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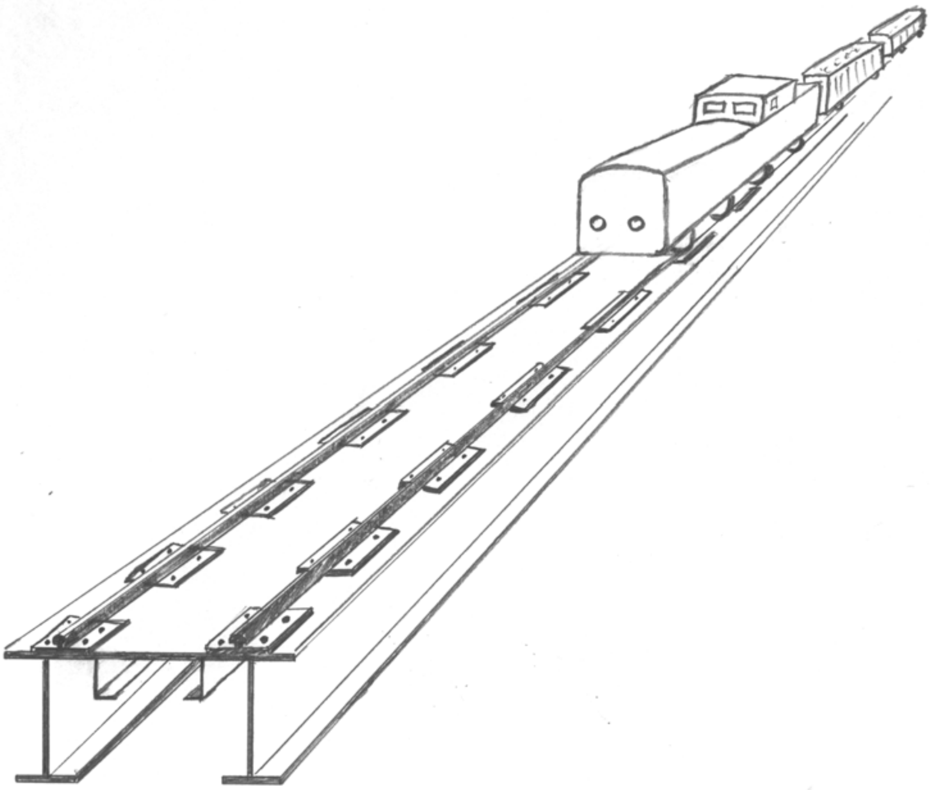

Eurocode versus Swedish national codes for steel bridges

Comparison of design calculations for the railway bridge over Kvillebäcken

Master's Thesis in the International Master's Programme Structural Engineering

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Steel and Timber Structures

CHALMERS UNIVERSITY OF TECHNOLOGY

Göteborg, Sweden 2005

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Cover:
Illustration of the railway bridge over Kvillebäcken

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ABSTRACT

The Swedish Rail Administration in Gothenburg initiated this Master's Thesis. The purpose was to obtain a clearer view of the transition from the Swedish codes to Eurocode (EC). An existing steel bridge was chosen as an object for study and comparison. The bridge is a single-track steel bridge located on Hisingen in Gothenburg. The structure consists of two I-beams with one upper flange. It is simply supported with a free span of 18 m.

The calculations that had been made during the design were analyzed. These calculations were performed with BRO 94, BV BRO, edition 4 and BSK 99 and an upgrade to the codes valid at present time, BRO 2004, BV BRO, edition 7 and BSK 99, was therefore made. After this, design calculations where performed according to EC. The upgrade and calculations were performed on the superstructure, in terms of ultimate limit state (ULS), serviceability limit state (SLS) and fatigue strength. The differences between the codes that emerged during the calculations were noted. Finally, the results were compared regarding the degree of utilization.

There were no major differences in the calculation principles between the codes. There were, however, a few differences concerning the loads and capacity. Loads prescribed for SLS and fatigue are lower in EC than in BV BRO. When calculating the permissible capacity in ULS, BSK 99 was more restrictive than EC. Moreover, when calculating the fatigue strength of the bridge, in the case of combined stresses, BSK 99 was more restrictive. Eurocode also ignores the parallel stresses in the longitudinal direction of the weld when checking the fatigue strength.

The Eurocode document contains a great deal of information. It is divided into many different parts and requires a great deal of work. The different parts refer to various documents the whole time. This makes it difficult to obtain a clear view of the document. A database in which the designer can search for the parts for a specific project would make the work easier.

Key words: Comparison, BV BRO, BRO 2004, BSK 99, Utilization

Eurocode kontra Svenska koder för stålbroar
Jämförelse av dimensioneringsberäkningar för järnvägsbron över Kvillebäcken
HENRIK JONSSON, JOHAN LJUNGBERG
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Chalmers tekniska högskola

SAMMANFATTNING

Detta examensarbete var initierat av Banverket Region Väst. Syftet var främst att få en överblick avseende skillnader som bör beaktas vid övergången från de svenska normerna till Eurocode (EC). En befintlig stålbro valdes som objekt. Bron är en stålbalkbro med ett spår och ligger på Hisingen i Göteborg. Den består av två I-balkar med gemensam överfläns. Den har en spännvidd på 18 meter och är fritt upplagd.

Beräkningarna som hade utförts vid dimensioneringen analyserades. Dessa beräkningar var utförda med BRO 94, BV BRO, utgåva 4 och BSK 99, varför en uppdatering till normer som gäller idag, BRO 2004, BV BRO, utgåva 7 och BSK 99, utfördes. Därefter utfördes beräkningar enligt EC. Uppdatering och beräkningar utfördes för brons överbyggnad gällande brottgränstillstånd, brukgränstillstånd och utmattningshållfasthet. De skillnader som dök upp i koderna under beräkningen noterades. Slutligen jämfördes resultaten med avseende på utnyttjandegrad.

Det var inga större skillnader i beräkningsprinciper mellan koderna. Däremot fanns en del skillnader avseende laster och kapacitet. Laster som rekommenderas för kontroll i brukgränstillstånd och för utmattning är lägre i EC än i BV BRO. Vid beräkning av tillåten kapacitet i brottgränstillstånd var BSK 99 mer restriktiv än EC. Även vid beräkning av brons utmattningshållfasthet, vid fallet med kombinerade spänningar, var BSK 99 mer restriktiv. Dessutom ignorerar EC parallella normalspänningar i svetsens längdriktning vid kontroll av utmattningshållfasthet.

EC dokumentet är väldigt omfattande och tungt att arbeta med. Det består av väldigt många delar och hänvisningar mellan delarna återkommer ständigt. Detta gör det svårt att få en överblick över dokumentet. En databas för att kunna söka efter de delar som behövs för det aktuella projektet skulle underlätta arbetet.

Nyckelord: Jämförelse, BV BRO, BRO 2004, BSK 99, Utnyttjandegrad

Contents

Preface

This International Master's Thesis was carried out from October 2004 until Mars 2005, at the Department of Civil and Environmental Engineering, Steel and Timber Structures, Chalmers University of Technology in Gothenburg, Sweden.

The Thesis was developed based on the suggestion from the Swedish Rail Administration. The work has been carried out at Chalmers University of Technology, in cooperation between Steel and Timber structures and Ramböll Sverige AB.

We would like to thank our examiner Mohammad Al-Emrani (Chalmers), our supervisors Magnus Bäckström (Chalmers) and Staffan Gustafsson (Ramböll Sverige AB), for their support and guidance throughout the thesis. A special thanks also goes out to Peter Lidemar (Swedish Rail Administration) for the initiation of this Master's Thesis.

Göteborg Mars 2005

Henrik Jonsson and Johan Ljungberg

Notations

Roman upper case letter

A	Area
C	Detail category
CL	Centre line
D	Dynamic coefficient in BV BRO
E	Young's modulus in EC
E_k	Young's modulus in BSK
F	Reaction force
F_{Brake}	Breaking force
$F_{\mathrm{Re}action}$	Reaction force at support
F_{Wind}	Wind force
$F_{w.Rd}$	Shear stress capacity for welds In EC
$F_{R\|\|}$	Shear stress capacity for welds in BSK
G	Permanent load
GC	Centre of gravity
I	Moment of inertia
L	Length
L_{best}	Determinant length according to BV BRO
L_{cr}	Characteristic length in EC
L_{Φ}	Determinant length according to EC
M	Moment
N	Normal force
N_R	Number of loading cycles
P_{Axle}	Force due to the axle of the train

P_{Nosing} Nosing force

Q_D Dominant variable load

Q_O Other variable loads

S First moment of area

W Flexural resistance

Roman lower case letter

a Deign section, length, spacing between transverse stiffeners

b Distance, Thickness of section

b_w Depth of the web

c Length

d Depth of the web

e Eccentricity

e_a Allowed eccentricity by the codes

$e1$ Eccentricity due to braking

$e2$ Eccentricity due to reaction force at support

f_{Rail} Factor for load increase due to rail displacement

f_{rd} Design value of the fatigue strength in BSK

$f_{rd||}$ Design value of the fatigue strength in BSK

$f_{rd\perp}$ Design value of the fatigue strength in BSK

f_{rk} Characteristic value of fatigue strength

f_u The nominal ultimate tensile strength of the weaker part

f_{uk} Characteristic value of ultimate tensile strength in BSK

f_{rvd} Design value of the fatigue strength in BSK

f_y Characteristic yield strength in EC

f_{yd}	Design yield strength
f_{yk}	Characteristic yield strength in BSK
h	Height of longitudinal stiffener
h_w	Height of web
i	Radius of gyration
k_τ	Buckling factor for shear
l_c	Characteristic length in BSK
m	3 or 5, depending on number of loading cycles
s	Width of load
$s1$	Distance between CL and web
$s2$	Distance between CL and track
t	Thickness
t_w	Thickness of web
q_u	Distributed load in kN/m
q_{vk}	Distributed load for load model SW/2
z	Height

Greek upper case letters

B_w	Is the appropriate correlation factor
ΔM	Variation spectra of moment
ΔV	Variation spectra of shear force
$\Delta\sigma_E$	Stress range of fatigue in EC
$\Delta\sigma_C$	Reference value of fatigue strength in EC
$\Delta\tau_C$	The design capacity of shear in fatigue
$\Delta\tau_E$	Stress range of fatigue in EC
$\Delta\sigma_{71}$	Stress range of load model LM 71

Φ	Dynamic factor in EC, variable in the reduction of buckling according to EC
Φ_2	Dynamic factor in EC, dynamic factor in fatigue
Φ_3	Dynamic factor in EC

Greek lower case letters

α	Imperfection factor in EC, variable in the reduction of buckling according to BSK, factor applied on the train load
β_1	Variable for a group
χ	Reduction factor for buckling in EC
φ	Reduction factor for butt weld in weld class WA and WB in BSK
φ'	Depends on permitted vehicle speed
φ''	Depends on span length
γ	Load factor in EC
γ_{Ff}	Partial safety factor of fatigue loading
γ_{Mf}	Partial safety factor of fatigue
$\gamma_{M,i}$	The partial safety factor in EC, is defined for each case to be checked
γ_{M2}	Partial factor of resistance
γ_m	Partial safety factor with regard to the uncertainty in determining the resistance
γ_n	Partial safety factor for safety class
γ_{steel}	Weight of steel in kN/m^3
κ	Standardized stress spectra
λ	Damage equivalence factor
λ_c	Slenderness parameter in BSK
λ_w	Web slenderness in BSK

λ_1 Is a factor for different type of girder that takes into account the damaging effect of traffic and depends on the length (span) of the influence line or area, variable for slenderness in EC

λ_2 Is a factor that takes into account the traffic volume

λ_3 Is a factor that takes into account the design life of the bridge

λ_4 Is a factor to be applied when the structural element is loaded by more than one track

$\overline{\lambda}$ Slenderness in EC

$\overline{\lambda}_w$ Web slenderness in EC

π Pi

$\sigma(z)$ Normal stress at the height z

σ_{rd} Stress response in the checked section

$\sigma_{rd\parallel}$ Stress range of fatigue in BSK

$\sigma_{rd\perp}$ Stress range of fatigue in BSK

$\sigma_{\parallel}$ Parallel stress

$\sigma_{\perp}$ Perpendicular stress

τ_{rd} Shear stress response in the checked section

$\tau_{rd\parallel}$ Shear stress range of fatigue in BSK

$\tau_{rd\perp}$ Shear stress range of fatigue in BSK

$\tau_{\parallel}$ Parallel shear stress

$\tau_{\perp}$ Perpendicular shear stress

ω Reduction factor for shear buckling

ω_c Reduction factor for buckling in BSK

ω_v Reduction factor for shear buckling

ψ Load combination factor in EC

$\psi\gamma$ Load combination factor in BV BRO

$\psi\gamma_G$ Load combination factor in BV BRO

$\psi\gamma_{\max}$ Higher value of load factor in BV BRO

$\psi\gamma_{\min}$ Lower value of load factor in BV BRO

Times New Roman upper case letter

E Effect of design load

L Span length

R Design value of resistance

U Utilization factor

Times New Roman lower case letter

a length in welds

1 Introduction

This Master's Thesis is a comparison regarding design calculations between the Swedish national building codes and Eurocode (EC). A railway steel bridge is chosen as an object for investigation of the differences between the codes.

1.1 Background

The work with EC has been going on for some time now, and the work is about to be completed, hence, the preparations using EC is now increasing. When the EC document is introduced as standard regulations, it will replace the national codes all throughout Europe. It will be a period of transition with a lot of question marks. The Swedish Rail Administration is well aware of this fact. The reason this task has been assigned to us is for the local Swedish Rail Administration in Gothenburg to get a clearer view of the differences to be, between the national Swedish codes used today and the EC document. However, the main office of the Rail Administration has a somewhat better knowledge about the new regulations, due to participation in committees processing Eurocode. (Peter Lidemar [23])

1.2 The Swedish Rail Administration (Banverket)

The Swedish Rail Administration is the authority responsible for rail traffic in Sweden. The Rail Administration follows and conducts the development in the railway sector. It supports Parliament and the Government with railway issues. The organisation is responsible for the operation and management of state track installations i.e. co-ordinate the local, regional and inter-regional railways. It also provides support for research and development in the railway sector. (www.banverket.se [18])

1.3 Steel bridges

Bridges are often built in steel, especially if the bridge is crossing over water. All types of bridges are built in steel, from small walkway bridges to suspension bridges with great span lengths. (SBI [5])

The main reason for using a steel structure is the short production time on site [23]. The parts can be put together in a factory and then transported to the scene. At LECOR Stålteknik AB in Kungälv this method is used (Tennce Carlsson [24]). In Figure 1.1 a steel beam with vertical stiffeners, ready to be delivered, is shown. More photos from the production of steel beams and steel bridges at LECOR can be seen in Appendix E.

Figure 1.1 Photo of a steel beam with vertical stiffeners, ready to be delivered.

This method is often used in densely populated areas where a minimum of disturbance is required. With modern technology, the noise from steel bridges can be reduced; also this makes the steel bridge useable in populated areas (www.sbi.se [20]). Another reason for choosing a steel bridge is when a bridge has to be replaced and the old supports will be used. In that case no extra weight should be added to the existing supports, which can cause new settlements. In some cases it is not possible to build a concrete bridge due to its geographical location, hence, a steel bridge is the only option. The possibility to build aesthetic and slender structures, with high strength, is a possibility that attracts many designers and architects. [23]

1.4 Eurocode

The work with EC began in 1975. The Commission of the European Community initiated an action program in the field of construction. This initiation was based on Article 95 of the Treaty of Rome, the objective was to eliminate technical obstacles to trade between countries and also to unite the appearance of technical specifications. The Commission continued the work for 15 years, with the assistance of a steering committee containing representatives of the EU member states. This led to, a first publication of the European codes in the 1980's.

In 1989 an agreement between CEN (European Committee of Standardization) and the European commission was made. The agreement gave CEN the task of preparations and publications of the Eurocodes.

At present time, EC is available as a pre-standard (ENV or prEN) but the work, to convert the documents in to full European standards (EN) is in progress. During the transmission time the EN will be valid beside the National Standards, and a NAD (National Application Document) for each country is developed. These NAD documents are written by each country and will be used during the transmission period. Publications of completed parts of EN are expected between 2002 and 2006. (www.sis.se [19])

The aim with a united European standard is to simplify and improve the work of buildings and structures. By these measures the clients, consultants and contractors will be able to perform their work in any country within EU and EFTA. This will increase the competition, over the borders in Europe, and in the long run lead to better structures for less cost.

