Structural Behavior of Concrete Shear Keys in the Nanchang Red Valley Immersed Tunnel

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Preface

This master's dissertation is the result of research performed at Tongji University in Shanghai, P.R. China, and Ghent University, Belgium. The thesis document is for the largest part written at the Geotechnical Department of Tongji University in Shanghai and on the construction site of the Red Valley immersed tunnel in Nanchang. The subject of this dissertation fits in the context of ongoing research at Tongji University about the structural behavior of the Jianxi Nanchang Red Valley immersed tunnel.

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Abstract

The immersed tunnel technique is a common technique for crossing rivers, lakes and sea in the People’s Republic of China. Although historically most immersed tunnel construction occurred in the Netherlands, the United States and Japan, the construction of immersed tunnels in China has increased rapidly in the last decades. Currently the People’s republic of China has no National Standard design code for immersed tunnels and their design is often accompanied by numerical studies and physical tests.

The focus of this dissertation is the behavior of the concrete shear keys of a physical scale model of the Nanchang Red valley immersed tunnel under static loads. The Nanchang Red valley immersed tunnel is under construction during the period in which this dissertation is written (2015-2016). Because concrete shear keys are loaded most heavily when the immersed tunnel is loaded in the lateral direction, the loading case that is of prior interest is lateral loading of the tunnel elements in combination with the axial load of the tunnel elements that is inherent to immersed tunnels due to their construction process.

The structural behavior of the joint of a scale model of the Nanchang Red Valley immersed tunnel is assessed. Tests are conducted through numerical analysis, and can later be compared with results from physical scale model testing. For both the numerical model and the physical test, a geometrical scale of 1:5 is used relative to the real tunnel prototype. The overall dimensions of the cross-section of both the numerical model and the physical scale model are 6 m × 1,66 m.

The numerical analysis is performed by using the finite element (FE) software package Abaqus. The FE model is composed of the ends of 2 adjacent tunnel elements, and their mutual joint. Its purpose is to predict the structural behavior of the joint under lateral loading that will occur in the physical scale model test. The results of the numerical analysis can be useful in the design of the physical scale model experiment. After the physical scale model tests, the numerical results can be verified and numerical parameters can be calibrated further, so that numerical modeling of future immersed tunnel projects can be performed more reliably.
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#### Latin symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>area</td>
</tr>
<tr>
<td>C_{10}</td>
<td>first empirical shear constant in Mooney-Rivlin model</td>
</tr>
<tr>
<td>C_{21}</td>
<td>second empirical shear constant in Mooney-Rivlin model</td>
</tr>
<tr>
<td>d</td>
<td>scalar degradation variable</td>
</tr>
<tr>
<td>D</td>
<td>undamaged elasticity matrix</td>
</tr>
<tr>
<td>D_1</td>
<td>compressibility constant in Mooney-Rivlin model</td>
</tr>
<tr>
<td>E</td>
<td>modulus of elasticity</td>
</tr>
<tr>
<td>F</td>
<td>load</td>
</tr>
<tr>
<td>F()</td>
<td>yield function</td>
</tr>
<tr>
<td>f</td>
<td>strength</td>
</tr>
<tr>
<td>G</td>
<td>flow potential</td>
</tr>
<tr>
<td>H_r</td>
<td>material hardness</td>
</tr>
<tr>
<td>I_1</td>
<td>first invariant of Green</td>
</tr>
<tr>
<td>I_2</td>
<td>second invariant of Green</td>
</tr>
<tr>
<td>K</td>
<td>stiffness</td>
</tr>
<tr>
<td>p</td>
<td>contact pressure</td>
</tr>
<tr>
<td>p̅</td>
<td>effective hydrostatic pressure</td>
</tr>
<tr>
<td>q̅</td>
<td>Mises equivalent effective stress</td>
</tr>
<tr>
<td>r</td>
<td>principal stress weight factor</td>
</tr>
<tr>
<td>R^2</td>
<td>coefficient of determination</td>
</tr>
<tr>
<td>S</td>
<td>scale factor</td>
</tr>
<tr>
<td>Ṡ</td>
<td>deviatory part of effective stress tensor</td>
</tr>
<tr>
<td>t</td>
<td>time</td>
</tr>
<tr>
<td>W</td>
<td>strain energy potential</td>
</tr>
<tr>
<td>x</td>
<td>horizontal, lateral direction</td>
</tr>
<tr>
<td>y</td>
<td>vertical direction</td>
</tr>
<tr>
<td>ẏ</td>
<td>fraction of lateral load</td>
</tr>
<tr>
<td>z</td>
<td>horizontal, axial direction</td>
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</table>

#### Greek symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
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<tbody>
<tr>
<td>α</td>
<td>significance level</td>
</tr>
<tr>
<td>α</td>
<td>angular rotation</td>
</tr>
<tr>
<td>α</td>
<td>concrete dimensionless material constant (1)</td>
</tr>
<tr>
<td>β</td>
<td>concrete yield condition parameter</td>
</tr>
<tr>
<td>γ</td>
<td>concrete dimensionless material constant (2)</td>
</tr>
<tr>
<td>ε</td>
<td>strain</td>
</tr>
<tr>
<td>ε</td>
<td>element of a (numerical) set</td>
</tr>
<tr>
<td>ϵ</td>
<td>flow rule eccentricity</td>
</tr>
<tr>
<td>η</td>
<td>dimensionless stress of concrete</td>
</tr>
<tr>
<td>μ</td>
<td>viscosity parameter</td>
</tr>
<tr>
<td>μ</td>
<td>coefficient of friction</td>
</tr>
<tr>
<td>τ</td>
<td>shear stress</td>
</tr>
<tr>
<td>σ</td>
<td>normal stress</td>
</tr>
<tr>
<td>φ</td>
<td>bar diameter</td>
</tr>
<tr>
<td>ψ</td>
<td>flow rule dilatation angle</td>
</tr>
</tbody>
</table>
\( \xi \)  
dimensionless strain of concrete

**Subscripts**
- \( c \): compression
- \( E \): stiffness
- \( f \): axial (load, on tunnel face)
- \( i \): initial
- \( l \): lateral (stiffness)
- \( L \): length
- \( s \): lateral (load, on tunnel side)
- \( t \): tension
- \( u \): ultimate value
- \( v \): viscous
- \( x \): displacement
- \( y \): yield
- \( 0 \): initial

**Superscripts**
- \( el \): elastic
- \( pl \): plastic

**Abbreviations, acronyms and units**
- CI: confidence interval
- cm: centimeter
- FE: finite element
- HWL: high water level
- HPC: high performance concrete
- HZMB: Hong Kong-Zhuhai-Macao bridge tunnel
- kN: kilonewton
- LWL: low water level
- m: meter
- max: maximum
- min: minimum
- mm: millimeter
- N: Newton
- MPa: megapascal
- rad: radians
- sec: seconds
I Introduction

1 Problem definition

The immersed tunneling technique is a common technique for crossing rivers, lakes and sea in the People's Republic of China. Currently the People's republic of China has no National Standard design code for immersed tunnels and their design is often accompanied by numerical studies and physical tests. An immersed tunnels is composed of prefabricated elements that are placed in trenches that have been dredged in river or sea bottoms, and that are afterwards interconnected. The joints that connect the adjacent tunnel elements are considered to be the weakest elements in the whole tunnel.

To assess the behavior of the joint of the Nanchang Red Valley immersed tunnel, a physical scale model with a geometric scale of 1:5 will be built on the Red Valley tunnel construction site and will be loaded laterally until failure. Two quasi-static loading cases will be considered: combined axial and vertical loading, and combined axial and lateral loading. This dissertation focuses on the latter loading case.

Prior to testing of the physical scale model, its behavior is difficult to predict. Nevertheless decisions have to be made concerning the design of the scale model and the test setup. Prior to the test, the ultimate load of the physical scale model is unknown, so that it is not known which loading configuration is best suited for the scale model test. Also, the location and specifications of the strain gauges and pressure sensors depend on the damage phenomena that are expected to occur during the physical scale model test as well as the pressures that will occur locally.

A numerical (FE) model is used to predict the behavior of the 1:5 physical scale model. The following research questions are formulated.

2 Main research question

What is the structural behavior of the concrete shear keys in a 1:5 physical scale model of the Nanchang Red Valley tunnel under static lateral loading?
3 Sub-questions

1. How can the materials and the structural configuration of the joint of an immersed tunnel be modeled using FE software?
2. How can we predict the ultimate lateral load that can be applied to the 1:5 physical scale model of the Red Valley immersed tunnel using FE simulation;
3. How can the damage phenomena that will occur in the concrete shear keys in the 1:5 physical scale model of the Red Valley immersed tunnel be predicted;
4. How can the distribution of the externally applied lateral loads over the structural elements in the immersed joints be assessed?

4 Objective

The objective of this dissertation is to construct a numerical model with which the structural behavior that will occur in a physical scale model of the Nanchang Red Valley immersed tunnel can be predicted. The aim of the numerical modeling is to provide useful information for the set-up of the physical scale model and to predict certain aspects of the response of the scale model. In a later stage the results from the physical model can provide useful feedback to further enhance initial numerical models, so that future numerical analyses of immersed tunnel joints can be performed more accurately.

5 Structure of the report

Part I of the report contains general information on this master’s dissertation. Part II of this report is an introduction on immersed tunneling, their construction technique and on immersed joints. In part III the Nanchang Red Valley project is introduced. The design of this project is the starting point for the numerical and physical simulations. Part IV explains the design of the numerical model, and contains the results from the numerical tests. Part V contains final conclusions.
II  Immersed tunneling

1  Abstract

An immersed tunnel is composed of prefabricated elements. The elements are placed in trenches that have been dredged in river or sea bottoms, and are afterwards interconnected. The joints that connect the adjacent tunnel elements are considered to be the weakest elements in the whole tunnel. Whereas steel immersed tunnels have a structural system that is composed of stiffened structural steel plates working compositely with interior concrete, concrete tunnels have passively reinforced and/or pre-stressed concrete as the main structural material [1][2].

In the following paragraphs, an overview of selected literature concerning immersed tunneling is provided. As there is not many literature available on the structural behavior of the joints, the focus is primarily on the tunnel structure and on the construction method of immersed tunnels.

2  Definition

An immersed tunnel is a passageway below water level, that consists of one or more prefabricated elements that are floated to the site, installed one by one, and interconnected under water. An immersed tunnel is generally installed in a trench that has been dredged previously at the bottom of a river, lake or sea, and connects terminal structures that are located on land.

The space between the trench bottom and the soffit of the tunnel can be a gravel or a sand bed. After placement of the elements, the tunnel trench is backfilled and the completed tunnel is usually covered with a protective layer of stone/rock over the roof [1][3].

Two main types of immersed tunnels exist, namely steel and concrete immersed tunnels. The structural system of steel tunnels is made of stiffened structural steel plates, working compositely with interior concrete. Concrete tunnels on the other hand, have pre-stressed and/or passively reinforced concrete as the main structural material [1][2].
3 Comparison to other tunnel types

Currently, 3 types of tunnels exist, namely cut-and-cover tunnels, subsurface excavation tunnels and immersed tunnels. The possibility of a fourth type of tunnel, i.e. floating tunnel, is being examined but has currently not been built for human transportation [4].

Cut-and-cover tunnels, also called surface tunnels or open excavation tunnels, are built by excavating a trench and constructing the tunnel structure or placing the prefabricated elements in the trench. After the completion of the underground structure, the trench is backfilled with soil. Cut-and-cover tunnels are shallow tunnels, typically less than 30 meters below ground level [2].

Subsurface excavation tunneling comprises two different tunneling techniques: mechanical tunneling and conventional tunneling. In mechanical tunneling, the excavation is done by using full-face tunnel boring machines (TBMs). Conventional tunneling uses small drill holes in which explosives are placed, together with mechanical excavators (but not full-face TBM's) [2].

Contrary to cut-and-cover tunnels, an immersed tunnel only functions to cross rivers, lakes and sea. Unlike subsurface excavation tunnels, they rest on the river- or seabed. The immersed tunnel's prefabricated elements are placed in trenches that have been dredged at the river or sea bottom, and that are interconnected after placement. The joints that connect the adjacent tunnel elements are considered to be the weakest elements in the whole tunnel [1].

Compared to cut-and-cover tunnels and subsurface excavation tunnels, immersed tunnels have some specific advantages [2][3]:

- possibility of non-circular cross-section and versatility of cross-section geometry
- tunnel construction is possible with ground conditions that are not feasible for subsurface excavation, such as soft alluvial deposits in for example river estuaries.
- prefabricated construction in dry docks under normal working conditions promotes construction quality
- placement of tunnel elements on the bed of the waterway yields more shallow construction, and thus shorter tunnel approaches (Figure II-1)
- construction, placement and further detailing of different parts of the tunnel can happen simultaneously
- fewer in-situ joints.

These advantages can make immersed tunnel more viable for river or sea passages than other tunneling methods concerning total project cost, operational aspects and technical feasibility.
Nevertheless, some possible limitations of immersed tunnels can be mentioned [2]:

- possibility of environmental disturbance to the water body bed
- need for suitable sites for the casting yard or construction yard
- influence of weather and tidal conditions during placements of immersed tunnel segments
- need for stable soil on river- or seabed to maintain the dug trench.

4 Construction method

4.1 Overview of the construction process

As the nature and magnitude of the loads on the joints between the segments arise from or are dependent on the construction method, a brief summary is given of the practical construction of immersed tunnels.

The construction of an immersed tunnel is composed of following phases [2][3][5][6]:

1. trench excavation
2. foundation preparation
3. tunnel element fabrication
4. transportation and lowering of the element
5. positioning and connecting the element
6. backfilling of the trench
7. complementary works in the tunnel.

Not all of these steps or phases have to be performed subsequently. For example, foundation preparation and tunnel element fabrication are typically two phases that happen simultaneously. Also, while one tunnel element is transported, handled and placed, the next element and its foundation can already be constructed.

1. Trench excavation. The most common method of excavation of the trenches for immersed tunnels is by using a clamshell dredger. Accuracy of dredging and potential sloughing of the sides needs to be taken into account, so that the necessary bottom width and trench profile are maintained during the lowering and placing of the elements and placing of the foundation materials.
The purpose of the excavation is to make space for the prefabricated tunnel body, the sand or gravel foundation under the body and the protective backfill at the sides and the top of the tunnel.

2. **Foundation preparation.** Foundation treatment methods depend on local geologic conditions. Pile foundations are used when differential settlements are feared due to varying stiffness of the subsoil along the tunnel length, due to vibrations of the tunnel or due to serious sediment inclusion in areas with extreme soft subsoil.

An alternative to constructing a pile foundation is to use sand jetting. In this technique, the tunnel elements are placed directly on the sea- or riverbed and sand is dispersed horizontally in clearance areas between the tunnel elements and the subsoil through openings in the bottom slab.

A third alternative is the construction of a gravel bed prior to placement of the element, with or without the use of special grout. This third technique gives a higher foundation stiffness than the sand jetting technique, but requires more accuracy during the construction of the foundation layer.

The usage of sand as foundation material is not advised in areas with seismic activities, as the loading capacity of the foundation may be diminished due to liquefaction of the sand.

3. **Tunnel element fabrication.** Tunnel elements are fabricated off-site, usually in dry docks or in specially constructed casting basins. Tunnel elements are normally between 80 m and 150 m long, and can consist of several interconnected segments. The length of segments usually varies between 15 and 25 m. Prior to transportation, the ends of each element are closed by bulkheads to make the element watertight. The bulkheads are set back a nominal distance from the end of the element, resulting in a small space at the ends of the adjoining sections that is filled with water when elements are interconnected. This space requires dewatering after the connection with the previously installed element is made.

A dry dock for the elements of a 1,3 km long immersed tunnel underneath a river in Nanchang, China is shown in Figure II-2.

![Figure II-2: casting basin in Nanchang, China](image)

An overview of different production steps in the casting basin of the Nanchang tunnel is depicted in Figure II-3.
4. **Transportation and lowering of the element.** The elements are towed into position over the excavated trench. For concrete immersed tunnel elements, the draught of a segment is usually just shy of the height of the elements. Lowering of the tunnel elements is done with purpose-built catamarans, with pontoons on top of the elements or with cranes. To lower an element into its position on the subsoil, the element is sunk either under its own weight or by using temporary ballasting. During lowering and positioning of the elements, the position of two segments relative to each other can be monitored by using survey towers. These towers can also be used for interior access (Figure II-4). When the tunnel elements is sunk, significant water pressures act on the bulkheads of the tunnel elements.

5. **Positioning and connecting the element.** After placing the element in its position, it is connected with the previously placed element or structure with which it has to be joined. Once the element is in its final position and butted up against the adjacent element, the water that is entrapped between the bulkheads is pumped out (Figure II-5). Buttoning up of the elements against each other can be done by using jacks that are mounted on the outside of the elements. The water pressure on the end of every newly installed element is a very important factor in pushing the elements against each other. After remaining foundation and backfilling work, the joint can be completed. A typical joint between two tunnel elements consists of watertight bulkheads, joint seals and gaskets, room and provisions for the horizontal and vertical shear keys and vertical and horizontal adjustment devices such as wedges and jacks.
Figure II-5: positioning adjacent elements and dewatering voids between bulkheads [5]

6. Backfilling the trench. A locking fill (sand or coarser material) is placed in the trench to about half the height of the elements to ensure their position after connection. Ordinary backfill is also placed to fill the trench, to a depth of about 1.5 m to maximum 2 m above the tube. This ordinary backfill is typically material that was excavated from the trench. The ballast on top of the tunnel functions as a protection of the structure and also prevents uplift.

Figure II-6: backfill material is laced besides and over the tunnel [3]

7. Complementary works in the tunnel. As soon as the tunnel elements have been brought to rest on the permanent foundation and after the ballast has been applied to prevent uplift, the complementary works inside the tunnel can be performed. They include casting of ballast concrete and removal of water in the tunnel, installation of remaining seals and joints and construction of the remaining joint structure. The installation of shear keys in the element joints is done in this phase of the tunnel construction.

4.2 Immersed joints

An immersed joint consists of structural mechanisms to transfer loads across the joint, and elements to ensure watertightness of the joint.

4.2.1 Structural configuration

The transfer of non-axial loads over the segment joint is ensured by horizontal and vertical shear keys (either in concrete or in steel). Longitudinal compression forces are transferred through infill concrete between the adjacent elements and through the Gina gasket. In case they occur, extension forces in the longitudinal direction of the tunnel can be transferred with passive or pre-stressed cables.

