity. The lower the water-cement ratio the lower the permeability (Ref. 45). Admixtures which improve the workability of low water-cement ratio mixes may be used to good advantage. Air entrainment admixtures do so and are especially useful because they increase the density and reduce segregation and bleeding (Ref. 116). The entrained air bubbles help to keep the aggregate particles from touching so that they are truly surrounded by the cement paste thus sealing off any pores in the aggregate particles.

2.32 Shrinkage Resistance

Resistance to shrinkage must also be considered in designing the mix. Two types of shrinkage occur during the hardening of concrete. One is due to the drying out of concrete and the other to reduction in temperatature.

The less water incorporated in the mix the less there is to dry out. Therefore the lowest water-cement ratio that will permit good placement should be used. The water-cement ratio can be reduced by the addition of chemical admixtures such as Plastocrete by Sika or, if the available aggregates are deficient in fines, the addition of finely divided materials.

Another means of controlling shrinkage is to use an expansive cement or an admixture which causes expansion of the concrete during hardening. This expansion compensates for the shrinkage due to drying.

Thermal shrinkage occurs because the temperature of the concrete as it hardens is greater than the service temperature of the hardened concrete. This may be due to the temperature of the material when mixed and placed or due to the heat released by the chemical reaction during hydration. The amount of this shrinkage can be reduced by chilling the plastic concrete before placing, by replacing a portion of the portland cement with a material such as pozzolan which will act as a cement but does not produce as much heat during its hydration, using as little portland cement as possible while still meeting the requirements for strength, impermeability, and workability, spraying cooling water on the forms after the concrete has been placed, and curing with cool water sprays (Ref. 15). Where extreme measures are required to minimize temperature shrinkage cracks cooling water can be piped through the new concrete lowering the temperature at the center thus keeping the entire pour at a more uniform temperature (Ref. 73). While the reference by Janssen describes this method being used for prefabricating sunken tube tunnel sections, no reference has been found of its use for a cast-in-place lining.

2.33 Chemical Resistance

To construct a watertight tunnel not only must it be structurally adequate and watertight at the time of completion but, ideally, it should remain so throughout its useful life. For this reason, before an appropriate concrete mix design can be prepared, it is necessary to determine the environment, both external and internal, to which the concrete will be exposed during its life. By altering the constituents of the mix it can be made resistant to many of the aggressive chemicals present in soils and groundwater. For instance, the use of a Type II or V portland cement will offer resistance to sulfate attack, or Type I may be used with the addition of pozzalons that have been field tested and found suitable.

To determine the external conditions a careful site investigation is helpful, but during excavation the ground conditions should be monitored so that the mix design can be altered when conditions change. It is also advisable to study surrounding conditions which, through the years, might alter the impurities carried by the groundwater. Resistance of the concrete to the external environment is extremely important since the exterior of the tunnel is inaccessible once it has been completed.

The internal environment will be the result of the use to which the tunnel is put. A tunnel used by motor vehicles will contain exhaust gases and dripped oil and gasoline. One used for electrically run vehicles will be subject to strong electrical currents. Just as tunnels which had been constructed for horse-drawn vehicles later became tunnels for coal-burning or gasoline-burning engines, so new tunnels designed for today's equipment may well be used for very different equipment a hundred years from now. We cannot design for this change but since the internal surfaces are accessible they could probably be treated or coated to protect them from any new aggresive conditions so that designing for the anticipated use should be adequate.

The compatibility of the concrete with any other material to be embedded in it must be considered. If a chemical reaction between the two is possible, it is likely that it will deteriorate the concrete. Aluminum, for example, reacts principally with the alkali hydroxides in the cement. The resultant compound occupies more space that its constituents and therefore it can crack and spall the concrete. Iron when sufficient oxygen is present, is also expansive in concrete. Aluminum items must be coated but iron need not be unless oxygen will be available at the location of the iron.

2.40 PREVENTION OF CORROSION OF REINFORCING STEEL

To maintain the integrity of the concrete for the many years of service expected of a transportation tunnel corrosion of the reinforcing steel must be prevented. The iron oxide produced occupiesa much greater volume than the iron that has been oxidized, so the concrete around the steel will crack. If this corrosion of the reinforcing steel continues for a long time the tensile strength of the section will be reduced.

If corrosion of the reinforcing steel is taking place it is generally first noticed as a rusty line on the surface of the concrete parallel to the steel. The corrosion product, having a greater volume, is cracking the thin section between the bar and the surface and some of the rust has seeped through. Generally the alkaline nature of portland cement deters the oxidation and adequate cover of the steel is required by drawings or specifications to keep water and oxygen away from the steel. If, however, the aggregate used is too large or too little vibration is used, the cover on the bars may be porous. This must be prevented by good design and placement practices. The graph of Figure 2 shows the relationship between the thickness of cover over reinforcing steel and permeability of concrete.

The higher the stress in the steel the more rapid the corrosion. To minimize the effects of corrosion, intermediate grade steel should be used with higher than the theoretical design area. This also aids in the reduction of shrinkage cracks.

If ground conditions are particularly conducive to corrosion of the steel it may be necessary to use cathodic protection.

2.50 JOINT DESIGN

Every joint in the concrete is a potential leakage path. Therefore, for watertight concrete, particular care must be taken to make all joints watertight. Since this is expensive, it is advantageous to design the structure and the concrete mix to reduce the length of the joints to a minimum.

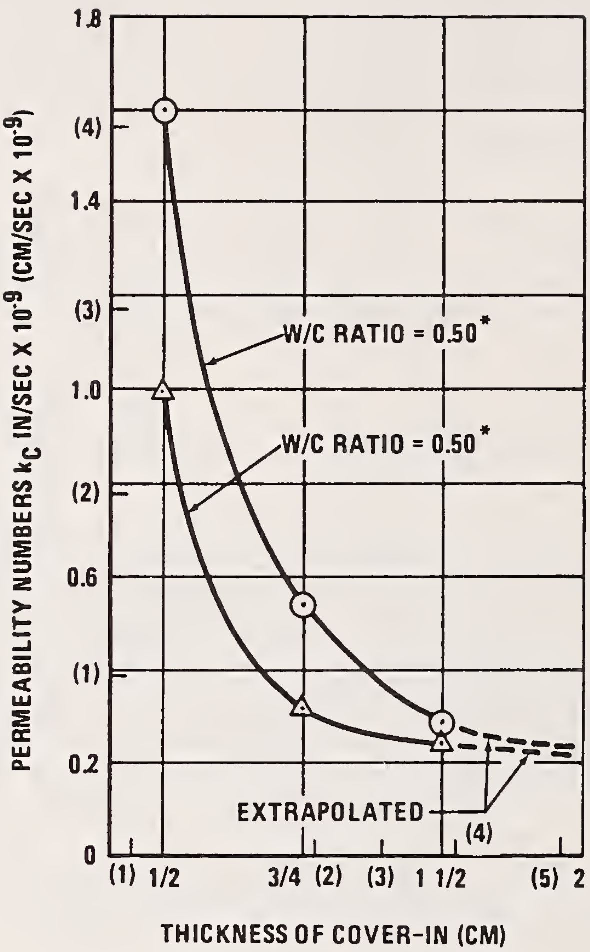


Figure 2 - THICKNESS OF COVER AND PERMEABILITY
(From Birkmyer, Ref. 14)

2.51 Types of Joints

Large concrete structures such as concrete lined tunnels are usually divided by joints to facilitate construction and to prevent the formation of cracks due to drying shrinkage, creep, temperature stresses, water pressure and differential settlement. There are three types of joints used to accomplish this.

Construction joints are joints required by placing limitations. Slabs and the walls above them are usually placed separately with a construction joint between, which will, hopefully, function as though no joint were there. Also it is generally impractical and unwise to place long longitudinal pours. A long pour is more likely to develop shrinkage cracks, but a well designed concrete mix with good control of the concrete from design through curing will make longer pours possible.

In cut-and-cover structures of the rectangular box type there is, commonly, a longitudinal construction joint between the invert slab and the walls and frequently between the walls and the roof. In bored tunnels if the walls are high and straight as in railroad tunnels there may be a joint between invert slab and side walls, and side walls and arch. If the tunnel is horseshoe in shape and not high, the joint just above the invert slab will probably be all that is required. Concrete for circular tunnels is sometimes placed without any longitudinal construction joints. If the concrete is shrinkage controlled this is the ideal situation, reducing the joints to a minimum.

Contraction joints are joints which are designed to open when the concrete shrinks due to drying or a decrease in temperature. In other words they control the location of shrinkage cracks. In waterproof construction they must be bridged with an elastic membrane, such as a waterstop, to prevent the flow of water.

Expansion joints are joints designed to be open under normal temperature and at least partially closed when the temperature of the concrete increases. These are intended to prevent excessive compressive stress in the concrete. In tunnels these are seldom necessary because temperature variations are usually small (Ref. 4). However, near the tunnel entrances temperature variations may be sufficient to require expansion joints.

2.52 Waterstops

Waterstops are metal, rubber, or plastic strips placed in joints to prevent the flow of water when the joint opens. Metal waterstops are rarely used today (Ref. 119). The type and magnitude of the anticipated joint movement and required chemical resistance determine the materials used and the design of the central portion of the waterstop. The ability of the material to withstand extensive elongation and return essentially to its original configuration is important. The materials used in the waterstops must retain this ability to a sufficient degree to fulfill the waterstopping function for the anticipated useful life of the structure. Materials are sometimes tested for the retention of this ability after artificial, accelerated aging. A report on natural rubber indicated a life of 100 years at 20°C and 400 years at 10°C (Ref. 44).

Various cross sections have been developed in an attempt to make the edges of the strip hold in the concrete when contraction of the concrete opens the joint. The size of waterstops is usually between 3 and 12 inches (75 and 300 mm) wide and 3/32 and 1/2 inch (2 and 13 mm) thick.

The configuration customarily used for rubber waterstops is the dumbbell profile. This has a round bead at each edge and may or may not have a hollow center bulb. Polyvinyl chloride (PVC) waterstops usually have ribbed flanges with or without a hollow center bulb. If considerable differential movement is anticipated a waterstop having a thin, weak longitudinal section can be chosen (Ref. 119). When movement occurs the bulb splits forming a U-shaped configuaration and a differential movement of approximately the bulb circumference can be accommodated without rupture of the waterstop. Typical configurations of metal, rubber and PVC waterstops are shown in Figure 3.

STUVA, the Studiengesellschaft fur unterirdische Verkehrsanglagen e.V. (Research company for underground traffic systems) in Koln, West Germany, has developed and tested new configurations for improving the sealing properties of the dumbbell profile. These include a cross shaped edge consisting of 3 tee-shaped arms in place of the round edge bead, ribbed edge bead with a hard core, and ribbed hollow bead into which material could be injected after placement, developing positive pressure against the hardened concrete (Ref. 108). The latter design makes possible a good seal even where small defects exist in the concrete around the bead.

If waterstops are to do the job for which they are intended they must be properly installed. During the first concrete pour the waterstop must be secured so that its center line is centered in the joint, its embedded edge secured so that it will not be shifted during placement of concrete and its extended half is protected from concrete, curing compounds and all other foreign matter. The sealing of joints between pieces of waterstop and the proper making and sealing of intersections, corner turns, and other changes of direction is very critical. The extended half must be supported so that it will not curl under when concrete is poured against it. This can be done by wiring the edge to the reinforcing steel or the forms every 12 inches (300 mm) or as required by the conditions of placement.

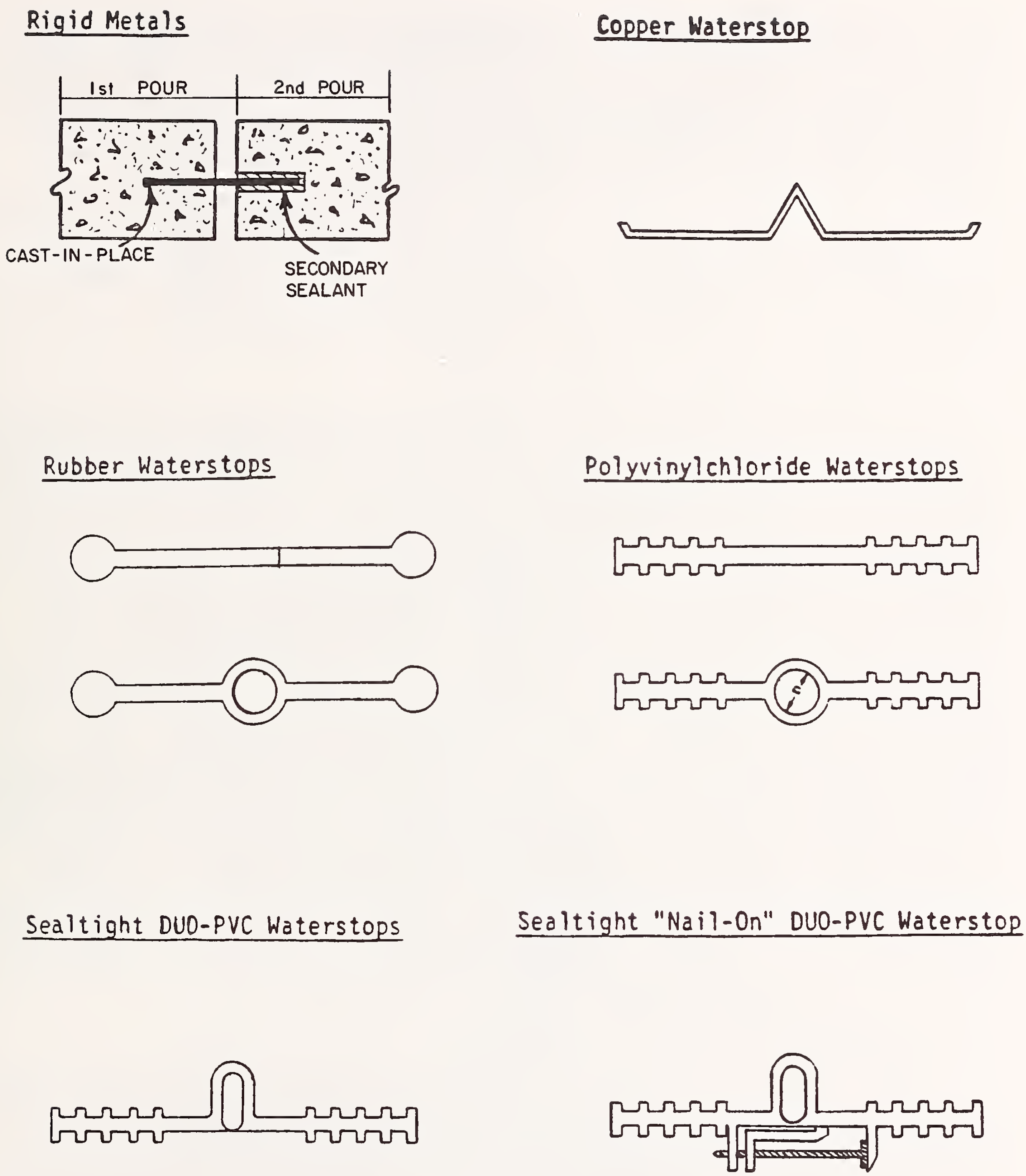


Figure 3 - TYPICAL WATERSTOP CONFIGURATIONS
(From Stilling - Ref. 119)

For horizontal joints where the concrete is dropped upon the extended position it is sometimes recommended that a layer of grout be placed over the waterstop just prior to the concrete placement to prevent excessive movement and to insure against honeycombing and voids. A special form of waterstop has been developed for such joints (Ref. 88). It's much shorter and wider than those discussed above with keys to grip the concrete. It is less susceptible to deflection by the second pour.

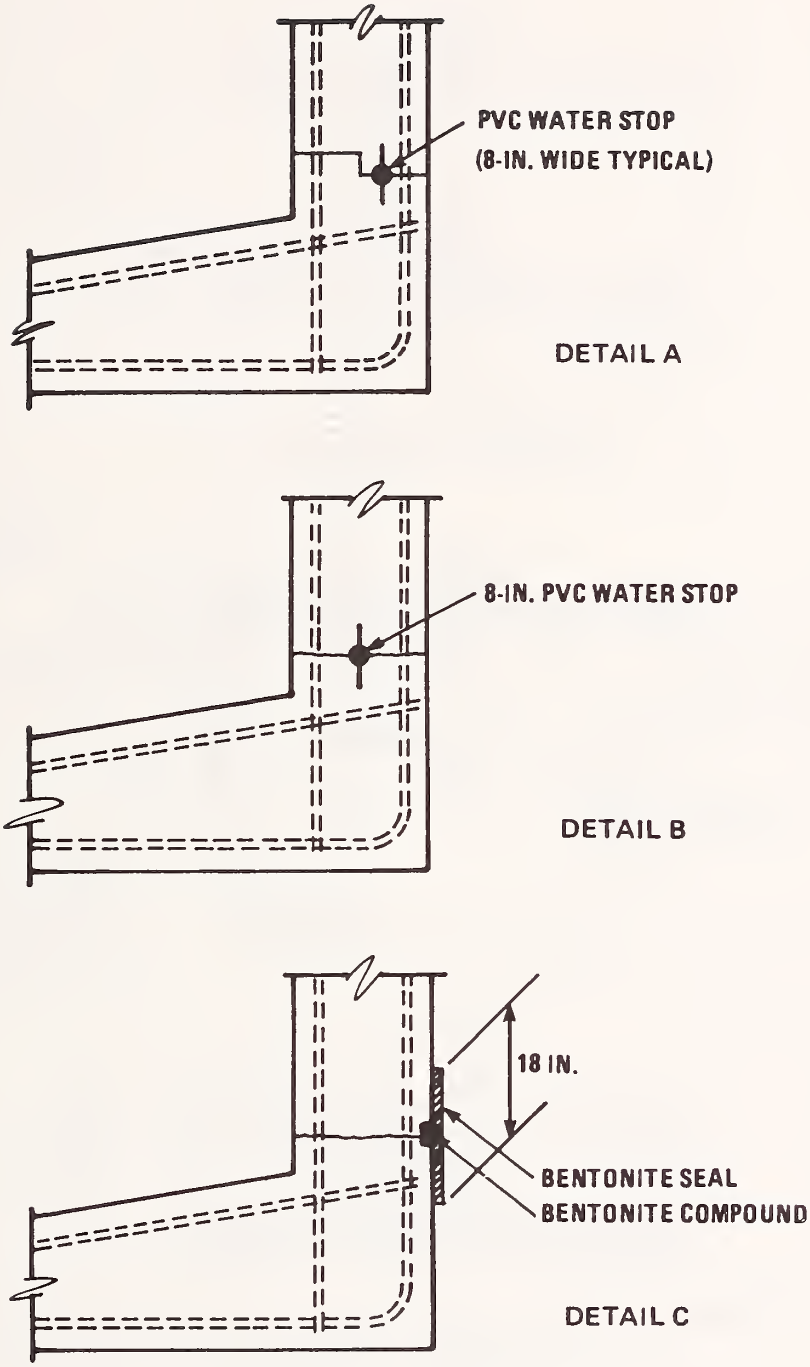
The concrete around the waterstops must be sufficiently vibrated to avoid honeycombs and voids and to insure complete bonding of the concrete to the waterstop.

2.53 Design of Construction Joints

If the concrete is reinforced the reinforcement must be continuous through construction joints. If the fresh concrete can be placed so that it bonds perfectly with the hardened concrete, the joint will be watertight without any additional materials. Since this is difficult extra precautions are usually required. These may include of the installation of a waterstop across the joint or a bentonite backing outside the joint. In cut-and-cover construction the bentonite backing may be applied just before the backfilling operation. A channel is blocked out across the joint during placement of both concrete pours and later filled with bentonite compound or tube, and a bentonite strip 18 inches (450 mm) wide, or as required, is centered over the joint and secured. Then the backfill is placed. This detail can be used for the longitudinal joints and the wall and roof construction joints. With slight modification this joint can be used even when concrete is placed against the excavation or excavation support. The bentonite application is simpler to apply than a waterstop and has the advantage that it is generally self-healing and will fill small cracks in the concrete itself whereas if the waterstop is overstressed and fails repair is difficult. Typical construction joint details are shown in Figure 4.

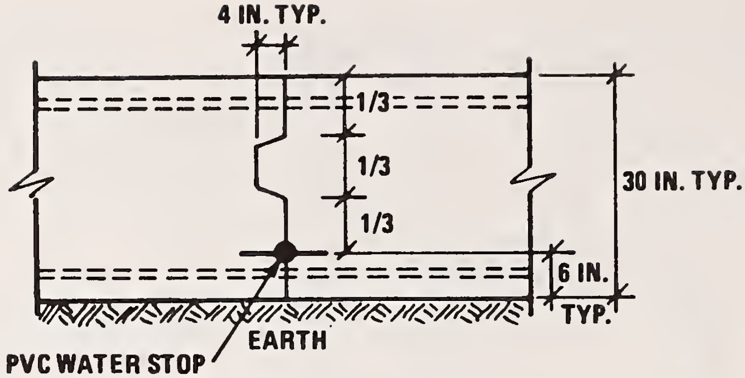
2.54 Design of Contraction Joints

Contraction, or control, joints are intended to permit the concrete to contract without the development of random cracks. If reinforced, the reinforcing steel crossing the joint is treated with a bond breaker so that as contraction occurs there is less tensile strength at the joint making it a favorable location for a crack to occur. The weakness of this plane is further enhanced by chases along each face of the concrete as shown in Figure 5. These chases are then waterproofed. The exterior chase may be treated with bentonite panel and compound as described for construction joints and the interior by fillers and sealants or by expansive caulking yarn and a layer of keeping yarn (Ref. 14).

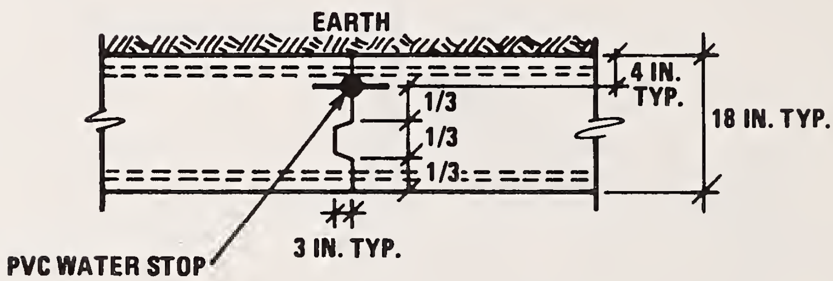


LONGITUDINAL JOINTS (1 mi = 25.4 mm.)

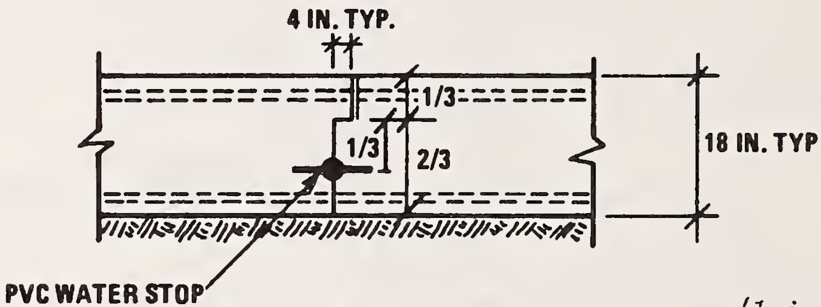
Figure 4 - CONSTRUCTION JOINT DETAILS
(From Birkmyer, Ref. 14)



DETAIL A – INVERT JOINT



DETAIL B – ROOF JOINT

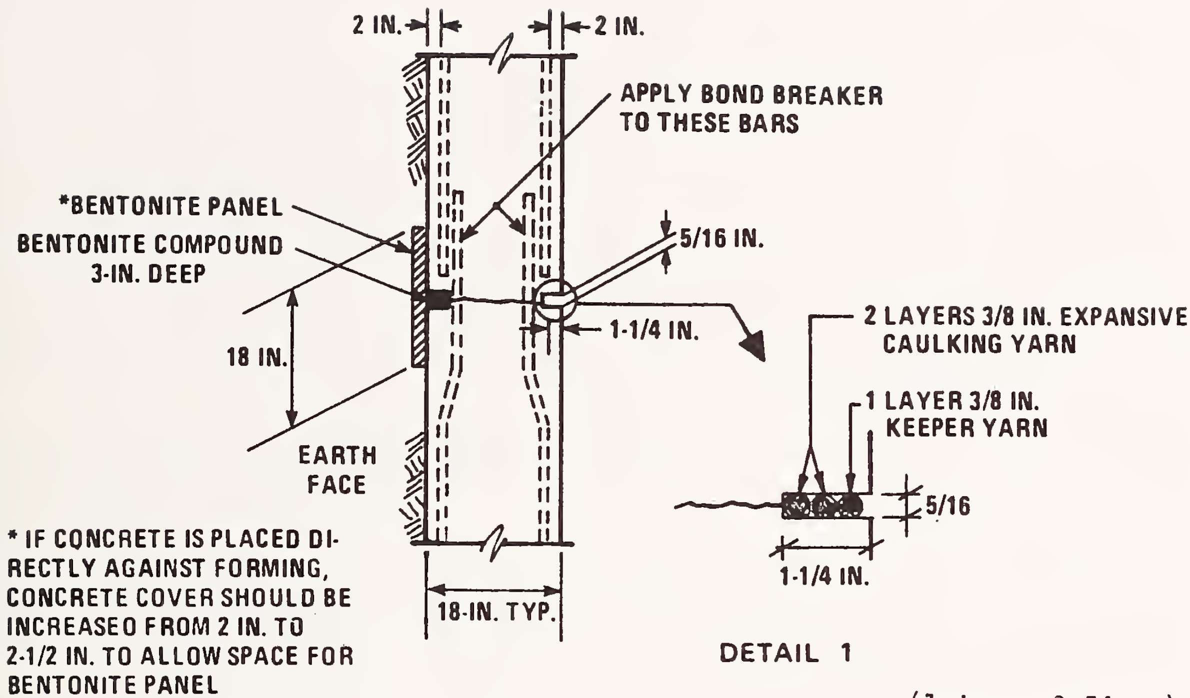


(1 in. = 2.54 cm)

DETAIL C WALL JOINT

TRANSVERSE JOINTS

Figure 4 - CONSTRUCTION JOINTS (continued)
(From Birkmyer, Ref. 14)



(1 in. = 2.54 cm)

Figure 5 - CONTRACTION JOINT DETAIL
(From Birkmyer, Ref. 14)

The distance between contraction joints is dependent on the design of the concrete mix and the anticipated control of the mixing, transporting, placing, and curing. They should be placed so that no large cracks occur rupturing the waterstop or permitting the bentonite to wash through the crack.

2.55 Design of Expansion Joints

In long runs of concrete where normal temperature ranges are great it is necessary to leave a space from time to time so that as the temperature of the concrete increases the compressive stresses in the concrete do not become too great. No reinforcing steel crosses an expansion joint. Usually the first pour is made and when ready for the second pour a premolded joint filler is placed against hardened concrete and fresh concrete is placed against the joint filler. When completed, a joint sealer may be applied at the surfaces. Then as the concrete expands the joint filler is compressed reducing the compression stresses that would otherwise develop in the concrete. Many joint fillers are resilient and return to their original thickness as the temperature drops. However, if an expansion joint is required below the water table a waterstop should be used to prevent leakage before the filler has returned to its proper dimensions or in the event that the filler is overstressed and breaks down. It is very important to be sure the center bulb is not bonded to the concrete and is centered in the joint.

Temperatures in underground structures below the water table rarely vary greatly so the protection provided by a waterstop may not be needed even where an expansion joint is considered advisable.

2.60 MIXING AND TRANSPORTING THE CONCRETE

The same mixing and transporting practices needed for highquality structural concrete are needed for watertight concrete.

2.61 Mixing the Concrete

Thorough mixing is extremely important since it improves plasticity resulting in better placing and distributes the constituent materials uniformly. The fine and coarse aggregates must be mixed to produce as compact a mass as possible with the cement paste, including any admixures used, completely surrounding each aggregate particle. During hydration of the portland cement considerable heat is generated. To prevent temperature shrinkage cracks it is well to chill the constituent materials so that the temperature of the curing concrete will not appreciably exceed the normal ambient temperature of the structure in use (Ref. 13). This cooling is also helpful if the weather is hot, especially in cut-and-cover construction.

2.62 Transporting the Concrete

During transportation of the concrete to the point of placement, segregation must be prevented. The means of doing this are well established. In cut-and-cover construction there are no more difficulties than in deep building foundations.

In under-the-roof construction and bored tunnels the transporting of concrete becomes more complicated. Except in very large and long tunels where a portable batch plant may be placed in the tunnel itself the mixed concrete must be conveyed from portal or shaft of the tunnel to the location of the pour. This is usually done by hopper car, agitator car, or pumping. If a hopper car is used, and the distance is long, or there is a delay in placing, the aggregates may settle producing a poor quality, permeable concrete. If the time from addition of water to the dry ingredients to the time of placement is excessive the plasticity of the mix will have been reduced and the addition of water to restore the plasticity will produce a more permeable concrete. Pumping concrete long distances tends to dry out the mixture and an admixture may be required to aid the flow.

2.70 PLACING AND CURING THE CONCRETE

2.71 Placing the Concrete

Before placement begins the same precautions should be taken as with any high-quality concrete.

1. If placed against the ground surface there should be no projections into the design volume of the concrete.
2. There should be no standing water within the area of placement.
3. Running water should be collected and carried to a drain so that it will not mix with the concrete.
4. All debris must be cleaned out of the placing volume.
5. Waterstops extending from a previous pour should be examined for damage and adequately supported to prevent dispacement during the pour.
6. When pouring against hardened concrete, the hardened concrete should be well roughened, cleaned, moistened, and treated with cement mortar just before placement.

During placement the precautions still resemble those for high-quality structural concrete.

1. Place carefully along waterstops so as not to bend them.
2. Place in a manner that will not cause segregation of the large aggregate. This causes porous concrete.
3. Deposit concrete as nearly as practical in its final position.

4. Place in shallow enough lifts so that they can be well compacted and if internal vibrators are used the top portion of the plastic lift below can be reached by the vibrators.
5. Vibrate the concrete to compact it thoroughly using internal vibrators or vibrators attached to the forms or both. Compaction eliminates rock pockets and large air bubbles, unifies each lift with the still plastic lift below, completely surrounds the reinforcing steel and other embedded items with the concrete, and moves enough cement paste to the faces and the top surface to seal these surfaces.
6. Use special care in compaction along joints and around the reinforcing steel, and other embedded items.
7. Once pouring has begun on a section it should be completed without interruption.

In bored tunnels one of the most difficult parts of the lining to place is the crown.

2.72 Curing the Concrete

To obtain watertight concrete moist curing is essential. If the concrete is allowed to dry before sufficient hydration of the cement has taken place drying shrinkage will occur. A minimum of seven days of water or fog curing should be required. Steam curing hastens the development of strength but retards the drop in permeability. In fact, without the application of fog curing after steam curing the permeability may still be unacceptably high at the end of 28 days (Ref. 89). Furthermore if the partially hydrated cement paste is dried beyond a certain point permeability will increase, probably due to shrinkage cracks within the hydrated cement paste forming passages between capillaries in the paste. For this reason after moist curing has been completed the concrete should be protected from excessive drying conditions such as dry air draughts through the tunnel. During the hydration period moist curing with cooled water can also help to keep the temperature of the concrete down to prevent temperature shrinkage (Ref. 13). Figure 6 shows the effect of curing on permeability.

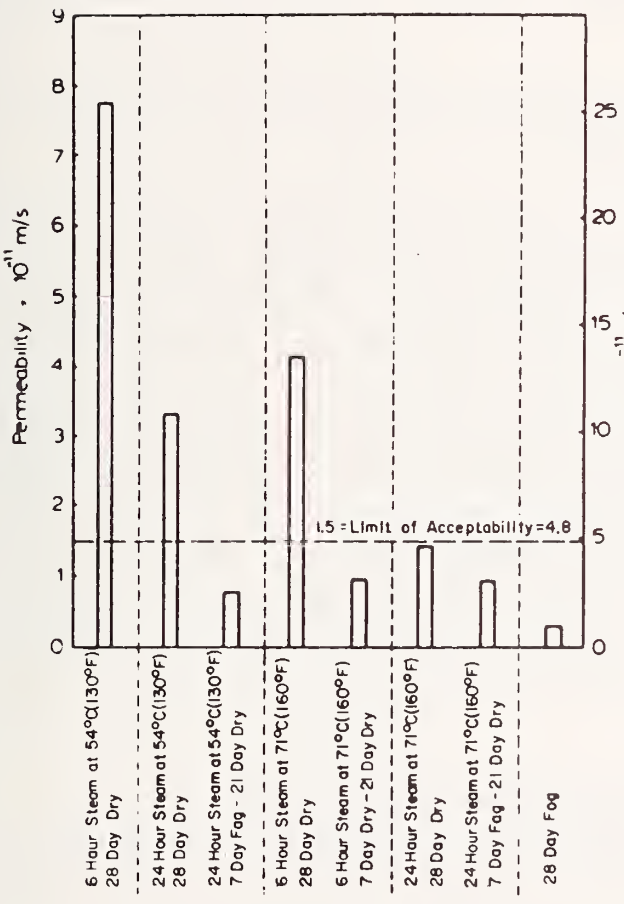
2.80 REPAIR OF DEFECTS

Control of the concrete is easier in cut-and-cover construction than in under-the-roof construction or tunneling. In the latter, working conditions make placement of concrete difficult. If a cut-and-cover project is covered by a continuous traffic deck, transportation and placing difficulties may be close to those encountered in tunneling. Even in well-supervised cut-and-cover work without continuous decking there can still be defects which can endanger the impervious integrity of the concrete such as honeycombing, bolt holes, pop-outs, and cracks

RELATION OF CURING TIME TO PERMEABILITY*

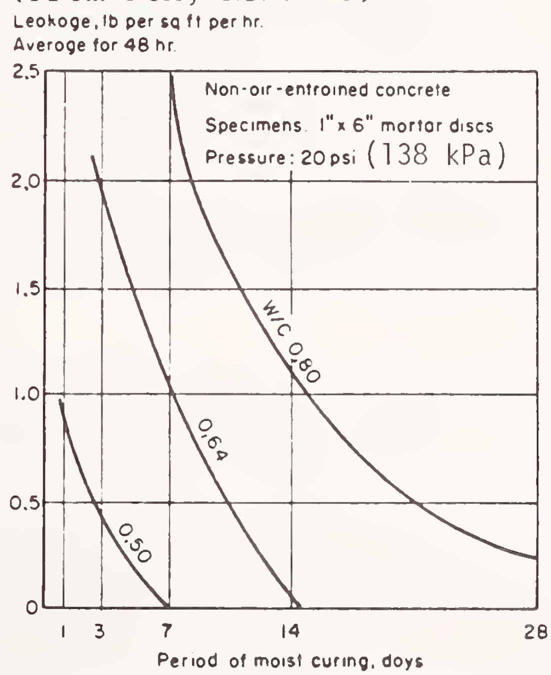
Days of Curing	Coefficient of Permeability
Fresh Paste	1,150,000,000
1	36,300,000
2	2,050,000
3	191,000
4	23,000
5	5,900
7	1,380
12	195
24	46

*Source: Corrosive Metals in Concrete, ACI Publication SP-49



Permeability of Steam Cured Concrete (After Higginson, 1961)

(From PCA, Ref. 45)



Effect of water-cement ratio and curing on watertightness. Note that leakage is reduced as the water-cement ratio is decreased and the curing period increased.

Figure 6 - EFFECT OF CURING ON CONCRETE PERMEABILITY

in the cured concrete. To ensure watertightness of the finished structure these defects must be repaired. Surface damages which occur during construction must also be repaired. If the defect is visible its location and nature are obvious but if it is disclosed merely by a wet inner surface or dripping water determination of its nature may be more difficult. The location of the leak, however, indicates where the porous path reaches the surface so the defective concrete can be exposed and removed.

The repairs must be thoroughly and permanently bonded to the sound concrete, be as impermeable as the sound concrete, be free of shrinkage cracks, be as resistant to freeze-thaw cycles, sulfate attack, and meet any other criteria required of the original concrete for the structure (Ref. 141).

All unformed surfaces in the original structure should be finished so that the surface is hard and smooth and any cracks which develop must be sealed. Cracks will usually have to be widened before filling and sealing.

All formed concrete surfaces should be carefully examined for defects as soon as the forms have been stripped. If possible, the necessary repairs should be performed within 24 hours since the bond with green concrete is generally stronger than that with fully hardened concrete (Ref. 141). Tie holes and pop-outs must be filled and sealed, all porous concrete (honeycomb, aggregate segregation) must be removed and replaced with sound concrete, and all cracks must be sealed. The American Concrete Institute and the Bureau of Reclamation have published procedures for the repair of surface defects which are quite comprehensive. The procedures cover preparation of the surface to which the patch is to be applied, types of patching materials, their applications, finishing, and curing as necessary. That such a procedure be carefully followed is of paramount importance if the concrete is to be watertight (Ref. 1, 31).

Various resins such as epoxy and polyester as well as synthetic rubber and bituminous materials used in the repair mortar have proved helpful in bonding and sealing of concrete repairs (Ref. 141).

If water is actually flowing into the structure, measures must be taken to channel the flow to a drain. If the water comes through the concrete obviously the concrete is porous in that area and this concrete must be removed. Then the water can be panned and piped to a drain, the area patched around the pipe, the patch cured and then fast-setting grout pumped under low pressure into the pipe to fill and seal the space behind the lining and the pipe itself filled level with the inner surface of the lining.

If water leaks through a joint in the concrete, the repair procedures taken should be suitable to the type of joint and the reason the joint is not watertight.

2.90 INNOVATIONS

For many years a length of tunnel could be lined with reinforced concrete more rapidly than it could be excavated; now, due to new developments in tunneling techniques, excavation often outstrips the lining placement. Studies are being performed to develop techniques for more rapid concreting using new developments in concrete technology. These include regulated-set portland cements, expansive portland cements, polymer-impregnated concrete, polymer concrete, ferro-cement, and fiber-reinforced concrete.

2.91 Regulated-Set Portland Cements

Regulated-set portland cements have been investigated to permit more rapid advance of slipforms. Regulated-set cements are portland cements modified to develop early usable strengths. These cements require special handling. Usually citric acid is added to the mix and the mixing water is cooled nearly to freezing to extend the handling time. The permeabilities of conventional mixes using these cements range from 1.0×10^{-10} m/sec to 3.0×10^{-10} m/sec. Mixes designed as pumpable, low-void-volume mixes can reduce the permeability to approximately 6.0×10^{-11} m/sec at 14 days when fog curing is used (Ref. 62). One great disadvantage of the regulated-set portland cements now available is that the handling time, even with set retarders, is extremely short. The handling times are as short as 15 or 20 minutes (Ref. 61) which is very useful for highway patching and similar work and adequate for casting-shop and laboratory work, but too short for the transportation difficulties and uncertain conditions in most tunnels.

2.92 Expansive Cements

One of the drawbacks with portland cement concrete is the development of cracks in the hardened concrete due to drying shrinkage. To reduce the cracking it has been traditional to place large concrete areas in a "checkerboard" sequence so that plastic concrete is placed against hardened concrete which has already done any drying shrinkage it is subject to. The plastic concrete fills the volume of the shrinkage of the earlier placed concrete.

In recent years larger and larger areas have been placed without shrinkage cracks using some expansive cement in the mix. These mixes can be simply shrinkage compensating or they can actually prestress the concrete if it is confined.

In tunnels where the lining must be placed shortly after the tunnel is excavated, such as in soft ground, this checkerboard arrangement can not be used so the expansive cements could be extremely useful.

2.93 Polymer-Impregnated Concrete

Hardened concrete can be impregnated with a suitable monomer and that monomer polymerized either by radiation, thermal-catalytic techniques, or a combination. A polymer-impregnated concrete (PIC) can also be produced by adding the monomer to the concrete mix and subsequently polymerizing the hardened concrete. Polymerizing changes the properties of the concrete. To illustrate this, Table 2, reproduced from "Concrete-Polymer Materials, First Topical Report", is shown on the following page. This table shows some of the outstanding alterations in the properties of the concrete so treated (Ref. 118).

All the strength values are higher, water permeability is reduced to zero, chemical resistance is greatly improved, and the freeze-thaw durability is improved. This is only one of many possible polymer-impregnated concretes. The properties of concretes containing different polymers vary considerably. One grave drawback is the effect of high temperatures on the strength properties. A study of concrete impregnated with polymethyl-methacrylate (PMMA) showed an 80% loss of compressive strength after heating to 500°F (260°C) and cooling to room temperatures, and a 95% loss if heated to 1500°F (816°C). The polymer decomposed at 1000°F (538°C). From analysis of the products of combustion it was concluded that PMMA-PIC did not pose a serious health hazard when exposed to moderate or severe fire conditions (Ref. 28). Regulated-set concretes retain 30 to 40% of their original strength when heated to 660°F (350°C) and cooled (Ref. 62). This temperature is used as the temperature of a moderate to severe tunnel fire condition (Ref. 64). So in fire conditions the regulated-set concrete functions somewhat better than concrete impregnated with PMMA.

When loaded as a beam, polymer-impregnated concrete reinforced with steel fibers in lieu of reinforcing bars can sustain much greater loads with a given deflection than unimpregnated concrete. When it fails, it fails in a plastic manner and the steel fibers themselves break rather than pull out so the entire strength of the steel is utilized (Ref. 9).

Though polymer-impregnated concretes possess unique structural and durability properties the cost of producing them is very high since, in addition to costs of producing high-quality concrete, there is the cost of monomer handling, impregnation, and polymerization equipment. At present it is still suitable only for precast items though some consideration has been given to impregnation of existing concrete structures to improve structural and other qualities so impregnation of cast-in-place concrete could be feasible in the future (Ref. 9).

Table 2

Summary of Properties of Concrete-Polymer Material

(From Steinberg, et al, Ref. 118)

	Concrete control specimen (type II cement)	Concrete with up to 6.7 wt % loading of polymethyl methacrylate Co^{60} gamma radiation polymerized
Compressive strength, psi	5,267	20,255
Tensile strength, psi	416	1,627
Modulus of elasticity, psi	3.5×10^{6}	6.3×10^{6}
Modulus of rupture, psi	739	2,637
Flexural modulus of elasticity, psi	4.3×10^{6}	6.2×10^{6}
Coefficient of expansion, in./in.-°F	4.02×10^{-6}	5.36×10^{-6}
Thermal conductivity at 73°F (23°C), Btu/ft-hr-°F	1.332	1.306
Water permeability, ft/yr	6.2×10^{-4}	0
Water absorption, %	5.3	0.29
Freeze-thaw durability		
Number of cycles	590	2,420
% wt loss	26.5	0.5
Hardness-impact ("L" hammer)	32.0	55.3
Corrosion by 15% HCl (84-day exposure), % wt loss	10.4	3.6
Corrosion by sulfates (300-day exposure), % expansion	0.144	0
Corrosion by distilled water	severe attack	no attack

1 psi = 6.9 kPa $1^{o}F$ = 0.55C.   1 ft. = 0.305 m.

2.94 Polymer Concrete

Polymer concrete contains no portland cement or other mineral binder but uses only polymer binders with the aggregates. The monomers used are polymerized by radiation, or thermal-catalytic or promotor-catalytic techniques (Ref. 79). Given the same care in mix design, mixing, transporting, placing, and curing, polymer concretes have much higher compressive and tensile strengths, are more impermeable, and far less susceptible to chemical attack. They do age more rapidly than portland cement concrete though this is still a very slow process. The aging results from the splitting of the polymer molecules. There are many different resins, modifiers, and hardeners which can be used and the properties of the concrete vary considerably (Ref. 42).

One particularly disadvantageous property of the polymer concretes is the contraction of the material during condensation of the resin. This reduction in linear dimension can be as much as 0.1% for furfural-acetone polymer concretes. Epoxy-polymer concretes show the least shrinkage. This shrinkage ceases as the hardening process is completed (Ref. 42). Until some meansof avoiding this shrinkage is developed the polymer concretes would be more useful in precast elements than in cast-in-place linings.

Heat affects the different types of polymer concretes in different ways. One type, polyester-polymer concrete, burns. Many others show decomposition of the polymer with evolution of gases (Ref. 42). Some tests have been done to evaluate the effects of geothermal temperatures on certain polymer concretes in which the effects of temperatures up to 238°C were studied (Ref. 79). These reported about a 60% reduction in compressive strength after 28 days of exposure and a loss in weight (Ref. 79).

There is not yet enough known of the aging of the materials in tunnel environments to warrant large scale use as the structural and waterproofing components of the tunnel.

Because of the high cost of polymer concrete it is unlikely to replace portland cement concrete but it has a potential for use as an exterior layer in underground construction with a portland cement concrete interior lining. This would increase flexural strength, greatly reduce permeability, and give improved corrosion protection to the reinforcement (Ref. 89).

2.95 Ferro-Cement

Ferro-cement has been in use for over 150 years in the constructon of boat hulls. It has shown excellent durability and watertightness. It consists of layers of wire mesh each having a thin coating of hydraulic cement mortar placed by hand or by shotcreting equipment. Because of the high steel content (about 6 percent) and good distribution of the steel reinforcing it is essentially crack free. Its value in underground construction derives from the ease in which it can be adjusted to unusual spatial designs (Ref. 89).

2.96 Fiber-Reinforced Concrete

Even at best the placing of reinforcing steel bars in tunnels is difficult. For continuous slipforming of the concrete liner directly behind the excavation this would be virtually impossible. In lieu of the reinforcement customarily used to improve the tensile and flexural strength of concrete, discontinuous, discrete fibers may be used (Ref. 63). They are generally 1/2 to 4 inches (15 to 100 mm) long. Thicknesses vary with the materials used. Many different materials have been used but the most frequently used materials are steel, alkali resistant glass, plastics, and asbestos. These fibers are added to the mix before placement. The resultant hardened concrete behaves essentially as an isotropic material highly resistant to cracking (Ref. 89).

Steel-fiber-reinforced concrete has been used for pavements, patching, refractories, mine tunnel linings, housing and school buildings among other structures (Ref. 70). The fibers are of various configurations. They may be flat, round, Duoform (alternately round and flat), crimped, or irregular from a hot melt. They may have plain ends, enlarged ends, or bent ends (Ref. 63). Using aggregate greater than 3/8-inch (10 mm) maximum size causes the steel fibers to ball considerably, the amount of balling increasing as the aggregate size increases. Additional fines are usually required to improve pumpability. Careful design is required to obtain a minimum void ratio in the mix and it is recommended that the cement paste volume exceed the void volume by from 2 to 5% (Ref. 97).

Glass fibers are usually alkali-resistant, chopped, multifilament strands (Ref. 97). These fibers have been used for concrete boat hulls. The filament diameter, bundle size and filament dispersion can be varied to suit conditions (Ref. 70).

Plastic fibers studied include nylon, polypropylene, polyethylene, rayon acetate, Saran, Orlon, and Dacron. Plastic fibers help the concrete to hold together since they bond well with concrete, are flexible, and can elongate considerably under load (Ref. 70).

Asbestos fibers have a natural affinity for water so a higher water-cement ratio is needed to make a workable mix producing a weaker concrete. The use of a very high percentage of fiber produces products such as piping, house siding, and wallboard (Ref. 70).

2.97 Microwave Curing

A means of obtaining high early strength studied in France is the use of microwaves to raise the temperature of the new concrete accelerating the hardening (Ref. 41). In general the specimens did not reach the 28-day strength of conventionally cured specimens but at 2 hours some specimens had 1/3 the 28-day strength of the control. The report gives no indication of the permeability of the concrete produced nor of the drying and temperature shrinkage which may be high enough to cause cracks. Furthermore, the technology for using microwaves for even the large areas of precast items uniformly and safely is yet to be developed and possible use for cast-in-place concrete work is still further in the future.

3.00 APPLIED WATERPROOFING ENVELOPES

3.10 GENERAL

When the underground structure is not of itself sufficiently water-resistant, an envelope of waterproofing material may be placed around the outside of the structure to prevent the infiltration of the groundwater into the structure. This is done frequently in cut-and-cover construction but may also be applied to soft ground or rock tunnels. When the groundwater head is not great or when the only protection required is from water seeping through the ground from above, often only the roof or roof and walls receive waterproofing. However, when structures are below the water table they may be completely surrounded by a membrane made continuous by overlapping and sealing the sections of the membrane.

