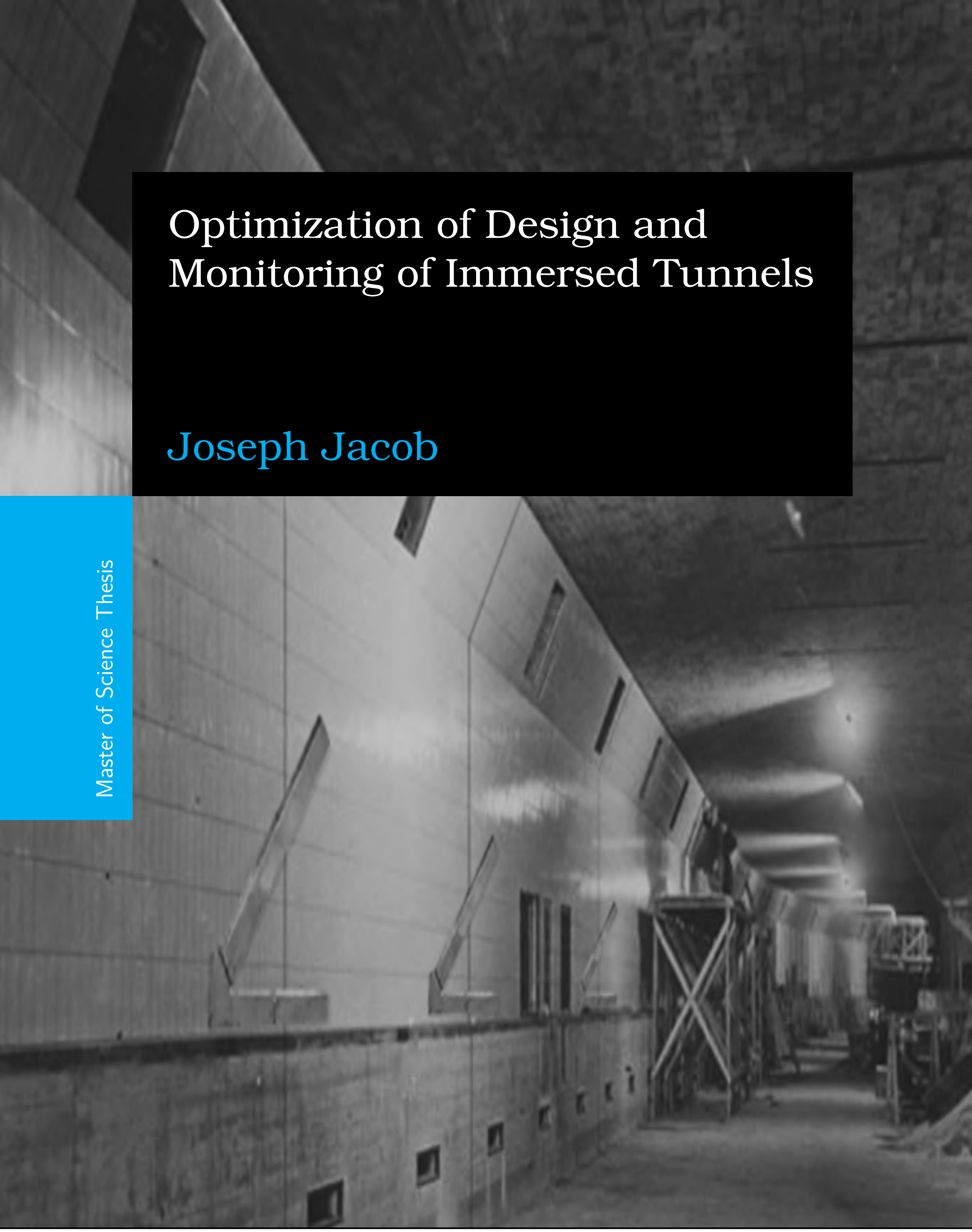


Optimization of Design and Monitoring of Immersed Tunnels

Joseph Jacob

Master of Science Thesis

TU Delft
Delft University of Technology

Optimization of Design and Monitoring of Immersed Tunnels

Master of Science Thesis

For the degree of Master of Science in Geo-Engineering at Delft University of Technology

Joseph Jacob

November 9, 2022

Supervisors: Dr. Ir. W. Broere
Dr. Ing. M.Voorendt
Ir. X.H. Zhang

Faculty of Civil Engineering and Geosciences (CiTG) · Delft University of Technology

Preface

This thesis, "Optimization of Design and Monitoring of Immersed Tunnels" is written to fulfill the graduation requirement of MSc Civil Engineering, specializing in Geo-Engineering, at the Delft University of Technology. I was involved in this project from March 2022 to Oct 2022. I am proud to say that after three enriching years I will conclude my study as a master student with this thesis. I have always been drawn to tunnelling as it requires great expertise from various fields. Throughout this thesis, I was able to implement the knowledge gained from my lectures and further expand my insight into the field of sensors and structural engineering. I hope this report gives the reader an idea of the importance of innovation and monitoring within immersed tunnels.

First and foremost I would like to thank my supervisors Wout Broere, Mark Voorendt and Xuehui Zhang. This thesis would have been an impossible task without your constant support, insight and flexibility. Xuehui thank you so much for our daily conversations and guidance. It has been a great pleasure to talk to you! Last but not the least, I would like to express my gratitude to my friends who constantly motivated me and picked me up when times got tough. Finally, to my family, words could never express what you mean to me. I have missed you everyday.

Delft, University of Technology — Joseph Jacob
November 9, 2022

Abstract

Many immersed tunnels have already reached or are about to reach the halfway point of their intended lifespan of 100 years. Around this time, several structural problems were noticed during inspection which have led to performance deterioration and durability issues of the tunnel. Often large- scale maintenance was required to meet the design life of immersed tunnels. To counter these problems and improve the constructability of tunnels several design changes have been developed over the past two decades. Furthermore, the current monitoring plan lacks in terms of frequency, accuracy and often requires tunnel tube closure. This thesis aims to improve the life cycle operation of immersed tunnels based on long-term structural problems encountered during their design life.

To achieve this, common structural problems encountered were initially identified by reviewing literature and maintenance reports of past projects. Excess deformations, joint gap opening/closure, concrete cracking, corrosion and leakage were determined as the five typical structural problems. These problems were then further analysed based on construction technique, geotechnical engineering and design flaw from a mechanical and structural perspective. It was observed that the main reason for excessive deformations was sub-soil stiffness. The thermal movement of concrete produced by seasonal temperature variations significantly facilitated the joint opening and closure. Large strains in the concrete induced by differential displacements along longitudinal and transverse directions ultimately led to concrete cracking. Chloride ingress and carbonation of concrete were identified as the primary causes of corrosion of steel components. Finally, a loss of prestressing of rubber gaskets driven by relaxation of the rubber gasket and differential settlements formed leakage channels.

Once the structural problems were identified, recent design changes were inventoried based on element material, cross-section design, transverse prestressing, foundation treatment, waterproofing and joint formation. The primary objective of this step was to understand the impact of these design changes in mitigating structural problems. Semi-rigid elements, seismic joint, bellow joint, crown seal joint, steel and discrete shear key were developed to withstand significant large differential displacements and restrain the opening of joints. Emergence of concrete cracks was mitigated by incorporating the use of utility tubes, transverse pre-stressing, full section casting and new reinforcement detailing. Utilizing corrosion-resistant steel and cathodic protection, corrosion was controlled. The presence of an additional barrier by double seal and enhanced underwater connection provided by V-wedge, key and deployable element were used to limit leakages.

Following the structural problem study, an improved monitoring plan was developed to serve as an early warning system for structural problems. Firstly, the parameters to be monitored were determined based on how they influenced structural problems. These parameters were vertical displacement, transverse displacement, joint width, crack width, shear key force, leakage detection, chloride diffusivity, pH in concrete, concrete strain and shear modulus of soil. Subsequently, threshold limits for each parameter were set to implement quick remedial actions once these limits were crossed. The sensors and monitoring techniques that were used to monitor these parameters were distributed optic fiber sensor, fiber optic extensometer, pressure cell, vibrating wire strain gauge, fiber optic pH sensor, optic fiber chloride sensor, swellable polymeric optic sensor and multichannel analysis of surface waves, hydrostatic level and terrestrial laser scanner. These sensors were narrowed down even more based on accuracy, frequency and how helpful they were to contractors in current-day practice.

Remediation measures specific to each structural problem were also briefly discussed. In the case of concrete cracking, measures to be implemented are concrete replacement and removal of polluted environment. Ground improvement techniques help to mitigate excess deformations. Installing a steel limit block, prestressing cables across joints and injecting resin are the necessary steps to take in case of joint gap opening/closure. The necessary steps to perform during leakages are adding/replacement of rubber gaskets, adopting an external steel plate, drilling and injection method. Finally, applying coats, cathodic protection, replacing/repairing concrete and using plastic plates/ rubber rings help mitigate corrosion.

Table of Contents

Chapter 1

Introduction

1-1 Motivation

Immersed tunnels have been used in many places worldwide to cross waterways (mostly canals and harbors) instead of bridges or bored tunnels. Since the first immersed tunnel was constructed in 1910 for traffic use, there are now over 150 immersed tunnels in use worldwide (Lunniss and Baber, 2013). In the Netherlands alone, there are over 25 road and rail immersed tunnels in service by year 2022, with the oldest being the Maastunnel in Rotterdam opened in 1942. Thus, immersed tunnels form an integral part of the road and railway network.

A large number of these tunnels have reached or are about to reach half of their design life of 100 years. Large-scale maintenance is usually needed around this time to ensure their performance. For instance, according to Van Montfort (2018) the First Coen Tunnel which was opened to service in 1966 had undergone large-scale maintenance from 2013 to 2014. Similarly, based on maintenance records more and more existing immersed tunnels around the world are showing signs of performance deterioration. Many early immersed tunnels have shown excessively large differential settlements far beyond the anticipated value in the preliminary design stage which resulted in concrete cracking, leakage et al. For example, the Shanghai Outer Ring tunnel in China placed on soft ground was observed to have unexpectedly large joint deformations which caused substantial overcompression and subsequent damage of the Gina and Omega gasket.

Besides differential displacements, there were other problems that deteriorated the serviceability of immersed tunnels. Corrosion of gasket fixature bolts was one such common issue that reduced the watertightness of joints. These issues are becoming more and more evident during inspection phase. Due to the various problems associated with immersed tunnels in operation, maintenance activity requires special attention. Van Montfort (2018) classified the maintenance of immersed tunnels into two categories:

- Structural maintenance: Maintenance of concrete structures, such as tunnel tubes, foundation and connections.

- Operational maintenance: Maintenance of internal finishings such as road asphalt, mechanical and electrical installations et al.

The focus of this thesis is on structural problems encountered during operation of immersed tunnels. During structural maintenance tunnels have to be closed resulting in monetary losses and traffic hindrance to the tunnel user. Hence, taking preventive measures to reduce the need for structural maintenance is more cost effective. An efficient monitoring plan aids in identifying these problems at an early stage. However, the present monitoring plan lacks in terms of measurement accuracy and frequency. For example, deformation monitoring of immersed tunnels was, in most cases, limited to settlement monitoring of limited points longitudinally by manual levelling, measured at yearly or multi-yearly intervals. This failed to capture the transverse deformation pattern and short-term daily or seasonal tunnel behavior. Additionally, there have been major and minor changes to the design philosophies of immersed tunnels in recent decades aimed at improving construction methodology and countering structural problems. Hong Kong-Zhuhai-Macau bridge tunnel is one such immersed tunnel where a double seal was used at the immersion joint in place of the conventional single metal-rubber strip for better watertightness (Lin et al, 2022). This report studies the impact of these changes in design philosophies on long-term behavior of immersed tunnel and an improved monitoring plan is proposed based on the shortcomings of the current strategy.

1-2 Problem Analysis

An immersed tunnel is a form of tunnel that can be used as a waterway crossing. It can function as a road, railway or pipeline. Bridges and bored tunnels are other common alternatives. Lunniss and Baber (2013) stated that the choice between tunnels or bridges depends on waterway operations and environmental reasons. For instance, bridges have to be higher above the water level than the tunnels lying below them. Besides, bridges are prone to stoppages during adverse climatic conditions. Among tunnels, since an immersed tunnel is placed directly on a trench lying at the bed of the waterway, it is relatively shallower than a bored tunnel which lies much deeper in the subsoil as illustrated in Figure 1-1. Thus, immersed tunnels are relatively cheaper than bored tunnels.

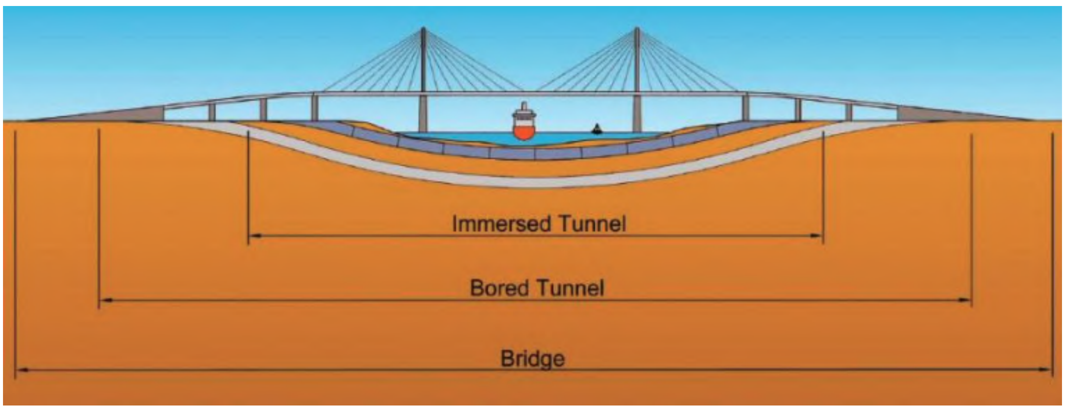


Figure 1-1: Layout of immersed tunnels (Van Montfort, 2018)

Immersed tunnels can be classified into three main categories based the on type of element material selected. They are as follows:

1. Steel shell tunnels
2. Reinforced concrete immersed tunnels
3. Steel concrete sandwich (SCS) tunnels

There are two main types of steel shell tunnels, single and double steel shells. A single steel shell has an outer steel plate stiffened internally which serves as a permanent watertight membrane and provides the necessary structural strength. In double steel, an exterior steel shell serves as the watertight membrane and is used in combination with a reinforced concrete interior lining to fulfill the structural requirements of the immersed tunnel (Gursoy, 1995).

Similar to steel shells, concrete tunnels can be classified into two types, monolithic and segmental concrete tunnels. Before the 1960s, the tunnel elements were built as monolithic reinforced concrete elements of approximately 100 m in length. Building such large concrete sections led to early thermal shrinkage cracking which goes right across the concrete section and creates leakage paths. These leakages reduced the long-term durability of concrete tunnels. Thus, these tunnels had an external waterproofing membrane to make them watertight. To avoid the use of a membrane, which was time-consuming as well as expensive, segmental concrete tunnel was developed in the Netherlands by dividing the element into several individual segments each about 20–25 m long. These segments could be cast without any early thermal shrinkage cracks. These segments were temporarily prestressed together to form a continuous element while being towed and this prestress was cut once placed (Lunniss and Baber, 2013). The steel concrete sandwich tunnels were recently developed by the Japanese. The cross-section comprises a sandwich of concrete placed between two steel plates spaced roughly 300 mm apart. These plates are connected by shear studs and a fluid self-compacting concrete mix used to fill the gap between them. The steel shells carry the tensile stress while the concrete takes the compressive stresses. The steel concrete sandwich is very strong structurally and can withstand large deflections without rupturing (Lunniss and Baber, 2013).

Construction of immersed tunnels is a lengthy process that takes several years to complete. Initially, the immersed tunnel elements are prefabricated at a dry dock. Concrete and steel shells differ in the way these elements are manufactured. Steel shells are made of several modules approximately 4-5 m long. A steel plate with internal stiffeners is rolled to create each module. Afterward, these are joined together by welding to create a single 25 m-long element. Subsequently, the elements are transported to an outfitting facility where internal concrete is placed while they are afloat. In the case of concrete tunnels, elements are constructed by concrete pouring in a casting facility. Segmental concrete elements have an additional step where the elements are prestressed before the dock is flooded and elements are floated to site (Lunniss and Baber, 2013).

At the site, a trench is excavated and an appropriate foundation layer is prepared. This is because immersed tunnels are often located in poor geological conditions. Additionally, if the tunnel is placed on the dredged surface directly there will be a misalignment at the joints as the tolerances that currently available dredging equipment can achieve are inadequate. Depending on the type of foundation layer applied, elements are either placed directly onto the layer or on temporary supports and the gap between the element and trench is under-filled. Once positioned, these elements are joined together to allow articulation of the tunnel while maintaining watertightness (Lunniss and Baber, 2013).

1-3 Problem Definition

Ideally, immersed tunnels should function satisfactorily for their design life of 100 years without requiring any maintenance operations. However, based on recent maintenance activities performed on immersed tunnels existing for more than 50 years several structural issues have risen. Ensuring the life cycle operation of immersed tunnels is a major political, technical and financial challenge for the stakeholders involved. The common structural problems which have been noticed are excessive deformations, concrete cracking, leakage, corrosion, joint gap opening and closure.

Excessive deformations could result in tunnel alignment variation, joint movements, damage to shear keys and concrete cracks. Joint opening and closure could threaten the watertightness of the tunnel. Additionally, these cracks provide a route for water and soil to enter the tunnel. The sand leak could result in greater settlements. Besides, a water leak could cause visual discomfort to the user and endanger road safety if frozen over. Some of these leaks can pass through the joints and be restrained within the gaskets. This ponding of water within the gaskets can lead to corrosion of the gasket components. Furthermore, the reinforcement can undergo corrosion mainly due to carbonation and chloride ingress. Once the corrosion rate becomes substantial the capacity of structural components is further reduced. In summary, these are significant problems because they affect the tunnel's structural integrity, longevity, watertightness and result in high maintenance activity costs.

1-4 Objective

The main objective of the thesis is to improve the life cycle operation of immersed tunnels based on the long-term structural problems encountered during their design life. Additionally, based on the review of these problems feedback to existing and newly emerging design philosophies along with a new monitoring plan to maintain the life cycle safety of the immersed tunnel is proposed. This monitoring plan would further aid in the early detection of structural issues.

1-5 Research Questions

Based on the problem definition given in Section 1-3, the main research question for this thesis is :

How can the life cycle operation of immersed tunnels be improved?

To perform a detailed study on the main research question, several sub-questions were formed to address it. The sub-questions are as follows :

- What are typical structural problems encountered in the operation of existing immersed tunnels?

  The purpose of this sub-question is to identify the typical structural problems of early immersed tunnels by analyzing them based on construction technique, geotechnical engineering and design flaw from a mechanical and structural perspective. By analysing them further, the impact of these problems on the life-cycle serviceability and safety of immersed tunnels can be understood.

- How do these investigations on structural problems feedback to both conventional and recent design philosophies of immersed tunnels?

  This question aims to analyse the feasibility of recent design changes based on the structural problem study. An inventory of new design changes in the past two decades based on cross-section design, element material, transverse prestressing, foundation treatment, waterproofing and joint formation is to be prepared.

- How can structural problems be observed by monitoring?

  This question aims to identify what parameters directly cause and influence the severity of structural problems. Threshold limits for these parameters have to be determined so that monitoring these parameters can serve as an early warning system.

- What improvements can be made to the current monitoring plan?

  The goal of this sub-question is to identify defects in conventional monitoring methods. An inventory of potential sensors and monitoring techniques that can be installed in current and new immersed tunnels is prepared. Finally, based on the usefulness of these sensors to the contractor an improved monitoring plan is proposed compared to current practice.

1-6 Research Approach

Firstly, literature work and project information on immersed tunnelling projects before 2000 were reviewed to gain a general understanding of immersed tunnels. Furthermore, an inventory of structural problems in existing immersed tunnels is prepared. The structural problems were then analysed to identify the potential causes of these problems based on construction techniques, geotechnical engineering and design flaw from mechanical and structural perspective. The first step of the approach is used to answer sub-question 1 of the research questions.

Secondly, an inventory of innovation in immersed tunnel design will be formulated by analysing the design changes implemented over the past two decades. Based on the structural problem study, the impact of these new design changes on structural behaviour and mitigating structural problems will be investigated. This step addresses sub-question 2.

Thirdly, an overview of current monitoring techniques used in conventional practice before 2000 is made. Based on the structural problems, parameters that have to be monitored for early detection of the structural problems are identified. Threshold limits are then set for these parameters which when surpassed served as an early warning system for structural problems. Sub-question 3 is answered in this step.

Fourthly, an improved monitoring plan to bridge the present gaps between safety assurance and practical monitoring work is developed for current and existing immersed tunnels. An inventory of sensors and monitoring techniques that can be used is proposed by reviewing studies done over the past two decades for structural health monitoring of all tunnel types. These sensors are further shortlisted based on usefulness to a contractor constrained by a limited budget. Furthermore, a brief summary of possible remedial measures are given. This approach addresses sub-question 4.

1-7 Report Structure

This section outlines the report structure of the thesis.

In Chapter 2, a summary of the problems of existing tunnels is given. Case studies are also specified to illustrate the common structural problems in detail. The first step of the research approach is addressed in this chapter.

In Chapter 3, an overview of new design trends within the past two decades is provided. This chapter helps to understand design changes in element material, cross-section design, transverse prestressing, foundation treatment method, waterproofing and joint formation. The structural problem study will be used as a feedback loop to further analyse the new design changes. In this chapter, the research approach's second step is discussed.

The conventional monitoring methods used before the year 2000 are briefly summarised in Chapter 4. Afterward, the threshold limits and reasoning behind why vertical displacement, transverse displacement, joint width, crack width, shear key force, leakage detection, chloride diffusivity, pH in concrete, concrete strain and shear modulus of soil were chosen as the parameters to be monitored are discussed. Finally, a brief description of the working principle and applicability of potential sensors and monitoring techniques that are suitable for monitoring these parameters are addressed. The third and a part of fourth step of the research approach is covered in this chapter.

The aim of Chapter 5 is to propose the improved monitoring plan required to provide for a better structural safety check. The sensors and monitoring techniques are further shortlisted based on usefulness to contractors. Finally, a short description of possible remedial actions that can be taken in case of the occurrence of structural problems is provided. This chapter deals with the remainder of the research approach's fourth step.

In Chapter 6, conclusions from this research are presented and future recommendations are provided.

Chapter 2

Typical Structural Problems Of Tunnels Faced During Design Life

In this chapter, the first research question is answered by discussing the common structural issues encountered in immersed tunnels and the lessons that can be learned from them. This chapter provides the necessary background to check the feasibility of new design changes in Chapter 3 and to propose an improved monitoring plan in Chapter 5. Firstly, Section 2-1 gives information on a few projects that encountered structural problems during their operational period followed by Section 2-2 which gives a detailed description of typical problems that are encountered by immersed tunnels.

2-1 Case Studies

2-1-1 Maas Tunnel

It is a monolithic concrete immersed tunnel that is composed of nine elements, each approximately 61.35 m long having a total length of 1373 m and immersed length of 584 m. The tunnel has a width of 24.77 m and a height of 8.39 m. The tunnel was placed on a sand bed made using the sand jetting method. It was the first immersed tunnel constructed in the Netherlands. The problems encountered by the tunnel are:

- Settlement:
  Table 2-1 shows that the majority of the settlements occurred in the initial years.
- Damaged concrete:
  Upon inspection of the tunnel floor and car deck, it was found that the concrete was severely damaged. Concrete spalling (35% of the surface area), high chloride levels near reinforcement and severely corroded reinforcement were noticed at the top side of the tunnel floor. Booltink et al. (2018) stated that due to cracks in the bottom side of the

Table 2-1: Maas tunnel settlements (Grantz, 2001)

Element number	1	2	3	4	5	6	7	8	9
During construction	18+			35					
During 27 years after construction	21	22	14	10	12	16	19	24	21
Totals	39			45					

car deck water penetrated through them resulting in spalling of concrete. Moreover, during rainy and snowy days the concrete was exposed to moist conditions whereas became dry during warm days.

- Reinforcement corrosion:
  According to Blom et al. (2018) chloride intrusion from de-icing salts and ventilation systems led to reinforcement corrosion. Besides, the reinforcement was depassivated due to the presence of water and oxygen resulting in high corrosion rates.

- Concrete cracking during repair work:
  When the damaged concrete was replaced it was observed that concrete cracking was possible during the maintenance work. Usually, the reinforcement can redistribute the loads and prevent crack formation. Here, the embedded concrete was removed so the reinforcement was not effective and the formation of cracks was possible. Blom et al. (2018) confirmed this hypothesis by performing FEM simulations as seen in Figure 2-1.

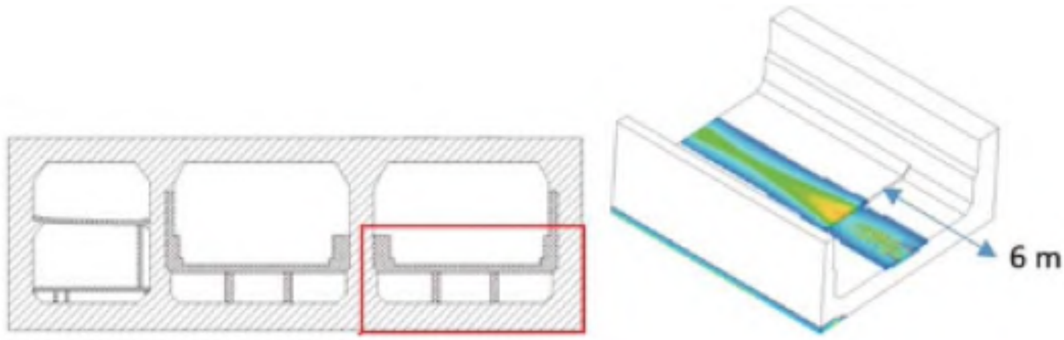


Figure 2-1: 3D FEM simulations to check effectiveness of reinforcement (Blom et al., 2018)

2-1-2 Rotterdam Metro Tunnel

It is a monolithic concrete tunnel with element lengths varying between 50 – 70 m in the land section while for the river section it is between 72 – 93 m. It has 23 immersion joints on land and 10 on the river side. It was the first tunnel to use both the Gina gasket and Omega seal at the immersion joint. The problems it encountered are:

- Deformed Gina gasket:

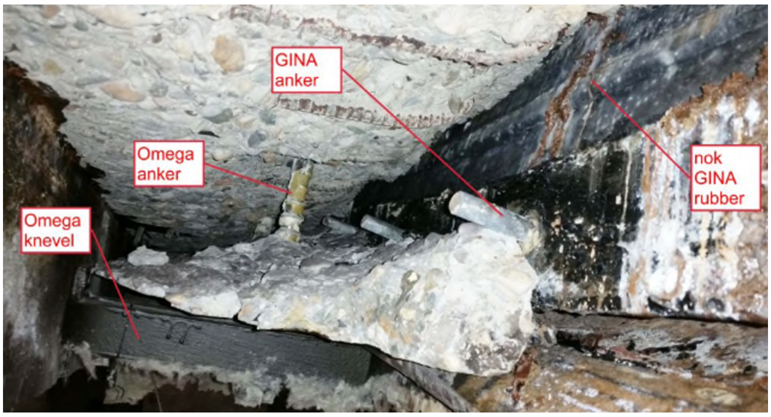


Figure 2-2: Damage picture in early 2015 showing damaged Gina gasket and Omega seal of Rotterdam Metro Tunnel (Molendijk et.al, 2019)

From various inspectionsm it was noticed that the Gina gasket was damaged and locally shifted as the bolts of the Gina gasket had failed as shown in Figure 2-2. Molendijk et al. (2019) claimed that neither differential settlements nor hydrostatic pressure were high enough to cause the failure. Since the cofferdams were filled with sand after the tunnel elements were immersed the joints were filled with sand. Furthermore, the joint widths were found to vary between 90 -130 mm based on water depth and the hardness of the rubber. This was combined with a temperature-induced semi-annual joint movement of 5 -10 mm. Rahadian (2017) assumed that the failure had occurred due to a large surcharge load from the overlying sand in the immersion joints. Due to the joint gap's exposure to over 50 years of loading and unloading cycles arising from seasonal thermal expansion and contraction of tunnel elements, there was an increase in load stress. This, in turn, was hypothesised to further densify soil and increase soil stress. This reduced the mobility of the Gina gasket and forced it to shift inwards. Based on experimental and Plaxis FEM investigations conducted by Rahadian (2017) it was found that the gasket moved continually inwards as the soil density and stresses increased with time. The mechanism behind the inward movement is illustrated in Figure 2-3.

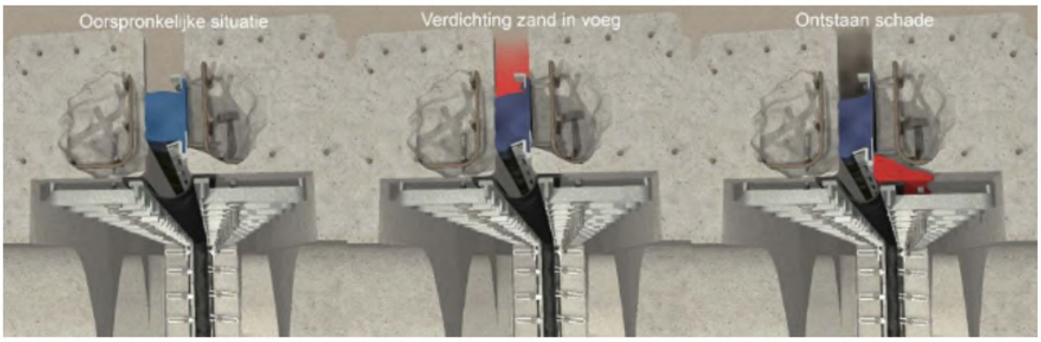


Figure 2-3: Joint movement (left), compaction of sand in joint (middle) and damage occurs (right) in Rotterdam Metro Tunnel (Molendijk et.al, 2019)

- Leakage at immersion joints:
At the start of 2015, a water leak was noticed at the roof of one of the immersion joints. This was found to occur due to the deformed Gina gasket. By the end of 2018, 13 immersion joints in total required quick repair.

- Concrete deterioration:
Further inspections performed at the immersion joint revealed that the concrete was damaged. This could be attributed to a combination of increased edge stresses due to joint gap opening and leakage. Figure 2-4 shows the location of the damaged concrete locations.

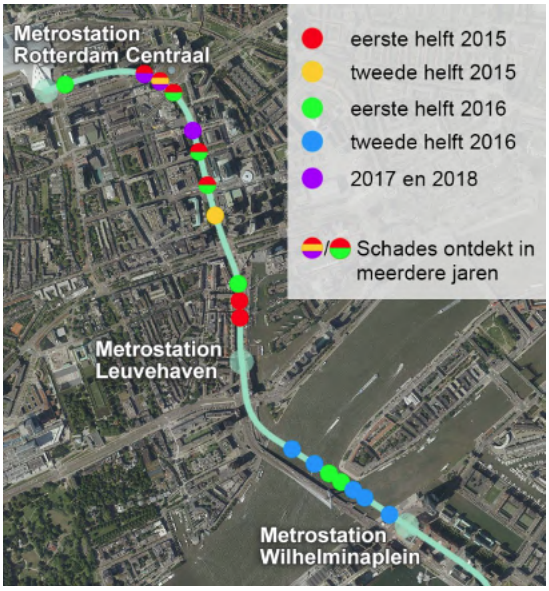


Figure 2-4: Location of damaged concrete in Rotterdam Metro Tunnel (Molendijk et.al, 2019)

2-1-3 First Coen Tunnel

It is a monolithic concrete tunnel with a total length of 1283 m and an immersed length of 587 m. It has 6 tunnel elements having a length of 90 m. It has two tubes with two lanes in each. The tunnel element has a height of 7.74 m and a width of 23.33 m. It encountered the following problems:

- Joint gap opening/ closure:
Based on the report 'Lekkage in Tunnels' by Leeuw (2008) joint gaps greater than 10 mm at certain points were noticed. This led to the tearing of the Omega seal. Cracks of 2 -3 mm were also formed on the Omega seal.

- Leakage at immersion joint:
Two leakages of rates 9 m^3/day were observed at the immersion joints in 2010. This resulted in sand and water leak in the east tube as depicted in Figure 2-5. It was found to occur due to the presence of cracks in the concrete collar. A rectangular concrete

dowel was used at the immersion joint as the shear key. The First Coen tunnel is the only tunnel that has shown a failure of the shear key according to COB (2014).

Figure 2-5: Sand and water leak in east tube of First Coen Tunnel (Leeuw, 2008)

2-1-4 First Heinenoord Tunnel

It is a segmental concrete tunnel with a total length of 1064 m and an immersed length of 574 m founded on a sand bed installed by the sand flow method. The tunnel consists of 5 suspended concrete elements of approximately 115.5 meters long. Each element is further divided into 6 segments of about 19 m with the closure joint situated between elements 4 and 5. It has a twin tube with 3 lanes each. The height is 8.8 m and the width is 30.7 m. The tunnel faced the following problems:

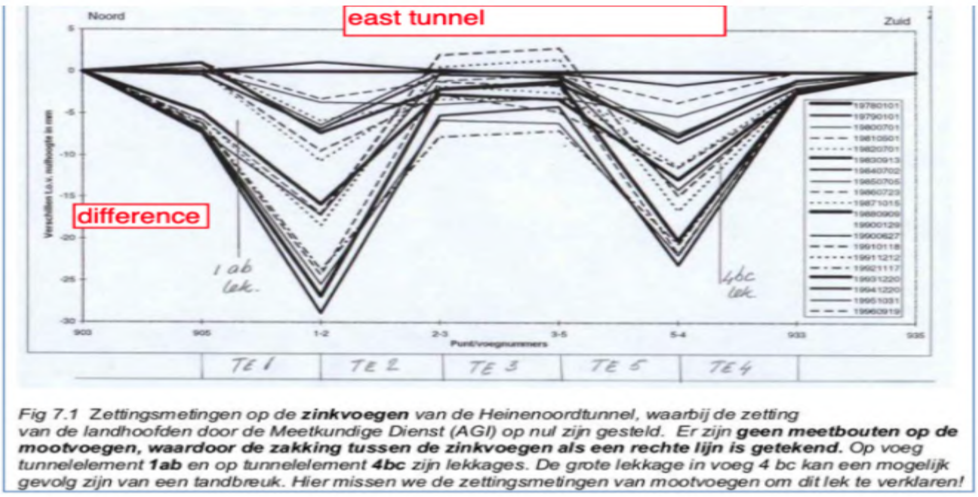


Figure 2-6: Leakage at expansion joints of First Heinenoord Tunnel (Leeuw, 2008)

- Leakage at expansion joints:
  There were leakage concerns in the expansion joints of tunnel element 1 since the construction phase in 1966. Leakages were detected at the expansion joints, joint ab of element 1 and joint bc of element 4, as shown in Figure 2-6. They arose because of the substantial differential settlements that occurred at the expansion joints. At the time of measurement in 1990, this resulted in a high leakage rate of 4 to 5 m^3 per hour.

- Corrosion of bolts:
  Based on endoscopic research done by Nebest (2018) it was found that the bolts were severely corroded with a layer of about 5 -10 mm thick rust, see Figure 2-7.

a)  b) 

Figure 2-7: a) Corroded bolts and b) Deformed Omega seal at immersion joint in Heinenoord tunnel (Nebest, 2018)

- Omega seal deformation:
  The Omega seal had considerable folds as a result of the large settlement of around 40 mm at the connection between element 1 and the northern approach ramp, see Figure 2-7.

2-1-5 Limfjord Tunnel

It is a monolithic concrete having a total length of 945 m and an immersed length of 553 m. It is a twin tube with three lanes each. Each element has a width of 27.4 m and a height of 8.54 m. It is Denmark's first motorway tunnel. The tunnel experienced the following problems:

- Leakage at immersion joints:
  Leakages were noticed immediately after elements were installed and they continued to occur during the operation phase as shown in Figure 2-8. Water leaked through the existing shrinkage cracks in the tunnel roof and outer walls. This was caused mainly due to structural damage and lack of adhesion of the membrane to the structures. Moreover, the length of the tunnel increased annually during the initial 25-year period by 1 mm due to temperature variations. These longitudinal deformations along with settlements resulted in tensile stresses which led to cracks in the concrete. This further contributed to leakages.

- Large differential settlements:
  Settlements amounting up to 130 mm were recorded which was significantly greater

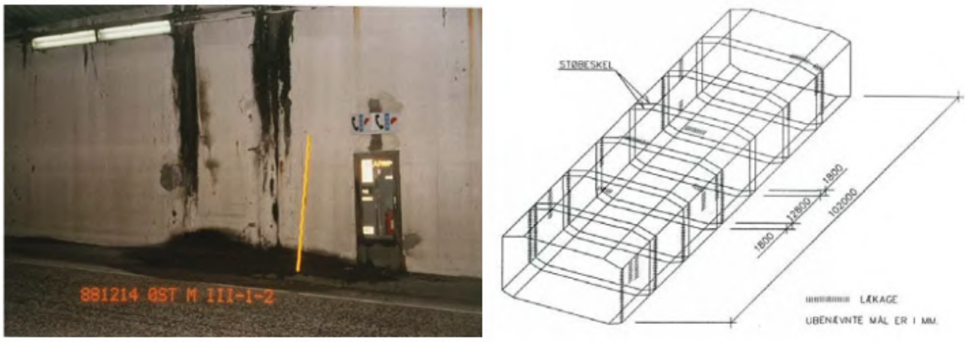


Figure 2-8: Water leakage at the short sections in Limfjord Tunnel (Dimensions are shown in mm) (SEG, 2019)

than the originally predicted settlements of 30 mm. The settlement history is shown in Figure 2-9. These additional settlements were mainly caused due to creep settlement within the uncompacted sand fill below the tunnel elements IV and V (SEG, 2019). The Presence of Gyttja soil lying below the sand would contribute to further long-term creep settlements.

