REPORT

Principles report recalculation IJssel bridge A12

40B-100 and 40B-111

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1 preface

1.1 Rationale for the project

Verification studies conducted by Rijkswaterstaat Major Projects and Maintenance (RWS GPO) it turned out that various details in the main supporting construction of the IJssel bridge near Westervoort in the In all probability, highway A12 will have insufficient (residual) life on fatigue. The problems mainly arise in various welded and riveted connection details, with relative low fatigue ratings. Based on these findings, in the period April-May 2017, met urgent inspections were carried out on the bridge. On the basis of these inspections it has been established that there is on this there is currently no measurable fatigue damage to the main supporting structure. Nevertheless, in all likelihood, the bridge is in all probability insufficient and serves it be reinforced to guarantee a residual life of at least 30 years. Till the moment The reinforcement will ensure the safety of the bridge by means of an intensified inspection regime.

To determine the actual scope of the problem and the required mitigation and / or control measures requires an overall recalculation of the bridge (on both strength and fatigue) and a reinforcement design.

In this report, the principles for the recalculation of the bridge are determined.

1.2 Objective

The recalculation of the IJssel bridge including the design and specification of the required reinforcement and / or control measures aims to restore the bridge to the requirement standard safety for an intended residual life of at least 30 years. Assessment takes place according to NEN 8700 / NEN 8701, RBK 1.1 and ROK 1.4 based on usage level for the existing / maintained structural parts and rebuilding level for reinforcements, including 30 years of residual life on fatigue.

1.3 Scope delimitation

The IJssel bridge in the A12 consists of 3 parallel bridge constructions: 2 steel composite to the west (bridges) and steel structures (main bridges) with year of construction 1961 and 1964 and 1 located to the east fully concrete bridge with year of construction 1990.

The bridge structures to be assessed and strengthened within this assignment are limited to the reinforced concrete bridges and the steel bridges. Recalculation and strengthening of the eastern whole concrete bridge is outside the scope of this assignment. Where reference is made in this document to a western or eastern IJssel bridge is therefore referred to 1 of the 2 steel / composite concrete bridges, and not the adjacent fully concrete bridge.

Figure 1 - Overview of the various separate bridge structures of the IJssel Bridge [request]

Both bridges consist successively of 3 bridge sections: 2 bridges with 3 fields each are completely above the Northern floodplain, and a main bridge with 5 fields, of which the in the middle the main bridge of 105m over the river IJssel. In total, both bridges are 536 each meter long.

The bridges are steel-concrete bridges consisting of two steel main beams and cross beams interacting with the concrete deck that protrudes beyond the main girders. At the location of the consoles are present at the end crossbars, but not at the location of the other crossbars.

The main bridges consist of 2 steel main girders with variable construction height with in between cross bars with an orthotropic steel deck with open stiffeners (bulbs). Outside the main girders the orthotropic riding deck is supported by cantilever brackets that are an extension of each crossbeam.

Bridge Bridge Main bridge

Figure 2 - Side view of the IJssel bridge [A.46205A]

The top code of RWS for the eastern bridge is 40B-100. RWS top code for the western part located bridge is 40B-111.

The main supporting structures and driving floors of both bridges are calculated in their entirety on static strength and stability, from the northernmost to the southernmost point of support, including supports, jacking points and joints. In addition, the main supporting structures including the cross bars calculated for fatigue. The steel orthotropic floor of the main bridges is not necessary be calculated on fatigue because it is maintained by means of a inspection regime. Based on the recalculation, if necessary, a final reinforcement design for the bridges.

Based on the support reactions in the current and reinforced situation, the substructure (pillars and pile foundations) of both bridges are assessed by means of a comparison with the original design loads with the premise that the strength of the substructure is normative has not changed. If necessary, advice will be given on the extent to which further recalculation of the substructure is required and to what extent the normative strength of the substructure may have changed.

Figure 3 - The IJssel Bridge seen from the south bank of the IJssel [request]

Figure 4 - The IJssel Bridge seen from the south bank of the IJssel [Cyclomedia]

Figure 5 - Aerial photo IJssel bridge (2017) [Cyclomedia]

1.4 Project phasing

The following phases are distinguished within the assignment, in which the (sub) products below must be delivered:

Phase 1: Drafting a memorandum of principles (Part A paragraph 1.11 RBK)

Phase 2: Preparing and executing recalculation: Parts B to F are removed complete paragraph 1.11 of the RBK (B. archive research, C. constructive inspection, D. research (where necessary), E. verification calculation, F. assessment). The entire construction will be on both strength and fatigue are calculated and tested (with the exception of the orthotropic samples driving floor on fatigue).

Phase 3: Designing reinforcements: For the parts for which conclusions are made in Phase 2 that they do not comply, reinforcement measures must be designed at VO level and estimated according to the SSK-2010 system. Phasing and estimation of nuisance from implementation are part of this phase.

Phase 4: Assessment of construction in future situation and final design (DO, in in accordance with Chapter 7 of the ROK) reinforcement measures: final calculations made of the reinforced bridge construction with an arithmetic substantiation of the reinforcements. Final design drawings are drawn up on the basis of whose design can be elaborated into an implementation design. A proposal for one optimal implementation phases are drawn up.

Phase 5: Drafting demand specification for the realization contract: Support the Client when tendering for the realization contract and testing it.

This report gives the result of **Phase 1 (note of principles)** .

2 Inventory of archive data

2.1 General history

Below is an overview of the history of the IJssel Bridge.

- 1941 Construction of embankment / abutments
- 1943 Construction ceased due to World War II
- 1961 Opening of the eastern IJssel bridge
- 1964 Opening of the western IJssel bridge
- 1975 Widen both bridges with inspection paths on extended consoles
- 1977 Preservation of the eastern IJssel bridge
- 1985 Repair concrete pillars
- 1989 Conservation of the western IJssel bridge
- 1990 Repair concrete deck / curved edges on both bridges and apply ZOAB
- 1990 Driving direction of the eastern IJssel bridge reversed due to the opening of the third IJssel bridge (concrete bridge)
- 2007 Restore concrete pillars and abutments

Some of these events are explained in more detail below

1941-1943 Build substructure

Before the Second World War, construction of the land additions to the A12 started.
According to the Rijkswaterstaat image bank, the abutments and possibly also the pillars for the 2 e made world war. In 1943 the construction of the A12 is suspended in connection with the two e world.

Figure 6 - Construction of embankment / land fill (1941) [Source: https://beeldbank.rws.nl, Rijkswaterstaat / Auke Leen]

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1959-1961 Build the eastern IJssel bridge
After the Second World War, the eastern IJssel bridge was first realized in 1961. The individual bridge sections are manufactured in Dordrecht (main bridges) and Rotterdam (bridges) and these are finished pontoons brought to the location in the IJssel and riveted together.

Figure 7 - Construction of bridges [Source: https://beeldbank.rws.nl, Rijkswaterstaat / Auke Leen]

Figure 8 - Construction of the bridge [Source: https://beeldbank.rws.nl, Rijkswaterstaat / Multimedia Department Rijkswaterstaat]

Figure 9 - Bridge concrete decking [Source: https://beeldbank.rws.nl, Rijkswaterstaat / Auke Leen]

Figure 10 - Construction of the northern approach [Source: https://beeldbank.rws.nl, Rijkswaterstaat / Multimedia Department Rijkswaterstaat]

1964 Construction of the western IJssel bridge completed
The western IJssel bridge was built immediately after the eastern one and was completed in 1964.

1975 Widening of the bridge with inspection paths
In 1975 the bridge was widened by extending the consoles with a steel profile, with walkways on it for an inspection path. This was done over the entire length of the two bridges, on both sides of every bridge.

1990 Build 3 e IJsselbrug
In 1990, the third e IJsselbrug (concrete bridge) were completed. With the availability of this bridge is large-scale maintenance on the eastern bridge. After performing the maintenance, the direction of the eastern bridge has changed, so that traffic now drives from Arnhem to Germany.

2.2 General construction description

The IJssel Bridge is located in the A12 over the IJssel near Arnhem / Velp. The bridge consists of several separate bridges, both steel bridges (main bridges) and steel-concrete bridges (bridges).

The Eastern Bridge now houses the main carriageway of the A12 with 2 lanes in the direction of Germany. The Westelijke Brug provides space for a parallel carriageway between the Velperbroek junction and the exit Westervoort. In the past, the lane layout has been different, as in section 2.1 explained. Both bridges have a largely identical design and each consists of 3 bridge sections separated by joints: 2 bridges, each with 3 fields that are completely above the Northern floodplain, and a main bridge with 5 fields, the middle of which crosses the main bridge of 105m the river IJssel. In total, both bridges are 536 meters long.

2.2.1 Bridging

Both bridges (East and West) each consist of 2 separate parts with 3 squares on 4 supports (separated by a joint) with spans of approximately 40 meters. The The main supporting structure of the bridges consists of 2 steel main girders with rivets in between crossbars and crossbeams with a concrete floor. The steel construction and the concrete deck work constructively together.

Figure 11 - Side view from the west on the bridges [A.22646]

Figure 12 - Cross section of one of the bridges [A.22646]

A more detailed description of the construction of the bridges is described in chapter 8.1.

2.2.2 Main bridges

The main bridge is an all-steel construction with 5 fields on 6 supports, consisting of 2 variable height main girders with crossbars riveted and crossbeams in between then a steel orthotopic floor with open continuous stiffeners (bulbs).

Figure 13 - Side view from west on main bridges [A.22646]

Figure 14 - Cross section of one of the main bridges [A.22646]

A more detailed description of the construction of the main bridge is described in chapter 7.1.

2.2.3 Substructure

The substructure consists of 10 concrete pillars with masonry cladding, and 2 concrete abutments with masonry cladding. The concrete abutments consist, from bottom to top, of foundation piles with a slant 1: 4, a concrete floor, concrete walls finished with masonry, with a concrete on top L-wall with wing walls.

Figure 15 - Cross section and front and rear view of northern abutment [C.4348]

The pillars A, B, C, D, E, F, G and K are based on piles and consist of a concrete floor slab, concrete walls finished with masonry and a concrete top.

Figure 16 - Side and top view and cross section of pillars A, B, C, D, E, F, G and K [C.4461]

The river pillars H and J are basically the same as the other pillars, but are based on steel, on an underwater concrete floor with sheet piling all around.

Figure 17 - Side and top views and cross section of pillars H and J [C.4461]

3 Preconditions and principles for recalculation

3.1 Available documents

Rijkswaterstaat, as client, has produced a large number of drawings, calculations, specifications, inspection reports and other documents made available. An overview of all available documents are attached in Annexes A to D.

- Appendix A: Drawings
- Appendix B: Calculations
- Appendix C: Specifications
- Appendix D: Other documents (Inspection reports, investigations, etc.)

References to these documents are indicated in square brackets [xxx] in this report. The reference designations are included in the relevant Annexes. As much as possible uses the document numbers of Rijkswaterstaat.

3.2 Standards and guidelines

3.2.1 Standards

code	title	Organization
	Design new construction	
NEN-EN 1990 + A1 + A1 / C2: 2011 NEN-EN 1990 + A1 + A1 / C2 / NB: 2011	Foundations of the structural design	NAND
NEN-EN 1991-1-1 + C1: 2011 NEN-EN 1991-1-1 + C1 / NB: 2011	Loads on structures - Part 1-1: General loads - Volumic weights, own weight and taxes imposed on buildings	NAND
NEN-EN 1991-1-4 + A1 + C2: 2011 NEN-EN 1991-1-4 + A1 + C2 / NB: 2011	Loads on structures - Part 1-4: General taxes - Wind load	NAND
NEN-EN 1991-1-5 + C1: 2011 NEN-EN 1991-1-5 + C1 / NB: 2011	Loads on structures - Part 1-5: General loads - Thermal load	NAND
	Loads on structures - Part 1-7: General	

NEN-EN 1991-1-7 + C1 + A1: 2015 NEN-EN 1991-1-7 + C1 / NB: 2011	taxes - Extraordinary taxes: shock loads and explosions	NAND
NEN-EN 1991-2 + C1: 2015 NEN-EN 1991-2 + C1: 2011 / NB: 2011	Loads on structures - Part 2: Traffic load on bridges	NAND
NEN-EN 1992-1-1 + C2: 2011 NEN-EN 1992-1-1 + C2: 2011 / NB: 2016	Design and calculation of concrete structures - Part 1-1: General rules and rules for buildings	NAND
NEN-EN 1992-2 + C1: 2011 NEN-EN 1992-2 + C1: 2011 / NB: 2016	Design and calculation of concrete structures - Part 2: Bridges	NAND
NEN-EN 1993-1-1 + C2 + A1: 2016 NEN-EN 1993-1-1 + C2 + A1: 2016 / NB: 2016	Steel Structure Design and Calculation - Part 1-1: General rules and rules for buildings	NAND
NEN-EN 1993-1-5 + C1: 2012 NEN-EN 1993-1-5 + C1: 2012 / NB: 2011	Steel Structure Design and Calculation - Part 1-5: Constructive plate fields	NAND
NEN-EN 1993-1-8 + C2 + C11: 2016 NEN-EN 1993-1-8 + C2 + C11: 2016 / NB: 2011	Steel Structure Design and Calculation - Part 1-8: Design and calculation of connections	NAND
NEN-EN 1993-1-9 + C2: 2012 NEN-EN 1993-1-9 + C2: 2012 / NB: 2011	Steel Structure Design and Calculation - Part 1-9: Fatigue	NAND

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NEN-EN 1993-2 + C1: 2011 NEN-EN 1993-2 + C1: 2011 / NB: 2011	Steel Structure Design and Calculation - Part 2: Steel bridges	NAND
NEN-EN 1994-1-1 + C1: 2011 NEN-EN 1994-1-1 + C1: 2011 / NB: 2012	Design and calculation of steel-concrete structures - Part 1-1: General rules and rules for buildings	NAND
NEN-EN 1994-2 + C1: 2011 NEN-EN 1994-2 + C1: 2011 / NB: 2011	Design and calculation of steel-concrete structures - Part 2: General rules and rules for bridges	NAND
	(Additional) Assessment of existing construction	
NEN 8700: 2011	Constructive safety assessment of a existing building work during renovation and remodeling - Principles	NAND
NEN 8701: 2011	Constructive safety assessment of a existing building work during renovation and remodeling - Taxes	NAND

Table 1 - Standards overview

3.2.2 Guidelines

Code	Title	Organization
RTD 1006: 2013	Guideline Assessment of Artworks RBK v 1.1 (2013); May 27, 2013 (referred to as RBK in report)	RWS
RTD 1001: 2017	Guideline for Artworks SKIRT v 1.4 (2017); April 2017 (referred to as ROK in report)	RWS
RTD 1007-2: 2014	Requirements for joint transitions v3.0 (2014; December 1 2014)	RWS
RTD 1012: 2017	Requirements for bridge supports (2017); February 21, 2017	RWS

Table 2 - Overview of guidelines

3.2.3 Contract documents

Code	Title	Organization
Demand specification 31128403	Recalculation and Design Reinforcements IJsselbrug A12	RWS
TNO-2017-R10480	RBK Staal dated 31-1-2017 - Start document for NEN committee 3510010204 Existing Steel Construction for drawing up NEN 8703	TNO
TNO-2017-R10405	Detail categories for fatigue rivet joints and of orthotope riding decks with open stiffeners	TNO

Table 3 - Overview of contract documents

3.2.4 Standards and regulations at the time of the design of the existing bridge

The bridge was designed in the late 1950s according to VOSB 1963 (Source [BBV-0010-01]). This is based on of Load class 60. The later extended consoles are also calculated in accordance with the VOSB. Also the substructure is designed for this load (Source [Archive Calculation Foundation]).

Code	Title	Organization
VOSB 1963	Regulations for the Design of Steel Bridges (VOSB 1963)	NAND

Table 4 - Overview of other standards and guidelines at the time of the bridge's design

3.3 Residual life, consequence and reliability class

3.3.1 Residual life and reference period

The residual life and reference period are shown in the table below for the various assessment levels. The actual road layout is calculated for the traffic load, where distinction is made in the V1 normal situation and V2 emergency. The latter situation applies a reduced reference period of 1 month per 5 years of residual life.

Security level	Residual life [year]	Reference period [year]
New construction	100	100
Renovation	**30**	**30**
Usage - V1 normal situation	**30**	**30**
Use - V2 emergency	**30**	**0.5**
Disapproval	1	15

Table 5 - Remaining life and reference period

The existing components are tested at the level of “use”. The reinforcements are designed at the level of "renovation". A residual life of 30 years is assumed (from 1/1/2021).

3.3.2 Consequence class

The bridge is a bridge over a main waterway.

The bridge is classified in consequence class CC3, in accordance with table RBK 1.1 pag. 8 art. 1.3.

3.3.3 Trust class

The construction is classified on the basis of consequence class CC3, in reliability class 3, RC3.

4 Material data

4.1 Structural steel

4.1.1 Material Properties

Different qualities of structural steel are used in the IJssel Bridge. In chapter 7.1 (main bridge) and Chapter 8.1 (bridging) determines the applied steel quality per construction component. In main lines is applied for the main beams and the steel deck LQmc 52 and the other parts in Qmc 37. The later extended consoles are made in Fe 360

The material properties of the structural steel used are given in the table below.
The properties have been determined on the basis of RBK Staal-Startdocument NEN 8703 of TNO [TNO-2017-R10480].

Steel type	Yield stress f_y [N / mm²]	Tensile strength f_u [N / mm²]	Elasticity modulus E_s [N / mm $_2$]
Qmc 37	215	340	210000
LQmc 52	350	510	210000
Fe 360	240	360	210000

Table 6 - Overview of material properties of structural steel

4.1.2 Cross-section classification

The cross sections are classified in accordance with NEN-EN 1993-1-1 art. 5.5. Because it depends on the steel tension, it is determined in the verification calculation.

4.1.3 Partial material factors

The partial factors to be used for determining the resistance of the components under consideration are in the table below. In accordance with the specifications [BR 205] and [BR 104], mild steel is used, for which, in accordance with the RBK Steel Start Document NEN 8703 [TNO-2017-R10480], the partial factors for steel bridges according to NEN-EN 1993-2 may be used (NEN-EN 1993-2 / NB table NB.2).