To clarify the structural configuration of immersion joints, some practical examples from existing projects are mentioned.
The cross-section of the immersion joint of the Hong Kong-Zhuhai-Macao Bridge Tunnel (HZMB) in China is shown in Figure II-7. The pre-stressed cables that connect the segments of the tunnel element are indicated in the enlarged top part of this figure. In the lower part of the figure, the horizontal and vertical concrete shear keys are indicated.

![Diagram of HZMB Tunnel Immersion Joint](image)

**Figure II-7: details and cross-section of the immersion joint of HZMB Tunnel [7]**

Gina and omega seals in the top and bottom slab of the tunnel elements together with the shear keys in the side walls are shown in a detailed vertical section of the immersion joint and shear keys of the Coatzacoalcos project in Veacruz, Mexico (Figure II-8).

To transfer tension forces and to provide security against leakage of an immersion joints due to settlements and earthquake-induced loads, passive tendons are sometimes installed over the immersion joints (not shown in Figure II-8). These tendons are activated if the minimum compression of the Gina gasket under normal conditions is reached, and prevent the loss of compression force in the Gina gasket [8].

The couplers between immersion joints are indicated in a schematic design drawing of an immersion joint of the Osaka South Port tunnel in Japan (Figure II-9).
4.2.2 Elements for watertightness

The watertightness of immersed concrete tunnels depends on the quality of the joints and the absence of full-dept cracks and in the concrete. Some immersed tunnels use watertight enveloping membranes as a secondary waterproofing system and to protect the structural concrete against aggressive chemical agents. In that case, the watertightness also depends on the quality of this waterproofing [3][10].
To ensure this watertightness of joints, water seals and gaskets are installed in the immersed joints. All joints between the tunnel elements are to be tightly closed. The use of rubber seals makes it possible to maintain some degree of flexibility of the immersed joints.

Waterstop gaskets are installed prior to floating of the tunnel elements and they provide an initial seal upon connection of the elements after sinking. For this primary watertight sealing, different types are used worldwide. In the 1960’s, the Gina sealing was invented in the Netherlands [11]. Different types of gaskets are shown in Figure II-10. In this figure, (a) and (c) are Gina gaskets, which are common in Western Europe and in China.

Omega seals, on the other hand, serve as secondary water-retaining elements. In order to check whether leakage occurs between the Gina seal and the omega seal, pressure tests can be used. Pipework to perform such tests were previously indicated in Figure II-11 (number (11) in the figure).

Some details of the elements that ensure watertightness of a typical immersion joint in Western Europe and in China is shown in Figure II-11. The Gina gaskets (4) and omega seal (14) are mounted with clamping systems (3) and (15). The omega clamping system and the seal counterplate (6) is attached to the concrete with anchor bars (9). In this figure, (10) indicates structural reinforcement within the concrete slab.

After the omega seal is fixed, infill concrete (13) is cast on the inside of the tunnel. A compressible joint filler and surface seal (17) is to ensure watertightness and at the same time maintain some degree of flexibility of the immersed joint.
Part of the clamping system for the Gina gaskets on an element under construction of the Nanchang project is indicated with arrows in Figure II-12.

In order to avoid full-depth cracks in the structural concrete (due to, for example, shrinkage), the tunnel elements can be composed of different segments, that are interconnected with expansion joints. The length of each segment depends on the practical length of a single concrete pour and of the risk for shrinkage cracks, and is typically in the range of 20 m. The vertical joint between two segments is provided with a cast-in flexible waterstop (Figure II-13). In this way, the tunnel element can be subjected to flexural deformations without developing longitudinal tensile strain at the location of the expansion joints, which may cause cracking of the concrete. Longitudinal pre-stressing is also sometimes applied to avoid uncontrollable full-depth cracks. Crack inducers at the location of the segment joints can force tensile cracks to occur at the location of segment joints. For the Coatzacoalcos project, an injection hole at the location of the segment joints allow for the repair of such cracks [7][8][10].

An example of rubber waterstops that ensure watertightness between two different casts in the Nanchang project is shown in the middle of Figure II-14.
4.2.3 Shear keys

Shear keys of immersed tunnel have a dual purpose: to avoid discontinuous displacements over the immersed joints in longitudinal and vertical direction, and to transfer shear forces between the tunnel elements. Since neither the water seals, nor longitudinal tendons over immersion joints are suited to take up shear, the transfer of shear forces is the primary function of the shear keys.

The structural configuration of the immersion joint of the HZMB Tunnel in China and a representation of the shear keys in the walls of the Coatzacoalcos project in Veacruz, Mexico were shown in Figure II-7 and Figure II-8.

The vertical shear keys transfer shear forces between adjacent joint under longitudinal bending, and the horizontal concrete reinforced shear keys in the ballasted concrete bear horizontal forces, such as seismic shear forces. The pre-stressed cables will work in for example during seismic events and keep the displacements of segmental joints within their waterproofing limits. Shear key forces can also be expected as a result of foundation stiffness variations, sedimentation loads on the tunnel or gravel bed surface intolerances [7][12].

During transportation and positioning of elements, segments are held together by using longitudinal tendons in order to maintain the integrity of the tunnel element. The tendons can be either passive or pre-stressed. If the tendons are pre-stressed, the differential displacements at the immersion joints can be expected to be lower than in the case where the tendons are passive. The latter case requires more heavy shear keys [8].

The dimensions of the shear keys are an important design issue for the overall tunnel structure, because they can be governing for the wall thickness and the overall structural dimensions of the tunnel [12].

For the Nanchang project, provisions to attach the vertical steel shear keys prior to sinking are shown in Figure II-15.
The horizontal concrete shear keys are only installed after sinking and connecting of the elements. Again for the Nanchang project, a schematic view of the vertical steel rebars with which the concrete shear keys are connected to the ends of the tunnel elements is shown in Figure II-16.
III Design starting points

In this chapter the design starting points for the physical scale model and so also for the FE model are elaborated.

1 Nanchang Red Valley tunnel

The Nanchang Red Valley tunnel construction project comprises the construction of a 1.3 km long immersed tunnel under the Ganjian river in the city Nanchang, which is the capital of China’s Jianxi Province. The tunnel is intended for vehicle traffic in two directions, and has 3 compartments: one for each direction of vehicle traffic and one central compartment for evacuation purposes and for technical provisions.

Counting from West to East, the immersed tunnel is composed of 12 segments in total: 9 segments of 115 m, followed by a segment of 90 m, one of 108 m and a last one of 90 m, respectively. An overview of the construction site is shown in Figure III-1. The elements of the tunnel are constructed in two dry docks, shown on the bottom left of Figure III-1. The site where the tunnel elements are immersed and connected lies approximately 8.5 km from the casting basin, and is indicated as Red Valley Tunnel (红谷隧道) on the right part of Figure III-1.

Figure III-1: overview of the Nanchang construction site

The HPC research group of Tongji University has been asked to assess the behavior of the joints of the Nanchang tunnel under static loads. The assessment of static loads is useful to evaluate the
structural behavior of the immersed joint, for example to find the internal force distribution of loads over the internal load-carrying parts of the immersed joint. Also, during seismic analyses seismic loads are commonly translated into static loads. A first step in the assessment of the Nanchang tunnel segments under seismic loading thus can thus be to investigate the behavior of the segments under static loading.

In order to assess the behavior of the Nanchang Red Valley immersed tunnel under static loads, two modeling techniques are used: performing numerical analyses on a geometrically scaled Abaqus FE software model and by performing a 1:5 physical scale model test at the Nanchang construction site. The aim of the numerical model is to predict the behavior of the physical scale model.

2 Immersed joint shear key configuration

Both the physical scale model and the numerical model of the Nanchang Red Valley immersed tunnel are based on the final design plans for the actual construction project of the tunnel prototype. Some parts of the design of the actual construction project on which both models are based are elaborated. A schematic overview of the locations of shear keys on the faces of 2 adjacent tunnel elements for the Red Valley tunnel is shown schematically in Figure III-2.

![Figure III-2: schematic overview of locations of shear keys on tunnel element](image)

2.1 Tunnel geometry

2.1.1 Cross section

The tunnel elements of the prototype have a total width of 30 m and an overall height of 8.3 m. A cross-section of a tunnel element for the Nanchang Red Valley tunnel project is shown in Figure III-3.
2.1.2 Shear keys

A plan view of the configuration of the concrete shear keys between two tunnel elements is shown in Figure III-4. The teeth of the shear keys are separated by rubber supports, as indicated in Figure III-4. The remaining void between the shear keys of two different elements is filled with high-density polyethylene (HDPE). On the left hand side of the figure one can see the connection location for the Gina gasket and for the omega seal.

2.1.3 Gina gasket and omega seal

The configuration of Gina gasket and omega seal in a vertical section through the immersed joint is shown in Figure III-5. The dimensions of the Gina gaskets in the Red Valley tunnel prototype are also indicated in this figure. The influence of the omega seal on the overall structural behavior of the joint is considered to be negligible, and the omega seal will not be considered any further. The influence of the Gina gasket on the other hand is considered to be of structural importance. The position of a Gina gaskets on a tunnel element is shown in Figure III-6.
2.1.4 Length profile

A schematic representation of the tunnel's length profile is shown in Figure III-7. The expected high and low water level in the river are indicated in this figure as HWL and LWL respectively. Note that in the plane of this figure the horizontal and vertical scale are not equal. In Figure III-7, the open sections at both ends of the tunnel represent the abutments.

Because the behavior of the joints is modeled, the dimensions of the cross section and of the components in the joint (such as the shear keys and the Gina water stops) are of primary interest for the design of the numerical model. In this case the length profile is useful because the water loads on the tunnel bulkheads after immersion affect the lateral load that will be transferred
though the immersed joint. An assessment of these loads for the tunnel prototype has been made in the design stage of the tunnel prototype, and is discussed below.

### 2.2 Axial load

Under the influence of lateral loads on the side of a tunnel element, the concrete shear keys will be loaded most heavily when the axial load on the tunnel face is minimal. When this is the case, the forces that the Gina gaskets exert on the opposite tunnel element is also minimal. According to Coulomb's friction law, this reduces the friction between the Gina gasket and the tunnel face. This has as a result that more load is taken up by the concrete shear keys. One of the loads that is of importance to model the behavior of the concrete shear keys is thus the minimal vertical load on the bulkheads.

A preliminary assessment of the total axial water load on the bulkheads of the Red Valley tunnel just after sinking was made by the Dutch firm Trelleborg. Their results are attached in Addendum I. These results, that are based on historic data of water heights in the Ganjian river, show that the lowest load that is to be expected on a bulkhead just after sinking is 8779 kN. This value will be used to determine the loads that are used in the numerical and physical modeling.
IV Numerical modeling of the Red Valley tunnel

Numerical modeling of the Red Valley tunnel is conducted using FE software package Abaqus. Just like the physical scale model, the dimensions of the numerical model are downscaled geometrically by a factor 5 compared to the actual Red Valley tunnel construction project (prototype). This way, the dimensions in the numerical model and the dimensions of the scale model are the same, and results of the numerical simulation can be compared directly to the results of the scale model without taking into account further scaling effects between both models.

Only the behavior of the immersed joint is simulated, so both the numerical model and the physical scale model comprise 2 adjacent tunnel elements and their common connection. The joint of the numerical model contains Gina gaskets, concrete shear keys with rebars but contrary to the physical model, the numerical model does not contain steel shear keys (cf. infra).

1 The Abaqus software package

Abaqus is a suite of engineering simulation programs based on the FE method. With Abaqus both linear and nonlinear simulations of structural systems can be performed, with a wide range of materials. In a nonlinear analysis Abaqus automatically chooses appropriate load increments to ensure that an accurate solution is obtained. For the simulation of concrete shear key behavior, the package Abaqus/Standard is used. This is a general-purpose analysis suited for solving static responses. Contrary to Abaqus/Explicit, the Abaqus/Standard package solves a system of equations implicitly at each load increment, performing iterations until the solution converges. It has a wide range of material models and has a robust capacity for solving contact problems, and its solution technique is unconditionally stable [13].

The model is preprocessed with the product Abaqus/CAE. This is an interactive graphical environment in which geometries are created and/or imported, and meshed. The material properties, interaction properties, loads and boundary conditions are assigned to the geometry in Abaqus/CAE. It is also used to submit the analysis and to do the postprocessing.
2 Model design

2.1 Dimensional similitude

The scale factor for geometric length of both the physical and numerical model is 1:5. This implies a scale factor of 1/5 for all length quantities, or $S_L = 1/5$. The scale factor for displacements is chosen to be equal to the scale factor for length ($S_x = S_L$). The scale model will use the same materials as the actual prototype, which implies that the E-moduli of the materials in the models and prototype will be identical. This implies $S_E = 1$. From the 3 similitude parameters $S_L$, $S_x$ and $S_E$, all other necessary similitude parameters can be obtained. The similitude parameters for the FE model (and also for the physical scale model) are shown in Table IV-1.

<table>
<thead>
<tr>
<th>quantity</th>
<th>dimensions</th>
<th>scale factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>length</td>
<td>$L$</td>
<td>$S_L$ = 1/5</td>
</tr>
<tr>
<td></td>
<td>$x$</td>
<td>$S_x = S_L$</td>
</tr>
<tr>
<td>area</td>
<td>-</td>
<td>$S_L^2$ = 1/25</td>
</tr>
<tr>
<td>volume</td>
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<td>$S_L^3$ = 1/125</td>
</tr>
<tr>
<td>stress</td>
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</tr>
<tr>
<td>strain</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td>E-modulus</td>
<td>$F L^{-2}$</td>
<td>$S_E$ = 1</td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>-</td>
<td>1</td>
</tr>
<tr>
<td>load</td>
<td>$F$</td>
<td>$S_E S_L^2$ = 1/25</td>
</tr>
<tr>
<td>point load</td>
<td>$F L^{-1}$</td>
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</tr>
<tr>
<td>distributed load</td>
<td>$F L^{-2}$</td>
<td>$S_E$ = 1</td>
</tr>
<tr>
<td>area load</td>
<td>$F L$</td>
<td>$S_E S_L^3$ = 1/125</td>
</tr>
<tr>
<td>moment</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

2.2 Model geometry

The cross-section of the numerical model is scaled geometrically by a factor $S_L = 1/5$ compared to the Nanchang Red Valley tunnel prototype. All dimensions that were mentioned in part III are downscaled by a factor 5, and no changes are made to the overall geometry of the tunnel cross section. An overview of the 2 segments out of which the numerical model is composed was shown previously in Figure III-2 (page 29), with the difference that the steel shear keys are omitted in the numerical model. The rubber supports between the concrete shear keys are also not included in the numerical model. The dimensions of the cross sections of the two tunnel elements are depicted in Figure IV-1. In this figure, the positioning of the concrete shear keys relative to the cross section of the tunnel elements is also indicated for the numerical model. The tunnel elements of both the numerical model and scale model have a depth of 1800 mm measured perpendicular to the plane of Figure IV-1. The dimensions of the concrete shear keys of the FE model are depicted in Figure IV-2. A plan view of the configuration of the concrete shear keys and their dimensions in the numerical model is shown in Figure IV-3. A drawing of the reinforcement bars in the numerical model's concrete shear keys is shown in Figure IV-4. The configuration of the reinforcement bars in
the shear keys in the FE model is taken the same as the configuration in the design of the physical scale model test.
2.3 Boundary conditions

The boundary conditions of the overall numerical model are depicted in Figure IV-5. A convention for the numbering of the two tunnel elements (1 and 2) is also shown in this figure. The bottom part of tunnel element 1 is constrained so it cannot move in the vertical (y-) direction. Rigid body translation of tunnel element 2 is prevented by constraining the indicated area of tunnel element 2 in all three the x-, y- and z-directions (Figure IV-5). These boundary conditions are defined to mimic the boundary conditions of the physical scale model on the Nanchang construction site and do not take into account certain phenomena that might occur with the tunnel prototype. For example differential settlements just after immersion of the tunnel elements is not considered, and thus spring boundary conditions are not considered.

To model the connection between the concrete shear keys and the tunnel elements, tie constraints are used. This means that the relative displacement of all points that the common points on the surfaces of the concrete shear keys and the tunnel elements is set to be zero in all directions. The
same technique is used to model the connection between the Gina gaskets and the concrete tunnel elements.

In the numerical simulation the Gina seal is modeled using six separate straight Gina gaskets. In the tunnel prototype as well as in the physical scale model, the Gina seal is composed of one large part that is connected in a continuous manner to tunnel element 2 (see also Figure III-6 on p. 31). In order to model interaction between the separate parts out of which the Gina seal is composed, the faces that make up the boundaries between the different Gina parts are assigned local boundary conditions stating that these faces can have no movement out of their plane. Referring to Figure IV-6 this means that the hatched surface on the horizontal shear key cannot move in the x-direction, the hatched surface on the vertical shear key cannot move in the y-direction and the hatched areas on the tilted Gina cannot move in the local z’-direction. This simplification of the connections may introduce a more stiff behavior of the Gina seals in the x-y-plane of the numerical model. The effect hereof to the overall solution is assumed to be negligible.

![Figure IV-6: Gina gasket local boundary conditions](image)

### 2.4 Loads

#### 2.4.1 Considered loading case

The load case under investigation in both the numerical and physical scale model is depicted in Figure IV-7. The global coordinate system that will be used throughout the analysis is depicted on the left hand side in Figure IV-7. The x-direction is defined as the lateral direction and the z-direction as the longitudinal one. The y-direction is defined as the vertical direction. The loading case consists of an axial load \( F_x \) on the tunnel face of tunnel element 1 and a lateral load \( F_y \) on the tunnel side of element 1. During the analysis, \( F_x \) is held constant and \( F_y \) is increased incrementally up to the point where complete damage of the concrete shear keys occurs. In the physical scale model test, the horizontal load on the side of the tunnel will be applied through a steal beam with a flange with of 400 mm. Therefore the load on the side of tunnel element \( F_y \) in the numerical model is also applied over a rectangular area with a width of 400 mm along the z-axis.
2.4.2 Axial load on tunnel face

As previously mentioned in paragraph III.2.2, the lowest load that is to be expected on a bulkhead of the tunnel prototype after sinking is 8779 kN. Taking into account the similitude constraints set out in Table IV-1, this load must be scaled by $S_e S_l^2 = 1/25$. The axial load to be exerted on the tunnel face of the scale model and on the numerical model thus becomes

$$F'_t = \frac{8779 \text{ kN}}{25} = 351.16 \text{ kN}.$$  \hspace{1cm} (1)

For practical reasons during the loading of the physical scale model, a load of 360 kN will be exerted. So for both the scale model and the numerical model

$$F_t = 360 \text{ kN}.$$  \hspace{1cm} (2)

In a first stage in the numerical modeling, a lateral load of both 351.16 kN and of 360 kN will be applied in 2 different numerical models, so that it can be judged if the influence of rounding of the lateral load to 360 kN has significant effect on the results of the FE calculations. During the physical scale model testing the axial load will be set to 360 kN.