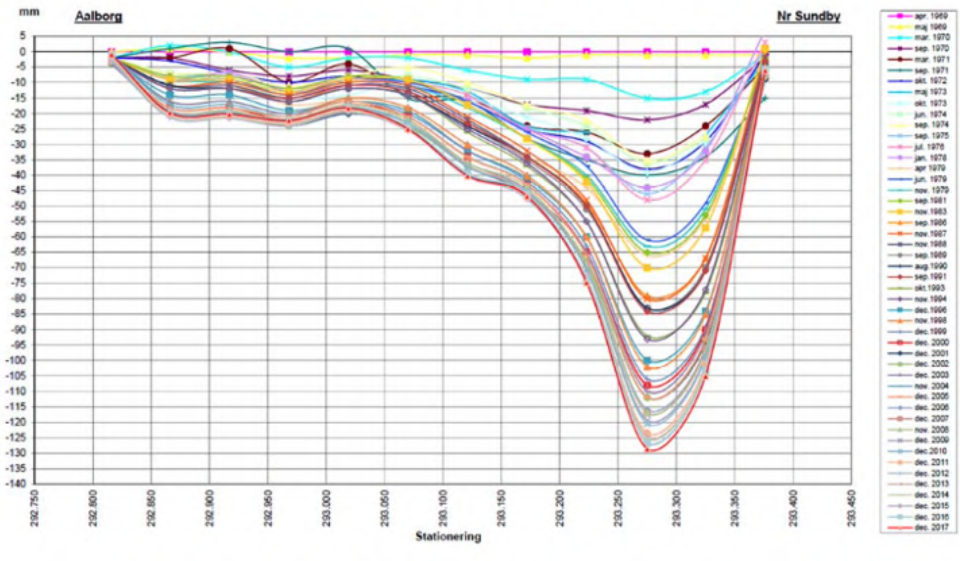


Figure 2-9: Recorded settlements in Limfjord Tunnel during the period 1969 – 2017 (SEG, 2019)

- Reinforcement corrosion:
  The reinforcement was corroded due to prolonged exposure of the tunnel interior to salt water sprays from de-icing salts on the roads and leakage of water through the outer walls and top slab of the tunnel, see Figure 2-10. As a result, the expensive concrete repair was done after a period of 25-30 years. No significant corrosion of reinforcement had been observed in later inspections (SEG, 2019).

Figure 2-10: Reinforcement corrosion from inspection in 1988 (SEG, 2019)

- Delamination of concrete:
  In the late 1990s, the tunnel ceiling had a significant repair in which corroded reinforcement and concrete were removed up to a depth of 200 mm and restored with new reinforcement and concrete. Delamination between the repaired and original concrete occurred sometime after the initial reinforcement/concrete replacement, therefore the repair was strengthened with anchors in 2010. Although delamination was visible in some spots, the quality of the interface between the repaired and original concrete in the roof was deemed suitable for properly transferring shear forces (SEG, 2019).

2-1-6 Kil tunnel

It is a segment concrete tunnel that has a total length of 405 m and an immersed length of 335 m. The longitudinal dimension consists of three elements of length 111.5 m and is further divided into 5 segments. The cross-section consists of a double tube each having a two-lane highway and bike lane. The element has a width of 31 m and a height of 8.75 m. The tunnel faced the following problems:

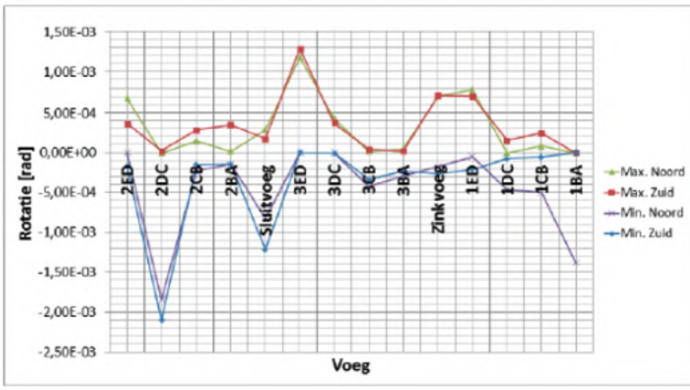


Figure 2-11: Extreme rotations measured at expansion joint of Kil tunnel in south and north (Kil, n.d.)

- Large rotations at expansion joints:
  Figure 2-11 shows the amount of joint rotations at each joint.

- Concrete cracking:
  Gavin et al. (2019) pointed out that cracks were formed in the wall between segments 2C and 2D. Moreover, FEM simulations performed by Yang (2020) predicted that the joint between 1A and 1D would also be prone to cracking. These cracks were formed due to longitudinal differential settlements which resulted in longitudinal bending of the tunnel. Moreover, additional bending was induced during the foundation treatment which involved slurry pressure. This bending induced additional internal force which exceeded the tensile strength of the structure. As a result, cracks were formed at the expansion joints.

  In addition to wall cracking, Leeuw (2008) reported that a tooth fracture occurred in the expansion joint, see Figure 2-12. Large differential settlements, lack of soil support beneath the tunnel and poor structural design all contributed to this. Because of compaction and gravity, sand beneath the sloped regions had sunk. As a result, there were hollow spaces under the tunnel body. Due to this lack of soil support in some areas, the force was redistributed within the concrete tooth. This was further aggravated due to the piping of sand under the tunnel through the leaks present in the structure. Finally, dimensions of tooth reinforcement were set based on differential settlements and failed to take into account the additional line loads that could arise from large joint rotations.

Figure 2-12: Cracking at expansion joint of Kil Tunnel (Gavin et al., 2019)

- Large longitudinal differential settlements:
  More than 80% of the predicted settlements occurred in the Kiltunnel around 34 years after completion. This was much greater than the initial design predicted settlements over the past 40 years. Initially, the settlements were small and concentrated in element 2. When maintenance work was carried out in 2001, large quantities of sand, around 7 m^3, were found in the pumping chamber. The settlements were concentrated at the expansion joints in elements 1 and 2 as shown in Figure 2-13. Even though the settlement limit was crossed in 1977, functionality problems arose only in 2001. Furthermore, based on the results of monitoring settlements in the north and south directions there

was no bending or torsion of the segment as the difference between the north and south settlements did not exceed 5 mm (Yang, 2020).

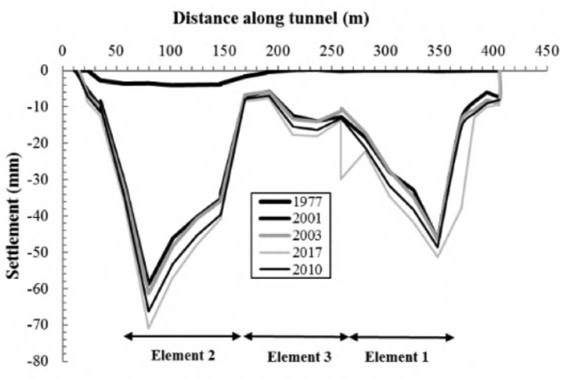


Figure 2-13: Longitudinal vertical settlement of Kil Tunnel measured between 1977 and 2017 (Gavin et al., 2019)

Gavin et al. (2019) concluded that the possible reasons for the large magnitude of settlements were:

i) *Presence of soft soil zones underneath the tunnel*
The geophysical investigation which was carried out later by Gavin et al. (2019) confirmed the presence of soft soils. Under segments, 2B to 2E and segment 1B presence of soft soils was detected. These soft zones were potentially old deeper channels of the river that were infilled with soft sediments or areas of poorly compacted sand that were backfilled into the excavated trench before tunnel placement.

ii) *Cyclic loading effects*
For both cyclic loading and creep effects on soils, Gavin et al. (2019) described the overall behaviour as simple power-law expressions and it was found that the maximum total settlements can be in the of range 10 to 90 mm for the 100-year design life of the structure, see Figure 2-14. Since the tunnel was situated in a tidal river, Yang (2020) claimed that the settlement fluctuated about 6 mm due to seasonal tidal changes accounting for 6 - 9% of the final settlement. In the research of Van Amsterdam (2019), it was already mentioned that initial settlements at the edges of element 3 due to temperature fluctuations were between 0.06 and 0.08 mm and this increased to 0.55 mm in 2018.

iii) *Creep effects*
Van Amsterdam (2019) claimed that creep is not the main driver of the time dependent settlements at the Kiltunnel.

iv) *Leakage*
Based on the settlement profile, the differential settlement between the ends of segments 2D and 2C are 36.1 mm and 12.5 mm respectively. As a result, the outer concrete wall was under tremendous tension while the internal wall at joint 2CD was under significant compression. This could have led to a tooth fracture. Due to tooth fracture at the

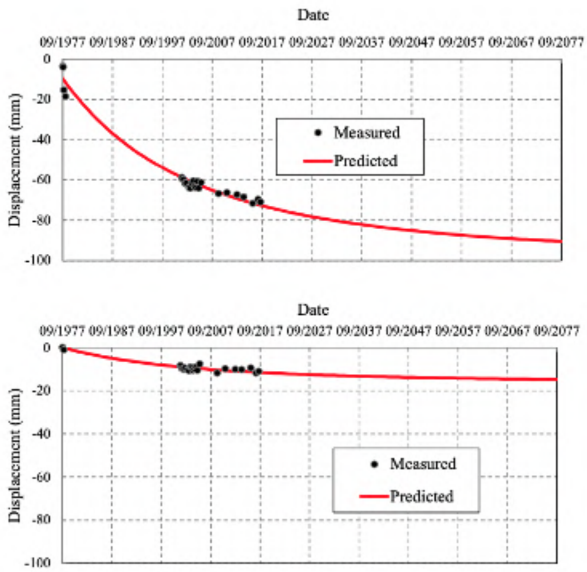


Figure 2-14: Measured and predicted settlement for segment 2C (left) and segment 2E (right) of Kil Tunnel (Gavin et al., 2019)

expansion joints, a water inflow of 1.5 l/s and a sand quantity of approximately 10 l per week was observed. The presence of leakage at the expansion joints is illustrated in Figure 2-15. Yang (2020) demonstrated through FEM simulations that the leaks do not impact the settlement. However, the leaks changed the composition of the base layer which may affect settlements. Due to leakage inflow, there was greater water pressure at the periphery of the pancake which meant that more sand was washed away and a nearby clay layer might have flowed in. This foundation layer change was found to be a minor factor in settlement.

Figure 2-15: Leaky expansion joint with crack in concrete cover of Kil Tunnel identified in 2001 (left) and in December 2019 (right) (Yang, 2020)

v) *Relocation of levees*
Both the east and west sides of the levee moved further east and west, widening the river channels. This widening was due to dikes being displaced as a result of the Dordtse

Kil river being canalized. The soft soil under elements 1 and 2 suffered from a loading-unloading-reloading process as a result of the levee relocation which could result in large settlements. High water content, large void ratio, high compressibility and low bearing capacity are all features of soft soil. The soil will exhibit large rebound and compression deformation during unloading and reloading cycles. Furthermore, soil rebound and compression consolidation lasts for a long time in clays. This results in long-term settlements. Yang (2020) validated this assumption through FEM investigations and concluded that it was the key factor behind the settlements.

- Corrosion of gasket fixture bolts:
  Based on endoscopic research done to understand the condition of the clamping structure it was noticed that all the chambers underneath the floor were filled with water. Moreover, it was found that the Omega gasket fixture bolts were corroded due to exposure of gasket fixture bolts to water leakages as seen in Figure 2-16.

a) 

b) 

Figure 2-16: a) Corroded clamping structure at the wall b) Rusted Omega gasket joint fixture bolts of Kil Tunnel (Leeuw, 2013)

2-1-7 Shanghai Outer Ring Tunnel

It is a monolithic concrete tunnel that consists of seven elements each having the length of approx. 108 m and numbered E1 to E7 from west to east. It is a 2882 m long highway tunnel with an immersed length of 736 m. The cross-section consists of three tubes with eight lanes and two service galleries for escape and utilities. The element has a cross-sectional width of 43 m and a height 9.55 m each. The tunnel faced the following problems:

- Large differential settlement
  The accumulated settlement curve of the tunnel since 2003 is given in Figure 2-17. The largest settlement occurred at the joint between E5 and E6 reaching 230 mm in 2008. By 2004 this settlement had reached 120 mm accounting for 50% of the total settlement. The central part of E7 experienced a relatively large settlement of 40 mm in 2007. The largest differential settlement of 172 m occurred between the ends of element E6.

By grouting into the foundation under the E5/E6 joint and element E7 in 2005 and 2007, respectively, the settlements at these critical locations were controlled according to Wang et al (2020).

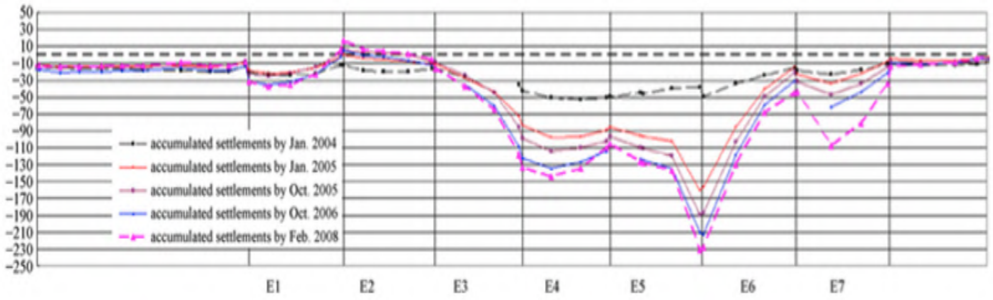


Figure 2-17: Accumulated settlement of Shanghai Outer Ring Tunnel measured from north hole points (compared with original data in October 2003) (Wang et al, 2020)

- Deformed Omega and Gina gaskets

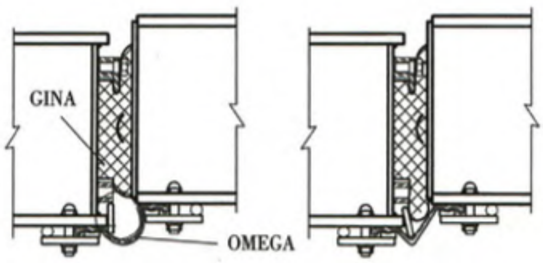


Figure 2-18: : Damage of Omega gasket of Shanghai Outer Ring Tunnel due to over compression of GINA gasket a) GINA bolts are sheared (b) Clamping and bolts squeeze Omega (Bai and Lu, 2016)

After 3 years of tunnel operation, it was found that compression of the Gina gasket at the joints between elements E3/E4 exceeded the maximum allowable compression amount proposed in the design and Gina gasket positioning plates at individual positions were dislocated. As a result, the Omega rubber profile was broken in multiple places, see Figure 2-18. This abnormality was noticed when the omega gasket showed a continuous bulging phenomenon. On further inspection via endoscopy, it was noticed that the Gina gasket was cracked and extruded sideways. Consequently, during the extrusion process, a lateral force was generated which cut off the bolts connecting the clamping plate. This made the counter plate and bolts fall on the Omega gasket (Bai and Lu, 2016). Furthermore except for joints between elements E1 /E2 and E6 /E7, the compression of the GINA gasket of other joints exceeded the maximum design compression during operation. Moreover, the amount of compression displayed an increasing trend, especially in summer, see Figure 2-19. This was due to the thermal expansion of the immersed tunnels in summer which reduced the joint spacing and compressed the

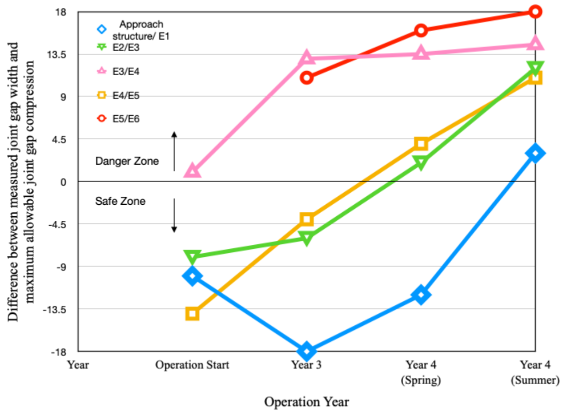


Figure 2-19: Variation of GINA gasket compression of Shanghai Outer Ring Tunnel (Bai and Lu, 2016)

Gina waterstop (Bai and Lu, 2016). The compression of the immersion joints is related to the settlement history. Therefore, the excessive compression between the elements caused by differential settlement was the main reason for Gina gasket deformation.

Table 2-2 provides an overview of the immersed tunnels which provided critical information about these problems.

2-2 Common Structural Problems

The quality of immersed tunnels usually deteriorates once they reach half the age of their 100-year design life. Hence, large-scale maintenance is often needed. Maintenance involves replacing, repairing and maintaining parts of the structures. These actions aid in maintaining the accessibility of the region and the quality of the immersed tunnel. The ideal way to limit the need for maintenance is early detection of structural problems by applying a robust monitoring plan. Identifying the common structural problems and analysing them from the perspective of structural, geotechnical engineering and construction technique is necessary to adopt an appropriate monitoring strategy. The common problems which are encountered during the lifetime of immersed tunnels are discussed further in the following sub-sections.

2-2-1 Excessive deformations

Immersed tunnels are pretty lightweight having only approximately 10% more weight than water when fully ballasted. This is much lighter than the displaced soil. Furthermore, immersed tunnels are usually placed on soft soil strata. Despite this, settlements are often quite large, exceeding the values predicted by detailed geotechnical assessments. Based on analysis of long-term behaviour of past immersed tunnel projects it was found that relative displacements occurred across the length of the tunnel. Owing to the stiffness properties of the tunnel, these deformations tend to concentrate at the joints. Furthermore, Zhang and Broere (2022) describe three possible directions in which they can act.

Table 2-2: Overview of immersed tunnels between 1941 and 2003 which encountered structural problems

Tunnel Name	Year	Location	Typical Problems
Maas Tunnel	1941	Rotterdam, Netherlands	Settlement, reinforcement corrosion, concrete spalling and cracking
Rotterdam Metro tunnel	1965	Rotterdam, Netherlands	Leakage at immersion joint, concrete deterioration, deformed Gina gasket, joint gap opening/ closure
First Coen tunnel	1966	Amsterdam, Netherlands	Leakage at immersion joint, cracking in concrete collar, cracking of Omega seal, joint gap opening/ closure
First Heinenoord Tunnel	1969	Barendrecht, Netherlands	Leakage at expansion joints, corrosion of bolts, Omega seal deformation
Limfjord Tunnel	1969	Aalborg, Denmark	Large differential settlements, leakage at immersion joints, reinforcement corrosion, delamination of concrete
Kil Tunnel	1978	Dordrecht, Netherlands	Large differential settlements, leakage at expansion joints, corrosion of gasket fixture bolts, concrete cracking, large rotation at expansion joints
Shanghai Outer Ring Tunnel	2003	Shanghai, China	Large differential settlement, deformed Omega and Gina gaskets

1. Vertical uneven settlements at two sides of a joint. They are described in this section.

2. Horizontal displacements resulting in joint gap opening and closure, discussed in section 2-2-2

3. Transverse segment/ element drift
   Before the locking fill is placed, elements are quite sensitive to lateral displacement as the effective weight of the tunnel at the bottom is only 7 -10%. Consequently, the friction between the element and foundation is very low. Grantz (2001) stated that tunnel elements or segments can be displaced transversely due to poor back-filling operations. As lateral displacement at joints is rarely monitored this displacement is not further discussed in this chapter. Moreover, Zhang and Broere (2022) suggested that due to inadequacy of conventional monitoring it is yet to understand if transverse displacement contributes to collar damage.

Grantz (2001) identified the causes of settlements as:

- Sub-soil conditions
  Stiffness of the underlying soil layers is the most important factor that influences the magnitude of settlements. If consolidated sands and silts with little or no rebound are present in the sub-soil, relatively small and immediate settlement occurs. On the other hand, the presence of thick layers of compressible soils such as clay with a considerable rebound in the sub-soil leads to large settlements which continue for many years due to creep effects. If these settlements are uniform, the life cycle of the tunnel is not affected.

- Siltation
  Siltation of the dredged trench can obstruct the placement of an element to the proper elevation if allowed to accumulate sufficiently. This results in substantial differential settlements and thus alters the tunnel alignment. Similarly, river silt can neutralise the negative buoyancy of an element causing it to float off. This is quite severe in rivers with heavy bed loads. The type of foundation layer installed in the trench for the tunnel element to finally rest on also affects the amount of siltation.

  Before installing a jetted sand or sand flow foundation, there is a period where the tunnel element is placed for a few days. During this period, fine materials can flow into the space beneath the tunnel element. This can be mitigated to an extent by direct removal with dredges, protecting the space under the element with hanging curtains and using the sand jetting method to remove water and silt from the trench as the foundation is being laid by pumping a layer of sand and water. A gravel bedding course has the advantage of being laid before the element is placed. This significantly reduces the time available for fine-grained material to contaminate the gravel layer.

- Method of tunnel foundation construction
  Screeded gravel beds and jetted sand or sand flow methods are conventionally used in immersed tunnel foundations in USA and Europe respectively. However, the main factor determining the choice of foundation method depends on local geology, environment and other considerations. The degree of settlement in these two methods is determined by the thickness and consolidation of foundation materials as well as the gap left under

each element before load transfer to the bedding. For instance, the sand flow treatment method was applied to Shanghai Outer Ring Tunnel and Zhujiang Tunnel. Due to different sub-soil conditions, the largest settlement in the Shanghai Outer Ring Tunnel was about 8 times larger than that of the Zhujiang Tunnel (Wang et al, 2020).

- Surcharge
  A high surcharge load rising from backfill can result in higher settlements. Ted Williams Tunnel and Fort McHenry Tunnel showed settlements up to 15 cm when the backfill load was placed. Additionally, in fully transversed ventilated tunnels, a large ventilation building is required at either one or both ends of the tunnel. To achieve this, a dike may be constructed on top of the element, see Figure 2-20. This results in further surcharge load and makes the joint between immersed elements and approach structures or ventilation buildings prone to differential settlements.

Figure 2-20: West cofferdam Fort McHenry tunnel. Tunnel elements with closure dike on right (Grantz, 2001)

- Trench dredging methods
  Different types of equipment are used for dredging resulting in different magnitude of settlement based on soil type. Hydraulic cutter-head suction dredges are used for shallower water while clamshell bucket dredges are commonly used in deeper water. In sands, the dredging equipment doesn't have much effect on the settlement. For stiff clays, the cutter-head disrupts the bed less than a grab bucket. This is because grab buckets leave a more irregular bottom with large voids that take longer for the foundation material to fill and stabilize. Blasting methods are used in rock layers. The gravel foundation layer is later placed on the blasted surface. Long-term settlements can arise from the gradual migration of foundation material into the fissures and broken rock located at the trench bottom.

- The geometry of tunnel element
  Based on the shape of the element, contact width can differ. American tunnels are mostly steel tunnels with an octagonal shape. This makes the plan width bigger than the contact width. For instance, two-lane Hampton Roads elements only have a contact width of 44% of the plan width. On the other hand, the four-lane Fort McHenry and Ted Williams Tunnel elements have contact widths that are approximately 70% of the

projected plan width. Concrete tunnels commonly used in Europe tend to adopt a rectangular box shape. They have a corresponding contact width of approximately 100% of the plan width. A reduced contact width means increased pressure due to the weight of the element and protective backfill is concentrated in a smaller area. This results in increased settlements during back-filling.

- Dynamic loading
  Tidal variation, wave action, vehicle loading, earthquake loads et. al. all come under dynamic loading. Yang (2020) pointed out that cyclic large amplitude tidal variation or wave action can result in daily pore pressure oscillations and settlements, especially in weak layers like silt or clays. If the foundation is insufficiently compacted then it can result in scour effects. The scour effect is greater in the slopes due to less compaction and gravity effects. Vehicle and pedestrian loads can also lead to significant settlements. Tingstad tunnel, located in Sweden, experienced a large settlement of 50 mm due to vehicle load. Earthquakes are rare but if it occurs the vibrations and shock loading can cause liquefaction of sand flow foundation and hence, substantial settlements. In soft soils, earthquakes increase the pore pressure resulting in further consolidation.

- Voids left around the element after placing the element and backfilling
  If voids are present in the bedding layer this can be prone to water flow and hence, a locally attenuated bed. Even though this hasn't resulted in a long- term settlement problem, care must be taken to eliminate this factor.

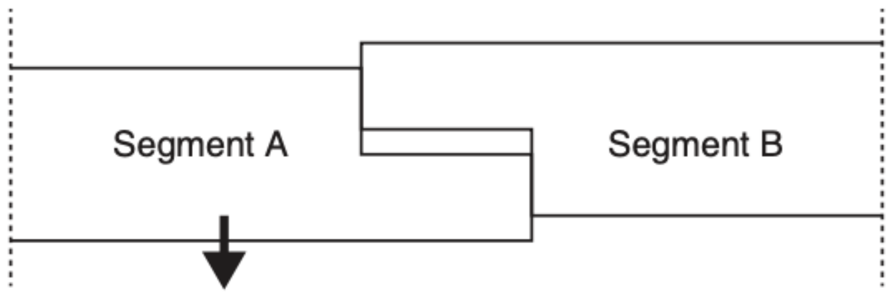


Figure 2-21: Differential settlement between two adjacent segments where segment A settles more than segment B

Rather than the magnitude of total settlements, differential settlements and joint rotations are more detrimental. If settlement between two successive segments is different, then differential settlement takes place between them, see Figure 2-21. Differential settlements can be due to:

- Variable soil profile
  Due to spatial variability of soil profiles, a section of the elements may be placed on soft compressible soil while the other is on relatively stiffer incompressible soil. The section on the more compressible soil settles more than on the stiffer soil (Wu, 2017).

- Variations in foundation loading
  Since the water depth varies along the length of the tunnel,different hydraulic loads act on them which could result in differential settlements. Moreover, deeper tunnel

elements could suffer from larger siltation effects. These will alter the loading on the elements (Wu, 2017).

- Variations in-situ conditions
  Stress history could vary along the soil profile. For instance, a part of the tunnel could have been the site of a large structure that was demolished later or a sloped bed where a large thickness of overburden soil was removed to form a level ground. These varying soil stress state changes before and after loading would result in differential settlements or swelling (Wu, 2017).

- Sudden change in structural form
  Changes in structural form can be noticed at connections between immersed part to adjacent ventilation or cut-and-cover tunnels. The structural loadings acting on the cut-and-cover section are usually higher than those on the immersed section. Therefore, construction is carried out in different ways to accommodate the difference in loading. The cut-and-cover section tends to be stiffer while the immersed section is flexible. If a suitable connection is not made between the two sections, large shear loads would act at the joint resulting in large differential settlements (Lunniss and Baber, 2013).

Differential settlements can be classified as follows:

1. Differential settlements longitudinally

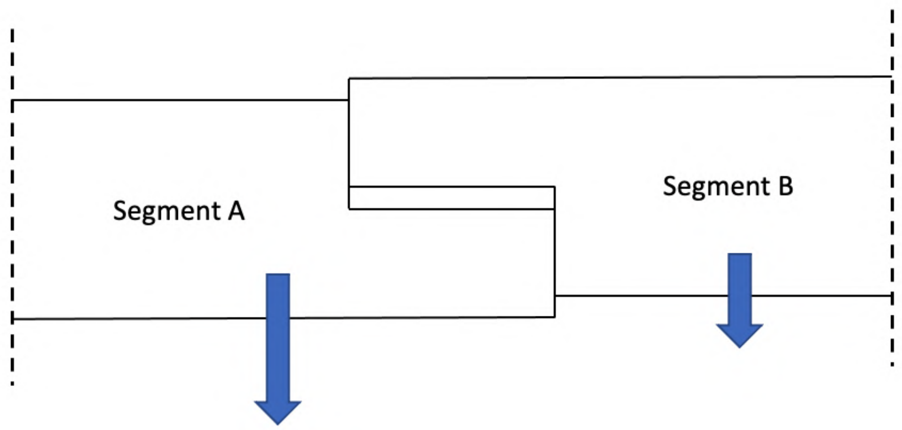


Figure 2-22: Longitudinal differential settlement between segments

These settlements can lead to changes in tunnel alignment, gasket deformation, concrete cracking and shear key damage due to settlement-induced large bending or shear stresses. Figure 2-22 illustrates the change in tunnel alignment when segment A settles more than segment B. Concrete tooth fractures due to large differential settlements at expansion joints were noticed for the first time at the Kil tunnel when a sand leak was noticed by the manager (Leeuw, 2008).

2. Uneven settlement transversely
   Figure 2-23 shows a tunnel element where point A settles more than C leading to transverse uneven settlement. This can potentially lead to torsion of tunnel cross-section along with the same effects of longitudinal differential settlements (Vervuurt et al., 2013).

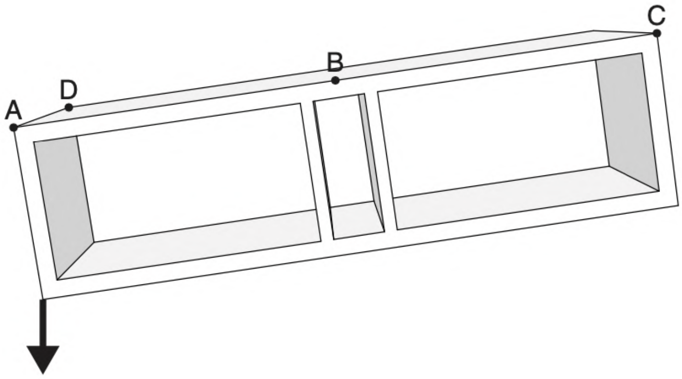


Figure 2-23: Transverse uneven settlement

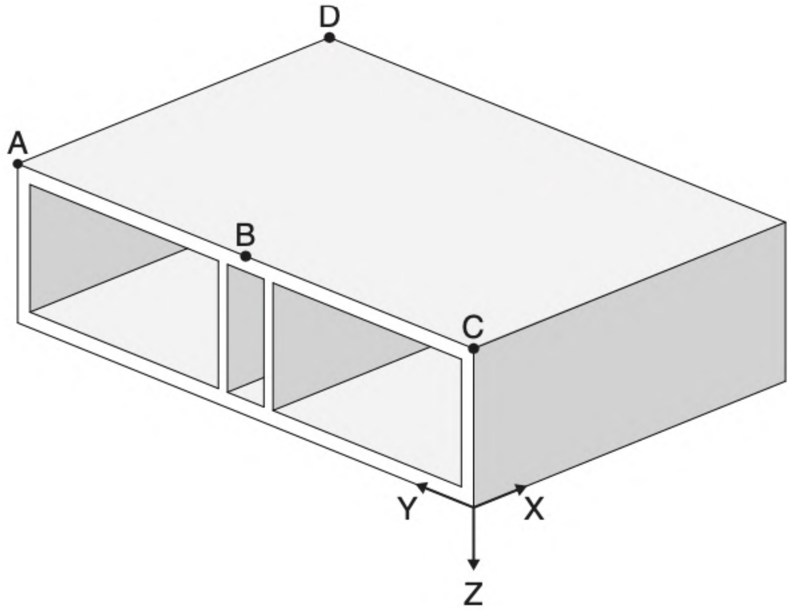


Figure 2-24: Tunnel section profile

Another consequence that can occur for both longitudinal and transverse differential deformations is the rotation of the tunnel body. The rotation of a segment can occur within a segment or between two consecutive segments. If two displacement measurement points within a segment are uneven, the segment exhibits rotation around the x-axis (points A and C in Figure 2-24) or y-axis (points A and D in Figure 2-24). This leads to a reduction in contact surface and hence, reduced tensile force transfer (Kil, n.d.). On the other hand, the segment undergoes bending or torsion if three or more measurement points within an element are unequal as shown in Figure 2-25.

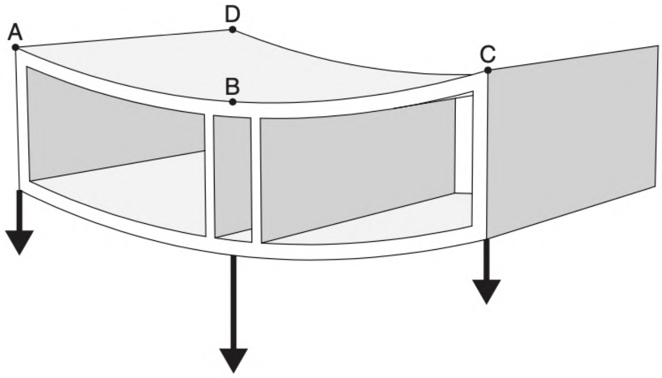


Figure 2-25: Bending of tunnel element

When the rotation of two consecutive segments differs there is a joint rotation occurring, see Figure 2-26. This results in compression and tensile zone. For a positive joint rotation, the compressive force is at the bottom of the cross-section while for a negative rotation, the compression is at the top. Since the expansion joint is unable to transfer tensile forces, the tension zone is fictional. This results in additional compression forces within the reduced contact zone (Yang, 2020). When the segments are stretched an arching effect is created which introduces a normal force on the cross-section. The normal forces at the top and base of the cross-section form a couple which introduces another bending moment. This bending moment acts opposite to the direction of the joint rotation, see Figure 2-27. This rotation effect can cause tunnel misalignment, shear key damage, concrete cracking, gasket deformation and leakage.

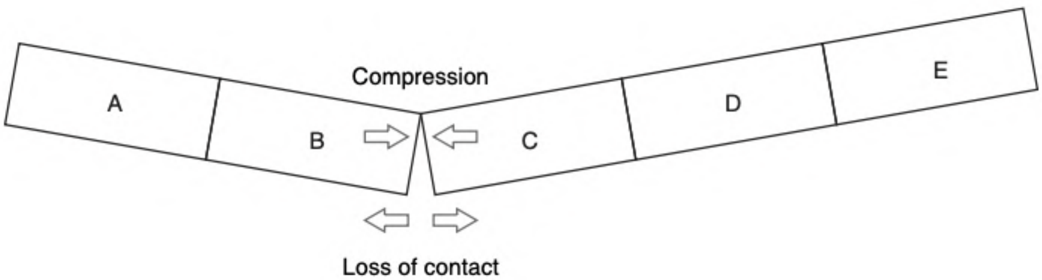


Figure 2-26: Excessive rotation between consecutive segments

In general, steel shell tunnels are more tolerant of differential settlements than concrete tunnels. The ductility and composite nature in which steel shell works with reinforced concrete help the tunnel to resist hydrostatic and earth pressure loads. As a result, the steel shell is flexible in both longitudinal and transverse directions. This was noticed in Hampton Road tunnels which underwent vertical differential settlement of 0.3 m over a distance of around 300 m and still did not have any leakage along the element body or joints. Whereas, concrete is brittle in nature which makes it less tolerant to differential settlement (Grantz, 2001). According to Fugro (2018), if the differential settlement between adjacent segments or elements is beyond 5 mm there is a probability of 80% that significant deformation has taken place.

Whether this deformation is significant to the immersed tunnel has to be investigated further. For transverse displacements, the threshold is higher and is set as ±10 mm.

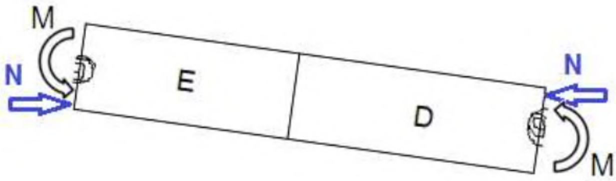


Figure 2-27: Opposing moment (Schols, 2012)

2-2-2 Joint gap opening and closure

A gap can arise in the joints due to temperature-induced longitudinal movements, rotation of the tunnel body and differential settlements. Concrete tunnel elements and segments that have not been piled are susceptible to longitudinal movement as a result of temperature variation causing expansion in summer and contraction of the concrete body in winter. This results in the opening and closure of the joints (Leeuw, 2008). When the concrete shrinks expansion joints get filled with soil from outside or dirt from inside the tunnel and when the joint expands again it exerts pressure on the adjacent segment. Over time, joints become wider and rubber gaskets stretch and deform. Even though newer tunnels use ACME rubber profiles fitted in small square recesses in the expansion joints to counter this, the rubber profile can become loose.