Description	Symbol □ $_m$ [-]	
Cross section capacity		
Cross-section resistance to the crossing the yield strength including locally pleating	0 $_{M0}$	1.00
Resistance of bars to instability determined at review	□ $_{M1}$	1.00
Resistance of sections in tension to fracture	□ $_{M2}$	1.25
Capacity of connections		
Resistance of bolts, rivets, pins, welds and crush resistance of plates	□ $_{M2}$	1.25

Table 7 - Overview of structural steel partial factors

4.2 Rivets

4.2.1 Material properties

In addition to welding, the IJsselbrug also uses various rivet connections. In the past, the steel type of rivets selected for the material in which it was used. The review on strength of the rivet is therefore related to the steel type of the sheet material. In the cutlery [BR 2370] and [BR 2755] it is therefore stated that the rivets used in Qmc 52 are manufactured must be made of "high-quality steel".

Figure 18 - Material properties according to specifications [BR 2370]

This is also reflected in the various drawings, where two types of steel are used for the rivets LQmc 34 and LQmc 42. The material properties of the rivets used are stated in the table below. The properties have been determined on the basis of table 6.3 from the RBK Staal-TNO start document NEN 8703 [TNO-2017-R10480].

Steel type Rivet	Period of time from	Old standard	Calculation value strength f_{ur} [N / mm²]
(L) Qmc 34	1952	V 1035 -IV	400
LQmc 42	1952	V 1035 -IV	600

Table 8 - Overview of material properties for rivets

4.2.2 Partial material factors

The partial factors to be used for determining the resistance of the components under consideration are in the table below. These are used in accordance with NEN-EN 1993-1-8.

Description	Symbol □ m [-]	
Capacity of connections		
Resistance of bolts, rivets, pins, welds and crush resistance of plates	□ M2	1.25

Table 9 - Compound partial factors overview

4.3 Concrete

4.3.1 Material properties

Concrete is mainly used in the bridges (concrete deck) and the substructure of both bridges (abutments / pillars).

Concrete deck

The concrete deck consists partly of a reinforced concrete deck and partly of a prestressed concrete deck. In Both parts use the same concrete with a cube compressive strength at 28 days of at least 45 N / mm² [BR 205] AND [BR 104]. This corresponds to a concrete grade K450.

Ramp edges
The same specification also mentions another concrete mixture for the cut edges with a lower cement content. However, there are no requirements for cube compressive strength in the specifications, which assumes the lowest quality according to the GBV 1950. This is assumed to be the case the sheep sides have not been supervised, so K150 is assumed. For the calculation it is assumed that the curved sides do not cooperate constructively.

Abutments and pillars [1]
According to specifications [BR 73], the abutments and pillars consist of different types of concrete, namely concrete for the reinforced concrete piles, underwater concrete, concrete for the work floors, fat concrete behind masonry (with shrinkage reinforcement), reinforced concrete and stamped concrete, each with a different cement content. However, the specifications do not make any demands on the cube compressive strength, as a result of which the lowest quality according to the GBV 1930 is assumed (f_{ck} = 8 N / mm²).

[1] For the time being, no construction calculations of the substructure are being made. Based on a comparison of support reactions between the design calculation and the recalculation will be advised whether it is necessary to also consider the substructure in more detail.

Norm	Quality	Current name cf Eurocode	$f_{ck, cube}$ [N / mm²]	f_{ck} [N / mm²]	f_{cd} [N / mm²]	f_{ctm} [N / mm²]	f_{ctd} [N / mm²]
GBV 1930		C8 / 10	10	8	5.3	1.20	0.56
GBV 1950 K150		C8 / 10	10	8	5.3	1.20	0.56
GBV 1962 K450		C32 / 40	40.2	32	21.3	3.02	1.41

Table 10 - Concrete material properties

4.3.2 Shrink and creep properties

Shrinkage and creep calculation are based on RH = 80% and CEM 32.5 N with cement class S. De concrete pressure stress remains below 45% of f_{ck} (t_0), so that linear creep may be assumed. The stiffness of concrete depends on the type of load and the construction phasing and is explained in more detail at the description of the modeling of the bridges.

4.3.3 Partial material factors

The following material factors are used for concrete, in accordance with NEN-EN 1992-1-1.

Design situation	γ_C
Permanent and temporary	1.50
Out of the ordinary	1.20
Fatigue	1.50

Table 11 - Overview of partial material factors for concrete

4.4 Reinforcing steel

4.4.1 Material properties

Different qualities of reinforcing steel are used in the IJssel Bridge.

Concrete deck
According to the specifications [BR 205] and [BR 104], both QR24 and QR42 are used in the concrete deck, whereby QR 42 has been used as main reinforcement and QR 24 as secondary reinforcement for the non-structural elements

Abutments and pillars [1]
The specifications [BR73] and the drawings of the substructure do not represent a reinforcing steel type. In the cutlery it is reported that there is a deviation from GBV 1930 (with regard to keeping the concrete). On this basis, it is assumed that the reinforcement has been designed in accordance with GBV 1930. Er Therefore, in accordance with table 2.6, reinforcing steel type 1 is assumed.

[1] For the time being, no construction calculations of the substructure are being made. Based on a comparison of support reactions between the design calculation and the recalculation will be advised whether it is necessary to also consider the substructure in more detail.

The material properties of the rebar are summarized in the table below. The material properties of the reinforcement are in accordance with the RBK art. 2.6.1 (table 2.6) is maintained.

Reinforcing steel type	f_{yk} [N / mm²]	f_{yd} [N / mm²]	Ductility class	Slippery/ ribbed
1. B	220	191	B	Slippery
QR 24	240	209	B	Ribbed
QR 42	420	300 [1]	B	Ribbed

[1] Based on the voltage allowed in the original standard, a lower one is used value for f_{yd} derived then which follows from art. 3.2.7 (2) based on the information given here value of f_{yk} .

Table 12 - Overview of material properties of reinforcing steel

4.4.2 Partial material factors

The material factors in accordance with the Eurocode do not always apply to older rebar types. In the RBK art. 2.6.1 contains both the characteristic and calculation values of the material strengths, which should be used.

4.5 Prestressing steel

4.5.1 Material Properties

In the IJsselbrug, prestressing reinforcement has been applied in the concrete deck at the location of the intermediate supports the shape of prestressing bars Ø 26 mm [BR 205], [BR 104]. According to drawing [A.27268] these are Dywidag bars. In accordance with Appendix B4 of the RBK, Dywidag are prestressing bars from 1960 and later QP80 / 105. This corresponds to the force of 30 tons (permanent) stated in [BBV-0010-00]. In this calculation is based on 10% span. It should be noted that the status of this calculations are unknown, the calculations give the impression that they are draft calculations.

Assuming A_p = 531 mm² and a force of 30 tons, a work preload of 565 is found N / mm², which corresponds to QP105. The initial prestressing is 622 N / mm² (33 tons).

The material properties of the prestressing steel are summarized in the table below. The material properties of the prestressing are in accordance with the RBK art. 2.6.1, Table 2.7, maintained.

Steel grade	f_{hp} [N / mm²]	f_{pk} / □ s [N / mm²]	$f_{p0.1k}$ [N / mm²]	f_{pd} [N / mm²]	□ uk [%]	Allowable initial-preload [1]	Allowable work voltage [2]
QP105	1030	936	785	713	3.5	670	567

[1] The permissible initial voltage given is the voltage as permitted by the design standard. This is the value that was supposedly designed / prestressed at the time.
[2] The permissible operating pretension given is the voltage as permitted in accordance with the design standard. This is the value that was supposedly designed at the time. Friction will make the work preload even lower.

Table 13 - Overview of material properties of prestressing steel

The minimum compressive strength of the concrete, at the time of tightening the prestressing steel, is according to the specifications 40 N / mm² (eastern bridge) and 37.5 N / mm² (western bridge). The prestressing channels are after it cocked injected.

4.5.2 Partial material factors

In the RBK art. 2.6.1, table 2.7, shows both the characteristic and the calculation values of the material strengths to be used. The partial factors correspond to the standard factors according to NEN-EN 1992-1-1 art. 2.4.2.4.

Design situation	□ s
Permanent and temporary	1.1
Out of the ordinary	1.0
Fatigue	1.1

Table 14 - Overview of partial material pre-stress factors

5 Construction phasing

5.1 Bridging

The two two bridges each have a length of 120 m. The construction of the bridge is standing described on the drawings and in the guide in folder 1900 folder 4c [H105] of the archive of Rijkswaterstaat. The bridge was made in parts in the factory and then transported by boat. With help the bridge sections of the river have rolled from a roller conveyor to the abutment. The concrete deck of bridges consists of a prestressed concrete deck at the intermediate supports and a reinforced one concrete deck at the location of the other parts. According to the specifications [BR 205] and [BR 104] and the assembly drawing [H 105-51] the construction phasing consists of the following steps;

1. Placing steel construction. The top flange of the main beam is not yet in this phase connected to the body over a length of 13 m at each intermediate support point (for the preload);
2. Pouring prestressed concrete deck;
3. Applying the prestressing force;
4. Riveting the top flange of the main beam to the body at the intermediate supports;
5. Jacking the bridge with 400 mm at the intermediate supports;
6. Pouring reinforced concrete deck;
7. Lower intermediate supports to the original height;
8. Installing cut edges, pavement and road furniture on the deck.

After this last stage, shrinkage and creep of the concrete deck and the prestressing loss of the prestressing will occur performance. A number of construction phases are shown in the figures below.

Figure 19 - Rolling the bridges to the abutment

Figure 20 - Temporary supports during mounting

Figure 21 - Pouring prestressed deck (before jacking) and reinforced concrete deck (after jacking) of the bridges

During construction, the steel construction is temporarily supported midway through the spans.
It is assumed that the placing on sow is (almost) voltage-free, so that this is not taken into account
is taken into account in the calculation. This also applies to the curvature of the tee during prestressing.
This is not done in the design calculation either.

The following construction phases are used for modeling in SCIA Engineer. For phase 3
made use of a separate model of the prestressed concrete deck + top flange (T-piece) of the
main beam. The results of this model are manually combined with the results of the
global model. For further explanation of the modeling, see chapter 8.4.

Phase Model	Change of construction	Tax case
Phase 1 Global model	Add steel construction with tpv main beam intermediate support only lower flange and web	Own weight of steel
Phase 2 Global model -		Bulk density prestressed concrete deck
Phase 3 Model concrete deck	Separate model with prestressed concrete deck (eg = 0) + top flange main beam (T)	Preload
Phase 4 Global model	Add top flange main beam at intermediate support + prestressed concrete deck	Jacking
Phase 5 Global model -		Bulk density reinforced concrete deck
Phase 6 Global model Add reinforced concrete deck (eg = 0)		Indulgences
Phase 7 Global model -		Asphalt pavement
Phase 8 Global model -		Other permanent taxes
Phase 9 Global model -		Shrink and creep
Phase 10 Global model -		Prestressing loss
Phase 11 Global model -		Variable taxes

Table 15 - Overview of modeled construction phases

5.2 Main bridge

The construction of the main bridge is described in the drawings and in the guide in folder 1900 folder 4c [H105] from the Rijkswaterstaat archive. The bridge is made in parts in the factory and then per boat landed. Drawing [H105-52-A] gives an overview of the construction phasing and consists of the subsequent phases.

1. Placing the loose pieces of the part 0-7;
2. Rolling section 0-7 to the southern abutment;
3. Changing the roller track and placing the part 7-9;
4. Placing the section 9-12;
5. Placing the section 12-14;
6. Placing and rolling part 27-34 (as under 1 and 2);
7. Placing the section 20-27 (as under 3.4 and 5);
8. Placing the keep 14-20.

A number of construction phases are shown in the figures below.

Figure 22 - Situation after rolling part 0-7 and placing part 7-9

Figure 23 - Placing section 9-12

Figure 24 - Placing the keep 14-20

During construction, the construction at the spans across the flood plains is temporary supported. The parts between river pier H and axis 14 and between river pier J and axis 20 collars in the first body, after which the keep will be hung between them and the beam will be the continuous construction go to work.

With the help of a construction phase analysis, the difference in (Von Mises) tension was investigated of the own weight is between a calculation with and without taking into account the construction phasing.

Construction phase	Field	Support center (pillar H)
Without construction phases	□ $_s$ = 52.5 N / mm²	□ $_s$ = 38.3 N / mm²
With construction stages	□ $_s$ = 47.6 N / mm²	□ $_s$ = 51.3 N / mm²

It follows from the calculation that a (limited) difference in stresses occurs in the field (□□ $_s$ = 5 N / mm²). At the point of support of pillar H, the difference is somewhat larger (□□ $_s$ = 13 N / mm²), which is approximately 4% of the yield stress is. The influence on the calculation time seems limited for the time being, so it has been decided to use the construction phases for the main bridge to be included in the Finite Element Model.

6 Taxes

6.1 Permanent taxes

The permanent loads are divided into the load due to the weight of the steel, the prestressed concrete and the reinforced concrete, the prestressing and associated losses, jacking and releasing the intermediate supports, shrinkage and creep of the concrete deck and the other permanent ones taxes. The numbering of the load cases is in order of the construction phases of the detained. For the main bridge, only BG1 and BG9 apply.

6.1.1 Own weight steel (BG1)

The own weight of the steel construction is automatically generated by SCIA Engineer. Hereby is based on the following volumic weight.

- Structural steel □ $_s$ = 7850 kg / m ^{3}

For the steel parts a surcharge will be charged for extra steel such as rivets, gusset plates, connecting plates and stiffening ribs. This supplement is determined per part, and included in Appendix M.

For the main bridge, a distinction is made between two construction phases, BG1a corresponds to its own weight of part 0-14 and part 20-34, BG1b corresponds to the dead weight of part 14-20, see also paragraph 7.4.1.1.

6.1.2 Own weight of prestressed concrete (BG2) and reinforced concrete (BG5)

The concrete deck of the bridges has been chosen for the load due to the (bulk) weight of the concrete as line loads on the steel beams. It works at the time of depositing concrete not yet together with the steel as a cooperating steel-concrete beam. This causes the full load on the concrete carried by the steel structure alone.

The loads from the concrete deck have been translated into loads on the main beams and the cross beams. In order to determine the load on the different beams, the load has been cut into for simplification one part because of the concrete deck with a constant thickness of 200 mm and one part because of the rib above the main beams (variable 0-100 mm).

The constant thickness deck is assumed to be loaded through a tax envelope at 45 ° to the side members and cross members. In Figure 25 is shown the payment, where the steel construction is shown in green and the direction of the tax payment in red. It cantilevered part of the deck outside the main beams is transferred directly to the main beam, with exception to the part at the joint transitions, where a steel console is present. Here is the load also distributed at 45 ° to the console and the main beam. The rib is assumed to be this transfers its load directly to the main beam.

Figure 25 - Principle tax payment bulk density concrete (top view deck)

In the calculation, a distinction is made between the tax due to the depositing of the prestressed concrete and the reinforced concrete, because they occur at different times in the construction phases. In

BG2 takes into account the weight of the prestressed concrete, in BG5 the weight of
charged the reinforced concrete deck. The following is taken into account
volumic weight:

- Precast concrete □ $_c$ = 2500 kg / m ^{3}

The width of the prestressed section is 6 m on either side of the intermediate supports (total 12
m per intermediate support point). The rest is reinforced concrete.

6.1.3 Pre-tension (BG3) and pre-tension loss (BG10)

The effect of bias (BG3) is modeled as a load in a separate finite
element model, because at the time of prestressing the top flange of the main beam and the
crossbeam is not connected to the rest of the construction. The results from tax case BG3
are added manually to the global model. The pretension losses (BG8) occur as the
top flange and crossbars are connected to the rest of the structure and are therefore in it
global model applied. See also the explanation in section 5.1 .

Build up prestressing
Preloads are applied from 6 m up to 6 m after each intermediate support point in the longitudinal direction
from the bridge. In the deck, 44 Dywidag rods Ø26 mm are used per intermediate support point [A.27268],
[C.9366], [BR 205], [BR 104].

Figure 26 - Top view bridging deck with prestressed part [A.27268]

The center-to-center distances of the prestressing bars vary from 170 mm to 315 mm, measured horizontally
and are located 100 mm from the bottom of the concrete deck. Spot the rib (thickened part) above
in the main beams, the pretension is therefore eccentric in the concrete deck, in the other part centric. The
rods are straight.

Figure 27 - Deck cross-section with pre-load position [C.6399]

The bars are threaded and anchored in accordance with specifications [BR 205], [BR 104] anchor plate and a nut. The bars are tensioned and anchored on one side. The casing tubes Ø40 mm have been injected with cement mortar after prestressing. The rod ends after injection burned down and concreted.

Preload force and losses

In accordance with Appendix B4 of the RBK, Dywidag prestressing bars from 1960 and later are QP80 / 105. This comes corresponds to the force of 30 tons stated in calculation [BBV-0010-00] (permanent, after time-dependent to lose). This calculation assumes 33 tons of initial prestressing force and approximately 10% time-dependent losses due to creep, etc. In accordance with RBK 2.6.1, article 3.1.4 (7), based on the pretension losses as charged at the time of construction. The losses seem somewhat low, however, because in accordance with the RVB 1962 alone, relaxation caused about 8-10% loss will occur. On the other hand, due to the applied anchorage method (anchor plate with nut) and straight bars hardly wedge and friction occur. Also, prestressing was only allowed if the concrete had a minimum cube compressive strength of 37.5 N / mm² (western bridge) and 40 N / mm² (eastern bridge), as a result of which part of the curing shrinkage has already taken place. When the losses are in accordance with the RVB would be determined, approximately 17% loss is found due to shrinkage, creep and relaxation and approximately 3% wedge set by wedge set + friction.

In consultation with Rijkswaterstaat, it was decided to initially assume a 10% pretension loss in accordance the design calculations. Based on the load case bias loss, it will be assessed whether this has a lot of influence on the assessments and it will be decided, in consultation with Rijkswaterstaat, whether one lower / upper limit approach is necessary.

