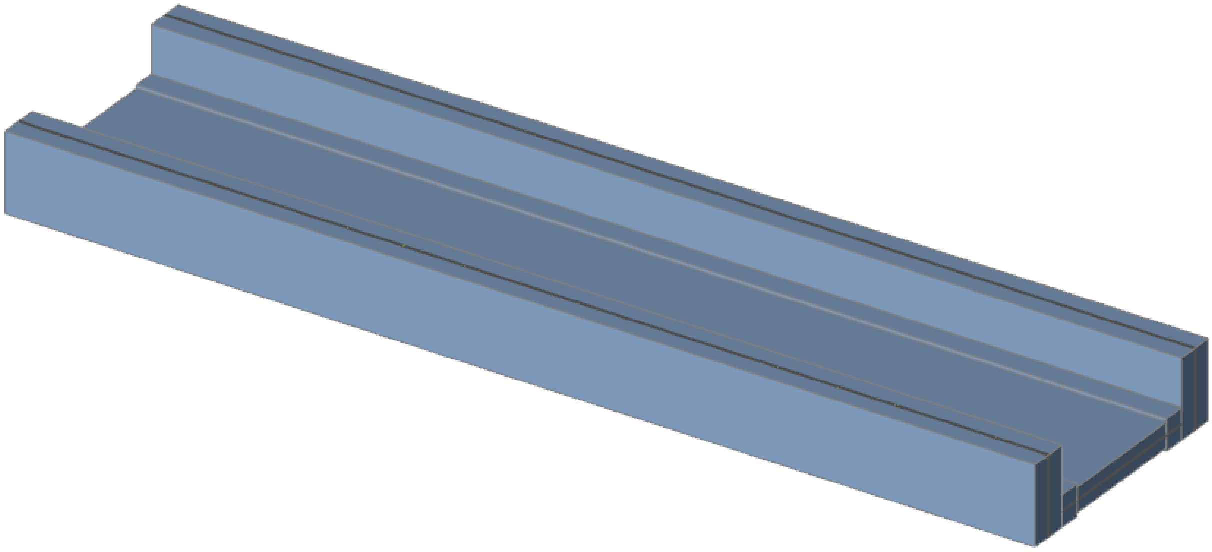
Shear and torsion in a prestressed through railway bridge

A comparison between the analytical solution and SCIA



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Preface

This thesis is written as partial fulfilment of the master programme Structural Engineering at Delft University of Technology. The research is facilitated by Witteveen+Bos and supported by Delft University of Technology.

With the arrival of the Eurocode the calculations for concrete shear resistance have become increasingly conservative. This imposes questions on organisations like Prorail, who manage the railway infrastructure in the Netherlands, whether their existing structures still meet the current regulations. In order to get a better insight into this problem, the shear resistance of two prestressed through railway bridges is reassessed by means of a hand calculation.
The reassessment could be a typical assignment for an engineering firm like Witteveen+Bos, but because hand calculations are too time consuming a finite element programme is used in practice. Because nowadays we completely rely on these programs, a comparison is made between the analytical solution and SCIA engineer.

I would like to thank my graduation committee for their help, guidance and feedback. Firstly I would like to thank prof. dr. ir. D.A. Hordijk for his enthusiasm and feedback. Secondly I want to express my gratitude towards dr. ir. C. van der Veen for his regular advice, thoroughness and insights. Thirdly I want to thank dr. ir. M.A.N. Hendriks for his time, knowledge and suggestions.
Furthermore I would like to thank Witteveen+Bos for providing their facilities and expertise. And last but not least, I would like to expresses my gratitude towards my daily supervisor, ir. E. Jongstra, for his guidance, time and support.

Bob Wouterlood

Delft, 22th of June 2018

Summary

With the arrival of the Eurocode, the calculations regarding shear resistance, have become increasingly conservative compared to former concrete standards. For example in *Voorschriften Beton 1974 (VB 74),* a former concrete standard, a shear resistance combination of concrete, stirrups and prestress was allowed. Whereas the Eurocode assumes the shear resistance of the concrete is zero if it's standalone contribution is insufficient. Meaning that once stirrups are required, the total applied shear stress is controlled by the stirrups.
Prorail is the party in the Netherlands which is responsible for the construction and maintenance of the railway infrastructure. The change in regulations results in concerns for parties like Prorail and in particular to the shear resistance of concrete railway bridges constructed according to the VB 74.

A through railway bridge consists of a relatively thin floor and two prestressed girders. Whenever a train drives over the floor, a great deal of the loading is spread in transverse direction, causing large shear forces and torsion in the two prestressed girders. Because this combination can be critical for the shear resistance, two prestressed through railway bridges, constructed according to the VB 74, are investigated in this master thesis.
It is verified with hand calculations whether or not these existing structures can guarantee structural safety by considering three shear resistance checks; the risk of shear tension failure, capacity of the stirrups and resistance against fatigue. Ultimately it is concluded, that the largest unity check is 1,01 and that both bridges can guarantee structural safety regarding shear resistance.

The reassessment of an through bridge, is an typical assignment for engineering firm such as Witteveen+Bos. But because hand calculations are too time consuming, the design loads are determined with SCIA Engineer (FEA program). However in the earlier days FEA programs were not available and torsion in the girders was derived from a set of differential equations (analytical solution). Because structural engineers from today completely rely on programs like SCIA, a comparison is drawn up between SCIA and the analytical solution for torsion in the girder.

The plate, beam model 1A and 1B are the three types of models available in SCIA to model a through bridge. The plate model consists of a 2D-floor and 2D-girder, where the beam models form a combination of a 2D-floor and 1D-girder. But the plate and beam model 1B have in common that rigid connections are applied every ¼ meter between the girder and floor.
The analytical solution is derived with the assumption that the bridge is divided into strips with a length of 1,0 meter, which is implemented in the plate and beam models by reducing the E-modulus of the floor to roughly a third (cracked floor). For the governing load combination, this leads to values for torsion, which remain 10-15%, 40% and 10% behind the analytical solution for respectively the plate and beam model 1A and 1B. The large deviation of beam model 1A is remarkable and can be explained from the fact that no rigid connections between the girder and floor are applied, resulting in a loss of bending and torsional stiffness of the girder.

To conclude, the analytical solution is based on a number of assumptions, like no load distribution of the floor in longitudinal direction. In reality loads will be as well distributed in longitudinal as transverse direction and the analytical solution therefore needs to be considered as an safe upper limit.

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1 Introduction

1.1 Background

The Dutch Department of Waterways and Public Works (Rijkswaterstaat) is responsible for the maintenance and construction of traffic infrastructure in the Netherlands. A large number of Rijkswaterstaat's concrete bridges were built between 1960 and 1980. The typical concrete standards used at the time to construct these bridges are the *Gewapend Betonvoorschriften (GBV 1962)* and the *Voorschriften Beton (VB 1974)*. Compared to the current concrete standard (the Eurocode), the GBV and the VB are less conservative and strict.

A similar conclusion was drawn by recent research conducted by TNO, TU Delft and Rijkswaterstaat (1). This research showed that concrete traffic bridges, constructed before 1974, may have insufficient shear resistance due to a difference in load models and shear resistance calculations adopted at that time. Since shear failure is a brittle mechanism which shows no warning up to the point of failure, a solution needed to be found. As a result a number of investigations were conducted on existing structures and the most important findings were bundled by Rijkswaterstaat in the form of a guideline. This guideline, *Richtlijnen Beoordeling Kunstwerken* (2), is an addition to the Eurocode specifying a set of rules that is applicable to Rijkswaterstaat's existing structures.

Prorail is the party in the Netherlands that is responsible for the maintenance and construction of railway infrastructure. With the Eurocode shifting towards more conservative regulations, Prorail may expect issues with their existing railway structures. In order to investigate this effect, two prestressed through railway bridges are analysed in this master thesis.

1.2 Research objective

As mentioned earlier, the load models and the shear resistance calculations have become more conservative with the arrival of the Eurocode. Provided that the critical stresses in a through bridge results from a combination of shear, torsion and bending, these type of structures may have insufficient shear resistance. The loads are determined analytically and the structural safety is verified based on a number of Eurocode checks (Figure 1).

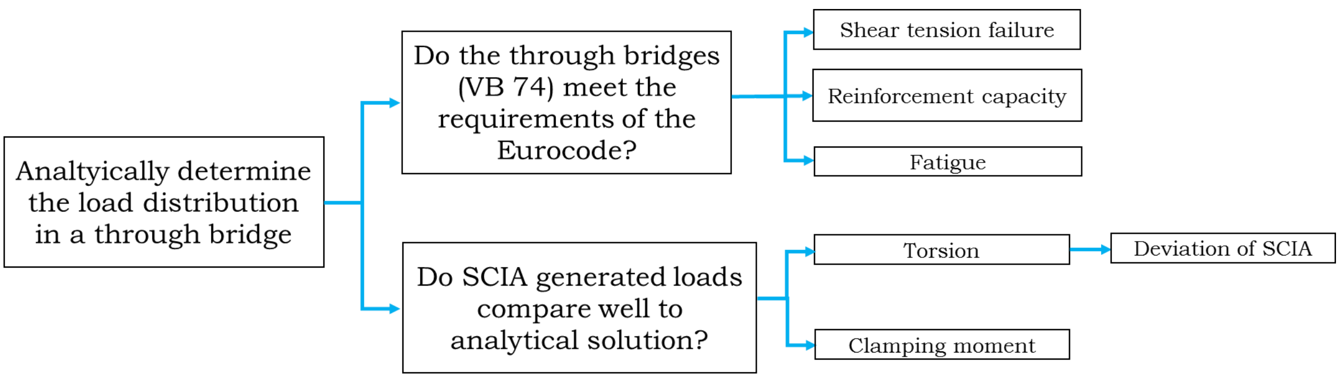


Figure 1: Overview of the research process

The reassessment of an existing through railway bridge could be a typical assignment for an engineering firms such as Witteveen+Bos. Due to the availability of finite element models (FEM) and computers with necessary computational power, it is expected a structural engineering will execute the reassessment using a FEA program (*SCIA engineer* in the case of Witteveen+Bos).
In the earlier days these models didn't exist yet and through bridges were designed with hand calculations. The difficulty back then lied within in establishing torsion in the prestressed girders, for which a number of differential equations needed to be derived (analytical solution). The second part of this research will therefore focus on the comparison between SCIA and the analytical solution with respect to torsion in the girder. The ultimate goal is to establish the deviation between SCIA and the analytical solution (w.r.t. to torsion) for the governing load combination.

1.3 Assumptions

Since the shear resistance of the through bridge is assumed to be governing, the checks performed on the bridge will be limited to shear related checks only. Concludingly the risk of shear tension failure, the capacity of the stirrups and the resistance against fatigue are assumed to be the governing checks in the reassessment of the bridge.

The comparison between SCIA and the analytical solution only focuses on torsion and clamping moments in the girder. The explanation for this is two folded. The first reason for this is, as mentioned above, the fact that torsion and the corresponding clamping moment are important design loads in the through bridge. The second reason is that a number of differential equations, for torsion and the clamping moment, are derived which are based on a number of assumptions. It will be interesting to compare with SCIA, whether these assumptions are realistic or too conservative. An overview of these assumptions can be found in paragraph 3.3.2.

1.4 Research questions

1. What is the shear resistance of a fully prestressed through railway bridge, designed according to the VB 74, when applying the load models and the calculation procedures of the Eurocode?

   a. What are the differences in applied loads and combinations?
   b. What is the risk of shear tension failure in the girder?
   c. What is the shear capacity of the applied stirrups?
   d. What is the concrete and reinforcement resistance against fatigue?

2. What is the difference between a SCIA model and the analytical solution with respect to torsion in the girder?

   a. What type of models in SCIA are possible for a through bridge?
   b. Which assumptions (of the analytical solution) have to be taken into account when modelling?
   c. What is the deviation between SCIA and the analytical solution with respect to torsion in the girder?

1.5 Methodology

The research questions will be answered on the basis of a literature study, an analytical solution and three SCIA models.

The literature study discusses the following topics:

- The difference between the load models of the Eurocode and VOSB.
- The difference between load combinations at ultimate and serviceability limit state in the Eurocode and the VB 74.
- The manner in which the Eurocode and the VB 74 deal with shear resistance calculations.
- The mechanisms of shear failure.

The analytical solution deals with:

- An approach to determine bending, shear and torsion in the through girder analytically.
- The calculation of the maximum principal stress, in order to verify whether or not shear tension failure will occur.
- A reinforcement capacity calculation to verify if an adequate number of stirrups is applied in the girder.
- A fatigue resistance calculation on concrete, reinforcement and prestress steel.

SCIA focuses on the following topics:

- The different type of models that can be used to model a through bridge.
- Processing the assumptions of the analytical solution into the SCIA models.
- Comparing SCIA and the analytical solution for torsion and the clamping moment in the girder.
- Establishing the deviation (for torsion), due to the governing load combination, between SCIA and the analytical solution.

1.6 Case study

The case study considers two simply supported single track fully prestressed through bridges which are positioned below the emplacement of train station the Hague HS. They were built in 1994 in order to construct a tram tunnel which passes underneath the train station. In the Netherlands the railway track is classified with a geocode and an accompanying kilometre set. The bridges are classified as geocode 536 and lie on km 61.4.

This case study considers one short and one long bridge, which from now on are indicated as bridge A and B. Even though both bridges are simply supported, they are constructed with a small cantilever of 1,0 meter at both sides. The most important properties were found in the design reports (3) and a summary is presented in Table 1.

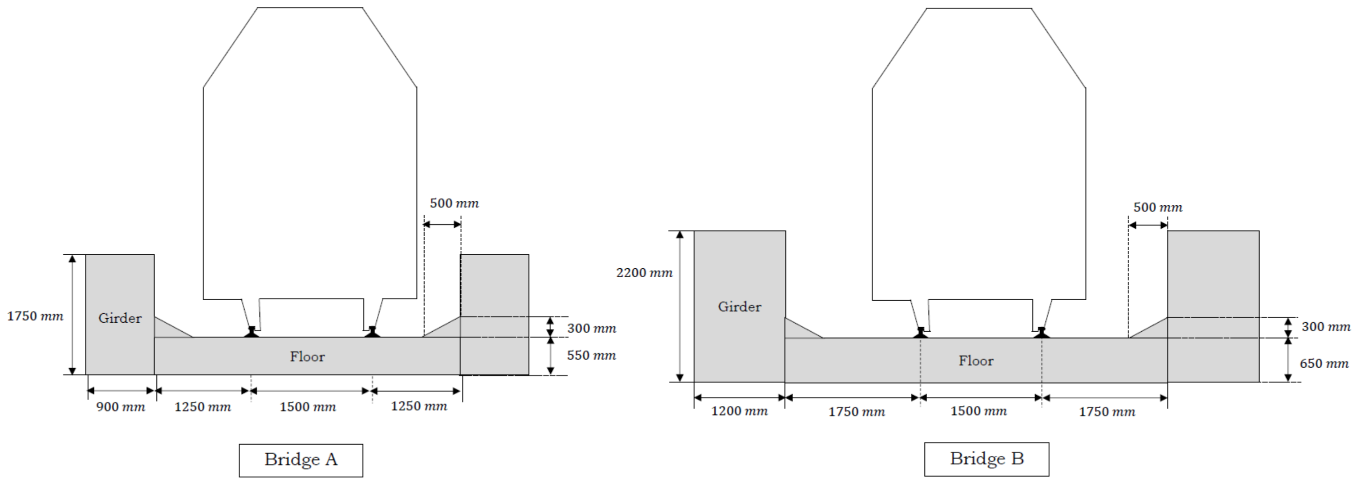


Figure 2: Cross section of bridge A (left) and bridge B (right)

Table 1: Properties of concrete, reinforcement and prestress steel of bridge A and B

Property	Bridge A	Bridge B
Support system	Simply supported	Simply supported
Span	21 m	31,5 m
Total width	5,8 m	7,4 m
Total height	1,75 m	2,2 m
Reinforced concrete (floor)	B35 (C25/30)	B35 (C25/30)
Prestressed concrete (girder)	B45 (C35/45)	B45 (C35/45)
Reinforcement	FeB500 (S435)	FeB500 (S435)
Longitudinal Prestress system	Type: Cona Multi Strands: Ø12,9 Quality: FeP1860 Tendon: 19 strands Total: 6 tendons A_p: 1900 mm²	Type: Cona Multi Strands: Ø15,7 Quality: FeP1770 Tendon: 18 strands Total: 8 tendons A_p: 2700 mm²
Transverse Prestress system (near the supports)	Type: Dywidag Diameter: Ø36 Quality: FeP1230 Tendon: 4	Type: Dywidag Diameter: Ø36 Quality: FeP1230 Tendon: 6

2 Literature study

2.1 Through bridge

A single track through bridges consist of two prestressed girders which span in longitudinal direction. The bottom side of the girders are connected with a floor, on which a railway track is mounted. The floor transfers the majority of the applied loading in the transverse direction towards the girders. The girders then transfer the load towards the supports.

Usually the girders are prestressed in longitudinal direction and thereby indirectly prestress the floor in longitudinal direction. High anchor forces at the end of the bridge can lead to large local tensile splitting forces. But bridge A and B are therefore equipped with splitting reinforcement and transverse prestressing near the supports. Besides prestress the girders are equipped with stirrups and longitudinal reinforcement in order to transfer shear forces and bending moments. The floor is subjected to significantly smaller loads and is therefore equipped with longitudinal reinforcement only, which spans in as well transverse as longitudinal direction.

The usage of a through bridge has a number of advantages (4):

- The bridge has a very limited construction height which is independent of the span. The main reason for this is the low-lying floor. This has a major advantage, namely the elevation of the railway track remains limited, meaning the length of the entrance and exists towards the bridge can be reduced.
- The supports can be applied asymmetrically, this enables one to realise a difficult junction with infrastructure below.
- The main girders have a noise protecting ability, making the bridge suitable for rural areas.
- When the bridge is subjected to an upward bending moment, the floor will function as a compression flange, making the structure suitable for continuous spans.

The usage of a through bridge also has a number of disadvantages (4):

- Because most of the concrete is in the lower section, the concrete will not be used to an optimum when the bridge is subjected to a downward bending moment at midspan.
- End-cross members can be applied to increase the effective width of the bridge near the supports. However, these end-cross members also cause additional torsion in the girders. As mentioned earlier the combination of shear, torsion and bending can be very critical for a through bridge.

2.2 Shear failure mechanisms

Since the shear resistance calculations, in the VB 74 and the Eurocode, are based on failure mechanisms, it is important to have a basic understanding of them. This paragraph will elaborate on the different types of shear failure.

2.2.1 Flexural shear failure

When a reinforced concrete member is subjected to a shear force and a bending moment, a flexural-shear crack can occur. Initially a bending moment causes vertical cracks in the cross-section which under the influence of a shear force start to grow under a certain angle. Four different zones can be distinguished in this type of failure (Figure 3).

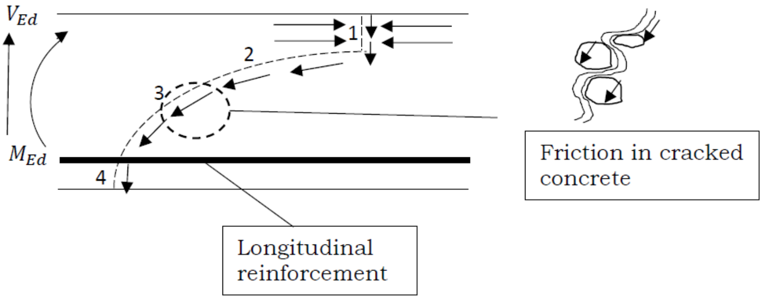


Figure 3: Flexural-shear crack

1. **Uncracked compression zone:** Due to the high compressive stresses this zone is capable of transferring large shear stresses. Hence zone 1 has a large contribution to the total shear resistance.
2. **Small crack width:** The upper part of the curved crack (close to the uncracked section) shows little deformation, which enables the concrete to resist relatively large tensile stresses.
3. **Frictional forces in the cracked concrete:** The development of a flexural-shear crack leads to a parallel shift of the cracked faces, which causes frictional stresses. This mechanism contributes to the shear resistance.
4. **Stirrups and longitudinal reinforcement:** Due to the parallel shift of these faces, vertical displacement leads to tension in the stirrups and activation of the dowel effect in the longitudinal reinforcement.

When the cracks keep on growing, the stirrups will start to yield, the height of the uncracked compression zone decreases and vertical equilibrium can no longer be guaranteed. Ultimately a brittle failure mechanism occurs. (5)

2.2.2 Shear tension failure

In a section without shear reinforcement or with limited bending stresses (e.g. a prestressed section), shear tension failure is likely to occur. In order to explain this an element with a normal and shear stresses acting on it needs to be considered. The combined effect of these stresses can be expressed by a tensile and compressive principal stress. When the tensile principal stress exceeds the tensile strength of the concrete, a shear tension crack will develop. (6)

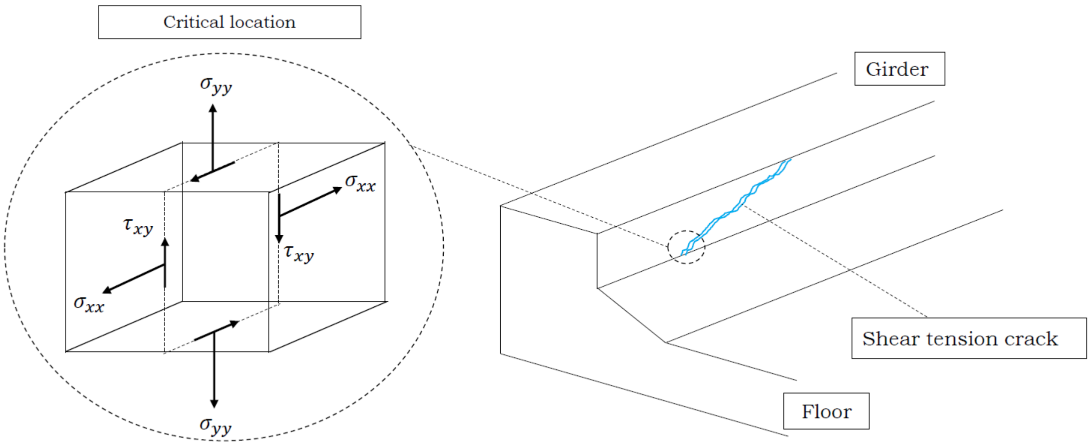


Figure 4: Shear tension crack in the through girder

2.2.3 Shear compression failure

A concrete beam can be strengthened by applying stirrups. In case the beam is loaded with a shear force, diagonal cracks will develop and intersect with the applied stirrups. This changes the internal force distribution, which can be approximated by the truss analogy.

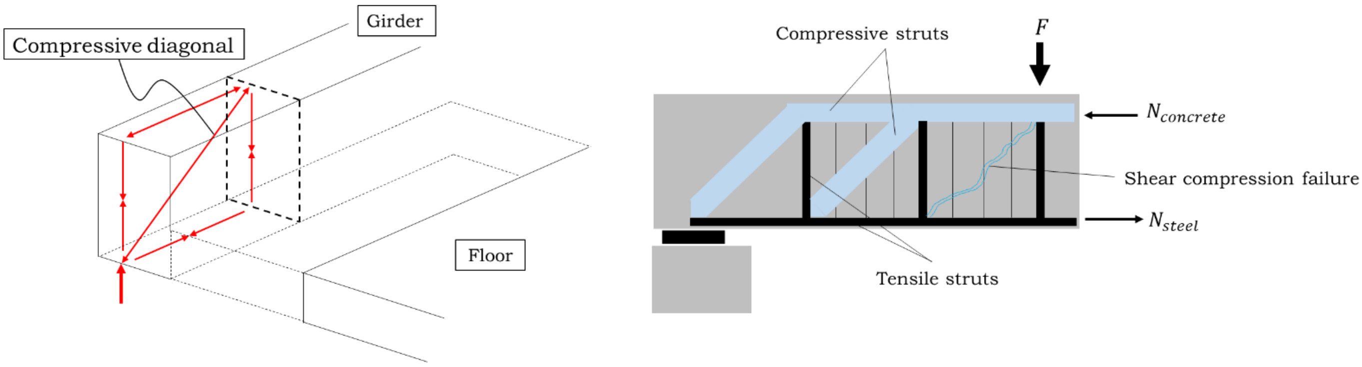


Figure 5: Truss analogy in the girder (left) and shear compression failure (right)

The analogy assumes an equilibrium of compressive and tensile struts. The stirrups and longitudinal reinforcement become tensile struts, whereas the diagonal and the compression zone become compressive struts. Before the stirrups get to yield, the compressive capacity of the diagonal is reached, causing a brittle failure mechanism. In the through bridge, the compressive diagonal is formed in the girder which means the maximum shear loading on the bridge is governed by the dimensions of the girder. This failure mechanism forms the upper limit of the shear capacity and definitely needs to be checked in the reassessment.

2.3 Shear resistance calculations

One of the main reasons for this research topic was the change in concrete regulations, in particular with respect to shear resistance. This paragraph will elaborate on the differences in shear resistance calculations between the Eurocode and VB 74 for prestressed structures.

2.3.1 VB 74

According to the VB 74 the total shear stress in a section is computed using a combination of torsion and shear (equation [1]). The shear force is reduced by the vertical component of the prestress and torsion is divided by the torsional constant $(\psi/(b^2h)$ in order to get a stress.

$$\tau_{Ed} = \frac{V_{Ed} - P_{\infty} * \sin \propto}{b * h} + \psi \frac{T_{Ed}}{b^2 h} \qquad [1]$$

The total shear resistance depends on a combination of concrete, prestress and stirrups (7). The contribution of each depends on whether the structure is subjected to fatigue. The shear resistance calculations are performed at ultimate limit state (Table 2).

Table 2: Overview shear resistance calculations for prestressed structures (VB 74)

Structure	Shear resistance	
Prestressed element without shear reinforcement	$\boldsymbol{\tau_{Ed} \leq \tau_1}$	
	No Fatigue	Fatigue
	$\tau_1 = 0{,}5 * f_b + 0{,}15 * \frac{P_\infty}{A_b}$	$\tau_1 = 0{,}15 * \frac{P_\infty}{A_b}$
Prestressed element with shear reinforcement	$\boldsymbol{\tau_{Ed} \leq \tau_1 + \tau_s}$	
	No Fatigue	Fatigue
	$\tau_1 = \tau_s + 0{,}5 * f_b + 0{,}15 * \frac{P_\infty}{A_b}$	$\tau_1 = \tau_s + 0{,}15 * \frac{P_\infty}{A_b}$
Maximum shear resistance (capacity compressive diagonal)	$\boldsymbol{\tau_{Ed} \leq \tau_2}$	
	$\tau_2 = 0{,}25 * f_{bk} \leq 9{,}0\ N/mm^2$	

Where:

- $f_b =$ Characteristic tensile strength concrete
- $f_{bk} =$ Characteristic compressive strength concrete
- $\tau_s =$ Shear resistance stirrups
- $P_\infty/A_b =$ Compressive prestress

Notable is the resistance of concrete, which is equal to half the tensile strength. It may be taken into account whenever a structure is not subjected to fatigue. Besides this an additional 15% of the prestress is added to the shear resistance, independently of a structure being loaded in fatigue. The ultimate shear resistance is limited by the capacity of the concrete diagonal, which in case of the VB 74 is taken as 25% of the characteristic compressive strength.

The check on shear tension failure is performed at serviceability limit state. This type of failure mechanism leads to pure tension in the concrete and no tension in the stirrups. As a result the maximum principal stress is checked against the concrete shear resistance ($0{,}5 * f_b$).

An overview of the checks and their corresponding limit states is provided in paragraph 2.4.3 and 2.4.4 for the VB 74 and Eurocode respectively.

2.3.2 Eurocode

In contradiction to the VB 74 the Eurocode does not allow a combination of concrete and steel in the resistance calculations. Initially the shear resistance of the concrete is considered. As long as loads do not exceed this capacity, only minimum shear reinforcement is required. It is even allowed, for structures with bending stresses smaller than the tensile design strength (f_{ctd}), to determine the concrete shear resistance using the shear tension failure mechanism. For structures with larger bending stresses, the flexural shear failure mechanism is governing.
The resistance against torsion is based on a thin walled beam, where the maximum shear stress is equal to the tensile design strength (f_{ctd}). The Eurocode converts this maximum stress, by using the cross-sectional area and an internal lever, into a maximum torsional moment.

For the case where concrete shear capacity is insufficient, stirrups need to be applied. The capacity of all the stirrups intersected by the compressive diagonal should be in balance with the applied shear force and torsion. Ultimately the maximum resistance is determined by the capacity of the compressive diagonal.

Table 3: Overview shear resistance calculations for prestressed structures (Eurocode)

<table>
<tr><th>Structure</th><th colspan="2">Total shear resistance</th></tr>
<tr><td rowspan="5">Prestressed element without shear reinforcement</td><td>Shear</td><td>Torsion</td></tr>
<tr><td>$\sigma_b \leq f_{ctk;0,05}/\gamma_c$</td><td rowspan="4">$T_{Rd,c} = 2t_{ef}f_{ctd}A_k$</td></tr>
<tr><td>Shear tension failure:
$V_{Rd,c} = \frac{I * b}{S}\sqrt{(f_{ctd}{}^2 + \alpha_l \sigma_{cp} f_{ctd})}$</td></tr>
<tr><td>$\sigma_b > f_{ctk;0,05}/\gamma_c$</td></tr>
<tr><td>Flexural shear failure:
$V_{Rd,c} = \left(C_{Rd,c}k(100\rho_l f_{ck})^{\frac{1}{3}} + 0{,}15\sigma_{cp}\right)bd$
$V_{Rd,min} = [v_{min} + 0{,}15\sigma_{cp}]bd$</td></tr>
<tr><td rowspan="2">Prestressed element with shear reinforcement</td><td colspan="2">$\frac{V_{Ed}}{V_{Rd,c}} + \frac{T_{Ed}}{T_{Rd,c}} =$</td></tr>
<tr><td>$\leq 1{,}0$: Min. Reinforcement
$> 1{,}0$: V_{Ed} transferred by stirrups</td><td>$\leq 1{,}0$: Min. Reinforcement
$> 1{,}0$: T_{Ed} transferred by stirrups</td></tr>
<tr><td rowspan="2">Maximum shear resistance (capacity compressive diagonal)</td><td>$V_{Rd,max} = \frac{\alpha_{cw}\, b_w z v_1 f_{cd}}{\cot\theta + \tan\theta}$</td><td>$T_{Rd,max} = 2v\alpha_{cw}f_{cd}A_k t_{ef}\sin\theta\cos\theta$</td></tr>
<tr><td colspan="2">$\frac{V_{Ed}}{V_{Rd,max}} + \frac{T_{Ed}}{T_{Rd,max}} \leq 1{,}0$</td></tr>
</table>

Note: The contribution of prestress is taken into account for as well the shear tension as the flexural-shear failure mechanism.

Where:

$\sigma_{cp} =$ Compressive prestress
$f_{ctd} =$ Concrete tensile design strength
$V_{Rd,max} =$ Maximum shear capacity
$T_{Rd,max} =$ Maximum torsional capacity

In the Eurocode the check on shear tension failure is performed at ultimate limit state. As mentioned before this type of failure purely leads to tension in the concrete. By rewriting the equation in Table 3, it can be concluded that the maximum tensile principal stress cannot grow larger than the concrete tensile design strength (paragraph 4.4).

2.4 Loads

This paragraph elaborates on the different type of load models for train traffic considered by the VB 74 (which refers to the VOSB 1963) and the Eurocode. Besides load models an overview of load factors and combinations is presented.

2.4.1 Load models VOSB 1963

The VOSB (8) defines the load models for railway traffic. The VOSB 150 is a combination of an distributed load of 80 kN per meter and three grouped 150 kN concentrated loads with 1,5 m spacings. This group of concentrated loads is repeated with an interval of at least 17 meters. Respectively the concentrated and distributed loads represent the axle loads of a locomotive and loaded carriages. In contrast to the VOSB 150, the VOBS 250 and 270 only consider axle loads.

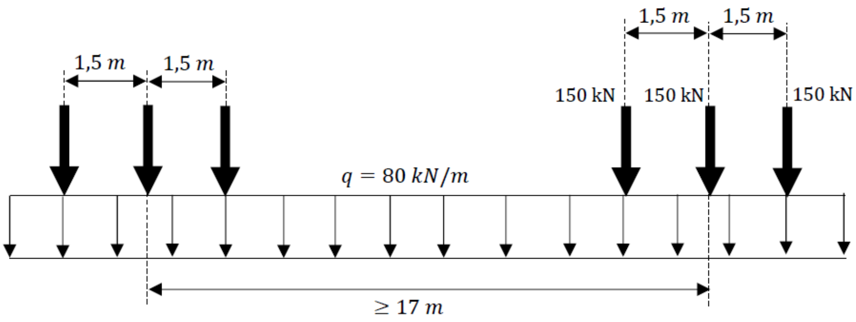


Figure 6: Load model VOSB 150

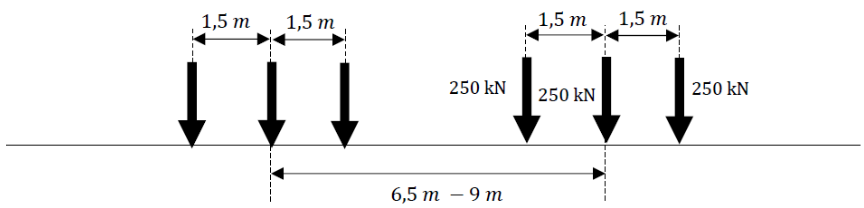


Figure 7: Load model VOSB 250

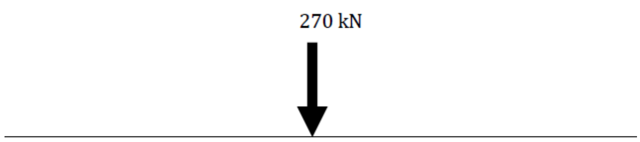


Figure 8: Load model VOSB 270

The characteristic values need to be multiplied with a dynamic factor, which takes into account the dynamic loading due to passing traffic. The factor is determined with equation [2], which originates from NS-guideline 1008 (9), and is different from the dynamic factor in the Eurocode. For bridge A and B a value of respectively 1,195 and 1,155 is found.

$$1{,}10 \leq 1 + \frac{10 * (1{,}2 - d)}{20 + L} \leq 1{,}50 \qquad [2]$$

Where:

L = Span of the bridge between the supports
d = Height of the ballast bed measured from the bottom of the sleeper

According to the design reports the VOSB 150 is the governing load model and the other two load models do not need to be considered. By using influence lines, the most unfavourable position of VOSB 150 on the bridge is established.

2.4.2 Load models Eurocode

Eurocode 1 (10) specifies three different load models for railway traffic that can be applied on a bridge:

- Load model 71: Represents loading on the bridge due to normal railway traffic.
- SW/0: Represents normal traffic on a bridge with a continuous span.
- SW/2: Represents heavy railway traffic on a bridge.

Because the two considered bridges are simply supported, only load model 71 and SW/2 are applicable.

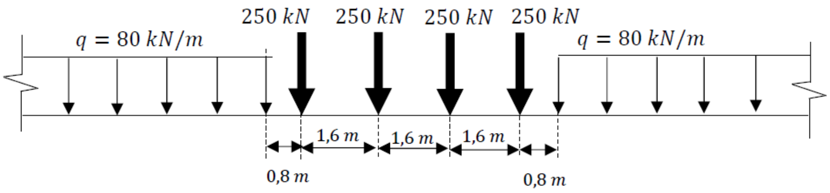


Figure 9: Load model 71

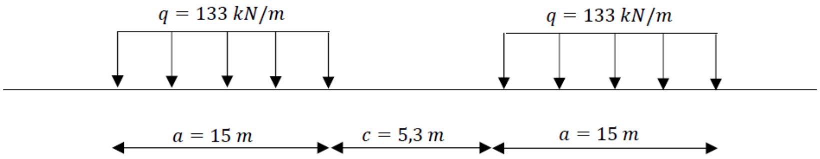


Figure 10: Load model SW/0

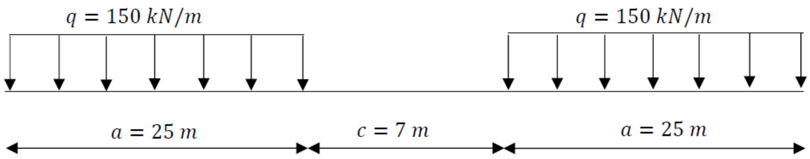


Figure 11: Load model SW/2

The dynamic factor for a carefully maintained railway track is determined with equation [3]. For bridge A and B a dynamic factor of respectively 1,136 and 1,077 is found. Relatively seen this leads to a decrease in dynamic loading of 5-7% compared to the VB 74.

$$\Phi = \frac{1{,}44}{\sqrt{L_\Phi - 0{,}2}} - 0{,}82 \qquad [3]$$

Where:

$L_\Phi = length\ determined\ from\ table\ 6.2\ in\ Eurocode\ 1$

Besides dynamic loading, load model 71 and SW/0 are multiplied with a factor α. This factor takes into account the traffic that is either heavier or lighter than normal traffic. A value of 1,21 for α is specified in the Dutch National Annex to Eurocode 1 (11).

The finite element program SCIA Engineer is used to find the most unfavourable position of LM71 and SW/2 on both bridges (Appendix C). The influence lines for bending, shear and torsion are presented in Figure 12 for a section at 0,8d. The most unfavourable position of LM71 and SW/2 on bridge A are as follows:

- LM71: Positioning the group of concentrated loads at the start of the bridge generates maximum torsion. The distributed load of 80 kN/m is then best positioned between the end of the concentrated loads and the other support, in order to maximize shear and bending.
- SW/2: The minimum length of this load model is 25 meters and is therefore best positioned at the start of the bridge to maximize torsion and until the end support to maximize bending and shear.

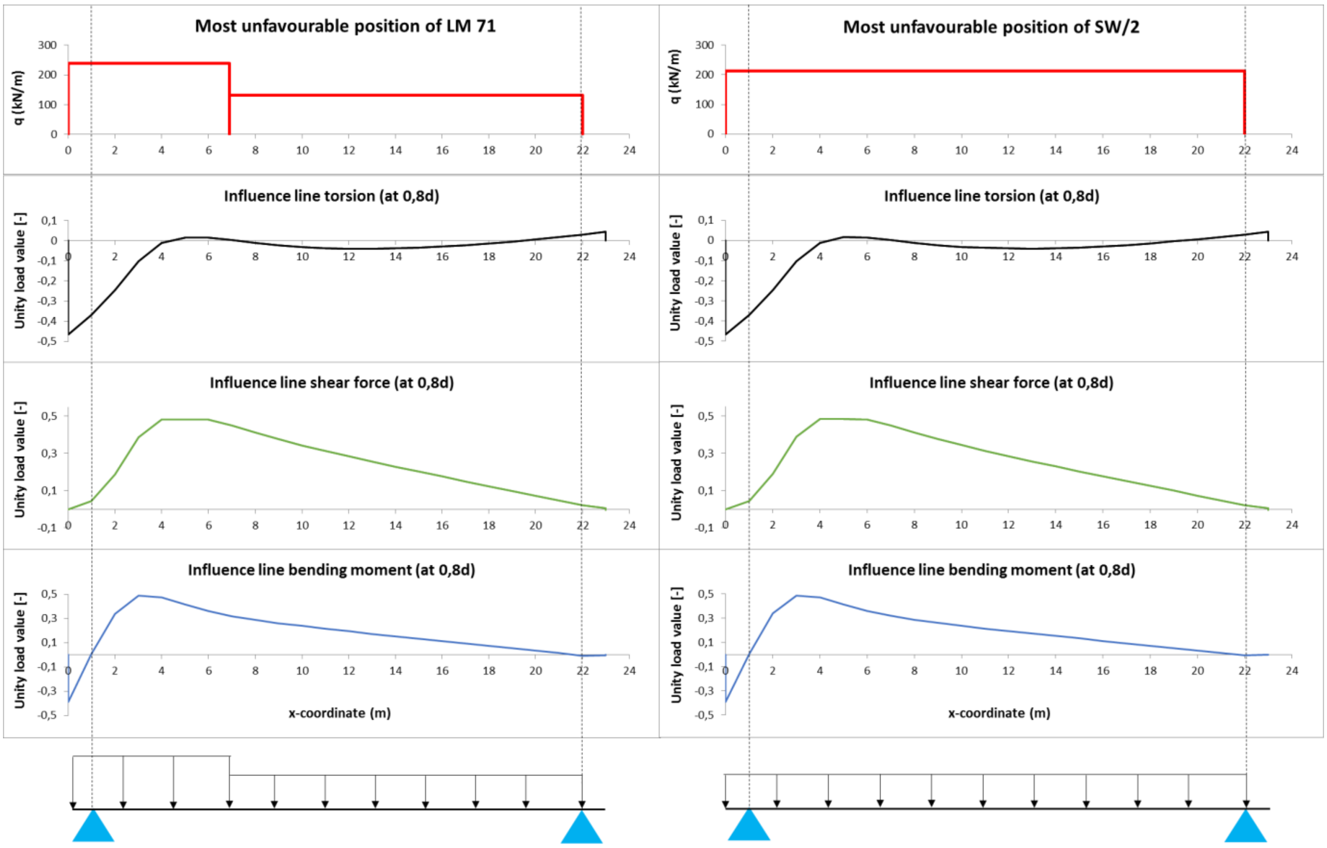


Figure 12: Bridge A: Most unfavourable position of LM71 (left) and SW/2 (right) at a section 0,8d

2.4.3 Load combinations in the VB 74

Part A of the VB 74 (12) defines the load combinations at serviceability and ultimate limit state:

$$Q_{d,tot} = \gamma * Q_{k,tot} = 1{,}0 * Q_{k,tot}$$ Serviceability limit state

$$Q_{d,tot} = \gamma * Q_{k,tot} = 1{,}7 * Q_{k,tot}$$ Ultimate limit state

The design load ($Q_{d,tot}$) is a product of the safety factor and a combination of all loads ($Q_{k,tot}$). The value for the safety factor at serviceability and ultimate limit state is respectively 1,0 and 1,7. In addition to the load combination it is important to know which limit state is considered for the performed checks. An overview of the checks and their corresponding limit states is therefore presented in Table 4.

Table 4: Limit states of the three shear resistance checks (VB 74)

Shear resistance check	State
Shear tension failure	SLS
Reinforcement capacity	ULS
Fatigue	SLS

2.4.4 Load combinations in the Eurocode

A fundamental difference between the Eurocode and the VB 74 lies within the usage of safety factors. The Eurocode maintains a combination of partial load and material safety factors. For example at ultimate limit state, the material factors for concrete, reinforcement and prestress steel can be derived from the Dutch National Annex to Eurocode 2 (13), see Table 5.

Table 5: Partial material factors (ULS)

Concrete	Reinforcement	Prestress steel
$\gamma_c = 1{,}50$	$\gamma_s = 1{,}15$	$\gamma_p = 1{,}10$

The risk of shear tension failure and the capacity of the reinforcement are checked at ULS. Eurocode 0 (14) considers two load combinations in this state, one where permanent loads are governing (6.10a) and another where the variable actions on the structure are governing (6.10b).

$$\gamma_G * G_k + \gamma_Q * \psi_0 * Q_k + \gamma_P * P_k$$ [6.10a]

$$\xi\gamma_G * G_k + \gamma_Q * Q_k + \gamma_P * P_k$$ [6.10b]

The Dutch National Annex to Eurocode 0 (15) contains a table which defines the partial load factors for a number of different load cases at ULS. The values for a railway bridge in consequence class 3 are summarized in Table 6. The only missing factor is the one that accounts for simultaneous action, table A2.3 in Eurocode shows that a value of 0,80 may be taken for ψ_0.

Table 6: Partial load factors for a railway bridge in CC3 (ULS)

Type of loading	Load factor 6.10a	Load factor 6.10b
Self-weight	$\gamma_G = 1{,}40$	$\xi\gamma_G = 1{,}25$
LM71	$\gamma_Q = 1{,}50$	$\gamma_Q = 1{,}50$
SW/2	$\gamma_Q = 1{,}25$	$\gamma_Q = 1{,}25$
Prestress	$\gamma_P = 1{,}00$	$\gamma_P = 1{,}00$
Support-settlement	$\gamma_{sp} = 1{,}20$	$\gamma_{sp} = 1{,}20$

The fatigue resistance calculation is performed at ULS, but in this case all partial load factors are taken equal to 1,0. Additionally the OVS (regulations from Prorail) requires that the factor α, which takes deviations from normal traffic into account, should be taken equal to 1,0 as well. Ultimately, a characteristic load combination is considered in the fatigue resistance calculation:

$$\gamma_G * G_k + \gamma_Q * Q_k + \gamma_P * P_k \qquad \text{ULS Characteristic}$$

$$= G_k + Q_k + P_k$$

An overview of the checks and their corresponding limit states is presented in Table 7.

Table 7: Limit states of the three shear resistance checks (Eurocode)

Shear resistance check	State
Shear tension failure	ULS
Reinforcement capacity	ULS
Fatigue	ULS characteristic ($\gamma_G = \gamma_Q = \gamma_P = 1{,}0$)

Evidently the difference in load models, factors and combinations justifies a reassessment on the shear resistance of the through girder.

3 Analytical solution

One of the main objectives of this master thesis is to understand the force distribution in the through bridge. The best way to explain this is by determining the forces analytically. The first part of this paragraph elaborates on the forces one can expect in a through bridge and second part explains in detail how torsion, suspension forces and clamping moments can be evaluated.

3.1 Distribution of forces

Its assumed a concentrated load is applied on the middle of the floor. The load spreads at an angle of 45° outwards towards the girders and becomes a distributed load. The interaction between the floor and the girder creates a reactional force which is called the suspension force. This force transfers the applied vertical loading of the floor into the girder. The girder subsequently transfers the applied vertical loading in the longitudinal direction towards the supports, which means the suspension force generates a shear force and a bending moment in the girder (Figure 13).

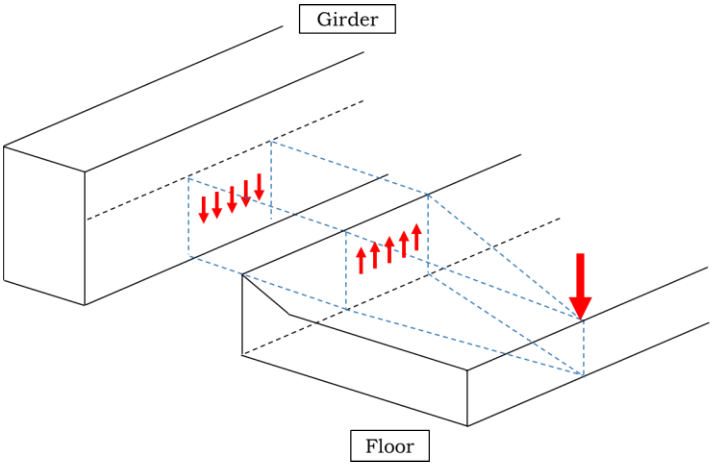


Figure 13: Suspension force due to concentrated load

Because the floor is assumed to be fully restrained by the girder, a second reactional force is present. Due to the rigid connection a bending moment is generated called the clamping moment. As the girder is free to rotate around the longitudinal axis, this moment causes a rotation of the girder and the floor (Figure 14).

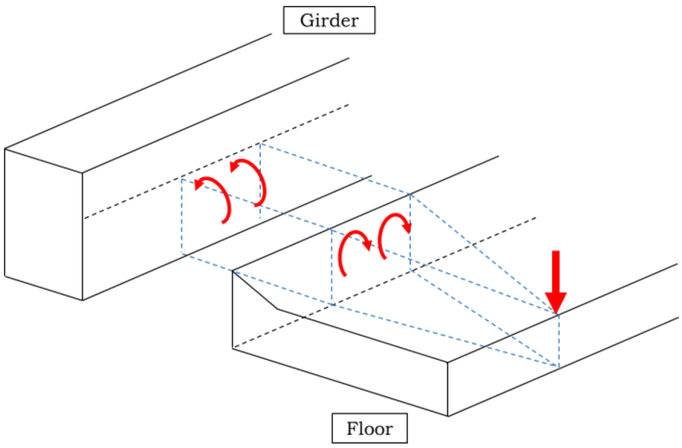


Figure 14: Clamping moment due to concentrated load

The self-weight of the bridge causes a continuous constant loading. Every part of the girder will undergo exactly the same rotation. As the girder is nowhere restricted to rotate about its longitudinal axis, no torsion is induced.

But if a train passes over the bridge, the loaded parts of the floor will deflection and the girder will undergo a certain rotation. Conversely, the unloaded parts of the bridge will hardly deflect nor rotate. This effect, where the unloaded parts of the bridge counteract the rotation of the loaded parts, generates torsion in the girder.

Analytically determining the shear force and bending moment is basic knowledge of mechanics and is not further explained. Yet the derivations of torsion, suspension force and clamping moment are somewhat more complex and are therefore discussed in the next paragraphs.

3.2 Suspension force

Figure 15 presents the top view of a through bridge. Clearly visible are the girder, the floor and the railway track which is mounted on the floor. When a train passes, the track is loaded with axle loads (in this case taken as a distributed line loads) which spread at angle of 45° outwards towards the girders.

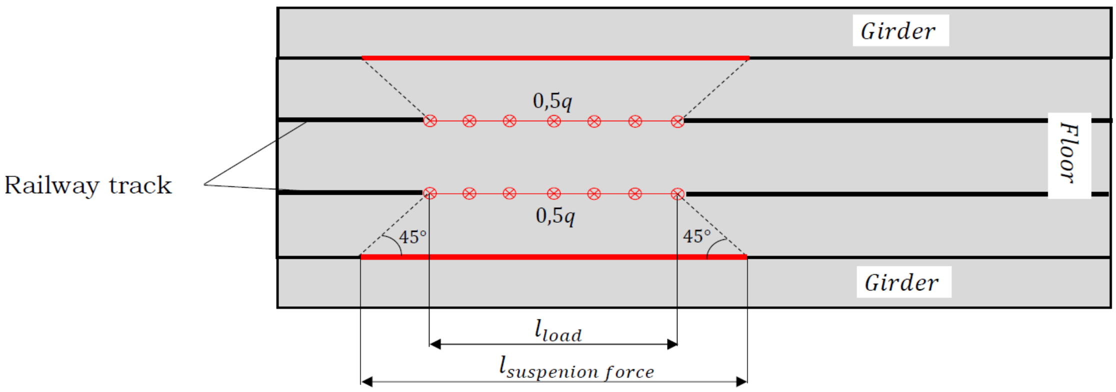


Figure 15: Top view of a through bridge, indicating the distribution of loads

The bending stiffness of the floor is assumed to be insignificant in longitudinal direction, but significant in transverse direction. The floor can therefore be divided into strips with a length of 1,0 meter. Figure 16, a typical cross-section of the bridge, depicts a typical strip of length 1,0 meter. The suspension force is found by dividing the applied loading on the track by the distributed length of the suspension force.

$$Q_{yy} = \frac{0{,}5q * l_{load}}{l_{suspension\ force}} \qquad [4]$$

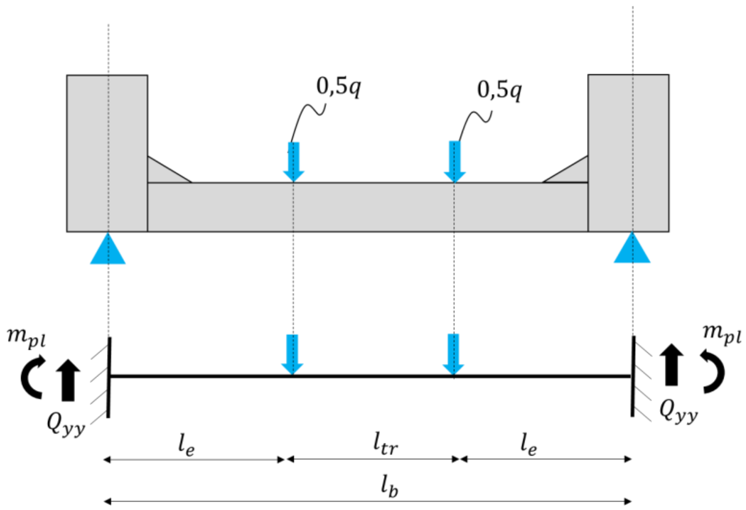


Figure 16: Transverse section of through bridge with loading on the railway track

3.3 Torsion

3.3.1 Primary load

The applied vertical load on the strip causes besides a suspension force an additional load, namely the primary load denoted by m_{pl}. If no loading is present at the strip, the primary load goes to zero. Yet the clamping moment, which is the reactional moment between the girder and the floor, does not go to zero.
This difference is explained by Figure 17; section A has a primary load whereas section B does not. The primary load is only present for strips with loading, where the clamping moment is the reactional force in the girder and spreads over the entire length of the bridge.

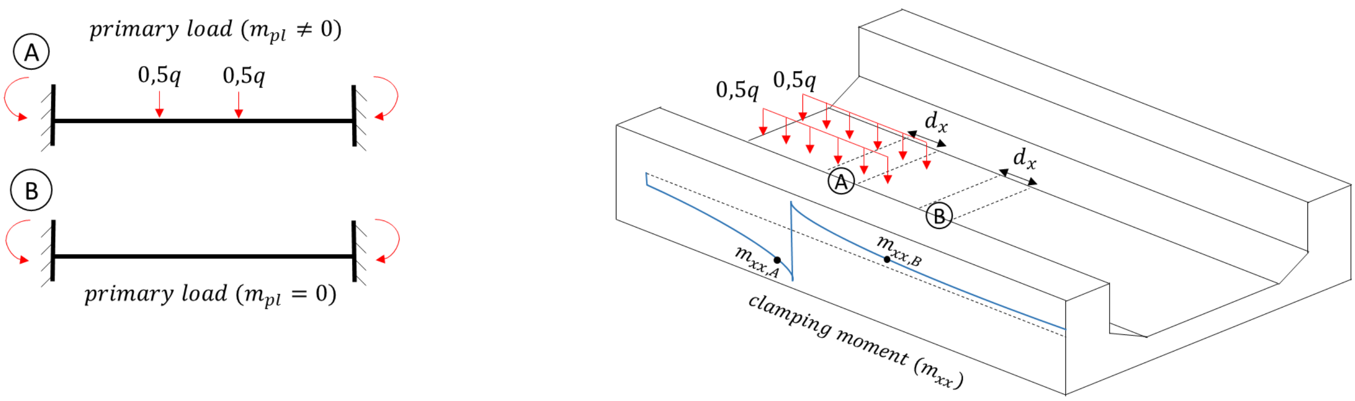


Figure 17: Primary load (left) and clamping moment (right) due to loading on the bridge

3.3.2 Differential equation and general solution

By using the master thesis of R.T.J. de Groot (4) a differential equation for torsion in the through girder is derived. The floor is divided into elements with a length dx and the following assumptions are made:

1. The torsional stiffness of the floor is neglected in both the longitudinal and transverse direction.
2. The load on the floor is distributed predominately in the transverse direction. Hence only the bending stiffness of the floor in transverse direction is assumed to be significant.
3. The connection between the floor and the main girder is assumed to be in the centre line of the main girder. The floor prevents the girder from bending in the transverse direction.
4. The supports are considered in the centre of the girder and they do not restrain a rotation about the longitudinal axis.
5. There is no difference in deflection between the two girders.
6. Only pure torsion is considered, no warping.

Figure 18 considers an element with length dx, where a load on the floor generates a clamping moment (m_{xx}) which leads to a change in rotation ($d\varphi$). A set of kinematic, constitutive and equilibrium equations is combined to derive a general differential equation for torsion.

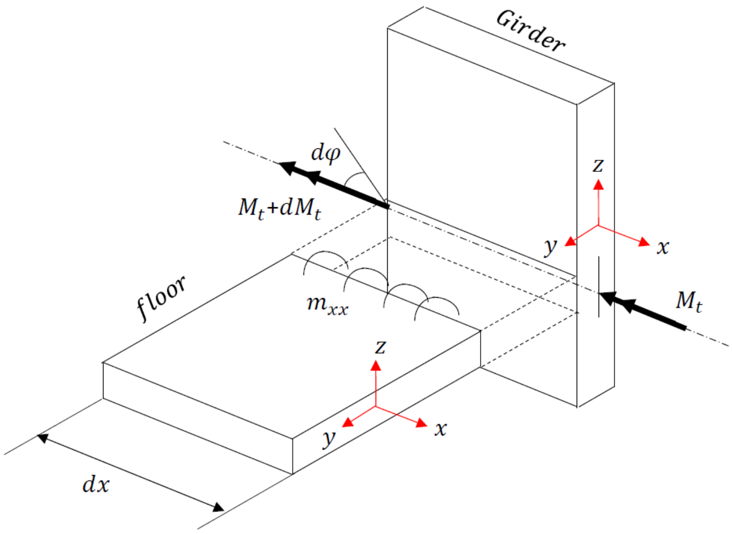


Figure 18: Element with length dx

$$\frac{d^2M_t}{dx^2} - \omega^2 M_t - \frac{dm_{pl}}{dx} = 0 \qquad [5]$$

Where:

$M_t =$ Torsion
$m_{pl} =$ Primary load
$\omega^2 = s_{pl}/GI_t$ Ratio between bending stiffness of the floor and torsional stiffness of the girder

The differential equation is solved by using a homogeneous and particular solution. The function contains constants which can be solved by determining the boundary conditions for specific cases.

$$M_t(x) = GI_t\omega * (C_1 \sinh \omega x + C_2 \cosh \omega x) \quad [6]$$

3.3.3 Torsion due to a train

Imagine a train which is about to leave the bridge, but the last carriage is still positioned on the bridge. Schematically this would look like Figure 19. In order to derive a function for torsion in girder I and II, two transitional conditions and two boundary conditions are needed.

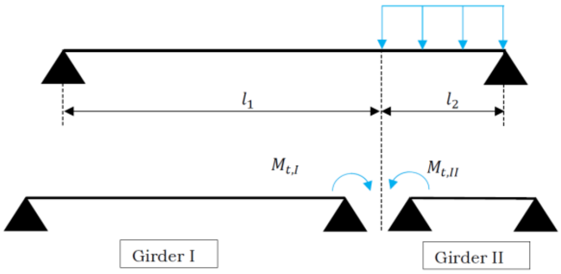


Figure 19: Train positioned at the right end of the bridge

Directly above the supports, the girder is not restricted and free to rotate. This means that torsion always equals zero above the supports. Thus the two boundary conditions for girder I and II, are respectively zero torsion at $x = 0$ and $x = l$.

BC1: $M_{t,I}(0) = 0$

BC2: $M_{t,II}(l) = 0$

Because the girder is split into two, transitional conditions are needed as well. In the transitional section ($x = l_1$), torsion and rotation in girder I should equal torsion and rotation in girder II.

TC1: $\varphi_I(l_1) = \varphi_{II}(l_1)$

TC2: $M_{t,I}(l_1) = -M_{t,II}(l_1)$

Torsion in girder I ($0 \leq x \leq l_1$) and II ($l_1 < x \leq l$) are defined by:

$$M_{t,I}(x) = \frac{m_{pl}}{\omega}\left(\frac{\tanh \omega l_1 * \tanh \omega l_2}{\tanh \omega l_1 + \tanh \omega l_2}\right)\left(\frac{\sinh \omega x}{\sinh \omega l_1}\right) \quad [7]$$

$$M_{t,II}(x) = \frac{m_{pl}}{\omega}\left(\frac{\tanh \omega l_1 * \tanh \omega l_2}{\tanh \omega l_1 + \tanh \omega l_2}\right)\left(\cosh \omega(x - l_1) - \frac{\sinh \omega(x - l_1)}{\tanh \omega l_2}\right) \quad [8]$$

The course of the torsion is graphically presented by Figure 20. The horizontal axis represents the x-coordinate along the girder whereas the vertical axis holds an expression for the ratio between the actual present torsion ($M_t(x)$) and the applied primary load (m_{pl}). From equation [7] and [8] it can be derived that the maximum torsion, found at midspan, goes to: $m_{pl}/2\omega$. This also becomes visible from the envelope in Figure 20 which is plotted with $m_{pl} = 1$ and $\omega = 0{,}24$.

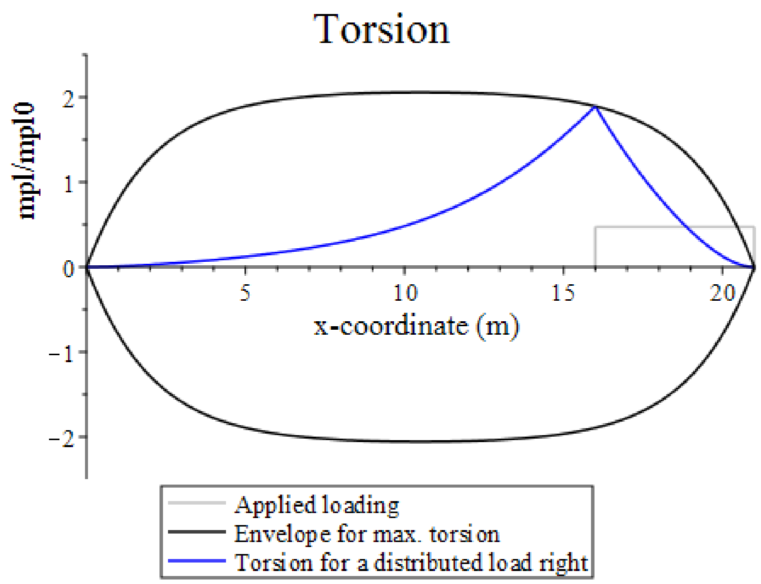


Figure 20: Torsion in the girder due to a train load at the right end

3.4 Clamping moment

A clamping moment generates a change in rotation and thus a change in torsion. In other words the derivative of torsion is equal to the clamping moment.

$$m_x(x) = \frac{dM_t(x)}{dx} \qquad [9]$$

The function for a train load at the right edge of the bridge, can be derived by taking the derivatives of equation [7] and [8].

$$m_{x,I}(x) = \frac{dM_{t,I}(x)}{dx} = m_{pl}\left(\frac{\tanh\omega l_1 * \tanh\omega l_2}{\tanh\omega l_1 + \tanh\omega l_2}\right)\left(\frac{\cosh\omega x}{\sinh\omega l_1}\right) \qquad [10]$$

$$m_{x,II}(x) = \frac{dM_{t,II}(x)}{dx} = m_{pl}\left(\frac{\tanh\omega l_1 * \tanh\omega l_2}{\tanh\omega l_1 + \tanh\omega l_2}\right)\left(\sinh\omega(x - l_1) - \frac{\cosh\omega(x - l_1)}{\tanh\omega l_2}\right) \qquad [11]$$

The clamping moment in the girder is graphically presented by Figure 21. The vertical axis is the ratio between clamping moment ($m_{xx}(x)$) and the primary load (m_{pl}). Remarkable is that the clamping moment is negative under loading and has a sign switch at the interface loaded-unloaded. This can be explained from the fact that the negative clamping moment under loading is counteracted by the unloaded parts of the bridge. Therefore the difference between the maximum and minimum clamping moment is equal to the applied primary load.

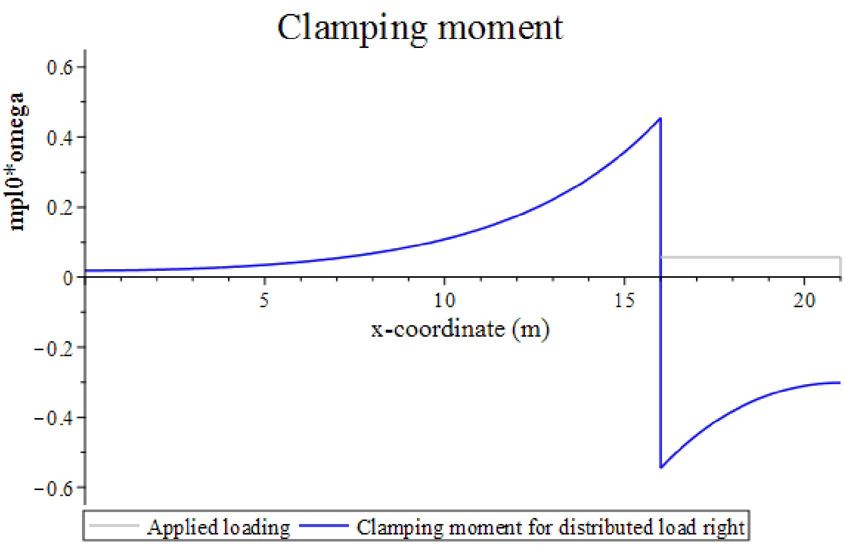


Figure 21: Clamping moment in the girder due to train load at the right end

The derivation of the differential equation and the corresponding graphs for torsion and clamping moment are discussed in detail in Appendix A.

4 Shear tension failure

In chapter 4 to 6 the shear resistance of the two fully prestressed through bridges, designed according to the VB 74, are investigated based on a number of Eurocode shear resistance checks. The final goal is to check whether the structural safety of these existing structures is comprised by the change in load models and calculation procedures of the Eurocode. This particular chapter will focus on the risk of shear tension failure.

Characteristic for the through bridge is the combination of shear, torsion and bending. At a section, just above the connection between the floor and the girder, the combination of normal and shear stresses is expected to be governing (Figure 22). This combination of stresses is called the principal stress and the maximum tensile principal stress can cause shear tension failure. The normal and shear stresses are induced by the following loads:

- Horizontal normal stress (σ_{xx}): Due to normal forces and bending from external loads, prestress and restrained deformations.
- Vertical normal stress (σ_{yy}): Due to suspension forces and clamping moments originating from loads on the floor.
- Shear stress (τ_{xy}): Due to torsion and shear from external loads, prestress and restrained deformations.

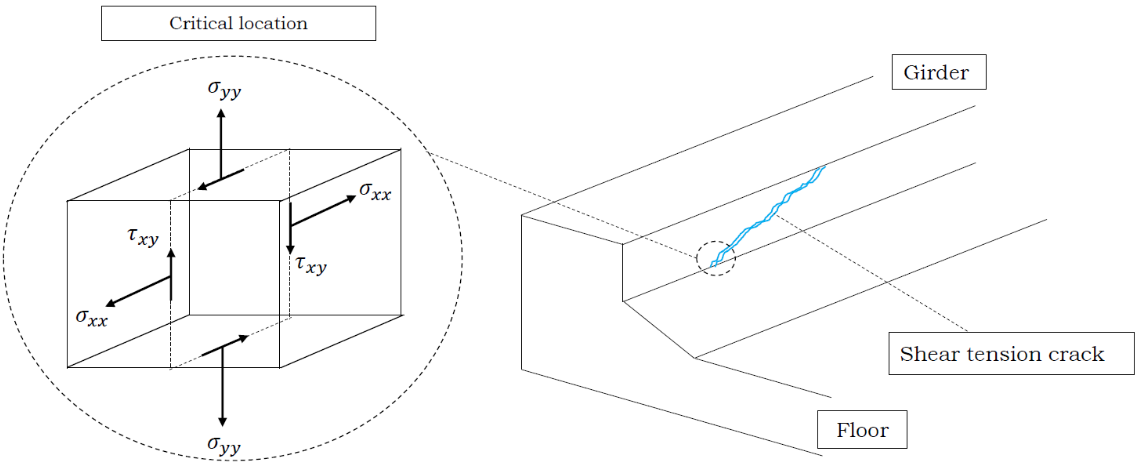


Figure 22: Critical location for shear tension failure

This chapter focuses on determining the normal and shear stress and eventually finding the maximum tensile principal stress. Ultimately the maximum stress is checked against the tensile design strength of concrete to assess the risk of shear tension failure.

4.1 Horizontal normal stress

The horizontal normal stress is due to bending and prestress. Using the formula's in Figure 23 the forces are converted into a normal stress, causing a compressive stress at the critical location. One should keep in mind that the horizontal prestress force is introduced above the supports and from there spreads at angle of 45°. When determining horizontal normal stress one should take into account the position of the considered section and the load spread of the prestress over the cross-section of the bridge (chapter 3, Appendix C).

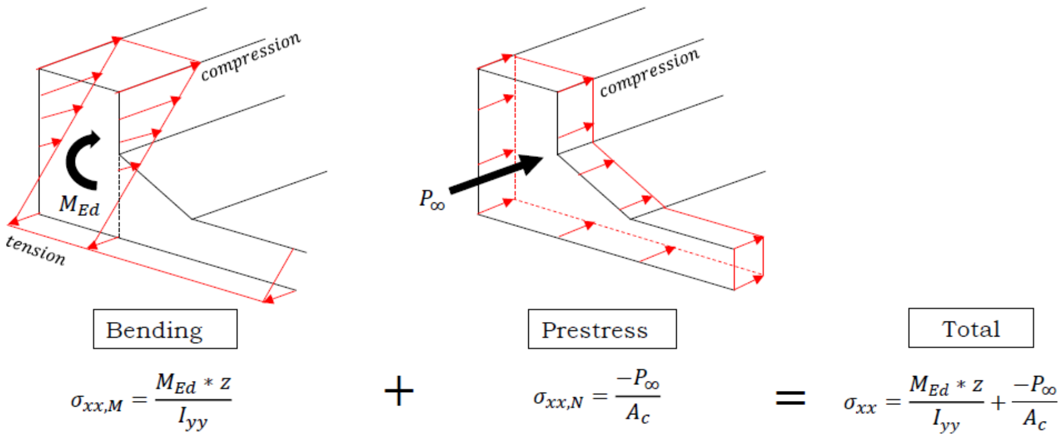


Figure 23: Horizontal normal stress due to bending and prestress

4.2 Vertical normal stress

The vertical normal stress is caused by the suspension force and the clamping moment. Because the transfer of forces takes places at the point where the floor is connected to the girder, both forces get a certain eccentricity with respect to the centre of the girder. The suspension force generates an additional bending moment, whereas the clamping moment can be freely moved to the centre of the girder. Consequently the maximum vertical normal stress can be found at the inside of the girder due to the additional bending moment.

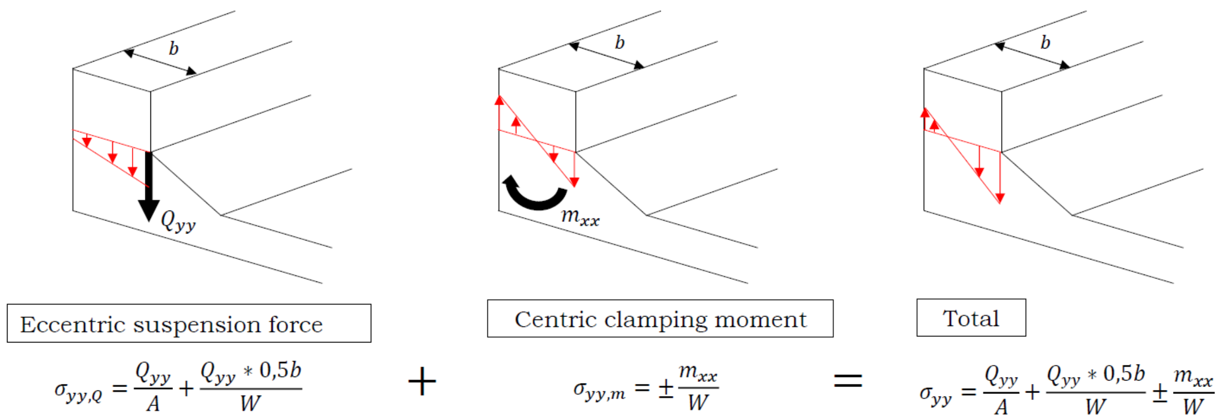
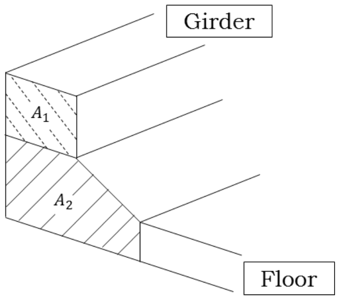


Figure 24: Vertical normal stress due to suspension force and clamping moment

This is a linear elastic calculation in which the girder is assumed to remain uncracked. The entire cross-section will therefore contribute to the load transfer of the suspension force and clamping moment. A ratio is introduced which describes the distribution of vertical normal stress over a section above (A_1) and below (A_2) the throat. Once the girder is cracked, for example in a stirrup calculation, this distribution is no longer valid.

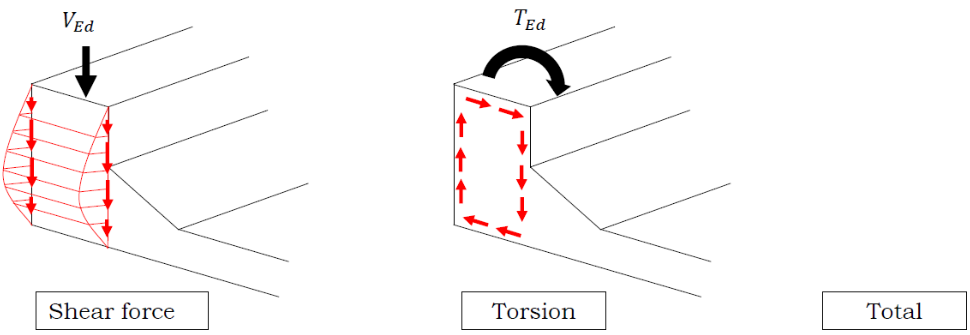


$$\sigma_{yy} = \frac{A_1}{A_1 + A_2} * \sigma_{yy,tot}$$

Figure 25: Load distribution of vertical normal stress in LE calculation

4.3 Shear stress

Shear stress in the girder is caused by shear forces and torsion from external loading. The shear forces generates a stress which is zero at the top and the bottom and has a maximum at the neutral axis. But from the outside to the inside of the girder the stresses remain uniform.
The stress due to torsion is determined by simplifying the girder into a thin walled box girder. Unlike the shear force, this shear stress is constant with height and rotates with the direction of the applied torsional moment. Ultimately the maximum shear stress for the critical section is found at the inside of the girder.



$$\tau_{xy,V} = \frac{V_{Ed} * S}{b * I} \quad + \quad \tau_{xy,T} = \pm \frac{M_{xy}}{2 * A_m * t_{ef}} \quad = \quad \tau_{xy} = \frac{V_{Ed} * S}{b * I} \pm \frac{M_{xy}}{2 * A_m * t_{ef}}$$

Figure 26: Shear stress due to shear and torsion

4.4 Principal stress

An element under stress has at least three planes, called the principal planes. At each plane a normal vector acts called the principal direction. The stresses acting parallel to the normal vectors are the so called principal stresses. Each stressed element has three principal stresses, namely the tensile and compressive principal stress and the maximum shear stress (where the normal stresses equal zero). The principal stresses are determined using Mohr's circle. Respectively the tensile and compressive stress are expressed by equations [12] and [13].

$$\rho_1 = \frac{\sigma_{xx} + \sigma_{yy}}{2} + \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \tau_{xy}{}^2} \qquad [12]$$

$$\rho_2 = \frac{\sigma_{xx} + \sigma_{yy}}{2} - \sqrt{\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \tau_{xy}{}^2} \qquad [13]$$

Equation 6.4 in Eurocode 2 (16) expresses the resistance of a structure against shear tension failure as:

$$V_{Rd,c} = \frac{I * b_w}{S}\sqrt{(f_{ctd})^2 + \sigma_{cp} * f_{ctd}} \qquad [14]$$

The maximum allowable principal stress is not explicitly mentioned in the Eurocode. By replacing the compressive prestress (σ_{cp}) with the normal stress (σ) and converting the shear force $(V_{Rd,c})$ into a stress (τ_{xy}), a maximum allowable tensile principal stress can be found:

$$f_{ctd} = \frac{\sigma}{2} + \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau_{xy}{}^2} \qquad [15]$$

The girder is constructed with B45, which according to the design report has a characteristic tensile strength of $2{,}0\ N/mm^2$. Consequently the maximum allowable tensile stress at ultimate limit state is equal to $f_{ctd} = 2{,}0/1{,}5 = 1{,}33\ N/mm^2$.

4.4.1 Eurocode

In the design reports the maximum tensile principal stress is found at 0,8d from the support for bridge A and above the support for bridge B. For the calculation in the Eurocode, the same locations are assumed to be critical. Additionally the combination of shear force and torsion are quite likely to be at a maximum at these locations.

Paragraph 2.4.2 presents SCIA generated influence lines in order to establish the most unfavourable position of LM71 and SW/2. The top left and right images of Figure 27 summarize the positions for respectively LM71 and SW/2 that result in maximum loading (at a section 0,8d) in bridge A. The bottom image corresponds with the maximum negative load case (SW/2 min.). By fully loading the cantilevers, negative torsion and shear is generated, which in combination with the prestress load case may lead to a critical principal stress.

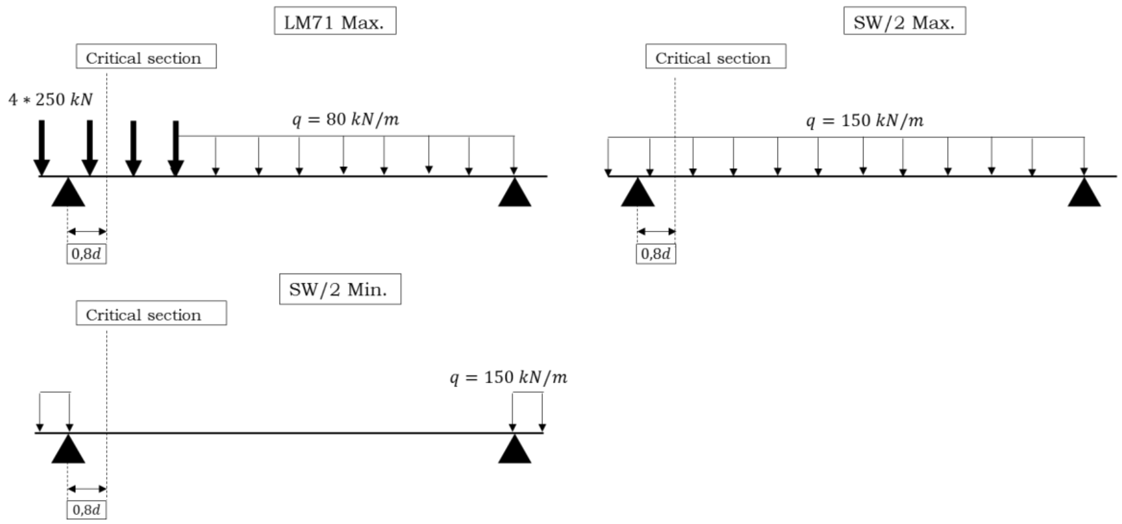


Figure 27: Bridge A: Position of LM71 and SW/2 generating the most unfavourable loading at section 0,8d

Table 8 shows the maximum tensile principal stresses and the unity checks for bridge A and B. For both bridges, load model 71 in combination 6.10b is governing. Respectively a unity check of 0,97 and 0,77 for bridge A and B is found. These values are acceptable and according to the Eurocode there is no risk of shear tension failure. A more detailed version of this calculation is presented in Appendix C.

Table 8: Eurocode tensile principal stresses and unity checks for a section at 0,8d (A) and above the support (B)

	Bridge A		Bridge B	
Load combination	6.10b | 0,8d		6.10b | support	
	ρ1 [N/mm2]	U.C.	ρ1 [N/mm2]	U.C.
LM71 Max.	1,29	**0,97**	1,02	**0,77**
SW/2 Min.	0,97	0,73	0,88	0,66
Self-weight + prestress	0,11	0,08	0,13	0,10
SW/2 Max.	1,24	0,93	0,96	0,72

4.4.2 VB 74

The design reports considers multiple positions of load model VOSB 150. But only two positions are governing for the check on shear tension failure. The maximum loading due to VOSB 150 is found, when the group of concentrated loads is positioned at $1/3L$ and the distributed load runs between the supports. The maximum negative load combination (VOSB 150 min.) is found, when the cantilevers and the run-up plates are fully loaded by the concentrated and distributed loads (Figure 28). Eventually the loads in the design reports are evaluated with an ANSYS model.

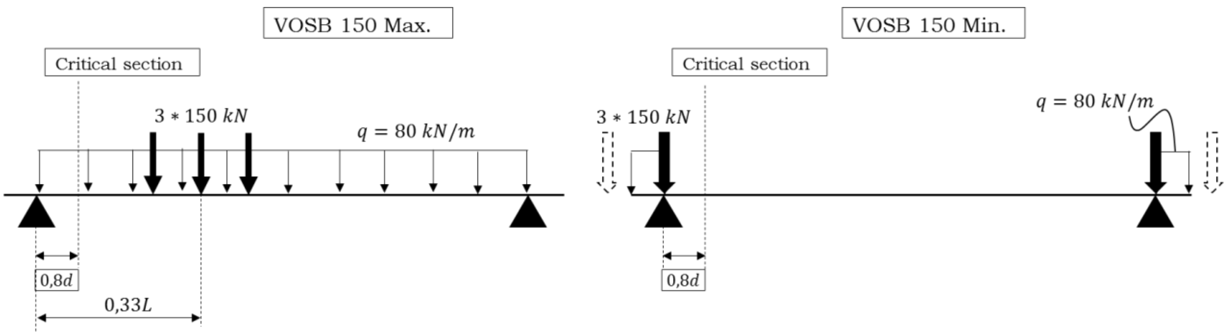


Figure 28: Bridge A: Positions of VOSB 150 causing maximum and minimum loading at 0,8d

As mentioned in paragraph 2.3.1 a structure is able to resist shear tension failure (according to the VB 74) when the maximum tensile principal stress remains smaller than half the characteristic tensile strength: $0{,}5 * f_{ctk;0{,}05} = 1{,}0\ N/mm^2$. As presented in Table 9 the unity check's for shear tension failure stay well within the acceptable limits.

Table 9: VB 74 tensile principal stresses and unity checks for a section at 0,8d (A) and above the support (B)

	Bridge A		Bridge B	
Location	0,8d		support	
Load combination	ρ1 [N/mm2]	U.C.	ρ1 [N/mm2]	U.C.
VOSB 150 Max.	0,60	0,60	0,59	0,59
VOSB 150 Min.	0,93	**0,93**	0,88	**0,88**
Self-weight + Prestress	0,51	0,51	0,67	0,67

In contrast to the Eurocode calculation, the maximum tensile principal stress in both bridges is found when the cantilevers are fully loaded. An explanation for this is found in the design report. The introduction of prestress load in ANSYS leads to large tensile stresses and strains in the first part of the floor, causing rotations of the girder. This rotation generates torsion of a much larger magnitude than expected in reality. Now because torsion due to prestress and the minimum load combination act in the same direction, the governing tensile principal stress is found when the cantilevers are fully loaded by VOSB 150. Even though it is acknowledge in the design report that the values for torsion due to prestress are unrealistically high, it is assumed the values will form an safe upper limit (Appendix B).

It appears the two through bridges are designed with possible conservative values for torsion. It may therefore be interesting to verify the risk of shear tension failure in a VB 74 bridge which is designed more accurately.

5 Reinforcement

A crucial difference between the VB 74 and the Eurocode is the possibility to combine the shear resistance of concrete and stirrups. In the days of the VB 74 this was allowed, but with the arrival of the Eurocode it was no longer. The Eurocode assumes the shear resistance of the concrete is zero if it's standalone contribution is insufficient. Meaning that once stirrups are required, the total applied shear stress is controlled by the stirrups.

$\frac{V_{Ed}}{V_{Rd,c}} + \frac{T_{Ed}}{T_{Rd,c}}$	$< 1{,}0$	Minimum amount of stirrups suffices
	$> 1{,}0$	Total shear stress controlled by the applied stirrups
$\frac{V_{Ed}}{V_{Rd,max}} + \frac{T_{Ed}}{T_{Rd,max}}$	$< 1{,}0$	Capacity of the compressive diagonal sufficient
	$> 1{,}0$	Capacity of the compressive diagonal insufficient

The first rule determines, for a combination of shear and torsion, whether or not shear reinforcement is necessary. The second one checks if the capacity of the diagonal is sufficient. Paragraph 5.1 and 5.2 determine the required amount of shear and longitudinal reinforcement (in the two bridges) according to the Eurocode. Paragraph 5.3 presents an overview of the VB 74 reinforcement calculations and the corresponding unity checks. And a detailed version of these calculations can be found in appendix D.

5.1 Shear reinforcement

With the application of stirrups the internal force distribution changes and is best described by the truss analogy. A compressive diagonal forms under an angle θ intersecting multiple stirrups. The vertical component of the diagonal is taken up by the stirrups, whereas the horizontal components result in a compression zone in the concrete and a tensile force in the longitudinal reinforcement.

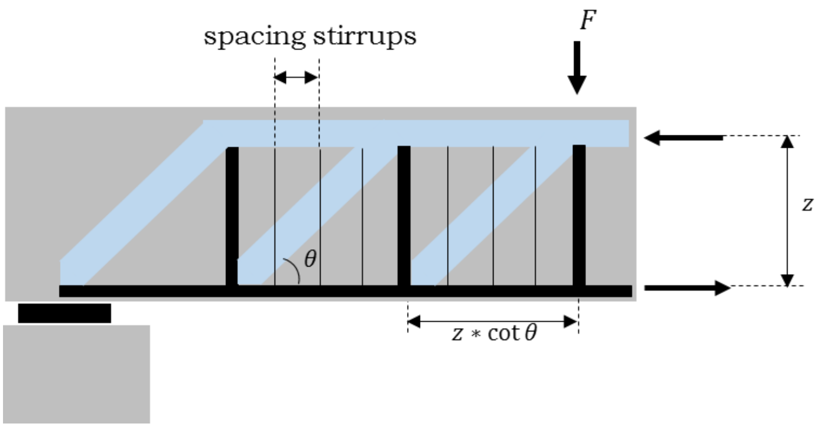


Figure 29: Truss analogy – Compressive diagonal intersecting multiple stirrups

Bridge A is equipped with an outer and inner stirrup, whereas bridge B only has one stirrup. The NS-guideline 1015 (17) defines a number of zones in the girder which have a specific load transferring function:

- Zone I: Shear and torsion
- Zone II: Shear
- Zone III: Shear, torsion and suspension loads

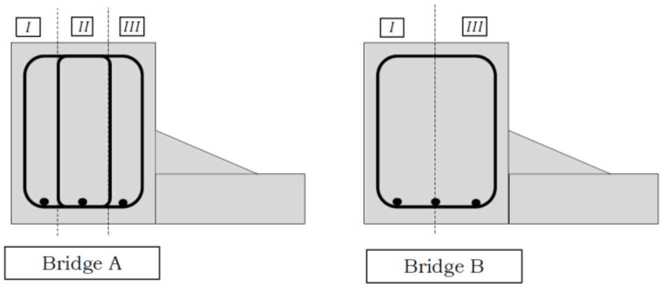


Figure 30: Shear reinforcement zones for bridge A (left) and bridge B (right)

The shear force causes a constant stress over the width of the critical section. In line with the guideline it is expected that 50% of the load goes through the outer stirrup and the other 50% through the inner stirrup.

Torsion creates a constant shear stress which rotates with the direction of the applied torsional moment. Since the girder is simplified to a structure with thin walls, the accompanying stresses are only present in zone I and III. For bridge A, this means only the outer stirrups transfers the shear stress due to torsion.

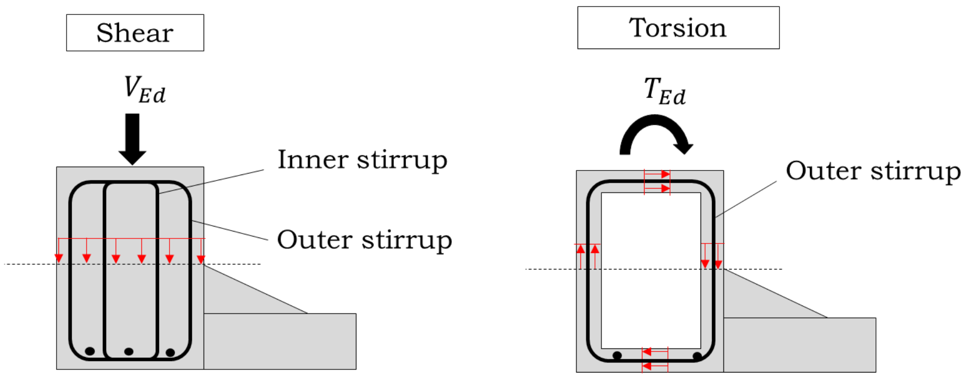


Figure 31: Shear stress in the stirrups due to shear and torsion

The required amount of shear reinforcement is determined per unit length, by dividing the load with the horizontal length of the diagonal ($z * \cot\theta$). Respectively the required amount of reinforcement (according to the Eurocode) is defined by equation [16] and [17] for shear and torsion.

$$\frac{A_V}{s} = \frac{V_{Ed}}{z * \cot\theta * f_{ywd}} \quad [16]$$

$$\frac{A_T}{s} = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta} \quad [17]$$

The NS-guideline 1015 assumes all load transfer from the floor to the girder occurs in zone III. Therefore one leg of the outer stirrup should be able to withstand the vertical normal stresses caused by the suspension force and the clamping moment. Additionally the guideline sets that the suspension reinforcement should be designed using a yield strength of 220 MPa, even though stirrups of FeB500 are applied. The explanation for this lies in the fact that a great part of the loading in the suspension reinforcement is due to mobile loading, making the reinforcement more sensitive to fatigue. By limiting the maximum yield stress to 220 MPa there should be sufficient resistance against fatigue loading.

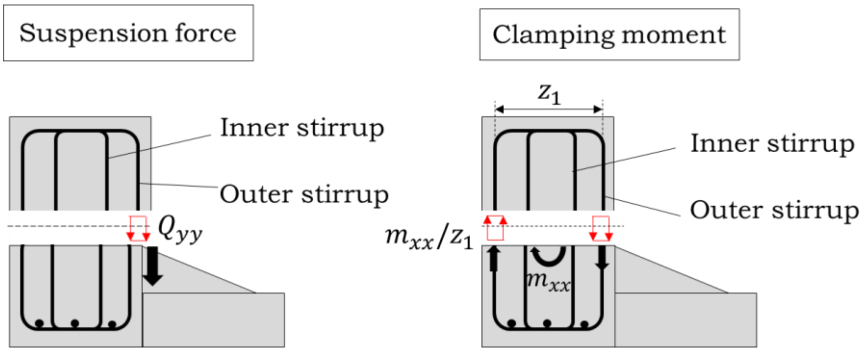


Figure 32: Vertical normal stress in the stirrups due to suspension force and clamping moment

The suspension force and clamping moment act at the upper side of the connection between the floor and the girder. At ultimate limit state, the girder is assumed as cracked, meaning the stirrups should be able to transfer 100% of the suspension loads from the floor into the girder. Initially it is assumed that the suspension loads are transferred by the outer stirrup leg only. But if the capacity is insufficient, the ductile outer stirrup (FeB500) will start to yield and a part of the loading is transferred to the inner stirrup leg. In that case a distribution over the outer and inner stirrup leg may be taken into account for the suspension loads.

Equation [18] and [19] can be used to determine the required amount of shear reinforcement for the suspension force and the clamping moment. Since the loads are obtained using strips with a length of $dx = 1{,}0\ m$, the required amount of reinforcement is found by dividing the loads by a $1000\ mm$ and the reduced yield strength. Additionally the clamping moment is divided by internal lever z_1, to transfer the bending moment into a shear force.

$$\frac{A_Q}{s} = \frac{Q_{yy}}{1000 * f_{yk}} \qquad [18]$$

$$\frac{A_m}{s} = \frac{m_{xx}/z_1}{1000 * f_{yk}} \qquad [19]$$

The angle of the diagonal (θ) can be freely chosen between 21,8° and 45°. Increasing the angle maximizes the load on the shear reinforcement and minimizes the load on the longitudinal reinforcement. Decreasing the angle has the opposite effect. For both bridges a section near the supports and at midspan are considered. The critical loading near the supports is mainly due to torsion and shear, whereas at midspan it is due to bending. The optimal capacity of the shear and longitudinal reinforcement is reached when an angle of 21,8° is used near the support and an angle of 45° is used at midspan.

The outer stirrup in bridge A does not have sufficient capacity to transfer all suspension loads. Therefore the calculations on the shear reinforcement are performed with a distribution of 65% and 35% of the suspension loads over the outer and inner stirrup respectively. By comparing the total required amount of shear reinforcement according to the Eurocode to the applied amount of shear reinforcement (VB 74), a unity check is established. Bridge A holds a value of 1,01 and 1,00 for respectively the outer and inner stirrup, where bridge B only has one stirrup which fulfils the requirements with a unity check of 0,90 (Table 10).

Even though the shear reinforcement is subjected to larger loads in the Eurocode, the checks for the applied amount of shear reinforcement stay just within the acceptable limits. Especially the possibility to vary the angle of the compressive diagonal and redistribute the suspension loads over the outer and inner stirrup, gives the opportunity to maximize the capacity of the present reinforcement.

Table 10: Eurocode unity checks for shear reinforcement in bridge A and B

Type of reinforcement	Bridge A		Bridge B	
	U.C. @ 0,8d	U.C @ 0,5L	U.C. @ support	U.C @ 0,5L
	$\theta = 21{,}8°$	$\theta = 45°$	$\theta = 21{,}8°$	$\theta = 45°$
Outer stirrup	**1,01**	0,16	**0,90**	0,36
Inner stirrup	1,00	0,28		

5.2 Longitudinal reinforcement

In paragraph 5.1 the truss analogy explained how a shear force creates a tensile force in the longitudinal reinforcement. Besides shear the girder is subjected to a prestress and an applied bending moment due to external loading. The bending moment causes an additional tensile force in the longitudinal reinforcement, whereas the prestress increases the tensile capacity of the reinforcement in two ways. Firstly, the prestress subjects the reinforcement to a compressive stress leading to larger loads before the reinforcement starts to yield. Secondly, when tendons are present in a tension zone, they contribute to the tensile capacity by withstanding tensile forces until yielding (f_{pd}).

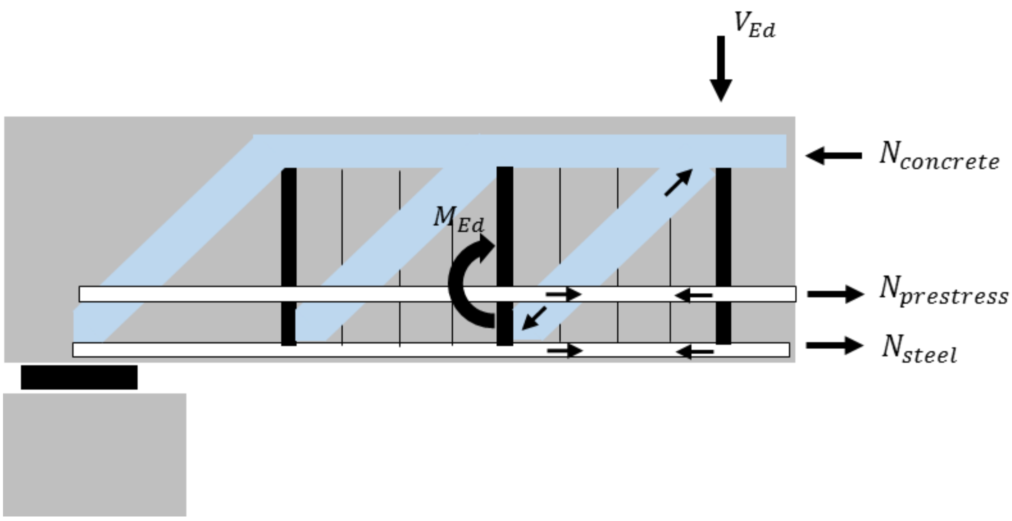


Figure 33: Shear and bending cause a tensile force in the longitudinal reinforcement and tendons

Since this check is performed at ultimate limit state, the girder is assumed to be cracked. Due to cracking the tensile strength of the concrete reduces to zero and possible tensile stresses need to be transferred by a combination of tendons and longitudinal reinforcement. This approach changes the internal force distribution as explained by the next section.

A section at 0,8d from the support is considered where the prestressing tendons are located at various heights across the section (Figure 34). When the girder cracks the compression zone will get a certain height (x) and compressive strain (ε_c). The strains in prestressing tendons and reinforcement are rewritten and expressed in terms of x and ε_c. With the Young's Modulus and cross-sectional area these strains are converted into forces. Eventually two equations guaranteeing equilibrium of horizontal forces and bending moments are solved in order to find suiting values for the two unknowns.

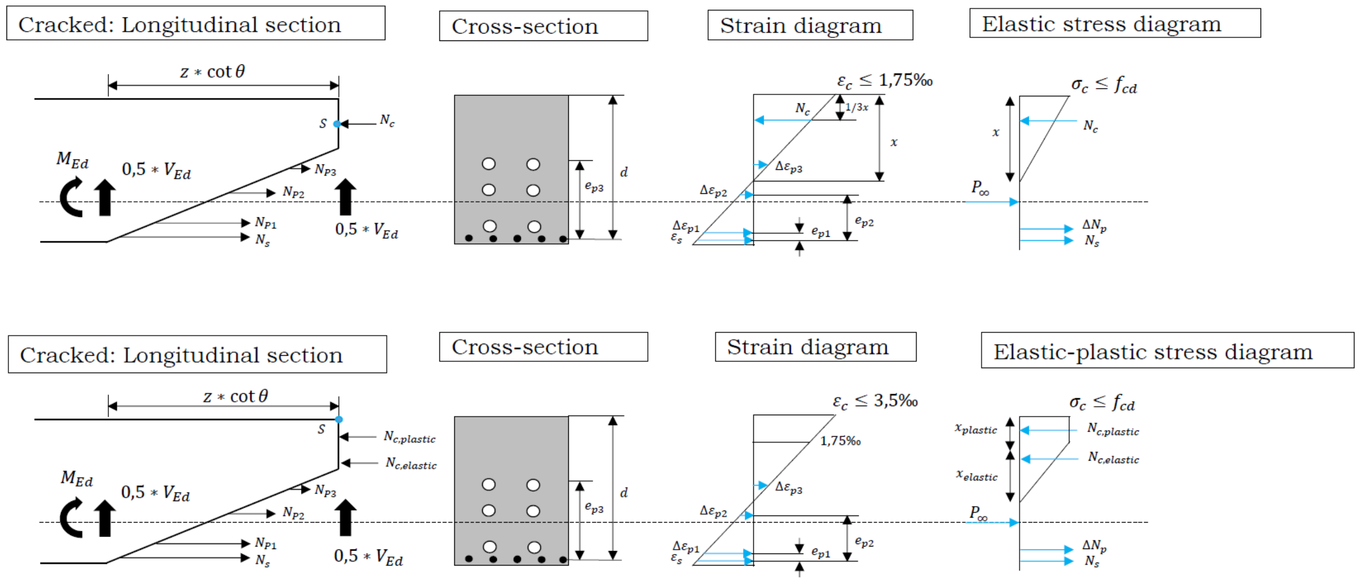


Figure 34: Stress and strain diagram for a section at 0,8d loaded in the elastic stage (top) and elastic-plastic stage (bottom)

Based on the obtained concrete strain and height of the compression zone, three stress-strain diagrams are possible:

1. Elastic ($\varepsilon_{c,top} \leq 1{,}75‰$): Up to a strain of 1,75‰ concrete behaves linearly elastic. The stresses and strains have a linear course and the maximum stress is equal to the compressive design strength (f_{cd}).
2. Elastic-plastic ($1{,}75‰ < \varepsilon_{c,top} \leq 3{,}5‰$): Beyond a strain of 1,75‰ the concrete starts to deform plastically and the maximum stress remains equal to the concrete design strength. A strain of 3,5‰ is the value for which a structure is about to fail. For this strain the height of the compression zone is limited by the Eurocode and the rotational capacity should be ensured. But because the prestress and reinforcement are most likely to yield at a lower strain, the yield strain of steel is maintained as the maximum allowable.
3. Compression only $(x > h_{girder})$: When the height of the compression zone exceeds the height of the cross-section, only compressive stresses are present in the girder. Naturally the unity check for longitudinal reinforcement goes to zero.

Ultimately the concrete strain and height of the compression zone are used to calculate the additional tensile force in the longitudinal reinforcement and tendons. The unity checks according to the Eurocode are presented in Table 11 and are well within the acceptable limits. Noteworthy is the constant value for prestressing steel. This is due to the fact that the additional tensile stress is relatively small compared to the yield stress. Hence the unity check holds a base value which is roughly equal to: $\sigma_{p\infty}/f_{pd}$.

Table 11: Eurocode unity checks for longitudinal reinforcement and prestress in bridge A and B

Type of reinforcement	Bridge A		Bridge B	
	U.C. @ 0,8d	U.C @ 0,5L	U.C. @ support	U.C @ 0,5L
	$\theta = 21{,}8°$	$\theta = 45°$	$\theta = 21{,}8°$	$\theta = 45°$
Longitudinal reinforcement	**0,26**	0,18	0,09	**0,56**
Prestressing steel	0,71	0,71	0,67	0,71

In paragraph 2.2.3 concerns are addressed regarding the capacity of the compressive diagonal. This capacity, depending on the dimension of the girder, determines the maximum shear resistance of the through bridges and is therefore important to check in the reassessment. Table 12 presents the Eurocode unity checks for the capacity of the compressive diagonal, which is most critical for a section near the support, but with a maximum of 0,59 stays well within the acceptable limits.

Table 12: Eurocode unity checks on the compressive diagonal capacity in bridge A and B

Capacity	Bridge A		Bridge B	
	U.C. @ 0,8d	U.C @ 0,5L	U.C. @ support	U.C @ 0,5L
	$\theta = 21{,}8°$	$\theta = 45°$	$\theta = 21{,}8°$	$\theta = 45°$
Compressive diagonal	**0,59**	0,11	**0,57**	0,05

5.3 VB 74

This paragraph presents an overview on the VB 74 reinforcement calculations and unity checks. With the help of Table 13 a couple of differences, with respect to the Eurocode, can be explained:

- A combination of prestress and stirrup shear resistance is taken into account in the VB 74 for the prestressed through bridges. Conversely the Eurocode assumes the total applied shear stress is controlled by the stirrups.
- The VB 74 assumes that only torsion generates an additional tensile forces which needs to be transferred by longitudinal reinforcement. The Eurocode assumes that this additional tensile force is due to a combination of shear and torsion.
- In case torsion and shear act on a structure simultaneously, the VB 74 allows one to reduce the combined amount of reinforcement to 85% of the separately required amount of reinforcement. The Eurocode does not allow such a reduction.

Table 13: Reinforcement calculations in the VB 74

Design load	Type of reinforcement	Formula
Shear	Stirrup	$A_V = \frac{(\tau_{V,ed} - \tau_{1p}) * b * h * 0{,}9}{0{,}9 * h * f_{ywd}}$ $\tau_{1p} = 0{,}15 * \frac{P_\infty}{A}$
Torsion	Stirrup	$A_T = \frac{(\tau_{T,ed} - \tau_{1p}) * 2 * A_k * t}{2 * A_k * f_{ywd}}$
	Longitudinal reinforcement	$A_T = \frac{(\tau_{T,ed} - \tau_{1p}) * 2 * A_k * t * u_k}{2 * A_k * f_{ywd}}$
Shear + Torsion	Stirrup	$A_{V,V+T} = 0{,}85 * V_{Ed} - \frac{\tau_{V,ed}}{\tau_{V,ed} + \tau_{T,ed}} * V_{1p}$ $A_{T,V+T} = 0{,}85 * T_{Ed} - \frac{\tau_{V,ed}}{\tau_{V,ed} + \tau_{T,ed}} * T_{1p}$
Suspension force	Stirrup	$A_Q = \frac{Q_{yy}}{f_{yk}}$
Clamping moment	Stirrup	$A_m = \frac{m_{xx}}{b * f_{yk}}$

Where:

$\tau_{V,ed}$ = Shear stress due to shear force
$\tau_{T,ed}$ = Shear stress due to torsion
τ_{1p} = Shear resistance contribution of prestress
A_k = Area enclosed by the fictitious box girder
t = Wall-thickness of the fictitious box girder
u_k = Perimeter of the fictitious box girder
V_{1p} = Resistance against shear due to prestress
T_{1p} = Resistance against torsion due to prestress

Table 14 compares the required amount of reinforcement according to the VB 74 and the applied amount of reinforcement in the form of a unity check. It can be concluded that as well the outer stirrup as the longitudinal reinforcement are not designed with a large overcapacity, whereas the inner stirrup does have a lot of extra capacity. But one should understand that these unity checks do not say much. The reinforcement is designed based on the VB 74, which means the applied amount of reinforcement will always exceed the required amount. The results from the Eurocode are therefore compared to the applied amount of reinforcement rather than to these unity checks.

Table 14: VB 74 Unity checks for reinforcement

Type of reinforcement	Bridge A		Bridge B	
	U.C.	Location	U.C.	Location
Outer stirrup	**0,96**	0,8d	0,82	support
Inner stirrup	0,19	0,8d	N/A	
Longitudinal reinforcement	0,96	0,8d	**0,96**	0,5L

6 Fatigue

The bridge is at rest when no train or other variable loads are present. In that case the maximum stress in the bridge is formed by a combination of self-weight and prestress. But when a train passes over the bridge, the maximum stress increases significantly. During the life span of the bridge thousands of trains pass over the bridge subjecting it to an even larger number of stress fluctuations. These fluctuations can cause small cracks in materials such as steel and concrete. With an increasing number of load cycles, small cracks will develop into larger ones eventually leading to failure of the structure. This phenomenon is called fatigue.

During the design of the bridges there was little knowledge on fatigue, however a stress range limitation on the longitudinal and shear reinforcement was included. Compared to the allowable stress ranges in the Eurocode, the recorded values in the design reports are significantly higher. Additionally the Eurocode holds a fatigue resistance check for concrete, something which is not considered in the design reports.

The difference in fatigue resistance calculations methods calls for a reassessment of the fatigue resistance of the through girders according to the Eurocode. This calculation is briefly explained in paragraph 6.1 to 6.4 and a more detailed version can be found in appendix F. In the final paragraph (6.5) of this chapter, the fatigue resistance calculations according to the NS-guideline 1016 (18), which is used in addition to the VB74, are presented.

6.1 Cracking of the girder

According to the OVS (19) a prestressed structure should remain uncracked at serviceability limit state. The maximum tensile bending stresses in the girders are therefore checked for a characteristic, frequent and quasi-permanent load combination and should remain below the following limits:

- Quasi-permanent: No tension is allowed in the entire cross-section.

- Frequent: No tensile stresses are allowed in the tendon zone and for the non-tendon zone the stresses should be limited to: $\sigma_b < 0{,}5 * f_{ctk;0{,}05} = 1{,}0\ N/mm^2$.

- Characteristic: In the tendon zone the tensile stresses are limited to $\sigma_b < 0{,}5 * f_{ctk;0{,}05} = 1{,}0\ N/mm^2$ and in the non-tendon zone the tensile stresses should remain smaller than: $\sigma_b < 0{,}75 * f_{ctk;0{,}05} = 1{,}5\ N/mm^2$

In appendix E the tensile bending stresses at midspan are determined at serviceability limit state. It turns out that girder A fulfils the requirements of the OVS, meaning the girder is uncracked and no additional checks on crack width are necessary. Girder B however does not fulfil the requirements for the characteristic and frequent load combination (due to larger loads from LM 71) and therefore has to be assumed as cracked.

Now that girder B is cracked not only the crack width needs to be verified, but also a different approach is used in the fatigue resistance calculations (paragraph 6.3 and 6.4). The cracked girder approach (Appendix D) is used to determine the concrete strain and height of the compression zone for a characteristic load combination on girder B. Ultimately it is concluded that the compression zone is larger than the height of the cross-section, meaning the checks on crack width are no longer necessary. Yet the cracking does have an influence on the way the fatigue resistance calculations are performed, this is discussed in more detail later on.

6.2 Fatigue Loading

At the start of this research, fatigue verification according to annex NN in Eurocode 2 (20) was allowed by the OVS. But with the arrival of the new version of the OVS, the fatigue verification needs to be performed according to the general part of Eurocode 2. Because the new OVS was published towards the end of this research, the calculations are performed according to annex NN.

Because the load combination for fatigue verification is not explicitly mentioned in Eurocode 0 it is derived from annex NN.3. The maximum, minimum (due to LM71) and permanent load combination at ultimate limit state need to be considered. But even though this is an ultimate limit state check all partial load factors and factor α (LM71) are equal to 1,0. The characteristic load combinations are expressed as:

$$\sum G + P \qquad \text{Permanent}$$

$$\sum G + P + Q_{LM71,max} \qquad \text{Maximum LM71}$$

$$\sum G + P + Q_{LM71,min} \qquad \text{Minimum LM71}$$

The positions for the maximum and minimum load combination of LM71 are presented at 0,8d and midspan by Figure 35.

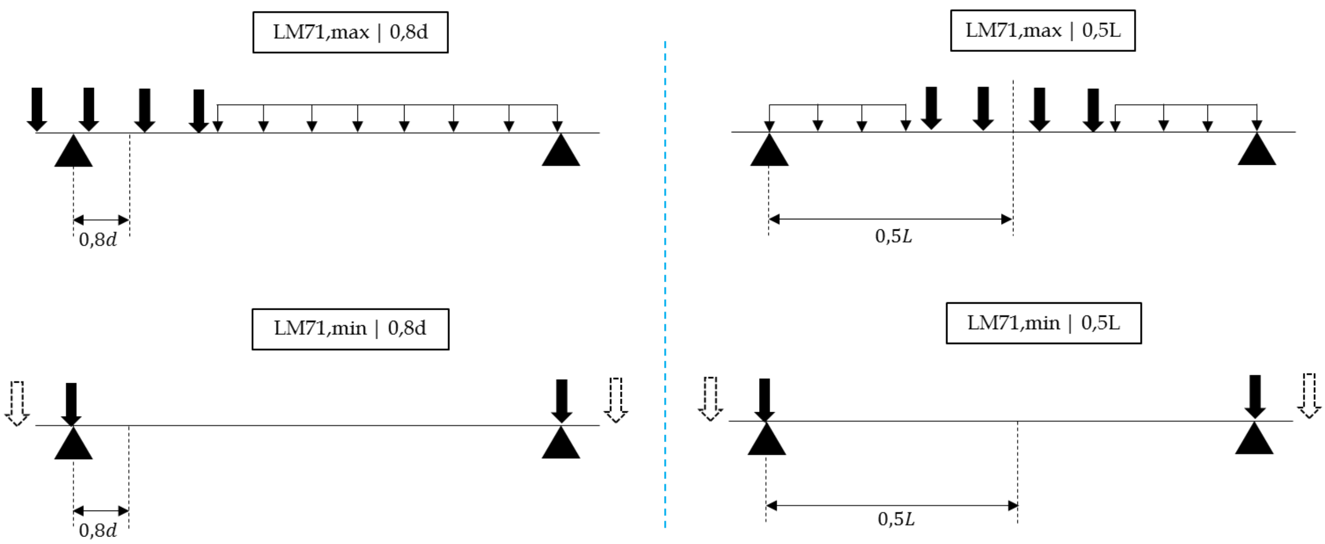


Figure 35: Maximum and minimum load combination at 0,8d (left) and 0,5L (right)

6.3 Concrete

Regardless of whether a structure is cracked or uncracked, the Eurocode states fatigue resistance calculations on concrete under compression need to be performed. The first step is to combine torsion, clamping moment, suspension and shear force into one total shear force (Appendix D). Considering that the total shear force has two reactional forces at the start and the end of the diagonal (Figure 36) the total bending moment can be expressed by:

$$M_{Ed,tot} = M_{Ed} + 0{,}5 * V_{Ed} * z * \cot\theta_{fat} \qquad [20]$$

According to Eurocode 2 (16) the angle of the compressive diagonal used in the ultimit limit state calculation maybe converted into θ_{fat} with equation [21].

$$\tan\theta_{fat} = \sqrt{\tan\theta} \qquad [21]$$

The angle of the compressive diagonal, under fatigue loading, becomes respectively 32,3° and 45° for a section near the support and at midspan.

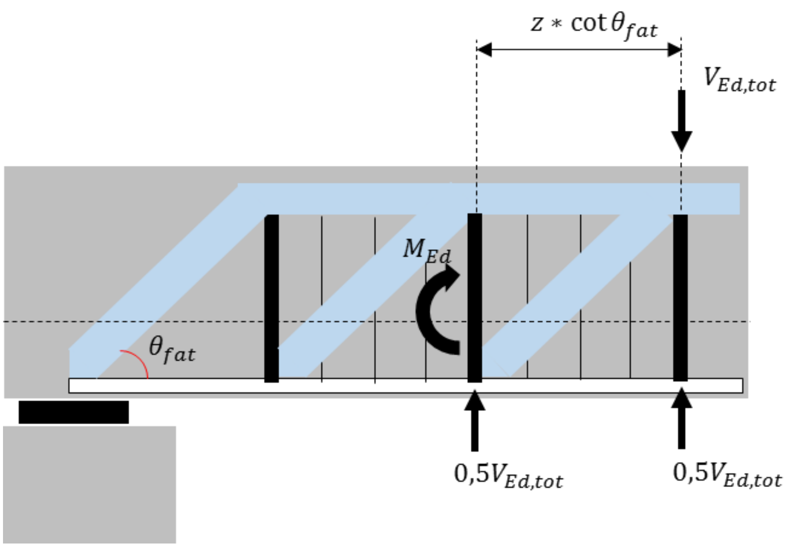


Figure 36: Total bending moment in the girder and angle of the compressive diagonal under fatigue loading

For the uncracked girder A, the maximum compressive stress is found at the top fibre. With equation [22] the stresses are computed for the maximum, minimum and permanent characteristic load combination.

$$\sigma_{c,top} = -\frac{P_\infty}{A_c} - \frac{M_{Ed,tot}}{W_{top}} \qquad [22]$$

Girder B is cracked meaning the tensile strength of the concrete is reduced to zero and the additional tensile forces are transferred by the longitudinal reinforcement and prestressing steel. The cracked girder approach maintains two unknown variables, the height of the compression zone and the concrete strain. Based on the height two solutions are possible:

1. $x < h$: A part of the cross-section is loaded in compression and the other in tension. The tensile stresses are transferred by the prestress and longitudinal reinforcement. Based on the concrete strain the stress fluctuations in the concrete, longitudinal reinforcement and tendons can be determined.

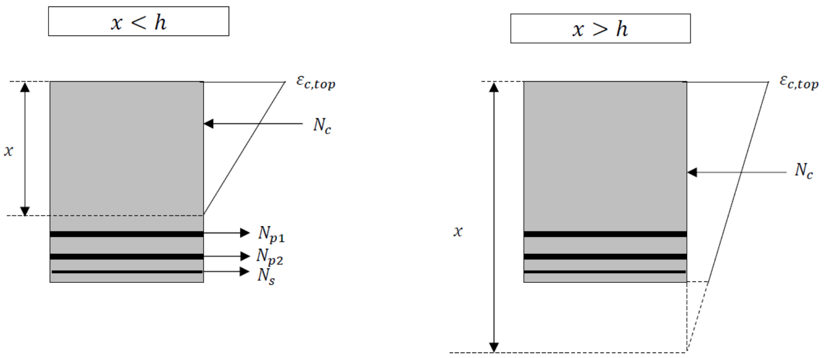


Figure 37: Compression zone smaller (left) or larger (right) than the cross-sectional height

2. $x > h$: The entire cross-section is under compression and there are no (additional) tensile forces in the prestress and longitudinal reinforcement. By interpolating on the graph in Figure 38 the compressive stress in the top fibre is obtained.

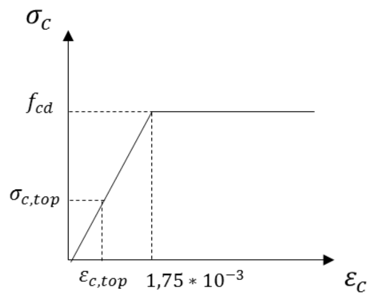


Figure 38: Stress-strain diagram for C35/45

Ultimately the maximum, minimum and permanent load combinations of the fatigue verification result in the second solution for girder B. In other words, when the girder is loaded in fatigue the entire cross-section is under compression, like girder A. No fatigue calculation has to be performed on the prestressing steel and longitudinal reinforcement because for both girders they are under compression. Yet the concrete stress fluctuation in the top fibre of the girder still need to be determined by interpolating on the stress-strain diagram in Figure 38.

Eventually the damage equivalent stress method from annex NN.3.2 in Eurocode 2 is applied. The stress range due to mobile loading is determined and corrected with a factor λ_c. This factor takes into account the annual traffic volume, life span, number of railway tracks and the length of the bridge. The final check is presented in the form of equation [23].

$$14 * \frac{1 - E_{cd,max,equ}}{\sqrt{1 - R_{equ}}} \geq 6 \qquad [23]$$

The results of the fatigue verification calculations are presented in Table 15, from which it can be concluded that the most critical section is at midspan in bridge B. This section is subjected to large bending moments, which causes large compressive stress fluctuations at the top of the girder, which ultimately leads to the most critical unity check.

Table 15: Concrete fatigue resistance of girder A and B

Unity Check	Bridge A		Bridge B	
	U.C. @ 0,8d	U.C. @ 0,5L	U.C. @ support	U.C. @ 0,5L
$14 * \frac{1 - E_{cd,max,equ}}{\sqrt{1 - R_{equ}}} > 6$	13,71 > 6	9,36 > 6	22,82 > 6	**7,46 > 6**

6.4 Shear reinforcement

In principle, the strains in the shear reinforcement of uncracked girder A remain zero, making the reinforcement non-sensitive to fatigue. However for cracked girder B the opposite is true. But girder A could crack due to other external loading, such as thermal actions. For this calculation both girders are assumed to be cracked and the resistance against stress fluctuations in the shear reinforcement are investigated.

The NS-guideline 1015 defines a number of zones which have a specific load transferring function. Zone III (outer stirrup) transfers a combination of shear, torsion and the suspension loads, where zone II (inner stirrup) only transfers shear forces. But from paragraph 5.1 it is learned that a part of the suspension loads is transferred by the inner stirrup as well. With the maximum and minimum characteristic load combinations, the stress range in the outer and inner stirrups is determined.

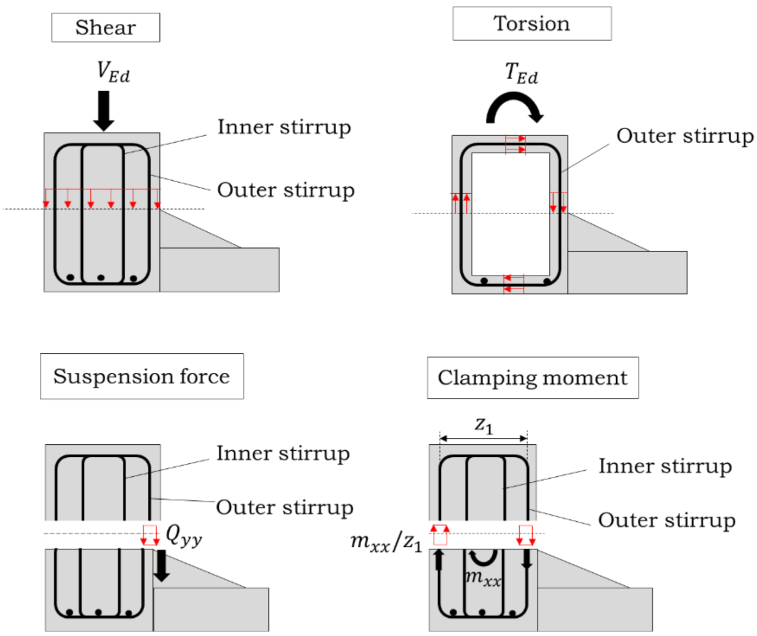


Figure 39: Stress in the outer and inner stirrup due to shear, torsion, suspension force and clamping moment

Annex NN.3.1 in Eurocode 2 is used to compute the damage equivalent stress in the stirrups. The stress range is multiplied with the dynamic factor and a correction factor. This last factor accounts for the annual traffic volume, life span, number of railway tracks and the length of the bridge.

$$\Delta\sigma_{s,equ} = \lambda_s * \Phi * \Delta\sigma_{s,71} \qquad [24]$$

The stress range is verified against a critical stress range at N^* cycles. The S-N curve of S435 steel is composed with help of section 6.8.4 in Eurocode 1. It becomes clear from the graph that the stress range should remain smaller than $\Delta\sigma_{critical} = 113{,}6\ N/mm^2$.

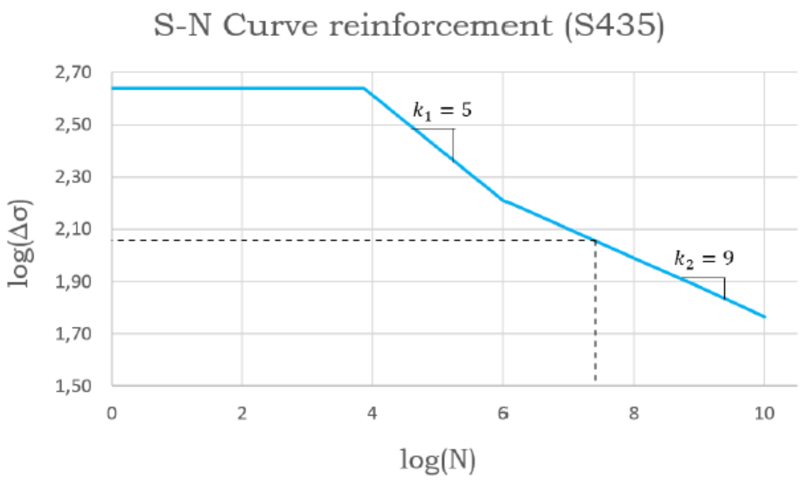


Figure 40: S-N curve for S435 steel

The unity check is determined by dividing the damage equivalent stress with the critical stress range. Additionally a partial load an material factor for fatigue are included.

$$U.C. = \frac{\gamma_{F,fat} * \Delta\sigma_{s,equ}(N^*)}{\Delta\sigma_{Rsk}(N^*)/\gamma_{s,fat}} \qquad [25]$$

The unity checks for shear reinforcement stay well within the acceptable limits (Table 16) and there is no risk of fatigue failure of the stirrups. The largest unity check is found at the inner stirrup at 0,8d. This can be explained from the fact that this stirrup is loaded with 35% of the suspension loads. These loads are mainly due to mobile loading, which impose a large stress fluctuation on the stirrup.

Table 16: Bridge A & B: Unity checks for fatigue in the shear reinforcement

Type of reinforcement	Bridge A		Bridge B	
	U.C. @ 0,8d	U.C @ 0,5L	U.C. @ support	U.C @ 0,5L
	$\theta = 21{,}8°$	$\theta = 45°$	$\theta = 21{,}8°$	$\theta = 45°$
Outer stirrup	0,50	0,22	**0,51**	0,19
Inner stirrup	**0,75**	0,21		

Since it can be confusing which calculation procedure is applied on the bridges, an overview is presented in Figure 41. The difference between bridge A and B is that the latter one has to be assumes as cracked, because it does not fulfil the requirements of the OVS. Consequently this changes the approach on determining the stress fluctuations in concrete and reinforcement.

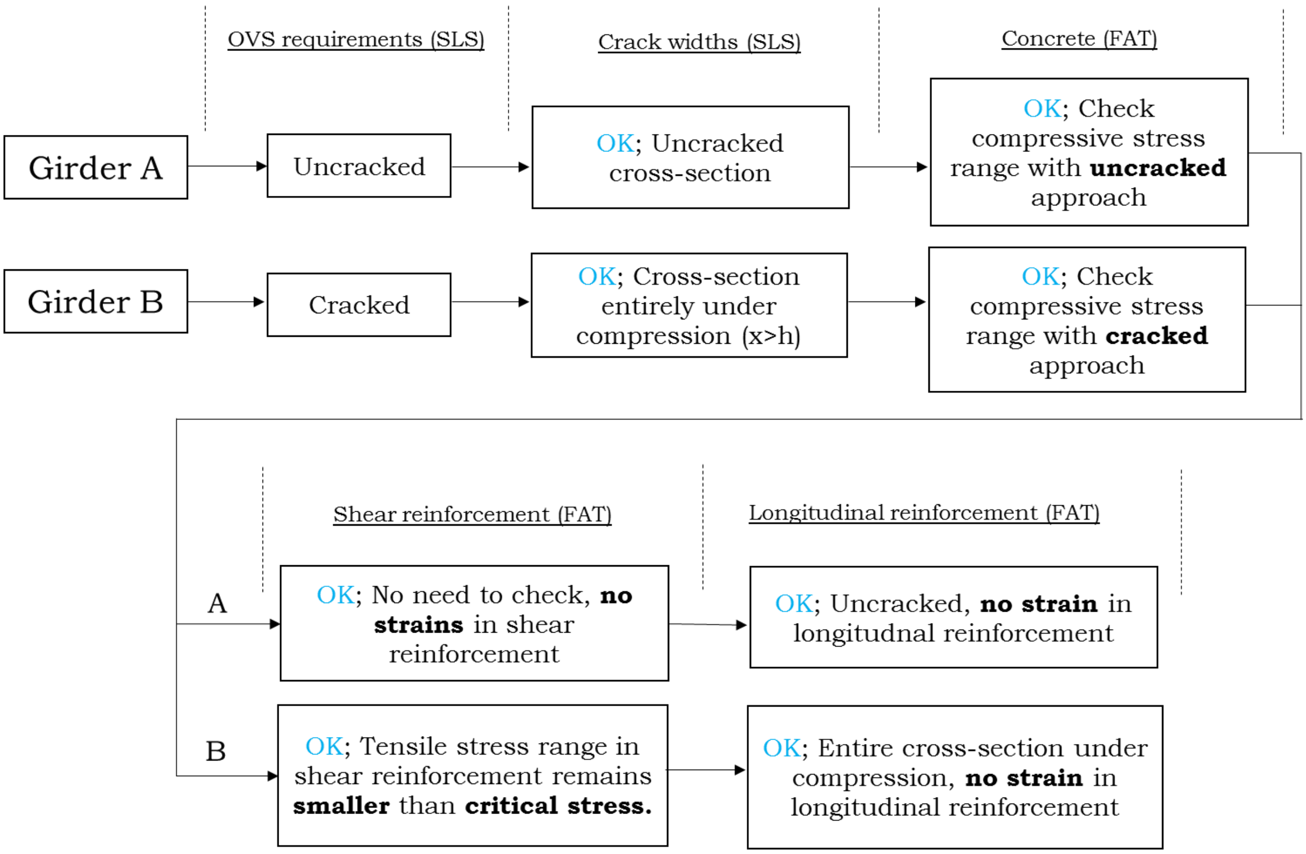


Figure 41: Overview of Eurocode fatigue calculations on bridge A and B

6.5 VB 74 & NS-Guideline 1016

The NS-Guideline 1016 (18) is used in addition to VB 74 and focuses on fatigue resistance calculations in railway structures. As described by this guideline, back in the day there was little knowledge on fatigue and especially on concrete fatigue resistance. This paragraph gives a brief overview of the available calculations in the guideline and the ones that are applied in the design reports.

6.5.1 Shear and longitudinal reinforcement

The calculations are supposed to be performed with real train loads. However a couple of coefficients are introduced, in order to keep on using the same load models as in the strength verification calculations (VOSB 150). Ultimately the stress fluctuation due to the VOSB load model should remain smaller than the critical stress range:

$$\Delta\sigma_{VOSB} \leq \Delta\sigma_{crit} \qquad [26]$$

The critical stress range for a VOSB load model is determined by equation [27]:

$$\Delta\sigma_{crit} = \frac{\Delta f_{ak} * k_A * k_N}{\gamma_{fat}} * \frac{1}{k_T * \lambda_T * \beta_T} \qquad [27]$$

Where:

Δf_{ak}	=	The steel fatigue resistance determined with experiments
k_A	=	Reduction factor for welds and arcs
k_N	=	Factor accounting for the number of axle loads that deviate from the total number of axle loads
γ_{fat}	=	Safety material factor (for steel equal to 1,4)
λ_T	=	Factor that takes the span of the bridge into account
k_T	=	Factor that takes into account either heavier or lighter railway traffic
β_T	=	Factor accounting for the number of railway tracks

In the design report, equation [27] results in a critical stress range of 182,2 N/mm^2, which is significantly higher than the acceptable critical stress range in the Eurocode which holds a value of 113,6 N/mm^2. Furthermore the fatigue calculation in the Eurocode is performed at ultimate limit state, but with all partial load factors taken equal to 1,0, whereas the VB 74 determines the stress range due to the VOSB load model at serviceability limit state.

6.5.2 Concrete

As mentioned before, the knowledge on the behaviour of concrete under fatigue loading, was at the time of the VB 74 very limited. The result is a limitation of the maximum tensile and compressive concrete stress at serviceability limit state (by the NS-guideline):

Maximum tensile principal stress under fatigue loading:

$$\rho_1 \leq 0{,}6 f_b \qquad [28]$$

Where:

ρ_1	=	Maximum tensile principal stress
f_b	=	Concrete tensile strength

Maximum compressive stress under fatigue loading:

$$f'_{bfat} = 0{,}4 * f'_{b} + 0{,}35 * \sigma'_{b,min} \leq 0{,}6 * f'_b \qquad [29]$$

Where:

f'_{bfat}	=	Maximum compressive stress under fatigue loading
σ'_{bmin}	=	Minimum compressive stress in the considered fibre, in case of cracking take $\sigma'_{bmin} = 0$
f'_b	=	Concrete compressive strength

There are two differences between the Eurocode and the NS-Guideline. Firstly, the guideline assumes that concrete under fatigue loading has some resistance against tension, whereas the Eurocode assumes it has none. Secondly, the fatigue resistance of concrete under compression in the guideline is a simple stress limitation (due to a lack of knowledge), whereas the Eurocode holds a damage equivalent stress check. And even though a stress limitation for compression is defined by the NS-guideline, it is not checked in the design reports.

The overall conclusion which can be drawn is that, due to more knowledge on fatigue, the Eurocode has more extensive and conservative fatigue resistance calculations than a combination of the VB 74 and the NS-guideline.

7 SCIA & Analytical Solution

The design or reassessment of a concrete through railway bridge is a typical assignment for an engineering firm such as Witteveen+Bos. Nowadays the design loads on a bridge are determined using finite element programs rather than performing time-consuming hand calculations. The risk however with FE models is that one needs as well a proper understanding of the program as the structural behaviour, in order to obtain correct results. Additionally a good structural engineer has a critical attitude against FEA-generated results.

In the case of Witteveen+Bos, the FEA program *SCIA Engineer* is used to determine the critical design loads in the bridge. Because engineers rely on the results generated by SCIA, this chapter holds a comparison between SCIA and the analytical solution with a focus on torsion in the girder. The ultimate goal is to establish the deviation between SCIA and the analytical solution (w.r.t. to torsion) for the governing load combination. A more detailed version of this comparison can be found in appendix G.

7.1 Earlier research

R.T.J. de Groot (4) compared the results from the analytical solution with a 2D and 3D DIANA model. Based on this research the following can be concluded:

- Torsion in the girder strongly depends on the E-modulus of the floor in transverse direction. By reducing the E-modulus of the floor, torsion in the girder is increased.
- When the girder is connected to the floor, there is a vertical eccentricity between the two corresponding nodes. This eccentricity causes an additional torsional moment which influences the results. A horizontal connection between these two elements is advised.
- Fully prestressed structures are hardly exposed to torsion due to self-weight. This is because prestress counteracts the deflections and rotations caused by self-weight. But for structures with partial prestressing, torsion due to self-weight may become an issue.
- The connection between the girder and the floor turns out to have a large influence on the results. By using solid-elements (3D) the exact geometry can be modelled and this problem is excluded. Under a double line load of 80 kN/m, the values for torsion in a 3D and 2D model remain respectively 30% and 50% behind the analytical solution.

7.2 Plate model

Three types of models are possible when modelling a through bridge in SCIA. The first one is a plate model, which is entirely constructed of 2D-elements. The floor and the voute are modelled in XY-plane and because these elements have overlapping nodes they are automatically connected by SCIA. The girder is modelled in XZ-plane, which means the centreline runs in a different direction than the centreline of the floor and voute. To solve this problem, NS-Guideline 1015 advises to use rigid connections between the centreline of the girder and the edge of the voute. In order to establish smooth graphs for torsion, the rigid connection is applied every 0,25 m to let it coincide with the mesh, which consists of squares of 0,25 m.

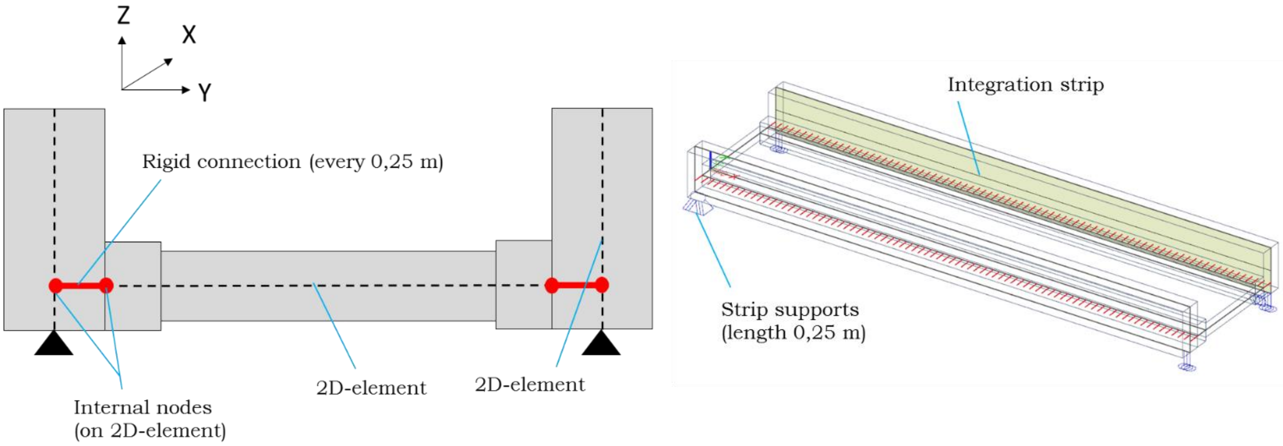


Figure 42: Cross-section and 3D-view of the plate model

A rigid connection in SCIA means rotations of the connected nodes are identical. Additionally the rotation determines the orientation of the connection line. The deformation of the nodes are identical as well, but due to the rotation node 2 undergoes an additional deformation of $0{,}5b * \varphi$.

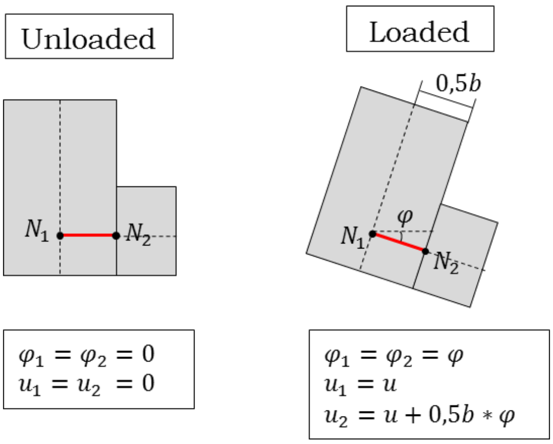


Figure 43: Deformation behaviour of a rigid connection

7.3 Beam model 1A

Beam model 1A consists of a floor, modelled as a 2D-element, which spans between the centrelines of the girder. The girder is modelled as a 1D-element on the long edge of the floor. The 2D-element has 4 nodes (one on each corner), whereas the 1D-element has 2 nodes (one at the start and the end of the girder). SCIA automatically connects overlapping nodes, which means a connection between the girder and floor is established at each corner of the structure. The concern however arises that the rotations and deformations of the floor and girder are only coupled at the corners of the structure and not along the entire length. To verify whether this is the case or not, an alternative beam model is introduced in paragraph 7.4 which applies rigid connections between the 1D and 2D-element.

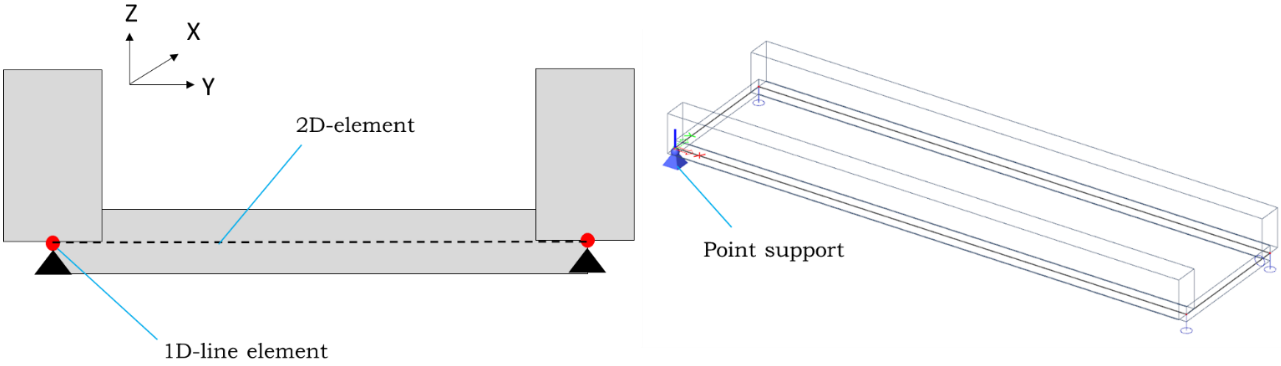


Figure 44: Cross-section and 3D-view of beam model 1A

The dimensions entered in SCIA are simplified compared to the original cross-section. A simple calculation (in appendix G) shows that the cross-sectional area is exactly the same in SCIA and the sectional modulus is an insignificantly 5,6% smaller than the original cross-section. Yet the torsional stiffness of the girder is 29,7% less in SCIA than the original torsional stiffness. Therefore the shear modulus of the girder is manually adjusted in SCIA.

7.4 Beam model 1B

Beam model 1B forms a combination of the plate model and beam model 1A. The floor and voute are modelled as in the plate model (2D) where the girder is modelled as a line-element (1D). But in contrast to model 1A, the centreline of the girder does not coincide with the centre of the floor. This means that rigid connections between the 1D and 2D-element need to be applied to form a connection between these two elements. To do so, internal nodes are added to the 1D and 2D-elements and rigid connections are formed every ¼ meter. An advantage of this model is that the geometry is no longer simplified compared to the real cross-section and that the shear modulus does not need to be increased manually. However a disadvantage is that the connections formed have a vertical eccentricity, which according to de Groot can induce additional bending and torsional moments.

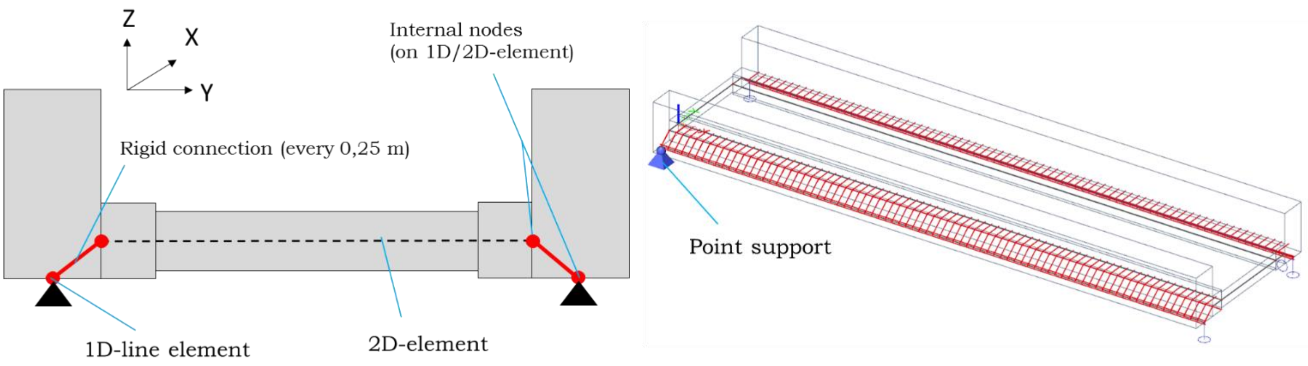


Figure 45: Cross-section and 3D-view of beam model 1B

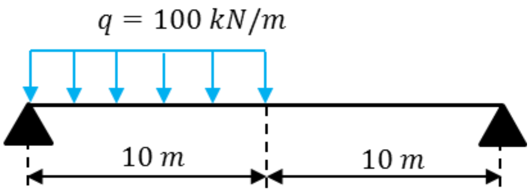
All three models are evaluated using the Mindlin theory for 2D-elements. In contrast to Kirchhoff, the Mindlin theory is applicable to thick plates and takes shear deformation into account. In order to get correct values for torsion it is essential that shear deformation in the 2D-elements is taken into account.

7.5 Torsion

In this paragraph torsion in the girder is established for the analytical solution, plate and beam models. A number of load cases are reviewed for a bridge with a span of 20 meters and the cross-sectional properties of bridge A.

7.5.1 Distributed mobile load

In Figure 46 torsion along the length of the girder is plotted for the analytical solution, plate and beam model 1A and 1B. Noteworthy is that beam model 1B and the plate model follow roughly the same course, where beam model 1A stays a bit behind. Even more controversial is the maximum torsion found by the analytical solution is more than twice as large as the maximum value obtained by SCIA.



Property	Girder	Floor
E	$34.000\ MPa$	$34.000\ MPa$

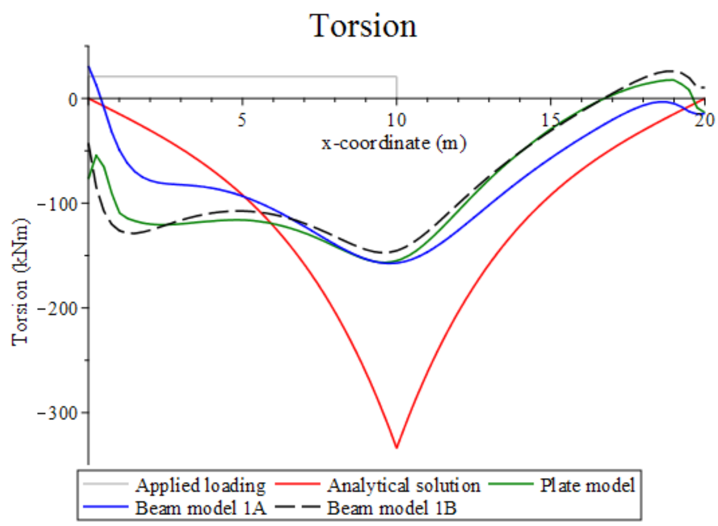


Figure 46: Double line load of 100 kN/m (left) and torsion graphs with uncracked floor (right)

During the derivation of the analytical solution a number of assumptions are made (paragraph 3.3.2). One of the most important ones is that the floor is divided into strips with a length $dx = 1{,}0\ m$. This assumption is processed in the SCIA models by reducing the E-modulus of the floor to $11.200\ MPa$. The floor could thereby be considered as cracked, resulting in a load distribution in the floor comparable with the strip method. This results in graphs for torsion presented by Figure 47. In order to keep a fair comparison the girder and floor will have an E-modulus of $34.000\ MPa$ and $11.200\ MPa$ from this point forward.

Property	Girder	Floor
E	$34.000\ MPa$	$11.200\ MPa$

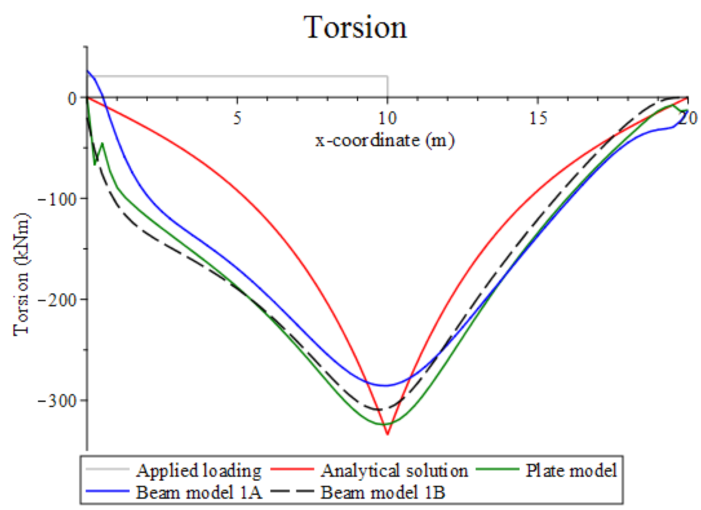


Figure 47: Torsion graphs for a double line load of 100 kN/m (cracked floor)

7.5.2 Local mobile load

A double local line load causes torsion in the girder as displayed by Figure 48. The plate and beam model 1B compare rather well to the analytical solution and the maximum torsion obtained is roughly the same. Contrastingly beam model 1A stays behind and has a maximum which is 40% lower. It is expected that the connection established between the 1D and 2D-element in model 1A results in a loss of stiffness.

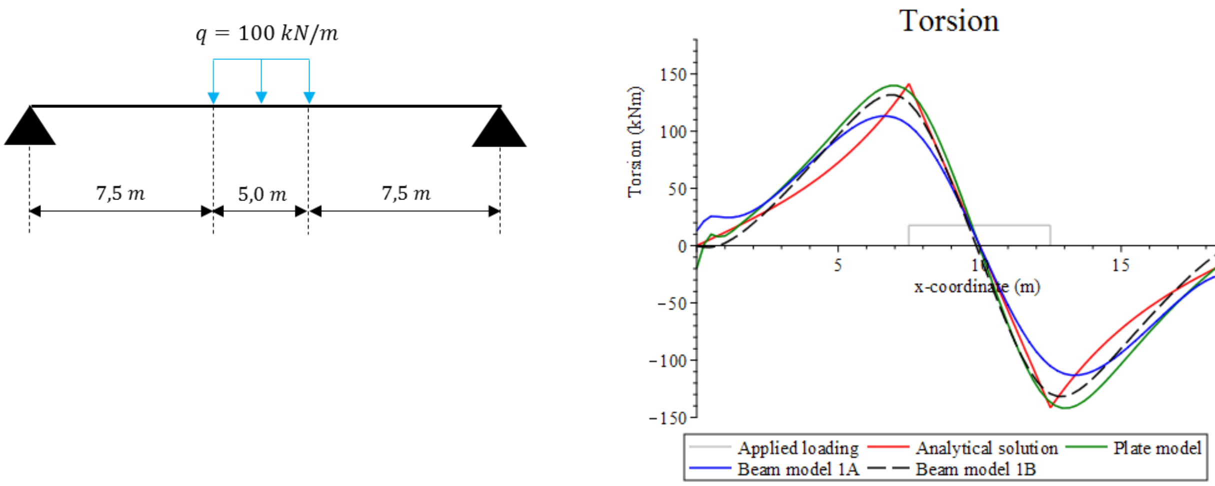


Figure 48: Torsion graphs for a double local line load of a 100 kN/m (cracked floor)

7.5.3 Self-weight

The analytical solution assumes there is no torsion due to self-weight. The driving idea behind that is that a constant load generates a constant deflection and rotation. Consequently no change in rotation means no torsion. SCIA however shows there is torsion due to self-weight, which can be explained by the fact that at midspan the deflections and rotations are larger than near the supports. An alternative load case, which results in the same deflection behaviour, is applied for the analytical solution.

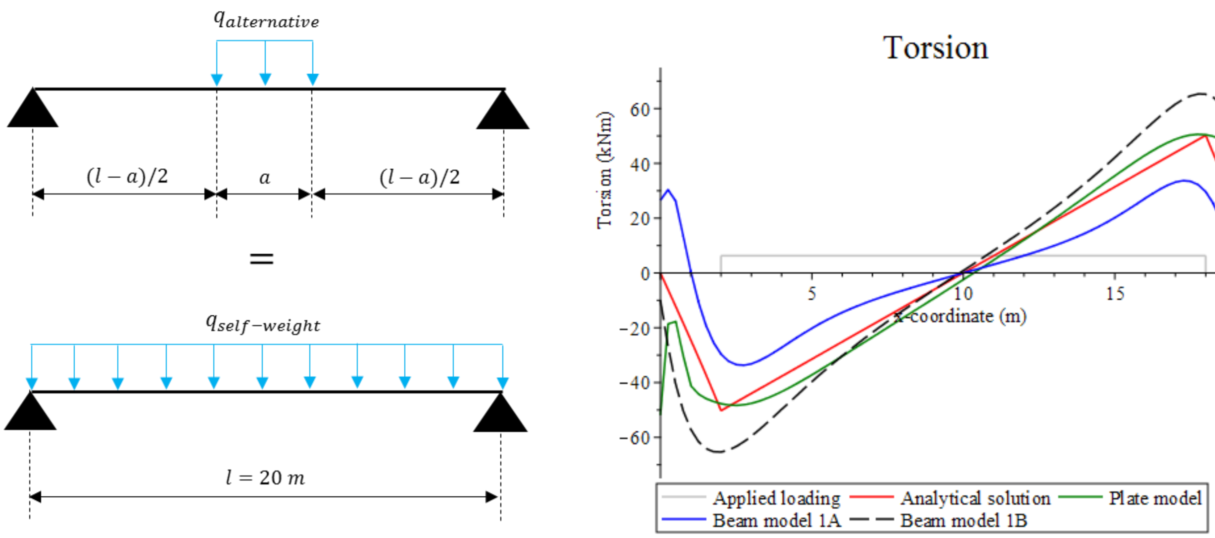


Figure 49: Torsion graphs for self-weight (cracked floor)

Figure 49 indicates that the plate model and analytical solution follow the same course and have roughly the same maximum. On contrary beam model 1A remains far behind, where beam model 1B presents a graph which exceeds the analytical solution. This last phenomenon is remarkable, as model 1B is expected to follow the same course as the plate model. But because point supports are applied, large reactional forces are present near the end of the bridge increasing the values for torsion. Unfortunately strip methods are no better solution, because they induce a large counteracting bending moment near the supports which influence the rotations and therefore torsion in the girder.

7.5.4 Prestress

Prestress consists of a horizontal force, an upward acting distributed load and a bending moment due to an eccentricity of the tendons. Because torsion is only generated by deflection of the bridge and rotation of the girders, the horizontal force is excluded from this calculation. Similar to self-weight, an alternative load case needs to be applied in order to find an analytically determined graph for torsion.

Figure 50 presents the graphs for torsion due to prestress. The results are quite similar to torsion due to self-weight, but with an opposite sign. The conclusion earlier drawn by R.T.J. de Groot can be confirmed; fully prestressed structures are hardly exposed to torsion due to self-weight.

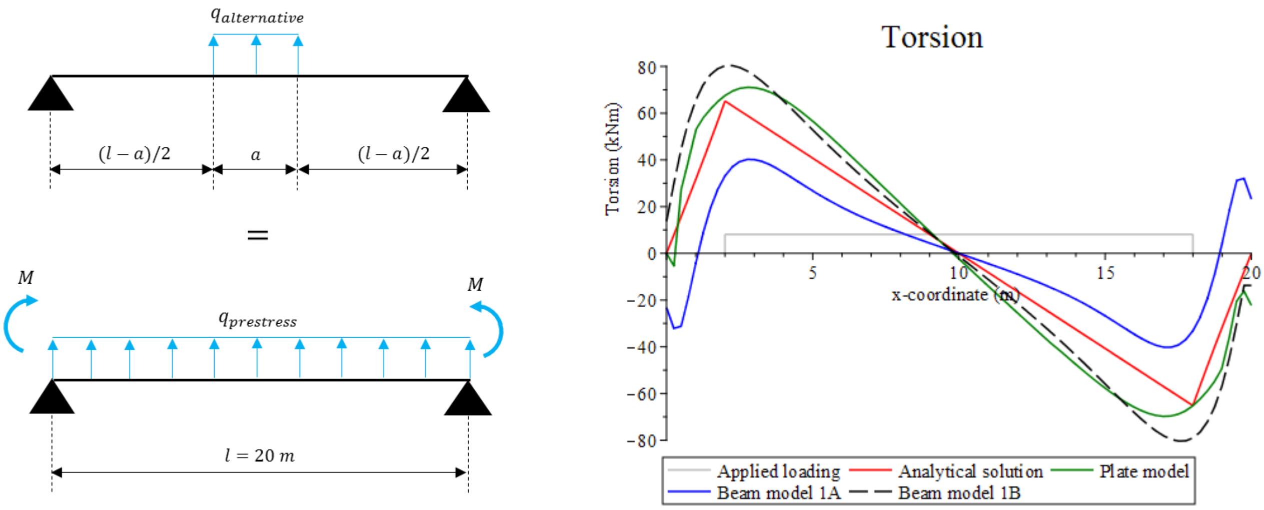
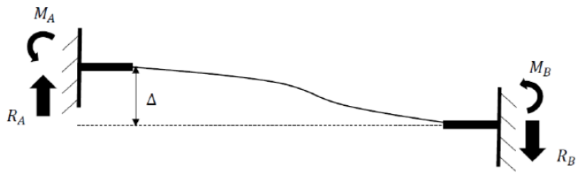


Figure 50: Torsion graphs for prestress (cracked floor)

7.5.5 Support settlement

It can be likely that one of the supports settles more than others. Therefore a load case is introduced which takes a settlement of $5\,mm$ into account. The analytical solution assumes no load distribution in longitudinal direction and considers a strip with length $dx = 1{,}0\,m$ and a deflection Δ. From the plate and beam models it becomes clear that the support settlement is not only distributed in transverse direction but also in longitudinal direction. The analytical solution is thus too simplistic and therefore too conservative (Figure 51).



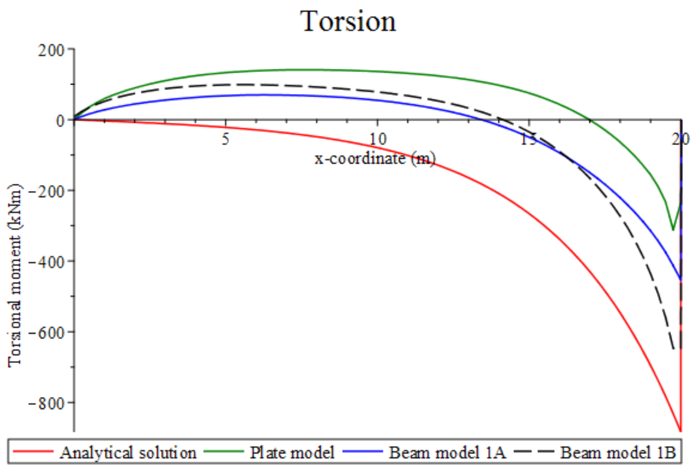


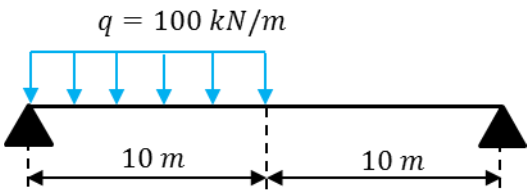
Figure 51: Transverse section support settlement (left) and torsion graphs modelled with cracked floor (right)

7.6 Clamping moment

Besides torsion the clamping moment is an important design load. In the case of Witteveen+Bos SCIA is used to determine the clamping moments in the girder. With the functions derived in appendix A, a comparison between the analytical solution and SCIA is drawn up.

7.6.1 Distributed mobile load

A load case as depicted by Figure 52 is considered. Due to this load case the clamping moment shows a sign switch at the end of the distributed mobile load. The explanation for this lies in the fact that the clamping moment under loading is counteracted by the unloaded part of the bridge, which causes the sign switch. From the right image of Figure 52 it can be concluded that the plate and beam model 1A contain this jump as well, but that the maximum values for the clamping moment remain behind. Contrastingly beam model 1B shows a different course for the clamping moment, which can be explained from the manner in which this model is constructed.



Property	Girder	Floor
E	$34.000\ MPa$	$11.200\ MPa$

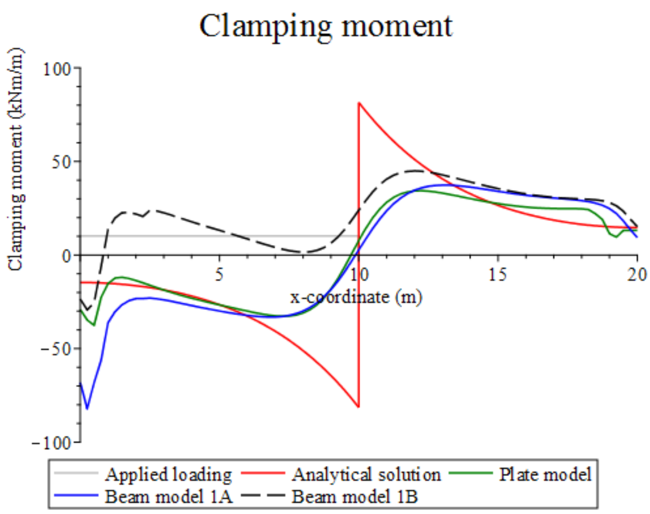


Figure 52: Double line load of 100 kN/m (left) and clamping moment graphs (right)

Since the clamping moment can only be derived from a 2D-element, section A-A (Figure 53) in the plate model and section B-B in the beam model 1B are used to find values for the clamping moment. However the analytical solution assumes the floor is fully restrained in the centre of the girders, which means the clamping moment in model 1B is not derived from the correct location resulting in a different course.

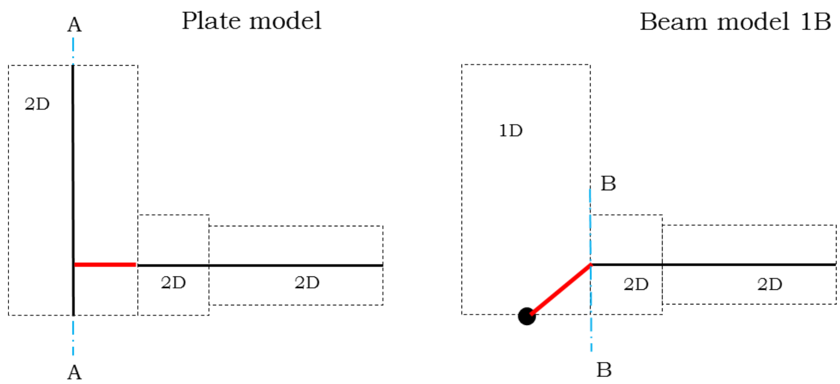


Figure 53: Sections used in plate model and beam model 1B to determine clamping moments

The assumption of the analytical solution, that the bending stiffness of the floor in longitudinal direction is negligible, is incorporated in the SCIA models. The floor is modelled orthotropic with an E-modulus in transverse and longitudinal direction of respectively 11.200 MPa and 100 MPa. The jump becomes better visible and the maximum clamping moment corresponds better with the analytical solution (Figure 54).

Property	Girder	Floor
E_{long}	$34.000\ MPa$	$100\ MPa$
E_{trans}	$34.000\ MPa$	$11.200\ MPa$

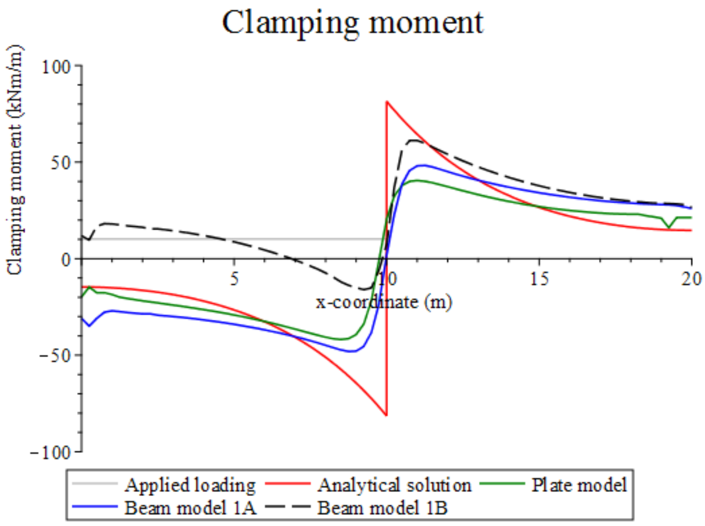
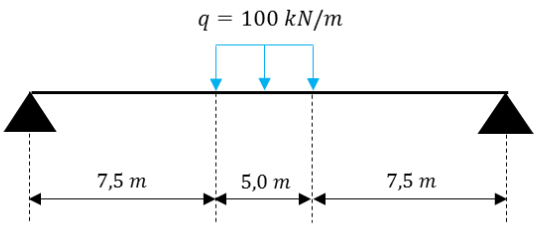


Figure 54: Clamping moment graphs for a double line load of 100 kN/m (orthotropic)

7.6.2 Local mobile load

A local mobile load generates graphs for the clamping moment as presented by Figure 55. Similar to the previous load case, the floor is modelled orthotropic with a longitudinal bending stiffness of a $100\ MPa$. Under loading the clamping moments in the plate and beam model 1A are larger than in the analytical solution. Unfortunately no direct explanation is found for the fact that these models deliver larger values than the analytical solution. But in practice, the bending stiffness of the floor (in a SCIA model) will not be reduced to a $100\ MPa$. Which means there will always be some load transfer in longitudinal direction, resulting in less conservative values for the clamping moment. From this point of view the analytical solution will form a safe upper limit. Beam model 1B results in values which remain smaller than the analytical solution, but no conclusions can be drawn from this fact since the results are obtained at a different location.



