Behaviour of segment joints in immersed tunnels under seismic loading

Final thesis

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Preface

This thesis is the result of the research performed to finish my Master “Hydraulic Structures” at the faculty of Civil Engineering and Geosciences at Delft University of Technology. The research is carried out in cooperation with the engineering company Tunnel Engineering Consultants (TEC), which is partly situated at the office of Royal Haskoning in Rotterdam and where I performed most of the thesis work. This thesis consists of an overview and conclusions drawn from the research performed on the behaviour of immersed tunnels in seismic environment.

The base for the subject of this research is my visit to Lima a year ago, where I participated in a multidisciplinary project to improve the port of Salaverry in Peru. There I was confronted with the effects of earthquakes on structures which were unknown territory to me until then. Back in the Netherlands I decided that I wanted to include this subject in my master thesis. Also my growing interest in immersed tunnels during my study in Delft, which was further stimulated during a visit at an immersion procedure of the Bjørvika Tunnel in Oslo, was an important drive to choose the subject for this research. The combination of both subjects forms the base of the research performed that finally resulted in this thesis.

Finishing this research was not possible without the necessary guiding and critical notes of my graduation committee during the period of research. I would like to thank Prof. Drs. Ir. J.H. Vrijling, Dr. Ir. K.J. Bakker and Dr.Ir. C.B.M. Blom of Delft University of Technology for their work and also Ir. H. de Wit and Ir. E. van Putten of TEC for their guidance and information at the office. Also my collegeas at the office of Royal Haskoning need to be thanked for the support and pleasant time during my thesis work.

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Abstract

The immersed tunnel technique is a common used technique for river crossings in the Netherlands, but also in the United States and Japan. 80% of the immersed tunnels are built in these 3 countries. The design of the immersed tunnels differs for the different countries, because of the possibility of seismic loading on the tunnel structure. This seismic loading can occur at the west coast of the United States and in Japan, because of their position close to tectonic earth plates.

The immersed tunnels in the Netherlands are built with elements of 100-150 meter consisting of segments of 20-25 meters. For transportation these segments are pre-stressed forming one element, after immersion on the river bottom they are released again. In this way the tunnel is flexible and can follow the settlement differences of the river bed. The water tightness in the joints is secured with rubber sealing profiles in the segment joints (W9U) and compressible rubber profiles in the immersion joint (Gina).

In areas where risk of seismic activity is severe, the immersed tunnels are built different. The pre-stressed tunnel elements are kept as one element, the same as they are for transportation, to prevent joint opening and leakage. This results in large deformations in the immersion joints and Gina gaskets and stresses in the tunnel structure. For this reason a research is performed on the behaviour of the segmental immersed tunnel subjected to seismic loading and especially on the sensitive segment joint.

Most earthquakes are created by the movement of tectonic plates. The release of energy involved with this movement creates seismic waves which will propagate through the soil. When these waves reach an immersed tunnel, it will respond to the soil movements with a certain behaviour. The design seismic wave that causes the tunnel to deform is the shear wave (S-wave). When this wave propagates parallel to the tunnel axis, the tunnel will bend in lateral direction causing the snaking effect. This can occur in horizontal and vertical direction, but the horizontal direction is determining. When the wave reaches the tunnel under an angle of 45 °, the tunnel can be deformed in axial direction. This is called the worming effect.

Both the worming and snaking effect are modeled to determine the influence of the different seismic design parameters like the wave length of the seismic wave, the construction depth and the tunnel element length. Other important model properties are the stiffness of the rubber gaskets in the segment joints and the immersion joints. All the properties and loads due to the seismic soil movement are included in two different beam models to determine the worming and snaking effect for a representative immersed tunnel.

The modeling of the worming and snaking effect shows that the gasket properties and the seismic wave length are the most important parameters. The compressibility of the Gina gasket and the elongation capacity of the W9U determines the total axial deformation of the immersed tunnel. For the snaking effect the rotation capacity and the number of Gina gaskets are of
importance. The segment joints act relatively stiff and are of less importance until their moment capacity is reached. Then joint opening and leakage could occur.

It can be concluded that the propagation speed and wave length of the seismic wave are important factors when modeling the worming and snaking effect of immersed tunnels. It is not clear from literature what the seismic design wave is for tunnels in soft soil. This needs to be investigated further to make a final judgment on the behaviour of the segment joints. For the used design parameters the tunnel can withstand the worming effect, but snaking could cause joint opening and leakage based on the free-field approach. For the snaking effect a soil-tunnel-interaction approach should be performed to determine the real tunnel response and make a final judgment of the behaviour of the segment joint.
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1 Introduction

1.1 Introduction

The immersed tunnel technique is a technique that is more than a century old and is used all over the world. It was first used in the United States at the beginning of the 20th century and was followed later on in Europe, especially in the Netherlands. With a total of 29 the Netherlands is the country with the most immersed tunnels, together with the United States (29) and Japan (25). These three countries cover about 80% of the total amount of immersed tunnels in the world. Other countries do not reach more than 10 immersed tunnels each.

In the Netherlands the immersed tunnels are not likely to encounter earthquakes because it is not situated in a seismic sensitive area. In other places in the world, where the tunnels are build in seismic areas in the subsoil, the damage on these underground structures is very minimal compared to structures on the surface. A couple of reasons for this phenomenon are[4]:

- The damage of surface structures is more visual
- There are less underground structures, then surface structures
- Underground structures involve high costs. This often ensures good quality
- The structures are constrained by the soil, so no self-vibration occurs

The most important reason is the last one mentioned, because an underground structure can not be subjected to resonance, because the surrounding soil will prevent this. This is also an important subject for this research, the interaction between tunnel and surrounding soil during the passage of an earthquake.

The aim of this research is to describe the behaviour of immersed tunnels which are subjected to seismic loading. The focus will be on the behaviour of the joints, especially the expansion- or segment joint; the connection between the concrete segments of an immersed tunnel element. These joints can be constructed in different ways and therefore performance during earthquake loading will differ for these joint types. The research will investigate in what conditions these segment joints can perform.

1.2 Problem definition

The connection between two tunnel segments in an immersed tunnel is in most cases formed by a rubber sealing (W9U) and pre-stressed cables for the transportation phase. In the Dutch situation these cables are cut after the immersion, leaving the tunnel flexible and be able to follow the variations in ground stiffness. In a seismic environment pre-stressed cables are commonly kept in place in final position. This is done because the soil movements during earthquake passage can cause too much bending or tension in the joints because of which leakage can occur. On the other hand, with pre-stressing the tunnel is stiffer and has to resist more ground movements and loads on the structure will become heavier. Therefore the structure needs larger dimensions.

Also different types of shear keys are available. Shear keys are used to transfer the shear load from one segment to another in case of settlement differences or differences in overburden. The shear keys can be situated in the outer- or inner walls and in the floor- or roof slab.
Depending on the magnitude of the earthquake loading and on the height of the settlement differences.

The problem for the segment joint is that it is not clear in which situation a specific segment joint combination is the best option. In this research the capacity of the segment with the Dutch approach will be determined to come up with a classification for the best segment joint in different seismic environments.

1.3 **Main research question**

In previous paragraph a problem is layed out. For this study we can come up with the following main question:

“What design parameters determine the limits of the existing segment joints in immersed tunnels in seismic areas?”

1.4 **Sub-question**

To answer the main question several sub-questions are formulated in order to perform a thorough investigation in different aspects of the problem. The sub questions will be the following:

- What is the behaviour of an immersed tunnel in a seismic environment in general?
- What are the existing segment joints and what is their capacity?
- Which parameters determine the design of a segment joint and how dominant is their influence?
- How can the immersed tunnel be modeled in seismic environment and what is its response?
- What are the limits of the existing segment joints?

1.5 **Objective**

The objective of this research is to develop a classification of the different segment joints based on seismic design parameters. This way the limitations of the flexible joint types are mapped so that the right joint can be chosen during a design process. Based on the conditions in the area where the tunnel is planned, the best segment joint can be chosen.

1.6 **Structure report**

The structure of this thesis will be explained here in this section. In chapter 2 an introduction will be given regarding the principles of immersed tunnels, the origin and design parameters of earthquakes and the types of segment joint available. In chapter 4 some starting points are stated as a base for further modeling. In chapter 5 the possible tunnel response to earthquakes is summed up. In Chapter 6 one of the tunnel responses, the worming effect, is examined in more detail. In chapter 7 an other important tunnel response, the snaking effect, is explained. Chapter 8 handles the behaviour of the concrete shear keys that are used in segment joints when subjected to seismic loads. In chapter 9 a classification will be proposed, describing in what conditions the segment joints can be used. Finally is chapter 10 the most important conclusions and recommendations that resulted from this research are presented.
2 Immersed tunnels in seismic environment

2.1 Immersed tunnels

The immersed tunnel technique is a very old technique that was first used on full-scale in the United States for the Detroit River Railway Tunnel in 1910[1]. The principle is to build tunnel elements on shore in a dry dock, transport these elements over water to the building site and immerse these elements to the river/sea-bottom where they are connected. The immersed tunnel elements in the USA where made out of steel as opposed to the later immersed tunnels in Europe. In Europe the price of steel was too high to build with this material, therefore reinforced concrete was used. The first concrete immersed tunnel built in the Netherlands was the Maastunnel in Rotterdam in 1942. After this successful project about 28 more immersed tunnels were build. Also in the rest of the world this technique is used to build tunnels for waterway crossings.

Concrete immersed tunnels, which are of interest in this research, are built using elements of 100-150 meters. These lengths cannot be casted in one piece because the enormous stresses that occur in the concrete during the hardening process will cause enormous cracks. Because of these cracks leakage can occur in the tunnel. To avoid this problem, the elements are casted in segments of about 20 meters in length and expansion joints are used (Figure 2.1-1). Another reason for this segmental building is to handle the settlement differences due to varying subsoil. To secure the water tightness of the expansion joint a rubber profile “W9U” is used between the segments (Figure 2.1-2). To transport the segments as one tunnel element the tunnel is prestressed with post-tension cables over the full length.
When the element is build up, temporary bulkheads are placed in the front and rear end of the element and it is floated up. Then the element is transported to the building site and immersed in an excavated trench at the bottom of the river/sea (Figure 2.1-3). This is done using ballast tanks filled with water creating a negative buoyancy resulting in the lowering of the element.

![Figure 2.1-3: Immersion of tunnel element [2]](image)

To connect the elements on the bottom, they are pulled together and the gasket (Gina) between the elements is pressed together resulting in a watertight sealing (Figure 2.1-4). Then the water is pumped out between the bulkheads and due to hydrostatic pressure the element is pushed more against the previous element and the gasket is sealed completely. Finally a second water sealing, the “Omega” profile, is placed and the bulkheads can be removed. From this point the tunnel interior can be finished. The ballast concrete is put in place, the temporary ballast tanks are removed and installations are placed.

![Figure 2.1-4: Connecting elements [2]](image)

There are many advantages for immersed tunnels as a water crossing in comparison to bored tunnels or bridges. Some of these advantages are[3]:

- Immersed tunnels do not need a lot of overburden due to their own weight, so the alignment can be shorter in comparison to a bored tunnel or bridge resulting in cost reductions (Figure 2.1-5)
- Immersed tunnels do not have a circular shape cross section, so the cross section can be used more efficient. This is ideal for wide highways or combined rail and road tunnels.
- Tunnel elements are built on shore in a dry dock, where decent quality control can be performed. This will result in a more reliable product.
- Because the elements are not built on site, there is no need for working area on site. In this way immersed tunnels are ideal for urban areas.
- A tunnel does not have a height limitation for ship traffic crossing the connection.

![Figure 2.1-5: Alignment comparison [ITA, 2009]](image)
2.2 Earthquakes

2.2.1 Origin of earthquakes

There are many types of earthquakes occurring in the earth body. The type of earthquake depends on the location on earth where it occurs and the geological conditions. They can be divided in the following types[4]:

- Tectonic earthquakes; occur due to tectonic plates of the earth crust moving along side each other
- Volcanic earthquakes; occur due to volcanic activity
- Collapse earthquakes; occur due to the collapse of mines and caverns.
- Explosion earthquakes; occur due to explosion of chemical and nuclear devices

The most common earthquake is the tectonic earthquake. This is also the most important earthquake when building in the underground (Figure 2.2-1).

Figure 2.2-1: Movement tectonic plates[12]

At some places on earth these tectonic plates are diverging (black) and in other places they are sliding alongside each other or converging (red). Due to the friction between these converging plates, they can get stuck and later on be released again. As a result of this release an earthquake occurs and energy is released (Figure 2.2-2). The rupture plane is called a fault and can be observed directly on the ground surface or in deeper laying plate joints.

Figure 2.2-2: Origin of tectonic earthquake [U.S. Geological Survey , 2009]
2.2.2  **Ground behaviour under the influence of earthquakes**

Due to energy release at the origin of an earthquake the surrounding ground will response to this in the form of ground movement. This movement can be divided into two groups[5]: 1) ground shaking and 2) geotechnical ground failure.

1)  **Ground shaking**

Ground shaking is caused by the propagating seismic waves through the soil mass. These seismic waves are described in the next paragraph.

2)  **Geotechnical ground failure**

The ground failure due to earthquake loading can be divided into 3 mechanisms: slope instability, fault displacement and liquefaction.

Ground shaking can cause landslides to occur, which we call slope instability. When this phenomenon occurs nearby a tunnel, this will result in shear displacement and may cause collapse of the cross section of the tunnel.

Fault displacement is the sliding of plates alongside each other. When this occurs on the lining of a tunnel, this would inevitably damage the tunnel. Therefore good soil investigation is required when selecting the location of a tunnel to avoid this phenomenon.

Liquefaction is the phenomenon where the soil deposits have lost their strength and appear to flow as fluids. Due to the ground shaking of the earthquake, the sediments will rearrange resulting in an increasing water pore pressure. This will cause the soil to act as a fluid and will be a threat to the tunnel structure in the underground. This can be avoided by using a gravel bed, soil improvement or a piled foundation.

2.2.3  **Seismic waves**

Seismic waves can be divided into two groups[6]: body waves and surface waves. Body waves are waves propagating in the interior of the earth in all directions. Surface waves only occur at the surface of the earth crust.

There are two types of body waves: Primary (P) - waves and Secondary (S) -waves (Figure 2.2-3). P-waves work in longitudinal direction by compressing and extending the soil body in the same direction as the wave propagation. The P-waves travel twice as fast as the S-waves. S-waves are shear waves and work in transverse direction, so perpendicular to the wave propagation. This particle motion can be in horizontal or vertical direction, so we can divide the shear wave further in SH-waves and SV-waves.

![Figure 2.2-3: Body waves](Kramer, 1996)
The surface waves can also be divided into two groups: Love waves and Rayleigh waves (Figure 2.2-4). Love waves cause horizontal shearing and Rayleigh waves move the ground with an elliptic rolling movement, like the orbital motion of ocean waves.

![Love wave and Rayleigh wave](image)

**Figure 2.2-4: Surface waves [Kramer, 1996]**

The velocity of the body waves and surface waves are different and depend on the type of soil in which they are propagating. In hard rock the propagation speed is very high, in the range of several kilometers per second and in loose soils the speed is more in the range of hundreds of meters per second. The fastest waves are the P-waves, which are about 2 times as fast compared to the S-waves. The speed of the surface waves is much lower than that of the S-waves. If an accelerogram (registration of seismic soil movements) is examined, first the P-waves can be distinguished, followed by the S-waves and last the surface waves will arrive. For the propagation velocity of the body waves *Kramer* [6] gives the following formulas:

\[
P\text{-waves: } C_p = \sqrt{\frac{G(2-2\nu)}{\rho(1-2\nu)}}
\]

\[
S\text{-waves: } C_s = \sqrt{\frac{G}{\rho}} \quad \text{G = shear modulus}
\]

\[\text{with } \nu = \text{poisson' s ration} \]

\[\rho = \text{density} \]

### 2.2.4 Design earthquake

#### 2.2.4.1 Size of earthquakes

To categorize earthquakes there are two ways to describe the size of an earthquake; the intensity or the magnitude[6].

The intensity of an earthquake is a qualitative description of the effects of an earthquake on a specific location. This is based on the observed damage and human reactions at this location. It is not a very accurate scaling because of the subjective approach with a human interpretation. The most common used intensity scale is the Modified Mercalli Intensity (MMI), which is scaled from I till XII for weak to strong earthquakes.

A more quantitative approach of categorizing an earthquake is the magnitude scale. This scale is based on the measurements of ground motion in the surrounding of the earthquake. This is related to the energy release and in this way not dependent on the place of observation. The different types of magnitude scales that are used are the local magnitude scale $M_l$ (Righter),...
body wave magnitude $M_B$, surface wave magnitude $M_s$ and the moment magnitude scale $M_W$.

The Righter scale is used more in news bulletins and the moment magnitude scale is preferred by seismologists. The two wave magnitude scales are related to the seismic waves in ground body, but they are not used a lot.

### 2.2.4.2 Seismic hazard analysis

To determine the design earthquake for a structure in a specific area a seismic hazard analysis can be performed. There are two types of analysis for this purpose: the Deterministic Seismic Hazard Analysis (DSHA) and the Probabilistic Seismic Hazard Analysis (PSHA)\[6\].

With the DSHA, the nearby potential earthquake sources are examined and with zoning relation the size of the earthquake on the building site is determined.

![Figure 2.2-5: Seismic hazard map Europe (Source: Institute of Earth Sciences Spain, 2009)](image)

For the PSHA all earthquake sources in the area and their probabilistic distributions of occurrence are included. This will result in a seismic hazard curve with a distribution of the Peak Ground Acceleration (PGA) of the hard bedrock as a function of the recurrence of all the earthquake sources. With this curve the design earthquake with a certain return period can be determined. This is performed for the whole world by the United Nations, but can also be selected per area, for example for Europe (Figure 2.2-5). These maps are freely available on the internet and the input is provided by local governments. The recurrence time for this map is 475 years and the higher PGA’s are marked in red/purple.
For the design phase of the structure two design earthquake levels can be distinguished[4]:

The Operational Design Earthquake (ODE)
The ODE is the earthquake that will probably occur once or twice during the lifetime of a structure. During and after the occurrence of this earthquake the structure needs to stay intact and little or no damage may arise. A common return period for this design level is once every 75 to 100 years. From the seismic hazard curve a design peak ground acceleration and ground displacement can be subtracted.

The Maximal Design Earthquake (MDE)
The MDE is the earthquake that has little chance to occur during the lifetime of a structure. During and after the occurrence of a MDE the structure needs to remain it’s function so people are able to escape from the structure. Large damage due to this earthquake is allowed, but people may not be effected during their flee. A common return period for this design level is once every 750 to 1000 years. Again the seismic hazard curve will give a design peak ground acceleration and ground displacement.

2.2.4.3 Code based design earthquake
Another way to determine the design earthquake is to make use of codes like the Uniform Building Code (UBC) or the Eurocode[7]. This is a less accurate way to determine a design earthquake but a common way for seismic design of structures. In these codes earthquake response spectra are given (Figure 2.2-6) for different soil types (A till E for hard rock to soft alluvial material). It gives the frequency-dependent amplification of the ground motion for different local soil conditions.

![Earthquake response spectra](Eurocode 8.1, 2004)

Figure 2.2-6: Earthquake response spectra [Eurocode 8.1, 2004]
2.3 Segment joints

2.3.1 Types of segment joint

The tunnel elements that are constructed in concrete as described in paragraph 2.1 are mostly used in Europe and Asia. There are two ways of constructing these tunnel elements: monolithic or segmental (Figure 2.3-1). The monolithic tunnel is cast in one part with several pours and in this way encounters cracking of the concrete during the hardening process of the concrete. To ensure water tightness of the tunnel elements a membrane is placed around the tunnel structure with bitumen or steel. The other way of constructing a tunnel element is by segmentation of the elements. With this approach the tunnel is cast per segment to avoid large cracking.

The tunnel elements of an immersed tunnel in the Netherlands are normally built with segments of about 20-25 m length[8]. This is done to prevent major cracks arising during the hardening process of the concrete. Between the individual segments a joint is used to connect the segments forming a tunnel element. This segment joint has three important functions:

- Allowing differential movements in longitudinal and transverse directions and relative rotation
- Resist static and dynamic earth and water loads
- Secure water tightness

The transfer of compression forces in longitudinal direction is ensured by casting the joints concrete to concrete. Extension forces in longitudinal direction are transferred with pre-stressed cables or through sealing profiles. Shear forces in transverse direction are transferred through the segment joint using shear keys. The water tightness is secured by a sealing profile which also needs to be capable of resisting water pressures due to high water depths.