If a tunnel or structure is constructed using a primary and secondary lining the membrane waterproofing may be placed between these two linings. This is sometimes done in bored tunnels. Here the outer structure usually is designed to withstand the earth load and the secondary or inner structure is designed to support the hydrostatic load.

The waterproofing membrane should be selected so that its useful life in the specific application will be 100 years or more.

Extremes of temperature; changes in temperature; chemical reaction with groundwater, cement paste, or tunnel wastewater impurities; microbiological reactions; presence of oxygen; soil movement; or a combination of these are the primary causes of the deterioration of organic materials. If any of these conditions are present organic materials should be avoided in the membrane. Therefore, unless conditions are known to be non-aggressive toward the organic material to be used, mineral or synthetic materials are to be preferred.

The waterproofing envelope may consist of any of a number of materials or combinations of materials. The most common are:

1. Built-up membrane consisting of hot-applied bituminous cementing materials alternating with fabrics or felts saturated with similar bituminous material.
2. Preformed multi-layered boards.
3. Plastic and synthetic rubber sheets.
4. Bentonite clay applied in sheets or sprayed on.
5. Cold, liquid-applied membranes, especially the elastomers
6. Cementitious coatings applied to the inside of the structure by the plaster-coat method or the iron-coat method.

7. Brick set in asphalt mastic.
8. Corrugated aluminum sheeting.

The membrane waterproofing system must be able to accommodate small, slow movements at any cracks in the concrete (Ref. 24). If earth is to be placed against a surface to be waterproofed, the membrane must also resist backfill damage, either by itself or by the addition of a protective covering.

The membrane waterproofing system must have a permeability at the water head anticipated such that it will be "impervious". Sowers and Sowers (Ref. 117) give the following relative values of permeability of soils:

Relative Permeability	Permeability mm/sec	Permeability ft/hr
Very Permeable	Over 1	Over 11.8
Medium Permeability	1 to $1x10^{-2}$	$1.18x10^{1}$ to $1.18x10^{-1}$
Low Permeability	$1x10^{-2}$ to $1x10^{-4}$	$1.18x10^{-1}$ to $1.18x10^{-3}$
Very Low Permeability	$1x10^{-4}$ to $1x10^{-6}$	$1.18x10^{-3}$ to $1.18x10^{-6}$
Impervious	Less than $1x10^{-6}$	Less than $1.18x10^{-6}$

Therefore, the material must have a permeability less than $1x10^{-6}$ mm/sec (less than $1.18x10^{-5}$ ft/hr) to meet the "impervious" requirement.

3.20 BITUMINOUS MATERIALS FOR WATERPROOFING

Bituminous materials had been used for waterproofing by the Egyptians and Babylonians many centuries before the birth of Christ. Bitumen was also used at that time as a mortar for the laying of brick. Bitumens include asphalts, tars, pitches and asphaltites (ASTM Definition D-8).

These materials are used today in tunnels for both purposes. In the New York subways brick set in asphalt mastic was used for waterproofing the jack arches as well as in other areas where the groundwater conditions are severe. This system has recently been replaced in New York by the use of an elastomer. Built-up waterproofing consisting of alternate layers of bituminous material, heated so that it is liquid, and bituminous saturated fabric was also used in the New York subways and in many other tunnels. These are built up in place, but preformed multi-layered waterproofing panels are also available which afford more rapid installation and do not require the use of hot mastic. One of the advantages of the bituminous waterproofing materials is that they also insulate against stray electrical currents.

3.21 Brick Set in Asphalt Mastic

Historically, brick set in asphalt mastic was used to waterproof the early New York City cut-and-cover subways in soil below the water table. Until a few years ago, New York City Transit Authority (NYCTA) still used this method for roof, sidewalls, and invert where groundwater conditions were severe (Ref. 93). See Figure 7 for NYCTA waterproofing standards.

The old New York City subway sections constructed by the cut-and-cover method used structural steel columns and beams for the primary support. Unreinforced concrete arches (called jack arches) were placed between the steel members leaving the inside flanges exposed. These exposed flanges were later painted for corrosion protection. Because the concrete does not bond imperviously with the flat surfaces of the structural steel the contact areas between structural steel and concrete are potential routes for water seepage and an unusually effective waterproofing is required to prevent this. The brick in asphalt mastic has proved to be very effective in this application but with today's high labor rates this time consuming method would be extremely expensive. Based on bid prices received by NYCTA from 1967 to 1971, escalated to 1980, this waterproofing method would now cost about $8.00 per square foot compared to about $1.00 to $1.50 per square foot for other methods.

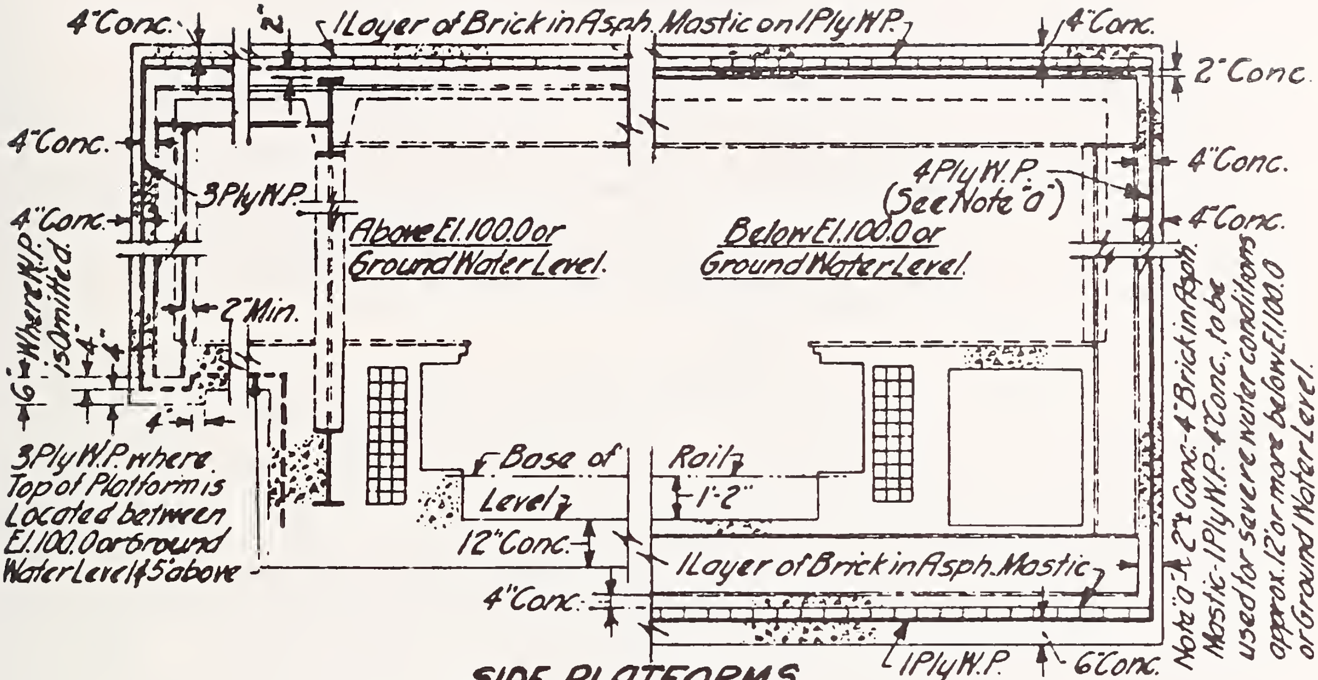
The system uses high compressive strength common brick, with low water absorption, laid up with 1/4-inch hot asphalt mastic joints horizontally and with the vertical joints filled with the hot asphalt mastic by pouring. Each brick is entirely covered with the asphalt. A single layer of asphalt saturated fabric is usually specified to be placed with a mopping of hot asphalt at each side prior to the laying of the brick and a 4-inch layer of concrete placed over the brick to protect it.

The NYCTA now specifies Oreloid, an elastometric compound that can be troweled on a vertical surface or poured and screeded on a horizontal surface for waterproofing in lieu of brick and mastic.

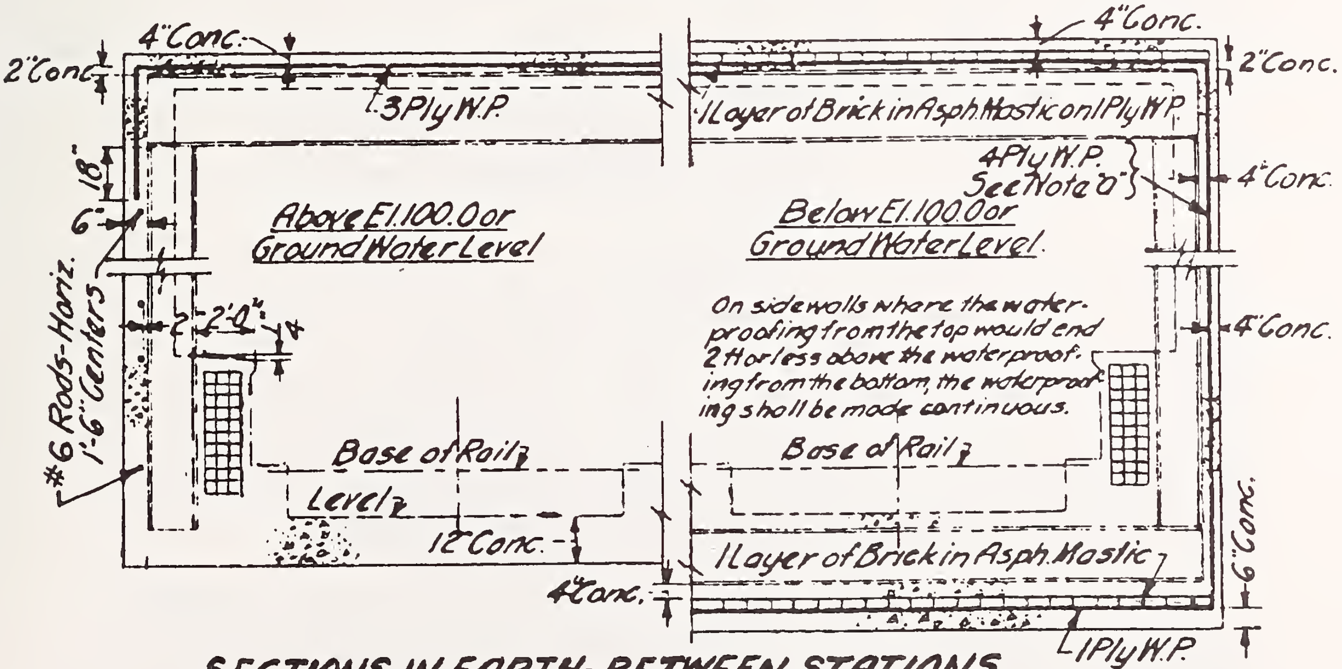
3.22 Built-Up Membrane Waterproofing Systems

Built-up membrane waterproofing consists of a series of alternating applications of a cementing material and plies sheets of woven fabric or felt, or a mat.

As the plying cement for hot-applied bituminous-based built-up waterproofing the choice is generally between properly prepared natural asphalt, asphalt derived from asphaltic petroleum, coal tar pitch, and coal tar enamel. Coal tar pitch is prepared from bituminous coal, and coal tar enamel is coal tar



SIDE PLATFORMS



SECTIONS IN EARTH-BETWEEN STATIONS

Figure 7 - BRICK AND MASTIC AND 3-PLY WATERPROOFING DETAILS FORMERLY USED BY NEW YORK CITY TRANSIT AUTHORITY (From Ref. 93)

pitch to which mineral fibers have been added to make the material less susceptible to temperature extremes and more stable in running water. These materials have performed well buried in soils for 50 to 100 years. The coal tar products show somewhat better waterproofing properties than do the asphalts. The asphalts are somewhat more resistant to weathering which is not of significance in tunnel waterproofing.

The plies may be of woven organic or inorganic fibers, felted organic or inorganic fibers, or they may be mats of uniformly distributed glass fibers. These sheets are then impregnated with bituminous material compatible with the plying cement used. Recently other types of plies have been developed for tunnel work such as the copper foils and jute burlap strips bituminized on both sides which were used in the Ferrara water chamber of the Upper Rhine power station and in some of the tunnels of the Walensee highway (Ref. 114).

The number of plies required for the installation depend on the ground conditions. A 3-ply system has been frequently used and found to be satisfactory. This would consist of a layer of hot bituminous material mopped on the surface to be waterproofed followed immediately by the application of a "ply" of bituminous impregnated sheeting worked well into the hot bituminous material, followed by another layer of cementing material and another "ply", followed by another mopping with hot bituminous material and another "ply", followed by a final mopping of hot bituminous material completely covering the third "ply". The successive plies are so arranged that no two seams are coincidental and within each layer the sheeting is lapped a minimum of 1 inch (25 mm). The surface to which the membrane is attached must be clean, dry and reasonably flat since the sheets are not very elastic and the cementing material usually is applied less than 1/16 inch (1-1/2 mm) thick and is not designed to smooth the surface. Figure 8 shows typical multi-ply systems specified for the American Railway Engineering Association.

The application of this waterproofing membrane is somewhat difficult on vertical surfaces due to the sagging of hot asphalt or coal tar. This can be partially overcome by doing the applications in sections and backfilling the section promptly to support the membrane (Ref. 15). This also serves to reduce the need of scaffolding. Another means of keeping it in place is to secure the plies to the wall so that they help to hold the hot tar in place. Frequently after the membrane is in place a thin concrete layer is placed over the membrane to protect it from damage and then backfilling is accomplished.

This method is very effective for cut-and-cover tunnels where the waterproofing can be placed on the outside of the completed structure but occasionally it is also used in tunneling where it is placed between primary and secondary linings.

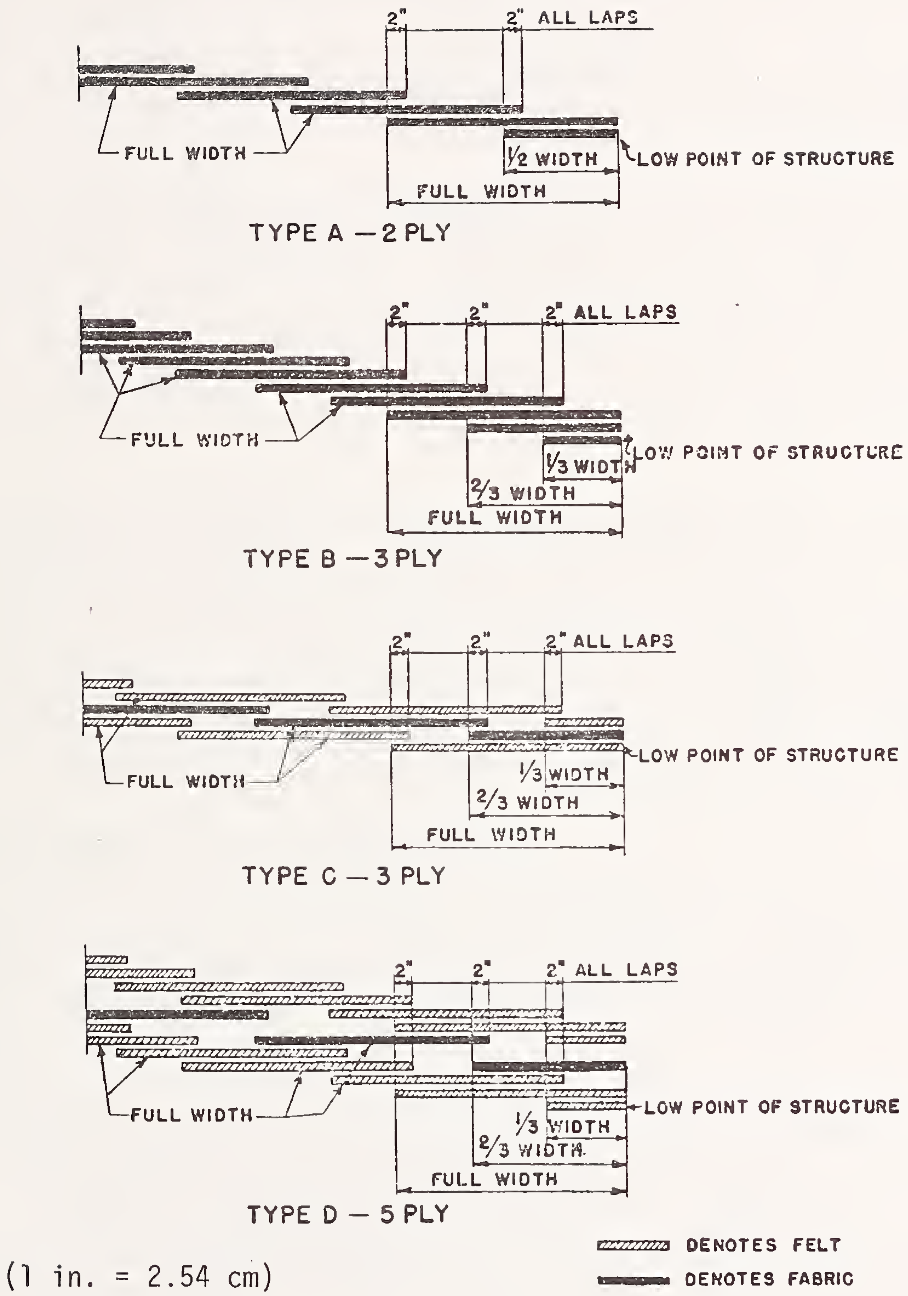


Figure 8 - TYPICAL BUILT-UP WATERPROOFING MEMBRANE SYSTEMS (From American Railway Engineering Association Manual, Ref. 4)

When the membrane is on the exterior and the surrounding soil settles the waterproofing membrane may be stripped off. To avoid this the waterproofing membrane can be sprayed with a bituminous antifriction layer having a thickness of 8 to 10 mm (0.3 to 0.4 in.). This new material allows the soil to slip easily along the wall thus preserving the integrity of the waterproofing (Ref. 43).

The New York City Transit Authority specifies the use of asphalt and asphalt impregnated woven cotton (Ref. 93). American Railway Engineering Association allows a choice of asphalt or coal tar pitch with woven cotton fabric or felt saturated with the appropriate bituminous substance (Ref. 4). Another multi-ply system uses coal tar enamel, impregnated felt plies as the outer plies and fiberglass reinforcing mats as the inner plies. It is stated that these applications should last over 100 years (Ref. 15).

With more rapid tunneling methods being developed more rapid means of waterproofing the tunnels become necessary. To meet this need, during the late sixties, a machine was developed and used by Sika International to apply bituminized strip to the primary tunnel lining using hot bitumen (Ref. 114). In the Ferrera water chamber of the Upper Rhine power station the strip used was copper foils and jute burlap strip bituminized on both sides. Using a similar machine, bituminized burlap sheets were applied in other tunnels including a highway tunnel at San Bernardino, Switzerland, another at Grancia, and a railway tunnel at Keferberg near Zurich. An improved machine was developed for use in the Munich underground railway because a rate of 15 m/shift was required. The tunnel was circular in section. The machine consisted of a heavy tubular scaffolding mounted on wheels. Two revolving arms each carrying a laying cradle were mounted centrally within the main structure. The machine is so constructed that sufficient space is left clear in the center of the tunnel so as not to impede normal tunneling traffic. The machine moves on rails and is powered by pneumatic motors. The two cradles are so oriented that the overlapping joints of the first layer are covered by the second layer without a repositioning of the machine. Each cradle is controlled by an operator standing on an operating platform attached to the cradle. A cross-beam carries the roll of bituminized burlap strip which is pressed against the tunnel face into hot bitumen. Each cradle lays the strip from the base of the wall to the apex, drops rapidly to the base opposite and lays its next strip, starting in an inverted position, from that base to overlap, at the apex, the strip just laid. While this operation is proceeding the second cradle is laying the second ply covering the joints on the first ply. Then the machine is advanced and the next section is placed. After being mechanically applied the plies are smoothed down by hand.

In another development the bituminized rolls pass through an electrically-heated bitumen bath and then directly onto the tunnel surface (Ref. 43).

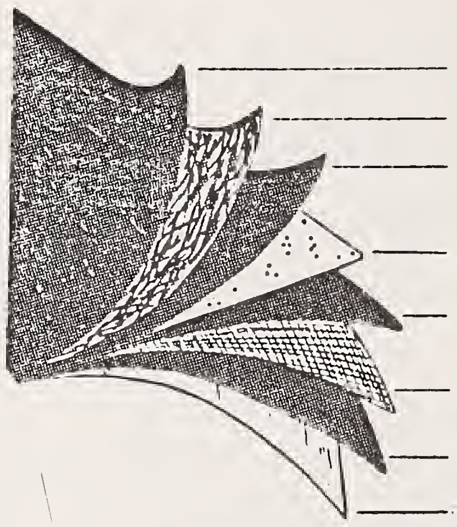
For Oslo's T-Banen underground railway a more efficient and less expensive waterproofing system was developed (Ref. 43). Here the conditions were much like those in Amsterdam where it was necessary to guard against changes in the groundwater level so that it was deemed essential that the tunnel be completely waterproofed. This was done with a two-ply bituminous membrane which was shown to be capable of withstanding the enormous water pressures encountered without being detached from the concrete face of the tunnel. The tunnel was first lined with reinforced concrete, then two layers of Ruberoid Vitrix were welded to the concrete, after which a secondary lining of concrete was applied. To apply the membrane to the roof of the tunnel a roller device was developed enabling one worker to roll out the material overhead while another beside him heated it with a torch welding it to the concrete. This simplified the difficult overhead application. Corners and ramps had to be reinforced with strips of bitumen-backed copper foil to prevent leaking. Occasionally sufficient water penetrated the outer concrete skin so that there was a danger of pushing the membrane away from the skin before the inner lining was placed. To avoid this, special temporary "draining strips" were applied to the invert to carry the water away. After placement of the secondary liner the drain channels were sealed.

3.23 Preformed Multi-Layered Board

A number of preformed multi-layered vapor and waterproofing materials are available. Some come as semi-rigid boards usually 4 feet by 8 feet, and some come in rolls. In general they resemble preassembled built-up membranes. Installation is much faster however.

The material consists of a minimum of one sheet of reinforcing material (to resist puncture) embedded in an elastomeric material composed of bituminous and synthetic resin materials. There may be any number of other plies including exterior protective coatings. An example of a multi-layered board is shown in Figure 9. The welding together of the various constituents, since it is done under controlled factory conditions, is superior to that in the built-up membranes. Usually an overlap of two to six inches is recommended by the manufacturer. Some products are butted with a sealing tape over the seams. Some are self-bonding though these require a prime coat on the substrate.

In all cases it is necessary to make sure the concrete is free of all foreign substances and loose surface material, any cracks that have opened have been repaired, any honeycombing has been chipped out and replaced by sound concrete, and all



Note: 1 mil. = 0.001 in.
1 in. = 25.4 mm.

Figure 9 = TYPICAL PERFORMED MULTI-LAYERED BOARD SHOWING TYPES OF PLIES (From W.R.Meadows, Melnar product data sheet) construction joints are tight. Frequently special treatment of construction joints before installation of the member is recommended by the manufacturer.

The substrate is then coated with the primer or a bonding agent. If the membrane is not the self-bonding type it is usually also coated with the bonding agent. Then the material is placed in position and rolled to make a good, complete bond with the substrate. The sealing of the joints in the membrane must be carefully done. Backfilling can usually be done immediately. Under certain conditions a protective course is recommended.

3.24 Cold-Applied Bituminous Materials

At Bochum (U-Bahn) in Germany where a perfect water seal was required, the tunnel was first lined with a sprayed-on concrete and then a waterproofing membrane was applied (Ref. 72). As reported in Peter Schulze's paper, Tiefbaumt der Stadt Bochum, this consisted of a sprayed bitumen-latex emulsion worked into which was a 2-millimeter thick "aspylan" foil. This was covered by another sprayed-on layer of the bitumen-latex. A secondary lining of 300 millimeters of concrete was then placed.

Studies are being made on spraying systems with bitumen based polyurethane materials (Ref. 43). This could give increased speed and flexibility as well as greatly reduced cost.

3.30 PLASTIC AND SYNTHETIC RUBBER SHEETING

With the growth of the plastics industry new materials have become available which show great promise of being useful for waterproofing underground structures. These include impervious plastic and synthetic rubber sheeting. Many advances have been made in the use of these sheets in the waterproofing of roofs. Frequently, the single thickness of plastic or synthetic rubber has given better service than the standard 3-ply bituminized system.

The materials which are most promising both in cost and properties are polyethylene, polyvinyl chloride, butyl rubber, Hypalon, and Neoprene. Their advantages over built-up membranes include:

- Resistance to many chemicals
- Resistance to bacteria and other organisms
- Superior elasticity and elongation before rupture
- Resistance to aging
- Wide temperature range of usefulness
- Electrical resistance
- Light weight

Application possible without use of hot mastics
Resistance to puncture
Self-healing, to some degree especially the rubbers
Flexibility.

3.31 Polyethylene Sheeting

Polyethylene sheets can be joined by heat fusion to make watertight sheets of any size and shape desired. If given proper treatment it can also be bonded using epoxy or other adhesives. This latter property makes it possible to attach the sheets to almost any surface.

In cut-and-cover the sheets may be applied to the outside of the structure, but they must then be given a protective covering. On the walls this may be asbestos cement board, concrete plank, insulation board, 3-inch (75 mm) concrete blocks or common brick in mortar, or similar material. On the invert a 2-inch (50 mm) layer of concrete is sufficient and on the roof which will be covered with earth a minimum of 3 inches (75 mm) of protective concrete should be used. If the roof will be subject to damage as is the case with tunnels under water where ships' anchors and other objects may damage the protective concrete other suitable protection should be employed. If no space is available between the excavation support and the wall the sheeting is placed against the form before the concrete is placed. Care must be taken in placing the reinforcing steel and the concrete so that the sheet is not ruptured. All joints in the sheeting must be completely sealed and tested before backfilling or concrete placing is undertaken.

In tunneled structures the sheeting is usually placed after the primary lining whether it be segmented lining, cast-in-place concrete lining, or shotcrete. Then the secondary liner is installed thus protecting the sheeting. Again care is required in placing the inner liner to prevent damage to the sheeting.

In Norway in tunnels (Ref. 60) where minimum maintenance is a prime criterion a special system for installing a four-layer polyethylene sheet membrane has been developed. After the tunneling operation concrete is placed to give a reasonably smooth surface on which to apply the sheeting. The sheeting is placed in two 2-ply layers each being held against the tunnel profile with reinforcing steel cut to a length such that its ends are supported on the foundations on opposite sides of the tunnel. The two sets of rods are staggered. These hold the sheets in place without the need for bonding or otherwise fastening them to the smoothing concrete. The structural lining is then placed.

3.32 Polyvinyl Chloride Sheeting

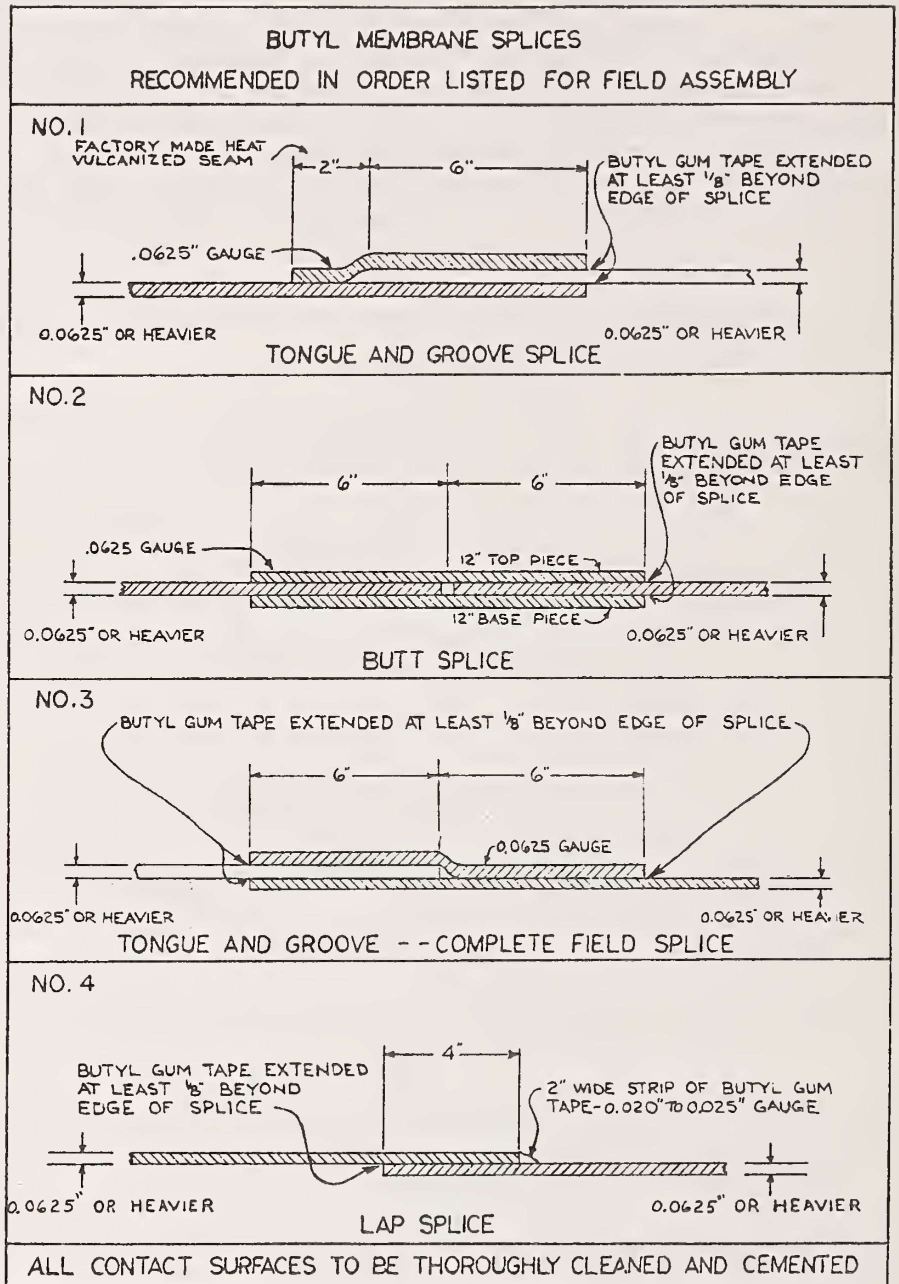
Polyvinyl chloride (PVC) sheeting has properties quite similar to the polyethylene and can be used similarly to the polyethylene. There are some specially designed types of sheeting manufactured which can be fastened to the forms before placing the concrete. These have T-shaped projections which interlock with the plastic concrete and which are then integral with the concrete when it has hardened. An example of this type of material is B. F. Goodrich's Koroseal Lok-Rib. There are adhesive and welding techniques for sealing the sheets of material to one another. The material is very expensive and would not be economically practical in a tunnel unless conditions were extremely difficult and required this mechanical bond with the concrete.

In Switzerland mechanization of the laying of the sheeting has been developed by modifying the machine used to lay bituminized waterproofing (Ref. 114). One of the modifications permitted the entire circumference of the tunnel to be covered in one sweep. The sheets were sprayed at the site with an adhesive which, when heated during application, sealed the sheets to the structure and sealed the overlaps so that it became a jointless lining over the entire surface. In this application the tunnel surface to which the lining was bonded had to be quite smooth.

Another method of attaching PVC has been described by A. Pedduzzi (Ref. 103). This method is applicable to rougher surfaces than the mechanized method just described. Special lath is manufactured of the same PVC material as the sheeting. These lathing strips are used to fasten a plastic foam base mat to the tunnel faces by means of nails through the lower portion of the lath section. These mats prevent damage of the sheeting by the rough shotcreted tunnel surface. A sheet of PVC at least 80 mils (2 mm) thick is bonded to the upper portion of the lath, made flat to receive the sheeting, using a hot-air process. The joints between the sheets are also heat-sealed using a double seaming process, the two seams being separated by a plastic wire. The seams can then be tested by using compressed air and any necessary remedial work done.

3.33 Butyl Rubber Sheeting

Butyl rubber (isobutene-isoprene rubber, IIR) has great flexibility and does not stiffen with age so it adapts well to surface irregularities. It also has a tendency to cold flow so it is able to fill small crevices, and to self-heal small holes and around puncturing nails or projections. It is easily cut and fitted around openings, penetrations and other irregularities. It can be spliced with cold self-vulcanizing butyl cement and/or unvulcanized butyl gum tape. Typical membrane splices recommended by the American Railway Engineering Association are shown in Figure 10. The application of butyl rubber to tunnel



(1 in. = 2.54 cm.)

Figure 10 - TYPICAL MEMBRANE SPLICING DETAILS (From American Railway Engineering Association Manual, Ref. 4)

surfaces is accomplished in much the same way as plastic waterproofing sheeting is, but the rubber is highly abrasion resistant and rarely needs protection during backfilling or concrete placement. Of the synthetic materials butyl rubber is the most impervious and most frequently used in the United States.

3.34 Hypalon Sheeting

Hypalon is a chlorosulfonated polyethylene manufactured by DuPont. It can be vulcanized, at room temperature, into a tough, chemically resistant rubber. Hypalon has the greatest elongation at break of the synthetic materials commonly used in waterproofing and therefore can successfully accommodate more differential structural movement than the other materials. It is, however, one of the most expensive sheetings.

3.35 Neoprene Sheeting

Neoprene, the DuPont trademark for a chloroprene rubber, has properties similar to natural rubber, but since it is chlorinated, it resists burning and biodegradation. It can be vulcanized in-place, or the sheets may be prevulcanized, to obtain the required properties, both chemical and physical. The mechanical properties of Neoprene vary with the fillers used as well as the degree of vulcanization. If vulcanized to a hardness of a rubber heel (Shore A-45) the hardness will increase with time till it becomes brittle which is typical for any polymer that can be cross-linked three dimensionally, Figure 10. This would not be acceptable in underground structures which are expected to have a useful life of 100 to 150 years during which the material should remain soft; the membrane must be prepared in such a way that this will not happen.

3.40 COLD, LIQUID-APPLIED WATERPROOFING

Where the infiltration of groundwater is through slightly porous concrete and the waterhead low, a coating on the inside of the structure may be sufficient to stop the leak. Some proprietary materials are said to develop sufficient adhesion and film strength to seal out moisture moving through a concrete wall (Ref. 24). This, however, is not a good, planned procedure where complete watertightness is desired, but rather a remedial measure.

Used as a membrane on the outside of a concrete structure or sandwiched between two lining layers, many of these materials can do an effective job. Sprayed or rolled onto the surface they are less expensive and faster to apply than sheet materials and can much more readily be applied to unusual, non-flat surfaces. The sheet materials can be applied more rapidly than the built-up membranes so the advantage over the traditional built-up membrane means of waterproofing is tremendous. Furthermore no hot bituminous material need be handled.

One of the disadvantages of cold, liquid-applied waterproofing is the difficulty in obtaining a uniform thickness throughout the coating and preventing pinholes. This usually entails several separate applications which defeats some of the advantage of rapid placement. Also it is usual to specify a minimum thickness with a greater average thickness than needed under the water head.

The Water and Power Resources Service has recently been using some of these liquid-applied materials in lieu of the customary asphalt and felt membranes for waterproofing roofs (Ref. 24). Liquid-applied Neoprene coated with a weather-resistant top coat and liquid-applied silicone rubber have been used. They have also found that Neoprene performs well on the upstream faces of dams being impervious as well as resistant both to sunlight and to abrasion.

A urethane waterproofing membrane manufactured by Gates Engineering is a two-component material, 100% solids which can be troweled, squeegeed, or spray applied and which cures into a water resistant rubbery coating. Fifty-pound roofing felt may be applied to the coating while still tacky to protect the membrane during backfill. The material has excellent adhesion to good, dry concrete rarely needing a prime coat.

A tar modified polyurethane, 2-component, 2-coat system, offered by Universal Protective Coatings performs much the same as the Gates product. It contains 85% solids by volume. The manufacturer recommends applying vertical protection board over the membrane by spot bonding.

Plas-Chem has a series of elastomeric coatings of both oneand two-component types. Included in these are a one-component Hypalon material and a two-component butyl rubber which can be spray applied giving good impermeability in a two-coat system of approximately 20 mils (0.5 mm). A hydrocarbon modified urethane membrane specifically intended for waterproofing concrete below grade is also included. This is a two-component material.

3.50 BENTONITE PANELS AND SPRAY

Another material which has shown excellent results when used as waterproofing for structures below the groundwater table is the clay material, bentonite, either applied in panels or sprayed on the surface to be waterproofed. There are only two manufacturers of bentonite waterproofing materials in the United States, namely: American Colloidal Company, manufacturers of bentonite panels; and Effective Building Products, Inc., manufacturers of the spray appplied material.

Bentonite is a highly plastic clay resulting from the decomposition of volcanic ash, and it is found in quantity in Wyoming. Chemically it consists primarily of sodium montmorillonite. It is inorganic, nontoxic, fire-resistant, and unaffected by freeze-thaw cycles. When unconfined its wet volume can be from 10 to 25 times its dry volume and it is then in a gel state rather than a solid state (Ref. 124). This property is the one that makes the mineral useful in waterproofing. When wet it expands and fills small voids and cracks preventing the flow of water through them. Cracks that develop in the structure, due to either temperature or drying shrinkage, are filled and closed and even cracks due to movement of the structure may be filled by the expanding bentonite.

Bentonite is affected by salt, acid, and alkali in the groundwater, but the manufacturers of the waterproofing bentonite materials can supply special bentonite formulations and application instructions that will give satisfactory results in some of these instances.

When bentonite is used for waterproofing the force due to the swelling of the bentonite when wet must be taken into account either in the design of the structure or the application of the material.

During storage of any bentonite product great care must be taken to keep the product dry in order to prevent any premature expansion. Application may be made in slightly damp conditions and on green concrete, if additional concrete or backfill will be placed immediately. However, effective application cannot be made under conditions of standing or running water. If placed in-the-dry, backfill or concrete need not be placed immediately if adequate waterproof covering is placed over the bentonite.

All penetrations and construction joints must be treated with additional thicknesses of the material since these are locations where some leakage will probably occur and surplus bentonite will be needed to fill the crevices.

Installation of panels or spray does not require special trades, but must be done under careful supervision to ensure proper coverage. It is often desirable to have the assistance of the manufacturer's representative during application.

Unlike bituminous, rubber, and plastic materials, bentonite is not an electrical insulator.

3.51 Bentonite Panels

Bentonite panel envelopes are used in cut-and-cover construction. The panel material consists of granular bentonite sealed inside a smooth face sheet of biodegradable corrugated kraft paper. These panels are usually four feet square and 3/16 inch thick. American Colloid Company, manufacturer of Volclay Panels, gives the results of permeability testing using standard soil testing procedures with a static 60-foot (18-m) head as:

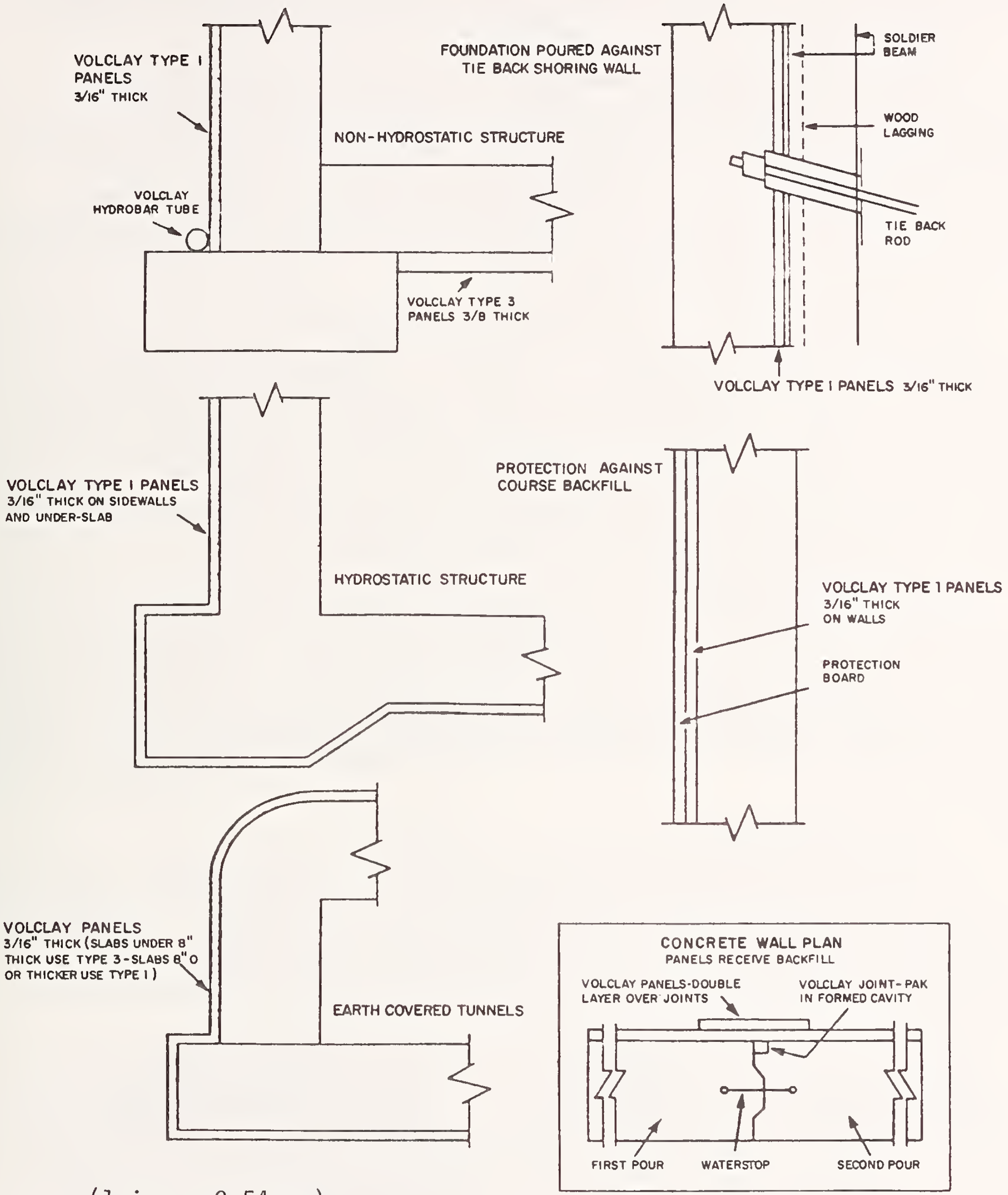
Time in Minutes	Permeability ft/hr	Permeability mm/sec
0 to 78	$1x10^{-7}$	$8.47x10^{-9}$
78 to 122	$2x10^{-7}$	$1.69x10^{-8}$
122 to 172	$1x10^{-7}$	$8.47x10^{-9}$
172 to 1324	$0.5x10^{-7}$	$4.23x10^{-9}$

These permeability figures are considerably less than the "impervious" figure of $1x10^{-6}$ mm/sec of Sowers & Sowers (Ref. 117).

The panels may be installed at any temperature and are effective at any temperature. The installation may be either on the finished concrete surfaces, on lagging, or on any other reasonably smooth surface against which the concrete will be placed. All panels must be overlapped with adjacent panels a minimum of 1-1/2 inches to insure continuity of waterproofing film. Typical bentonite panel installations are shown in Figure 11.

If the invert slab is to be waterproofed using these panels, special precautions are necessary. If the ground surface is uneven, a means of smoothing it must be found so that no tears will occur. A subslab of unreinforced concrete is an ideal surface and, in addition, it provides a clean level surface on which to prepare reinforcing steel, forms, embedded conduit, etc., for the base slab pour. If the bentonite panels are to be placed on the ground and it is wet or very damp one or more layers of 4-mil (0.1-mm) polyvinyl sheeting should be placed between the ground and the panels to prevent premature swelling of the bentonite. The panels should be stapled together or secured in some manner so that subsequent placing of concrete will not dislocate the panels. Once the panels are placed the concrete may be placed. If concrete cannot be placed immediately the panels must be protected from moisture, foot traffic, and construction operations which may damage the panels.

In cut-and-cover operations the concrete walls may be placed directly against the temporary excavation support or they may be completely formed leaving work space between the wall and the temporary support. If the concrete is to be cast directly against the support bentonite panels can be attached to the support if it is sufficiently smooth. If there will be work space between the structural wall and the temporary excavation support, the panels should be attached directly to the structural wall after curing. Obtaining the necessary lap of the



(1 in. = 2.54 cm)

Figure 11 - TYPICAL BENTONITE PANEL INSTALLATIONS
(From American Colloid Company's
Volclay Panels brochure)

wall panels with the slab panels is important and takes good supervision. If backfilling cannot be done immediately the installed panels must be protected from moisture.

Backfilling must be done with care to protect the kraft paper from damage and movement. If the backfill contains coarse or irregular gravel 1/4-inch hardboard or similar material may be needed to preserve the integrity of the panels during the backfilling operation. In order to prevent the installed bentonite layer from loosening, the backfill should be compacted to a minimum of 85% of the maximum as determined by the Modified Proctor test.

When applied to the roof of a structure the panels must be covered with a minimum of two feet of compacted backfill or an equivalent layer of concrete or other material to prevent movement of the bentonite by heavy traffic. Pressure will not "squeeze" the bentonite out but traffic loading could move it. The panels should be secured to the roof so that there will be no movement during the backfilling operation. It is usual to place a 6-inch layer of sand as a drain channel over the panels before placing and compacting the backfill.

If the construction is done "under-the-roof" the installation may be similar but in the reverse order. In this type of construction and any cut-and-cover construction using slurry walls as final permanent walls, the continuity of the bentonite envelope must be maintained by the grout or treated soil that contacts the exterior surface of the permanent excavation support walls.

3.52 Bentonite Spray

Bentonite may also be applied in spray form. When used this way usually a 1/4to 3/8-inch layer is applied. Effective Building Products of St. Paul, Minnesota, gives the following permeability data for its spray-on bentonite product, "Bentonize":

Water Head		Total Elapsed Time	Coefficient of Permeability	
Feet	Meters	Days	ft/hr	mm/sec
10	3.05	9	1.74×10^{-7}	1.47×10^{8}
40	12.20	11	1.95×10^{-8}	1.65×10^{9}
80	24.40	20	5.46×10^{-9}	4.62×10^{-10}

An advantage of the spray-applied product is that the bentonite may be applied to irregular surfaces such as are produced by poured-in-place concrete construction. It may also be used to waterproof irregular surfaces such as block masonry and spalled, honeycombed, or oozed concrete surfaces. Thus no means of eliminating sharp points and hollows need be used before the application of the waterproofing and the smoothing and filling of the surface may be done at the same time as the concrete for the slab or wall is placed. Typical applications of the sprayed-on bentonite and associated products are illustrated in Figure 12. The sketch shows the application of Bentonite directly on lagging. Here the recommended thickness is 3/8 inch. The second sketch shows an underground structure with Bentonize applied directly to the exterior of the concrete wall and roof slab. If the concrete is not green a necessary precaution for proper adhesion is that it be misted with alcohol to remove surface dust before application of the bentonite. Also shown in this sketch is the application of Waterstop-Plus, the Bentonize manufacturer's joint compound for sealing cold joints in concrete. Since this material expands when in contact with water, it forms a pressure seal so it does not depend on adhesion to the concrete to seal the joint. If the joint moves, the material, being flexible and under pressure, continues to keep the joint watertight. The bottom sketches show another product manufactured by Effective Building Products for use in conjunction with waterproofing membranes other than bentonite. This formulation, Leak Localizer, is similar to Waterstop-Plus. Its purpose is to localize any leaks through the non-bentonite membrane so that repairs can be made without the need for extensive investigations to determine the source of the leak.