EC contains calculation regulations for buildings and structures. EC consists of 10 design standards in 58 parts and approximately 6000 pages. The main standards are listed below. (www.eurocodes.co.uk [21])

Eurocode	Basis of Structural design
Eurocode 1	Action on Structures
Eurocode 2	Design of Concrete Structures
Eurocode 3	Design of Steel Structures
Eurocode 4	Design of Composite Steel And Concrete Structures
Eurocode 5	Design of Timber Structures
Eurocode 6	Design of Masonry Structures
Eurocode 7	Geotechnical Design
Eurocode 8	Design of Structures For Earthquake Resistance
Eurocode 9	Design of Aluminium Structures

1.5 Aim of the thesis

The aim of this Master's Thesis is to investigate the difference between Swedish national codes (Bro 2004, BV BRO, edition 7 and BSK 99) and Eurocode (EC). This is applied on the design calculations of the superstructure of a steel bridge located in Gothenburg Sweden. The differences will be enlightened and compared, regarding the degree of utilization.

1.6 Method

The Swedish Rail Administration has approved the design calculations and the bridge has already been built. The calculations that the designer made were handed to us. In the calculations the Swedish national codes, BRO 94, BV BRO, edition 4 and BSK 99 was used. Due to the fact that the codes, which are not valid today, have been used, an upgrade to the codes valid at present time (BV BRO, edition 7, BRO 2004 and BSK 99) had to be made. This was done, taking the existing calculations and put them into a Mathcad document. The calculations were translated into English and are presented in Appendix C. These calculations, were upgraded to the Swedish codes valid at present time, see Appendix A. After this a calculation was made with EC, see Appendix B.

To receive the sectional forces, Matlab was used. Matlab-files, received from the department of Structural Mechanics at Chalmers, for calculating sectional forces in the bridge was used. Results from the Matlab calculation are presented in Appendix D.

1.7 Limitations

The superstructure, of the bridge, will be analyzed using the already existing design calculations. There will be no extra calculations done. Only the main beam is checked and further checks concerning the bearings are therefore left out. The beam will be checked in the ultimate limit state, the serviceability limit state and for its fatigue strength.

2 The railway bridge over Kvillebäcken

The bridge, is located on Hisingen, an Island in Gothenburg, see Figure 2.1 (www.eniro.se [22]). It passes over a small stream called Kvillebäcken, see Appendix E. The bridge is a part of Hamnbanan, which leads out to the harbour of Gothenburg. Most of the cargo unloaded at the harbour has to take Hamnbanan before it is transported further out to its destination. About 60 trains per day run on this track. The bridge is located between two busy road bridges.

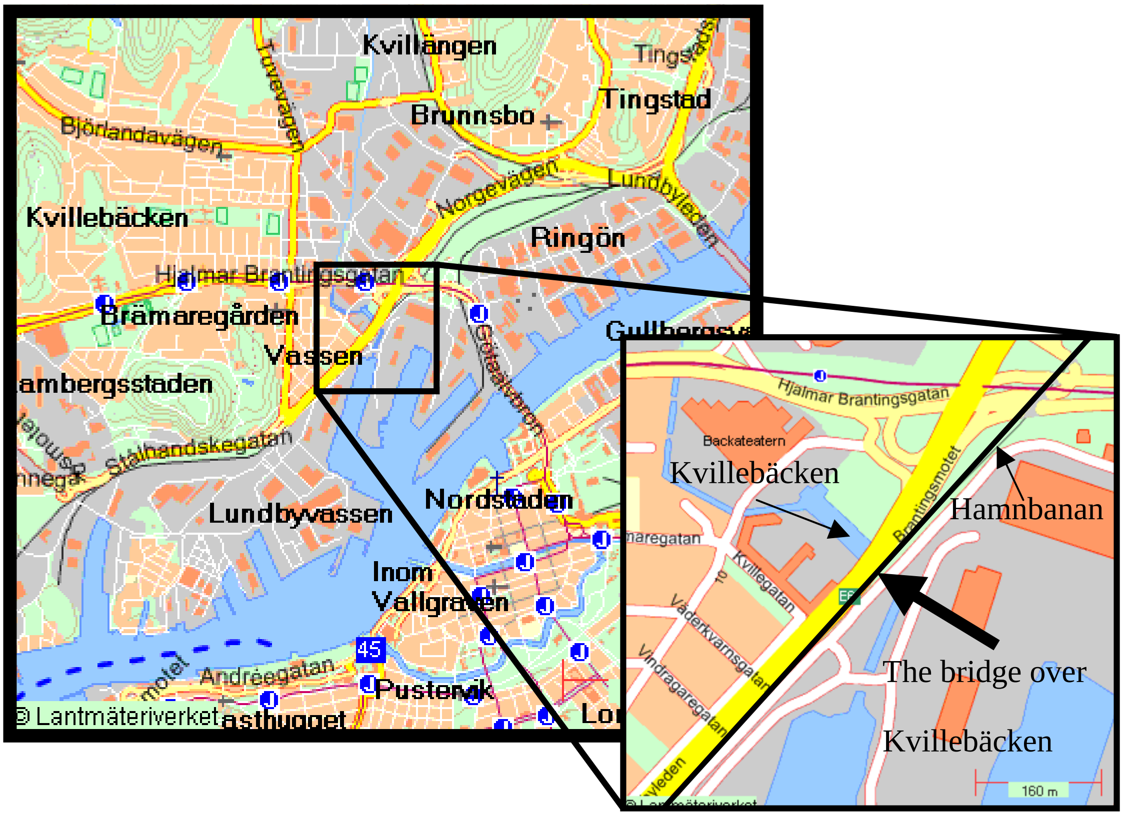

Figure 2.1 Location of the bridge.

The reasons for choosing a steel bridge in this particular case was due to the fact that it is located in a restricted area, between two busy road bridges. The Rail Administration also wanted the train traffic to get back to normal as quick as possible, i.e. a short production time was needed. The railway was temporary led in another route, during construction time.

The bridge is a single-track bridge with the total length of 18.8 m. It, is simply supported with a free span of 18 m, see Figure 2.2.

Figure 2.2 The bridge length measures

The cross-section of the load carrying structure consists of two I-beams (1), which are connected with one upper flange (2). Two longitudinal stiffeners (3) are placed inside the webs. The height of the cross-section is 1303 mm and the width of the upper flange is 2300 mm. The cross-section of the load carrying structure can be seen in Figure 2.3.

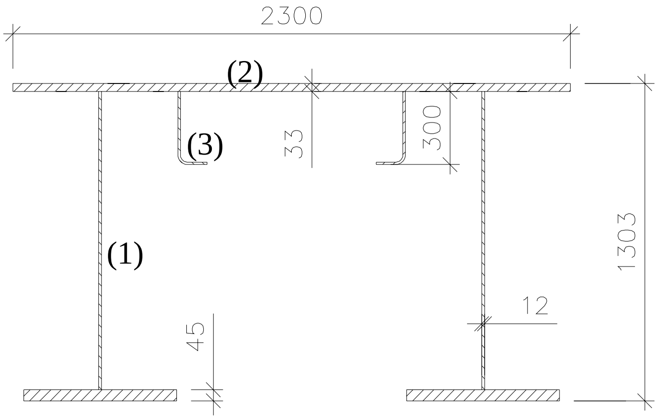

Figure 2.3 Cross section of the load carrying structure

The longitudinal stiffeners, prevents the upper flange from buckling. It also takes care of some of the local forces, which will affect the cross-section.

The bridge is equipped with vertical stiffeners (4), welded to the structure, see Figure 2.4. These vertical stiffeners are placed inside the webs, every third meter and prevent the webs of the main beam from buckling. Transverse beams (5), welded to the vertical stiffeners are placed every sixth meter. These beams prevent the bottom flanges from lateral torsional buckling. The rail (6) is resting on plates (7), which are connected to the upper flange by bolts. The rail is placed slightly inside the webs. Inside the rail, the derailment protection (8) is located. These are bolted to the upper flange above the longitudinal stiffeners.

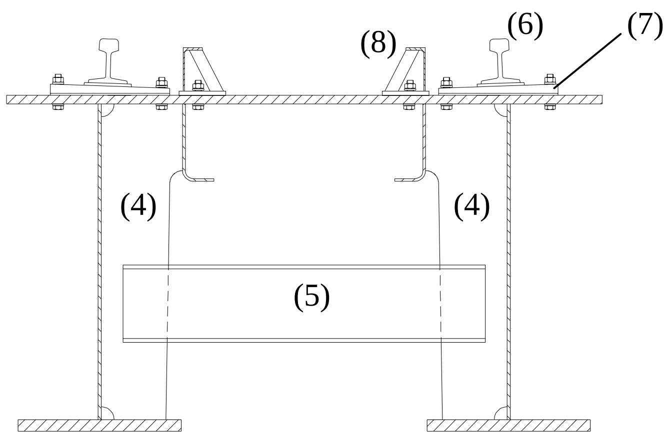

Figure 2.4 Cross-section shown with vertical stiffeners, transverse beams, railing and derailment protection

Along both sides of the bridge there are walkway bridges, which are not intended for pedestrians. The walkway bridges, are resting on cantilever beams, which are bolted to vertical plates, see Figure 2.5. The vertical plates are welded to the structure and are placed every sixth meter. These plates are only there for supporting the cantilever beams.

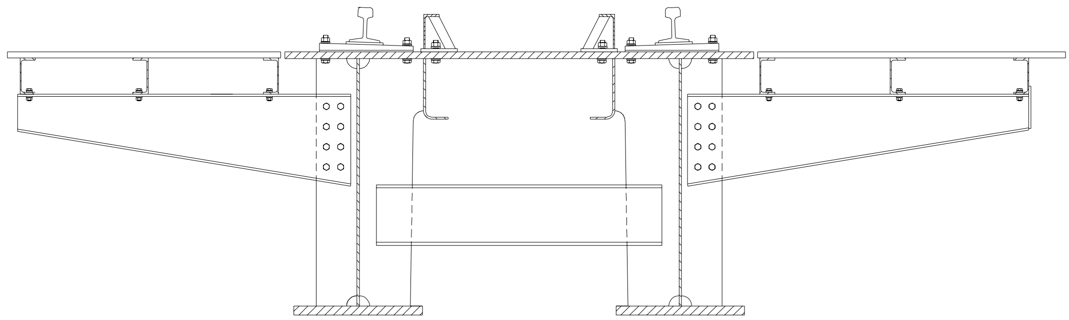

Figure 2.5 Cross-section, with walkway bridges.

At the support, the cross-section is provided with a continuous stiffener, covering almost the entire space between the two webs. It is also provided with a lifting device, to take the load from the hydraulic jack in case of a bearing change, see Figure 2.6.

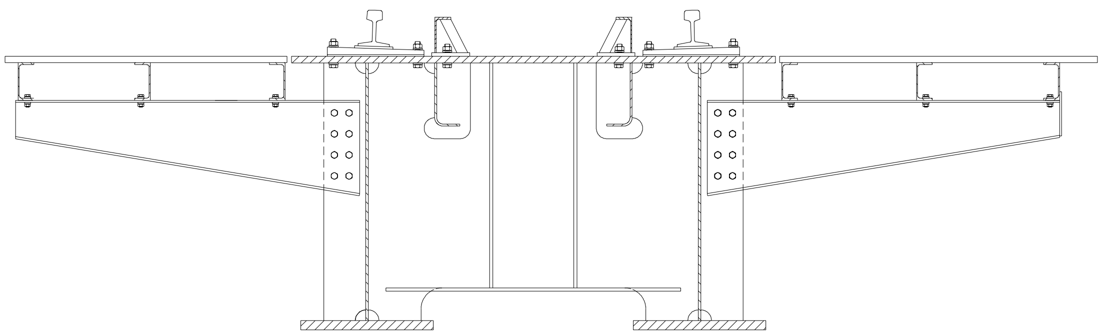

Figure 2.6 Cross-section at the support

3 Calculation principles

This section contains calculation principles used to check the capacity of the cross-section. It contains principles for the ultimate limit state, serviceability limit state and for fatigue strength. The principles presented here is the same used by the designer, who made the calculations handed out to us, however, more pictures and explanations have been added. A translated version of the original calculation can be seen in Appendix C.

3.1 Loads acting on the cross-section

The calculations are carried out on half of the cross-section, see Figure 3.1. This is because the vertical contribution from horizontal loads will act on one of the beams. This will be explained later.

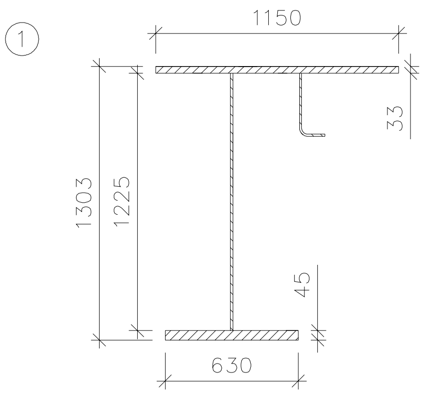

Figure 3.1 Cross-section in calculations

The cross-section is symmetrical. But if the rail is displaced during replacement, i.e. not placed exactly as it is shown on the blueprint, see Figure 3.2, this gives higher stresses and must be accounted for in the calculations. This is done using a rail-factor, which is multiplied with the load.

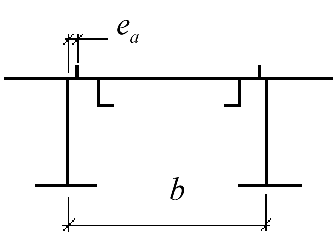

Figure 3.2 Load increase due to rail displacement

The rail-factor f_{Rail} is calculated as shown in equation (3.1).

$$f_{Rail} = \frac{\frac{b}{2} + e_a}{b} \qquad > 0.5 \tag{3.1}$$

Where b is the distance between the webs and e_a is the allowed eccentricity by the codes. The magnitude of the rail factor is larger than 0.5 and multiplied with the total vertical train load acting on the bridge.

Transverse horizontal loads, which do not act in shear centre, have to be transformed into vertical loads, due to the eccentricity. Two vertical forces take care of the moment that the eccentricity results in. An example of this, when the wind load acts on the train, is presented in Figure 3.3.

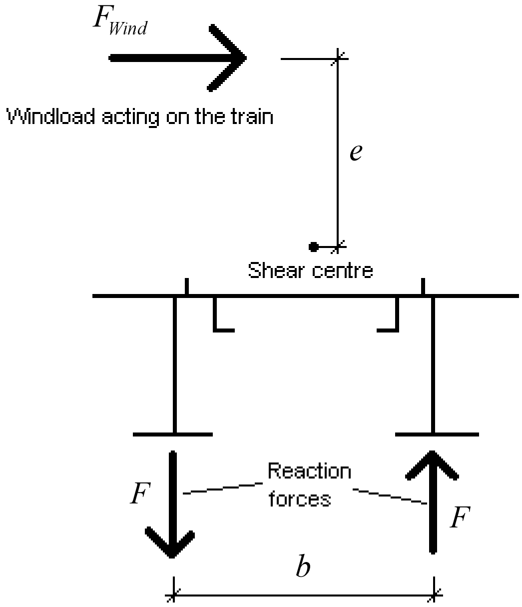

Figure 3.3 Vertical forces due to eccentricity of transverse horizontal loads

The vertical forces are calculated with a moment equation around the shear centre, see equation (3.2).

$$2 \cdot F \cdot \frac{b}{2} = F_{wind} \cdot e \,, \qquad F = \frac{F_{wind} \cdot e}{b} \tag{3.2}$$

Where e is the eccentricity of the load and b is the distance between the reaction forces.

A moment is caused by the longitudinal horizontal force due to eccentricity to the centre of gravity of the beam. This moment is the same in every section along the beam. An example of this is the braking force, which is acting in the top of the rail, as shown in Figure 3.4.