Property	Girder	Floor
E_{long}	$34.000\ MPa$	$100\ MPa$
E_{trans}	$34.000\ MPa$	$11.200\ MPa$

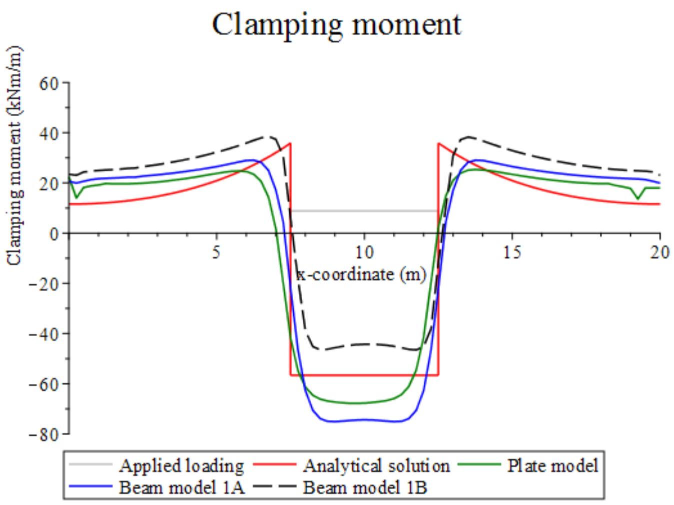


Figure 55: Clamping moment graphs for a local line load of 100 kN/m (orthotropic)

7.7 Deviation of SCIA

The objective of this chapter is to draw up a comparison between the analytical solution and SCIA, with a focus on torsion in the through girder. The deviation of SCIA (w.r.t. the analytical solution) is established by considering the governing load combination of self-weight, ballast, LM71 and prestress acting on bridge A and B. Since the support settlement load case gives unrealistically high values for torsion in the analytical solution, it is not taken into account in this calculation. Additionally the cross-sectional properties remain constant throughout this research. Which means the girder is assumed uncracked ($E = 34.000\ MPa$) and the floor is assumed cracked ($E = 11.200\ MPa$).

The difference in torsion between SCIA and the analytical solution is established at a section at 0,8d. But the values for torsion in this section can deviate, due to the influence of the supports. Therefore it is decided to also take into account the maximum torsion in the through girder, which is not bound to a specific location.

Table 17: Bridge A: The total deviation of maximum torsion in the plate and beam model 1A & 1B

LC	Type	% of Total loading	Deviation to analytical solution (%)		
			Plate	Beam	
				1A	1B
1	Self-weight	20,7%	-10%	-14%	-7%
2	Ballast	19,3%	-5%	-9%	-4%
3+5a	Load model 71	48,4%	0%	-10%	-2%
9	Prestress	11,6%	0%	-5%	2%
	Total	100,0%	**-16%**	**-38%**	**-11%**

The results in Table 17 describe the difference in maximum torsion between the analytical solution and the plate, beam model 1A and 1B in bridge A. To find these percentages, one needs two values per load case. Namely the difference in maximum torsion (w.r.t. the analytical solution) of each single load case and the contribution to the total amount of torsion. By multiplying these two factors an weighed average is found, which can be summoned to a total deviation per model.

Table 18: Bridge A & B: The total deviation of torsion in the plate and beam model 1A & 1B at 0,8d

LC	Bridge A - Deviation in torsion (%)			Bridge B - Deviation in torsion (%)		
	Plate	Beam		Plate	Beam	
		1A	1B		1A	1B
Self-weight	-9%	-15%	-4%	-10%	-23%	-8%
Ballast	0%	-5%	0%	0%	-7%	1%
Load model 71	-9%	-17%	-3%	6%	-5%	13%
Prestress	5%	-1%	11%	1%	-4%	11%
Total deviation [%]	**-14%**	**-39%**	**4%**	**-4%**	**-39%**	**17%**

Table 18 presents the deviations per model at 0,8d whereas Table 19 presents the deviations per model for maximum torsion (in bridge A & B). The first impression one gets, is that the results from the plate model and beam model 1B are closer to the analytical solution than the results from the beam model 1A. One can also conclude, by taking a closer look at Table 18, that there is a large difference between the analytical solution and beam model 1B for the prestress load case. Because the results in this table are generated at 0,8d, the supports are likely to have an influence. Therefore more value is attached to Table 19 which considers maximum torsion in the girder and is therefore not bound to a specific location.

Table 19: Bridge A & B: The total deviation of maximum torsion in the plate and beam model 1A & 1B

LC	Bridge A - Deviation in torsion (%)			Bridge B - Deviation in torsion (%)		
	Plate	Beam		Plate	Beam	
		1A	1B		1A	1B
Self-weight	-10%	-14%	-7%	-9%	-17%	-4%
Ballast	-5%	-9%	-4%	-2%	-6%	-2%
Load model 71	0%	-10%	-2%	2%	-9%	1%
Prestress	0%	-5%	2%	-1%	-9%	1%
Total deviation [%]	**-16%**	**-38%**	**-11%**	**-9%**	**-41%**	**-4%**

Respectively the total deviation of the plate, beam model 1A and 1B goes to 10-15%, 40% and 10% (as seen in Table 18 and Table 19). At first it looks like beam model 1B approximates the analytical solution slightly better than the plate model. But with the knowledge from paragraph 7.5 the contrary can be concluded. This is due to the fact that for self-weight and prestress beam model 1B exceeds the analytical solution and thereby compensates for its exactly larger deviation from the analytical solution.

Beam model 1A shows a deviation of 40% from the analytical solution. The most plausible explanation for this lies in the fact that no rigid connections are applied in beam model 1A, resulting in a loss of bending and torsional stiffness at the 1D (girder) and 2D (floor) interface.

R.T.J. de Groot concluded that 2D and 3D models of a through bridge remain respectively 50% and 30% behind on the analytical solution. However it should be noted that de Groot considered statically undetermined structures with end-cross members, where this thesis focuses on statically determined structures without end-cross members. Additionally de Groot was looking for the best possible FE model for a through bridge rather than the deviation between the theory and a FE model. Which means both researches have a different scope and purpose and cannot be compared one on one.

Finally one should realize that the analytical solution forms a safe upper limit when calculating torsion in a through bridge. The strip method and the negligence of the longitudinal bending stiffness of the floor are quite conservative assumptions. In reality a load on the floor will not only spread in transverse but also in longitudinal direction.

8 Conclusions & recommendations

8.1 Conclusions

The first research question of this master thesis focuses on the shear resistance of two fully prestressed through bridges, which are designed according to the VB 74. Three shear resistance calculations, according to the Eurocode, are performed to verify whether or not the structural safety of the two existing through bridges is compromised:

Shear tension failure:

- The Eurocode verifies the risk of shear tension failure at ultimate limit state by determining the maximum tensile principal stress. If this stress exceeds the concrete tensile design strength (f_{ctd}), shear tension failure in the girder will occur. It turns out the governing unity check is found for a section at 0,8d in bridge A. In this section the maximum load combination due to LM71 is governing and leads a unity check of 0,97. Especially the combination of shear and torsion becomes critical for a section which is close to the supports.

Reinforcement capacity:

- In the Eurocode the capacity of the reinforcement is verified at ultimate limit state. During the reassessment it is concluded that the outer stirrup of bridge A at a section 0,8d turns out to hold the governing unity check of 1,01. According to NS-Guideline 1015 this stirrup should be able to withstand a combination of shear, torsion and suspension loads, which at 0,8d reaches a maximum. The unity check is assumed to be just within the acceptable limits and therefore it is concluded that sufficient shear reinforcement is applied in both bridges.
- During the ULS reinforcement calculation the girder of the bridge is assumed to be cracked. This changes the internal force distribution, where the tensile strength of concrete is reduced to zero, and possible tensile stresses are transferred by a combination of longitudinal reinforcement and tendons. Ultimately a maximum unity check for the longitudinal reinforcement is found in bridge B at midspan. With a value of 0,56 it can be concluded sufficient longitudinal reinforcement is applied in both bridges.
- The maximum shear resistance of the through bridge is governed by the capacity of the compressive diagonal in the girders. With the arrival of the Eurocode it is possible to vary the angle of the diagonal between 21,8° and 45°, which means large loads can be applied on the diagonal when small angels are chosen. From the maximum unity check of 0,59 in bridge A, it can be concluded that the compressive diagonals in both bridges have sufficient capacity.

The OVS holds a number of stress limitations in serviceability limit state which are applicable to newly constructed railway bridges. These stress limitations are applied on the two existing through bridges, in order to verify whether the girder is cracked or not. Bridge A turns out to fulfil the requirements and remains uncracked, where bridge B must be assumed as cracked.

Fatigue:

- The fatigue resistance calculations in the Eurocode focus on the compressive and tensile stress fluctuations in the concrete and reinforcement respectively. But if the girders remain uncracked, no strains are present in the reinforcement, meaning only fatigue resistance calculations need to be performed on concrete. But girder A is assumed to be cracked as well, in order to verify if the reinforcement has enough fatigue resistance in case the girder cracks due to other external loads (e.g. thermal actions). Using the damage equivalent stress approach and an ULS characteristic load combination, the following can be concluded:
    - Concrete: The maximum unity check for concrete is obtained in bridge B at midspan. The large bending moments in this section generate large compressive stress fluctuations in the top fibre of the girder.
    - Shear reinforcement: The inner stirrup in bridge A, mainly loaded due to suspension loads, holds with a value of 0,75 the largest unity check for fatigue. Evidently the suspension loads are mainly due to mobile loading, increasing the stress fluctuations on these stirrups.
    - Longitudinal reinforcement: Bridge A is uncracked, meaning no strains and therefore tensile stress ranges are present in the longitudinal reinforcement. Bridge B is cracked but since the entire cross-section is under compression, the longitudinal reinforcement is not sensitive to fatigue as well.

Table 20: Unity checks for shear resistance calculations performed according to the Eurocode (A&B)

Shear resistance calculation	Eurocode Unity Check	
	Bridge A	Bridge B
A. Shear tension failure	0,97	0,77
B. Reinforcement capacity		
Shear reinforcement	**1,01**	0,90
Longitudinal reinforcement	0,26	0,56
Compressive diagonal	0,59	0,57
C. Fatigue	Uncracked	Cracked
Concrete	9,36>6	**7,46>6**
Shear reinforcement	0,75	0,51
Longitudinal reinforcement	0,00	0,00

The heavier load models and more conservative shear resistance calculations in the Eurocode, created the expectancy at the start of this research that a problem would arise concerning the shear resistance. The following two reasons explain why this not the case:

- Angle of the compressive diagonal (θ): In the Eurocode one has the freedom to vary the angle of the compressive diagonal between 21,8° and 45°, enabling one to optimize the capacity of the applied amount of concrete, reinforcement and prestress.
- Redistribution: The suspension loads need to be transferred by the outer stirrup (zone III) according to NS-guideline 1015. But since this is an ultimate limit state calculation, plasticity of the stirrups may be taken into account. Which means that if the ductile outer stirrup (FeB500) has insufficient capacity, it will start to yield and loading is transferred to the inner stirrup. The calculations in bridge A are therefore performed with a suspension load distribution of 65% and 35% over the outer and inner stirrup respectively.

The second research question focuses on drawing up a comparison between the analytical solution and SCIA, with a focus on torsion in the through girder. Based on the results in appendix G, the following conclusions can be drawn:

- Modelling the floor as cracked, by reducing the E-modulus to roughly a third ($E_{floor} = 11.200\ MPa$), makes the plate and beam models approximate the strip method of the analytical solution.
- As opposed to the analytical solution, self-weight and prestress do cause torsion in the girder. By making use of an alternative load case, a graph for torsion can even be found for the analytical solution. But one should keep in mind that for fully prestressed structures the deflections and rotations of these two load cases cancel each other out, resulting in hardly any torsion.
- A support settlement leads to maximum torsion at the location of settling which gradually decreases towards the unloaded support. In contrast to SCIA the analytical solution assumes no load spread in longitudinal direction for this load case, resulting in values for torsion which are too conservative.
- One of the assumptions during the derivation of the analytical solution is that the bending stiffness of the floor in longitudinal direction is negligible. In SCIA the floor is modelled as orthotropic with hardly any bending stiffness in longitudinal direction. This results in values for the clamping moment which compare rather well to the analytical solution for the plate and beam model 1A.
- A simply supported through bridge, loaded over half the length, shows increasing values for torsion up to a span of 30 meters.
- Increasing the girder height or width, makes it a relatively stiffer part of the structure. Stiffer parts of a structure attract more load, resulting in an (almost) linear relation between these dimensions and maximum obtained torsion.
- Applying a governing combination of self-weight, ballast, LM71 and prestress on bridge A and B, results in values for torsion which are 10-15%, 40% and 10% smaller than the analytical solution for respectively the plate, beam model 1A and 1B. However beam model 1B exceeds the analytical solution for the self-weight and prestress load case due to large reactional forces at the supports and therefore compensates for its deviation.
- R.T.J. de Groot concluded that a 2D and 3D model of a through bridge results in values for torsion which remain respectively 50% and 30% behind the analytical solution. However de Groot focussed on the best possible FE model and considered statically undetermined structures with end-cross members.
- There is a clear difference in deviation between beam model 1A and 1B. Because model 1A is constructed without rigid connections, there is a loss in bending and torsional stiffness of the girder, which results in a larger deviation from the analytical solution than model 1B.
- The analytical solution assumes the bridge to be divided into small strips and neglects the longitudinal bending stiffness of the floor. In reality loads will be as well distributed in longitudinal as transverse direction and the analytical solution therefore needs to be considered as a safe upper limit.

8.2 Recommendations

When *SCIA Engineer* is used to model a through bridge, it is recommended that the plate model is used. The reason for this is three folded. Firstly it only deviates 10-15% from the analytical solution. Secondly it has the ability to compute clamping moments at the centre of the girder, resulting in more accurate graphs for the clamping moment than beam model 1B (which obtains these values at the connection between girder and floor). And thirdly the plate model takes much less time to construct than beam model 1B.
But a 3D-model is the ultimate recommendation when modelling a through bridge, because the exact geometry can be modelled, leading to the most accurate results for torsion.

Because the scope of this research is limited to two fully prestressed through bridges which are simply supported, a number of recommendations is given below for further research:

- **Statically undetermined bridge**: This research completely focuses on a statically determined bridge. But a statically undetermined structure has an intermediate support which changes the course of the bending moment, shear force and torsion. Additionally the statically undetermined bridges usually have longer spans than the maximum span of 31,5 meters considered in this research. For further research it may be interesting to investigate the shear resistance of a fully prestressed statically undetermined through bridge. Furthermore it is advised to investigate the deviation between SCIA and the analytical solution and compare the result with statically determined bridges.

- **GBV 1962:** The GBV 1962 is the precursor of the VB 74. At the time of the GBV there was little knowledge on steel/concrete resistance against fatigue. Additionally the shear resistance calculations in the GBV are very different from the Eurocode. Since there are still several through railway bridges in the Netherlands which are designed according to the GBV 1962, it may be interesting to reassess the shear resistance of these structures in a research. Especially fatigue is ought to be critical in this reassessment.

- **3D modelling:** As concluded by R.T.J. de Groot, a 3D-model of a through bridge obtains the best results for torsion. It is highly recommended that, another graduate student models a simply supported fully prestressed through bridge in 3D and compares this to the analytical solution. Evidently it will be interesting to see if a 3D-model has indeed a better accuracy than a 2D-model. But one should keep in mind that it is not possible to create a 3D-model in SCIA, meaning a program like DIANA or ANSYS needs to be used.

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Appendix A - Torsion and clamping moment

1 Introduction

A passing train on a through bridge will lead to a local deflection of the floor. Because of the stiff connection between the floor and the girder, the loaded parts of the bridge will get a certain rotation. Yet the unloaded parts of the bridge have no intention to rotate and will counteract this rotation. This causes torsion in the through girder.

In chapter 2 a differential equation for torsion is derived. The solution to the equation is in a general form, containing two constants. In order to find a value for these constants, chapter 4 focuses on establishing the boundary conditions for a couple of basic load cases. Torsion along the length of the girder is presented in a number of graphs for these basic cases.
Because the floor is ought to be fully restrained in the heart of the girder, a reactional force and moment in the centre will be the result. The moment is the so-called clamping moment and is also of great importance in order to assess the critical loads in the through girder. A function for the moment can be found by taken the derivative of the torsional function. Chapter 5 considers the same basic load cases (as chapter 4) and shows a number of graphs for the clamping moment along the length of the girder.
The load cases on the bridge, such as load model 71, are different from the mentioned basic load cases. Therefore chapter 6 searches for a combination of basic cases, which have the same effect as the original load case.

A great deal of the essential knowledge in this appendix, such as the derivation of differential equation and the graphs for torsion, was collected through a master thesis by R.T.J. de Groot (4).

2 Derivation of the differential equation for torsion

A single track through bridge is considered which is simply supported and has a constant cross-section. The bridge is symmetrically supported and doesn't have any end cross members.

The distribution of torsion along the length of the main girder depends on the torsional stiffness of the girder, the bending stiffness of the floor and the presence of an end cross member. Before the differential equation can be derived a couple of assumptions need to be made.

2.1 Assumptions

The method of Liptak is used to derive the differential equation. The floor is divided into elements with a length dx and the following assumptions are made:

1. The torsional stiffness of the plate is neglected in as well longitudinal as transverse direction.
2. The load on the floor is mainly distributed in transverse direction. Hence only the bending stiffness of the floor in transverse direction is assumed to be significant.
3. The connection between the floor and the main girder is assumed to be in the centre line of the main girder. The floor prevents the girder from bending in the transverse direction.

4. The supports are considered in the centre of the main girder and they do not restrain a rotation about the longitudinal axis.
5. There is no difference in deflection between the two girders.
6. Only pure torsion is considered, no torsional warping.

2.2 The bending stiffness coefficient

Because the connection between the floor and the girders is assumed to be infinitely stiff, the floor can only deflect between the two girders. The stiff connection causes a restraining moment, called the clamping moment (m_x).

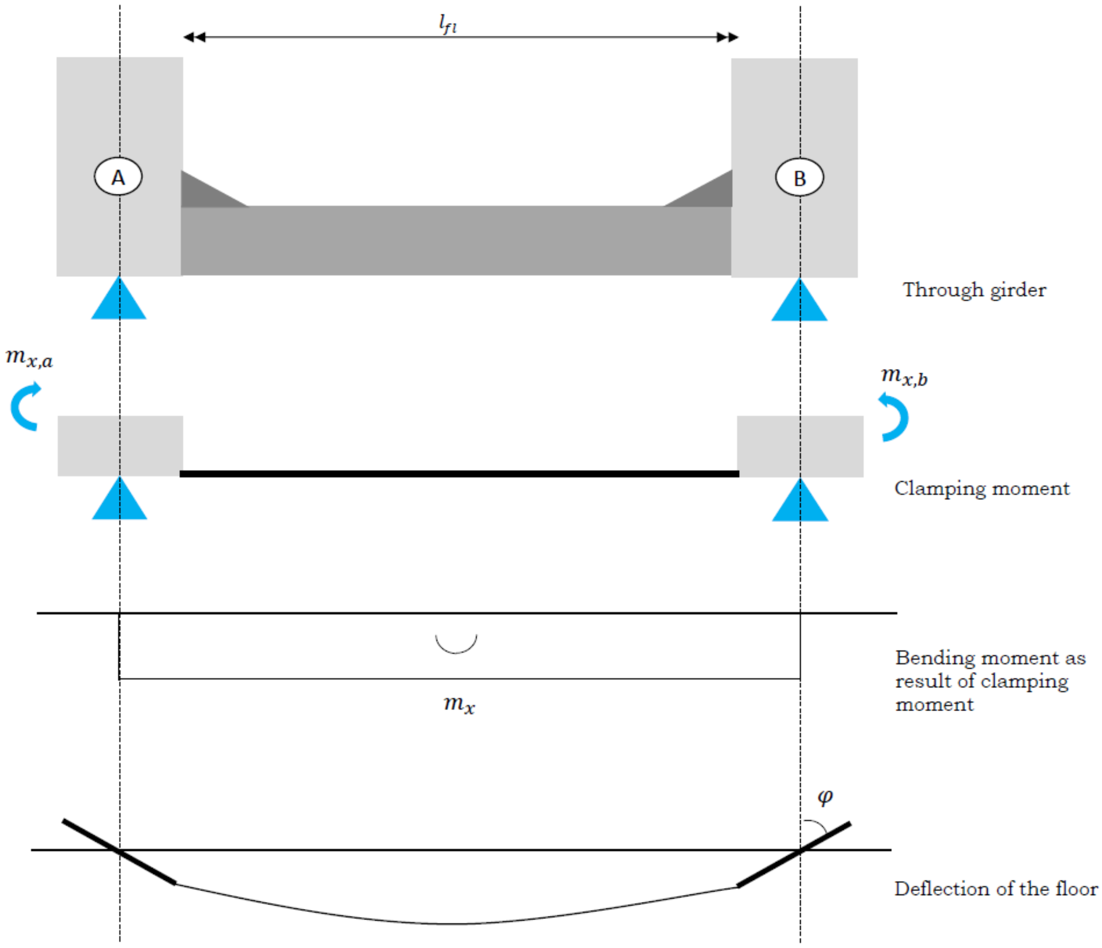

Figure A-1: Bending moment and deflection due to the clamping moment

Using basic mechanics the rotation at the supports can be written as (assume symmetrical loading $m_{x,a} = m_{x,b}$):

$$\varphi_b = \frac{m_{x,a} * l_{fl}}{6EI} + \frac{m_{x,b} * l_{fl}}{3EI} = \frac{m_x * l_{fl}}{2EI}$$

$$m_x = \frac{2EI}{l_{fl}} * \varphi_b = s_{pl} * \varphi_b$$

The bending stiffness coefficient is found by rewriting the rotational stiffness with the moment of inertia per meter.

$$s_{pl} = \frac{2E * \frac{1}{12} * 1 * t^3}{l_{fl}} = \frac{Et^3}{6l_{fl}} \qquad [\text{ A.1 }]$$

Equation [A.1] present the bending stiffness coefficient, where t is the thickness of the floor.

2.3 Differential equation for torsion

Before starting the derivation of the differential equation, one must understand the difference between the clamping moment m_x and the primary load m_{pl}. The latter one is a bending moment resulting from load on a strip with length d_x, if no loading is present at the element, the primary load goes to zero. Yet the clamping moment is the actual moment between the girder and the floor.
The difference between the two is clearly indicated by Figure A-2, section A has a primary load and section B doesn't. Even though the primary load is absent in section B, the clamping moment is still present.

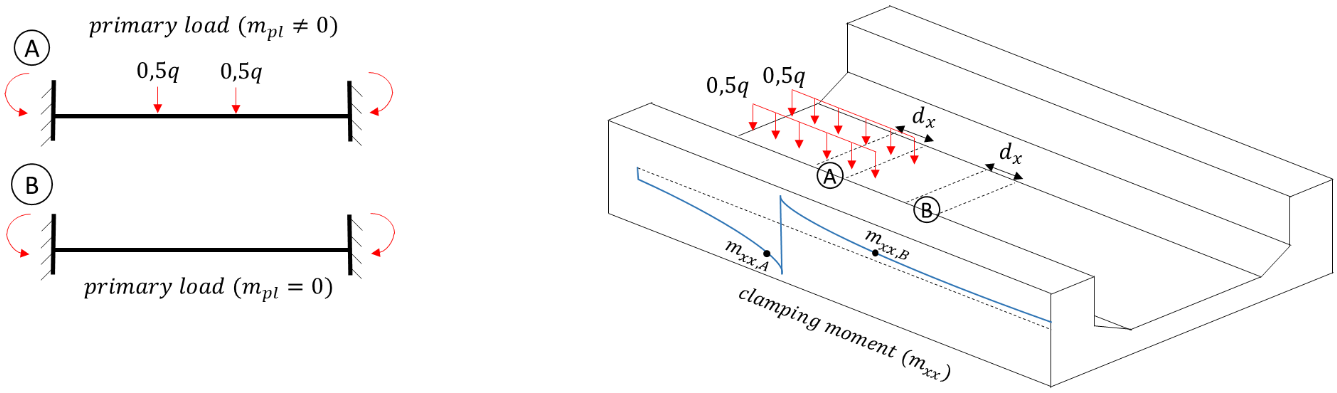

Figure A-2: Primary load (left) and clamping moment (right) due to loading on the bridge

Now let's consider an element with length dx.

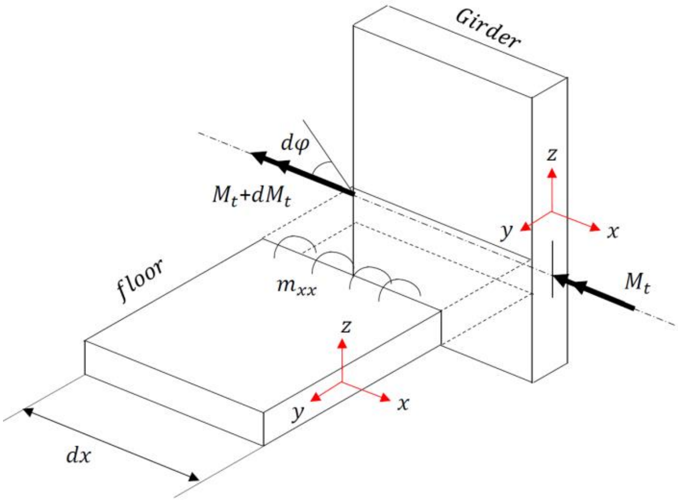

Figure A-3: Element with length dx

The following equations are applicable to the main girder:

Kinematic equation:

$$\phi = \frac{d\varphi_{mg}}{dx} \qquad [\text{ A.2 }]$$

Constitutive equation:

$$M_t = GI_t * \phi \qquad [\text{ A.3 }]$$

Equilibrium equation:

$$M_t + m_x dx = M_t + dM_t$$

$$m_x = \frac{dM_t}{dx} \qquad [\text{ A.4 }]$$

Kinematic boundary condition:

$$\varphi_{mg} = \varphi_{fl} = \varphi \qquad [\text{ A.5 }]$$

Equilibrium boundary condition:

$$m_{x,mg} = m_{x,fl} = m_x \qquad [\text{ A.6 }]$$

Meaning of the used symbols:

$M_t = torsional\ moment\ in\ the\ main\ girder\ (Nmm)$
$\varphi_{mg} = \varphi_x = rotation\ of\ the\ main\ girder\ (rad)$
$\varphi_{fl} = \varphi_x = rotation\ of\ the\ floor\ (rad)$
$GI_t = torsional\ stiffness\ of\ the\ main\ girder\ (Nm^2)$

Let's assume a constant distributed load, q, acts on the floor.

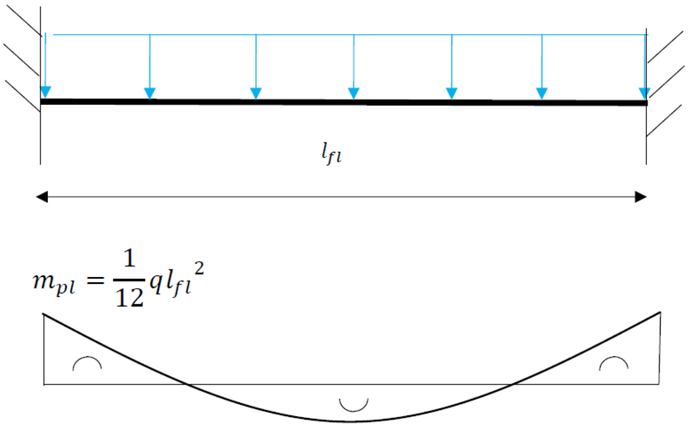

Figure A-4: Primary moment for a distributed load acting on the floor

$$\varphi_{fl} = \frac{m_x * l_{fl}}{2EI} - \frac{q * {l_{fl}}^3}{24EI} = \frac{m_x * l_{fl}}{2EI} - \frac{\left(\frac{1}{12} * q * {l_{fl}}^2\right) * l_{fl}}{2EI}$$

$$= \frac{m_x * l_{fl}}{2EI} - \frac{m_{pl} * l_{fl}}{2EI}$$

$$m_x = m_{pl} + \frac{2EI}{l_{fl}} * \varphi_{fl} = m_{pl} + s_{pl} * \varphi_{fl} \qquad [\ A.7\ ]$$

Substituting equation [A.2] and [A.3] into equation [A.4]:

$$m_x = GI_t * \frac{d^2\varphi_{mg}}{dx^2} \qquad [\ A.8\ ]$$

Equation [A.7] must be equal to equation [A.8], the differential equation for the rotation is derived:

$$\frac{d^2\varphi}{dx^2} - \omega^2\varphi - \frac{m_{pl}}{GI_t} = 0 \qquad [\ A.9\ ]$$

$$\frac{d^2M_t}{dx^2} - \omega^2 M_t - \frac{dm_{pl}}{dx} = 0 \qquad [\ A.10\ ]$$

Rewriting equation [A.9] with the torsional moment gives equation [A.10]. The ratio of the bending stiffness and torsional stiffness is expressed by $\omega^2 = s_{pl}/GI_t \quad (1/m^2)$.

2.4 Solution to the differential equation

The second order differential equation is solved by using a homogeneous solution and a particular solution. The general solution for rotation and torsion have now been derived. Both functions contain two constants which can be solved by determining the boundary conditions.

$$\varphi(x) = C_1 \sinh(\omega x) + C_2 \cosh(\omega x) + \frac{m_{pl}}{s_{pl}} \qquad [\text{ A.11 }]$$

$$M_t(x) = GI_t\omega * (C_1 \sinh(\omega x) + C_2 \cosh(\omega x)) \qquad [\text{ A.12 }]$$

3 Primary moment and load distribution

Because the bending stiffness of the floor is only significant in transverse direction, it can be divided into strips with a length dx. A load acting on the strips results in a primary load in the girder. This chapter holds an explanation on how to determine the primary load for the different load cases.

3.1 Primary load

Five different types of loading are distinguished:

1. A constant distributed area load, for example self-weight
2. A concentrated load, for example the axle load of a train.
3. An evenly distributed line load, for example an empty carriage.
4. An uneven displacement of the girder, for example the settlement of one support.
5. The reactional forces of run-up plates which act at the start and end of the bridge.

3.1.1 Distributed area load

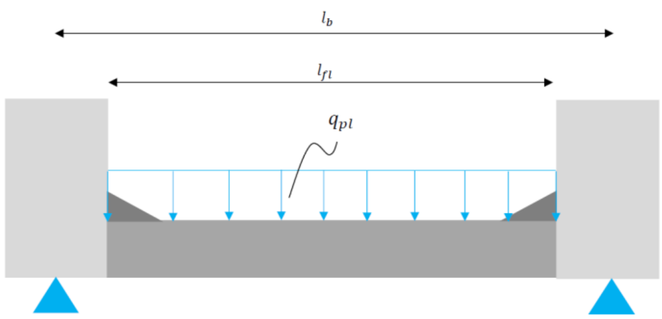

Figure A-5: Constantly distributed area load

Let's assume the length $dx = 1{,}0\, m$. At the point where the floor is connected to the girder, the rotation should equal zero. With this knowledge the primary moment can be derived.

$$\frac{1}{24} * \frac{q_{pl} * {l_{fl}}^3}{EI} - \frac{1}{2} * \frac{m_{pl} * l_{fl}}{EI} = 0$$

$$m_{pl} = \frac{1}{12} * q_{pl} * {l_{fl}}^2 \qquad [\text{ A.13 }]$$

3.1.2 Concentrated load

The same approach is applicable for a concentrated load. The rotation at the centre of the girders is ought to be zero. Use the *forget-me-nots* for a cantilevering beam.

$$\frac{P * l_e{}^2}{2EI} + \frac{P * l_{tr} * l_e}{EI} - \frac{m_{pl} * l_b}{2EI} = 0$$

$$m_{pl} = \frac{P * l_e * (l_e + 2 * l_{tr})}{l_b} \qquad [\text{A.14}]$$

The lengths l_{tr} and l_b are, the typical track width of 1,50 m and the centre to centre distance of the girders respectively.

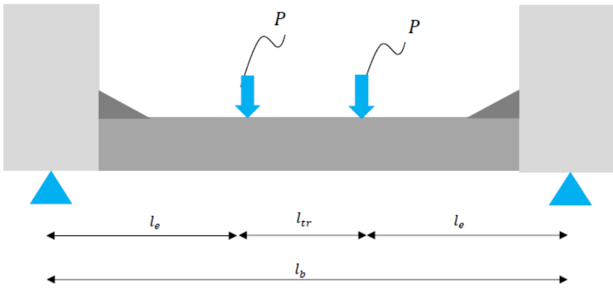

Figure A-6: Concentrated load

3.1.3 Distributed line load

The derivation of the evenly distributed line load is exactly the same as for the concentrated load. The value for the concentrated load P is replaced with the line load value p.

$$m_{pl} = \frac{p * l_e * (l_e + 2 * l_{tr})}{l_b} \qquad [\text{A.15}]$$

3.1.4 Support settlement

For a through bridge a settlement of one of the supports can be very likely. Now let's assume a settlement of Δ of one of the supports.

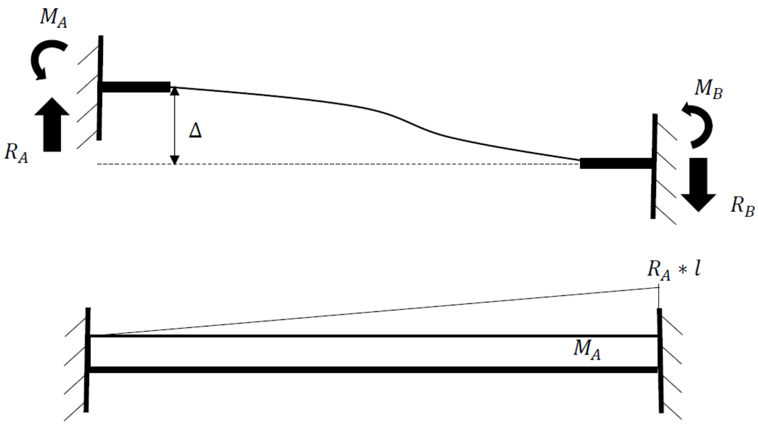

Figure A-7: Transverse section for the settlement of one support

The rotation between A and B:

$$\theta_{AB} = \frac{1}{EI}(Area_{AB}) = \frac{1}{2} * R_A * l^2 - M_A L = 0$$

$$R_A = +\frac{2M_A}{L}$$

Deflection in point B:

$$\theta_{AB} = \frac{1}{EI}(Area_{AB}) * \overline{X_B} = \frac{1}{2} * R_A * l^2 * \left(\frac{1}{3}L\right) - M_A L * \left(\frac{1}{2} * L\right) = -\Delta$$

$$R_A L^3 - 3M_A L^2 = -6EI\Delta$$

Substitute R_A by earlier found expression:

$$m_{pl} = M_A = M_B = 6EI\Delta/L^2 \qquad [\,A.16\,]$$

3.1.5 Run-up plate reactional force

At either side of the bridge two run-up plates are applied with a width l_2. The assumption is made that each run-up plate generates two reactional forces on the bridge, which results in a transverse section as in Figure A-8.

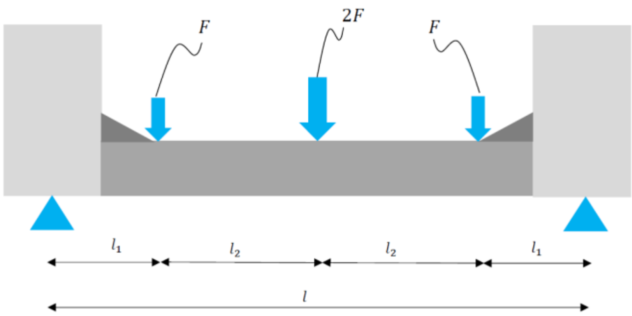

Figure A-8: Reactional forces of two run-up plates

Using *forget-me-nots* once again the primary load is derived:

$$m_{pl} = F * \left(\frac{{l_1}^2 + 2 * l_1 l_2 + 0{,}25 * l^2}{l}\right) \qquad [\text{ A.17 }]$$

3.2 Load distribution of a concentrated load

Because the floor is divided into strips with length d_x $(= 1{,}0\ m)$, the primary load for concentrated loads becomes rather large. The strip approach doesn't allow any distribution of the concentrated loads in longitudinal direction. To solve this problem, this paragraph will search for a standard distribution length for relevant concentrated loads. In the master thesis of R.T.J. de Groot (4), a standard function for concentrated loads is found:

$$m_{pl}(x) = \frac{m_{pl,0}}{2} * (\cos\frac{\pi x}{c} + 1) \qquad [\text{ A.18 }]$$

Where c $(= 4{,}5\ m)$ is the distribution length for a concentrated load acting on a single track bridge.

Taking the integral of this function leads to:

$$\int_0^{+2c} \frac{m_{pl,0}}{2} * (\cos\frac{\pi x}{c} + 1)\, dx = \frac{m_{pl,0}}{2} * [-\frac{\pi}{c}\left(\sin\left(\frac{\pi x}{c}\right) + x\right]_{-c}^{+c} = \frac{m_{pl,0}}{2} * [0 + 2c - 0 - 0] = m_{pl,0} * c$$

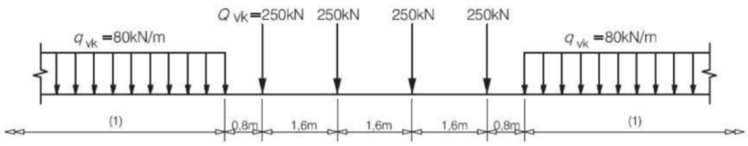

Figure A-9: Load model 71

It is desirable to convert the four concentrated loads from LM71 into a distributed line load. To do this, a function describing the total effect of the four loads is necessary. The area under the graph should equal $4 * m_{pl,0} * c$ and the distance between the first and last concentrated load is 4,8 meters.

$$\int_0^{+2c+4,8} A * m_{pl,0} * (\cos\frac{\pi x}{c+2,4} + 1)\, dx = A * m_{pl,0} * [-\frac{\pi}{c+2,4}\left(\sin\left(\frac{\pi x}{c+2,4}\right) + x\right]_{0}^{+2c+4,8}$$

$$= A * m_{pl,0} * [0 + 2c + 4,8 - 0 - 0] = A * m_{pl,0} * (2c + 4,8) = 4m_{pl,0} * c$$

The function for the total load goes to:

$$m_{pl}(x) = 1,30 * m_{pl,0} * (\cos\frac{\pi x}{c} + 1) \qquad [\text{ A.19 }]$$

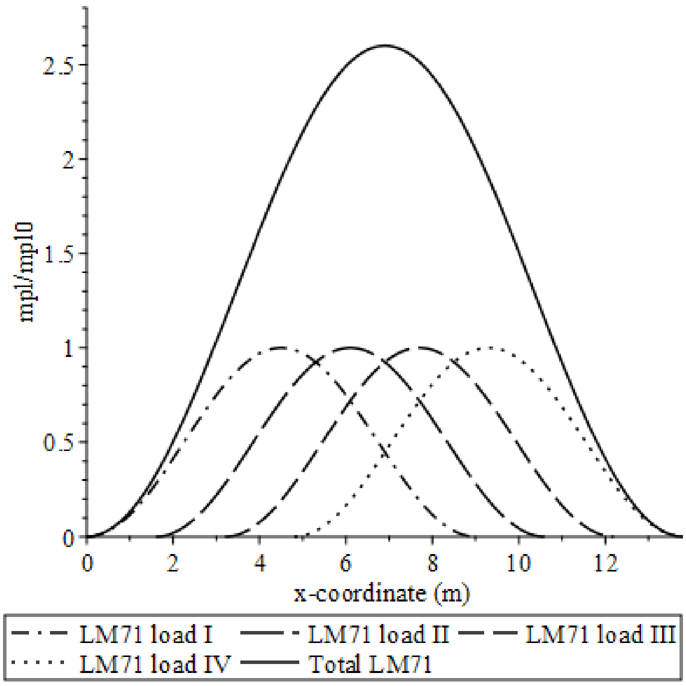

Figure A-10: Load distribution of LM71

Figure A-10 present a visualisation of the mentioned graphs. The combination of the four forces leads to a maximum load of $2,60 * m_{pl0}$ and acts over a length of $2c + 4,8 = 2 * 4.5 + 4.8 = 13,8\, m$. The total effect can be simplified to a constant distributed load, which has the same area under the graph. Hence the four concentrated loads of LM71 are replaced by a constant distributed load of $2,60m_{pl0}$ over 6,9 $meters$.

4 Torsional moment in the main girder

In chapter 2 the differential equation for torsion is derived and a general solution is presented. This chapter will focus on deriving the boundary conditions for different basic load cases.

Before one can continue, a sign convention must be established. Let's consider a through girder which is divided into two. The sign convention depends on the section that is regarded. For example if torsion acts on the right section, a moment from z to y is positive and from y to z is negative.

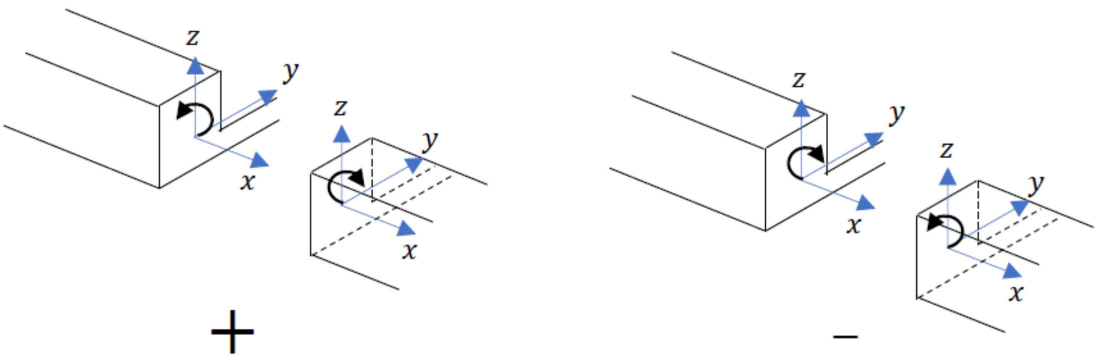

Figure A-11: Sign convention for torsion

4.1 Torsional moment at the edge of the bridge

If a torsional moment is applied at the start or end of a simply supported bridge with no end cross members, the primary moment (m_{pl}) equals zero. The solution to the differential equation reduces to a homogeneous one. Two situations are considered, one with a torsional moment at $x = 0$ and one with a torsional moment at $x = l$.

Boundary conditions if M_{pl} acts at $x = l$:

$x = 0 \qquad M_t = 0 \qquad \rightarrow \qquad C_1 = 0$

$x = l \qquad M_t = M_{pl} \qquad \rightarrow \qquad C_2 = \frac{M_{pl}}{GI_t\omega \sinh \omega l}$

The equations for rotation and torsion go to:

$$\varphi(x) = \frac{M_{pl}}{GI_t\omega} * \frac{\cosh \omega x}{\sinh \omega l} \qquad [\text{ A.20 }]$$

$$M_t(x) = M_{pl} * \frac{\sinh \omega x}{\sinh \omega l} \qquad [\text{ A.21 }]$$

Boundary conditions if M_{pl} acts at $x = 0$:

$$x = 0 \qquad M_t = M_{pl} \rightarrow \qquad C_1 = -\frac{M_{pl}}{GI_t\omega}$$

$$x = l \qquad M_t = 0 \rightarrow \qquad C_2 = -\frac{M_{pl}}{GI_t\omega} * \frac{1}{\tanh \omega l}$$

The equation for rotation and torsion goes to:

$$\varphi(x) = \frac{M_{pl}}{GI_t\omega} * \left(\sinh \omega x - \frac{\cosh \omega x}{\tanh \omega l}\right) \qquad [\text{ A.22 }]$$

$$M_t(x) = M_{pl} * \left(\cosh \omega x - \frac{\sinh \omega x}{\tanh \omega l}\right) \qquad [\text{ A.23 }]$$

Figure A-12 displays the course of torsion along the length of the girder. The y-axis expresses which fraction of the originally applied primary load is present. Torsion at one side of the bridge leads to a very local rotation which is counteracted by the entire through girder. This explains that the maximum torsion is found at the edge of loading and no torsion if found at the other side.

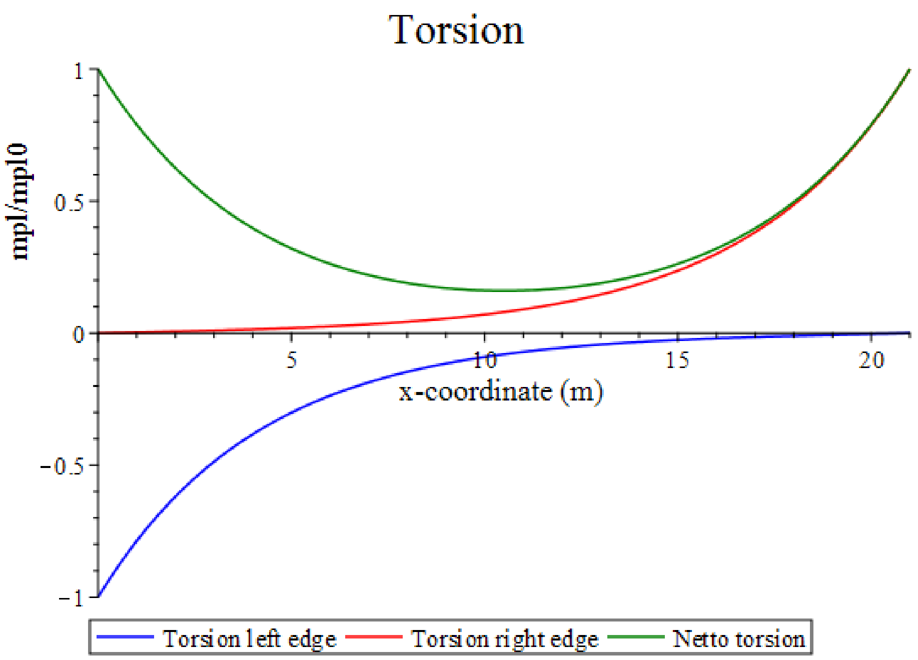

Figure A-12: Torsion applied at the start/end of the bridge

4.2 Concentrated torsional moment at an arbitrary position

The axle load of the train can cause a torsional moment at an arbitrary position. The situation is simplified, by splitting the girder into two and applying a torsional moment at the edge of the girder 1 and 2.

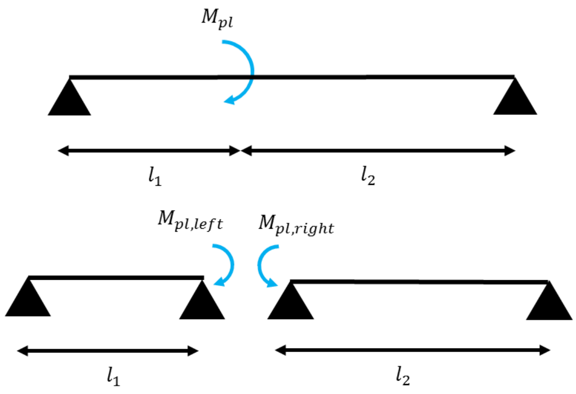

Figure A-13: Torsional moment spreads over two girders with length l_1 and l_2

The derivation in paragraph 4.1 is used to find an expression for torsion.

For the girder I with $0 \leq x \leq l_1$:

$$M_{t,I}(x) = M_{pl,left} * \left(\frac{\sinh \omega x}{\sinh \omega l_1}\right) \quad [\text{ A.24 }]$$

For the girder II with $l_1 < x \leq l$:

$$M_{t,II}(x) = M_{pl,right} * \left(\cosh \omega(x - l_1) - \frac{\sinh \omega(x - l_1)}{\tanh \omega l_2}\right) \quad [\text{ A.25 }]$$

The transitional condition between the two girders is equal to:

$$M_{pl} = M_{pl,left} - M_{pl,right}$$

Using the transitional condition, the distribution of the concentrated torsional moment over the two girders can be determined:

$$M_{pl,left} = \frac{\tanh(\omega l_1)}{\tanh(\omega l_1) + \tanh(\omega l_2)} * M_{pl} \quad [\text{ A.26 }]$$

$$M_{pl,right} = \frac{\tanh(\omega l_2)}{\tanh(\omega l_1) + \tanh(\omega l_2)} * M_{pl} \quad [\text{ A.27 }]$$

Figure A-14 shows the way torsion is distributed over the girder if a concentrated torsional moment is applied at $x = 5{,}0\ meters$.

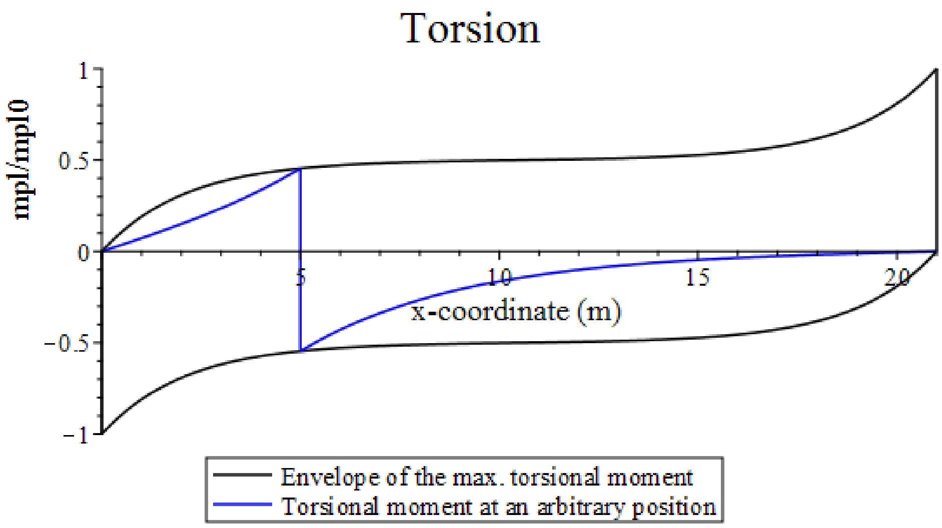

Figure A-14: Torsion for a concentrated torsional moment at an arbitrary position

4.3 Constant distributed load

The self-weight of the bridge causes a constant distributed load on the girder, which means the primary load m_{pl} is constant. This causes a constant rotation of the girder and since there are no parts of the girder counteracting this rotation, the torsional moment is expected to be zero.
Yet in reality the self-weight leads to a larger deflection at midspan compared to a section near the supports. A difference in deflection leads to a difference in rotation, which means the torsion due to self-weight is not expected to be zero after all. Chapter 6 searches for an alternative load case which is not constant.

4.4 Distributed line load at the edge of the bridge

A train can cause a distributed line load at the beginning or end of the bridge. Figure A-15 illustrates such a situation where a train is leaving the bridge.

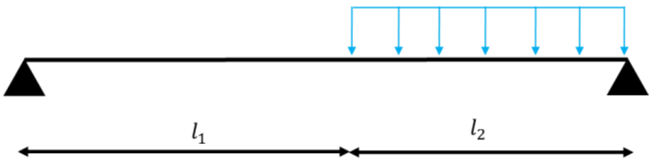

Figure A-15: The bridge with an unloaded length l1 and loaded length l2

Transitional condition:

$$\varphi_{left} = \varphi_{right}$$

The general solution is: (see chapter 2.4):

$$\varphi(x) = C_1 \sinh(\omega x) + C_2 \cosh(\omega x) + \frac{m_{pl}}{s_{pl}}$$

$$M_t(x) = GI_t\omega * (C_1 \sinh(\omega x) + C_2 \cosh(\omega x))$$

The boundary conditions for girder I with $0 < x < l_1$:

$x = 0$	$M_t = 0$	→	$C_1 = 0$
$x = l_1$	$M_t = M_{t,transition}$	→	$C_2 = \frac{M_{t,transition}}{GI_t\omega \sinh \omega l}$
$0 < x < l_1$	$m_{pl} = 0$		

The solution goes to:

$$M_{t,I}(x) = M_{t,transition} * \frac{\sinh \omega x}{\sinh \omega l_1} \quad [\ A.28\ ]$$

$$\varphi_{left} = \frac{M_{t,transition}}{GI_t \omega} * \frac{1}{\tanh \omega l_1} \quad [\ A.29\ ]$$

The boundary conditions for girder II with $l_1 < x < l$:

$x = 0 \qquad M_t = M_{t,transition} \quad \rightarrow \quad C_1 = -\frac{M_{t,transition}}{GI_t \omega}$

$x = l \qquad M_t = 0 \quad \rightarrow \quad C_2 = -\frac{M_{t,transition}}{GI_t \omega} \frac{1}{\tanh \omega l}$

$l_1 < x < l \qquad m_{pl} = constant$

The solution goes to:

$$M_{t,II}(x) = M_{t,transition} \left(\cosh \omega (x - l_1) - \frac{\sinh \omega (x - l_1)}{\tanh \omega l_2} \right) \quad [\ A.30\ ]$$

$$\varphi_{right} = -\frac{M_{t,transition}}{GI_t \omega} * \frac{1}{\tanh \omega l_2} + \frac{m_{pl}}{\omega^2 GI_t} \quad [\ A.31\ ]$$

Since $\varphi_{left} = \varphi_{right}$ the transitional moment can expressed as:

$$M_{t,transition} = \frac{m_{pl}}{\omega} \left(\frac{\tanh \omega l_1 * \tanh \omega l_2}{\tanh \omega l_1 + \tanh \omega l_2} \right) \quad [\ A.32\ ]$$

Using the earlier established sign convention, one can understand that an oncoming train causes negative torsion and a train leaving the bridge causes positive torsion. The black graph in Figure A-16 is the envelope defining maximum torsion, which has an upper limit value of $m_{pl0}/2\omega$.

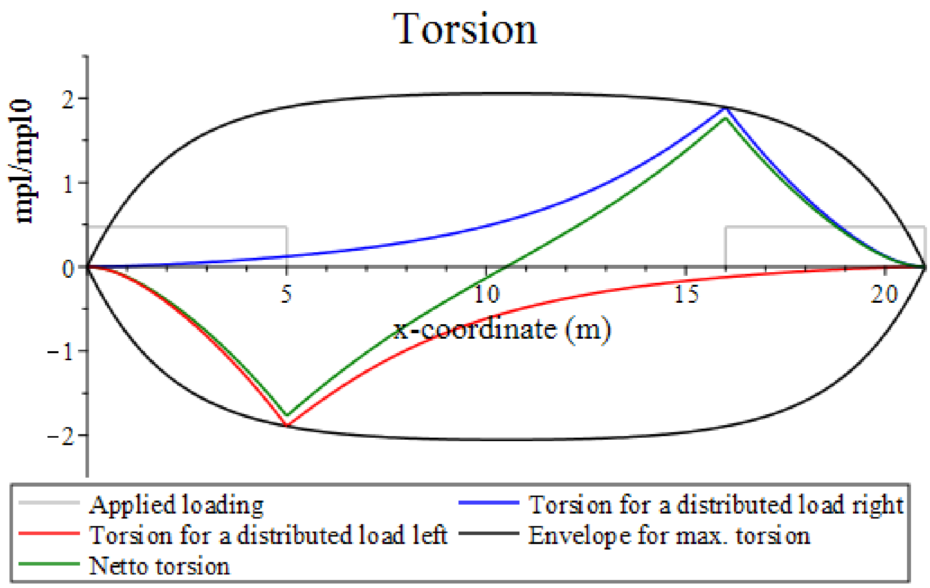

Figure A-16: Torsion for a distributed mobile line load

4.5 Local mobile load

Load model 71 consists of a distributed load and four concentrated loads. The concentrated loads could be heavy axle loads of a locomotive. Let's assume such a locomotive enters the bridge. Until the moment the last axle load is above the support, the expression for $M_{t,transition}$ in chapter 4.4 remains valid. By substituting the length l_2 with the length of the mobile load l_b, the expression goes to:

$$M_{t,transition} = \frac{m_{pl}}{\omega}\left(\frac{\tanh \omega l_1 * \tanh \omega l_b}{\tanh \omega l_1 + \tanh \omega l_b}\right) \quad [\text{ A.33 }]$$

But when the locomotive starts to move away from the support, torsion will no longer increase. The parts of the girder which are no longer loaded will now contribute to the load transfer.
In order to establish the torsional moment at midspan, the bridge is simplified to a girder with the length l_b. The unloaded parts can then be schematized as end-cross member. The expression for girders with end-cross members is obtained from (4).

$$M_{t,midspan} = \frac{m_{pl}}{2\omega}(1 + \sinh \omega l_b - \cosh \omega l_b) \quad [\text{ A.34 }]$$

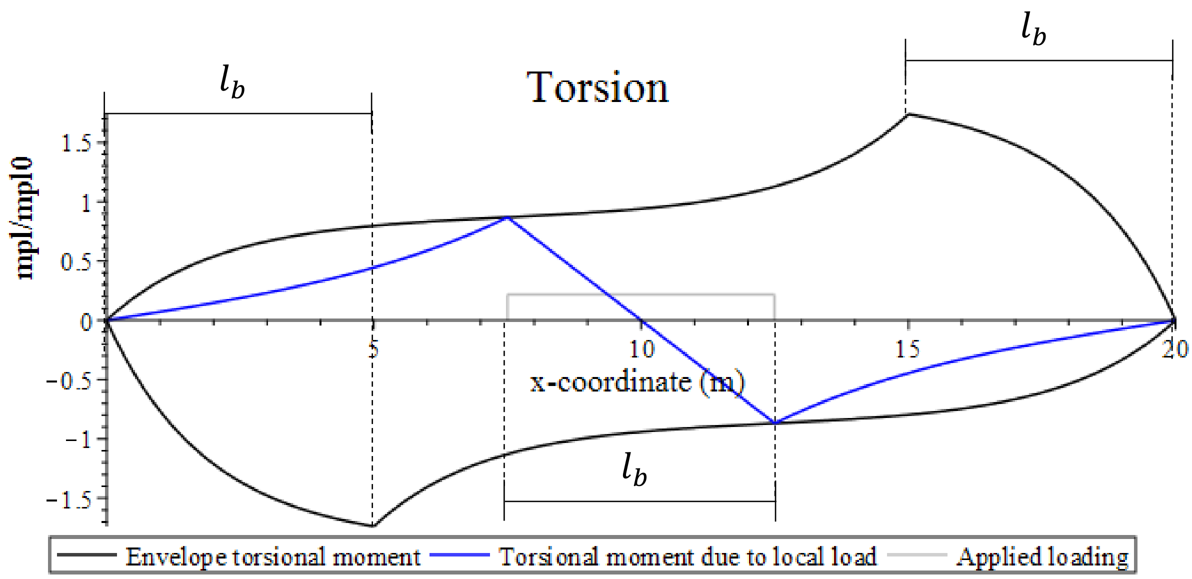

Figure A-17: Torsion due to a local mobile load

The graph above describes the torsional moment when a local mobile load with length l_b is applied on the bridge. A peak in the envelope appears at a distance l_b from the supports.

5 Clamping moment

Loads on the floor will spread in transverse direction towards the main girders. This results in a suspension force and a clamping moment.

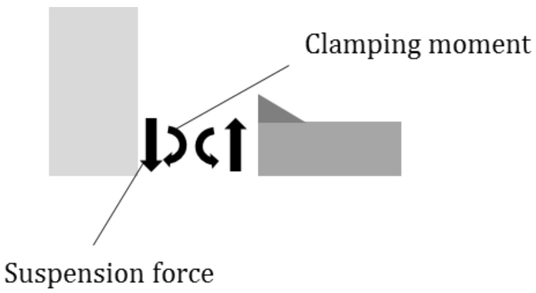

Figure A-18: Reactional forces between girder and floor

Figure A-19 presents a top view of bridge A, with a line load acting on the floor. Because the line load originates from a train loading, it is split into two separate line loads with the characteristic track distance of $1{,}50\ m$ between them. The load is assumed to spread under an angle of 45° towards the girders, meaning the primary load is present over a length: $l_{load} + 1{,}70\ m$. Earlier mentioned is that the clamping moment can even be present if the primary load is absent. Contrastingly a suspension force can only be found for sections at which the primary load is active.

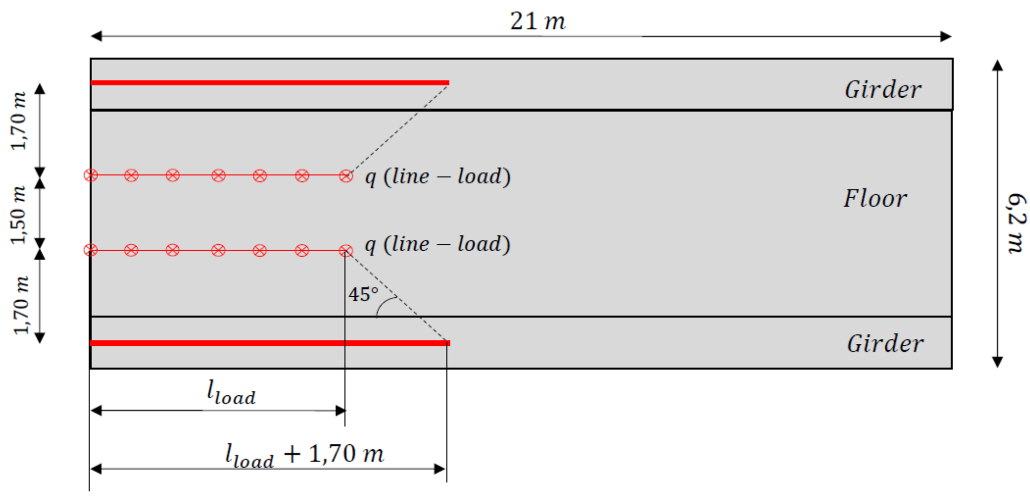

Figure A-19: Top view bridge A: Line-load acting on the floor

In the derivation of the differential equation one learned that the difference in torsion is equal to the clamping moment. In other words the clamping moment is the derivative of the torsion function. This chapter focuses on finding the function for the clamping moment. But before one can continue, a sign convention must be established.

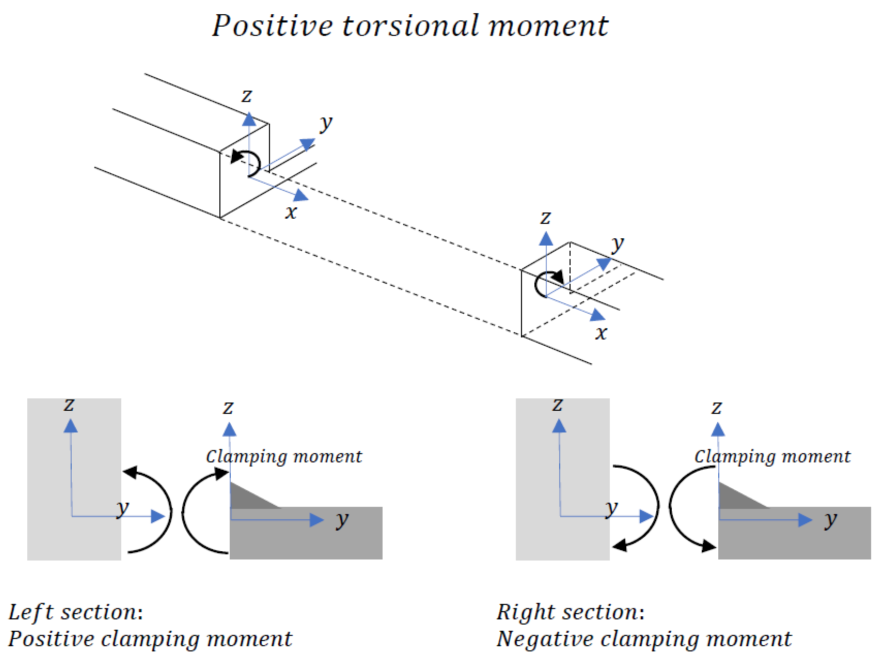

Figure A-20: Sign convention clamping moment

Let's have a look at the positive torsional moment and the corresponding reactional force. In the left section the reactional force is a downward bending moment, whereas in the right section it is an upward acting bending moment. From here onwards, the downward bending moment (from z to y) is a positive clamping moment.

5.1 Clamping moment due to torsion at the edge

Let's consider torsion at the edge of the bridge (chapter 4.1). The course of the clamping moment can be found by taking the derivative of the torsion function:

$$\frac{d}{dx}\cosh \omega x = \omega \sinh \omega x$$

$$\frac{d}{dx}\sinh \omega x = \omega \cosh \omega x$$

M_{pl} acts at $x = l$

$$M_t(x) = M_{pl} * \frac{\sinh \omega x}{\sinh \omega l}$$

$$m_x(x) = \frac{dM_t(x)}{dx} = M_{pl}\omega * \frac{\cosh \omega x}{\sinh \omega l} \quad [\text{ A.35 }]$$

M_{pl} acts at $x = 0$

$$M_t(x) = M_{pl} * \left(\cosh \omega x - \frac{\sinh \omega x}{\tanh \omega l}\right)$$

$$m_x(x) = \frac{dM_t(x)}{dx} = M_{pl}\omega * \left(\sinh \omega x - \frac{\cosh \omega x}{\tanh \omega l}\right) \quad [\text{ A.36 }]$$

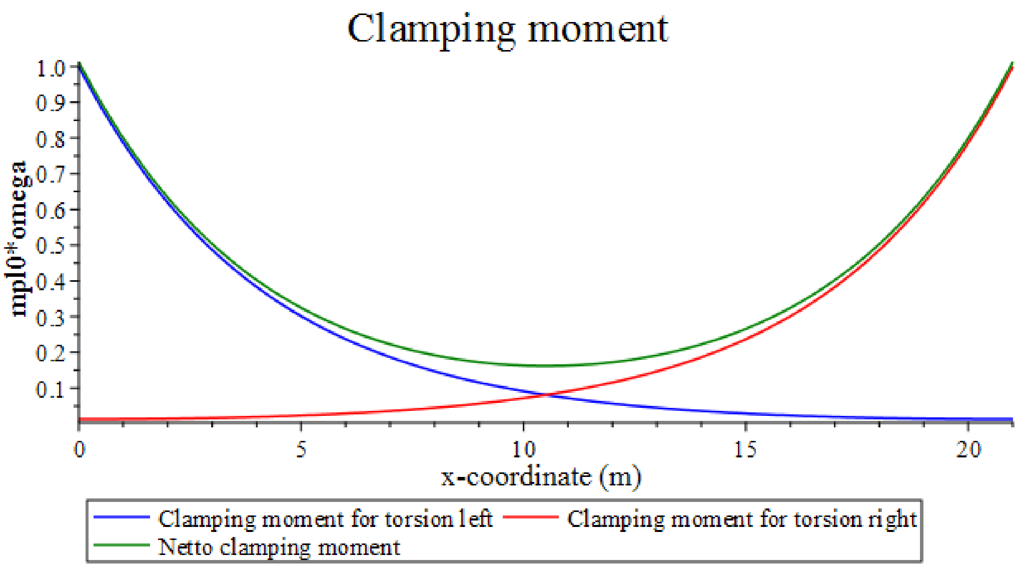

Figure A-21: Clamping moment course due to torsion at the start/end of the bridge

Both edge-moments generate a positive clamping moment with a maximum at the position of loading and a gradually decrease towards the opposite support.

5.2 Clamping moment due to a concentrated torsional moment

The approach in chapter 4.2 is used to find a function for the clamping moment.

For the girder I with $0 \leq x \leq l_1$:

$$M_{t,I}(x) = M_{pl,left} * \left(\frac{\sinh \omega x}{\sinh \omega l_1}\right)$$

$$m_{x,I}(x) = \omega * M_{pl,left} * \frac{\cosh \omega x}{\sinh \omega l_1} \quad [\text{ A.37 }]$$

For the girder II with $l_1 < x \leq l$:

$$M_{t,II}(x) = M_{pl,right} * \left(\cosh \omega(x - l_1) - \frac{\sinh \omega(x - l_1)}{\tanh \omega l_2}\right)$$

$$m_{x,II}(x) = \omega * M_{pl,right} * \left(\sinh \omega(x - l_1) - \frac{\cosh \omega(x - l_1)}{\tanh \omega l_2}\right) \quad [\text{ A.38 }]$$

Since the function that is used to determine the distribution of the torsional moment over girder I and II ($M_{pl,left}$ & $M_{pl,right}$) is independent of x, the distribution remains unchanged.

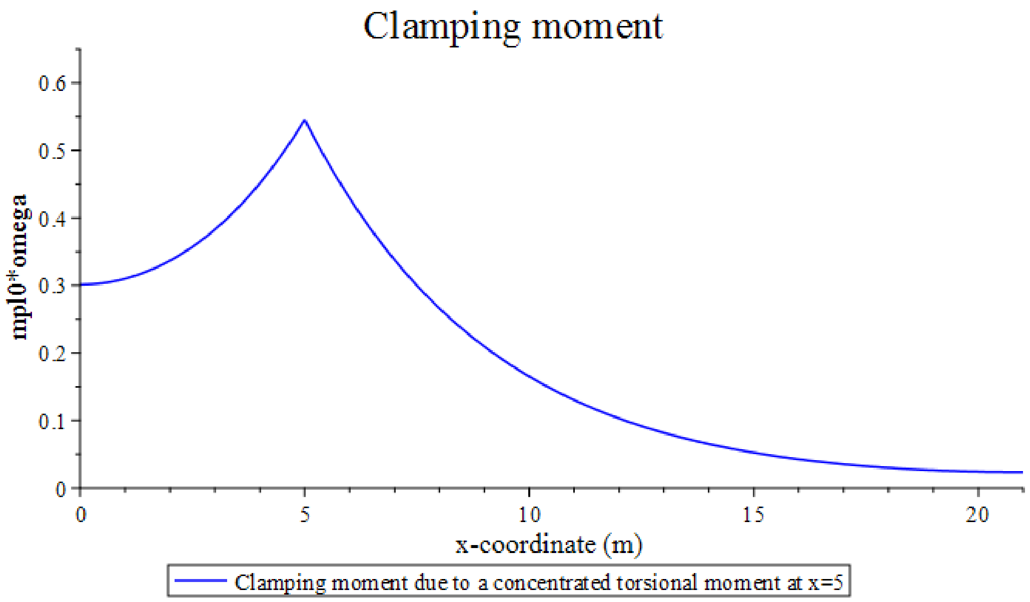

Figure A-22: Clamping moment for a concentrated torsional moment at an arbitrary position

If a concentrated torsional moment is applied at 5,0 meters from the left support, the resulting clamping moment follows the blue graph in Figure A-22.

5.3 Clamping moment due to a distributed line load at the edge

The basis of this derivation can be found in chapter 4.4.

The solution for girder I with $0 < x < l_1$:

$$M_{t,I}(x) = M_{t,transition} * \frac{\sinh \omega x}{\sinh \omega l_1}$$

$$m_{x,I}(x) = \omega M_{t,transition} * \frac{\cosh \omega x}{\sinh \omega l_1} \qquad [\ A.39\ ]$$

The solution for girder II with $l_1 < x < l$:

$$M_{t,II}(x) = M_{t,transition}\left(\cosh \omega(x - l_1) - \frac{\sinh \omega(x - l_1)}{\tanh \omega l_2}\right)$$

$$m_{x,II}(x) = \omega M_{t,transition} * \left(\sinh \omega(x - l_1) - \frac{\cosh \omega(x - l_1)}{\tanh \omega l_2}\right) \qquad [\ A.40\ ]$$

The transitional moment ($M_{t,transition}$) is independent of x and remains unchanged in the derivation for the clamping moment. Respectively a load applied on l_1 or l_2 leads to a clamping moment according to the red and blue graph in Figure A-23. One should notice that the jump of the clamping moment equals the primary load.

$$|m_{x,left}| + |m_{x,right}| = m_{pl}$$

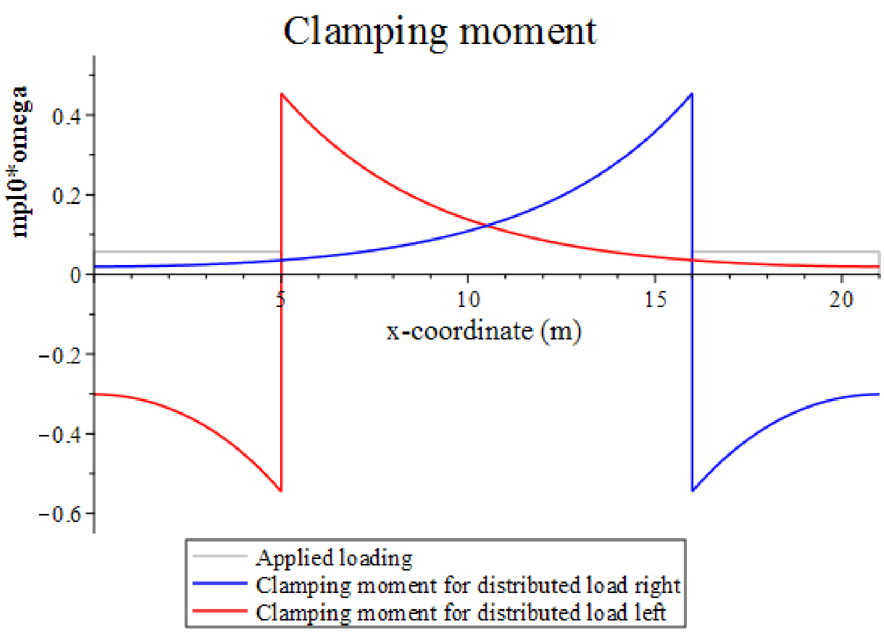

Figure A-23: Clamping moment due to a distributed load at one end

5.4 Clamping moment due to a local mobile load

Since the application of a local mobile load is a more complex version of the load case which considers a distributed line load, equations [A.39] and [A.40] from chapter 5.3 can be used. Both functions need to be multiplied with the maximum torsional moment (Figure A-17) to find a clamping moment of the right order of magnitude.

$$m_{x,I}(x) = \omega * M_{t,max} * M_{pl,left} * \frac{\cosh \omega x}{\sinh \omega l_1} \qquad [\ A.41\ ]$$

$$m_{x,II}(x) = \omega * M_{t,max} * M_{pl,right} * \left(\sinh \omega (x - l_1) - \frac{\cosh \omega (x - l_1)}{\tanh \omega l_2} \right) \qquad [\ A.42\ ]$$

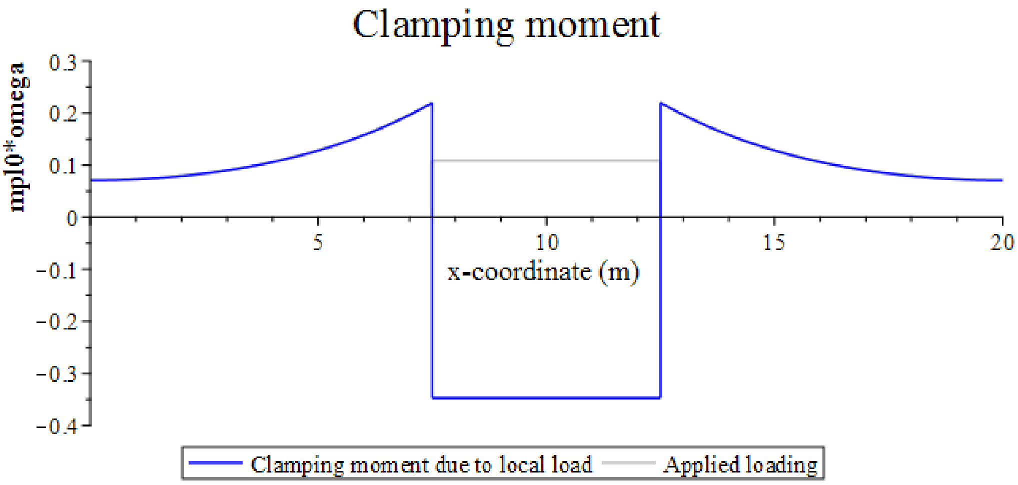

Figure A-24: Clamping moment due to a local mobile load

6 Load cases

A couple of basic load cases have been discussed throughout this appendix. Because the load cases acting on the bridge are more complex, they are broken down into a combination of these basic cases. It is recommended to read this chapter in combination with appendix G.

6.1 Self-weight

As mentioned before the theory does not assume torsion due to constant loading such as self-weight. But application of a finite element program (appendix G) proves that self-weight does cause torsion in the girder. Especially near the supports, the structure is relatively stiff and attracts quite some torsion. An alternative load case is applied which has the same deflection pattern as self-weight.

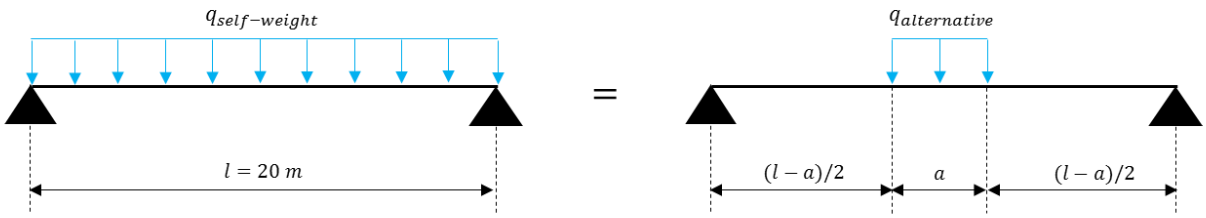

Figure A-25: Alternative load case for self-weight

With some trial and error it established that (for bridge A) the alternative load case has the same deflection pattern when the length approximates 16 m and the load 105% of the self-weight. Additionally the values for torsion of the FE model and theory compare rather well for this assumption.

6.2 Load on cantilevers

Load case 4 consists of a double distributed line load (due to a train) and a concentrated load (due to the reactional force of two run-up plates). This load case needs to be converted into a torsional moment which can be applied at the start and end of the bridge.

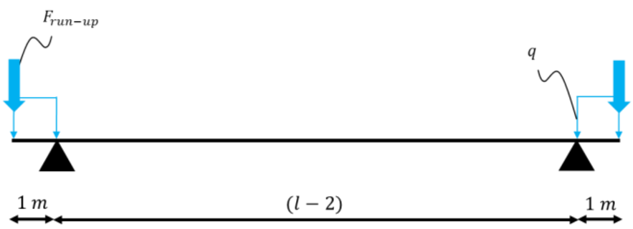

Figure A-26: LC 4: Mobile loading on the cantilevers of the bridge

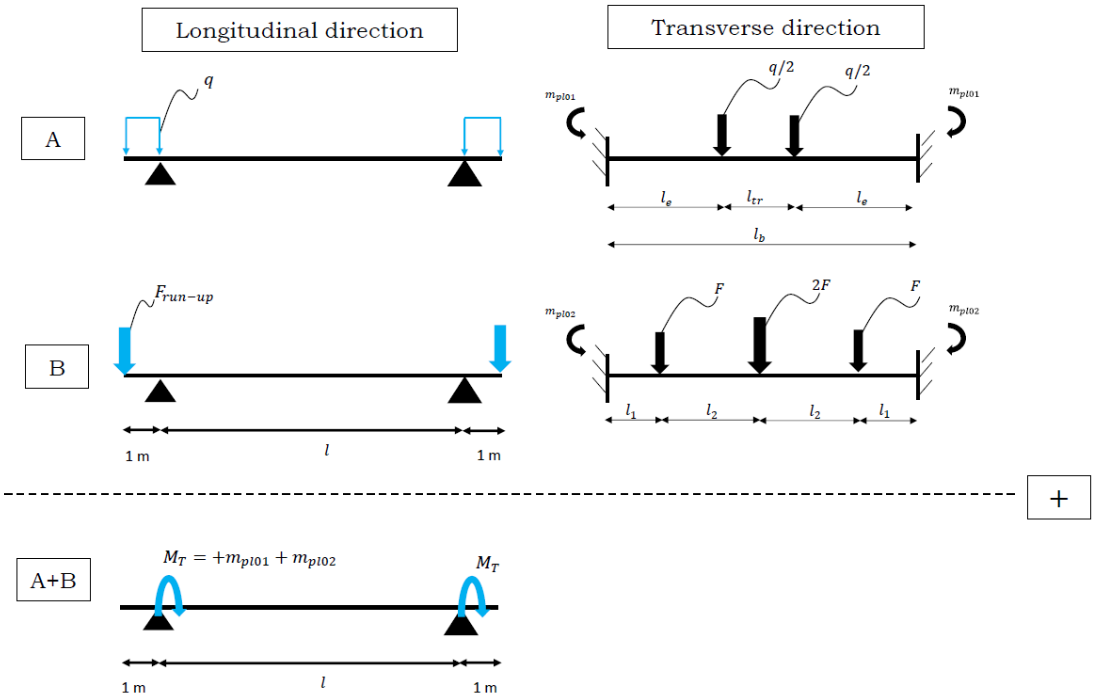

Figure A-27: Loading on the cantilevers converted into torsional moment at the start/end

The primary load for the distributed and concentrated load are determined with the knowledge from paragraph 3.1. The primary loads are added together and applied as a torsional moment on the start and end of the bridge (Figure A-27). This alternative load case comes down to one of the basic load cases and torsion along the length of the girder can now be plotted.

6.3 Settlement of supports

The occasion where one support settles more than the others it very likely. Therefore a load case is taken into account which assumes a support settlement of 5 mm. The primary load derived in paragraph 3.1 should be used to determine torsion in the girder. But because the theory assumes there is no load distribution in longitudinal direction, unrealistically high values for torsion are obtained. Because the SCIA plate model (appendix G) matches best with the theory, this model is used to find the correct values for torsion and the clamping moment.

Note: The support settlement load case is the only case for which SCIA is used to find appropriate values for torsion and the clamping moment. For all the other load cases in the upcoming calculations the analytical solution is used.

6.4 Prestress

Torsion consists of a horizontal prestressing force, upward distributed load and a bending moment due to eccentricity of the tendons. Because torsion is only generated by deflection of the floor and rotations of the girder, the horizontal prestressing force is excluded from the torsion calculation.

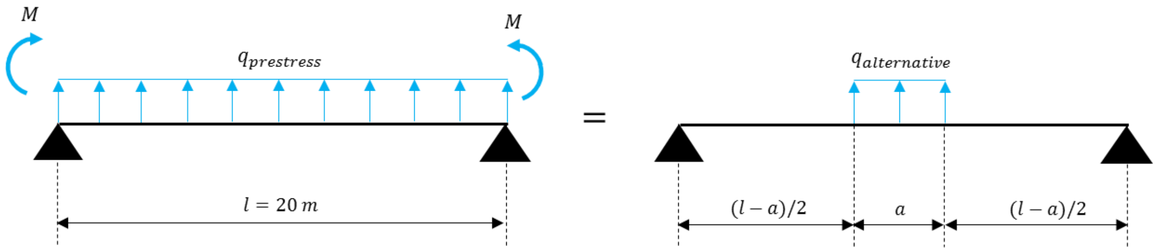

Figure A-28: Alternative load case for prestress

Like self-weight and alternative loading is applied on the bridge. From appendix G it is learned that by taking $a = 16\ m$ and $q_{alternative} = 102\% * q_{prestress}$, the same deflection pattern is found as the original case. Because the bending moment counteracts the upward deflection of the prestressing force, the alternative load case has roughly the same distributed load but is present over a shorter length. Appendix G also teaches one that the torsional graphs of the alternative case match pretty well with results from SCIA.

6.5 Bridge A

Table A-1: Torsion load cases bridge A

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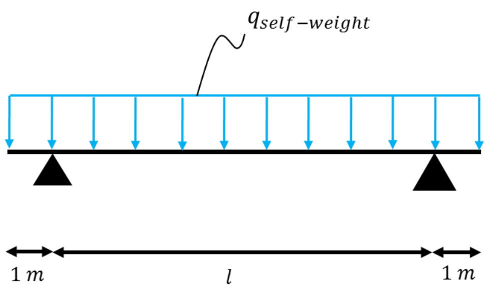
<td rowspan="2">
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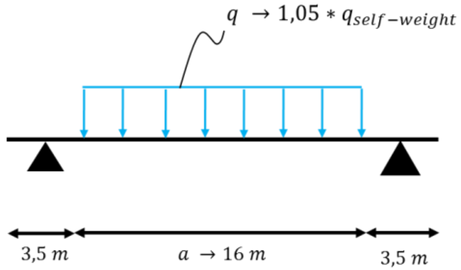
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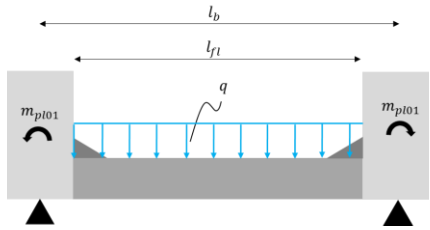
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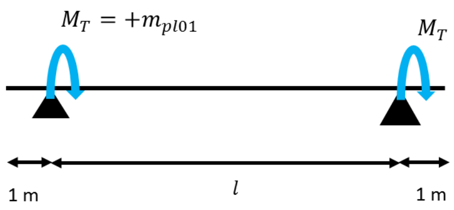
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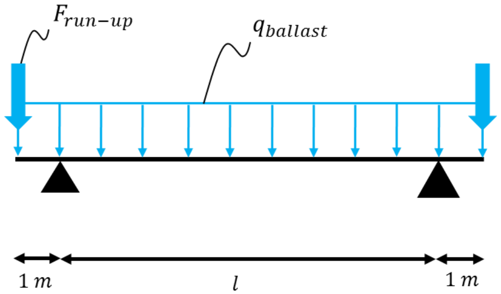
<td rowspan="2">
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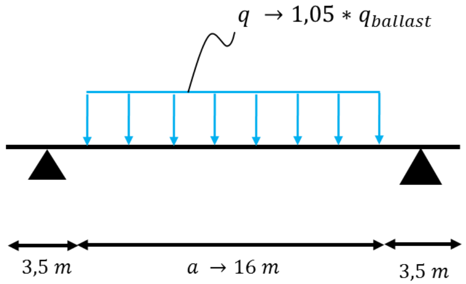
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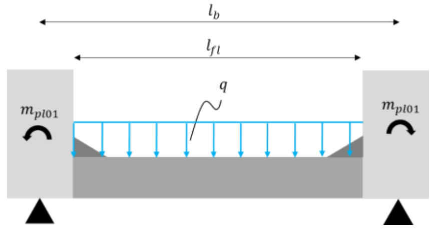
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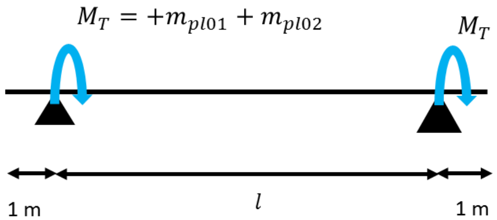
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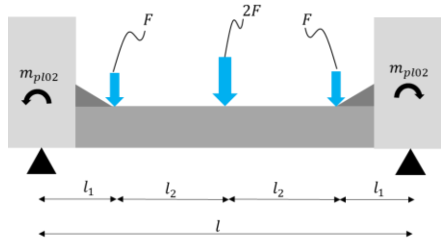
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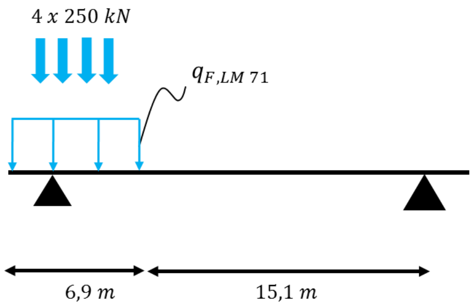
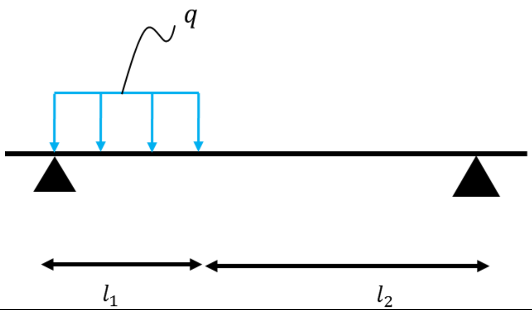
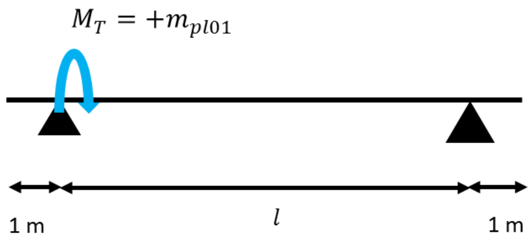
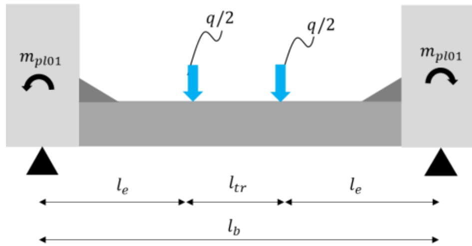
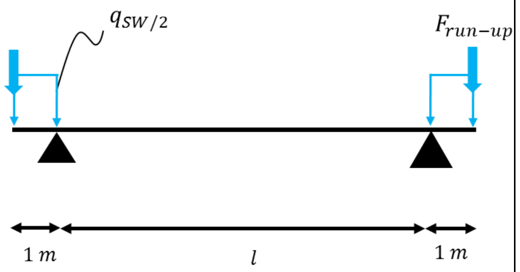
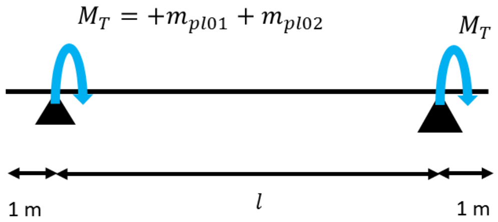
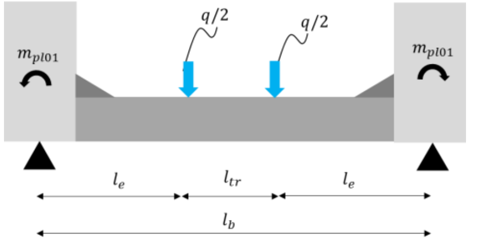
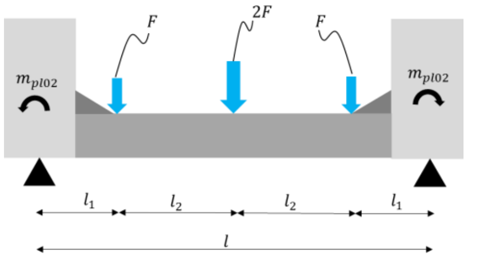
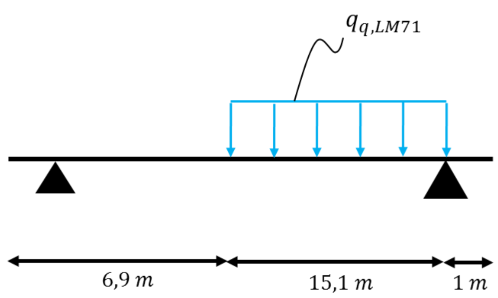
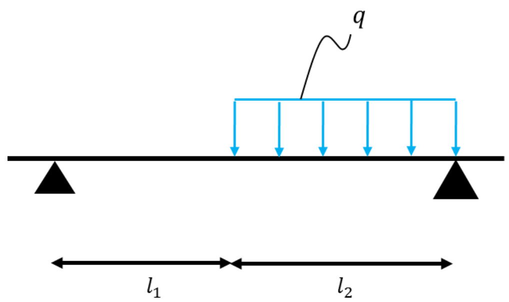
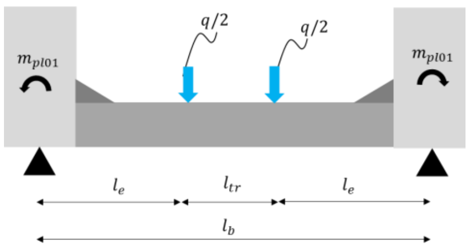
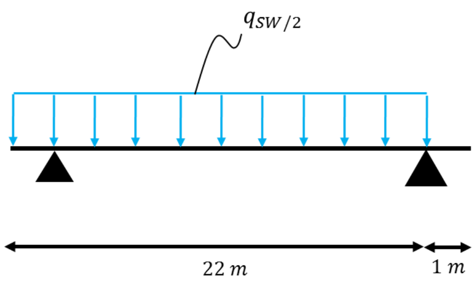
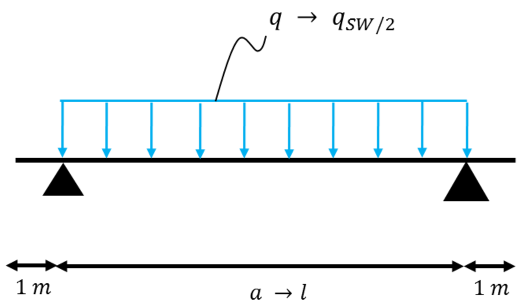
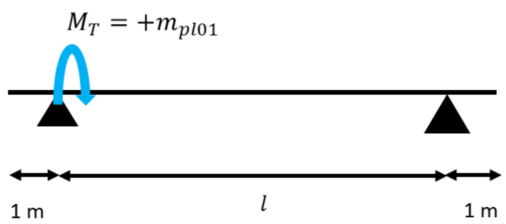
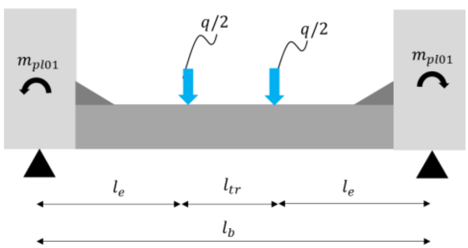
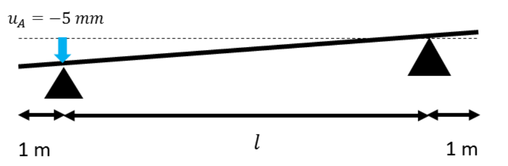
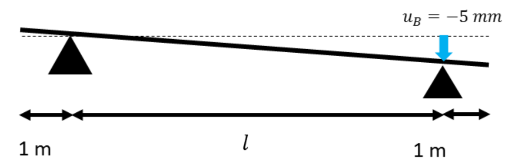
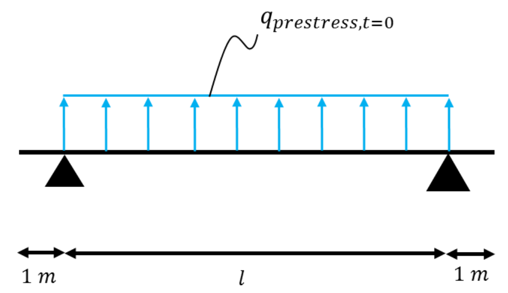
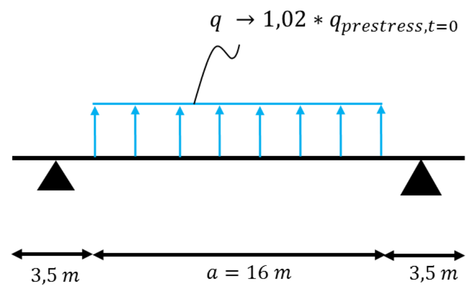
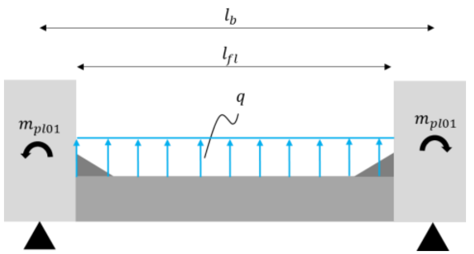
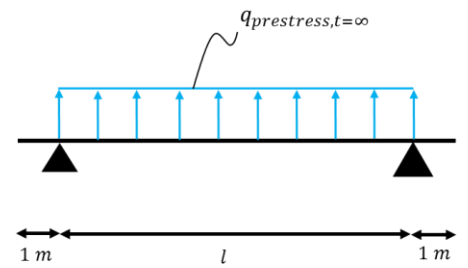
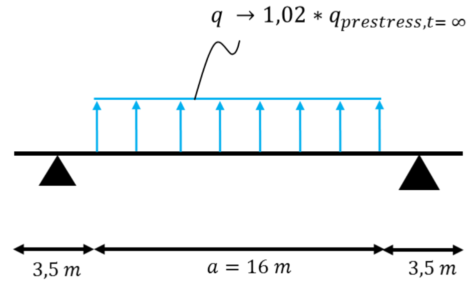
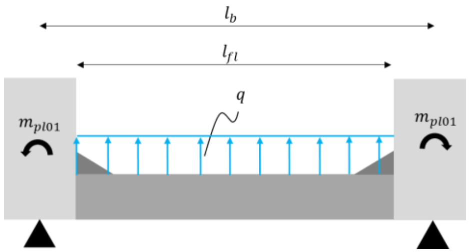
3– Con. Mobile Load (LM71)		**A.**  **B.** 	**A. & B.** 

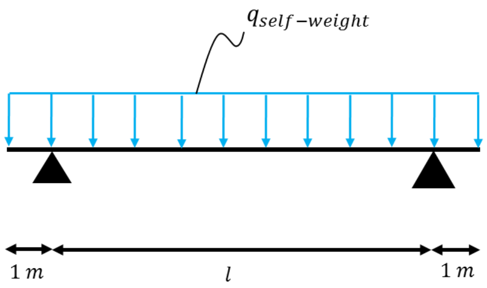
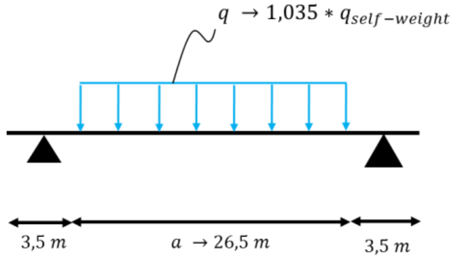
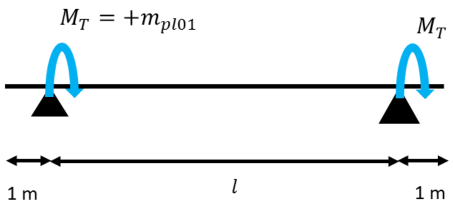
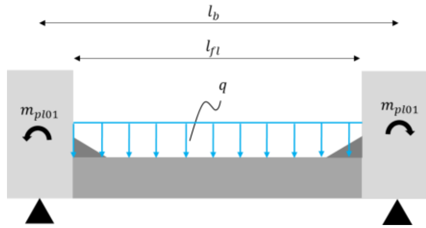
4– Cantilever fully loaded (SW/2)			**A.**  **B.** 

5a – Contin. Mobile load (LM71)		**A.** 	**A.** 
5b – Contin. Mobile load (SW/2)		**A.**  **B.** 	**A & B.** 

6 – Settlement support max.		**Torsion obtained from SCIA Plate Model**	
7 – Settlement support min.		**Torsion obtained from SCIA Plate Model**	
8 – Prestress at t=0		**A.** 	**A.** 
9 – Prestress at $t = \infty$		**A.** 	**A.** 

6.6 Bridge B

Table A-2: Torsion load cases bridge B

LC	Load scheme	Longitudinal loading	Transverse loading
1 - Self weight		**A.**  **B.** 	**A. & B.** 

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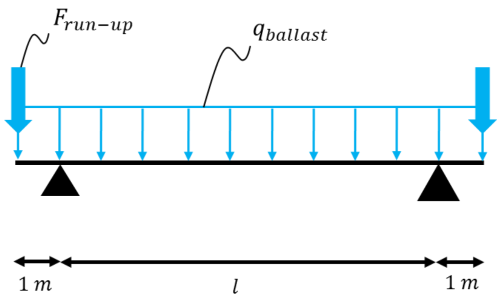
<td rowspan="2">2-Ballast</td>
<td rowspan="2">
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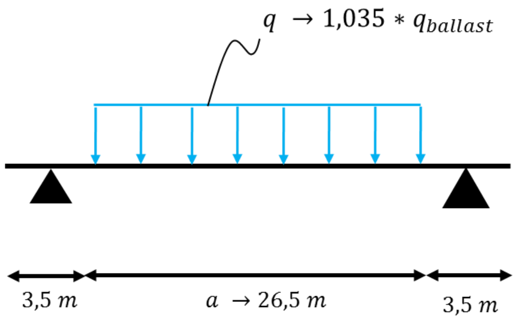
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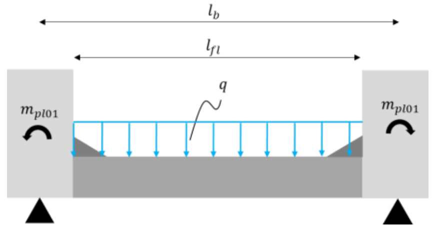
<td>A.

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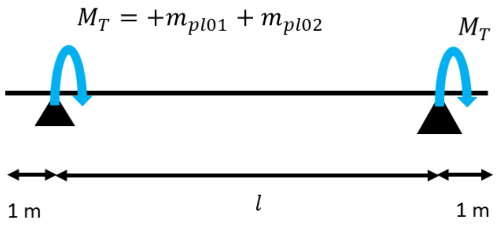
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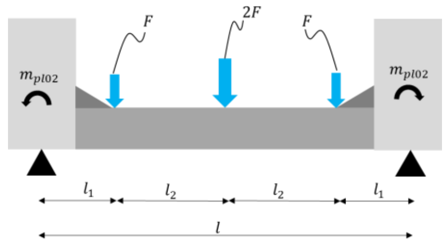
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<table>
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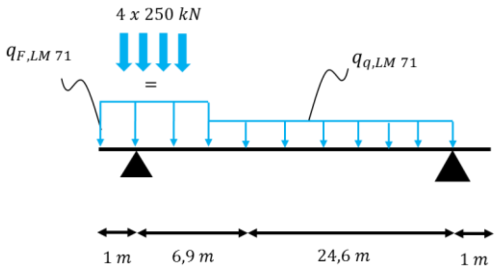
<td rowspan="3">3a–
Mobile Max
(LM 71)</td>
<td rowspan="3">
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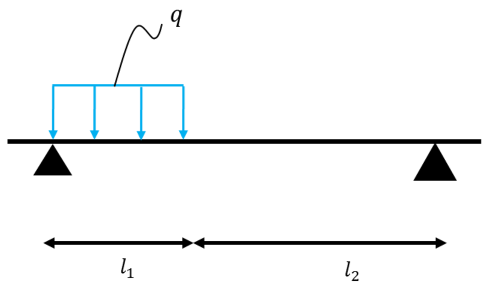
<td>A.

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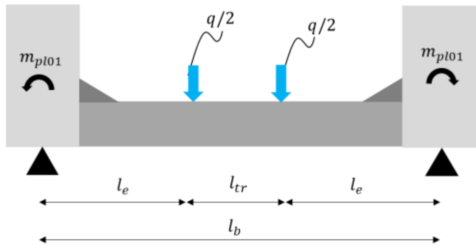
<td rowspan="2">A. & B.

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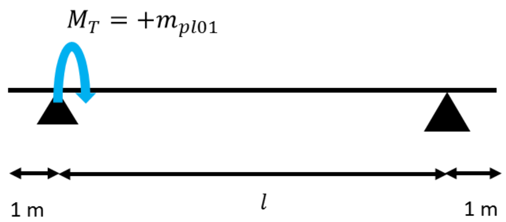
<td>B.

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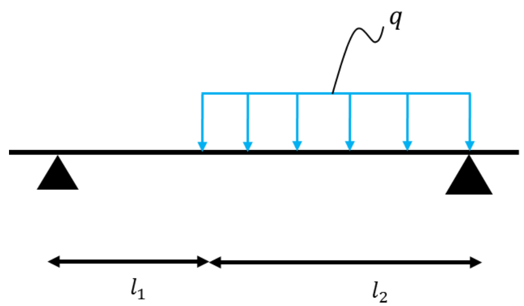
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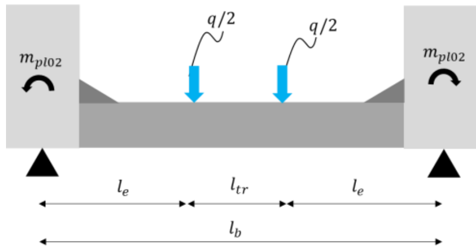
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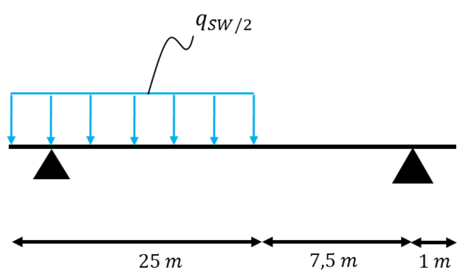
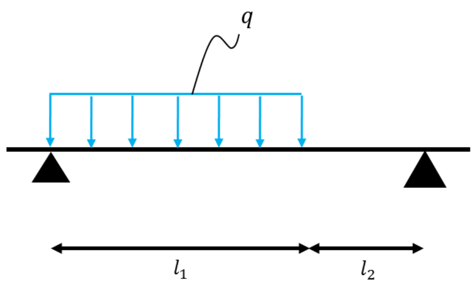
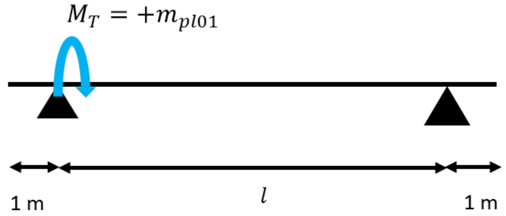
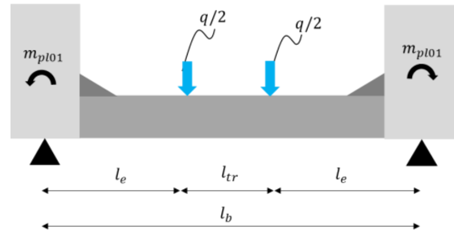
</table>

3b – Mobile Max (SW/2)		**A.**  **B.** 	**A. & B.** 

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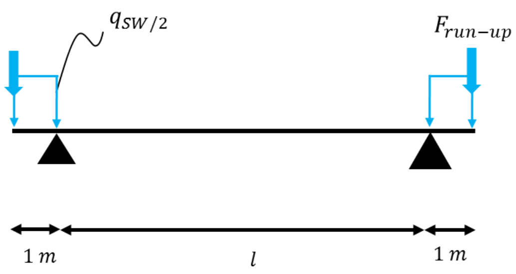
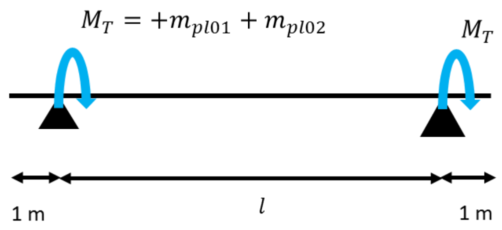
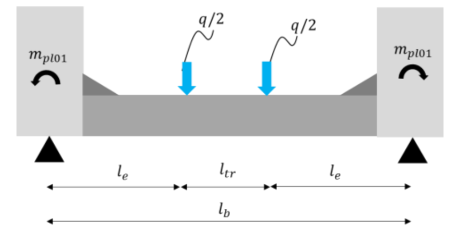
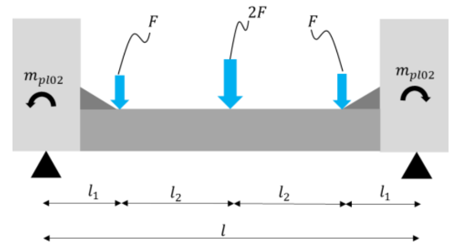
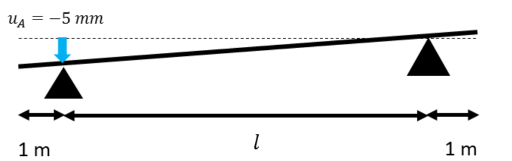
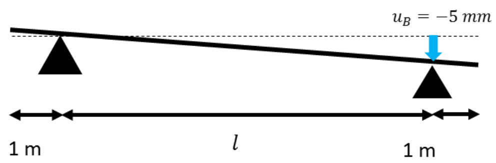
<tr><td>4– Cantilever fully loaded (SW/2)</td><td></td><td></td><td>A.

B.
</td></tr>
<tr><td>5 – Settlement support max.</td><td></td><td colspan="2">Torsion obtained from SCIA Plate Model</td></tr>
<tr><td>6 – Settlement support min.</td><td></td><td colspan="2">Torsion obtained from SCIA Plate Model</td></tr>
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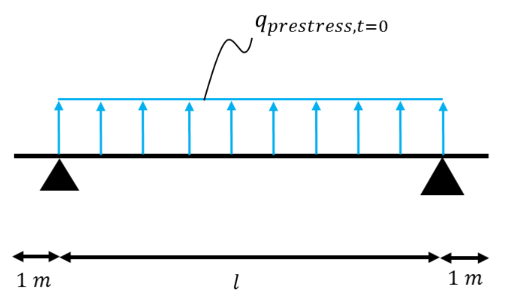
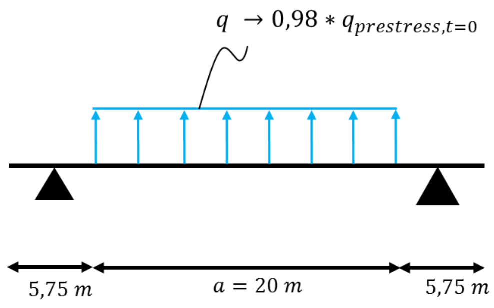
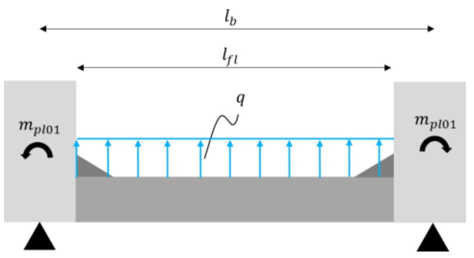
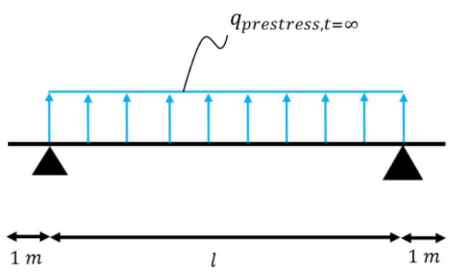
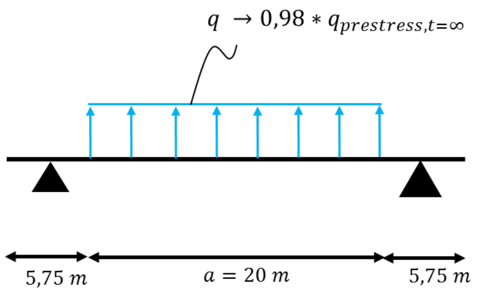
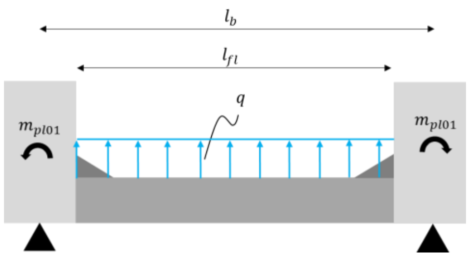
7 – Prestress at t=0		**A.** 	**A.** 
8 – Prestress at $t = \infty$		**A.** 	**A.** 

7 Maple sheets

7.1 Torsion

restart;

Torsion at the edge of the bridge

Parameters

The torsion spreads ver the entire lengt l:

$l := 21 :$

Torsion:

$mpl0 := 1;$

$$mpl0 := 1 \tag{1}$$

Ratio between the bending and torsional stiffness:

$\text{omega} := 0.24 :$

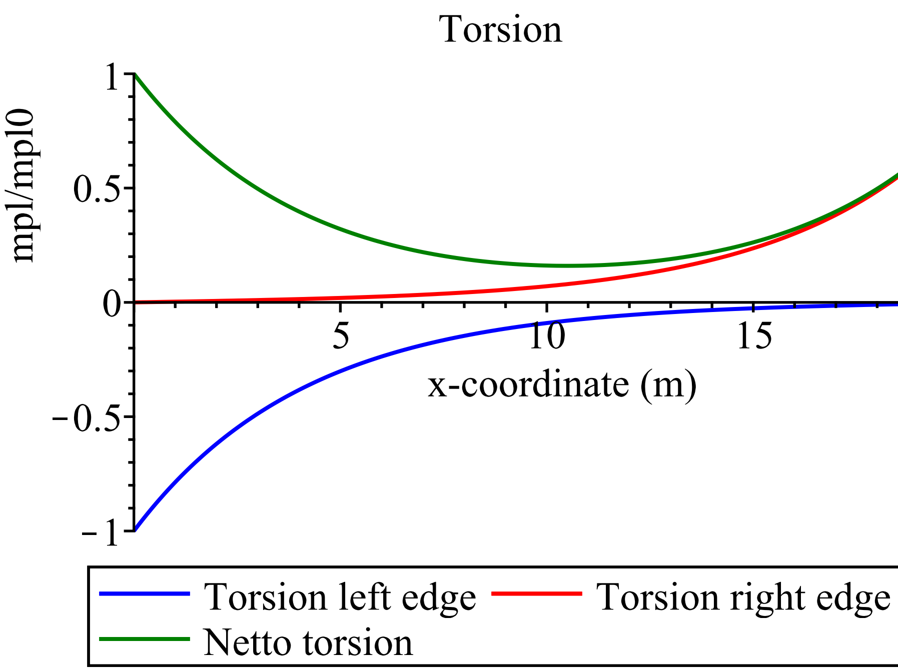
Functions

$$Mt0 := -mpl0 \cdot \left(\cosh(\text{omega} \cdot x) - \frac{\sinh(\text{omega} \cdot x)}{\tanh(\text{omega} \cdot l)} \right) :$$

$$MtL := \frac{mpl0 \cdot (\sinh(\text{omega} \cdot x))}{\sinh(\text{omega} \cdot l)} :$$

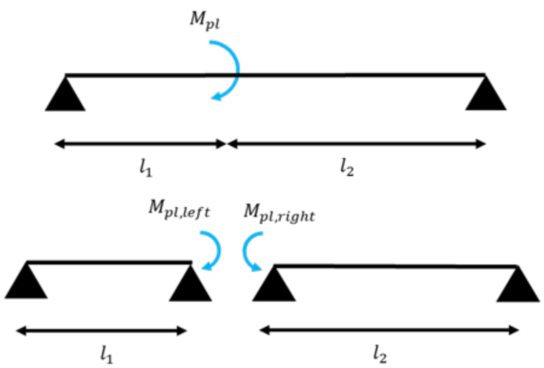
Graphs

```
with(plots) :
A := plot(Mt0, x = 0 ..l, style = line, color = "Blue", legend = ["Torsion left edge"], labels
    = ["x-coordinate (m) ", "mpl/mpl0"], labeldirections = [horizontal, vertical]) :
B := plot(MtL, x = 0 ..l, style = line, color = "Red", legend = ["Torsion right edge"], legendstyle
    = [location = bottom], title = ["Torsion"]) :
C := plot(MtL − Mt0, x = 0 ..l, style = line, color = "Green", legend = ["Netto torsion"]) :
display(A, B, C);
```


Torsion
mpl/mpl0
1
0.5
0
−0.5
−1
5
10
15
20
x-coordinate (m)
Torsion left edge
Torsion right edge
Netto torsion

restart;

Concentrated torsion at an arbitrary position



Parameters

Position of the concentrated torsional moment:

$a := 5 : \; l := 21 : \quad l1 := a : \qquad l2 := l - a :$

Primary load:

$Mpl := 1$

$$Mpl := 1 \tag{1}$$

Ratio between the bending and torsional stiffness:

$\text{omega} := 0.24 :$

Functions

$$Mpleft := \frac{\tanh(\text{omega} \cdot l1)}{\tanh(\text{omega} \cdot l1) + \tanh(\text{omega} \cdot l2)} \cdot Mpl :$$

$$Mpright := \frac{\tanh(\text{omega} \cdot l2)}{\tanh(\text{omega} \cdot l2) + \tanh(\text{omega} \cdot l1)} \cdot Mpl :$$

$$Mt1 := \frac{Mpleft \cdot \sinh(\text{omega} \cdot x)}{\sinh(\text{omega} \cdot l1)} :$$

$$Mt2 := -Mpright \cdot \left(\cosh(\text{omega} \cdot (x - l1)) - \frac{\sinh(\text{omega} \cdot (x - l1))}{\tanh(\text{omega} \cdot l2)} \right) :$$

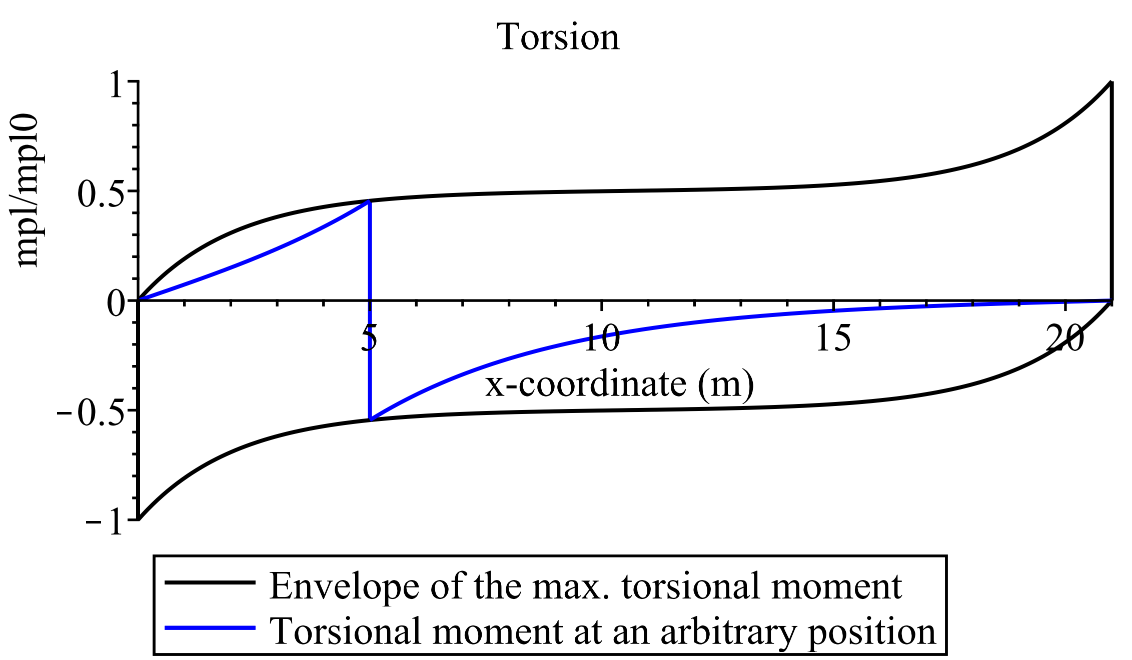
$$Me1 := \frac{\tanh(\text{omega}\cdot x)}{\tanh(\text{omega}\cdot x) + \tanh(\text{omega}\cdot (l - x))} :$$

$$Me2 := -\frac{\tanh(\text{omega}\cdot (l - x))}{\tanh(\text{omega}\cdot (l - x)) + \tanh(\text{omega}\cdot x)} :$$

$x := l1 :$
$a := Mt1 :$
$b := Mt2 :$
$x :=$'x':

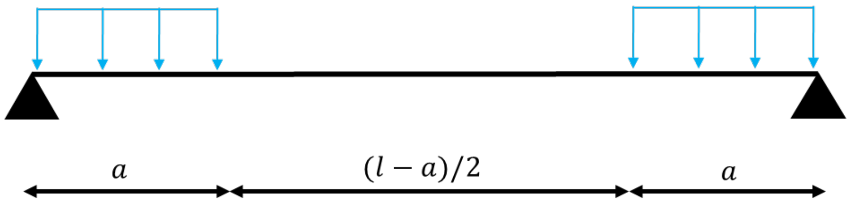
Graphs

```
with(plots) :
A := plot(Mt1, x = 0 ..l1, style = line, color = "Blue", legend
      = ["Torsional moment at an arbitrary position"]) :
B := plot(Mt2, x = l1 ..l, style = line, color = "Blue", legendstyle = [location = bottom]) :
C := plots[display](plottools[line]([l1, a], [l1, b]), color = blue) :
E := plot(Me1, x = 0 ..l, style = line, color = "Black") :
F := plot(Me2, x = 0 ..l, style = line, color = "Black", title = "Torsion") :
H := plots[display](plottools[line]([0, 0], [0, -Mpl]), color = black, legend
      = ["Envelope of the max. torsional moment "]) :
J := plots[display](plottools[line]([l, 0], [l, + Mpl]), color = black, labels = ["x-coordinate (m) ",
     "mpl/mpl0"], labeldirections = [horizontal, vertical]) :
display({A, B, C, E, F, H, J});
```



restart;

Torsion - Distributed load at the start/end



Parameters

Distributed load has a lenght of a:

$a := 5:\ l := 21:\quad l2 := a:\quad l1 := l - l2:$

Clamping moment:

$mpl0 := 1:$

Ratio between the bending and torsional stiffness:

$\text{omega} := 0.24:$

Functions

$$Mtrans := \frac{mpl0}{\text{omega}} \cdot \left(\frac{\tanh(\text{omega}\cdot x)\cdot\tanh(\text{omega}\cdot(l-x))}{\tanh(\text{omega}\cdot x) + \tanh(\text{omega}\cdot(l-x))} \right):$$

$$Mt1pos := \frac{Mtrans\cdot\sinh(\text{omega}\cdot x)}{\sinh(\text{omega}\cdot l1)}:$$

$$Mt2pos := Mtrans\cdot\left(\cosh(\text{omega}\cdot(x-l1)) - \frac{\sinh(\text{omega}\cdot(x-l1))}{\tanh(\text{omega}\cdot l2)}\right):$$

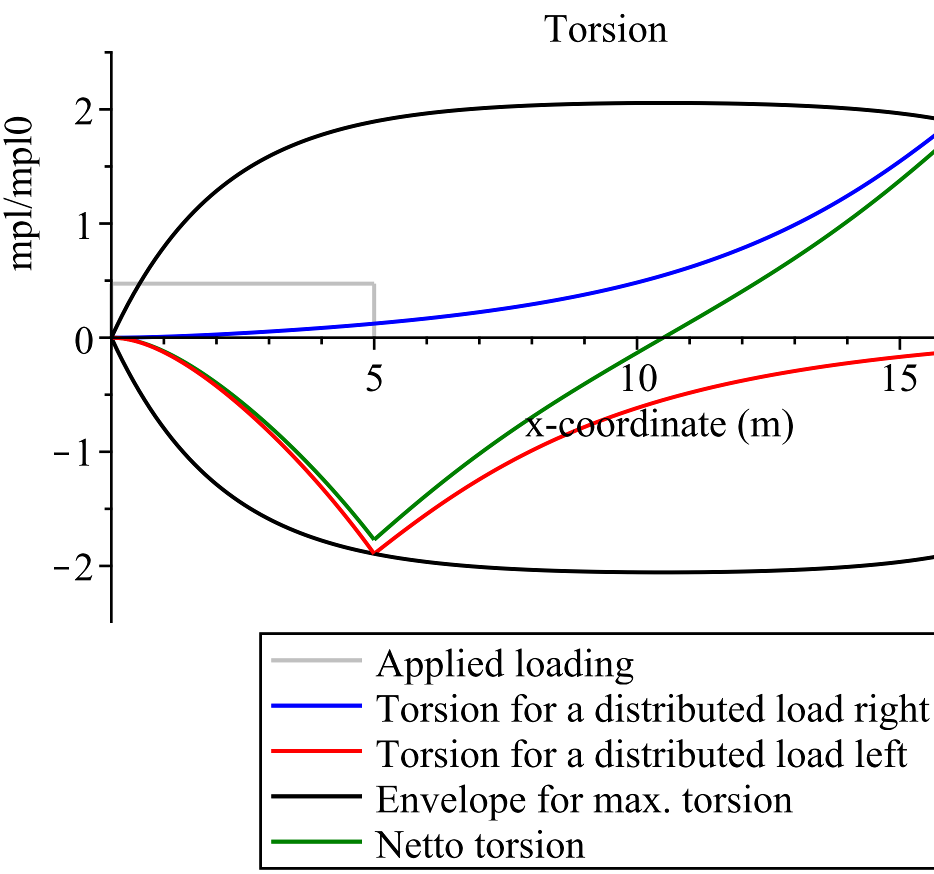
$x := l1:$
$C1 := Mt1pos:$
$x :=\text{'}x\text{'}:$

$$Mt1neg := -\frac{Mtrans\cdot\sinh(\text{omega}\cdot x)}{\sinh(\text{omega}\cdot l2)}:$$

$$Mt2neg := -Mtrans\cdot\left(\cosh(\text{omega}\cdot(x-l2)) - \frac{\sinh(\text{omega}\cdot(x-l2))}{\tanh(\text{omega}\cdot l1)}\right):$$

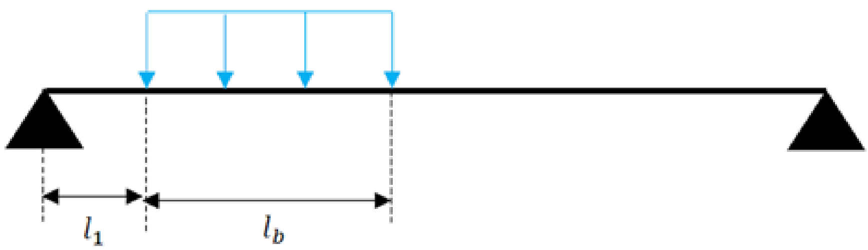
Graphs

```
with(plots) :
A := plot(Mt1pos, x = 0 ..l1, style = line, color = "Blue", legend
     = ["Torsion for a distributed load right"]) :
B := plot(Mt2pos, x = l1 ..l, style = line, color = "Blue", legendstyle = [location = bottom]) :
C := plot(Mtrans, x = 0 ..l, style = line, color = "Black", legend = ["Envelope for max. torsion"],
     legendstyle = [location = bottom]) :
E := plot(-Mtrans, x = 0 ..l, style = line, color = "Black") :
F := plot(Mt1neg, x = 0 ..l2, style = line, color = "Red", legend = ["Torsion for a distributed load left"],
     legendstyle = [location = bottom]) :
H := plot(Mt1pos + Mt1neg, x = 0 ..l2, style = line, color = "Green") :
J := plot(Mt1pos + Mt2neg, x = l2 ..(l - l2), style = line, color = "Green") :
K := plot(Mt2pos + Mt2neg, x = (l - l2) ..(l), style = line, color = "Green", legend = ["Netto torsion"],
     legendstyle = [location = bottom], title = ("Torsion")) :
G := plot(Mt2neg, x = l2 ..l, style = line, color = "Red", labels = ["x-coordinate (m) ", "mpl/mpl0"],
     labeldirections = [horizontal, vertical]) :

a1 := plots[display](plottools[line]([0, 0], [0, C1/4]), color = grey, legend = ["Applied loading"],
     legendstyle = [location = bottom]) :
b1 := plots[display](plottools[line]([0, C1/4], [a, C1/4]), color = grey) :
c1 := plots[display](plottools[line]([a, C1/4], [a, 0]), color = grey) :
a11 := plots[display](plottools[line]([l1, 0], [l1, C1/4]), color = grey) :
b11 := plots[display](plottools[line]([l1, C1/4], [l, C1/4]), color = grey) :
c11 := plots[display](plottools[line]([l, C1/4], [l, 0]), color = grey, view = [0 ..21, -2.5 ..2.5]) :

display({A, B, C, E, F, G, H, J, K, a1, b1, c1, a11, b11, c11});
```


Torsion
mpl/mpl0
2
1
0
−1
−2
5
10
15
20
x-coordinate (m)
Applied loading
Torsion for a distributed load right
Torsion for a distributed load left
Envelope for max. torsion
Netto torsion

restart;

Torsional due to local load



Parameters

The length of the local mobile load is equal to a:

$lb := 5:$ $l := 20:$ $l1 := 7.5:$ $l2 := l - lb - l1:$

Primary load:

$mpl1 := 1:$

Ratio between the bending and torsional stiffness:

$\text{omega} := 0.24:$

Functions

$$MtransII := \frac{mpl1}{\text{omega}} \cdot \left(\frac{\tanh(\text{omega} \cdot x) \cdot \tanh(\text{omega} \cdot lb)}{\tanh(\text{omega} \cdot x) + \tanh(\text{omega} \cdot lb)} \right):$$

$$MtransIII := \frac{mpl1}{\text{omega}} \cdot \left(\frac{\tanh(\text{omega} \cdot (-x + l)) \cdot \tanh(\text{omega} \cdot lb)}{\tanh(\text{omega} \cdot (-x + l)) + \tanh(\text{omega} \cdot lb)} \right):$$

$$Mpleft := \frac{\tanh(\text{omega} \cdot l1)}{\tanh(\text{omega} \cdot l1) + \tanh(\text{omega} \cdot l2)}:$$

$$Mpright := \frac{\tanh(\text{omega} \cdot l2)}{\tanh(\text{omega} \cdot l1) + \tanh(\text{omega} \cdot l2)}:$$

$x := lb:$
$C := MtransII:$
$x :='x':$

$$MenvI := \frac{\tanh(\text{omega}\cdot(x))}{\tanh(\text{omega}\cdot(x)) + \tanh(\text{omega}\cdot(l - lb - x))}\cdot C:$$

$$MenvII := -\frac{\tanh(\text{omega}\cdot(l-x))}{\tanh(\text{omega}\cdot(l-x)) + \tanh(\text{omega}\cdot(x - lb))}\cdot C:$$

$$MxyI := \frac{\sinh(\text{omega}\cdot(x))}{\sinh(\text{omega}\cdot(l1))}\cdot Mpleft\cdot C:$$

$$MxyII := -\left(-\frac{\sinh(\text{omega}\cdot(x - l1 - lb))}{\tanh(\text{omega}\cdot(l2))} + \cosh(\text{omega}\cdot(x - l1 - lb))\right)\cdot C\cdot Mpright:$$

$x := l1:$
$E1 := MxyI:$
$x :='x':$

$x := l1 + lb:$
$F1 := MxyII:$
$x :='x':$

Graphs

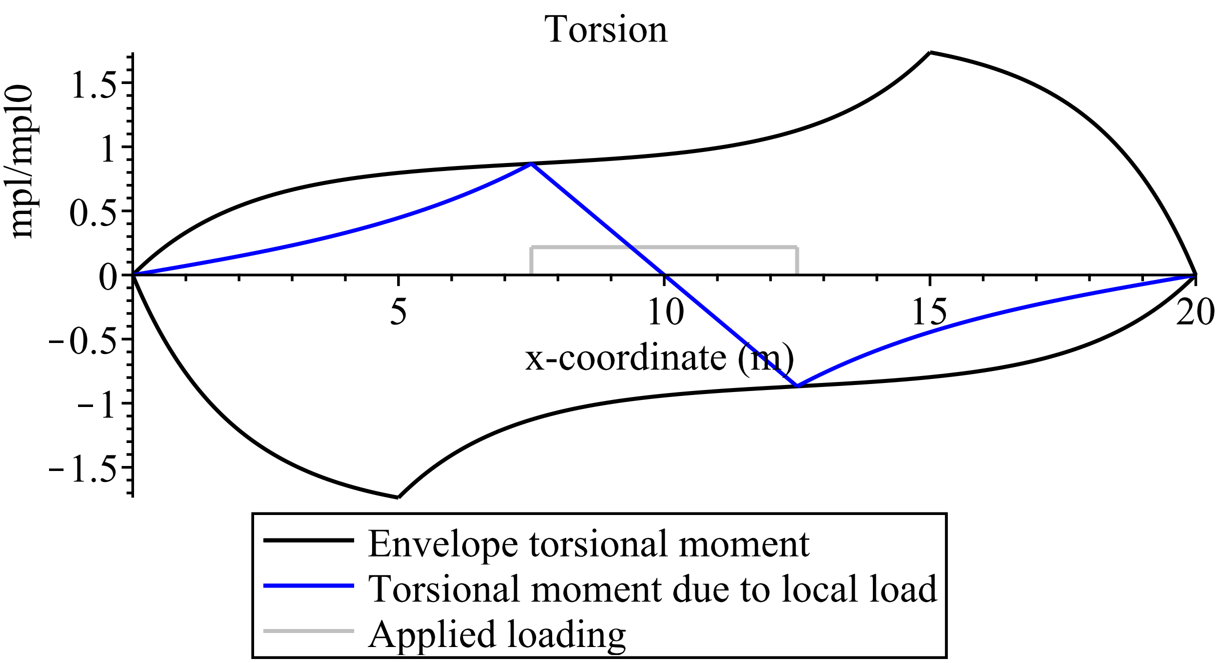
with(*plots*) :
K := *plot*(−*MtransII*, *x* = 0 ..*lb*, *color* = "Black", *legend* = ["Envelope torsional moment"], *legendstyle* = [*location* = *bottom*], *labels* = ["x-coordinate (m) ", "mpl/mpl0"], *labeldirections* = [*horizontal*, *vertical*]) :
L := *plot*(*MtransIII*, *x* = *l* − *lb* ..*l*, *color* = "Black") :
M := *plot*(*MenvI*, *x* = 0 ..*l*−*lb*, *color* = "Black") :
N := *plot*(*MenvII*, *x* = *lb* ..*l*, *color* = "Black") :
P := *plot*(*MxyI*, *x* = 0 ..*l1*, *color* = "Blue", *legend* = ["Torsional moment due to local load"], *legendstyle* = [*location* = *bottom*], *title* = ("Torsion")) :
Q := *plot*(*MxyII*, *x* = *l1* + *lb* ..*l*, *color* = "Blue") :

$R := plots[display]\left(plottools[line]\left([l1, 0], \left[l1, \frac{C}{8}\right]\right), color = grey, legend = [\text{"Applied loading"}], legendstyle = [location = bottom]\right):$

$S := plots[display]\left(plottools[line]\left(\left[l1, \frac{C}{8}\right], \left[l1 + lb, \frac{C}{8}\right]\right), color = grey, legend = [\text{"Applied loading"}], legendstyle = [location = bottom]\right):$

$T := plots[display]\left(plottools[line]\left(\left[l1 + lb, \frac{C}{8}\right], [l1 + lb, 0]\right), color = grey, legend = [\text{"Applied loading"}], legendstyle = [location = bottom]\right):$

U := *plots*[*display*](*plottools*[*line*]([*l1*, *E1*], [*l1* + *lb*, *F1*]), *color* = *blue*) :
display(*K*, *L*, *M*, *N*, *P*, *Q*, *R*, *S*, *T*, *U*);


Torsion
mpl/mpl0
1.5
1
0.5
0
−0.5
−1
−1.5
5
10
15
20
x-coordinate (m)
Envelope torsional moment
Torsional moment due to local load
Applied loading

7.2 Clamping moment

restart;

Clamping moment - Torsion at the edge of the bridge

Parameters

The moments are spread over the entire lengt l:

$l := 21 :$

Primary load:

$mpl0 := 1;$

$$mpl0 := 1 \quad (1)$$

Ratio between the bending and torsional stiffness:

$\text{omega} := 0.24 :$

Functions

$$mx0 := -mpl0 \cdot \text{omega} \cdot \left(\sinh(\text{omega} \cdot x) - \frac{\cosh(\text{omega} \cdot x)}{\tanh(\text{omega} \cdot l)} \right) :$$

$$mxL := mpl0 \cdot \text{omega} \cdot \frac{(\cosh(\text{omega} \cdot x))}{\sinh(\text{omega} \cdot l)} :$$

Graphs

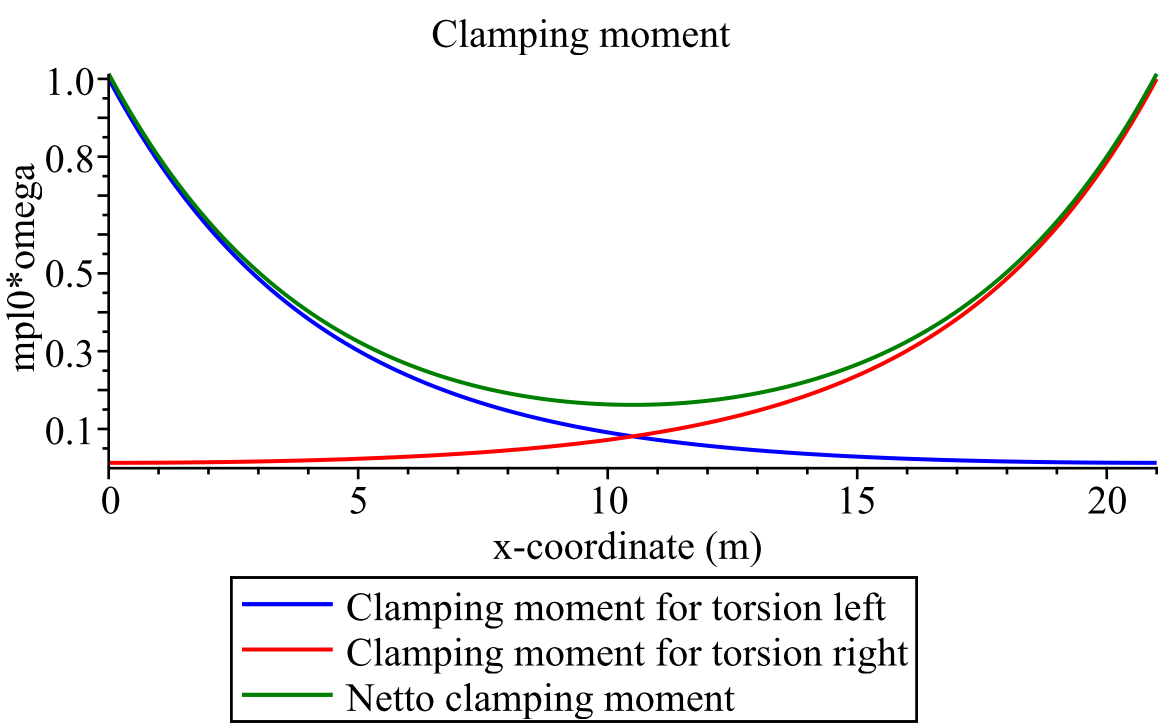
with(*plots*) :

$A := plot\left(\frac{mx0}{mpl0 \cdot \text{omega}}, x = 0 .. l, style = line, color = \text{"Blue"}, legend = [\text{"Clamping moment for torsion left"}], labels = [\text{"x-coordinate (m) "}, \text{"mpl0·omega"}], labeldirections = [horizontal, vertical]\right) :$

$B := plot\left(\frac{mxL}{mpl0 \cdot \text{omega}}, x = 0 .. l, style = line, color = \text{"Red"}, legend = [\text{"Clamping moment for torsion right"}], legendstyle = [location = bottom], title = [\text{"Clamping moment"}]\right) :$

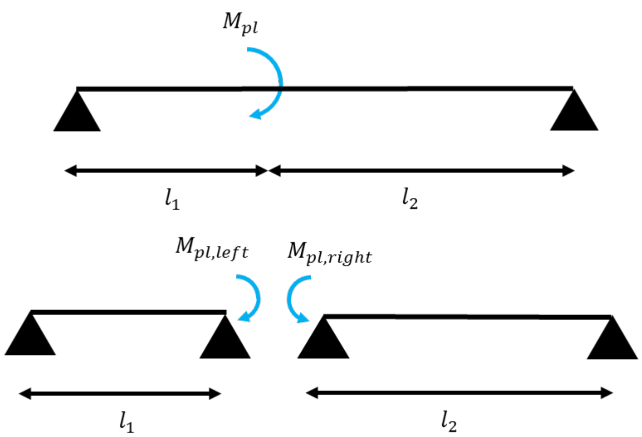
$C := plot\left(\frac{(mx0 + mxL)}{mpl0 \cdot \text{omega}}, x = 0 .. l, style = line, color = \text{"Green"}, legend = [\text{"Netto clamping moment"}]\right) :$

display(*A*, *B*, *C*);



restart;

Clamping moment - Concentrated torsional moment



Parameters

The moments are spread over the entire lengt l:

$l := 21 : \quad l1 := 5 : \qquad l2 := l - l1 :$

Primary load:

$mpl0 := 1;$

$$mpl0 := 1 \tag{1}$$

Ratio between the bending and torsional stiffness:

$\text{omega} := 0.24 :$

Functions

$$Mpleft := \frac{\tanh(\text{omega} \cdot l1)}{\tanh(\text{omega} \cdot l1) + \tanh(\text{omega} \cdot l2)} \cdot mpl0 :$$

$$Mpright := \frac{\tanh(\text{omega} \cdot l2)}{\tanh(\text{omega} \cdot l2) + \tanh(\text{omega} \cdot l1)} \cdot mpl0 :$$

$$mx0 := -mpl0 \cdot \text{omega} \cdot \left(\sinh(\text{omega} \cdot (x - l1)) - \frac{\cosh(\text{omega} \cdot (x - l1))}{\tanh(\text{omega} \cdot l2)} \right) \cdot Mpright :$$

$$mxL := mpl0 \cdot \text{omega} \cdot \frac{\cosh(\text{omega} \cdot x)}{\sinh(\text{omega} \cdot l1)} \cdot Mpleft :$$

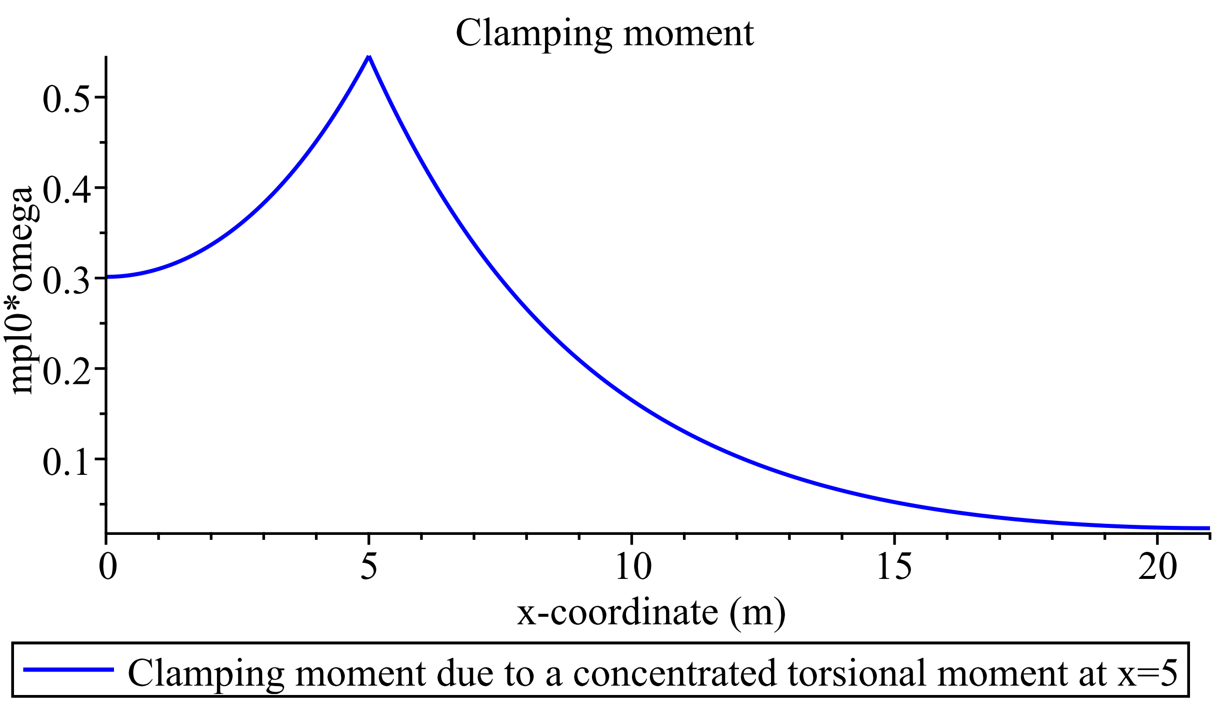
Graphs

with(*plots*) :

$A := plot\left(\dfrac{mx0}{mpl0 \cdot \text{omega}}, x = l1 .. l, style = line, color = \text{"Blue"}, legend = [\text{"Clamping moment due to a concentrated torsional moment at x=5"}], labels = [\text{"x-coordinate (m) "}, \text{"mpl0·omega"}], labeldirections = [horizontal, vertical]\right) :$

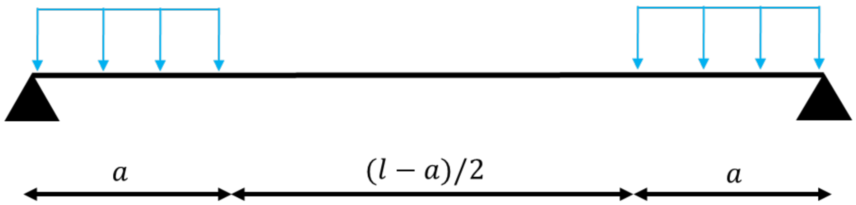
$B := plot\left(\dfrac{mxL}{mpl0 \cdot \text{omega}}, x = 0 .. l1, style = line, color = \text{"Blue"}, title = \text{"Clamping moment"}\right) :$

display(*A*, *B*);



restart;

Clamping moment - Distributed load at the start/end



Parameters

The distributed load has a length of a:

$a := 5 : \; l := 21 : \quad l2 := a : \quad l1 := l - l2 :$

Primary load:

$mpl0 := 1 :$

Ratio between the bending and torsional stiffness:

$\text{omega} := 0.24 :$

Functions

$$Mtrans := \frac{mpl0}{\text{omega}} \cdot \left(\frac{\tanh(\text{omega} \cdot l1) \cdot \tanh(\text{omega} \cdot (l2))}{\tanh(\text{omega} \cdot l1) + \tanh(\text{omega} \cdot (l2))} \right) :$$

$$mxL1 := \text{omega} \cdot \frac{Mtrans \cdot \cosh(\text{omega} \cdot x)}{\sinh(\text{omega} \cdot l1)} :$$

$$mxL2 := \text{omega} \cdot Mtrans \cdot \left(\sinh(\text{omega} \cdot (x - l1)) - \frac{\cosh(\text{omega} \cdot (x - l1))}{\tanh(\text{omega} \cdot l2)} \right) :$$

$x := l1 :$
$C1 := mxL1 :$
$x :=$'x':

$$mx01 := -\frac{Mtrans \cdot \text{omega} \cdot \cosh(\text{omega} \cdot x)}{\sinh(\text{omega} \cdot l2)} :$$

$$mx02 := -Mtrans \cdot \text{omega} \cdot \left(\sinh(\text{omega} \cdot (x - l2)) - \frac{\cosh(\text{omega} \cdot (x - l2))}{\tanh(\text{omega} \cdot l1)} \right) :$$

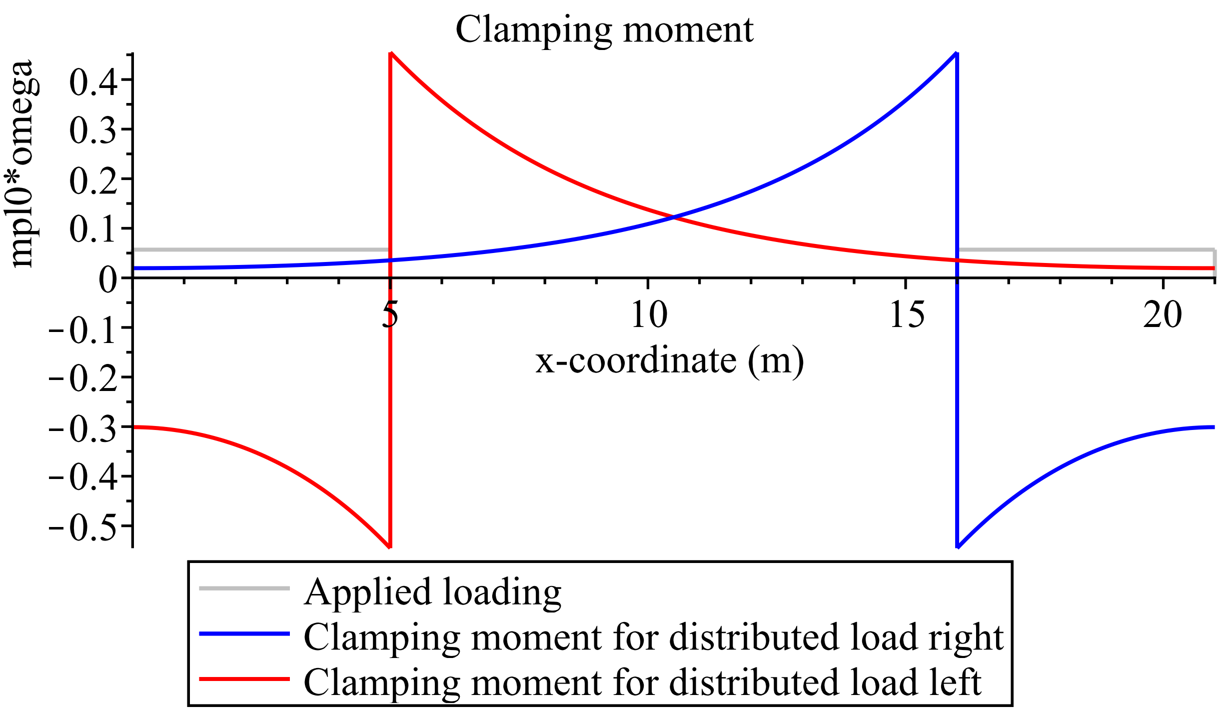
$x := a :$
$C3 := mx01 :$
$C4 := mx02 :$
$x :=$'x':

$x := l - a :$
$C5 := mxL1 :$
$C6 := mxL2 :$
$x :=$'x':

Graphs

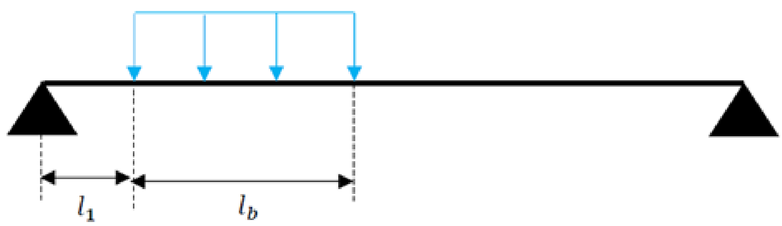
$with(plots) :$
$A := plot(mxL1, x = 0 ..l1, style = line, color =$ "Blue", $legend$
= ["Clamping moment for distributed load right"]) :
$B := plot(mxL2, x = l1 ..l, style = line, color =$ "Blue", $legendstyle = [location = bottom]) :$
$F := plot(mx01, x = 0 ..l2, style = line, color =$ "Red", $legend$
= ["Clamping moment for distributed load left"], $legendstyle = [location = bottom]) :$
$G := plot(mx02, x = l2 ..l, style = line, color =$ "Red", $labels =$ ["x-coordinate (m) ", "mpl0·omega"],
$labeldirections = [horizontal, vertical]) :$

$H := plots[display](plottools[line]([a, C4], [a, C3]), color = red) :$
$J := plots[display](plottools[line]([l - a, C5], [l - a, C6]), color = blue) :$
$a1 := plots[display]\left(plottools[line]\left([0, 0], \left[0, \frac{C1}{8}\right]\right), color = grey, legend =\right.$ ["Applied loading"],
$\left. legendstyle = [location = bottom]\right) :$
$b1 := plots[display]\left(plottools[line]\left(\left[0, \frac{C1}{8}\right], \left[a, \frac{C1}{8}\right]\right), color = grey\right) :$
$c1 := plots[display]\left(plottools[line]\left(\left[a, \frac{C1}{8}\right], [a, 0]\right), color = grey\right) :$
$a11 := plots[display]\left(plottools[line]\left([l1, 0], \left[l1, \frac{C1}{8}\right]\right), color = grey\right) :$
$b11 := plots[display]\left(plottools[line]\left(\left[l1, \frac{C1}{8}\right], \left[l, \frac{C1}{8}\right]\right), color = grey\right) :$
$c11 := plots[display]\left(plottools[line]\left(\left[l, \frac{C1}{8}\right], [l, 0]\right), color = grey, title =\right.$ ["Clamping moment"] $\Big) :$
$display(\{A, B, F, G, H, J, a1, b1, c1, a11, b11, c11\});$


Clamping moment
mpl0*omega
0.4
0.3
0.2
0.1
0
−0.1
−0.2
−0.3
−0.4
−0.5
5
10
15
20
x-coordinate (m)
Applied loading
Clamping moment for distributed load right
Clamping moment for distributed load left

restart;

Clamping moment due to local load



Parameters

The length of the local mobile load is equal to a:

$lb := 5 :$ $l := 20 :$ $l1 := 7.5 :$ $l2 := l - lb - l1 :$

Primary load:

$mpl1 := 1 :$

Ratio between the bending and torsional stiffness:

$\text{omega} := 0.24 :$

Functions Torsion

$$MtransII := \frac{mpl1}{\text{omega}} \cdot \left(\frac{\tanh(\text{omega} \cdot x) \cdot \tanh(\text{omega} \cdot lb)}{\tanh(\text{omega} \cdot x) + \tanh(\text{omega} \cdot lb)} \right) :$$

$$Mpleft := \frac{\tanh(\text{omega} \cdot l1)}{\tanh(\text{omega} \cdot l1) + \tanh(\text{omega} \cdot l2)} :$$

$$Mpright := \frac{\tanh(\text{omega} \cdot l2)}{\tanh(\text{omega} \cdot l1) + \tanh(\text{omega} \cdot l2)} :$$

$x := lb :$
$C := MtransII :$
$x :='x':$

$$MxyI := \frac{\sinh(\text{omega} \cdot (x))}{\sinh(\text{omega} \cdot (l1))} \cdot Mpleft \cdot C :$$

$$MxyII := -\left(-\frac{\sinh(\text{omega} \cdot (x - l1 - lb))}{\tanh(\text{omega} \cdot (l2))} + \cosh(\text{omega} \cdot (x - l1 - lb)) \right) \cdot C \cdot Mpright :$$

Functions Clamping Moment

$$mxxI := \text{omega}\cdot \frac{\cosh(\text{omega}\cdot (x))}{\sinh(\text{omega}\cdot (l1))}\cdot Mpleft\cdot C:$$

$$mxxII := -\text{omega}\cdot \left(-\frac{\cosh(\text{omega}\cdot (x - l1 - lb))}{\tanh(\text{omega}\cdot (l2))} + \sinh(\text{omega}\cdot (x - l1 - lb))\right)\cdot Mpright\cdot C:$$

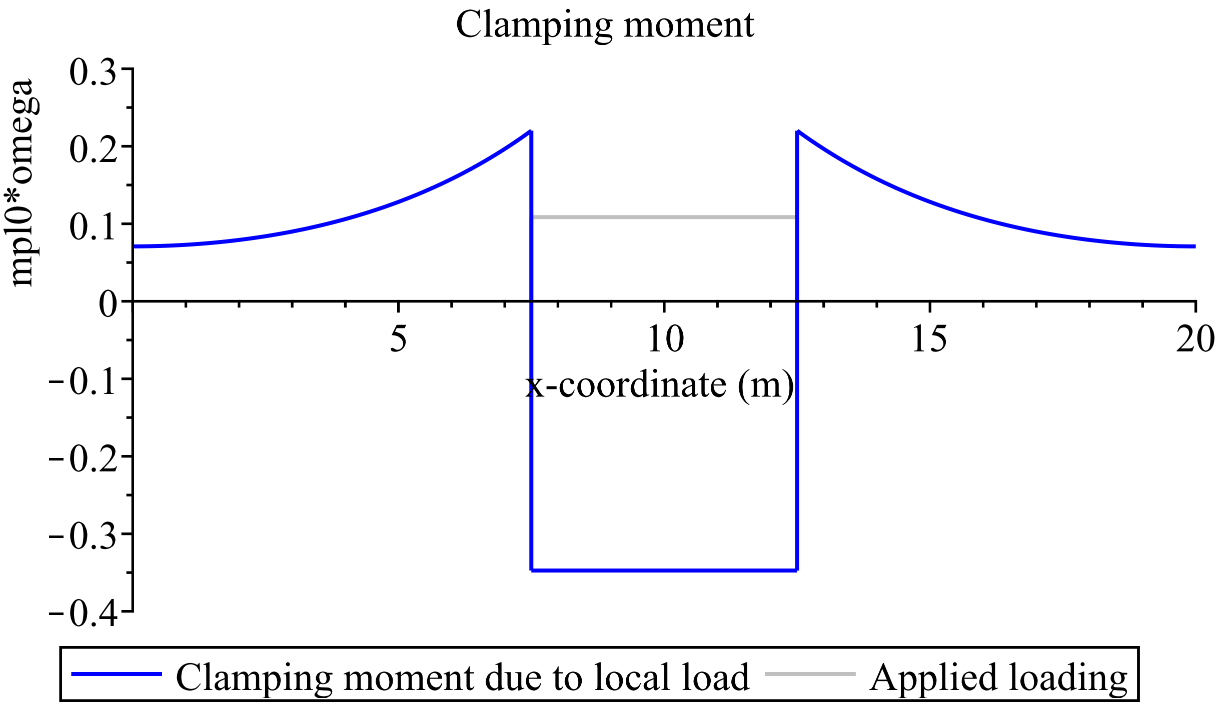
$x := l1:$
$E1 := MxyI:$
$E2 := mxxI:$
$x :=$'x':

$x := l1 + lb:$
$F1 := MxyII:$
$F2 := mxxII:$
$x :=$'x':

$$mxxderivative := \frac{(E1 - F1)}{lb}:$$

Graphs

```
with(plots) :
P1 := plot(mxxI, x = 0 ..l1, color = "Blue", legend = ["Clamping moment due to local load"], legendstyle
      = [location = bottom], title = ("Clamping moment"), labels = ["x-coordinate (m) ", "mpl0·omega"],
      labeldirections = [horizontal, vertical]) :

P2 := plot( -mxxderivative, x = l1 ..l1 + lb, color = "Blue") :
P3 := plot(mxxII, x = l1 + lb ..l, color = "Blue", view = [0 ..20, -0.4 ..0.3]) :
R := plots[display](plottools[line]( [l1, 0], [l1, C/16]), color = grey, legend = ["Applied loading"],
      legendstyle = [location = bottom]) :
S := plots[display](plottools[line]([l1, C/16], [l1 + lb, C/16]), color = grey, legend
      = ["Applied loading"], legendstyle = [location = bottom]) :
T := plots[display](plottools[line]([l1 + lb, C/16], [l1 + lb, 0]), color = grey, legend
      = ["Applied loading"], legendstyle = [location = bottom]) :

V := plots[display](plottools[line]([l1, -mxxderivative], [l1, E2]), color = blue) :
U := plots[display](plottools[line]([l1 + lb, -mxxderivative], [l1 + lb, F2]), color = blue) :
display(P1, P2, P3, R, S, T, V, U);
```


Clamping moment
mpl0*omega
0.3
0.2
0.1
0
−0.1
−0.2
−0.3
−0.4
5
10
15
20
x-coordinate (m)
Clamping moment due to local load
Applied loading

Appendix B – Shear tension failure VB 74 (SLS)

1 Introduction

Critical for the through bridge is a combination of shear, torsion and bending. At a section, just above the connection between the floor and the girder, the combination of normal and shear stresses is expected to be governing. This combination of stresses is called the principal stress. The tensile principal stress cannot exceed the tensile strength of the concrete, but if it does, a shear tension crack will develop. This is a very sudden and brittle failure mechanism, which should by avoided by all means.

