WATER INTRUSION IN UNDERGROUND STRUCTURES

by

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ABSTRACT

This thesis presents a study of the permissible groundwater infiltration rates in underground structures, the consequences of this leakage and the effectiveness of mitigation measures. Design guides and codes do not restrict, address or make clear recommendations for permissible inflows in underground space. Owners, with the help of engineers, typically make decisions based on costs or specifications from past projects without looking at consequences of excessive groundwater infiltration and mitigation costs.

The Author has reviewed the published leakage rates for tunnels in comparison with current international standards. After examining over one-hundred case studies, the Author infers that water leakage is the principal damage causing degradation on tunnel linings. International standards for permissible leakage rates (transit tunnels) are consistent with class A definitions of CIRIA (1979) and are approximately 0.1-2 gpm/100,000 SF (0.05-1.2 L/day/SM). The most common cause of leakage (based on numerous case studies) in cast-in-place lining is due to cracks that develop from shrinkage of concrete during curing and to the inability of the structure to accommodate movements due to thermal changes. Individual sources of leakage may be allowable within the permissible rates, however can cause damage to tunnel structure and to the surrounding environment (consolidation and differential settlement). Spalling is one of most common structural damages due to groundwater infiltration. The presence of water can cause unpleasant stains, resulting in erosion and corrosion over time. Formation of icicles, ice and water ponding can affect public safety in a tunnel and jeopardize operations.

To mitigate leakage in underground structures and tunnels one may control and/or eliminate the inflow. Chemical grouting is one of the most common measures. However, its application has been unsuccessful in 43% of cases reported by ITA-AITES (2001). Inappropriate material selection for each particular application is major contributing factor for the lack of success.

The Author focused this thesis on highway and rail tunnels, and established recommended permissible leakage rates for such underground structures based on international standards and experiences. These recommended rates can serve as guidelines for future tunnel design specifications or to compare recorded inflow rates with international standards.

Thesis Supervisor: **Andrew J. Whittle**
Title: **Professor of Civil and Environmental Engineering**

To Mikhail and Svetlana,

my Parents

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BIOGRAPHY

The Author, Alex Nazarchuk, P.E., is a licensed civil engineering in California. He worked in construction management, in Santa Monica, California, prior to attending MIT. He conducted field engineering, project and construction management on numerous projects.

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He was born in 1983 in Kiev, Ukraine and moved to Los Angles, California in 1995.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

The purpose of this thesis is to determine a range of permissible groundwater infiltration rates in underground structures, identifying the consequences of excessive leakage and reviewing potential mitigation measures. Chapter 2 presents the background on methods of waterproofing in relation to construction. Design guides and codes do not restrict, address or make clear recommendations for permissible inflows in underground space. Owners, with the help of engineers, typically make decisions based on costs or specifications from past projects without looking at consequences of excessive groundwater infiltration and mitigation costs.

Chapter 3 examines allowable leakage rates that were established for specific projects (Gould 1989, Deutche Bahn, MWATA 2002, etc.) and reviews the recommendations from design guides (CIRIA 1979, Haack 1991, AFTES 1989, and FHWA 2005). It also presents a review of published leakage performance in comparison with permissible rates.

In an underground structure with numerous leaks, the measured infiltration rate has to be evaluated with respect to broad international guidelines, permissible rates established by specific project designs, and potential consequences. In Boston, the local media have reported extensively on the excessive leakage for the recently completed Central Artery/Tunnel (CA/T) tunnels. The Author has reviewed the published leakage rates for these tunnels and in comparison with current international standards.

Chapter 4 considers the consequences of tunnel leakage and methods for mitigating leakage. If mitigation measures are needed, but not implemented, potential consequences of excessive infiltration are identified.

Appendix A gives a guide to the most commonly used terminology for tunnel construction and organizations.

2. HISTORY

2.1. Tunneling and Underground Space

There is a long history of underground construction for mining and military applications. Much more intensive use of underground space dates to the 19th century with the momentum of economic development and the emergence of new technologies for underground construction. This led to a dramatic increase in underground space use for transportation (roads, waterways, and railways), and in other fields such as hydroelectric power facilities (Sterling et al. 2001). Recent tunneling megaprojects such as the $15 B Central Artery/Tunnel (CA/T) Project in Boston Massachusetts, $11.5 B Chanel Tunnel and Channel Tunnel Rail Link (CTRL) in London, and the ongoing $16.8 B Second Avenue Subway Line in New York City are the latest examples of underground transportation facilities built to relieve urban congestion.

Underground space offers possibilities for infrastructure development that are difficult (or impossible), environmentally undesirable or less profitable to install above ground. Another fundamental characteristic of underground space lies in the natural protection it offers to the facilities. This protection is simultaneously mechanical, thermal, and acoustic. The containment created by underground structures has the advantage of protecting the surface environment from the risks and/or disturbances inherent in certain types of activities (Sterling et al. 2001). This paper will address the use of underground space for road/highway and rail/subway tunnels.

2.2. Construction of Tunnels

To approach a conceptual design and construction of a tunnel one needs to evaluate the geology. Potential construction options for tunnels are a function of the ground type (Wood 2000). Table 2-1 illustrates the main alternatives for tunneling. Tunnel construction in soft ground and rock are discussed in this thesis. For strong rock tunnels the options are drill-and-blast or advance by a Tunnel Boring Machine (TBM) with a rotating rock cutter face. Weak rock tunneling can be achieved either with a TBM or with open-face construction (NATM) using a shotcrete liner in combination with reinforcing mesh, bolts, dowels, or anchors. Construction means and methods for soft soil tunneling are addressed in section 2.2.1. Inflows of water during the construction phases are addresses in this thesis in section 4.8.

Table 2-1, Options for Tunneling, (Wood 2000)

Ground type	*Excavation*	*Support*
Strong rock	Drill-and-blast or TBM	Nil or rockbolts +
Weak rock	TBM or roadheader	Rockbolts, shotcrete etc.
Squeezing rock	Roadheader	Variety of means of support depending on conditions
Overconsolidated clay	Open-face shielded TBM or roadheader	Segmental lining or shotcrete etc.
Weak clay, silty clay	EPB closed-face machine	Segmental lining
Sands, gravels	Closed-face slurry machine	Segmental lining

2.2.1. Tunneling in Soft Ground

Tunneling is soft ground can be performed with slurry machines, earth pressure balance (EPB), or simply by open-face excavation. The heading and bench method of construction is principally a hand-mining operation used for large diameter tunnels. Mining is the most economical method for short tunnels with diameters less than 15 feet.

Two types of closed-face shield tunnel boring machines are used in soft ground conditions. 1) EPB shields use the excavated soil with additives within a pressurized chamber at the face. The face pressure is controlled by the rate of advance and the speed of the screw conveyor, which is used to remove the soil from the face. 2) Slurry support shields use pressurized bentonite slurry at the cutting face to create a near impermeable layer, which seals the face. This can be used in nearly all soft soil conditions but is best used in more permeable sandy soils. Soil stratum may often have boulders thus a stone crusher may be included in the machine, as shown in Figure 2-1. Practical limits of operating a slurry machine in soft ground depend of face stability. EBP shield method is preferred for finer soils and soft soils, for example deep clay layer below the water table (Sweeney 2006). In EBP tunneling the face is supported with a mud, formed from the excavated soil. The soil enters the excavation chamber through openings in the cutterhead. Water and polymer foam are added to lubricate the excavated blocks of soil. This helps to prevent heat development due to friction with rotation of the cutterhead.

The practical limits for operating a slurry machine in soft ground may be related directly to face stability. Face stability is reduced by the extent of percolation of the slurry into the ground. For example if the soil permeability of granular soil is estimated as d_{10}^2, where d_{10} represents grain size corresponding to the 10% smallest fraction of the soil. Wood (2000) represents the effectiveness of slurry in term of grain size in Figure 2-2 for any particular combination of soil type and ground water pressures. EPB machine will be preferred for soils (finer) with practical limits shown in Figure 2-2.

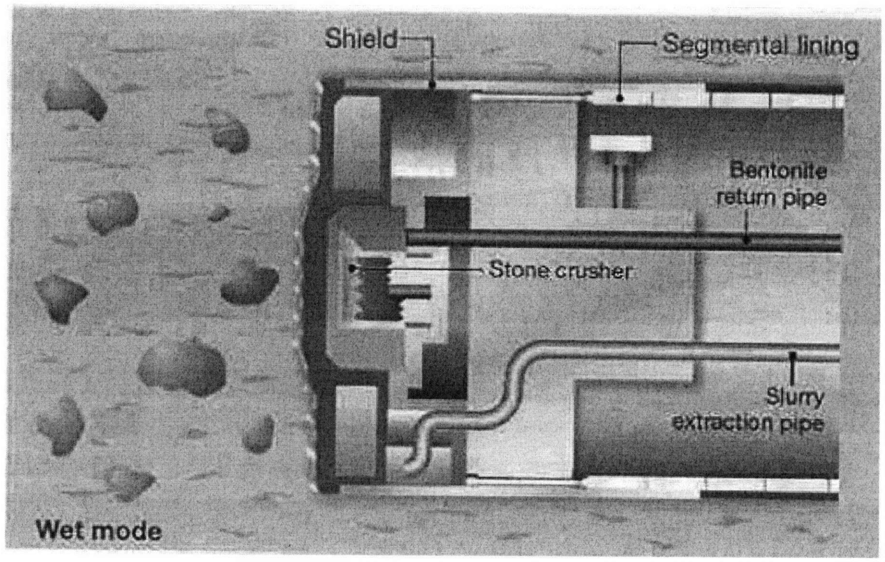


Figure 2-1, Conventional Mixshield mode with slurry face support, (Kolymbas 2005)

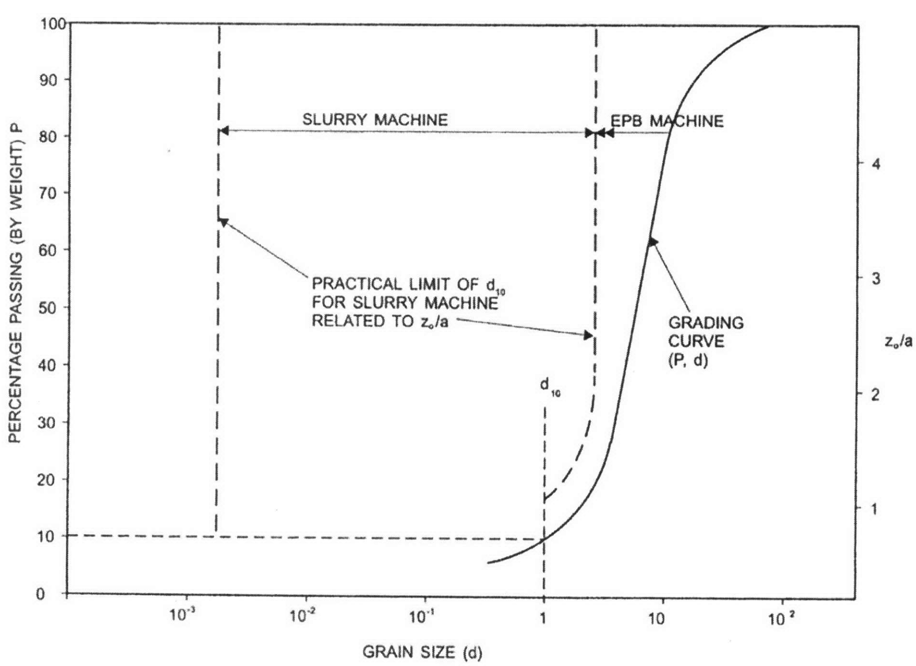


Figure 2-2, Selection between EPB or slurry machine, (Wood 2000)

2.2.2. Cut-and-Cover

Cut-and-cover is a method of construction is shallow tunnels (up to 100-165ft or 30-40m deep) where a trench is excavated, covered with a top slab, and later backfilled. Two construction methods of cut-and-cover tunneling are available: top-down method and traditional bottom-up method. Large cut-and-cover boxes are often used for underground metro stations even if the running tunnels are bored, with machines.

Top-down method consists of earth retaining walls constructed from ground level, using slurry walling, secant bored piles, or other permanent lateral support. A shallow excavation is then made to allow the top stab to be constructed. A top slab is cast, spanning the two walls. The surface is then reinstated except for glory hole, access openings. This allows quick restoration of roadways. Excavation machinery is lowered into the glory holes for mining operations, and the main excavation is carried out under the permanent tunnel roof, followed by construction of the base slab.

In the more traditional bottom-up construction, the excavation is supported by temporary or permanent walls and braced by internal preloaded struts (e.g. cross-lot or rakers) or external supports (e.g. prestressed tie back anchors). The permanent concrete structure is built inside the excavation and support elements are removed during infilling operations.

Shallow tunnels are frequently constructed with cut-and-cover method, while deep tunnels are bored with TBM or mined.

2.2.3. Diaphragm Walls

Numerous cut and cover tunnels are used with the construction of diaphragm walls. The term "diaphragm walls" refers to the final condition when the slurry is replaced by tremied concrete that acts as a structural system either for temporary excavation support or as part of the permanent structure (Konstantakos 2000). The walls provide temporary support for the excavation, but can also be included as part of the permanent structure. For example in the Boston Central Artery/Tunnel (CA/T) Project a variety of diaphragm wall, referred to as a Soldier Pile and Tremmie Concrete (SPTC) walls was used through much of the I-93 reconstruction and formed the permanent side walls for the tunnel boxes (Christian 2007). The height of the slurry walls extends from street level in Boston down as much as 120 ft (37 m) where they are embedded into rock. The strength of the walls derives from heavy, 36 in (91 cm) deep, steel I-beams that are employed as soldier piles. These piles, which are embedded in rock at their base and serve as the support point for the tunnel's floor and roof framing, are the vertical members that carry the weight of the tunnel components down to the foundation. They also are the members that give the slurry wall its bending strength and stiffness to withstand lateral earth and hydrostatic loads. The piles are spaced between four and six feet apart depending on the location. The space between the piles is filled with concrete. The slurry wall thickness is nominally 42" and is intended to provide 3" of concrete cover over the surface of the pile flanges (FHWA 2005). The soldier piles are designed to fully resist the horizontal pressures exerted on the retaining wall. A conservative assumption is that the tremie concrete acts only as a lagging system and doesn't contribute to the bending stiffness of the wall. Most other diaphragm wall use rebar cages for flexural support, instead of soldier piles.

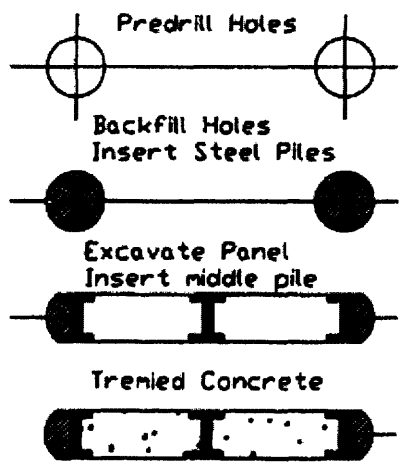


Figure 2-3, Construction Sequence SPTC (Konstantakos 2000)

Watertightness is very important especially since slurry walls are either used as part of the permanent underground basement structures or are selected in part to control groundwater flow during excavation. Significant leakage can occur at joints especially at the soldier pile, while minimal seepage can occur through the concrete. Waterproofing system designs, discussed in section 2.5, are critical for underground structures with slurry walls. Differential movement at panel joints may results in dampness around joints (Konstantakos 2000). Improper cleaning at the bottom of the trenching can result in leaking construction joints. Material remaining at the bottom of the trench is pushed up during concreting. Part of the material may reach the top of the panel where it is later chipped and removed, but some can get entrapped between the panel joints. When exposed, water can easily leak through these contaminated joint sections. (Konstantakos 2000)

2.2.4. TBM- Tunnel Boring Machine

TBMs excavate circular cross sections with a rotating cutterhead equipped with disc cutters. To press the cutterhead against the rock, the TBM is jacked at the tunnel wall by means of extendable outriggers. The support can be installed almost immediately after the excavation. Shotcrete temporary liners may be implemented for support, while permanent liners are discussed in section 2.3. TBM advances in strokes where stops are needed, mainly for the maintenance of the excavation cutting tools. TBMs are often protected against cave-ins by cylindrical steel shields. (Kolymbas 2005)

Tunneling in rock with TBM can be classified with unshielded and shield TBM. Unshielded TBM, where provisions are made between face and tail of the machine to protect laborers and equipment against rock falls but where no continues support is provided by the machine. A shield TBM, supports itself by using weak bentonite or cement grout outside the TBM for the purpose of steering the machine, where ground over-cutting is needed. (Wood 2000)

2.2.5. Tunneling in Rock

The words 'drilling', 'boring' and 'cutting' are used in denoting rock excavation (Kolymbas 2005). The traditional system of advancing rock tunnels has been by drilling and blasting and this method continues to be generally adopted for short tunnels, hard rock tunnels and for tunnels in variable ground. (Wood, 1989) The principle behind blasting in a tunnel is to obtain the greatest excavation length for the minimum explosive charge. The pattern of drill holes is selected to suit the rock and the explosive. Cut holes are arranged towards the centre of the face, usually inclined towards each other in order to remove a cone or wedge. Fractures in rock are created by explosives, causing channels for the groundwater to flow towards the tunnel cross section. Washington Metropolitan Area Transit Authority (WMATA) is experiencing groundwater intrusion in their subway tunnels partially due to these construction methods. Groundwater is recharged due to open rock joints. (Gould 1987)

2.3. Permanent Linings

The typical thickness of a permanent lining is at least 10 in (25 cm). For reinforced and watertight linings a minimum thickness of 14 in (35 cm) with exposition joints spaced 25 to 40 ft (8 to 12 m). Designers may not know all the loads acting on the liners. Apart from the loads exerted by the surrounding ground, the permanent lining is exposed to a series of loads such as: shrinkage, temperature difference. For concrete linings, the following structural design specifications are suggested. (ITA 1988)

(1) The thickness of a lining of cast-in-place concrete may have a lower limit of 10-12 in (25-30 cm). The following lower limits may be recommended:

- 8 in (20 cm), if lining is unreinforced;
- 10 in (25 cm), if lining is reinforced;
- 12 in (30 cm) for watertight concrete.

(2) Reinforcement may be desirable for crack control, even when it is not required for covering inner stresses. On the other hand, reinforcement may cause concrete-placement problems or long-term durability problems due to steel corrosion. Mesh reinforcement in the lining may be used for crack control.

(3) The recommended minimum cover of reinforcement is:

- 1.2 in (3.0 cm) at the outer surface if a waterproof membrane is provided.
- 2 in -2.4 in (5.0 cm-6.0 cm) at the outer surface if it is directly in contact with the ground and groundwater.
- 1.6 in-2 in (4.0 cm-5.0) cm at the inner tunnel surface.
- 2 in (5.0 cm) for the tunnel invert and where water is aggressive.

ITA commented on the temperature effects, stating that tension stresses may be somewhat controlled by working joints and by additional surface reinforcement in concrete exposed to low temperatures. ITA emphasized on the requirements for achieving long-term durability by the absence of aggressive water and the limited use of concrete additives for accelerating the setting (ITA 1988).

An initial lining of shotcrete may be considered to provide stability of the tunnel only when the long-term durability of the shotcrete is preserved. Shotcrete lining is a good temporary measure. Some tunnels implement shotcrete as a permanent lining in the structure, the final lining has a reduced thickness and is identified as singleshell or monocoque lining (Kolymbas 2005). The major setback with monocoque lining is the sealing against hydrostatic forces, groundwater. Shotcrete linings usually become cracked and thus are semi-permeable. A waterproofing system must be implemented as a part of permanent tunnel lining, such systems are discussed in the subsequent sections.

2.4. Water-Resistant Tunnel Linings and Waterproofing

Water-resistance will be defined in this thesis as prevention of a limited passage of water through a use of a membrane, coating or physical properties. In the 1988 Guidelines for the Design of Tunnels report, ITA recommends sealing against water using waterproofing sheets, discussed in 2.5.3, under the following conditions:

- When aggressive water action threatens to damage concrete and steel.
- When the water pressure level is more than 50 ft (15 m) above the crown.
- When there is a possibility of freezing of ingressing water along the tunnel section close to the portals.
- When the inner installations of the tunnel must be protected.

One must select a waterproofing system, defined in Section 2.5, to protect a structure from groundwater infiltration. The protection of an underground structure can be performed using a positive side, exterior, or interior (negative), waterproofing system. Negative systems are typically used in rehabilitation while a positive system is a preferred system. A positive system would protect a tunnel from water inflows while providing a tight waterproof membrane (Russell 2007).

The technology of waterproofing intends to protect tunnels and underground structures against unintended seepage water or moisture, leakage from water basins and chemicals contained in the groundwater. The reliability of waterproofing measures is of prime importance for tunnels which are under permanent hydrostatic pressure, are difficult and inaccessible for repairs, and must be continuously used and operated in support of daily activities of metropolitan residents worldwide.

If a tunnel alignment is selected below a groundwater table, the following requirements must be considered: (Haack and Emig, 2002; Russell, 2007)

- Type of structure and type of construction (bored, cut-and-cover, etc.).
- Durability for the lifetime of the structure (normally 100 years for tunnels).
- Maximum elevation of groundwater, after construction.
- Adequate resistance of the waterproof element to any material in contact with it, for example soil, insitu concrete, and others.
- The ability to accept any state of stress and deformation anticipated in the protected structure (this is clearly related its depth of burial of the structure).
- Insensitivity to temperatures which might occur during construction or under service condition.
- Simple workability of the applied waterproof sheets to prevent leaks, especially at their connections.
- Track record of waterproofing system.
- Shape details of the protected structure should be designed taking into account the special properties of the waterproofing system.
- Long term maintenance and repair work must be feasible and cost efficient.
- The waterproof system must be either a multi-layer one or provide reliability controls.
- A sufficient margin of safety, especially at the joints/seems, must be provided.
- Environmental considerations at the site, including pH, chemicals and hydrocarbons.

The design of a watertight system depends on the information provided by the structural and geotechnical engineers about groundwater levels, thermal stresses, soil loads, settlements, heave and displacements, shrinkage of concrete etc. One must also decide on the sequence of construction before a proper design of a watertight system can be achieved. Preconstruction services of an experienced tunneling contractor are essential to achieve a watertight design with proper construction waterproofing details.

Tunnel linings provide waterproofing against groundwater flow, only if the waterproofing is correctly designed and constructed. Many designers do not address waterproofing design initially; they approach it as and adjunct detail at the last stage of design or during construction. Waterproofing details need to be addressed in the early stages of design. Tunnel movements need to be addressed in waterproofing details and expansion joints are needed.

There is a wide choice of sealing materials for sealing joints between tunnel elements. Engineers will select sealants on the basis of cost and durability to meet particular criteria concerning: (a) capacity to tolerate relative movement between elements, (b) hydraulic pressure, and application to wet surfaces and under pressure (Wood 2000). It is critical to provide waterproofing in tunnel portals, rail shafts, vents, and stations (Russell 2008). Haack and Emig (2002) state that "joints are also unnecessary where concrete linings are built to protect structures in solid rock". This statement cannot be applied to highway tunnels that undergo daily temperature cycles with significant contraction and expansion of structural elements. In these cases, joint sealing is always required.

Some tunneling construction projects implement grouting within the joints, during construction as a first mitigation method once leaks are detected. Additional waterproofing options are (sealants provided in a liquid or plastic state; materials) caulked into the joint space; and preformed gaskets compressed between precast tunnel segments.

2.5. Waterproofing Systems

Waterproofing is a defined in this thesis as a coating or a membrane that prevents the free passage of all water through a medium. A waterproofing system is a component of a water-resistant tunnel.

There are numerous waterproofing systems available in the market place. They are the following categories: liquid system toweled or mopped, panel system sheet membranes, epoxy systems, sprayed systems, or hybrid systems (Russell 2007). These systems can be applied to the exterior of cut-and-cover tunnels, and must fully enclose the cross-section. For example the Marina Central Expressway (MCE) in Singapore, is a planned 10-lane highway tunnel is to be constructed by traditional cut-and-cover methods. Part of the tunnel will be below the sea bed within soft clay strata. In design recommendations for the tunnel, Russell (2007) recommended an exterior (positive) poly-rubber gel waterproofing system, enclosing the entire cross-section.

A tunnel situated above the groundwater table is typically protected against downward percolating water with an umbrella waterproofing, as shown in Figure 2-4. Tunnels below the groundwater table encounter hydrostatic pressures and a more comprehensive waterproofing must be applied.

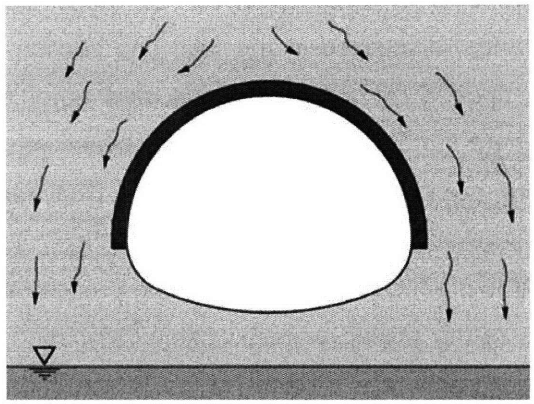


Figure 2-4, Umbrella Waterproofing, (Kolymbas 2005)

Kolymbas (2005) advises that for water pressure of 43.5 psi (3 bar), water-tight concrete may be used and for pressures above 43.5 psi (3 bar), and up to 218 psi (15 bar), watertight membranes should be used in conjunction with the liner. The membrane is typically set between the outer and the inner linings, as shown in Figure 2-4. In rock with pressures higher than 218 psi (15 bar) gaskets linings must be applied.

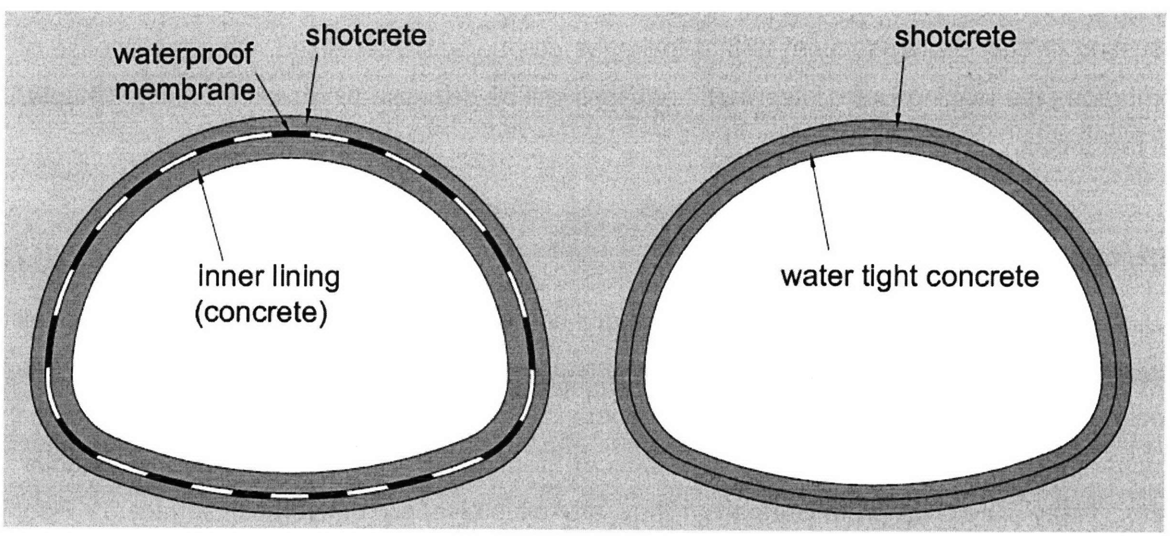


Figure 2-5, Principles of Tunnel Waterproofing, (Kolymbas 2005)

2.5.1. Waterproof/Water-Resistant Concrete

Concrete cannot be entirely waterproof, and this term is often misused in the industry. Concrete mix design can incorporate additives to reduce pore size of the concrete and prevent passage of water through the coatings; however water vapor can still pass. This system is not flexible and needs additional membranes, discussed in the following section, to provide a watertight product. Shrinkage cracking, caused by concrete curing, and other cracking due to thermal activity and differential settlement can provide passages for water intrusion. Fissures may due to tensile stresses, temperature gradients, creep and shrinkage (Kolymbas 2005).

In concrete the hydration heat is produced during the setting process and can cause early-age cracking. Approximately 25% of the mixing water is chemically bonded; the remaining water occupies the pores. Shrinkage cracks are caused by the evaporation of the free water. Low water content, in a given concrete mix design and water reducing admixtures, help keep the porosity small.

In 1988 ITA recommended the use of special specifications of concrete mixtures to achieve watertightness of concrete. ITA stated that the final quality of the concrete, avoidance of shrinkage stresses, and temperature gradients during setting is much more important than theoretical computations of crack widths (ITA 1988).

Kolymbas (2005) states that an advantage of watertight concrete (as compared with membrane, discussed in succeeding section) is that leakages can be easily localized, whereas in case of membranes the leaking water is spread. Leakages will be discussed in greater detail in Chapter 3 of this thesis.

2.5.2. Liquid System

Liquid systems are typically applied with a hot mix of melted rubber, tar, polymer and/or asphalt emulsions. These systems have a long track history with trained installers. The membrane is effective in penetrating, easy to apply on horizontal surfaces and leaves no seams. Some of the disadvantages include susceptibility to chemical breakdown, the necessity of dry surfaces, and/or 7-day cured concrete surfaces. They are difficult apply on vertical surfaces. Poor elongation capacity allows for separation from substrate. Any movement of the substrate surface causes the membrane to split and the system will be compromised.

2.5.3. Sheet Membrane Systems (positive): Rubber, Neoprene, Rubber/HDPE

Sheet membrane systems are typically manufactured from rubber, neoprene, high density polyurethane (HDPE), and a rubber/HDPE composite. The membrane is manufactured in 6 to 10 ft (2 to 3 m) wide self adhesive rolls. The concrete surface does not have to completely cure for the membrane application. The sheets can be applied against temporary excavation support systems, sheet piles or secant piles. These systems often leak due to improperly sealed seams during construction. A dry, clean surface is essential and the process requires skilled contractors. These types of membrane systems are able to resist elongation.

2.5.4. Sheet Membrane System (negative): HDPE, PVC

Sheet membrane systems comprised of High Density Polyurethane (HDPE), or Poly Vinyl Chloride (PVC) are often implemented in bored tunnels. These systems differ from the self adhered systems, discussed in 2.5.3, in that they are attached to the rock or shotcrete liner of the tunnel. They are widely used in construction of NATM. The system is installed as a close or open bag system: a closed bag system does not allow any water to penetrate it and is watertight; while an open bag system allows for drainage form the exterior of the liner to reduce hydrostatic pressure and is also watertight. The system is difficult to erect and often leaks due to unforeseen penetrations by reinforcing steel. The system, if properly installed, is an excellent system but requires intensive inspection. During the membrane construction the system is highly flammable. In addition the system is relatively rigid and has limited extensibility. This system is not suited for cut-and-cover constructions (Russell 2007).

2.5.5. Epoxy Liner Systems

These systems are rigid, with very limited elongation capabilities and minimal flexibility. Any movement in the structure can breach the membrane. The concrete has to cure for 18 days prior to application. The system is extremely flammable and explosive due heavy vapor density and low flash point (Russell 2007).

2.5.6. Sprayed Coatings Systems

Cold Applied Neoprene can be sprayed onto concrete surfaces immediately after the concrete forms are removed. It is good elastic membrane but is very susceptive to hydrocarbon attacks. There have been few applications of neoprene for tunnels. An alternative system is hot applied polyurea spray coatings which are new to the market. These systems have moderate elongation properties but can produce shear cracks. There are very few experienced installers and the costs are a lot less competitive compared to other systems (Russell 2007).

2.5.7. Cementations Waterproofing System

Cementitious coating is a rigid system which is susceptible to cracks and leaks with the movement of an underground structure. Exterior coatings are limited in application. They can be quickly installed after the concrete forms are removed.

2.5.8. Poly Rubber Gels

A composite of recycled rubber and copolymers produces flexible membranes that remain plastic. This system has been introduced on the market within the last decade and has been mostly used in Asia. It is typically applied cold by spray, however the material is extremely viscous. The systems have excellent resistance to hydrocarbons and sulfate/sulfite attacks. The material is extremely adhesive and is excellent in elongation and flexibility. This system was recommended for the invert slab, walls, and roof of Marina Central Expressway, Singapore, a planned 10 lane highway tunnel to be constructed by traditional cut-and-cover methods (Russell 2007).

2.6. Grouting Methods

Grouting includes the injection of a hardening/foaming/expanding fluid or mortar into the ground to improve its stiffness, strength and/or impermeability. There are numerous patterns of grout propagation: low pressure, compression and jet grouting. In low pressure or permeation grouting, the grout material is injected into the pores of the soil with minimal disturbance to the soil skeleton. The resulting grouted regions around a point source are spherical for isotropic soil as ellipsoidal (for anisotropic flow). It is possible to establish a control pressure at which the ground will fracture to continuously monitor the volumes of grout injected. Kolumbas (2005) defines the maximum pressure for permeation grouting as $\alpha\gamma h$, where γh, is the overburden pressure and α is an emperical factor. For grounds with very high or very low strength α may vary between α = 0.3 – 3.0.

During compensation grouting the ground is fractured at higher pressures and the grout propagates into the cracks causing compression of the soil voids. This type of grouting may be applied to compensate for surface settlements caused by tunneling. In jet grouting a grout jet protrudes from a nozzle into the surrounding soil. With an initial pressure between 4430 and 8862 psi (300 and 600 bar) the process mixes the soil and grout. Neither of these processes can be used to achieve long term watertightness of underground facilities, but both can control flow during construction operations.

2.7. Waterstops and Joints

Cold joints and structural intrusions are the most common causes of leakage in underground structures. Problems often arise due to lack of detailing and installation of proper gaskets or waterstops in the structure. While structural penetrations such as tie-down anchors, pipe or utility connections are often difficult to seal. All intrusions share the same commonality in that they breach the waterproof membrane and require special treatment to prevent the water ingress. The materials used for sealing penetrations and cold joints are referred to as waterstops. In cold and hot applied rubber neoprene and poly rubber gels membrane systems, minimal additional effort is required to seal penetrations and cold joints. The Poly Gel Rubber and cold applied Neoprene are the only products that do not require additional treatment for penetrations (Russell 2007). In order to ensure a tight structure, extra seals may be added to all penetrations. Sheet membrane, Epoxy and Bentonite membranes are the most difficult to seal and require additional materials and procedures to seal around structural connections.