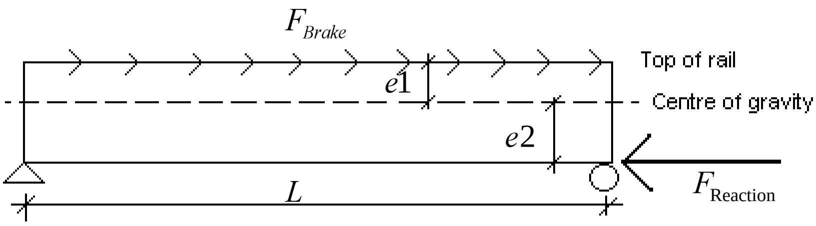

Figure 3.4 Longitudinal horizontal forces acting on the beam

If the eccentricity from gravity centre, to the braking force is smaller than the eccentricity from the centre of gravity, to the reaction force at the bearing, there will be a negative moment in the beam. This is a favourable action and will not be accounted for in the calculation. The moment is calculated as shown in equation (3.3).

$$M = F_{\text{Brake}} \cdot L \cdot e1 - F_{\text{Reaction}} \cdot e2 \tag{3.3}$$

3.2 Ultimate limit state

After all loads are accounted for, the first thing to do is to calculate the stress distribution over the cross-section. This is done for the largest vertical load, in the section with the highest stresses, in this case in the middle of the beam. Since the stress distribution is linear, it is easy to calculate the stress at any point of the cross-section, see Figure 3.5.

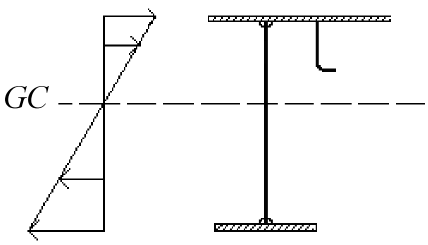

Figure 3.5 Stress distribution over the cross-section

The stress distribution is calculated, using Naviers formula, according to equation (3.4).

$$\sigma(z) = \frac{N}{A} + \frac{M}{I} z \tag{3.4}$$

3.2.1 Local effects

When calculating the stresses in the longitudinal stiffeners the local effects must be considered. To the stresses from global bending, the stresses from local actions are added. The nosing force, is such a case, see Figure 3.6. The nosing force is a single point load applied at the top edge of the rail in the transverse direction, simulating the train hitting the rail.

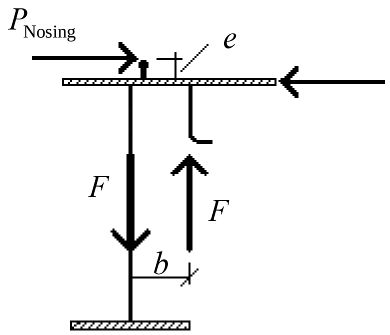

Figure 3.6 Local forces due to nosing force

The reaction force in the longitudinal stiffener due to the nosing force is calculated according to equation (3.5).

$$F = \frac{P_{\text{Nosing}} \cdot e}{b} \tag{3.5}$$

Since the rail is not placed exactly over the web of the beam, there will be a reaction force in the longitudinal stiffener, caused by the axle force from the train. When calculating this reaction force, the principle is the same as the previous case, see Figure 3.7.

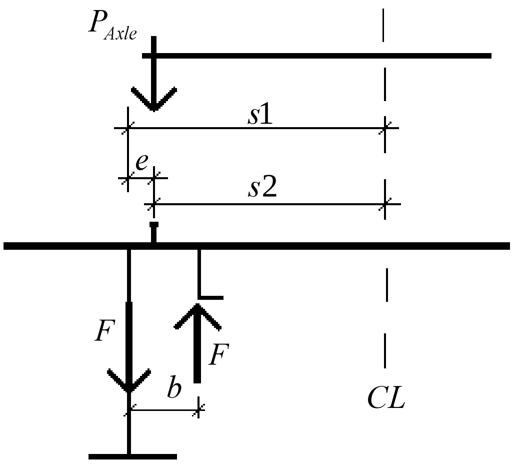

Figure 3.7 Local forces due to eccentricity of axle force

When calculating the force the allowed eccentricity e_a, according to Figure 3.2, should also be added to the actual eccentricity e. The reaction force in the longitudinal stiffener is calculated according to equation (3.6) and (3.7).

$$e = s1 - s2 \tag{3.6}$$

$$F = \frac{P_{Axle} \cdot (e + e_a)}{b} \tag{3.7}$$

The longitudinal stiffener is connected to vertical stiffeners at a certain distance L. To be able to calculate the stresses in the longitudinal stiffener due to the nosing- and axle force, the longitudinal stiffener is considered as a continuous beam with support at every vertical stiffener. The load is placed in the middle of the spans to get the largest stresses, see Figure 3.8.

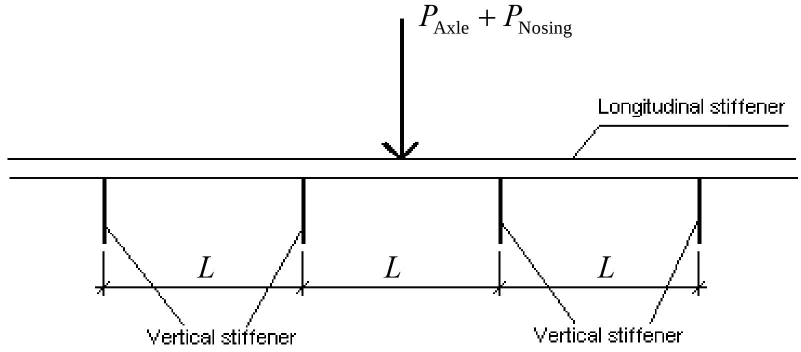

Figure 3.8 Nosing- and axle force acting on longitudinal stiffener which rest on the vertical stiffeners

When calculating the capacity of the longitudinal stiffener, stress from global bending is added to stress caused by nosing- and axle force.

3.2.2 Buckling

Buckling is checked for four parts of the cross-section, the longitudinal stiffener with effective upper flange, the plate between the longitudinal stiffeners, the flange of the longitudinal stiffener and the shear buckling of web.

When buckling of the longitudinal stiffener with effective upper flange is checked see Figure 3.9, the stresses caused by global bending are considered. The effective cross-section is stiffened at every position of a vertical stiffener. This makes the distance between the vertical stiffeners the buckling length. An average value of the stresses over the cross-section is calculated. The stress capacity over the section is reduced due to the risk of buckling and is compared to the average stress value.

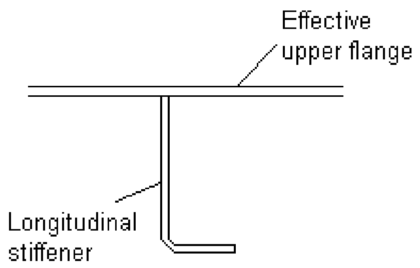

Figure 3.9 Longitudinal stiffener with effective flange

For the plate between the longitudinal stiffeners the free width of the part is calculated, see Figure 3.10. Then the stresses from global and local effects, acting on this part, are added together. The part is checked for its cross-section class and the capacity of the part is confirmed.

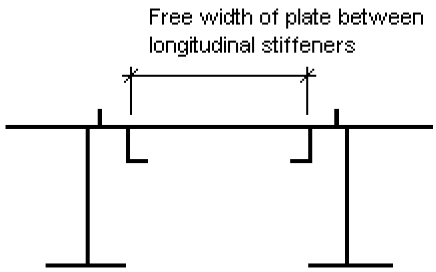

Figure 3.10 Free width of the plate between longitudinal stiffeners

The flange of the longitudinal stiffener is checked for lateral buckling. The flange is checked with the contribution of one third of the stiffeners web, see Figure 3.11. The part is taken out of the cross section and calculated as a simply supported beam with supports at the vertical stiffeners. This makes the buckling length also the distance between the vertical stiffeners. The stresses from local and global effects are calculated and transformed into a normal force. The capacity of the part is reduced due to the risk of buckling, and the capacity is then checked against the normal force in the part.

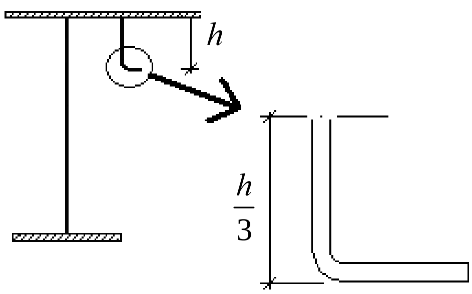

Figure 3.11 The flange of the longitudinal stiffener with one third of the web

The web of the beam is checked concerning shear buckling. This is checked close to the support $h_w / 2$, where the compression strut is possible, see Figure 3.12. The load is placed so the highest shear force occurs in the checked section. Since the web is slender the capacity must be reduced due to the risk of buckling.

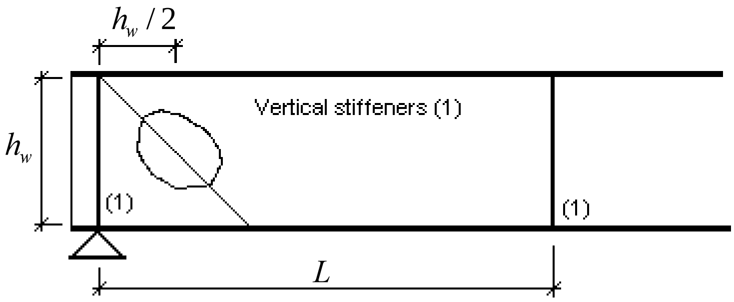

Figure 3.12 Shear buckling close to the support

3.2.3 Welds

Two welds are checked in the ultimate limit state. The butt weld between the upper flange and the web, and the fillet weld between the lower flange and the web, see Figure 3.13.

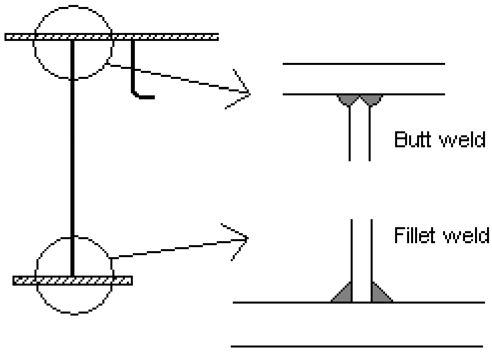

Figure 3.13 Welds between web and flanges

The welds are checked for the highest shear stress in their design sections a, see Figure 3.14. The design section for the fillet weld is the diagonal of the weld. For the butt weld the design section is the web.

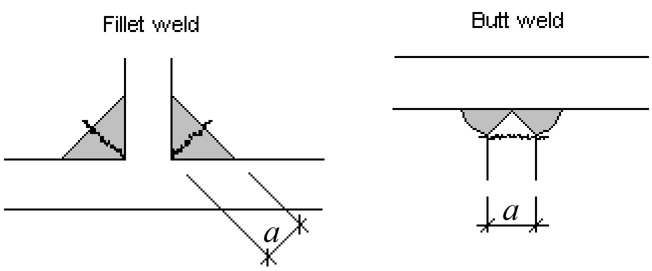

Figure 3.14 Design sections for butt-and fillet welds

3.3 Serviceability limit state

In the serviceability limit state the beam is only checked for maximum allowed displacement. For a simply supported bridge this occurs in the mid span and the actual displacement is checked against the maximum allowed displacement given by the code.

3.4 Fatigue strength

The beam is checked for its fatigue strength in three points. The checks are made for the butt weld between the web and the upper flange, the fillet weld between the vertical stiffener and the web and for the boltholes in the upper flange. For every certain detail a capacity is provided by the codes, together with the number of load cycles, the capacity of the checked section can be determined.

When checking the butt weld between the upper flange and the web, three stresses are considered, stress from local axle load, stress from global bending and global shear stress.

The stress from axle load is calculated as normal force acting on the weld. This stress will act perpendicular to the weld. The rail, is resting on plates, which are placed on top of the upper flange, see Figure 3.15.

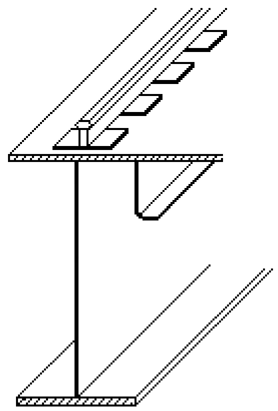

Figure 3.15 Plates on top of the upper flange

The load is transferred through the plates and then there will be a load distribution through the upper flange. The load distribution and calculation of the compression stresses acting on the butt weld is presented in Figure 3.16 and equation (3.8).

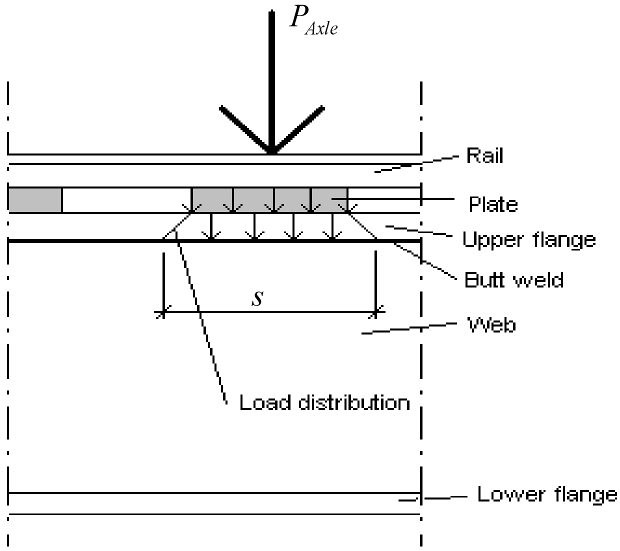

Figure 3.16 Load distribution in upper flange

$$\sigma_{\perp} = \frac{P_{Axle}}{t_w \cdot s} \tag{3.8}$$

Where t_w is the thickness of the web and s is the width of the load

The stress range, due to global bending are calculated for the greatest variation of moment, ΔM, in this case the mid span. This stress will act in the parallel direction of the weld. The stress is calculated using equation (3.9).

$$\sigma_{\parallel} = \frac{\Delta M}{W} \tag{3.9}$$

Where ΔM is the moment variation and W is the flexural resistance.

The stress range, due to shear is calculated using the highest variation of shear force, ΔV, in this case, close to the support. This stress is acting in the parallel direction of the weld. The shear stress $\tau_{\parallel}$ is calculated according to Jourawskis formula, see equation (3.10).

$$\tau_{\parallel} = \frac{\Delta V \cdot S}{I \cdot b} \tag{3.10}$$

Where ΔV is the shear force variation, S is the first moment of area, I is the moment of inertia and b is the thickness of the section.

All stresses are calculated for the sections with the largest load effects, see Figure 3.17. In case of combined stresses, the stresses are checked using an interaction formula, where the value is restricted to a certain limit. If the value exceeds the given limit, the weld must be checked in all sections, using the real values of the stresses acting in these sections.

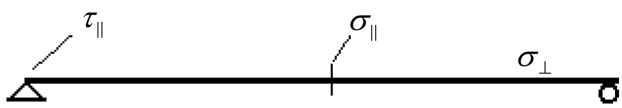

Figure 3.17 Worst sections for each individual stress

The stress, $\sigma_{\perp}$, caused by the axle load, is acting on each position of the plates carrying the rail. Therefore, this stress will be the same for every section along the beam.

At the lower flange, the weld between the vertical stiffener and the web is the worst case, see Figure 3.18.

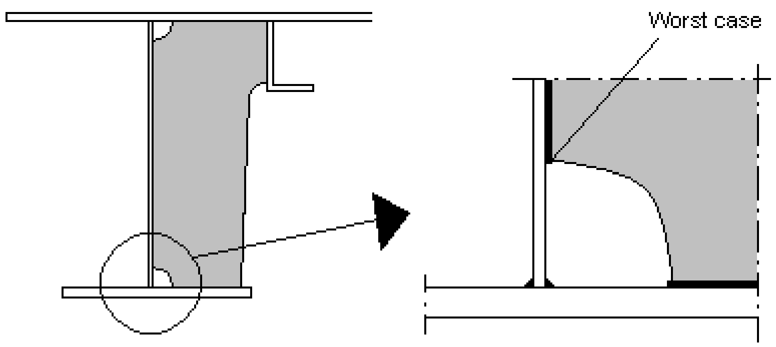

Figure 3.18 Worst case for lower flange

The weld is checked for stresses caused by global bending, i.e. from moment and shear force. To be on the safe side, the stresses are calculated at the point where the web meets the flange. The principle of the calculations is the same as for the upper butt weld.

The boltholes in the upper flange are checked for its fatigue strength concerning global bending. The upper flange is only subjected to compression stress; therefore it is only this stress that has to be considered.

4 Comparison

This section contains the differences between Swedish codes and EC. The differences that have been noticed during the calculations with BV BRO, edition 7, BRO 2004, BSK 99 and the EC document will be enlightened. The references in this chapter will be to one of these four documents. Complete calculations according to the Swedish codes and EC can be found in Appendix A and B respectively.

