# EXPERIMENTAL DETERMINATION OF BEARING CAPACITY TRANSVERSELY PRESTRESSED CONCRETE DECK SLABS 


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## A thesis submitted to the Department of Civil Engineering in conformity with the requirements for the degree of Master of Science

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## Preface

This research project on determining the bearing capacity of transversely prestressed concrete deck slabs is carried out as a Master's thesis project. The thesis is part of the study Civil Engineering with specialization Structural Engineering - Concrete Structures at Delft University of Technology.

The success of this thesis would not have been possible without the contributions and guidance of the thesis committee. To this regard I would like to express my sincere gratitude to my supervisor Dr. ir. C. van der Veen, whose valuable guidance, inspiring discussions and great patience during the research helped to complete the project.

Furthermore I would like to thank Ir. C. Quartel who arranged a visit to the factory of Spanbeton where the girders of the scale model were produced. It was very instructive to see the procedure of making prefabricated girders in practice.
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## Summary

In the Netherlands the traffic has grown rapidly over the years. The codes which were used to design the structures did not take into account the high values of today's traffic. Therefore Rijkswaterstaat started doing research on all structures built before 1975 .

One of the investigated structures is the Van Brienenoord bridge. The bridge deck consists of prestressed girders with a transversely prestressed concrete slab between them. During the review of the old structures, it was found that according to the current codes the Van Brienenoord bridge does not meet the requirements for structural safety. Although the bridge is loaded beyond its calculated capacity, it still is in a good condition. Explanation for this is the occurring phenomenon of compressive membrane action.

Some countries have incorporated compressive membrane action into their codes for the design of structures by an empirical method. However, these codes do not take into account the presence of transverse prestressing in the slabs. The issue with the Van Brienenoord bridge is the relative slenderness of the concrete slab, the span to depth ratio is high. The foreign design codes set a limit on the slenderness of a slab in order to make use of the code. Because this requirement is not met, the codes assume that the occurring amount of compressive membrane action is too little to take into account as beneficial for the bearing capacity. However, the bearing capacity is increased by the presence of transverse prestressing. So the exact bearing capacity of the slender transversely prestressed slab remains unknown.

This project includes a design of scale model which represents the properties of the Van Brienenoord bridge, where the effect of the transverse prestressing on the amount of compressive membrane action can be investigated. Also the bearing capacity of the slabs are predicted via analytical methods and finite element analysis.

Two analytical methods are used to calculate the bearing capacity of the slabs. Although these methods have not been validated with a lot of tests, the results are promising. Especially in comparison with Eurocode 2, the methods give results which are 7 to 9 times higher.

During the finite element analysis it is demonstrated that the transverse prestressing level influences the bearing capacity of the slabs. The occur-
rence of a compression arch is clearly visible when the strains are examined. Also the behaviour of the slabs changes when the level is adapted. The first moment of cracking delays when the prestressing level increases.

The properties of the interface between the girders and the slabs are very important for both the stiffness of the 2D model as well as the failure load. Because the capacity of the interface can change the failure mechanism, it is very important to monitor the occurring failure mechanism during the experiment.

A very important parameter of the experiment is the skewness of the interface. When the forces of the skew interface are decomposed, an extra vertical force loads the slabs. The extra loading results in a lower capacity of the interface. The investigation of this parameter via finite element analysis demonstrates that the capacity of the skew interface is considerably lower than the capacity of a straight interface.

When the failure loads during the experiment reach the values of the prediction in this thesis, the Van Brienenoord bridge will meet the requirements of structural safety. This is demonstrated by scaling back to the dimensions of the bridge. Add to this the increased concrete quality due to ongoing hydration and then the capacity increases even more. That would mean that strengthening the bridge is not necessary yet.

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## Chapter 1

## Introduction

Rijkswaterstaat is the executive arm of the Dutch Ministry of Infrastructure and the Environment. On behalf of the Minister and State Secretary, Rijkswaterstaat is responsible for the design, construction, management and maintenance of the main infrastructure facilities in the Netherlands. [9] An important part of the infrastructure facilities are the structures. In the Netherlands a lot of these structures are built in the sixties and seventies of the twentieth century. Over the years the traffic has grown rapidly, both the number of vehicles and their weight. This results in higher peak stresses and an increase of the average stress. The codes which were used to design the structures did not take into account the high values of today's traffic. Therefore Rijkswaterstaat started doing research on all structures built before 1975.

One of the investigated structures is the Van Brienenoord bridge. The Van Brienenoord bridge connects the city of Rotterdam with the southern part of the Netherlands by crossing the river the Nieuwe Maas. With twelve lanes and over 250.000 vehicles a day, the bridge is part of the busiest road of the Netherlands. The bridge deck consists of prestressed girders with a transversely prestressed concrete slab between them. During the review of the old structures, it was found that according to the current codes the Van Brienenoord bridge does not meet the requirements for structural safety. Although the bridge is loaded beyond its calculated capacity, it still is in a good condition. Explanation for this is the occurring phenomenon of compressive membrane action.

Several researchers have investigated the phenomenon of compressive membrane action and its positive effect on the bearing capacity of reinforced concrete slabs is demonstrated. Some countries have incorporated compressive membrane action into their codes for the design of structures by an empirical method. However, these codes do not take into account the presence of transverse prestressing in the slabs. Only some experiments have been carried out to investigate the effect of compressive membrane
action with transverse prestressing. From these experiments it can be concluded that transverse prestressing increases the effect and therefore giving the slabs a higher bearing capacity.

The issue with the Van Brienenoord bridge is the relative slenderness of the concrete slab, the span to depth ratio is high. The foreign design codes set a limit on the slenderness of a slab in order to make use of the code. Because this requirement is not met, the codes assume that the occurring amount of compressive membrane action is too little to take into account as beneficial for the bearing capacity. However, the bearing capacity is increased by the presence of transverse prestressing. So due to the limitations of the design codes, the exact bearing capacity of the slender transversely prestressed slab remains unknown. And therefore it is not clear whether the Van Brienenoord bridge needs to be strengthened or not.

### 1.1 Objective

The goal of this project is to design a scale model which represents the properties of the Van Brienenoord bridge, where the effect of the transverse prestressing on the amount of compressive membrane action can be investigated. The scale model has to be scaled down with a factor 1:2. Former experiments have shown that in reinforced slabs the phenomenon of compressive membrane action occurs. In theory the presence of prestressing should enlarge the effect of compressive membrane action, but what is effect on the slender slab of the Van Brienenoord bridge? This project will determine the bearing capacity of the slabs of the scale model with analytical methods and investigate the behaviour of the slabs by finite element analysis. Special attention will be paid to the properties of the concrete to concrete interface between the top flange of the girder and the slabs.

### 1.2 Outline of the thesis

The thesis consists of six chapters. Chapter $\mathbf{1}$ is an introduction to the investigation and states the objective. Chapter 2 gives an overview of the important components of the Van Brienenoord bridge. In chapter 3 the design of the scale model is fully elaborated, all the important considerations are described. In chapter 4 the bearing capacity of the concrete slabs are determined analytically, resulting in failure loads for the experiment. Also the loads are scaled back to the dimensions of the Van Brienenoord bridge. Chapter 5 describes the finite element analysis of the scale model. And in chapter 6 the conclusions and recommendations are given.

## Chapter 2

## Van Brienenoord bridge

The scale model which is going to be tested during the experiment is based on the dimensions of the Van Brienenoord bridge. Therefore a clear overview is necessary of the different parts of the bridge deck, so the properties of the scale model can be determined properly.

### 2.1 General information

The Van Brienenoord bridge connects the city of Rotterdam with the southern part of the Netherlands by crossing the river the Nieuwe Maas. With twelve lanes and over 250.000 vehicles a day, the bridge is part of the busiest road of the Netherlands. Figure 2.1 shows an aerial view of the bridge and its surroundings.

Plans to cross the river the Nieuwe Maas go back to the early thirties. However, only after the war these plans could actually be elaborated. In 1961 the construction started by clearing the site for building. Around 1962, the city council determined the name of the bridge. The Van Brienenoord bridge owes its name to the underlying Brienenoord Island, the place of A.W. Baron van Brienen. The bridge is completely constructed at the site. To build the arch bridge, two temporary support pillars were placed in the river. Construction was finished in 1965 and opened for traffic festively by Queen Juliana on February 1st 1965. Figure 2.2 shows the bridge just a few years after the opening.

Even in the seventies, short after the bridge was opened, it became clear that the bridge would become too small to handle the rapidly growing traffic. Already then, the possibilities to increase the number of lanes on the bridge were investigated. First it was thought to use the existing bicycle lanes as traffic lanes, but soon it became clear that this would not result in sufficient capacity. A solution with major planological impact was necessary, widening the bridge or building a tunnel next to the bridge. The work group "Van Brienenoord-corridor" was created to investigate the planological im-


Figure 2.1: Aerial view of the Van Brienenoord bridge
pact. For a number of reasons the choice was finally made in favour of the construction of a second bridge next to the existing one and the simultaneous expansion of the roads to and from the bridge. This proved to be the solution with the least drastic effects for the surroundings and one which was both financially and aesthetically sound. In 1985 a start was made with the final design and construction started in 1987. The date for opening the bridge to traffic is May 1990. The new arched bridge has not been built on the spot, as was the old one. It was assembled by Grootint in Zwijndrecht. After completion, the entire arch bridge was brought by boat to its ultimate destination. Figure 2.3 shows the transport of the arch bridge.

The rapidity of the growth of traffic is best shown in the next figures: in the year just after the opening in 1965, only 30000 vehicles passed the bridge per day. In 1975 the number grew to 90000 per day and after completing of the second bridge to 145000 per day. When the second bridge and the connecting roads were finished, the Van Brienenoord bridge consisted of four tracks with each three traffic lanes, so a total of twelve traffic lanes. Each bridge has a track for trough traffic separated from the track for local traffic, which have to exit immediately after crossing the bridge. Consequently there are no conflicts between the two types of traffic, resulting in better
road safety and better traffic flow. The capacity of the Van Brienenoord bridge at the onset of congestion was determined to be 180000 vehicles per day.


Figure 2.2: Van Brienenoord bridge in the seventies


Figure 2.3: Transportation of the arch bridge in February 1989

### 2.2 Construction of the deck

A closer look to the Van Brienenoord bridge shows that it consists of several bridges. From south to north nine approach spans of 50 meters, the arch bridge of 300 meters, the bascule bridge, the bascule pit and another nine approach spans of 50 meters. The total length of the bridges is 1320 meters. Figure 2.4 shows an overview of the Van Brienenoord bridge.


Figure 2.4: Overview of the Van Brienenoord bridge

In this thesis the concrete approach spans of 50 meters are of interest. The bridge deck is built up from simply supported longitudinal prestressed girders with a span of 50 meters. To connect the girders a concrete slab is cast in between them. The slab can be relative slender due to the presence of transverse prestressing. Figure 2.5 shows the deck during construction in $1962[9$. The holes for the transverse prestressing cables are clearly visible in the girders. The reinforcement ratio of the slab can be low due to the presence of transverse prestressing. At the supports a transversely prestressed end cross beam restrains the crosswise translation and rotation of the girders. Also two diaphragms are placed at one third and two third of the span, providing a higher stiffness in transverse direction of the bridge and a more beneficial force distribution.


Figure 2.5: Construction of the deck in 1962

### 2.3 Materials

The materials used in the Van Brienenoord bridge all have an old coding with respect to the applied strength class. Therefore in table 2.1 the old strength class is converted to an equivalent strength class which is specified in the current design codes. The strength class of the prestressing steel in the Van Brienenoord bridge is unknown, only the working force known. 40 tons wires are applied.

|  | Original strength class | Equivalent strength class |
| :--- | :--- | :--- |
| Concrete girder | B45 | $\mathrm{C} 35 / 45$ |
| Concrete slab | B 35 | $\mathrm{C} 28 / 35$ |
| Reinforcing steel | QR24 | FeB240 |

Table 2.1: Strength class of the Van Brienenoord bridge

Due to the fact that the Van Brienenoord bridge is built in the early sixties, the concrete strength is much higher than the original design strength. Therefore an additional material research is done by Witteveen+Bos [11]to determine the current concrete compressive and tensile strength. The research consisted of drilling out cores and testing the samples for strength in the laboratory. Table 2.2 shows the results of the material research. In this research project, the effect of increased concrete quality is not investigated.

|  | Value | Unit |
| :--- | :--- | :--- |
| Compressive strength | 84.6 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Splitting tensile strength | 4.3 | $\mathrm{~N} / \mathrm{mm}^{2}$ |

Table 2.2: Strength of cores tested in laboratory

### 2.4 Section properties

### 2.4.1 Girders

The bridge deck consists of nine girders. Figure 2.6 shows the cross section of a girder with the dimensions, where sub-figure 2.6a shows a part of the original hand made drawing.


Figure 2.6: Drawing cross-section of the girder

In order to design an accurate scale model of the Van Brienenoord bridge, the section properties of the girders have to be known. These are listed in table 2.3 .

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Concrete area | $A_{c}$ | 1127000 | $\mathrm{~mm}^{2}$ |
| Neutral axis to top fibre | $z_{c, \text { top }}$ | 1265 | mm |
| Neutral axis to bottom fibre | $z_{c, \text { bottom }}$ | 1735 | mm |
| Moment of inertia | $I_{c}$ | $1.3197 \cdot 10^{12}$ | $\mathrm{~mm}^{4}$ |
| Section modulus top | $W_{c, \text { top }}$ | $1.0432 \cdot 10^{9}$ | $\mathrm{~mm}^{3}$ |
| Section modulus bottom | $W_{c, \text { bottom }}$ | $0.7606 \cdot 10^{9}$ | $\mathrm{~mm}^{3}$ |
| Weight of the girder | $q_{G}$ | 2.8175 | $\mathrm{ton} / \mathrm{m}$ |

Table 2.3: Section properties girder Van Brienenoord bridge

### 2.4.2 Slabs

The design of the bridge deck positions the concrete slabs between the girders. To create a stable connection between the parts, transverse prestressing is applied. This is the only continuous connecting part, because no continuing reinforcing steel is applied from the girder to the slabs. The bearing capacity must be ensured by the transverse prestressing steel and a concrete to concrete interface. The dimensions of the slabs are depicted in figure 2.7 in cm . The height of the slab is 200 mm and the width is 2100 mm .


Figure 2.7: Dimensions of the slabs in cm
The amount of transverse prestressing steel is varying over the length of the girders. Drawing C10107 shows the side view of half a girder. Present are 38 prestressing cables divided over a length of 24670 mm . The average center to center distance is:

$$
\text { c.t.c.average }=\frac{24670}{38}=649 \mathrm{~mm}
$$

The maximum center to center distance is:

$$
\text { c.t.c. } \max =800 \mathrm{~mm}
$$

In the section of materials the prestressing steel is described. The amount of transverse prestressing steel is:

$$
A_{p}=\frac{400 \cdot 10^{3}}{1262 \cdot 649}=0.4884 \mathrm{~mm}^{2} / \mathrm{mm}
$$

Figure 2.7 shows a slight inclination in the slope of the slab. In combination with a skew edge of the girders, the interface is not a connection between two vertical planes. The skewness of the interface needs to be added to the scale model, because it can influence the bearing capacity.

### 2.4.3 End cross beams

Drawing C10367 shows details of the ends of the girders. Figure 2.8 shows a part of the drawing which depicts one end cross beam. The dimensions are in cm , so the width is 700 mm .


Figure 2.8: Dimensions of an end cross beam in cm

## Chapter 3

## Design of the experiment

### 3.1 Introduction

To verify the theory of compressive membrane action in transversely prestressed concrete slabs, an experiment will be carried out. The experiment focuses on the bearing capacity of the slab. In order to investigate the behaviour of compressive membrane action, the model has to be loaded until failure. Therefore also the occurring failure mechanism can be investigated. The experiment set-up is a part of the Van Brienenoord bridge which is scaled down. The dimensions of the scale model will be determined in this chapter.

### 3.2 Scale model

### 3.2.1 Materials

The materials of the Van Brienenoord bridge are discussed in the previous chapter. Due to the ongoing hydration of the concrete, the current strength is higher than the strength class used for the design. Therefore higher concrete classes must be used in the scale model to represent the current quality of the materials. The concrete strength class of the slabs is C45/55. The strength of the girders is higher, due to the better conditions in which they are casted. This represents the situation of the Van Brienenoord bridge where the girders also have a higher concrete strength class. The concrete strength class of the girders is C53/65. This quality is not described in Eurcode 2, however the properties are calculated according to Eurocode 2.

$$
\begin{align*}
& f_{c m}=f_{c k}+8[M P a]  \tag{3.1}\\
& f_{c t m}=2.12 \cdot \ln \left(1+\left(\frac{f_{c m}}{10}\right)\right) \tag{3.2}
\end{align*}
$$

$$
\begin{equation*}
E_{c m}=22 \cdot\left(\frac{f_{c m}}{10}\right)^{0.3} \tag{3.3}
\end{equation*}
$$

The properties of the concrete are specified in tables 3.1 and 3.2 .

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Characteristic compressive cylinder strength | $f_{c k}$ | 45 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Characteristic compressive cube strength | $f_{c k, \text { cube }}$ | 55 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Mean value of cylinder compressive strength | $f_{c m}$ | 53 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Mean value of axial tensile strength | $f_{c t m}$ | 3.8 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Secant modulus of elasticity | $E_{c m}$ | 36000 | $\mathrm{~N} / \mathrm{mm}^{2}$ |

Table 3.1: Properties of concrete C45/55

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Characteristic compressive cylinder strength | $f_{c k}$ | 53 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Characteristic compressive cube strength | $f_{c k, \text { cube }}$ | 65 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Mean value of cylinder compressive strength | $f_{c m}$ | 61 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Mean value of axial tensile strength | $f_{c t m}$ | 4.16 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Secant modulus of elasticity | $E_{c m}$ | 37846 | $\mathrm{~N} / \mathrm{mm}^{2}$ |

Table 3.2: Properties of concrete C53/65
The steel in the scale model will consist of reinforcing steel and prestressing steel. For the calculation of the amount of stirrups the properties of the reinforcing steel are necessary. The quality of the reinforcing steel is B500B. For the prestressing two types of steel are used, strands and bars. Strands are used to prestress the girders and bars are used to prestress the slabs and end cross beams. Tables $3.3,3.4$ and 3.5 specify the properties of the steel qualities used.

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Characteristic yield strength | $f_{y k}$ | 500 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Design value of modulus of elasticity | $E_{s}$ | 200000 | $\mathrm{~N} / \mathrm{mm}^{2}$ |

Table 3.3: Properties of reinforcing steel B500B

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Characteristic tensile strength | $f_{p k}$ | 1860 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Characteristic 0,1\% proof-stress | $f_{p 0.1 k}$ | 1640 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Design value of modulus of elasticity | $E_{p}$ | 195000 | $\mathrm{~N} / \mathrm{mm}^{2}$ |

Table 3.4: Properties of prestressing steel Y1860S

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Characteristic tensile strength | $f_{p k}$ | 1100 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Characteristic 0,1\% proof-stress | $f_{p 0.1 k}$ | 900 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Design value of modulus of elasticity | $E_{p}$ | 205000 | $\mathrm{~N} / \mathrm{mm}^{2}$ |

Table 3.5: Properties of prestressing steel Y1100H

### 3.2.2 Scale factors

Due to the massive dimensions of the Van Brienenoord bridge, the experiment is carried out on a scale model. Making use of a scale model demands exact determination of the scaled dimensions. Therefore scale factors are introduced. Savides [12] has investigated the different scale factors for the required parameters. Table 3.6 shows the expressions of the scale factors. In the expressions the $p$ is an abbreviation for prototype, which means in this experiment the parameter of the Van Brienenoord bridge. The $m$ in the expressions represents the parameter of the scale model.

| Scale factor | Expression |
| :--- | :--- |
| Length $(L)$ | $\lambda_{L}=L_{p} / L_{m}=x$ |
| Stress $(\sigma)$ | $\lambda_{\sigma}=\sigma_{p} / \sigma_{m}=1$ |
| Force $(F)$ | $\lambda_{F}=F_{p} / F_{m}=x^{2}$ |
| Strain $(\epsilon)$ | $\lambda_{\epsilon}=\epsilon_{p} / \epsilon_{m}=1$ |
| Moment $(M)$ | $\lambda_{M}=M_{p} / M_{m}=x^{3}$ |
| Area $(A)$ | $\lambda_{A}=A_{p} / A_{m}=x^{2}$ |
| Section modulus $(S)$ | $\lambda_{S}=S_{p} / S_{m}=x^{3}$ |
| Moment of inertia $(I)$ | $\lambda_{I}=I_{p} / I_{m}=x^{4}$ |
| Mass density $(\rho)$ | $\lambda_{\rho}=\rho_{p} / \rho_{m}=1 / x^{2}$ |

Table 3.6: Scale factors

Some comments have to be made to the scaling factors. The factors are determined based on geometry and are linear. However, when investigating punching shear failure non-linear terms are present like the perimeter. Also it is unknown to what degree the scale effects influence the compressive membrane action. He [6] describes the scale factors as conservative, so they are used for designing the scale model.

In the experiment the model will be scaled down 1:2. The experiment focuses on the bearing capacity of the slab, so the length and height are scaled down exactly with a factor $x=2$. Making use of the table of Savides, all required parameters can be calculated.