Table 2-3: Typical Gina gasket deformation capacity (Sinha, 2017)

Gina gasket type	SLS range (mm)	ULS range (mm)
G150 - 125	30 - 90	25 - 95
G225 - 275	40 - 135	40 - 142
G220 - 205	50 - 157	40 - 167
G300 - 370	60 - 190	50 - 200

As long as the Gina gasket remains prestressed, limited joint opening or closure is acceptable. The deformation capacity of typical Gina profiles used in the current immersion joint design is given in Table 2-3. Deformation caused by the opening and closure of these joints in the long term is often compensated at the immersion joints. However, the Gina gasket can undergo overcompression and decompression leading to gasket damage if the limit is exceeded. Correspondingly, the Omega seal gets compressed and stretched. Figure 2-28 shows the effects of joint opening on rubber gaskets. At these extreme joint openings, leakages occur due to a lack of prestressing between the Gina gasket and counter plate. These leakages will further deteriorate the durability of immersed tunnels and can accelerate concrete degradation by allowing ingress of chlorides and water. At the expansion joints, the integrity of the rubber waterstop deteriorates due to this cyclic movement (Zhang and Broere, 2022).

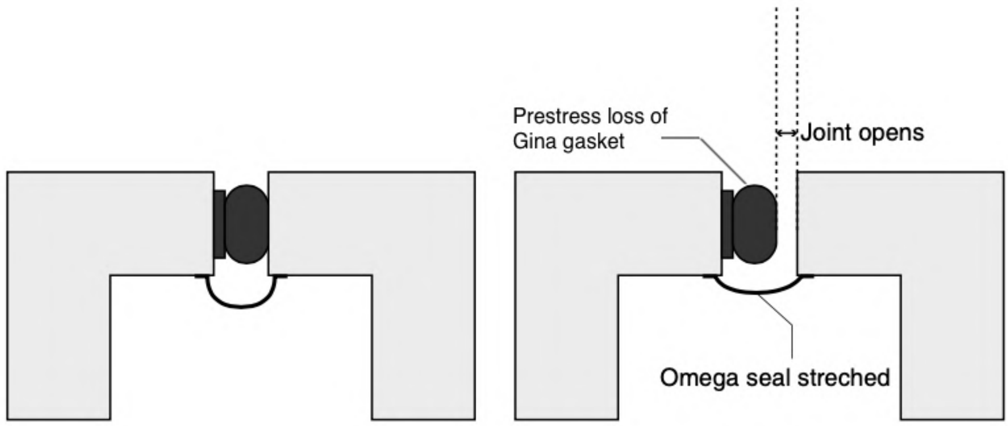


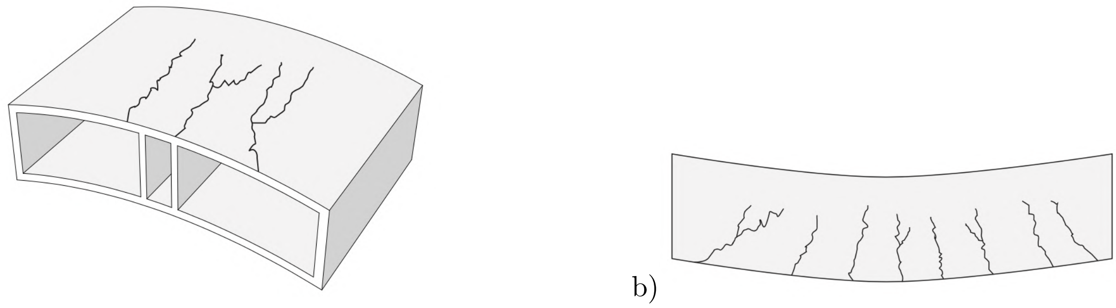
Figure 2-28: Joint gaps in normal status (left) and Gina gasket decompression and Omega seal stretching as joint opens (right)

Furthermore, variation in joint widths across the joints can result in joint rotations. These rotations put added pressure on the joints as shown previously in Figure 2-26. Depending on the type of joint rotations, the edges of joints come under pressure as the floor or roof elongates. For small joint rotations, no problem arises. However, there is a high-stress concentration on the bearing surface of the joint surface which becomes bigger when it is combined with differential settlements. According to Sinha (2017), when the horizontal displacements are greater than 10 mm the joint gap undergoes significant opening. As a result, a high line load develops on the concrete collar and cracking takes place. Moreover, the concrete might peel off, see Figure 2-29 (Schols, 2012).

Figure 2-29: Delaminated concrete (Schols, 2012)

2-2-3 Concrete cracking

Concrete cracks are caused due to thermal effects, bending and twisting of the element due to differential settlements along various parts of the body, Alkali-Silica Reaction (ASR), sulphate attack, surface damage by frost-thaw and inadequate reinforcement. Longitudinal cracks are caused by transverse bending, see Figure 2-30. The temperature gradient across the thickness of slabs and walls results in restrained movements which contribute significantly to transverse bending moments. During winter when the concrete shrinks, tensile stresses are created in the concrete. Since concrete has low tensile strength, shrinkage cracks could emerge. On the other hand, transverse cracks are due to longitudinal bending, see Figure 2-30 (Sinha, 2017).

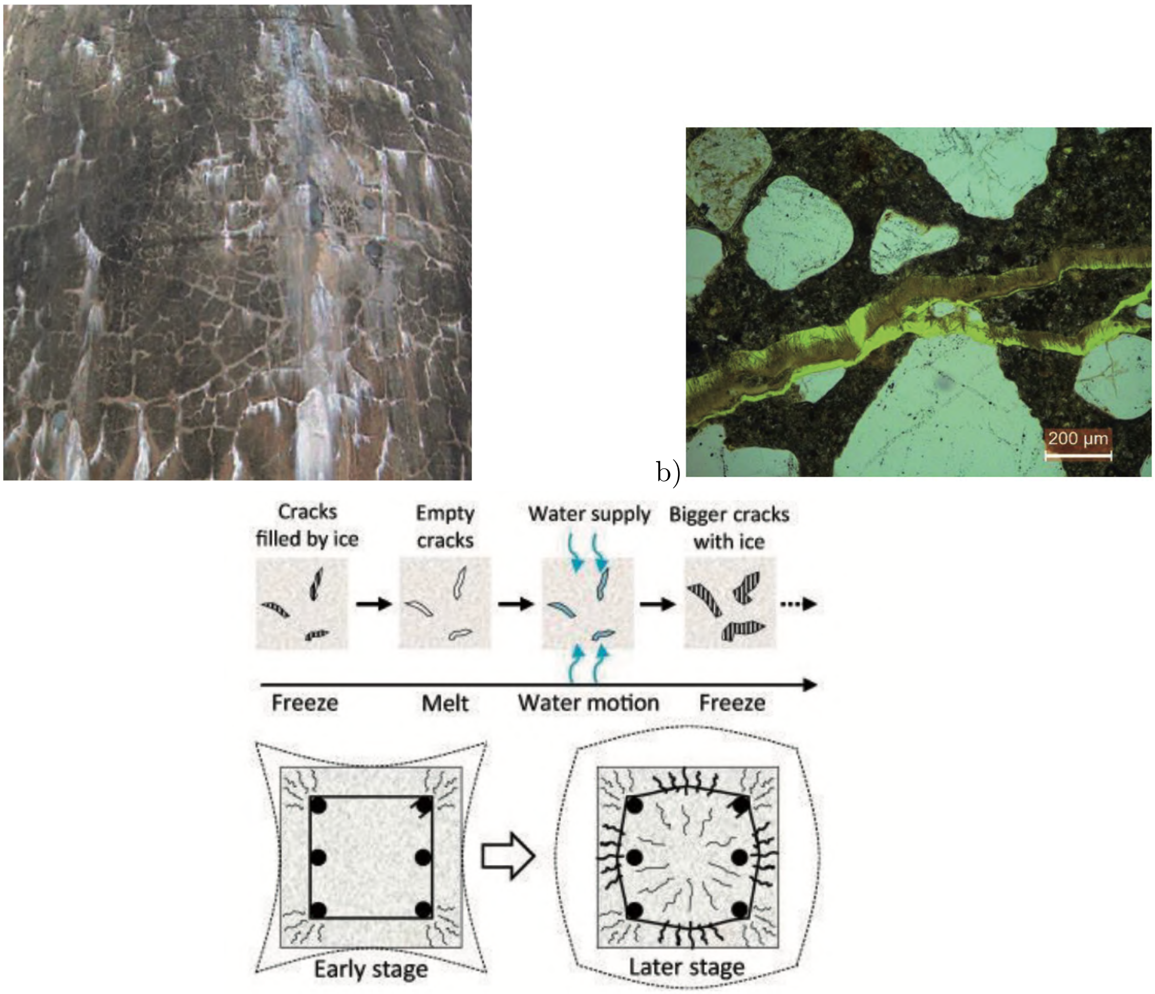


a) b)

Figure 2-30: a) Transverse bending b) Longitudinal bending of a tunnel element

COB (2021) stated that ASR cracks are caused due to presence of reactive aggregates in the concrete which form a silica gel by reacting with alkali hydroxide in concrete. Subsequently, this gel adsorbs water present in the environment resulting in an expansive pressure on concrete. These cracks are usually noticed as random map cracking and spalled concrete on the concrete surface. As it is a gradual reaction, these cracks are usually observed after 5 to 30 years. Sulphate attacks on the concrete surface result in formation of products such as ettringite and thaumasite. These sulphates are caused due to polluted marine environments such as harbours or inadequate concrete mix design. When the concrete hardening temperatures are greater than 70° C no exterior sulphate source is needed. This results in delayed ettringite formation (DEF). Cracks due to sulfate attacks are noticed on the tunnel exterior (COB, 2021). Permeability of concrete increases due to poor execution or mix design. When de-icing salts are applied on the concrete surface during winter, melt water seeps into the pores of the concrete. When this water freezes it puts additional pressure on the surrounding concrete. During multiple such freeze-thaw cycles, the concrete cracks and crumbles. This surface damage due to frost-thaw is noticed on the tunnel interior (COB, 2021). Figure 2-31 illustrates the effect of ASR cracks, sulphate attack and surface damage by frost-thaw.

To better understand the structural mechanism of the impact of various loading conditions on cracking behaviour at expansion joints, Parwani (2014) performed finite element method (FEM) simulations. The effect of loads due to hydrostatic pressure, shear force and rotation at the expansion joint were analysed using 3D FEM simulations of a twin tube immersion tunnel. It was found that shear stress was concentrated at the walls of cross-section of the expansion joint. Around 30-35% of the shear force was found to pass through the inner and outer walls depicted in Figure 2-32. Since the inner wall contains lesser reinforcement, it



a) b) c)

Figure 2-31: a) ASR cracking b) Sulphate attack c) Surface damage due to frost thaw cycles (COB, 2021)

is more prone to damage. Among the various loads, the bending moment was found to be more critical than the shear force for tunnel stability. A limitation of these analyses was that no soil structure interaction was considered and only linear elastic analyses were performed. Moreover, torsion and rotation effects representing field conditions were not considered.

The presence of cracks would provide a pathway for leakages to occur. Corrosive materials such as chloride ions and carbon dioxide could enter and initiate reinforcement corrosion. Due to marine and highly corrosive environments outside and within the tunnel respectively, strict limits on flexural crack widths were proposed by Lunniss and Baber (2013). On the external surface, a limit of 0.15 - 0.2 mm was specified while in interior faces it was between 0.2 - 0.25 mm for concrete tunnels without protective membrane. A stricter control would be needed if a protective membrane was used. Through-section cracking is not allowed in concrete tunnels.

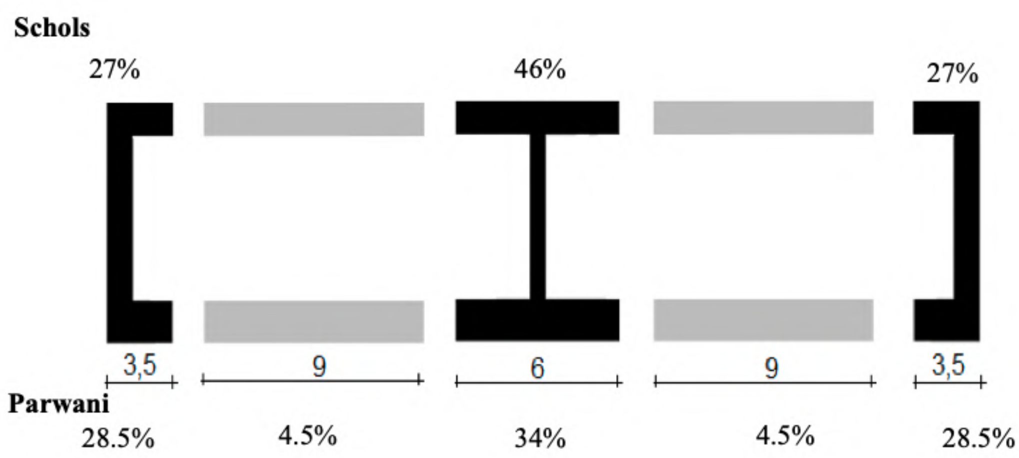


Figure 2-32: Percentage of shear force that passes through parts of the cross-section (Parwani, 2014)

2-2-4 Corrosion

Corrosion of steel is a common risk present in reinforced structures. In the case of immersed tunnels, this causes material degradation, reduces watertightness and threatens the structural integrity of the immersed tunnels. A good corrosion prevention strategy is needed so that the tunnel reaches their design life.

The four main types of corrosion mechanisms taking place in immersed tunnels are :

- Chloride-induced uniform corrosion: This type of corrosion is initiated when chlorides from sources such as concrete mix, sea water and de-icing salts penetrate through the concrete and reach reinforcement steel. They penetrate the concrete as ions dissolved in the water-filled pores (Mayer et al, 2018).
- Carbonation-induced uniform corrosion: Carbonation is a chemical reaction between carbon dioxide in air and water present in pores of the concrete to form calcium hydroxide, sodium, potassium and calcium. This reduces the pH level in concrete which increases the corrosion rate of reinforcement (COB, 2021).
- Microcell, macrocell and pitting corrosion: During uniform corrosion occurs several micro-cells are formed. In contrast, when the chloride ion concentration is high it breaks down the passive film protecting the steel locally to form macro cells where the pits act as anode and the depassivated steel section behaves as the cathode. This type of corrosion is referred to as pitting corrosion (COB, 2021).
- Electro-galvanic corrosion: When two dissimilar metals come in electrical contact under water, one metal acts as the corroding anode while the other is the protected cathode. Such corrosion is noticed when a steel frame comes in contact with the rebar cage of reinforced concrete due to lack of separation (Lunniss and Baber, 2013).

Corrosion is noticed at the following locations in immersed tunnels:

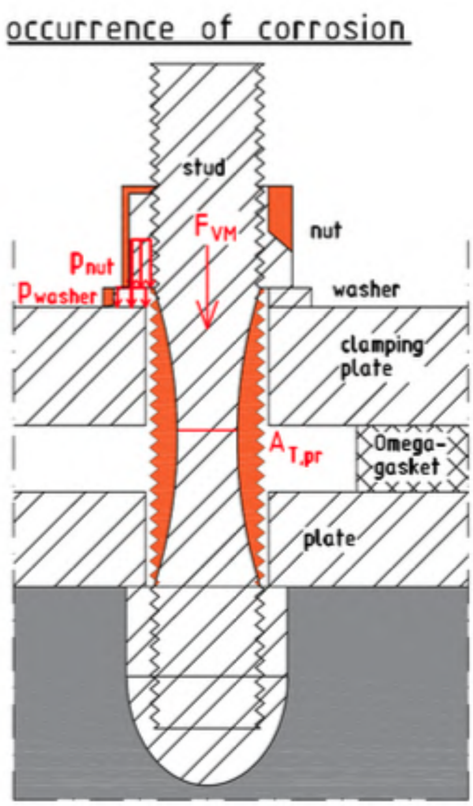


Figure 2-33: Cross section of the bolt, showing where the structure is deteriorated when corrosion occurs (Van Montfort, 2018)

- Gasket fixture components
  When corrosion occurs at the gasket fixture components, the compression value of gaskets reduces which can result in soil/water leaks. Van Montfort (2018) stated that corrosion at the stud, nut and washer can affect the functioning of the clamping structure, see Figure 2-33. When corrosion becomes large, according to Van Montfort (2018) the following can happen at these locations:

  1. Stud - The stud loses the capacity to transfer loads as cross-section reduces. This in turn results in cracks. Usually, the first signs of corrosion are observed at the studs. When one stud breaks, the pressure on the stud next to it increases. As a result, the chances of the next stud failing increase. This is referred to as the 'zipper effect',
  2. Nut - This can lead to two possible scenarios. In situation 1, the surface area of the nut reduces and this results in an increased pressure (p_{nut}) on the washer. Consequently, the washer becomes damaged. In situation 2, the force becomes so large that the nut breaks.
  3. Washer - Certain parts of the washer are lost due to corrosion. Subsequently, the pressure on the clamping plate (p_{washer}) becomes excessive and the plate is damaged.

- Steel end frame

  The steel end frame is exposed to the saline environment outside the immersion joint which causes corrosion. Even though the environment around the steel frame is usually inert since the immersion joint is buried within a backfill, corrosion can still take place. In buried environments such as polluted harbours corrosive bacteria such as sulfate-reducing bacteria (SRB) and sulfur-oxidizing bacteria (SOB) can give rise to pitting corrosion with a rate of upto 2 mm per year (Lunniss Baber, 2013). These microbes produce biogenic acids and other metabolites which corrode the concrete surface and increase the permeability of concrete. Furthermore, it reduces the pH value of concrete and forms a biofilm on the concrete surface (COB, 2021). This aggravates the corrosion even more. Inside the tunnel, the steel frame is considered to be in a damp environment.

  Steel end frames usually have protective coatings to mitigate this corrosion effect. This coating in the long term can deteriorate exposing the steel end frame. It is important to recognize the possible rate of corrosion to fully understand their effects on the structure and establish an appropriate protective system. The corrosion rate of exposed steel in saltwater can be quick, at roughly 0.33 mm per year, although it is predicted to gradually decrease over months to a stable rate of around 0.11 mm per year after around two years (Sinha, 2017). Lunniss and Baber (2013) stated that if corrosion of the steel frame supporting the Gina gasket progresses beneath the gasket, watertightness can be compromised. This is a very rare scenario but if the steel deteriorates the gasket's compression would cause it to always push into any void.

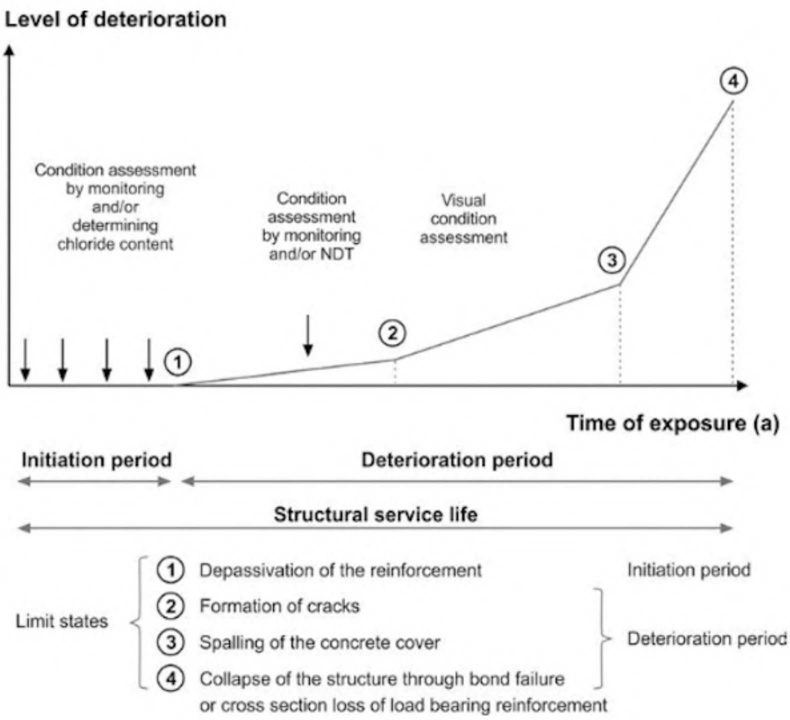


Figure 2-34: Deterioration of reinforcement over service life (Mayer et al, 2018)

- Reinforcement

  Steel present within reinforced concrete has a thin passive layer due to the high pH value of concrete pore solution to protect against corrosion. This passive layer often gets damaged due to carbonation, chloride ingress, large concrete cracks, insufficient concrete cover and poor concrete quality due to poor mix design or execution. It often takes a few years before the reinforcement becomes severely damaged and is noticed as brown spots at the concrete surface or concrete spalling (COB, 2021). There are two main phases of deterioration of reinforcement as shown in Figure 2-34.

- Initiation period: In this period, carbonation and chloride ingress can take place which destroys part of the passive layer. However, the functionality of the structure is not affected and no noticeable changes are present.
- Deterioration phase: In this phase, the functionality of the reinforcement is reduced and deterioration rate increases with time. If it is not stopped, it can lead to concrete cracking, leaks and loss of structural strength of the tunnel.

2-2-5 Leakage

The construction of immersion joints is an expensive process. Even though optimising design does help drive down costs, innovative methods in reducing and repairing leakages as well as dealing with construction difficulties are more effective. Based on Parwani (2014), leakages up to 4 m^3/hr could be pumped out without any issues. However, higher leakage rates would lead to problems. Leakages can occur due to gasket fixture bolt corrosion, concrete cracking, differential settlements and Gina gasket over compression and decompression. According to Gavin et al (2019), tunnel owners' biggest challenge is the leakage caused by differential settlements. Moreover, they have been found to occur mainly at the joints. According to Van Montfort (2018), leakage can occur through four possible routes in an immersed tunnel.

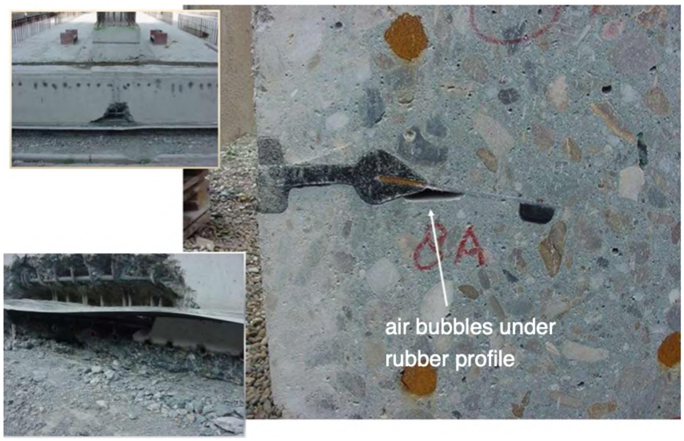


Figure 2-35: Lack of workmanship during concreting (Leeuw, 2013)

1. Concrete in the roof, wall or floor
   Watertightness in monolithic concrete tunnels is achieved through an external steel membrane while in segmental tunnels it is through the concrete. Leaks through the concrete occur only in small amounts. Since concrete is porous, small amounts of water

could leak through it. But it is not large enough to cause maintenance issues. Furthermore, the wall thickness of immersed tunnels is about 1 m thick and this makes penetration of water through concrete highly unlikely. Cracks which arise during construction are small and can be directly repaired (Van Montfort, 2018). Based on an inspection of several immersed tunnels, Leeuw (2013) claimed that leaks through walls are rare because there is no air entrapment around the vertical rubber joint profile during concreting. But lack of craftsmanship during concreting can result in air bubbles, see Figure 2-35.

2. Closure joints
   Leakages through closure joints have only been reported once so far for the Second Benelux tunnel.

3. Expansion joints
   When gaps are formed at the expansion joint, water leaks are first stopped by a waterstop. The mechanisms of the waterstop are explained in detail in section 3-7-2. As the joint gap increases the injectable waterstop gets stretched and loses adhesion to the concrete. This forms a pathway for water ingress as shown in Figure 2-36.

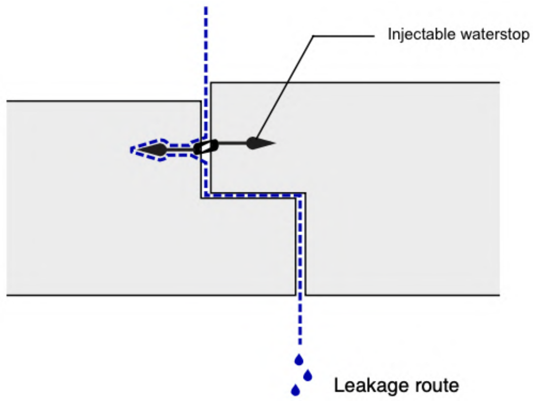


Figure 2-36: Leakage of water at expansion joint

4. Immersion joints
   Among the joints, leakages are often located at the expansion joints rather than at the immersion joints. This is because the deformation capacity of immersion joints is greater than expansion joints (Gavin et al., 2019). However, the First Coen tunnel did exhibit leakage through immersion joints. Van Montfort (2018) identified four possible leakage mechanisms at immersion joints, illustrated in Figures 2-37 and 2-38:

- *Crack in the concrete frame*: Large differences in vertical shear, eccentric forces or a combination of both can create concrete cracks. This would lead to leakage around the Gina gasket and Omega seal.
- *Piping*: Loosening of concrete behind the steel plate opposite of Gina gasket leads to creation of a leakage route around the Gina gasket and Omega seal. The leak would be small in this mechanism.

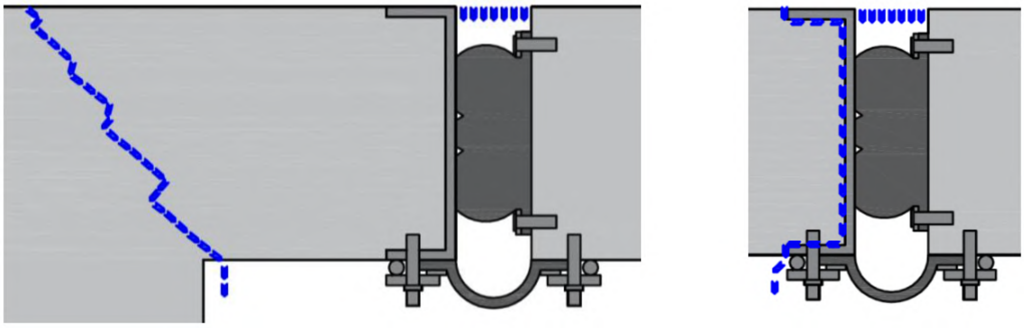


Figure 2-37: Leakage around the immersion joint: crack in the concrete frame(left) and piping (right) (Van Montfort, 2018)

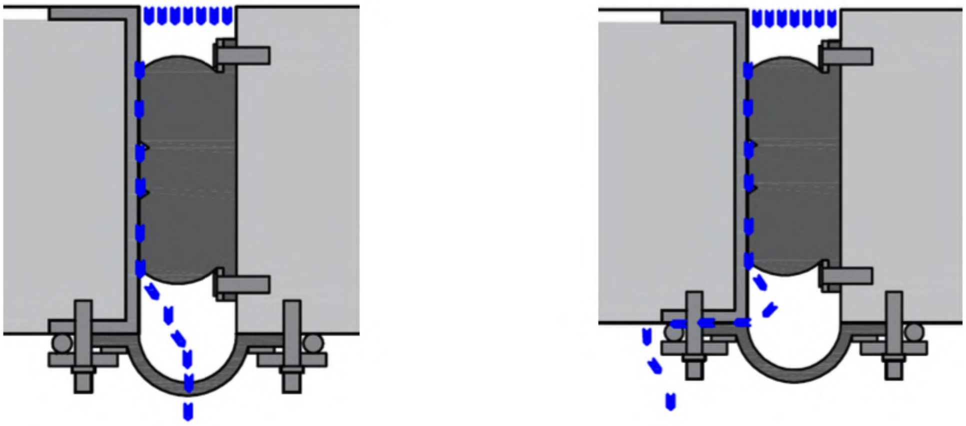


Figure 2-38: Leakage through the immersion joints: Failure of Gina gasket and leakage through Omega seal (left) and corrosion of the clamping structure of the Omega seal (right) (Van Montfort, 2018)

- *Failure of Gina gasket and leakage through the Omega seal*: Gina gasket is not pressed enough to the counter plate and the Omega seal is damaged possibly due to differential movements. Hence, a leakage route through the immersion joint is created.

- *Failure of the Gina gasket and corrosion of the clamping structure of the Omega seal*: Gina gasket is not pressed enough to the counter plate and the clamping component of the Omega seal such as bolts is corroded and have lost their functionality. A leakage route through the immersion joint is created.

Mechanisms 3 and 4 are leakages through the immersion joint. They can occur only if both the Gina and Omega gasket fail. Gina gasket can fail in four ways as depicted in Figure 2-39.

- *Water passes through the gasket due to lack of pressure*: When joint widths increase the pressure of the Gina gasket decreases. Similarly, the pressure reduces as relaxation of the Gina gasket takes place over time. This decrease is linear on a logarithmic scale. This means it is constant for periods ten times as long as the previous one. The Gina gasket supplier Trelleborg specifies relaxation of the rubber to take place at 6 % per decade. This means that after 100 years around 53.7 % of the initial compressive force is still present (COB, 2014).

- *Gina gasket is pushed inward due to increased soil pressure*: Soil can accumulate on the joint gap in the roof over time. As they expand during summer the soil becomes compressed. This leads to an additional load on the Gina gasket which ultimately ends up pushing the gasket inwards. Furthermore, it creates an additional load in the Gina gasket.

- *Lack of contact between Gina gasket and opposite tunnel alignment*: Due to displacements, tunnel elements can lose their alignment. This makes the Gina gasket lose contact with the opposite tunnel element.

- *Water passes through cracks in the gasket*: When the Gina gasket gets overcompressed for long periods, cracks emerge in the Gina gasket. This generates a leakage route through the Gina gasket.

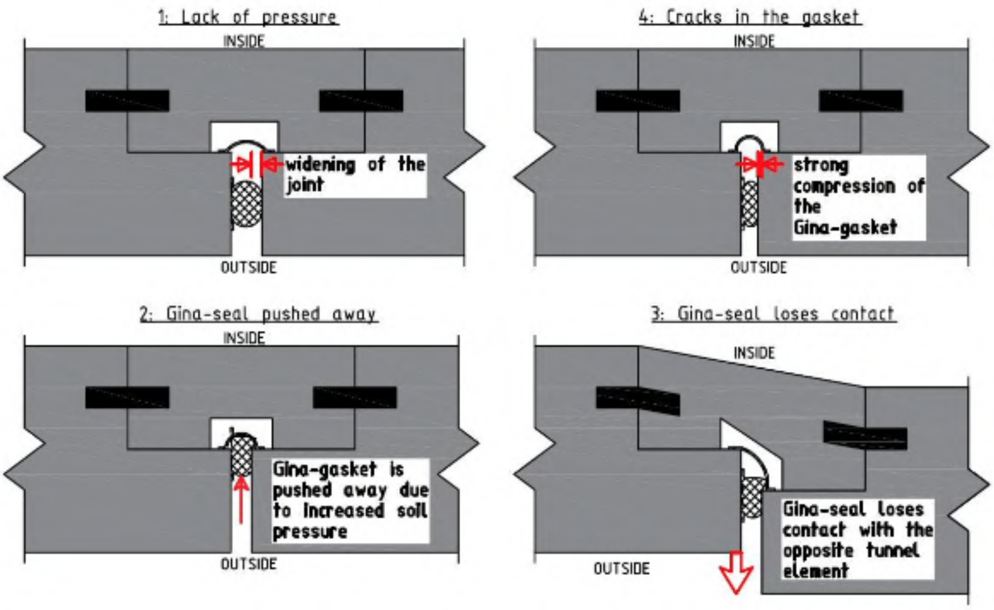


Figure 2-39: Failure mechanism of Gina gasket (Van Montfort, 2018)

Omega gasket has three possible modes of failure see Figure 2-40.

- *Water passes through the gasket due to insufficient clamping pressure*: The gasket can have insufficient clamping pressure due to corrosion of the clamping structure or relaxation of the gasket. This allows water to pass through the gasket.

- *The flange of Omega seal is pulled out*: Due to insufficient clamping pressure or movement of tunnel elements, the flange gets pulled out.

- *Water passes through cracks in the Omega seal*: When there are large joint movements, the gasket strains. This results in cracks in the seal.

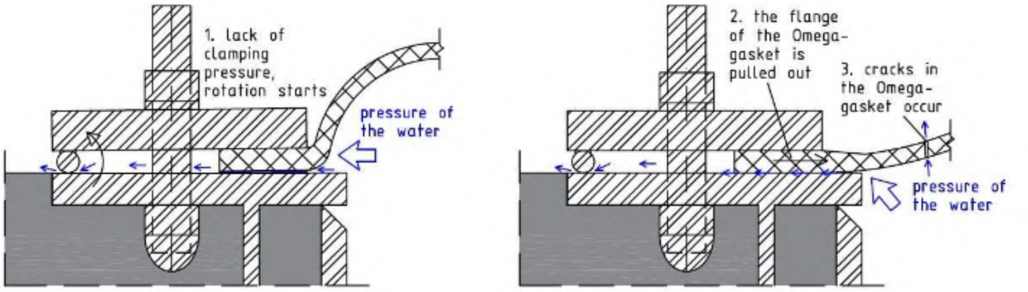


Figure 2-40: Failure mechanism of Omega seal (Van Montfort, 2018)

Leakage can lead to aesthetic problems, corrosion of reinforcement, ice on roadways, deterioration of tunnel finish, lifelong pumping costs and settlement. Seepage from hairline cracks can cause visual discomfort to the tunnel user. High leakage rates can gradually compromise the structural integrity of the tunnel if reinforcement undergoes significant corrosion. This will reduce the life span of immersed tunnels. During winter, leaks at the road surface can freeze and form ice which would endanger road safety. For example, the Drecht tunnel suffered ice formation due to leaking water. This resulted in the road closure and monetary losses. Moreover, tunnel finishings such as a metal wall, ceiling panels, mechanical piping et al can undergo long-term damage. Pumping to remove leaks will further increase costs (Grantz, 2001). Sand can be carried through leaks as well. Depending on the amount of sand involved, a sand leak might induce settlement. Therefore, sand leaks must be stopped as quickly as possible (Leeuw, 2008).

Appendix A gives an overview of potential reasons for these problems based on construction technique, geotechnical engineering and design flaw from a mechanical and structural perspective.

Chapter 3

Recent Changes In Immersed Tunnel Design

This chapter specifies new design changes in immersed tunnels based on a detailed review of tunnel projects that have been constructed or are under construction in the recent two decades. The goal of this chapter is to assess the potential effects of these new design philosophies on long-term behavior based on the structural problem study done in the previous chapter. An overview of how the development of recent design changes is linked to structural problems has been addressed in Appendix B by identifying the structural problem addressed, improvements and structural changes caused corresponding to each design change.

3-1 Catalogue of Recent Immersed Tunnel Projects

An inventory of completed and ongoing immersed tunnel projects over the recent two decades has been catalogued in Appendix C. Immersed tunnels gained significant popularity in the 1960s in Europe, after which the number of immersed tunnels increased worldwide. An important trend that has been noticed over recent decades is that longer and deeper immersed tunnels have been constructed. For instance, the 18 km long Fehmarn crossing currently being built connecting Germany and Denmark.

Until 2010, the majority of immersed tunnels which had been built were constructed in the Netherlands, the United States and Japan. However, there has been a geographic shift in the rate of immersed tunnels built. The People's Republic of China has experienced the fastest growth in the construction of immersed tunnels since the 2000s. Since the first immersed tunnel which was built in 1993, the country has experienced remarkable growth. This rapid growth can be attributed to high investment and the emergence of cities adjacent to estuaries and rivers. The number of immersed tunnels built in each country since 2000, as well as those under construction, is illustrated in Figure 3-1.