4.1 Loads

This section contains the principle differences of the loads between the Swedish national codes BV BRO, edition 7, BRO 2004 and the EC document.

4.1.1 Self-weight

Self-weight of steel is the only permanent load acting on the bridge. The self-weight according to EC is slightly higher than BRO 2004. In BRO 2004 the value of the weight γ_{steel} is provided to 77 kN/m^3 and in EC it is suggested that a mean value between 77-78.5 kN/m^3 is used for γ_{steel}. (EN 1991-1-1, [7])

4.1.2 Train load

In BV BRO one type of train load is recommended, BV 2000, while there are two different load models that have to be considered in EC, LM 71 and SW/2.

The principle of train load BV 2000 in BV BRO and LM 71 in EC is the same. It consists of two uniformly distributed loads q_u and four axle loads P_{Axle}, see Figure 4.1.

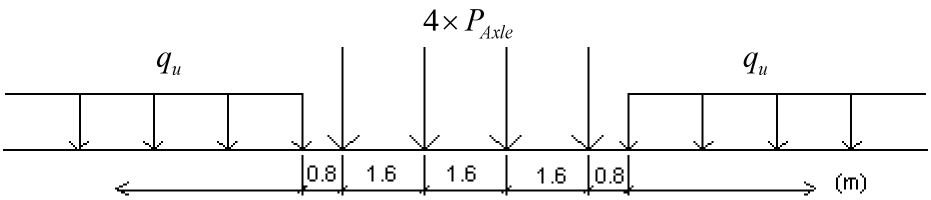

Figure 4.1 Principle train load for BV 2000 and LM 71.

The train load BV 2000 has characteristic values of P_{Axle} = 330 kN and q_u = 110 kN/m. In BV BRO there is also a train load for ore traffic, with characteristic values of P =350 kN and q_u =120 kN/m.

In EC the train load LM 71 has characteristic values of P = 250 kN and q_u = 80 kN/m. These characteristic values are multiplied with a factor α to get the classified values. The recommended values of α are 0,75-0,83-0,91-1,00-1,21-1,33-1,46. The designer chooses the value of α, depending on, the traffic situation on the railway line and which country the railway is located in.

Load model SW/2, given by EC, consists of two uniformly distributed loads q_{vk}, which represent the static effect due to heavy rail traffic, see Figure 4.2.

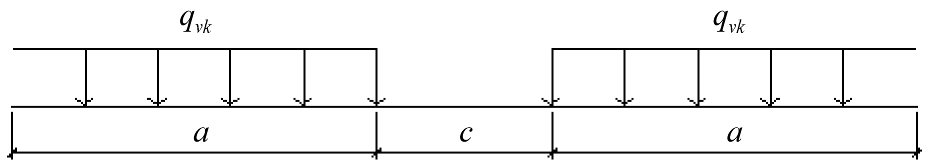

Figure 4.2 Load model SW/2

Where q_{vk} = 150 kN/m, c = 7.0 m and a = 25 m. This load case suites for continuous bridges, since, it results in greater moments over supports.

The load model, which provides the greatest response, in every section along the beam, is chosen as design load. This means that different loads can be design load for different sections along the beam. (prEN 1991-2, [8])

4.1.3 Dynamic factor

According to BV BRO the static train load shall be multiplied by a dynamic coefficient D, see equation (4.1)

$$D = 1.0 + \frac{4}{8 + L_{best}} \tag{4.1}$$

Where the value of L_{best} is the determinant length of the structural member considered. A guide how to decide the determinant length for the structural member can be found in tables in BV BRO. The determinant length depends on different conditions, for example continuous or simply supported structural member. This formula for D is the same for every check, only the value of L_{best} varies.

In EC, there has to be determined, whether a dynamic analysis is required. This, is done by use of a flow chart, see Figure 4.3.

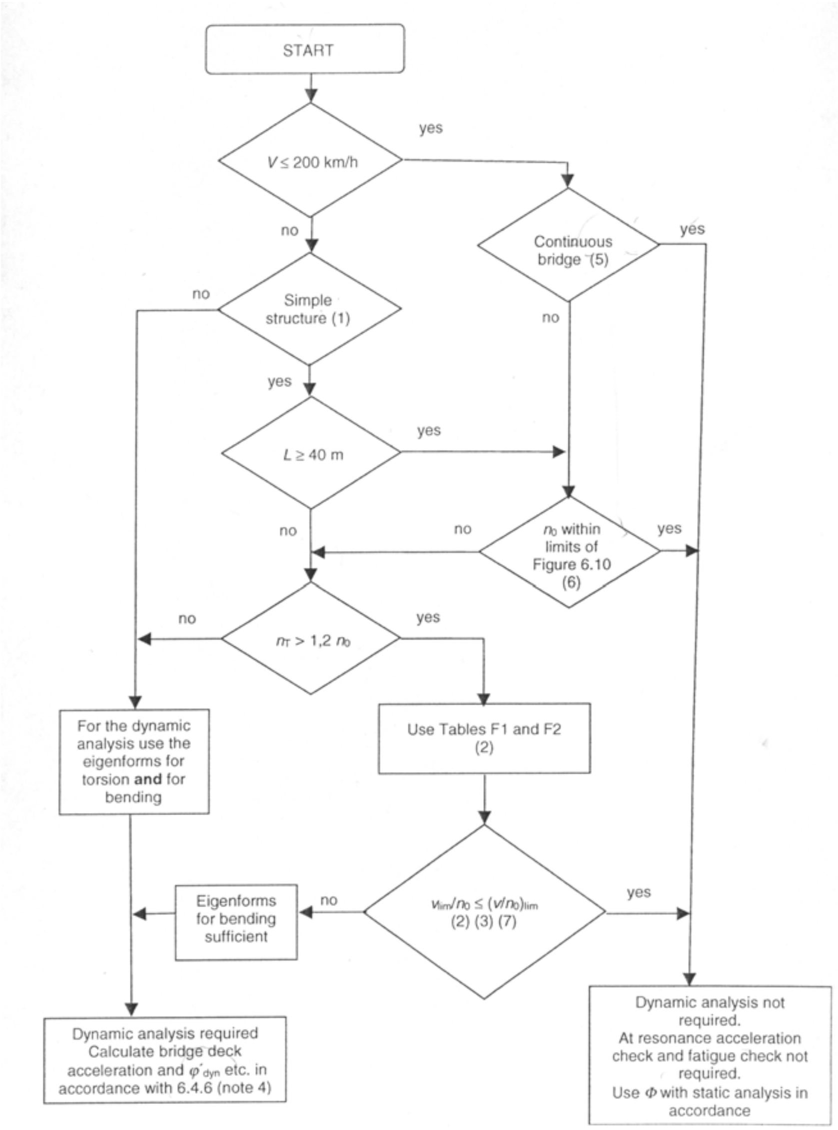

Figure 4.3 Flow chart for determining whether a dynamic analysis is required.

If a dynamic analysis is required, a description on how to consider this is carefully described in EC.

If a dynamic analysis is not required the procedure resembles BV BRO. The static train load is multiplied by a dynamic factor Φ. There are two different cases to consider, which are shown in equation (4.2) and (4.3).

For carefully maintained track:

$$\Phi_2 = \frac{1.44}{\sqrt{L_\Phi} - 0.2} + 0.82 \qquad \text{with } 1.00 \le \Phi_2 \le 1.67 \qquad (4.2)$$

For track with standard maintenance:

$$\Phi_3 = \frac{2.16}{\sqrt{L_\Phi} - 0.2} + 0.73 \qquad \text{with } 1.00 \le \Phi_3 \le 2.0 \tag{4.3}$$

Where the value of L_Φ is the determinant length of the structural member to be considered. A guide for how to decide the determinant length for each structural member can be found in tables in EC in the same way as in BV BRO. [8]

4.1.4 Derailment load

The derailment load is a load case, which considers the effects on the bridge in case of derailment of the train on the bridge.

In BV BRO the derailment load is provided with a specific load case, and the centerline of the bridge shall be displaced in the transverse direction.

According to EC there are two design situations to consider.

Design situation I: The train load LM 71 shall be displaced in the transverse direction.

Design situation II: The train load LM 71 is balancing on the edge of the bridge.

For the bridge over Kvillebäcken the derailment protection prescribe the distance the load can be displaced in the transverse direction. [8]

4.1.5 Nosing force

The nosing force is a single point load applied at the top edge of the rail in the transverse direction.

The characteristic value of the nosing force is the same for both codes, P_{Nosing} =100 kN. In EC however, the nosing force shall be multiplied by the same factor α applied on the vertical train load, see section 4.1.2. [8]

4.1.6 Wind load

In BRO 2004 the given characteristic value of the force induced by the wind velocity acting on the bridge is 1.8 kN/m^2 up to a height of 10 m. For greater heights than 30 m the value is 2.6 kN/m^2. Between 10 m and 30 m the value is interpolated linearly. The wind load acting on the train is given by BV BRO to 60 % of the wind load acting on the bridge and the height of the train is set to 4 m.

In EC the force induced by wind velocity acting on the bridge is recommended to 6 kN/m^2. For the wind load acting on the train there is no recommendations to be found. In this case the principle of calculating wind load on buildings had to be used. This value was afterwards multiplied with the force coefficients for wind, which are prescribed for bridges. (ENV 1991-2-4 [9])

4.1.7 Brake- and acceleration force

The principle of taking out the horizontal loads due to braking and acceleration is the same for both codes. However, in EC acceleration is referred to as traction. The values are given in kN/m and are then multiplied by the length of the bridge to get the value in kN.

In BV BRO the value of the braking force due to train load BV 2000 is 27 kN/m but ≤ 5400 kN. The acceleration force is 30 kN/m but ≤ 1000 kN.

In EC it is referred to as traction and braking force. The traction force due to train load LM 71 is 33 kN/m but ≤ 1000 kN. The braking force is 20 kN/m but ≤ 6000 kN. These values should be multiplied by the factor α, see section 4.1.2. [8]

4.1.8 Fatigue load

In BV BRO the fatigue load is given as a specific load case. In this load case the loads and load factors are defined. The load is multiplied by the same dynamic coefficient D presented in section 4.1.3.

In EC the simplified fatigue load model for railway bridges is used. This consists of the characteristic values of train load LM 71. The load effect $\Delta\sigma_{71}$ is multiplied by a dynamic factor Φ_2, and a damage equivalence factor λ, see equation (4.4). [8]

$$\Delta\sigma_E = \lambda \cdot \Phi_2 \cdot \Delta\sigma_{71} \tag{4.4}$$

Where $\Delta\sigma_E$ is the stress range in the checked section.

In EC the dynamic factor according to section 4.1.3 can be used, or a specific dynamic factor for fatigue Φ_2 can be chosen, see equation (4.5). This factor considers the maximum permitted vehicle speed allowed on the bridge, and the determinant length of the structural member considered. [8]

$$\Phi_2 = 1 + \frac{1}{2} \cdot (\varphi' + \frac{1}{2} \cdot \varphi'') \tag{4.5}$$

φ' Depends on permitted vehicle speed

φ'' Depends on span length

The damage equivalence factor λ, see equation (4.6), considers the span length of the bridge, the traffic volume on the bridge, the design life of the bridge and the number of tracks on the bridge. The values are received from tables given in EC. (ENV 1993-2:1997, [14])

$$\lambda = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4 \tag{4.6}$$

4.1.9 Load combinations

In BV BRO the load combinations for different design situations are given in a table. For every load to consider a load factor $\psi\gamma$ is presented with an upper and a lower value. The upper value is to be used for the dominant variable load and the lower values for the other variable loads, see equation (4.7).

$$\psi\gamma_G \cdot G + \psi\gamma_{\max} \cdot Q_D + \sum \psi\gamma_{\min} \cdot Q_O \tag{4.7}$$

In EC the load factor γ is given to the dominant variable load. For other variable loads the load factor γ is combined with the combination factor ψ, see equation (4.8).

$$\gamma \cdot G + \gamma \cdot Q_D + \sum \psi \cdot \gamma \cdot Q_O \tag{4.8}$$

The example in ULS is shown Table 4.1.

Table 4.1 Load factors and combination factors according to BV BRO and Eurocode for self-weight, train load and wind load in ULS

Load		**BV BRO**	**EC**
G	Permanent load	$\psi\gamma$ = 1.05	γ = 1.05
Q_D	Trainload (Dominant variable)	$\psi\gamma_{\max}$ = 1.4	γ = 1.45
Q_O	Wind load (Other variable)	$\psi\gamma_{\min}$ = 0.6	γ = 1.5 ψ = 0.75

In this case the permanent load and train load has almost the same factor. However, for the wind load, the characteristic value is increased according to EC while it is decreased according to BV BRO. These two different ways of combining the loads also have great effects when forces due to local effects are calculated, see Appendix A and B sections 2.2.3 and 2.2.4. (EN 1990:2002, [6])

4.2 Ultimate limit state (ULS)

This section contains the principle differences in ULS, between the BSK 99 and the EC document.

4.2.1 Characteristic yield strength

For the same steel material, the characteristic yield strength is different according to the codes, in BSK f_{yk} and in EC f_y. This is because of the different material thickness in the tables, BSK has more intervals than EC. An example for steel with quality class S355N is shown in Table 4.2. (ENV 1993-1-1:1992, [10])

Table 4.2 Characteristic yield strength values of steel for quality class S355N according to BSK 99 and EC.

S335N	BSK 99		EC 3	
	t	f_{yk}	t	f_y
	[mm]	[MPa]	[mm]	[MPa]
	-16	355	-40	355
	(16)-40	345		
	(40)-63	335	(40)-100	335
	(63)-80	325		
	(80)-100	315		

4.2.2 Design value of the yield strength

The design value of the strength is different in the codes, see Appendix A and B section 2.1.6.

In BSK the design strength is calculated according to equation (4.9).

$$f_{yd} = \frac{f_{yk}}{\gamma_m \gamma_n} \tag{4.9}$$

γ_m Partial safety factor with regard to the uncertainty in determining the resistance, chosen to 1.0 or 1.1.

γ_n Depends of the prescribed or chosen safety class, which result in one of the values 1.0, 1.1 or 1.2. According to BV BRO, bridges are always designed in safety class 3, and therefore γ_n is 1.2.

In EC the design strength is calculated according to equation (4.10).

$$f_{yd} = \frac{f_y}{\gamma_{M,i}} \tag{4.10}$$

$\gamma_{M,i}$ The partial safety factor is defined specifically for each design case to be checked.

When the design calculation is performed it is stated in EC, which of the partial safety factors that shall be used. For example, the partial factors presented in SS EN 1993-2 (NAD) is listed below.

$\gamma_{M0} = 1{,}0$	For cross-section what ever class
$\gamma_{M1} = 1{,}0$	For members of instability
$\gamma_{M2} = 1{,}1$	But not higher than 0,9 f_u/f_y for load carrying capacity of net cross-section
$\gamma_{M2} = 1{,}2$	For connections
$\gamma_{M3} = 1{,}2$	
$\gamma_{M3,ser} = 1{,}1$	For serviceability limit state
$\gamma_{M4} = 1{,}1$	
$\gamma_{M5} = 1{,}1$	For welded connections
$\gamma_{M6} = 1{,}0$	
$\gamma_{M7} = 1{,}1$	

4.2.3 The dynamic factor

The dynamic factor D can in BV BRO be chosen to 1.5 on the safe side, with L_{best} set to zero. In EC this is not possible in the same way, since the formula for the dynamic factor is different. The example is showed below. For this bridge, the choices can be seen in Appendix A and B section 2.2.4.

In BSK the dynamic factor calculated according to equation (4.11).

$$D = 1 + \frac{4}{(8 + L_{best})} \qquad L_{best} = 0 \Rightarrow D = 1.5 \tag{4.11}$$

In EC the dynamic factor is calculated according to equation (4.12).

$$\Phi_2 = \frac{1.44}{\sqrt{L_\phi} - 0.2} + 0.82 \qquad L_\phi = 0 \Rightarrow \Phi_2 = -6.38 \tag{4.12}$$

The value according to BV BRO is reasonable, while the value from EC is not usable in a design situation. For short length of structural members a dynamic analysis according to section 4.1.3 has to be done or a value between $1.00 \le \Phi_2 \le 1.67$ can be chosen. To be on the safe side the value of 1.67 should be used. [8]

4.2.4 Buckling resistance of members

The reduction factor in BSK ω_c and in EC χ, is calculated in similar ways, but the formulas look different, see Appendix A and B section 2.2.2.