### 3.2.3 Deck slabs

The main objective of the experiment is finding the bearing capacity of the cast in-situ transversely prestressed deck slabs. Therefore it is most important to represent the slab of the Van Brienenoord bridge properly. By using the scale factors of Savides the dimensions of the height and width can be determinded.

$$
\begin{align*}
& h_{m}=\frac{h_{p}}{x}=\frac{200}{2}=100 \mathrm{~mm}  \tag{3.4}\\
& w_{m}=\frac{w_{p}}{x}=\frac{2100}{2}=1050 \mathrm{~mm} \tag{3.5}
\end{align*}
$$

## Prestressing system

The deck slabs need to be transversely prestressed with varying levels of prestressing, $1.25 \mathrm{~N} / \mathrm{mm}^{2}$ and $2.50 \mathrm{~N} / \mathrm{mm}^{2}$. To reduce the time of preparing the scale model and demolishing after the experiment, the prestressing steel will be post-tensioned and unbonded. Two types of prestressing systems are considered, namely: post-tensioning of strands and post-tensioning of bars. The two systems both have different advantages and disadvantages. Decisive for the choice between the two systems is the necessity of anchors when using strands. Due to the small center to center distance of the strands, a lot of anchors are required which is very costly. When prestressing bars are used, the bars can be anchored by just a steel plate which is a lot cheaper. Another advantage of bars is the introduction of the prestressing force in the slabs. The steel plates are positioned at the edge of the outer girders and do not require additional length to introduce the prestressing force, while the anchors of strands are several decimeters long. This would require some additional concrete to the outer girders.

The prestressing bars are anchored at one side and stressed from the other side. The stressing is achieved by pulling out the bar and then applying a nut. When the bar is released, the proper amount of prestressing is present. In this way it is easy to vary the prestressing level from $2.50 \mathrm{~N} / \mathrm{mm}^{2}$ to $1.25 \mathrm{~N} / \mathrm{mm}^{2}$ just by loosening the nuts. This is also an advantage of the prestressing bars over the strands.

## Dimensions prestressing system

As described in chapter 2, the transverse prestressing bars in the Van Brienenoord bridge are positioned with an average center-to-center distance of 649 mm and a maximum center-to-center distance of 800 mm . To model the most unfavourable situation, the maximum center-to-center distance will be scaled
down for the experiment.

$$
\begin{equation*}
\text { c.t.c. } \cdot m=\frac{c . t . c \cdot p}{x}=\frac{800}{2}=400 \mathrm{~mm} \tag{3.6}
\end{equation*}
$$

One prestressing bar will load a concrete area of:

$$
A_{c, b a r}=100 \cdot 400=40000 \mathrm{~mm}^{2}
$$

The bars are dimensioned on the situation of the highest transverse prestressing level of $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}$. The total force working on one bar is:

$$
F_{b a r}=40000 \cdot 2.50=100 k N
$$

The prestressing bars used in the scale model are made by DYWIDAGSystems. The minimum cross section of one bar is:

$$
A_{p, b a r}=\frac{100 \cdot 10^{3}}{900}=112 \mathrm{~mm}^{2}
$$

The bar with the smallest diameter of DYWIDAG-Systems is $\varnothing 15 \mathrm{~mm}$. The area of this bar is $A_{p}=177 \mathrm{~mm}$, the yield strength is 136 kN and the ultimate strength is $195 k N[3]$. This type of bar will be selected to prestress the deck slabs. Appendix G contains the documentation sheet of the DYWIDAG Prestressing Steel Threadbar System.

$$
\begin{equation*}
A_{p} / m m=\frac{177 \cdot 1000}{400 \cdot 1000}=0.4425 \mathrm{~mm}^{2} / \mathrm{mm} \tag{3.7}
\end{equation*}
$$

To apply the bars to the slabs and girders, ducts are necessary. The minimum diameter of the duct for this type of bar is $\varnothing 20 \mathrm{~mm}$. However, to ease the fitting, a more practical diameter is $\varnothing 50 \mathrm{~mm}$. When the girders are positioned, it is easy to fit the bars in the ducts of the slabs. Therefore the ducts in the slabs can be $\varnothing 40 \mathrm{~mm}$, in this way the amount of voids is reduced.
The prestressing bars are anchored by steel plates. The dimensions of the plates are 100 mm by 130 mm . To absorb irregularities at the edge of the girder, felt is placed between the steel plate and the concrete. In this way the prestressing is applied uniformly. The stress of the concrete behind the anchor plate is:

$$
\sigma_{c p}=\frac{100 \cdot 10^{3}}{100 \cdot 130}=7.70 \mathrm{~N} / \mathrm{mm}^{2}
$$

The amount of prestressing necessary per bar for the two load situations is specified in table 3.7 .

| $\sigma_{c p}$ | $F_{p}$ |
| :--- | :--- |
| $1.25 \mathrm{~N} / \mathrm{mm}^{2}$ | $50 k N$ |
| $2.50 \mathrm{~N} / \mathrm{mm}^{2}$ | 100 kN |

Table 3.7: Prestressing force per bar

## Reinforcement

The applied reinforcement in the Van Brienenoordbridge is $\varnothing 8-200 \mathrm{~mm}$ in longitudinal direction and $\varnothing 8-250 \mathrm{~mm}$ in transverse direction. The ratio of reinforcement is kept the same in the prototype bridge and in the scale model.

## Longitudinal direction

$$
\rho_{p}=\frac{\frac{1}{4} \cdot \pi \cdot 8^{2} \cdot \frac{1000}{200}}{200 \cdot 1000}=0.00126
$$

To keep the reinforcement ratio the same $\varnothing 4-100 \mathrm{~mm}$ can be applied:

$$
\rho_{m}=\frac{\frac{1}{4} \cdot \pi \cdot 4^{2} \cdot \frac{1000}{100}}{100 \cdot 1000}=0.00126
$$

## Transverse direction

$$
\rho_{p}=\frac{\frac{1}{4} \cdot \pi \cdot 8^{2} \cdot \frac{1000}{250}}{200 \cdot 1000}=0.00101
$$

To keep the reinforcement ratio the same $\varnothing 4-125 \mathrm{~mm}$ can be applied:

$$
\rho_{m}=\frac{\frac{1}{4} \cdot \pi \cdot 4^{2} \cdot \frac{1000}{125}}{100 \cdot 1000}=0.00101
$$

The reinforcement ratio is lower than the minimum reinforcement ratio, also the bar diameter necessary to achieve the right ratio is very small. If bigger bars are used, the reinforcement would not be scaled down exactly and therefore the experiment would be influenced in a too favourable way. Because of these issues it is decided to neglect the low amount of reinforcement and only apply transverse prestressing.

### 3.2.4 Girders

The total length of the scale model is $12 m$, it is divided in one main span of 10.95 m and two cantilevers of 0.525 m . The cantilevers ensure the transmission of prestressing from the steel to the concrete.

## Cross section

The shape of the cross section is determined by the shape of the Van Brienenoord bridge. The prefabricated girders are made in the factory of Spanbeton. By adapting an existing mould of Spanbeton, the costs will be lower than making a complete new mould. Therefore some restrictions exist on the cross section of the girder. Figure 3.1b shows the dimensions of the
mould of Spanbeton. In consultation with Spanbeton, the cross section of the girder will be produced according to figure 3.1a.


Figure 3.1: Dimensions of the center girders
In the Van Brienenoord bridge the slab between the girders has a thickness of 200 mm . Scaling down with a factor 1:2, the slab thickness becomes 100 mm in the scale model. To ensure the proper behaviour of the connection between the girder and slab, the top flange is also reduced to a thickness of 100 mm . Another restriction is the thickness of the web, the minimum practical thickness for casting is 150 mm . After the restrictions, the most important parameter is the moment of inertia in longitudinal direction. The girders must have an equivalent stiffness to the Van Brienenoord bridge. This can be achieved by scaling down the moment of inertia in longitudinal direction according to Savides.

$$
\begin{equation*}
I_{x x, m}=\frac{I_{x x, p}}{x^{4}}=\frac{1.3197 \cdot 10^{12}}{2^{4}}=82.4813 \cdot 10^{9} \mathrm{~mm}^{4} \tag{3.8}
\end{equation*}
$$

Due to the restrictions it is not possible to meet the required value. The moment of inertia in longitudinal direction with the dimensions according to figure 3.1a has a moment of inertia of $I_{x x}=70.2295 \cdot 10^{9} \mathrm{~mm}^{4}$. This is lower than the calculated value, the reduction is:

$$
\begin{equation*}
\text { Reduction of } I_{x x}=\left(1-\frac{70.2295}{82.4813}\right) \cdot 100 \%=15 \% \tag{3.9}
\end{equation*}
$$

The moment of inertia in transverse direction also needs to be scaled down according to Savides. The smallest width will be checked per meter length, which is the web.

$$
\begin{equation*}
I_{y y, m}=\frac{I_{y y, p}}{x^{3}}=\frac{\frac{1}{12} \cdot 1000 \cdot 200^{3}}{2^{3}}=83.33 \cdot 10^{6} \mathrm{~mm}^{4} / \mathrm{m} \tag{3.10}
\end{equation*}
$$

Due to the restrictions the thickness of the web cannot be smaller than 150 mm . This results in a minimum moment of inertia in transverse direction which is sufficient.

$$
\begin{equation*}
I_{y y}=\frac{1}{12} \cdot 1000 \cdot 150^{3}=281.25 \cdot 10^{6} \mathrm{~mm}^{4} / \mathrm{m} \tag{3.11}
\end{equation*}
$$

The outer girders will have to introduce the transverse prestressing force into the slabs. They must also give some horizontal restraint to introduce compressive membrane action. Both issues are solved by casting an additional part of concrete to the top flange of the outer girders. The extra parts of 125 mm provide a straight surface where the steel plates can be positioned and they also provide more stiffness in transverse direction. Figure 3.2 shows the additional part of concrete at the outer girders.


Figure 3.2: Additional part concrete outer girders

## Prestressing steel

To determine the required amount of prestressing steel, two stages are considered namely:

- $\mathrm{t}=0$ : Self weight of the girder + prestressing force $P_{m 0}$
- $\mathrm{t}=\infty$ : Self weight of the girder + prestressing force $P_{m \infty}+$ weight of slabs + point load

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Concrete area | $A_{c}$ | 342900 | $\mathrm{~mm}^{2}$ |
| Neutral axis to top fibre | $z_{c, \text { top }}$ | 565 | mm |
| Neutral axis to bottom fibre | $z_{c, \text { bottom }}$ | 735 | mm |
| Moment of inertia | $I_{c}$ | $70.2295 \cdot 10^{9}$ | $\mathrm{~mm}^{4}$ |
| Section modulus top | $W_{c, \text { top }}$ | $124.300 \cdot 10^{6}$ | $\mathrm{~mm}^{3}$ |
| Section modulus bottom | $W_{c, \text { bottom }}$ | $95.550 \cdot 10^{6}$ | $\mathrm{~mm}^{3}$ |
| Weight of the girder | $q_{G}$ | 0.857 | ton $/ \mathrm{m}$ |

Table 3.8: Section properties girder scale model

The experiment focuses on the bearing capacity of the slab, so the girder must be able to take the full point load without cracking. Therefore the limits of prestressing become:

$$
\begin{aligned}
& t=0: \\
& 1 \geq \sigma_{c} \geq-f_{c d} \\
& \sigma_{c, \text { top }}=-\frac{P_{m 0}}{A_{c}}+\frac{P_{m 0} \cdot e}{W_{c, t o p}}-\frac{M_{G}}{W_{c, \text { top }}} \\
& \sigma_{c, \text { bottom }}=-\frac{P_{m 0}}{A_{c}}-\frac{P_{m 0} \cdot e}{W_{c, \text { bottom }}}+\frac{M_{G}}{W_{c, \text { bottom }}} \\
& t=\infty: \\
& f_{c t m} \geq \sigma_{c} \geq-f_{c d} \\
& \sigma_{c, \text { top }}=-\frac{P_{m \infty}}{A_{c}}+\frac{P_{m \infty} \cdot e}{W_{c, \text { top }}}-\frac{M_{G}+M_{S}+M_{Q}}{W_{c, t o p}} \\
& \sigma_{c, \text { bottom }}=-\frac{P_{m \infty}}{A_{c}}-\frac{P_{m \infty} \cdot e}{W_{c, b o t t o m}}+\frac{M_{G}+M_{S}+M_{Q}}{W_{c, b o t t o m}}
\end{aligned}
$$

The losses at $t=\infty$ for strands are assumed to be $15 \%$.
The girder must not fail under an exceptionally large load of $1375 k N$. However, the load is not positioned directly on top of the girder. The eccentricity of the load is geometrically determined according to figure 3.3. So the girder is loaded by:

$$
\begin{equation*}
F_{\text {girder }}=\frac{1425}{375+1425} \cdot F=0.8 \cdot F=0.8 \cdot 1375=1100 k N \tag{3.12}
\end{equation*}
$$



Figure 3.3: Distribution of load over the girders
The length of the cantilevers in the structural model is determined according to figure 3.4.


Figure 3.4: Cantilever length
Appendix A contains a Maple file which describes the force distribution of the girder. The results of the file, the force distribution of the girder, are discussed in appendix B. The following parameters were adapted to find the correct stress distribution:

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Prestressing force | $\mathrm{P}_{\text {m0 }}$ | 4951 | kN |
| Eccentricity | e | 389 | mm |
| Applied point load | F | 1100 | kN |

Table 3.9: Parameters adapted for design girders
The amount of prestressing steel can be calculated with the prestressing force. Spanbeton uses strands of $\varnothing 15.7 \mathrm{~mm}$ with an area of $150 \mathrm{~mm}^{2}$. The number of strands becomes:

$$
\begin{aligned}
& A_{p, \text { min }}=\frac{4951 \cdot 10^{3}}{1395}=3549 \mathrm{~mm}^{2} \\
& \# \text { strands }=\frac{3549}{150}=23.7
\end{aligned}
$$

The required amount of prestressing steel to satisfy the stress conditions is 24 strands of $\varnothing 15.7 \mathrm{~mm}$. The eccentricity of the strands used in the calculation is 389 mm . Figure 3.5 shows the distribution of the strands which results in the required eccentricity. The center of gravity of the strands is can be calculated with the distribution.
$z=\frac{3 \cdot 47+2 \cdot 60+3 \cdot 100+4 \cdot 150+2 \cdot(200+250+360+660+900+1200)}{24}=346 \mathrm{~mm}$

## Transmission length

The cantilevers at both sides of the girder function as length over which the prestressing force can be transferred from the steel to the concrete. According to NEN-EN-1992-1-1 cl. 8.10.2 the length of the cantilevers is sufficient to transfer the prestressing force. The dimensions of the cantilever are depicted in figure 3.4 .

The stress in the prestressing cable:

$$
\begin{aligned}
\sigma_{p m 0} & =\frac{4951 \cdot 10^{3}}{24 \cdot 150}=1375 \mathrm{~N} / \mathrm{mm}^{2} \\
\sigma_{p m \infty} & =\frac{0.85 \cdot 4951 \cdot 10^{3}}{24 \cdot 150}=1169 \mathrm{~N} / \mathrm{mm}^{2}
\end{aligned}
$$

It is assumed that the full design value of the tensile strength of the prestressing steel is activated (which is a conservative approach):

$$
\sigma_{p d}=\frac{1860}{1.1}=1691 \mathrm{~N} / \mathrm{mm}^{2}
$$

$f_{c t d}(t)$ is the design tensile strength at time of release:

$$
f_{c t d}(t)=\alpha_{c t} \cdot 0.7 \cdot \frac{f_{c t m}}{\gamma_{c}}=1.0 \cdot 0.7 \cdot \frac{4.16}{1.5}=1.94 \mathrm{~N} / \mathrm{mm}^{2}
$$



Figure 3.5: The distribution of the strands

At release of the strands, the prestressing force is assumed to be transferred to the concrete by a constant bond stress $f_{b p t}$ :

$$
f_{b p t}=\eta_{p 1} \cdot \eta_{1} \cdot f_{c t d}(t)=3.2 \cdot 1.0 \cdot 1.94=6.2 \mathrm{~N} / \mathrm{mm}^{2}
$$

The basic value of the transmission length $l_{p t}$ is:

$$
\begin{aligned}
& l_{p t}=\alpha_{1} \cdot \alpha_{2} \cdot \varnothing \cdot \frac{\sigma_{p m 0}}{f_{b p t}}=1.0 \cdot 0.19 \cdot 15.7 \cdot \frac{1375}{6.2}=662 \mathrm{~mm} \\
& l_{p t 1}=0.8 \cdot l_{p t}=530 \mathrm{~mm} \\
& l_{p t 2}=1.2 \cdot l_{p t}=795 \mathrm{~mm}
\end{aligned}
$$

The bond strength for anchorage in ultimate limit state is:

$$
f_{b p d}=\eta_{p 2} \cdot \eta_{1} \cdot f_{c t d}=1.2 \cdot 1.0 \cdot \frac{f_{c t k, 0.05}}{\gamma_{c}}=1.2 \cdot 1.0 \cdot \frac{0.7 \cdot 4.16}{1.5}=2.3 \mathrm{~N} / \mathrm{mm}^{2}
$$

The total anchorage length for anchoring a strand having a stress $\sigma_{p d}$ :

$$
l_{b p d}=l_{p t 2}+\alpha_{2} \cdot \varnothing \cdot \frac{\left(\sigma_{p d}-\sigma_{p m \infty}\right)}{f_{b p d}}=795+0.19 \cdot 15.7 \cdot \frac{(1691-1169)}{2.3}=1472 \mathrm{~mm}
$$

Figure 3.6 shows the transmission lengths and the anchorage length in ultimate limit state. The dotted line represents the strands at release and the other line the strands at ultimate limit state.


Figure 3.6: Ultimate limit state with respect to anchorage failure

The position of the first flexural crack in ultimate limit state has to be determined. The cracking moment has to be based on the characteristic tensile strength of the concrete $f_{c k, 0.05}$.

$$
\begin{equation*}
M_{c r, b o t t o m}=\left(f_{c k, 0.05}+\sigma_{c p, b o t t o m}\right) W_{c, b o t t o m} \tag{3.13}
\end{equation*}
$$

Figure 3.7 shows the location of the first flexural crack in ultimate limit state. The crack occurs under the point load when it is applied at 2900 mm , so outside the anchorage length $l_{b p d}=1472 \mathrm{~mm}$.


Figure 3.7: Moment distribution at first flexural crack in ULS [Nmm]

## Reinforcing steel

Determination of the shear capacity of the girder starts with checking if the girder is cracked or uncracked in the ultimate limit state. Therefore the cracking moment must be known. The design flexural cracking moment:

$$
\begin{align*}
& M_{c r}=f_{r} \cdot \frac{I_{c}}{y}=\left(-\sigma_{c p}+f_{c t d}\right) \cdot W_{c}  \tag{3.14}\\
& M_{c r, t o p}=\left(\frac{0.85 \cdot 4951 \cdot 10^{3}}{342900}+\frac{0.7 \cdot 4.16}{1.5}\right) \cdot 124.300 \cdot 10^{6}=1766.8 \mathrm{kNm} \\
& M_{c r, \text { bottom }}=\left(\frac{0.85 \cdot 4951 \cdot 10^{3}}{342900}+\frac{0.7 \cdot 4.16}{1.5}\right) \cdot 95.550 \cdot 10^{6}=1358.2 \mathrm{kNm}
\end{align*}
$$

The force distribution in the ultimate limit state:

$$
\begin{equation*}
M_{E d} \sim 1.35 \cdot q_{G+S}+1.35 \cdot F+1.0 \cdot P_{m \infty} \tag{3.15}
\end{equation*}
$$

From these conditions the cracked and uncracked parts can be obtained resulting in figure 3.8. The moment distribution is depicted in section B.4.


Figure 3.8: Overview of the cracked and uncracked parts
Table B. 4 shows the maximum values of the moments and shear force in the ultimate limit state. The maximum amount of shear force in the uncracked zone occurs when the point load is applied at $2 d_{p}$ from the support: $V_{E d}=1280.2 k N$. In the zones which are uncracked in the ultimate limit state, the design value for the shear resistance of the concrete is calculated by NEN-EN-1992-1-1 cl. 6.2.2(2).

$$
\begin{gather*}
V_{R d, c}=\frac{I \cdot b_{w}}{S} \sqrt{f_{c t d}^{2}+\alpha_{l} \cdot \sigma_{c p} \cdot f_{c t d}}  \tag{3.16}\\
V_{R d, c}=\frac{70.2295 \cdot 10^{9} \cdot 150}{70.90 \cdot 10^{6}} \sqrt{1.94^{2}+1.0 \cdot \frac{0.85 \cdot 4951 \cdot 10^{3}}{342900} \cdot 1.94}=780 \mathrm{kN}
\end{gather*}
$$

The shear resistance of the concrete is not high enough for the occurring shear force. However, for the experiment the shear resistance can be determined by taking the mean value of axial tensile strength. This results in a capacity which is sufficient for the experiment.

$$
V_{R d, c}=\frac{70.2295 \cdot 10^{9} \cdot 150}{70.90 \cdot 10^{6}} \sqrt{4.16^{2}+1.0 \cdot \frac{0.85 \cdot 4951 \cdot 10^{3}}{342900} \cdot 4.16}=1229 \mathrm{kN}
$$

Structures with shear reinforcement, prestressed with straight prestressing In the zone cracked in bending the calculation of the required shear reinforcement is almost the same as for reinforced concrete. The only difference is the presence of the axial compressive stress $\sigma_{c p}$ which results in an increase of shear resistance.