Assuming A_p = 531 mm² and a force of 33 tons, an initial prestress of 622 N / mm², well below the permissible initial tension of 670 N / mm² according to table 2.7 of the RBK. The work preload, including time-dependent losses, comes down to the original calculation at 565 N / mm², which corresponds to the working preload associated with QP105 ..

$_0 = 622/ _2$

$_0 = \cdot _{,?\,0} = 531 _2 \cdot 622/ _2 = 3300 /$

$_{,\infty} = 565/ _2$

$_{,\infty} = \cdot _{,\infty} = 531 _2 \cdot 565/ _2 = 3000 /$

To enter the prestressing force into the model, the prestressing force is split into two construction phases, to take into account the construction phases.

1. Initial prestressing force at t = 0 after curing concrete (BG3);
2. Preload loss at t = ∞ (BG 8).

As mentioned, tax case BG3 is considered in a separate model. Above mentioned powers have been introduced as a line load across the width of the concrete deck (9.44 m), including in the global model for the pretension losses is an opposite force of 10% of the initial pretension force apprehended. At the location of the ribs, the eccentricity of the pretension has been taken into account (e_p) relative to the system line of the concrete deck (center of the concrete deck), by entering one line moment. In the oblique part, the eccentricity is assumed linear, decreasing to 0 mm on the spot from the thin part of the rib.

Project related

$e_{p,b}$ =
0 mm

$e_{p,a}$ =
100 mm

$e_{p,b}$ =
0 mm

preload

e_p

q_p

system line

m_p

Figure 28 - Eccentricity of the pretension relative to the system line [C.6399]

q_p = 44 cables * 330 kN / cable / 9.44 m = 1538 kN / m
$m_{p,a}$ = 1538 kN / m * 100 mm = 154 kNm / m
$m_{p,b}$ = 1538 kN / m * 0 mm = 0 kNm / m

6.1.4 Jacking (BG4) and draining (BG6)

After adding the slabs for the precast concrete deck, the bridges are added
jacked at the location of the intermediate supports. To this end, an imposed distortion of the 4 is applied in SCIA
bearings of the intermediate supports applied of 400 mm in z-direction according to drawing [A.23791]
and calculation [BBV-0010-00].

After adding the slabs for the reinforced concrete deck, the bridges are added
lowered at the location of the intermediate supports. To this end, an imposed distortion is imposed in SCIA
of the 4 supports of the intermediate supports applied from -400 mm in z-direction.

6.1.5 Asphalt pavement (BG7)

Based on the archive information, the asphalt thickness of the various bridges is as good as possible
obsolete, both in accordance with the design and the current thickness.

The design thickness of the pavement for both bridges is 50 mm [BR 2571], [BR 3126], [A.50973]. In
1996 this was replaced on the steel bridges for ZOAB (30-35 mm) on DAB (25 mm) with two Parafor
membrane layers (2x5 mm) [VI Asphalt 1998]. This brings the total thickness to approx. 70 mm. For the
it was not possible to determine the thickness of the asphalt pavement applied, although it
it is plausible that the same thickness has been applied here, since the decks are in line with each other and side
from the design had the same design thickness of the asphalt.

It was therefore decided to determine the asphalt thickness during an inspection of the western bridge. Hereby is the height difference between the top of the asphalt and the top of the side of the curb (bridging) and the edge strip (main bridge) measured and the thickness of the asphalt pavement is derived from this. The designation west and east in the table below, refers to the flank side on the west and east sides, in both cases of the western bridge.

Bridge			Main bridge		
Location	west	east	Location	west	east
LH north	85		Pillar F		80
Pillar A	105		Pillar G	75	60
Pillar B	105		Pillar H	75	95
Pillar C	115		Pillar J	100	90
Pillar D	85	75	Pillar K.	90	80
Pillar E	85	85	LH south	85	70
Pillar F	65				

Table 16 - Asphalt thickness based on height measurements bk scrap edge - bk pavement

As can be seen, the thicknesses are greater than conforming specifications. It has been agreed in consultation with Rijkswaterstaat based on the average asphalt thickness per bridge. For the distribution of the wheel loads based on 70 mm, in accordance with the specifications.

The table below gives an overview of the applied asphalt thicknesses. The current is calculated asphalt thickness.

	Asphalt thickness design	Source	Current asphalt thickness	tax	Source
Main bridge east	50 mm	BR 2571, A.50937	82 mm	1.89 kN / m²	Measurement based on height bk bark side
Main bridge west	50 mm	BR 3126, A.50937	82 mm	1.89 kN / m²	Measurement based on height bk bark side
1 e aanbrug	50 mm	A.50937	102.5 mm	2.36 kN / m²	Measurement based on height bk bark side
2 e aanbrug	50 mm	A.50937	85 mm	1.96 kN / m²	Measurement based on height bk bark side

Table 17 - Overview of asphalt thickness in accordance with design and current asphalt thickness

Based on the specific gravity of the asphalt pavement, the load is maintained as follows.

- Specific weight of asphalt pavement $\rho_a = 2300\ kg / m^3$
- Asphalt width $b_a = 8250$ mm

Initially, the bridges will be charged with the high load. If it appears that the bridge does not meet strength, a distinction will be made between the 2 bridges.

NB The build-up of the asphalt pavement in accordance with the ROK, as required in, is not taken into account Article 2.4.1. of the RBK. This item only applies to concrete bridges, not to steel-concrete or steel bridges.

6.1.6 Other permanent taxes (BG8)

In addition to the pavement, various permanent loads are present on the bridge. Appendix E gives the calculation of the resting taxes and the associated source on which the tax is based. The results of this calculation are summarized below.

Ramp edge (BG8a)
The shear side is loaded by the load on the guardrail (both bridges) and on the concrete (all at the bridges).

$q_{guardrail}$

p_{edge} $p_{bark\ side}$

$q_{guardrail}$

Edge
(h = 500 mm)

Figure 29 - Loads on the cut edge of the main bridge (left) and bridge (right)

Schampkant concrete deck	$p_{grazing\ side}$ =	5.88 kN / m² over 595 mm width
Edge edge	p_{edge} =	12.5 kN / m² over 150 mm width
Guardrail	$q_{guardrail}$ =	0.60 kN / m

In the SCIA model, the load on the edge of the cut edge is translated into a line load and a moment on the edge of the (structural) concrete deck

Edge edge	q_{edge} =	0.15 m * 12.5 kN / m² =	1.88 kN / m $_1$
	m_{edge} =	1.88 kN / m1 * ½ * 0.15 m =	0.14 kNm / m $_1$

Normal console inspection path next to the bridge deck (BG8b)
The load from the inspection path is caused by the console, the handrail, the slatted floor and the cable tray for lighting.

$q_{handrail}$ $p_{inspection\ path}$ $q_{console}$

$q_{handrail}$ $p_{inspection\ path}$ $q_{console}$

$F_{cable\ tray}$ 705 mm

$F_{cable\ tray}$ 470 mm

Figure 30 - Loads inspection path main bridge (left) and bridge (right)

Handrail	$q_{handrail}$ =	0.20 kN / m
Inspection path	$p_{inspection\ path}$ =	0.40 kN / m²
Cable tray lighting	$q_{cable\ tray}$ =	0.75 kN / m
Console	$q_{console}$ =	0.11 kN / m

For the main calculation (global model) the above loads are translated to a load (point load and moment) at the end of the original console (main bridge) or the edge of the (structural) concrete deck, see appendix E. The center-to-center size for the bridges is 2.0 m, for the main bridge between 1.75 m and 1.80 m.

Reinforced console inspection pad with lighting (BG8c)

A lighting mast is present at the location of the "reinforced" consoles. The consoles are also something here longer and have a larger diameter (280 * 280 * 11 mm). The other taxes are as above described.

$q_{handrail}$ $F_{lighting}$ $p_{inspection\ path}$

$q_{handrail}$ $F_{lighting}$ $p_{inspection\ path}$

705 mm $F_{cable\ tray}$

$F_{cable\ tray}$ 470 mm

Figure 31 - Loads inspection path at reinforced main bridge console (left) and bridge (right)

Lighting:	$F_{lighting}$ =	3.00 kN
Console:	$q_{console}$ =	1.00 kN / m

The above loads are translated into a load (point load and moment) at the end of the original console (main bridge) or the edge of the (structural) concrete deck), see Appendix E. The reinforced consoles with lighting are at a height of approx. 20 m [A.50937].

Inspection path under the bridge (BG8d)

The inspection path under the bridge is charged as 2 point loads on the cross beams, c 700 mm, symmetrical to the center of the cross bars.

700 mm

$F_{inspection\ path}$ $F_{inspection\ path}$

Figure 32 - Taxes inspection path o / d bridge

Inspection path $F_{\text{inspection path}}$ = 0.60 kN (2x per cross beam)

NUON hot water pipe (BG8e)

Two NUON hot water pipes were installed under the western bridge in 2014. These are only applied under the western main bridge, from pillar F to pillar K.

Figure 33 - Location of the hot water pipe on the western main bridge (shown in red) [20140219 1713021-WerkPl-001 Bridge pipe]

The hot water pipe is suspended from the cross beams by means of a steel frame. Per the support is suspended from two crossbars. Leadership supports have a variable center to center distance of approx. 9 to 12 m. A support has been created at pillar G (“Fixed point”) which is only connected to pillar G. Here, the pipe is therefore not suspended from the bridge. Ter at this point, the horizontal loads are transferred in the longitudinal direction of the pipe. This one so do not cross the bridge.

The figures below show the construction structure of the suspension of the pipes.

Figure 34 - Front view of hot water pipe suspension on cross beams [20140219 1713021-WerkPl-001 Bridge pipe]

Figure 35 - Side view of hot water pipe suspension on cross beams [20140219 1713021-WerkPl-001 Bridge pipe]

In Strackee's [912-275.R01] calculation, only variable loads are applied to the hot water pipe given. No permanent loads are given, these are due to the finite element model determined. Appendix E defines the weight of the frame and the tube (Steel tube Ø 450 mm, wall thickness 7.11 mm, PE inner tube Ø 300 mm, insulation with PUR).

Frame weight:	2.1 kN per frame
Tube weight:	0.50 kN / m

These loads are translated to the 4 suspension points of the frame. The cross bars carry 2 frames.

Hot water pipe load	1.86 kN (4x per cross beam)

680 1220 680

Figure 36 - Point loads due to weight of hot water pipe and suspension frame

For the variable load, see section 6.8 .

6.1.7 Shrink and creep (BG9)

After pouring, the concrete will shrink and creep. The shrinkage will result in a tension in the composite steel-concrete cross-sections. The creep results in a decrease in stiffness leading to stretching will suffer.

The effects of creep are taken into account using the modulus ratios n_L for the concrete in accordance with NEN-EN 1994-2 art. 5.4.2.2. Depending on the creep coefficient and type of load the stiffness of the concrete is adjusted per construction phase.

The shrinkage effects are included as an imposed deformation on the deck. For the shrinkage calculation RH = 80% and CEM 32.5 N with cement class S. The concrete pressure stress remains below 45% of $f_{ck}(t_0)$, so that linear creep may be assumed. In the calculation is complete shrinkage load applied in the last construction phase. The occurring shrinkage between the various construction phases have been neglected for the calculation, in accordance with the original design calculations. It is assumed that prestressing has taken place at $t_{0,T}$ = 21 days after pouring concrete, given the required minimum cube compressive strength of 37.5 N / mm² (west bridge) [BR 104] and 40 N / mm² (east bridge) [BR 205] before starting with prestressing.

The following stretches result from shrinkage from the calculation.

Dehydration Shrinkage: □ $_{cd}$ (∞) = 0.178 ‰
Autogenous shrinkage: □ $_{cd}$ (∞) = 0.055 ‰

Total rack shortening □□ = 0.233 ‰

The rack shortening is entered in SCIA as an imposed rack shortening □□ = 0.233 mm / m

6.2 Traffic load

The traffic load is determined, in accordance with the demand specification, for an assessment on actual use (situation AII in accordance with the RBK).

6.2.1 Lane layout

In accordance with the request, the current lane layout will be used for the static calculation of the two bridges (2 lanes), with each existing lane being lane 1 for slow / heavy traffic can be (V1 Normal situation). Given the symmetry of the bridge, lane 1 is only on the west side apprehended. In addition, the situation in an emergency (V2) is considered, where the bridge is fictitious can be classified. Here too, lane 1 is only maintained on the west side.

6.2.2 Lane layout V1 - Normal situation

The lane layout for the normal situation (BM1) is, for the static calculation, taken from the figure 13 of the demand specification. The normal actual lane layout consists of two lanes of 3,625 m (center line to center line) positioned symmetrically to the center. On a redress strip with a width of 0.42 m is present on both sides emergency lanes and as such are regarded as unloaded residual space. The lanes are loaded over a width of up to 3.0 m. BM2 can be recorded anywhere due to a emergency.

In the sections below, we look at what the is the guiding lane layout for both BM1 and BM2.

Lane layout for the main beam

For the main beams, the loads of BM1 are placed in the middle of the lanes, see Figure 37 .

□ Q * 600 kN

□ Q * 400 kN

□ q * 9.0 kN / m²

□ q * 2.5 kN / m²

3.0 m 0.625 m 3.0 m

0.6 m 0.42 m 3,625 m 3,625 m 0.42 m 0.60 m

Lane 2 Lane 1

Figure 37 - Actual lane layout V1 - Normal situation - Main beam load

For the reduction factors, see section 6.2.9. The load from a single axis is BM2 not normative for the main beam.

Lane layout for crossbeam

For the maximum field moment in the cross beam, the UDL load is as much as possible towards it placed in the middle, see Figure 38 . The load on the overhang is for the maximum field moment omitted, in accordance with the comment in NEN-EN 1991-2 art 4.3.2 (4). The position of the wheel loads lanes 1 and 2 are placed 0.5 m apart (local assessment) and using a beam calculation is the most decisive position determined for the field moment. One of the heavy wheels exactly in the middle of the cross beam.

2.0 m
0.35 m 2.0 m 0.5 m □ Q * 600 kN
□ Q * 400 kN
□ q * 9.0 kN / m²
□ q * 2.5 kN / m²
2.85 m 2.85 m
0.6 m 0.42 m 3,625 m 3,625 m 0.42 m 0.60 m

Figure 38 - Actual lane layout V1 - Normal situation - Load crossbeam

For the reduction factors, see section 6.2.9. Starting from a center to center distance of the cross beams of 1.8 m, and the fact that there is no UDL and 2 nd axis is possible concluded that the load from BM2 for the field moment in the cross beam will not be decisive.

Lane layout for console and concrete deck cantilever

For the maximum point of support in the console (main bridge) and in the cantilever part of the concrete deck (bridging) for BM1 are the wheel loads of lane 1 on the edge of the actual lane apprehended.

For BM1, assuming a redress strip of 0.42 m and a wheel width of 0.40 m, that the center of the wheel is 0.58 m from the main beam. Lane 2 loading is not for the console relevant and has therefore been omitted.

□ Q * 600 kN 0.58 m

□ q * 9.0 kN / m²

3.0 m

0.6 m 0.42 m 3,625 m Lane 2 3,625 m Lane 1 0.42 m 0.60 m

Figure 39 - Actual lane layout V1 - Normal situation - Load console / concrete deck cantilever (BM1)

In addition to BM1, BM2 can also be decisive for the consoles and the concrete deck. For BM2 assuming one of the wheels can be right next to the vehicle barrier (emergency). In the verification calculation will first check which load is decisive (BM1 / BM2).

2.0 m 0.3 m

□ Q * 400 kN

Figure 40 - BM2 at the overhang - Normal situation - Load support moment of console / cantilever concrete deck (BM2)

For the reduction factors, see section 6.2.9 .

Lane layout for steel deck with bulbs

There are two locations for the maximum field and support moments in the steel deck and the bulbs considered:

- in the field between the cross bars
- in the field between the consoles

Lane layout for steel deck with bulbs between the cross bars

The UDL load in the transverse direction is as much as possible for the steel deck between the cross beams positioned in the center of the cross beam, taking into account the actual lane layout, see Figure 39. The UDL load is shown in separate load cases on the even and odd fields placed from the local model.

2.0 m

2.0 m 0.5 m □ Q * 600 kN

□ Q * 400 kN

□ q * 9.0 kN / m²

□ q * 2.5 kN / m²

3.0 m 3.0 m

0.6 m 0.42 m 3,625 m 3,625 m

0.42 m 0.60 m

Figure 41 - Actual lane layout V1 - Normal situation - Load of steel deck with bulbs between cross bars (BM1)

The position of the wheel loads of lanes 1 and 2 are placed transversely 0.5 m apart (local assessment) and using SCIA the most decisive position was determined for the maximum voltage in the field and at the point of support (cross beam). For this purpose, the axle load is in steps of 0.1 m shifted until the maximum voltage is found (see Appendix F.3). Only the normative positions will be used in the model. This is the same for the maximum tensile stress in the field with center tandem system at 0.5 m from the crossbeam. For the maximum compressive stress at the location of the the support point corresponds to the center tandem system at 0.1 m from the cross beam.

0.5 m 0.1 m

Figure 42 - Actual lane layout V1 - Normal situation - Load of steel deck with bulbs between cross beams (BM1) position longitudinal direction

Variation of the position of the tandem system in the transverse direction hardly influences the maximum tension and is therefore taken as the same as when testing the cross beams.

BM2 can be decisive for the steel deck and the bulbs between the cross bars. In transverse direction is the load with one of the wheels therefore placed on the middle bulb.

2.0 m

□ $_Q$ * 400 kN

Figure 43 - BM2 - Normal situation V1 - Load steel deck with bulbs between cross beams (BM2)

In the longitudinal direction, the decisive positions for the field and point of support determined by moving the load in steps of 0.1 m, see appendix F.3. In front of the field moment gives an axle load at 0.8 m from the cross beam the maximum tension. For the maximum tension at the point of support, the axle load is 1.1 m from the cross beam.

0.8 m 1.1 m

Figure 44 - BM2 - Normal situation V1 - Load steel deck with bulbs between cross beams (BM2) longitudinal position

The calculation shows that BM2 does not lead to higher voltages than BM1 (see appendix F.3), so this further is not included for further calculations.