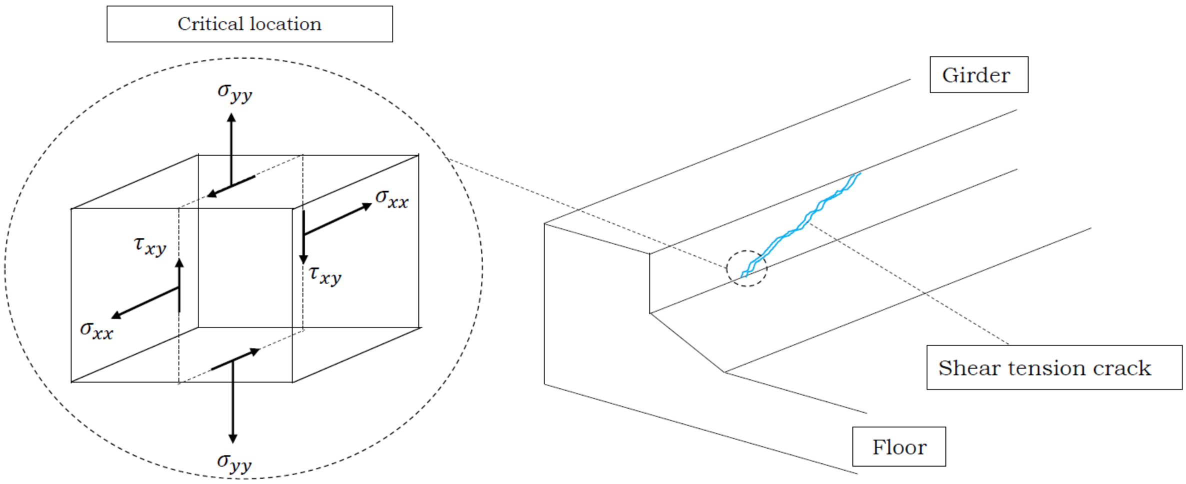


Figure B-1: Shear tension crack in a through bridge

Bridge A (3) and B (21) were designed using a combination of the VB 74 (7) and the VOSB 1963 (8). The load models used at the time, are quite different from the Eurocode load models. One of the objectives in this master thesis is to verify whether or not the change in load models enhances the risk of shear tension failure. Hence the purpose of this appendix is presenting an overview of the maximum tensile principal stresses, which were found in the design reports of bridge A and B.

According to the VB 74, the check on shear tension failure, is performed at serviceability limit state. Part F of the standard states that the safety factors for self-weight (γ_g), ballast (γ_r) and mobile loads (γ_q) are all equal to 1,0. Additionally the tensile principal stress cannot grow larger than half the characteristic tensile strength. However if it does, a shear tension crack will develop.

$$U.C. = \frac{\rho_1}{0.5 * f_{ctk}} \leq 1{,}0 \qquad [\text{ B.1 }]$$

Where:
$\rho_1 = maximum\ tensile\ principal\ stress$
$f_{ctk} = characteristic\ concrete\ tensile\ strength$

2 Load cases

The VOSB 1963 defines 3 different load models for railway traffic. Yet in the design report only the VOSB 150 is considered. It consists of a group of concentrated loads (with 150 kN per load) and a distributed load of 80 kN/m. The minimum spacing between the group of concentrated loads is 17 meters.

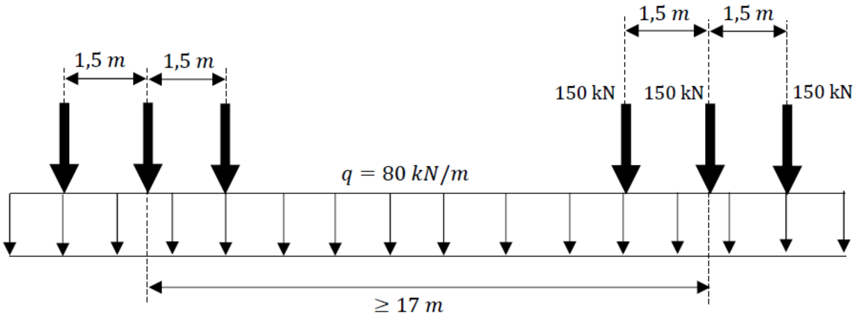


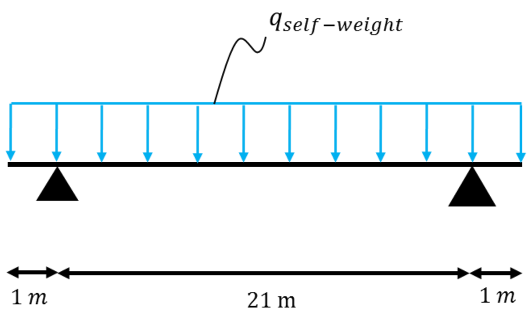
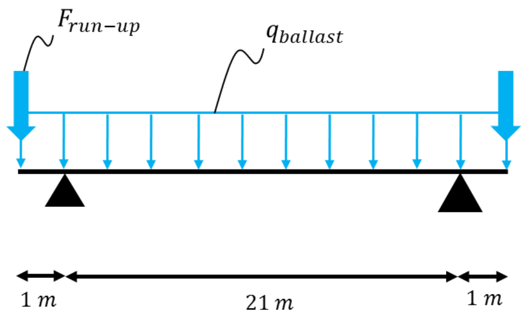
Figure B-2: VOSB 150

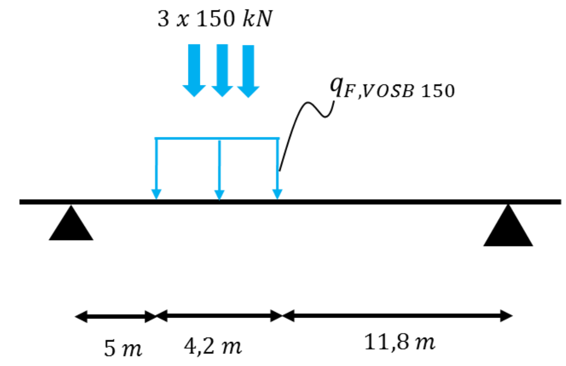
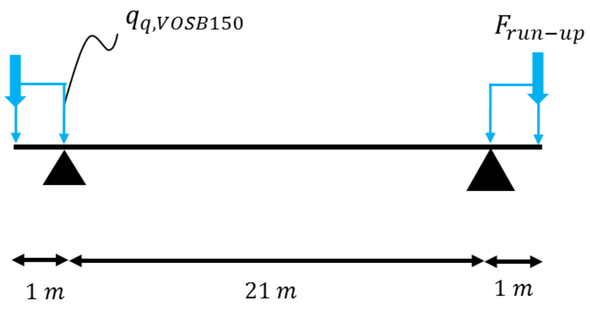
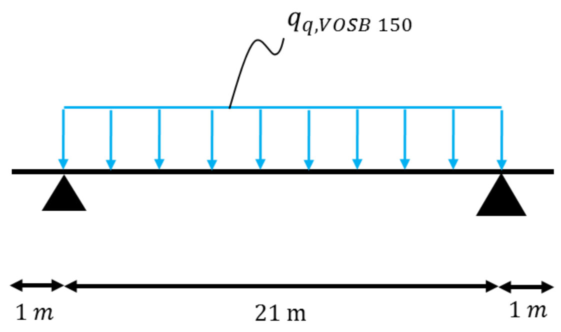
The following remarks can give one a better understanding of the applied load cases (presented by Table B-1 and Table B-2):

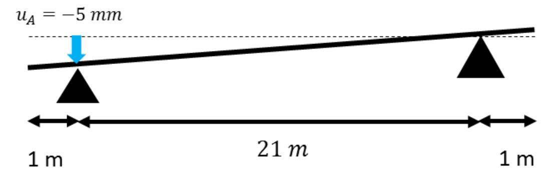
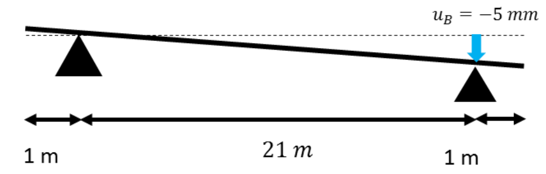
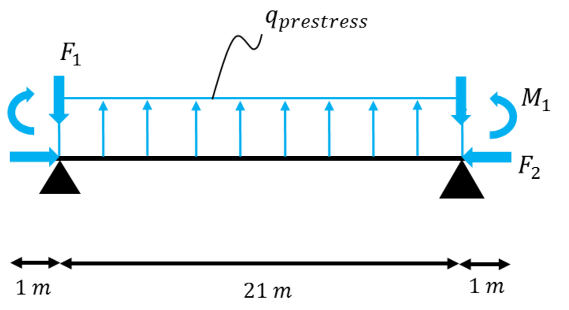
- **VOSB 150:** For bridge A, the concentrated and distributed loads are split into LC 3 and LC 5 respectively.
- **Run-up plate:** In the run-up to the bridge two plates at either side are applied. These plates are supported by the abutment and the bridge. The reactional force creates a concentrated load on the bridge.
- **Settlement:** The soil on which the structure is founded is heterogeneous. It could be likely that one of the supports settles more than the others, therefore a settlement of 5 mm of one of the supports is taken into account.
- **Prestress:**
    - The initial prestress reduces over time due to creep, shrinkage and relaxation. On average the reduction for construction A and B, is 0,912 and 0,915.
    - Because the centre of gravity of the applied tendons does not coincide with the centre of gravity of the structure, an additional bending moment is present.

2.1 Bridge A

Table B-1: Load cases bridge A (SLS)

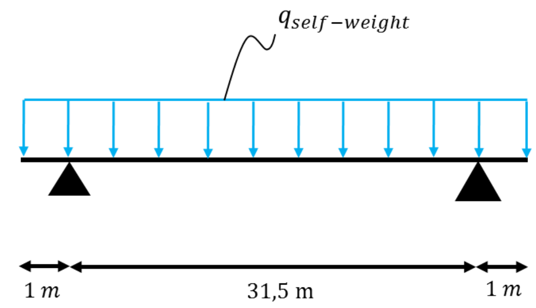
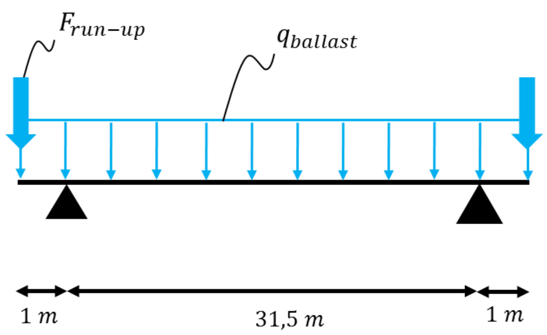
LC	Load parameters	Load	Load scheme
1 - Self weight	$A_b = 5{,}5\ m^2$ $\gamma_{concrete} = 25\ kN/m^3$	$q_{s.w.} = 137{,}5\ kN/m$	
2- Ballast	$h_{ballast} = 0{,}65\ m$ $b_{ballast} = 4{,}0\ m$ $\gamma_{ballast} = 18\ kN/m^3$ $q_{s.w.\ run-up} = 9{,}0\ kN/m$ $l_{run-up} = 4{,}0\ m$ $b_{run-up} = 1{,}6m$	$q_{ballast} = 11{,}7\ kN/m^2$ $F_{run-up\ plate} = 111\ kN$	

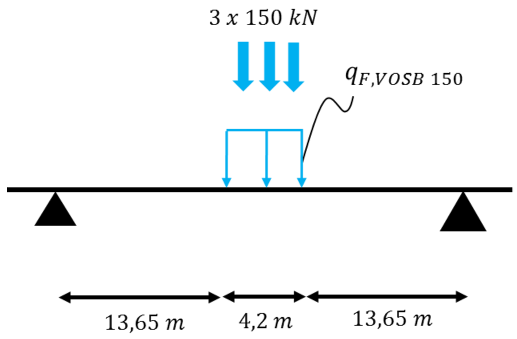
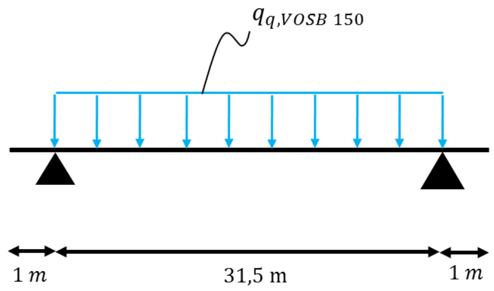
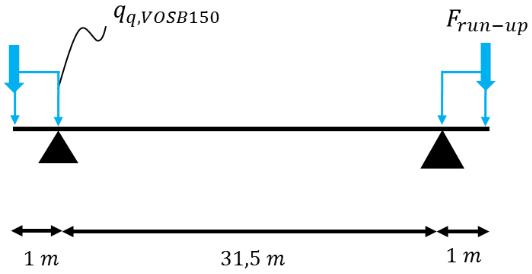
3 – Con. Mobile Load (VOSB 150)	$l = 4{,}2\ m$ $b = 2{,}85\ m$ $F_{VOSB\ 150} = 150\ kN$ $\Phi_{dynamic} = 1{,}195$	$q_{F,VOSB\ 150} = \frac{3 * 150 * 1{,}195}{2{,}85 * 4{,}2} = 44{,}9\ kN/m^2$	
4 – Cantilever fully loaded (VOSB 150)	$q_{VOSB\ 150} = 80\ kN/m$ $\Phi_{dynamic} = 1{,}195$	$F_{run-up\ plate} = 191\ kN$ $q_{q,VOSB} = \frac{1{,}195 * 80}{2{,}85} = 33{,}5\ kN/m^2$	
5 – Contin. Mobile load (VOSB 150)		$q_{q,VOSB} = 33{,}5\ kN/m^2$	

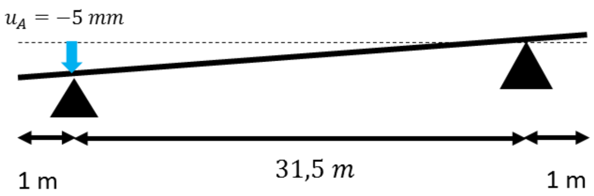
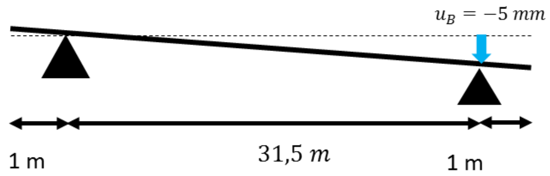
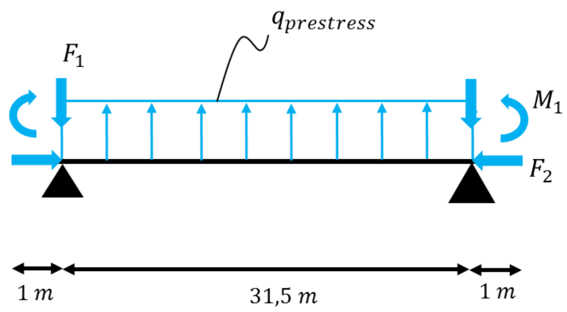
6 – Settlement support min.		$u_a = -5\ mm$	
7 – Prestress at t=0		$u_b = -5\ mm$	
8 – Prestress at t=0		$q_{P0} = 101\ kN/m$ $F_1 = 1115\ kN$ $F_2 = 13200\ kN$ $M_1 = 294\ kNm$	
9 – Prestress at $t = \infty$	$\frac{P_\infty}{P_0} = 0,912$	$q_{P0} = 92\ kN/m$ $F_1 = 1017\ kN$ $F_2 = 12038\ kN$ $M_1 = 268\ kNm$	

2.2 Bridge B

Table B-2: Load cases bridge B (SLS)

LC	Load parameters	Load	Load scheme
1 - Self weight	$A_b = 8{,}63\ m^2$ $\gamma_{concrete} = 25\ kN/m^3$	$q_{s.w.} = 216\ kN/m$	
2- Ballast	$h_{ballast} = 0{,}55\ m$ $b_{ballast} = 4{,}0\ m$ $\gamma_{ballast} = 18\ kN/m^3$ $q_{s.w.\ run-up} = 9{,}0\ kN/m$ $l_{run-up} = 4{,}0\ m$ $b_{run-up} = 1{,}6m$	$q_{ballast} = 9{,}9\ kN/m^2$ $F_{run-up\ plate} = 99\ kN$	

3 – Mobile Max (VOSB 150)	$l = 4{,}2\ m$ $b = 3{,}0\ m$ $F_{VOSB\ 150} = 150\ kN$ $q_{VOSB\ 150} = 80\ kN$ $\Phi_{dynamic} = 1{,}155$	$q_{F,VOSB\ 150} = \frac{3 * 150 * 1{,}155}{3{,}0 * 4{,}2} = 41{,}3\ kN/m^2$	
		$q_{q,VOSB\ 150} = \frac{80 * 1{,}155}{3{,}0} = 30{,}8\ kN/m^2$	
4 – Cantilever fully loaded (VOSB 150)	$q_{VOSB\ 150} = 80\ kN/m$ $\Phi_{dynamic} = 1{,}155$	$F_{run-up\ plate} = 185\ kN$ $q_{q,VOSB} = \frac{1{,}155 * 80}{3{,}0} = 30{,}8\ kN/m^2$	

5 – Settlement support max.		$u_a = -5\ mm$	
6 – Settlement support min.		$u_b = -5\ mm$	
7 – Prestress at t=0		$q_{P0} = 145{,}2\ kN/m$ $F_1 = 2290\ kN$ $F_2 = 22826\ kN$ $M_1 = 5045\ kNm$	
8 – Prestress at $t = \infty$	$\frac{P_\infty}{P_0} = 0{,}915$	$q_{P0} = 132{,}9\ kN/m$ $F_1 = 1048\ kN$ $F_2 = 20886\ kN$ $M_1 = 4616\ kNm$	

3 Principal stresses

The critical location for shear tension failure is at the upper side of the connection between the floor and the main girder (Figure B-3). The critical principal stress consists of:

- σ_{xx}: The horizontal normal stress, due to normal force and bending from external forces, prestressing and restrained deformations.
- σ_{yy}: The vertical normal stress, is caused by a suspension force and clamping moment from the floor.
- τ_{xy}: The shear stress, caused by torsion of the mobile load and shear force of external forces, prestressing and restrained deformations.

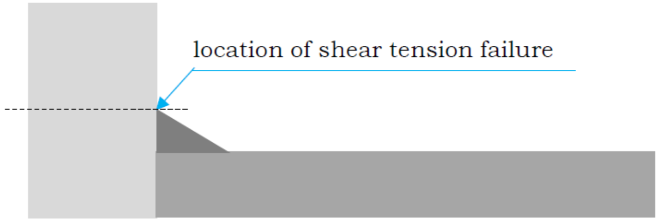


Figure B-3: Location of shear tension failure

With Mohr's circle the normal and shear stresses are combined to a principal stress, equation [B.2] and [B.3] define an expression for the two characteristic principal stresses.

$$\rho_1 = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + {\tau_{xy}}^2} \qquad [\ B.2\ ]$$

$$\rho_2 = \frac{\sigma_x + \sigma_y}{2} - \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + {\tau_{xy}}^2} \qquad [\ B.3\ ]$$

3.1 Results

Earlier it has been established that the connection between the floor and girder leads to critical stress combinations. Respectively for bridge A and B, the most critical location in longitudinal direction, is at 0,8d from the support and right above the support. The normal and shear stresses for these two locations are shown in Table B-3 and Table B-4.

Table B-3: Bridge A: Normal and shear stresses at 0,8d

		Torsion		Shear force	Normal force	Bending moment floor		Shear force floor	
LC	type	Mxy	τxy	τxy	σxx	Myy	σyy	Qyy	σyy
1	self-weight	5,8	0,06	0,43	-0,31	13,2	0,10	-36,9	-0,04
2	ballast	-17,2	-0,17	0,19	-0,11	10,9	0,08	-31,6	-0,04
3	Conc. Mobile Load	8,6	0,08	0,11	-0,08	10	0,07	-3,7	0,00
4	Cant. Mobile Load	-53,7	-0,52	-0,02	0,09	21,3	0,16	-32,3	-0,04
5	Contin. Mobile Load	-2,3	-0,02	0,32	-0,23	23,7	0,18	-49,8	-0,06
6	Support Settlement max.	26,1	0,25	0,01	0,01	20,9	0,15	10,5	0,01
7	Support Settlement min.	-26,1	-0,25	-0,01	-0,01	-20,9	-0,15	-10,5	-0,01
8	Prestress t=0	-120	-1,17	-0,91	-4,82	-20,7	-0,15	-16,5	-0,02
9	Prestress t = ∞	-109	-1,06	-0,82	-4,39	-18,8	-0,14	-15	-0,02
LC 1+ LC 2+ LC 3+ LC 5+ LC 6 + LC 8		-88	-0,86	0,24	-5,11	60	0,44	-127	-0,14
LC 1+ LC 2+ LC 3+LC 5+LC 7 + LC 8		-140,2	-1,36	0,22	-5,13	18,1	0,13	-147,5	-0,16
LC 1+ LC 2+ LC 4+ LC 6 +LC 8		-148	-1,44	-0,21	-4,71	47,5	0,35	-105,3	-0,12
LC 1+LC 2+LC 4+LC 7 + LC 8		-200,2	-1,95	-0,23	-4,73	5,7	0,04	-126,3	-0,14
LC 1+ LC 7		-114,2	-1,11	-0,48	-5,13	-7,5	-0,06	-53,4	-0,06

Table B-4: Bridge B: Normal and shear stress above the support

		Torsion		Shear force	Normal force	Bending moment floor		Shear force floor	
LC	type	Mxy	τxy	τxy	σxx	Myy	σyy	Qyy	σyy
1	self-weight	0,8	0,00	0,62		1,6	0,01	-112	-0,09
2	ballast	-12,3	-0,07	0,14		-15,4	-0,06	-40,7	-0,03
3	Mobile Max (VOSB 150)	13,6	0,07	0,31		-27,3	-0,11	-68,5	-0,06
4	Mobile Min (VOSB 150)	-76,9	-0,42	-0,01		-8,5	-0,04	-34,1	-0,03
5	Support Settlement max.	29,5	0,16	0,00		-33,3	-0,14	12,7	0,01
6	Support Settlement min.	-29,5	-0,16	0,01		33,3	0,14	-12,7	-0,01
7	Prestress t=0	-112	-0,62	-1,10	-5,29	115,6	0,48	-114	-0,10
8	Prestress t = ∞	-102	-0,56	-1,00	-4,84	105,7	0,44	-104	-0,09
LC 1+ LC 2+ LC 3+ LC 5+ LC 6 + LC 8		-70	-0,39	0,07	-4,84	31	0,13	-313	-0,26
LC 1+ LC 2+ LC 3+LC 5+LC 7 + LC 8		-129	-0,71	0,08	-4,84	98	0,41	-338	-0,28
LC 1+ LC 2+ LC 4+ LC 6 +LC 8		-161	-0,88	-0,25	-4,84	50	0,21	-278	-0,23
LC 1+LC 2+LC 4+LC 7 + LC 8		-220	-1,21	-0,24	-4,84	117	0,49	-304	-0,25
LC 1+ LC 7		-111	-0,61	-0,48	-5,29	117	0,49	-226	-0,19

Remarkable are the large shear stresses due to prestress. The large contribution of the shear force can be explained. At the support the angle between the prestressing tendons and the horizontal is at a maximum, making this the location with the largest vertical component. The vertical component is obviously the acting shear force.

The large contribution of torsion to the shear stress cannot directly be explained. According to the design report, the introduction of the prestress in ANSYS leads to large tensile stresses in the first part of the floor. This causes strains of the floor and a rotation in the girder. Eventually this rotation generates torsion in the girder which is of a much larger magnitude than in reality. According to the design report the values for torsion form a safe upper limit for the design of a through bridge.

Table B-5: Bridge A: Principal stresses at 0,8d

		Total			Principal stresses	
LC	type	σyy	σxx	τxy	ρ1	ρ2
1	self-weight					
2	ballast					
3	Conc. Mobile Load					
4	Cant. Mobile Load					
5	Contin. Mobile Load					
6	Support Settlement max.					
7	Support Settlement min.					
8	Prestress t=0					
9	Prestress t = ∞					
LC 1+ LC 2+ LC 3+ LC 5+ LC 6 + LC 8		0,32	-5,11	1,10	0,53	-5,32
LC 1+ LC 2+ LC 3+LC 5+LC 7 + LC 8		0,16	-5,13	1,58	0,60	-5,57
LC 1+ LC 2+ LC 4+ LC 6 +LC 8		0,25	-4,71	1,65	0,75	-5,21
LC 1+LC 2+LC 4+LC 7 + LC 8		0,10	-4,73	2,18	**0,93**	-5,57
LC 1+ LC 7		0,06	-5,13	1,59	0,51	**-5,58**

Table B-6: Bridge B: Principal stress above the support

		Total			Principal stresses	
LC	type	σyy	σxx	τxy	ρ1	ρ2
1	self-weight					
2	ballast					
3	Mobile Max (VOSB 150)					
4	Mobile Min (VOSB 150)					
5	Support Settlement max.					
6	Support Settlement min.					
7	Prestress t=0					
8	Prestress t = ∞					
LC 1+ LC 2+ LC 3+ LC 5+ LC 6 + LC 8		0,27	-4,84	0,46	0,31	-4,88
LC 1+ LC 2+ LC 3+LC 5+LC 7 + LC 8		0,48	-4,84	0,79	0,59	-4,95
LC 1+ LC 2+ LC 4+ LC 6 +LC 8		0,31	-4,84	1,13	0,55	-5,08
LC 1+LC 2+LC 4+LC 7 + LC 8		0,51	-4,84	1,45	**0,88**	-5,21
LC 1+ LC 7		0,47	-5,29	1,09	0,67	**-5,49**

The vertical normal stress, consisting of the suspension force and the clamping moment, is initially assumed constant with the height of the beam. Yet in reality these stresses vary with height, with a maximum at the bottom and zero at the top. Because a section just above the throat (Figure B-3) is considered, the vertical normal stresses are reduced by using Timoshenko's theory of elasticity.

3.2 Unity check

In the design reports, three locations are checked for shear tension failure. Only the normal and shear stresses leading to the most critical stress combination have been presented in the previous chapter. However Table B-7 and Table B-8 show the unity check for all three locations of both bridges.

Table B-7: Unity check's bridge A

Location	ρ_1	$0.5 * f_b$	U.C.
0,8d	0,93 N/mm^2	1,0 N/mm^2	0,93/1,0=**0,93**
2,0d	0,62 N/mm^2	1,0 N/mm^2	0,62/1,0=0,62
0,5L	0,48 N/mm^2	1,0 N/mm^2	0,48/1,0=0,48

Table B-8: Unity check's bridge B

Location	ρ_1	$0.5 * f_b$	U.C.
Support	0,88 N/mm^2	1,0 N/mm^2	0,88/1,0=**0,88**
0,25L	0,56 N/mm^2	1,0 N/mm^2	0,56/1,0=0,56
0,5L	0,52 N/mm^2	1,0 N/mm^2	0,52/1,0=0,52

Appendix C – Shear tension failure in the Eurocode (ULS)

1 Introduction

Bridge A and B are designed using the load models in VOSB 1963. Nowadays the Eurocode is applicable, which uses different load models for railway traffic. The objective of this appendix is to check whether or not shear tension failure is an issue, if the bridges are subjected to load models from the Eurocode. In order to get a proper understanding of the transfer of forces, this check is performed with a hand calculation.

1.1 ULS

The risk of shear tension failure is an ultimate limit state check in the Eurocode. Eurocode 0 (14) considers two load combinations, one where the self-weight of the structure is governing (6.10a) and another where the variable action on the structure is governing (6.10b).

$$\gamma_G * G_k + \gamma_Q * \Psi_0 * Q_k + \gamma_P * P_k \qquad [\ 6.10a\ ]$$

$$\xi\gamma_G * G_k + \gamma_Q * Q_k + \gamma_P * P_k \qquad [\ 6.10b\ ]$$

Table C-1: Load factors for load combination 6.10a and 6.10b

Load case	Load factor 6.10a	Load factor 6.10b
1- Self-weight	1,40	1,25
2- Ballast	1,40	1,25
3- Conc. Mobile load (LM71)	1,20	1,50
4- Cantilevers fully loaded (SW/2)	1,00	1,25
5a- Cont. Mobile load (LM71)	1,20	1,50
5b- Cont. Mobile load (SW/2)	1,00	1,25
6a- Settlement support max	1,20	1,20
6b- Settlement support min	1,20	1,20
7a- Prestress at $t = 0$	1,00	1,00
7b– Prestress at $t = \infty$	1,00	1,00

All load factors are derived from Table NB.14 in the Dutch National Annex to Eurocode 0 (15). The factor Ψ_0 describes the chance at simultaneous variable actions which is equal to 0,80.

1.2 Typical cross-section

Respectively the typical cross-section of bridge A and bridge B are shown in Figure C-1 and Figure C-2. The cross-sections are constant along the length of the bridge.

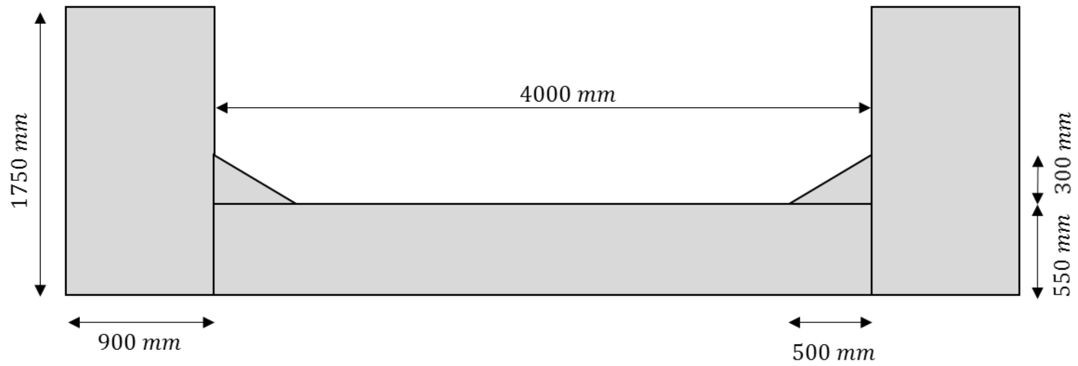


Figure C-1: Typical cross-section bridge A

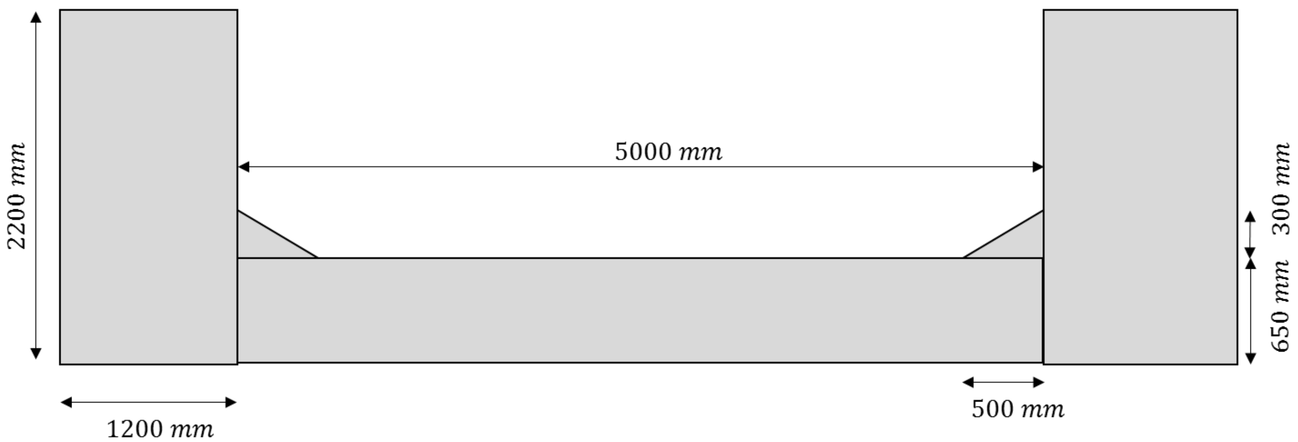


Figure C-2: Typical cross-section bridge B

2 Load cases

The railway traffic can be approximated by a couple of load models according to the Eurocode. For a simply supported bridge, load model 71 and SW/2 are applicable.

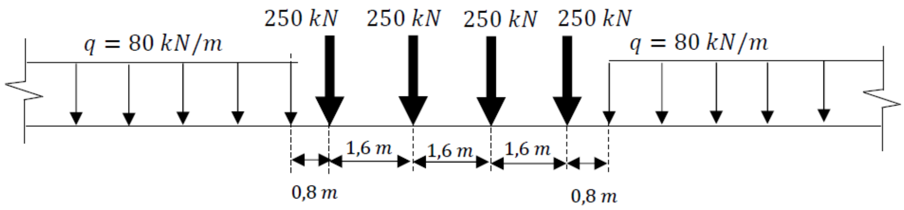


Figure C-3: Load model 71

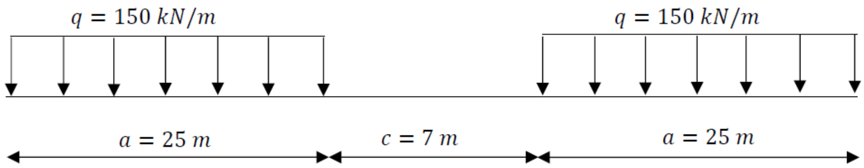


Figure C-4: Load model SW/2

According to the design reports the critical location for shear tension failure is at 0,8d from the support for bridge A and right above the support for bridge B. For the Eurocode calculation the locations are assumed to be the same.

To find the most unfavourable position of load model 71, influence lines are generated with SCIA. The influence lines for shear, torsion and bending are established for the mentioned critical sections of bridge A and B. Positioning the group of concentrated loads at the start of the bridge results in maximum torsion in the critical section. Positioning the distributed load between the group of concentrated loads and the end support, leads to maximum bending and shear force in the critical section and hardly influences the maximum torsion.

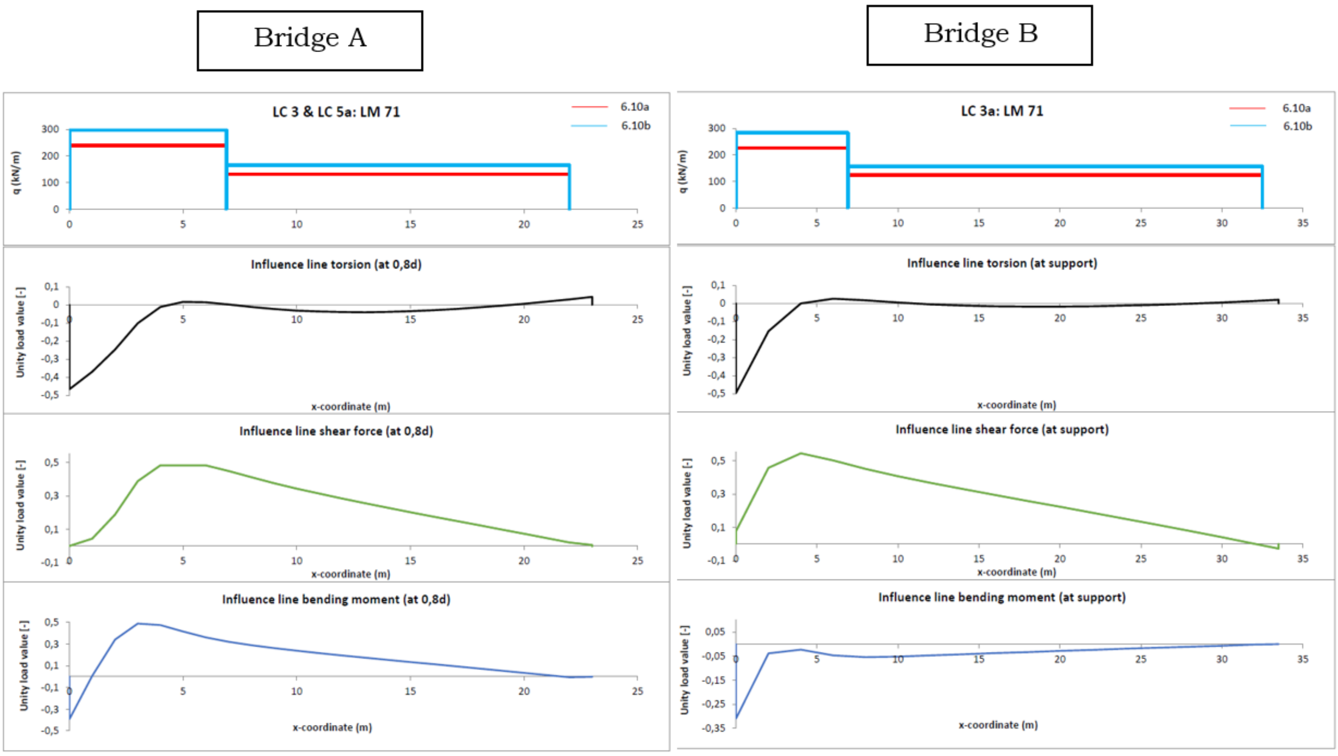


Figure C-5: : Influence lines for torsion, shear and bending at 0,8d in bridge A (left) and above the support in bridge B (right)

A number of remarks are made to give one a better understanding of the applied load case (presented by Table C-2 and Table C-3):

- **LM 71:**
  - The group of concentrated loads is distributed over 6,9 meters (Appendix A).
  - To take into account the passing of heavier trains, LM71 is multiplied with a factor $\alpha = 1{,}21$.

- **SW/2:** Heavy railway traffic is simulated with load model SW/2. Because it has a length of at least 25 meters, the model does not fit between the supports of bridge A (see LC 5b in Table C-2).

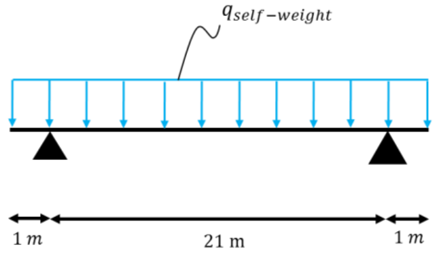
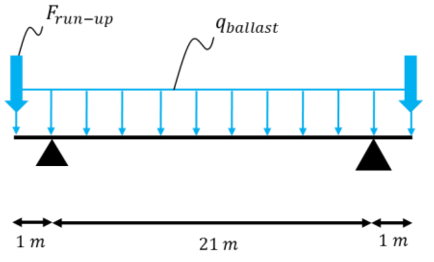
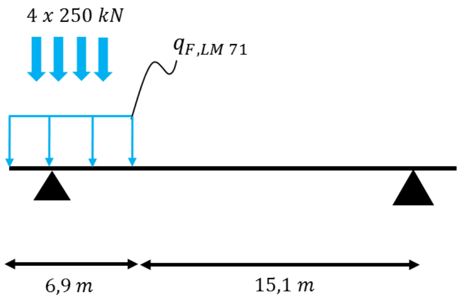
- **Dynamic factor**: A passing train causes a fluctuation in stresses and gives a certain acceleration to the bridge. This causes an additional load that is taken into account with the dynamic factor. Section 6.4 in Eurocode 2 – part 2 (20) is consulted for the dynamic factor.

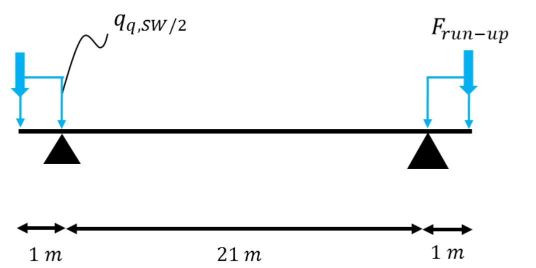
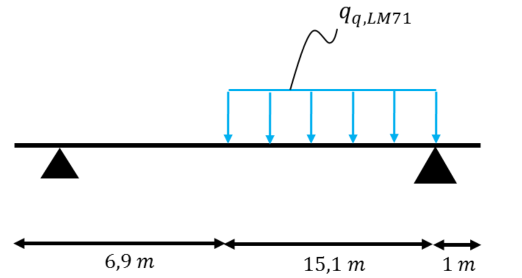
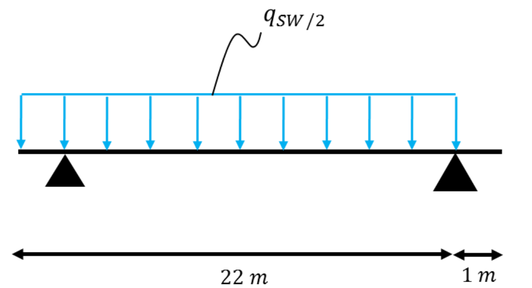
$$Brige\ A: \qquad \phi_2 = \frac{1{,}44}{\sqrt{21 - 0{,}2}} + 0{,}82 = 1{,}136$$

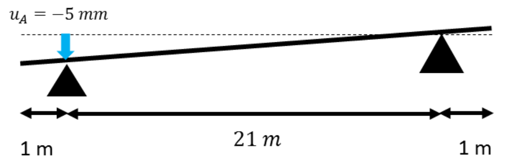
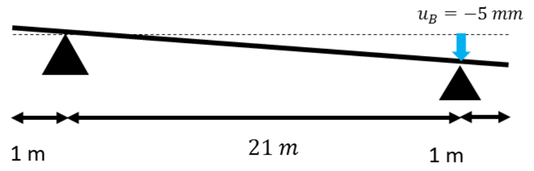
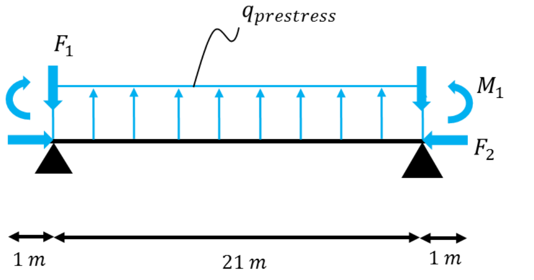
$$Brige\ B: \qquad \phi_2 = \frac{1{,}44}{\sqrt{31{,}5 - 0{,}2}} + 0{,}82 = 1{,}077$$

2.1 Bridge A

Table C-2: Load cases bridge A (ULS)

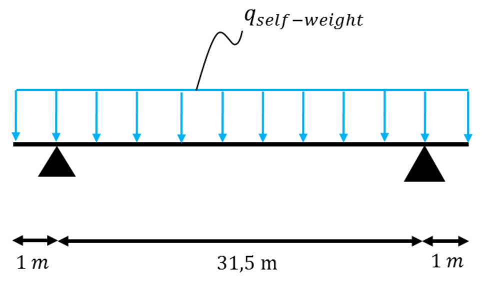
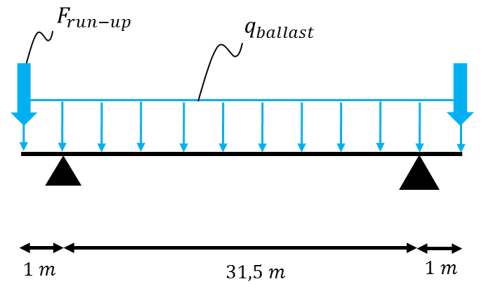
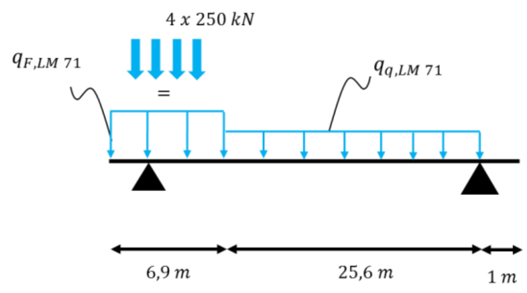
LC	Load parameters	Load Factors	6.10a	6.10b	Load scheme
1 - Self weight	$A_b = 5{,}5\ m^2$ $\gamma_{concrete} = 25\ kN/m^3$	<u>6.10a</u> $\gamma_G = 1{,}4$ <u>6.10b</u> $\gamma_G = 1{,}25$	$q_{s.w.} = 193\ kN/m$	$q_{s.w.} = 172\ kN/m$	
2- Ballast	$h_{ballast} = 0{,}65\ m$ $b_{ballast} = 4{,}0\ m$ $\gamma_{ballast} = 18\ kN/m^3$ $q_{s.w.\ run-up} = 9{,}0\ kN/m$ $l_{run-up} = 4{,}0\ m$ $b_{run-up} = 1{,}6m$	<u>6.10a</u> $\gamma_G = 1{,}4$ <u>6.10b</u> $\gamma_G = 1{,}25$	$q = 16{,}4\ kN/m^2$ $F_{run-up} = 155\ kN$	$q = 14{,}6\ kN/m^2$ $F_{run-up} = 139\ kN$	
3– Con. Mobile Load (LM71)	$l = 6{,}9\ m$ $\Phi = 1{,}136$ $F_{LM71} = 250\ kN$	<u>6.10a</u> $\gamma_Q = 1{,}2$ $\propto = 1{,}21$ <u>6.10b</u> $\gamma_Q = 1{,}5$ $\propto = 1{,}21$	$q_{F,LM71} = 239{,}1\ kN/m$	$q_{F,LM71} = 298{,}8\ kN/m$	

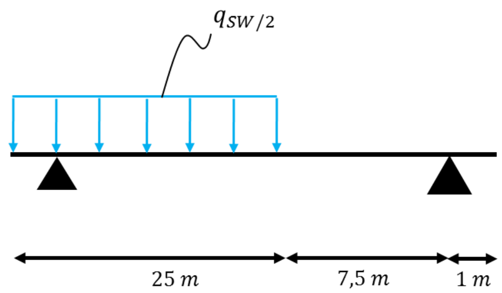
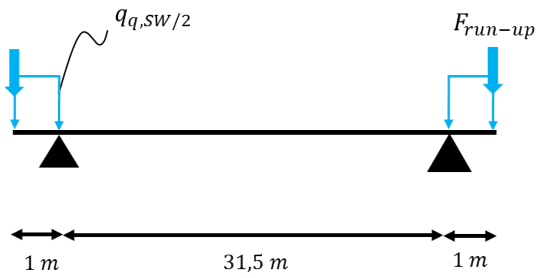
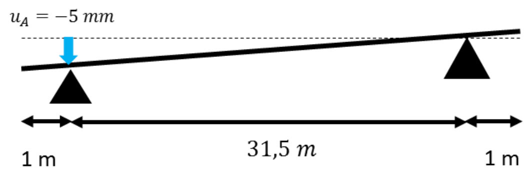
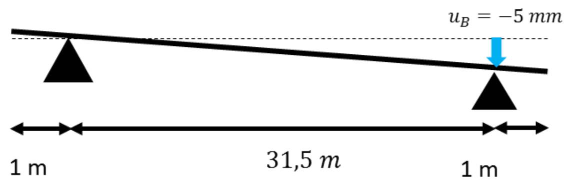
4– Cantilever fully loaded (SW/2)	$q_{SW/2} = 150\ kN/m$ $\Phi = 1{,}136$	6.10a $\gamma_Q = 1{,}0$ 6.10b $\gamma_Q = 1{,}25$	$q = 170{,}4\ kN/m$ $F_{run-up} = 341\ kN$	$q = 213\ kN/m$ $F_{run-up} = 426\ kN$	
5a – Contin. Mobile load (LM71)	$q_{LM71} = 80\ kN/m$ $\Phi = 1{,}136$	6.10a $\gamma_Q = 1{,}2$ $\propto = 1{,}21$ 6.10b $\gamma_Q = 1{,}5$ $\propto = 1{,}21$	$q_{q,LM71} = 132\ kN/m$	$q_{q,LM71} = 165\ kN/m$	
5b – Contin. Mobile load (SW/2)	$q_{SW/2} = 150\ kN/m$ $\Phi = 1{,}136$	6.10a $\gamma_Q = 1{,}0$ 6.10b $\gamma_Q = 1{,}25$	$q = 170{,}4\ kN/m$	$q = 213\ kN/m$	

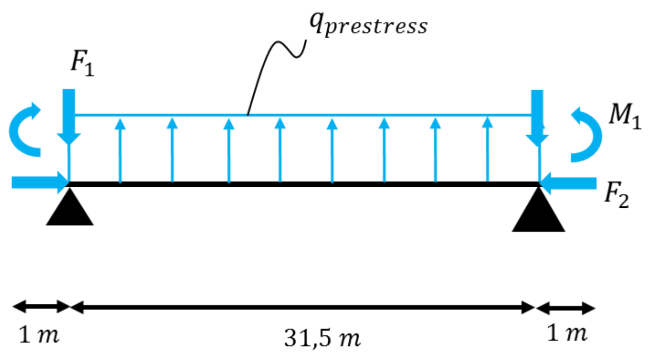
6 – Settlement support max.		<u>6.10a</u> $\gamma_Q = 1{,}20$ <u>6.10b</u> $\gamma_Q = 1{,}20$	$u_a = -5\ mm$	
7 – Settlement support min.		<u>6.10a</u> $\gamma_Q = 1{,}20$ <u>6.10b</u> $\gamma_Q = 1{,}20$	$u_b = -5\ mm$	
8 – Prestress at t=0		<u>6.10a</u> $\gamma_P = 1{,}00$ <u>6.10b</u> $\gamma_P = 1{,}00$	$q_{P0} = 101\ kN/m$ $F_1 = 1115\ kN$ $F_2 = 13200\ kN$ $M_1 = 294\ kNm$	
9 – Prestress at $t = \infty$	$\frac{P_\infty}{P_0} = 0{,}912$	<u>6.10a</u> $\gamma_P = 1{,}00$ <u>6.10b</u> $\gamma_P = 1{,}00$	$q_{P0} = 92\ kN/m$ $F_1 = 1017\ kN$ $F_2 = 12038\ kN$ $M_1 = 268\ kNm$	

2.2 Bridge B

Table C-3: Load cases bridge B (ULS)

LC	Load parameters	Load factors	Load 6.10a	Load 6.10b	Load scheme
1 - Self weight	$A_b = 8{,}63\ m^2$ $\gamma_{concrete} = 25\ kN/m^3$	6.10a $\gamma_G = 1{,}4$ 6.10b $\gamma_G = 1{,}25$	$q_{s.w.} = 302\ kN/m$	$q_{s.w.} = 269{,}6\ kN/m$	
2- Ballast	$h_{ballast} = 0{,}55\ m$ $b_{ballast} = 5{,}0\ m$ $\gamma_{ballast} = 18\ kN/m^3$	6.10a $\gamma_G = 1{,}4$ 6.10b $\gamma_G = 1{,}25$	$q_{ballast} = 13{,}9\ kN/m^2$ $F_{run-up\ plate} = 139\ kN$	$q_{ballast} = 12{,}4\ kN/m^2$ $F_{run-up\ plate} = 124\ kN$	
3a– Mobile Max (LM 71)	$l = 6{,}9\ m$ $\Phi = 1{,}077$ $b = 3{,}0\ m$ $F_{LM\ 71} = 250\ kN$ $q_{\ LM\ 71} = 80\ kN/m$	6.10a $\gamma_Q = 1{,}2$ $\propto = 1{,}21$ 6.10b $\gamma_Q = 1{,}5$ $\propto = 1{,}21$	$q_{F,LM\ 71} = 226{,}6\ kN/m$ $q_{q,LM\ 71} = 125{,}1\ kN/m$	$q_{F,LM\ 71} = 283{,}3\ kN/m$ $q_{q,LM\ 71} = 156{,}4\ kN/m$	

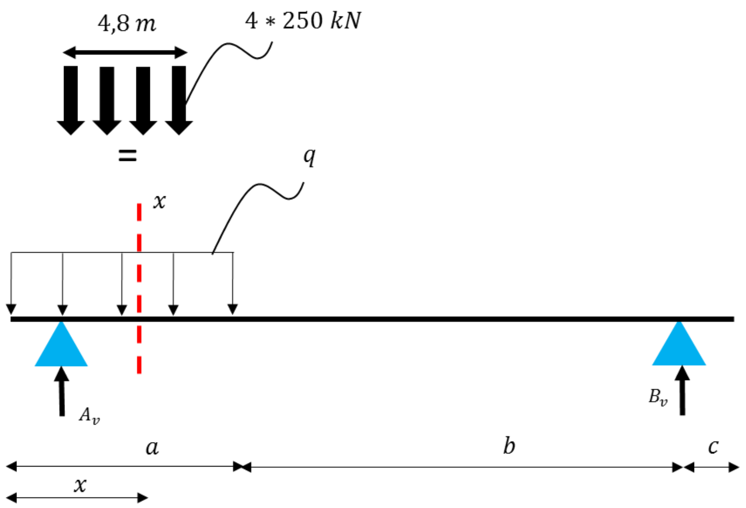
3b – Mobile Max (SW/2)	$\Phi = 1{,}077$ $b = 3{,}0\ m$ $q_{sw/2} = 150\ kN/m$	<u>6.10a</u> $\gamma_Q = 1{,}0$ <u>6.10b</u> $\gamma_Q = 1{,}25$	$q_{sw/2} = 161{,}6\ kN/m$	$q_{sw/2} = 202\ kN/m$	
4 – Cantilever fully loaded (SW/2)	$\Phi = 1{,}077$ $b = 3{,}0\ m$ $q_{sw/2} = 150\ kN/m$	<u>6.10a</u> $\gamma_Q = 1{,}0$ <u>6.10b</u> $\gamma_Q = 1{,}25$	$F_{run-up} = 323\ kN$ $q_{sw/2} = 161{,}6\ kN/m$	$F_{run-up} = 404\ kN$ $q_{sw/2} = 202\ kN/m$	
5 – Settlement support max.		<u>6.10a</u> $\gamma_Q = 1{,}20$ <u>6.10b</u> $\gamma_Q = 1{,}20$	$u_a = -5\ mm$		
6 – Settlement support min.		<u>6.10a</u> $\gamma_Q = 1{,}20$ <u>6.10b</u> $\gamma_Q = 1{,}20$	$u_b = -5\ mm$		

7 – Prestress at t=0		<u>6.10a</u> $\gamma_P = 1{,}00$ <u>6.10b</u> $\gamma_P = 1{,}00$	$q_{P0} = 145{,}2\ kN/m$ $F_1 = 2290\ kN$ $F_2 = 22826\ kN$ $M_1 = 5045\ kNm$	
8 – Prestress at $t = \infty$	$\frac{P_\infty}{P_0} = 0{,}915$	<u>6.10a</u> $\gamma_P = 1{,}00$	$q_{P0} = 132{,}9\ kN/m$ $F_1 = 1048\ kN$ $F_2 = 20886\ kN$ $M_1 = 4616\ kNm$	

3 Normal and shear stresses

3.1 Horizontal normal stress

The horizontal normal stress is due to prestressing, external forces and restrained deformations. Through an example the procedure to calculate the horizontal normal stresses is explained. Let's consider a section at 0,8d from the supports in bridge A. For this particular example load case 3 is considered in load combination 6.10a.



Load parameters
$x = 2{,}36\ m$
$a = 6{,}9\ m$
$b = 15{,}1\ m$
$c = 1{,}0\ m$
$q = 239{,}1\ kN/m$

Figure C-6: LC 3 acting on bridge A (combination 6.10a)

First one needs to determine reaction force A_v (keep in mind that loads are distributed over the two girders).

$$A_v = 0{,}5 * \frac{q * a * \left(b + \frac{a}{2}\right)}{(a + b - c)} = 728{,}7\ kN$$

The bending moment at $x = 2{,}36\ m$:

$$M_x = A_v * (x - c) - 0{,}5q * x^2 * 0{,}5 = 658\ kNm$$

In the design report the centre of gravity is calculated for the entire structure. It is positioned at 629 mm (z) above the bottom fibre. When loading the cross-section with a downward acting bending moment the bottom fibres are subjected to tension and the top fibres to compression. The critical section lies 221 mm above the neutral axis and is subjected to a compressive stress.

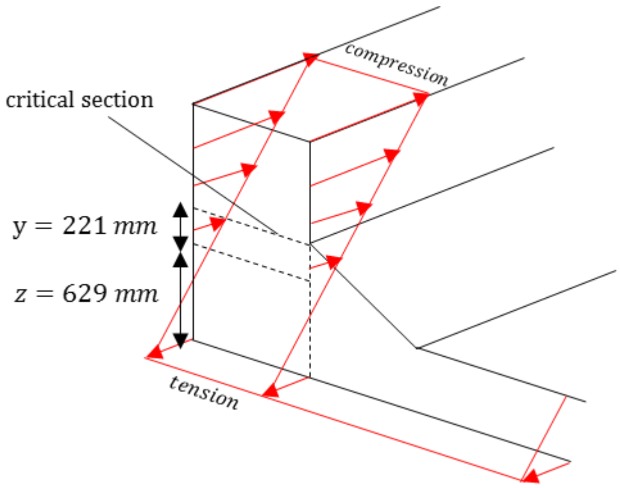


Figure C-7: Horizontal normal stress due to an acting bending moment

It is important to understand that the entire cross-section of the bridge is loaded with the bending moment. Which means that the bending moment resistance of the floor should also be taken into account. Therefore the moment of inertia for half the cross-section is computed:

$$I_{yy,girder} = \frac{1}{12} * b_{girder} * h_{girder}{}^3 + b_{girder} * h_{girder} * \left(\frac{h_{girder}}{2} - z\right)^2 = 0{,}497\ m^4$$

$$0{,}5 * I_{yy,floor} = 0{,}5 * \left(\frac{1}{12} * b_{floor} * h_{floor}{}^3 + b_{floor} * h_{floor} * \left(z - \frac{h_{floor}}{2}\right)^2\right) = 0{,}166\ m^4$$

Moment of inertia: $0{,}5 * I_{yy,bridge} = I_{yy,girder} + 0{,}5 * I_{yy,floor} = 0{,}663\ m^4$

Equation [C.1] is used to calculate the horizontal normal stresses due to bending.

$$\sigma_{xx} = \frac{M * z}{I_{yy}} \qquad [\ C.1\ ]$$

$$\sigma_{xx} = -\frac{658\ kNm * 0{,}221\ m}{0{,}663\ m^4} = -0{,}22\ N/mm^2$$

Figure C-8 shows a top view of the bridge. The prestressing forces are introduced in the girders and spread under an angle of 45°. The truss model shows that these large prestressing forces lead to tensile splitting forces, but because transverse prestress and tensile splitting reinforcement is applied this will not be a problem. When calculating the horizontal normal stress it is important to take into account over which width the prestress force is distributed.

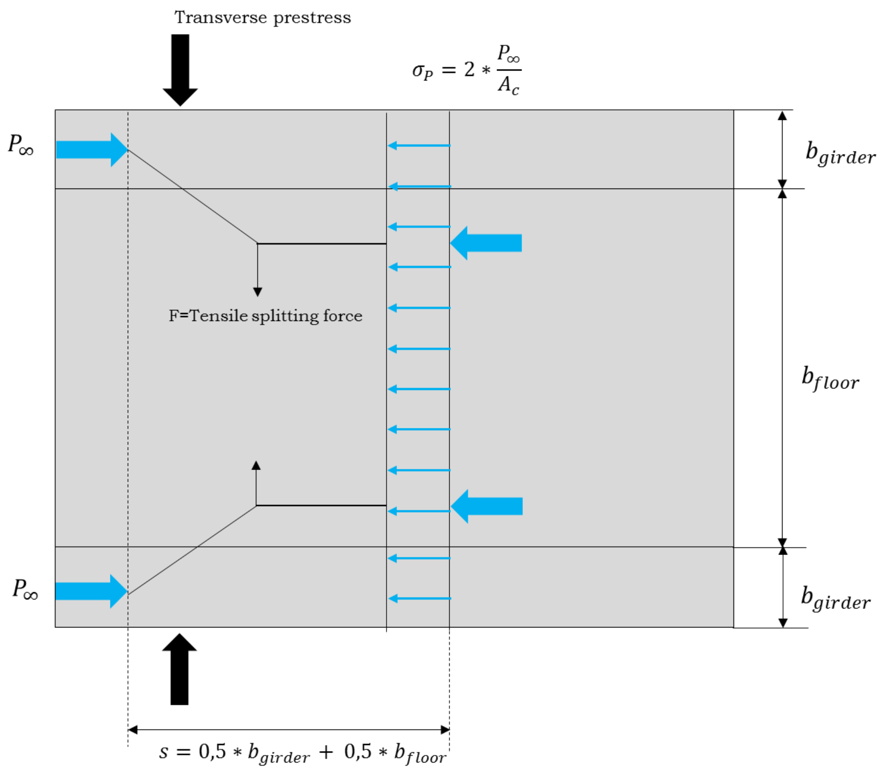


Figure C-8: The distribution of prestress over the bridge (top view)

At a section 0,8d from the support the prestress force is spread over a width which is equal to 0,8d (spread 1:1). From the centre of the girder this results in a total width of 1,81 m over which the prestress force is present.

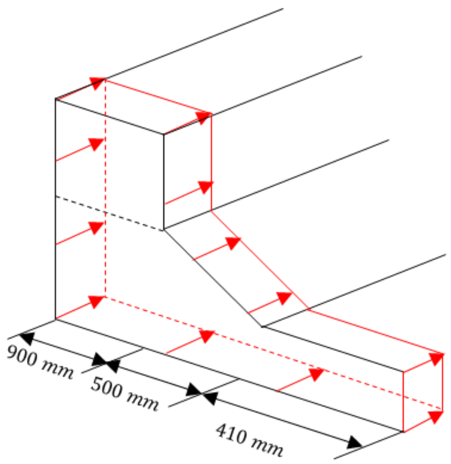


Figure C-9: Compressive normal stress due to prestress

With the corresponding area and equation [C.2] the horizontal normal stress is computed:

$$\sigma_{xx} = \frac{P}{A} \qquad [\ C.2\ ]$$

Area:

$$A_{prestress} = b_{girder} * h_{girder} + 0{,}5 * b_{voute} * h_{voute} + x * h_{floor} =$$

$$A_{prestress} = 900 * 1750 + 0{,}5 * 500 * 300 + 910 * 550 = 2{,}15 * 10^6\ mm^2$$

$$\sigma_{xx} = \frac{-12038\ kN}{2{,}15\ m^2} = -5{,}60\ N/mm^2$$

The horizontal normal stresses for bridge A and B and both load combinations are presented by Table C-4 and Table C-5. Remarkable are the bending stresses in bridge B, which are close to zero for a lot of the load case. This has to do with the bridge being simply supported and the bending moment being determined above the supports.

Table C-4: Horizontal normal stress bridge A (combination 6.10a & 6.10b) – 0,8d

		Prestress - σxx [N/mm2]		Bending moment - σxx [N/mm2]		Total - σxx [N/mm2]	
LC	**type**	6.10a	6.10b	6.10a	6.10b	6.10a	6.10b
1	**self-weight**			-0,41	-0,37		
2	**ballast**			-0,11	-0,10		
3	**Conc. Mobile Load**			-0,22	-0,27		
4	**Cant. Mobile Load**			0,07	0,07		
5a	**Contin. Mobile Load**			-0,16	-0,20		
5b	**Contin. Mobile Load (SW/2)**			-0,37	-0,46		
6	**Support settelement max**			0,00	0,00		
7	**Support settelement min**			0,00	0,00		
8	**Prestress t=0**	-6,14	-6,14	0,35	0,35		
9	**Prestress t = ∞**	-5,60	-5,60	0,32	0,32		
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		-5,60	-5,60	-0,59	-0,63	-6,19	-6,22
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		-5,60	-5,60	-0,59	-0,63	-6,19	-6,22
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		-5,60	-5,60	-0,14	-0,08	-5,73	-5,67
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		-5,60	-5,60	-0,14	-0,08	-5,73	-5,67
LC 1 + LC 8		-6,14	-6,14	-0,06	-0,02	-6,20	-6,15
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		-5,60	-5,60	-0,57	-0,61	-6,17	-6,21
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		-5,60	-5,60	-0,57	-0,61	-6,17	-6,21

Table C-5: Horizontal normal stress bridge B (combination 6.10a & 6.10b) - support

		Prestress - σxx [N/mm2]		Bending moment - σxx [N/mm2]		Total - σxx [N/mm2]	
LC	**type**	6.10a	6.10b	6.10a	6.10b	6.10a	6.10b
1	**self-weight**			0,01	0,01		
2	**ballast**			0,01	0,01		
3a	**Mobile Max. (LM71)**			0,00	0,01		
3b	**Mobile Max. (SW/2)**			0,00	0,00		
4	**Mobile Min. (SW/2)**			0,02	0,02		
5	**Support settelement max**			0,00	0,00		
6	**Support settelement min**			0,00	0,00		
7	**Prestress t=0**	-8,65	-8,65	-0,41	-0,41		
8	**Prestress t = ∞**	-7,91	-7,91	-0,38	-0,38		
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-7,91	-7,91	-0,36	-0,36	-8,27	-8,27
LC 1 + LC 2 + LC 3a + LC 6 + LC8		-7,91	-7,91	-0,36	-0,36	-8,27	-8,27
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-7,91	-7,91	-0,35	-0,35	-8,26	-8,26
LC 1 + LC 2 + LC 4 + LC 6 + LC8		-7,91	-7,91	-0,35	-0,35	-8,26	-8,26
LC 1 + LC 7		-8,65	-8,65	-0,41	-0,41	-9,05	-9,06
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-7,91	-7,91	-0,36	-0,36	-8,27	-8,27
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		-7,91	-7,91	-0,36	-0,36	-8,27	-8,27

3.2 Vertical normal stress

In appendix A it is explained how a load on the floor can lead to a clamping moment and suspension force in the girder. Both these loads contribute to the vertical normal stress. An example is given for load case 3.

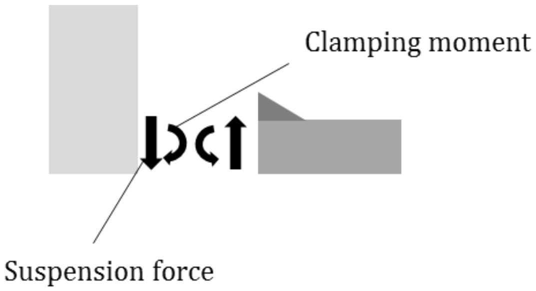
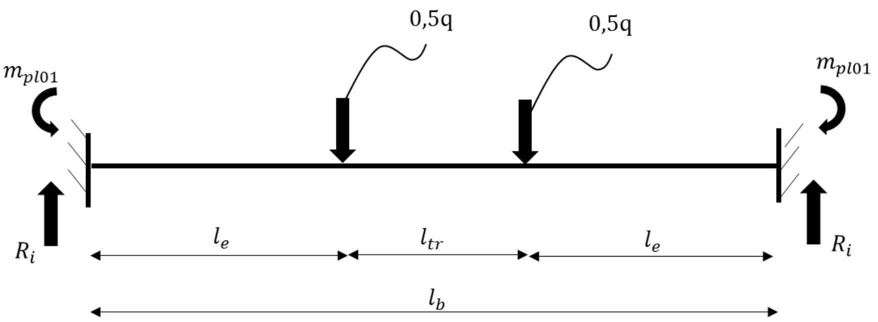


Figure C-10: Clamping moment and suspension force acting on the girder

Firstly the reactional forces, m_{pl01} and R_i, are determined by considering a transverse section of the bridge. The line load representing LC 3 is distributed over the two rails with a spacing of $l_{tr} = 1{,}50\ m$. The floor is assumed to be fully restrained in the centre of the girders ($l_b = 4{,}9\ m$).



$l_e = 1{,}7\ m$
$l_{tr} = 1{,}5\ m$
$l_b = 4{,}9\ m$
$q = 239{,}1\ kN/m$

Figure C-11: Transverse section of LC 3 acting on bridge A

$$m_{pl0} = \frac{0{,}5q * l_e * (2 * l_{tr} + l_e)}{l_b} = 195\ kNm$$

(Chapter 3.1 Appendix A)

$$Q_{yy} = R_i = 0{,}5q = 120\ kN$$

The suspension force spreads under angle of 45° towards the girders. Because LC 3 is present in the considered section (0,8d), the suspension force is taken as $Q_{yy} = 120\ kN$.

In appendix A it is explained that the clamping moment is the derivative of the torsion function. A number of formula's are derived for a couple of basic load cases. In order to establish the clamping moment for the considered example, LC 3 needs to be split into two separate cases.

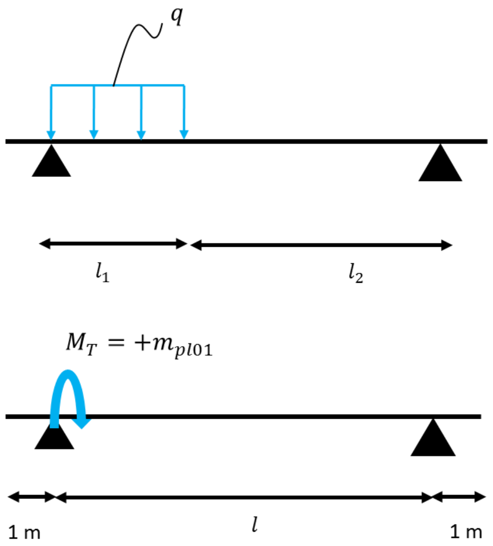


Figure C-12: Distributed line load (top) and torsional moment (bottom) representing LC 3

The distributed line load and torsional moment respectively have a contribution of $-49{,}8\ kNm$ and $+33{,}8\ kNm$ (Figure C-13). The total clamping moment then equals: $m_{xx} = -16{,}0\ kNm$.

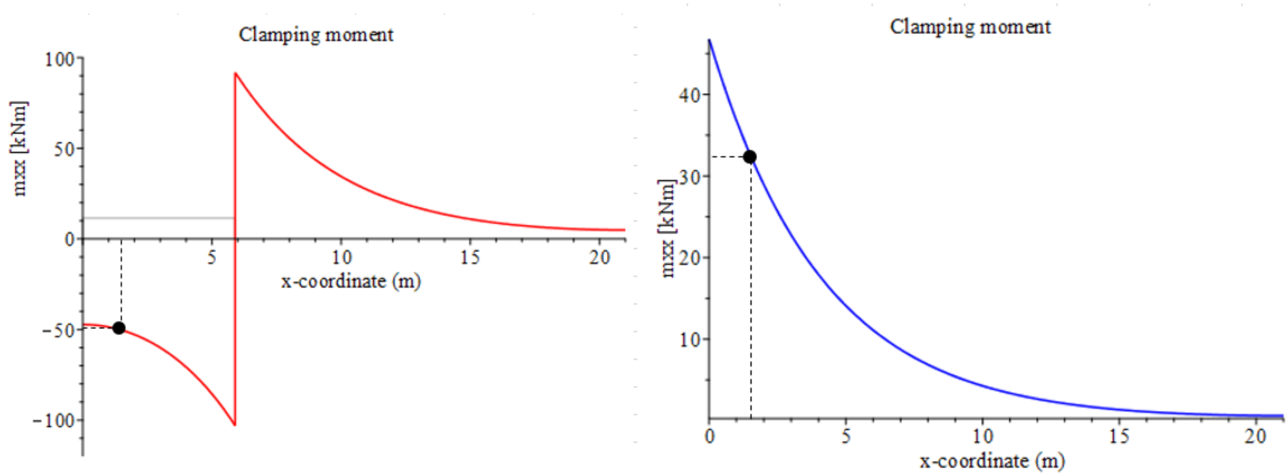


Figure C-13: Clamping moment due to distributed line load (left) and torsional moment (right)

A linear elastic calculation is assumed, which means that the girder remains uncracked. In that case the entire cross-section will contribute to the load transfer of the suspension force and clamping moment. A ratio is therefore introduced which describes the distribution of vertical normal stress over a section above and below the throat. When the girder is cracked, for example in a stirrup calculation, this distribution is no longer valid.

$$A_1 = b_{girder} * \left(h_{girder} - h_{floor} - 0{,}3\right) = 0{,}81\ m^2$$

$$A_2 = b_{girder} * \left(h_{floor} + 0{,}3\right) + 0{,}5 * 0{,}5 * 0{,}3 + 0{,}5 * h_{floor} = 1{,}115\ m^2$$

Ratio: $\frac{A_1}{A_1+A_2} * 100\% = 42\%$

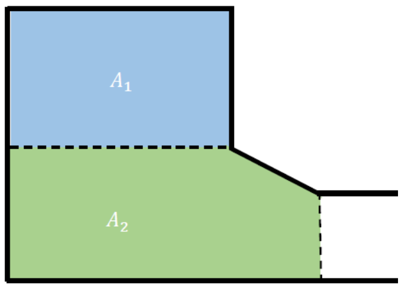


Figure C-14: Area above the throat section (A1) and area below the throat section (A2)

The suspension force could be seen as an eccentric load, acting at the edge of the girder. The result is a centric suspension force and additional bending moment due to the eccentricity of the force. A maximum tensile stress is therefore obtained on the 'inside' of the girder.

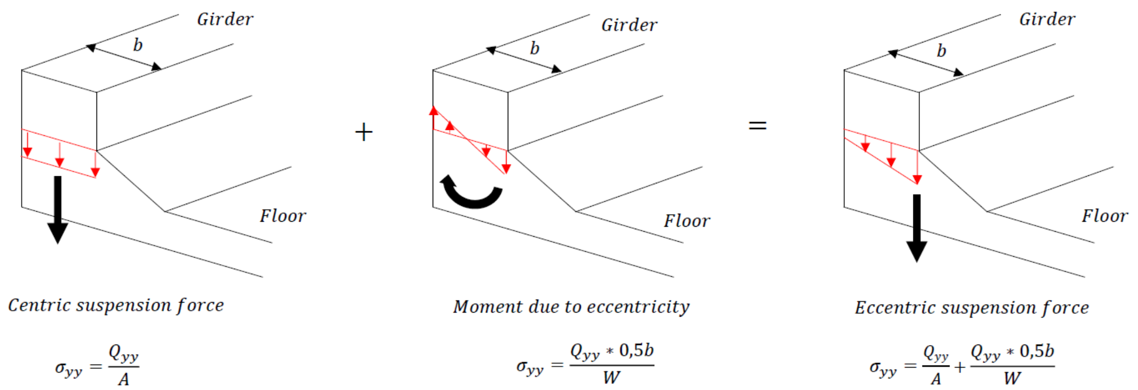


Figure C-15: Vertical normal stress due to the suspension force

The sectional properties are based on a strip with a length $dx = 1{,}0\ meters$.

Area: $A = b_{girder} * 1 = 0{,}90\ m^2$

Section modulus: $W = \frac{1}{6} * 1 * {b_{girder}}^2 = 0{,}135\ m^3$

The clamping moment acts at the centre of the girder, causing a compressive stress left and tensile stress right. The critical location for shear tension failure is therefore at the right edge (inside) of the girder.

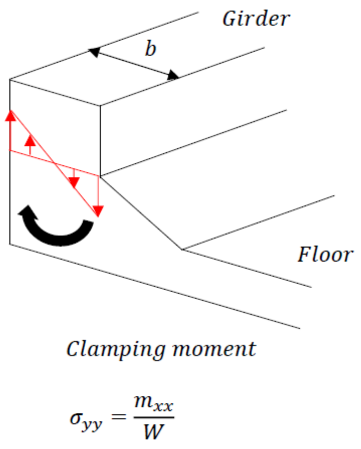


Figure C-16: Vertical normal stress due to the clamping moment

Equation [C.3] and [C.4] describe the vertical normal stress due to suspension force and clamping moment.

$$\sigma_{yy,Q_{yy}} = \frac{Q_{yy}}{A} + \frac{Q_{yy} * 0{,}5 * b_{girder}}{W} \quad [\ C.3\ ]$$

$$\sigma_{yy,m_{xx}} = \frac{m_{xx}}{W} \quad [\ C.4\ ]$$

The vertical normal stress for LC 3 equals:

$$\sigma_{yy} = \left(\frac{-16{,}0 * 10^6}{1{,}35 * 10^8} + \frac{120 * 10^3}{9{,}0 * 10^5} + \frac{120 * 10^3 * 0{,}5 * 900}{1{,}35 * 10^8}\right) * 42\% = 0{,}17\ N/mm^2$$

The vertical normal stresses for bridge A and B, are for as well load combination 6.10a as 6.10b presented in in Table C-6 and Table C-7.

Table C-6: Vertical normal stress bridge A (combination 6.10a & 6.10b) – 0,8d

		Suspension force - σyy [N/mm2]		Clamping moment - σyy [N/mm2]		Total - σyy [N/mm2]	
LC	**type**	6.10a	6.10b	6.10a	6.10b	6.10a	6.10b
1	**self-weight**	0,07	0,06	0,06	0,05		
2	**ballast**	0,06	0,05	0,06	0,05		
3	**Conc. Mobile Load**	0,22	0,28	-0,05	-0,06		
4	**Cant. Mobile Load**	0,00	0,00	0,17	0,19		
5a	**Contin. Mobile Load**	0,00	0,00	0,09	0,11		
5b	**Contin. Mobile Load (SW/2)**	0,16	0,20	0,12	0,15		
6	**Support settelement max**	-0,03	-0,03	0,31	0,31		
7	**Support settelement min**	0,03	0,03	-0,31	-0,31		
8	**Prestress t=0**	-0,10	-0,10	0,09	0,09		
9	**Prestress t = ∞**	-0,09	-0,09	0,08	0,08		
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		0,24	0,28	0,54	0,54	0,78	0,81
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		0,30	0,34	-0,07	-0,08	0,23	0,27
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		0,01	0,00	0,67	0,68	0,69	0,68
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		0,08	0,06	0,06	0,07	0,14	0,13
LC 1 + LC 8		-0,02	-0,03	0,14	0,14	0,12	0,11
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		0,17	0,20	0,62	0,64	0,79	0,84
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		0,24	0,26	0,01	0,03	0,25	0,29

Table C-7: Vertical normal stress bridge B (combination 6.10a & 6.10b) – support

		Suspension force - σyy [N/mm2]		Clamping moment - σyy [N/mm2]		Total - σyy [N/mm2]	
LC	**type**	6.10a	6.10b	6.10a	6.10b	6.10a	6.10b
1	**self-weight**	0,10	0,09	0,07	0,06		
2	**ballast**	0,06	0,05	0,04	0,04		
3a	**Mobile Max. (LM71)**	0,19	0,23	-0,01	-0,01		
3b	**Mobile Max. (SW/2)**	0,13	0,17	0,05	0,06		
4	**Mobile Min. (SW/2)**	0,13	0,17	0,12	0,15		
5	**Support settelement max**	-0,44	-0,44	0,47	0,47		
6	**Support settelement min**	0,44	0,44	-0,47	-0,47		
7	**Prestress t=0**	-0,09	-0,09	0,06	0,06		
8	**Prestress t = ∞**	-0,09	-0,09	0,05	0,05		
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-0,18	-0,15	0,62	0,61	0,44	0,46
LC 1 + LC 2 + LC 3a + LC 6 + LC8		0,69	0,72	-0,31	-0,32	0,38	0,40
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-0,23	-0,22	0,75	0,77	0,52	0,55
LC 1 + LC 2 + LC 4 + LC 6 + LC8		0,64	0,65	-0,18	-0,16	0,46	0,49
LC 1 + LC 7		0,00	-0,01	0,13	0,12	0,13	0,11
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-0,23	-0,22	0,68	0,68	0,44	0,46
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		0,64	0,65	-0,25	-0,25	0,38	0,40

3.3 Shear stress

Let's assume a shear force and torsional moment act on the girder as in Figure C-17. Where the shear force causes a constant shear stress, torsion leads to a flow of shear stresses. At the 'inside' of the girder, the shear stress due to both loads act in the same direction, resulting in the largest shear stress in the critical section. To simplify the check on shear tension failure, it is assumed that maximum shear stress due to these two forces always appear at the 'inside' of the girder.

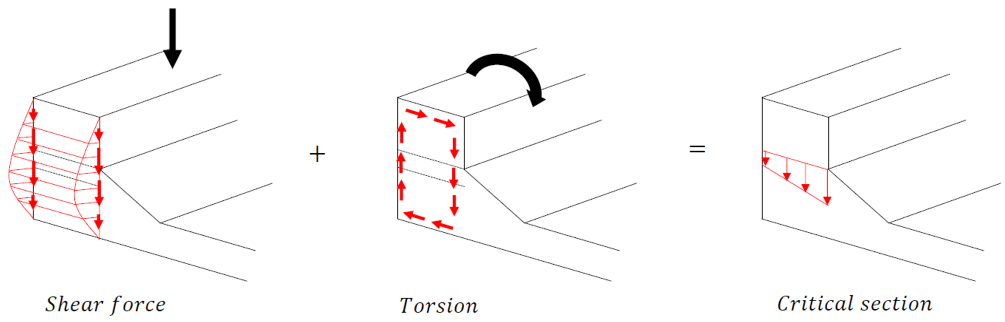


Figure C-17: The combined shear stress due to shear force and torsion

The shear force can be derived from the reactional forces in chapter 3.1.
At $x = 2{,}36\,m$ the shear force equals:

$$V_x = A_v - 0{,}5 * q * x = 446{,}5\,kN$$

In the same way as in chapter 3.2, the load case is split into two separate cases. Appendix A can be used to establish torsion in the girder.

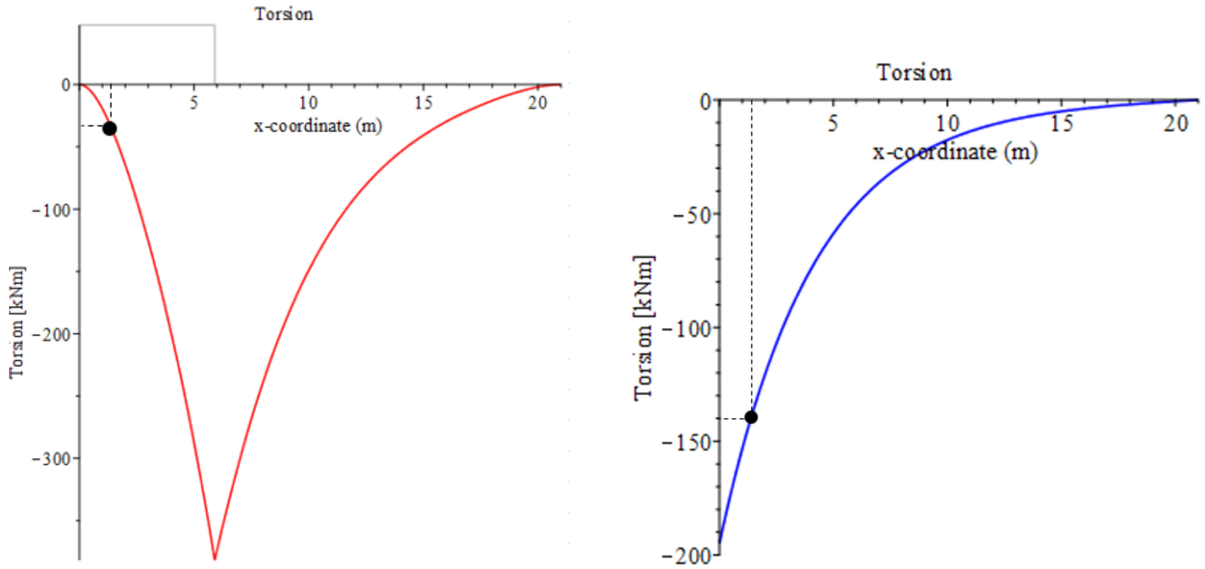


Figure C-18: Torsion due to distributed line load (left) and torsional moment (right)

The distributed line load and torsional moment respectively have a contribution of $-33{,}4\,kNm$ and $-140{,}6\,kNm$. The total torsion equals: $M_{xy} = -174\,kNm$.

The girder can be schematized as a tube with thin walls. An effective wall thickness of 150 mm is adopted from the design report (3). The shear stresses are computed using equation [C.5] and [C.6].

$$\tau_{xy,V_x} = \frac{V_x * S}{b * I_{yy}} \qquad [\text{ C.5 }]$$

$$\tau_{xy,M_{xy}} = \frac{M_{xy}}{2 * A_m * t_{ef}} \qquad [\text{ C.6 }]$$

Where:

$S = statical\ moment\ of\ area\ (mm^3)$
$b = widht\ of\ the\ girder\ (mm)$
$I_{yy} = moment\ of\ inertia\ (mm^4)$
$A_m = area\ enclosed\ by\ the\ heartlines\ of\ the\ walls\ (mm^2)$
$t_{ef} = effective\ thickness\ of\ the\ walls\ (mm)$

Statical moment of Area:	$S = b_{girder} * \left(h_{girder} - h_{floor} - 0{,}3\right)^2 * 0{,}5 = 0{,}3645\ m^3$
Area enclosed by walls:	$A_m = \left(b_{girder} - t_{ef}\right) * \left(h_{girder} - t_{ef}\right) = 1{,}2\ m^2$

The total torsional resistance is formed by the combination of the girder and the effective part of the floor. Since only torsion in the girder is considered, a ratio is introduced that takes this distribution into account. The notes by P.C.J. Hoogenboom (22) are used to calculate the torsional constants of the girder and floor.

Girder	Floor
$a = \frac{h_{girder}}{2} = 0{,}875\ m \quad \& \quad b = \frac{b_{girder}}{2} = 0{,}45\ m$	$a = \frac{0.5 * b_{floor}}{2} = 1{,}0\ m \quad \& \quad b = \frac{h_{floor}}{2} = 0{,}275\ m$
$\alpha = \left[\frac{16}{3} - 3{,}36 * \frac{b}{a} * \left(1 - \frac{b^4}{12a^4}\right)\right] = 3{,}62$	$\alpha = \left[\frac{16}{3} - 3{,}36 * \frac{b}{a} * \left(1 - \frac{b^4}{12a^4}\right)\right] = 4{,}41$
$I_{t,girder} = \alpha * ab^3 = 0{,}29\ m^4$	$I_{t,floor} = \alpha * ab^3 = 0{,}09\ m^4$

Ratio:	$Ratio = \frac{I_{t,girder}}{I_{t,girder} + I_{t,floor}} = 75{,}9\%$

Combing equation [C.5] and [C.6] and the ratio, the total shear stress for LC 3 can be computed:

$$\tau_{xy} = \left(\frac{446{,}5\ kN * 0{,}36\ m^3}{0{,}663\ m^4 * 0{,}9\ m}\right) + \left(\frac{|-174\ kNm|}{2 * 1{,}2\ m^2 * 0{,}15\ m}\right) * 75{,}9\% = 0{,}64\ N/mm^2$$

The shear stresses for bridge A and B and both load combinations are presented in Table C-8 and Table C-9.

Table C-8: Shear stress bridge A (combination 6.10a & 6.10b) – 0,8d

LC	type	Shear force - τxy [N/mm2]		Torsion - τxy [N/mm2]		Total - τxy [N/mm2]	
		6.10a	6.10b	6.10a	6.10b	6.10a	6.10b
1	self-weight	0,54	0,48	-0,22	-0,20		
2	ballast	0,18	0,16	-0,19	-0,17		
3	Conc. Mobile Load	0,27	0,34	-0,37	-0,46		
4	Cant. Mobile Load	0,00	0,00	-0,47	-0,52		
5a	Contin. Mobile Load	0,22	0,27	0,04	0,05		
5b	Contin. Mobile Load (SW/2)	0,48	0,60	-0,21	-0,26		
6	Support settelement max	0,00	0,00	-0,45	-0,45		
7	Support settelement min	0,00	0,00	0,45	0,45		
8	Prestress t=0	-0,56	-0,56	0,08	0,08		
9	Prestress t = ∞	-0,51	-0,51	0,07	0,07		
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		0,70	0,74	-1,12	-1,16	1,82	1,90
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		0,70	0,74	-0,22	-0,26	0,91	1,00
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		0,21	0,13	-1,26	-1,26	1,46	1,39
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		0,21	0,13	-0,36	-0,36	0,56	0,49
LC 1 + LC 8		-0,03	-0,08	-0,14	-0,12	0,17	0,20
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		0,68	0,72	-1,00	-1,01	1,68	1,73
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		0,68	0,72	-0,10	-0,11	0,78	0,83

Table C-9: Shear stress bridge B (combination 6.10a & 6.10b) – support

LC	type	Shear force - τxy [N/mm2]		Torsion - τxy [N/mm2]		Total - τxy [N/mm2]	
		6.10a	6.10b	6.10a	6.10b	6.10a	6.10b
1	self-weight	1,08	0,96	-0,33	-0,29		
2	ballast	0,25	0,22	-0,16	-0,14		
3a	Mobile Max. (LM71)	0,57	0,71	-0,23	-0,29		
3b	Mobile Max. (SW/2)	0,54	0,68	-0,17	-0,21		
4	Mobile Min. (SW/2)	0,00	0,00	-0,37	-0,46		
5	Support settelement max	0,00	0,00	-0,60	-0,60		
6	Support settelement min	0,00	0,00	0,60	0,60		
7	Prestress t=0	-1,03	-1,03	0,00	0,00		
8	Prestress t = ∞	-0,95	-0,95	0,00	0,00		
LC 1 + LC 2 + LC 3a + LC 5 + LC8		0,95	0,95	-1,32	-1,33	2,27	2,27
LC 1 + LC 2 + LC 3a + LC 6 + LC8		0,95	0,95	-0,12	-0,13	1,07	1,08
LC 1 + LC 2 + LC 4 + LC 5 + LC8		0,38	0,23	-1,46	-1,50	1,83	1,73
LC 1 + LC 2 + LC 4 + LC 6 + LC8		0,38	0,23	-0,26	-0,30	0,64	0,53
LC 1 + LC 7		0,04	-0,07	-0,33	-0,29	0,37	0,37
LC 1 + LC 2 + LC 3b + LC 5 + LC8		0,92	0,91	-1,26	-1,24	2,18	2,16
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		0,92	0,91	-0,06	-0,05	0,98	0,96

4 Principal stresses

An element under stress has at least three planes, called the principal planes. At each plane a normal vector acts, perpendicular to the plane, called the principal direction. The stresses acting parallel to the normal vectors are the so called principal stresses. Each stressed element has three principal stresses, namely the tensile and compressive principal stress and the maximum shear stress (where the normal stresses equal zero).

$$\rho_1 = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}{}^2} \qquad [\text{ C.7 }]$$

$$\rho_2 = \frac{\sigma_x + \sigma_y}{2} - \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}{}^2} \qquad [\text{ C.8 }]$$

Respectively Table C-10 and Table C-11 present the tensile principal stresses in bridge A and B. Remarkable are the lower stresses in bridge B in comparison to bridge A. The explanation for this lies in the fact that for a section above the supports the horizontal normal stress due to prestress is larger than at a distance 0,8d from the support. This contribution increases the resistance against shear tension failure, meaning that even though bridge B is subjected to larger shear stresses it eventually has a lower tensile principal stress.

Table C-10: Tensile principal stresses bridge A (combination 6.10a & 6.10b) – 0,8d

		Tensile princ. stress - ρ1 [N/mm2]	
LC	type	6.10a	6.10b
1	self-weight		
2	ballast		
3	Conc. Mobile Load		
4	Cant. Mobile Load		
5a	Contin. Mobile Load		
5b	Contin. Mobile Load (SW/2)		
6	Support settelement max		
7	Support settelement min		
8	Prestress t=0		
9	Prestress t = ∞		
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		1,22	1,29
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		0,36	0,42
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		1,00	0,97
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		0,19	0,17
LC 1 + LC 8		0,12	0,11
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		1,18	1,24
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		0,34	0,39

Table C-11: Principal stresses bridge B (combination 6.10a & 6.10b) – support

		Tensile princ. stress - ρ1 [N/mm2]	
LC	type	6.10a	6.10b
1	self-weight		
2	ballast		
3a	Mobile Max. (LM71)		
3b	Mobile Max. (SW/2)		
4	Mobile Min. (SW/2)		
5	Support settelement max		
6	Support settelement min		
7	Prestress t=0		
8	Prestress t = ∞		
LC 1 + LC 2 + LC 3a + LC 5 + LC8		1,00	1,02
LC 1 + LC 2 + LC 3a + LC 6 + LC8		0,51	0,53
LC 1 + LC 2 + LC 4 + LC 5 + LC8		0,88	0,88
LC 1 + LC 2 + LC 4 + LC 6 + LC8		0,50	0,52
LC 1 + LC 7		0,14	0,13
LC 1 + LC 2 + LC 3b + LC 5 + LC8		0,96	0,96
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		0,49	0,50

In order to study whether or not the correct critical location for bridge B was chosen, a section at 0,8d from the support is considered. For this section the contribution of prestress to the horizontal normal stress is less than before, but likewise are the shear stresses due to torsion. Ultimately the maximum tensile principal stress at 0,8d from the support is 0,98 N/mm^2, which is slightly smaller than the stress found at a section above the support. This means the correct critical location was assumed for bridge B.

In Eurocode 2 (16) a formula for shear tension failure is given in section 6.2.2.

$$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd})^2 + \sigma_{cp} * f_{ctd}} \qquad [\text{ C.9 }]$$

Where:
$\sigma_{cp} = compressive\ stress\ due\ to\ working\ prestress$ (N/mm^2)
$f_{ctd} = concrete\ tensile\ design\ strength$ (N/mm^2)

The formula can be rewritten by taking σ_{cp} as the normal stress and f_{ctd} as tensile principal stress. Equation [C.5] is applied to convert the shear force into a shear stress.

$$\tau_{xy} = \sqrt{(\rho_1)^2 + \sigma * \rho_1}$$

$$\rho_1 = -\frac{\sigma}{2} + \sqrt{\left(\frac{\sigma}{2}\right)^2 + {\tau_{xy}}^2} \qquad [\text{ C.10 }]$$

To conclude, shear tension failure will not be an issue if tensile stresses remain smaller than the concrete tensile design strength. In the Dutch National Annex of Eurocode 2 (13), the coefficient for long-term loading and the safety factor for concrete are found.

$$f_{ctd} = \propto_{ct} * \frac{f_{ctk;0.05}}{\gamma_c} = 1{,}0 * \frac{2{,}0}{1{,}5} = 1{,}33\ N/mm^2$$

Where:

$\alpha_{ct} = coefficient\ for\ long\ term\ loading$
$\gamma_c = safety\ factor\ for\ concrete$

For both bridges load combination 6.10b of LM71 is governing. The unity check for shear tension failure for bridge A and B are as follows:

Bridge A: $U.C. = \frac{\rho_1}{f_{ctd}} = \frac{1{,}29}{1{,}33} = 0{,}97$

Bridge B: $U.C. = \frac{\rho_1}{f_{ctd}} = \frac{1{,}02}{1{,}33} = 0{,}77$

With a maximum unity check of 0,97 and 0,77, there is no risk of shear tension failure for both bridges when applying the load models and safety factor from the Eurocode.

5 Spreadsheets

5.1 Bridge A - 6.10a – 0,8d

Normal stresses

		Prestress		Bending moment		Total hor. normal stress
LC	type	P [kN]	σxx [N/mm2]	M [kNm]	σxx [N/mm2]	σxx [N/mm2]
1	self-weight			1237	-0,41	
2	ballast			344	-0,11	
3	Conc. Mobile Load			658	-0,22	
4	Cant. Mobile Load			-213	0,07	
5a	Contin. Mobile Load			487	-0,16	
5b	Contin. Mobile Load (SW/2)			1098	-0,37	
6	Support settelement max			0	0,00	
7	Support settelement min			0	0,00	
8	Prestress t=0	-13200	-6,14	-1055	0,35	
9	Prestress t = ∞	-12038	-5,60	-962	0,32	
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		-12038	-5,60	1765	-0,59	-6,19
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		-12038	-5,60	1765	-0,59	-6,19
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		-12038	-5,60	406	-0,14	-5,73
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		-12038	-5,60	406	-0,14	-5,73
LC 1 + LC 8		-13200	-6,14	182	-0,06	-6,20
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		-12038	-5,60	1717	-0,57	-6,17
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		-12038	-5,60	1717	-0,57	-6,17

		Suspension force		Clamping moment		Suspension force excen.		Total ver. Normal stress
LC	type	Qyy [kN]	σyy [N/mm2]	mxx [kNm]	σyy [N/mm2]	Myy [kNm]	σyy [N/mm2]	σyy [N/mm2]
1	self-weight	39	0,02	18	0,06	17	0,05	
2	ballast	33	0,02	20	0,06	15	0,05	
3	Conc. Mobile Load	120	0,06	-16	-0,05	54	0,17	
4	Cant. Mobile Load	0	0,00	55	0,17	0	0,00	
5a	Contin. Mobile Load	0	0,00	28	0,09	0	0,00	
5b	Contin. Mobile Load (SW/2)	85	0,04	38	0,12	38	0,12	
6	Support settelement max	-18	-0,01	98	0,31	-8	-0,03	
7	Support settelement min	18	0,01	-98	-0,31	8	0,03	
8	Prestress t=0	-52	-0,02	28	0,09	-23	-0,07	
9	Prestress t = ∞	-47	-0,02	26	0,08	-21	-0,07	
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		126	0,06	173	0,54	57	0,18	0,78
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		162	0,08	-24	-0,07	73	0,23	0,23
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		6	0,00	216	0,67	3	0,01	0,69
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		42	0,02	19	0,06	19	0,06	0,14
LC 1 + LC 8		-13	-0,01	46	0,14	-6	-0,02	0,12
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		92	0,04	200	0,62	41	0,13	0,79
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		128	0,06	3	0,01	57	0,18	0,25

Shear and principal stresses

LC	type	Shear force Vz [kN]	Shear force τxy [N/mm2]	Torsion Mxy [kNm]	Torsion τxy [N/mm2]	Total shear stress τxy [N/mm2]
1	self-weight	880	0,54	-104	-0,22	
2	ballast	300	0,18	-91	-0,19	
3	Conc. Mobile Load	447	0,27	-174	-0,37	
4	Cant. Mobile Load	0	0,00	-221	-0,47	
5a	Contin. Mobile Load	358	0,22	18	0,04	
5b	Contin. Mobile Load (SW/2)	781	0,48	-100	-0,21	
6	Support settelement max	0	0,00	-214	-0,45	
7	Support settelement min	0	0,00	214	0,45	
8	Prestress t=0	-923	-0,56	37	0,08	
9	Prestress t = ∞	-842	-0,51	34	0,07	
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		1142	0,70	-530	-1,12	1,82
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		1142	0,70	-103	-0,22	0,91
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		338	0,21	-596	-1,26	1,46
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		338	0,21	-169	-0,36	0,56
LC 1 + LC 8		-43	-0,03	-67	-0,14	0,17
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		1118	0,68	-475	-1,00	1,68
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		1118	0,68	-48	-0,10	0,78

LC	type	Total σxx [N/mm2]	Total σyy [N/mm2]	Total τxy [N/mm2]	Principal stress ρ1 [N/mm2]	Principal stress ρ2 [N/mm2]
1	self-weight					
2	ballast					
3	Conc. Mobile Load					
4	Cant. Mobile Load					
5a	Contin. Mobile Load					
5b	Contin. Mobile Load (SW/2)					
6	Support settelement max					
7	Support settelement min					
8	Prestress t=0					
9	Prestress t = ∞					
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		-6,19	0,78	1,82	1,22	-6,63
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		-6,19	0,23	0,91	0,36	-6,31
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		-5,73	0,69	1,46	1,00	-6,05
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		-5,73	0,14	0,56	0,19	-5,79
LC 1 + LC 8		-6,20	0,12	0,17	0,12	-6,20
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		-6,17	0,79	1,68	1,18	-6,56
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		-6,17	0,25	0,78	0,34	-6,26

Parameters

Bending stiffness coefficient		
E	34000	[N/mm2]
S_{pl}	2,36E+08	

Torsional stiffness coefficient		
v	0,20	
G	1,42E+04	[N/mm2]
$I_{t,girder}$	2,88E+11	[mm4]
GI_t	4,08E+15	[Nmm2]
ω	0,24	

Shear stress parameters		
I_{yy}	6,63E+11	[mm4]
S	3,65E+08	[mm]
b_{girder}	900	[mm]
A_M	1200000	[mm2]
t_{ef}	150	[mm]

Sectional properties		
b_{girder}	900	[mm]
h_{girder}	1750	[mm]
t_{floor}	550	[mm]
b_{floor}	4000	[mm]
$0{,}5 * A_{prestress}$	2150500	[mm2]
$0{,}5 * I_{yy}$	6,63E+11	[mm4]
y	221	[mm]
z	629	[mm]

Normal stress parameters		
A_1	810000	[mm2]
A_2	1115000,00	[mm2]
$A_1/(A_1+A_2)$	42,1%	
$A_{Q_{yy}}$	900000	[mm2]
$W_{m_{xx}}$	135000000	[mm3]
$eccentricity$	0,45	[m]

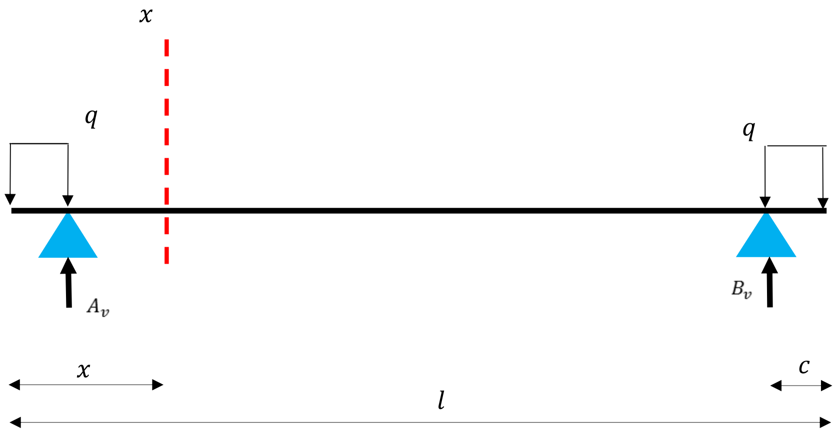
Torsional stiffness girder		
a	0,875	[m]
b	0,45	[m]
$\propto$	3,62	[-]
$I_{t,girder}$	0,29	[m4]

Torsional stiffness floor		
a	1,00	[m]
b	0,275	[m]
$\propto$	4,41	[-]
$I_{t,floor}$	0,09	[m4]
$I_{t,grider}/(I_{t,floor} + I_{t,girder})$	75,86%	[%]

LC 1

Cantilevers loaded

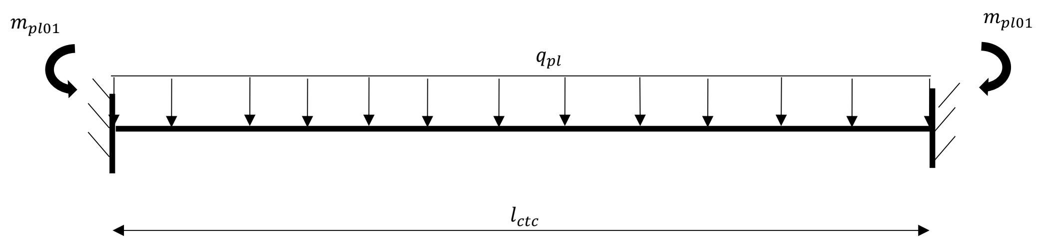
Loading (long. direction)



Load & Reaction forces		
qbridge	192,5	[kN/m]
0,5q	96,3	[kN/m]
Av	96,3	[kN]
Bv	96,3	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	48,1	[kN/m]
mpl01	64,2	[kNm]

Measurements		
lctc	4,0	[m]

Shear Force

Vz	0,0	[kN]

Bending moment

Mx	-48,1	[kNm]

Torsion

Mxy,M	-46,0	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	-46,0	[kNm]	

Suspension force

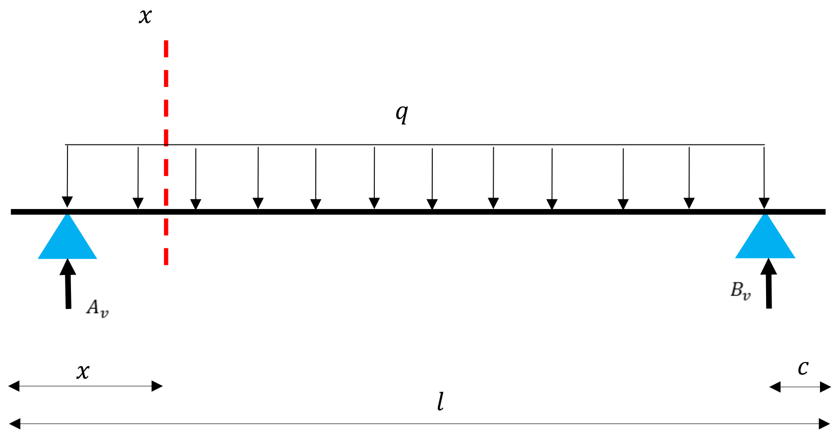
Qyy	0,0	[kN]

Clamping moment

mxx,M	11,3	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	11,3	[kNm]	

Midspan loaded

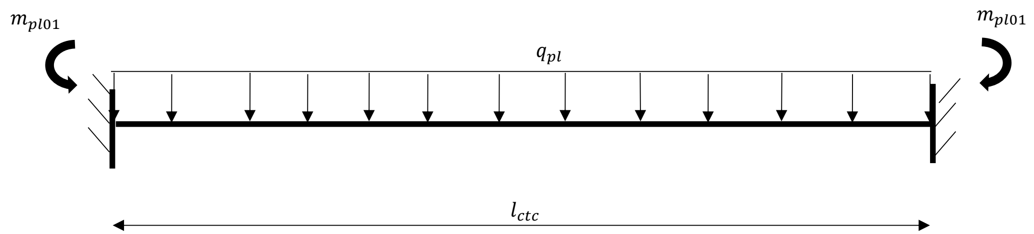
Loading (long. direction)



Load & Reaction forces		
qbridge	192,5	[kN/m]
0,5q	96,3	[kN/m]
Av	1010,6	[kN]
Bv	1010,6	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	48,1	[kN/m]
mpl02	64,2	[kNm]

Measurements		
lctc	4,0	[m]

Shear Force

Vz 879,7 [kN]

Bending moment

Mx 1285,4 [kNm]

Torsion

Mxy, alt	-58,2	[kNm]	(due to alternative load case)
Mxy,subtot	-58,2	[kNm]	

Suspension force

Qyy 38,5 [kN]

Clamping moment

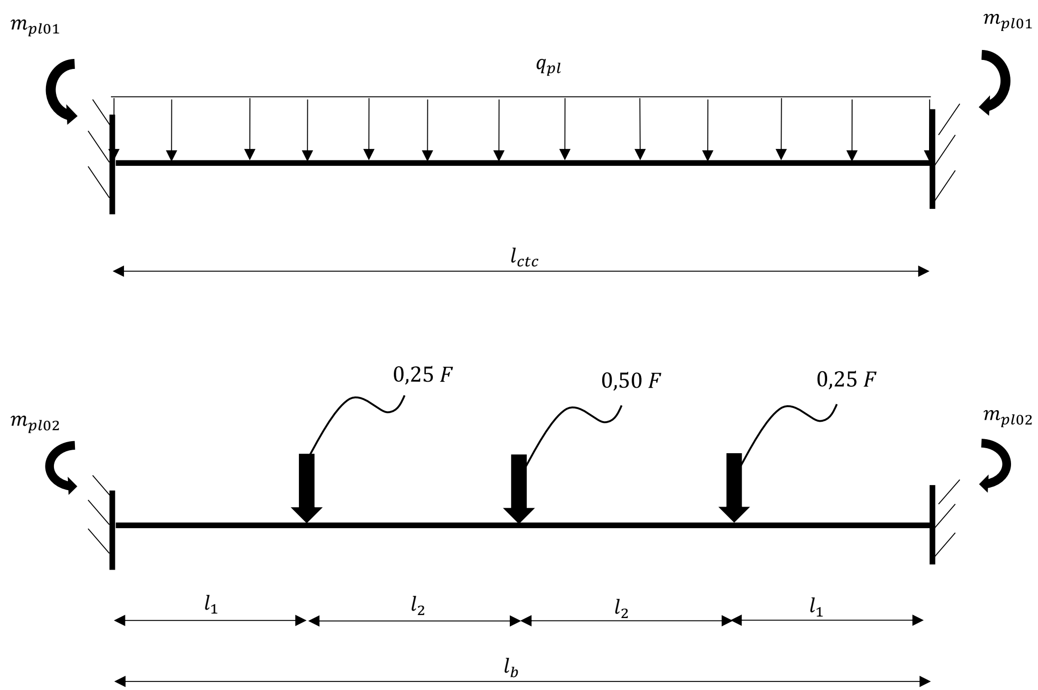
mxx, alt	6,4	[kNm]	(due to alternative load case)
mxx,subtot.	6,4	[kNm]	

LC 2

Cantilevers loaded

Loading (long. direction)

Loading (transverse direction)



Load & Reaction forces		
q	16,4	[kN/m2]
0,5q	32,8	[kN/m]
F	155	[kN]
0,5F	77,5	[kN]
Av	110,3	[kN]
Bv	110,3	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	16,4	[kN/m]
0,25F	38,8	[kN]
mpl01	21,9	[kNm]
mpl02	77,1	[kNm]
MT	99,0	[kNm]

Measurements		
lctc	4,0	[m]
l1	0,95	[m]
l2	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

Bending moment

Mx	-93,9	[kNm]

Clamping moment

mxx,M	17,5	[kNm]
mxx,subtot.	17,5	[kNm]

(due to torsional moment at both ends)

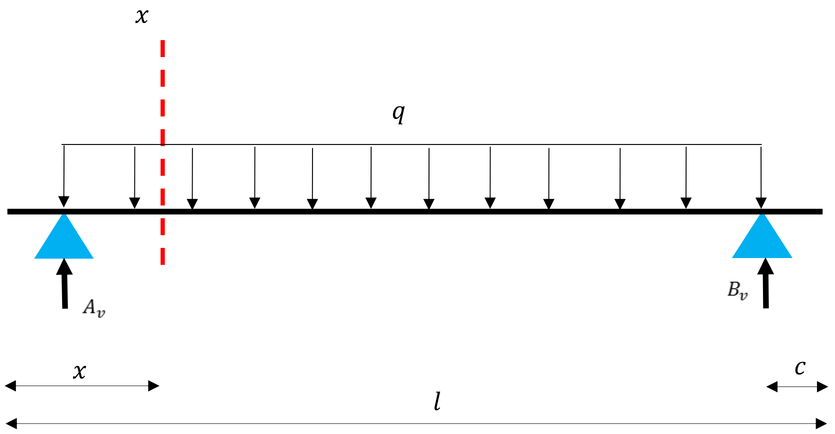
Torsion

Mxy,M	-71,0	[kNm]
Mxy,subtot	-71,0	[kNm]

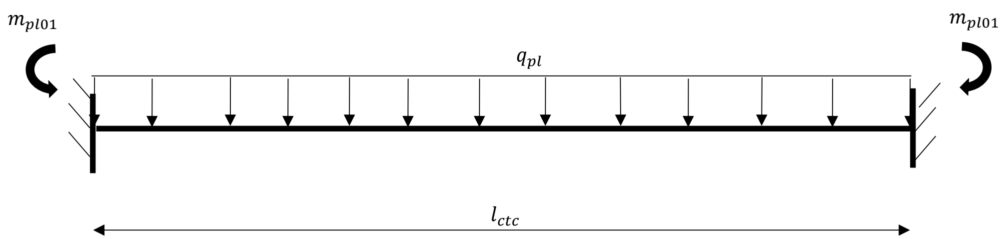
(due to torsional moment at both ends)

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces

q	16,4	[kN/m2]
0,5q	32,8	[kN/m]
Av	344,4	[kN]
Bv	344,4	[kN]

Measurements

x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces

qpl	16,4	[kN/m]
mpl01	21,9	[kNm]

Measurements

lctc	4,0	[m]

Shear Force

Vz	299,8	[kN]

Suspension force

Qyy	32,8	[kN]

Bending moment

Mx	438,1	[kNm]

Torsion

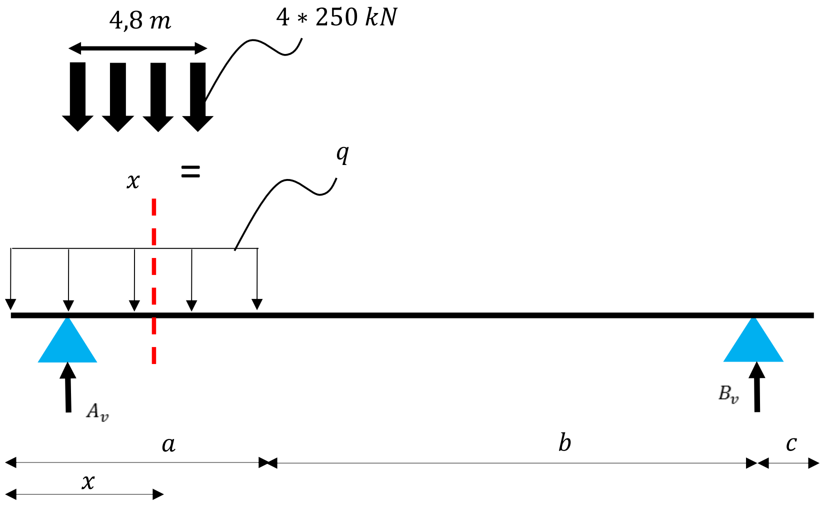
Mxy,alt	-19,9	[kNm]	(due to alternative load case)
Mxy,subtot	-19,9	[kNm]	

Clamping moment

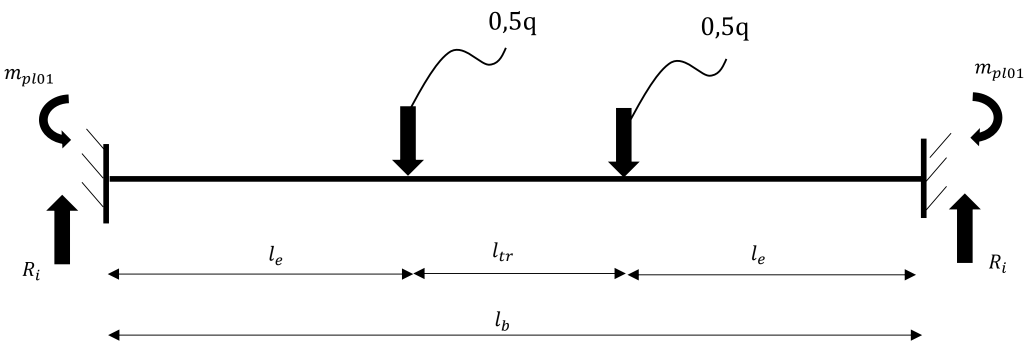
mxx,alt	2,2	[kNm]	(due to alternative load case)
mxx,subtot.	2,2	[kNm]	

LC 3

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces

q	239,1	[kN/m]
0,5q	119,6	[kN/m]
Av	728,7	[kN]
Bv	96,2	[kN]

Measurements

x	2,36	[m]
a	6,9	[m]
b	15,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces

p	119,6	[kN/m]
mpl01	194,9	[kNm]

Measurements

le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 446,5 [kN]

Bending moment

Mx 658,1 [kNm]

Torsion

Mxy,M	-140,6	[kNm]	(due to torsional moment at one end)
Mxy,q	-33,4	[kNm]	(due to distributed load)
Mxy,tot	-174,0	[kNm]	

Suspension force

Qyy 119,6 [kN]

Clamping moment

mxx,M	33,8	[kNm]	(due to torsional moment at one end)
mxx,q	-49,8	[kNm]	(due to distributed load)
mxx,tot	-16,0	[kNm]	

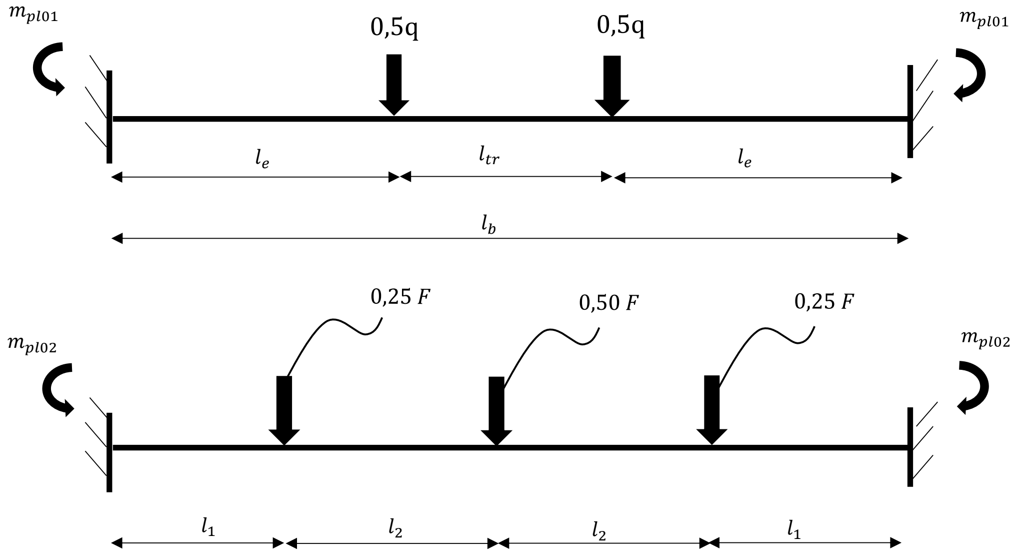
LC 4

Loading (long. direction)

Load & Reaction forces		
q	170,4	[kN/m]
0,5q	85,2	[kN/m]
F	341	[kN]
0,5F	170,5	[kN]
Av	255,7	[kN]
Bv	255,7	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	85,2	[kN/m]
0,25F	85,3	[kN]
mpl01	138,9	[kNm]
mpl02	169,7	[kNm]
MT	308,6	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]
l1	0,95	[m]
l2	1,5	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

Bending moment

Mx -213,1 [kNm]

Clamping moment

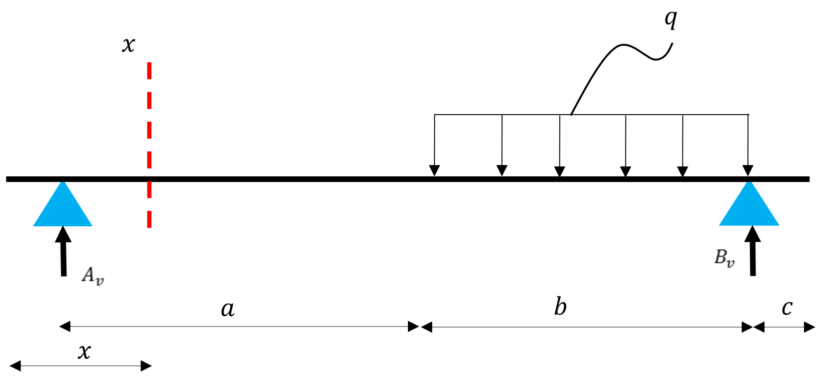
mxx,M	54,5	[kNm]	(due to torsional moment at both ends)
mxx,tot	54,5	[kNm]	

Torsion

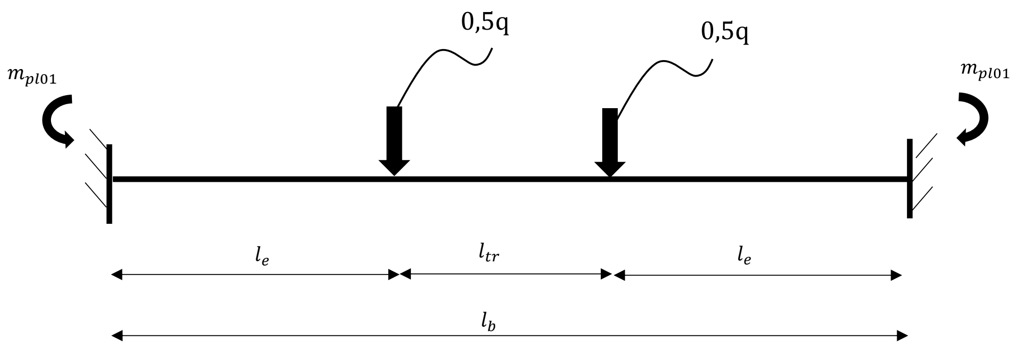
Mxy,M	-221,3	[kNm]	(due to torsional moment at both ends)
Mxy,tot	-221,3	[kNm]	

LC 5a

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	132	[kN/m]
0,5q	66,0	[kN/m]
Av	358,3	[kN]
Bv	638,3	[kN]

Measurements		
x	2,36	[m]
a	5,9	[m]
b	15,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
0,5q	66,0	[kN/m]
mpl01	107,6	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 358,3 [kN]

Bending moment

Mx 487,3 [kNm]

Torsion

Mxy,q	18,4	[kNm]	(due to distributed load)
Mxy,tot	18,4	[kNm]	

Suspension force

Qyy 0,0 [kN]

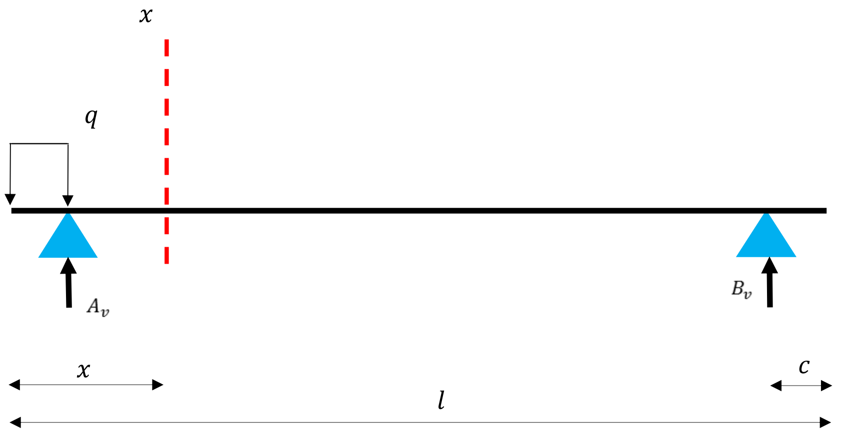
Clamping moment

mxx,q	27,5	[kNm]	(due to distributed load)
mxx,tot	27,5	[kNm]	

LC 5b

Cantilevers loaded

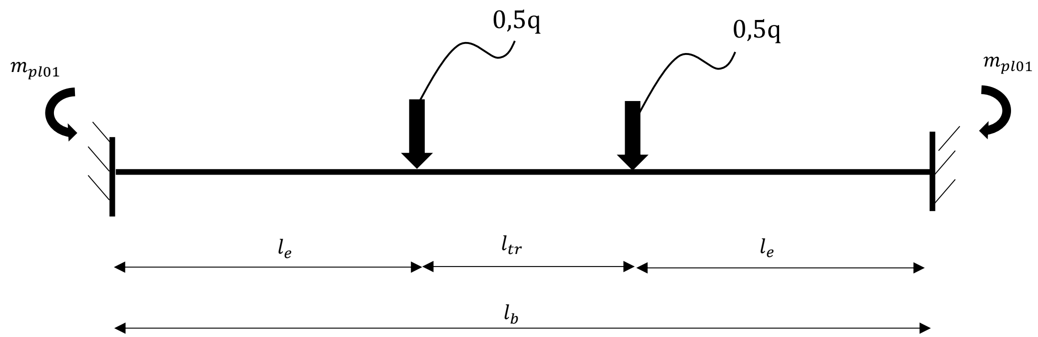
Loading (long. direction)



Load & Reaction forces		
q	170,4	[kN/m]
0,5q	85,2	[kN/m]
Av	87,2	[kN]
Bv	-2,0	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	85,2	[kN/m]
mpl01	138,9	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 2,0 [kN]

Bending moment

Mx -39,8 [kNm]

Torsion

Mxy,M	-100,2	[kNm]	(due to torsional moment at one end)
Mxy,subtot	-100,2	[kNm]	

Suspension force

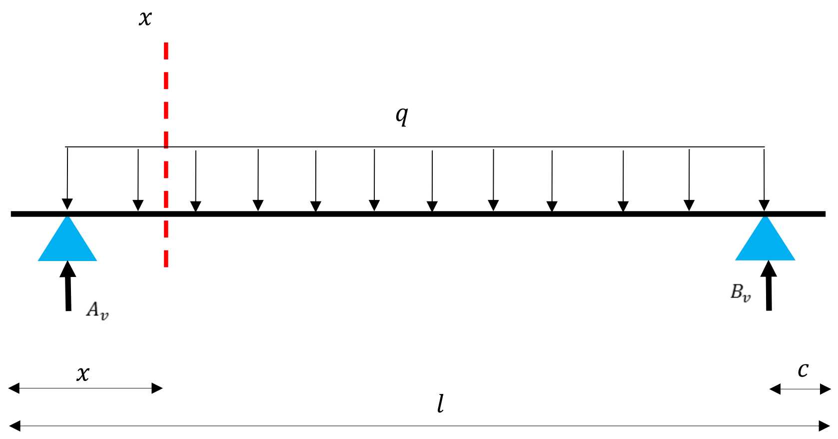
Qyy 0,0 [kN]

Clamping moment

mxx,M	24,1	[kNm]	(due to torsional moment at one end)
mxx,subtot.	24,1	[kNm]	

Midspan loaded

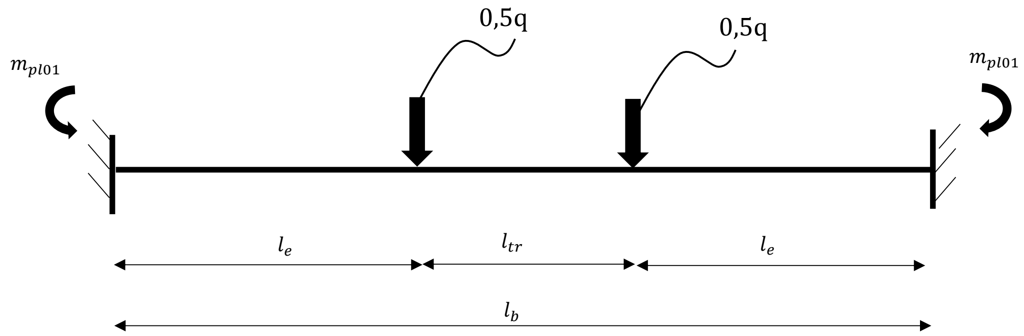
Loading (long. direction)



Load & Reaction forces		
q	170,4	[kN/m]
0,5q	85,2	[kN/m]
Av	894,6	[kN]
Bv	894,6	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	85,2	[kN/m]
mpl01	138,9	[kNm]

Measurements		
lb	4,9	[m]
ltr	1,5	[m]
le	1,7	[m]

Shear Force

Vz	778,7	[kN]

Bending moment

Mx	1137,9	[kNm]

Torsion

Mxy, alt	0,0	[kNm]	(due to alternative load case)
Mxy,subtot	0,0	[kNm]	

Suspension force

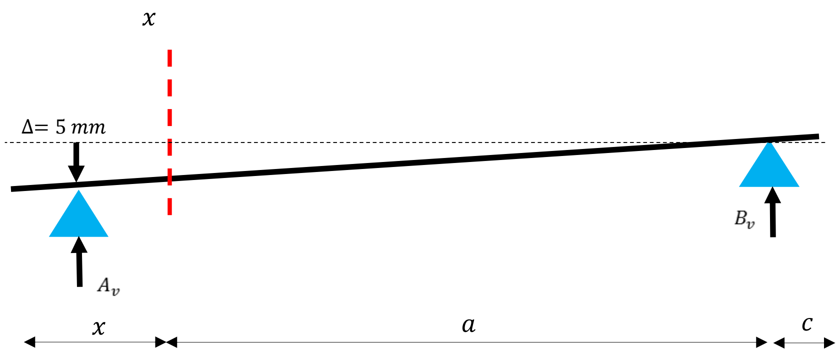
Qyy	85,2	[kN]

Clamping moment

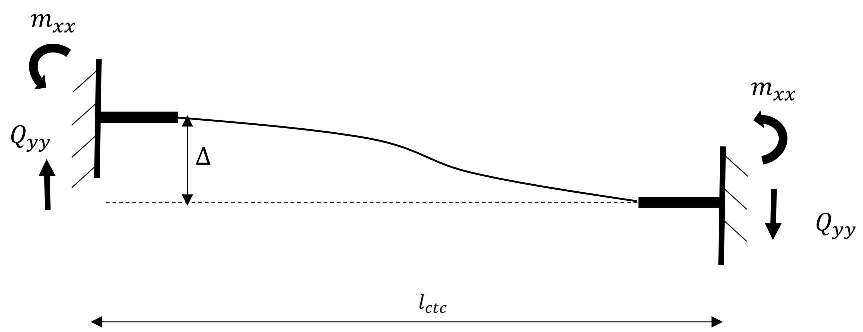
mxx, alt	13,8	[kNm]	(due to alternative load case)
mxx,subtot.	13,8	[kNm]	

LC 6

Loading (long. direction)



Loading (transverse direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γ	1,2	[-]

Measurements		
x	2,36	[m]
a	19,6	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Shear Force

Vz	0,0	[kN]

Suspension force

Qyy	-15,0	[kN]
Qyy,tot	-18,0	[kN]

Bending moment

Mx	0,0	[kNm]

Clamping moment

mxx,Δ	82,0	[kNm]
mxx,tot	98,4	[kNm]

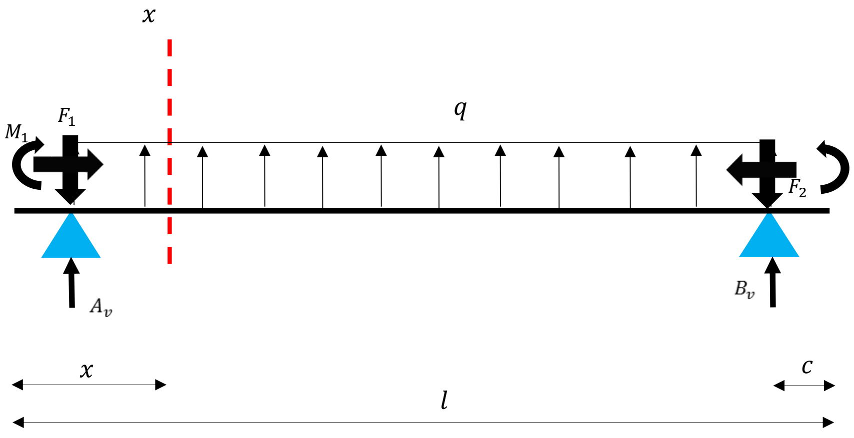
Torsion

Mxy,Δ	-178,0	[kNm]
Mxy,tot	-213,6	[kNm]

LC 8

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	1115	[kN]
F2	-13200	[kN]
M1	294	[kNm]
q	-101	[kN]
Av	55	[kN]
Bv	55	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Shear Force

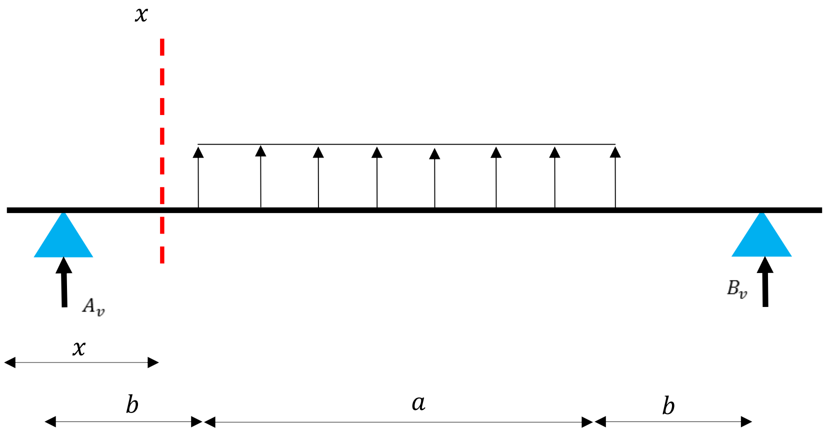
Vz	-923,1	[kN]

Bending moment

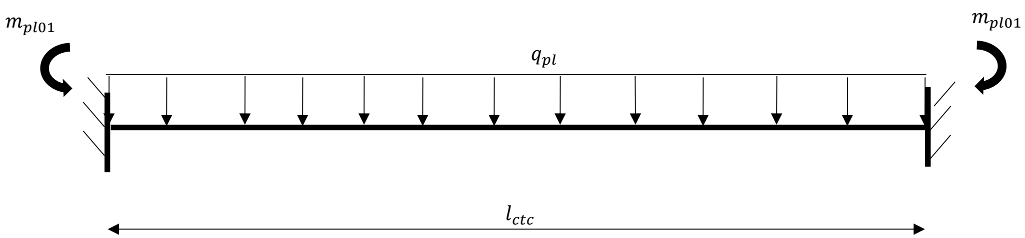
Mx	-1054,9	[kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	2,36	[m]
a	16,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	-51,5	[kN/m]
mpl01	-68,7	[kNm]

Measurements		
lctc	4,0	[m]

Torsion

Mxy,alt	37,3	[kNm]	(due to alternative load case)
Mxy,tot	37,3	[kNm]	

Suspension force

Qyy	-51,5	[kN]

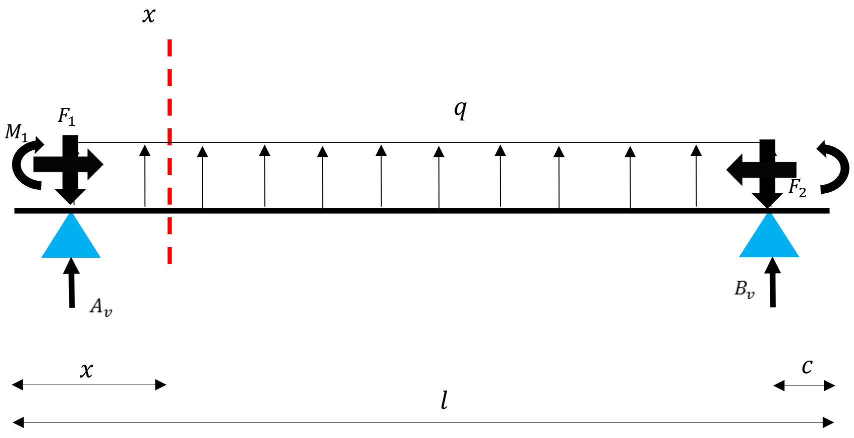
Clamping moment

mxx,alt	28,4	[kNm]	(due to alternative load case)
mxx,tot	28,4	[kNm]	

LC 9

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	1017	[kN]
F2	-12038	[kN]
M1	268	[kNm]
q	-92	[kN]
Av	50	[kN]
Bv	50	[kN]
P∞/P0	0,912	[-]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Shear Force

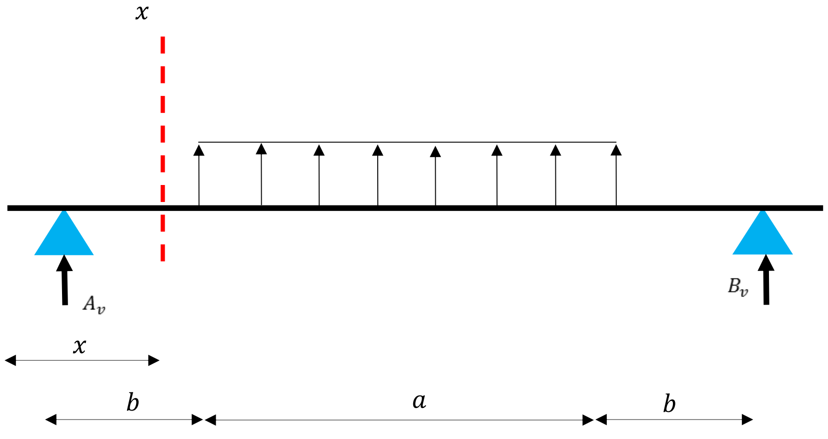
Vz -841,9 [kN]

Bending moment

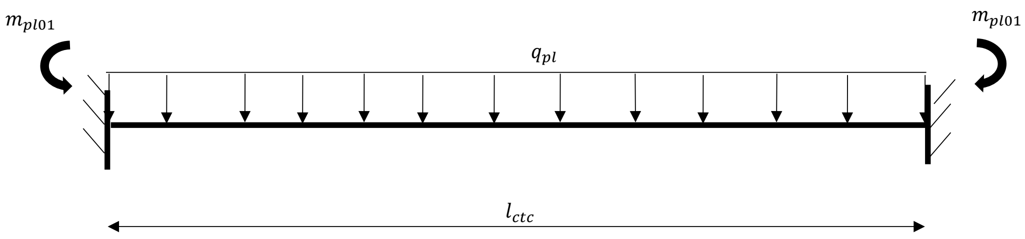
Mx -962,0 [kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	2,36	[m]
a	16,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	-47,0	[kN/m]
mpl01	-62,6	[kNm]

Measurements		
lctc	4,0	[m]

Torsion

Mxy,alt	34,0	[kNm]	(due to alternative load case)
Mxy,tot	34,0	[kNm]	

Suspension force

Qyy	-47,0	[kN]

Clamping moment

mxx,alt	25,9	[kNm]	(due to alternative load case)
mxx,tot	25,9	[kNm]	

5.2 Bridge A - 6.10b – 0,8d

Normal stresses

LC	type	Prestress		Bending moment		Total hor. normal stress
		P [kN]	σxx [N/mm2]	M [kNm]	σxx [N/mm2]	σxx [N/mm2]
1	self-weight			1106	-0,37	
2	ballast			306	-0,10	
3	Conc. Mobile Load			822	-0,27	
4	Cant. Mobile Load			-224	0,07	
5a	Contin. Mobile Load			609	-0,20	
5b	Contin. Mobile Load (SW/2)			1373	-0,46	
6	Support settelement max			0	0,00	
7	Support settelement min			0	0,00	
8	Prestress t=0	-13200	-6,14	-1055	0,35	
9	Prestress t = ∞	-12038	-5,60	-962	0,32	
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		-12038	-5,60	1881	-0,63	-6,22
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		-12038	-5,60	1881	-0,63	-6,22
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		-12038	-5,60	226	-0,08	-5,67
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		-12038	-5,60	226	-0,08	-5,67
LC 1 + LC 8		-13200	-6,14	51	-0,02	-6,15
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		-12038	-5,60	1822	-0,61	-6,21
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		-12038	-5,60	1822	-0,61	-6,21

LC	type	Suspension force		Clamping moment		Suspension force excen.		Total ver. Normal stress
		Qyy [kN]	σyy [N/mm2]	mxx [kNm]	σyy [N/mm2]	Myy [kNm]	σyy [N/mm2]	σyy [N/mm2]
1	self-weight	34	0,02	16	0,05	15	0,05	
2	ballast	29	0,01	18	0,05	13	0,04	
3	Conc. Mobile Load	149	0,07	-20	-0,06	67	0,21	
4	Cant. Mobile Load	0	0,00	61	0,19	0	0,00	
5a	Contin. Mobile Load	0	0,00	34	0,11	0	0,00	
5b	Contin. Mobile Load (SW/2)	107	0,05	47	0,15	48	0,15	
6	Support settelement max	-18	-0,01	98	0,31	-8	-0,03	
7	Support settelement min	18	0,01	-98	-0,31	8	0,03	
8	Prestress t=0	-52	-0,02	28	0,09	-23	-0,07	
9	Prestress t = ∞	-47	-0,02	26	0,08	-21	-0,07	
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		148	0,07	172	0,54	67	0,21	0,81
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		184	0,09	-25	-0,08	83	0,26	0,27
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		-1	0,00	218	0,68	-1	0,00	0,68
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		35	0,02	21	0,07	16	0,05	0,13
LC 1 + LC 8		-17	-0,01	44	0,14	-8	-0,02	0,11
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		105	0,05	205	0,64	47	0,15	0,84
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		141	0,07	8	0,03	64	0,20	0,29

Shear and principal stresses

LC	type	Shear force Vz [kN]	Shear force τxy [N/mm2]	Torsion Mxy [kNm]	Torsion τxy [N/mm2]	Total shear stress τxy [N/mm2]
1	self-weight	786	0,48	-93	-0,20	
2	ballast	267	0,16	-81	-0,17	
3	Conc. Mobile Load	558	0,34	-218	-0,46	
4	Cant. Mobile Load	0	0,00	-246	-0,52	
5a	Contin. Mobile Load	448	0,27	23	0,05	
5b	Contin. Mobile Load (SW/2)	976	0,60	-125	-0,26	
6	Support settelement max	0	0,00	-214	-0,45	
7	Support settelement min	0	0,00	214	0,45	
8	Prestress t=0	-923	-0,56	37	0,08	
9	Prestress t = ∞	-842	-0,51	34	0,07	
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		1217	0,74	-548	-1,16	1,90
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		1217	0,74	-121	-0,26	1,00
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		211	0,13	-600	-1,26	1,39
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		211	0,13	-173	-0,36	0,49
LC 1 + LC 8		-137	-0,08	-56	-0,12	0,20
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		1187	0,72	-479	-1,01	1,73
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		1187	0,72	-52	-0,11	0,83

LC	type	Total σxx [N/mm2]	Total σyy [N/mm2]	Total τxy [N/mm2]	Principal stress ρ1 [N/mm2]	Principal stress ρ2 [N/mm2]
1	self-weight					
2	ballast					
3	Conc. Mobile Load					
4	Cant. Mobile Load					
5a	Contin. Mobile Load					
5b	Contin. Mobile Load (SW/2)					
6	Support settelement max					
7	Support settelement min					
8	Prestress t=0					
9	Prestress t = ∞					
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		-6,22	0,81	1,90	1,29	-6,70
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		-6,22	0,27	1,00	0,42	-6,37
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		-5,67	0,68	1,39	0,97	-5,97
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		-5,67	0,13	0,49	0,17	-5,71
LC 1 + LC 8		-6,15	0,11	0,20	0,11	-6,16
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		-6,21	0,84	1,73	1,24	-6,61
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		-6,21	0,29	0,83	0,39	-6,31

Parameters

Bending stiffness coefficient		
E	34000	[N/mm2]
S_{pl}	2,36E+08	

Torsional stiffness coefficient		
v	0,20	
G	1,42E+04	[N/mm2]
$I_{t,girder}$	2,88E+11	[mm4]
GI_t	4,08E+15	[Nmm2]
ω	0,24	

Shear stress parameters		
I_{yy}	6,63E+11	[mm4]
S	3,65E+08	[mm]
b_{girder}	900	[mm]
A_M	1200000	[mm2]
t_{ef}	150	[mm]

Sectional properties		
b_{girder}	900	[mm]
h_{girder}	1750	[mm]
t_{floor}	550	[mm]
b_{floor}	4000	[mm]
$0{,}5 * A_{prestress}$	2150500	[mm2]
$0{,}5 * I_{yy}$	6,63E+11	[mm4]
y	221	[mm]
z	629	[mm]

Normal stress parameters		
A_1	810000	[mm2]
A_2	1115000,00	[mm2]
$A_1/(A_1+A_2)$	42,1%	
$A_{Q_{yy}}$	900000	[mm2]
$W_{m_{xx}}$	135000000	[mm3]
$eccentricity$	0,45	[m]

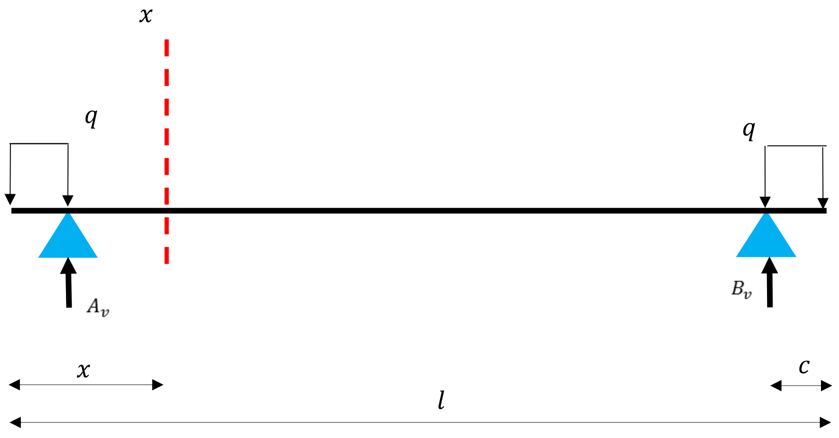
Torsional stiffness girder		
a	0,875	[m]
b	0,45	[m]
$\propto$	3,62	[-]
$I_{t,girder}$	0,29	[m4]

Torsional stiffness floor		
a	1,00	[m]
b	0,275	[m]
$\propto$	4,41	[-]
$I_{t,floor}$	0,09	[m4]
$I_{t,grider}/(I_{t,floor} + I_{t,girder})$	75,86%	[%]

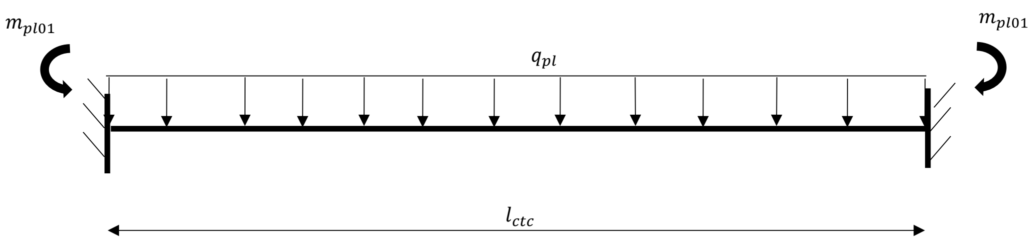
LC 1

Cantilevers loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
qbridge	172,0	[kN/m]
0,5q	86,0	[kN/m]
Av	86,0	[kN]
Bv	86,0	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	43,0	[kN/m]
mpl01	57,3	[kNm]

Measurements		
lctc	4,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -43,0 [kNm]

Torsion

Mxy,M	-41,1	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	-41,1	[kNm]	

Suspension force

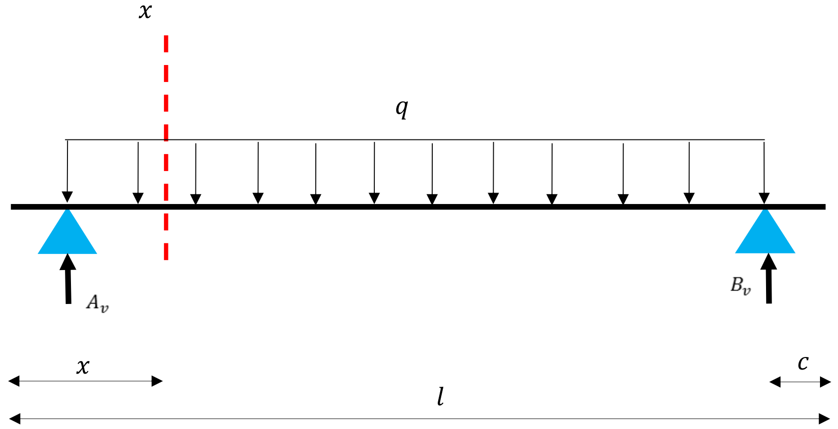
Qyy 0,0 [kN]

Clamping moment

mxx,M	10,1	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	10,1	[kNm]	

Midspan loaded

Loading (long. direction)



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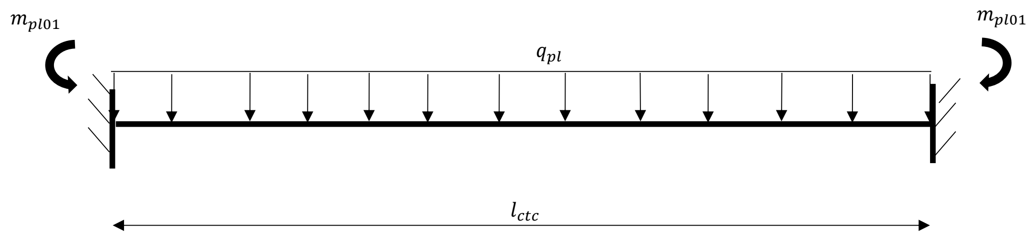


Load & Reaction forces		
qbridge	172,0	[kN/m]
0,5q	86,0	[kN/m]
Av	903,0	[kN]
Bv	903,0	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	43,0	[kN/m]
mpl02	57,3	[kNm]

Measurements		
lctc	4,0	[m]

Shear Force

Vz 786,0 [kN]

Bending moment

Mx 1148,5 [kNm]

Torsion

Mxy, alt	-52,0	[kNm]	(due to alternative load case)
Mxy,subtot	-52,0	[kNm]	

Suspension force

Qyy 34,4 [kN]

Clamping moment

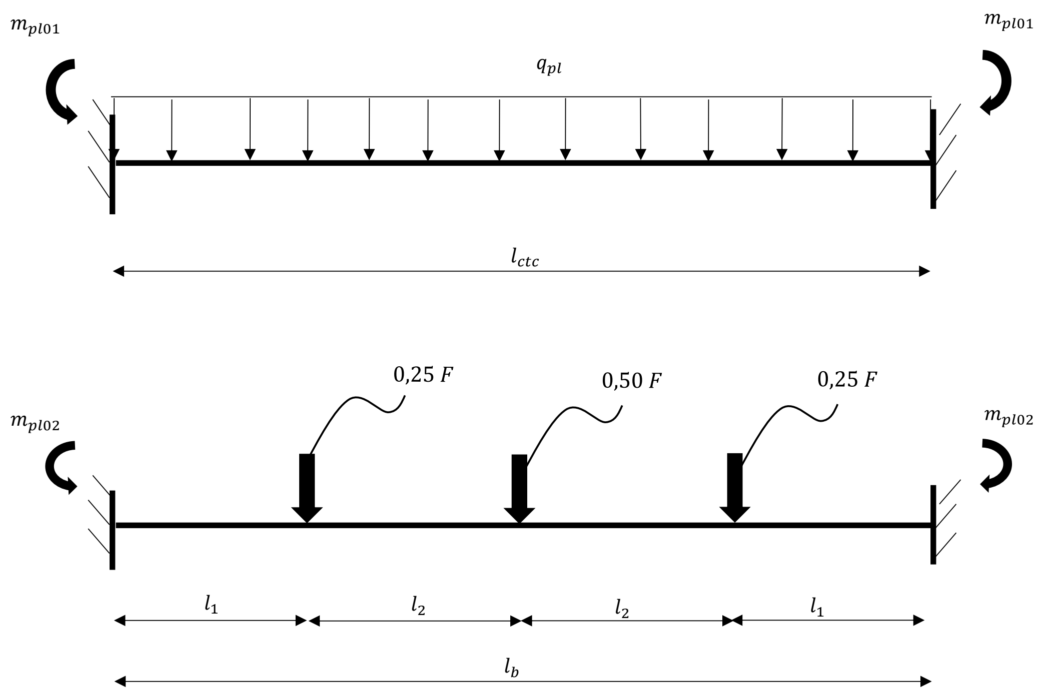
mxx, alt	5,7	[kNm]	(due to alternative load case)
mxx,subtot.	5,7	[kNm]	

LC 2

Cantilevers loaded

Loading (long. direction)

Loading (transverse direction)



Load & Reaction forces		
q	14,6	[kN/m2]
0,5q	29,2	[kN/m]
F	139	[kN]
0,5F	69,5	[kN]
Av	98,7	[kN]
Bv	98,7	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	14,6	[kN/m]
0,25F	34,8	[kN]
mpl01	19,5	[kNm]
mpl02	69,2	[kNm]
MT	88,6	[kNm]

Measurements		
lctc	4,0	[m]
l1	0,95	[m]
l2	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

Bending moment

Mx -84,1 [kNm]

Torsion

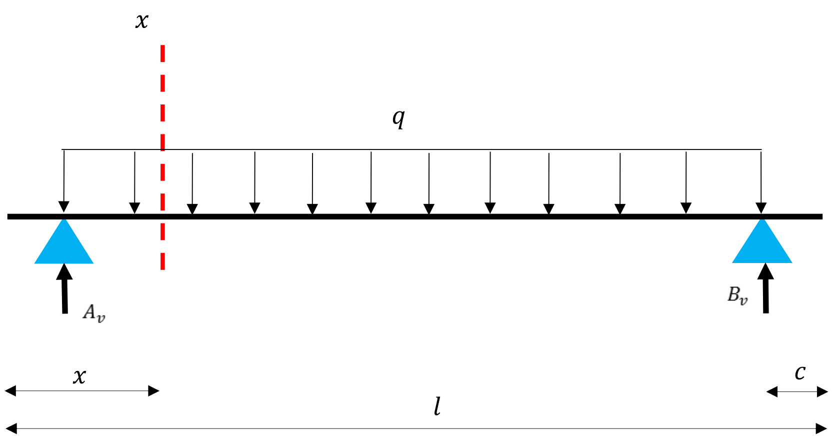
Mxy,M	-63,5	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	-63,5	[kNm]	

Clamping moment

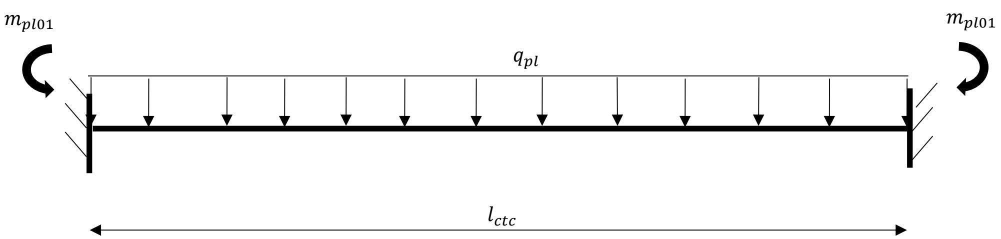
mxx,M	15,6	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	15,6	[kNm]	

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	14,6	[kN/m2]
0,5q	29,2	[kN/m]
Av	306,6	[kN]
Bv	306,6	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	14,6	[kN/m]
mpl01	19,5	[kNm]

Measurements		
lctc	4,0	[m]

Shear Force

Vz 266,9 [kN]

Suspension force

Qyy 29,2 [kN]

Bending moment

Mx	390,0	[kNm]

Torsion

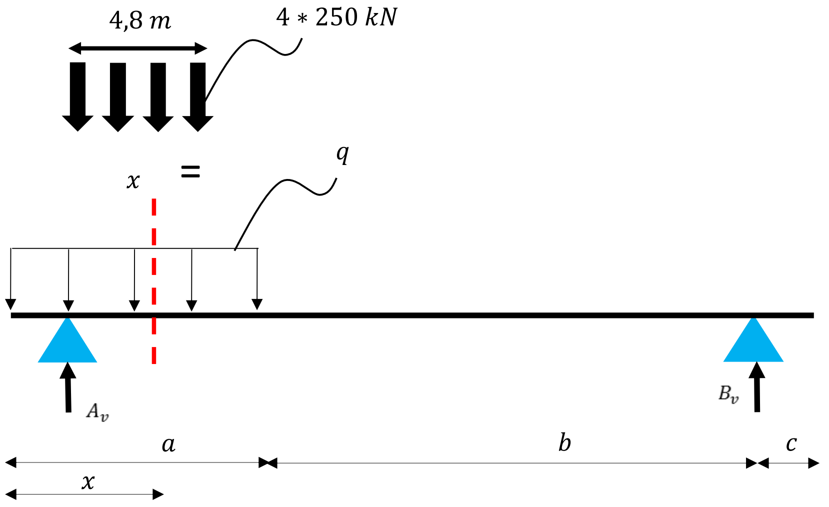
Mxy,alt	-17,7	[kNm]	(due to alternative load case)
Mxy,subtot	-17,7	[kNm]	

Clamping moment

mxx,alt	1,9	[kNm]	(due to alternative load case)
mxx,subtot.	1,9	[kNm]	

LC 3

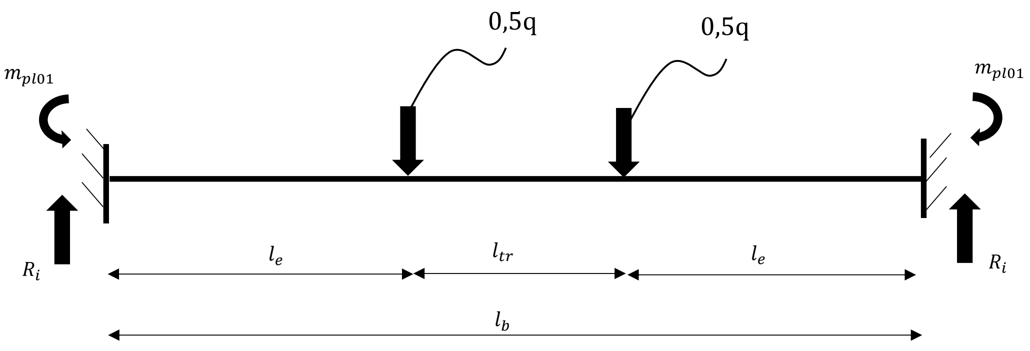
Loading (long. direction)



Load & Reaction forces		
q	298,8	[kN/m]
0,5q	149,4	[kN/m]
Av	910,6	[kN]
Bv	120,3	[kN]

Measurements		
x	2,36	[m]
a	6,9	[m]
b	15,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
p	149,4	[kN/m]
mpl01	243,6	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz	558,0	[kN]

Suspension force

Qyy	149,4	[kN]

Bending moment

Mx	822,4	[kNm]

Clamping moment

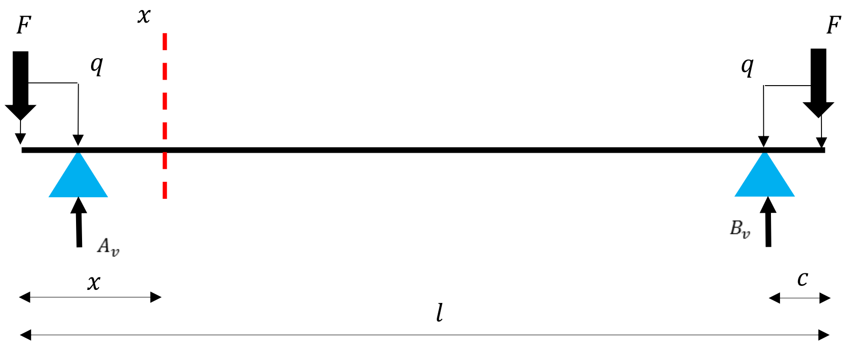
mxx,M	42,2	[kNm]	(due to torsional moment at one end)
mxx,q	-62,3	[kNm]	(due to distributed load)
mxx,tot	-20,1	[kNm]	

Torsion

Mxy,M	-175,8	[kNm]	(due to torsional moment at one end)
Mxy,q	-41,7	[kNm]	(due to distributed load)
Mxy,tot	-217,5	[kNm]	

LC 4

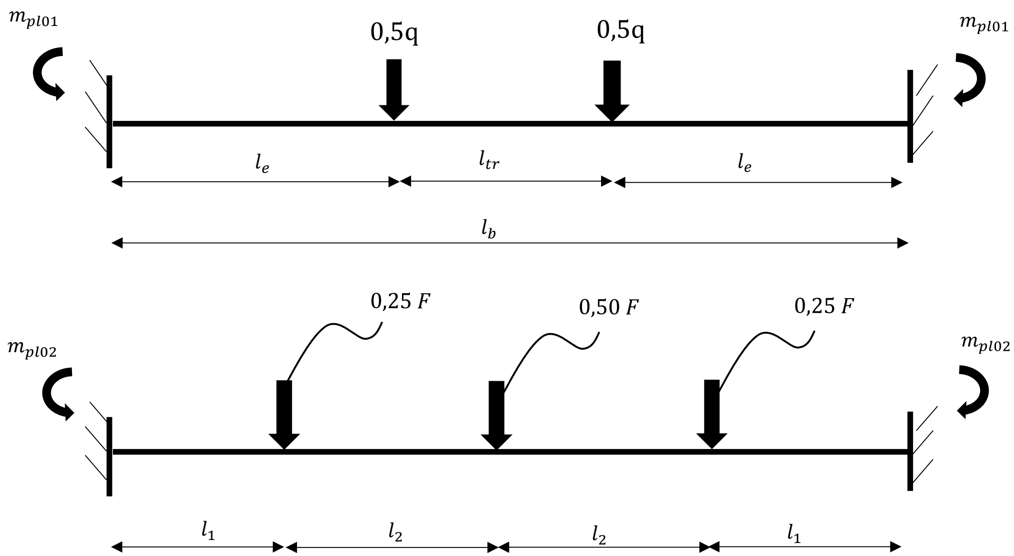
Loading (long. direction)



Load & Reaction forces		
q	213	[kN/m]
0,5q	106,5	[kN/m]
F	341	[kN]
0,5F	170,5	[kN]
Av	277,0	[kN]
Bv	277,0	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	106,5	[kN/m]
0,25F	85,3	[kN]
mpl01	173,7	[kNm]
mpl02	169,7	[kNm]
MT	343,4	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]
l1	0,95	[m]
l2	1,5	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

Bending moment

Mx -223,8 [kNm]

Clamping moment

mxx,M	60,6	[kNm]
mxx,tot	60,6	[kNm]

(due to torsional moment at both ends)