In the uncracked zone both the uncracked compression zone and the uncracked tensile zone contribute to shear resistance. Because the cracks do not proceed to the outer fibres of the beam, they hardly open. Therefore the crack width is small and the shear reinforcement is only lightly stressed. If the shear force is higher than the force that causes tensile splitting shear failure, which implies that shear reinforcement has to be applied, the contribution of the concrete is somewhat higher than in case of shear bending failure. NEN-EN 1992-1-1 could take this positive effect into account by allowing for a larger rotation of the compressive diagonal concrete struts. This is not accounted for in the code: it is prescribed that the calculation of the required amount of shear reinforcement should be carried out following the same procedure as for the cracked zone. The design value for the shear
resistance of the concrete is calculated by NEN-EN-1992-1-1 cl. 6.2.2(1) and is the maximum value of:

$$
\begin{align*}
& V_{R d, c}=\left[C_{R d, c} \cdot k \cdot\left(100 \cdot \rho_{l} \cdot f_{c k}\right)^{\frac{1}{3}}+k_{1} \cdot \sigma_{c p}\right] \cdot b_{w} \cdot d  \tag{3.17}\\
& V_{R d, c, \text { min }}=\left(v_{\text {min }}+k_{1} \cdot \sigma_{c p}\right) \cdot b_{w} \cdot d \tag{3.18}
\end{align*}
$$

The effective depth $d_{p}$ is determined for the strands in the tension zone only, so the four strands at the top in figure 3.5 are not taken into account. The calculation of the shear resistance:

$$
\begin{aligned}
& d=1300-205=1095 \mathrm{~mm} \\
& k=1+\sqrt{\frac{200}{d}}=1+\sqrt{\frac{200}{1095}}=1.43<2.0 \\
& \rho_{l}=\frac{3600}{342900}=0.010 \\
& \sigma_{c p}=\frac{0.85 \cdot 4951 \cdot 10^{3}}{342900}=12.3 \mathrm{~N} / \mathrm{mm}^{2}>0.2 \cdot f_{c d}=0.2 \cdot \frac{53}{1.5}=7.1 \mathrm{~N} / \mathrm{mm}^{2} \\
& V_{R d, c}=\left[0.12 \cdot 1.43 \cdot(100 \cdot 0.010 \cdot 53)^{\frac{1}{3}}+0.15 \cdot 7.1\right] \cdot 150 \cdot 1095=281 \mathrm{kN} \\
& v_{\text {min }}=0.035 \cdot k^{\frac{3}{2}} \cdot f_{c k}^{\frac{1}{2}}=0.0035 \cdot 1.43^{\frac{3}{2}} \cdot 53^{\frac{1}{2}}=0.44 \\
& V_{R d, c, \text { min }}=(0.44+0.15 \cdot 7.1) \cdot 150 \cdot 954=247 \mathrm{kN}
\end{aligned}
$$

Since $247 k N$ is much lower than the occurring shear force, stirrups have to be applied. The amount of stirrups is calculated according to NEN-EN-1992-1-1 cl. 6.2.3(3) For members with vertical shear reinforcement, the shear resistance $V_{R d}$ is the smaller value of:

$$
\begin{align*}
& V_{R d, s}=\frac{A_{s w}}{s} \cdot z \cdot f_{y w d} \cdot \cot \theta  \tag{3.19}\\
& V_{R d, \text { max }}=\alpha_{c w} \cdot b_{w} \cdot z \cdot v_{1} \cdot \frac{f_{c d}}{\cot \theta+\tan \theta} \tag{3.20}
\end{align*}
$$

This requirement sets the limits of $V_{R d, s}$ and $V_{R d, \max }$ :

$$
\begin{equation*}
V_{E d} \leq V_{R d, s}, V_{R d, \max } \tag{3.21}
\end{equation*}
$$

$V_{E d}$ is maximum at the location of the first flexural crack. The location is depicted in figure 3.8 and the maximum value is calculated in section B.4.

$$
V_{E d}=1187 \mathrm{kN}
$$

$$
\begin{aligned}
& z=0.9 d=0.9 \cdot 1095=986 \mathrm{~mm} \\
& \theta=25.2^{\circ}
\end{aligned}
$$

The applied stirrups have a diameter of 10 mm .

$$
A_{s w}=2 \cdot \frac{1}{4} \cdot \pi \cdot 10^{2}=157 \mathrm{~mm}^{2}
$$

The amount of shear reinforcement per $m m$ is:

$$
\frac{A_{s w}}{s}=\frac{V_{E d}}{z \cdot \cot \theta \cdot f_{y w d}}=\frac{1187 \cdot 10^{3}}{0.9 \cdot 1095 \cdot \cot 25.2 \cdot \frac{500}{1.15}}=1.304 \mathrm{~mm}^{2} / \mathrm{mm}
$$

So the maximum spacing of the stirrups is:

$$
\begin{aligned}
& s=\frac{157}{1.304}=120.4 \mathrm{~mm} \\
& s_{\text {applied }}=120 \mathrm{~mm}
\end{aligned}
$$

The shear resistance $V_{R d}$ is the smaller value of:

$$
\begin{gathered}
V_{R d, s}=\frac{157}{120} \cdot 0.9 \cdot 1095 \cdot \frac{500}{1.15} \cdot \cot 25.2=1192 \mathrm{kN} \\
V_{R d, \max }=1.25 \cdot 150 \cdot 0.9 \cdot 1095 \cdot 0.6 \cdot\left(1-\frac{53}{250}\right) \cdot \frac{\frac{53}{1.5}}{\cot 25.2+\tan 25.2}=1189 \mathrm{kN}
\end{gathered}
$$

Figure 3.9 shows the shear reinforcement of the girders.


Figure 3.9: Reinforcement of the girders by Spanbeton

### 3.2.5 Transversely prestressed end cross beam

## Cross section

After the prestressed girders are placed the end cross beams can be casted. The end cross beams provide stiffness in transverse direction by stabilizing the girders. The presence of transverse prestressing increases the restraint of the girders. By scaling down the end cross beam of the Van Brienenoord bridge, the dimensions can be determined.

$$
w_{m}=\frac{w_{p}}{x}=\frac{700}{2}=350 \mathrm{~mm}
$$

The width of the end cross beams is 350 mm , the height is dependant of the height of the girders. There are some restrictions to the height of the end cross beams. During the experiment the bottom side of the slabs need to be inspected to investigate the cracking pattern, so there has to be space to reach the bottom side. Therefore an opening of 300 mm is located at the bottom, which is the same height as the bottom flange. Between the top flange of the girder and the end cross beams there has to be space for loading close to the support. If this space was filled by the end cross beams, they would support the slabs and this results in a too favourable ultimate load. By having openings at the top and bottom of the girder, the end cross beams have a cross section of $350 \times 810 \mathrm{~mm}$. Figure 3.10 depicts an overview of the dimensions of the end cross beams.

## Prestressing steel

Because of the presence of transverse prestressing in the bridge deck slab, the end cross beams need to be transversely prestressed as well. The amount of prestressing must be approximately the same for a proper force transfer. The slab is prestressed to $2.50 \mathrm{~N} / \mathrm{mm}^{2}$, so the end cross beam will get the same amount. The total force necessary to prestress one end cross beam to the same level as the slabs is:

$$
P_{m}=A_{c} \cdot \sigma_{c p}=350 \cdot 810 \cdot 2.5=708.8 \mathrm{kN}
$$

To introduce the prestressing force uniform, eight bars per end cross beam are present. Therefore the force per prestressing bar is:

$$
F_{b a r}=\frac{708.8}{8}=88.6 \mathrm{kN}
$$

Appendix Gcontains the documentation sheet of the DYWIDAG Prestressing Steel Threadbar System.


Figure 3.10: Dimensions end cross beams

### 3.2.6 Concrete to concrete interface

The girders and the slabs are connected by a concrete to concrete interface, so the interfaces act as boundary conditions on the slabs. Therefore the bearing capacity of the slabs is highly influenced by the properties of the interface. The properties can be split up into two main parameters, namely: the roughness and the skewness of the interface. The calculations of the influence on the bearing capacity are described in section 4.3, this section describes the properties applied in the scale model.

## Roughness

One of the basic mechanisms for shear transfer is frictional resistance in joint interfaces. The resistance depends on concrete to concrete friction. In joints to precast elements the roughness of the joint faces may vary and the shear resistance may concern uncracked as well as cracked conditions. The roughness of joint faces can be controlled by treatment of the fresh concrete. Joint faces can by classified with regard to its natural roughness, roughness after special treatment or even specially formed shear keys[7]. Eurocode 2 distinguishes four surface classes for concrete to concrete interfaces.

The roughness of the interface of the scale model is introduced by placing specially formed shear keys in the moulds. Over the whole length of the interface, this special pattern is present. The shear keys are have the shape
of a teardrop and are positioned in a rotated pattern. The height of the teardrops is $1-2 \mathrm{~mm}$ and the length and width are $30 \times 10 \mathrm{~mm}$. The pattern is depicted in figure 3.11 .


Figure 3.11: Ruukki DIN 59220 teardrop pattern

## Skewness

The skewness of the interface in the Van Brienenoord bridge is caused by two issues, the skewness of the edge of the girders and the slope of the deck. The edge of the girders is not a vertical plane, but lies under an angle with a slope of approximately $1: 100$. In addition, the slabs which connect with the girders lie also under an angle. Because the slope of the deck is difficult to obtain in the scale model, these properties are represented by a skew interface. The skewness of the interface is 5 mm over the height of 100 mm , so a slope of $1: 20$. To investigate the influence of the skewness, not all interfaces are skew. The scale model consists of three slab of which two have skew interfaces. Figure 3.12 shows the skewness of the interface and figure 3.13 shows the location of the skew interfaces.


Figure 3.12: Skewness of the interface


Figure 3.13: Location skew interfaces

### 3.3 Loading

The scale model will be subjected to two types of loading, namely: transverse prestressing and a wheel load. The prestressing system is described in section 3.2.3. The wheel load will be represented by a jack which is hanging on a frame. By varying the position of the load, the scale model can be tested over the whole surface. The influence of the following parameters can be investigated by changing the loading of the experiment:

- Transverse prestressing level

1. $1.25 \mathrm{~N} / \mathrm{mm}^{2}$
2. $2.50 \mathrm{~N} / \mathrm{mm}^{2}$

- Position of wheel load

1. Center of slab
2. Edge of slab

- Amount of wheel loads

1. One
2. Two

- Concrete to concrete interface

1. Straight
2. Skew

- Influence transverse prestressing steel

1. Between two prestressing bars
2. On top of a prestressing bar

### 3.4 Overview experiment set-up

The drawings are made by A. Bosman.


Figure 3.14: Top view experiment set-up


Figure 3.15: Side view experiment set-up


Figure 3.16: Front view experiment set-up

## Chapter 4

## Analytical calculations

### 4.1 Bearing capacity deck slab

### 4.1.1 Introduction

The bearing capacity of the deck slab will be determined by an experiment. The dimensions of the scale model which will be tested during the experiment are determined in chapter 3 . This chapter will give a prediction for the failure load of the slab cast between the girders, calculated analytically. The purpose of an analytical calculation is to verify the output of the finite element analysis which will be done afterwards. It will also create more insight in the occurring failure mechanisms and their theoretical background.

### 4.1.2 Calculation method

The starting point of the calculation of the bearing capacity is the report of De Rooij: Loading capacity of laterally restrained prestressed concrete slabs [10]. This report describes two failure mechanisms, namely: bending and punching shear failure. The theory of both failure mechanisms is elaborated in this report. However, when the theory is compared with experimental findings, only punching shear failure is considered. All the slabs tested in the experiments failed by punching shear, therefore only this failure mechanism is considered in the analytical findings. The report shows a flowchart for calculating the punching capacity of a concrete slab and is depicted in figure 4.1.

The flowchart is designed to calculate the ultimate punching load $P_{u}$ on a transversely prestressed concrete slab. The calculation is done by a script written in Matlab, which solves the equations of the theory by iteration. The script is based on the theory of the Kinnunen and Nylander punching model. Hewitt and Batchelor modified the model, so it incorporated the effect of compressive membrane action. Hewitt and Batchelor proposed the usage of a restraint factor $\eta$ to express unspecified values of the horizontal force


Figure 4.1: Flowchart for calculating punching capacity of a concrete slab
$F_{b}$ and moment $M_{b}$ caused by lateral restraint in terms of the maximum possible values $F_{b, \max }$ and $M_{b, \max }$.

In the thesis written by Wei, Assessment of real loading capacity of concrete slabs[15], this theory is explained. In the report of He, Punching behaviour of composite decks with transverse prestressing [6], the way to make use of prestressing in the model is presented. De Rooij 10 adjusted the script to meet the boundary conditions of the models evaluated in his thesis. Various scripts have been elaborated and compared with the experiments carried out by He [6]. By comparing various scripts and experiments, two calculation methods fit the output best.

1. Hewitt \& Batchelor model + manually adjusting the restraint factor
2. Hewitt \& Batchelor model + variable restraint by calculation

The difference in the variants on the Hewitt and Batchelor model are the use of the restraint factor. The prestressing force increases the compressive membrane action and therefore the restraining of the slab. In all the variants the restraint factor is dependent on the prestressing level of the concrete.

The first method uses a graph following from the experiment He 6] did. He found a relation between the transverse prestressing force and the restraint factor. Different values for the restraint factor $\eta$ are taken depending on the level of prestressing. When the restraint factor value is chosen it does not change during the calculation. The restraint factor represents the total confinement level of the slab, including the prestressing force.


Figure 4.2: $\eta$-TPL Relationship [6]

The second method uses the method of superposition. It consists of two calculations done separately, which result in $P_{u a}$ and $P_{u b}$. The difference is the adjustment of the restraint factor $\eta . P_{u a}$ is calculated with a restraint factor of 0.2 , which corresponds with the proportion of the surrounding concrete only. $P_{u b}$ corresponds with the proportion of the prestressing of the concrete and is calculated with a restraint factor of:

$$
\begin{equation*}
\eta=\frac{F_{p}}{F_{b, \max }} \tag{4.1}
\end{equation*}
$$

### 4.1.3 Input calculation

The Matlab scripts have to be adjusted for the scale model of the Van Brienenoord bridge. Therefore all the properties have to be adjusted. The input is listed in this section.

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Diameter of the slab | c | 1050 | mm |
| Major axis load area | r 1 | 200 | mm |
| Minor axis load area | r 2 | 200 | mm |
| Area of prestressing steel | Ap | 0.4425 | $\mathrm{~mm}^{2} / \mathrm{mm}$ |
| Height of the slab | h | 100 | mm |
| Compressive strength of concrete | fck | 45 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Cube compressive strength of concrete | fcube | 55 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Compressive stress of concrete | sigmap | $1.25-2.50$ | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Tensile strength of prestressing steel | fpk | 1100 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Young's modulus of prestressing steel | Es | 195000 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Restraint factor | eta | variable | - |

Table 4.1: Input of scale model for Matlab scripts

Method 1 uses figure 4.2 to determine the restraint factor.

| $\sigma_{c p}\left[N / \mathrm{mm}^{2}\right]$ | $\eta[-]$ |
| :--- | :--- |
| 1.25 | 0.35 |
| 2.50 | 0.45 |

Table 4.2: Determination restraint factor $\eta$

The parameters listed above are fixed values. The dimensions of the scale model can be measured precisely. The transverse prestressing of the concrete can be adjusted to the right magnitude during the experiment, so that parameter is controlled as well. The concrete quality is the only parameter subjected to change. Although it can be changed in the calculation
afterwards, it is useful to investigate the influence of the concrete quality beforehand.

### 4.1.4 Eurocode 2

To draw conclusions from the results of the calculations, it is useful to compare the output of the scripts with the calculation of punching shear from the Eurocode. The maximum punching shear load is calculated according to NEN-EN-1992-1-1 cl. 6.4.4. The punching shear resistance of a slab should be assessed for the basic control section according to 6.4.2. The design punching shear resistance may be calculated as follows:

$$
\begin{equation*}
v_{R d, c}=C_{R d, c} \cdot k \cdot\left(100 \rho_{l} \cdot f_{c k}\right)^{\frac{1}{3}}+k_{1} \cdot \sigma_{c p} \geq\left(v_{m i n}+k_{1} \cdot \sigma_{c p}\right) \tag{4.2}
\end{equation*}
$$

The perimeter of the load area $u$ :

$$
u=2\left(c_{1}+c_{2}\right)+4 \pi d=2 \cdot(200+200)+4 \pi \cdot 50=1428 m m
$$

The scaling coefficient $k$ :

$$
k=1+\sqrt{\frac{200}{d}}=1+\sqrt{\frac{200}{50}}=3 \rightarrow k=2.0
$$

The reinforcement ratio $\rho_{l}$ :

$$
\rho_{l}=\sqrt{\rho_{l y} \cdot \rho_{l z}}=\sqrt{\frac{0.4425}{100} \cdot 0}=0
$$

Due to the fact that the reinforcement ratio of the slab is zero, the governing capacity of the punching shear resistance depends only on $v_{\text {min }}$ and the amount of transverse prestressing $\sigma_{c p}$. The transverse prestressing level differs between $1.25 \mathrm{~N} / \mathrm{mm}^{2}$ and $2.50 \mathrm{~N} / \mathrm{mm}^{2}$, which influences the resistance against punching shear failure. The higher the transverse prestressing, the higher the resistance against punching shear failure. Therefore the two levels have to be considered separately, starting with $1.25 \mathrm{~N} / \mathrm{mm}^{2}$ :

$$
\sigma_{c p}=\frac{\left(\sigma_{c y}+\sigma_{c z}\right)}{2}=\frac{1.25+0}{2}=0.625 \mathrm{~N} / \mathrm{mm}^{2}
$$

The resistance against punching shear failure is:

$$
v_{R d, c 1.25, \min }=0.035 \cdot 2.0^{\frac{3}{2}} \cdot \sqrt{45}+0.1 \cdot 0.625=0.73 \mathrm{~N} / \mathrm{mm}^{2}
$$

The resistance against punching shear failure when the transverse prestressing level is $2.50 \mathrm{~N} / \mathrm{mm}^{2}$ :

$$
\sigma_{c p}=\frac{\left(\sigma_{c y}+\sigma_{c z}\right)}{2}=\frac{2.50+0}{2}=1.25 \mathrm{~N} / \mathrm{mm}^{2}
$$

The resistance against punching shear failure is:

$$
v_{R d, c 2.50, \text { min }}=0.035 \cdot 2.0^{\frac{3}{2}} \cdot \sqrt{45}+0.1 \cdot 1.25=0.79 \mathrm{~N} / \mathrm{mm}^{2}
$$

The maximum punching shear load can be calculated according with:

$$
\begin{aligned}
& v_{R d, c}=v_{E d}=\frac{V_{E d}}{u \cdot d}=\frac{1.35 \cdot P_{u}}{1428 \cdot 50} \\
& P_{u, 1.25}=\frac{1428 \cdot 50}{1.35} \cdot 0.73=38.6 \mathrm{kN} \\
& P_{u, 2.50}=\frac{1428 \cdot 50}{1.35} \cdot 0.79=41.8 \mathrm{kN}
\end{aligned}
$$

### 4.1.5 Results

Method 1: Manually adjusting the restraint factor

| Concrete class | $\sigma_{c p}\left[\mathrm{~N} / \mathrm{mm}^{2}\right]$ | $\eta[-]$ | $P_{u}[\mathrm{kN}]$ |
| :--- | :--- | :--- | :--- |
| C40/50 | 1.25 | 0.35 | 244.2 |
| C40/50 | 2.50 | 0.45 | 295.7 |
| C45/55 | 1.25 | 0.35 | 268.8 |
| C45/55 | 2.50 | 0.45 | 328.5 |
| C50/60 | 1.25 | 0.35 | 292.7 |
| C50/60 | 2.50 | 0.45 | 361.2 |

Table 4.3: Method 1: $P_{u}$ for various concrete classes


Figure 4.3: Method 1: $\mathrm{P}_{u}$ for various concrete classes

## Method 2: Variable restraint by calculation

In method 2 the restraint factor $\eta$ is output from the calculation.

| Concrete class | $\sigma_{c p}\left[\mathrm{~N} / \mathrm{mm}^{2}\right]$ | $\eta[-]$ | $P_{u}[\mathrm{kN}]$ |
| :--- | :--- | :--- | :--- |
| C40/50 | 1.25 | 0.35 | 315.4 |
| C40/50 | 2.50 | 0.48 | 362.9 |
| C45/55 | 1.25 | 0.33 | 329.3 |
| C45/55 | 2.50 | 0.45 | 377.5 |
| C50/60 | 1.25 | 0.32 | 343.5 |
| C50/60 | 2.50 | 0.42 | 392.3 |

Table 4.4: Method 2: $P_{u}$ for various concrete classes


Figure 4.4: Method 2: $\mathrm{P}_{u}$ for various concrete classes

## Supplemental results

The results described in the previous paragraph are calculated with $f_{c k}$ and $d=50 \mathrm{~mm}$. However, in the experiment the results can be higher, because the mean value of the compressive strength is higher than the characteristic value. Therefore the calculations have also been made with $f_{c m}$. Another influencing parameter is the effective depth. With the approximation of $d=50 \mathrm{~mm}$ a lower bound for the bearing capacity is found, because in this approximation the lower half does not contribute to the strength. However, during the experiment the lower half of the slab will also contribute to the strength, so also the upper bound of the bearing capacity will be determined with $d=100 \mathrm{~mm}$. The concrete class of the slab in the experiment is C45/55, this is not adapted in the supplemental results.

| Method 1 with $f_{c m}$ | $\eta[-]$ | $P_{u}[\mathrm{kN}]$ |
| :--- | :--- | :--- |
| $\sigma_{c p}=1.25 \mathrm{~N} / \mathrm{mm}^{2}, d=50 \mathrm{~mm}$ | 0.35 | 307.5 |
| $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}, d=50 \mathrm{~mm}$ | 0.45 | 381.0 |
| $\sigma_{c p}=1.25 \mathrm{~N} / \mathrm{mm}^{2}, d=100 \mathrm{~mm}$ | 0.35 | 509.4 |
| $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}, d=100 \mathrm{~mm}$ | 0.45 | 535.1 |

Table 4.5: Supplemental results Method 1

| Method 2 with $f_{c m}$ | $\eta[-]$ | $P_{u}[\mathrm{kN}]$ |
| :--- | :--- | :--- |
| $\sigma_{c p}=1.25 \mathrm{~N} / \mathrm{mm}^{2}, d=50 \mathrm{~mm}$ | 0.31 | 352.4 |
| $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}, d=50 \mathrm{~mm}$ | 0.41 | 401.4 |
| $\sigma_{c p}=1.25 \mathrm{~N} / \mathrm{mm}^{2}, d=100 \mathrm{~mm}$ | 0.33 | 616.2 |
| $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}, d=100 \mathrm{~mm}$ | 0.42 | 701.9 |

Table 4.6: Supplemental results Method 2

## Horizontal restraining force

The horizontal force which restrains the slab is output from the scripts. This value is important for the shear capacity of the concrete to concrete interface because the extra membrane force increases the capacity.

| $\sigma_{c p}\left[N / \mathrm{mm}^{2}\right]$ | $F_{b}[k N]$ | $P_{u}[k N]$ |
| :--- | :--- | :--- |
| 1.25 | 339.1 | 268.8 |
| 2.50 | 467.4 | 328.5 |

Table 4.7: Membrane force at failure

### 4.1.6 Conclusion

From the comparison with the Eurocode it can be concluded that both models result in a higher resistance against punching shear failure. This result can be explained by the conservative calculation in the Eurocode. Both models take into account the full prestressing in contrary to the Eurocode which uses: $k_{1} \cdot \sigma_{c p}=0.1 \sigma_{c p}$. So only $10 \%$ of the prestressing is used for the resistance against punching shear. The ratio between calculation method 1 and the Eurocode is approximately 7 and the ratio between calculation method 2 and the Eurocode is approximately 9 .