For the reduction factors, see section 6.2.9 .

Lane layout for steel deck with bulbs between consoles

For the maximum stresses in the steel deck with bulbs, a load close to the edge of the steel deck. Here there are different bulbs / strips under the deck and there is also one edge strip present.

For BM1, assuming a redress strip of 0.42 m and a wheel width of 0.40 m, that the center of the wheel is 0.58 m from the main beam. Lane 2 is as much as possible towards the edge of the lane 2 moved.

□ Q * 600 kN 0.58 m

2.0 m 0.20 m

□ q * 9.0 kN / m²

□ q * 2.5 kN / m²

3.0 m 3.0 m

0.6 m 0.42 m 3,625 m Lane 2 3,625 m Lane 1 0.42 m 0.60 m

Figure 45 - Actual lane layout V1 - Normal situation - Load of steel deck with bulbs between consoles (BM1)

The design positions of the tandem systems for the maximum stresses in the field and on site of the support point was determined using SCIA by sliding it in steps of 0.1 m (see Appendix F.3). Only the normative positions are saved in the model due to the calculation time. For the maximum tensile stress in the field, the center of the tandem system stands at 0.4 m from the crossbeam the maximum compressive stress at the point of support is set at 0.1 m from the tandem heart crossbeam.

0.4 m 0.1 m

Figure 46 - Actual lane layout V1 - Normal situation - Load of steel deck with bulbs between consoles (BM1) position long direction

In addition to BM1, BM2 can also be decisive for the bulbs between the consoles. For BM2 assuming one of the wheels can be directly next to the vehicle barrier (accident).

2.0 m 0.3 m

$\square_{Q}$ * 400 kN

Figure 47 - BM2 - Normal situation V1 - Load steel deck with bulbs between the consoles (BM2)

Also for BM2 the normative positions for the tension in the field and at the point of support are of the bulbs determined by moving the tandem system in steps of 0.1 m (see appendix F.3). In front of the maximum tensions in the field, the tandem system stands at 0.8 m, for the maximum tensions location of the support point at 1.1 m. Only the normative positions will be in the model applied because of the calculation time.

0.8 m

1.1 m

Figure 48 - BM2 - Normal situation V1 - Load of steel deck with bulbs between the consoles (BM2) longitudinal position

BM2 appears to give slightly greater tensions than BM1. Both are included in the calculation model, because BM1 must also be combined with other loads.

For the reduction factors, see section 6.2.9 .

Lane layout for concrete deck

Moments and normal forces in the longitudinal direction of the bridge are decisive for the concrete deck. The design forces are found by maximizing the concrete deck between the cross beams tax. The lanes are therefore positioned as far as possible to the center; see Figure 50 .

BM2 does not appear to be the norm for the concrete deck above BM1. Because the axle loads of BM1 on lanes 1 and 2 are close to each other, the spread wheel prints overlap and deliver this higher taxes on the wheel prints of BM2.

2.0 m

2.0 m 0.5 m □ Q * 600 kN

□ Q * 400 kN

□ q * 9.0 kN / m²

□ q * 2.5 kN / m²

3.0 m 3.0 m

0.6 m 0.42 m 3,625 m 3,625 m 0.42 m 0.60 m

Figure 49 - Actual lane layout V1 - Normal situation - Concrete deck load (BM1)

The position of the wheel loads of lanes 1 and 2 have been determined at various normative positions. The positions are determined on the basis of the normative forces for the assessment.

The maximum normal force in the concrete occurs due to global effects. For this are the load models placed in the global field between the supports. An analysis shows the concrete deck in the characteristic combination remains almost completely under pressure. Only locally above the main beams on site there is some draft of the intermediate supports.

For the maximum point of support, above the crossbars and near the main beam, the load systems placed above the cross beams. The moments in the deck appear to be relatively low due to the limited center to center distance from the cross beams and the cooperation of the steel with the concrete.

The maximum field moments occur due to local effects. The load models are available at this location the global fields and supports placed between two crossbars (local field). Analysis shows that these moments are also relatively low. Shifting the axle loads has no noteworthy influence on the moments, which assumes a tandem system in the middle of the field.

Figure 50 - Actual lane layout V1 - Normal situation - Concrete deck load (BM1) longitudinal position

6.2.3 Lane Layout V2 - Emergency

For the emergency, the theoretical lane layout according to NEN-EN should be used
1991-2, but with a reduced reference period of 6 months. The width between the
vehicle barriers are equal to the bridges for the main bridge, namely 8.09 m. The width of the
driving distance may be measured from 0.3 m from the inside of the vehicle barrier, in accordance with RBK art 1.8.
The bridge deck between the vehicle barrier is divided into 2 lanes with a width of $w_l = 3.0$.
In accordance with the normal situation, the redress strip becomes an unloaded residual space
apprehended. The lane layout with associated load systems is selected in such a way that the
construction is loaded as unfavorably as possible for the element under consideration. For the reduction factors
please refer to section 6.2.9 .

Lane layout for the main beam
For the main beams, the loads are positioned so that they are on one side as much as possible
the carriageway girder, however 0.3 m from the vehicle barrier.

□ Q * 600 kN

□ Q * 400 kN

□ q * 9 kN / m²

□ q * 2.5 kN / m²

0.3 m 1.49 m 3 m 3 m 0.3 m

Figure 51 - Fictitious lane layout V2 - Emergency - Main girder load

Lane layout for crossbeam
The position of the UDL load and the axle loads for the bending field moment in the cross beam is
estimated on the basis of a beam calculation and will be validated in the final model. In front of
In this situation the UDL load on the overhang works favorably and has been omitted. The wheel loads
stand for this situation at 0.5 m from each other.

0.35 m 2.0 m 0.5 m 2.0 m

□ Q * 400 kN □ Q * 600 kN

□ q * 9 kN / m²

□ q * 2.5 kN / m²

1.195 m 1.5 m 3 m 1.2 m 1.195 m

Figure 52 - Actual lane layout V2 - Emergency - Load crossbar

Lane layout for console and concrete deck cantilever

For the maximum point of support in the console (main bridge) and in the cantilever part of the concrete deck (bridging), the wheel loads of BM1 are maintained on the edge of the fictional lane, with the edge of the wheel 0.3 m from the vehicle barrier. Assuming a distance from the vehicle barrier of 0.30 m and a wheel width of 0.40 m, the center of the wheel is 0.695 m from the main beam. The lane 2 load is not relevant to the cantilever and has been omitted.

0.695 m

$\square_Q$ * 600 kN

$\square_q$ * 9 kN / m²

0.3 m 1.49 m 3 m 3 m 0.3 m

Figure 53 - Fictional lane layout V2 - Emergency - Load console / concrete deck cantilever (BM1)

For BM2 the wheel loads are placed against the vehicle barrier. Since the load case BM2 - emergency is exactly equal to BM2 - normal situation, it will not be charged further, since the reduction factors for the emergency situation are lower.

2.0 m 0.3 m

$\square_Q$ * 400 kN

Figure 54 - BM2 at the overhang V2 - Emergency - Console load / cantilever concrete deck (BM2)

Lane layout for steel deck and bulbs

There are two locations for the maximum field and support moments in the steel deck and the bulbs considered:

- in the field between the cross bars
- in the field between the consoles

Lane layout for steel deck with bulbs between the cross bars

The load from BM1 and BM2 in the emergency situation (V2) will affect the steel deck and the bulbs between the crossbars are not normative with respect to the normal situation (V1), due to the higher ones reduction factor for V2 due to the reduced service life that may be used. Because the axes are equidistant in both situations, V1 will always be decisive.

Lane arrangement for bulbs between the consoles

For the maximum voltages in the bulbs, the situation in case of an emergency (V2) is possible are normative above the normal situation (V1), because the load is more to the outside of the deck and can therefore put more stress on the edge strips. The load in the situation V2 emergency is therefore placed on the edge of the deck between the consoles.

Widthwise, the loads are maintained as described under the load on the console, however now also with lane 2. Here the wheel loads of BM1 are on the edge of the fictional lane with the edge of the wheel 0.3 m from the vehicle barrier. Assuming a distance to the vehicle barrier of 0.30 m and a wheel width of 0.40 m, the center of the wheel is 0.695 m from the main girder.

0.695 m
□ Q * 600 kN
0.2 m
□ Q * 600 kN
□ q * 9 kN / m²
□ q * 2.5 kN / m²
0.3 m 1.49 m 3 m 3 m 0.3 m

Figure 55 - Fictional lane layout V2 - Emergency - Steel deck load with bulbs between consoles (BM1)

The design positions of the tandem systems for the maximum stresses in the field and on site of the support point was determined using SCIA by sliding it in steps of 0.1 m (see Appendix F.3). Only the normative positions are saved in the model due to the calculation time. For the tensions in the field, the center of the tandem system at 0.4 m from the cross beam is decisive, for the compressive stress at the point of support is the tandem center at 0.1 m from the cross beam normative.

0.4 m

0.1 m

Figure 56 - Fictional lane layout V2 - Emergency - Steel deck load with bulbs between consoles (BM1) long direction position

For the reduction factors, see section 6.2.9.

BM2, just like the console, is never decisive in an emergency (V2) is above the normal situation (V1), since the position is equal, but the reduction factors in the normal situation are lower.

Lane layout for concrete deck
For the concrete deck, the situation in an emergency (V2) is not decisive above the normal situation (V1). In the V2 situation, the loads are lower because of the higher reduction factor, but the wheel loads are the same distance apart.

Traffic loads in situation V2 have therefore not been included for the concrete deck.

6.2.4 Vertical traffic load - Tax model 1

The analysis of the bridge is based on a bridge without load limitation. Between the vehicle barriers are based on the traffic load in accordance with NEN-EN 1991-2. In the first instance the loads without reductions are described.

Basic traffic load values
In each lane i you have to count on a tandem system with 2 axles · $_{Qi}$ · Q_{ik} and an even distributed load □ $_{qi}$ · q_i . In accordance with table 4.2 of NEN-EN 1991-2, the following characteristic must be met value of the tax.

Position	Tandem system (TS)	Evenly distributed tax (CFP)
	Axle load Q_i [kN]	q_i (or q_{rk}) [kN / m²]
Lane 1	300	9.0
Lane 2	200	2.5

Table 18 - LM 1: characteristic values

For □ $_{Qi}$ and □ $_{qi}$, see Table 20 .

Load distribution main bridge
Assuming the thickness of the steel deck of 10 mm + 70 mm asphalt, the main bridge will be a spread of the wheel loads to the center of the structural cover plate see NEN-EN 1991-2 art 4.3.6 (3). This means a wheel area of 0.55x0.55 meters (instead of 0.4x0.4 meters).

Increase tax spread
Based on the minimum thickness of the concrete deck of 200 mm + 70 mm asphalt, for the

bridging a spread maintained from the wheel loads to the heart of the concrete structure, see NEN-
EN 1991-2 art 4.3.6 (2). This means a wheel area of 0.74x0.74 meters (instead of 0.4x0.4 meters).

6.2.5 Vertical traffic load - Tax model 2

In accordance with article 4.3.3 of NEN-EN 1991-2, the following characteristic value of the load must be applied to be arrested.

$\square_{Q} * Q_{ak} = \square_{Q1} \cdot Q_{ak}$

at which:
Q_{ak} = 400 kN (axle load)
1_{Q1} see Table 20

LM2 may be reduced by the client under certain conditions. In the
demand specification this is not specified and is therefore not applied. Where it becomes relevant
only one wheel charged.

In the vicinity of expansion joints, an additional magnification factor for dynamic effects,
equal to that charged for fatigue, see section 6.10.6.

Load distribution main bridge
Assuming the thickness of the steel deck of 10 mm + 70 mm asphalt, the main bridge will be a
spread of the wheel loads to the center of the structural cover plate see NEN-EN 1991-
2 art 4.3.6 (3). This means a wheel area of 0.50x0.75 meters (instead of 0.35x0.60 meters).

Increase tax spread
Based on the minimum thickness of the concrete deck of 200 mm + 70 mm asphalt, for the
bridging a spread maintained from the wheel loads to the heart of the concrete structure, see NEN-
EN 1991-2 art 4.3.6 (2). This means a wheel area of 0.69x0.94 meters (instead of 0.35x0.60 meters).

6.2.6 Vertical traffic load - Tax model 3

No special vehicles have been established for the bridge by the client.

6.2.7 Vertical traffic load - Tax model 4

A crowd of people on the bridge is not relevant to the IJssel Bridge and has therefore not been taken into account.

6.2.8 Influence length

The traffic load may be reduced depending on the influence length of the part to be tested turn into. In Appendix L, the influence line of moment $M_{y\ is}$ of a number of normative points displayed. The influence lengths below are used in the calculation.

Part	Main bridge	Bridge
Main spar field 1	45 m	40 m
Main girder field 2 (positive moment)	50 m	40 m
Main girder field 2 (negative moment)	155 m	80 m
Main spar field 3	105 m	
Main beam support point 2	95 m	80 m
Main beam support point 3	155 m	
Cross beam / console	12 m	15 m
Bulb, concrete deck, steel deck	<2.5 m	<2.5 m

Table 19 - Influence length per part

6.2.9 Correction factors

The correction factors on the traffic taxes are taken into account in the taxes.

Correction factor for less than normal load from freight traffic
Depending on the number of trucks per year, a correction factor may apply in accordance with NEN-EN 1991-2 less than normal loads from freight traffic are applied. According to information from Rijkswaterstaat may assume 1510000 trucks per lane in 2050. The reduction factor may be applied to all lanes.

Number of trucks per year per lane for heavy traffic N_{obs} [a]	□ $_{Q1}$ and □ $_{q1}$					
	Length of span or influence length (L)					
	20 m	30 m	50 m	100 m	≥200 m	□ $_{qr}$
2000000	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**
1510000	**0.99**	**0.99**	**0.99**	**0.99**	**0.99**	**0.97**
200000	0.97	0.97	0.97	0.95	0.95	0.9
20000	0.95	0.95	0.94	0.89	0.88	0.8
2000	0.91	0.91	0.91	0.82	0.81	0.7
200	0.88	0.88	0.87	0.85	0.74	0.6

[a] *Intermediate values may be interpolated.*

Table 20 - Correction factors *□ $_{Q1}$, □ $_{q1}$ and □ $_{qr}$*

The correction factor is equal to 0.99 for all parts of both bridges (0 m ≤ L ≤ 155 m).

Trend reduction

NEN 8701 (Table 2) gives a trend reduction in the load size for LM1 and LM2 if assumed of a shorter (residual) lifespan. When a (residual) lifespan of 30 years is assumed from 1 January 2021 (demand specification requirement), a trend reduction is found as shown in the table below in the column at 2051.

The value for $\square_{trend}$ depends on the influence length of the part to be considered. For LM2 the trend reduction, independent of the influence length, equal to $\square_{trend}$ = 0.98.

Influential length L [m]	Reduction factor $\square_{trend}$						
	2010	2020	2030	2040	2050	2051	2060
0	1.00	1.00	1.00	1.00	1.00	**1.00**	1.00
20	0.89	0.91	0.93	0.96	0.98	**0.98**	1.00
30	0.87	0.90	0.92	0.95	0.97	**0.97**	1.00
50	0.82	0.86	0.89	0.93	0.96	**0.96**	1.00
75	0.78	0.83	0.87	0.91	0.96	**0.96**	1.00
100	0.76	0.81	0.85	0.90	0.95	**0.96**	1.00
150	0.75	0.80	0.85	0.90	0.95	**0.96**	1.00
≥200	0.75	0.80	0.85	0.90	0.95	**0.96**	1.00

[a] Linear interpolation is allowed for other periods and influence lengths. Before taxes through the single axis or the single wheel (of LM2), regardless of the influence length, the value for L = Keep 20 m. Two pendulum axes placed in line with each other count as one as.

Table 21 - Trend reduction factor

The table below gives the reduction factor $\square_{trend}$ for LM1 per component .

Part	Main bridge		Bridge	
	L	$\square_{trend}$	L	$\square_{trend}$
Main spar field	45 m - 105 m	0.96	40 m	0.97
Main girder intermediate support / field 2 (negative moment)	95 m - 155 m	0.96	80 m	0.96
Cross beam / console	12 m / 15 m	0.99	12 m	0.99
Bulb, concrete deck, steel deck	<2.5 m	1.00	<2.5 m	1.00

Table 22 - Correction factors $\square_{trend}$ *per component*

Reduction at large span

In addition, a reduction factor $\square_{L}$ applies to bridges with a large span , in accordance with NEN 8701 paragraph 5.1.4. For construction parts with an influence length L greater than or equal to 100 meters, correction factors $\square_{qi}$ and $\square_{qr}$ of NEN-EN 1991-2 are multiplied by the reduction factor $\square_{L}$.

$\square_{L}$ = 1.2 - 0.002 * L for 100 m ≤ L <200 m

$\square_{L}$ = 0.8 for L ≥ 200 m

Only at the location of support point 3 and field 2 (negative moment) and 3 of the main bridge is the Influence length greater than 100 m. The following reduction factors apply.

Part	Main bridge	
	L	□ trend
Main girder field 2 (negative moment)	150 m	0.90
Main spar field 3	105 m	0.99
Main beam support point 2	155 m	0.89

Table 23 - Correction factors □ *L per part*

Traffic tax reduction factor due to reference period

In the load combinations, a reduction on the traffic tax may be applied if the reference period is less than 100 years. The calculations assume “normal use” (V1) a reference period of 30 years, for the situation in an “emergency” (V2), 6 is assumed months (1 month per 5 years residual life).

Depending on the influence length (L), according to table 1 of NEN 8701, the □ factor for one reference period of 30 years (V1 normal use) or 6 months (V2 emergency) is used.