2.4.3 Lateral loads on tunnel side

The lateral load on the tunnel face is a free parameter in the numerical and physical tests. Before the numerical analysis, the ultimate load that can be applied on the side of the tunnel element is unknown. In a preliminary FE model, a first loading scheme is used in which load on the side of the tunnel element is applied in increments of 100 kN. In later modeling stages different load increments will be used, and the steps at which the loads are applied are changed to smaller values at larger lateral loads. The lateral loading increments in the numerical model are discussed in a later section.
3 Materials and interaction behavior

To ease the construction process of the scale model on the construction site, the concrete and steel types used in the physical scale model will be the same as concrete and steel types used in the construction of the actual Red Valley tunnel construction.

The concrete type for both the tunnel elements and the concrete shear keys used in the physical scale model is C40. The properties of this material are shown in Table IV-2. The concrete in the FE model is modeled using these parameters. Other concrete parameters are explained in paragraph IV3.1.

The steel type used for the rebars in the concrete shear keys is Q345. The elastic parameters that are used for the steel rebars in the numerical model are shown in Table IV-3.

The description of the rubber material is discussed further on in paragraph IV3.4.

<table>
<thead>
<tr>
<th>Table IV-2: plain concrete material parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>concrete type</td>
</tr>
<tr>
<td>mass density</td>
</tr>
<tr>
<td>Young’s modulus</td>
</tr>
<tr>
<td>Poisson ratio</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table IV-3: steel reinforcement material parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>steel type</td>
</tr>
<tr>
<td>mass density</td>
</tr>
<tr>
<td>Young’s modulus</td>
</tr>
<tr>
<td>Poisson ratio</td>
</tr>
</tbody>
</table>

For all materials in the FE model C3D8R elements are used, except for the steel rebars, who are composed of beam elements. The C3D8R element in Abaqus is an 8-node linear brick element with reduced integration, and is used with hourglass control to avoid zero-energy modes. For the steel rebars the T3D2 element type is used. These are linear 3D-truss elements with nodes at each end of each reinforcement bar.

3.1 Plain concrete material model

There are three material models that are commonly used to simulate the behavior of plain and reinforced concrete in Abaqus: the concrete cracking model, the concrete smeared cracking model and the concrete damaged plasticity model [13]. For the numerical model of the Nanchang Red Valley tunnel, a concrete damaged plasticity model is used. This model is chosen because, unlike the concrete smeared cracking model, it can be used without having to assume monotonic straining of the concrete element under examination. Furthermore the concrete damaged plasticity model allows for nonlinear behavior both in tension and in compression. Contrary to the cracking model for concrete, it takes into account both tensile cracking and compressive failure.

The concrete damaged plasticity model assumes low confining pressures, which is assumed to be the case in the loading scheme of the Red Valley model tests. In order to explain certain parameters
that occur in the material model, a brief overview of the governing constitutive relations is provided, based entirely on the plastic damage model for concrete by Lubliner (1988) and by Lee and Fenves (1998) [14][15][16][17]. The uniaxial loading case is formulated first, and is then extended to the multiaxial case. In the preprocessing stage of the numerical model, the uniaxial case is most important because the damaged plasticity model in Abaqus requires uniaxial parameters as input. Abaqus automatically converts these parameters to the multiaxial case, that is also explained briefly.

3.1.1 Uniaxial loading

The concrete damaged plasticity model assumes a decomposition of the strain into an elastic part $\varepsilon^{\text{el}}$ and a plastic part $\varepsilon^{\text{pl}}$:

$$
\varepsilon = \varepsilon^{\text{el}} + \varepsilon^{\text{pl}}.
$$

The assumed behavior of concrete in uniaxial tension and compression is shown in Figure IV-8 and Figure IV-9. The material behavior is such that for both the compression and the tension case, the initial elastic stiffness $E_0$ is reduced when the softening branch is reached. This reduction of the initial stiffness is proportional to the damage that occurs in the concrete material and is reflected into the constitutive relations by the damage parameters $d_t$ and $d_c$, where subscripts $t$ and $c$ denote tension and compression, respectively. Both damage parameters are scalar degradation variables that range from 0 to 1, where the value 0 denotes no damage and the value 1 denotes fully damaged material.

For the uniaxial case, $\tilde{\varepsilon}_t^{\text{pl}}$ and $\tilde{\varepsilon}_c^{\text{pl}}$ are the equivalent plastic strains in tension and compression, and characterize the damaged state of the material. $\tilde{\varepsilon}_t^{\text{ck}}$ and $\tilde{\varepsilon}_c^{\text{in}}$ are defined as the cracking strain in tension and the inelastic (crushing) strain in compression, respectively.

The stress-strain relations under uniaxial tension and compression loading are (Figure IV-8 and Figure IV-9):

$$
\sigma_t = (1 - d_t) E_0 \left( \varepsilon_t - \tilde{\varepsilon}_t^{\text{pl}} \right) 
$$

$$
\sigma_c = (1 - d_c) E_0 \left( \varepsilon_c - \tilde{\varepsilon}_c^{\text{pl}} \right).
$$

The yield condition in the multiaxial case will in a later paragraph be defined in terms of effective uniaxial stresses, which are defined as

$$
\bar{\sigma}_t = E_0 \left( \varepsilon_t - \tilde{\varepsilon}_t^{\text{pl}} \right) 
$$

$$
\bar{\sigma}_c = E_0 \left( \varepsilon_c - \tilde{\varepsilon}_c^{\text{pl}} \right).
$$
The stress-strain relations under uniaxial tension and compression loading as depicted in Figure IV-8 and Figure IV-9 are implemented into the FE model in Abaqus by specifying the cracking strain $\varepsilon_{t}^{ck}$ and the inelastic (crushing) strain $\varepsilon_{t}^{in}$ as a tabular function of the inelastic stresses $|\sigma_{t}|$. Inelastic stresses are defined as stresses for which the strains reach values beyond the linear elastic branch of the stress-strain curves.

The values of $\varepsilon_{t}^{ck}$ and $\varepsilon_{t}^{in}$ for C40 concrete are calculated using Appendix C of the Chinese Code for Design of Concrete Structures GB 50010-2010 [18]. The values that are used in the numerical model are shown in Table IV-4. A graphical representation of the values in Table IV-4 is depicted in Figure IV-10. It is noted that, contrary to Figure IV-8 and Figure IV-9, these graphs only depict uniaxial inelastic strains $\varepsilon_{t}^{ck}$ and $\varepsilon_{t}^{in}$, in function of the absolute stresses beyond the linear elastic branch.

The calculation of the tensile case of these values is explained in Addendum II of this document. The calculations for the compressive case are similar to the tensile case.
Table IV-4: inelastic stresses $|\sigma_c|$ as functions of the cracking strain and inelastic strain in numerical model

<table>
<thead>
<tr>
<th>Inelastic tensile stress [N/mm²]</th>
<th>Cracking Strain $\varepsilon_c^{ck}$ [-]</th>
<th>Inelastic compressive Stress [N/mm²]</th>
<th>Inelastic Strain $\varepsilon_c^{in}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2,3900</td>
<td>0</td>
<td>13,7092</td>
<td>0</td>
</tr>
<tr>
<td>2,0000</td>
<td>7,9482E-05</td>
<td>25,7386</td>
<td>4,0732E-04</td>
</tr>
<tr>
<td>0,7939</td>
<td>2,9552E-04</td>
<td>26,7995</td>
<td>8,2404E-04</td>
</tr>
<tr>
<td>0,4689</td>
<td>5,4542E-04</td>
<td>22,3577</td>
<td>1,7092E-03</td>
</tr>
<tr>
<td>0,2238</td>
<td>1,3921E-03</td>
<td>13,4059</td>
<td>3,4799E-03</td>
</tr>
<tr>
<td>0,1716</td>
<td>1,9927E-03</td>
<td>5,6548</td>
<td>7,7942E-03</td>
</tr>
<tr>
<td>0,0889</td>
<td>4,9848E-03</td>
<td>3,9156</td>
<td>1,0820E-02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2,3433</td>
<td>1,7277E-02</td>
</tr>
</tbody>
</table>

Figure IV-10: uniaxial strains in function of inelastic stresses $\sigma_c$

Under uniaxial loading conditions the effective plastic strain rates are both defined as positive scalars. They are given as

$$\dot{\varepsilon}_t^{pl} = \dot{\varepsilon}_t^{pl}$$  \hspace{1cm} (8)

$$\dot{\varepsilon}_c^{pl} = -\dot{\varepsilon}_c^{pl}$$  \hspace{1cm} (9)

for uniaxial tension and for uniaxial compression, respectively. $\varepsilon_t^{pl}$ is the uniaxial plastic strain, and $\varepsilon_c^{pl}$ is its time derivative. It is noted that the tensile strains are defined as positive.

Given these effective plastic strain rates, the equivalent plastic strains under uniaxial loading are
where the integration is carried out over the time step over which the numerical calculation is performed. For the analysis at hand, typical time steps are between $10^{-4}$ sec and 0.25 sec.

Equations (8)-(11) define the equivalent plastic strains under uniaxial loading conditions. As shown in Figure IV-8 and Figure IV-9, the degradation of the elastic stiffness is characterized by the two uniaxial damage variables $d_t$ and $d_c$, who are assumed to be mutually independent. These damage variables are assumed to be a direct function of the equivalent plastic strains:

$$d_t = d_t \left( \varepsilon_t^{PL} \right)$$

$$d_c = d_c \left( \varepsilon_c^{PL} \right).$$

For the practical implementation of the concrete damaged plasticity model in Abaqus, the damage variables $d_t$ and $d_c$ are defined directly as tabular functions of the cracking strain and inelastic crushing strain $\varepsilon_t^{ck}$ and $\varepsilon_c^{in}$. The tabular functions that are implemented in the numerical model are mentioned in Table IV-5. These values are for concrete C40 and were calculated on the basis of Appendix C of the Chinese code for design of concrete structures GB 50010-2010 [18] (see also Addendum II). The numerical values in Table IV-5 are depicted graphically in Figure IV-11.

Because the constitutive relations for tensile behavior are based on damage parameters in function of the equivalent plastic strain, Abaqus has to convert the cracking strain values $\varepsilon_t^{ck}$ to equivalent plastic strains. Based on Figure IV-8, the relation between the cracking strain values $\varepsilon_t^{ck}$ and the plastic strain values $\varepsilon_t^{PL}$ is

$$\varepsilon_t^{PL} = \varepsilon_t^{ck} - \frac{d_t}{(1 - d_t) E_0} \sigma_t.$$  

Similarly for the compressive case the relation between the inelastic strain values $\varepsilon_c^{in}$ and the inelastic (crushing) strain $\varepsilon_c^{in}$ is

$$\varepsilon_c^{PL} = \varepsilon_c^{in} - \frac{d_c}{(1 - d_c) E_0} \sigma_c.$$  

$$
\varepsilon_t^{PL} = \int_0^t \dot{\varepsilon}_t^{PL} \, dt
$$

$$
\varepsilon_c^{PL} = \int_0^t \dot{\varepsilon}_c^{PL} \, dt
$$
3.1.2 Multiaxial conditions

The concrete damaged plasticity model for uniaxial conditions is extended to the multiaxial case.

The stress-strain relations of the damaged plasticity model for the case of general tree-dimensional multi-axial loading are given by

\[ \sigma = (1 - d) D_0^{el} \cdot (\varepsilon - \varepsilon^{pl}), \]

where \( D_0^{el} \) is the initial undamaged elasticity matrix, \( \sigma \) and \( \varepsilon \) are the stress and strain vectors and \( \varepsilon^{pl} \) is the plastic part of the strain vector. \( d \) is the scalar degradation variable \( (0 < d < 1) \), where \( d = 0 \) denotes no damage and \( d = 1 \) denotes fully damaged material.

The plastic behavior will be formulated in terms of effective stress, which is denoted as
\[ \bar{\sigma} := D_0^{pl} \cdot (\varepsilon - \varepsilon^{pl}). \] (17)

For the multiaxial case, the equivalent plastic strain vector is defined as
\[ \dot{\varepsilon}^{pl} := \begin{bmatrix} \dot{\varepsilon}_t^{pl} \\ \dot{\varepsilon}_c^{pl} \end{bmatrix} \] (18)

where it is assumed that for the multiaxial case the equivalent plastic strains in tension and compression are given by
\[ \dot{\varepsilon}_t^{pl} := r(\bar{\sigma}) \varepsilon_{\text{max}}^{pl} \] (19)
\[ \dot{\varepsilon}_c^{pl} := -\left(1 - r(\bar{\sigma})\right) \varepsilon_{\text{min}}^{pl} \] (20)

where \(\varepsilon_{\text{max}}^{pl}\) and \(\varepsilon_{\text{min}}^{pl}\) are the maximum and minimum eigenvalues of the 3x3 plastic strain rate tensor \(\dot{\varepsilon}^{pl}\), and \(r(\bar{\sigma})\) is a stress weight factor that takes into account the sign of the principal stresses.

\[ r(\bar{\sigma}) := \frac{\sum_{i=1}^{3}(\bar{\sigma}_i)}{\sum_{i=1}^{3}|\bar{\sigma}_i|}; \quad 0 \leq r(\bar{\sigma}) \leq 1. \] (21)

\(r(\bar{\sigma}) = 1\) when all principal stresses are positive and it is 0 when all principal stresses are negative. In equation (21), the operator \langle \cdot \rangle is defined by \(\langle x \rangle := \frac{1}{2}(|x| + x)\).

Finally, the concrete damaged plasticity model assumes following relation between the multiaxial scalar degradation variable \(d\) and its uniaxial counterparts \(d_t\) and \(d_c\):
\[ (1 - d) = (1 - d_c) (1 - r(\bar{\sigma}) d_t). \] (22)

3.1.3 Yield condition

The plastic-damage concrete model uses a yield condition in terms of effective stresses of the form
\[ F(\bar{\sigma}, \dot{\varepsilon}^{pl}) \leq 0 \] (23)

where
\[ F(\bar{\sigma}, \dot{\varepsilon}^{pl}) = \frac{1}{(1-\alpha)} \left( \bar{q} - 3\alpha \bar{p} + \beta \dot{\varepsilon}^{pl} \langle \bar{\sigma}_{\text{max}} \rangle - \gamma (-\bar{\sigma}_{\text{max}}) \right) - \bar{\alpha} \dot{\varepsilon}^{pl}. \] (24)

In equation (24), \(\bar{p}\) is the effective hydrostatic pressure and \(\bar{q}\) is the Mises equivalent effective stress:
\[ \bar{p} = -\frac{1}{3} \bar{\sigma} \cdot I. \] (25)
\[ \bar{q} = \left( \frac{3}{2} \bar{S} \cdot \bar{S} \right)^{\frac{1}{2}} \]  

(26)

where

\[ \bar{S} = \bar{\sigma} + \bar{p} \mathbf{I} \]

is the deviatory part of the effective stress tensor \( \bar{\sigma} \). \( \bar{\sigma}_{\text{max}} \) is the maximum eigenvalue of \( \bar{\sigma} \). The function \( \beta(\bar{\varepsilon}^{\text{pl}}) \) is given by

\[ \beta(\bar{\varepsilon}^{\text{pl}}) = \frac{\bar{\sigma}_c \left( \bar{\varepsilon}_c^{\text{pl}} \right)}{\bar{\sigma}_t \left( \bar{\varepsilon}_t^{\text{pl}} \right)} (1 - \alpha) - (1 + \alpha) \]

in which the effective tensile and compressive stresses \( \bar{\sigma}_t \left( \bar{\varepsilon}_t^{\text{pl}} \right) \) and \( \bar{\sigma}_c \left( \bar{\varepsilon}_c^{\text{pl}} \right) \) are defined by equations (6) and (7).

The coefficient \( \alpha \) is a dimensionless material constant, and is determined by the initial equibiaxial compressive yield stress \( \sigma_{\text{b0}} \) and the initial uniaxial compressive yield stress \( \sigma_{c0} \), as

\[ \alpha = \frac{\sigma_{\text{b0}} - \sigma_{c0}}{2\sigma_{\text{b0}} - \sigma_{c0}}. \]  

(27)

The ratio \( \frac{\sigma_{\text{b0}}}{\sigma_{c0}} \) is an important parameter in the concrete damaged plasticity model in Abaqus. Typical experimental values of the ratio \( \frac{\sigma_{\text{b0}}}{\sigma_{c0}} \) range from 1.10 to 1.16 [13][15]. In the numerical model of the Nanchang Red valley tunnel, a value \( \frac{\sigma_{\text{b0}}}{\sigma_{c0}} = 1.16 \) is used. This value is based on investigations by Liu, Xu and Chen [16].

The dimensionless material constant \( \gamma \) is expressed in the material model in terms of a parameter \( K_c \):

\[ \gamma = \frac{3(1 - K_c)}{2K_c - 1} \]  

(28)

where it is reasonable to assume that \( K_c \) is constant. A typical value for plain concrete is \( K_c = \frac{2}{3} \) [13][15][16].

3.1.4 Flow rule

The flow rule in the plastic-damage concrete model is

\[ \dot{\bar{\varepsilon}}^{\text{pl}} = \dot{\lambda} \frac{\partial G(\bar{\sigma})}{\partial \bar{\sigma}} \]  

(29)

where the flow potential \( G \) is expressed as

\[ G = \sqrt{(\epsilon \sigma_{t0} \tan \psi)^2 + \bar{q}^2} - \bar{p} \tan \psi. \]  

(30)
\( \sigma_{\text{fu}} \) is the uniaxial tensile stress at failure, \( \psi \) is the dilatation angle and \( \epsilon \) is the eccentricity. Both \( \psi \) and \( \epsilon \) are related directly to the shape of the flow potential function, and \( \epsilon \) defines the rate at which the flow potential function approaches its asymptote.

For the Nanchang Red Valley tunnel numerical model, the values \( \psi = 30^\circ \) and \( \epsilon = 0.1 \) are assumed, in accordance with Liu, Xu and Chen [16].