During transportation of the tunnel elements to the building site the segment joint is kept very stiff by using pre-stressed cables (red dotted lines) to provide bending capacity. In its final position the segment joint can vary from stiff to flexible depending on the cutting of the pre-stressed cables. As described previously in Dutch tunnel structures (Figure 2.3-2) the pre-stressed cables are cut after immersion. This way the tunnel can follow the settlement differences of the foundation bed. The movements of the segments are relatively small and can be followed by the sealing profile (W9U).

In seismic environments the tunnel structure is subjected to ground shaking resulting in a tunnel response. One of the response modes is the worming effect, which can cause the segment joints to open. If the opening becomes too large, the sealing profile cannot follow this movement and leakage may occur. To prevent this, the pre-stressed cables are kept in place after immersion (Figure 2.3-2). This way the segment joint is sealed by compression. The stiff segment joint is common used in seismic areas like Japan, therefore it can be called a “Japanese” segment joint.
Between the flexible Dutch segment joint and the stiff "Japanese" segment joint a more intermediate joint is also possible. In this type the shear force is brought from one segment to the other by shear keys in the inner walls (Figure 2.3-4). This way the segment joint has more shear capacity to withstand earthquake loading, but is still flexible. The pre-stressed cables are cut after immersion and the water tightness is secured with a sealing profile (W9U).

For an immersed tunnel project in Korea, the “Busan – Geoje Fixed Link”, this type of intermediate segment joint was used. For this tunnel in a seismic environment transverse and axial forces were expected. The shear keys in the inner walls could transfer these transverse forces from segment to segment. The axial forces causing the worming effect in the tunnel structure results in compression and extension of the segment joint. It is assumed that the W9U could not follow this elongation due to the extension, so an Omega-profile is used as a second water sealing like it is done at the immersion joint with a smaller size. This Omega-profile has more tensile capacity and can be elongated more in comparison to the W9U. This way a larger joint opening of the segment joint can be handled during earthquake loading.

2.3.2 Water sealing

To secure water tightness of the immersed tunnel, water sealing profiles are used at the segment joints. Two types of water sealing profiles are used depending on the properties of the segment joint. If the segment joint is stiff, so the joint is compressed by pre-stressed cables, a compressible water sealing is used. When the segment joint is flexible like in the Dutch structures, a more flexible water sealing profile is used, usually the W9U profile.

2.3.2.1 Compressible water sealing

The compressible water sealing is used when the segment joint is cast concrete to concrete. Between the segments an expanding hydrophilic waterstop is used to secure water tightness. A
common used profile is the Hydrotite profile[9]. This profile can expand up to 4 - 8 times its original volume when exposed to water. This way all gaps between the segments are filled up (Figure 2.3-5). This profile has no capacity to transfer axial forces through the segments, so it can only by used for immersed tunnels where the pre-stressed cables are kept in place.

![Figure 2.3-5: Expanding hydrophilic waterstop [Kasei, 2007]]

2.3.2.2 Flexible water sealing
For the flexible segment joint the water tightness is secured with a flexible water sealing profile. Most used profile for this situation is the W9U-profile. This profile is placed between the segments at construction and poured into the concrete. This way the segments are forming one piece from a water tightness point of view.

This type of water sealing profile was first designed by “Vredestein”, a Dutch producer of rubber profiles. The first profiles where called W9A, W9B and W9C (depending on the size) and were used in immersed tunnels in the Netherlands[10]. Later on the company “Trelleborg Bakker” took over the production of this rubber profile and it is now produced as the W9U profile. This profile is made out of rubber which is heated and then shaped by pushing it in a mold. When it is still hot, on either both side a steel plate is pushed in. These bendable steel plates are used to increase the adhesion between the profile and the concrete. The adhesive property of the rubber itself is not very good, therefore extra steel plates are used.

There are different types of W9U profiles[2] available depending on the size and injection capacity (Figure 2.3-6). The W9U is the standard profile with a width of 350 mm. This type is commonly used in Dutch immersed tunnels. If more movement in the segment joint is expected, the W9CU-profile can be used. This profile has a width of 500 mm and can be elongated more. Also a smaller W10U profile is available for expansion joints, but this is not used in immersed tunnels because of its restrictions.

![Figure 2.3-6: W9U-profiles [Trelleborg, 2007]]
After years of using the W9U profiles the problem arose that the concrete was not poured well around the profile. Because of the large amount of reinforcement bars around the profile and the width of the steal plate on the profile, the space below the profile is not very well reachable during pouring of the concrete. This problem results in pores, cracks and even rock pockets in the concrete and water tightness is not secured anymore. To solve this problem the W9U-i profiles are designed (Figure 2.3-7).

![Figure 2.3-7: W9U-i profiles [Trelleborg, 2007]](image1)

These profiles have the same dimensions as the “normal” profiles, but have an extra injection hose at the end of the profile (Figure 2.3-8). After hardening of the concrete segments, this hose can be injected with epoxy to fill the holes and cracks. This way the segment joint is watertight.

![Figure 2.3-8: W9U-i profile [Leeuw, 2008]](image2)

Due to temperature differences in the concrete the segment can shrink, opening the segment joint. Also settlement differences between segments can result in opening of the segment joint. The sealing profile has to follow opening of the joint and be elongated. Also due to the tunnel response on earthquake loading as described in previous paragraphs, the segment joint can open and the W9U profile is elongated. The elongation capacity of the profile depends on the initial opening of the joint and the water pressure acting on the profile. This water pressure causes arch forming of the rubber profile which reduces the stress in the rubber (Figure 2.3-9).

![Figure 2.3-9: Opening of segment joint [Trelleborg, 2005]](image3)
For the different W9U-profiles design graphs are produced by the company Trelleborg Bakker to determine the elongation of the profile (Figure 2.3-10). Different elongation tests are performed resulting in design graphs for Serviceability Limit State (SLS) and Ultimate Limit State (ULS) conditions. Depending on the existing water pressure and the initial gap between the segments the elongation of the profile can be determined. The common initial gap for immersed tunnels is 0.

![Application data W9CU & W9CU](image)

**Figure 2.3-10: Design graph W9CU elongation [Trelleborg, 2007]**

Immersed tunnels in the Netherlands are built relatively shallow, so the water pressure will be below the 30 meter water column. This equals a pressure of 0.3 MPa and stays in the save area of the design graph, below and left of the lines. In seismic area an earthquake can cause quick movements of the tunnel segments and water can be enclosed. This will result in higher water pressures and can have effect on the sealing profile.

### 2.3.3 Shear keys

As a result of earthquake loading, the ground around a tunnel structure will move. This movement results in longitudinal and transversal forces on the tunnel structure. The transverse forces work in horizontal en vertical direction on the cross section of the tunnel structure and also create a rotation moment on the structure (Figure 2.3-11). To transfer these forces in the segment joint from one segment to another, shear keys are used. The shear keys can be situated in the outer walls or in the inner walls of the tunnel structure depending on the size of the shear forces.
2.3.3.1 Shear keys in outer walls

In the Dutch structures the shear key is situated in the outer wall, like a spigot and socket joint of sewer tubes. The shear capacity of this joint type depends on the thickness of the walls and the floor- and roof slab, which is about 1 meter in common situations. The capacity to withstand, for example, vertical shear force depends on how much reinforcement in this shear key can be included (Figure 2.3-12).

![Figure 2.3-11: Forces on tunnel structure](image)

Figure 2.3-11: Forces on tunnel structure

![Figure 2.3-12: Shear key in Dutch segment joint](image)

Figure 2.3-12: Shear key in Dutch segment joint

The more reinforcement in the shear key, the bigger the shear forces that can be transferred. On the other hand, with more reinforcement the casting of concrete will become difficult and pores in the concrete can arise. This can weaken the shear key. Another point of attention for this type of shear key is the accessibility in case of damage. In comparison to shear keys in the inner walls, this type of shear key is difficult to reach when damage occurs. So repair of this shear key after cracking is difficult. However the only known crack of this type of shear key is at the Kiltunnel[10] near Dordrecht. Due to large settlement differences the shear key cracked and leakage occurred. This leakage was solved by boring to the crack and inject it with a sealing fluid.
2.3.3.2 Shear keys in inner walls

As described in previous paragraphs another possibility of transferring the shear force from segment to segment is to make use of shear keys in the inner walls and on the inside of the roof, floor and outer walls (Figure 2.3-15). This type of placing of the shear key has an somewhat equal capacity in vertical direction in comparison to the previously described shear key in the outer walls. The capacity of the horizontally placed shear keys have a bigger capacity in comparison to the shear keys placed in the outer walls from previous paragraph. The horizontal and vertical shear capacity of this type of shear keys is somewhat equal. By placing of the shear keys this way the rotation moment can also be taken better compared to the Dutch segment joint.
Another advantage of this type of placement of the segment joint is that the shear keys have good accessibility. In case of an earthquake, which can cause damage to the shear keys, they can be repaired relatively easily. They can be reached from the inside of the tunnel without danger for leakage. The sealing profile is placed outside of the shear keys in the outer walls and floor/roof slab.

In the segment joint type for seismic environment, the “Japanese” segment joint, no shear key is included. The shear forces are transferred from segment to segment by the friction between the segments. The segments are pushed to each other due to the pre-stressed cables and this way the shear forces are transferred.
3 Design starting points

3.1 Geometry of tunnel

In this research the immersed tunnel is modeled to determine the effect of seismic waves on the tunnel structure. To gain more insight a representative tunnel is used during this modeling in calculations in following chapters. A short description of the tunnel properties is given here.

Immersed tunnels are mostly used as river crossings. To give a reality based comparison, in the following calculations a river crossing is also used. In the tunnel is placed in a soil layer of loose soil like sand or alluvial deposits under a river. Under this loose soil layer most of the time a hard bedrock is situated (Figure 3.1-1).

![Figure 3.1-1: Geometry of tunnel](image1)

The length of the entire tunnel is not determined, because only the part that lies below the river is of interest in this research. At the point where the tunnel reaches the surface, the abutments, the tunnel is build with the cut and cover method. This is a rigid structure with a firm foundation, so this is less affected by seismic loading. In this research only the immersed parts below the river are looked at and the abutments are not taken into account. The behaviour of the tunnel elements are subjected to seismic loading with the boundary condition that they are compressed between the rigid abutments.

![Figure 3.1-2: Tunnel element](image2)
The properties of a single tunnel element are fixed to be able to model the tunnel response. A tunnel element is constructed by coupling of tunnel segments (Figure 3.1-2). One single tunnel element consist 6 segments and the element has the following properties:
- Segment length \( L_s = 20 \text{ m} \)
- Element length \( L_t = 6 \times 20 = 120 \text{ m} \)

### 3.2 Cross section tunnel

For the modeling of seismic loading on a tunnel structure a specific cross section needs to be chosen to be able to make calculations. A typical cross section is used based on a reference project in Mexico; the “Coatzacoalcos Immersed Tunnel”. Several reference projects of immersed tunnels in seismic environments are described in paragraph 3.5.

The cross-section of the tunnel is drawn (Figure 3.2-1) and basic properties are listed in Table 1.

![Cross section of representative tunnel](image)

**Figure 3.2-1: Cross section of representative tunnel**

<table>
<thead>
<tr>
<th>Table 1: Tunnel parameters</th>
<th>Outer dimensions</th>
<th>Inner dimensions</th>
<th>Width slabs</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tunnel parameters</strong></td>
<td><strong>Total area</strong></td>
<td><strong>Moment of inertia</strong></td>
<td><strong>Concrete properties</strong></td>
</tr>
<tr>
<td>Tunnel height ( H_t = 9.3 \text{ m} )</td>
<td>Surface area ( A_c = 77.35 \text{ m}^2 )</td>
<td>( I_{zz} = 1040 \text{ m}^4 )</td>
<td>Type = C28/35</td>
</tr>
<tr>
<td>Tunnel width ( W_t = 24.5 \text{ m} )</td>
<td>Perimeter ( O_c = 67.6 \text{ m} )</td>
<td>( I_{yy} = 4774 \text{ m}^4 )</td>
<td>( E_c = 31000 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td><strong>Inner dimensions</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Traffic lane width ( TL_w = 10 \text{ m} )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Traffic lane height ( TL_h = 7 \text{ m} )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Escape lane width ( EL_w = 1,5 \text{ m} )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Escape lane height ( EL_h = 7 \text{ m} )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Width slabs</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof slab width ( W_r = 1 \text{ m} )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floor slab width ( W_f = 1,3 \text{ m} )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outer wall width ( W_{ou} = 1 \text{ m} )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inner wall width ( W_{in} = 0,5 \text{ m} )</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### 3.3 Geotechnical parameters

As described in paragraph 3.1 most of the immersed tunnels are situated in the subsoil of a river. To have a representative site for the modeling with seismic loading, in the following calculations a river crossing is also used (Figure 3.3-1).

A typical subsoil layering for such a situation is first a very thin layer of alluvial material and sediments. Below this, a layer of loose sand or/and clay is situated (20–30 m.). Below this layer of loose material, a layer of densely packed sand can be distinguished (20–30 m.). At last below this dense sand the hard bedrock starts. This bedrock can be kilometers thick, so this can be seen as the boundary of the soil profile.

The layering of the soil is presented very straightforward and simplified. In reality the layers are more mixed with parts of clay or sand in between. The line between two layers is also not this straight. But to give a good impression of the tunnel behaviour due to soil movement, a simplified layering needs to be chosen.

![Figure 3.3-1: soil profile](image)

### 3.4 Loads

The purpose of this research is to determine the effects of seismic activity on the tunnel structure. Not all the loads that can influence the tunnel design are of interest for this research so not all the loads are taken into account for further modeling. The summation below gives the load that are included in the further modeling, but also the load that are not included.

**Included loads:**
- Static loads (soil pressure, water pressure, buoyancy etc.)
- Seismic loads (soil movement)

**Not included loads:**
- Loads in the tunnel interior due to temperature differences
- Falling anchor
A common design load for immersed tunnels is the impact of a falling anchor of a passing ship. This anchor can perforate the tunnel roof, so this is a standard load that immersed tunnel designs have to be able to withstand. This type of load has no relation with the seismic activity so it is not included in further modeling.

The loads that are included in the modeling of the tunnel for seismic activity are now described separately. The loads are included in the calculations if necessary.

### 3.5 Reference projects

To make a choice for a design tunnel for the further modeling, it is good to have a look at other immersed tunnel projects which are situated in seismic areas. For these projects the important parameters are required to have a good view on these projects, like the soil properties, seismic parameters, tunnel dimensions, type of sealing profiles, shear keys, etc.

The projects that are described below are projects where TEC was involved in design or acted as reviewer of the proposed design. This way the information was available to make a good comparison. The projects are “Busan-Geoje Fixed Link” in South Korea, “Coatzacoalcos Immersed Tunnel” in Mexico, and the “HongKong-Zhuhai-Macao Bridge” (HZMB) China which consists of bridges and an immersed tunnel part. The geotechnical cross sections of the reference projects are included in appendix A.

#### 3.5.1 Busan-Geoje (Korea)

The Busan-Geoje is the first offshore concrete immersed tunnel which is immersed to the large depth of 48 meters. In this project they had to deal with large wave heights and a very soft clay layer. Also they had to deal with severe earthquake risk. The following properties of the tunnel are known:

<table>
<thead>
<tr>
<th>Soil conditions</th>
<th>Soft clay (20 m.) + hard bedrock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic parameters (MDE – 750 year):</td>
<td>PGA = 1.54 m/s²</td>
</tr>
<tr>
<td>Foundation:</td>
<td>Gravel</td>
</tr>
<tr>
<td>Tunnel dimensions:</td>
<td>Length = 18 x 180 = 3240 m</td>
</tr>
<tr>
<td>Cross section:</td>
<td>B x H = 9.75 x 25.9</td>
</tr>
<tr>
<td>Immersion joint:</td>
<td>Gina-profile: Yokohama + Omega</td>
</tr>
<tr>
<td>Segment joints</td>
<td>Shear keys in inner- and outer wall, floor and roof</td>
</tr>
<tr>
<td>Shear keys in inner- and outer wall</td>
<td>W9CU-I waterstop + extra omega + inflatable sealing (repair)</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.5.2 Busan

Figure 3.5-1: Geotechnical cross section Busan (m)

3.5.3 Coatzacoalcos (Mexico)

The Coatzacoalcos tunnel is situated in a heavy seismic environment and the soil conditions are difficult. The following properties of the tunnel are known:

<table>
<thead>
<tr>
<th>Soil conditions</th>
<th>Clay and sand + hard bedrock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic parameters (MDE – 1000 year):</td>
<td>PGA = 2.0 m/s²</td>
</tr>
<tr>
<td>Foundation:</td>
<td>Gravel</td>
</tr>
<tr>
<td>Tunnel dimensions:</td>
<td>Length = 6 x 138 = 830 m</td>
</tr>
<tr>
<td>Cross section:</td>
<td>B x H = 9.2 x 25.1</td>
</tr>
<tr>
<td>Immersion joint:</td>
<td>Gina ETS 320/370 + Omega</td>
</tr>
<tr>
<td>Shear keys in inner- and outer walls and floor</td>
<td></td>
</tr>
<tr>
<td>Segment joints</td>
<td>Pre-stressed joint</td>
</tr>
<tr>
<td>Crack inducer for earthquake damage</td>
<td></td>
</tr>
<tr>
<td>Hydrotite waterstop + injection hose</td>
<td></td>
</tr>
</tbody>
</table>

3.5.4 Coatzacoalcos

Figure 3.5-2: Geotechnical cross section Coatzacoalcos (m)

3.5.5 HZMB (China)

In this project the offshore conditions also need to be taken into account for the tunnel design. Important factor for this tunnel project is the large construction depth, because is needed to lay below an approach channel for large vessels (300 ton). This causes high water pressures on the
tunnel (40-50 m) and also a lot of sedimentation on top of the tunnel can be expected. The following properties of the tunnel are known:

<table>
<thead>
<tr>
<th>Soil conditions</th>
<th>Clay and sand + hard bedrock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic parameters (MDE – 750 year):</td>
<td>PGA = 1.5 m/s²</td>
</tr>
<tr>
<td>Tunnel dimensions:</td>
<td>Length = 32 x 180 = 5760 m</td>
</tr>
<tr>
<td>Cross section:</td>
<td>B x H = 11.5 x 37.9</td>
</tr>
<tr>
<td>Immersion joint:</td>
<td>Gina ETS 320/370 + Omega</td>
</tr>
<tr>
<td></td>
<td>Shear keys in inner- and outer walls, floor and roof</td>
</tr>
<tr>
<td>Segment joints</td>
<td>W9CU-I waterstop + extra omega + inflatable sealing (repair)</td>
</tr>
<tr>
<td></td>
<td>Shear keys in inner- and outer wall</td>
</tr>
</tbody>
</table>

Figure 3.5-3: Geotechnical cross section HZMB (m)

3.5.1 Used reference project

The Coatzacoalcos tunnel in Mexico is a project which is situated in a severe seismic area. The probability of occurrence of a large earthquake is high and the Peak Ground Acceleration, Velocity and Displacement are also high. This is why in further modeling in following chapters this reference project is used as case study. For this project a lot of information is available at TEC which can be used for the examination of tunnel response on seismic activity.
4 Tunnel response

The response of the tunnel to seismic waves in the underground can be separated in different modes. This is first explained in the first paragraph and after that the two important response modes worming and snaking are handled in detail in the following paragraphs.

4.1 Introduction of tunnel response

4.1.1 Seismic waves

From paragraph 2.2 it is given that the snaking effect of earthquakes is caused by the movement of seismic waves through the ground. The seismic waves that effect the soil are body waves and surface waves. The body waves traveling through the soil can be separated in P-waves (pressure waves, axial direction) and S-waves (shear waves, lateral direction). Surface waves only propagate at the surface and can be separated in Rayleigh-waves and Love-waves.

For the modeling of the immersed tunnel in seismic environment in this research, the origin of the earthquake (fault) and the site of the tunnel are not at close range. The propagation of the seismic waves will take place in the hard bedrock. The damping in the upper alluvial layer is much greater than in the bedrock. Because of this the surface waves will be damped out very fast and will not reach the tunnel site. So in the further modeling the surface waves are neglected and only body waves are taken into account.
The body waves propagate in the earth interior through the hard bedrock and finally reach the site where the immersed tunnel is situated (Figure 4.1-3). The soft layers on top of the bedrock have a lower stiffness in comparison to the bedrock stiffness. When the body waves traveling in the bedrock reach the soft layer, refraction takes place in the direction of the surface due to the varying stiffness[13]. As a result of this phenomenon the seismic waves can arrive at the immersed tunnel under an angle. Also the propagation speed will decrease in the soft layers.

The body waves exert different effects on the tunnel and its behaviour. The effects depend on the type of seismic wave (P- or S-wave), their propagation speed and the angle of the incoming wave related to the tunnel axis as described above. An incoming seismic wave with a certain angle $\phi$ can be separated in a longitudinal part and a perpendicular part (Figure 4.1-4). The effect of the wave on the tunnel of each direction has to be taken into account.