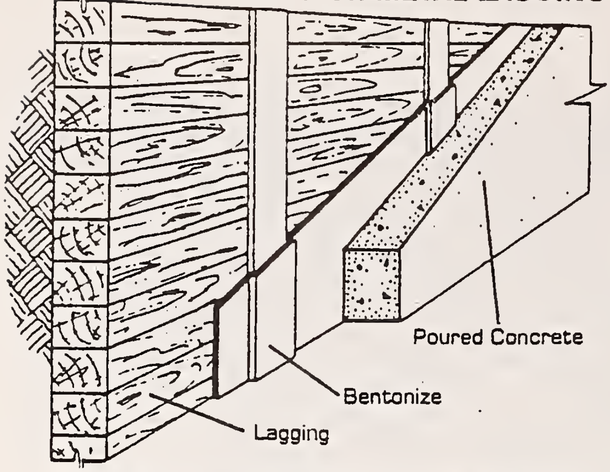
Care should be taken in placing concrete and backfill against the waterproofing to prevent these materials from striking the waterproofed surface directly and scouring the waterproofing off.

Ideally, backfill or placing of concrete should follow application immediately. If this is not possible the spray applied product requires the same sort of protection as do the panels.

3.53 Maintenance of Bentonite Waterproofing Envelopes

The maintenance of bentonite envelope waterproofing is not difficult. Generally, if small leaks develop, they heal themselves by the movement of the bentonite gel into the cracks through which the water flows. If a major leak occurs it will be noted on the interior of the tunnel right where it is, since water cannot flow along the exterior of the tunnel lining through the bentonite gel layer. Therefore, bentonite slurry injected through a hole drilled at the location of the leak will stop the leak. Then the drilled hole can be sealed with a hydraulic cement plug and the tunnel will again be leak-free.

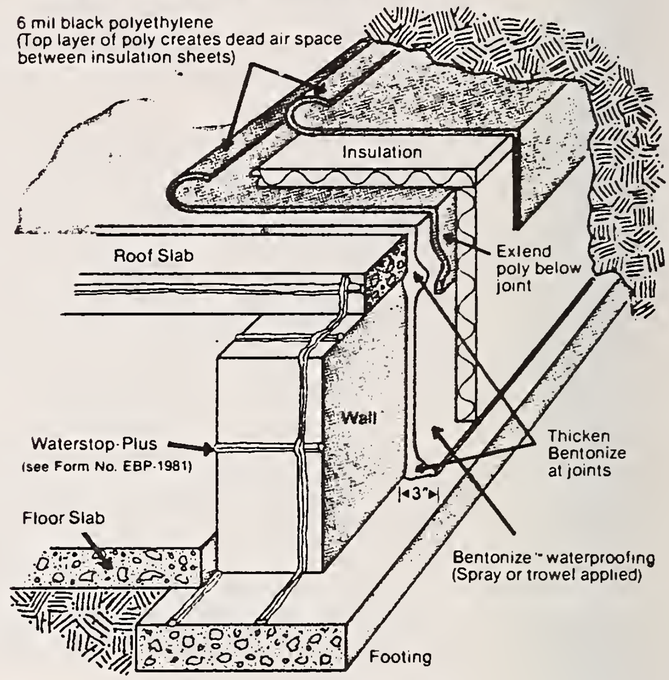
WOOD OR METAL LAGGING



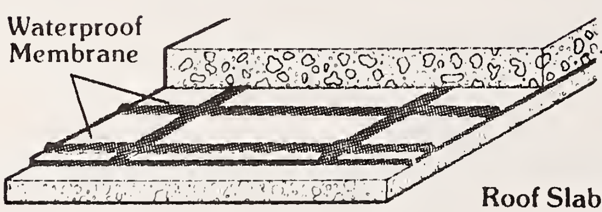
The Bentonize™ Waterproofing System used on a lagging type operation.

COMPLETE VERTICAL AND HORIZONTAL WATERPROOFING

The Bentonize™ Waterproofing System may be used on horizontal, vertical, or tunnel applications, including projects that require complete envelope sealing.

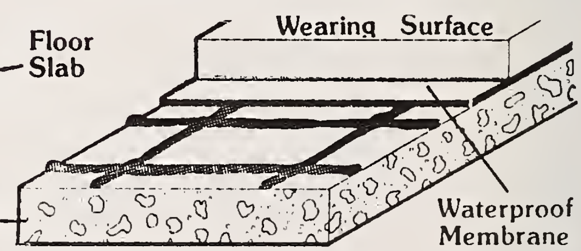


FLOOR SLAB



Leak Localizer is placed over the waterproof membrane in a square grid pattern to isolate any water leak coming through the waterproof membrane.

TUNNEL PLAZA OR UNDERGROUND ROOF SLAB



Leak Localizer placed in a manner which will isolate and stop the horizontal migration of water under the waterproof membrane, facilitating easy repair.

Figure 12 - TYPICAL INSTALLATIONS OF BENTONITE SPRAY AND ASSOCIATED PRODUCTS
(From Effective Building Products, Inc. Data Sheets)

3.60 CEMENTITIOUS WATERPROOF COATINGS

Plaster coat waterproofing and a variation of it iron coat method have several advantages. They can be applied on the inside of concrete structures; they can be applied on wet surfaces; leaks, if they occur, are localized right where they come through; and leaks can be repaired easily by a reapplication of the waterproofing coating. Furthermore, if the porosity of the concrete is due to insufficient curing, the coating, being on the inside, allows the groundwater to continue the hydration of the concrete through the years thus developing a more impervious concrete.

The concrete surface to which the coating is to be applied must be clean and well roughened so that there is good adhesion. The coatings must be moist cured. Furthermore the workers applying the coatings must be skilled in their application. The number of such skilled workers is decreasing.

3.61 Plaster Coat Method

The plaster coat usually consists of a mixture of cement, sand, and possibly a waterproofing admixture which is troweled on the prepared surface to a depth of 3/4 to 1 inch (20 to 25 mm). If leaks still occur the coating may be removed and replaced or greater thicknesses used. This method has been known to withstand a head of 65 feet (19.8 m.) (Ref. 99).

3.62 Iron Coat Method

The Bay Area Rapid Transit District specified the Conrad Solvig's iron coat method of waterproofing for cross-passages and access shafts in Oakland. The coating specified consisted of a mixture of portland cement, fine aggregate, water, pulverized iron, and a chemical oxidizing agent. Recesses, cracks and intersections of vertical and horizontal surfaces should be packed with a grout containing the pulverized iron. Three brush or sprayed coatings are applied and must be moist cured. Usually a protective coating is then applied consisting of cement, water, and sand, to cover the "rusty" coat. The iron coat can also be troweled on. In some applications alternate brushings of the pulverized iron with the oxidizing agent and the iron-cement slurry are used. If done well the iron coat method can withstand a water head of seventy feet.

3.63 Non-Metallic, Nonshrink, Cementitious Coating

Recently United States Grout Corporation has developed a non-metallic, nonshrink, cementitious coating which can be troweled, brushed, or sprayed on the interior surface. It is a fast-setting, hydraulic cement-based mortar. It is said to be superior to the iron coat method. It does not need the skill required by the plaster or iron coat methods and a single coat

1/8 in. (3 mm.) thick is recommended. The manufacturer's published data states that this thickness will resist a 50-foot (15.2 m.) head of water.

3.70 MISCELLANEOUS WATERPROOFING ENVELOPES

In Norway a method of shielding the tunnel from groundwater intrusion has been developed using corrugated aluminum sheets (Ref. 60). Aluminum sheeting is relatively inexpensive and is durable under tunnel conditions unless the groundwater contains certain aggressive impurities. This shielding consists of an arch of aluminum sheet the ends of which are braced against the foundation at both sides of the invert. Since the sheet is normally quite thin (0.71 mm) the groundwater must be protected from freezing since an ice load could easily damage the aluminum. This is done by gluing watertight polyethylene bags filled with rock wool to the installed sheet and placing a second sheet against the insulation braced in the same manner as the first. For drainage the subgrade and drains must also be insulated. Figure 13 illustrates the placement of frost insulation to protect earth surrounding the tunnel and the drains from freezing.

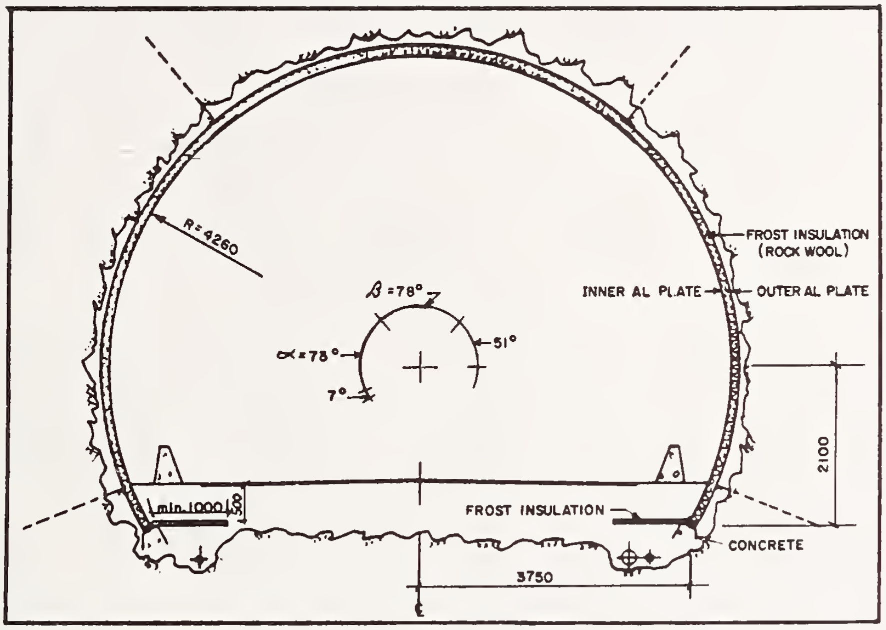


Figure 13 - ROAD TUNNEL CROSS SECTION SHOWING A TYPICAL LINING DESIGN USING ALUMINUM SHEETS FOR WATER SHIELDING (All measurements in millimeters)

(From Grønhaug, Ref. 60)

4.00 SEGMENTED TUNNEL LININGS

4.10 CONSTRUCTION OF SEGMENTED LININGS

Tunneling systems utilizing shields and segmented tunnel linings to construct tunnels in soft ground have been used since 1865 (Ref. 109). Although there have been many technical improvements in the equipment used and the fabrication of linings, the basic construction technique remains unchanged and is still an attractive alternative to use of a primary and seconondary lining/support system. The segmented lining often combines both an initial ground support and permanent support in one. For some tunnel-uses a non-structural finish lining may be placed inside.

Segmented linings of many types, shapes and materials have been used, but basic procedure of construction for all is similar. A continuous complete lining is erected in the tail of a shield, one ring at a time. The rings, which may vary from 18 inches (0.5 m) to 4 feet (1.2 m) in length, consist of a number of pieces, or segments. Each ring is erected in the tail of the shield adjacent to the last previously erected ring. The shield is basically a circular working chamber of steel enabling the tunnelers to excavate the ground and erect the lining in safety; the shield supports the ground until the lining is built. After each ring is erected, hydraulic jacks mounted in the circumference of the shield shove against the erected ring (and through it to the completed lining behind), propelling the shield forward the length of one more ring. Sufficient soil is excavated at the face for one shove, the shield jacks are retracted, and another cycle begins. Figure 14 shows a segmented lining being erected in the tail of a soft ground tunnel shield. Note the shove jacks in the lower right corner.

In soil that is loose, or permeable and water bearing, the segments must be bolted together for structural continuity, resisting bending moments, and maintaining the integrity of joint seals. In soil that is relatively dry and firm, such as London's blue clays, it may not be necessary to firmly connect the segments or rings together. Unfortunately such benevolent ground conditions are rare in most U. S. coastal cities where transportation tunnels have been built in the past, or are contemplated in the future. Since this study is on the control of groundwater, discussion in this section will be limited to bolted segmented linings when they are used in soft ground tunnels. Typical bolted segments of steel, cast iron and concrete are shown in Figure 15. A discussion on unbolted linings for use in rock tunnels will follow the discussion of bolted linings in soft ground.

The reason for discussing the construction of segmented linings is that the nature of the lining, consisting of many

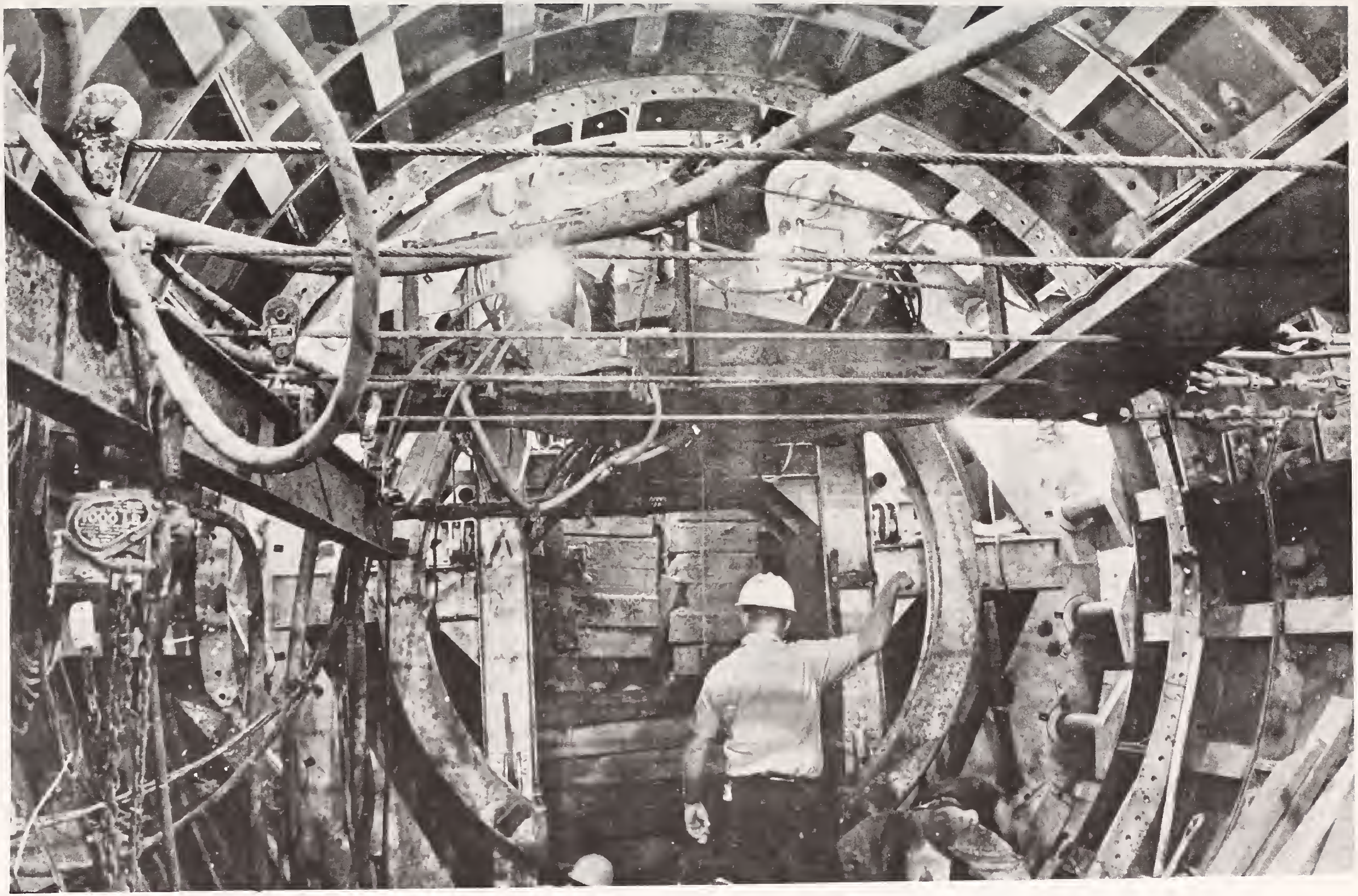


Figure 14 - SEGMENTED TUNNEL LINING ERECTED IN THE TAIL OF A SHIELD
(Courtesy of MacLean-Grove & Co.)

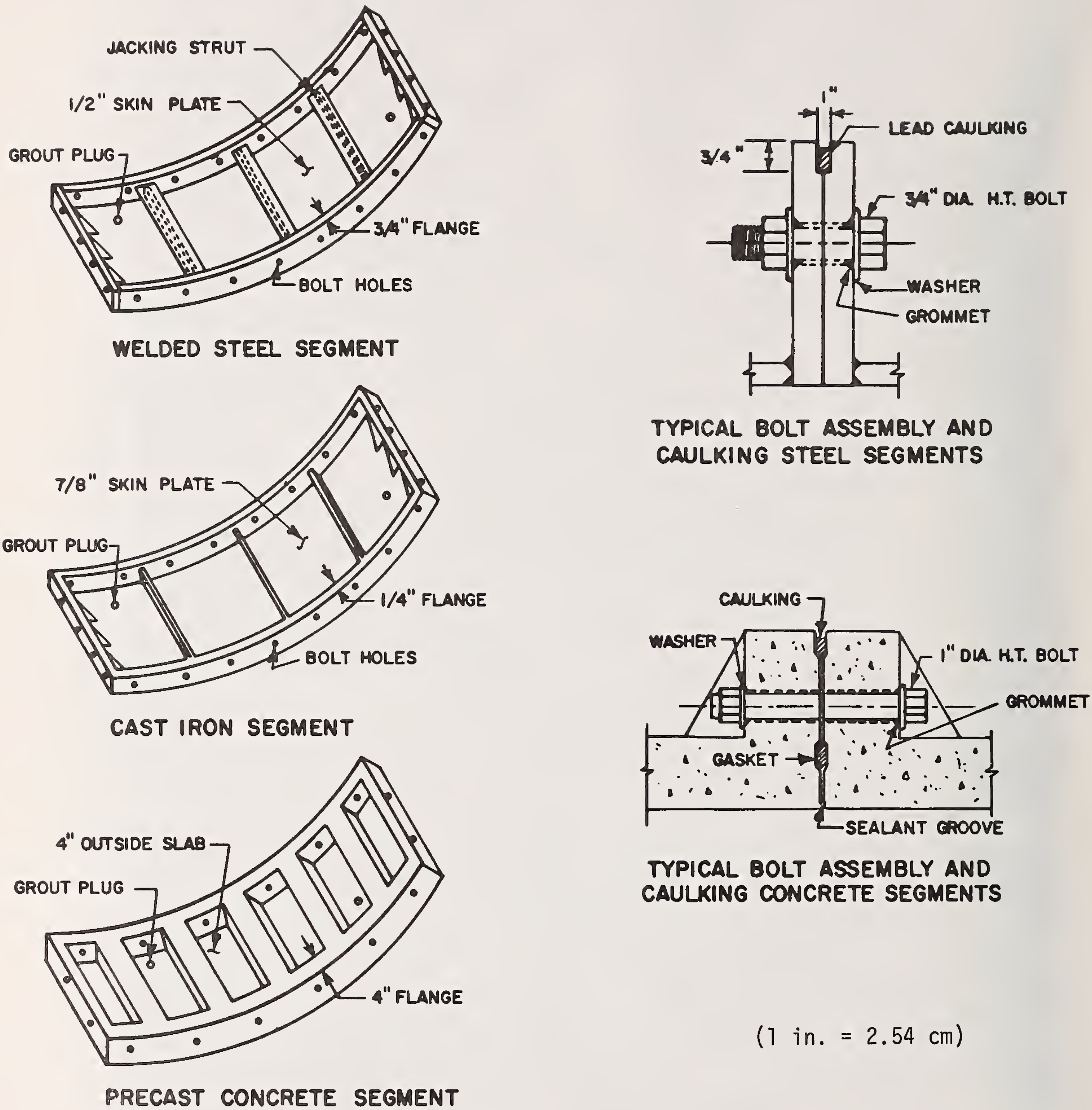


Figure 15 - TYPICAL CHANNEL SECTION SEGMENTS
(From Birkmyer, Ref. 17)

individual, contiguous, prefabricated pieces limits the methods of waterproofing for groundwater control in the permanent structure. Covered in Volume 1 of this study are numerous diverse methods of controlling groundwater during construction, many of which could be used in conjunction with this type of tunnel contruction. Keeping water out of the permanent structure however, means sealing thousands of short joints and bolt holes. To give an idea of the relative lengths of joints to be sealed, a typical single track rapid transit cast-in-place tunnel lining would have from one to five linear feet (0.3 to 1.5 m) of joints for each foot (0.3 m) of tunnel, depending on the number and lengths of pours. A comparable segmented lining would have twenty to forty linear feet (6.1 to 12.2 m) of joints per foot (0.3 m) of tunnel, depending on the size of segments. It is not practical to surround the structure with a continuous membrane outside the segmented lining, and it would add considerably to the cost to place a membrane inside the lining and then add another structural cast-in-place concrete lining inside that.

4.20 CAST IRON SEGMENTED LININGS

Important factors affecting the type of sealing used for segmented linings are the fabrication, configuration and material used in the segments. The earliest segmented linings were made of cast iron with a continuous skin or web 3/4 to 1-1/2 inches (19 to 38 mm) thick on the outside circumference of the ring. The sides, or flanges, of the segment give it stiffness and permit bolting to adjacent segments. Depending on the size of the segment, additional stiffeners are added between flanges to keep the web thickness and segment weight to reasonable values. In casting the segments, provisions are made for handling holes and holes in the skin to place gravel and/or grout in the annular space between the ring and the ground left by the passage of the shield skin. A sample of cast iron segment details is given in Figure 16.

Cast iron is highly resistant to both chemical and electrolytic corrosion. All outside segment flange surfaces are machined to a close tolerance insuring a tight fit and a lining that is reasonably water-resistant when bolted together. In addition to machining flange faces a caulking groove is cast and machined on the inside flange edges. After bolting adjacent flanges together, a strip of lead is forced into the groove with a small air hammer (caulking tool) having an appropriately sized tip. If any leakage occurs the lead can be removed and reset even in the presence of water inflow. Effectiveness of the caulking depends on pressure of the lead in the groove rather than on adhesion. Bolt holes are another possible source of leakage as they are larger than the bolts to allow for fabrication and erection tolerance in bolting segments and rings together. Grommets or gaskets are placed under washers at head and nut ends. When tightened these grommets are squeezed into

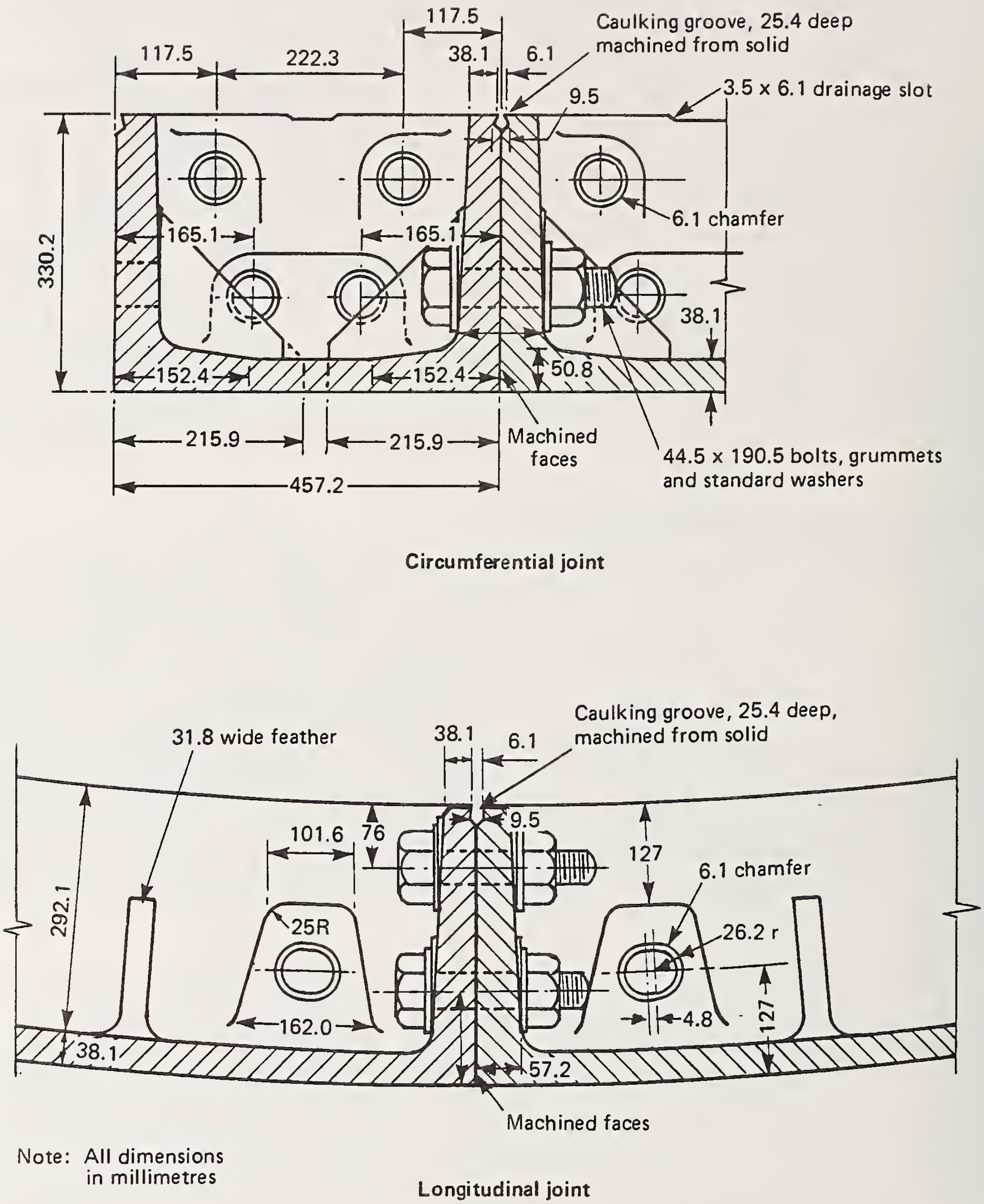


Figure 16 - CLYDE TUNNEL LINING
(From TRRL Report
335, Ref. 34)

the space between bolt and hole. Prior to about 1960 hemp gaskets dipped in red lead were used for grommets but these have been replaced by polyethelene gaskets which are more efficient and much easier to handle.

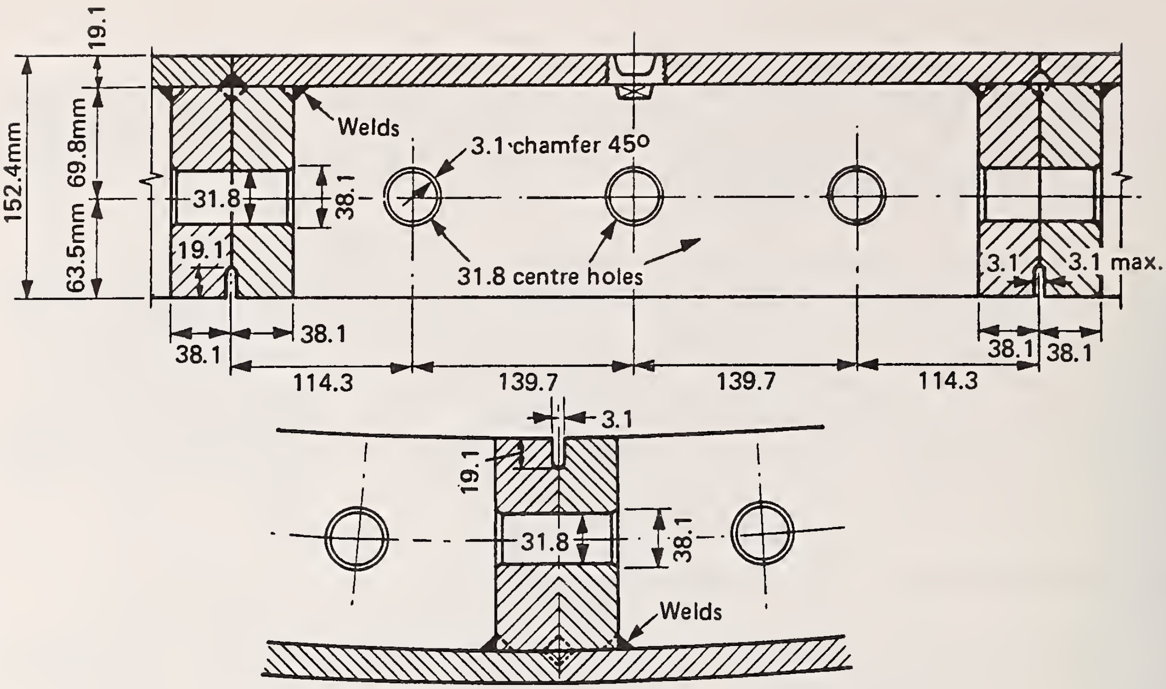
Alternates to cast iron segments are cast steel or ductile iron which are less brittle but much more costly. These alternate cast rings have been used for short sections of cast iron segment tunnels. They are usually limited to areas of high stress such as transitions from rock to soft ground or midriver sumps as in the Hudson River crossings. The cast steel and ductile iron rings receive the same caulking and grommet treatment as the cast iron rings.

4.30 FABRICATED STEEL SEGMENTED LININGS

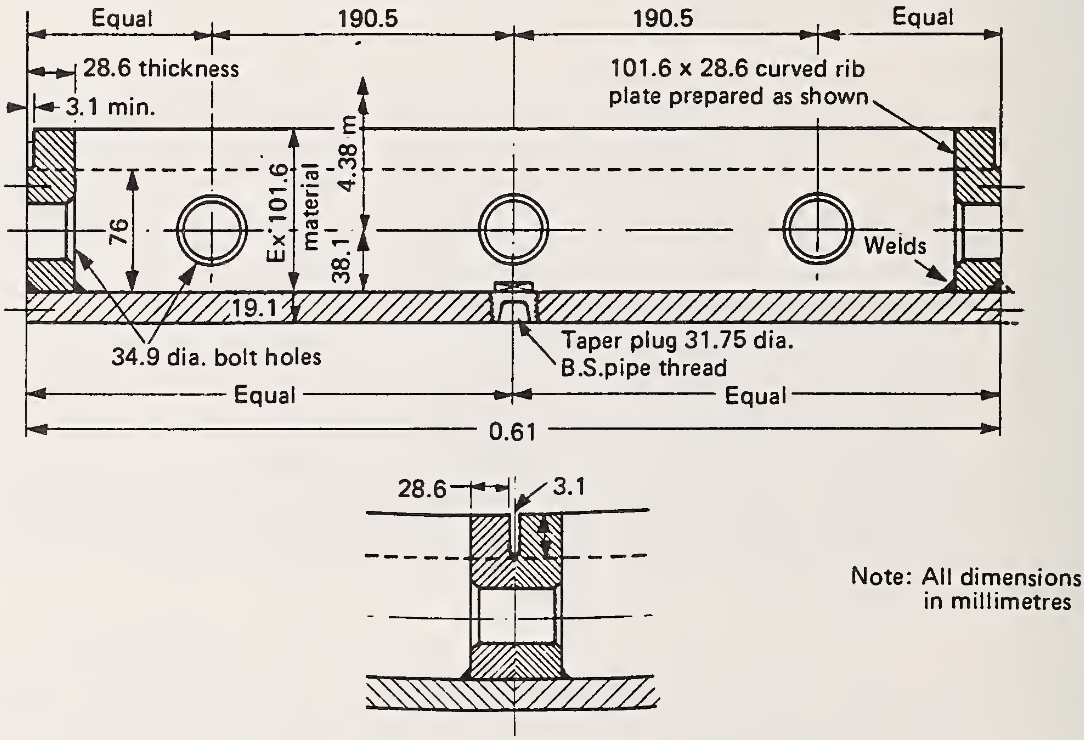
Improvements in steel fabrication and shop material handling techniques have made fabricated steel segments a competitive alternative, and they are now used in place of cast iron for many tunnels. All soft ground tunnels on the BART and some on the WMATA systems used fabricated steel segmented linings (See Figure 1). As steel is more susceptible to corrosion than cast iron it must be covered by a protective coating and bonded to a cathodic protection system.

Fabricated steel segments generally require the same waterproofing procedures as cast iron assuming they are used as a single lining. In a situation where a secondary cast-in-place concrete lining is required for a smooth finish or other reasons, the sealing of segments can be less stringent. Depending on the head of water, machining of flanges, grommets, and even caulking may be omitted. In some cases an edge sealing strip of epoxy or soft plastic may be used to reduce water inflow until the secondary lining is place. In a compressed air driven tunnel where the secondary lining is to be placed behind the face while the tunnel is still pressurized, no special precautions are needed (unless there is considerable air leakage) as the air pressure will keep the tunnel dry until the secondary lining is placed. Thus it can be seen that the constrution method can sometimes have an effect on the measures used to keep the permanent structure dry. A sample of steel segment details is given in Figure 17.

Where a secondary lining is not required, fabricated steel segments will be coated against corrosion, the segment flanges machined, bolts grommeted, and the flanges caulked (usually with lead). On the BART system, the owner, BART, let a separate contract for manufacture of all soft ground tunnel segmented linings, which were then delivered to individual construction contractors, as required for installation. The lining contract which was bid competitively, included designs for fabricated steel, cast iron and precast concrete linings. The bidders



Detail of Dungeness 'A' tunnel lining



Detail of Dungeness 'B' tunnel lining

Figure 17 - DUNGENESS TUNNEL LINING
(From TRRL Report 335,
Ref. 34)

of fabricated steel and cast iron had some advantage over those bidding concrete. Because of the anticipated difficulty of sealing concrete segments against relatively high heads of water, only a portion of the total tunnel lengths involved could be bid as precast concrete. The contract was let for all segmented linings to be of fabricated steel.

On the WMATA system the linings were not let on a separate contract but left to the individual construction contractor to furnish. The contractors were given the option of providing a fabricated steel segment lining, a cast iron and ductile iron segmented lining, or a two stage primary lining with a secondary cast-in-place lining. Since individual construction contract packages were comparatively small compared to the system-wide contract on BART, and the cost to mobilize a lining fabrication plant is high, most contractors elected the two stage support and lining system.

For the fabricated steel segment liner on BART, and both the steel and cast iron linings for WMATA, the following water protection measures were specified. Segments were covered on the outside with a two-component, chemically-cured, coal tar epoxy coating, and on the inside with a two-part, self-curing inorganic zinc silicate coating. Bolt grommets were supplied made of polymerized plastic, and lead caulking strips were used in the caulking grooves between segments. It is safe to say that this is typical, common practice for waterproofing cast iron or fabricated steel segment tunnel linings in the United States today. Placing of lead caulking between steel segments of a BARTD tunnel is illustrated in Figure 18.

4.40 PRECAST CONCRETE SEGMENTED LININGS

There is a greater variety of configurations and sealing methods for precast concrete segments than for cast iron or steel. Many studies have been done on the concrete segment shapes, erection procedures and sealing methods, and there will probably be many more in the future. The potential for savings that are possible with precast concrete have spurred the search for improvements in these areas. In England a number of improvements have been made and used in expanding precast concrete linings for firm ground. See Figure 19 for an example of expanding linings. While these are not directly applicable to the problems of bolted and sealed linings necessary for water bearing ground, many lessons have been learned on shapes able to withstand combined ground and shield shove loads without cracking. Some of these lessons have been applied to use of segmented linings in rock tunnels to be discussed later.

With steel and cast iron segments the major concern with leakage is through the joints, the plates themselves are completely watertight if fabricated properly. This is primarily

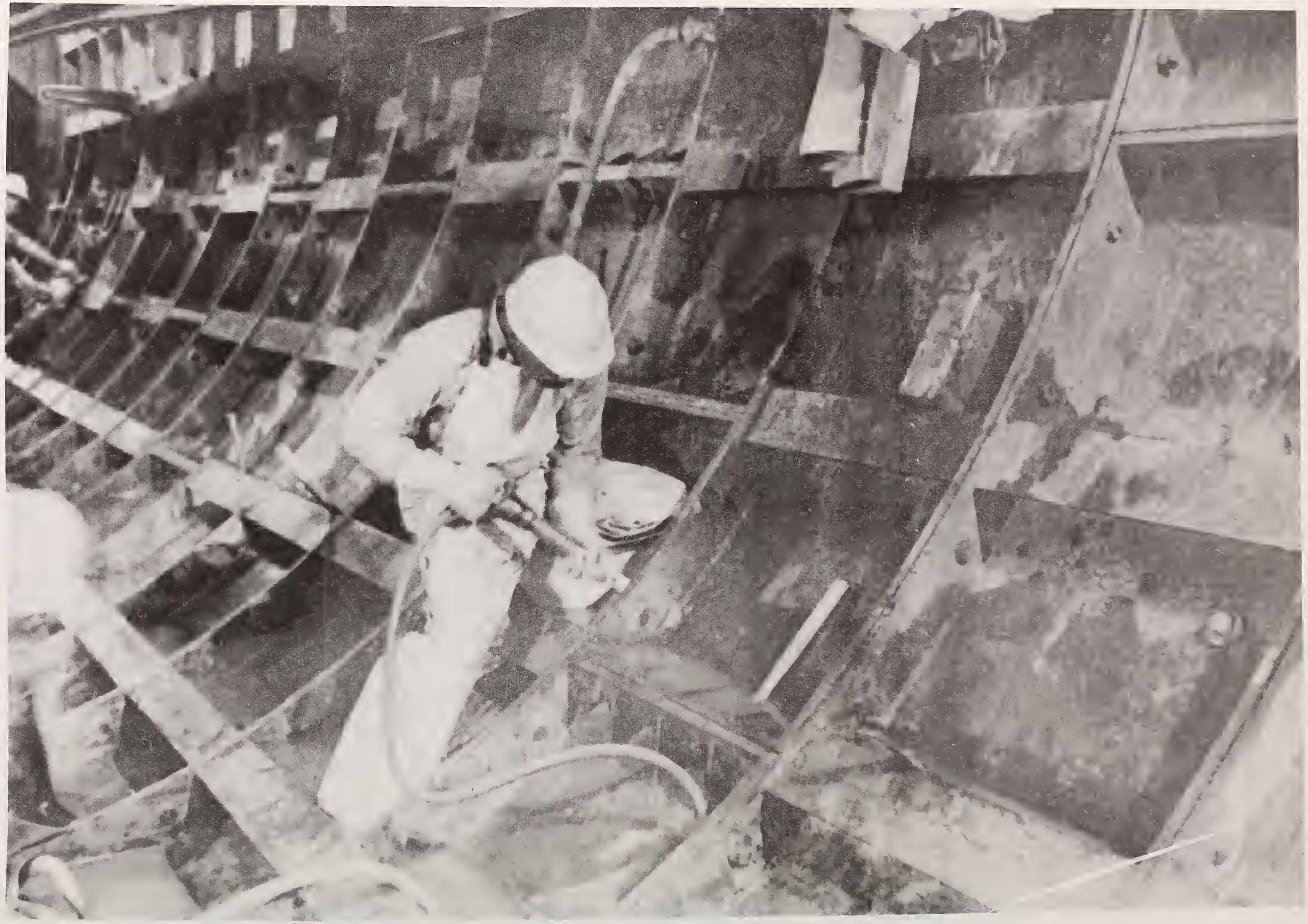


Figure 18 - PLACING LEAD CAULKING IN BARTD TUNNEL FABRICATED STEEL LINING
(Courtesy of MacLean-Grove & Co.)

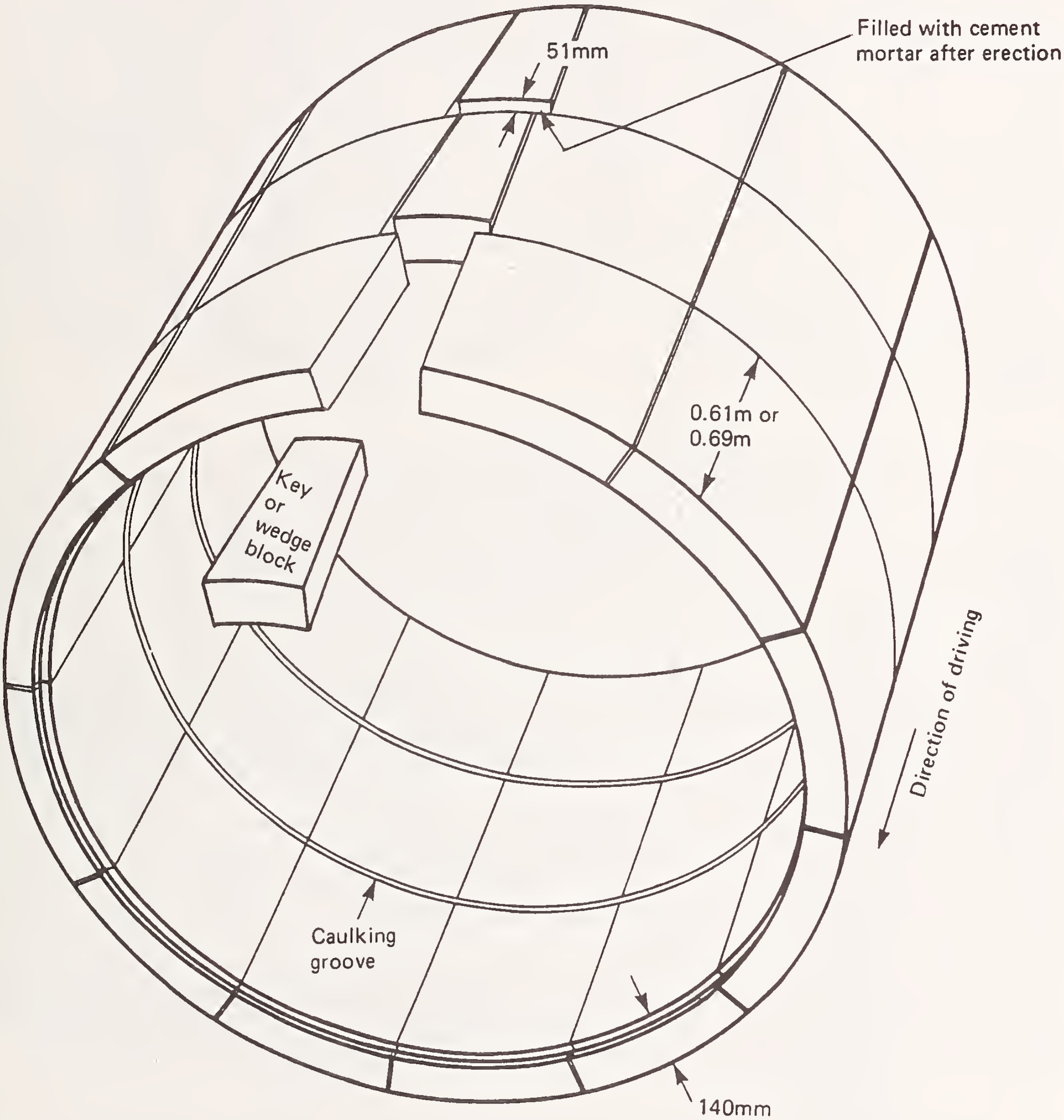


Figure 19 - WEDGE BLOCK EXPANDED CONCRETE LINING
(From TRRL Report 335, Ref. 34)

true of concrete segments also but concrete being brittle, there is more of a possibility of leakage through cracks caused by improper handling, shield shove loads, and ground loads. The concrete material in segmented linings is generally superior to a cast-in-place concrete lining. Precast segments are made in casting yards where mixing, placing and curing of the concrete is performed under much more favorable conditions than cast-in-place tunnel concrete. Although the art of casting with steel forms and controlled materials, ideal water-cement ratios and steam curing results in very accurately dimensioned segments, they cannot match that achieved by machined metal plates. To compensate for this, designers of concrete segments often specify a sealing strip or gasket on all flange faces in addition to caulking. This will be discussed in greater detail with other methods of waterproofing concrete segments. Other designers have come up with composite segments, basically of concrete, but with embedded steel plate or angles for flange or bolt bearing surfaces.

As stated previously, there are more segment configurations possible with precast concrete than with fabricated steel or cast iron. Some forms of bolted concrete linings are similar in shape to steel linings with heavy webs, flanges and stiffeners arranged in a parallel configuration. The segments designed for BART and Metro de Caracas are examples of this. Other shapes of bolted segments have been devised in Europe and Japan. See Figures 20, 21 and 22 for examples of concrete segments from England and Japan. Unfortunately translated literature of current Japanese practices in this field is scarce, but there has been considerable research done in Japan on fabricating and sealing concrete segments. Reports have been made on some innovative European segmented linings. In Germany, two segment designs referred to as the Rhine Tunnel segments and the German Tee Rib, utilize curved bolts to avoid use of pockets for placing bolts. Recently a very efficient type of segment was used in a section of the Munich subway beneath the river Isar. See Figure 23. It is not the purpose of this report to discuss all the possible shapes of tunnel lining; this has already been done (Ref. 17, 34, 47). However the shape of the segments often has some bearing on the type of waterproofing used and the procedures for its installation.