### 3.1.5 Viscoplastic regularization

Because the plastic-damage concrete model exhibits softening behavior and because the FE analysis is conducted in Abaqus using an implicit solver method, the stiffness degradation inherent to the material model may lead to severe convergence difficulties. Some convergence difficulties can be overcome by using viscoplastic regularization. This regularization permits stresses to be outside the yield surface during convergence. This is done by replacing the inviscid strain tensor \( \mathbf{\varepsilon}^\text{pl} \) by a viscoplastic strain tensor \( \mathbf{\dot{\varepsilon}}^\text{pl} \). The viscoplastic strain tensor \( \mathbf{\dot{\varepsilon}}^\text{pl} \) is defined through the viscoplastic strain rate tensor \( \mathbf{\dot{\varepsilon}}^\text{pl} \):

\[
\mathbf{\dot{\varepsilon}}^\text{pl} = \frac{\partial \mathbf{\varepsilon}^\text{pl}}{\partial t} = \frac{1}{\mu} (\mathbf{\varepsilon}^\text{pl} - \mathbf{\varepsilon}^\text{pl})
\]

(31)

where this time \( \mu \) is a viscosity parameter representing the relaxation time of the viscoplastic system.

A viscous stiffness degradation variable \( d_v \) for the viscoplastic system is defined similarly through

\[
\dot{d}_v = \frac{\partial d_v}{\partial t} = \frac{1}{\mu} (d - d_v)
\]

(32)

with \( d \) the inviscous degradation variable from equation (22). The stress strain relation (16) for the viscoplastic model thus becomes

\[
\mathbf{\sigma} = (1 - d_v) \mathbf{D}^\text{pl} \cdot (\mathbf{\varepsilon} - \mathbf{\varepsilon}^\text{pl}).
\]

(33)

Using viscoplastic regularization with a value of the viscosity parameter that is small compared to the characteristic time increment in which the calculation takes place may help improve the convergence of the model in the softening branch, without compromising results. The solution of the viscoplastic system converges to that of the inviscid case as \( \frac{t}{\mu} \to \infty \). Because for the analysis at hand typical time steps are between \( 10^{-4} \) sec and 0.25 sec, a viscosity parameter \( \mu = 10^{-5} \) was initially suggested. Nevertheless, this value caused severe convergence difficulties beyond a side load \( F_s > 250 \) kN, while preliminary results indicated that the ultimate load of the concrete shear keys had not yet been reached. Therefore, the viscosity parameter was adjusted to \( \mu = 10^{-3} \). This way numerical stability of the calculations is ensured up to failure of the concrete shear keys.
3.1.6 Summary

The parameters of C40 concrete for the plastic-damage concrete model in the Nanchang Red Valley tunnel numerical model are summarized in Table IV-6. Other variables were mentioned in Table IV-4 and Table IV-5.

<table>
<thead>
<tr>
<th>$\psi$ [°]</th>
<th>$\epsilon$ [-]</th>
<th>$\sigma_{00}/\sigma_{c0}$ [-]</th>
<th>$K_c$ [-]</th>
<th>$\mu$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>0,1</td>
<td>1,16</td>
<td>2/3</td>
<td>$10^{-3}$</td>
</tr>
</tbody>
</table>

3.2 Concrete shear key teeth interaction behavior

To model the mechanical behavior that occurs when the teeth of the concrete shear keys make contact due to the lateral loading, the Abaqus default “hard” contact model is used. With this method the surfaces transmit no contact pressure unless the elements of both surfaces make contact (Figure IV-12). Additionally, no penetration between the element nodes of one surface into the element of another surface is allowed [19].

![Figure IV-12: Contact pressure-clearance relationship for “hard” contact [20]](image)

The condition of no penetration is enforced by the default constraint enforcement method, namely the linear penalty method. The physical pressure-overclosure relationship of the linear penalty method is depicted in Figure IV-13. With the linear penalty method Abaqus will by default set the penalty stiffness $K_n$ to 10 times a representative underlying element stiffness. The Abaqus Analysis User’s guide mentions that contact penetrations of element nodes into adjacent contacting elements resulting from the default penalty stiffness will not significantly affect the results in most cases; however, these penetrations can sometimes contribute to some degree of stress inaccuracy [20].

![Figure IV-13: linear penalty method [21]](image)
In the linear penalty method the intrusion of the nodes of one element of a concrete shear key into an element on an adjacent shear key is 'penalized' by a reaction force on the intruding node, proportional to the penalty stiffness $K_{in}$ of the element that is being intruded.

The separation of elements after they have made contact during the numerical analysis is set to be allowed. The contact definition allows for partial contact and is defined in all directions.

### 3.3 Steel reinforcement

#### 3.3.1 Steel material

Steel is a homogeneous and isotropic material that can yield and that is not expected to show brittle behavior during the Nanchang scale model test. For the numerical analysis this material is modeled using a standard material definition in Abaqus, and only a brief description of its material properties is given. The physical properties of Q345 steel used in the numerical model have been summarized previously in Table IV-3. A plastic model with isotropic hardening is chosen. Again, the plastic behavior is described by defining the inelastic strain as a tabular function of stress outside the linear elastic region. The plastic behavior as defined in the numerical model is given in Table IV-7. This data is shown graphically in Figure IV-14.

<table>
<thead>
<tr>
<th>stress [N/mm²]</th>
<th>Inelastic strain [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>283</td>
<td>0</td>
</tr>
<tr>
<td>403</td>
<td>0.2787</td>
</tr>
</tbody>
</table>

![Table IV-7: inelastic strain in function of stress beyond elastic region](image)

**Figure IV-14: steel plastic behavior definition in numerical model**

#### 3.3.2 Defining reinforcement bars in Abaqus

In the numerical model the concrete shear keys contain reinforcement (Figure IV-4). To simulate reinforcement in Abaqus, the "embedded elements" method is used. In this method the rebars are defined as truss elements, and their nodes are tied to the nodes of the concrete material in which they are embedded. This way, the rebars follow the displacements of the concrete elements in
which they are embedded and contribute to the mass and stiffness of the concrete elements by which they are surrounded [13].

### 3.4 Rubber material model

To simulate the behavior of the rubber Gina gaskets, a Mooney-Rivlin material model is used. Because rubber is a hyperelastic material, a strain energy potential ($W$), rather than a Young’s modulus and Poisson’s ratio is used to relate stresses to strains.

$$W = C_{10}(I_1 - 3) + C_{01}(I_2 - 3)$$  \hspace{1cm} (34)

where $I_1$ and $I_2$ are the first and second invariants of Green and are measures of the distortion that occurs in the material. $C_{10}$ and $C_{01}$ are empirical material constants that are related to the shear behavior of the material. $W$ is the strain energy potential. Modeling the behavior of a rubber material requires a third parameter, $D_1$, that relates to the compressibility of the material. For an incompressible material, $D_1$ equals zero [22][23].

For the numerical simulations of the Nanchang tunnel project, the constants $C_{10}$ and $C_{01}$ were determined previously through numerical calibration studies by the Tongji University HPC group. In these calibrations, a Gina gasket was modeled in Abaqus with different parameter values for $C_{10}$ and $C_{01}$, until the force-compression curve of the numerical model fits the curve of the Gina for the Gina gaskets that are used in the tunnel prototype. These numerical calibrations have led to the values

$$C_{10} = 0,5820$$
$$C_{01} = 0,0291$$  \hspace{1cm} (35)

and they are verified as follows.

According to Dan [23], a relation between empirical constants $C_{10}$ and $C_{01}$ is

$$\log(C_{10} + C_{01}) + \log(6) = 0,0198 H_r - 0.5432$$  \hspace{1cm} (36)

where $H_r$ is the hardness of the rubber material. Furthermore, according to Zuo [24] one can assume a ratio $\frac{C_{01}}{C_{10}} = 0,05$. Taking into account the obtained values for $C_{10}$ and $C_{01}$ (equation (35)), the hardness $H_r$ can be obtained from equation (36), yielding $H_r = 56$ Shore A. This value is in accordance with the model of the Hong Kong–Zhuhai–Macao immersed tunnel performed by Xiao [1], where the rubber hardness is in the range 55-60 Shore A, and thus the values for $C_{10}$ and $C_{01}$ are deemed realistic. The parameters of the rubber material definition in the numerical model are summarized in Table IV-8.

<table>
<thead>
<tr>
<th>Table IV-8: numerical model rubber material parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>mass density [kg/mm³]</td>
</tr>
<tr>
<td>1,15E-06</td>
</tr>
</tbody>
</table>

50
3.5 Rubber-concrete interaction behavior

To model the friction behavior that will occur between the Gina gaskets and concrete tunnel element 1, a Coulomb friction model is used. The frictional behavior between the surfaces is thus characterized by a coefficient of friction $\mu$.

$$ \tau = \mu \ p $$

Where $\tau$ is the shear stress value, $\mu$ is the Coulomb friction parameter and $p$ is the contact pressure between the two surfaces. The Coulomb friction parameter $\mu$ between the rubber Gina gasket and between the concrete of the immersed tunnel is taken as 0.3 based on [1]. For the normal pressure $p$ “hard” contact model is used just like with the concrete shear keys, and the condition of no penetration is again enforced by the default linear penalty method.

4 Numerical test results

4.1 Preliminary calculations

Prior to the numerical calculations, the maximal lateral load that the numerical model can withstand before failure of the concrete shear keys is hard to assess. In order to estimate the ultimate lateral load, a preliminary numerical model is used in which lateral loads increments of 100 kN are applied. These calculations are done with relatively large load steps and very limited output requests in order to limit the calculation time. Beyond a lateral load of 300 kN, the preliminary model shows severe convergence problems, and values of the damage parameter of the concrete shear keys $d_1$ approximate the value 1 at numerous locations on the concrete shear keys. This result is seen as an indication that the ultimate load of the numerical model is between 300 kN and 400 kN and is used to decide upon the magnitude of the load increments in more detailed numerical models, that can confirm this first estimation of the ultimate lateral load.

4.2 Load increments

The numerical model is loaded with the axial-lateral loading case indicated previously in Figure IV-7 (page 38). This loading case consists of an axial load $F_x$ in the negative z-direction and a lateral load $F_s$ in the positive x-direction. It is emphasized that the lateral load $F_s$ is applied on a strip with width 400 mm only, and not on the whole side area of the tunnel element 1. For the convention of the local coordinate system and the definition of tunnel element 1 and tunnel element 2, reference is made to Figure IV-7. The areas in the numerical model of the tunnel face (without bulkheads) and of the part of the tunnel side on which load is applied are shown in Table IV-9.

<table>
<thead>
<tr>
<th>Table IV-9: area of tunnel face and tunnel side of numerical model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area [mm²]</td>
</tr>
<tr>
<td>tunnel face</td>
</tr>
<tr>
<td>tunnel side</td>
</tr>
</tbody>
</table>
The loads in the numerical model are applied in 9 steps, and the length of every step is chosen to be 1 second. To improve convergence, the load is incremented linearly over the length of every step. The load steps are shown graphically in Figure IV-15 for the axial load $F_f$ on the tunnel face and on Figure IV-16 for the lateral load $F_s$ on the tunnel side.

For the axial loading, two different cases are considered separately. Both an axial load $F_f = 351.16$ kN and $F_f = 360$ kN are applied on the tunnel face (cfr. equations (1) and (2)), in 2 distinct models. This allows to evaluate to a limited extent the influence of axial load on the behavior of the immersed joint and the concrete shear keys, and the effect of rounding off the axial load value in the physical scale model tests.

<table>
<thead>
<tr>
<th>Table IV-10: loading cases</th>
</tr>
</thead>
<tbody>
<tr>
<td>axial load [kN]</td>
</tr>
<tr>
<td>load case 1</td>
</tr>
<tr>
<td>load case 2</td>
</tr>
</tbody>
</table>

The lateral loads are applied in increments of 100 kN, up to the point where the axial load reaches 200 kN. Beyond 200 kN, the load increments are reduced to 25 kN until the total load reaches 300 kN, after which the increments become 10 kN.
4.3 Ultimate lateral load

The ultimate lateral load that the model of the Red Valley tunnel can withstand can be assessed in different ways. On a local level, the damage parameters of the concretes shear keys can be used. On a macroscopic level the relative displacement the concrete shear keys in tunnel element 1 relative to a fixed coordinate system is used to assess the ultimate lateral load. The relation between the applied lateral loads and the maximum of the lateral (x-)displacements of all points on the concrete shear keys in tunnel element 1 is shown in Figure IV-17. Both the case of a lateral load of 351.16 kN and of a lateral load of 360 kN are included in this figure.
For both values of the axial load, the load-displacement curves are identical up to a lateral load of 250 kN, and a clear linear elastic behavior is present at a lateral load up to about 200 kN (region ○ in Figure IV-17), after which the stiffness degrades significantly. For the 360 kN axial loading case, the displacement is 0.105 mm at lateral load of 100 kN, from which the initial lateral stiffness of the joint can be obtained as

\[ k_{l1} = \frac{100 \text{ kN}}{0.105 \text{ mm}} = 952.4 \text{ kN/mm} \] (38)

At a lateral load of 210 kN, a first relatively sudden loss of lateral stiffness seems to occur. A second and more sudden increase in lateral displacement occurs just after the lateral load of 250 kN. At this stage the initial lateral stiffness has degraded significantly (region ○). Between the application of the lateral loads of 250 kN and 300 kN, the stiffness has degraded to approximately

\[ k_{l2} = \frac{259.6 \text{ kN} - 299.3 \text{ kN}}{0.531 \text{ mm} - 1.087 \text{ mm}} = 71.4 \text{ kN/mm} \] (39)

These results show that the stiffness of the joint is reduced by 92.5% when the lateral load reaches two thirds of the ultimate value at failure \( F_u \).

Beyond a lateral load of 300 kN (region ○), the maxima of the displacements of all points on the shear keys in tunnel element 1 becomes very large compared to the applied lateral load increments. At a lateral load of 314 kN the load-displacement relation becomes asymptotical, indicating complete loss of stiffness \( k_{l3} = 0 \text{ kN/mm} \) and thus failure of the concrete shear keys. Under an axial load of 360 kN, the ultimate lateral load that can be applied to the model, resulting in complete damage of the concrete shear keys is thus estimated to be

\[ F_u := 314 \text{ kN} \] (40)

The influence of the two slightly different axial loads is only significant to a limited extent in regions ○ and ○ of the load-displacement curve. Sudden decrease of stiffness is inherent to damage behavior of brittle materials, which clearly occurs in region ○ of the load-displacement curve. In that part the load-displacement curves of load case 1 and load case 2 are no longer identical, as the sudden decrease in stiffness occurs at different lateral loads. This indicates that the axial compression load does have an influence on the load-displacement behavior of the immersed joint. The fact that the ultimate load for loading case 2 is larger than for loading case 1 is explained by the fact that under larger compression loads, the Gina gaskets take up additional loads through friction between concrete element 1 and the Gina gaskets according to Coulomb’s friction law.

### 4.4 Concrete shear key damage

The results of the numerical model are used to predict the damage behavior of the concrete shear keys. Damage is defined as a change in material properties that adversely affects the performance (in this case the stiffness) of the material, and in this case cracking is considered to be a manifestation of damage. According to Lubliner [15], cracking initiates at a point where the maximal
principal plastic strain is positive and the tensile equivalent plastic strain $\varepsilon_{pl}^t$ is greater than zero. An estimation is made of the loads at which visible macro-cracks in the concrete shear keys occur.

The convention with which the concrete shear keys are indicated is shown in Figure IV-18. Looking down on the x-z plane (plan view), the four shear keys are numbered in a clockwise manner. The interlocking parts of the respective shear keys are labeled in Figure IV-18. They are defined as the shear key teeth. The global coordinate system was defined previously in Figure IV-7. In what follows, the axial load on the tunnel face is always 360 kN.

![Figure IV-18: concrete shear key indication convention](image)

4.4.1 Occurrence of cracks in concrete shear keys

The maximal principal plastic strain and the tensile equivalent plastic strain of the concrete shear keys at a lateral load $F_s = 79$ kN are indicated in Figure IV-19 and Figure IV-20, respectively. These results suggest that cracking initiates in the upper left corner of part 1.2 of shear key 1, as at this location the maximal principal plastic strain and the tensile equivalent plastic strain $\varepsilon_{pl}^t$ are both significantly positive. At the upper left corner of part 1.2 of shear key 1, the maximum principal plastic strain and the equivalent tensile plastic strain both reach values above $10^{-4}$, which is deemed sufficient to initiate cracking [14]. At the upper left corner of shear key part 1.4, the maximum principal plastic strain and equivalent plastic strain both locally reach maxima as well, although the values are smaller than at the location of shear key part 1.2.

Based on these results it can be concluded that visual cracking of the concrete shear keys is expected to initiate in the upper left corner of part 1.2, and in a later stage in the upper left corner of part 1.4. Cracking is expected to be visibly present in the scale model test at a lateral load for which $F_s > 80$ kN ($F_s/F_u > 0.25$). In the results of the FE model this is confirmed by the values for the tensile concrete damage parameter $d_{ct}$, which at this lateral load reaches a value of about 0.5 at the upper left corner of part 1.2 (Figure IV-21).

The evolution of the maximal principal plastic strain and of tensile equivalent plastic strain $\varepsilon_{pl}^t$ with $F_s$ while $F_f = 360$ kN is shown graphically in Addendum III and Addendum IV, respectively. The results in Addendum III and Addendum IV are for lateral loads $F_s \geq 100$ kN ($F_s/F_u \geq 0.32$). For shear key part 1.4 both these strains reach values above $10^{-4}$ at a lateral load of 100 kN (Addendum III and Addendum IV).
At the upper left corner of shear key part 1.4, the maximum principal plastic strain and equivalent plastic strain both locally reach maxima as well, although the values are smaller than at the location of shear key part 1.2.