In BSK the reduction factor for buckling is calculated according to equation (4.13).

$$\omega_c = \frac{\alpha - \sqrt{\alpha - 4.4 \cdot \lambda_c^2}}{2.2 \cdot \lambda_c^2} \qquad \text{Reduction factor for buckling} \tag{4.13}$$

$$\alpha = 1 + \beta_1 \cdot (\lambda_c - 0.2) + 1.1 \cdot \lambda_c^2$$

$$\lambda_c = \frac{l_c}{\pi \cdot i} \cdot \sqrt{\frac{f_{yk}}{E_k}} \qquad \text{Slenderness parameter}$$

$$i = \sqrt{\frac{I}{A}} \qquad \text{Radius of gyration}$$

$$\beta_1 = 0.49 \qquad \text{For group c}$$

In EC the reduction factor for buckling is calculated according to equation (4.14). (prEN 1993-1-1:2004, [11])

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}^2}} \qquad \text{Reduction factor for buckling} \tag{4.14}$$

$$\Phi = 0.5 \cdot \left[1 + \alpha \cdot \left(\overline{\lambda} - 0.2\right) + \overline{\lambda}^2\right]$$

$$\overline{\lambda} = \frac{L_{cr}}{i \cdot \lambda_1} \qquad \text{Slenderness for cross-section class 1, 2 and 3}$$

$$\lambda_1 = \pi \cdot \sqrt{\frac{E}{f_y}}$$

$i = \sqrt{\frac{I}{A}}$ Radius of gyration

$\alpha = 0.49$ Imperfection factor

If the characteristic strength is given the same value, in the different codes ($f_{yk} = f_y$), then the magnitude of the reduction factors are almost the same for this bridge.

BSK

f_{yk} =345 Mpa $\Rightarrow \omega_c = 0.8714$

EC

f_y =345 Mpa $\Rightarrow \chi = 0.8736$

4.2.5 Buckling of the plate between the longitudinal stiffeners

EC has four classes of the cross-section and the fourth class concern buckling of the cross-section before yielding. BSK is similar, but has three classes and the third concern buckling of the cross-section before yielding. In the third class according to the Swedish codes, the thickness of the member is reduced, however, in EC the part of the member that buckles is cut out of the cross-section, see Figure 4.4.

The bridge plate between the stiffeners is in cross-section class 2 (BSK) and class 3 (EC). This result in that the member does not buckle, see Appendix A and B section 2.3. [11]

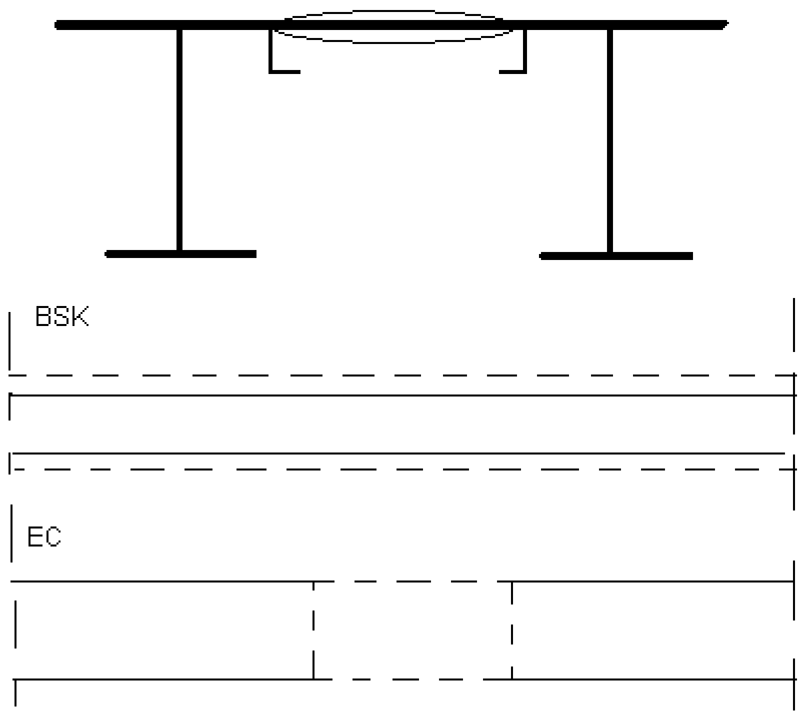

Figure 4.4 Reduction of cross-section due to buckling

4.2.6 Shear buckling of the web

Here the differences is the partial safety factor applied on the characteristic strength, in EC it is 1.0. According to BV BRO and BSK, γ_n is always 1.2 for bridges, see Appendix A and B section 2.4. The formulas for reduction of the design resistance look different but result in almost the same values for this case, but the calculation of the buckling factor for shear k_τ is the same for both codes, see equation (4.15-4.18) (K18, [4]). Another difference is that EC offers choices for verifying the capacity, the simple post-critical method and the tension field method. Bellow, the calculations of the reduction factor can be seen.

BSK

$k_\tau = 5.34 + 4 \cdot (b_w / a)^2$	Buckling factor for shear	(4.15)
$\lambda_w = \dfrac{0.81}{\sqrt{k_\tau}} \cdot \dfrac{b_w}{t_w} \cdot \sqrt{\dfrac{f_{yk}}{E_k}}$	Web slenderness	
$\omega_v = \dfrac{0.5}{\lambda_w}$	Reduction factor for shear buckling	(4.16)
b_w	Depth of the web	
a	Spacing between transverse stiffeners	
t_w	Thickness of web	

EC (Simple post-critical method), [10]

$k_\tau = 5.34 + 4 / (a / d)^2$	Buckling factor for shear	(4.17)
$\bar{\lambda}_w = \dfrac{d / t_w}{37.4 \cdot \sqrt{\dfrac{235}{f_y}} \cdot \sqrt{k_\tau}}$	Web slenderness	
$\omega = \dfrac{0.9}{\bar{\lambda}_w \cdot \sqrt{3}}$	Reduction factor for shear buckling	(4.18)
d	Depth of the web	
a	Spacing between transverse stiffeners	
t_w	Thickness of web	

4.2.7 Welds

The main difference, in this section about welds, is the calculating of the shear capacity.

In BSK the shear capacity is calculated according to equation (4.19).

$$F_{R\parallel} = \frac{0.6 \cdot \varphi \cdot f_{uk}}{1.2 \cdot \gamma_n} \tag{4.19}$$

$\varphi = 0.9$	Reduction factor for butt weld in weld class WA and WB
$f_{uk} = 490\,\mathrm{MPa}$	Characteristic value of ultimate tensile strength
$\gamma_n = 1.2$	Partial factor regarding safety class
$F_{R\parallel} = 183.75\,\mathrm{MPa}$	In this case, see Appendix A section 2.5

In EC the shear capacity is calculated according to equation (4.20).

$$F_{w.Rd} = \frac{f_u / \sqrt{3}}{\mathrm{B}_w \cdot \gamma_{M2}} \tag{4.20}$$

$f_u = 490\,\mathrm{MPa}$	The nominal ultimate tensile strength of the weaker part
$\mathrm{B}_w = 0.9$	Is the appropriate correlation factor
$\gamma_{M2} = 1.2$	Partial factor of resistance
$F_{w.Rd} = 261.95\,\mathrm{MPa}$	In this case, see Appendix B section 2.5

It is the extra factor of 1.2 applied on the characteristic strength in BSK, which result in the large difference of the capacity. Also the factor of 0.9 is applied differently in the codes. The calculations can be followed in Appendix A and B section 2.5. (prEN 1993-1-8:2002, [12])

4.3 Serviceability limit state

In serviceability limit state the bridge is checked for maximum allowed displacement. The procedures are the same but the loads and allowed limits differ between the codes.

In BV BRO a specific load case, including train load and wind load, is provided. The limit for maximum displacement is L/800.

In EC one load is used. It is the characteristic value of train load LM 71. The limit for maximum displacement is L/600. [6]

4.4 Fatigue strength

The principle of calculating the fatigue strength is the same for both codes. There are however some differences that must be enlightened.

4.4.1 Number of load cycles and capacity

The section that is checked for its fatigue strength has to be designed for a number of load cycles.

In BV BRO the number of cycles that should be used is specified. It is also specified a standardized stress spectra $\kappa = 2/3$. The stress spectra consider the number of times the structure is affected by full loading cycles. This makes the value of the capacity greater according to BSK than EC. The detail category for the section checked is received from BSK. The capacity is received from a table in BSK using the number of cycles, stress spectra and detail category.

In EC it is up to the designer to know how many loading cycles that are going to affect the structure. The detail category for the section checked is received from EC in the same way as in BSK. The capacity is calculated using the number of cycles and the detail category according to equation (4.21). (prEN 1993-1-9:2002,[13])

$$\Delta\sigma_C = C \cdot \left[\frac{2 \cdot 10^6}{N_R}\right]^{\frac{1}{m}} \quad \text{For normal stress} \qquad (4.21)$$

C Detail category

N_R Number of loading cycles

m 3 or 5, depending on number of loading cycles

Where, $\Delta\sigma_C$ is the design capacity for normal stress of the checked section. The design capacity for shear $\Delta\tau_C$ is calculated according equation (4.22).

$$\Delta\tau_C = \frac{\Delta\sigma_C}{\sqrt{3}} \quad \text{For shear stress} \qquad (4.22)$$

4.4.2 Stress capacity of welds

The stresses are calculated in the same way in both codes, see section 3.4.

In Figure 4.5 the stresses considered according to BSK is shown.

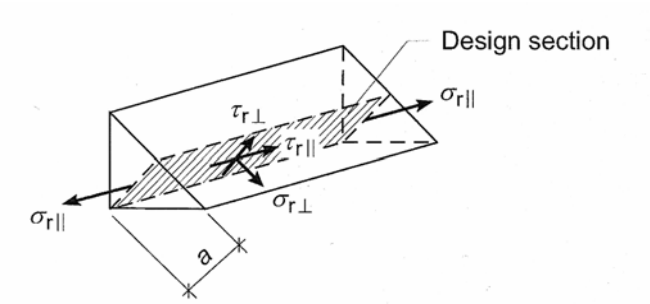

Figure 4.5 Relevant stresses in welds according to BSK 99

In BSK the design conditions are

$\sigma_{rd} \leq f_{rd}$ For normal stress

$\tau_{rd} \leq f_{rvd}$ For shear stress

Where σ_{rd} and τ_{rd} are the stress response in the checked section. The design values of the fatigue strength, f_{rd} and f_{rvd} are calculated as shown in equation (4.23) and (4.24).

$$f_{rd} = \frac{f_{rk}}{1.1 \cdot \gamma_n}$$ Design value for normal stress (4.23)

$$f_{rvd} = 0.6 \cdot f_{rd}$$ Design value for shear stress (4.24)

In BSK the value of the partial safety factor γ_n for fatigue strength is 1.2 for all bridges.

The stresses are checked individually and in case of combined stresses with an interaction formula, see equation (4.25).

$$\sqrt{\frac{\sigma_{rd\parallel}^{2}}{f_{rd\parallel}^{2}} + \frac{\sigma_{rd\perp}^{2}}{f_{rd\perp}^{2}} + \frac{\tau_{rd\parallel}^{2}}{f_{rvd}^{2}} + \frac{\tau_{rd\perp}^{2}}{f_{rvd}^{2}}} \leq 1.10 \qquad (4.25)$$

In EC, the normal stress $\sigma_{\parallel}$ parallel to the longitudinal direction of the weld, is not considered, see Figure 4.6. The stresses $\sigma_{\perp}$ and $\tau_{\perp}$ perpendicular to the weld are combined according to equation (4.26). The only stress checked, that is parallel to the direction of the weld, is $\tau_{\parallel}$, see equation (4.27).

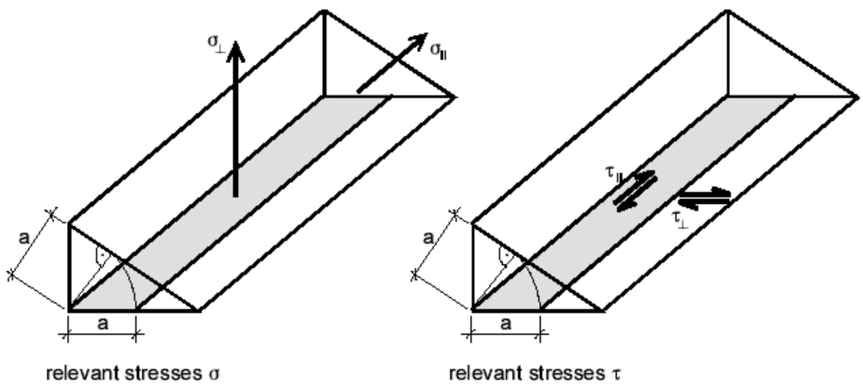

Figure 4.6 Relevant stresses in welds according to EC

$$\Delta\sigma_E = \sqrt{\sigma_{\perp}^2 + \tau_{\perp}^2} \qquad \text{Normal stress} \qquad (4.26)$$

$$\Delta\tau_E = \tau_{\parallel} \qquad \text{Shear stress} \qquad (4.27)$$

The stresses $\Delta\sigma_E$ and $\Delta\tau_E$ are verified using the condition in equation (4.28) and (4.29).

$$\frac{\gamma_{Ff} \cdot \Delta\sigma_E}{\Delta\sigma_C / \gamma_{Mf}} \leq 1.0 \qquad \text{For normal stress} \qquad (4.28)$$

$$\frac{\gamma_{Ff} \cdot \Delta\tau_E}{\Delta\tau_C / \gamma_{Mf}} \leq 1.0 \qquad \text{For shear stress} \qquad (4.29)$$

Where: γ_{Ff} Partial safety factor for fatigue loading.

γ_{Mf} Partial safety factor for fatigue strength.

$\Delta\sigma_C$ and $\Delta\tau_C$ Capacity according to section 4.4.1.

According to EC the designer has the opportunity to choose the partial safety factor for fatigue γ_{Mf}. Depending on the consequence of failure and the safety concept, one of the values in Table 4.3 can be chosen.

Table 4.3 Partial safety factor for fatigue strength γ_{Mf} according to EC

Safety concept	**Consequence of failure**	
	Low consequence	High consequence
Damage tolerant concept	1.00	1.15
Safe life concept	1.15	1.35

In case of combined stresses it shall be verified that the condition given in equation (4.30) is fulfilled. [13]

$$\left(\frac{\gamma_{Ff}\cdot\Delta\sigma_E}{\Delta\sigma_C/\gamma_{Mf}}\right)^3+\left(\frac{\gamma_{Ff}\cdot\Delta\tau_E}{\Delta\tau_C/\gamma_{Mf}}\right)^5\leq 1.0 \qquad (4.30)$$

This condition is valid unless otherwise are stated in the detail category.

5 Results and conclusions

In this section the effect of design load is compared with the design resistance for BSK and EC. Utilization factors, are presented for every check in the calculations, see Table 5.1. Each case can be found in Appendix A and B.

E Effect of design load

R Design value of resistance

U=E/R Utilization factor

Table 5.1 Effect of design load, design value of resistance and utilization factors for calculation according to BSK and EC

Case	**Appendix A, B**			**BSK**			**EC**	
	Chapter	Unit	E_{BSK}	R_{BSK}	U_{BSK}	E_{EC}	R_{EC}	U_{EC}
ULS								
Whole cross-section								
Stresses in top flange	2.1.6	MPa	120	288	0,42	126	355	0,36
Stresses in bottom flange	2.1.6	MPa	161	288	0,56	170	355	0,48
Longitudinal stiffener								
Average stress	2.2.1	MPa	112	288	0,39	118	355	0,33
Buckling resistance	2.2.2	MPa	112	247	0,45	118	309	0,38
Stresses of local effect upper	2.2.5	MPa	20	288	0,07	30	355	0,08
Stresses of local effect lower	2.2.5	MPa	106	288	0,37	160	355	0,45
Lateral buckling	2,2.6	kN	114	259	0,44	120	333	0,36
Plate								
Stresses	2.3	MPa	140	288	0,49	156	355	0,44
Web								
Shear buckling	2.4	MN	1,38	1,61	0,86	1,45	2,00	0,72
Welds								
Butt weld	2.5.1	MPa	94	184	0,51	98	262	0,38
Fillet weld	2.5.2	MPa	108	184	0,59	114	262	0,43
SLS								
Vertical displacement	2.6	mm	22,4	22,5	0,996	20,0	30,0	0,67
FATIGUE								
Butt weld								
Stresses due to axle pressure	2.7.1.1	MPa	37	50	0,73	24	31	0,79
Stresses due to bending	2.7.1.2	MPa	59	113	0,52	not considered		
Stresses due to shear	2.7.1.3	MPa	49	68	0,72	31	48	0,65
Compilation Max	2.7.1.4		1,15	1,10	1,05	0,61	1,00	0,61
Compilation in section	2.7.1.4		0,93	1,10	0,85			
Fillet weld								
Stresses due to bending	2.7.2.1	MPa	84	80	1,05	54	59	0,91
Stresses due to shear	2.7.2.2	MPa	59	48	1,22	37	34	1,09
Holes in upper flange								
Stresses due to bending	2.7.3	MPa	61	90	0,67	39	78	0,50

In Table 5.2 the ratios between the different codes concerning the effects of design load, design value of resistance and utilization is presented.