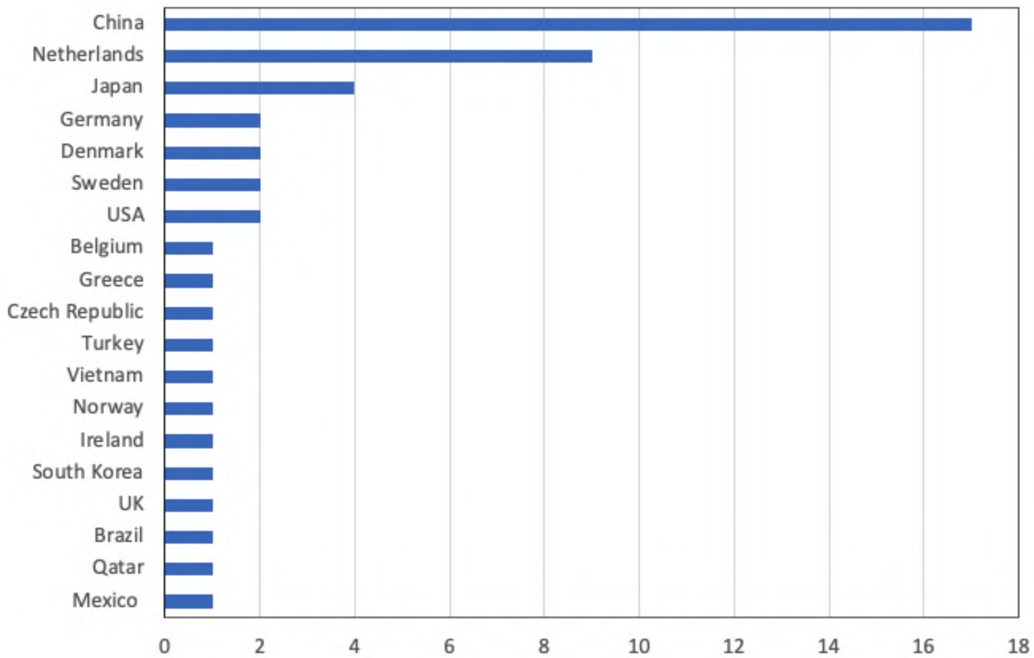


Figure 3-1: Number of tunnels constructed by country since year 2000

3-2 Element Material

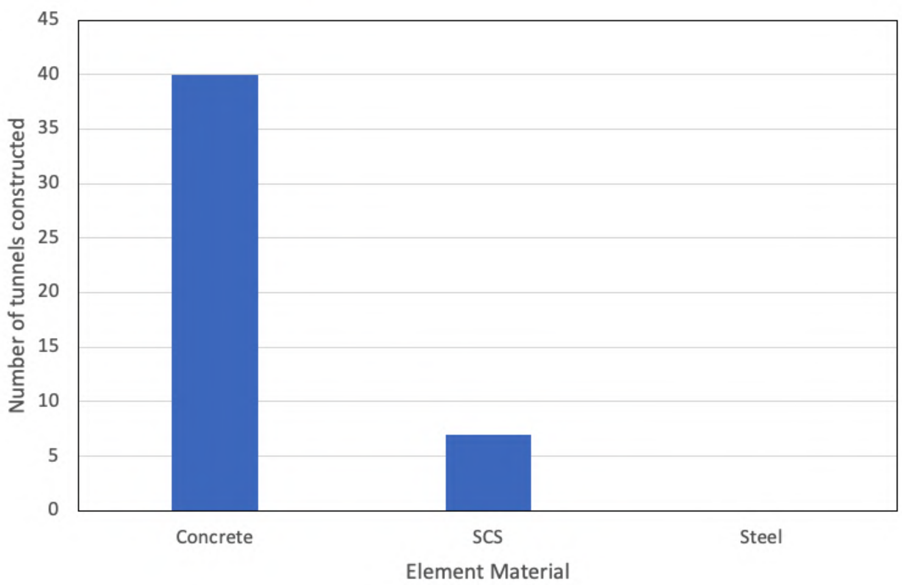


Figure 3-2: Number of tunnels constructed by element material in recent two decades

This section describes the developments in element materials being used in recent immersed tunnels. The conventional element material used for immersed tunnels are concrete, steel shell and steel-concrete-sandwich (SCS) tunnels. Figure 3-2 shows that in recent decades, there has been no reported use of steel shells in immersed tunnel design. Instead, steel concrete sandwich tunnels have gained popularity with over 7 such tunnels in use so far. The Söderstrom tunnel in Sweden was the first tunnel outside of Japan to use a steel concrete sandwich in its immersed tunnel design, see Figure 3-3. Finally, concrete tunnels have emerged as the most popular element material as 40 such tunnels have used it in their design worldwide. Within concrete tunnels, segmental tunnels are preferred by a slight margin over monolithic tunnels.

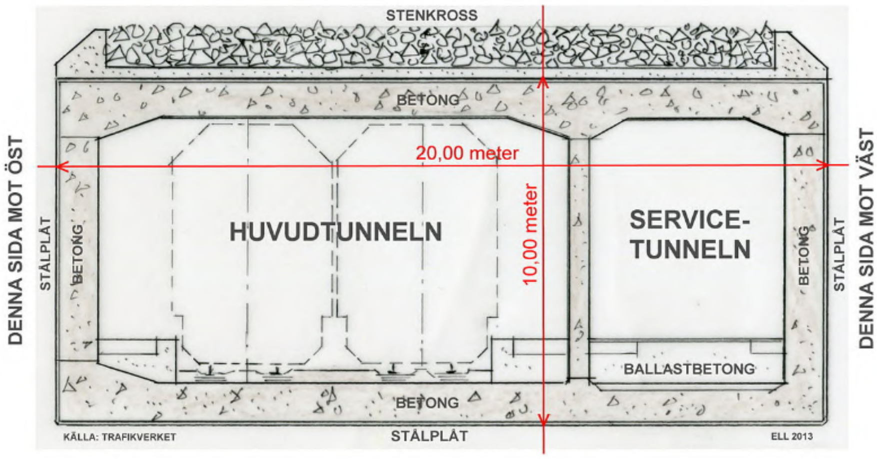


Figure 3-3: Steel concrete sandwich design used in Söderstrom tunnel (Vliet et al, 2022)

The decline in the use of steel shells can be related to the fact that most countries outside of the US may not have the required shipbuilding tradition, materials and fabrication skills and would have to import them to manufacture steel shells. Moreover, steel shells offer less flexibility in cross- section design as individual steel tubes have to be built for multiple lanes. The availability of raw materials and higher flexibility in cross-section design have led to the widespread popularity of concrete tunnels. Besides, the ductility and structural capacity of SCS tunnels to accommodate large differential settlements have led to rising popularity especially in seismic zones. Since it is a relatively recent development, improvements in the ease of manufacturing these tunnels would lead to wider use in the coming years.

Conventionally, segments were prestressed temporarily during transportation and immersion, after which the prestressing cables were cut off. This enabled the segments to retain flexibility upon immersion. However, they were not usually used in seismic regions and deep waters due to their reduced deformation capacity. To overcome this, a new form of elements was developed called semi-rigid elements. They are elements in which the prestressed cables are not cut once the segments have been immersed, thereby increasing the overall stiffness of the tunnel. The element is partly rigid due to permanent prestressing and partially flexible because it is made up of segments. The prestressed cables add to the longitudinal compressive stress and increased bending rigidity of the expansion joints. Furthermore, the increased stress raises friction of expansion joints which improves their anti-shear bearing ability. As a result, semi-rigid elements can adjust to large differential settlements. The use of semi-rigid elements not only improves the structural performance of expansion joints but also lowers construction costs and time by reusing the prestressed cables.

Hu et al.(2018) stated that these elements were first used to speed up the construction of the Piet Hein tunnel in Amsterdam and not for structural reasons. Given the specific geological conditions and load-bearing capacity of expansion joints, Hong Kong-Zhuhai-Macao Bridge (HZMB) was the first immersed tunnel to use such elements for structural purposes. These elements were significantly efficient in limiting differential settlements. The expansion joint details of the HZMB tunnel are shown in Figure 3-4.

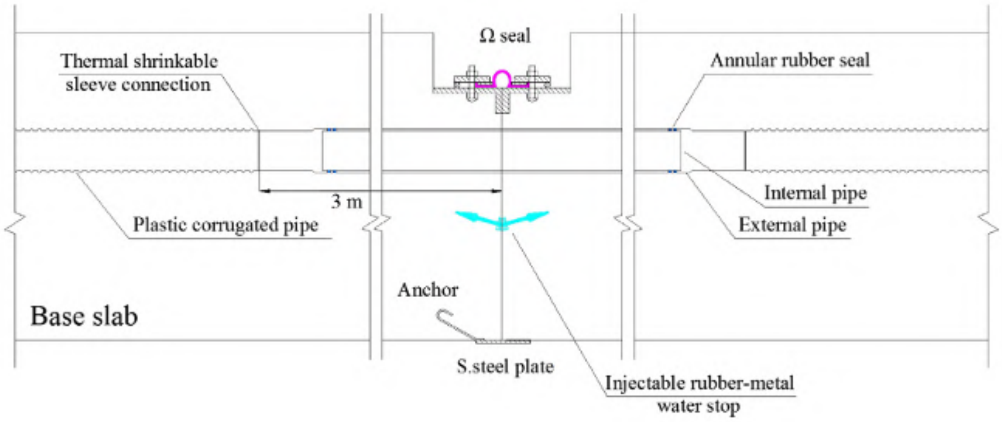


Figure 3-4: Expansion joint details of semi rigid elements used in HZMB (Hu et al.,2018)

Although semi-rigid elements have several advantages, Hu et al.(2018) claimed that their use can lead to long-term structural problems. The location and number of prestressed cables should be carefully evaluated because the structural functioning shifts from temporary during transportation to permanent after immersion. The structural qualities and watertightness of expansion joints may be affected as a result of this. The prestress level of remaining cables should also be carefully determined, as it impacts the structural rigidity and deformation ability of the elements. The opening of expansion joints cannot be controlled properly if the prestress level is too low. On the other hand, differential settlements will primarily focus on immersion joints if the prestress level is too high as the rigidity of expansion joints would be similar to that of the elemental body. Furthermore, in a marine environment, high-strength wires may experience stress corrosion induced by chloride ingress if the prestress level is too high. This could result in chain and sudden fracture of prestressed cables, jeopardizing tunnel safety.

Finally, the prestress loss of cables can affect the durability of semi-rigid elements. Uniform prestress losses do not affect structures but asymmetrical prestress losses produced by material faults in wires, tensioning errors in prestressed cables, grouting errors in ducts and other factors can cause joints to open. As a result, ensuring the construction quality of prestressed cables, duct grouting and the joint connection is crucial. Furthermore, if considerable openings of expansion joints develop, an effective and unique control approach is to cut off a few prestressed cables based on calculations. However, this can result in element vibration and shock, stress redistribution in remaining cables, and diminished anti-shearing performance of expansion joints. Despite these disadvantages, with the development of the economy in coastal regions, semi-rigid elements can prove beneficial for immersed tunnels.

3-3 Cross Section Design

This section presents the trends in cross-section design employed over the past two decades. According to Zhigang et al (2016), cross-section design is the most difficult part in immersed tunnel design as it affects tunnel scale, technical difficulties, service level and building cost. Due to limited space in immersed tunnels, cross section design must be optimized to satisfy all of these needs. As immersed tunnels become longer and larger such as the Fehmarn crossing and Busan–Geoje Tunnel the cross-section design becomes more challenging. The cross-sectional layout of immersed tunnels can be divided into rectangular and circular shapes having one or multiple traffic bores along with or without utility tubes. Utility tube contains facilities such as ventilation, lighting, surveillance, drainage, power supply, escape and rescue. Different possible layouts are shown in Figure 3-5. The span and thickness of the slab are determined based on the layout of the traffic bore and utility tube. Placing more partition walls to increase the number of traffic bores helps to increase the longitudinal rigidity of the tunnel by providing higher bending and shear capacity of immersion joints. Moreover, it helps decrease the span of a slab, reduce reinforcement quantity and structural design difficulties and risks such as crack formation. However, it reduces traffic capacity and increases the weight and width of the element. Therefore, several structural analysis for several layouts is needed to select the appropriate layout for the cross-section.

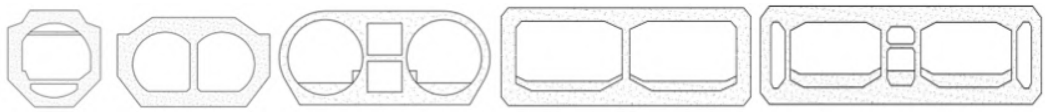


Figure 3-5: Possible cross sectional layouts of immersed tunnel (Zhigang et al., 2016)

Steel shells often have a circular section whereas concrete tunnels typically have a rectangular shape. Circular shapes are no longer employed because there hasn't been any reported use of steel tunnels in recent years anywhere in the world. The popularity among designers to select this shape can be attributed to the flexibility offered for traffic bores to satisfy traffic demand and suitability for traffic or rail envelopes. Among the recent tunnels for which cross-section details were gathered, 75% of the tunnels had a utility tube. It was noticed that the immersed tunnels built solely for rail transport did not adopt an utility tube. Furthermore, it was noticed that only 7 out of 34 tunnels used more than one utility tube. Finally, 76% of the utility tubes were placed centrally between the road tubes and 24% on the sides. A utility tube if placed centrally helps to reduce the overall cross-section width and costs. However, adding utility tubes to the sides creates an easier escape route. If one of the tubes is on fire the other is not affected.

3-4 Transverse Prestressing

The demand for wider spans in immersed concrete tunnels has become essential due to rising traffic demands. Furthermore, added costs associated with dredging and lengthening approach structures to increase tunnel height make it more economical to expand tunnel width than tunnel depth. However, as tunnels become wider bending moments increases at a rate of square of span length and so deflection of the span becomes a design issue. Besides, when reinforcement is higher than the reinforcement ratio limit of 0.02 concrete compressive strength is controlling the design which could result in brittle failure before the full strength of steel is used. Thus, reinforced concrete tunnels have a limited span length and low slenderness for the slab. Transversely prestressing the element is an innovative solution to make wider concrete tunnels by increasing the capacity and slenderness of the slabs. Furthermore, it helps reduce costs as less overall concrete is utilised (Hanusaik, 2021). Figure 3-6 shows the reinforcement detailing for transverse prestressing.

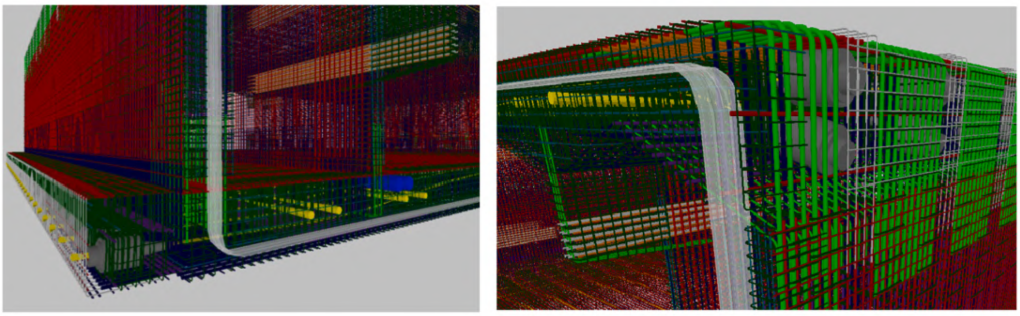


Figure 3-6: Transverse reinforcement detailing for floor and base slab (Putten and Os, 2022)

Hanusaik (2021) pointed out that this technique is only occasionally used in the industry owing to the difficulty of using it in underwater environments with varied loading conditions acting on the immersed tunnel. When post-tensioned tendons are used the curvature generates extra distributed loads that help resist high hydrostatic pressures and backfilling weight which acts on the tunnel when fully immersed. However, once prestressed until the elements are immersed the increased prestressing loads are primarily offset by the structure's weight. For immersed tunnels, the tunnel's self-weight only makes up 10-40 % of the final loading compared to the dead load of prestressed bridges which constitutes 80–90%. As a result, pre-stressing in the initial stages before transportation generates significant tensile stresses and might cause severe cracking. Additionally, the constructability of the element becomes more complex when transverse prestressing reinforcement is incorporated. In the long term, post-tensioning and time dependent losses are to be considered. Friction effects between the tendon and duct in which it is placed reduce the prestress level of the tendon. Asymmetrical prestress loss can further result in cracks. Finally, concrete creep, shrinkage and relaxation can further widen small cracks in the long term (Hanusaik, 2021).

Hanusaik (2021) proposed two possible solutions to mitigate cracks. Firstly, additional reinforcement is provided at the top slab, a critical area in tension, to improve the structural capacity of the slab as shown in Figure 3-7. An increase in steel quantity increases the resistance of the slab to cracking as the compressive strength limit is reached only at a higher load. Care should be given to ensure that the maximum reinforcement ratio limit is not exceeded.
Secondly, temporary tendons connecting the top and bottom slabs can be inserted to simulate the final loading conditions as illustrated in Figure 3-8. They are referred to as temporary because the tendons could be loosened to accommodate developing external forces and prevent overloading the element. These tendons are taken out once the tunnel is fully submerged, and the final stage is built with no extra tendons. Compared to the previous option, temporary

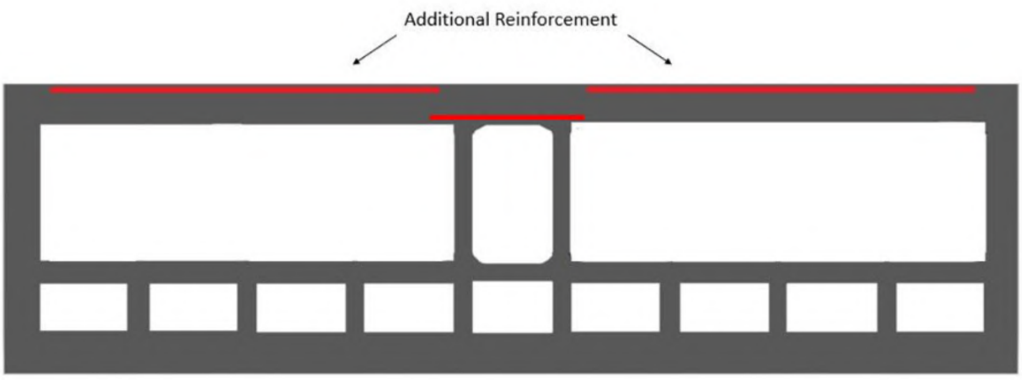


Figure 3-7: Additional reinforcement layout (Hanusaik, 2021)

tendons give a slight economic advantage as less amount of reinforcement is used. Nonetheless, this method requires the separation of ballast tanks into two separate ones to enable a connection for the tendon in the middle.

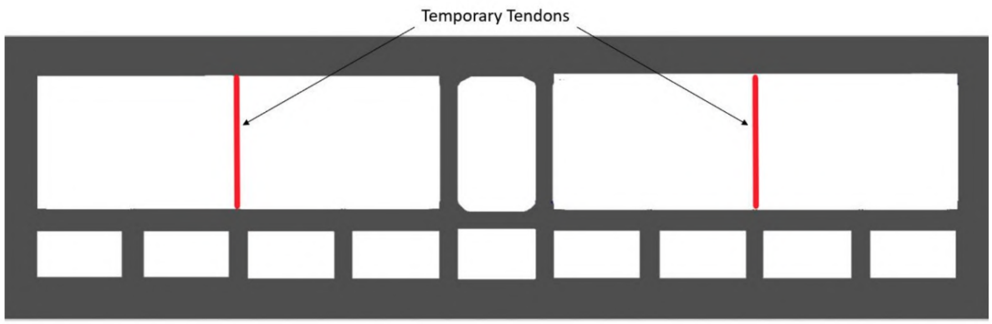


Figure 3-8: Temporary tendon placement layout (Hanusaik, 2021)

Based on recent projects, prestressing has been done to reduce the overall height or increasing the overall capacity of particular sections to withstand large external loads such as the Louis-Hippolyte tunnel and Bjorvika tunnel. With the increasing demand for wider spans for immersed tunnels such as the proposed Shenzhong tunnel with a span of approximately 18.5 m, transverse prestressing can serve as an efficient solution for immersed tunnels in the future.

3-5 Foundation Treatment Methods

Immersed tunnels are usually located in poor geological conditions with weak soil. To maintain alignment and avoid unwanted loading condition effects, foundation treatment is performed. The conventional foundation treatment methods used are sand, gravel, grouted and piled foundations. The method is chosen based on factors such as water depth, material availability, flow characteristics and seismic conditions. This section provides a comparison of the various foundation treatment methods in use (Lunniss and Baber, 2013).

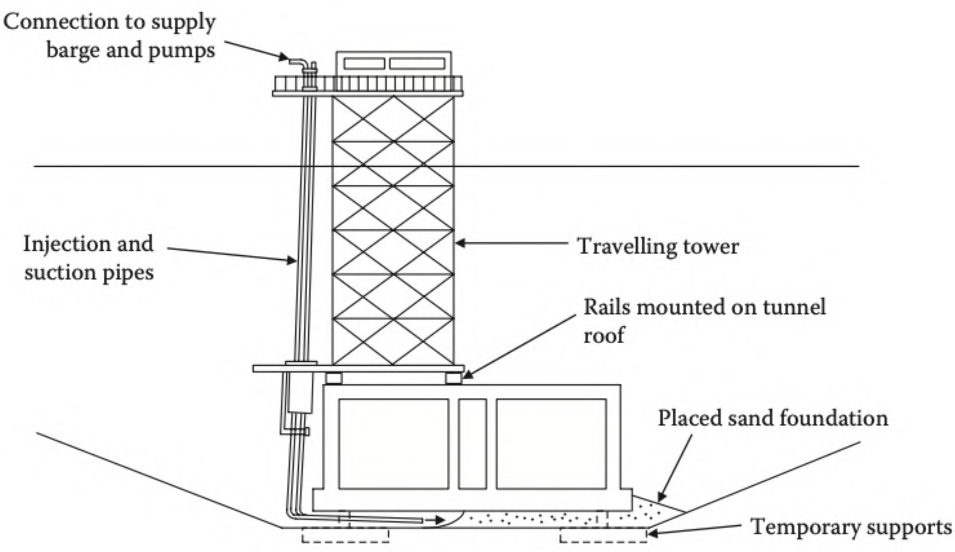


Figure 3-9: Sand jetting equipment mounted on roof of tunnel (Lunniss and Baber, 2013)

Sand foundations can be classified as sand jetting and sand flow. The two methods differ only in the way the sand injection is carried out. In the case of sand jetting, the equipment is mounted on the tunnel roof and can move longitudinally along the length of the tunnel as shown in Figure 3-9. For injecting the sand, elements are supported on hydraulic rams and the primary end of the adjacent element which creates a space of 600 -1000 mm above the bed. The tunnel is then fitted with a delivery pipe that runs alongside the wall and injects sand into this space (Lunniss and Baber, 2013).

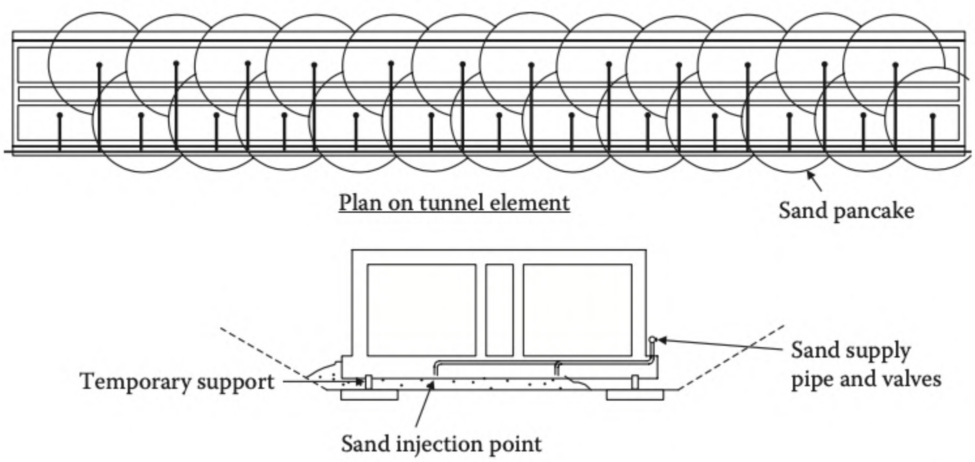


Figure 3-10: Sand flow method (Lunniss and Baber, 2013)

To avoid the use of equipment on the tunnel roof delivery pipes are cast into the tunnel floor for the sand flow method. These pipes are further connected to exterior pipes that are attached to a pumping plant near the shore to avoid using a floating plant. A sand pancake of around 12 m in diameter is formed under the tunnel element through each injection point as depicted in Figure 3-10. Sand foundations are usually applied in regions with low currents and normal seismic activity. However, these methods need diver activity which becomes increasingly undesirable at greater depths. In highly seismic zones, gravel or grouted solutions are more appropriate. Gravel foundations are conventionally placed by using a fall pipe, grab or dumping from barges followed by utilising a screeding frame or shoe to produce a closer tolerance finish, see Figure 3-11. The gravel bed is formed before the elements are immersed. In deep water or strong current conditions, it becomes challenging to place gravel foundations with high accuracy. For such circumstances, a grout layer of approximately 100-150 mm thickness is placed above the gravel bed. Hence, it becomes possible to screed the gravel bed to a rough tolerance, place the tunnel components on temporary supports and fill the space between the gravel and tunnel with a thin coating of grout (Lunniss and Baber, 2013).

Figure 3-11: Construction of gravel bed (Wang et al, 2020)

Finally, pile foundations are applied in regions of very soft subsoils where the degree of settlement has to be controlled. Since it is impossible to keep all of the pile heads at the same level while laying a pile foundation, Wang et al (2020) stated that the most crucial task is to fill the space between the pile top and tunnel bottom. The various conventional methods developed to eliminate this gap can be divided into the adjustable pile head method and pile capping beam method. The adjustable pile head method is often applied using an adjustable head made of a separate concrete component attached to the remainder of the pile through a nylon sleeve. Upon immersion, the adjustable pile head will be raised to the bottom of the tubes by grouting into the nylon sleeve as shown in Figure 3-12. The second technique is typically used with steel or concrete pile capping beams built underwater to hold a pile group together. Typically, cushion material made of cement will be used to fill the space between the capping beams and the element, see Figure 3-13. Compared to other foundation methods, great accuracy has to be maintained during pile driving and immersion.

It has been observed that no novel foundation solution has been introduced since 2000. Figure 3-14 depicts the use of foundation treatment methods used worldwide since 2000. Based on the catalogue of recent immersed tunnels, it can be seen that gravel foundations have been

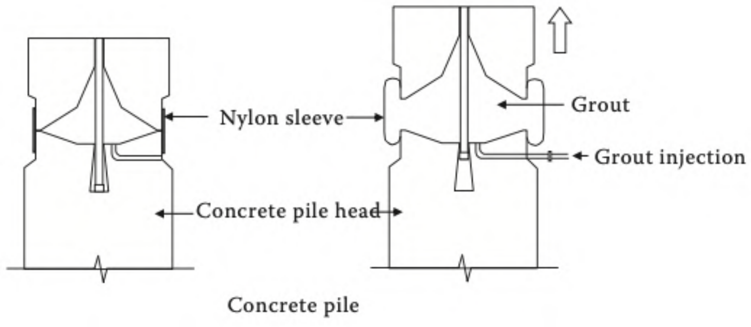


Figure 3-12: Adjustable pile head method (Lunniss and Baber, 2013)

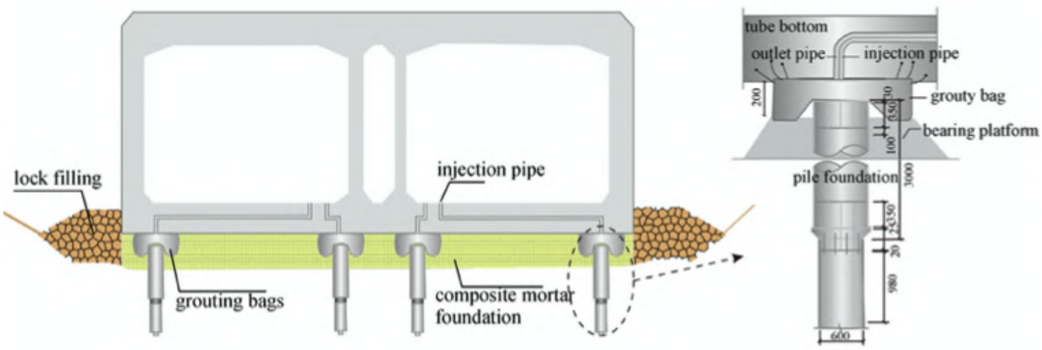


Figure 3-13: Grouting bag used for Changhong Tunnel (Wang et al, 2020)

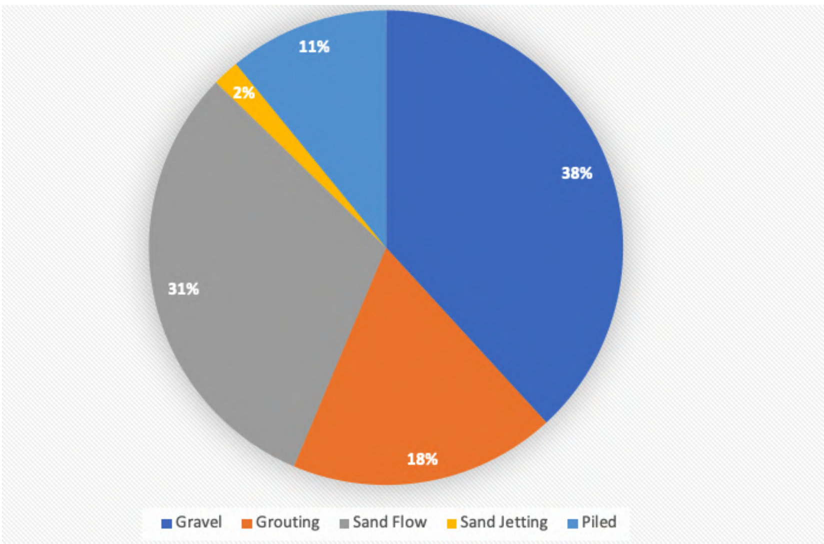


Figure 3-14: Foundation treatment method used in past twenty years

used the most. This can be attributed to the rising demand for immersed tunnels in areas with deeper waters, strong currents and high seismicity where gravel foundations are suitable. It was also noticed that sand foundations closely followed gravel foundations in terms of usage. Due to the elimination of sand jetting equipment, flexibility in choosing raw material and faster injection rates, the sand flow method has been universally chosen over sand jetting whenever sand foundation was used. The only immersed tunnel which used sand jetting in the past two decades is the Thu Thiem tunnel in Vietnam. Compared to other solutions, the piled foundation offers better settlement control in similar geological conditions. However, pile foundations are less chosen due to higher initial installation costs and complexity in construction (Lunniss and Baber, 2013).

3-6 Waterproofing Design

As there has been no reported use of steel shells in recent decades, this section describes the developments for waterproofing measures of concrete and SCS tunnels. Since concrete is a porous material, water can leak through the walls and slabs. The likelihood of leakage through a concrete surface is relatively minimal as long as cracks do not form. Cracks can occur in reinforced concrete in the form of either flexural or through-section cracks. When reinforced concrete bends the opposite ends of an element are put in tension and compression resulting in flexural cracks. These cracks do not threaten the watertightness of the structure as they extend to only a limited depth from the surface. On the other hand, through-section cracking can cause leakages. They are formed due to restricted structural movement and thermal gradient effects. As concreting of the tunnel element is done in several stages, joints between walls and slabs become potential locations of through-section cracking. Additionally, differences in temperature across the cross-section of walls and slabs can generate through-section cracks. Furthermore, concrete is exposed to a corrosive environment inside and outside the tunnel which can also facilitate cracking. The primary waterproofing measures used to tackle these issues are improvements in concrete mix design, concreting technique and use of external membranes (Lunniss and Baber, 2013).

Innovations within the concrete mix design involved the use of cement replacement materials such as pulverized fly ash (PFA), ground granulated blast furnace slag (GGBFS) and micro silica to reduce permeability and heat of hydration. By reducing concrete permeability, the leakage path for water is blocked whereas reducing the heat of hydration helps to limit the formation of early age shrinkage cracks. Lunniss and Baber (2013) stated that PFA and GGBFS were used to replace around 50% and 70% of the cement content.

To limit through section cracking, concrete cooling and full section casting were the two main developments in concreting technique. A system of cooling pipes is cast into the reinforced concrete close to construction joints as shown in Figure 3-15. The cooled water is then passed through these pipes to absorb the heat generated during curing. By limiting the thermal differential the expansion of concrete is limited. The second technique was first applied to the Øresund Tunnel. It involves concreting the walls and slabs of a tunnel segment in one continuous sequence. This way restraint to concrete movement is avoided. Moreover, it eliminates the need for cast-in cooling pipes (Lunniss and Baber, 2013).

Finally, for monolithic concrete tunnels external membranes are applied to prevent the development of through-section cracks. Usually, a tough membrane of steel or hard plastic is applied underneath the tunnel element. Steel membranes are usually around 6-10 mm thick

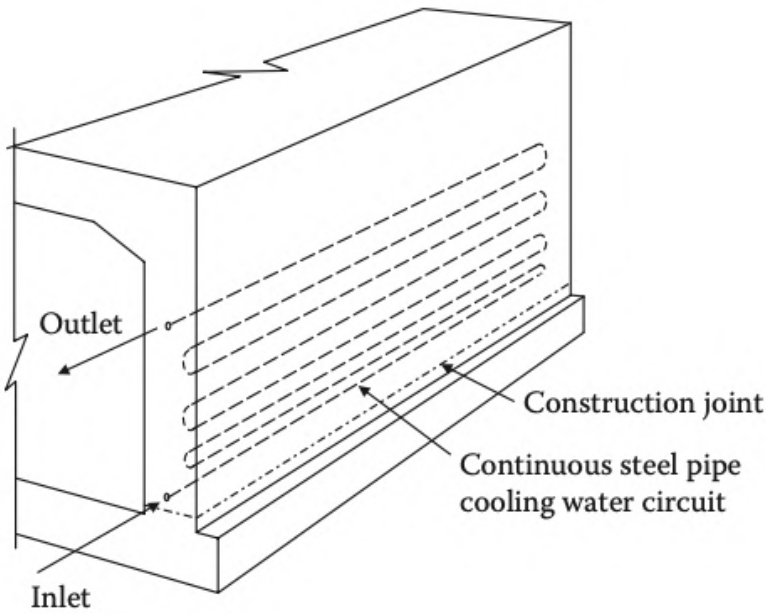


Figure 3-15: Arrangement of cooling pipes (Lunniss and Baber, 2013)

with only enough shear connectors to secure them to the concrete without producing a composite action. Tough plastic membranes such as high-density polyethylene (HDPE) sheets provide a cheaper alternative to steel membranes. However, they are more prone to puncture due to improper handling. For the roof and walls, flexible sheets or spray-applied membranes are used. Flexible sheet membranes such as bituminous sheets require high workmanship to ensure equal bonding. On the other hand, spray-applied elastomer membrane is cheaper and provides better adhesion to the concrete. The high demand for workmanship to ensure proper adhesion of the membrane and difficulty in identifying the source of leakage in case of lack of adhesion makes external membranes, not an ideal waterproofing measure (Lunniss and Baber, 2013).

Cathodic protection and corrosion resitant steel are two recent design changes used to limit corrosion induced cracking. Cathodic protection (CP) is another strategy used to mitigate reinforcement corrosion. The basic principle of the cathodic protection system is that an anode is connected to the steel reinforcement and a corrosion cell is created based on the relative potential between the two components. Hence, the corrosion of the anode takes place and the reinforcement remains undamaged. This system can be further applied to steel components of immersion joints and steel exteriors of SCS tunnels. Corrosion detection monitors could be installed at the relevant parts to activate the cathodic protection before the onset of corrosion. The cathodic protection was used for the Bjørvika Tunnel with the anodes placed on the exterior of the tunnel as shown in Figure 3-16. The main disadvantages of CP systems are that constant monitoring is needed and the electrical components needed to be replaced every 15-20 years (SEG, 2019).

Another defense mechanism adopted against reinforcement corrosion is a coating of the rein-

Figure 3-16: Arrangement of anodes on the exterior of Bjørvika Tunnel (Lunniss and Baber, 2013)

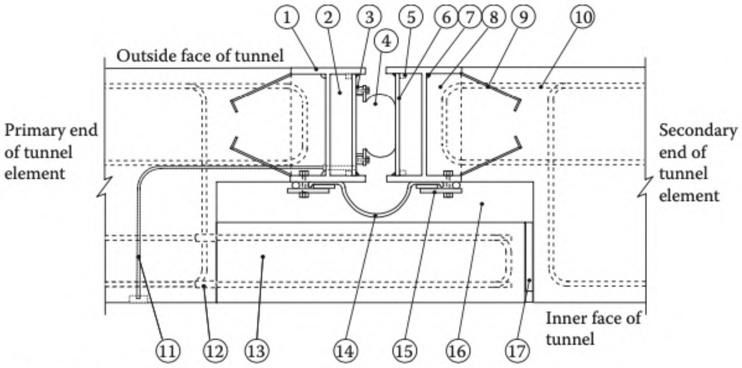
forcement in the most exposed zones (Braestrup, 2016). This has become a popular corrosion mitigation measure in Scandinavian countries. A coating with epoxy paint 500 microns thick is applied to the reinforcement. This coating ensures that corrosion does not take place during construction, transportation and immersion. Furthermore, in polluted environments which may contain sulfate-reducing bacteria (SRB) coatings provide better protection than CP systems. These coatings have a short lifetime of around 40 years in marine environments and are only efficient if they are re-applied. Since they have to be re-applied it is an expensive durability measure. If the coatings contain defects they cause more harm as accelerated corrosion takes place.

3-7 Joint Formation

This section indicates the various developments in joint formation. Aside from construction joints, the main purpose of joints in immersed tunnels is to allow a connection between elements and segments as well as ensure the watertightness of the structure. Additionally, shear keys aim to reduce differential settlements and allow for the transfer of loads. Immersion, expansion and closure joints have undergone several developments over the past decades to overcome common structural problems and provide greater ease in construction.

3-7-1 Immersion Joints

Immersion joints are the joints formed between individual tunnel elements. Before immersion, a continuous rubber gasket called Gina gasket is installed outside the tunnel at the primary end of the element. This gasket also called primary seal is designed to provide watertightness and function as a permanent seal. It has a nose and body with low and high stiffness



Item no	Component
1	Immersion joint end frame
2	Grout infill to void
3	Gina gasket clamping system—spacer, clamping bar, and securing bolt
4	Gina gasket
5	Welded spacer block to position the counterplate
6	Welded counterplate
7	Grout injection tube (passes through stiffener plate)
8	Stiffener plate
9	Anchor bar to secure the frame to concrete (can use shear studs)
10	Structural reinforcement within concrete slab
11	Pressure test pipework
12	Couplers on reinforcement for connection infill section bars
13	Infill concrete, cast after Omega seal is fixed
14	Omega seal
15	Omega seal clamping system—clamping bar, spacer, and torqued bolt
16	Void
17	Compressible joint filler and surface seal

Figure 3-17: Components of immersion joints in concrete tunnel :section through roof slab (Lunniss and Baber, 2013)

respectively. They are fastened to a continuous steel frame that is cast onto the end of the tunnel element via a simple clamping bar arrangement and goes all the way around it. The clamping arrangement is illustrated in Figure 3-17. Once the elements are placed on the foundation layer they are pulled together and the softer nose makes contact with a counter plate of the adjacent tunnel element. Subsequently, the joint space is dewatered to provide an initial watertight seal. As a result of this dewatering, an out-of-balance hydrostatic pressure is created which further compresses the Gina gasket.