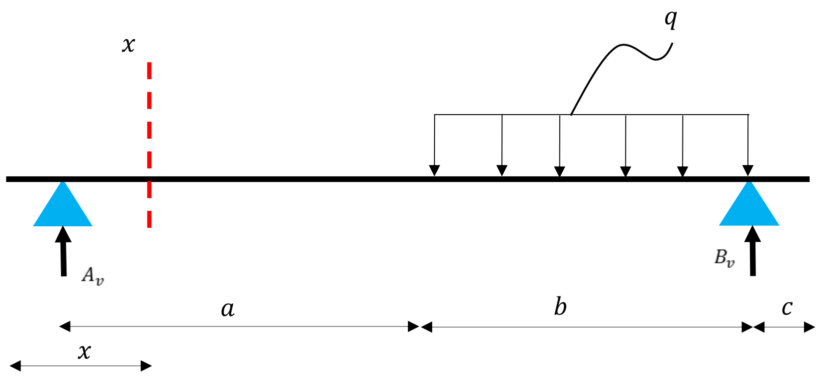
Torsion

Mxy,M	-246,3	[kNm]
Mxy,tot	-246,3	[kNm]

(due to torsional moment at both ends)

LC 5a

Loading (long. direction)



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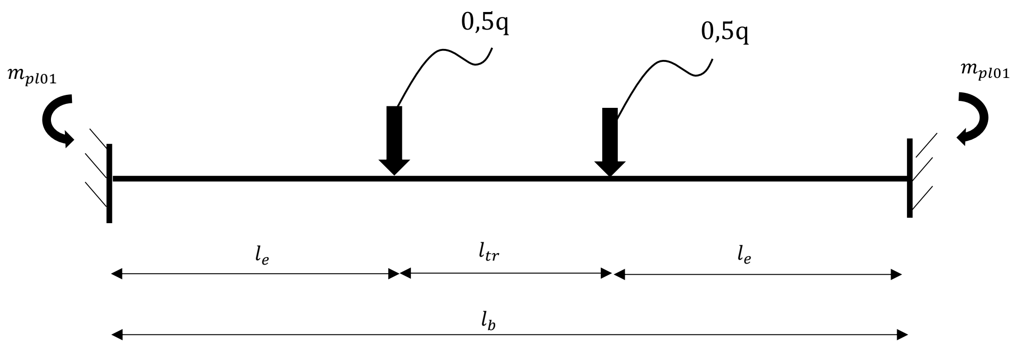


Load & Reaction forces		
q	165	[kN/m]
0,5q	82,5	[kN/m]
Av	447,9	[kN]
Bv	797,9	[kN]

Measurements		
x	2,36	[m]
a	5,9	[m]
b	15,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
0,5q	82,5	[kN/m]
mpl01	134,5	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 447,9 [kN]

Bending moment

Mx 609,1 [kNm]

Torsion

Mxy,q	23,0	[kNm]	(due to distributed load)
Mxy,tot	23,0	[kNm]	

Suspension force

Qyy 0,0 [kN]

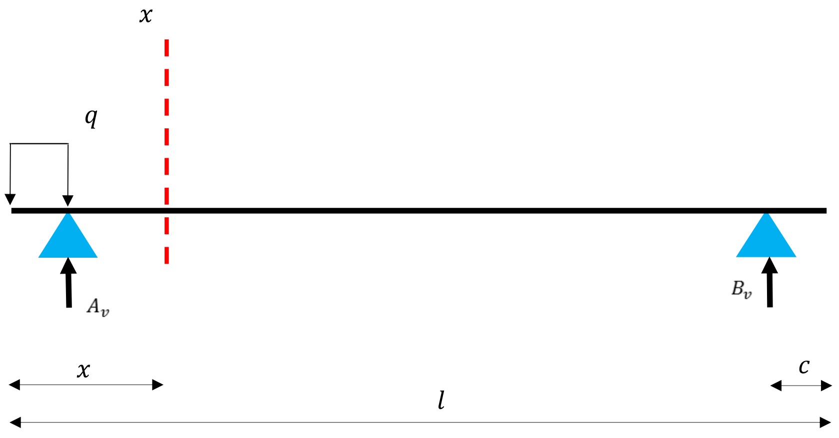
Clamping moment

mxx,q	34,4	[kNm]	(due to distributed load)
mxx,tot	34,4	[kNm]	

LC 5b

Cantilevers loaded

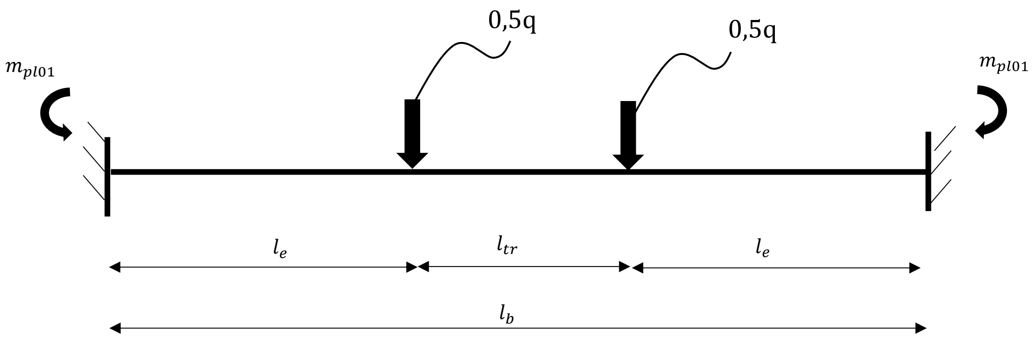
Loading (long. direction)



Load & Reaction forces		
q	213	[kN/m]
0,5q	106,5	[kN/m]
Av	109,0	[kN]
Bv	-2,5	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	106,5	[kN/m]
mpl01	173,7	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 2,5 [kN]

Bending moment

Mx -49,8 [kNm]

Torsion

Mxy,M	-125,3	[kNm]	(due to torsional moment at one end)
Mxy,subtot	-125,3	[kNm]	

Suspension force

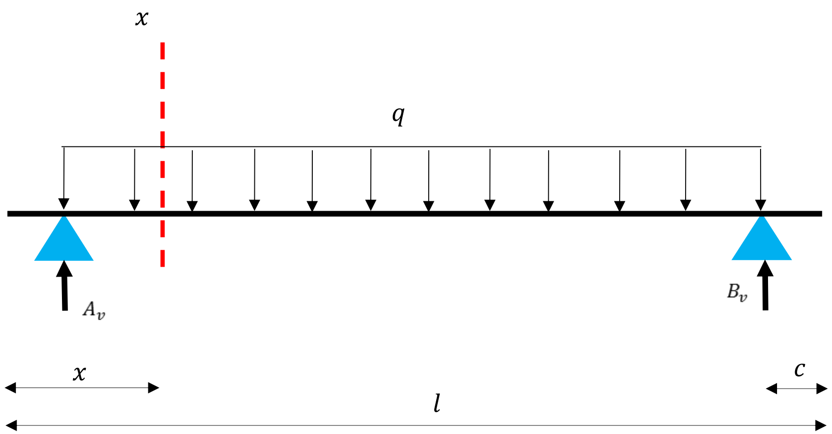
Qyy 0,0 [kN]

Clamping moment

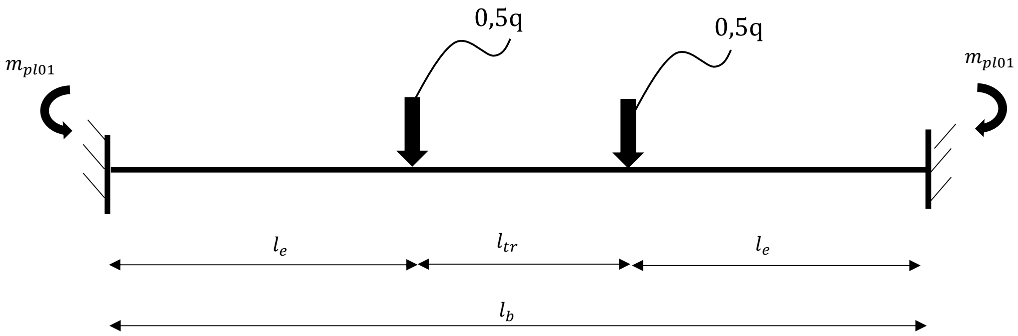
mxx,M	30,1	[kNm]	(due to torsional moment at one end)
mxx,subtot.	30,1	[kNm]	

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	213	[kN/m]
0,5q	106,5	[kN/m]
Av	1118,3	[kN]
Bv	1118,3	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
0,5q	106,5	[kN/m]
mpl01	173,7	[kNm]

Measurements		
lb	4,9	[m]
ltr	1,5	[m]
le	1,7	[m]

Shear Force

Vz 973,4 [kN]

Bending moment

Mx 1422,3 [kNm]

Torsion

Mxy, alt 0,0 [kNm] (due to alternative load case)

Mxy,subtot 0,0 [kNm]

Suspension force

Qyy 106,5 [kN]

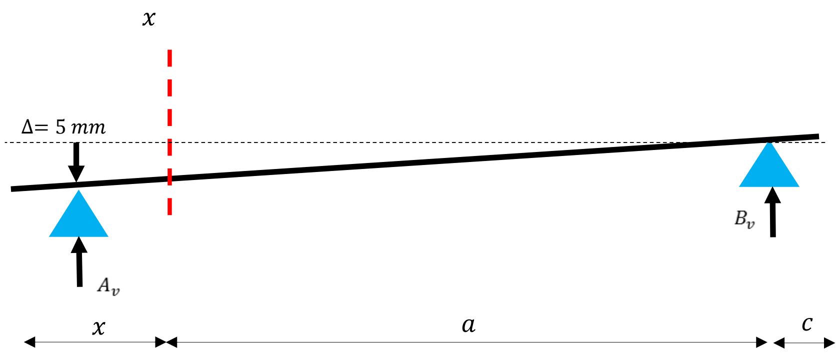
Clamping moment

mxx, alt 17,2 [kNm] (due to alternative load case)

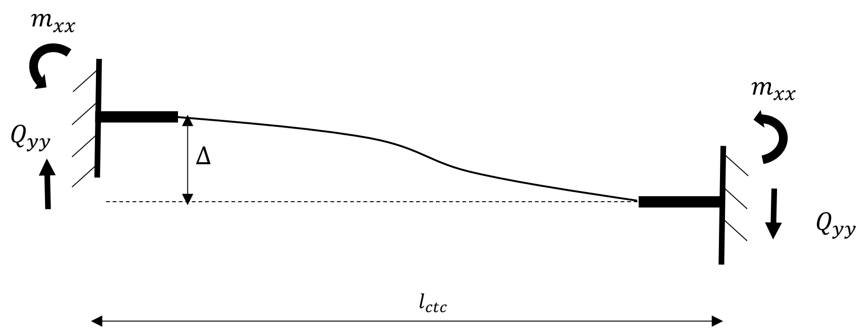
mxx,subtot. 17,2 [kNm]

LC 6

Loading (long. direction)



Loading (transverse direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γ	1,2	[-]

Measurements		
x	2,36	[m]
a	19,6	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Shear Force

Vz	0,0	[kN]

Bending moment

Mx	0,0	[kNm]

Torsion

Mxy,Δ	-178,0	[kNm]
Mxy,tot	-213,6	[kNm]

Suspension force

Qyy	-15,0	[kN]
Qyy,tot	-18,0	[kN]

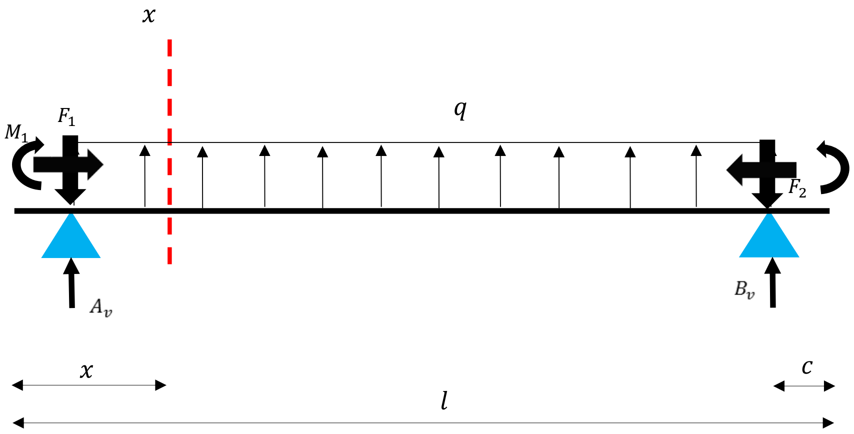
Clamping moment

mxx,Δ	82,0	[kNm]
mxx,tot	98,4	[kNm]

LC 8

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	1115	[kN]
F2	-13200	[kN]
M1	294	[kNm]
q	-101	[kN]
Av	55	[kN]
Bv	55	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Shear Force

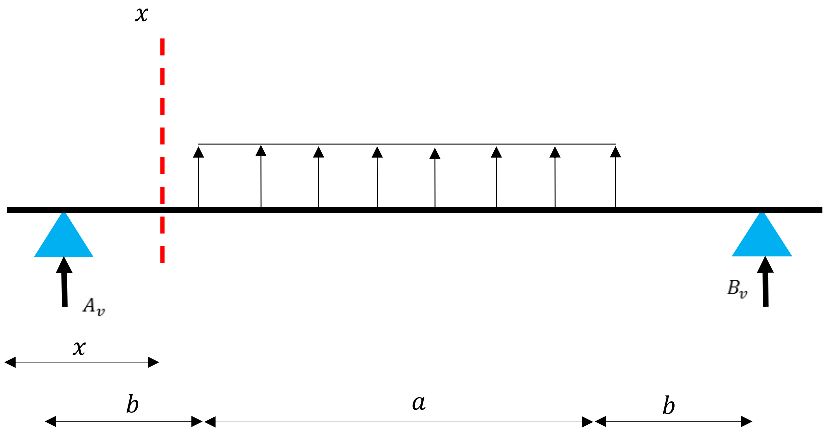
Vz	-923,1	[kN]

Bending moment

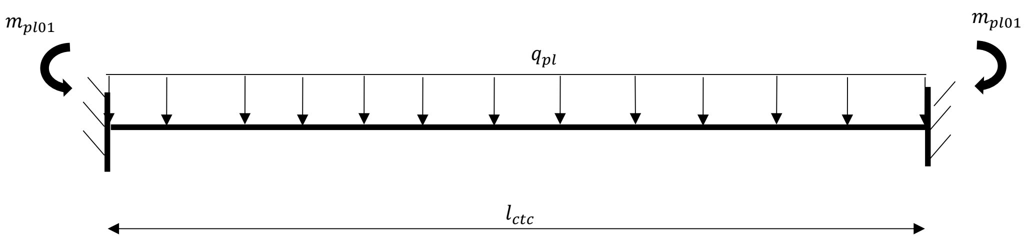
Mx	-1054,9	[kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	2,36	[m]
a	16,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	-51,5	[kN/m]
mpl01	-68,7	[kNm]

Measurements		
lctc	4,0	[m]

Torsion

Mxy,alt	37,3	[kNm]	(due to alternative load case)
Mxy,tot	37,3	[kNm]	

Suspension force

Qyy	-51,5	[kN]

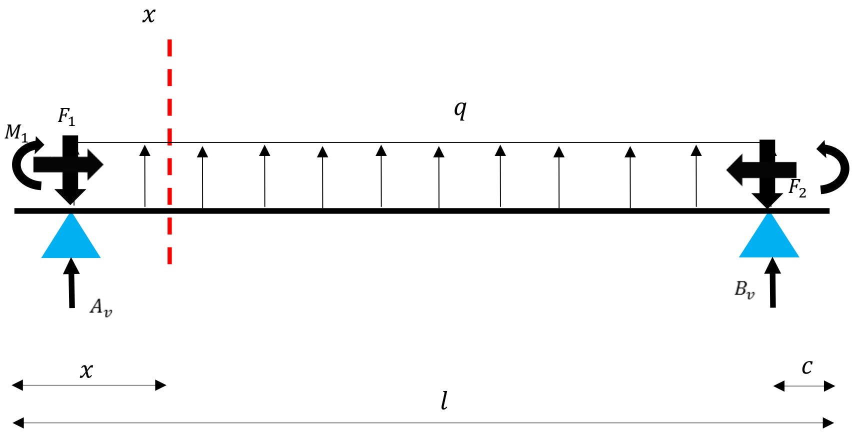
Clamping moment

mxx,alt	28,4	[kNm]	(due to alternative load case)
mxx,tot	28,4	[kNm]	

LC 9

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	1017	[kN]
F2	-12038	[kN]
M1	268	[kNm]
q	-92	[kN]
Av	50	[kN]
Bv	50	[kN]
P∞/P0	0,912	[-]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Shear Force

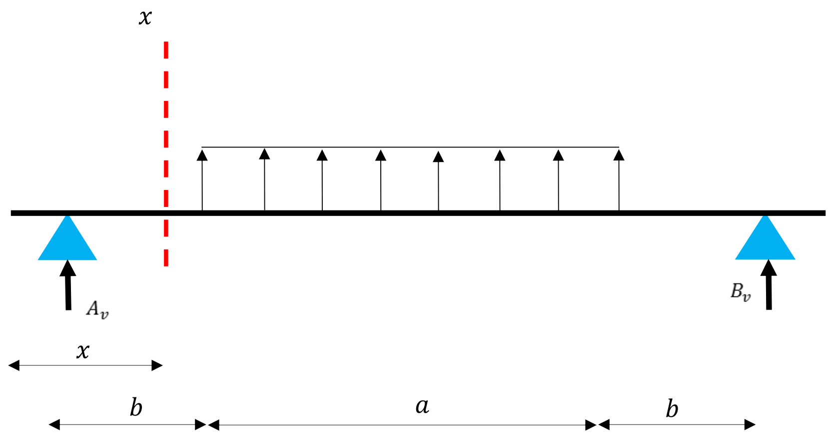
Vz	-841,9	[kN]

Bending moment

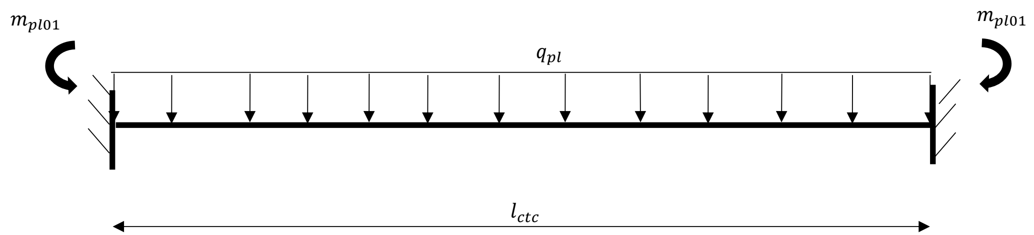
Mx	-962,0	[kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	2,36	[m]
a	16,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	-47,0	[kN/m]
mpl01	-62,6	[kNm]

Measurements		
lctc	4,0	[m]

Torsion

Mxy,alt	34,0	[kNm]
Mxy,tot	34,0	[kNm]

(due to alternative load case)

Suspension force

Qyy	-47,0	[kN]

Clamping moment

mxx,alt	25,9	[kNm]
mxx,tot	25,9	[kNm]

(due to alternative load case)

5.3 Bridge B - 6.10a – support

Normal stresses

LC	type	Prestress P [kN]	Prestress σxx [N/mm2]	Bending moment M [kNm]	Bending moment σxx [N/mm2]	Total hor. normal stress σxx [N/mm2]
1	self-weight			-76	0,01	
2	ballast			-87	0,01	
3a	Mobile Max. (LM71)			-57	0,00	
3b	Mobile Max. (SW/2)			-40	0,00	
4	Mobile Min. (SW/2)			-202	0,02	
5	Support settelement max			0	0,00	
6	Support settelement min			0	0,00	
7	Prestress t=0	-22826	-8,65	5045	-0,41	
8	Prestress t = ∞	-20886	-7,91	4616	-0,38	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-20886	-7,91	4397	-0,36	-8,27
LC 1 + LC 2 + LC 3a + LC 6 + LC8		-20886	-7,91	4397	-0,36	-8,27
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-20886	-7,91	4252	-0,35	-8,26
LC 1 + LC 2 + LC 4 + LC 6 + LC8		-20886	-7,91	4252	-0,35	-8,26
LC 1 + LC 7		-22826	-8,65	4970	-0,41	-9,05
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-20886	-7,91	4413	-0,36	-8,27
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		-20886	-7,91	4413	-0,36	-8,27

LC	type	Suspension force Qyy [kN]	Suspension force σyy [N/mm2]	Clamping moment mxx [kNm]	Clamping moment σyy [N/mm2]	Suspension force excen. Myy [kNm]	Suspension force excen. σyy [N/mm2]	Total ver. Normal stress σyy [N/mm2]
1	self-weight	59	0,02	33	0,07	35	0,07	
2	ballast	35	0,01	21	0,04	21	0,04	
3a	Mobile Max. (LM71)	113	0,05	-3	-0,01	68	0,14	
3b	Mobile Max. (SW/2)	81	0,03	23	0,05	48	0,10	
4	Mobile Min. (SW/2)	81	0,03	59	0,12	48	0,10	
5	Support settelement max	-265	-0,11	227	0,47	-159	-0,33	
6	Support settelement min	265	0,11	-227	-0,47	159	0,33	
7	Prestress t=0	-57	-0,02	28	0,06	-34	-0,07	
8	Prestress t = ∞	-52	-0,02	26	0,05	-31	-0,06	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-110	-0,05	303	0,62	-66	-0,14	0,44
LC 1 + LC 2 + LC 3a + LC 6 + LC8		420	0,17	-150	-0,31	252	0,52	0,38
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-143	-0,06	365	0,75	-86	-0,18	0,52
LC 1 + LC 2 + LC 4 + LC 6 + LC8		388	0,16	-88	-0,18	233	0,48	0,46
LC 1 + LC 7		2	0,00	61	0,13	1	0,00	0,13
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-143	-0,06	330	0,68	-86	-0,18	0,44
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		388	0,16	-124	-0,25	233	0,48	0,38

Shear and principal stresses

LC	type	Shear force Vz [kN]	Shear force τxy [N/mm2]	Torsion Mxy [kNm]	Torsion τxy [N/mm2]	Total shear stress τxy [N/mm2]
1	self-weight	2378	1,08	-322	-0,33	
2	ballast	547	0,25	-160	-0,16	
3a	Mobile Max. (LM71)	1258	0,57	-230	-0,23	
3b	Mobile Max. (SW/2)	1202	0,54	-164	-0,17	
4	Mobile Min. (SW/2)	0	0,00	-362	-0,37	
5	Support settelement max	0	0,00	-589	-0,60	
6	Support settelement min	0	0,00	589	0,60	
7	Prestress t=0	-2287	-1,03	0	0,00	
8	Prestress t = ∞	-2093	-0,95	0	0,00	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		2091	0,95	-1301	-1,32	2,27
LC 1 + LC 2 + LC 3a + LC 6 + LC8		2091	0,95	-123	-0,12	1,07
LC 1 + LC 2 + LC 4 + LC 5 + LC8		833	0,38	-1433	-1,46	1,83
LC 1 + LC 2 + LC 4 + LC 6 + LC8		833	0,38	-255	-0,26	0,64
LC 1 + LC 7		91	0,04	-322	-0,33	0,37
LC 1 + LC 2 + LC 3b + LC 5 + LC8		2035	0,92	-1235	-1,26	2,18
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		2035	0,92	-57	-0,06	0,98

LC	type	Total σxx [N/mm2]	Total σyy [N/mm2]	Total τxy [N/mm2]	Principal stress ρ1 [N/mm2]	Principal stress ρ2 [N/mm2]
1	self-weight					
2	ballast					
3a	Mobile Max. (LM71)					
3b	Mobile Max. (SW/2)					
4	Mobile Min. (SW/2)					
5	Support settelement max					
6	Support settelement min					
7	Prestress t=0					
8	Prestress t = ∞					
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-8,27	0,44	2,27	1,00	-8,83
LC 1 + LC 2 + LC 3a + LC 6 + LC8		-8,27	0,38	1,07	0,51	-8,40
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-8,26	0,52	1,83	0,88	-8,63
LC 1 + LC 2 + LC 4 + LC 6 + LC8		-8,26	0,46	0,64	0,50	-8,31
LC 1 + LC 7		-9,05	0,13	0,37	0,14	-9,07
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-8,27	0,44	2,18	0,96	-8,79
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		-8,27	0,38	0,98	0,49	-8,38

Parameters

Bending stiffness coefficient

E	34000	[N/mm2]
S_{pl}	3,11E+08	

Torsional stiffness coefficient

v	0,20	
G	1,42E+04	[N/mm2]
$I_{t,girder}$	8,35E+11	[mm4]
GI_t	1,18E+16	[Nmm2]
ω	0,16	

Shear stress parameters

I_{yy}	1,73E+12	[mm4]
S	9,38E+08	[mm]
b_{girder}	1200	[mm]
A_M	2000000	[mm2]
t_{ef}	200	[mm]

Sectional properties

b_{girder}	1200	[mm]
h_{girder}	2200	[mm]
t_{floor}	650	[mm]
b_{floor}	5000	[mm]
$0,5 * A_{prestress}$	2640000	[mm2]
$0,5 * I_{yy}$	1,73E+12	[mm4]
y	142	[mm]
z	808	[mm]

Normal stress parameters

A_1	1500000	[mm2]
A_2	1540000,00	[mm2]
$A_1/(A_1+A_2)$	49,3%	
$A_{Q_{yy}}$	1200000	[mm2]
$W_{m_{xx}}$	240000000	[mm3]
$eccentricity$	0,6	[m]

Torsional stiffness girder

a	1,1	[m]
b	0,6	[m]
$\propto$	3,51	[-]
$I_{t,girder}$	0,83	[m4]

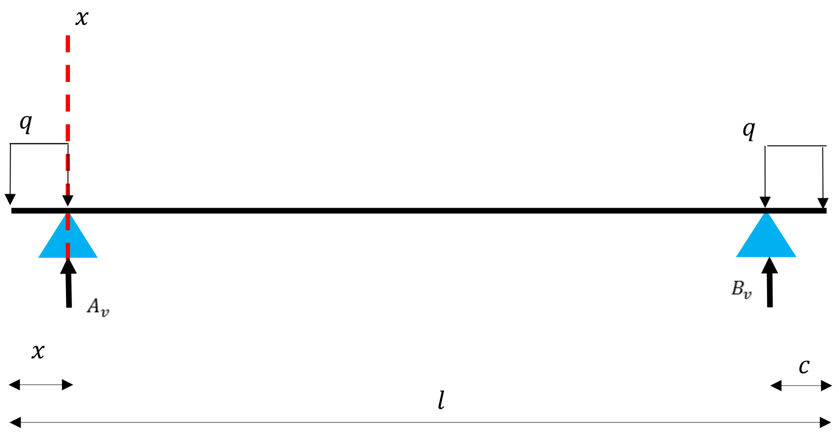
Torsional stiffness floor

a	1,25	[m]
b	0,325	[m]
$\propto$	4,46	[-]
$I_{t,floor}$	0,19	[m4]
$I_{t,grider}/(I_{t,floor} + I_{t,girder})$	81,35%	[%]

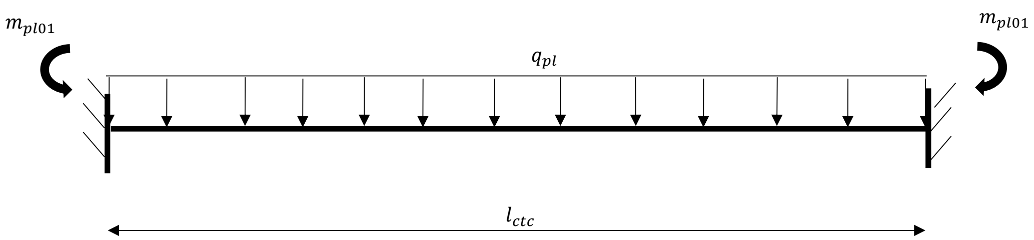
LC 1

Cantilevers loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
qbridge	302,0	[kN/m]
0,5q	151,0	[kN/m]
Av	151,0	[kN]
Bv	151,0	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	60,4	[kN/m]
mpl01	125,8	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -75,5 [kNm]

Torsion

Mxy,M	-125,8	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	-125,8	[kNm]	

Suspension force

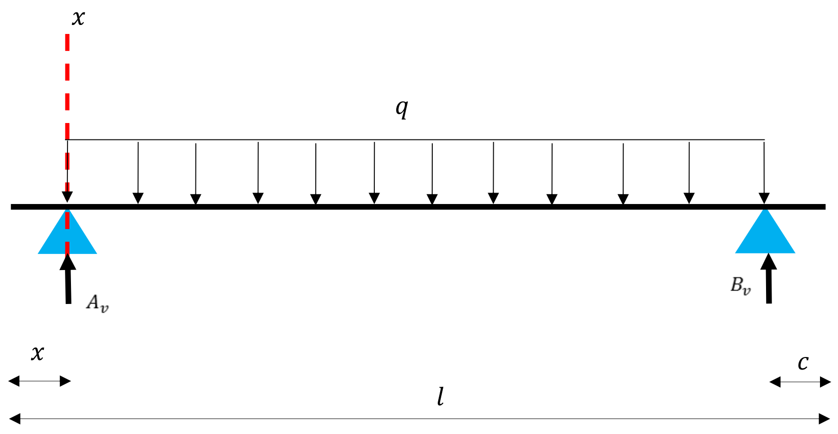
Qyy 0,0 [kN]

Clamping moment

mxx,M	20,4	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	20,4	[kNm]	

Midspan loaded

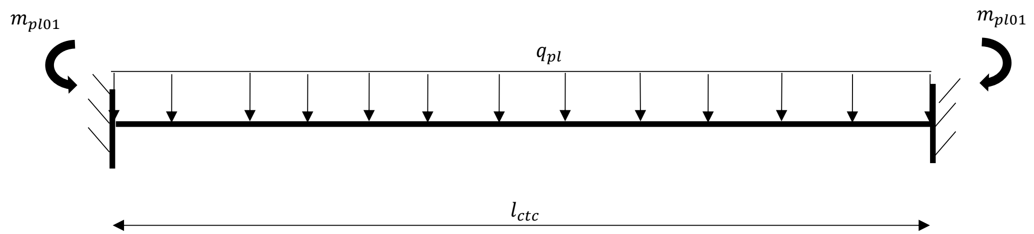
Loading (long. direction)



Load & Reaction forces		
qbridge	302,0	[kN/m]
0,5q	151,0	[kN/m]
Av	2378,3	[kN]
Bv	2378,3	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	60,4	[kN/m]
mpl02	125,8	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz 2378,3 [kN]

Bending moment

Mx 0,0 [kNm]

Torsion

Mxy, alt	-196,6	[kNm]	(due to alternative load case)
Mxy,subtot	-196,6	[kNm]	

Suspension force

Qyy 59,1 [kN]

Clamping moment

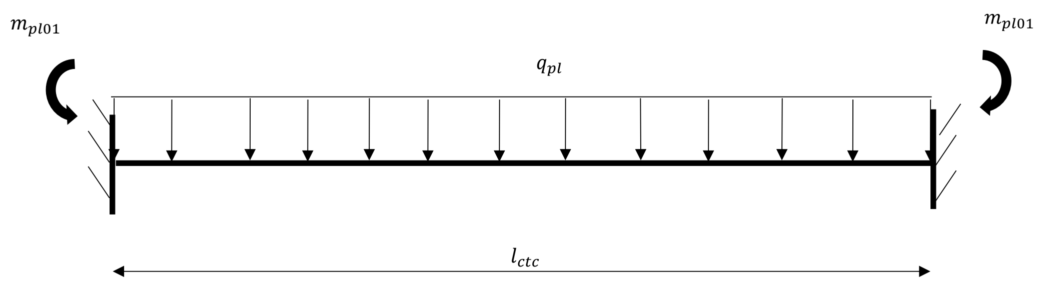
mxx, alt	12,5	[kNm]	(due to alternative load case)
mxx,subtot.	12,5	[kNm]	

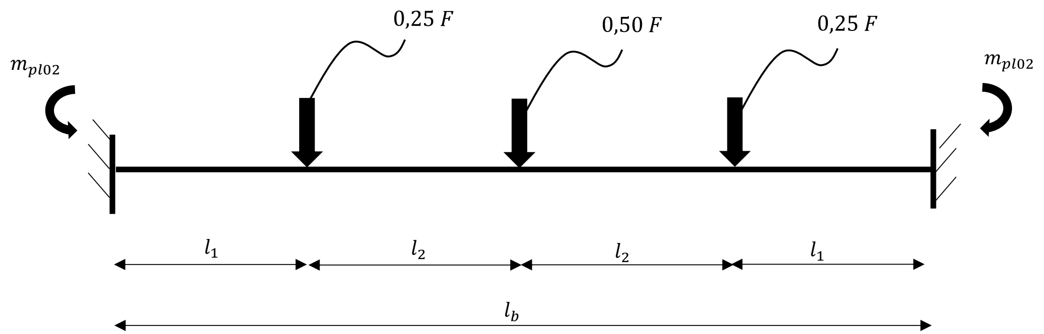
LC 2

Cantilevers loaded

Loading (long. direction)

Loading (transverse direction)





Load & Reaction forces

q	13,9	[kN/m2]
0,5q	34,8	[kN/m]
F	139	[kN]
0,5F	69,5	[kN]
Av	104,3	[kN]
Bv	104,3	[kN]

Measurements

x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces

qpl	13,9	[kN/m]
0,25F	34,8	[kN]
mpl01	29,0	[kNm]
mpl02	85,3	[kNm]
MT	114,3	[kNm]

Measurements

lctc	5,0	[m]
l1	1,1	[m]
l2	2,0	[m]
lb	6,2	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

Bending moment

Mx	-86,9	[kNm]

Torsion

Mxy,M	-114,3	[kNm]
Mxy,subtot	-114,3	[kNm]

(due to torsional moment at both ends)

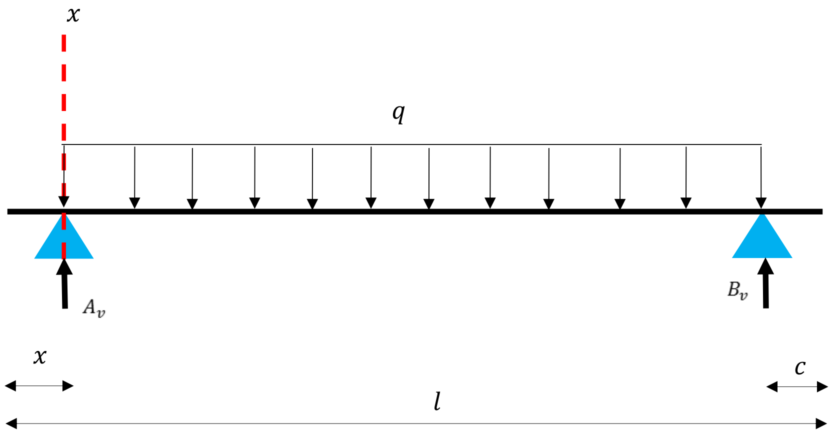
Clamping moment

mxx,M	18,5	[kNm]
mxx,subtot.	18,5	[kNm]

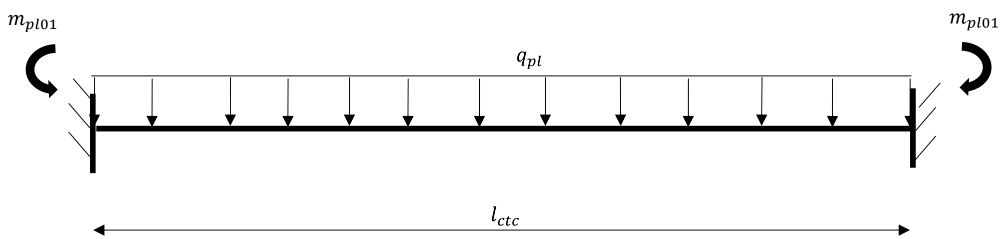
(due to torsional moment at both ends)

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	13,9	[kN/m2]
0,5q	34,8	[kN/m]
Av	547,3	[kN]
Bv	547,3	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	13,9	[kN/m]
mpl01	29,0	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz	547,3	[kN]

Suspension force

Qyy	34,8	[kN]

Bending moment

Mx	0,0	[kNm]

Torsion

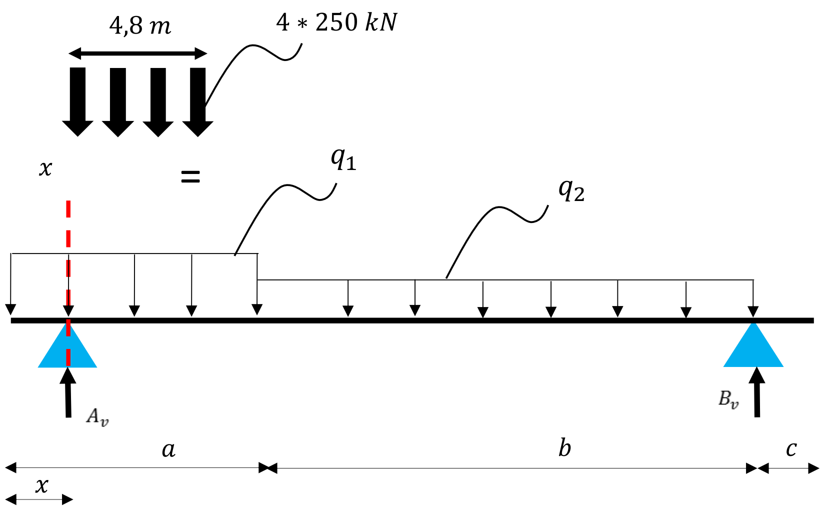
Mxy,alt	-45,3	[kNm]	(due to alternative load case)
Mxy,subtot	-45,3	[kNm]	

Clamping moment

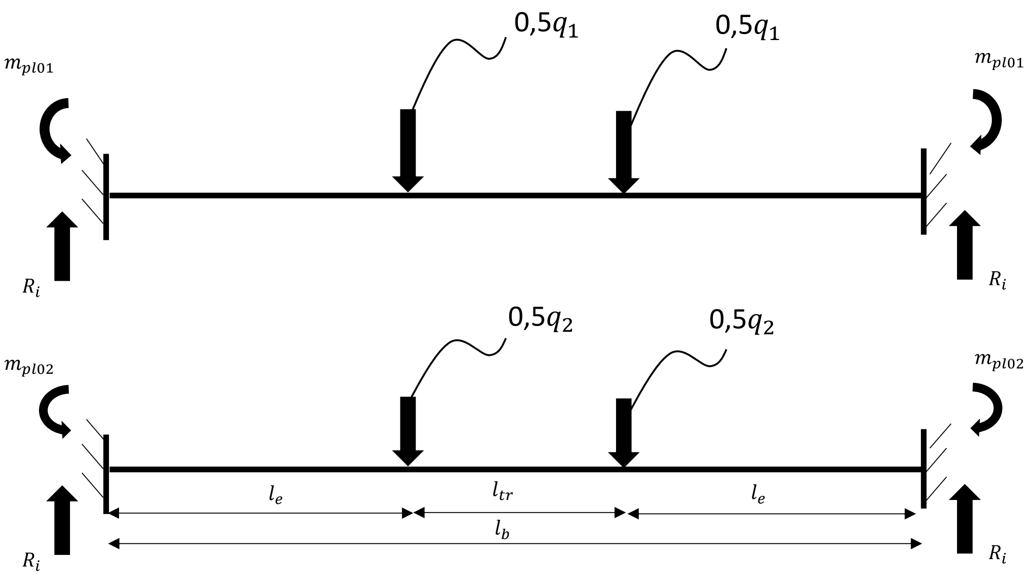
mxx,alt	2,9	[kNm]	(due to alternative load case)
mxx,subtot.	2,9	[kNm]	

LC 3a

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q1	226,6	[kN/m]
0,5q1	113,3	[kN/m]
q2	125,1	[kN/m]
0,5q2	62,55	[kN/m]
Av	1371,6	[kN]
Bv	1011,4	[kN]

Measurements		
x	1,00	[m]
a	6,9	[m]
b	25,6	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
0,5q1	113,3	[kN/m]
0,5q2	62,55	[kN/m]
mpl01	229,8	[kNm]
mpl02	126,8	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]

Shear Force

Vz	1258,3	[kN]

Bending moment

Mx	-56,7	[kNm]

Torsion

Mxy,M	-229,8	[kNm]	(due to torsional moment at one end)
Mxy,q1	0	[kNm]	(due to distributed load)
Mxx,q2	0	[kNm]	(due to distributed load)
Mxy,tot	-229,8	[kNm]	

Suspension force

Qyy	113,3	[kN]

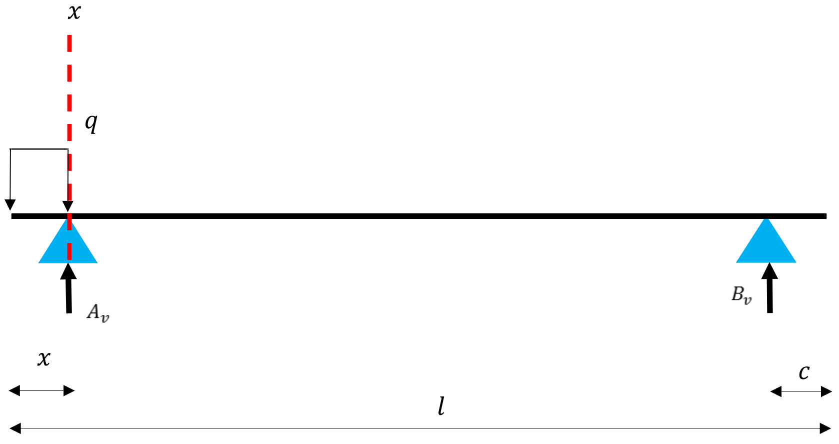
Clamping moment

mxx,M	36,8	[kNm]	(due to torsional moment at one end)
mxx,q1	-89,4	[kNm]	(due to distributed load)
mxx,q2	49,3	[kNm]	(due to distributed load)
mxx,tot	-3,3	[kNm]	

LC 3b

Cantilevers loaded

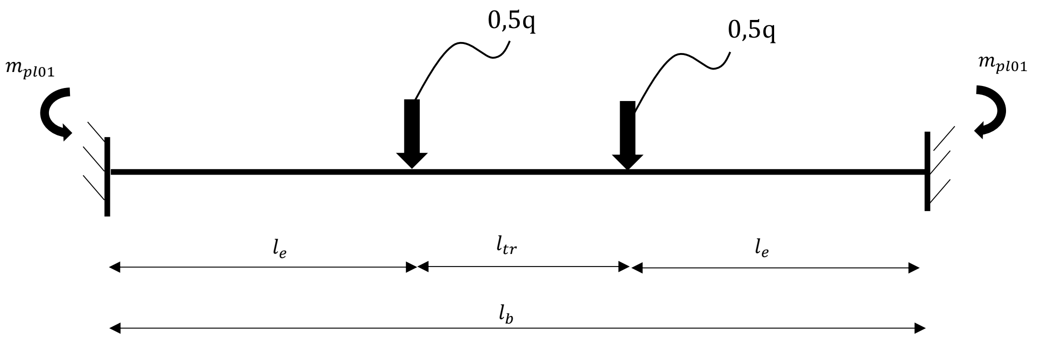
Loading (long. direction)



Load & Reaction forces		
q	161,6	[kN/m]
0,5q	80,8	[kN/m]
Av	82,1	[kN]
Bv	-1,3	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	80,8	[kN/m]
mpl01	163,8	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]

Shear Force

Vz	1,3	[kN]

Bending moment

Mx	-40,4	[kNm]

Torsion

Mxy,M	-163,8	[kNm]	(due to torsional moment at one end)
Mxy,subtot	-163,8	[kNm]	

Suspension force

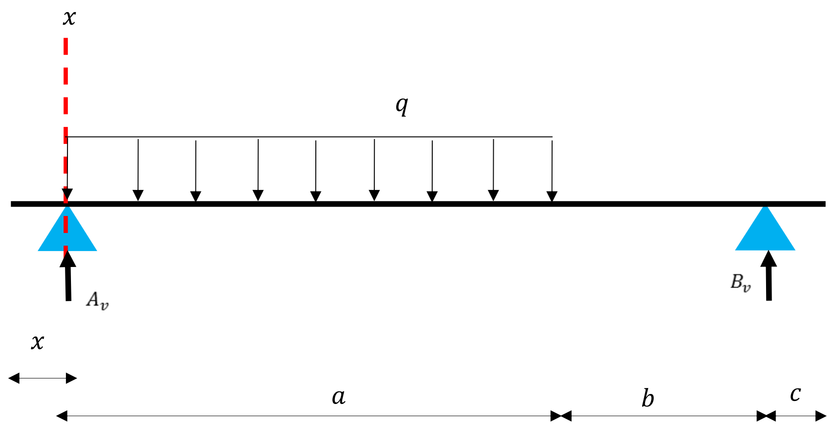
Qyy	0,0	[kN]

Clamping moment

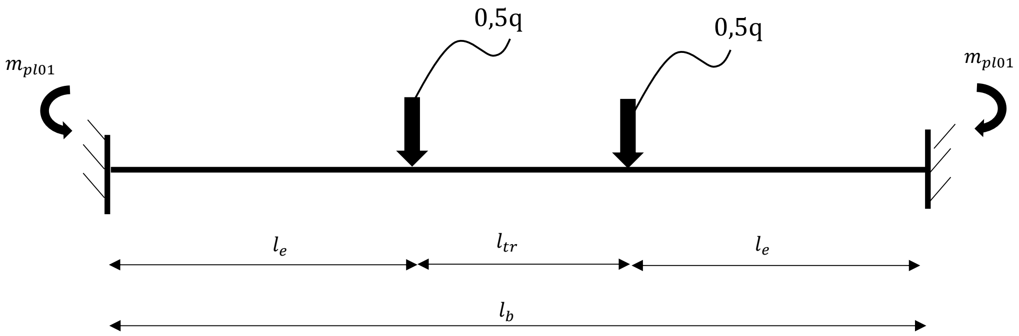
mxx,M	26,2	[kNm]	(due to torsional moment at one end)
mxx,subtot.	26,2	[kNm]	

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	161,6	[kN/m]
0,5q	80,8	[kN/m]
Av	1200,5	[kN]
Bv	738,7	[kN]

Measurements		
x	1,00	[m]
a	24	[m]
b	7,5	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
0,5q	80,8	[kN/m]
mpl01	163,8	[kNm]

Measurements		
lb	6,2	[m]
ltr	1,5	[m]
le	2,4	[m]

Shear Force

Vz 1200,5 [kN]

Bending moment

Mx 0,0 [kNm]

Torsion

Mxy, alt	0,0	[kNm]	(due to distributed load)
Mxy,subtot	0,0	[kNm]	

Suspension force

Qyy 80,8 [kN]

Clamping moment

mxx, alt	-3,2	[kNm]	(due to distributed load)
mxx,subtot.	-3,2	[kNm]	

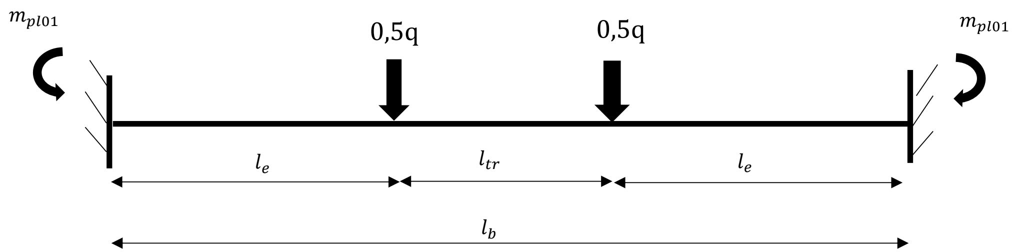
LC 4

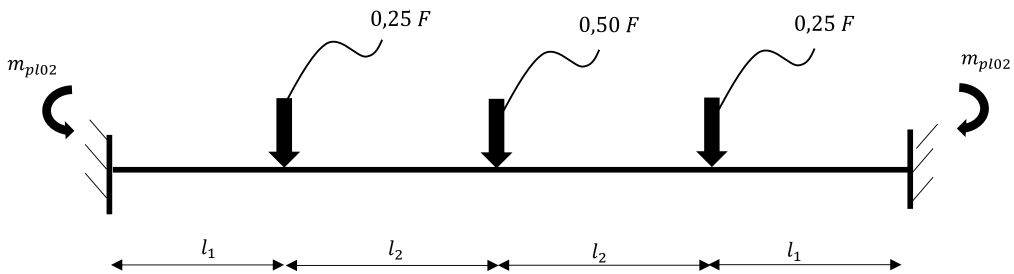
Loading (long. direction)

Load & Reaction forces		
q	161,6	[kN/m]
0,5q	80,8	[kN/m]
F	323	[kN]
0,5F	161,5	[kN]
Av	242,3	[kN]
Bv	242,3	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)





Load & Reaction forces		
0,5q	80,8	[kN/m]
0,25F	80,8	[kN]
mpl01	163,8	[kNm]
mpl02	198,2	[kNm]
MT	362,1	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]
l1	1,1	[m]
l2	2,0	[m]

Shear Force

Vz	0,0	[kN]

Suspension force

Qyy	80,8	[kN]

Bending moment

Mx	-201,9	[kNm]

Clamping moment

mxx,M	58,7	[kNm]
mxx,tot	58,7	[kNm]

(due to torsional moment at both ends)

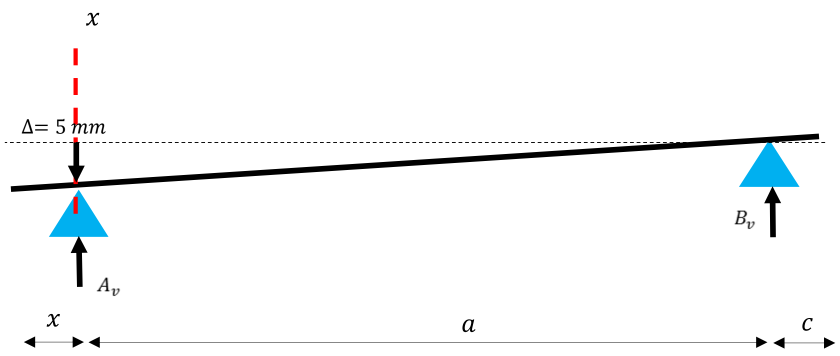
Torsion

Mxy,M	-362,1	[kNm]
Mxy,tot	-362,1	[kNm]

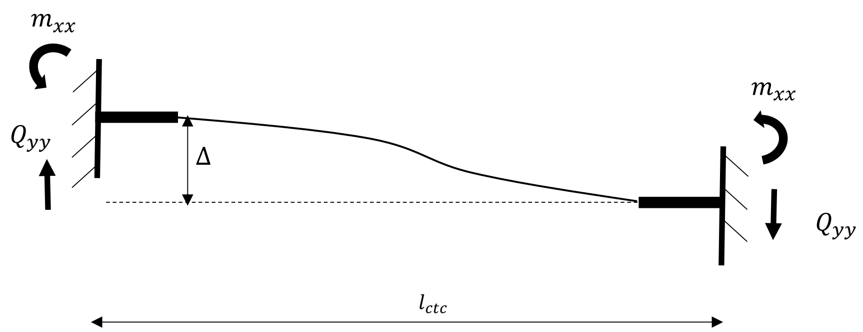
(due to torsional moment at both ends)

LC 6

Loading (long. direction)



Loading (transverse direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γ	1,2	[-]

Measurements		
x	1,00	[m]
a	31,5	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

Vz	0,0	[kN]

Bending moment

Mx	0,0	[kNm]

Torsion

Mxy,Δ	-491,0	[kNm]
Mxy,tot	-589,2	[kNm]

Suspension force

Qyy	-221,0	[kN]
Qyy,tot	-265,2	[kN]

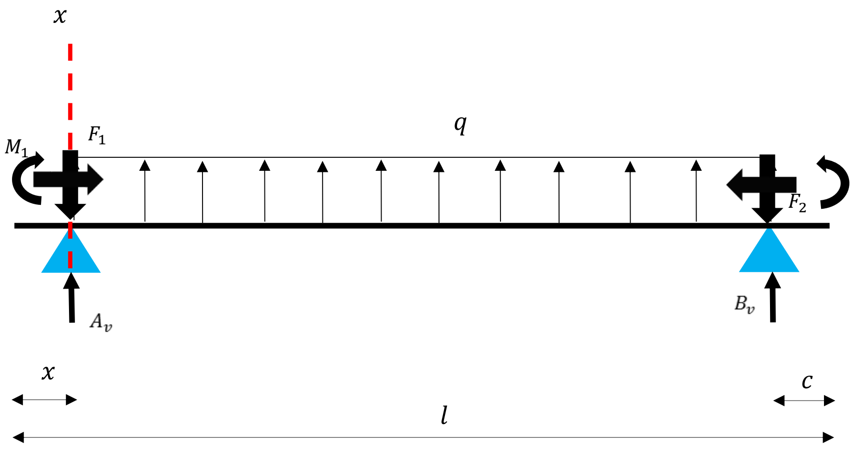
Clamping moment

mxx,Δ	189,0	[kNm]
mxx,tot	226,8	[kNm]

LC 8

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	2290	[kN]
F2	-22826	[kN]
M1	5045	[kNm]
q	-145,2	[kN]
Av	3	[kN]
Bv	3	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

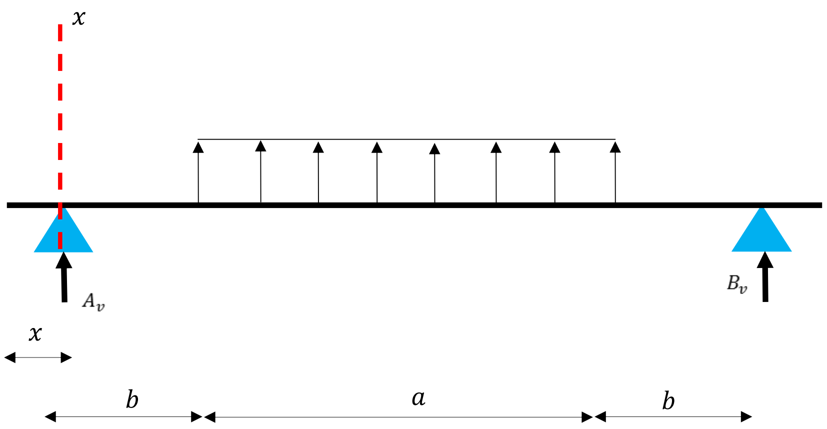
Vz -2286,9 [kN]

Bending moment

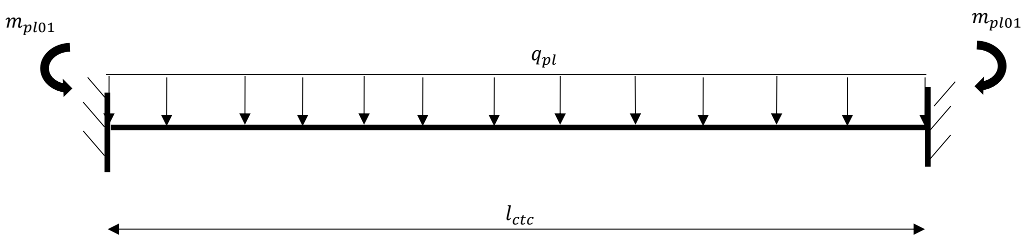
Mx 5045,0 [kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	1,00	[m]
a	20,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	-56,9	[kN/m]
mpl01	-118,6	[kNm]

Measurements		
lctc	5,0	[m]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,tot	0,0	[kNm]	

Suspension force

Qyy	-56,9	[kN]

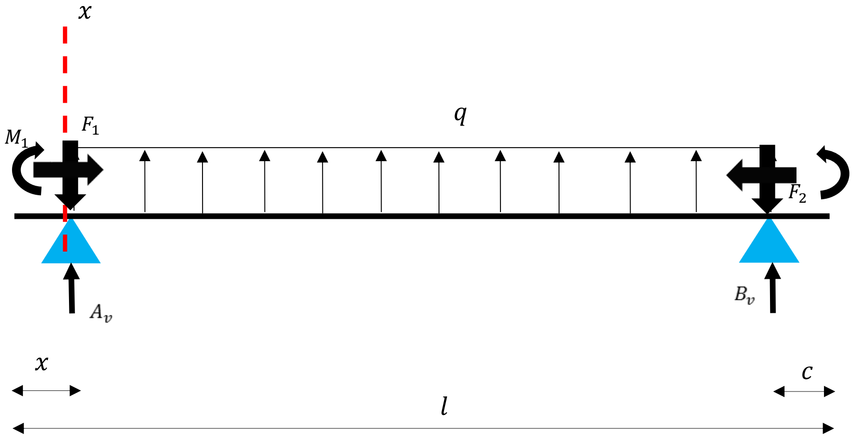
Clamping moment

mxx,alt	28,0	[kNm]	(due to alternative load case)
mxx,tot	28,0	[kNm]	

LC 9

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	2095	[kN]
F2	-20886	[kN]
M1	4616	[kNm]
q	-133	[kN]
Av	3	[kN]
Bv	3	[kN]
P∞/P0	0,915	[-]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

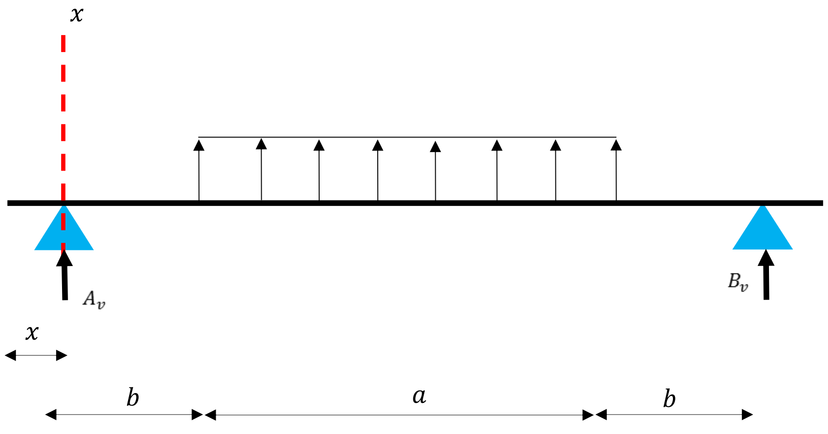
Vz -2092,5 [kN]

Bending moment

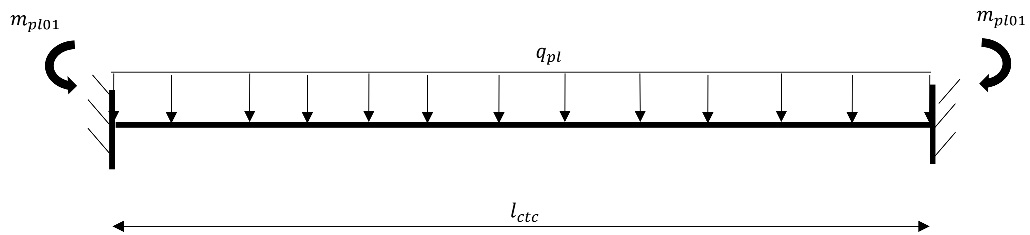
Mx 4616,2 [kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	1,00	[m]
a	20,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	-52,1	[kN/m]
mpl01	-108,5	[kNm]

Measurements		
lctc	5,0	[m]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,tot	0,0	[kNm]	

Suspension force

Qyy	-52,1	[kN]

Clamping moment

mxx,alt	25,6	[kNm]	(due to alternative load case)
mxx,tot	25,6	[kNm]	

5.4 Bridge B - 6.10b - support

Normal stresses

LC	type	Prestress P [kN]	Prestress σxx [N/mm2]	Bending moment M [kNm]	Bending moment σxx [N/mm2]	Total hor. normal stress σxx [N/mm2]
1	self-weight			-67	0,01	
2	ballast			-78	0,01	
3a	Mobile Max. (LM71)			-71	0,01	
3b	Mobile Max. (SW/2)			-51	0,00	
4	Mobile Min. (SW/2)			-253	0,02	
5	Support settelement max			0	0,00	
6	Support settelement min			0	0,00	
7	Prestress t=0	-22826	-8,65	5045	-0,41	
8	Prestress t = ∞	-20886	-7,91	4616	-0,38	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-20886	-7,91	4400	-0,36	-8,27
LC 1 + LC 2 + LC 3a + LC 6 + LC8		-20886	-7,91	4400	-0,36	-8,27
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-20886	-7,91	4219	-0,35	-8,26
LC 1 + LC 2 + LC 4 + LC 6 + LC8		-20886	-7,91	4219	-0,35	-8,26
LC 1 + LC 7		-22826	-8,65	4978	-0,41	-9,06
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-20886	-7,91	4421	-0,36	-8,27
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		-20886	-7,91	4421	-0,36	-8,27

LC	type	Suspension force Qyy [kN]	Suspension force σyy [N/mm2]	Clamping moment mxx [kNm]	Clamping moment σyy [N/mm2]	Suspension force excen. Myy [kNm]	Suspension force excen. σyy [N/mm2]	Total ver. Normal stress σyy [N/mm2]
1	self-weight	53	0,02	29	0,06	32	0,07	
2	ballast	31	0,01	19	0,04	19	0,04	
3a	Mobile Max. (LM71)	142	0,06	-4	-0,01	85	0,17	
3b	Mobile Max. (SW/2)	101	0,04	29	0,06	61	0,12	
4	Mobile Min. (SW/2)	101	0,04	73	0,15	61	0,12	
5	Support settelement max	-265	-0,11	227	0,47	-159	-0,33	
6	Support settelement min	265	0,11	-227	-0,47	159	0,33	
7	Prestress t=0	-57	-0,02	28	0,06	-34	-0,07	
8	Prestress t = ∞	-52	-0,02	26	0,05	-31	-0,06	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-92	-0,04	297	0,61	-55	-0,11	0,46
LC 1 + LC 2 + LC 3a + LC 6 + LC8		439	0,18	-157	-0,32	263	0,54	0,40
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-132	-0,05	374	0,77	-79	-0,16	0,55
LC 1 + LC 2 + LC 4 + LC 6 + LC8		398	0,16	-79	-0,16	239	0,49	0,49
LC 1 + LC 7		-4	0,00	57	0,12	-2	-0,01	0,11
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-132	-0,05	330	0,68	-79	-0,16	0,46
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		398	0,16	-124	-0,25	239	0,49	0,40

Shear and principal stresses

LC	type	Shear force Vz [kN]	Shear force τxy [N/mm2]	Torsion Mxy [kNm]	Torsion τxy [N/mm2]	Total shear stress τxy [N/mm2]
1	self-weight	2123	0,96	-288	-0,29	
2	ballast	488	0,22	-142	-0,14	
3a	Mobile Max. (LM71)	1573	0,71	-287	-0,29	
3b	Mobile Max. (SW/2)	1502	0,68	-205	-0,21	
4	Mobile Min. (SW/2)	0	0,00	-453	-0,46	
5	Support settelement max	0	0,00	-589	-0,60	
6	Support settelement min	0	0,00	589	0,60	
7	Prestress t=0	-2287	-1,03	0	0,00	
8	Prestress t = ∞	-2093	-0,95	0	0,00	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		2092	0,95	-1306	-1,33	2,27
LC 1 + LC 2 + LC 3a + LC 6 + LC8		2092	0,95	-128	-0,13	1,08
LC 1 + LC 2 + LC 4 + LC 5 + LC8		519	0,23	-1472	-1,50	1,73
LC 1 + LC 2 + LC 4 + LC 6 + LC8		519	0,23	-294	-0,30	0,53
LC 1 + LC 7		-164	-0,07	-288	-0,29	0,37
LC 1 + LC 2 + LC 3b + LC 5 + LC8		2021	0,91	-1224	-1,24	2,16
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		2021	0,91	-46	-0,05	0,96

LC	type	Total σxx [N/mm2]	Total σyy [N/mm2]	Total τxy [N/mm2]	Principal stress ρ1 [N/mm2]	Principal stress ρ2 [N/mm2]
1	self-weight					
2	ballast					
3a	Mobile Max. (LM71)					
3b	Mobile Max. (SW/2)					
4	Mobile Min. (SW/2)					
5	Support settelement max					
6	Support settelement min					
7	Prestress t=0					
8	Prestress t = ∞					
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-8,27	0,46	2,27	1,02	-8,83
LC 1 + LC 2 + LC 3a + LC 6 + LC8		-8,27	0,40	1,08	0,53	-8,40
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-8,26	0,55	1,73	0,88	-8,59
LC 1 + LC 2 + LC 4 + LC 6 + LC8		-8,26	0,49	0,53	0,52	-8,29
LC 1 + LC 7		-9,06	0,11	0,37	0,13	-9,07
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-8,27	0,46	2,16	0,96	-8,78
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		-8,27	0,40	0,96	0,50	-8,38

Parameters

Bending stiffness coefficient

E	34000	[N/mm2]
S_{pl}	3,11E+08	

Torsional stiffness coefficient

v	0,20	
G	1,42E+04	[N/mm2]
$I_{t,girder}$	8,35E+11	[mm4]
GI_t	1,18E+16	[Nmm2]
ω	0,16	

Shear stress parameters

I_{yy}	1,73E+12	[mm4]
S	9,38E+08	[mm]
b_{girder}	1200	[mm]
A_M	2000000	[mm2]
t_{ef}	200	[mm]

Sectional properties

b_{girder}	1200	[mm]
h_{girder}	2200	[mm]
t_{floor}	650	[mm]
b_{floor}	5000	[mm]
$0{,}5 * A_{prestress}$	2640000	[mm2]
$0{,}5 * I_{yy}$	1,73E+12	[mm4]
y	142	[mm]
z	808	[mm]

Normal stress parameters

A_1	1500000	[mm2]
A_2	1540000,00	[mm2]
$A_1/(A_1+A_2)$	49,3%	
$A_{Q_{yy}}$	1200000	[mm2]
$W_{m_{xx}}$	240000000	[mm3]
$eccentricity$	0,6	[m]

Torsional stiffness girder

a	1,1	[m]
b	0,6	[m]
$\propto$	3,51	[-]
$I_{t,girder}$	0,83	[m4]

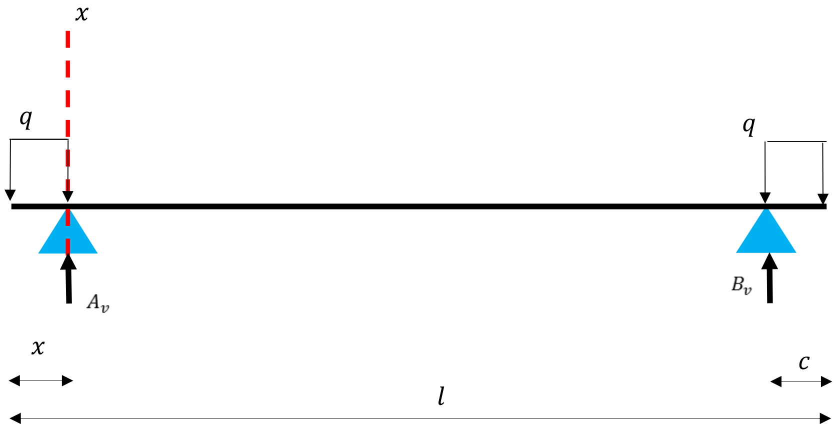
Torsional stiffness floor

a	1,25	[m]
b	0,325	[m]
$\propto$	4,46	[-]
$I_{t,floor}$	0,19	[m4]
$I_{t,grider}/(I_{t,floor} + I_{t,girder})$	81,35%	[%]

LC 1

Cantilevers loaded

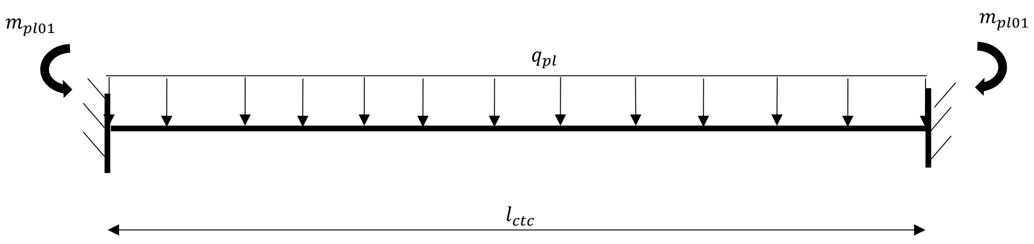
Loading (long. direction)



Load & Reaction forces		
qbridge	269,6	[kN/m]
0,5q	134,8	[kN/m]
Av	134,8	[kN]
Bv	134,8	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	53,9	[kN/m]
mpl01	112,3	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -67,4 [kNm]

Torsion

Mxy,M	-112,3	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	-112,3	[kNm]	

Suspension force

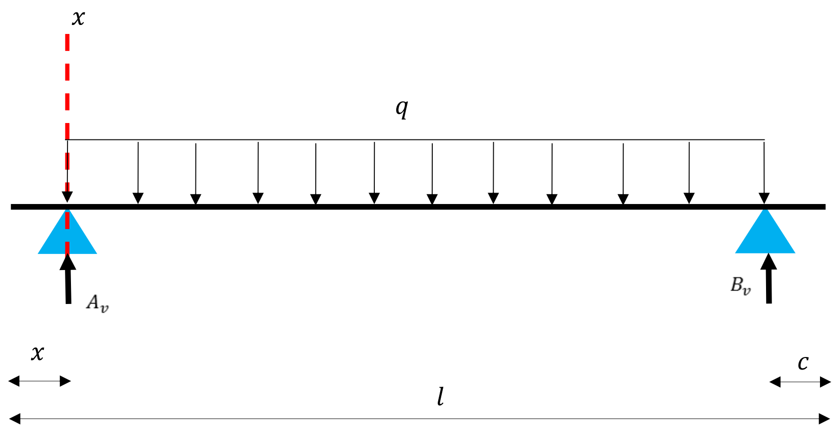
Qyy 0,0 [kN]

Clamping moment

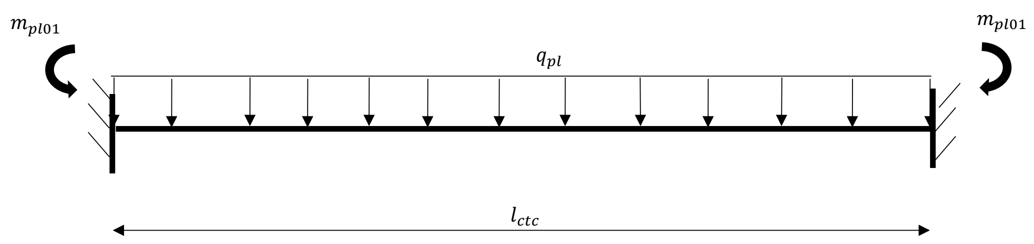
mxx,M	18,2	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	18,2	[kNm]	

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
qbridge	269,6	[kN/m]
0,5q	134,8	[kN/m]
Av	2123,1	[kN]
Bv	2123,1	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	53,9	[kN/m]
mpl02	112,3	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz 2123,1 [kN]

Bending moment

Mx 0,0 [kNm]

Torsion

Mxy, alt	-175,5	[kNm]	(due to alternative load case)
Mxy,subtot	-175,5	[kNm]	

Suspension force

Qyy 52,8 [kN]

Clamping moment

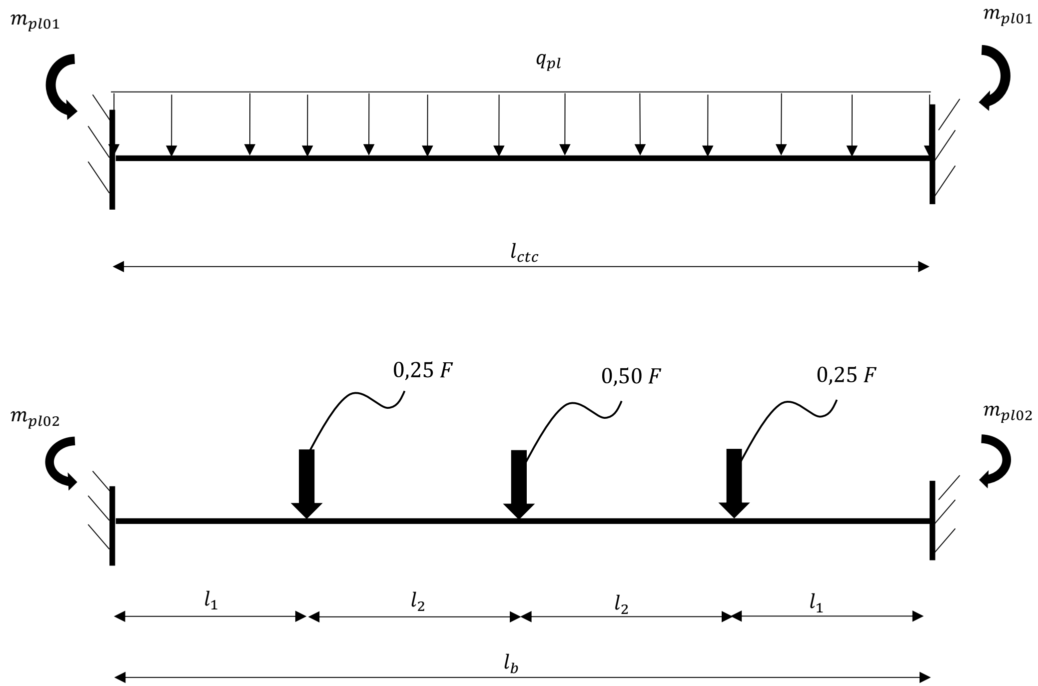
mxx, alt	11,1	[kNm]	(due to alternative load case)
mxx,subtot.	11,1	[kNm]	

LC 2

Cantilevers loaded

Loading (long. direction)

Loading (transverse direction)



Load & Reaction forces		
q	12,4	[kN/m2]
0,5q	31,0	[kN/m]
F	124	[kN]
0,5F	62	[kN]
Av	93,0	[kN]
Bv	93,0	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	12,4	[kN/m]
0,25F	31,0	[kN]
mpl01	25,8	[kNm]
mpl02	76,1	[kNm]
MT	101,9	[kNm]

Measurements		
lctc	5,0	[m]
l1	1,1	[m]
l2	2,0	[m]
lb	6,2	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

Bending moment

Mx	-77,5	[kNm]

Torsion

Mxy,M	-101,9	[kNm]
Mxy,subtot	-101,9	[kNm]

(due to torsional moment at both ends)

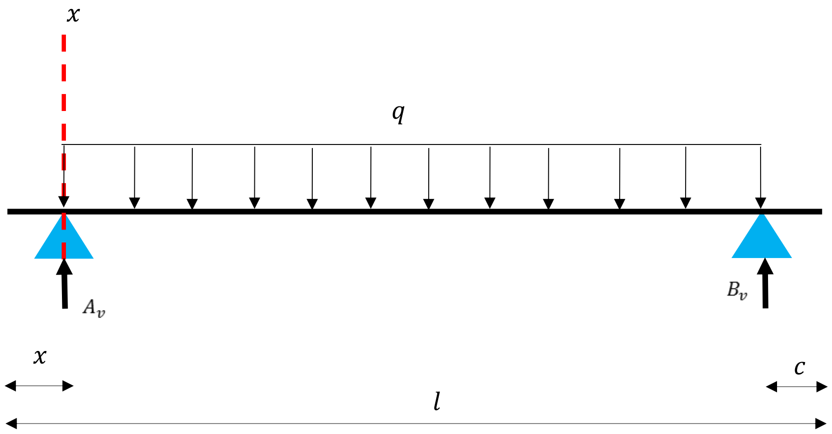
Clamping moment

mxx,M	16,5	[kNm]
mxx,subtot.	16,5	[kNm]

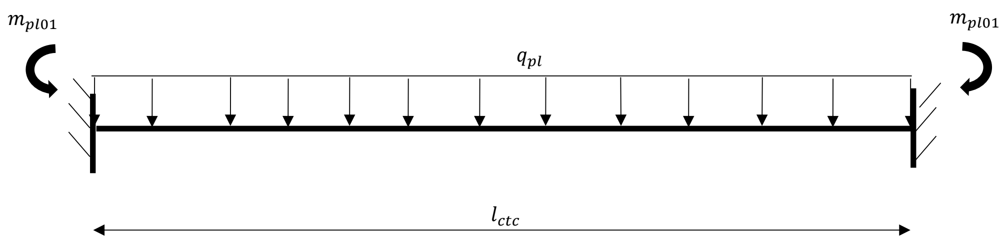
(due to torsional moment at both ends)

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	12,4	[kN/m2]
0,5q	31,0	[kN/m]
Av	488,3	[kN]
Bv	488,3	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	12,4	[kN/m]
mpl01	25,8	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz	488,3	[kN]

Suspension force

Qyy	31,0	[kN]

Bending moment

Mx	0,0	[kNm]

Torsion

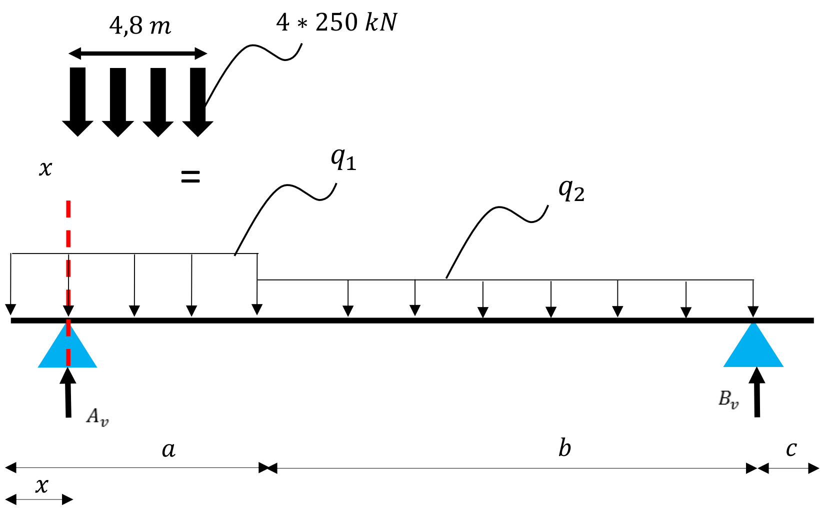
Mxy,alt	-40,3	[kNm]	(due to alternative load case)
Mxy,subtot	-40,3	[kNm]	

Clamping moment

mxx,alt	2,6	[kNm]	(due to alternative load case)
mxx,subtot.	2,6	[kNm]	

LC 3a

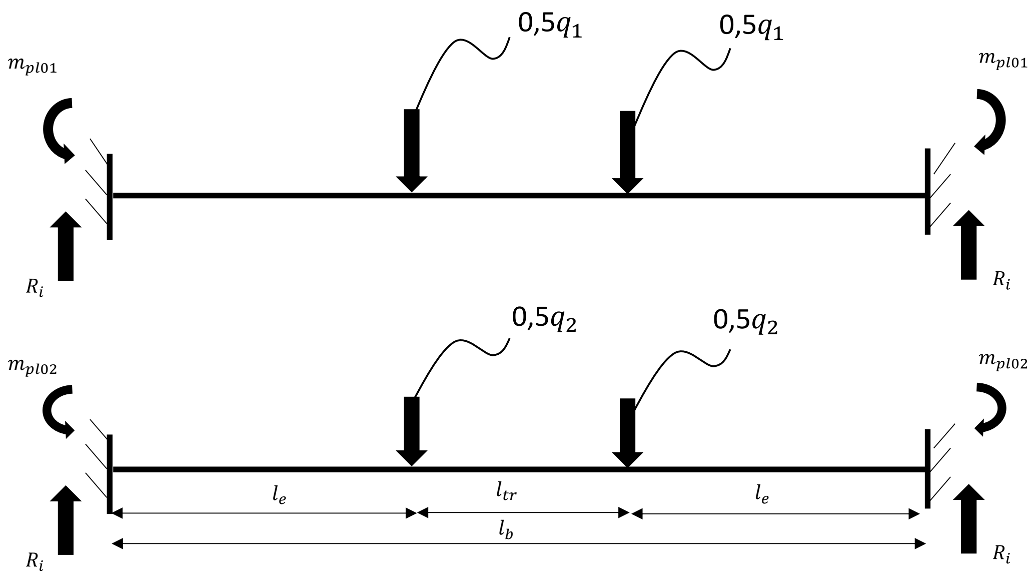
Loading (long. direction)



Load & Reaction forces		
q1	283,3	[kN/m]
0,5q1	141,7	[kN/m]
q2	156,4	[kN/m]
0,5q2	78,2	[kN/m]
Av	1714,8	[kN]
Bv	1264,5	[kN]

Measurements		
x	1,00	[m]
a	6,9	[m]
b	25,6	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q1	141,7	[kN/m]
0,5q2	78,2	[kN/m]
mpl01	287,2	[kNm]
mpl02	158,6	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]

Shear Force

Vz	1573,2	[kN]

Bending moment

Mx	-70,8	[kNm]

Torsion

Mxy,M	-287,2	[kNm]	(due to torsional moment at one end)
Mxy,q1	0	[kNm]	(due to distributed load)
Mxx,q2	0	[kNm]	(due to distributed load)
Mxy,tot	-287,2	[kNm]	

Suspension force

Qyy	141,7	[kN]

Clamping moment

mxx,M	46,0	[kNm]	(due to torsional moment at one end)
mxx,q1	-111,7	[kNm]	(due to distributed load)
mxx,q2	61,7	[kNm]	(due to distributed load)
mxx,tot	-4,0	[kNm]	

LC 3b

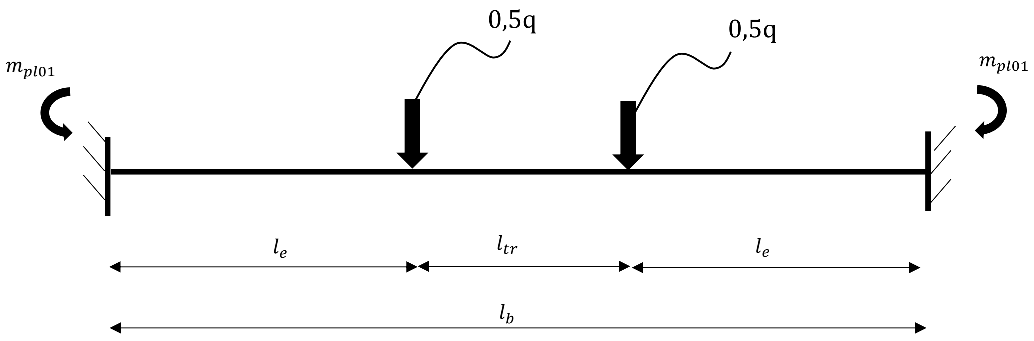
Cantilevers loaded

Loading (long. direction)

Load & Reaction forces		
q	202	[kN/m]
0,5q	101,0	[kN/m]
Av	102,6	[kN]
Bv	-1,6	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	101,0	[kN/m]
mpl01	204,8	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]

Shear Force

Vz 1,6 [kN]

Bending moment

Mx -50,5 [kNm]

Torsion

Mxy,M	-204,8	[kNm]	(due to torsional moment at one end)
Mxy,subtot	-204,8	[kNm]	

Suspension force

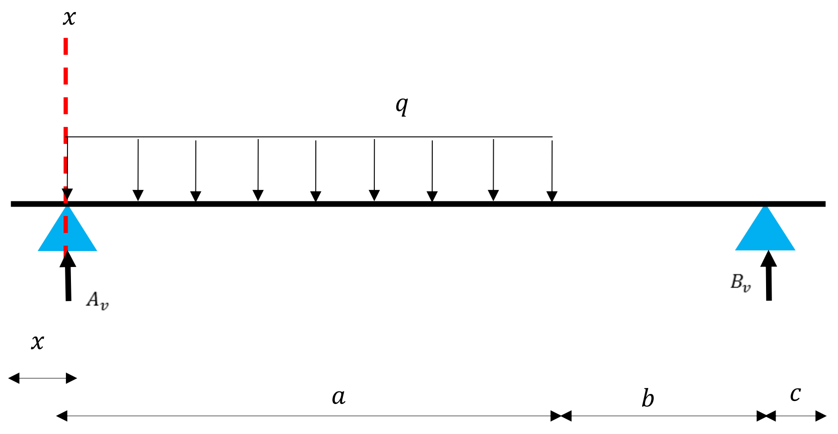
Qyy 0,0 [kN]

Clamping moment

mxx,M	32,8	[kNm]	(due to torsional moment at one end)
mxx,subtot.	32,8	[kNm]	

Midspan loaded

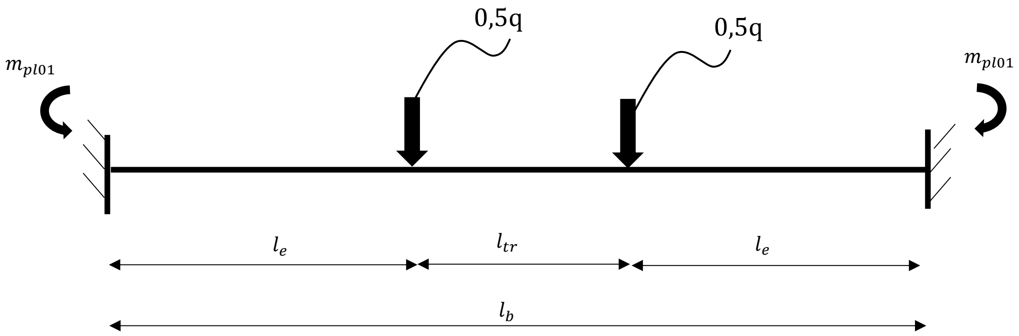
Loading (long. direction)



Load & Reaction forces		
q	202	[kN/m]
0,5q	101	[kN/m]
Av	1500,6	[kN]
Bv	923,4	[kN]

Measurements		
x	1,00	[m]
a	24	[m]
b	7,5	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	101,0	[kN/m]
mpl01	204,8	[kNm]

Measurements		
lb	6,2	[m]
ltr	1,5	[m]
le	2,4	[m]

Shear Force

Vz 1500,6 [kN]

Bending moment

Mx 0,0 [kNm]

Torsion

Mxy, alt	0,0	[kNm]	(due to distributed load)
Mxy,subtot	0,0	[kNm]	

Suspension force

Qyy 101,0 [kN]

Clamping moment

mxx, alt	-4	[kNm]	(due to distributed load)
mxx,subtot.	-4,0	[kNm]	

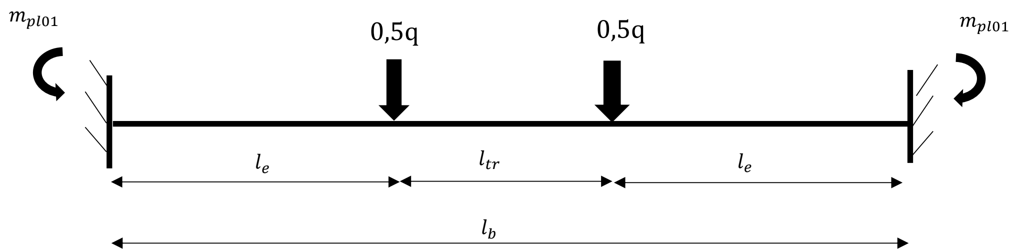
LC 4

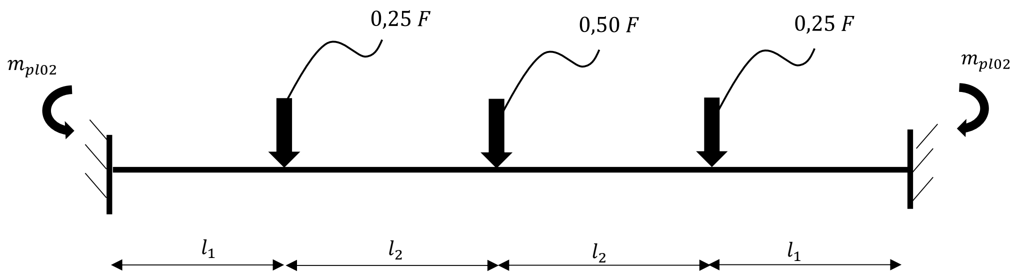
Loading (long. direction)

Load & Reaction forces		
q	202	[kN/m]
0,5q	101,0	[kN/m]
F	404	[kN]
0,5F	202	[kN]
Av	303,0	[kN]
Bv	303,0	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)





Load & Reaction forces		
0,5q	101,0	[kN/m]
0,25F	101,0	[kN]
mpl01	204,8	[kNm]
mpl02	247,9	[kNm]
MT	452,7	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]
l1	1,1	[m]
l2	2,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -252,5 [kNm]

Torsion

Mxy,M	-452,7	[kNm]	(due to torsional moment at both ends)
Mxy,tot	-452,7	[kNm]	

Suspension force

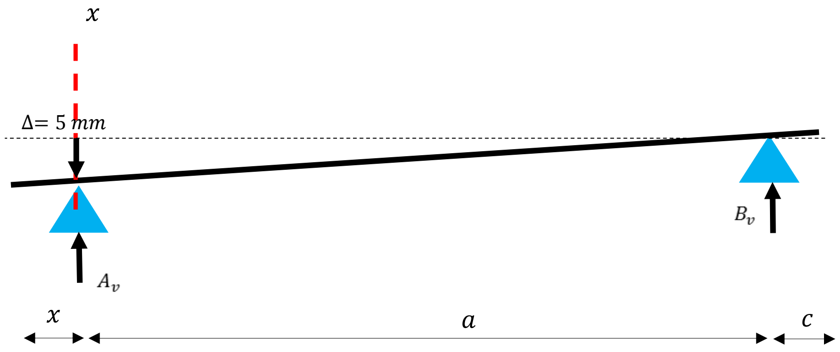
Qyy 101,0 [kN]

Clamping moment

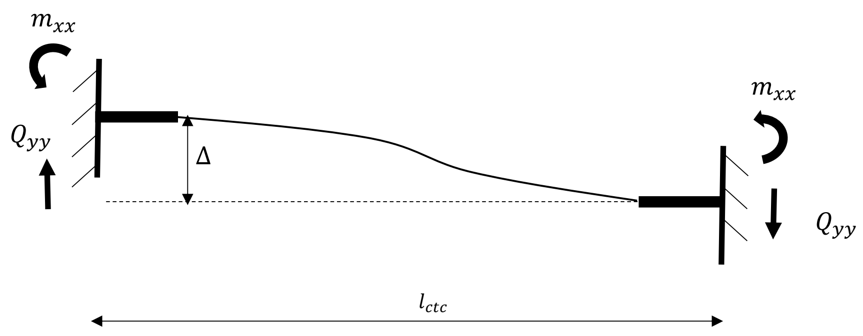
mxx,M	73,4	[kNm]	(due to torsional moment at both ends)
mxx,tot	73,4	[kNm]	

LC 6

Loading (long. direction)



Loading (transverse direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γ	1,2	[-]

Measurements		
x	1,00	[m]
a	31,5	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

Vz	0,0	[kN]

Bending moment

Mx	0,0	[kNm]

Torsion

Mxy,Δ	-491,0	[kNm]
Mxy,tot	-589,2	[kNm]

Suspension force

Qyy	-221,0	[kN]
Qyy,tot	-265,2	[kN]

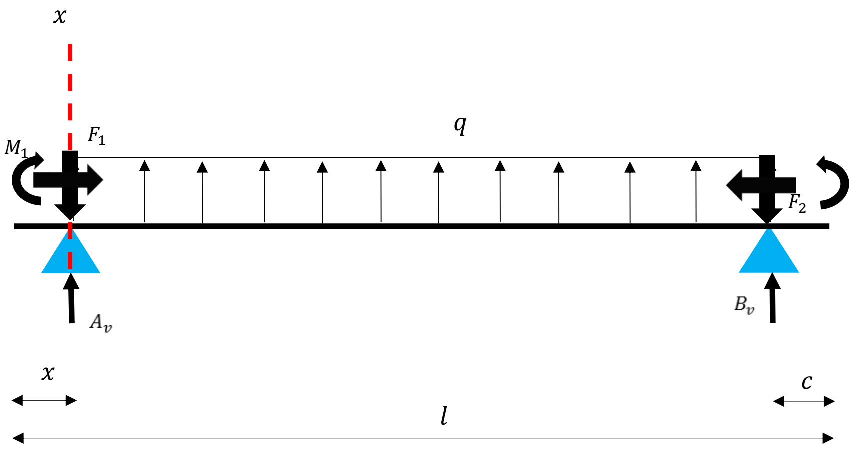
Clamping moment

mxx,Δ	189,0	[kNm]
mxx,tot	226,8	[kNm]

LC 8

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	2290	[kN]
F2	-22826	[kN]
M1	5045	[kNm]
q	-145,2	[kN]
Av	3	[kN]
Bv	3	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

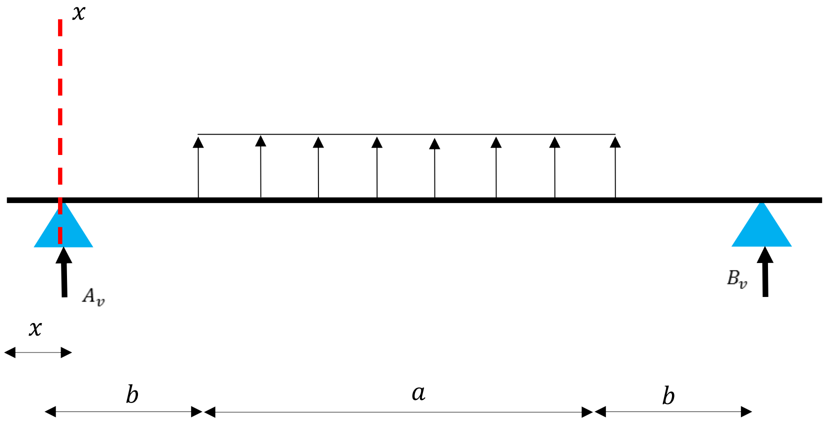
Vz -2286,9 [kN]

Bending moment

Mx 5045,0 [kNm]

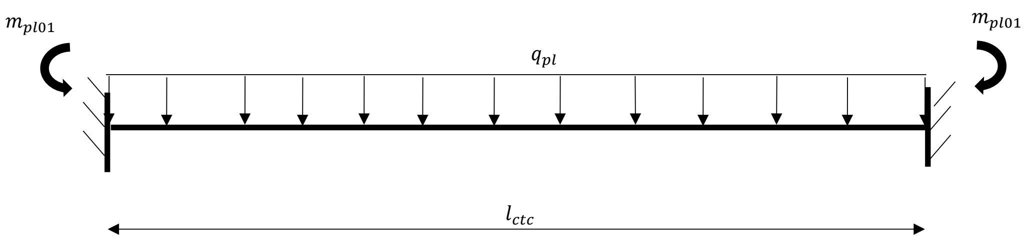
Floor loaded

Loading (long. direction)



Measurements		
x	1,00	[m]
a	20,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	-56,9	[kN/m]
mpl01	-118,6	[kNm]

Measurements		
lctc	5,0	[m]

Torsion

Mxy,alt	0,0	[kNm]
Mxy,tot	0,0	[kNm]

(due to alternative load case)

Suspension force

Qyy	-56,9	[kN]

Clamping moment

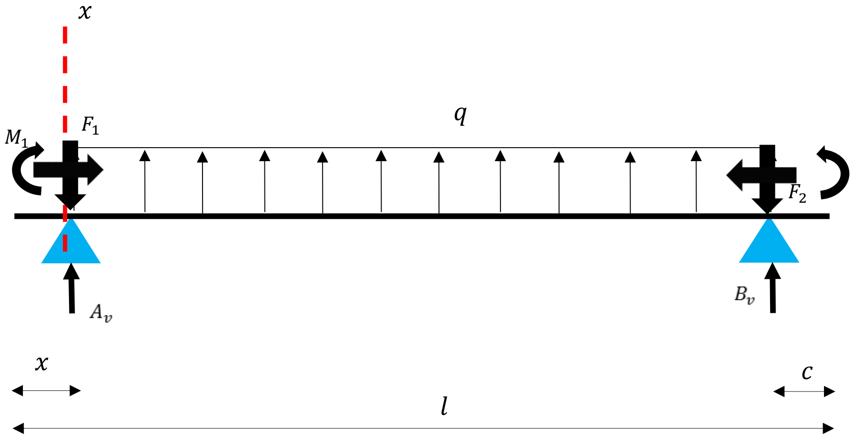
mxx,alt	28,0	[kNm]
mxx,tot	28,0	[kNm]

(due to alternative load case)

LC 9

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	2095	[kN]
F2	-20886	[kN]
M1	4616	[kNm]
q	-133	[kN]
Av	3	[kN]
Bv	3	[kN]
P∞/P0	0,915	[-]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

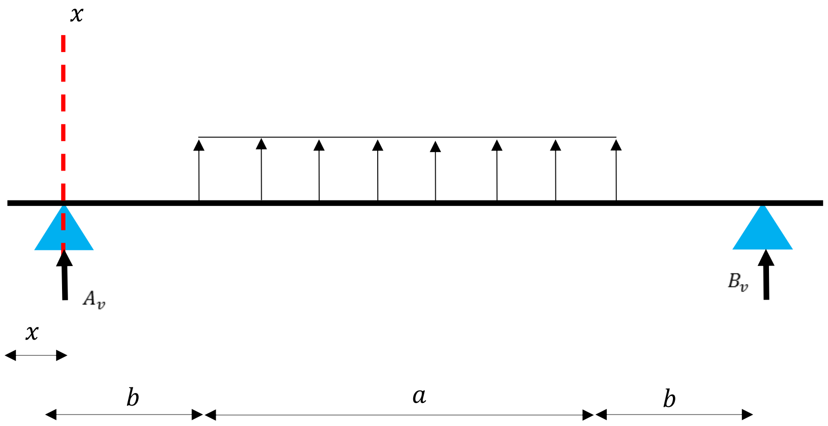
Vz	-2092,5	[kN]

Bending moment

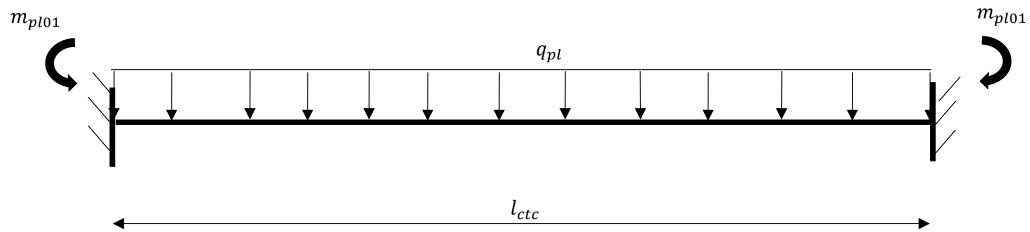
Mx	4616,2	[kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	1,00	[m]
a	20,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	-52,1	[kN/m]
mpl01	-108,5	[kNm]

Measurements		
lctc	5,0	[m]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,tot	0,0	[kNm]	

Suspension force

Qyy	-52,1	[kN]

Clamping moment

mxx,alt	25,6	[kNm]	(due to alternative load case)
mxx,tot	25,6	[kNm]	

5.5 Bridge B - 6.10b – 0,8d

Normal stresses

LC	type	Prestress P [kN]	Prestress σxx [N/mm2]	Bending moment M [kNm]	Bending moment σxx [N/mm2]	Total hor. normal stress σxx [N/mm2]
1	self-weight			3366	-0,28	
2	ballast			712	-0,06	
3a	Mobile Max. (LM71)			2412	-0,20	
3b	Mobile Max. (SW/2)			2371	-0,19	
4	Mobile Min. (SW/2)			-253	0,02	
5	Support settelement max			0	0,00	
6	Support settelement min			0	0,00	
7	Prestress t=0	-22826	-6,64	1347	-0,11	
8	Prestress t = ∞	-20886	-6,08	1232	-0,10	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-20886	-6,08	7723	-0,63	-6,71
LC 1 + LC 2 + LC 3a + LC 6 + LC8		-20886	-6,08	7723	-0,63	-6,71
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-20886	-6,08	5058	-0,42	-6,49
LC 1 + LC 2 + LC 4 + LC 6 + LC8		-20886	-6,08	5058	-0,42	-6,49
LC 1 + LC 7		-22826	-6,64	4713	-0,39	-7,03
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-20886	-6,08	7681	-0,63	-6,71
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		-20886	-6,08	7681	-0,63	-6,71

LC	type	Suspension force Qyy [kN]	Suspension force σyy [N/mm2]	Clamping moment mxx [kNm]	Clamping moment σyy [N/mm2]	Suspension force excen. Myy [kNm]	Suspension force excen. σyy [N/mm2]	Total ver. Normal stress σyy [N/mm2]
1	self-weight	53	0,02	25	0,05	32	0,07	
2	ballast	31	0,01	15	0,03	19	0,04	
3a	Mobile Max. (LM71)	142	0,06	-17	-0,03	85	0,17	
3b	Mobile Max. (SW/2)	101	0,04	21	0,04	61	0,12	
4	Mobile Min. (SW/2)	0	0,00	56	0,12	0	0,00	
5	Support settelement max	-24	-0,01	114	0,23	-14	-0,03	
6	Support settelement min	24	0,01	-114	-0,23	14	0,03	
7	Prestress t=0	-57	-0,02	29	0,06	-34	-0,07	
8	Prestress t = ∞	-52	-0,02	27	0,05	-31	-0,06	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		149	0,06	163	0,34	90	0,18	0,58
LC 1 + LC 2 + LC 3a + LC 6 + LC8		197	0,08	-64	-0,13	118	0,24	0,19
LC 1 + LC 2 + LC 4 + LC 5 + LC8		8	0,00	236	0,49	5	0,01	0,50
LC 1 + LC 2 + LC 4 + LC 6 + LC8		56	0,02	9	0,02	33	0,07	0,11
LC 1 + LC 7		-4	0,00	54	0,11	-2	-0,01	0,10
LC 1 + LC 2 + LC 3b + LC 5 + LC8		109	0,04	201	0,41	65	0,13	0,59
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		157	0,06	-26	-0,05	94	0,19	0,20

Shear and principal stresses

LC	type	Shear force Vz [kN]	Shear force τxy [N/mm2]	Torsion Mxy [kNm]	Torsion τxy [N/mm2]	Total shear stress τxy [N/mm2]
1	self-weight	1893	0,86	-242	-0,25	
2	ballast	435	0,20	-117	-0,12	
3a	Mobile Max. (LM71)	1331	0,60	-261	-0,27	
3b	Mobile Max. (SW/2)	1329	0,60	-159	-0,16	
4	Mobile Min. (SW/2)	0	0,00	-343	-0,35	
5	Support settelement max	0	0,00	-358	-0,36	
6	Support settelement min	0	0,00	358	0,36	
7	Prestress t=0	-2039	-0,92	49	0,05	
8	Prestress t = ∞	-1865	-0,84	44	0,05	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		1793	0,81	-934	-0,95	1,76
LC 1 + LC 2 + LC 3a + LC 6 + LC8		1793	0,81	-217	-0,22	1,03
LC 1 + LC 2 + LC 4 + LC 5 + LC8		463	0,21	-1015	-1,03	1,24
LC 1 + LC 2 + LC 4 + LC 6 + LC8		463	0,21	-299	-0,30	0,51
LC 1 + LC 7		-146	-0,07	-193	-0,20	0,26
LC 1 + LC 2 + LC 3b + LC 5 + LC8		1792	0,81	-831	-0,85	1,66
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		1792	0,81	-115	-0,12	0,93

LC	type	Total σxx [N/mm2]	Total σyy [N/mm2]	Total τxy [N/mm2]	Principal stress ρ1 [N/mm2]	Principal stress ρ2 [N/mm2]
1	self-weight					
2	ballast					
3a	Mobile Max. (LM71)					
3b	Mobile Max. (SW/2)					
4	Mobile Min. (SW/2)					
5	Support settelement max					
6	Support settelement min					
7	Prestress t=0					
8	Prestress t = ∞					
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-6,71	0,58	1,76	0,98	-7,12
LC 1 + LC 2 + LC 3a + LC 6 + LC8		-6,71	0,19	1,03	0,34	-6,86
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-6,49	0,50	1,24	0,71	-6,71
LC 1 + LC 2 + LC 4 + LC 6 + LC8		-6,49	0,11	0,51	0,15	-6,53
LC 1 + LC 7		-7,03	0,10	0,26	0,11	-7,04
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-6,71	0,59	1,66	0,95	-7,07
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		-6,71	0,20	0,93	0,33	-6,83

Parameters

Bending stiffness coefficient

E	34000	[N/mm2]
S_{pl}	3,11E+08	

Torsional stiffness coefficient

v	0,20	
G	1,42E+04	[N/mm2]
$I_{t,girder}$	8,35E+11	[mm4]
GI_t	1,18E+16	[Nmm2]
ω	0,16	

Shear stress parameters

I_{yy}	1,73E+12	[mm4]
S	9,38E+08	[mm]
b_{girder}	1200	[mm]
A_M	2000000	[mm2]
t_{ef}	200	[mm]

Sectional properties

b_{girder}	1200	[mm]
h_{girder}	2200	[mm]
t_{floor}	650	[mm]
b_{floor}	5000	[mm]
$0{,}5 * A_{prestress}$	3436500	[mm2]
$0{,}5 * I_{yy}$	1,73E+12	[mm4]
y	142	[mm]
z	808	[mm]

Normal stress parameters

A_1	1500000	[mm2]
A_2	1540000,00	[mm2]
$A_1/(A_1+A_2)$	49,3%	
$A_{Q_{yy}}$	1200000	[mm2]
$W_{m_{xx}}$	240000000	[mm3]
$eccentricity$	0,6	[m]

Torsional stiffness girder

a	1,1	[m]
b	0,6	[m]
$\propto$	3,51	[-]
$I_{t,girder}$	0,83	[m4]

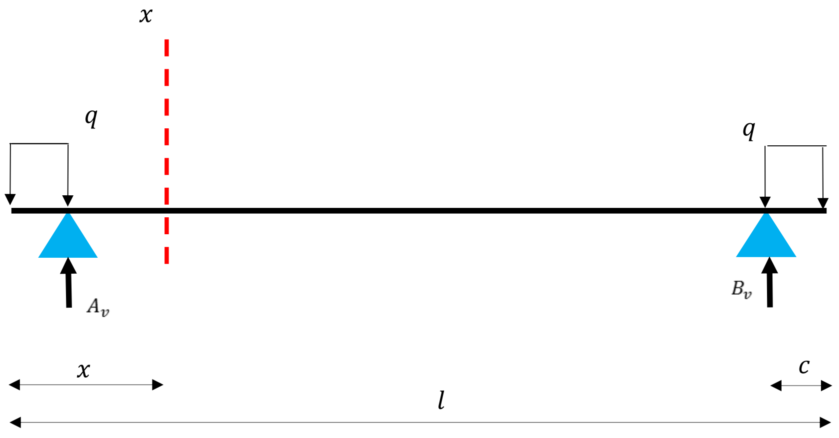
Torsional stiffness floor

a	1,25	[m]
b	0,325	[m]
$\propto$	4,46	[-]
$I_{t,floor}$	0,19	[m4]
$I_{t,grider}/(I_{t,floor} + I_{t,girder})$	81,35%	[%]

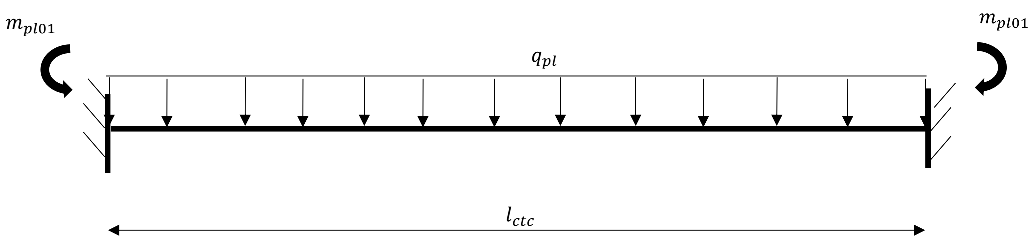
LC 1

Cantilevers loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
qbridge	269,6	[kN/m]
0,5q	134,8	[kN/m]
Av	134,8	[kN]
Bv	134,8	[kN]

Measurements		
x	2,71	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	53,9	[kN/m]
mpl01	112,3	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -67,4 [kNm]

Torsion

Mxy,M	-85,0	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	-85,0	[kNm]	

Suspension force

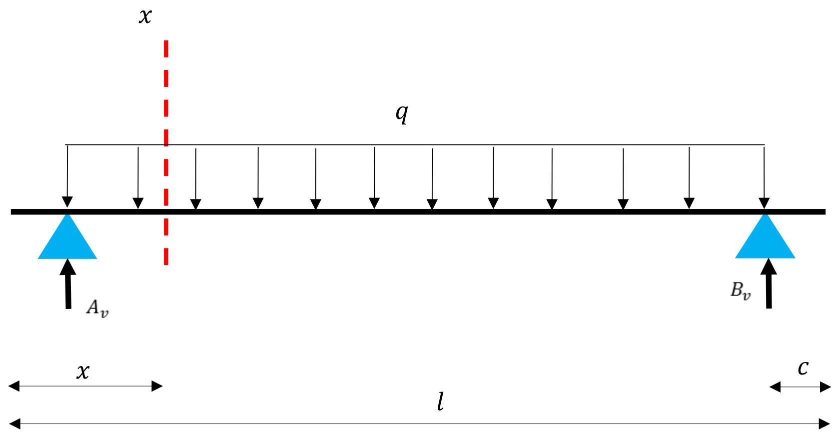
Qyy 0,0 [kN]

Clamping moment

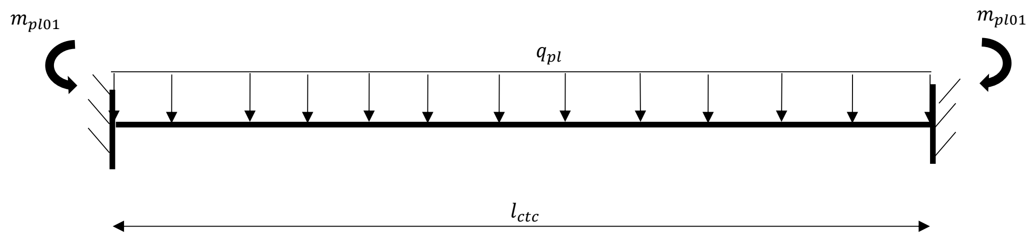
mxx,M	13,9	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	13,9	[kNm]	

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
qbridge	269,6	[kN/m]
0,5q	134,8	[kN/m]
Av	2123,1	[kN]
Bv	2123,1	[kN]

Measurements		
x	2,71	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	53,9	[kN/m]
mpl02	112,3	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz 1892,6 [kN]

Bending moment

Mx 3433,4 [kNm]

Torsion

Mxy, alt	-156,5	[kNm]	(due to alternative load case)
Mxy,subtot	-156,5	[kNm]	

Suspension force

Qyy 52,8 [kN]

Clamping moment

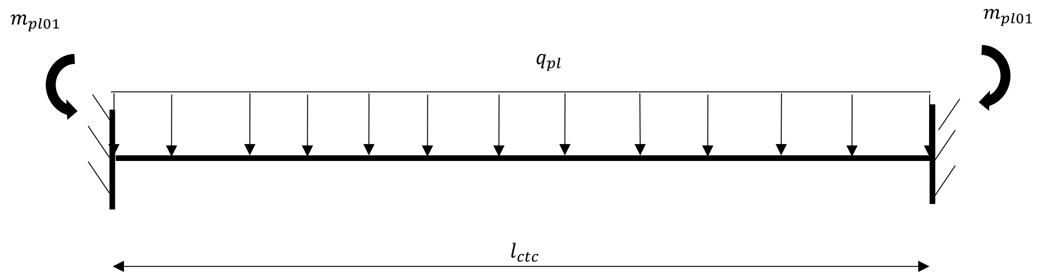
mxx, alt	11,1	[kNm]	(due to alternative load case)
mxx,subtot.	11,1	[kNm]	

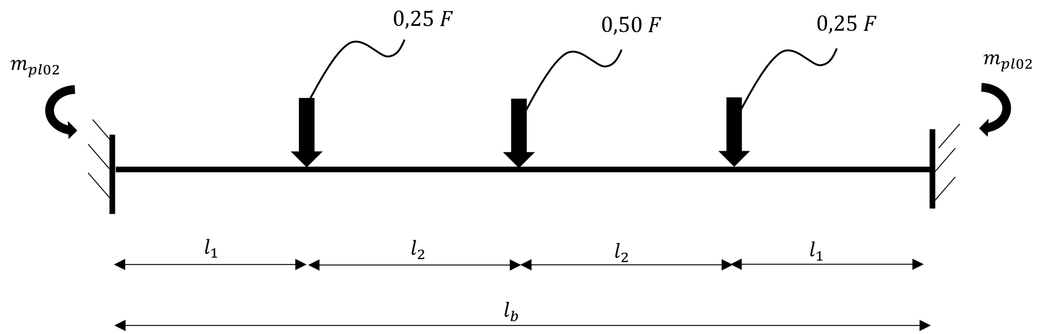
LC 2

Cantilevers loaded

Loading (long. direction)

Loading (transverse direction)





Load & Reaction forces		
q	12,4	[kN/m2]
0,5q	31,0	[kN/m]
F	124	[kN]
0,5F	62	[kN]
Av	93,0	[kN]
Bv	93,0	[kN]

Measurements		
x	2,71	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	12,4	[kN/m]
0,25F	31,0	[kN]
mpl01	25,8	[kNm]
mpl02	76,1	[kNm]
MT	101,9	[kNm]

Measurements		
lctc	5,0	[m]
l1	1,1	[m]
l2	2,0	[m]
lb	6,2	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

Bending moment

Mx	-77,5	[kNm]

Torsion

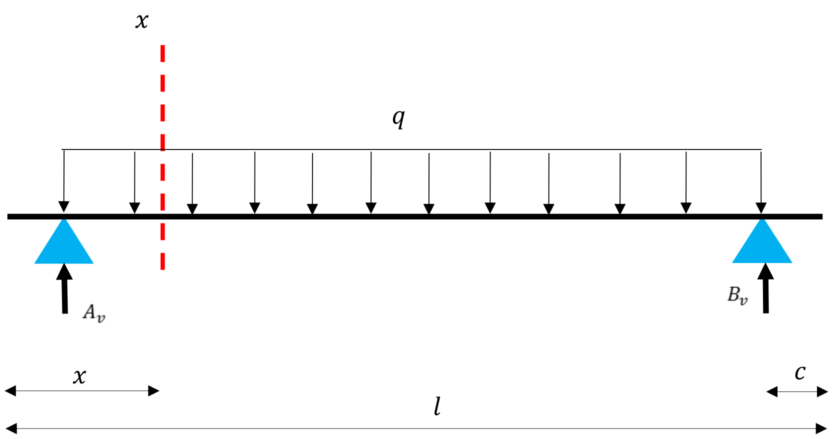
Mxy,M	-77,1	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	-77,1	[kNm]	

Clamping moment

mxx,M	12,6	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	12,6	[kNm]	

Midspan loaded

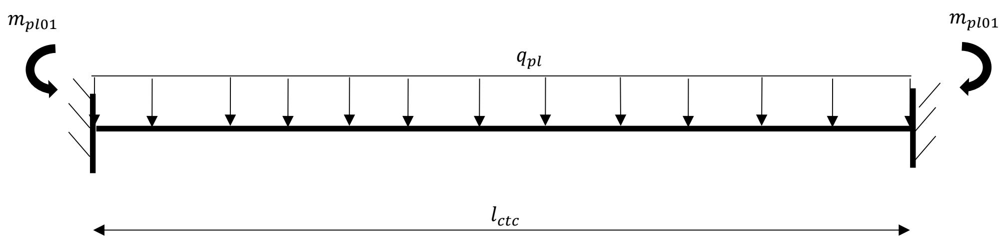
Loading (long. direction)



Load & Reaction forces		
q	12,4	[kN/m2]
0,5q	31,0	[kN/m]
Av	488,3	[kN]
Bv	488,3	[kN]

Measurements		
x	2,71	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	12,4	[kN/m]
mpl01	25,8	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz	435,2	[kN]

Suspension force

Qyy	31,0	[kN]

Bending moment

Mx	789,6	[kNm]

Torsion

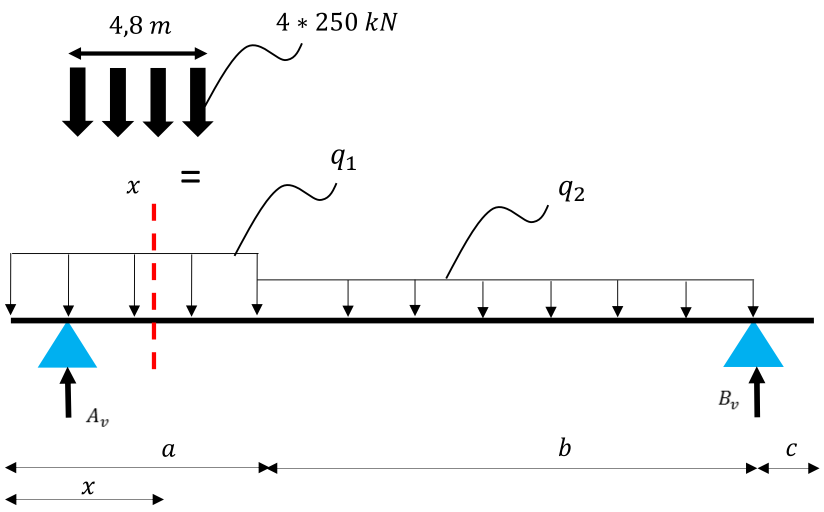
Mxy,alt	-39,9	[kNm]	(due to alternative load case)
Mxy,subtot	-39,9	[kNm]	

Clamping moment

mxx,alt	2,6	[kNm]	(due to alternative load case)
mxx,subtot.	2,6	[kNm]	

LC 3a

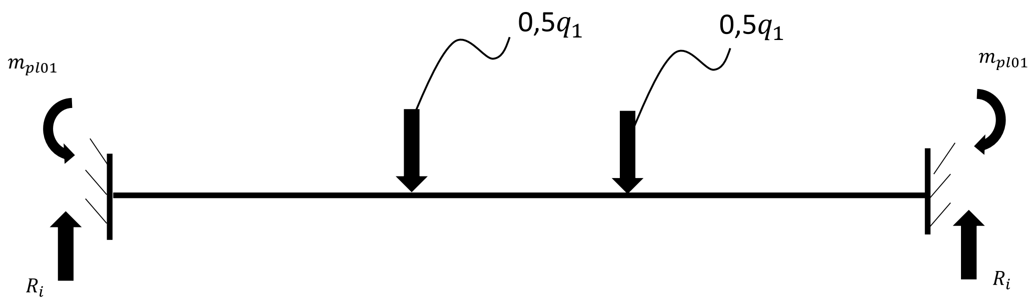
Loading (long. direction)

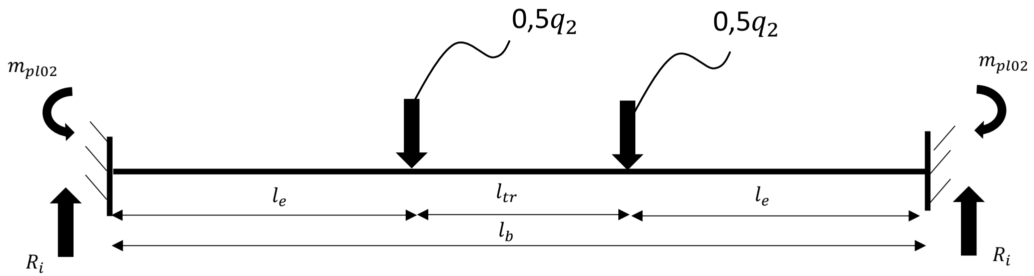


Load & Reaction forces		
q1	283,3	[kN/m]
0,5q1	141,7	[kN/m]
q2	156,4	[kN/m]
0,5q2	78,2	[kN/m]
Av	1714,8	[kN]
Bv	1264,5	[kN]

Measurements		
x	2,71	[m]
a	6,9	[m]
b	25,6	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)





Load & Reaction forces		
0,5q1	141,7	[kN/m]
0,5q2	78,2	[kN/m]
mpl01	287,2	[kNm]
mpl02	158,6	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]

Shear Force

Vz 1331,0 [kN]

Bending moment

Mx 2412,2 [kNm]

Torsion

Mxy,M	-218,4	[kNm]	(due to torsional moment at one end)
Mxy,q1	-96,1	[kNm]	(due to distributed load)
Mxx,q2	53,1	[kNm]	(due to distributed load)
Mxy,tot	-261,4	[kNm]	

Suspension force

Qyy 141,7 [kN]

Clamping moment

mxx,M	35,0	[kNm]	(due to torsional moment at one end)
mxx,q1	-115,9	[kNm]	(due to distributed load)
mxx,q2	64	[kNm]	(due to distributed load)
mxx,tot	-16,9	[kNm]	

LC 3b

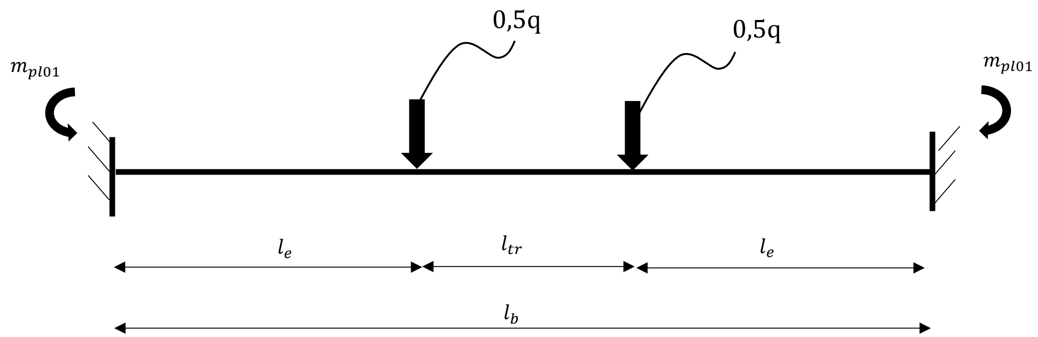
Cantilevers loaded

Loading (long. direction)

Load & Reaction forces		
q	202	[kN/m]
0,5q	101,0	[kN/m]
Av	102,6	[kN]
Bv	-1,6	[kN]

Measurements		
x	2,71	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	101,0	[kN/m]
mpl01	204,8	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]

Shear Force

Vz	1,6	[kN]

Bending moment

Mx	-47,8	[kNm]

Torsion

Mxy,M	-155,8	[kNm]	(due to torsional moment at one end)
Mxy,subtot	-155,8	[kNm]	

Suspension force

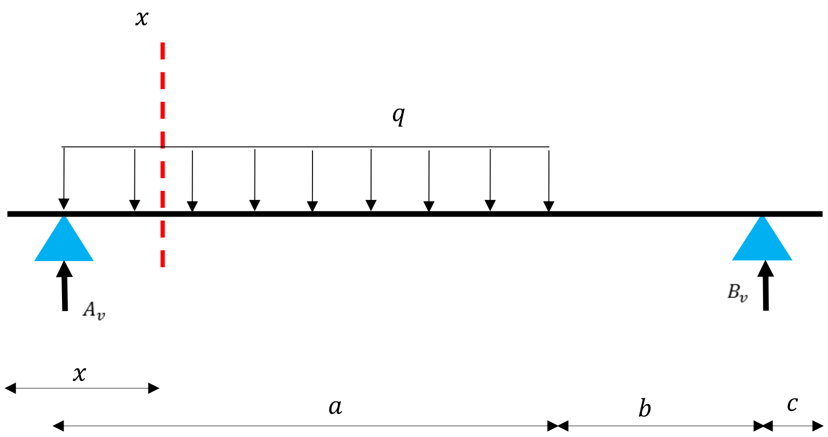
Qyy	0,0	[kN]

Clamping moment

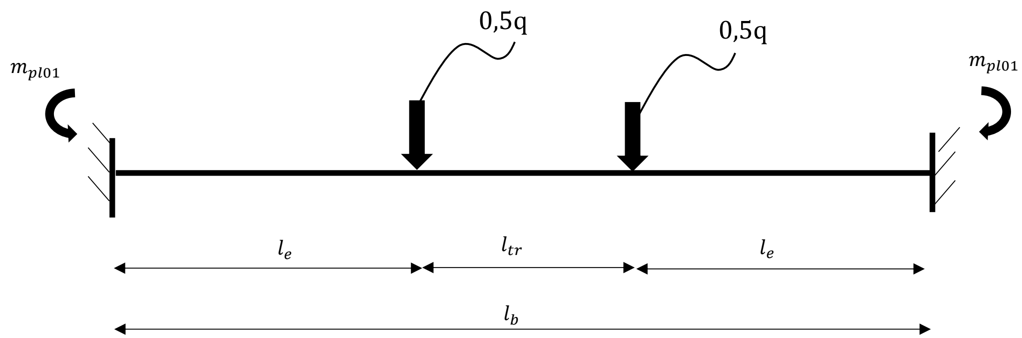
mxx,M	24,9	[kNm]	(due to torsional moment at one end)
mxx,subtot.	24,9	[kNm]	

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	202	[kN/m]
0,5q	101	[kN/m]
Av	1500,6	[kN]
Bv	923,4	[kN]

Measurements		
x	2,71	[m]
a	24	[m]
b	7,5	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
0,5q	101,0	[kN/m]
mpl01	204,8	[kNm]

Measurements		
lb	6,2	[m]
ltr	1,5	[m]
le	2,4	[m]

Shear Force

Vz 1327,9 [kN]

Bending moment

Mx 2418,3 [kNm]

Torsion

Mxy, alt	-3,2	[kNm]	(due to distributed load)
Mxy,subtot	-3,2	[kNm]	

Suspension force

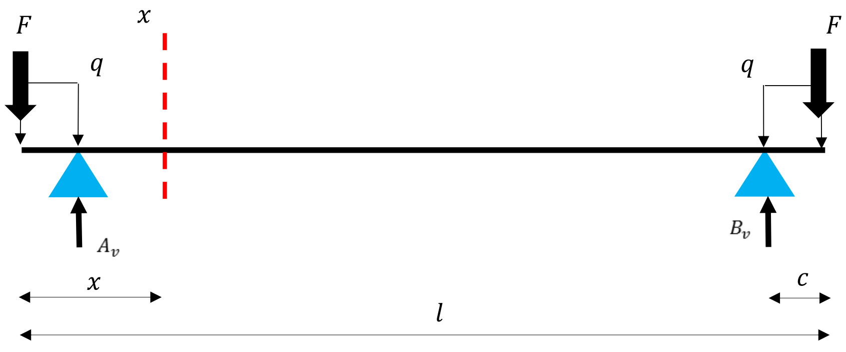
Qyy 101,0 [kN]

Clamping moment

mxx, alt	-4,2	[kNm]	(due to distributed load)
mxx,subtot.	-4,2	[kNm]	

LC 4

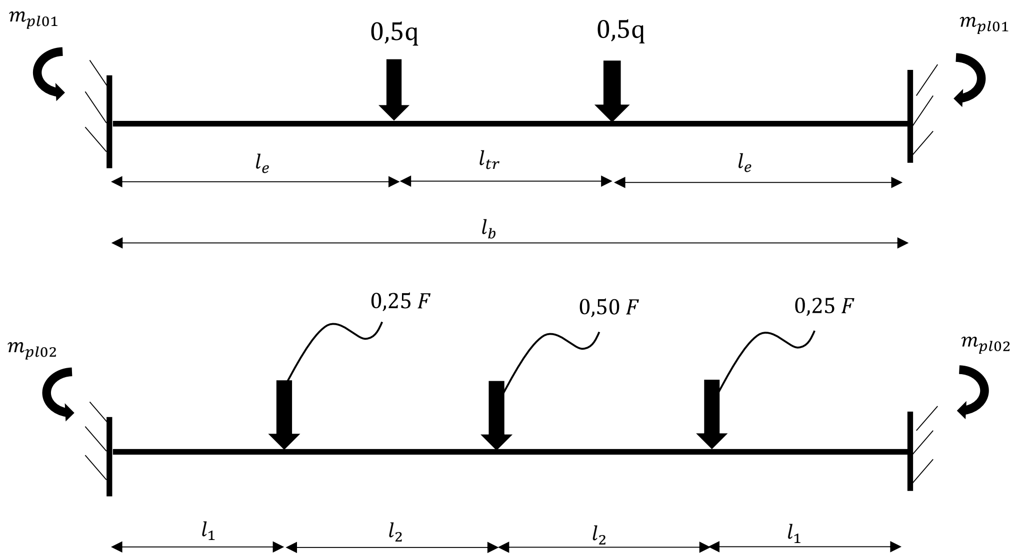
Loading (long. direction)



Load & Reaction forces		
q	202	[kN/m]
0,5q	101,0	[kN/m]
F	404	[kN]
0,5F	202	[kN]
Av	303,0	[kN]
Bv	303,0	[kN]

Measurements		
x	2,71	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	101,0	[kN/m]
0,25F	101,0	[kN]
mpl01	204,8	[kNm]
mpl02	247,9	[kNm]
MT	452,7	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]
l1	1,1	[m]
l2	2,0	[m]

Shear Force

Vz	0,0	[kN]

Bending moment

Mx	-252,5	[kNm]

Torsion

Mxy,M	-342,7	[kNm]	(due to torsional moment at both ends)
Mxy,tot	-342,7	[kNm]	

Suspension force

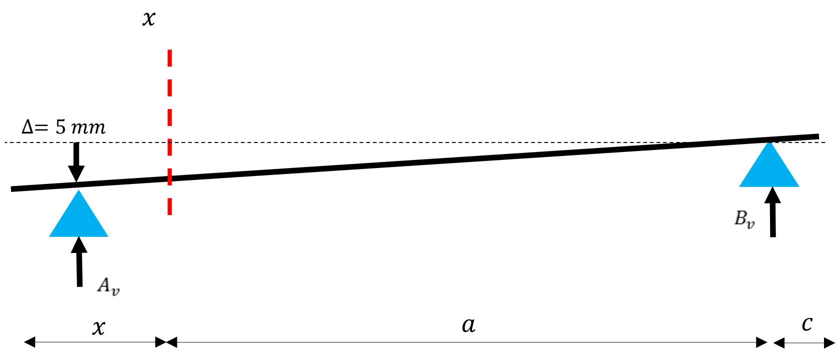
Qyy	0,0	[kN]

Clamping moment

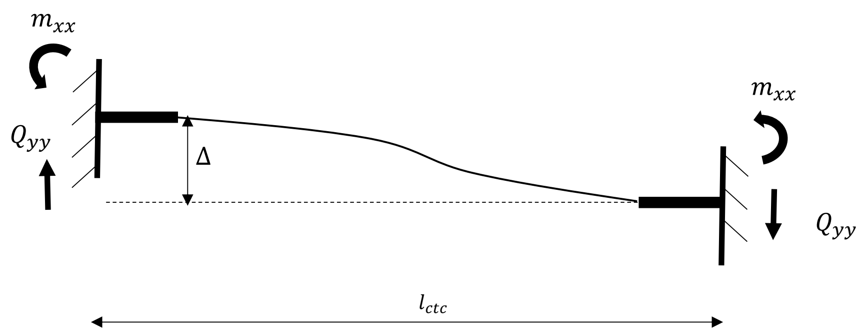
mxx,M	56,1	[kNm]	(due to torsional moment at both ends)
mxx,tot	56,1	[kNm]	

LC 6

Loading (long. direction)



Loading (transverse direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γ	1,2	[-]

Measurements		
x	2,71	[m]
a	29,8	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

Vz	0,0	[kN]

Suspension force

Qyy	-20,0	[kN]
Qyy,tot	-24,0	[kN]

Bending moment

Mx	0,0	[kNm]

Clamping moment

mxx,Δ	94,6	[kNm]
mxx,tot	113,5	[kNm]

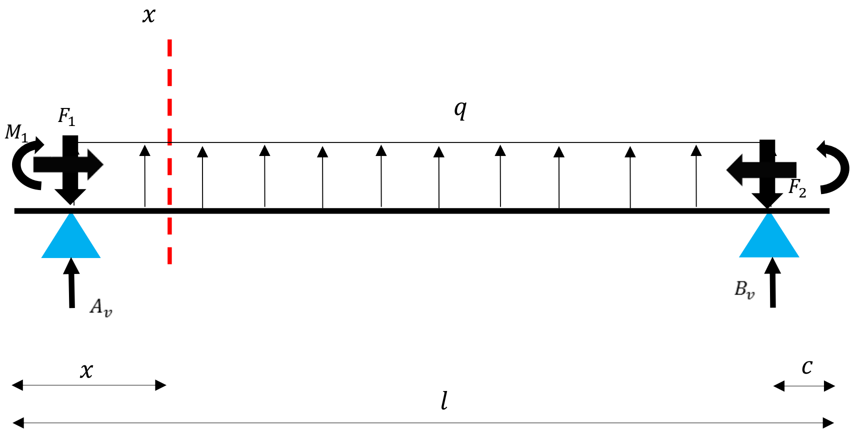
Torsion

Mxy,Δ	-298,5	[kNm]
Mxy,tot	-358,1	[kNm]

LC 8

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	2290	[kN]
F2	-22826	[kN]
M1	5045	[kNm]
q	-145,2	[kN]
Av	3	[kN]
Bv	3	[kN]

Measurements		
x	2,71	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

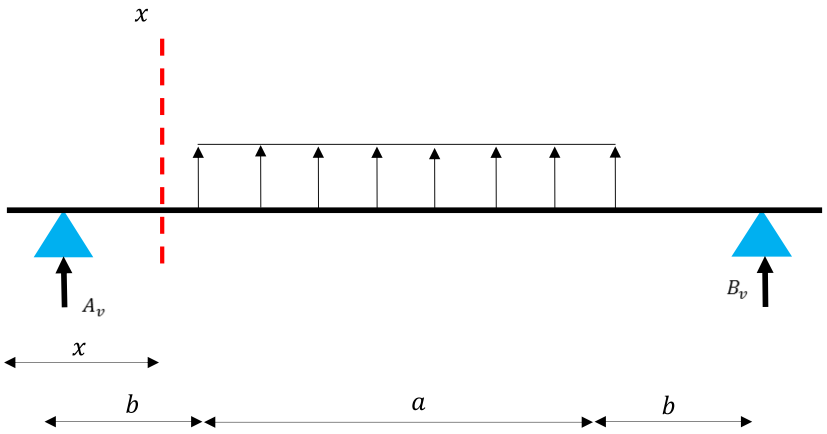
Vz -2038,6 [kN]

Bending moment

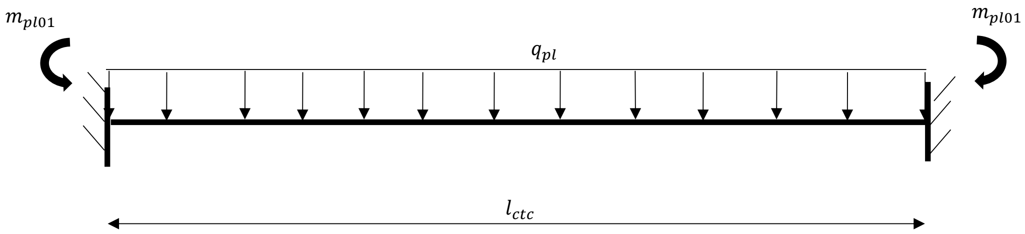
Mx 1346,7 [kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements

x	2,71	[m]
a	20,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces

qpl	-56,9	[kN/m]
mpl01	-118,6	[kNm]

Measurements

lctc	5,0	[m]

Torsion

Mxy,alt	48,5	[kNm]	(due to alternative load case)
Mxy,tot	48,5	[kNm]	

Suspension force

Qyy	-56,9	[kN]

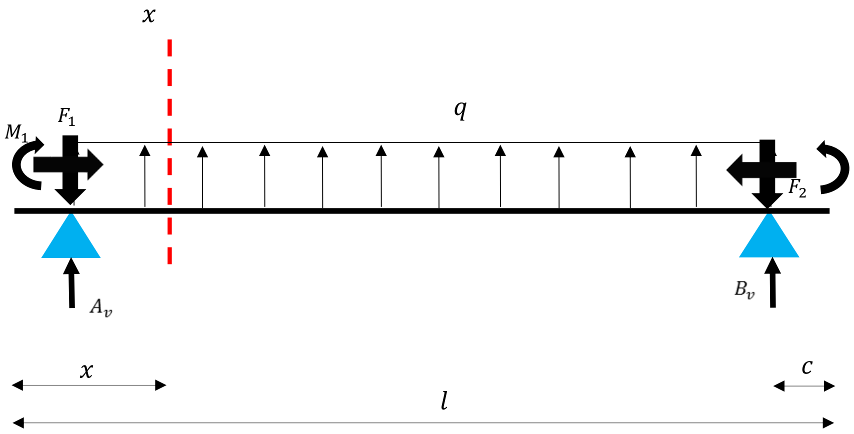
Clamping moment

mxx,alt	29,1	[kNm]	(due to alternative load case)
mxx,tot	29,1	[kNm]	

LC 9

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	2095	[kN]
F2	-20886	[kN]
M1	4616	[kNm]
q	-133	[kN]
Av	3	[kN]
Bv	3	[kN]
P∞/P0	0,915	[-]

Measurements		
x	2,71	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

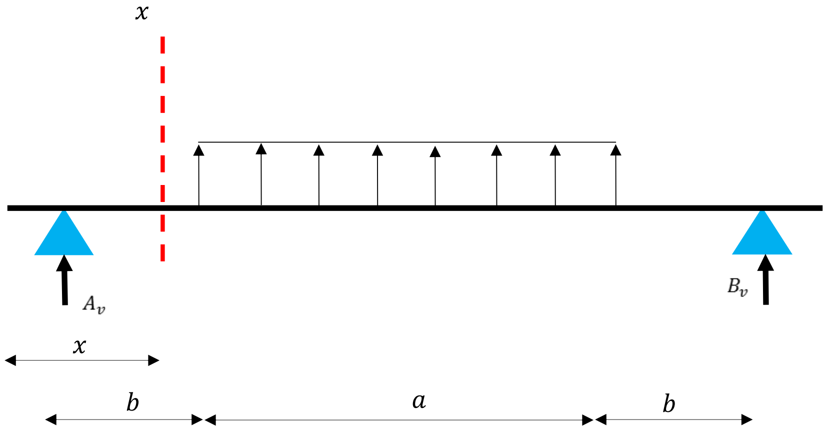
Vz -1865,3 [kN]

Bending moment

Mx 1232,2 [kNm]

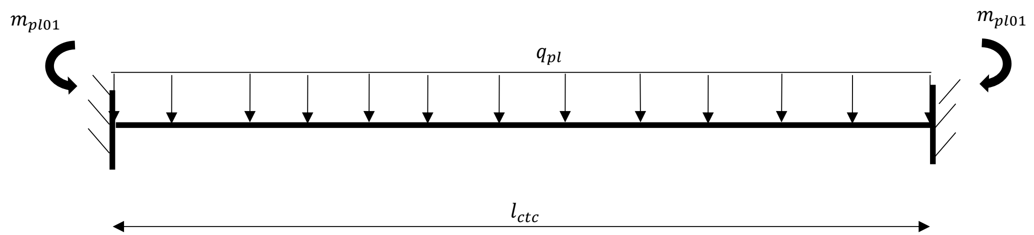
Floor loaded

Loading (long. direction)



Measurements		
x	2,71	[m]
a	20,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	-52,1	[kN/m]
mpl01	-108,5	[kNm]

Measurements		
lctc	5,0	[m]

Torsion

Mxy,alt	44,4	[kNm]	(due to alternative load case)
Mxy,tot	44,4	[kNm]	

Suspension force

Qyy	-52,1	[kN]

Clamping moment

mxx,alt	26,6	[kNm]	(due to alternative load case)
mxx,tot	26,6	[kNm]	

Appendix D – Reinforcement capacity (ULS)

1 Introduction

A load on the floor of the through bridge, due to for example a passing train, leads to torsion, shear force and suspension forces in the girder. The occurring sectional forces in the girder need to be transferred, even when the structure is cracked, from the floor into the girder. Therefore shear and longitudinal reinforcement is applied. During the design of the bridges different rules were valid for the application of reinforcement then nowadays.

According to VB 74 (7) the shear resistance of prestressed structures, not prone to fatigue, depend on the combined shear resistance of concrete and stirrups. For structures that are prone to fatigue, the shear resistance of concrete is neglected. The contribution of prestress to the shear resistance is something which is unrelated to this and is always taken into account.

Table D-1: Shear resistance calculations for prestressed structures according to VB 74 and the Eurocode

<table>
<tr><th></th><th>VB 74</th><th>Eurocode</th></tr>
<tr><td>Limit State</td><td>ULS</td><td>ULS</td></tr>
<tr><td rowspan="4">Prestressed structure without shear reinforcement</td><td>No fatigue</td><td>No fatigue/fatigue</td></tr>
<tr><td>$\tau_d = \tau_c + \frac{0{,}15 * P_\infty}{A_b}$</td><td rowspan="3">A. $\sigma_{xx} > f_{ctk;0,05}//\gamma_c$ Concrete shear capacity limited by flexural shear failure.
B. $\sigma_{xx} < f_{ctk;0,05}/\gamma_c$ Concrete shear capacity limited by shear tension failure.</td></tr>
<tr><td>Fatigue</td></tr>
<tr><td>$\tau_d = \frac{0{,}15 * P_\infty}{A_b}$</td></tr>
<tr><td rowspan="4">Prestressed structure with shear reinforcement</td><td>No fatigue</td><td>No fatigue/fatigue</td></tr>
<tr><td>$\tau_d = \tau_s + \tau_c + \frac{0{,}15 * P_\infty}{A_b}$</td><td rowspan="3">All loading transferred by the stirrups, no contribution of the concrete taken into account.</td></tr>
<tr><td>Fatigue</td></tr>
<tr><td>$\tau_d = \tau_s + \frac{0{,}15 * P_\infty}{A_b}$</td></tr>
</table>

Where:

$f_{ctk;0,05} = characteristic\ concrete\ tensile\ strength\ (N/mm^2)$
$\sigma_{xx} = tensile\ bending\ stresses\ (N/mm^2)$
$\tau_d = total\ shear\ resistance\ (N/mm^2)$
$\tau_c = concrete\ shear\ resistance\ (N/mm^2)$
$\tau_s = reinforcement\ shear\ resistance\ (N/mm^2)$
$P_\infty = working\ prestress\ at\ t = \infty\ (N)$

It becomes clear from Table D-1 that both concrete standards check the shear resistance at ultimate limit state. However a fundamental difference lies in the combination of concrete and reinforcement when determining the shear resistance. The VB allows such a combination depending on whether or not the structure is subjected to fatigue. The Eurocode on the other hand does not allow a combination. Whenever the shear resistance of the concrete is exceeded, the entire loading needs to be transferred by the stirrups.

The objective of this appendix is to verify if enough stirrups and longitudinal reinforcement has been applied in the through girder, according to the load models and calculation procedures of the Eurocode.

2 Concrete

The first step in the calculation procedure is to determine the shear and torsional resistance of the concrete and whether stirrups are necessary or not. Respectively section 6.2 and 6.3 of Eurocode 2 (16) are needed to determine this resistance.

All the calculations in this appendix will be explained using the maximum load combination (6.10b) of LM71 acting on bridge A. The considered section is 0,8d from the support and the accompanying loads can be found in appendix C.

2.1 Concrete shear resistance

The shear resistance of a structure without stirrups is limited by either shear tension failure or flexural shear failure. For a structure equipped with stirrups, the maximum shear resistance is limited by the capacity of the concrete diagonal.

2.1.1 Shear tension failure

When the tensile bending stresses remain limited ($< f_{ctk;0,05}/\gamma_c$), the shear tension failure mechanism becomes governing.

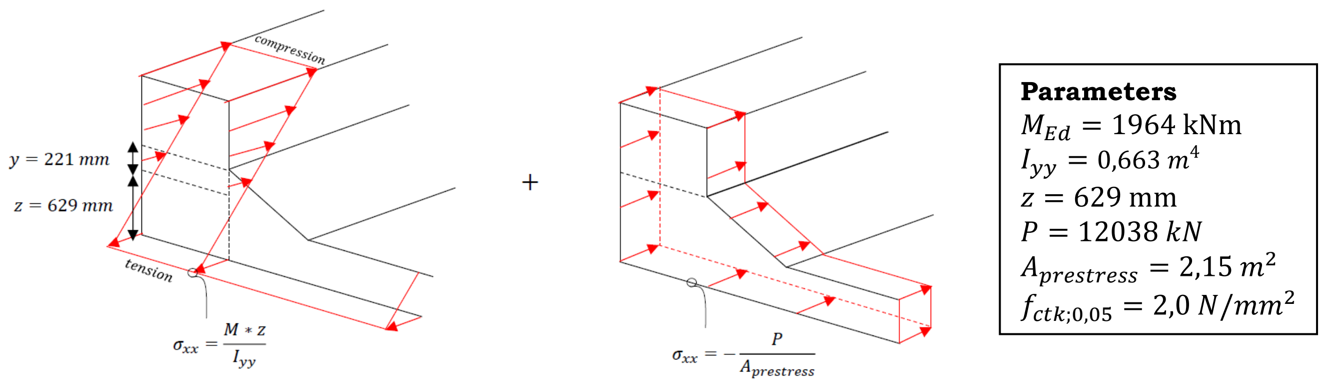


Figure D-1: Bridge A: Maximum tensile bending stress at the bottom of the girder

The maximum tensile bending stresses for LM71 are found in the bottom fibre:

$$\sigma_{xx} = \frac{1881 * 10^6 * 629}{0,663 * 10^{12}} - \frac{12038 * 10^3}{2,15 * 10^6} = -3,82\ N/mm^2$$

The stress is actually compressive and remains smaller than the tensile strength:

$$\sigma_{xx} < \frac{f_{ctk;0.05}}{\gamma_C} = -2,52 < 1,33\ N/mm^2$$

Hence shear tension failure is governing:

$$V_{Rd,c} = \frac{I_{yy} * b}{S} * \sqrt{(f_{ctd})^2 + \propto_l * \sigma_{cp} * f_{ctd}} \qquad [\text{ D.1 }]$$

Where:

$\propto_l = 1{,}0\ for\ this\ type\ of\ prestress\ system$

$f_{ctd} = f_{ctk;0{,}05}/\gamma_c\ (N/mm^2)$

$I_{yy} = moment\ of\ inertia\ of\ the\ girder\ (mm^4)$

$S = statical\ moment\ of\ area\ above\ the\ neutral\ axis\ (mm^3)$

$\sigma_{cp} = compressive\ stress\ due\ to\ prestress\ (N/mm^2)$

Table D-2: Sectional and material properties of the through girder

Parameter	Value
h_{girder}	$1{,}75\ m$
b_{girder}	$0{,}9\ m$
f_{ctd}	$1{,}33\ N/mm^2$
f_{ck}	$35\ N/mm^2$
f_{cd}	$23{,}3\ N/mm^2$

Parameters:

$$\sigma_{cp} = \frac{P}{A} < 0{,}2 * f_{cd} = \frac{12038 * 10^3}{2{,}15 * 10^6} = 5{,}60 > 4{,}66\ N/mm^2\ (use\ 4{,}66\ N/mm^2)$$

$$I_{yy} = \frac{1}{12} * b_{girder} * h_{girder}{}^3 = 0{,}402\ m^4$$

$$S = (h_{girder} - z_{neutral\ axis})^2 * 0{,}5 * b_{girder} = 0{,}565\ m^3$$

Eventually the shear tension failure resistance of the girder goes to:

$$V_{Rd,c} = \frac{0{,}402 * 10^{12} * 900}{0{,}565 * 10^9} * \sqrt{(1{,}33)^2 + 1{,}0 * 4{,}66 * 1{,}33} = 1806\ kN$$

2.1.2 Flexural shear failure

When the tensile bending stresses grow larger than the concrete tensile strength ($> f_{ctk;0,05}/\gamma_c$), the flexural shear failure mechanism becomes governing. This shear resistance is expressed by equation [D.2] with a minimum expressed by equation [D.3].

$$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right] * b * d \qquad [\ D.2\ ]$$

$$V_{Rd,c} = \left(v_{min} + k_1 * \sigma_{cp}\right) * b * d \qquad [\ D.3\ ]$$

Table D-3: Applied reinforcement in bridge A at 0,8d & 0,5L

Type of reinforcement	0,8d	0,5L
Outer stirrup	$\emptyset 16 - 150$	$\emptyset 16 - 150$
Inner stirrup	$\emptyset 12 - 150$	$\emptyset 12 - 150$
Longitudinal reinforcement	$14\emptyset 16$	$8\emptyset 12$

Parameters:

$C_{Rd,c} = 0,18/\gamma_c = 0,12$ (according to the Dutch National Annex (3))

$d = h - \emptyset_{stirrup} - 0,5 * \emptyset_{flexural} - c = 1691\ mm$

$k = 1 + \sqrt{200/d} = 1,34 \leq 2,0$

$$\rho_l = \frac{A_{sl}}{b * d} = \frac{0,25 * \pi * 14 * 16^2}{900 * 1691} = 0,0018 < 0,02$$

$v_{min} = 0,035 * k^{\frac{3}{2}} * f_{ck}^{\frac{1}{2}} = 0,32$ (according to the Dutch National Annex (3))

$k_1 = 0,15$ (according to the Dutch National Annex (3))

The flexural shear resistance goes to:

$$V_{Rd,c} = \left[0,12 * 1,34 * (100 * 0,0018 * 35)^{\frac{1}{3}} + 0,15 * 4,66\right] * 900 * 1691 = 1516\ kN$$

With a minimum of:

$$V_{Rd,c} = (0,32 + 0,15 * 4,66) * 900 * 1691 = 1551\ kN$$

The eventual flexural shear resistance is 1551 kN. However for this particular example the shear tension resistance remains governing.

2.1.3 Maximum shear resistance

A concrete structure can be strengthen by applying stirrups. In case a beam is loaded with a shear force, a compressive diagonal will form which intersects a number of stirrups. This internal force distribution can be approximated by the truss analogy.

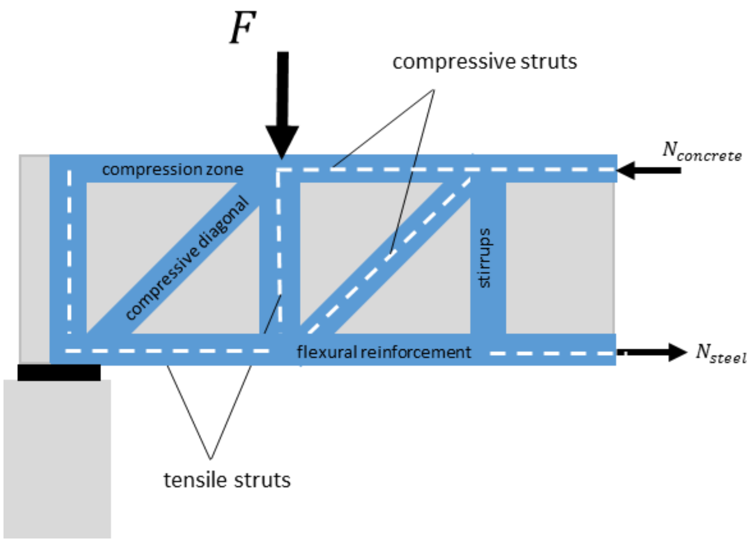


Figure D-2: Truss analogy

The analogy assumes an equilibrium of compressive and tensile struts. The shear and longitudinal reinforcement form tensile struts, whereas the diagonal and the compression zone form compressive struts. Before the stirrups get to yield, the compressive capacity of the diagonal is reached, causing a brittle failure mechanism. This paragraph focuses on determining the maximum shear resistance based on the capacity of the compressive diagonal.

The maximum shear resistance is given by equation [D.4]:

$$V_{Rd,max} = \propto_{cw} * b * z * v_1 * f_{cd}/(\cot\theta + \tan\theta) \quad [\text{ D.4 }]$$

Parameters:

$\propto_{cw} = \left(1 + \frac{\sigma_{cp}}{f_{cd}}\right) = 1{,}20$ (according to the Dutch National Annex (3))

$z = 0{,}9 * d = 1522\ mm$

$v_1 = v = 0{,}6 * \left[1 - \frac{f_{ck}}{250}\right] = 0{,}52$ (according to the Dutch National Annex (3))

$\theta = 21{,}8°$

The angle of the compressive diagonal, expressed by θ, may be chosen between 21,8° and 45°. Choosing a small angle will decrease the load on the stirrups and increase the load on the longitudinal reinforcement. A large angle has the opposite effect on the reinforcement. Because for the example at 0,8d the stirrups are governing over the longitudinal reinforcement, an angle of 21,8° is chosen.

The maximum shear resistance, based on the capacity of the diagonal is:

$$V_{Rd,max} = 1,20 * 900 * 1522 * 0,52 * 23,3/(\cot 21,8 + \tan 21,8) = 6867\ kN$$

2.2 Concrete torsion resistance

Resistance against torsion of a structure without shear reinforcement is limited by the torsion resistance of concrete. For a structure with shear reinforcement, the torsion resistance is limited by the capacity of the compressive diagonal.

2.2.1 Torsion resistance

The solid rectangular profile of the girder is simplified to a thin walled tube. The thickness of the walls is established by dividing the cross-sectional area by the perimeter.

$$t_{ef} = \frac{A}{u} = \frac{1750 * 900}{(2 * 1750 + 2 * 900)} = 297\ mm > 2(h_{girder} - d) = 118\ mm$$

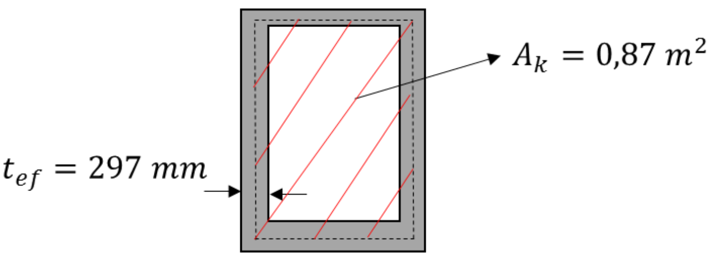


Figure D-3: Girder simplified to thin walled tube

A_k is the area enclosed by the wall centre-lines:

$$A_k = (h_{girder} - t_{ef}) * (b_{girer} - t_{ef}) = 0{,}87\ m^2$$

The torsion resistance of the concrete is expressed by [D.5]:

$$T_{Rd,c} = 2 * t_{ef} * f_{ctd} * A_k \qquad [\ D.5\ ]$$

Using the previously defined parameters, the resistance goes to:

$$T_{Rd,c} = 2 * 297 * 1{,}33 * 2 * 0{,}87 * 10^6 = 693\ kNm$$

2.2.2 Maximum torsion resistance

Like the maximum shear resistance, the maximum resistance against torsion is limited by the capacity of the compressive diagonal.

$$T_{Rd,max} = 2 * \nu * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta \qquad [\ D.6\ ]$$

Using the previously defined parameters, the maximum resistance goes to:

$$T_{Rd,max} = 2 * 0{,}52 * 1{,}20 * 23{,}3 * 0{,}87 * 10^6 * 297 * \sin 21{,}8 * \cos 21{,}8 = 2612\ kNm$$

2.3 Total concrete resistance

To determine, for a structure subjected to shear and torsion, whether or not stirrups should be applied, equation [D.7] is used.

$$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}} \leq 1{,}0 \qquad [\ D.7\ ]$$

For a value below 1.0, a minimum amount of stirrups suffices (see chapter 3.5). A value above 1.0 requires that all the loading is transferred by shear reinforcement. With the application of stirrups, the capacity of the compressive diagonal needs to be checked as well. This is done with equation [D.8].

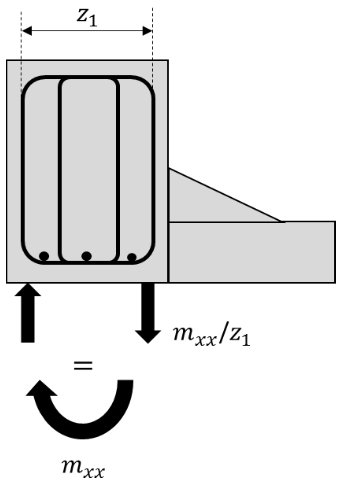
$$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed}}{V_{Rd,max}} \leq 1{,}0 \qquad [\ D.8\ ]$$

Before one can continue, it is important to understand that for a through bridge the shear force consists of three components. The shear force in longitudinal direction, the suspension force and the clamping moment. All three generate a vertical load in the girder, which needs to be taken into account when assessing the total shear resistance of the concrete.

The suspension force is a load per meter (due to the strip method), which is directly added to shear force in longitudinal direction.

The clamping moment needs to be split into two forces with an internal lever. Figure D-4 shows that the clamping moment is split over the two legs of the outer stirrup. The distance z_1 equals:

$$z_1 = b - 2 * \left(c + 0.5 * \varnothing_{stirrup}\right) = 814\ mm$$



Loads due to LM71
$V_{Ed} = 1217$ kN
$m_{xx} = 172\ kNm/m$
$Q_{yy} = 148\ kN/m$
$T_{Ed} = -548\ kNm$
$M_{Ed} = 1881\ kNm$

Figure D-4: The clamping moment split into shear force

The total shear force for LM71 becomes:

$$V_{Ed,tot} = V_{Ed} + Q_{yy} + \frac{m_{xx}}{z_1} = 1576\ kN$$

Check whether or not stirrups are necessary:

$$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed,tot}}{V_{Rd,c}} = \frac{548}{693} + \frac{1576}{1806} = 1{,}66 > 1{,}00 \rightarrow \quad Apply\ stirrups$$

Because stirrups are applied, the capacity of the concrete diagonal needs to be checked. Since the diagonal spreads over a horizontal length of $z * \cot\theta$, the clamping moment and suspension force (which are per meter) are multiplied with this horizontal length.

$$V_{Ed,tot} = V_{Ed} + \left(Q_{yy} + \frac{m_{xx}}{z_1}\right) * z * \cot\theta = 2583\ kN$$

Check if the concrete diagonal has sufficient capacity:

$$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,c}} = \frac{548}{2612} + \frac{2583}{6867} = 0{,}59 < 1{,}00 \rightarrow \quad Diagonal\ has\ sufficient\ capacity$$

3 Stirrups

The combination of shear force, torsion, suspension force and the clamping moment requires the application of stirrups. This chapter elaborates on the calculation procedure that determines the required amount of reinforcement for each individual design load.

3.1 Shear force

Shear force generates a uniform shear stress in the girder. Meaning an equal amount of shear stress goes through the inner and the outer stirrup. A distribution of 25% shear stress per stirrup leg is assumed.