2.7.1. Waterstops in Slurry Walls

In slurry walls waterstops are inserted with the help of the end stops, they are used to mitigate leaks. The usual remedy for excessively leaking joints is to grout behind the joint once the movements of the wall have stabilized stopped. However, additional differential deflections can occur between adjacent panels if too much grout is inserted behind the wall, and thus sealing will not be effective.

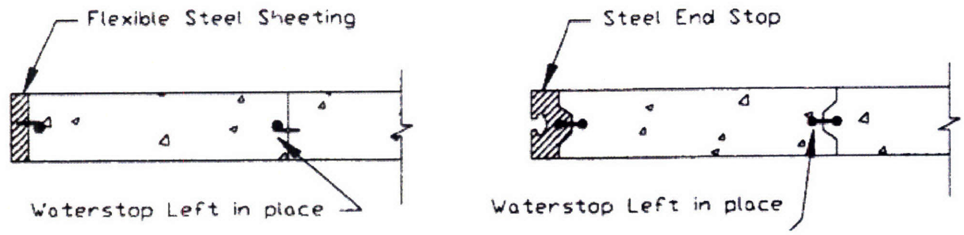


Figure 2-6, Flexible sheet pile with male waterstop notch (Left), Keyed steel end stop with chemical waterstop (Right), (Konstantakos 2000)

2.7.2. PVC and Vinyl Waterstops

The traditional waterstop for concrete cold joints are made of vinyl, PVC or rubber and have ribbed or beaded cross-section as shown in Figure 2-7. These waterstops are partially placed in the primary pour and are held in position in the subsequent pour. This placement is across the joint providing a second barrier to the inflow of water. Beaded-sections should only be used where limited movement is expected (Poole 2008). These water stops rely on the ribs, as shown in Figure 2-7, to create a tortuous path for water to follow across a cold joint. These waterstops are often ineffective due to damage or displacement during the concrete pour and craftsmanship during installation (can be easily torn or folded). Beaded waterstops should not be used for pipe or utility connections (Russell 2007). Waterstops must be securely positioned in the forms to prevent deflection or misalignment during concrete placement. This is achieved by fastening the outer flanges of the waterstop to the adjacent reinforcing steel (Greensteak 2008). Ribbed centerbulb is the most versatile type of waterstop available. The centerbulb accommodates lateral, transverse, and shear movement. Ribbed centerbulb waterstop can be used in expansion, construction, and control joints, Figure 2-8 shows a detail of a ribbed centerbulb waterstop.

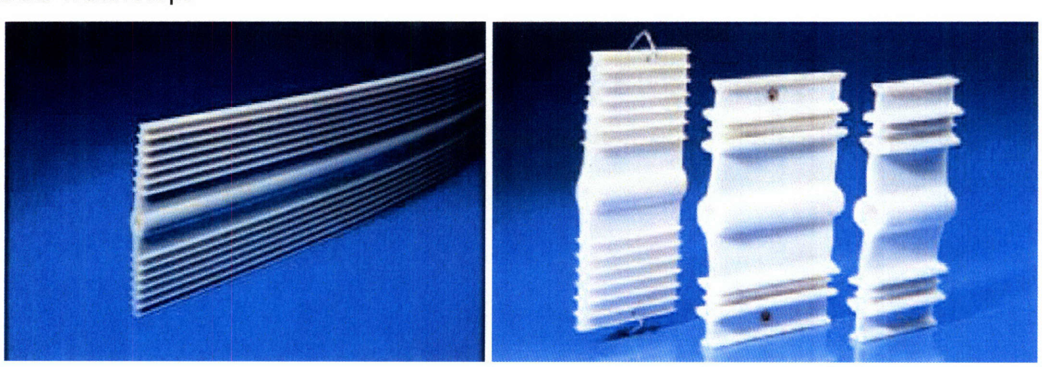


Figure 2-7, PVC Waterstops, (Greenstreak 2008)

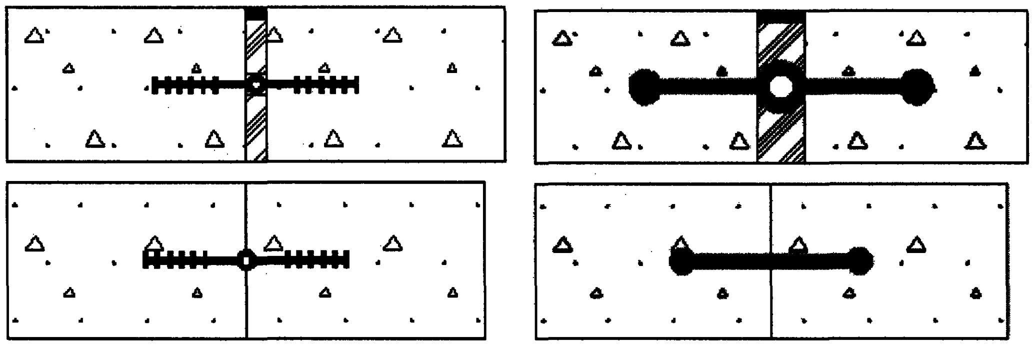


Figure 2-8, Ribbed Centerbulb Waterstop (Left) Dumbbell and Centerbulb Waterstop (Right) (Poole 2008)

2.7.3. Swelling Rubber Waterstops

An alternative to the traditional water stop at cold joints is the use of swelling rubber waterstops. The swelling rubber is a polymerized rubber that swells up to 300% when in contact with water (Russell 2007). Swelling rubberstop require 72 hours for the material to swell thereby preventing premature failure of the waterstop as a result of the water stop material being wet from rainfall or other means during construction. Swelling rubber waterstop can be most effective. They require minimal attention during installation and are generally nailed to the cured primary pour. Swelling rubber waterstops have also been recommended for the Marina Central Expressway, Singapore (Russell 2007).

2.7.4. Injection Waterstops

Injection waterstops are designed to allow the injection of grouts into a leaking cold joint. A tube is placed in the joint, with junction boxes placed at intervals to allow for the tube to be injected at a later time when the joint leaks. The use of injection tubes for waterproofing joints in walls and penetrations has had little success. They are extremely sensitive to the installation and in particular to the care taken in the installation. They must be protected from concrete infilling and can only be implemented for a one time injection. Injection tubes should be used as a backup system and not as a primary waterproofing of construction joints. (Russell 2007)

3. LEAKAGE RATES

Water-resistant tunnels and waterproofing systems are equally over-and-under emphasized. Measures to achieve watertightness are overdone by some owners and not properly executed by others (Kolymbas 2005). Some droplets of water may be tolerable in certain cases and absolute water tightness is rare. What should be avoided (especially in road tunnels) are inflows creating water puddles, black ice and icicles. It should also be taken into account that, in the end, every water-resistant lining will have a few leaks. Kolymbas (2005) recommends for tunnel operators and owners to focus on the reduction of the related damage (e.g. with drainage of the leaking water and vents for grouting) and provisions for an easy repair. In addition sealing to prevent all seepage is extremely expensive and the designer has to justify his specification of water proofing relative to these costs.

The success or failure of waterproofing depends on the owner's perception of water tightness and the established permissible leakage rate identified in the contract documents. The Author focuses on international permissible and established leakage rates, and presents specified guidelines for permissible rates, rates established for specific project and recorded inflow rates in existing tunnels. However specifying a leak rate does not guarantee that a higher leak rate will not be encountered during operations. Appropriate lining and waterproofing systems must be selected to achieve the specified rates.

The intensity of leakage in a transit tunnel is the result of numerous factors. The causes include: permeability of the surrounding ground, permeability of the lining, and groundwater conditions (total head and recharge of the groundwater table). Further discussion on consequences of inflows is presented in Chapter 4 of this thesis. Figure 3-1 introduces the general concept of water infiltration into a tunnel. As the liner becomes less pervious or if systematic drainage is omitted, the head retained outside the liner is maximized and water within the tunnel is a minimum. On the other hand, either systematic drainage or inadvertent leakage will decrease the surrounding water pressures acting outside the lining. In rock or soft ground the external water pressures acting on the lining and leakage through the lining are produced by an interaction of the following factors: (1) the permeability of the surrounding ground and of the tunnel lining, (2) the provisions for systematic drainage to the tunnel or inadvertent leakage through the lining, (3) the tunnel head acting from the groundwater source, divided in two components: head loss in the surrounding ground and increment of head loss causing flow through the lining (Gould 1989).

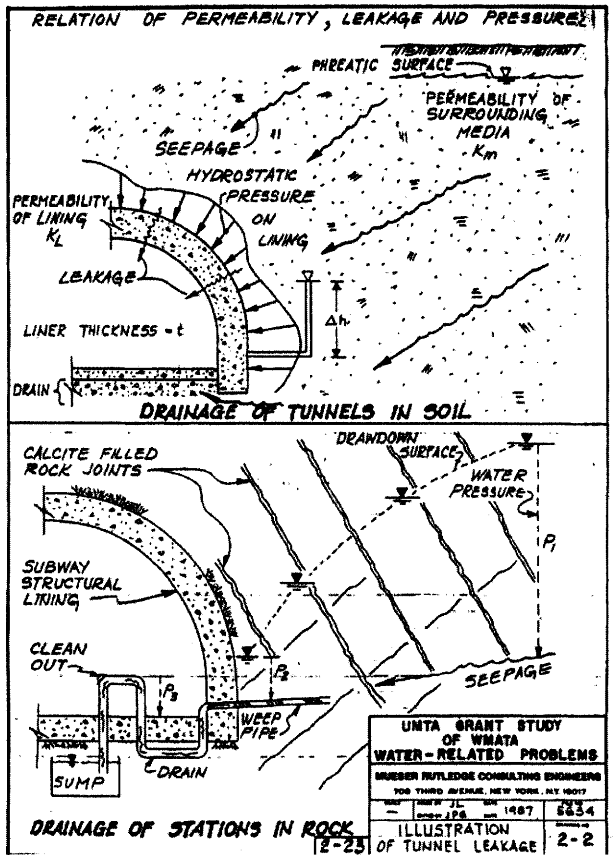


Figure 3-1, Relationship of Permeability, Leakage and Pressure, (Gould 1989)

3.1. Units of Measurement and Conversions

In the United States, the unit utilized to express the intensity of water infiltration is a flow rate, gallons per minute (gpm), for a given tunnel length (linear feet) or as a function of a tunnel interior surface area, expressed in square foot (SF or ft^2). Some rates are presented in a gpm per 10,000 SF or 100,000 SF. In Europe and Asia the rate is articulated as liters (L) per day for a square meter (SM or m^2). A point source leak is expressed as liters (L) for each minute or gallons per minute (gpm).

To convert from a rate of L/day/m^2 to gpm/ft^2, one must multiply a given number by a factor of $1.68x10^{-5}$ and vice versa. ($1.68x10^{-5}$ L/day/m^2= 1 gpm/ft^2)

To switch from a given flow rate for tunnel length (gpm/ft or L/day/m) to a flow rate as a function of a surface area (gmp/ft^2 or L/day/m^2), one must know an average tunnel diameter or circumference. An assumption of a circumference or diameter range may be used, for example inner diameters of 15 to 21 ft (4.6 to 6.5 m) may be assumed for a typical tunnel. Please note this cannot be assumed in all cases, one must know the characteristics of a given tunnel. Once the diameter or the circumference is established, a flow rate per linear increment must be divided by a circumference to obtain a rate as a function of the surface area.

For transit tunnels of approximate 18 feet inside diameter, one may divide the value of infiltration as gpm per 1000 lineal feet by 40 to obtain approximate leakage in gallons per square foot of tunnel surface per day (Gould 1989). Leakage in liters per square meter per day (L/m^2/d) is approximately equal numerically to the value of leakage in gpm per 1000 lineal feet (Gould 1989). Figure 3-4 and Table 3-2 presented in the succeeding sections use the above conversions. The Author concluded that a rate expressed in *English Units* (gpm/100,000 ft^2) is very close to a rate presented in the units of *International System of Units (SI)* (L/day/m^2). However one cannot state or that the rates are interchangeable due to uncertainties in the assumptions used.

3.2. Established Permissible/Allowable Leakage Rates

A permissible leakage rate is often identified as the amount of water that is acceptable to the owner for a safe operation of a given tunnel. Tunnel designers typically develop criteria for water tightness, which is defined as permissible leakage in this thesis.

The Author examined case studies in various tunnels with permissible leakage rates established for operation and maintenance. Case studies focusing on transit road tunnels are emphasized in this thesis. Figure 3-2 illustrates the established permissible leakage rates established by various international owners and agency specified standards and guidelines. Succeeding sections present allowable standards established internationally. Permissible rates for waste-water and other utility tunnel presented in this thesis to contrast with required performance of transit tunnels. Permissible rates very significantly and are dependant on various factors (including operation criteria). Details of these rates are provided in subsequent sub-sections.

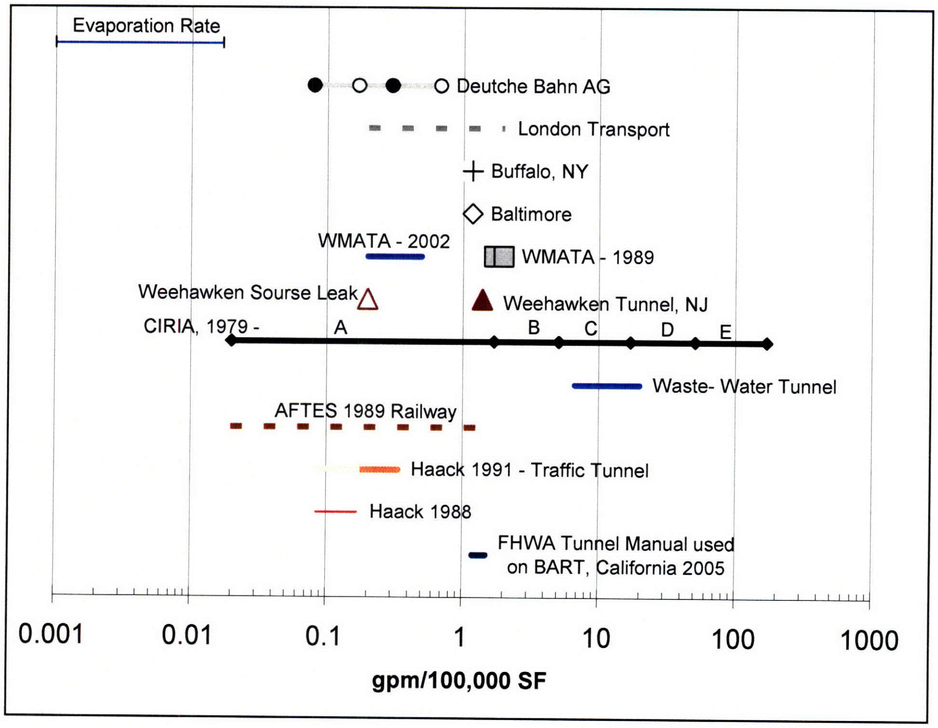


Figure 3-2, International Permissible Leakage Rates *(English Units)*

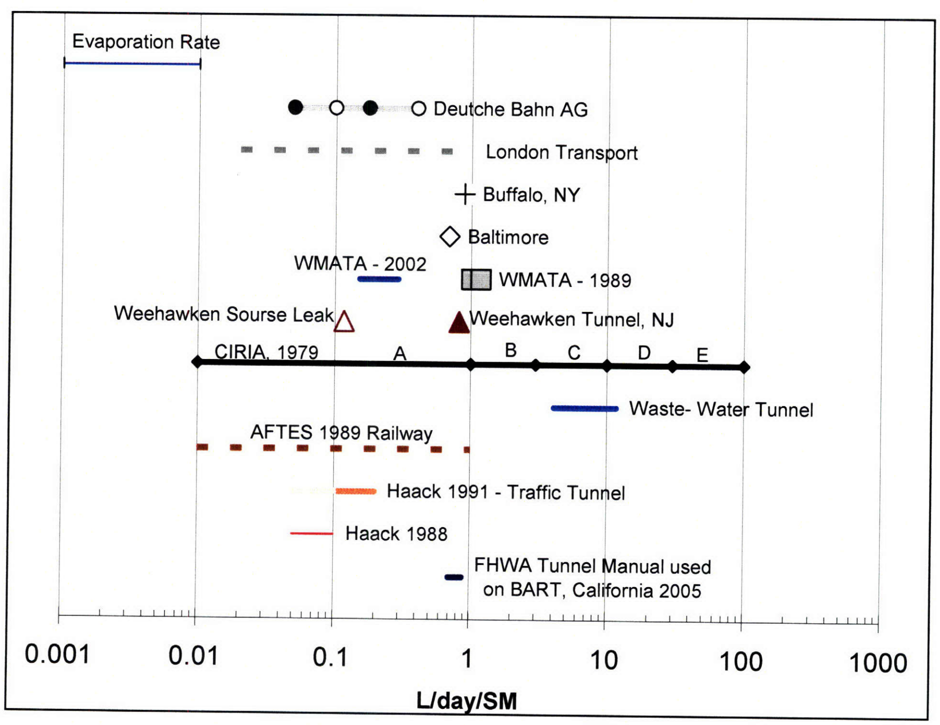


Figure 3-3, International Permissible Leakage Rates *(Metric Units)*

3.2.1. United Kingdom (UK) Classification

CIRIA identified a need for standardisation and recommended that one may adopt a waterproofing classification system similar to the one presented in Table 3-1, an introduction of a formal classification. CIRIA did not attempt to associate any standard rates with a specific tunnel usage; it was left to the engineer's judgment and owner's budget. Transit tunnels are assumed to be in class 0, A, and B, while utility tunnels are in classes C, D, and E. A detailed classification would enable a direct comparison between different waterproofing system and construction of water-resistant tunnels in various geological settings. Subsequent sections present specified permissible rates for tunnels identified by various international agencies. Grán (1988) cited CIRIA's classification in the table below.

Table 3-1, UK Tunnel Waterproofing Classification, (CIRIA 1979)

CIRIA Class.	Maximum permissible leakage (litre/d/m^2)
O	Nothing visible
A	1
B	3
C	10
D	30
E	100
U	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 1m or 100mm sides.
3. Examples of notation:
   A/all : B/1 = Class A overall, Class B over 1m square
   A/all : B/100 = Class A overall, Class B over ~~100m square.~~ 100mm square

3.2.2. United States (US) Standards and Metro Case Studies

For transit tunnels in the United States, the infiltration permitted by specifications is usually stated in terms of leakage per 100 or 200 lineal feet of single tunnel. This allowable value is equal to approximately 0.1 gpm per 100 lineal feet, or, on the horizontal scale in Figure 3-4, approximately 1 gpm per 1000 lineal feet of tunnel. Assuming a tunnel diameter of 15 to 21 ft (4.6 to 6.5 m) this rate is equal to approximately 1.2 to 1.5 gmp per 100,000 square feet (SF)

(0.7 to 0.9 L/day/m^2). Leakage rates which are specified for 1000 lineal feet typically average the local inflows created by individual large leaks. (Gould 1989)

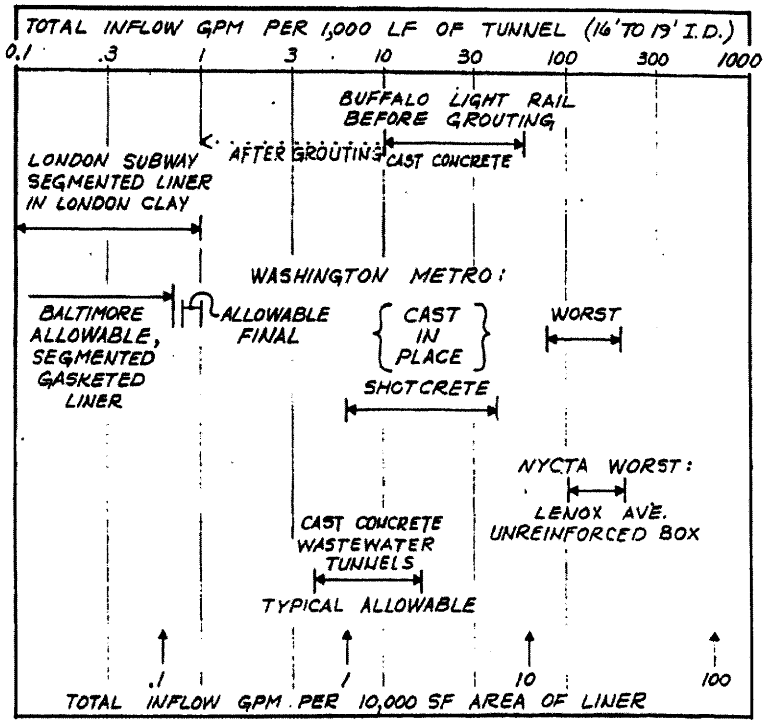


Figure 3-4, Experience of Tunnel Leakage Rates, Permissible and Recorded, (Gould 1989)

The FHWA Tunnel Maintenance and Rehabilitation Manual (FHWA 2005) was used to establish an allowable intrusion rate in the Bay Area Rapid Transit (BART) system in California. The FHWA Manual was adopted by other tunnel owners as a workable criterion. The maximum permissible rate, approximately 1 gpm per 1000 ft of tunnel, FHWA claims that this offers a practical point of reference to evaluate how successful the project is in achieving the specified requirement for a dry tunnel (FHWA 2005). It can be argued an inflow rate per linear segment is not sufficient information. An additional flow rate (function of a surface area) should be specified and considered for a source leak and overall tunnel surface area. A permissible inflow rate per liner tunnel length may be acceptable for one type of a tunnel with specific type of waterproofing system (segmental lining in this case for BART), and not applicable for a cut-and-cover tunnel with a varying alignment and cross-sectional areas.

Reported and published permissible leakage rates in the United States typically fall between 0.2 to 2 gpm per 100,000 ft^2. Various metro railway system rates are presented in Figure 3-4, Table

3-2, and Table 3-6. Gould (1989) reports for the Washington DC Metropolitan Area Transit Authority (WMATA) that the initial established leakage rates, 1.7-2 gpm/100,000SF, were revised to a maximum permissible inflow rate of 1.89 L/day/500ft (0.2-0.5 gpm/100,000SF) (Gould 1989 and WMATA 2002).

Table 3-2, Tunnel leakage Case Histories, (Gould 1989)

SYSTEM	TYPE OF TUNNEL LINING	PERMISSIBLE LEAKAGE	LEAKAGE EXPERIENCE
Wash. D.C. Metro WMATA	Cast concrete and shot-crete on ribs	0.2gpm per 250lf or 0.1gpm in any 100lf	Worst condition found in shallow rock with extensive re-charge, max. about 100gpm/1000lf.
	Segmented pre-cast concrete		Appears to meet specification limitation in first two sections completed.
Baltimore Subway	Segmented pre-cast or metal liner	.07gpm per 100lf	Reported "as no measurable infiltration "in segmented gasketed liner. Extensive grouting of cast concrete required to meet specification limitation.
Buffalo Light Rail NFTA	Cast concrete	0.9gpm per 1000lf or .05gpm in any 30lf	Local maximum 250gpm in 300lf. Typically 10 to 60gpm per 1000lf. Reduced to specified limit by cement grout outside lining and by acrylamide grout in cracks in cast concrete.
London Transport	Segmented pre-cast concrete or metal liner	Not stipulated	Generally 0.1 to 1gpm per 1000lf. Metal liner used for modern sub-aqueous tunnels. Tunnels in London clay usually yield insignificant leakage.
Typical Waste-Water Tunnel	Typically cast-in-place concrete	4 to 12gpm per 1000lf	Exceptional cracking can occur when high internal water pressures deform the lining outward against yielding ground.

3.2.3. German Standards

Watertightness of subsurface facilities is highly emphasized in German codes. Permissible leakage standard are strict and have increased with time and costs of a given project. High standard of execution requires a longer construction period as well as the use of high-quality and, therefore, more expensive materials. In addition, the work must be carried out by highly trained workers with the greatest possible care (Haack 1991).

The definition "bone dry," was used by German clients, but no longer stipulated in construction contracts at least not in the case of tunnels built by mining methods. One may specify that the tunnel must be dry enough to ensure that it can be used without disturbance for its planned service life. Consideration of a wider range of possible types of use results in the sort of picture presented in the Table 3-3.

Table 3-3, Required degree of tightness related to the use to which the tunnel or construction is planned, (Hack 1991)

Degree of Required Watertightness	Nature of Use of Tunnel/Structure	Likely Damage/Problems to be Expected
Higher	People present for lengthy periods	Chronic illnesses
	Storage of goods affected by moisture (paper, foodstuffs)	The quality of the goods is diminished or goods may be entirely spoiled
	Traffic tunnel sections affected by frost (portal zones, pedestrian tunnels)	Icicle formation in the clearance profile; reduced traffic safety
	Frost-free traffic tunnel sections	Damage to the building material, possibly resulting in reduced stability
	Utility tunnels	Damage to the building material, possibly resulting in reduced stability, corrosion of lines
Lower	Sewage tunnels	Damage to the building material, possibly resulting in reduced stability; eventual added load for the clarification plant; environmental pollution

The German League of Cities established starting points for permissible leakage rates. An attempt was made to define permissible leakage rates in underground railway constructions in a large number of German cities, as shown in Table 3-4. The permissible leakage rates provided below did not address the type of waterproofing system used. Thus Haack (1991) states that these limits are conceptual and can hardly be checked in practice.

German Federal Railway tunnelling guidelines (1984) did not define any permissible leakage water rates. However three classes of tightness were identified Table 3-5. The relevant moisture characteristics correspond to those contained in lines 1 to 3 of Table 3-4. The permissible leakage water rate for railway tunnels, given in line 4 of Table 3-4, is too high for a German Federal Railway specification. The German Railway and German League of Cities specification apply to facilities with waterproofing systems and some water-resistant structures.

In the United States and in other countries, the permissible leakage water rate is defined in conjunction with the stated reference length. Table 3-6 presents examples of underground railway systems classified by such a system. This allows a greater leakage rate for the shorter reference length than for the longer tunnel section. This method, with reference lengths appears more suitable compared to Table 3-4.

Table 3-4, Permissible daily leakage water rates, depending upon the use of subsurface facilities, according to findings of the Otto-Graf-Institute, (Haack 1991)

Line	Moisture Characteristics	Purpose	Permissible Daily Leakage Water Rate (l/sq. m)
0	1	2	3
1	Completely dry	Storerooms, restrooms	0.001
2	Substantially dry	Underground/tramway tunnels	0.01
3	Capillary penetration of moisture	Road, pedestrian tunnels	0.1
4	Weak trickling water	Rail tunnels	0.5
5	Trickling water	Sewage tunnels	1.0

Table 3-5, Water Tightness Classes of the German Railway, (Haack 1991)

Tightness Class	Moisture Characteristics	Use of Tunnel	Definition
1	2	2	4
2	Completely dry	Storerooms and workrooms, restrooms	The wall of the lining must be so that that no moist patches are detectable on the inside.
3	Substantially dry	Frost-endangered tunnel sections	The wall of the lining must be so tight that only slight, isolated patches of moisture can be detected on the inside (e.g., as a result of discoloration. After touching such slightly moist patches with the dry hand, no traces of water should be detectable on it. If a piece of blotting paper or newspaper is placed upon a patch, it must on no account become discolored as a result of absorbing moisture.
4	Capillary moisture penetration	Tunnel sections and rooms for which Tightness Class 1 or 2 is not required	The wall of the lining must be so tight that only isolated, locally restricted patches of moisture occur. Restricted patches of moisture are such that they reveal that the wall has been penetrated by moisture, and a piece of blotting paper or newspaper discolors if placed upon it—but there is no trickling water evident.

One may note that the United States leakage permissible rates are greater that the German equivalents. For example German rate, 0.01 L/m^2 (1.7×10^{-7} gpm/ft^2), (Table 3-4 line 2) is 90 times smaller than Washington D.C. rate of 0.9 L/m^2 (1.5×10^{-5} gpm/ft^2).

The maximum leakage rates established by Buffalo, New York (Haack 1991) is of the same value shown in Table 3-4, for the capillary penetration of moisture for road and pedestrian tunnels. Table 3-7 summarizes proposed permissible leakage water rates for operating different types of tunnels (Haack 1991). Comparing Table 3-3 and Table 3-6 it can be seen that the permissible leakage rates for short reference lengths have been increased for rooms accommodating persons and for underground railway tunnels; retained for road and pedestrian tunnels; and reduced for railway tunnels. One can conclude that the recommended permissible leakage rates for large reference lengths are half as high as for rates for short reference lengths.

Finally, water tightness classification for German Railway Company, Deutsche Bahn AG (DB) are presented in Table 3-8. DB is the operating company for 746 tunnels, of which 491 were constructed between 1840 and 1940. The classes of water-tighness and respective permissible water leakage rates are shown in Table 3-8.

Table 3-6, Permissible daily leakage rates in various United States Cities, Austria and Belgium, (Haack 1991)

Underground Railway Systems	Short Section		Long Section	
	Daily Leakage Rate (l/sq. m)	Reference Length (m)	Daily Leakage Rate (l/sq. m)	Reference Length (m)
1	2	3	4	5
Washington, D.C. (U.S.A.)	10.7	3.5	0.9	80
San Francisco (U.S.A.)			0.9	80
Atlanta (U.S.A.)			0.9	80
Boston (U.S.A.)			1.8	35
Baltimore (U.S.A.)	5.3	3.5	0.7	35
Buffalo (U.S.A.)	0.4	10	0.2	1,000
Melbourne (Australia)	0.25	10	0.1	1,100
Antwerp (Belgium)	0.25	10	0.1	100

Table 3-7, STUVA's proposal for determining use- and length-related permissible daily leakage water rates in Germany, (Haack 1991)

Tightness Class	Moisture Characteristics	Intended Use	Definition	Permissible Daily Leakage Water Quantity (l/sq. m), Given a Reference Length of:	
				10 m	100 m
1	2	3	4	5	6
1	Completely dry	Storerooms and workrooms, restrooms	The wall of the lining must be so tight that no moist pat hes are detectable on the inside.	0.02	0.01
2	Substantially dry	Frost-endangered sections of traffic tunnels; station tunnels	The wall of the lining must be so tight that only slight, isolated patches of moisutre can be detected on the inside (e.g., as a result of discoloration). After touching such slightly moist patches with a dry hand, no traces of water should be detectable on it. If a piece of blotting paper or newspaper is placed upon a patch, it must on no account become discolored as a result of absorbing moisture.	0.1	0.05
3	Capillary wetting	Route sections of traffic tunnels for which Tightness Class 2 is not required	The wall of the lining must be so tight that only isolated, locally restricted patches of moisture occur. Restricted patches of moisture reveal that the wall is wet, leading to a discoloration of a piece of blotting paper or newspaper if placed upon it—but no trickling water is evident.	0.2	0.1
4	Weak trickling water	Utility tunnels	Trickling water permitted at isolated spots and locally.	0.5	0.2
5	Trickling water	Sewage tunnels		1.0	0.5

3.2.4. France - Specifications

AFTES (1989) established a maximum permissible rate of 1 L/day/m^2, for railway tunnels with cast-iron segments.

Table 3-8, Classification of Watertightness by Deutsche Bahn AG (German Rail) for their Underground Facilities, (AITES 2001)

Tight-ness Class	Moisture Characteristics	Use of Tunnel	Definition	Acceptable leakage rate (l/day/sq.m) at the reference length	
				10m	100m
1	Completely dry	Stores, workrooms, rest rooms	The wall of the lining must be tight so that no moist patches are detected on the intrados	Nil	Nil
2	Substantially dry	Frost-endangered underground sections	The wall of the lining must be tight so that only slight, isolated patches of moisture can be detected on the intrados, e.g. result of discolouration. After touching such slight moist patches with a dry hand, no traces of water should be detected. A sheet of blotting paper placed on a patch, should not discolour as a result of absorbed moisture.	0.2	0.05
3	Capillary moisture penetration	Underground sections and rooms which do not require class 1 or class 2.	The wall of the lining must be tight so that only isolated, locally restricted patches of moisture are to be seen. Restricted patches of moisture are areas at which a penetration of moisture could be registered. A sheet of blotting paper will discolour as it is soaked with water, but no trickling water is to penetrate the intrados.	0.4	0.1

3.3. Contract Documents and Technical Specifications

Maximum permissible rates should be determined in the contract documents and set by the owner prior to bidding the project. This will help eliminate misunderstandings and subsequent disputes from the very outset. The owner has to establish a quantitative value of maximum allocable leakage using the description of the moisture characteristics provided in the preceding sections. Vague specifications, for example a *dry tunnel*, will create confusion and general claims. One must be aware that low permissible leakage rates (tightness class 1 presented in Table 3-7) can be achieved if properly planned and executed with an acceptable waterproofing systems. To achieve such a rate a great monetary investment must be made. To save money on construction some owners will select a tightness class 2 (Table 3-7), and chose a more economical waterproofing system.

Technical specification may include information on grout injections to be carried out to achieve the specified tightness. This is typically done during construction and before the project is turned over and accepted by the owner.

If the degree of tightness specified in the contract is still not achieved, then long-term effective supplementary measures should be undertaken, discussed in Chapter 5 of this thesis.

3.4. Means and Methods of Leakage Measurements

CIRIA (1979) recommended the following the descriptions to be tied with quantifications of an observed leaks. Haack (1991) added an additional term, **Past Moisture** defined as staining arising from former moisture.

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	-	drop of water which fall at a rate of at least L/min; (Note: 1 L/day is 3 lo 4 drips/min)
Continuous Leak	-	A trickle or jet of water. (Note: Dips become a continuous trickle when they fall at a rate of about 300/min).

3.5. Reported Leakage Rates – Case Histories

International reported leakage rates are presented in Figure 3-5, while Chapter 4 of the thesis will address consequences, possible sources and causes of water inflow. Subsequent sections describe the geology, cross-section, construction means, lining type and waterproofing systems used in each case.