Bulkheads are then opened and a secondary Omega seal is installed within this bulkhead space. The Omega seal forms a secondary barrier to water ingress. The Omega seal is fastened onto the steel end frame by a bar and bolt arrangement as shown in Figure 3-17. Van Montfort (2018) stated that the Omega seal was installed either before or after the consolidation period which is generally 4 weeks after the elements are laid on the foundation layer. Due to varying soil stiffness, consolidation can lead to differential settlements. Hence, it is more convenient to install the Omega seal after the consolidation is complete so that shape of the gasket does not change. Afterward, infill concrete is placed in the interior of the immersion joint. Typically, this infill is created continuously with the tunnel element's concrete on one side while allowing space for expansion and contraction on the other side. It also serves as a rocking slab to eliminate differential settlements.

The Omega gasket must be able to elongate to allow a certain range of movement during

Table 3-1: Maximum elongation for different standards types of Omega seals (Sinha, 2017)

Omega seal type	Maximum allowable elongation in ULS (mm)	Elongation at Break (mm)
OS 240-40	60	85
OS 300-70	65	80
OS 360-100 & OS 400-100	90	110

the operational period of the immersed tunnel. The maximum elongation for different types of Omega seals specified by Sinha (2017) for ultimate limit state (ULS) conditions is given in Table 3-1.

As more immersed tunnels are being constructed in larger water depths and highly seismic zones, there is a need for new joint design to ensure watertightness. Even though larger rubber gaskets can sustain enough compressive force at deeper water depths, due to transverse displacements and other issues, they are unable to avoid water leakage. Seismic, bellow and crown seal joints are new types of joints developed to absorb the large displacements that cannot be absorbed by traditional immersion joints.

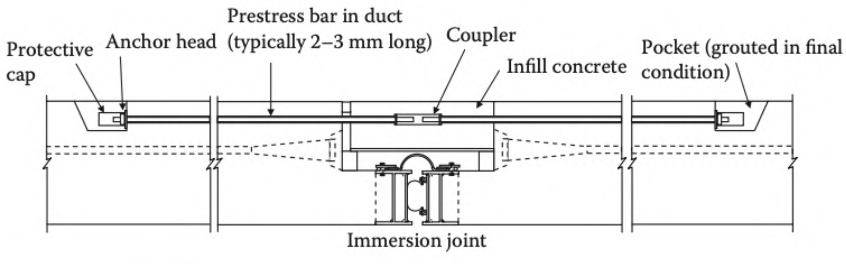


Figure 3-18: Seismic immersion joint (Lunniss and Baber, 2013)

Conventional immersion joints can accommodate all potential movements in moderate seismic zones. However, the joints could lose their watertightness under intense seismic loading. Seismic joints were developed to withstand this loading. The seismic joint can be classified as seismic immersion joint or seismic terminal joint based on the location of installation. In contrast to terminal seismic joints, where just one seismic joint is used to connect the approach structures to the immersed tunnel structure, seismic immersion joints are positioned at each immersion joint. The seismic immersion joint consists of high tensile bars or cables strapped across the joints as depicted in Figure 3-18. These cables are made long enough to allow some movement by straining but their high stiffness ensures that the amount of joint opening is controlled.

Lunniss and Baber (2013) pointed out that during a seismic event there is a chance that the seismic joint will rapidly open and close, damaging the bar or coupler-bearing plate. Bellow joints developed in Japan provide an alternate solution to this design issue. Bellow joints consist of a wave-shaped steel plate as shown in Figure 3-19 located inside the Gina gasket. The steel plate will stretch and compress as the joint opens and closes during seismic activity to ensure that water does not leak inside. Furthermore, as the steel plates are manufactured

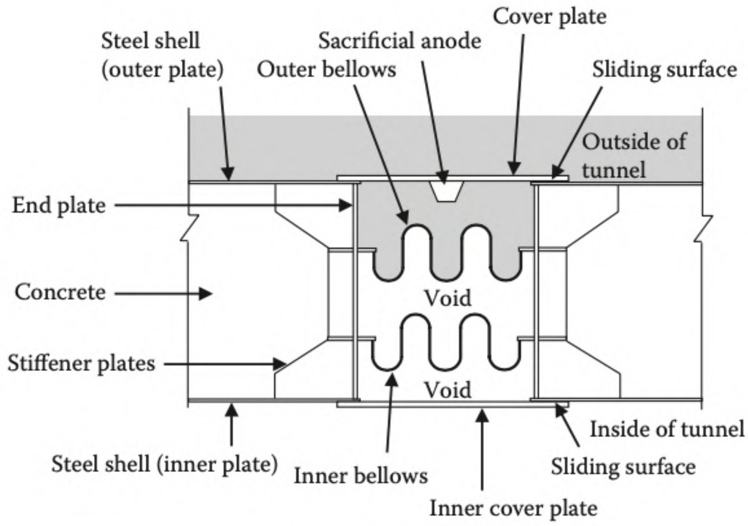


Figure 3-19: Bellow joint developed in Japan (Lunniss and Baber, 2013)

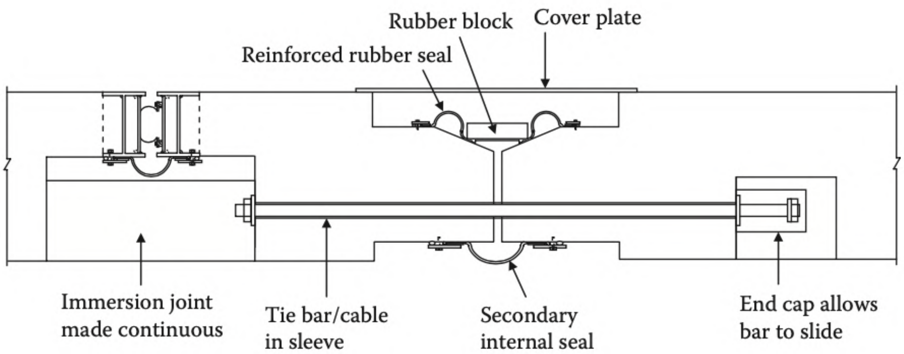


Figure 3-20: Crown seal joint in Yumeshima tunnel (Lunniss and Baber, 2013)

in factories, a reliable connection can be achieved between elements.

Similarly, another immersion joint developed in Japan for the Yumeshima tunnel to withstand large differential settlements is the crown seal joint as illustrated in 3-20. It is also quite effective in reducing lateral deformation according to Xiao et al (2017). This joint is constructed next to an immersion joint which is made continuous. Besides, it has a prestressing member across the joint to avoid excessive joint opening and a double seal that allows for greater watertightness.

3-7-2 Expansion Joints

Expansion joints are the match-cast joints located between consecutive segments of the segmental concrete tunnel. These joints have no reinforcement passing through them and are debonded to allow for joint rotation, opening and closure of joints (Lunniss and Baber, 2013). These joints also referred to as segment or dilation joints are illustrated in Figure 3-21.

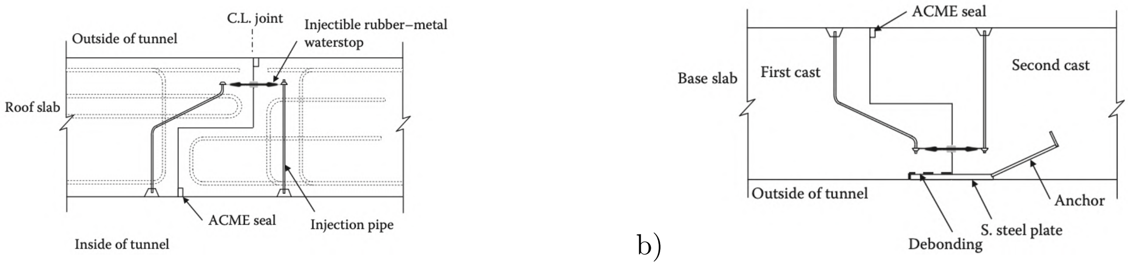


Figure 3-21: Typical expansion joint arrangement through a) tunnel roof and b) base slab (Lunniss and Baber, 2013)

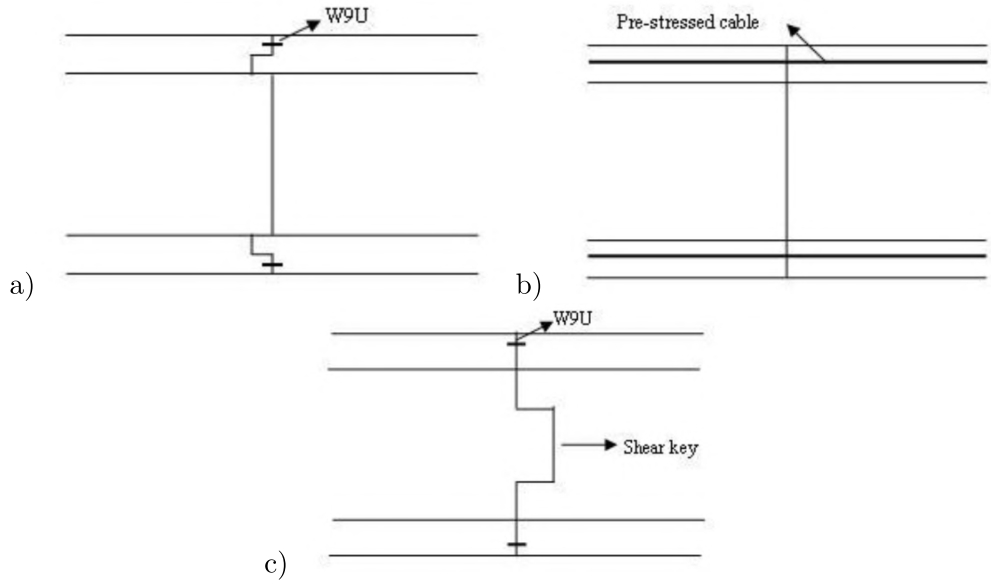


Figure 3-22: Side view of a) Dutch 'flexible' joint b) Japanese 'stiff' joint c) Intermediate joint (Van Oorsouw, 2010)

Van Oorsouw (2010) further classified expansion joints based on their flexibility. Dutch expansion joints are joints where the prestressing cables used for the transportation of tunnel elements are cut once immersed. This enables the tunnel to follow small differential settlements of the foundation. In seismic zones, where the opening of expansion joints can become significantly large, rubber sealing profiles fail to maintain watertightness. To prevent this, the stiff Japanese expansion joint was introduced where the prestressed cables are not cut after immersion. No shear keys are present in these joints and shear transfer occurs through friction between the segments. The prestressing cables push these segments closer to transfer shear forces. Finally, an intermediate joint having stiffness in between stiff and flexible expansion joint is also possible. Here, discrete shear keys are placed in the inner walls of the expansion joints to increase shear capacity and maintain flexibility. The prestressing cables are cut once the elements are immersed. The three types of expansion joints are shown in Figure 3-22.

The watertightness of expansion joints is achieved through a primary grout injectable rubber-metal waterstop across the match-cast face. The main components of a rubber stop are as follows:

- Rubber bulb at the joint gap that allows joint opening without tearing of the waterstop.
- Rubber flanges on both sides of the central bulb to provide a long water path.
- Metal steel plates that have injectable foam sponges that extend further into the concrete. These improve the adhesion between the rubber profile and concrete.

The water leakage path can be used to describe the injectable waterstop principle. Water from outside initially enters the expansion joint and travels in a straight line until it reaches the waterstop. The waterstop's presence causes the water to turn 90 degrees to the left or right and proceed toward its ends. The concrete obstructs the water's progress if the rubber is compressed against it. If the obstruction fails, the water will advance to the steel plate sections at the waterstop's ends which can stop the water flow as the epoxy resin has been injected into them.

Figure 3-23: W9U-i profile (Leeuw, 2008)

For flexible expansion joints, the injectable waterstop consists of a W9U profile having a width of 350 mm. Several of these W9U profiles based on size and injection capacity were developed by the company "Trelleborg Bakker". It used to be the standard rubber profile used in Dutch immersed tunnels. Over the years, cracks were noticed in the concrete around the W9U profiles. This was because a significant amount of reinforcement was used around the W9U profile and steel plate. This hindered the flow of concrete during pouring operations. This issue was solved by developing W9U-i profiles. These profiles came with an injection tube at the end of the profile as shown in Figure 3-23. This tube is used to inject epoxy resin to seal the gaps and cracks once the segments have hardened, thereby making the joint watertight (Van Oorsouw, 2010).

When the expansion joints opens the rubber profile tends to follow the opening. Moreover, external water pressure will cause the rubber profile to form an arch to reduce stress on the rubber. Thus, the elongation capacity of the rubber profile depends on water pressure and initial joint opening. The design graph provided by Trelleborg Bakker to determine the elongation capacity of the W9U and W9U-i profiles for Serviceability Limit State (SLS) and

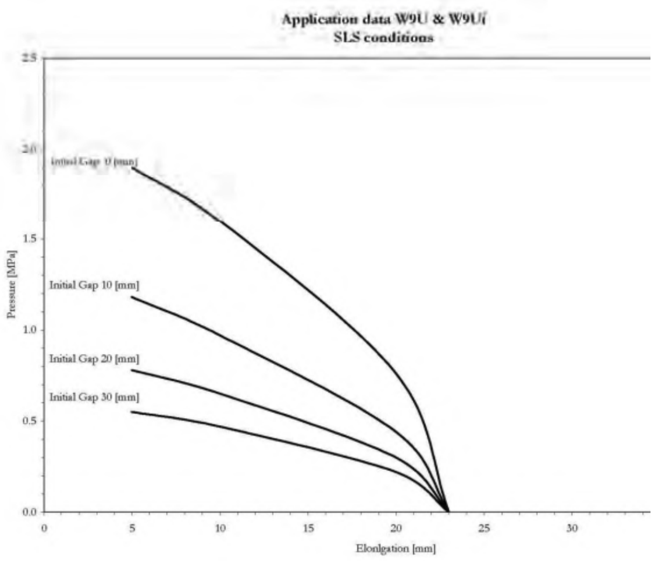


Figure 3-24: Elongation capacity of the W9U and W9U-i profiles for SLS conditions (Leeuw, 2008)

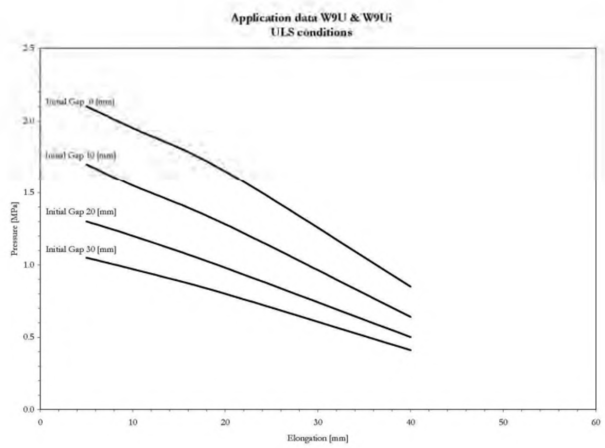


Figure 3-25: Elongation capacity of the W9U and W9U-i profiles for ULS conditions (Leeuw, 2008)

Ultimate Limit State (ULS) conditions are shown in Figure 3-24 and 3-25 respectively. The elongation value can be identified based on in-situ hydrostatic pressure and initial expansion joint gap. The initial joint gap for immersed tunnels is usually considered to be 0.

To ensure further watertightness at the expansion joints secondary barriers are used. Hydrophilic seal is one such secondary barrier placed on the inside of the injectable water stop. When thermal movements of individual segments are combined along with settlements, joint gaps become wider. This seal fails to close the joint gap until a large amount of seepage has occurred during such openings. Another good approach is to apply a High-Density Polyethylene (HDPE) ACME seal externally at the joint surface along the tunnel perimeter to avoid soil seeping into the joint gap. This allows joints to articulate freely. On the wall and roof, this seal is applied once the segments are cast and at the base slab, a stainless plate is used at one of the adjacent tunnel segments as shown in Figure 3-21. An anchor or shear stud is used to hold this plate in place while on the opposite side a debonding material is used to allow the plate to slide freely during the joint gap opening and closure (Lunniss and Baber, 2013).

Lin et al.(2018) claimed that the watertightness capacity of the traditional single seal consisting of a rubber metal waterstop is not reliable. This is further aggravated by the fact that it is the only primary water barrier. The problems encountered are structural defects, joint openings and injectability of epoxy resin. Firstly, due to improper concreting, voids can form at the surface of the waterstop which could cause leakage. In addition, at great water depths, the hydrostatic pressure can intensify the occurrence of joint leakage. Secondly, joint openings stretch the rubber of the injectable waterstop and the thickness of the rubber becomes thinner. Afterward, the rubber and concrete are no longer compressed against each other tightly eventually creating a passage for leakages. Finally, due to the difficulty in installing the injection pipe in a dense rebar arrangement, some of the tubes would not be 100% injectable. Moreover, the additional ACME seal used as the secondary barrier placed at the perimeter fails to function in highly seismic zones, significant differential displacements and at large water depths.

Figure 3-26: New design of prefab reinforcement around the rubber metal joint strip in Caland tunnel (Leeuw, 2008)

The main developments in expansion joints over the past two decades were focused on reinforcement detailing and using a double Omega seal. Firstly, before the 2000s, reinforcement at the expansion joints was designed to withstand its own self-weight and not the support forces of an adjacent segment. In the case of large differential settlements, there is a line load acting at the expansion joint which can cause fracture at the joint tooth. Therefore, after 2002, reinforcement in the tooth was greatly strengthened to support this load as depicted in Figure 3-26.

Secondly, a grout injectable water stop and ACME seal are secondary barriers that cannot withstand large displacements during an earthquake and strong hydrostatic pressures at considerable depths, necessitating a more durable solution. The double seal as depicted in Figure 3-27 was introduced to solve this design issue. The expansion joint in this instance features an injectable waterstop on the outside and an Omega seal inside the joint. The Omega seal acts as a secondary waterstop to stop any potential seepage water and the injectable waterstop acts as the first waterproofing measure at the junction. Owing to the larger tensile

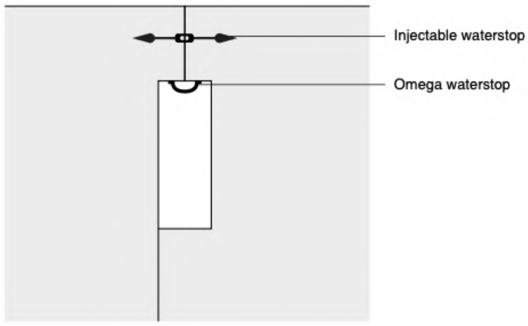


Figure 3-27: Layout of double seal

capacity of the Omega seal to withstand large differential settlements compared to the W9U-i profile, the seal can withstand larger openings of the expansion joints in larger water depths and seismic zones. Furthermore, the water sealing effect of the double seal can be monitored using embedded pipes with pressure meters in the region between the injectable waterstop and Omega gasket (Lin et al, 2022). These double seals have been successfully applied for the Hong Kong Zhuhai Macao Bridge tunnel and Dalian Bay Subsea tunnel in China. To ensure appropriate reliability during an earthquake at great water depths, it is recommended to add Omega seals to every expansion joint in future immersed tunnels.

3-7-3 Closure Joints

The final joint formed by leaving a space of 0.5-1.5 m between either immersed tunnel and an approach structure or two adjacent tunnel elements is called the closure joint. There are several conventional approaches to executing the closure joint. It can be done either in dry or submerged conditions. The dry joint is the simplest approach wherein the construction sequence is such that the immersed part of the tunnel is constructed first before the approach structures. This allows one end of the immersed tunnel to be exposed and the connection to be made in dry conditions. If the construction sequence only allows for the closure joint to be made underwater, the most popular among the conventional methods is the in-situ expansion/ construction joint arrangement depicted in Figure 3-28.

This joint is constructed by blocking space of 0.5-1.5 m between the tunnel elements using wedges or jacks. This wedge block system is used to maintain the compression force at the joints throughout the structure before the closure joint is dewatered. The elements are braced by placing wedges against either the internal or external walls. Once the elements are braced, closure formwork panels are placed around the external perimeter of the tunnel. Subsequently, concreting is performed to fill the void at the joint. Finally, the temporary wedges or jacks are either dismantled or broken from within to transfer the load directly onto the permanent concrete (Lunniss and Baber, 2013). To reduce construction time and complexity associated with conventional closure joints, the three innovative closure joints that have been introduced since 2000 are V-wedge, key element and deployable element.

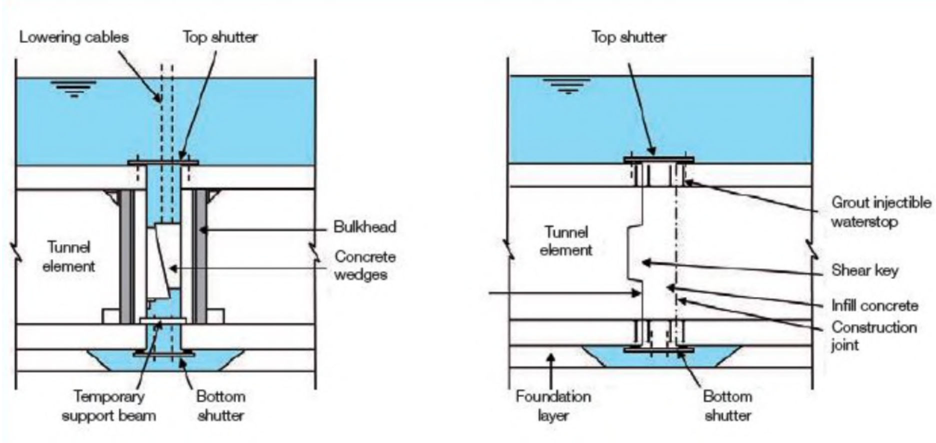


Figure 3-28: Expansion/ construction joint arrangement (Sinha, 2017)

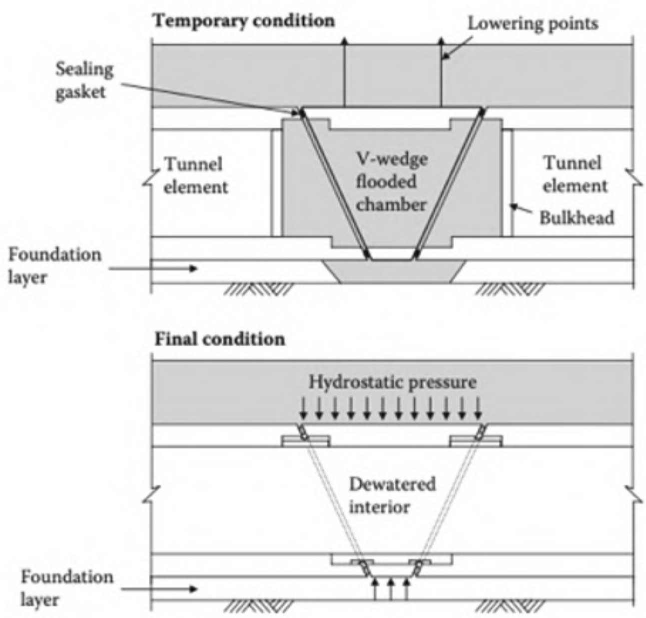


Figure 3-29: V-wedge closure joint (Lunniss and Baber, 2013)

The V-wedge was developed in Japan to avoid the use of external underwater formwork and reduce diver activity. This reduces construction time. Furthermore, the joint is constructed based on the hydrostatic pressure difference between the base and roof slab to preserve the segment's stability. Moreover, the rubber gasket around the perimeter of the previously placed tunnel parts is compressed by the dead weight of the V-block to ensure watertightness. The construction methodology is illustrated in Figure 3-29.

The V-Wedge closure joint was further developed in Japan into the key element method. Instead of using V-blocks, wedge-shaped end frames were used as shown in Figure 3-30. The main advantage of this method was that the marine activities were further reduced and no special immersion equipment was needed to construct the joint. Furthermore, a stretch rubber waterstop was used to accommodate the construction deviations arising from immersion activity. The waterstop has a hollow structure consisting of an external rubber body that is meant to be joined to the key element's opposite end frame. It is folded and placed on the steel end frame of the element adjacent to the key element. The initial connection is made by pumping air into the gasket which thereby increases the water pressure between the bulkheads. Afterward, the pressure between the bulkheads is released and the gasket is filled with mortar. Finally, the weight of the key component further increases the pressure, thereby improving watertightness.

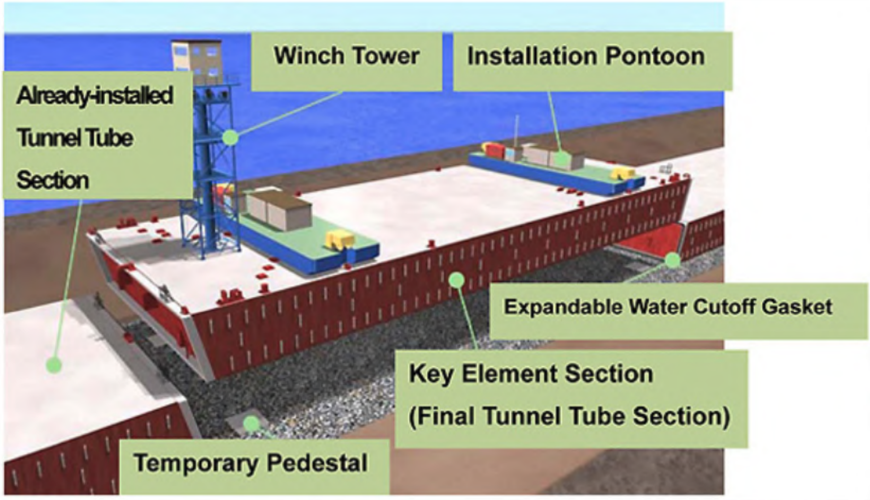


Figure 3-30: Key element method (Sinha, 2017)

Recently, the deployable element method was developed for the HZMB project as the existing closure joints were unable to provide a suitable underwater connection. The basic principle of this method is that the closure joint element remains smaller than its final size during transportation and immersion. The structure has two deployable parts which can extend from the main body along the longitudinal direction to attach and detach to the adjacent tunnel elements when necessary. For a permanent connection steel plates are welded from inside the tunnel and concrete is poured to form a steel-concrete-steel composite structure. The construction methodology is illustrated in Figure 3-31.

The main advantage of this method is that the construction process is short and reversible and provides a strong connection at the joint. The closure joint can be formed within a day and reduces risk by eliminating diver activity. Jacks are used at the joints to ensure that sufficient Gina gasket compression is achieved. In case of any misalignment, the deployable element can be detached to make a reconnection and ensure installation accuracy.Lin et al. (2018) pointed out that the method does come with difficulties. Firstly, a large crane ship is needed for placing the element. Secondly, if special foundation treatment is not performed the differential foundation stiffness between the deployable and adjacent element can result in significant differential settlement. Finally, the jacks used for making the connection cannot be removed and reused. The first two shortcomings can be addressed by meeting two conditions.

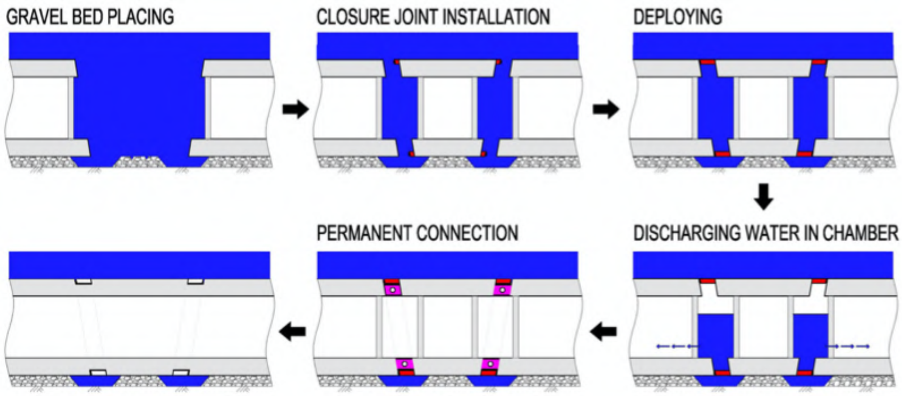


Figure 3-31: Construction procedure of deployable element method (Lin et al., 2018)

One is that standard immersion rigs can be used for immersion and the other is to balance its dead weight through buoyancy when immersed. This can be done by either making length of the deployable element identical to that of a typical element or sufficiently long. Figure 3-32 illustrates the possible longitudinal layout options depending on the design's structural criteria. The third disadvantage can be solved by placing the jacks at corresponding locations of the corbel supporting the bulkheads or placing them on the inner side of the tunnel. Thus, the jacks can be removed once the permanent connection is made.

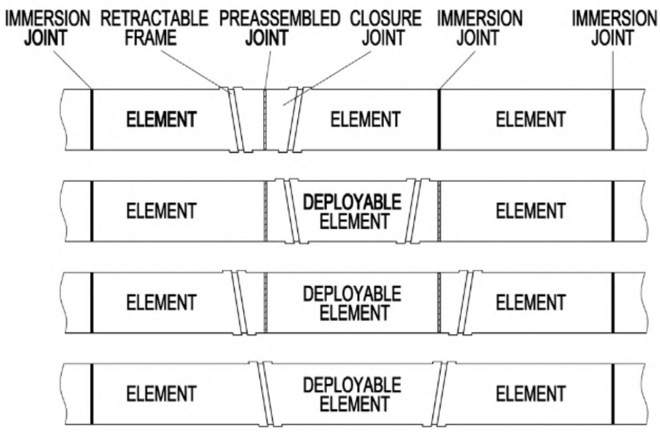


Figure 3-32: Possible longitudinal forms of deployable element (Lin et al., 2018)

3-7-4 Shear keys

The rubber gaskets do not offer much shear resistance at the immersion joints because they have lower axial and shear stiffness than the tunnel concrete body. Thus, shear keys are used to prevent horizontal or vertical displacement of the tunnel elements concerning one another and to prevent joint components from being damaged. For concrete tunnels, shear keys present in the walls transfer the vertical shear forces while those in the roof and floor transfer horizontal shear forces. Shear load transfer in steel tunnels is achieved by making the structure continuous across joints.

The traditional shear keys that were used to achieve load transfer are dowel and half-joint shear keys. Dowel shear keys are circular hollow steel sections filled with concrete. They are cast into the infill concrete on one side of the immersion joint for allowing shear transfer as shown in Figure 3-33. The main limitation of this shear key is that it can be placed within a limited depth of concrete and has low shear capacity. In case of high loads, shear stresses will concentrate at these keys and requires a high amount of reinforcement to prevent shear failure.

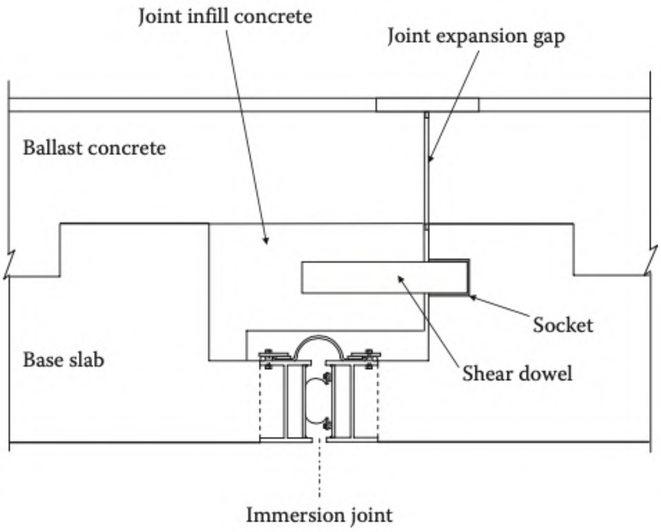


Figure 3-33: Dowel shear key (Lunniss and Baber, 2013)

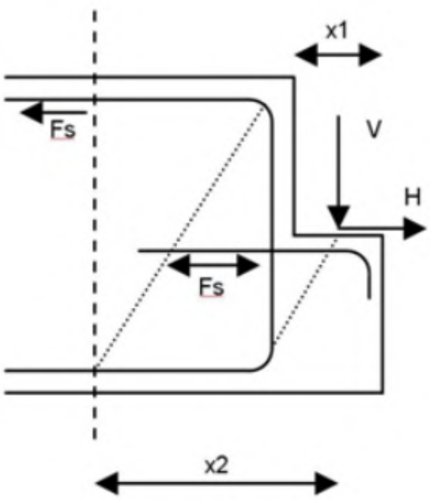


Figure 3-34: Load distribution in concrete collar (Van Oorsouw, 2010)

At the expansion joints, shear keys are generally placed on the outer wall to form a spigot and socket-type joint as shown in Figure 3-21. These shear keys are also called concrete collar or half-joint shear keys. When a vertical load V acts on the collar, the load is transmitted diagonally to the key as shown in Figure 3-34. The resistance to this load is provided by the horizontal tensile force (F_s) from the horizontal reinforcement. Conversely, this horizontal capacity is reduced by horizontal force H which is a function of vertical load V and friction coefficient μ. Van Oorsouw (2010) suggested that for static loading, the shear capacity for a concrete collar is 1000 kN/m. Applying a typical segment length of 25 m and friction coefficient varying between 0.1 - 0.5, vertical shear capacity is in the range of 16 and 24 MN while horizontal shear capacity is in the range of 6 and 9 MN.

The main challenge with this design ultimately is assessing the area effective in shear load transfer. For span widths, more than 10 m, a significant section of the half joint is ineffective in transferring shear. Due to this shear lag, shear loads are concentrated close to the section of the walls and require appropriate reinforcement detailing to avoid shear cracking. The shear capacity of this key depends on the amount of reinforcement and thickness of walls and slabs. The greater the reinforcement, the greater the capacity to withstand shear loads. Nevertheless, concrete casting becomes difficult with more reinforcement and this can lead to the formation of pores and cracks in concrete. Furthermore, in case of cracking, this type of shear key is difficult to access for repair since it is located in the outer walls.

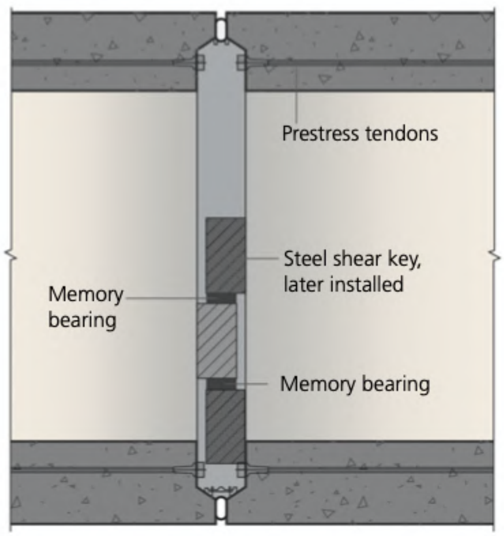


Figure 3-35: Side section view of steel shear key arrangement at immersion joint (Lin et al, 2022)

To overcome the limitations of conventional dowel and concrete shear keys, steel and discrete shear keys have been developed. Steel shear keys are a three-part shear keys where two keys are fixed to the secondary end of one element and one to the primary end of the following tunnel element as shown in Figure 3-35. Each key consists of a box or deep I-section fixed on a base plate which is then fastened to the end faces of the tunnel element using bolts. The box section is reinforced by stiffening ribs within its hollow interior. The keys are fitted at the immersion joints once immersed but locking of the shear keys is postponed until all loads have been imposed and initial settlements have occurred. Since space is required for immersion joint components in the top and bottom slabs, the depth of the three-part keys is less than one-third of the entire height. Moreover, any horizontal misalignment between tunnel elements due to variation in settlements across the section or tolerance inaccuracies in the bearing surface can result in lateral eccentric loading. This could result in torsion effects on the element. As a result, the combination of eccentric loading and shallow depth of shear keys can generate high tensile stresses at the extreme fiber of the shear keys. To transfer this tension back to the element, a considerable amount of reinforcement would be needed to fix the base plate of the steel shear key to the concrete structure. This is an important design consideration to be taken into account while using steel shear keys (Lunniss and Baber, 2013).