Figure IV-19: maximal principal plastic strain $[\cdot]$ at $F_s = 79 \text{kN}$

Figure IV-20: tensile equivalent plastic strain $\varepsilon_{i}^{pl} \ [\cdot]$ at $F_s = 79 \text{kN}$
Following the reasoning that was set out above, the results in Addendum III and Addendum IV show the occurrence of cracks in tooth 1.2 and 1.4 of shear key 1. Also, the numerical results in Addendum III and Addendum IV indicate that cracking behavior occurs in shear key parts 4.1 and 1.3 at lateral loads \( F_s \geq 120 \text{kN} (F_s/F_u \geq 0.38)\). Above this load both the maximal principal plastic strain and tensile equivalent plastic strain reach values above \(10^{-4}\). At parts 2.2 and 2.4 of shear key 2 cracking seems to occur when the lateral load \( F_s \) reaches values above 140 kN (\(F_s/F_u > 0.46\)). It becomes clear that damage at high lateral loads due to cracking is expected to be most severe at part 1.2 of concrete shear key 1. At lateral loads \( F_s \) beyond 146 kN, the maximal principal plastic strain and tensile equivalent plastic strain both reach values above \(10^{-3}\) at shear key teeth 2.2 and 2.4, with concrete tensile damage parameter \(d_t\) locally above 95% (Figure IV-22) in the upper left corners of all three of the loaded tooth of concrete shear key 1. Referring to Figure IV-17, it is around this point in region \(\Box\) that the macroscopic behavior of the joint starts to deviate from the linear part, and that a degradation of the stiffness of the immersed joint is observed.
4.4.2 Joint stiffness degradation

A significant reduction in stiffness was noted in part ② of the load-displacement curve in Figure IV-17 at lateral load values for which $F_s/F_u > 0.8$ (or $F_s > 250$ kN). The values of maximal principal plastic strain and of tensile equivalent plastic strain in Addendum III and Addendum IV indicate that at this load cracking occurs at all four concrete shear keys and the concrete tensile damage parameter reaches values up to 0.99 in all shear key teeth (Figure IV-23). It is concluded that at this point complete degradation of the shear key teeth is taking place.
Although the physical scale model tests have to confirm the actual degradation behavior, the numerical results indicate that part 1.4 is most likely to first encounter complete failure. It is in this part that a continuous region is formed where the principal plastic strain and tensile equivalent plastic strain are both significantly positive. Based on the numerical results shown in Addendum III and Addendum IV, this happens at a lateral load for which $F_s/F_u$ is approximately 0.96 (or $F_s = 300$ kN). The load-displacement curve in Figure IV-17 indeed shows that at a lateral load of 300 kN the stiffness of the joint has diminished significantly, and the relative lateral displacement of the 2 sets of concrete shear keys at this load reaches values of over 1 mm. To illustrate the deformation field of the concrete shear keys, the absolute displacement in the x-direction of the shear keys (relative to a fixed external coordinate system) at a lateral load $F_s = 300$ kN is shown in Figure IV-24. The attention is drawn to the fact that the lateral displacement of concrete shear key teeth 1.4 and 4.3 is larger than that of the other shear key teeth.

The evolution of the Mises stress in all rebars is depicted in Figure IV-25. The load regions ① and ② that were defined in Figure IV-17 are shown in this figure, as well as the yield stress of the Q345 reinforcement steel, $f_y = 283$ N/mm². Figure IV-25 depicts that in region ① the rebars show a behavior that is approximately linear, as is to be expected. In this region the behavior of the rebars is also linear-elastic. In region ② the slope with which the Mises stresses increases is augmented, up to the point where the yield strength of the first steel rebar is reached according to the Mises criterion. This happens at a lateral load of 0.86 $F_u$ which is considered to be at a sufficiently high load. In case the rebars would already yield in region ①, this could have raised concerns about the design of the physical scale model test. The transient behavior that is observed between regions ① and ② in the load-displacement curve Figure IV-17 can also be found in Figure IV-25.
The direction in which cracks propagate is assumed to be orthogonal to that of the maximum principal plastic strain, as explained in the next paragraph.

### 4.4.3 Crack directions

Using the concrete plastic damage model by Lubliner [15], the direction of the cracks is assumed to be orthogonal to the direction of the maximum principal plastic strain at the location where damage has occurred. The direction and magnitude of the maximum principal strain that occurs at each element of the mesh of the concrete shear keys at a lateral load of 79 kN is shown in Figure IV-26. For lateral load values of 100 kN, 200 kN, 300 kN and 314 kN the direction and magnitude of the maximum principal strain is shown in Figure IV-27. The direction in which the propagation of cracks is expected to occur is also indicated in Figure IV-27.

When the lateral load is between 0 kN and 314 kN, the direction of the maximum principal plastic strain at the damaged points of concrete shear key 1 make an angle of 30° to 45° with the z-axis. During crack initiation in part 1.2 and 1.4 of shear key 1, the direction of crack initiation in the upper left corner of elements 1.2 and 1.4 as indicated in Figure IV-27. Similarly, when at a later stage cracking behavior occurs at shear key parts 4.1, 1.3, 2.2 and 2.4, the cracking initiation and later propagation is expected to also occur in the direction indicated at the respective locations in Figure IV-27.

### 4.4.4 Summary

Macroscopic cracking behavior of the concrete shear keys is expected to be present at a lateral load for which \( F_s/F_u > 0.25 \) at shear key 1. The expected locations, loads and crack directions at cracking initiation based on the results of FE simulation are summarized in Table IV-11. A significant reduction in overall joint stiffness is observed at lateral load values for which \( F_s/F_u > 0.8 \). Although the physical tests have to confirm the actual degradation behavior, the numerical results indicate that part 1.4 is likely to first encounter complete failure. It is underlined that the absolute values of the lateral load at which these damage phenomena occur have little significance. The relative load

![Figure IV-25: maximum Mises stress of rebars in all concrete shear key teeth](image-url)
$F_s/F_u$ is considered to be a more useful parameter to indicate the load at which damage is expected to occur. Also, the initiation of cracking depends on the load distribution of the lateral loads, which is discussed in the following paragraph.

Table IV-11: estimated location, load and direction of crack initiation of concrete shear keys

<table>
<thead>
<tr>
<th>location</th>
<th>absolute load [kN]</th>
<th>relative load $F_s/F_u [-]$</th>
<th>direction (relative to pos. x-axis) [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2</td>
<td>80</td>
<td>0.25</td>
<td>30-45 (↗)</td>
</tr>
<tr>
<td>1.4</td>
<td>100</td>
<td>0.32</td>
<td>30-45 (↗)</td>
</tr>
<tr>
<td>4.1</td>
<td>120</td>
<td>0.38</td>
<td>110-125 (↘)</td>
</tr>
<tr>
<td>1.3</td>
<td>120</td>
<td>0.38</td>
<td>30-45 (↗)</td>
</tr>
<tr>
<td>2.2</td>
<td>140</td>
<td>0.45</td>
<td>40-50 (↗)</td>
</tr>
<tr>
<td>2.4</td>
<td>140</td>
<td>0.45</td>
<td>40-50 (↗)</td>
</tr>
</tbody>
</table>

Figure IV-26: direction and magnitude of maximum principal strain [.] at $F_s = 79$ kN
Figure IV-27: direction and magnitude of maximum principal strain [-]
4.5 Load distribution

In the analysis of the structural behavior of the concrete shear keys, one of the issues of interest is the way the applied external forces are distributed as internal forces in the joint of the immersed tunnel. In both the physical and the numerical model of the Nanchang Red Valley tunnel, the applied lateral forces are expected to be carried primarily by the concrete shear keys, and to a less extent by the Gina gasket and by the steel shear keys. In the following paragraphs, the load distribution in the immersed joint is assessed.

4.5.1 Influence of steel shear keys

The structural members in the immersed joint that can transfer loads between the two tunnel elements are hatched in Figure IV-28. They include the concrete shear keys, the steel shear keys and the Gina gaskets. It was mentioned previously that the influence of the vertical steel shear keys on the stress distribution under lateral loads in the immersed joint is assumed to be negligible, and the steel shear keys are even omitted from the numerical calculations completely. In the physical scale model of the Nanchang Red Valley Tunnel, however, the steel shear keys are present. To assess whether the omission of the steel shear keys in the numerical model can be justified, the vertical displacement of the two tunnel elements relative to each other is discussed.

As long as the relative vertical displacement (i.e. the displacement in the y-direction) of the 2 tunnel elements in the numerical model without steel shear keys at the location of the omitted steel shear keys is small enough, the omission of the steel shear keys can be justified. In the case of small vertical displacements, the steel shear keys only just make contact and do not transfer any forces, because there is no friction between the outer steel plates. Also, because the vertical shear keys do not exert reaction forces on one another they cannot deform, and interlocking of deformed parts of the steel shear keys cannot contribute to the transfer of lateral forces between the two tunnel elements. This explains why the relative vertical displacement of the two tunnel elements in the numerical model without concrete shear keys is used to assess whether the omission of the steel shear keys in the numerical model is justified.

The results of the numerical model indicate that the displacements in the vertical (y-)direction of the model without steel shear key are at a maximum when the lateral load is at its maximum of 314 kN. The absolute displacements of tunnel elements 1 and 2 at this lateral load are shown in Figure IV-29. These displacements are measured relative to a fixed coordinate system outside the tunnel. The results depicted in this figure indicate that the relative displacement of the two tunnel elements is smaller than 0,06 mm at all the locations where steel shear keys are attached to the concrete elements. The numerical results also show that the rotation around the x-axis of the tunnel elements is negligibly small. This indicates that the steel shear keys will only just make contact and thus will not contribute to the overall load transfer that occurs in the joints.

It can be argued that the forces transferred between the steel shear keys in the numerical model do have significance in the sense that they represent a force transfer that is the result of the deformation of the concrete elements and of the other structural elements under the applied...
lateral loads, as will also be the case in the scale model tests. In this sense the loads that are taken up by the steel shear keys can be used as a prediction of the contribution the steel shear keys will make in the physical scale model test. Still it is decided not to take the steel shear keys into account in the numerical model, because the boundary conditions of the numerical model are idealized, and will be different in the actual physical modeling of the tunnel on the construction site in Nanchang. This means that the small reaction forces from the steel shear keys would not be significant predictions of the reaction forces in the physical scale model, and it is concluded that the omission of the steel shear keys is justified.

Figure IV-28: load transfer members in immersed joint

Figure IV-29: absolute displacement in y-direction [mm] of tunnel element 1 (a) and element 2 (b)

4.5.2 Concrete shear keys reaction forces

The loading case of combined axial and lateral load is considered, as indicated earlier in Figure IV-7. The loads that are taken up by the concrete shear keys and by the Gina gaskets are calculated separately, as well as the internal force distribution between and within the concrete shear keys.
The effect of the steel shear keys is neglected as explained in the above paragraph. The lateral loads are expected to be taken up primarily by the concrete shear keys and to less extent by the Gina gaskets. The results from the FE analysis will confirm this assumption.

When lateral load is applied on the side of tunnel element 1 (Figure IV-7), the vertical faces on the side of the teeth of the concrete shear keys make contact, and contact pressures perpendicular to these vertical faces occur. This results in a force transfer through the joint of the immersed tunnel. Figure IV-30 shows contact pressures on the side of concrete shear keys 1 and 2. For all four the concrete shear keys, the numerical results show a variation of the pressure over the contact area. This variation is not considered to correspond to the real pressure variation that will occur in the physical scale model test. In the physical scale model test, the teeth of the concrete shear keys are separated by rubber supports (Figure IV-31). These rubber supports, that are not present in the numerical model, will distribute the load that is transferred between the teeth, so that the exact distribution of the contact stresses in the numerical model will not correspond to the distribution in the physical scale model test.

![Figure IV-30: contact pressures on side of concrete shear keys [N/mm²]](image)

In Figure IV-31: rubber supports in physical scale model

In Figure IV-18 the convention with which the concrete shear key parts are numbered was shown. This figure is shown again below.
The reaction forces on the different teeth of the concrete shear keys are found by averaging out the contact pressure over the contact area between the concrete shear keys. A detail of the numerical result for the contact pressure on a side of concrete shear key part 1.2 is shown in Figure IV-32. This figure shows a detail of Figure IV-30 and is for an axial load \( F_f = 360 \text{ kN} \) and a lateral load of \( F_s = 100 \text{ kN} \). At this load, there is a certain area on the side of the concrete shear key teeth that is not in contact with the teeth of other shear keys, as can be seen in for example Figure IV-19 (page 56). The part of the side concrete shear key part 1.2 that is not in contact with the side of shear key part 4.1 is hatched in Figure IV-32. Theoretically, the contact pressure should be zero over this complete hatched area. Nevertheless the numerical results indicate a contact pressure that varies between 0 and 1.09 \( \text{N/mm}^2 \). This gives an indication of the inaccuracy of the numerical results for local contact pressures on a small scale. The contact forces \( F_c \) are evaluated by averaging out the contact pressures weighted by area, and multiplying the average contact pressure with the total contact area:

\[
F_c := \sum_{i=1}^{n} \sigma_i \cdot A_i
\]  

(41)

where \( i \) is the number of distinct contact pressure intervals inside the region that makes contact with another shear key.

The numerical result of contact pressures are always linked to a certain area and cannot be extracted from the post processed numerical result in the form of a graph or a table. The numerical output of the contact pressures takes a form as in Figure IV-32, and consists of areas in which the contact pressure is in a certain interval. Because only the boundary values of each interval are known and not the exact pressure at every point, \( \sigma_i \) is only the assumed average contact pressure over each area, taken as the middle of each interval. This might induce an additional error in the calculation of the contact pressures.
The calculation of the contact pressures is elaborated for concrete shear key 1. For the calculation of the contact force of shear key part 1.2, the first term in the right hand side of equation (41) is \( \sigma_1 = \frac{1}{2} (0.8696 + 1.087) = 0.9783 \text{ N/mm}^2 \) and \( A_1 = 333 \text{ mm}^2 \) (Figure IV-32). Doing a similar calculation for the other areas in Figure IV-32 and applying equation (41) yields a reaction force on the side of shear key part 1.1 of

\[
F'_{c\,1.2} = \sum_{i=1}^{8} \sigma_i \cdot A_i = 18,69 \text{ kN}.
\]

Similar calculations for concrete shear key 4.1 yield \( F'_{c\,4.1} = 18,11 \text{ kN} \). Because the forces \( F_{c\,1.2} \) and \( F_{c\,4.1} \) form a reaction pair, they have to be equal. The best estimate for both \( F_{c\,1.2} \) and \( F_{c\,4.1} \) is taken as the average value

\[
F_{c\,1.2} = F_{c\,4.1} = 18,40 \text{ kN}.
\]

This value and its physical sense for shear key part 1.2 is indicated in Figure IV-33. The values for \( F_{c\,1.2} \) and \( F_{c\,1.4} \) are also indicated in this figure and have been obtained by performing similar calculations as for \( F_{c\,1.2} \).

The above value for \( F_{c\,1.2} \) and \( F_{c\,4.1} \) is calculated by averaging out the values for both shear key parts. Both values of the reaction forces are calculated by using an approximate method, and the difference between both values is an indication of the error that is induced by using this method. To indicate this error, a point estimate of the coefficient of variation is used. This estimation is made by using a point estimate for the standard deviation and for the mean:

\[
\bar{C}_v := \frac{s}{\bar{x}} = \frac{\sqrt{\sum_{i=1}^{n} (x_i - \bar{x})^2}}{\bar{x}}
\]

where in this case \( n = 2, \bar{x} \) and \( s \) are the sample average and the sample variance, respectively. For \( F_{c\,1.2} \) and \( F_{c\,4.1} \) one finds

\[
\bar{C}_v = 0.022
\]

which is considered to be reasonably small. This measure is an indicator for the combined error in the contact pressures calculation in the numerical model, the error induced by averaging out the pressure intervals and the error in the calculation of the areas \( A_i \) in equation (41).
So for concrete shear key tooth 1.2, the best estimate for the reaction force on its left side is 18.40 kN acting in the negative x-direction. On its right side, the reaction force is zero because tunnel element 1 is only loaded in the positive x-direction and pressure on shear key 1 is only applied by shear key 4 in the positive x-direction. For the same reason the shear key parts 1.3 and 1.4 only have reaction forces in the negative x-direction. For shear key part 1.1 the reaction forces are all zero because it has a free edge on its left hand side in Figure IV-33. Similar calculations for the shear key parts 1.3 and 1.4, and for the concrete shear keys 2, 3 and 4 yield the results shown in Table IV-12 at an axial load of 360 kN and at a lateral load of 100 kN.

Addendum V shows the reaction forces on the loaded sides of concrete shear keys under lateral loads in the range 0.03F_u - 0.9F_u. The axial load on the tunnel elements is always 360 kN. The percentage of the total lateral load that is taken up by each shear key part and by the Gina is indicated in the tables in Addendum V. The coefficient of variation is calculated for each pair of concrete shear key teeth that make contact, and is shown for each shear key part. Figure IV-33 shows the calculated reaction forces on the teeth of concrete shear key 1 at an axial load of 100 kN. The point of application of the resultant force is drawn based on the pressure distributions on the sides of the shear key teeth.

It is noted that the teeth of opposite concrete shear keys form action-reaction pairs, so that the load acting on shear key part 1.2 equals the load acting on shear key part 4.1. The tables in Addendum V thus only show the reaction forces of shear keys 1 and 2. The reaction forces on the teeth of shear keys 3 and 4 are equal in magnitude and opposite in physical direction compared to the corresponding teeth of keys 1 and 2.
<table>
<thead>
<tr>
<th>Shear key part</th>
<th>load [kN]</th>
<th>percentage</th>
<th>c, [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>0,00</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>1.2</td>
<td>18,40</td>
<td>18,4%</td>
<td>0,022</td>
</tr>
<tr>
<td>1.3</td>
<td>18,09</td>
<td>18,1%</td>
<td>0,001</td>
</tr>
<tr>
<td>1.4</td>
<td>12,42</td>
<td>12,4%</td>
<td>0,012</td>
</tr>
<tr>
<td>2.1</td>
<td>0,00</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>2.2</td>
<td>16,00</td>
<td>16,0%</td>
<td>0,004</td>
</tr>
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<td>2.3</td>
<td>15,31</td>
<td>15,3%</td>
<td>0,008</td>
</tr>
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<td>2.4</td>
<td>13,89</td>
<td>13,9%</td>
<td>0,008</td>
</tr>
<tr>
<td>GINA</td>
<td>5,89</td>
<td>5,9%</td>
<td>-</td>
</tr>
<tr>
<td>SUM =</td>
<td>100,00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The reaction forces on concrete shear key 2 at a lateral load of 100 kN is shown in Figure IV-34.

![Figure IV-34: reaction forces on teeth of concrete shear key 2 (F_s = 100 kN)](image)

Figure IV-33 and Figure IV-34 show that the reaction forces in the numerical model do not act centrically on the loaded face of the concrete shear key teeth. This can be explained by the fact that, due to the eccentric application of the lateral load (Figure IV-7), tunnel element 1 rotates slightly relative to element 2. This causes the concrete shear key teeth of opposite shear keys also to rotate in a horizontal plane relative to each other, thus inducing eccentric forces on the loaded sides of the shear key teeth. Secondly, numerical inaccuracies due to the discretization of the contact behavior can also play a role.