Some notes have to be added to the calculation methods using the Matlab scripts. In the report of De Rooij 10, the lowest transverse prestressing level is $2 \mathrm{~N} / \mathrm{mm} 2$. Below that value the calculation methods are not compared with experimental data. When method 1 is evaluated in the report, it can be seen that the results differ the most at low levels of prestressing. The accuracy improves when the prestressing increases. Method 2 overestimates the maximum punching load with approximately $10 \%$, but the results have the same behaviour as the experimental data. The difference between the experiments and the calculation method is constant at every prestressing level.

Another note to the calculation methods is the use of a restraint factor. In method 1 this factor is chosen in accordance with empirical data. In method 2 the restraint factor is calculated and depends on the prestressing force, which is added to the empirical data representing the surrounding
concrete. Therefore the restraint factor is higher. However, the effect of the surrounding concrete is not calculated accurately in both calculation methods. So the exact restraint factor can only be obtained by comparing the experimental results with the theoretical results afterwards. For this comparison the transverse displacement of the girders should be monitored during the experiment.

HB n, Pu, prestressing steel


Figure 4.5: Results De Rooij: method 1

HB m2 t2, prestressing steel


Figure 4.6: Results De Rooij: method 2

### 4.2 Capacity Van Brienenoord bridge

The bearing capacity of the Van Brienenoord bridge is the final output of the experiment. The results obtained by the Matlab scripts can be calculated back to the ultimate load on the bridge. Several methods can be used to do the calculation of scaling the results of the experiment:

- Scaling according to the size effect coefficient $k$
- Input of Van Brienenoord bridge into Matlab scripts

The slab in the scale model is unreinforced, so the effective depth of the slab is small. In the Van Brienenoord bridge some reinforcement is present, which enlarges the effective depth and therefore the scaling factor. If the reinforcement of the Van Brienenoord bridge would be neglected, the comparison with the scale model is more realistic. Neglecting the reinforcement of the Van Brienenoord bridge will result in a lower bound of the real capacity of the deck. Another reason to neglect the reinforcement is the fact that the Matlab scripts do not take into account the reinforcing steel.

### 4.2.1 Scaling according to the size effect coefficient $k$

Determination of the size effect coefficient:
Scale model:

$$
\begin{aligned}
& d=\frac{100}{2}=50 \mathrm{~mm} \\
& k=1+\sqrt{\frac{200}{d}}=1+\sqrt{\frac{200}{50}}=3.0 \rightarrow k=2.0 \\
& u=2 \cdot(200+200)+4 \pi \cdot 50=1428 \mathrm{~mm} \\
& d=\frac{200}{2}=100 \mathrm{~mm} \\
& k=1+\sqrt{\frac{200}{d}}=1+\sqrt{\frac{200}{100}}=2.4 \rightarrow k=2.0 \\
& u=2 \cdot(200+200)+4 \pi \cdot 100=2057 \mathrm{~mm}
\end{aligned}
$$

Van Brienenoord bridge:

The size effect coefficient is equal for both the scale model and the Van Brienenoord bridge. Therefore also the resistance against shear is the same, because $k$ is the only parameter which is dependent on the dimensions. The ultimate load can be determined by $v_{R d, c}=v_{E d}$. Due to the fact that the resistance is equal, also the stress due to the load must be equal.

$$
\begin{equation*}
v_{E d, s c a l e m o d e l}=v_{E d, V a n B r i e n e n o o r d b r i d g e} \tag{4.3}
\end{equation*}
$$

$$
\begin{align*}
& \frac{1.35 \cdot P_{u, s c a l e}}{u \cdot d}=\frac{1.35 \cdot P_{u, V a n B r}}{u \cdot d} \rightarrow \frac{P_{u, s c a l e}}{1428 \cdot 50}=\frac{P_{u, V a n B r}}{2057 \cdot 100} \\
& P_{u, V a n B r}=\frac{2057 \cdot 100}{1428 \cdot 50} \cdot P_{u, s c a l e}=2.880 \cdot P_{u, s c a l e} \tag{4.4}
\end{align*}
$$

| Calculation method 1 | $P_{u, \text { scalemodel }}[k N]$ | $P_{u, \text { VanBrienenoordbridge }}[k N]$ |
| :--- | :--- | :--- |
| $\sigma_{c p}=1.25 \mathrm{~N} / \mathrm{mm}^{2}$ | 268.8 | 774.1 |
| $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}$ | 328.5 | 946.1 |
| Calculation method 2 |  |  |
| $\sigma_{c p}=1.25 \mathrm{~N} / \mathrm{mm}^{2}$ | 329.3 | 948.4 |
| $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}$ | 377.5 | 1087.2 |

Table 4.8: Scaling back ultimate load Van Brienenoord bridge

### 4.2.2 Van Brienenoord bridge into Matlab scripts

## Input

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Diameter of the slab | c | 2100 | mm |
| Major axis load area | r 1 | 200 | mm |
| Minor axis load area | r 2 | 200 | mm |
| Area of prestressing steel | Ap | 0.4884 | $\mathrm{~mm}{ }^{2} / \mathrm{mm}$ |
| Height of the slab | h | 200 | mm |
| Compressive strength of concrete | fck | 45 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Cube compressive strength of concrete | fcube | 55 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Compressive stress of concrete | sigmap | $1.25-2.50$ | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Tensile strength of prestressing steel | fpk | 1262 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Young's modulus of prestressing steel | Es | 195000 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Restraint factor | eta | variable | - |

Table 4.9: Input of Van Brienenoord bridge for Matlab scripts

## Results

| Calculation method 1 | $P_{u, \text { VanBrienenoordbridge }}[k \mathrm{~N}]$ |
| :--- | :--- |
| $\sigma_{c p}=1.25 \mathrm{~N} / \mathrm{mm}^{2}$ | 883.1 |
| $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}$ | 1138.6 |
| Calculation method 2 |  |
| $\sigma_{c p}=1.25 \mathrm{~N} / \mathrm{mm}^{2}$ | 850.8 |
| $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}$ | 1124.9 |

Table 4.10: Ultimate load Van Brienenoord bridge, loading plate $200 \times 200$
The results of table 4.10 are calculated with a loading plate of $200 \mathrm{~mm} \times$ 200 mm . However, according to NEN-EN 1991-2 cl. 4.3.2, Load Model 1 consists of four wheel loads with a surface of $400 \mathrm{~mm} \times 400 \mathrm{~mm}$. The bigger loading surface influences the bearing capacity of the slab positively. Therefore the calculations are also done with a bigger loading surface, the results are in table 4.11.

| Calculation method 1 | $P_{u, \text { Van Brienenoordbridge }}[\mathrm{kN}]$ |
| :--- | :--- |
| $\sigma_{c p}=1.25 \mathrm{~N} / \mathrm{mm}^{2}$ | 1008.7 |
| $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}$ | 1324.1 |
| Calculation method 2 |  |
| $\sigma_{c p}=1.25 \mathrm{~N} / \mathrm{mm}^{2}$ | 1014.8 |
| $\sigma_{c p}=2.50 \mathrm{~N} / \mathrm{mm}^{2}$ | 1384.3 |

Table 4.11: Ultimate load Van Brienenoord bridge, loading plate $400 \times 400$

### 4.2.3 Conclusion

The difference between the up-scaled value of the scale model and the value of the Van Brienenoord bridge can be explained by the amount and type of prestressing steel applied. When the Van Brienenoord bridge dimensions are inserted in the Matlab script, strands are used so the steel quality is higher. On the other hand, the amount of steel per millimeter is less than in the scale model. The prestressing level of the concrete remains the same. When comparing the calculation methods, method 2 matches the outcome of the scale model and the Van Brienenoord bridge best.

Using a larger loading plate influences the bearing capacity of the slab. From the calculations with plates of $200 \times 200$ and $400 \times 400$, it results in an increase of approximately $15 \%$ with the larger plates. This increase is important when the experimental results are known. Then the results can be scaled to determine the bearing capacity of the Van Brienenoord brigde.

### 4.3 Concrete to concrete interface

In a concrete to concrete interface subjected to shear loading, various mechanisms interact and influence each other. The occurring mechanisms are:

- Adhesive bonding
- Mechanical interlocking
- Friction
- Dowel action

When no interface reinforcement is applied in the interface, the behaviour is brittle. Failure occurs at small slips of the interface. However, when reinforcement is applied the behaviour of the interface becomes more ductile. This results in larger slip of the interface at failure.


Figure 4.7: Dimensions + shear distribution of the interface

### 4.3.1 Capacity of vertical interface

The maximum punching shear load is calculated according to NEN-EN-1992-1-1 cl. 6.2.5:

$$
\begin{align*}
& v_{R d i}=c \cdot f_{c t d}+\mu \cdot \sigma_{n}+\rho \cdot f_{y d} \cdot(\mu \sin \alpha+\cos \alpha) \leq 0.5 \cdot v \cdot f_{c d}  \tag{4.5}\\
& f_{c t d}=\alpha_{c t} \cdot \frac{f_{c t k, 0.05}}{\gamma_{c}}=1.0 \cdot \frac{2.7}{1.5}=1.8 \mathrm{~N} / \mathrm{mm}^{2}
\end{align*}
$$

The stress per unit area caused by the minimum external normal force across the interface that can act simultaneously with the shear force is $\sigma_{n}$. This normal force can be caused by the transverse prestressing, but also from the compressive membrane action. The minimum external force is the prestressing. The calculation of the capacity of the interface with the prestressing only results in a lower bound. If the compressive membrane action is also taken into account, this results in an upper bound. The values of the restraining force are given in table 4.7 .

The calculations of the upper bound are done with $f_{c t d}=f_{c t m}$, because $f_{c t d}$ is too conservative for the experiment. This approximation causes another increase of the capacity of the upper bound.

Prestressing (lower bound)

- $\sigma_{n 1.25}=1.25 \mathrm{~N} / \mathrm{mm}^{2}$
- $\sigma_{n 2.50}=2.50 \mathrm{~N} / \mathrm{mm}^{2}$

Prestressing + compressive membrane action (upper bound)

- $\sigma_{n 1.25}=\frac{339.1 \cdot 10^{3}}{1450 \cdot 100}=2.34 \mathrm{~N} / \mathrm{mm}^{2}$
- $\sigma_{n 2.50}=\frac{467.4 \cdot 10^{3}}{1450 \cdot 100}=3.22 \mathrm{~N} / \mathrm{mm}^{2}$

The reinforcement which crosses the interface is not bonded to the concrete. Therefore the steel cannot be taken into account for shear capacity.

$$
\begin{aligned}
& \rho=0 \\
& v=0.6 \cdot\left(1-\frac{f_{c k}}{250}\right)=0.6 \cdot\left(1-\frac{45}{250}\right)=0.48 \\
& v_{R d, \max }=0.5 \cdot 0.48 \cdot \frac{50}{1.5}=8.0 \mathrm{~N} / \mathrm{mm}^{2}
\end{aligned}
$$

The maximum applied load can be calculated with:

$$
\begin{aligned}
& v_{E d}=\frac{\beta \cdot V_{E d}}{z \cdot b}=\frac{1.0 \cdot 1.35 \cdot \frac{1}{2} \cdot P_{u, i}}{100 \cdot 1450}<v_{R d i} \\
& P_{u i}<\frac{2 \cdot 100 \cdot 1450}{1.35} \cdot v_{R d i}
\end{aligned}
$$

The area loaded in shear is determined by spreading of the load under $45^{\circ}$. Half of the total load is taken by $b=1450 \mathrm{~mm}$ and the height $z=$ 100 mm . Figure 4.7 shows the distribution of spreading.

| Surface class | $c$ | $\mu$ | $v_{R d, 1.25}$ | $v_{R d, 2.50}$ | $P_{u, 1.25}[k N]$ | $P_{u, 2.50}[k N]$ |
| :--- | :--- | :--- | :--- | :--- | :--- | :--- |
| Very smooth | 0.25 | 0.5 | 1.08 | 1.70 | 230.9 | 365.2 |
| Smooth | 0.35 | 0.6 | 1.38 | 2.13 | 296.4 | 457.6 |
| Rough | 0.45 | 0.7 | 1.69 | 2.56 | 362.0 | 549.9 |
| Indented | 0.50 | 0.9 | 2.03 | 3.15 | 435.0 | 676.7 |

Table 4.12: Capacity of vertical interface (lower bound)

| Surface class | $c$ | $\mu$ | $v_{R d, 1.25}$ | $v_{R d, 2.50}$ | $P_{u, 1.25}[k N]$ | $P_{u, 2.50}[k N]$ |
| :--- | :--- | :--- | :--- | :--- | :--- | :--- |
| Very smooth | 0.25 | 0.5 | 1.85 | 2.29 | 396.3 | 490.9 |
| Smooth | 0.35 | 0.6 | 2.35 | 2.88 | 504.6 | 618.0 |
| Rough | 0.45 | 0.7 | 2.85 | 3.47 | 612.9 | 745.2 |
| Indented | 0.50 | 0.9 | 3.46 | 4.25 | 742.4 | 912.5 |

Table 4.13: Capacity of vertical interface (upper bound)

### 4.3.2 Skewness of the interface

The concrete to concrete interface is not a connection between two vertical planes. A certain amount of skewness is present. The skewness of the interface is 5 mm horizontal over the height of 100 mm , so $1: 20$. Therefore an additional vertical force loads the concrete deck slab, which reduces the shear capacity.


Figure 4.8: Skewness of the interface


Figure 4.9: Decomposition of forces

The vertical component created by the shape of the interface can be calculated according to figure 4.9 .

- $F_{H}$ is the force caused by transverse prestressing.
- $F_{V}$ is the vertical component caused by the shape of the interface.
- $F_{\perp}$ is the force working perpendicular to the plane of the interface.

$$
\begin{align*}
& \tan \alpha=\frac{F_{v}}{F_{H}}=\frac{5}{100}  \tag{4.6}\\
& f_{V}=f_{H} \cdot \frac{5}{100} \tag{4.7}
\end{align*}
$$

The total reduction has to be calculated over the length of the interface:

$$
\begin{equation*}
\frac{1}{2} \cdot F_{V}=f_{V} \cdot l_{\text {interface }}=f_{V} \cdot 1450 \tag{4.8}
\end{equation*}
$$

### 4.3.3 Capacity of skew interface

## Reduction due to transverse prestressing

The presence of transverse prestressing causes a horizontal force in the deck slab. Due to the skewness of the interface a part of the prestressing loads the slab extra in vertical direction. This extra load reduces the resistance of the interface.

$$
\begin{align*}
& \frac{1}{2} \cdot F_{V}=f_{V} \cdot l_{\text {interface }}  \tag{4.9}\\
& F_{V, 1.25}=2 \cdot 100 \cdot 1.25 \cdot \frac{5}{100} \cdot 1450=18.1 \mathrm{kN} \\
& F_{V, 2.50}=2 \cdot 100 \cdot 2.50 \cdot \frac{5}{100} \cdot 1450=36.3 \mathrm{kN}
\end{align*}
$$

## Reduction due to membrane action

Just like the transverse prestressing, the membrane forces causes an extra reduction of the resistance of the interface.

$$
\begin{align*}
& \frac{1}{2} \cdot F_{V}=f_{V} \cdot l_{\text {interface }}  \tag{4.10}\\
& F_{V, 1.25}=2 \cdot 339.1 \cdot \frac{5}{100} \cdot 1450=49.2 \mathrm{kN} \\
& F_{V, 2.50}=2 \cdot 467.4 \cdot \frac{5}{100} \cdot 1450=67.8 \mathrm{kN}
\end{align*}
$$

## Capacity of the skew interface

The capacity of the skew interface can be calculated by taking the vertical interface and reducing the capacity with the losses due to transverse prestressing and membrane action. The results are collected in table 4.14 and 4.15.

| Surface class | $P_{u, 1.25}[k N]$ | $P_{u, 1.25, \text { skew }}[k N]$ | $P_{u, 2.50}[k N]$ | $P_{u, 2.50, \text { skew }}[k N]$ |
| :--- | :--- | :--- | :--- | :--- |
| Very smooth | 230.9 | 212.8 | 365.2 | 328.9 |
| Smooth | 296.4 | 278.3 | 457.6 | 421.3 |
| Rough | 362.0 | 343.9 | 549.9 | 531.6 |
| Indented | 435.0 | 416.9 | 676.7 | 640.4 |

Table 4.14: Capacity skew interface (lower bound)

| Surface class | $P_{u, 1.25}[k N]$ | $P_{u, 1.25, \text { skew }}[k N]$ | $P_{u, 2.50}[k N]$ | $P_{u, 2.50, \text { skew }}[k N]$ |
| :--- | :--- | :--- | :--- | :--- |
| Very smooth | 396.3 | 347.1 | 490.9 | 423.1 |
| Smooth | 504.6 | 455.4 | 618.0 | 550.2 |
| Rough | 612.9 | 563.7 | 745.2 | 677.4 |
| Indented | 742.4 | 693.2 | 912.5 | 844.7 |

Table 4.15: Capacity skew interface (upper bound)

### 4.3.4 Conclusion

According to section 3.2 .6 the interface used in the scale model belongs to surface class "smooth" of Eurocode 2. The height of the teardrops in the patterned plate is 2 mm , the interface would belong to surface class "rough" if the roughness is at least 3 mm . Eurocode 2 describes the surface class 'smooth' as follows:
"a slipformed or extruded surface, or a free surface left without further treatment after vibration"

When the values of the lower bounds in tables 4.3, 4.4 and 4.15 are compared, it can be concluded that the interface has enough capacity to resist the wheel load until failure when the high level of prestressing is applied. If the prestressing level is lowered to $1.25 \mathrm{~N} / \mathrm{mm}^{2}$ the capacity of the interface is lower than the bearing capacity of the slab, so the scale model will fail at the interfaces.

Comparing the values of the upper bounds in tables 4.5, 4.6 and 4.15, it can be concluded that the capacity of the interface is enough when the effective depth $d$ is 50 mm . When the maximum value is taken for $d(=100 \mathrm{~mm})$,
the results of method 1 (table 4.5) can be taken only by the straight interfaces. However, the results of method 2 (table 4.6) are too high even for the straight interfaces, so the scale model fails at the interfaces.

## Chapter 5

## Finite Element Analysis

### 5.1 Introduction

The test set-up of the scale model is not only analysed analytically, but also by using a finite element model. During this analysis it is possible to investigate the behaviour of the concrete slab which is loaded. Using incremental step sizes, the decrease of stiffness due to cracking can be found. Also the influence of the interface between the slab and the girder can be investigated. The results will be presented in load-displacement curves which can depict the different stages of the structure.

The software used to analyse the finite element models are:

- midas FX+ Version 3.1.0
- Mesh Editor Release 9.4.4
- DIANA Release 9.4.4


### 5.2 Model theses Bakker and De Rooij

### 5.2.1 Thesis Bakker

Bakker[2] investigated the influence of compressive membrane action in both 2D and 3D models. The 2D model consists of plane stress elements and focuses on the failure mechanism bending. For the 3D model, Bakker used an axisymmetric model which represents punch or a combination between punch and bending. Figure 5.1 shows the plane stress model and figure 5.2 shows the axisymmetric model.


Figure 5.1: Plane stress model Bakker


Figure 5.2: Axisymmetric model Bakker

One of the parameters Bakker investigated is the influence of tension softening on the failure mechanism. When comparing the finite element models with experiments, Bakker found a change in failure mechanism. Making use of tension softening instead of brittle material behaviour results in a structure which is too ductile. The models do not fail in punching shear, but fail in bending. This is contradicting with the experiments.

### 5.2.2 Thesis De Rooij

De Rooij[10] investigated the presence of transverse prestressing on concrete slabs. Starting point of his investigation is the axisymmetric model of Bakker. De Rooij added transverse prestressing to this model. Problem with the model is the fact that it is axisymmetric, so the prestressing is ap-
plied in radial direction to guarantee equally distribution. The prestressing force is not inserted as an external force but as an imposed deformation which introduces the right amount of prestressing. The model with imposed deformation as prestressing is depicted in figure 5.3 .


Figure 5.3: Axisymmetric model De Rooij
De Rooij compared the finite element model with the experiments carried out by He [6]. At higher prestress levels, De Rooij found lower values for the ultimate capacity than He. From this can be concluded that the 2D axisymmetric model does not give a good indication of the behaviour of a transverse prestressed concrete slab. De Rooij recommends to investigate the influence of prestressing in a 3D model. In this way the prestressing cables can be inserted in only one direction instead of radial direction. The advantage is that the concrete is not influenced by the favourable uniform prestressing in the other direction. Also the boundary conditions can be modelled more realistically in a 3D model.