Memory bearings are placed at the vertical steel shear key of the immersion joint to lock the shear key once settlements have stabilized and to reduce the eccentric loading effect by ensuring there is sufficient contact between the keys to allow for a uniform bearing pressure. For significant differential settlements, the shear force transmitted through shear keys is large and concrete cracking can take place. The memory bearing protects the shear keys by allowing itself to be compressed thus enabling differential settlement within a particular range to take place and redirecting the load to the foundation. The bearing maintains a constant reactive force during compression that can be pre-set to be no greater than the structural capacity (Lin et al, 2019). Based on experimental studies done by Xiao et al (2017) and Yuan (2019), it was found that the shear capacity of the immersion joint is not the sum of all vertical steel shear keys. This is because not all the shear keys carry the shear force simultaneously. This reduced shear capacity can also be attributed to the fact that bearings were not used to ensure even distribution of shear forces. Hence, different shear keys were activated at different shear displacements. Moreover, the Gina gasket contributes significantly to the shear capacity of the joint. Xiao et al (2017) pointed out that the maximum shear capacity obtained for a single box shear key was 19 MN.

Discrete shear keys address the shear lag issue encountered in steel shear key due to limited depth. It is usually considered the most favoured solution as they are placed in the slabs and walls. This ensures effective shear transfer across the structure as shown in Figure 3-36. Additionally, they can be applied at both immersion and expansion joints.

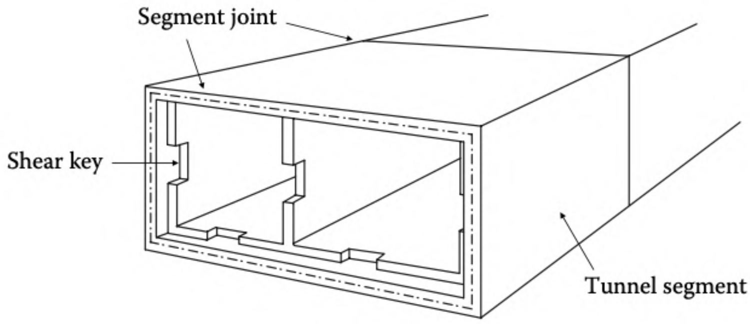


Figure 3-36: Discrete shear key (Lunniss and Baber, 2013)

The shear capacity of these keys is based on the width and height of their dimensions. The width of shear keys is limited by the width of the inner walls and partial width of the outer walls unlike the full width in the case of the concrete collar. For the height, approximately one-third of the tunnel height is used. The shear load transfer within this key works similarly to the way of the concrete collar as shown in Figure 3-37. Similarly, the horizontal force H due to friction between the segments reduces the shear capacity of this key. Four discrete keys are acting at the joint, the total shear capacity in the horizontal and vertical directions is found to vary between 16-24 MN based on the friction coefficient (Van Oorsouw, 2010).

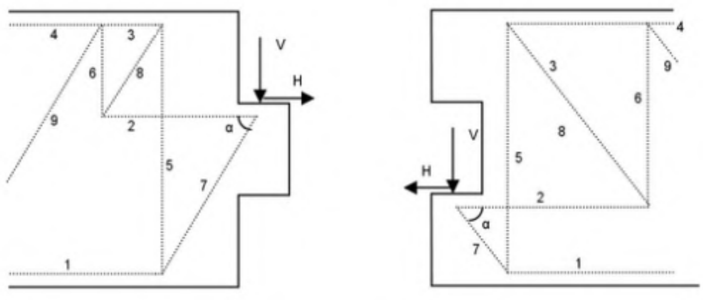


Figure 3-37: Load transfer mechanism in discrete shear keys (Van Oorsouw, 2010)

At expansion joints, shear keys are installed at recesses of previously cast segments. The effect of thermal expansion and contraction can cause lateral restraint to the segment and produce cracks. One possible way to solve this issue is by installing a compressible layer in both the vertical and horizontal faces. Another important consideration is that a significant amount of reinforcement is required beneath the bearing surface of shear keys to withstand the shear load. Therefore, reinforcement detailing should be chosen appropriately to ensure that there is no hindrance to concreting (Lunniss and Baber, 2013).

Compared to concrete collars placed at expansion joints, discrete shear keys have a much higher shear capacity. The shear capacity of the joint is provided by four shear keys instead of a single shear key. Moreover, the shear capacity is equal in both directions and so is more suitable for seismic zones. Since the dimensions of discrete shear keys are larger than the concrete collar, the size and amount of reinforcement can be increased to further extend the shear capacity. Besides, they can withstand the rotational moment generated in the case of joint opening and differential settlements much better. Another benefit is that these joints have much better accessibility since they are placed within the tunnel. In case of any damage arising in the shear keys, maintenance can be easily carried out without posing a leakage risk. The water-sealing rubber profile in these keys is placed on the outer walls and slabs.

Discrete shear keys offer greater shear capacity compared to steel shear keys. However, Van Oorsouw (2010) stated that when the concrete shear keys are subject to cyclic loading in seismic zones strength of the structure decreases and strains increase at each load step as shown in Figure 3-38. As a result of this reduced shear capacity, cracks form and even a fracture of the discrete shear key could take place. In such scenarios, the steel shear key allows the structure to behave in a ductile manner. During each load, more strains will be generated while strength remains the same. Cracks generated due to the plastic behaviour can be repaired and is preferred over the brittle failure of discrete shear keys. Additionally, the steel shear key offers ease of installation at greater depths compared to the discrete shear key where complex concreting operations have to be performed. The steel sections can be prefabricated with higher accuracy and ease.

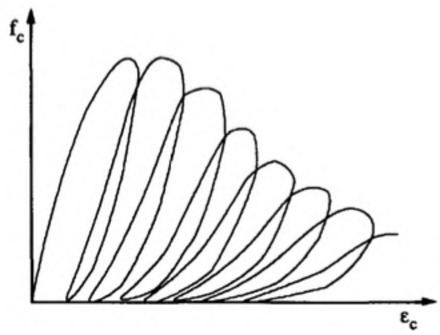


Figure 3-38: Stress strain curve for concrete under cyclic loading (Van Oorsouw, 2010)

Chapter 4

Potential Sensors And Monitoring Techniques For An Immersed Tunnel Monitoring Plan

In this chapter, the third and part of the fourth research question are answered by addressing how structural problems can be observed through monitoring and providing a catalogue of potential sensors and monitoring techniques that can be applied. In Section 4-1 an overview of conventional monitoring techniques used before the 21^{st} century is provided. In the following Section 4-2, defects of conventional monitoring plan specific to each main aspect of monitoring are discussed. The parameters to be monitored, threshold limits and the reasons for choosing them are given in Section 4-3. Finally, Section 4-4 outlines the working principle and installation method of potential sensors and monitoring techniques that can be applied to immersed tunnel monitoring.

4-1 Brief Introduction Of Development In Conventional Immersed Tunnel Monitoring

During the service life of tunnels, monitoring various aspects of immersed tunnels such as deformations, leakage rates and corrosion is a good strategy to examine the structural integrity of the tunnel. In the conventional monitoring methods, referred to as the methods used before the 21^{st} century, monitoring for longitudinal settlements was done using manual levelling with a total station, invar ruler and strain gauges, see Figure 4-1. These techniques rarely examine longitudinal and lateral tunnel movement owing to difficulties in attaining acceptable measurements (Zhang and Broere, 2022).

Extensometer was another instrument at times used to understand the tunnel deformation by measuring the joint gap opening. By arranging the extensometer in a certain configuration, this device can be applied in existing and new tunnels to understand the degree of joint

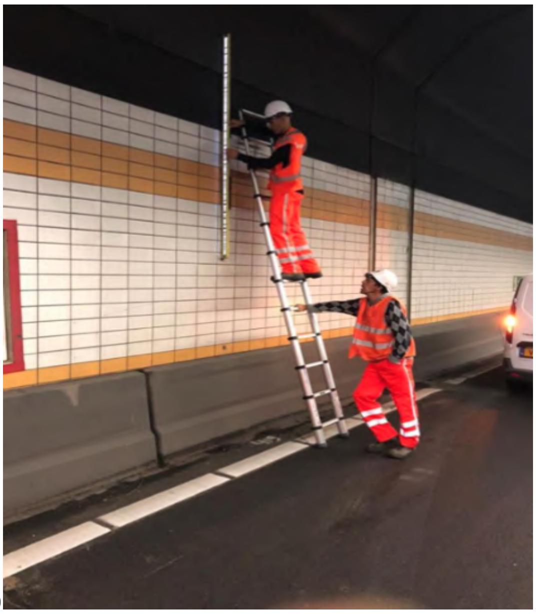
a)  b) 

Figure 4-1: Invar ruler in 1st Heinenoord Tunnel (Fugro, 2018) and b) Total station used for monitoring deformation behaviour, (Trimble 2020)

rotation. A more detailed description of this instrument is given in section 4-6. To check for leakages and corrosion when cracks are formed visual inspections were performed. The degree of damage determined by this method was subjective and varied based on the experience of maintenance personnel. Furthermore, these inspections failed to capture the full extent of the damage as only the exterior of the affected surface can be examined. The leakages and corrosion associated with rubber gaskets and their components were checked using intrusive methods using an endoscope or a keyhole operation (Van Montfort, 2018). This was because the space in front of the Omega profile was concreted once individual elements were connected. These methods are explained further in section 4-2.

In conventional monitoring methods, tunnel tubes had to be fully or partially closed for a temporary period to obtain the monitoring data. This resulted in both traffic hindrance and economic losses. This limited the monitoring frequency to annual or multi-annual intervals. The low-frequency measurements obtained using these techniques fail to capture the unexpected structural deformation and thus, quick remedial measures cannot be executed in time. For instance, there was no monitoring data for the Maas Tunnel in the Netherlands from 1941 to 1968, indicating that there was a 27-year period during which there was no information available about tunnel settlement. It was noted that the majority of the settlements occurred during these initial years (Grantz, 2001). Another example is the Shanghai Outer Ring tunnel, where damage to the Gina gasket was identified only when a thorough manual inspection was carried out because the normal monitoring technique failed to detect the joint deformation in time (Bai and Lu, 2016). Such incidents can be treated if a real-time monitoring system was present to monitor tunnel behaviour before irreparable damage occurs.

Over the past two decades, more attention has been provided to structural health monitoring and has led to the development of different types of sensors such as fiber optic sensors, laser scanners et al. that can be used to monitor various parameters of immersed tunnels. Therefore, it is essential to create a new monitoring system with high frequency, accuracy, limited traffic hindrance and provide a better understanding of behaviour of immersed tunnels by making use of potential sensors in existing and new immersed tunnels.

4-2 Main Aspects In Immersed Tunnel Monitoring

4-2-1 Deformation sensing

Tunnel deformations can be further categorised into segment body and joint deformation based on the region of the tunnel body affected. One common effect of deformations for both regions is concrete cracking. Traditionally, differential settlements were monitored manually using total station, invar ruler and strain gauges. Inspections for cracks were conducted annually according to SEG (2019). If major cracking occurs, the development of crack widths was monitored using strain gauges. As addressed in section 4-1, these methods have low frequency, efficiency, hinders traffic and fail to capture full tunnel behaviour.

- Element/ segment body deformation
  The two main deformation effects acting along the body are torsion, vertical and transverse displacement. Conventionally, torsion and transverse displacements were rarely monitored.

- Joint deformation
  Deformation at the joints can be characterised by joint openings and rotation. Conventionally, extensometers were installed at the joints to only measure joint width. However, extensometers can be used to measure joint rotations as well.

4-2-2 Contact force sensing

Shear keys are crucial to ensure that shear transfer occurs at the joints and to prevent excessive differential displacement between tunnel elements. The capacity of shear keys is reduced due to tunnel deformation. Furthermore, shear keys undergo failure once the capacity of shear keys is exceeded. Conventionally, contact forces at the shear keys were not monitored. Measuring the contact force between the shear keys helps calculate the forces acting on the shear key.

4-2-3 Leakage detection

Leakages are the most common structural problem encountered during the service life of immersed tunnels. Traditionally, the presence of leakages was checked by visually checking for drops at the joints and tunnel body. In case of larger leakage rates, which usually occurred at the joints, the watertightness of rubber gaskets was checked. As the space above the Omega profile was concreted once the individual elements were connected, it was difficult to examine the watertightness of the Omega profile. According to Van Montfort (2018), the two conventional methods used to check for leakages in front of the Omega gasket are as follows:

- Endoscope
  In this method, an endoscope which consists of a small camera along with a light source is used. A hole is drilled at the joint into which the endoscope is inserted and pictures are taken to examine for leakages as shown in Figure 4-2. As sight is limited, the quality of pictures taken is often poor.

a) 

b) 

Figure 4-2: a) Endoscope inserted in joint above barrier b) image of corroded bolt present in Drecht tunnel (Nebest, 2018)

a) 

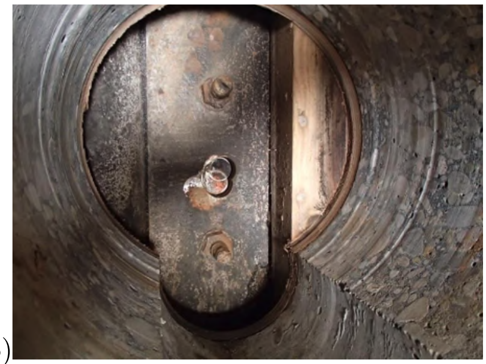
b) 

Figure 4-3: a) Keyhole operation being executed in Kil tunnel (de Haan and Bras, 2018) b) Picture of the clamping structure of the Omega-seal in the Kil Tunnel (Van Montfort, 2018)

- Keyhole operation
  In the second method, the concrete cover above the Omega profile is removed. This provides better visibility for inspection even though the process is more time-consuming as seen in Figure 4-3.

Both the above-mentioned methods are intrusive and time-consuming. If there are water leaks in the space between Gina and Omega gasket, the pipework for pressure testing can be used as entry points for checking the presence of leaks (Lunniss and Baber, 2013).

4-2-4 Corrosion sensing

Based on the structural problem study corrosion has been noticed at the gasket fixture components, steel end frame and reinforcement. At low rates of uniform corrosion of 0.11 mm per year, functionality of attacked components is not affected. However, if the rate is significantly large such as 2 mm per year for pitting corrosion then the structural capacity of affected components can be lost. Traditionally, they are noticed only during visual inspections when concrete cracks, spalls or have brown spots on the surface. To determine the state of corrosion within gasket components and steel end frame, endoscopes or keyhole operations were executed. Moreover, the images obtained through the endoscopic method showed only the presence of corrosion and were not sufficient to determine the extent of corrosion. For examining the state of corrosion in reinforcement, the concrete cover within the structure had to be removed.

4-3 Parameters To Be Monitored

The parameters to be monitored, corresponding threshold limits and the reasoning behind why these parameters were chosen are listed as follows :

- Vertical displacement (Settlement): To understand the differential settlement behaviour of immersed tunnels, vertical displacements of points located on either side of immersion and expansion joints are measured. Based on the structural problem study it was noticed that the predicted design settlement was often exceeded during its service life. These settlements varied between adjacent segments and elements resulting in differential settlements. They affected the tunnel alignment, capacity of shear keys and rubber gaskets. Furthermore, due to transverse differential settlements, the tunnel can undergo torsion. This torsion creates additional loads at the joints and longitudinal cracks. Torsion can be measured by determining vertical displacements at three points A, B and C present on either end of the segment as shown in Figure 2-24. The threshold limit for differential settlement is set as ± 5 mm between adjacent segments/ elements.

- Transverse displacement: Differential transverse displacements lead to increased stresses at the joint and deformation of rubber gaskets. This parameter was rarely monitored in conventional monitoring plans. It can be measured by installing a sensor block at joints located in the roof and setting a threshold limit of ± 10 mm. This threshold helps to limit variations in tunnel alignment, reduce joint stresses and gasket deformation

- Joint width: Widths of the joint expand and reduce due to longitudinal movements. Limited openings can be accommodated by the rubber gasket. However, when the openings are too large the prestress at the Gina gasket is reduced and gaps are formed. These gaps create a pathway for water to enter the tunnel resulting in higher leakage rates. Similarly, if the joint width is negligible it means that the Gina gasket has undergone overcompression. If the joint widths along the wall vary it means that joints have undergone rotation. These rotations further aggravate the effects of joint gaps. According to Fugro (2018), the limit of joint widths can be set at 10 mm. As it is difficult to check the watertightness of the injectable waterstop at expansion due to lack of space, monitoring joint widths would give a preliminary warning about the possible failure of the waterstop. Furthermore, monitoring joint widths help determine if the gaskets have been overcompressed or decompressed.

- Crack width and concrete strain: Differential displacements, large joint gaps and temperature variations of concrete result in high strain values which eventually result in concrete cracking. These cracks provide a route for water to leak into the tunnel. Additionally, if the cracks are significant structural integrity becomes compromised. According to Lunniss and Baber (2013), crack width limits of 0.2 mm can be used to ensure the durability of the tunnel. Furthermore, a maximum tensile strain limit of 0.0002 can be set to ensure cracks do not form. This limit can be set as an indicator of the formation of rust due to corrosion in steel.

- Shear key force: Failure of shear keys can result in substantial differential settlement and reduces the structural capacity of immersed tunnels. Experimental studies done by Xiao et al (2017) showed that shear forces are not evenly distributed in shear keys. In the conventional monitoring plan, shear stresses were not monitored. Placing sensors at the shear keys which measure contact stress would help understand the development of shear forces at the joint and whether the capacity of shear keys is exceeded. Based on the studies done by Xiao et al (2017) and Van Oorsouw (2010), the shear capacity for steel shear keys can be set at 19 MN and 16-24 MN for concrete shear keys based on the friction coefficient.

- Leakage detection: Leaks are usually noticed at the joints and poses the main challenge for tunnel owners to avoid or mitigate. To a certain limit, leaks can be pumped out without any issue. However, Parwani (2014) stated that flow rates greater than 4 m^3/hr it becomes a risk to tunnel safety. Reinforcement corrosion, visual discomfort to the drivers and soil intrusion resulting in settlement occur at large leakage rates. Furthermore, during winter the water can freeze over and form ice on the roads resulting in tunnel tube closure. Quick detection of leaks would help take quick remediation measures.

- Chloride diffusivity and pH in concrete: As aforementioned corrosion is caused mainly due to chloride ingress and carbonation of concrete. This corrosion leads to reduced structural capacity of load-bearing components, concrete spalling and cracking. Corrosion during the initiation phase is hard to detect as no visible damage is present on the concrete. Current investigation methods examine the state of corrosion only in the deterioration phase. Remediation measures at this stage are considerably more costly and time-consuming compared to measures taken during corrosion deterioration stage. According to Hire et al (2022), adopting appropriate corrosion monitoring methods can help save up to 35% of the cost of immersed tunnels. Monitoring environmental parameters of the concrete such as Cl^- levels and pH helps to asses the state and rate of corrosion in tunnels. According to Lunniss and Baber (2013), the chloride diffusivity limit should be set as 7 x 10^{-12} m^2/ sec. Liu et al (2018) suggested that reinforcement corrosion can occur when pH is between 11.3 and 12.1.

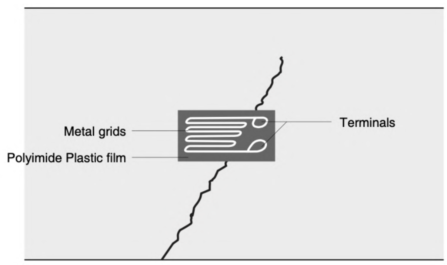
- Shear modulus of soil: Grantz (2001) claimed that the stiffness of underlying soil was the most important factor which contributes to settlements. Moreover, natural processes such as scouring can alter the foundation stiffness. The shear modulus of soil can be used to define soil stiffness. This parameter is rarely monitored in conventional methods and therefore, determining the stiffness properties of underlying soil would help understand occurring settlements and predict settlements in immersed tunnels. Checking for the presence of soft soils underneath the tunnel would be a good indicator for long-term creep settlements. In seismic zones, the stiffness of cohesionless soils such as sands would help predict the liquefaction potential of soil.

4-4 Inventory Of Potential Sensors

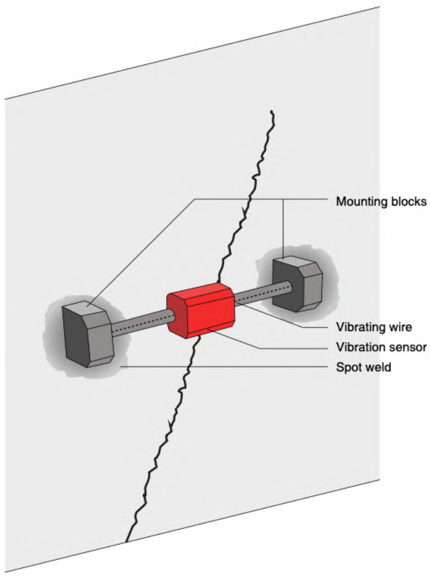
This section describes the working principle and installation of sensors that can be used to monitor the parameters stated above.

4-4-1 Strain Gauge

Strain gauges are the most conventional and used sensors for monitoring the structural behaviour of immersed tunnels. They capture the strain resulting from bending, torsion, shear, compression and tension of the elements and/or segments. Electrical strain gauge (ESG), vibrating wire strain gauge (VWSG) and optical fibre bragging grating (FBG) based strain gauges are three common commercial strain gauges based on electrical, mechanical and optical principle respectively.



a)



b)

Figure 4-4: a) Electrical strain gauge and b) vibrating wire strain gauge installed across crack width

Electrical strain gauges form the most common and inexpensive strain sensor among strain gauges. It consists of a thin foil grid attached to a thin plastic film bonded to the monitored component using an adhesive as shown in Figure 4-4. The foil changes resistance in response to changes in the strain of the monitored component (Ştefănescu, 2011). These strain gauges are fragile and need considerable care during installation. Unless properly sealed they are prone to damage by water. Furthermore, they are affected by electromagnetic interference and hence are suitable only for short-term monitoring (Zalt et al, 2007).

VWSGs consist of a pre-tensioned steel vibrating wire placed between two lugs as shown in Figure 4-4 with a sensor placed on top of the wire. The sensor measures the natural frequency of the vibrating wire which is linearly proportional to strain of the wire. They can be installed across cracks and along joints by welding to measure concrete strains and crack widths. These sensors are inexpensive, immune to electromagnetic interference and can be applied for long-term monitoring as the wire does not decay over time (Zalt et al, 2007). However, they require long lengths of wire to carry out measurements and obtain the output.

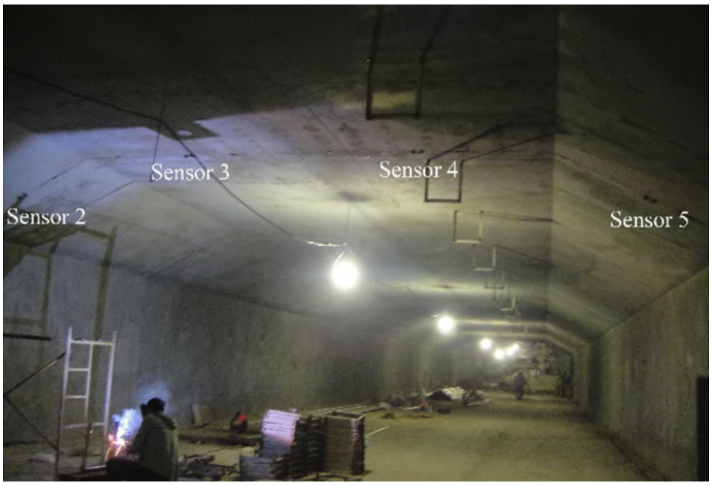


Figure 4-5: Field installation of FBG sensors (Wei et al., 2018)

Fiber Bragg gratings (FBG) strain gauges are lightweight, highly accurate sensors. They are used to measure strain and temperature along its length to obtain a continuous distribution of strain. They provide real-time monitoring of strains. Similar to VWSG strain gauges, they don't undergo decay in the long term. For instance, Wei et al (2018) used FBGs to monitor concrete strain caused due to tidal loads at daily intervals in Zhoushan immersed tunnel. The sensors can be installed along the walls as shown in Figure 4-5 to measure circumferential concrete strains (Fan and Bao, 2021). However, they can not be used to measure local strains and are expensive to install.

4-4-2 Extensometer

Extensometers are instruments that measure strain directly, eliminating interference from other components and improving accuracy. They can operate in large gauge lengths and thus allows to measure large joint widths compared to strain gauges which are limited to short gauge lengths and local strain. Extensometers can be divided into two categories: contact and non-contact. Fiber optic extensometers are one example of contact extensometers. These sensors are placed inside a protective covering and spot welded across the joints. Figures 4-6 and 4-7 illustrate the layout and working principle of the fiber optic extensometer respectively. The basic principle is that as the joints open and close, the sensors measure the joint width. The working principle of fiber optic sensors is explained in more detail in section 4.4.3. According to the websites of Zwick Roell (n.d.) and Industrial Physics (n.d.), contact extensometers are durable devices that can work up to gauge lengths of 100 mm. Even though a

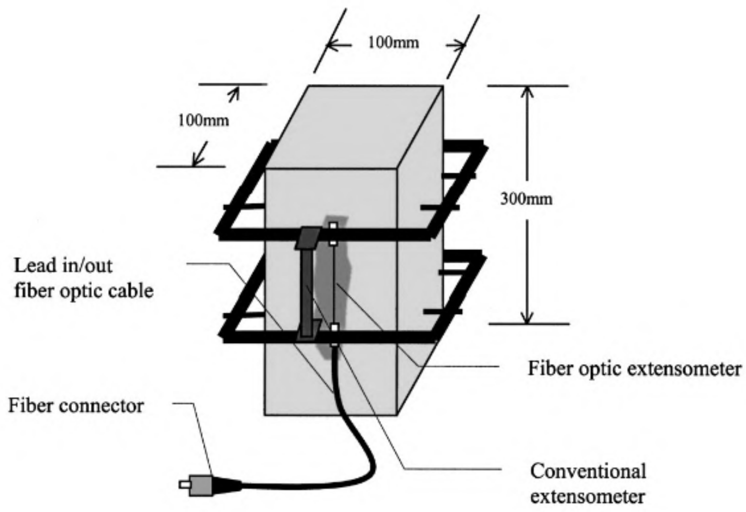


Figure 4-6: Layout of surface mounted fiber optic extensometer (Yuan et al, 2002)

non-contact extensometer can work in higher displacement ranges upto 1000 mm, it would not be economically feasible to set up a non-contact extensometer at each joint.

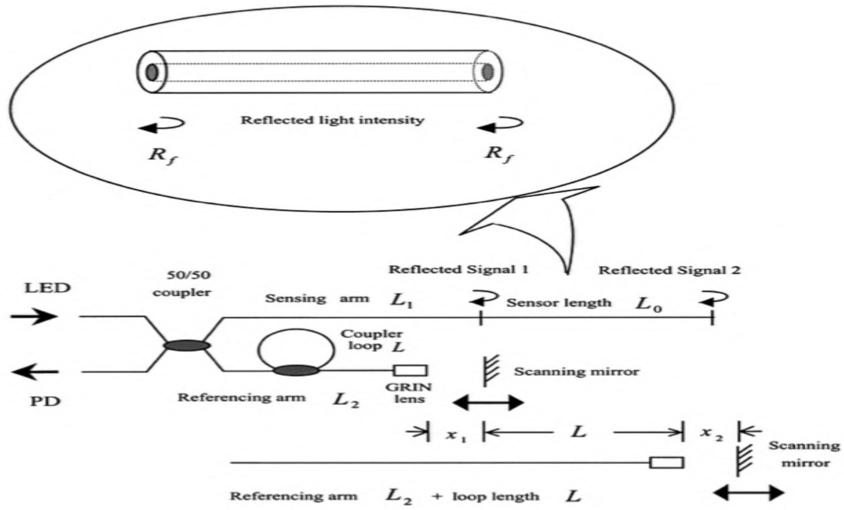


Figure 4-7: Measurement principle of fiber optic extensometer (Yuan et al, 2002)

4-4-3 Distributed Optic Fiber Sensor

Distributed optic fiber sensor (DOFS) is a new type of sensor increasingly being applied in structural health monitoring. The system consists of a continuous optic fiber cable and a signal interrogator as shown in Figure 4-8. The optic fiber is attached to the target structure to serve the dual functionality of the sensor and signal transmission medium. In scenarios where the monitoring location is largely inaccessible, the fiber itself is extended to a remote-control data-taking device while the sensing part of the fiber remains attached to the structure. The interrogator serves as both the light source and receiver for the investigation of Brillouin frequency shift.

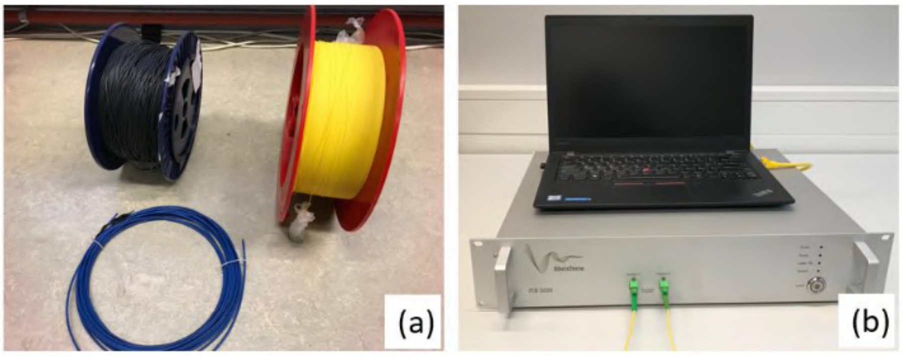


Figure 4-8: (a) Optic fiber cables (b) interrogator with operation computer (Zhang and Broere, 2022)

The basic principle of this sensor is that when light travels along the optic fiber, scattering takes place. One form of scattering is Brillouin scattering wherein the frequency of the backscattered light is different from the initial propagating light frequency. This Brillouin frequency shift can be interrogated constantly to obtain distributed strain and temperature information along the fiber cable. The relation between the Brillouin frequency shift $\Delta\nu$, strain (ϵ) and environmental temperature variation (ΔT) is given as follows:

$$\Delta\nu = C_{\varepsilon}\varepsilon + C_t\Delta T \tag{4-1}$$

where C_{ε} and C_t are strain and temperature sensitivity coefficients respectively which are constant properties of the fiber. For field monitoring, the layout suggested by Zhang and Broere (2022) for the First Heinenoord Tunnel can be adopted. It is a special layout consisting of two fiber lines as depicted in Figure 4-9 to obtain the displacement indirectly from the fiber strain. In this layout, the optic fiber was prestressed so as to detect extension and contraction. For the unstressed section, PVC duct was used to protect the cable. Moreover, to reduce installation difficulty the same layout was utilised for all monitored joints. This layout allows for simultaneous measurement of both horizontal joint opening and differential settlement of immersion and expansion joints at sub-millimetre accuracy. Transverse displacements can be examined by installing the sensors on the roof of utility ducts as installation on the roof of the traffic tube can be difficult due to limited access and temporary tunnel closure.

DOFS can also be used to monitor pitting and uniform corrosion in the corrosion deterioration stage along the entire length of the fiber. Furthermore, the fiber is resistant to corrosion and can withstand high tensile loading. Here the optical fiber is tightly wound across the steel rebar before concreting, see Figure 4-10. Once rust is formed during rebar corrosion, it affects the strain of the optical fiber, see Figure 4-11. This strain is measured by the optical fiber which indicates that the reinforcement is undergoing corrosion (Fan and Bao, 2021). During in-situ application, the small bar diameters can result in attenuation of the intensity

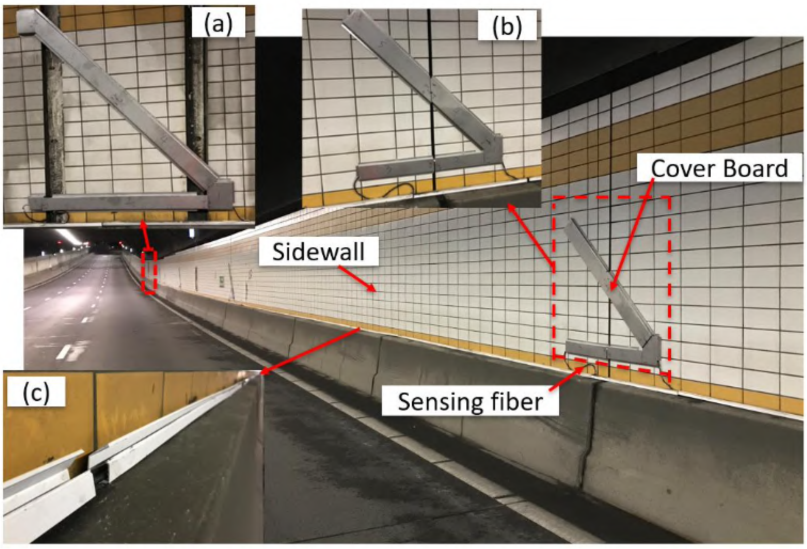


Figure 4-9: Sensor layout in First Heinnoord Tunnel at a) Immersion joint b) Expansion joint c) Loose fibre present in protective PVC duct (Xuehui and Broere, 2022)

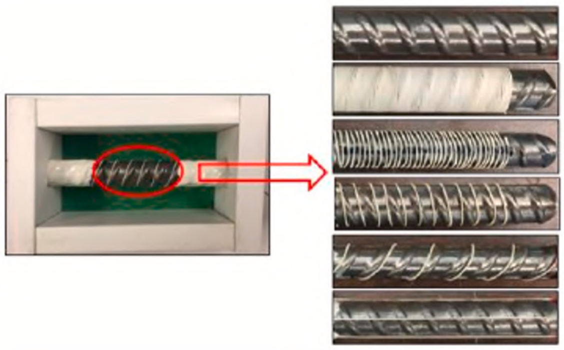


Figure 4-10: Sensor installation of DOFS (Fan and Bao, 2021)

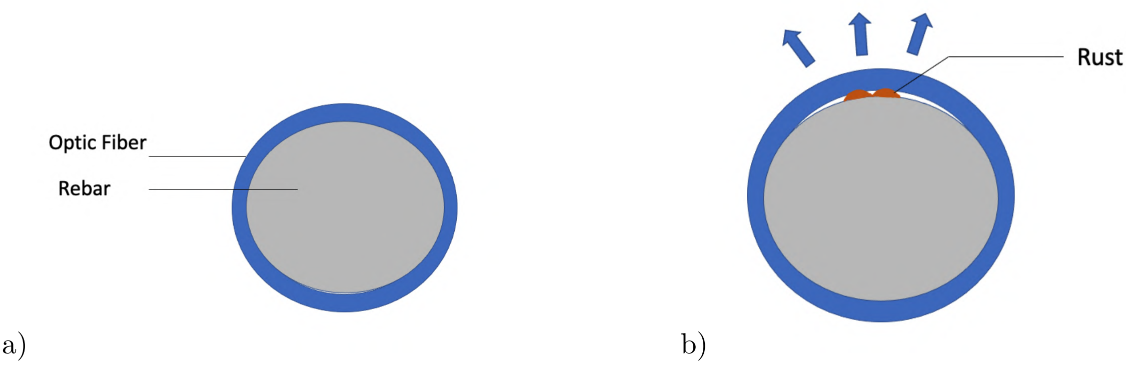


Figure 4-11: DOFS a) without straining b) strained after formation of rust

of light due to the macro-bending of the cable over long distances. Furthermore, the spatial resolution reduces over long distances (Fan and She, 2021).

4-4-4 Swellable polymeric fiber optic sensor

Fiber optic moisture sensors provide a potential solution to detect leakages. Different attempts in designing this sensor have been undertaken in the past decade. Among them, developed by Bremer et al. (2014) is a swellable polymeric fibre optic sensor to detect leakages in sewage pipes. The sensor consists of a rod coated with hydrogel, optical fibre and a device for micro-bending the fibre as shown in Figure 4-12. A nylon thread was helically twisted to create micro bends. The hydrogel coating swells when it comes in contact with water and makes the optical fiber press against the nylon thread. Subsequently, the light being transmitted through the fiber becomes attenuated. This attenuation is then related to the presence of water. The sensor was covered with a felt wick to be protected against external damage. Based on experimental studies, it was found that in the presence of water the sensor showed a light attenuation value of 34.2 dB. Moreover, this sensor functioned well in highly alkaline environments with pH values up to 13.4. The long-term suitability of this sensor is yet to be investigated. Due to the small size of this sensor, it can be installed in the space in front of the Omega gasket for automatically detecting leakages. This would eliminate the need for intrusive methods to check for leakages in future tunnels.

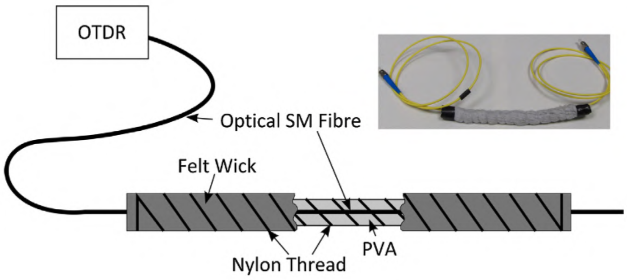


Figure 4-12: Layout of fibre optic swellable polymeric sensor (Bremer et al., 2014)

4-4-5 Spectrometer

Chloride concentration and pH of concrete are indicators of the onset of corrosion which can be monitored using spectrometers. The working principle of spectrometers is such that a continuous light beam is transmitted along the optical fiber. Once the light reaches the target sample, a part of the light is reflected by the sample and collected by the fiber. The intensity, frequency and phase of the reflected light wave are then assessed to monitor the chloride concentration and pH level (Fan and Bao, 2021).