	□ factor a			
Reference period	Length of span or influence length L			
	20 m	50 m	100 m	≥ 200 m
100 years	1.00	1.00	1.00	1.00
50 years	0.99	0.99	0.99	0.99
30 years	**0.99**	**0.99**	**0.98**	**0.97**
15 years	0.98	0.98	0.96	0.96
1 year	0.95 b	0.94 b	0.89	0.88
6 months	**0.93 b**	**0.92 b**	**0.84**	**0.84**
1 month	0.91 b	0.91 b	0.81	0.81

a For other influence lengths and reference period, it may be interpolated linearly.
b see note under table 1 in NEN 8701

Table 24 - Reduction factor for the reference period for renovation in accordance with NEN 8701

Project related

The table below gives the □-factor per part for each part.

Part	Main bridge			Bridge		
	L	□ factor (30 years)	□ factor (6 months)		□ factor (30 years)	□ factor (6 months)
Main spar field 1	45 m	0.99	0.92	40 m	0.99	0.92
Main girder field 2 (pos. M)	50 m	0.99	0.92	40 m	0.99	0.92
Main girder field 2 (neg. M)	150 m	0.98	0.84	80 m	0.98	0.87
Main spar field 3	105 m	0.98	0.84			
Main beam support point 2	95 m	0.98	0.85	80 m	0.98	0.87
Main beam support point 3	155 m	0.97	0.84			
Cross beam / console	12 m	0.99	0.93	15 m	0.99	0.93
Bulb, concrete deck, steel deck	<2.5 m	0.99	0.93	<2.5 m	0.99	0.93

Table 25 - Correction factors □ *factor per component*

At the end of Appendix E, a complete overview of the various reduction factors is included
will be applied to traffic taxes. In the first instance, the
reduction factors for the (positive) field moments. Should it turn out that one of the parts with a
longer influence length (supports, negative field moments) will not suffice, then the extra reduction will occur
will still be charged.

6.2.10 Horizontal traffic loads

Braking and acceleration power
The braking and acceleration forces have been determined in accordance with article 4.4.1 of NEN-EN 1991-2.

$Q_{lk} = 0.6 \cdot 1_{Q1} (2 \cdot Q_{1k}) + 0.1 \cdot \square_{q1} \cdot q_{1k} \cdot w_1 \cdot L$
$180\square_{Q1}$ (kN) $\leq Q_{lk} \leq 800$ kN

In which:
L is the length of the superstructure or the part considered
w_1 is the width of the (theoretical) lane 1

Assuming a width of lane 1 $w_1 = 3.0$ m and the values for the traffic load $Q_{1k} = 300$ kN and $q_{1k} = 9.0$ kN / m this amounts to:

$Q_{lk} = \square_{Q1}$ 360 kN + 2.7 kN / m $\square_{q1}$ L ≤ 800 kN

This gives

Part	L	Q_{lk}
Main bridge	295 m	$\square_{Q1} \cdot 360 + 2.7 \cdot \square_{q1} \cdot 295 \leq 800$ kN
Bridge	120 m	$\square_{Q1} \cdot 360 + 2.7 \cdot \square_{q1} \cdot 120 \leq 800$ kN

Table 26 - Braking and acceleration force

The braking force is charged as a line load, running along the longitudinal axis of any one
lane can engage. For this, lanes 1 or 2 are maintained as in the previous section
displayed, with the brake load on 1 lane at the same time. A distinction is hereby made
made in the normal situation and the situation in an emergency.

The horizontal traffic loads are reduced by the same reduction factors as for the vertical traffic loads.

Centrifugal and other transverse forces

No centrifugal forces apply for the IJssel Bridge. The effects of transverse direction that occur due to braking or skewing in an oblique direction for calculation neglected. The magnitude of the load (25% of the brake load) is considerably lower than the wind load, see Appendix G.

6.2.11 Vertical loads on inspection paths

Inspection path next to the bridge deck

For the inspection paths next to the bridge deck, according to NEN-EN 1991-2 art. 4.1 that the effects of taxes for conducting inspections where relevant should be separately identified.
However, the question specification does not make any statement about this. This applies in accordance with NEN-EN 1991-2 art 5.2.3 (1) for areas of the deck of road traffic bridges that are separated by railings and that are not be part of the road, a pedestrian tax must be charged in accordance with Article 5.3, with the exception of the official vehicle. However, the inspection path does not apply to the IJssel Bridge separated by railings. In addition, the inspection path is in principle not accessible to audience.

The inspection paths (grid floor) next to the bridge deck were originally designed at 2.0 kN / m² [BBV-0010-00]. In consultation with Rijkswaterstaat, it has been decided to use the design load of 2.0 kN / m² or a point load of 7.0 kN in accordance with NEN-EN 1991-2 art 5.3.2.2 .. For the global calculation combined with traffic, no tax is charged on it inspection path.

0.8 kN / m 7 kN
0.8 kN / m
2.0 kN / m²

700 mm
$F_{inspection\ path}$ $F_{inspection\ path}$

Figure 57 - Loads inspection path next to the bridge (left) and under the bridge (right)

A line load must also be taken into account for the railings along the inspection path of 0.8 kN / m, which can be horizontal or vertical once. It will not work for the console are normative in relation to the point load of 7 kN.

Inspection path under the bridge deck
No design load is known for the inspection path under the bridge deck. In consultation with Rijkswaterstaat it has been agreed to assume an inspection path in the interior of bridges in accordance with NEN-EN 1991-2 art 5.2.3 (2). This is subject to a load of 2 kN / m² or a concentrated load of 3 kN. This one tax applies only to local assessments, the global calculation is combined with traffic no tax charged on the inspection path.

6.2.12 Horizontal loads on inspection paths

In accordance with Article 5.4 (1), the horizontal load Q $_{flk}$ only has to be charged for pedestrian bridges are being brought. So no horizontal load on the inspection paths has been charged.

6.3 Extraordinary taxes

6.3.1 General

In accordance with NEN 8701 and the RBK (art. 4.4.1 (1)), it is not necessary to assess whether a construction should be rejected or when assessed at the level of use, only to take into account the extraordinary taxes originally charged and the NEN 8701 in 4.4.2 (1) given exceptional load by road vehicles on the supporting substructure.

Extraordinary loads have not been taken into account during the design, so in principle they are no need to be charged.

However, client has requested to consider the following extraordinary taxes [demand specification]:

- Collision forces on vehicle barriers (section 6.3.2);
- Impact loads from river and canal traffic on the deck (section 6.3.3).

The other extraordinary taxes must be described and considered qualitatively, including a global risk assessment in relation to the costs of removing that risk. This concerns the following taxes.

- Impact loads from river and canal traffic on the substructure (section 6.3.4);
- Impact load on the supporting substructure by road vehicles (section 6.3.5);
- Shock load on *the superstructure* by road vehicles (section 6.3.6);
- Traffic accident on the bridge deck (section 6.3.7) .

Extraordinary taxes are considered in the extraordinary tax combination.

6.3.2 Collision forces on vehicle barriers

The horizontal force transmitted by the vehicle barrier is based on NEN-EN 1991-2 article 4.7.3.3 . Table 27 shows the horizontal forces for different classes of vehicle barriers.

Class	Horizontal force [kN]
a	100
B	200
C	400
D	600

Table 27 - Classes for the horizontal force transmitted by the vehicle barrier

A steel H2 vehicle barrier is present on the bridge decks. This can be done in accordance with NEN-EN 1991-2 a class A vehicle barrier is assumed.

The horizontal force is distributed over a length of 0.5 m and engages at a height of 100 mm below the top of the completed vehicle barrier, or 1.0 m above the roadway, the lowest of both options are adhered to. This amounts to 0.7 m for both bridges.

Figure 58 - Vehicle barrier at the bridges (left) and the main bridge (right)

The vertical force which simultaneously adversely affects the horizontal impact force must be equal taken at $0.5 \square_{Q1} Q_{1k}$.

The support construction should be 1.75 times the characteristic local resistance of the crash barrier can resist. A style IPE 100 with a resistance of 9.3 kNm has been used. In accordance with the ROK at least 24 kNm must be used for the characteristic local resistance per style. In the first instance will be calculated with the ROK requirement of 24 kNm. If this cannot be met will be reverted to the legally obtained level and will be calculated with 9.3 kNm.

6.3.3 Shock load from river and canal traffic on the deck

The deck construction of the main bridge can be approached at the river span.

Acceptance tax for steel construction deck
The acceptance load on the deck is determined in accordance with NEN-EN 1991-1-7 article 4.6.2 (4). The value for the equivalent static force is 1 MN. For the dimensions of the engagement surface (axb) of the tax must be retained:

- a = 0.25 m
- b = 3.0 m

6.3.4 Impact load by river and canal traffic on the substructure

The substructure of the main bridge (river pillars) can in principle be approached by shipping traffic. The acceptance load on the substructure (pillars H and J) is determined in accordance with the replacement text of the ROK for NEN-EN 1991-1-7, article 4.6.2 (1) and articles 4.6.1 (2) and (3) of the NEN-EN 1991-1-7. The tax will be considered qualitative.

The acceptance tax is based on a CEMT class IV (Class Europe) in accordance with the document “Waterways in the Netherlands” of Rijkswaterstaat (dated October 2017), which is applicable for the Geldersche IJssel.

Frontal collision
The maximum impact force is determined in accordance with the ROK. The impact load should be applied to a height of 1.50 meters above the maximum navigable water level.

The sailing speed required for the calculation was initially estimated for the consideration of approx. 1.5 m / s. In any calculation, this will have to be reported by Rijkswaterstaat, as well the maximum navigable water level.

$F_{dx} = 3.3$ □E + 5.6 [MN]

$E = 0.55 mv_t{}^2 = 0.55 * 1500$ tons $* 6,8^2 = 38.1$ MNm
m = 1500 tons (in accordance with ROK table 5-3)
$v_t = v_v + v_w = 5.3 + 1.5 = 6.8$ m / sec
v_v = sailing speed (in accordance with ROK table 5-4)
= 5.3 m / sec
v_s = flow rate (estimate)
= 1.5 m / sec

$F_{dx} = 3.3$,138.1 + 5.6 [MN] = 11.8 MN

Impact at an angle
In addition to a collision at the head of the pillar, an impact load at an angle is also possible. At one impact at an angle, the impact force must be decomposed into a component perpendicular to the construction and a component parallel to the construction. To determine the angle of entry, is in the figure below (fictional) shows a CEMT IV class ship sailing into the pier. Enter perpendicular on the high speed pillar is not realistic. A real angle of entry on the pillar has been estimated as between the 0 ° and 45 °.

0 °

45 °
25 °

Figure 59 - Angle of approach on the pillar

It follows, as an example for an angle of 45 °, an impact load of:

F_{dy} = □ * F_{dx} * cos (□)
= 0.78 * 11.8 MN * cos (45 °) = 6.5 MN (Perpendicular to the pillar)

F_R = F_{dx} * sin (□)
= 11.8 MN * sin (25 °) = 8.3 MN (friction parallel to the pillar)

The impact area (wxh) must be assumed
$b = b_{pier}$ and h = 0.5 m in a frontal collision and
b = 0.5 m and h = 1.0 m with a side collision.

in which:
b_{pier} is the width of the obstacle in the waterway.

Project related

Consideration of impact load on the substructure by river and canal traffic
It is not known whether and if so on which impact loads the pillars in the design have been calculated. For the frontal impact loads from river and canal traffic are expected to be able to withstand the pillar given its length in the longitudinal direction.

For a collision at an angle, the load perpendicular to the pillar will be partly absorbed by the ground behind the pillar, since behind the pillars there is at least 11.0 m + NAP ground. This means that the full load will not have to be paid via the pillar.

Figure 60 - Section across the river pillars

However, it cannot be said in advance whether the construction will comply with this. To be sure about this the substructure should be calculated.

If it would follow from such a calculation that the construction should be reinforced, then economically speaking, an inhibition work is most obvious. However, it is expected that this is not desirable for passage. An alternative would be to widen the pillar, but this will go to all probability lead to disproportionate costs.

It is recommended to maintain the current situation on the basis of the legally obtained level.

6.3.5 Impact load on the supporting substructure by road vehicles

In accordance with the RBK serves for the exceptional load by road vehicles on the supporting vehicle substructure to take into account at least 75% of the design value according to 4.3.1 of NEN-EN 1991-1-7 are being brought. If this cannot be met, the construction must be reinforced to 100%.

Traffic category		F_{dx} [a] [kN]	F_{dy} [a] [kN]	d_b [m]
Motorways, state roads and major roads		2000	1000	20
National highways in rural areas		1500	750	15
Roads in urban areas		1000	500	10
Courtyards and parking garages with access for:	cars	100	50	4
	trucks (> 3.5 tons)	200	100	5

[a] x = direction parallel to the road axis, y = perpendicular to the road axis

Table 28 - Design values of equivalent static forces due to impact loads from vehicles against elements that support structures across or adjacent to roads

According to the ROK, bridges of Rijkswaterstaat should be classified as “Motorways, provincial and main roads ”. Given the nature of the road, this seems to be a very demanding one existing construction. It is proposed to use “roads in urban areas” for this away, which should be based on F_{dx} = 1000 kN and F_{dy} = 500 kN. This is still one conservative assumption.

In principle, a collision can only take place at the abutments. Given the nature of the construction (solid concrete abutment) the abutment will be amply able to absorb the impact load and locally damaged at most, see figure below.

Figure 61 - Roads under the bridge at the northern abutment (left) and the southern abutment (right)

No further analysis of the impact load on the supporting substructure is recommended road vehicles, given the abutment's generous structural capacity and small size risk of collision.

6.3.6 Impact load on the superstructure by road vehicles

The impact load on the superstructure by road vehicles is in principle only possible at the location of the underpasses take place right next to the abutments. According to Article 4.3.2 of NEN-EN 1991-1-7 the following taxes are to be withheld.

Traffic category		F_{dx} [a] [kN]	$F_{a,□}$ [kN]
Motorways, state roads and major roads		2000	600
National highways in rural areas		1500	450
Roads in urban areas		1000	450
Courtyards and parking garages with access for:	cars	100	
	trucks (> 3.5 tons)	200	

[a] x = direction parallel to the road axis,

Table 29 - Design values of equivalent static forces as a result of an impact load on the superstructure

Here too (conservatively), “Roads in urban areas” is assumed and therefore a tax applies of F_{dx} = 1000 kN and $F_{a,□}$ = 450 kN. The above load may be reduced depending on the height, according to the figure below, where at h_1 = 7 m the load has decreased to zero.

Figure 62 - Reduction factor for vehicle impact loads on horizontal structural members above motorways, depending on the clearance height h

According to drawing [C.4348], the ground level at the northern abutment is at approximately 10.1 + NAP located. The bottom support is located at 17.2 m + NAP [A.22656], so that the clearance height is approx

7.1 m.

According to drawing [C4349], the ground level at the southern abutment is at approximately 13.70+ NAP located. The bottom of the beam is located at approx. 21.5 m + NAP [A.92198], so that the clearance height is approx. 7.8 m.

approx.7.1 m

approx.7.8 m

Figure 63 - Drive-through height h at the under-roads

In view of the clearance height at the location of the underpasses, the impact load on the superstructure are zero. It is therefore not necessary to apply an impact load to the superstructure by road vehicles to charge.

6.3.7 Traffic accident on the bridge deck

The ROK 1.4 provides an additional requirement that an option of one must also be counted on traffic accident on the bridge deck as an extraordinary load. The outer wheels of the heaviest tandem system ($2Q_{1k}$) to be placed on the edge of the bridge deck, regardless of the presence of a guide structure. The traffic load on the rest of the bridge may be equal to the representative value of the traffic load are taken less the tandem system that is on the edge.

This tax case will mainly affect locally the consoles and the concrete deck. Starting from a comparison with Load Model 2 (2x200 kN) the following comparison can be made for it moment on the cantilever.

Accident:	Axis 1: $M_{Ed} = 1.00 * 150 \text{ kN} * (1.795 + 0.40) =$	329 kNm
	Axis 2: $M_{Ed} = 1.00 * 150 \text{ kN} * (1.795-0.30-0.50) * 0.60 / 1.80 =$	50 kNm
	$M_{Ed} = 1.00 * 9 \text{ kN/m}^2 * 1.80 * (1.795-0.30) * (1.795-0.30) / 2 = 18 \text{ kNm} +$	
	Total =	397 kNm
BM2:	$M_{Ed} = 1.25 * 200 \text{ kN} * (1.795-0.30) =$	374 kNm

0.2 m 2.0 m

300 kN

2.0 m 0.3 m

400 kN

Figure 64 - Traffic accident on the bridge deck (red) and BM2 at the crossing (green)

The moment in the extraordinary situation is slightly higher. However, this has not yet been taken into account with the lower load factor for the permanent loads in the extraordinary load combination. As a result, this tax case in the combination will ultimately not be decisive for the combination with BM2. This tax case is therefore not considered any further.

6.4 Wind load

6.4.1 General

The wind load is described in NEN-EN 1991-1-4 chapters 4 and 8. Wind loads result in forces in the x, y, and z directions.

- x direction: in width direction of the bridge deck, perpendicular to the span;
- y direction: in span direction;
- z direction: the direction perpendicular to the deck.

In accordance with ROK section 5.4, article 4.3.2 (2), the terrain category must be assumed terrain category II - unbuilt. The IJssel Bridge is located in wind area III. There is no bill taken with the concrete bridge, leaving the eastern steel bridge in the lee of the two outer bridges.

The following basic values apply to both bridges. The basic wind speed v_b is equal to the fundamental value of the basic wind speed $v_{b;0}$ because the values c_{dir} and c_{season} in NEN-EN 1991-1-4 art. 4.2 are set to 1.0. However, a reduced residual life of 30 has been taken into account year, which may be assumed for 0.96 for c_{prob}, see section 6.4.4.

$v_{b,0}$ = 24.5 m / s
v_b = 23.6 m / s (c_{dir} = 1.0, c_{season} = 1.0, c_{prob} = 0.96 based on 30 years)
$v_{b,0}$ * = 23.0 m / s

In addition, the following factors have been applied, see also Appendix G.
terrain factor k_r = 0.209 (Terrain category II, unbuilt)
roughness factor = 0.870-0.927 (depending on reference height z_e).
orography factor c_o (z) = 1.0
turbulence factor k_l = 1.0
construction factor $c_s c_d$ = 1.0 *

The extreme thrust q_p (z) therefore varies between 0.71 kN / m² and 0.77 kN / m² (depending on roughness factor, see appendix G).