The sealing of concrete segments is subject to much study and experimentation. While various types of adhesive sealers have proven effective for low heads of water, higher pressures require caulking and/or gasket seals on joints that are rigidly bolted as with steel and cast iron. Instead of lead caulking in flange grooves, asbestos-cement or epoxy-elastomerics are most commonly used, but other types of caulking have been investigated and used. In 1968 testing of two new alternative caulking systems were performed for BART (Ref. 15). The first consisted of a caulking yarn seal in the root of a caulking groove covered with an epoxy adhesive. This system showed

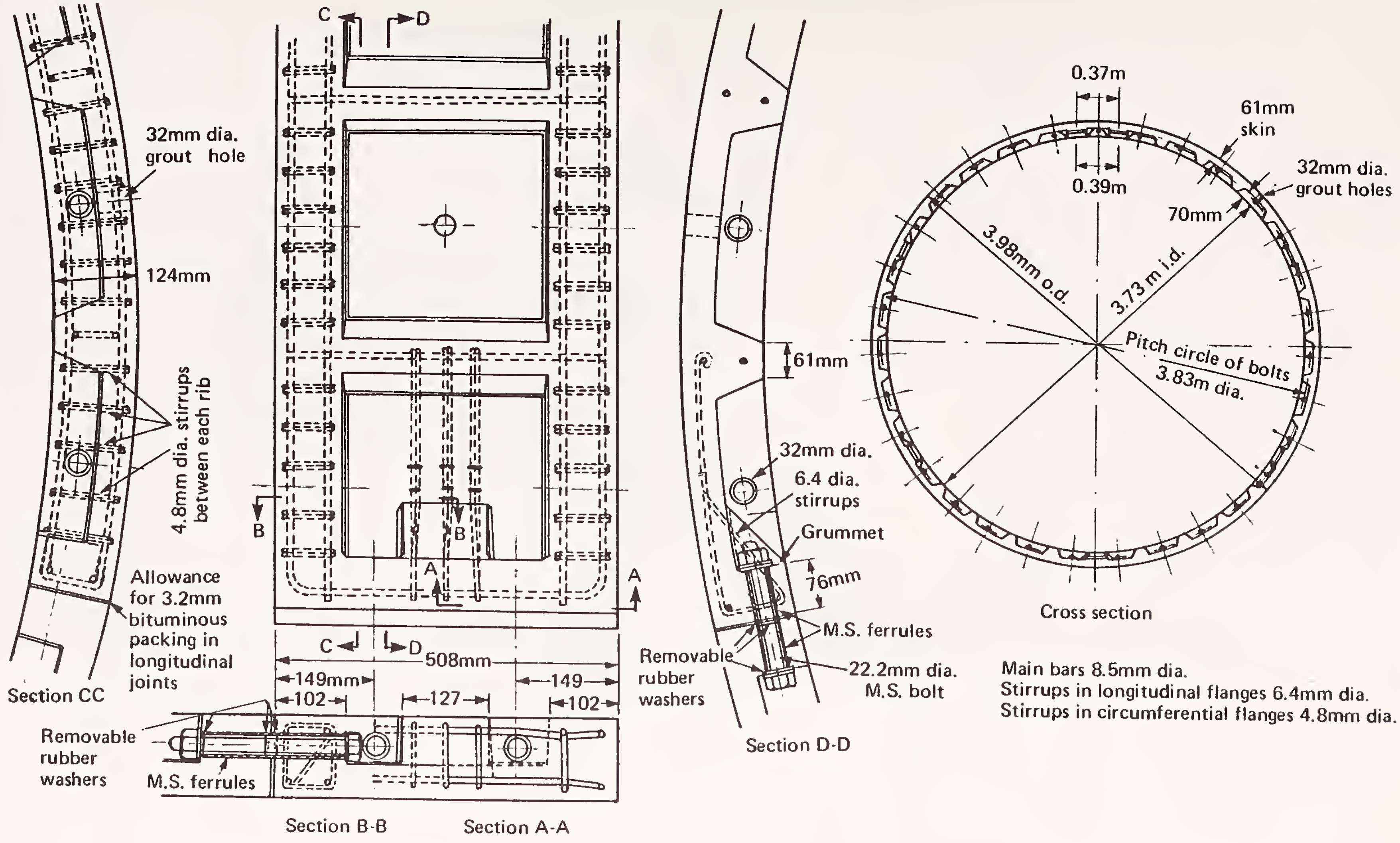


Figure 20 - LTE BOLTED LINING FOR CENTRAL LINE EXTENSION TO ILFORD
(From TRRL Report 335, Ref. 34)

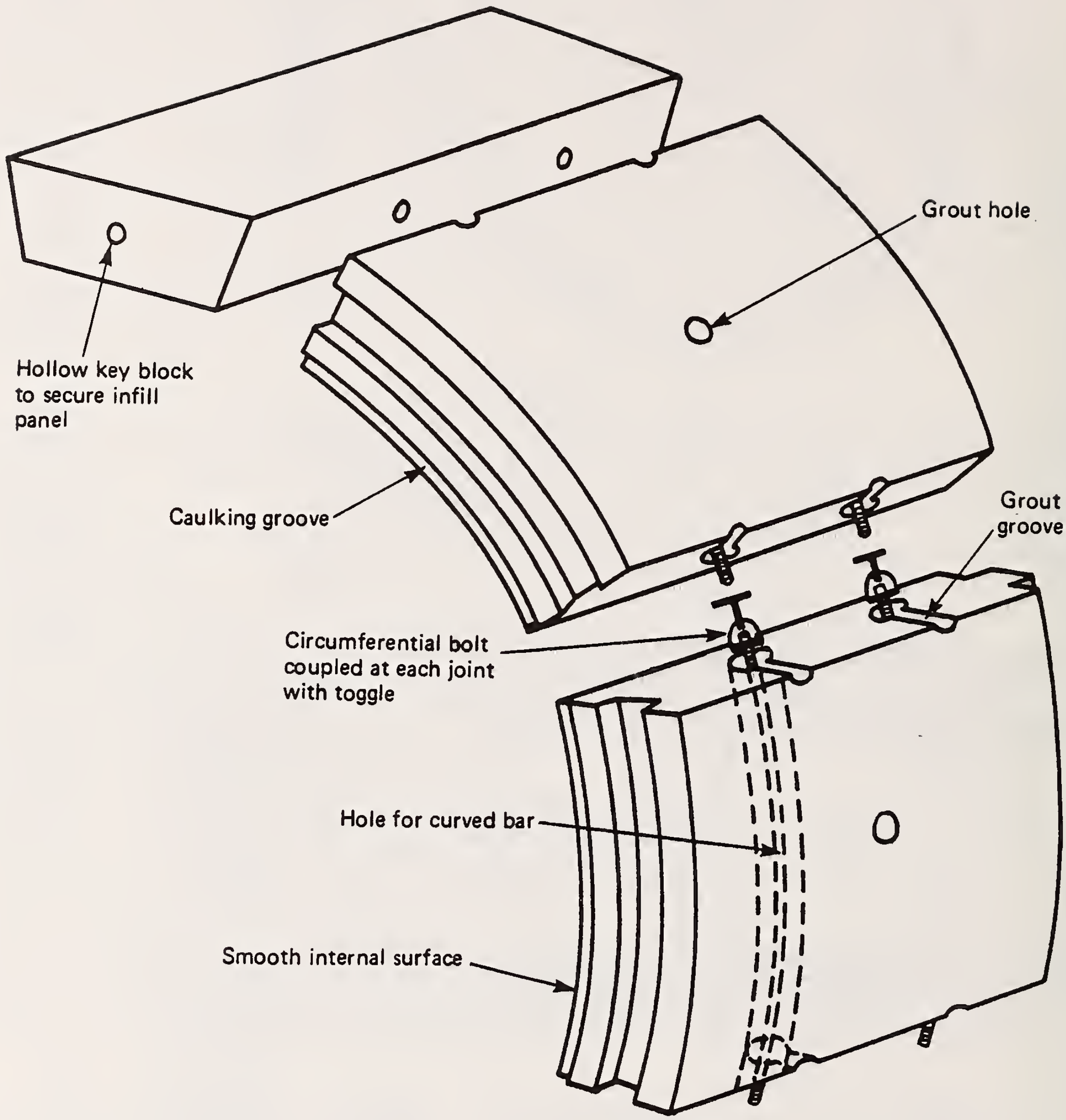
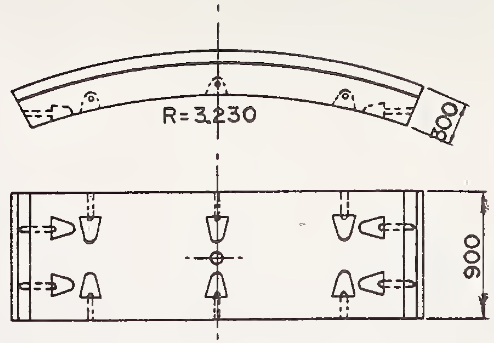
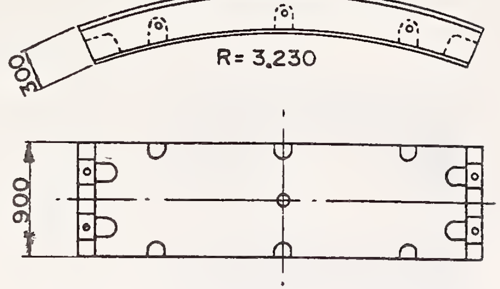


Figure 21 - CHARCON UNIVERSAL GROUTED SMOOTH BORE CONCRETE LINING
(From TRRL Report 335, Ref. 34)

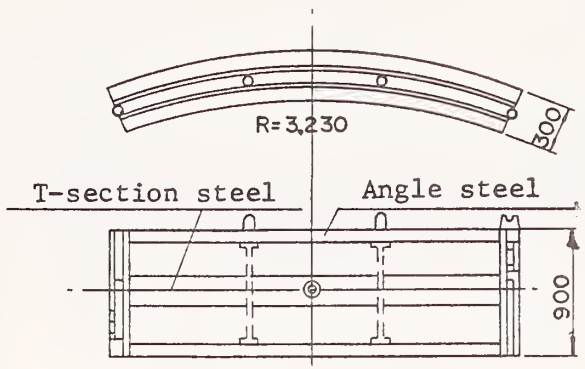


Smooth bore reinforced
concrete segment
(curved bolts)

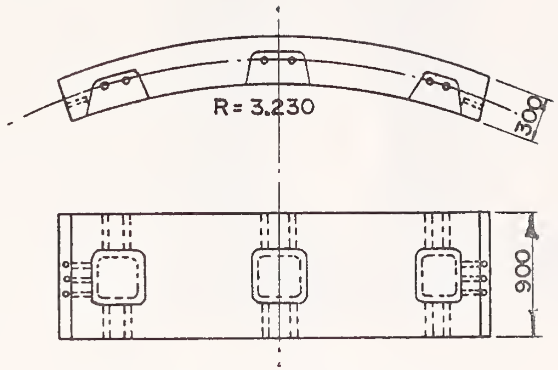


Smooth bore reinforced
concrete segment
(straight bolts)

Note: Units are m. and mm.

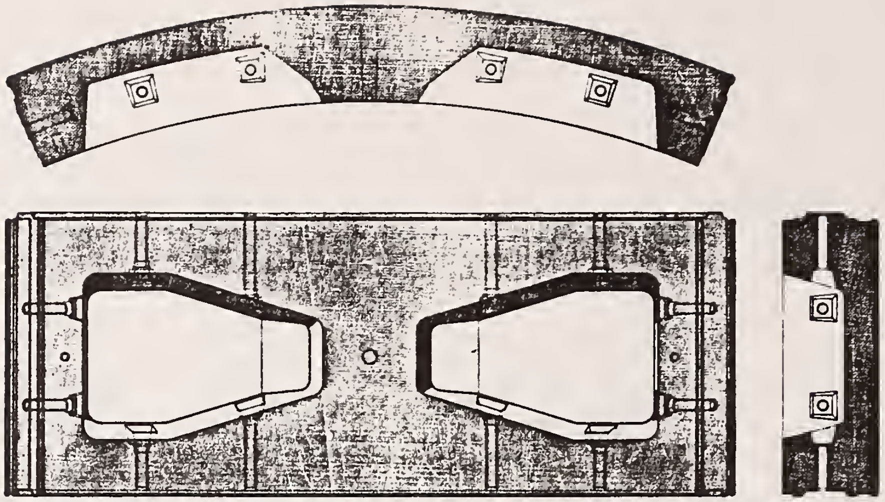


Smooth bore composite
steel and concrete segment
(pin and mortise)



Ribbed-type reinforced
concrete segment
(straight bolts)

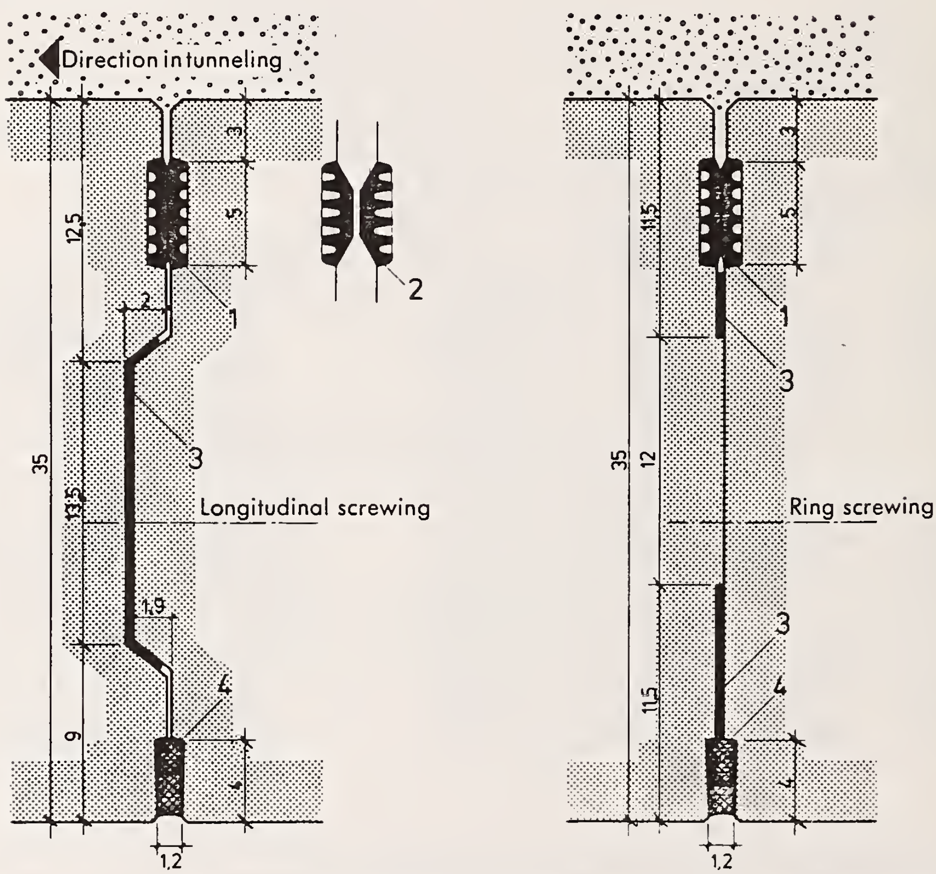
Figure 22 - TYPES OF CONCRETE SEGMENTS USED BY JAPANESE NATIONAL RAILWAYS
(Courtesy of Japan Tunnelling Society)



Precast segment

Note: Units in cm.

Forming of joints



1 Compressed solid rubber seal, 2 Compressed solid rubber seal prior to mounting, 3 Plastic packing, 4 Sealing of joints

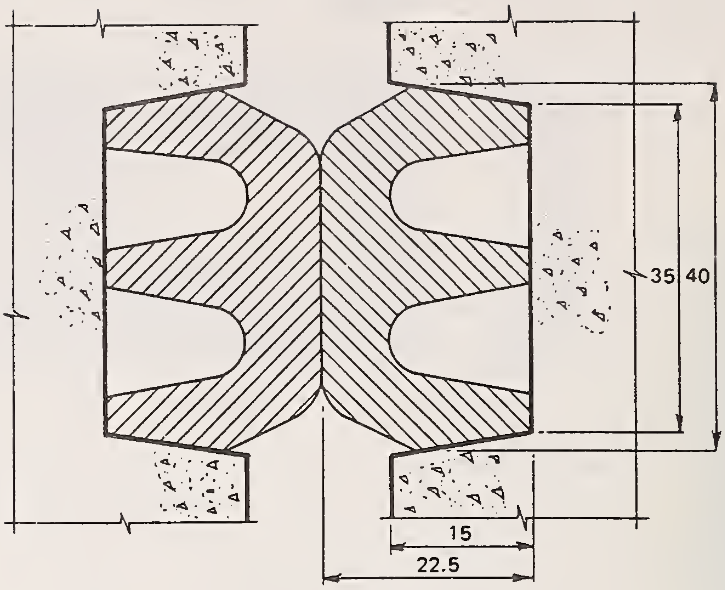
Figure 23 - CONCRETE SEGMENTS - ISAR TUNNEL - MUNICH
(Courtesy of Wayss & Freytag)

limited success for applications where little or no joint movement occurred. The second test used several shapes of neoprene "O" rings with epoxy adhesive. This method proved a more effective seal than the first. In Ref. 17, Birkmyer suggests a variation to this second test method utilizing an inflatable "O" ring pressurized by injecting a liquid filler.

Gasket seals fall into two categories. They may be fairly firm and preformed or soft with adhesive qualities. Examples of preformed gaskets are butyl rubber as used in the Tokyo and Munich subways and cellular neoprene as used on some WMATA tunnels. For the concrete segments of the Metro de Caracas, the contractors were given a choice of preformed gaskets of butyl rubber, polysulfide, or neoprene. Soft plastic adhesive gaskets include polysulfide, semi-cured butyl rubber and coal tar epoxy. These may be sprayed or troweled on the segment joint face. In the United States, Commercial Shearing Inc. has developed a gasket material called TSE consisting of a two-component butyl-polymer material for use on its steel liners. It can be manufactured in various degrees of hardness for spraying or extrusion as a formed gasket. Gasket seals, whether formed or plastic, are usually considered a primary sealant to be followed by caulking. The soft plastic seals are more subject to damage and contamination by dirt under adverse tunneling conditions. A variety of current and proposed preformed gasket seals are illustrated in Figure 24.

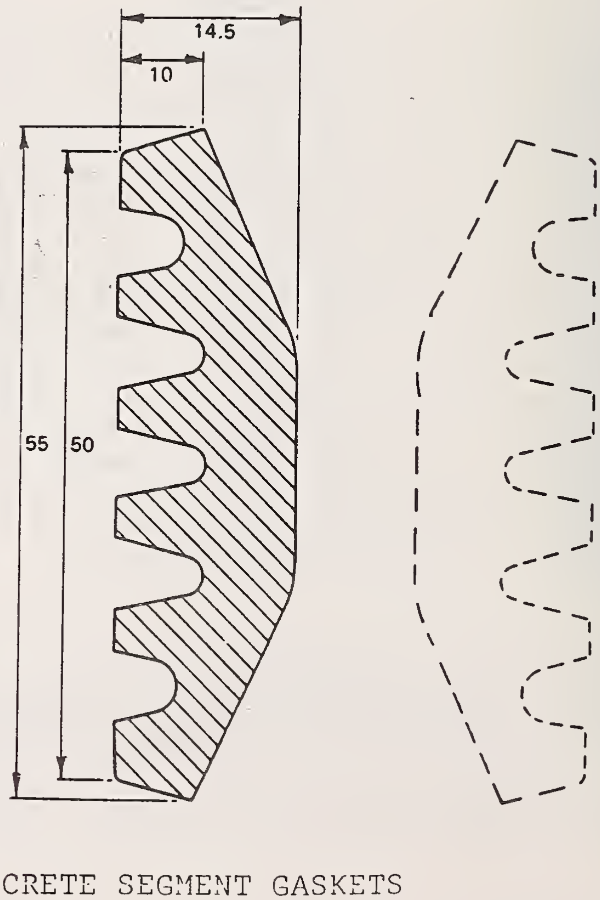
An interesting practical demonstration for comparing the use of fabricated steel and precast concrete tunnel lining segments was recently completed in Baltimore. The Urban Mass Transportation Administration (UMTA) sponsored the demonstration in the construction of the twin tubes of the Lexington Market Tunnels of Baltimore Metro system. The parallel tubes, each about 1,500 feet (460 m) long, were driven and lined under one contract, one with a fabricated steel segmented lining and the other with a precast concrete segmented lining. Compressed air was used for groundwater control during construction of each. The final sealing of the steel segments consisted of a molded epoxy compression seal, placed between flanges, outside the bolt circle, and traditional lead caulking in the flange caulking grooves. The concrete segments were sealed using a preformed gasket in grooves in the flanges. The gasket was the same configuration used successfully in the Isar River Tunnel of the Munich subway and illustrated in Figure 23. Both sealing systems were successful in their respective tubes.

A report being prepared for the Department of Transportation by W. D. Wightman, et al. of UTD Corporation gives considerable detail on this large scale comparison experiment. Overall progress in the two tubes averaged the same rate per day for excavation and lining. After adjustments for minor differences in the tubes the estimated construction costs per foot were approximately equal, with the slight material cost



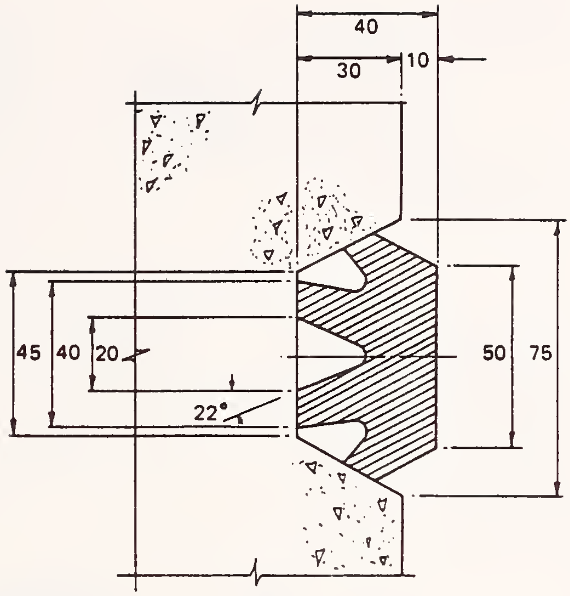
Proposed flexible joint gasket between bolted concrete segments, Ahmed Hamdi Tunnel, Egypt (Courtesy Sir William Halcrow and Partners)

Note: Units in mm.



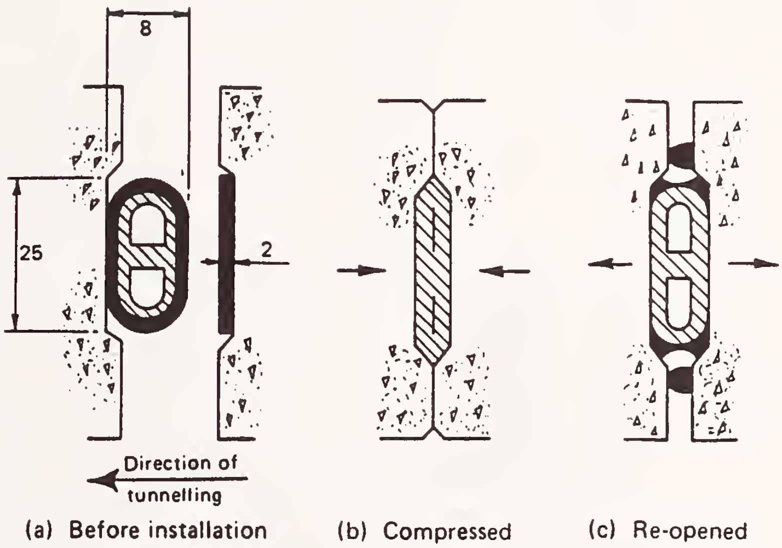
Joint gasket between bolted concrete segments, Isar River Tunnel, Munich, West Germany

Figure 24 - SAMPLE CONCRETE SEGMENT GASKETS
(From CIRIA Report 81, Ref. 25)



*Proposed joint gasket for concrete segments, West Germany (*Courtesy *STUVA)*

Note: Units in mm.

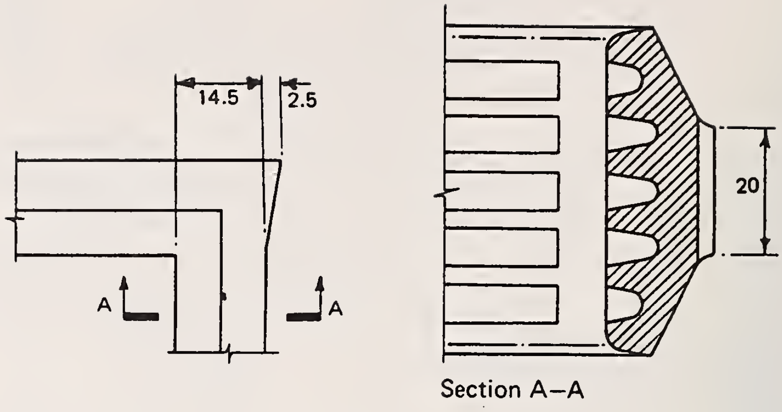


Proposed flexible joint gasket between concrete segments, Japan

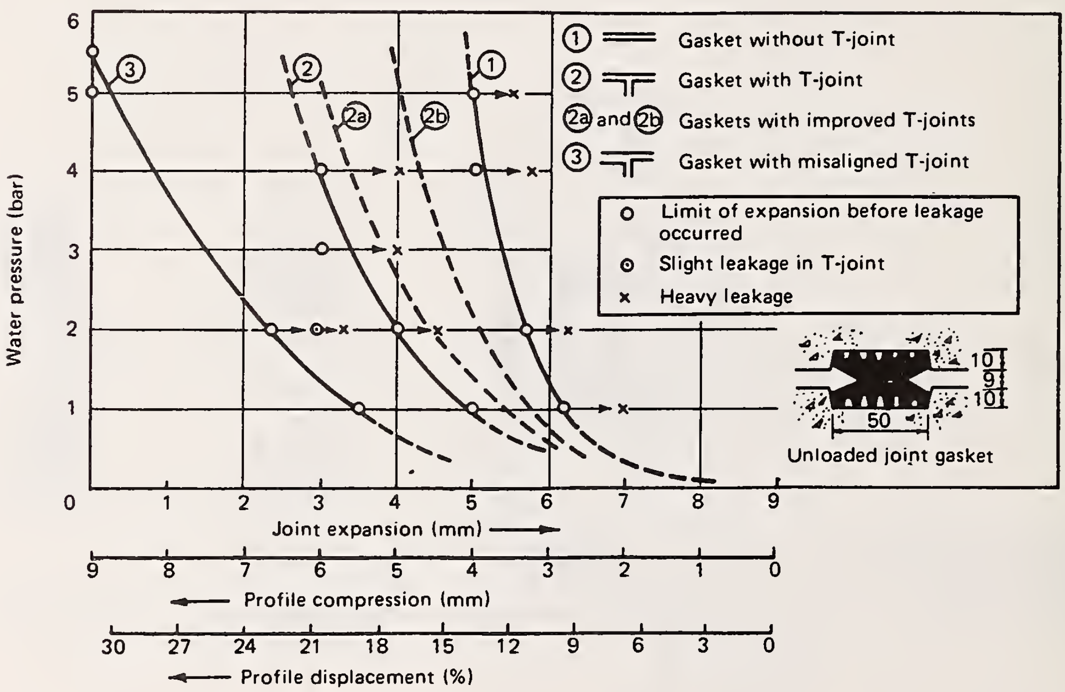
Figure 24 - SAMPLE CONCRETE SEGMENT GASKETS (continued)
(From CIRIA Report 81, Ref. 25)

Joint gasket between bolted concrete segments, with improved corner detail, West Germany

(Courtesy STUVA)



Note: Units in mm.

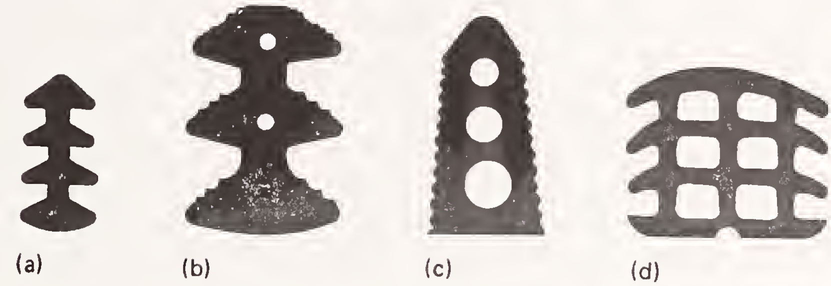


Results of tests on joint gaskets, West Germany (Courtesy STUVA)

Figure 24 - SAMPLE CONCRETE SEGMENT GASKETS (continued)
(From CIRIA Report 81, Ref. 25)

Leakage duct gasket, UK (Courtesy Colebrand Ltd)

Experimental leakage duct gaskets, West Germany (Courtesy STUVA)



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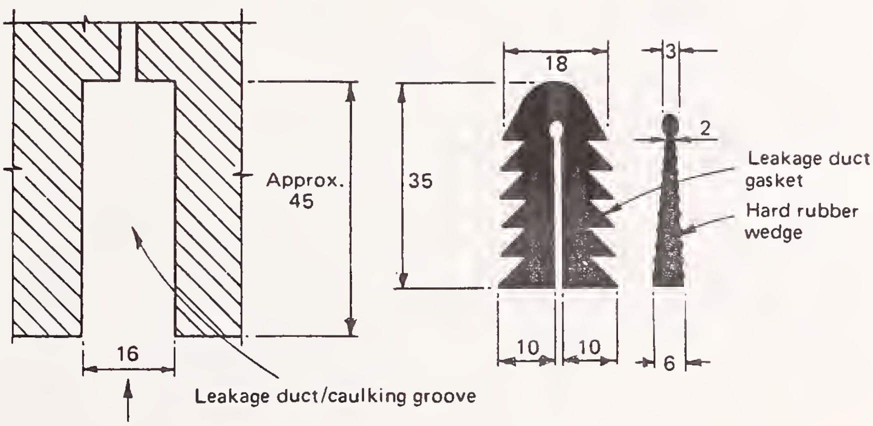


Proposed leakage duct gasket, West Germany (Courtesy STUVA)

Figure 24 - SAMPLE CONCRETE SEGMENT GASKETS (continued)
(From CIRIA Report 81, Ref. 25)

advantage of the concrete segments offset by higher lining erection costs. The report, however, points out that in both tubes the major cost item was the purchase of lining material. Since the high cost of the precast segment forms is spread over a relatively short tunnel length, the concrete lining should prove more cost effective for longer tunnels, or in a system where the forms can be used for several contracts.

While there are potential savings in the use of concrete segments, there are some drawbacks to their use and some precautions that must be observed. The first and most obvious is their weight which is considerably greater than steel or even cast iron. This means heavier erection equipment and more difficulty in an already crowded shield. This is particularly true in a small tunnel. The use of polymerized concrete which increases concrete strength considerably is being studied and may aid in reducing concrete segment thickness. The questions of dimensional integrity and brittleness have been mentioned previously. While gaskets may help to overcome the drawback of not having machined faces, it may be difficult when handling and placing segments to keep from ripping or contaminating these gaskets. Another problem in handling concrete segments is the possible spalling of edges and corners in transporting or setting the segments. As long as precautions are taken to overcome these difficulties, there is no reason why concrete segments cannot be a competitive tunnel lining in the United States just as they are in Europe and Japan.

4.50 SEGMENTED TUNNEL LININGS IN ROCK

While segmented tunnel linings have been used almost exclusively in soft ground tunneling, there have been several rock tunnels built in the United States in the last few years using precast concrete segments. Each of these tunnels was excavated by a mining machine with a shield cover and the concrete segment lining erected within the tail. This type of lining is more adaptable to a mining machine tunnel which leaves a smoother bore than drill and blast excavation. As rock can be considered firm ground these tunnel linings are not bolted as in soft ground tunnels. Pea gravel and cement grout backpacking insure a fairly uniform load on the lining. While there has been a mixed degree of success in these several tunnels, it should be considered that each tunnel differed from the others and because of the small number of such tunnels built to date the construction method is still in the experimental stage.

The Castaic Tunnel No. 2 was completed in 1969 for the Metropolitan Water District of Southern California (MWDSC). It had a rough bore of 26 feet (7.9 m) in a marginal rock material that bordered on being soft ground, and was lined with 4-foot wide (1.2 m), 1-foot (0.3 m) thick, composite 3-segment rings. Each segment consisted of unreinforced concrete with two 5-inch

(127 mm) steel ribs embedded close to the inner face and protruding from the ends of the concrete. The ribs were temporarily bolted during construction and later welded to form continuous circular ribs with concrete between. The project set a record at the time, for completed tunnel, with an average of 112 feet (34 m) per day (Ref. 135).

The 5.5-mile (8.8 km) long San Fernando Tunnel, also built for the MWDSC, through ground similar to Castaic Tunnel, was only 2000 feet (610 m) short of completion in 1971 when a disastrous natural gas explosion and fire killed 17 men and delayed completion more than two years. The lining system consisted of precast, reinforced concrete segments. The segments were 4 feet (1.2 m) wide and 10 inches (254 mm) thick, with four segments to the ring for the 22-foot (6.7 m) excavated diameter. The segments were manufactured by the contractor with a unique no-slump concrete and mobile casting machine method. The rate of tunnel construction exceeded 3500 feet (1070 m) one month and set a record of 277 feet (69 m) in one day. As in the Castaic Tunnel, the mining machine shield was propelled forward by jacking against the lining (Ref. 106). Both the Castaic and San Fernando Tunnels were constructed in the relatively soft rock of the California Coastal Range.

Excavation of the hard rock Buckskin Mountain Tunnel for the Water and Power Resource Service's Central Arizona Project was begun in 1976. Unlike the Castaic and San Fernando tunneling machines, a full face tunnel boring machine (TBM) was used, 23 feet (7.1 m) in diameter, which gripped the tunnel sidewalls to advance forward without applying any pressure against the segmented lining. The 5-foot (1.5 m) wide, four-segment rings were erected in the shield tail 40 feet (12 m) behind the face. The segments were 7 inches (178 mm) thick and were reinforced on both faces with welded wire fabric. The four-segment ring was arranged to have staggered longitudinal joints obtained by using a slightly tapered pattern, with the arch and invert segments fitting opposite hand to the side segments. The segments were not bolted and had to be held in place until backpacked. The invert was placed on pea gravel bedding. After the ring was completely erected pea gravel was placed through the lining followed by neat cement grout to complete the backpacking. All joints were tongue and groove with a foamed strip joint filler set back from the inside face with the remainder of the joint filled with neat cement grout. A caulking groove on the inside face was filled with an elastomeric joint filler to provide a watertight seal (Ref. 105, 136).

The Park River Auxiliary Tunnel is being constructed for the Corps of Engineers at Hartford, Connecticut as the final stage of a flood control project. It is a 22-foot (6.1 m) I.D. inverted siphon tunnel, 9,100 feet (2,800 m) long, and up to 200 feet (60 m in depth. When the construction contract was being let, three alternate excavation and lining procedures were designated: 1) drill and blast excavation with a variable thickness cast-in-place lining, 2) machine boring with a constant thickness, but variably reinforced lining, and, 3) machine boring with reinforced precast concrete lining. The third option with a precast concrete lining proved to be least expensive.

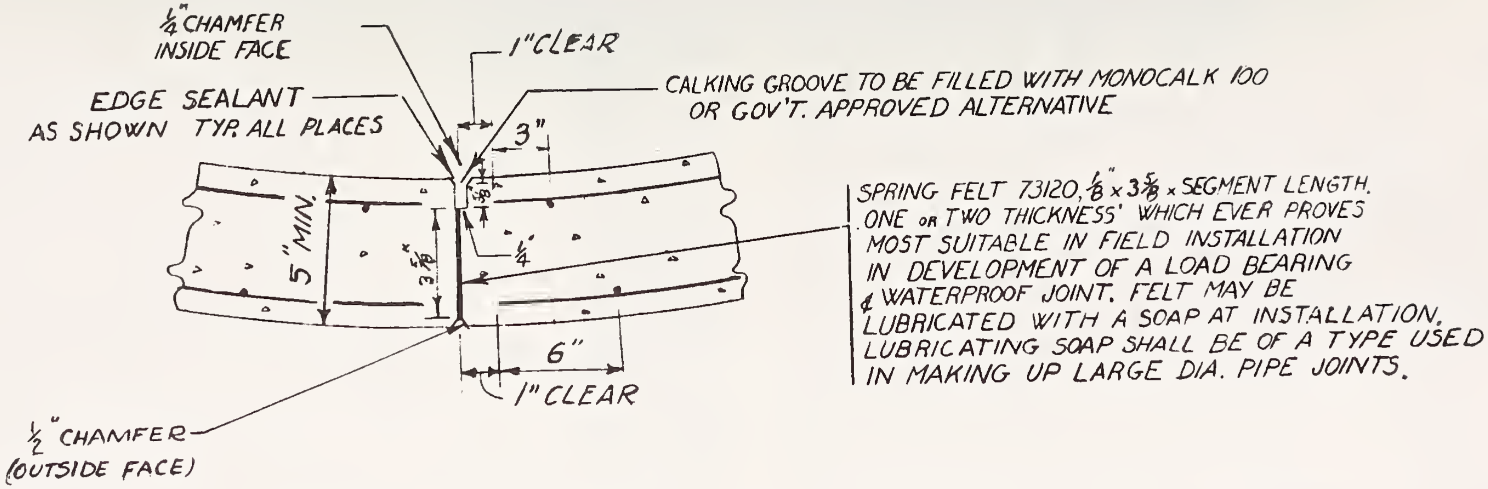
The four-segment rings are 6 feet (1.8 m) wide and 9 inches (229 mm) thick. The segments are reinforced in both faces with ship-lapped transverse joints and rounded longitudinal joints. The tapered longitudinal joints provide a staggered (saw tooth) pattern as in the Buckskin Mountain Tunnel. The transverse joints are sealed with a continuous "O" ring gasket and butyl rubber sealant backup strips, while the rounded longitudinal joints are sealed with a neoprene gasket. Both have caulking grooves on the inside face filled with an epoxy sealant.

A recent example of a rock tunnel with concrete segmented lining that has run into unexpected difficulty is the Stillwater Tunnel being constructed for the Water and Power Resource Service in Utah. The excavation and lining procedures are similar to the Buckskin Tunnel though this tunnel is considerably smaller with a 9-foot (2.7 m) excavated diameter.

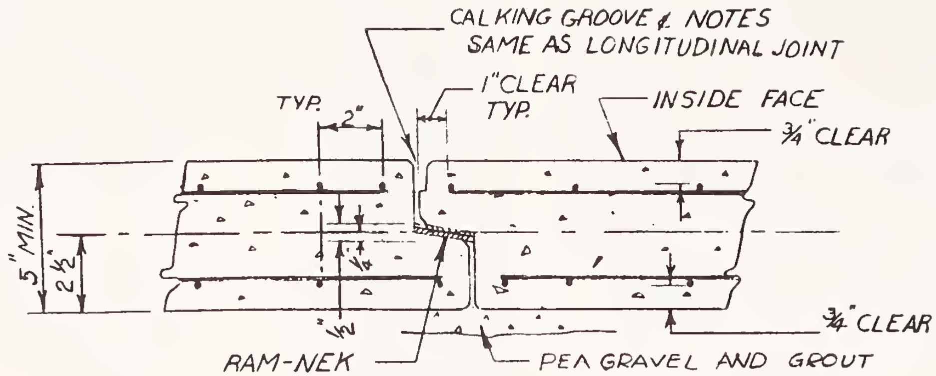
The four-segment rings are 3-feet(0.9m) wide with a 5-inch (127 mm) segment thickness. The circumferential joints are ship-lapped while the longitudinal joints are butt joints. These are detailed in Figure 25. Unlike the Buckskin Tunnel lining where the joints are at the quarter points these joints are on the vertical and horizontal diameters. All segments are reinforced with wire fabric on both faces, and the lower segments have continuous haunches for setting rail for tunnel equipment. A preformed filler is used between segments at the joints, and caulking grooves at the inside face are filled with a polymer sealant.

The total tunnel length of 42,000 feet (12,800 m) was to have 36,000 feet (11,000 m) excavated by a TBM and lined with precast segments, and 6,000 feet (1,800 m) excavated by drill and blast, and lined with cast-in-place concrete. As excavation proceeded with the TBM it was found that unexpected internal squeezing rock pressures were present. The pressures exerted on the TBM when it shut down on weekends made excavation difficult and on some occasions hand mining around the shield was needed. The higher than anticipated pressure also cracked the lining in some areas. As a result excavation was halted after 7,000 feet (2,100 m) had been driven for reevaluation of design and construction procedures.

In general there have been problems, other than those caused by squeezing ground, connected with using unbolted precast concrete segmented linings in tunnels. Transporting,



LONGITUDINAL JOINTS



(1 in. = 2.54 cm)

CIRCUMFERENTIAL JOINTS

Figure 25 - JOINT CONFIGURATION AND SEALING DETAILS
STILLWATER TUNNEL
(Courtesy of Harrison Western)

handling, and supporting the heavy segments, as well as sealing problems and the prevention of cracked corners should be improved with additional experience and experimentation. This type of lining has many advantages over a two-stage process of temporary support and cast-in-place secondary lining. Casting and curing the concrete in a yard produces a stronger, more dense and water-resistant concrete than can possibly be placed in the tunnel. Other important factors are improved safety and production. The entire concrete pour crew works in a plant outside the tunnel. With proper equipment the ring can be rapidly erected and backpacked, producing a completed tunnel instead of one with merely temporary support. Since the excavation and support procedures are uniform throughout the tunnel the crews and production need not vary depending on varying needs for temporary support, and the crews can develop greater proficiency performing uniform operations. There is no need for safety miners to keep the back scaled or retimbering crews to reblock or add sets for taking increased weight. Cleanup is considerably simpler and the light colored walls reduce lighting requirements. A flat invert can be cast in the lining, as was done in Buckskin Mountain, Park River, and Stillwater with inserts to simplify setting rail. Sidewall inserts can aid in carrying ventilation, compressed air and water piping, and power lines.

Although this type of lining has not yet been used in transportation tunnels in rock in the United States there is no reason why it could not be. Development of this system for rapid transit tunnels including improved waterproofing methods should be encouraged, and should lead to lower tunneling costs.

5.00 OTHER METHODS OF CONTROLLING GROUNDWATER

5.10 CHEMICAL GROUTING FOR CUT-AND-COVER AND SOFT GROUND TUNNELS

5.11 General

Groundwater flows through the interconnected pores, cracks and fissures which occur in many natural soils and rocks, and man-made fills. If these paths could be plugged in a continuous curtain surrounding the structure no groundwater could infiltrate the structure. In theory this can be done by impregnating the soil or rock mass around the structure with an appropriate impervious "grout". At the time of injection grouts are in a fluid state. After injection they become fully or partially solid by the action of polymerization, chemical reaction, deflocculation, or expansion due to saturation. These solidified grouts change the engineering properties of the soil or rock mass (Ref. 53). The grout can be made up of very fine particles such as cement or clay held in suspension during injection, or it may be composed of chemicals in solution. The solution type are called nonparticulate or chemical grouts and generally have lower viscosities than the particulate grouts.

In the fluid state these chemical grouts can penetrate into very fine voids and cracks in soil and rock where particulate grouts cannot enter. One of the major uses of chemical grouting in soft ground tunneling is the control of groundwater flow. It has the advantage of changing the engineering properties of the ground mass with little disturbance to the existing structures. This is of particular importance when excavating in urban areas (Ref. 53). Figure 26 shows soil limits for grout injectibility of various types of grout.

The chemical grouting philosophy in England and the Continent is quite different from that in the United States. In Europe a careful soil investigation followed by a well-planned grouting program is the most common primary means of groundwater control during tunneling in soft grounds with compressed air used as a last resort. The grouting programs envisioned during design are left sufficiently flexible for the grouting contractor to plan the actual procedures to be used (Ref. 29). If the voids in the soil are sufficiently large, an initial grouting is performed using particulate grouts such as cement or bentonite. Then a second grouting with nonparticulate chemical grout is used to seal the small pores so that the ground is essentially impervious. If this is successful in controlling groundwater flow during excavation it may contribute appreciably to the watertightness of the finished underground structure providing a durable grout has been used. In fact, the attitude in Europe is that the main advantage in grouting for groundwater control is that if it is successful it is less costly than any

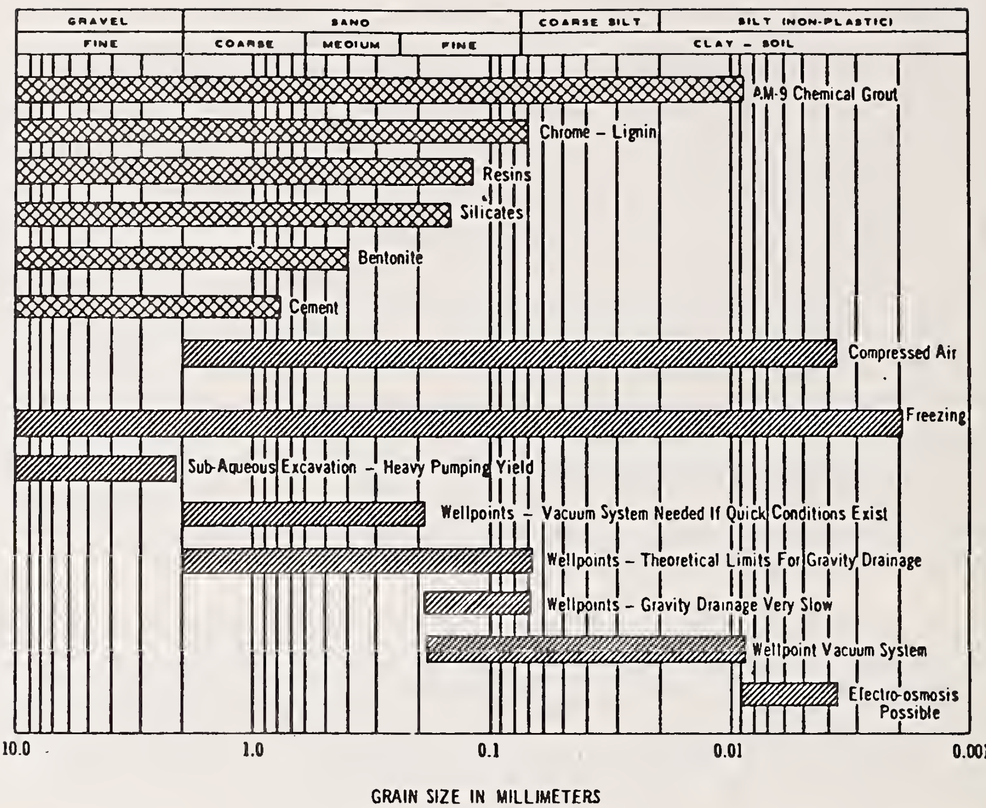


Figure 26 - SOIL LIMITS FOR GROUT INJECTIBILITY
(From Am. Cyanamid, Ref. 2)

other sealing method and does not require enlarging the tunnel to accommodate the sealing materials. Grouting after excavation is not considered as successful as grouting ahead of the tunnel face (Ref. 60). In the United States grouting is usually used only in real problem areas where water cannot be controlled by other means (Ref. 124).

In cut-and-cover tunnel construction the soil outside of the excavation ground supports (sheet piling, soldier piles and lagging, slurry walls, etc.) can be grouted to control the influx of groundwater both during and after construction (Ref. 48). The pressure used must not be great enough to damage the ground support system.

5.12 Required Characteristics

In selecting a grout to reduce the permeability of a soil certain characteristics of the soil and of the grout must be considered.

The primary site investigation should note the various types of soils which will be encountered, their in-situ porosity and permeability, pore-size distribution, grain-size gradation, temperature, soil and groundwater chemistry including pH, and groundwater flow characteristics (Ref. 68, 95).

With this knowledge of the soils to be treated the grouts can be selected by choosing for each soil the grout whose characteristics will best suit the soil and placement conditions. Sometimes it is advantageous to grout first with a particulate grout, a viscous chemical grout, or a chemical grout containing suspended particles, followed by a second treatment with a grout of lower viscosity. The first treatment fills the larger voids quickly and economically and the second treatment, with the more expensive, low viscosity materials, makes the mass essentially impervious.

The main significant characteristics used in selecting a grout are its viscosity, setting time, strength, permeability, durability, toxicity, flammability, and cost. Selection of the most advantageous grouting system is a very difficult task and requires the assistance of experts in the grouting field with extensive and varied experience.

Viscosity

Viscosity is the internal resistance of a fluid to flow. The closer the viscosity of the grouting fluid is to that of water, which is 1.0, the finer the voids which will be penetrated by it. The finer the voids filled the more impervious the treated soil. Therefore in the selection of a grout for preventing the infiltration of groundwater into an underground structure the viscosity of the grout must be chosen to meet the requirements of the soil to be treated. If two grouts are equally desirable for an application the one with the lower viscosity will enter the soil pores at a lower pressure and more rapidly than the more viscous, and would probably be the better choice, if costs are similar.

Setting Time

Generally, treatment of only a given thickness of soil around the structure is required to seal off the groundwater flow. With this in mind it is important to have the grout set as soon as this penetration is reached. Grout flowing beyond this zone is essentially wasted and uneconomical. Knowing the characteristics of the soil formation to be grouted -- grain size, pore size, porosity, permeability -- and the characteristics of the grout -- viscosity, grain size if particulate -- it is possible to judge fairly accurately the time for the grout to penetrate the required distance into the formation. If set does not take place rapidly once the grout is in the location desired, groundwater may wash it out or dilute it so that it will not set, the grout ingredients will then enter the groundwater, and the soil in the area will still be pervious. If setting takes place too rapidly the necessary volume of soil will not be infiltrated and gaps in the grout curtain will permit leakage.

Controlled setting times are also necessary to insure the filling of the finer voids. The fluid takes the path of least resistance filling the larger voids. When the grout sets in the larger voids it prevents further flow through the large voids and the fluid grout is thus forced to go into the finer voids.

The temperature of the soil mass affects the setting time and must be considered in the design of the grout to be used. Setting times in use vary from a few seconds to many hours.

Strength

The intrinsic strength of the gelled grout and the strength of the treated ground must be sufficiently great so that the water head to which the structure could be subjected will not damage the integrity of the grout curtain by stripping it from the soil or rock particles.

Permeablity

Permeablity of the gelled grout alone is relatively unimportant. The permeablity of the treated soil is the criterion which must be used in grout selection. A grout which will give satisfactory results used in coarse sands and gravels may not decrease the permeablity of fine sand or silts at all even though the intrinsic permeability of the gelled grout itself may be very near zero.

Durability

For permanent control of the groundwater around a structure using chemically grouted soil as the primary control the gel or solidified grout must not deteriorate with time or under the influence of the soil and groundwater chemistry. The useful life of transportation tunnels is generally around 100 to 150 years. Therefore, in the selection of chemical grouting as a primary means of preventing the infiltation of groundwater into the underground structure, the durabililty of the grout is a prime criterion. Chemical grouting has not been used for a sufficiently long time that its durability in field conditions is well established.