Waves which travel at a certain distance from the source can be seen as a wave front with a large width. Perpendicular waves will move the tunnel as a whole over this wave front, so no local deformation on different parts of the tunnel is found. In this way there are no force differences in the tunnel parts and the joints are not loaded. The only important perpendicular
wave is the shear wave perpendicular to the tunnel axis which causes racking as will be described in the following paragraph.

Longitudinal waves will result in deformations in opposite direction over the different parts of the tunnel. This results in forces in the longitudinal and lateral direction which will determine the behaviour of the tunnel. So the main focus will be on these longitudinal waves.

4.1.2 Tunnel behaviour

When the seismic waves reach the immersed tunnel, the tunnel will respond on the soil movement in the surrounding soil. This response can be separated in different deformation modes of the tunnel. The three different types of deformation[5] are: axial extension and compression, longitudinal bending and racking.

4.1.2.1 Axial extension and compression

Due to the P-waves the ground will be compressed and extended in the direction of the wave propagation, as described in the previous paragraph. If this propagation is parallel to the axis of the immersed tunnel, this will result in compression and tension of the tunnel elements (Figure 4.1-5). Also the S-wave on an angle $\varphi$ of 45 degrees will cause the tunnel to be compressed and tensioned in axial direction. This phenomenon is also called the “worming effect”, because of the similarities to the movements of a worm.

![Compression-extension](image)

(a) Compression-extension

Figure 4.1-5: Worming effect [Hashash Y.M.A, 2001]

4.1.2.2 Longitudinal bending

The S-waves will move the ground in lateral direction in relation to the wave propagation. The effect of this motion to an immersed tunnel will be that the tunnel follows this motion. The tunnel will be bended resulting in compression on one side and tension on the other side in the same cross-section (Figure 4.1-6). This deformation of the tunnel is also known as the “snaking effect” related to the moving motion of a snake.
4.1.2.3 Racking

The last deformation mode of an immerse tunnel is in the cross-sectional plane of the tunnel. When shear waves propagate in a direction perpendicular to the tunnel axis, the structure will have a distortion in the cross-sectional plane (Figure 4.1-7). The S-wave will reach the tunnel at the floor and propagate in the direction of the surface. This deformation mode is called “racking” of the tunnel structure.

The focus of this research is on the behaviour of the segment joints in an immersed tunnel during the passing of seismic waves. The different deformation modes due to these passing waves will cause different loading cases for the segment joints. For the deformation mode where racking occurs, the loading will be in the cross sectional plane. The segment joint is more vulnerable for loading in axial direction. Therefore the main focus will be on the worming and snaking behaviour. The phenomenon of racking will not be investigated in more detail.

4.1.3 Worming

The worming effect due to seismic waves consists of the tension and compression of the tunnel structure in longitudinal direction. This effect is caused by two different seismic waves:

- P-wave in longitudinal direction ($\varphi = 0^\circ$)
- S-wave on an angle of incoming (max. for $\varphi = 45^\circ$)

The tunnel structure will be deformed in longitudinal direction because in some regions the tunnel is compressed and in other parts the tunnel is lengthened by tension. This phenomenon is drawn below.
This deformation of the whole structure will be resisted by the immersion joints and segment joints between the concrete segments. The immersion joints are formed by the GINA profiles and they are relatively flexible in comparison to the concrete elements and can handle a certain deformation. The segment joints behave more stiff because there is no rubber profile in between the segments. The focus of this research will be on the effects of worming on segment joints. This deformation mode will result in two load cases; compression and tension of the segment joint.

4.1.3.1 Axial compression
The compression of the segment joint is a positive load, because concrete acts better when it is compressed instead of under tension. Due to the fact that the immersed tunnel is situated below water level, an axial compression from the water pressure is already present. This water pressure combined with the pressure of the seismic wave, gives a certain amount of compression on the segment joint. The segment joint has to withstand this compression, so the restriction for this load case is the compressive strength of the used concrete.

Another phenomenon that takes place during axial compression is the loading and unloading of the axial force. The seismic waves generate compression and tension while travelling through the soil. This way compression and tension will both work on a segment joint. This causes loading and unloading of the axial force and hammering of the segments can occur. In this case the segments are hitting each other and bridling of the concrete can occur. This may reduce the strength of the segment joint.

4.1.3.2 Axial tension
The other effect that takes place during the worming effect is the axial tension of the joints. The immersion joints with the GINA-profile can bear no tension, because the profile is only connected on one side and sealed by compression due to water pressure. When tension occurs due to the worming effect on the immersion joint, first the water pressure has to be overruled but then joint opening could occur. To prevent this immersion joint opening, in seismic areas sometimes couplers are used to maximize the relaxation of the GINA-profile (Figure 4.1-9). This way the GINA-profile can be compressed and released to a certain level and joint opening is prevented. With this restriction of relaxation, water tightness is guaranteed. The couplers are not used in every immersed tunnel in seismic area, but only when there is doubt whether the Gina’s can achieve water tightness at all times.

Figure 4.1-8: Worming effect on tunnel structure

This deformation of the whole structure will be resisted by the immersion joints and segment joints between the concrete segments. The immersion joints are formed by the GINA profiles and they are relatively flexible in comparison to the concrete elements and can handle a certain deformation. The segment joints behave more stiff because there is no rubber profile in between the segments. The focus of this research will be on the effects of worming on segment joints. This deformation mode will result in two load cases; compression and tension of the segment joint.

4.1.3.1 Axial compression
The compression of the segment joint is a positive load, because concrete acts better when it is compressed instead of under tension. Due to the fact that the immersed tunnel is situated below water level, an axial compression from the water pressure is already present. This water pressure combined with the pressure of the seismic wave, gives a certain amount of compression on the segment joint. The segment joint has to withstand this compression, so the restriction for this load case is the compressive strength of the used concrete.

Another phenomenon that takes place during axial compression is the loading and unloading of the axial force. The seismic waves generate compression and tension while travelling through the soil. This way compression and tension will both work on a segment joint. This causes loading and unloading of the axial force and hammering of the segments can occur. In this case the segments are hitting each other and bridling of the concrete can occur. This may reduce the strength of the segment joint.

4.1.3.2 Axial tension
The other effect that takes place during the worming effect is the axial tension of the joints. The immersion joints with the GINA-profile can bear no tension, because the profile is only connected on one side and sealed by compression due to water pressure. When tension occurs due to the worming effect on the immersion joint, first the water pressure has to be overruled but then joint opening could occur. To prevent this immersion joint opening, in seismic areas sometimes couplers are used to maximize the relaxation of the GINA-profile (Figure 4.1-9). This way the GINA-profile can be compressed and released to a certain level and joint opening is prevented. With this restriction of relaxation, water tightness is guaranteed. The couplers are not used in every immersed tunnel in seismic area, but only when there is doubt whether the Gina’s can achieve water tightness at all times.
In the segment joints the water tightness is usually guaranteed with the W9U-profile which is cast in the concrete on both sides of the segment joint. This way a connection is made between the two segments which can be tensioned and compressed. The pre-stressing cables for the transportation phase are cut after immersion of the tunnel element, so in final phase those cables are not functioning anymore.

The second load case due to the worming effect is the tension on the segment joint. The W9U-profile in the segment joint will hardly prevent this, since it has a very low tensile capacity. This capacity depends on the water depth and is investigated by the producer of the profile. It has to be investigated how much safety is included here and what tensile capacity is acceptable.

The problems as described above for the worming effect in an immersed tunnel are looked at in more detail in chapter 4.2.

4.1.4 Horizontal and vertical snaking
The other deformation mode that occurs due to the passing of seismic waves is the snaking effect. Snaking is the bending of the tunnel structure in lateral direction. This is caused by shear waves traveling in longitudinal direction of the tunnel (S-wave ($\phi = 0^\circ$)). These S-waves have a horizontal and a vertical component, so we can distinguish horizontal (SH-waves) and vertical
(SV-waves) shear waves. Similar to the waves, the deformation of the tunnel structure will be in horizontal and vertical direction. The principle of these deformations is similar for the horizontal and vertical deformation, but the impact forces on the tunnel structure and joints will be different for each situation.

Due to the deformation of the tunnel structure the segment joints will be loaded with bending and shear forces. Again we can distinguish two loading cases for the segment joints caused by the snaking effect.

4.1.4.1 Rotation of segment joint
The third loading case is the rotation of the joints due to the bending deformation. The rotation of the immersion joints can partly be taken by the GINA-profiles. The rotation of the segment joints will cause more problems, because there is no compressible rubber profile in between the segments to take the deformations. Only the sealing profile W9U is there to guarantee the water tightness. Joint opening could occur because of bending on the outside and compression of the segment joint on the inside.

The opening of the segment joint needs to be followed by the W9U profile similar to the situation with the tension load in the worming effect. The tensile capacity of the profile is the restriction for this joint opening. For the compression on the other side of the segment joint the compression strength of the concrete is decisive. The total pressure needs to be below the compressive strength of the concrete to prevent failure.
4.1.4.2 Shear load on segment joint

Due to the snaking effect, the tunnel structure will bend. Depending on the rotation stiffness of the joints the tunnel will follow the deformation and a certain moment distribution will be spread over the tunnel. Due to the deformation, a shear problem arises in the joints as drawn in Figure 4.1-13.

Figure 4.1-13: Shear load due to snaking

The shear load depends on the opening of the joint and the pre-stressing in axial direction due to the initial water pressure. This shear load needs to be transferred from segment to segment with the help of shear keys in the joint. The loading of these shear keys is the last load case and determines the shear capacity of the segment joint.

The problem described above, the rotation of the segment joint and the shear loading of the segment joint, are worked out in more detail in chapter 4.3. The capacity of different shear keys are examined in chapter Fout! Verwijzingsbron niet gevonden.

4.1.5 Modeling of tunnel behaviour

To predict the behaviour of the tunnel structure in the underground due to earthquake loading different models can be used. Common used tunnel behaviour models are the analytical model, the multi-mass spring model and the finite element model[11]. In these model series each model is more complex than the previous one, resulting in a more accurate approach of the tunnel behaviour.

4.1.5.1 Analytical model

The analytical model describes the tunnel structure as an infinite long elastic beam supported by an elastic foundation in the form of soil springs (Figure 4.1-14). The stiffness of these springs depends on the modulus of sub grade reaction of the subsoil. The seismic waves will deform the soil springs and the tunnel structure will respond to this based on its own flexibility. The limitation of this model is the fact that the beam is modeled continuously while the immersed tunnel is in discontinue in real time.

Figure 4.1-14: Analytical model [Kiyomiya, 1995]
4.1.5.2 Multi-mass spring model

The second model of tunnel behaviour to earthquake loading is the multi-mass spring model. For this model the surface is divided into planes perpendicular to the tunnel axis (Figure 4.1-15). Each plane has got a mass and is connected with the base rock by a spring and a dashpot. The soil layer above this base rock vibrates in 1st shear mode. All the planes are connected to one another along the tunnel axis with springs and dashpots. The spring constant is based on the push-pull resistance in axial or transversal direction depending on the response direction.

![Multi-mass spring model](image)

Figure 4.1-15: Multi-mass spring model [Kiyomiya, 1995]

4.1.5.3 The Finite element model

The finite element model is a numerical 3D-model to predict the tunnel behaviour due to seismic loading. The ground is modeled with block elements and the immersed tunnel is modeled using beam elements. In this model the response of the immersed tunnel and the surface layer are calculated at the same time including their interaction. The only disadvantage of this method is the complexity of the calculation; it needs large memory capacity of the computer.

4.1.5.4 Choice of model

The three models mentioned deal with the soil-structure interaction for the complete tunnel in seismic environment. The spring stiffness of the immersion joint is included in the spring model and the finite element model, but the segment joints are not taken into account. The elements are modeled as one piece pretending there is no segment joint. For the very stiff “Japanese” joint this can be accepted but if the joint is more flexible, it is not a good approach anymore. A model which gives more insight in the behaviour of both joint types is required.

For the modeling of the tunnel response in the following chapters the tunnel is modeled as a beam on elastic foundation. For the immersion joints and the segment joints the properties will
be determined to include the spring stiffness of these joints in the model. This beam model will be explained in more detail in chapter 4.2 and 4.3 for the worming and snaking effect.
4.2 Worming effect

4.2.1 Introduction

In previous chapters the worming effect is described as one of the tunnel responses that can occur due to seismic loading of an immersed tunnel. The worming effect is caused by both pressure waves (P-waves) and shear waves (S-waves) and results in the tension and compression of the tunnel structure in longitudinal direction. The tunnel structure will be deformed in longitudinal direction where in some region the tunnel is compressed and in other parts the tunnel is lengthened by tension. The idea is roughly sketched in Fout! Verwijzingsbron niet gevonden.

![Figure 4.2-1 Worming effect on tunnel structure (side view or top view)](image)

The worming effect can be divided into two problems as described in previous chapter. The two loading cases for the joints are:
1. Axial compression
2. Axial tension

The axial compression of the segment joints is caused by the compressing part of the seismic wave in the surrounding soil passing the tunnel. This is subjected from the soil by shear stress on the tunnel walls, roof & floor and the compressing of the joints. The axial tension in the tunnel is caused the same way by the pressure wave, but with tension. The tunnel and its joints are lengthened by this loading.

There are different approaches to describe the deformation of the tunnel structure due to passing seismic waves. Hashash et al. proposed two approaches to describe the behaviour of the underground structures under seismic loading[5]: free-field deformation and soil-structure interaction. With the free-field approach the ground strains by seismic waves are described with the absence of structures. In this approach the interaction between the underground structure and the surrounding ground is ignored. In the soil-tunnel interaction approach the interaction of the tunnel in the surrounding soil is taken into account. Here the tunnel stiffness and the ground stiffness are included.

4.2.2 Seismic design wave

It was stated in the paragraph 4.1 that the worming effect is caused by two types of seismic waves in the soft top layer where the immersed tunnel is situated. The two types of waves with their angle of inclination causing the maximal worming effect are:

- P-wave in longitudinal direction ($\varphi = 0^\circ$)
- S-wave on an angle of incoming (max. for $\varphi = 45^\circ$)
The propagation velocity of the P-wave is about two times higher than the propagation speed of the S-waves in the same medium. But the effect of the P-wave in comparison to the S-wave is much smaller. From different literature (Eurocode 8 and different seismic analysis of tunnel structures[14],[15]) it is stated that the S-wave is the decisive wave causing the worming effect and also the snaking effect for immersed tunnels.

The travelling speed of seismic waves varies depending on the soil properties. The propagation speed varies between 100 m/s for soft alluvial materials till 6000 m/s for hard rock like Basalt and Granite. Most of the immersed tunnels are placed in an earlier excavated trench of sand and covered later on with a sand layer or a rock bed. This whole package is below the water level, so it is fully saturated. In this case we are dealing with loose- till medium dense, saturated sand or partly clay layers.

As described in paragraph 4.1 the seismic waves will propagate from the source to the site through the hard bedrock and at site location causing the shaking of soft top layers. Here the S-wave will encounter refraction; it will be bended in the direction of the surface. Due to this refraction the waves will loose energy and with decreasing depth the soil stiffness will also decrease. As a result of those two facts the propagation speed of the S-waves will decrease as they approach the immersed tunnel. So for the soft top layers on top of the hard bedrock, the following properties of the S-wave are known[7]:

- S-waves velocity $C_s$: 100-350 m/s (soft alluvial soil layers)
- Design S-wave velocity: $C_s$: 200 m/s

The wave length $L$ of the S-wave depends on the predominant natural period $T$. For a soft layer on top of hard bedrock Idriss and Seed (1968) recommended for this period $T$:

$$T = \frac{4H}{C_s}$$

with $H$ = soil deposit thickness over rigid bedrock

From previous paragraph 3.3 the thickness of soft top layer is known: $H= +/-$ 50 m. This proposed thickness of the soft layer is not a fixed value. In other situation the thickness can be thicker, so $H= 50 – 150$ m. With this thickness, the natural period $T$ be calculated:

$$T = \frac{4H}{C_s} = \frac{4 \times (50 - 150)}{200} = 1 – 3 \, s.$$ 

Another way to determine the dominant period $T$ is to use a Fourier type approach with existing measurements. In Figure 4.2-2 [6] the Gilroy record is drawn for rock and soil type layer. The Gilroy earthquake is a representative earthquake for a lot of seismic design approaches because a lot of measurements are performed during this earthquake. The graphs show that for rock a smaller dominant period can be found than for soft soil. For soft soil the peak is between 1 - 2 seconds.
Power et al.[16] provides a relation between the peak ground velocity (PGV) and the peak ground acceleration (PGA) in the article of Hashash et al.[5] For soft soils the ratios are given:

\[
\frac{v_{max}}{a_{max}} = 140\text{–}270 \quad \text{with } v_{max} (cm/s) \text{ and } a_{max} (g)
\]

The acceleration \(a_{max}\) is given above in gravitation load \(g\). This can be rewritten in m/s\(^2\) by multiplying this with the gravitation acceleration of \(9,81 = 10\text{ m/s}^2\). The velocity is written in cm/s, so this can also be rewritten in m/s by multiplying the value with 0.01. The total relation is now 1000 higher than when it is written in seconds. In this way it counts:

\[
\frac{v_{max}}{a_{max}} = 0.140\text{–}0.270 \quad \text{with } v_{max} (m/s) \text{ and } a_{max} (m/s^2)
\]

The relation between the maximum velocity and the maximum acceleration determines the dominant period \(T\) if we neglect the damping factor. The acceleration function is the derivative of the velocity function, so the relation is:

\[
\frac{v_{max}}{a_{max}} = \frac{T}{2\pi} \rightarrow T = 2\pi \times \frac{v_{max}}{a_{max}}
\]

With the two formulas above the bandwidth for the predominant period \(T\) in soft soil can be determined:

\[
T = 2\pi \times \frac{v_{max}}{a_{max}} = 2\pi \times (0.140\text{–}0.270) = 0.88\text{–}1.70s.
\]

Also from this approach the predominant period is: \(T = 1\text{–}2\text{ s}\).

The wave length \(L_s\) of the S-wave can be calculated with the following formula:
\[ L_s = C_s \times T \]

For the worming effect a longer wave length has a negative effect on the loads on the tunnel, so for worming the biggest period is used.

Worming: \[ L_s = C_s \times T = 200 \times 2 = 400 \text{m} \]

For snaking a short wave length causes negative effect, so for snaking we use the following wave length:

Snaking: \[ L_s = C_s \times T = 200 \times 1 = 200 \text{m} \]

Those two wave lengths for the worming and snaking effect are used in the following chapters for calculations.

### 4.2.3 Free-field deformation approach

In the free-field deformation approach the deformation of the tunnel is equal to the deformation of the surrounding soil and the interaction is not taken into account. This approach is a good approach to get first insight in the deformation of the tunnel. In Figure 4.2-3 the free-field deformation along a tunnel axis due to a sinusoidal shear wave is given.

\[ u_x = D \sin \phi \sin(2\pi x/L \cos \phi) \]

\[ u_y = D \cos \phi \sin(2\pi x/L \cos \phi) \]

The axial deformation of the ground and in this approach also of the tunnel consists of a part with longitudinal strain and bending strain proposed by St John and Zahrah[17]:

\[ \varepsilon_x = \varepsilon_{axial} + \varepsilon_{bending} = \frac{V_s}{C_s} \times \sin \phi \times \cos \phi + r \times \frac{a_s}{C_s} \cos^3 \phi \]
With \( r \) is the half width of the tunnel, \( \phi \) is the angle of incidence with respect to the tunnel axis, \( V_s \) is the peak particle velocity for shear waves, \( C_s \) is the effective propagation velocity of the shear wave and \( A_s \) is the peak particle acceleration of the shear wave. The equation is worked out in detail in Appendix B.