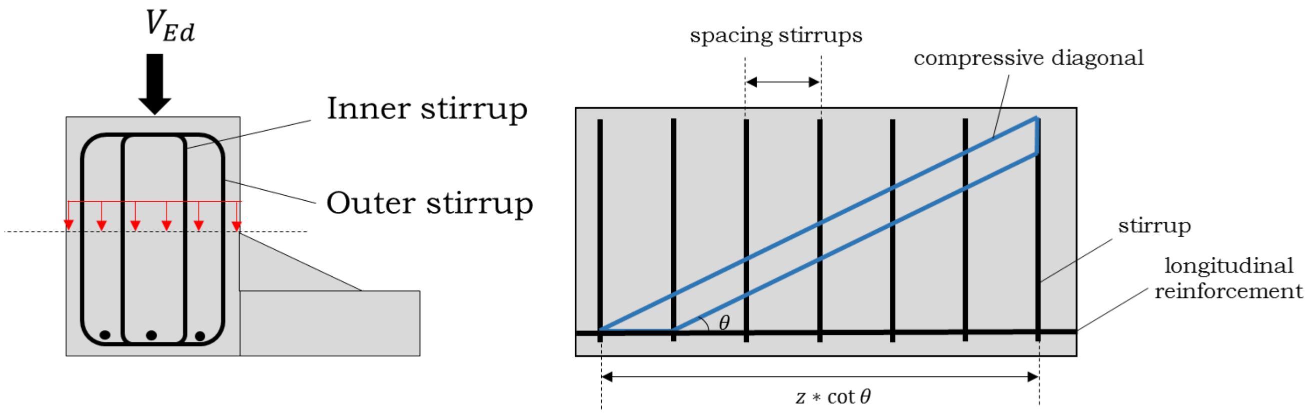


Figure D-5: Shear stress in the girder (left) Compressive diagonal intersects multiple stirrups (right)

When a concrete structure with shear reinforcement is loaded, an equilibrium will be set between the compressive diagonal and the stirrups. The compressive diagonal intersects a number of stirrups (right Figure D-5). This effect is taken into account by dividing the shear force with the horizontal length of the diagonal ($z * \cot\theta$).

$$A_V = \frac{V_{Ed}}{z * \cot\theta * f_{ywd}} \qquad [\text{ D.9 }]$$

Parameters:

$$f_{ywd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1{,}15} = 435\ N/mm^2$$

For the considered example the required amount of outer/inner stirrup is:

$$A_v = \frac{1217 * 10^3}{1522 * \cot 21{,}8 * 435} = 0{,}74\ mm^2/mm$$

$$A_v = 0{,}25 * 0{,}74 = 0{,}18\ mm^2/mm\ (per\ leg)$$

3.2 Torsion

The shear stress due to torsion, can be found by simplifying the rectangular cross-section to a thin walled tube. The shear stress in the walls needs to be taken up by the outer stirrup. Equation [D.10] defines the shear stress in the wall. By taking the yield strength and the number of stirrups intersected by the diagonal into account, the equation can be rewritten to [D.11].

$$\tau = \frac{T_{Ed}}{2 * t_{ef} * A_k} \qquad [\text{ D.10 }]$$

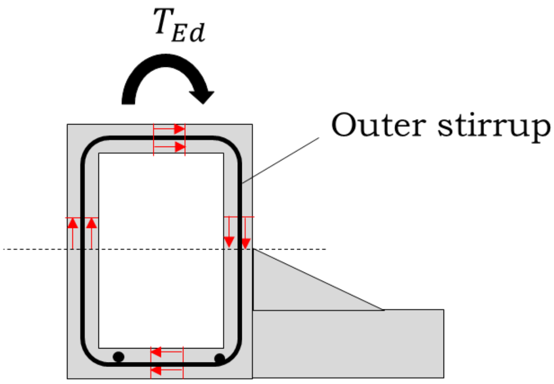


Figure D-6: Shear stress in the girder due to torsion

$$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta} \qquad [\text{ D.11 }]$$

For the considered example the required amount of outer stirrup is:

$$A_T = \frac{548 * 10^6}{2 * 0{,}87 * 10^6 * 435 * \cot 21{,}8} = 0{,}29\ mm^2/mm\ (per\ leg)$$

3.3 Suspension force

The suspension force acts at the upper side of the connection between the floor and the girder. At ultimate limit state, the girder could crack, meaning the stirrups should be able to transfer 100% of the suspension force from the floor into the girder. Initially the suspension force is assumed to be transferred by the outer stirrup leg. If it turns out there is insufficient capacity, the capacity of the inner stirrup leg is exploited as well.

NS-guideline 1015 (17) used to be the governing guideline for designing through bridges. Because a great part of the load in the suspension reinforcement is due to mobile loads, the reinforcement is more sensitive to fatigue. Besides crack widths in the concrete will be more difficult to control. As a measurement the guideline recommends to design suspension reinforcement with a characteristic yield strength of 220 MPa, even though stirrups of FeB500 are applied.

$$A_Q = \frac{Q_{yy}}{1000 * f_{yk}} \qquad [\text{ D.12 }]$$

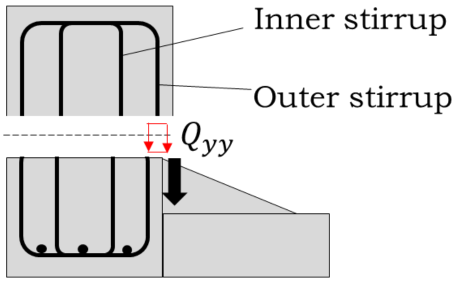


Figure D-7: Vertical normal stress in the girder due to a suspension force

The required amount of outer stirrup (per leg) is:

$$A_Q = \frac{148 * 10^3}{1000 * 220} = 0{,}67\ mm^2/mm\ (per\ leg)$$

3.4 Clamping moment

The clamping moment acts at the heart of the girder. Dividing the moment by z_1 delivers a vertical force in the leg of the outer stirrup. Like the suspension force, this vertical component needs to be transferred entirely by the outer stirrup leg. And if the outer stirrup leg has insufficient capacity, a part of the load transfer is done by the inner stirrup leg. Ultimately equation [D.13] is used to determine the required amount of reinforcement.

$$A_m = \frac{m_{xx}/z_1}{1000 * f_{yk}} \qquad [\text{ D.13 }]$$

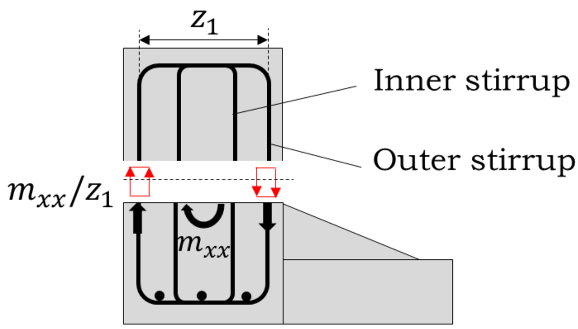


Figure D-8: Vertical normal stress in the girder due to a clamping moment

For the considered example the required amount of outer stirrup is:

$$A_m = \frac{172 * 10^6/814}{1000 * 220} = 0{,}96\; mm^2/mm\;(per\;leg)$$

3.5 Minimum reinforcement

When a value smaller than 1.0 is found for the combination of shear and torsion (equation [D.7]) , a minimum amount of reinforcement needs to be applied. Section 9.2.1.1 in Eurocode 2 (16) holds a formula that is used to determine this minimum amount.

$$\rho_{w,min} = (0{,}08 * \sqrt{f_{ck}})/f_{yk} \qquad [\text{ D.14 }]$$

With the knowledge that the characteristic yield strength of the reinforcement is 500 MPa, the minimum amount of reinforcement goes to:

$$\rho_{w,min} = (0{,}08 * \sqrt{35})/500 = 0{,}00095$$

$$A_{s,min} = \rho_{w,min} * b = 0{,}85\; mm^2/mm$$

$$A_{s,min} = 0{,}25 * 0{,}85 = 0{,}21\; mm^2/mm\;(per\;leg)$$

4 Longitudinal reinforcement & Prestress

The truss analogy explains in what way a shear force can cause a tensile force in the longitudinal reinforcement. The shear force generates compression in the diagonal, which can be split into a horizontal and vertical component. These components have to make equilibrium with reactional tensile forces. Respectively the horizontal and vertical tensile forces are taken up by the longitudinal and shear reinforcement.

Prestress introduces a compressive stress in the girder and thus in the longitudinal reinforcement. This enlarges the resistance of the reinforcement against tensile forces. Additionally prestressing tendons are able to take up a part of the tensile forces themselves, when there present in a tension zone. Initially the tendons are subjected to an average stress ($\sigma_{p\infty}$), but when the tendons are present in a tension zone they are able to take up an additional force until they yield (f_{pd}).

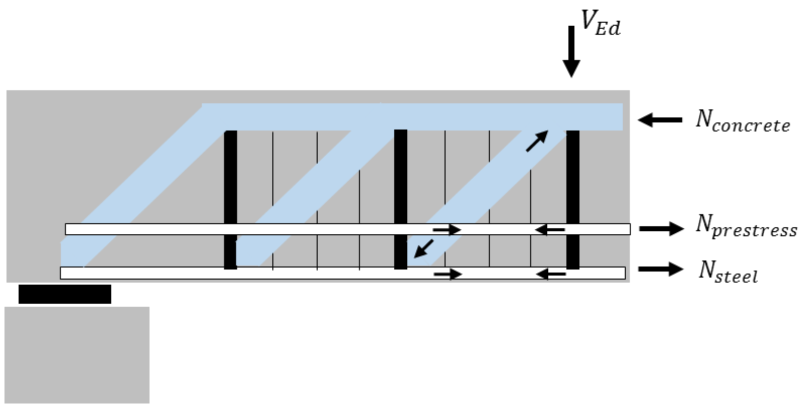


Figure D-9: Shear force causing a tensile force in the longitudinal reinforcement and tendons

Let's consider a section in bridge A at 0,8d from the support. Figure D-10 present the strains and the internal forces for this considered section. Besides a shear force, a bending moment is present in the section. The height of the compression zone (x) and the compressive concrete strain in the top fibre (ε_c) are the two unknown variables.

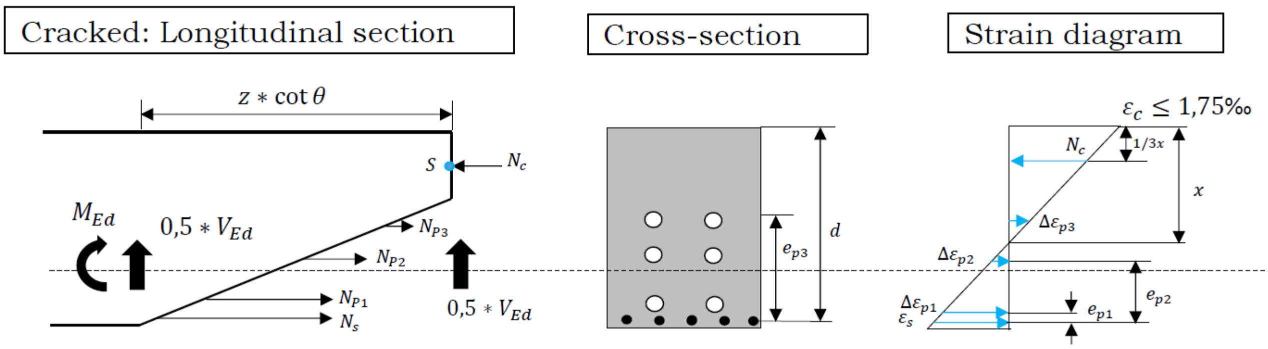


Figure D-10: Longitudinal section girder (left), cross-section (middle) and strain diagram (right)

Before one can start, a magnitude for the shear force and bending moment need to be established. In agreement with chapter 2.3 the bending moment equals 1881 kNm.

The total shear force is a combination of the suspension force, clamping moment, shear and torsion. The latter one needs to be rewritten in the form of a shear force:

$$V_{T_{Ed}} = \frac{T_{Ed} * z}{2 * A_k} \quad [\text{ D.15 }]$$

Total shear force:

$$V_{Ed,tot} = V_{Ed} + \left(Q_{yy} + \frac{m_{xx}}{z_1}\right) * z * \cot\theta + \frac{T_{Ed} * z}{2 * A_k} \quad [\text{ D.16 }]$$

With the values presented in chapter 2.3 the total shear force goes to 3060 kN.

With the knowledge in the reader prestressed concrete (23), a set of equations is derived for which the height of the compression zone and the maximum strain can be determined. The first step is to express the strain in the reinforcement and prestress in terms of x and ε_c.

$$\varepsilon_s = \frac{\varepsilon_c}{x} * (d - x)$$

$$\Delta\varepsilon_{p1} = \frac{\varepsilon_c}{x} * (d - x - e_{p1})$$

$$\Delta\varepsilon_{p2} = \frac{\varepsilon_c}{x} * (d - x - e_{p2})$$

$$\Delta\varepsilon_{p3} = \frac{\varepsilon_c}{x} * (d - x - e_{p3})$$

With the Young Modulus of the reinforcement and prestress steel, the strains are converted into stresses. Subsequently the stresses are multiplied with the cross-sectional areas to find the corresponding forces.

When the concrete is loaded in tension, a part is taken up by the present tendons. But the transfer of forces completely depends on the actual bond factor between the prestress and concrete, which is expressed as:

$$\xi_1 = \sqrt{\xi * \frac{\emptyset_{longitudinal}}{\emptyset_{prestress}}}$$

Where:

$\emptyset_{prestress} = 1{,}6\sqrt{A_p}$ (according to Eurocode 2)

$\xi = 0{,}5$ (according to table 6.2 in Eurocode 2)

The forces can be expressed as:

$$N_s = A_s * E_s * \varepsilon_s$$

$$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$$

$$N_{p2} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p2} * E_p * \xi_1)$$

$$N_{p3} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p3} * E_p * \xi_1)$$

$$N_c = 0{,}5 * \frac{\varepsilon_c}{1{,}75 * 10^{-3}} * f_{cd} * b * x$$

An equilibrium of horizontal forces is required:

$$\sum H = N_c - N_s - N_{p1} - N_{p2} - N_{p3} = 0$$

Additionally an equilibrium of moments is required. Note: Half the shear force is being taken into account at the start and end of the compressive diagonal (Figure D-10).

$$\sum M_{|S} = M_{Ed} + 0{,}5 * V_{Ed} * z * \cot\theta - N_s * \left(d - \frac{1}{3}x\right) - N_{p1} * \left(d - \frac{1}{3}x - e_{p1}\right)$$

$$-N_{p2} * \left(d - \frac{1}{3}x - e_{p2}\right) - N_{p3} * \left(d - \frac{1}{3}x - e_{p3}\right) = 0$$

The two equations for equilibrium are solved and a solution for x and ε_c is found. Based on the maximum strain, three stress and strain diagrams are possible as depicted by Figure D-11:

1. Elastic stage ($\varepsilon_{c,top} \leq 1{,}75‰$): A strain of $1{,}75‰$ causes a compressive stress equal to the design strength of concrete (f_{cd}). The course of the stress diagram is linear.
2. Elastic-plastic stage ($1{,}75‰ < \varepsilon_{c,top} \leq 3{,}5‰$): Up to a strain of $1{,}75‰$ the course of the stress diagram remains linear, beyond this point the concrete will start to deform plastically and the maximum stress is equal to the design strength of the concrete (f_{cd}). Note: A strain of $3{,}5‰$ is the value for a structure which is about to fail. The height of the compression zone is limited by the Eurocode and rotational capacity should be ensured. But because the prestress and reinforcement yield at a lower strain, the yield strain of steel is maintained as the maximum allowable.
3. Compression only ($x > h$): When the height of the compression zone exceeds the height of the cross-section, only compressive stresses are present in the girder. Naturally the unity check for longitudinal reinforcement goes to zero.

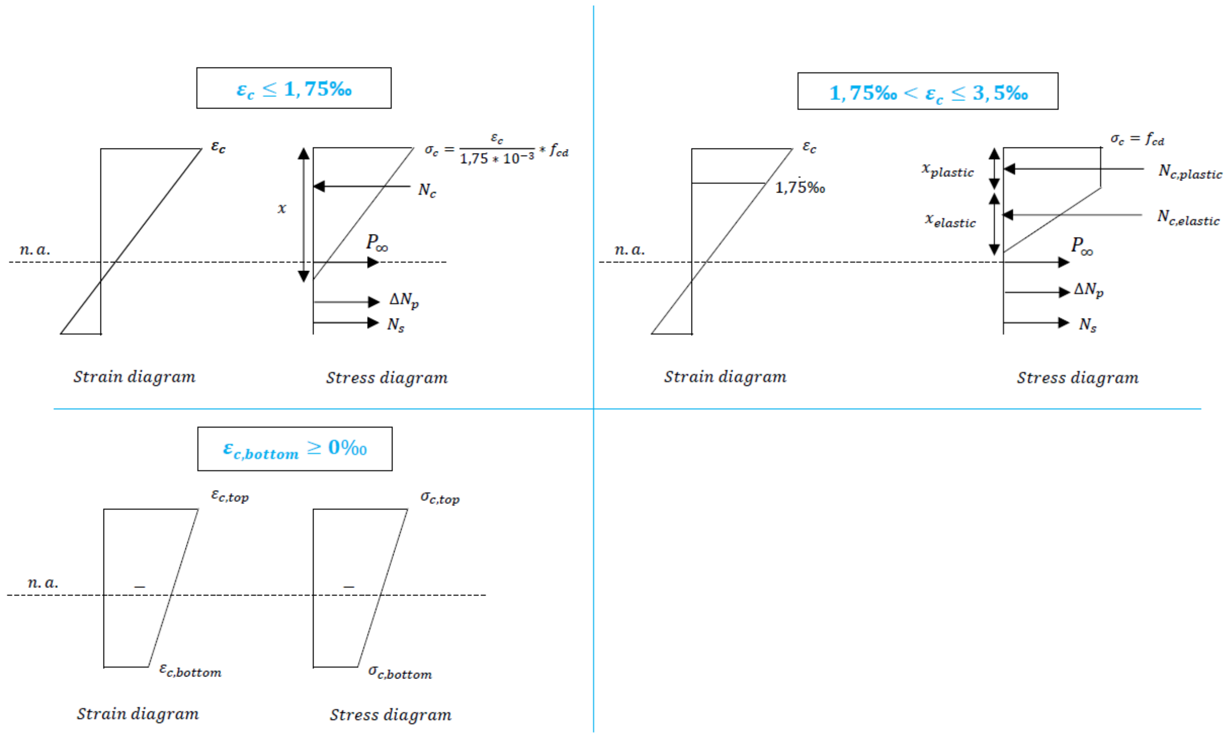


Figure D-11: Stress and strain diagrams for possible solution of x and ε_c

For the considered example the height of the compression zone goes to 1263 mm and the concrete strain to $1{,}61 * 10^{-3}$. Meaning the girder is loaded within the elastic limits of the concrete.

Parameters
$A_p = 1900\ mm^2$
$A_{flex} = 14\text{Ø}16$
$E_s = 210\ GPa$
$E_p = 205\ GPa$
$\xi_1 = 0{,}34$
$P_\infty = 12038\ kN$
$d = 1691\ mm$
$e_{p1} = 189\ mm$
$e_{p2} = 584\ mm$
$e_{p3} = 979\ mm$
$f_{pd} = 1522\ N/mm^2$

Longitudinal reinforcement:

$$N_s = A_s * E_s * \frac{\varepsilon_c}{x} * (d - x) = 322\ kN$$

Prestressing steel:

$$N_{p1} = 2 * A_p * \left(\frac{P_\infty}{6A_p} + \frac{\varepsilon_c}{x} * (d - x - e_{p1}) * E_p * \xi_1\right) = 4093\ kN$$

$$N_{p2} = 2 * A_p * \left(\frac{P_\infty}{6A_p} + \frac{\varepsilon_c}{x} * (d - x - e_{p2}) * E_p * \xi_1\right) = 3960\ kN$$

$$N_{p3} = 2 * A_p * \left(\frac{P_\infty}{6A_p} + \frac{\varepsilon_c}{x} * (d - x - e_{p3}) * E_p * \xi_1\right) = 3827\ kN$$

Ultimately the unity check's for the longitudinal reinforcement and prestressing steel go to:

$$UC_{reinf.} = \frac{N_s}{A_s * f_{ywd}} = \frac{322}{0{,}25 * \pi * 16^2 * 14 * 435 * 10^{-3}} = 0{,}26$$

$$UC_{prestr.} = \frac{\max(N_{p1}; N_{p2}; N_{p3})}{2 * A_p * f_{pd}} = \frac{4093}{2 * 1900 * 1522 * 10^{-3}} = 0{,}71$$

5 Results

Bridge A is equipped with an outer and inner stirrup, whereas bridge B only has one stirrup. Based on this configuration, the NS-guideline 1015 defines a number of zones in the girder, which have a specific load transferring function.

- Zone I: Shear and torsion
- Zone II: Shear
- Zone III: Shear, torsion and suspension.

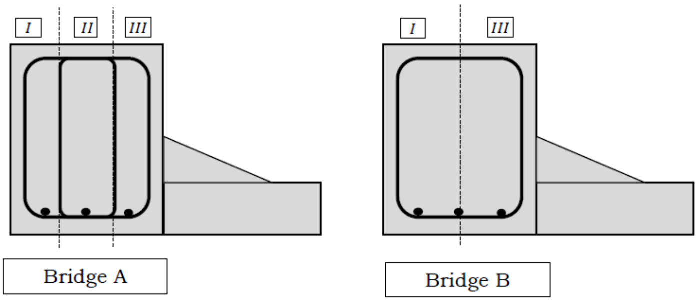


Figure D-12: Shear reinforcement zones for bridge A (left) and bridge B (right)

To find the combined amount of shear reinforcement, the maximum in each zone needs to be determined:

Zone I
$$= 0{,}85 * (0{,}25 * A_V + A_T)$$
$$= 0{,}25 * A_V$$
$$= A_T$$
$$= 0{,}25 * A_{min}$$

Zone II
$$= 0{,}25 * A_V$$
$$= 0{,}25 * A_{min}$$

Zone III
$$= 0{,}80 * (0{,}25 * A_V + A_T + A_m + A_Q)$$
$$= 0{,}25 * A_V + A_m + A_Q$$
$$= A_T + A_m + A_Q$$
$$= 0{,}85 * (0{,}25 * A_V + A_T)$$
$$= 0{,}25 * A_{min}$$

Where:

$A_V = required\ amount\ of\ stirrups\ due\ to\ shear\ force$
$A_T = required\ amount\ of\ stirrups\ due\ to\ torsion$
$A_m = required\ amount\ of\ stirrups\ due\ to\ clamping\ moment$
$A_Q = required\ amount\ of\ stirrups\ due\ to\ suspension\ force$
$A_{min} = minimum\ amount\ of\ stirrups$

Figure D-13 shows the amount of shear and longitudinal reinforcement that is applied in the girder of bridge A and B.

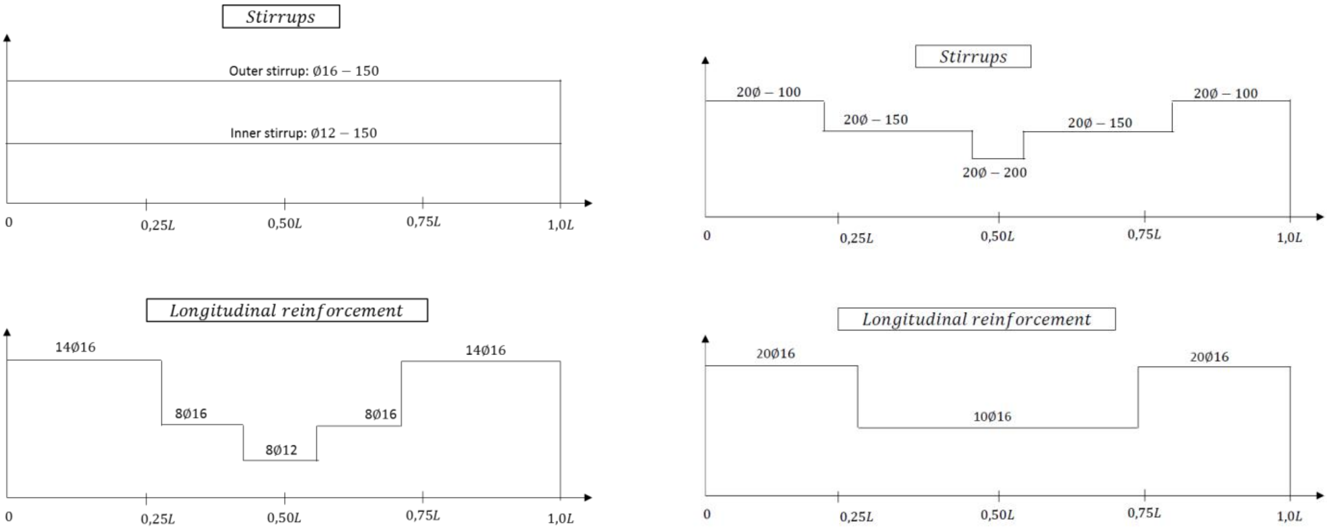


Figure D-13: Shear and longitudinal reinforcement applied in the through girder, Bridge A (left) and Bridge B (right)

For the considered example the required amount of reinforcement per zone is:

Zone I	$= 0{,}40\ mm^2/mm\ (shear + torsion)$
Zone II	$= 0{,}21\ mm^2/mm\ (minimum\ amount)$
Zone III	$= 1{,}92\ mm^2/mm\ (torsion + supension)$

Unity checks for a section at 0,8d in bridge A:

Outer stirrup

$$U.C. = \frac{1{,}92 * 10^3\ mm^2/m}{0{,}25 * \pi * 16^2 * 1000/150} = 1{,}43$$

Inner stirrup

$$U.C. = \frac{0{,}21 * 10^3\ mm^2/m}{0{,}25 * \pi * 12^2 * 1000/150} = 0{,}28$$

Longitudinal Reinforcement

$$U.C. = \frac{322}{0{,}25 * \pi * 16^2 * 14 * 435 * 10^{-3}} = 0{,}26$$

Prestressing steel

$$U.C. = \frac{4093}{2 * 1900 * 1522 * 10^{-3}} = 0{,}71$$

Remarkable is the high unity check for the outer stirrup and the relatively low unity check for the inner stirrup. This difference is caused by the fact that the outer stirrup has to transfer all suspension loads. Because this is an ultimate limit state calculation and steel with ductility class B is applied, one can expect that the outer stirrup will start to deform plastically. Consequently a part of the loading will be transferred towards the inner stirrup. Therefore this calculation is performed again, but this time with a distribution of 35% and 65% of the suspension loads over the inner and outer stirrup.

This results in unity checks which are just within acceptable limits:

Outer stirrup
$$U.C. = \frac{1{,}35 * 10^3\ mm^2/m}{0{,}25 * \pi * 16^2 * 1000/150} = 1{,}01$$

Inner stirrup
$$U.C. = \frac{0{,}755 * 10^3\ mm^2/m}{0{,}25 * \pi * 12^2 * 1000/150} = 1{,}00$$

The angle θ can be freely chosen between 21,8° and 45°. Increasing the angle maximizes the load on the stirrups and minimizes the load on the longitudinal reinforcement. Decreasing the angle has the opposite effect. For both bridges a section near the support and at midspan are regarded. The critical loading near the support is mainly due to torsion and shear, whereas at midspan it is due to bending. The optimal capacity of the shear and longitudinal reinforcement is reached when an angle of 21,8° is used for the sections near the supports and angle of 45° is used at midspan.

Table D-4: Bridge A: Unity check for shear and longitudinal reinforcement

Type of reinforcement	**U.C. @ 0,8d**	**U.C @ 0,5L**
	$\theta = 21,8°$	$\theta = 45°$
Outer stirrup	**1,01**	0,16
Inner stirrup	1,00	0,28
Longitudinal reinforcement	0,26	0,18
Prestressing steel	0,71	0,71

Table D-5: Bridge B: Unity check for shear and longitudinal reinforcement

Type of reinforcement	**U.C. @ support**	**U.C. @ 0,5L**
	$\theta = 21,8°$	$\theta = 45°$
Stirrup	**0,90**	0,36
Longitudinal reinforcement	0,09	0,56
Prestressing steel	0,67	0,71

Table D-4 and Table D-5 present the unity checks for prestress, stirrups and longitudinal reinforcement. In light of the results a number of things need to be explained:

- The unity check of the prestressing steel doesn't range much. The explanation for this lies in the fact that the additional tensile stress is relatively small compared to the yield stress. The unity check holds a base value which roughly equal to: $\sigma_{p\infty}/f_{pd}$.
- Even though the Eurocode loads are larger than the design report loads, unity checks for the shear and longitudinal reinforcement are just within acceptable limits. The possibility to freely choose the angle θ, gives on the opportunity to optimize the capacity of the applied reinforcement.
- The largest unity check for shear reinforcement is found in bridge A, despite the larger loads in bridge B. But bridge B is also equipped with 2,3 times more stirrups, whereas the loads are not 2,3 times larger.

6 Maple sheets

6.1 Equilibrium of forces – elastic

restart;

Equilibrium of forces in a through girder

Parameters

Concrete

$epsiloncelas := 1.75 \cdot 10^{-3}:$ $fcd := \frac{35}{1.5}:$ $\text{theta} := 21.8:$

Prestress steel

$Ap := 1900:$ $Pinf := 12038 \cdot 10^{3}:$ $sigmainf := \frac{Pinf}{6 \cdot Ap}:$ $Ep := 205 \cdot 10^{3}:$ $fpd := 1522:$ $zeta1 := \left(\frac{0.5 \cdot diameter}{(1.6 \cdot (Ap)^{0.5})}\right)^{0.5}:$

Reinforcement steel

$diameter := 16:$ $n := 14:$ $As := 0.25 \cdot 3.1415 \cdot diameter^{2} \cdot n:$ $Es := 210 \cdot 10^{3}:$ $fywd := 435:$

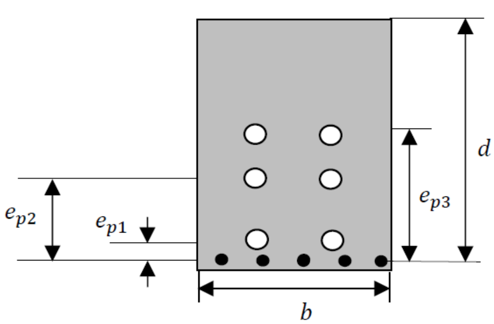
Load

$Med := 2926 \cdot 10^{6};$ $Ved := 0.5 \cdot 3191 \cdot 10^{3};$

$$Med := 2926000000$$

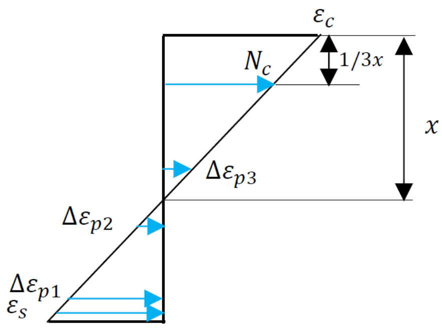
$$Ved := 1.5955000\ 10^{6} \quad \textbf{(1)}$$

Dimensions



$d := 1691:$ $ep1 := 189:$ $ep2 := \frac{(585 + 582)}{2}:$ $ep3 := 979:$ $z := 0.9 \cdot d:$ $b := 900:$

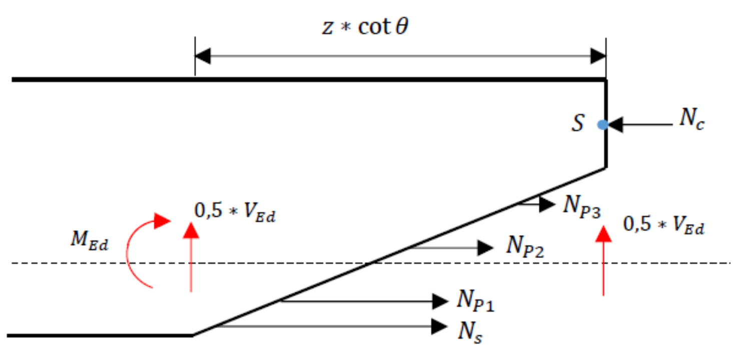
Strains



$$epsilons := \frac{epsilonc}{x} \cdot (d - x) :$$

$$epsilonp1 := \frac{epsilonc}{x} \cdot (d - x - ep1) :$$
$$epsilonp2 := \frac{epsilonc}{x} \cdot (d - x - ep2) :$$
$$epsilonp3 := \frac{epsilonc}{x} \cdot (d - x - ep3) :$$

Forces



$$Ns := As \cdot Es \cdot epsilons :$$
$$Np1 := 2 \cdot Ap \cdot (sigmainf + zeta1 \cdot epsilonp1 \cdot Ep) :$$
$$Np2 := 2 \cdot Ap \cdot (sigmainf + zeta1 \cdot epsilonp2 \cdot Ep) :$$
$$Np3 := 2 \cdot Ap \cdot (sigmainf + zeta1 \cdot epsilonp3 \cdot Ep) :$$

$$Nc := \frac{0.5 \cdot epsilonc}{epsiloncelas} \cdot fcd \cdot b \cdot x :$$

Equilibrium of horizontal forces

$H := Nc - Ns - Np1 - Np2 - Np3 :$

$eq1 := H = 0 :$

Equilibrium of moment

$MS1 := Med + Ved \cdot z \cdot \cot\left(\frac{\text{theta} \cdot 3.1415}{180}\right) :$

$MS2 := Ns \cdot \left(d - \frac{1}{3} \cdot x\right) + Np1 \cdot \left(d - \frac{1}{3} \cdot x - ep1\right) + Np2 \cdot \left(d - \frac{1}{3} \cdot x - ep2\right) + Np3 \cdot \left(d - \frac{1}{3} \cdot x - ep3\right) :$

$eq2 := MS1 = MS2 :$

Solution for epsilonc and x

$solution := solve(\{eq1, eq2\}, \{epsilonc, x\});$

$solution := \{x = 1213.498972, epsilonc = 0.001691263617\}, \{x = -67.08803449 + 920.6259193\,I, epsilonc = 0.0002329345928 - 0.001559515108\,I\}, \{x = -67.08803449 - 920.6259193\,I, epsilonc = 0.0002329345928 + 0.001559515108\,I\}$ **(2)**

$x := 1213.498972 :$ $epsilonc := 0.001691263617 :$

Forces

Reinforcement steel

$print(\text{round}(0, Ns \cdot 10^{-3}));$

393 **(3)**

Max. prestress force

$\text{round}(0, Np1 \cdot 10^{-3});$

4119 **(4)**

$\text{round}(0, Np2 \cdot 10^{-3});$

3974 **(5)**

$\text{round}(0, Np3 \cdot 10^{-3});$

3828 **(6)**

Concrete force

$print(\text{round}(0, Nc \cdot 10^{-3}));$

12314 **(7)**

Unity check

Reinforcement steel

$$UC := parse\left(sprintf\left(\text{"\%.2f"}, \frac{Ns}{(As \cdot fywd)}\right)\right);$$

$$UC := 0.32 \tag{8}$$

Prestress steel

$$UC := parse\left(sprintf\left(\text{"\%.2f"}, \frac{Np1}{(2 \cdot Ap \cdot fpd)}\right)\right);$$

$$UC := 0.71 \tag{9}$$

6.2 Equilibrium of forces – elastic-plastic

restart;

Equilibrium of forces in a through girder

Parameters

Concrete

$epsiloncelas := 1.75 \cdot 10^{-3}:$ $fcd := \frac{35}{1.5}:$ $\text{theta} := 21.8:$

Prestress steel

$Ap := 1900:$ $Pinf := 12038 \cdot 10^{3}:$ $sigmainf := \frac{Pinf}{6 \cdot Ap}:$ $Ep := 205 \cdot 10^{3}:$ $fpd := 1522:$ $zeta1 := \left(\frac{0.5 \cdot diameter}{\left(1.6 \cdot (Ap)^{0.5}\right)}\right)^{0.5}:$

Reinforcement steel

$diameter := 16:$ $n := 14:$ $As := 0.25 \cdot 3.1415 \cdot diameter^{2} \cdot n:$ $Es := 210 \cdot 10^{3}:$ $fywd := 435:$

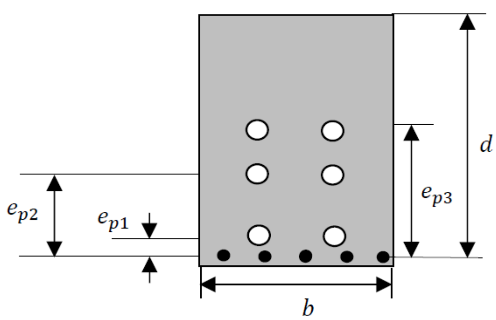
Load

$Med := 2784 \cdot 10^{6};$ $Ved := 0.5 \cdot 3404 \cdot 10^{3};$

$$Med := 2784000000$$

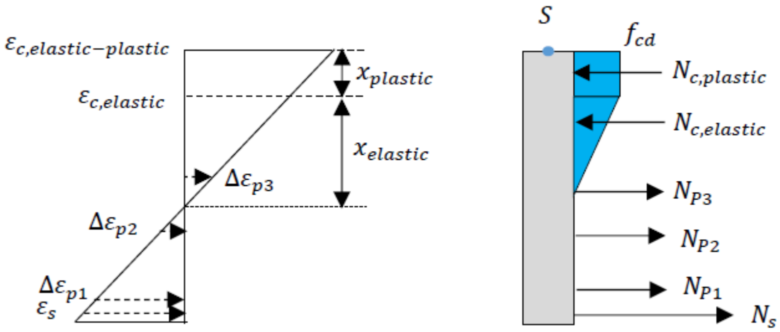
$$Ved := 1.7020000\ 10^{6} \qquad \textbf{(1)}$$

Dimensions



$d := 1691:$ $ep1 := 189:$ $ep2 := \frac{(585 + 582)}{2}:$ $ep3 := 979:$ $z := 0.9 \cdot d:$ $b := 900:$

Strains

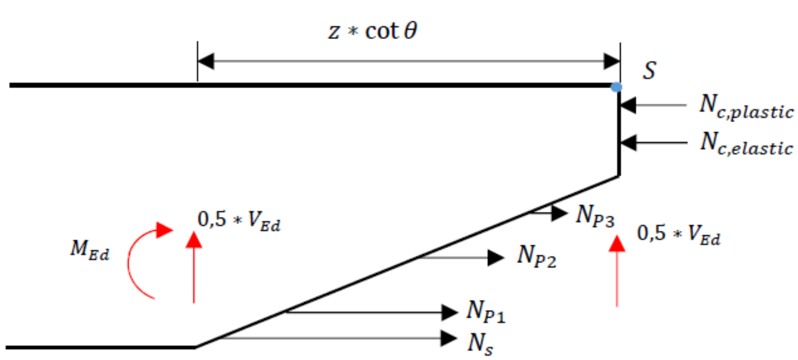


$$epsilons := \frac{epsilonc}{x} \cdot (d - x) :$$

$$epsilonp1 := \frac{epsilonc}{x} \cdot (d - x - ep1) :$$
$$epsilonp2 := \frac{epsilonc}{x} \cdot (d - x - ep2) :$$
$$epsilonp3 := \frac{epsilonc}{x} \cdot (d - x - ep3) :$$
$$xelastic := \frac{epsiloncelas}{epsilonc} \cdot x :$$

$$xplastic := x - xelastic :$$

Forces



$Ns := As \cdot Es \cdot epsilons :$
$Np1 := 2 \cdot Ap \cdot (sigmainf + zeta1 \cdot epsilonp1 \cdot Ep) :$
$Np2 := 2 \cdot Ap \cdot (sigmainf + zeta1 \cdot epsilonp2 \cdot Ep) :$
$Np3 := 2 \cdot Ap \cdot (sigmainf + zeta1 \cdot epsilonp3 \cdot Ep) :$

$Ncelastic := 0.5 \cdot fcd \cdot b \cdot xelastic :$
$Ncplastic := fcd \cdot b \cdot xplastic :$

Equilibrium of horizontal forces

$H := Ncelastic + Ncplastic - Ns - Np1 - Np2 - Np3 :$

$eq1 := H = 0 :$

Equilibrium of moment

$MS1 := Med + Ved \cdot z \cdot \cot\left(\frac{\text{theta} \cdot 3.1415}{180}\right) :$

$MS2 := Ns \cdot (d) + Np1 \cdot (d - ep1) + Np2 \cdot (d - ep2) + Np3 \cdot (d - ep3) - Ncelastic \cdot \left(xplastic + \frac{1}{3} \cdot xelastic\right) - Ncplastic \cdot 0.5 \cdot xplastic :$

$eq2 := MS1 = MS2 :$

Solution for epsilonc and x

$solution := solve(\{eq1, eq2\}, \{epsilonc, x\});$

$solution := \{x = 1176.689932, epsilonc = 0.001757781489\}, \{x = 157.9765575 - 551.5007266\,I, epsilonc = 0.0005160996105 + 0.0004987772805\,I\}, \{x = 1746.855876 + 339.6809586\,I, epsilonc = -0.06321977950 + 0.01545609979\,I\}, \{x = 1746.855876 - 339.6809586\,I, epsilonc = -0.06321977950 - 0.01545609979\,I\}, \{x = 157.9765575 + 551.5007266\,I, epsilonc = 0.0005160996105 - 0.0004987772805\,I\}$ **(2)**

$x := 1176.689932 :$ $epsilonc := 0.001757781489 :$

Forces

Reinforcement steel

$print(\text{round}(0, Ns \cdot 10^{-3}));$

454 **(3)**

Max. prestress force

$print(\text{round}(0, Np1 \cdot 10^{-3}));$

4141 **(4)**

$print(\text{round}(0, Np2 \cdot 10^{-3}));$

3985 **(5)**

$print(\text{round}(0, Np3 \cdot 10^{-3}));$

3830 **(6)**

Concrete force

$print(\text{round}(0, (Ncelastic) \cdot 10^{-3}));$

12301 **(7)**

$print(\text{round}(0, (Ncplastic) \cdot 10^{-3}));$

109 **(8)**

Unity check

Reinforcement steel

$$UC := parse\left(sprintf\left(\text{"\%.2f"}, \frac{Ns}{(As \cdot fywd)}\right)\right);$$

$$UC := 0.37 \tag{9}$$

Prestress steel

$$UC := parse\left(sprintf\left(\text{"\%.2f"}, \frac{Np1}{(2 \cdot Ap \cdot fpd)}\right)\right);$$

$$UC := 0.72 \tag{10}$$

7 Spreadsheets

7.1 Bridge A - 6.10b – 0,8d

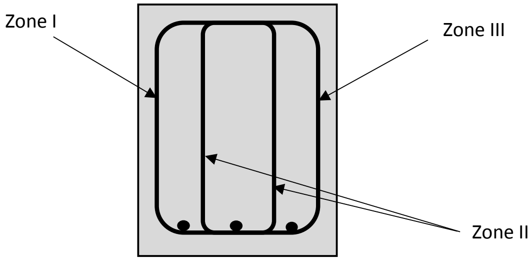
Forces

LC	type	Prestress P [kN]	Prestress σxx [N/mm2]	Bending moment M [kNm]	Bending moment σxx [N/mm2]	Total hor. normal stress σxx [N/mm2]
1	self-weight			1106	1,05	
2	ballast			306	0,29	
3	Conc. Mobile Load			822	0,78	
4	Cant. Mobile Load			-224	-0,21	
5a	Contin. Mobile Load			609	0,58	
5b	Contin. Mobile Load (SW/2)			1373	1,30	
6	Support settelement max			0	0,00	
7	Support settelement min			0	0,00	
8	Prestress t=0	-13200	-6,14	-1055	-1,00	
9	Prestress t = ∞	-12038	-5,60	-962	-0,91	
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		-12038	-5,60	1881	1,78	-3,81
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		-12038	-5,60	1881	1,78	-3,81
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		-12038	-5,60	226	0,21	-5,38
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		-12038	-5,60	226	0,21	-5,38
LC 1 + LC 8		-13200	-6,14	51	0,05	-6,09
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		-12038	-5,60	1822	1,73	-3,87
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		-12038	-5,60	1822	1,73	-3,87

LC	type	Suspension force Qyy [kN]	Clamping moment mxx [kNm]	Shear force [kN]	Torsion [kNm]
1	self-weight	34	16	786	-93
2	ballast	29	18	267	-81
3	Conc. Mobile Load	149	-20	558	-218
4	Cant. Mobile Load	0	61	0	-246
5a	Contin. Mobile Load	0	34	448	23
5b	Contin. Mobile Load (SW/2)	107	47	976	-125
6	Support settelement max	-18	98	0	-214
7	Support settelement min	18	-98	0	214
8	Prestress t=0	-52	28	-923	37
9	Prestress t = ∞	-47	26	-842	34
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		148	172	1217	-548
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		184	-25	1217	-121
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		-1	218	211	-600
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		35	21	211	-173
LC 1 + LC 8		-17	44	-137	-56
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		105	205	1187	-479
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		141	8	1187	-52

Reinforcement

Load combination	Suspension force A_Q [mm2/mm]	Clamping moment A_m [mm2/mm]	Shear force A_V [mm2/mm]	Torsion A_T [mm2/mm]	Total amount of stirrups Zone I [mm2/mm]	Zone II [mm2/mm]	Zone III [mm2/mm]
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9	0,67	0,96	0,74	0,29	0,40	0,75	1,35
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9	0,00	0,00	0,00	0,00	0,21	0,21	0,21
LC 1 + LC 2 + LC 4 + LC 6 + LC 9	0,01	1,22	0,13	0,32	0,32	0,46	1,11
LC 1 + LC 2 + LC 4 + LC 7+ LC 9	0,00	0,00	0,00	0,00	0,21	0,21	0,21
LC 1 + LC 8	0,00	0,00	0,00	0,00	0,21	0,21	0,21
LC 1 + LC 2 + LC 5b + LC 6 + LC 9	0,48	1,15	0,72	0,25	0,37	0,75	1,31
LC 1 + LC 2 + LC 5b + LC 7 + LC 9	0,00	0,00	0,00	0,00	0,21	0,21	0,21



Type of reinforcement	Required amount (mm2/m)	Applied reinforcement	Applied amount (mm2/m)	U.C.
Outer stirrup	1349	Ø16-150	1340	1,01
Inner stirrup	755	Ø12-150	754	1,00

Load combination	Unity check [-] Longitudinal reinforcement	Prestress steel
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9	0,26	0,71
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9	0,04	0,69
LC 1 + LC 2 + LC 4 + LC 6 + LC 9	0,00	0,69
LC 1 + LC 2 + LC 4 + LC 7+ LC 9	0,00	0,69
LC 1 + LC 8	0,00	0,69
LC 1 + LC 2 + LC 5b + LC 6 + LC 9	0,22	0,70
LC 1 + LC 2 + LC 5b + LC 7 + LC 9	0,00	0,69

Parameters

Sectional properties

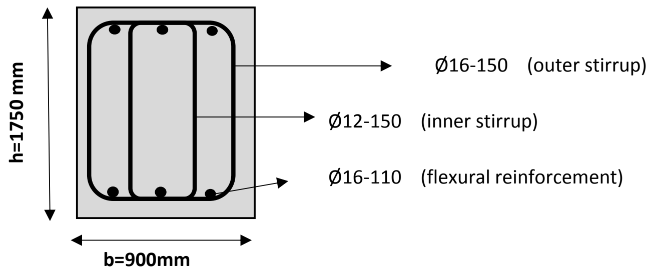
b_{girder}	900	[mm]
h_{girder}	1750	[mm]
t_{floor}	550	[mm]
b_{floor}	4000	[mm]
$0{,}5 * A_{bridge}$	2150500	[mm2]
$0{,}5 * I_{yy}$	6,63E+11	[mm4]
y	629	[mm]
z	629	[mm]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1691	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_{∞}	=	-12038	[kN]
V_{Ed}	=	1217	[kN]
T_{Ed}	=	548	[kNm]
Q_{yy}	=	148	[kN/m]
m_{xx}	=	172	[kNm/m]
M_{Ed}	=	1881	[kNm]

Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2,0	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9



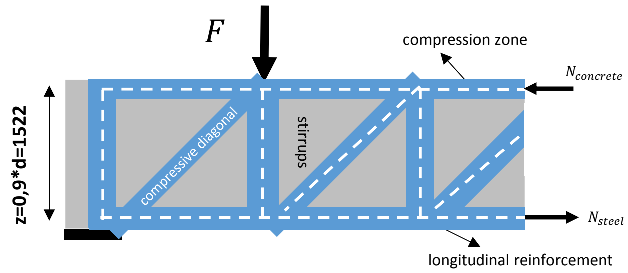
n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	14	16	201	110	2815

Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	



$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s}$	=	6933	[kN]

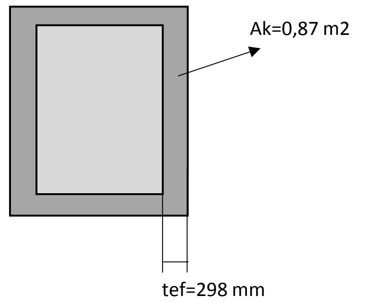
z	=	1522	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0018	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1516	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1551	[kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	6867	[kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	1806	[kN]

Shear capacity concrete

Max. tensile bending stress	=	-3,81 [N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	1806 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	6867 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

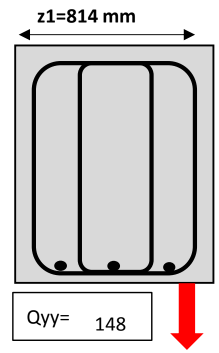
$Area\ (A)$	=	1,58 [m2]	
$Perimeter\ (u)$	=	5,3 [m]	
$t_{ef} = A/u$	=	298 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	118 [mm]	
A_k	=	0,87 [m2]	
u_k	=	4,11 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	2612 [kN]	*(Capacity compressive diagonal)*

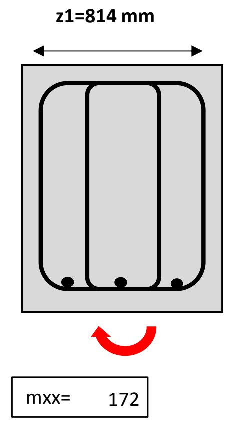
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814 [mm]
$V_{Ed,sup}$	=	148 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	563 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	2583 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	211	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	803	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,59	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	1,66	>	1,00	*(Capacity of concrete insufficient, apply stirrups)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	0,74	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	0,29	[mm2/mm]

Minimum reinforcement

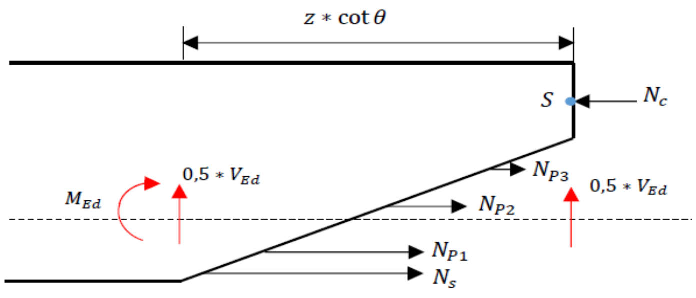
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	0,85 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	0,67	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	0,96	[mm2/mm]

Equilibrium of forces

ε_c	=	1,61E-03	< 1,75E-03	[elastic stage]
x	=	1263,3	[mm]	
$M_{Ed} - M_{P\infty}$	=	2843	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	3060	[kN]	
$\sigma_{p\infty}$	=	1056	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,339	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	322	[kN]	
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	4093	[kN]	
e_{p1}	=	189	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,26 [-]

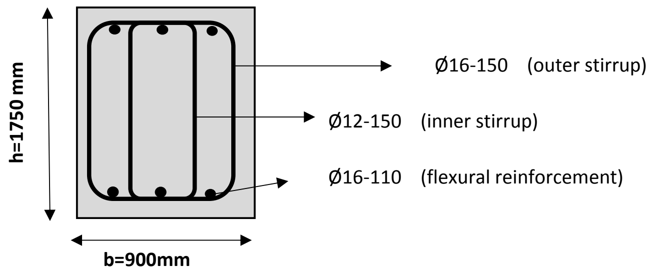
Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,71 [-]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1691	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_∞	=	-12038	[kN]
V_{Ed}	=	1217	[kN]
T_{Ed}	=	121	[kNm]
Q_{yy}	=	184	[kN/m]
m_{xx}	=	25	[kNm/m]
M_{Ed}	=	1881	[kNm]

LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9



Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

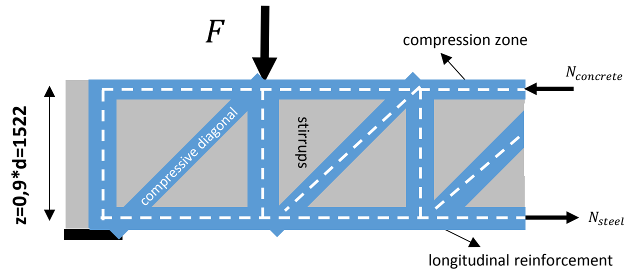
n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	14	16	201	110	2815

Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	



Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0018	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1516	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1551	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	6867	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	1806	[kN]

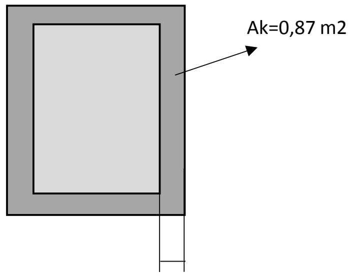
$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 6933 \text{ [kN]}$$

z	=	1522	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)

$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-3,81 [N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	1806 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	6867 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

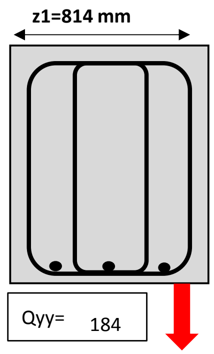
$Area\ (A)$	=	1,58 [m2]	
$Perimeter\ (u)$	=	5,3 [m]	
$t_{ef} = A/u$	=	298 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	118 [mm]	
A_k	=	0,87 [m2]	
u_k	=	4,11 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	2612 [kN]	*(Capacity compressive diagonal)*

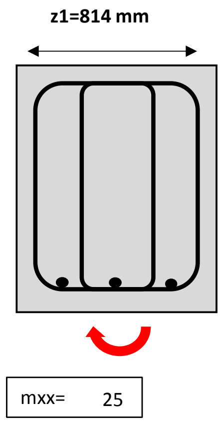
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814 [mm]
$V_{Ed,sup}$	=	184 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	700 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	2035 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	31	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	118	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,34	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,97	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

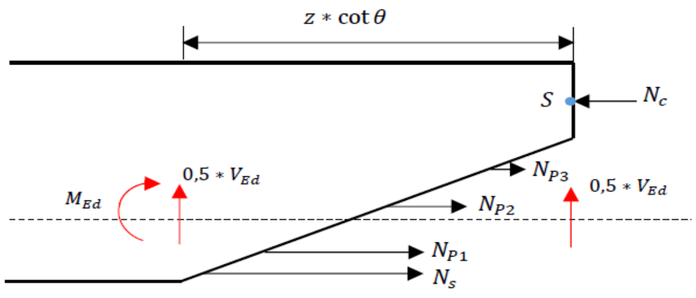
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	0,85 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	1,24E-03	<	1,75E-03 [elastic stage]
x	=	1585,8	[mm]	
$M_{Ed} - M_{P_\infty}$	=	2843	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	2140	[kN]	
$\sigma_{p\infty}$	=	1056	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,339	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	49	[kN]	
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	3996	[kN]	
e_{p1}	=	189	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,04 [-]

Prestressing steel

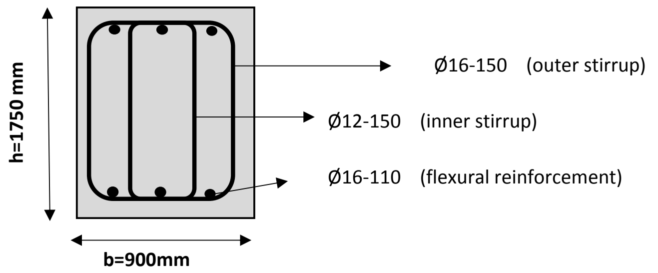
$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,69 [-]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1691	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_∞	=	-12038	[kN]
V_{Ed}	=	211	[kN]
T_{Ed}	=	600	[kNm]
Q_{yy}	=	1	[kN/m]
m_{xx}	=	218	[kNm/m]
M_{Ed}	=	226	[kNm]

Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

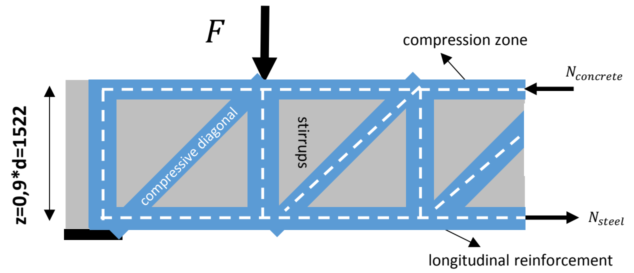
LC 1 + LC 2 + LC 4 + LC 6 + LC 9



n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	14	16	201	110	2815



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	

Shear capacity concrete

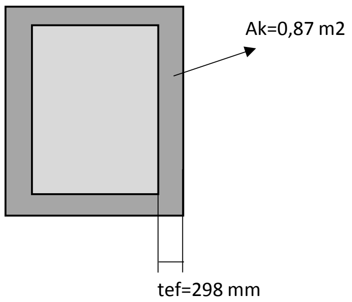
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0018	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1516	[kN]
$V_{Rd,min} = \left[v_{min} + k_1 * \sigma_{cp}\right\} * b * d$	=	1551	[kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	6867	[kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	1806	[kN]

$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s}$ = 6933 [kN]

z	=	1522	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-3,81 [N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	1806 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	6867 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

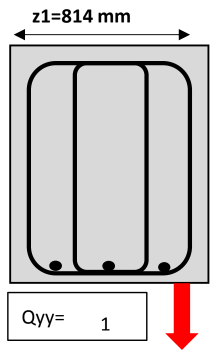
$Area\ (A)$	=	1,58 [m2]	
$Perimeter\ (u)$	=	5,3 [m]	
$t_{ef} = A/u$	=	298 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	118 [mm]	
A_k	=	0,87 [m2]	
u_k	=	4,11 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \alpha_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	2612 [kN]	*(Capacity compressive diagonal)*

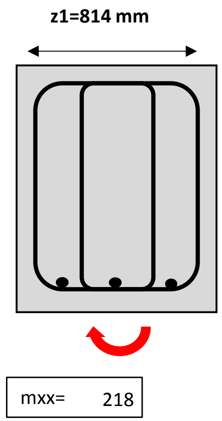
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814 [mm]
$V_{Ed,sup}$	=	1 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	5 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	1236 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	268	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	1020	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,41	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	1,13	>	1,00	*(Capacity of concrete insufficient, apply stirrups)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	0,13	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	0,32	[mm2/mm]

Minimum reinforcement

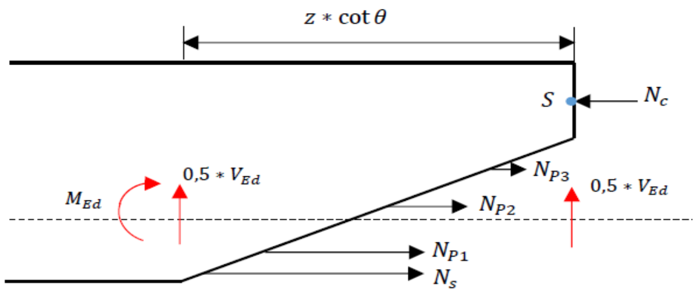
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	0,85 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	0,01	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	1,22	[mm2/mm]

Equilibrium of forces

ε_c	=	9,00E-04	<	1,75E-03	[elastic stage]
x	=	2139	[mm]		
$M_{Ed} - M_{P_\infty}$	=	1188	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	1759	[kN]		
$\sigma_{p\infty}$	=	1056	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,339	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	4013	[kN]		
e_{p1}	=	189	[mm]		



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

Prestressing steel

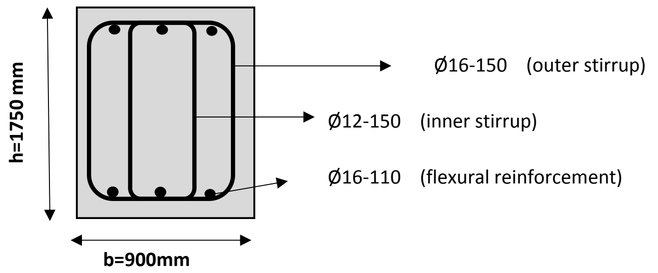
$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,69 [-]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1691	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_{∞}	=	-12038	[kN]
V_{Ed}	=	211	[kN]
T_{Ed}	=	173	[kNm]
Q_{yy}	=	35	[kN/m]
m_{xx}	=	21	[kNm/m]
M_{Ed}	=	226	[kNm]

Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

LC 1 + LC 2 + LC 4 + LC 7+ LC 9



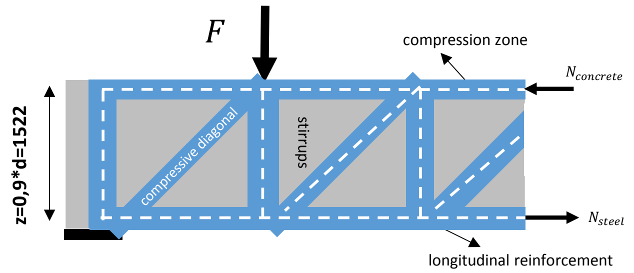
n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	14	16	201	110	2815

Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	



Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b*d)$	=	0,0018	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1516	[kN]
$V_{Rd,min} = \left[v_{min} + k_1 * \sigma_{cp}\right\} * b * d$	=	1551	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	6867	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{\left(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd}\right)}$	=	1806	[kN]

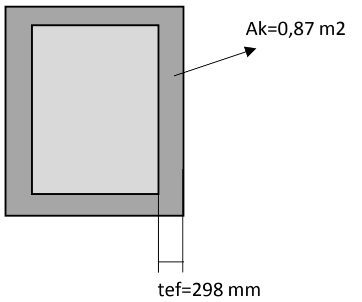
$V_{Rd,s} = z * f_{ywd} * \cot\theta * \dfrac{A_{sw}}{s}$ = 6933 [kN]

z	=	1522	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)

$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-3,81	[N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	1806	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	6867	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

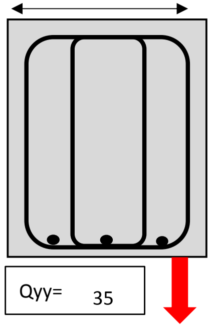
$Area\ (A)$	=	1,58	[m2]	
$Perimeter\ (u)$	=	5,3	[m]	
$t_{ef} = A/u$	=	298	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	118	[mm]	
A_k	=	0,87	[m2]	
u_k	=	4,11	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	2612	[kN]	*(Capacity compressive diagonal)*

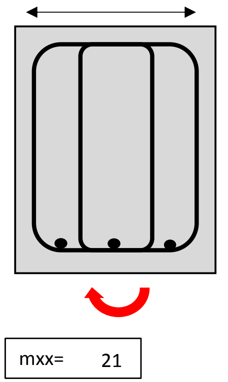
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814	[mm]
$V_{Ed,sup}$	=	35	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	132	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	442	[kN]

Clamping moment

$V_{Ed,clamp.} = \dfrac{m_{xx}}{z_1}$	=	26	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	99	[kN]





Total capacity concrete

$\dfrac{T_{Ed}}{T_{Rd,max}} + \dfrac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,13	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\dfrac{T_{Ed}}{T_{Rd,c}} + \dfrac{V_{Ed}}{V_{Rd,c}}$	=	0,40	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \dfrac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \dfrac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

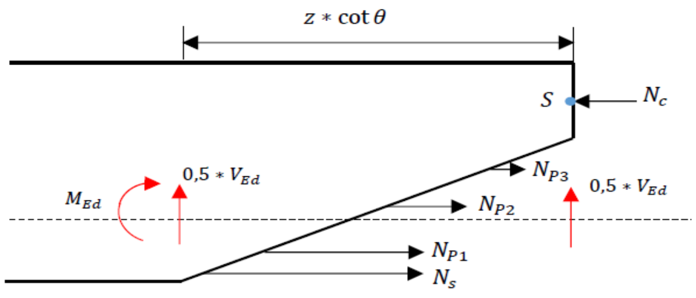
$\rho_{w,min} = \dfrac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	0,85	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	7,10E-04	<	1,75E-03 [elastic stage]
x	=	2701,5	[mm]	
$M_{Ed} - M_{P_\infty}$	=	1188	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	593	[kN]	
$\sigma_{p\infty}$	=	1056	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,339	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]	
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	4013	[kN]	
e_{p1}	=	189	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

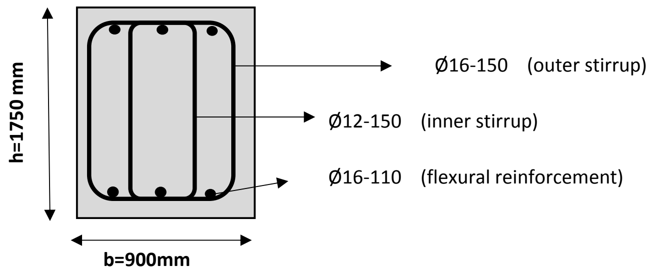
Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,69 [-]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1691	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_∞	=	-12038	[kN]
V_{Ed}	=	137	[kN]
T_{Ed}	=	56	[kNm]
Q_{yy}	=	17	[kN/m]
m_{xx}	=	44	[kNm/m]
M_{Ed}	=	51	[kNm]

LC 1 + LC 8



Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

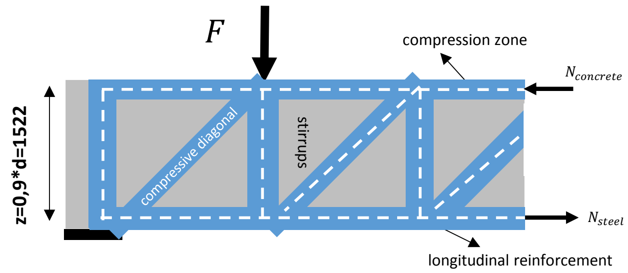
n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	14	16	201	110	2815

Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	



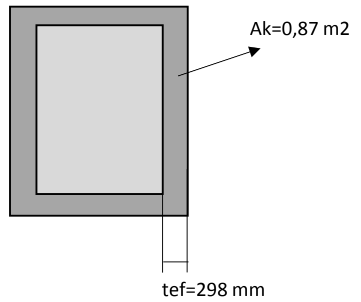
Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b*d)$	=	0,0018	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1516	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1551	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	6867	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	1806	[kN]

$V_{Rd,s} = z * f_{ywd} * \cot\theta * \dfrac{A_{sw}}{s}$	=	6933	[kN]	
z	=	1522	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-3,81 [N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	1806 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	6867 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

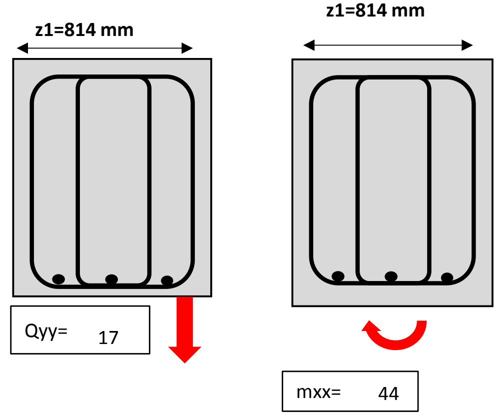
$Area\ (A)$	=	1,58 [m2]	
$Perimeter\ (u)$	=	5,3 [m]	
$t_{ef} = A/u$	=	298 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	118 [mm]	
A_k	=	0,87 [m2]	
u_k	=	4,11 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	2612 [kN]	*(Capacity compressive diagonal)*

Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814 [mm]
$V_{Ed,sup}$	=	17 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	65 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	407 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	54	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	205	[kN]



Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,08	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,20	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

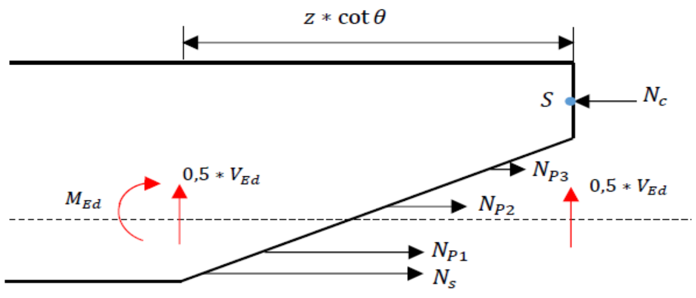
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	0,85 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \dfrac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \dfrac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	**6,80E-04**	<	1,75E-03	[elastic stage]
x	=	**2813,7**	[mm]		
$M_{Ed} - M_{P_\infty}$	=	1013	[kNm]		
$V_{Ed} = V_{Ed,tot} + \dfrac{T_{Ed} * z}{2 * A_k}$	=	456	[kN]		
$\sigma_{p\infty}$	=	1056	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \dfrac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,339	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	4013	[kN]		
e_{p1}	=	**189**	[mm]		



Longitudinal Reinforcement

$U.C. = \dfrac{N_s}{A_s * fywd}$ = 0,00 [-]

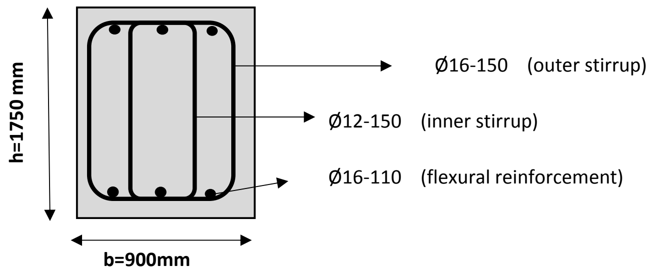
Prestressing steel

$U.C. = \dfrac{N_{p1}}{2 * A_p * fpd}$ = 0,69 [-]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1691	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_{∞}	=	-12038	[kN]
V_{Ed}	=	1187	[kN]
T_{Ed}	=	479	[kNm]
Q_{yy}	=	105	[kN/m]
m_{xx}	=	205	[kNm/m]
M_{Ed}	=	1822	[kNm]

LC 1 + LC 2 + LC 5b + LC 6 + LC 9

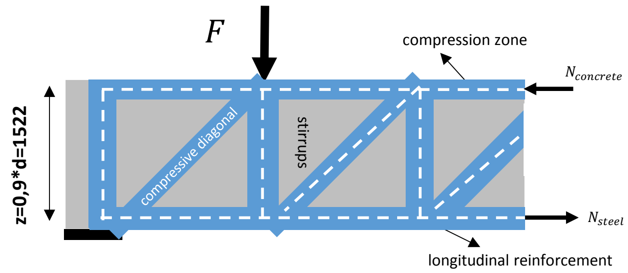


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	14	16	201	110	2815



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	16	402	150	2,68
Inner stirrups	2	12	226	150	1,51 +
	4		628		4,19

Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0018	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1516	[kN]
$V_{Rd,min} = \left[v_{min} + k_1 * \sigma_{cp}\right\} * b * d$	=	1551	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	6867	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{\left(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd}\right)}$	=	1806	[kN]

$V_{Rd,s} = z * f_{ywd} * \cot\theta * \dfrac{A_{sw}}{s}$ = 6933 [kN]

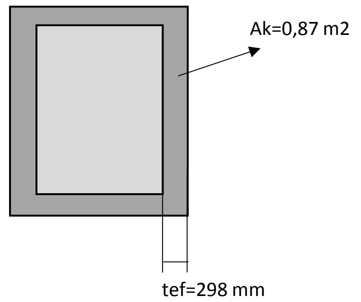
z	=	1522	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress = -3,81 [N/mm2] *(Concrete capacity is limited by shear tension failure)*

$V_{Rd,c}$ = 1806 [kN] *(Concrete shear capacity)*

$V_{Rd,max}$ = 6867 [kN] *(Capacity compressive diagonal)*



Torsion capacity concrete

$Area\ (A)$ = 1,58 [m2]

$Perimeter\ (u)$ = 5,3 [m]

$t_{ef} = A/u$ = 298 [mm]

$t_{ef,lower\ limit} = 2 * (h - d)$ = 118 [mm]

A_k = 0,87 [m2]

u_k = 4,11 [m]

$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$ = 693 [kN] *(Concrete torsion capacity)*

$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$ = 2612 [kN] *(Capacity compressive diagonal)*

Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$ = 814 [mm]

$V_{Ed,sup}$ = 105 [kN/m]

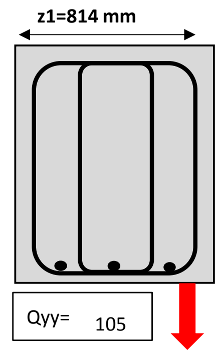
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$ = 400 [kN]

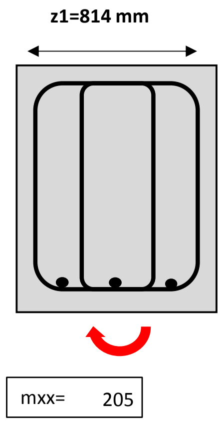
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$ = 2546 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$ = 252 [kN/m]

$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$ = 959 [kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$ = 0,55 < 1,00 *(Capacity of compressive diagonal is sufficient)*

$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$ = 1,55 > 1,00 *(Capacity of concrete insufficient, apply stirrups)*

Stirrups

Stirrups due to Shear Force = $A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$ = 0,72 [mm2/mm]

Stirrups due to Torsion = $A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$ = 0,25 [mm2/mm]

Minimum reinforcement

$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$ = 0,000947 [-]

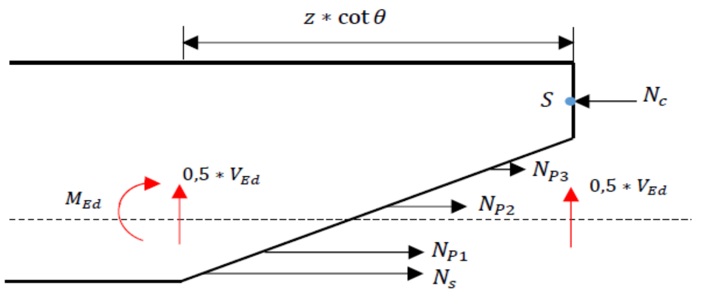
$A_{s,min}$ = 0,85 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	0,48	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	1,15	[mm2/mm]

Equilibrium of forces

ε_c	=	1,55E-03	<	1,75E-03	[elastic stage]
x	=	1302,2	[mm]		
$M_{Ed} - M_{P_\infty}$	=	2784	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	2963	[kN]		
$\sigma_{p\infty}$	=	1056	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,339	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	274	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	4076	[kN]		
e_{p1}	=	189	[mm]		



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,22 [-]

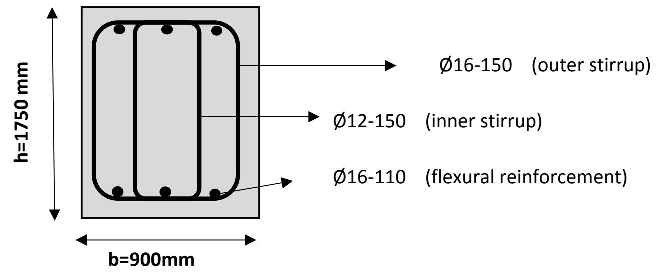
Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,70 [-]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1691	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_∞	=	-12038	[kN]
V_{Ed}	=	1187	[kN]
T_{Ed}	=	52	[kNm]
Q_{yy}	=	141	[kN/m]
m_{xx}	=	8	[kNm/m]
M_{Ed}	=	1822	[kNm]

LC 1 + LC 2 + LC 5b + LC 6 + LC 9

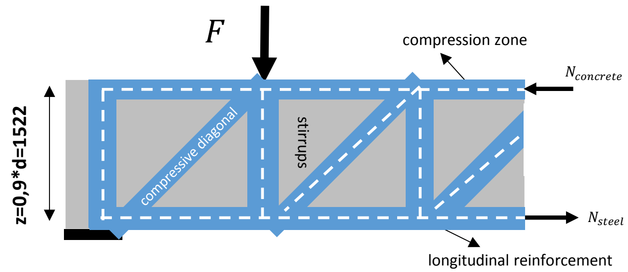


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	14	16	201	110	2815



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	

Shear capacity concrete

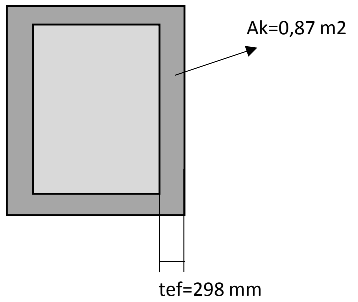
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0018	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1516	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1551	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	6867	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	1806	[kN]

$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 6933 \text{ [kN]}$$

z	=	1522	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-3,81	[N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	1806	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	6867	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

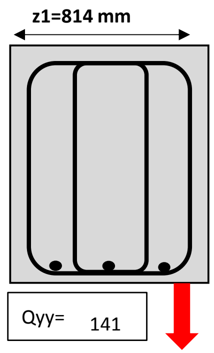
$Area\ (A)$	=	1,58	[m2]	
$Perimeter\ (u)$	=	5,3	[m]	
$t_{ef} = A/u$	=	298	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	118	[mm]	
A_k	=	0,87	[m2]	
u_k	=	4,11	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	2612	[kN]	*(Capacity compressive diagonal)*

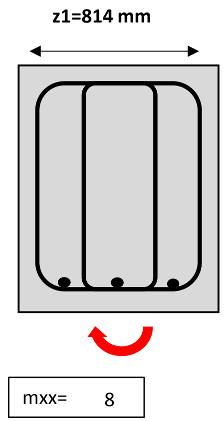
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814	[mm]
$V_{Ed,sup}$	=	141	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	537	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	1762	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	10	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	38	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,28	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,82	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

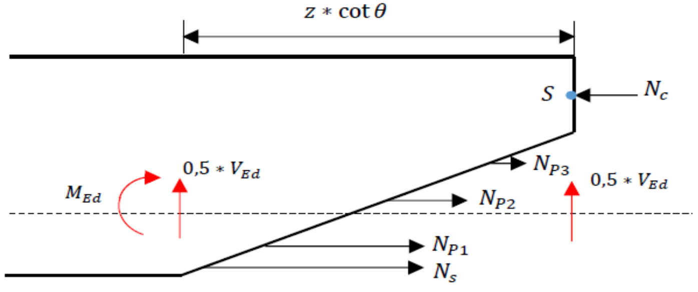
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	0,85	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	**1,10E-03**	<	1,75E-03	[elastic stage]
x	=	**1737**	[mm]		
$M_{Ed} - M_{P_\infty}$	=	2784	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	1807	[kN]		
$\sigma_{p\infty}$	=	1056	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,339	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	3974	[kN]		
e_{p1}	=	**189**	[mm]		



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,69 [-]

7.2 Bridge A - 6.10b – 0,5L

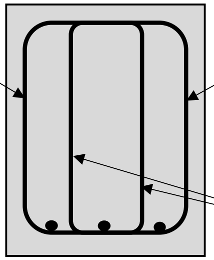
Forces

LC	type	Prestress P [kN]	Prestress σxx [N/mm2]	Bending moment M [kNm]	Bending moment σxx [N/mm2]	Total hor. normal stress σxx [N/mm2]
1	self-weight			4698	4,46	
2	ballast			1526	1,45	
3	Conc. Mobile Load			4523	4,29	
4	Cant. Mobile Load			-224	-0,21	
5a	Contin. Mobile Load			2050	1,94	
5b	Contin. Mobile Load (SW/2)			5844	5,54	
6	Support settelement max			0	0,00	
7	Support settelement min			0	0,00	
8	Prestress t=0	-13200	-4,80	-5274	-5,00	
9	Prestress t = ∞	-12038	-4,38	-4810	-4,56	
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		-12038	-4,38	7987	7,57	3,20
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		-12038	-4,38	7987	7,57	3,20
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		-12038	-4,38	1190	1,13	-3,25
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		-12038	-4,38	1190	1,13	-3,25
LC 1 + LC 8		-13200	-4,80	-576	-0,55	-5,35
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		-12038	-4,38	7258	6,88	2,51
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		-12038	-4,38	7258	6,88	2,51

LC	type	Suspension force Qyy [kN]	Clamping moment mxx [kNm]	Shear force [kN]	Torsion [kNm]
1	self-weight	34	8	0	0
2	ballast	29	5	0	0
3	Conc. Mobile Load	149	-68	0	0
4	Cant. Mobile Load	0	13	0	0
5a	Contin. Mobile Load	0	57	0	0
5b	Contin. Mobile Load (SW/2)	107	21	3	-174
6	Support settelement max	1	8	0	140
7	Support settelement min	-1	-8	0	-140
8	Prestress t=0	-52	9	0	0
9	Prestress t = ∞	-47	8	0	0
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9		167	19	0	140
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9		165	2	0	-140
LC 1 + LC 2 + LC 4 + LC 6 + LC 9		18	43	0	140
LC 1 + LC 2 + LC 4 + LC 7+ LC 9		16	26	0	-140
LC 1 + LC 8		-17	17	0	0
LC 1 + LC 2 + LC 5b + LC 6 + LC 9		124	50	3	-33
LC 1 + LC 2 + LC 5b + LC 7 + LC 9		122	34	3	-314

Reinforcement

Load combination	Suspension force A_Q [mm2/mm]	Clamping moment A_m [mm2/mm]	Shear force A_V [mm2/mm]	Torsion A_T [mm2/mm]	Total amount of stirrups Zone I [mm2/mm]	Zone II [mm2/mm]	Zone III [mm2/mm]
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9	0,00	0,00	0,00	0,00	0,21	0,21	0,21
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9	0,00	0,00	0,00	0,00	0,21	0,21	0,21
LC 1 + LC 2 + LC 4 + LC 6 + LC 9	0,00	0,00	0,00	0,00	0,21	0,21	0,21
LC 1 + LC 2 + LC 4 + LC 7+ LC 9	0,00	0,00	0,00	0,00	0,21	0,21	0,21
LC 1 + LC 8	0,00	0,00	0,00	0,00	0,21	0,21	0,21
LC 1 + LC 2 + LC 5b + LC 6 + LC 9	0,00	0,00	0,00	0,00	0,21	0,21	0,21
LC 1 + LC 2 + LC 5b + LC 7 + LC 9	0,00	0,00	0,00	0,00	0,21	0,21	0,21



Type of reinforcement	Required amount (mm2/m)	Applied reinforcement	Applied amount (mm2/m)	U.C.
Outer stirrup	213	Ø16-150	1340	0,16
Inner stirrup	213	Ø12-150	754	0,28

Load combination	Unity check [-] Longitudinal reinforcement	Prestress steel
LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9	0,18	0,71
LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9	0,18	0,71
LC 1 + LC 2 + LC 4 + LC 6 + LC 9	0,00	0,69
LC 1 + LC 2 + LC 4 + LC 7+ LC 9	0,00	0,69
LC 1 + LC 8	0,00	0,69
LC 1 + LC 2 + LC 5b + LC 6 + LC 9	0,08	0,70
LC 1 + LC 2 + LC 5b + LC 7 + LC 9	0,10	0,70

Parameters

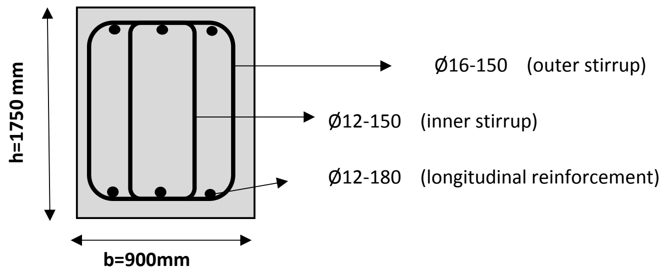
Sectional properties

b_{girder}	900	[mm]
h_{girder}	1750	[mm]
t_{floor}	550	[mm]
b_{floor}	4000	[mm]
$0,5 * A_{bridge}$	2750000	[mm2]
$0,5 * I_{yy}$	6,63E+11	[mm4]
y	629	[mm]
z	629	[mm]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1693	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_∞	=	-12038	[kN]
V_{Ed}	=	0	[kN]
T_{Ed}	=	140	[kNm]
Q_{yy}	=	167	[kN/m]
m_{xx}	=	19	[kNm/m]
M_{Ed}	=	7987	[kNm]

LC 1+ LC 2+ LC 3+ LC 5a+ LC 6 + LC 9



Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2,0	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

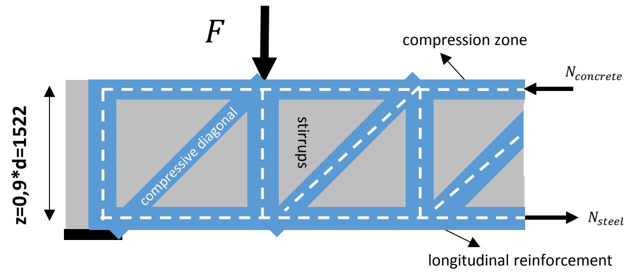
n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	8	12	113	180	905

Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	16	402	150	2,68
Inner stirrups	2	12	226	150	1,51 +
	4		628		4,19



Shear capacity concrete

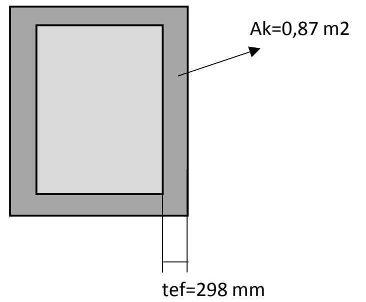
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,38	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,19	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0006	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1315	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1489	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	9869	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{\left(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd}\right)}$	=	1763	[kN]

$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} \quad = \quad 2776 \text{ [kN]}$$

z	=	1524	[mm]	
θ_{stp}	=	90	[°]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[°]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[°]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	3,20	[N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	1489	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	9869	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

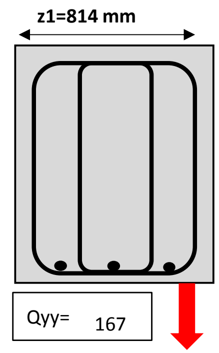
$Area\ (A)$	=	1,58	[m2]	
$Perimeter\ (u)$	=	5,3	[m]	
$t_{ef} = A/u$	=	298	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	114	[mm]	
A_k	=	0,87	[m2]	
u_k	=	4,11	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	3749	[kN]	*(Capacity compressive diagonal)*

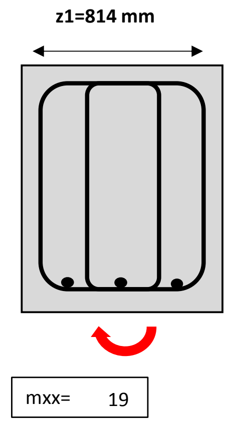
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814	[mm]
$V_{Ed,sup}$	=	167	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	254	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	289	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	23	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	35	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,07	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,33	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

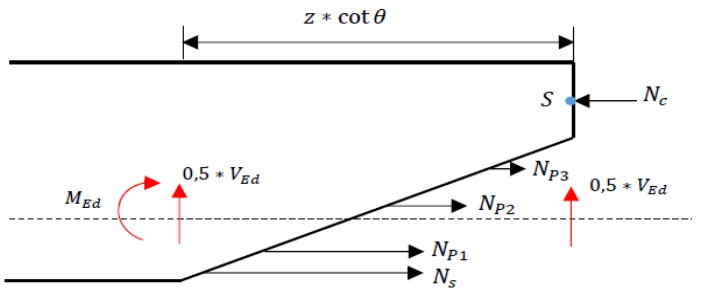
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	0,85	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	1,51E-03	< 1,75E-03	[elastic stage]
x	=	1351,4	[mm]	
$M_{Ed} - M_{P_\infty}$	=	12796	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	411	[kN]	
$\sigma_{p\infty}$	=	1056	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,293	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	72	[kN]	
$N_{p1} = 3 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	6119	[kN]	
e_{p1}	=	82	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,18 [-]

Prestressing steel

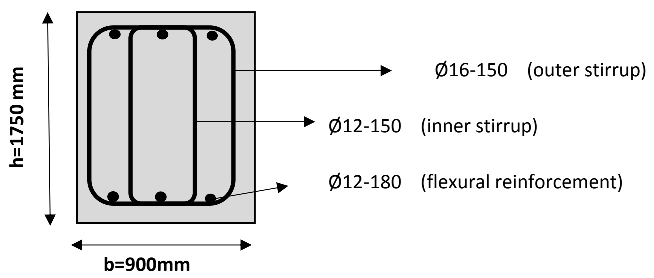
$U.C. = \frac{N_{p1}}{3 * A_p * fpd}$ = 0,71 [-]

LC 1+ LC 2+ LC 3 + LC 5a + LC 7 + LC 9

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1693	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_{∞}	=	-12038	[kN]
V_{Ed}	=	0	[kN]
T_{Ed}	=	140	[kNm]
Q_{yy}	=	165	[kN/m]
m_{xx}	=	2	[kNm/m]
M_{Ed}	=	7987	[kNm]

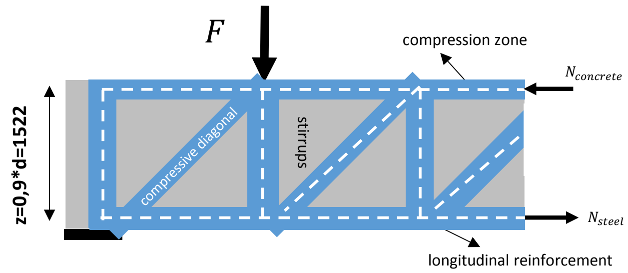
Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				



n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	8	12	113	180	905



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	

Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,38	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,19	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0006	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1315	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1489	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	9869	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	1763	[kN]

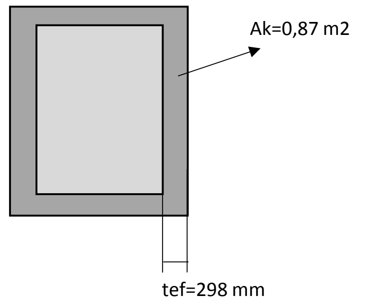
$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} \quad = \quad 2776 \text{ [kN]}$$

z	=	1524	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)

$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	3,20	[N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	1489	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	9869	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

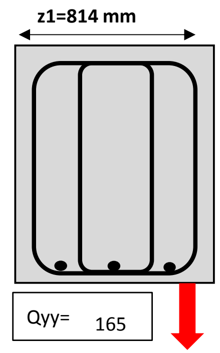
$Area\ (A)$	=	1,58	[m2]	
$Perimeter\ (u)$	=	5,3	[m]	
$t_{ef} = A/u$	=	298	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	114	[mm]	
A_k	=	0,87	[m2]	
u_k	=	4,11	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	3749	[kN]	*(Capacity compressive diagonal)*

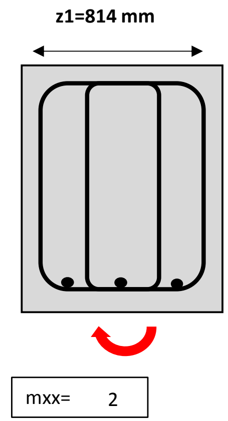
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814	[mm]
$V_{Ed,sup}$	=	165	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	251	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	254	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	2	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	3	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,06	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,31	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

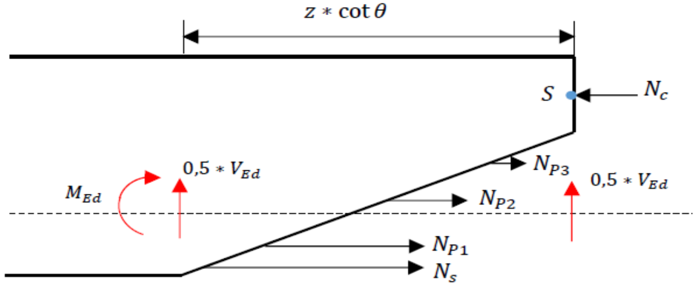
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	0,85	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	1,50E-03	< 1,75E-03	[elastic stage]
x	=	1356,2	[mm]	
$M_{Ed} - M_{P_\infty}$	=	12796	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	376	[kN]	
$\sigma_{p\infty}$	=	1056	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,293	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	71	[kN]	
$N_{p1} = 3 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	6116	[kN]	
e_{p1}	=	82	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,18 [-]

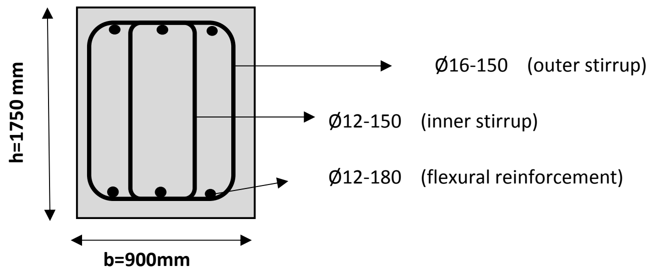
Prestressing steel

$U.C. = \frac{N_{p1}}{3 * A_p * fpd}$ = 0,71 [-]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1693	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_∞	=	-12038	[kN]
V_{Ed}	=	0	[kN]
T_{Ed}	=	140	[kNm]
Q_{yy}	=	18	[kN/m]
m_{xx}	=	43	[kNm/m]
M_{Ed}	=	1190	[kNm]

LC 1 + LC 2 + LC 4 + LC 6 + LC 9

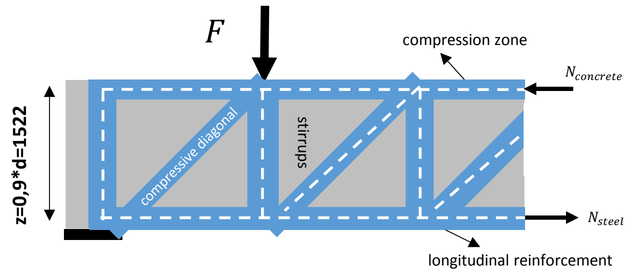


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	8	12	113	180	905



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	16	402	150	2,68
Inner stirrups	2	12	226	150	1,51 +
	4		628		4,19

Shear capacity concrete

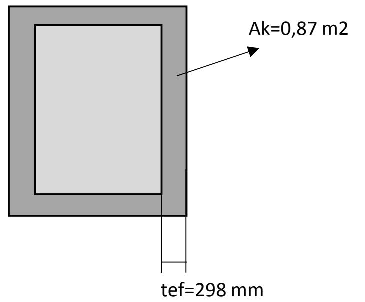
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,38	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,19	[-]
$\rho_l = A_{sl}/(b*d)$	=	0,0006	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1315	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1489	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	9869	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	1763	[kN]

$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 2776 \text{ [kN]}$$

z	=	1524	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	3,20 [N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	1489 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	9869 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

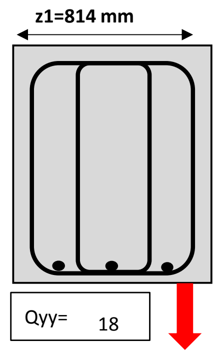
$Area\ (A)$	=	1,58 [m2]	
$Perimeter\ (u)$	=	5,3 [m]	
$t_{ef} = A/u$	=	298 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	114 [mm]	
A_k	=	0,87 [m2]	
u_k	=	4,11 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	3749 [kN]	*(Capacity compressive diagonal)*

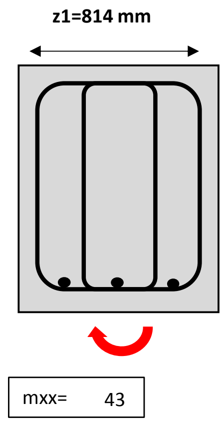
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814 [mm]
$V_{Ed,sup}$	=	18 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	27 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	108 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	53	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	81	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,05	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,25	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

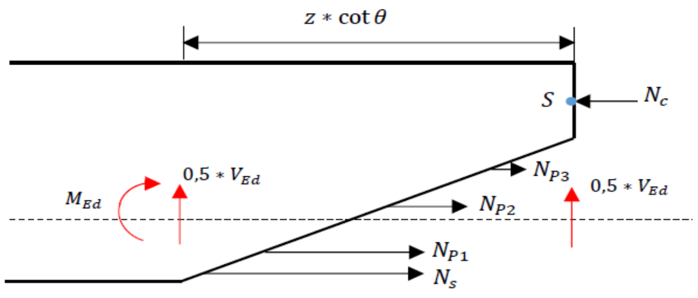
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	0,85 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	6,60E-04	<	1,75E-03	[elastic stage]
x	=	2983	[mm]		
$M_{Ed} - M_{P_\infty}$	=	6000	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	230	[kN]		
$\sigma_{p\infty}$	=	1056	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,293	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]		
$N_{p1} = 3 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	6019	[kN]		
e_{p1}	=	82	[mm]		



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

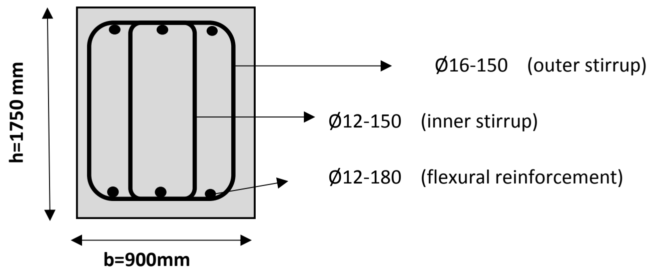
Prestressing steel

$U.C. = \frac{N_{p1}}{3 * A_p * fpd}$ = 0,69 [-]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1693	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_{∞}	=	-12038	[kN]
V_{Ed}	=	0	[kN]
T_{Ed}	=	140	[kNm]
Q_{yy}	=	16	[kN/m]
m_{xx}	=	26	[kNm/m]
M_{Ed}	=	1190	[kNm]

LC 1 + LC 2 + LC 4 + LC 7+ LC 9

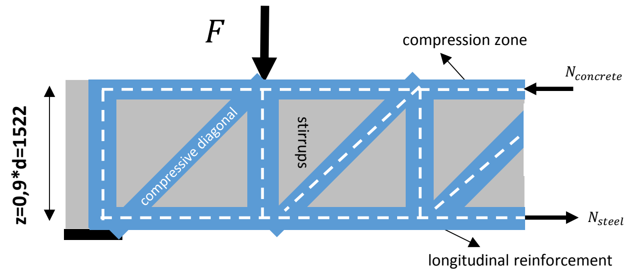


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	8	12	113	180	905



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	

Shear capacity concrete

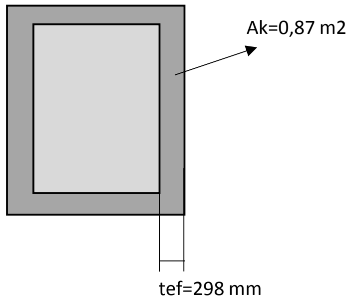
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,38	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,19	[-]
$\rho_l = A_{sl}/(b*d)$	=	0,0006	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1315	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1489	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	9869	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{\left(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd}\right)}$	=	1763	[kN]

$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 2776 \text{ [kN]}$$

z	=	1524	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	3,20 [N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	1489 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	9869 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

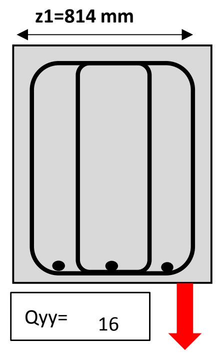
$Area\ (A)$	=	1,58 [m2]	
$Perimeter\ (u)$	=	5,3 [m]	
$t_{ef} = A/u$	=	298 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	114 [mm]	
A_k	=	0,87 [m2]	
u_k	=	4,11 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	3749 [kN]	*(Capacity compressive diagonal)*

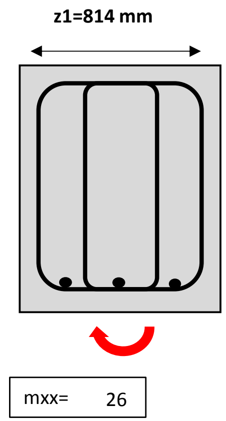
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814 [mm]
$V_{Ed,sup}$	=	16 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	24 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	73 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	32	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	49	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,04	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,23	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

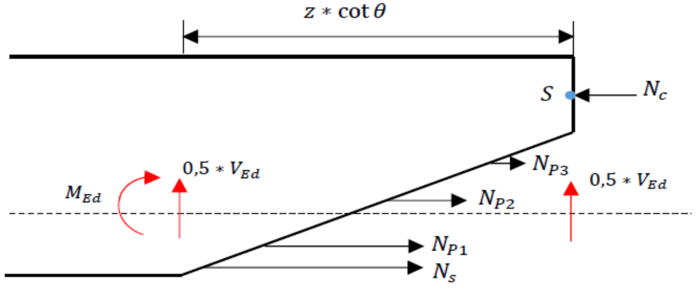
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	0,85 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	6,60E-04	<	1,75E-03	[elastic stage]
x	=	2990	[mm]		
$M_{Ed} - M_{P_\infty}$	=	6000	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	195	[kN]		
$\sigma_{p\infty}$	=	1056	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,293	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]		
$N_{p1} = 3 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	6019	[kN]		
e_{p1}	=	82	[mm]		



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

Prestressing steel

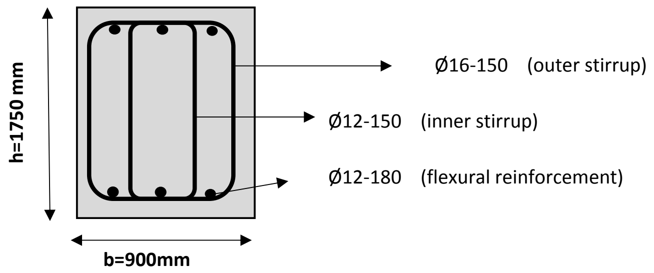
$U.C. = \frac{N_{p1}}{3 * A_p * fpd}$ = 0,69 [-]

LC 1 + LC 8

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1693	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_∞	=	-12038	[kN]
V_{Ed}	=	0	[kN]
T_{Ed}	=	0	[kNm]
Q_{yy}	=	17	[kN/m]
m_{xx}	=	17	[kNm/m]
M_{Ed}	=	-576	[kNm]

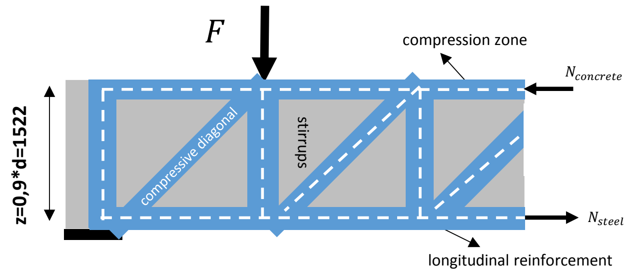
Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				



n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	8	12	113	180	905



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	

Shear capacity concrete

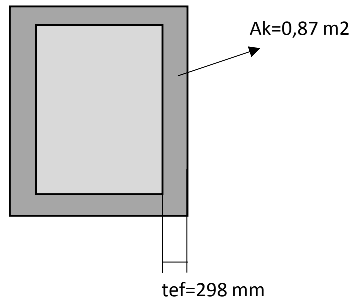
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,38	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,19	[-]
$\rho_l = A_{sl}/(b*d)$	=	0,0006	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1315	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1489	[kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	9869	[kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	1763	[kN]

$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s}$ = 2776 [kN]

z	=	1524	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	3,20	[N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	1489	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	9869	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

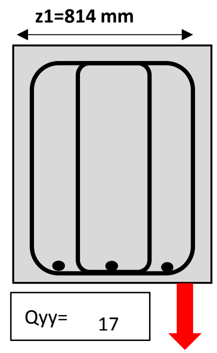
$Area\ (A)$	=	1,58	[m2]	
$Perimeter\ (u)$	=	5,3	[m]	
$t_{ef} = A/u$	=	298	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	114	[mm]	
A_k	=	0,87	[m2]	
u_k	=	4,11	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	3749	[kN]	*(Capacity compressive diagonal)*

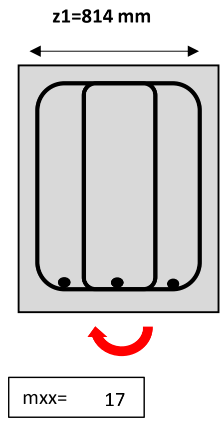
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814	[mm]
$V_{Ed,sup}$	=	17	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	26	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	58	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	21	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	32	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,01	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,03	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

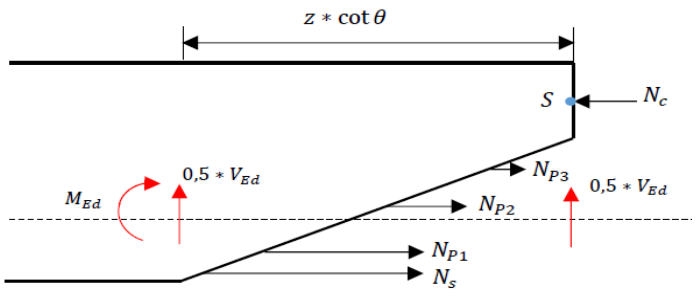
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	0,85	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	5,70E-04	<	1,75E-03 [elastic stage]
x	=	3467	[mm]	
$M_{Ed} - M_{P_\infty}$	=	4234	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	58	[kN]	
$\sigma_{p\infty}$	=	1056	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,293	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]	
$N_{p1} = 3 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	6019	[kN]	
e_{p1}	=	82	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

Prestressing steel

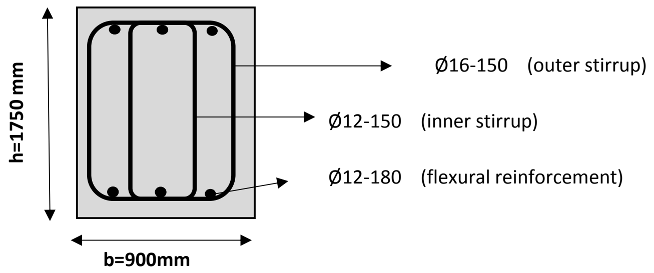
$U.C. = \frac{N_{p1}}{3 * A_p * fpd}$ = 0,69 [-]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1693	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_∞	=	-12038	[kN]
V_{Ed}	=	3	[kN]
T_{Ed}	=	33	[kNm]
Q_{yy}	=	124	[kN/m]
m_{xx}	=	50	[kNm/m]
M_{Ed}	=	7258	[kNm]

Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

LC 1 + LC 2 + LC 5b + LC 6 + LC 9



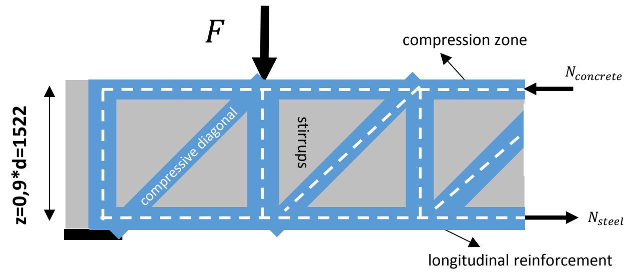
n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	8	12	113	180	905

Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	



Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,38	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,19	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0006	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1315	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1489	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	9869	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}^{\ 2} + \propto_l * \sigma_{cp} * f_{ctd})}$	=	1763	[kN]

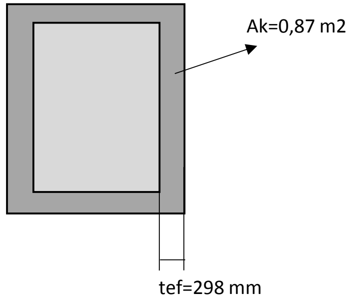
$V_{Rd,s} = z * f_{ywd} * \cot\theta * \dfrac{A_{sw}}{s}$ = 2776 [kN]

z	=	1524	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)

$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	3,20	[N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	1489	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	9869	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

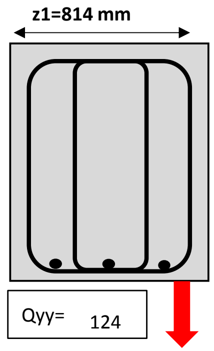
$Area\ (A)$	=	1,58	[m2]	
$Perimeter\ (u)$	=	5,3	[m]	
$t_{ef} = A/u$	=	298	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	114	[mm]	
A_k	=	0,87	[m2]	
u_k	=	4,11	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	3749	[kN]	*(Capacity compressive diagonal)*

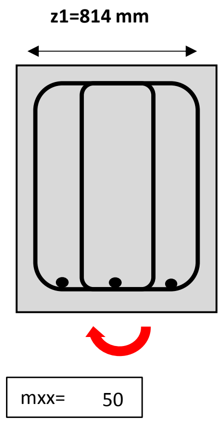
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814	[mm]
$V_{Ed,sup}$	=	124	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	189	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	286	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	62	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	94	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,04	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,17	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

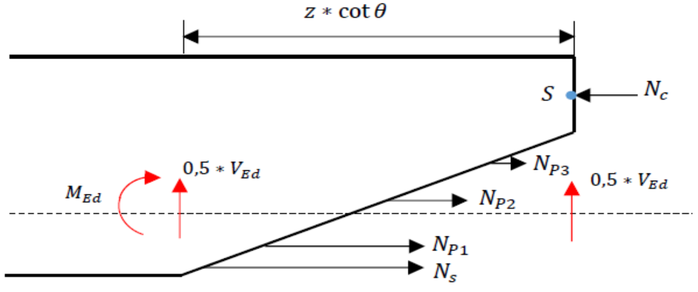
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	0,85	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	1,34E-03	< 1,75E-03	[elastic stage]
x	=	1505,7	[mm]	
$M_{Ed} - M_{P_\infty}$	=	12067	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	315	[kN]	
$\sigma_{p\infty}$	=	1056	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,293	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	32	[kN]	
$N_{p1} = 3 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	6051	[kN]	
e_{p1}	=	82	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,08 [-]

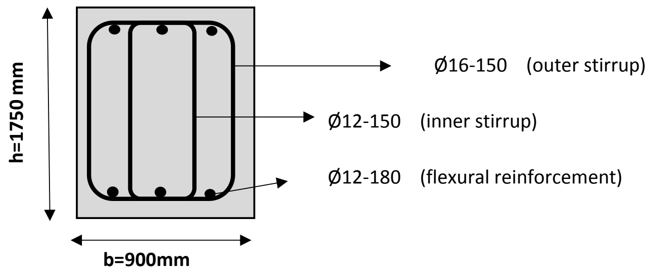
Prestressing steel

$U.C. = \frac{N_{p1}}{3 * A_p * fpd}$ = 0,70 [-]

Geometry			
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1693	[mm]
Area	=	1,58	[m2]
Area prestressing tendon Ap	=	1900	[mm2]

Loads			
P_{∞}	=	-12038	[kN]
V_{Ed}	=	3	[kN]
T_{Ed}	=	314	[kNm]
Q_{yy}	=	122	[kN/m]
m_{xx}	=	34	[kNm/m]
M_{Ed}	=	7258	[kNm]

LC 1 + LC 2 + LC 5b + LC 6 + LC 9

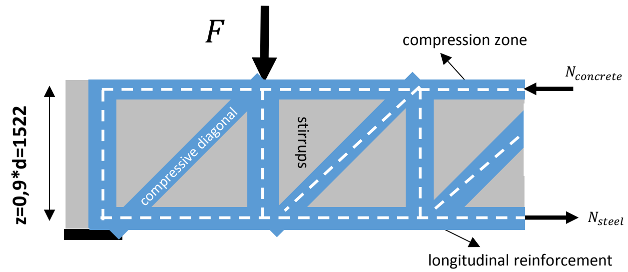


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1522	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	629	mm
Iyy	=	4,02E+11	mm4
S	=	5,65E+08	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	8	12	113	180	905



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]	
Outer stirrups	2	16	402	150	2,68	
Inner stirrups	2	12	226	150	1,51	+
	4		628		4,19	

Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,34	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,32	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,38	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,19	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0006	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	1315	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	1489	[kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	9869	[kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	1763	[kN]

$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s}$ = 2776 [kN]

z	=	1524	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	24,1	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)

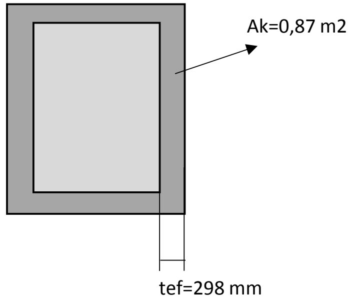
$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress = 3,20 [N/mm2] *(Concrete capacity is limited by flexural shear failure)*

$V_{Rd,c}$ = 1489 [kN] *(Concrete shear capacity)*

$V_{Rd,max}$ = 9869 [kN] *(Capacity compressive diagonal)*



Torsion capacity concrete

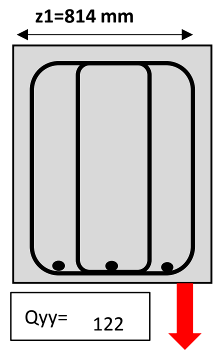
$Area\ (A)$	=	1,58	[m2]	
$Perimeter\ (u)$	=	5,3	[m]	
$t_{ef} = A/u$	=	298	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	114	[mm]	
A_k	=	0,87	[m2]	
u_k	=	4,11	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	693	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	3749	[kN]	*(Capacity compressive diagonal)*

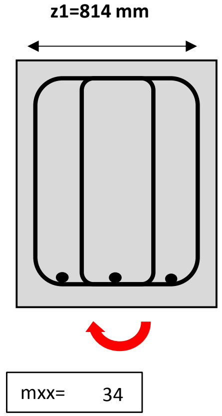
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814	[mm]
$V_{Ed,sup}$	=	122	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	186	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	251	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	41	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	62	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,11	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,56	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

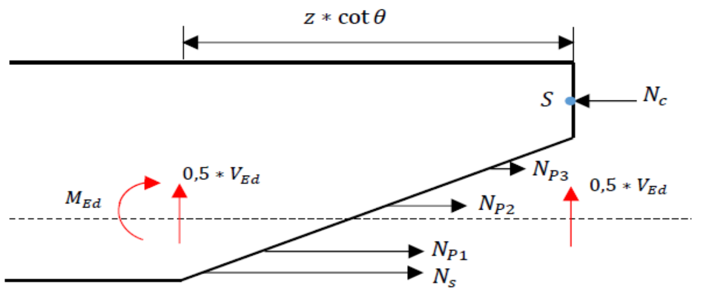
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	0,85	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	1,37E-03	<	1,75E-03 [elastic stage]
x	=	1473,7	[mm]	
$M_{Ed} - M_{P\infty}$	=	12067	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	524	[kN]	
$\sigma_{p\infty}$	=	1056	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,293	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	39	[kN]	
$N_{p1} = 3 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	6063	[kN]	
e_{p1}	=	82	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,10 [-]

Prestressing steel

$U.C. = \frac{N_{p1}}{3 * A_p * fpd}$ = 0,70 [-]

7.3 Bridge B - 6.10b - support

Forces

LC	type	Prestress P [kN]	Prestress σxx [N/mm2]	Bending moment M [kNm]	Bending moment σxx [N/mm2]	Total hor. normal stress σxx [N/mm2]
1	self-weight			-67	-0,03	
2	ballast			-78	-0,04	
3a	Mobile Max. (LM71)			-71	-0,03	
3b	Mobile Max. (SW/2)			-51	-0,02	
4	Mobile Min. (SW/2)			-253	-0,12	
5	Support settelement max			0	0,00	
6	Support settelement min			0	0,00	
7	Prestress t=0	-22826	-8,65	5045	2,36	
8	Prestress t = ∞	-20886	-7,91	4616	2,16	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-20886	-7,91	4400	2,06	-5,85
LC 1 + LC 2 + LC 3a + LC 6 + LC8		-20886	-7,91	4400	2,06	-5,85
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-20886	-7,91	4219	1,97	-5,94
LC 1 + LC 2 + LC 4 + LC 6 + LC8		-20886	-7,91	4219	1,97	-5,94
LC 1 + LC 7		-22826	-8,65	4978	2,33	-6,32
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-20886	-7,91	4421	2,07	-5,84
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		-20886	-7,91	4421	2,07	-5,84

LC	type	Suspension force Qyy [kN]	Clamping moment mxx [kNm]	Shear force [kN]	Torsion [kNm]
1	self-weight	53	29	2123	-288
2	ballast	31	19	488	-142
3a	Mobile Max. (LM71)	142	-4	1573	-287
3b	Mobile Max. (SW/2)	101	29	1502	-205
4	Mobile Min. (SW/2)	101	73	0	-453
5	Support settelement max	-265	227	0	-589
6	Support settelement min	265	-227	0	589
7	Prestress t=0	-57	28	-2287	0
8	Prestress t = ∞	-52	26	-2093	0
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-92	297	2092	-1306
LC 1 + LC 2 + LC 3a + LC 6 + LC8		439	-157	2092	-128
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-132	374	519	-1472
LC 1 + LC 2 + LC 4 + LC 6 + LC8		398	-79	519	-294
LC 1 + LC 7		-4	57	-164	-288
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-132	330	2021	-1224
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		398	-124	2021	-46

Reinforcement

Load combination	Suspension force A_Q [mm2/mm]	Clamping moment A_m [mm2/mm]	Shear force A_V [mm2/mm]	Torsion A_T [mm2/mm]	Total amount of stirrups [mm2/mm]
LC 1 + LC 2 + LC 3a + LC 5 + LC8	0,42	1,21	1,00	0,41	2,54
LC 1 + LC 2 + LC 3a + LC 6 + LC8	0,00	0,00	0,00	0,00	0,57
LC 1 + LC 2 + LC 4 + LC 5 + LC8	0,60	1,53	0,25	0,46	2,72
LC 1 + LC 2 + LC 4 + LC 6 + LC8	0,00	0,00	0,00	0,00	0,57
LC 1 + LC 7	0,00	0,00	0,00	0,00	0,57
LC 1 + LC 2 + LC 3b + LC 5 + LC8	0,60	1,35	0,97	0,38	2,82
LC 1 + LC 2 + LC 3b + LC 6 + LC 8	0,00	0,00	0,00	0,00	0,57

Type of reinforcement	Required amount (mm2/m)	Applied reinforcement	Applied amount (mm2/m)	U.C.
Outer stirrup	2818	Ø20-100	3142	**0,90**

Load combination	Unity check [-]	
	Longitudinal reinforcement	Prestress steel
LC 1 + LC 2 + LC 3a + LC 5 + LC8	0,06	0,67
LC 1 + LC 2 + LC 3a + LC 6 + LC8	**0,09**	**0,67**
LC 1 + LC 2 + LC 4 + LC 5 + LC8	0,00	0,67
LC 1 + LC 2 + LC 4 + LC 6 + LC8	0,00	0,67
LC 1 + LC 7	0,00	0,67
LC 1 + LC 2 + LC 3b + LC 5 + LC8	0,09	0,67
LC 1 + LC 2 + LC 3b + LC 6 + LC 8	0,04	0,67

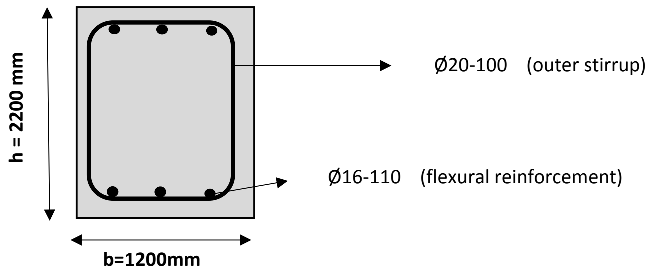
Parameters

Sectional properties		
b_{girder}	1200	[mm]
h_{girder}	2200	[mm]
t_{floor}	650	[mm]
b_{floor}	5000	[mm]
$0{,}5 * A_{bridge}$	2640000	[mm2]
$0{,}5 * I_{yy}$	1,73E+12	[mm4]
y	808	[mm]
z	808	[mm]

Geometry			
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

Loads			
P_∞	=	-20886	[kN]
V_{Ed}	=	2092	[kN]
T_{Ed}	=	1306	[kNm]
Q_{yy}	=	92	[kN/m]
m_{xx}	=	297	[kNm/m]
M_{Ed}	=	4400	[kNm]

LC 1 + LC 2 + LC 3a + LC 5 + LC8

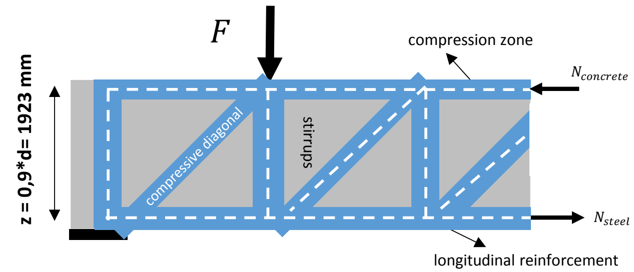


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2,0	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	20	16	201	110	4021

Stirrups	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	100	6,28
+	2		628		6,28



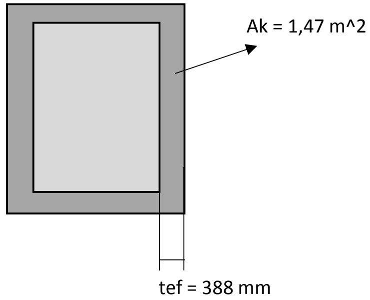
Shear capacity concrete			
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0016	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2508	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	11570	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{\left(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd}\right)}$	=	3102	[kN]

$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 13143 \text{ [kN]}$$

z	=	1923	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	25,7	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-5,84 [N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	3102 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	11570 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

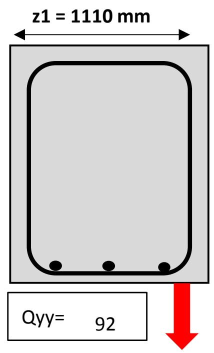
$Area\ (A)$	=	2,64 [m2]	
$Perimeter\ (u)$	=	6,8 [m]	
$t_{ef} = A/u$	=	388 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126 [mm]	
A_k	=	1,47 [m2]	
u_k	=	5,25 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	5724 [kN]	*(Capacity compressive diagonal)*

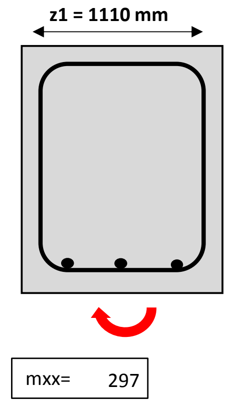
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110 [mm]
$V_{Ed,sup}$	=	92 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	442 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	3818 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	267	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	1284	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,56	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	1,65	>	1,00	*(Capacity of concrete insufficient, apply stirrups)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	1,00	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	0,41	[mm2/mm]

Minimum reinforcement

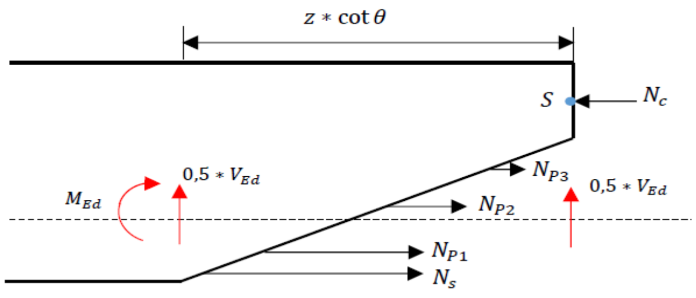
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	1,14 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	0,42	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	1,21	[mm2/mm]

Equilibrium of forces

ε_c	=	1,30E-03	< 1,75E-03	[elastic stage]
x	=	1952,9	[mm]	
$M_{Ed} - M_{P_\infty}$	=	-216	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	4672	[kN]	
$\sigma_{p\infty}$	=	967	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	103	[kN]	
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5218	[kN]	
e_{p1}	=	200	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,06 [-]

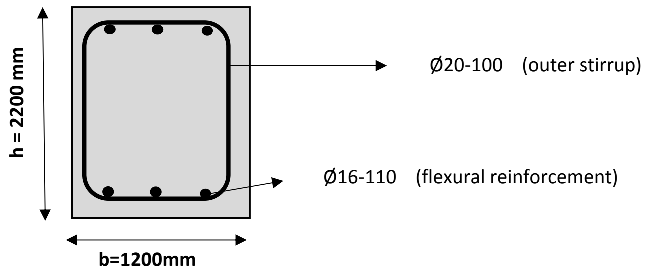
Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,67 [-]

Geometry			
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

Loads			
P_∞	=	-20886	[kN]
V_{Ed}	=	2092	[kN]
T_{Ed}	=	128	[kNm]
Q_{yy}	=	439	[kN/m]
m_{xx}	=	157	[kNm/m]
M_{Ed}	=	4400	[kNm]

LC 1 + LC 2 + LC 3a + LC 6 + LC8

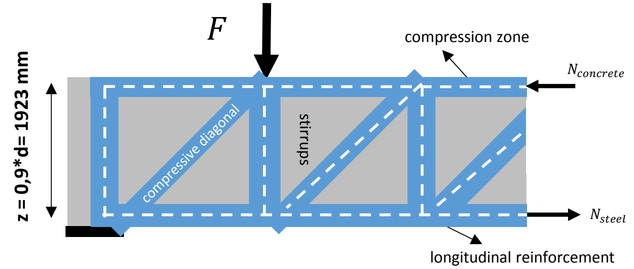


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	808 mm
Iyy	=	1,06E+12 mm4
S	=	1,16E+09 mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	20	16	201	110	4021



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	100	6,28
+					
	2		628		6,28

Shear capacity concrete

Coefficient Crd,c	=	0,12
Coefficient k1	=	0,15
Coefficient v1=v	=	0,52 [N/mm2]
Coefficient k	=	1,31
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66 [N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66 [N/mm2]
$\propto_{cw}$	=	1,20 [-]
$\rho_l = A_{sl}/(b*d)$	=	0,0016
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2508 [kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	2587 [kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	11570 [kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102 [kN]

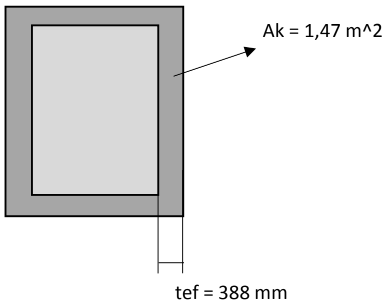
$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 13143 \text{ [kN]}$$

z	=	1923	[mm]	
θ_{stp}	=	90	[°]	(angle of the stirrups)
$\theta_{diagonal}$	=	25,7	[°]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[°]	(assumed angle comp. diagonal)

$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-5,84 [N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	3102 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	11570 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

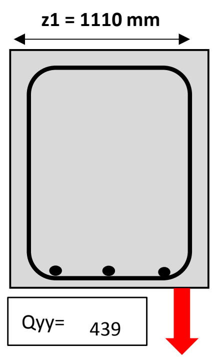
$Area\ (A)$	=	2,64 [m2]	
$Perimeter\ (u)$	=	6,8 [m]	
$t_{ef} = A/u$	=	388 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126 [mm]	
A_k	=	1,47 [m2]	
u_k	=	5,25 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	5724 [kN]	*(Capacity compressive diagonal)*

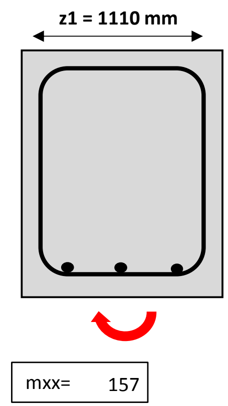
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110 [mm]
$V_{Ed,sup}$	=	439 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	2109 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	4879 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	141	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	678	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,44	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,95	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

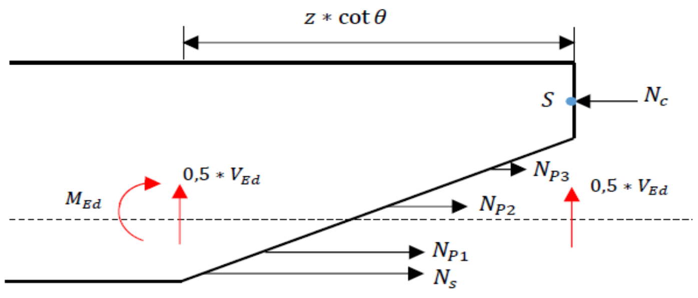
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	1,14	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	1,36E-03	< 1,75E-03	[elastic stage]
x	=	1868,8	[mm]	
$M_{Ed} - M_{P_\infty}$	=	-216	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	4963	[kN]	
$\sigma_{p\infty}$	=	967	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	165	[kN]	
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5238	[kN]	
e_{p1}	=	200	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,09 [-]

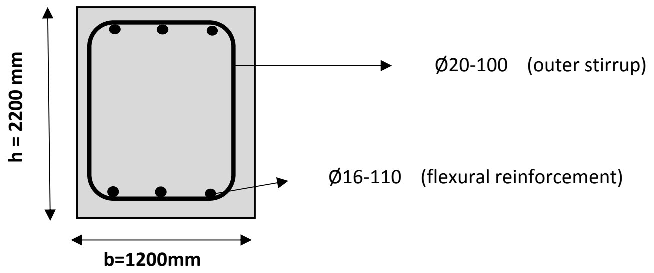
Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,67 [-]

Geometry			
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

Loads			
P_∞	=	-20886	[kN]
V_{Ed}	=	519	[kN]
T_{Ed}	=	1472	[kNm]
Q_{yy}	=	132	[kN/m]
m_{xx}	=	374	[kNm/m]
M_{Ed}	=	4219	[kNm]

LC 1 + LC 2 + LC 4 + LC 5 + LC8

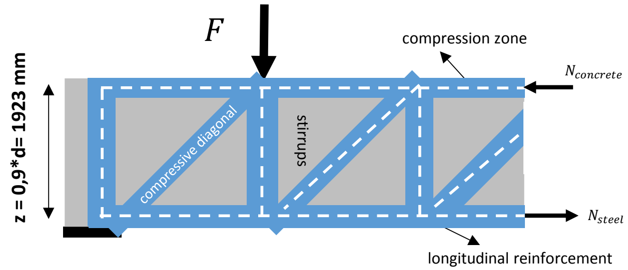


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	20	16	201	110	4021



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	100	6,28
+					
	2		628		6,28

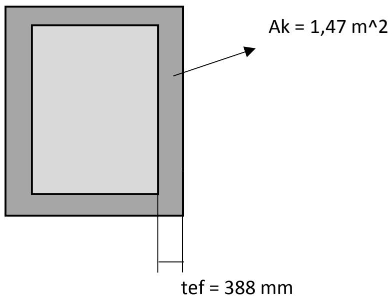
Shear capacity concrete			
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0,035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0016	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2508	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	11570	[kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s}$ = 13143 [kN]

z	=	1923	[mm]	
θ_{stp}	=	90	[°]	(angle of the stirrups)
$\theta_{diagonal}$	=	25,7	[°]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[°]	(assumed angle comp. diagonal)
				$21,8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-5,84 [N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	3102 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	11570 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

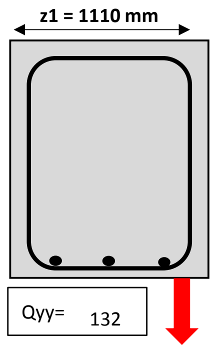
$Area\ (A)$	=	2,64 [m2]	
$Perimeter\ (u)$	=	6,8 [m]	
$t_{ef} = A/u$	=	388 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126 [mm]	
A_k	=	1,47 [m2]	
u_k	=	5,25 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	5724 [kN]	*(Capacity compressive diagonal)*

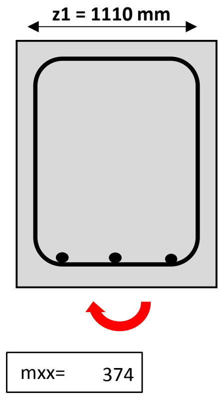
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110 [mm]
$V_{Ed,sup}$	=	132 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	637 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	2776 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	337	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	1620	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,50	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	1,29	>	1,00	*(Capacity of concrete insufficient, apply stirrups)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	0,25	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	0,46	[mm2/mm]

Minimum reinforcement

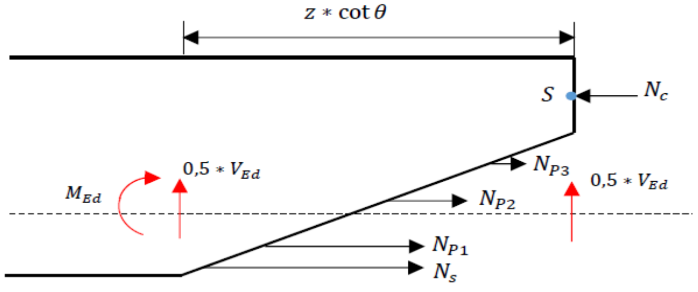
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	1,14 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	0,60	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	1,53	[mm2/mm]

Equilibrium of forces

ε_c	=	1,11E-03	<	1,75E-03	[elastic stage]
x	=	2266,8	[mm]		
$M_{Ed} - M_{P_\infty}$	=	-397	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	3738	[kN]		
$\sigma_{p\infty}$	=	967	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5221	[kN]		
e_{p1}	=	200	[mm]		



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

Prestressing steel

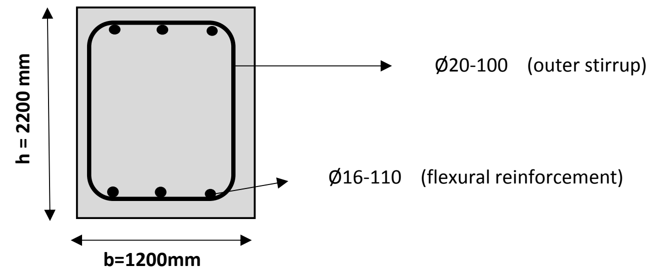
$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,67 [-]

LC 1 + LC 2 + LC 4 + LC 6 + LC8

Geometry			
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

Loads			
P_{∞}	=	-20886	[kN]
V_{Ed}	=	519	[kN]
T_{Ed}	=	294	[kNm]
Q_{yy}	=	398	[kN/m]
m_{xx}	=	79	[kNm/m]
M_{Ed}	=	4219	[kNm]

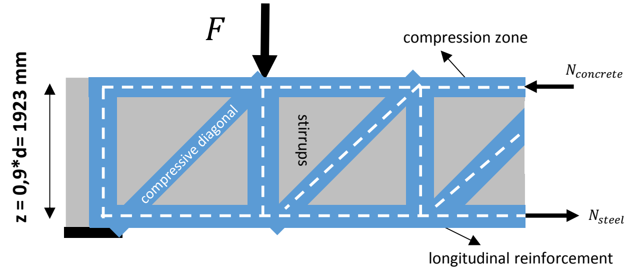
Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				



n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	20	16	201	110	4021



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	100	6,28
+					
	2		628		6,28

Shear capacity concrete

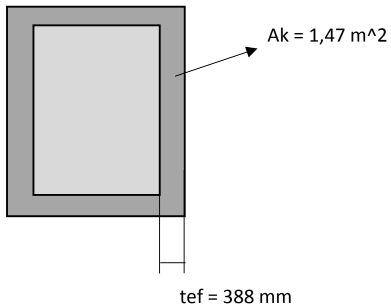
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0016	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2508	[kN]
$V_{Rd,min} = \left[v_{min} + k_1 * \sigma_{cp}\right\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	11570	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 13143 \text{ [kN]}$$

z	=	1923	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	25,7	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-5,84	[N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	3102	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	11570	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

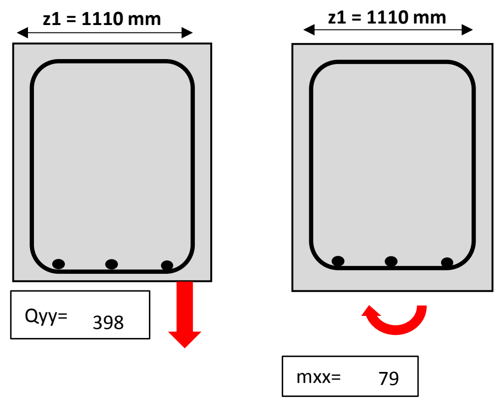
$Area\ (A)$	=	2,64	[m2]	
$Perimeter\ (u)$	=	6,8	[m]	
$t_{ef} = A/u$	=	388	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126	[mm]	
A_k	=	1,47	[m2]	
u_k	=	5,25	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	5724	[kN]	*(Capacity compressive diagonal)*

Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110	[mm]
$V_{Ed,sup}$	=	398	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	1913	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	2778	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	72	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	346	[kN]



Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,29	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,51	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

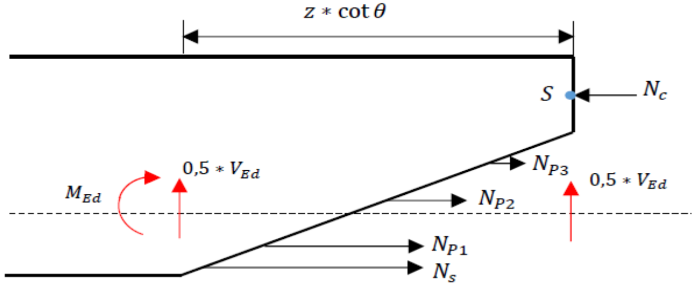
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	1,14	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	9,90E-04	<	1,75E-03	[elastic stage]
x	=	2522	[mm]		
$M_{Ed} - M_{P_\infty}$	=	-397	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	2970	[kN]		
$\sigma_{p\infty}$	=	967	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5221	[kN]		
e_{p1}	=	200	[mm]		



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

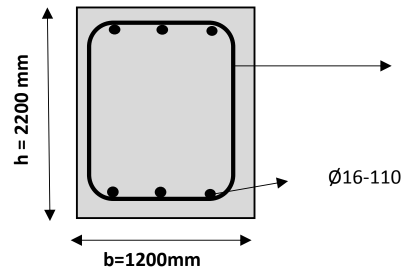
Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,67 [-]

Geometry			
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

Loads			
P_∞	=	-20886	[kN]
V_{Ed}	=	164	[kN]
T_{Ed}	=	288	[kNm]
Q_{yy}	=	4	[kN/m]
m_{xx}	=	57	[kNm/m]
M_{Ed}	=	4978	[kNm]

LC 1 + LC 7

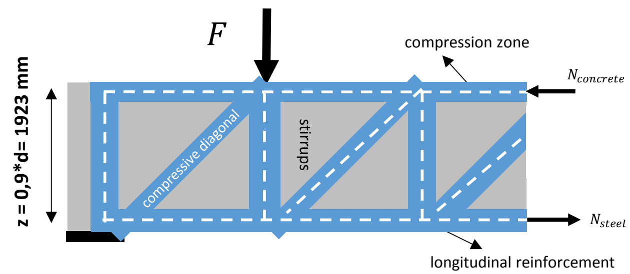


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement

	n [rebars]	$\varnothing_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	20	16	201	110	4021



Stirrups

	n [sections]	$\varnothing_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	100	6,28
+					
	2		628		6,28

Shear capacity concrete

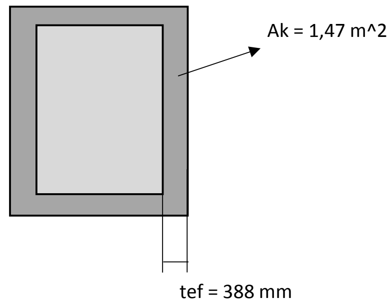
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b*d)$	=	0,0016	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2508	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	11570	[kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s}$ = 13143 [kN]

z	=	1923	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	25,7	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-5,84 [N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	3102 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	11570 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

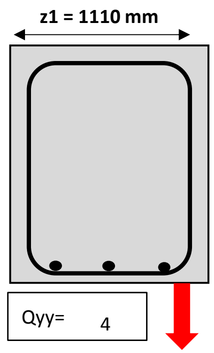
$Area\ (A)$	=	2,64 [m2]	
$Perimeter\ (u)$	=	6,8 [m]	
$t_{ef} = A/u$	=	388 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126 [mm]	
A_k	=	1,47 [m2]	
u_k	=	5,25 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	5724 [kN]	*(Capacity compressive diagonal)*

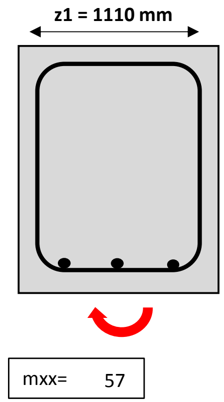
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110 [mm]
$V_{Ed,sup}$	=	4 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	20 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	434 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	52	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	250	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,09	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,26	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

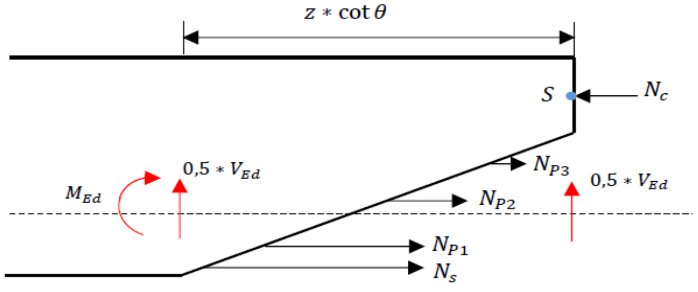
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	1,14 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	7,70E-04	<	1,75E-03	[elastic stage]
x	=	3230	[mm]		
$M_{Ed} - M_{P_\infty}$	=	361	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	622	[kN]		
$\sigma_{p\infty}$	=	967	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5221	[kN]		
e_{p1}	=	200	[mm]		



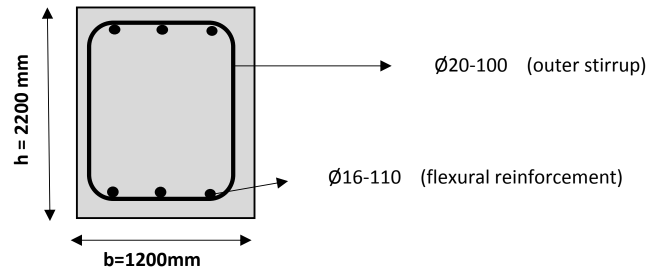
Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,67 [-]

LC 1 + LC 2 + LC 3b + LC 5 + LC8



Geometry

height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

Loads

P_∞	=	-20886	[kN]
V_{Ed}	=	2021	[kN]
T_{Ed}	=	1224	[kNm]
Q_{yy}	=	132	[kN/m]
m_{xx}	=	330	[kNm/m]
M_{Ed}	=	4421	[kNm]

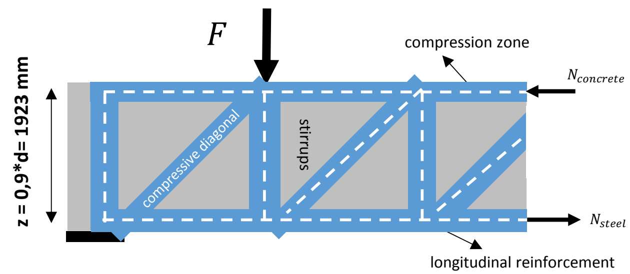
Materials

compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	20	16	201	110	4021



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	100	6,28
+					
	2		628		6,28

Shear capacity concrete

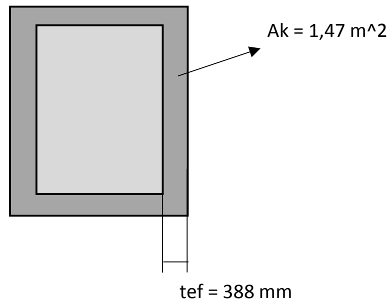
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b*d)$	=	0,0016	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2508	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	11570	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 13143 \text{ [kN]}$$

z	=	1923	[mm]	
θ_{stp}	=	90	[°]	(angle of the stirrups)
$\theta_{diagonal}$	=	25,7	[°]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[°]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-5,84 [N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	3102 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	11570 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

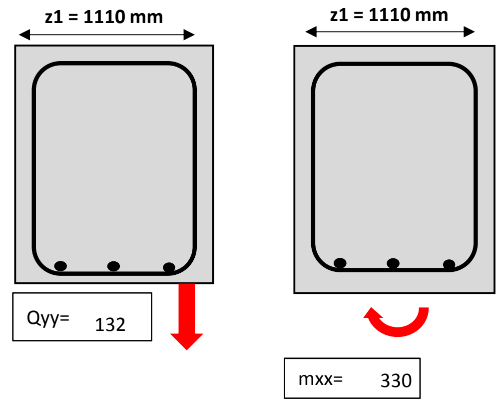
$Area\ (A)$	=	2,64 [m2]	
$Perimeter\ (u)$	=	6,8 [m]	
$t_{ef} = A/u$	=	388 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126 [mm]	
A_k	=	1,47 [m2]	
u_k	=	5,25 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	5724 [kN]	*(Capacity compressive diagonal)*

Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110 [mm]
$V_{Ed,sup}$	=	132 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	637 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	4086 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	297	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	1428	[kN]



Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,57	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	1,60	>	1,00	*(Capacity of concrete insufficient, apply stirrups)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	0,97	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	0,38	[mm2/mm]

Minimum reinforcement

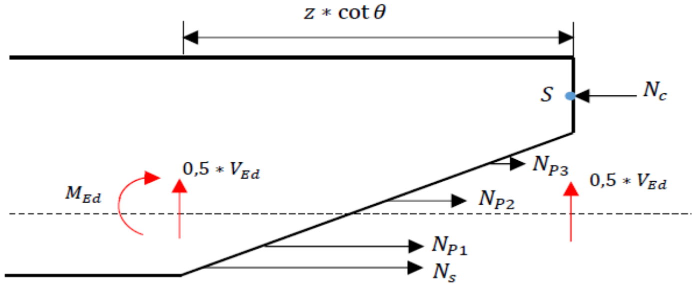
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	1,14 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	0,60	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	1,35	[mm2/mm]

Equilibrium of forces

ε_c	=	1,35E-03	<	1,75E-03	[elastic stage]
x	=	1888,2	[mm]		
$M_{Ed} - M_{P_\infty}$	=	-195	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	4886	[kN]		
$\sigma_{p\infty}$	=	967	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	150	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5233	[kN]		
e_{p1}	=	200	[mm]		



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,09 [-]

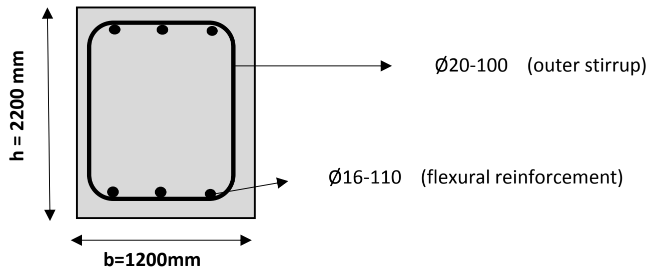
Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,67 [-]

Geometry			
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

Loads			
P_{∞}	=	-20886	[kN]
V_{Ed}	=	2021	[kN]
T_{Ed}	=	46	[kNm]
Q_{yy}	=	398	[kN/m]
m_{xx}	=	124	[kNm/m]
M_{Ed}	=	4421	[kNm]

LC 1 + LC 2 + LC 3b + LC 5 + LC8

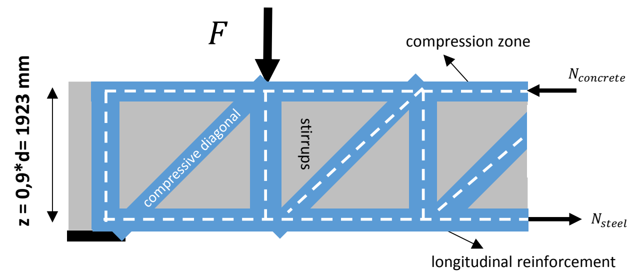


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	20	16	201	110	4021



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	100	6,28
+					
	2		628		6,28

Shear capacity concrete

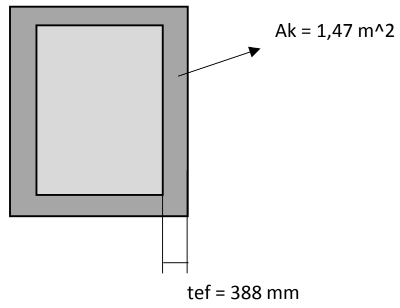
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0016	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2508	[kN]
$V_{Rd,min} = \left[v_{min} + k_1 * \sigma_{cp}\right\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	11570	[kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 13143 \text{ [kN]}$$

z	=	1923	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	25,7	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	21,8	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	-5,84	[N/mm2]	*(Concrete capacity is limited by shear tension failure)*
$V_{Rd,c}$	=	3102	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	11570	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

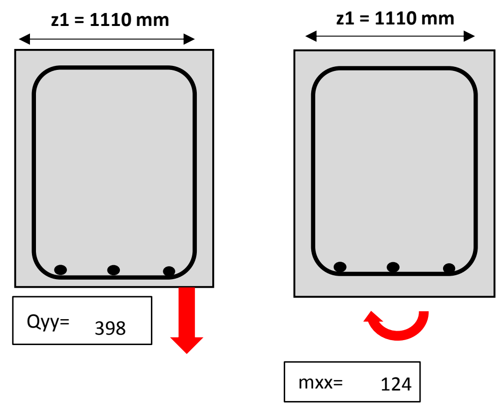
$Area\ (A)$	=	2,64	[m2]	
$Perimeter\ (u)$	=	6,8	[m]	
$t_{ef} = A/u$	=	388	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126	[mm]	
A_k	=	1,47	[m2]	
u_k	=	5,25	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	5724	[kN]	*(Capacity compressive diagonal)*

Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110	[mm]
$V_{Ed,sup}$	=	398	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	1913	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	4473	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	112	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	539	[kN]



Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,39	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,85	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

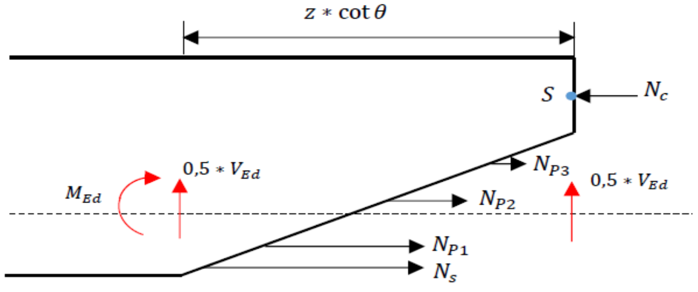
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	1,14	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	1,26E-03	<	1,75E-03 [elastic stage]
x	=	2000,6	[mm]	
$M_{Ed} - M_{P_\infty}$	=	-195	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	4503	[kN]	
$\sigma_{p\infty}$	=	967	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	73	[kN]	
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5208	[kN]	
e_{p1}	=	200	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,04 [-]

Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,67 [-]

7.4 Bridge B - 6.10b – 0,5L

Forces

LC	type	Prestress P [kN]	Prestress σxx [N/mm2]	Bending moment M [kNm]	Bending moment σxx [N/mm2]	Total hor. normal stress σxx [N/mm2]
1	self-weight			16652	7,79	
2	ballast			3767	1,76	
3a	Mobile Max. (LM71)			12769	5,97	
3b	Mobile Max. (SW/2)			11994	5,61	
4	Mobile Min. (SW/2)			-253	-0,12	
5	Support settelement max			0	0,00	
6	Support settelement min			0	0,00	
7	Prestress t=0	-22826	-5,26	-12964	-6,07	
8	Prestress t = ∞	-20886	-4,81	-11862	-5,55	
LC 1 + LC 2 + LC 3a + LC 5 + LC8		-20886	-4,81	21326	9,98	5,17
LC 1 + LC 2 + LC 3a + LC 6 + LC8		-20886	-4,81	21326	9,98	5,17
LC 1 + LC 2 + LC 4 + LC 5 + LC8		-20886	-4,81	8305	3,89	-0,93
LC 1 + LC 2 + LC 4 + LC 6 + LC8		-20886	-4,81	8305	3,89	-0,93
LC 1 + LC 7		-22826	-5,26	3688	1,73	-3,53
LC 1 + LC 2 + LC 3b + LC 5 + LC8		-20886	-4,81	20551	9,61	4,80
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		-20886	-4,81	20551	9,61	4,80

LC	type	Suspension force Qyy [kN]	Clamping moment mxx [kNm]	Shear force [kN]	Torsion [kNm]
1	self-weight	53	14	0	0
2	ballast	31	5	0	0
3a	Mobile Max. (LM71)	142	-14	0	0
3b	Mobile Max. (SW/2)	101	-26	0	0
4	Mobile Min. (SW/2)	0	12	0	0
5	Support settelement max	1	7	0	203
6	Support settelement min	-1	-7	0	-203
7	Prestress t=0	-57	-19	0	0
8	Prestress t = ∞	-52	-17	0	0
LC 1 + LC 2 + LC 3a + LC 5 + LC8		175	-5	0	203
LC 1 + LC 2 + LC 3a + LC 6 + LC8		172	-19	0	-203
LC 1 + LC 2 + LC 4 + LC 5 + LC8		33	21	0	203
LC 1 + LC 2 + LC 4 + LC 6 + LC8		31	7	0	-203
LC 1 + LC 7		-4	-5	0	0
LC 1 + LC 2 + LC 3b + LC 5 + LC8		134	-16	0	203
LC 1 + LC 2 + LC 3b + LC 6 + LC 8		132	-31	0	-203

Reinforcement

Load combination	Suspension force A_Q [mm2/mm]	Clamping moment A_m [mm2/mm]	Shear force A_V [mm2/mm]	Torsion A_T [mm2/mm]	Total amount of stirrups [mm2/mm]
LC 1 + LC 2 + LC 3a + LC 5 + LC8	0,00	0,00	0,00	0,00	0,57
LC 1 + LC 2 + LC 3a + LC 6 + LC8	0,00	0,00	0,00	0,00	0,57
LC 1 + LC 2 + LC 4 + LC 5 + LC8	0,00	0,00	0,00	0,00	0,57
LC 1 + LC 2 + LC 4 + LC 6 + LC8	0,00	0,00	0,00	0,00	0,57
LC 1 + LC 7	0,00	0,00	0,00	0,00	0,57
LC 1 + LC 2 + LC 3b + LC 5 + LC8	0,00	0,00	0,00	0,00	0,57
LC 1 + LC 2 + LC 3b + LC 6 + LC 8	0,00	0,00	0,00	0,00	0,57

Type of reinforcement	Required amount (mm2/m)	Applied reinforcement	Applied amount (mm2/m)	U.C.
Outer stirrup	568	Ø20-200	1571	0,36

Load combination	Unity check [-] Longitudinal reinforcement	Prestress steel
LC 1 + LC 2 + LC 3a + LC 5 + LC8	0,55	0,71
LC 1 + LC 2 + LC 3a + LC 6 + LC8	0,56	0,71
LC 1 + LC 2 + LC 4 + LC 5 + LC8	0,00	0,67
LC 1 + LC 2 + LC 4 + LC 6 + LC8	0,00	0,67
LC 1 + LC 7	0,00	0,67
LC 1 + LC 2 + LC 3b + LC 5 + LC8	0,45	0,70
LC 1 + LC 2 + LC 3b + LC 6 + LC 8	0,45	0,70

Parameters

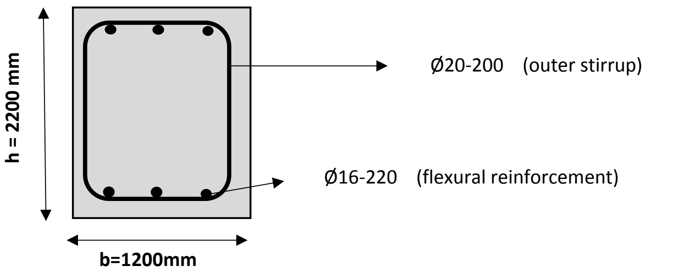
Sectional properties

b_{girder}	1200	[mm]
h_{girder}	2200	[mm]
t_{floor}	650	[mm]
b_{floor}	5000	[mm]
$0,5 * A_{bridge}$	4340000	[mm2]
$0,5 * I_{yy}$	1,73E+12	[mm4]
y	808	[mm]
z	808	[mm]

Geometry			
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

Loads			
P_{∞}	=	-20886	[kN]
V_{Ed}	=	0	[kN]
T_{Ed}	=	203	[kNm]
Q_{yy}	=	175	[kN/m]
m_{xx}	=	5	[kNm/m]
M_{Ed}	=	21326	[kNm]

LC 1 + LC 2 + LC 3a + LC 5 + LC8



Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2,0	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

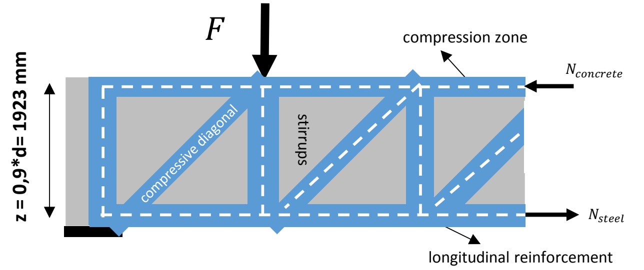
n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	10	16	201	220	2011

Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	200	3,14
+					
	2		628		3,14



Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b*d)$	=	0,0008	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2361	[kN]
$V_{Rd,min} = \left[v_{min} + k_1 * \sigma_{cp}\right\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	16778	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{\left(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd}\right)}$	=	3102	[kN]

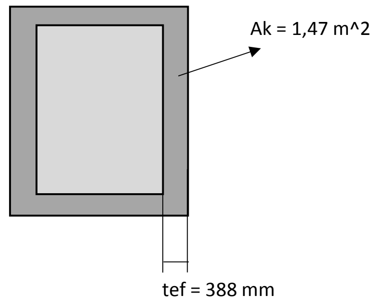
$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 2628 \text{ [kN]}$$

z	=	1923	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	21,8	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)

$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	5,17 [N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	2587 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	16778 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

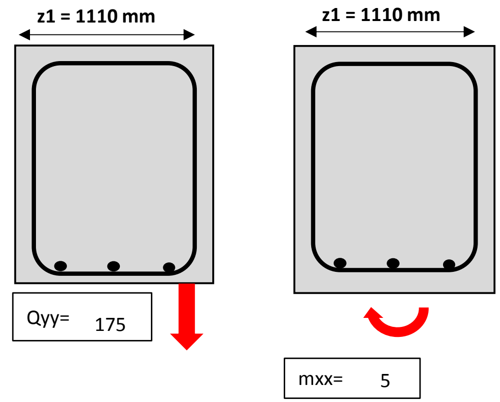
$Area\ (A)$	=	2,64 [m2]	
$Perimeter\ (u)$	=	6,8 [m]	
$t_{ef} = A/u$	=	388 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126 [mm]	
A_k	=	1,47 [m2]	
u_k	=	5,25 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	8300 [kN]	*(Capacity compressive diagonal)*

Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110 [mm]
$V_{Ed,sup}$	=	175 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	336 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	344 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	4	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	8	[kN]



Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,04	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,20	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

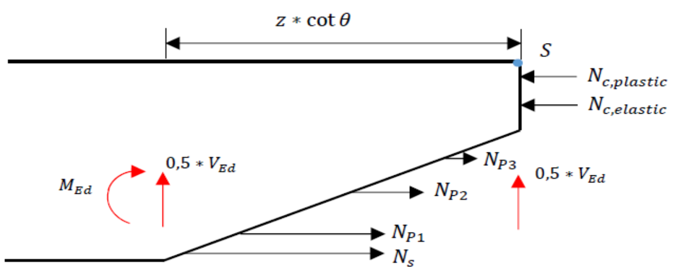
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	1,14	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	2,10E-03	>	1,75E-03	[elastic-plastic stage]
x	=	1381,1	[mm]		
$M_{Ed} - M_{P_\infty}$	=	33189	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	477	[kN]		
$\sigma_{p\infty}$	=	967	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	485	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5576	[kN]		
e_{p1}	=	76	[mm]		



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,55 [-]

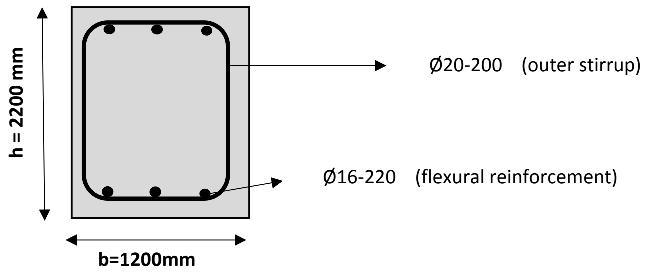
Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,71 [-]

Geometry				Loads			
height h	=	2200	[mm]	P_{∞}	=	-20886	[kN]
width b	=	1200	[mm]	V_{Ed}	=	0	[kN]
cover	=	35	[mm]	T_{Ed}	=	203	[kNm]
effective height	=	2137	[mm]	Q_{yy}	=	172	[kN/m]
Area	=	2,64	[m2]	m_{xx}	=	19	[kNm/m]
Area prestressing tendon Ap	=	2700	[mm2]	M_{Ed}	=	21326	[kNm]

Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

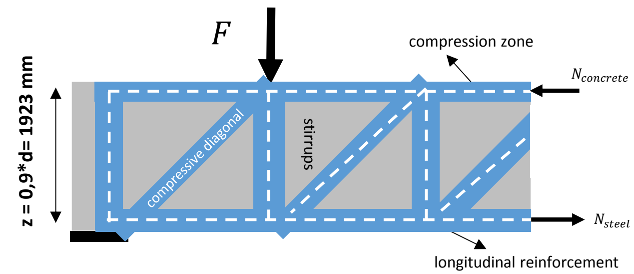
LC 1 + LC 2 + LC 3a + LC 6 + LC8



n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement

	n [rebars]	$\O_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	10	16	201	220	2011



Stirrups

	n [sections]	$\O_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	200	3,14
					+
	2		628		3,14

Shear capacity concrete

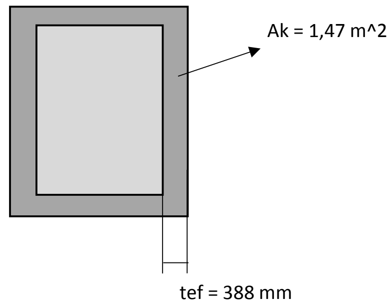
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0008	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2361	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	16778	[kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{({f_{ctd}}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s}$ = 2628 [kN]

z	=	1923	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	21,8	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	5,17 [N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	2587 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	16778 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