Toxicity

A material which is toxic to the persons handling it without protective clothing and equipment or in a space with limited ventilation is less desirable for use than a material which does not present such hazards. By scrupulous adherence to safety regulations these hazards can be overcome and the material can be used. However, if the material can enter the groundwater in sufficient concentration to be harmful to plant or animal life or detrimental to existing structures it may be neccessary to abandon its use. This could happen due to poor application or improper selection of the grouting materials. If the grout material is slow in setting it may be diluted by the groundwater such that the concentration of ingredients is too low to set or it may be completely or partially washed away by the groundwater. Also if the soil or groundwater chemical conditions are such that they cause a degradation of the gel, in time the components may pollute the groundwater to a dangerous degree and the impermeablity of the treated soil will be lost. By close supervision of the application of properly chosen grouts these problems can be avoided.

Flammability

It would be unthinkable to use a material which would be flammable after placing and setting in an underground structure. However, some good grouts contain flammable ingredients. If these are highly volatile they may become explosive when mixed with air during handling for injection. Again by scrupulous adherence to safety regulations these hazards can be overcome.

Cost

The costs of the grouting material, the equipment for injecting it, and the labor needed to place it are all important economic considerations. These must be considered along with the above characteristics in the selection of possible grout systems. Costs of grouted soils in the United States vary from about $100/cy to $500/cy of grouted soil, depending on the type of chemical grout used and the characteristics of the soil. If the grouting procedure interferes with excavation as in the heading of a tunnel the effective price could be even higher. Relative costs are summarized in Volume 1 of this report.

5.13 Types of Chemical Grouts

There are several types of chemical grouts in use today. They are generally all more expensive than the cement and bentonite grouts. They make soils impervious by filling the voids and binding together the granular components in a gel network. Discussions of several of the more promising chemical grouts follow.

Silicate-Based Grouts

The silicate-based grouts are the most frequently used. When first used, a concentrated solution of sodium silicate was injected into the ground followed by a separate injection of the reactant, calcium chloride. A reaction between the two chemicals took place immediately, producing a strong gel (Joosten Process, 1925). Because concentrated solutions were needed for this process the viscosities of the solutions were high precluding penetration into relatively fine sands. The infiltration into coarser materials was slow, not deep, and required considerable pressure. Therefore in treating soils many closely spaced holes were required when using this process. In addition, close control of the mixing of the ingredients within the ground could not be obtained so there was no assurance that complete gelation had taken place (Ref. 126). Many modifications have been made on this process and many successful applications have been completed.

It is now possible to use a silcate solution with the reactant already mixed into it before injection thus saving much time and labor by using a single phase (one-shot) grouting procedure. This also assures the proper proportions of the ingredients within the ground mass exclusive of any changes caused by the groundwater. There are a number of chemicals which have been found to be practical as reactants instead of calcium chloride. By varying the concentrations and the constituents it is possible to produce a wide range of setting times and strengths. For waterproofing purposes a very dilute solution of sodium silicate is adequate and can be as inexpensive as a particulate grout. A dilute solution has a low viscosity so it penetrates fine soils well but is considered a "soft gel" since it adds little to the strength of the soil treated. Of the chemical grouting systems in use the silicates are the least expensive and least toxic. One authority mentions that some miners have complained of skin rash when handling silicate treated soils (Ref. 88). The silicate-based grouts have good durability but do suffer some from leaching and syneresis. Syneresis is an exudation of the water from the gel, producing shrinkkage of the gel. In fine soils this phenomenon does not occur. If the application is correctly done and the grout well adapted to the soil to be treated syneresis should be negligible.

Plant-Product-Based Grouts

Many of the available grouts contain petrochemicals as primary ingredients or necessary reactants. With the increase in cost of these materials and their decreasing availability, grouts derived from other products should be considered for development and wider use.

Lignin-based grouts utilize a by-product of the wood processing industries. These grouts give satisfactory results in groundwater control but do not develop sufficient mechanical strength to improve the structural properties of weak soils. The lignin-base is generally nontoxic but the reactant used to produce the set, a chromium compound, is highly toxic. The chromium in the set grout is in a nontoxic state but, since it is difficult to determine if complete polymerization has taken place and since some leaching of the poisonous hexavalent chromium may occur, there may be some contamination of the environment by the toxic material. There are a number of ways to keep the amount of leaching at a minimum (Ref. 126). These lignin-based grouts have the advantages of low viscosity, a wide range of setting times, easily regulated by changing the dilution, and a basic material which is plentiful, not petrochemical in nature, and inexpensive. As soon as the reactant is added to the base solution the viscosity starts to increase. The increase continues until gelation is complete (Ref. 88). Studies are being done to bring safer conditions to the application of lignin-based grouts.

Two other grouting materials can be obtained from plant products: colophane, the residual product of the distillation of pitch from fir trees to obtain turpentine; and furfural which can be obtained from corn cobs and other vegetable matter. The colophane is only useful in waterproofing and is more expensive than the soft silicate gels that it could replace. The furfural, however, holds great promise for use in the grouting of pervious soils but has been used very little.

Acrylamides

In the 1960's numerous studies were performed using resins, which could be polymerized, as grouts for injection into soils to improve their physical qualities. One grout, AM-9, developed by American Cyanamid Company, worked well and was used in many applications all over the world (Ref. 2, 130, 134). This grout consists of two organic monomers, acrylamide and N, N'-methylenebisacrylamide, and a two-part catalyst system, all in aqueous solution. It has the advantage of a viscosity of only 1.5 cP, which is very close to that of water, so that it can be used successfully to control the flow of groundwater through very fine soils. This low viscosity is maintained almost to the moment of complete solidification. Gel time can be controlled very easily and accurately using varying amounts of the catalyst initiator and/or a fifth ingredient which can inhibit setting.

The water becomes integrated into the space lattice of the gel structures. If the gel is subjected to conditions of low humidity some of this water will evaporate and the gel will shrink. When subjected to wet conditions again the gel will reabsorb the water returning to approximately the permeability of the original gel but the strength of the stabilized mass is not reattained.

The ingredients of the acrylamide grouts are highly toxic and must be handled with complete protection. Acrylamide can penetrate unbroken skin, and prolonged contact with the material will affect the central nervous system. No ungelled acrylamide should be allowed to enter the groundwater.

Phenoplast Resins

Phenoplast resins are polycondensates obtained through the reaction of phenol on an aldehyde. No suitable combinations of phenol and aldehyde for use in waterproofing soils near the surface were found until the 1960's. Most combinations needed elevated temperatures or an acid environment to produce the set and, since surface soils are usually alkaline and within a rather narrow range of moderate temperatures, these were not suitable. In the 1960's a resorcinal-formaldehyde type was developed which could be used both for consolidation and for water-proofing in average soils since set could occur in an alkaline environment at ambient temperatures.

These grouts have low viscosities, about 1.5 cP. This viscosity remains almost constant up to the point of set. Setting time is controlled mainly by the amount of dilution of the grout - the more concentrated solutions setting faster - though some control can be obtained by the use of expensive accelerators.

All the raw materials are toxic and caustic. If the proper porportions are used these should be completely combined into a practically inert mass. This material shows more promise than polyacrylamides (Ref. 126). It was used in grouting the "Beauchamp" sands encountered during the construction of the Paris Area Regional Express Transportation System (Ref. 81, 107). Another application of this grouting material was in the Boulby Potash Mine shaft. There the Bunter sandstone was first injected with a cement grout till the groundwater flow was reduced and then new holes were drilled and the sandstone was grouted with resorcinal-formaldehyde grout (Ref. 35).

Polyurethane

Another material injected into sands to reduce their permeability is polyurethane foam. The foam is produced by mixing polyisocyanates with polyols or polyether. To produce the foam, blowing agents and surfactants are added. This is a two-shot injection process and therefore more expensive. However, the isocyanate can also react with the groundwater to form a foam. This would be a one-shot injection. The blowing agents and surfactants are used to make sure set will take place where no groundwater contacts the injected materials. The foam produced is closed cell so it offers no path to water flow (Ref. 81). The isocyanate is toxic and gas masks must be worn during handling. During application this is the most hazardous of the chemical grouting systems and cannot be used in confined spaces even with "good ventilation" (Ref. 81). A grout of this type was used successfully in the Canelles Dam in Spain. The viscosity of the grout is somewhat high and it is expensive but it is very useful in the treatment of rivulets in karstic and similar conditions (Ref. 126).

Other Chemical Grouts

Many more chemical grouts have been developed, tested, and used successfully but most of these are appreciably more expensive than the ones discussed here. When improved load capacity is required as well as groundwater control it may be economical to use one of these but where only groundwater control is required a "soft silica gel" is probably the most advantageous since it is easy to handle, has fairly low viscosity, low toxicity, and low cost.

The aminoplast resin grouts consist of urea, formaldehyde, and a catalyst. The catalyst must be acid so this grout cannot be used in the calcareous soils or rock which are most frequently encountered. They are, however, not petrochemical derivatives and are low in cost.

A number of colloidal solutions have been tried with success. Some authorities consider these to be "chemical" grouts since the suspended particles are less than 1 micron in size. In this report the inorganic material "bentonite" is considered a particulate grout even when the particles are minute. A number of organic materials have been tried but they have generally been too viscous. They can be used for water-proofing some sands, but, being organic in nature, they may eventually be destroyed by soil bacteria (Ref. 126).

Nonaqueous grouts have also been tried. They are all petrochemically derived, they are viscous and flammable, and some need to be dissolved in a solvent or heated before application.

Emulsions, suspensions of very fine droplets of one liquid in another, have also been used. Bitumens are the materials most commonly used, dispersed in water. These can be used in fine-grain, clayey sands. They leave a jelly-like substance in the soil of low permeability and low strength (Ref. 56). Their advantages are their relatively low price, ease of placement, nontoxicity of ingredients, and good long term stability. They do not add to the strength of the ground mass treated and are petrochemically derived. Various resins and waxes can be put into an aqueous emulsion. These materials are all quite expensive.

Another group of chemical grouts react with the salts in the soil and groundwater. Some require only the presence of groundwater to effect a solidification. An example of this is the polyurethane discussed previously.

Combinations of Grouting Systems

Besides the advantages of filling large voids with particulate or high viscosity grouts and then injecting low viscosity chemical grouts to complete the waterproofing, it is sometimes useful to mix two chemical grouts together before injection. An example of this is the addition of sodium silicate to phenoplast resin grouts to set the material quickly so that it will not be washed out by the groundwater.

Bentonite, when added to chemical grouts before injection, though not imparting strength to the treated mass, sets up sufficiently to prevent the continued migration, under the force of gravity, of the less viscous, slower setting grouts.

Choice of a Grouting System

For every conceivable underground condition there would appear to be a grout or combination of grouts that could seal the voids. The exception to this is ground with such a low permeability that water will not drain from it. The matter of selecting the method best suited to the conditions encountered is very complex. It requires a thorough knowledge of the great variety of materials that can be used separately or in various combinations to produce the result required in the underground conditions on the project. Most of the expertise in this field is to be found within the grout injection contracting firms or the grout manufacturing companies.

In selecting a grouting system, Einstein and Schnitter suggest five steps be taken (Ref. 5). These are:

1. Detailed investigation of site geology, hydrology and ground chemistry.
2. Choice of a group of possible grouts.
3. Laboratory tests made on these grouts.

4. Laboratory tests made on these grouts with soil.
5. Field testing these grouts for final selection.

The grout selection should be done by an expert in grouting. Field testing is important because of the numerous factors which influence the effectiveness of a grout. In fact the gel time for a grout is frequently determined at the project site after field testing, so all influencing factors are included.

Where the permeability of the ground formation is so low that the water will not drain out of the ground, it is extremely difficult or impossible to inject grout and some other means of controlling the groundwater must be used (Ref. 56). Another condition which causes difficulty when grouting is dry soil. Sometimes it is necessary to inject the soil with water before the grout will penetrate it (Ref. 88). Figure 27 shows typical grout applications for various situations.

5.14 Grouting Methods

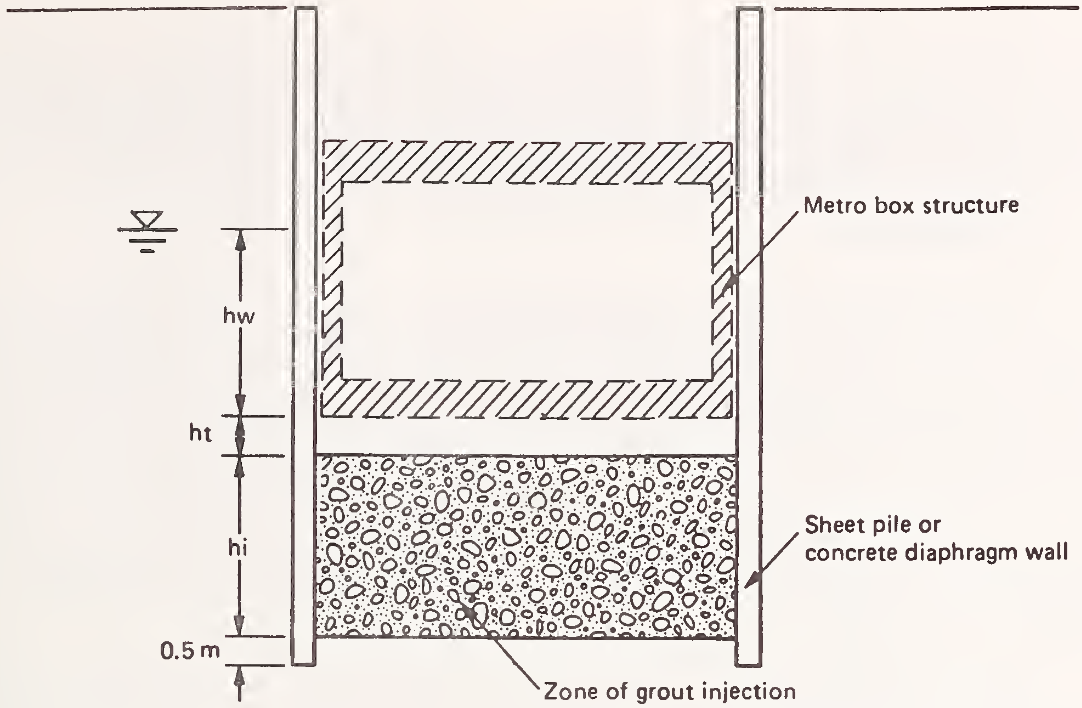
A relatively thin vertical grouted layer, placed to prevent horizontal flow of groundwater, is called a "curtain". A relatively thin horizontal grouted layer, placed to prevent vertical flow of water, is called a "blanket". The required depth of the blanket is dependent on its location and purpose. In Europe in cut-and-cover construction this is occasionally used to seal off the groundwater which would rise through the floor of a cut-and-cover structure constructed utilizing diaphragm walls or steel sheet piling for ground support (Ref. 95). This grouting is done before excavation of the ground between the walls. It is done at a depth such that the weight of the grouted soil remaining after excavation balances the maximum pressure of the groundwater including a safety factor. A half meter layer of ungrouted soil above the grouted material may be included in the design as a buffer against the upward migration of the grout. This treatment should be permanent. It can sometimes serve two purposes -- it keeps the excavation, and later the structure, dry and may eliminate or reduce the need for additional compensating structure weight or piles in tension to prevent uplift of the structure by the groundwater (Ref. 125). See Figure 28.

Another common European use of the grout blanket is as a canopy above the finished structure, but there its usual purpose is strengthening a shallow or weak cover though it can also function to complete the impervious envelope consisting of slurry walls, bottom blanket, and canopy (Ref. 95). See Figure 29.

In placing a grout envelope around an underground structure some systematic method must be used to insure continuity of the envelope. Selection of the method to be used in any situation depends on the underground site conditions, the place in

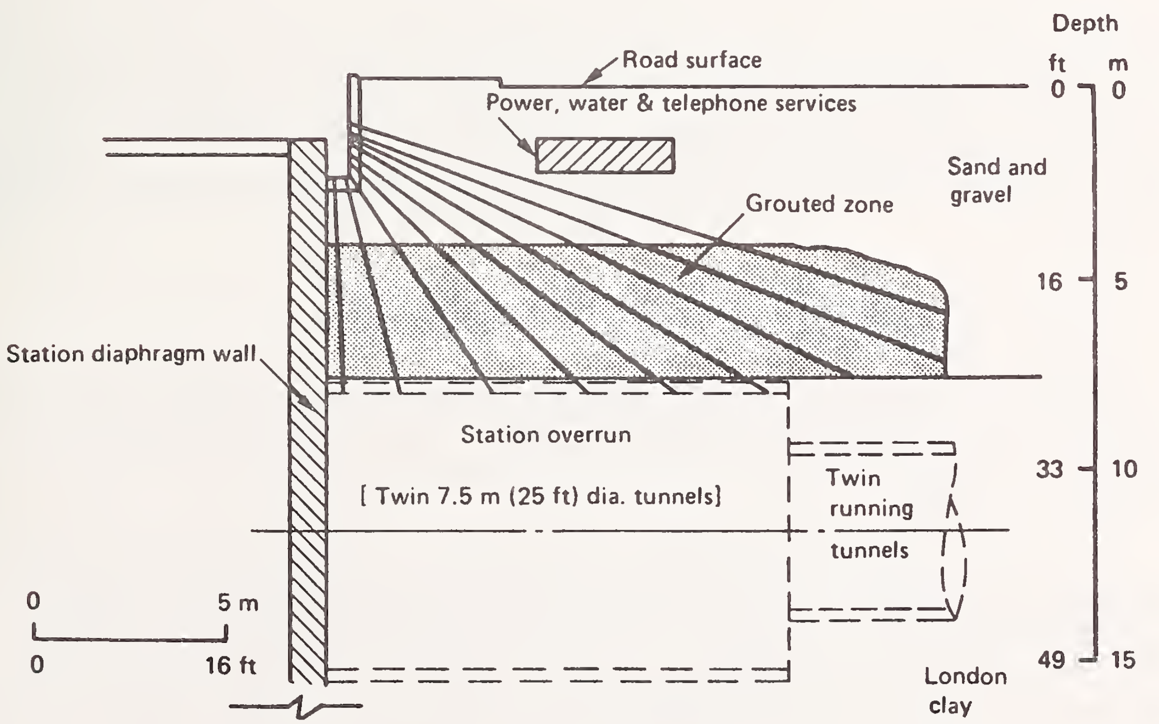
		WATERTIGHTENING	STRENGTHENING
SOILS	Coarse Sands, Gravels	Clay-Cement Bentonite-Cement + Silica Gel	Cement-Clay Cement-Bentonite + Gel
	Sands	Bentonite Gel Silica Gel	Silica Gel + Cement-Bentonite Silica Gel (Tough)
	Silty Sands	Silica Gel (Very Diluted) Acrylamide Resin	Silica Gel, Tough with Low Viscosity Phenolic Resins
ROCKS	Voids (Openings)	Clay (or Bentonite)-Cement	Cement-Clay (or Bentonite) + Weighting Material
	Karst, Large Fissures	Clay (or Bentonite)-Cement + Weighting Material Aerated or Swelling Grouts Hot Asphalt	Grout Foam with Cement
	Medium Fissures	Bentonite-Cement Bentonite-Cement + Fine Weighting Material	Cement Cement-Bentonite Cement-Bentonite + Weighting Material
	Fine Fissures	Bentonite-Cement Very Fine Silica Gel + Bentonite-Cement (Very Fine)	Very Fine Cement Very Fine Cement-Bentonite
	Very Fine Fissures	Bentonite Gel-Silica Gel Acrylamide Resins	Silica Gel Phenolic Resins
STRUCTURES	Filling		Cement-Clay (or Bentonite) + Weighting Material = Fine
	Gluing	Bentonite-Cement	Cement + Admixtures to Make Fluid and Stable (Bentonite)
	Regeneration of Masonry with Open or Exposed Holes	Bentonite-Cement (Very Fine) Aerated Grouts	Very Fine Cement Very Fine Cement-Bentonite
	Active Cracks or Fissures	Acrylamide Resins Mixed Gels Latex Emulsions	
	Fine Fissures or Cracks	Acrylamide Resins	Epoxy Resins, Polyesters ...

Figure 27 - TYPICAL GROUT APPLICATIONS
(From Clough, Ref. 29)



(After Ferrand et al. 1976)

Figure 28 - GROUTING THE BASE OF A BRACED CUT



(After Moller, 1976)

Figure 29 - GROUTING ABOVE THE CROWN IN A MIXED SOIL PROFILE

the construction sequence that it will be performed, the grouts to be used, the above ground site conditions, and the available injection equipment. The most commonly used methods for placing grout to assure complete coverage under a variety of ground conditions is discussed in Volume 1 of this report.

5.15 Application

The application of chemical grouts is a very specialized process which requires strict quality control and constant monitoring. The materials must be accurately measured and pumping pressures must be accurately controlled. Records should be kept of the elevations grouted, type of grouts, amounts injected, and the pressures (Ref. 95). Besides checking the pressure on the grout lines the ground surface and existing structures in the vicinity of the grouting should be watched for surface heave. Some grouting companies use electronic monitoring equipment which will sound an alarm as soon as there is an elevation change of a specified magnitude (Ref. 124).

All ambient conditions which can alter the physical properties of the chemical grout must be observed so that adjustments can be made in the porportioning and pressures. These include temperature, entrained air in the solution, metal in the soil, impurities in the mixing water, exposure of the solution to sunlight, and the use of filler materials in suspension (Ref. 124).

Effective chemical grouting is an art. It requires the expert judgment of the grouting specialist for interpreting the pumping records and field measurements in the light of his knowledge of available equipment, grout types, possible grout variations and combinations, ground formation types, and his experience with the interactions of these variables (Ref. 95). The grouting operation should be kept flexible so that as conditions change the grouting expert is free to alter the program to suit.

5.16 Verification of Grout Application

The solidified grout is not within easy reach for verification of its in-situ characteristics but some means have been devised for determining some of the in-situ characteristics though these tests are still infrequently used and not particularly reliable.

In most cases in Europe both owner and grouting contractor depend on careful construction control and monitoring during injection to prove the effectiveness of the grouting. In the United States contractors are usually told how much grout to use or the desired compressive strength, but tests after grouting are rarely performed (Ref. 68).

After a chemical grout application has been completed it would be helpful to determine the permeability of the treated ground, whether there is proper distribution of the grout in the soil, how complete the gelification is, and what materials, if any, are moving from the grouted area into the untreated soil and the groundwater.

If permeability tests had been made on the ground mass before injection and like tests are made after injection the effectiveness of the treatment can be estimated (Ref. 53). If the injection is made behind leaking ground support walls or tunnel lining the verification of successful change in the permeability is self-evident.

If the grout "take" of the formation is close to the anticipated take it is a good sign. The amount of grout the ground will take is dependent on the porosity of the ground, the rate of flow in the ground, and the setting time. A thorough investigation of the subsurface conditions should supply a fairly accurate figure for the porosity and for the permeability of the formation and these with the viscosity of the grout and the injection pressures determines a good figure for the rate of flow and in most grouts setting times can be closely controlled. The rate of flow, the length of grout hole, and the setting time determine the grout volume and the volume times the porosity gives approximately the volume of grout that should be taken.

Sampling of the grouted soil is difficult since most chemical grouts do not adhere strongly to the sands and silts they have infiltrated and the sample falls apart before its properties have been tested (Ref. 68). Therefore, the tests must be "in situ" if their results are to be meaningful. Some strength tests have proven fairly reliable, but they are not significant for groundwater control.

Some geophysical tests have shown potential as aids in monitoring grouting activities. They are the two types which are most commonly used for remote determination of soil changes with depth; namely, electrical resistivity and seismic refraction. Since the resistivity methods are more sensitive to changes in soil pore fluid, they may be adapted to monitoring distribution during grouting. The feasibility of using resistivity measurements to determine the progress of grouting has been tested in small-scale laboratory tests which show a marked change in soil resistivity upon grouting. After grouting the seismic methods potentially should perform well since they are more responsive to changes in soil strength and elasticity. The seismic refraction methods do not penetrate below a hard layer, so they might be useful in determining the depth to the top of a grouted formation. More work is needed in testing these geophysical methods in full-scale field operations (Ref. 68).

So far no tests seem to have been developed to provide information on the completeness of the solidifying reaction or what materials were migrating from the treated area. If these two things could be determined the safeness of chemical grouting could be established and its use become more widespread.

Grouting of soils can be an effective method of keeping groundwater out of tunnels during construction and keeping the permanent structure dry on a long-term basis. Due to the high costs involved ($100 to $500/cy) it is not economical to substitute this method for the other waterproofing methods described in this report, unless the grout is used for groundwater control and ground reinforcing during the construction period as well as for preventing the infiltration of groundwater into the permanent structure.

5.20 CONSOLIDATION GROUTING OF ROCK TUNNELS

Cement grout has been used for filling voids behind tunnel linings for more than a hundred years. It is still the most inexpensive and most commonly used material for grout sealing of leaks through rock fissures (consolidation grouting) and filling voids between a cast-in-place lining and the excavated rock (contact grouting). Contact grouting will be discussed in sub-section 5.30.

In general, relatively large volumes of water leakage can be tolerated during rock tunnel construction, particularly if the tunnel is being driven "uphill", with the grade rising from portal or shaft to the heading. In this situation the water is usually channeled in the invert to a sump near the portal or shaft and pumped to a settling basin and subsequent disposal. If the tunnel is being driven downgrade, the water must be pumped out of the tunnel from the heading, often in stages. It is not uncommon to handle hundreds of gallons per minute (100 gpm = 6.3 litres/sec) in aggregate leakage from heading and tunnel walls. When leakage at the heading becomes unmanagable, interferes with excavation and support operations, or has a deteriorating effect on the rock, consolidation grouting is employed to seal the leakage. In this case it is a water control measure to aid construction. However, whether the grout is placed during excavation, prior to concreting the lining, or after the lining is placed, an important objective is to keep groundwater from the permanent structure. In most rock tunnels the concrete lining and grout are the only measures taken to prevent water inflow. In unlined railroad tunnels, powerhouses or underground storage caverns, consolidation grouting may be the sole deterrent to water inflow.

5.21 Grouting During Excavation

Most rock is relatively impervious and water leakage takes place through joints, cracks, or shear zones of fractured rock. If a regular grid pattern of shallow holes is drilled and injected with grout to reduce rock permeability it is referred to as blanket or area grouting. This may serve a dual purpose of consolidating fractured rock around a tunnel as well as sealing off water. Very often individual seams are grouted after being exposed by excavation, either near the heading or prior to concreting. One or more grout nipples are placed into the seam and cemented in place. A grout nipple is a short length of pipe with a simple valve and coupling. The rest of the seam is sealed off, if possible, with burlap and wooden wedges. Grouting begins at the lowest level, continues till grout appears at the next nipple which is left open to permit water and air to escape. The first nipple is closed to prevent the grout from reentering the tunnel and the grout hose connected in turn to each of the remaining nipples. If the water finds another flow path into the tunnel the process must be repeated. It may be necessary to drill holes into the rock to try to intercept the flow path away from the rock surface. While urban transportation tunnels are not usually deep, some rock tunnels may be far below the surface, requiring high grout pressures to displace the groundwater present. Pressures in excess of 1000 psi (6,895 kPa) have been required for grouting some deep tunnels.

A rule of thumb used in some specifications for grouting pressures in rock calls for 1 psi per foot of overburden (22.6 kPa/m). This is conservative as it does not consider tensile and shear strength of the rock mass. The actual pressure that the rock is capable of withstanding is dependent on several variables and may change in a particular rock mass due to its non-homogeneity. Changes in tensile strength, jointing, stratification, weathering or altering of joints all affect this maximum pressure. Figure 30 shows approximate rock grouting pressures. This can be used as a rough guide, but water pressure tests should be made during the grouting program to determine actual field values to be followed.

5.22 Pregrouting Rock Before Excavation

If large water inflows requiring immediate grouting are anticipated in a particular area, the contractor may, either by specification or through prudence, drill exploratory holes out from the face in the direction of the drive to intercept the flow before excavation. When these exploratory drill holes indicate a large water flow is present, the contractor can delay excavation to consolidate the ground ahead of, and above, the excavation with a pattern of long drill grout holes. Although this is time consuming, excavating into a high flow zone of 10000 gpm (63 litres/sec), or more, could have disastrous consequences.

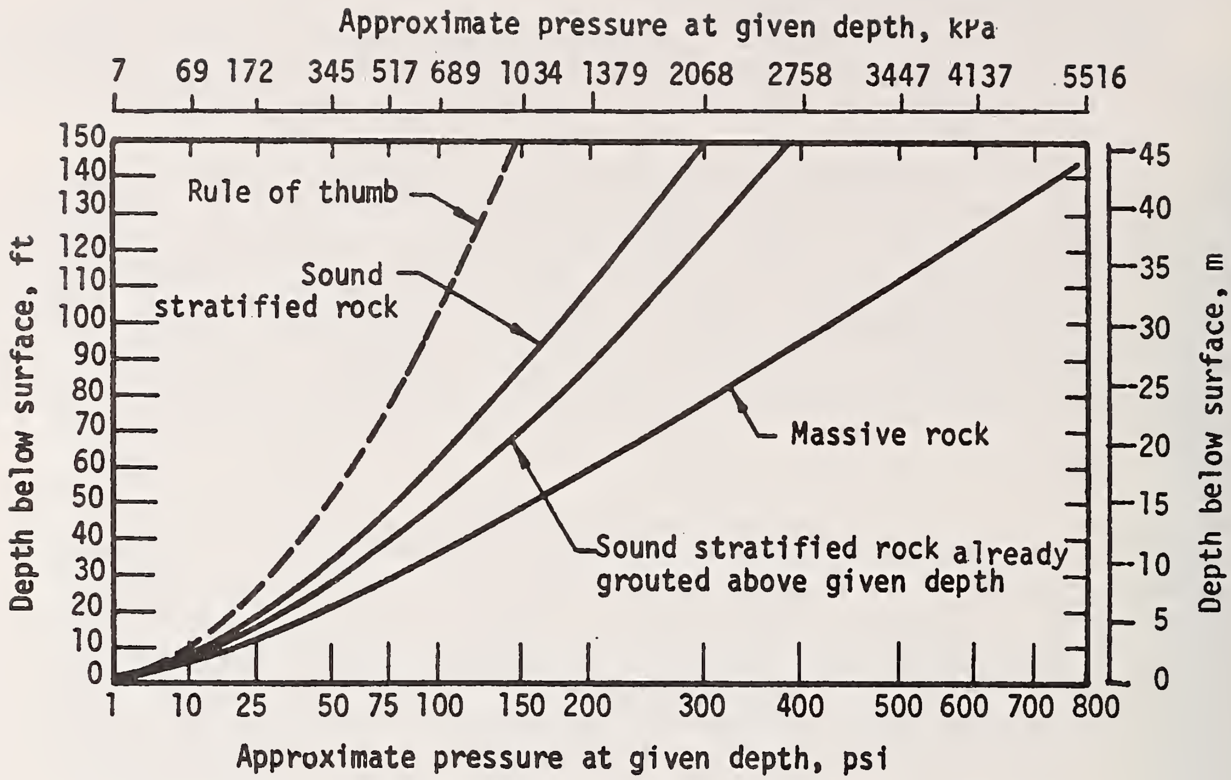


Figure 30 - Rough Guide for Rock Grouting Pressures
(After Creager, Justin and Hinds, 1945)

The following methods of pregrouting rock have been developed largely in connection with dam construction but can be modified for pregrouting of rock tunnels. While this type of complete coverage cannot be justified for most tunnels, the type of procedures used can be adapted to pregrouting difficult water areas of a tunnel either from the face or surface. The usual procedure in tunneling is to grout from a face or occasionally a small pilot tunnel when the need arises. One of several types of grouting described below may be used.

Stage Grouting. Holes are drilled and grouted in preset depth increments or stages from the rock surface, starting with the shallowest depth, relatively low pressure and the most viscous grout mix. After each stage of grouting, the drill hole is jetted and cleaned before the grout hardens to avoid the need of redrilling. The next stage is then drilled by extending the hole. Successive stages have higher pressures and thinner grout mixes to reach all cracks and fissures. At various stages, water pressure tests are made to aid in selecting the proper combination of grout mix and pressure. The first set of holes grouted are relatively far apart. A second set of holes are then drilled between the holes of the first set and the grout take compared to that in the original set. If deemed necessary a third set are drilled splitting the first and second set of holes.

Series Grouting. This is similar to stage grouting with shallow holes drilled and grouted first, except that all sets of holes are drilled and grouted at each stage. Holes are not washed out but deeper staged holes are drilled through the grouted zone. This results in more drilling than other methods but assures that each successive zone is grouted tight before advancing to the next zone depth.

Packer Grouting. With this method grout is placed at a particular depth by providing a packing in the grout hole above and below the area to be grouted. The packing isolates a section of perforated grout pipe limiting the grouting zone and providing a seal to prevent grout from coming back into the tunnel. The holes are drilled their full length and grouting is begun at the bottom of the hole progressing toward the rock surface in stages, with lower pressures in the later stages.

Circuit Grouting. Using this method, grout is pumped to the bottom of the grout hole through a grout pipe filling the grout hole. All zones are exposed to the grout under pressure at the same time. A return line empties unused grout into a holding tank.

5.23 Grout Mixes and Pumping Equipment

As mentioned previously cement grouts are most commonly used for rock tunnels although other grouting materials such as silicate grouts and resin grouts have also been used. The advantages of cement are its strength when set and its low cost. The biggest drawbacks to cement grouts are the difficulty in controlling set time and its limitation of flowing in narrow cracks. While Type I portland cement is most often used for grout, any of the other types may be used where conditions warrant. Type II or Type V may be used where the groundwater contains soluble sulphates. Type III, scalped, has particles approximately one-third to one-quarter of the size of the Type I particles, enabling flow in less permeable ground, and has high early strength. Air-entrained cements are not recommended by the Corps of Engineers who have had extensive grouting experience. They have reported difficulty in pumping such grouts. Resin gypsum cements are more expensive but have very fast set times.

Sand, clay, fly ash and even sawdust have been used as fillers in cement grout mixes to extend the cement. Cement-sand mixes are used for filling large voids but are not recommended for grouting in rock with small cracks. Cement-clay grouts using local native clays are likewise not recommended for filling small cracks, as they often contain fine sands. Bentonite, while more expensive, does not have this drawback. The larger the voids to be filled the thicker the mix, requiring a higher cement content, and possibly mixed with sand. Thinner seams require much thinner neat cement mixes and sometimes require a chemical grout not restricted by cement particle size. Chemical grouts may also be required in situations where faster or more controlled set time is needed than is possible with cement. There are situations where cement grouts can be used but groundwater flow conditions are such that the grout would flow long distances before setting. Under such conditions a smaller quantity of more expensive, but fast setting grouts can prove more economical. Occasionally it may be desirable to use both, a chemical grout with fast set time to seal off flow and a cement grout to fill the bulk of the voids.

As a general rule, to prevent bleeding of cement grout from a seam, the lowest possible water-cement ratio of grout that will flow, either neat or with a filler, should be used. Neat cement-grout mixes vary from about five gallons (19 liters) per sack of cement to twenty or thirty gallons (76 or 114 liters) per sack. For pumping cement grouts with sand fillers, cement-sand ratios vary from 1:0.5 to 1:2 by weight depending on the void to be filled and the type of equipment used. Cement-clay-water ratios may vary from 1:2:3 to 1:3:6 by weight. Additives which may be used with cement clays include set time accelerators or retarders. The most commonly used accelerator is calcium chloride. If grout must travel a long distance calcium lignosulfonate can be used as a set retarder, but in grouting most tunnels it is more usual to need to accelerate set time.

Among the chemical grouts that have been used in rock tunnels are silicate grouts (waterglass), polyester resins, and AM-9. The earliest chemical grouts used in tunnels were sodium silicate solutions. When used with proper reagents such as sodium bicarbonate or sodium chloride these solutions set up as a gel. Varying the type and quantity of reagent controls the grout set time. Silicate grouts are among the safest chemical grouts whether considering handling by the workmen or possible groundwater contamination. They were the only chemical grouts permitted in Japan at one time and served very well in driving the Seikan Tunnel (Ref. 74). This 32.3 mile (52 km) long,double track railroad tunnel beneath the Tsugaru Strait was one of the most challenging tunnel projects in the world. The silicate grout was particularly adaptable to the heavy salt water infiltration of this site.

Another difficult tunnel grouting job was the Susquehanna Water Tunnel in Baltimore (Ref. 134). Decomposed rock and mud seams were successfully pregrouted with AM-9, a polymer grout. The grout holes were drilled in the face to a depth of 20 to 30 feet (6 to 9 m). Excavation proceeded till a grout plug of about 7 feet (2 m) remained. Test holes were used to determine if another round of grout holes would be required and if so, the cycle began again.

The United States Bureau of Mines with the cooperation of several coal mine companies have been experimenting with the use of polyester resin impregnation of mine roof rock (Ref. 36, 94).

While the major consideration in these experiments is strengthening the rock through consolidation, the techniques could be used in tunnels in similar sedimentary rock for water control as well as rock reinforcement.

High pressure grouting, often needed to force water out of deep seams, is placed by a positive displacement pump. Grouting equipment includes a mixing chamber, either mounted in fixed position to discharge into the pump or a separate unit feeding the pump by pipe or hose. A return line recirculates grout back to the mixer when the voids are grouted to refusal. Where relatively low grout pressures, less than 150 psi (1030 kPa), will suffice, a progressive helical cavity pump (Moyno) is a compact efficient alternate to the larger piston pumps.

Sample rock tunnels constructed for various agencies in the last decade are listed in Table 3, showing the project specifications for grout mixes and grouting equipment. All of the sample tunnel specifications used call for cement grouts as is common in most rock tunnels.

5.30 ANNULAR SPACE GROUTING AND CONTACT GROUTING OF TUNNEL LINING

5.31 Annular Space Grouting

In tunnel excavation utilizing a shield and primary tunnel lining, the lining is erected in the protection of the tail of the shield. Some types of primary linings are expandable, that is, they can be expanded to the full excavated ground diameter after leaving the tail of the shield. Most bolted linings are not expandable, consequently they are 4 to 6 inches (100 to 150 mm) smaller in diameter that the outside of the shield. As the lining leaves the tail of the shield a void of at least 2 or 3 inches (50 to 75 mm) remains around its circumference. To prevent settlement of the lining and the ground above the tunnel it is imperative that this void be filled.

This situation more commonly exists in shield-driven, soft ground tunnels with soil of relatively short stand-up time, but there are times when a shield encased TBM is used for rock tunneling with a primary tunnel lining (Ref. 105, 136). The primary lining can consist of ribs and lagging, liner plates (with or without ribs), or a segmented lining. Traditionally the fill ing of voids in earth tunnels consists in placing pea gravel (3/8 inch or 10 mm maximum) under the invert to keep the lining from settling, then up the sides and over the arch of the lining. This is done by blowing the gravel through the lining with a gravel pan and air hose attached to threaded holes in the lining. Back from the heading 50 to 100 feet (15 to 30 m) cement grout is pumped out into the void through the same holes.

Table 3: SAMPLE TUNNEL CONSOLIDATION GROUTING SPECIFICATIONS

Tunnel	Owner	Date	Pipe Size	Equipment Cap.	Pumping Pressure	Mix Design, Etc.
Warm Springs Dam & Lake Sonoma Outlet Works Sonoma County California, USA	U.S. Corps of Engineers	1973	1-1/2" Dia.	Pump: Positive Disp. Similar to 'Moyno' Mixer, Sump, Hoses, Valves, Gages, etc.	Max of 30 psi at collar of hole	Cement and water with proportions to be approved. Sand in some cases only.
Thompson Yarra Tunnel Thompson River Development Melbourne, Australia	Melbourne & Metropolitan Board of Works	1968	As directed	Max. 400 psi Pump: Duplex piston type	As directed	Cement and water as directed. Sand if necessary.
Angeles Tunnel California Aqueduct Los Angeles County California, USA	Department of Water Resources, State of California	1966	By Engineer	Pump: Moyno Mixer: Harris Krough 50 GPM at 250 psi to 300 psi	Up to 100 psi	Portland cement and water.
Glendora Tunnel Foothill Feeder Los Angeles, California, USA	The Metropolitan Water District of Southern California	1965	1-1/2" Dia. Holes	As required to maintain service pressure capability not less than 1500 psi	Service pressures up to 1500 psi	As approved by Engineer. Mixture of Portland cement, calcium chloride, sawdust, or other approved material and water.
White Rock Tunnel Upper American River Project California, USA	Sacramento Municipal Utility District, Sacramento, California	1964	Min. Dia. 1-1/2"	Max of 200 psi Pump, Mixer, Hoses, Gages, valves, etc.	As directed	Cement and water.
Homestake Tunnel Lake and Pitkin Counties Colorado, USA	City of Aurora and City of Colorado Springs, Colorado	1963	1-1/2" Dia. Black Steel Pipe Std. Wt. (Sch. 40) ASTM A-120	Pump: Moyno Mixer, Agitator Tanks, Hoses: 1-1/2" Dia.	As directed	Cement and water as directed.
Upper Rubicon Dams, Tunnels, Access Roads, Etc. Upper American River Project California, USA	Sacramento Municipal Utility District, Sacramento, California	1962	Min. Dia. of 1-1/2"	Max of 200 psi Pump: Duplex piston type Mixer: Etc.	As directed	Cement and water.
Wachusett-Marlborough Tunnel Marlborough, Massachusetts USA of Massachusetts	Metropolitan District Commission, Commonwealth	1958	Up to 2-1/2"	As approved by Engineer.	Up to a max. of 100 psi to 300 psi as required.	Cement and water as directed.
West Delaware Tunnel Delaware System Delaware County New York, USA	Board of Water Supply, City of New York	1955	1-1/2" to 2-1/2" Dia. Steel	As approved by Engineer.	Up to 100 psi for large spaces. Up to 500 psi for small spaces.	Cement and sand as directed.

1" = 25.4 mm; 1 psi = 6.9 kPa

It would be naive to believe that the grout disperses itself evenly through the gravel to form a water-resistant, uniform, concrete lining around the tunnel. Grout follows the path of least resistance and will disperse through any voids around the tunnel rather than force its way through the small interstices of the gravel. These paths may include the space between the lining and the shield tail unless a grout dam or seal is provided. The dispersal through the gravel is further discouraged by the presence of fines from the soil mixed with the gravel by settlement or by forceful shooting of the gravel into the void. The method of gravel placing does not promote uniform distribution, but leaves pockets of gravel with voids or settled soil between. This nonuniform distribution has been observed outside tunnels originally backpacked in this manner and later uncovered by subsequent construction in the area (Ref. 128).

The primary purpose of providing gravel packing and grout in the annular space is to prevent settlement. If the space is filled in a timely and efficient manner it has served its purpose. Any sealing off of water access is a secondary consideration and cannot be reliably controlled. At the same time, any grout that is taken by voids around the tunnel helps to reduce the ground permeability caused, at least in part, by the tunnel excavation operations. The filling of the annular space around the tunnel therefore cannot be considered a reliable waterproofing method, but by filling voids, this grouting reduces to some extent the permeabilty of the surrounding soil.

5.32 Contact Grouting of Concrete Linings in Soft Ground Tunnels

Depending on the strength of the primary lining in a soft ground tunnel and the construction methods employed, placing of the permanent concrete lining may take place a short distance back from the heading or may be delayed till completion of excavation. Generally it is advantageous to use a stronger and more costly primary lining to avoid the periodic disruption and mutual interference of excavation, cleanup, and concrete operations. When the loads on the tunnel are great, the added cost of the stronger lining may be considered too high. Another situation where a lining may be concreted during excavation is in a tunnel being driven with compressed air to prevent water inflow. The cost of sealing the primary lining can be saved by placing concrete while the tunnel is still under air pressure. To be economical however, this saving must overcome the high additional cost of concrete labor working in compressed air.

In either case, after the tunnel concrete lining is in place it is necessary to grout any remaining voids between the primary and secondary linings. The primary lining, whether it be ribs and lagging, liner plates, or segments, does not have a smooth inside surface against which to place concrete. No matter how conscientious the contractor may be, it is difficult to fill all voids when pumping concrete into a long steel form, even with form doors and vibrators. The most difficult area is the arch where air can become trapped above the concrete between ribs or segment flanges. These voids must later be filled with a sand-cement grout. Occassionally a small percentage of bentonite is added to the grout. Eliminating these voids by filling them with grout is a definite aid to promoting watertightness in the lining by plugging off potential water courses.

There are two methods of providing grout holes through the lining depending on the contractor's preference. The contractor may place short sections of pipe behind the forms prior to concreting, or he may elect to drill grout holes through the concrete. Grouting procedures are similar in either case. Grout holes are staggered with one hole per 50 to 150 sq. ft. (5 to 15 m^2). Grout is pumped into a hole with just enough pressure to make it flow. When it reaches the next open grout pipe the first is closed and the grout hose moved to the second. If a hole does not take any grout at the maximum pressure, usually under 30 psi (207 kPa), it is considered full.

Care must be taken not to use too high a pressure to avoid cracking the concrete lining. For this reason contact grouting should not begin till the concrete has reached its design strength. A low pressure grout pump, either a positive displacement piston pump or a progressive helical cavity (Moyno) pump, can be used to place the grout. The one and one-half-inch (38 mm) size of grout pipes and hose is the most common size used.

5.33 Contact Grouting of Concrete Linings in Rock Tunnels

Rock tunneling, where a cast-in-place concrete lining is to be used, is almost invariably a two-phase project, with the lining placed after the excavation is completed. If support or reinforcement of the rock is needed during excavation it is provided by temporary measures such as steel ribs, rock bolts, shotcrete or a combination of these measures. This support or rock reinforcement is varied as the need requires (Ref. 146). While a number of useful studies have been made to place a continuous cast-in-place concrete lining in the tail of a TBM, it has not yet become a practical reality (Ref. 46, 97). When this is accomplished it will eliminate the need for a separate temporary lining, and reduce the construction time and cost of rock tunneling for many projects where it can be appropriately used.

Contact (or backfill) grouting of concrete linings in rock tunnels is similar to that described for soft ground tunnels. For tunnels excavated by drill and blast methods more grout will be required than for comparable TBM excavated tunnels in rock, or for soft ground tunnels. Blasting creates more uneveness of the rock surface increasing the chance of leaving voids. Where lagging, blocking or cribbing are required with steel ribs the probability for leaving voids increases. In such cases it is advisable to place a grout pipe from the area to the form to insure that the area is not missed during grouting.

In extremely bad ground where continuous lagging is required it is important to place grout pipes through the lagging to insure grouting any voids between lagging and rock as well as between concrete and lagging. Drilling through the concrete (and lagging if required) can be used as an alternate to setting grout pipes prior to concrete placing. The location of potential problem areas should be carefully recorded to aid in locating drill holes later through the lining. As in the case of soft ground tunnels it is important to fill all voids with grout to eliminate possible water courses behind the lining.

Equipment, piping, and grout mixes are similar to those described for soft ground tunnels. Table 4 shows sample rock tunnel contact grouting specifications for these items.

5.40 DIAPHRAGM CUTOFF WALLS

5.41 General Description

While diaphragm cutoff walls are often used for temporarily controlling groundwater during construction they remain in the ground for the life of the structure. It is appropriate therefore to review the effects of such walls on the permanent structure in terms of groundwater control. Diaphragm cutoff walls are used in cut-and-cover tunneling projects for a number of purposes and may serve several on a particular project. They may be used: 1) as an alternative to underpinning structures adjacent to the excavation, 2) to cut off or reduce groundwater flow into the excavation site, 3) as temporary ground support walls, and 4) as structural outer walls of a permanent structure. This type of construction is relatively expensive, but can be most cost effective when incorporating a combination of all uses above for a given structure (Ref. 144). A number of recent studies and reports describe in detail how such walls are constructed and utilized. It is recommended that the reader refer to these works for additional details. (See Ref. 18, 53, 54, 55, 86, 87, 125, 143, 144, 145, 148, 149).