This strain needs to be below the concrete strain of the tunnel structure. The tensile strain of the concrete is lower in comparison to the compressive strain, so the determined ground strain of the formula above needs to be below the tensile strain of the concrete:

\[
\varepsilon_s \leq \varepsilon_{\text{allowable}} = \varepsilon_{\text{concrete-tensile}}
\]

This deformation of the surrounding soil is the passage of seismic waves travelling from source to the site location. The design seismic wave for the further worming modeling is determined in previous paragraph:

- S-wave on an angle of incoming (max. for \( \phi = 45^\circ \))
- Design S-wave velocity: \( C_s \): 200 m/s

To determine the ground strains the seismic parameters are needed. As a reference the seismic parameters are used from the reference project Coatzacoalcos Tunnel (see paragraph 3.4). The parameters are:

- Peak Ground Acceleration (PGA) = 2,0 m/s²
- Peak Ground Velocity (PGV) = 0,35 m/s
- Peak Ground Displacement (PGD) = 0,2 m

With the properties of the decisive S-wave under an angle of 45 degrees as stated above, the axial strain can be determined for the free-field approach:

\[
\varepsilon_s = \varepsilon_{\text{axial}} + \varepsilon_{\text{bending}} = \frac{V_s}{2 \times C_s} + r \times \frac{a_s}{C_s^2} \frac{\sqrt{2}}{4}
\]

\[
\varepsilon_s = \frac{0,35}{2 \times 200} + 12,25 \times \frac{2,0}{200^2} \times \frac{\sqrt{2}}{4} = 0,00109 = 1,09 \%
\]

The capacity of the concrete tunnel depends on the tensile strain that can be subjected in the tunnel structure. The problem for the concrete structure is the allowable crack width, which may not be too large, to prevent corrosion of the reinforcement. The strain in the concrete depends on the strain that can be taken by the reinforcement steel. For crack width calculations the steel stress is mostly taken lower than the 435 N/mm² for FeB500, more around 200-220 N/mm² (Figure 4.2-4). In this way the allowable strain will also be half of the elastic strain limit, so \( \varepsilon_{\text{allowable}} = 0.85 \% \).
If we now compare the axial strain with the allowable strain, it can be seen that it does not fulfill the requirements:
\[ \varepsilon_{\text{ax}} \leq \varepsilon_{\text{allowable}} = \varepsilon_{\text{concrete-tensile}} \]

For this conservative approach the strain will work over the entire element length and causes deformation in the immersion joints. The total deformation will be:
\[ w = \varepsilon_{\text{ax}} \times l_{\text{element}} = 0,00109 \times 12000 = 13 \, \text{mm} \]

This is a very large deformation for an immersion joint which probably can not be taken by the Gina gasket.

The free-field approach is a very conservative approach for this situation. The immersed tunnel is situated in a relative soft soil layer, so a stiff construction in a loose environment. The assumptions for free-field approach are the other way around, a flexible tunnel which follows the surrounding moving soil. In this way a very conservative design is needed to fulfill the requirements.

In real time the tunnel structure will not follow the movement of the surrounding soil but will resist it and will partly deform due to the rubber Gina gasket in the immersion joints. So the free-field approach is not recommended for immersed tunnels in soft soils. For this situation a soil-structure interaction approach is a better approach, which is discussed in the following paragraph.

### 4.2.4 Soil-structure interaction approach

The moving soil around the tunnel structure will result in forces on the tunnel structure and will create deformation. The height of these forces depends on the friction between soil and the tunnel, the soil-tunnel interaction. In earlier studies on seismic effects on pipelines in the subsoil, the interaction is described with a slider/spring model (Figure 4.2-5). The soil has a linear behaviour till a certain limit is reached. Above this limit slip will start and the soil will behave plastic. The behaviour differs for different soil types, for sand the graph for the shear stress – displacement relation is drawn in Figure 4.2-5. In this graph the spring/slider behaviour can be recognized.
Slip between the structure and the surrounding soil occurs when a certain relative displacement is reached, the difference between the soil movement and the structure movement. If this relative displacement is above 10 mm the slip will start and the maximum shear stress $\tau_{\text{max}}$ is acting between soil and structure. This relative displacement limit of 10 mm is not depending on the diameter of the structure, it is a common used value for example with axial loaded piles (Source: K.J. Bakker & B. Everts).

The maximum shear stress along the tunnel walls depends on the normal stress on the tunnel due to the effective soil stress. The normal stress and the shear stress along the tunnel walls are drawn in Figure 4.2-7.

To calculate the maximum friction against a structure surface the Coulomb’s formula is used:

$$\tau_{\text{max}} = \sigma \times f = \sigma \times \tan(\delta)$$

with $\delta = \phi$ (conservative value for horizontal surfaces)[18]

The maximal shear stress can be determined: $\tau_{\text{max}} = \sigma \times f = \sigma \times 0.58$

The angle of internal friction $\phi$ differs for different soil types. In this approach the angle $\phi$ is chosen of 30 ° for loose sand. When the tunnel is situated in for example clay this angle will be different and also the cohesion factor should be included there.

The maximum shear stress depends on the construction depth of the tunnel, because with increasing construction depth the normal stress on the tunnel walls will increase too. Taking the stress on the floor, walls and top layer separately (Figure 4.2-7) or a representative average stress and multiply this with the tunnel circumference, the shear stress per running meter tunnel can be calculated.
The shear stress will act over a certain part of the tunnel, depending on relative displacement and so the stiffness of the tunnel. This can be explained by drawing the extreme situations where the present situation needs to be in between.

The seismic wave which caused the worming effect is the S-wave with an inclination angle of 45˚ as described earlier. This S-wave can be described as a sinus function with an amplitude and a wave length. In this approach we do not include the time part, because we are looking at a standing wave:

$$u_x = u_{x,\text{max}} \times \sin\left(\frac{2\pi \times x}{L}\right)$$

The S-waves approaches the tunnel axis under an angle, but to examine the worming effect of the tunnel, the ground movement in axial direction is needed. With the help of Figure 4.2-3 the soil deformation in axial direction can be described:

$$u_x = u_{x,\text{max}} \times \sin\left(\frac{2\pi \times x}{L_x}\right) \quad \text{with} \quad u_{x,\text{max}} = D \times \sin(\phi) \quad \text{and} \quad L_x = L / \cos(\phi)$$

The inclination angle $\phi$ of 45˚ causes the wave length to grow with $\sqrt{2}$.(Figure 4.2-9). The used wave length $L_x$ will now be:

$$L_x = L_x \times \sqrt{2} = 400 \times \sqrt{2} = 566 \text{ m} \approx 600 \text{ m}$$

To get a better modeling the axial wave length is increased till 600 m. In this way one entire wave is spread over 5 tunnel elements of 120 meters. This is drawn in Figure 4.2-10.
The soil movement along the tunnel is equal if we look from a zero crossing to one side, it is symmetrical. In this way we only have to look at one part of the tunnel, so at the half of a wave length. This is spread over 2,5 tunnel element.

The soil will move along the tunnel floor, roof and walls and cause the tunnel to deform depending on the axial stiffness \( EA \) of the tunnel. The difference between the soil deformation and the tunnel deformation, the relative displacement, determines the shear stress on the tunnel. This shear stress is the load on the tunnel and together with the tunnel stiffness \( EA \) determines the tunnel movement. And again this influences the relative displacement and thus the shear stress. It is an iterative process to calculate the final deformation of the tunnel structure. This process can be explained better with some figures. For this explanation we distinguish three situations:

- Tunnel structure is very stiff \( \rightarrow EA = \infty \)
- Tunnel structure is very flexible \( \rightarrow EA = 0 \)
- Tunnel structure is between stiff and flexible \( \rightarrow EA = EA_{\text{normal}} \)

In this approach the principle will be explained on a qualitative way, with only the stiffness of the concrete structure taken into account. The joint properties and their stiffness as they are in real situation are not taken into account.

**Stiff tunnel \( (EA = \infty) \)**

When the tunnel stiffness is infinite large, the tunnel will not move due to the loading and only the soil deformation is present (Figure 4.2-11). The soil deformation is directed to the right part of the tunnel. The relative displacement is in this situation equal to the soil movement (Figure 4.2-12).
Figure 4.2-12: Relative displacement in axial direction (stiff tunnel $EA = \infty$)

The relative displacement is almost over the entire part of the tunnel above the 10 mm, which was stated as the slip limit, so the shear stress is almost over the entire part maximal (Figure 4.2-13). As a result of this, the total normal force in the right part of the tunnel is caused by the maximal shear stress over the entire length, thus over $\frac{1}{2} L$.

Figure 4.2-13: Shear stress in axial direction (stiff tunnel $EA = \infty$)

Flexible tunnel ($EA = 0$)

When the tunnel stiffness is very low, the tunnel will follow the deformation of the soil and the tunnel will be deformed. The deformation of the tunnel follows the deformation of the surrounding soil completely. (Figure 4.2-14). The soil deformation will be the same as explained above.

Figure 4.2-14: Soil- & tunnel deformation in axial direction (flexible tunnel $EA = 0$)
The relative displacement is zero for this situation and also the shear stress will be zero, because these are direct related. In this situation no resistance of the tunnel is present, no shear stress takes place and the tunnel will act like it is equal to the soil and moves equal with the soil movement.

*Normal tunnel (EA = \( EA_{normal} \))*

When the tunnel stiffness is more in the direction of the actual stiffness of an immersed tunnel, the tunnel deformation due to the soil movement is in between the two situations as described above. The both deformations are plotted in Figure 4.2-15.

**Figure 4.2-15: Soil- & tunnel deformation in axial direction (normal tunnel EA= \( EA_{normal} \))**

When we now look at the relative displacement (Figure 4.2-16), again most of the part is above the slip limit and the maximum shear stress is present here. The shift of the shear stress in the left direction to the right direction is now moved more to the left compared to situation with the flexible tunnel. In that situation it was in the middle of this part of the tunnel.
The different stiffness modes as described above are summed up for the shear stress in Figure 4.2-18. When the tunnel stiffness decreases from infinite stiff to flexible the shear stress will change from directed to one side (right) into two-side directed. In Figure 4.2-19 the normal force also gives the same behaviour. If the tunnel is stiff, the normal axial force is only directed to the right, but when it behaves more flexible, it shifts also to the left part.

The position of the shift from negative to positive shear stress depends on the stiffness of the tunnel EA and can vary between 0 and \( \frac{1}{4} L \), the left and middle of this tunnel part. The movement can be expressed as a factor \( \alpha \) (Figure 4.2-20).
In the description above the tunnel was modeled as a solid concrete tube, without any joints in between, the simple tube approach with infinite length. In reality an immersed tunnel is build with immersion joints and segment joint, which both have their specific properties. These properties or better joint stiffness influence the total stiffness of the tunnel structure completely. In the simple model as described above the joint stiffness can not be included. For this purpose an elaborated beam model is proposed in next paragraph, but with the same principle as stated in this paragraph. In the elaborated model the factor $\alpha$ as described above will vary between 0 and $\frac{1}{4}L$ depending on the stiffness of the joints.
4.2.5 Axial beam model

4.2.5.1 Overall beam model

The tunnel structure is subjected to shear stress on the outside tunnel walls caused by the S-waves as described in previous paragraph. The shear stress works in two directions and causes the tunnel to deform as drawn in Figure 4.2-21. At one part the tunnel will be compressed and on the other part a joint opening can occur.

To determine the tunnel deformation and the joint opening the structure and its response needs to be modeled. This modeling will be done with a beam model loaded in axial direction. In this modeling the same approach is used as described in paragraph 4.2.4.

![Figure 4.2-21: Schematic shear distribution](image)

Because we are only modeling the axial loading the tunnel structure with the shear stress can be modeled with the axial beam model as seen in Figure 4.2-22.

![Figure 4.2-22: Axial beam model](image)

The beam model has 5 beams with the length of the tunnel elements enclosed by two fixed hinges at both sides. The two fixed hinges represents the compressed tunnel parts in which direction the tunnel will be deformed. These can be seen as static points.

The beams are supported by rolling hinges, because the only loading on the model is due to the shear stress $\tau_{\text{max}}$, the interaction between soil and tunnel. Between the beams there are Gina gaskets which are modeled as translation springs. Their spring stiffness depends on the initial compression and the compression or tension due to the seismic loading. This is described in paragraph 4.2.5.2. In the middle of the 5 beams the tunnel is tensioned and the rest is compressed due to the fixation at the end and the shear stress which is directed to the outside part. How much of the shear stress is working on the tunnel depends on the total stiffness of the tunnel structure including the joints, which is expressed with the alpha factor.
Due to the equality on both sides of the whole model we can focus on one side for further modeling Figure 4.2-23. This is also already done in paragraph 4.2.4 and will be used again in this axial beam model. The focus will be on the right part of the tunnel model in Figure 4.2-22. In this case the segment joints in the middle part will be tensioned and can be seen as translation springs with the spring stiffness which will be determined in paragraph 4.2.5.3. The rest of the joints will be compressed due to the rightward directed shear stress. The spring stiffness of those segment- and immersion joints will be determined in the following paragraphs.

![Figure 4.2-23: Axial beam model one-sided](image)
4.2.5.2 Immersion joint

In previous chapters and paragraphs the immersion joints are described as sealing Gina gaskets. In the Dutch situation these Gina gaskets are attached to one tunnel element and compressed by the other element due to the water pressure at immersion phase and total compression of the tunnel in final phase. In earthquake sensitive areas the immersion joint can be extended with a coupler to prevent the Gina from total relaxation. When this happens, leakage could occur and the tunnel can collapse. The couplers are tensioned at the final stage of finishing the tunnel and in this way there is tension included in the tendons.

During the phase when the tunnel is in use, the Gina gasket will encounter relaxation due to the properties of the rubber. This results in the decrease of tension in the tendons, but it will still be enough to compress the Gina gasket and prevent leakage. When the tunnel is loaded in axial direction during the passage of seismic waves, the immersion joint will encounter compression and tensioning. When the joint is tensioned, the coupler starts to work and the tendons are tensioned again.

![Diagram of Gina gasket with coupler](image)

**Figure 4.2-24: Gina gasket with coupler (Anastasopoulos, 2007)**

For immersion joints with couplers as also described in paragraph 4.1.3, Van Putten determined formulas for spring stiffness\[4]. The spring stiffness of the axial translation spring for the Gina gasket can be described with the following formula:

\[ k_u = 2k_0 \times \left( B_t + H_t \right) \]

This reflects the length of the Gina gasket times the initial spring stiffness \( k_0 \). This initial spring stiffness can be determined with the force-compression graph which is created by the manufacturer (Figure 4.2-25).
4.2.5.2.1 Compression Gina gasket

From the graph it can be seen that the gasket behaves more stiff when it is compressed more. The force on the gasket due to the water pressure in immersion phase gives an initial compression (red line). The tangent (pink line) in this point gives the initial spring stiffness $k_0$ for this situation. In every situation new spring stiffness can be determined because of the non-linear relation, but this would be too complex to model. For this reason the force-compression graph is simplified in a graph with 3 parts with different spring stiffness’s (Figure 4.2-26).

![Force-compression graph Gina gasket](image)

**Figure 4.2-26: Simplified force-compression graph Gina gasket**
The middle section represents the initial spring stiffness $k_0$. More in the direction of the axis's origin the gasket is subjected to relaxation, so a lower spring stiffness $k_{\text{relaxation}}$ is found. When the gasket is compressed more, the gasket acts more stiff, so a higher spring stiffness $k_{\text{compression}}$ is found. These different spring stiffness's are used for the worming modeling in different loading phases.

For different construction depths these simplified force-compression graphs differ as well. With the varying construction depths, the initial water pressure is different and so is the initial spring stiffness. The initial spring stiffness increases with increasing construction depths. The force-compression graphs for varying construction depths are given in appendix E. It can be seen that from the construction depth of 40 meter, the initial spring stiffness is almost equal to the highest compression spring stiffness. An overview of the spring stiffness's is given in Table 2.

### Table 2: Properties Gina gasket

<table>
<thead>
<tr>
<th>Water depth (m)</th>
<th>Initial normal force $F_0$ (kN/m)</th>
<th>Initial compression $c_0$ (mm)</th>
<th>Start relaxation part (mm)</th>
<th>Start compression part (mm)</th>
<th>Spring stiffness $k_u$ middle part graph (MN/m)</th>
<th>Relaxation capacity Gina (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>250</td>
<td>78</td>
<td>70</td>
<td>103</td>
<td>750</td>
<td>58</td>
</tr>
<tr>
<td>15</td>
<td>451</td>
<td>93</td>
<td>83</td>
<td>103</td>
<td>1200</td>
<td>73</td>
</tr>
<tr>
<td>20</td>
<td>585</td>
<td>98</td>
<td>83</td>
<td>106</td>
<td>2100</td>
<td>78</td>
</tr>
<tr>
<td>30</td>
<td>920</td>
<td>107</td>
<td>95</td>
<td>110</td>
<td>3700</td>
<td>87</td>
</tr>
<tr>
<td>40</td>
<td>1260</td>
<td>112</td>
<td>102</td>
<td>115</td>
<td>6000</td>
<td>92</td>
</tr>
</tbody>
</table>

The stiffness’s in the relaxation part and in the fully compressed part are fixed for varying water depths. For the modeling the following spring stiffness’s are used:

- $k_{\text{relaxation}} = 220 \text{ MN/m}$
- $k_{\text{compression}} = 6760 \text{ MN/m}$

All of the properties in Table 2 are based on the force-compression graph of a Gina ETS 180-220. This is a commonly used Gina gasket for immersed tunnels. Other Gina gaskets are the smaller ETS 130-160 and the larger ETS 200-260 SN. For these gaskets the force-compression graphs are given in appendix D. For these other gaskets the principle is similar and the spring stiffness’s can also be determined for the three different areas as described above, but in further modeling only the ETS 180-220 Gina gasket is used.

4.2.5.2.2 Relaxation Gina gasket

It was mentioned in previous paragraph that with the relaxation of the Gina gasket the spring stiffness decreases rapidly. If this relaxation becomes too large, and compression approaches the minimal required compression of the gasket, the water tightness of the joint can become critical. In this case the outside water pressure can result in water intrusion in the tunnel. To
prevent this from happening, a minimal compression in the Gina gasket is retained by using a coupler as mentioned in earlier paragraph. With this coupler the Gina gasket can be compressed freely and also relaxation can take place. When the relaxation become too large, almost in the direction of the minimal compression of the Gina, the coupler will prevent further relaxation. Below this minimal compression of the Gina the coupler couples the concrete elements and therefore no more relaxation can take place. For this relaxation pattern the same graph as for compression can be drawn. The force-compression graph for this relaxation path can be modeled as drawn in Figure 4.2-27.

When the maximum relaxation is reached and the coupler is starts working, the stiffness is much larger than the stiffness of the Gina gasket. That is the reason of the vertical propagation of the relaxation path, relaxation is prevented by the coupler and tendons. In reality the line will be some what inclined because the tendons and coupler also aren’t infinitely stiff, but much larger than the relaxation stiffness of the Gina gasket. But also the tendons and couplers have got a certain capacity, which need to be checked. This capacity can be increased by increasing the number of tendons.

\[
\begin{align*}
\text{Figure 4.2-27: Relaxation of Gina gasket with coupler}
\end{align*}
\]

In the first part of the graph in Figure 4.2-27 where relaxation takes place, the Gina gasket can be modeled as a translation spring with constant spring stiffness. For this part the model as described in paragraph 4.2.5.1 consists of the translation spring for the Gina gasket on tension. This modeling is permitted until the coupler starts to prevent the relaxation.

When the coupler is working the spring stiffness is very large compared to the spring stiffness in previous modeling. In this situation the tendons of both concrete sections are coupled and stiffness depends on the steel properties. This stiffness is very large, so the connection can be modeled as a fixed connection.
4.2.5.3 Segment joint

In paragraph 2.3 the segment joint is described as loose concrete sections connected with a W9U sealing profile. This joint is cast concrete to concrete, so if this joint is compressed, this joint will simply act as concrete under compression. When this joint is tensioned the only connection that is working is the sealing profile W9U. In this joint no coupler is used because there is no space available and it will probably be too expensive to apply all of the segment joints in the tunnel with these couplers. This W9U profile will act as a translation spring until the tension capacity of the profile is reached. The spring stiffness of this sealing profile is very low. This tension/compression behaviour can be drawn in a force-displacement graph (Figure 4.2-28).

![Diagram of force-displacement graph W9CU](image)

**Figure 4.2-28: Force-displacement graph W9CU**

The segment joint of a tunnel in use is compressed due to the initial pre-stressing caused by water depth. We start here with an initial compression force $F_0$. If the segment joint is compressed more, this would be no problem because the concrete can have a lot of compression. When the joint is tensioned, first relaxation takes place till the origin of the axis (see arrows). From that point the sealing profile starts to stretch and it can be seen as a translation spring in axial direction. The profile can elongate till a maximum elongation based on the water depth and initial gap given in the graph of the manufacturer (Figure 4.2-29). For immersed tunnels this gap is always zero and the water depth from 0 till 40 m (0-0.4 MPa).