The relative rotation of the tunnel elements is clearly visible in the numerical results. The absolute displacement in the z-direction of tunnel element 1 at a lateral load of 100 kN ($F_s/F_u=0,32$) is shown in Figure IV-35. At this lateral load the numerical results show that the relative displacement of the very left wall and very right wall of tunnel element 1 is 0,7 mm. At the same lateral load, the relative displacement of the outer walls of tunnel element 2 is approximately zero (order of magnitude $10^{-3}$ mm). This results in a rotation of tunnel element 1 relative to tunnel element 2 of $1,1 \times 10^{-4}$ rad.
The displacement fields of tunnel element 1 in the z-direction at lateral loads in the range 10 kN-314 kN look similar to Figure IV-35. The angular rotation around the y-axis of tunnel element 1 relative to tunnel element 2 for varying lateral loads is depicted in Figure IV-36. A clockwise rotation around the y-axis is defined as positive. The load regions that were defined in the load-displacement curve of the immersed joint (Figure IV-17) are also indicated in Figure IV-36.

The evolution of the angular rotation in Figure IV-36 shows that at small lateral loads the relative rotation of the 2 tunnel elements approximates zero, as is to be expected. At larger lateral loads up to the failure load, the relative rotation reaches values around $5 \times 10^{-4}$ rad, which corresponds to a differential compression of 3 mm of the Gina gaskets on the left and right walls of tunnel element 1.

Although for the reaction forces mentioned in Addendum V the coefficients of variance are reasonably small, the approximate character of the calculation of the reaction forces based on the numerical contact stresses is underlined. Nevertheless the results of the reaction force calculations are valuable in the sense that, firstly, they give an indication of the relative distribution of the reaction forces between the concrete shear keys and the Gina-gasket, and secondly that they can provide an estimate of the relative force distribution between the teeth of the concrete shear keys.
For the physical scale model tests this information can be for example used for the choice of force sensors on the shear key teeth. The extent to which the load distribution from the numerical calculations comply with the actual distribution in the physical scale model test will have to be confirmed by the physical scale model tests.

4.5.3 Load distribution between Gina gasket and concrete shear keys

At an axial load of 360 kN and a lateral load of 100 kN ($F_s/F_u = 0.32$), the concrete shear keys together account for about 94% of the transfer of lateral forces through the numerical model immersed joint (Table IV-12). The Gina gaskets account for about 6% of the lateral force transfer between tunnel element 1 and tunnel element 2 of the numerical model. This force transfer is through friction between the compressed Gina gaskets and tunnel element 1.

The fraction of the applied lateral load that is taken up by the Gina gaskets in function of the fraction of the ultimate lateral load $F_s/F_u$ is depicted graphically in Figure IV-37. At lateral loads up to 64% of the ultimate lateral load ($F_s < 0.64F_u = 200$ kN), the force-displacement curve of the numerical model is within the boundaries of linear region $\circ$, as can be seen in Figure IV-17 (page 53). In this region the lateral load that the Gina gaskets bear is within the range 4%-8% of the total lateral load. At higher loading, when the lateral load reaches values beyond 64% of the ultimate lateral load ($F_s > 0.64F_u = 250$ kN) and thus when the stiffness of the immersed joint degrades, the fraction of the lateral load that is taken up by the Gina gasket drops to a value as low as 3,5%. Nevertheless as the lateral load approaches the failure load, the fraction of lateral load that the shear keys carry increases again to about 8%.

![Figure IV-37: fraction of lateral load taken up by Gina gaskets](image)

The fraction of the lateral load that is taken up by the Gina gaskets shows no particular pattern. The only conclusion that is drawn from the data in Figure IV-37 is that the fraction of the applied load that is taken up by the Gina gaskets is an estimated 5,6% of the total applied lateral load, which is the average fraction of the seven data points depicted in Figure IV-37. The concrete shear keys thus account for 94,4% of the applied lateral load in the axial-lateral loading case when the lateral load ranges from 0,03 $F_u$ to 0,9 $F_u$. 

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4.5.4 Relation between load distribution and concrete shear key damage

The load distribution between the different concrete shear key teeth at varying lateral loads are shown in Addendum V. The relation between the load distribution in the tables in Addendum V and the cracking behavior that was described in paragraph IV4.4.1 is examined.

The results in Addendum V show that at a lateral load of 100 kN, shear key tooth 1.2 (and thus also tooth 4.1) is loaded most heavily, carrying 18.4% of the applied lateral load, followed by tooth 1.3 (and thus also 4.2). Shear key 1.4 only takes up 6.4% of the applied lateral load.

Referring to paragraph IV4.4.1, these concrete shear key teeth are also the ones at which visible damage is expected to occur first. Indeed, the values of the maximal principal plastic strain and of the tensile equivalent plastic strain in Addendum III and Addendum IV indicate the occurrence of macro cracks in the corner of shear key teeth 1.2 and 1.4, and to a secondary extent at tooth 1.3 (see also Figure IV-19 and Figure IV-20). Although teeth 1.2 and 1.3 are loaded more heavily than tooth 1.4, visible damage is expected to occur at teeth 1.3 and 1.4 first, and only in a later stage at tooth 1.2. This is attributed to the smaller cross-section and of tooth 1.2 compared to the adjacent shear key teeth.

The results in Addendum III and Addendum IV thus show that visible damage is expected to occur first at shear key teeth 1.3 and 1.4, but not at 4.1 and 4.2 although the reaction forces are the same. This is explained by the fact that the point of application of the reaction forces acting on the concrete shear key teeth 1.2 and 1.3 is located further away from the body of the concrete shear keys than is the case for teeth 4.1 and 4.2. This induces additional bending moments at the base of the concrete shear key teeth, larger tension forces and thus damage behavior earlier in the loading process.

4.5.5 Load distribution between the concrete shear key teeth

Table IV-13 summarizes the distribution of the applied lateral load $F_s$ over the concrete shear keys and the Gina gasket for different values of $F_s$. From this table it appears that while at lower lateral loads, shear key parts 1.2 and 1.3 are loaded most heavily, teeth 2.2 and 2.3 carry relatively more load at larger lateral force. The extent to which this is true will be verified. The values from Table IV-13 are depicted graphically in Figure IV-38.

<table>
<thead>
<tr>
<th>$F_s/F_u$</th>
<th>10</th>
<th>58</th>
<th>100</th>
<th>146</th>
<th>169</th>
<th>200</th>
<th>225</th>
<th>250</th>
<th>280</th>
</tr>
</thead>
<tbody>
<tr>
<td>shear key part 1.1</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>1.2</td>
<td>21.3%</td>
<td>18.9%</td>
<td>18.4%</td>
<td>16.6%</td>
<td>14.6%</td>
<td>13.5%</td>
<td>13.8%</td>
<td>14.3%</td>
<td>16.5%</td>
</tr>
<tr>
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<td>20.8%</td>
<td>18.2%</td>
<td>18.1%</td>
<td>17.8%</td>
<td>17.9%</td>
<td>17.6%</td>
<td>14.8%</td>
<td>15.0%</td>
<td>15.1%</td>
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<tr>
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<td>12.6%</td>
<td>12.4%</td>
<td>12.4%</td>
<td>12.5%</td>
<td>10.0%</td>
<td>12.1%</td>
<td>12.9%</td>
<td>10.0%</td>
</tr>
<tr>
<td>2.1</td>
<td>0%</td>
<td>0%</td>
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<td>0%</td>
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<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>2.2</td>
<td>13.2%</td>
<td>15.0%</td>
<td>16.0%</td>
<td>16.9%</td>
<td>17.1%</td>
<td>18.5%</td>
<td>18.7%</td>
<td>18.9%</td>
<td>17.6%</td>
</tr>
<tr>
<td>2.3</td>
<td>15.2%</td>
<td>14.4%</td>
<td>15.3%</td>
<td>17.0%</td>
<td>17.6%</td>
<td>18.7%</td>
<td>19.7%</td>
<td>18.8%</td>
<td>18.9%</td>
</tr>
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<td>2.4</td>
<td>11.4%</td>
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<td>13.9%</td>
<td>14.8%</td>
<td>15.2%</td>
<td>16.0%</td>
<td>17.1%</td>
<td>16.6%</td>
<td>13.8%</td>
</tr>
<tr>
<td>GINA</td>
<td>5.4%</td>
<td>7.8%</td>
<td>5.9%</td>
<td>4.5%</td>
<td>5.1%</td>
<td>5.7%</td>
<td>3.7%</td>
<td>3.5%</td>
<td>8.1%</td>
</tr>
</tbody>
</table>
Figure IV-38: fraction of $F_s$ taken up by concrete shear key parts for varying $F_s$

Figure IV-38 shows the fraction of the lateral load $F_s$ that is taken up by each concrete shear key tooth as a function of the applied lateral load. It appears that the fraction of the lateral load that is carried by the teeth of shear key 2 increases with the applied lateral load up to the point where $F_s = 0.72 F_u$. Only tooth 2.3 shows a deviation from the linear trend at a lateral load of 10 kN. This load range $F_s \leq 0.72F_u$ comprises the linear region $\text{○}_1$ and partly the transition region between part $\text{○}_1$ and part $\text{○}_2$ of the immersed joint's force-displacement curve. In the linear region $\text{○}_1$ the fraction of the lateral load that is carried by the tooth of shear key teeth 1.2 and 1.3 seems to decrease linearly. The behavior of tooth 1.4 seems to be approximately constant when $F_s \leq 0.54 F_u$, but varies strongly beyond that point.

Linear regression is performed on the data for the individual shear key teeth, and statistical tests on the individual coefficients indicate whether the load taken by the shear key teeth can indeed be concluded to be a linear function of the applied lateral load in the respective load regions that are mentioned in the above paragraph. The results of the linear regression analysis are shown in Table IV-14. This table shows the linear regression coefficients in linear force-displacement region $\text{○}_1$ and for shear key 2 also part of the transition region between region $\text{○}_1$ and region $\text{○}_2$. The significance of these coefficients was tested using t-tests. The significance level $\alpha$ at which both the intercept and the slope were found to be nonzero is included in Table IV-14. Also, 95% confidence intervals (CI) for the regression slope are included.

Table IV-14: linear regression analysis on shear key teeth reaction forces

<table>
<thead>
<tr>
<th>shear key tooth</th>
<th>loading range $F_s/F_u$</th>
<th>slope [-]</th>
<th>intercept [-]</th>
<th>slope 95% CI</th>
<th>regression significant at $\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.2</td>
<td>[0 ; 0.72]</td>
<td>0.0785</td>
<td>0.1324</td>
<td>[0.066 ; 0.090]</td>
<td>0.001</td>
</tr>
<tr>
<td>2.3</td>
<td>[0.18 ; 0.72]</td>
<td>0.1000</td>
<td>0.1236</td>
<td>[0.088 ; 0.112]</td>
<td>0.001</td>
</tr>
<tr>
<td>2.4</td>
<td>[0 ; 0.72]</td>
<td>0.0769</td>
<td>0.1131</td>
<td>[0.065 ; 0.089]</td>
<td>0.001</td>
</tr>
<tr>
<td>1.2</td>
<td>[0 ; 0.64]</td>
<td>-0.1244</td>
<td>0.2174</td>
<td>[-0.156 ; -0.093]</td>
<td>0.001</td>
</tr>
<tr>
<td>1.3</td>
<td>[0.18 ; 0.64]</td>
<td>-0.0137</td>
<td>0.1852</td>
<td>[-0.021 ; -0.006]</td>
<td>0.01</td>
</tr>
</tbody>
</table>
The results of this regression analysis show that based on the data shown in Figure IV-38 it can be concluded that the lateral load that is taken up by shear key teeth 2.2 and 2.4 increases linearly with the applied lateral load when $F_s/F_u$ is in the range $[0 ; 0.72]$. The same conclusion holds for shear key tooth 2.3 for $F_s/F_u$ in the range $[0.18 ; 0.72]$. Both the slope and the intercept are significant at a significance level of $\alpha = 0.001$. The regression for the data on shear key 2 is shown graphically in Figure IV-39. The regression equations are included in his graph. In these equations, $y$ is defined as the fraction of lateral load that is taken up by each shear key tooth separately. The coefficients of determination $R^2$ are found to be close to 1.

![Figure IV-39: tooth of shear key 2 linear regression](image)

Table IV-14 indicates a clear decreasing linear trend for the fraction of lateral load in function of the applied load for shear key teeth 1.2 when the loads are in the linear region ①. For tooth 1.3 the fraction of the lateral load that is carried by this shear key tooth is found to be only slightly decreasing when $F_s/F_u$ is in the range $[0.18 ; 0.64]$. The data for tooth 1.2, 1.3 and 1.4 are depicted in Figure IV-40 together with the linear regression of the shear key teeth in the regions in which they were found to be significant.
Concerning the load distribution between the concrete shear key teeth, it is concluded that the lateral load carried by concrete shear key tooth 2.2, 2.3 and 2.4 increases linearly with the applied lateral load $F_s$ up to the point where $F_s/F_u = 0.72$. Only shear key 2.3 shows a deviation from the linear trend at a lateral load of 10 kN, in an early stage of the lateral loading.

The fraction of the total applied lateral load that is carried by shear key tooth 1.2 is found to decrease linearly with the applied lateral load in the linear branch $\circ$ of the overall load-displacement curve of the immersed joint. Teeth 1.3 and 1.4 only show a mild decreasing trend over the largest part of the linear region $\circ$ of the overall load-displacement curve of the immersed joint (Figure IV-40).

### 4.5.6 Load distribution between the concrete shear keys

The same procedure is used to examine the reaction forces on the complete concrete shear keys. The numerical results for the total reaction forces on all teeth of concrete shear key 1 and separately of concrete shear key 2 are depicted in Figure IV-41.

For concrete shear key 2 it was found in the previous paragraph that the fraction of the lateral load that is carried by all of its teeth increases linearly with the applied lateral load up to the point where $F_s = 0.72 F_u$, with the exception of 1 point at an early stage in the lateral loading. The same linear behavior is expected to occur for the load carried by concrete shear key 2 as a whole. Figure IV-41 indeed shows a clear linear trend up to $F_s = 0.72 F_u$. This is confirmed by the results of a statistical analysis in Table IV-15. The statistical tests are similar to the ones in the previous paragraph and the regression coefficients of shear key 2 are significant with a significance level of 0.001.
For concrete shear key 1, a clear linear trend in the range $F_s/F_u \in [0; 0.72]$ is also confirmed by the results in Table IV-15, this time at a significance level of $0.001$ which is considered to be sufficiently low. Again, this load range $F_s/F_u \in [0; 0.72]$ comprises the linear region ① and the transition region between part ① and part ② of the immersed joint's force-displacement curve.

The fraction of lateral load carried by the Gina gasket was already found to be approximately constant throughout the analysis, and is also included in Figure IV-41.

### Table IV-15: linear regression analysis on shear key reaction forces

<table>
<thead>
<tr>
<th>shear key</th>
<th>loading range $F_s/F_u$</th>
<th>regression slope [-]</th>
<th>regression intercept [-]</th>
<th>slope 95% CI</th>
<th>regression significant at $\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$[0; 0.72]$</td>
<td>-0.1988</td>
<td>0.5498</td>
<td>[-0.235; -0.162]</td>
<td>0.001</td>
</tr>
<tr>
<td>2</td>
<td>$[0; 0.72]$</td>
<td>0.2294</td>
<td>0.3833</td>
<td>[0.202; 0.257]</td>
<td>0.001</td>
</tr>
</tbody>
</table>

The fit for both shear keys in the range in which they were found to be linear is shown in Figure IV-42. The regression formulae and $R^2$ values are included in this figure. A trend line for the Gina gasket indeed shows that the load transferred by the Gina gaskets is indeed approximately constant.
A regression model is constructed for concrete shear key 1 and for concrete shear key 2 in the load range $F_s/F_u \in [0; 0.89]$ (or $F_s \in [0\text{ kN}; 280\text{ kN}]$). The results are shown in Figure IV-43. In this model the behavior of the shear keys is assumed to be bilinear and the load that the Gina gasket carries is assumed to be constant.

The regression model for concrete shear key 1 is

$$y = \begin{cases} 
-0.20\left(\frac{F_s}{F_u}\right) + 0.55; & F_s/F_u < 0.72 \\
0.08\left(\frac{F_s}{F_u}\right) + 0.35; & F_s/F_u \geq 0.72 
\end{cases}$$  \hspace{1cm} (43)$$

and for shear key 2

$$y = \begin{cases} 
0.23\left(\frac{F_s}{F_u}\right) + 0.38; & F_s/F_u < 0.72 \\
-0.18\left(\frac{F_s}{F_u}\right) + 0.68; & F_s/F_u \geq 0.72
\end{cases}$$ \hspace{1cm} (44)$$

In these equations $y$ is defined as the fraction of lateral load that is taken up by each shear key. The parts of the equation where $F_s/F_u \geq 0.72$ are based on 3 data points only and are scaled in order to comply with the value of the first linear equation at $F_s/F_u = 0.72$. This has as a result that the second branch of both bilinear equations are statistically less significant.
4.5.7 Consequences for physical scale model pressure gauges

In order to measure the reaction forces on the physical scale model, force sensors are used. The results from the previous paragraphs are useful to evaluate which type of sensor is deemed suitable.

It was mentioned in paragraph IV4.5.2 that the exact pressure distribution on the sides of the concrete shear keys of the numerical model will not correspond to the real values in the physical scale model test, but can be used to estimate the distribution of reaction forces on the teeth of the concrete shear keys. The distribution of reaction forces on the concrete shear keys at varying lateral loads are shown in Addendum V.

Because the contact area between the sides of the concrete shear keys in the numerical model is known (see for example Figure IV-32), an estimate of the average contact pressures on the side of the concrete shear keys can be calculated. The calculated average contact pressures on the loaded sides on the concrete shear key teeth in MPa for varying lateral loads are shown in Table IV-16.
Table IV-16: calculated average contact pressures on concrete shear key teeth [N/mm²]

<table>
<thead>
<tr>
<th>$F_s$ [kN]</th>
<th>100</th>
<th>146</th>
<th>169</th>
<th>200</th>
<th>225</th>
<th>250</th>
<th>280</th>
</tr>
</thead>
<tbody>
<tr>
<td>shear key part 1.1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1.2</td>
<td>1,60</td>
<td>2,11</td>
<td>2,15</td>
<td>2,35</td>
<td>2,70</td>
<td>3,10</td>
<td>4,01</td>
</tr>
<tr>
<td>1.3</td>
<td>1,57</td>
<td>2,26</td>
<td>2,63</td>
<td>3,06</td>
<td>2,90</td>
<td>3,26</td>
<td>3,68</td>
</tr>
<tr>
<td>1.4</td>
<td>1,08</td>
<td>1,58</td>
<td>1,84</td>
<td>1,75</td>
<td>2,36</td>
<td>2,80</td>
<td>2,43</td>
</tr>
<tr>
<td>2.1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2.2</td>
<td>1,39</td>
<td>2,14</td>
<td>2,52</td>
<td>3,22</td>
<td>3,67</td>
<td>4,12</td>
<td>4,28</td>
</tr>
<tr>
<td>2.3</td>
<td>1,33</td>
<td>2,16</td>
<td>2,59</td>
<td>3,25</td>
<td>3,85</td>
<td>4,09</td>
<td>4,60</td>
</tr>
<tr>
<td>2.4</td>
<td>1,21</td>
<td>1,87</td>
<td>2,23</td>
<td>2,79</td>
<td>3,35</td>
<td>3,60</td>
<td>3,37</td>
</tr>
</tbody>
</table>

The force sensors will be placed between the concrete shear keys and the rubber supports (Figure IV-31), and thus have to be sufficiently thin. Additionally the sensors have to be able to measure the loads accurately at all loading stages.