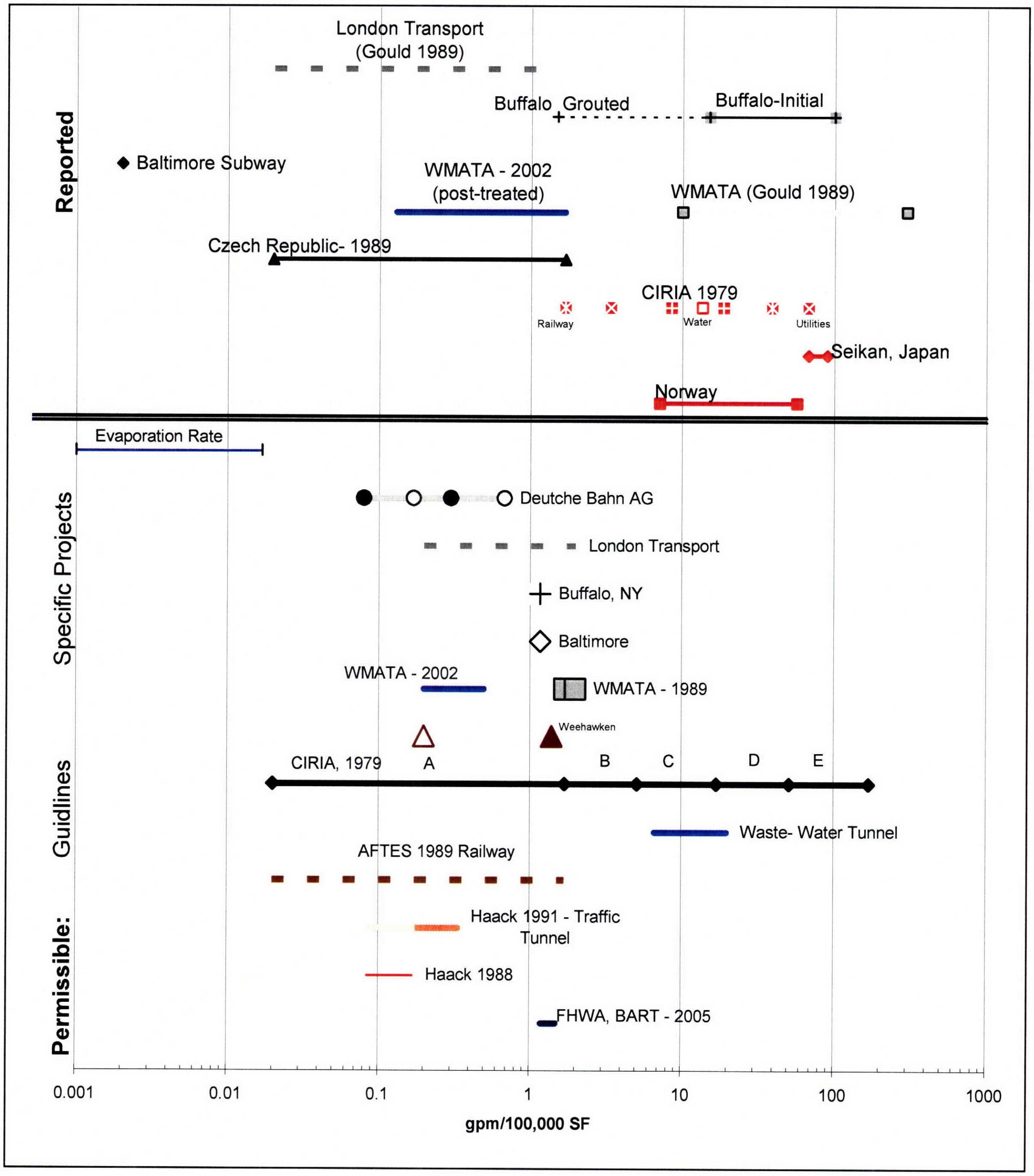


Figure 3-5, Reported Leakage Rates versus Permissible Rates *(English Units)*

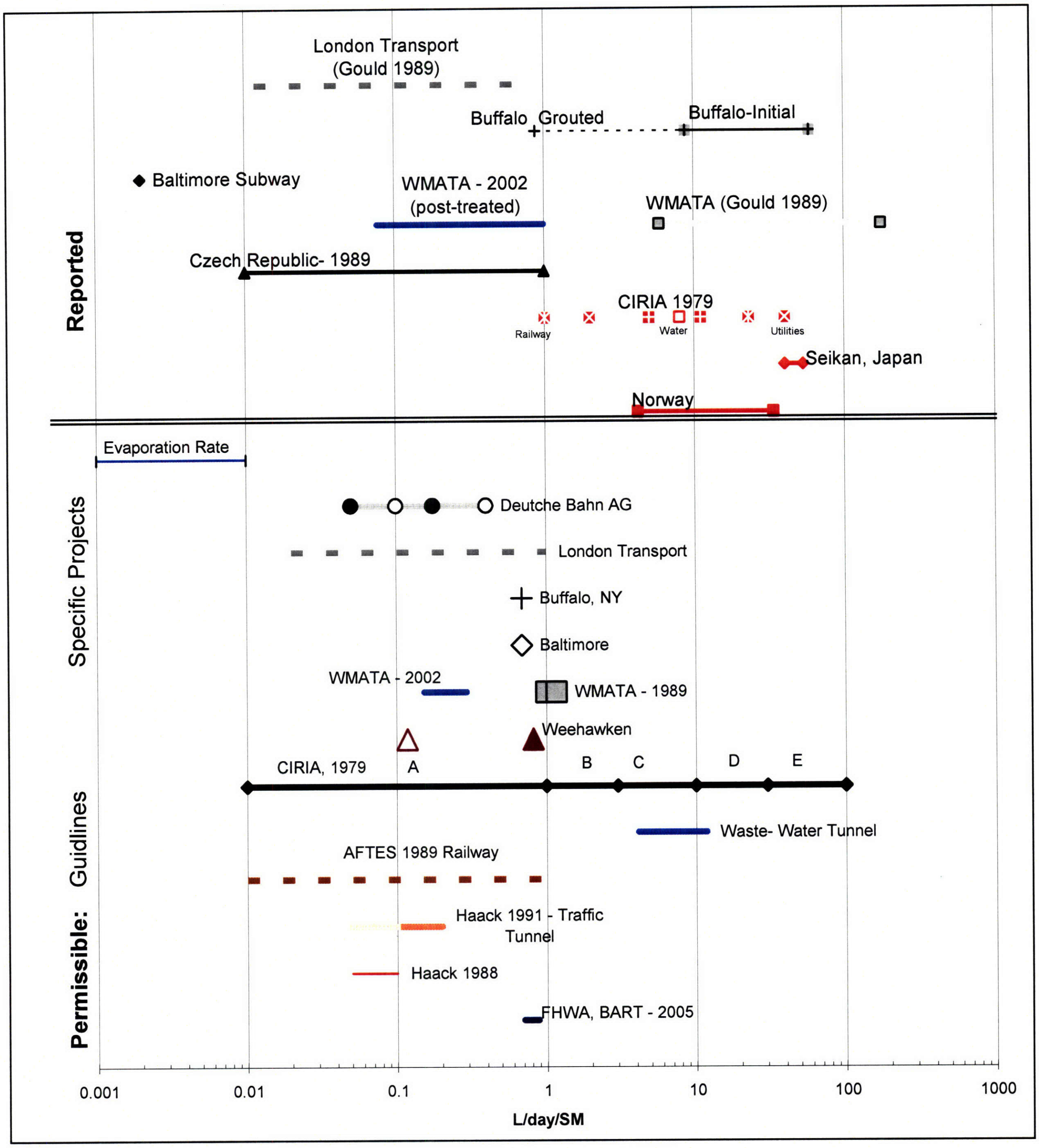


Figure 3-6, Reported Leakage Rates versus Permissible Rates *(Metric Units)*

3.5.1. United Kingdom

CIRIA (1979) presented examples of inward leakage flow rates which have been deemed satisfactory or specified for various tunnels completed in the UK, Table 3-9. No characteristics of the specific tunnels were given in this report.

Table 3-9, Examples of Inward Leakage Flow Rates in Existing Tunnels (CIRIA 1979)

Purpose of tunnel	Flows and units in the manner specified or reported	Equivalent to litre/d/m^2
Water transfer	1 litre/m length/m i.d./h	8
Sewerage	150 gal/in. dia./mile/day	5
Sewerage	30 litre/m dia./100m length/½h	5
Sewerage	0.06 litre/mm perimeter/day/10m length	6
Sewerage	0.25 gal/in. dia./100ft/h	11
Railway	6 litre/s/mile (4.5m i.d.)	23
Railway	0.07 gal/min/100ft (17ft 10in. i.d.)	1.5
Railway	70 gal/d/100ft (12ft 8in. i.d.)	1
HT cables	1 gal/yd^2/d above tunnel axis 6 gal/yd^2/d below tunnel axis	6 40
HT cables	2.3 gal/yd^2/d – sensibly dry above knee joint	15
PO cables	0.5 litre/min/5m length (7ft 0in. i.d.) 2.5 litre/min/50m length (7ft 0in. i.d.)	21 11
PO cables and equipment	0.1 litre/min/2m length (19ft 6in. i.d.) 0.25 litre/min/10m length (19ft 6in. i.d.)	4 2
Water transfer (National Water Council Specification(2))	0.5 litre/lineal m/m nominal bore/30 min	8

3.5.2. United States

Precast concrete liners were part of the early construction, of the Washington D.C. Metro (WMATA) and had low leakage rates. However, they were not used on the entire system. In the later construction stages, cast-in-place (in situ) concrete linings were implemented. One section of the WMATA Red Line experienced high inflow rates with the new construction. Gould (1989) presents values for local inflows greater than 1 gpm inflow for a lineal foot, equivalent to values of 200 to 500 gpm for 1000 feet, averaged over longer tunnel section, (150 to 300 gpm/100,000SF). The measured leak rates were up to 100 times the permissible inflow rates. Mitigation measures (section 5) were taken to reduce the high inflow rates. Individual leaks were sealed by grouting and caulking. In 2002 the measured inflows were three (3) times greater than the permissible rates and ranged from 0.5 to 6.2 L/day for a 500ft reference length (0.1-1.7

gpm/100,000SF). Figure 3-7 illustrates examples of measured leakage rates along the tunnel alignment versus permissible leakage rates.

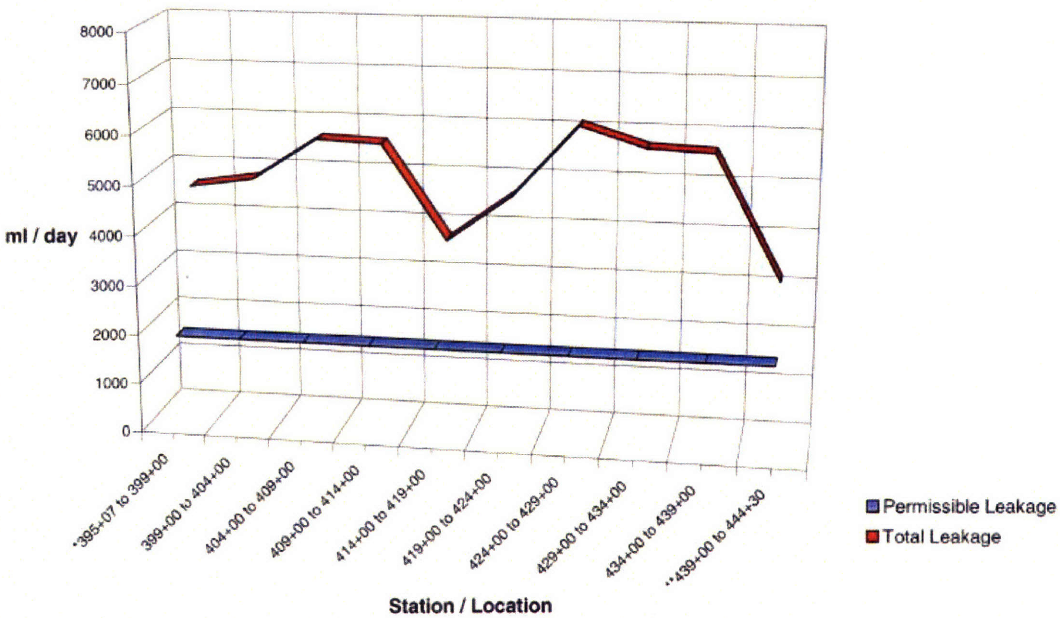


Figure 3-7, Leakage along Sections of WMATA Red Line (Bethedsa to Medical Center Station), (WMATA 2002)

One may state that there is resemblance between watertightness and particular lining types. For example, the **Baltimore** Metro experience with segmented, gasketed concrete or cast-iron linings is similar to that of London Transport. In both cases the leakage averaged out below the specified limit of 1 gpm per 1000 lineal feet. Extensive grouting behind the linings and local remedial grouting for individual leaks was performed. Further detail about the Baltimore Subway and others are presented in Table 3-2.

3.5.3. Czech Republic

Measured inflow rates for a rail tunnel in Prague with tunnel diameter of 5.1m and station diameter of 7.8m are presented by Grán (1989). The tunnel was constructed with a precast concrete element lining, grouting was implemented between the rock and the lining. Sand layers were encountered at the surface with slow moving groundwater, and rock layer with joints at the tunnel invert.

Stations were constructed via cut-and-cover and experienced watertightness problems, where only 10 out of 33 stations did not have leakage. The main tunnels were driven. The majority of inflows were at the cut-and-cover constructions of reinforced in situ concrete. In 1983 the following inflow measurements were taken along an 18 km aliment, including 9 driven stations and 14 stations via cut-and cover (Grán 1989). The alignment was divided into 30 sections for inspections. The results are presented below according to CIRIA classification system (presented in Table 3-1).

- 2 sections - class 0
- 6 sections - class A
- 13 sections - class B
- 9 sections - class C

The majority of the leaks were found in the connections, expansions joints, construction joints, vents and shafts (Gran 1989). Roof slabs for the stations were constructed with precast beams. Original waterproofing, up to 1983, consisted from layers of bitumen isolation. Post 1983 foils of softened PVC were implemented, where foils were covered by special fabric from glass fibers for protection during concrete casting construction.

3.5.4. Norway

In Norway 30 subsea tunnels were built during the past 2-3 decades. Nilsen (2001) reports leakage rates for four (4) road tunnels. The subsea tunnels are constructed in Geneiss rock with faults and very challenging ground conditions. The tunnels were constructed via drill and blast method. Ellingsoy, Kvulsund, Godoy and Freifjord Projects were completed between 1987 and 1992. The main rock is. Tunnel length ranges from 1.6 to 5.2 km with a cross-sectional are of 43 to 70 m^2. The liners are comprised of in situ concrete or fiber reinforced shotcrete with rock bolts; ribs were also implemented.

Eleven (11) tunnels had reported leakage rates after completion of construction, ranging from 8.5 to 460 L/min/km. After grouting and during the operation the rates in four (4) tunnels were reported from 90 to 280 L/min/km (7 to 15 gpm/1000ft). Converted to a flow rate as a function of surface area the infiltration rates will range 4.2 to 33.5 L/day/SM (7.1 to 57 gpm/100,000SF). In the Elingson tunnel a maximum flowrate of 400 L/min was encountered from a single probe at a depth of 1 km below the sea level.

3.5.5. Japan

Fourteen (14) years of inflow data was collected in Seikan, Japan; a 55 km subsea rail tunnel with 5 m diameter. Reported leakage rates ranged between 24 to 32 m^3/min/55km or 40 to 53 L/day/m^2 (68 to 90 gpm/100,000ft^2). Approximately half of the tunnel alignment is undersea. Tunnel invert elevations are at depth of 240 m below the groundwater table (Ikuma 2001).

3.6. Estimated Leakage Rates for CA/T Project in Boston

Boston's Central Artery Project (CA/T) is a large vehicular concrete box tunnel constructed via a cut-and-cover method using SPTC wall. Portions of the tunnel alignment are beneath the Boston Inner Harbor (Ted Williams Tunnel). The groundwater table is near or at the surface, the total head is approximately 70ft, (21.3 m) (Christian 2007).

In September 2004 a breach occurred in one of panel located the deepest section of the tunnel, spewing water onto the roadway and creating a crisis of public confidence in the project. The tunnel was closed to traffic during the initially emergency procedures to staunch the flow. The breach occurred where the soil outside the tunnel is till, comprised of sands and gravels with little or no silt and clay, having permeability much higher than other tills and overlaying clays. Faulty construction by means of a concrete over-pour intruded into a secondary panel. Portions of the soil lying under the excess mass were never excavated and were incorporated in the final wall. Part of this material was till through which the flow eventually occurred (Christian 2007).

The tunnel alignment is approximately 17 miles (89,800 ft). Groundwater inflows experienced in the tunnel are estimated between 0.3 to 1.2 gpm/100,000 SF with means approximately 0.6 gpm/100,000 SF (Whittle, 2008).

Tremendous local interest has been generated by leakage experienced in the CA/T tunnels. Figure 3-9 compares the leakage rate to the international norms described above. One can conclude that the CA/T rates are within CIRIA Class A Guidelines and are within the reported rates by WMATA and observed elsewhere. The inflows at CA/T are within the range recommended by Haack (1991) for road tunnels.

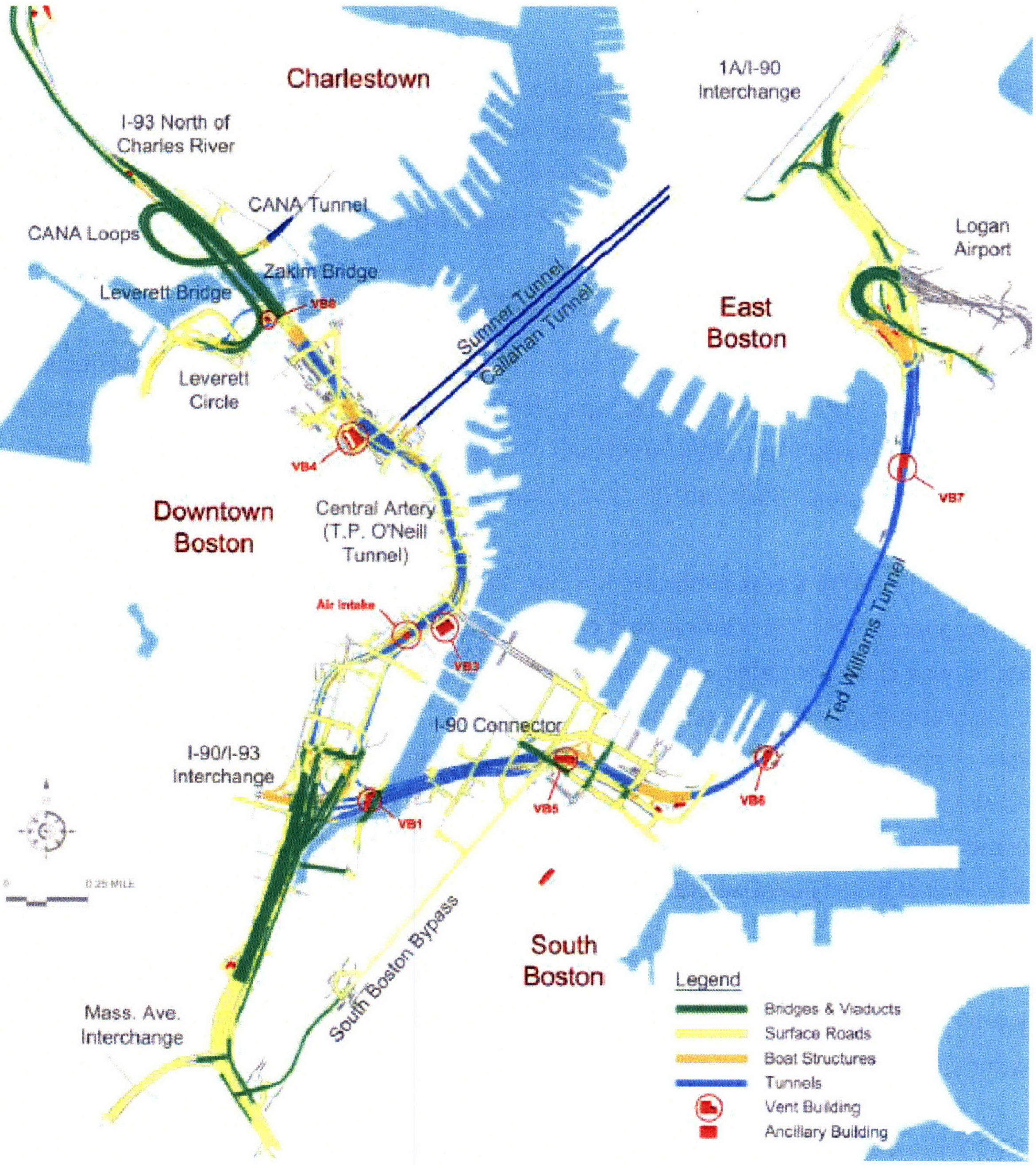


Figure 3-8, Plan of CA/T Project in Boston

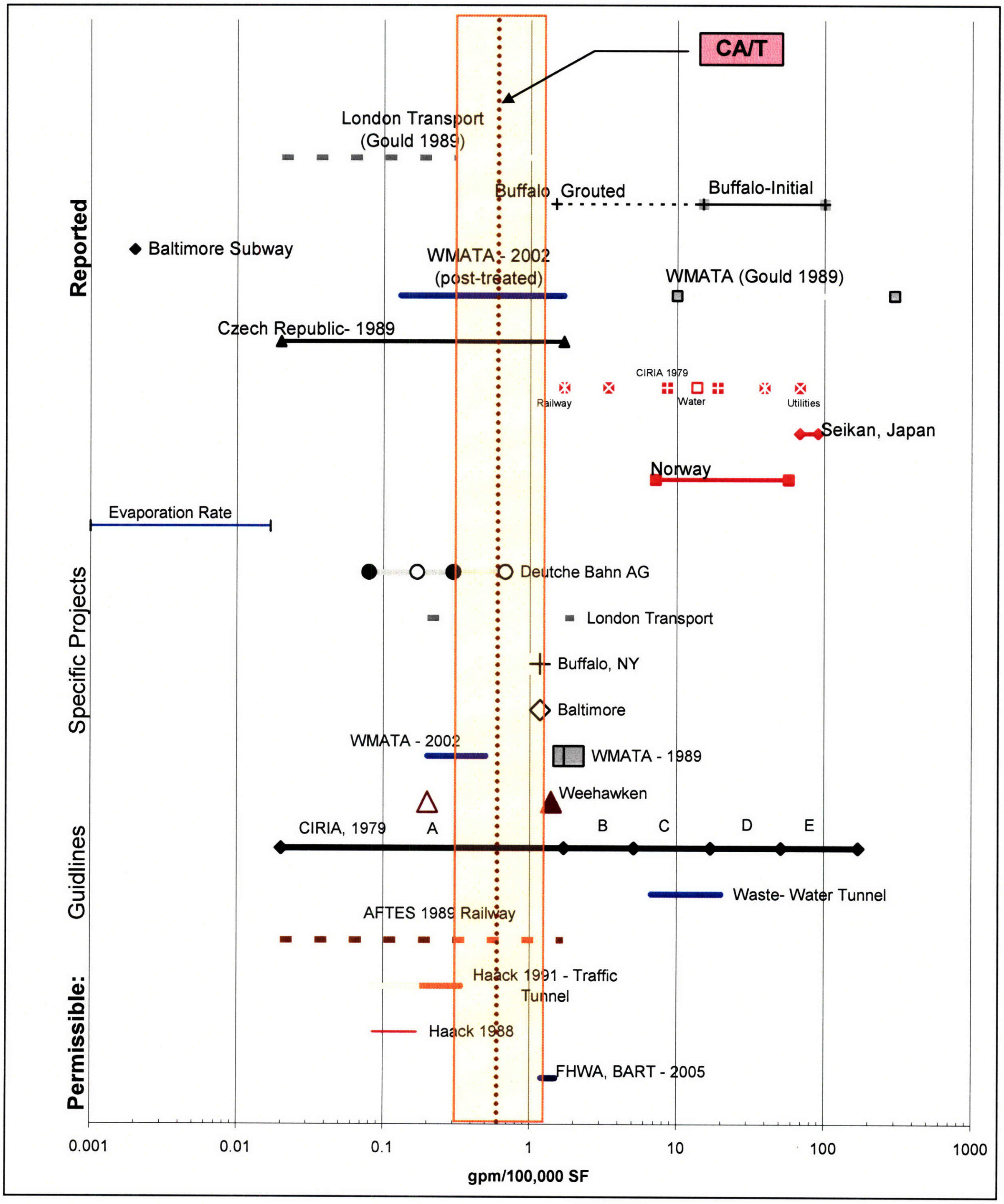


Figure 3-9, CA/T Estimated Inflow Rates Contrasted to International Rates

4. CONSEQUENCES OF LEAKS

Water always finds the weakest path with time. (Smith 2001)

Leakage in an underground structure may be in the form groundwater infiltration or exfiltration of fluid that is transported in a utility tunnel. This chapter focuses on the most common type of leakage encountered, water infiltration into structures (highway and rail tunnels). A photograph of a leak in a Red-line subway tunnel is presented in Figure 4-1, where calcification and corrosion are the two visible consequences. Groundwater inflows contribute to structural deteriorations (e.g.. cracks and loss of strength) and vice versa, cracks (caused by stresses above the allowable design) accede to groundwater infiltration and additional crack formations.

Figure 4-1, Red-line, between Porter Square and Harvard Square, Cambridge, MBTA, (Russell 2007)

Let's consider the case where the owner and the engineer have specified a permissible leakage rate for a given tunnel. However, due to various factors this rate is exceeded and design requirements are not met. The potential contributing factors causing excessive leakage are: environment, design, materials, application and the use of an underground structure. The next question to be addressed is: What are the effects of experiencing higher than expected water inflows? This chapter will review the consequences of groundwater inflows onto an underground structures and particularly tunnels. Water leakage can contribute to long term structural deterioration, damage to mechanical-electrical-plumbing (MEP) equipment. It can potentially cause cause lowering of the groundwater table and induce damage to neighboring structures and facilities, and in some case safety hazards. It can also produce a disposal problem, where large volumes of water are added to into the stormwater or sewer system.

Once a structure begins to leak, the maintenance work drastically increases. This can produce inconvenience as the structure has to be temporary closed for retrofit work or operated under restricted conditions. Both the owner/operator and the user experience monetary consequences, where the owner pays for repairs and may lose revenue (in tolls fees or rail tickets) or where the patron has to find alternative routes of transportation (e.g. for road or rail tunnel), which will add time and possibly costs in lost travel time. Additional consequences include shortening the life of the structure. Figure 4-3 shows an example of spalling, caused by ground water infiltration, which lead to structural degradation of the roof slab and reduction in the life of the structure.

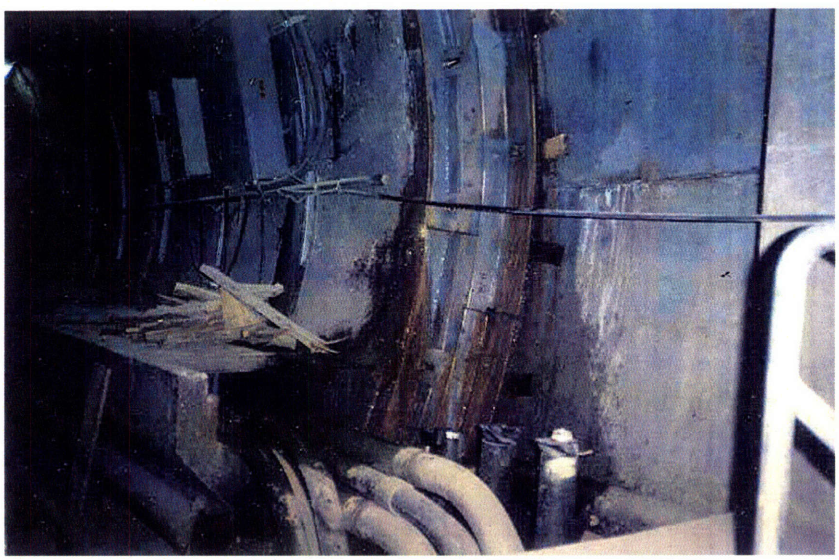


Figure 4-2, Seismic Joint, 7th Street and Flower Street, Los Angeles, MTA, (Russell)

Engineers should take the time before specifying proper waterproofing systems; decisions should be made based on a review of past project specifications and the success rate. In the UK it is no longer acceptable to plead *not guilty* if the waterproofing system does not function properly (Smith 2001).

Individual sources of leakage may be allowable within permissible ranges, however they can still cause damage to the tunnel structure and/or to the surrounding environment. Figure 4-2 shows an example of stains and potential damage to the conduits in a Los Angles subway tunnel, resulting from a leaking joint. The presence of water can cause unpleasant stains, resulting in erosion and corrosion over time. Formation of icicles, ice and water ponding will impede safety of users in a tunnel. Drops of water in a highway tunnel are unacceptable. If one has a drop of water fall on their vehicle or sees drops of water in the tunnel he/she will be greatly concerned for public safety (not knowing what are the engineering consequences of the leak or drop). Thus media and public are typically concerned with regard to public safety when water inflows are not addressed with a great deal of caution.

This chapter discusses consequences of leakages during tunnel operation and once construction is completed. A brief overview on possible effects of leakage during construction (CIRIA 1979) is also presented in Section 4.8.

Figure 4-3, Roof of Park Avenue Tunnel, New York City, (Russell)

4.1. Contributing Factors - Why Leaks Occur?

Water leakage is one of contributing factors to tunnel deformation (Asakura et al. 2001). Figure 4-4 provides a general overview of deformations and causes, which are further discussed in the preceding subsections.

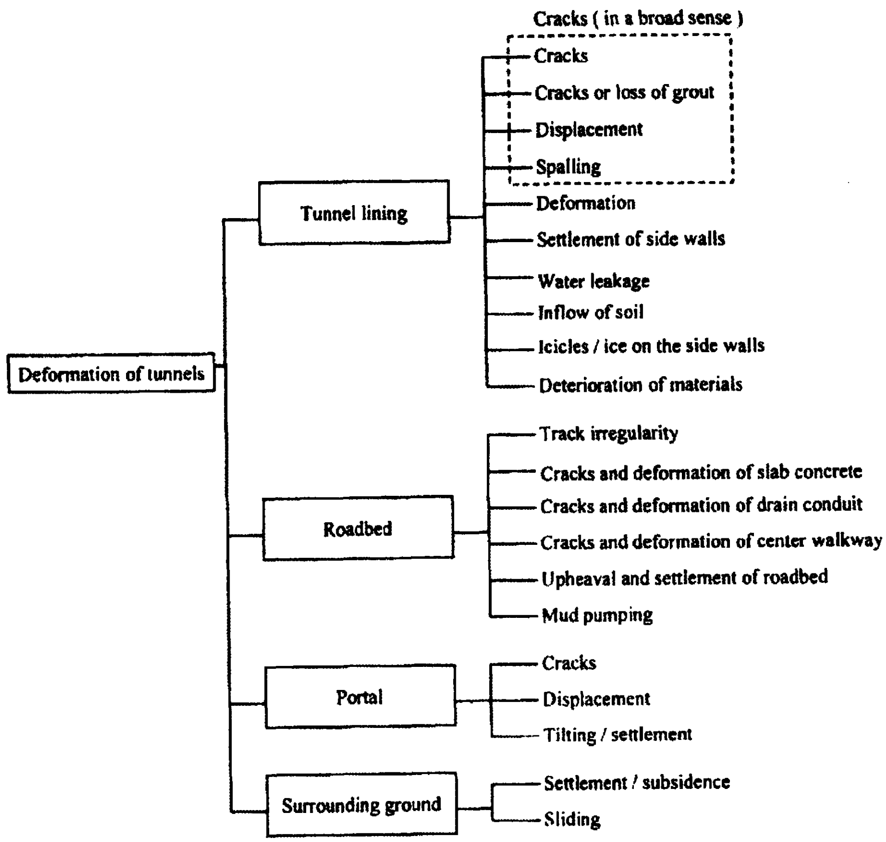


Figure 4-4, Classification of deformation of tunnels (Asakura et al. 2001)

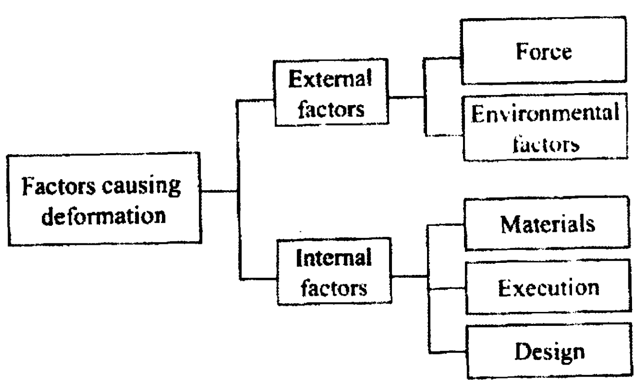


Figure 4-5, Classification of factors causing the deformation of tunnels (Asakura et al. 2001)

4.1.1. Design, Selection of Materials

Although individual materials may be excellent in their own right, it is always important to establish if they will function as specified in diverse situations and locations (i.e. material interactions needs to be considered). Other contributing factors which may be omitted during design are: material quality and consistency in addition to fatigue resistances. The location of possible movement in joints needs to be analyzed relative to the change in adjacent stratigraphy and with respect to the structure. Suitable joint materials based on short and long term properties and construction methods will affect leakage rates. For example a connection between a station constructed via top-down and a driven tunnel creates a potential source of significant leakage in WMATA.

When designing an underground structure such as a tunnel the designer must acknowledge the following:

- Concrete is not completely impermeable;
- Slurry wall concrete is not the best quality, and some flow pathways are inevitable.
- Groundwater, under pressure, will find a path to gradually permeates the liner;
- Good waterproofing is a must to obtain a water-resistant structure;
- Permeability of soil increases during construction, creating a better passage for water inflows;
- In a highway tunnel air temperature changes and circulates the tunnel numerous times per day, variation in annual ambient temperature and the exposure of the structure to thermal loading during and after construction contributes to movements (Smith 2001). When the structure wants to move but an inadequate expansion joints, cracks will form allowing for future water infiltrations. Designers often do not consider an adequate amount of expansion joints. Some structural engineers do not design tunnels as bridges or highways. Russell (2008) reports highway tunnels undergo temperature changes up to 50°F within 24 hours.
- Material should be selected based on expected deformation.
- Cut and cover tunnels can move within the fist 2 year after construction (due to re-equilibration of groundwater flow patterns and ground movements).
- Some tunnels may do not allow for thermal expansion. For example in a cut and cover SPTC constructed tunnel the concrete wants to pull apart from the fixed soldier

piles, creating cracks and possibilities for water to infiltrate between steel and concrete, if improperly waterproofed.

If the designer does not pay a great deal of attention to the above factors, the tunnels will experience groundwater infiltration.

4.1.2. Construction

Poor working conditions, productivity pressure on crews, and poor quality control may contribute to the quality of workmanship. Cure times of chemicals and concrete may be jeopardized and improper fittings of water stops are two examples of contributing factors to a tunnel with leaks. Working during extreme weather and wet conditions contribute to the outcome of a tunnel with excessive inflows. Cleanliness of the joints before filler application has to be considered for proper bonding and watertightness. Cracks are discussed in Section 4.4.2.