### 5.3 2D model

### 5.3.1 Introduction

Although the recommendation is to create a 3 D model of the scale model to investigate the bearing capacity of the slab, the starting point of the finite element analysis will be a 2D model. This model is a cross section in transverse direction of the deck and will give information about the amount of arching action in the slab.

### 5.3.2 Properties

## Plane strain

The 2D model represents a cross section of the scale model. Using the plane strain state the cross section is approximated most realistic. Plane strain is
defined to be a state of strain in which the strain normal to the $\mathrm{x}-\mathrm{y}$ plane, $\epsilon_{z}$, and the shear strain $\gamma_{x z}$ and $\gamma_{y z}$ are assumed to be zero. In plane strain, one deals with a situation in which the dimension of the structure in one direction, the z-coordinate direction, is very large in comparison with the dimensions of the structure in the other two directions ( $x$ - and $y$-coordinate axes). The applied forces act in the $\mathrm{x}-\mathrm{y}$ plane and do not vary in the z direction, the loads are uniformly distributed with respect to the large dimension and act perpendicular to it. Plane strain elements are characterized by the fact that their thickness $t$ is equal to unity and that the strain components perpendicular to the element face are zero: $\epsilon_{z z}=0$. Figure 5.4 [4] shows the conditions with respect to position and loading plane strain elements must fulfil.


Figure 5.4: Characteristics of plane strain elements

## Geometry

The 2D model represents a cross section of the scale model. The scale model consists of four girders connected by three slabs cast in between. To provide stiffness in transverse direction, end cross beams are applied at the ends of the supports. The influence of these end cross beams is difficult to model in a 2D model, since their influence is not constant over the length of the girders. The influence of the end cross beams is averaged out over the length of the girders by filling the space between the girders with an elastic material. The properties of the elastic material are discussed in the paragraph of materials. Figure 5.5 shows the geometry of the 2D model. The units used to create the model are $N$ for the forces and $m m$ for the dimensions. Plane strain elements have a unit thickness, so the thickness of the 2 D model is 1 mm .


Figure 5.5: Geometry 2D model

## Elements

The elements used for the 2D model have to be plane strain elements.


Figure 5.6: Plane strain elements for 2D model
The CT12E element (figure 5.6a) is a six-node triangular isoparametric plane strain element. It is based on quadratic interpolation and area integration. This is done because triangular elements are integrated with an integration rule different from the Gauss rule, which is a two dimensional integration rule specifically for triangles. The area coordinates correspond in a natural way to this integration scheme. The polynomial for the displacements $u_{X}$ and $u_{Y}$ can be expressed as:

$$
\begin{equation*}
u_{i}(\xi, \eta)=a_{0}+a_{1} \xi+a_{2} \eta+a_{3} \xi \eta+a_{4} \xi^{2}+a_{5} \eta^{2} \tag{5.1}
\end{equation*}
$$

The CQ16E element (figure 5.6b) is an eight-node quadrilateral isoparametric plane strain element. It is based on quadratic interpolation and Gauss integration. The polynomial for the displacements $u_{X}$ and $u_{Y}$ can be expressed as:

$$
\begin{equation*}
u_{i}(\xi, \eta)=a_{0}+a_{1} \xi+a_{2} \eta+a_{3} \xi \eta+a_{4} \xi^{2}+a_{5} \eta^{2}+a_{6} \xi^{2} \eta+a_{7} \xi \eta^{2} \tag{5.2}
\end{equation*}
$$

## Materials

The materials used in the 2D model are two different strength classes for concrete. The slab consists of $\mathrm{C} 45 / 55$ and the girder consist of C53/65. The tabulated properties:

| C45/55 |  |  |  |
| :--- | :--- | :--- | :--- |
| Young's Modulus | $E_{c m}$ | 36000 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Characteristic compressive cylinder strength | $f_{c m}$ | 53 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Mean value of axial tensile strength | $f_{c t m}$ | 3.8 | $\mathrm{~N} / \mathrm{mm}^{2}$ |

Table 5.1: Properties concrete C45/55

| C53/65 |  |  |  |
| :--- | :--- | :--- | :--- |
| Young's Modulus | $E_{c m}$ | 37846 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Characteristic compressive cylinder strength | $f_{c m}$ | 61 | $\mathrm{~N} / \mathrm{mm}^{2}$ |
| Mean value of axial tensile strength | $f_{c t m}$ | 4.2 | $\mathrm{~N} / \mathrm{mm}^{2}$ |

Table 5.2: Properties concrete C53/65

The presence of the end cross beam is included via an elastic material connecting the girders. This material represents the influence of the end cross beams at the supports of the scale model, to the area which is loaded. The width over which the end cross beam is smeared out is determined in figure 5.7. By spreading over an angle of $45^{\circ}$ the width becomes 1450 mm . The total width of both end cross beams is $2 \cdot 350=700 \mathrm{~mm}$. The elastic material consists of plane strain elements, so they have a unit thickness. Therefore the properties have to be inserted in the Young's modulus.

$$
E_{\text {endcrossbeams }}=\frac{700}{1450} \cdot 37846=18270 \mathrm{~N} / \mathrm{mm}^{2}
$$



Figure 5.7: Width over which the end cross beams are smeared out

Non-linear material properties Compressive membrane action can only occur when the concrete structure behaves non-linear, the concrete cracks under a certain amount of deformation resulting in membrane forces. To include the non-linearity, both physical and geometrical non-linearity are applied in the 2D model. The physical non-linear properties of concrete define the behaviour of the concrete if the maximum compressive or tensile stress is reached. DIANA [4] has several models based on total strain, also called the 'Total Strain crack models', which describe the tensile and compressive behaviour of a material with one stress-strain relationship.


Figure 5.8: Physical non-linearity concrete

As described in section 5.2.1, Bakker [2] recommends to use brittle behaviour of concrete in tension. Figure 5.8a shows the predefined tension softening function of this behaviour. The stress drops to zero when the
cracking strain is reached. Due to the sudden drop of stress, this material property can require a lot of iterations, because new equilibrium has to be obtained.

For concrete in compression the constant compression function is chosen to model the crushing behaviour of concrete. Figure 5.8b shows the predefined compression function of this behaviour. This model is chosen because of the uncertainties in the strength of the concrete. The prescribed strength class used for the slabs is C45/55. However, in the experiment the actual strength of the concrete is probably higher and will be determined by taking test cubes. The actual strength can be included into a model to analyse the experiment results.

## Mesh

The experiment focuses mainly on the slabs between the girders. According to the theory, the slabs have to crack before the phenomenon of compressive membrane action is activated. To investigate the cracking of the slabs in detail a fine mesh is chosen, five elements over the height of the slabs. The girders and the end cross beams are designed to remain uncracked during the experiment, meaning they will behave elastic. Therefore a more coarse mesh is chosen for these materials. Figure 5.9 shows the mesh of the 2D model.


Figure 5.9: Mesh + supports 2D model

## Supports

The supports of the scale model are positioned at the end cross beams. Because the 2D model is a cross section of the scale model, it is difficult to specify the boundary conditions. The girders will deform due to the point load applied on the slab, but this will have minor influence on the slabs. Therefore it is a justifiable assumption to pin one girder and apply rollers on the others. In this way the point load will go from the slab through the girders to the supports in stead of via bending to the support in the experiment. The supports are depicted in figure 5.9 .

## Loads

The first load applied to the scale model is transverse prestressing. This will load the slabs between the girders to enhance the bearing capacity. The load is varied between $1.25 \mathrm{~N} / \mathrm{mm}^{2}$ and $2.50 \mathrm{~N} / \mathrm{mm}^{2}$. Also the end cross beams are prestressed during the experiment. They will have the same amount of prestressing as the slabs, so a maximum of $2.50 \mathrm{~N} / \mathrm{mm}^{2}$. Taking into account this amount of prestressing, the elastic material between the girders must be prestressed as well. Not the full prestressing will be present, but it will be smeared out just like the calculation of the Young's modulus.

$$
\sigma_{c p, \text { endcrossbeams }}=\frac{700}{1450} \cdot 2.50=1.21 \mathrm{~N} / \mathrm{mm}^{2}
$$

The point load will determine the bearing capacity of the concrete slabs. This load is introduced as a displacement to the nodes which are loaded. The surface loaded by the jack is $200 \times 200 \mathrm{~mm}$, so in the 2 D model the displacement has a lenght of 200 mm . In plane strain the assumption is that the load is present over the whole length of the structure in stead of a square $200 \times 200$. This is one of the disadvantages of using the 2 D model. By taking into account an effective width over which the load is distributed like in figure 5.7, the load can be calculated back to the failure load of the scale model. Figure 5.10 shows the loads on the 2D model.


Figure 5.10: Loads 2D model

## Calculation method

The numerical method which is used is Regular Newton-Raphson and the convergence criterion is the displacement norm. The loads are applied in midas FX+ for DIANA. With Mesh Editor the loads are reduced to load increments. The size of the increments is determined by trial and error. The transverse prestressing force is applied in one load step, the displacement representing the wheel load is applied in steps of 0.01 mm . Appendix $F$ shows the DCF-file in which the exact loading of the 2 D model is given.

To determine the load caused by the increasing displacement, the reaction forces in y-direction are summed. Then the results can be presented in load-displacement curves.

### 5.3.3 Results

The results from the 2D model are load-displacement curves from the plane strain state. However, the approximation is that there is an effective width over which the load is spread. Therefore the load in the load-displacement curves are multiplied with the effective width. In this way the influence of spreading is taken into account.

## Parametric study

To investigate the main factors influencing the bearing capacity of the slabs a parametric study was conducted. The influence of the variables transverse prestressing level and angle of spreading were investigated by comparing various values. For every set of parameters, the legend shows what is changed in the set.

## Restraint by surrounding concrete

By comparing a simple supported transversely prestressed slab with the scale model will show the influence of lateral restraint. The dimensions of the slab are the same as the slab in the scale model. Also the transverse prestressing level is identical $\sigma_{c p}=2,50 \mathrm{~N} / \mathrm{mm}^{2}$. Only the boundary conditions vary. Figure 5.11 depicts the simple supported slab.


Figure 5.11: Simple supported transversely prestressed slab


Figure 5.12: Results influence surrounding concrete
The influence of the surrounding concrete is clearly visible in figure 5.12. By laterally restraining the concrete the slab has a higher stiffness, which is due to the occurring phenomenon of compressive membrane action. The graph of the unrestrained slab only has one drop in stiffness. This is because the second drop represents the cracking of the slab at the supports, which does not happen when the slab can rotate freely.

## Transverse prestressing level

Another important parameter for the experiment is the transverse prestressing level. As stated in previous sections, this level varies from $1.25 \mathrm{~N} / \mathrm{mm}^{2}$ to $2.50 \mathrm{~N} / \mathrm{mm}^{2}$. During the finite element analysis also a model is tested which is not prestressed. This clearly shows the effect of transverse prestressing. The load is positioned at the center slab at midspan.


Figure 5.13: Results influence transverse prestressing level
One can easily see the influence of the transverse prestressing level in figure 5.13. By increasing the prestressing level, the bearing capacity increases. The slope of the graphs are equal, because the same amount of restraint is present. The difference between the models is the elastic branch of the load-displacement curves. With an increasing transverse prestressing level, the elastic branch extends. In other words, the first crack is being delayed when the transverse prestressing level increases. Another positive effect of increasing the transverse prestressing level is the drop in force when a crack occurs. Because the drop is smaller, the bearing capacity is higher at the same displacement.

Both the delaying of cracks and small losses of stiffness are beneficial for the serviceability limit state, because higher loads can be applied with the same deflections of the slab.


Figure 5.14: Strains and corresponding crack pattern at various load steps

Figure 5.14 depicts the strains and corresponding crack pattern at various load steps. These figures show the behaviour of the concrete slabs under the wheel load. The elastic branch of the load-displacement curve ranges from 0 mm to 0.35 mm . During the elastic branch the stiffness of the slab is constant, because of the absence of cracks (figure 5.14b). The strains in this part are all below the cracking strains which gives a smooth distribution. The maximum strains are located at the edges of the wheel load and at the connection with the girder (figure 5.14a). The compression arch is becoming visible.
When the displacement of the wheel load is growing from 0.35 mm to 0.36 mm the tensile strength of the concrete is exceeded. Due to the brittle tension curve, the stress at that position drops to zero which results in a crack. The cracks are located at the bottom of the slab under the edges of the wheel load (figure 5.14 d ). The compression arch becomes more visible, because the concrete part under the load contributes less to the total stiffness (figure 5.14 c .
After the first crack occurred, the slab picks up stiffness again until the tensile strength of the concrete is exceeded at the connection with the girder. Figure 5.14 f shows also cracks at both connections with the girders. From this moment on the compression arch becomes most beneficial, because mostly compression is present in the concrete. Only at the start of the cracks, the concrete is in tension. Figure 5.14 e shows the compression arch clearly in dark blue. Only a few irregularities are present, which represent the cracks.
The final stage is when the slab fails. Figure 5.14 h shows that the concrete under the wheel load is failed in tension. Therefore the model can not find equilibrium any more and stops the calculation.
NOTE: When the experiment is carried out, a rubber block is used as load surface to simulate a tire. This will result in a less sharp displacement field and also a crack under the center of the load. Therefore applying the load directly at the slab underestimates the capacity in comparison with the experiment.

## Angle of spreading

The angle of spreading is assumed to be $45^{\circ}$ in figure 5.7. However, the effective width of the slab can only be obtained from the experiment. Therefore this parameter is also investigated in advance. Three angles of spreading are investigated, namely: $30^{\circ}, 45^{\circ}$ and $60^{\circ}$. These are assumed to be the lower and upper bounds of the spreading angle.

By changing the angle of spreading, also the effective width over which the spreading occurs changes. The effective width is calculated with the
following equation:

$$
\begin{equation*}
\text { effective width }=200+2 \cdot\left(100+\frac{1050}{2}\right) \cdot \tan \alpha \tag{5.3}
\end{equation*}
$$

With the new effective width, the corresponding influence of the end cross beams can be calculated. Table 5.3 shows the properties for the Young's modulus and transverse prestressing level of the end cross beams for various amounts of spreading.

| Angle of spreading [ ${ }^{\circ}$ ] | Effective width $[\mathrm{mm}]$ | $E\left[\mathrm{~N} / \mathrm{mm}^{2}\right]$ | $\sigma_{c p}\left[\mathrm{~N} / \mathrm{mm}^{2}\right]$ |
| :--- | :--- | :--- | :--- |
| 30 | 922 | 28733 | 1.90 |
| 45 | 1450 | 18270 | 1.21 |
| 60 | 2365 | 11202 | 0.74 |

Table 5.3: Properties of end cross beams for various amounts of spreading


Figure 5.15: Results influence angle of spreading
Although the properties of the end cross beams are less rigid with an increasing angle of spreading, the total stiffness of the structure is higher. This is due to the multiplication with the larger effective width. The larger effective width lowers the stresses in the concrete, which results in a higher bearing capacity at a certain displacement of the wheel load. The actual amount of spreading can be determined in the experiment, but can also be output of a 3D model.

| Angle [] | Load [kN] | Ratio [-] | Effective width [mm] | Ratio $[-]$ |
| :--- | :--- | :--- | :--- | :--- |
| 30 | 223.0 | 0.63 | 922 | 0.64 |
| 45 | 354.2 | 1.00 | 1450 | 1.00 |
| 60 | 580.1 | 1.64 | 2365 | 1.63 |

Table 5.4: Comparison ratio capacity and effective width

Table 5.4 shows the ratios of the capacity and the effective width. The increase in capacity should be directly related with the increase in effective width, due to the extra portion of the slab which is taken into account. Table 5.4 shows the ratios numerically and figure 5.16 shows the ratios graphically. In both comparisons the ratios are determined by dividing the results with the values of $45^{\circ}$. The ratio of the effective width can be calculated easily for every value of the angle, but a change in angle requires an extra analysis of the model to determine the capacity. Therefore only three angles are tested. Figure 5.16 shows that the ratios are directly related.


Figure 5.16: Comparison ratio capacity and effective width

## Loading center or outer slab

The position of the load is important for the experiment, if one situation gives a very high result this should be known. Therefore the influence of loading the different slabs is investigated. Both variants have a transverse prestressing level of $2.50 \mathrm{~N} / \mathrm{mm}^{2}$ and the wheel load is applied at the middle of the slab.


Figure 5.17: Results influence loading center or outer slab

Between the two loading positions is a small difference in stiffness. This difference can be explained by the amount of restraint given by the surrounding concrete. The center slab is located between two slabs and four girders, so the horizontal support is more than for the outer slab where only one girder gives horizontal support. However, the difference is not very large, which means that the extra parts cast at the sides of the outer girders are functioning. They are placed to introduce horizontal support and thereby introduce compressive membrane action.

### 5.4 Influence concrete to concrete interface

In section 3.2 .6 and 4.3 the properties of the concrete to concrete interface are described. In this section the properties are inserted in the 2D model of section 5.3.

### 5.4.1 Interface elements

The CL12I element (figure 5.18) is an interface element between two lines in a two-dimensional configuration. The local $x y$ axes for the displacements are evaluated in the first node with $x$ from node 1 to node 2 . Variables are oriented in the $x y$ axes. The element is based on quadratic interpolation.


Figure 5.18: Interface elements for 2D model

### 5.4.2 Coulomb friction criterion

The behaviour of the interface between the slabs and the girders is governed by a frictional behaviour. Dry friction resists relative lateral motion of two solid surfaces in contact. Coulomb friction is a model used to calculate the force of dry friction. It is governed by the equation:

$$
\begin{equation*}
F_{t}=\mu F_{n} \tag{5.4}
\end{equation*}
$$

where:

- $F_{t}$ is the force of friction exerted by each surface on the other. It is parallel to the surface, in a direction opposite to the net applied force.
- $\mu$ is the coefficient of friction, which is an empirical property of the contacting materials.
- $F_{n}$ is the normal force exerted by each surface on the other, directed perpendicular (normal) to the surface.


Figure 5.19: Coulomb friction criterion
In DIANA the Coulomb friction model can be adapted by varying the cohesion $c$ and the coefficient of friction $\mu$. It is also possible to introduce a cut-off tensile strength, because the default value corresponds to the apex of the Coulomb friction criterion $f_{t}=c / \tan \phi$.

### 5.4.3 Properties

The properties of the concrete to concrete interface are specified in this section, starting with the stiffness of the interface elements. An initial dummy penalty stiffness is given to the interface elements in order to keep them initially inactive before the cut-off is reached. The dummy stiffness in normal and tangential directions are determined with the following formulas:

$$
\begin{align*}
& k_{n}=1000 \cdot \frac{E_{\text {element }}}{l_{\text {element }}}  \tag{5.5}\\
& k_{t}=1000 \cdot \frac{G_{\text {element }}}{l_{\text {element }}}  \tag{5.6}\\
& k_{n}=1000 \cdot \frac{36000}{20}=1.8 \cdot 10^{6} \mathrm{~N} / \mathrm{mm}^{3} \\
& k_{t}=1000 \cdot \frac{15000}{20}=0.75 \cdot 10^{6} \mathrm{~N} / \mathrm{mm}^{3}
\end{align*}
$$

The cut-off tensile strength is in case of a concrete to concrete interface not $f_{c t m}$, because the bonding strength is lower than in ordinary concrete. For the cut-off strength a value of $f_{\text {ctk,0.05 }}$ is used.

$$
f_{c t k, 0.05}=2.7 N / m^{2}
$$

The properties of the roughness of the interface are determined in section 4.3. From the patterned plates used in the mould follows that the roughness is of category 'Smooth' in Eurocode 2. The corresponding parameters are:

$$
\begin{aligned}
& c=0.35 \cdot f_{c t m}=0.35 \cdot 3,8=1,33 \mathrm{~N} / \mathrm{mm}^{2} \\
& \mu=\tan \phi=0.6
\end{aligned}
$$

## Behaviour after gap appearance

For the behaviour after cracking of the interface, three Mode-II models are examined:

- Brittle
- Constant Shear Retention
- Aggregate interlock relation of Walraven and Reinhardt

For the brittle model, no properties have to be inserted. The calculation just ends when somewhere in the interface the limit stress is reached. The Constant Shear Retention model requires a value for the reduced stiffness after occurrence of a gap. Two values are examined, $1 \%$ and $10 \%$ of the tangential stiffness. The aggregate interlock model of Walraven and Reinhardt requires the compressive strength of the concrete as input, which is $f_{c m}=53 \mathrm{~N} / \mathrm{mm}^{2}$.

### 5.4.4 Results

Before the parametric study can be conducted, the influence of the four Mode-II models has to be investigated. Figure 5.20 shows the load-displacement curves of the models with a straight interface and a transverse prestressing level of $2.50 \mathrm{~N} / \mathrm{mm}^{2}$.


Figure 5.20: Influence Mode-II models after gap appearance
From the graphs the behaviour of the different models can be obtained. By comparing each model with the curve where no interface is applied, the influence of the interface properties can be investigated. The brittle curve immediately ends when it starts to deviate from the one without interfaces. The explanation for the sudden stop is the brittle behaviour of the model, it ends the calculation when the limit stress is reached. The behaviour underestimates the capacity of the interface, because some shear capacity is present when the interface opens. Therefore one of the other models is more appropriate.

The Constant Shear Retention model allows some shear capacity when the interface opens. The amount of resistance can be inserted via the reduced stiffness. From the graphs can be obtained that a reduced stiffness of $1 \%$ of the tangential stiffness results in an interface which is decisive for the failure load. The 2D model fails at the positions of the interface due to the interface properties. However, when a reduced stiffness of $10 \%$ of the tangential stiffness is inserted in the 2D model, the interface is not decisive any more, the slab fails due to cracking. During the experiment can be obtained whether the interface or the slab fails, and then the percentage can be adapted in the Constant Shear Retention model to the correct value.