Diaphragm cutoff walls are constructed by excavating a narrow deep slot in earth, which is filled with a bentonite slurry of proper consistency to prevent the sides of the slot from caving. Excavation in soft ground is usually performed by a specially constructed hydraulically operated grab bucket, mounted on the kelly bar of a crane. A drill or large chisel is used to break up boulders. A structural concrete wall is constructed in the slot. The wall can be cast-in-place in sections, or can be built of precast concrete members set into the slurry trench. These walls are then braced with wales and struts, or with wales and tiebacks as the excavation proceeds.

Such walls have been built in short sections for some rapid transit sections in Washington and Atlanta to support existing

Table 4: SAMPLE TUNNEL CONTACT GROUTING SPECIFICATIONS

Tunnel	Owner	Date	Pipe Size	Equipment Cap.	Pumping Pressure	Mix Design, Etc.
Pacheco Tunnel - Reach 2 Central Valley Project California, USA	U.S. Dept. of Interior Bureau of Reclamation	1976	By Contracting Officer	Pump: Duplex Piston Type, Helical Screw Rotor Type, with 150 psi press capacity	Not more than 30 psi	Either of: 1. Cement and water 2. Cement, sand and water. Add Bentonite 2% by wt. of cement
Buckskin Mountains Tunnel Central Arizona Project Arizona, USA	U.S. Dept. of Interior Bureau of Reclamation	1974	By Contracting Officer	Pump: Duplex Piston Type, Helical Screw Rotor Type etc., with 150 psi press capacity	Not more than 30 psi	Either of: 1. Cement and water 2. Cement, sand and water. Add Bentonite 2% by wt. of cement
Warm Springs Dam & Lake Sonoma Outlet Works Sonoma County California, USA	U.S. Corps of Engineers	1973	1-1/2" Dia.	Pump: Air driven 15 GPM or more slurry capacity. Mixer, Sump, Hoses, Valves, Gauges, etc.	Max. of 30 psi at collar of hole	Either of: 1. 1 part cement and 2 parts min. filler and sand, (proportions to be approved). And when sand not possible: 2. Cement, water and mineral filler (fly ash)
Tonner Tunnels No. 1 & No. 2 Yorba Linda Feeder Los Angeles, California, USA	The Metropolitan Water District of Southern California	1972	2" Dia. Black Steel Pipe Std. Wt. (Sch. 40) ASTM A-120-69	As approved by Engineer	As directed by Engineer but not more than 50 psi	Either of: 1. Portland cement and water. 2. Portland cement, sand and water. Mix proportion to be determined by Engineer. Either of above to contain 2% of Bentonite by wt. of cement.
San Fernando Tunnel Foothill Feeder Los Angeles, California, USA	The Metropolitan Water District of Southern California	1969	1-1/2" & 2" Dia. Black Steel Pipe, Std. Wt. (Sch. 40) ASTM A-120-65	As approved by the Engineer	As directed by Engineer but not more than 50 psi	Either of: 1. Portland cement and water 2. Portland cement, sand and water. Either of above to contain 2% of Bentonite by wt. cement. Mix of proportion to be determined by Engineer.
Thompson Yarra Tunnel Thompson River Development Melbourne, Australia	Melbourne & Metropolitan Board of Works	1968	As directed Black Steel Pipe	Pump: As approved Mixer: As approved	25 psi or lower as directed	Cement, sand and water as directed.
Castaic No. 1, Castaic No. 2 Saugus and Placerita Tunnels Foothill Feeder Los Angeles, California, USA	The Metropolitan Water District of Southern California	1966	1-1/2" & 2" Dia. Black Steel Pipe, Std. Wt. (Sch. 40) ASTM A-120	As approved by the Engineer	As directed by Engineer but not more than 50 psi	Either of: 1. Portland cement and water 2. Portland cement, sand and water. Either of above to contain 2% of Bentonite by wt. of cement. Mix proportion to be determined by Engineer.
Angeles Tunnel California Aqueduct Los Angeles County California, USA	Department of Water Resources, State of California	1966	By Engineer	Pump: Moyno Mixer: Harris Krough 50 GPM at 250 psi to 300 psi	Max. of 30 psi	Water, cement, Pozzolan and sand with 2% Bentonite by wt. of cement

1 psi = 6.9 kPa   1 gpm = 0.63 l./s.   1" = 25.4 mm.

Table 4: SAMPLE TUNNEL CONTACT GROUTING SPECIFICATIONS (cont'd)

Tunnel	Owner	Date	Pipe Size	Equipment Cap.	Pumping Pressure	Mix Design, Etc.
Glendora Tunnel Foothill Feeder Los Angeles, California, USA	The Metropolitan Water District of Southern California	1965	2" Dia. Black Steel Pipe, Std. Wt. (Sch. 40) ASTM A-120	As approved by the Engineer	As directed by Engineer but not more than 30 psi	Either of: 1. Portland cement and water 2. Portland cement, sand and water. Either of above to contain 2% of Bentonite by wt. of cement. Mix proportion to be determined by Engineer.
White Rock Tunnel Upper American River Project California, USA	Sacramento Municipal Utility District, Sacramento, California	1964	Min. Dia. of 1-1/2"	Max of 200 psi Pump, Mixer, Hoses, Valves, Gages, etc.	Max. of 30 psi	Cement, sand and water. Not more than 3 parts sand to 1 part cement.
Homestake Tunnel Lake and Pitkin Counties Colorado, USA	City of Aurora and City of Colorado Springs, Colorado	1963	1-1/2" Dia. Black Steel Pipe Std. wt. (Sch. 40) ASTM A-120	Pump: Moyno Mixer, Agitator Tanks, Hoses: 1-1/2" Dia.	As directed	Cement, sand and water as directed, not more than 3 parts sand to 1 part cement.
Upper Rubicon Dams, Tunnels, Access Roads, Etc. Upper American River Project California, USA	Sacramento Municipal Utility District, Sacramento, California	1962	Min. Dia. of 1-1/2"	Max of 200 psi Pump: Duplex Piston Type Mixer	As directed	Either of: 1. Cement and water 2. Cement, sand and water (As determined by Engineer)
Wachusett-Marlborough Tunnel Marlborough, Massachusetts USA	Metropolitan District Commission, Commonwealth of Massachusetts	1958	Up to 2-1/2"	As approved by Engineer	Up to a max. of 100 psi to 300 psi as required	Cement, sand and water as directed.
Harold D. Roberts Tunnel Summit and Park Counties Colorado, USA	Denver Municipal Water Works, Denver, Colorado	1956	2" Dia. Black Steel Pipe Std. wt. (Sch. 40) ASTM A-120-54	As approved by Engineer	As directed by Engineer but not more than 100 psi	Cement, sand and water.
West Delaware Tunnel Delaware System Delaware County New York, USA	Board of Water Supply, City of New York	1955	1-1/2" to 2-1/2" Dia. Steel	As approved by Engineer	Up to 100 psi for large spaces. Up to 500 psi for small spaces.	Cement, sand and water as directed.

1 psi = 6.9 kPa 1 gpm = 0.63 l./s. 1" = 2.54 mm.

structures. Station and line section portions of the Red Line Extention in Boston, presently under construction, include diaphragm walls. Acting as a semi-rigid ground support with appropriate bracing, this type of wall prevents outward movement of the soil beneath the existing structure. Movement can occur when a flexible support wall is used without underpinning the structure. While this use serves its purpose, if the remaining ground support walls permit drainage into the excavation these short sections of solid wall cannot be effective in aiding to control groundwater. In New York, a rapid transit line section was constructed using continuous diaphragm walls as temporary ground support and as an alternate to underpinning. The excavation invert was in rock, but as the walls were not sealed to the rock, they were ineffective in keeping water out of the excavation. In San Francisco several stations were constructed with deep cast-in-place diaphragm walls on all sides. They were designed to serve all four basic functions listed above. They proved competent for these purposes and were cost effective compared to other construction methods (Ref. 129).

5.42 Cast-in-Place Reinforced Concrete Diaphragm Walls

The earliest and most common type of cast-in-place diaphragm walls are cast in alternate panels and reinforced with cages of reinforcing steel. This was first used in Europe but has been slow in being accepted in the United States, partly becaused it involves different construction methods and practices from the more standard soldier pile and lagging method.

Alternate slots are excavated along the wall line, utilizing bentonite slurry, more commonly known as driller's mud. The slurry, which is a stable suspension of powdered bentonite in water, is used to keep the excavated slot stable and prevent sloughing. The slurry replaces the excavated material until the concrete is poured by tremie methods. The installation procedure requires equipment for slurry mixing, circulation and cleaning, together with special slot excavating equipment. This equipment includes hydraulic clamshells for earth excavation and chisels and drills for breaking up boulders. The excavated slot is narrow, 1' to 3' x about 20' long (0.3 m to 1.0 m x 6 m) and can be quite deep, 80' (24 m) or more.

Preassembled reinforcing steel cages are lowered into the slurry in the excavated slot, and concrete is tremied into the slot, displacing the bentonite slurry.

Once the initial alternating slots have been concreted, the remaining spaces between the completed sections are excavated, reinforcing steel installed, and tremie concrete placed in the same manner. Various key and waterstop configurations have been used to make the connection watertight. Embedded items such as bearing plates for wale installation or pipe segments for tieback installation can be attached to the reinforcing cages prior to concreting to facilitate later installation of the bracing system.

No forms are used in constructing a cast-in-place diaphragm wall. The tremie concrete fills the excavated slot, displacing the bentonite slurry which is pumped from the top, filtered and reused in another slot. The concrete wall faces take on the rough features of the excavated slot. A coating of slurry is left which is later removed from the inside face. That slurry remaining on the outside acts as a permanent seal similar to the use of bentonite panels described in Section 3. To supplement this seal it is sometimes necessary to use special joint treatment between panels.

Several imaginative joint details have been developed. Some details have included transmitting shear and splicing of rebars across joints, but we will limit our discussion to those details for waterproofing joints. The following sample joints are illustrated in Figure 31: A) The earliest joints were simple butt joints roughened by scraping tools or bucket teeth. B) Next came the half round or interlocking pipe joint, which is still widely used today. A steel pipe whose diameter is the same as the width of the slot is placed at each end of the slot. During the concrete pour the pipe is partially rotated to prevent bonding to the concrete. After the initial set, it is carefully raised, leaving a circular hole. When the adjacent panel is poured a half round joint is formed lengthening the path of possible water seapage. C) The next joint shown was developed in Japan as a variation of the interlocking pipe joint. A corrugated steel plate is attached to the pipe to create a wavy, half round joint. D) and E) These joint variations include one or two key tubes much smaller than the interlocking pipe. These can be grouted after the adjacent panel is placed to seal possible leaks. F) This joint shows a variation using a waterstop in the smaller tube which becomes half embedded in the first pour and is protected in the void of the tube while the adjacent slot is excavated. It is then half embedded in the second pour.

5.43 Cast-in-Place Soldier Pile and Tremie Concrete Walls

The soldier pile and tremie concrete system developed in the United States provides a continuous structural wall consisting of soldier piles spaced at predetermined intervals with casting-slurry concrete wall panels (usually unreinforced) between the piles. The wall thickness is equal to the depth of the soldier pile section, normally a rolled steel beam. The construction procedure starts with the placing of soldier piles in predrilled holes, kept open with bentonite slurry. Vertical alignment of the pile section is critical in this system to avoid too long a span for the unreinforced concrete. The hole is backfilled with lean concrete, which displaces the slurry. Slots are then dug between the soldier beams, again utilizing slurry. A special

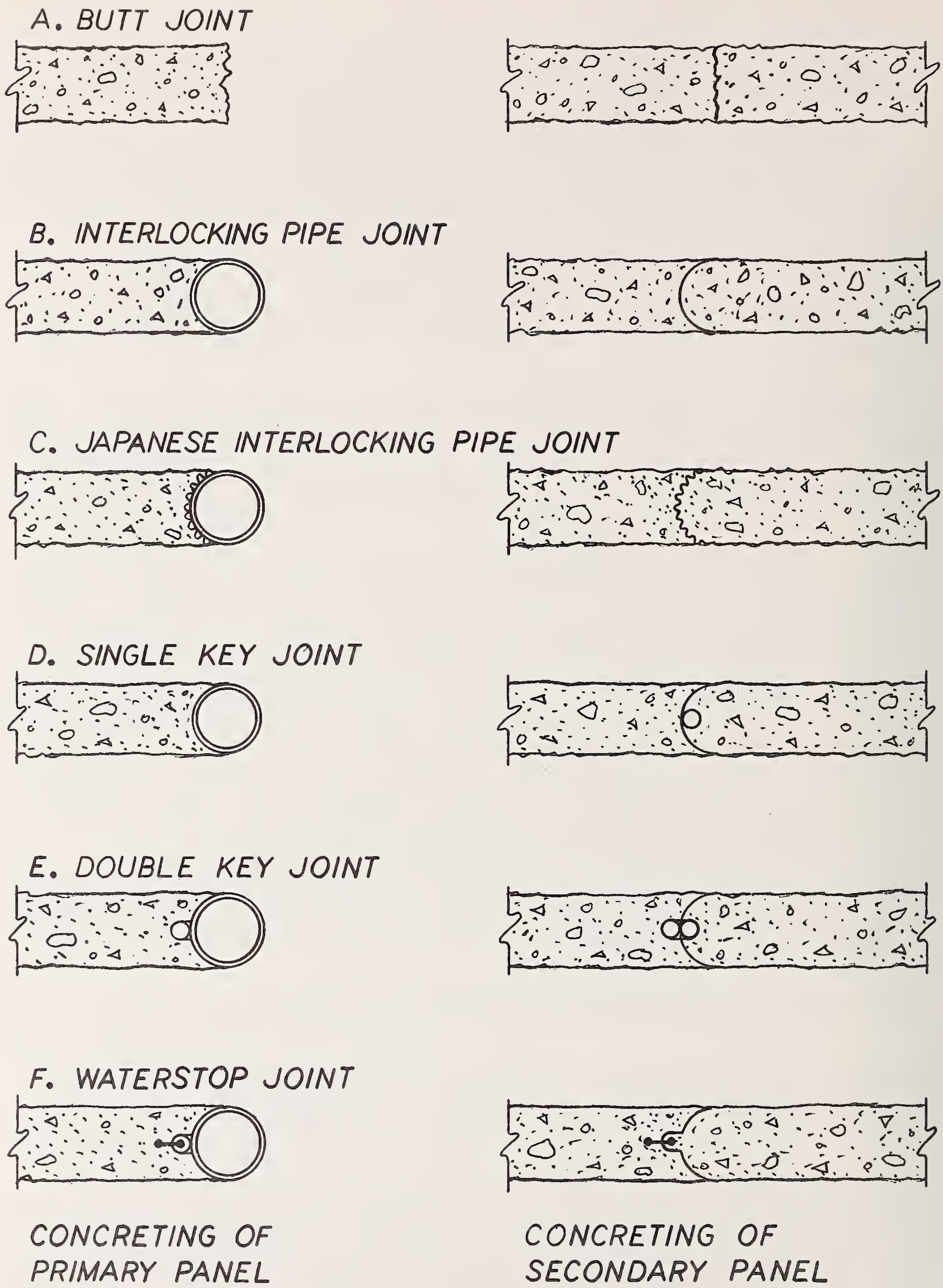


Figure 31 - CAST-IN-PLACE SLURRY WALLS - TYPICAL JOINT DETAILS

digging bucket is used and the sides of the soldier beams serve as guides. Tremie concrete is placed in the excavated slot. An alternate method is sometimes used where every other pile is placed in drilled holes and a double length slot is excavated. The center pile is placed in the slot and tremie concrete is used to fill the double slot using two tremie pipes. Care must be exercised to raise the levels of concrete simultanelously to avoid dislocating the center pile.

The walls are usually constructed from 18 to 36 inches (0.46 to 0.91 m) thick with soldier piles at 5 to 6 feet (1.5 to 1.8 m) on centers. An alternate arrangment is to place piles farther apart and place a layer of reinforcing near the inside face to span from pile to pile. For both arrangements, the soldier piles form the main vertical structural members with the concrete acting as impervious lagging to hold the soil between piles and transmit the soil load to the piles. The piles are blocked to the bracing wales as the ground is excavated between walls. This blocking of steel wales to steel piles is similar to the procedures used in bracing soldier pile and lagging supported excavations, and may explain its acceptance by United States contractors.

Care must be taken after the slot has been excavated in cleaning the soldier piles prior to placing concrete. As this is done below the surface of the slurry it is equally difficult to perform and to inspect. Any dirt remaining on the pile will contaminate the concrete and affect the bonding of the concrete to the steel. It is this bonding action which prevents water inflow at the steel-concrete joint of each panel. There are no concrete to concrete joints as in the case of the reinforced panel diaphragm walls where waterstops can be used. This is one of the drawbacks of the soldier pile wall, the difficulty of preventing leakage at the steel-concrete interfaces. Although the outer-face coating of bentonite slurry helps seal the wall, as in the reinforced concrete wall, the coating is thin and it is still necessary to have good bond at the piles.

5.44 Precast Concrete Segment Diaphragm Walls

Several variations of a basic precast wall system have been used successfully in Europe, since 1970 under the patented "Panasol" system of Soletanche and the "Prefasif" system of S.I.F. - Bachy (Ref. 18, 125). Precast walls can vary in configuration and size and include tongue and groove continuous paneling, and precast tee beam and panel combinations. The wall excavation procedure is similar to the cast-in-place slurry wall system. A trench is excavated using a special slow setting bentonite-cement grout slurry. Upon completion of the slot, the precast panels are lowered into the trench and aligned, and the grout allowed to set. The grout slurry is an important component of the system since the segments do not completely fill the trench as in the case of the cast-in-slurry walls. The setting of the grout which is at least equal in strength to the surrounding soil, assures elimination of voids and filling of all irregularities in the trench, thus minimizing potential settlement in the adjacent soil. The predominant use of this system to date has been in relatively shallow to medium depth excavations utilizing tieback bracing. The wall system appears to be readily adaptable to the use of wales and struts which may be required for deep cuts.

This system has many advantages for incorporation into the completed structure over the cast-in-place diaphragm wall system. Since the sections are cast in a yard, better control and uniformity of concrete can be achieved. There is no concern that reinforcing is coated with slurry. A good finish can be cast on the inside face eliminating the need, in many cases, for a finish wall, and waterproofing can be placed on the outer face. Dowels, keys, recesses and bearing plates can be incorporated into the casting as required.

This type of diaphragm wall has not been used yet in the United States There was a precast wall designed as an alternate ground support for a recent cut-and-cover sewer project in Baltimore, but the contractor elected to use a cast-in-place wall. An extensive study of the design and potential use of precast panel diaphragm walls for transportation tunnels in the United States was competed by Martin, et al in 1977 for the Department of Transportation (Ref. 86, 87).

The grout slurry used for setting the precast panels in the slot serves more than one purpose. Besides acting as a filler in the sides of the slot, the slurry is impervious and, leaving a wider gap on the outside face, provides a thicker water seal than the thin coating on the cast-in-place wall. In addition, if a deep slot is required for water cutoff, it is not necessary to extend the precast panels to the bottom of the slot. The panels are set sufficiently below the invert to meet structural requirements with the less expensive hardened grout slurry below to support the panels and fulfill the impervious cutoff requirements.

If additional treatment is deemed necessary at the joints, the ends of the panels can be cast with various shapes of keys and joint shapes similar to those shown for cast-in-place walls, and left slightly open to be filled with grout slurry or a hole for grouting.

5.45 Use of Diaphragm Walls as Permanent Groundwater Control

To be of maximum effectiveness in controlling groundwater for a completed cut-and-cover structure a diaphragm wall must: 1) be continuous on all sides of the structure, 2) not have leakage through the concrete or joints, and 3) be firmly bedded and sealed in a continuous impervious ground layer of clay, silt or rock. If all these are present, groundwater will not be able to reach the structure inside the walls. The practical problem involved is the difficulty of being certain that these conditions have been met. There is no problem checking for leakage in the walls above the invert during excavation and correcting defects by replacing concrete, or grouting through the walls. If there is leakage in the wall below the invert, or the wall bottom is not effectively sealed, it may not be detected unless the quantity of flow is high.

For a diaphragm wall that is deep, but does not extend into an impervious layer, or if there is leakage below the invert, the walls will still aid in reducing groundwater flow to some extent. A deep diaphragm lengthens the path of groundwater flow and subsequently increases the impedence to flow. Although water will continue to flow down under the wall and up to the invert, the quantity of flow will be greatly reduced. Volume 1 of this report, described how this factor aids in controlling groundwater during construction and greatly reduces the need for pumping water from the excavation. Eventually, after the structure is built, water pressure under the structure and that outside the walls will reach a state of equilibrium. If there is any leakage into the structure during its lifetime however, the potential maximum flow will be considerably less than it would be without the diaphragm walls.

As long as the diaphragm wall joints are sealed, there is no basic difference in performance between the three types of walls discussed. The precast panels, due to the need to transport and lift them into place, are made narrower than cast-in-place reinforced wall panels and therefore have more joints to seal. As mentioned previously the soldier pile and tremie concrete walls may be difficult to seal at the steel-concrete interfaces.

5.50 Permanent Lowering of Groundwater Table

There is no question that a rapid transit system constructed above the permanent groundwater table costs less to construct and less to maintain. The costs of the subway section of the Edmonton, Alberta rapid transit system bear this out. Most major cities in the United States are either on coastal plains or adjacent to lakes or large rivers. One possible means for controlling groundwater for a transportation tunnel is to lower the groundwater table permanently. Continuous pumping for a hundred years or more would hardly be practical but lowering by drainage may be possible in some areas. While this system has not been used for a transportation system, it has been used as a temporary construction aid for some difficult tunnel construction projects.

In 1976 an infiltration tunnel was excavated in a new suburb of Stockholm to control the groundwater level (Ref. 49). In this situation the tunnel was used to raise and then maintain water elevation at a fixed level, but the same principles could be used to lower the water table. The system consists of a tunnel through rock with a series of holes drilled radially upward from the tunnel to the rock-soil surface. Water level is maintained in a shaft used as a surge chamber. With the use of a supply system this can be used to add or remove water as required.

The difficulty with using such a system for most transportation tunnels would be the possible adverse effects on nearby structures. Most of these tunnels are constructed in built-up, urban areas, often with old buildings on spread footings. In some of these areas, even temporary lowering of the water table for construction can cause settlement if compressible soils are present.

This method of controlling groundwater could be very effective under the proper ground conditions. It should be considered in situations where lowering the groundwater will not have an unfavorable effect on other structures.

6.00 SUNKEN TUBE TUNNELS

6.10 INTRODUCTION

Preventing groundwater inflow into any tunnel is important. In a sunken tube tunnel 50 to 100 feet below the surface of a river or harbor it is absolutely critical.

Sunken tube tunnels are unique in design, construction, and sealing techniques. Among the few characteristics they share in common with tunnels constructed by more conventional cut-and-cover or tunneling methods are their purpose and interior appearance. While driving through a completed underwater highway tunnel it would not be apparent which construction method had been used, but construction methods are so vastly similar that the sunken tube tunnel would probably have been built by a company specializing in marine construction rather than by a tunneling contractor. Such tunnels have been constructed for highways, railroads, rapid transit systems, pedestrian travel, sewers, and various pipelines.

Sunken tubes are built in long units usually varying from 200 to 400 feet (60 to 120 m) in length, though there have been notable exceptions to this. They may be constructed of a circular steel shell with a concrete lining inside, or a precast reinforced box section which may, or may not, be prestressed. They may be constructed in a shipyard (if steel) or dry dock (if concrete), launched, and towed to the tunnel location one unit at a time. A dredged channel is previously prepared to receive the units on a graded gravel bed or concrete landing pads. The tube unit is sunk carefully into position by adding ballast, and secured and sealed to the previous unit. Details of construction and sealing methods will be discussed later.

In sunken tube tunnels there is no distinction between the methods employed to keep the tunnel dry during construction or after as they are the same. The development of this type of tunnel and the construction methods used are not as widely known as other types of tunneling, but are integrally tied to the unusual and stringent, groundwater control methods employed. A brief discussion of this development and the construction methods is therefore warranted.

In several of the noted references, the sunken tube tunnel is referred to as a "submerged tunnel", "immersed tube", or "immersed tunnel". These terms are all considered synonymous with "sunken tube tunnel".

6.11 Conditions Suitable for Constructing Sunken Tubes

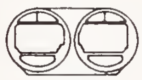
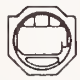
There are three basic methods that have been used for constructing underwater tunnels. In the past the most common method has been the shield driven tunnel, usually used with compressed air as described in Section 4.16 of Volume 1. Many tunnels in this country and abroad have been constructed this way. Another method is to place fill half way across the body of water (river), dig a trench and construct half the tunnel by cut-and-cover methods. The fill is removed, and the operations repeated for the other half. This method has not been widely used, but was employed on the first subaqueous vehicular tunnel in the United States. This was built in Chicago, in 1866 (Ref. 88). One obvious drawback to this type of tunnel is the adverse effect on shipping with more than half the river blocked for the entire construction period.

Sunken tubes, under proper conditions, are often an attractive alternative to a driven tunnel, particularly if the high cost of compressed air work is necessary for the driven tunnel. More than forty sunken tube transportation tunnels have been constructed since 1910, with over half of these completed in the last fifteen years. In addition, a number of sewer and pipeline tunnels have been constructed as sunken tubes. It is necessary that the river or harbor bottom be relatively stable. Strong currents can cause heavy deposits of silt over the tunnel, while turbulent flow could remove the tunnel blanket and even undermine the tunnel. The depth of water must be sufficient to float the tube units. The units are large, cumbersome and can present considerable resistance to flow. While it is desirable to tow and lower the units in still water these operations have been performed with currents of 2 to 4 knots (Ref. 37).

It is not necessary that units be constructed at the site as long as there is a suitable waterway by which they can be transported. Very often the area near the tunnel site is congested and it is desirable to locate the fabrication plant elsewhere. If a graving dock is to be constructed by dewatering and excavating, it is usually advantageous to locate in a less built-up area where the dewatering required for the deep basin cannot affect existing structures. Probably the most extreme example of fabricating off-site was the Chesapeake Bay tunnels whose 37 tubes, each 300 feet (91 m) long, were fabricated in Orange, Texas, and towed to Chesapeake Bay through the Inland Waterway (Ref. 88). In an area where several tunnels are constructed savings can be realized by reuse of fabrication facilities. Some dry docks in the Netherlands have been used to build units for several tunnels.

6.20 A BRIEF HISTORY OF SUNKEN TUBE TUNNELS

A number of reports and papers have been written on the history of sunken tube tunnels. (See Ref. 22, 37, 38, 88.) This history will be briefly summarized here. An excellent table of data on all transportation tunnels built by this method from 1910 to 1975 from Culverwell (Ref. 37), is reproduced here as Figure 32.

No	YEAR	NAME	FORM	LOCATION	TUBE LENGTH	CROSS-SECTION	No OF LANES	TYPE
1	1910	DETROIT RIVER	RAILWAY	MICHIGAN, U.S.A./ ONTARIO, CANADA	800 m		2x2 TRACKS	S
2	1914	HARLEM RIVER	RAILWAY	NEW YORK, U.S.A.	329 m		4x1 TRACKS	S
3	1927	FREIDRICHSHAFEN	PEDESTRIAN FOOTWAY	BERLIN, GERMANY	120 m		—	R
4	1928	POSEY	ROAD	CALIFORNIA, U.S.A.	742 m		2	R
5	1930	DETROIT-WINDSOR	ROAD	MICHIGAN, U.S.A./ ONTARIO, CANADA	670 m		2	S
6	1940	BANKHEAD	ROAD	ALABAMA, U.S.A.	610 m		2	S
7	1941	MAAS	ROAD	ROTTERDAM, NETHERLANDS	587 m		2x2	R
8	1942	STATE STREET	RAILWAY	ILLINOIS, U.S.A.	61 m		2x1 TRACKS	S
9	1950	WASHBURN	ROAD	TEXAS, U.S.A.	457 m		2	S
10	1952	ELIZABETH RIVER (1)	ROAD	VIRGINIA, U.S.A.	638 m		2	S
11	1953	BAYTOWN	ROAD	TEXAS, U.S.A.	780 m		2	S
12	1957	BALTIMORE	ROAD	MARYLAND, U.S.A.	1920 m		2x2	S
13	1957	HAMPTON ROADS	ROAD	VIRGINIA, U.S.A.	2091 m		2	S
14	1958	HAVANA	ROAD	CUBA	520 m		2x2	P
15	1959	DEAS ISLAND	ROAD	BRITISH COLUMBIA, CANADA	629 m		2x2	R
16	1961	RENDSBURG	ROAD	WEST GERMANY	140 m		2x2	R
17	1962	WEBSTER STREET	ROAD	CALIFORNIA, U.S.A.	732 m		2	R
18	1962	ELIZABETH RIVER (2)	ROAD	VIRGINIA, U.S.A.	1056 m		2	S

NOTES:
1. Date given is the date of completion.
2. *Denotes part of an underground railway system.
3. Form of tunnel is denoted as follows:
   S - Steel shell
   R - Reinforced concrete box
   P - Wholly or partly prestressed concrete box

Figure 32 - SUNKEN TUBE TRANSPORTATION TUNNELS (1910-1975) (From Culverwell, Ref. 37)

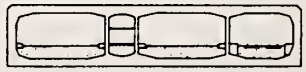
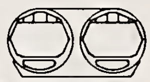
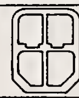
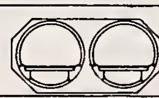
No	YEAR	NAME	FORM	LOCATION	TUBE LENGTH	CROSS-SECTION	No OF LANES	TYPE
19	1963	CHESAPEAKE BAY (a) THIMBLE SHOAL TUNNEL (b) BALTIMORE CHANNEL TUNNEL	ROAD	VIRGINIA, U.S.A.	(a) 1750 m (b) 1661 m		2	S
20	1966	COEN	ROAD	AMSTERDAM, NETHERLANDS	540m		2 x 2	R
21	1967	BENELUX	ROAD	ROTTERDAM, NETHERLANDS	745 m		2 x 2	R
22	1967	LAFONTAINE	ROAD	QUEBEC, CANADA	768 m		2 x 3	P
23	1967	VIEUX-PORT	ROAD	MARSEILLES, FRANCE	273 m		2 x 2	R
24	1968	TINGSTAD	ROAD	GOTHENBURG, SWEDEN	452 m		2 x 3	R
25	1968	ROTTERDAM METRO	RAILWAY *	ROTTERDAM, NETHERLANDS	1040m		2x1 TRACKS	R
26	1969	IJ RIVER	ROAD	AMSTERDAM, NETHERLANDS	790m		2x2	R
27	1969	SCHELDT E 3 (J.F.K. TUNNEL)	ROAD/ RAILWAY	ANTWERP, BELGIUM	510 m		2x3 ROAD 2 TRACKS	P
28	1969	HEINENOORD	ROAD	BARENDRECHT, NETHERLANDS	614 m		2x3	R
29	1969	LIMFJORD	ROAD	ARLBORG/JUTLAND, DENMARK	510 m		2 x 3	R
30	1969	PARANA (HERNANDIAS)	ROAD	ARGENTINA	2356 m		2	R
31	1970	BAY AREA RAPID TRANSIT	RAILWAY *	CALIFORNIA, U.S.A.	5825 m		2 TRACK	S
32	1972	CROSS-HARBOUR TUNNEL	ROAD	HONG KONG	1602 m		2 x 2	S
33	1973	EAST 63rd ST. TUNNEL	RAILWAY *	NEW YORK, U.S.A.	2 x 229m		4 x 1 TRACKS	S
34	1973	WANGAN SEN	ROAD	TOKYO, JAPAN	1035 m		2x3	R
35	1973	I 10 MOBILE RIVER	ROAD	ALABAMA, U.S.A.	747m		2x2	S
36	1974	KEIHIN CANAL	ROAD	KAWASAKI, JAPAN	660 m		2x2	S
37	1975	ELBE	ROAD	HAMBURG, GERMANY	1057m		3x2	R

Figure 32 - SUNKEN TUBE TRANSPORTATION TUNNELS (1910-1975) (continued)
(From Culverwell, Ref. 37)

The first recorded attempt to place a sunken tube was in the form of an experiment by two British engineers Wyatt and Hawkins, early in the 19th century. They actually built and joined two sections made of brickwork at the start of a crossing, of the Thames River. The units were 9 feet (2.75 m) in diameter,and 25 feet (7.6 m) long. Being unable to obtain sufficient interest and backing, the scheme was abandoned. In 1894, a 260-foot (79 m) length of 6-foot (1.8 m) diameter sewer tunnel was set in Boston Harbor by the Boston Metropolitan Sewer District. Six units were used consisting of riveted steel shells with brick lining.

The generally acknowledged forerunner of current sunken tube practices in the United States is the Detroit River railway tunnel connecting Detroit, Michigan, with Windsor, Ontario. Ten steel, twin-tube sections, each side 18 feet (5.5 m) in diameter averaged 262 feet (80 m) in length. Built at a nearby shipyard, they were towed to location and sunk into place by controlled flooding. Divers were used to bolt the sections together. A grout seal was used outside a rubber gasket. Then tremie concrete was placed outside the steel lining and the trench backfilled. This tunnel was completed in 1910 and the era of sunken tube tunnels had begun. In 1914, a four-tube railway tunnel was constructed across the Harlem River in New York City by the sunken tube method. Figure 33 shows one of the sections being floated into position.

In 1928 the first sunken tube highway tunnel, the Posey Tunnel, was completed linking Oakland and Alameda under the Oakland Inner Harbor in California. While this tunnel also had a circular shape, it was constructed of reinforced concrete. Twelve, single tube, two lane units, each 203 feet (62 m) long were used. With the exception of this tunnel and the parallel Webster Street Tunnel completed in 1962 all sunken tube tunnels built in the United States to date have been of the steel shell type, most of them circular in shape. Much of the development of the steel shell type of sunken tube has been credited to the engineering firm of Parsons, Brinckerhoff, Quade & Douglas, and the contracting firm of Merritt-Chapman & Scott (Ref. 37).

European, Canadian and Japanese sunken tube tunnels have been developed separately and independently of the steel shell type used in the United States The tunnels in these countries are built in a rectangular shape of precast concrete with either reinforcing steel or prestressing strands, as dictated by design requirements. This type of structure can accommodate more lanes or tracks than a circular steel shell. The first tunnel so constructed was the Maas Tunnel in Rotterdam completed in 1941. The rectangular cross section, 81 feet (24.8 m) wide by 27 feet (8.3 m) high, contained two two-lane highway compartments and one compartment each for pedestrians and bicycles. The units were sunk to rest on concrete pads. Hydraulic jacks were used to set the unit to grade about 3 feet (1 m) off the bottom of

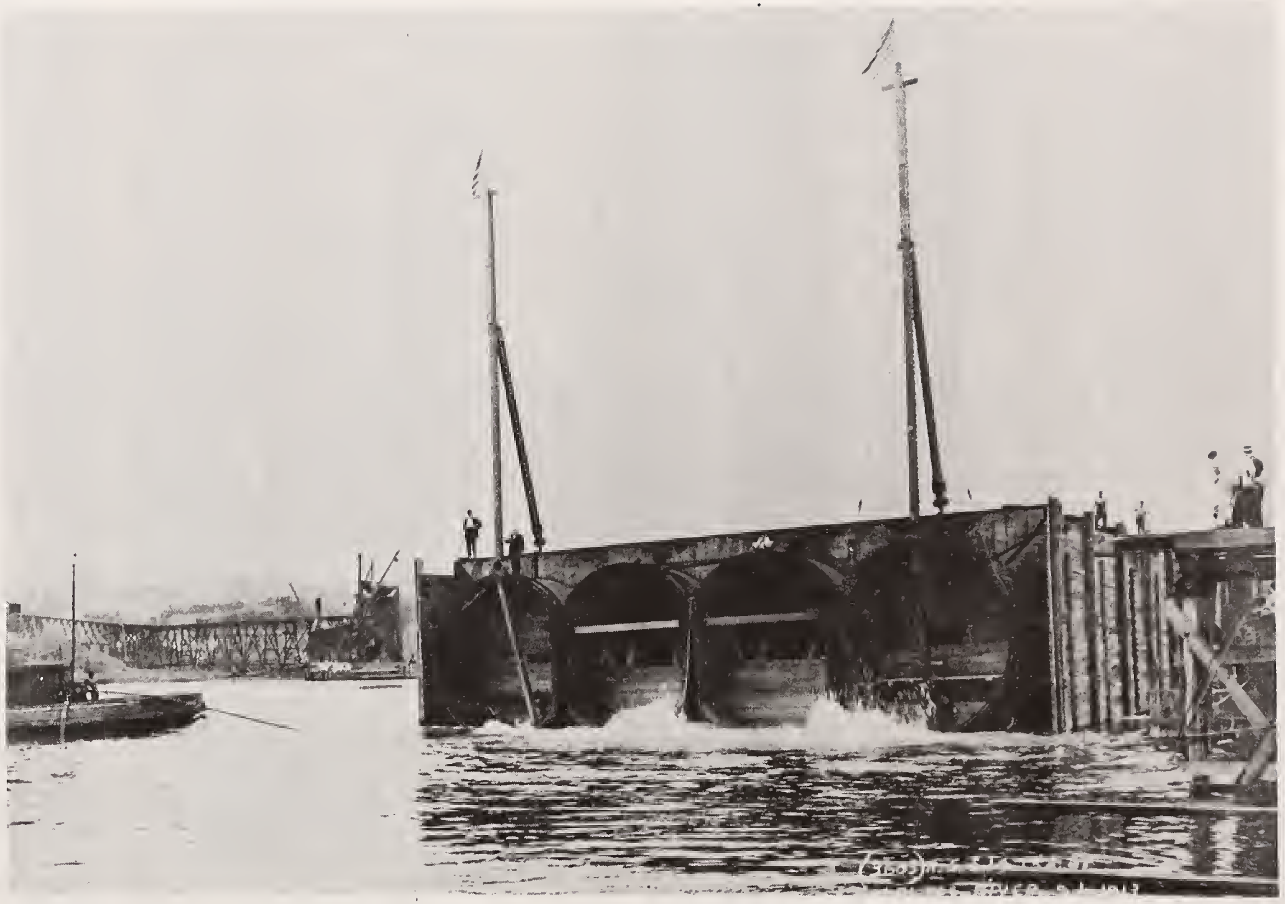


Figure 33 - SECTION OF THE HARLEM RIVER SUNKEN TUBE TUNNEL BEING TOWED TO THE TUNNEL SITE (1913)
(Courtesy of E. S. Plotkin)

the trench and a sand foundation was jetted into this space. Figure 34 shows the cross section of the Maas Tunnel. Figure 35 illustrates the method of sand jetting the foundation for a rectangular box section.

This set the pattern for future development of this type of sunken tube tunnel. Various improvements on the construction techniques used will be discussed in the next subsection. The largest individual units used to date of this type were those of the Scheldt Tunnel in Antwerp, Belgium, which contained two three-lane highway compartments and one two-track railroad compartment. Completed in 1969, the units were 27 feet (8 m) high, 157 feet (48 m) wide, 377 feet (115 m), long and weighed 55,000 tons each (Ref. 22). The longest individual tube sections used were the straight portions of the Hemspoor Railway Tunnel in Amsterdam, 879 feet (268 m) long for each of four units. The rectangular section 70 x 29 feet (21 x 9 m) contains three single track compartments.

The shortest and the longest sunken tube transportation tunnels built to date are both rapid transit tunnels in the United States. The shortest is the State Street Tunnel in Chicago built in 1942. Only 200 feet long (61 m), it consists of but one section under the Chicago River. The longest tunnel is the Bay Area Rapid Transit tunnel under San Francisco Bay. Completed in 1970, it consists of 57 sections varying from 273 to 366 feet (83 to 112 m) in length for a total of 19,110 feet (5,825 m).

An interesting application of the sunken tube method is in connection with a section of the Rotterdam Metro built in 1968. A section of tunnel 3,412 feet (1,040 m) in length was placed beneath the Maas River by the sunken tube method. The connecting land section, about 7900 feet (2,400 m) long, was on flat land consisting of silt, sand and gravel with a high water table. A canal was dug for the full length of the route including city streets. The ground was supported by sheet piling with one row of bracing at the ground surface. Precast units varying in length from 250 to 300 feet (75 to 90 m) were floated into position using guide rails set below the bracing level. Due to the poor ground conditions precast concrete piles with telescoping caps were driven to support the sections after they were lowered into position. See Figure 36. It should also be noted that this land section included several curves and four stations.

6.30 CONSTRUCTION METHODS

6.31 General

As mentioned previously, there has been parallel, and for the most part, separate development of two basic methods of constructing sunken tube units. The earlier steel shell method

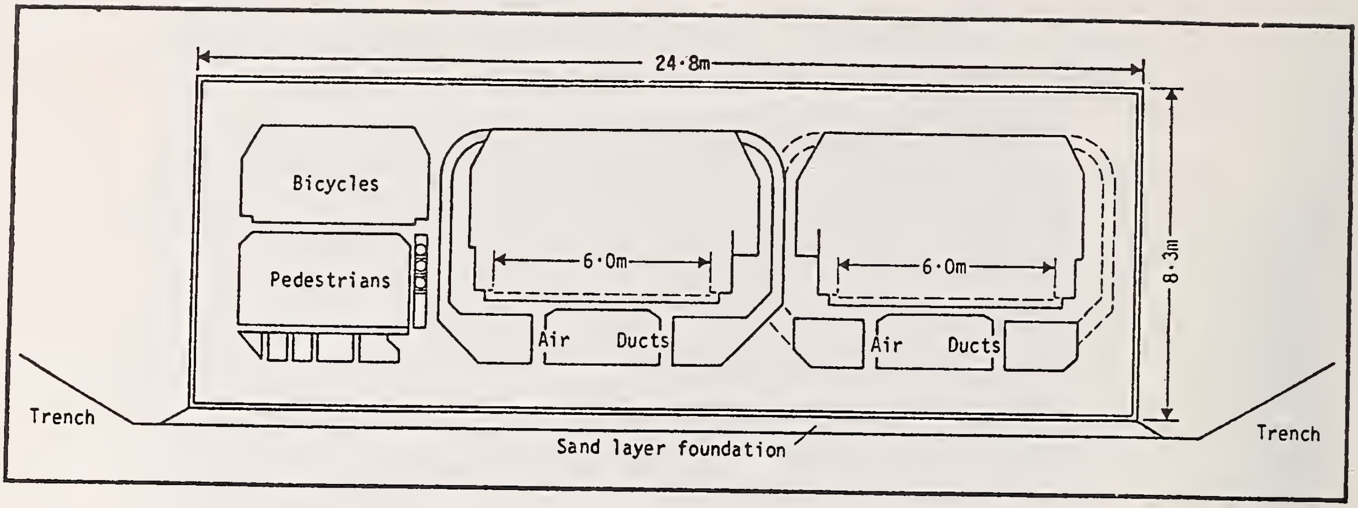


Figure 34 - CROSS SECTION OF THE MAAS TUNNEL, ROTTERDAM
(From Culverwell, Ref. 37)

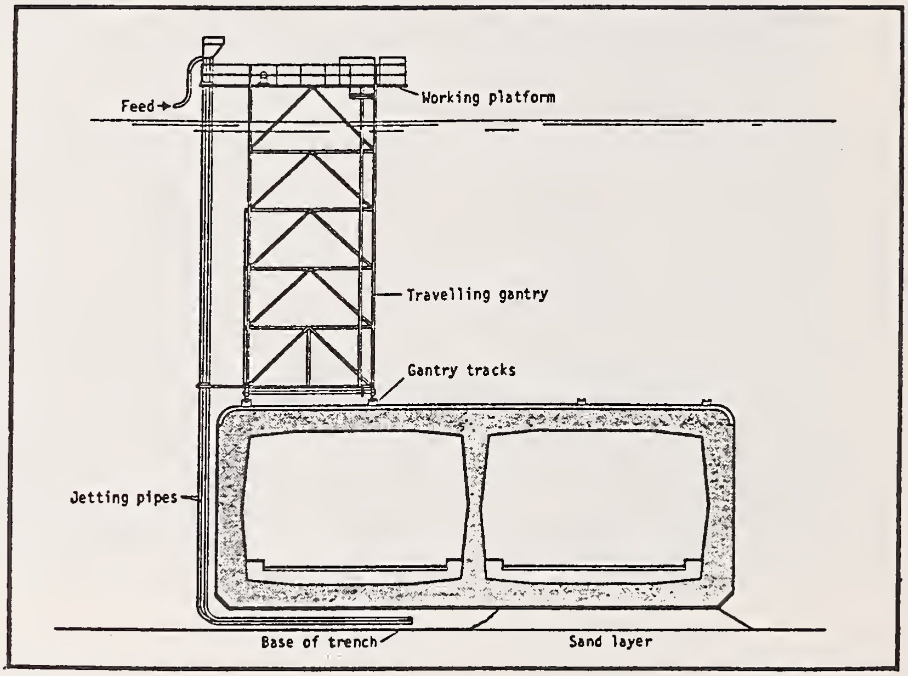
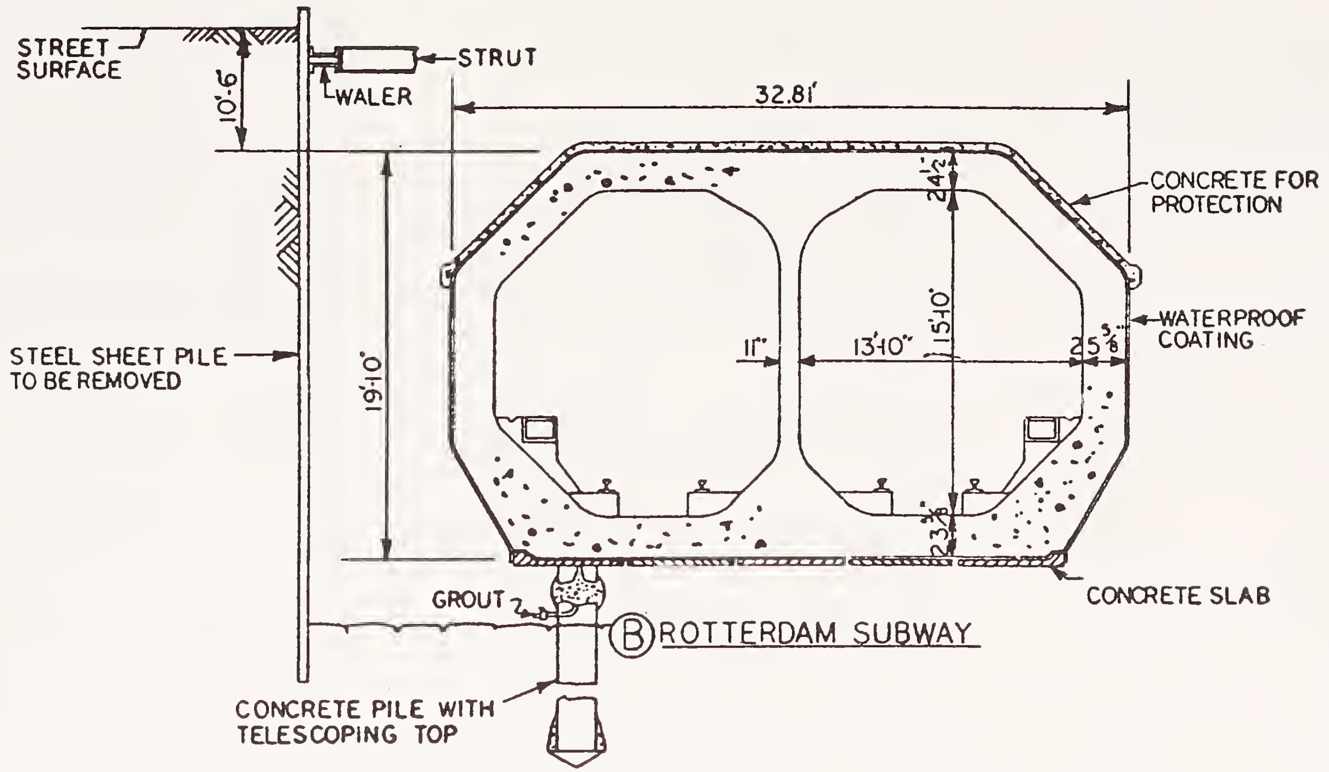


Figure 35 - SAND JETTING THE FOUNDATION FOR A RECTANGULAR BOX SECTION
(From Culverwell, Ref. 38)



(1 ft. = 0.3 m)

Figure 36 - CROSS SECTION OF ROTTERDAM METRO - PORTION BUILT IN CANAL (From Mayo, Ref. 88)

was begun and developed in the United States; the rectangular reinforced concrete method was first used in Netherland and is now used by most countries outside the United States Actually both methods use reinforced concrete as the major structural component of the completed tunnel, and occasionally a precast concrete box section will utilize a steel covering to act as both outside form surface and waterproofing membrane. The major difference between the two is in the fabrication sequence. The steel shell is constructed with sufficient strength to withstand launching and towing with a nominal amount of concrete in the invert to act as ballast. Additional concrete is added with the unit tied up to a dock, and the concreting is completed after the unit has been sunk in place. With a precast unit all structural concrete is placed while the unit is in a dry dock. Pontoons are then used to help float the unit to the tunnel site.