As Figure IV-33 and Figure IV-34 show, the contact pressures are not uniform over the side area of the concrete shear keys. This means that the average values in Table IV-16 are not conservative enough to make a decision on which force sensors can be used. Instead of a uniform distribution, a triangular stress distribution over the contact area between the concrete shear keys is assumed. A theoretical triangular stress distribution on the teeth of concrete shear key 2 is shown in Figure IV-44. This stress distribution is in accordance with the numerical results for the contact stresses for shear key 2, shown in Figure IV-34. Nevertheless it is underlined that the triangular load distribution is an extreme situation that is used to estimate the maximum pressure that could occur locally.

![Figure IV-44: theoretical triangular load distribution on shear key 2](image)

The use of a triangular distribution is justified by the fact that at the contact surface between two concrete shear key teeth, the contact pressure can become zero, but cannot become negative (no tension forces can be transmitted). Although the exact pressure distribution that will occur in the physical scale model test is unknown, assuming zero pressure at one point on each contact surface is assumed to be a fairly conservative approach. This corresponds to the point where the concrete shear key teeth just start to detach at one point. The numerical results also indicate that the contact area between the concrete shear key teeth does not diminish during the loading steps, so that besides the variation of stresses, a reduction in contact area is not taken into account in the estimation of the maximum contact stress.
The maximum occurring contact stresses at the shear key teeth for the case of a triangular stress distribution are found by doubling the values in Table IV-16 and are shown in Table IV-17.

<table>
<thead>
<tr>
<th>shear key part</th>
<th>F_s [kN]:</th>
<th>100</th>
<th>146</th>
<th>169</th>
<th>200</th>
<th>225</th>
<th>250</th>
<th>280</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1.2</td>
<td>3,20</td>
<td>4,22</td>
<td>4,30</td>
<td>4,70</td>
<td>5,40</td>
<td>6,20</td>
<td>8,02</td>
<td></td>
</tr>
<tr>
<td>1.3</td>
<td>3,15</td>
<td>4,53</td>
<td>5,26</td>
<td>6,11</td>
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<td>6,51</td>
<td>7,37</td>
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</tr>
<tr>
<td>1.4</td>
<td>2,16</td>
<td>3,15</td>
<td>3,67</td>
<td>4,39</td>
<td>4,73</td>
<td>5,60</td>
<td>4,86</td>
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<tr>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
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<tr>
<td>2.2</td>
<td>2,78</td>
<td>4,28</td>
<td>5,03</td>
<td>6,43</td>
<td>7,33</td>
<td>8,23</td>
<td>8,56</td>
<td></td>
</tr>
<tr>
<td>2.3</td>
<td>2,66</td>
<td>4,31</td>
<td>5,18</td>
<td>6,50</td>
<td>7,70</td>
<td>8,18</td>
<td>9,21</td>
<td></td>
</tr>
<tr>
<td>2.4</td>
<td>2,42</td>
<td>3,75</td>
<td>4,45</td>
<td>5,57</td>
<td>6,70</td>
<td>7,20</td>
<td>6,73</td>
<td></td>
</tr>
</tbody>
</table>

This means that in the lateral loading the maximal occurring pressure is expected to be $2 \cdot 4,6 \approx 9,2$ MPa.

During the physical scale model test, Standard FlexiForce® A201-100 sensors will be used. These sensors are relatively thin (0,20 mm) and can measure forces up to 445 N. The contact area of the circular sensor is 71,3 mm², so that the maximal pressure that can be measured by the sensors is 6,2 MPa. Nevertheless the supplier's user manual mentions that forces up to 4450 N (62 MPa) can be measured by adjusting the drive voltage and the resistance the electronic measuring circuit [25][26].

If the sensors are placed on the location where the contact pressure is maximal, and the parameters of the electronic measuring circuit are not altered during the tests, the A201-100 sensors are expected only to yield accurate results up to the point where the contact pressure reaches 6,2 MPa. This is estimated to happen at a lateral load $F_s$ of just over 190 kN (Table IV-16 and Figure IV-39).

It is concluded that the sensor is expected to be appropriate to measure the contact stresses between the concrete shear keys and the rubber supports in the linear region $\odot$ of the immersed joint's force-displacement curve, up to an estimated lateral load of 190 kN. In order to obtain accurate results of the contact stresses at lateral loads beyond 190 kN, the sensors must not be placed at locations where the contact pressure is expected to be maximal. In that case it is suggested to place the sensors close to the shear key body for shear keys 1 and 2. An alternative is to alter the drive voltage and the resistance the electronic measuring circuit during the physical scale model test.
4.5.8 Discussion of FE model load distribution

No literature was found on the internal distribution of lateral load over the concrete shear keys and over the concrete shear key teeth, and the physical scale model tests will have to confirm to which extent the findings from the FE model are correct. Nevertheless some remarks can already be made concerning the predicted load distribution based on the distribution of lateral loads over the individual shear key teeth (Figure IV-38, p. 73) and on the distribution of lateral loads over the shear keys themselves (Figure IV-43, p. 78). Both Figure IV-38 and Figure IV-43 are reiterated below for the sake of reading comfort.

A possible source of errors that can occur at all loading stages is the fact that contact between the shear key teeth is through discrete finite elements. As described in paragraph IV3.2, the real contact behavior was modeled by using the penalty method, that relies on an inherent stiffness that is automatically attributed to individual elements. The discretization of a continuous contact phenomenon to a discrete situation with the penalty method might induce errors in the results for the contact pressures, and thus also in the results for the load distributions of the shear keys, the shear key teeth and the Gina gaskets. The calculation of reaction forces by calculating the average contact pressure for every shear key tooth can also contribute to inaccuracies.

Figure IV-38 showed the fraction of $F_s$ that is taken up by the concrete shear key teeth for $F_s$ in the linear region $\circ$ and the stiffness reduction region $\circ$. Remarks are made concerning 3 regions:

- $F_s/F_u < 0.18$
- $F_s/F_u \in [0.18; 0.64]$
- $F_s/F_u > 0.72$

When $F_s/F_u < 0.18$ (this is a part of region $\circ$) the fraction of the lateral load that each loaded shear key tooth carries varies around 15.5%, and is in the range 11% - 22%. Although the physical scale model tests will have to confirm the actual behavior, it can be expected intuitively that at small
lateral loads, when no degradation of the stiffness has occurred and when there is no relative rotation of the tunnel elements (Figure IV-36), each of the 6 loaded shear key teeth carries approximately 1/6th of the lateral loads. The numerical results in Figure IV-38 clearly show that this is not the case.

These possible inaccuracies might be ascribed to inherent flaws of element discretization and the use of the penalty method. Also, in the region $F_s/F_u < 0.18$ the results diverge from the linear behavior that was observed in the rest of region ①. Whether the calculations of the contact pressures are less accurate at smaller lateral loads and thus at smaller pressures is nevertheless unclear. Another important influencing factor is the cross-sectional shape of the loaded concrete shear key tooth 1.4. This shear key tooth has a different cross-section than the other five loaded shear key teeth, and has a smaller lateral stiffness. Furthermore the absolute amount of reinforcement in tooth 1.4 is smaller than the other teeth of concrete shear keys 1 and 2. It is expected that during loading the stiffest elements take up most loads. This can explain why concrete shear key tooth 1.4 is loaded least heavily at almost all loading stages in Figure IV-38, and thus partly explain the imbalance between the different shear key teeth individually.

In the region where $F_s/F_u \in [0.18 ; 0.64]$ (still region ①) the variance of the load fraction on the individual shear key teeth has diminished compared to the region where $F_s/F_u < 0.18$, and the load fraction for each tooth varies around 15.8%. Additional to the element discretization, a possible factor that plays a role in the non-uniform distribution of the lateral loads is the fact that tunnel element 1 rotates relative to tunnel element 2 (as shown by Figure IV-36). A relative rotation causes a movement in the positive x-direction of the teeth of shear key 4 relative to the teeth of shear key 1. Similarly the rotation causes a movement in the negative x-direction of the teeth of shear key 3 relative to the teeth of shear key 2. This relative displacement is larger for the outer shear key teeth than for the inner ones and thus can cause differential contact pressures when it is constrained. This effect is nevertheless not directly visible in the results in Figure IV-38.

In the region where $F_s/F_u > 0.72$ (region ②) a different behavior can be observed for all shear key teeth compared to region ①. The teeth of shear key 2, who carry the largest fraction of the lateral load in region ①, take up a decreasing fraction of the lateral load in region ②. This might be attributed to the stiffness degradation of the concrete shear keys that occurs in this region (the stiffness of the shear key teeth that carry the largest part of the load degrades first, so that the other teeth take up more load). The rotation behavior of the tunnel elements and thus also of the shear keys might also play a role, although the extent to which this holds is difficult to assess.

Figure IV-43 shows the fraction of $F_s$ that is taken up by the concrete shear key 1 and concrete shear key 2 for $F_s$ in the linear region ① and the stiffness reduction region ②. Again, similar remarks can be made concerning 3 regions:

- $F_s/F_u < 0.18$
- $F_s/F_u \in [0.18 ; 0.64]$
- $F_s/F_u > 0.72$
In the region where $F_s/F_u < 0.18$ the load distribution over the 2 shear keys again is not equal, and this while at small lateral loads the relative rotation of the tunnel elements is found to be very small (Figure IV-36). Again these possible inaccuracies might be ascribed to inherent flaws of the discretization of the contact between the surfaces of the concrete shear key teeth.

When $F_s/F_u \in [0.18 ; 0.64]$, the fraction of the load on both shear key teeth varies around 47%. Again in this region this value approximates the load fraction that each shear key would carry if they each would carry the same load: $(100\%-5.6\%)/2 = 47.4\%$. Besides the discretization inaccuracies, again the rotation of the tunnel elements and shear keys relative to each other might be an influencing factor for the observed trend in the fraction of the load that each shear key bears.

When $F_s/F_u > 0.72$ again inverse behavior is observed compared to region ①. Additional to the discretization of contact behavior, both the stiffness degradation and the relative rotation of the tunnel elements can play a role in the observed behavior at this loading stage. The clear change of behavior at the point where the stiffness degrades endorses the argument of load redistribution due to stiffness degradation.

4.5.9 Proposal of simplified model for damage assessment

Based on the FE results concerning the load distributions between the concrete shear key teeth, a more simplified FE model can be proposed to assess the damage behavior of the concrete shear key teeth.

A proposal for a simplified FE model for the assessment of damage to the concrete shear key teeth is by considering only tunnel element 2 and its shear keys, shear key 1 and shear key 2. The model does not include shear keys 3 and 4 nor the Gina gaskets. Figure IV-45 shows the geometry of the simplified numerical model. The load is now applied directly onto the sides of the concrete shear keys as uniformly distributed pressures.
The Gina gaskets were found earlier to carry an estimated 5,6% of the total lateral load. Figure IV-37 indeed shows that in this load range \( F_s/F_u \) in the range 0,20 to 0,5 the load fraction of the Gina is approximately equal to this value.

Based on the results for the loads on the individual shear key teeth in Figure IV-38, a possible loading scheme for the fraction of the lateral load that is applied on each shear key tooth is depicted in Figure IV-46. This load distribution is expected to yield most conform results for the damage behavior of the shear key teeth in the load range were \( 0,20 < F_s/F_u < 0,50 \) because in this loading range they correspond best to the values from Figure IV-38. In the Figure IV-46 the fractions for all the shear key teeth together add up to 94,4%.

The equivalent absolute load and the equivalent relative load at which the alternative FE model shows that cracking initiates is summarized in Table IV-18. The same criteria for cracking initiation...
were used as in paragraph IV4.4. In Table IV-18 the equivalent lateral load is defined as the sum of the loads that are applied directly on the concrete shear key teeth, multiplied by 0.944⁻¹ to account for the additional load that the Gina gaskets would carry.

<table>
<thead>
<tr>
<th>Shear key tooth</th>
<th>equivalent absolute load [kN]</th>
<th>equivalent relative load $F_s/F_u [-]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4</td>
<td>65</td>
<td>21%</td>
</tr>
<tr>
<td>1.2</td>
<td>70</td>
<td>22%</td>
</tr>
<tr>
<td>1.3</td>
<td>70</td>
<td>22%</td>
</tr>
<tr>
<td>2.2</td>
<td>75</td>
<td>24%</td>
</tr>
<tr>
<td>2.4</td>
<td>75</td>
<td>24%</td>
</tr>
<tr>
<td>2.3</td>
<td>80</td>
<td>25%</td>
</tr>
</tbody>
</table>

The results in Table IV-18 indicate the occurrence of cracking at lower loads than was predicted in paragraph IV4.4. This is mainly ascribed to the fact that the simplified model does not take into account the interaction between the concrete shear keys of opposite tunnel elements. In the simplified model concrete shear keys 1 and 2 can deform freely and thus show cracking behavior at smaller equivalent lateral loads. This is considered to be a drawback of the simplified numerical model.

Although tooth 1.4 is loaded with a smaller fraction of the total load, cracking is expected to occur first at this tooth due to its smaller lateral stiffness. Tooth 1.2 and 1.3 are loaded more heavily than the teeth of shear key 2 and thus also show damage first. The evolution of the concrete tensile damage parameter $d_t$ with the equivalent lateral load is depicted graphically in Figure IV-47. This figure shows that cracking occurs more uniformly compared to the results that were found with the initial, more elaborated numerical model.

The simplified model shows significantly reduced computation time and less convergence problems due to the omission of contact behavior between the concrete shear key teeth. Nevertheless it does not take into account the observed relative rotation of the tunnel elements and redistribution of loads due to stiffness degradation, nor other physical interaction between the shear key teeth on opposite tunnel elements.
4.5.10 Summary

The reaction forces on the loaded sides of the concrete shear key tooth are calculated for lateral loads up to 90% of the lateral load at failure. The results are included in Addendum V.

The fraction of the lateral load that is transferred through the immersed joint by the Gina gaskets is estimated to be 5.6%. The fraction of the lateral load that is transferred through the concrete shear keys is thus estimated to be 94.4%.

The distribution of the loads between the concrete shear key teeth was found to vary throughout the analysis. At small lateral loads, tooth 1.2 and tooth 1.3 of shear key 1 are loaded most heavily. A linear trend is found for the relative load that is carried by each tooth individually for concrete shear key 1, in most of the region where the lateral load is in part ① of the immersed joint's force-displacement curve (Figure IV-40). Similarly a linear trend is found for the relative load that is carried by each individual tooth of concrete shear key 2 in almost the complete region where the lateral load is in part ① of the immersed joint's force-displacement curve (Figure IV-39).
For the concrete shear keys as entities, it is found that while at smaller lateral loads reaction forces are largest at concrete shear key 1, the relative amount of load that concrete shear key 1 carries decreases linearly with the applied lateral load when it is in the linear region ① and when it is in the transition zone between regions ① and ② of the load-displacement curve. In region ② on the other hand, the relative amount of load that concrete shear key 1 carries is concluded to increase linearly with the applied lateral load. For concrete shear key 2 it is found that the opposite holds: a linear increase up to the point where the lateral load reaches region ② of the load-displacement curve, and a linear decrease beyond this point (Figure IV-43). Bilinear regression equations were constructed for both shear keys (equations (43) and (44)). Remarks were made concerning the outcome of the numerical results, especially at small lateral loads, and the physical scale model tests can be used to verify to which extent the findings from the FE model are correct. Based on the results for the internal load distribution in the immersed joint, a reduced but strongly simplified FE model of the immersed joint was proposed with which the damage behavior of the immersed joint can be assessed to a limited extent.
The main objective of this dissertation was to predict the structural behavior of the concrete shear keys that will occur in a 1:5 physical scale model of the Nanchang Red Valley immersed tunnel, using a FE model of an immersed joint.

Several material models for the different constituents of the immersed joint were investigated and the used material models were elaborated. For plain concrete, the damaged plasticity model according to Lubliner [15] and Lee [17] was used. Steel reinforcement was defined in the FE model by using the embedded regions method, for which the steel material behavior was assumed plastic with isotropic hardening. For the rubber of the Gina gaskets, a Mooney-Rivlin material model was used. For the modeling of concrete, viscoplastic regularization was necessary to overcome convergence problems when the stiffness of the concrete has degraded significantly.

Dimensional similitude between the FE model and the tunnel prototype was obtained by using the scaling parameters $S_x = S_z = 1/5$ and $S_E = 1$. The reinforcement ratio of the FE model was nevertheless lowered compared to the tunnel prototype to comply with the design of the physical scale model test. The (scaled) axial load acting on the FE model's immersed joint was also altered to comply with the design of the physical scale model test.

The degradation of the stiffness of the immersed joint was investigated. Linear force-displacement behavior, incremental stiffness degradation and complete loss of stiffness occurred in 3 distinct regions in the load-displacement curve of the immersed joint.

Cracking behavior was assessed using the damaged plasticity model for concrete. Visible cracking of the concrete shear keys is expected to occur in the physical scale model at a lateral load of above 25% of the ultimate lateral load. The damage that occurred in the FE model was noticed to be in accordance with the degradation of the joint stiffness.

The reaction forces on the loaded side of the concrete shear key teeth were calculated for lateral loads up to 90% of the lateral load at failure. The distribution of the applied lateral load between the Gina gaskets and the concrete shear keys was studied. The fraction of the applied lateral load
that is transferred through the immersed joint by the Gina gaskets was concluded to be 5.6\% on average, with limited fluctuations.