Also in the aggregate interlock model of Walraven and Reinhardt the interface is decisive for failure of the 2D model. Via the input of the com-
pressive strength of the slab, the amount of shear resistance is calculated.
After comparison of the different Mode-II models, both Constant Shear Retention models are used to conduct the parametric study. By taking both models, two situations are investigated: one where the interface is decisive and one where the slab is decisive.

## Constant Shear Retention 10\%

By investigating the development of membrane force in the slabs, the capacity of the interface can be calculated. Figure 5.21 depicts the development of the membrane force plotted against the displacement of the wheel load. The shear capacity of the Coulomb friction model can be calculated with:

$$
\begin{equation*}
t_{t}=c+\mu \cdot t_{n} \tag{5.7}
\end{equation*}
$$

Following from figure 5.20 the capacity of the interface is enough to make the slab decisive for the failure load. However, figure 5.22 shows that the capacity of the interface crosses the line of the loading. The crossing represents the loading exceeds the capacity of the interface, which would mean that the interface is decisive. The explanation for this contradiction is equation 5.7, due to the properties of the interface after reaching the limit stress, equation 5.7 can not be used any more. The capacity of the interface is larger than calculated with the equation, due to the constant shear retention part.


Figure 5.21: Membrane force development CSR $10 \%$


Figure 5.22: Capacity total deck CSR $10 \%$

If the slab is decisive for the failure load, the finite element analysis can be compared with the failure load calculated in tables 4.5 and 4.6. The values of those tables are added to figure 5.22. From the graphs can be seen that the loading almost reaches the capacity of the analytical results, which is promising for both predictions.

The capacity of the failure load is also calculated according to Eurocode 2. By inserting the transverse prestressing force and the membrane action into the equation, the capacity exceeds the value of section 4.1.4 Eurocode 2 still underestimates the capacity of the slab when it is compared with the other methods.


Figure 5.23: Horizontal opening interface CSR $10 \%$

Figure 5.23 shows the horizontal opening of the interface plotted against the vertical displacement of the wheel load. The interface starts to open when the wheel load has a vertical displacement of 0.3 mm , from that point
on the horizontal opening of the interface increases. The maximum opening of the interface is 0.21 mm .

## Constant Shear Retention 1\%

From figure 5.20 can be seen that the capacity of the interface is decisive for the failure load. Also for the constant shear retention value of $1 \%$ of the tangential stiffness the development of membrane action is investigated. Figure 5.24 shows the development and with equation 5.7 the capacity of the interface is calculated. Because the interface is decisive for the failure load, the loading can not grow after the capacity of the interface is reached. When the graphs of the capacity of the interface and the loading cross, the calculation ends.


Figure 5.24: Membrane force development CSR 1\%


Figure 5.25 : Capacity total deck CSR $1 \%$

Again, the capacity of the failure load is calculated according to Eu-
rocode 2. Also in this comparison Eurocode 2 underestimates the capacity of the slab, which can be seen by the crossing of the graphs of capacity and loading.


Figure 5.26: Horizontal opening interface CSR 1\%

Figure 5.26 shows the horizontal opening of the interface plotted against the vertical displacement of the wheel load. The interface starts to open when the wheel load has a vertical displacement of 0.3 mm , from that point on the horizontal opening of the interface increases. The maximum opening of the interface is less than the Constant Shear Retention model with $10 \%$ stiffness, because shear failure of the interface occurs at a lower displacement of the wheel load.

## Parametric study

After the taking a closer look to the capacity of the slabs and interfaces, the parametric study can be conducted. Two parameters will be investigated, namely:

- Influence of skewness interface
- Influence of distance between load and interface

Both investigations will be conducted with the Constant Shear Retention of $1 \%$, because if the interface is not decisive there will not be a difference in bearing capacity. The transverse prestressing level will be $2.50 \mathrm{~N} / \mathrm{mm}^{2}$ during the investigation.

## Skewness of interface

The skewness of the interface will be investigated by loading the two outer slabs of the 2 D model, in order to ensure the equal amount of restraint.


Figure 5.27: Results influence skewness of interface
Figure 5.27 shows the comparison between the skew and the straight interface. In chapter 4 it was determined that the capacity of the skew interface would be lower, due to the extra vertical force. This reduction in capacity is demonstrated again with this comparison of interface skewness. However, the amount of reduction is very much depending on the input parameters.

## Distance load to interface

The distance between the load and the interface is an important parameter, because this also influences the effective width of the slab. When the load is positioned next to the interface, the amount of spreading is limited, which results in a lower capacity. The investigation is performed on the skew interface, because the previous parametric study stated that the skew interface has the lowest capacity. Also the influence of the position of the interface is investigated, both the interfaces next to the center girders as well as next to the outer girders are tested. Assuming that the load is positioned next to the interface, the amount of spreading is minimum.

$$
\begin{aligned}
& \text { effective width }=200+2 \cdot\left(100+\frac{200}{2}\right) \cdot \tan 45=600 \mathrm{~mm} \\
& E_{\text {endcrossbeams }}=\frac{700}{600} \cdot 37846=44154 \mathrm{~N} / \mathrm{mm}^{2} \\
& \sigma_{\text {cp,endcrossbeams }}=\frac{700}{600} \cdot 2.50=2.92 \mathrm{~N} / \mathrm{mm}^{2}
\end{aligned}
$$



Figure 5.28: Influence of distance load to interface

Figure 5.28 shows the influence of the position of the load. When the load is applied close to the interface the connection behaves brittle. This brittle behaviour can be explained by the fact that the interface is loaded in almost pure shear. Failure occurs even before the first crack in the slab occurs, this results in a lower failure load of the 2D model when loading next to the interface.

The influence of loading next to the center or outer girder is small. The amount of horizontal restraint is different in both cases, but the difference in stiffness is small. The minor difference can be explained by the fact that the interface is loaded in almost pure shear, the influence of restraint contributes less in this situation.

### 5.5 Conclusion

In this chapter various parameters which influence the behaviour of the transversely prestressed concrete deck slabs are investigated. The model is a 2D model and represents a cross section of the scale model in longitudinal direction. Due to this assumption only in plane forces can be evaluated.

Both the influence of lateral restraint and an increasing level of transverse prestressing enhances the behaviour of the concrete slabs. The lateral restraint ensures an overall higher stiffness of the structure, while the increasing transverse prestressing level delays the moment when the first crack occurs. Both enhancements are very beneficial for the serviceability limit state, because a higher load can be applied to an uncracked structure.

An important parameter to investigate during the experiment is the angle of spreading of the load. This angle determines the effective width and has a large influence on both the stiffness of the structure as well as the bearing capacity. Because a larger concrete area is taken into account with a larger angle, the stresses are lower which results in a higher bearing
capacity.
A limitation of the model is the selected compression curve for the concrete. The ideal compression curve was chosen because the exact properties of the concrete are still unknown. By using this curve only a maximum stress is introduced. After reaching this value, the strains can increase and the stress remains constant. This results in equal failure loads when different levels of transverse prestressing are applied, because the slabs only fail in tension. In reality the concrete would also crush in compression.

The properties of the interface between the girders and the slabs are very important for both the stiffness of the 2 D model as well as the failure load. Various models have been investigated for the behaviour of the interface after a gap appears. The Constant Shear Retention model has been investigated in more detail. Two values of the input parameter 'reduced stiffness' resulted in two different failure mechanisms. When the parameter is $10 \%$ of the interface stiffness, the slab is decisive for the failure load. But when the parameter is $1 \%$ of the interface stiffness, the interface becomes decisive. During the experiment can be obtained which failure mechanism occurs and then the model can be fine tuned.

A very important parameter of the experiment is the skewness of the interface. In the previous chapter the forces of the skew interface are decomposed, which resulted in an extra vertical force on the slabs. The extra loading resulted in a lower capacity of the interface. The investigation of this parameter via finite element analysis demonstrated again that the capacity of the skew interface is considerably lower than the capacity of a straight interface.

The punching capacity is the lowest when loading next to the interface. When the load is applied close to the interface the connection behaves brittle. This brittle behaviour can be explained by the fact that the interface is loaded in almost pure shear. Failure occurs even before the first crack in the slab occurs, which results in a lower failure load of the 2 D model.

## Chapter 6

## Conclusions and recommendations

### 6.1 Conclusions

The main part of the research project is the effect of transverse prestressing on compressive membrane action in the slender slabs of the Van Brienenoord bridge. Due to the high slenderness of the slabs, the foreign design codes state that the amount of compressive membrane action is too small to take into account. However, the hypothesis is that the transverse prestressing would enhance the compressive membrane action.

Two analytical methods are used to calculate the bearing capacity of the slabs. Although these methods have not been validated with a lot of tests, the results are promising. Especially in comparison with Eurocode 2, the methods give results which are 7 to 9 times higher. This increase in capacity can be explained by the occurring phenomenon of compressive membrane action.

During the finite element analysis it is demonstrated that the transverse prestressing level influences the bearing capacity of the slabs. The occurrence of a compression arch is clearly visible when the strains are examined. Also the behaviour of the slabs changes when the level is adapted. The first moment of cracking delays when the prestressing level increases, which is beneficial in the serviceability limit state. A disadvantage of a higher prestressing level is a more brittle behaviour of the structure, the failure load is reached at a smaller deflection of the slab.

The properties of the interface between the girders and the slabs are very important for both the stiffness of the 2D model as well as the failure load. Various models have been investigated for the behaviour of the interface after a gap appears. The Constant Shear Retention model has been investigated in more detail. Two values of the input parameter 'reduced stiffness' resulted in two different failure mechanisms. When the parameter is $10 \%$ of the
interface stiffness, the slab is decisive for the failure load. But when the parameter is $1 \%$ of the interface stiffness, the interface becomes decisive. Because the capacity of the interface can change the failure mechanism, it is very important to monitor the occurring failure mechanism during the experiment. Afterwards, the parameter of the interface can be obtained by comparing the experiment and finite element model.

A very important parameter of the experiment is the skewness of the interface. When the forces of the skew interface are decomposed, an extra vertical force loads the slabs. The extra loading results in a lower capacity of the interface. The investigation of this parameter via finite element analysis demonstrates that the capacity of the skew interface is considerably lower than the capacity of a straight interface. Therefore the tests on the slabs supported by skew interfaces is indispensable and determines the lower bound of the capacity of the scale model.

The experiment will be carried out to determine the remaining bearing capacity of the Van Brienenoord bridge. When the failure loads during the experiment reach the values of the prediction in this thesis, the Van Brienenoord bridge will meet the requirements of structural safety. This is demonstrated by scaling back to the dimensions of the bridge. Add to this the increased concrete quality due to ongoing hydration and then the capacity increases even more. That would mean that strengthening the bridge is not necessary yet.

### 6.2 Recommendations

The recommendations focus on the finite element analysis. Two modifications can be made to improve the 2D model. The first modification is taking another compression curve for the concrete slabs. The ideal plastic compression curve just introduces a maximum value for the compressive strength. When that value is reached, the compressive strain increases while the stress remains constant. This results in similar failure loads with varying transverse prestressing levels. The only difference is that the failure load occurs at a larger displacement. By taking test cubes from the concrete of the slabs during the experiment set-up and inserting these properties into the 2 D model, the failure load will not be equal at varying prestressing levels due to crushing of the concrete.

The second modification to the 2 D model is to use the exact angle of spreading which can be measured during the experiment. In the finite element analysis a lower and upper bound are taken, which results in a certain range of effective widths. By measuring the effective width in the experiment and inserting this value in the 2D model's properties, the output from the calculation is more accurate.

Although the two modifications improve the 2 D model, it still does not
represent the experiment accurately. The explanation for this is the absence of the failure mechanism punching shear in a 2 D model. Because only a cross section of the scale model is analysed, the three dimensional effect is not included in the finite element analysis. The only way to include this mechanism is to create a 3 D model.

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## Appendix A

## Maple force distribution girders

```
restart:
w}1:=(\textrm{C}1*\mp@subsup{\textrm{x}}{}{\wedge}3+\textrm{C}2*\mp@subsup{\textrm{x}}{}{\wedge}2+\textrm{C}3*\textrm{x}+\textrm{C}4+(1/24)*\textrm{q}*\mp@subsup{\textrm{x}}{}{\wedge}4)/\textrm{EI}
w2 := (C5*x^3+C6*x^2+C7*x+C8+(1/24)*q*x^4)/EI:
w}3:=(\textrm{C}9*\mp@subsup{\textrm{x}}{}{\wedge}3+\textrm{C}10*\mp@subsup{\textrm{x}}{}{\wedge}2+\textrm{C}11*\textrm{x}+\textrm{C}12+(1/24)*q*\textrm{x}^4)/\textrm{EI}
w4 := (C13*x^3+C}14*\mp@subsup{x}{}{\wedge}2+\textrm{C}15*\textrm{x}+\textrm{C}16+(1/24)*q*x^4)/EI
phi1 := -(diff(w1, x)):
phi2 := -(diff(w2, x)):
phi3 := -(diff(w3, x)):
phi4 := -(diff(w4, x)):
M1 := EI*(diff(phi1, x)):
M2 := EI*(diff(phi2, x)):
M3 := EI*(diff(phi3, x)):
M4 := EI*(diff(phi4, x)):
V1 := diff(M1, x):
V2 := diff(M2, x):
V3 := diff(M3, x):
V4 := diff(M4, x):
x := 0:
eq1 := M1 = -P*e:
eq2 := V1 = 0:
x := x1:
eq3 := w1 = 0:
eq4 := w1 = w2:
```

```
eq5 := phi1 = phi2:
eq6 := M1 = M2:
x := x2:
eq7 := w2 = w3:
eq8 := phi2 = phi3:
eq9 := M2 = M3:
eq10 := V2 = V3+F:
x := x3:
eq11 := w3 = 0:
eq12 := w3 = w4:
eq13 := phi3 = phi4:
eq14 := M3 = M4:
x := l:
eq15:= M4 = -P*e:
eq16 := V4= 0:
solution := solve({eq1, eq2, eq3, eq4, eq5 , eq6 , eq7 , eq8,
eq9, eq10, eq11, eq12, eq13, eq14, eq15, eq16},
{C1, C10, C11, C12, C13, C14, C15, C16, C2, C3, C4, C5,
C6, C7, C8, C9}): assign(solution): x := 'x':
w := piecewise(x< x1, w1, x1 < x and x < x2, w2,
x}2<\textrm{x}\mathrm{ and }\textrm{x}<\textrm{x}3, w3, x > x3, w4):
M := piecewise(x< x1, M1, x1 < x and x < x2, M2,
x2<x and x < x3, M3, x > x3, M4):
V := piecewise(x< x1, V1, x1 < x and x < x2, V2,
x}2<\textrm{x}\mathrm{ and }\textrm{x}<\textrm{x}3, V3, x> x3, V4)
x1 := 525:
x2 := 6000:
x3 := l-x1:
l := 12000:
# Parameters
# t=0
A := 342900:
E := 37846:
Ic := 70.229454348*10^9:
EI := E*Ic:
l:= 12000:
```

```
zctop := 565:
zcbot := 1300-zctop:
e := zcbot - 346;
Wcbot := Ic/zcbot;
Wctop := Ic/zctop;
Mcrt := (4.155+P/(.85*A))*Wctop:
Mcrb := (4.155+P/(.85*A))*Wcbot:
q}:=25.0*A*10^(-6)
P}:=0
F := 0*10^ 3:
plot(-w, x = 0 .. l);
plot ({Mcrt, -M, -Mcrb}, x = 0 .. l);
x := x1: print(M2): x := x2: print(M2): x := 'x':
plot(V, x = 0 .. l );
x := x1: print(V1, V2): x := x2: print(V2): x := 'x':
# t=infinity
q}:=((A+1050*100)*25.0)*10^(-6)
P}:=(1-.15)*P
F}:=1100*10^3
plot(-w, x = 0 .. l);
plot({Mcrt, -M, -Mcrb}, x = 0 .. l );
x := x1: print(M2): x := x2: print(M2): x := ' }\textrm{x}'=\mp@code{:
plot(V, x = 0 .. l );
x := x1: print(V1, V2): x := x2: print(V2): x := 'x':
# Shear force
x2 := x1+2*(zctop+e );
q}:=25.0*(1.35*(A+1050*100))*10^(-6)
P}:=4951*(1-.15)*10^3
F}:=(1.35*1100)*10^3
plot({Mcrt, -M, -Mcrb}, x = 0 .. l );
x := x1: print(M2): x := x2: print(M2): x := 'x':
plot(V, x = 0 .. l); x := x2: print(V2): x := ' }\textrm{x}'
```


## Appendix B

## Force distribution girders

Appendix A contains a Maple file which describes the force distribution of the girder. The results of the file, the force distribution of the girder, are discussed in this appendix. The following parameters were adapted to find the correct stress distribution:

| Description | Sign | Value | Unit |
| :--- | :--- | :--- | :--- |
| Prestressing force | $\mathrm{P}_{\text {m }}$ | 4951 | kN |
| Eccentricity | e | 389 | mm |
| Applied point load | F | 1100 | kN |

Table B.1: Parameters adapted for design girders

In the figures below all the situations are reviewed. The effect of prestressing is not taken into account in the distribution, it is included when the stresses are calculated.

## B. 1 Determination maximum prestressing force

$\mathbf{t}=\mathbf{0}$ : Self weight of the girder + prestressing force $P_{m 0}$


Figure B.1: Structural model at $t=0$


Figure B.2: Moment distribution at $t=0$ [Nmm]


Figure B.3: Shear distribution at $t=0[\mathrm{~N}]$


Figure B.4: The location of the cross sections

| $t=0$ | CrosssectionA | CrosssectionB |
| :--- | :--- | :--- |
| Moment [kNm] | -1.2 | 127.3 |
| Shear force [kN] | 46.9 | 0.0 |

Table B.2: Values at cross sections $t=0$

## Values at cross sections

## Cross section A

$$
\begin{aligned}
\sigma_{c, \text { top }} & =-\frac{P_{m 0}}{A_{c}}+\frac{P_{m 0} \cdot e}{W_{c, \text { top }}}-\frac{M_{G}}{W_{c, \text { top }}} \\
\sigma_{c, \text { top }} & =-\frac{4951 \cdot 10^{3}}{342900}+\frac{4951 \cdot 10^{3} \cdot 389}{124.300 \cdot 10^{6}}-\frac{-1.2 \cdot 10^{6}}{124.300 \cdot 10^{6}}=0.99 \mathrm{~N} / \mathrm{mm}^{2} \\
\sigma_{c, \text { bottom }} & =-\frac{P_{m 0}}{A_{c}}-\frac{P_{m 0} \cdot e}{W_{c, b o t t o m}}+\frac{M_{G}}{W_{c, \text { bottom }}} \\
\sigma_{c, \text { bottom }} & =-\frac{4951 \cdot 10^{3}}{342900}+\frac{4951 \cdot 10^{3} \cdot 389}{95.550 \cdot 10^{6}}-\frac{-1.2 \cdot 10^{6}}{95.550 \cdot 10^{6}}=-34.50 \mathrm{~N} / \mathrm{mm}^{2}
\end{aligned}
$$

## Cross section B

$$
\begin{aligned}
\sigma_{c, \text { top }} & =-\frac{P_{m 0}}{A_{c}}+\frac{P_{m 0} \cdot e}{W_{c, \text { top }}}-\frac{M_{G}}{W_{c, \text { top }}} \\
\sigma_{c, \text { top }} & =-\frac{4951 \cdot 10^{3}}{342900}+\frac{4951 \cdot 10^{3} \cdot 389}{124.300 \cdot 10^{6}}-\frac{127.3 \cdot 10^{6}}{124.300 \cdot 10^{6}}=-0.05 \mathrm{~N} / \mathrm{mm}^{2} \\
\sigma_{c, \text { bottom }} & =-\frac{P_{m 0}}{A_{c}}-\frac{P_{m 0} \cdot e}{W_{c, \text { bottom }}}+\frac{M_{G}}{W_{c, b o t t o m}} \\
\sigma_{c, \text { bottom }} & =-\frac{4951 \cdot 10^{3}}{342900}+\frac{4951 \cdot 10^{3} \cdot 389}{95.550 \cdot 10^{6}}-\frac{127.3 \cdot 10^{6}}{95.550 \cdot 10^{6}}=-33.16 \mathrm{~N} / \mathrm{mm}^{2}
\end{aligned}
$$

## B. 2 Determination minimum prestressing force

$\mathbf{t}=\infty$ : $1.35 \times$ Self weight of the girder $+1.0 \times$ prestressing force $P_{m \infty}$ $+1.35 \times$ weight of slabs $+1.35 \times$ point load)


Figure B.5: Structural model at $t=\infty$ situation 1


Figure B.6: Moment distribution at $t=\infty$ situation 1 [Nmm]


Figure B.7: Shear distribution at $t=\infty$ situation 1 [ N$]$


Figure B.8: The location of the cross sections

| $t=\infty$ situation 1 | Crosssection $A$ | CrosssectionB |
| :--- | :--- | :--- |
| Moment $[\mathrm{kNm}]$ | -1.6 | 3177.5 |
| Shear force $[\mathrm{kN}]$ | 611.3 | 550.0 |

Table B.3: Values at cross sections $t=\infty$ situation 1

## Values at cross sections

## Cross section A

$$
\sigma_{c, \text { top }}=-\frac{P_{m \infty}}{A_{c}}+\frac{P_{m \infty} \cdot e}{W_{c, \text { top }}}-\frac{M_{G}+M_{S}+M_{Q}}{W_{c, \text { top }}}
$$

$$
\begin{gathered}
\sigma_{c, \text { top }}=-\frac{0.85 \cdot 4951 \cdot 10^{3}}{342900}+\frac{4951 \cdot 10^{3} \cdot 389}{124.300 \cdot 10^{6}}-\frac{-1.6 \cdot 10^{6}}{124.300 \cdot 10^{6}}=0.84 \mathrm{~N} / \mathrm{mm}^{2} \\
\sigma_{c, \text { bottom }}=-\frac{P_{m \infty}}{A_{c}}-\frac{P_{m \infty} \cdot e}{W_{c, \text { bottom }}}+\frac{M_{G}+M_{S}+M_{Q}}{W_{c, \text { bottom }}} \\
\sigma_{c, \text { bottom }}=-\frac{0.85 \cdot 4951 \cdot 10^{3}}{342900}+\frac{4951 \cdot 10^{3} \cdot 389}{95.550 \cdot 10^{6}}-\frac{-1.6 \cdot 10^{6}}{95.550 \cdot 10^{6}}=-29.33 \mathrm{~N} / \mathrm{mm}^{2}
\end{gathered}
$$

## Cross section B

$$
\begin{gathered}
\sigma_{c, \text { top }}=-\frac{P_{m \infty}}{A_{c}}+\frac{P_{m \infty} \cdot e}{W_{c, t o p}}-\frac{M_{G}+M_{S}+M_{Q}}{W_{c, t o p}} \\
\sigma_{c, \text { top }}=-\frac{0.85 \cdot 4951 \cdot 10^{3}}{342900}+\frac{4951 \cdot 10^{3} \cdot 389}{124.300 \cdot 10^{6}}-\frac{3177.5 \cdot 10^{6}}{124.300 \cdot 10^{6}}=-24.73 \mathrm{~N} / \mathrm{mm}^{2} \\
\sigma_{c, \text { bottom }}=-\frac{P_{m \infty}}{A_{c}}-\frac{P_{m \infty} \cdot e}{W_{c, \text { bottom }}}+\frac{M_{G}+M_{S}+M_{Q}}{W_{c, \text { bottom }}} \\
\sigma_{c, \text { bottom }}=-\frac{0.85 \cdot 4951 \cdot 10^{3}}{342900}+\frac{4951 \cdot 10^{3} \cdot 389}{95.550 \cdot 10^{6}}-\frac{3177.5 \cdot 10^{6}}{95.550 \cdot 10^{6}}=3.94 \mathrm{~N} / \mathrm{mm}^{2}
\end{gathered}
$$

## B. 3 Determination maximum shear force

$\mathbf{t}=\infty: 1.35 \times$ Self weight of the girder $+1.0 \times$ prestressing force $P_{m \infty}$ $+1.35 \times$ weight of slabs $+1.35 \times$ point load)


Figure B.9: Structural model at $t=\infty$ situation 2

The purpose of this stage is to calculate the maximum shear force working on the scale model. For this calculation the ultimate limit state is required.