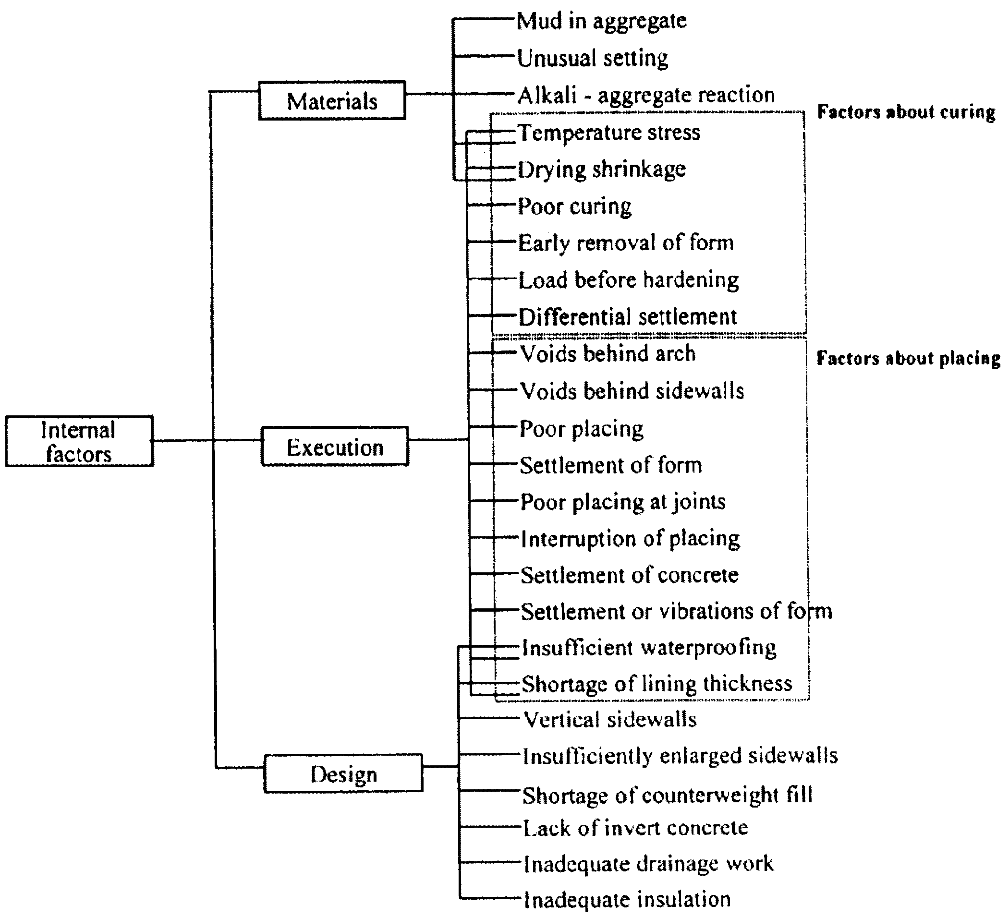


Figure 4-6, Classification of Internal Factors- example (Asakura et al. 2001)

4.1.3. Environment (External Factors)

Structural interactions with the environment and the proper design assumptions have to be made: The maximum groundwater table during a wet season has to be considered; in addition one must consider man-made fluctuations. Some tunnels will leak if the design water table is exceeded; greater hydrostatic pressures will be experienced causing stresses the liner cannot accommodate, creating cracks and passages for groundwater infiltration. Differential settlement (short and long term) must be considered because it can create movement of the structure causing water inflow.

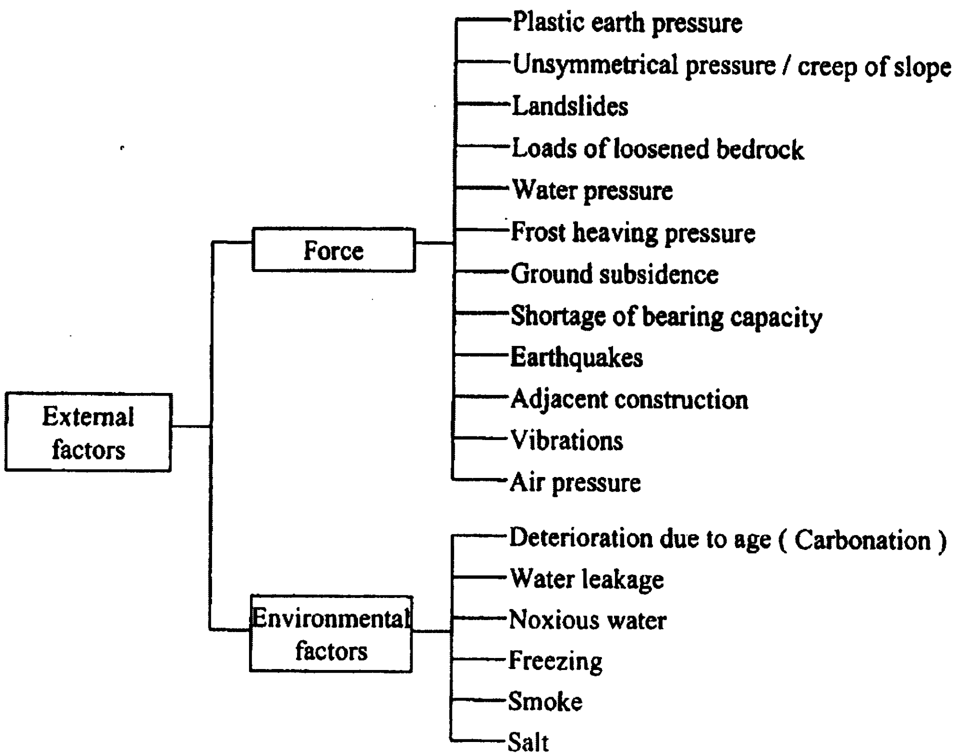


Figure 4-7, Classification of External Factors (Asakura et al. 2001)

4.2. ITA (1991) Case Studies

Report on the Damaging Effects of Water on Tunnels during Their Working Life was published by ITA (1991) to educate the engineering community about adverse effects of water on tunnel structures. The group collected case histories from around the world concerning problems caused by water originating from the surroundings of the tunnel. The following classification were established:

- ***A*** - External effects (on the surroundings of the tunnels, but not affecting the structure).

- ***B*** - Structural Effects (affecting the structural adequacy of the tunnel).

- ***C*** - Functional Effects (affecting the functional adequacy of the tunnel).

Case histories, mainly relating to ***B*** and ***C***, were collected and presented. Only few case histories for type ***A*** effect were presented, although it is known that such effects exist. Table 4-2 lists the nature of the problems identified, the category of the damage and the case histories that apply.

Table 4-1, Reference list of case histories included in the ITA (1991) Report

Reference	Country	Location of Tunnel (in or between what town(s))	Tunnel Use	Type of Lining	Ground Type	Date(s) Built	Damage Class
A1	Austria	Mittersill/Matrei	Road	In-situ concrete	Rock	1962–67	BC
A2	Austria	Golling/Werfen	Road	In-situ concrete	Rock	1962–67	C
A3	Austria	Bischofshofen/Poham	Railway	Masonry	Rock	1874	C
C1	Cuba	Havana Bay	Road	Precast concrete		1958	BC
EG1	Egypt	Ahmed Hamdi	Road	Precast concrete	Rock/clay	1980	B
UK1	England	Blackwall	Road	Cast Iron	Varied	1892–97	C
UK2	England	Southampton	Railway	Masonry	Clay/sand	1846–47	BC
UK3	England	Old Street/Moorgate	Light Railway	Cast Iron	Clay/sand	1899–1924	B
UK4	England	Blisworth	Canal	Masonry	Rock/clay	1794-05	B
UK5	England	Dartford/Thurrock	Road	Precast concrete	Rock	1972–80	B
UK6	England	Abbeystead	Raw Water	Precast concrete	Rock	1975–79	AC
D1	Federal Republic of Germany	Berlin	Road	In-situ concrete	Sand	1978	BC
D2	Federal Republic of Germany	Frankfurt	Railway	Masonry		1849	B
D3	Federal Republic of Germany	Frankfurt	Railway	Masonry	Rock	1849	B
D4	Federal Republic of Germany	Cologne	Railway	Masonry	Rock	1910	BC
D5	Federal Republic of Germany	Cologne	Railway	Masonry	Rock	1902	BC
D6	Federal Republic of Germany	Cologne	Railway	Masonry	Rock	1886	BC
D7	Federal Republic of Germany	Hanover	Light Railway	RC/Cast iron	Clay/sand	1976–85	B
D8	Federal Republic of Germany	Stuttgart	Road	In-situ concrete	Clay	1958	B
D9	Federal Republic of Germany	Nuebiberg	Road	In-situ concrete	Cut/cover	1960–61	BC
D10	Federal Republic of Germany	Hamburg	Light Railway	In-situ concrete		1960	AB
D11	Federal Republic of Germany	Bochum	Light Railway	In-situ concrete	Rock/clay	1974–77	BC
F1	France	Pagny-sur-Meuse	Railway	Masonry	Rock	1846–51	B
F2	France	Pagny-sur-Meuse	Railway	Masonry	Rock	1846–51	B
F3	France	Marseille	Railway	Masonry	Rock	1912–14	B
F4	France		Canal	Masonry	Rock	1914–45	BC
F5	France	Paris	Light Railway	Masonry	Rock	1941–48	BC
F6	France	Nice/Digne	Railway	Masonry	Rock	1880	BC
F7	France	Sabart	Hydro Gallery	In-situ concrete	Rock	1929	B
HK1	Hong Kong	Hong Kong	Light Railway	Precast concrete	Clay	1980	BC
J1	Japan	Shimonoseki/Moji	Railway	In-situ concrete	Rock	1944	BC
J2	Japan	Shimonoseki/Moji	Railway	In-situ concrete	Rock	1974	BC
J3	Japan	Shimonoseki/Moji	Road	PC/In-situ concrete	Rock	1958	BC
J4	Japan	Tokyo	Light Railway	Precast concrete	Clay/sand	1965–76	BC
J5	Japan	Mikuni	Road	In-situ concrete	Rock	1959	B
J6	Japan	Uebonmachi/Nipponbashi	Railway	Precast concrete	Clay/sand	1970	BC
N1	Norway	Oslo	Road	In-situ concrete	Rock	1970	C
CH1	Switzerland	Basel/Olten	Road	In-situ concrete	Rock/clay	1963–69	B
CH2	Switzerland	Simplon/Brig	Road	In-situ concrete	Rock	1964	B
CH3	Switzerland	Basel/Olten	Railway	Masonry/concrete	Clay	1912–16	BC
CH4	Switzerland	Andeer/Sufers	Road	In-situ concrete	Rock	1966–69	BC
ET1	United Arab Emirates	Dubai/Deira	Road	In-situ concrete		1972–75	B
US1	United States of America	Detroit, Michigan	Raw Water	PC/In-situ concrete	Rock	1948–54	B
US2	United States of America	Sterling Heights, Michigan	Raw Sewage	In-situ concrete	Silt	1970–72	B
US3	United States of America	Sterling Heights, Michigan	Raw Sewage	In-situ concrete	Clay/sand	1972–73	AB
S1	Sweden	Stockholm	Rail (Metro)	Shotcrete/unlined	Rock	1957–58	BC
S2	Sweden	Jarpen	Hydro	Shotcrete	Rock	1940	BC
S3	Sweden	Stockholm	Telecommunications	Shotcrete/unlined	Rock	1961–75	C

Table 4-2, Problems identified by the case histories, the category of damage and the case histories that apply, (ITA 1991)

Nature of Defect	Classification of Damage	Relevant Case Histories
Deterioration of mortar internally	B	J1, J2, F1, F5, F6
Corrosion of reinforcement	B	CH1, D1, ET1, EG1, J3, J4, HK1
Degradation/Reduction in strength of concrete	B	CH1, F7, J1, ET1, HK1, UK5
Swelling soil—lifting/ damage to invert	B	CH3, D4, D8, F1, F2, UK2
Erosion of mortar (masonry lining)	B	D2, D3, D4, D5, D6, F1, F2, UK4
Loss of support due to fines transport	B	US1, US2, US3, J4
Dissolution of bitumen external layer by toluol in ground water	B	D10
Rising water table—lack of tightness	B	J4
Chemical action on lining/cast iron	C	UK3, J5, J6
Frost damage and other icing effects	C	A1, A2, A3, CH2, CH3, CH4, D1, D5, D6, D9, N1, F1, F2
Damage to surface finishes	C	A2, C1, F4, UK1, S3
Corrosion of internal fittings	C	A1, A2, C1, CH3, D5, D10, D11, F4, F5, J1, J2, J3, J4, S1, S3
Salt deposits, masonry sealing	C	D11, F3, F6, S1
Clogging drainage due to fines	C	A1, C1, D4, CH3, UK2
Cracks in track/road slab	C	J1, J2, J3, UK2, UK5
Coal tar inflows	C	UK1
Ingress of dissolved gases	C	UK6
No reported damage or effect	-	D7, D8

4.2.1. Segmental Concrete Tunnels

Four out of six case studies with segmentally lined concrete tunnels had damage resulting from the infiltration of groundwater containing chlorides. In these cases, the chloride contamination of the concrete has caused electrolytic action with the steel reinforcement, causing corrosion and consequent cracking of the surrounding concrete. In case history, J4, the inflow of fines into the tunnel has led to settlement. In case history C1, drainage ducts and pumps became blocked.

4.2.2. German Road Tunnels

Case D1 a reinforced concrete tunnel constructed in 1979. A number of joints in the upper wall area as well as in the ceiling were leaking. In general, the leaks are situated above the groundwater table, potentially caused by rain and thaw water seeping into the tunnel. The effects due to inflows were: wetness, corrosion of steel reinforcement in the area of concrete crack, icicles in ceiling and wall areas, and ice formations on the road surface.

Case D9 describes a steel-reinforced concrete tunnel constructed via a cut-and-cover method in he 1960's. Transversal cracks in ceiling slab were observed causing seepage of surface water into block joint creating icing.

4.3. Affects on Functionality (Service) and Safety

The infiltration of water into the structure creates numerous operational problems to the facility. One of the primary problems is the service loss during shutdowns for maintenance and safety precautions. Leakage in rail tunnels causes ice build up on track or tunnel crown and has the potential for a train to derail. For example, ice formations (shown in Figure 4-8), have to be removed prior to operating the Kenmore Station in Boston. Water leakage and frost effect service of vehicles and roadways and may directly affect vehicles (metal corrosion). Splashing water may cause vehicles to skid (Asakura, et al. 2001).

Icecles of great size may form and may fall on vehicles due to wind, vibrations in the tunnel. The formation of icecles occurs in mountainous or high latitude environments in addition to winter climates. Icicles may form in exit shafts and potentially fall on public users or maintanence workers.

Figure 4-8, Ice formations, Kenmore Station, Green-line, Boston MBTA (Russell 2007)

Hazard (vehicle skidding and loss of friction) are generated by formed ice (black ice) on the on the carriage way in road tunnel. Ventilation and opening in shafts can be clogged with icing, at cold temperatures.

The presence of water in the exhaust plenum creates the potential for the mixing of exhaust gases to create sulfate, and sulfite compounds that will attack the concrete lining and steel supports including the ceiling supports hangers. Corrosion effects are discussed in succeeding section.

4.3.1. Mechanical and Electrical Equipment

Mechanical equipment can be affected by inflows and damp atmosphere unless protected. Groundwater leakage cannot be tolerated in any form in areas where there is water sensitive electrical, electronic, or special operation equipment. Electrical equipment may initiate combustion or explosion of gases if not isolated or properly sealed (CIRIA 1979).

Electrical conduits may transmit water into an underground structure. Outlets are often heavily calcified (depending on geology) as a result of evaporation of water at the end of the conduit and in many cases (WMATA 2002) the conduits have been completely filled with calcification due to evaporation of the water.

4.4. Structural Effects

Groundwater infiltration can affect the structural adequacy of the tunnel. AITES-ITA (2001) concluded, from 157 cases, that water leakage is the principal cause of damage to and degradation of tunnel linings. An example roof slab spalling, caused by groundwater inflows is shown in Figure 4-9.

Figure 4-9, Ashmont Station, Red-line, Boston, MBTA (Russell 2007)

4.4.1. Corrosion mechanisms

Water leakage accelerates corrosion. Bracher, et al. (2004) described three types of chloride-induced corrosion mechanisms in concrete segmental tunnels.

In deep tunnels, the rate groundwater inflow can be drastic, caused by high hydrostatic pressures. Concentrations of chlorides, sulphates and other deleterious materials may be in the groundwater. Inflows of such concentrates cause corrosion to reinforcing steel and tunnel metals, an example of corrosion is shown in Figure 4-10 and Figure 4-13.

To offset possible black ice formations in highway tunnels, maintenance crews will often use road salts. Salts will accelerate corrosion; chlorides will penetrate concrete in the road slab and concrete linings. Chlorides in road salt cause corrosion of reinforcing steel (Russell 2000). Degradation, as a result of rebar corrosion and was recorded at the Sumner Tunnel in Boston (Figure 4-11).

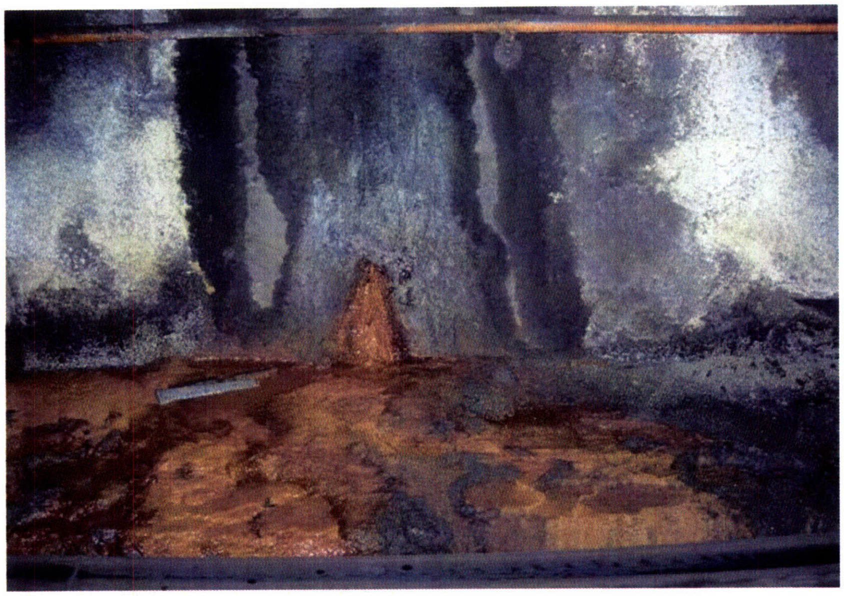


Figure 4-10, Red-line between Alewife and Davis Square, Cambridge, MBTA (Russell 2007)

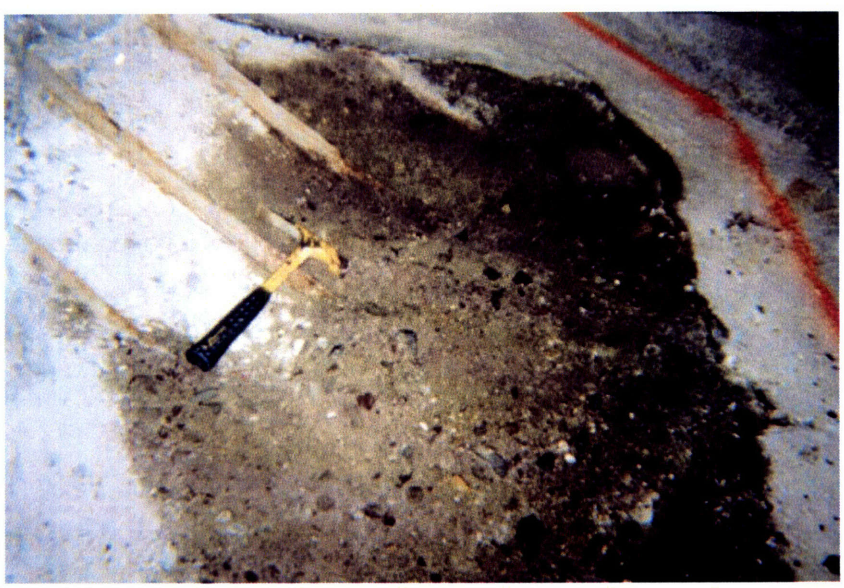


Figure 4-11, Total Slab Degradation, Sumner Tunnel, Boston (Russell 1989)

- Transpiration

  If the outer surface of the concrete is below the water table and becomes saturated, a water vapor gradient is set up between the dry interior and the saturated area. As water evaporates from the internal face, more water is drawn through the concrete and, upon evaporation; salts are deposited at a point close to the saturation front. This process can be identified as transpiration and is schematically shown in Figure 4-12. These salts, such as chlorides and sulphates, can build up their concentrations and cause corrosion of reinforcement and attack the concrete paste matrix.

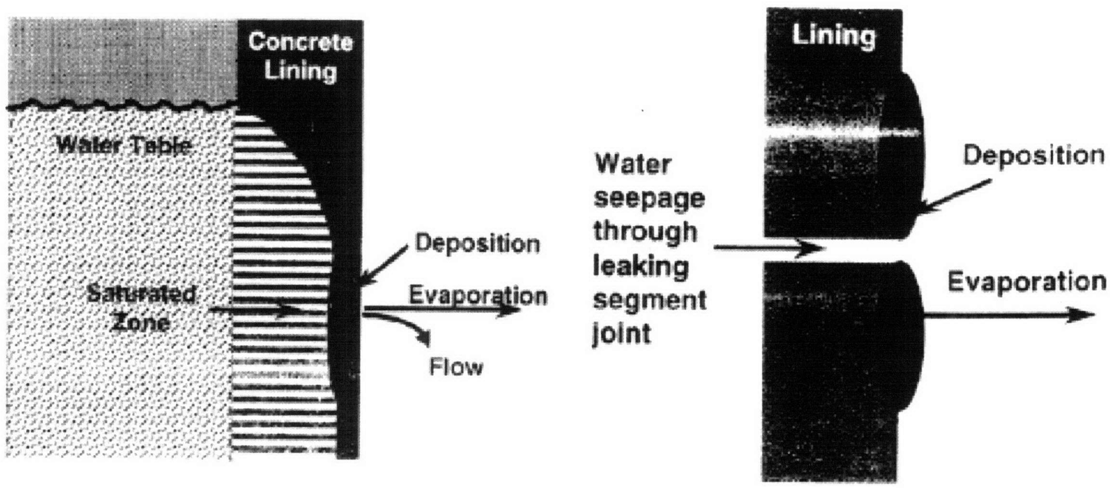


Figure 4-12, Transpiration Schematic (left), Leakage through Joints and Cracks (right), (Bracher, et al. 2004)

- Air-Borne Chlorides

  In marine and coastal environments air-borne chloride ions contribute to corrosion. These salts are transported through ventilation systems and movement of vehicles (in highway tunnels). In railway tunnels, high air pressure, created by moving trains, forces chlorides and others deleterious material into the lining. (Bracher, et al. 2004)

  Once the temperature of the surface of the tunnel lining falls below the dew point, moisture will condense on the surface of the concrete and the chlorides will go into solution. As the dew evaporates, deposits of chlorides will be left on the concrete surfaces and build up in concentration, due to cyclic wetting and drying.

The sealing of segmental joints relies upon compression seal, or gaskets. When gaskets are misaligned during construction, a leaking joint will be created. When leaks occur through liner cracks or joints, the groundwater will penetrate on the inner face of the tunnel and be absorbed into the concrete. As the inflow water evaporates, deposits of chlorides are left behind. Cycles of wetting and drying will lead to build ups of high concentrations of chlorides and sulphates. Back diffusion of chlorides into the concrete from the internal face of the lining will occur, with the resultant corrosion of reinforcement and spalling concrete cover zone.

Bracher, et al. (2004) states that this type of corrosion mechanism is the fastest acting and most damaging. Corrosion damage is encountered in tunnels constructed less than a decade ago in Asia.

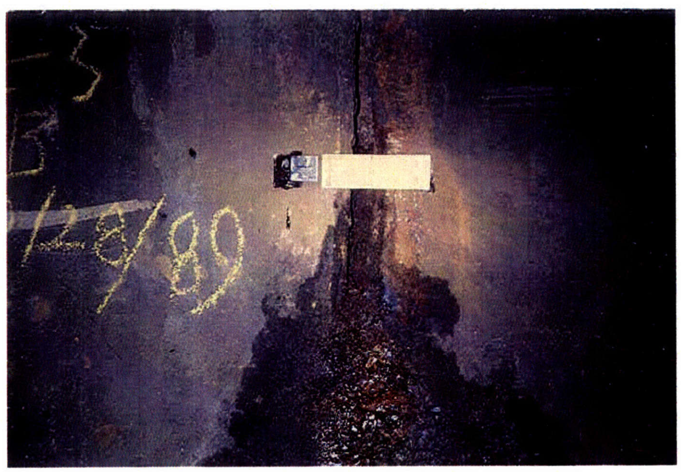


Figure 4-13, Crack Monitoring Gage, Chicago (Russell 1989)

- Electrified Tunnels

  In electrified tunnels, the electrolysis created by the stray currents can accelerate the corrosion of the electro-mechanical sub system (Russell 2000).

4.4.2. Construction and Cracks

The most common cause of leakage in cast-in-place lining is due to cracks that develop from shrinkage of concrete during curing. Construction joints and areas of poor workmanship (e.g.. honeycombing) are also areas of concern. Leakage to or from these linings will cause deterioration of the reinforcing steel and subsequent spalling of the concrete, as discussed in the following sections.

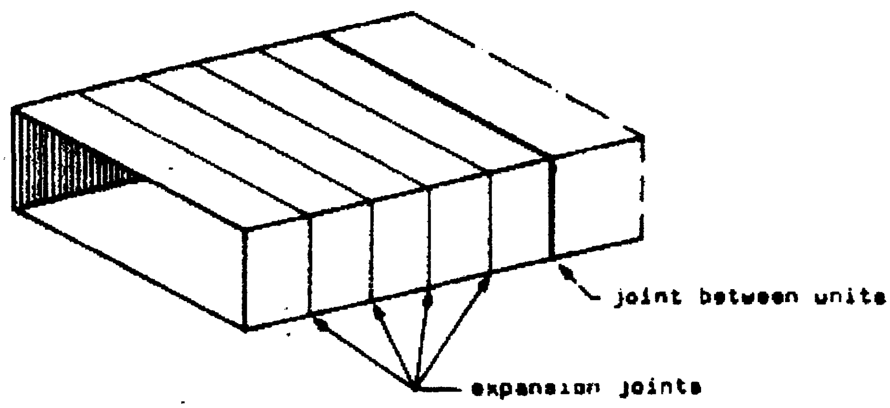


Figure 4-14, Scheme of Immersed Tunnel, (Janssen 1978)

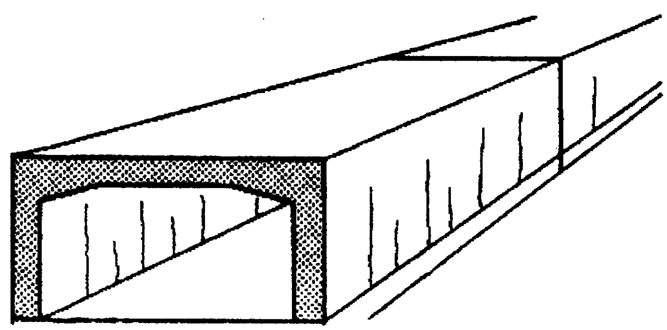


Figure 4-15, Cracks in the tunnel walls caused by temperature influence, (Janssen 1978)

Figure 4-14 shows an immersed tunnel with units of 70-120 m., and expansion joints at 20 m. centers. Cracking in the concrete can originate from:

- Dehydration

  When the tunnel segments are still in the dry dock, micro-cracks may appeal on the outside (Figure 4-15). After the immersion and placing of the segments the crack will be closed again by the swelling of the concrete in water (Janssen 1978).

- Temperature changes

During a change in ambient temperature, a temperature gradient will occur in the walls, the roof and the floor. Concrete hydration heat is due to a chemical reaction that takes place during casting of new concrete. During tunnel construction, the hydration can cause two following problems:

(1) Higher temperature will arise in the core of the floor, the walls and the roof than in the outer layers of these segments. This phenomenon will lead to a temperature gradient, cooling down of the outer layers will cause the concrete on the outside to shrink at a higher rate than the concrete in the core. Consequently tensile stresses will occur creating potential cracking.

Cracking as a result of rapid temperature change, *thermal shock*, will result when formwork is removed (in-situ or precast concrete) from a concrete wall. Thermal shock occurs when a thermal gradient causes different parts of a given structure to expand by varying amounts. Cracks form at a given point in a structure, when thermal stress exceeds the strength of the material (tensile strength of concrete). For example: during the night time the surface temperature of the concrete can decrease considerably. Janssen (1978) states and confirms by experience that in concrete tunnels, a temperature difference of 15-20 °C causes cracking. The stress zone expands during cooling down, a crack will progress further inward; and vice versa (when temperature in the core decreases, the crack will close).

(2) During the pouring of the wall upon the floor, difficulties arise because the fresh concrete will note behave in the same way as the older concrete of the floor. The deformation of the new mass of concrete is interfered with by the floor slab. In the wall, hydration will increase the temperature. The heat dissipation to the floor is only limited and the floor temperature will lag behind. At first the wall can freely expand but in the process of hardening of the concrete, the cohesion with the floor will come into play. When temperature increases in the wall due to hydration, compressive stresses will occur in the wall and tensile stresses in the floor. During cooling down, the opposite takes place: compressive stresses will be generated in the floor and tensile stresses in the wall, leaking to cracking as shown in Figure 4-15. This phenomenon will occur when the wall cools down first.

4.4.3. Micro-Cracks

Micro-cracks are typically generated by thermal stresses triggered by construction and formwork removal. Micro-cracks allow penetration deleterious substances and expose more surface area to attack, resulting in concrete deterioration (following section 4.4.4) which causes increase in liner permeability. Micro-cracks greater than 80 microns in width will cause an increase in the water permeability of concrete (Bracher, et al. 2004).

4.4.4. Deterioration (aggressive water, freezing, etc.)

Ambient water, humidity, frost, salinity, and chemically aggressive water, and freezing can all cause deterioration to a structure. Water flows are usually the source of deterioration due to the progressive physical decay produced by the circulation of water and chemical action that may be present (e.g. sulphate water) (Pelizza, et al. 2001). The inflow of acid water along with cycles of drying and wetting (discussed in preceding sections) contribute to deterioration, leading to concrete spalling and reducing the strength of the liner. Spalling may not be immediate problem. However deterioration in the long-term will lead to spalling.

Leakage in rail tunnels causes tie deterioration, corrosion (Section 4.4.1) of reinforcing steel. Water leakage accelerates corrosion to rails, metal plates and shortens replacement periods. Figure 4-17 shows a subterranean parking garage in Boston, where aggressive water inflows may have caused deterioration to some the steel plates and utility conduits.

4.4.5. Mechanics of Spalling

Spalls are flakes of a material that are broken off a larger solid body and can be produced by a corrosion and/or weathering. Spalling is the process of surface failure in which spall is detached. Spalling of concrete can lead to loss in structural integrity of the structure and significant shortening of the life of the structure and create overhead safety hazards (e.g. Figure 4-18). A schematic example of a spalling mechanism is shown in Figure 4-16.

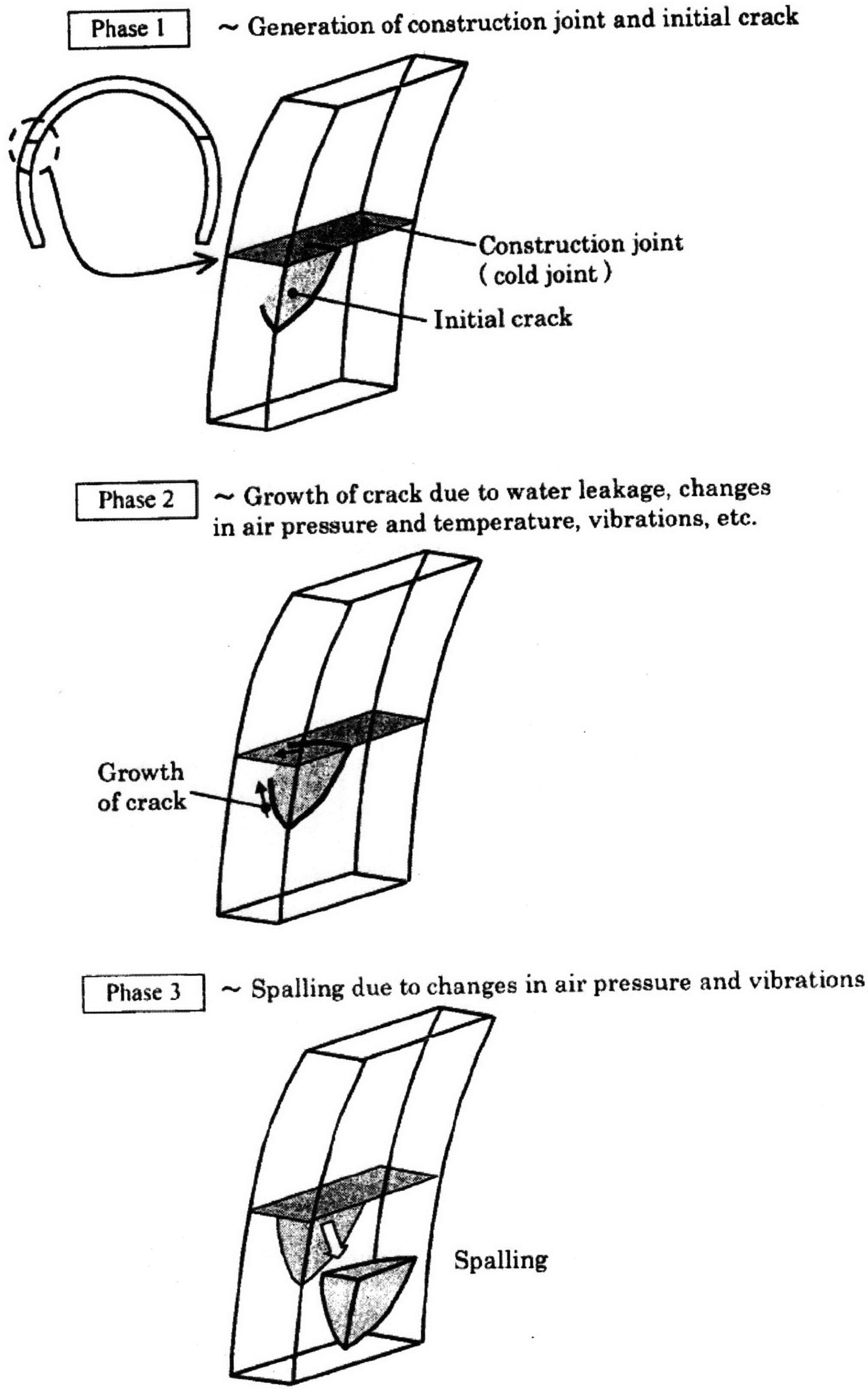


Figure 4-16, Mechanics of Spalling in the Fukuoka Tunnel, (Asakura et al. 2001)

Figure 4-17, Waterfront Underground Parking Garage, Boston, MA, (Russell 2007)

Figure 4-18, Concrete Spall Fell on the Track at Ashmont Station, Red-line, Boston MBTA, (Russell 2007)

Spalling in Mechanical Weathering

Spalling is a common mechanism of weathering in concrete, and occurs at the surface when there are large shear stresses under the surface. This form of mechanical weathering can be caused by freezing and thawing, unloading, thermal expansion and contraction or salt deposition. Freeze-thaw weathering is caused by moisture freezing inside cracks in a liner. During the freezing cycle the volume expands, causing large forces which cracks spall off the outer surface. As this cycle repeats the surface repeatedly undergoes spalling.