An optic fiber chloride sensor is a type of spectrometer that can be used to assess the onset of rebar corrosion by measuring the chloride concentration. The layout of a silver nitrate (AgNO3) based chloride sensor to monitor chloride concentration in reinforced concrete is shown in Figure 4-13. The sensor is made of an input/output fiber, porous membrane and stainless steel sleeve containing the indicator solution. Chloride ions present in the concrete pass through the membrane and react with silver ions present in the solution. Silver chloride is formed as a precipitate which changes the colour of the anionic dye present. Light is injected along the cable and the intensity of reflected light varies due to the colour change.

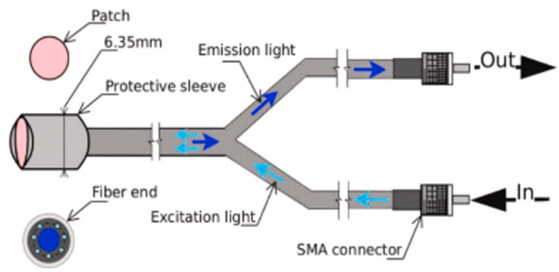


Figure 4-13: Layout of optic fiber chloride sensor (Dhouib et al., 2019)

The intensity of reflected lights is analysed by a spectrometer to monitor the chloride concentration. The intensity of reflected light increases with chloride concentration (Dhouib et al, 2019).

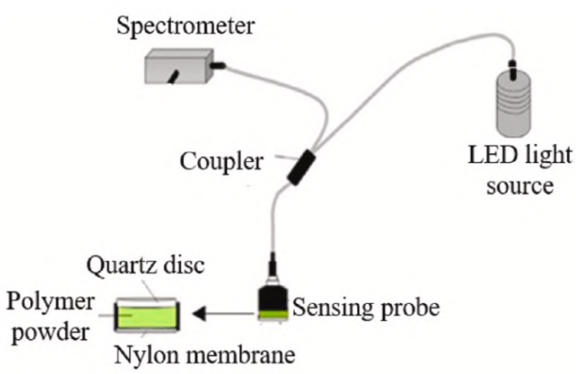


Figure 4-14: Layout of pH sensor based on fluorescent polymer (Fan and Bao, 2021)

Carbonation leads to a reduction of pH value and the onset of corrosion. A fiber optic pH sensor is another type of spectrometer that can be used to monitor pH values ranging between 10-13. The sensor consists of a quartz disc, fluorescent polymer powder and porous nylon membrane. The dye within the sensing probe comes into contact with the surrounding concrete pore solution via the porous membrane. As the pH value is reduced the fluorescent intensity of the dye increases. This correlation between pH and fluorescene intensity can be used for long-term monitoring of pH in concrete (Nguyen et al, 2014).

Both the sensors can be installed by embedded within reinforced concrete before concreting. More research has to be conducted to improve the long-term stability of spectrometer-based sensors. The polymer in the pH sensor is prone to degradation over time in the highly alkaline environment of immersed tunnels. Compared to sensors that measure strain and temperature, sensors based on monitoring Cl^- level and pH value have shorter service life (Fan and She, 2021).

4-4-6 Terrestrial Laser Scanning (3D Point cloud scanning)

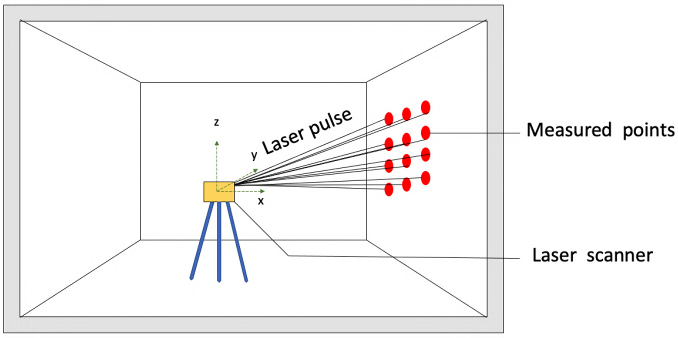


Figure 4-15: Measuring deformation with laser scanner

Terrestrial laser scanning, also known as 3D point cloud scanning, is a non-contact method commonly used in engineering surveying. It provides millions of 3D points with mm-level accuracy and has a very high point density of around 1 million points per second, see Figure 4-15. Laser scanners can be used to scan the entire cross-section of the tunnel throughout its length to appropriately comprehend the deformation behaviour of the immersed tunnel such as joint displacements.

The fundamental concept of this scanner is based on LiDAR (Light Detection and Ranging) technique. LiDAR is used to determine how far each point is from the lens. The laser scanner consists of a transmitter/ receiver of the laser beams, scanning and timing device as shown in Figure 4-15. The laser system sends out a beam of electromagnetic radiation. This radiation is reflected from the surface of the target object and is received by the scanner. The timing device then calculates the flight time Δt. Based on the speed of emitted beam and flight time, distance d can be determined (Wang et al, 2014).

$$d = \frac{c\Delta t}{2} \tag{4-2}$$

where c is the speed of light. Afterward, the relative position of the reflective surface to the device is found by knowing the direction and angle of the light. In addition to coordinate information, Yu et al (2018) claimed that the intensity information of various point clouds can also be extracted. The intensity is the strength of the electronic signal obtained by converting the reflected optical power. This variation in intensity in tunnel surroundings can be used to identify possible areas of water leakage. Artificial intelligence processes the big data collected during routine scans of the cross-section of the tunnel and identifies the damaged areas and leaks present. This reduces the time needed for visual checks and eliminates inefficiencies.

According to Wang et al (2014), the spatial resolution of the laser scanner is $\pm$ 6 mm within a maximum distance of around 50 m. Additionally, the monitoring does not require the installation of temporary reference points and hence is non-contact. To carry out the measurements, the equipment should be fixed in different positions along the tunnel. Moreover, it would require the closure of the tunnel tube to avoid inaccuracies in measurements. Thus, the monitoring frequency has to be limited to yearly intervals. Another drawback of this technique

Figure 4-16: Laser scanner mounted on a van (Bovaida et al, 2012)

is that the computational cost of analysing millions of 3D points is much higher than other techniques. Future developments in computing power and algorithms would help overcome this issue. Advances in mobile laser scanning would enable the monitoring to be done without hindering traffic (Che et al, 2019). Figure 4-16 illustrates a laser scanner mounted on a van.

4-4-7 Pressure Cell

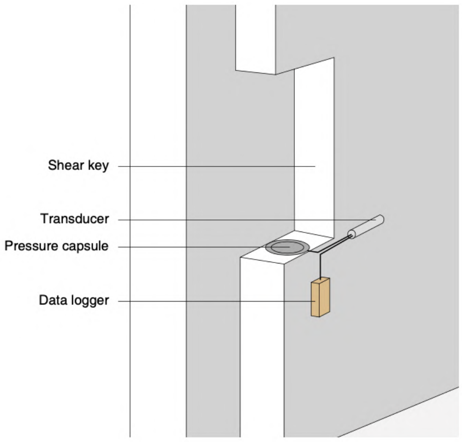


Figure 4-17: Layout of pressure cell at shear key

Pressure cells are sensors that can be used to measure contact force acting on the shear key. For instance, pressure cells were applied in the Honggu tunnel in between shear keys to monitor the contact force (Xu et al, 2017). Among the various types available, concrete pressure cell is appropriate for this purpose. It is made of a flat circular capsule that is sandwiched between two steel discs. This capsule is attached to a pressure transducer by a long stainless tube as shown in Figure 4-17. The pressure transducer has a cable that is protected against external damage and connected to a data logger to provide either manual or real-time monitoring data. The shear key exerts pressure on the cell which is then transmitted through the fluid in the stress capsule to an internal diaphragm present in the pressure transducer. Variations in pressure make the diaphragm deflect, thereby changing the tension of the vibrating wire. This change in tension changes the frequency of the wire. By relating frequency to tension, contact force can be measured (Encardio, 2021). Generally, the structural performance of shear keys is only checked when structural problems arise. This makes it difficult to implement early remediation measures. Hence, a pressure cell makes it possible to take fast and reliable readings without having their output disturbed by external noises. Additionally, these readouts can be done digitally at remote locations.

4-4-8 Hydrostatic level

Hydrostatic Levelling Cells (HLC) is an automated system used for monitoring vertical deformation in structures. For instance, it was used to monitor the uneven settlement in the Honggu tunnel (Xu et al, 2017). The system consists of a data logger, hydrostatic level cells, tubes and a reservoir as shown in Figure 4-18. The series of hydrostatic level cells are connected by tubes containing fluid and air to create a continuous circuit as shown in Figure 4-18. One of the cells acts as a reference cell and is placed outside the influence zone of expected movement. The other cells are placed within the influence zone. The reference cell is located close to the fluid reservoir and is placed at a higher elevation than the other cells to generate a hydrostatic pressure. As the elements experience vertical movements the level of cells subsequently changes. This level change results in a change in pressure relative to the reference cell. The sensor present in each cell measures this pressure. These pressure readings are converted to vertical displacements. In the case of settlements, the cells report increased pressure while for heave it reports decreased pressure (GEO-Instruments, 2019).

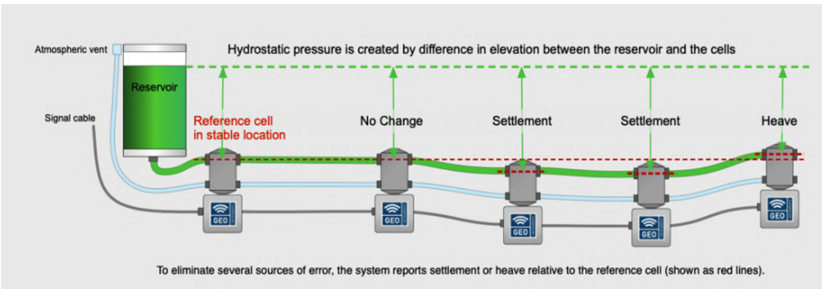


Figure 4-18: Layout of hydrostatic levelling cell (GEO-Instruments, 2019)

They can be installed at the joints to obtain real-time monitoring of differential settlement of segments and/or elements in immersed tunnels. After installation, the system can be run for many years with minimal maintenance requirements (GEO-Instruments, 2022). The system offers a very high resolution of 0.024 mm and has a working range of 20 to 50 cm. Besides, these readings are reliable as it is taken concerning a reference cell and hence avoid error due to reservoir maintenance. The readings can be transferred to cloud servers so that they can be read remotely (GEO-Instruments, 2019). Despite these advantages, the big volume of this system makes it difficult to install. Furthermore, the pressure sensors within each sensor have limited tolerance for minimum and maximum pressure. Hence, it fails to provide a sufficient recordable range if the cells are at large differential elevations (Yin and Huang, 2015).

4-4-9 Multichannel analysis of surface waves (MASW)

This is a geophysical method that can be used to examine the foundation quality by analysing the shear wave velocity of surface waves. The method requires a set of geophones, a seismic source and a hammer. The surface waves are generated by striking the seismic source using a sledgehammer and they are received by geophones, see Figure 4-19. The geophones are placed in a suitable configuration away from the source as shown. A wide range of frequencies is generated by the hammer. Depending on the intensity of frequency, waves are influenced by soil material at various depths. This frequency dependence is called dispersion. The various surface wave surveys generated are converted into velocity and frequency correlations called dispersion curves using this dispersion (Long et al, 2020). Based on the shear wave velocity, V_s, the small strain shear modulus G_0 of soil can be obtained using the following equation:

$$G_0 = \rho V_s^2 \tag{4-3}$$

Here ρ is the density. G_0 is then further used to characterise the underlying soil. The shear wave velocity distribution in the depth range between 10 m and 100 m can be pretty accurately measured by analyzing the amplitude spectra of surface waves of the Rayleigh type (Gavin et al, 2019).

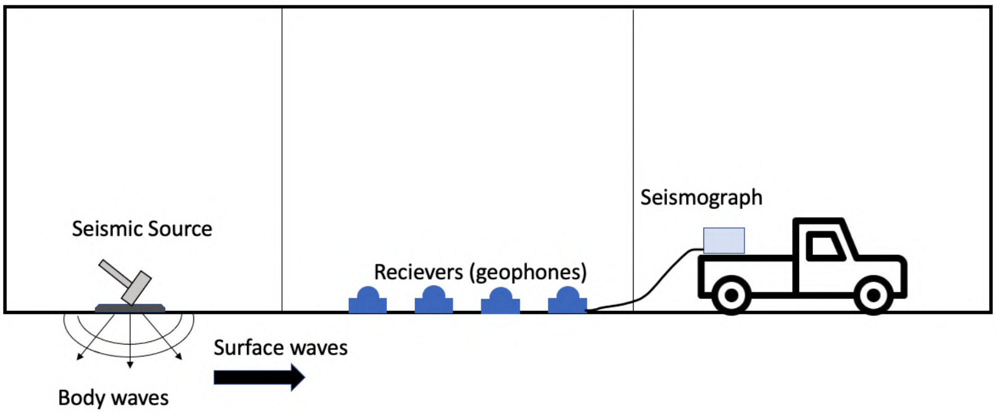


Figure 4-19: Representation of a typical MASW setup

Detailed soil investigation data is often needed to determine the soil types and its corresponding mechanical properties for calculating the magnitude of settlement. It was observed that this data is often unavailable in older immersed tunnels. In such scenarios, MASW can be used to understand the stiffness properties of the underlying soil. Additionally, geological effects such as scour can alter the soil profile. Periodic investigation using MASW at annual intervals would help understand if there are any variations. Finally, this is a quick, cost-effective and non-intrusive investigation technique (Gavin et al, 2019).

4-5 Concluding Remarks

Conventionally, settlements were monitored using total station, invar ruler and strain gauges. Extensometers were used to measure the width of the joint opening. Leakages concrete cracks and corrosion were checked for by visual inspections. In case of large leakage rates, endoscopic and key hole operations were done to further inspect the joints. Upon review of these conventional techniques, it can be concluded that they failed to capture full tunnel behaviour such as contact force at shear keys and transverse displacements. Furthermore, the accuracy of measurements varied depending on the subjectivity of personnel involved in monitoring. Additionally, monitoring often required tunnel tube closure resulting in additional costs and traffic disruption.

Vertical displacement, transverse displacement, joint width, crack width, shear key force, leakage detection, chloride diffusivity, pH in concrete, concrete strain and shear modulus of soil were chosen as the parameters to overcome the defects of conventional monitoring techniques. Strain gauge, extensometer, distributed optic fiber sensor, sellable polymeric fiber optic sensor, pH sensor, optic fiber chloride sensor, terrestrial laser scanner, pressure cell, hydrostatic level and multichannel analysis of surface waves were the potential sensors and monitoring techniques that could monitor these parameters.

Chapter 5

Development Of An Improved Monitoring Plan

In section 5-1, possible remediation measures that can be taken to mitigate the effects of long-term structural problems are given. Finally, an improved monitoring plan that aims to overcome the defects of conventional monitoring strategy is presented in Section 5-2. It describes the accuracy, frequency, position, timing of installation and benefits of the sensors.

5-1 Remediation Measures

5-1-1 Settlements

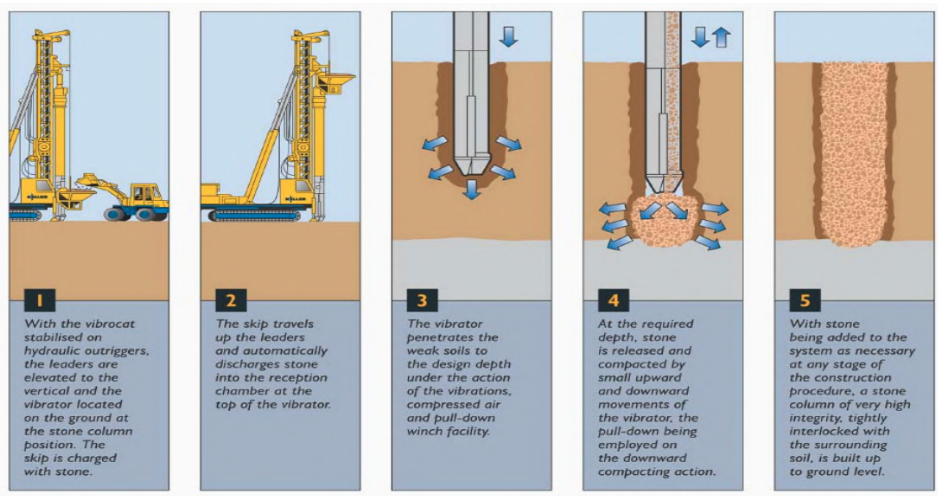


Figure 5-1: Construction sequence of stone column installation (McCabe et al, 2007)

The best strategy to control settlement would to be slow down the settlement rate by improving soil properties. High settlement rates are often found in soft soils and seismic zones. The techniques that are commonly applied are ground replacement, stone columns, soil mixing and sand compaction piles (SCP). Ground replacement is the simplest method where the poor soil is excavated and replaced by better quality soil. This method becomes limited by the depth to which dredging equipment can operate. Stone columns are installed to provide drainage pathways for the dissipation of excess pore water pressure during seismic events and to increase the rate of consolidation settlements. The methodology of installing stone columns is illustrated in Figure 5-1. This method is costly and requires large operating facilities to execute. Stone columns have been installed as deep as 30 meters below the seabed (Lunniss and Baber, 2013).

Figure 5-2: Methodology of soil mixing (Topolnicki, 2016)

Soil compaction also known as soil mixing offers the deepest range of ground improvement of up to 70 m below the seabed. Eight augers are arranged in a square pattern. These augers penetrate the soil and inject cement as it is withdrawn. The methodology is illustrated in Figure 5-2. The mixing of soil with cement improves the bearing capacity of soil and resistance against liquefaction. SCP is used to densify the ground by inserting a vibrating sleeve. Once it has reached a certain depth, the sleeve is withdrawn and sand is released simultaneously during withdrawal. These four methods are executed using a significant amount of empiricism. A robust approach is required to ensure the effectiveness of the foundation (Lunniss and Baber, 2013).

5-1-2 Leakages

Leakages along the tunnel body tend to be small and can be pumped out quite easily. At the expansion joints, the drilling and injection technique mentioned in Leeuw (2013) can be used employed to close the leak. Two holes are drilled at an angle above the waterstop as shown in Figure 5-3. Afterward, a sealant is applied to the holes to seal the leak. This method can be executed quickly but requires the closure of the traffic tube to work safely.

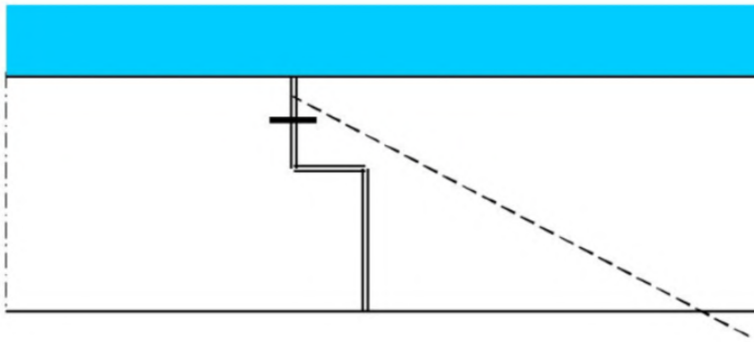
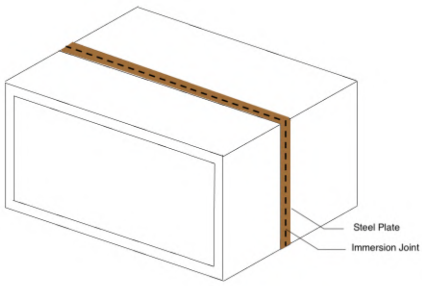
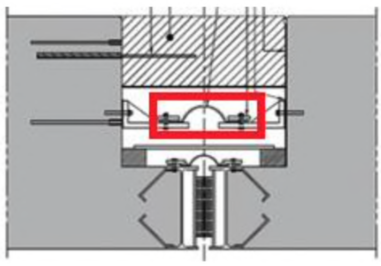


Figure 5-3: Drilling and injection method to seal leaks at expansion joints (Leeuw, 2013)

At the immersion joints, maintenance work is not as straightforward as expansion joints. Research has to be done on which component of the immersion joint needs to be maintained. Removal of ballast concrete, asphalt and installations such as ventilation make it difficult to replace damaged parts of the immersion joint. Moreover, it is not easy to close tunnel tubes for research as it would result in traffic hindrances. Also, removing parts such as the Omega gasket and shear keys makes the maintenance operation riskier. Shear keys are crucial to prevent differential settlement between elements. Therefore, the removal of shear keys requires construction of temporary shear keys (Van Montfort, 2018).



a)



b)

Figure 5-4: a) Protective steel plates outside immersion joint b) Additional watertight layer above Omega seal (Van Montfort, 2018)

As it is difficult to replace the Gina gasket, the only option would be to freeze the soil behind the Gina gasket to create a temporary watertight layer. Moreover, the lack of space available at the Gina gasket makes the option to replace it less feasible. A possible alternative would be to place external protection steel plates at the joints on the roof and sides of the tunnel to prevent water and sand leaks as shown in Figure 5-4 a). If the Omega seal is identified to be the damaged component, either part of the Omega seal can be replaced or the current state has to be conserved. Omega seal can be replaced only if it is proven that the Gina gasket is watertight. If the Omega seal is prone to damage in the future, corrosion inhibitors can be used to preserve the current state of the seal. The latter option is cheaper than the former but is less reliable. If it is difficult to achieve watertightness at the Omega seal another watertight gasket layer can be placed above the Omega seal as shown in Figure 5-4 b). This method eliminates the need for future maintenance on the Omega and Gina gasket. However, it is a costly method and requires the third layer to function well (Van Montfort, 2018).

5-1-3 Concrete cracking

The amount of work required to repair the cracks depends on the size and depth of penetration of cracks within the concrete section. If cracks are small then the developments of these cracks has to be monitored using strain gauges (SEG, 2019). For most cases of cracks formed due to frost-thaw the repair work is minimal and can be fixed by removing the top layer by a few millimetres. Once removed the rough surface is smoothened with cement mortar. On the other hand, for ASR cracks the only remedy would to stop them from progressing is to make the structure dry. This is not possible for immersed tunnels so the focus should be prevention by using non-reactive aggregates in the mix design. As sulphate attacks originate from polluted environments, removing and replacing this environment will help prevent further deterioration. However, if the damage is expensive the affected concrete has to be replaced (COB, 2021). Since the size of cracks generated due to differential settlement often increases with settlement rate ground improvement techniques help to control the damage.

5-1-4 Joint gap opening and closure

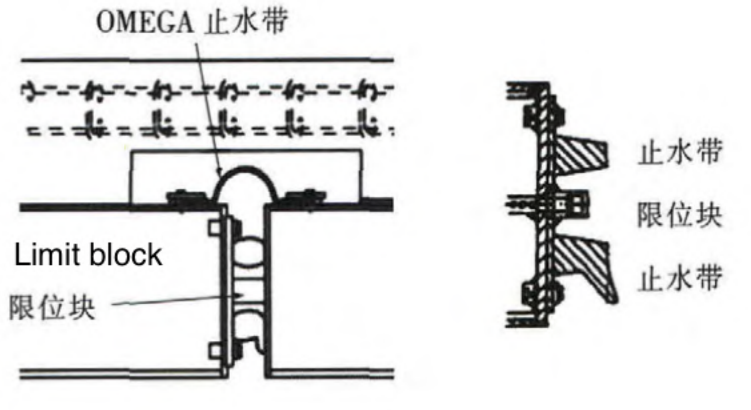


Figure 5-5: Limit block used in BART tunnel (Bai and Lu, 2016)

Due to excessive compression and opening of the joints, the rubber gaskets can get damaged. To ensure the watertightness of the joint is maintained the joint movement must be limited. The compression can be limited by using steel limit block as used in Bay Area Rapid Transit (BART), see Figure 5-5. On the other hand, the joint opening can be limited by using pre-tensioned cables across the joint or injecting high pressurised grout/ epoxy resin to fill the gap (Bai and Lu, 2016).

5-1-5 Corrosion

For chloride-induced corrosion, the possible mitigation measures are applying a coating to the reinforcement, the concrete cover repair by either replacing or cleaning of reinforcement. However, the concrete cover has to be replaced in the case of carbonated concrete cover. Before placing a new concrete cover application of an alkali-based primer within an epoxy-based resin to the reinforcement is suggested. Both the mentioned measures can be combined with cathodic protection to prevent further corrosion. In case of electro galvanic corrosion, the corroded part have to be replaced and plastic rings or plates can be used to prevent contact between the rebar cage and steel end frame (COB, 2021).

5-2 Improved Monitoring Plan

The proposed monitoring plan aims to overcome the limitations of the conventional plan while maintaining tunnel durability and safety of its users. Moreover, monitoring data from sensors and monitoring techniques used would provide whether preventive and/or remedial measures can be carried out. The aspects that are monitored in the proposed monitoring plan are deformation, contact force, leakage and corrosion. To measure these aspects the parameters to be monitored are crack width, joint width, joint rotation, vertical displacement, transverse displacement, concrete strain, leakage detection, chloride diffusivity, pH level, shear key force and shear modulus of soil.

Initially, structural cracks are inspected in periodic intervals by visual inspection. If cracks are detected then the development of cracks widths is to be monitored in real-time using a vibrating wire strain gauge. These sensors are cheap and can be used for long-term monitoring of crack widths from remote locations. According to Modares and Waksmanski (2013), these sensors have an accuracy of ± 0.1 to ± 0.5 %, a resolution of 1 $\mu\varepsilon$ and a sensing range of ± 3000 $\mu\varepsilon$. As differential settlements often exceed the initially predicted value it can generate concrete cracks. Based on the size of crack widths the contractor can carry out remediation measures on time. However, these sensors can only be used for detecting cracks present within the interior of existing and new tunnels.

Joint widths can be monitored in real-time using fiber optic extensometers. The extensometers are placed in a protective casing and the steel wire is prestrained across two points close to the joints. Additionally, by placing two extensometers close to the top and bottom of the wall along the joint, joint width can be translated into joint rotation measurements. The top and bottom of the wall are chosen since joint rotations would be the greatest at these locations due to differential displacement. For existing tunnels, these sensors can be installed during the annual maintenance work window. On the other hand, for new tunnels, it is best to install once initial settlements have taken place after the connection between all the elements has been made. This sensor provides the contractor information on the deformation state of the rubber gaskets and provides a preliminary warning on the watertightness of joints in case the joint width limits have been surpassed. Moreover, the joint widths can be measured remotely without having to be on-site in real-time. According to the websites of Zwick Roell (n.d.) and Industrial Physics (n.d.), contact extensometers are durable devices that can work up to gauge lengths of 100 mm and provide resolution of 0.1 mm and accuracy Class 0.5 to ISO 9513.

DOFS can be applied at the tunnel joints for measuring vertical displacements as well as joint widths. This location is chosen for installation because displacements tend to concentrate at the joints. Transverse displacements are measured by installing the sensors on the roof of the utility tube. In the absence of a utility tube, the sensor can only be installed during traffic tube closure. It should be noted that the layout varies for expansion and immersed joints. Vertical and transverse displacements can be monitored at hourly intervals by using DOFS. Moreover, DOFS has an accuracy of 2 $\mu\varepsilon$ and a resolution of 0.2 m up to 2 km and 0.5 m up to 25 km (Zhang and Broere, 2022). These sensors can be applied for both existing and new tunnels in a similar timing as fiber optic extensometer. These sensors provide the client and contractor a way to measure displacements remotely, eliminate observational errors using manual levelling instruments and avoid the need for traffic tube closure. Additionally, it helps the contractor to keep a better check on tunnel alignment.

For leakage detection behind the Omega seal, the built-in water pressure gauges can be used. If the internal water pressure between Omega and Gina gasket is not high that means there is no constant leakage and the watertightness performance of the Gina gasket is not compromised. To check for leakages in front of the Omega gasket, a swellable polymeric fiber optic sensor can be installed. These sensors can be installed in existing tunnels during the annual maintenance check by removing the concrete cover. For new tunnels, these sensors have to be installed before the infill concrete is placed. The main benefit of these sensors is that they allow the contractor to detect the presence of leakages in real-time. This eliminates the need for intrusive inspection techniques resulting in traffic hindrance. However, the accuracy of this sensor was not found during the literature review.

It is not possible with currently available sensors to capture the complete corrosion process during the operational period of the immersed tunnel. As a result, a combination of sensors is used to monitor the rate and onset of corrosion in real-time. DOFS can be tightly wound around the reinforcement before concreting is done in the construction dock. These sensors once installed measure the concrete strain as rust is formed. Depending on the amount of rust formed, concrete strain increases which allow the contractor to detect the rate of localised corrosion along the reinforcement. Moreover, these sensors are inexpensive to install and allows the contractor to accurately determine the rate of corrosion. Optic fiber chloride sensor and fiber optic pH sensor are two sensors that can be used to measure the onset of corrosion in real-time. According to Zhang and Zhou (2018), fiber optic pH sensors have an accuracy of up to 0.2 pH. The accuracy of the chloride sensor was not available during the literature study. A network of these sensors can be created at discrete locations spread throughout the reinforcement before concreting operations in the dock. The main benefit of these sensors is that they can detect corrosion before it becomes visible as spalling or concrete cracking on the tunnel interior. This helps the contractor to carry out quick remediation measures and hence, substantially reduce costs. The main drawback of the available corrosion sensors is that they can only be installed in future tunnels.

The shear key force is monitored in real time by placing a pressure cell on the contact surface between the shear keys at the joint as shown in Figure 4-17. On the website of Geokon (n.d.) it was specified that pressure cells can work within ranges of stress up to 5 MPa and have an accuracy of $\pm 0.025\%$ F.S. The flexibility in the installation of these sensors differs for concrete and steel shear keys. For concrete shear keys, the cell is placed on the female shear keys once the elements have been immersed and before the construction of male shear keys to connect the elements. Whereas, the steel shear keys can be easily removed even after immersion to place the cell. Therefore, pressure cells can be used in both existing and new tunnels with steel shear keys whereas this installation is only applicable to new tunnels in the case of concrete shear keys. Since shear keys play a key role in limiting differential displacements monitoring the performance of shear keys helps the contractor to either prevent or take quick remediation action before significant damage occurs.

A rough idea of foundation soil stiffness can be obtained using MASW investigation at annual intervals. Compared to direct borehole investigation methods MASW provides an accuracy of 85-90%. The resolution of soil properties can be increased by reducing the spacing between the geophones. For instance, a spacing of 0.3 m would have a vertical resolution of 0.6 m. This would compromise obtaining information about deeper soils (Gavin et al, 2019). Furthermore, if a significant change in soil stiffness is observed, in-situ CPT testing has to be done close to the immersed tunnels to gather detailed information on soil properties. The geophysical investigation can be done by partially closing the bike lane as shown in Figure 4-19. If bike lanes are not available partial closure of a tunnel tube is required to carry out measures. This method is especially beneficial when soil investigation data of soil underneath the tunnel is unavailable to the concerned parties. Furthermore, it can be used to determine whether ground improvement has to be performed in the case of soft or liquefiable soils or scenarios where soil profile has been altered due to scour effects. This method can be applied to both existing and new tunnels as it is non-intrusive.

Figure 5-6 and 5-7 illustrates the locations where sensors specified in the aforementioned plan are to be installed. Depending on the budget available to the contractor the number of sensors adopted can vary. For a contractor having a limited budget, based on a broad perspective of costs associated the monitoring strategy and remediation measures to be adopted are given in Table 5-1. If extra budget is available the additional strategy and remediation measures that are to be implemented are given in Table 5-2.

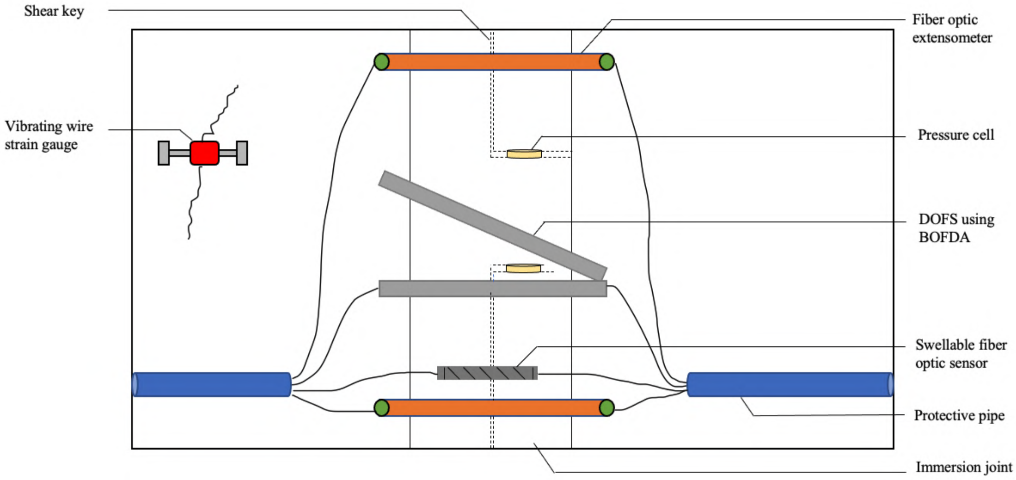


Figure 5-6: Side view of layout of sensors at an immersion joint

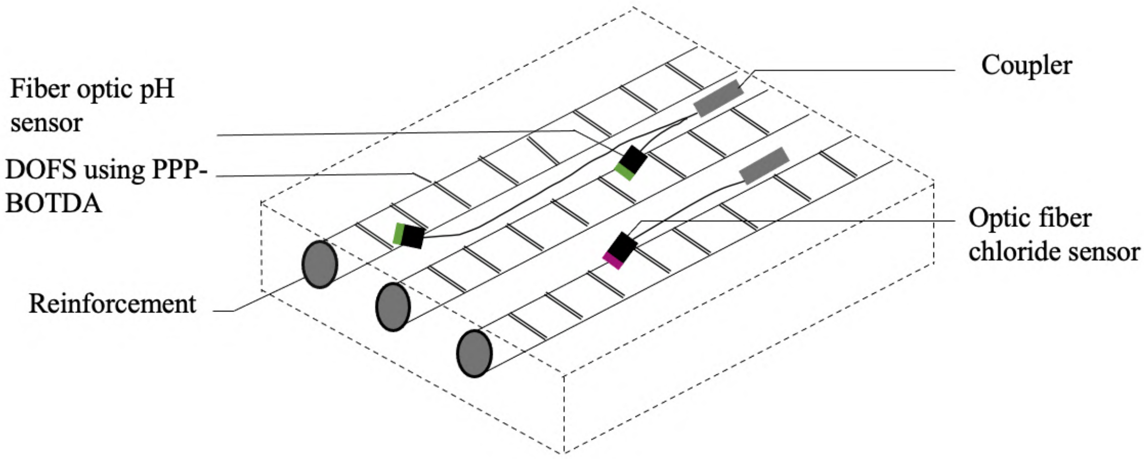


Figure 5-7: Layout of sensors embedded within tunnel section

Table 5-1: Overview of monitoring strategy and remediation measures specific to each structural problem for limited budget

Structural Problem	**Strategy to assess problem**	**Parameter monitored**	**Threshold Limit**	**Remediation measures**
Concrete cracking	• Visual inspection to check for cracks in tunnel interior • VWSG attached across crack by adhesion to monitor crack development	Crack width	0.25 mm	Concrete replacement Replacing polluted environment
Joint gap opening/ closure	• Two fiber optic extensometers are prestrained across two points along top and bottom of wall joint.	Joint width	10 mm	Steel limit block Prestressed cables across joint Injecting resin to fill gap
Excess deformations	• DOFS is applied at joints along wall and roof to measure deformations along vertical, transverse and longitudinal direction • Shear loads acting on shear keys are measured using pressure cells to ensure shear keys do not fail. • Geophysical soil investigation is carried out using MASW to understand soil behaviour.	Differential settlement Transverse displacement Joint width Shear key force Shear modulus	5 mm 10 mm 10 mm 19 MN (steel shear key) 24 MN (concrete shear key) –	Ground replacement Stone columns Soil mixing Sand compaction piles

Table 5-2: Overview of additonal monitoring strategy and remediation measures specific to each structural problem

Structural Problem	**Strategy to assess problem**	**Parameter monitored**	**Threshold Limit**	**Remediation measures**
Leakage	• Leaks behind the Omega seal measured using built in pressure gauges. In front of Omega, swellable polymeric fiber optic sensor is used	Leakage detection	34.2 dB	Drill and injection Replacing gaskets Additional gasket External steel plates
Corrosion	• DOFS is tightly wound around rebar before concreting to determine pitting corrosion rate by measuring concrete strain • Optic fiber chloride sensor and fiber optic pH sensor are used to determine onset of corrosion.	Concrete strain Chloride diffusivity pH	0.0002 7 x 10 $^{-12}$ m^2/sec 11.3 -12.1	Coating Concrete cover repair/ replacement Cathodic protection Plastic plates/ rings

Chapter 6

Conclusions and Recommendations

The goal of this research is to optimise the design and monitoring of immersed tunnels. This in turn improves the life cycle operation of tunnels. To achieve this goal, the research was divided mainly into three parts.