The force coefficients depend on the height of the main beams and vary between 2.00 and 2.29 for wind without traffic, including surcharge for the cross slope. For wind with traffic, the force coefficient between 2.08 and 2.32. For the full calculation, see Appendix G.

* NB In the strength tests, an optimization of the wind load has been implemented if a specific one assessment is not sufficient. The construction factor $c_s c_d$ = 0.85 has been used in accordance with the specification Rijkswaterstaat.

6.4.2 Bridge

The wind load on the deck construction is worked out in accordance with NEN-EN 1991-1-4 art. 8.

Wind load x direction

Reference surface

A bridge deck with flat beams is assumed. The reference area for the combinations without traffic load is determined in accordance with the draft version of the national annex (2015) of NEN-EN 1991-4 and is equal to the sum of:

1) the area affected by the wind of the height of the floor construction (concrete deck or driving floor with troughs) with the projection of the first main girder projecting beneath it plus the projections of the projecting parts of the second, third and subsequent main girders multiplied by coefficients of 2/3, (2/3) 2, etc. resp. the projected surface of the sleeve affected by the wind;
2) the projected surface from above the bridge deck protruding structural elements such as beams, footpath or ballast track, and
3) if applicable, the projected area from above described under 1) solid protruding obstacles or sound barriers, or in the absence thereof 0.3 m in front any open bridge railing or guardrail.

1. surface floor with the first and second main beam

The main beam has a height of 2.1 m. The concrete deck, excluding the bark side, has a height of 0.30 m, bringing the total height to 2.4 m.

m
5
2
0.2
m
3
0

2.1 m

Figure 65 - Bridge reference height

The second main beam, including rib, protrudes 2.2 m below the bottom of the deck. This will be 2/3 taken away.

2. surface of structural elements protruding from above the bridge deck

The scab side protrudes 0.225 m above the concrete deck.

3. surface of fixed obstacles

There are no fixed obstacles other than the vehicle barrier and the handrail. 0.3 m is assumed for each open bridge railing or crash barrier, a total of 4 x 0.3 m = 1.2 m.

The total height of this comes at d_{until} = 2.4 + 2/3 * 0.225 + 1.2 + 2.2 m m m = 5.29 ($A_{ref,x}$ = 635 m^2)

The reference area for load combinations with traffic load is based on a
2.0 m traffic band, measured from the top of the road surface. Assuming an asphalt thickness of 70 mm
this brings the total reference height d_{to} = 2.4 + 2/3 * 2.2 + 0.07 + 2.0 = 5.94 m ($A_{ref;x}$ = 712 m^2)

Reference height

The reference height is equal to the distance from the lowest ground level to the center of the bridge deck (without railing). The lowest ground level is at the location of the pillars at approximately 10.1 m + NAP [C.4350]. The bottom of the main beam is located at 21.3 m + NAP [A.22656]. Starting from a height of 2.625 m from the bottom of the main beam to the top of the side of the bar is the reference height 21.3 + 2.625 / 2 - 10.1 = 12.5 m.

Wind load (traffic dominant)

The wind load is detailed in Appendix G.1.

According to. For wind load combined with traffic, where traffic is the dominant load NEN-EN 1991-1-4 art. 8.1 for wind the highest value of $\psi_0 \cdot F_{wk}$ and F^*_w (with ψ_0 = 1.0) are considered.
F^*_w should be determined by replacing the basic value of the basic wind speed $v_{b;0}$ by $v^*_{b;0}$. For $v^*_{b;0}$ is maintained at 23 m / s.

Wind load with traffic	$\psi_0 \cdot F_{wk}$ = 2.6 kN / m_1
Wind load with traffic	**F^*_w = 8.1 kN / m_1**

It turns out that the value of F^*_w is decisive above $\psi_0 \cdot F_{wk}$.

Wind load (wind dominant)

For the wind load combined with road traffic, where wind is the dominant load, must account is taken of a wind load equal to F_{wk} with and without road traffic. For the situation with road traffic on (parts of) the bridge where road traffic is present, F_{wk} may be reduced to the wind load F^*_w.

Wind load without traffic:	F_{wk} = 7.3 kN / m_1
Wind load with traffic:	F^*_w = 8.1 kN / m_1

Billed wind load (wind load according to standard)

The wind load with traffic (wind dominant) is in this case equal to the wind load with traffic (traffic dominant). The tax combinations with traffic as the dominant tax will therefore always be are normative above the combination with wind dominant (with traffic).

The wind load without traffic (F_{wk}) is lower than the wind load with traffic (F^*_w), due to the lower height. The load combinations with wind load without traffic will therefore not be decisive.

In principle, it is therefore only necessary to calculate with a wind load F^*_w (with traffic).

Charged wind load (wind load with reduced traffic load)

The above means that there is simultaneously extreme wind load (F^*_w) and extreme traffic load occurs in the tax combinations in accordance with NEN-EN 1990. Rijkswaterstaat has discussed this with TNO, because this is very conservative and will therefore lead to high tensions.

At the request of Rijkswaterstaat, an extra set of tax combinations is also applied in the calculation taking into account a reduced traffic load (ψ_{red} = 0.9), if this is simultaneous occurs with F^*_w. Normal combinations are calculated for the combinations with Fwk.

The substantiation for the combination factor is as follows.
Imagine a storm with F * w that lasts 1/2 day 4 times a year, that is during the (residual) lifespan 4 * 1/2 days * 30 years = 2 months. To arrive at a reference period, we must sit a factor of 10 higher, therefore 20 months (+/- 2 years). This results in accordance with table 1 of NEN 8701 in approx. 90% of the traffic load.

The above reduction is applied in a separate set of load combinations and is used alone if a specific part is (or is not) sufficient. When these custom combinations are used this will be explicitly stated.

Load in y direction
According to NEN-EN 1991-1-4, for wind force in the y direction, 40% of the wind force in x direction to be taken into account. The wind force in the y direction occurs simultaneously with an equally great wind force in the x direction.

The wind forces in y direction $F^*_{w;y}$ is taken into account as an evenly distributed load $p^*_{w,y}$ on deck and is equal to (see also Appendix G.1):

Wind load with traffic	**$p^*_{w,y} = 0.34$ kN / m $_2$**

Load in z direction
Wind load in the z direction, perpendicular to the deck, is determined in Appendix G.1. The tax is in charged as an evenly distributed load on the deck.

In addition, an eccentricity of e = b / 4 has also been taken into account, by dividing the load into a constant wind load of $F^*_{w,z}$ and a progressive wind load (□F = 0) for the eccentricity. Both calculations are made once with a wind load without eccentricity and one wind load with eccentricity, with the highest load on the side with lane 1 apprehended. Calculations are also made with both an upward and a downward orientation wind load.

The load is charged as an evenly distributed load $p^*_{w,z}$ (see Figure 66). The load on the inspection paths is translated into a line load and line moment on the edge of the concrete deck.

Wind load with traffic (5 °)	**$p^*_{w,z} = 0.51$ kN / m $_2$ (constant)**
	$p^*_{w,z} = \pm 0.77$ kN / m $_2$ (progressing for eccentricity)

In addition to the above reduction, Rijkswaterstaat has also asked for the adjusted version load combinations with reduced traffic load assuming a reduced angle □ of the wind with the horizontal. In accordance with NEN-EN 1991-1-4, ± 5 ° should be assumed. This is a generic rule in the Eurocode that is probably on the conservative side for NL and TNO is here agree in concept. Rijkswaterstaat is proposing to reduce this to 2.5 ° + the influence of the slope of the bridge deck, and also to include this in the additional load combination.

Wind load with traffic (2.5 °)	**$p^*_{w,z} = 0.44$ kN / m $_2$ (constant)**
	$p^*_{w,z} = \pm 0.66$ kN / m $_2$ (progressing for eccentricity)

□$p^*_{w,z}$

$p^*_{w,z}$

−□$p^*_{w,z}$

Figure 66 - Wind load z direction deck

6.4.3 Main bridge

The wind load on the deck construction of the main bridge is worked out in a similar way as before the bridges. Because the main beam varies in height, the wind load will be at six locations determined, namely at the support points and at the center of the spans.
The wind load is interpolated in a straight line for the intermediate areas.

Reference surface
The reference area for the combinations without traffic load is equal to the sum of the following three parts:

1. surface floor with the first and second main beam
The first and second main beams have a variable height between 2.4 m and 5.3 m, excluding the edge strip protruding above deck. For the 2nd main beam, 2/3 of the height is placed under the deck charged.

0.22 m

0.15 m

m

5.3
-
2.4

Figure 67 - Main bridge reference height

The hot water pipes that hang between the main beams are for the calculation neglected. The tubes are round and will therefore catch relatively little wind. Moreover, they hang between the main girders, so that they are only blown by wind to a limited extent.

2. surface from above the bridge deck protruding constructional parts
The cut side has a height of about 0.22 m above the deck + 0.15 m on the other side above it asphalt.

3. surface of fixed obstacles
There are no fixed obstacles other than the vehicle barrier and the handrail. 0.3 m is assumed for each open bridge railing or crash barrier, a total of 4 x 0.3 m = 1.2 m.

The height for determining the reference surface therefore varies from d_{to} = 2.4 + 2/3 * 2.4 + 0.37 + 1.2 m = 5.57 m to d_{to} = 5.3 + 2/3 * 5.3 + 0.37 + 1.2 m = 10.40 m.

The reference area for load combinations with traffic load is based on a 2.0 m traffic band, measured from the top of the road surface. The total height including traffic band varies from d_{to} = 2.4 + 2/3 * 2.4 + 0.37 + 2.0 = 6.37 m to d_{to} = 5.3 + 2/3 * 5.3 + 0.37 + 2.0 = 11.2 m.

Reference height
The reference height is equal to the distance from the lowest ground level to the center of the bridge deck (without railing). The lowest ground level is at the location of the floodplain at approx. 10.1 m + NAP [C.4350] and at about 6.5 m + NAP at the river span (low water) [C.4352]. The position of the center of the beam varies by location.

Wind load (traffic dominant)
The wind load is detailed in Appendix G.2. For wind loads combined with traffic, where traffic is the dominant load, according to NEN-EN 1991-1-4 art. 8.1 the highest value for wind of □$_0 \cdot F_{wk}$ and F^*_w (with □$_0$ = 1.0) are considered. F^*_w should be determined by the fundamental value of the basic wind speed $v_{b;0}$ to be replaced by $v^*_{b;0}$.

	Pillar F & LH south [kN / m1]	Fields 1e and 5e span [kN / m1]	Pillar G and K [kN / m1]	Fields 2nd and 3rd span [kN / m1]	Pillar H and J [kN / m1]	Field 3e span [kN / m1]
Wind load with traffic (□$_0 \cdot F_{wk}$)	2.8	2.8	2.8	3.5	5.9	3.5
Wind load with traffic (F^*_w)	**8.9**	**8.9**	**9.0**	**11.1**	**18.6**	**11.0**

Table 30 - Wind load x direction (traffic dominant)

It turns out that the value of F * w is decisive above □ 0 · F wk .

Wind load (wind dominant)
For the wind load combined with road traffic, where wind is the dominant load, must account is taken of a wind load equal to F_{wk} with and without road traffic. For the situation with road traffic on (parts of) the bridge where road traffic is present, F_{wk} may be reduced to the wind load F^*_w .

	Pillar F & LH south [kN / m 1]	Fields 1 e and 5 e span [kN / m 1]	Pillar G and K [kN / m 1]	Fields 2 nd and 3 rd span [kN / m 1]	Pillar H and J [kN / m 1]	Field 3 e span [kN / m 1]
Wind load without traffic (F_{wk})	7.9	7.9	8.0	10.3	18.0	10.0
Wind load with traffic (F_{wk} , F^*_w)	9.4	9.4	9.5	11.2	19.6	11.6
Wind load with traffic (min (F_{wk} , F^*_w))	**8.9**	**8.9**	**9.0**	**11.1**	**18.6**	**11.0**

Table 31 - Wind load x direction (wind dominant)

Billed wind load (wind load according to standard)
The wind load with traffic (wind dominant) is in this case equal to the wind load with traffic (traffic dominant). The tax combinations with traffic as the dominant tax will therefore always be are normative above the combination with wind dominant (with traffic).

The wind load without traffic (F_{wk}) is lower than the wind load with traffic (F^*_w), due to the lower height. The load combinations with wind load without traffic will therefore not be decisive.

In principle, it is therefore only necessary to calculate with a wind load F^*_w (with traffic).

Charged wind load (wind load with reduced traffic load)
The above means that there is simultaneously extreme wind load (F^*_w) and extreme traffic load occurs in the tax combinations in accordance with NEN-EN 1990. Rijkswaterstaat has discussed this with TNO, because this is very conservative and will therefore lead to high tensions.

At the request of Rijkswaterstaat, an extra set of tax combinations is also applied in the calculation taking into account a reduced traffic load (□ red = 0.9), if this is simultaneous occurs with F^*_w . Normal combinations are calculated for the combinations with Fwk.

The substantiation for the combination factor is as follows.
Imagine a storm with F * w that lasts 1/2 day 4 times a year, that is during the (residual) lifespan 4 * 1/2 days * 30 years = 2 months. To arrive at a reference period, we must sit a factor of 10 higher, therefore 20 months (+/- 2 years). This results in accordance with table 1 of NEN 8701 in approx. 90% of the traffic load.

The above reduction is applied in a separate set of load combinations and is used alone if a specific part is (or is not) sufficient. When these custom combinations are used this will be explicitly stated.

In addition, in a number of cases calculations were also made with a reduced construction factor of $c_s c_d$ = 0.85 and a b / d to ratio, where b is based on two steel bridges. Because there is hardly any space is present between the two bridge decks, it is assumed that this is more in accordance with reality. This optimization has also been implemented only if a specific part is not sufficient.

Load in y direction
According to NEN-EN 1991-1-4, for wind force in the y direction, 40% of the wind force in x direction to be taken into account. The wind force in the y direction occurs simultaneously with an equally great wind force in the x direction.

The wind force in y direction $F^*_{w;y}$ is taken into account as an evenly distributed load $p^*_{w,y}$ on deck and is equal to (see also Appendix G.2):

	Pillar F & LH south [kN / m 2]	Fields 1 e and 5 e span [kN / m 2]	Pillar G and K [kN / m 2]	Fields 2 nd and 3 rd span [kN / m 2]	Pillar H and J [kN / m 2]	Field 3 e span [kN / m 2]
Wind load with traffic ($q^*_{w,y}$)	0.38	0.38	0.39	0.48	0.80	0.47

Table 32 - Wind load y direction (traffic dominant)

For the x direction, 40% of the load from the previous section is used.

Load in z direction
Wind load in the z direction perpendicular to the deck is specified in Appendix G.2. The tax is in charged as an evenly distributed load on the deck.

In addition, an eccentricity of e = b / 4 has been taken into account, by dividing the load into one constant wind load of $F^*_{w,z}$ and a varying wind load (□F = 0) for the eccentricity. Both calculations are made once with a wind load without eccentricity and one wind load with eccentricity, with the highest load on the side with lane 1 apprehended. Calculations are also made with both an upward and a downward orientation wind load.

The tax is charged as an evenly distributed tax $p^*_{w,z}$. The tax on the inspection paths have been translated into a line load and line moment on the edge of the concrete deck.

	Fields 1 e and 5 e span [kN / m 2]	Fields 2 nd and 3 rd span [kN / m 2]	Field 3 e span [kN / m 2]
Wind load (constant) ($p^*_{w,z}$)	0.52	0.51	0.56
Wind load (expired) (□$p^*_{w,z}$)	± 0.77	± 0.77	± 0.84

Table 33 - Wind load z direction (traffic dominant, □ = 5 °))

In addition to the above reduction, Rijkswaterstaat has also asked for the adjusted version load combinations with reduced traffic load assuming a reduced angle □ of the wind with the horizontal. In accordance with NEN-EN 1991-1-4, ± 5 ° should be assumed. This is a generic rule in the Eurocode that is probably on the conservative side for NL and TNO is here agree in concept. Rijkswaterstaat is proposing to reduce this to 2.5 ° + the influence of the slope of the bridge deck, and also to include this in the additional load combination.

	Fields 1 e and 5 e span [kN / m^2]	Fields 2 nd and 3 rd span [kN / m^2]	Field 3 e span [kN / m^2]
Wind load (constant) ($p^*_{w,z}$)	0.45	0.46	0.50
Wind load (expired) (□$p^*_{w,z}$)	± 0.67	± 0.69	± 0.74

Table 34 - Wind load z direction (traffic dominant, □ = 2.5 °))

□$p^*_{w,z}$

$p^*_{w,z}$

−□$p^*_{w,z}$

Figure 68 - Wind load z direction deck

6.4.4 Wind load reduction factor due to reference period

The wind load in accordance with NEN-EN 1991-1-4 applies for a design life of 50 years. At one For other design life, the wind speed should be adjusted with a c_{prob} factor. Compliant NEN-EN 1991-1-4 + A1 + C2: 2011 art. 4.2 Note. (4) applies:

= (1 - · (- (1 -)) / 1 - · (- (0.98)))

In which:
K = 0.281 (based on wind area III)
n = 0.5 (based on wind area III)

p = 1/30 years (renovation, use) (NB The taxes in appendix G already include this factor)
c_{prob} = 0.96

The reduction factor should be applied to the wind speed:

6.5 Temperature load

The temperature load is determined in accordance with NEN-EN 1991-1-5 and the accompanying National Annex. In front of the temperature load on the structural models of the main and bridge bridges, in accordance with the NEN-EN 1991-1-5 distinguished in the following components.

- Uniform temperature component;

• Vertical temperature component;
• Temperature difference between parts of the main supporting structure.

Uniform temperature component.

The uniform temperature component is based on a type 1b deck construction (steel deck) for the main bridge and type 2 (concrete deck on steel beams) for the roadway. In Appendix H, the Calculation of the temperature components is shown.

This gives the same uniform temperature component for both types. This amounts to one temperature increase in summer of 35 ° C and a temperature decrease in winter of -36 ° C.