Fire typically causes damage of the concrete lining due to spalling. The spalling is due to the fast rise of temperature combined with the humidity of the concrete and the structure of its pores. From 100°C onwards the water entrapped in the pores of the concrete transforms to vapor, whose increased pressure spalls the concrete (Kolymbas 2005). In contrast rocks do not conduct heat well, so when they are exposed to extreme heat the outer most layers becomes much hotter than the rock underneath causing different thermal expansion. This differential expansion causes spalling. This mechanism of weathering causes the outer surface of the rock to fall off, it can be analogues to concrete liners in extreme weather conditions.

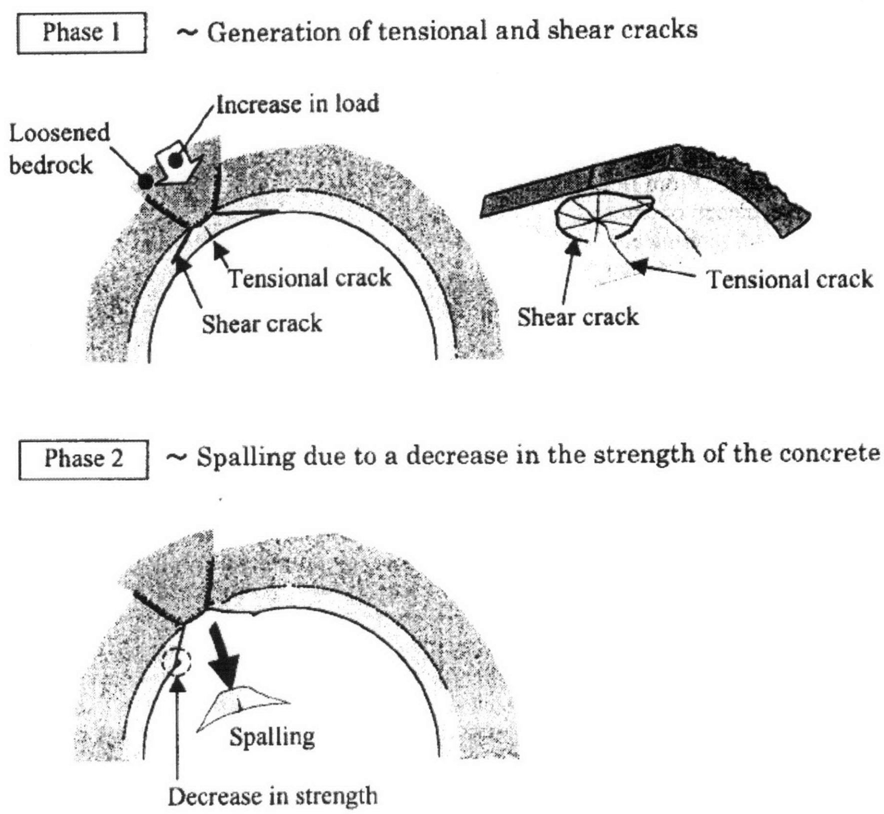


Figure 4-19, Mechanism of Spalling in the Rebunhama Tunnel, (Asakura et al. 2001)

Salt Spalling

Salt spalling occurs in concrete as dissolved salt is carried through the material in water and crystallizes inside the material near the surface as the water evaporates during the drying cycle. During the crystallization process, the salt expands within the cracks and creates shear stresses which break away spall from the surface.

Corrosion

Spalling occurs due to corrosion when a concrete or steel discards particles of corrosion products as part of the corrosion reaction progresses. A large volume change occurs during the reaction process. Rebar can expand drastically contributing to spalling of the concrete cover layers (in reinforced concrete liners). Figure 4-21 shows an example of spalling due to rebar corrosion in a Boston subway tunnel.

Figure 4-20, Emergency Exit Stairwell, Andrew Square, Red-line, Boston, MBTA, (Russell 2006)

Figure 4-21, Red-line, Boston, MBTA, (Russell 2006)

Case Studies

Incidents of ground collapse in the upper parts of tunnel have taken place in Japan (e.g. Oyamano Tunnel). In 1999, spalling incidents occurred in Fukoka Tunnel and Kita-Kyusyu Tunnel along Shinkansen Line, and Rebunhama Tunnel – Muroran Line. In all the spalling accidents the cracks progressed gradually over several years, where cracks originally formed and then dropped. A schematic of the spalling mechanism in the Rebunhama Tunnel is presented in Figure 4-19 while a local example in Boston, Sumner Tunnel collapse is shown in Figure 4-22.

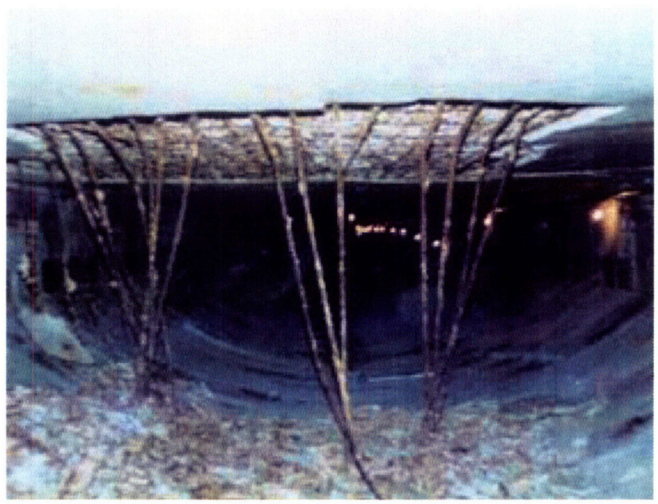


Figure 4-22, Sumner Tunnel, Boston, (Russell 1989)

Water leakage along cracks, cycles of drying and wetting, and cycles of freezing and thawing are the cause of crack propagation. In the Shinkansel line variations in air pressures and vibrations due to train are also contributing factors.

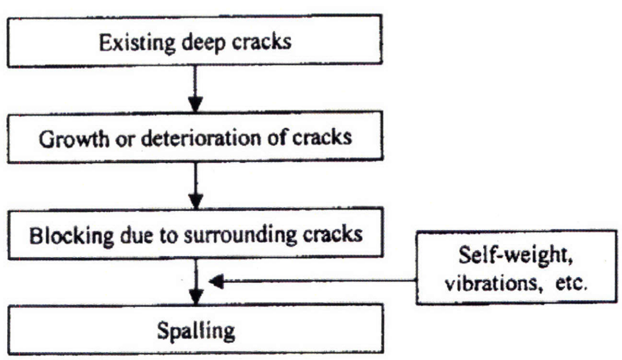


Figure 4-23, Mechanics of Spalling, (Asakura et al. 2001)

Fukuoka Tunnel, constructed in 1975, a rail tunnel with a total length of approximately 8.5km constructed via cast-in-place reinforced concrete. The tunnel depths are in the order of 100m with schist geology. In 1999 a power failure occurred inside the tunnel causing a train to stop. A lining block spalled. Cracks formed over a wide area inside the lower portion of the arch due to vibrations of the supports. Additional contributing factor was the quality of construction, where cracks formed due to removal of form and insufficient formwork cleaning with a lack of a release agent. Water leakage over a long period of time and varying temperatures, air pressure changes, train vibrations, caused the cracks to gradually propagate. Finally, the lining block fell due to train vibrations. Aschematic of the spalling mechanism is shown in Figure 4-16 (Asakura et al. 2001).

4.4.6. Drainage System

A section of a subway tunnel (Seoul, Korea), 870m long with a horseshoe shaped crossecton (8.2m wide), constructed via drill and blast, within a granitic rock terrain. High groundwater pressures formed onto the secondary liner due to the malfunctions of the drainage system. The change in crack displacement corresponded to the groundwater pressures. Cracks contributed to inflow of groundwater.

If a given tunnel design incorporates a drainage system to relieve hydrostatic pressure, and such system fails, the liner will experience excessive hydrostatic pressures. These pressures may be greater than the design allowable, causing the liner to potentially fail. The drains may fail due to clogging by groundwater. Clogging of drains due to calcite deposits can present a long term maintenance problem. This can affect pressure relief drains, as well as drainage systems inside the tunnel.

4.5. External Effects - Lowering Groundwater Table

Some tunnels use drains as mitigation measures in reducing the hydrostatic pressures, which contributes to a lowering of a groundwater table. Groundwater inflows (unintentional leaks) into tunnels can reduce the volume of local water supplies. Lowering the groundwater levels in soils can cause consolidation and settlement. Trees and vegetation might be adversely affected. The reduction in the water table level as a result of the introduction of a new tunnel is likely.

4.5.1. Consolidation, Settlement and Long Term Affects

Lowering the groundwater table potentially impairs stability of adjacent structures due to consolidation of clays (in certain geologies). Ground movements and surface settlement can occur (Wood 2001). Long term movements are generally associated with ground consolidation caused by changes in effective ground stress and associated with flow of water towards the tunnel. Piezometric changes need to be evaluated in calculating long term consolidation settlement. Any inflows of water into a tunnel may cause ground movements effecting structures.

4.5.2. Kennington, South London

Gourvenec et al. (2005) reported a field study finding around an approximately 80 year old tunnel in London Clay at a site in Kennington, South London. A borehole investigation was performed incorporating a program of in situ pore water pressure monitoring, and laboratory triaxial testing. The presence of a tunnel beneath the site allowed the investigation of its influence on the local soil and groundwater conditions. The London Clay is up to 150 m thick in areas, but within central London depths between 30 and 100 m are more common. Variation of pore pressure in the London Clay versus the distance from an old tunnel was examined. One may predict that long term seepage through the segmental tunnel lining will cause a reduction in pore water pressures in the near vicinity. Gourvenec et al. (2005) verified the previous statement by measuring pore water pressures close to the tunnel. Thus an investigation of stress changes during construction and consolidation period were conducted, this was done by looking at variations in the geotechnical properties of the London Clay within the vicinity of the tunnel.

Gourvenec et al. (2005) concluded that the presence of the tunnel had an influence on the local pore water pressures. However minor evidence of groundwater seepage towards the tunnel was evident from the piezometer data. A slight reduction in pore water pressures was recorded by the piezometers installed behind the lining. Gourvenec et al. (2005) implied that far-field conditions are reached within about 1.5 m of the tunnel. In fact Gourvenec et al. (2005) observed unexpectedly high pore pressures adjacent to the tunnel lining.

The geology at Kennington potentially resulted from the higher permeability associated with the sandiness of the clay (less permeable tunnel relative to the usual London Clay). The overall conclusion was that tunnels running through very low-permeability London Clay may act as drains, while others such as the Kennington section (within in the basal London Clay deposits) appear to be effectively impermeable.

4.6. Ground Loss into Tunnel

It is difficult for soil particles to flow inside and through the tunnel liner; groundwater does not have enough velocity for such internal erosion. This phenomenon may occur when soils will flow into drains or into a highly permeable liner.

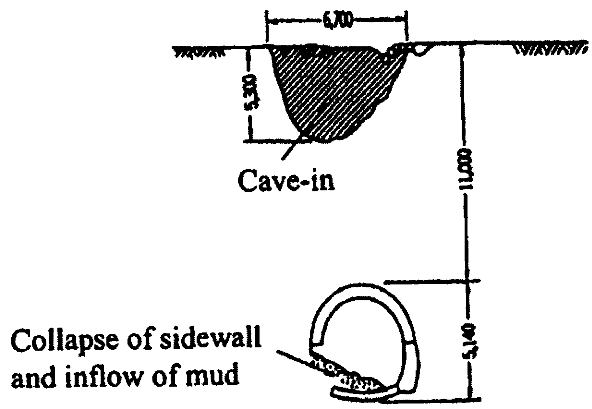


Figure 4-24, Cross Section of Iwamoto Tunnel, (Asakura et al. 2001)

When the soil in the surrounding ground flows into a tunnel along with the leakage water the drainage functions decreases (clogged drains). This causes mud pumping and deterioration of the road bed (Asakura et al. 2001). Voids are formed under the invert slab (highway tunnel) and potentially influence stability of vehicles in motion.

Inflows of water can cause loss of ground into the tunnel, either as suspended solid or in solution, impacting the permeability and the stability of the ground and neighboring structures.

A breach/leak occurred in September 2004 in the CA/T tunnel slurry wall. Groundwater and saturated sand entered a through-wall void and caused flooding on the right two lanes of the roadway. Accumulated sand was removed from the walkway, roadway, and drain lines. The soil was described as a uniform clean, fine sand. A layer of sand approximately 18 inches deep was reported in the void space between the architectural panels and the slurry wall for a distance of 20 ft on either side of the leak. The wall breach was described as an estimated 12-in by 12-in square hole at the slurry wall face (Figure 4-25) (FHWA 2005; Christian 2007).

a) Slurry Wall breach

b) Sand in roadway

c) Breach in Slurry Wall Panel

d) Flow Rate Dissipating

Figure 4-25, CA/T Slurry Wall Breach – September 2004, (FHWA 2005)

Tokyo Electric and Power Company used Iwamoto tunnel (Japan) this utility tunnel as tailrace, downstream part of a dam. The tunnel length is 438m with horseshoe cross-section, 4.24m in height and width. The tunnel was built in 1923 with unreinforced concrete. The tunnel passes through bedrock, where overlaying layers of cobble are at the surface. The tunnel is on average 1.5m below the top of bedrock and 11m below the grade. In 1990 the left sidewall failed in a trapezoid shape, along a construction joint, and the ground surface of the tunnel caved in (Figure 4-24). The results of inspection concluded that the accumulated underground water penetrated thought the ground and washed away the fines where weathering had been progressing. Due to wash-away of the fines, voids were formed behind the sidewall. The flow-in gravel and water pushed against the sidewall and failed it along the construction joints. The soils flowed into the tunnel space, collapsing the ground surface.

4.7. Disposal of Large Inflow Volumes

Environmental regulating agencies (e.g. EPA) may not allow large volumes of inflow water permanently pumped. Lowering the groundwater storage supply is one of the greater concerns in addition to causing potential consolidation and settlement.

4.8. Possible Effects of Leakage during Construction

Large inflows of water can cause decreased efficiency in working; can wash away caulking, sealing for grouting, grout, the cement constituent of partially set concrete; can make joint surfaces wet and dirty (and thus prevent the adhesion of sealants); and can burst open caulked or sealed joints while they are only partially set (CIRIA 1979). In some areas groundwater inflows might be polluted. Such flows will requite proper collection disposal and transfer.

5. MITIGATION METHODS

There are a range of mitigation measure that can be used to mitigate leakage in operating underground structures and tunnels. Prior to commencing any significant mitigation measures an inspection and investigation must be performed to determine the sources of inflow, temperature, ingress quantity and quality of inflow (e.g. pH and conductivity). The material selected for treatment schemes must be compatible with the surrounding environment. This chapter addresses the control and elimination of ingress groundwater during the operational life of the structure. The treatment of inflow during construction is described in detail by Kolymbas (2005) and Powers (1992).

Operational costs (e.g. pumping discharge costs) and maintenance costs are the main motivator for decisions to mitigation leakage problems. Contractors and owners will often perform mitigation measures to meet the required permissible leakage specifications. The majority of mitigation measures require a great deal of skilled labor and experience to improve watertightness. The success rates, based on over one-hundred case studies, are discussed in this chapter.

5.1. Leakage Control and Sealing Methods

Groundwater infiltration into a tunnel is the principal cause for deterioration of tunnel liners, proper mitigation of leaks is vital in the protection of the underground structure. AITES-ITA (2001) identified the four following categories of repairs required to control or eliminate the leakage:

Surface Sealing Methods: Applied to the inner surface of the tunnel lining, becoming a part of the lining surface. Surface sealing methods are only feasible with very low infiltration rates.

Conduction Methods: Applied inside the tunnel, where it is acceptable to allow controlled drainage or channeling of the water towards the tunnel invert and along the tunnel toward a sump for disposal.

Lining Reinstatement: Measure taken to establish or re-establish the impermeability of the tunnel lining.

Eliminator at the Source: Measure undertaken outside the tunnel lining within the surround ground mass.

The methods above may be used in conjunction with one another and additional methods (depending on the specific situations encountered). Some methods may not be classed in the above categories. The lining type will control the implementation method, however the categories above will assist one in selecting the appropriate mitigation measures.

Thermal and frost damage may be reduced and eliminated by the use of heat. Surface sealing methods involve simple applications in accordance with manufactures' specification. The other three categories of repair methods require significant skill and expertise.

Basic ideas for counter measures against water leakages through tunnel linings are presented in Table 5-1, countermeasure against frost and spalling are presented in Table 5-2 and Figure 5-1, respectfully. Countermeasure against leakage, frost and spalling will help mitigate the damage consequences addressed in Chapter 4.

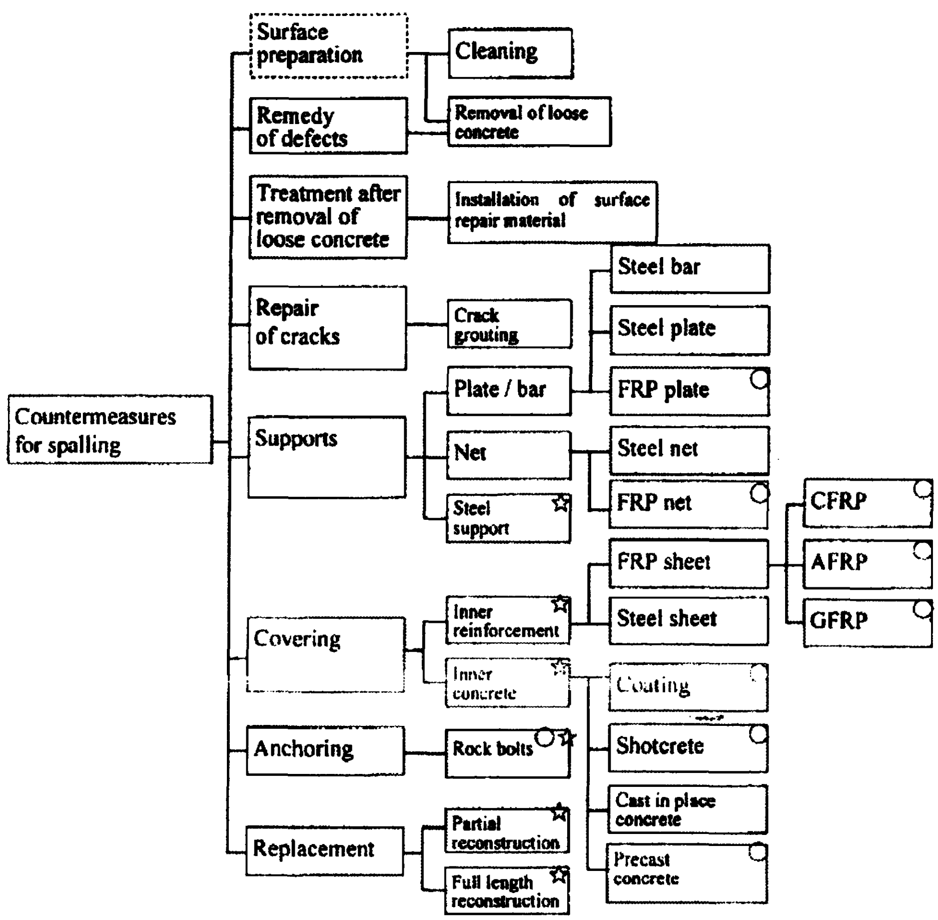
Table 5-1, Leakage Control Methods in Linings (no leaking due to freezing of water), (Akasura 2001)

Factors for selecting appropriate methods / Methods	Form of leakage [1]	Linear				Planar				Remarks
	Amount of leakage [2]	Small		Large		Small		Large		
	Allowance of inner section [3]	A	U	A	U	A	U	A	U	
Measure to control linear leakage	Water conveyance gutter	O		O		Δ		Δ		Applied for linear leakage such as the one at the joints of linings
	Grooving		O	O	O		O		O	Either V- or U-shaped grooves are cut. When a V-shaped arch is constructed, care should be taken so that no capping material may spall. This method is also used prior to a planar leakage control method.
	Flow cut-off	Δ	Δ							Either grooving and capping, or pointing is used. Applicable only where the leaking is only dripping, and the area of leakage is limited. (Care should be taken to prevent the dropping of
Measure to control planar leakage	Shotcreting					O		O		Combined use of wire netting, anchors and water conveyance channels (grooves) is necessary (for preventing the dropping of material).
	Coating					Δ	Δ			Applicable only where the leakage is of a minor degree (care should be taken to prevent the dropping of material)
	Waterproof board							O		
	Waterproof membrane					O		O		Applied in such cases as inner lining and renovation
Back grouting				O	O			O	O	Applied where the earth cover is small, and surface water or rainwater flows direct into the tunnel via a cavity behind the tunnel as if it were a
Lowering of water level				O	O			O	O	Applied where the groundwater level is high, and earth materials are discharged due to seepage or cyclic loading by vehicles, causing structural damage to the tunnel

Table 5-2, Freezing damage control methods, (Akasura 2001)

Factors for selecting appropriate methods / Methods	Form of leakage and freezing[1] / Allowance of inner section[2]	Linear A	Linear U	Planar A	Planar U	Remarks
Thermal insulation	Insertion of U-shaped insulating materials	○*	○**			※ Effective where the temperature is not so low ※※ A cut wider than the width of a U-shaped groove is required for inserting the insulating materials.
	Surface insulation	○		○		
	Double-coating insulation			○	○	Adopted in combination with renovation where reinforcement of the lining is necessary.
Heating (by electric heater)		○				Power source is required where the temperature is extremely low and leakage occurs locally.

Note 1: Form of leakage and freezing
Linear: Leakage or freezing is distributed linearly.
Planar: Leakage or freezing is distributed in a plane.
Note 2: Allowance of inner section
A(Available): Lining surface has an allowance for applying insulating materials
U(Unavailable): Lining surface has no allowance for applying insulating materials



○ New materials and methods being developed and applied; ☆ Applicable as a countermeasures for earth pressure.

Figure 5-1, Countermeasures for spalling, (Akasura 2001)

5.2. Conduction of Water Leakage

This method involves the treatment of leakage through a tunnel lining by drainage, channeling and disposal of water. This method can be implemented if the tunnel operations and the structural integrity are not jeopardized by the installation of waterproofing system.

The inflows are typically collected at the invert, and than channeled to a sump for disposal, ITA-AITES (2001) examined twenty-four (24) case studies subdivided into three categories:

5.2.1. Channeling of Leakage Water

This repair procedure involves installation of drains and gutter (materials: steel, fiberglass, or PVC) at the leaking crack or joints of a tunnel, to create a channel and capture the water directing it to the drain at the tunnel invert. Edges of the drain may be sealed by compression and caulking compound or by the use of adhesives. There were 9 cases studies for this repair method, these include two cases that incorporate radial drainage pipes drilled through the lining into the surrounding ground. Strainers to filter the water and prevent drain blockage were incorporated. This method is only applicable to a structure where the primary waterproofing system is a concrete liner (no water proofing membrane systems are implemented). According to AITES (2001) these systems can be installed in areas of localized leakage. Construction and installation can cause minimal and no interference with the tunnel operations and the public. The effectiveness of the systems are dependent upon the quality of the sealing of the gutter and drain joints and edges. Drainage channels can become clogged with fines or calcitic deposits, while those located along sidewalls and in the road slab can be susceptible to vehicular damage. Water in the drains may freeze (freezing temperatures) causing movements within the drain. Gutters and drains that extend beyond the inner tunnel lining are often intrusive and highly visible and may impede other maintenance and operations.

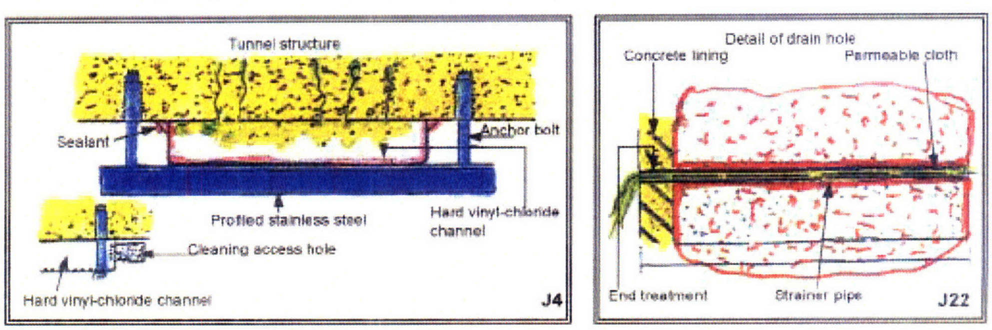


Figure 5-2, Channeling of Leakage Water, (ITA-AITES 2001)

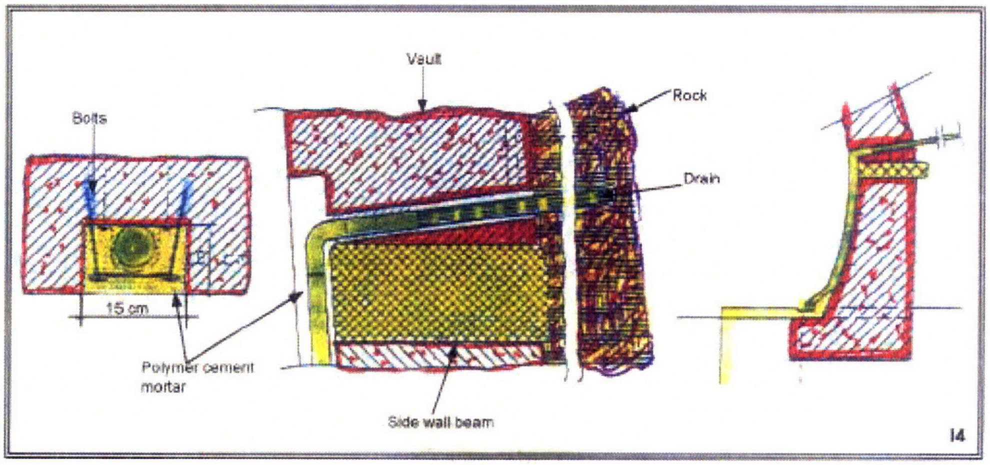


Figure 5-3, Channeling of Leakage Water, (ITA-AITES 2001)

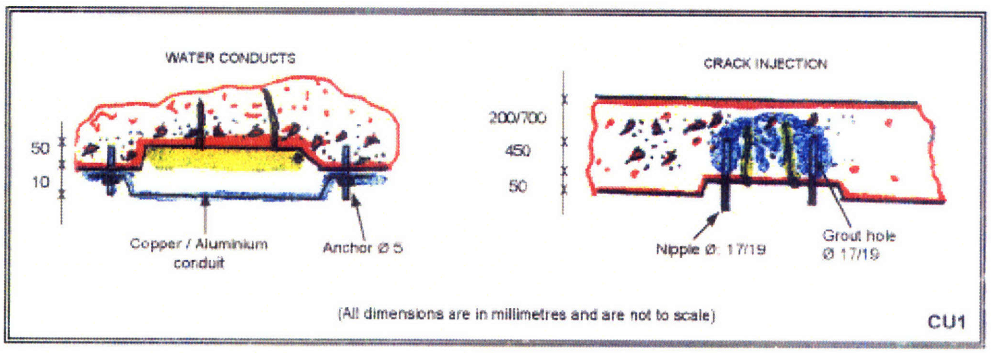


Figure 5-4, Channeling of Leakage Water, (ITA-AITES 2001)

ITA-AITES (2001) proposed the following measures to improve efficiency of the system:

- Proper sizing of channel cross section to accommodate calcitic deposits that buildup and sedimentation of the solids in the drain water;
- Appropriate selection of materials and location of the gutter and drains for specific tunnel conditions;
- Providing for easy maintenance of drain for long term effectiveness;
- Minimal use of gutters and drain in horizontal locations;
- Use of radial drain through the liner where leakage through the liner walls is encountered.

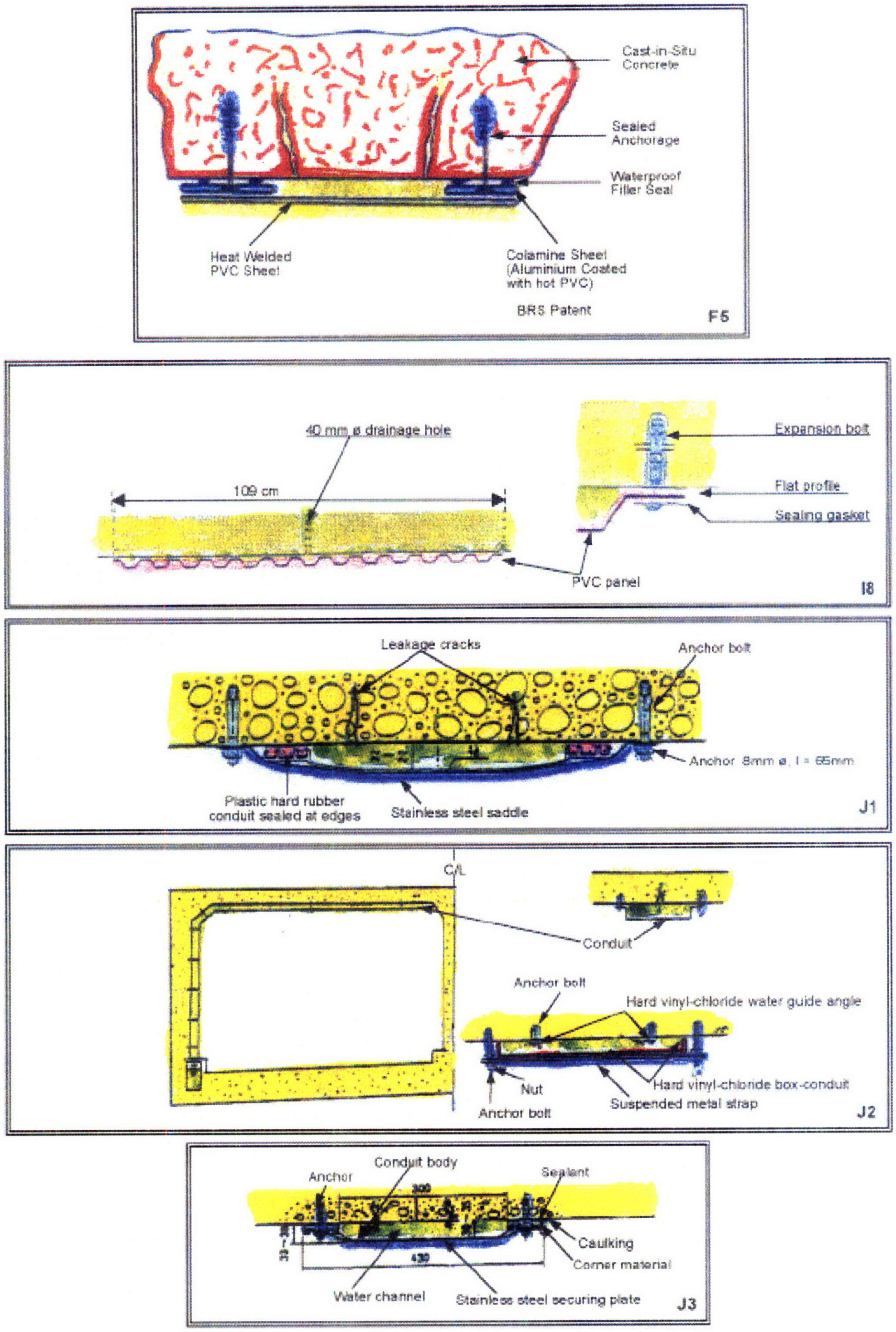


Figure 5-5, Channeling of Leakage Water, (ITA-AITES 2001)

5.2.2. Inner Shell

This technique uses a waterproof sheet membrane attached to the existing inner liner surface, which is experiencing inflows through a network of dripping cracks or joints. The proposed membrane will drain the water to the tunnel invert and the collecting system. Haack (1991) recommends an umbrella-like drain-off device, which captures the leakage water and transfers it to the tunnel drainage system. Thermal protection may be provided by the membrane layers, to limit ice formation. Mechanical damage, fire and frost must be considered and are addressed with an interior shell (in-situ concrete, shotcrete, etc.)

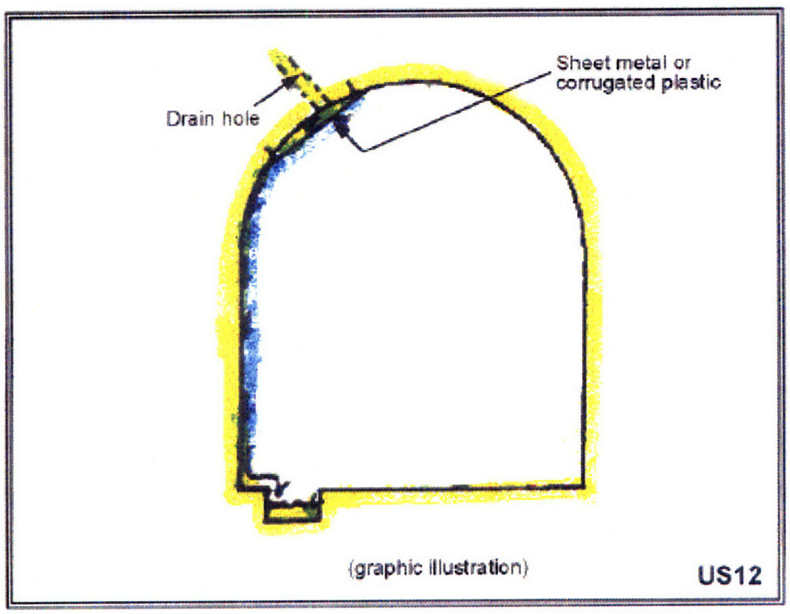


Figure 5-6, Inner Shell-Inflow Collection, (ITA-AITES 2001)

PVC or HDPE waterproofing membranes can be fixed onto the intrados without perforation. Construction can be accomplished by heat-welding the sheet onto synthetic washers (nailed prior to membrane installation). Another alternative are a semi-rigid foams, which may be fixed by anchors passing through the membrane, provided the waterproofing is maintained at each perforation by a mechanical device or sealing of attached elements. These operations are complex and require skilled workers.