Firstly, the common structural problems encountered during the operation period were identified by reviewing the literature of past projects. They are excess deformations, joint gap opening/closure, concrete cracking, corrosion and leakage. These problems were further analysed based on construction technique, geotechnical and structural engineering to find the key factor causing these problems. The deformations in the vertical direction were mostly affected by the stiffness of the subsoil conditions. Whereas, poor backfilling operations caused deformations in the transverse direction. Deformations in the longitudinal direction led to opening and closure of joints. These were influenced by seasonal temperature variations. All of these deformations cause tunnel bending, which results in concrete cracking. Besides, these movements reduce the contact force of rubber gaskets at the joints which further reduces watertightness. This leads to the formation of entry channels for dirt and water. Furthermore, chloride ingress and carbonation were determined to be the main reasons for the corrosion of steel components.

In the second part, an inventory of recent design changes since 2000 was formulated based on element material, cross-section design, transverse prestressing, foundation treatment, waterproofing and joint formation. This helped to understand the impact of these changes in mitigating structural problems. It was observed that there were no recent changes in the foundation treatment method. For element material, semi-rigid elements were developed which were partially rigid due to permanent prestressing and partially flexibly due to the use of segments. These elements improved the deformation capacity of expansion joints enabling the tunnel to large differential settlements. Under cross-section, an increase in the adoption of utility tubes was noticed. This tube primarily serves as an escape tube and installation of mechanical and electrical equipment. Nevertheless, these tubes reduced the shear stress acting on the inner walls which helps reduce the possibility of crack formation. Due to an increase in traffic, the demand for wider tunnels is increasing. Transverse prestressing increase the bending resistance and slenderness of concrete slabs, assisting in meeting this demand. The main improvements in waterproofing the tunnel are full section casting, cathodic protection and corrosion resistant steel. The entire element is constructed in a single pour in full-section casting, thereby, eliminating through-section cracking during construction. Cathodic protection and corrosion-resistant steel are two measures aimed at mitigating corrosion. Cathodic protection helps to mitigate reinforcement corrosion of attached steel components for approximately 15-20 years. On the other hand, corrosion-resistant steel is more expensive but offers a corrosion prevention period of 40 years. Seismic, bellow and crown seal joints with high deformation capacity were developed as alternatives for the traditional immersed joint which fails to maintain watertightness at large water depths and seismic zones. At the expansion joint, the double seal serves as an additional watertightness barrier. On the other hand, new reinforcement detailing is used to strengthen the expansion joint. V-wedge, key and deployable elements were developed to reduce construction time and improve the watertightness as an alternative to traditional closure joints. Finally, steel and discrete shear keys have much higher shear capacity than conventional shear keys. This enables the tunnel to withstand large deformations.

Finally, an improved monitored plan was developed to serve as an early warning system for structural problems. It was noticed that the conventional monitoring plan lacked frequency, accuracy, capturing complete tunnel behaviour and required full or partial closure of traffic tubes. The parameters that were to be monitored by the sensors were identified as vertical displacement, transverse displacement, joint width, crack width, concrete strain, leakage detection, chloride diffusivity, pH in concrete and shear key force. Subsequently, threshold limits were set as $\pm$5 mm, $\pm$10 mm, 10 mm, 0.25 mm, 0.0002, 34.2 dB, 7 x 10 $^{-12}$ m^2/sec, 11.3 -12.1, 19 MN for steel shear key and 24 MN for concrete shear key respectively. The main monitoring aspects to be monitored are deformations, and contact force at shear keys. Placing DOFS along the wall joints halfway up the tunnel and the roof joints helps to measure deformations in vertical, transverse and longitudinal directions. As differential settlements also result in joint rotations, placing two fiber optic extensometers along the wall joint helps to measure joint width which can then be translated into joint rotations. Besides, differential movements can also cause cracks. VWSG aids in monitoring the development of these cracks. As shear keys play a major role in limiting differential displacements, pressure cells are to be placed between keys to monitor acting shear forces. Additionally, geophysical investigation using multichannel analysis of surface waves can be used to determine underlying soil properties. If the contractor is not restricted by a limited budget, the additional aspects to be monitored are corrosion and leakage. Even though corrosion is ever present, the applicability of available corrosion sensors such as pH sensor, chloride sensor and DOFS is restricted to new tunnels. It is advised to include corrosion sensing in the main monitoring plan if non-destructive techniques not influenced by surrounding effects are developed. For detecting leaks, swellable polymeric sensors are to be placed in front of Omega seals. All the aforementioned sensors offer real-time monitoring except for MASW. Other than concrete cracking and leakage detection, the sensors mentioned in the improved monitoring plan can detect changes before the structural problem arises.

The sensors can also be used to check how their impact on failure modes. The effectiveness of the semi-rigid elements, discrete/ steel shear key, seismic, bellow and crown seal joint in withstanding differential displacements can be determined by using DOFS and pressure cells. VWSG can be used to track how well transverse prestressing and full-section casting prevents crack formation. A fiber optic extensometer and swellable polymeric sensor can be used to check if the watertightness is maintained in seismic/ bellow joints, double seal, V wedge, key and deployable element. Finally, the feasibility of cathodic protection and corrosion-resistant steel in preventing reinforcement corrosion can be observed using DOFS, fiber optic pH sensor and optic fiber chloride sensor.

The improved monitoring plan helps to carry out quick remediation measures once structural problems emerge. Ground improvement techniques are highly effective in controlling settlements. However, the methodology of this technology relies on empiricism. Installing steel limit block, filling the gap by injecting high pressurised grout or epoxy resin and using prestressed cables across the joints helps limit joint gap opening/closing. The measures differ based on the size and depth of concrete cracks. For small cracks, the top layer can be removed by a few millimetres and smoothened with cement mortar. For deep cracks, concrete replacement and removal of the polluted environment outside the tunnel are advised. Corrosion is inevitable for all reinforced structures. In the case of carbonated concrete, the cover has to be replaced whereas applying a coating to the reinforcement delays the onset of choride-induced corrosion. Finally, leaks at the expansion joints can be sealed by drilling and injection methods. For immersion joints, replacement of rubber gaskets, installing an additional gasket or protective steel plates outside the joints are advised.

6-1 Recommendations

Field and numerical investigation of the effectiveness of recent design changes in addressing the structural problems has to be examined.

The service life of sensors tend to be shorter than the design life of immersed tunnels. As a result, the possible replacement of damaged sensors has to be considered before installing the sensors. Moreover, long term behaviour of all sensors considered in this report has to be examined. Current work on long-term stability of sensors is limited to laboratory experiments. However, field applications would subject these sensors to various scenarios such as mechanical loading, wet/dry cycles, thermal gradients and high pH values. Therefore, work has to be done to extend and assess the service life of current sensors.

Most sensors that are currently being developed are suitable only for new immersed tunnels rather than existing ones. It is essential to develop sensors that can monitor corrosion without using destructive techniques. Non-destructive techniques such as infrared thermography, ground penetration radar and magnetic flux detection with a magnetometer could provide a potential solution. These techniques have not been discussed in this report. Future research has to be done to limit the effect of factors such as moisture content, magnetic interference from the surrounding environment and concrete cover depth on the monitoring data.

A large number of sensors have been deployed for monitoring tunnel behaviour. This increase in sensors not only increases the complexity of installation but makes them more prone to damage. The development of a single sensor that can monitor several parameters can help reduce cost, difficulty in installation and save time.

Improvements in computation power and the development of mobile laser scanners can help measure concrete strains, crack widths and displacements accurately. Furthermore, relating the intensity variation of reflected signals could aid in leakage detection.

Cost-benefit analysis of sensors and monitoring techniques mentioned in the improved monitoring plan has to be performed. This would help fuel further research in the monitoring immersed tunnels and convince contractors of their usefulness.

Appendix A

Analysis Of Structural Problems

This Appendix aims to provide the reader a brief summary of possible reasons for common structural problems based on construction method, geotechnical engineering, and mechanical and structural design flaws. This summary form the basis for analysing feasibility of design changes.

Table A-1: Reasons for structural problems based on construction technique, geotechnical engineering and design flaw

Problem	Construction	Geotechnical	Design Flaw
Excess deformations	• Poor backfilling operations: Due to poor backfilling, tunnel body can be displaced transversely. • Tunnel foundation construction: The construction procedure followed for different foundation types can affect the amount of silt deposited which causes differential settlement. • Trench dredging method: For different soil types and water depths, different equipment are used. The tolerance level achieved if poor can cause differential settlement.	• Sub-soil conditions: Stiffness of soil affects magnitude of settlement. For consolidated sands and silts, immediate settlement occurs. For clays large creep settlements takes place. Spatial variability of soil causes differential settlements. • Dynamic loading: Earthquakes cause liquefaction of sand layer leading to large settlements. Cyclic tidal and wave loads changes pore pressure causing settlement in silt and clays. • Current and bed loads: In waterways with fast currents and high bed loads, siltation effect can be higher.	• Surcharge loading: Due to surcharge loads above immersed tunnels, surcharge loading increases. Differential settlements occur if loading is large • Tunnel shape: Contact between foundation and tunnel is low for circular steel shell while 100% contact maintained for rectangular concrete tunnel shape. Reduced contact leads to higher stress on soil which causes higher settlements during filling stage. • Element material : Steel tunnel can tolerate more differential settlement than concrete tunnel due to ductile nature while concrete is brittle.

Problem	Construction	Geotechnical	Design Flaw
Joint gap opening/ closing	• Low gasket compression: During immersion, if adequate gina gasket compression is not achieved then deformation capacity of immersion joint reduces making joints to open.	• Spatial variability: Differential horizontal movement between adjacent elements/ segments due to varying soil stiffness properties makes joints open wider. • Tunnel body rotation: For large rotations loss of contact of gaskets at joints which widens joint. • Soil intrusion: When the concrete shrinks at the expansion joints, soil fills in roof joint and later gets compressed when concrete expands. Over time it causes joints to be wider and rubber gaskets to stretch.	• Thermal movement of concrete: Tunnel not connected to piles are prone to longitudinal movement. Varying temperature causes expansion in summer and contraction of concrete in winter. This results in joint opening/ closing. • Deterioration of rubber gaskets: In long term due to creep, relaxation and cyclic movement of joints, deformation capacity of gasket reduces over time. This causes joints to open as prestressing of Gina gasket is reduced.
Concrete cracking	• Poor backfilling operations: Due to this transverse displacements may occur resulting in increased stress at the joint causing joint and shear key damage. This reasoning has not been proven yet as monitoring of transverse displacement is rarely done. • Poor workmanship: This makes concrete porous and prone to surface cracks due to frost-thaw cycles.	• Spatial variability: Differential settlement can lead to additional load in the concrete collar which can generate cracks in collar • Polluted environment: Such areas can be a source of sulphates resulting in sulphate attacks leading to map cracking.	• Temperature gradient: Gradient across section contribute to transverse bending moments and creates longitudinal cracks. During winter when concrete shrinks, tensile stresses are created in concrete and shrinkage cracks could emerge. • Inadequate reinforcement detailing: Inner walls contain lesser reinforcement compared to outer walls. This makes inner walls prone to cracking when shear stress is large. • Concrete mix design: Inappropriate selection of aggregates can make it susceptible to ASR cracks, frost-thaw damage.

Problem	Construction	Geotechnical	Design Flaw
Corrosion	• Poor workmanship: Improper welding of steel membrane in monolithic tunnel creates a new leakage route. This leads to intrusion of corrosive components. If reinforcement cage and steel frame not properly separated, electro-galvanic corrosion could take place. Finally, poor concreting can increase permeability of concrete providing an easy route for chlorides to penetrate	• Chloride ingress: Corrosion occurs through chloride ingress from external and internal environment • Carbonation of concrete: In long term, exposure of concrete to inside atmosphere can make it lose alkalinity around reinforcement causing general corrosion. • Exposed components: Long term exposure of internal steel components causes general corrosion. • Polluted buried environment: Presence of corrosive material such as SRB in buried environment can cause pitting corrosion.	• Deterioration of protective coatings: Coatings of steel frames deteriorate in long term leading to increased corrosion rates. • Inadequate membrane properties : Poor adhesive and durable properties of external membrane applied for monolithic elements. This creates a route for chloride ingress. • Concrete mix design: Presence of chlorides in concrete mix causes early onset of reinforcement corrosion.
Leakage	• Poor workmanship: If concreting not done properly air bubbles form around rubber profile. This allows water to pass around rubber profile. • Poor placing operations: Possible damage to external membrane during placing can allow water to bypass the membrane	• Spatial variability: Spatial variability causes differential settlements which causes joints to open, Gina gasket to lose prestressing and clamping force of omega seal is reduce. Several leakage paths are formed. • Soil intrusion: Soil can deposit on the roof joint over time. Due to cyclic movement soil compress and pushes Gina gasket inward. Water leaks through Gina gasket.	• Relaxation of gasket: In long term due to relaxation of Gina gasket prestressing is reduced allowing for water to leak.

Appendix B

Overview Of Recent Design Changes

This Appendix describes how recent design changes are related to common structural problems. Moreover, improvements and structural changes brought about by each design change to mitigate these problems.

Table B-1: Structural problem addressed along with improvements and structural changes introduced by recent design changes

Design change	Structural problem addressed	Improvements	Structural changes
Semi rigid elements	• Segmental tunnel used less in seismic zone and poor geologic conditions due to low deformation capacity. It would lead to high differential settlements	• Can withstand higher differential settlements • Reduced construction time and cost	• Increases shear capacity and bending rigidity of expansion joints of segmental concrete tunnels.
Utility tube	• Reduced number of partition walls leads to increased shear stress acting on the inner walls. This could lead to concrete cracking.	• increases bending moment and shear capacity of joints provide an escape route • Location for installing sensors	• Decreases span and thickness of slab • Reduces reinforcement quantity.
Transverse prestressing	• Bending moments and span deflection increases as tunnels becomes wider. • If reinforcement higher than reinforcement ratio limit, compression capacity of concrete reduces. • These ultimately result in concrete cracking.	• Can construct wider tunnels instead of deeper tunnels to meet rising demand • Reduces overall construction cost.	• Increases bending capacity of concrete slabs and makes slabs more slender

Design change	Maintenance problem addressed	Improvements	Structural changes
Full section casting	• Concreting of elements when done in multiple pours, leads to formation of through-section cracks at joints between walls and slabs. • The through-section cracks form leakage paths.	• Prevents formation of through-section cracks and leakage by concreting in one single pour. • Eliminates need of cast-in cooling pipes.	• Restraint to structural movement during concreting is avoided.
Cathodic protection	• Environment inside and outside of tunnel can induce reinforcement corrosion through chloride ingress and carbonation. This reduces concrete durability and strength • Presence of cracks will further increase	• Prevents reinforcement corrosion.	• Anode attached to steel reinforcement creates a corrosion cell. • Anode corrodes and reinforcement stays intact.
Corrosion resistant steel	• Environment inside and outside of tunnel can induce reinforcement corrosion through chloride ingress and carbonation. This reduces concrete durability and strength • Presence of cracks will further increase	• Ensures reinforcement corrosion does not begin during concreting and after immersion.	• Corrosive materials cannot attack reinforcement directly. Thus, delays initiation of corrosion
Seismic joint	• Traditional joints are prone to large joint openings in highly seismic zones. This leads to leakages	• They can withstand large differential settlements induced by large seismic loading in highly seismic zones. • This joint also limits the amount of joint opening to ensure watertightness	• High tensile bars provided at joints to make joint continuous. These allows limited joint opening by straining.

Design change	Maintenance problem addressed	Improvements	Structural changes
Bellow joint	• Traditional joints are prone to large joint openings in highly seismic zones. This leads to leakages • At great water depths achieving a good connection between elements is difficult resulting in joint openings.	• Similar to seismic joints, bellow joints limits joint opening and helps absorb large differential settlements. • It ensures tensile bars are not damaged during rapid opening and closing by using steel plates. • Steel plates preformed in factories helps to achieve a good connection between elements.	• Wave shaped steel plate used to ensure joint opens and closes smoothly during earthquakes.
Crown seal joint	• At larger water depths, transverse settlements cause leakages at immersion joints.	• Effective in absorbing differential settlements and transverse displacements.	• Immersion joint is made continuous • Double omega seal used to provide higher watertightness • Prestressed bar across joint to limit joint opening by straining.
Double seal	• Due to joint openings, improper concreting and poor functioning of injection pipes traditional single seal expansion joints fails to maintain watertightness. • Secondary barrier such as secondary injection tubes, ACME seal loses functionality at large differential displacements and water depths.	• Omega seal provides an additional secondary barrier against leakage in case the waterstop fails. • The omega seal can withstand large differential settlements, joint openings and water pressure due to its high tensile capacity	• expansion joint consists of an injectable waterstop on the outside and omega seal inside of the joint
Reinforcement detailing	• Inadequate reinforcement detailing at tooth resulted in fracture at expansion joints during large differential settlements	• The new reinforcement detailing was able to withstand own weight and support forces of adjacent segment. This helps prevent fracture.	• Improved reinforcement detailing used to strengthen tooth at expansion joint.

Design change	Maintenance problem addressed	Improvements	Structural changes
V wedge	• Construction of closure joint is a long complex process. • If wedges not installed properly, compression is not maintained at joints and watertightness is compromised.	• Avoids use of external underwater formwork and reduces diver activity. • Ensure watertightness by using V block to maintain compression of rubber gasket.	• The hydrostatic pressure between base and roof slab used to maintain segment stability. • Dead weight of V-block used to compress rubber gasket.
Key element	• Special immersed equipment needed to construct closure joint. • Misalignment can result in poor installation accuracy. This results in joint opening and leakage.	• No special immersion equipment required and reduces construction time. • Uses a stretch rubber waterstop to provide greater tolerance during immersion.	• Stretch rubber waterstop is attached to steel frame of element adjacent to key element. Mortar is pumped into waterstop to make connection between elements
Deplo-yable element	• Misalignment can result in poor installation accuracy. This results in joint opening and leakage. • In long term Gina gasket can loses compression value due to relaxation.	• Reduces construction time and can be executed in a day • Construction process is reversible • Provides a strong underwater connection to maintain watertightness	• Size of closure joint element remains smaller during immersion. Once immersed, deployable sections extend to make connection with adjacent element • Jacks used at joints to ensure gina gasket compression is maintained.
Steel shear key	• During large differential displacements concrete shear keys experience brittle failure • Complexity in carrying out in-situ concreting operations reduces capacity of shear keys and can leads to cracks	• It can accommodate large differential settlement without failing. Ductile nature of steel prevents brittle failure of keys. • Provides higher shear capacity than conventional shear keys. • Provides higher accuracy and ease of installation at greater water depths as keys can be preformed and installed on site.	• Three part shear key which are attached to the element ends using bolts. • Significant reinforc-ement used within keys to withstand lateral eccentric loading and compensate for shallow depth of keys. • Memory bearing placed between keys to reduce eccentric loading effect.

Design change	**Maintenance problem addressed**	**Improvements**	**Structural changes**
Discrete shear key	• Difficulty in inspecting shear keys for cracks as traditional shear keys are positioned in the outer walls. • Generally, shear keys are ineffective in shear load transfer for large slab widths due to shear lag effects and reduced size. This results in increased shear stresses on walls leading to potential shear cracks.	• Maintenance can be carried out as keys are placed inside the tunnel. • Provides highest shear capacity among available shear keys • Shear capacity is identical in vertical and horizontal direction making it suitable for seismic zones.	• Location of keys within slabs and walls ensure effective shear transfer across joints. • Shear capacity is provided by four shear keys instead a single shear key • Size of shear keys are larger than the other shear keys.

Appendix C

Catalogue Of Immersed Tunnels In The Past Two Decades

This Appendix provides an inventory of completed and ongoing immersed tunnel projects in the recent two decades. Project information of these tunnels were reviewed so as to collect detailed information on design changes.

Table C-1: List of completed and ongoing (u/c) immersed tunnels since 2000

Name	Location	Open Year	Element type	Cross section	Foundation type
Oresund Tunnel	Denmark	2000	Segmental concrete	Rectangular with a central escape tube	Gravel
Chang Hong Tunnel	China	2002	Monolithic concrete	Rectangular with a central escape tube	Piled and grouting
Niigata	Japan	2002	Monolithic concrete	Rectangular with two escape tubes on sides	Grouting and gravel
Aktion Preveza	Greece	2003	Segmental concrete	Rectangular without an escape tube	Grouting and gravel
Fort Point Channel	USA	2003	Segmental concrete	Rectangular without an escape tube	Piled
Warnow tunnel	Germany	2003	Segmental concrete	Rectangular without an escape tube	Sand flow

Name	Location	Open Year	Element type	Cross section	Foundation type
Shanghai outer ring road tunnel	China	2003	Monolithic concrete	Rectangular with two central escape tubes	Sand flow
Second Benelux Tunnel	Netherlands	2003	Segmental concrete	Rectangular with a central escape tube	Gravel
Vltava Tunnel	Czech Republic	2004	Segmental concrete	Rectangular without an escape tube	Gravel
Caland Tunnel	Netherlands	2004	Monolithic concrete	Rectangular with a central escape tube	Gravel
Dordtsche Kil (HSL)	Netherlands	2005	Segmental concrete	Rectangular with two central escape tubes	Sand flow
Oude Maas (HSL)	Netherlands	2005	Segmental concrete	Rectangular with two central escape tubes	Sand flow
Shin-Wakato	Japan	2007	Steel concrete sandwich	Rectangular with two escape tubes on sides	Grouting and gravel
Yumeshima	Japan	2007	Steel concrete sandwich	Rectangular with two escape tubes on sides	Piled
Roertunnel	Netherlands	2008	Segmental concrete	Rectangular with a central escape tube	Gravel
Marmaray Tunnel	Turkey	2008	Steel concrete sandwich	Rectangular without an escape tube	Grouting and gravel
Thu Thiem Tunnel	Vietnam	2009	Segmental concrete	Rectangular with two escape tubes on sides	Sand jetting
Bjørvika Tunnel	Norway	2010	Segmental concrete	Rectangular without an escape tube	Scraded gravel

Name	Location	Open Year	Element type	Cross section	Foundation type
Limerick	Ireland	2010	Segmental concrete	Rectangular without an escape tube	Sand flow
Busan–Geoje Tunnel	South Korea	2010	Segmental concrete	Rectangular with a central escape tube	Grouting and gravel
Luntou–Bio Island Tunnel	China	2010	Monolithic concrete	Rectangular with a central escape tube	Sand flow
Bio Island University City Tunnel	China	2010	Monolithic concrete	Rectangular with a central escape tube	Sand flow
Naha Tunnel	Japan	2010	Steel concrete sandwich	Rectangular with two side escape tubes	Grouting and gravel
New Tyne Tunnel	UK	2011	Segmental concrete	Rectangular with a central escape tube	Sand flow
Second Coen	Netherlands	2013	Segmental concrete	Rectangular with a central escape tube	Piled
Shenjiamen Port Subsea Tunnel	China	2014	Monolithic concrete	Rectangular without an escape tube	Grouting and gravel
Zhoutouzui Tunnel	China	2015	Monolithic concrete	Rectangular with a central escape tube	Sand flow
Haihe Tunnel	China	2015	Monolithic concrete	Rectangular with a central and two side escape tubes	Grouting and gravel
Second Midtown Tunnel	USA	2016	Monolithic concrete	Rectangular with a side escape tube	Gravel

Name	Location	Open Year	Element type	Cross section	Foundation type
Dongping Tunnel	China	2017	Monolithic concrete	Rectangular with a central escape tube	Sand flow
Honggu Tunnel	China	2017	Monolithic concrete	Rectangular with a central escape tube	Sand flow
Coatzacoalcos	Mexico	2017	Segmental concrete	Rectangular with a central escape tube	Gravel
Söderstrom	Sweden	2017	Steel concrete sandwich	Rectangular with a side escape tube	Piled
North-South Line metro (NS)	Netherlands	2018	Segmental concrete	Rectangular without an escape tube	Sand flow
Hong Kong Zhuhai Macao Bridge Tunnel (HZMB)	China	2018	Segmental concrete	Rectangular with a central escape tube	Piled and gravel
Marieholm Tunnel	Sweden	2020	Monolithic concrete	Rectangular with a central escape tube	Sand flow
North-South Corridor Cross Harbour Tunnels	China	u/c	Segmental concrete	Rectangular without an escape tube	Sand flow
Dalian Bay Subsea Tunnel	China	u/c	Segmental concrete	Rectangular with a central escape tube	Gravel
Fehmarn Belt Tunnel	Denmark, Germany	u/c	Segmental concrete	Rectangular with a central escape tube	Gravel
Sharq Crossing	Qatar	u/c	Steel concrete sandwich	Rectangular	—

Name	Location	Open Year	Element type	Cross section	Foundation type
Shenzhong Link	China	u/c	Steel concrete sandwich, segmental concrete	Rectangular	Gravel
Shatin to Central Link	China	u/c	Segmental concrete	Rectangular with an escape tube	Gravel
Scheldt Tunnel	Belgium	u/c	Concrete	Rectangular with a central escape tube	—
Santos-Guarujà Crossing	Brazil	u/c	Monolithic concrete	Rectangular with central escape tube	—
Chebi Road Tunnel	China	u/c	Monolithic concrete	Rectangular with central escape tube	Sand flow
Yuliangzhou Tunnel	China	u/c	Monolithic concrete	Rectangular with central escape tube	Sand flow
A24 Blankenburg Link	Netherlands	u/c	Segmental concrete	Rectangular with central escape tube	Sand flow

[1] u/c refers to tunnels under construction

[2] The escape tube may also serve as a utility tube for tunnel installations

References

Bai, Y., & Lu, H.-H. (2016, 09). Damage analysis and repair technology of omega gasket in immersed tube tunnel. , *33*, 87-92.

Blom, C., Lukovic, M., Taffijn, E., Zanten, D., & Gellweiler, W. (2018). Repair strategy of the deteriorated immersed maastunnel. In *High tech concrete: Where technology and engineering meet* (pp. 1951–1958). Springer.

Boavida, J., Oliveira, A., & Santos, B. (2012). Precise long tunnel survey using the riegl vmx-250 mobile laser scanning system. In *Proceedings of the 2012 riegl international airborne and mobile user conference, orlando, fl, usa* (Vol. 27).

Booltink, R., Leenders, F., & Biemans, C. (2018). Modifying and renovating the 75 year old maastunnel to meet with modern standards. In *High tech concrete: Where technology and engineering meet* (pp. 1933–1941). Springer.

Braestrup, M. (2016). Danish strait crossings: Lillebælt, storebælt, øresund and femern bælt. In *Marine concrete structures* (pp. 287–319). Elsevier.

Bremer, K., Meinhardt-Wollweber, M., Thiel, T., Werner, G., Sun, T., Grattan, K., & Roth, B. (2014). Sewerage tunnel leakage detection using a fibre optic moisture-detecting sensor system. *Sensors and Actuators A: Physical*, *220*, 62–68.

Che, E., Jung, J., & Olsen, M. J. (2019). Object recognition, segmentation, and classification of mobile laser scanning point clouds: A state of the art review. *Sensors*, *19*(4), 810.

COB. (2014). Instandhouding zinkvoegen.

de Haan, H., & Bras, A. (2018). To renovate the kiltunnel.

Dhouib, M., Conciatori, D., & Sorelli, L. (2019). Optical fiber chloride sensor for health monitoring of structures in cold regions. In *Cold regions engineering 2019* (pp. 391–397). American Society of Civil Engineers Reston, VA.

Fan, L., & Bao, Y. (2021). Review of fiber optic sensors for corrosion monitoring in reinforced concrete. *Cement and Concrete Composites*, *120*, 104029.

Fan, L., & Shi, X. (2022). Techniques of corrosion monitoring of steel rebar in reinforced concrete structures: A review. *Structural Health Monitoring*, *21*(4), 1879–1905.

Fugro. (2018). Deformatiemeting.

Gavin, K., Broere, W., Kovačević, M., & de Haas, K. (2019). Investigation of the remaining life of an immersed tube tunnel in the netherlands. In *Tunnels and underground cities:*

Engineering and innovation meet archaeology, architecture and art (pp. 4831–4838). CRC Press.

GEO-Instruments. (n.d.). *Operation advantages installation - geo-instruments.* Retrieved July 18, 2022, from https://www.geo-instruments.com/wp-content/uploads/hydrostatic-level-system.pdf

GEO-Instruments. (2022). *Settlement monitoring using hydrostatic levelling cells: Geo-instruments uk.* Retrieved July 18, 2022, from https://www.geo-instruments.co.uk/news/settlement-monitoring-using-hydrostatic-levelling-cells

GEOKON. (n.d.). *Contact type (vw).* Retrieved July 20, 2022, from https://www.geokon.com/4810

Grantz, W. C. (2001). Immersed tunnel settlements. part 1: nature of settlements. *Tunnelling and Underground Space Technology*, *16*(3), 195–201.

Gursoy, A. (1995). Immersed steel tube tunnels: An american experience. *Tunnelling and Underground Space Technology*, *10*(4), 439–453.

Hu, Z., Xie, Y., Xu, G., Bin, S., Liu, H., & Lai, J. (2018). Advantages and potential challenges of applying semi-rigid elements in an immersed tunnel: A case study of the hong kong-zhuhai-macao bridge. *Tunnelling and Underground Space Technology*, *79*, 143–149.

Kil, S. (n.d.). Settlement analysis of kiltunnel.

Lee, H., & Park, H. (2013). Measurement of maximum strain of steel beam structures based on average strains from vibrating wire strain gages. *Experimental Techniques*, *37*(2), 23–29.

Leeuw, L. (2008). Lekkage in tunnels. *Rijkswaterstaat Bouwdienst.*

Leeuw, L. (2013). Zinkvoegen in tunnels.

Lin, M., Lin, W., Van Stee, J., & Peng, X. (2018). Improving the effectiveness of the injectable waterstop used for the segmented joint of immersed tunnel. In *The thirteenth isope pacific/asia offshore mechanics symposium.*

Lin, M., Lin, W., Wang, Q., & Wang, X. (2018). The deployable element, a new closure joint construction method for immersed tunnel. *Tunnelling and Underground Space Technology*, *80*, 290–300.

Lin, M., Lin, W., Yin, H., Liu, X., & Liu, K. (2019). Memory bearing: A novel solution to protect element joints from differential settlement for immersed tunnels with deep alignment. *Tunnelling and Underground Space Technology*, *88*, 144–155.

Lin, M., Liu, X., Yin, H., Tian, Y., Gao, J., Lin, W., & Liu, M. (2022). Semi-rigid system for immersed tunnel. *Proceedings of the Institution of Civil Engineers-Structures and Buildings*, *175*(2), 190–200.

Liu, X., Niu, D., Li, X., Lv, Y., & Fu, Q. (2018). Pore solution ph for the corrosion initiation of rebars embedded in concrete under a long-term natural carbonation reaction. *Applied Sciences*, *8*(1), 128.

Long, M., Trafford, A., McGrath, T., & O'Connor, P. (2020). Multichannel analysis of surface waves (masw) for offshore geotechnical investigations. *Engineering geology*, *272*, 105649.

López-Higuera, J. M., Cobo, L. R., Incera, A. Q., & Cobo, A. (2011). Fiber optic sensors in structural health monitoring. *Journal of lightwave technology*, *29*(4), 587–608.

Lunniss, R., & Baber, J. (2013). *Immersed tunnels.* CRC Press.

Mayer, T., Harnisch, J., Ebell, G., & Dauberschmidt, C. (2018, 07). Korrosionsmonitoring von stahlbetonbauwerken. *Beton- und Stahlbetonbau*, *113*. doi: 10.1002/best.201800026

McCabe, B. A., McNeill, J. A., & Black, J. A. (2007). Ground improvement using the vibro-stone column technique.

Modares, M., & Waksmanski, N. (2013). Overview of structural health monitoring for steel bridges. *Practice Periodical on Structural Design and Construction*, *18*(3), 187–191.

Molendijk, A., Dalmeijer, R., Taffijn, E., Hol, M., & van Nes, P. (2019). Zinkvoegen klaar voor de toekomst.

Nebest. (2018). Rapportage aanvullend onderzoek zinkvoegen heinenoordtunnel.

Nguyen, T. H., Venugopala, T., Chen, S., Sun, T., Grattan, K. T., Taylor, S. E., ... Long, A. E. (2014). Fluorescence based fibre optic ph sensor for the ph 10–13 range suitable for corrosion monitoring in concrete structures. *Sensors and Actuators B: Chemical*, *191*, 498–507.

Parwani, K. (2014). Quantifying the impact of loads on connections between segments of an immersed tunnel.

Physics, I. (2022). *Clip-on extensometer.* Retrieved from https://industrialphysics.com/product/clip-on-extensometer/

Rahadian, R. (2017). Investigation on stress increase of sand inside immersed tunnel joint gap.

Rite, E. (2021). *Pressure cells: Types, working principle, amp; applications.* Retrieved July 18, 2022, from https://www.encardio.com/blog/pressure-cells-types-working-principle-applications/

Roell, Z. (n.d.). *Extensometers: Strain measurement devices.* Retrieved from https://www.zwickroell.com/accessories/extensometers/

Schols, I. (2012). Segmentvoegcapaciteit van de kiltunnel.

SEG. (2019). Limfjord tunnel assessment and retrofitting.

Sinha, T. (2017). Alternative of an immersion joint.

Ştefănescu, D. M. (2011). Strain gauges and wheatstone bridges—basic instrumentation and new applications for electrical measurement of non-electrical quantities. In *Eighth international multi-conference on systems, signals & devices* (pp. 1–5).

Topolnicki, M. (2016). General overview and advances in deep soil mixing. In *Xxiv geotechnical conference of torino design, construction and controls of soil improvement systems* (pp. 25–26).

Trimble. (2020). *Tunnel technology for surveyors.* Retrieved from https://tunneling.trimble.com/node/1426

van Amsterdam, B. (2019). Probabilistic analysis of immersed tunnel settlement using cpt and masw.

van der Vliet, C. C., Kuiper, R. R., Heijmans, R. R., & Luttikholt, A. A. (2022). Golden horn unkapanı highway tube tunnel–istanbul: Underwater bridge design in seismic environment. *Tunnelling and Underground Space Technology*, *120*, 104273.

van Montfort, R. (2018). Insufficiency of immersion joints in existing immersed tunnels: Case study on functioning of gina-seal and omega-seal in the kil tunnel.

Van Oorsouw, R. (2010). Behaviour of segment joints in immersed tunnels under seismic loading.

van Putten, E., & van Os, P. (2022). The a24 blankenburg connection: An innovative design concept for an immersed tunnel project in a busy port. *Tunnelling and Underground Space Technology*, *124*, 104312.

Vervuurt, A., Braam, C., & Schols, I. (2013). Zettingsgedrag en capaciteit van mootvoegen in zinktunnels. *Land+ Water: vakblad voor civiel-en milieutechniek,(12) 2013*.

Vilanova Hanusaik, N. (2021). Analysis and optimization of transverse post-tensioning in immersed tunnels.

Wang, S., Zhang, X., & Bai, Y. (2020). Comparative study on foundation treatment methods of immersed tunnels in china. *Frontiers of Structural and Civil Engineering*, *14*(1), 82–93.

Wang, W., Zhao, W., Huang, L., Vimarlund, V., & Wang, Z. (2014). Applications of terrestrial laser scanning for tunnels: a review. *Journal of Traffic and Transportation Engineering (English Edition)*, *1*(5), 325–337.

Wei, G., Lu, S.-j., Wang, Z., & Huang, X. (2018). A theoretical model for the circumferential strain of immersed tunnel elements under tidal load. *Geotechnical and Geological Engineering*, *36*(3), 1633–1645.

Wu, X. (2017). Impact of spatial variability of subsoil stiffness on immersed tunnels.

Xiao, W., Yu, H., Yuan, Y., Taerwe, L., & Xu, G. (2017). Compression-shear behavior of a scaled immersion joint with steel shear keys. *Tunnelling and Underground Space Technology*, *70*, 76–88.

Xu, X.-c., Liu, S.-y., Tong, L.-y., Guo, J., & Yang, R.-r. (2017). Analysis of inland immersed tunnel health monitoring system.

Xuehui, Z., & Broere, W. (2022). Design of a distributed optic fiber sensor system for measuring immersed tunnel joint in immersed tunnels.

Yang, M. (2020). Settlement data analysis and prediction for kiltunnel.

Yin, J., & Huang, H. (2015). Real time monitoring method for the longitudinal settlement of shield tunnel using wireless inclinometer. In *2015 information technology and mechatronics engineering conference* (pp. 274–280).

Yu, P., Wu, H., Liu, C., & Xu, Z. (2018). Water leakage diagnosis in metro tunnels by intergration of laser point cloud and infrared thermal imaging. *ISPRS-International Archives of the Photogrammetry, Remote Sensing and Spatial Information Sciences*, *42*, 2167–2171.

Yuan, L., Zhou, L.-m., Lau, K., Jin, W., & Demokan, M. (2002). Fiber optic extensometer for concrete deformation measurements. *Review of scientific instruments*, *73*(6), 2469–2474.

Zalt, A., Meganathan, V., Yehia, S., Abudayyeh, O., & Abdel-Qader, I. (2007). Evaluating sensors for bridge health monitoring. In *2007 ieee international conference on electro/information technology* (pp. 368–372).

Zhang, J., & Zhou, L. (2018). Preparation and optimization of optical ph sensor based on sol-gel. *Sensors*, *18*(10), 3195.

Zhigang, Z., Wei, L., Hai, J., & Xiaodong, L. (2016). Layout and design techniques of cross section for the large immersed tunnel. *Procedia engineering*, *166*, 37–44.