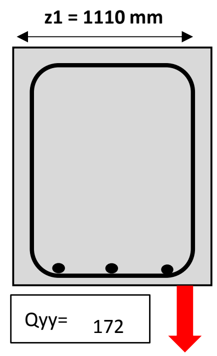
$Area\ (A)$	=	2,64 [m2]	
$Perimeter\ (u)$	=	6,8 [m]	
$t_{ef} = A/u$	=	388 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126 [mm]	
A_k	=	1,47 [m2]	
u_k	=	5,25 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	8300 [kN]	*(Capacity compressive diagonal)*

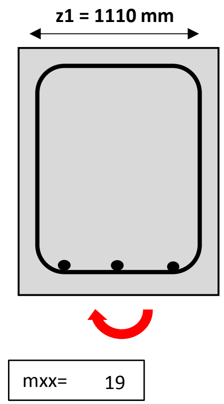
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110 [mm]
$V_{Ed,sup}$	=	172 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	331 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	364 [kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	17	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	33	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,05	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,21	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

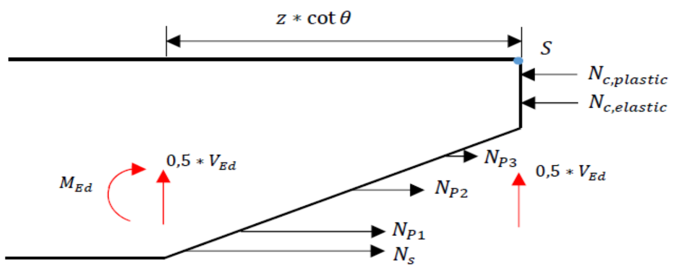
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	1,14 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	2,11E-03	>	1,75E-03	[elastic-plastic stage]
x	=	1379,8	[mm]		
$M_{Ed} - M_{P_\infty}$	=	33189	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	497	[kN]		
$\sigma_{p\infty}$	=	967	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	489	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5579	[kN]		
e_{p1}	=	76	[mm]		



Longitudinal Reinforcement

$$U.C. = \frac{N_s}{A_s * fywd}$$ = 0,56 [-]

Prestressing steel

$$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$$ = 0,71 [-]

Geometry

height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

Loads

P_{∞}	=	-20886	[kN]
V_{Ed}	=	0	[kN]
T_{Ed}	=	203	[kNm]
Q_{yy}	=	33	[kN/m]
m_{xx}	=	21	[kNm/m]
M_{Ed}	=	8305	[kNm]

Materials

compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

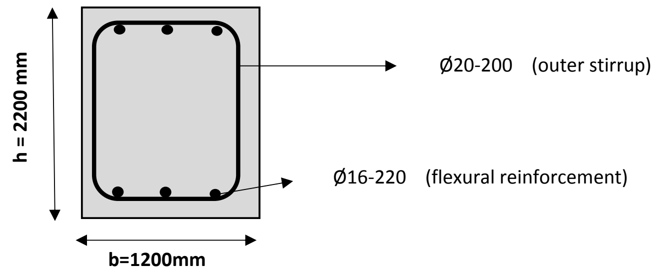
Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	10	16	201	220	2011

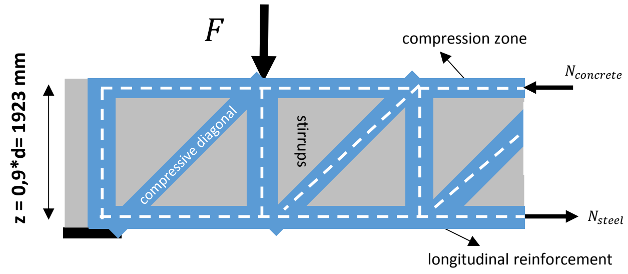
Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	200	3,14
+					
	2		628		3,14

LC 1 + LC 2 + LC 4 + LC 5 + LC8



n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3



Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0008	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2361	[kN]
$V_{Rd,min} = \left[v_{min} + k_1 * \sigma_{cp}\right\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	16778	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

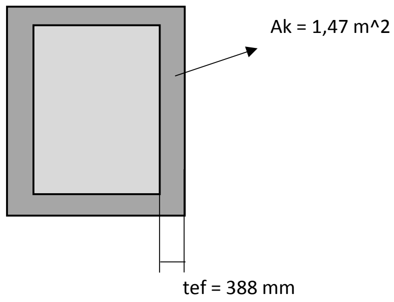
$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} \quad = \quad 2628 \text{ [kN]}$$

z	=	1923	[mm]	
θ_{stp}	=	90	[°]	(angle of the stirrups)
$\theta_{diagonal}$	=	21,8	[°]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[°]	(assumed angle comp. diagonal)

$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	5,17	[N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	2587	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	16778	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

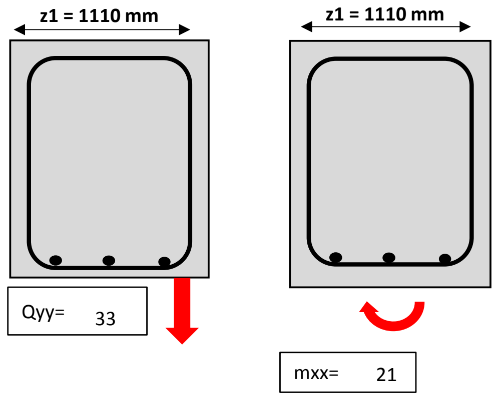
$Area\ (A)$	=	2,64	[m2]	
$Perimeter\ (u)$	=	6,8	[m]	
$t_{ef} = A/u$	=	388	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126	[mm]	
A_k	=	1,47	[m2]	
u_k	=	5,25	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	8300	[kN]	*(Capacity compressive diagonal)*

Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110	[mm]
$V_{Ed,sup}$	=	33	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	63	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	100	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	19	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	37	[kN]



Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,03	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,15	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

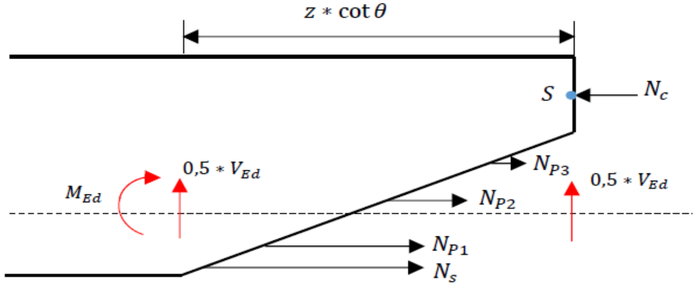
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	1,14	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	8,90E-04	<	1,75E-03	[elastic stage]
x	=	2870,7	[mm]		
$M_{Ed} - M_{P_\infty}$	=	20167	[kNm]		
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	233	[kN]		
$\sigma_{p\infty}$	=	967	[N/mm2]		
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]		
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]		
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5221	[kN]		
e_{p1}	=	76	[mm]		



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

Prestressing steel

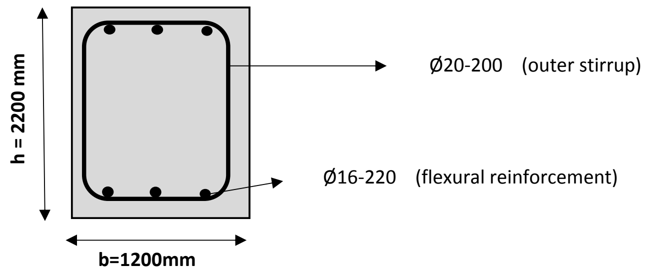
$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,67 [-]

Geometry			
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

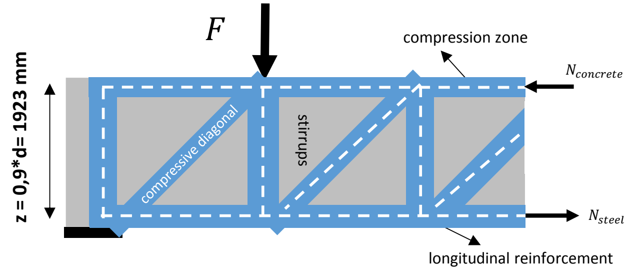
Loads			
P_∞	=	-20886	[kN]
V_{Ed}	=	0	[kN]
T_{Ed}	=	203	[kNm]
Q_{yy}	=	31	[kN/m]
m_{xx}	=	7	[kNm/m]
M_{Ed}	=	8305	[kNm]

Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

LC 1 + LC 2 + LC 4 + LC 6 + LC8



n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3



Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	10	16	201	220	2011

Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	200	3,14
+					
	2		628		3,14

Shear capacity concrete

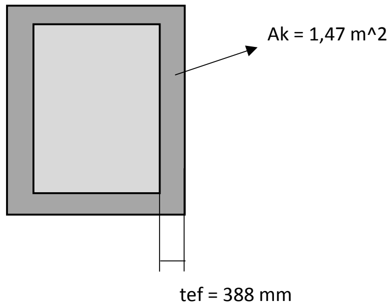
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0008	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2361	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	16778	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} \quad = \quad 2628 \text{ [kN]}$$

z	=	1923	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	21,8	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	5,17	[N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	2587	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	16778	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

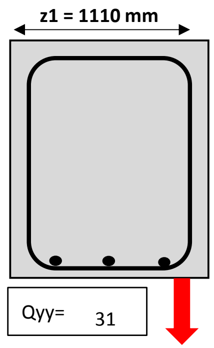
$Area\ (A)$	=	2,64	[m2]	
$Perimeter\ (u)$	=	6,8	[m]	
$t_{ef} = A/u$	=	388	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126	[mm]	
A_k	=	1,47	[m2]	
u_k	=	5,25	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	8300	[kN]	*(Capacity compressive diagonal)*

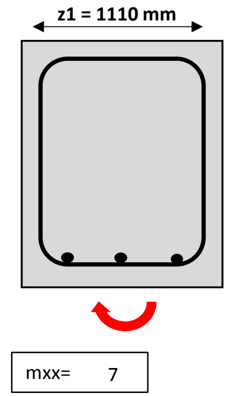
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110	[mm]
$V_{Ed,sup}$	=	31	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	59	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	71	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	6	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	12	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,03	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,15	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

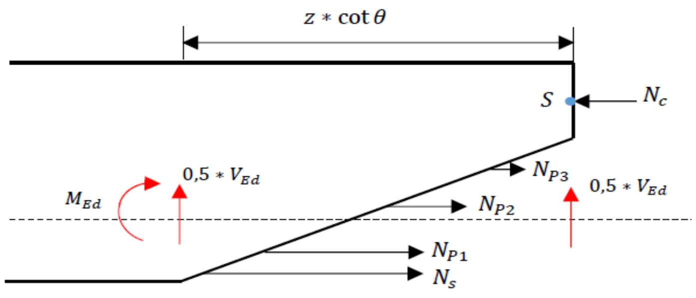
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	1,14	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	8,90E-04	< 1,75E-03	[elastic stage]
x	=	2874,7	[mm]	
$M_{Ed} - M_{P_\infty}$	=	20167	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	204	[kN]	
$\sigma_{p\infty}$	=	967	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]	
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5221	[kN]	
e_{p1}	=	76	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

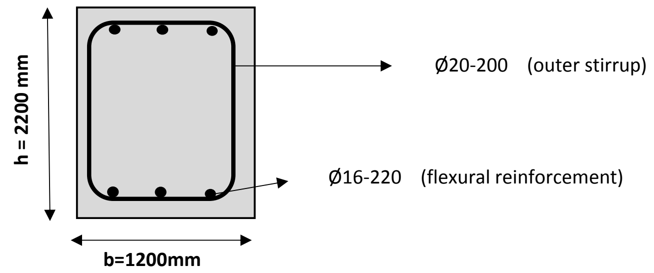
Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,67 [-]

Geometry				Loads			
height h	=	2200	[mm]	P_{∞}	=	-20886	[kN]
width b	=	1200	[mm]	V_{Ed}	=	0	[kN]
cover	=	35	[mm]	T_{Ed}	=	0	[kNm]
effective height	=	2137	[mm]	Q_{yy}	=	4	[kN/m]
Area	=	2,64	[m2]	m_{xx}	=	5	[kNm/m]
Area prestressing tendon Ap	=	2700	[mm2]	M_{Ed}	=	3688	[kNm]

Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

LC 1 + LC 7



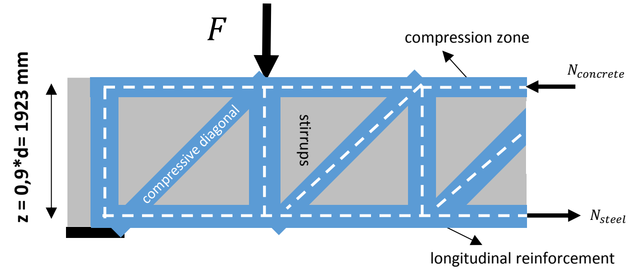
n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	10	16	201	220	2011

Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	200	3,14
+					
	2		628		3,14



Shear capacity concrete

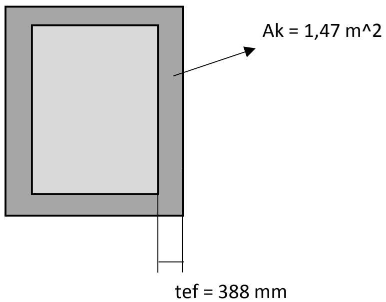
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b*d)$	=	0,0008	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2361	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	16778	[kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s}$ = 2628 [kN]

z	=	1923	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	21,8	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)
				$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	5,17	[N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	2587	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	16778	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

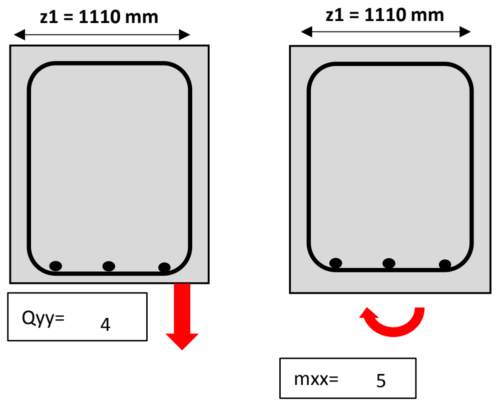
$Area\ (A)$	=	2,64	[m2]	
$Perimeter\ (u)$	=	6,8	[m]	
$t_{ef} = A/u$	=	388	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126	[mm]	
A_k	=	1,47	[m2]	
u_k	=	5,25	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	8300	[kN]	*(Capacity compressive diagonal)*

Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110	[mm]
$V_{Ed,sup}$	=	4	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	8	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	16	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	4	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	8	[kN]



Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,00	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,00	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

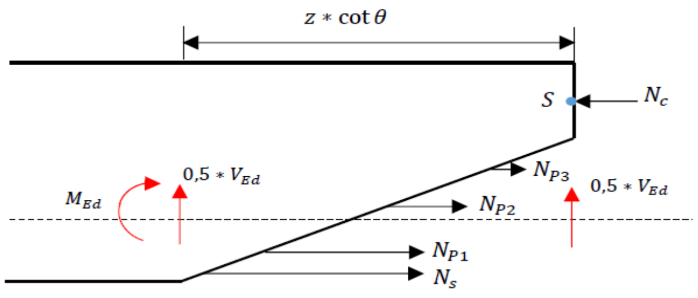
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	1,14	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	**7,10E-04**	<	1,75E-03 [elastic stage]
x	=	**3570,6**	[mm]	
$M_{Ed} - M_{P_\infty}$	=	15550	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	16	[kN]	
$\sigma_{p\infty}$	=	967	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	0	[kN]	
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5221	[kN]	
e_{p1}	=	**76**	[mm]	



Longitudinal Reinforcement

$U.C. = \frac{N_s}{A_s * fywd}$ = 0,00 [-]

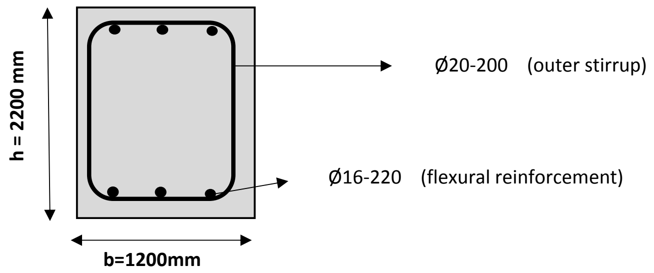
Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,67 [-]

Geometry			
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
Area	=	2,64	[m2]
Area prestressing tendon Ap	=	2700	[mm2]

Loads			
P_∞	=	-20886	[kN]
V_{Ed}	=	0	[kN]
T_{Ed}	=	203	[kNm]
Q_{yy}	=	134	[kN/m]
m_{xx}	=	16	[kNm/m]
M_{Ed}	=	20551	[kNm]

LC 1 + LC 2 + LC 3b + LC 5 + LC8

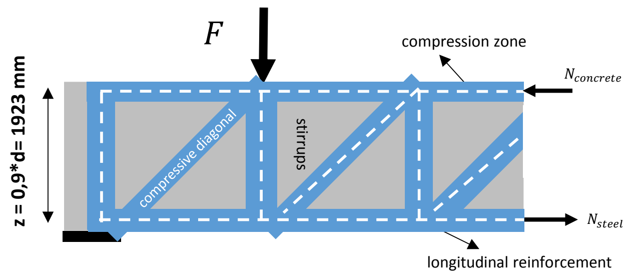


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	10	16	201	220	2011



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	200	3,14
+					
	2		628		3,14

Shear capacity concrete

Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b*d)$	=	0,0008	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2361	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \frac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	16778	[kN]
$V_{Rd,c} = \frac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

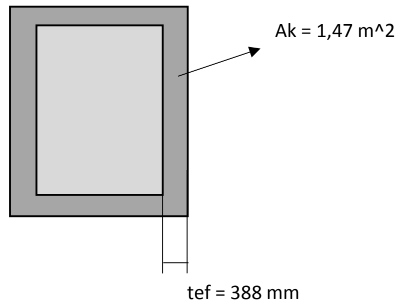
$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 2628\ [kN]$$

z	=	1923	[mm]	
θ_{stp}	=	90	[°]	(angle of the stirrups)
$\theta_{diagonal}$	=	21,8	[°]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[°]	(assumed angle comp. diagonal)

$21{,}8° \leq \theta \leq 45°$

Shear capacity concrete

Max. tensile bending stress	=	5,17	[N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	2587	[kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	16778	[kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete

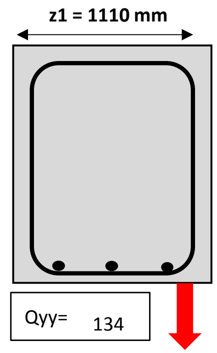
$Area\ (A)$	=	2,64	[m2]	
$Perimeter\ (u)$	=	6,8	[m]	
$t_{ef} = A/u$	=	388	[mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126	[mm]	
A_k	=	1,47	[m2]	
u_k	=	5,25	[m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519	[kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	8300	[kN]	*(Capacity compressive diagonal)*

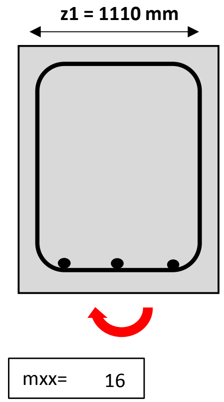
Suspension force

$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110	[mm]
$V_{Ed,sup}$	=	134	[kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	258	[kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	285	[kN]

Clamping moment

$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	14	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	27	[kN]





Total capacity concrete

$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,04	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,19	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups

Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

Minimum reinforcement

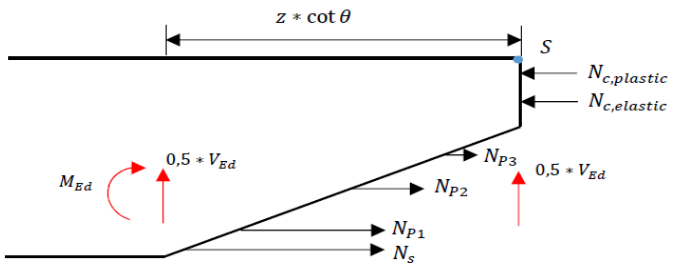
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947	[-]
$A_{s,min}$	=	1,14	[mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	1,95E-03	> 1,75E-03	[elastic-plastic stage]
x	=	1442,5	[mm]	
$M_{Ed} - M_{P_\infty}$	=	32413	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	418	[kN]	
$\sigma_{p\infty}$	=	967	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	396	[kN]	
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5508	[kN]	
e_{p1}	=	76	[mm]	



Longitudinal Reinforcement

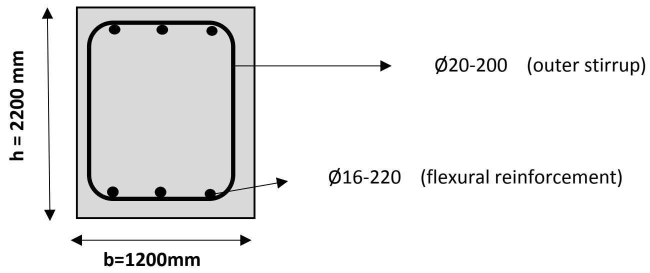
$U.C. = \frac{N_s}{A_s * fywd}$ = 0,45 [-]

Prestressing steel

$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$ = 0,70 [-]

Geometry				Loads			
height h	=	2200	[mm]	P_∞	=	-20886	[kN]
width b	=	1200	[mm]	V_{Ed}	=	0	[kN]
cover	=	35	[mm]	T_{Ed}	=	203	[kNm]
effective height	=	2137	[mm]	Q_{yy}	=	132	[kN/m]
Area	=	2,64	[m2]	m_{xx}	=	31	[kNm/m]
Area prestressing tendon Ap	=	2700	[mm2]	M_{Ed}	=	20551	[kNm]

LC 1 + LC 2 + LC 3b + LC 6 + LC 8

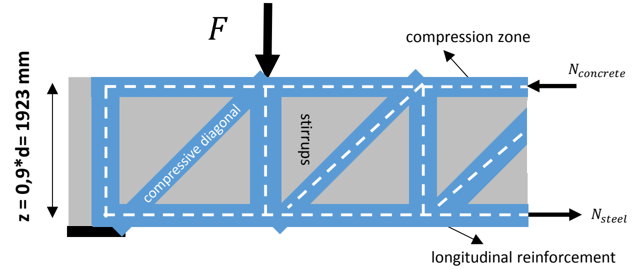


Materials							
compressive strength fck	=	35	[N/mm2]	fcd	=	23,3	[N/mm2]
yield strength stirrups fyk	=	500	[N/mm2]	fywd	=	435	[N/mm2]
yield strength suspension fyk,sup.	=	220	[N/mm2]	fctd	=	1,33	[N/mm2]
char. tensile strength fctk;0,05	=	2	[N/mm2]	fpd	=	1448	[N/mm2]
safety factor reinf. γs	=	1,15	[-]	Es	=	210000	[N/mm2]
safety factor concrete γc	=	1,50	[-]	Ep	=	205000	[N/mm2]
safety factor prestress γp	=	1,10	[-]				

n.a.	=	808	mm
Iyy	=	1,06E+12	mm4
S	=	1,16E+09	mm3

Longitudinal reinforcement

	n [rebars]	$\emptyset_{long}$ [mm]	A_{long} [mm2]	s_{long} [mm]	A_{sl} [mm2/mm]
Longitudinal reinfor.	10	16	201	220	2011



Stirrups

	n [sections]	$\emptyset_{stp}$ [mm]	A_{stp} [mm2]	s_{flr} [mm]	A_{sw} [mm2/mm]
Outer stirrups	2	20	628	200	3,14
+		2	628		3,14

Shear capacity concrete

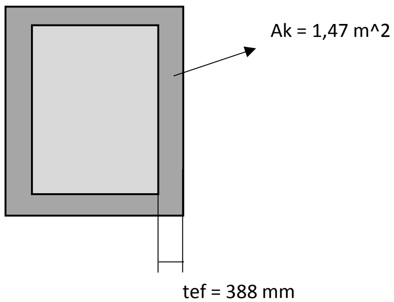
Coefficient Crd,c	=	0,12	
Coefficient k1	=	0,15	
Coefficient v1=v	=	0,52	[N/mm2]
Coefficient k	=	1,31	
$v_{min} = 0{,}035 * k^{\frac{3}{2}} * f_{ck}{}^{1/2}$	=	0,31	
$\sigma_{cp} = N_{Ed}/(bh)$	=	4,66	[N/mm2]
$\sigma_{cp,upper\ limit}$	=	4,66	[N/mm2]
$\propto_{cw}$	=	1,20	[-]
$\rho_l = A_{sl}/(b * d)$	=	0,0008	
$V_{Rd,c} = \left[C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}\right\} * b * d$	=	2361	[kN]
$V_{Rd,min} = [v_{min} + k_1 * \sigma_{cp}\} * b * d$	=	2587	[kN]
$V_{Rd,max} = \dfrac{\propto_{cw} * b_w * z * v_1 * f_{cd}}{\cot\theta + \tan\theta}$	=	16778	[kN]
$V_{Rd,c} = \dfrac{I * b_w}{S} * \sqrt{(f_{ctd}{}^2 + \propto_l * \sigma_{cp} * f_{ctd})}$	=	3102	[kN]

$$V_{Rd,s} = z * f_{ywd} * \cot\theta * \frac{A_{sw}}{s} = 2628 \text{ [kN]}$$

z	=	1923	[mm]	
θ_{stp}	=	90	[◦]	(angle of the stirrups)
$\theta_{diagonal}$	=	21,8	[◦]	(angle compr. diagonal)
$\theta_{assum.angle.}$	=	45	[◦]	(assumed angle comp. diagonal)

$21{,}8° \leq \theta \leq 45°$

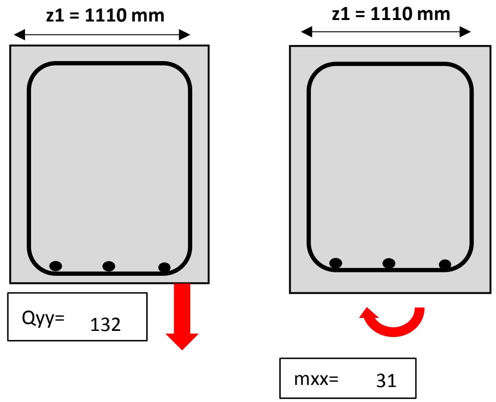
Shear capacity concrete			
Max. tensile bending stress	=	5,17 [N/mm2]	*(Concrete capacity is limited by flexural shear failure)*
$V_{Rd,c}$	=	2587 [kN]	*(Concrete shear capacity)*
$V_{Rd,max}$	=	16778 [kN]	*(Capacity compressive diagonal)*



Torsion capacity concrete			
$Area\ (A)$	=	2,64 [m2]	
$Perimeter\ (u)$	=	6,8 [m]	
$t_{ef} = A/u$	=	388 [mm]	
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126 [mm]	
A_k	=	1,47 [m2]	
u_k	=	5,25 [m]	
$T_{Rd,c} = t_{ef} * f_{ctd} * 2 * A_k$	=	1519 [kN]	*(Concrete torsion capacity)*
$T_{Rd,max} = 2 * v * \propto_{cw} * f_{cd} * A_k * t_{ef} * \sin\theta * \cos\theta$	=	8300 [kN]	*(Capacity compressive diagonal)*

Suspension force		
$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110 [mm]
$V_{Ed,sup}$	=	132 [kN/m]
$V_{Ed,sup,tot} = V_{Ed,sup} * z * \cot\theta$	=	253 [kN]
$V_{Ed,tot} = V_{Ed} + V_{Ed,sup,tot} + V_{Ed,clamp,tot}$	=	307 [kN]

Clamping moment			
$V_{Ed,clamp.} = \frac{m_{xx}}{z_1}$	=	28	[kN/m]
$V_{Ed,clamp,tot} = V_{Ed,clamp} * z * \cot\theta$	=	54	[kN]



Total capacity concrete					
$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed,tot}}{V_{Rd,max}}$	=	0,04	<	1,00	*(Capacity of compressive diagonal is sufficient)*
$\frac{T_{Ed}}{T_{Rd,c}} + \frac{V_{Ed}}{V_{Rd,c}}$	=	0,20	<	1,00	*(Minimum amount of reinforcement is allowed)*

Stirrups					
Stirrups due to Shear Force	=	$A_V = \frac{V_{Ed}}{z * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to Torsion	=	$A_T = \frac{T_{Ed}}{2 * A_k * f_{ywd} * \cot\theta}$	=	Min. Reinf.	[mm2/mm]

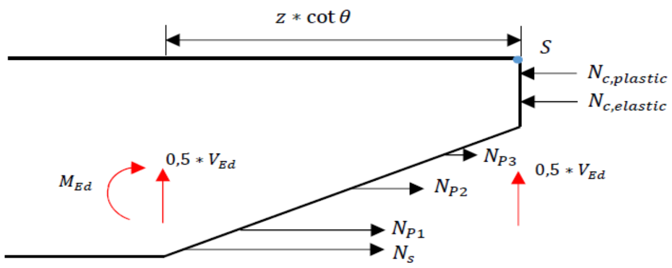
Minimum reinforcement		
$\rho_{w,min} = \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}$	=	0,000947 [-]
$A_{s,min}$	=	1,14 [mm2/mm]

Suspension reinforcement

Stirrups due to susp. force	=	$A_Q = \frac{V_{Ed,sup}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]
Stirrups due to clamp. mom.	=	$A_m = \frac{V_{Ed,clamp}}{1000 * f_{yk,sup}}$	=	Min. Reinf.	[mm2/mm]

Equilibrium of forces

ε_c	=	1,95E-03	> 1,75E-03	[elastic-plastic stage]
x	=	1440,9	[mm]	
$M_{Ed} - M_{P_\infty}$	=	32413	[kNm]	
$V_{Ed} = V_{Ed,tot} + \frac{T_{Ed} * z}{2 * A_k}$	=	440	[kN]	
$\sigma_{p\infty}$	=	967	[N/mm2]	
$\zeta_1 = \sqrt{\zeta * \frac{\emptyset_{long}}{\emptyset_{prestress}}}$	=	0,310	[-]	
$N_s = A_s * E_s * \varepsilon_s$	=	398	[kN]	
$N_{p1} = 2 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$	=	5510	[kN]	
e_{p1}	=	76	[mm]	



Longitudinal Reinforcement

$$U.C. = \frac{N_s}{A_s * fywd}$$ = 0,45 [-]

Prestressing steel

$$U.C. = \frac{N_{p1}}{2 * A_p * fpd}$$ = 0,70 [-]

Appendix E – Concrete stresses (SLS)

1 Introduction

A newly constructed railway bridge should fulfil the requirements set by the OVS. At serviceability limit state, the maximum tensile bending stresses in the prestressed girders are checked for a characteristic, frequent and quasi-permanent load combination. The OVS defines for each load combination the maximum allowable stress:

- Quasi-permanent: No tension is allowed in the entire cross-section.

- Frequent: No tensile stresses are allowed in the tendon zone and for the non-tendon zone the stresses should be limited to: $\sigma_b < 0{,}5 * f_{ctk;0{,}05} = 1{,}0\ N/mm^2$.

- Characteristic: In the tendon zone the tensile stresses are limited to $\sigma_b < 0{,}5 * f_{ctk;0{,}05} = 1{,}0\ N/mm^2$ and in the non-tendon zone the tensile stresses should remain smaller than: $\sigma_b < 0{,}75 * f_{ctk;0{,}05} = 1{,}5\ N/mm^2$

The tensile bending stresses in bridge A and B are determined at midspan (0,5L) because here the largest values are expected. Once the requirements of the OVS are met, one can assume the girder remains uncracked and that the requirements regarding crack width are met. However it could be possible, due to the heavier load models from the Eurocode, that the requirements are not met. In that case the girder must be assumed as cracked and the concrete tensile strength goes to zero. The cracked girder approach from appendix D then needs to be used to verify whether the crack widths remain within the acceptable limits.

2 Concrete stresses

Eurocode 0 (14) accounts for the characteristic, frequent and quasi-permanent load combination at serviceability limit state (Table E-1). The self-weight and ballast are the unfavourable permanent loads, where LM71 forms the leading variable load. In SLS all partial load factors can be taken equal to 1,0.

Table E-1: Load combinations at serviceability limit state

Load Combination	Permanent loading G_d		Variable load Q_d	
	Unfavourable	Favourable	Leading	Others
Characteristic	$G_{k,j,sup}$	$G_{k,j,inf}$	$Q_{k,1}$	$\psi_{0,i}Q_{k,i}$
Frequent	$G_{k,j,sup}$	$G_{k,j,inf}$	$\psi_{1,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$
Quasi-permanent	$G_{k,j,sup}$	$G_{k,j,inf}$	$\psi_{2,1}Q_{k,1}$	$\psi_{2,i}Q_{k,i}$

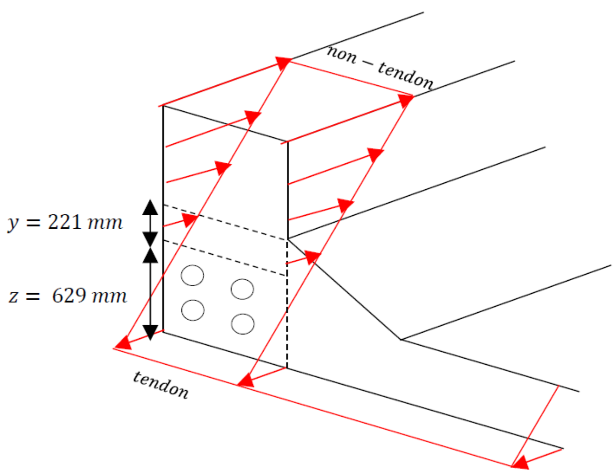
Table A2.3 in Eurocode 0 defines the factors for simultaneous load action. A summary of the relevant factors is presented in Table E-2.

Table E-2: Simultaneity factors for a single track bridge

Simultaneity factor	Self-weight	Prestress	Settlement support	LM71
$\psi_{0,1}$	1,00	1,00	1,00	0,80
$\psi_{1,1}$	1,00	1,00	1,00	0,80
$\psi_{2,1}$	1,00	1,00	1,00	0,00

Let's consider a cross-section of bridge A at 0,5L. The characteristic load combination could be expressed as:

$$\gamma_G * G_k + \gamma_Q * Q_k + \gamma_P * P$$



Parameters
$M_G = 4978\ kNm$
$M_Q = 4381\ kNm$
$M_P = -4810\ kNm$
$P_\infty = -12038\ kN$
$A_c = 2,75\ m^2$
$I_{yy} = 0,663\ m^4$
$z = 629\ mm$
$h = 1750\ mm$

Figure E- 1: Bending stresses in bridge A in the tendon and non-tendon zone

A maximum tensile stress is found in the non-tendon zone by only taking self-weight and prestress into account. Contrastingly the maximum tensile stresses in the tendon zone is formed by a combination of self-weight, prestress and LM71:

$$\sigma_{b,non-tendon} = -\frac{P}{A_c} - \frac{(M_G + M_P) * (h - z)}{I_{yy}} = -4{,}66\ N/mm^2 < 1{,}50\ N/mm^2$$

$$\sigma_{b,tendon} = -\frac{P}{A_c} + \frac{(M_G + M_P + M_Q) * z}{I_{yy}} = -0{,}06\ N/mm^2 < 1{,}00\ N/mm^2$$

In a similar way the characteristic, frequent and quasi-permanent load combinations are analysed for bridge A and B. The final results are presented in Table E-3 and Table E-4 for bridge A and B respectively. It can be concluded that girder A meets the requirements of the OVS and therefore remains uncracked. Girder B however does not meet the requirements for the characteristic and frequent load combination. An additional calculation on girder B is performed to verify whether or not the crack widths stay within the acceptable limits.

Table E-3: Maximum tensile bending stress in bridge A (section 0,5L)

Load Combination	Bridge A		
	Tendon	Non-Tendon	U.C.
Characteristic	$-0{,}06 < 1{,}00\ N/mm^2$	$-4{,}66 < 1{,}50\ N/mm^2$	O.K.
Frequent	$-0{,}89 < 0{,}00\ N/mm^2$	$-4{,}66 < 1{,}00\ N/mm^2$	O.K.
Quasi-permanent	$-4{,}22 < 0{,}00\ N/mm^2$	$-4{,}66 < 0{,}00\ N/mm^2$	O.K.

Table E-4: Maximum tensile bending stress in Bridge B (section 0,5L)

Load Combination	Bridge B		
	Tendon	Non-Tendon	U.C.
Characteristic	$1{,}30 > 1{,}00\ N/mm^2$	$-8{,}48 < 1{,}50\ N/mm^2$	NOT O.K.
Frequent	$0{,}50 > 0{,}00\ N/mm^2$	$-8{,}48 < 1{,}00\ N/mm^2$	NOT O.K.
Quasi-permanent	$-2{,}68 < 0{,}00\ N/mm^2$	$-8{,}48 < 0{,}00\ N/mm^2$	O.K.

3 Cracked girder

With the girder being cracked, tensile forces in the cross-section need to be transferred by the applied longitudinal reinforcement and prestress steel. Similar to the approach in appendix D, the concrete compressive strain (ε_c) and compression zone height (x) are the two unknowns which are solved for equilibrium of horizontal forces and bending moment.

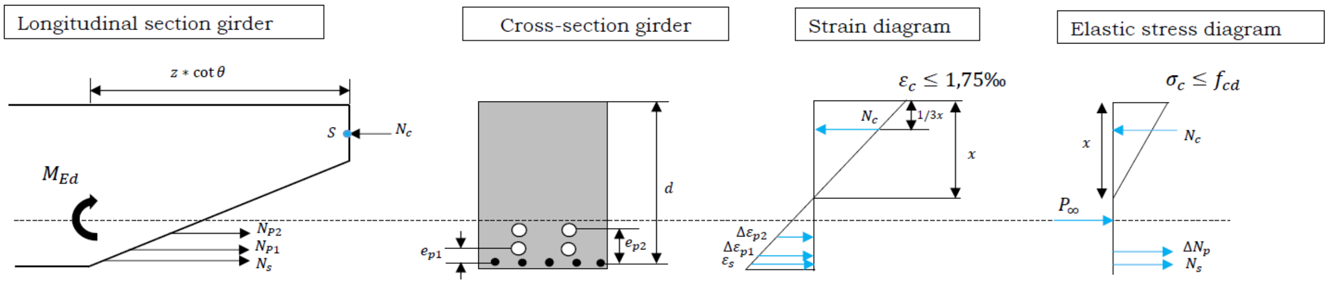


Figure E-2: Internal force distribution in a cracked girder at midspan (0,5L)

The strains are expressed as:

$$\varepsilon_s = \frac{\varepsilon_c}{x} * (d - x)$$

$$\Delta\varepsilon_{p1} = \frac{\varepsilon_c}{x} * (d - x - e_{p1})$$

$$\Delta\varepsilon_{p2} = \frac{\varepsilon_c}{x} * (d - x - e_{p2})$$

The forces are expressed as:

$$N_s = A_s * E_s * \varepsilon_s$$

$$N_{p1} = 4 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p1} * E_p * \xi_1)$$

$$N_{p2} = 4 * A_p * (\sigma_{p\infty} + \Delta\varepsilon_{p2} * E_p * \xi_1)$$

$$N_c = 0{,}5 * \frac{\varepsilon_c}{1{,}75 * 10^{-3}} * f_{cd} * b * x$$

Parameters
$A_p = 2700\ mm^2$
$A_{flex} = 10Ø16$
$E_s = 210\ GPa$
$E_p = 205\ GPa$
$\xi_1 = 0{,}31$
$P_\infty = 20886 kN$
$d = 2137\ mm$
$e_{p1} = 76\ mm$
$e_{p2} = 284\ mm$
$f_{pd} = 1448\ N/mm^2$
$f_{cd} = 23{,}3\ N/mm^2$

With a value of 24923 kNm the characteristic load combination (at midspan) results in the largest bending moment in girder B. Applying the equilibrium conditions the height of the compression zone and the compressive strain goes to 2252 mm and 1,15 ‰ respectively. It can be concluded that height of the compression is larger than the height of the girder. For that it can be safely said that the entire cross-section is under compression and the requirements regarding crack width are met.

4 Spreadsheets for concrete stresses

4.1 Stresses: Bridge A - characteristic loading – 0,5L

Load combination: Characteristic

		Prestress		Bending moment	σb [N/mm2]		U.C. [-]	
LC	type	P [kN]	σxx [N/mm2]	M [kNm]	Non-Tendon	Tendon	Non-Tendon	Tendon
1	self-weight			3755				
2	ballast			1223				
3	Conc. Mobile Load			3015				
5a	Contin. Mobile Load			1366				
6	Support settelement max			0				
7	Support settelement min			0				
8	Prestress t=0	-13200	-4,80	-5274				
9	Prestress t = ∞	-12038	-4,38	-4810				
LC 1 + LC 2 + LC 9		-12038	-4,38	169	-4,66		O.K.	
LC 1 + LC 2 + LC 3 + LC 5a + LC 6 + LC 9		-12038	-4,38	4551		-0,06		O.K.
LC 1 + LC 2 + LC 3 + LC 5a + LC 7 + LC 9		-12038	-4,38	4551		-0,06		O.K.

Parameters

Sectional properties

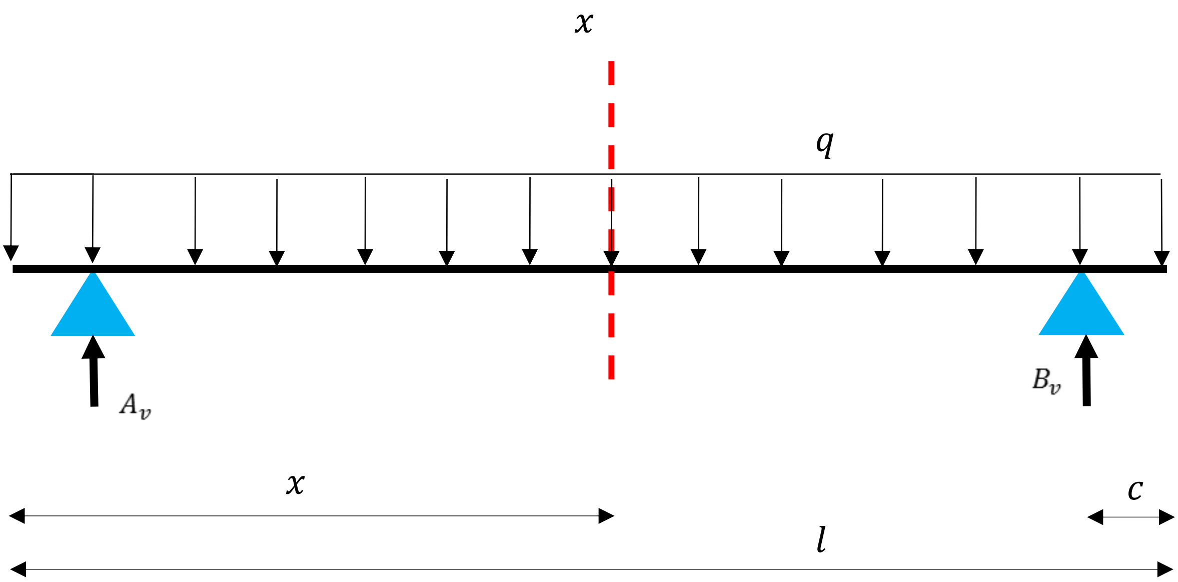
b_{girder}	900	[mm]
h_{girder}	1750	[mm]
$0{,}5 * A_{bridge}$	2750000	[mm2]
$0{,}5 * I_{yy}$	6,63E+11	[mm4]
y	629	[mm]
z_{bottom}	629	[mm]
z_{top}	1121	[mm]

Maximum bending stress

$f_{ctk;0,05} =$	2,00	[N/mm2]
$\sigma_{b,max,tendon} =$	1,00	[N/mm2]
$\sigma_{b,max,non-tendon} =$	1,50	[N/mm2]

LC 1

Loading (long. direction)



Load & Reaction forces		
qbridge	138	[kN/m]
0,5q	68,8	[kN/m]
Av	790,6	[kN]
Bv	790,6	[kN]
γG	1,00	[-] (NEN 1990 table A2.3)
ψ0	1,00	[-]

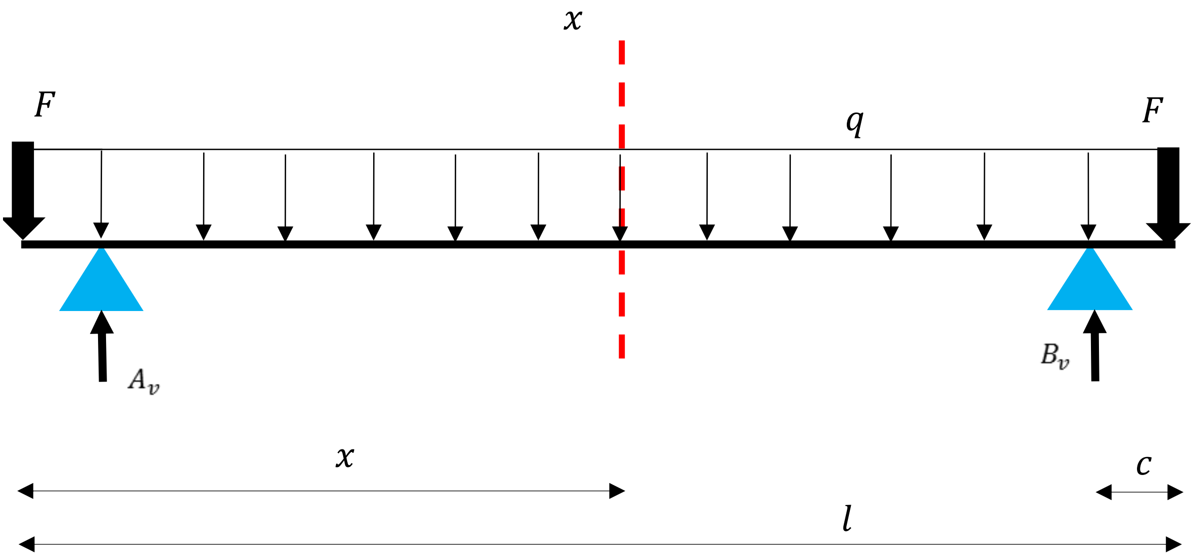
Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	3755,5	[kNm]

LC 2

Loading (long. direction)



Load & Reaction forces		
q	11,7	[kN/m2]
0,5q	23,4	[kN/m]
F	111	[kN]
0,5F	55	[kN]
Av	324,5	[kN]
Bv	324,5	[kN]
γG	1,00	[-]
ψ0	1,00	[-]

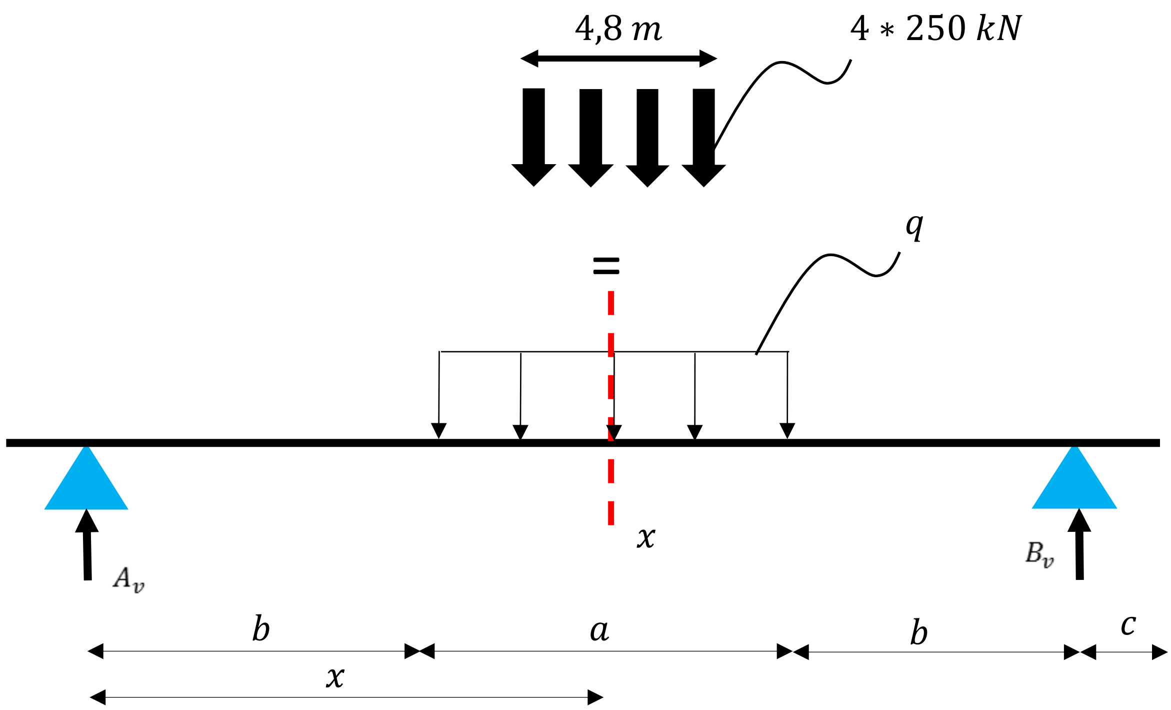
Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]
lctc	4,0	[m]

Bending moment

Mx	1222,8	[kNm]

LC 3

Loading (long. direction)



Load & Reaction forces		
q	199,2	[kN/m]
0,5q	99,6	[kN/m]
Av	343,6	[kN]
Bv	343,6	[kN]
γQ	1,00	[-]
α	1,21	[-]

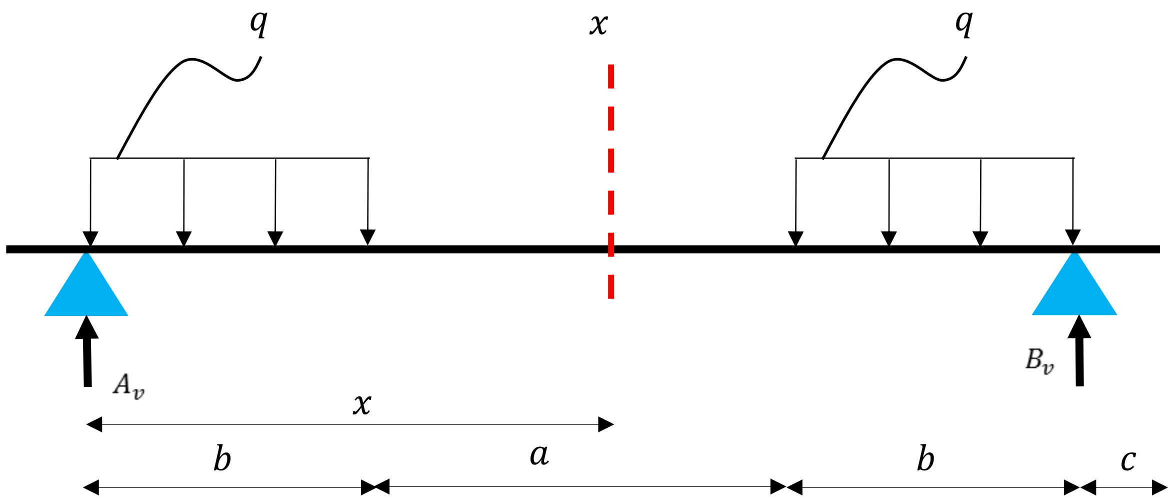
Measurements		
x	11,50	[m]
a	6,9	[m]
b	7,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	3015,4	[kNm]

LC 5a

Loading (long. direction)



Load & Reaction forces		
q	110	[kN/m]
0,5q	55	[kN/m]
Av	387,6	[kN]
Bv	387,6	[kN]
γQ	1,00	[-]
α	1,21	[-]

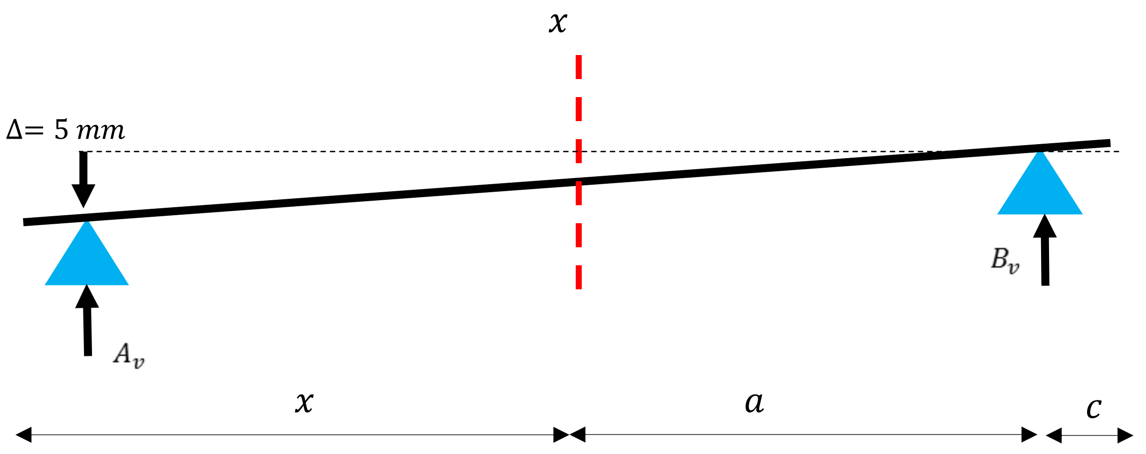
Measurements		
x	11,50	[m]
a	6,9	[m]
b	7,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	1366,4	[kNm]

LC 6

Loading (long. direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γG	1,00	[-]
ψ0	1,00	[-]

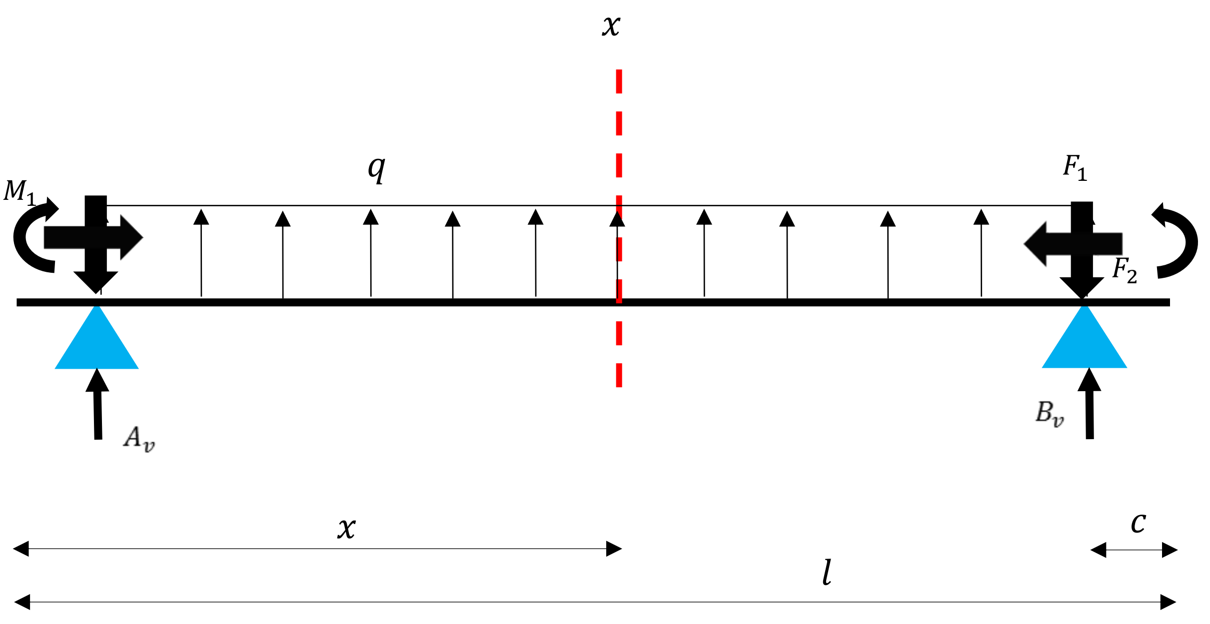
Measurements		
x	11,5	[m]
a	10,5	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	0,0	[kNm]

LC 8

Loading (long. direction)



Load & Reaction forces		
F1	1115	[kN]
F2	-13200	[kN]
M1	294	[kNm]
q	-101	[kN]
Av	55	[kN]
Bv	55	[kN]
yP	1,00	[-]
ψ0	1,00	[-]

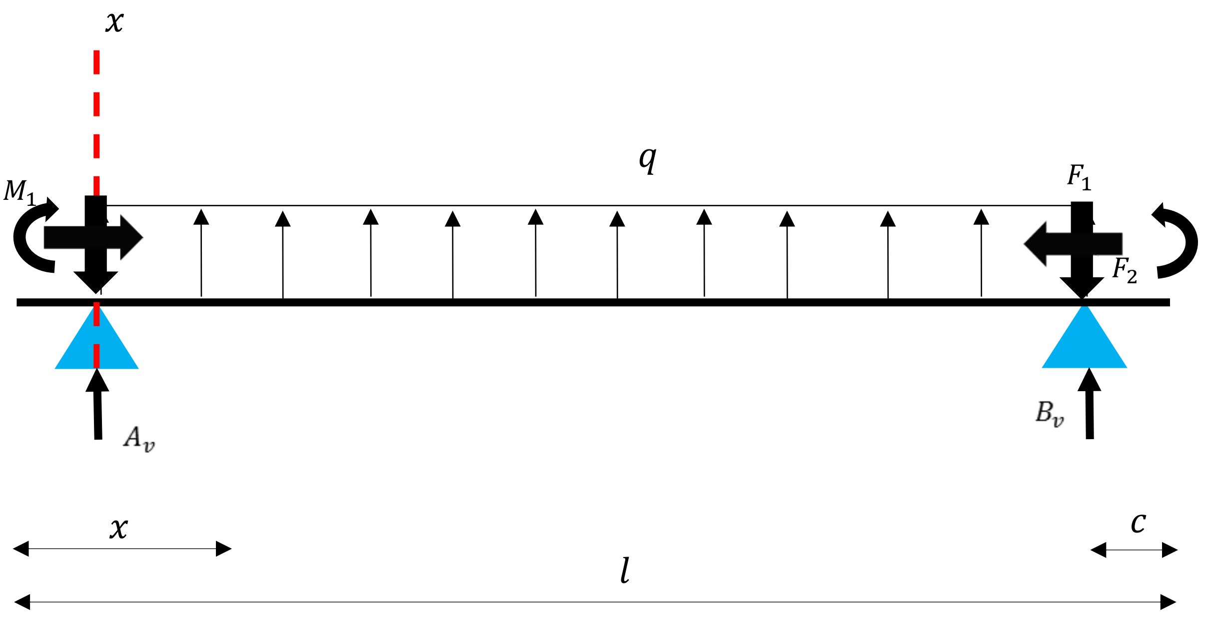
Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	-5273,6	[kNm]

LC 9

Loading (long. direction)



Load & Reaction forces		
F1	1016,88	[kN]
F2	-12038	[kN]
M1	268	[kNm]
q	-92	[kN]
Av	50	[kN]
Bv	50	[kN]
yP	1,00	[-]
P∞/P0	0,912	[-]
ψ0	1,00	[-]

Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	-4809,5	[kNm]

4.2 Stresses: Bridge A - frequent loading – 0,5L

Load combination: Frequent

LC	type	Prestress P [kN]	Prestress σxx [N/mm2]	Bending moment M [kNm]	σb [N/mm2] Non-Tendon	σb [N/mm2] Tendon	U.C. [-] Non-Tendon	U.C. [-] Tendon
1	self-weight			3755				
2	ballast			1223				
3	Conc. Mobile Load			2412				
5a	Contin. Mobile Load			1093				
6	Support settelement max			0				
7	Support settelement min			0				
8	Prestress t=0	-13200	-4,80	-5274				
9	Prestress t = ∞	-12038	-4,38	-4810				
LC 1 + LC 2 + LC 9		-12038	-4,38	169	-4,66		O.K.	
LC 1 + LC 2 + LC 3 + LC 5a + LC 6 + LC 9		-12038	-4,38	3674		-0,89		O.K.
LC 1 + LC 2 + LC 3 + LC 5a + LC 7 + LC 9		-12038	-4,38	3674		-0,89		O.K.

Parameters

Sectional properties

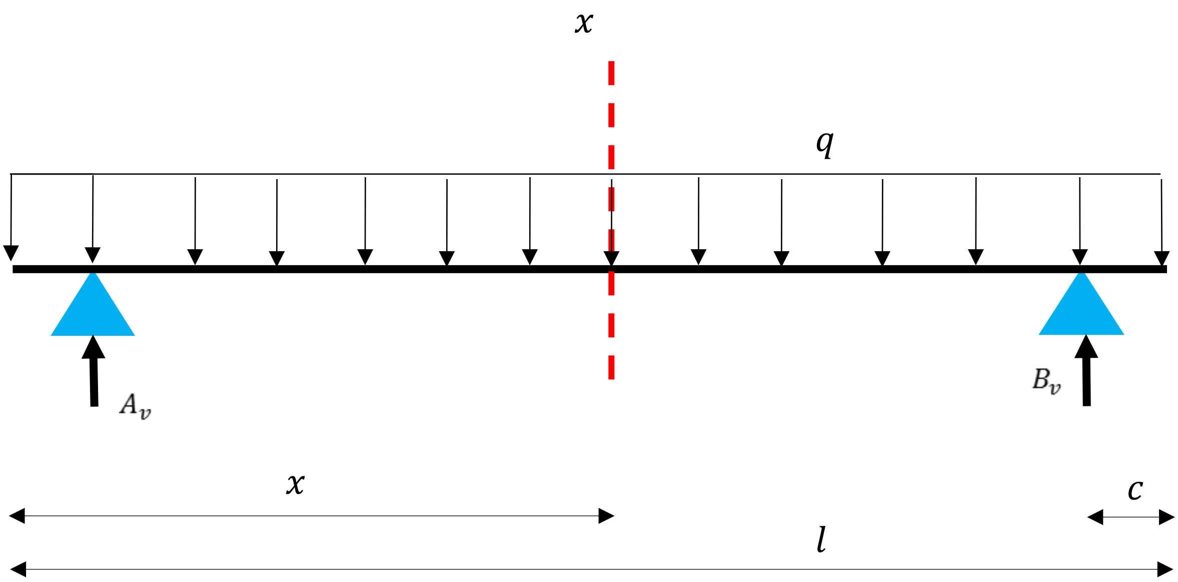
b_{girder}	900	[mm]
h_{girder}	1750	[mm]
$0{,}5 * A_{bridge}$	2750000	[mm2]
$0{,}5 * I_{yy}$	6,63E+11	[mm4]
y	629	[mm]
z_{bottom}	629	[mm]
z_{top}	1121	[mm]

Maximum bending stress

$f_{ctk;0,05} =$	2,00	[N/mm2]
$\sigma_{b,max,tendon} =$	0,00	[N/mm2]
$\sigma_{b,max,non-tendon} =$	1,00	[N/mm2]

LC 1

Loading (long. direction)



Load & Reaction forces		
qbridge	138	[kN/m]
0,5q	68,8	[kN/m]
Av	790,6	[kN]
Bv	790,6	[kN]
γG	1,00	[-] (NEN 1990 table A2.3)
ψ1	1,00	[-]

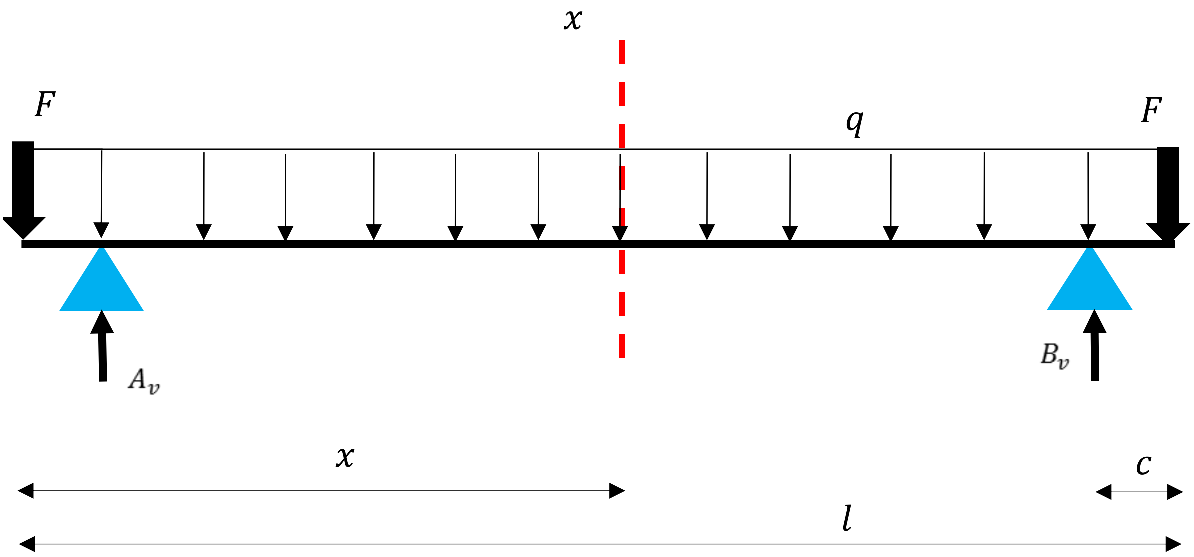
Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	3755,5	[kNm]

LC 2

Loading (long. direction)



Load & Reaction forces		
q	11,7	[kN/m2]
0,5q	23,4	[kN/m]
F	111	[kN]
0,5F	55	[kN]
Av	324,5	[kN]
Bv	324,5	[kN]
γG	1,00	[-]
ψ1	1,00	[-]

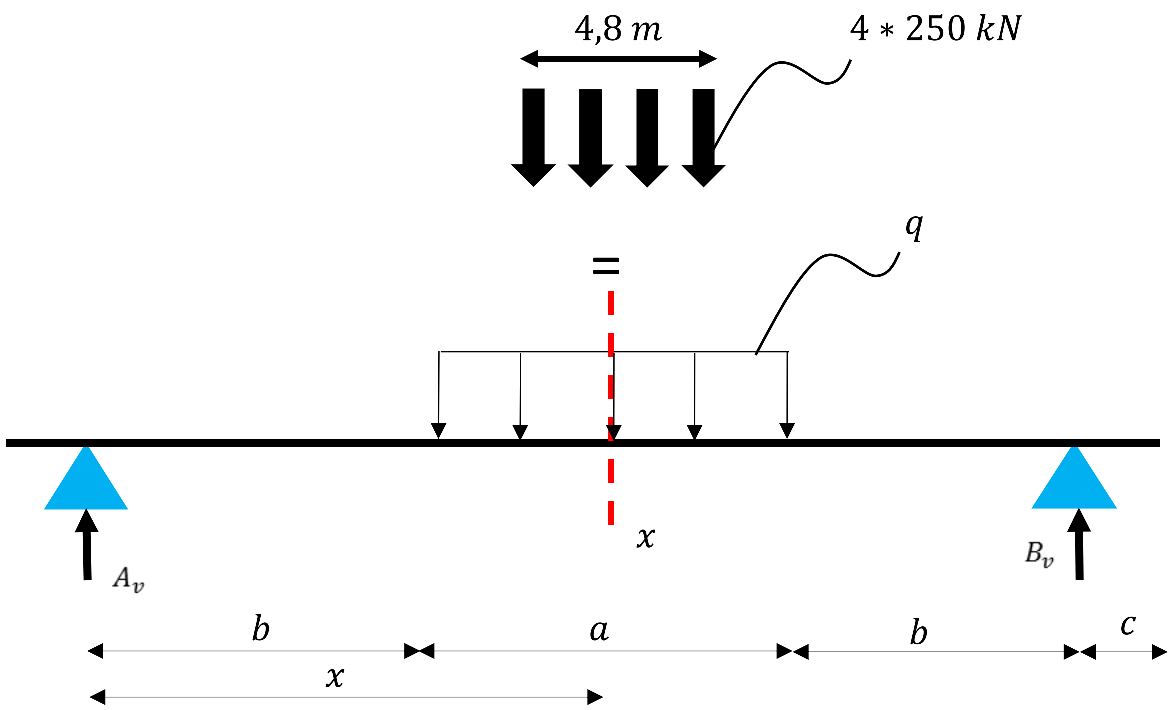
Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]
lctc	4,0	[m]

Bending moment

Mx	1222,8	[kNm]

LC 3

Loading (long. direction)



Load & Reaction forces		
q	159,4	[kN/m]
0,5q	79,7	[kN/m]
Av	274,9	[kN]
Bv	274,9	[kN]
γQ	1,00	[-]
ψ1	0,80	[-]
α	1,21	[-]

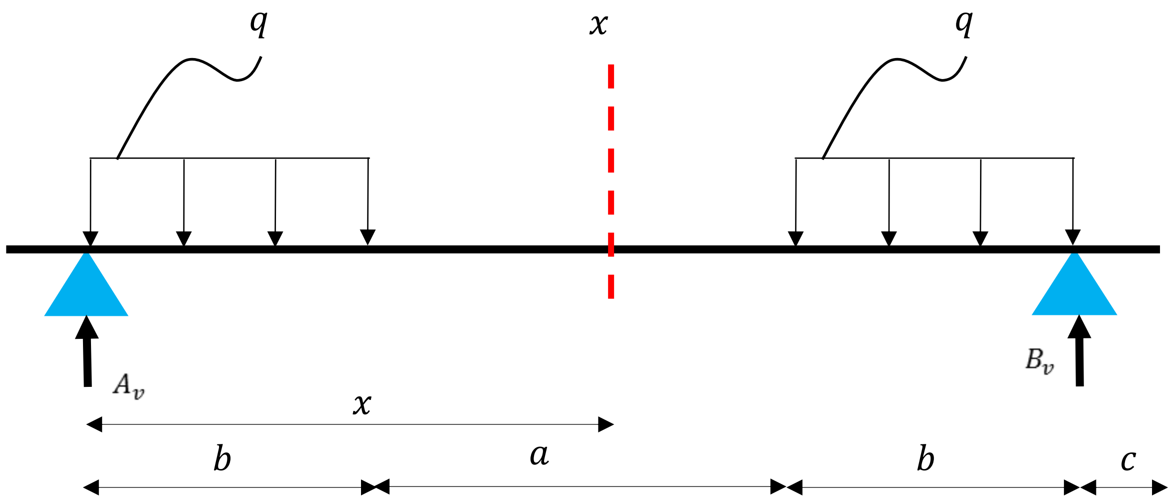
Measurements		
x	11,50	[m]
a	6,9	[m]
b	7,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	2412,4	[kNm]

LC 5a

Loading (long. direction)



Load & Reaction forces		
q	88	[kN/m]
0,5q	44	[kN/m]
Av	310,1	[kN]
Bv	310,1	[kN]
γQ	1,00	[-]
ψ1	0,80	[-]
α	1,21	[-]

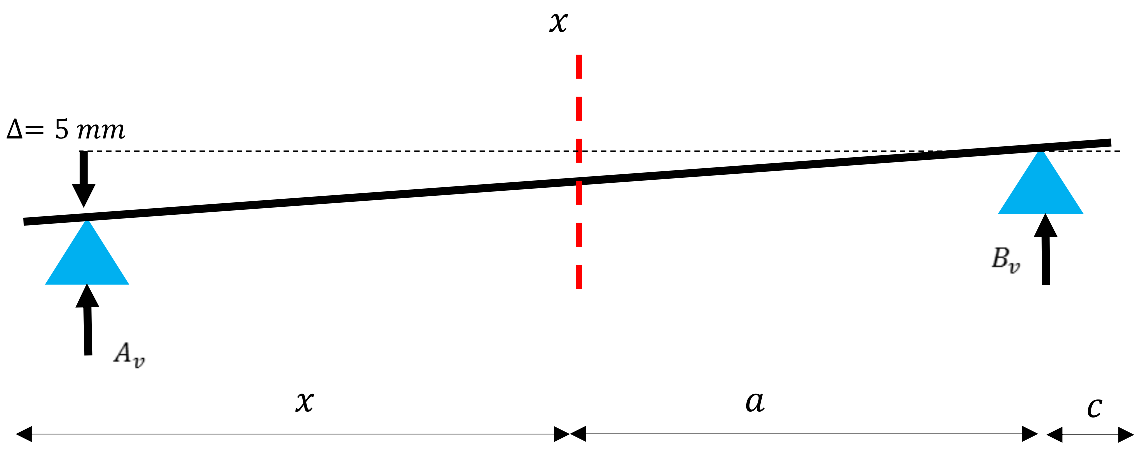
Measurements		
x	11,50	[m]
a	6,9	[m]
b	7,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	1093,1	[kNm]

LC 6

Loading (long. direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γG	1,00	[-]
ψ1	1,00	[-]

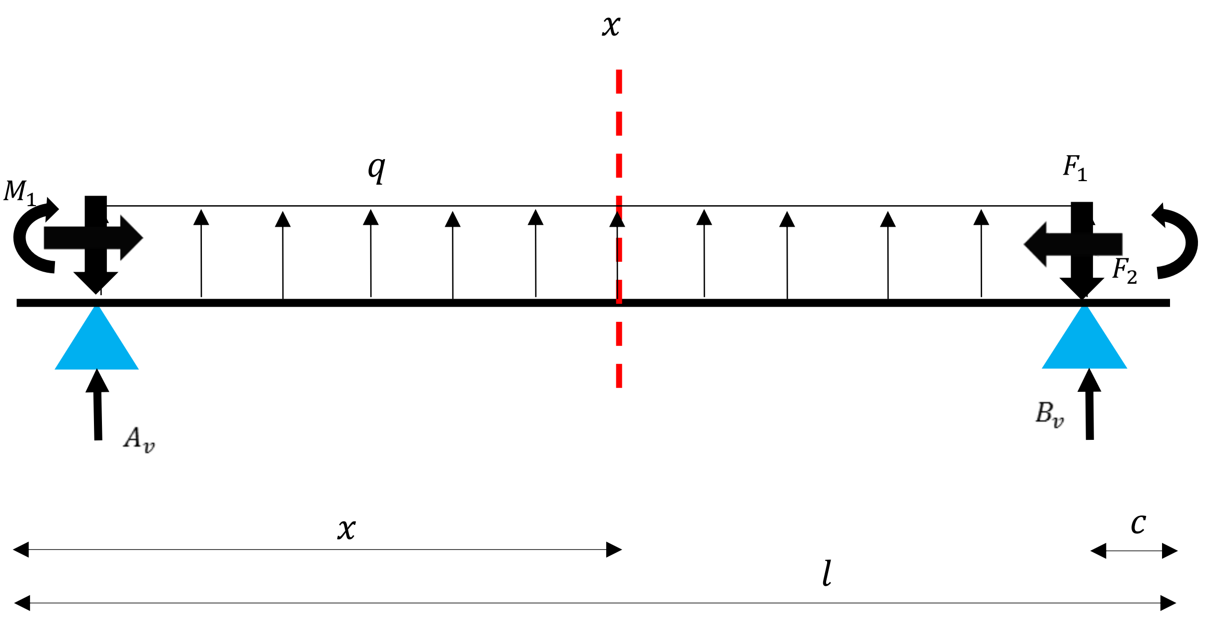
Measurements		
x	11,5	[m]
a	10,5	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	0,0	[kNm]

LC 8

Loading (long. direction)



Load & Reaction forces		
F1	1115	[kN]
F2	-13200	[kN]
M1	294	[kNm]
q	-101	[kN]
Av	55	[kN]
Bv	55	[kN]
γP	1,00	[-]
ψ1	1,00	[-]

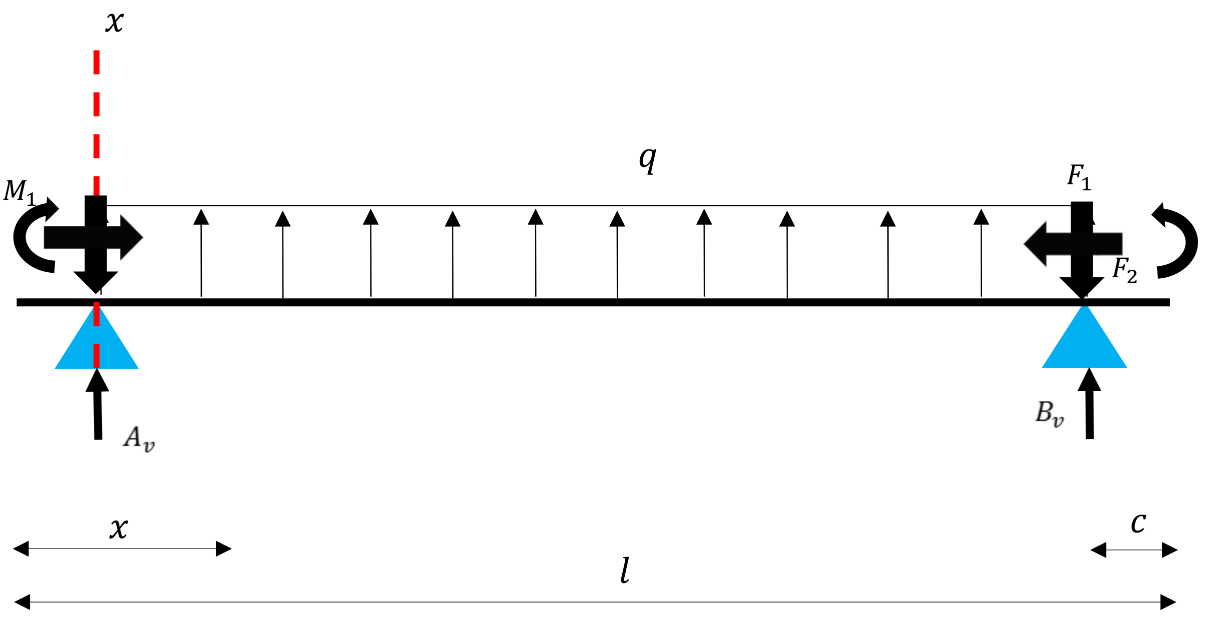
Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	-5273,6	[kNm]

LC 9

Loading (long. direction)



Load & Reaction forces		
F1	1016,88	[kN]
F2	-12038	[kN]
M1	268	[kNm]
q	-92	[kN]
Av	50	[kN]
Bv	50	[kN]
yP	1,00	[-]
P∞/P0	0,912	[-]
ψ1	1,00	[-]

Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx -4809,5 [kNm]

4.3 Stresses: Bridge A - quasi-permanent loading – 0,5L

Load combination: Quasi-permanent

		Prestress		Bending moment	σb [N/mm2]		U.C. [-]	
LC	type	P [kN]	σxx [N/mm2]	M [kNm]	Non-Tendon	Tendon	Non-Tendon	Tendon
1	self-weight			3755				
2	ballast			1223				
3	Conc. Mobile Load			0				
5a	Contin. Mobile Load			0				
6	Support settelement max			0				
7	Support settelement min			0				
8	Prestress t=0	-13200	-4,80	-5274				
9	Prestress t = ∞	-12038	-4,38	-4810				
LC 1 + LC 2 + LC 9		-12038	-4,38	169	-4,66		O.K.	
LC 1 + LC 2 + LC 3 + LC 5a + LC 6 + LC 9		-12038	-4,38	169		-4,22		O.K.
LC 1 + LC 2 + LC 3 + LC 5a + LC 7 + LC 9		-12038	-4,38	169		-4,22		O.K.

Parameters

Sectional properties

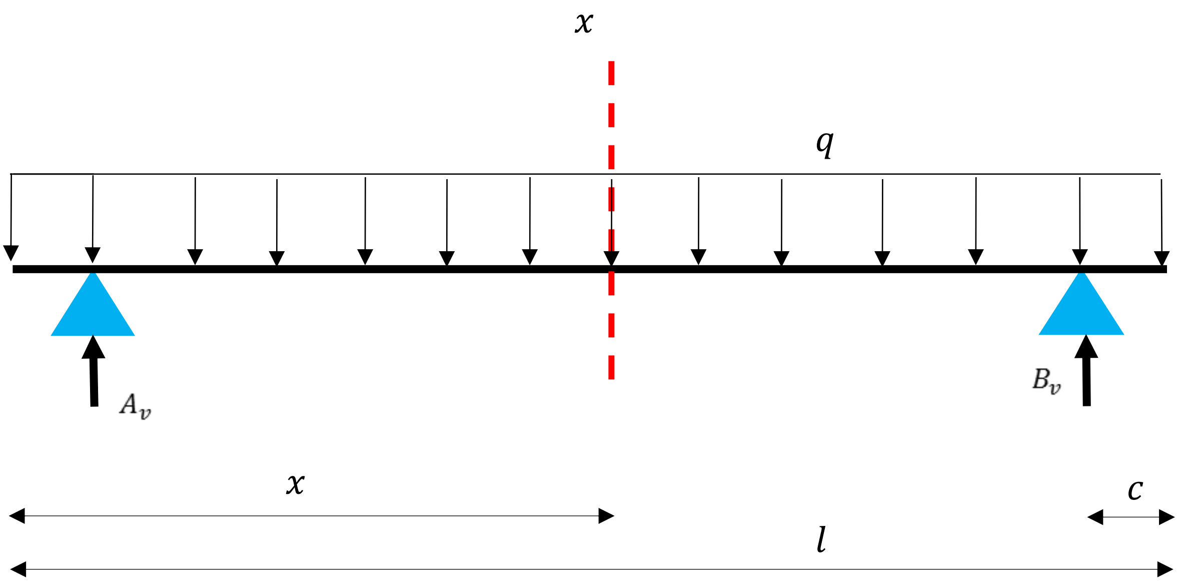
b_{girder}	900	[mm]
h_{girder}	1750	[mm]
$0{,}5 * A_{bridge}$	2750000	[mm2]
$0{,}5 * I_{yy}$	6,63E+11	[mm4]
y	629	[mm]
z_{bottom}	629	[mm]
z_{top}	1121	[mm]

Maximum bending stress

$f_{ctk;0,05}$	2,00	[N/mm2]
$\sigma_{b,max} = 0$	0,00	[N/mm2]

LC 1

Loading (long. direction)



Load & Reaction forces		
qbridge	138	[kN/m]
0,5q	68,8	[kN/m]
Av	790,6	[kN]
Bv	790,6	[kN]
γG	1,00	[-] (NEN 1990 table A2.3)
ψ2	1,00	[-]

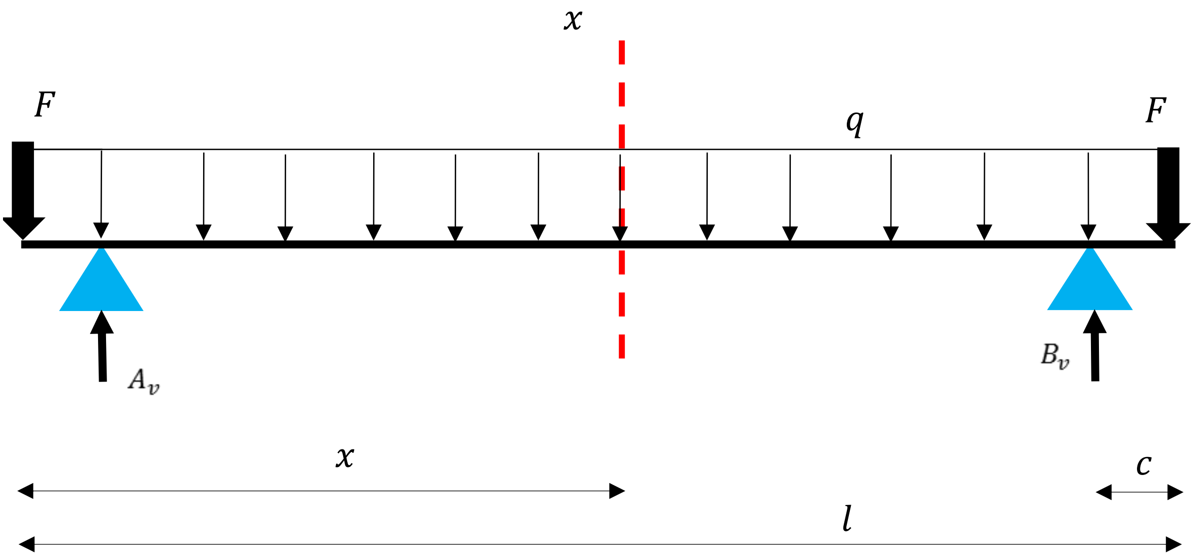
Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	3755,5	[kNm]

LC 2

Loading (long. direction)



Load & Reaction forces		
q	11,7	[kN/m2]
0,5q	23,4	[kN/m]
F	111	[kN]
0,5F	55	[kN]
Av	324,5	[kN]
Bv	324,5	[kN]
γG	1,00	[-]
ψ2	1,00	[-]

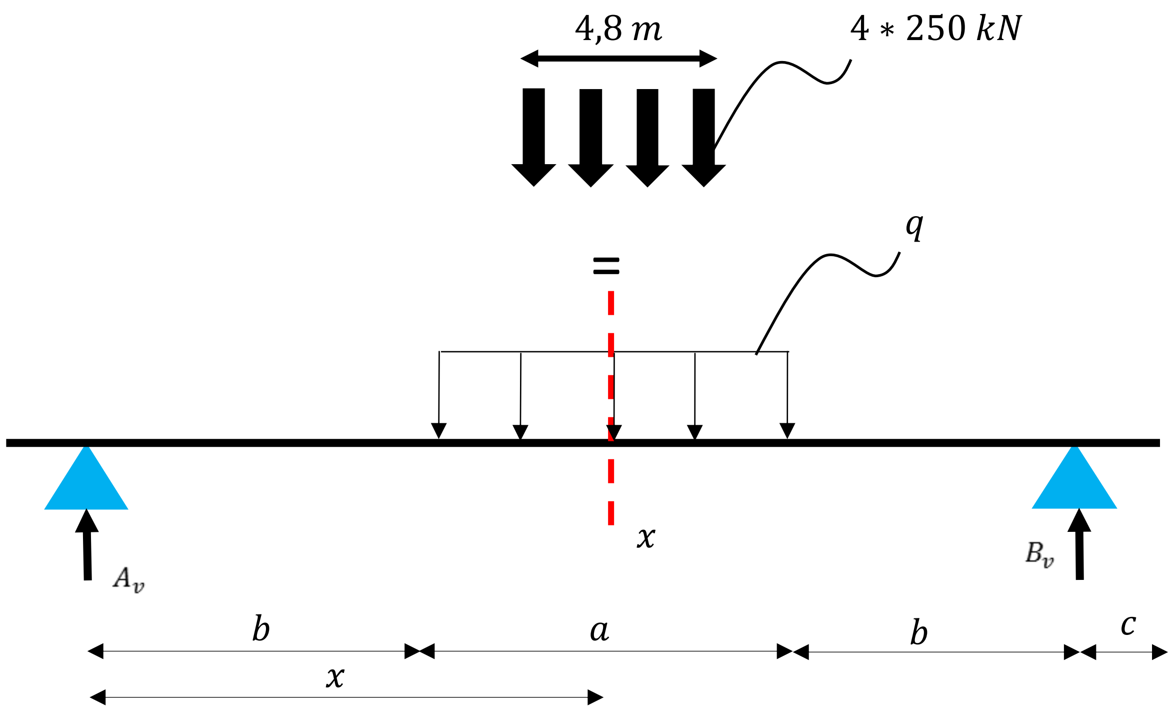
Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]
lctc	4,0	[m]

Bending moment

Mx	1222,8	[kNm]

LC 3

Loading (long. direction)



Load & Reaction forces		
q	0,0	[kN/m]
0,5q	0,0	[kN/m]
Av	0,0	[kN]
Bv	0,0	[kN]
γQ	1,00	[-]
ψ2	0,00	[-]
α	1,21	[-]

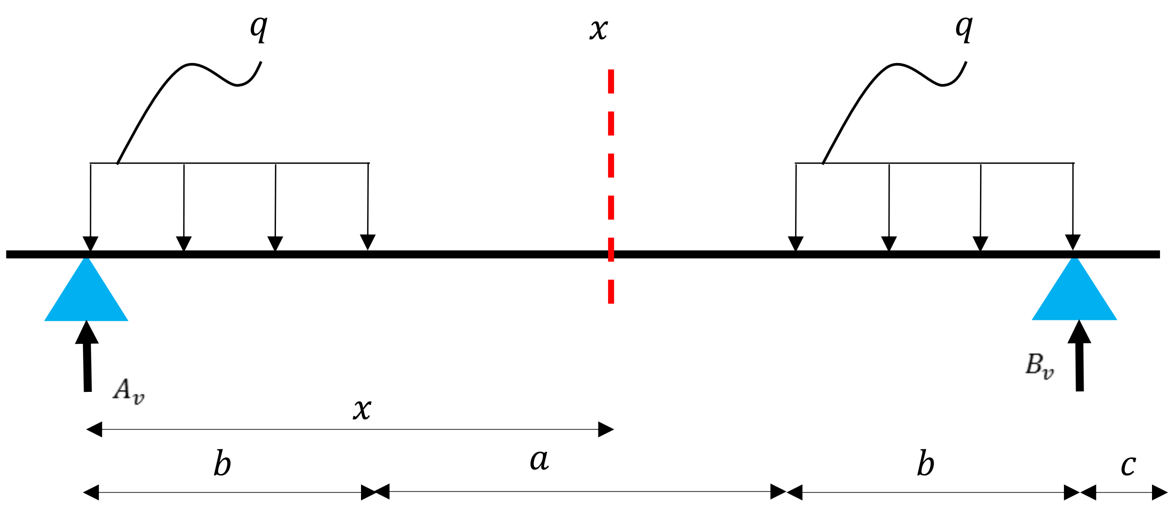
Measurements		
x	11,50	[m]
a	6,9	[m]
b	7,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	0,0	[kNm]

LC 5a

Loading (long. direction)



Load & Reaction forces		
q	0	[kN/m]
0,5q	0	[kN/m]
Av	0,0	[kN]
Bv	0,0	[kN]
γQ	1,00	[-]
ψ2	0,00	[-]
α	1,21	[-]

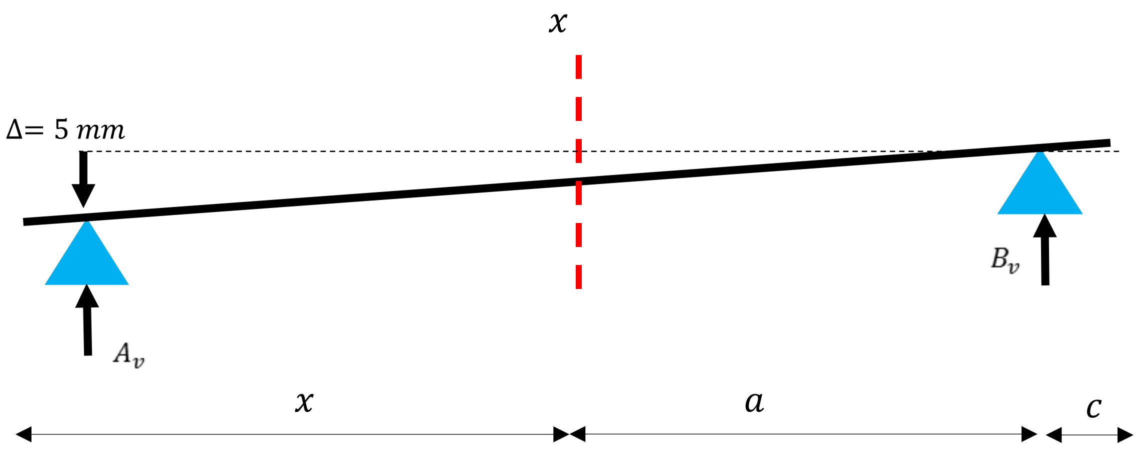
Measurements		
x	11,50	[m]
a	6,9	[m]
b	7,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	0,0	[kNm]

LC 6

Loading (long. direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γG	1,00	[-]
ψ2	1,00	[-]

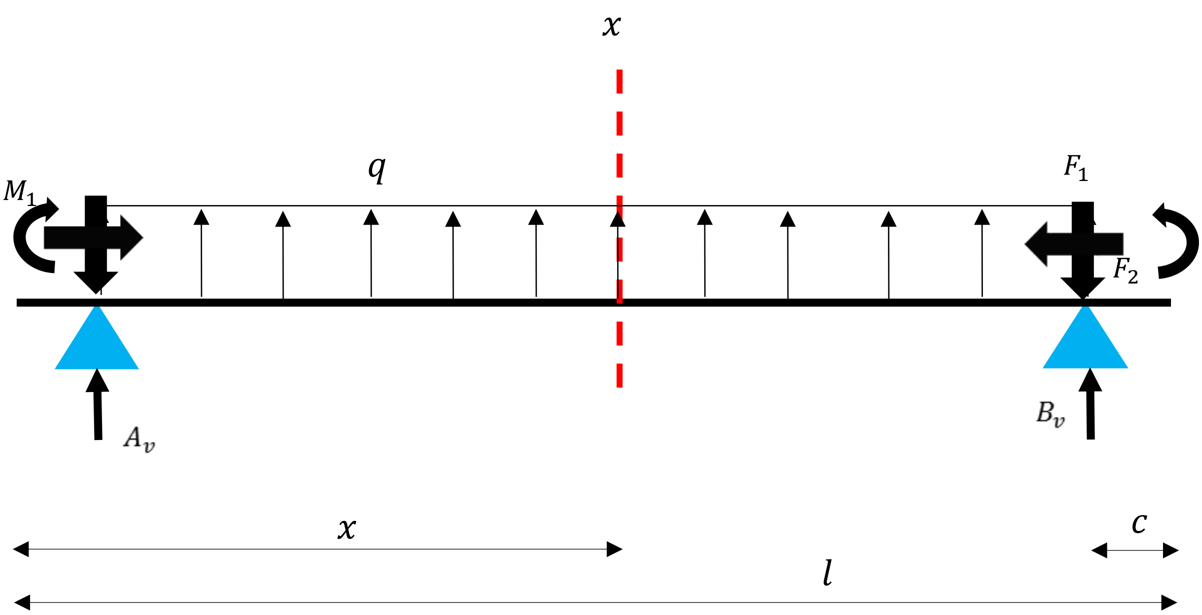
Measurements		
x	11,5	[m]
a	10,5	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	0,0	[kNm]

LC 8

Loading (long. direction)



Load & Reaction forces		
F1	1115	[kN]
F2	-13200	[kN]
M1	294	[kNm]
q	-101	[kN]
Av	55	[kN]
Bv	55	[kN]
yP	1,00	[-]
ψ2	1,00	[-]

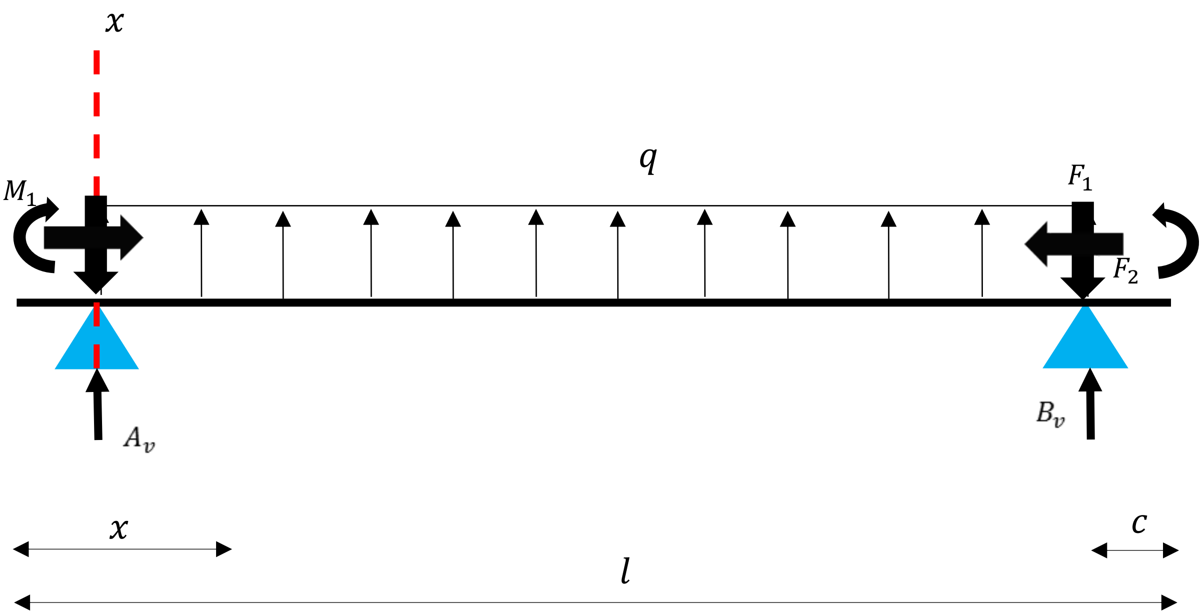
Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	-5273,6	[kNm]

LC 9

Loading (long. direction)



Load & Reaction forces		
F1	1016,88	[kN]
F2	-12038	[kN]
M1	268	[kNm]
q	-92	[kN]
Av	50	[kN]
Bv	50	[kN]
yP	1,00	[-]
P∞/P0	0,912	[-]
ψ2	1,00	[-]

Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Bending moment

Mx	-4809,5	[kNm]

4.4 Stresses: Bridge B - characteristic loading – 0,5L

Load combination: Characteristic

LC	type	Prestress P [kN]	Prestress σxx [N/mm2]	Bending moment M [kNm]	σb [N/mm2] Non-Tendon	σb [N/mm2] Tendon	U.C. [-] Non-Tendon	U.C. [-] Tendon
1	self-weight			13403				
2	ballast			3008				
3a	Mobile Max. (LM71)			8512				
5	Support settelement max			0				
6	Support settelement min			0				
7	Prestress t=0	-22826	-5,26	-12964				
8	Prestress t = ∞	-20886	-4,81	-11862				
LC 1 + LC 2 + LC 8		-20886	-4,81	4548	-8,48		O.K.	
LC 1 + LC 2 + LC 3a + LC 5 + LC 8		-20886	-4,81	13061		1,30		NOT O.K.
LC 1 + LC 2 + LC 3a + LC 6 + LC 8		-20886	-4,81	13061		1,30		NOT O.K.

Parameters

Sectional properties

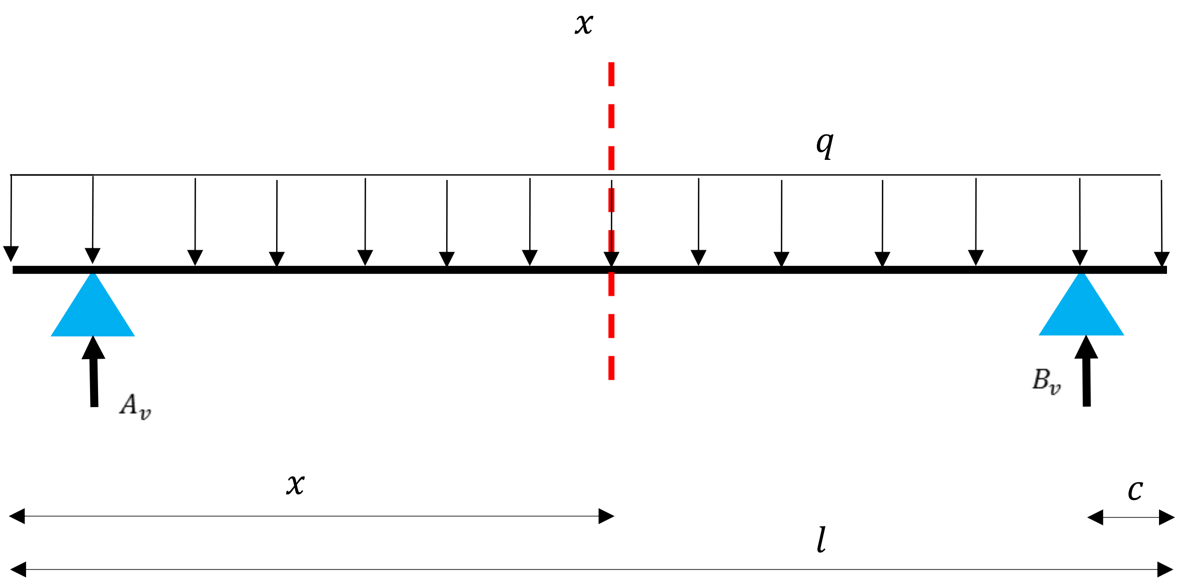
b_{girder}	1200	[mm]
h_{girder}	2200	[mm]
$0,5 * A_{bridge}$	4340000	[mm2]
$0,5 * I_{yy}$	1,73E+12	[mm4]
y	808	[mm]
z_{bottom}	808	[mm]
z_{top}	1392	[mm]

Maximum bending stress

$f_{ctk;0,05}$	2,00	[N/mm2]
$\sigma_{b,max,tendon} =$	1,00	[N/mm2]
$\sigma_{b,max,non-tendon} =$	1,50	[N/mm2]

LC 1

Loading (long. direction)



Load & Reaction forces		
qbridge	217	[kN/m]
0,5q	108,5	[kN/m]
Av	1817,4	[kN]
Bv	1817,4	[kN]
γG	1,00	[-] (NEN 1990 table A2.3)
ψ0	1,00	[-]

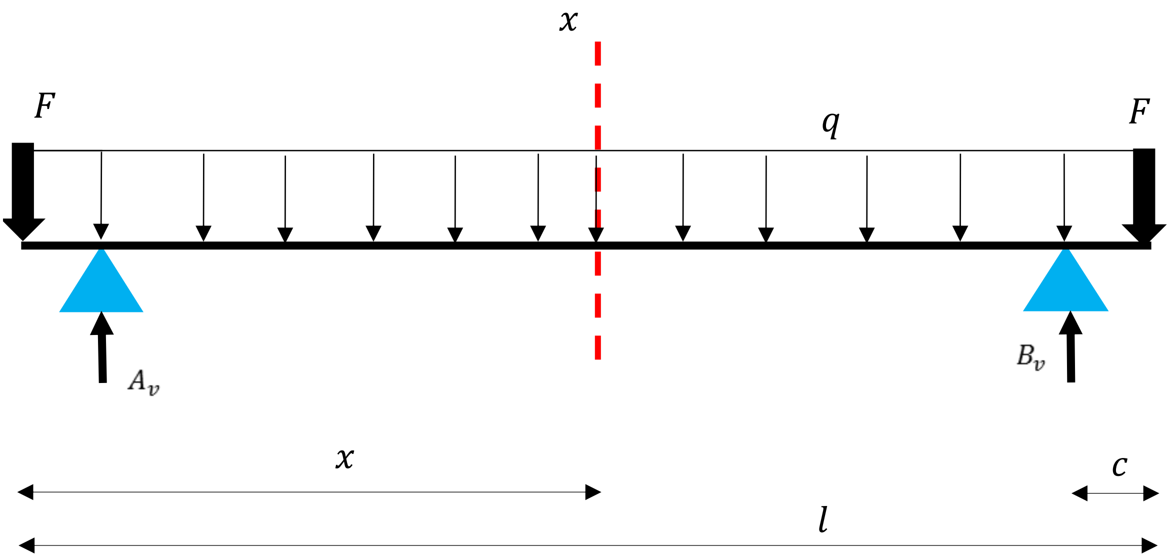
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	13403,1	[kNm]

LC 2

Loading (long. direction)



Load & Reaction forces		
q	9,9	[kN/m2]
0,5q	24,8	[kN/m]
F	99	[kN]
0,5F	50	[kN]
Av	464,2	[kN]
Bv	464,2	[kN]
γG	1,00	[-]
ψ0	1,00	[-]

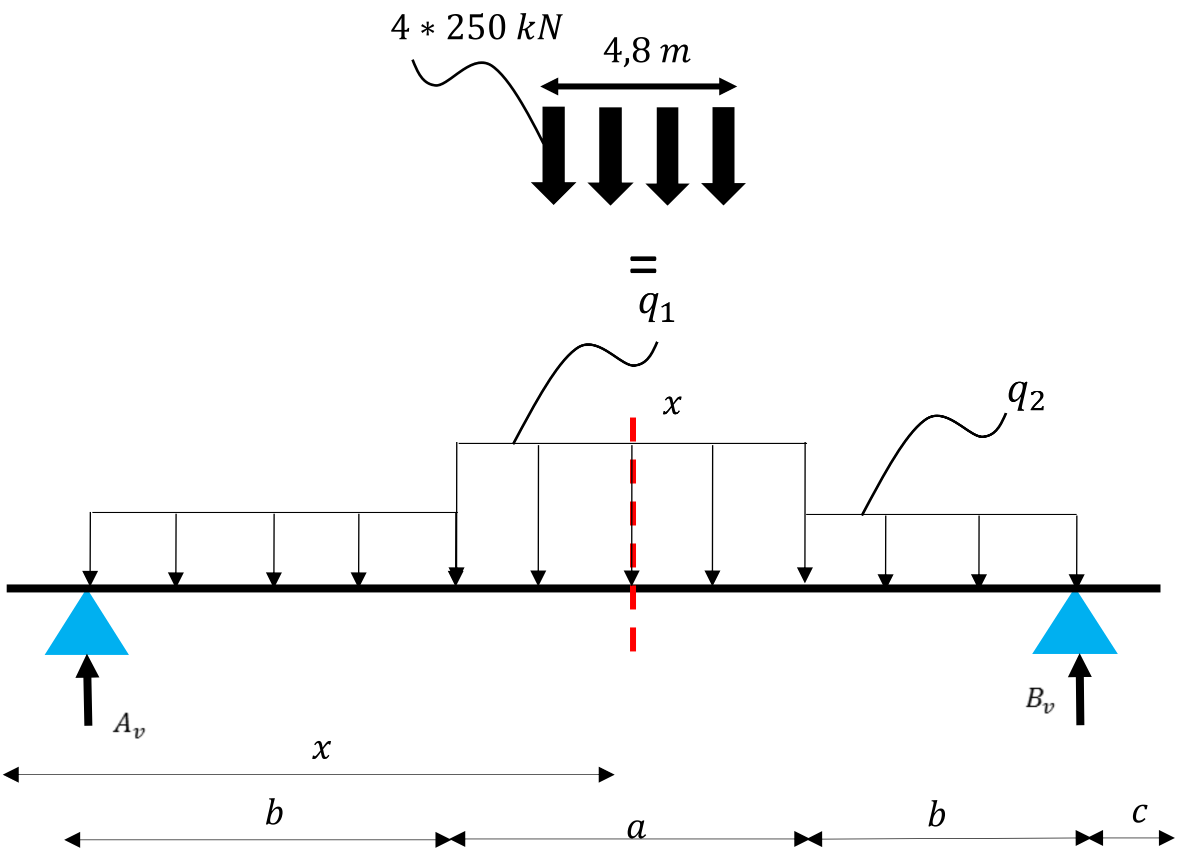
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]
lctc	5,0	[m]

Bending moment

Mx	3007,7	[kNm]

LC 3a

Loading (long. direction)



Load & Reaction forces		
q1	188,9	[kN/m]
q2	104,3	[kN/m]
0,5q1	94,4	[kN]
0,5q2	52,1	[kN]
Av	967,0	[kN]
Bv	967,0	[kN]
γQ	1,00	[-]
α	1,21	[-]

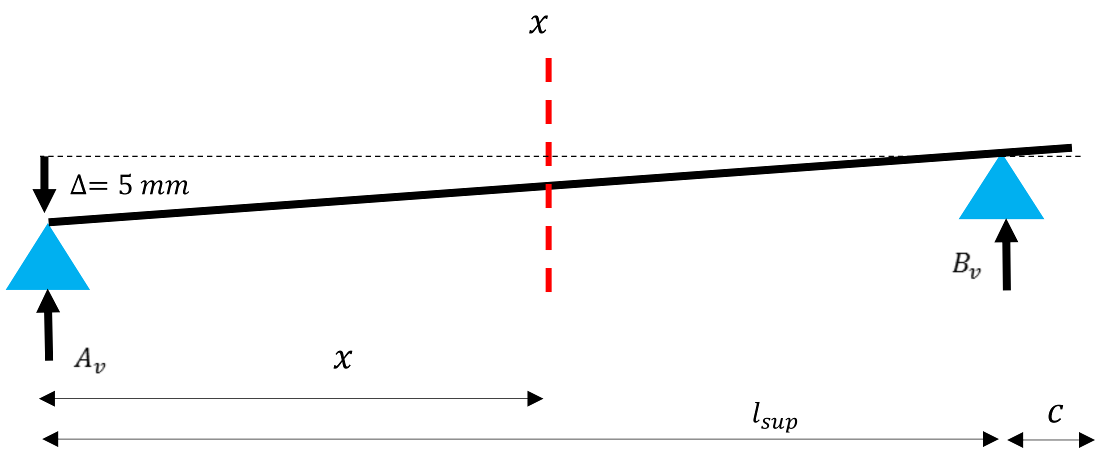
Measurements		
x	16,75	[m]
a	6,9	[m]
b	12,3	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	8512,4	[kNm]

LC 5

Loading (long. direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γG	1,00	[-]
ψ0	1,00	[-]

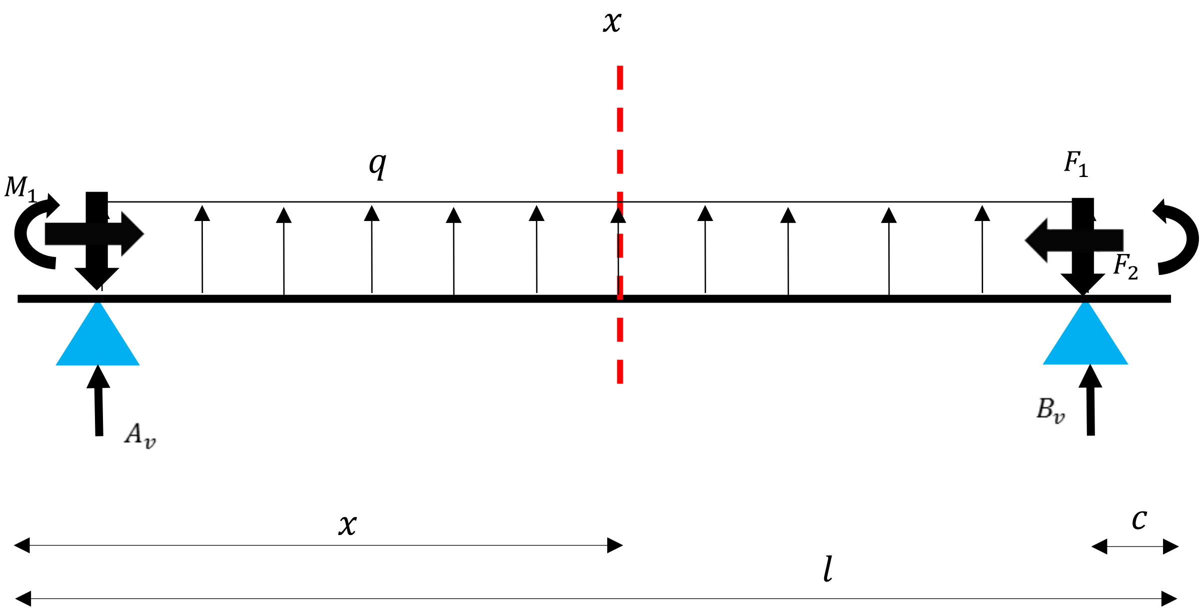
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	0,0	[kNm]

LC 7

Loading (long. direction)



Load & Reaction forces		
F1	2290	[kN]
F2	-22826	[kN]
M1	5045	[kNm]
q	-145,2	[kN]
Av	3	[kN]
Bv	3	[kN]
yP	1,00	[-]
ψ0	1,00	[-]

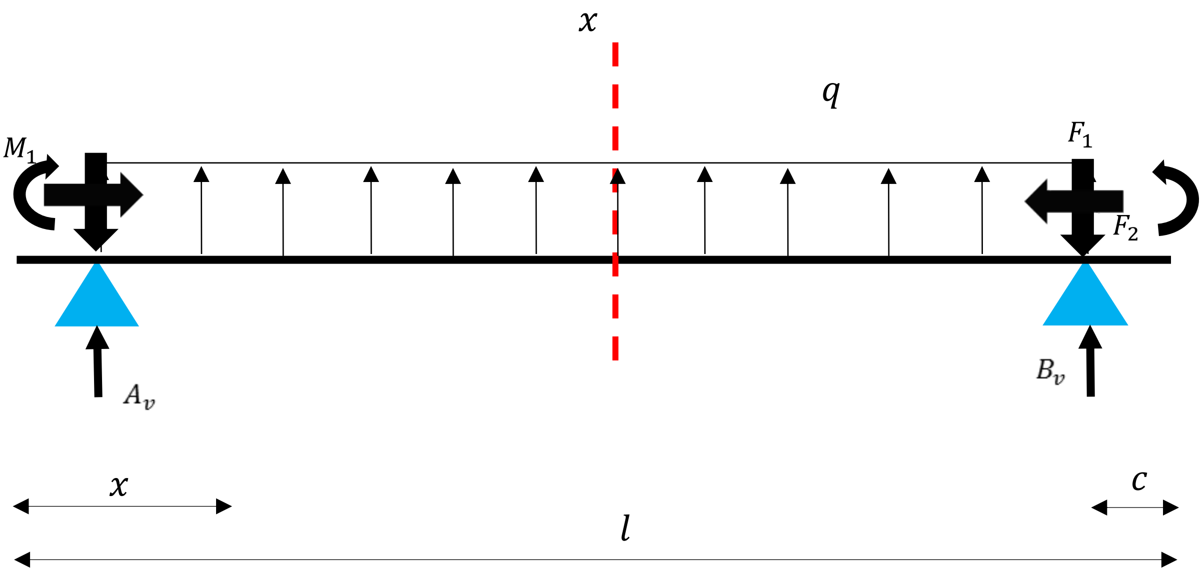
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	-12964,3	[kNm]

LC 8

Loading (long. direction)



Load & Reaction forces		
F1	2095	[kN]
F2	-20886	[kN]
M1	4616	[kNm]
q	-133	[kN]
Av	3	[kN]
Bv	3	[kN]
yP	1,00	[-]
P∞/P0	0,915	[-]
ψ0	1,00	[-]

Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	-11862,4	[kNm]

4.5 Stresses: Bridge B - frequent loading – 0,5L

Load combination: Frequent

LC	type	Prestress P [kN]	Prestress σxx [N/mm2]	Bending moment M [kNm]	σb [N/mm2] Non-Tendon	σb [N/mm2] Tendon	U.C. [-] Non-Tendon	U.C. [-] Tendon
1	self-weight			13403				
2	ballast			3008				
3a	Mobile Max. (LM71)			6810				
5	Support settelement max			0				
6	Support settelement min			0				
7	Prestress t=0	-22826	-5,26	-12964				
8	Prestress t = ∞	-20886	-4,81	-11862				
LC 1 + LC 2 + LC 8		-20886	-4,81	4548	-8,48		O.K.	
LC 1 + LC 2 + LC 3a + LC 5 + LC 8		-20886	-4,81	11358		0,50		NOT O.K.
LC 1 + LC 2 + LC 3a + LC 6 + LC 8		-20886	-4,81	11358		0,50		NOT O.K.

Parameters

Sectional properties

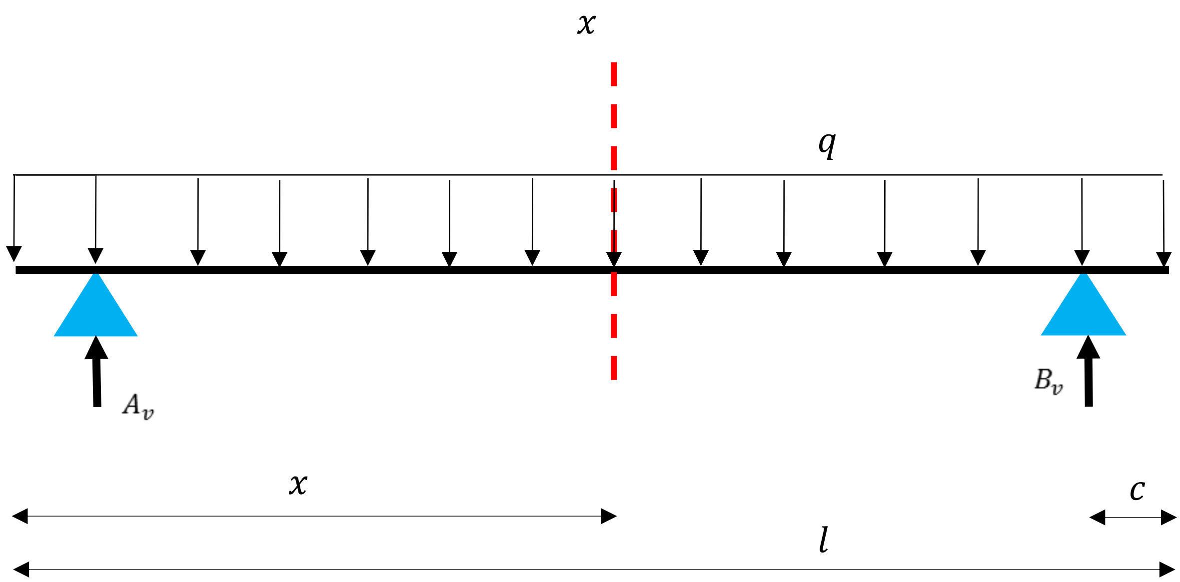
b_{girder}	1200	[mm]
h_{girder}	2200	[mm]
$0,5 * A_{bridge}$	4340000	[mm2]
$0,5 * I_{yy}$	1,73E+12	[mm4]
y	808	[mm]
z_{bottom}	808	[mm]
z_{top}	1392	[mm]

Maximum bending stress

$f_{ctk;0,05}$	2,00	[N/mm2]
$\sigma_{b,max,tendon} =$	0,00	[N/mm2]
$\sigma_{b,max,non-tendon} =$	1,00	[N/mm2]

LC 1

Loading (long. direction)



Load & Reaction forces		
qbridge	217	[kN/m]
0,5q	108,5	[kN/m]
Av	1817,4	[kN]
Bv	1817,4	[kN]
γG	1,00	[-] (NEN 1990 table A2.3)
ψ1	1,00	[-]

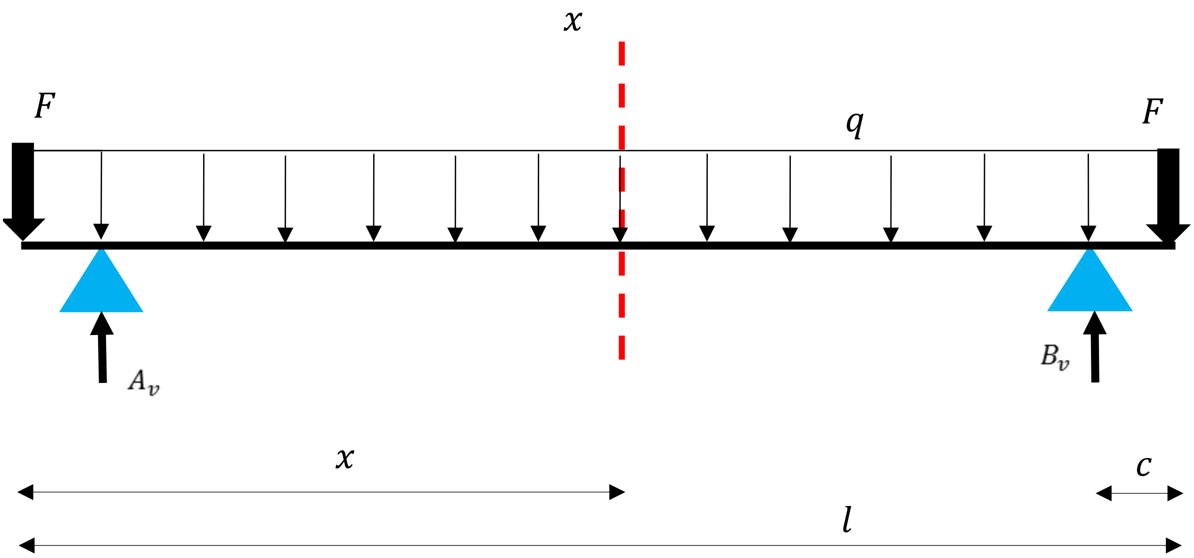
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	13403,1	[kNm]

LC 2

Loading (long. direction)



Load & Reaction forces		
q	9,9	[kN/m2]
0,5q	24,8	[kN/m]
F	99	[kN]
0,5F	50	[kN]
Av	464,2	[kN]
Bv	464,2	[kN]
γG	1,00	[-]
ψ1	1,00	[-]

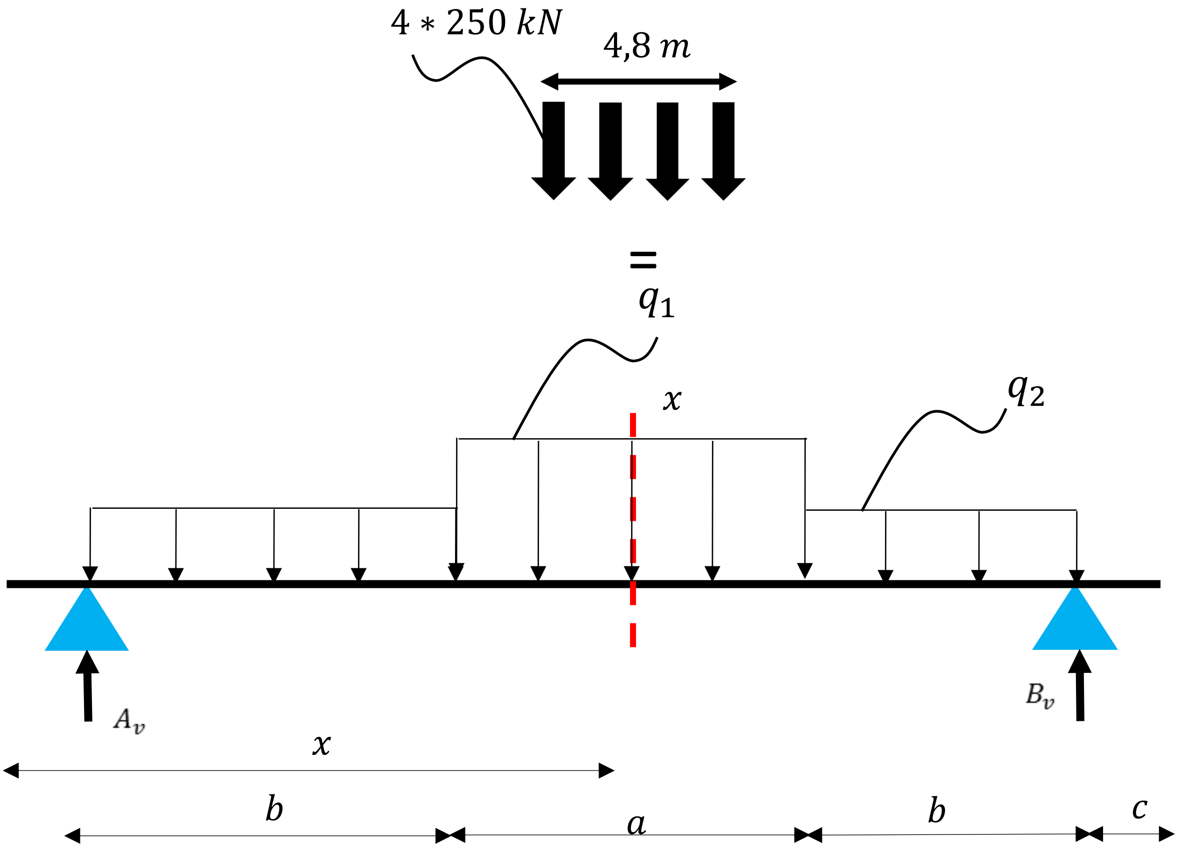
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]
lctc	5,0	[m]

Bending moment

Mx	3007,7	[kNm]

LC 3a

Loading (long. direction)



Load & Reaction forces		
q1	151,1	[kN/m]
q2	83,4	[kN/m]
0,5q1	75,5	[kN/m]
0,5q2	41,7	[kN/m]
Av	773,6	[kN]
Bv	773,6	[kN]
γQ	1,00	[-]
α	1,21	[-]
ψ1	0,80	[-]

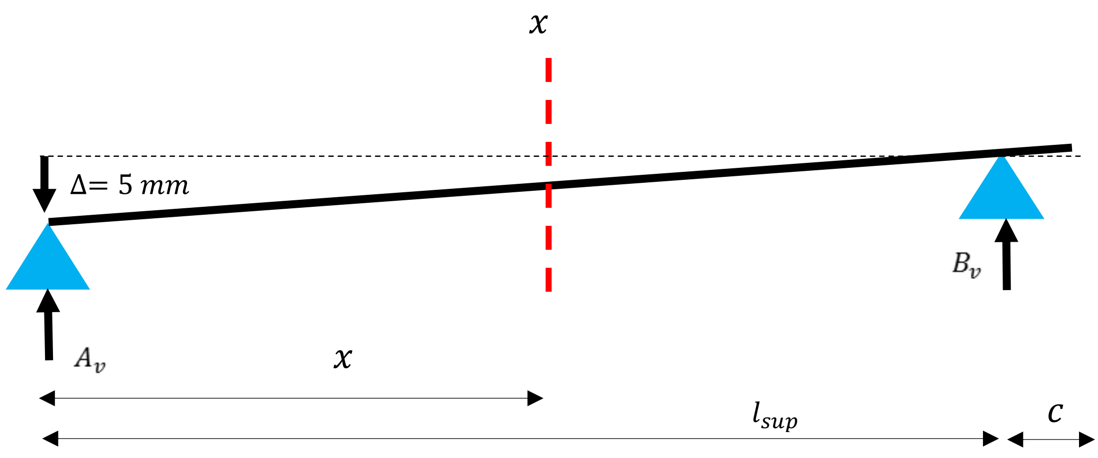
Measurements		
x	16,75	[m]
a	6,9	[m]
b	12,3	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	6809,9	[kNm]

LC 5

Loading (long. direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γG	1,00	[-]
ψ1	1,00	[-]

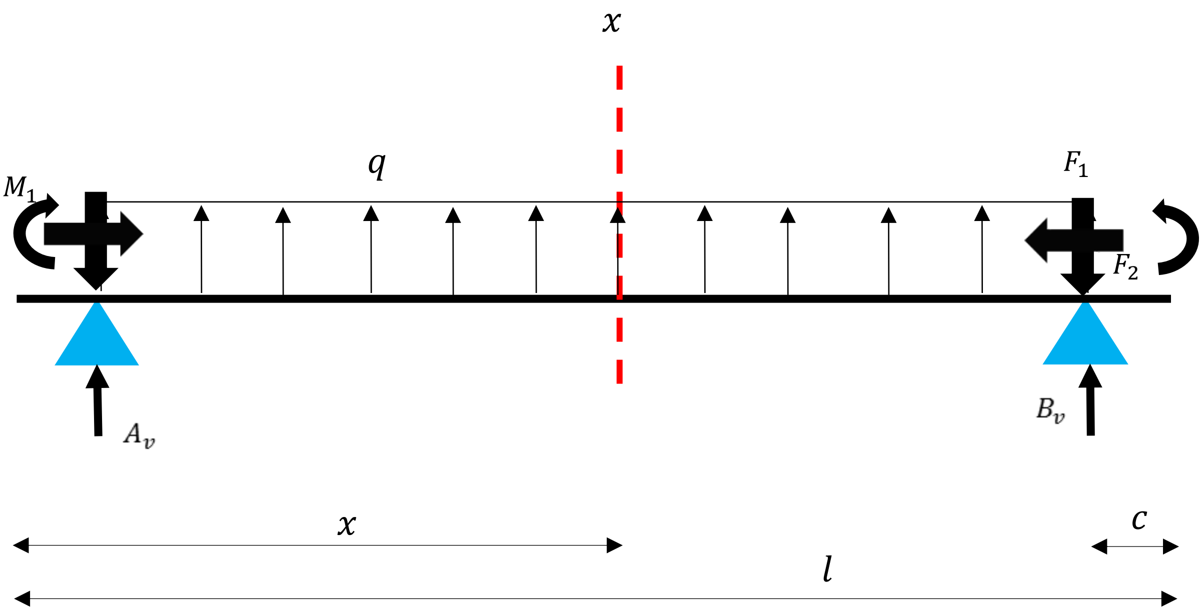
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	0,0	[kNm]

LC 7

Loading (long. direction)



Load & Reaction forces		
F1	2290	[kN]
F2	-22826	[kN]
M1	5045	[kNm]
q	-145,2	[kN]
Av	3	[kN]
Bv	3	[kN]
yP	1,00	[-]
ψ1	1,00	[-]

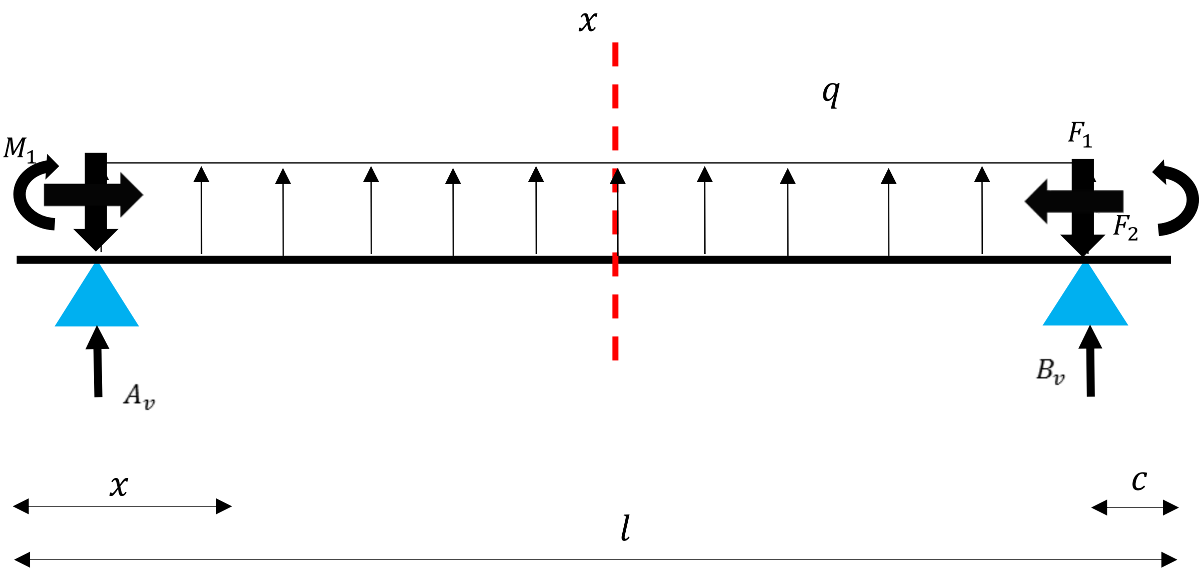
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	-12964,3	[kNm]

LC 8

Loading (long. direction)



Load & Reaction forces		
F1	2095	[kN]
F2	-20886	[kN]
M1	4616	[kNm]
q	-133	[kN]
Av	3	[kN]
Bv	3	[kN]
yP	1,00	[-]
P∞/P0	0,915	[-]
ψ1	1,00	[-]

Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	-11862,4	[kNm]

4.6 Stresses: Bridge B - quasi-permanent loading – 0,5L

Load combination: Quasi-permanent

		Prestress		Bending moment	σb [N/mm2]		U.C. [-]	
LC	type	P [kN]	σxx [N/mm2]	M [kNm]	Non-Tendon	Tendon	Non-Tendon	Tendon
1	self-weight			13403				
2	ballast			3008				
3a	Mobile Max. (LM71)			0				
5	Support settelement max			0				
6	Support settelement min			0				
7	Prestress t=0	-22826	-5,26	-12964				
8	Prestress t = ∞	-20886	-4,81	-11862				
LC 1 + LC 2 + LC 8		-20886	-4,81	4548	-8,48		O.K.	
LC 1 + LC 2 + LC 3a + LC 5 + LC 8		-20886	-4,81	4548		-2,68		O.K.
LC 1 + LC 2 + LC 3a + LC 6 + LC 8		-20886	-4,81	4548		-2,68		O.K.

Parameters

Sectional properties

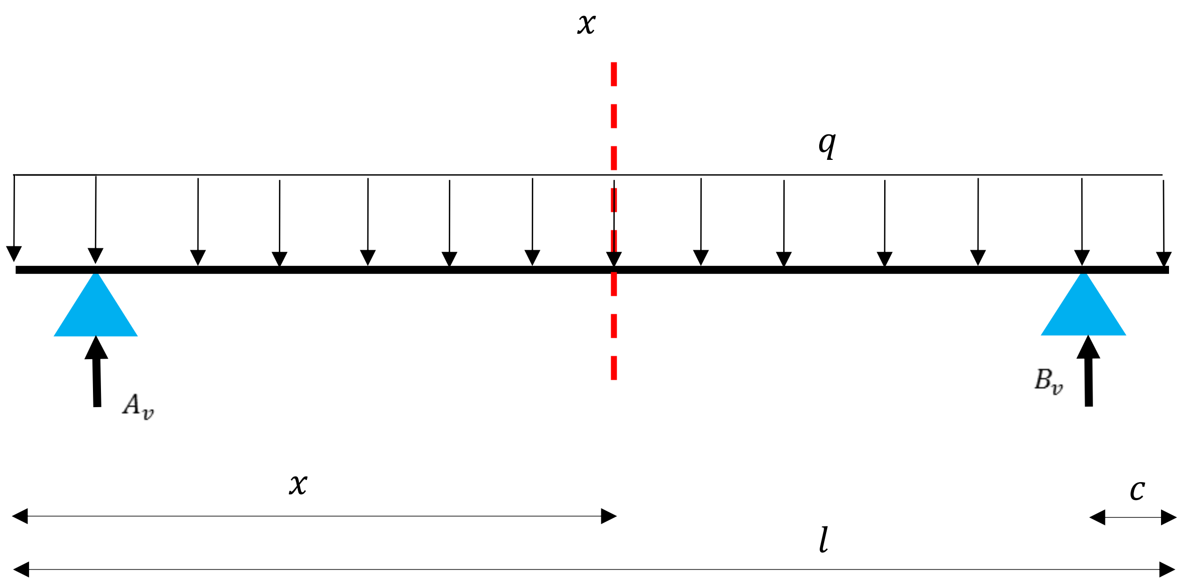
b_{girder}	1200	[mm]
h_{girder}	2200	[mm]
$0{,}5 * A_{bridge}$	4340000	[mm2]
$0{,}5 * I_{yy}$	1,73E+12	[mm4]
y	808	[mm]
z_{bottom}	808	[mm]
z_{top}	1392	[mm]

Maximum bending stress

$f_{ctk;0,05}$	2,00	[N/mm2]
$\sigma_{b,max} = 0$	0,00	[N/mm2]

LC 1

Loading (long. direction)



Load & Reaction forces			
qbridge	217	[kN/m]	
0,5q	108,5	[kN/m]	
Av	1817,4	[kN]	
Bv	1817,4	[kN]	
γG	1,00	[-]	(NEN 1990 table A2.3)
ψ2	1,00	[-]	

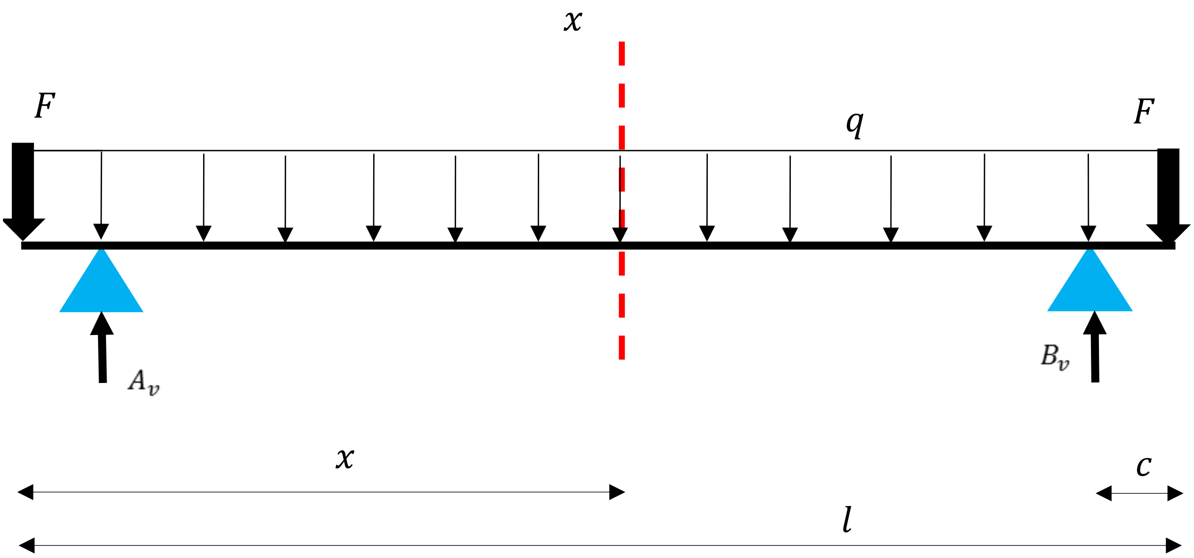
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	13403,1	[kNm]

LC 2

Loading (long. direction)



Load & Reaction forces		
q	9,9	[kN/m2]
0,5q	24,8	[kN/m]
F	99	[kN]
0,5F	50	[kN]
Av	464,2	[kN]
Bv	464,2	[kN]
γG	1,00	[-]
ψ2	1,00	[-]

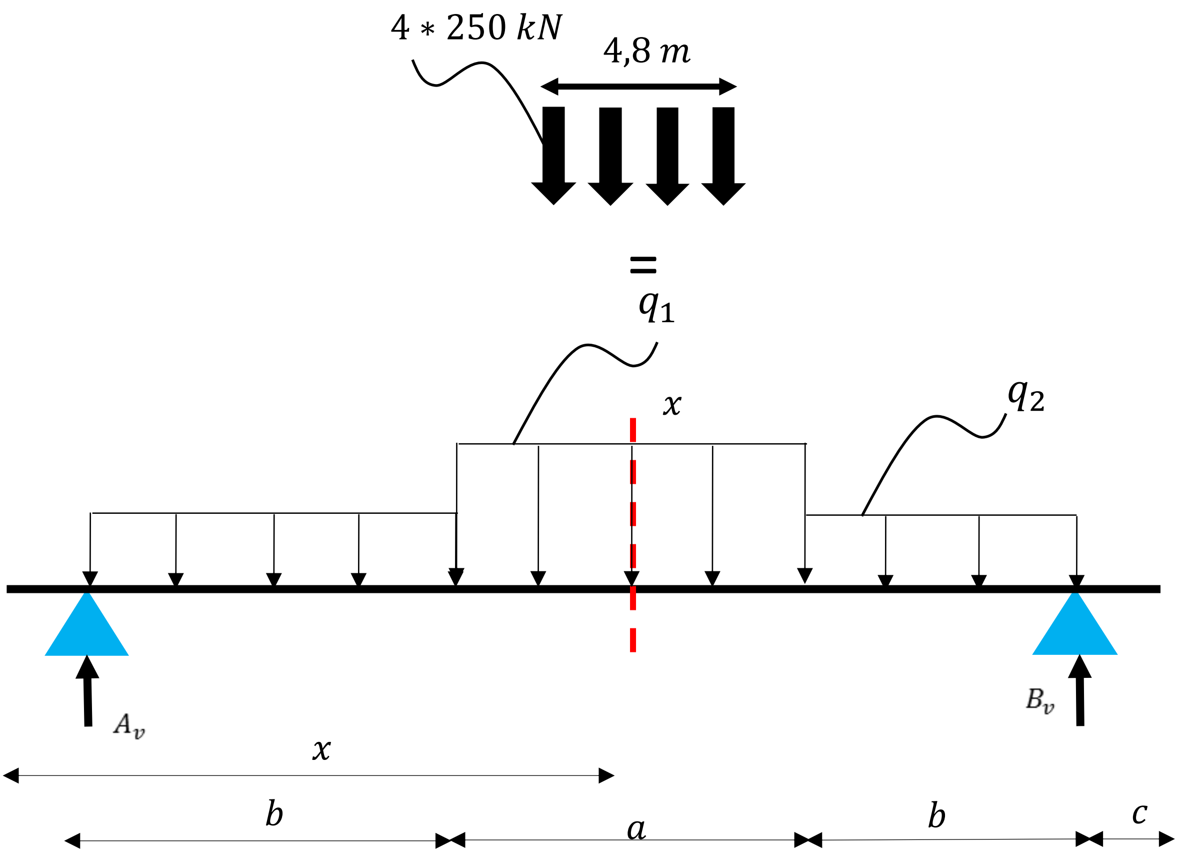
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]
lctc	5,0	[m]

Bending moment

Mx	3007,7	[kNm]

LC 3a

Loading (long. direction)



Load & Reaction forces		
q1	0,0	[kN/m]
q2	0,0	[kN/m]
0,5q1	0,0	[kN]
0,5q2	0,0	[kN]
Av	0,0	[kN]
Bv	0,0	[kN]
γQ	1,00	[-]
α	1,21	[-]
ψ2	0,00	[-]

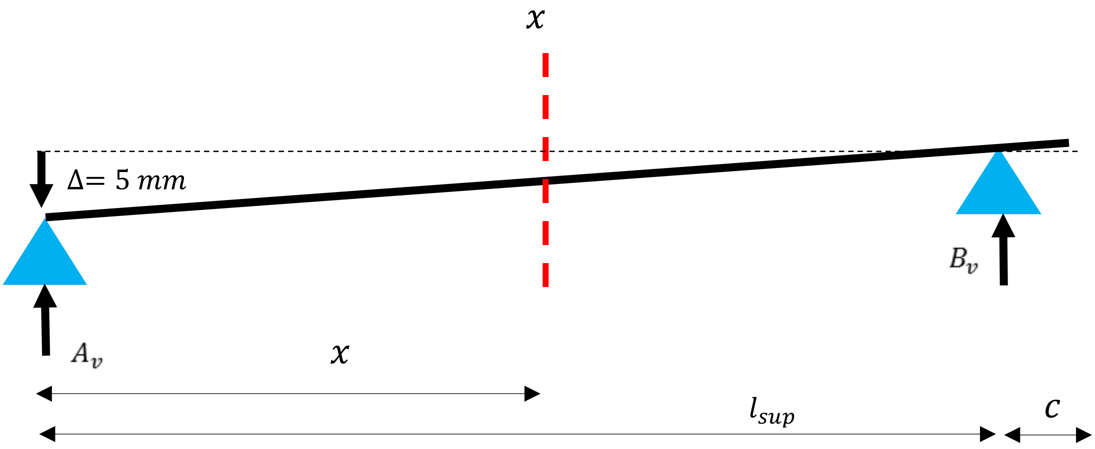
Measurements		
x	16,75	[m]
a	6,9	[m]
b	12,3	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	0,0	[kNm]

LC 5

Loading (long. direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γG	1,00	[-]
ψ2	1,00	[-]

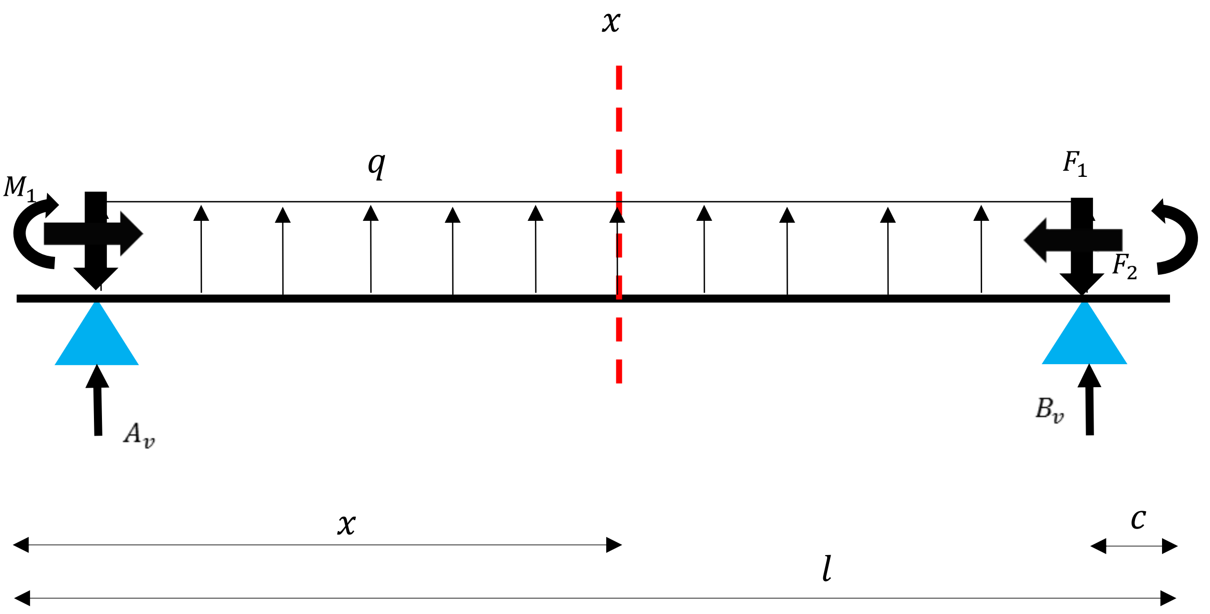
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	0,0	[kNm]

LC 7

Loading (long. direction)



Load & Reaction forces		
F1	2290	[kN]
F2	-22826	[kN]
M1	5045	[kNm]
q	-145,2	[kN]
Av	3	[kN]
Bv	3	[kN]
yP	1,00	[-]
ψ2	1,00	[-]

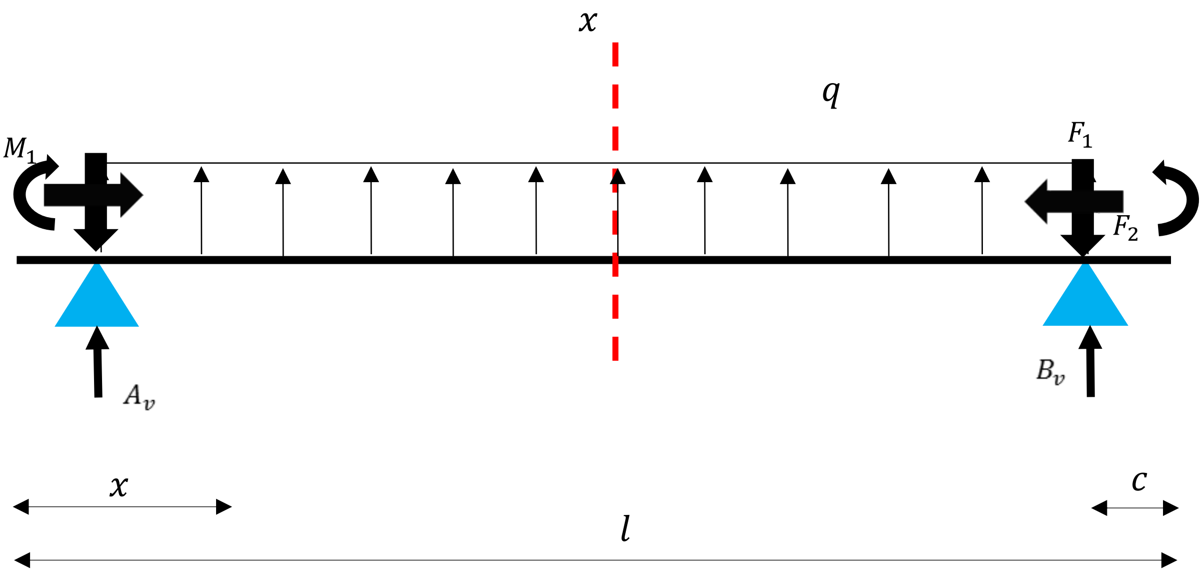
Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	-12964,3	[kNm]

LC 8

Loading (long. direction)



Load & Reaction forces		
F1	2095	[kN]
F2	-20886	[kN]
M1	4616	[kNm]
q	-133	[kN]
Av	3	[kN]
Bv	3	[kN]
yP	1,00	[-]
P∞/P0	0,915	[-]
ψ2	1,00	[-]

Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Bending moment

Mx	-11862,4	[kNm]

Appendix F – Fatigue (ULS)

1 Introduction

The through bridge is at rest when no variable loads act on it. At that moment the maximum stresses are caused by a combination of self-weight, prestress and ballast. But when a train passes the maximum stress increases significantly. During the life span of the bridge millions of trains will pass, causing an even greater number of stress fluctuation in the through girder. Materials loaded with a large number of stress fluctuations can develop small cracks. This phenomenon is called fatigue.

Fatigue calculations in the design report (3) are performed on the longitudinal and shear reinforcement by limiting the stress fluctuations to a maximum. However this maximum allowable stress fluctuation is a lot higher than what the Eurocode considers acceptable. Additionally the design report only performs fatigue checks on reinforcement, whereas according to the Eurocode concrete under compression should be checked for fatigue as well.

Due to the different calculation procedures and load models, a fatigue resistance calculation is performed on the through girder according to the Eurocode. Girder A remains uncracked (appendix E), leading to no strains in the shear and longitudinal reinforcement. This makes the reinforcement non-sensitive to fatigue, meaning only a fatigue resistance calculation on concrete under compression is necessary. Girder B however is cracked according to the requirements of the OVS. Besides a fatigue check on concrete, a fatigue check on the prestress steel, longitudinal and shear reinforcement is necessary.

2 Concrete

At the start of this research, fatigue verification according to annex NN in Eurocode 2 (20) was allowed by the OVS. But with the arrival of the new version of the OVS, the fatigue verifications needs to be performed according to the general part of Eurocode 2. Because the new OVS was published towards the end of this research, the calculations are performed using annex NN.

The load combination for fatigue verification is not explicitly mentioned in Eurocode 0 and is therefore derived from annex NN.3. The maximum, minimum and permanent characteristic load combination at ultimate limit state need to be considered. Even though this is an ULS check, all partial load factors are taken equal to 1,0. Additionally the factor α, (according to the OVS (19)) should be taken equal to 1,0 instead of 1,21.

$$\sum G + P \qquad \text{Permanent}$$

$$\sum G + P + Q_{LM71,max} \qquad \text{Maximum LM71}$$

$$\sum G + P + Q_{LM71,min} \qquad \text{Minimum LM71}$$

The maximum load combination of LM71 corresponds with heavy axle loads at the start of the bridge and a distributed load over the rest. Whereas he minimum load combination is found when the cantilever is loaded with heavy axle loads. Naturally the permanent load combination consist of self-weight, ballast and prestress.

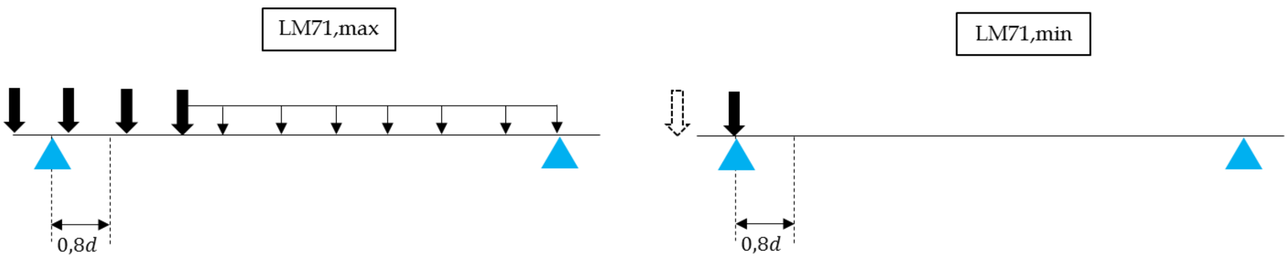


Figure F-1: Maximum and minimum loading due to LM71 (section 0,8d)

2.1 Uncracked girder

The next step, in order to determine the resistance of concrete against fatigue, is to determine the maximum compressive stress in the girder. Because girder A is uncracked and girder B is cracked, two different approaches need to be considered. This paragraph focuses on the uncracked approach, where the maximum compressive stress is found in the top fibre of the girder.

According to Eurocode 2 the angle of the compressive diagonal used at ultimate limit state (θ) maybe converted into θ_{fat} with equation [F.1].

$$\tan\theta_{fat} = \sqrt{\tan\theta} \quad \text{[F.1]}$$

The angle (under fatigue loading) for a section near the support and at midspan is respectively 32,3° and 45° (Table F-1).

Table F-1: Angle of the compressive diagonal under fatigue loading

Section	θ	θ_{fat}
0,8d	21,8°	32,3°
support	21,8°	32,3°
0,5L	45°	45°

In order to find the maximum compressive stress one needs to combine the torsion, clamping moment, suspension and shear force into one total shear force:

$$V_{Ed,tot} = V_{Ed} + \left(Q_{yy} + \frac{m_{xx}}{z_1}\right) * z * \cot\theta_{fat} + \frac{T_{Ed} * z}{2 * A_k} \quad \text{[F.2]}$$

This total shear force is then combined with a bending moment, in order to find the bending stress in the considered section (Chapter 4 Appendix D):

$$M_{Ed,tot} = M_{Ed} + 0{,}5 * V_{Ed} * z * \cot\theta_{fat} \quad \text{[F.3]}$$

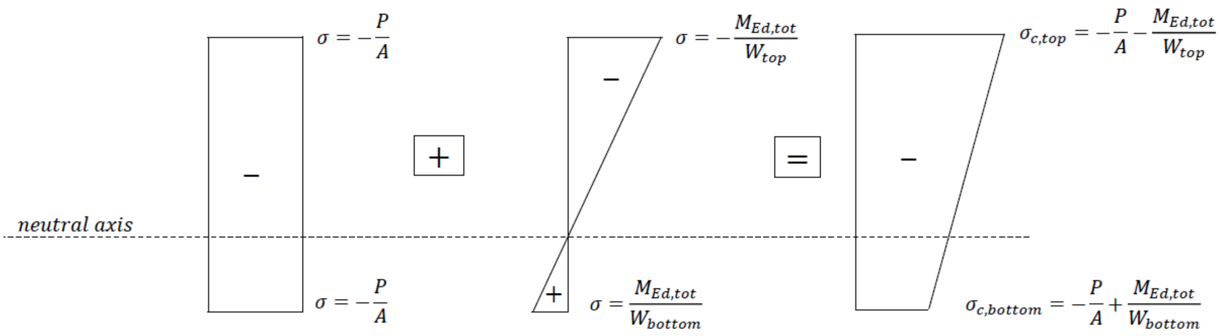


Figure F-2: Stress diagrams due to prestress and bending and their combination

Ultimately the compression in the top fibre is computed by equation [F.4]:

$$\sigma_{c,top} = -\frac{P_\infty}{A_c} - \frac{M_{Ed,tot}}{W_{top}} \qquad [\text{ F.4 }]$$

The characteristic load combinations in bridge A (at 0,8d and 0,5L), result in the compressive stresses in the top fibre as presented by Table F-2. A section at midspan is subjected to larger bending moments, which results in a larger stress fluctuation in the concrete.

Table F-2: Bridge A: Compressive stress in top fibre for different characteristic load combinations

Load Combination	$\sigma_{c,top}$ $[N/mm^2]$	
	0,8d ($\theta_{fat} = 32{,}3°$)	0,5L ($\theta_{fat} = 45°$)
Permanent	-7,29	-4,87
LM71, min	-7,54	-4,81
LM71, max	-10,40	-11,14

2.2 Cracked girder

As mentioned before girder B is cracked. The tensile bending strength of the concrete goes to zero and (possible) tensile forces are transferred by the longitudinal reinforcement and prestressing steel. Using the cracked girder approach, a value for the height of the compression zone and the concrete strain needs to be obtained (Appendix D). Based on this first value, two approach are possible:

1. $x > h$: Height of the compression zone is larger than the height of the cross-section. The entire section is under compression, meaning there are no (additional) tensile forces in the reinforcement and prestressing steel and the requirements for fatigue are met. However the concrete is subjected to a compressive stress fluctuation that cannot be neglected.

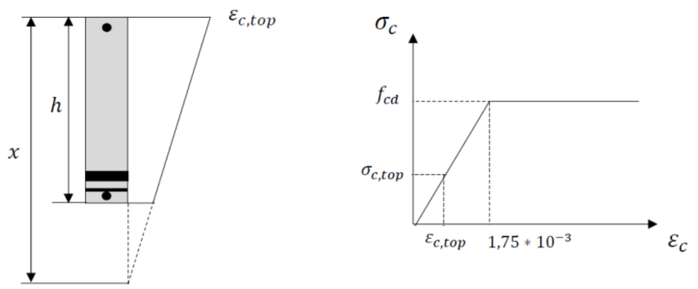


Figure F-3: Compression zone relative to the cross-section (left), stress-strain diagram (right)

   A strain of 1,75‰ corresponds with the concrete design strength and forms the upper limit of the elastic stage. A strain that is smaller than that corresponds with a compressive stress of:

$$\sigma_{c,top} = \frac{\varepsilon_{c,top}}{1{,}75 * 10^{-3}} * f_{cd} \qquad [\text{ F.5 }]$$

   By determining the strain for the permanent, maximum and minimum characteristic load case, the compressive stress fluctuation can be established.

2. $x < h$: Height of the compression zone is smaller than the height of the cross-section which means a part of the section will be loaded in tension. The stress fluctuations in the concrete, reinforcement and prestress steel need to be determined in order to perform a fatigue resistance calculation.

As it turns out the first approach is valid for all load combinations. Meaning the prestressing steel and longitudinal reinforcement in girder B fulfil the fatigue requirements. Table F-3 presents the accompanying concrete stresses in the top fibre.

Table F-3: Bridge B: Compressive stress in top fibre for different characteristic load combinations

Load Combination	$\sigma_{c,top}$ $[N/mm^2]$	
	support ($\theta_{fat} = 32{,}3°$)	0,5L ($\theta_{fat} = 45°$)
Permanent	-10,21	-9,87
LM71, min	-10,09	-9,84
LM71, max	-10,93	-14,13

2.3 Fatigue resistance

Let's consider an example for a section at 0,8d in girder A using the damage equivalent stress approach (annex NN.3.2 in Eurocode 2). The first step is to determine the fatigue design strength:

$$f_{cd,fat} = k_1 \beta_{cc}(t_0) * f_{cd} * \left(1 - \frac{f_{ck}}{400}\right) \quad [\text{ F.6 }]$$

Where:

$k_1 = 0{,}85$ (According to the Dutch National Annex)
$f_{ck} = 35\ N/mm^2$ (Design report (3))
$f_{cd} = 35/1{,}5 = 23{,}3\ N/mm^2$
$\beta_{cc}(t_0) = 1{,}0$ (Factor for concrete strength at first loading)

$$f_{cd,fat} = 0{,}85 * 1{,}0 * 23{,}3 * \left(1 - \frac{35}{400}\right) = 18{,}1\ N/mm^2$$

The Eurocode takes into account a certain correction factor, λ_c , which is applied on the stress fluctuation. This factor accounts for the annual traffic volume, lifespan, number of tracks, length and permanent loading.

$$\lambda_c = \lambda_{c,0} * \lambda_{c,1} * \lambda_{c,2,3} * \lambda_{c,4} \quad [\text{ F.7 }]$$

- The $\lambda_{c,0}$ factor accounts for the influence of the permanent load:

$\lambda_{c,0} = 0{,}94 + 0{,}2 * \frac{\sigma_{c,perm}}{f_{cd,fat}} \geq 1{,}0$ (for the compression zone)

$$\lambda_{c,0} = 0{,}94 + 0{,}2 * \frac{7{,}29}{18{,}1} = 1{,}02$$

- The $\lambda_{c,1}$ factor accounts for the critical length of the influence line:

$$\lambda_{c,1} = \lambda_{c,1}\,(2\ m) + [\lambda_{c,1}\,(20\ m) - \lambda_{c,1}(2\ m)] * \log(L - 0{,}3)$$

Where:

$\lambda_{c,1}\,(2\ m) = 0{,}70$ (According to table NN.3)
$\lambda_{c,1}\,(20\ m) = 0{,}75$ (According to table NN.3)
$L = 20\ m$ (Critical length of the influence line)

$$\lambda_{c,1} = 0{,}70 + [0{,}75 - 0{,}70] * (\log(20) - 0{,}3) = 0{,}75$$

- The $\lambda_{c,2,3}$ factor accounts for the annual traffic volume and the lifespan of the bridge:

$$\lambda_{c,2,3} = 1 + \frac{1}{8} * \log\left[\frac{Vol}{25 * 10^6}\right] + \frac{1}{8} * \log\left[\frac{N_{years}}{100}\right]$$

Where:

$$Vol = 25 * 10^6$$
$$N_{years} = 100$$

$$\lambda_{c,2,3} = 1 + \frac{1}{8} * \log\left[\frac{25 * 10^6}{25 * 10^6}\right] + \frac{1}{8} * \log\left[\frac{100}{100}\right] = 1{,}0$$

- The $\lambda_{c,4}$ factor accounts for the number of tracks on the bridge:

$\lambda_{c,4} = 1{,}0$ (Bridge A is a single track bridge)

The total correction factor goes to:

$$\lambda_c = 1{,}02 * 0{,}75 * 1{,}0 * 1{,}0 = 0{,}77$$

The largest and the smallest stress of the damage equivalent stress fluctuation need to be determined from equation [F.8] and [F.9].

$$\sigma_{cd,max,equ} = \sigma_{c,perm} + \lambda_c(\sigma_{c,max,71} - \sigma_{c,perm}) \qquad [\text{ F.8 }]$$

$$\sigma_{cd,min,equ} = \sigma_{c,perm} - \lambda_c(\sigma_{c,perm} - \sigma_{c,min,71}) \qquad [\text{ F.9 }]$$

$$\sigma_{cd,max,equ} = 7{,}29 + 0{,}77 * (10{,}40 - 7{,}29) = 9{,}67\ N/mm^2$$

$$\sigma_{cd,min,equ} = 7{,}29 - 0{,}77 * (7{,}29 - 7{,}54) = 7{,}48\ N/mm^2$$

The final check is to verify the fatigue resistance of concrete in the form of equation [F.10].

$$14 * \frac{1 - E_{cd,max,equ}}{\sqrt{1 - R_{equ}}} \geq 6 \qquad [\text{ F.10 }]$$

Where:

$$E_{cd,min,equ} = \gamma_{sd} * \frac{\sigma_{cd,min,equ}}{f_{cd,fat}}$$

$$E_{cd,max,equ} = \gamma_{sd} * \frac{\sigma_{cd,max,equ}}{f_{cd,fat}}$$

$$R_{equ} = \frac{E_{cd,min,equ}}{E_{cd,max,equ}}$$

$\gamma_{sd} = 1{,}0$ (Factor accounting for model uncertainties)

$$E_{cd,min,equ} = 1{,}0 * \frac{7{,}48}{18{,}1} = 0{,}41$$

$$E_{cd,max,equ} = 1{,}0 * \frac{9{,}67}{18{,}1} = 0{,}53$$

$$R_{equ} = \frac{0{,}41}{0{,}53} = 0{,}77$$

The final check shows one that concrete has sufficient resistance against fatigue:

$$14 * \frac{1 - 0{,}53}{\sqrt{1 - 0{,}77}} = 13{,}71 \geq 6$$

The fatigue resistance calculation is performed on the uncracked girder (A) and cracked girder (B). The results of the calculations are presented in Table F-4, from which it can be concluded that the most critical section is at midspan in bridge B. This section is subjected to large bending moments, which causes large stress fluctuations in the top fibre which ultimately leads to the most critical unity check.