The maximum elongation for this profile in ULS condition given by the manufacturer is 70 mm. Till this elongation the water tightness of the joint is guaranteed by the manufacturer. Above this value it is possible that tear of the profile could appear with leakage as consequence. The profile can elongated much more, the material has an elongation capacity of 300%, but manufacturer guarantees no water tightness above 20% elongation. The maximum tension force for this profile can be calculated:

- Tensile strength W9CU: $\sigma = 4$ MPa = 4 N/mm²
- Working area = Perimeter x thickness : $A = O \times d = 63600 \times 13 = 826800$ mm²
- Tensile force: $F_{\text{max}} = \sigma \times A = 4 \times 826800 = 3307200$ N = 3.3 MN
- Spring stiffness: $k = F/U = 3.3/0.07 = +/50$ MN/m
From Figure 4.2-29 it can be seen that the thickness of the profile is 13 mm which is used for the tension capacity in the calculation above. In real time the profile will be stretched and the thickness will be reduced. Because of this the thickness in the calculation is taken too positive and the spring stiffness too high. In reality this stiffness will be smaller, say half of the calculated stiffness.

Now the behaviour of the sealing profile is known, it can be included in the model. For the first part of the graph the spring stiffness is equal to the concrete, very high compared to the stiffness of the sealing profile. In the second part the stiffness is constant (tangent of graph = +/- 25 MN/m). In the last part there is no stiffness left, so there it is zero. These different stages of stiffness of the segment joint are included in the modeling as described above.

If we look at the model (Figure 4.2-23) with the tunnel elements, in all elements the segment joints are situated. Because of the axial loading in one direction most of the segment joints are compressed. In this way they just function like concrete connections as described above and can be left out in the model. In the first tunnel element from the left side a part will be tensioned due to the displacement due to the compression of the right part. This part is compressed due to the shear stress on this part of the tunnel. In the tensioned part the segment joints must be included and given the properties belonging to a tensioned W9U profile. The results of the above described worming model with joint properties are described in paragraph 4.2.6.
4.2.6  Response of axial beam model

The axial beam model for the worming effect as described in paragraph 4.2.4 is built in the framework program SCIA Engineer. In this model the soil-tunnel interaction approach handled in paragraph 4.2.4 is included. In the model the concrete tunnel elements are modeled as beams and the properties of the joints are also included as springs (Figure 4.2-31). On the right side the model is fixed with a fixed hinge. The other two hinges are rolling hinges to make sure that the loading is only axially directed. The only load on the beam model is the shear stress in axial direction. This stress is directed in rightward direction to the fixed hinge (Figure 4.2-32). On the left side the tunnel is attached to the other half part of the tunnel, which are symmetrical parts. Due to the fact that they are symmetrical, the left part of the tunnel is also fixed. It is attached to the other tunnel part with a W9U profile with low spring stiffness (Figure 4.2-30).

Figure 4.2-30: Axial beam model in two directions

Figure 4.2-31: Axial beam model with tunnel segments (one direction)

Figure 4.2-32: Axial beam model in framework program

To make a simple and understandable model only linear springs are included in the model. This way the loading and behaviour of the model can be noticed directly and changed if needed. It was seen in paragraph 4.2.5 that the Gina gasket and the W9U profiles don’t have linear spring stiffness, but behave different when they are compressed and tensioned. In the modeling the joints are taken with a fixed spring stiffness based on the fact if they are compressed or tensioned and in what range the load will be.

The beam model consists of 2.5 tunnel elements separated by Gina gaskets. The tunnel element that is split is taken at the left side, resulting in a Gina gasket in the upper right part of the model. This is done because the W9U profiles in the left part have the lowest tension capacity, thus be the weak link. Only a small part on the left part will be tensioned, so this will be the most undesired situation for the worming effect. If the segment joints were taken on the right side, they were compressed and functioned as concrete joints, so very stiff. In that situation the Gina gasket was on the left end of the model, which has higher tension stiffness than the W9U profile, so not the most undesired situation.

The only load on the beam model is the shear stress. As described in paragraph 4.2.4 the shear stress is a result of the relative displacement between tunnel and surrounding soil. It is an iterative process, because the shear stress influences the tunnel movement and the tunnel movement together with the soil movement influences the shear stress. With this iterative
The process the alpha factor is determined at what point the shear stress starts to work from left to right.

In this model first a representative length is taken for the shear stress and checked what the resulting tunnel displacements were. With this tunnel deformation and the fixed sinusoidal ground deformation as described in paragraph 4.2.4 the relative displacement over the whole beam model can be determined which determines a new shear stress distribution. This process is repeated a couple of times until the situation is fixed. The response of the model is given in Figure 4.2-33 and Figure 4.2-34 in the form of normal force and displacements of the joints.

**Figure 4.2-33: Normal force N in axial beam model (kN)**

**Figure 4.2-34: Displacements in axial beam model (mm)**

It can be seen that the biggest part of the beam model is under compression (red) and only a small part on the left side is under tension (blue). For this part the segment joints which are under tension need to have the properties of the sealing profile W9U (low spring stiffness). The segment joints which are subjected to compression in the red area have the properties of a normal concrete cross section (high spring stiffness). The two Gina gaskets which are situated in the compression zone (red), the two most right hinges, have the spring stiffness for full compression (see paragraph 4.2.5.2). The last Gina gasket is in the tension zone, so this joint has the relaxation spring stiffness in this beam model. This stiffness is still high in comparison to the stiffness of the W9U profiles.

All the segment joints which are not subjected to shear stress are situated in the blue tension zone on the left side. These segments need to be able to follow the elongation of the tunnel deformation. The tunnel is compressed over certain part due to the shear stress and this result in an elongation on the left side. This elongation needs to be redistributed over the W9U profiles with their low spring stiffness. It can be seen that the elongation of about +/- 120 mm is equally spread over the first three segment joint, each taking an elongation of +/- 40 mm. The water tightness of the W9U profiles is guaranteed till a elongation of 70 mm, so in this situation the tunnel will fulfill the requirements.
The tunnel deformation as calculated with the model in SCIA, is used in the analytical approach (paragraph 4.2.4). Again the difference between the soil deformation and the tunnel deformation is determined (Figure 4.2-35). It can be seen in the first 60-70 meters from the left that the relative displacement is varying from negative to positive a couple of times over this part (Figure 4.2-36). This means that the shear stress over this part is also changing from direction in left and right over this part (Figure 4.2-37). It is spread evenly over this part, so it can be stated that it shear stress is damped out due to this change of direction. For this first 60-70 meters the shear stress is taken zero. Over the rest working to the right part the shear stress is maximal, because the relative displacement is above the slip limit of 10 mm.

In Figure 4.2-37 it can be seen that the height of the soil deformation does not make a big difference for the shear stress on the tunnel and indirect on the worming effect. On a large part of the tunnel slip occurs and the shear stress is maximal. If the soil deformation is increased, the
slip remains and the shear stress remains the same. So the magnitude of the soil deformation, the amplitude of the S-wave, is in this case not a decisive parameter for this worming effect. This will be different when the soil- and tunnel deformations are small. If the deformations are in the range below the slip limit, so between 0 and 10 mm, an increase or decrease of the wave amplitude can make a difference.

The axial loading on the tunnel causes tension and compression in the structure. The tension in the sealing profiles determines the capacity of the tunnel. In the middle element which is tensioned (Figure 4.2-30), the element consists of 5 segment joints. In this way the tunnel can have a total elongation over one wave length of 5 x 70 = 350 mm. In the model only one side is checked, so on one side the total elongation can be 175 mm. This is the total compression what can be compressed in the Gina’s in the right part of the model.

In the compressed part the normal force increases from left to right (Figure 4.2-31). The maximum normal force at the upper right side is 444.000 kN. The compressive stress in the tunnel due to this normal force need to be below the maximum stress. Common used concrete class for immersed tunnels is C28/B35, with a compressive stress of 21 N/mm$^2$. The compressive stress in the model can be calculated:

$$\sigma = \frac{N}{A} = \frac{444 \times 10^6}{77.35 \times 10^6} = 5.74 \text{ N/mm}^2$$

The compressive stress is far below the maximum concrete stress, so this is not a decisive part in the worming behaviour. The compression can be increased much more, but the concrete will not fail. The compressive stress used in the above check is valid for static loading conditions. For dynamic loading in the norms this value is commonly increased with a factor for accidental loading. In this way some more capacity for the tunnel structure is present.

A part which can become critical is the compressed Gina’s in the immersion joints. These Gina’s will become stiffer if they are compressed more. This is a positive property, because than the relative compression decreases and the elongation at the left side will not become too big. But it is not checked in paragraph 4.2.5.2 what the maximum compression or force is for the Gina. It is possible that the rubber will start to behave plastic above a certain value. This can be examined in more detail in further studies.

The maximal shear stress which will be subjected to the tunnel depends on the effective soil pressure on the tunnel roof, floor and walls. If the overburden on the tunnel is increased, the effective stress will increase. In the calculation above a standard overburden on top of the tunnel wall is taken of 2 m. This results in an average effective soil stress of 60 kN/m$^2$. An overburden of 2 meters is realistic coverage for an immersed tunnel in the middle of a river/sea crossing. At the abutments the overburden can be higher if the tunnel needs to cross the dikes. In that case the overburden will be higher and the shear stress will also increase. But this is only a local increase and not decisive for the full length of a tunnel.

The design parameter that makes a difference for the worming effect is the wave length of the incoming wave. If the wave length increases, the shear stress is spread over a larger tunnel part and more forces can be subjected on the tunnel. The wave length of the S-waves depend on the...
propagation speed in the tunnel. If a soil layer is at relative short distance from the surface, than it mostly consists of loosely packed soil and the propagation speed for these layers is low (150-350 m/s). In deeper layers the soil is more densely packed and the propagation speed is higher here (400-600 m/s). In this way the worming effect will increase with increasing construction depth.

In this model for the worming effect the tunnel is checked for a static situation, the tunnel is lengthened in one form. In reality the tunnel is subjected to a passing axial wave and the opening and closing of joints take place all along the tunnel axis. Because of the high propagation speed of the passing wave the opening and closing of the joint take place very fast. Thinking of this fast movement the static approach used in the calculations above is probably a bit conservative. The actual joint openings will probably be a bit lower due to the mass inertia.

Also due to the fast wave propagation another problem arises. The closing of the segment joint will go fast resulting in the hammering of the concrete sections against each other. This can have impact on the outside and interior of the concrete sections. If this phenomenon occurs and what the size of the impact is, will not be worked out in detail in this research. It is only said that the problem can arise with this worming effect. A possible solution for this hammering can be a bearing to absorb the movements (Figure 4.2-38). These bearings are common used for vertical movement at bridges. These bearings are made of rubber and have steel plates inside to take the outside directed tension forces. For the tunnel these bearings used in vertical direction, but for the tunnel these bearings can be used in horizontal direction. The only problem will be to find the available space between the tunnel segments, because these are in most cases cast concrete to concrete.

Figure 4.2-38: Bridge bearings
4.2.7 Conclusions on worming effect

The worming effect of one entire axial wave is spread over the length of 600 meters, so over 5 tunnel elements. The next 5 elements the whole model is repeated equally, so only one part of 5 elements is examined. From the calculations and the iterations of the analytical approach performed in paragraph 4.2.6 it is clear that the amplitude of the axial directed wave has no influence on the worming effect. If the amplitude is increased the shear stress on the tunnel is limited and the tunnel will not be compressed or tensioned more. Only if the relative displacement is in the range between 0 and 10 mm, where the maximal shear stress is not reached yet, the amplitude of the incoming wave has an influence. Than the shear stress can increase till the maximal shear stress and the worming effect will be influenced. But when the maximal shear stress is finally reached, there is no difference any more and the tunnel will remain in its final deformation.

In contrary to the amplitude of the S-wave, the effective stress on the tunnel roof, floor and walls does have an effect on the worming behaviour of the tunnel. If the overburden on top of the tunnel is increased, the effective stress increases and finally the maximal shear stress will increase. This will result in larger forces, more compression and more tension in the segment joints in the tensioned part of the tunnel. This overburden can be increased until the maximal elongation of the W9U profiles in the segment joints is reached. Around the abutments where an immersed tunnel is situated below a dike, this overburden will be larger and therefore this will be a critical area. But this is outside the scope of the research; the focus for this research is on the immersed tunnel part below the river or sea.

The length of the incoming S-wave also determines the length of the wave parallel to the tunnel axis. The larger this wave length, the larger the maximal shear stress is spread over one tunnel part. In deeper layers the soil is denser and in these layers the propagation speed of S-waves will be higher. The direct relation with between propagation speed and length will result in a bigger wave length and in this way more loading of the tunnel and increase of the worming effect.

It was seen in the analytical approach in paragraph 4.2.4 that with the infinite stiff tunnel that the shear stress will work over the over exactly ½ L of an axial wave (alpha ->0). This results in more compression in the compressed part and elongation of the segment joints. If the number of flexible joints (Gina) is decreased (larger tunnel elements), the tunnel stiffness increases. But the compression part needs to be taken by a smaller number of Gina’s and their stiffness increases with increasing compression. In this way the compression will converge to a maximum or the Gina’s will reach there capacity.

The compressive strength of the concrete is checked for this static approach and it fulfills the requirements. The concrete tunnel parts can take the loads easily. It can be increased quite well, about 3 to 4 times. So this is not the decisive parameter in the design. What will be interesting is the effect of the dynamic behaviour. The segment joints will be compressed and tensioned rapidly and in this way hammering could occur. The effect of this for the interior of the concrete, brittle around the reinforcement for example, is outside the scope of this research.
4.3 Snaking effect

4.3.1 Introduction

From paragraph 4.1.4 it was clear that besides the worming effect the snaking effect is an important tunnel response, which has a major influence on the tunnel joints. The snaking effect can be separated in horizontal and vertical snaking, the direction where the tunnel will be bended in due to the passing seismic waves. Also the snaking effect depends on the stiffness and behaviour of both tunnel joints, the immersion- and the segment joint. The joint properties are determined for the snaking effect in this chapter, especially the rotation stiffness, to be able to model this tunnel response.

4.3.2 Seismic design wave

The snaking effect will move the tunnel in lateral direction resulting in forces and stresses in the tunnel structure. To determine these forces and eventually the loading of the segment joints, the structure and its response needs to be modeled. This modeling will be done with a same sort of beam model as used with the worming effect, but here the loading will be lateral directed.

For this modeling the governing seismic wave is needed. The governing seismic wave that causes the snaking has a certain wave length based on the propagation speed and the frequency. A conservative and reasonable design value for apparent propagation velocity is 2000 fps (600 m/s) to 3000 fps (900 m/s)[19]. Also in the seismic design of the reference project Coatzacoalcos a propagation speed for S-waves is $C_s = 900 \text{ m/s}$[20]. The typical frequency for these waves is around 0.8 Hz, so $T = 1 – 1.5$ s. The wave length $L$ for this seismic wave will be:

$$C = 600 – 900 \text{ (m/s)}$$

$$T = 1 – 1.5 \text{ (s)}$$

$$L = C*T = 600 – 1200 \text{ (m)}$$

For the modeling in coming paragraphs the propagation speed $C_s = 900 \text{ m/s}$ and the dominant period $T = 1 \text{ s.}$ is taken as a conservative approach. The wave length for further modeling will be:

$L_{\text{wave}} = 900 \text{ m}$

In paragraph 4.2.2 a more detailed approach to determine the design wave was used. In that paragraph it was stated that the design S-wave has the following properties:

- S-waves velocity $C_s$: 100-350 m/s (soft alluvial soil layers)
- Design S-wave velocity: $C_s$: 200 m/s
- Predominant period: $T = 1 – 2$ s.

This resulted in the following wave lengths for worming and snaking:

Worming: (large length = negative)  
$$L_s = C_s \times T = 200 \times 2 = 400 \text{ m}$$

Snaking: (small length = negative)  
$$L_s = C_s \times T = 200 \times 1 = 200 \text{ m}$$
This knowledge was gathered after the modeling for snaking was already performed and calculated. So the following model for the snaking effect is performed with the first stated wave length $L_{\text{wave}} = 900$ m. This will result in a bending what can be taken quite well by the tunnel. Later on the modeling is checked with the new wave length $L_{\text{wave}} = 200$ m and this has enormous effects on the tunnel behaviour. This difference is explained in more detail in paragraph 4.3.6.

### 4.3.3 Beam model for snaking

#### 4.3.3.1 Modeling of whole tunnel

In this model a wave length $L_{\text{wave}}$ of 900 m is used as determined in previous paragraph. We now look at a part of an immersed tunnel of 8 elements with an element length of $6 \times 20 = 120$ m. This way the wave length is equal to the length of almost 8 tunnel elements. In this model the connection to the abutments is not taken into account, only the part of the tunnel which is situated below the river or sea which is crossed. The support stiffness of the abutments is most of the time higher than below the river due to foundation at the abutments. The variation between those support stiffness causes high shear loads in the tunnel, but this will not be modeled in this research. The connection with the abutments is outside the scope of the research.

![Wave length and immersed tunnel element](image)

**Figure 4.3-1: Wave length and immersed tunnel element**

The part that is subjected to half a wave, 4 elements, can be modeled as a beam on supports with a subjected displacement due to the ground movement. Resistance of tunnel structure due to ground movements depend on the flexibility of the tunnel. When the tunnel is continue and there are no joints, the flexibility depends on the bending stiffness $EI$. This modeling is drawn below:

![First modeling of immersed tunnel part](image)

**Figure 4.3-2: First modeling of immersed tunnel part**

The subjected displacement causes bending of the tunnel structure and a certain moment pattern over the tunnel. This moment and shear load needs to be carried by the tunnel structure.
4.3.3.2 Immersion joints in tunnel structure

In reality the tunnel structure is not monolith over half the wavelength as described in the first model, but it has immersion joints between the elements with rubber Gina gaskets. These Gina gaskets are very flexible in comparison to the concrete elements. In this way the immersion joints can be modeled as rotation springs.

![Diagram of immersion joints](image)

Figure 4.3-3: Modeling of snaking with immersion joints

The rotation stiffness of the springs depends on the behaviour of the rubber Gina gasket. This bilinear behaviour depends on the initial compression and the length of the profile, so the tunnel dimensions width $B_t$ and height $H_t$. Rubber has the property to behave more stiff when it is compressed more. In Figure 4.3-4 the force-compression curve for the common used ETS 180/220 Gina gasket is given and the non-linear behaviour can be seen here.

![Force-compression curve](image)

Figure 4.3-4: Force-compression curve ETS 180/220 GINA profile
The Gina gasket has a certain compression due to the water pressure at construction depth. The
time per meter caused by this water pressure can be read in the graph and gives the initial
compression. The tangent (pink) at this point represents the initial stiffness $k_0$ which can be used
to determine the rotational spring stiffness. So for different construction depths the joint will
behave different. In a deeper situated tunnel this means more compression and a stiffer
behaviour of the immersion joint.

For immersion joints with couplers as described in paragraph 4.1.3, Van Putten determined the
formulas for spring stiffness\[4\]. The axial loading the immersion joint can be seen as a
translation spring. The snaking effect in horizontal and vertical direction can be seen as a
rotation spring. The formulas for spring stiffness depending on the tunnel dimensions $B_t$ and $H_t$
are:

\[
k_u = 2k_0 \left( B_t + H_t \right) \\
k_{\psi_h} = B_t^2 k_0 \left( B_t / 3 + H_t \right) \\
k_{\phi_v} = H_t^2 k_0 \left( H_t / 3 + B_t \right)
\]

The design formulas are explained in detail in appendix F. With these formulas the rotation
springs can be determined for the snaking model; the horizontal and vertical snaking with a
changing parameter water depth. In this way a new model is created with an adapted flexibility
which can be compared to the monolithic modeling in paragraph 4.3.3.1.

4.3.3.3 Segment joints in tunnel structure

In the previous model the properties of the joints between the concrete segments are not taken
into account. In this model the joints are looked at as a monolithic connection. Because the
tension cables at the segment joints are commonly cut after immersion, this assumption is not
right. The segment joints are not equipped with a Gina gasket like the immersion joint, but are
cast concrete to concrete. With this fact the segment joints should be modeled as hinges. But
the water pressure causes a new pre-stressing in axial direction of the tunnel and so the joint is
compressed. The rotation of the joint now depends on the elastic
behaviour of the concrete. So
we can model the segment joint with a rotation stiffness which is a bit higher compared to the
stiffness of the immersion joint. This leads to the following model:

![Figure 4.3-5: Modeling of snaking with immersion- and segment joints](image)

The spring stiffness for the segment joints is not a linear relation, but depends on the
compression due to water pressure and the joint opening. This relation needs to be put in to
another model to take into account the joint opening and reducing contact area in the segment
joint. This results in a M-$\phi$ relation for the joint and is worked out in more detail in chapter
4.3.4. The spring stiffness defined with the M-$\phi$ relation will be used afterwards in the beam
model.
4.3.4 M-φ relation of segment joint

4.3.4.1 Principle of joint opening

The segment joint is compressed due to an axial pre-stressing force caused by the water pressure. When this joint is subjected to a bending moment due to seismic loading this compression will decrease on the tension side. When the stress becomes zero, the joint will open on this tension side. This will shift the neutral line and the behaviour of the joint becomes non-linear. When the bending moment is still small and the joint is not opened yet, the joint will behave linear depending on the elasticity of the concrete connection. To describe the complete behaviour of the segment joint a M-φ relation is required.