Table 5.2 Ratios between the different codes concerning the effects of design load, design value of resistance and utilization

Case	Appendix A, B	Ratio BSK / EC		
	Chapter	E_{BSK}/E_{EC}	R_{BSK}/R_{EC}	U_{BSK}/U_{EC}
ULS				
Whole cross-section				
Stresses in top flange	2.1.6	0,95	0,81	1,17
Stresses in bottom flange	2.1.6	0,95	0,81	1,17
Longitudinal stiffener				
Average stress	2.2.1	0,95	0,81	1,17
Buckling resistance	2.2.2	0,95	0,80	1,19
Stresses of local effect upper	2.2.5	0,68	0,81	0,84
Stresses of local effect lower	2.2.5	0,66	0,81	0,82
Lateral buckling	2,2.6	0,95	0,78	1,22
Plate				
Stresses	2.3	0,90	0,81	1,11
Web				
Shear buckling	2.4	0,95	0,80	1,18
Welds				
Butt weld	2.5.1	0,95	0,70	1,36
Fillet weld	2.5.2	0,95	0,70	1,36
SLS				
Vertical displacement	2.6	1,12	0,75	1,49
FATIGUE				
Butt weld				
Stresses due to axle pressure	2.7.1.1	1,52	1,63	0,93
Stresses due to bending	2.7.1.2			
Stresses due to shear	2.7.1.3	1,57	1,41	1,12
Compilation Max	2.7.1.4	1,89	1,10	1,71
Compilation in section	2.7.1.4			
Fillet weld				
Stresses due to bending	2.7.2.1	1,57	1,35	1,16
Stresses due to shear	2.7.2.2	1,58	1,40	1,12
Holes in upper flange				
Stresses due to bending	2.7.3	1,57	1,15	1,36

All calculation principles are the same for both codes. There are no major differences in how the checks are performed. However, the differences occur when the design loads and the design resistance are determined.

In ULS, the values of utilization factors are slightly higher in BSK than in EC, except when looking at local effects (highlighted values). The reasons for higher utilization factors in BSK are explained by two factors. The design loads applying EC result in greater effects and the design resistance is higher than when applying BSK. Concerning the design loads, they are multiplied by load factors and combination factors which together result in larger effect, in comparison to BSK. The partial safety factor γ_M is set to 1.0 according to NAD(S)/SS-ENV 1993-2 and the characteristic yield strength f_y used in EC has the value of 355 MPa for thickness up to 40 mm. BSK uses the value of 1.2 for the partial safety factor γ_n, regarding all bridges. BSK also provide the value of 345 MPa for the characteristic yield strength f_{yk} for thickness between 16-40 mm and 355 MPa for 0-16 mm. However, notice that when checking the shear buckling, the same yield strength is used; it is only the partial safety factor that differs.

The local effects in the longitudinal stiffeners, see Table 5.1, BSK has smaller utilization factors than EC. The greater design loads in EC explain this. It is the α value applied on the nosing force, the larger dynamic factor, load factors and combination factors applied to the axle force, which result in the higher utilization factor for EC.

When it comes to welds, in ULS, the extra factor of 1.2 is applied on the shear capacity in BSK, which result in large difference. Also the fact that the factor of 0.9 is applied differently according to the codes increases the difference.

In SLS, the value of the utilization factor is almost 1 for BV BRO, while it is only 0.67 for EC. In EC the value of the design load for vertical displacement is given a smaller value than BV BRO. In EC, also the design limit has a higher value (L/600) than the one provided by BV BRO (L/800).

When the bridge is checked for its fatigue strength there are a few differences that will be enlightened. Some of the utilization factors in the calculation with BSK exceed 1. This is, when the upgrade from BV BRO, edition 4 to BV BRO, edition 7 was made. The number of loading cycles was increased from $1 \cdot 10^6$ to $2 \cdot 10^6$ this reduces the value of the design resistance, there will be no conclusions made on this fact.

The fatigue load recommended by EC is approximately 2/3 of the one given by BV BRO. When taking out the capacity of the checked section with BSK, a stress spectra is given by BV BRO. The stress spectra κ considers the number of loading cycles, the full load affects the structure. This makes the value of the capacity greater according to BSK than EC.

The major difference occurs when checking the butt weld between the upper flange and the web. In EC the parallel stresses $\sigma_{\parallel}$ in the longitudinal direction of a weld is not considered in fatigue. Another thing is that in EC the shear stresses $\sigma_{\perp}$ and $\tau_{\perp}$ are combined before the check is made. In BSK all stresses are checked individually. Finally there is a check in case of combined stress. These two formulas differ a great deal. The one recommended by BSK is more restrictive than the one in EC. This is clearly shown in the results; where BSK result in a utilization factor of 1.05 for the case with maximum stresses while EC gives a value of 0.61 for the same case, see Table 5.1.

When checking the fillet weld between the vertical stiffener and the web for its fatigue strength, regarding bending stresses. The utilization factor in calculation with BSK results in 1.05, while EC results in 0.91, see Table 5.1. The capacity provided by BSK is higher than the one given by EC. However, the load according to BV BRO is also much higher than the load provided by EC. When considering shear stresses the utilization factor exceeds 1 in both cases, this depends on the increased number of load cycles.

When checking the fatigue strength of the holes in the upper flange, the difference in utilization factor depends on the partial safety factor. In BSK the same factor is used in all checks, but in EC a smaller value can be chosen if a failure in the checked section do not affect the load carrying capacity of the structure. The holes are such a case, and the partial safety factor γ_{Mf} is chosen to 1.15, instead of 1.35, which is valid for the weld cases.

An interesting remark is that in ULS, the values in EC result in higher design loads and higher design capacity, than BV BRO and BSK. For fatigue it is vice versa.

6 General discussion and proposals for improvement

While working with the EC document, there have been many obstacles. One is that it contains so many parts referring to each other. As a result, it is sometimes difficult to find the necessary information. Almost every time, the documents refer to another part, and this part can refer to a third part and so on. This can be very frustrating, and the information needed can be difficult to interpret. This is due to many nations and people processing the EC document. A great deal of suggestions and ideas have to be considered and the code must be suitable for all nations within the EU and EFTA.

One example in EC is when the design train load shall be calculated, there is a list of values for α and a reference to the national application document, and by this document the α value can be chosen. A list of α values for each country and for different design situations could easily be presented in the EC document, which makes those kinds of references unnecessary.

When this Master's Thesis began, there was no guidance, of which EC documents that should be used. The only list of EC documents presented, was the general list, see section 1.4. There is no significant information about their content. A good thing to do would be to create a database, which can give the designer a list of the documents that will be used for a particular project. This would save a lot of time, since gathering the necessary documents takes time. Another idea to get EC more reasonable to work with would be to gather all information concerning a defined case in one document. For example gather all actions and loads on bridges in one document.

In BRO 2004, BV BRO and BSK 99 the regulations are clearly stated. This fact makes the documents very easy to follow. In EC the designer is given a lot more freedom, and the different parts often recommend the designer to design for the particular project. For an experienced designer this may only be a minor problem, however, a less experienced designer will have more difficulties using EC. However, in this particular case, it also has to be considered that a fair comparison was made, which limited the choices.

More freedom to the designer also results in greater responsibility for the client to state more precisely the actions and loads, which the object should be designed for. One example in this case is the fact that the bridge is located between two other bridges and sheltered from the wind load. In such a case the designer according to EC, could be able to disregard the wind load on the bridge. However, if one of the road bridges is removed, it will be exposed to wind load. This type of information has to be provided by the client. According to Swedish codes, the designer is only able to ignore well-documented loads i.e. the designer can prove that the load will not occur. For example if the bridge is straight, the centrifugal force is neglected. If piles on bedrock are used for the foundation, settlements are not possible, and support displacement loads are not necessary.

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Appendix A

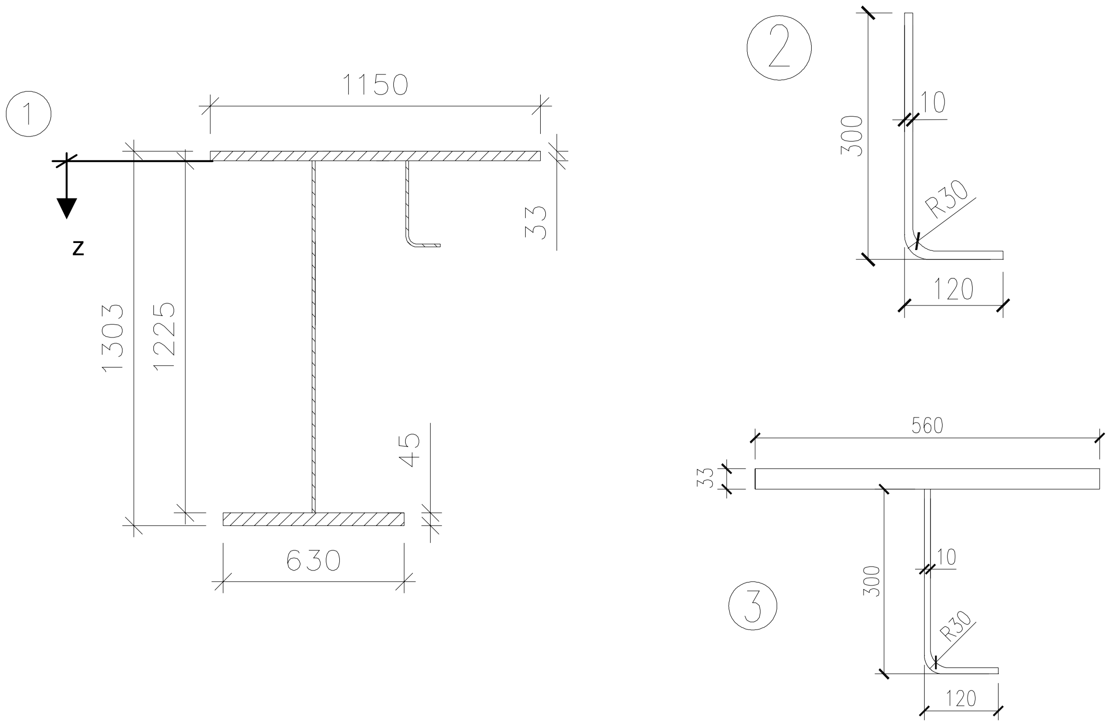
Calculation according to Swedish codes

Contents

Appendix A

1. Loads and actions on the bridge
According to Bro 2004 and BV BRO edition 7

The main beam is calculated as simply supported with half the bridge contributing



1.1 Cross-sectional constants

Table 1.1 Cross-sectional constants

Section nr	Section name	Web		Upper flange		Lower flange		Centre of gravity
		t	h	t	b	t	b	z
		mm	mm	mm	mm	mm	mm	mm
1	Main beam + stiff	12	1225	33	1150	45	630	524
2	Stiffener	10	290	0	0	10	105	185
3	Stiff + upper flange	10	290	33	560	10	105	19

Section nr	Section name		Area	Weight	Ix	W_{upper}	W_{lower}
			m^2	kg	m^4	m^3	m^3
1	Main beam + stiff		0,085	680	0,028	0,05099	0,03802
2	Stiffener		0,004	32	4E-05	0,0002	0,00033
3	Stiff + upper flange		0,0224	179	2E-04	0,0033	0,00061

Distance from the centre of gravity to lower edge of the beam 1225+45-504=766 mm
The shear centre is located 300mm over the top of the rail.

1.2 Permanent loads

Loads are calculated on half the bridge

1.2.1 Self-weight

Main beam in weight/m				680 kg/m
	Density, steel	77 kN/m^3		
For load combination IV load factor = 1.05				
	Load factor	1,05		
	Gravity constant	10 m/s^2		**7,14 kN/m**
	Cross beams		1.4*44.4/6	10,36
	Stiffener		15*8*0.28*1.225/3	13,72
	Stiffener		15*8*0.2*1.225/6	4,9
	Cantilever		10*8*0.45*1.8/6	10,8
	UPE 180		19.7*3	59,1
	Iron bars walkway		35*1.4	49
	Railing		60*2	120
	Screws and nuts		2*30*8*0.18*0.46/0.6	66,24
	Cables			50
				384 kg/m
			Load comb IV* 1.05 =	**4,03 kN/m**

The values of the self-weight are calculated for half the bridge

1.3 Variable actions

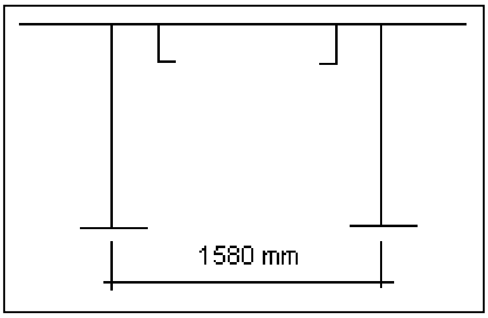
1.3.1 Traffic load

The bridge is designed according to train load BV 2000 (BV 21.2211)
Load regulations for railway bridges according to BV BRO edition 7

According to the technical report.
In design of the main beams, it is assumed that the rail can be displaced by maximum 20mm.

e = 20 mm

Distance between the webs = 1,58 m



Increase of load, caused by rail displacement F = (1.58/2+0.02)/1.58 **F = 0,513**
(If the load is displaced in the transverse direction, the load will slightly increase on one beam while it decreases on the other beam)

The bridge is placed in a straight line with a span length of L = 18 m

According to BV 21.2216
Dynamic contribution, D = 1+4/(8+L_{best}), where L_{best} = 18m **D = 1,154**

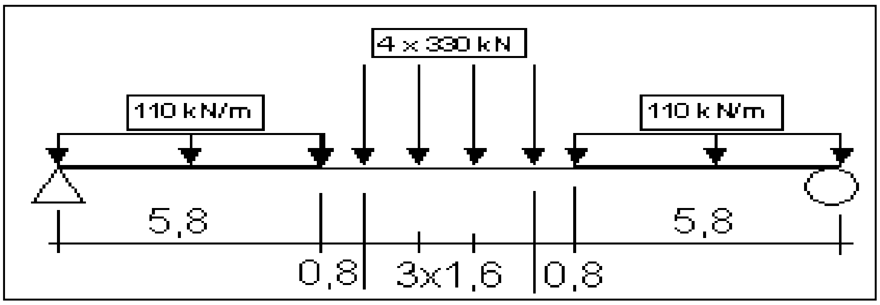
Load factor (LF), Load combination IV = **1,4** BV 2000

Design load factor in load-combination IV, (ULS), including rail displacement factor, dynamic factor and load factor

Load combination IV **F_{dim} = F*D*LF = 0,8281** BV 2000

Train load BV 2000

The traffic load consists of two uniformly distributed loads (110 kN/m) and four axis loads (330 kN)



P = 330 kN 4 pcs
q = 110 kN/m

Design load in load combination IV, ULS

$P_{dim} = P*F_{dim} = 330*F_{dim}$ **P_{dim} = 273,3 kN**
$q_{dim} = q*F_{dim} = 110*F_{dim}$ **q_{dim} = 91,1 kN/m**

1.3.2 Derailment load

According to BV BRO 21.36 it assumes that derailment load BV 2000 shall be displaced in the transverse direction, that in this case is defined by the derailment protection to 300 mm.

This is an accidental load case with load factor = 0,8
which gives the design force:

P_{dim} = 330*0.8*(1580/2+300)/1580*D P_{dim} = 152,3077 kN

The derailment load will not be the design force for the main beam,

1.3.3 Nosing force

According to BV BRO, 21.2222, the bridge shall be designed for a single nosing force which is acting at the top of the rail.