For either type of construction there are certain advantages in sunken tubes over conventional tunneling from a scheduling and construction sequence point of view. Two important factors in developing and maintaining an efficient construction schedule are the degree of repetition of operations and the sensitivity of critical path operations. Both forms of tunneling have a relatively high degree of recurring, cyclic operations, where workmen can develop skill through constant repetition of operations. Driven tunnels however are more sensitive to delay of critical path operations (i.e. at the tunnel face). When the tunnel face operation is delayed the whole project is delayed. Sunken tube tunnel construction has more parallel operations. A hold-up in one operation does not usually shut down all operations. Even sinking and joining units together usually contains slack time. With several units being constructed at any one time there is usually alternate work to do even if one operation is delayed.

6.32 Construction of Steel Shell Tube Units

Fabrication of the steel shell units is usually performed at a shipyard facility. An isometric view of a typical circular two-lane highway tunnel is shown in Figure 37. A unit such as this would have a steel shell 3/8 inch (10 mm) thick and about 30 feet (9 m) in diameter with a length of 250 to 350 feet (80 to 110 m). The steel shell would have tee stiffeners and other bracing to make it sufficiently rigid for launching and towing. Today all fabrication is by welding and preferably by machine welding. As the steel shell also serves as a waterproof membrane, watertight continuous welding is a more critical criterion than strength. Though the steel shell is a structural envelope during the construction stage, the reinforced concrete ring placed inside is the major structural component of the completed structure. To overcome buoyancy and to protect the steel shell from corrosion, additional unreinforced concrete is placed outside the structure. It is common practice to use thin steel plate about 1/4 inch (6 mm) thick as a back form braced to the

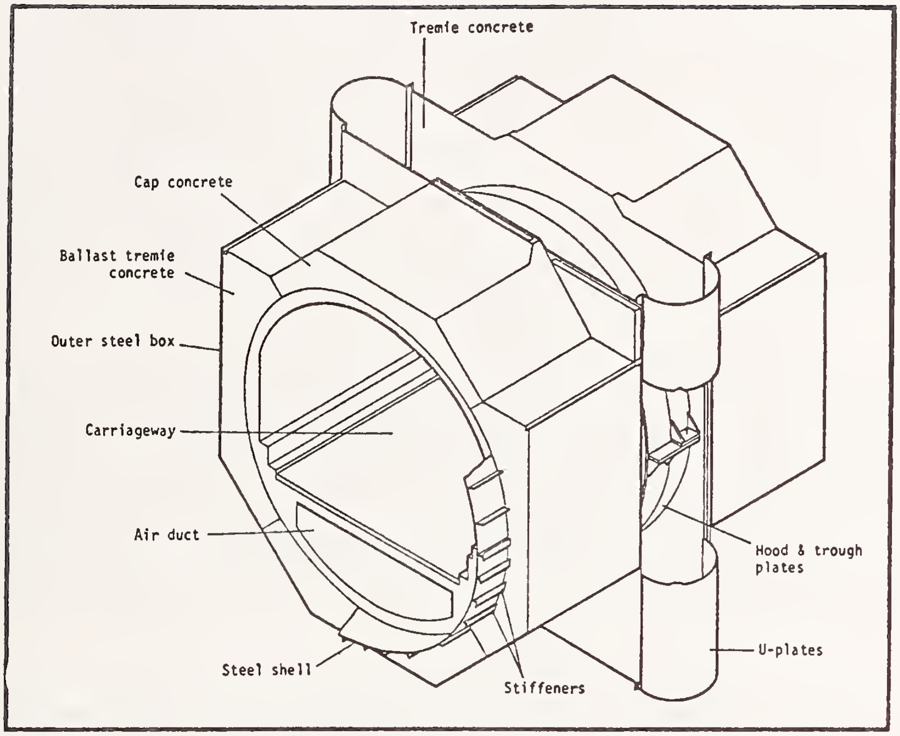


Figure 37 - ISOMETRIC VIEW OF A CIRCULAR STEEL-SHELL UNIT AND JOINT
(From Culverwell, Ref. 38)

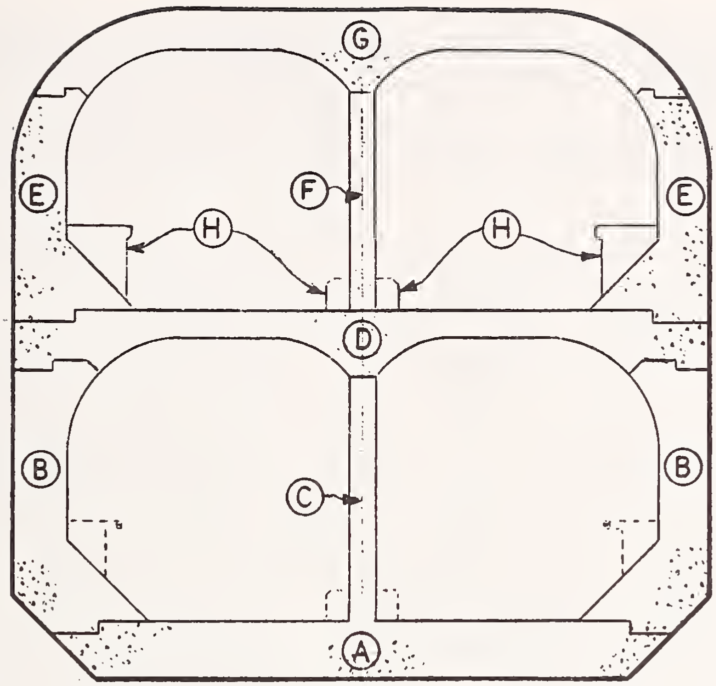
steel shell stiffeners, as shown in Figure 37. Protection ofthis steel is not critical as corrosion after the outer concrete is in place will not affect the integrity of the structure or the main shell.

After fabrication of the steel shell, a portion of the bottom protection concrete is placed between the inner and outer plates to act as a keel to keep the unit upright when afloat. Reinforcing steel for the structural lining is placed and temporary watertight bulkheads are attached to each end. The unit may be end or side-launched depending on the fabrication site. The unit is then towed to a dock or jetty near the tunnel site to place the remaining structural concrete and ballast concrete. When first launched the unit is extremely light and rides high on the water. The concrete and ballast added at the dock is designed to leave only 1 to 2 feet (0.3 to 0.6 m) of freeboard for towing to the tunnel site. The sequence of pours must be carefully scheduled and controlled to keep the unit afloat and prevent undue stress on the steel shell. See Figure 38 for typical pour sequences of steel shell tubes.

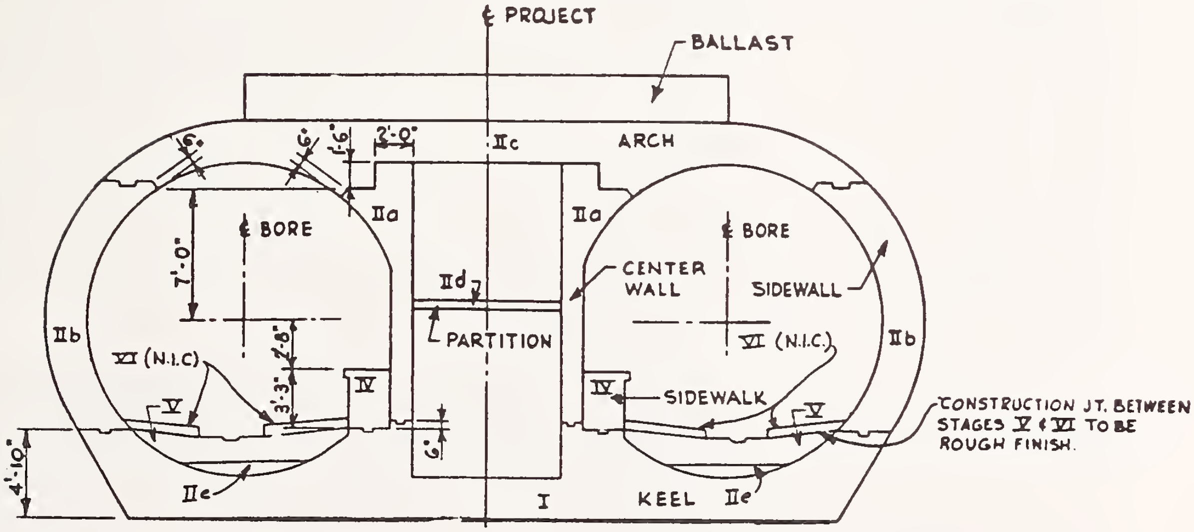
Dredging the tunnel trench is performed by lifts well ahead of the units being set in place. A sonar scanner is used to check the grade and slopes of the trench. After dredging, a foundation layer of gravel, 2 to 3 feet (0.6 to 0.9 m) thick, is spread with a hopper and drop pipe from a specially built barge which also screeds the gravel. The barge is anchored to the bottom to counteract tidal or flow changes. The screed is heavy, and an accuracy of 1 to 2 inches (25 to 50 mm) is possible with modern equipment.

The tube unit is outfitted with a survey target tower at the far end and a sonar scanner at the end to be connected to the existing tube. It is towed to the approximate sinking site. If necessary the gravel bed is cleaned of silt before setting the tube unit. A special lowering rig mounted on two barges straddles the tube unit and is anchored to the bottom. Additional ballast is added to the tube till it reaches a predetermined negative balance, which may vary from 150 to 650 tons. Heavy truss beams supported by the barges carry this additional weight and are adjustable for setting tube alignment. The tube is then lowered at least 10 feet (3 m) back from the last tube unit placed. When approximate grade is reached the sonar scanner is activated and the unit is slowly moved forward till it is about 2 feet (0.6 m) from the last tube. Divers then visually check location before engaging temporary coupling devices that will join the two units. The alignment of the rear end of the unit is controlled by sighting on the target tower projecting up out of the water. The coupling devices pull the units together.

If a gasket is used between units (see next section on sealing techniques) the space between the units can be drained to enable workmen to enter the space between to set the perma-



NYCTA EAST 63 STREET TUBE UNDER
EAST RIVER, NEW YORK CITY



BARTD RAPID TRANSIT TUBE UNDER SAN FRANCISCO BAY

Figure 38 - POUR SEQUENCES FOR TWO RECENT STEEL-SHELL
TUBE TUNNELS

nent seal which can be welded steel plates or a rubber Omega profile (to be discussed in next section). If a gasket is not used, divers place steel U-plate forms as shown in Figure 37 and tremie concrete is placed to encase the steel joint. Gravel and/or selected fill is placed around and above the unit, and the bulkhead of the new unit and adjoining bulk- head of the previously laid unit are removed. The tunnel is now ready to receive the next unit. If there is a possibility of scouring action above the tunnel a rock blanket or layer of rip-rap may be placed over the filled trench.

6.33 Construction of Rectangular Reinforced Concrete Tube Units

This type of sunken tube unit, developed in Europe, is constructed almost entirely in a basin or dry dock before being launched. For relatively short tunnels all units may be constructed together, for longer tunnels four to six units may be constructed at one time. To construct six 330-foot (100 m) long sections at one time requires a basin 1150 x 260 feet (350 x 80 m), 50 feet (15 m) below water level (Ref. 38). The construction of such a basin is an expensive and sometimes challenging project in itself. In the Netherlands several such basins have each been used for construction of a number of sunken tube tunnels. This helps considerably in reducing the overall project cost.

After construction the basin will be flooded to float the tube units. To avoid a vacuum effect between the flat bottomed tube and the floor of the basin a gravel base is laid prior to the construction. Concreting may be in three lifts of base slabs, walls, and roof or two lifts with the walls and roof poured together. Forms are usually stiff steel frames with wood surfacing. If a steel membrane is used it serves as the outer form, with temporary bracing. In earlier units, pours were continuous and monolithic for the length of the unit. More recently the unit is divided into 50- to 75-foot (15 to 23 m) sections with expansion joints between to reduce cracking and permit some flexibility of the unit. These units are held together with stressed cables until after the unit is in place and backfilled. If there is no outer steel membrane the joints between sections must be waterproofed.

After concreting and waterproofing, temporary bulkheads are added at the ends. Interior compartments are bulkheaded as ballast tanks which are filled to keep the unit in place until it is time to tow each individual unit to the tunnel site. Rubber seals, steel joint plates, couplers and hydraulic jacks are added and the basin is flooded.

Before each unit is to be sunk, it is outfitted with steel braced towers at each end to provide access to the unit and aid in alignment when the unit is lowered. Special pontoon rigs fitted with mooring anchors, lowering winches and ballast pumps are also attached to the tube unit. These will support the negative buoyancy weight of the unit during sinking. As in the case of the steel shell units this will be in the order of several hundred tons depending on the size of the unit, the water flow, and the water density. The sinking sequence is illustrated in Figure 39.

Because of the width of these flat-bottomed rectangular units they are not placed directly on a screeded gravel bottom as in the case of the circular steel shell. The method used for the foundation of the first rectangular tube, the Maas Tunnel, was developed by Christiani & Nielsen of Denmark and has been largely responsible for the success of this type of construction. The Maas Tunnel was lowered onto concrete pads at each end of the unit, with large jacks supporting the tube about 3 feet (1 m) off the bottom. From a barge above a pipe extended down under the tube and selected sand was tightly packed into the space by jetting. The jetting techniques have been improved over the years. On more recent tunnels the front end of the unit being sunk is supported on the end of the last one placed while the rear end sits on hydraulic jacks down to embedded concrete pads.

When the unit is in place, is on line vertically and about 8 inches (200 mm) from the last unit, divers attach coupling units. These hydraulic coupler jacks pull the unit forward squeezing the rubber Gina gasket of the new unit against the steel joint plate on the end of the last unit. Water is then removed from between the bulkheads creating a vacuum. Water pressure acting on the rear end of the new unit forces it ahead squeezing the gasket as far as it can, effectively sealing the units together till workmen can enter the space to complete the jointing and sealing.

6.34 Comparison of Steel Shell and Rectangular Reinforced Concrete Tubes

As described above, the major construction differences between steel shell and reinforced concrete tubes is in the sequence of operations, method of launching, and, to some extent, method of sinking. While each sunken tube is individual in design and construction, suiting the use and site conditions, the use of either of the two general types tend to be based mostly on the experience of the designers and contractors. Each of these two types of tubes has certain advantages and drawbacks.

Circular steel shells are used primarily for double lane highway and double track railroad tunnels. They are relatively light when launched and do not require a large dry dock or basin. Most of the concrete is placed while the unit is tied to an ordinary dock or jetty. The circular tunnel shape has an inherent strength not found in a rectangular tunnel whether it be a driven tunnel or sunken tube.

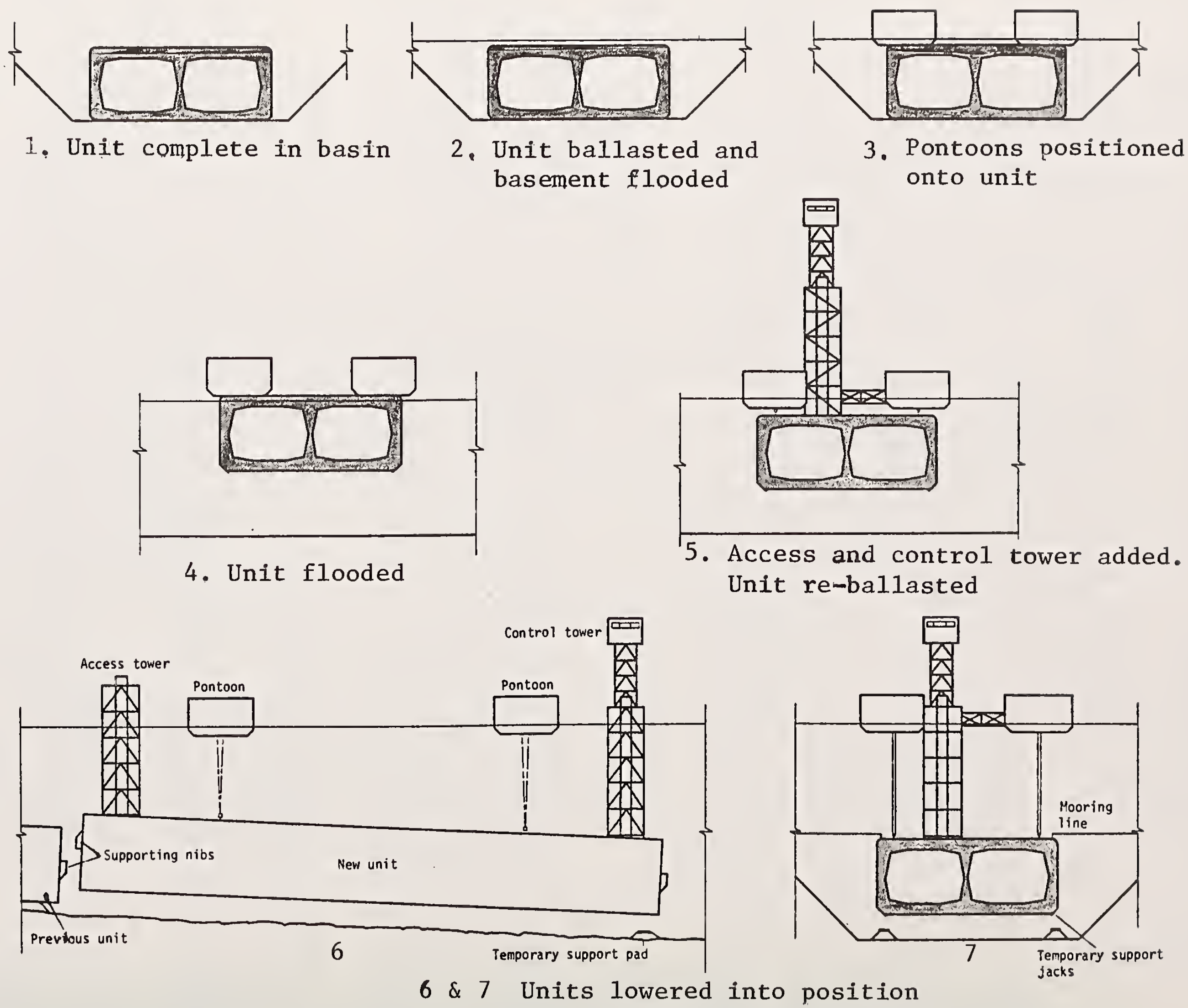


Figure 39 - TYPICAL SINKING SEQUENCE FOR A RECTANGULAR BOX TUNNEL (From Culverwell, Ref. 38)

On the other hand, the amount of concrete used in a sunken tube is usually based on buoyancy considerations rather than strength, so the strength of the steel shell is not necessarily an important factor. In long tunnels the space above and/or below the roadway compartment is used for ventilation. For relatively short tunnels, with all traffic in one direction, a piston effect is created by moving traffic eliminating the need for forced ventilation and separate vent compartments, although blowers may be required for slow or stopped traffic. In these situations not only is the additional space wasted and costly, but added concrete is needed to overcome the buoyancy of the extra unused space. In addition, with or without ventilation compartments, the circular shape is higher than an equivalent rectangular section with ventilation compartments on the side. Since the depth to the top of the tunnel is determined by shipping lane requirements this could result in added dredging and backfill.

The biggest advantage of rectangular shaped, reinforced concrete tubes is in constructing multi-lane tunnels. Several contain two-3 lane compartments; in addition to this, the Scheldt Tunnel also has a double track railroad compartment. Combining multiple lanes in one tunnel unit can result in considerable reduction in the cost per lane where such capacity is required. The actual size of ventilation compartment(s) can be determined and added to suit the length of tunnel. The major disadvantage of the reinforced concrete tubes is the need for a large and expensive casting basin. Although this need not be at the tunnel site, the small freeboard of these units at the time of launching makes it desirable to keep towing distance to a minimum and confined to calm inland waterways. If it is possible, as in the Netherlands, to re-use a casting basin for more than one tunnel, this helps to defray the high initial cost of the basin.

Although the development of these two types of sunken tubes have advanced along comparatively independent lines there has been a trend in recent years to combine the advantages of each. Two recent steel shell tunnels in the United States have diverged from the traditional circular shape to more economical use of space. The Bay Area Rapid Transit Tunnel under San Francisco Bay has an oval shape. The semi-circular ends each carry one track and are reduced to single track tunnel height. A rectangular compartment between contains a ventilation exhaust duct and a gallery for utilities and maintenance access. The 63rd Street Tunnel built for the New York City Transit Authority under the East River, had a modified rectangular shape. The four track-compartments were placed two above and two below. The upper corners were rounded and the lower corners bevelled inward to reduce the width at the base. The cross sections of both of these tunnels are shown in Figure 38.

An even more striking example of combining the two methods are the similar Ohgishima (Keihin Canal) Tunnel under Kawasaki Port (1974) and the nearby Tokyo Port Tunnel (1976)(Ref. 115). Each are rectangular shaped multi-lane vehiclar tunnels of reinforced concrete with an outer steel lining, giving a similar outward appearance. The units for the four-lane Ohgishima tunnel, constructed for Nippon Kokan Company (Japan's largest steel manufacturer) were fabricated by the steel shell method with a 1/2 inch (13 mm) thick steel lining. After launching the reinforced concrete lining was cast inside at a fitting pier. The six-lane Tokyo Port Tunnel units were constructed as reinforced concrete units with a thin steel membrane on the outside. All nine units were built together in a large casting yard, 2130 x 415 feet (650 x 126 m), formed by closing off the ends of an unused canal. These units were launched by flooding the casting yard as described in the last section.

6.40 SEALING METHODS

6.41 General

Some driven tunnels may be subject to a greater head of water than sunken tube tunnels. However, a sunken tube tunnel has a virtually limitless reservoir a few feet above it. The thin layer of cover would offer a little resistance to the flow of the water into the sunken tube, in the event of leakage. Tunnels driven beneath a body of water are usually further below the river or harbor bottom. This additional depth of cover offers greater resistance to the flow toward the tunnel. It is useful therefore to discuss the groundwater control methods used to keep these unusual tunnels dry. As in the case of other types of tunnels discussed, the total tunnel environment, design, and construction largely determine the types of waterproofing and sealing used. Due to the close proximity of an immense body of water to the tunnel, unusually stringent methods are employed to prevent leakage. In most sunken tube tunnels this consists of an extremely efficient and costly membrane outside the tubes and elaborate sealing methods of the relatively few joints between the long tube units. Recent alternatives to the use of an outer membrane will also be discussed.

6.42 Sealing Steel Shell Tube Tunnels

The methods used for waterproofing and sealing the joints of steel shell tunnels have been fairly well standardized, though some recent variations should be noted. It was mentioned previously that present-day steel shells are all welded construction with most of the welding done by machine. The steel shell constitutes a continuous membrane that is the primary waterproofing for this type of sunken tube. The concrete lining is placed inside the steel shell under adverse conditions similar to lining a driven tunnel. The problem of trying to place dense and impermeable concrete and good construction joints under such conditions makes the resulting concrete lining of questionable aid in helping the steel shell to keep the tunnel watertight. For most shell type tunnels corrosion of this steel membrane is prevented by the surrounding tremie concrete ballast. In the case of the Bay Area Rapid Transit Tunnel a cathodic protection system was provided for the shell as there was no outside ballast concrete. It was felt that this was sufficient to provide against underwater corrosion.

The most common method for sealing the joints between the units is for divers to place a U-plate cofferdam spanning the joint as shown in Figure 37. Tremie concrete is placed between the cofferdam and steel shell completely encasing the joint. The space between bulkheads is then dewatered and steel plates are welded across the joint gap to make the shell/membrane continuous and concrete is placed to make the lining continuous. Two recent steel shell tunels, the BART Tunnel and the second Mobile Tunnel used rubber gaskets for primary seals.

6.43 Sealing Reinforced Concrete Tube Tunnels

Reinforced concrete tube units are usually covered with a steel or multi-ply asphaltic membrane, or a combination of the two. If a steel membrane is used it consists of relatively thin welded plate, about 1/4 inch (6 mm), and is used as the base slab and outer wall form. After the roof is poured the steel membrane can be continued across the top or a multi-ply membrane may be used for the roof and covered with a thin layer, 3 inches (75 mm), of protective concrete. If ply membrane is used on the outer walls timber protection will most likely be used. In addition to the outer membrane, considerable care is exercised to make the structural concrete as dense and impervious as possible. In the Netherlands where a number of such tunnels have been constructed, the procedures for producing impervious concrete have been sufficiently perfected that for several recent tunnels the outer membrane has been eliminated (Ref. 73). The procedures for producing impervious concrete have been detailed in Section 2 of this report.

For a tunnel to be watertight no water may penetrate the concrete, the construction joints between pours, the expansion joints between sections, or the sealed joints between tube units. To prevent penetration through the concrete not only must the concrete be dense, but it must be free of cracks caused by heat of hydration. While the concrete is setting, uneven temperature gradients exist between the center of walls or slabs and the outer layers which tend to cool faster. Placing wall and roof concrete on a previously hardened base slab tends to cause vertical cracks in the wall near the slab. To combat this type of cracking, a series of cooling water pipes are embedded in the walls being placed. Cooling water is introduced in the bottom near the slab and is warmed by the concrete as it progresses upward in the wall. The temperatures are carefully monitored and the flow can be altered, stopped and started to match individual pour requirements. The steel cooling tubes remain in the concrete to act as permanent shrinkage steel. See Janssen, Ref. 73, for a more complete discussion of these methods. Additional measures that are used include selecting cement with a low heat of hydration and using timber facing on forms instead of steel because of its insulating properties.

To insure watertightness of the joint between slab and walls a waterstop is used and the cold joint is rough chipped and grouted with cement mortar. The concrete pour lengths in a unit average about 65 feet (20 m) with expansion joints between. Sealing of this type of joint is usually done in one of two ways in Holland. See Figure 40, Details A and B. An inner seal consists of a composite rubber/metal waterstop. On the outside face a multi-layer membrane may be used as shown in Detail A, or an additional seal of polyurethane putty may be used as shown in Detail B. The outer face seal was added as a backup measure because it was found that concrete around the waterstop occasionally had segregated gravel pockets. For the construction of the Hem Railway Tunnel foam rubber strips and thin embedded steel tubes were attached periodically to the waterstop. After concreting, epoxy resin was injected in the tubes to fill and consolidate any rock pockets. This method proved successful and the outer seal could be eliminated. It is shown in Detail C of Figure 40.

The joints between the sunken tube units are usually sealed with gaskets as shown in Detail D of Figure 40. The initial or primary gasket, known as the Gina profile, is made of hard rubber with a relatively soft triangular tip. It is mounted continuously around the circumference of the front end of a tube unit being sunk. When in place it bears against a corresponding continuous steel plate on the rear end of the preceding unit set. When water is removed from between the bulkheads the water pressure acting on the rear end of the new tube forces it forward compressing the seal. After removing water from between bulkheads a secondary continuous rubber seal known as an Omega profile is added as shown in Detail D of Figure 40. This shape allows some flexibility in movement not possible with the rigid shell type seals, permitting a degree of settlement or ground movement.

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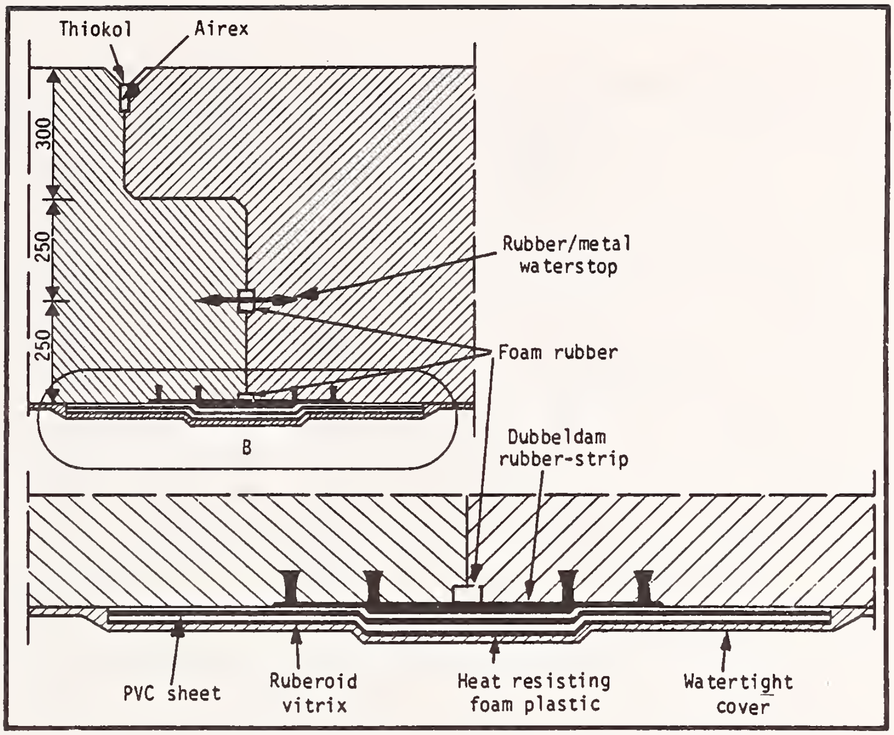
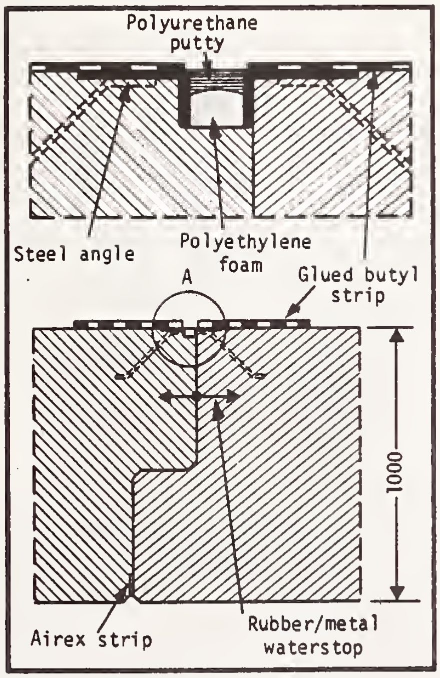


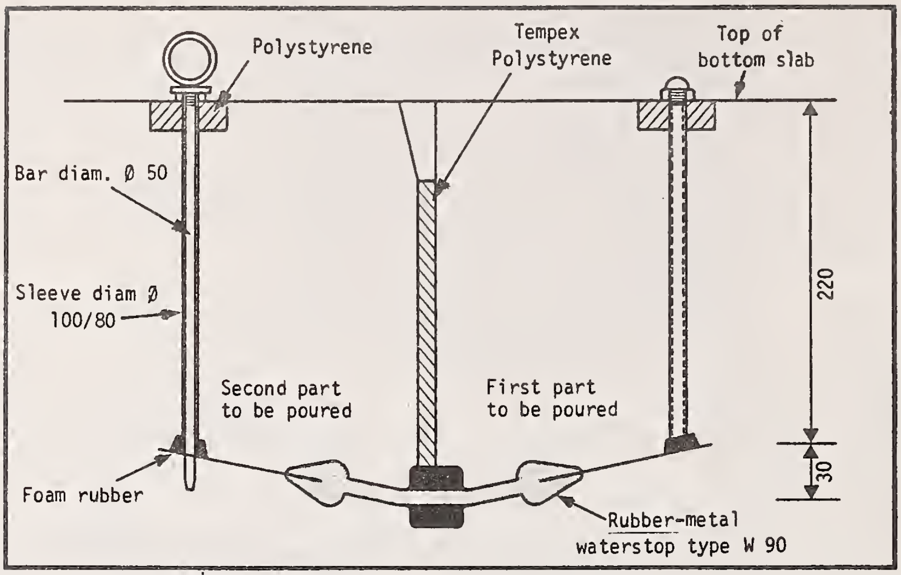
Detail A - EXPANSION JOINT WITH DUBBELDAM RUBBER STRIP

Note: Units in mm.



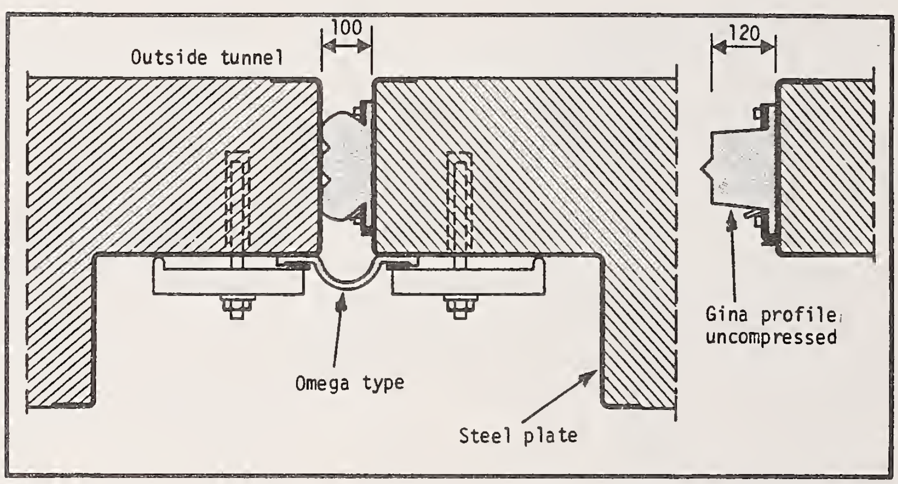
Detail B - EXPANSION JOINT WITH RUBBER/METAL WATERSTOP AND POLYURETHANE PUTTY

Figure 40 - JOINT SEALING DETAILS FOR CONCRETE BOX TUBES (From Janssen, Ref. 73)



Detail C - DETAIL OF INJECTION TUBES TO RUBBER/ METAL WATERSTOP

Note: Units in mm.



Detail D - DETAIL OF UNIT JOINT SEAL SHOWING GINA GASKET AND OMEGA GASKET

Figure 40 - JOINT SEALING DETAILS FOR CONCRETE BOX TUBES (continued) (From Janssen, Ref. 73)

7.00 CRITERIA FOR SELECTION OF GROUNDWATER CONTROL METHODS

7.10 INTRODUCTION

The previous sections have described various methods, materials, and applications for keeping groundwater out of underground structures in general and urban transportation tunnels in particular. They have been grouped and discussed in a manner to simplify classifying types of control methods with four basic design and construction methods: 1) cut-and-cover tunnels, 2) soft ground tunnels, 3) rock tunnels, and 4) sunken tube tunnels. This section will detail how the various control methods can be selected to be compatible with the design, construction methods, and temporary construction phase groundwater controls.

While groundwater control methods used during construction are generally the option of the contractor, the control methods that apply for the life of the permanent structure are rightfully the responsibility of the owner's design engineer. The owner may permit the contractor to substitute an alternate material for waterproofing if it is equal to that specified, but the tunnel design chosen often determines the basic type of groundwater control required.

7.20 BASIC TYPES OF TUNNELING METHODS

The decision to use one of the four basic tunnel construction methods is an option of the designer. Although in the United States, construction methods are generally left to the contractor, the design of the permanent structure, including the permanent groundwater control methods, are so different and interdependent with the basic construction method that the decision must be made before the structure is designed. There have been exceptions to this, but they are rare. In Washington, one contractor on the WMATA system submitted, and had approved, an alternate design for twin soft ground tunnels on a section originally designed as a two-track cut-and-cover box section. Even in such a situation the decision and responsibility for such a change rightfully rests with the designer.

The choice of which basic type of tunnel should be used for a proposed project must consider many factors of structure use, location, site conditions, and previous experience. If the tunnel is close to the surface, a cut-and-cover structure would likely be most cost effective. If the structure consists of single or two-lane (or rail) tunnels with the invert 50 feet (15 m) or more below the surface, a driven tunnel will probably be less expensive than cut-and-cover. However, if the tunnel is below a water table that cannot be lowered and compressed air is required that factor would lower the depth where cut-and-cover construction is competitive. Use of the newer slurry shields or earth pressure balance shields help to offset the higher tunneling costs in wet soil. Excess surface disruptions, high utility density, and underpinning requirements could raise the level where tunneling is competitive.

When there is a choice of tunneling in soft ground or rock, the rock tunnel is usually less expensive. If getting the tunnel down into bedrock means steep grades or additional tunnel length, it may not be worthwhile. Driving tunnel in a mixed face of earth and rock is considerably more expensive than either earth or rock separately. If the earth/rock interface is fairly close to the surface, cut-and-cover tunneling may be considered as the cost differential is not as great as it is in a driven tunnel.

Sunken tube tunneling is an alternative to a driving a tunnel under water. In recent years it is becoming a more attractive alternate in cost effectiveness, due in part, to improvements in construction and sealing techniques. On one project, a section of the Rotterdam Metro, this method was substituted for cut-and-cover tunneling by extending a canal down city streets. This would not be practical for most urban areas in the U.S. For a shallow, relatively short water crossing where shipping traffic is light, it is possible to fill in ground and use cut-and-cover tunneling, but on longer, deeper crossings the choice is between driven tunnels and sunken tubes. Driven tunnels are limited to single or double lane (or track) tubes. Compressed air work, formerly required for most underwater tunnel crossings has become increasingly more exensive, but the use of earth pressure balance shields should help to overcome that drawback. As discussed in Section 6, sunken tubes need not be as limited in width as are driven tunnels, and many multi-lane tunnels have been constructed by this method.

7.30 APPLICABILITY OF GROUNDWATER CONTROL METHODS

Once the basic tunneling method has been chosen the list of appropriate permanent groundwater control methods is reduced to a number of options compatible with that type of structure. Each of the general control methods described in Sections 2 through 6 will be discussed in regard to use in one or more of the four basic types of tunnels.

7.31 Water-Resistant Cast-In-Place Concrete Linings

The methods described for designing, mixing, transporting, placing and curing a dense impervious concrete lining as described in Section 2 should be used to some extent in all tunnels dependent on a strucural concrete support/lining. For cast-in-place linings of driven tunnels in earth or rock, or for steel shell sunken tube linings, the problems of placing in a confined, possibly wet environment may invalidate the effectiveness of such measures. These methods can be most effective in cut-and-cover tunnels, precast reinforced concrete sunken tubes, precast concrete segmented tunnel linings, and precast concrete panel slurry walls. With proper care in applications of these techniques it is possible to eliminate the need for additional outer membrane waterproofing. This has been accomplished in some cut-and-cover tunnel structures in the United States and in some sunken tube tunnels in the Netherlands. Even in tunnels where other waterproofing measures are used, good dense impervious concrete is an effective secondary groundwater control measure as well as good practice for achieving structural integrity.

In general, the use of good concreting practices are compatible with all methods of controlling groundwater during construction except as noted above when placing concrete under adverse conditions in a driven tunnel. If concrete must be placed in a tunnel under compressed air, additional delays are experienced bringing the concrete through an air lock. If the construction control method does not leave the tunnel sufficiently dry, water will have to be diverted in the area where concrete is being placed.

7.32 Applied Waterproofing Membranes

Section 3 describes a number of waterproofing membrane materials that can be used to place an impervious envelope on the outside of an underground structure. These are commonly used, and most easily applied, to cut-and-cover structures and sunken tube tunnels. On some cut-and-cover sections where reliance on water-resistant concrete can be expected a membrane may be used on the roof only to aid in protecting against the pressure of ponded groundwater. In order to use a membrane in a driven tunnel it is necessary to place a smooth outer lining to which the membrane is attached and then place a structural lining inside of that. The placing of the additional lining and membrane has been done in some tunnels in Europe. At current prices, for the size of tunnel required for single track rapid transit, it would add about $400 to $450 per linear foot (0.3 m) to the tunnel cost. The designer would have to decide for each particular tunnel whether this additional cost is warranted or whether other less costly measures would suffice.

The waterproofing membranes described fall into the following categories: 1) Built-up multi-ply membranes of hot-applied bituminous materials alternating with a suitable fabric material, 2) Preformed, multi-layered membranes, 3) Plastic or synthetic rubber sheets, 4) Bentonite clay sheets, spray or powder, 5) Cold, liquid-applied membranes, 6) Brick set in asphalt mastic, 7) Cementitious coating applied to the inside of the structure, and 8) Thin continuous metal membranes. The first six types above have all been used at times for cut-and-cover tunnels. Some materials such as brick in mastic and bentonite clay sheets are more easily applied to vertical surfaces. In general the bituminous materials are less expensive than single-sheet man-made membranes, but depending on the number of plies required, installation labor costs are higher. Item 7 above, cementitious coatings are not ordinarily used as a full original membrane but rather for repair and maintenance. A thin corrugated aluminum membrane (Item 8) was used for a tunnel in Norway, but a more common use is of steel membranes on the outside of sunken tube tunnels. For precast concrete sunken tubes, multi-ply membranes may be used in lieu of a steel membrane.

Whatever groundwater control measure is used during construction it must keep the tunnel effectively dry in order to place most membranes. This applies to predrainage or cutoff walls for cut-and-cover tunnels; trenching and sumping would likely not be sufficiently effective. In driven tunnels grouting, caulking or other means would have to be complete and effective prior to placing the initial lining for the membrane. As sunken tube tunnels are fabricated on dry land there is no problem of water control during application of the membrane.

7.33 Segmented Tunnel Lining Groundwater Control Measures

The measures used to prevent groundwater inflow into segment-lined tunnels are unique to that type of lining, as described in Section 4. Segmented linings can be used in driven tunnels in either earth or rock, but are more common in earth. The segments are usually fabricated of cast iron, structural steel or reinforced concrete. The segments should be fabricated, with flaw free casting or continuous welding, so that they themselves are impervious. In the casting of concrete segments the methods previously described for achieving impervious dense concrete can be realized in the consistent environment of a casting yard. Steel segments must be coated to prevent corrosion. A coal-tar epoxy is often used for an outer coating, with a zinc silicate coating used on the inside surfaces.

In addition to making the segments watertight, the joints between segments must be sealed. For cast iron and steel segments, the contact flanges are machined for an accurate, tight fit. Caulking grooves on the flange edges are filled with continuous lead strips. To prevent leakage through bolt holes in the flanges plastic grommets (washers) are tightened into the holes by steel washers at the head and nut. With precast concrete segments, bolts, if they are required, also have grommets for sealing bolt holes. Instead of lead, asbestos-cement or epoxy-elastomerics are used for caulking concrete segments. Because concrete segments cannot be fabricated to the close tolerances of cast iron or steel, it is common to also place a gasket seal between segments to prevent water inflow before final caulking. These gaskets are usually of man-made materials and may be preformed of fairly firm material or a softer material sprayed or troweled on the flange. Care must be exercised to prevent contamination of these gaskets in the tunnel environment. It is anticipated that current and future research will continue to improve the manufacture and sealing techniques of precast concrete segments.

Each of the groundwater control methods described in Volume 1 as applicable to driven tunnels are compatible with the methods described for sealing segmented linings, including predrainage, freezing, grouting, compressed air, slurry shields and earth pressure balance shields.

7.34 Chemical Grouting of Soils

Soils may be grouted to prevent water inflow into a tunnel, to strengthen the soil by consolidation, or a combination of both. Grout can be used for either a cut-and-cover tunnel or soft ground tunnel. Chemical grouting is more useful if done prior to excavation and therefore serves as a construction aid. To prevent groundwater inflow into the tunnel during its service life, grouts must be used that will not be susceptible to deterioration by any constituents of the local groundwater. While chemical grouting is more commonly used in Europe and Japan than in the U.S., it is a viable alternate groundwater control method in areas where the water table cannot be lowered. Chemical grouting of soils would not be used solely as a permanent structure groundwater control method, but if it is to be used during construction it should be considered also as a permanent control measure. The advantages of chemical grouts over the less expensive cement, or particulate grouts, is their ability to flow through very fine voids, and to have more easily controlled and a wider range of setting times.

The grouts that have been used for consolidating and sealing soil can be grouped in the following categories:
1) Silicate-based grouts, 2) Plant product-based grouts,
3) Acrylamides, 4) Phenoplast resins, 5) Aminoplast resins, and
6) Petrochemical derivative grouts. The choice and use of these grouts, either singly or in combinations is dependent on many factors of soil, groundwater flow, location and working conditions. Grouting is usually performed by a specialty contractor, and workmen must be skilled in the use of grouts to insure they perform effectively. During application constant monitoring of grouting, and sampling of grouted soil, is necessary to insure complete coverage and uniform dispersion of grout.

7.35 Consolidation Grouting of Rock Tunnels

Consolidation grouting is a primary means of preventing groundwater intrusion of rock tunnels. The most common grout used is cement grout occasionally containing a sand or other material filler. For special application for filling thin seams or where a controlled set time is required one of the chemical grouts mentioned above may be used in lieu of, or in conjunction with, a cement grout. In most rock formations, water flow takes place through joints, seams or cracks rather than through the rock material. The grout prevents this by physically filling joints and cracks in the rock around a tunnel.

Although grouting can be done from the surface before tunneling, provided the tunnel is not too deep, it is rarely ever done. Most rock tunnels are not wet for their entire length, so grouting is usually done from the tunnel on an as-needed basis. If water flow at the face is very severe, excavation may be halted to grout ahead of the tunnel. Otherwise grouting may be done between excavation and concrete placing or after concreting, depending on water flow and the type of rock. If the water has a deleterious effect on the rock, the tunnel should be grouted as soon as possible, even if the flow is not heavy. Other advantages of grouting before concreting are that it is easier to locate leakage sources and that concreting can be done in a dry environment. An advantage of waiting till after concreting is that the consolidation grouting can then be combined with the contact grouting instead of performing it as a separate operation.

7.36 Contact Grouting of Tunnel Linings

Used in soft ground and rock tunnels, contact grouting helps a cast-in-place concrete lining keep water out by filling any voids that may have been left between the concrete and the ground. Because of the difficulty of placing concrete between the forms and the ground in a driven tunnel, voids may be left no matter how competent the concrete crew may be. In a rock tunnel the excavated rock may be jagged and uneven. If primary rib support and blocking are used the profile is more uneven. In soft ground tunnels hollows exist in the primary linings of ribs and lagging or pressed liner plates. As the level of concrete rises, air is trapped in these hollows, particularly in the arch. Cement grout, possibly with a sand filler and a small percentage of bentonite, is pumped into a prescribed pattern of holes to fill these voids and eliminate possible water accumulation.

7.37 Diaphragm Cutoff Walls

Though diaphragm cutoff walls are useful in controlling groundwater for cut-and-cover structures in soil during the construction stage they are usually not sufficiently reliable to keep the permanent structure dry without other control measures.

Leakage through the wall above the invert can be found and repaired during excavation. Finding possible leakage sources through the wall below the invert or under the wall is extremely difficult, and repairing such leaks, even worse. With a small amount of leakage through the wall the volume of leakage possible in the structure is considerably less than it would be if the wall were not present, but the same head of water would be acting against the structure. A continuous reinforced concrete wall or precast concrete panel wall will probably leak less than a soldier pile and concrete wall provided special care is taken with the joints. The soldier pile and concrete wall is more susceptible to leakage at the many steel/concrete interfaces where continuous bonding is difficult when placing concrete under slurry. If the invert of the structure is in bedrock, all of the wall is exposed during excavation and any possible leakage can be repaired, but the problem of sealing the wall to rock is expensive and difficult. If the rock is not sound and water leakage can take place through the rock additional measures such as grouting, impervious concrete, or a membrane covering will still be required.