The segment joints in the Dutch situation, as mentioned in the literature study, have shear keys in the roof and bottom slab of the tunnel. In more seismic areas these shear keys are situated in the inner walls of the tunnel, the so called intermediate situation. In the modeling of the segment joint to determine the M-φ relation these shear keys are not taken into account, but a flat joint is used (Figure 4.3-6). In this situation the contact-area in the cross-section approaches reality and therefore it is accepted. The shear keys start working to resist the shear force when the joint is opened. Before that the shear force is transferred by friction on the contact-area between the segments. The transfer of shear force on the shear keys is discussed in later stage.

![Figure 4.3-6: Modeling of flat joint](image)

The influence area of the joint is half a segment length on both sides of the joint. So the joint is influenced by one segment width (Figure 4.3-7). The next segment joint will entail the next influence area.

![Figure 4.3-7: Influence zone per segment](image)

4.3.4.1.1 Behaviour phases

The segment joint is subjected to a normal force in the form of water pressure. This normal force is in the final situation reduced due to the relaxation of the GINA-profiles in the immersion
joints. This relaxation can lead up to 45-50% compared to the immersion phase. Therefore in the final situation where the tunnel is subjected to seismic loading, only 50% of the original normal force is taken into account.

Besides the normal force, a bending moment is caused by a water pressure difference and the segment joint is also subjected by an increasing bending moment due to seismic loading. The behaviour of the joint with an increasing bending moment can be divided in three phases:

- Linear elastic
- Non-linear elastic
- Non-linear plastic

For each phase the bending moment is increased and the joint rotation $\phi$ is calculated in small steps.

4.3.4.1.2 Linear elastic

In this phase the joint is compressed by the normal stress of the water. The bending stress due to the water is constant and is smaller than the normal force on tension side. The bending stress due to soil movement of the passing seismic wave is also small and is build up step by step. This bending tension is increased until it is equal to the normal compression. Up to this point the segment joint is closed. This phase with an increasing bending moment gives a linear relation in the $M-\phi$ graph.

**Figure 4.3-8: Linear elastic phase**

4.3.4.1.3 Non-linear elastic

When increasing the seismic bending moment the tensile stress becomes larger than the compression stress of the water. Because the reinforcement is cut, the joint can not take tension force and opening of the joint will begin. Due to this opening the contact area will decrease and the stress in the remaining contact area will increase. The neutral line will shift down.
Due to this decreasing compression zone the $M - \phi$ relation is not linear anymore, but it will act quadratic. With an increasing moment, the rotation of the joint will increase more. This can be seen in the $M - \phi$ graph in paragraph 4.3.4.2.

4.3.4.1.4 Non-linear plastic

With an increase of the bending moment, the compression strength limit of the concrete can be reached in the compression zone. When this happens, the concrete start to behave plastic. This can be seen in the figure below. This shows the situation where the rotation increases, but the moment is not anymore, but remains constant.

The behaviour of the segment joint as described above is true for the movement in vertical direction and it gives the vertical rotation stiffness of the joint. The approach for horizontal behaviour is equal with as the only difference that the water pressure is equal over the entire width of the joint. So there is no bending moment due to the water pressure difference, only due to the seismic loading.

4.3.4.2 $M - \phi$ graph

For the different phases as described in the previous paragraph the moment and joint rotation can be calculated step by step in a spread sheet. The parameter water depth is kept constant, so a constant axial normal force is working on the joint. For a water depth of 10 meters the $M - \phi$...
graph is drawn for the rotation in vertical- and horizontal direction. The graph is separated by a pink line in the linear part and the non-linear part.

The first part of the graph is linear and so a constant spring stiffness of the joint can be found here. In the non-linear part the tangent on the graph decreases, so the spring stiffness decreases also.
The difference between the graphs of vertical and horizontal rotation is that the horizontal graph has a bigger moment capacity and transformation from elastic to plastic behaviour takes a longer path. This is caused by the difference in height of the joint and the moment of inertia. The joint height for horizontal direction is 2.5 times bigger due to the height/width ratio and the moment of inertia is 4.6 times bigger in comparison to the vertical joint calculation (Figure 4.3-13).

When the water depth is increased, the axial compression also increases and so a bigger bending moment can be accepted for the joint. For the water depths 10, 15 and 20 meters the $M-\phi$ relation is drawn for the vertical (Figure 4.3-14) and horizontal direction (Figure 4.3-15).

![Figure 4.3-14: M-phi relation for 10, 15 and 20 m waterdepth (vertical plane)](image-url)
For different water depths the slope of the linear part is equal, but the length of the linear part is bigger. This way the moment capacity is higher for larger water depths. For the modeling of the segment joint the transformation from the elastic part to the plastic part is too complex to implement. For the snaking mode of paragraph 4.3.3.3 the behaviour of the segment joint is simplified as a bi-linear relation (Figure 4.3-16).

This way the constant stiffness for the elastic part can be determined for the different water depths and a maximum moment. At this maximum moment the joint will start to act as a plastic hinge, where rotation increases but the moment stays constant. The properties for the bi-linear relation for horizontal and vertical rotation are given in Table 3 and Table 4.

**Vertical rotation**

Table 3: Vertical segment joint properties

<table>
<thead>
<tr>
<th></th>
<th>Elastic</th>
<th>Plastic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint stiffness $k_\phi$ (kNm/rad)</td>
<td>$M_{max\ 10\ m}$ (kNm)</td>
<td>$M_{max\ 15\ m}$ (kNm)</td>
</tr>
<tr>
<td>$1.61*10^3$</td>
<td>75.000</td>
<td>125.000</td>
</tr>
</tbody>
</table>
### Horizontal rotation:

**Table 4: Horizontal segment joint properties**

<table>
<thead>
<tr>
<th></th>
<th>Elastic</th>
<th>Plastic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint stiffness $k_{\phi}$ (kNm/rad)</td>
<td>$7.4 \times 10^9$</td>
<td></td>
</tr>
<tr>
<td>$M_{\text{max}}$ 10 m (kNm)</td>
<td>200.000</td>
<td></td>
</tr>
<tr>
<td>$M_{\text{max}}$ 15 m (kNm)</td>
<td>325.000</td>
<td></td>
</tr>
<tr>
<td>$M_{\text{max}}$ 20 m (kNm)</td>
<td>450.000</td>
<td></td>
</tr>
</tbody>
</table>

The joint stiffness for the horizontal and vertical rotation is high compared to the stiffness of the immersion joint. This way most of the movement will take place in the immersion joints and not in the segment joints. If the tunnel is situated deeper, the water depth and pre-stressing of the tunnel increase. The stiffness of the GINA-profile increases and with it the rotation stiffness of the immersion joint. So with greater water depths the tunnel structure behaves more stiff. For these greater depths the same $M - \phi$ relation can be found, only with a larger elastic part, but with the same joint stiffness.

The moment capacities for the different water depths are high with a minimum of 75.000 kNm for vertical rotation. It is expected that the moment in the tunnel structure will be below the moment capacity. This way the segment joint will behave in the elastic part of the $M - \phi$ graph and the plastic behaviour would not be reached. This will be further examined in the next chapter.

#### 4.3.4.3 Joint opening

Equal to the determination of the $M - \phi$ relations, the opening of the segment joint can be calculated. In the elastic part of the rotation the joint will be closed by the water pressure, the joint opening is zero here. After this stage the joint will open and increases more and more when the moment increases constant. This can be seen in the graphs below for the joint opening in vertical and horizontal direction.

**Figure 4.3-17: Moment-opening (vertical joint)**
It can be seen in figure 27 and 28 that the joint opening is relatively small when it stays below the moment capacities which are determined in the previous paragraph. In that situation the opening will not exceed 5 to 10 mm. The tension capacity of the W9U rubber profile that is mostly used, is more than 10 times this elongation depending on the water pressure. Therefore opening of the joint should not be a problem if the moment capacity is not reached.

It must be noted that the joint is modeled without the rubber profile W9U. In case that joint opening appears, the rubber profile will be tensioned and opening will partly be prevented. How much is prevented, depends on the tension stiffness of the profile. In the $M - \phi$ relations calculated above, this rubber profile is not taken into account. So rotation in the non-linear part will be smaller in real time and the joint opening is also reduced.
4.3.5  Response of the beam model

4.3.5.1  Bending moment

The rotation stiffness for horizontal and vertical direction is determined in chapter 4.3.4 and will be used in the beam model to determine the response of a tunnel structure. Due to the passing seismic wave, the tunnel will undergo a deflection creating a bending moment in the tunnel. For different water depths the tunnel is modeled and checked if the elastic behaviour of the joints is acceptable. The calculations are performed using the same framework program as stated for the axial beam model for worming.

As described in paragraph 4.3.3.2 the stiffness of the GINA increases with increasing water depth. Using the force-compression curve (Figure 4.3-4) the initial compression \( k_0 \) can be read-out. The GINA ETS 180/220 is chosen here, because it is commonly used in immersed tunnels. Smaller or bigger profiles can be used, but they show a rather similar force-compression curve. For a first approach this is an acceptable profile.

With the initial compression the horizontal- and vertical rotation stiffness is calculated using formulas from paragraph 4.3.3.2. The following properties are used:

<table>
<thead>
<tr>
<th>Water depth (m)</th>
<th>Initial compression ( k_0 ) (MN/m/m)</th>
<th>Vertical rotation stiffness ( k_v ) (MN/m/rad)</th>
<th>Horizontal rotation stiffness ( k_h ) (MN/m/rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>11</td>
<td>26258</td>
<td>115328</td>
</tr>
<tr>
<td>15</td>
<td>18</td>
<td>42968</td>
<td>188719</td>
</tr>
<tr>
<td>20</td>
<td>31</td>
<td>74000</td>
<td>325015</td>
</tr>
<tr>
<td>30</td>
<td>55</td>
<td>131292</td>
<td>576640</td>
</tr>
<tr>
<td>40</td>
<td>89</td>
<td>212454</td>
<td>933109</td>
</tr>
</tbody>
</table>

For vertical and horizontal rotation the joint stiffness of the elastic part of the \( M - \phi \) relation is used. The joint properties are implemented in the beam model for a part of the tunnel consisting of a length of 4 tunnel elements (4*120m) as described in paragraph 4.3.3.3.

The beam model is given a deflection equal to the Peak Ground Displacement (PGD) of an representative earthquake. This earthquake is taken from the reference project Coatzacoalcos tunnel where for the Operational Design Earthquake (ODE) the PGD is 0,01g and for the Maximal Design Earthquake (MDE) the PGD is 0,02g. This displacement is the maximum displacement due to the earthquake acting in the middle of the beam model. To create the deflection, the tunnel is loaded with an evenly spread line load. For this situation the representative PGD of 0.01g is chosen, so about 0.1 m. Later on also the PGD of 0.02g is checked. An example is given in Figure 4.3-19 and Figure 4.3-20.
For different water depths the calculation described above can be performed and checked if it is still in the elastic part of the segment joint behaviour. For the vertical bending due to snaking the table below gives the bending moments $M_{\text{max}}$ for varying water depths:

**Table 6: Vertical bending moments beam model for 100 mm max. deflection**

<table>
<thead>
<tr>
<th>Water depth (m)</th>
<th>Equally spread line load q (kN/m)</th>
<th>Maximal bending moment $M_{\text{max}}$ (kNm)</th>
<th>Moment capacity for elastic behaviour $M_{\text{cap.}}$ (kNm)</th>
<th>Reduced moment capacity $M_{\text{cap, red.}}$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0,36</td>
<td>10.656</td>
<td>75.000</td>
<td>37.500</td>
</tr>
<tr>
<td>15</td>
<td>0,55</td>
<td>15.840</td>
<td>125.000</td>
<td>62.500</td>
</tr>
<tr>
<td>20</td>
<td>0,8</td>
<td>23.040</td>
<td>175.000</td>
<td>87.500</td>
</tr>
<tr>
<td>30</td>
<td>1,2</td>
<td>34.560</td>
<td>275.000</td>
<td>137.500</td>
</tr>
<tr>
<td>40</td>
<td>1,45</td>
<td>41.760</td>
<td>375.000</td>
<td>187.500</td>
</tr>
</tbody>
</table>

For horizontal snaking the same calculations can be made resulting in the horizontal bending moments $M_{\text{max}}$ for a part of the immersed tunnel structure consisting 4 elements:

**Table 7: Horizontal bending moments beam model for 100 mm max. deflection**

<table>
<thead>
<tr>
<th>Water depth (m)</th>
<th>Equally spread line load q (kN/m)</th>
<th>Maximal bending moment $M_{\text{max}}$ (kNm)</th>
<th>Moment capacity for elastic behaviour $M_{\text{cap.}}$ (kNm)</th>
<th>Reduced moment capacity $M_{\text{cap, red.}}$ (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>1,6</td>
<td>46.080</td>
<td>200.000</td>
<td>100.000</td>
</tr>
<tr>
<td>15</td>
<td>2,5</td>
<td>72.000</td>
<td>325.000</td>
<td>162.500</td>
</tr>
<tr>
<td>20</td>
<td>3,7</td>
<td>106.560</td>
<td>450.000</td>
<td>225.000</td>
</tr>
<tr>
<td>30</td>
<td>5,3</td>
<td>152.640</td>
<td>700.000</td>
<td>350.000</td>
</tr>
<tr>
<td>40</td>
<td>6,7</td>
<td>192.960</td>
<td>950.000</td>
<td>475.000</td>
</tr>
</tbody>
</table>

In the last two columns of Table 6 and Table 7 the moment capacity and reduced moment capacity are given for the segment joints. The moment capacity is determined in the previous
chapter for the point where the segment joint start behaving plastic. When the tunnel is in use a part of the moment capacity is used for the bending moments caused by static loading like settlement differences and soil layers on top of the tunnel. For this loading a conservative 50% of the moment capacity is taken, so a reduced moment capacity consists of half the full moment capacity.

It is clear from Table 6 and Table 7 that the acting bending moments in the tunnel structure are far below the reduced moment capacities of the segment joints. This means that the segment joints will behave in the linear elastic phase and the normal force compressing the segment joints is not overruled. This way no joint opening will occur at the segment connection.

If the deflection is doubled; a maximum displacement of 200 mm instead of 100 mm, the bending moments for horizontal and vertical snaking will also double. If we now look at the Table 6 and Table 7 the increased bending moments will still be far below the moment capacities, so elastic behaviour is justified. When the deflection will be increased more, the bending moment will become critical.

4.3.5.2 Shear force

The shear force due to the horizontal and vertical snaking depends on the bending moment as calculated in the previous paragraph. According to Hashash et al.[5] and Van Putten[4] the maximum shear force acting on the tunnel structure can be written as:

$$V_{\text{max}} = \left(\frac{2\pi}{L_{\text{wave}}}\right) * M_{\text{max}}$$

The shear force is a function of the maximum bending moment and the length of the seismic wave. The bending moments and shear forces in the middle part of a tunnel are not that big, because they are only subjected to ground movement. Around the abutments where the tunnel can be supported by piles and a cut-and-cover will be used, the structure will resist the movements more. In this area the forces and moments will be much greater, but this is outside of the scope of this research.

If the bending moments of the previous paragraph are used, the acting shear force can be determined. The biggest bending moment works in horizontal direction and at great depth. Also a doubled deflection is used. This way the maximum bending moment is approximately:

$$M_{\text{max}} = 400.000 \text{ kNm}$$

The acting shear force for this bending moment is:

$$V_{\text{max}} = \left(\frac{2\pi}{L_{\text{wave}}}\right) * M_{\text{max}} = \left(\frac{2\pi}{900}\right) * 400.000 = 2.800 \text{ kN}$$

This shear force needs to be taken by the horizontally placed shear keys in the segment joint.

When the tunnel is still compressed due to the axial normal force, the shear loads are transferred from segment to segment by friction. The friction force, in this case the shear force, depends on the axial force in the following relation:

$$V_{\text{friction}} = \mu_{\text{stat}} * N_{\text{axial}}$$

According to Koek the friction coefficient for static situation is $\mu_{\text{stat}} = 0.4$ for concrete to concrete contact[21]. So in deeper situated tunnels, the tunnel is compressed more and more friction could be realized.
In case the segment joint will be opened due to the worming effect (axial elongation), the segments cannot transfer shear force by friction. In this situation the shear force is transferred by the shear keys. The shear keys need to have enough capacity to resist the shear force. This capacity is discussed in chapter **Fout! Verwijzingsbron niet gevonden**.

### 4.3.6 Shortening of the wave length

It was explained in paragraph 4.3.2 that we are dealing with two wave lengths for the modeling of the snaking effect. The first is the large wavelength $L_{\text{wave}} = 900$ m and is used in previous paragraph for the modeling of snaking. For this wave length the tunnel performs well and it will not encounter leakage or joint opening.

After some time more investigation on the seismic wave properties was performed, which ended in a new seismic design wave of $L = 200$ m. The effect of this change will be explained here in this paragraph.

When the wave length is decreased from 900 till 200 m, it can be simplified to the spread of the wave over 8 elements changing in 2 elements. In the model for snaking in previous paragraph only half a wave is looked at, so 4 elements was given a fixed deflection. Now number of elements is decreased to 3, 2 and 1, but given the same deflection. This result in relative high moment in the tunnel due to the decrease of number of Gina’s, the relative flexible joint compared to the stiff segment joints. In Figure 4.3-21 the situation for equal deflection of the beams with reducing number of elements the moment is given. This is the horizontal deflection for a depth of 40 below water level.

The top one represents the model with 4 beams as described in previous paragraph. Here the moment is below the moment capacity for elastic behaviour. For the next three beams, number of elements in decreased but deflection is remained. In this way the moment will increase quadratic. Now these moments are not in the elastic behaviour anymore, so the tunnel fails. If we perform this for different water depths or for vertical deflection, the same extreme moments are found. It can be concluded that with this modeling for snaking the structure fails if the model is subjected to a seismic wave with wave length smaller than 900 m.
4.3.7 Conclusions on snaking effect

The performed modeling for snaking in this chapter is based on the free-field approach as described in paragraph 4.2.3, but here in lateral direction. In paragraph 4.2.3 it was a very conservative approach and later on displaced by the soil-interaction approach.

For the snaking in this chapter the free-field approach only fulfills the requirements if the wave length is above 900 m. If it is below 900 meter the segment joint will encounter plastic deformation due to the large rotation. The wave length of 900 m is too large and so the free-field approach is too conservative for this snaking modeling.

In further modeling of the snaking effect it is proposed to use the soil-tunnel interaction approach as described in paragraph 4.2.4, but here in lateral direction. What the effects are and what the relations between soil and tunnel are is not determined here. This will further be outside the scope of this research.
5 Behaviour of shear keys

5.1 Static shear capacity

The maximum shear force that can be taken by the segment joint depends on the type of shear key and the dimension of it. As described in paragraph 2.3 the three different segment joint are the Dutch spigot and socket joint, the intermediate joint with shear keys in the inner walls and the typical Japanese joint where the pre-stressing cables are kept in final situation. Here the static capacity of the Dutch and intermediate type is determined. The Japanese type of segment joint is not decisive in the shear capacity.

5.1.1 Dutch spigot and socket joint

In the Dutch spigot and socket joint the shear key is situated in the outer side of the wall (Figure 5.1-1). The capacity of the shear key is equal for horizontal and vertical direction, depending on the working length, because it works over the full length and height of the tunnel. In the floor, roof and outer walls the same type of shear key is used. Only one shear key in each direction can be used, because of the opposite projection to each other. So a downward vertical load is only taken by the shear key in the floor over the full width and not by the roof slab.

Due to Eurocode2 [22] section 5.6.4 and NEN6720[23] section 9.11.7 the shear keys can be calculated by using the strut and tie model. This way the pressure en tensile force in the structure can be modeled. When the shear key or notch is loaded by a vertical load \( V \) (Figure 5.1-1), a diagonal pressure force works in the direction of the reinforcement in the bottom and a horizontal tensile force \( (F_s) \) works in the shear key itself to protect the key from fracturing.

The horizontal reinforcement to take this horizontal tensile force determines the capacity of this shear key. The diameter of the bars cannot exceed 16 mm, because it needs to be bended in the upper corner and it then becomes difficult to bend. Also the center-to-center distance for these bars are limited to 100 mm. This way a certain type of reinforcement can be chosen and the capacity of the shear key depends on the force in this tension reinforcement.

The capacity of the horizontal tensile reinforcement is reduced by the horizontal force \( H \) which acts on the shear key. This horizontal force \( H \) is a function of the vertical load \( V \) with a friction...
coefficient \( \mu \) as described in paragraph 4.3.5.2. For concrete to concrete contact this is 0.4, but by using bitumen coating between the concrete segments this can be lowered to 0.1-0.2.