Table F-4: Concrete fatigue resistance for girder A & B

Bridge	$14 * \frac{1 - E_{cd,max,equ}}{\sqrt{1 - R_{equ}}}$		
	0,8d	support	0,5L
A	13,71 > 6		9,36 > 6
B		22,82 > 6	**7,46 > 6**

3 Shear reinforcement

In an uncracked section, the strains in the shear reinforcement remain zero, making the shear reinforcement non-sensitive to fatigue. A cracked girder however is different, in that case there are strains in the reinforcement making the stirrups sensitive to fatigue. Girder B is cracked and the fatigue check on the stirrups is therefore a necessity. Contrastingly girder A is uncracked and the check is unnecessary. However it is assumed as cracked (due to e.g. thermal action) to see whether or not the stirrups have enough fatigue resistance.

3.1 Stress range

First one needs to determine the stresses in the outer and inner stirrup. This calculation is quite similar to the approach in appendix D. Zone III is responsible for transferring torsion, shear and the suspension leads, where zone II only transfers shear forces. But from appendix D it can be learned that zone III on its own has insufficient capacity and that a distribution of respectively 35% and 65% of the suspension loads over zone II and III takes place. This distribution is also processed in this fatigue resistance calculation.

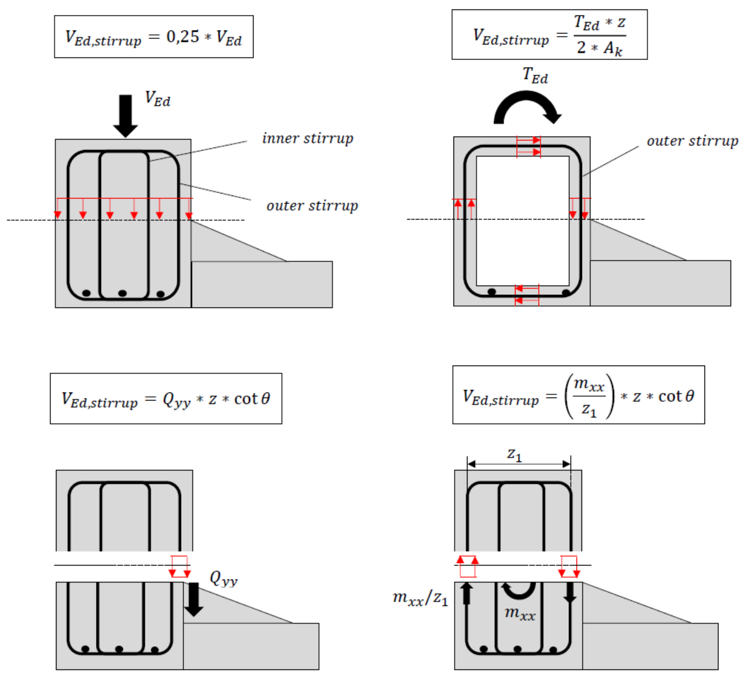


Figure F-4: Forces in the stirrups due to shear (left top), torsion (right top), suspension force (left bottom), clamping moment (right bottom)

Because the suspension loads are per meter, they are multiplied with the horizontal length of the diagonal. Taking the load distribution into account, the total shear fore in the outer and inner stirrup is expressed by [F.11] and [F.12].

$$V_{Ed,tot} = 0,25 * V_{Ed} + \frac{T_{Ed} * z}{2 * A_k} + z * \cot\theta * \left(Q_{yy} + \frac{m_{xx}}{z_1}\right) * 65\% \qquad [\text{ F.11 }]$$

$$V_{Ed,tot} = 0,25 * V_{Ed} + z * \cot\theta * \left(Q_{yy} + \frac{m_{xx}}{z_1}\right) * 35\% \qquad [\text{ F.12 }]$$

The maximum and minimum characteristic load combination are considered for bridge A and B. This results in the following stresses in the outer and inner stirrup:

Table F-5: Stresses in the outer and inner stirrup of girder A (0,8d & 0,5L)

Load Combination	Outer stirrup: σ_s $[N/mm^2]$		Inner stirrup: σ_s $[N/mm^2]$	
	0,8d	0,5L	0,8d	0,5L
LM71, min	207	72	10	21
LM71, max	283	105	124	52

Table F-6: Stresses in the outer stirrup of girder B (support & 0,5L)

Load Combination	Outer stirrup: σ_s $[N/mm^2]$	
	support	0,5L
LM71, min	104	24
LM71, max	182	52

3.2 Fatigue resistance

The annex NN.3.1 in Eurocode 2 (20) uses the damage equivalent stress approach to determine the resistance of reinforcement against fatigue. The equivalent stress is computed by equation [F.13].

$$\Delta\sigma_{s,equ} = \lambda_s * \Phi * \Delta\sigma_{s,71} \qquad [\text{ F.13 }]$$

The stress ranges due to LM71, including dynamic factor, can be derived from Table F-5 and Table F-6. The correction factor needs to be computed with equation [F.14].

$$\lambda_s = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 \qquad [\text{ F.14 }]$$

- The $\lambda_{s,1}$ factor accounts for the critical length of the influence line:

$$\lambda_{s,1} = \lambda_{s,1}\,(2\,m) + \left[\lambda_{s,1}\,(20\,m) - \lambda_{s,1}(2\,m)\right] * \log(L - 0{,}3)$$

Where:

$\lambda_{s,1}\,(2\,m) = 0{,}90$ (According to table NN.2)
$\lambda_{s,1}\,(20\,m) = 0{,}65$ (According to table NN.2)
$L = 20\,m$ (Critical length of the influence line)

$$\lambda_{s,1} = 0{,}90 + [0{,}65 - 0{,}90] * (\log(20) - 0{,}3) = 0{,}65$$

- The $\lambda_{s,2}$ factor accounts for the annual traffic volume:

$$\lambda_{s,2} = \sqrt[k_2]{\frac{Vol}{25 * 10^6}}$$

Where:

$Vol = 25 * 10^6$

$k_2 = 9{,}0$ (Table 6.3N in NEN 1992-1-1)

$$\lambda_{s,2} = \sqrt[9]{\frac{25 * 10^6}{25 * 10^6}} = 1{,}0$$

- The $\lambda_{s,3}$ factor accounts for the lifespan of the bridge:

$$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}}$$

Where:

$N_{years} = 100$

$$\lambda_{s,3} = \sqrt[9]{\frac{100}{100}} = 1{,}0$$

- The $\lambda_{c,4}$ factor accounts for the number of tracks on the bridge:

$\lambda_{c,4} = 1{,}0$ (Bridge A & B are single track bridges)

This yields in,

$\lambda_s = 0{,}65 * 1{,}0 * 1{,}0 * 1{,}0 = 0{,}65$

The damage equivalent stress in the outer stirrup of girder A (at 0,8) is equal to:

$\Delta\sigma_{s,equ} = 0{,}65 * (283 - 207) = 49{,}6\ N/mm^2$

The damage equivalent stress should remain smaller than the critical stress range at N^* cycles:

$$\gamma_{F,fat} * \Delta\sigma_{s,equ}(N^*) \leq \frac{\Delta\sigma_{Rsk}(N^*)}{\gamma_{S,fat}} \qquad [\text{ F.15 }]$$

With help of section 6.8.4 in NEN-EN 1991-1-1 the S-N curve for S435 steel is established. The structure is subjected to 25 million cycles, which gives one a critical stress range of 113,6 N/mm^2.

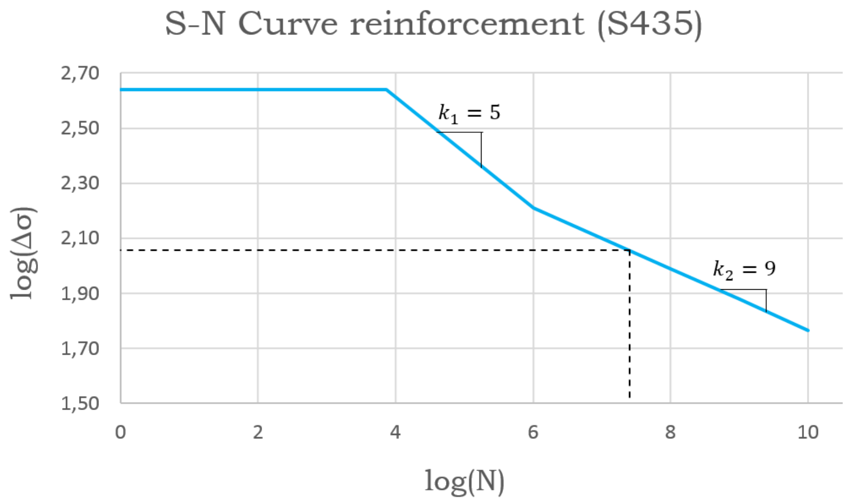


Figure F-5: S-N curve for S435 reinforcement

According to the Dutch National Annex to Eurocode 2 (13) the partial load and material factor can be taken as: $\gamma_{F,fat} = 1{,}0$ and $\gamma_{S,fat} = 1{,}15$.

The unity check for the outer stirrup in girder A at 0,8d eventually goes to:

$$U.C. = \frac{\gamma_{F,fat} * \Delta\sigma_{s,equ}(N^*)}{\Delta\sigma_{Rsk}(N^*)/\gamma_{s,fat}} = \frac{1{,}0 * 49{,}6}{113{,}6/1{,}15} = 0{,}50$$

Presented by Table F-7 and Table F-8 are the unity checks for fatigue in the shear reinforcement. The values stay well within the acceptable limits and there is no risk of fatigue failure of the stirrups.

Table F-7: Bridge A: Unity checks for fatigue in the shear reinforcement

Type of reinforcement	Unity check [-]	
	0,8d	0,5L
Outer stirrup	0,50	0,22
Inner stirrup	**0,75**	0,21

Table F-8: Bridge B: Unity checks for fatigue in the shear reinforcement

Type of reinforcement	Unity check [-]	
	support	0,5L
Stirrup	**0,51**	0,19

4 Longitudinal reinforcement

According to the OVS girder A remains uncracked, which means the entire section is under compression and there are no (additional) tensile forces in the longitudinal reinforcement and prestressing steel. A check on the fatigue resistance is thus irrelevant.

Girder B however turns out to be cracked, which makes one expect (additional) tensile forces in the reinforcement and prestressing steel. Yet in paragraph 2.2 it was concluded that the height of the compression zone is larger than the cross-sectional height. Consequently the entire section is under compression and the requirements regarding fatigue are automatically met for the longitudinal reinforcement and prestressing steel.

5 Spreadsheets

5.1 Fatigue: Bridge A – Characteristic loading – 0,8d

Forces

		Prestress	Bending moment
LC	type	P [kN]	M [kNm]
1	self-weight		884
2	ballast		245
3	LM 71 max (1/2)		453
5a	LM71 max (2/2)		335
5b	LM71 min		-41
6	Support settelement max		0
7	Support settelement min		0
8	Prestress t=0	-13200	-1055
9	Prestress t = ∞	-12038	-962
σperm (1+2+6+9)		-12038	167
σ71,min (1+2+5b+6+9)		-12038	126
σ71,max (1+2+3+5a+6+9)		-12038	956

		Suspension force	Clamping moment	Shear force	Torsion	Total shear force
LC	type	Qyy [kN]	mxx [kNm]	[kN]	[kNm]	[kN]
1	self-weight	29	13	628	-74	
2	ballast	23	14	214	-65	
3	LM 71 max (1/2)	82	-11	307	-120	
5a	LM71 max (2/2)	0	19	247	13	
5b	LM71 min	0	24	0	-96	
6	Support settelement max	15	82	0	-178	
7	Support settelement min	-15	-82	0	178	
8	Prestress t=0	-52	28	-923	37	
9	Prestress t = ∞	-47	26	-842	34	
σperm (1+2+6+9)		21	135	0	283	694
σ71,min (1+2+5b+6+9)		21	158	0	380	848
σ71,max (1+2+3+5a+6+9)		103	142	554	390	1563

Fatigue Parameters

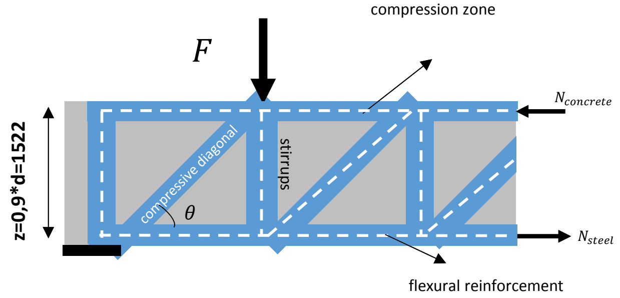
Geometry			
length L	=	21000	[mm]
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1691	[mm]
$0{,}5I_{yy}$	=	6,63E+11	[mm4]
y	=	628	[mm]
$0{,}5A$	=	2150500	[mm2]

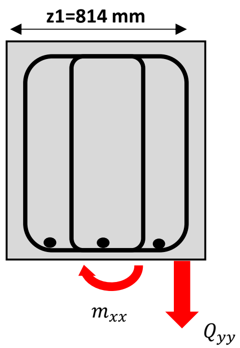
Reinforcement			
Øouter stirrup	=	16	[mm]
Øinner stirrup	=	12	[mm]
Øflexural reinf.	=	16	[mm]
spacing outer stirrup	=	150	[mm]
spacing inner stirrup	=	150	[mm]
n rebar flex. reinf.	=	14	[-]
$A_{outer\ stirrup}$	=	1,34	[mm2/mm]
$A_{inner\ stirrup}$	=	0,75	[mm2/mm]
$A_{flexural\ reinforcement}$	=	2815	[mm2]
A_p	=	1900	[mm2]

Internal lever arm			
$z = 0{,}9 * d$	=	1522	[mm]
$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	814	[mm]

Angle compr. diagonal			
θfat	=	32,3	[◦]
θ	=	21,8	[◦]

Box girder properties			
$Area\ (A)$	=	1,575	[m2]
$Perimeter\ (u)$	=	5,30	[m]
$t_{ef} = A/u$	=	297	[mm]
$t_{ef,lower\ limit} = 2 * (h - d)$	=	118	[mm]
A_k	=	0,88	[m2]
u_k	=	4,11	[m]





Stress range concrete

σperm (1+2+6+9)

M_{Ed} = 167 [kNm]

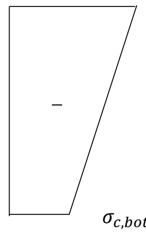
$V_{Ed,tot} = V_{Ed} + (Q_{yy} + m_{xx}/z_1) * z * \cot\theta + \frac{T_{Ed} * z}{2 * A_k}$ = 694 [kN]

$W_{top} = I_{yy}/(h-y)$ = 5,91E+08 [mm3]

$W_{bottom} = I_{yy}/(y)$ = 1,06E+09 [mm3]

$\sigma_{c,top}$ = -7,29 [N/mm2]

$\sigma_{c,bottom}$ = -4,65 [N/mm2]



$$\sigma_{c,top} = -\frac{P}{A} - \frac{M_{Ed} + V_{Ed} * 0,5 * z * \cot\theta}{W_{top}}$$

$$\sigma_{c,bottom} = -\frac{P}{A} + \frac{M_{Ed} + V_{Ed} * 0,5 * z * \cot\theta}{W_{bottom}}$$

σ71,min (1+2+5b+6+9)

M_{Ed} = 126 [kNm]

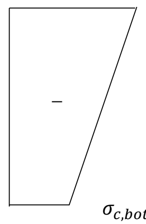
$V_{Ed,tot} = V_{Ed} + (Q_{yy} + m_{xx}/z_1) * z * \cot\theta + \frac{T_{Ed} * z}{2 * A_k}$ = 848 [kN]

$W_{top} = I_{yy}/(h-y)$ = 5,91E+08 [mm3]

$W_{bottom} = I_{yy}/(y)$ = 1,06E+09 [mm3]

$\sigma_{c,top}$ = -7,54 [N/mm2]

$\sigma_{c,bottom}$ = -4,51 [N/mm2]



$$\sigma_{c,top} = -\frac{P}{A} - \frac{M_{Ed} + V_{Ed} * 0,5 * z * \cot\theta}{W_{top}}$$

$$\sigma_{c,bottom} = -\frac{P}{A} + \frac{M_{Ed} + V_{Ed} * 0,5 * z * \cot\theta}{W_{bottom}}$$

σ71,max (1+2+3+5a+6+9)

M_{Ed} = 956 [kNm]

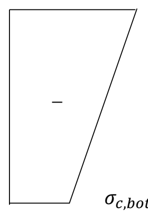
$V_{Ed,tot} = V_{Ed} + (Q_{yy} + m_{xx}/z_1) * z * \cot\theta + \frac{T_{Ed} * z}{2 * A_k}$ = 1563 [kN]

$W_{top} = I_{yy}/(h-y)$ = 5,91E+08 [mm3]

$W_{bottom} = I_{yy}/(y)$ = 1,06E+09 [mm3]

$\sigma_{c,top}$ = -10,40 [N/mm2]

$\sigma_{c,bottom}$ = -2,91 [N/mm2]



$$\sigma_{c,top} = -\frac{P}{A} - \frac{M_{Ed} + V_{Ed} * 0,5 * z * \cot\theta}{W_{top}}$$

$$\sigma_{c,bottom} = -\frac{P}{A} + \frac{M_{Ed} + V_{Ed} * 0,5 * z * \cot\theta}{W_{bottom}}$$

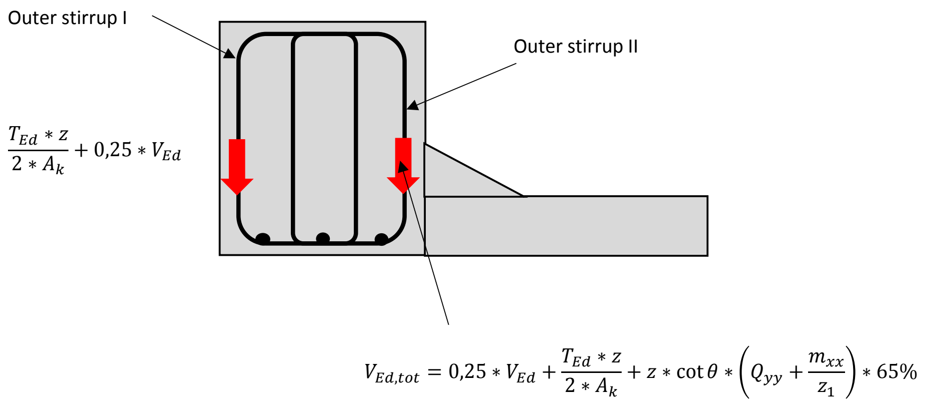
Stress range top fibre

Load combination	Total
	σc,t [N/mm2]
σperm (1+2+7+9)	-7,29
σ71,min (1+2+5b+7+9)	-7,54
σ71,max (1+2+3+5a+7+9)	-10,40

Stress range stirrups

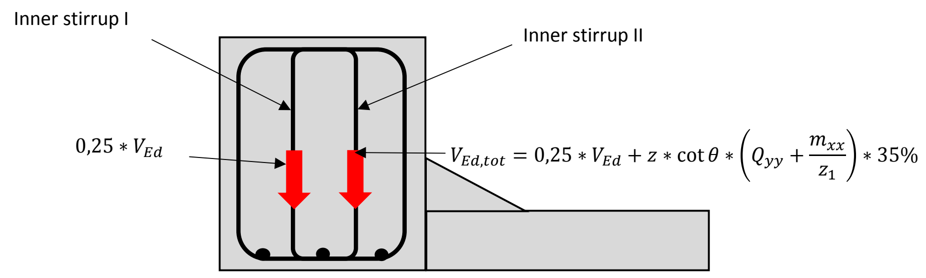
Stress range outer stirrup

		Total	
LC	type	VEd,tot [kN]	σstirrup [N/mm2]
1	self-weight		
2	ballast		
3	LM 71 max (1/2)		
5a	LM71 max (2/2)		
5b	LM71 min		
6	Support settelement max		
7	Support settelement min		
8	Prestress t=0		
9	Prestress t = ∞		
σperm (1+2+6+9)		537	167
σ71,min (1+2+5b+6+9)		666	207
σ71,max (1+2+3+5a+6+9)		913	283



Stress range inner stirrup

		Total	
LC	type	VEd,tot [kN]	σstirrup [N/mm2]
1	self-weight		
2	ballast		
3	LM 71 max (1/2)		
5a	LM71 max (2/2)		
5b	LM71 min		
6	Support settelement max		
7	Support settelement min		
8	Prestress t=0		
9	Prestress t = ∞		
σperm (1+2+6+9)		18	10
σ71,min (1+2+5b+6+9)		18	10
σ71,max (1+2+3+5a+6+9)		225	124



Concrete Fatigue verification (damage equivalent stress)

Fatigue strength

k_1	=	0,85	[-]	(NEN-EN 1992-1-1 NB)
$\beta_{cc}(t_0)$	=	1,00	[-]	
f_{ck}	=	35	[N/mm2]	(RBK)
$\gamma_{c,fat}$	=	1,50	[-]	(NEN-EN 1992-1-1 NB)
f_{cd}	=	23,3	[N/mm2]	
$f_{cd,fat}$	=	18,1	[N/mm2]	

Concrete stress

γ_{sd}	=	1,00	[-]
$\sigma_{c,perm}$	=	7,29	[N/mm2]
$\sigma_{c,max,71}$	=	10,40	[N/mm2]
$\sigma_{c,min,71}$	=	7,54	[N/mm2]
$\sigma_{cd,max,equ}$	=	9,67	[N/mm2]
$\sigma_{cd,min,equ}$	=	7,48	[N/mm2]
$E_{cd,min,equ}$	=	0,41	[-]
$E_{cd,max,equ}$	=	0,53	[-]

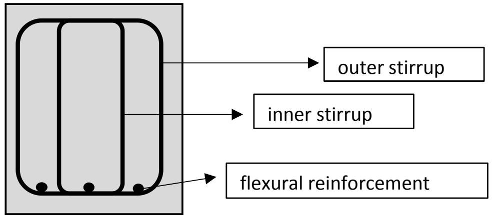
Unity check

$R_{equ} = \frac{E_{cd,min,equ}}{E_{cd,max,equ}}$	=	0,77 [-]			
$14 * \frac{1 - E_{cd,max,equ}}{\sqrt{1 - R_{equ}}} \geq 6$	=	13,71	>	6	OK

Correction factor λc

Factor for permanent stress	=	$\lambda_{c,0} = 0{,}94 + 0{,}2 * \sigma_{c,perm}/f_{cd,fat} \geq 1$	=	1,02 [-]
Factor for element type	=	$L = critical\ length\ influence\ line$	=	20,0 [m]
	=	$\lambda_{c,1}(2\ m) = according\ to\ table\ NN.3$	=	0,70 [-]
	=	$\lambda_{c,1}(20\ m) = according\ to\ table\ NN.3$	=	0,75 [-]
	=	$\lambda_{c,1}(L) = \lambda_{c,1}(2\ m) + [\lambda_{c,1}(20\ m) - \lambda_{c,1}(2\ m)] * (\log L - 0{,}3)$	=	0,75 [-]
Factor for volume and life	=	Vol	=	2,50E+07 [ton/year/tr]
	=	N_{years}	=	100 [year]
	=	$\lambda_{c,2,3} = 1 + \frac{1}{8} * \log\left(\frac{Vol}{25 * 10^6}\right) + \frac{1}{8} * \log\left(\frac{N_{years}}{100}\right)$	=	1,00 [-]
Factor for more than one track	=	$\lambda_{c,4} =$	=	1,00 [-]
Damage equivalent factor	=	$\lambda_c = \lambda_{c,0} * \lambda_{c,1} * \lambda_{c,2,3} * \lambda_{c,4} =$	=	0,77 [-]

Outer stirrup fatigue verification (damage equivalent stress)



Damage equivalent stress

$\Phi * \Delta\sigma_{s,71}$	=	76,33	[N/mm2]
$\Delta\sigma_{s,equ}$	=	49,60	[N/mm2]

Safety factors

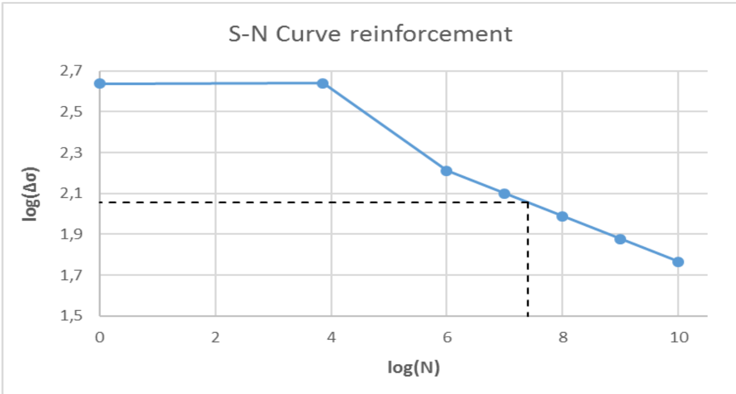
$\gamma_{F,fat}$	=	1,00	[-]	(NEN-EN 1992-1-1 NB)
$\gamma_{s,fat}$	=	1,15	[-]	(NEN-EN 1992-1-1 NB)

Correction factor λs

Factor for element type	=	$L = critical\ length\ influence\ line$	=	20,0	[m]	(OVS 00030-6)
	=	$\lambda_{s,1}(2\ m) = according\ to\ table\ NN.2$	=	0,90	[-]	
	=	$\lambda_{s,1}(20\ m) = according\ to\ table\ NN.2$	=	0,65	[-]	
	=	$\lambda_{s,1}(L) = \lambda_{s,1}(2\ m) + [\lambda_{s,1}(20\ m) - \lambda_{s,1}(2\ m)] * (\log L - 0{,}3)$	=	0,65	[-]	
Factor for volume	=	Vol	=	2,50E+07	[ton/year/tr]	
	=	slope of S-N line (table 6.3N NEN 1992-1-1), k_2	=	9,0	[-]	
	=	$\lambda_{s,2} = \sqrt[k_2]{\frac{Vol}{25 * 10^6}}$	=	1,0	[-]	
Factor for life	=	N_{years}	=	100	[years]	
	=	$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}}$	=	1,0	[-]	
Factor for more than one track	=	$\lambda_{s,4}$	=	1,0	[-]	
Damage equivalent factor	=	$\lambda_s = \lambda_{s,1} * \lambda_{s,2} * \lambda_{c,3} * \lambda_{s,4} =$	=	0,65	[-]	

S-N curve

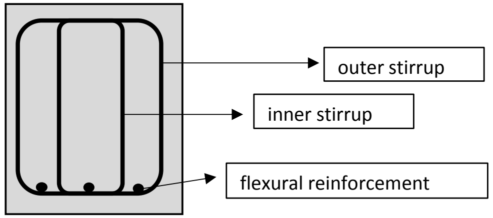
N^*	1,00E+06	[cycles]	(NEN-EN 1992-1-1)
k_1	5,0	[-]	(NEN-EN 1992-1-1)
k_2	9,0	[-]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N^*)$	162,5	[N/mm2]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N)$	113,6	[N/mm2]	(see graph)



Unity check

$U.C. = \gamma_{F,fat} * \Delta\sigma_{s,equ}(N) \leq \frac{\Delta\sigma_{risk}(N)}{\gamma_{s,fat}}$	=	0,50	<	1,00	OK

Inner stirrup fatigue verification (damage equivalent stress)



Damage equivalent stress

$\Phi * \Delta\sigma_{s,71}$	=	114,57	[N/mm2]
$\Delta\sigma_{s,equ}$	=	74,44	[N/mm2]

Safety factors

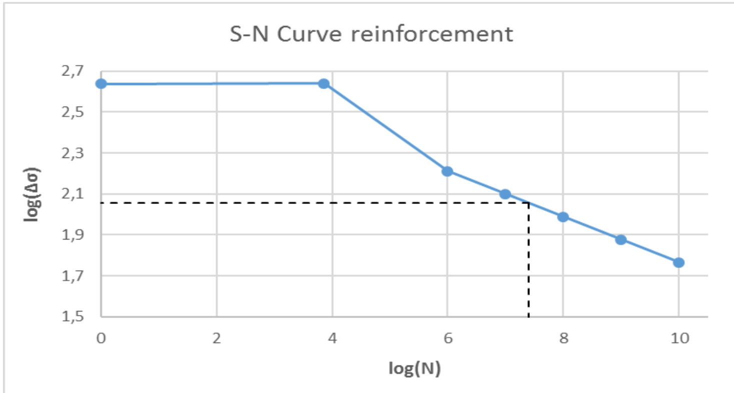
$\gamma_{F,fat}$	=	1,00	[-]	(NEN-EN 1992-1-1 NB)
$\gamma_{s,fat}$	=	1,15	[-]	(NEN-EN 1992-1-1 NB)

Correction factor λs

Factor for element type	=	$L = critical\ length\ influence\ line$	=	20,0	[m]	(OVS 00030-6)
	=	$\lambda_{s,1}(2\ m) = according\ to\ table\ NN.2$	=	0,90	[-]	
	=	$\lambda_{s,1}(20\ m) = according\ to\ table\ NN.2$	=	0,65	[-]	
	=	$\lambda_{s,1}(L) = \lambda_{s,1}(2\ m) + [\lambda_{s,1}(20\ m) - \lambda_{s,1}(2\ m)] * (\log L - 0{,}3)$	=	0,65	[-]	
Factor for volume	=	Vol	=	2,50E+07	[ton/year/tr]	
	=	slope of S-N line (table 6.3N NEN 1992-1-1), k_2	=	9,0	[-]	
	=	$\lambda_{s,2} = \sqrt[k_2]{\frac{Vol}{25 * 10^6}}$	=	1,0	[-]	
Factor for life	=	N_{years}	=	100	[years]	
	=	$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}}$	=	1,0	[-]	
Factor for more than one track	=	$\lambda_{s,4}$	=	1,0	[-]	
Damage equivalent factor	=	$\lambda_s = \lambda_{s,1} * \lambda_{s,2} * \lambda_{c,3} * \lambda_{s,4} =$	=	0,65	[-]	

S-N curve

N^*	1,00E+06	[cycles]	(NEN-EN 1992-1-1)
k_1	5,0	[-]	(NEN-EN 1992-1-1)
k_2	9,0	[-]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N^*)$	162,5	[N/mm2]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N)$	113,6	[N/mm2]	(see graph)



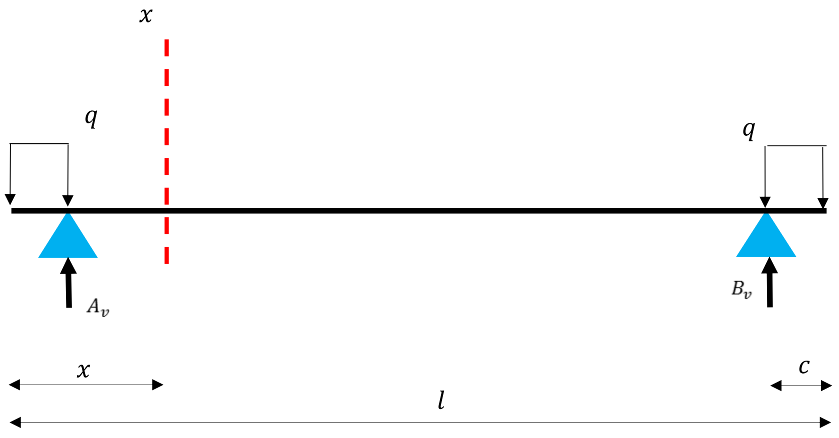
Unity check

$U.C. = \gamma_{F,fat} * \Delta\sigma_{s,equ}(N) \leq \frac{\Delta\sigma_{risk}(N)}{\gamma_{s,fat}}$	=	0,75	<	1,00	OK

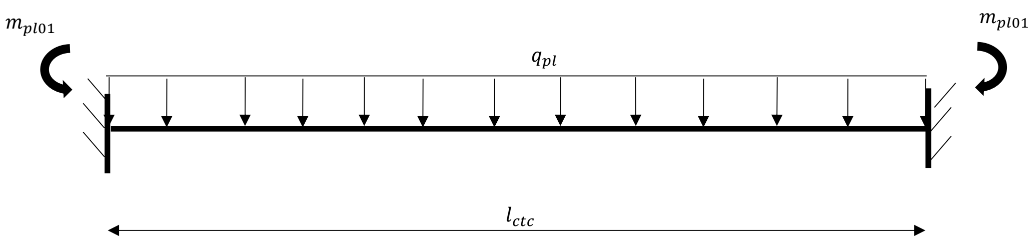
LC 1

Cantilevers loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces			
qbridge	138	[kN/m]	
0,5q	68,8	[kN/m]	
Av	68,8	[kN]	
Bv	68,8	[kN]	
γG	1,00	[-]	(NEN 1990 table A2.3)

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	34,4	[kN/m]
mpl01	45,8	[kNm]

Measurements		
lctc	4,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -34,4 [kNm]

Torsion

Mxy,M	-32,8	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	-32,8	[kNm]	

Suspension force

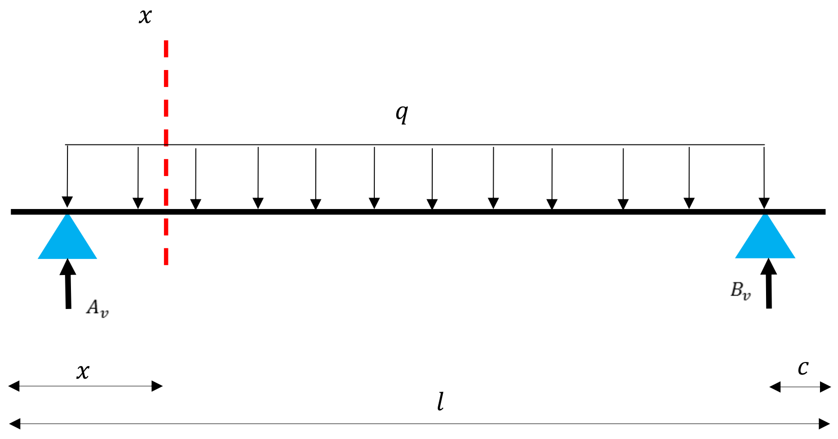
Qyy 0,0 [kN]

Clamping moment

mxx,M	8,1	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	8,1	[kNm]	

Midspan loaded

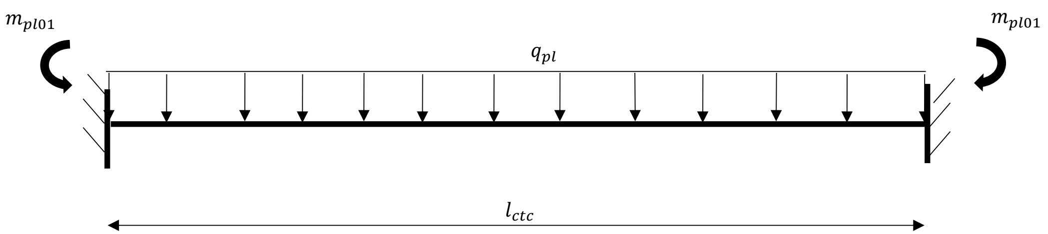
Loading (long. direction)



Load & Reaction forces		
qbridge	138	[kN/m]
0,5q	68,8	[kN/m]
Av	721,9	[kN]
Bv	721,9	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	34,4	[kN/m]
mpl01	45,8	[kNm]

Measurements		
lctc	4,0	[m]

Shear Force

Vz 628,4 [kN]

Bending moment

Mx 918,2 [kNm]

Torsion

Mxy, alt	-41,5	[kNm]	(due to alternative load case)
Mxy,subtot	-41,5	[kNm]	

Suspension force

Qyy 29,4 [kN]

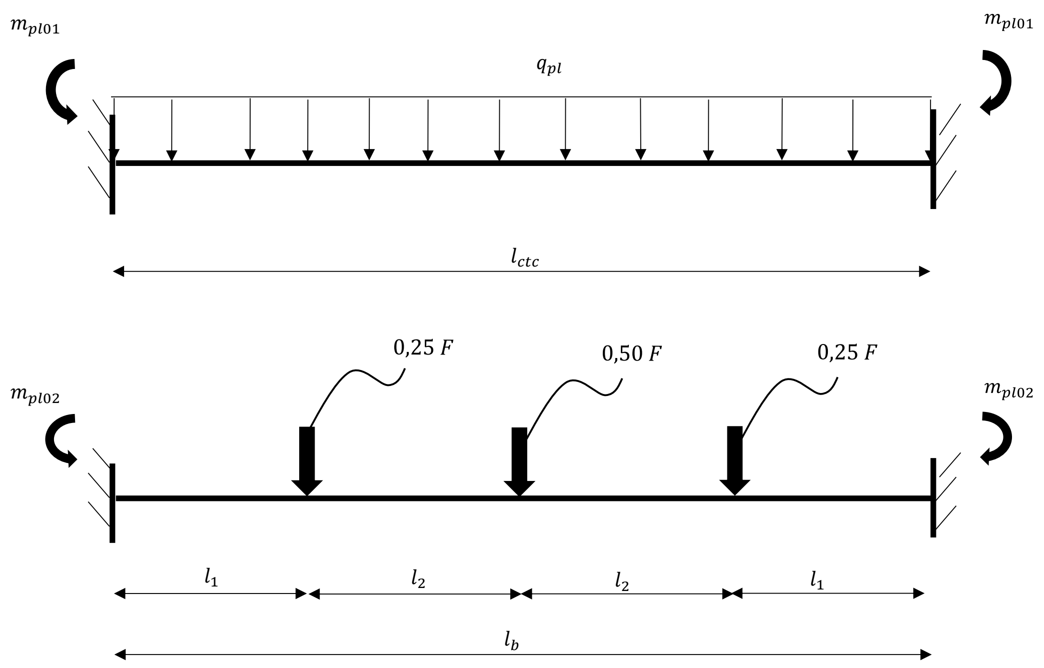
Clamping moment

mxx, alt	4,5	[kNm]	(due to alternative load case)
mxx,subtot.	4,5	[kNm]	

LC 2

Cantilevers loaded

Loading (long. direction)	Loading (transverse direction)



Load & Reaction forces		
q	11,7	[kN/m2]
0,5q	23,4	[kN/m]
F	111	[kN]
0,5F	55	[kN]
Av	78,8	[kN]
Bv	78,8	[kN]
γG	1,00	[-]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	11,7	[kN/m]
0,25F	27,7	[kN/m]
mpl01	15,6	[kNm]
mpl02	55,2	[kNm]
MT	70,8	[kNm]

Measurements		
lctc	4,0	[m]
l1	0,95	[m]
l2	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

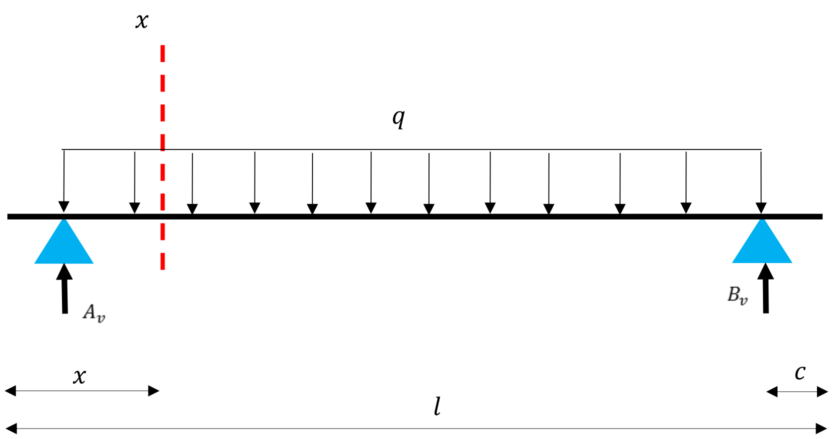
Bending moment		
Mx	-67,1	[kNm]

Torsion			
Mxy,M	-50,8	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	-50,8	[kNm]	

Clamping moment			
mxx,M	12,5	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	12,5	[kNm]	

Midspan loaded

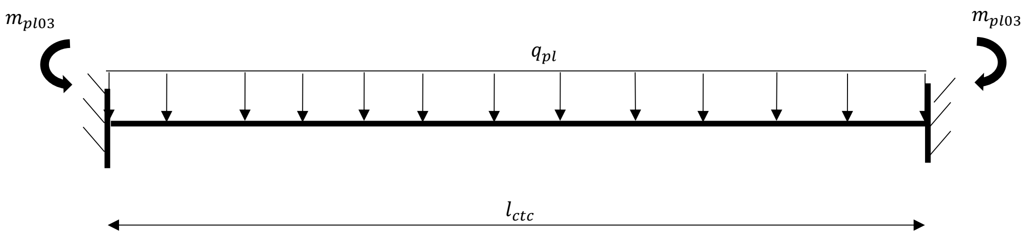
Loading (long. direction)



Load & Reaction forces		
q	11,7	[kN/m2]
0,5q	23,4	[kN/m]
Av	245,7	[kN]
Bv	245,7	[kN]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	11,7	[kN/m]
mpl02	15,6	[kNm]

Measurements		
lctc	4,0	[m]

Shear Force		
Vz	213,9	[kN]

Suspension force		
Qyy	23,4	[kN]

Bending moment

Mx	312,5	[kNm]

Torsion

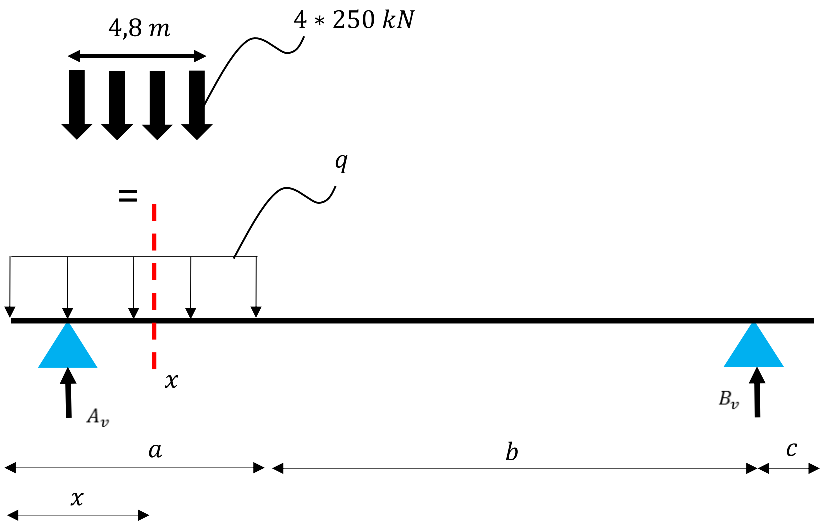
Mxy,alt	-14,2	[kNm]	(due to alternative load case)
Mxy,subtot	-14,2	[kNm]	

Clamping moment

mxx,alt	1,5	[kNm]	(due to alternative load case)
mxx,subtot.	1,5	[kNm]	

LC 3

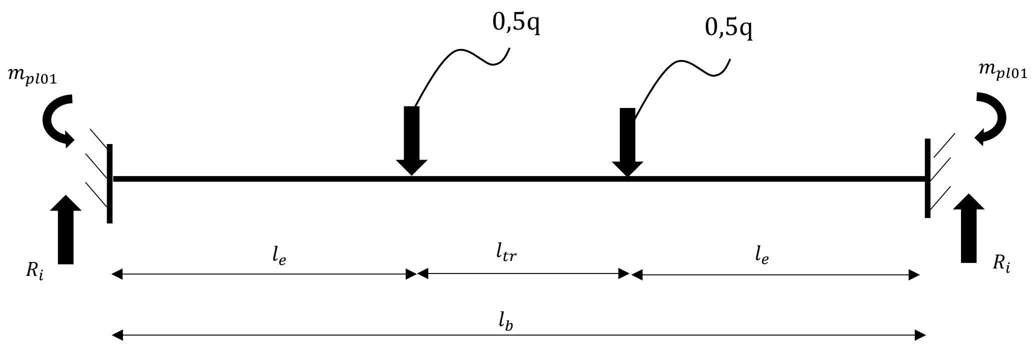
Loading (long. direction)



Load & Reaction forces		
q	164,6	[kN/m]
0,5q	82,3	[kN/m]
Av	501,7	[kN]
Bv	66,3	[kN]
γQ	1,00	[-]
α	1,00	[-]

Measurements		
x	2,36	[m]
a	6,9	[m]
b	15,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
0,5q	82,3	[kN/m]
mpl01	134,2	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 307,5 [kN]

Bending moment

Mx 453,1 [kNm]

Torsion

Mxy,M	-96,8	[kNm]	(due to torsional moment at one end)
Mxy,q	-23,0	[kNm]	(due to distributed load)
Mxy,tot	-119,8	[kNm]	

Suspension force

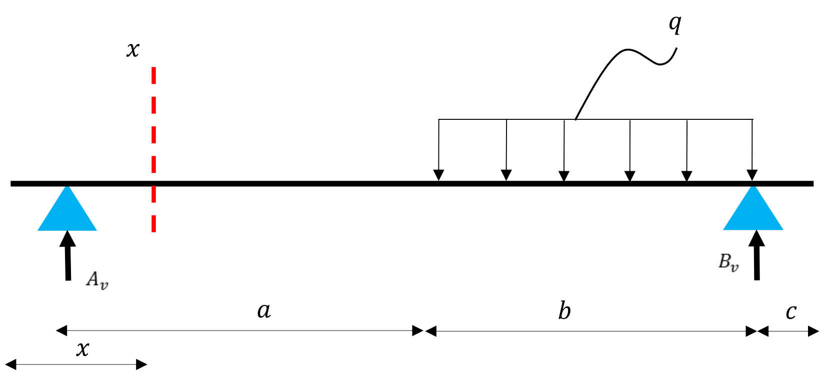
Qyy 82,3 [kN]

Clamping moment

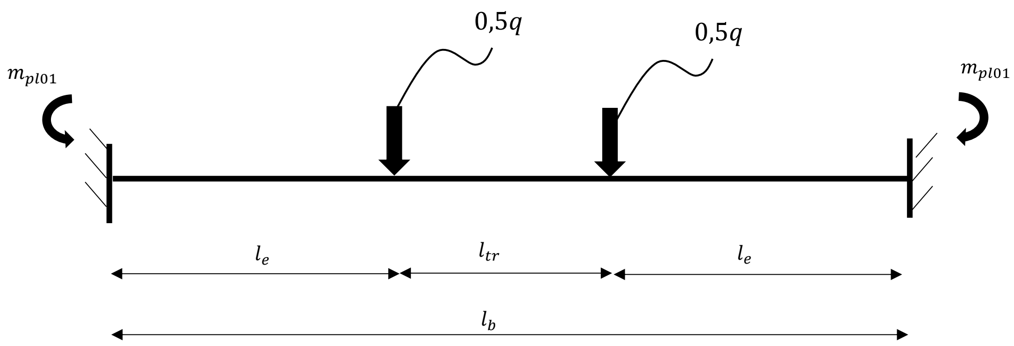
mxx,M	23,2	[kNm]	(due to torsional moment at one end)
mxx,q	-34,3	[kNm]	(due to distributed load)
mxx,tot	-11,1	[kNm]	

LC 5a

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces

q	91	[kN/m]
0,5q	45	[kN/m]
Av	246,7	[kN]
Bv	439,5	[kN]
γQ	1,00	[-]
α	1,00	[-]

Measurements

x	2,36	[m]
a	5,9	[m]
b	15,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces

0,5q	45,4	[kN/m]
mpl01	74,1	[kNm]

Measurements

le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 246,7 [kN]

Suspension force

Qyy 0,0 [kN]

Bending moment

Mx 335,5 [kNm]

Clamping moment

mxx,q	18,9	[kNm]
mxx,tot	18,9	[kNm]

(due to distributed load at one end)

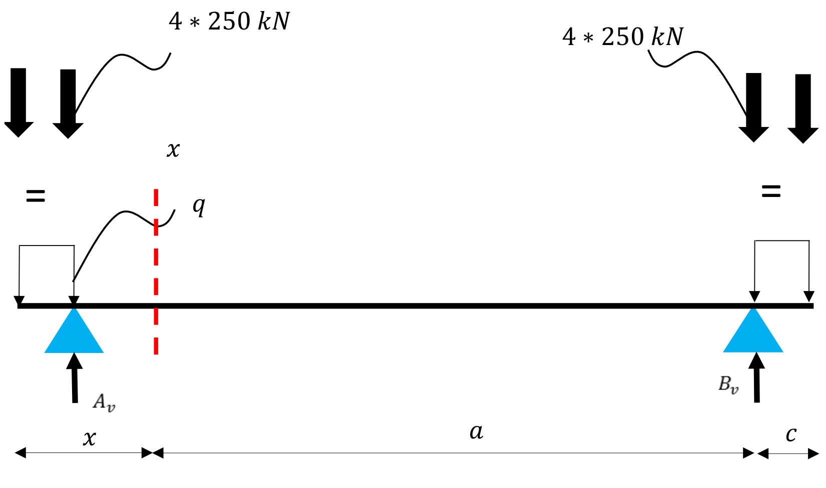
Torsion

Mxy,q	12,7	[kNm]
Mxy,tot	12,7	[kNm]

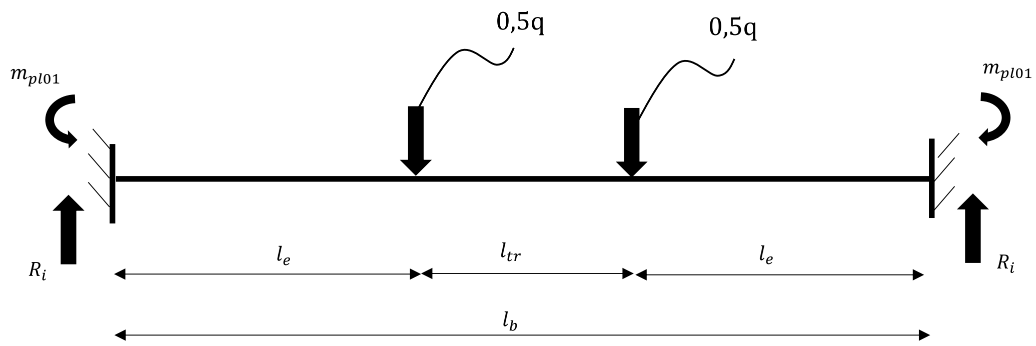
(due to distributed load at one end)

LC 5b

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	164,6	[kN/m]
0,5q	82,3	[kN/m]
Av	82,3	[kN]
Bv	82,3	[kN]
γQ	1,00	[-]
α	1,00	[-]

Measurements		
x	2,36	[m]
a	19,6	[m]
b	0,0	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
0,5q	82,3	[kN/m]
mpl01	134,2	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -41,2 [kNm]

Torsion

Mxy,M -96,2 [kNm] (due to torsion at both ends)

Mxy,tot -96,2 [kNm]

Suspension force

Qyy 0,0 [kN]

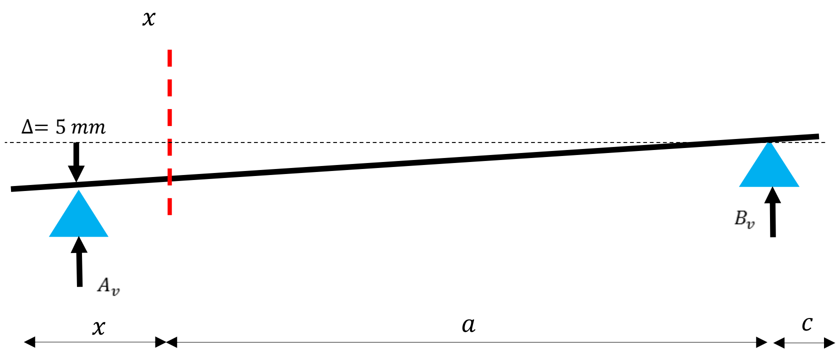
Clamping moment

mxx,M 23,7 [kNm] (due to torsion at both ends)

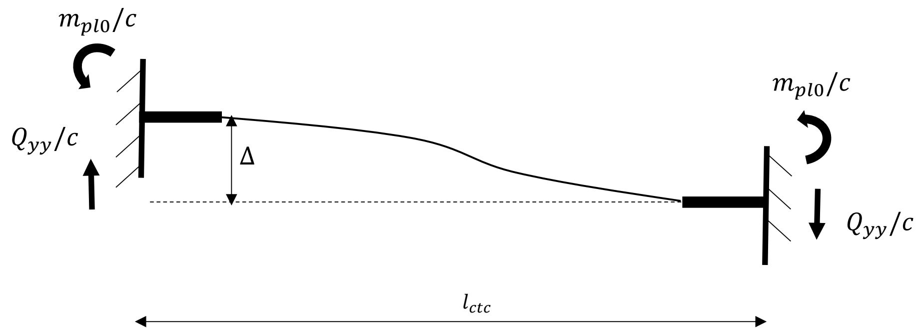
mxx,tot 23,7 [kNm]

LC 6

Loading (long. direction)



Loading (transverse direction)



Deflection & Reaction forces		
Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
yG	1,00	[-]

Measurements		
x	2,36	[m]
a	19,6	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Delfection & Reaction forces		
Δ	0,005	[m]

Shear Force

Vz	0,0	[kN]

Suspension force

Qyy	15,0	[kN]
Qyy,tot	15,0	[kN]

Bending moment

Mx	0,0	[kNm]

Clamping moment

mxx,Δ	82,0	[kNm]
mxx,tot	82,0	[kNm]

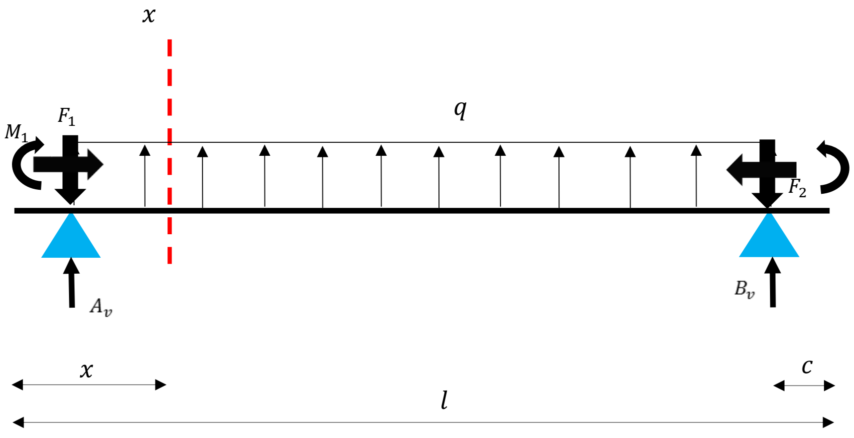
Torsion

Mxy,Δ	-178,0	[kNm]
Mxy,tot	-178,0	[kNm]

LC 8

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	1115	[kN]
F2	-13200	[kN]
M1	294	[kNm]
q	-101	[kN]
Av	55	[kN]
Bv	55	[kN]
yP	1,00	[-]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Shear Force

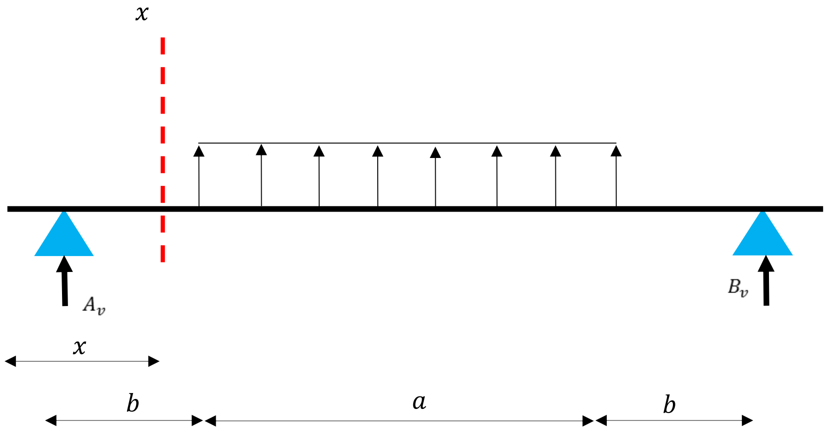
Vz	-923,1	[kN]

Bending moment

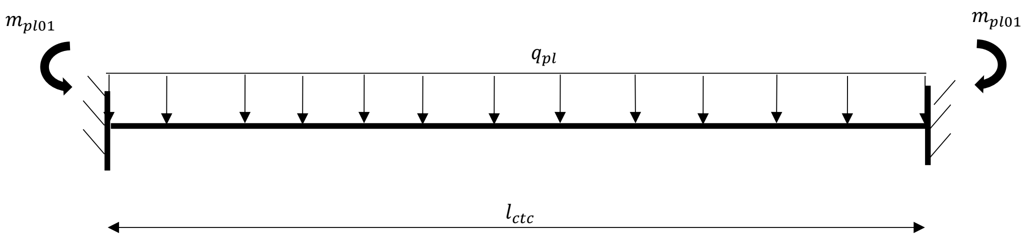
Mx	-1054,9	[kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	2,36	[m]
a	16,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	-51,5	[kN/m]
mpl01	-68,7	[kNm]

Measurements		
lctc	4,0	[m]

Torsion

Mxy,alt	37,3	[kNm]	(due to alternative load case)
Mxy,tot	37,3	[kNm]	

Suspension force

Qyy	-51,5	[kN]

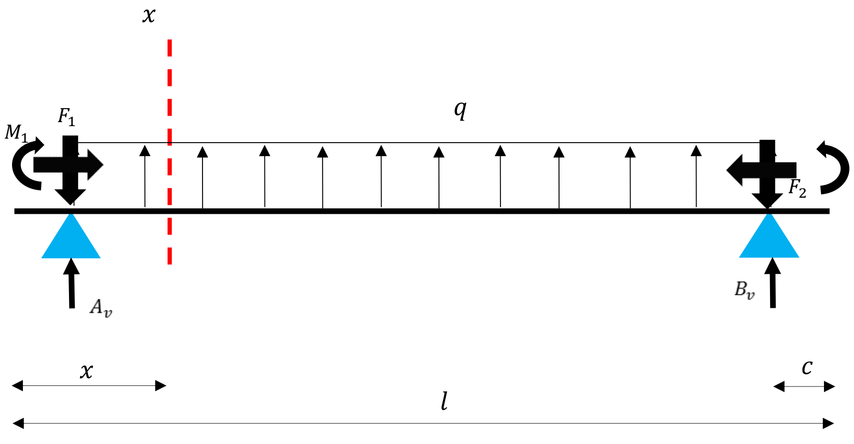
Clamping moment

mxx,alt	28,4	[kNm]	(due to alternative load case)
mxx,tot	28,4	[kNm]	

LC 9

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	1017	[kN]
F2	-12038	[kN]
M1	268	[kNm]
q	-92	[kN]
Av	50	[kN]
Bv	50	[kN]
P∞/P0	0,912	[-]
yP	1,00	[-]

Measurements		
x	2,36	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Shear Force

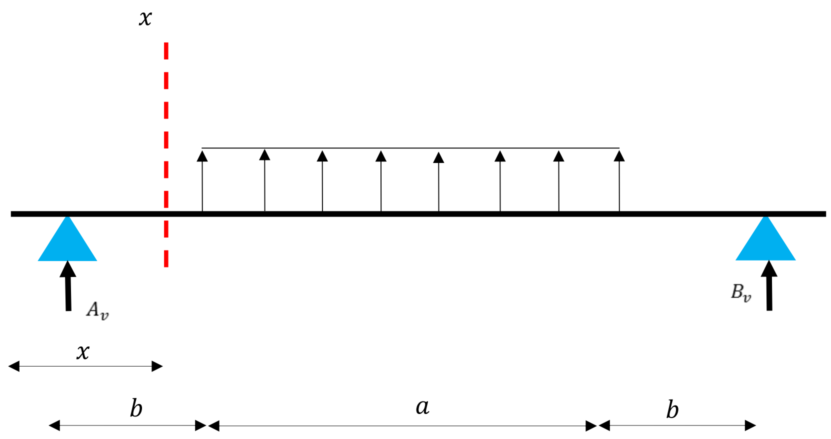
Vz	-841,9	[kN]

Bending moment

Mx	-962,0	[kNm]

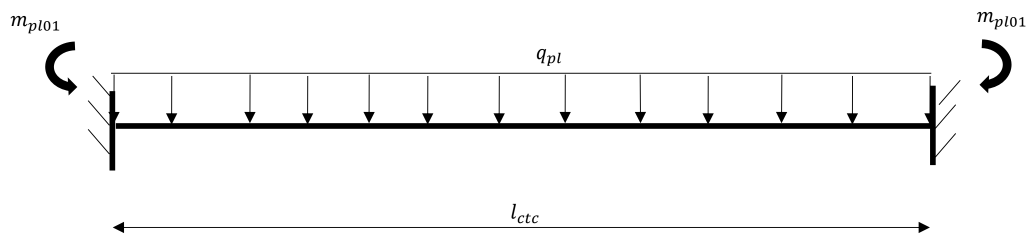
Floor loaded

Loading (long. direction)



Measurements		
x	2,36	[m]
a	16,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	-47,0	[kN/m]
mpl01	-62,6	[kNm]

Measurements		
lctc	4,0	[m]

Torsion

Mxy,alt	34,0	[kNm]	(due to alternative load case)
Mxy,tot	34,0	[kNm]	

Suspension force

Qyy	-47,0	[kN]

Clamping moment

mxx,alt	25,9	[kNm]	(due to alternative load case)
mxx,tot	25,9	[kNm]	

5.2 Fatigue: Bridge A – Characteristic loading – 0,5L

Forces

LC	type	Prestress P [kN]	Bending moment M [kNm]
1	self-weight		3755
2	ballast		1223
3	LM 71 max (1/2)		2492
5a	LM71 max (2/2)		1129
5b	LM71 min		-41
6	Support settelement max		0
7	Support settelement min		0
8	Prestress t=0	-13200	-5274
9	Prestress t = ∞	-12038	-4810
σperm (1+2+6+9)		-12038	169
σ71,min (1+2+5b+6+9)		-12038	128
σ71,max (1+2+3+5a+6+9)		-12038	3790

LC	type	Suspension force Qyy [kN]	Clamping moment mxx [kNm]	Shear force [kN]	Torsion [kNm]	Total shear force [kN]
1	self-weight	29	6	0	0	
2	ballast	23	4	0	0	
3	LM 71 max (1/2)	82	-38	0	0	
5a	LM71 max (2/2)	0	31	0	0	
5b	LM71 min	0	5	0	0	
6	Support settelement max	1	7	0	117	
7	Support settelement min	-1	-7	0	-117	
8	Prestress t=0	-52	9	0	0	
9	Prestress t = ∞	-47	8	0	0	
σperm (1+2+6+9)		7	26	0	117	160
σ71,min (1+2+5b+6+9)		7	31	0	117	170
σ71,max (1+2+3+5a+6+9)		89	19	0	117	274

Fatigue Parameters

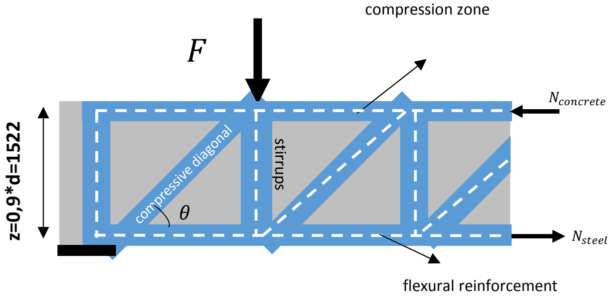
Geometry			
length L	=	21000	[mm]
height h	=	1750	[mm]
width b	=	900	[mm]
cover	=	35	[mm]
effective height	=	1691	[mm]
$0,5I_{yy}$	=	6,63E+11	[mm4]
y	=	628	[mm]
$0,5A$	=	2750000	[mm2]

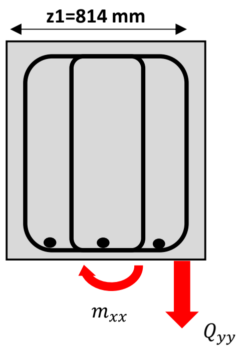
Reinforcement			
Øouter stirrup	=	16	[mm]
Øinner stirrup	=	12	[mm]
Øflexural reinf.	=	16	[mm]
spacing outer stirrup	=	150	[mm]
spacing inner stirrup	=	150	[mm]
n rebar flex. reinf.	=	14	[-]
$A_{outer\ stirrup}$	=	1,34	[mm2/mm]
$A_{inner\ stirrup}$	=	0,75	[mm2/mm]
$A_{longitudinal\ reinforcement}$	=	2815	[mm2]
A_p	=	1900	[mm2]

Internal lever arm			
$z = 0,9 * d$	=	1522	[mm]
$z_1 = b - 2 * c - 2 * 0,5 * Ø_{stp}$	=	814	[mm]

Angle compr. diagonal			
θfat	=	45,0	[◦]
θ	=	45,0	[◦]

Box girder properties			
$Area\ (A)$	=	1,575	[m2]
$Perimeter\ (u)$	=	5,30	[m]
$t_{ef} = A/u$	=	297	[mm]
$t_{ef,lower\ limit} = 2 * (h - d)$	=	118	[mm]
A_k	=	0,88	[m2]
u_k	=	4,11	[m]

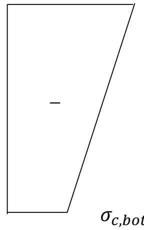




Stress range concrete

σperm (1+2+6+9)

M_{Ed}	=	169	[kNm]
$V_{Ed,tot} = V_{Ed} + (Q_{yy}+m_{xx}/z_1) * z * \cot\,\theta + \frac{T_{Ed} * z}{2 * A_k}$	=	160	[kN]
$W_{top} = I_{yy}/(h-y)$	=	5,91E+08	[mm3]
$W_{bottom} = I_{yy}/(y)$	=	1,06E+09	[mm3]
$\sigma_{c,top}$	=	-4,87	[N/mm2]
$\sigma_{c,bottom}$	=	-4,10	[N/mm2]

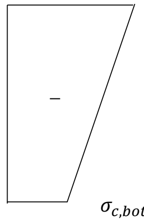


$$\sigma_{c,top} = -\frac{P}{A} - \frac{M_{Ed} + V_{Ed} * 0{,}5 * z * \cot\theta}{W_{top}}$$

$$\sigma_{c,bottom} = -\frac{P}{A} + \frac{M_{Ed} + V_{Ed} * 0{,}5 * z * \cot\theta}{W_{bottom}}$$

σ71,min (1+2+5b+6+9)

M_{Ed}	=	128	[kNm]
$V_{Ed,tot} = V_{Ed} + (Q_{yy}+m_{xx}/z_1) * z * \cot\,\theta + \frac{T_{Ed} * z}{2 * A_k}$	=	170	[kN]
$W_{top} = I_{yy}/(h-y)$	=	5,91E+08	[mm3]
$W_{bottom} = I_{yy}/(y)$	=	1,06E+09	[mm3]
$\sigma_{c,top}$	=	-4,81	[N/mm2]
$\sigma_{c,bottom}$	=	-4,13	[N/mm2]

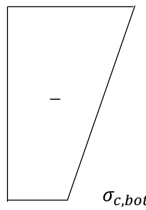


$$\sigma_{c,top} = -\frac{P}{A} - \frac{M_{Ed} + V_{Ed} * 0{,}5 * z * \cot\theta}{W_{top}}$$

$$\sigma_{c,bottom} = -\frac{P}{A} + \frac{M_{Ed} + V_{Ed} * 0{,}5 * z * \cot\theta}{W_{bottom}}$$

σ71,max (1+2+3+5a+6+9)

M_{Ed}	=	3790	[kNm]
$V_{Ed,tot} = V_{Ed} + (Q_{yy}+m_{xx}/z_1) * z * \cot\,\theta + \frac{T_{Ed} * z}{2 * A_k}$	=	274	[kN]
$W_{top} = I_{yy}/(h-y)$	=	5,91E+08	[mm3]
$W_{bottom} = I_{yy}/(y)$	=	1,06E+09	[mm3]
$\sigma_{c,top}$	=	-11,14	[N/mm2]
$\sigma_{c,bottom}$	=	-0,59	[N/mm2]



$$\sigma_{c,top} = -\frac{P}{A} - \frac{M_{Ed} + V_{Ed} * 0{,}5 * z * \cot\theta}{W_{top}}$$

$$\sigma_{c,bottom} = -\frac{P}{A} + \frac{M_{Ed} + V_{Ed} * 0{,}5 * z * \cot\theta}{W_{bottom}}$$

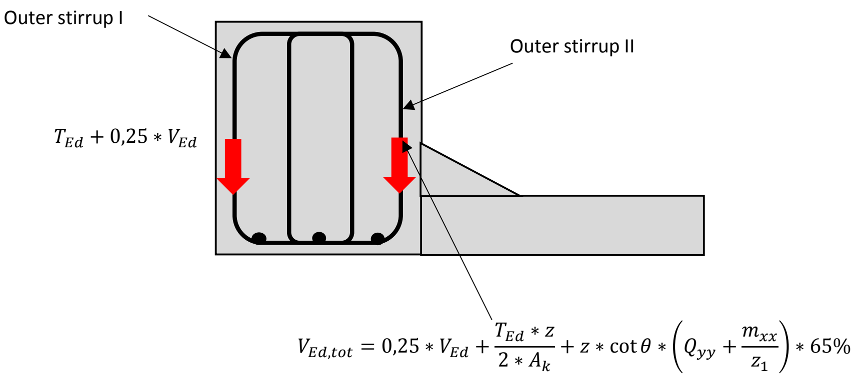
Stress range top fibre

Load combination	Total σc,t [N/mm2]
σperm (1+2+7+9)	-4,87
σ71,min (1+2+5b+7+9)	-4,81
σ71,max (1+2+3+5a+7+9)	-11,14

Stress range stirrups

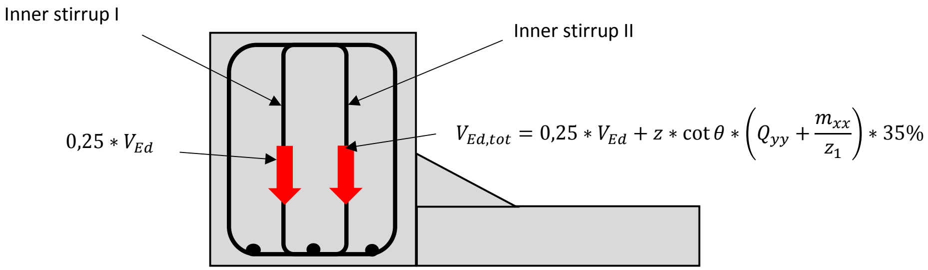
Stress range outer stirrup

		Total	
LC	type	VEd,tot [kN]	σstirrup [N/mm2]
1	self-weight		
2	ballast		
3	LM 71 max (1/2)		
5a	LM71 max (2/2)		
5b	LM71 min		
6	Support settelement max		
7	Support settelement min		
8	Prestress t=0		
9	Prestress t = ∞		
σperm (1+2+6+9)		140	68
σ71,min (1+2+5b+6+9)		146	72
σ71,max (1+2+3+5a+6+9)		213	105



Stress range inner stirrup

		Total	
LC	type	VEd,tot [kN]	σstirrup [N/mm2]
1	self-weight		
2	ballast		
3	LM 71 max (1/2)		
5a	LM71 max (2/2)		
5b	LM71 min		
6	Support settelement max		
7	Support settelement min		
8	Prestress t=0		
9	Prestress t = ∞		
σperm (1+2+6+9)		20	18
σ71,min (1+2+5b+6+9)		24	21
σ71,max (1+2+3+5a+6+9)		60	52



Concrete Fatigue verification (damage equivalent stress)

Fatigue strength

k_1	=	0,85	[-]	(NEN-EN 1992-1-1 NB)
$\beta_{cc}(t_0)$	=	1,00	[-]	
f_{ck}	=	35	[N/mm2]	(RBK)
$\gamma_{c,fat}$	=	1,50	[-]	(NEN-EN 1992-1-1 NB)
f_{cd}	=	23,3	[N/mm2]	
$f_{cd,fat}$	=	18,1	[N/mm2]	

Concrete stress

γ_{sd}	=	1,00	[-]
$\sigma_{c,perm}$	=	4,87	[N/mm2]
$\sigma_{c,max,71}$	=	11,14	[N/mm2]
$\sigma_{c,min,71}$	=	4,81	[N/mm2]
$\sigma_{cd,max,equ}$	=	9,57	[N/mm2]
$\sigma_{cd,min,equ}$	=	4,83	[N/mm2]
$E_{cd,min,equ}$	=	0,27	[-]
$E_{cd,max,equ}$	=	0,53	[-]

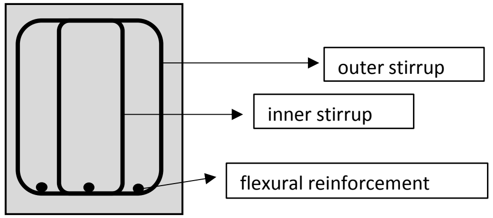
Unity check

$R_{equ} = \frac{E_{cd,min,equ}}{E_{cd,max,equ}}$	=	0,50 [-]			
$14 * \frac{1 - E_{cd,max,equ}}{\sqrt{1 - R_{equ}}} \geq 6$	=	9,36	>	6	OK

Correction factor λc

Factor for permanent stress	=	$\lambda_{c,0} = 0{,}94 + 0{,}2 * \sigma_{c,perm}/f_{cd,fat} \geq 1$	=	1,00 [-]
Factor for element type	=	$L = critical\ length\ influence\ line$	=	20,0 [m]
	=	$\lambda_{c,1}(2\ m) = according\ to\ table\ NN.3$	=	0,70 [-]
	=	$\lambda_{c,1}(20\ m) = according\ to\ table\ NN.3$	=	0,75 [-]
	=	$\lambda_{c,1}(L) = \lambda_{c,1}(2\ m) + [\lambda_{c,1}(20\ m) - \lambda_{c,1}(2\ m)] * (\log L - 0{,}3)$	=	0,75 [-]
Factor for volume and life	=	Vol	=	2,50E+07 [ton/year/tr]
	=	N_{years}	=	100 [year]
	=	$\lambda_{c,2,3} = 1 + \frac{1}{8} * \log\left(\frac{Vol}{25 * 10^6}\right) + \frac{1}{8} * \log\left(\frac{N_{years}}{100}\right)$	=	1,00 [-]
Factor for more than one track	=	$\lambda_{c,4} =$	=	1,00 [-]
Damage equivalent factor	=	$\lambda_c = \lambda_{c,0} * \lambda_{c,1} * \lambda_{c,2,3} * \lambda_{c,4} =$	=	0,75 [-]

Outer stirrup fatigue verification (damage equivalent stress)



Damage equivalent stress

$\Phi * \Delta\sigma_{s,71}$	=	33,07	[N/mm2]
$\Delta\sigma_{s,equ}$	=	21,49	[N/mm2]

Safety factors

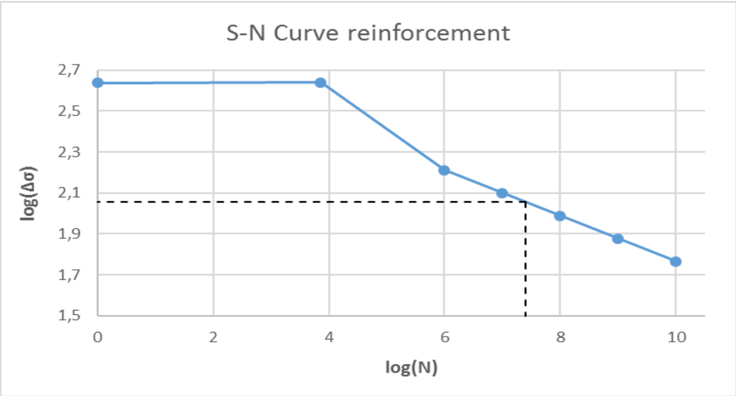
$\gamma_{F,fat}$	=	1,00	[-]	(NEN-EN 1992-1-1 NB)
$\gamma_{s,fat}$	=	1,15	[-]	(NEN-EN 1992-1-1 NB)

Correction factor λs

Factor for element type	=	$L = critical\ length\ influence\ line$	=	20,0	[m]	(OVS 00030-6)
	=	$\lambda_{s,1}(2\ m) = according\ to\ table\ NN.2$	=	0,90	[-]	
	=	$\lambda_{s,1}(20\ m) = according\ to\ table\ NN.2$	=	0,65	[-]	
	=	$\lambda_{s,1}(L) = \lambda_{s,1}(2\ m) + [\lambda_{s,1}(20\ m) - \lambda_{s,1}(2\ m)] * (\log L - 0{,}3)$	=	0,65	[-]	
Factor for volume	=	Vol	=	2,50E+07	[ton/year/tr]	
	=	slope of S-N line (table 6.3N NEN 1992-1-1), k_2	=	9,0	[-]	
	=	$\lambda_{s,2} = \sqrt[k_2]{\frac{Vol}{25 * 10^6}}$	=	1,0	[-]	
Factor for life	=	N_{years}	=	100	[years]	
	=	$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}}$	=	1,0	[-]	
Factor for more than one track	=	$\lambda_{s,4}$	=	1,0	[-]	
Damage equivalent factor	=	$\lambda_s = \lambda_{s,1} * \lambda_{s,2} * \lambda_{c,3} * \lambda_{s,4} =$	=	0,65	[-]	

S-N curve

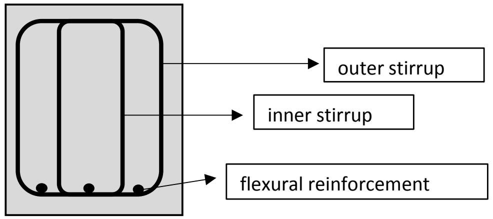
N^*	1,00E+06	[cycles]	(NEN-EN 1992-1-1)
k_1	5,0	[-]	(NEN-EN 1992-1-1)
k_2	9,0	[-]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N^*)$	162,5	[N/mm2]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N)$	113,6	[N/mm2]	(see graph)



Unity check

$$U.C. = \gamma_{F,fat} * \Delta\sigma_{s,equ}(N) \leq \frac{\Delta\sigma_{risk}(N)}{\gamma_{s,fat}}$$ = 0,22 < 1,00 OK

Inner stirrup fatigue verification (damage equivalent stress)



Damage equivalent stress

$\Phi * \Delta\sigma_{s,71}$	=	31,65 [N/mm2]
$\Delta\sigma_{s,equ}$	=	20,57 [N/mm2]

Safety factors

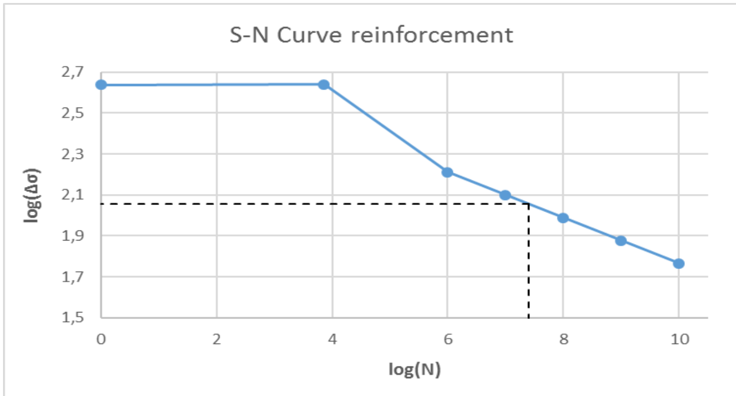
$\gamma_{F,fat}$	=	1,00 [-]	(NEN-EN 1992-1-1 NB)
$\gamma_{s,fat}$	=	1,15 [-]	(NEN-EN 1992-1-1 NB)

Correction factor λs

Factor for element type	=	$L = critical\ length\ influence\ line$	=	20,0 [m]	(OVS 00030-6)
	=	$\lambda_{s,1}(2\ m) = according\ to\ table\ NN.2$	=	0,90 [-]	
	=	$\lambda_{s,1}(20\ m) = according\ to\ table\ NN.2$	=	0,65 [-]	
	=	$\lambda_{s,1}(L) = \lambda_{s,1}(2\ m) + [\lambda_{s,1}(20\ m) - \lambda_{s,1}(2\ m)] * (\log L - 0{,}3)$	=	0,65 [-]	
Factor for volume	=	Vol	=	2,50E+07 [ton/year/tr]	
	=	slope of S-N line (table 6.3N NEN 1992-1-1), k_2	=	9,0 [-]	
	=	$\lambda_{s,2} = \sqrt[k_2]{\frac{Vol}{25 * 10^6}}$	=	1,0 [-]	
Factor for life	=	N_{years}	=	100 [years]	
	=	$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}}$	=	1,0 [-]	
Factor for more than one track	=	$\lambda_{s,4}$	=	1,0 [-]	
Damage equivalent factor	=	$\lambda_s = \lambda_{s,1} * \lambda_{s,2} * \lambda_{c,3} * \lambda_{s,4} =$	=	0,65 [-]	

S-N curve

N^*	1,00E+06 [cycles]	(NEN-EN 1992-1-1)
k_1	5,0 [-]	(NEN-EN 1992-1-1)
k_2	9,0 [-]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N^*)$	162,5 [N/mm2]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N)$	113,6 [N/mm2]	(see graph)



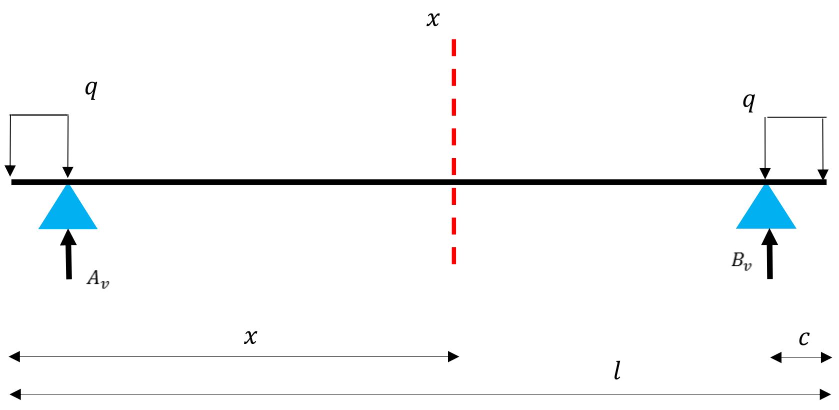
Unity check

$U.C. = \gamma_{F,fat} * \Delta\sigma_{s,equ}(N) \leq \frac{\Delta\sigma_{risk}(N)}{\gamma_{s,fat}}$	=	0,21	<	1,00	OK

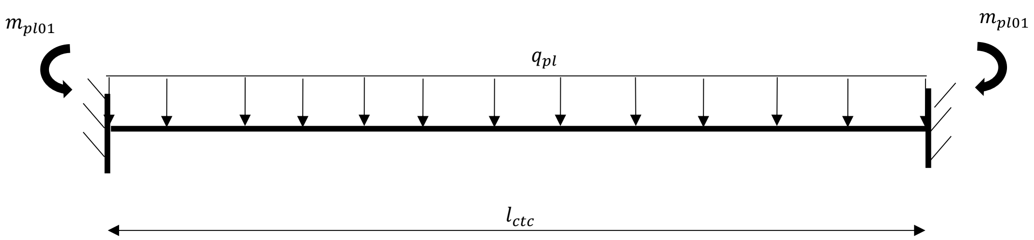
LC 1

Cantilevers loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces			
qbridge	138	[kN/m]	
0,5q	68,8	[kN/m]	
Av	68,8	[kN]	
Bv	68,8	[kN]	
γG	1,00	[-]	(NEN 1990 table A2.3)

Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	34,4	[kN/m]
mpl01	45,8	[kNm]

Measurements		
lctc	4,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -34,4 [kNm]

Torsion

Mxy,M	0,0	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	0,0	[kNm]	

Suspension force

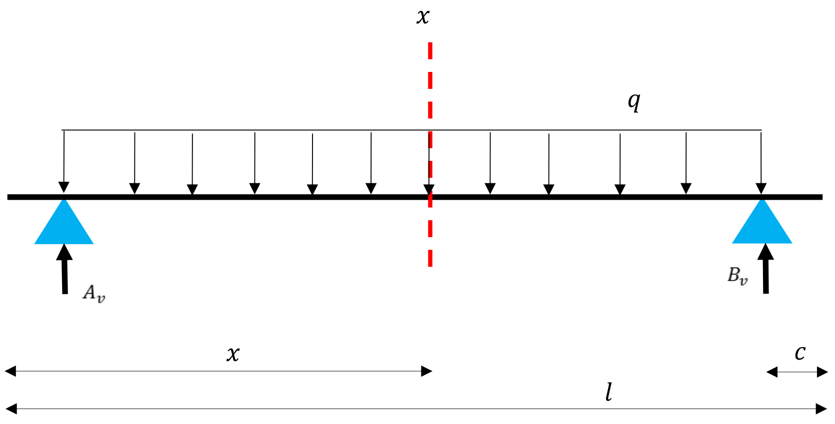
Qyy 0,0 [kN]

Clamping moment

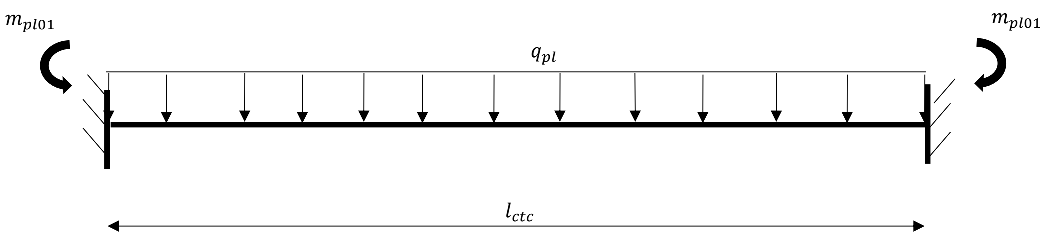
mxx,M	1,8	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	1,8	[kNm]	

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
qbridge	138	[kN/m]
0,5q	68,8	[kN/m]
Av	721,9	[kN]
Bv	721,9	[kN]

Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	34,4	[kN/m]
mpl01	45,8	[kNm]

Measurement		
lctc	4,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx 3789,8 [kNm]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,subtot	0,0	[kNm]	

Suspension force

Qyy 29,4 [kN]

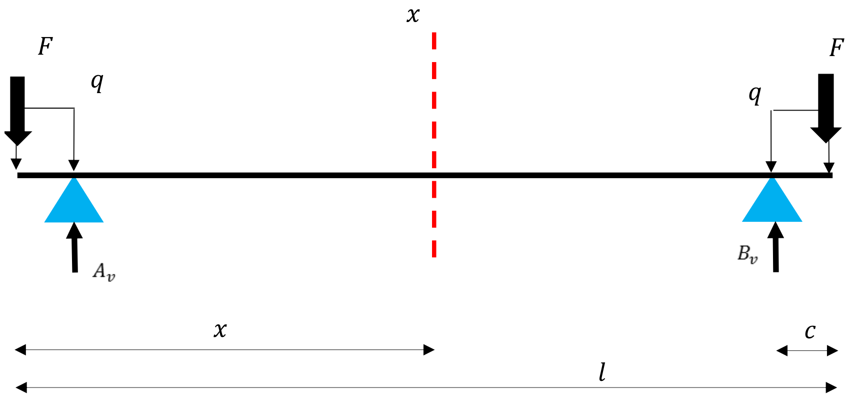
Clamping moment

mxx,alt	4,5	[kNm]	(due to alternative load case)
mxx,subtot.	4,5	[kNm]	

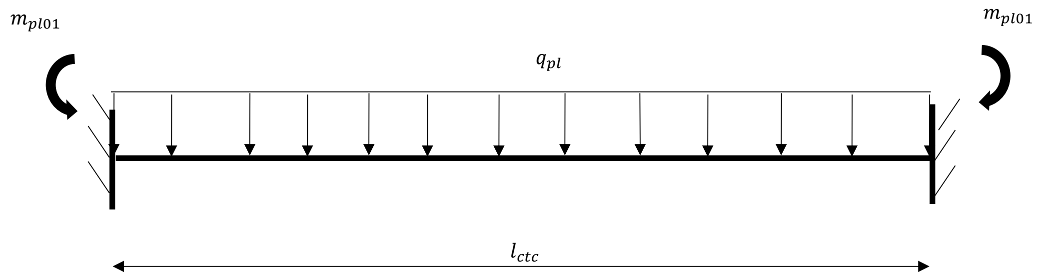
LC 2

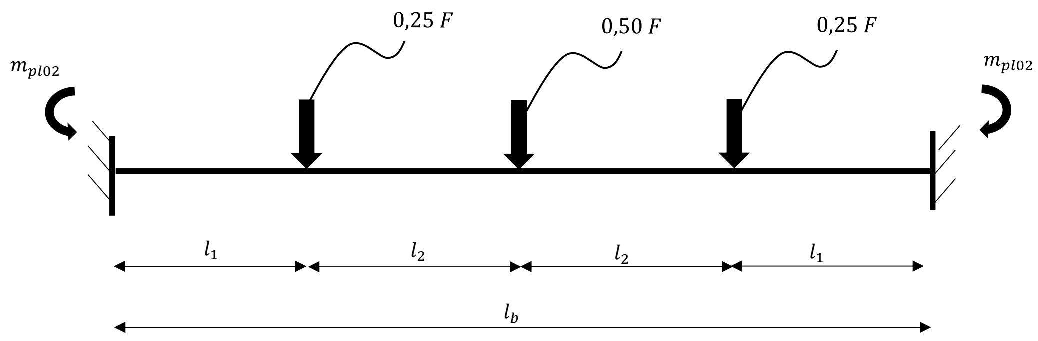
Cantilevers loaded

Loading (long. direction)



Loading (transverse direction)





Load & Reaction forces		
q	11,7	[kN/m2]
0,5q	23,4	[kN/m]
F	111	[kN]
0,5F	55	[kN]
Av	78,8	[kN]
Bv	78,8	[kN]
γG	1,00	[-]

Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	11,7	[kN/m]
0,25F	27,7	[kN/m]
mpl01	15,6	[kNm]
mpl02	55,2	[kNm]
MT	70,8	[kNm]

Measurements		
lctc	4,0	[m]
l1	0,95	[m]
l2	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

Bending moment

Mx	-67,1	[kNm]

Torsion

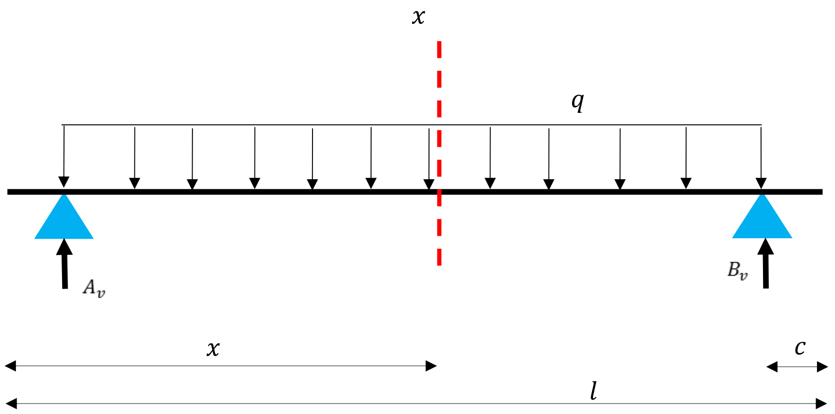
Mxy,M	0,0	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	0,0	[kNm]	

Clamping moment

mxx,M	2,8	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	2,8	[kNm]	

Midspan loaded

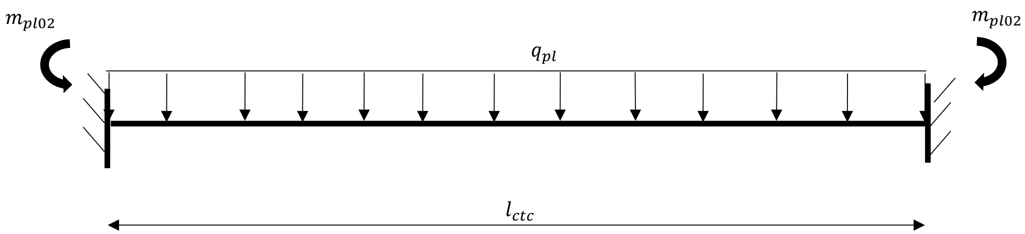
Loading (long. direction)



Load & Reaction forces		
q	11,7	[kN/m2]
0,5q	23,4	[kN/m]
Av	245,7	[kN]
Bv	245,7	[kN]

Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	11,7	[kN/m]
mpl03	15,6	[kNm]

Measurements		
lctc	4,0	[m]

Shear Force

Vz	0,0	[kN]

Suspension force

Qyy	23,4	[kN]

Bending moment

Mx	1289,9	[kNm]

Torsion

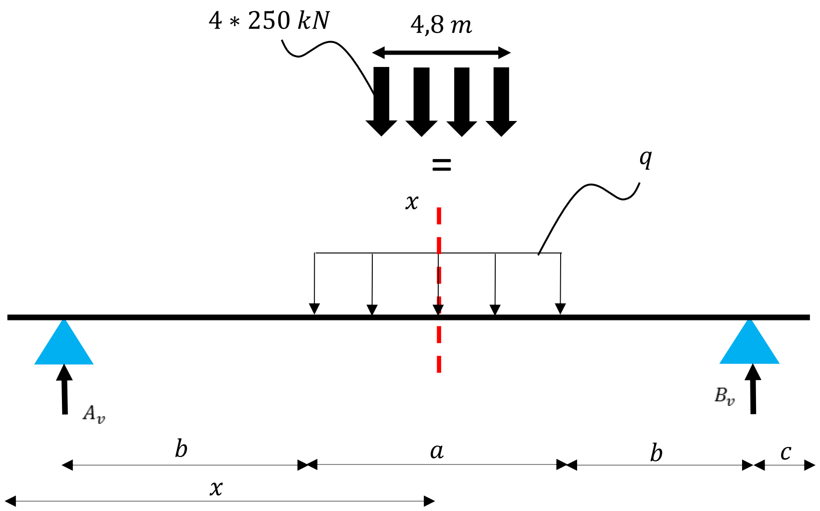
Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,subtot	0,0	[kNm]	

Clamping moment

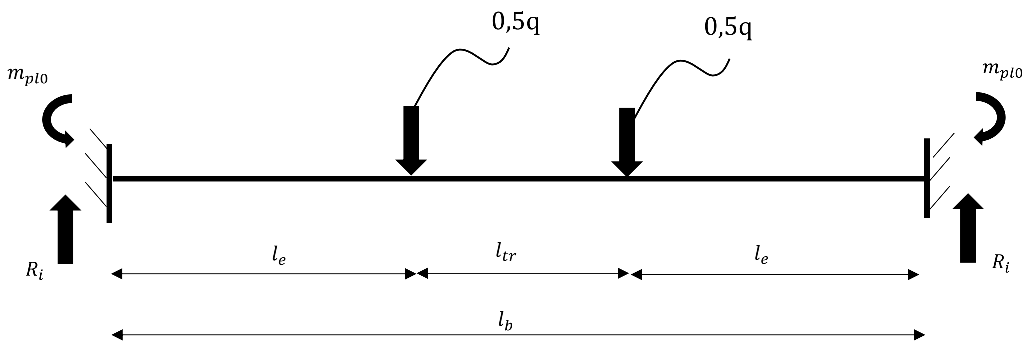
mxx, alt	1,5	[kNm]	(due to alternative load case)
mxx,subtot.	1,5	[kNm]	

LC 3

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces

q	164,6	[kN/m]
0,5q	82,3	[kN/m]
Av	284,0	[kN]
Bv	284,0	[kN]
γQ	1,00	[-]
α	1,00	[-]

Measurements

x	11,50	[m]
a	6,9	[m]
b	7,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces

p	82,3	[kN/m]
mpl0	134,2	[kNm]

Measurements

le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 82,3 [kN]

Bending moment

Mx 2492,1 [kNm]

Clamping moment

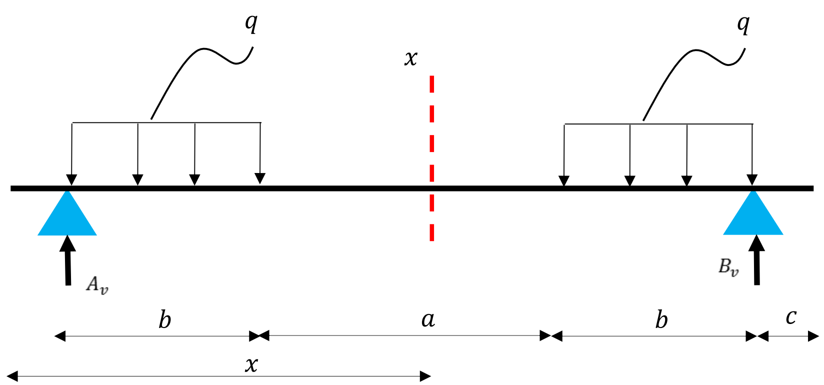
mxx	-37,7	[kNm]	(due to concentrated loads at midspan)
mxx,tot	-37,7	[kNm]	

Torsion

Mxy	0,0	[kNm]	(due to concentrated loads at midspan)
Mxy,tot	0,0	[kNm]	

LC 5a

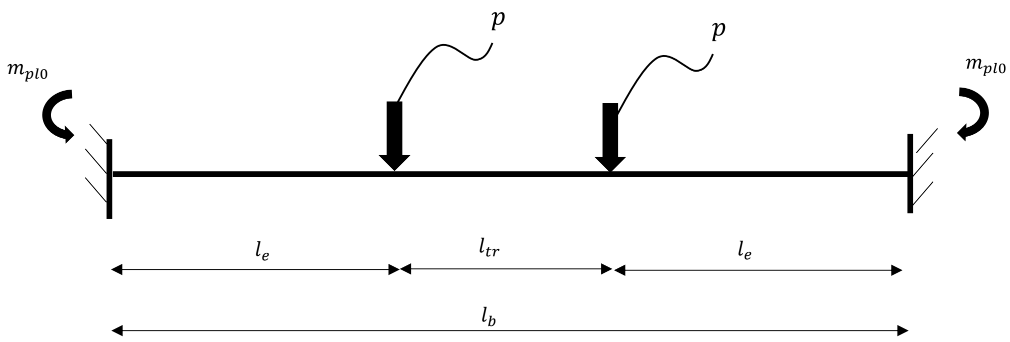
Loading (long. direction)



Load & Reaction forces		
q	91	[kN/m]
0,5q	45	[kN/m]
Av	320,4	[kN]
Bv	320,4	[kN]
γQ	1,00	[-]
α	1,00	[-]

Measurements		
x	11,50	[m]
a	6,9	[m]
b	7,1	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces		
p	45,4	[kN/m]
mpl0	74,1	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx 1129,2 [kNm]

Torsion

Mxy,q	0,0	[kNm]	(due to distributed load)
Mxy,q	0,0	[kNm]	(due to distributed load)
Mxy,tot	0,0	[kNm]	

Suspension force

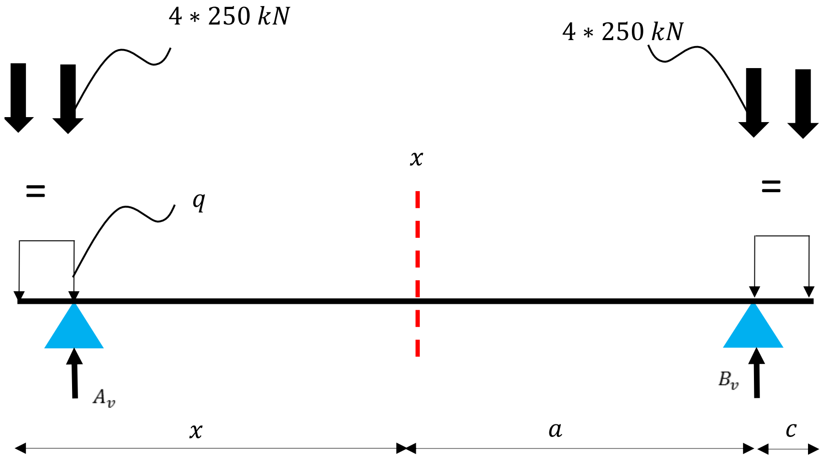
Qyy 0,0 [kN]

Clamping moment

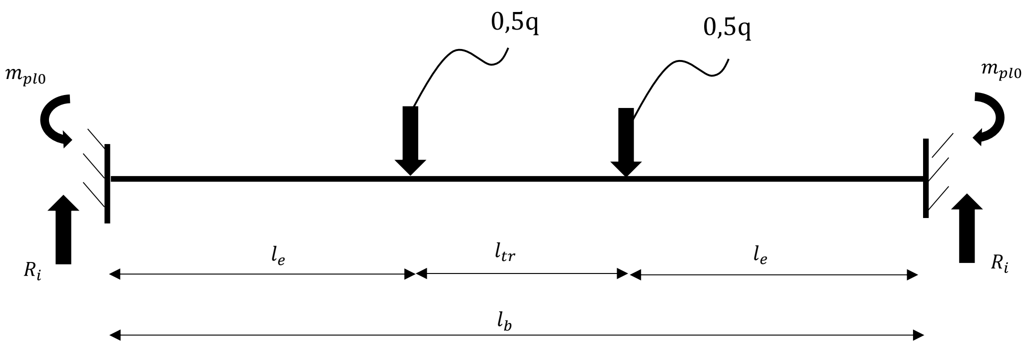
mxx,q	15,7	[kNm]	(due to distributed load)
mxx,q	15,7	[kNm]	(due to distributed load)
mxx,tot	31,4	[kNm]	

LC 5b

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	164,6	[kN/m]
0,5q	82,3	[kN/m]
Av	82,3	[kN]
Bv	82,3	[kN]
γQ	1,00	[-]
α	1,00	[-]

Measurements		
x	11,50	[m]
a	10,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
p	82,3	[kN/m]
mpl0	134,2	[kNm]

Measurements		
le	1,7	[m]
ltr	1,5	[m]
lb	4,9	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -41,2 [kNm]

Torsion

Mxy,M	0,0	[kNm]	(due to torsional moment at both sides)
Mxy,tot	0,0	[kNm]	

Suspension force

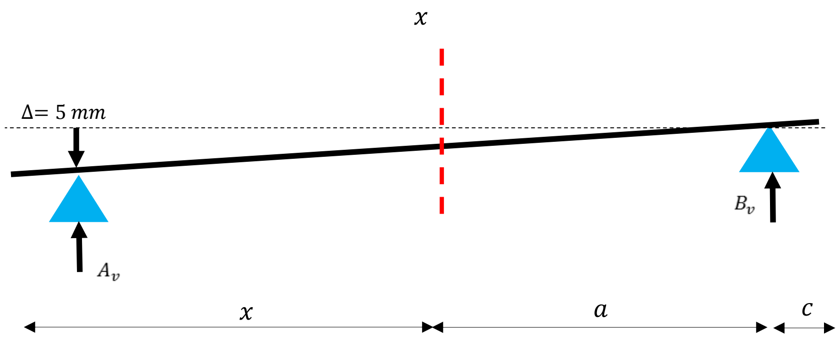
Qyy 0,0 [kN]

Clamping moment

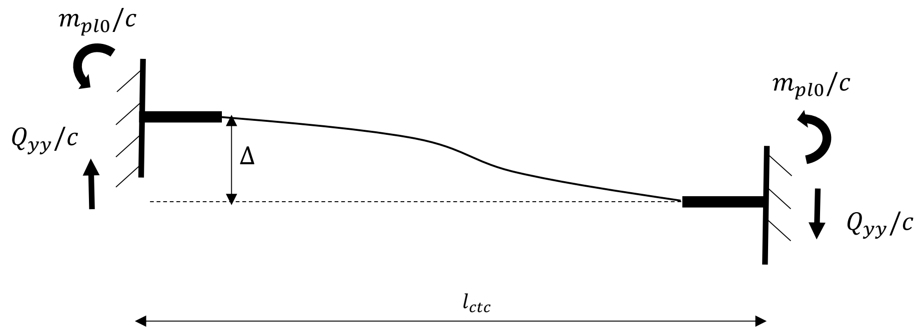
mxx,M	5,2	[kNm]	(due to torsional moments at both sides)
mxx,tot	5,2	[kNm]	

LC 6

Loading (long. direction)



Loading (transverse direction)



Deflection & Reaction forces

Δ	-5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
yG	1,00	[-]

Measurements

x	11,50	[m]
a	10,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Delfection

Δ	0,005	[m]

Shear Force

Vz	0,0	[kN]

Suspension force

Qyy	1,0	[kN]
Qyyt,tot	1,0	[kN]

Bending moment

Mx	0,0	[kNm]

Clamping moment

mxx,Δ	7,0	[kNm]
mxx,tot	7,0	[kNm]

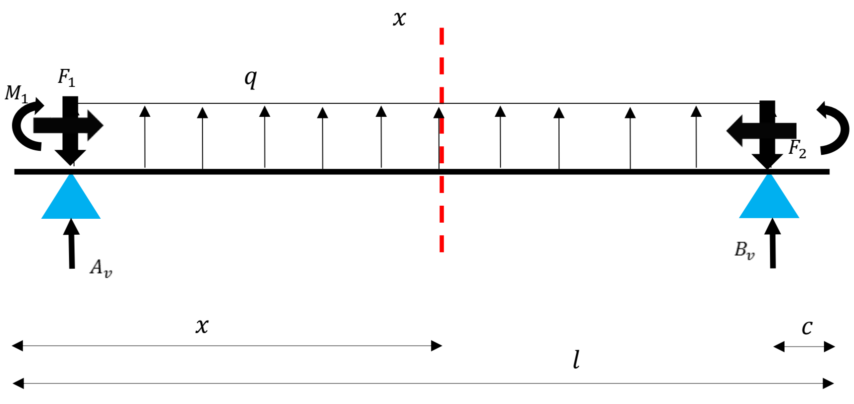
Torsion

Mxy,Δ	117,0	[kNm]
Mxy,tot	117,0	[kNm]

LC 8

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	1115	[kN]
F2	-13200	[kN]
M1	294	[kNm]
q	-101	[kN]
Av	55	[kN]
Bv	55	[kN]
yP	1,00	[-]

Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Shear Force

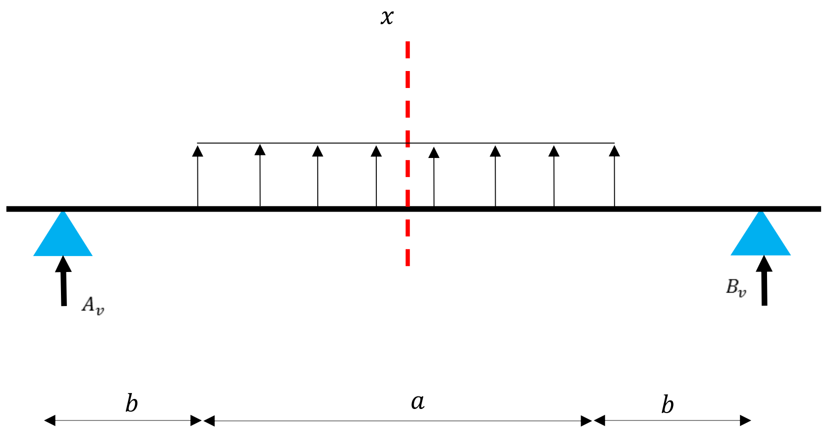
Vz	0,0	[kN]

Bending moment

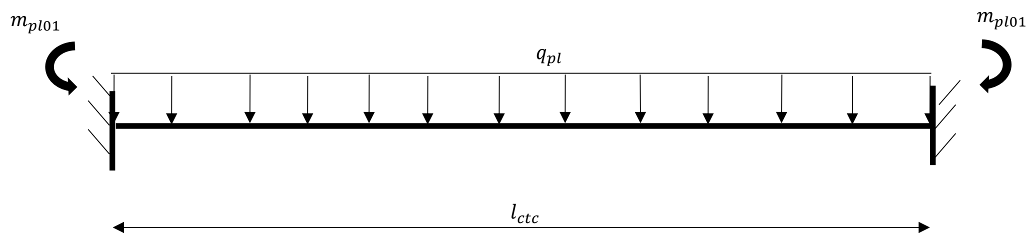
Mx	-5273,6	[kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	0,00	[m]
a	16,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Load & Reaction forces		
qpl	-51,5	[kN/m]
mpl01	-68,7	[kNm]

Measurements		
lctc	4,0	[m]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,tot	0,0	[kNm]	

Suspension force

Qyy	-51,5	[kN]

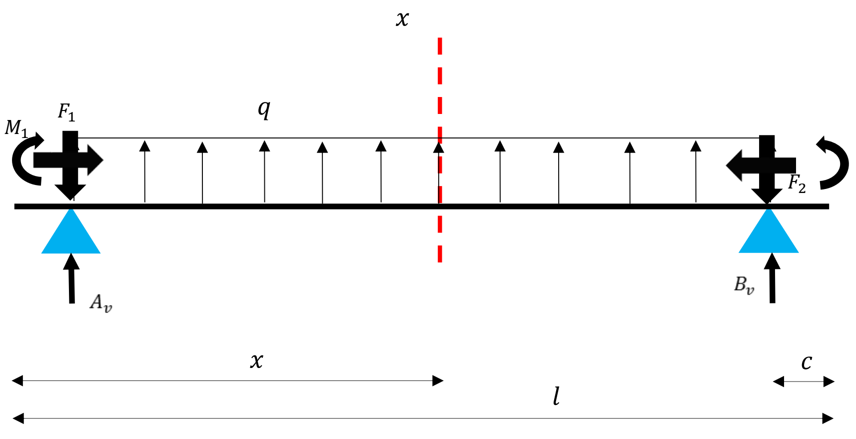
Clamping moment

mxx,alt	8,9	[kNm]	(due to alternative load case)
mxx,tot	8,9	[kNm]	

LC 9

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	1017	[kN]
F2	-12038	[kN]
M1	268	[kNm]
q	-92	[kN]
Av	50	[kN]
Bv	50	[kN]
P∞/P0	0,912	[-]
yP	1,00	[-]

Measurements		
x	11,50	[m]
c	1,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Shear Force

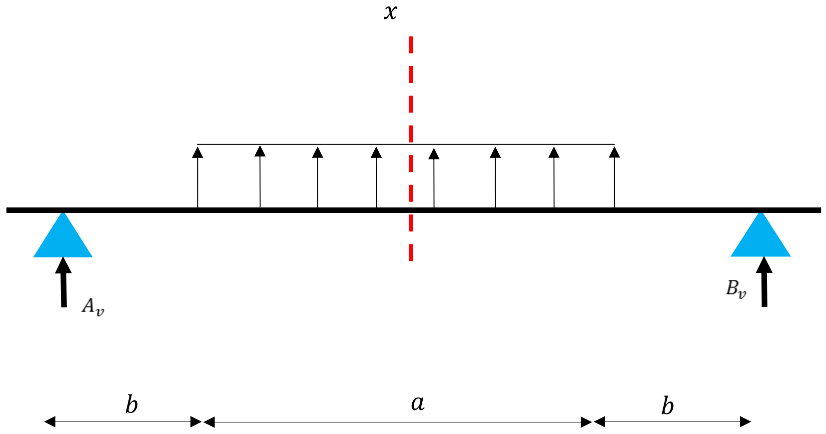
Vz	0,0	[kN]

Bending moment

Mx	-4809,5	[kNm]

Floor loaded

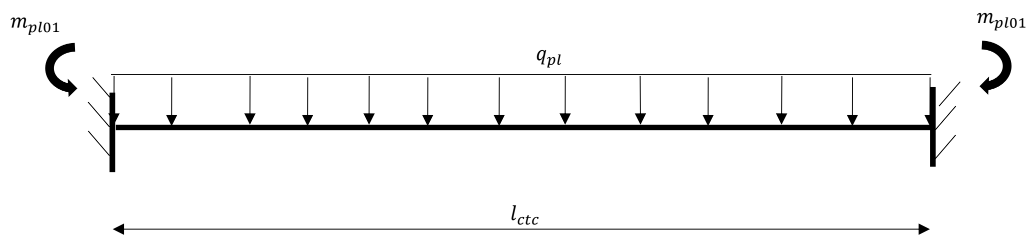
Loading (long. direction)



Measurements

x	0,00	[m]
a	16,0	[m]
l	23,0	[m]
lsup.	21,0	[m]

Loading (transverse direction)



Load & Reaction forces

qpl	-47,0	[kN/m]
mpl01	-62,6	[kNm]

Measurements

lctc	4,0	[m]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,tot	0,0	[kNm]	

Suspension force

Qyy	-47,0	[kN]

Clamping moment

mxx,alt	8,1	[kNm]	(due to alternative load case)
mxx,tot	8,1	[kNm]	

5.3 Fatigue: Bridge B – Characteristic loading – support

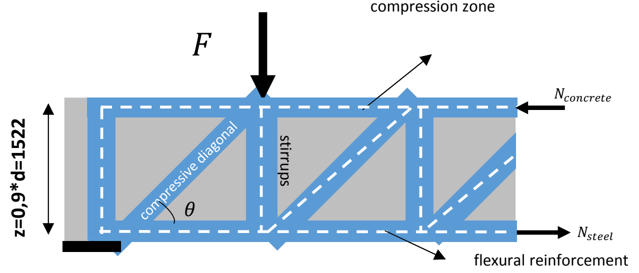
Forces

LC	type	Prestress P [kN]	Bending moment M [kNm]
1	self-weight		-54
2	ballast		-62
3a	Mobile Max. (LM71)		-39
3b	Mobile Min. (LM71)		-39
5	Support settelement max		0
6	Support settelement min		0
7	Prestress t=0	-22826	5045
8	Prestress t = ∞	-20886	4616
σperm (1+2+6+8)		-20886	4500
σ71,min (1+2+3b+6+8)		-20886	4461
σ71,max (1+2+3a+6+8)		-20886	4461

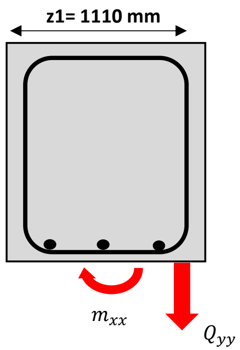
LC	type	Suspension force Qyy [kN]	Clamping moment mxx [kNm]	Shear force [kN]	Torsion [kNm]	Total shear force [kN]
1	self-weight	42	24	1699	-230	
2	ballast	25	15	390	-114	
3a	Mobile Max. (LM71)	78	-2	867	-158	
3b	Mobile Min. (LM71)	0	26	0	-158	
5	Support settelement max	-221	189	0	-491	
6	Support settelement min	221	-189	0	491	
7	Prestress t=0	-57	28	-2287	0	
8	Prestress t = ∞	-52	26	-2093	0	
σperm (1+2+6+8)		236	125	4	147	1159
σ71,min (1+2+3b+6+8)		236	99	4	12	1000
σ71,max (1+2+3a+6+8)		314	127	863	12	2173

Fatigue Parameters

Geometry			
length L	=	31500	[mm]
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
$0{,}5 I_{yy}$	=	1,73E+12	[mm4]
y	=	808	[mm]
$0{,}5 A$	=	4340000	[mm2]



Reinforcement			
Øouter stirrup	=	20	[mm]
Øflexural reinf.	=	16	[mm]
spacing outer stirrup	=	100	[mm]
n rebar flex. reinf.	=	20	[-]
$A_{outer\ stirrup}$	=	3,14	[mm2/mm]
$A_{longitudinal\ reinforcement}$	=	4021	[mm2]
A_p	=	1900	[mm2]



Internal lever arm			
$z = 0{,}9 * d$	=	1923	[mm]
$z_1 = b - 2 * c - 2 * 0{,}5 * \emptyset_{stp}$	=	1110	[mm]

Angle compr. diagonal			
θfat	=	32,3	[◦]
θ	=	21,8	[◦]

Box girder properties			
$Area\ (A)$	=	2,64	[m2]
$Perimeter\ (u)$	=	6,80	[m]
$t_{ef} = A/u$	=	388	[mm]
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126	[mm]
A_k	=	1,47	[m2]
u_k	=	5,25	[m]

Stress range concrete

σperm (1+2+6+8)

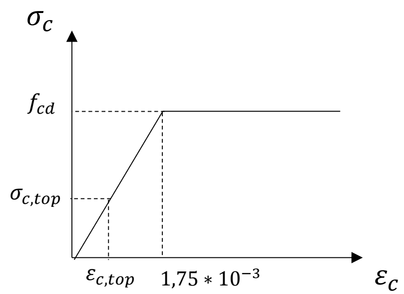
$M_{Ed} - M_{p\infty}$	=	-116	[kNm]
$V_{Ed,tot}$	=	1159	[kN]
x	=	3261	[mm]
$x > h$	=	Entire cross-section under compression	
$\varepsilon_{c,top}$	=	7,66E-04	[-]
$\sigma_{c,top}$	=	-10,21	[N/mm2]

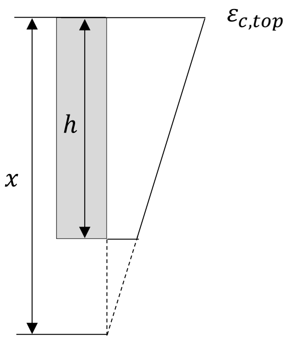
σ71,min (1+2+3b+6+8)

$M_{Ed} - M_{p\infty}$	=	-155	[kNm]
$V_{Ed,tot}$	=	1000	[kN]
x	=	3302	[mm]
$x > h$	=	Entire cross-section under compression	
$\varepsilon_{c,top}$	=	7,57E-04	[-]
$\sigma_{c,top}$	=	-10,09	[N/mm2]

σ71,max (1+2+3a+6+8)

$M_{Ed} - M_{p\infty}$	=	-155	[kNm]
$V_{Ed,tot}$	=	2173	[kN]
x	=	3040	[mm]
$x > h$	=	Entire cross-section under compression	
$\varepsilon_{c,top}$	=	8,20E-04	[-]
$\sigma_{c,top}$	=	-10,93	[N/mm2]





Fatigue calculation on long. reinforcement/prestress necessary?

Load combination	Total	
	x [mm]	Check
σperm (1+2+6+8)	3261	NO
σ71,min (1+2+3b+6+8)	3302	NO
σ71,max (1+2+3a+6+8)	3040	NO

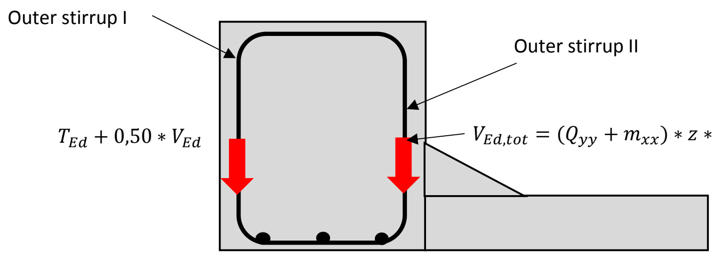
Stress range concrete

Load combination	Total
	$\sigma_{c,top}$ (N/mm^2)
σperm (1+2+7+9)	-10,21
σ71,min (1+2+5b+7+9)	-10,09
σ71,max (1+2+3+5a+7+9)	-10,93

Stress range stirrups

Stress range outer stirrup

	Total	
LC type	**VEd,tot [kN]**	**σstirrup [N/mm2]**
1 self-weight		
2 ballast		
3a Mobile Max. (LM71)		
3b Mobile Min. (LM71)		
5 Support settelement max		
6 Support settelement min		
7 Prestress t=0		
8 Prestress t = ∞		
σperm (1+2+7+9)	1157	121
σ71,min (1+2+5b+7+9)	998	104
σ71,max (1+2+3+5a+7+9)	1742	182



Concrete Fatigue verification (damage equivalent stress)

Fatigue strength

k_1	=	0,85	[-]	(NEN-EN 1992-1-1 NB)
$\beta_{cc}(t_0)$	=	1,00	[-]	
f_{ck}	=	35	[N/mm2]	(RBK)
$\gamma_{c,fat}$	=	1,50	[-]	(NEN-EN 1992-1-1 NB)
f_{cd}	=	23,3	[N/mm2]	
$f_{cd,fat}$	=	18,1	[N/mm2]	

Concrete stress

γ_{sd}	=	1,00	[-]
$\sigma_{c,perm}$	=	10,21	[N/mm2]
$\sigma_{c,max,71}$	=	10,93	[N/mm2]
$\sigma_{c,min,71}$	=	10,09	[N/mm2]
$\sigma_{cd,max,equ}$	=	10,78	[N/mm2]
$\sigma_{cd,min,equ}$	=	10,12	[N/mm2]
$E_{cd,min,equ}$	=	0,56	[-]
$E_{cd,max,equ}$	=	0,60	[-]

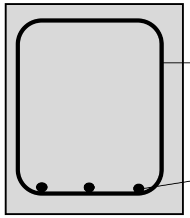
Unity check

$R_{equ} = \frac{E_{cd,min,equ}}{E_{cd,max,equ}}$	=	0,94 [-]			
$14 * \frac{1 - E_{cd,max,equ}}{\sqrt{1 - R_{equ}}} \geq 6$	=	22,82	>	6	OK

Correction factor λc

Factor for permanent stress	=	$\lambda_{c,0} = 0{,}94 + 0{,}2 * \sigma_{c,perm}/f_{cd,fat} \geq 1$	=	1,05 [-]
Factor for element type	=	$L = critical\ length\ influence\ line$	=	20,0 [m]
	=	$\lambda_{c,1}(2\ m) = according\ to\ table\ NN.3$	=	0,70 [-]
	=	$\lambda_{c,1}(20\ m) = according\ to\ table\ NN.3$	=	0,75 [-]
	=	$\lambda_{c,1}(L) = \lambda_{c,1}(2\ m) + [\lambda_{c,1}(20\ m) - \lambda_{c,1}(2\ m)] * (\log L - 0{,}3)$	=	0,75 [-]
Factor for volume and life	=	Vol	=	2,50E+07 [ton/year/tr]
	=	N_{years}	=	100 [year]
	=	$\lambda_{c,2,3} = 1 + \frac{1}{8} * \log\left(\frac{Vol}{25 * 10^6}\right) + \frac{1}{8} * \log\left(\frac{N_{years}}{100}\right)$	=	1,00 [-]
Factor for more than one track	=	$\lambda_{c,4} =$	=	1,00 [-]
Damage equivalent factor	=	$\lambda_c = \lambda_{c,0} * \lambda_{c,1} * \lambda_{c,2,3} * \lambda_{c,4} =$	=	0,79 [-]

Outer stirrup fatigue verification (damage equivalent stress)



Damage equivalent stress

$\Phi * \Delta\sigma_{s,71}$	=	77,85 [N/mm2]
$\Delta\sigma_{s,equ}$	=	50,58 [N/mm2]

Safety factors

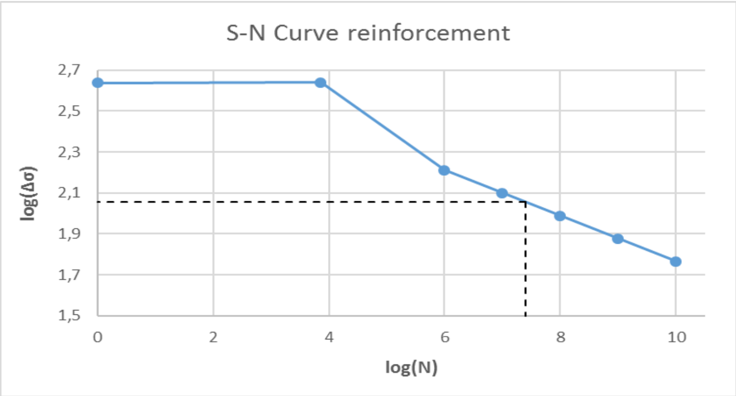
$\gamma_{F,fat}$	=	1,00 [-]	(NEN-EN 1992-1-1 NB)
$\gamma_{s,fat}$	=	1,15 [-]	(NEN-EN 1992-1-1 NB)

Correction factor λs

Factor for element type	=	$L = critical\ length\ influence\ line$	=	20,0 [m]	(OVS 00030-6)
	=	$\lambda_{s,1}(2\ m) = according\ to\ table\ NN.2$	=	0,90 [-]	
	=	$\lambda_{s,1}(20\ m) = according\ to\ table\ NN.2$	=	0,65 [-]	
	=	$\lambda_{s,1}(L) = \lambda_{s,1}(2\ m) + [\lambda_{s,1}(20\ m) - \lambda_{s,1}(2\ m)] * (\log L - 0{,}3)$	=	0,65 [-]	
Factor for volume	=	Vol	=	2,50E+07 [ton/year/tr]	
	=	slope of S-N line (table 6.3N NEN 1992-1-1), k_2	=	9,0 [-]	
	=	$\lambda_{s,2} = \sqrt[k_2]{\frac{Vol}{25 * 10^6}}$	=	1,0 [-]	
Factor for life	=	N_{years}	=	100 [years]	
	=	$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}}$	=	1,0 [-]	
Factor for more than one track	=	$\lambda_{s,4}$	=	1,0 [-]	
Damage equivalent factor	=	$\lambda_s = \lambda_{s,1} * \lambda_{s,2} * \lambda_{c,3} * \lambda_{s,4} =$	=	0,65 [-]	

S-N curve

N^*	1,00E+06 [cycles]	(NEN-EN 1992-1-1)
k_1	5,0 [-]	(NEN-EN 1992-1-1)
k_2	9,0 [-]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N^*)$	162,5 [N/mm2]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N)$	113,6 [N/mm2]	(see graph)



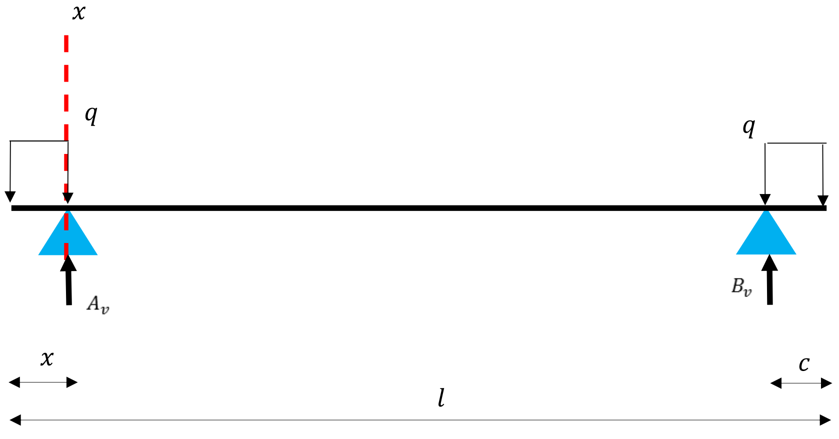
Unity check

$U.C. = \gamma_{F,fat} * \Delta\sigma_{s,equ}(N) \leq \frac{\Delta\sigma_{risk}(N)}{\gamma_{s,fat}}$	=	0,51	<	1,00	OK

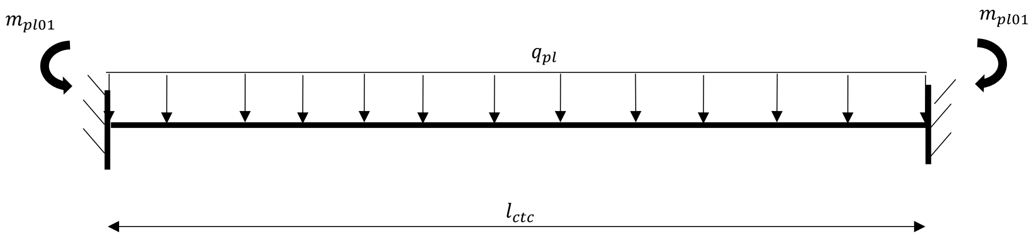
LC 1

Cantilevers loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
qbridge	216	[kN/m]
0,5q	107,9	[kN/m]
Av	107,9	[kN]
Bv	107,9	[kN]
γG	1,00	[-]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	43,2	[kN/m]
mpl01	89,9	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -53,9 [kNm]

Torsion

Mxy,M -89,9 [kNm] (due to torsional moment at both ends)

Mxy,subtot -89,9 [kNm]

Suspension force

Qyy 0,0 [kN]

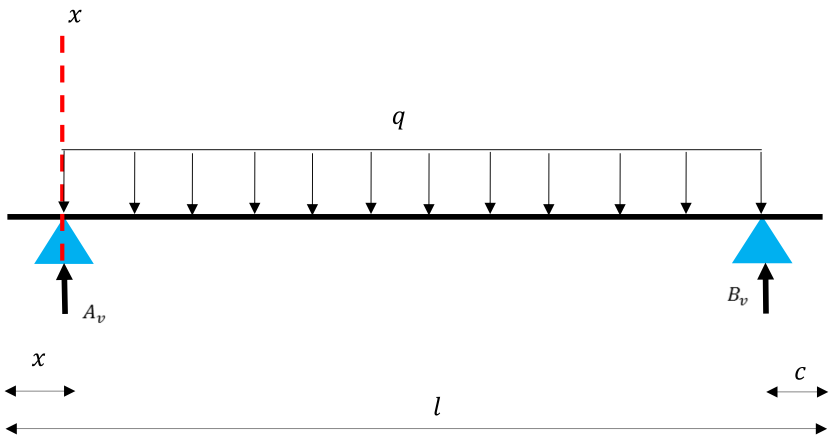
Clamping moment

mxx,M 14,6 [kNm] (due to torsional moment at both ends)

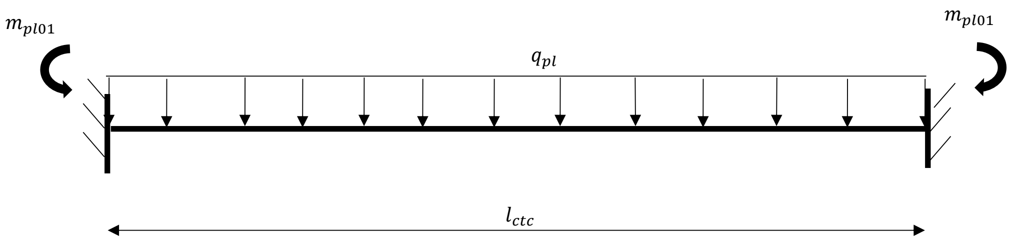
mxx,subtot. 14,6 [kNm]

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
qbridge	216	[kN/m]
0,5q	107,9	[kN/m]
Av	1699,0	[kN]
Bv	1699,0	[kN]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	43,2	[kN/m]
mpl01	89,9	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz 1699,0 [kN]

Bending moment

Mx 0,0 [kNm]

Torsion

Mxy,alt	-140,5	[kNm]	(due to alternative load case)
Mxy,subtot	-140,5	[kNm]	

Suspension force

Qyy 42,3 [kN]

Clamping moment

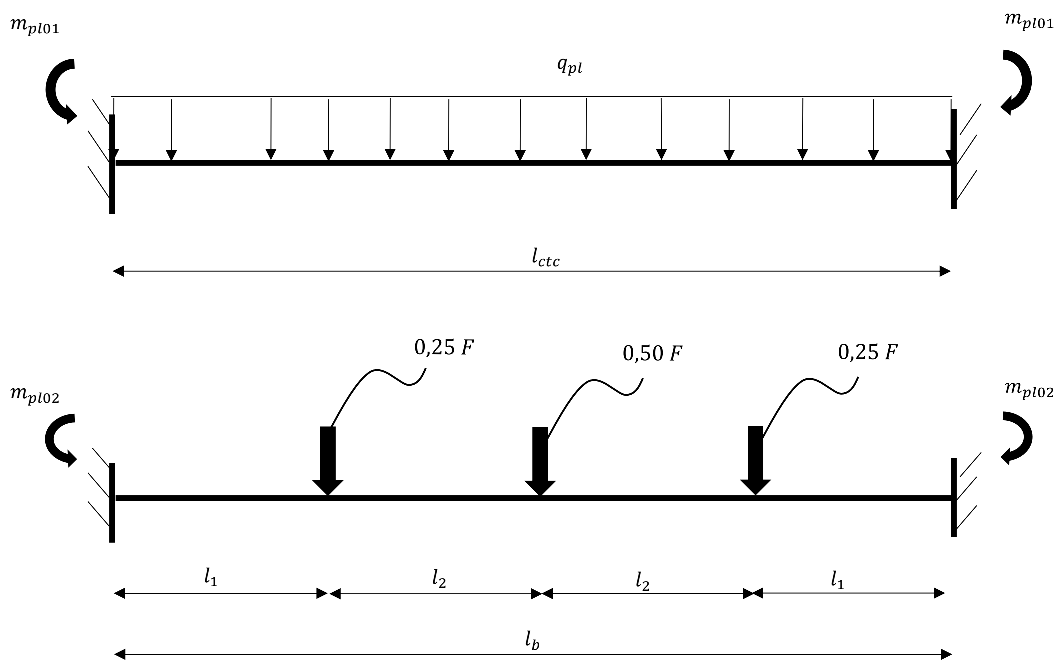
mxx,alt	8,9	[kNm]	(due to alternative load case)
mxx,subtot.	8,9	[kNm]	

LC 2

Cantilevers loaded

Loading (long. direction)

Loading (transverse direction)



Load & Reaction forces

q	9,9	[kN/m2]
0,5q	24,8	[kN/m]
F	99	[kN]
0,5F	50	[kN]
Av	74,4	[kN]
Bv	74,4	[kN]
γ_G	1,00	[-]

Measurements

x	1,0	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces

qpl	9,9	[kN/m]
0,25F	24,8	[kN/m]
mpl01	20,6	[kNm]
mpl02	61,0	[kNm]
mpl01+mpl02	81,6	[kNm]

Measurements

lctc	5,0	[m]
l1	1,1	[m]
l2	2,0	[m]
lb	6,2	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

Bending moment

Mx	-62,1	[kNm]

Torsion

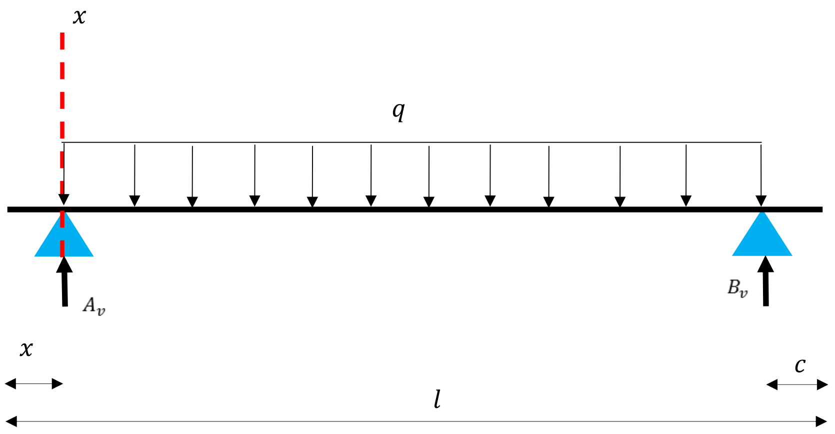
Mxy,M	-81,6	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	-81,6	[kNm]	

Clamping moment

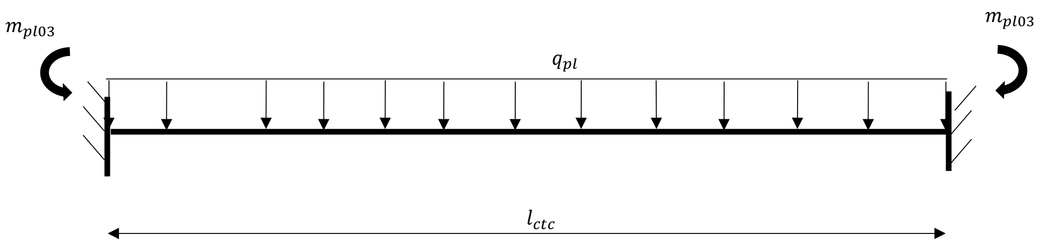
mxx,M	13,2	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	13,2	[kNm]	

Midspan loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces

q	9,9	[kN/m2]
0,5q	24,8	[kN/m]
Av	389,8	[kN]
Bv	389,8	[kN]

Measurements

x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces

qpl	9,9	[kN/m]
mpl01	20,6	[kNm]

Measurements

lctc	5,0	[m]

Shear Force

Vz	389,8	[kN]

Suspension force

Qyy	24,8	[kN]

Bending moment

Mx	0,0	[kNm]

Torsion

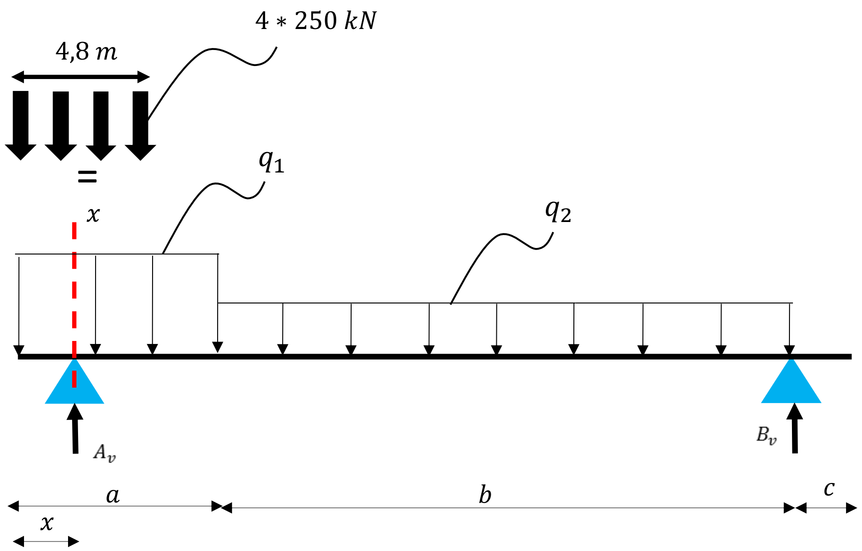
Mxy,incr.	-32,2	[kNm]	(due to alternative load case)
Mxy,subtot	-32,2	[kNm]	

Clamping moment

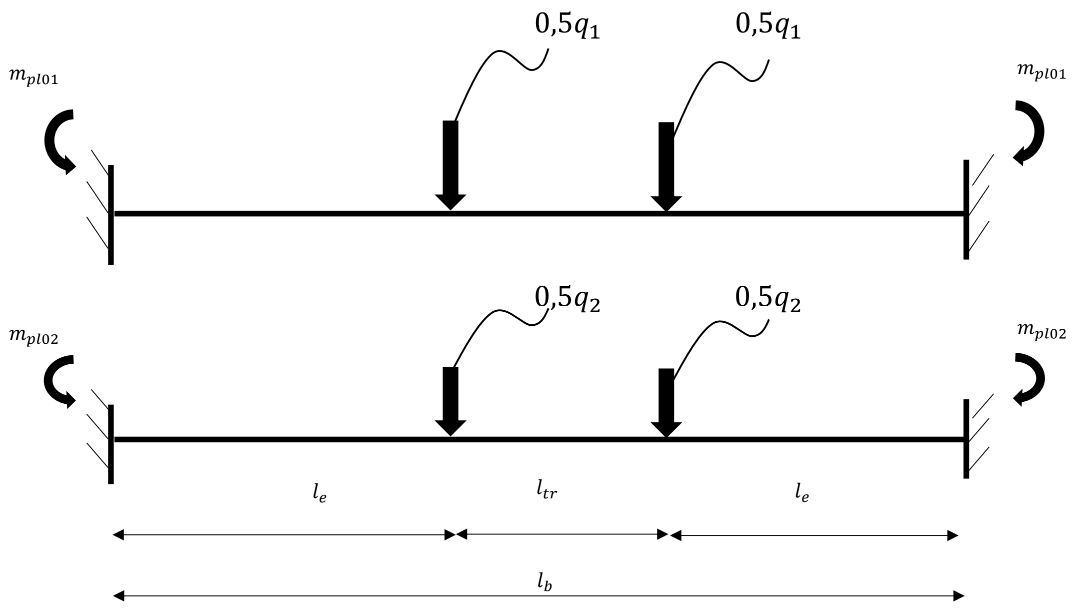
mxx,incr.	2,0	[kNm]	(due to alternative load case)
mxx,subtot.	2,0	[kNm]	

LC 3a

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q1	156,1	[kN/m]
q2	86,2	[kN/m]
0,5q1	78,0	[kN]
0,5q2	43,1	[kN]
Av	944,8	[kN]
Bv	696,6	[kN]
γQ	1,00	[-]
α	1,00	[-]

Measurements		
x	1,00	[m]
a	6,9	[m]
b	25,6	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
q1	78,0	[kN/m]
q2	43,1	[kN/m]
mpl01	158,3	[kNm]
mpl02	87,4	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]

Shear Force

Vz 866,7 [kN]

Bending moment

Mx -39,0 [kNm]

Suspension force

Qyy 78,0 [kN]

Torsion

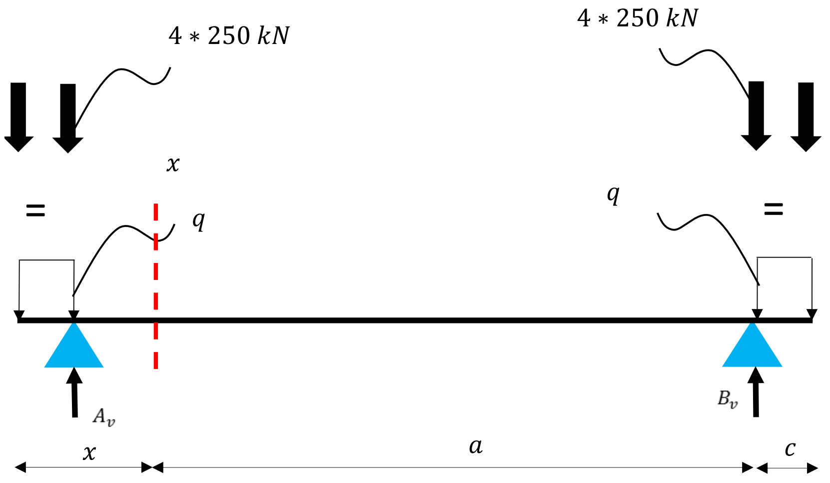
Mxy,M	-158,3	[kNm]	(due to torsional moment at one end)
Mxy,q1	0	[kNm]	(due to distributed load)
Mxx,q2	0	[kNm]	(due to distributed load)
Mxy,tot	-158,3	[kNm]	

Clamping moment

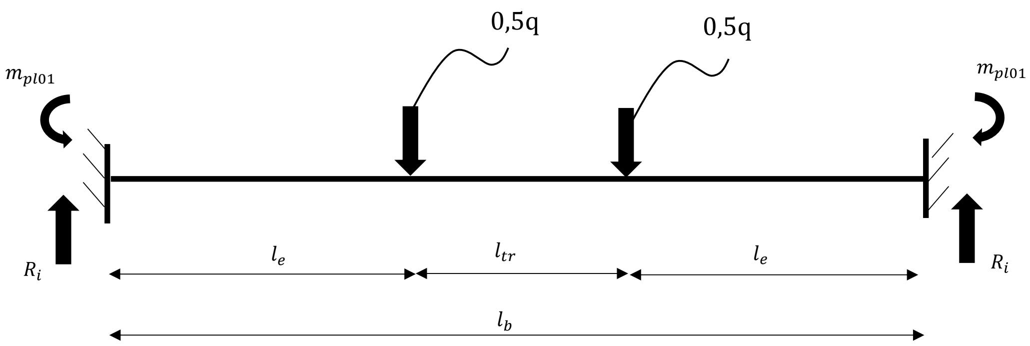
mxx,M	25,3	[kNm]	(due to torsional moment at one end)
mxx,q1	-61,6	[kNm]	(due to distributed load)
mxx,q2	34	[kNm]	(due to distributed load)
mxx,tot	-2,3	[kNm]	

LC 3b

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	156,1	[kN/m]
0,5q	78,0	[kN/m]
Av	78,0	[kN]
Bv	78,0	[kN]
γQ	1,00	[-]
α	1,00	[-]

Measurements		
x	1,00	[m]
a	31,5	[m]
b	0,0	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
p	78,0	[kN/m]
mpl01	158,3	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -39,0 [kNm]

Torsion

Mxy,M	-158,3	[kNm]	(due to torsional moment at both sides)
Mxy,tot	-158,3	[kNm]	

Suspension force

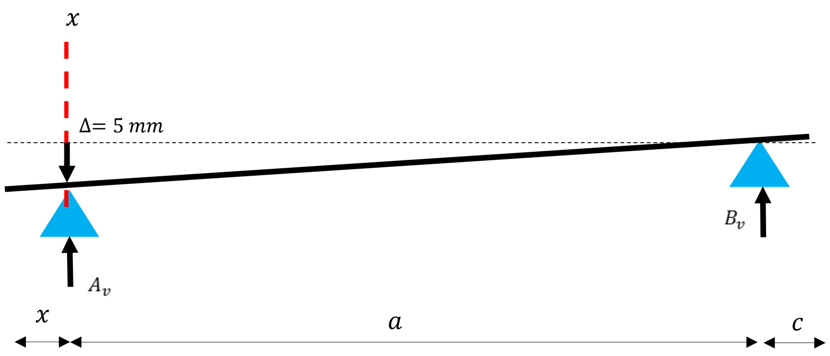
Qyy 0,0 [kN]

Clamping moment

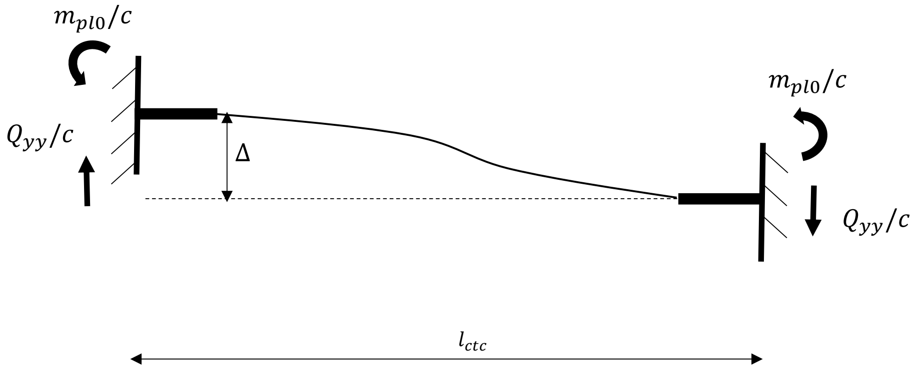
mxx,M	25,7	[kNm]	(due to torsional moment at both sides)
mxx,tot	25,7	[kNm]	

LC 5

Loading (long. direction)



Loading (transverse direction)



Deflection & Reaction forces		
Δ	5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γq	1,00	[-]

Measurements		
x	1,00	[m]
a	31,5	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Deflection		
Δ	0,005	[m]

Shear Force

Vz	0,0	[kN]

Suspension force

Qyy	-221,0	[kN]
Qyy,tot	-221,0	[kN]

Bending moment

Mx	0,0	[kNm]

Clamping moment

mxx,Δ	189,0	[kNm]
mxx,tot	189,0	[kNm]

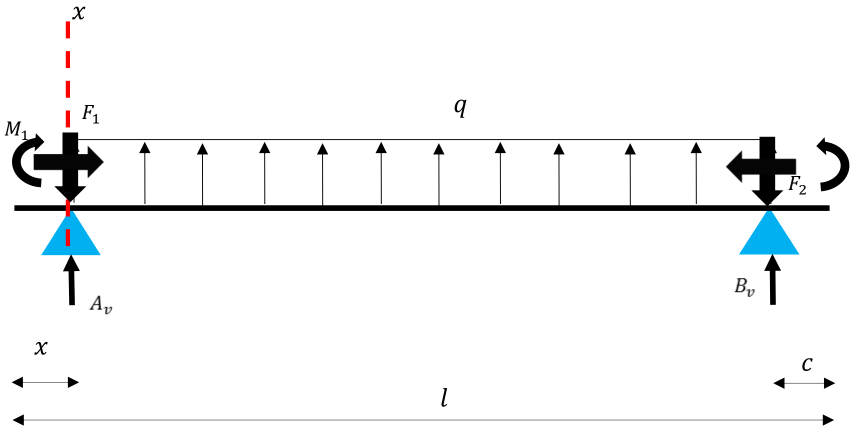
Torsion

Mxy,Δ	-491,0	[kNm]
Mxy,tot	-491,0	[kNm]

LC 7

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	2290	[kN]
F2	-22826	[kN]
M1	5045	[kNm]
q	-145,2	[kN]
Av	3	[kN]
Bv	3	[kN]
γP	1,00	[-]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

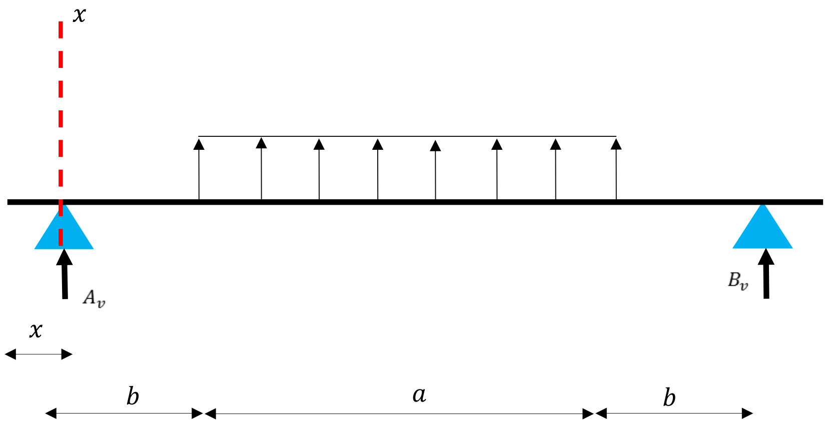
Vz -2286,9 [kN]

Bending moment

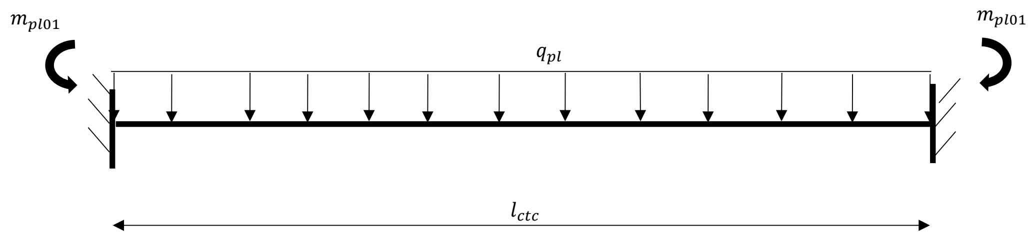
Mx 5045,0 [kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	1,00	[m]
a	20,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	-56,9	[kN/m]
mpl01	-118,6	[kNm]

Measurements		
lctc	5,0	[m]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,tot	0,0	[kNm]	

Suspension force

Qyy	-56,9	[kN]

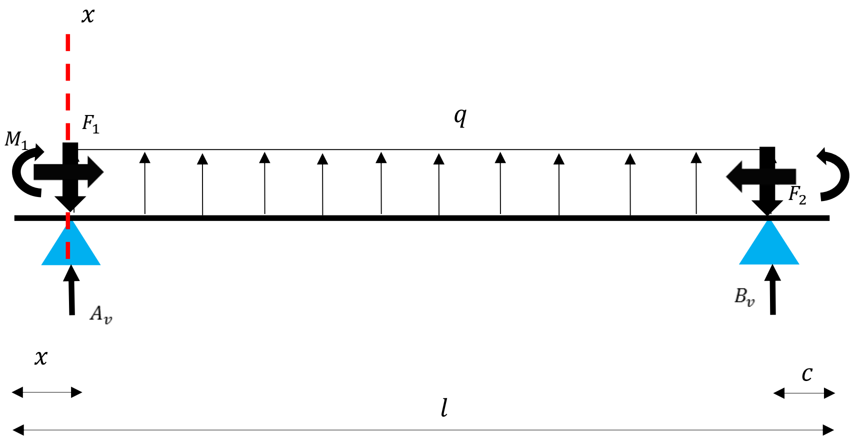
Clamping moment

mxx,alt	28,0	[kNm]	(due to alternative load case)
mxx,tot	28,0	[kNm]	

LC 8

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	2095	[kN]
F2	-20886	[kN]
M1	4616	[kNm]
q	-133	[kN]
Av	3	[kN]
Bv	3	[kN]
P∞/P0	0,915	[-]
γP	1,00	[-]

Measurements		
x	1,00	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

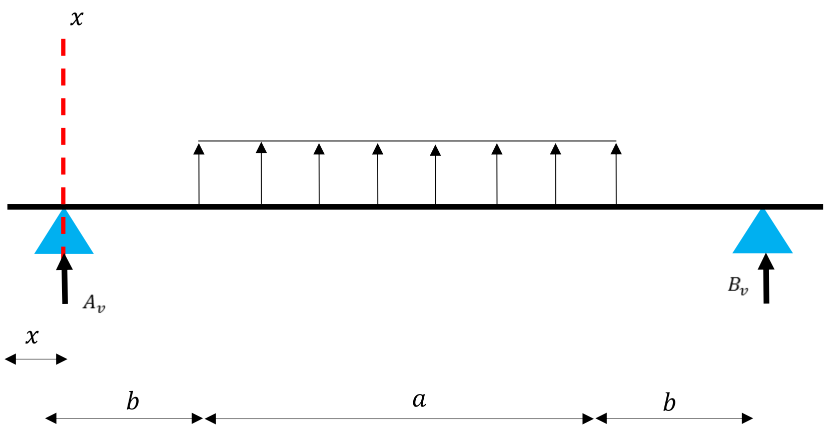
Vz	-2092,5	[kN]

Bending moment

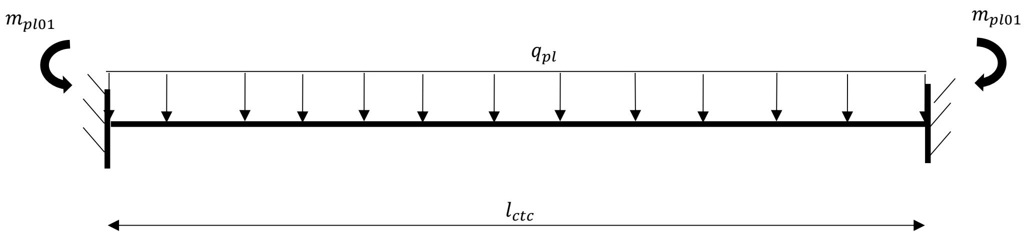
Mx	4616,2	[kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	1,00	[m]
a	20,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	-52,1	[kN/m]
mpl01	-108,5	[kNm]

Measurements		
lctc	5,0	[m]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,tot	0,0	[kNm]	

Suspension force

Qyy	-52,1	[kN]

Clamping moment

mxx,alt	25,6	[kNm]	(due to alternative load case)
mxx,tot	25,6	[kNm]	

5.4 Fatigue: Bridge B – Characteristic loading – 0,5L

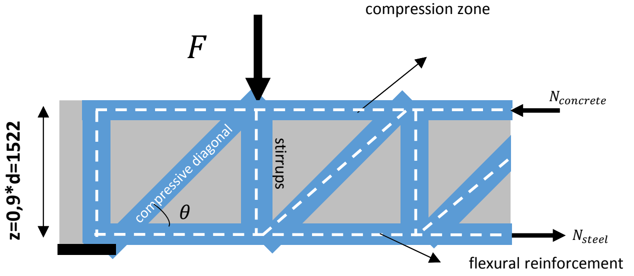
Forces

		Prestress	Bending moment
LC	type	P [kN]	M [kNm]
1	self-weight		13326
2	ballast		3008
3a	Mobile Max. (LM71)		7035
3b	Mobile Min. (LM71)		-39
5	Support settelement max		0
6	Support settelement min		0
7	Prestress t=0	-22826	-12964
8	Prestress t = ∞	-20886	-11862
σperm (1+2+6+8)		-20886	4471
σ71,min (1+2+3b+6+8)		-20886	4432
σ71,max (1+2+3a+6+8)		-20886	11506

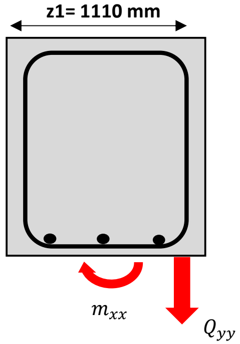
		Suspension force	Clamping moment	Shear force	Torsion	Total shear force
LC	type	Qyy [kN]	mxx [kNm]	[kN]	[kNm]	[kN]
1	self-weight	42	11	0	0	
2	ballast	25	4	0	0	
3a	Mobile Max. (LM71)	78	-8	0	0	
3b	Mobile Min. (LM71)	0	4	0	0	
5	Support settelement max	1	6	0	169	
6	Support settelement min	-1	-6	0	-169	
7	Prestress t=0	-57	-19	0	0	
8	Prestress t = ∞	-52	-17	0	0	
σperm (1+2+6+8)		14	8	0	169	151
σ71,min (1+2+3b+6+8)		14	4	0	169	144
σ71,max (1+2+3a+6+8)		92	16	0	169	315

Fatigue Parameters

Geometry			
length L	=	31500	[mm]
height h	=	2200	[mm]
width b	=	1200	[mm]
cover	=	35	[mm]
effective height	=	2137	[mm]
$0{,}5I_{yy}$	=	1,73E+12	[mm4]
y	=	808	[mm]
$0{,}5A$	=	4340000	[mm2]



Reinforcement			
Øouter stirrup	=	20	[mm]
Øflexural reinf.	=	16	[mm]
spacing outer stirrup	=	100	[mm]
n rebar flex. reinf.	=	20	[-]
$A_{outer\ stirrup}$	=	3,14	[mm2/mm]
$A_{flexural\ reinforcement}$	=	4021	[mm2]
A_p	=	1900	[mm2]



Internal lever arm			
$z = 0{,}9 * d$	=	1923	[mm]
$z_1 = b - 2 * c - 2 * 0{,}5 * Ø_{stp}$	=	1110	[mm]

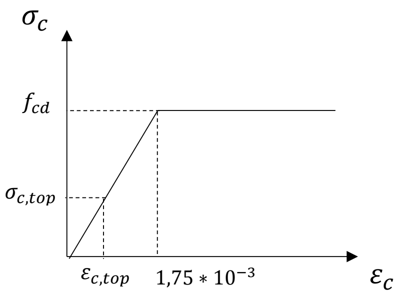
Angle compr. diagonal			
θfat	=	45	[◦]
θ	=	45	[◦]

Box girder properties			
$Area\ (A)$	=	2,64	[m2]
$Perimeter\ (u)$	=	6,80	[m]
$t_{ef} = A/u$	=	388	[mm]
$t_{ef,lower\ limit} = 2 * (h - d)$	=	126	[mm]
A_k	=	1,47	[m2]
u_k	=	5,25	[m]

Stress range concrete

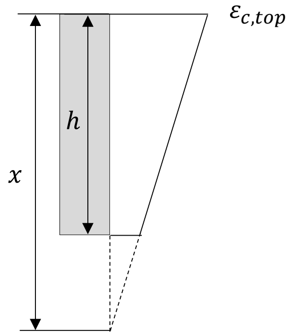
σperm(1+2+6+8)

$M_{Ed} - M_{p\infty}$	=	16334	[kNm]
$V_{Ed,tot}$	=	151	[kN]
x	=	3437	[mm]
$x > h$	=	Entire cross-section under compression	
$\varepsilon_{c,top}$	=	7,40E-04	[-]
$\sigma_{c,top}$	=	-9,87	[N/mm2]



σ71,min(1+2+3b+6+8)

$M_{Ed} - M_{p\infty}$	=	16295	[kNm]
$V_{Ed,tot}$	=	144	[kN]
x	=	3443	[mm]
$x > h$	=	Entire cross-section under compression	
$\varepsilon_{c,top}$	=	7,38E-04	[-]
$\sigma_{c,top}$	=	-9,84	[N/mm2]



σ71,max (1+2+3a+6+8)

$M_{Ed} - M_{p\infty}$	=	23369	[kNm]
$V_{Ed,tot}$	=	315	[kN]
x	=	2416	[mm]
$x > h$	=	Entire cross-section under compression	
$\varepsilon_{c,top}$	=	1,06E-03	[-]
$\sigma_{c,top}$	=	-14,13	[N/mm2]

Fatigue calculation on long. reinforcement/prestress necessary?