ITA-AITES (2001) suggests the following measures to be addressed for material construction of the inner protective shell:

Shotcrete: It is difficult to a achieve good bond between the shotcrete and the synthetic sheets in a shell form. The shotcrete must be rapid set using accelerators and/or special rapid hardening cements

Frame Structures: The minimum thickness of concrete frames are about 40cm, which may lead to an unacceptable loss of tunnel clearances in existing tunnels. In addition, the high cost of a temporary heavy steel framework is not cost effective for tunnels of short length. Lastly, the installation of this framework requires the closing of at least one lane of traffic, which is often not possible in urban environments where diversion of traffic cannot be provided for in the tunnel. Frame structures are usually unacceptable for rail tunnels due to the strict requirements for operating clearances for the rolling stock, and the general lack of alternative routes for the rail traffic.

Prefabricated Concrete Elements provide excellent quality control during fabrication and create very high strength permanent liners. These elements require heavy construction equipment to handle and are difficult to install in the typically restricted construction window provided by most tunnel operators, during which a complete closure of the tunnel is required. This system is best performed with total tunnel closure for the entire construction period, with no returning of the tunnel to service until the project is complete. The cost of this method is very high due to the requirements for setting up of a prefabricating plant, tunnel closure, heavy construction equipment, and extensive measures for diversion of traffic.

Inner shell systems are very efficient for the whole drainage surface, provided that the waterproofing membrane is not perforated accidentally during installation and provided that adequate quality control measures are implemented, particularly with respect to the sealing and heat-welding of the elements of the membrane. These systems must be protected from fire, depending on the flammability of the membrane materials used. The best method of protecting the membrane from fire is to use a cementitious material to provide an acceptable fire rating, as determined by the appropriate Authorities.

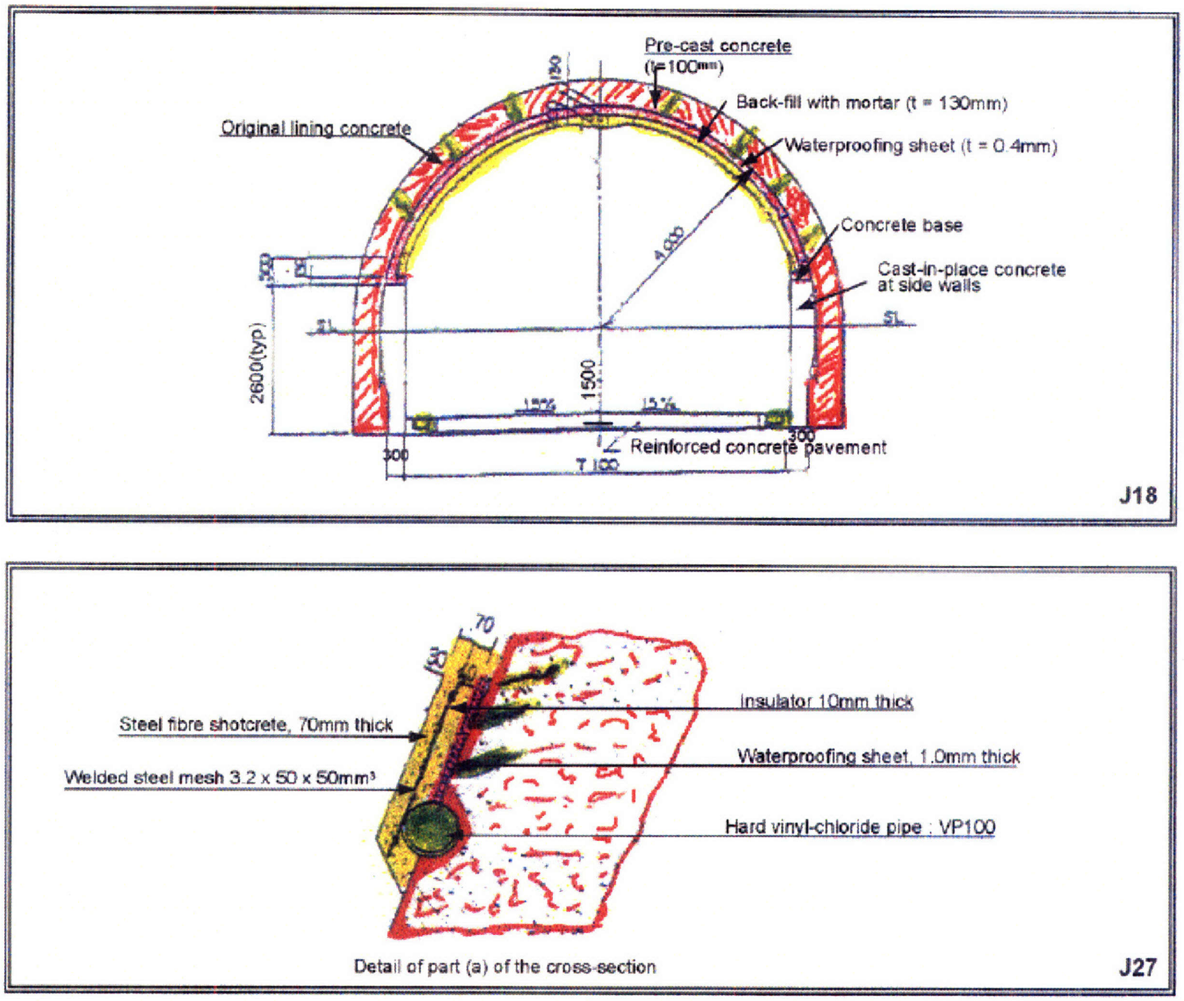


Figure 5-7, Inner Shell-Inflow Collection, (ITA-AITES 2001)

The efficiency of inner shells can be enhanced by placing a porous material between the membrane and the existing intrados, which provides a sufficient cross-sectional area for water drainage. The same effect can be achieved by using a membrane fitted with studs, which act as spacers and form channels far the water to flow through. Although the drains (collecting water from the water proofing membrane) can become clogged the overall system has superior long-term durability. These techniques generality provide an aesthetically pleasing geometrical and smooth internal lining. Staining and deposits associated with drips and general water ingress is no longer an issue.

For inner shell techniques to be effective, they must be carefully applied with appropriate specifications and proper supervision and inspection during the installation. System selection is site dependent and must satisfy the requirements of the operator and the physical environment of the tunnel. In addition to the treatment of water inflows, this technique improves the tunnel appearance and the comfort of the users. The technique is generally cost effective for tunnels with heavy traffic and with widespread cracking and water inflows.

ITA-AITES (2001) reports two case studies that have used the concept of a thin, waterproof, free-standing and independent shell. The systems comprised of:

(1) A waterproofing system protected by a heat-welded sheet protected by shotcrete;

(2) A shotcrete shell 10-30cm thick and causing only a limited reduction in the cross-section of the tunnel.

The shotcrete shell provides primary support to the waterproofing system, comprising an outer layer made up of an impermeable membrane sheet or, which serves as a surface on which the shotcrete is sprayed. These structures, which are easily damaged by collision with vehicles, are placed on reinforced concrete, shock-resistant substructures of about 2m in height. Waterproofing and drainage facilities collect and transport water to a drain collector system under the sidewalk.

Two different methods for controlling leakage have been applied in French tunnels since 1994:

IATES (2001) case F11 describes a structure made of lattice girders and conventional steel reinforcement, assembled outside the tunnel in 8-10m long modules. These modules support the waterproofing system fixed on the extrados. The modules were transported inside the tunnel and were attached to the concrete liner, shotcrete was then sprayed onto the inside surface.

In case F12 (AITES 2001) the inner shell comprised of three dimensional lattice-reinforcing panels (Figure 5-10) reinforced by pre-compressed micro-girders, which were either supported on the existing tunnel liner. The waterproofing system was placed on the outside of these structures. The low weight of the various elements, manufactured to the required thickness and curved according to the required tunnel profile, allowed them to be placed formed concrete substructure. One erected the structure is shotcreted.

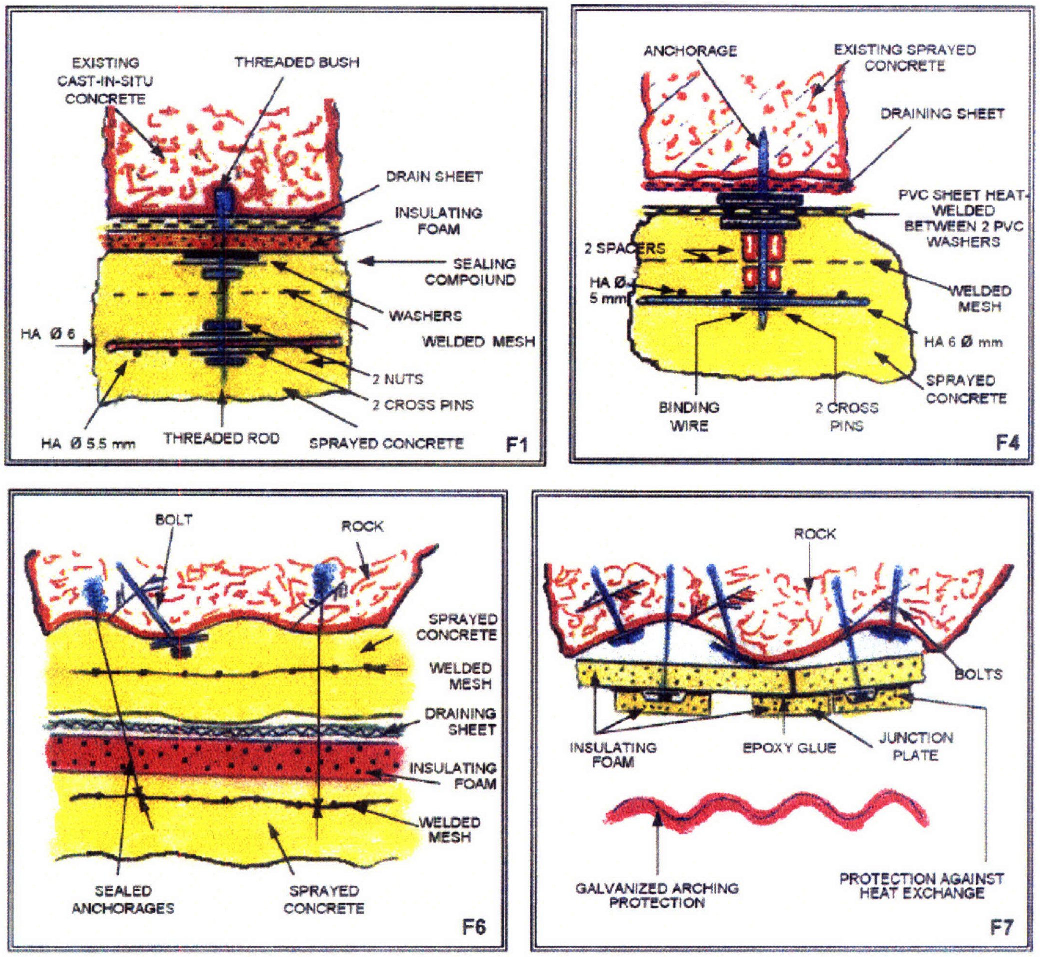


Figure 5-8, Inner Shell-Inflow Collection, (ITA-AITES 2001)

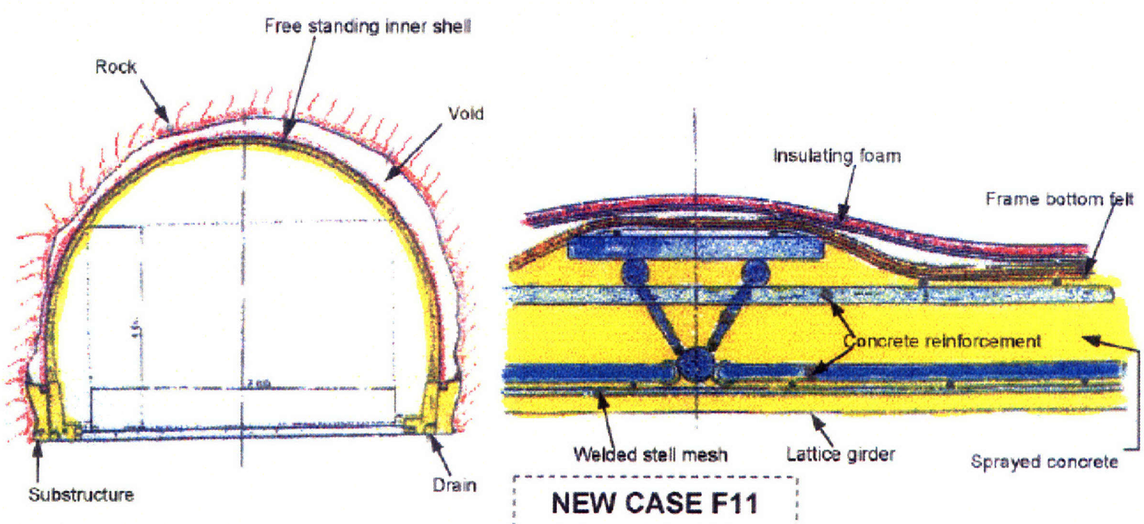


Figure 5-9, Inner Shell-Inflow Collection Case F11, ITA-AITES (2001)

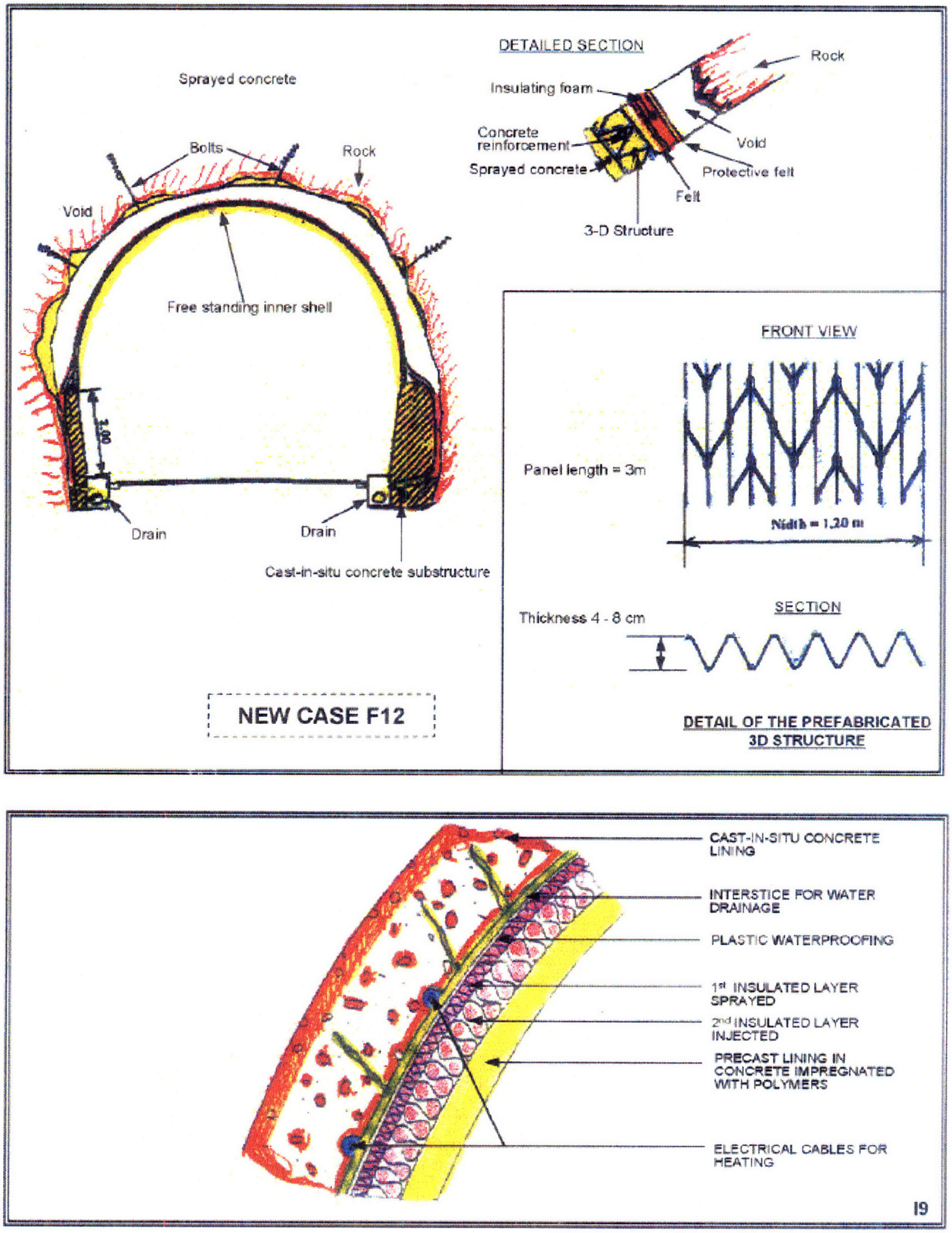
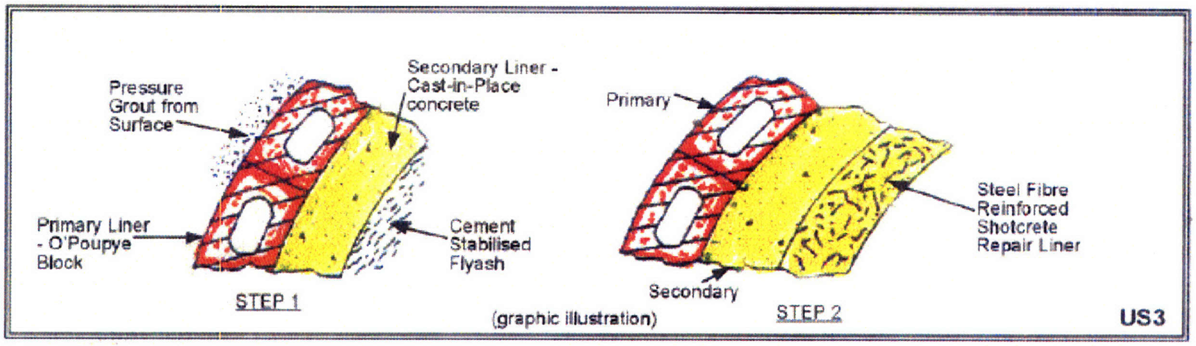


Figure 5-10, Inner Shell-Inflow Collection Case F12 and I9, ITA-AITES (2001)

5.2.3. Sprayed Membrane or Inner Lining

These techniques comprise the spraying of special mortars, reinforced by fibers or welded mesh fixed to the existing liner. The degree of waterproofing results from mortar; The reinforcement and the mortar itself are designed to limit cracking during curing and to provide some flexibility to accommodate active crack movements. Sealing capability of the liner is often supplemented by injections of grouts into the interface between the existing liner and the shotcrete; the shotcrete surface can also be coated to reduce porosity. Bracher (2004) recommends an inner coating to be applied on the inner face of the liner, for chloride and corrosion prevention.



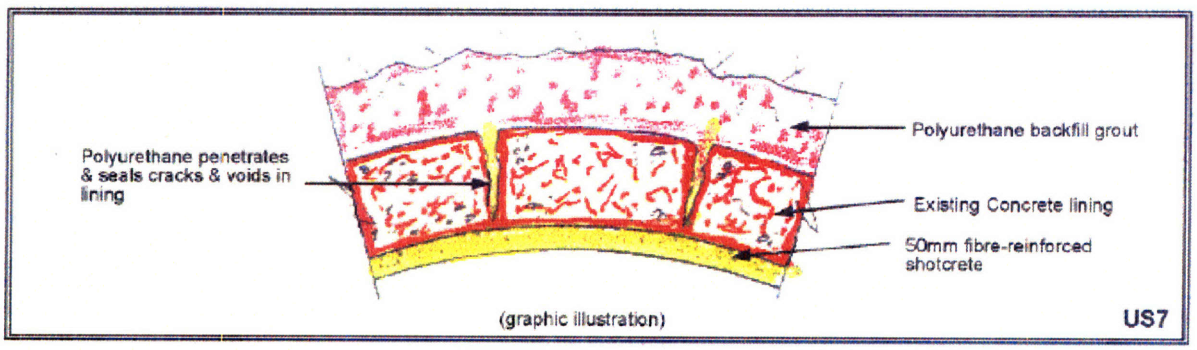


Figure 5-11, Sprayed Membrane or Inner Lining, (ITA-AITES 2001)

The application of these methods is not particularly difficult. It can be undertaken with minimal impact on tunnel operations. A major complexity is the formulation of the mortar mixture, which must be sufficiently dense to reduce the migration of water and to control shrinkage cracking. Leaking cracks or joints are sealed, as a separate operation, by the injection of particle or chemical grouts into the liner at controlled pressures to prevent spalling or damage to the existing liner. Mortar must be applied to dry surfaces (all water inflow from the leaking cracks or joints must be sealed).

These systems are efficient only for structures where the water inflow is very low (seep or standing drop). Grouting of active leaks in conjunction with the mortars can provide a long term satisfactory repair of the structure. A complete sealing of the structure is difficult to achieve, particularly where localized water inflows can occur.

There are some significant durability issues with sprayed linings. There must be careful control of shrinkage to reduce micro-cracking. Failure to limit the micro-cracking due to shrinkage will result in damage to the liner in areas where the tunnel is exposed to frost-thaw cycles. Overall, this type system is not very durable for situations where minimal infiltration is required.

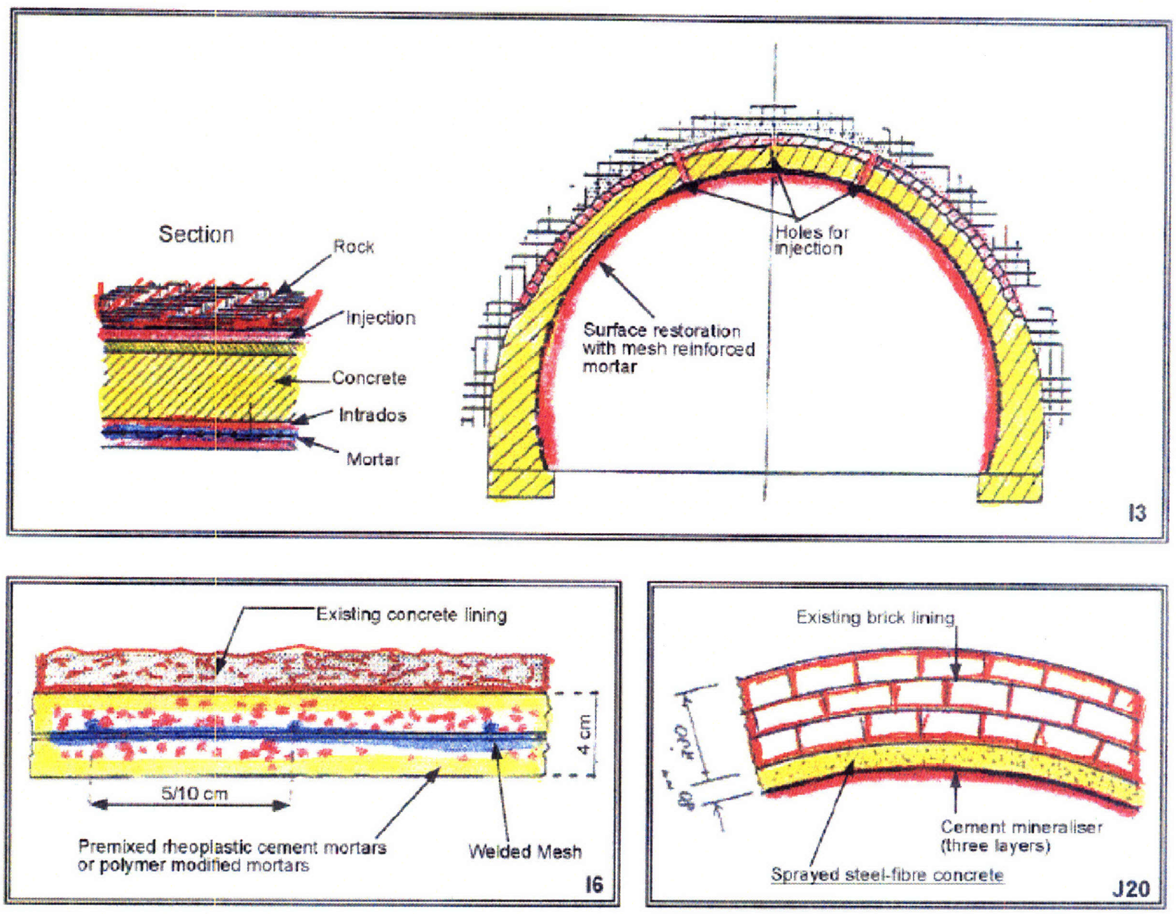


Figure 5-12, Sprayed Membrane or Inner Lining, (ITA-AITES 2001)

Sprayed coatings follow the contour of the original tunnel profile, as shown in Figure 5-12. If the waterproofing function is compromised and leakage reoccurs, water staining and deposits will appear. From an aesthetic perspective, untreated shotcrete provides an acceptable rough finish for applications such as rail tunnel sections. In public areas such as stations and in some highway tunnels a more aesthetically pleasing finish may be needed.

This technique requires detailed inspection of the existing conditions, the selection of a suitable mortar, and the use of qualified specialist applicators to achieve acceptable results. In order to be effective, all leaks must be sealed (grouted) prior to the application of the shotcrete.

5.3. Reinstatement of Tunnel Liner

Severe damage, which requires cutting out and replacing part of the tunnel liner, can be effective only if the water infiltration has been controlled or eliminated. This section discusses sealing of cracks and joints in the tunnel liner, through which leakage is encountered.

Material selection and proper installation are critical to successful sealing of the tunnel. Physical characteristics of the crack, joints and wet areas have to compatible and fit for the selected materials and its volume. ITA-AITES (2001) presents a list of materials used for the repair of leakage (Table 5-3). All materials used to stop water leakage into a tunnel will come into contact with the groundwater and grouting/injections may cause unacceptable contaminations. Toxic materials are generally prohibited. High-alumina cement used for temporary sealing of leaks should not be used as permanent repairs due to degradation over time. The grouting materials also have to be compatible with the lining and all of its ancillary components (e.g.. waterstops, waterproofing membranes and joint fillers). ITA-AITES (2001) recommends the following considerations of compatibility and suitability of grouting material:

- *Environmental compatibility of the ground materials;*
- *Width of crack or joints and the volume of repair;*
- *Compatibility of the material properties of grouting or injection material with the properties of the tunnel lining;*
- *Resistance to washout by infiltration ground water and resistance to mechanical and chemical attack;*
- *Injectability of the selected material into the are of leakage;*
- *Setting and rheological;*
- *Particle size distribution of cementitious materials;*
- *Grout performance at varying temperatures, humidities and pressures;*

- *Long term durability and strength;*
- *Availability and cost;*

Table 5-3, Application of Materials for Crack Sealing, (ITA-AITES 2001)

Procedure to apply	Types of Repair Materials for specific rates of leakage			
	Moisture Leakage at Apex/Faces of Cracks			
	dry	damp	Steady Flow	
			Free Flow	Pressure Flow
close	EP-perm.	EP-perm [1]		
	EP-inj	EP-inj [1]		
	PUR-inj	PUR-inj	PUR-inj	PUR-inj [2]
	CP-inj	CP-inj	CP-inj	CP-inj [3]
	CS-inj	CS-inj	CS-inj	CS- inj [3]
seal	EP-inj	EP-inj [1]		
	PUR-inj	PUR-inj	PUR-inj	PUR-inj [2]
	CP-inj	CP-inj	CP-inj	CP-inj [3]
	CS-inj	CS-inj	CS-inj	CS- inj [3]
rigid bond	EP-inj	EP-inj [1]		
	CP-inj	CP-inj	CP-inj	CP-inj [3]
	CS-inj	CS-inj	CS-inj	CS- inj [3]
elastic bond	PUR-inj	PUR-inj	PUR-inj	PUR-inj [2]

Legend:
EP-perm — Permeation with epoxy resin
EP-inj — Injection with epoxy resin
PUR-inj — Injection with polyurethane
CP-inj — Injection with cement paste
CS-inj — Injection with microfine cement suspension

Superscripts:
[1] only water tolerant materials
[2] previously inject rapid acting foam
[3] previously reduce water flow

5.3.1. Grouting of Cracks

The injection or grouting should fill the full depth of the crack as complete as possible, thereby providing the best seal to water intrusion and restoring integrity of the liner. Prior to grouting, the depth of the crack with relation to the lining must be known. Grout injection ports (nipples) need to be properly placed with correct angles to the surface of the lining. Crack surfaces are typically cleaned with high-pressure water prior to the packer installation. The type of fitting or packer varies depending on the injection method and equipment used. Spacing of the grouting ports is determined by the viscosity of the grout used, the porosity of the lining, and the injection range.*

The diameter of the drill hole is a function of the injection port. In additional to the injection ports, one (1) additional port is needed. Prior to commencing grouting, the crack is flushed with water to clean and wet the surfaces to be grouted.

Grouting of vertical cracks is commenced from the bottom port, working sequentially upwards to the top. When grouting horizontal cracks, grouting is started at one end of the crack and proceeds progressively to the other end. When the grout surfaces from the next port, injection is stopped and the injection port is sealed. The injection is moved to the next port and recommenced. The grout presses is maintained for a set duration at the last port to ensure complete filling before injection is stopped and the port is sealed.

At the end of the grouting operation and after the grout has set, the ports are removed and the drill holes are sealed using rapid hardening Portland cement mortar.

Cracks from 0.5mm (1/50 in) up to 3mm (1/8 in) in width, are typical of shrinkage processes and can be grouted with Portland cement suspension or paste. Admixtures may be added to swell the grout and/or improve the workability. Fine-grained sand is added to stabilize grout when injecting into large voids.

The paste is typically mixed by a colloidal mixer and is injected at a medium water/cement ratio and low pressures. A second round of grouting may be needed with a reduced water/cement ratio and a higher grouting pressure is recommended by ITA-AITES (2001). The spacing of the

* Grouting ports cannot interfere with the reinforcing bars.

injection holes for Portland cement past or suspension grout at temperatures of 20°C (70°F) are typically:

- 0.1m (4 in) - concrete with high porosity;
- 0.15m (6 in) - shotcrete and concrete of medium porosity;
- 0.3m (12 in) - top quality concrete with low porosity;

Microfine grout Portland cement with a controlled particle size distribution (maximum grain size $d_{95}<16\mu m$), can be used to grout narrow cracks smaller than 0.25mm (1/100 in). These grouts are less permeable, more durable, and have strength similar to chemical grouts. These grouts are non toxic and can be applied with standard cement grouting technology. Hardened grout is brittle and can not tolerate any movements. Surface preparation is similar to the regular cement grout. In high water inflows a pretreatment is necessary to prevent washing out of the grout.

Epoxy grouts are moisture sensitive and cannot be used in actively leaking cracks. Any water, contamination (e.g. silt or dust) in the bottom of the crack will significantly reduce the effectiveness of the grout. Epoxy grouts should only be used to fill dry cracks.

Chemical grouts typically consists of two components: urethane with sodium silicates and acrylomides; which combine to form a gel, solid precipitate or a foam. Chemical grouts are particularly appropriate for sealing cracks as narrow as 0.05mm (0.002 in) and are suitable for use in wet conditions. The grout is good for sealing construction joints and shrinkage cracks, they retain flexibility after curing (with good degree of elongation). A skilled and experienced subcontractor is a must have to complete a successful repair. Polyurethane grouts rapidly react with water, generating foams which increase in volume by multiples of 4-30. For example, Green Mountain International, LLC manufactures *P.L.U.G.S.* which is a two component injection resin that reacts in 90 to 120 seconds. It does not generate extreme exothermic temperatures and expands approximately thirty times (30x) the volume from liquid to foam. The Author tested this product and verified the rapid foaming and the high volumetric expansion properties. The product also has a 5% elongation. Another product by the same manufacturer is MG295 which is a single component low viscosity grout. MG295 has superior elongation (140%) but a longer rise and reaction time. In general foam grout is not totally rigid and can accommodate small movements during subsequent grout injections to fill the crack completely.

The surface preparation is similar to cement grouts. The port spacing ranges from 0.1m (4 in) for injections in dense concrete to 0.3m (12 in) for masonry.

If a given tunnel has a steady water flow through cracks, the grouting procedure would take at least two iterations. The first injection, with a one component grout reduces the inflow (quantity and velocity) of the seeping water, due to the urethane grout reaction with water and the foam product. The second round of grouting forms a soft elastic resin, which fills the crack. The first round of foam will be compressed, and the new grout will bind both face of the crack and form a seal.

A third round injection may be necessary if the water, confined in the concrete pores, react with the resin injected in the second stage to form a foam which causes a loss of binding between the resin and the lining. This bond loss will cause the water to seep through the cracks again. The third injection will require higher pressures of up to 250 bars (3600 psi) (ITA-AITES 2001). One may note that such pressure may be harmful to some old masonry liners.

ITA-AITES (2001) gives several examples of successful applications of chemical grouts:

- *Numerous tunnels in the USA were successfully treated by drilling 5/8-in. holes, equipped with rubber packers and one-way valves. Water reactive polyurethane foam grout was injected two to three times until polyurethane came out of the cracks.*
- *Austria has reported successfully stopping leakages in tunnel using a combined injection on Polyurethane and epoxy resin.*
- *Two subway tunnels, in Belgium were sealed by the use of one-component water reactive polyurethane.*
- *The construction joint between the slab and walls in a German motorway tunnel was sealed using injection pipes to insert a one-component water reactive polyurethane grout. The material was modified to provide low viscosity, deep penetrations, and high strength properties.*
- *Japan has used two-stage injection of polyurethane and epoxy resin to seal tunnel linings.*

5.3.2. Repair of Leaking Joints

Construction joints in cast-in-situ concrete linings vary in design: from a simple unkeyed joint through a key unbounded joint to a keyed, bonded and sealed joint. The sealed joint may incorporate one or more waterstops. Repairs to construction joints should consider waterstops and reinforcing steel.