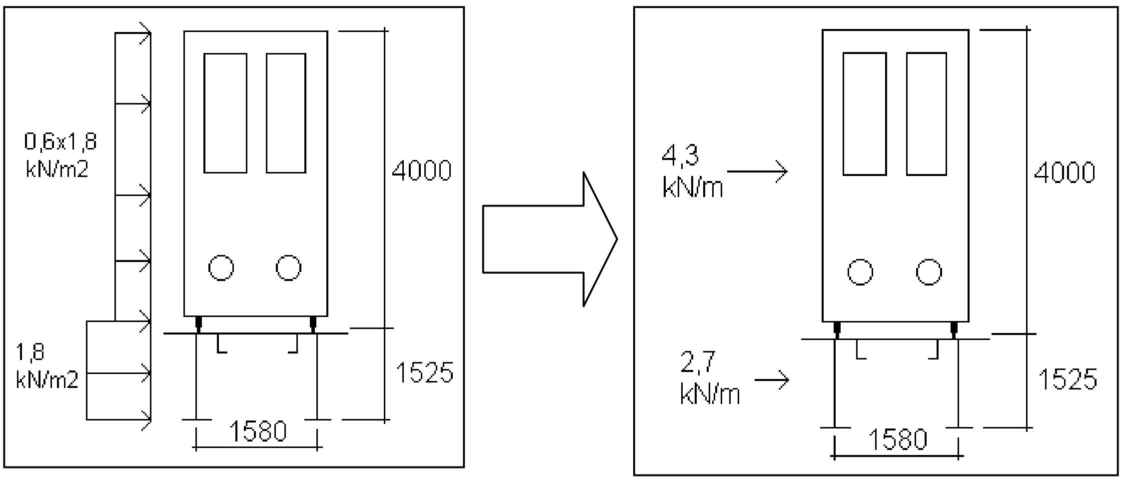
P_{nosing} = 100 kN

Due to the fact that the shear centre is located slightly above the upper edge of the rail, a load in the transverse direction will give a small contribution to the vertical load.
This load will not be used for design of the main beam, but will be used for design of local parts

1.3.4 Wind load

According to BRO 2004, 21.272   **Characteristic wind load   1,8 kN/m^2**
The pressure on the train is 60% of the characteristic value

The load is transformed from an area based value to a distributed load acting in the longitudinal direction of the bridge.



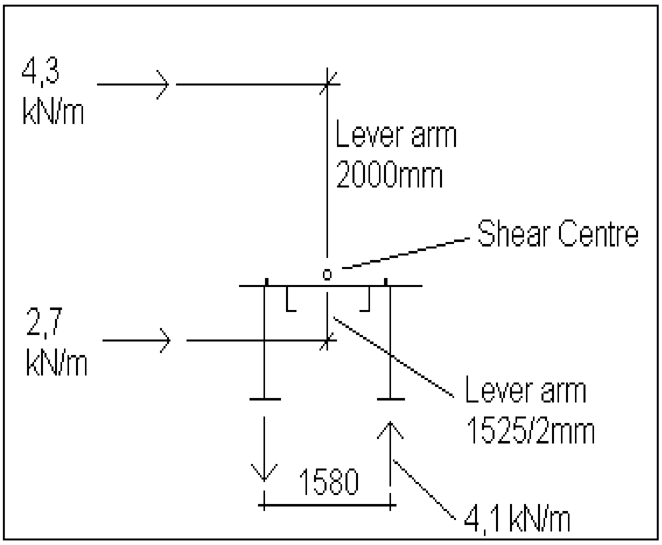
height of bridge	1,525 m	(From bottom flange to top of rail)		
height of train	4 m	reduction factor =	0,6	(Wind load on train)

Distributed loads in longitudinal direction

Characteristic load on the bridge	1.525*1,8 =	**2,7 kN/m**
Characteristic load on the train	4*1.8*0.6 =	**4,3 kN/m**

1.3.4.1 Vertical load increase due to wind.

Calculation on the safe side. Assume the shear centre is located in line with the top edge of the rail.



Moment calculation around shear centre results in:

(4.32*2-2.745*1.525/2)/1.58 = 4,1 kN/m

In load combination IV with trainload, this is calculated with a load factor = 0,6

Design vertical load due to wind, in load combination IV, ULS

$q_{wind,dim}$ = 0,6*4,1436 = **2,5 kN/m**

1.3.5 Horizontal force

Brake- and acceleration force

The acceleration force give a greater value than the braking force.

P_{dim}=	30 kN/m	(Acceleration force)
L	18 m	(Span length)

Design force

P_{max} = 18*30 540 kN

1.4 Fatigue load

In BV BRO edition 7, Table BV 22-1 load combination VI is used for receiving the fatigue load

Load factor for train load BV 2000	0,8
Load factor for wind load	0,6

Design load factor in load-combination VI (fatigue), including rail displacement factor, dynamic factor and load factor

F_{rdim} = F*D*LFr = **0,4732** BV 2000

Design loads for fatigue

Train axle loads	330*F_{rdim} =	**156,2**	**kN**
Uniformly distributed load	110*F_{rdim} =	**52,1**	**kN/m**
Vertical load due to wind	2.5*0.6 =	**1,5**	**kN/m**

2 Design of the main beam, BSK 99

Documents used in the design: BSK 99
K 18
BV BRO, edition 7
BRO 2004

2.1 Stresses over the cross-section, ULS

The first check is the stresses over the cross-section, the bridge is simply supported, so the largest response will appear in the middle of the beam. The bending moment will be calculated for each load, and then added together.

The formula of the bending moment is $M := q \cdot \frac{L^2}{8}$ where L = 18 m

All the actions in kN/m are from chapter 1.

2.1.1 Self-weight

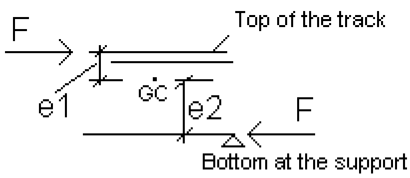
Self-weight $M_{self} := \frac{18^2}{8} \cdot (7.14 + 4.03)$ $M_{self} = 452.385$ kNm

2.1.2 Wind load

Wind load $M_{wind} := \frac{18^2}{8} \cdot 2.48$ $M_{wind} = 100.44$ kNm

2.1.3 Brake- and acceleration force

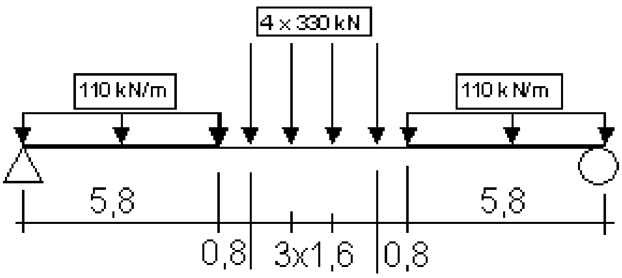
The brake- and acceleration force is acting in top of the track. The horizontal reaction force at the support acts in the bottom of the beam cross-section. The center of gravity is located closer to the top, therefore, the total moment of this force is negative, which is a favorable response, and is not added to the calculations.



e1 < e2 => Negative moment

2.1.4 Train load

The design train load is BV 2000:



Moment in mid span

Point loads $M_p := 330 \cdot (2 \cdot 6.6 + 1.6)$ $M_p = 4884$ kNm

Uniform $M_u := 110 \cdot \frac{5.8^2}{2}$ $M_u = 1850.2$ kNm

Total moment in mid span $M_{train} := 0.8281 \cdot (M_p + M_u)$

Fdim

$M_{train} = 5576.591$ kNm

2.1.5 The total moment

$M_{tot1} := M_{self} + M_{wind} + M_{train}$ $M_{tot1} = 6129.416$ kNm

$M_{tot} := \frac{M_{tot1}}{1000}$ $M_{tot} = 6.129$ MNm

2.1.6 Stresses over the cross-section

f_{yk} from BSK 99, Table 2:21b in accordance with the technical report.

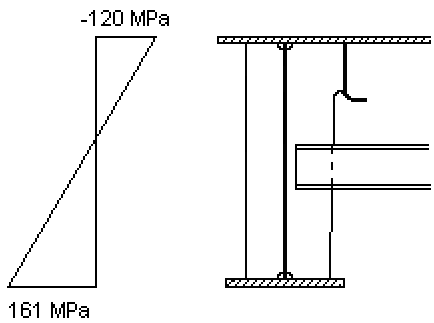
$f_{yk} := 345$ MPa $\gamma_n := 1.2$

$f_{yd} := \frac{f_{yk}}{\gamma_n}$ $f_{yd} = 287.5$ MPa

$W_{u1} := 0.05099$ m^3

$W_{l1} := 0.03802$ m^3

(Table 1.1 cross-sectional constants)



This give the following stresses.

$\sigma_{tf} := \frac{M_{tot}}{-W_{u1}}$ $\sigma_{tf} = -120.208$ MPa

$\sigma_{bf} := \frac{M_{tot}}{W_{l1}}$ $\sigma_{bf} = 161.216$ MPa

$|\sigma_{tf}| = 120.208$ MPa $<$ $f_{yd} = 287.5$ MPa OK! (Compression)

$\sigma_{bf} = 161.216$ MPa $<$ $f_{yd} = 287.5$ MPa OK! (Tension)

2.2 The longitudinal stiffeners with the effective flange, ULS

First the average stress over the cross-section is checked. Secondly the flexural buckling is confirmed. Then the stresses due to eccentricity of the nosing- and axle force are checked. Finally the flange of the longitudinal stiffener is verified for lateral buckling.

2.2.1 Average stress in longitudinal stiffener with effective flange

top flange $560 \cdot 33 \quad mm^2$ (Table 1.1 cross-sectional constants)

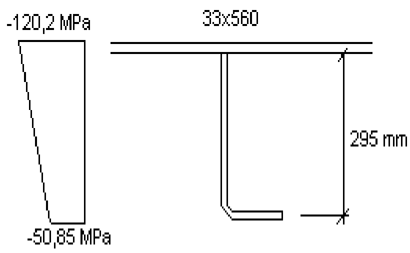
$\sigma_{tf} = -120.208$ MPa (compression)

The stress is linear over the cross section,

$$1303mm - \frac{45mm}{2} - \frac{33mm}{2} = 1.264m$$ (Distance between center of flanges)

$$\sigma_{ratio} := \frac{-(\sigma_{tf} - \sigma_{bf})}{1.264} \qquad \sigma_{ratio} = 222.645$$ MPa / meter

Stress in bottom flange of stiffener $$\sigma_{bs} := \sigma_{tf} + \left(\frac{0.033}{2} + 0.295\right) \cdot \sigma_{ratio}$$



$\sigma_{bs} = -50.854$ MPa Compression

Average stress in the longitudinal stiffener

$$\sigma_{average} := \frac{\sigma_{tf} + \sigma_{bs}}{2}$$

$\sigma_{average} = -85.531$ MPa

Total compression force in longitudinal stiffener

$$P_{tot} := 33 \cdot 560 \cdot \sigma_{tf} + 285 \cdot 10 \cdot \sigma_{average} + 110 \cdot 10 \cdot \sigma_{bs}$$

$P_{tot} = -2521150.937$ N

Average stress in the local cross-section

$$\sigma_{Sd} := \frac{P_{tot}}{(33 \cdot 560 + 10 \cdot 395)} \qquad f_{yk} := 345\,MPa \qquad \gamma_n := 1.2$$

$\sigma_{Sd} = -112.401$ MPa $\qquad f_{yd} := \dfrac{f_{yk}}{\gamma_n} \qquad f_{yd} = 287.5$ MPa

$|\sigma_{Sd}| = 112.401$ MPa $<$ $f_{yd} = 287.5$ MPa OK!

2.2.2 Buckling resistance of members

The longitudinal stiffeners are stiffened every third meter by the vertical stiffeners.

$f_{yk} := 345\ \text{MPa}$ $E_k := 210000\ \ \text{MPa}$ $\gamma_n := 1.2$

$f_{yd} := \frac{f_{yk}}{\gamma_n}$ $f_{yd} = 287.5\ \ \text{MPa}$ $L_c := 3\ \ \text{m}$

Stress capacity

$\sigma_{Rd} := \omega_c \cdot f_{yd}$

Actual stress

$\sigma_{Sd} = -112.401$ MPa

$I_3 := 0.00017$ $A_3 := 0.0224$ (Table 1.1 cross-sectional constants)

$$i := \left(\frac{I_3}{A_3}\right)^{0.5} \qquad i = 0.087$$

$$\lambda_c := \frac{L_c}{\pi \cdot i} \cdot \sqrt{\frac{f_{yk}}{E_k}} \qquad \lambda_c = 0.444$$ (BSK 99, eq 6:233a)

$\omega_c := 0.86$ Buckling curve C (BSK 99, Table 6:233)

Allowable Stress:

$\sigma_{Rd} := \omega_c \cdot f_{yd}$ $\sigma_{Rd} = 247.25$ Mpa

$\left|\sigma_{Sd}\right| = 112.401$ MPa $<$ $\sigma_{Rd} = 247.25$ Mpa OK!

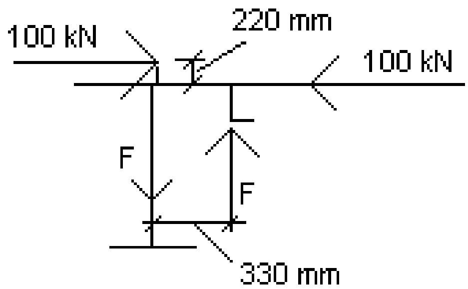
2.2.3 Forces caused by eccentricity of the nosing force

The nosing force acts at the top edge of the rail, i.e. 220 mm over the upper flange. This will give a moment that is taken by two forces, one in the web and one in the longitudinal stiffener.

$F_{nosing} := 100\ \ \text{kN}$

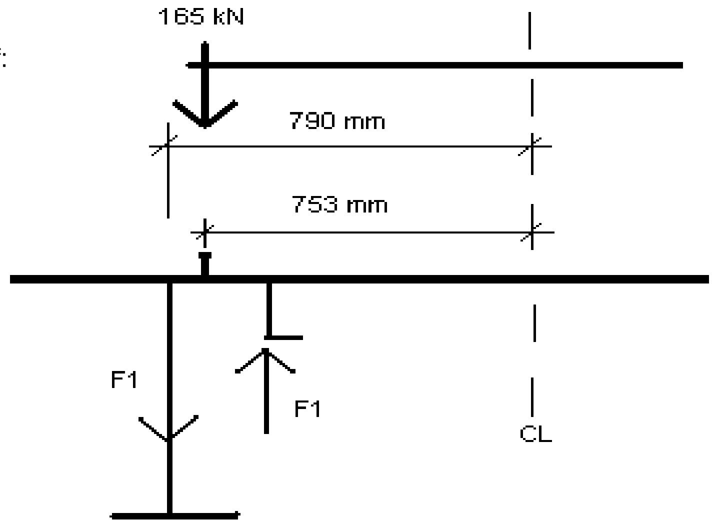
Load factor 1,4

$$F := \frac{\left(F_{nosing} \cdot 220\right)}{330} \cdot 1.4 \qquad F = 93.333\ \text{kN}$$



2.2.4 Forces caused by eccentricity of the axle force

The rail is located 753 mm from centre line (CL) of the bridge and the webs are located 790 mm from CL. This will give a moment that is taken by two forces, one in the web and one in the longitudinal stiffener. Also add the displacement eccentricity of 20 mm



This gives the eccentricity of:

$790 - 753 + 20 = 57$ mm

Calculate for one axle force.