If the limitation of the reinforcement is taken into account and the horizontal force due to friction is also included, the horizontal and vertical capacity can be calculated. In figure 32 and figure 33 the shear capacities for horizontal and vertical direction are drawn depending on the friction coefficient for the horizontal loading. The shear capacity per running meter for this type of shear key is 1000 kN. Taking into account the dimensions of the tunnel the shear capacity of the whole tunnel can be calculated.

You can see that the shear capacity is high for the vertical direction, fluctuating between 16 and 24 MN. This can be useful for loading due to settlement differences, but for the seismic loading it is not needed. The vertical shear force due to seismic waves is low compared to the horizontal shear load.
The horizontal shear capacity fluctuates between 6 and 9 MN for high and low friction between the segments. For the horizontal shear keys the static loading due to layers on top of the tunnel is not present, so all the capacity is available to resist the seismic loading.

With this fact it can be said that the shear capacity is not decisive for the choice if this type of shear key in the segment joint is usable in seismic areas. It must be noted that this only counts for these tunnel dimensions. With other tunnel dimensions the shear capacity will also change.

### 5.1.2 Shear keys in intermediate joint

For the intermediate joint the shear keys are situated in the inner walls and partly in the outer walls for vertical direction (Figure 5.1-4). For horizontal direction the shear keys are situated in the floor and roof slab. The capacity of the shear keys depends on its dimensions; the width is limited by the width of the inner walls and for the height commonly 1/3 of the tunnel height is taken. In this joint type the shear force is transferred by four shear keys with a small width in stead of one shear key over the full width like in the spigot joint.

![Figure 5.1-4: Intermediate segment joint - cross section & side view (male)](image)

The shear capacity for this type of segment joint is determined the same way as described in the previous paragraph. The strut and tie model is applied to determine the force in the structure and again the horizontal reinforcement (2) determines the shear capacity of the segment joint (Figure 5.1-5).

![Figure 5.1-5: Strut and tie approach shear key (male and female)](image)
The amount of reinforcement which can be placed in the top part of the shear key is limited by the width of the shear key and the positioning of the center line of the layers reinforcement in vertical direction. This center line needs to be placed as high as possible in the shear key, but by adding more layers of reinforcement this centerline moves down. Because of larger dimensions of this shear key compared to the Dutch joint, heavier and more reinforcement can be placed in this type.

Also for this type of shear key the shear capacity is reduced by the horizontal force $H$ cause by friction between the segments. Depending on the friction coefficient the shear capacity for one shear key fluctuates between 4 and 6 MN. In the case that there are 4 of these shear keys the capacity is 16 - 24 MN. In horizontal and vertical direction both have 4 shear keys, so the capacity is equal in both directions.

This shear capacity is far above the expected shear loading. So also for this type of shear key the shear force will not be decisive. Also the reinforcement in the shear keys could be increased a bit more, so the shear capacity could be even bigger.

### 5.2 Cyclic loading of shear keys

In the previous paragraph the shear capacity of the segment joint was examined for a static approach, so a long term loading in one direction. For the situation when the tunnel is in use this is true. This approach is not fully true if we look at the loading principle with seismic waves. In case of seismic loading the shear keys will encounter a constant cycle of loading, unloading, loading in opposite direction and back. This phenomenon is called cyclic loading of concrete structures and gives some complications.

When a concrete structure is subjected to cyclic loading, the maximum strength of the structure will decrease and the strain will increase at every loading step. This principle is drawn in the figure below (Figure 5.2-1).

![Stress strain curve for concrete under cyclic loading](image)

**Figure 5.2-1: Stress strain curve for concrete under cyclic loading**

The consequence of this concrete behaviour is that the capacity of the shear key will decrease in time during an earthquake. This can be a large decrease resulting in cracking and bridling of the concrete or even worse, the fracture of the shear key.
To prevent this problem from happening, the ductility of the structure needs to be increased. This means that the structure acts in a more ductile way. The strength will not decrease when it is cyclic loaded. Only the strain will increase more and more, so a plastic behaviour of the structure is the result. The plastic behaviour causes cracking, which can be spotted and repaired. This is preferred above a sudden collapse of the structure.

![Stress-strain diagram](image)

**Figure 5.2-2: Stress-strain diagrams for various types of confinement (Kappos, 1997)**

The increase of ductility can be accomplished by confinement of the reinforcement. In this way the concrete is compressed around the reinforcement and the bonding between steel and concrete is strengthened. This can be done by placing ties or spirals around the reinforcement[24]. The effect is drawn in Figure 5.2-2. Using better confinement increases the strength for cyclic loading, only plastic deformation occurs. In the figure below different confinement arrangements are drawn for axially loaded columns. The graphs give the pattern as described above. Also in Eurocode 8.1 for seismic design this confining of the reinforcement is advised.

![Concrete confinement diagram](image)

The whole improving of reinforcement and getting better confinement is a complex study. It entails the material behaviour of reinforced concrete. This paragraph was only meant to
describe the phenomenon, but calculations are not included yet. This is outside the scope of this research.
6 Classification

6.1 Introduction

In the problem analyses in chapter 1 it was described that the objective of this research was to come up with a classification for the different available segment joints, which was based on seismic design parameters. An example is given in Figure 6.1-1. This classification should be based on the PGA or PGD for the design location. Now both important tunnel responses are examined and the capacity of the shear keys is looked at, a classification can be made for both effects.

![Figure 6.1-1: Classification of segment joints](Image)

6.2 Classification worming

From chapter 4.2 it was concluded that the worming effect does not depend on the amplitude of the passing wave, because the maximum shear stress along the tunnel walls will not increase with increasing soil movement. Therefore a classification based on the amplitude of the wave or the PGD is not possible for the worming effect.

Another fact which was clear was the fact that the worming effect increases if the maximum shear stress increases or the working length of the shear stress on the tunnel walls. The maximum shear stress can be changed with the effective stress and directly related with the overburden. The length of the working shear stress is based on the wave length of the passing wave.

With these facts a more qualitative classification for the segment joints can be made based on the worming effect. This approach is not based on exact values, because it is too complex to include all different design parameters. It depends on the type of soil, construction depth, intensity of the earthquake etc. For the worming effect it makes no difference what type of segment joint is chosen, because the tunnel is only subjected to axial load. This load will be transferred through an equal concrete cross section; therefore this does not make a difference.
The effect of the overburden on the worming effect is drawn in Figure 6.2-1. With an increasing overburden the worming effect will grow rapidly but later be flattened out more due to the increasing stiffness of the compressed Gina profiles. The overburden can be increased until the elongation capacity of the W9U profiles in the segment joints is reached and tear of the profile and/or leakage could occur.

![Figure 6.2-1: Worming effect due to overburden](image)

The effect of the wavelength on the worming effect is drawn in Figure 6.2-2. The worming effect will grow rapidly with the increase of the wave length. This is caused by the working length where the maximal shear stress is subjected on the tunnel. The wave length can increase until the tension capacity of the W9U profile is reached.

![Figure 6.2-2: Worming effect due to wave length](image)
For parameters it can be stated that with an increasing stiffness of the tunnel structure, the worming effect will be less. Increasing stiffness can be accomplished by increasing the tunnel element length. In this way the number of Gina profiles is decreased and so the total compression will also be decreased. In this way the worming effect is also decreased. Another solution is to pre-tension the Gina’s so they already have a higher spring stiffness, and less compression could occur during the seismic wave passage. This also minimizes the worming effect.

6.3 Classification snaking

In contrary to the classification for the worming effect, the classification for the snaking effect can be based on the PGD. In Figure 6.3-1 the relation between the PGD and the snaking effect is given. With an increase of the PGD or amplitude of the S-wave, the snaking effect will increase quadratic. This is because the curvature of the wave is the second derivative of the amplitude.

![Figure 6.3-1: Snaking effect due to PGD](image)

To make a classification for the different segment types, the PGD can serve as the decisive parameter as proposed in paragraph 6.1. The capacity of the different segment joints is based on the shear capacity. The decisive movement for earthquakes is horizontal, so the horizontal capacity should be examined. For the segment joints with shear keys in the inner walls, the intermediate joint, the horizontal shear capacity will be larger than the spigot type shear keys. In this way the intermediate joint is able to encounter more PGD than the spigot shear key. The shear force and the PGD are directly related, because the third derivative of the amplitude of a wave (PGD) is the shear force $V$.

In the above described classification the free-field approach is used for the translation from soil movement to tunnel movement in snaking. In reality the tunnel resists the soil movement and
the tunnel deformation needs to be approached with the soil-tunnel interaction. This approach is outside the scope of this research and therefore it can not be included in this classification.
7  Conclusions and recommendations

In this chapter the most important conclusions drawn from the performed research are described. Also some recommendations are made to provide a direction for further research.

During this research certain uncertainties were encountered, which require further investigation.

7.1  Conclusions

- Immersed tunnels in seismic areas are subjected to seismic waves and will respond with either worming or snaking effect in terms of tunnel deformation.

- The worming effect is not influenced by the increase of the seismic wave amplitude, but by the seismic wave length and the overburden. If the axial wave amplitude increases over a tunnel part in the soil-tunnel interaction approach, the shear stress is already maximal and it will not increase more. The increasing wave length will increase the contact area for the maximum shear stress and compression and worming increases. If the overburden is increased, the effective stress is increased and thus the maximum shear stress. This results in more compression and more worming effect.

- For the snaking effect, the Peak Ground Displacement or amplitude of the S-wave is decisive. If this value increases the bending will increase and so the snaking effect is increased. Also the wave length is a decisive parameter, if this is decreased the snaking effect will increase.

- The elongation capacity of the W9U profile determines the capacity of the tunnel to resist the worming effect, it is the weakest link.

- There are two types of segment joints for segmental tunnels, the “Dutch” joint and the “Intermediate joint”. The “Dutch” joint has more vertical shear capacity and the “Intermediate” more horizontal shear capacity. The decisive movement for earthquakes is horizontal, so the “Intermediate” joint is more suitable for seismic areas.

- It is not totally clear from the literature what the decisive seismic design wave is for tunnels in soft soils. From the modeling for worming and snaking it is clear that the wave length is the decisive parameter for all further calculations.

7.2  Recommendations

For further research several recommendations can be made. The most important one is:

- The propagation speed, period and wave length of the design seismic wave is decisive for the worming and snaking effect for immersed tunnels. In this research it was not
completely clear what propagation speed should be used in soft soils. This needs to be investigated further to make a final judgment on the behaviour of segment joints.

Some other recommendations for further research are:

- For the snaking effect a soil-tunnel interaction approach should be used to determine the tunnel deformation (as has been done for the worming effect.) Now only the conservative free-field approach is used because in the first approach with the larger wave length it already fulfilled the requirements.

- The worming and snaking effect should be combined to determine the total deformation of the tunnel. It is expected that part of the effects are damped out due to their interaction. This should be investigated in more detail.

- The elongation capacity of the W9U profile should be examined in more detail. Now the conservative capacity of the manufacturer is used in the modeling.

- Different Gina profiles should be used in the modeling of worming and snaking, because this influences the spring stiffness of the joints and the tunnel stiffness. Now only the Gina ETS 180/220 is used, but smaller and larger Gina profiles are available.

- The length of the tunnel elements should be varied. Longer elements will result in less Gina profiles and in a stiffer tunnel. This will influence the worming and snaking effects.

- Investigation should be performed on the influence of the abutments on the tunnel behaviour. This involves stiff foundations and extra overburden. This will have influence on the loads and deformations in the tunnel structure.

- The effect of hammering of two concrete segments should be investigated. Now only the phenomenon is mentioned, but the material behaviour during cyclic loading can be of importance for the concrete segment joints and its capacity. Also the possible solution with dampers in the segment joint can be investigated in further research.

- Liquefaction as a result of seismic loading is not included in the research. Due to this phenomenon the loads on the tunnel can reduce and the tunnel response as well. But it generates other problems for the tunnel structure when it is floating up due to the liquefaction.
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Appendix A: Geotechnical cross sections reference projects
Coatzacoalcos Tunnel, Mexico
HZMB, China
Appendix B: Determination of seismic strains

In the article of st John and Zahrah[17] the seismic strains are determined using the theory of wave equations in one direction. From the theory of wave equation in one direction the following formulas are known:

$$\frac{du}{dx} = -\frac{1}{c} \frac{du}{dt} \quad \text{and} \quad \frac{d^2 u}{dx^2} = -\frac{1}{c^2} \frac{d^2 u}{dt^2}$$

The axial strain and curvature are also a function of the displacement $u$:

$$\varepsilon = \pm \frac{du}{dx} \quad \text{and} \quad \kappa = \pm \frac{d^2 u}{dx^2}$$

The peak ground velocity and acceleration are functions of time, so:

$$\frac{du}{dt} = \pm V_s \quad \text{and} \quad \frac{d^2 u}{dt^2} = \pm a_s$$

When these equations are combined, the formula for axial strain and bending strain are:

$$\varepsilon = \frac{du}{dx} = \pm \frac{1}{c} \frac{du}{dt} \pm \frac{1}{C_s} V_s = \pm \frac{V_s}{C_s} \quad \text{and} \quad \kappa = \frac{d^2 u}{dx^2} = \pm \frac{1}{c^2} \frac{d^2 u}{dt^2} \pm \frac{1}{C_s^2} a_s = \pm \frac{a_s}{C_s^2}$$

The equations above are valid for wave propagation in the direction of the tunnel axis. When the wave is approaching the tunnel axis under an angle, the wavelength will be decreased with $\cos \phi$, so in reality increased and the amplitude with $\sin \phi$ or $\cos \phi$ depending on the lateral or axial direction (figure below). When this is taken into account for the seismic strain equations from above it can be rewritten for axial strain due to the S-wave:

$$\varepsilon_s = \varepsilon_{\text{axial}} + \varepsilon_{\text{bending}} = \frac{V_s}{C_s} \times \sin \phi \times \cos \phi + r \times \frac{a_s}{C_s^2} \cos^3 \phi$$

With $r$ the distance between the neutral line and the extreme fiber of the tunnel cross section, $\phi$ is the angle of incidence with respect to the tunnel axis, $V_s$ is the peak particle velocity for shear waves, $C_s$ is the effective propagation velocity of the shear wave and $A_s$ is the peak particle acceleration of the shear wave.

For all different inclination directions and the strains in all directions the steps as described above are performed. This resulted in the different design formulas as summed up in the table below. The formulas are used for the free-field approach for worming effect in paragraph 4.2.3.
Strain and curvature due to body and surface waves (after St. John and Zaitrah, 1987)

<table>
<thead>
<tr>
<th>Wave type</th>
<th>Longitudinal strain</th>
<th>Normal strain</th>
<th>Shear strain</th>
<th>Curvature</th>
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<td>$\varepsilon_{n1} = \frac{V_p}{C_p} \sin \phi$</td>
<td>$\gamma = \frac{V_p}{C_p} \sin \phi \cos \phi$</td>
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<tr>
<td></td>
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<td>$\varepsilon_{n2} = \frac{V_p}{C_p}$ for $\phi = 90^\circ$</td>
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<td>$\varepsilon_{s2} = \frac{V_S}{2C_S}$ for $\phi = 45^\circ$</td>
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<td>$K_m = \frac{A_S}{C_S}$ for $\phi = 35^\circ$</td>
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</table>

The Poisson's ratio and dynamic modulus of a soil deposit can be computed from measured P- and S-wave propagation velocities in an elastic medium: $\nu = \frac{1}{2} \left( \frac{C_p}{C_s} \right)^2 - 1$ or $C_v = \sqrt{\frac{E}{\rho}} = \frac{(1 - \nu)(1 + 4\nu)}{(1 - 2\nu)} \frac{E}{\rho}$ and $E_v = \frac{E}{\rho}$, respectively.
Appendix C: Determination of shear stress

For the soil-tunnel interaction the shear stress is determined depending on the axial soil displacement, tunnel displacement and the relative displacement. Calculation is performed using a spreadsheet and differs for different tunnel stiffness. The following different phases are calculated:
- Stiff tunnel \((EA = \text{infinite})\)
- Flexible tunnel \((EA = 0)\)
- Normal tunnel \((EA=EA_{\text{normal}})\)
- Normal tunnel with joints \((EA = EA_{\text{normal+joints}})\)

\[
U_x = D \sin \phi \sin (2 \pi x / L \cos \phi)
\]

For \(\phi=45\):

\[
U_{\text{max}} = D \times \sqrt{2}
\]

\[
PGD = D = 0.2 \text{ m}
\]

Ground:

\[
u=u_{\text{max}} \sin (2 \pi x / L_{x})
\]

\[
L_{x} = 600 \text{ m}
\]

\[
u_{\text{max}} = 0.141 \text{ m}
\]

Tunnel:

\[
v=v_{\text{max}} \times \left((0.5L-x)/0.5L\right)
\]

Effective soil pressure = 60 kN/m²

circumference = 67600 mm

\[
f = \tan 0.8 \phi = 0.48
\]

\[
tau_{\text{max}} = 1946.88 \text{ N/mm}
\]

\[
tau_{\text{linear}} = (u-v)/10*tau_{\text{max}}
\]
**Stiff tunnel (EA = infinite)**

\[ v_{\text{max}} = 0,00007 \text{ m} \]

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<th>tunnel v</th>
<th>diff. displ. ( \Delta u )</th>
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<th>( F_i ) (kN)</th>
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**sum:** \( 564459 \text{ kN} \)

\( 564 \text{ MN} \)

\[ \text{EA} = 2397850000 \text{ MN/m} \]

\[ K_{\text{gina}} = 2000 \text{ MN/m} \]

\[ K_{\text{total}} = 999,999583 \text{ MN/m} \]

\[ v_{\text{max \_ calculated}}(\text{EA}) = 7,06206E-05 \text{ m} \]
Behaviour of segment joints in immersed tunnels under seismic loading
Flexible tunnel (EA = 0)

\[ v_{\text{max}} = 0.2698 \, \text{m} \]

Position | ground u | tunnel v | diff. displ. \( \Delta u \) | relative disp. \( \Delta u \) (mm) | \( \tau \) (N/mm) | \( F_i \) (kN)
---|---|---|---|---|---|---
0 | 0 | 270 | -270 | 270 | -1947 | -19469
10 | 15 | 261 | -246 | 246 | -1947 | -19469
20 | 29 | 252 | -222 | 222 | -1947 | -19469
30 | 44 | 243 | -199 | 199 | -1947 | -19469
40 | 58 | 234 | -176 | 176 | -1947 | -19469
50 | 71 | 225 | -154 | 154 | -1947 | -19469
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70 | 95 | 207 | -112 | 112 | -1947 | -19469
80 | 105 | 198 | -93 | 93 | -1947 | -19469
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110 | 129 | 171 | -42 | 42 | -1947 | -19469
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220 | 105 | 72 | 33 | 33 | 1947 | 19469
230 | 95 | 63 | 32 | 32 | 1947 | 19469
240 | 83 | 54 | 29 | 29 | 1947 | 19469
250 | 71 | 45 | 26 | 26 | 1947 | 19469
260 | 58 | 36 | 22 | 22 | 1947 | 19469
270 | 44 | 27 | 17 | 17 | 1947 | 19469
280 | 29 | 18 | 11 | 11 | 1947 | 19469
290 | 15 | 9 | 6 | 6 | 1127 | 11271
300 | 0 | 0 | 0 | 0 | 0 | 0

**sum:** 2698 mm  \hspace{1cm} **sum:** -1823 kN

-2 MN
Behaviour of segment joints in immersed tunnels under seismic loading

- Tunnel displacement
- Ground deformation
- Relative displacement
- Shear stress

Reeks 1
Normal tunnel \((EA=EA_{\text{normal}})\)

\[
v_{\text{max}} = 0.055 \text{ m}
\]

<table>
<thead>
<tr>
<th>Position (x)</th>
<th>ground (u)</th>
<th>tunnel (v)</th>
<th>diff. displ. (\Delta u)</th>
<th>relative displ. (\Delta u)</th>
<th>(\tau) (N/mm)</th>
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\[
\text{sum: } 436211 \text{ kN} \\
436 \text{ MN} \\
EA = 2397850 \text{ MN/m} \\
K_{\text{gina}} = 2000 \text{ MN/m} \\
K_{\text{total}} = 999,5831336 \text{ MN/m} \\
v_{\text{max, calculated (EA)}} = 0.054575203 \text{ m}
\]
Behaviour of segment joints in immersed tunnels under seismic loading – Final thesis
Normal tunnel with joints \((EA=EA_{normal+joints})\)

\[ v_{\text{max}} = 0.117 \text{ m} \]

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<th>Position x</th>
<th>soil def. (u)</th>
<th>tunnel (v)</th>
<th>diff. displ. (\Delta u)</th>
<th>relative disp. (\Delta u) (mm)</th>
<th>(\tau) (N/mm)</th>
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Behaviour of segment joints in immersed tunnels under seismic loading – Final thesis

- Tunnel displacement
- Soil displacement
- Diff. displacement
- Relative displacement
- Shear stress
Appendix D: Force-compression graphs Gina by Trelleborg
Appendix E: Simplified force-compression graphs Gina

Simplified Force-compression graphs Gina ETS 180-220

**D = 10 m.**

\[ k_c = 6760 \text{ MN/m} \]

\[ k_f = 220 \text{ MN/m} \]

\[ k_0 = 750 \text{ MN/m} \]

\[ F_0 = 250 \text{ kN/m} \]

**D = 15 m.**

\[ k_c = 6760 \text{ MN/m} \]

\[ k_f = 220 \text{ MN/m} \]

\[ k_0 = 1200 \text{ MN/m} \]

\[ F_0 = 415 \text{ kN/m} \]
Behaviour of segment joints in immersed tunnels under seismic loading

- Chart 1: 
  - D = 20 m.
  - $k_c = 6760 \text{ MN/m}$
  - $F_0 = 585 \text{ kN/m}$
  - $k_f = 220 \text{ MN/m}$
  - $k_0 = 2100 \text{ MN/m}$
  - Gina ETS 180-220

- Chart 2: 
  - D = 30 m.
  - $k_c = 6760 \text{ MN/m}$
  - $F_0 = 920 \text{ kN/m}$
  - $k_f = 220 \text{ MN/m}$
  - $k_0 = 3700 \text{ MN/m}$
  - Gina ETS 180-220
D = 40 m.