Therefore the load factors are included which are 1.35 . Only the value at cross section D is of interest according to NEN-EN-1992-1-1 cl. 6.2.2(6). Cross section D is located at $2 d$ from the support, which results in the highest value of $V_{E d}$ because the reduction factor is 1.0 at $2 d$.


Figure B.10: Shear distribution at $t=\infty$ situation $2[\mathrm{~N}]$


Figure B.11: The location of the cross sections

| $t=\infty$ situation 2 | Crosssection $A$ | CrosssectionD |
| :--- | :--- | :--- |
| Moment $[\mathrm{kNm}]$ | -2.1 | 2468.0 |
| Shear force $[\mathrm{kN}]$ | 1309.0 | 1280.2 |

Table B.4: Values at cross sections $t=\infty$ situation 2

## B. 4 Determination location first flexural crack in ULS

$\mathbf{t}=\infty: 1.35 \times$ Self weight of the girder $+1.0 \times$ prestressing force $P_{m \infty}$ $+1.35 \times$ weight of slabs $+1.35 \times$ point load)

The cracking moment of the bottom part of the girder is:

$$
M_{c r, b o t t o m}=\left(\frac{0.85 \cdot 4951 \cdot 10^{3}}{342900}+4.16\right) \cdot 95.550 \cdot 10^{6}=1570 \mathrm{kNm}
$$

The first flexural crack in the ultimate limit state occurs at 3050 mm according to figure B.12. The shear force at that location is depicted in figure B. 13 and is $V_{E d}=1187 \mathrm{kN}$.


Figure B.12: Location of the first flexural crack in ULS


Figure B.13: Shear force at location of the first flexural crack in ULS

## Appendix C

## Matlab method 1

```
function [Pu] = KN (c,B,d,rho,fck,fsy, Es, eta,fcube,h)
% c = diameter of the slab, given in [mm]
% r1 = major axis ellips of load area [mm]
% r2 = minor axis ellips of load area [mm]
% B = fictious diameter of the load area, given in [mm
% Ap = area of prestressing steel [mm2]
% h = height of conrete slab [mm]
% rho = reinforcement percentage
% d = average effective depth of the slab, given in [mm]
% fck = compressive strength of concrete, given in [N/mm2]
% sigmap = compressive stress in concrete due to prestressing [N/mm2]
% Fp = prestressing force [N]
% fpk = characteristic tensile strength of prestressing steel [N/mm2]
% fsy = remaining capacity of yield stress of prestressing steel [N/mm2]
% Es = modulus of elasticity of prestressing steel [N/mm2]
% eta = restraint factor
% fcube = compressive strength of concrete, measured on standard cubes,
% given in [N/mm2]
%
% The value of Pu will be return in [kN]
%
% Starting Assumption
c = 1050;
r1 = 200;
r2 = 200;
B}=(\textrm{r}1*\textrm{r}2)^0.5
Ap = 0.4425;
h = 100;
rho = Ap / h;
d = h / 2;
fck = 45;
fcube = 55;
sigmap = 2.5;
Fp = sigmap * h;
fpk = 1100;
fsy = fpk - (Fp / Ap);
Es =1.95* 10^5;
```

```
eta = 0.45;
ddelta = 1;
j = 0;
delta = 0.99*h;
while abs(ddelta/delta) > 0.01
j = j + 1;
if j > 300
break
end
Fc = 0.8 * 2/3 * fck * (h/2 - delta/4);
Ft = d * rho * fsy;
Fbmax = Fc - Ft;
Mbmax =( Ft * (2 * d - h) - Fc * (d - 13*h/16 - 3 * delta/32));
Fb}= eta * Fbmax
Mb}= eta * Mbmax
X = calX (c,B,d,Es,fsy,fcube,rho,Fb,Mb);
y = caly(c,B,d,Es,fsy,fcube,rho,Fb,X);
if B / d < 2
tasi = 0.0035 * (1 - 0.22*(B/d))*(1 + B/(2*y));
else
tasi = 0.0019 * ( 1 + B/(2*y));
end
delta1 = 1/2*tasi*(c-B);
ddelta = delta - delta1;
delta = delta1;
end
TA = calTA (c,B,d,y,X);
p1 = P1 (B,y,d,fcube,TA);
p2 = P2 (c, B,y,d, Es,fsy,X, rho, Fb);
Pu = (p1+p2)/2;
function [P1] = P1 (B,y,d,fcube,TA)
% B = diameter of the load area, given in [mm]
% y = depth of the compression zone in KN model, given in [mm]
% d = average effective depth of the slab, given in [mm]
% fcube = compressive strength of concrete, measured on standard cubes,
% given in [N/mm2]
% TA = tan(alpha)
%
% calculate ft
%
if B/d < 2
ft = 825* (0.35 + 0.3* (fcube / 150))*(1-0.22* (B / d));
else
ft = 460*(0.35 + 0.3*(fcube / 150));
end
%
% calculate falpha
%
falpha = TA * (1 - TA)/(1 + TA * TA );
%
% calculate P1
P1 = pi * (B / d) * (y / d) * (B + 2 * y) / (B + y) * ft * falpha * d * d / 1000;
function [P2] = P2 (c,B,y,d, Es, fsy,X,rho,Fb)
% c = diameter of the slab, given in [mm]
```

```
% B = diameter of the load area, given in [mm]
% y = depth of the compression zone in KN model, given in [mm]
% d = average effective depth of the slab, given in [mm]
% fsy = yield stress of reinforcing steel, given in [N/mm2]
% Es = modulus of elastisity of reinforcing steel, given in [N/mm2]
% X = boundary moment and normal force ratio
% rho = reinforcement percentage
% Fb = horizontal lateral normal force, given in [kN]
%
% Calculate kz
ky = 3*(c-B)/(2*(3*d - y));
kz = ky - 3 * X * c / (4 * (3 * d - y));
% Calculate R1 R2overBeta
if B / d< < 2
tasi = 0.0035 * (1 - 0.22*(B/d))*(1 + B/(2*y));
else
tasi = 0.0019*(1+B/(2*y));
end
rs = Es / fsy * tasi * (d - y);
C0 = B/2 + 1.8 * d;
if rs > C0
R1 = rho * fsy * d * ((rs - C0) + rs * log(c/(2 * rs)))/1000;
R2overBeta = rho * fsy * d * C0/1000;
else
R1 = rho * fsy * d * rs * log(c/(2*C0))/1000;
R2overBeta = rho * fsy * d * rs/1000;
end
%
% Calculate P2
%
P2 = 2 * pi / kz * (R1 + R2overBeta + Fb * (c/2/1000));
function [ta] = calTA (c,B,d,y,X)
ky = 3*(c-B)/(2*(3*d - y));
kz = ky - 3 * X * c / (4* (3 * d - y));
A = 1/4.7*(1+y/B)*log (c/(B+2*y));
ta = ((kz + 1)- sqrt((kz + 1)*(kz + 1) - 4 * (kz + A)*(A + 1)))/(2*(kz+A));
function [y] = caly(c,B,d,Es,fsy,fcube,rho,Fb,X)
dp = 10;
n = 0;
y = d;
while abs(dp)>0.1
n = n+1;
if n > 300
break
end
TA = calTA (c,B,d,y,X);
p1 = P1(B,y,d, fcube ,TA);
p2 = P2(c,B,y,d,Es, fsy ,X,rho ,Fb);
dp = p1 - p2;
y = y - dp/100;
end
function [X] = calX(c,B,d,Es,fsy,fcube,rho,Fb,Mb)
dX = 1;
k = 0;
```

```
X = 0;
X1 = 1;
while abs (dX) > 0.0000001
k = k + 1;
if k > 300;
break
end
y = caly(c,B,d, Es,fsy,fcube,rho,Fb,X);
TA = calTA (c,B,d,y,X);
p1 = P1 (B,y,d,fcube,TA);
p2 = P2 (c,B,y,d, Es, fsy,X, rho, Fb);
X1 = 4* pi*(Mb/((p1+p2)*1000/2));
dX = X1 - X;
X = X + dX/20;
end
```


## Appendix D

## Matlab method 2

## D. 1 Part A

```
function [Pu] = KN (c,B,d,rho,fck,fsy, Es, eta,fcube,h)
% c = diameter of the slab, given in [mm]
% r1 = major axis ellips of load area [mm
% r2 = minor axis ellips of load area [mm]
% B = fictious diameter of the load area, given in [mm]
% Ap = area of prestressing steel [mm2]
% h = height of conrete slab [mm]
% rho = reinforcement percentage
% d = average effective depth of the slab, given in [mm]
% fck = compressive strength of concrete, given in [N/mm2]
% sigmap = compressive stress in concrete due to prestressing [N/mm2]
% Fp = prestressing force [N]
% fpk = characteristic tensile strength of prestressing steel [N/mm2]
% fsy = remaining capacity of yield stress of prestressing steel [N/mm2]
% Es = modulus of elasticity of prestressing steel [N/mm2]
% eta = restraint factor
% fcube = compressive strength of concrete, measured on standard cubes,
% given in [N/mm2]
%
% The value of Pu will be return in [kN]
%
% Starting Assumption
c = 1050;
r1 = 200;
r2 = 200;
B}=(\textrm{r}1*\textrm{r}2)^0.5
Ap = 0.4425;
h = 100;
rho = Ap / h;
d = h / 2;
fck = 45;
fcube = 55;
sigmap = 2.5;
Fp = sigmap * h;
fpk = 1100;
```

```
fsy = fpk - (Fp / Ap);
Es = 1.95 * 10^ 5;
eta = 0.2;
ddelta = 1;
j = 0;
delta = 0.99 * h;
while abs(ddelta/delta) > 0.01
j = j + 1;
if j > 300
break
end
Fc = 0.8 * 2/3 * fck * (h/2 - delta/4);
Ft = d * rho * fsy;
Fbmax = Fc - Ft;
Mbmax =( Ft * (2 * d - h) - Fc * (d - 13*h/16 - 3 * delta/32));
Fb = eta * Fbmax;
Mb}=\mathrm{ eta * Mbmax;
X = calX (c,B,d,Es,fsy,fcube,rho,Fb,Mb);
y = caly(c,B,d,Es,fsy,fcube,rho,Fb,X);
if B / d< < 
tasi = 0.0035 * (1 - 0.22*(B/d))*(1 + B/(2*y));
else
tasi = 0.0019*(1+B/(2*y));
end
delta1 = 1/2*tasi*(c-B);
ddelta = delta - delta1;
delta = delta1;
end
TA = calTA (c,B,d,y,X);
p1 = P1 (B,y,d,fcube,TA);
p2 = P2 (c,B,y,d, Es, fsy ,X, rho,Fb);
Pu}=(\textrm{p}1+\textrm{p}2)/2
Fb,delta,p1,p2
function [P1] = P1 (B,y,d,fcube,TA)
% B = diameter of the load area, given in [mm]
% y = depth of the compression zone in KN model, given in [mm]
% d = average effective depth of the slab, given in [mm]
% fcube = compressive strength of concrete, measured on standard cubes,
% given in [N/mm2]
% TA = tan(alpha)
%
% calculate ft
%
if B/d< < 
ft = 825 * (0.35 + 0.3* (fcube /150))*(1 - 0.22 * (B / d));
else
ft = 460*(0.35 + 0.3*(fcube / 150));
end
%
% calculate falpha
%
falpha = TA * (1 - TA)/(1 + TA * TA);
%
% calculate P1
```

```
P1 = pi * (B / d) * (y / d) * (B + 2 * y) / (B + y) * ft * falpha * d * d / 1000;
function [P2] = P2 (c,B,y,d,Es, fsy,X,rho,Fb)
% c = diameter of the slab, given in [mm]
% B = diameter of the load area, given in [mm]
% y = depth of the compression zone in KN model, given in [mm]
% d = average effective depth of the slab, given in [mm]
% fsy = yield stress of reinforcing steel, given in [N/mm2]
% Es = modulus of elastisity of reinforcing steel, given in [N/mm2]
% X = boundary moment and normal force ratio
% rho = reinforcement percentage
% Fb = horizontal lateral normal force, given in [kN]
%
% Calculate kz
ky = 3*(c-B)/(2*(3*d - y));
kz = ky - 3 * X * c / (4 * (3 * d - y));
% Calculate R1 R2overBeta
if B / d< < 2
tasi = 0.0035*(1-0.22*(B/d))*(1 + B/(2*y));
else
tasi = 0.0019*(1+B/(2*y));
end
rs = Es / fsy * tasi * (d - y);
C0 = B/2 + 1.8 * d;
if rs > C0
R1 = rho * fsy * d * ((rs - C0) + rs * log(c/(2 * rs)))/1000;
R2overBeta = rho * fsy * d * C0/1000;
else
R1 = rho * fsy * d * rs * log(c/(2*C0))/1000;
R2overBeta = rho * fsy * d * rs/1000;
end
%
% Calculate P2
%
P2 = 2 * pi / kz * (R1 + R2overBeta + Fb * (c/2/1000));
function [ta] = calTA (c,B,d,y,X)
ky = 3*(c - B)/ (2*(3*d - y));
kz = ky - 3 * X * c / (4* (3 * d - y));
A = 1/4.7*(1+y/B)* log (c/(B+2*y));
ta = ((kz + 1)- sqrt((kz + 1)*(kz + 1) - 4 * (kz + A)*(A + 1)))/(2*(kz+A));
function [y] = caly(c,B,d,Es,fsy,fcube,rho,Fb,X)
dp = 10;
n = 0;
y = d;
while abs(dp)>0.1
n = n+1;
if n > 300
break
end
TA = calTA (c,B,d,y,X);
p1 = P1(B,y,d, fcube,TA);
p2 = P2(c,B,y,d,Es, fsy ,X,rho ,Fb);
dp = p1 - p2;
y = y - dp/100;
end
```

```
function [X] = calX(c,B,d,Es,fsy,fcube,rho,Fb,Mb)
dX = 1;
k = 0;
X = 0;
X1 = 1;
while abs (dX) > 0.0000001
k = k + 1;
if k > 300;
break
end
y = caly(c,B,d,Es,fsy,fcube,rho,Fb,X);
TA = calTA (c,B,d,y,X);
p1 = P1 (B,y,d,fcube,TA);
p2 = P2 (c,B,y,d, Es, fsy,X,rho,Fb);
X1 = 4* pi *(Mb/((p1+p2)*1000/2));
dX = X1 - X;
X = X + dX/20;
end
```


## D. 2 Part B

```
function [Pu] = KN (c, B, d, rho,fck,fsy, Es, eta,fcube,h)
% c = diameter of the slab, given in [mm]
% r1 = major axis ellips of load area [mm]
% r2 = minor axis ellips of load area [mm]
% B = fictious diameter of the load area, given in [mm
% Ap = area of prestressing steel [mm2]
% h = height of conrete slab [mm]
% rho = reinforcement percentage
% d = average effective depth of the slab, given in [mm]
% fck = compressive strength of concrete, given in [N/mm2]
% sigmap = compressive stress in concrete due to prestressing [N/mm2]
% Fp = prestressing force [N]
% fpk = characteristic tensile strength of prestressing steel [N/mm2]
% fsy = remaining capacity of yield stress of prestressing steel [N/mm2]
% Es = modulus of elasticity of prestressing steel [N/mm2]
% eta = restraint factor
% fcube = compressive strength of concrete, measured on standard cubes,
% given in [N/mm2]
%
% The value of Pu will be return in [kN]
%
% Starting Assumption
c = 1050;
r1 = 200;
r2 = 200;
B}=(\textrm{r}1*\textrm{r}2)^0.5
Ap = 0.4425;
h = 100;
rho = Ap / h;
d = h / 2;
fck = 45;
fcube = 55;
```

```
sigmap = 2.5;
Fp = sigmap * h
fpk = 1100;
fsy = fpk - (Fp / Ap)
Es = 1.95 * 10^5;
ddelta = 1;
j = 0;
delta = 0.99* h;
while abs(ddelta/delta) > 0.01
j = j + 1
if j > 300
break
end
Fc}=0.8*2/3* fck * (h/2 - delta/4)
Ft = d * rho * fsy
Fbmax = Fc - Ft;
Mbmax =(Ft * (2*d - h) - Fc * (d - 13*h/16-3* delta/32));
eta = Fp / Fbmax
Fb = eta * Fbmax
Mb}= eta * Mbmax
X = calX (c, B,d, Es, fsy ,fcube, rho ,Fb,Mb);
y = caly(c,B,d,Es,fsy,fcube,rho,Fb,X);
if B / d< < 2
tasi = 0.0035 * (1 - 0.22*(B/d))*(1 + B/(2*y));
else
tasi = 0.0019*( 1 + B/(2*y));
end
delta1 = 1/2*tasi*(c-B);
ddelta = delta - delta1;
delta = delta1;
end
TA = calTA (c,B,d,y,X);
p1 = P1 (B,y,d,fcube,TA);
p2 = P2 (c, B, y,d, Es, fsy ,X, rho ,Fb);
Pu = (p1+p2)/2;
function [P1] = P1 (B,y,d,fcube,TA)
% B = diameter of the load area, given in [mm]
% y = depth of the compression zone in KN model, given in [mm]
% d = average effective depth of the slab, given in [mm]
% fcube = compressive strength of concrete, measured on standard cubes,
% given in [N/mm2]
% TA = tan(alpha)
%
% calculate ft
%
    if B/d < 2
ft = 825*(0.35 + 0.3* (fcube /150))*(1-0.22*(B / d));
else
ft = 460* (0.35 + 0.3 * (fcube/150));
end
%
% calculate falpha
%
falpha = TA * (1 - TA)/(1 + TA * TA ;
```

```
%
% calculate P1
P1 = pi * (B / d) * (y / d) * (B + 2 * y) / (B + y) * ft * falpha * d * d / 1000;
function [P2] = P2 (c,B,y,d,Es,fsy,X,rho,Fb)
% c = diameter of the slab, given in [mm]
% B = diameter of the load area, given in [mm]
% y = depth of the compression zone in KN model, given in [mm]
% d = average effective depth of the slab, given in [mm
% fsy = yield stress of reinforcing steel, given in [N/mm2]
% Es = modulus of elastisity of reinforcing steel, given in [N/mm2]
% X = boundary moment and normal force ratio
% rho = reinforcement percentage
% Fb = horizontal lateral normal force, given in [kN]
%
% Calculate kz
ky = 3*(c - B)/(2*(3*d - y));
kz = ky - 3*X * c / (4* (3*d - y));
% Calculate R1 R2overBeta
if B / d< < 2
tasi}=0.0035*(1-0.22*(B/d))*(1 + B/(2*y))
else
tasi = 0.0019 * ( 1 + B/(2*y));
end
rs = Es / fsy * tasi * (d - y);
C0 = B/2 + 1.8 * d;
if rs > C0
R1 = rho * fsy * d * ((rs - C0) + rs * log(c/(2 * rs)))/1000;
R2overBeta = rho * fsy * d * C0/1000;
else
R1 = rho * fsy * d * rs * log (c/(2*C0))/1000;
R2overBeta = rho * fsy * d * rs/1000;
end
%
% Calculate P2
%
P2 = 2 * pi / kz * (R1 + R2overBeta + Fb * (c/2/1000));
function [ta] = calTA (c,B,d,y,X)
ky = 3*(c - B)/(2*(3*d - y));
kz = ky - 3 * X * c / (4 * (3 * d - y));
A = 1/4.7*(1+y/B)* log (c/(B+2*y));
ta = ((kz + 1)- sqrt((kz + 1)*(kz + 1) - 4* (kz + A )*(A + 1))) / (2*(kz+A));
function [y] = caly(c,B,d, Es,fsy,fcube, rho,Fb,X)
dp = 10;
n = 0;
y = d;
while abs(dp)>0.1
n = n+1;
if n > 300
break
end
TA = calTA (c,B,d,y,X);
p1 = P1(B,y ,d, fcube,TA);
p2 = P2(c,B,y,d,Es, fsy,X,rho, Fb);
dp = p1 - p2;
```

```
\(\mathrm{y}=\mathrm{y}-\mathrm{dp} / 100\);
end
function \([\mathrm{X}]=\) calX (c, B, d, Es, fsy, fcube, rho , Fb, Mb)
\(\mathrm{dX}=1\);
\(\mathrm{k}=0\);
\(\mathrm{X}=0\);
\(\mathrm{X} 1=1\);
while abs \((d X)>0.0000001\)
\(\mathrm{k}=\mathrm{k}+1\);
if k > 500;
break
end
\(y=\) caly (c, B, d, Es, fsy, fcube , rho, Fb,X);
\(\mathrm{TA}=\operatorname{calTA}(\mathrm{c}, \mathrm{B}, \mathrm{d}, \mathrm{y}, \mathrm{X})\);
\(\mathrm{p} 1=\mathrm{P} 1\) ( \(\mathrm{B}, \mathrm{y}, \mathrm{d}, \mathrm{fcube}, \mathrm{TA})\);
p2 = P2 (c, B, y, d, Es , fsy , X, rho , Fb) ;
\(\mathrm{X} 1=4 * \mathrm{pi} *(\mathrm{Mb} /((\mathrm{p} 1+\mathrm{p} 2) * 1000 / 2))\);
\(\mathrm{dX}=\mathrm{X} 1-\mathrm{X}\);
\(\mathrm{X}=\mathrm{X}+\mathrm{dX} / 20\);
end
```