Temperature component	Temperature difference (□ T) (a) Global warming	(b) Cooling
Even temperature component NT N	0 5 10 15 20 25 30 35 40	-40 -30 -20 -10 0
	35	-36
	2400 35	-36 2400

Figure 69 - Uniform temperature component main bridge

This load is applied to all parts of the construction, with the exception of the bottom edge of the portals. Because the supports on both sides of the portal are firmly modeled temperature load will quickly lead to excessive forces (prevented deformation). In in reality, the imposition will shift if the forces run too high. This is sufficient there is play in both the pivot bearings and the roller bearings. In addition, the bottom edge completely in the shade, so temperatures will never run as high as where in accordance with the standard must be taken into account.

Vertical temperature component
The calculation of the vertical temperature component is based on the steel deck (main bridge) of a pavement with a thickness of 70 mm. In the calculation, for the concrete deck (bridging) based on a concrete height of 200 mm. A thickness is assumed for the pavement of 70 mm.

Below are the results of the vertical temperature distribution over the height, where 0 mm corresponds to top of concrete / steel.

Main bridge

Temperature component	Temperature difference (□ T) (a) Global warming	(b) Cooling
Vertical temperature component MT M	0 2 4 6 8 10 12 14 16	-7 -6 -5 -4 -3 -2 -1 0
	0 15.0	-6.0 0.0 0100
	500 0.0	
	2400 0.0	0.0 2400

Figure 70 - Vertical temperature component main bridge

Bridge

Temperature component **Temperature difference (□ T)**

(a) Global warming **(b) Cooling**

Vertical temperature component

MT M

0 10 20 -10 -5 0

0 16.0 -4.0 0

120

120 4.0 200

200 2.9

400 0.0 -8.0 600

Figure 71 - Vertical temperature component bridge

Combination of uniform and vertical temperature component

In addition to the separate even and vertical temperature component, there should also be taken into account be held with a combination of both temperature loads. For this, the following combinations will be charged.

1.00 * □T M, heat (or □T M, cool) + 0.35 * □T N, heat (or □T N, heat)

0.75 * □T M, heat (or □T M, cool) + 1.00 * □T N, heat (or □T N, heat)

Main bridge

Combination 1

MT M + w N □T N w N = 0.35

□T b + w N □T N, exp = 27.3 ° C □T b - w N □T N, con = -18.6 ° C

□T 1 + w N □T N, exp = 12.3 ° C □T h1 - w N □T N, con = -12.6 ° C

□T o + w N □T N, exp = 12.3 ° C OT o - w N □T N, con = -12.6 ° C

0 5 10 15 20 25 30 -20 -15 -10 -5 0

0 27.3 -18.6 -12.6 0100

500 12.3

2400 12.3 -12.6 2400

Combination 2

w M □T M + □T N w M = 0.75

w M □T 1 + □T N, exp = 46.4 ° C w M □T 1 - □T N, con = -40.5 ° C

w M □T 2 + □T N, exp = 35.1 ° C w M □T h1 - □T N, con = -36.0 ° C

w M □T 3 + □T N, exp = 35.1 ° C w M □T o + □T N, exp = -36.0 ° C

0 10 20 30 40 50 -50 -40 -30 -20 -10 0

0 46.4 -40.5-36.0 0100

500 35.1

2400 35.1 -36.0 2400

Figure 72 - Combinations of uniform and vertical main bridge temperature component

Bridge

Temperature component		Temperature difference (□ T) (a) Global warming		(b) Cooling	
Combination 1		$\square T_1 + w_N \square T_{N,exp}$ =	28.3 ° C	$1T_1 - w_N \square T_{N,con}$ =	-16.6 ° C
$MT_M + w_N \square T_N$	$w_N = 0.35$	$2T_2 + w_N \square T_{N,exp}$ =	16.3 ° C	$\square T_{h1} - w_N \square T_{N,con}$ =	-12.6 ° C
		$\square T_o + w_N \square T_{N,exp}$ =	15.1 ° C	$\square T_h - w_N \square T_{N,con}$ =	-12.6 ° C
		$\square T_s + w_N \square T_{N,exp}$ =	12.3 ° C	$2T_2 - w_N \square T_{N,con}$ =	-20.6 ° C

0 10 20 30 -30 -20 -10 0

0 28.3 -16.6 0

120 16.3 -12.6 -12.6 120 200

200 15.1

400 12.3 -20.6 600

Combination 2		$w_M \square T_1 + \square T_{N,exp}$ =	47.1 ° C	$w_M \square T_1 - \square T_{N,con}$ =	-39.1 ° C
$w_M \square T_M + \square T_N$	$w_M = 0.75$	$w_M \square T_2 + \square T_{N,exp}$ =	38.1 ° C	$w_M \square T_{h1} - \square T_{N,con}$ =	-36.0 ° C
		$w_M \square T_o + \square T_{N,exp}$ =	37.3 ° C	$w_M \square T_h + \square T_{N,exp}$ =	-36.0 ° C
		$w_M \square T_s + \square T_{N,exp}$ =	35.1 ° C	$w_M \square T_2 + \square T_{N,exp}$ =	-42.0 ° C

0 20 40 60 -60 -40 -20 0

0 47.1 -39.1 0

120 38.1 -36.0 -36.0 120 200

200 37.3

400 35.1 -42.0 600

Figure 73 - Combinations of uniform and vertical temperature component bridge

Temperature load input SCIA

The input of temperature loads on beams must be entered in SCIA Engineer as a constant temperature and a gradient temperature, which runs linearly from the top of the profile to the bottom of the profile. It is therefore not possible to make a kink in the temperature trend about the height. Therefore, the vertical temperature component is divided into one average temperature ($\Delta T_{M,N}$), a linear gradient temperature ($\Delta T_{M,M}$) and a natural temperature ($\Delta T_{M,E}$).

Figure 74 - Dividing arbitrary temperature profile into an average, linear and own temperature

The average temperature component $\Delta T_{M,N}$ results in a (small) expansion, the linear course temperature component $\Delta T_{M,M}$ results in a curvature. Together, they result in exactly the same expansion and curvature as if the vertical temperature component ΔT_{M} had been introduced.

The remainder (ΔT_{M} - $\Delta T_{M,N}$ - $\Delta T_{M,M}$) is called its own temperature ($\Delta T_{M,E}$) and does not result in a curvature or expansion of the bridge and is further neglected. Only the component $\Delta T_{M,N}$ and $\Delta T_{M,M}$ are applied in the calculation. Annex H gives an indicative breakdown into one of the cross sections of the main beams. The temperature loads on the other cross sections will decrease with the further elaboration of the calculation can be worked out.

Temperature difference between parts of the main supporting structure

In addition to the aforementioned temperature effects, a horizontal one must also be taken into account temperature component, in accordance with Article 6.1.4.3. According to this article, account must be taken with a temperature difference of 5 ° C between the outer sides of the bridge. There is no reason to maintain a higher value. Given the height of the main beams, it was decided to do this maintain a temperature difference between the main beams and a constant for the protruding parts temperature trend. The temperature between the main beams has been kept at 0 ° C because of limitations in the calculation package. In SCIA it is not possible to have a gradient temperature over the width of a plate, only over the thickness the temperature can be variable be arrested. Given the limited voltages from this load case (approx. 8 Mpa), the error will be this does not have any significant consequences for the calculation in the UGT.

+ 2.5 ° C + 2.5 ° C 0 ° C 0 ° C 0 ° C -2.5 ° C -2.5 ° C

-2.5 ° C

2.5 ° C
+
0 ° C 0 ° C 0 ° C

0 ° C

Figure 75 - Horizontal temperature component

6.5.1 Temperature load reduction factor due to reference period

The temperature load in accordance with NEN-EN 1991-1-5 applies for a design life of 50 years. Bee another design life should be the maximum and minimum temperature in the shade adapted, in accordance with NEN-EN 1991-1-5 appendix A. This adjustment is, based on 30 years, included in the calculation of the temperature load.

6.6 Snow load

Snow load is not decisive and is not considered.

6.7 Settlements

The effects of settlements may normally be neglected to the extreme according to NEN-EN 1994-2 limit state of steel-concrete bridges, where all sections are class 1 or 2 and the moment resistance is not reduced by chicken. Class 4 profiles will also be built in the IJssel Bridge are present, so that in principle settlements must be charged. In accordance with the RBK and NEN 8700, the settlements must be based on the actual support settlements based on of measurements. However, these measurements have not been made available.

To view the sensitivity to fulcrum settlements, the indicative calculation is for the main bridge made from a fulcrum setting difference of 20 mm. For this, both supports are on one pillar 20 mm down. This results in a maximum tension in the main beam of 20 N / mm².

For the main bridge, the effect of a 20 mm support position of one of the two supports on the same pillar. This results in a maximum tension of 34 N / mm². Such a a large difference in settlement is unlikely, given the robustness of the pillars.

The tensions that follow from the settlements are so low that it has been decided to reduce the effects of settlements further negligible.

6.8 Variable load, hot water pipe

The variable load of the hot water pipe is based on the Strackee calculation [912-275.R01]. It should be noted, however, that in this calculation the horizontal loads are on the bridge through a frame that transfers its load on three crossbars. The other frames contribute their load to one cross beam in the calculation. This does not match the drawings and what has been done outside. In consultation with Rijkswaterstaat, the taxes have been determined as follows

Vertical load	F_z = 20 kN per tube, per suspension point
Longitudinal horizontal load:	F_x = 5 kN per tube, per suspension point

The load has been translated per suspension point into 4 point loads on the cross beams.

680 1220 680

Figure 76 - Variable load, hot water pipe

6.9 Load factors and combinations

6.9.1 Load factors

In the calculation, the bridges are tested with the load factors for Use (RBK). Any reinforcements is calculated with the load factors for Renovation (NEN 8700). With all calculations has assumed consequence class CC3.

	Permanent taxes			Traffic (with □ = 1)	Wind (with □ = 1)	Others changeable (with □ = 1)
	J $_{Gj, sup}$		J $_{Gj, inf}$			
	6.10a	6.10b (incl. ξ)	6.10a and 6.10b	□ $_{Q, 1}$	□ $_{Q, 1}$	□ $_{Q, 1}$
New construction	1.40	1.25	0.9	1.50	1.65	1.65
Renovation	**1.30**	**1.15**	**0.9**	**1.30**	**1.60**	**1.50**
Use	**1.25**	**1.15**	**0.9**	**1.25**	**1.50**	**1.30**
Disapproval	1.25	1.10	0.9	1.25	1.50	1.30

Table 35 - Partial load factors (□) for the extreme limit states STR and GEO in accordance with the RBK

6.9.2 Combination factors , □ factors

6.9.2.1 Combination factors according to standard

The □ factors are observed in accordance with NEN-EN 1990 table NB.16. The taxes “Footpaths”, Crowds of people, Special vehicles and Snow are irrelevant or not normative and are

Leave behind. See also the previous sections on the relevant taxes.

Added are the variable load on the inspection path (□ $_{0,1,2}$ = 0) and the variable load on the

hot water pipe (□ $_{0.1.2}$ = 1.0).

The load combinations gr3, gr4, gr5 and S are not relevant for the IJssel bridge (load not relevant or normative, see above) and have been omitted. Added is a tax combination I for load on the inspection path. The table below applies when using formula 6.10b. The cells with the dark blue background indicate the normative load in the relevant combination again. The dark text columns can be decisive and have been included in the calculation. The light text columns are never indicative and are therefore not included in the calculation.

						Tax combinations							
	gr1a		gr1b	gr2		W b		T b		I	I	A1 a, b	
TS (LM1)	1	1	0	0.8	0.8	0.8	0.64	0.8	0.64	0	0	0.8	0.64
UDL (LM1)	1	1	0	0.8	0.8	0.8	0.64	0.8	0.64	0	0	0.8	0.64
Single axis (LM2)	0	0	1	0	0	0	0	0	0	0	0	0	0
Horizontal load	0.8	0.8	0	1	1	0.64	0.8	0.64	0.8	0	0	0.64	0.8
Wind c F wk	0.3	0	0	0.3	0	0	0	0.3	0.3	0.3	0	0	0
F * w	0	1	0	0	1	1	1	0	0	0	1	0	0
Temperature	0.3	0.3	0	0.3	0.3	0.3	0.3	1	1	0.3	0.3	0	0
Inspection path	0	0	0	0	0	0	0	0	0	1	1	0	0
Impact on or under the bridge	0	0	0	0	0	0	0	0	0	0	0	1	1
Hot water pipe	1	1	1	1	1	1	1	1	1	1	1	1	1

a A1 = collision on or under the bridge and collision

b In these combinations is in the first column gr1a * □ 0 , and the second column gr2 □ * 0 . For the definition of the group traffic load gr1a and gr2 see NEN-EN 1991-

2 + C1

c Where traffic load is present on (parts of) the bridge, F * w may be used instead of F wk

Table 36 - □ factors for use in formula 6.10b

The combinations with 0.3 F wk are not decisive compared to F * w and are therefore not taken into account. The combinations with wind normative (W) are equal or lower as the combinations with traffic normative and are also not charged.
When using formula 6.10a the table below is used. Here only gr1a and gr2 normative.

							Tax combinations								
	gr1a		gr1b	gr2			W b			T b		I	I	A1 a, b	
TS (LM1)	0.8	0.8	0	0.64	0.64	0.8	0.8	0.64	0.64	0.8	0.64	0	0	0.8	0.64
UDL (LM1)	0.8	0.8	0	0.64	0.64	0.8	0.8	0.64	0.64	0.8	0.64	0	0	0.8	0.64
Single axis (LM2)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Horizontal load	0.64	0.64	0	0.8	0.8	0.64	0.64	0.8	0.8	0.64	0.8	0	0	0.64	0.8
Wind c F wk	0.3	0	0	0.3	0	0.3	0	0.3	0	0.3	0.3	0.3	0	0	0
F * w	0	1	0	0	1	0	1	0	1	0	0	0	1	0	0
Temperature	0.3	0.3	0	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0	0
Inspection path	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Impact on or under the bridge	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Hot water pipe	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

a A1 = collision on or under the bridge and collision

b In these combinations is in the first column gr1a * □ 0 , and the second column gr2 □ * 0 . For the definition of the group traffic load gr1a and gr2 see NEN-EN 1991-2 + C1

c Where traffic load is present on (parts of) the bridge, F * w may be used instead of F wk .

Table 37 - □ factors for use in formula 6.10a

6.9.2.2 Reduced combination factors

As also described in section 6.4 , the above combination factors in gr1a and gr2 mean that there are extreme wind load (F * w) and extreme traffic load occur simultaneously in the tax combinations. Rijkswaterstaat has consulted with TNO on this, because it is very conservative and will therefore lead to high voltages.

At the request of Rijkswaterstaat, an extra set of tax combinations is also applied in the calculation taking into account a reduced traffic load (□ red = 0.9), if this is simultaneous occurs with F * w . For the combinations with F wk , the normal traffic load is taken into account.

The above reduction is applied in a separate set of load combinations and is used alone if a specific part is (or is not) sufficient. When these custom combinations are used this will be explicitly stated. The table below applies when using formula 6.10b.

						Tax combinations							
	gr1a		gr1b	gr2		W b		T b		I	I	A1 a, b	
TS (LM1)	1	0.9	0	0.8	0.72	0.8	0.64	0.8	0.64	0	0	0.8	0.64
UDL (LM1)	1	0.9	0	0.8	0.72	0.8	0.64	0.8	0.64	0	0	0.8	0.64
Single axis (LM2)	0	0	1	0	0	0	0	0	0	0	0	0	0
Horizontal load	0.8	0.72	0	1	0.9	0.64	0.8	0.64	0.8	0	0	0.64	0.8
Wind c F wk	0.3	0	0	0.3	0	0	0	0.3	0.3	0.3	0	0	0
F * w	0	1	0	0	1	1	1	0	0	0	1	0	0
Temperature	0.3	0.3	0	0.3	0.3	0.3	0.3	1	1	0.3	0.3	0	0
Inspection path	0	0	0	0	0	0	0	0	0	1	1	0	0
Impact on or under the bridge	0	0	0	0	0	0	0	0	0	0	0	1	1
Hot water pipe	1	1	1	1	1	1	1	1	1	1	1	1	1

a A1 = collision on or under the bridge and collision

b In these combinations is in the first column gr1a * □ 0 , and the second column gr2 □ * 0 . For the definition of the group traffic load gr1a and gr2 see NEN-EN 1991-

2 + C1

c Where traffic load is present on (parts of) the bridge, F * w may be used instead of F wk

Table 38 - □ factors for use in formula 6.10b

The combinations with wind normative (W) are equal or lower as the combinations with traffic normative and have not been charged.

When using formula 6.10a the table below is used. Here only gr1a and gr2 normative.

	Tax combinations														
	gr1a		gr1b	gr2			W b			T b		I	I	A1 a, b	
TS (LM1)	0.8	0.72	0	0.64	0.576	0.8	0.8	0.64	0.64	0.8	0.64	0	0	0.8	0.64
UDL (LM1)	0.8	0.72	0	0.64	0.576	0.8	0.8	0.64	0.64	0.8	0.64	0	0	0.8	0.64
Single axis (LM2)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Horizontal load	0.64	0.576	0	0.8	0.72	0.64	0.64	0.8	0.8	0.64	0.8	0	0	0.64	0.8
Wind c F wk	0.3	0	0	0.3	0	0.3	0	0.3	0	0.3	0.3	0.3	0	0	0
F * w	0	1	0	0	1	0	1	0	1	0	0	0	1	0	0
Temperature	0.3	0.3	0	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0	0
Inspection path	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Impact on or under the bridge	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Hot water pipe	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

a A1 = collision on or under the bridge and collision

b In these combinations is in the first column gr1a * □ 0 , and the second column gr2 □ * 0 . For the definition of the group traffic load gr1a and gr2 see NEN-EN 1991-2 + C1

c Where traffic load is present on (parts of) the bridge, F * w may be used instead of F wk .

Table 39 - □ factors for use in formula 6.10a

6.9.3 Load combinations

6.9.3.1 Load combinations according to standard

Ultimate limit state (use)

The table below shows the load combinations for the permanent design situation (6.10a and 6.10b) and the extraordinary design situation (6.11b) for the assessment at the level “Use”, based on of the combination factors according to the standard.