Leaking construction joints, with an incorporated water stop, will most likely have a problem associated with poorly compacted concrete around the waterstop. The entire length of the joint has to be treated, because the specific location of water ingress is very difficult to identify.

Repairs of rigid bonded and unbounded construction joints can be executed with cement paste or microfine cement suspension paste. Water proofing of all other types of construction joints is preferably done with chemical grouts, as described in the preceding subsections. ITA-AITES (2001) recommends that the grouting pressures and viscosity to be confirmed by tests and field trials to assure complete filling and sealing of the joint.

Compression/expansion joints allow for movements in the tunnel lining, caused by shrinkage or temperature changes (thermal stresses). Unlike construction joints, a compression joint incorporates a joint filler, which acts as a bond breaker, which is then covered by a sealing material placed along the surface of the joint. Construction joints of thickness greater than 0.3m (12 in) should be sealed with permeation grouting discussed in the succeeding subsection.

The filler and the sealing materials have to be removed from the compression joint to seal the leaking joint, while a special adhesive sealing is applied around the surface of the joint to contain the grout.

Two case studies from Germany report applications of polyurethane foam and epoxy grouts in tunnels. In the fist case, rail transit tunnel, having water seepage though the joints. The majority of the joints were successfully sealed, while others did not meet Deutsche Bahn AG permissible leakage rates (Chapter 3). In the second case of two highway tunnels, construction and expansion joints could not be sealed. After three (3) grout injections some of the joints were still allowing water infiltration.

5.3.3. Rehabilitation by Permeation Grouting

Moist areas in concrete linings generally arise from adverse properties of the linings such as lack of compaction, poor aggregate selection, or a cold joint inside the lining, all of which increase the porosity of the lining. If the hydrostatic pressures in the granular skeleton of the concrete are high, a permeating injection of the grout may not successfully seal the leakage. Water tolerant epoxy resins or urethane are the two grouting materials suitable for permeation grouting.

A confinement of the moist areas within the impermeable concrete is the first step in the process. The barrier is typically constructed by the injection of grout through two rows of ports installed around the moist areas. The depth of the drilled holes for the grouting, which forms the barrier, would typically alternate between 40% and 80% of the lining thickness. The permeation grout injection holes, within the confines of the barrier, may be drilled at depth alternating between 60% and 75% of the lining thickness. Following the grouting of the injection ports, the surface area of the lining (and a 0.5 m or 20 in area outside the barrier) needs to be sealed. If running water is encountered, injections should be done with urethane grout to reduce the inflow. Following this, progressive grouting of the moist area is carried out to complete the seal. Grouting procedures are presented in the preceding subsections.

5.3.4. Concluding Remarks on Grouting (ITA-AITES 2001)

The rate of success of the case histories reported in ITA-AITES (2001) varies between 40% and 60%. Some of the failures are attributed to difficult working conditions. Temperature changes, varying quantities of leakage water and water vapor are the main factors affecting the outcome.

5.4. Elimination of Leakage at the Source

This section considers modifications of the soil/rock surrounding the tunnels through grouting procedures. The influence of grain size distribution on the groutablility of the soil mass is illustrated in Figure 5-13.

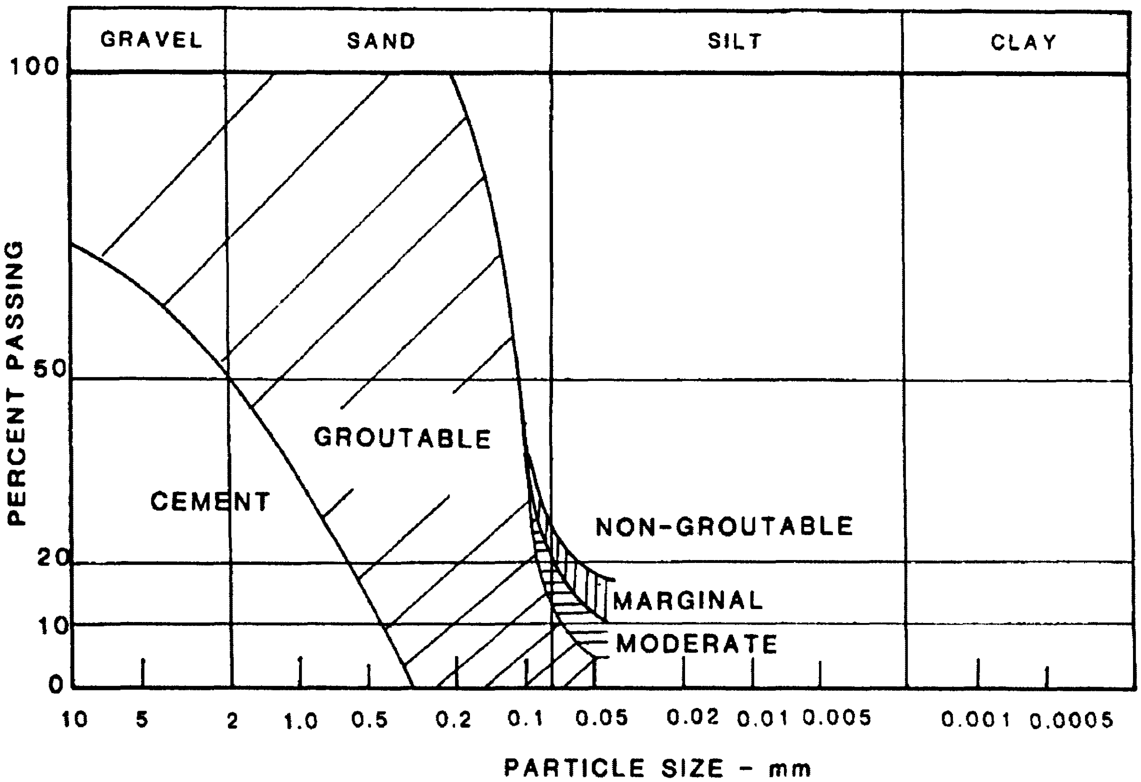


Figure 5-13, Grain-size ranges for groutable soils, (Baker 1982)

There are main methods of grouting for groundwater control in soil masses:

(1) Permeation grouting is the most common method used. Grout is pumped under low pressure to fill the voids in the soil strata and hence, reduce hydrostatic conductivity. The two main types of grouts are chemical or cementitious materials.

(2) Displacement grouting, also known as compaction or compression grouting, is a process in which grout is injected under high pressure to displace the existing soil, creating a soil-cement bulb. Overlapping of cement bulbs can create areas of very low permeability within the soil mass.

(3) Jet (Replacement) grouting is a process that uses high pressure jets to remove the soil in a specific location with cementitious soil.

The grouting method must be determined depending on the type of grout one is using. Groutablity of soil is a function of fines, or the amount of soil passing the No. 200 sieve. Figure

5-14 illustrates the relationship between injection method and grain size distribution. Horizontal lines illustrate the range of successful grouting in relation to grain size.

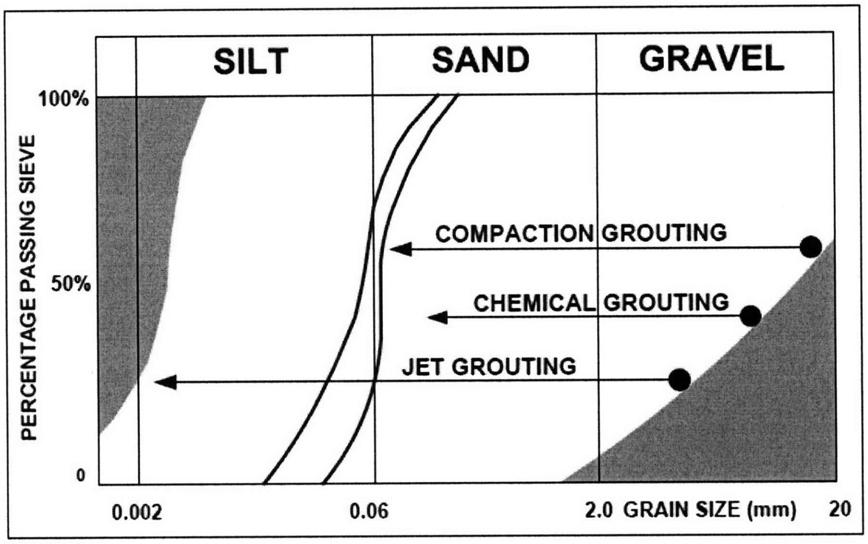


Figure 5-14, Grouting Method and Soil Type, (ITA-AITES 2001)

The grouts available for source leaks are similar to ones identified in section 5.3.1, consisting of:

(1) Particle Grouts

- Cementious materials;
- Non-flexible after curing;
- Structure movement requires reinjection;
- Used for displacement and replacement grouting of surrounding soil spectra;

(2) Chemical Grouts

- Flexible after curing;
- Good track record in USA and Europe;
- Good for: fine sands, silts, clays, and potentially fractured rock;
- More expensive than particle grouts;
- Use for permeation grouting;
- Used in seismic activity zone;
- Toxic and are regulated by local environmental standards;

Table 5-4, Particle and Chemical Grout and specifications, (ITA-AITES 2001)

Description	Viscosity	Toxicity	Strength	Remarks
Particle Grout				
Flyash Type F;C	High (50 cps- 2:1)	Low	High	Nonflexible
Type I Cement	High (50 cps-2:1)	Low	High	Nonflexible
Type III Cement	Med (15 cps-2:1)	Low	High	Nonflexible
Microfine Cement	Low (8 cps-2:1)	Low	High	Nonflexible
Microfine Cement/silicate	Low (10 cps-2:1)	Low	High	Nonflexible
Bentonite	High (50 cps-2:1)	Low	Low	Semi-flexible
Chemical Grout				
Acrylamides	Low (2:1)	High	Low	Flexible
Acrylates	Low	Low	High	Flexible, poor success record
Silicates	Low (6 cps)	Low	High	Nonflexible, high shrinkage
Lignosulfates	Med (8 cps)	High	Low	Flexible, high cost
Polyurethane (MDI)	High (100 cps)	Med	Low	Flexible, low shrinkage, water reactive
Polyurethane(TDI)	High (300 cps)	Med	Low	Flexible

5.5. Effectiveness of Repairs

ITA-AITES (2001) Working Group No. 6 (Maintenance and Repair) published a report analyzing 157 cases with tunnel damages, of which 123 had damages caused by groundwater infiltration. The group concluded that water leakage is the principal damage to and degradation on tunnel linings.

Summary of the case history data is given in Appendix A of ITA-AITES (2001) report. An analysis of the leakage data is presented in Figure 5-15. Continuous leaks and drips compose the majority of the reported inflows. One may argue that this is not the major form of leakage, because it can be expected that the more severe cases of damage involving major repair works would be recorded in more detail that the less critical cases (small leaks).

Figure 5-16 shows that only 57% of the repairs were successful; One concludes that up to 43% of cases no permanent repair was accomplished.

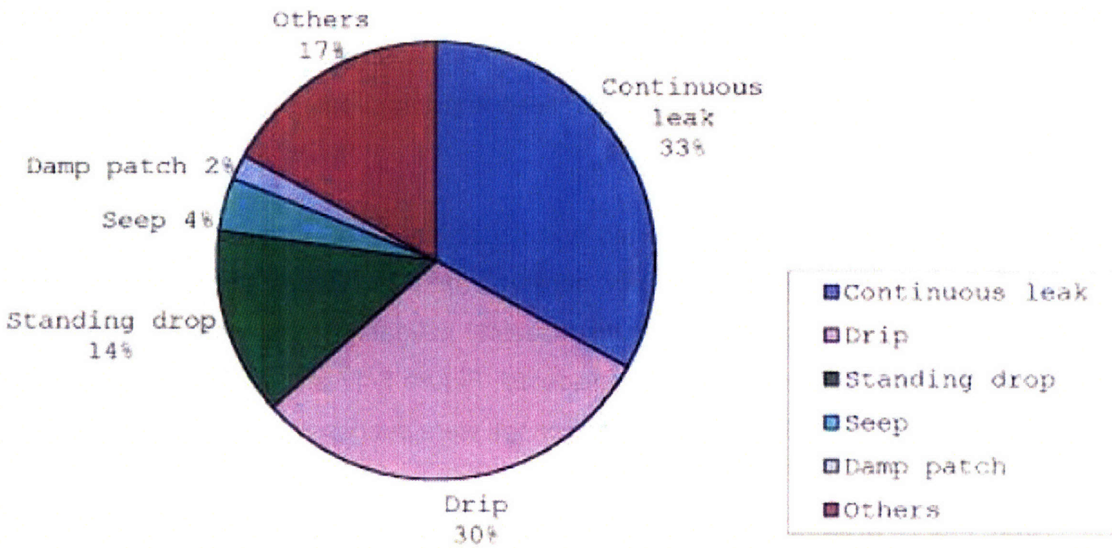


Figure 5-15, Type of Leakage Reported, (Total No. of Cases: 106), (ITA-AITES 2001)

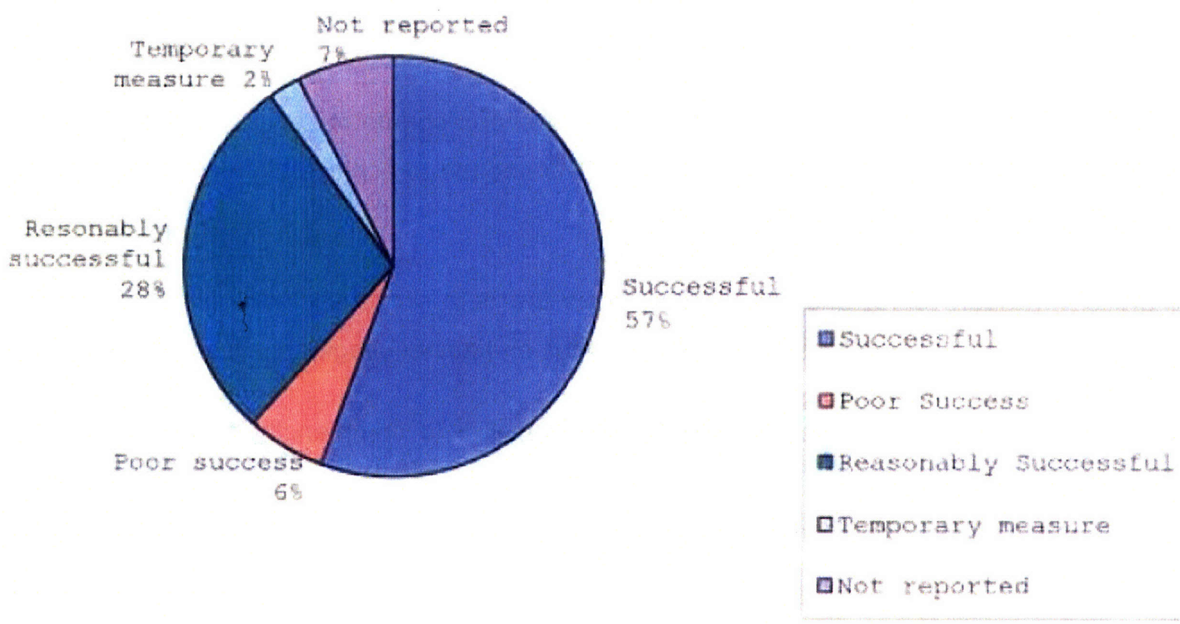


Figure 5-16, Leakage Repair Effectiveness, (Total No. of Cases: 81), (ITA-AITES 2001)

Effectiveness of repairs as a function of leakage quantity is presented in Table 5-5. No firm conclusion can be made with regard to cases of low inflows (because of quantity of reported data). The reported 100% success of repairs in seep and damp patch is regarded as misleading by ITA-AITES (2001).

Although a number of cases were reported as unsuccessful, the tunnel continued to operate efficiently. Some of the repairs were not achieved due to access limitations imposed by operations. However the number of the unsuccessful mitigation measures is alarmingly high (43%), considering the number of cases considered. Figure 5-17 shows the degree of success for a specific repair method, 54% to 59% of the cases were successful.

Table 5-5, Degree of Success in Treating Various Inflows, (ITA-AITES 2001)

Type of leak	No. of Cases	% Degree of Success		
		Successful %	Reasonably Successful %	Poor Success %
Continuous	35	60	23	9
Drip	32	53	31	3
Standing drop	15	47	33	7
Seep	4	100	-	-
Damp patch	2	100	-	-
Others	18	50	4	6

ITA-AITES (2001) stresses the importance using the appropriate material and repair method. The case histories highlight that an improvement is needed to achieve higher success rates of repairs.

Figure 5-18, Figure 5-19, and Figure 5-20 show the details of the study conducted by ITA-AITES (2001). Tunnel construction methods, linings and tunnel usage are presented.

The group recommended investigating the damage prior to commencing repairs or material selection, taking into account all tunnel materials (lining, soil, etc.), and a selection of a qualified contractor to perform the repairs. All repairs are site specific and no one solution can be universally applied.

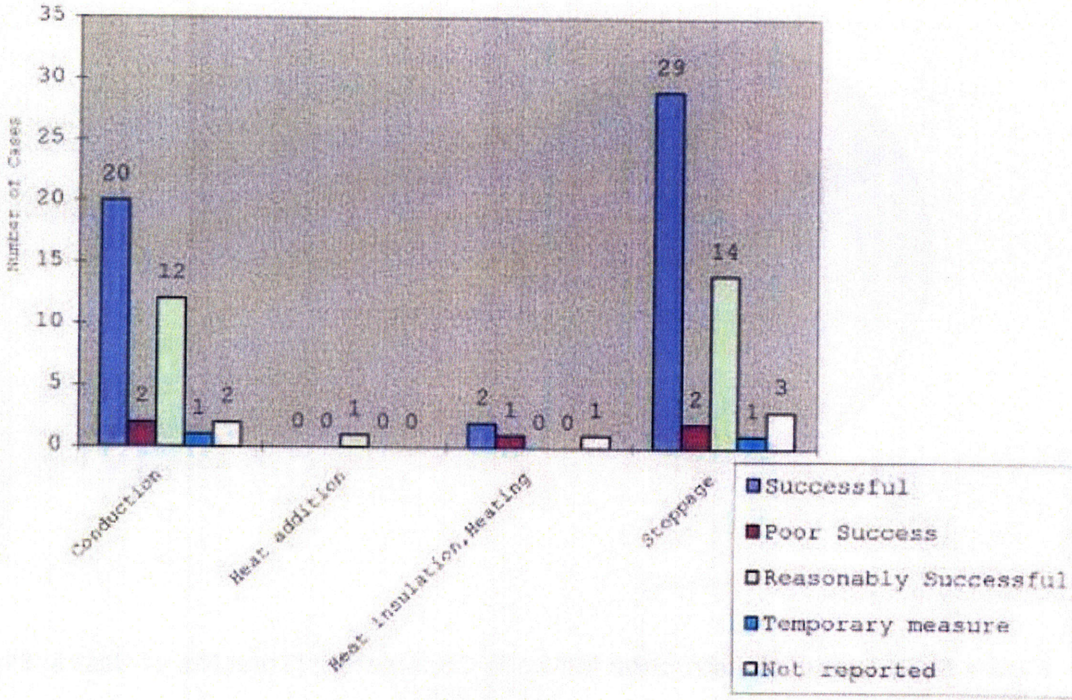


Figure 5-17, Mitigation, Repair Effectiveness, (Total No. of Cases: 91), (ITA-AITES 2001)

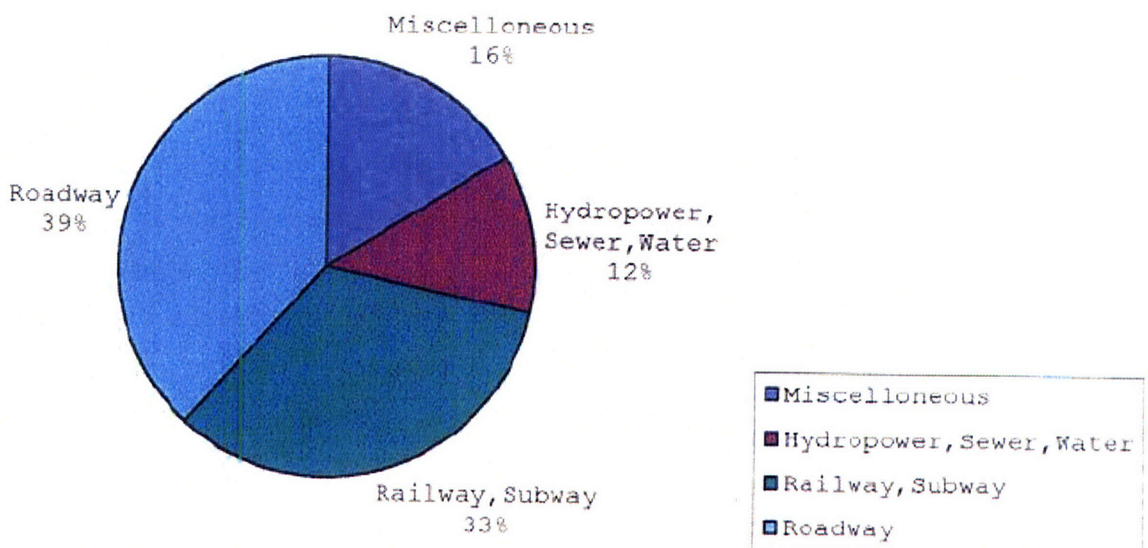


Figure 5-18, Types of Tunnels Considered, (Total No. of Cases: 81), (ITA-AITES 2001)

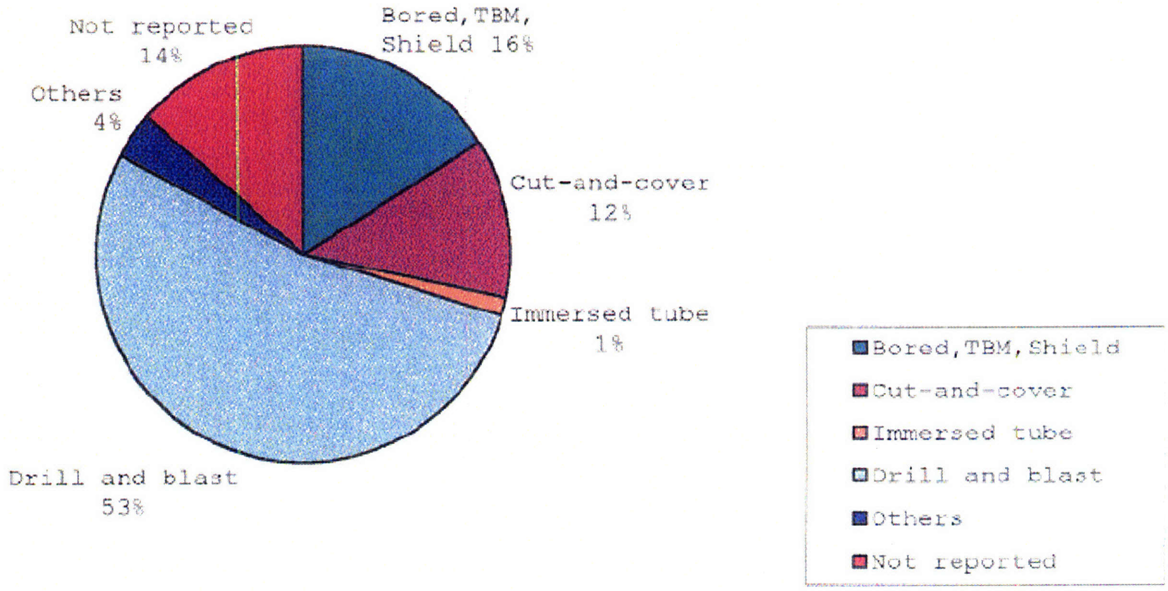


Figure 5-19, Tunnel Construction Methods Considered, (Total No. of Cases: 91), (ITA-AITES 2001)

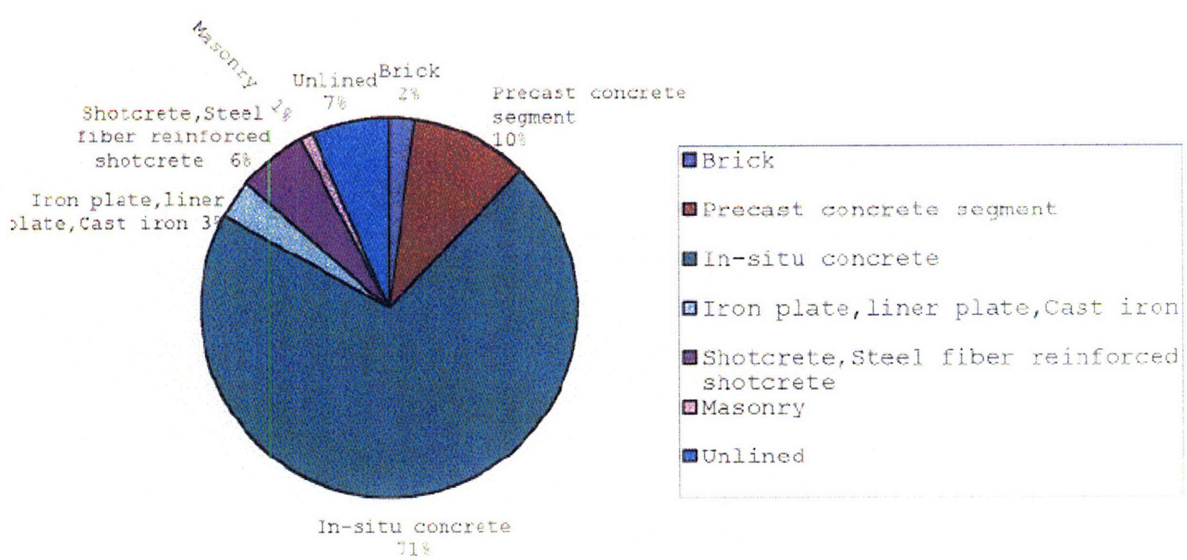


Figure 5-20, Types of Lining Considered, (Total No. of Cases: 96), (ITA-AITES 2001)

6. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

After examining over one-hundred case studies, the Author infers that water leakage is the principal damage causing degradation on tunnel linings. International standards for permissible leakage rates (transit tunnels) are consistent with class A definitions of CIRIA (1979) and are approximately 0.1-2 gpm/100,000 SF (0.05-1.2 L/day/SM). Numerous projects in US, Germany, UK, and Czech Republic specified and recorded leakage rates below CIRIA class A.

The intensity of leakage in underground structures is the result of: permeability of the surrounding ground (environment), permeability of the lining and/or the waterproofing system (materials), groundwater conditions (total head and recharge of the groundwater table) and the use of the underground space.

The most common cause of leakage (based on numerous case studies) in cast-in-place lining is due to cracks that develop from shrinkage of concrete during curing and inability of the structure to accommodate movements due to thermal changes.

Individual sources of leakage may be allowable within the permissible rates. However these can cause damage to the tunnel structure and to the surrounding environment (consolidation and differential settlement). Spalling is one of most common structural damages due to groundwater infiltration. The presence of water can cause unpleasant stains, resulting in erosion and corrosion over time. Formation of icicles, ice and water ponding can affect public safety in a tunnel and jeopardize operations.

To mitigate leakage in underground structures and tunnels one may control and/or eliminate the inflow. Chemical grouting is one of the most common measures. However, its application has been unsuccessful in 43% of cases reported by ITA-AITES (2001). Inappropriate material selection for each particular application is major contributing factor for the lack of success.

The Author focused this thesis on highway and rail tunnels, and established recommended permissible leakage rates for such underground structures based on international standards and experiences. These recommended rates (Appendix C) can serve as guidelines for future tunnel design specifications or to compare recorded inflow rates with international standards.

Leaking basements and subterranean parking structures (below the groundwater table) have not been addressed in research. Insufficient technical research has been conducted on the subject of leak mitigation in retaining walls and/or slurry walls for basement and subterranean garages. Insurance companies, building owners, and general public with non-technical background need know the consequences of water infiltration and mitigation measures to control and eliminate inflows.

Designs of retaining walls for basements and subterranean structures (dating back decades) often contain insufficient drainage behind the wall and lack of necessary wall thickness. Building codes evolved with time, thus older designs are often deficient in waterproofing details. As structures gain value with time owners will be faced with a problem statement: Is the leak acceptable? How does it compare to others? What are the consequences? How do I repair it? Hydrostatic pressures in the adjacent soil provide the driving force for infiltration and are often amplified during heavy rain events. Other contributing factors are poor maintenance of the site drainage system, clogged gutters and structural movements.

In conducting future research one will need to examine designs recommendation by various Authors used today. Case histories of slurry wall performance can be used, as a starting point, to identify the scope of potential problems (e.g. Konstantakos 2000). There are two strategies that are commonly considered:

(1) Permanent solutions involving proactive maintenance and strengthening of walls (Edwards 2006). Remedial solutions include structural rehabilitation using fiber reinforced polymer (FRP) bonded to the wall. These FRP methods are similar to inner shell methods for tunneling and can also address leakage. Further research can be conducted by looking into polymers that are typically used to wrap failing walls.

(2) Temporary solutions include fiberglass sheets attached to trap and collect water in floor drains and for extraction through sump pumps. This is a poor long term solution as the problem is hidden from observation. Often temporary solutions include installation of perimeter interception and drainage systems (Edwards 2006).

A chart, similar to the ones presented in Chapter 0, with permissible infiltration rates from design guides and particular project construction documents needs to be developed. The chart should also include recorded infiltration data into basement and subterranean structures.

Property owners, real estate investors, and insurance companies are all interested in solutions to leaking basements. Inspections during property sales and insurance appraisals will ignite questions. For example the majority of zoning requirements for new developments require a certain number of parking stalls per square foot of office space or a certain number of parking stalls per bedrooms for a residential development thus requiring construction of underground parking structures. Assuming that a significant fraction of subterranean garages recently built (since the 1990's) are poorly maintained, there is likely to be large maintenance deficit in the future years. In Southern California the majority of condominiums or apartment projects have subterranean parking structures. One can expect leakage problems in the in the near future. Although ground water approximately 40 to 60 feet below surface in some parts of Los Angles, drainage issues may exist due to run off and over irrigation (Stirbys, et al 1999).

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APPENDIX A: TERMINOLOGY

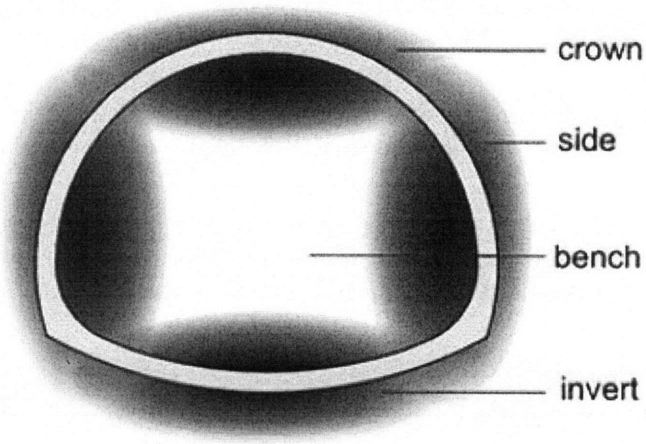


Figure A-1, Elements of a Tunnel Cross Section, (Kolymbas 2005)

Central Artery/Tunnel Project (CA/T) (unofficially termed Big Dig) includes two main sections of tunnel: the Central Artery; 4.3 mile long reconstruction of Interstate 93 through downtown Boston, and the 'Tunnel' a 3.9 mile log extension of Interstate 90 eastward from I-93 to Logan Airport, passing through the Ted Williams tunnel below Boston Harbor. The Big Dig began construction in 1991 and was completed in 2006.

Diaphragm wall – another term used for slurry wall;

Drill-and-blast – a method of disintegration rock by drilling small diameter holes on a planned layout packing these holes with explosives and then firing to a fixed program to shatter the rock in a desired form.

Dry space – underground structures, such as: road and railway tunnels and subterranean parking garages which require dry space with zero or minimal permissibly groundwater inflow rates.

EPB – earth pressure balance; closed-face tunnel boring machine, generally used in soft ground condition.

Heading – heading of a tunnel comprises: excavation, support of the cavity and removal of the excavated earth (Kolymbas 2005).

In-situ concrete – concrete which is poured on site (in-pace), as a part of a permanent structure.

Joints – expansion joints, shrinkage joints and construction joints are normally used to divide extended structures into blocks of limited deformation. For example, since 1960 many subway tunnels below the groundwater table have been built with short block lengths. (Haack and Emig 2002)

Leakage – ground water intrusion, infiltration, and inflow are all synonyms used to identify leakage in underground structures.

Mining – process of excavation in a soft ground cut-and cover, or rock extraction from a rock tunnel.

NATM – New Austrian Tunneling Method constitutes a method where surrounding rock or soil formation of a tunnel are integrated into an overall ring-like support structure. Thus the ground formation itself contributes to the tunnel support by internal load redistribution.

Precast concrete – structural components are manufactured in a offsite plant and later brought to the building site for assembly. Precast concrete is typically used for segmental lining.

Roadheader – also know as boom cutters are tools used for moderate rock strength and for laminated or joined rock. The cutter is mounted on the extension arm, boom, of the excavator and millcuts the rock into small pieces. Figure A-2 below shows the roadheader.

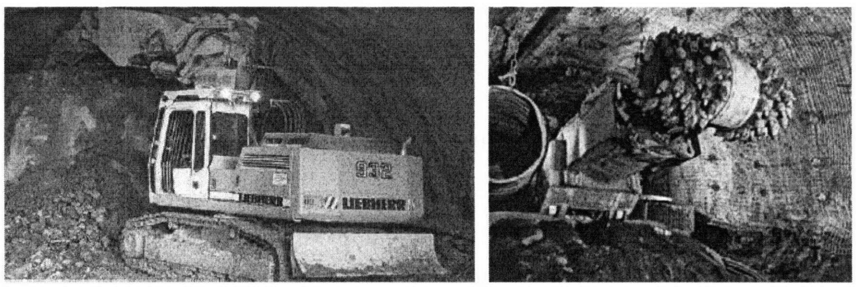


Figure A-2, Excavator (left); Roadheader (right), (Kolymbas 2005)

Segmental lining – tunnel lining segments manufactured at an offsite facility and then installed in a tunnel as permanent lining.