7.38 Permanent Lowering of the Water Table

Though this has never been used to keep a transportation tunnel dry during its service life, it is possible under certain conditions to use a series of small tunnels and drill holes to lower the water table at the tunnel site and keep it low. In most U.S. cities this would not be practical because of effects on existing structures. It could be applied to either cut-and-cover tunnels, soft ground tunnels, or rock tunnels under proper conditions.

7.39 Sunken Tube Tunnel Control Measures

As in the case of tunnels with segmented linings, the groundwater control measures used for sunken tube tunnels are dependent to a large degree on the way the tunnel is designed and constructed and is unique to that type of tunnel construction. This type of tunnel is also unusual in that the water control measures during construction and for the life of the structure are the same. Due to the continual presence of a large body of water directly above a sunken tube tunnel very stringent waterproofing measures are used. Two basic types of tubes, steel shell and precast concrete, are each fabricated in long units of several hundred feet each, towed to the tunnel site, and lowered into place, as detailed in Section 6.

The steel shell type uses its continuously welded shell as waterproof membrane. After lowering a unit into position with temporary bulkheads at each end it is sealed to the preceding unit. This is usually done by having divers set a U-plate cofferdam spanning the joint and then tremie concrete is poured to encase the joint. After the space between the bulkheads is dewatered, steel plates are welded across the gap to make the steel membrane continuous.

Precast concrete sunken tube units may have a thin steel outer membrane, a multi-ply membrane, or a combination of the two. Some recent units in the Netherlands, contructed of dense impervious concrete were considered sufficiently waterproof that the membrane could be eliminated. Double rubber seals are usually used between precast units, the outer called a Gina type seal and the inner an Omega type seal. These give this type of tube a degree of flexibility.

7.40 SAMPLES OF CURRENT PRACTICE IN SPECIFYING CONTROL METHODS

To sample current practice of tunnel groundwater protection methods several dozen individual transportation and other tunnel contract specifications were reviewed. Three tables have been prepared showing the basic type of structure, waterproofing and other control methods specified, maximum allowable leakage (where specified) and specified treatment of tunnel leakage. There is one table each for cut-and-cover structures, soft ground tunnel structures, and rock tunnel structures. Tunnels from seven rapid transit systems comprise most of the entries, but tunnels from four other local and federal agencies are also presented. Each table is presented in reverse chronological order with the more recent projects listed first. Because of the amount of data and variety of water protection methods used, it was impractical to list all data in a single page format. Tunnel linings, water protection methods, and joint treatments are detailed on a second sheet of each table and letter-coded for use in the table, as applicable to each tunnel project. Where alternate protection methods are allowed by specification, all are listed. Most agencies will permit substitution of contractor-proposed alternate protection methods provided the contractor and/or the material manufacture can prove, through tests or previous installations, that it is equivalent to the method specified.

7.41 Cut-and-Cover Structures

Sample specification summaries of methods for controlling groundwater in cut-and-cover underground structures are given in Table 5. Many of the structures listed rely to a large extent on thick, good quality concrete walls and slabs as a primary deterrent to water intrusion, although it is not specifically listed as a waterproofing method. The required membranes are more of a secondary defense to keep water from any shrinkage cracks or other cracks that might develop. The concrete box structures are highly stable, water resistant structures if the concrete is placed properly and any minor defects are corrected.

The exception to this, as mentioned in Section 3, is the line section of the New York City Transit Authority. Built as a composite structure of structural steel bents at 5-foot (1.5 m) centers with unreinforced concrete between, this

Table 5: SAMPLE CUT & COVER UNDERGROUND STRUCTURES & METHODS SPECIFIED FOR CONTROL OF GROUNDWATER

Owner & Year	Use of Structure	Type of Structure	W.P. & Protection Coverage*			Allowable Leakage	Leakage Treatment	See Note No.
			Roof	Sides	Invert			
MARTA Atlanta 1976	Rapid Transit Station	C.I.P. Conc. Box	B or E, G & M	B or E, G & M	M (Only)	Not Specified	Injected Waterproofing	4
MARTA Atlanta 1976	Rapid Transit Line	C.I.P. Conc. Box	S, G & M	S or T J & M	M (Only)	Not Specified	Injected Waterproofing	4
Edmonton 1975	Rapid Transit Station	C.I.P. Conc. Box	C & K	N & U	None	Not Specified	Repair Membrane	3
WMATA 1975	Rapid Transit Line	C.I.P. Conc. Dbl. Box	B, G & O	B, L & O	B. H, L & O	For 1 Yr. No Standing or Free Water on Exp. Conc.	Bentonite Slurry or Epoxy Injection or, Patch if Accessable	
New York 1972	Rapid Transit Line	C.I.P. Conc. Dbl. Box Below G.W.	B, D, G & M	B, D, G & M	B, D, G & M	Not Specified	Repair or Replace Portions not Watertight	
WMATA 1967-1975	Rapid Transit Line	C.I.P. Conc. Dbl. Box	A, I & R	A, L & Q	None	Not Specified	Bentonite Slurry or Epoxy Injection	1
WMATA 1967-1975	Rapid Transit Station	C.I.P. Conc. w/Arch Roof	B, G & O	E. L, O & Q	None	Not Specified	Bentonite Slurry or Epoxy Injection	1
BARTD 1967	Rapid Transit Station	C.I.P. Conc. Box-Multi Level	F, G & M	F, J & M	F, H & M	0.2 Gal/Min / 250 Lin Ft 0.15 Gal/Hr/1000 S.F. 6 Ml/Hr/M^2	Repair or Replace W.P. Waterproofing	
BARTD 1965	Rapid Transit Line	C.I.P. Conc. 2-Single or 1-Dbl Box	F, G & M	F, J & M (Jt. Only)	M (Only)	0.2 Gal/Min / 250 Lin Ft 0.55 Gal/Hr/1000 S.F. 22.3 Ml/Hr/M^2	Repair or Replace Waterproofing	2

*Letter designations from specified method listing. See next page.
#1: Install 2 mil polyethelene both sides of vert. bentonite. Remove inside film before pouring concrete.
#2: Pipe sleeves w/polysulfide sealant for openings.
#3: Very low ground water conditions.
#4: Pipe sleeves w/rubber base caulk @ openings.

1 gpm = 0.63 l./s. 1 ft. = 0.305 m.

Table 5 SAMPLE CUT & COVER UNDERGROUND STRUCTURES & METHODS SPECIFIED FOR CONTROL OF GROUNDWATER

(Continued)

WATERPROOFING

A	Asphalt dampproofing
B	Built-up asphalt membrane
C	Built-up asphalt membrane, reinforced
D	Brick in asphaltic mastic
E	Bentonite panels (continuous)
F	Butyl rubber membrane

WATERPROOFING PROTECTION

G	Concrete slab
H	Concrete slab, both sides of membrane
I	Hardboard
J	Insulation board
K	Cement powder
L	Plywood at joint niche

JOINT TREATMENT

M	PVC waterstop at external joints
N	Metal waterstop, 12 in. (300 mm) wide at ext. joints
O	Bentonite, triangular tube seal in niche
P	Premolded filler
Q	Bentonite panels, 2 ft. (0.6 m) wide, single
R	Bentonite panels, 2 ft. (0.6 m) wide, double
S	Bentonite panels, 4 ft. (1.2 m) wide, single
T	Membrane, 4 ft. (1.2 m) wide
U	Wet grout

structure specification requires a stringent waterproofing membrane of brick in asphaltic mastic and built-up asphalt ply membrane. Although the concrete specifications are as exacting as other agencies, the steel-concrete interfaces are difficult to bond completely and may allow possible leakage. Thus it is necessary to rely on more severe waterproofing measures, which, though relatively expensive, have proved effective over many years of use.

The other waterproofing methods specified cover a range of membrane materials including built-up asphalt membranes, with or without reinforcing, butyl rubber, and bentonite panels. There is no apparent trend in the use of materials for the years covered and they more likely reflect the preference of the designers and the locations of the project. One WMATA specification requires merely dampproofing, and one on MARTA requires no waterproofing except at joints. Most require membranes on the roof or roof and sides only, relying on a heavy base slab, needed to counteract buoyancy to be sufficiently water resistant.

Most structures require PVC waterstop at joints, but some specify bentonite in niches with bentonite panels covering the outer concrete face at the joint. The Edmonton station specifications required a metal waterstop.

Only on the BART system were specific allowable leakage requirements given. All specifications called for leakage repair but only a few designated specific remedies of injecting bentonite slurry or epoxy.

7.42 Soft Ground Tunnel Structures

Sample soft ground tunnel specifications on groundwater control methods are given in Table 6. While the majority of cut-and-cover structures are cast-in-place reinforced concrete, this is not necessarily so in soft ground tunnels. There are a number of possible ground support/lining combinations, which may affect the type of waterproofing or sealing methods that can be used. Specified primary and secondary linings (and possible alternates) are listed in code as are the specified waterproofing measures. A list of code definitions is given on the second sheet of Table 6.

Of the seven tunnels listed, five are rapid transit line tunnels and two are sewer main tunnels. Three have cast-in-place concrete linings specified, three have segmented linings specified, and the seventh has several alternate possible linings. The three tunnels with segmented linings are transit tunnels and have a partial concrete lining of invert and walkway only. In general, grouting is required outside the lining, and between the primary lining and concrete on projects where there is a full concrete lining.

Table 6: SAMPLE SOFT GROUND TUNNEL STRUCTURES & METHODS SPECIFIED FOR CONTROL OF GROUNDWATER

Owner & Year	Use of Structure	Type of Structure	Linings: Primary Second*	W.P. & Protection Coverage* Arch	W.P. & Protection Coverage* Invert	Allowable Leakage	Leakage Treatment	See Note No.
CCSF Dept. of Public Works 1979	Sewer Main Tunnel	Circular	B & E or C & F or D & F	H, I & S H, I & Q H, I & Q	H, I & S H, I & Q H, I & Q	$\frac{\text{1 Gal/Min}}{\text{10,000 Ft}^2}$ & No Visible Running Water $\frac{\text{6 Gal/Hr}}{\text{1,000 S.F.}}$ 244 Ml/Hr/M^2	Repair w/Injected Epoxy Mortar	
WMATA 1977	Rapid Transit Line	Circular w/Invert Single Track	A & E	H, I, M & O	H, I, M & O	$\frac{\text{0.2 Gal/Min}}{\text{250 Lin Ft}}$ $\frac{\text{.85 Gal/Hr}}{\text{1,000 S.F.}}$ 34.5 Ml/Hr/M^2	Pressure Grout	
Baltimore 1976	Rapid Transit Line	Circular Tube Single Track	C & G	H, J, K P & T	H, J, K P & T	@ Joints $\frac{\text{.000025 Gal/Min}}{\text{Lin Ft Joint}}$	Bentonite Slurry to Create Envelope or Epoxy Injection	
WMATA 1973	Rapid Transit Line	Circular Tube Single Track	C or D & G	H, J, K, R & T	H, J, K, R & T	$\frac{\text{0.2 Gal/Min}}{\text{250 lin ft}}$	Recaulk Leak Joints Patch Damaged Coating	
BARTD 1965	Rapid Transit Line	Circular Tube Single Track	C & G	H, J, K R & T	H, J, K R & T	$\frac{\text{0.2 Gal/Min}}{\text{250 Lin Ft}}$ $\frac{\text{0.9 Gal/Hr}}{\text{1,000 S.F.}}$ 36.5 Ml/Hr/M^2	Recaulk Leak Joints Patch Damaged Coating	1
Metro Dist. Commission Boston 1959	Sewer Main Tunnel	Circular	B & E	H, I & M	H, I & M	Not Specified	Not Specified	
New York 1955	Rapid Transit Line	Dbl Box Steel Sets w/Lagging	A & E	I, L & N	None	Not Specified	Repair or Replace Portions not Watertight	2

*Letter designations from specified methods listing. See next page.

#1: Segments supplied by BARTD

#2: a) Mixed face tunnel. Arch in soft ground
b) Extensive track drainage provisions. Weep holes in invert.

Table 6 SAMPLE SOFT GROUND TUNNEL STRUCTURES & METHODS SPECIFIED FOR CONTROL OF GROUNDWATER
(Continued)

TUNNEL LINING

Primary:
A Steel ribs and lagging
B Steel liner plates
C Fabricated steel segmented lining
D Precast concrete segmented lining

Secondary:
E Cast-in-place lining, structural
F Cast-in-place lining, finish only
G Cast-in-place invert and walkway only

WATERPROOFING

H Grouting of voids behind lining
I Contact grouting of concrete lining
J Segments - external coat - coal tar epoxy
K Segments - internal coat - zinc silicate
L Concrete blocking, steel supports to roof

JOINT TREATMENT

M PVC waterstop
N Copper waterstop
O Bentonite panels & triangular niche tube
P Joint sealant (to be approved)
Q Caulking (to be approved)
R Lead caulking
S Rubber or Neoprene waterstops
T Bolts with washers and plastic grommets

Cast-in-place linings require PVC or copper waterstop at the joints although the contractor on the New York Subway project received approval to omit the copper waterstop and provide remedial grouting where required instead. Those projects calling for steel segmented linings required protective coatings inside and outside. Most segmented linings additionally required plastic grommets for connecting bolts and caulking between segments. On the Baltimore tunnel a joint sealant was specified in lieu of caulking.

Where the allowable leakage permitted in the transit tunnels was specified, it was comparable to those given for cut-and-cover structures in Table 5. The allowable leakage given for the San Francisco sewer tunnel was about eight times that allowed for transit tunnels. Leakage treatment requirements were also comparable to those for cut-and-cover tunnels, consisting of pressure grouting, bentonite slurry, or epoxy injection.

7.43 Rock Tunnel Structures

Table 7 gives sample specifications for groundwater control in completed rock tunnel structures. As in the case of driven soft ground tunnels the primary and secondary linings of rock tunnels may have an effect on the choice of groundwater control measures. While a number of the tunnels researched had alternate primary lining choices, they all specified cast-in-place concrete secondary linings. Though some segmented linings have been built in rock tunnels, there are very few and these have been discussed in Section 4 of this report.

All tunnels specified consolidation grouting of rock (where required) and contact grouting of the secondary lining. Most required waterstops for joints though some called for bituminous, sponge rubber, or paraplastic fillers. The Melbourne Underground Rail Loop Authority (MURLA) rapid transit projects required plastic grommets for bolts and caulking for those sections using steel liner plates as a primary lining. The New York Subway tunnels in rock did not require any waterproofing for the inverts which were relatively thin slabs. Weep holes in the invert were specified with the tunnel drainage system designed to take the anticipated leakage.

The only tunnel specification in this group to designate an allowable leakage was the MURLA rapid transit. It is interesting to note that this requirement was considerably more severe than those given for U.S. rapid transit tunnels in the previous tables. Those specifications that gave particular requirements for leakage treatment mentioned repairing or replacing portions not watertight, or specified grouting to seal off leaks.

Table 7: SAMPLE ROCK TUNNEL STRUCTURES & METHODS SPECIFIED FOR CONTROL OF GROUNDWATER

Owner & Year	Use of Structure	Type/ Linings of Structure*	W. P. & Protection Coverage* Arch & Sides	Invert	Allowable Leakage	Leakage Treatment	See Note No.
CCSF Dept. of Public Works 1979	Sewer Main Tunnel	B & F Circular	G,H & L	G, H & L	Not Specified	Re-Grout	
U.S. Bureau of Reclam. 1975	Water Tunnel	B & D or or C & F Circular	G, H, M & R	G, H,	Not Specified M & R	Repair w/Grout	
Corps of Engineers 1975	Water Tunnel	B, C, or or D & F Circular	G, H, L & R	G, H,	Not Specified L & R	Not Specified	
MURLA Melbourne 1973	Rapid Transit	A, D & F or A, C & F Circular	G, H & L or G, H, K L, S & T	G, H & L or G, H, K L, S & T	5 $ml/Hr/m^2$	Steel liner: Repair or Replace Coating	
WMATA 1967-75	Rapid Transit Line	E & F Circular	G, H & L	L	Not Specified	Contact Grouting	
WMATA 1967-75	Rapid Transit Line	E & F Horseshoe	G, H & L	I, J & L	Not Specified	Contact Grouting	
WMATA 1967-75	Rapid Transit Station	B, E & F Horseshoe	G, H & L	L	Not Specified	Contact Grouting	
New York 1969	Rapid Transit Line	B & F Horseshoe (4 track)	G, H & L	None	Not Specified	Repair or Replace Portions not Watertight	1
New York 1969	Rapid Transit Station	B & F Arch Roof	G, H & L	None	Not Specified	Repair or Replace Portions not Watertight	1
BARTD 1965	Rapid Transit Line	B & F Horseshoe	G, H, P & Q	P & Q	Not Specified	Contact Grouting	
New York 1955	Rapid Transit Line	B & F Dbl. H.S.	G, H & O	None	Not Specified	Repair or Replace Portions not Watertight	1

*Letter designations from specified methods listing. See next page.
#1: Extensive track drainage provisions. Weep holes in invert.

Table 7 SAMPLE ROCK TUNNEL STRUCTURES & METHODS SPECIFIED FOR CONTROL OF GROUNDWATER

(Continued)

TUNNEL LINING

Primary:

A Steel ribs

B Steel ribs and lagging (as req'd)

C Steel liner plates

D Shotcrete

E Rock bolt reinforcement

Secondary:

F Cast-in-place lining, structural

WATERPROOFING

G Consolidation grouting of rock (as req'd)

H Contact grouting of concrete lining

I Porous subgrade

J Polyethelene and waterproof paper

K Coal tar epoxy on liner plates

JOINT TREATMENT

L PVC waterstop

M Rubber waterstop

N Steel waterstop

O Copper waterstop

P Bituminous filler

Q Paraplastic filler

R Sponge rubber and mastic filler

S Caulking (as approved)

T Bolts with washers and plastic grommets

8.00 MAINTENANCE REQUIREMENTS DURING THE LIFE OF A TUNNEL

The previous sections of this report have described methods of preventing water inflow into a wide range and variety of tunnels. The main reasons for preventing water inflow were discussed in Section 1. An important reason for keeping tunnels and underground structures dry is to reduce the cost of repairs due to possible deterioration of the structure and its contents over the life of the structure.

It has been pointed out how each method of waterproofing or groundwater control can be used, the conditions in which its use is applicable, the care needed in application for the method to be effective, and the limitations of its use. There has been some discussion of its cost relative to other possible methods or different materials for the same construction conditions, and this is also covered in Volume 3.

As ideal applications are rarely accomplished, possible failure or deterioration of the chosen water-protecting system must be considered. As an example a WMATA system of drains to prevent buildup of water pressure around transit tunnels is failing due to precipitates clogging drainage lines. Means for restoring the integrity of the water-proofing when failure occurs must be planned. The ease or difficulty with which the maintenance can be accomplished and the cost of performing maintenance may be major factors in the selection of the waterproofing means to be used.

Major changes in the use of the structure or changes in the loads to which the structure is subjected because of ground movements, added or subtracted surface-structure loads, or changes in water table elevation might cause failure of a properly applied waterproofing.

This section will present means of maintaining the watertightness of the structure during its useful service life.

8.10 DESIGN AND CONSTRUCTION PROCEDURES TO REDUCE MAINTENANCE REQUIREMENTS

8.11 Backup Waterproofing Method

Any of the waterproofing systems discussed in this report could be successfully used alone if appropriate for the underground conditions and construction methods, and if ideally applied. "Ideal" application is usually not achieved because working conditions underground are not ideal, materials do not always meet specifications, and supervision of the work is less than perfect. Therefore, for a completely dry tunnel, a backup waterproofing system may be considered in the design. This is particularly advisable if the structure will be at considerable depth, in a highly pervious aquifer, or covered with surface structures. One writer suggests that if the water table is higher than the tunnel axis more protection than just good concrete is needed (Ref. 125).

Using two barriers to groundwater infiltration increases the cost and the time of construction. Rigid specifications and capable supervision also increase cost and construction time. Therefore the consideration of less stringent construction controls with a backup waterproofing system added should be examined.

8.12 Cast-in-Place Concrete

Procedures for constructing water-resistant concrete are discussed in Section 2. Only the highlights will be repeated here.

Structural Design for Watertightness. The structural design must be based on all loads to which the structure will be subjected. Where bentonite waterproofing is used the pressure caused by its expansion must be considered. The loads to be included are ground and surcharge, hydrostatic, and, for the invert slab, the hydrostatic uplift and the possible span load if some of the soft ground under the structure shifts.

Concrete Design. The concrete mix must be designed for low porosity, low shrinkage, and good resistance to the chemicals in the ground and to those to be expected from the intended use. Only high quality ingredients should be specified.

Reinforcing Steel. Corrosion of reinforcing steel and other embedded items must be kept at a minimum. Intermediate grade steel should be used with a cross-sectional area greater than the theoretical design area. Increase in stress increases corrosion when conditions are favorable for corrosion. To permit good consolidation of concrete outside the steel and between adjacent bars, maximum aggregate size should not exceed 1/3 the space. The steel should be protected by adequate cover from stray currents, especially direct current. In certain conditions cathodic protection may be required.

Joints. Construction joints should be made tight by roughening the hardened concrete, cleaning it, moistening it, and placing a cement grout layer before placing the next pour. To back up this construction procedure either waterstops or bentonite backing should be used on the joints.

Contraction joints should have waterstops with bentonite in the exterior chase and caulking in the interior chase.

Expansion joints should have highly resilient premolded joint filler and flexible waterstops.

Mixing and Transporting. For watertightness, the specifications should call for thorough mixing and chilling of ingredients when needed. Transportation should be done in such a way that the large aggregate is not segregated.

Placing. The specifications should be specific about placement of the concrete.

a. Preparation requirements for placement:
   1. Flowing water has been diverted from placement area.
   2. Reinforcing steel clearances are as designed.
   3. Debris and standing water have been removed from behind the forms where the concrete is to be placed.
   4. The forms are cleaned and oiled.
   5. Waterstops are adequately supported.
   6. Hardened concrete to receive new concrete is properly prepared.

b. Placing requirements:
   1. Consolidate around waterstops.
   2. Prevent segregation of large aggregate.
   3. Deposit as near to final position as possible.
   4. Place in lifts the vibrators can reach through.
   5. Vibrate for thorough consolidation.
   6. Consolidate carefully along joints and around reinforcing steel and other embedded items.
   7. Place the whole pour without interruption.

Continuous Placement. Tunnels, in which the permanent lining consists of continuously-poured concrete, will probably not be completely watertight. Even with the use of expansive cements the lengths of continuous pours are sufficiently long that shrinkage cracks are probable. In addition, the placement of concrete within a tunnel is always difficult, often resulting in concrete of inferior quality. The design should include another means of restraining the groundwater if the advantages of continuous lining outweigh its disadvantages. If no other means of waterproofing is included in the design, remedial measures will have to be taken if the tunnel is to be dry.

8.13 Applied Waterproofing Envelopes

The design drawings and specifications should require the following:

1. Membrane comprising the envelopes must be "impermeable" at the anticipated head of water.

2. Membrane must be capable of accommodating small movements at cracks.

3. Membrane must be capable of resisting the damaging effects of the backfilling operation or it must be given

a protective cover, if it will be subjected to backfill abrasion and punching loads.

4. The application instructions of the manufacturer must be followed strictly.

5. Most applied membranes must be installed by experienced workmen under capable supervision.

6. For sheet materials there must be adequate lapping and the laps must be completely sealed.

7. Liquid applied membranes must not be less than the specified thickness and must be free of pinholes.

8. Penetrations through the membrane must be given special attention.

9. Application-surface irregularities and cracks should be repaired so that the membrane may be placed on a smooth surface which will not tear the sheet material or make uniform coverage by liquid applied materials difficult. If this cannot be done, as in a rock excavation, a cushioning material must be placed before a sheet membrane is installed.

10. If bentonite is used it must be kept completely dry until it is encased between the structure and the ground.

8.14 Segmented Tunnel Linings

Cast Iron, Cast Steel, and Ductile Iron Segmented Linings. To ensure a dry tunnel with minimum maintenance, in tunnels built using cast iron, cast steel, or ductile iron segmented linings, special design attention should be given to the following items.

1. Close fit of mating flanges.

2. Grommets of a material which will not be affected by ground chemistry or chemicals produced by the use to which the tunnel is put. It should also be capable of the deformation required to fill the space between the bolt and the bolt hole without a tendency to flow and it should not be adversely affected by vibration.

3. Specific requirements for the tightening of the bolts.

4. Good directions for placing the caulk.

5. Specifications for handling procedures for the brittle castings.

Fabricated Steel Segmented Linings. For tunnels using a fabricated steel segmented lining special design consideration should be given to the first four items above as needed for cast iron segmented linings. In addition, a well-designed cathodic protection system is required and suitable coatings for the exterior and interior faces, to help prevent any attack on the steel from ground chemistry and chemicals produced by the use to which the tunnel is put if no secondary concrete lining is used.

Precast Conrete Segmented Linings. For tunnels using a precast concrete segmented lining the following items should be given special consideration in the design stage to ensure a dry tunnel with minimum maintenance.

1. Concrete-mix designed for low porosity as well as strength.

2. Design and construction of casting forms and specification of casting procedures to insure close dimensional tolerances.

3. Water curing required after steam curing to ensure low permeability.

4. Gasket material selected for long life in the ground chemistry environment and for good resiliency over the entire anticipated life.

5. Grommet material specified as for metal segmented linings.

6. Caulking materials chosen for good performance in the interior tunnel environment.

7. Definitive specifications for tightening the bolts.

8. Requirements for handling the brittle segments.

9. Means of preventing damage to the gaskets during installation.

Segmented Tunnel Linings in Rock. For rock tunnels using a segmented lining, items 1, 2, 5 and 7 under the above paragraph should be given special consideration in the design stage. In addition the following items require attention.

1. Backpacking must be well placed with the distribution of the gravel controlled as well as present methods permit. This is a primary requisite for the equal load distributions on the lining.

2. Well-grouted gravel backpacking will help in keeping the tunnel dry.

3. Material for joint filler should be selected for long life in the local ground environment and for good resiliency during its lifetime.

4. Design of mating edges of the segments should be able to accommodate movements caused by differences of transverse and longitudinal loading.

8.15 Other Waterproofing Methods

Chemical Grouting. In chemical grouting the important design procedures start with a thorough subsurface investigation along the tunnel route. This investigation should include the determination of types of ground formations that will be encountered with their in-situ porosities, permeabilities, pore-size distributions, temperatures, grain-size gradations, ground and groundwater chemistries including pH's, and groundwater flow characteristics.

Then the grouting expert can decide what grouting systems are applicable, including the type of grout, whether particulate or chemical .

The next step in the procedure should be the testing of these systems in relation to the site conditions anticipated. From this a tentative selection is made and grouting can be started.

A complete investigation of the entire length of the route is often impractical before excavation starts. Therefore, as the tunnel is advanced it may be necessary to drill pilot holes ahead of the face to determine more precisely the nature of the ground. Since sudden changes in the ground formations occur, the grouting program must be kept flexible and amenable to sudden changes.

Consolidation Grouting of Rock Tunnels. The grouting of fissures exposed during excavation in tunnels driven through rock is not usually designed ahead of excavation as is soft ground injection grouting. The grouting program is usually planned by the contractor subject to approval by the engineer. During initial subsurface investigation the knowledge that fissured rock will be encountered may be ascertained. With this knowledge, contractor and engineer will be prepared to deal with it when it is exposed.

Planning an extensive grouting program starts with determination of void size, chemistry of ground and groundwater, and water flow characteristics. The sizes of the voids control the maximum particle size of the grout. The chemical properties of the ground and groundwater may influence the selection of grout ingredients which must be able to resist any aggressive chemicals. Water flow may control the required setting time for the grout which in turn may be the design criterion for adding a chemical grout to a particulate grout. The design of the minimum pressure to be used in grout placement should be based on the pressure of the water to be displaced by the grout. The maximum pressure should be determined based on the characteristics and condition of the rock mass.

If high water inflows ahead of the tunneling face are suspected, the design should call for exploratory drilling ahead of the face and if excessive water is found a grouting pattern should be designed to grout sufficiently outside the tunnel excavation line to cut off the water flow. Essentially the same criteria are used to develop this pregrouting system as the grouting of exposed fissures.

The consolidation grouting program must be kept flexible so that it can be changed to suit changing rock characteristics. Sometimes it is necessary to start with a plan and then add more holes, longer holes, different grouts, etc., until the right grouting coverage is achieved.

<u>Annular Space Grouting.</u> Linings erected in the protection of the tail of the tunneling shield are surrounded by an annular void approximately two to three inches (50 to 75 mm) wide. To keep the ground from settling into this space it must be filled. This is done by blowing pea gravel, until refusal, into the space soon after the shield is moved forward. To consolidate the gravel and to complete the filling of the void, grout is injected, until refusal, into the gravel in the annular space. It is not possible to place the gravel uniformly in the space and blowing the gravel into the annular void loosens some of the surrounding ground mixing it with the gravel. In other grouting operations it is customary to ascertain the characteristics of the material to be grouted before designing the grouting system to be used. In annular space grouting the characteristics of the gravel and ground mixture may vary significantly within very short distances. Therefore, the grout-design-related characteristics of the material to be grouted are unknown. This makes normal grout-design procedures inoperable and experience governs design.

In order to have reasonable control over the void-filling operations, the specifications should call for monitoring of the volume of muck removed during tunneling, the volume of gravel placed, and the volume of grout injected. This monitoring need not attempt to be exact. If the volume of muck exceeds the anticipated volume then volume the of the void around the lining is greater than the theorectical volume. Ground, not intended to be excavated, has fallen from the arch and/or sidewalls of the tunnel. By comparing the volume of gravel and grout placed, allowing for the voids in the gravel before placement, with the theorectical volume of the annular space adjusted for excess muck (in turn adjusted for swell due to excavation) it can be determined approximately how successful the operation has been. If there is considerable discrepancy between these volumes, settlement is likely to occur. A program of monitoring surface elevations should also be specified.

Appropriate grouting pressures can and must be selected based on the actual conditions; they must be high enough to overcome the water pressure, but not high enough to overstress the lining.

Contact Grouting of Concrete Linings. Whether in soft ground or rock, cast-in-place concrete linings, by the nature of their placement and the nature of the surface of the excavated tunnel, have irregular volumes of voids around them. A cement grout, usually incorporating a filler such as sand, is injected into the voids to form a solid contact between ground and lining. If properly done the operation aids in controlling groundwater flow into the tunnel. The grout mixture is customarily chosen by the designer to fit the anticipated voids. This choice is usually based on the experience of the contractor and engineer with similar tunnels.

As with annular space grouting, appropriate grouting pressures can and must be selected based on the actual conditions; they must be high enough to overcome the water pressure but not high enough to overstress the lining.

Slurry Walls.

a. Cast-in-Place Concrete Slurry Walls. If slurry walls are to be watertight the design of the concrete mix must be done from a different approach than that used for conventional structural concrete and must be based on a full knowledge of slurry wall construction.

The fresh concrete must flow well so that it will move from the tremie pipe outward along the trench and meet the ends of the panel or the concrete from the adjacent tremie pipes. If flow is slow, mud may be trapped within the concrete. The entrapped material is essentially impervious, but it weakens the panel. Under loading this may permit cracking which, in turn, would permit water inflow. Another problem produced by slow flow is the development of cavities at the ends of panels because the concrete did not flow to the ends. This obviously increases the wall's permeability.

With "flowable" concrete, segregation and bleeding are common defects. Such concrete may block the tremie pipe or mix with the bentonite. To eliminate these problems the concrete mix should be designed to incorporate more fines than usually needed. Experienced workers can usually determine visually, before placement, if concrete will segregate or bleed, so the specifications should require personnel experienced in slurry wall construction to supervise placement.

The specifications should require placing of the entire panel without interruption. Cold joints and contaminated concrete which may leak, can be caused by interruption during concreting or raising the end of the tremie pipe too close to the concrete-slurry interface. Since the placement of the entire panel must be done without interruption, another mix-design criterion which is different from good structural concrete design is that setting time should be prolonged to ensure the first concrete poured will still be plastic when the panel is completed. Some of the first pour may be at the top surface of the concrete and must move upward as a plastic material and in any event the first pour will still be moving within the mass even if it does not reach the surface.

Some adjustments are advised to make up for the less-than-perfect construction conditions. For example, it has been recommended that the allowable bond stress be reduced by 20% and splice lengths be approximately doubled (Ref. 149). This is to compensate for the possibility of slurry clinging to the reinforcing steel and allowing slippage of the concrete along the bars, producing shrinkage cracks. A reduction in allowable compressive stress is also suggested. This is primarily for strength reasons, but anything that permits weakness in the concrete is likely to produce leakage through cracking. To accommodate the excavation equipment, slurry walls are usually thicker than cut-and-cover walls would be at a given site. Their thickness may also be increased by the sloughing off of the soil into the slurry during excavation. With thicker wall sections, less strength is needed in the concrete for the same loading so the increase in width must be sufficient to compensate for reduction in allowable stress.

The reinforcing bars may interfere with the upward flow of the concrete so the reinforcing cage should be designed for smooth flow of the concrete. Interference of flow by the bars can cause pockets and other defects which reduce the strength of the concrete and may cause porosity of the wall. Therefore, the bars should be spaced as widely as possible and laps should be kept to a minimum. In addition, the clearances at the faces should be increased since the concrete near the ground surfaces may mix with the bentonite slurry, reducing its protective ability.

The construction joints in a cast-in-place, continuous slurry wall are difficult to design so that they will be watertight. The early methods used did not permit reinforcing steel to pass through the joint giving horizontal continuity. Therefore, these methods can only be used when the vertical steel is the principal reinforcement. The design of the joint must be based on the loads to be resisted at the joint and the watertightness required of the joint. There are a number of joint designs to guide the engineer including special provisions to transfer loads from one panel to the next and to put a water sealing section, such as a waterstop, across the joint. In all cases, however, the slurry has wetted the concrete surface and a coating may be left that allows water to leak through. One advantage that mitigates this is that the bentonite mud cake on the unexcavated face of the wall will tend to fill and seal the leakage path if it is not too wide.

b. Soldier Pile and Cast-in-Place Tremie Concrete Walls. To seal the soldier piles to the cast-in-place concrete is a construction problem rather than a design procedure. Specifications should require that after excavation of the slot the soldier pile must be cleaned as thoroughly as possible within the slurry. Fortunately the bentonite behind the wall will seep through and may seal small cracks that remain.

c. Precast Concrete Segment Walls. Control of concrete in a casting yard is far superior to that tremied into a slurry filled slot. This is one of the reasons the use of precast concrete segment walls has come into practice. The design considerations for casting watertight concrete panels are essentially the same as for precast concrete segmented linings.

The precast concrete panels themselves can be considered impermeable if properly designed and constructed so the joints are the only groundwater control problem for field control. To make the joints watertight the slurry mix can be designed so that after a predetermined period of time the slurry will self-harden into an impervious grout behind the panels and through the joints in a continuous mass. The contractor must decide if it is preferable to use the altered slurry for excavation or to use the normal bentonite slurry for excavation and displace it with a heavier, self-hardening fluid just before placement of the segments.

If the project conditions require continuity of horizontal reinforcing steel, the design can be facilitated by a study of the many panel designs, in use or suggested, that are intended to transfer loads from panel to panel. Some have special means for making watertight joints.

8.16 Sunken Tube Tunnels.

The environment of a sunken tube tunnel imposes a more severe burden on the tunnel's waterproofing systems than does that of a cut-and-cover or driven tunnel. The tunnel sections, waterproofing, and bulkheads control the groundwater while the sections are lowered into place, but sealing the sections to one another must be done under water which poses entirely different problems from those confronting the designer of a cut-and-cover or driven tunnel. Many of the concerns of designers of tunnels using different types of construction are the same though their solutions may be different. The major design concerns for watertightness of sunken tube tunnels include:

1. Impermeability of walls.

2. Corrosion protection for the steel shell, if used.

3. Watertightness of joints between sections.

4. Watertightness of expansion joints, if used.

For the structural steel shells, the designer must specify a testing method to ensure that all welded joints have complete watertight welds before concreting. The steel itself is impervious, but a means of protecting it from corrosion must be designed. This is usually done by surrounding it with tremie concrete or by using a cathodic (electrical current) protection. This should ensure impermeability of the walls for the life of the tunnel.

The design for the waterproofing of the joints between successive structural steel shells may include welding of a U-plate cofferdam across the joint on the exterior as a form for tremie concrete, and the welding of a steel plate across the inside of the joint to make the steel continuous with the shell. More recent designs use a heavy rubber gasket as first seal with a steel plate welded across the inside of the joint as above.

Reinforced concrete as the structural element of the tube sections is favored in Europe. The waterproofing design considerations for these tubes are somewhat more like cut-and-cover structures. The designer may choose good, watertight concrete if the rigid controls and special techniques necessary to produce it are available. Steel can be used as exterior forms and left in place as a waterproofing membrane or the exterior can be coated with a waterproofing material such as a multi-ply asphalt membrane.

Since sections are long it has been found to be advisible to include expansion joints at intervals. Construction joint design is especially important if the concrete is to be impervious. This presents the same problems as in cast-in-place concrete liners in cut-and-cover construction, and requires much the same design and construction techniques. Present design uses waterstops within the concrete. The outer face should be sealed as a backup measure, using polyurethane putty or a multi-ply membrane, since the placement of concrete around a waterstop is difficult, frequently producing gravel pockets. The designer may choose to use a new method -- to inject an epoxy sealant into the waterstop location through preset pipes in order to fill the voids in the gravel and consolidate the material. The volume treated becomes impervious and the exterior sealant may be eliminated.

The joints between sections are usually designed to be sealed using large gaskets which provide the first seal while the space between bulkheads is dewatered, and a secondary continuous rubber seal which is placed inside the tunnel to complete the sealing.

8.20 NORMAL MAINTENANCE PROGRAM

The heart of the normal maintenance program for the watertightness of a tunnel is efficient monitoring procedures based on regular inspection. If no program is established, workers inspecting or maintaining track, road bed, signals, ventilation system, lighting, or other tunnel functions will probably not report water until it inconveniences them in their work. Excessive water flow in covered drains may not be noticed.

8.21 Preventive Maintenance

If the tunnel must stay dry, maintenance measures must be used to prevent leakage.

Preventive maintenance is particularly valuable and necessary in bolted segmented linings. Bolts are not corrosion-resistant except in unusual circumstances. They are expected to corrode and replacement will be necessary. The routine inspection reports should include comments on the conditions of the bolts, nuts, washers, and grommets. Corrosion of the metal, deterioration of grommets, and loosening of bolts due to vibration should be observed and reported. This should be done by spot check with special comment on those items near areas where leakage is occurring. Some areas may show no deterioration so fewer checks would be made in these areas, and areas showing any corrosion would be more closely monitored. Then, at a convenient time, a work crew could replace the bolts, nuts, washers and grommets in the deteriorating areas and reseat the caulking if necessary, preventing water infiltration. If only an occasional bolt is corroding in an area, it may be just as well to wait to replace it when other repair becomes necessary.

Caulking in the joints may also be monitored in a similar way.

If monitoring is not done, a regular maintenance crew should periodically patrol the tunnel correcting sealant deficiencies as they are noticed.

Another item for preventive maintenance inspection is the interior coating on fabricated steel segments if exposed. If this coating deteriorates, chips, or flakes, the corrosion prone steel is exposed to the corrosive environment of the tunnel. In a structure which lasts a great many years, corrosion of the steel plate could reach such proportions as to reduce the strength of the plate past the point of failure. Re-coating need not be done frequently, but when large areas show damage to the interior protective coating, the coating should be replaced.

8.22 Monitoring Leakage

First it must be decided how dry the tunnel must be kept. In general, it is impractical to repair every small moisture intrusion or minor dripping when first visible, since materials and equipment must be gathered, a crew sent out to do the repairs, and the use of the tunnel restricted until the repairs are completed. This is an expensive procedure and an inconvenience to the users of the facility. It is more economical to wait until a number of small leaks exist (maximim allowable flow) or until a major leak (more than maximum allowable flow) is noted. Frequently, small leaks heal by solidification, in the water's flow path, of material carried by the water seeping through.

Tunnels are customarily inspected routinely for numerous hazards, signs of structural or architectural deterioration, oil slick, malfunctions, etc. During this routine inspection, any sign of water leakage should be noted on the inspection report sheet with the approximate amount of flow and its location. Of course, if the flow is large, immediate notification of the maintenance manager should be made. Also noted in the report should be the absence or presence of soil in the water.

The amount of flow in litres/day or gallons per hour is difficult to estimate without considerable experience. The inspectors who would observe leakage while performing their other inspecting duties may not be trained for that sort of determination. For this reason, a scale of descriptive leakage should be established to give a certain uniformity to the reports of all inspectors. This could be similar to that prepared by CIRIA and quoted in Section 1 of this volume of the report (Ref. 25). These should be keyed to common experience. A suggested list is given.

1. Visible shiny water film, possibly dripping slightly, affecting an area less than 1 m^2

2. Constant seepage from a narrow crack

3. Sheet flow from area less than 1 m^2

4. Sheet flow from area more than 1 m^2

5. Flow more than faucet leak but less than garden hose flow

6. Crack or joint flow equivalent to moderate garden hose flow or more - notify maintenance manager immediately!

The maintenance manager should be mindful of leaks that increase in volume since this indicates increasing deterioration of the waterproofing system and the possibility of immenent structural damage. He should also be watching for reports of soil present in the flow. This indicates loss of ground above the tunnel with a possibility of settlement of utilities or structures above the tunnel. Loss of ground also increases the permeability of the ground mass and decreases its strength. This decrease in strength may result in a shift in load distribution on the tunnel lining producing stresses unanticipated at the time of design. A decrease in volume of flow would indicate that the groundwater was depositing soil particles or precipitated chemicals in the cracks and pores and the integrity of the watertightness might eventually be restored.

The maintenance manager should compare consecutive reports as they are received and if an individual leak is becoming significantly larger, a new large leak appears, or if leaks in one area are getting more numerous or have remained about the same size for a long period of time, maintenance procedures should be started and all significant leaks and leaks of long duration should be repaired. Of course, if a large leak is reported that needs immediate attention, consideration should be given to repairing all other leaks within the section of the tunnel shut down for leak repairs. If other tunnel repair work causes shut-down this is a good time to repair leaks within the area.

Another means of determining the presence of significant leakage into a tunnel is to maintain a record of the output of the sump pumps. If in dry weather output is significant, it is obvious that water is flowing in the area of the tunnel that feeds the pump. If no washing or other water-producing work is being done in that area, it must be checked for leaks. This program is likely to flag only large individual leaks or numerous smaller ones. The drainage system, including the sump pumps, has been designed for anticipated maximum flows from water entering through stations and tunnel entrances during inclement weather. If pump output exceeds this appreciably the tunnel should be checked for leakage.

8.23 Repair Methods

Once it has been decided that the time has come to stem the flow of water, the repair method chosen will depend on the waterproofing method originally used, the type of failure of that waterproofing, and the type of ground surounding the tunnel at the location of the leak.

One repair method that is useful in many situations is the injection of particulate or chemical grout into the ground surrounding the tunnel. The characteristics of the soil determine the most useful type of grout, as discussed in Section 5. Therefore an investigation of the ground mass should be made as described in sub-section 5.10. A grouting system is then chosen, holes are drilled through the leaking areas into the ground mass, and grout is injected until the leaking stops. Usually there will be enough information from the tunnel construction phase that a group of grouting systems for routine maintenance can be established for the major formations surrounding the tunnel. These would be used unless leakage is catastrophic or cannot be controlled by the specified system. Then it would be necessary to resort to the assistance of an expert and the soil investigation.

Repair of leaking, water-resistant-concrete linings is accomplished in much the same manner as repairs of defects at the time of construction (sub-section 2.80). An alternate means of stopping the flow of groundwater through the concrete is injection of particulate or chemical grout behind the concrete in the area of the flow plus a saftey zone behind it.

The applied waterproofing membranes that are in sheet form cannot normally be replaced or repaired under wet conditions. For these materials, it is generally best to inject grout behind the waterproofing if soil conditions permit. Unfortunately, the location of the leak in the membrane may be some distance from the appearance of water within the tunnel. The water may travel long distances between the waterproofing membrane and the structural or inner tunnel lining before leaking through a porous place in the inner lining. Therefore, several injections may be needed before the leakage is eliminated.

The maintenance of bentonite envelope waterproofing is usually not difficult. Small leaks heal themselves by the movement of the bentonite gel into the cracks through which the water flows. For major leaks, bentonite slurry is injected through a hole drilled at the location of the leak. Then the drilled hole can be sealed with a hydraulic cement plug and the tunnel will again be leak-free.

One of the advantages of cementitious waterproof coatings is that leaks, if they occur, are localized right where they come through. They can be repaired easily by a reapplication of the waterproofing coating on the cleaned and roughened existing surface even though wet.

Where chemical or particulate grouting of the ground mass has been used as the primary groundwater control method, regrouting will usually suffice to stop the leak. It may be found, during examination of the surrounding ground, that a different grout should be used than was used originally.

If the means of controlling groundwater is permanent lowering of the groundwater table by drainage, and leakage occurs, the drainage system must be checked and if flow is lower than its capacity, any clogging of the system must be cleaned out. If the flow is near full capacity, a source of unanticipated water supply must be sought and controlled by some other means.

In a tunnel with a bolted segmented lining, if the bolts corrode, they will loosen and leakage will occur at the seams. The bolts may also be loosened by the heavy vibrations associated with transportation tunnels. This is expected and is easily corrected if the interior of the lining is exposed. Grommets, washers, and nuts associated with the bolts would be replaced at the time the nuts are tightened or replaced. If the interior is covered with concrete, the concrete will probably prevent this corrosion.

If the interior caulking between segments loosens, all that is required is that it be reinstalled. Since this caulking is most frequently lead, a small air hammer does the job. Some additional lead may be required to make the joint completely watertight again.

In sunken tubes, the section joints are the most probable locations for leaks. These are usually inaccessible since the exterior is usually covered by a clay or concrete blanket and the interior by concrete lining or architectural finish, and the grouting of the blanket is the most usable means of restoring watertightness.

8.24 Unusual Maintenance

Unusual maintenance is required when neglect has caused major deterioration of the tunnel, differential settlement due to tectonic or other ground forces has occurred, the lining has been damaged by a major fire in the tunnel, or there has been some other occurrence which jeopardizes the integrity of the tunnel.

When a fire occurs in a tunnel, the lining is often damaged. The fire may produce cracks or porosity in the tunnel lining material; warp or crack metal segmented linings, even opening the seams; destroy the waterproofing ability of an applied waterproofing membrane; or, in some other way, destroy the integrity of the waterproofing over an extensive area without a collapse of the tunnel. The extent of the damage will determine the necessary remedial work. Portions of the tunnel linings that have been damaged must be replaced. If the integrity of the waterproofing has been breached it would probably be difficult or impractical to replace. Extensive grouting of the soil outside may be necessary.

If deterioration of the tunnel lining has progressed until there is such strength loss that the lining fails structurally or if earth movements cause major cracks or breaks in the lining, water inflow will be the maximum which can flow through the exposed ground formation. "Maintenance" then becomes major structural repair. To accomplish this work, the tunnel must be dewatered. This can frequently be done using the same methods as were used during the construction period.

If the means of preventing water infiltration during construction was compressed air, the damaged section can be cut off from the rest of the tunnel with temporary bulkheads and pressurized until the restoration work has been completed. If slurry walls were used as water control it is sometimes possible to box the area around the damaged tunnel with slurry walls or slurry cutoff walls, dewater, replace the original slurry wall with formed concrete, and replace the interior tunnel finish. Sheet piles, especially if grouted, can be used in lieu of slurry wall. If the tunnel had been built by cut-and-cover methods it may be possible to reach the affected area from ground level. Local ground freezing may be employed under appropriate conditions even though it may not have been used in the original construction.

A versatile tool for stopping the flow of groundwater into a damaged tunnel is quick-setting injection grout.

Construction procedures for rebuilding the damaged section would be similar to the construction procedures used for the original tunnel.

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

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This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

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This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

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