	6.10a		6.10b						6.11b	
	gr1a	gr2	gr1a	gr1b	gr2	T b		I	A1 a, b	
Own weight	0.9 / 1.25	0.9 / 1.25	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15 0 , 9 / 1.15		1	1
Resting load	0.9 / 1.25	0.9 / 1.25	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15 0 , 9 / 1.15		1	1
Shrink and creep	1	1	1	1	1	1	1	1	1	1
Preload	1	1	1	1	1	1	1	1	1	1
TS (LM1)	1	0.8	1.25	0	1	1	0.8	0	0.8	0.64
UDL (LM1)	1	0.8	1.25	0	1	1	0.8	0	0.8	0.64
Single axis (LM2)	0	0	0	1.25	0	0	0	0	0	0
Horizontal load	0.8	1	1	0	1.25	0.8	1	0	0.64	0.8
Wind c F wk	0	0	0	0	0	0.45	0.45	0	0	0
F * w	1.5	1.5	1.5	0	1.5	0	0	1.5	0	0
Temperature	0.39	0.39	0.39	0	0.39	1.3	1.3	0.39	0	0
Inspection path	0	0	0	0	0	0	0	1.3	0	0
Impact on or under the bridge	0	0	0	0	0	0	0	0	1	1
Hot water pipe	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1	1

a A1 = collision on or under the bridge and collision

b In these combinations is in the first column gr1a * □ 0 , and the second column gr2 □ * 0 . For the definition of the group traffic load gr1a and gr2 see NEN-EN 1991-2 + C1

c Where traffic load is present on (parts of) the bridge, F * w may be used instead of F wk . For the combinations with □ 0 * F wk is assuming the wind load F * w with a load factor 1.5 * □ 0 * F wk / F * w = 1.5 * 0.3 * 1.056 = 0.48.

Table 40 - Load combinations for the ultimate limit state (use)

Project related

Limit state (renovation)

The table below shows the load combinations for the permanent design situation (6.10a and 6.10b) and the extraordinary design situation (6.11b) for checking the reinforcements at the level "Renovation", based on the combination factors according to the standard.

	6.10a		6.10b						6.11b	
	gr1a	gr2	gr1a	gr1b	gr2	T b		I	A1 a, b	
Own weight	0.9 / 1.3	0.9 / 1.3	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0 , 9 / 1.15	1	1
Resting load	0.9 / 1.3	0.9 / 1.3	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0 , 9 / 1.15	1	1
Shrink and creep	1	1	1	1	1	1	1	1	1	1
Preload	1	1	1	1	1	1	1	1	1	1
TS (LM1)	1.04	0.832	1.3	0	1.04	1.04	0.832	0	0.8	0.64
UDL (LM1)	1.04	0.832	1.3	0	1.04	1.04	0.832	0	0.8	0.64
Single axis (LM2)	0	0	0	1.3	0	0	0	0	0	0
Horizontal load	0.832	1.04	1.04	0	1.3	0.832	1.04	0	0.64	0.8
Wind c F_{wk}	0	0	0	0	0	0.48	0.48	0	0	0
F^*_w	1.6	1.6	1.6	0	1.6	0	0	1.6	0	0
Temperature	0.45	0.45	0.45	0	0.45	1.5	1.5	0.45	0	0
Inspection path	0	0	0	0	0	0	0	1.5	0	0
Impact on or under the bridge	0	0	0	0	0	0	0	0	1	1
Hot water pipe	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1	1

a A1 = collision on or under the bridge and collision

b In these combinations is in the first column gr1a * □ 0 , and the second column gr2 □ * 0 . For the definition of the group traffic load gr1a and gr2 see NEN-EN 1991-2 + C1

c Where traffic load is present on (parts of) the bridge, F^*_w may be used instead of F_{wk} . For the combinations with □ 0 * F_{wk} is assuming the wind load F^*_w with a load factor 1.6 * □ 0 * F_{wk} / F^*_w = 1.6 * 0.3 * 1.056 = 0.51.

Table 41 - Load combinations for the ultimate limit state (conversion)

Project related

Characteristic load combinations

The characteristic load combinations are determined according to formula 6.14b and are shown in the table below view. The characteristic load combinations are used for the calculations of the deflections.

Characteristic load combinations (6.14)

	gr1a	gr1b	gr2	W b		T b		I
Own weight	1	1	1	1	1	1	1	1
Resting load	1	1	1	1	1	1	1	1
Shrink and creep	1	1	1	1	1	1	1	1
Preload	1	1	1	1	1	1	1	1
TS (LM1)	1	0	0.8	0.80	0.64	0.8	0.64	0
UDL (LM1)	1	0	0.8	0.8	0.64	0.8	0.64	0
Single axis (LM2)	0	1	0	0	0	0	0	0
Horizontal load	0.8	0	1	0.64	0.8	0.64	0.8	0
Wind c F wk	0	0	0	0	0	0.3	0.3	0
F * w	1	0	1	1	1	0	0	1
Temperature	0.3	0	0.3	0.3	0.3	1	1	0.3
Inspection path	0	0	0	0	0	0	0	1
Impact on or under the bridge	0	0	0	0	0	0	0	0
Hot water pipe	1	1	1	1	1	1	1	1

b In these combinations is in the first column gr1a * □ 0 , and the second column gr2 □ * 0 . For the definition of the group traffic loads gr1a and gr2 see NEN-EN 1991-2 + C1

c Where traffic load is present on (parts of) the bridge, F * w may be used instead of F wk . For the combinations with □ 0 * F wk assume the wind load F * w with a load factor 1.0 * □ 0 * F wk / F * w = 1.0 * 0.3 * 1.056 = 0.32.

Table 42 - Characteristic load combinations

Frequent load combinations

The frequent load combinations are determined according to formula 6.15b and are in the table below view. The frequent load combinations are used for the calculations of the cracking in concrete. The combinations shown in light blue are not normative.

Frequent load combinations (6.15b)

	gr1a	gr1b	gr2	W.	T	I
Own weight	1	1	1	1	1	1
Resting load	1	1	1	1	1	1
Shrink and creep	1	1	1	1	1	1
Preload	1	1	1	1	1	1
TS (LM1)	0.8	0	0.8	0.4	0.4	0
UDL (LM1)	0.8	0	0.8	0.4	0.4	0
Single axis (LM2)	0	0.8	0	0	0	0
Horizontal load	0.8	0	0.8	0.4	0.4	0
Wind c F wk	0	0	0	0	0	0
F * w	0	0	0	0.6	0	0
Temperature	0.3	0	0.3	0.3	0.8	0.3
Inspection path	0	0	0	0	0	0
Impact on or under the bridge	0	0	0	0	0	0
Hot water pipe	1	1	1	1	1	1

c Where traffic is present on (parts of) the bridge, F * w may have been used instead from F wk .

Table 43 - Frequent load combinations

Quasi-permanent tax combinations

The quasi-permanent load combinations are determined according to formula 6.16b and are shown in the table below view. The quasi-permanent load combinations are used for the calculations of shrinkage and crawl into the concrete. The combinations shown in light blue are not normative.

Quasi-permanent tax combinations (6.16b)	gr1a	gr1b	gr2	W.	T	I
Own weight	1	1	1	1	1	1
Resting load	1	1	1	1	1	1
Shrink and creep	1	1	1	1	1	1
Preload	1	1	1	1	1	1
TS (LM1)	0.4	0	0	0.4	0.4	0
UDL (LM1)	0.4	0	0	0.4	0.4	0
Single axis (LM2)	0	0	0	0	0	0
Horizontal load	0.4	0	0	0.4	0.4	0
Wind F_{wk}	0	0	0	0	0	0
F^*_w	0	0	0	0	0	0
Temperature	0.3	0.3	0.3	0.3	0.3	0.3
Inspection path	0	0	0	0	0	0
Impact on or under the bridge	0	0	0	0	0	0
Hot water pipe	1	1	1	1	1	1

Table 44 - Quasi-permanent tax combinations

6.9.3.2 Reduced load combinations

Ultimate limit state (use) with reduced combination factors

The table below shows the load combinations for the permanent design situation (6.10a and 6.10b) and the extraordinary design situation (6.11b) for the assessment at the level "Use", based on of the reduced combination factors as described in section 6 .9.2.2. Therefore, for the combinations with F * w assume a reduced traffic load (□ = 0.9).

		6.10a						6.10b					6.11b	
	gr1a		gr2		gr1a		gr1b	gr2		T b		I	A1 a, b	
Own weight	0.9 / 1.25	0.9 / 1.25	0.9 / 1.25	0.9 / 1.25	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0 .9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	1	1
Resting load	0.9 / 1.25	0.9 / 1.25	0.9 / 1.25	0.9 / 1.25	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0 .9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	0.9 / 1.15	1	1
Shrink and creep	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Preload	1	1	1	1	1	1	1	1	1	1	1	1	1	1
TS (LM1)	1	0.9	0.8	0.72	1.25	1,125	0	1	0.9	1	0.8	0	0.8	0.64
UDL (LM1)	1	0.9	0.8	0.72	1.25	1,125	0	1	0.9	1	0.8	0	0.8	0.64
Single axis (LM2)	0	0	0	0	0	0	1.25	0	0	0	0	0	0	0
Horizontal load	0.8	0.72	1	0.9	1	0.9	0	1.25	1,125	0.8	1	0	0.64	0.8
Wind c F wk	0.45	0	0.45	0	0.45	0	0	0.45	0	0.45	0.45	0	0	0
F * w	0	1.5	0	1.5	0	1.5	0	0	1.5	0	0	1.5	0	0
Temperature	0.39	0.39	0.39	0.39	0.39	0.39	0	0.39	0.39	1.3	1.3	0.39	0	0
Inspection path	0	0	0	0	0	0	0	0	0	0	0	1.3	0	0
Impact on or under the bridge	0	0	0	0	0	0	0	0	0	0	0	0	1	1
Hot water pipe	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1	1

a A1 = collision on or under the bridge and collision

b In these combinations is in the first column gr1a * □ 0 , and the second column gr2 □ * 0 . For the definition of the group traffic load gr1a and gr2 see NEN-EN 1991-2 + C1

c Where traffic load is present on (parts of) the bridge, F * w may be used instead of F wk . For the combinations with □ 0 * F wk , the wind load F * w with a is assumed

load factor 1.5 * □ 0 * F wk / F * w = 1.5 * 0.3 * 1.056 = 0.48.

Table 45 - Load combinations for the ultimate limit state (use) with reduced combination factors

6.10 Fatigue

6.10.1 Explanation of approach to fatigue calculation

This section will first outline the approach to fatigue calculation, without going into detail about the numbers, loads, vehicle combinations, etc.

The fatigue test is basically carried out as follows:

1. Input unit loads (2x50 kN) in Scia Engineer;
2. Output influence lines of N / M / V or voltages of unit load to be considered part;
3. Determination of influence lines of stresses in section and / or connection;
4. Calculate influence lines unit load to influence lines of truck (configurations);
5. Determination of voltage ripples per configuration using the rainflow method;
6. Calculation and testing of fatigue damage.

Steps 1 and 2 will be performed in SCIA Engineer. Step 3 is performed with Excel. The steps 4 through 6 are performed using a Python calculation tool, so that all truck combinations for all years are fully developed, including all reductions and surcharges per year. In this way, there are no conservative simplifications in the calculations applied.

1. Input unit costs in scia engineer

In Scia Engineer, for unit lanes 1 and 2, a unit load becomes centric in the lane separately entered, so that the influence line can be implemented per part. For the eastern bridge (HRB) in 1990 the direction of travel was reversed. The calculation therefore takes into account a period from 1964-1989, with traffic traveling towards Utrecht and a period from 1990 to 2050, where traffic travels to Germany. For the western bridge (PRB), the traffic is always in the towards Germany.

Heart lane

2x50 kN

2x50 kN

1.8125 m

1.8125 m

0.6 m 0.42 m

3,625 m
(lane 1)

3,625 m
(lane 2)

0.42 m

0.60 m

Figure 77 - Positioning of unit loads, viewing direction towards Utrecht (eastern bridge 1964-1990)

Heart lane

2x50 kN

2x50 kN

1.8125 m

1.8125 m

0.6 m 0.42 m

3,625 m
(lane 2)

3,625 m
(lane 1)

0.42 m

0.60 m

Figure 78 - Positioning of unit loads, viewing direction to Germany (east bridge 1990-2051, west bridge 1964-2051)

In principle, this does not matter for the position of the unit costs. These remain in the same place. However, the direction has been reversed, so instead of a unit load that runs from south to north, there is in the second period assumed a unitary load that runs from north to south.

It is noted that the (local) distribution of the wheel loads according to figure 4.6 of NEN-EN 1991-2 of applicable for local parts. Since the driving deck does not have to be tested for fatigue inside this assignment, the application of the local variation of the wheel loads was not applied. The difference in ripple due to this variation will be negligibly small for the main components.

2. Output of NMV influence lines or voltages for the part to be considered

After entering the unit load, the influence line can be used in SCIA for a certain cross-section determined by leaving the unit load with a load step size of 1.0 m over the model move. A step size of 0.1 m is used for the cross beams if the axle load is close by comes from the cross beam to be tested. The influence line then follows for the indicated cross-section (N / M / V or the voltage). Two influence lines (N / M / V or voltage) are determined for each component; one of lane 1 and one for lane 2.

3. Determination of influence lines of stresses in section and / or connection

Based on the influence line of the truck configuration, the bar forces can be (N / V / M) translated into steel stresses in the extreme fibers of a cross-section or in a joint. In the local plate models, the tensions are read directly in step 2 and this translation is not necessary. Weld stresses are determined perpendicular to the throat of the weld and not based on the Von Mises tensions.

To be able to translate the influence lines of the unit load into influence lines for vehicle configurations, the linear interpolation increment is reduced from 1m to 0.1m.

The result are lines of influence of tensions in the fibers to be considered in the cross-section / connection based on taxes in the reference year 2000.

Figure 79 - Influence line tensions in the cross section (for illustration)

The locations to be tested will be determined on the basis of the strength calculation and in advance with Rijkswaterstaat are discussed.

4. Calculate influence lines unit load to influence lines of truck configurations

Using a calculation tool (Python), the influence line of one axis can be moved, copied and enlarged based on axle configurations and axle loads of a particular truck. Thus, the influence lines of the various trucks are determined for the reference year 2000 (see section 6.10.2) .

In addition, the influence lines of the various trucks can also be moved and copied to determine the influence lines for the various truck configurations (overtaking trucks and convoys, see section 6.10.3). This is shown in simplified form below:

Figure 80 - Translation of unit loads into truck configurations

The axle loads and axle distances of the various vehicles are known and are combined with the results of the unit loads to determine the influence line of a part for one truck configuration.

In order to obtain the correct results from the rainflow calculation in step 5, it is necessary that the influence lines of the vehicle combinations follow each other so that a cyclic movement occurs. To this to reach the part of the influence line from the beginning to the highest tension and on pasted the end of the influence line.

5. Determination of voltage ripples per configuration using the rainflow method
Using the rainflow method, based on the influence line for stresses, the various stress ripples are determined that are required to calculate the fatigue damage. It result is voltage ripple per truck configuration for the reference year 2000. The calculation tool for the rainflow method will be submitted for verification before all fatigue calculations are performed.

6. Calculation and testing of fatigue damage
As a last step, the voltage ripples of the reference year 2000 are translated into the individual ones years (1964-2051), so that the voltage ripple can be corrected for the development over time (see section 6.10.4) and the numbers per year (see appendix K). This may also include any other corrections are taken into account for the dynamic effects (see section 6.10.5) and the magnification factor at joint transitions (see section 6.10.6).

The damage is then determined per year on the basis of a calculation using the Miner rule in accordance with NEN-EN 1993-1-9 Appendix A.
The result is a graphic representation of damage development over time up to 2051.

Figure 81 - Evolution of fatigue damage over time (illustrative)

The assessment of the main supporting structure is initially based on the “safe life” method with high consequences of failure (γ_{Mf} = 1.35). In addition, in accordance with the tender, the fatigue damage of the main supporting structure presented γ_{Mf} = 1.15 and γ_{Mf} = 1.00. The damage will are determined for the year 2018 and for the year 2051. For locations with fatigue damage> 1.0 as of 2018, the influence lines per lane will be included as an appendix to the calculation report.

For the main carriageway, the direction of travel changed in 1990. Therefore, for the HRB, the chosen the following approach. The damage development is calculated once with traffic direction Utrecht (indicated with "until 1989), and once with traffic to Germany (indicated with" from 1990). In both calculations, it is assumed (fictitiously) that the entire period (1964-2050) is being driven in the same direction. Then the damage is determined by the correct periods with the add the correct direction of travel.

- HRB west:
  - Damage period 1964-1989, calculation HRB west (until 1989, traffic towards Utrecht) +
  - Damage period 1990-2050, calculation HRB west (from 1990, traffic towards Germany)
- HRB east:
  - Damage period 1964-1989, calculation HRB east (until 1989, traffic towards Utrecht) +
  - Damage period 1990-2050, calculation HRB east (from 1990, traffic towards Germany)

6.10.2 Detailed load model for fatigue

For the fatigue calculation of both bridges, in accordance with the tender, use is made of the Annex K contains an overview of historical and future numbers of trucks per year (N_{obs}). It should be noted that there is a difference in numbers between the western and eastern bridge, due to it being in 1990 the direction of travel changed on the western bridge. Both bridges will therefore separate on fatigue are assessed.

The number of vehicles per year is specified per bridge by the client in the tender and is split up in three periods, 1964 to 1989, 1990 to 2017 and 2018 to 2050. An overview of the number vehicles is given in Appendix K.

The tax is based on Appendix A2 of NEN 8701 for the situation “long distance” and the current and historical lane layout. The axle loads are split into three periods, namely the period opening until 1990, 1991 until 2010 and 2011 until the end of the reference period. It is noted that this time slots differ from the time slots defined for the numbers. This makes for the however, do not calculate it, because the damage calculations are worked out per year.