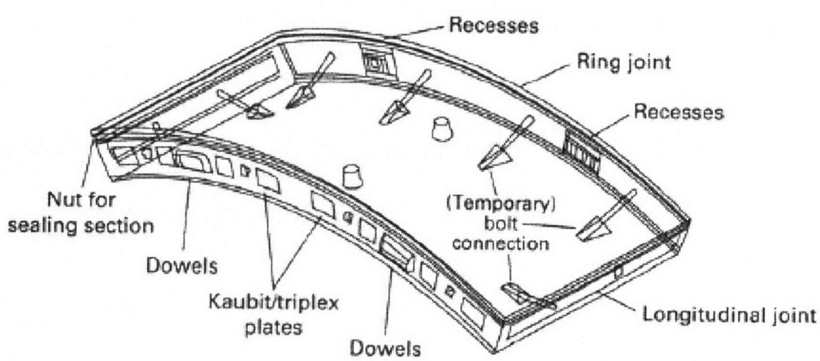


Figure A-3, Lining Segment, (Kolymbas 2005)

SF – square foot (feet);

Shield – implies a device to provide an immediate support to the ground. A self propelling device for providing ground support around the excavation.

Shotcrete – In tunneling, shotcrete is applied to seal freshly uncovered surfaces and for the support of cavities.

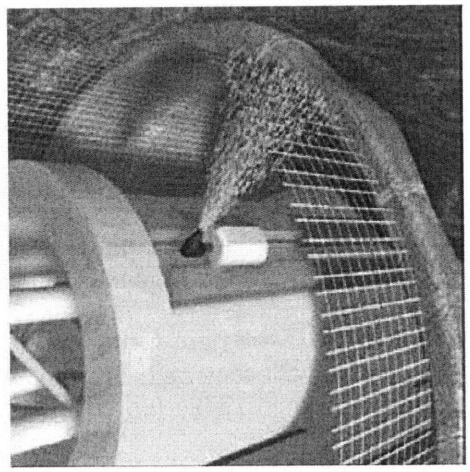


Figure A-4, Shotcrete Application, (Kolymbas 2005)

Slurry wall – is a type of wall used for temporary and permanent support of excavation. It is used for tunnels construction in areas of soft earth with a high groundwater table. A special clamshell-shaped digger is used to excavate the slurry trench guided by the guide walls. The trench is kept filled with bentonite slurry at all times to prevent collapse. Once the first trench is completed to design depth, or bedrock, an adjacent trench is dug in the same manner. Eventually, once a particular length is reached, a reinforcing cage, or I-beams, may be lowered into the slurry-filled pit and the pit is filled with concrete from the bottom up using tremie pipes. The concrete displaces the bentonite slurry which is pumped out and recycled.

SM – square meter(s);

Soldier Pile Tremmie Concrete (SPTC) Wall – see section 2.2.3;

Tunnel – is defined as a road, highway, or rail, underground passageway in varying geology. Tunneling or tunnelling is defined as a construction activity of creating a tunnel.

Tunnel Boring Machines (TBM) – describes a full-face machine used for advancing tunnels in rock, usually making a circular cut.

Underground structure – is defined as a tunnel, or any subterranean structure needing waterproofing.

Watertight – no water may penetrate through the concrete, the working and expansion joints (precast or in situ concrete), the seal between tunnel units (submerged tunnel), or gaskets (segmental linings).

Water-resistance – prevention of a limited passage of water through a use of a membrane, coating or physical properties.

Waterproofing systems – a coating or a membrane that prevents the free passage of all water through a medium. A waterproofing system is a component of a water-resistant tunnel.

TUNNELING ORGANIZATIONS

AFTES – Association Française des Tunnels et de l'Espace Souterrain - the French tunnelling association, was established in January 1972. (www.aftes.asso.fr/)

CIRIA – The Construction Industry Research and Information Association is an industrial co-operative research association set up and operated by the British construction industry with the support of the United Kingdom Government. Its main role is to organize cooperation within the industry and between it and Government for identifying, financing and managing research, other investigations, and the collection and dissemination of information. CIRIA published Report 81:*Tunnel Waterproofing* in 1979. (www.ciria.org.uk/)

ITA-AITES – The International Tunneling and Underground Space Association is an organization founded in 1974, comprising currently 52 member nations and 284 affiliate members, aiming to encourage the use of the subsurface for the benefit of public, environment and sustainable development, and to promote advances in planning, design, construction, maintenance and safety of tunnels and underground space.
(www.ita-aites.org)

Federal Transit Administration (FTA) – an agency within the US DOT that provides financial and technical assistance to local public transit systems; (www.fta.dot.gov)
FHWA performs research in the areas of automobile safety, congestion, highway materials and construction methods. (www.fhwa.dot.gov)

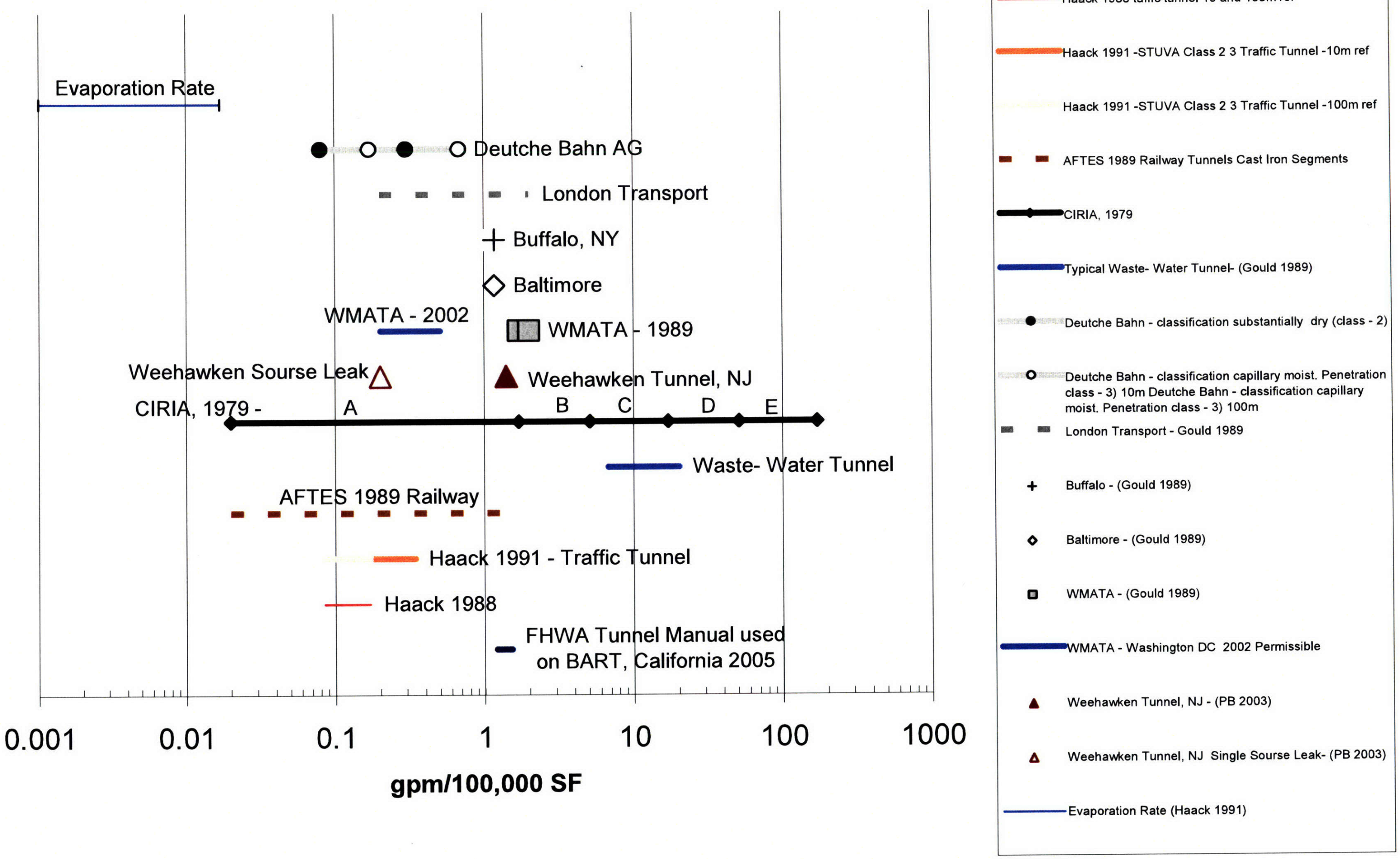
The Federal Highway Administration (FHWA) – is a division of the United States Department of Transportation that specializes in highway transportation. FHWA oversees federal funds used for constructing and maintaining the National Highway System. This funding mostly comes from the federal gasoline tax and mostly goes to State departments of transportation. FHWA oversees projects using funds to ensure that federal requirements for project eligibility, contract administration and construction standards are adhered to.

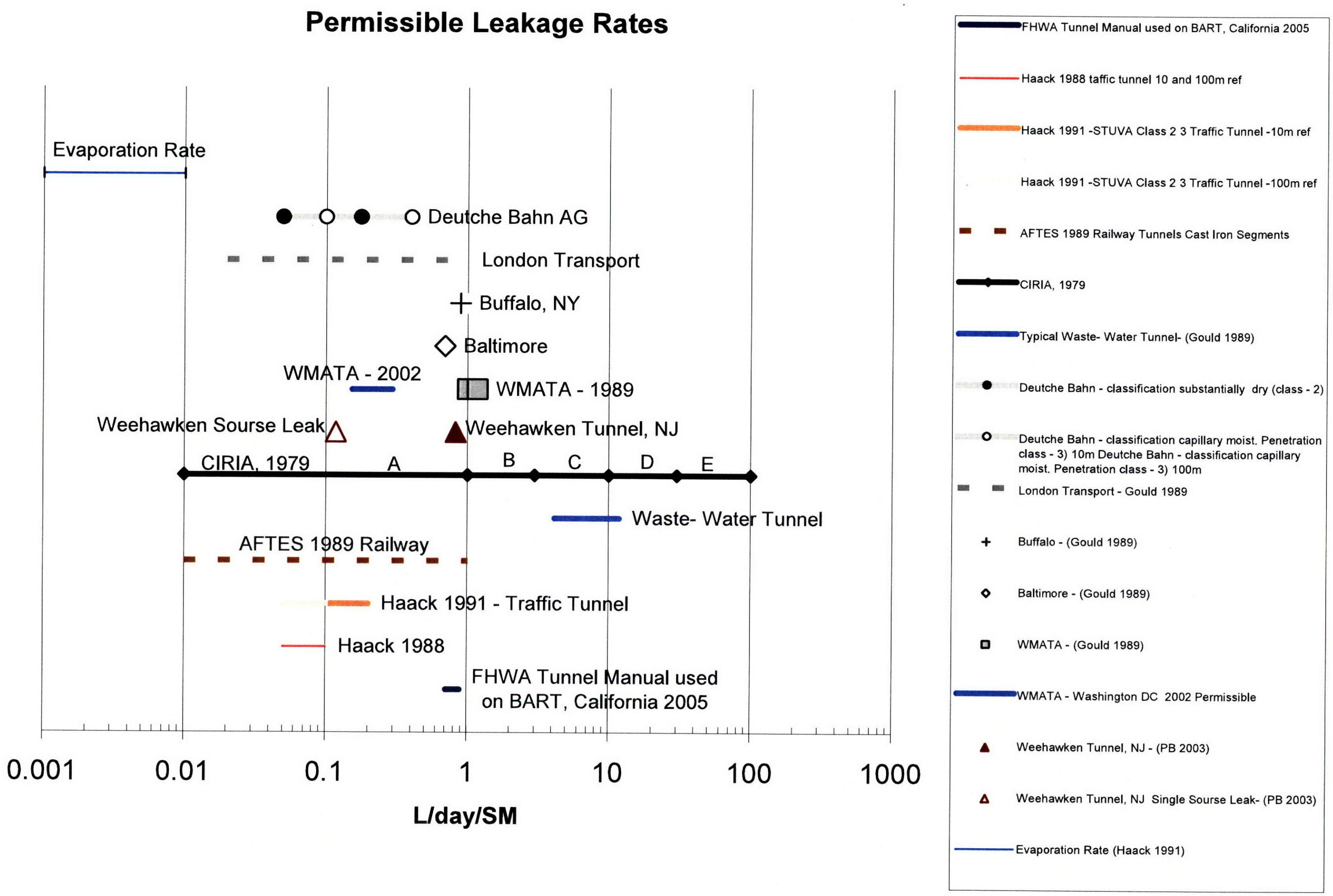
Urban Mass Transportation Administration (UMTA) – In 1991, the agency was renamed the **Federal Transit Administration (FTA)**.

US Department of Transportation (DOT) – is a federal Cabinet department of the United States government concerned with transportation. It was established in 1966 and is administered by the United States Secretary of Transportation. Its mission is to "Serve the United States by ensuring a fast, safe, efficient, accessible and convenient transportation system that meets our vital national interests and enhances the quality of life of the American people, today and into the future." (www.dot.gov)

APPENDIX B: PERMISSIBLE LEAKAGE RATES

Permissible Leakage Rates




Permissible Leakage Rates
Evaporation Rate
Deutche Bahn AG
London Transport
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WMATA - 1989
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Haack 1991 - Traffic Tunnel
Haack 1988
FHWA Tunnel Manual used on BART, California 2005
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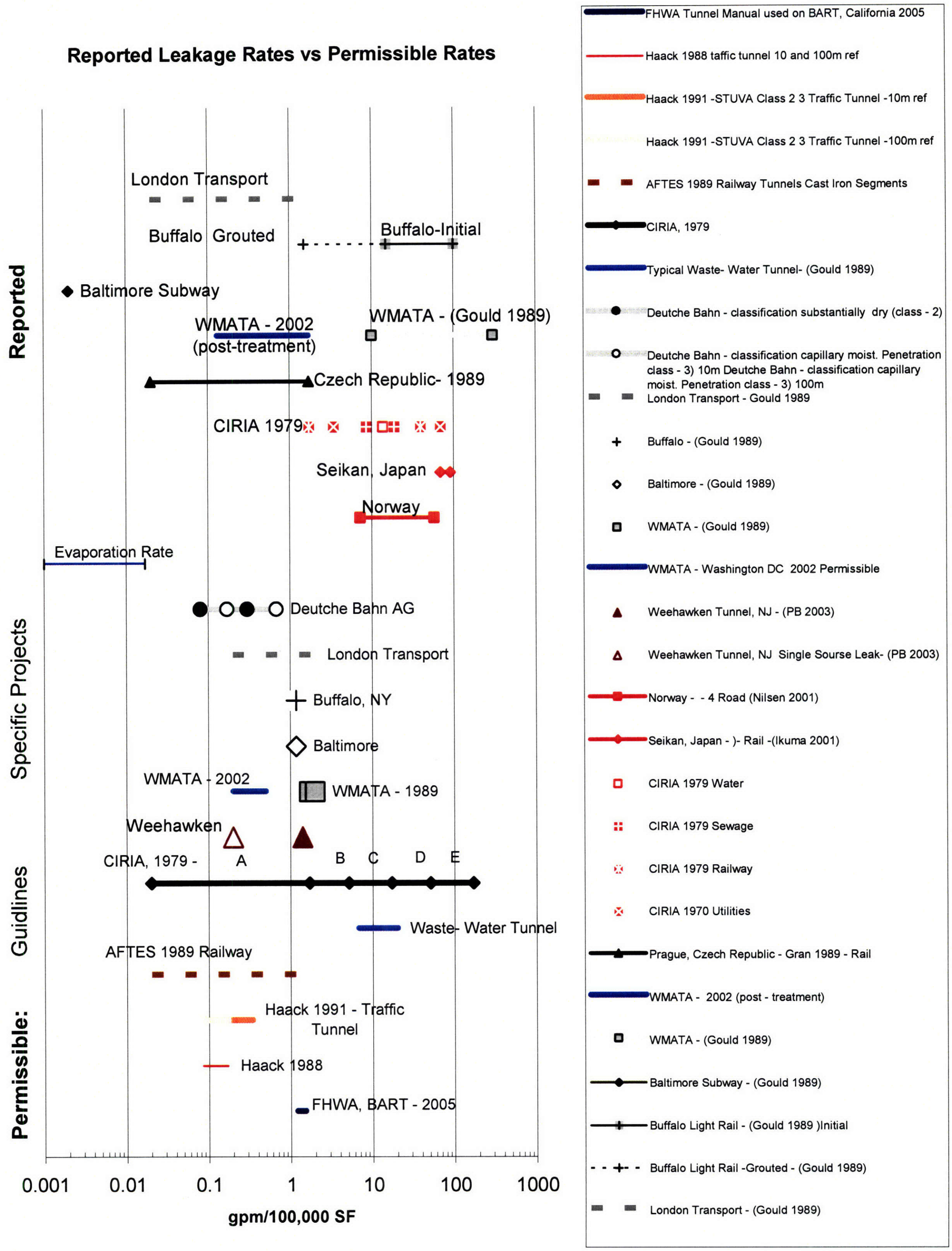
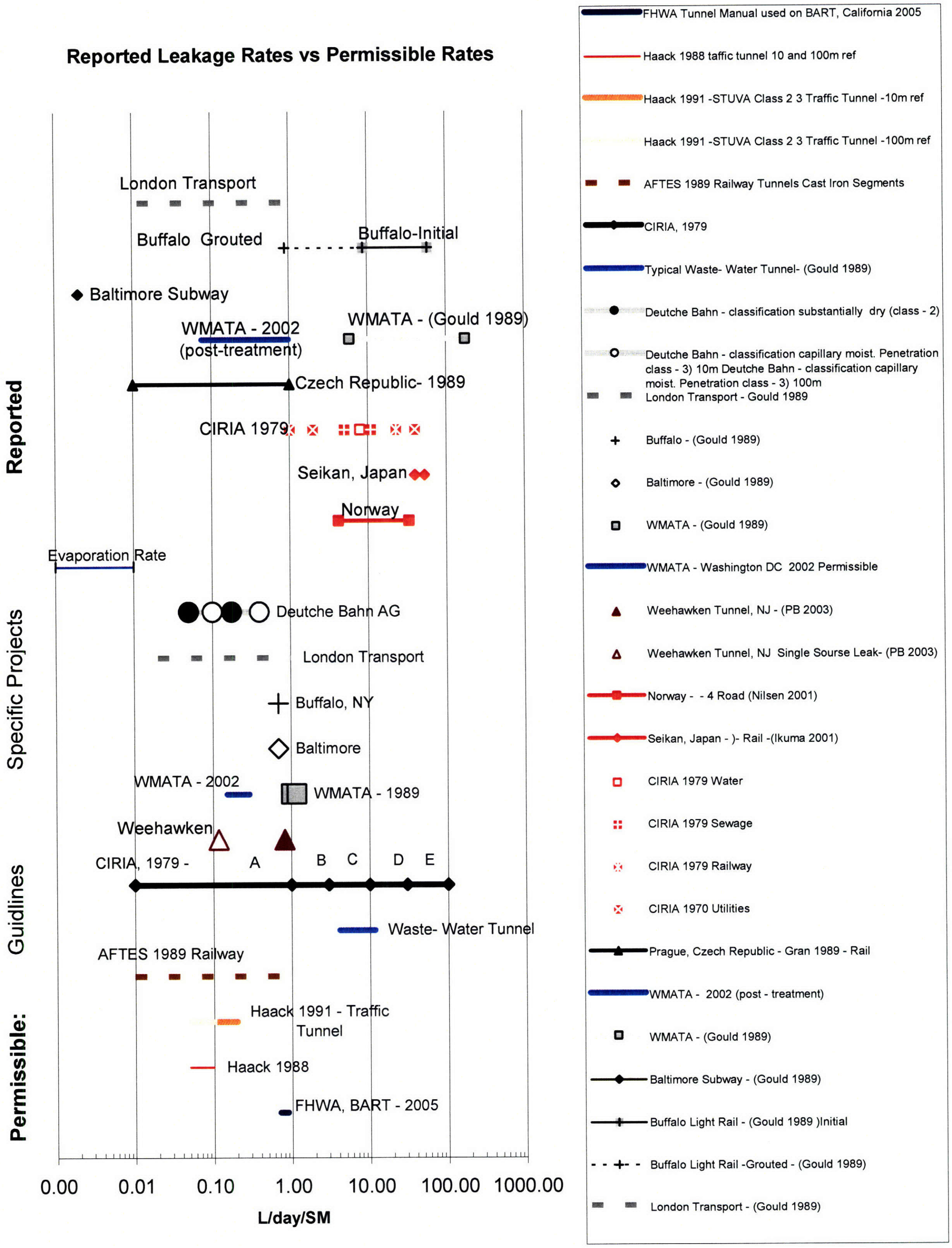
L/day/SM
FHWA Tunnel Manual used on BART, California 2005
Haack 1988 taffic tunnel 10 and 100m ref
Haack 1991 -STUVA Class 2 3 Traffic Tunnel -10m ref
Haack 1991 -STUVA Class 2 3 Traffic Tunnel -100m ref
AFTES 1989 Railway Tunnels Cast Iron Segments
CIRIA, 1979
Typical Waste- Water Tunnel- (Gould 1989)
Deutche Bahn - classification substantially dry (class - 2)
Deutche Bahn - classification capillary moist. Penetration class - 3) 10m Deutche Bahn - classification capillary moist. Penetration class - 3) 100m
London Transport - Gould 1989
Buffalo - (Gould 1989)
Baltimore - (Gould 1989)
WMATA - (Gould 1989)
WMATA - Washington DC 2002 Permissible
Weehawken Tunnel, NJ - (PB 2003)
Weehawken Tunnel, NJ Single Sourse Leak- (PB 2003)
Evaporation Rate (Haack 1991)

APPENDIX C: REPORTED LEAKAGE RATES VS PERMISSIBLE RATES


Reported Leakage Rates vs Permissible Rates
Reported
Specific Projects
Guidlines
Permissible:
London Transport
Buffalo Grouted
Buffalo-Initial
Baltimore Subway
WMATA - 2002 (post-treatment)
WMATA - (Gould 1989)
Czech Republic- 1989
CIRIA 1979
Seikan, Japan
Norway
Evaporation Rate
Deutche Bahn AG
London Transport
Buffalo, NY
Baltimore
WMATA - 2002
WMATA - 1989
Weehawken
CIRIA, 1979 - A B C D E
Waste- Water Tunnel
AFTES 1989 Railway
Haack 1991 - Traffic Tunnel
Haack 1988
FHWA, BART - 2005
0.001 0.01 0.1 1 10 100 1000
gpm/100,000 SF
FHWA Tunnel Manual used on BART, California 2005
Haack 1988 taffic tunnel 10 and 100m ref
Haack 1991 -STUVA Class 2 3 Traffic Tunnel -10m ref
Haack 1991 -STUVA Class 2 3 Traffic Tunnel -100m ref
AFTES 1989 Railway Tunnels Cast Iron Segments
CIRIA, 1979
Typical Waste- Water Tunnel- (Gould 1989)
Deutche Bahn - classification substantially dry (class - 2)
Deutche Bahn - classification capillary moist. Penetration class - 3) 10m Deutche Bahn - classification capillary moist. Penetration class - 3) 100m
London Transport - Gould 1989
Buffalo - (Gould 1989)
Baltimore - (Gould 1989)
WMATA - (Gould 1989)
WMATA - Washington DC 2002 Permissible
Weehawken Tunnel, NJ - (PB 2003)
Weehawken Tunnel, NJ Single Sourse Leak- (PB 2003)
Norway - - 4 Road (Nilsen 2001)
Seikan, Japan -)- Rail -(Ikuma 2001)
CIRIA 1979 Water
CIRIA 1979 Sewage
CIRIA 1979 Railway
CIRIA 1970 Utilities
Prague, Czech Republic - Gran 1989 - Rail
WMATA - 2002 (post - treatment)
WMATA - (Gould 1989)
Baltimore Subway - (Gould 1989)
Buffalo Light Rail - (Gould 1989)Initial
Buffalo Light Rail -Grouted - (Gould 1989)
London Transport - (Gould 1989)



Reported Leakage Rates vs Permissible Rates
Reported
Specific Projects
Guidlines
Permissible:
London Transport
Buffalo Grouted
Buffalo-Initial
Baltimore Subway
WMATA - 2002 (post-treatment)
WMATA - (Gould 1989)
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CIRIA 1979
Seikan, Japan
Norway
Evaporation Rate
Deutche Bahn AG
London Transport
Buffalo, NY
Baltimore
WMATA - 2002
WMATA - 1989
Weehawken
CIRIA, 1979 -
A
B
C
D
E
Waste- Water Tunnel
AFTES 1989 Railway
Haack 1991 - Traffic Tunnel
Haack 1988
FHWA, BART - 2005
0.00
0.01
0.10
1.00
10.00
100.00
1000.00
L/day/SM
FHWA Tunnel Manual used on BART, California 2005
Haack 1988 taffic tunnel 10 and 100m ref
Haack 1991 -STUVA Class 2 3 Traffic Tunnel -10m ref
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Baltimore - (Gould 1989)
WMATA - (Gould 1989)
WMATA - Washington DC 2002 Permissible
Weehawken Tunnel, NJ - (PB 2003)
Weehawken Tunnel, NJ Single Sourse Leak- (PB 2003)
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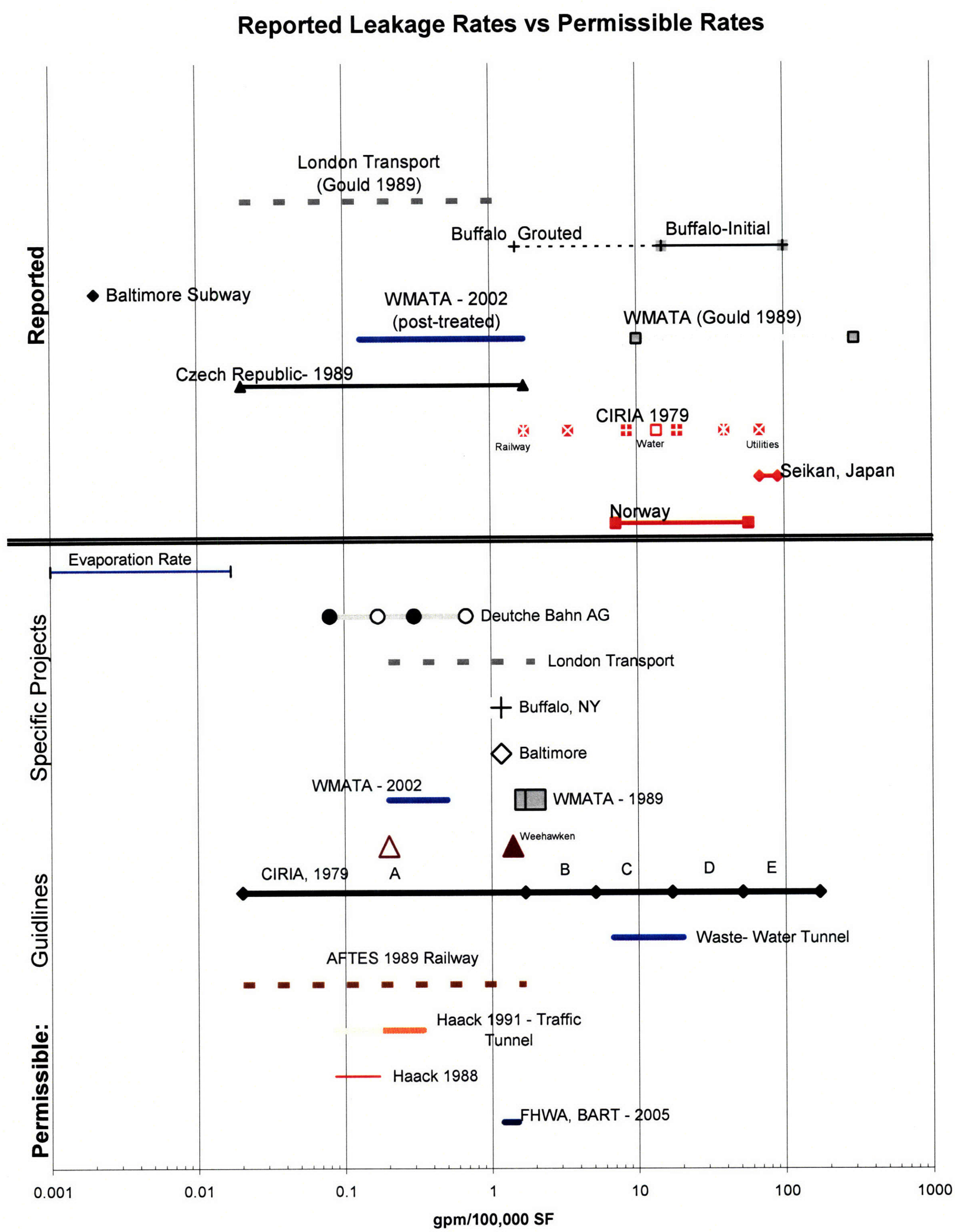
CIRIA 1979 Water
CIRIA 1979 Sewage
CIRIA 1979 Railway
CIRIA 1970 Utilities
Prague, Czech Republic - Gran 1989 - Rail
WMATA - 2002 (post - treatment)
WMATA - (Gould 1989)
Baltimore Subway - (Gould 1989)
Buffalo Light Rail - (Gould 1989)Initial
Buffalo Light Rail -Grouted - (Gould 1989)
London Transport - (Gould 1989)


Reported Leakage Rates vs Permissible Rates
Reported
London Transport (Gould 1989)
Buffalo_Grouted
Buffalo-Initial
Baltimore Subway
WMATA - 2002 (post-treated)
WMATA (Gould 1989)
Czech Republic- 1989
CIRIA 1979
Railway
Water
Utilities
Seikan, Japan
Norway
Evaporation Rate
Specific Projects
Deutche Bahn AG
London Transport
Buffalo, NY
Baltimore
WMATA - 2002
WMATA - 1989
Weehawken
Permissible: Guidlines
CIRIA, 1979
A
B
C
D
E
Waste- Water Tunnel
AFTES 1989 Railway
Haack 1991 - Traffic Tunnel
Haack 1988
FHWA, BART - 2005
0.001
0.01
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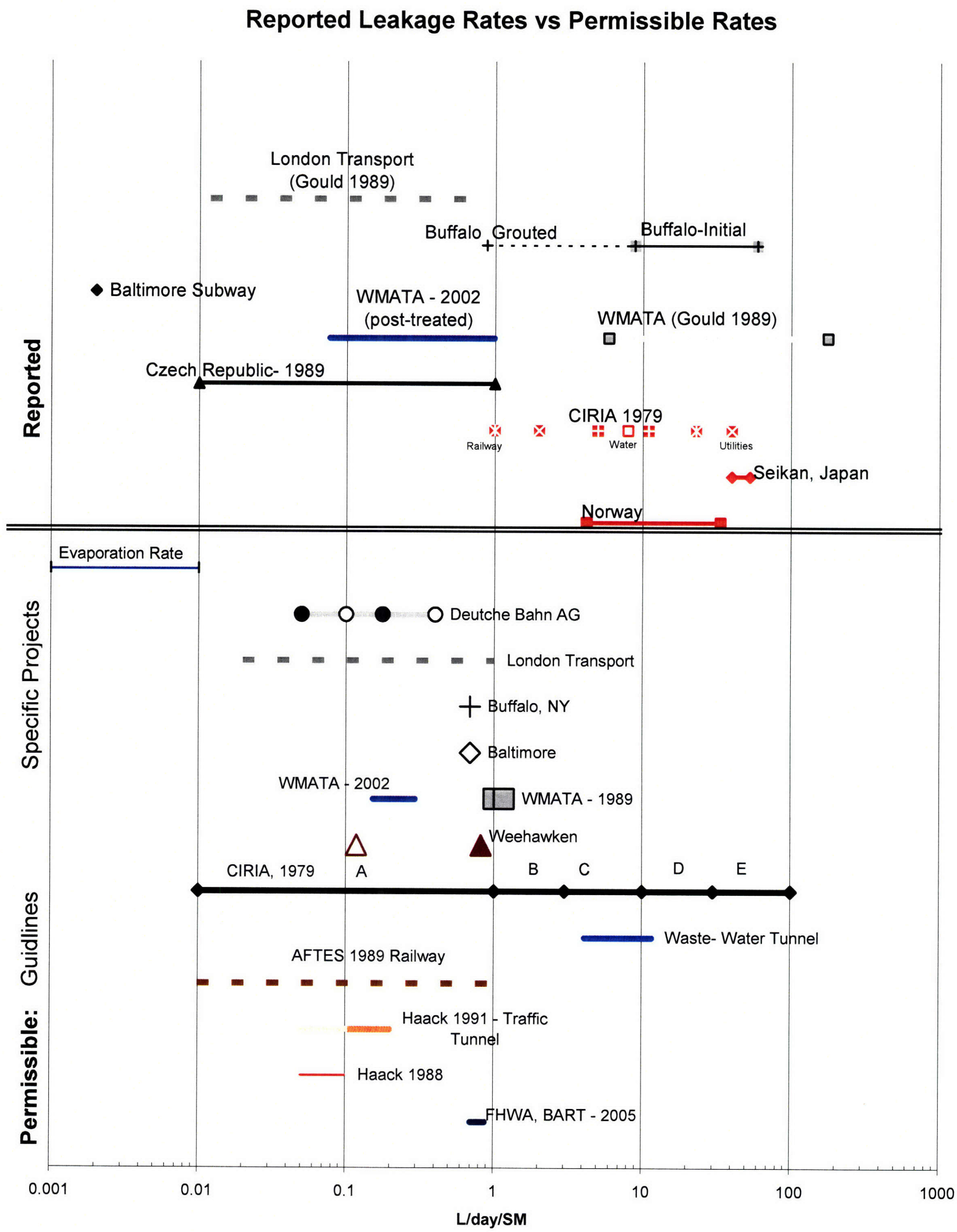
10
100
1000
gpm/100,000 SF


Reported Leakage Rates vs Permissible Rates
Reported
London Transport
(Gould 1989)
Buffalo Grouted
Buffalo-Initial
Baltimore Subway
WMATA - 2002
(post-treated)
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Czech Republic- 1989
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Railway
Water
Utilities
Seikan, Japan
Norway
Evaporation Rate
Specific Projects
Deutche Bahn AG
London Transport
Buffalo, NY
Baltimore
WMATA - 2002
WMATA - 1989
Weehawken
Permissible: Guidlines
CIRIA, 1979
A
B
C
D
E
Waste- Water Tunnel
AFTES 1989 Railway
Haack 1991 - Traffic
Tunnel
Haack 1988
FHWA, BART - 2005
0.001
0.01
0.1
1
10
100
1000
L/day/SM

APPENDIX D: REPORTED LEAKAGE RATES AND PERMISSIBLE RATES VS CA/T

Reported Leakage Rates and Permissible Rates vs CA/T