Load combination	Total	
	x [mm]	Check
σperm (1+2+6+8)	3437	NO
σ71,min (1+2+3b+6+8)	3443	NO
σ71,max (1+2+3a+6+8)	2416	NO

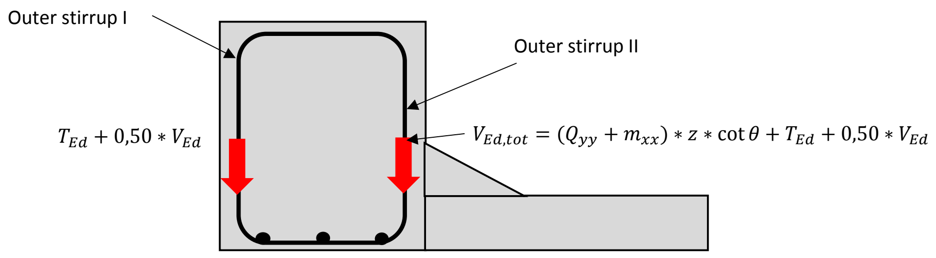
Stress range concrete

Load combination	Total
	$\sigma_{c,top}$ (N/mm^2)
σperm (1+2+7+9)	-9,87
σ71,min (1+2+5b+7+9)	-9,84
σ71,max (1+2+3+5a+7+9)	-14,13

Stress range stirrups

Stress range outer stirrup

		Total	
LC	type	VEd,tot [kN]	σstirrup [N/mm2]
1	self-weight		
2	ballast		
3a	Mobile Max. (LM71)		
3b	Mobile Min. (LM71)		
5	Support settelement max		
6	Support settelement min		
7	Prestress t=0		
8	Prestress t = ∞		
σperm (1+2+7+9)		151	25
σ71,min (1+2+5b+7+9)		144	24
σ71,max (1+2+3+5a+7+9)		315	52



Concrete Fatigue verification (damage equivalent stress)

Fatigue strength

k_1	=	0,85 [-]	(NEN-EN 1992-1-1 NB)
$\beta_{cc}(t_0)$	=	1,00 [-]	
f_{ck}	=	35 [N/mm2]	(RBK)
$\gamma_{c,fat}$	=	1,50 [-]	(NEN-EN 1992-1-1 NB)
f_{cd}	=	23,3 [N/mm2]	
$f_{cd,fat}$	=	18,1 [N/mm2]	

Concrete stress

γ_{sd}	=	1,00 [-]
$\sigma_{c,perm}$	=	9,87 [N/mm2]
$\sigma_{c,max,71}$	=	14,13 [N/mm2]
$\sigma_{c,min,71}$	=	9,84 [N/mm2]
$\sigma_{cd,max,equ}$	=	13,22 [N/mm2]
$\sigma_{cd,min,equ}$	=	9,85 [N/mm2]
$E_{cd,min,equ}$	=	0,54 [-]
$E_{cd,max,equ}$	=	0,73 [-]

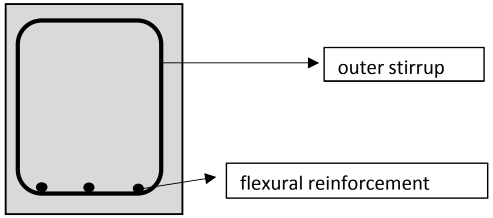
Unity check

$R_{equ} = \frac{E_{cd,min,equ}}{E_{cd,max,equ}}$	=	0,74 [-]			
$14 * \frac{1 - E_{cd,max,equ}}{\sqrt{1 - R_{equ}}} \geq 6$	=	7,46	>	6	OK

Correction factor λc

Factor for permanent stress	=	$\lambda_{c,0} = 0{,}94 + 0{,}2 * \sigma_{c,perm}/f_{cd,fat} \geq 1$	=	1,05 [-]
Factor for element type	=	$L = critical\ length\ influence\ line$	=	20,0 [m]
	=	$\lambda_{c,1}(2\ m) = according\ to\ table\ NN.3$	=	0,70 [-]
	=	$\lambda_{c,1}(20\ m) = according\ to\ table\ NN.3$	=	0,75 [-]
	=	$\lambda_{c,1}(L) = \lambda_{c,1}(2\ m) + [\lambda_{c,1}(20\ m) - \lambda_{c,1}(2\ m)] * (\log L - 0{,}3)$	=	0,75 [-]
Factor for volume and life	=	Vol	=	2,50E+07 [ton/year/tr]
	=	N_{years}	=	100 [year]
	=	$\lambda_{c,2,3} = 1 + \frac{1}{8} * \log\left(\frac{Vol}{25 * 10^6}\right) + \frac{1}{8} * \log\left(\frac{N_{years}}{100}\right)$	=	1,00 [-]
Factor for more than one track	=	$\lambda_{c,4} =$	=	1,00 [-]
Damage equivalent factor	=	$\lambda_c = \lambda_{c,0} * \lambda_{c,1} * \lambda_{c,2,3} * \lambda_{c,4} =$	=	0,79 [-]

Outer stirrup fatigue verification (damage equivalent stress)



Correction factor λs

Factor for element type	=	$L = critical\ length\ influence\ line$	=	20,0 [m]	(OVS 00030-6)
	=	$\lambda_{s,1}(2\ m) = according\ to\ table\ NN.2$	=	0,90 [-]	
	=	$\lambda_{s,1}(20\ m) = according\ to\ table\ NN.2$	=	0,65 [-]	
	=	$\lambda_{s,1}(L) = \lambda_{s,1}(2\ m) + [\lambda_{s,1}(20\ m) - \lambda_{s,1}(2\ m)] * (\log L - 0{,}3)$	=	0,65 [-]	
Factor for volume	=	Vol	=	2,50E+07 [ton/year/tr]	
	=	slope of S-N line (table 6.3N NEN 1992-1-1), k_2	=	9,0 [-]	
	=	$\lambda_{s,2} = \sqrt[k_2]{\frac{Vol}{25 * 10^6}}$	=	1,0 [-]	
Factor for life	=	N_{years}	=	100 [years]	
	=	$\lambda_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}}$	=	1,0 [-]	
Factor for more than one track	=	$\lambda_{s,4}$	=	1,0 [-]	
Damage equivalent factor	=	$\lambda_s = \lambda_{s,1} * \lambda_{s,2} * \lambda_{c,3} * \lambda_{s,4} =$	=	0,65 [-]	

Damage equivalent stress

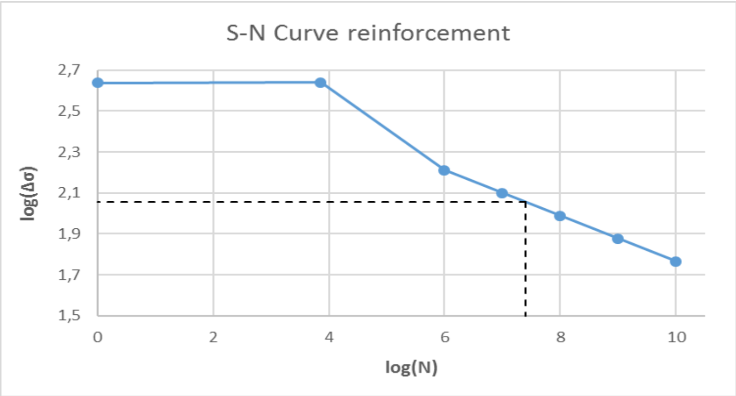
$\Phi * \Delta\sigma_{s,71}$	=	28,28	[N/mm2]
$\Delta\sigma_{s,equ}$	=	18,38	[N/mm2]

Safety factors

$\gamma_{F,fat}$	=	1,00	[-]	(NEN-EN 1992-1-1 NB)
$\gamma_{s,fat}$	=	1,15	[-]	(NEN-EN 1992-1-1 NB)

S-N curve

N^*	1,00E+06	[cycles]	(NEN-EN 1992-1-1)
k_1	5,0	[-]	(NEN-EN 1992-1-1)
k_2	9,0	[-]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N^*)$	162,5	[N/mm2]	(NEN-EN 1992-1-1)
$\Delta\sigma_{risk}(N)$	113,6	[N/mm2]	(see graph)



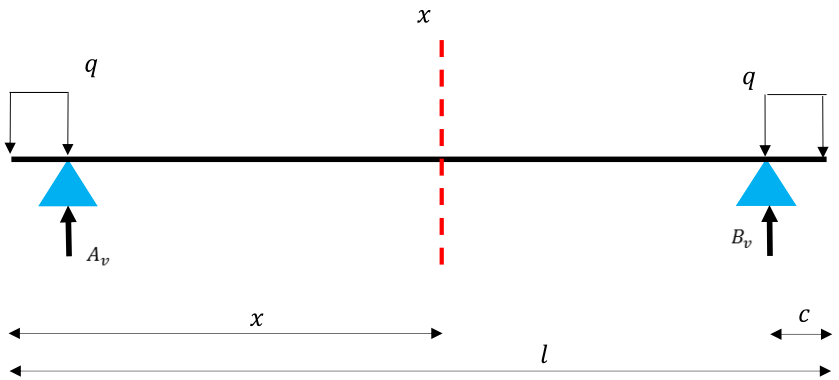
Unity check

$U.C. = \gamma_{F,fat} * \Delta\sigma_{s,equ}(N) \leq \frac{\Delta\sigma_{risk}(N)}{\gamma_{s,fat}}$	=	0,19	<	1,00	OK

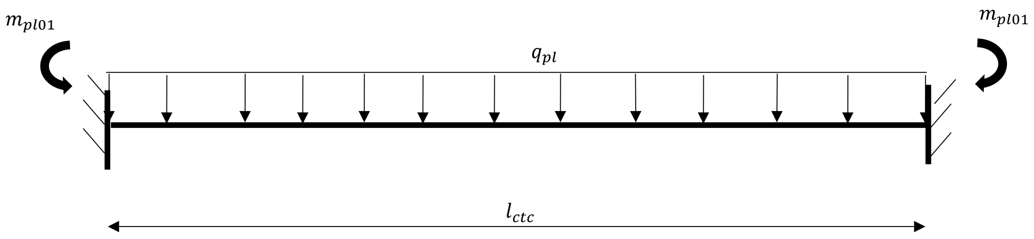
LC 1

Cantilevers loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
qbridge	216	[kN/m]
0,5q	107,9	[kN/m]
Av	107,9	[kN]
Bv	107,9	[kN]
γG	1,00	[-]

Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	43,2	[kN/m]
mpl01	89,9	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -53,9 [kNm]

Torsion

Mxy,M	0,0	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	0,0	[kNm]	

Suspension force

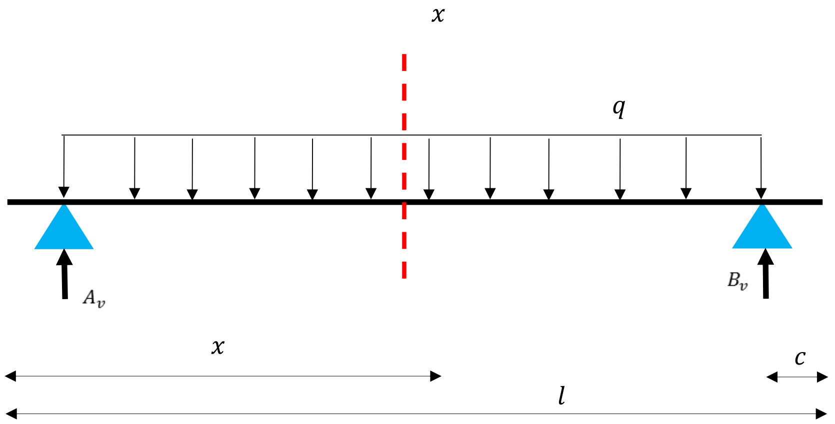
Qyy 0,0 [kN]

Clamping moment

mxx,M	2,3	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	2,3	[kNm]	

Midspan loaded

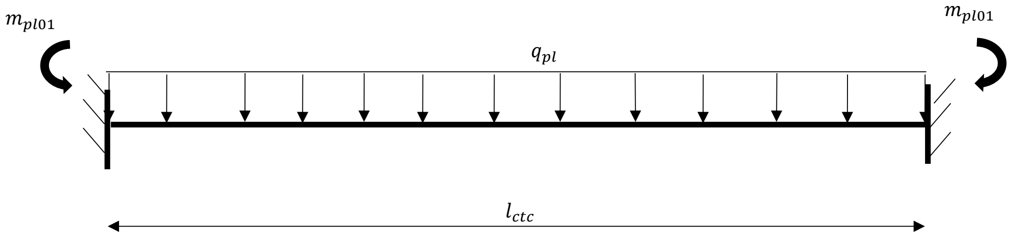
Loading (long. direction)



Load & Reaction forces		
qbridge	216	[kN/m]
0,5q	107,9	[kN/m]
Av	1699,0	[kN]
Bv	1699,0	[kN]

Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	43,2	[kN/m]
mpl01	89,9	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx 13379,9 [kNm]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative loading)
Mxy,subtot	0,0	[kNm]	

Suspension force

Qyy 42,3 [kN]

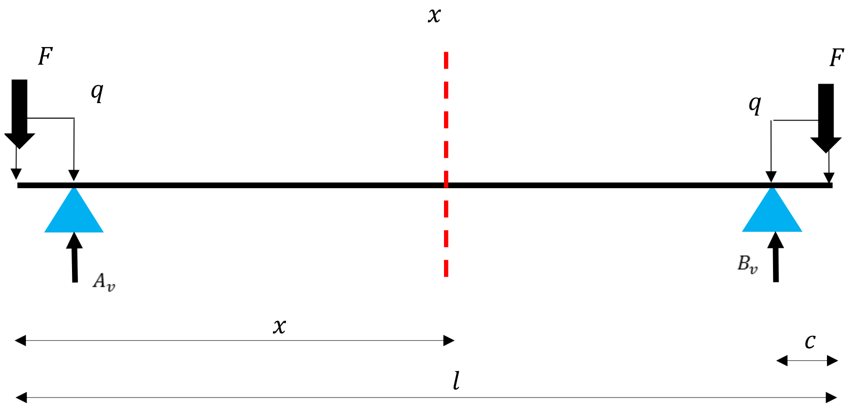
Clamping moment

mxx, alt	8,9	[kNm]	(due to alternative loading)
mxx,subtot.	8,9	[kNm]	

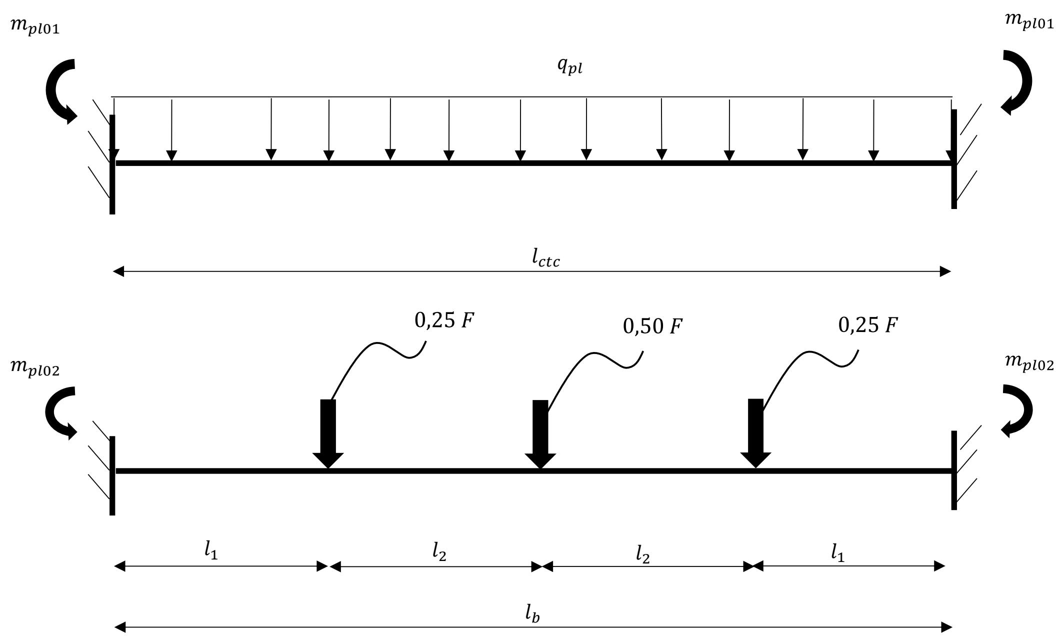
LC 2

Cantilevers loaded

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	9,9	[kN/m2]
0,5q	24,8	[kN/m]
F	99	[kN]
0,5F	49,7	[kN]
Av	74,4	[kN]
Bv	74,4	[kN]
γG	1,00	[-]

Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	9,9	[kN/m]
0,25F	24,8	[kN]
mpl01	20,6	[kNm]
mpl02	61,0	[kNm]
mpl01+mpl02	81,6	[kNm]

Measurements		
lctc	5,0	[m]
l1	1,1	[m]
l2	2,0	[m]
lb	6,2	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 0,0 [kN]

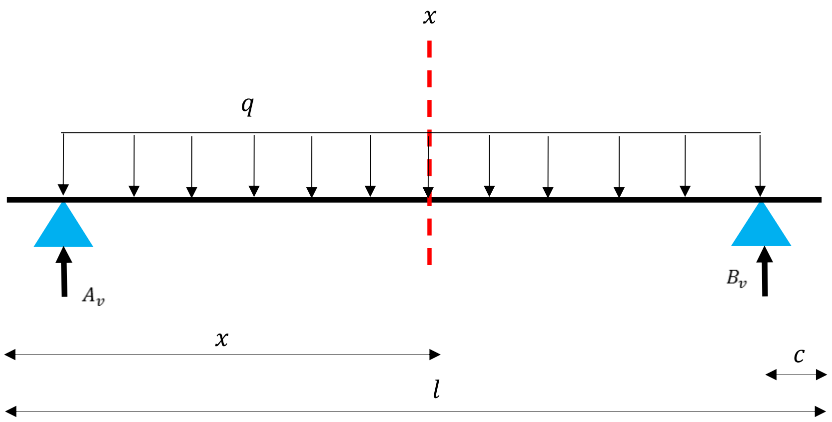
Bending moment		
Mx	-62,1	[kNm]

Torsion			
Mxy,M	0,0	[kNm]	(due to torsional moment at both ends)
Mxy,subtot	0,0	[kNm]	

Clamping moment			
mxx,M	2,1	[kNm]	(due to torsional moment at both ends)
mxx,subtot.	2,1	[kNm]	

Midspan loaded

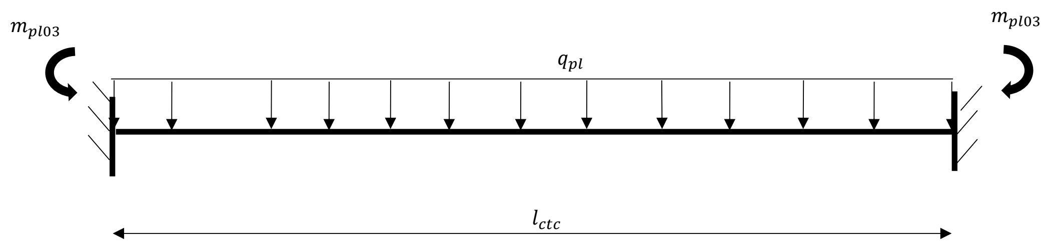
Loading (long. direction)



Load & Reaction forces		
q	9,9	[kN/m2]
0,5q	24,8	[kN/m]
Av	389,8	[kN]
Bv	389,8	[kN]

Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Loading (transverse direction)



Load & Reaction forces		
qpl	9,9	[kN/m]
mpl01	20,6	[kNm]

Measurements		
lctc	5,0	[m]

Shear Force		
Vz	0,0	[kN]

Suspension force		
Qyy	24,8	[kN]

Bending moment

Mx	3069,8	[kNm]

Torsion

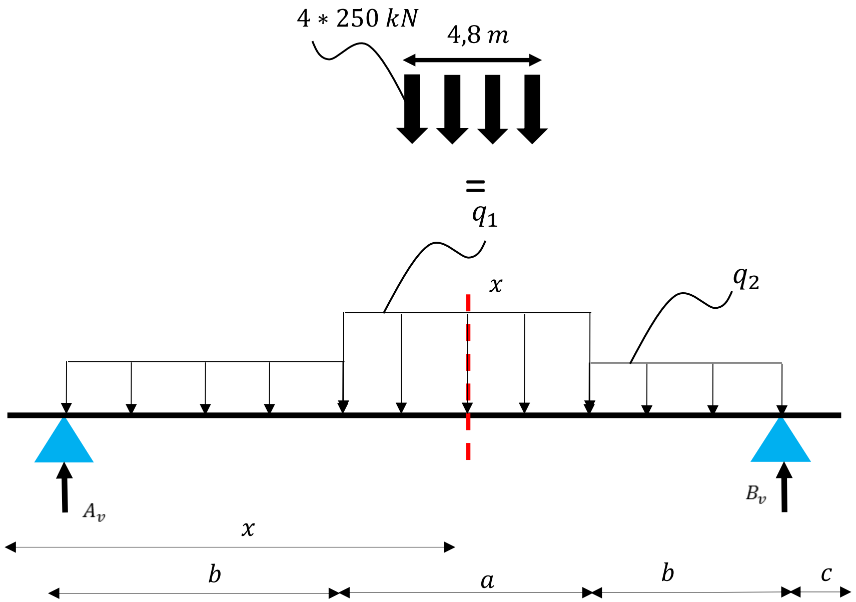
Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,subtot	0,0	[kNm]	

Clamping moment

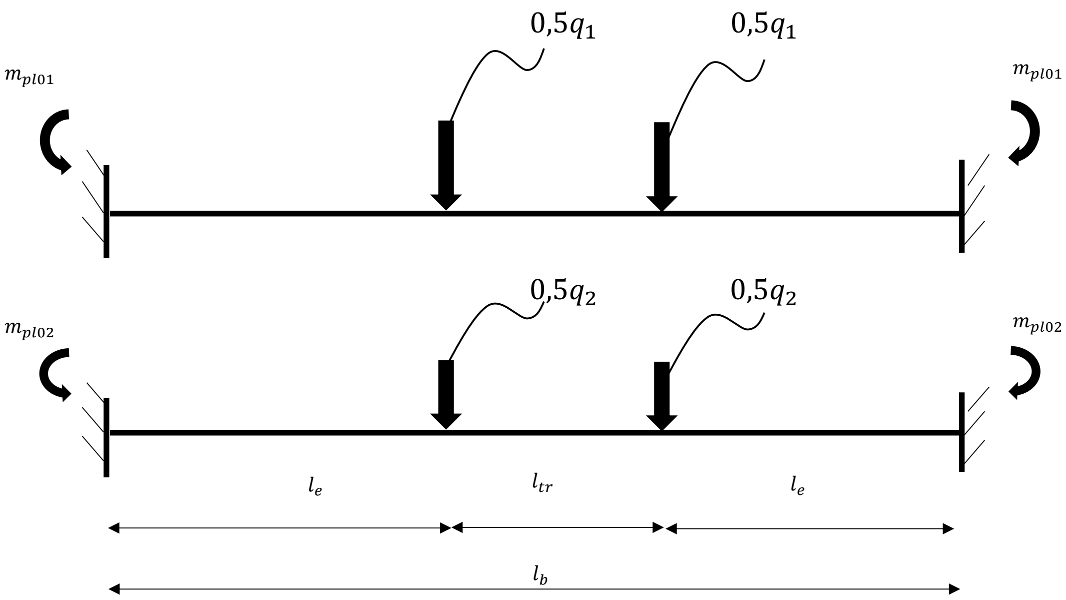
mxx,incr.	2,0	[kNm]	(due to alternative load case)
mxx,subtot.	2,0	[kNm]	

LC 3a

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q1	156,1	[kN/m]
q2	86,2	[kN/m]
0,5q1	78,0	[kN]
0,5q2	43,1	[kN]
Av	799,1	[kN]
Bv	799,1	[kN]
γQ	1,00	[-]
α	1,00	[-]

Measurements		
x	16,75	[m]
a	6,9	[m]
b	12,3	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
0,5q1	78,0	[kN/m]
0,5q2	43,1	[kN/m]
mpl01	158,3	[kNm]
mpl02	87,4	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]

Shear Force

Vz 0,0 [kN]

Suspension force

Qyy 78,0 [kN]

Bending moment

Mx 7035,0 [kNm]

Torsion

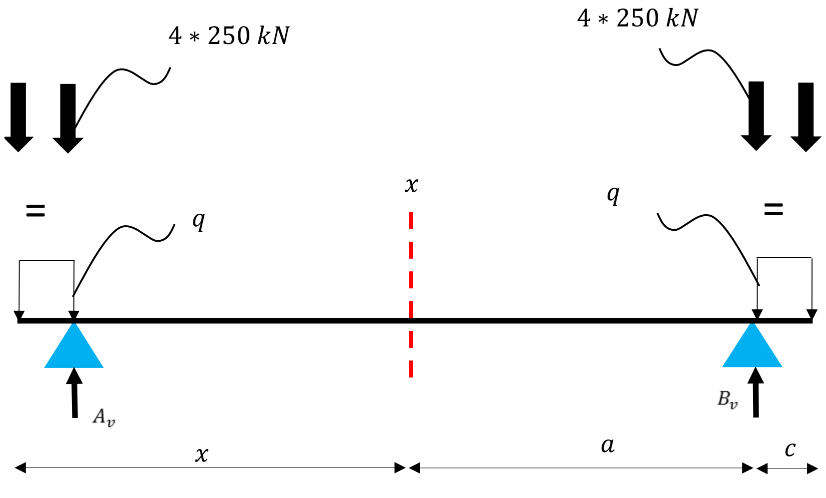
Mxy,q1	0	[kNm]	(due to distributed load)
Mxy,q2	0	[kNm]	(due to distributed load)
Mxy,tot	0,0	[kNm]	

Clamping moment

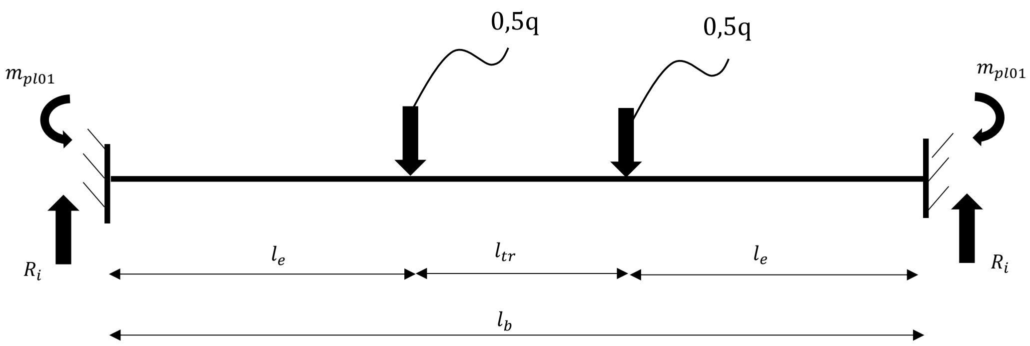
mxx,q1	-57,5	[kNm]	(due to distributed load)
mxx,q2	49,6	[kNm]	(due to distributed load)
mxx,tot	-7,9	[kNm]	

LC 3b

Loading (long. direction)



Loading (transverse direction)



Load & Reaction forces		
q	156,1	[kN/m]
0,5q	78,0	[kN/m]
Av	78,0	[kN]
Bv	78,0	[kN]
γQ	1,00	[-]
α	1,00	[-]

Measurements		
x	16,75	[m]
a	15,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
0,5q	78,0	[kN/m]
mpl01	158,3	[kNm]

Measurements		
le	2,4	[m]
ltr	1,5	[m]
lb	6,2	[m]

Shear Force

Vz 0,0 [kN]

Bending moment

Mx -39,0 [kNm]

Torsion

Mxy,M	0,0	[kNm]	(due to torsion at both sides)
Mxy,tot	0,0	[kNm]	

Suspension force

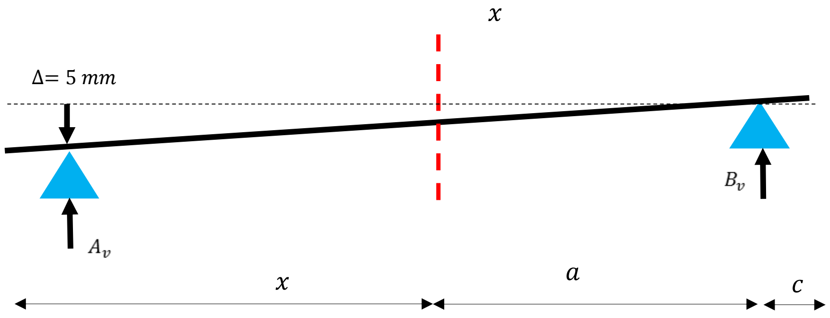
Qyy 0,0 [kN]

Clamping moment

mxx,M	4,1	[kNm]	(due to torsion at both sides)
mxx,tot	4,1	[kNm]	

LC 5

Loading (long. direction)



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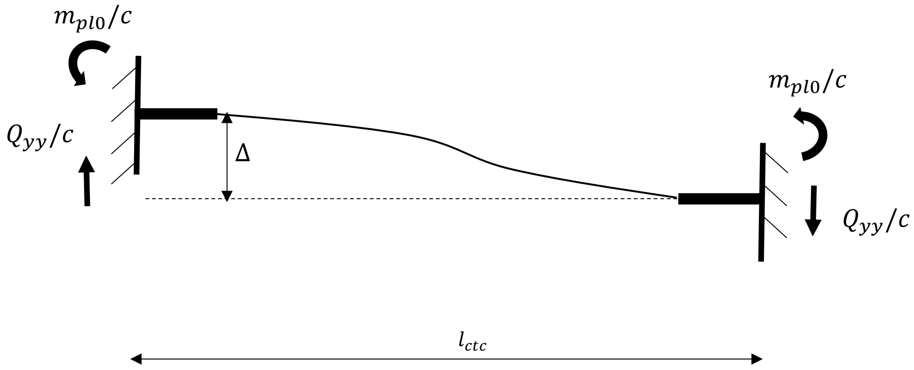


Deflection & Reaction forces		
Δ	5	[mm]
Av	0,0	[kN]
Bv	0,0	[kN]
γq	1,00	[-]

Measurements		
x	16,75	[m]
a	15,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Deflection		
Δ	0,005	[m]

Shear Force

Vz	0,0	[kN]

Bending moment

Mx	0,0	[kNm]

Torsion

Mxy,Δ	169,3	[kNm]
Mxy,tot	169,3	[kNm]

Suspension force

Qyy	1,0	[kN]
Qyyt,tot	1,0	[kN]

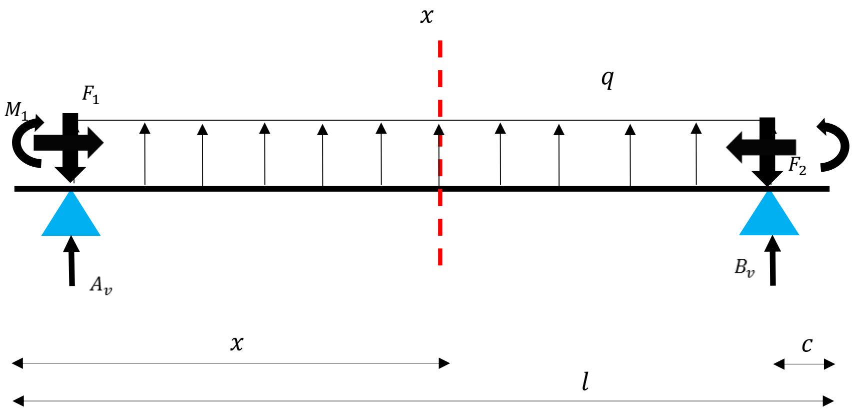
Clamping moment

mxx,Δ	6,2	[kNm]
mxx,tot	6,2	[kNm]

LC 7

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	2290	[kN]
F2	-22826	[kN]
M1	5045	[kNm]
q	-145,2	[kN]
Av	3	[kN]
Bv	3	[kN]
γP	1,00	[-]

Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

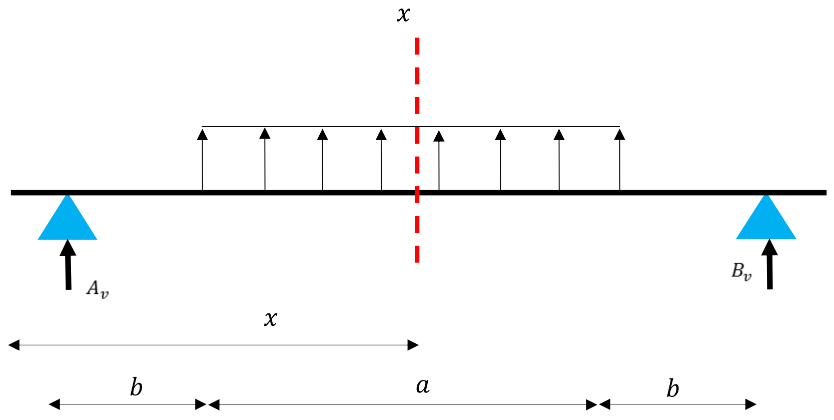
Vz	0,0	[kN]

Bending moment

Mx	-12964,3	[kNm]

Floor loaded

Loading (long. direction)



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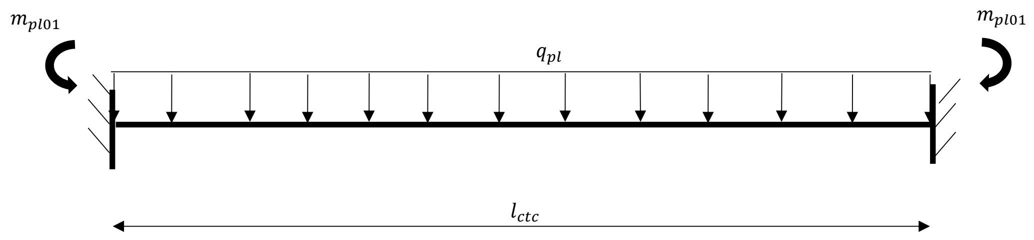


Measurements		
x	16,75	[m]
a	20,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	-56,9	[kN/m]
mpl01	-118,6	[kNm]

Measurements		
lctc	5,0	[m]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,tot	0,0	[kNm]	

Suspension force

Qyy	-56,9	[kN]

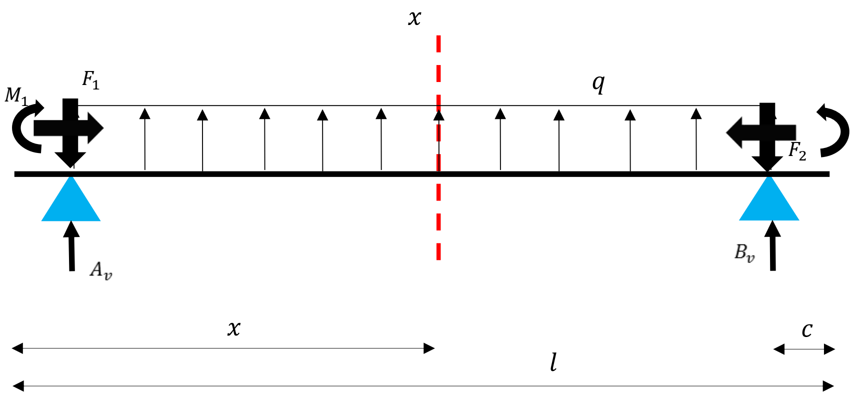
Clamping moment

mxx,alt	-18,5	[kNm]	(due to alternative load case)
mxx,tot	-18,5	[kNm]	

LC 8

Girder loaded

Loading (long. direction)



Load & Reaction forces		
F1	2095	[kN]
F2	-20886	[kN]
M1	4616	[kNm]
q	-133	[kN]
Av	3	[kN]
Bv	3	[kN]
P∞/P0	0,915	[-]
γP	1,00	[-]

Measurements		
x	16,75	[m]
c	1,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Shear Force

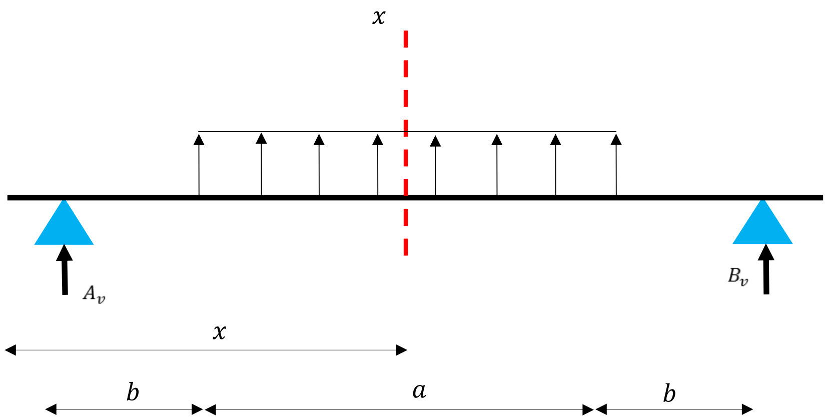
Vz	0,0	[kN]

Bending moment

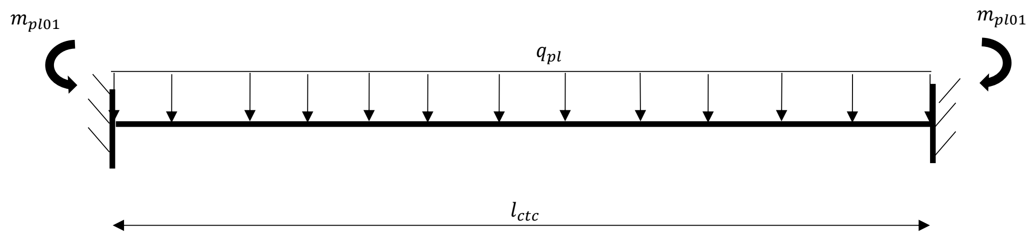
Mx	-11862,4	[kNm]

Floor loaded

Loading (long. direction)



Loading (transverse direction)



Measurements		
x	16,75	[m]
a	20,0	[m]
l	33,5	[m]
lsup.	31,5	[m]

Load & Reaction forces		
qpl	-52,1	[kN/m]
mpl01	-108,5	[kNm]

Measurements		
lctc	5,0	[m]

Torsion

Mxy,alt	0,0	[kNm]	(due to alternative load case)
Mxy,tot	0,0	[kNm]	

Suspension force

Qyy	-52,1	[kN]

Clamping moment

mxx,alt	-16,9	[kNm]	(due to alternative load case)
mxx,tot	-16,9	[kNm]	

Appendix G – SCIA Model vs. Analytical solution

1 Introduction

The design of a concrete through railway bridge is a typical assignment for an engineering firm such as Witteveen+Bos. Nowadays the design loads on the bridge are determined using FEA programs rather than performing time-consuming hand calculations. The risk with FEA programs is that one needs as well a proper understanding of the program as the structural behaviour, in order to obtain correct results. Additionally a good structural engineer should have a critical attitude against the FEA-generated results to verify whether they are correct or not.

In the case of Witteveen+Bos the FEA program *SCIA Engineer* is used to determine the critical design loads. The objective of this appendix is to draw up a comparison between the results generated with SCIA and the analytical solution. Chapter 3 considers a number of load cases for which torsion in the girder is compared between three SCIA models and the analytical solution. Chapter 4 does the same but then focuses on the clamping moment, whereas chapter 5 researches the influence of dimensions on torsion. Finally chapter 6 tries to establish what the accuracy of the SCIA models is compared to the theory.

The ultimate goal of this appendix is to prove that SCIA and the analytical solution compare rather well and that the current way of designing a bridge (using FE models) is a safe and acceptable method.

2 Models

The master thesis of R.T.J. de Groot (4) focuses on torsion in a through bridge and makes the comparison between a 2D and 3D DIANA model and the analytical solution. Because a number of helpful conclusions and recommendation is provided in this thesis, an overview is given in the first part of this chapter. The second part will elaborate on three SCIA models that simulate a through bridge.

2.1 Conclusions & recommendations

- In contradiction to the analytical solution it turns out that self-weight does cause torsion in the girder.
- Torsion in the girder strongly depends on the E-modulus of the floor in transverse direction. By reducing the E-modulus of the floor, torsion in the girder increases.
- According to the analytical solution, the floor is distributed into strips. With the reduced E-modulus of the floor this assumption turns out to be correct. However, under a double line load of 80 kN/m, the values for torsion in the 2D-model remain 50% behind on the theoretical values.
- When the girder is connected to the floor, there is a vertical eccentricity between the two corresponding nodes. This eccentricity causes an additional torsional moment which influences the results. A horizontal connection between these two elements is advised.
- Fully prestressed structures are hardly exposed to torsion due to self-weight. This is because prestress counteracts the deflections and rotations caused by self-weight. But for structures with partial prestressing, torsion due to self-weight might become an issue.
- Under a line-load the distribution of torsion in the FEM model compares rather well to the analytical solution. For both methods torsion approximates a limit value for bridges longer than 40 meters.
- The connection between the girder and the floor turns out to have a large influence on the results. By using solid-elements the exact geometry can be modelled and this problem is excluded. Hence solid-elements lead to better results than 2D-elements. But still the values for torsion in the solid-model (under a double line load of 80 kN/m) remain 30% behind the values obtained by the analytical solution.

2.2 Plate model

The plate model is entirely constructed out of 2D-elements and is designed in a number of steps. The first step is modelling the through floor in the XY-plane with a certain thickness. Then the triangular shaped section of the bridge (voute) is modelled in the XY-plane on the long edge of the floor. Because SCIA automatically connects overlapping nodes, no further action has to be under taken to connect these two plates. The girder is then modelled in the XZ-plane and translated to its correct position. One should notice that the centre-lines of the girder run in Z-direction, where the centre-lines of the other two elements run in Y-direction. Which means the girder is not yet connected to the structure. To solve this problem the NS-Guideline 1015 (17) advises to use a rigid connection between the centreline of the girder and the edge of the triangular shaped section (red line in Figure G-1).

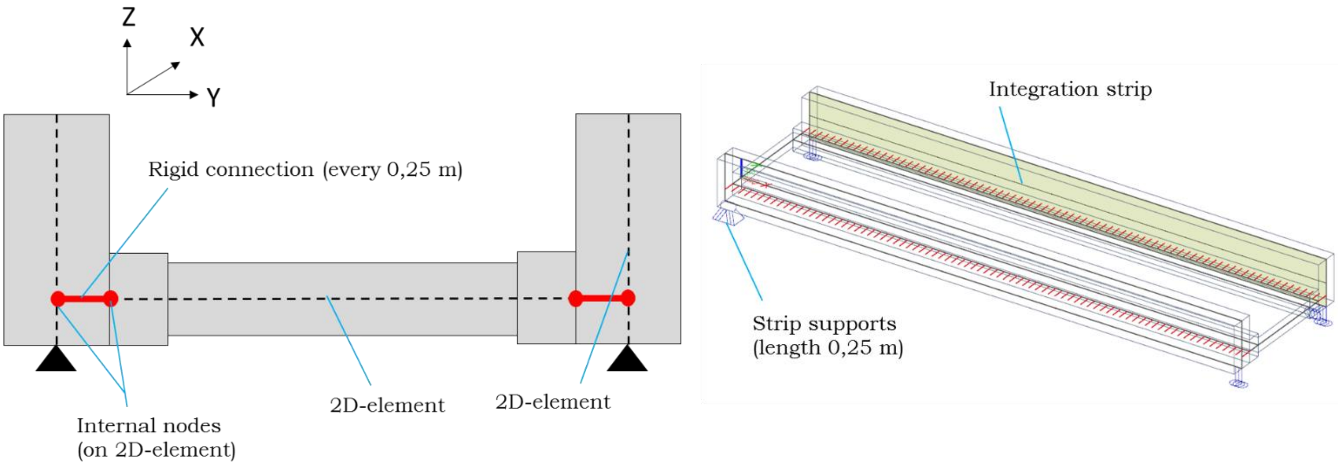


Figure G-1: Cross-section and 3D-view of the plate model

A rigid connection in SCIA means that the rotation for the two connected nodes are identical. Additionally the rotation determines the orientation of the connection line. The deformation of the nodes are identical as well, but due the rotation node 2 undergoes an additional deformation of $0{,}5b * \varphi$.

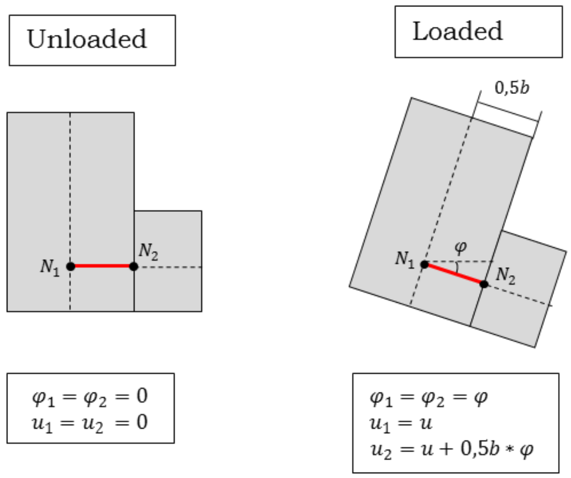


Figure G-2: Deformation behaviour of the rigid connection

The 2D-elements in SCIA are evaluated with 4-node quadrilateral and 3-node triangle elements. Each node has 6 degrees of freedom which represent the displacement and rotations in respectively X, Y and Z direction. Because the plate model only consists of simple forms, the model is evaluated with 4-node square elements. After some trial and error, it turns out that a mesh with squares of 0,25 m forms the right balance between computational time and accurate results.

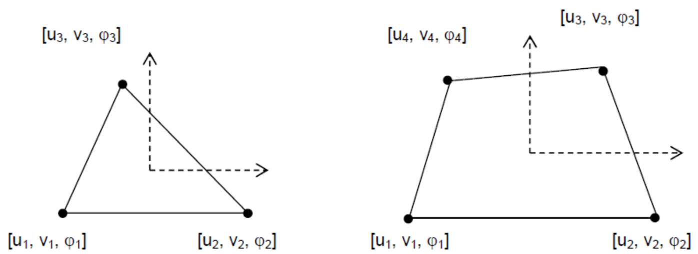


Figure G-3: 3-node triangle element (left) and 4-node quadrilateral element (right)

The application of point supports in the plate model results in large stresses and strains near the supports. To avoid this problem, so called strip supports are applied, which are line supports with a length of 0,25 meters (Figure G-4). With this length the supports coincide with the mesh, eliminating large stresses near the supports.

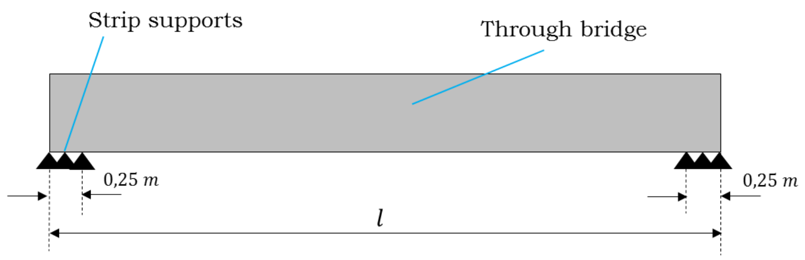


Figure G-4: Strip supports applied in the plate model

A similar problem arises when the rigid connections are applied every meter. The graphs for torsion become rather volatile. To solve this problem, the connections are applied every 0,25 meter, to let the connections coincide with the mesh as well. Evidently this leads to smoothers graphs for torsion.

2.3 Beam model 1A

In beam model 1A the floor is modelled as a 2D-element which spans from centre girder to centre girder. The girder itself is modelled as a 1D-element on the long edge of the floor. The 1D-element consists of 2 nodes (one at the start and one at the end of the girder), where the 2D-element consists of 4 nodes (one on each corner). As mentioned before SCIA automatically connects overlapping nodes, meaning the floor and girder are connected at the corners. The concern however arises that the rotations and deformations of the floor and girder are only coupled at the corners of the structure and not along the entire length. To verify whether this is the case or not, an alternative beam model is introduced in paragraph 2.4 which applies rigid connections between the 1D and 2D-elements.

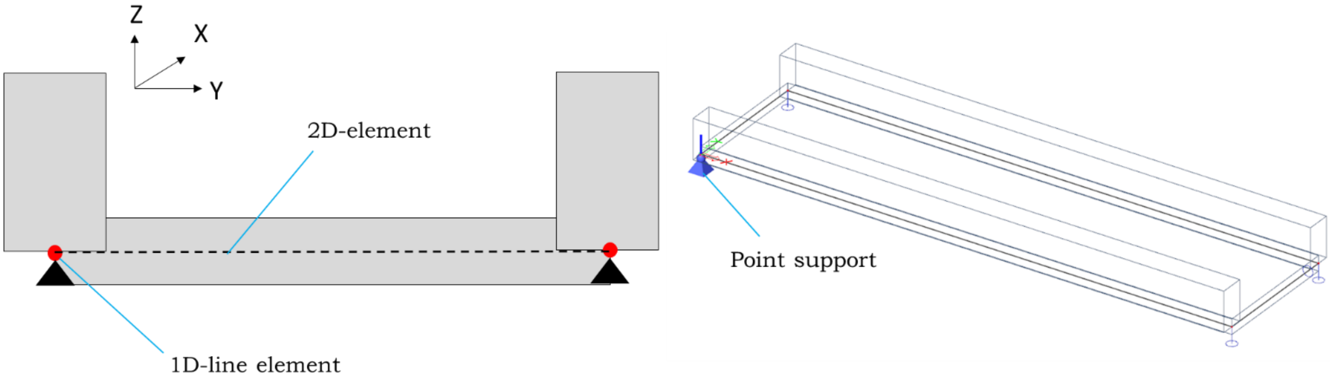


Figure G-5: Cross-section and 3D-view of the beam model 1A

The cross-section entered in SCIA does not have the same dimensions as the original cross-section. The height of the girder is reduced with half the thickness of the floor. Equivalently the width of the floor is increased with the width of the girder (floor now spans from centre to centre). With the spreadsheet in paragraph 8.1 it is established that the cross-sectional area in SCIA is exactly the same whereas the section modulus is 5,6% smaller than the original cross-section. Yet the torsional stiffness of the girder has a value which is 29,7% less than the original value. Which means the shear modulus needs to be manually adjusted to:
$G_{girder} = 18376\ N/mm^2$ instead of $G = 14176\ N/mm^2$.

2.4 Beam model 1B

Beam model 1B forms a combination of the plate model and beam model 1A. The floor and voute are modelled as in the plate model (2D) where the girder is modelled as a line-element (1D). But in contrast to model 1A, the centreline of the girder does not coincide with the centre of the floor. This means that rigid connections between the 1D and 2D-element need to be applied to form a connection between these two elements. To do so, internal nodes are added to the 1D and 2D-elements and rigid connections are formed every ¼ meter. An advantage of this model is that the geometry is no longer simplified compared to the real cross-section and that the shear modulus does not need to be increased manually. However a disadvantage is that the connections formed have a vertical eccentricity, which according to de Groot can induce additional bending and torsional moments.

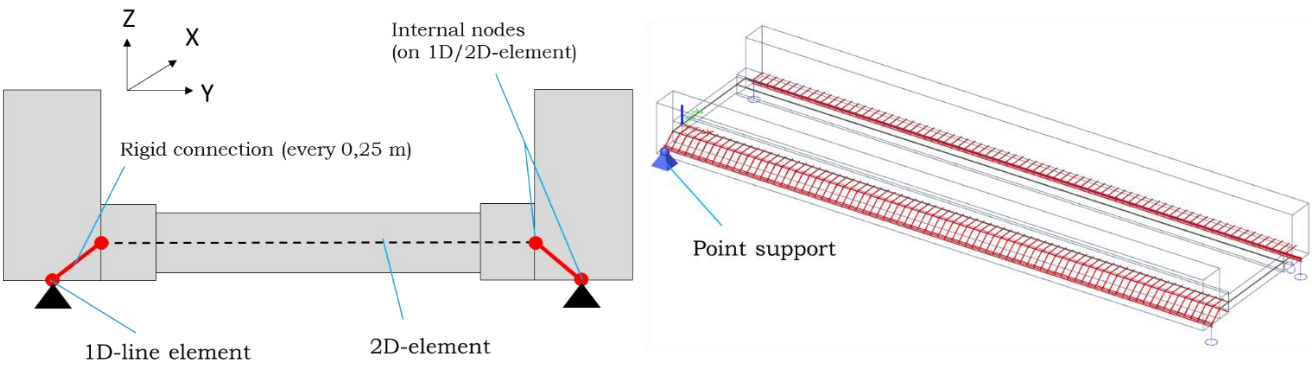


Figure G-6: Cross-section and 3D-view of the beam model 1B

2.5 Mindlin vs. Kirchhoff

The Kirchhoff theory assumes that vertical lines remains straight and perpendicular to the neutral plane of the plate during bending. Therefore normal strains and stress remain perpendicular to the surface. This explains that the shear deformation in an Kirchhoff element is not included ($\gamma_{xz} = \gamma_{yz} = 0$).
Contrastingly the Mindlin theory retains the assumption that vertical lines remain straight, but no longer perpendicular to the neutral plane during bending. Therefore normal strains and stresses do not remain perpendicular to the neutral plane. As a consequence additional strains arise ($\gamma_{xz} = \gamma_{yz} \neq 0$), causing shear deformation.

In principal Kirchhoff elements apply to thin plates where shear deformation can be neglected, whereas Mindlin elements apply to thick plates where shear deformation is of significance. Obviously thick plates are applied in all three SCIA models and shear deformation is essential to determine the correct values for torsion.

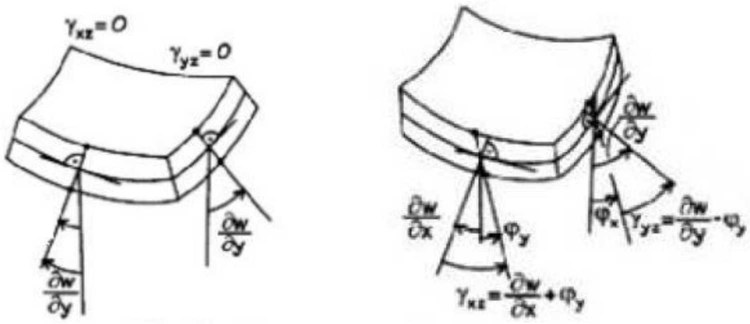


Figure G-7: Kirchhoff element (left) and Mindlin element (right)

3 Torsion

3.1 Distributed mobile load

Let's assume a bridge with a span of 20 meters and the same cross-sectional properties as bridge A, is loaded with a double distributed line load of 100 kN/m. Because this load simulates a train, the line loads are present with 1,50 meters spacing.

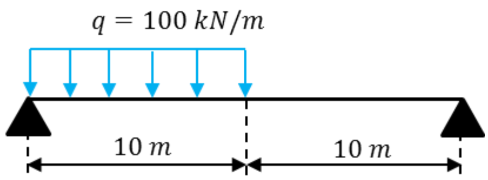


Figure G-8: A double distributed line load of 100 kN/m

Torsion along the length of the girder is plotted for the analytical solution, the plate and beam model 1A and 1B. Noteworthy is that beam model 1B and the plate model follow roughly the same course and beam model 1A stays behind. Even more controversial is the maximum torsion found by the analytical solution which is more than twice as large as the maximum value obtained by SCIA.

Property	Girder	Floor
E	34.000 MPa	34.000 MPa

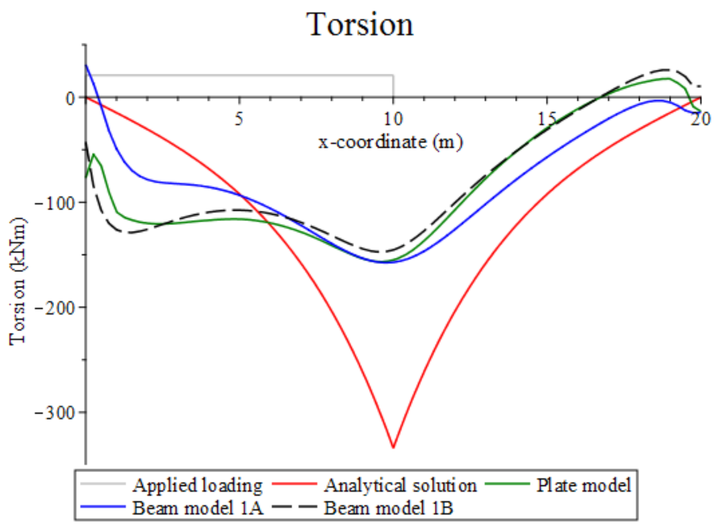


Figure G-9: Torsion due to a double distributed line load (uncracked floor)

During the derivation of the analytical solution (Appendix A) a number of assumptions are made. One of the most important ones is that the floor is divided into strips, meaning the load applied on the floor spreads quickly towards the girders. By reducing the Young's Modulus of the floor, more load goes towards the girders and torsion increases.

According to the Eurocode the Young's Modulus of a cracked section may be reduced to a fictitious value. Table NB.1 in the National Annex to Eurocode 2 (13) holds an expression for these fictitious values. From this table it is derived that the reduced E-modulus for C35/45 is equal to $E_{cracked} = 11.200\ N/mm^2$. To process this assumptions into SCIA, the E-modulus of the floor is reduced to $11.200\ N/mm^2$.

Property	Girder	Floor
E	$34.000\ MPa$	$11.200\ MPa$

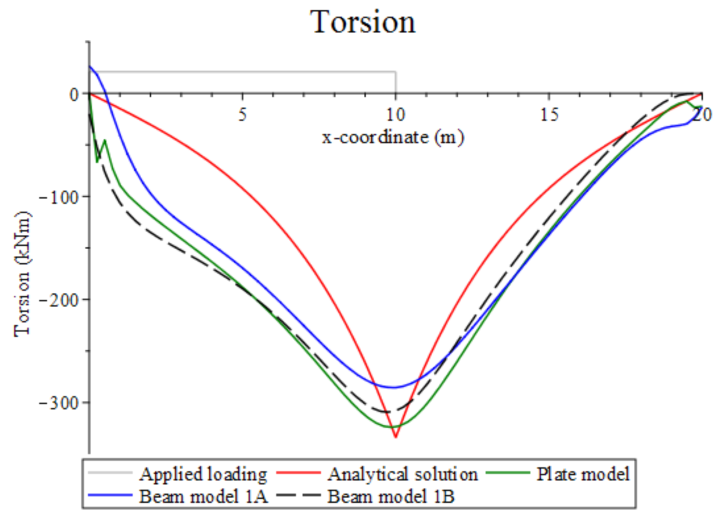


Figure G-10: Torsion due to a double distributed line load (cracked floor)

Figure G-10 shows torsion for the half-loaded bridge, where the floor in SCIA is modelled as cracked. Due to this more load is transferred to the girders causing larger values for torsion. With this modification the plate and beam models in SCIA approximate the strip method used by the analytical solution.

3.2 Local mobile load

The second considered load case is a double local line load of a 100 kN/m, which spans from $x = 7{,}5\ m$ to $x = 12{,}5\ m$. Like the previous load case, the floor is modelled as cracked.

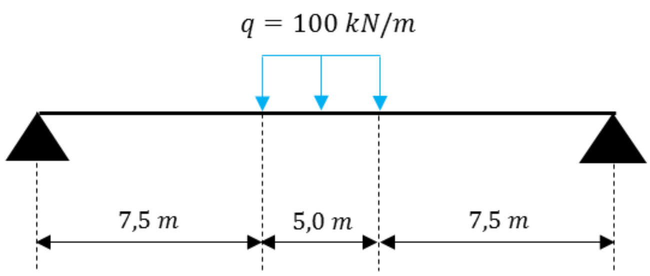


Figure G-11: A double local line load of 100 kN/m

The plate model and beam model 1B compare rather well to the analytical solution and the maximum torsion obtained is roughly the same. Contrastingly beam model 1A stays behind and has a maximum which 40% lower. It is expected that the connection established between the 1D and 2D-element in model 1A results in a loss of stiffness.

Property	Girder	Floor
E	$34.000\ MPa$	$11.200\ MPa$

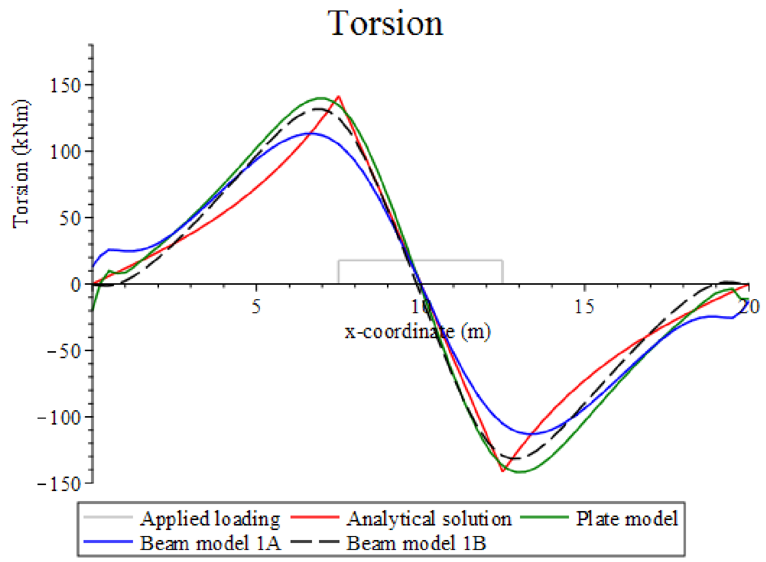


Figure G-12: Torsion due to a local mobile load (cracked floor)

3.3 Self-weight

The analytical solution assumes that for a constant loading (e.g. self-weight) the deflection of the floor and the rotation of the girder are constant. No deviation in rotation means that there is no torsion. However the SCIA models show that the floor (under influence of self-weight) deflects more at midspan than near the supports. Which means there is a deviation in rotation and therefore torsion.

In order to find a graph for the analytical solution, an alternative load case is introduced. A local load is applied over a length a with a value $q_{alternative}$. The sheet in paragraph 8.2 is used to find a value for these two parameters which lead to the same deflection behaviour as the original load case.

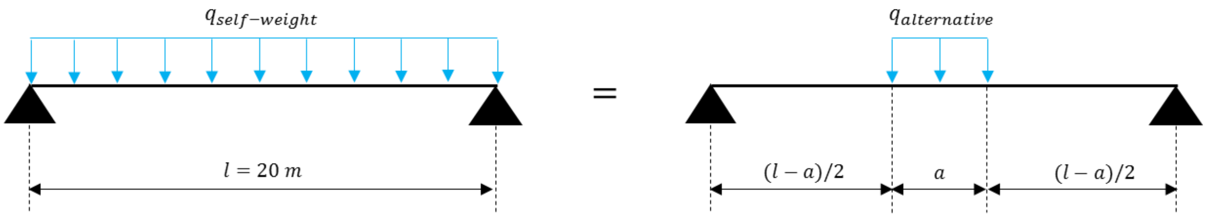


Figure G-13: Alternative load case for self-weight in longitudinal direction

But besides an alternative load case in longitudinal direction, an alternative case in transverse direction is needed as well. The primary load on a strip is established by applying the entire self-weight as a distributed load on the floor.

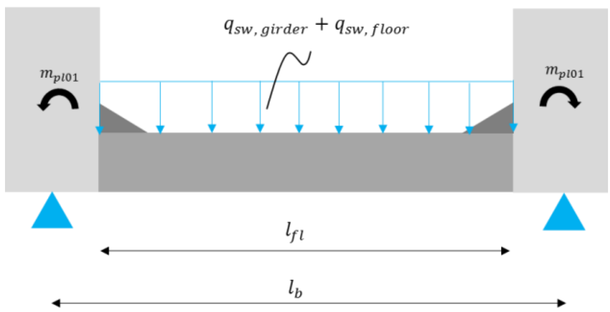


Figure G-14: Alternative load case for self-weight in transverse direction

With some trial and error it is established that, when the alternative load case in longitudinal direction approximates 16 meters with a value of 105% of the original loading ($q_{alternative} = q_{self-weight} * 1{,}05$), that the analytical curve for torsion matches best with the SCIA models.

Property	Girder	Floor
E	$34.000\ MPa$	$11.200\ MPa$

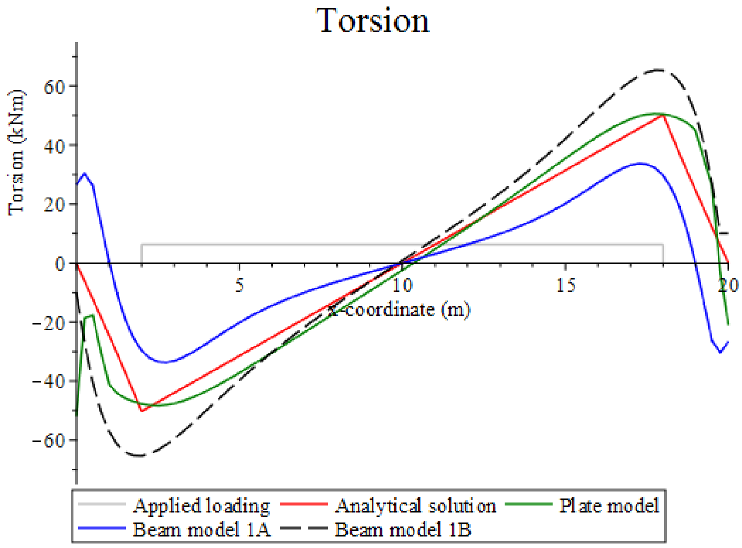


Figure G-15: Torsion due to self-weight (cracked floor)

Figure G-15 shows the graphs for torsion due to self-weight. It turns out the plate model and analytical solution follow the same course and have roughly the same maximum. Beam model 1A shows a graph which remains far behind, where beam model 1B presents a graph which exceeds the analytical solution. This last phenomenon is remarkable, as model 1B is expected to follow the same course as the plate model. But because point supports are applied, large reactional forces are present near the end of the bridge increasing the values for torsion. Unfortunately strip supports are no better solution, because they induce a large counteracting bending moment near the supports which influences the rotation and therefore torsion in the girder.

3.4 Prestress

Prestress basically has the opposite effect on the bridge as self-weight does. It generates an upward deflection and rotation in the girders that counteracts the rotation due to self-weight. But the similarity between the two is that the analytical solution assumes there is no torsion due to constant loading. Therefore an alternative load case needs to be applied.

The prestress consists of a horizontal prestressing force, an upward acting distributed load and bending moment due to a small eccentricity of the tendons. Because torsion is only generated by deflection of the bridge and rotation of the girders, the horizontal force is excluded from this calculation. The downward acting bending moments at the edge of the bridge, leads to a smaller length of the alternative load case. With some trial and error, the same deflection behaviour is found by taking $a = 16\,m$ and $q = 1{,}02 * q_{prestress}$.

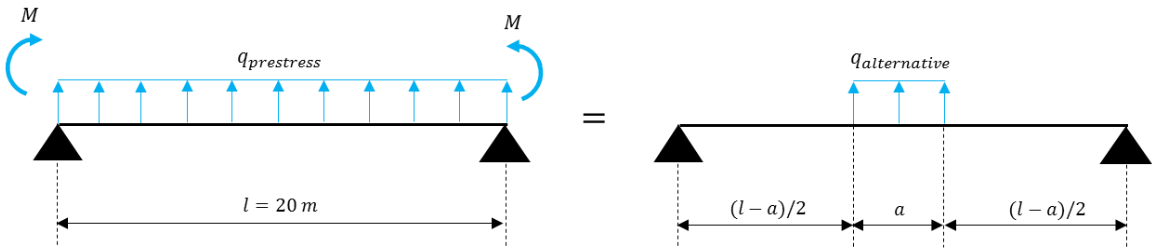


Figure G-16: Alternative load case for prestress in longitudinal direction

The upward acting distributed load is a result of the drape of the tendons and is originally present in the girders. But for this alternative load case, it is increased by 2% and the load is distributed over the floor in order to find a primary load.

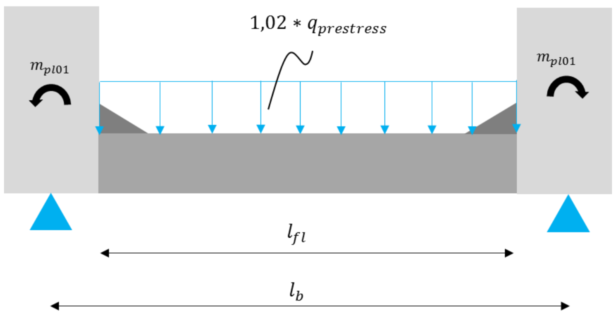


Figure G-17: Alternative load case for prestress in transverse direction

Property	Girder	Floor
E	$34.000\ MPa$	$11.200\ MPa$

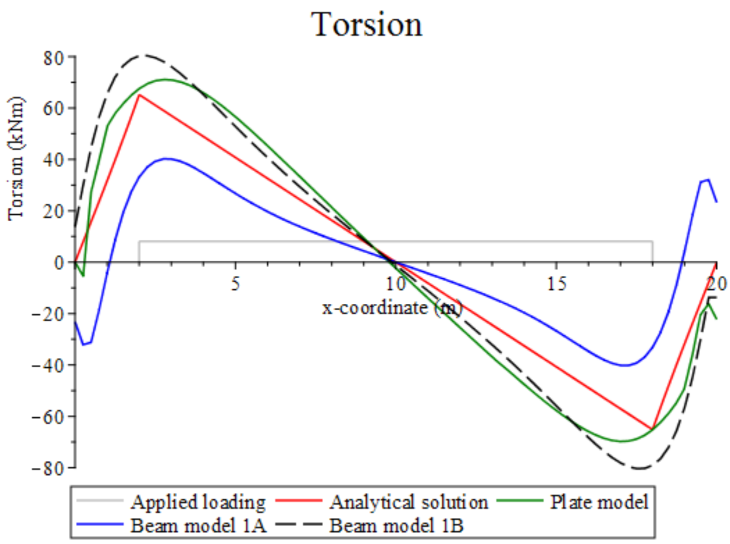


Figure G-18: Torsion due to prestress (cracked floor)

Figure G-18 presents the graphs for torsion in the girder due to prestress. The results are quite similar to torsion due self-weight, but then with an opposite sign. The plate model and analytical solution align pretty well, beam model 1A remains behind and beam model 1B exceeds the analytical solution near the supports. As concluded by R.T.J. de Groot, torsion in fully prestressed structures due to self-weight and prestress is negligible.

3.5 Settlement of supports

It can be likely that one of the supports settles more than others. Therefore a load case is introduced which takes 5 mm of support settlement into account. This subjects the bridge to a deformation in as well longitudinal as transverse direction. But because the analytical solution assumes no load distribution in longitudinal direction, the support settlement case is reduced to a strip $dx = 1{,}0\ m$ with a deflection Δ (Figure G-19). In chapter 3.1 in appendix A the primary load for this case is derived:

$$m_{pl0} = 6EI\Delta/L^2 \qquad [\text{ G.1 }]$$

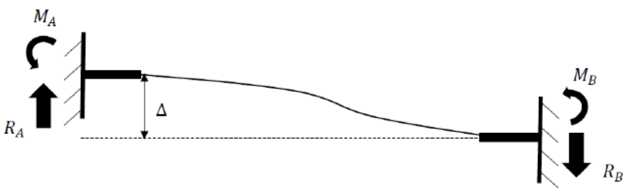


Figure G-19: Transverse section for support settlement load case

Noteworthy in Figure G-20 is that the shape of the analytical solution and the SCIA models compare rather well but that the values for torsion are far off. From the SCIA models it becomes clear that the settlement of the support isn't just a deflection in transverse direction but also in longitudinal direction. This means that the load transfer happens in both directions and that the simplified case by the analytical solution is too conservative.

Property	Girder	Floor
E	34.000 MPa	11.200 MPa

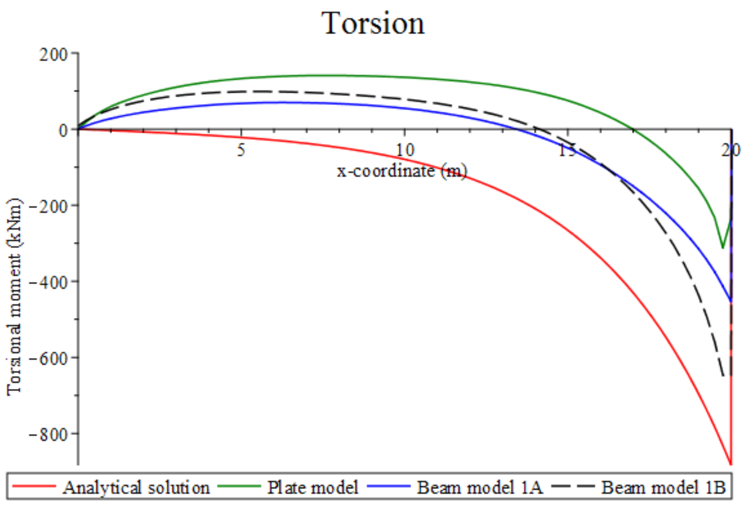


Figure G-20: Torsion due to support settlement (cracked floor)

4 Clamping moment

Besides torsion the clamping moment is an important design load. A function for this load is derived in Appendix A to determine the course along the length of the girder. Because Witteveen+Bos uses SCIA to determine the clamping moment, it is interesting to see whether this matches with the analytical solution or not.

4.1 Distributed line load

The same type of loading as in paragraph 3.1 is considered, a double line load of 100 kN/m on a bridge with a span of 20 meters. In the plate model the clamping moment is derived from the internal forces in the 2D-element that represents the girder. In beam model 1A the clamping moment is obtained by taking a section on the 2D-element which represents the floor. The similarity between these two is that the clamping moment is derived from a 2D-element which is present at the centre of the girder. However in beam model 1B this not possible, because there is no 2D-element present at the centre of the girder. The clamping moment is therefore derived at the interface between the voute (2D) and the girder (1D), see Figure G-30.

Property	Girder	Floor
E	34.000 MPa	11.200 MPa

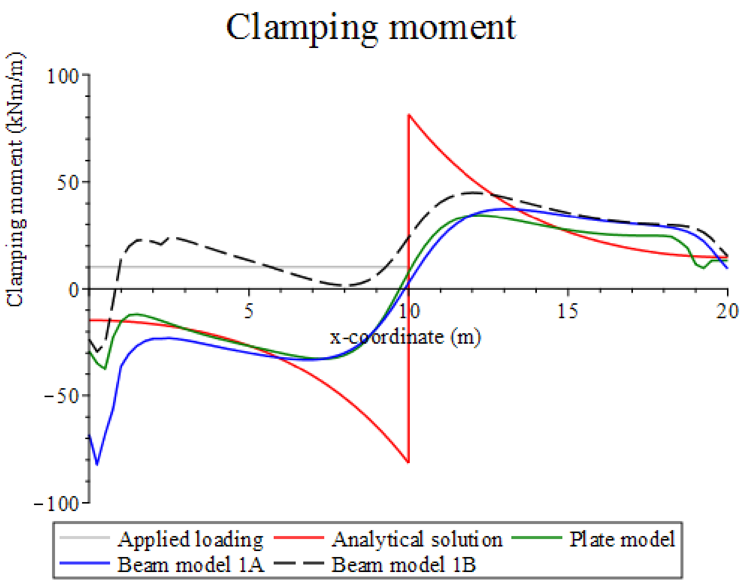


Figure G-21: Clamping moment for a double distributed line load (isotropic)

The analytical solution shows a jump (and sign switch) at the end of the distributed load. This can be explained from the fact that the clamping moment under loading is counteracted by the clamping moment under the unloaded part. Which means that the clamping moment has the same magnitude but with an opposite sign.

According to Figure G-21 the plate and beam model 1A align pretty well. Like the analytical solution these two models show a jump in clamping moment at the loaded-unloaded interface, but the maximums stay a bit behind. Beam model 1B shows a different result, which can be devoted to the fact that the clamping moment is obtained at a different location.

During the derivation of the clamping moment function, the assumption is made that the floor has no bending stiffness in longitudinal direction. Therefore the floor is now modelled as orthotropic (paragraph 8.4), with a cracked E-modulus in transverse direction and an E-modulus of 5.000 MPa in longitudinal direction. This last value is established using the national annex to Eurocode 2, which defines the lower limits for fictitious E-moduli.

Property	Girder	Floor
E_{long}	34.000 MPa	5.000 MPa
E_{trans}	34.000 MPa	11.200 MPa

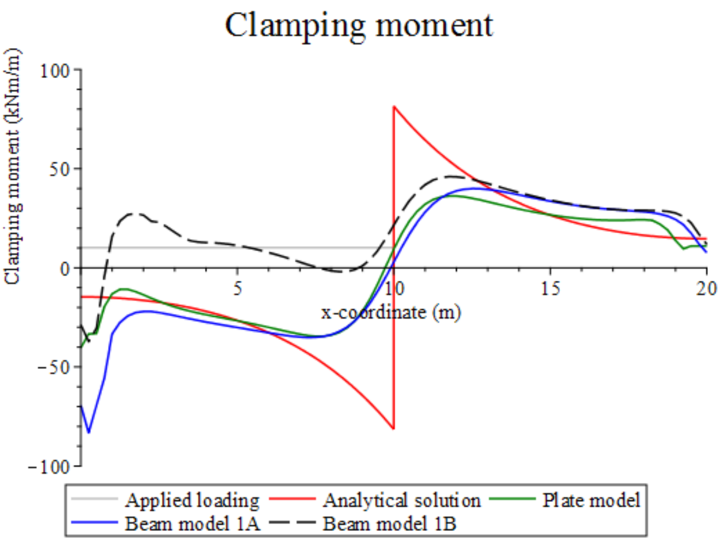


Figure G-22: Clamping moment for a double distributed line load (orthotropic, Elong=5.000 MPa)

As one can see the change in bending stiffness in longitudinal direction has increased the clamping moment slightly. But to see what the effect is when the floor has no longitudinal bending stiffness at all, the E-modulus is further reduced to $E_{long} = 100\ MPa$.

Property	Girder	Floor
E_{long}	34.000 MPa	100 MPa
E_{trans}	34.000 MPa	11.200 MPa

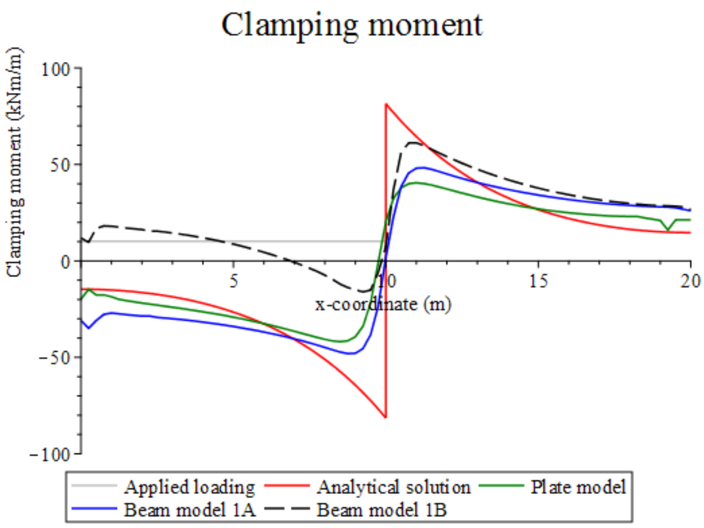


Figure G-23: Clamping moment for a double distributed line load (orthotropic, Elong=100 MPa)

From Figure G-23 it can be concluded that by eliminating the longitudinal bending stiffness of the floor, the clamping moment graphs of the plate and beam model 1A align well with the analytical solution. Beam model 1B is less suitable for determining the clamping moment, because it is not possible to extract results at the centre of the girder.

4.2 Local mobile load

The same load case as in paragraph 3.2 is considered, but this time for the clamping moment. Figure G-24 indicates that under loading the clamping moment is negative. At the interface loaded/unloaded the clamping moment makes a jump and becomes positive again. The explanation for this is similar to the previous paragraph, where the loaded parts of the bridge generate a clamping moment that is counteracted by the unloaded parts.

Property	Girder	Floor
E_{long}	34.000 MPa	100 MPa
E_{trans}	34.000 MPa	11.200 MPa

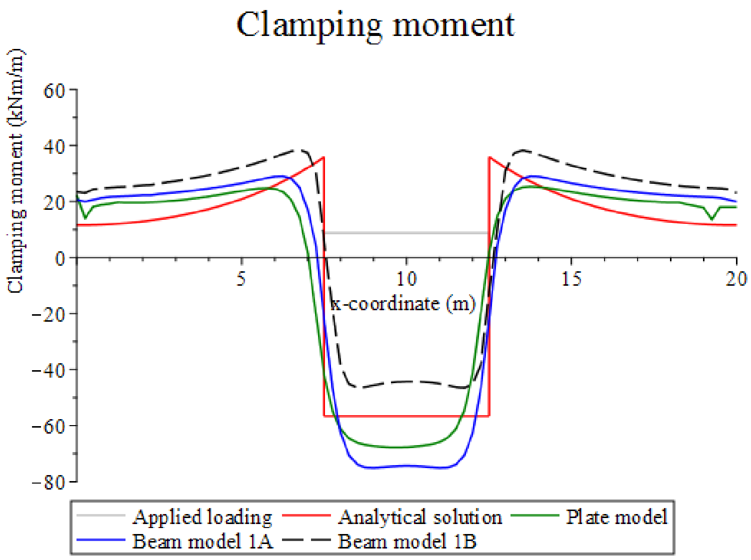


Figure G-24: Clamping moment for a local double distributed line load (orthotropic, Elong=100 MPa)

Under loading the clamping moments in the plate and beam model 1A are larger than in the analytical solution. Unfortunately no direct explanation is found for the fact that these models deliver larger values than the analytical solution. But in practice, the bending stiffness of the floor (in a SCIA model) will not be reduced to a 100 MPa. Which means there will always be some load transfer in longitudinal direction, resulting in less conservative values for the clamping moment. From this point of view the analytical solution will form a safe upper limit. Beam model 1B results in values which remain smaller than the analytical solution, but no conclusions can be drawn from this fact since the results are obtained from at a different location.

5 Dimensions

In this chapter a number of dimensions are changed to see what the effect is on torsion in the girder.

5.1 Length

For different bridge spans ranging from 0 to 40 meters, the maximum torsion for half loaded bridges is researched. For example a bridge of 30 meters is loaded with a double distributed line load of 100 kN/m over 15 meters.

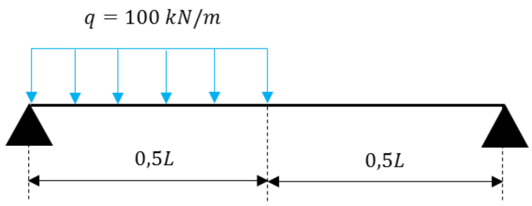


Figure G-25: Distributed line load on half the girder

It can be concluded that for a bridge span between 0 and 30 meters, the maximum value for torsion keeps on increasing with the span. If the bridge span keeps on increasing beyond 30 meters there is no longer an increase in torsion. All three SCIA models and the analytical solution agree on this. But in general torsion in SCIA remains behind on the analytical solution for this mobile load case.

Property	Girder	Floor
E	$34.000\ MPa$	$11.200\ MPa$

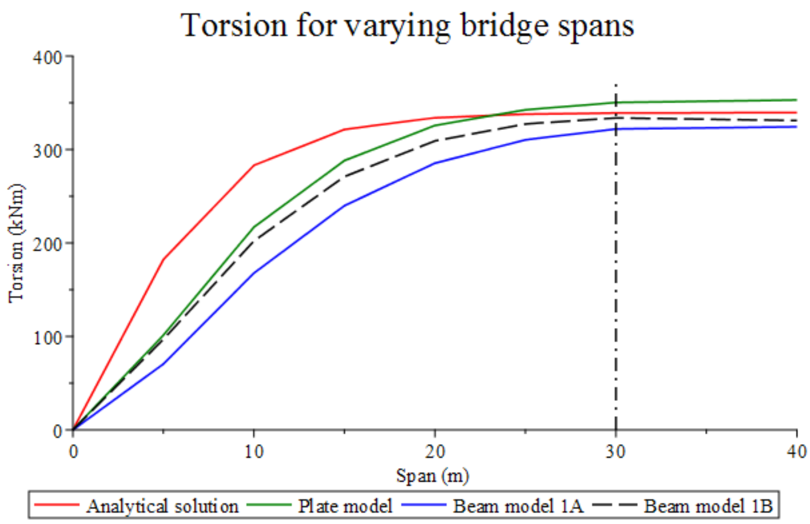
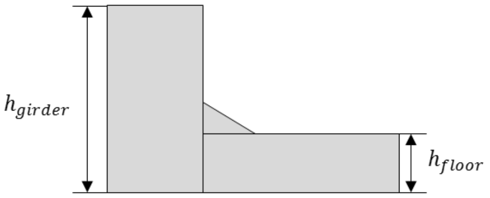


Figure G-26: Maximum torsion for different bridge spans (cracked floor)

5.2 Height ratio

Again let's consider the bridge and load case from paragraph 3.1. This paragraph focuses on the influence of the girder height and floor thickness on the maximum torsion, by expressing these two dimensions in the form of a ratio: h_{girder}/h_{floor} .

Property	Girder	Floor
E	$34.000\ MPa$	$11.200\ MPa$



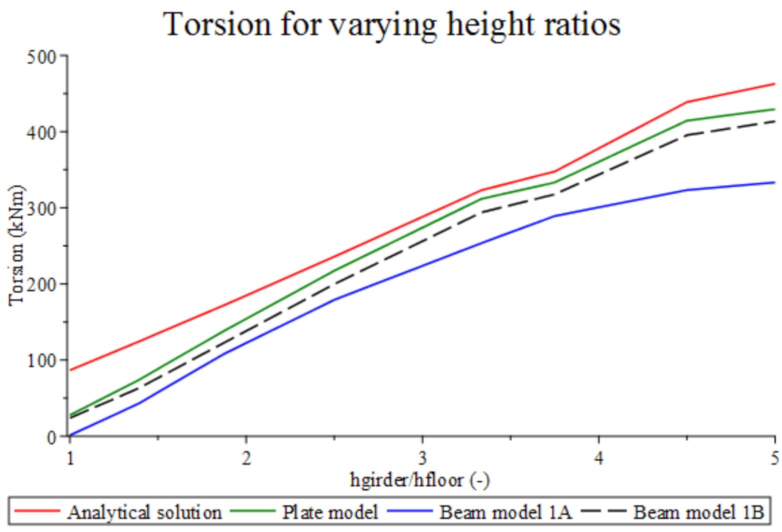


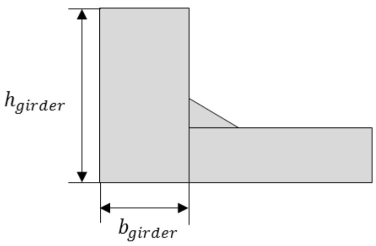
Figure G-27: Maximum torsion for a half loaded girder with varying girder height and floor thickness (cracked floor)

With an increase in height ratio, the maximum torsion increases as well. It turns out there is linear correlation between the height ratio and the maximum obtained torsion. This can be explained from the point of view that a larger height ratio corresponds with a relatively stiffer girder. Basic mechanics teaches one that stiffer parts in the structure always attract more load, which in this case increases torsion in the girder.

5.3 Girder height & width

One last time the half-loaded bridge from paragraph 3.1 is considered. This time the girder height and width are varied, to observe the influence on the maximum torsion. In order to properly understand what the effect of both parameters is, they are varied separately.

Property	Girder	Floor
E	$34.000\ MPa$	$11.200\ MPa$



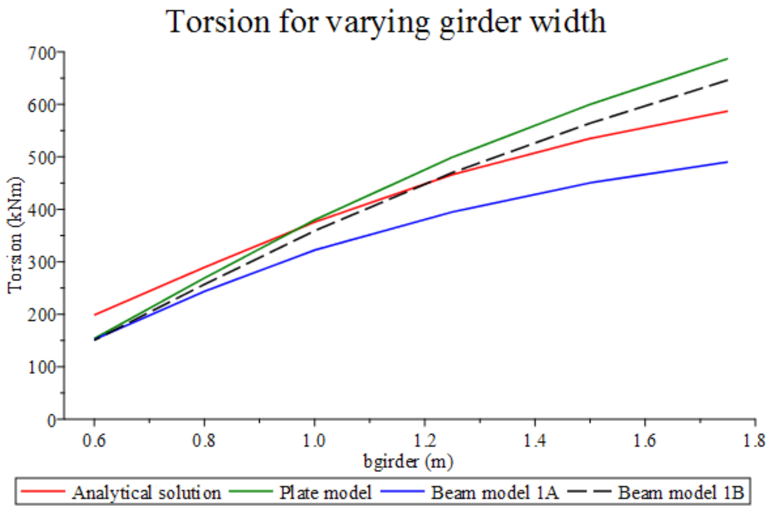


Figure G-28: Maximum torsion for increasing girder width

Property	Girder	Floor
E	$34.000\ MPa$	$11.200\ MPa$

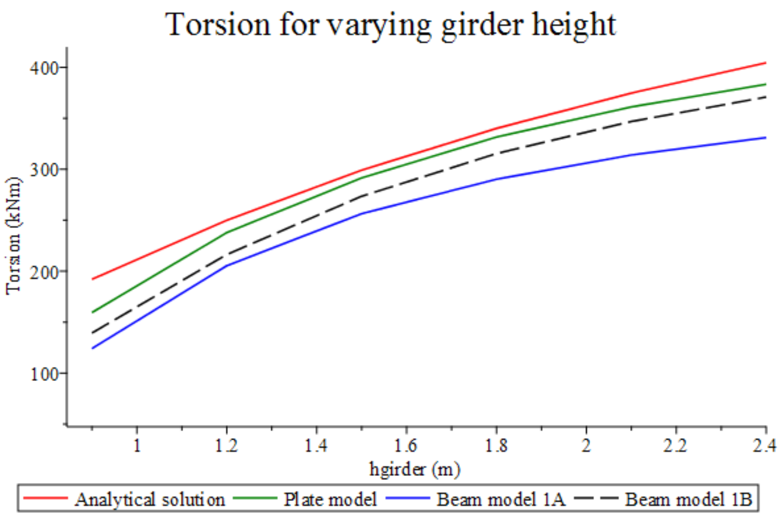


Figure G-29: Maximum torsion for increasing girder height

The three SCIA models and the analytical solution agree on the fact that whenever the height or the width of the girder increases, the maximum torsion increases as well. As mentioned before, when girder dimensions increase, it becomes a relatively stiffer part of the structure attracting more torsion.

6 Deviation of the model

The main objective of this appendix is drawing up a comparison between the analytical solution and the three SCIA models, with respect to torsion in a through girder. This chapter will focus on the deviation between SCIA and the analytical solution, by considering the governing combination of self-weight, ballast, LM71 and prestress acting on bridge A and B. Because the support settlement load case gives unrealistically high values for torsion, it is not considered in this calculation. In order to get a fair comparison, the cross-sectional properties will be fixed throughout this research. Which means that the girder is assumed to remain uncracked ($E = 34.000\ MPa$) whereas the floor is assumed to be cracked ($E = 11.200\ MPa$).

The difference in torsion between SCIA and the analytical solution is established in a section at 0,8d. But the values for torsion in this section can deviate, due to for example the influence of the supports. Therefore it is decided to also take into account the maximum torsion in the through girder, which is not bound to a specific location.

Table G-1: Bridge A: The difference in torsion between SCIA and the analytical solution at 0,8d

LC	Type	Torsion [kNm]				Difference [%]		
		Analytical	Plate	Beam		Plate	Beam	
				1A	1B		1A	1B
1	Self-weight	-93,0	-55,5	-32,5	-77,0	-40%	-65%	-17%
2	Ballast	-81,0	-79,8	-59,9	-80,9	-1%	-26%	0%
3 + 5a	Load model 71	-195,0	-160,5	-124,9	-184,9	-18%	-36%	-5%
9	Prestress	34,0	52,3	29,7	76,8	54%	-13%	126%

Table G-2: Bridge A: The difference in maximum torsion between SCIA and the analytical solution

LC	Type	Maximum Torsion [kNm]				Difference [%]		
		Analytical	Plate	Beam		Plate	Beam	
				1A	1B		1A	1B
1	Self-weight	-117,0	-59,6	-36,7	-77,2	-49%	-69%	-34%
2	Ballast	-108,9	-82,5	-59,9	-84,3	-24%	-45%	-23%
3 + 5a	Load model 71	-272,9	-271,8	-219,3	-261,5	0%	-20%	-4%
9	Prestress	65,2	62,4	36,5	78,3	-4%	-44%	20%

Table G-1 presents torsion at 0,8d whereas Table G-2 shows the maximum torsion in the girder. The first impression one gets, is that the results from the plate model and beam model 1B are closer to the analytical solution than the results from the beam model 1A. One can also conclude, by taking a closer look at Table G-1, that there is a large difference between the analytical solution and the plate and beam model 1B for the load case prestress. Because the results in this table are generated at 0,8d, the supports are likely to have an influence. Therefore more value is put to Table G-2 which considers maximum torsion in the girder.

Table G-3: Bridge A: The total deviation of maximum torsion in the plate and beam model 1A & 1B

LC	Type	% of Total loading	Deviation to analytical solution (%)		
			Plate	Beam	
				1A	1B
1	Self-weight	20,7%	-10%	-14%	-7%
2	Ballast	19,3%	-5%	-9%	-4%
3+5a	Load model 71	48,4%	0%	-10%	-2%
9	Prestress	11,6%	0%	-5%	2%
	Total	100,0%	**-16%**	**-38%**	**-11%**

Because not every load case has the same contribution to the total torsion, a weighed average is introduced to determine the total deviation. For example when considering the maximum torsion in bridge A, self-weight results in a deviation of -49% in the plate model (Table G-2). By taking into account that self-weight generates 20,7% of the total torsion, the deviation caused by this load case is $-49\% * 0{,}207 = -10\%$. Ultimately this results, for maximum torsion in bridge A, in a total deviation of -16% for the plate model, -38% for the beam model 1A and -11% for beam model 1B (Table G-3).

Table G-4: Bridge A & B: The total deviation of torsion in the plate and beam model 1A & 1B at 0,8d

LC	Bridge A - Deviation in torsion (%)			Bridge B - Deviation in torsion (%)		
	Plate	Beam		Plate	Beam	
		1A	1B		1A	1B
Self-weight	-9%	-15%	-4%	-10%	-23%	-8%
Ballast	0%	-5%	0%	0%	-7%	1%
Load model 71	-9%	-17%	-3%	6%	-5%	13%
Prestress	5%	-1%	11%	1%	-4%	11%
Total deviation [%]	**-14%**	**-39%**	**4%**	**-4%**	**-39%**	**17%**

Table G-5: Bridge A & B: The total deviation of maximum torsion in the plate and beam model 1A & 1B

LC	Bridge A - Deviation in torsion (%)			Bridge B - Deviation in torsion (%)		
	Plate	Beam		Plate	Beam	
		1A	1B		1A	1B
Self-weight	-10%	-14%	-7%	-9%	-17%	-4%
Ballast	-5%	-9%	-4%	-2%	-6%	-2%
Load model 71	0%	-10%	-2%	2%	-9%	1%
Prestress	0%	-5%	2%	-1%	-9%	1%
Total deviation [%]	**-16%**	**-38%**	**-11%**	**-9%**	**-41%**	**-4%**

Respectively the total deviation of the plate, beam model 1A and 1B goes to 10-15%, 40% and 10% (as seen in Table G-4 and Table G-5). At first it looks like beam model 1B approximates the analytical solution slightly better than the plate model. But with the knowledge from chapter 3 the contrary can be concluded. This is due to the fact that for self-weight and prestress, beam model 1B exceeds the analytical solution and thereby compensates for its exactly larger deviation from the analytical solution.

Beam model 1A shows a deviation of 40% from the analytical solution. The most plausible explanation for this lies in the fact that no rigid connections are applied in beam model 1A, resulting in a loss of bending and torsional stiffness at the 1D (girder) and 2D (floor) interface.

R.T.J. de Groot concluded that 2D and 3D models of a through bridge remain respectively 50% and 30% behind on the analytical solution. However it should be noted that de Groot considered statically undetermined structures with end-cross members, where this thesis focuses on statically determined structures without end-cross members. Additionally de Groot was looking for the best possible FE model for a through bridge rather than the deviation between the theory and a FE model. Which means both researches have a different scope and purpose and cannot be compared one on one.

Finally one should realize that the analytical solution forms a safe upper limit when calculating torsion in a through bridge. The strip method and the negligence of the longitudinal bending stiffness of the floor are quite conservative assumptions. In reality a load on the floor will not only spread in transverse but also in longitudinal direction. SCIA on the other hand has the ability to distribute loads in both directions and is therefore closer to the real structural behaviour.

7 Conclusions & Recommendations

7.1 Conclusions

Based on the results found in this appendix the following conclusions can be drawn:

- Modelling the floor as cracked, by reducing the E-modulus to roughly a third ($E_{floor} = 11.200\,MPa$), makes the plate and beam models approximate the strip method of the analytical solution.
- As opposed to the analytical solution, self-weight and prestress do cause torsion in the girder. By making use of an alternative load case, a graph for torsion can even be found for the analytical solution. But one should keep in mind that for fully prestressed structures the deflections and rotations of these two load cases cancel each other out, resulting in hardly any torsion.
- A support settlement leads to maximum torsion at the location of settling which gradually decreases towards the unloaded support. In contrast to SCIA the analytical solution assumes no load spread in longitudinal direction for this load case, resulting in values for torsion which are too conservative.
- One of the assumptions during the derivation of the analytical solution is that the bending stiffness of the floor in longitudinal direction is negligible. In SCIA the floor is modelled as orthotropic with hardly any bending stiffness in longitudinal direction. This results in values for the clamping moment which compare rather well to the analytical solution for the plate and beam model 1A.
- A simply supported through bridge, loaded over half the length, shows increasing values for torsion up to a span of 30 meters.
- Increasing the girder height or width, makes it a relatively stiffer part of the structure. Stiffer parts of a structure attract more load, resulting in an (almost) linear relation between these dimensions and maximum obtained torsion.
- Applying a governing combination of self-weight, ballast, LM71 and prestress on bridge A and B, results in values for torsion which are 10-15%, 40% and 10% smaller than the analytical solution for respectively the plate, beam model 1A and 1B. However beam model 1B exceeds the analytical solution for the self-weight and prestress load case due to large reactional forces at the supports and therefore compensates for its deviation.
- R.T.J. de Groot concluded that a 2D and 3D model of a through bridge results in values for torsion which remain respectively 50% and 30% behind the analytical solution. However de Groot focussed on the best possible FE model and considered statically undetermined structures with end-cross members.
- There is a clear difference in deviation between beam model 1A and 1B. Because model 1A is constructed without rigid connections, there is a loss in bending and torsional stiffness of the girder, which results in a larger deviation from the analytical solution than beam model 1B.
- The analytical solution assumes the bridge to be divided into small strips and neglects the longitudinal bending stiffness of the floor. In reality loads will be as well distributed in longitudinal as transverse direction and the analytical solution therefore needs to be considered as a safe upper limit.

7.2 Recommendations

For modelling a through bridge in SCIA the plate model is recommended over the two beam models. Three reasons are provided to support this recommendation:

- **Deviation:** As mentioned in paragraph 7.1, beam model 1A remains 40% behind the analytical solution due to the absence of rigid connections. The plate and beam model 1B remain respectively 10-15% and 10% behind the analytical solution. However beam model 1B exceeds the analytical solution for the self-weight and prestress load case due to large reactional forces at the supports and therefore compensates for its deviation. Additionally chapter 3 shows that the plate model aligns better with the analytical solution than beam model 1B.

- **Clamping moment:** To find the clamping moment in the through girder 2D-elements are needed. The plate model is entirely constructed of 2D-elements and the clamping moment is established from the centre of the girder (section A-A in Figure G- 30). Beam model 1B uses a 1D-element to model the girder and therefore does not have the ability to compute the clamping moment at the centre. As an alternative the clamping moment is derived at the interface between the girder and the floor (section B-B), which is not in agreement with the analytical solution which assumes the floor to be fully restrained in the centre of the girder. Additionally chapter 4 shows beam model 1B is not very suitable for determining clamping moments.

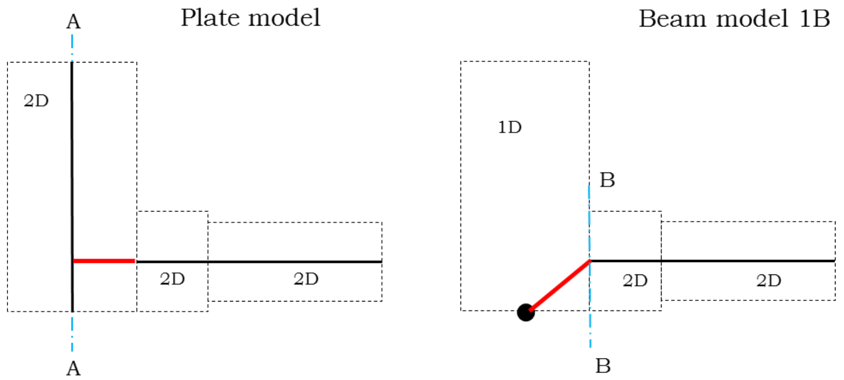


Figure G- 30: Sections used in plate model and beam model 1B to determine clamping moments

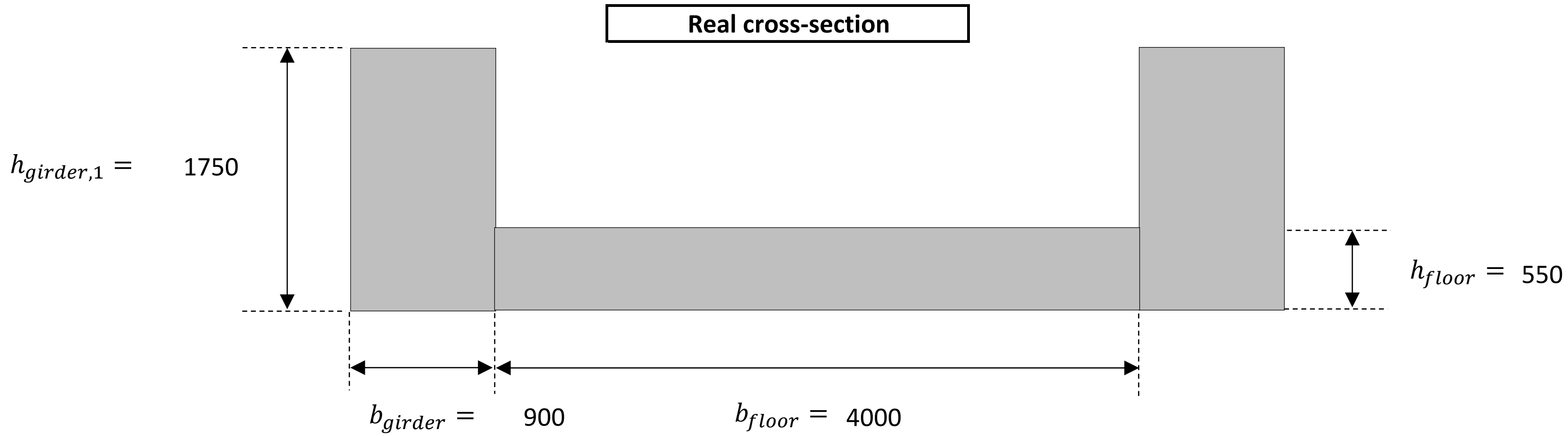
- **Modelling time:** Each internal node that is applied on a 1D-element has to be inserted manually. Consequently each rigid connection between the 1D and 2D-element has to be applied manually as well. The plate model is constructed by applying one rigid connection which is duplicated multiple times. Hence the beam model 1B is much more time consuming to model than the plate model.

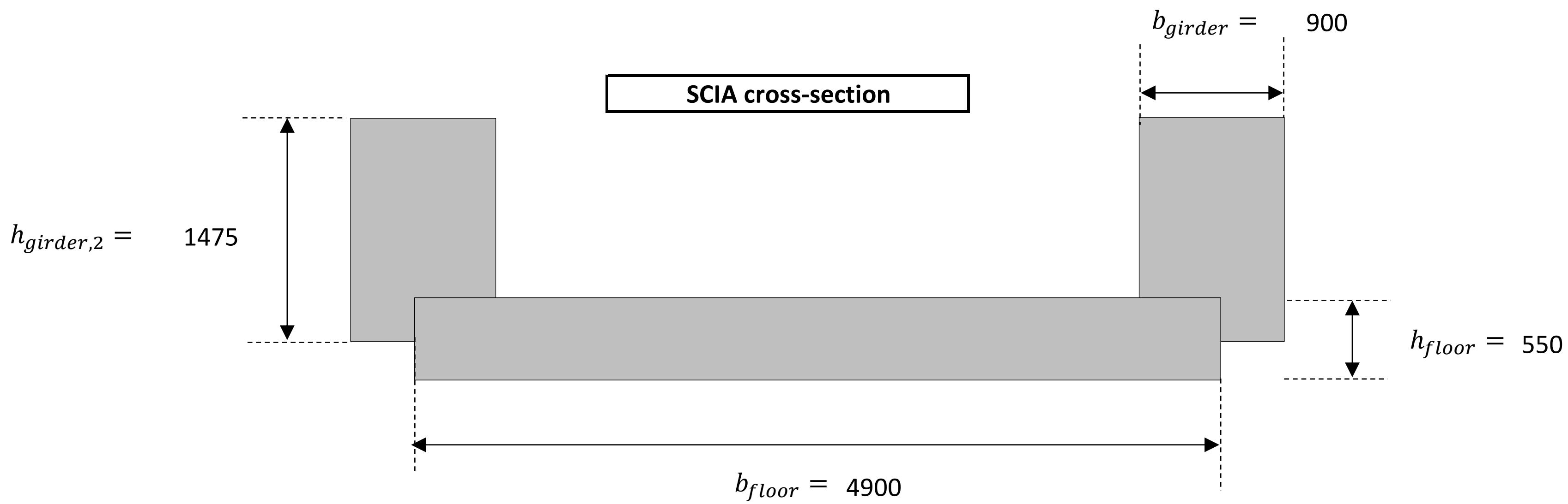
To conclude, the plate model is recommended to model a through bridge. But because this model still forms a simplification of the real structure, it is strongly recommended another graduate student models the through bridge in 3D. It is expected that a 3D model generates better results for torsion, because the geometry of the structure is no longer simplified. With this model a better understanding of the differences between the analytical solution and the real structural behaviour is obtained.

8 Attachments

8.1 Cross-sectional properties

Cross-sectional dimensions





Cross-sectional properties (2/2)

Area

A_{real}	=	5,35E+06	[mm2]
A_{SCIA}	=	5,35E+06	[mm2]
Δ_{Area}	=	0,00	[%]

Position neutral axis

Real cross-section

A_1	=	1,58E+06	[mm2]	y_1	=	875	[mm]	$A_1 * y_1$	=	1,38E+09	[mm3]
A_2	=	2,20E+06	[mm2]	y_2	=	275	[mm]	$A_2 * y_2$	=	6,05E+08	[mm3]
A_3	=	1,58E+06	[mm2]	y_3	=	875	[mm]	$A_3 * y_3$	=	1,38E+09	[mm3]
$\sum$	=	5,35E+06	[mm2]	$\sum$	=	2,03E+03	[mm]	$\sum$	=	3,36E+09	[mm3]

Neutral axis above bottom fibre = 628 [mm]

SCIA cross-section

A_1	=	1,33E+06	[mm2]	y_1	=	1012,5	[mm]	$A_1 * y_1$	=	1,34E+09	[mm3]
A_2	=	2,70E+06	[mm2]	y_2	=	275	[mm]	$A_2 * y_2$	=	7,41E+08	[mm3]
A_3	=	1,33E+06	[mm2]	y_3	=	1012,5	[mm]	$A_3 * y_3$	=	1,34E+09	[mm3]
$\sum$	=	5,35E+06	[mm2]	$\sum$	=	2,30E+03	[mm]	$\sum$	=	3,43E+09	[mm3]

Neutral axis above bottom fibre = 641 [mm]

Cross-sectional properties (2/2)

Moment of inertia

Real cross-section			
$I_{yy,girder}$	=	4,98E+11	[mm4]
$I_{yy,floor}$	=	3,3E+11	[mm4]
$I_{yy,tot}$	=	1,33E+12	[mm4]

SCIA cross-section			
$I_{yy,girder}$	=	4,24E+11	[mm4]
$I_{yy,floor}$	=	4,29E+11	[mm4]
$I_{yy,tot}$	=	1,28E+12	[mm4]

Sectional modulus

Real cross-section			
W_{top}	=	1,18E+09	[mm3]
W_{bottom}	=	2,11E+09	[mm3]

SCIA cross-section			
W_{top}	=	1,15E+09	[mm3]
W_{bottom}	=	1,99E+09	[mm3]

Difference		
ΔW_{top}	=	-2,59%
ΔW_{bottom}	=	-5,60%

Torsional stiffness

Real cross-section			
a_{girder}	=	875	[mm]
b_{girder}	=	450	[mm]
a_{floor}	=	2000	[mm]
b_{floor}	=	275	[mm]
$I_{t,girder}$	=	2,88E+11	[mm4]

SCIA cross-section			
a_{girder}	=	737,5	[mm]
b_{girder}	=	450	[mm]
a_{floor}	=	2450	[mm]
b_{floor}	=	275	[mm]
$I_{t,girder}$	=	2,22E+11	[mm4]

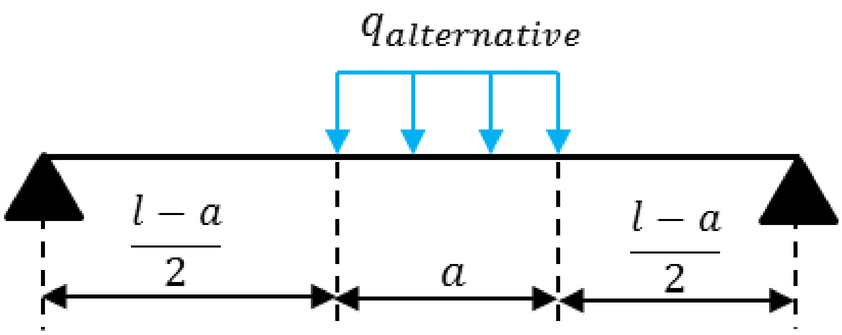
Modified shear modulus

E_{cm}	=	34000	[Mpa]
v	=	0,20	[-]
G	=	14167	[Mpa]
$I_{t,real}/I_{t,SCIA}$	=	1,2971	[-]
$G_{modified}$	=	18376	[Mpa]

8.2 Deflection due to self-weight

restart;

Deflection due to self-weight & alternative load-case:



Which values are chosen for length *a* and alternative load *qalternative,* to obtain a similar deflection behaviour as the self-weight load case.

Parameters

$E := 34000 : \ Iy := 1.3265 \cdot 10^{12} : \quad EI := E \cdot Iy :$

Alternative load over length a:

$a := 18000 : \ l := 20000 :$

Distributed load due to self-weight:

$qselfweight := 137.5 :$

Distributed load due to alternative load case:

$qalternative := 140 :$

Functions for alternative load case

Displacement functions

$w1 := c1 + c2 \cdot x + c3 \cdot x^2 + c4 \cdot x^3 :$

$w2 := d1 + d2 \cdot x + d3 \cdot x^2 + d4 \cdot x^3 + \frac{qalternative \cdot x^4}{24 \cdot EI} :$

$w3 := e1 + e2 \cdot x + e3 \cdot x^2 + e4 \cdot x^3 :$

Rotation functions

$phi1 := -diff(w1, x) :$
$phi2 := -diff(w2, x) :$

$phi3 := -diff(w3, x) :$

Bending moment functions

$M1 := EI \cdot diff(phi1, x) :$
$M2 := EI \cdot diff(phi2, x) :$
$M3 := EI \cdot diff(phi3, x) :$

Shear force functions

$V1 := diff(M1, x) :$
$V2 := diff(M2, x) :$
$V3 := diff(M3, x) :$

Functions for self-weight

Displacement functions

$$w7 := j1 + j2 \cdot x + j3 \cdot x^2 + j4 \cdot x^3 + \frac{qselfweight \cdot x^4}{24 \cdot EI} :$$

Rotation functions

$phi7 := -diff(w7, x) :$

Bending moment functions

$M7 := EI \cdot diff(phi7, x) :$

Shear force functions

$V7 := diff(M7, x) :$

Boundary conditions for alternative load case

Boundary conditions at x=0

$x := 0 :$
$eq1 := w1 = 0 :$
$eq2 := M1 = 0 :$

Transition condition at x=(l-a)/2

$x := \frac{(l-a)}{2}$:
$eq3 := w1 = w2$:
$eq4 := phi1 = phi2$:
$eq5 := M1 = M2$:
$eq6 := V1 = V2$:

Transition condition at x=(l+a)/2

$x := \frac{(l+a)}{2}$:
$eq7 := w2 = w3$:
$eq8 := phi2 = phi3$:
$eq9 := M2 = M3$:
$eq10 := V2 = V3$:

Boundary conditions at x=L
$x := l$:
$eq11 := w3 = 0$:
$eq12 := M3 = 0$:

Boundary conditions for self-weight

Boundary conditions at x=0

$x := 0$:
$eq25 := w7 = 0$:
$eq26 := M7 = 0$:

Boundary conditions at x=L
$x := l$:
$eq27 := w7 = 0$:
$eq28 := M7 = 0$:

Solution

$solution := solve(\{eq1, eq2, eq3, eq4, eq5, eq6, eq7, eq8, eq9, eq10, eq11, eq12, eq25, eq26, eq27, eq28\}, \{c1, c2, c3, c4, d1, d2, d3, d4, e1, e2, e3, e4, j1, j2, j3, j4\})$:
$assign(solution)$:
$x :='x'$:

Graphs

Displacement graph

$with(plots):$

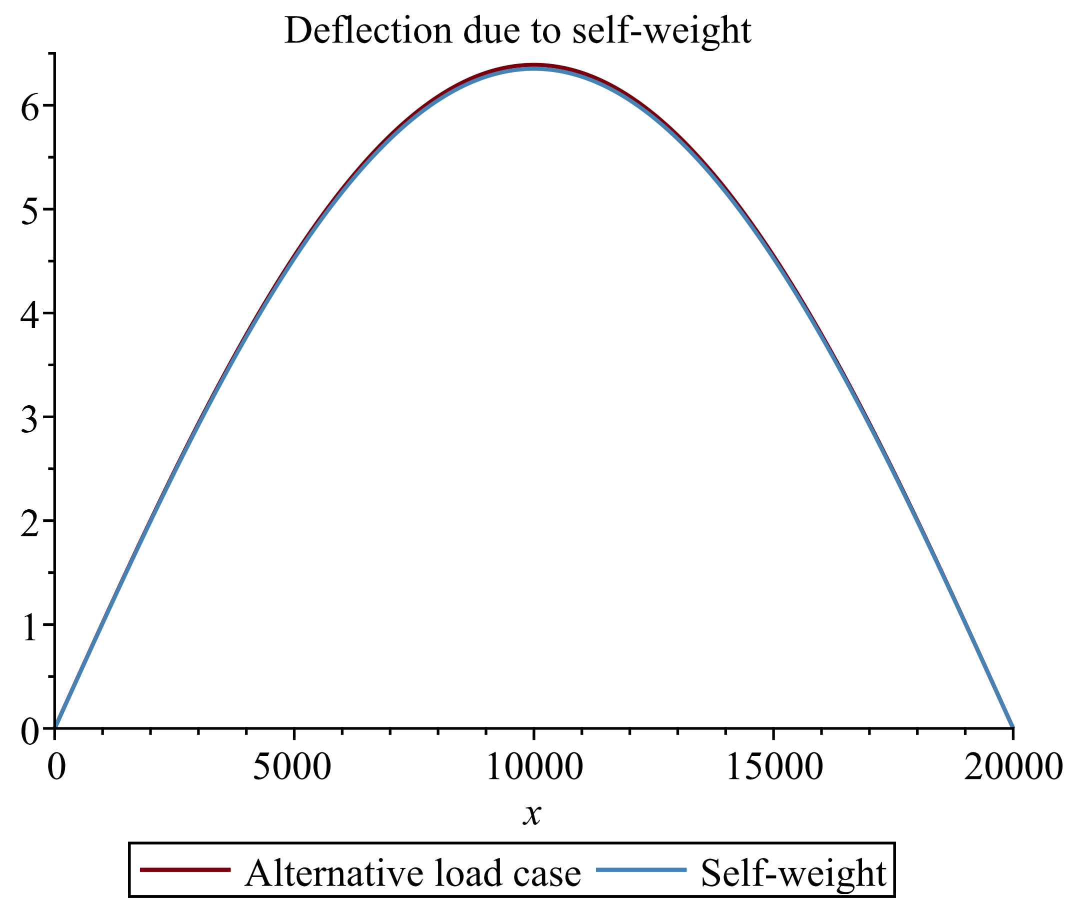
$A := plot\left(w1, x = 0 .. \frac{(l-a)}{2}, style = line\right):$

$B := plot\left(w2, x = \frac{(l-a)}{2} .. \frac{(l+a)}{2}, y = 0 ..6.5, labels = [\text{"x-coordinate (mm) "}, \text{"deflection (mm)"}], labeldirections = [horizontal, vertical], style = line, title = (\text{"Deflection due to self-weight"})\right):$

$C := plot\left(w3, x = \frac{(l+a)}{2} ..l, \quad legend = [\text{"Alternative load case"}], legendstyle = [location = bottom], style = line\right):$

$E := plot(w7, x = 0 ..l, legend = [\text{"Self-weight"}], legendstyle = [location = bottom], style = line, color = \text{"SteelBlue"}, labels = [\text{"x-coordinate (mm) "}, \text{"Deflection (mm)"}], labeldirections = [horizontal, vertical]):$

$display(\{A, B, C, E\});$

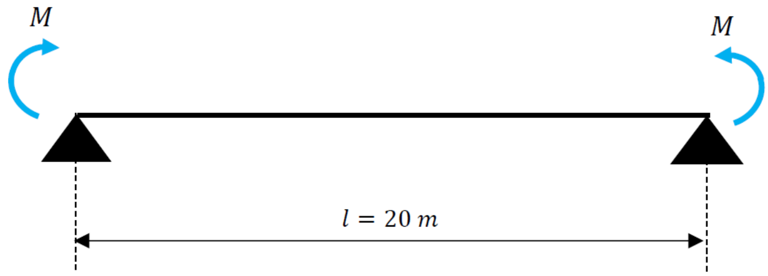


8.3 Deflection due to prestress

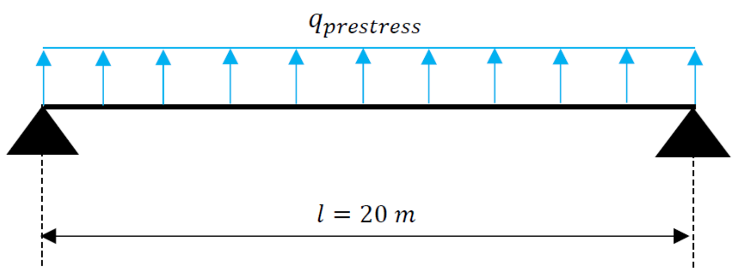
restart;

Deflection due to prestress:

A) Deflection due to bending moment MA



B) Deflection due to constant upward acting prestress q



Parameters

Parameters of the girder

$E := 34000 :\ Iy := 1.3265 \cdot 10^{12} :\ \ EI := E \cdot Iy :\qquad l := 20000 :$

Applied loading

$MA := 268 \cdot 10^{6} :\qquad q1 := -92.1 :$

Functions A

Displacement function

$w1 := c1 + c2 \cdot x + c3 \cdot x^2 + c4 \cdot x^3 + \frac{1}{2} \cdot \frac{MA \cdot x^2}{EI} :$

Rotation function

$phi1 := -diff(w1, x) :$

Bending moment function

$M1 := EI \cdot diff(phi1, x) :$

Shear force functions

$V1 := diff(M1, x):$

Functions B

Displacement function

$w5 := j1 + j2 \cdot x + j3 \cdot x^2 + j4 \cdot x^3 + \frac{2 \cdot q1 \cdot x^4}{24 \cdot EI}:$

Rotation function

$phi5 := -diff(w5, x):$

Bending moment function

$M5 := EI \cdot diff(phi5, x):$

Shear force function

$V5 := diff(M5, x):$

Boundary conditions A

Boundary conditions at x=0

$x := 0:$
$eq1 := w1 = 0:$
$eq2 := M1 = MA:$

Boundary conditions at x=L
$x := l:$
$eq3 := w1 = 0:$
$eq4 := M1 = MA:$

Boundary conditions B

Boundary conditions at x=0

```
x := 0 :
eq17 := w5 = 0 :
eq18 := M5 = 0 :
```

Boundary conditions at x=L

```
x := l :
eq19 := w5 = 0 :
eq20 := M5 = 0 :
```

Solution

```
solution := solve( {eq1, eq2, eq3, eq4, eq17, eq18, eq19, eq20}, {c1, c2, c3, c4, j1, j2, j3, j4}) :
assign(solution) :
x :='x':
```

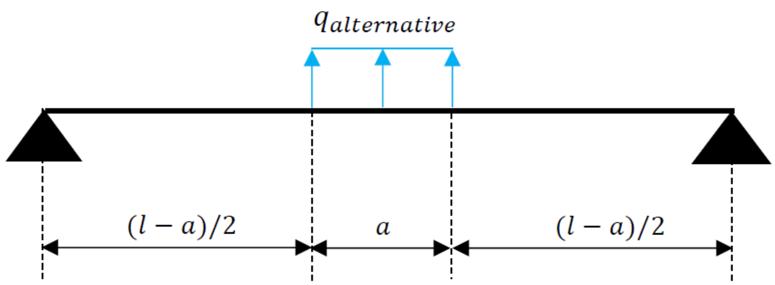
Graphs

Displacement graph

```
with(plots) :
```

```
Tot1 := plot(w1 + w5, x = 0 ..l, color = "Blue", legend = ["Prestress in girder"], legendstyle = [location
    = bottom]) :
```

Deflection due to alternative loading:



Parameters

Applied loading & length

$q3 := -2 \cdot 92.1 \cdot 0.98 :$ $\quad a := 18000 :$

Functions C

Displacement functions

$w6 := k1 + k2 \cdot x + k3 \cdot x^2 + k4 \cdot x^3 :$

$$w7 := l1 + l2 \cdot x + l3 \cdot x^2 + l4 \cdot x^3 + \frac{q3 \cdot x^4}{24 \cdot EI} :$$

$w8 := m1 + m2 \cdot x + m3 \cdot x^2 + m4 \cdot x^3 :$

Rotation functions

$phi6 := -diff(w6, x) :$
$phi7 := -diff(w7, x) :$
$phi8 := -diff(w8, x) :$

Bending moment functions

$M6 := EI \cdot diff(phi6, x) :$
$M7 := EI \cdot diff(phi7, x) :$
$M8 := EI \cdot diff(phi8, x) :$

Shear force functions

$V6 := diff(M6, x) :$
$V7 := diff(M7, x) :$
$V8 := diff(M8, x) :$

Boundary conditions C

Boundary conditions at x=0

$x := 0:$
$eq21 := w6 = 0:$
$eq22 := M6 = 0:$

Transition condition at x=(l-a)/2

$x := \frac{(l-a)}{2}:$
$eq23 := w6 = w7:$
$eq24 := phi6 = phi7:$
$eq25 := M6 = M7:$
$eq26 := V6 = V7:$

Transition condition at x=(l+a)/2

$x := \frac{(l+a)}{2}:$
$eq27 := w7 = w8:$
$eq28 := phi7 = phi8:$
$eq29 := M7 = M8:$
$eq30 := V7 = V8:$

Boundary conditions at x=L

$x := l:$
$eq31 := w8 = 0:$
$eq32 := M8 = 0:$

Solution

$solution := solve(\{eq20, eq21, eq22, eq23, eq24, eq25, eq26, eq27, eq28, eq29, eq30, eq31, eq32\}, \{k1, k2, k3, k4, l1, l2, l3, l4, m1, m2, m3, m4\}):$
$assign(solution):$
$x :='x':$

Graphs

Displacement graph

$with(plots):$

$D1 := plot\left(w6, x = 0 .. \frac{(l-a)}{2}, color = \text{"Red"}, labels = [\text{"x-coordinate (mm) "}, \text{"Deflection(mm)"}], labeldirections = [horizontal, vertical]\right):$

$$D2 := plot\left(w7, x = \frac{(l-a)}{2} .. \frac{(l+a)}{2}, color = \text{"Red"}, title = [\text{"Deflection due to Prestressing"}]\right):$$

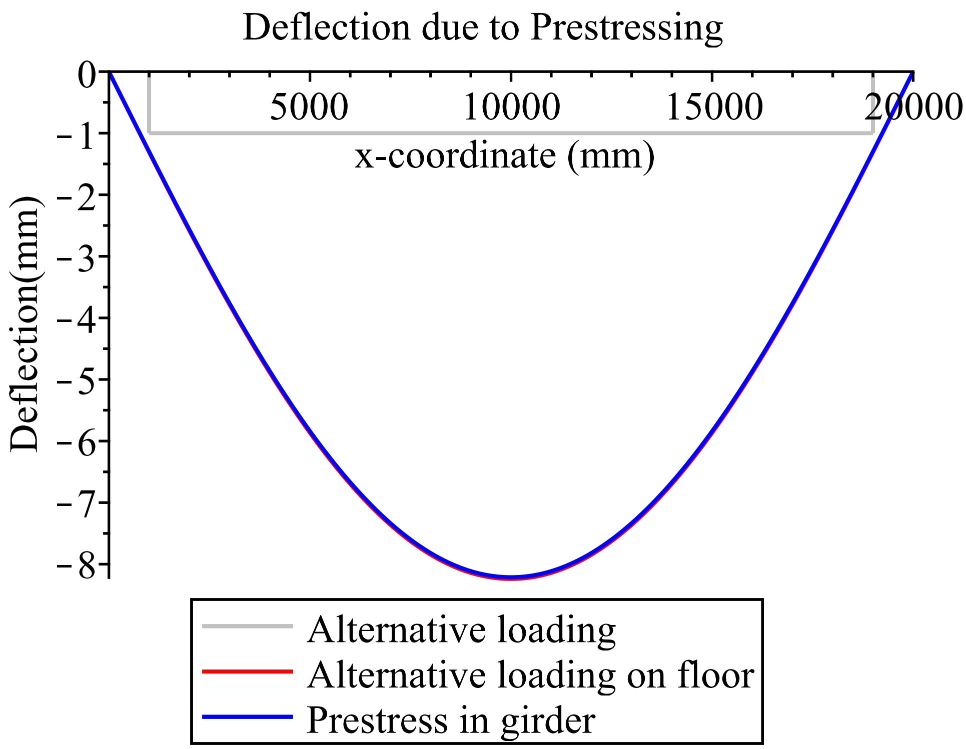
$$D3 := plot\left(w8, x = \frac{(l+a)}{2} ..l, color = \text{"Red"}, legend = [\text{"Alternative loading on floor"}], legendstyle = [location = bottom]\right):$$

$$E1 := plots[display]\left(plottools[line]\left(\left[\frac{(l-a)}{2}, 0\right], \left[\frac{(l-a)}{2}, -1\right]\right), color = grey, legend = [\text{"Alternative loading"}], legendstyle = [location = bottom]\right):$$

$$E2 := plots[display]\left(plottools[line]\left(\left[\frac{(l-a)}{2}, -1\right], \left[\frac{(l+a)}{2}, -1\right]\right), color = grey\right):$$

$$E3 := plots[display]\left(plottools[line]\left(\left[\frac{(l+a)}{2}, -1\right], \left[\frac{(l+a)}{2}, 0\right]\right), color = grey\right):$$

$display(\{D1, D2, D3, Tot1, E1, E2, E3\});$



8.4 Orthotropic floor properties

Orthotropy

Properties

Height	h	=		=	650	mm
Youngs Modulus	E_1	=		=	100	N/mm²
	E_2	=		=	11200	N/mm²
Poisson Ratio	ν_{12}	=		=	0,20	[-]
	ν_{21}	=	$\nu_{12} * E_2 / E_1$	=	0,50	[-]
Shear modulus	G_{12}	=	$\sqrt{(E_1 * E_2)} / (2 * (1 + \sqrt{(\nu_{12} * \nu_{21})}))$	=	402	N/mm²
	G_{13}	=	$E_1 / (2 * (1 + \nu_{12}))$	=	42	N/mm²
	G_{23}	=	$E_2 / (2 * (1 + \nu_{21}))$	=	3733	N/mm²

Parameters (plate elements)

D_{11}	=	$(E_1 * h^3) / (12 * (1 - (\nu_{12} * \nu_{21})))$	=	3	MNm
D_{22}	=	$(E_2 * h^3) / (12 * (1 - (\nu_{12} * \nu_{21})))$	=	285	MNm
D_{12}	=	$\nu_{21} * D_{11}$	=	1	MNm
D_{33}	=	$(G_{12} * h^3) / 12$	=	9	MNm
D_{44}	=	$G_{13} * h / 1{,}2$	=	23	MN/m
D_{55}	=	$G_{23} * h / 1{,}2$	=	2022	MN/m

Parameters (shell elements)

d_{11}	=	$(E_1 * h) / (1 - (\nu_{12} * \nu_{21}))$	=	72	MN/m
d_{22}	=	$(E_2 * h) / (1 - (\nu_{12} * \nu_{21}))$	=	8089	MN/m
d_{12}	=	$\nu_{21} * d_{11}$	=	36	MN/m
d_{33}	=	$G_{12} * h$	=	261	MN/m

8.5 SCIA plate model – Bridge A

1. 2D-elementen

1.1. 2D-elementen - Trogvloer

Naam	Laag	Type	Element type	Materiaal	Dikte type	D. [mm]
Trogvloer	Laag1	vloer (90)	Standaard	C35/45 (gescheurd)	constant	550

1.1.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K1	0,000	0,000	0,000
K2	23,000	0,000	0,000
K3	23,000	3,000	0,000
K4	0,000	3,000	0,000

1.2. 2D-elementen - Voute 1

Naam	Laag	Type	Element type	Materiaal	Dikte type	D. [mm]
Voute 1	Laag1	vloer (90)	Standaard	C35/45 (gescheurd)	constant	700

1.2.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K1	0,000	0,000	0,000
K2	23,000	0,000	0,000
K5	23,000	-0,500	0,000
K18	0,000	-0,500	0,000

1.3. 2D-elementen - Voute 2

Naam	Laag	Type	Element type	Materiaal	Dikte type	D. [mm]
Voute 2	Laag1	vloer (90)	Standaard	C35/45 (gescheurd)	constant	700

1.3.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K3	23,000	3,000	0,000
K4	0,000	3,000	0,000
K7	23,000	3,500	0,000
K203	0,000	3,500	0,000

1.4. 2D-elementen - Trogbalk 1

Naam	Laag	Type	Element type	Materiaal	Dikte type	D. [mm]
Trogbalk 1	Laag1	vloer (90)	Standaard	C35/45 (ongescheurd)	constant	900

1.4.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K9	23,000	-0,950	1,400
K10	0,000	-0,950	1,400
K11	0,000	-0,950	-0,350
K12	23,000	-0,950	-0,350

1.5. 2D-elementen - Trogbalk 2

Naam	Laag	Type	Element type	Materiaal	Dikte type	D. [mm]
Trogbalk 2	Laag1	vloer (90)	Standaard	C35/45 (ongescheurd)	constant	900

1.5.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K13	0,000	3,950	-0,350
K14	23,000	3,950	-0,350
K15	23,000	3,950	1,400
K16	0,000	3,950	1,400

2. Materialen

Naam	Type	ρ [kg/m³]	Dichtheid in natte toestand [kg/m³]	E_{mod} [MPa]	μ	α [m/mK]	$f_{c.k.28}$ [MPa]	Kleur
C35/45 (ongescheurd)	Beton	2500,0	2600,0	3,4100e+04	0.2	0,00	35,00	
C35/45 (gescheurd)	Beton	2500,0	2600,0	1,1200e+04	0.2	0,00	35,00	

Verklaring van symbolen	
Dichtheid in natte toestand	De waarde van de dichtheid van het kenmerk nieuwe toestand wordt alleen gebruikt als een samengesteld dek wordt ingevoerd en rekening wordt gehouden met de belasting van het eigengewicht.

Wapening EC2

Naam	Type	ρ [kg/m³]	E_{mod} [MPa]	G_{mod} [MPa]	α [m/mK]	$f_{y.k}$ [MPa]
B 400A	Betonstaal	7850,0	2,0000e+05	8,3333e+04	0,00	400,0

3. Integratiestrook

Naam	2D-element	Effectieve breedte geometrie	Effectieve breedte definitie	Breedte (totaal) [mm]
CM1	Trogbalk 1	Constant symmetrisch	Breedte	1750,0
CM2	Trogbalk 2	Constant symmetrisch	Breedte	1750,0

4. Ondersteuningen op 2D elementranden

Naam	2D-element	Oors	Pos x_1 [m]	X	Y	Z	Rx	Ry	Rz
	Rand	Coör	Pos x_2 [m]						
Sle1	Trogbalk 1	Vanaf begin	1,000	Vast	Vast	Vast	Vrij	Vrij	Vrij
	1	Abso	1,250						
Sle2	Trogbalk 2	Vanaf begin	1,000	Vrij	Vrij	Vast	Vrij	Vrij	Vrij
	1	Abso	1,250						
Sle3	Trogbalk 2	Vanaf begin	22,000	Vrij	Vast	Vast	Vrij	Vrij	Vrij
	1	Abso	22,250						
Sle4	Trogbalk 1	Vanaf begin	22,000	Vrij	Vrij	Vast	Vrij	Vrij	Vrij
	1	Abso	22,250						

5. Belastingsgevallen

5.1. Belastingsgevallen - LC 1 - Line load - 0,5L

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep
LC 1 - Line load - 0,5L		Permanent Standaard	LG1

5.1.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL1	LC 1 - Line load - 0,5L	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte
FL2	LC 1 - Line load - 0,5L	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte

5.2. Belastingsgevallen - LC 2 - Local line load

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep
LC 2 - Local line load		Permanent Standaard	LG1

5.2.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL3	LC 2 - Local line load	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte
FL4	LC 2 - Local line load	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte

5.3. Belastingsgevallen - LC 3 - Self-weight

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep	Richting
LC 3 - Self-weight		Permanent Eigen gewicht	LG1	-Z

5.4. Belastingsgevallen - LC 4 - Prestress

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep
LC 4 - Prestress		Permanent Standaard	LG1

5.4.1. Moment in knoop

Naam	Knoop	Belastingsgeval	Systeem	Rich	Type	Waarde - M [kNm]
M1	K387	LC 4 - Prestress	GCS	My	Moment	268,00
M2	K388	LC 4 - Prestress	GCS	My	Moment	-268,00
M3	K390	LC 4 - Prestress	GCS	My	Moment	-268,00
M4	K389	LC 4 - Prestress	GCS	My	Moment	268,00

5.4.2. Lijnlast op 2D elementrand

Naam	2D-element Belastingsgeval	Type Systeem	Rich Verdeling	Waarde - P_1 [kN/m] Waarde - P_2 [kN/m]	Pos x_1 [m] Pos x_2 [m]	Loc Coör	Rand Oors
LFS1	Trogbalk 1 LC 4 - Prestress	Kracht LCS	Y Gelijkmatig	-92,10	1,000 22,000	Lengte Abso	1 Vanaf begin
LFS2	Trogbalk 2 LC 4 - Prestress	Kracht LCS	Y Gelijkmatig	-92,10	1,000 22,000	Lengte Abso	1 Vanaf begin

5.5. Belastingsgevallen - LC 6 - LM 71 1/2

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep
LC 6 - LM 71 1/2		Permanent	LG1

Naam	Omschrijving	Actie type	Lastgroep
	Spec	Belastingtype	
		Standaard	

5.5.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL5	LC 6 - LM 71 1/2	Z	Kracht	Gelijkmatig	-298,80	Alle	Auto	GCS	Lengte
FL6	LC 6 - LM 71 1/2	Z	Kracht	Gelijkmatig	-298,80	Alle	Auto	GCS	Lengte

5.6. Belastingsgevallen - LC 7 - LM 71 2/2

Naam	Omschrijving	Actie type	Lastgroep
	Spec	Belastingtype	
LC 7 - LM 71 2/2		Permanent	LG1
		Standaard	

5.6.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL7	LC 7 - LM 71 2/2	Z	Kracht	Gelijkmatig	-165,00	Alle	Auto	GCS	Lengte
FL8	LC 7 - LM 71 2/2	Z	Kracht	Gelijkmatig	-165,00	Alle	Auto	GCS	Lengte

8.6 SCIA beam model 1A – Bridge A

1. 2D-elementen

1.1. 2D-elementen - Trogvloer

Naam	Laag	Type	Element type	Materiaal	Dikte type	D. [mm]
Trogvloer	Laag1	vloer (90)	Standaard	C35/45 (vloer)	constant	550

1.1.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K1	0,000	0,000	0,000
K2	23,000	0,000	0,000
K3	23,000	4,900	0,000
K4	0,000	4,900	0,000

1.1.2. 2D-element interne randen

Naam	2D-element 1	Lengte [m]	Vorm	Knoop	Rand
Rand1	Trogvloer	23,000	Lijn	K1 K2	Lijn
Rand2	Trogvloer	23,000	Lijn	K3 K4	Lijn

2. Staven

2.1. Staven - Trogbalk 1

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Trogbalk 1	CS1 - Rechthoek (1475; 900)	C35/45 (balk)	23,000	K1	K2	Algemeen (0)

2.1.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K1	0,000	0,000	0,000
K2	23,000	0,000	0,000
K9	1,000	0,000	0,000
K10	22,000	0,000	0,000

2.2. Staven - Trogbalk 2

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Trogbalk 2	CS1 - Rechthoek (1475; 900)	C35/45 (balk)	23,000	K3	K4	Algemeen (0)

2.2.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K3	23,000	4,900	0,000
K4	0,000	4,900	0,000
K11	22,000	4,900	0,000
K12	1,000	4,900	0,000

3. Knoopondersteuningen

Naam	Knoop	Systeem	Type	X	Y	Z	Rx	Ry	Rz
Sn1	K9	GCS	Standaard	Vast	Vast	Vast	Vrij	Vrij	Vrij
Sn2	K10	GCS	Standaard	Vrij	Vrij	Vast	Vrij	Vrij	Vrij
Sn3	K12	GCS	Standaard	Vrij	Vrij	Vast	Vrij	Vrij	Vrij
Sn4	K11	GCS	Standaard	Vrij	Vast	Vast	Vrij	Vrij	Vrij

4. Materialen

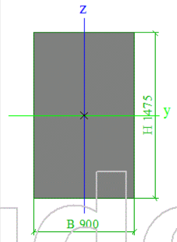
Naam	Type	ρ [kg/m³]	Dichtheid in natte toestand [kg/m³]	E_{mod} [MPa]	μ	α [m/mK]	$f_{c.k.28}$ [MPa]	Kleur
C35/45 (balk)	Beton	2500,0	2600,0	3,4100e+04	0.2	0,00	35,00	
C35/45 (vloer)	Beton	2500,0	2600,0	1,1200e+04	0.2	0,00	35,00	

Verklaring van symbolen	
Dichtheid in natte toestand	De waarde van de dichtheid van het kenmerk nieuwe toestand wordt alleen gebruikt als een samengesteld dek wordt ingevoerd en rekening wordt gehouden met de belasting van het eigengewicht.

Wapening EC2

Naam	Type	ρ [kg/m³]	E_{mod} [MPa]	G_{mod} [MPa]	α [m/mK]	$f_{y.k}$ [MPa]
B 400A	Betonstaal	7850,0	2,0000e+05	8,3333e+04	0,00	400,0

5. Doorsneden

CS1		
Type	Rechthoek	
Uitgebreid	1475; 900	
Vorm type	Dikke wanden	
Onderdeelmateriaal	C35/45 (balk)	
Bouwwijze	beton	
Kleur	■	
A [m²]	1,3275e+00	
A_y [m²], A_z [m²]	1,1063e+00	1,1063e+00
A_L [m²/m], A_D [m²/m]	4,7500e+00	4,7500e+00
$c_{Y.UCS}$ [mm], $c_{Z.UCS}$ [mm]	450	738
α [deg]	0,00	
I_y [m⁴], I_z [m⁴]	2,4068e-01	8,9606e-02
i_y [mm], i_z [mm]	426	260
$W_{el.y}$ [m³], $W_{el.z}$ [m³]	3,2634e-01	1,9912e-01
$W_{pl.y}$ [m³], $W_{pl.z}$ [m³]	0,0000e+00	0,0000e+00
$M_{pl.y.+}$ [Nm], $M_{pl.y.-}$ [Nm]	0,00e+00	0,00e+00
$M_{pl.z.+}$ [Nm], $M_{pl.z.-}$ [Nm]	0,00e+00	0,00e+00
d_y [mm], d_z [mm]	0	0
I_t [m⁴], I_w [m⁶]	2,2223e-01	0,0000e+00
$β_y$ [mm], $β_z$ [mm]	0	0
Afbeelding		

Verklaring van symbolen	
A	Gebied
A_y	Afschuifoppervlak in hoofd y-richting
A_z	Afschuifoppervlak in hoofd z-richting
A_L	Omtrek per eenheidslengte
A_D	Uithardingsoppervlakte per eenheidslengte
$c_{Y.UCS}$	Zwaartepunt coordinaten in Y-richting van het invoer assen systeem
$c_{Z.UCS}$	Zwaartepunt coordinaten in Z-richting van het invoer assen systeem
$I_{Y.LCS}$	Tweede moment van het gebied rond de YLCS as
$I_{Z.LCS}$	Tweede moment van het gebied rond de ZLCS as
$I_{YZ.LCS}$	Product moment van het gebied in het LCS systeem
α	Rotatiehoek van het hoofd assen systeem
I_y	Tweede moment van het gebied rond de hoofd y-as
I_z	Tweede moment van het gebied rond de hoofd z-as
i_y	Traagheidsstraal rond de hoofd y-as
i_z	Traagheidsstraal rond de hoofd z-as

Verklaring van symbolen	
$W_{el.y}$	Elastische doorsnede modulus rond de hoofd y-as
$W_{el.z}$	Elastische doorsnede modulus rond de hoofd z-as
$W_{pl.y}$	Plastische doorsnede modulus rond de hoofd y-as
$W_{pl.z}$	Plastische doorsnede modulus rond de hoofd z-as
$M_{pl.y.+}$	Plastisch moment rond de hoofd y-as voor een positief My moment
$M_{pl.y.-}$	Plastisch moment rond de hoofd y-as voor een negatief My moment
$M_{pl.z.+}$	Plastisch moment rond de hoofd z-as voor een positief Mz moment
$M_{pl.z.-}$	Plastisch moment rond de hoofd z-as voor een negatief Mz moment
d_y	Afschuif middencoordinaat in hoofd y-richting gemeten vanaf het zwaartepunt - Niet berekend of vereenvoudigd
d_z	Afschuif middencoordinaat in hoofd z-richting gemeten vanaf het zwaartepunt - Niet berekend of vereenvoudigd
I_t	Torsie constante - Niet berekend of vereenvoudigd
I_w	Welvings constante - Niet berekend of vereenvoudigd
$β_y$	Mono-symmetrische constante rond de hoofd y-as
$β_z$	Mono-symmetrische constante rond de hoofd z-as

6. Belastingsgevallen

6.1. Belastingsgevallen - LC 1 - 0,5L Loaded

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep
LC 1 - 0,5L Loaded		Permanent Standaard	LG1

6.1.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL1	LC 1 - 0,5L Loaded	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte
FL2	LC 1 - 0,5L Loaded	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte

6.2. Belastingsgevallen - LC 2 - Local line load

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep
LC 2 - Local line load		Permanent Standaard	LG1

6.2.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL3	LC 2 - Local line load	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte
FL4	LC 2 - Local line load	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte

6.3. Belastingsgevallen - LC 3 - Self-weight

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep	Richting
LC 3 - Self-weight		Permanent Eigen gewicht	LG1	-Z

6.4. Belastingsgevallen - LC 4 - Prestress

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep
LC 4 - Prestress		Permanent Standaard	LG1

6.4.1. Moment op staaf

Naam	Staaf Belastingsgeval	Systeem Rich	Waarde - M [kNm] Type	Pos x [m]	Coör Oors	Herh (n) dx [m]
M1	Trogbalk 1 LC 4 - Prestress	LCS My	268,00 Moment	1,000	Abso Vanaf begin	1
M2	Trogbalk 2 LC 4 - Prestress	LCS My	268,00 Moment	1,000	Abso Vanaf begin	1
M3	Trogbalk 1 LC 4 - Prestress	LCS My	-268,00 Moment	1,000	Abso Vanaf einde	1
M4	Trogbalk 2 LC 4 - Prestress	LCS My	-268,00 Moment	1,000	Abso Vanaf einde	1

6.4.2. Lijnlast

Naam	Staaf Belastingsgeval	Type Systeem	Rich Verdeling	Waarde - P_1 [kN/m] Waarde - P_2 [kN/m]	Pos x_1 [m] Pos x_2 [m]	Coör Loc	Oors	Exc ey [m] Exc ez [m]
Lijnlast1	Trogbalk 1 LC 4 - Prestress	Kracht LCS	Z Gelijkmatig	92,10	1,000 22,000	Abso Lengte	Vanaf begin	0,000 0,000
Lijnlast2	Trogbalk 2 LC 4 - Prestress	Kracht LCS	Z Gelijkmatig	92,10	1,000 22,000	Abso Lengte	Vanaf begin	0,000 0,000

6.5. Belastingsgevallen - LC 6 - LM 71 1/2

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep
LC 6 - LM 71 1/2		Permanent Standaard	LG1

6.5.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL5	LC 6 - LM 71 1/2	Z	Kracht	Gelijkmatig	-149,40	Alle	Auto	GCS	Lengte
FL6	LC 6 - LM 71 1/2	Z	Kracht	Gelijkmatig	-149,40	Alle	Auto	GCS	Lengte

6.6. Belastingsgevallen - LC 7 - LM 71 2/2

Naam	Omschrijving Spec	Actie type Belastingtype	Lastgroep
LC 7 - LM 71 2/2		Permanent Standaard	LG1

6.6.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL7	LC 7 - LM 71 2/2	Z	Kracht	Gelijkmatig	-82,50	Alle	Auto	GCS	Lengte
FL8	LC 7 - LM 71 2/2	Z	Kracht	Gelijkmatig	-82,50	Alle	Auto	GCS	Lengte

8.7 SCIA beam model 1B – Bridge A

1. 2D-elementen

1.1. 2D-elementen - Voute 1

Naam	Laag	Type	Element type	Materiaal	Dikte type	D. [mm]
Voute 1	Laag1	vloer (90)	Standaard	C35/45 (vloer)	constant	700

1.1.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K617	0,000	0,950	0,000
K618	23,000	0,950	0,000
K623	0,000	0,450	0,000
K624	23,000	0,450	0,000

1.2. 2D-elementen - Trogvloer

Naam	Laag	Type	Element type	Materiaal	Dikte type	D. [mm]
Trogvloer	Laag1	vloer (90)	Standaard	C35/45 (vloer)	constant	550

1.2.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K617	0,000	0,950	0,000
K618	23,000	0,950	0,000
K809	0,000	3,950	0,000
K810	23,000	3,950	0,000

1.3. 2D-elementen - Voute 2

Naam	Laag	Type	Element type	Materiaal	Dikte type	D. [mm]
Voute 2	Laag1	vloer (90)	Standaard	C35/45 (vloer)	constant	700

1.3.1. Knopen

Naam	Coördinaat X [m]	Coördinaat Y [m]	Coördinaat Z [m]
K809	0,000	3,950	0,000
K810	23,000	3,950	0,000
K813	0,000	4,450	0,000
K814	23,000	4,450	0,000

2. Staven

Naam	Doorsnede	Materiaal	Lengte [m]	Beginknoop	Eindknoop	Type
Trogbalk 1	CS1 - Rechthoek (1750; 900)	C35/45	23,000	K1	K2	Algemeen (0)
Trogbalk 2	CS1 - Rechthoek (1750; 900)	C35/45	23,000	K716	K717	Algemeen (0)

3. Puntondersteuningen op staaf

Naam	Type	Coör	Pos x [m]	dx [m]	X	Y	Z	Rx	Ry	Rz
		Systeem	Oors	Herh (n)						
Sb1	Standaard	Abso	1,000		Vast	Vast	Vast	Vrij	Vrij	Vrij
		GCS	Vanaf begin	1						
Sb2	Standaard	Abso	1,000		Vrij	Vrij	Vast	Vrij	Vrij	Vrij
		GCS	Vanaf begin	1						
Sb3	Standaard	Abso	1,000		Vrij	Vrij	Vast	Vrij	Vrij	Vrij
		GCS	Vanaf einde	1						
Sb4	Standaard	Abso	1,000		Vrij	Vast	Vast	Vrij	Vrij	Vrij
		GCS	Vanaf einde	1						

4. Materialen

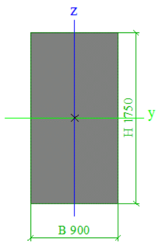
Naam	Type	ρ [kg/m³]	Dichtheid in natte toestand [kg/m³]	E_{mod} [MPa]	μ	α [m/mK]	$f_{c.k.28}$ [MPa]	Kleur
C35/45	Beton	2500,0	2600,0	3,4100e+04	0.2	0,00	35,00	
C35/45 (vloer)	Beton	2500,0	2600,0	1,1200e+04	0.2	0,00	35,00	

Verklaring van symbolen	
Dichtheid in natte toestand	De waarde van de dichtheid van het kenmerk nieuwe toestand wordt alleen gebruikt als een samengesteld dek wordt ingevoerd en rekening wordt gehouden met de belasting van het eigengewicht.

Wapening EC2

Naam	Type	ρ [kg/m³]	E_{mod} [MPa]	G_{mod} [MPa]	α [m/mK]	$f_{y.k}$ [MPa]
B 400A	Betonstaal	7850,0	2,0000e+05	8,3333e+04	0,00	400,0

5. Doorsneden

CS1		
Type	Rechthoek	
Uitgebreid	1750; 900	
Vorm type	Dikke wanden	
Onderdeelmateriaal	C35/45	
Bouwwijze	beton	
Kleur		
A [m²]	1,5750e+00	
A_y [m²], A_z [m²]	1,3125e+00	1,3125e+00
A_L [m²/m], A_D [m²/m]	5,3000e+00	5,3000e+00
$c_{Y.UCS}$ [mm], $c_{Z.UCS}$ [mm]	450	875
α [deg]	0,00	
I_y [m⁴], I_z [m⁴]	4,0195e-01	1,0631e-01
i_y [mm], i_z [mm]	505	260
$W_{el.y}$ [m³], $W_{el.z}$ [m³]	4,5937e-01	2,3625e-01
$W_{pl.y}$ [m³], $W_{pl.z}$ [m³]	0,0000e+00	0,0000e+00
$M_{pl.y.+}$ [Nm], $M_{pl.y.-}$ [Nm]	0,00e+00	0,00e+00
$M_{pl.z.+}$ [Nm], $M_{pl.z.-}$ [Nm]	0,00e+00	0,00e+00
d_y [mm], d_z [mm]	0	0
I_t [m⁴], I_w [m⁶]	2,8808e-01	0,0000e+00
$β_y$ [mm], $β_z$ [mm]	0	0
Afbeelding		

Verklaring van symbolen	
A	Gebied
A_y	Afschuifoppervlak in hoofd y-richting
A_z	Afschuifoppervlak in hoofd z-richting
A_L	Omtrek per eenheidslengte
A_D	Uithardingsoppervlakte per eenheidslengte
$c_{Y.UCS}$	Zwaartepunt coordinaten in Y-richting van het invoer assen systeem
$c_{Z.UCS}$	Zwaartepunt coordinaten in Z-richting van het invoer assen systeem
$I_{Y.LCS}$	Tweede moment van het gebied rond de YLCS as
$I_{Z.LCS}$	Tweede moment van het gebied rond

Verklaring van symbolen	
	de ZLCS as
$I_{YZ.LCS}$	Product moment van het gebied in het LCS systeem
α	Rotatiehoek van het hoofd assen systeem
I_y	Tweede moment van het gebied rond de hoofd y-as
I_z	Tweede moment van het gebied rond de hoofd z-as
i_y	Traagheidsstraal rond de hoofd y-as
i_z	Traagheidsstraal rond de hoofd z-as
$W_{el.y}$	Elastische doorsnede modulus rond de hoofd y-as

Verklaring van symbolen	
$W_{el.z}$	Elastische doorsnede modulus rond de hoofd z-as
$W_{pl.y}$	Plastische doorsnede modulus rond de hoofd y-as
$W_{pl.z}$	Plastische doorsnede modulus rond de hoofd z-as
$M_{pl.y.+}$	Plastisch moment rond de hoofd y-as voor een positief My moment
$M_{pl.y.-}$	Plastisch moment rond de hoofd y-as voor een negatief My moment
$M_{pl.z.+}$	Plastisch moment rond de hoofd z-as voor een positief Mz moment
$M_{pl.z.-}$	Plastisch moment rond de hoofd z-as voor een negatief Mz moment

Verklaring van symbolen	
d_y	Afschuif middencoordinaat in hoofd y-richting gemeten vanaf het zwaartepunt - Niet berekend of vereenvoudigd
d_z	Afschuif middencoordinaat in hoofd z-richting gemeten vanaf het zwaartepunt - Niet berekend of vereenvoudigd
I_t	Torsie constante - Niet berekend of vereenvoudigd
I_w	Welvings constante - Niet berekend of vereenvoudigd
β_y	Mono-symmetrische constante rond de hoofd y-as
β_z	Mono-symmetrische constante rond de hoofd z-as

6. Belastingsgevallen

6.1. Belastingsgevallen - LC 1 - 0,5L Loaded

Naam	Omschrijving	Actie type	Lastgroep
	Spec	Belastingtype	
LC 1 - 0,5L Loaded		Permanent	LG1
		Standaard	

6.1.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL1	LC 1 - 0,5L Loaded	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte
FL2	LC 1 - 0,5L Loaded	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte

6.2. Belastingsgevallen - LC 2 - Local line load

Naam	Omschrijving	Actie type	Lastgroep
	Spec	Belastingtype	
LC 2 - Local line load		Permanent	LG1
		Standaard	

6.2.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL3	LC 2 - Local line load	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte
FL4	LC 2 - Local line load	Z	Kracht	Gelijkmatig	-100,00	Alle	Auto	GCS	Lengte

6.3. Belastingsgevallen - LC 3 - Self-weight

Naam	Omschrijving	Actie type	Lastgroep	Richting
	Spec	Belastingtype		
LC 3 - Self-weight		Permanent	LG1	-Z
		Eigen gewicht		

6.4. Belastingsgevallen - LC 4 - Prestress

Naam	Omschrijving	Actie type	Lastgroep
	Spec	Belastingtype	
LC 4 - Prestress		Permanent	LG1
		Standaard	

6.4.1. Lijnlast

Naam	Staaf	Type	Rich	Waarde - P_1 [kN/m]	Pos x_1 [m]	Coör	Oors	Exc ey [m]
	Belastingsgeval	Systeem	Verdeling	Waarde - P_2 [kN/m]	Pos x_2 [m]	Loc		Exc ez [m]
Lijnlast1	Trogbalk 1	Kracht	Z	92,10	1,000	Abso	Vanaf begin	0,000
	LC 4 - Prestress	LCS	Gelijkmatig		22,000	Lengte		0,000
Lijnlast2	Trogbalk 2	Kracht	Z	92,10	1,000	Abso	Vanaf begin	0,000
	LC 4 - Prestress	LCS	Gelijkmatig		22,000	Lengte		0,000

6.4.2. Moment op staaf

Naam	Staaf	Systeem	Waarde - M [kNm]	Pos x [m]	Coör	Herh (n)
	Belastingsgeval	Rich	Type		Oors	dx [m]
M1	Trogbalk 1	LCS	268,00	1,000	Abso	1
	LC 4 - Prestress	My	Moment		Vanaf begin	
M2	Trogbalk 2	LCS	268,00	1,000	Abso	1
	LC 4 - Prestress	My	Moment		Vanaf begin	
M3	Trogbalk 1	LCS	-268,00	1,000	Abso	1
	LC 4 - Prestress	My	Moment		Vanaf einde	
M4	Trogbalk 2	LCS	-268,00	1,000	Abso	1
	LC 4 - Prestress	My	Moment		Vanaf einde	

6.5. Belastingsgevallen - LC 6 - LM 71 1/2

Naam	Omschrijving	Actie type	Lastgroep
	Spec	Belastingtype	
LC 6 - LM 71 1/2		Permanent	LG1
		Standaard	

6.5.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL5	LC 6 - LM 71 1/2	Z	Kracht	Gelijkmatig	-149,40	Alle	Auto	GCS	Lengte
FL6	LC 6 - LM 71 1/2	Z	Kracht	Gelijkmatig	-149,40	Alle	Auto	GCS	Lengte

6.6. Belastingsgevallen - LC 7 - LM 71 2/2

Naam	Omschrijving	Actie type	Lastgroep
	Spec	Belastingtype	
LC 7 - LM 71 2/2		Permanent	LG1
		Standaard	

6.6.1. Vrije lijn last

Naam	Belastingsgeval	Rich	Type	Verdeling	Waarde - P_1 [kN/m]	Geldigheid	Selecteer	Systeem	Locatie
FL7	LC 7 - LM 71 2/2	Z	Kracht	Gelijkmatig	-82,50	Alle	Auto	GCS	Lengte
FL8	LC 7 - LM 71 2/2	Z	Kracht	Gelijkmatig	-82,50	Alle	Auto	GCS	Lengte