- $F_0 = 1260$ kN/m
- $k_c = 6760$ MN/m
- $k_0 = 6000$ MN/m
- $k_I = 220$ MN/m

Gina ETS 180-220
Appendix F: Rotation stiffness Gina gasket

The rotation stiffness of the Gina gasket is based on the initial spring stiffness \( k_0 \) and the dimensions of the seal. The height \( H_t \) and width \( B_t \) are given in the figure:

![Diagram of Gina gasket](image)

The rotation around the vertical axis, so horizontal rotation stiffness, can be schematized with the following figure and calculated:

\[
F_1 = k_0 \times u = k_0 \times \varphi \times \frac{1}{2} B \times H = \frac{1}{2} k_0 \varphi B H
\]

\[
F_2 = 2 \times \frac{1}{2} \times \frac{1}{2} B^2 \times \varphi \times k_0 = \frac{1}{2} k_0 \varphi B^2
\]

\[
M = 2 \times F_1 \times B + 2 \times F_2 \times \frac{2}{3} \times \frac{1}{2} B = 2 \times F_1 \times B + 2 \times F_2 \times \frac{1}{3} \times B
\]

\[
M = 2 \times \left( \frac{1}{2} k_0 \varphi BH \right) \times B + 2 \times \left( \frac{1}{2} k_0 \varphi B^2 \right) \times \frac{1}{3} \times B
\]

\[
M = k_0 \varphi B^2 H + k_0 \varphi \frac{1}{3} B^3
\]

\[
M = k_0 \varphi B^2 \left( H + \frac{1}{3} B \right)
\]

\[
k_\varphi = \frac{M}{\varphi} = k_0 B^2 \left( H + \frac{1}{3} B \right)
\]

The rotation stiffness in vertical direction will be exactly the same, only the parameters \( H \) and \( B \) will change place. The vertical rotation stiffness will now be:

\[
k_\varphi = k_0 H^2 \left( B + \frac{1}{3} H \right)
\]
Appendix G: Spreadsheet calculations shear key capacity

General information

<table>
<thead>
<tr>
<th>Cross-sectional Dimensions General</th>
<th>Surface area general (without segment joint)</th>
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<tr>
<td><strong>Width slabs</strong></td>
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<tr>
<td>Roof slab</td>
<td>Roof slab 24,5 m² (Wr*Tuh)</td>
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<tr>
<td>Floor slab</td>
<td>Floor 31,85 m² (Wf*Tuh)</td>
</tr>
<tr>
<td>Outer wall</td>
<td>Outer wall 14 m² (Wou*Tlh)</td>
</tr>
<tr>
<td>Inner wall</td>
<td>Inner wall 7 m² (Win*Tlh)</td>
</tr>
</tbody>
</table>

| **Inner dimensions**                                |                                             |
| Traffic lane width                                  | Traffic lane width 10 m (Tlw)               |
| Traffic lane height                                 | Traffic lane height 7 m (Tlh)               |
| Escape lane width                                   | Escape lane width 1,5 m (Elw)              |
| Escape lane height                                  | Escape lane height 7 m (Elh)               |

| **Outer dimensions**                                |                                             |
| Tunnel height                                       | Tunnel height 9,3 m (Tuh)                  |
| Tunnel width                                        | Tunnel width 24,5 m (Tuw)                 |

| Surface area general (without segment joint)        |                                             |
| Roof slab                                          | 24,5 m² (Wr*Tuh)                            |
| Floor slab                                         | 31,85 m² (Wf*Tuh)                           |
| Outer wall                                         | 14 m² (Wou*Tlh)                             |
| Inner wall                                         | 7 m² (Win*Tlh)                              |

Total 77,35 m²
Dutch segment joint - Vertical

Example (cement 2007, nr.8, Opinie: Cees Tandtechniek)

**Vertical capacity**

Vertical capacity

Shear key

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Horizontal force H</th>
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<td>h1 600 mm</td>
<td>H = \alpha * V</td>
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<td>h2 600 mm</td>
<td>\alpha = 0,3</td>
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<tr>
<td>x1 300 mm</td>
<td>normal alpha = 0,3-0,5</td>
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<tr>
<td>x2 0 mm</td>
<td>With bitumen alpha = 0,1-0,2</td>
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<table>
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<th>y</th>
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<tr>
<td>B</td>
<td>0,5 L</td>
<td>0,50 Qmax</td>
</tr>
<tr>
<td>C</td>
<td>0,5 L</td>
<td>1 Qmax</td>
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**Vertical shear capacity tunnel**

Vtot 18318 kN

18,3 MN

**Properties "trekband"**

Ø16
-100
d 16 mm
h.o.h . 100 mm
As 201 2

**Hoek drukdiagonaal**

\[ \theta = 60 \text{ degrees (between 30 and 60)} \]

1,04
7198 rad. (VBC art.8.1.4)
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<tr>
<th>Parameter</th>
<th>Value</th>
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<td>$A_{s,t}$</td>
<td>201 mm</td>
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<tr>
<td>$f_s$</td>
<td>435 m²</td>
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<tr>
<td>$F_s$</td>
<td>875 kN</td>
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\[
\begin{align*}
A_{s,t} \times f_s \times 2 &= 346 \text{ mm} \\
A_{s,t} \times f_s \times 4 &= 1087 \text{ mm} \\
A_{s,t} \times f_s \times 2 &= 1433 \text{ mm}
\end{align*}
\]

\[
\begin{align*}
F_s \times 0.8 &= 8V \\
996 \text{ kN per meter} &= F_s \frac{V}{\tan \theta} \\
0.9 \text{ kN per meter} &= F_s \frac{V}{\tan \theta} \\
V &= 24424 \text{ kN} \\
24.4 \text{ MN}
\end{align*}
\]
Q-load spreading

Reinforcement spreading

Output alpha

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<th>α</th>
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<th>0,3</th>
<th>0,4</th>
<th>0,5</th>
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<td>0,78</td>
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<td>1125,1</td>
<td>996,9</td>
<td>894,9</td>
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<td>16,4</td>
<td>14,9</td>
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Shear capacity (kN) vs. alpha
Diameter output

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<td>100</td>
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<td>875</td>
<td>1367 kN</td>
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<td>0,88</td>
<td>0,88</td>
<td>0,88 V</td>
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Degrees theta

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<td>0,959931</td>
<td>0,872665</td>
<td>0,785398</td>
<td>0,698132 rad</td>
</tr>
<tr>
<td>Fs,V</td>
<td>0,58</td>
<td>0,70</td>
<td>0,84</td>
<td>1,00</td>
<td>1,19 V</td>
</tr>
<tr>
<td>Fs =</td>
<td>0,88</td>
<td>1,00</td>
<td>1,14</td>
<td>1,30</td>
<td>1,49 V</td>
</tr>
<tr>
<td>V =</td>
<td>996,9</td>
<td>874,4</td>
<td>767,8</td>
<td>672,8</td>
<td>586,3 kN</td>
</tr>
<tr>
<td>Vtot</td>
<td>18318</td>
<td>16068</td>
<td>14109</td>
<td>12362</td>
<td>10773 kN</td>
</tr>
<tr>
<td></td>
<td>18,3</td>
<td>16,1</td>
<td>14,1</td>
<td>12,4</td>
<td>10,8 MN</td>
</tr>
</tbody>
</table>
Dutch segment joint - Horizontal

### Horizontal capacity

**Shear key**

\[ F_s = 0.88 \text{ V} \]

\[ V = 996.9 \text{ kN per meter tunnel width} \]

<table>
<thead>
<tr>
<th>Part</th>
<th>Length</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Part A</td>
<td>0.25 H</td>
<td>1 Qmax</td>
</tr>
<tr>
<td>Part B</td>
<td>0.5 H</td>
<td>0.50 Qmax</td>
</tr>
</tbody>
</table>

### Horizontal shear capacity tunnel (full width)

Vertical tunnel height = 9.3 m

\[ V_{tot} = 9271 \text{ kN} \]

9.3 MN

### Horizontal shear capacity tunnel

\[ V_{tot} = 6953 \text{ kN} \]

7.0 MN

---

**Q-load spreading**

- \( Q_{max} \)
- \( (1/2) Q_{max} \)
- \( Q_{max} \)

**Reinforcement spreading**

- A
- B
- A
\begin{align*}
\text{Output alpha} \\
\quad \alpha & \quad 0,1 \quad 0,2 \quad 0,3 \quad 0,4 \quad 0,5 \\
\text{Fs} & \quad 0,68 \quad 0,78 \quad 0,88 \quad 0,98 \quad 1,08 \text{ V} \\
\text{V} & \quad 1291,2 \quad 1125,1 \quad 996,9 \quad 894,9 \quad 811,8 \text{ kN} \\
\text{Vtot} & \quad 9006 \quad 7848 \quad 6953 \quad 6242 \quad 5662 \text{ kN} \\
\end{align*}

\begin{figure}
\centering
\includegraphics[width=\textwidth]{output_alpha.png}
\caption{Output alpha}
\end{figure}

\begin{align*}
\text{Diameter output} \\
\quad d & \quad 8 \quad 10 \quad 12 \quad 16 \quad 20 \text{ mm} \\
\text{h.o.h.} & \quad 100 \quad 100 \quad 100 \quad 100 \quad 100 \text{ mm} \\
\text{As} & \quad 50 \quad 79 \quad 113 \quad 201 \quad 314 \text{ mm}^2 \\
\text{As,tot} & \quad 503 \quad 785 \quad 1131 \quad 2011 \quad 3142 \text{ mm}^2 \\
\text{Fs} & \quad 219 \quad 342 \quad 492 \quad 875 \quad 1367 \text{ kN} \\
\text{Fs} & \quad 0,88 \quad 0,88 \quad 0,88 \quad 0,88 \quad 0,88 \text{ V} \\
\text{V} & \quad 249,2 \quad 389,4 \quad 560,7 \quad 996,9 \quad 1557,6 \text{ kN} \\
\text{Vtot} & \quad 1738 \quad 2716 \quad 3911 \quad 6953 \quad 10865 \text{ kN} \\
\end{align*}

\begin{figure}
\centering
\includegraphics[width=\textwidth]{diameter_output.png}
\caption{Diameter output}
\end{figure}
### Degrees theta

<table>
<thead>
<tr>
<th>Theta</th>
<th>60</th>
<th>55</th>
<th>50</th>
<th>45</th>
<th>40</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1,047198</td>
<td>0,959931</td>
<td>0,872665</td>
<td>0,785398</td>
<td>0,698132</td>
</tr>
<tr>
<td>Fs_V</td>
<td>0,58</td>
<td>0,70</td>
<td>0,84</td>
<td>1,00</td>
<td>1,19</td>
</tr>
<tr>
<td>Fs</td>
<td>0,88</td>
<td>1,00</td>
<td>1,14</td>
<td>1,30</td>
<td>1,49</td>
</tr>
<tr>
<td>V</td>
<td>996,9</td>
<td>874,4</td>
<td>767,8</td>
<td>672,8</td>
<td>586,3</td>
</tr>
<tr>
<td>Vtot</td>
<td>6953</td>
<td>6099</td>
<td>5356</td>
<td>4693</td>
<td>4089</td>
</tr>
</tbody>
</table>

| Shear capacity (kN) | 7,0 | 6,1 | 5,4 | 4,7 | 4,1 |

<table>
<thead>
<tr>
<th>Shear capacity (MN)</th>
<th>0,0</th>
<th>1,0</th>
<th>2,0</th>
<th>3,0</th>
<th>4,0</th>
<th>5,0</th>
<th>6,0</th>
<th>7,0</th>
<th>8,0</th>
</tr>
</thead>
<tbody>
<tr>
<td>degrees theta</td>
<td>60</td>
<td>55</td>
<td>50</td>
<td>45</td>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

![Graph showing shear capacity vs. degrees theta]
Intermediate segment joint – Vertical

\[ P1 = 30\% \quad P2 = 20\% \quad 20\% = P3 \quad P4 = 30\% \]

Capacity Shear key

- no. of keys: 4
- Dimensions:
  - \( h1 = 4650 \text{ mm} \)
  - \( h2 = 3000 \text{ mm} \)
  - \( h3 = 4650 \text{ mm} \)
  - \( x1 = 1500 \text{ mm} \)
  - \( x2 = 0 \text{ mm} \)

\[ H = \alpha \cdot V \quad \alpha = 0.3 \]
<table>
<thead>
<tr>
<th>Note</th>
<th>Normal force</th>
<th>Note</th>
<th>Normal force</th>
<th>Note</th>
<th>Normal force</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>tan α = -0.58 V</td>
<td>1</td>
<td>tan α = -0.84 V</td>
<td>1</td>
<td>tan α = -1.00 V</td>
</tr>
<tr>
<td>2</td>
<td>V / tan α = 0.58 V</td>
<td>2</td>
<td>V / tan α = 0.84 V</td>
<td>2</td>
<td>V / tan α = 1.00 V</td>
</tr>
<tr>
<td>3</td>
<td>V / tan α = 0.58 V (Ltruss / Htruss)</td>
<td>3</td>
<td>V / tan α = 0.84 V (Ltruss / Htruss)</td>
<td>3</td>
<td>V / tan α = 1.00 V (Ltruss / Htruss)</td>
</tr>
<tr>
<td>4</td>
<td>V = 1.00 V</td>
<td>4</td>
<td>V = 1.00 V</td>
<td>4</td>
<td>V = 1.00 V</td>
</tr>
<tr>
<td>5</td>
<td>V = 1.00 V</td>
<td>5</td>
<td>V = 1.00 V</td>
<td>5</td>
<td>V = 1.00 V</td>
</tr>
<tr>
<td>6</td>
<td>V = 1.00 V</td>
<td>6</td>
<td>V = 1.00 V</td>
<td>6</td>
<td>V = 1.00 V</td>
</tr>
<tr>
<td>7</td>
<td>a = -1.15 V</td>
<td>7</td>
<td>a = -1.31 V</td>
<td>7</td>
<td>a = -1.41 V</td>
</tr>
<tr>
<td>8</td>
<td>a = -1.15 V</td>
<td>8</td>
<td>a = -1.31 V</td>
<td>8</td>
<td>a = -1.41 V</td>
</tr>
<tr>
<td>9</td>
<td>a = -1.15 V</td>
<td>9</td>
<td>a = -1.31 V</td>
<td>9</td>
<td>a = -1.41 V</td>
</tr>
</tbody>
</table>

Shear key internal: 2 23 %
Capacity: 4711 kN

Shear key external: 2 27 %
Capacity: 5401 kN

Shear capacity segment joint

V_{to}t = 20224 kN
20.2 MN

V_{to}t = 15577 kN
15.6 MN

V_{to}t = 13649 kN
13.6 MN
<table>
<thead>
<tr>
<th>Shear key Coatzacoalcos internal wall</th>
<th>TYPE A</th>
<th>Shear key Coatzacoalcos external wall</th>
<th>TYPE A</th>
</tr>
</thead>
<tbody>
<tr>
<td>B+C: additional</td>
<td></td>
<td>B+C: additional</td>
<td></td>
</tr>
<tr>
<td>d</td>
<td>19,05</td>
<td>d</td>
<td>25,4</td>
</tr>
<tr>
<td>h.o.h.</td>
<td>100</td>
<td>h.o.h.</td>
<td>100</td>
</tr>
<tr>
<td>As</td>
<td>285</td>
<td>As</td>
<td>507</td>
</tr>
<tr>
<td>layers</td>
<td>3</td>
<td>layers</td>
<td>6</td>
</tr>
<tr>
<td>no. Bars</td>
<td>4</td>
<td>no. Bars</td>
<td>2</td>
</tr>
<tr>
<td>As,tot</td>
<td>3420</td>
<td>As,tot</td>
<td>6080</td>
</tr>
<tr>
<td>As,tot</td>
<td>9501</td>
<td>As,tot</td>
<td></td>
</tr>
<tr>
<td>fs</td>
<td>435</td>
<td>Fs</td>
<td>4133</td>
</tr>
<tr>
<td>Fs, max</td>
<td>4739 kN</td>
<td>(As,tot * fs)</td>
<td></td>
</tr>
</tbody>
</table>

| Note 2                               |        | Note 2                               |        |
| h.o.h.                               |        | h.o.h.                               |        |
| As                                   | 285    | As                                   | 507    |
| layers                               | 4      | layers                               | 4      |
| no. Bars                             | 4      | no. Bars                             | 2      |
| As,tot                               | 4560   | As,tot                               | 4054   |
| As,tot                               | 10894  | As,tot                               | 2280   |
| As,tot                               | 10894  | As,tot                               |        |
| fs                                   | 435    | Fs                                   | 4133   |
| Fs, max                              | 4739 kN| (As,tot * fs)                        |        |

| Percentage                           |        | Percentage                           |        |
|                                       |        |                                       |        |
Intermediate segment joint – Vertical

alpha = 60 degrees (between 30 and 60)  
1,047198 rad.

<table>
<thead>
<tr>
<th>Note</th>
<th>Normal force</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>- V / tan a</td>
</tr>
<tr>
<td>2</td>
<td>V / tan a</td>
</tr>
<tr>
<td>3</td>
<td>V / tan a</td>
</tr>
<tr>
<td>4</td>
<td>(Ltruss/Htruss)</td>
</tr>
<tr>
<td>5</td>
<td>V</td>
</tr>
<tr>
<td>6</td>
<td>V</td>
</tr>
<tr>
<td>7</td>
<td>- V / sin a</td>
</tr>
<tr>
<td>8</td>
<td>- V / sin a</td>
</tr>
<tr>
<td>9</td>
<td>- V / sin a</td>
</tr>
</tbody>
</table>

Normal force

-0.58 V
0.58 V
0.58 V
#DIV/0!
1.00 V
1.00 V
-1.15 V
-1.15 V
-1.15 V

Shear key external
Capacity 2745 kN

Shear capacity segment joint

\( V_{tot} \) 10978 kN

11.0 MN
<table>
<thead>
<tr>
<th></th>
<th>Shear key Coatzacoalcos internal wall</th>
<th>TYPE A</th>
</tr>
</thead>
<tbody>
<tr>
<td>B+C:</td>
<td>Ø16 -100</td>
<td>Ø25 -100</td>
</tr>
<tr>
<td></td>
<td>d 16 mm</td>
<td>d 25 mm</td>
</tr>
<tr>
<td>Note 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>h.o.h.</td>
<td>100 mm</td>
<td>100 mm</td>
</tr>
<tr>
<td>As</td>
<td>201 mm²</td>
<td>As 491 mm²</td>
</tr>
<tr>
<td>layers</td>
<td>2 layers</td>
<td>4 layers</td>
</tr>
<tr>
<td>no. Bars</td>
<td>4 no. Bars</td>
<td>2 no. Bars</td>
</tr>
<tr>
<td>As,tot</td>
<td>1608 mm²</td>
<td>As,tot 3927 mm²</td>
</tr>
<tr>
<td>As,tot</td>
<td></td>
<td>5535 mm²</td>
</tr>
<tr>
<td>fs</td>
<td>435 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Fs, max</td>
<td>2408 kN</td>
<td>(As,tot * fs)</td>
</tr>
<tr>
<td>Percentage</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>