Concerning the fraction of the applied lateral load that is taken up by concrete shear keys 1 and 2, bilinear models were proposed for both concrete shear keys. On the level of individual shear key teeth it was found that at lower lateral loads shear key teeth 1.2 and 1.3 are loaded most heavily, and that this shifts to shear key teeth 2.2 and 2.3 at larger lateral loading. The relation between the applied lateral load and the fraction of load that is taken up by individual shear key teeth showed to be linear over large but distinct ranges of the applied lateral load. The load distribution between and within the concrete shear keys showed to be in accordance with the FE model’s damage behavior and the stiffness degradation of the immersed joint. The load distribution that was obtained from the numerical model was used to make preliminary predictions for the load sensors for the physical scale model test on the construction site in Nanchang. Some important remarks were made concerning the accuracy of the outcome of the numerical results, especially at small lateral loads. Based on the results for the internal load distribution in the immersed joint, a reduced but strongly simplified FE model of the immersed joint was proposed with which the damage behavior of the immersed joint can be assessed to a limited extent.

Some recommendations can be made concerning the calibration of future FE models

- Modification of the mesh of the concrete shear keys could further enhance the numerical results
- Contact methods other than the penalty method can be investigated to simulate contact behavior between the concrete shear keys
- Adding the rubber supports between shear key teeth to the model could better represent the real contact conditions
- Modifications to the viscoplastic regularization parameter $\mu$ can have an effect on the accuracy of the FE results
- The uniaxial damage parameters $d_t$ and $d_c$ for concrete can be calculated using different methods, and can be calibrated further with results from physical tests.
VI References


VII Addenda
Addendum I  Estimated loads on bulkhead of Red Valley tunnel prototype

The information below was provided by the firm Trelleborg.
Addendum II  
**Uniaxial tensile material parameters of C40 concrete**

The uniaxial material parameters $\varepsilon^{\text{el}}$ and $\varepsilon^{\text{pl}}$ as a function of the inelastic stresses $|\sigma_c|$ for C40 concrete in tension that are mentioned in paragraph IV3.1.1 are calculated below based on the Chinese Design Code for Concrete Structures GB 50010:2010 [18].

Appendix C of the Chinese Design Code for Concrete Structures defines the stress-strain curve of concrete in uniaxial tension as

$$\eta = \begin{cases} 
1.2 \xi - 0.2 \xi^6 ; & \xi \leq 1 \\
\frac{\xi}{\alpha_t (\xi - 1)^{1.7} + \xi} ; & \xi > 1 
\end{cases} \quad (45)$$

where

$$\eta = \frac{\sigma}{f_{tr}} \quad (46)$$

$$\xi = \frac{\varepsilon_t}{\varepsilon_{tr}} \quad (47)$$

and with

- $f_{tr}$: representative value of the concrete tensile strength, here taken as $f_{tk}$;
- $\alpha_t$: dimensionless parameter for the descending branch of the stress-strain curve, taken from table C.2.3 from GB 50010:2010 (Figure VII-1);
- $\varepsilon_{tr}$: peak strain value in the stress-strain curve, to be interpolated in table C.2.3 with respect to $f_{tr}$.

<table>
<thead>
<tr>
<th>$f_{tr}$ ($\text{N/mm}^2$)</th>
<th>1.0</th>
<th>1.5</th>
<th>2.0</th>
<th>2.5</th>
<th>3.0</th>
<th>3.5</th>
<th>4.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_{tr}$ ($10^{-6}$)</td>
<td>65</td>
<td>81</td>
<td>95</td>
<td>107</td>
<td>118</td>
<td>128</td>
<td>137</td>
</tr>
<tr>
<td>$a_t$</td>
<td>0.31</td>
<td>0.70</td>
<td>1.25</td>
<td>1.95</td>
<td>2.81</td>
<td>3.82</td>
<td>5.00</td>
</tr>
</tbody>
</table>

**Figure VII-1**: dimensionless material constants for concrete tensile constitutive relation [18]

For C40 concrete, according to the Chinese Design Code

$$f_{tr} = f_{tk} = 2.39 \text{ N/mm}^2$$

$$E_c = 3.25 \times 10^4 \text{ N/mm}^2$$

$$v = 0.2.$$  \hspace{1cm} (48)

With linear interpolation of the values in table C.2.3 from GB 50010 (Figure VII-1) this yields
With the parameters (48)-(49) the stress-strain curve (45) of concrete C40 can be constructed. This curve is shown graphically in Figure VII-2.

![Graph](image)

**Figure VII-2: uniaxial stress-strain relation of C40 concrete under tension**

Equation C.2.3-2 of the Chinese Design Code for Concrete structures defines the tensile damage parameter \( d_t \) as

\[
d_t = \begin{cases} 
1 - p_t(1,2 - 0,2\xi^5) & ; \xi \leq 1 \\
1 - \frac{\rho_t}{\alpha_t(\xi - 1)^{1,7} + \xi} & ; \xi > 1 
\end{cases}
\]  

(50)

with

\[
\rho_t = \frac{f_{tr}}{E_c \varepsilon_{tr}} = 0,705
\]  

(51)

From Figure IV-8 it can be seen that based on the uniaxial stress-strain relation, the uniaxial tensile plastic strain \( \varepsilon_{t,p} \) can be found as

\[
\varepsilon_{t,p} = \varepsilon_t - \frac{\sigma_t}{(1 - d_t)E_o}.
\]  

(52)
Once $\varepsilon_t^{pl}$ is known through equation (52), the cracking strain $\varepsilon_t^{ck}$ can be found by using equation (14) from paragraph IV3.1.1:

$$\varepsilon_t^{ck} = \varepsilon_t^{pl} + \frac{d_t \sigma_t}{(1 - d_t)E_0}$$

or, alternatively by reasoning on the stress-strain relation in Figure IV-8:

$$\varepsilon_t^{ck} = \varepsilon_t - \frac{\sigma_t}{E_0}$$

By using the stresses and strains from the constitutive relation (45) together with equation (50)-(53) the tensile stress in the inelastic region in function of the cracking strain $\varepsilon_t^{ck}$ is found. The inelastic tensile stress $\sigma_t$ and the cracking strain $\varepsilon_t^{ck}$ in function of a number of tensile strain values $\varepsilon_t$ is mentioned in Table VII-1. These are the values that are mentioned in Table IV-4 in paragraph IV3.1.1.

Table VII-1: uniaxial elastic strains in function of inelastic stresses

<table>
<thead>
<tr>
<th>$\varepsilon_t$ [-]</th>
<th>$\sigma_t$ [N/m²]</th>
<th>$\varepsilon_t^{ck}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,04E-04</td>
<td>2,39</td>
<td>0</td>
</tr>
<tr>
<td>3,20E-04</td>
<td>0,79</td>
<td>2,96E-04</td>
</tr>
<tr>
<td>5,60E-04</td>
<td>0,47</td>
<td>5,46E-04</td>
</tr>
<tr>
<td>1,40E-03</td>
<td>0,22</td>
<td>1,39E-03</td>
</tr>
<tr>
<td>2,00E-03</td>
<td>0,17</td>
<td>1,99E-03</td>
</tr>
<tr>
<td>5,00E-03</td>
<td>0,09</td>
<td>5,00E-03</td>
</tr>
</tbody>
</table>
Addendum III  Evolution of maximal principal plastic strain in concrete shear keys numerical model at $F_i = 360$ kN
Addendum IV     Evolution of tensile equivalent plastic strain $\varepsilon_{pl}^e$ in concrete shear keys numerical model at $F_t = 360$ kN.
Addendum V  

Reaction forces on concrete shear keys and Gina

**Table VII-2: reaction forces on concrete shear keys and Gina under lateral load of 10 kN**

<table>
<thead>
<tr>
<th>Shear key part</th>
<th>load [kN]</th>
<th>percentage</th>
<th>$c_v$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>0</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>1.2</td>
<td>2,13</td>
<td>21,3%</td>
<td>0,035</td>
</tr>
<tr>
<td>1.3</td>
<td>2,08</td>
<td>20,8%</td>
<td>0,000</td>
</tr>
<tr>
<td>1.4</td>
<td>1,29</td>
<td>12,9%</td>
<td>0,004</td>
</tr>
<tr>
<td>2.1</td>
<td>0</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>2.2</td>
<td>1,32</td>
<td>13,2%</td>
<td>0,004</td>
</tr>
<tr>
<td>2.3</td>
<td>1,52</td>
<td>15,2%</td>
<td>0,157</td>
</tr>
<tr>
<td>2.4</td>
<td>1,14</td>
<td>11,4%</td>
<td>0,017</td>
</tr>
<tr>
<td>GINA</td>
<td>0,54</td>
<td>5,4%</td>
<td>-</td>
</tr>
<tr>
<td>SUM =</td>
<td>10,00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table VII-3: reaction forces on concrete shear keys and Gina under lateral load of 58 kN**

<table>
<thead>
<tr>
<th>Shear key part</th>
<th>load [kN]</th>
<th>percentage</th>
<th>$c_v$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>0</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>1.2</td>
<td>10,98</td>
<td>18,9%</td>
<td>0,052</td>
</tr>
<tr>
<td>1.3</td>
<td>10,58</td>
<td>18,2%</td>
<td>0,001</td>
</tr>
<tr>
<td>1.4</td>
<td>7,30</td>
<td>12,6%</td>
<td>0,004</td>
</tr>
<tr>
<td>2.1</td>
<td>0</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>2.2</td>
<td>8,69</td>
<td>15,0%</td>
<td>0,001</td>
</tr>
<tr>
<td>2.3</td>
<td>8,37</td>
<td>14,4%</td>
<td>0,002</td>
</tr>
<tr>
<td>2.4</td>
<td>7,58</td>
<td>13,1%</td>
<td>0,004</td>
</tr>
<tr>
<td>GINA</td>
<td>4,50</td>
<td>7,8%</td>
<td>-</td>
</tr>
<tr>
<td>SUM =</td>
<td>58,00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table VII-4: reaction forces on concrete shear keys and Gina under lateral load of 100 kN**

<table>
<thead>
<tr>
<th>Shear key part</th>
<th>load [kN]</th>
<th>percentage</th>
<th>$c_v$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>0</td>
<td>0,0%</td>
<td>0</td>
</tr>
<tr>
<td>1.2</td>
<td>18,40</td>
<td>18,4%</td>
<td>0,022</td>
</tr>
<tr>
<td>1.3</td>
<td>18,09</td>
<td>18,1%</td>
<td>0,001</td>
</tr>
<tr>
<td>1.4</td>
<td>12,42</td>
<td>12,4%</td>
<td>0,012</td>
</tr>
<tr>
<td>2.1</td>
<td>0</td>
<td>0,0%</td>
<td>0</td>
</tr>
<tr>
<td>2.2</td>
<td>16,00</td>
<td>16,0%</td>
<td>0,004</td>
</tr>
<tr>
<td>2.3</td>
<td>15,31</td>
<td>15,3%</td>
<td>0,008</td>
</tr>
<tr>
<td>2.4</td>
<td>13,89</td>
<td>13,9%</td>
<td>0,008</td>
</tr>
<tr>
<td>GINA</td>
<td>5,89</td>
<td>5,9%</td>
<td>-</td>
</tr>
<tr>
<td>SUM =</td>
<td>100</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table VII-5: reaction forces on concrete shear keys and Gina under lateral load of 146 kN

<table>
<thead>
<tr>
<th>Shear key part</th>
<th>load [kN]</th>
<th>percentage</th>
<th>$c_v$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>0,00</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>1.2</td>
<td>24,29</td>
<td>16,6%</td>
<td>0,002</td>
</tr>
<tr>
<td>1.3</td>
<td>26,03</td>
<td>17,8%</td>
<td>0,001</td>
</tr>
<tr>
<td>1.4</td>
<td>18,13</td>
<td>12,4%</td>
<td>0,006</td>
</tr>
<tr>
<td>2.1</td>
<td>0,00</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>2.2</td>
<td>24,61</td>
<td>16,9%</td>
<td>0,002</td>
</tr>
<tr>
<td>2.3</td>
<td>24,80</td>
<td>17,0%</td>
<td>0,010</td>
</tr>
<tr>
<td>2.4</td>
<td>21,55</td>
<td>14,8%</td>
<td>0,000</td>
</tr>
<tr>
<td>Gina</td>
<td>6,60</td>
<td>4,5%</td>
<td>-</td>
</tr>
<tr>
<td>SUM</td>
<td>146,00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table VII-6: reaction forces on concrete shear keys and Gina under lateral load of 169 kN

<table>
<thead>
<tr>
<th>Shear key part</th>
<th>load [kN]</th>
<th>percentage</th>
<th>$c_v$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>0</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>1.2</td>
<td>24,73</td>
<td>14,6%</td>
<td>0,048</td>
</tr>
<tr>
<td>1.3</td>
<td>30,24</td>
<td>17,9%</td>
<td>0,006</td>
</tr>
<tr>
<td>1.4</td>
<td>21,12</td>
<td>12,5%</td>
<td>0,007</td>
</tr>
<tr>
<td>2.1</td>
<td>0</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>2.2</td>
<td>28,94</td>
<td>17,1%</td>
<td>0,002</td>
</tr>
<tr>
<td>2.3</td>
<td>29,80</td>
<td>17,6%</td>
<td>0,000</td>
</tr>
<tr>
<td>2.4</td>
<td>25,61</td>
<td>15,2%</td>
<td>0,004</td>
</tr>
<tr>
<td>Gina</td>
<td>8,55</td>
<td>5,1%</td>
<td>-</td>
</tr>
<tr>
<td>SUM</td>
<td>169,00</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table VII-7: reaction forces on concrete shear keys and Gina under lateral load of 200 kN

<table>
<thead>
<tr>
<th>Shear key part</th>
<th>load [kN]</th>
<th>percentage</th>
<th>$c_v$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>0,00</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>1.2</td>
<td>27,02</td>
<td>13,5%</td>
<td>0,008</td>
</tr>
<tr>
<td>1.3</td>
<td>35,15</td>
<td>17,6%</td>
<td>0,001</td>
</tr>
<tr>
<td>1.4</td>
<td>20,09</td>
<td>10,0%</td>
<td>0,054</td>
</tr>
<tr>
<td>2.1</td>
<td>0,00</td>
<td>0,0%</td>
<td>-</td>
</tr>
<tr>
<td>2.2</td>
<td>36,98</td>
<td>18,5%</td>
<td>0,002</td>
</tr>
<tr>
<td>2.3</td>
<td>37,38</td>
<td>18,7%</td>
<td>0,000</td>
</tr>
<tr>
<td>2.4</td>
<td>32,06</td>
<td>16,0%</td>
<td>0,000</td>
</tr>
<tr>
<td>Gina</td>
<td>11,32</td>
<td>5,7%</td>
<td>-</td>
</tr>
<tr>
<td>SUM</td>
<td>200,00</td>
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</tr>
</tbody>
</table>
Table VII-8: reaction forces on concrete shear keys and Gina under lateral load of 225 kN

<table>
<thead>
<tr>
<th>Shear key part</th>
<th>load [kN]</th>
<th>percentage</th>
<th>c_v [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>0</td>
<td>0.0%</td>
<td>-</td>
</tr>
<tr>
<td>1.2</td>
<td>31.04</td>
<td>13.8%</td>
<td>0.015</td>
</tr>
<tr>
<td>1.3</td>
<td>33.38</td>
<td>14.8%</td>
<td>0.001</td>
</tr>
<tr>
<td>1.4</td>
<td>27.18</td>
<td>12.1%</td>
<td>0.011</td>
</tr>
<tr>
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<td>0</td>
<td>0.0%</td>
<td>-</td>
</tr>
<tr>
<td>2.2</td>
<td>42.17</td>
<td>18.7%</td>
<td>0.001</td>
</tr>
<tr>
<td>2.3</td>
<td>44.30</td>
<td>19.7%</td>
<td>0.000</td>
</tr>
<tr>
<td>2.4</td>
<td>38.55</td>
<td>17.1%</td>
<td>0.001</td>
</tr>
<tr>
<td>Gina</td>
<td>8.38</td>
<td>3.7%</td>
<td>-</td>
</tr>
<tr>
<td><strong>SUM =</strong></td>
<td><strong>225,00</strong></td>
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</tr>
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</table>

Table VII-9: reaction forces on concrete shear keys and Gina under lateral load of 250 kN

<table>
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<tr>
<th>Shear key part</th>
<th>load [kN]</th>
<th>percentage</th>
<th>c_v [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>0</td>
<td>0.0%</td>
<td>-</td>
</tr>
<tr>
<td>1.2</td>
<td>35.68</td>
<td>14.3%</td>
<td>0.008</td>
</tr>
<tr>
<td>1.3</td>
<td>37.46</td>
<td>15.0%</td>
<td>0.004</td>
</tr>
<tr>
<td>1.4</td>
<td>32.22</td>
<td>12.9%</td>
<td>0.007</td>
</tr>
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<td>0</td>
<td>0.0%</td>
<td>-</td>
</tr>
<tr>
<td>2.2</td>
<td>47.34</td>
<td>18.9%</td>
<td>0.010</td>
</tr>
<tr>
<td>2.3</td>
<td>47.05</td>
<td>18.8%</td>
<td>0.016</td>
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<tr>
<td>2.4</td>
<td>41.42</td>
<td>16.6%</td>
<td>0.001</td>
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<tr>
<td>Gina</td>
<td>8.83</td>
<td>3.5%</td>
<td>-</td>
</tr>
<tr>
<td><strong>SUM =</strong></td>
<td><strong>250,00</strong></td>
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</tbody>
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Table VII-10: reaction forces on concrete shear keys and Gina under lateral load of 280 kN

<table>
<thead>
<tr>
<th>Shear key part</th>
<th>load [kN]</th>
<th>percentage</th>
<th>c_v [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>0</td>
<td>0.0%</td>
<td>-</td>
</tr>
<tr>
<td>1.2</td>
<td>46.13</td>
<td>16.5%</td>
<td>0.005</td>
</tr>
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<td>1.3</td>
<td>42.36</td>
<td>15.1%</td>
<td>0.027</td>
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<td>1.4</td>
<td>27.94</td>
<td>10.0%</td>
<td>0.134</td>
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<tr>
<td>2.1</td>
<td>0</td>
<td>0.0%</td>
<td>-</td>
</tr>
<tr>
<td>2.2</td>
<td>49.21</td>
<td>17.6%</td>
<td>0.045</td>
</tr>
<tr>
<td>2.3</td>
<td>52.95</td>
<td>18.9%</td>
<td>0.024</td>
</tr>
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<td>2.4</td>
<td>38.72</td>
<td>13.8%</td>
<td>0.232</td>
</tr>
<tr>
<td>Gina</td>
<td>22.70</td>
<td>8.1%</td>
<td>-</td>
</tr>
<tr>
<td><strong>SUM =</strong></td>
<td><strong>280,00</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>