## Appendix E

## DAT-file

## E. 1 2D model

```
Translated from FX+ for DIANA neutral file (version 1.2.0)
'DIRECTIONS'
    1 1.00000E+000 0.00000E +000 0.00000E +000
    2 0.00000E+000 1.00000E +000 0.00000E +000
    3 0.00000E+000 0.00000E+000 1.00000E+000
'COORDINATES'
    1 1.92500E+003-1.00000E+002 0.00000E+000
12105 4.32054E+003 -2.99419E+002 0.00000E+000
'MATERI'
    1 NAME "C45/55"
        YOUNG 3.60000E+004
        POISON 2.00000E-001
        THERMX 1.00000E-005
        DENSIT 2.40000E-009
        TOTCRK ROTATE
        TENCRV BRITTL
        TENSTR 3.80000E+000
        OOMCRV CONSTA
        COMSTR 5.30000E+001
    2 NAME "C53/65"
        YOUNG 3.78460E+004
        POISON 2.00000E-001
        THERMX 1.00000E-005
        DENSIT 2.40000E-009
    3 NAME "Endcrossbeam"
        YOUNG 1.82700E+004
        POISON 2.00000E-001
'GEOMET'
    1 NAME "Slab"
    2 NAME "Girder"
    3 NAME "Endcrossbeam"
'DATA'
    1 NAME "Slab"
'ELEMENTS'
```

```
CONNECT
    1 CQ16E 1
3863 CQ16E 10767 12100 10696 11209 10623 11800 10624 12087
    993 CT12E 2971 3279 2794 3061 2795 3485
3873 CT12E 10906 12105 10767 12090 10768 12096
MATERI
/ 1-795 / 1
/ 796-2094 / 2
/ 2095-3873 / 3
DATA
/ 1-3873 / 1
GEOMET
/ 1-795 / 1
/ 796-2094 / 2
/ 2095-3873 / 3
'LOADS'
CASE 1
ELEMEN
87 EDGE KSI2
    FORCE 2.50000E+000
    DIRECT 1
1968 EDGE ETA2
    FORCE -2.50000E+000
    DIRECT 1
888 EDGE KSI1
    FORCE -1.21000E+000
    DIRELM NORMAL
1859 EDGE KSI1
    FORCE -1.21000E+000
    DIRELM NORMAL
CASE 2
DEFORM
1326 TR 2 -1.00000E+001
DEFORM
1325 TR 2 -1.00000E+001
    'GROUPS'
ELEMEN
    399 "Auto-Mesh(Face)" / 1-795 /
    400 "Auto-Mesh(Face)-1" / 796-2094 /
    401 "Auto-Mesh(Face)-2" / 2095-3873 /
    'SUPPOR'
3092 TR 1
/ 3092 3898 5052 6291 1326 1331 1003-1007 1327-1330 1321
998-1002 1322-1325 / TR 2
/ 3092 3898 5052 6291 / TR 3
'UNITS'
FORCE N
LENGTH MM
    'END'
```


## E. 2 2D model including interfaces

```
Translated from FX+ for DIANA neutral file (version 1.2.0).
'DIRECTIONS'
    1 1.00000E+000 0.00000E +000 0.00000E +000
    2 0.00000E+000 1.00000E+000 0.00000E+000
    3 0.00000E+000 0.00000E+000 1.00000E +000
'COORDINATES'
    1 1.92500E+003 -1.00000E+002 0.00000E+000
12171 5.52850E+003 - 7.00000E+001 0.00000E+000
'MATERI'
    1 NAME "EN(RC) _C45/55"
        YOUNG 3.60000E+004
        POISON 2.00000E-001
        THERMX 1.00000E-005
        DENSIT 2.40000E-009
        TOTCRK ROTATE
        TENCRV BRITTL
        TENSTR 3.80000E+000
        OOMCRV CONSTA
        COMSTR 5.30000E+001
    2 NAME "C53/65"
        YOUNG 3.78460E+004
        POISON 2.00000E-001
        THERMX 1.00000E-005
        DENSIT 2.40000E-009
    3 NAME "Endcrossbeam"
        YOUNG 1.82700E+004
        POISON 2.00000E-001
    4 NAME "ctc"
        FRICTI
        DSTIF 1.80000E+006 7.50000E+005
        FRCVAL 1.33000E+000 6.00861E-001 6.00861E-001
        GAP
        GAPVAL 2.70000E+000
        MODE2 1
        MO2VAL 7.50000E+004
'GEOMET'
    1 NAME "ctc"
        CONFIG PSTRAI
        ZAXIS 0.00000E+000 0.00000E+000 1.00000E+000
    2 NAME "Slab"
    3 NAME "Girder"
    4 NAME "Endcrossbeam"
'DATA'
    1 NAME "Slab"
'ELEMENTS'
CONNECT
3874 CL12I 6 329 5 12111 12122 12110
3903 CL12I 1888 2211 1887 12161 12168 12156
    1 CQ16E 1 1 325 2 441 117 442 116
```

```
3863 CQ16E 10767 12100 10696 11209 10623 11800 10624 12087
    993 CT12E 2971 3279 2794 3061 2795 3485
3873 CT12E 10906 12105 10767 12090 10768 12096
MATERI
/ 1-795 / 1
/ 796-2094 / 2
/ 2095-3873 / 3
/ 3874-3903 / 4
DATA
/ 1-3903 / 1
GEOMET
/ 3874-3903 / 1
/ 1-795 / 2
/ 796-2094 / 3
/ 2095-3873 / 4
'LOADS'
CASE 1
ELEMEN
87 EDGE KSI2
    FORCE 2.50000E+000
    DIRECT 1
1968 EDGE ETA2
    FORCE 2.50000E+000
    DIRECT 1
888 EDGE KSI1
    FORCE -1.21000E+000
    DIRELM NORMAL
1859 EDGE KSI1
    FORCE -1.21000E+000
    DIRELM NORMAL
CASE 2
DEFORM
356 TR 2 -1.00000E+001
DEFORM
360 TR 2 -1.00000E+001
'GROUPS'
ELEMEN
    399 "Auto-Mesh(Face)" / 1-795 /
    400 "Auto-Mesh(Face)-1" / 796-2094 /
    401 "Auto-Mesh(Face)-2" / 2095-3873 /
    402 "Interface Element" / 3874-3903 /
'SUPPOR'
3092 TR 1
/ 3092 3898 5052 6291 356 351 28-32 352-355 361
33-37 357-360 / TR 2
/ 3092 3898 5052 6291 / TR 3
'UNITS'
FORCE N
LENGTH MM
```


## Appendix F

## DCF-file

```
*FILOS
    INITIA
*NONLIN
    EXECUT
    BEGIN EXECUT
        BEGIN LOAD
            LOADNR 2
            STEPS EXPLIC SIZES 0.001(1000)
            END LOAD
            ITERAT MAXITE 50
    END EXECUT
    BEGIN OUTPUT
        FXPLUS
        FILE "2d11"
        DISPLA TOTAL TRANSL GLOBAL
        FORCE REACTI TRANSL GLOBAL
        FRACTU
        STRAIN CRACK GREEN
        STRAIN ELASTI GREEN GLOBAL
        STRAIN PLASTI GREEN GLOBAL
        STRAIN TOTAL GREEN GLOBAL
        STRESS TOTAL CAUCHY GLOBAL
            STRESS TOTAL TRACTI LOCAL
    END OUTPUT
    TYPE GEOMET
*END
```


## Appendix G

## DYWIDAG Prestressing Steel Threadbar System

## DYWIDAG Prestressing Steel Threadbar System



## DYWIDAG Prestressing Steel Threadbar System

DYWIDAG Prestressing Steel Threadbar is a high tensile alloy steel bar which features a coarse right-hand thread over its full length. The system is proven worldwide and offers versatility in a range of applications.

Manufactured in accordance with the German Certificate of Approval (Deutsches Institut für Bautechnik), the system also offers general conformance with BS 4486 : High Tensile Steel Bars for Prestressing of Concrete. During the steel making process, the threadbars are hot rolled, quenched and tempered, followed by cold working and further tempering, to achieve the necessary performance.

DYWIDAG Prestressing Steel Threadbars, $15 \mathrm{~mm}-75 \mathrm{~mm} \varnothing$ are suitable for all static loading applications. Additionally, for post-tensioning and dynamic applications, DYWIDAG Prestressing Steel Threadbars $26.5 \mathrm{~mm}-40 \mathrm{~mm} \varnothing$, see note ( c ) below, offer a fatigue resistance in excess of 2 million load cycles over a tensile range of $630-682 \mathrm{~N} / \mathrm{mm}^{2}$ as specified in the European Technical Approval No. ETA - 05/0123 and ETAG 013. Stress relaxation when loaded to $70 \% \mathrm{fpu}$ is less than $3.5 \%$ over a 1000 hour period in accordance with BS4486.

Key features of the system are:

- Fully threaded bar - can be cut and coupled at any point.
- Coarse pitch threadform ( $\mathrm{d} / 2$ ), right-hand, with two faces ensuring the thread is self cleaning. Ideal for construction site use.
- Low relaxation steel - minimum relaxation during service life.
- Prestressing grade steel - high strength offers weight savings and reduced working diameters.


## Technical Data for Prestressing Steel Threadbar

| Nominal <br> Diameter | Steel <br> Grade | Ultimate <br> Strength <br> fpu | $\mathbf{0 . 1 \% ( a )}$ <br> Proof <br> Strength | $\mathbf{7 0 \%}(\mathbf{b})$ <br> Ultimate <br> Strength | $\mathbf{5 0 \%}$ <br> Ultimate <br> Strength | Cross <br> Sectional <br> Area | Diameter <br> Over <br> Threads | Thread <br> Pitch | Bar <br> Weight |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| $\mathbf{m m}$ | $\mathbf{N / m \mathbf { m } ^ { 2 }}$ | $\mathbf{k N}$ | $\mathbf{k N}$ | $\mathbf{k N}$ | $\mathbf{k N}$ | $\mathbf{m m}^{\mathbf{2}}$ | $\mathbf{m m}$ | $\mathbf{m m}$ | $\mathbf{k g / m}$ |
| 15 | $900 / 1100$ | 195 | 159 | 136 | 98 | 177 | 17 | 10 | 1.44 |
| 20 | $900 / 1100$ | 345 | 283 | 241 | 173 | 314 | 23 | 10 | 2.56 |
| 26.5 | $950 / 1050$ | 579 | 523 | 405 | 290 | 551 | 30 | 13 | 4.48 |
| 32 | $950 / 1050$ | 844 | 764 | 591 | 422 | 804 | 36 | 16 | 6.53 |
| 36 | $950 / 1050$ | 1069 | 967 | 748 | 535 | 1018 | 40 | 18 | 8.27 |
| 40 | $950 / 1050$ | 1320 | 1194 | 924 | 660 | 1257 | 45 | 20 | 10.21 |
| 47 | $950 / 1050$ | 1822 | 1648 | 1275 | 911 | 1735 | 52 | 21 | 14.10 |
| 57 | $835 / 1035$ | 2671 | 2155 | 1870 | 1335 | 2581 | 64 | 21 | 20.95 |
| 65 | $835 / 1035$ | 3447 | 2771 | 2413 | 1724 | 3318 | 71 | 23 | 27.10 |
| 75 | $835 / 1035$ | 4572 | 3645 | 3200 | 2286 | 4418 | 82 | 24 | 35.90 |

(a) $0.1 \%$ Proof Stress also referred to, in general terms, as Yield Strength - Ty.
(b) For geotechnical applications $75 \%$ fpu may be used for proof testing.
(c) Approval Standards: $\varnothing 26.5-47 \mathrm{~mm}$ (grade $950 / 1050 \mathrm{~N} / \mathrm{mm}^{2}$ ) ETA 05/0123 and ETAG 013. Øs 15 \& 20 mm (grade $900 / 1100 \mathrm{~N} / \mathrm{mm}^{2}$ ) formtie approvals. $\varnothing \mathrm{s} 57-75 \mathrm{~mm}$ (grade $855 / 1035 \mathrm{~N} / \mathrm{mm}^{2}$ ) system approval.

Modulus of Elasticity: $E=205,000 \mathrm{~N} / \mathrm{mm}^{2}+/-5 \%$.
Stock Lengths: $15 \mathrm{~mm}-20 \mathrm{~mm} \varnothing$ bars, $6.0 \mathrm{~m} ; 26.5 \mathrm{~mm}-75 \mathrm{~mm} \varnothing$ bars, 12.0 m . Tolerances $+/-50 \mathrm{~mm}$.
All bar diameters can be cut to length to suit customer requirements.

## Couplers for Threadbars

Couplers enable prestressing steel threadbars to be coupled or extended, reliably and efficiently. Coupler strength (for bar $\emptyset s 26.5-47 \mathrm{~mm}$ ) $=1.27 \times$ Yield Strength, which equates to $1.15 \times$ Ultimate Strength, in accordance with German Approval Certificates. Coupler strengths for other prestressing steel bar grades (bar $\varnothing \mathrm{s} 15 \& 20 \mathrm{~mm}$, and $57-75 \mathrm{~mm}$ ) exceed the published Ultimate Bar Strengths and are covered by separate approvals (see note C, Technical Data).

Precautions should be taken to ensure that the coupler remains centrally located. This can be achieved through the use of grub screws and/or a centre pin. Marking the two bars with paint or similar at half a coupler length prior to engagement provides visual confirmation of centralisation and is recommended as good working practice.

paint - visual indicator


## DYWIDAG Prestressing Steel Threadbar Accessories

| Nominal <br> Diameter | Steel <br> Grade | Recessed Plate | Domed Nut |  | Flat Plate | Lock Nut |  | Hexagonal Nut | Coupler |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Stock Size* | AF | Length | Stock Size* | AF | Length | AF | Length | Dia. | Length |  |
| mm | N/mm ${ }^{2}$ | mm | mm | mm | mm | mm | mm | mm | mm | mm | mm |
| 15 | $900 / 1100$ | $\mathrm{n} / \mathrm{a}$ | $\mathrm{n} / \mathrm{a}$ | $\mathrm{n} / \mathrm{a}$ | $120 \times 120 \times 12$ | 30 | 30 | 30 | 50 | 30 | 105 |
| 20 | $900 / 1100$ | $\mathrm{n} / \mathrm{a}$ | $\mathrm{n} / \mathrm{a}$ | $\mathrm{n} / \mathrm{a}$ | $120 \times 120 \times 20$ | 36 | 30 | 36 | 70 | 40 | 130 |
| 26.5 | $950 / 1050$ | $130 \times 130 \times 35$ | 46 | 55 | $130 \times 130 \times 35$ | 46 | 25 | 46 | 80 | 50 | 170 |
| 32 | $950 / 1050$ | $160 \times 160 \times 40$ | 55 | 70 | $160 \times 160 \times 40$ | 55 | 35 | 55 | 90 | 60 | 200 |
| 36 | $950 / 1050$ | $180 \times 180 \times 45$ | 60 | 90 | $180 \times 180 \times 45$ | 60 | 35 | 60 | 110 | 68 | 210 |
| 40 | $950 / 1050$ | $220 \times 220 \times 50$ | 70 | 115 | $220 \times 220 \times 50$ | 50 | 25 | 70 | 120 | 70 | 245 |
| 47 | $950 / 1050$ | $260 \times 260 \times 50$ | 80 | 135 | $260 \times 260 \times 50$ | 60 | 30 | 80 | 140 | 83 | 270 |
| 57 | $835 / 1035$ | n/a | n/a | n/a | $285 \times 285 \times 65$ | 90 | 35 | 90 | 120 | 95 | 240 |
| 65 | $835 / 1035$ | n/a | n/a | n/a | $325 \times 325 \times 70$ | 90 | 40 | 100 | 130 | 105 | 260 |
| 75 | $835 / 1035$ | n/a | n/a | n/a | $370 \times 370 \times 80$ | 105 | 50 | 105 | 145 | 114 | 290 |

*Anchor plates can be supplied in any size to suit customer requirements.
Nuts and Couplers

## Precautions with Prestressing Steel Threadbars

| Do | Do Not |
| :--- | :--- |
| $\checkmark$ Handle with care during loading and installation. | $\times$ Neglect or throw the bars around. |
| $\checkmark$ Cut using a high speed abrasive wheel. | $\times$ Use oxy-acetylene to cut to length. |
| $\checkmark$ Keep dry and free from corrosion. | $\times$ Allow contact with corrosive soils or atmospheres. |
| $\checkmark$ Check safe working loads. | $\times$ Subject to impact or shear loading. |
| $\checkmark$ Support bars during handling to prevent undue bending. | $\times$ Weld or allow welding sparks in contact. |
| $\checkmark$ Use only Ordinary Portland Cement for grouting. | $\times$ Store near to high induction sources. |
| $\checkmark$Take corrosion protection measures where service life <br> is greater than 2 years. | Use special cements for grouting as corrosive agents <br> may be present in the mix. |

Detailed recommendations available on request.

## Corrosion Protection

For applications involving service life in excess of 2 years, or shorter lifespans in aggressive environments, sufficient corrosion protection measures are essential. Factory pregrouted encapsulation featuring a bar grouted within a plastic sheath, in accordance with BS 8081, offers a practical and durable solution for permanent geotechnical and structural applications.

## Applications



## Stressing Equipment

| Jack Selection Chart |  |  | Bar Diameter mm |  |  |  |  |  |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
| Capacity | Stroke | Weight | 15 | 20 | 26.5 | 32 | 36 | 40 | 47 | 57 | 65 | 75 |
| 300kN | 50 mm | 21 kg | $\checkmark$ | $\checkmark$ | $\checkmark$ |  |  |  |  |  |  |  |
| 600kN | 50 mm | 36 kg |  |  | $\checkmark$ | $\checkmark$ | $\checkmark$ |  |  |  |  |  |
| 600 kN | 100 mm | 44kg |  |  | $\checkmark$ | $\checkmark$ | $\checkmark$ |  |  |  |  |  |
| 1100 kN | 50 mm | 46 kg |  |  | $\checkmark$ | $\checkmark$ | $\checkmark$ | $\checkmark$ |  |  |  |  |
| 1100 kN | 150 mm | 54 kg |  |  |  | $\checkmark$ | $\checkmark$ | $\checkmark$ | $\checkmark$ |  |  |  |
| 1500 kN | 100 mm | 140kg |  |  |  |  |  | $\checkmark$ | $\checkmark$ | $\checkmark$ |  |  |
| 3000 kN | 250 mm | 400kg |  |  |  |  |  |  |  | $\checkmark$ | $\checkmark$ | $\checkmark$ |
| 4000KN | 250mm | 650kg |  |  |  |  |  |  |  |  | $\checkmark$ | $\checkmark$ |

## Stressing Dimensions

| Nominal Diameter | Steel Grade | Dimension A | Dimension B |  |
| :---: | :---: | :---: | :---: | :---: |
| mm | N/mm ${ }^{2}$ | mm | mm |  |
| 15 | 900/1100 | 50 | 50 |  |
| 20 | 900/1100 | 70 | 55 |  |
| 26.5 | 950/1050 | 40 | 60 |  |
| 32 | 950/1050 | 50 | 70 |  |
| 36 | 950/1050 | 70 | 85 |  |
| 40 | 950/1050 | 90 | 125 | Dimension $\mathbf{B}$ is the minimum threadbar projection required for stressing. |
| 47 | 950/1050 | 105 | 135 |  |
| 57 | 835/1035 | 120 | 140 |  |
| 65 | 835/1035 | 130 | 150 |  |
| 75 | 835/1035 | 145 | 170 |  |

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