$\frac{330}{2} = 165$ kN

Dynamic factor

$L_{bestloc} := 4.5$ (Table BV 21.2216a in BV BRO)

$D := 1 + \frac{4}{(8 + L_{bestloc})}$ $D = 1.32$

Load factor = 0.7

Calculating the bending moment

$M := 0.057 \cdot 165 \cdot 0.7 \cdot 1.32$ $M = 8.69$ kNm

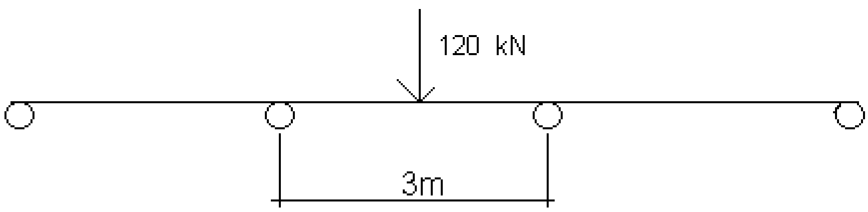
This gives a vertical force of

$F1 := \frac{M}{0.33}$ $F1 = 26.334$ kN

2.2.5 Stresses in longitudinal stiffener with effective flange, caused by axle- and nosing force

Calculate the longitudinal stiffener as a continuous beam with support every third meter

The total force $F_{tot} := F + F1$ $F_{tot} = 119.667$ kN



Calculation of the moment

Table value on safe side
Elementary case

$M := F_{tot} \cdot 3 \cdot 0.18$ $\quad$ $M = 64.62$ kNm

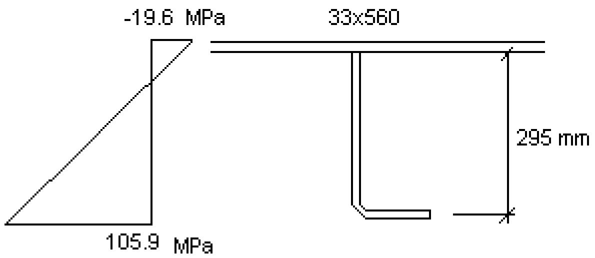
M = Ftot*L*0.18

Stresses in the longitudinal stiffener

$W_{l3} := 0.00061 m^3$ $\quad$ $W_{u3} := 0.0033 m^3$ $\quad$ (Table 1.1 cross-sectional constants)

Lower edge $\quad \sigma_{stiff} := \dfrac{M \cdot 10^{-3}}{W_{l3}}$ $\quad \sigma_{stiff} = 105.935$ MPa $\quad$ (tension)

Upper edge $\quad \sigma_{flange} := \dfrac{M \cdot 10^{-3}}{-W_{u3}}$ $\quad \sigma_{flange} = -19.582$ MPa $\quad$ (compression)



$|\sigma_{flange}| = 19.582$ MPa $\quad < \quad f_{yd} = 287.5$ MPa $\quad$ OK!

$\sigma_{stiff} = 105.935$ MPa $\quad < \quad f_{yd} = 287.5$ MPa $\quad$ OK!

2.2.6 Lateral buckling in the flange of the longitudinal stiffener

The flange of the longitudinal stiffener will be checked with one third of the stiffener's web contributing. The axle- and nosing force will not be included, because they give tension stresses in the flange, section 6:23 in BSK 99.

$N_{Rcd} := \omega_c \cdot A \cdot f_{yd}$

$\sigma_{bs} = -50.854$ MPa

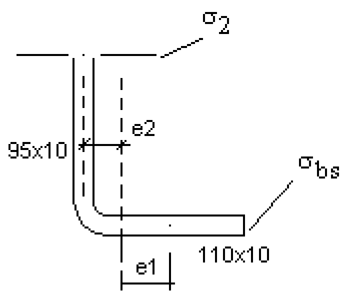
$\sigma_{tf} = -120.208$ MPa

$\sigma_{ratio} = 222.645$ MPa/m

$\sigma_2 := \sigma_{bs} - \sigma_{ratio} \cdot 0.095$ $\quad$ $\sigma_2 = -72.005$ MPa

$A_2 := 95 \cdot 10 + 110 \cdot 10$ $\quad$ $A_2 = 2050$ mm^2

$e_2 := \dfrac{(110 \cdot 10 \cdot 50)}{2050}$ $\quad$ $e_2 := 26.829$ mm



$e_1 := \frac{110}{2} - e_2 - 5$ $\qquad e_1 = 23.171$ mm

$I_2 := e_2{}^2 \cdot 95 \cdot 10 + e_1{}^2 \cdot 110 \cdot 10 + \frac{\left(10 \cdot 110^3\right)}{12}$ $\qquad I_2 = 2383556.911$ mm^4

$i_2 := \sqrt{\frac{I_2}{A_2}}$ $\qquad i_2 = 34.099$ mm

$Lc := 3000$ mm

$\lambda_c := \frac{Lc}{\pi \cdot i_2} \cdot \sqrt{\frac{f_{yk}}{E_k}}$ $\qquad \lambda_c = 1.135$

$\omega_c := 0.44$ $\qquad$ Buckling curve C

$N_{Rcd} := 0.44 \cdot \frac{345}{1.2} \cdot 2050 \cdot 10^{-3}$ $\qquad N_{Rcd} = 259.325$ kN

Average stress in the lower third of the longitudinal stiffener

$\sigma_{aver} := \frac{\left(\sigma_{bs} + \sigma_2\right)}{2}$ $\qquad \sigma_{aver} = -61.43$ MPa

$N_{Scd} := \left(\sigma_{bs} \cdot 110 \cdot 10 + \sigma_{aver} \cdot 95 \cdot 10\right) \cdot 10^{-3}$ $\qquad N_{Scd} = -114.298$ kN

$\left|N_{Scd}\right| = 114.298$ kN $<$ $N_{Rcd} = 259.325$ kN $\qquad$ OK!

2.3 Buckling of the plate between the longitudinal stiffeners, ULS

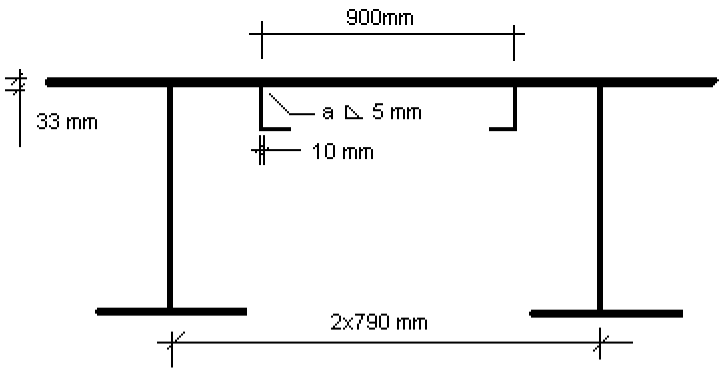
Stresses in the longitudinal plate.

$\sigma_{flange} = -19.582\ \text{MPa}$ Stress from nosing- and axle force

$\sigma_{tf} = -120.208\ \text{MPa}$ Stress from global bending

Total stress $\sigma_{total} := \sigma_{flange} + \sigma_{tf}$

$\sigma_{total} = -139.79$ MPa



The free length of the flange

$E_k := 210000$ MPa

$f_{yk} := 345$ MPa

$2\cdot 790 - 2\cdot 330 - 10 - 2\cdot 5 = 900$ mm $b_f := 900$ mm $t_f := 33$

From Table 6:211a in BSK.

$\beta_{fel} := 1.14\cdot\sqrt{\frac{E_k}{f_{yk}}}$ $\beta_{fel} = 28.126$

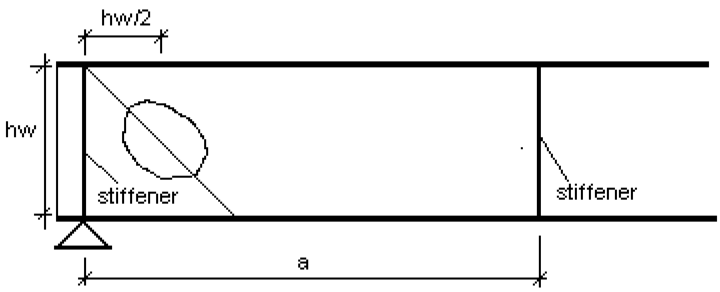
$\beta_f := \frac{b_f}{t_f}$ $\beta_f = 27.273$

$\beta_f < \beta_{fel}$ => Class 2

$|\sigma_{total}| = 139.79$ MPa $<$ $f_{yd} = 287.5$ MPa OK! No risk of buckling.

2.4 Shear buckling of the web, ULS

The magnitude of the shear force is taken from the Matlab calculation. The buckling is checked for the highest shear force at x = hw/2 = 0.6m from the support.
Chapter :26 from K 18 is used.



The Matlab calculation gives at x=0.6 m.

$t_w := 12\text{mm}$ => $f_{.yk} := 355$ MPa

$V_{sd} := 1.378$ MN

$f_{yd} := \frac{355}{1.2}$ $f_{yd} = 295.833$

$h_w := 1.225$ m

$b_w := h_w - 2\cdot\sqrt{2}\cdot 0.005$ m $b_w = 1.211$ m

$t_w := 0.012$ m

$a := 3$ m

Vertical web stiffeners every third meter

K 18:26 gives

$$\kappa_\tau := 5.34 + 4.00\cdot\left(\frac{b_w}{a}\right)^2 \qquad \kappa_\tau = 5.992$$ K18, eq 18:26e

$$\lambda_w := \frac{0.81}{\sqrt{\kappa_\tau}}\cdot\frac{b_w}{t_w}\cdot\sqrt{\frac{f_{yk}}{E_k}} \qquad \lambda_w = 1.353$$ K18, eq 18:26d

For web in fatigue Table K18:26a, (column 3)

$$\omega_v := \frac{0.5}{\lambda_w} \qquad \omega_v = 0.369$$

$$V_{Rd} := \omega_v\cdot h_w\cdot t_w\cdot f_{yd} \qquad V_{Rd} = 1.607 \text{ MN}$$

$V_{sd} := 1.378$ MN < $V_{Rd} = 1.607$ MN OK!

2.5 Welds between the web and the flanges, ULS

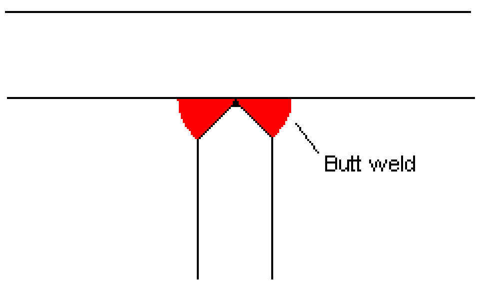
The welds, both between the upper flange and the web, and between the lower flange and the web will be checked for the highest shear force. This is a static analysis in ultimate limit state.
The capacity will be cheched by BSK 99, section 6:3.

2.5.1 Butt welds between web and upper flange

Butt welded connection against upper flange

Calculate weld as if weld has the same strength as the material

Weld class WB



S 355N, $f_{uk} := 490$ MPa $\gamma_n := 1.2$

$\gamma_m := 1.0$

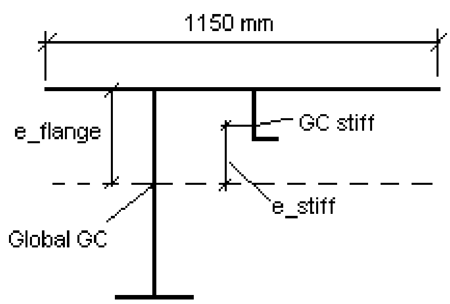
$f_{ud} := \dfrac{f_{uk}}{1.2 \cdot \gamma_m \cdot \gamma_n}$ $f_{ud} = 340.278$ $\phi := 0.9$

$f_{wd} := \phi \cdot f_{ud}$ $f_{wd} = 306.25$ MPa (BSK 99, eq 6:32a)

$f_{R.para} := 0.6 \cdot f_{wd}$ $f_{R.para} = 183.75$ MPa

The largest magnitude of the shear force is over the support.

$V_{max} := 1.458$ MN (train, wind, self)



$A_{flange} := 1.150 \cdot 0.033$ $A_{flange} = 0.038$ m^2

$e_{flange} := 0.524 + \dfrac{0.033}{2}$ $e_{flange} = 0.54$ m

$A_{stiff} := 0.395 \cdot 0.010$ $A_{stiff} = 0.004$ m^2

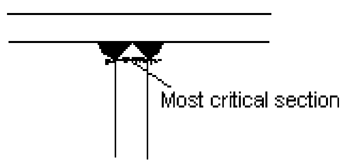
$e_{stiff} := 0.524 - 0.185$ $e_{stiff} = 0.339$ m

$S_1 := A_{flange} \cdot e_{flange} + A_{stiff} \cdot e_{stiff}$ $S_1 = 0.022$ m^3

$I := 0.02838$ m^4

$b := 0.012$ m

The most critical section is the web, since it is most narrow.

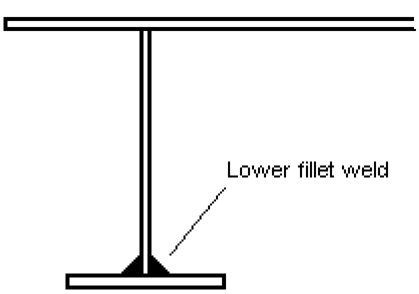


$$\tau_{Sd.para} := \frac{V_{max} \cdot S_1}{I \cdot b} \qquad \tau_{Sd.para} = 93.548 \quad \text{MPa}$$

$$\tau_{Sd.para} = 93.548 \quad \text{MPa} \quad < \quad f_{R.para} = 183.75 \quad \text{MPa} \quad \text{OK!}$$

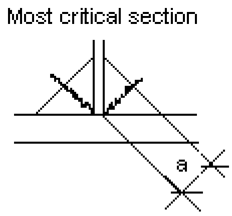
2.5.2 Fillet weld between web and lower flange

$$S_s := 630 \cdot 45 \cdot \left(766 - \frac{45}{2}\right) \qquad S_s = 21078225 \quad \text{mm}^3$$



Thickness of weld, a = 5mm

$b_s := 2 \cdot 0.005 \quad \text{m}$



The highest value of the shear force is recived over the support.

$V_{max} = 1.458 \quad \text{MN}$ (from Matlab calculation)

$$\tau_{S.para} := \frac{V_{max} \cdot S_s \cdot 10^{-9}}{I \cdot b_s} \qquad \tau_{S.para} = 108.288 \quad \text{MPa}$$

$\phi := 0.9$

$$f_{wd1} := \frac{\phi \cdot 490}{1.2 \cdot 1.2} \qquad f_{wd1} = 306.25 \quad \text{MPa}$$

$$f_{R.para} := 0.6 \cdot f_{wd1} \qquad f_{R.para} = 183.75 \quad \text{MPa}$$

$$\tau_{S.para} = 108.288 \quad \text{MPa} \quad < \quad f_{R.para} = 183.75 \quad \text{MPa} \quad \text{OK!}$$

2.6 Max allowed displacement, SLS

According to BV BRO edition 7, section 12.421, the max displacement is limitid to L / 800

For this bridge $L := 18000$

$$\frac{L}{800} = 22.5 \quad \text{mm}$$

This is valid in load combination V:C with $\psi\gamma$ = 1,0

The loads used are: Train load BV 2000, $\psi\gamma$ = 1,0
Wind load $\psi\gamma$ = 0,4

This is checked in the Matlab calculation and gives a displacement of 22,4mm.
This is very close to the allowed value, however, when considering the wind-load it was assumed that the shear center was located in line with the upper edge of the rail, but it is acctualy located 300mm above the upper edge of the rail.
This makes the calculation on the safe side.

2.7 Fatigue

When checking the bridge for fatigue strength the self-weigth is excluded, since the self-weight do not vary. The largest response variation in stresses is considered.

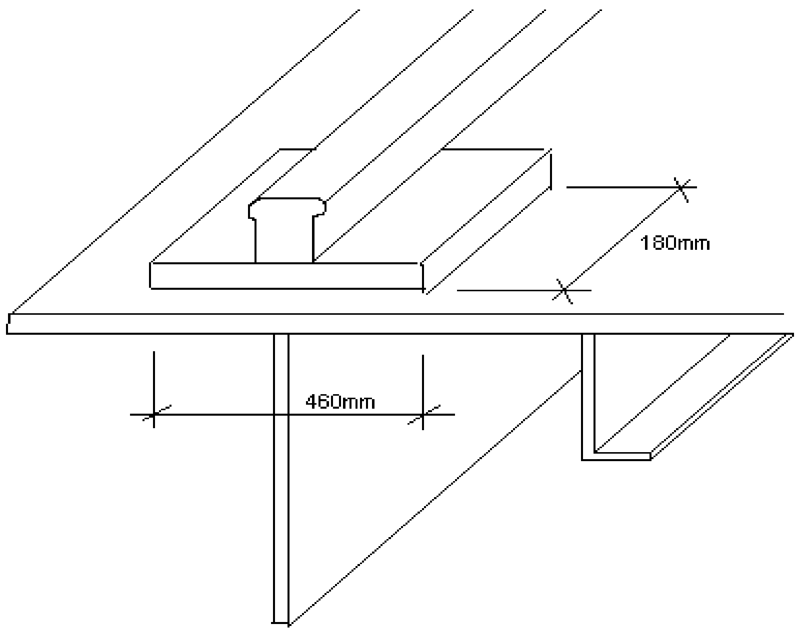
Loads used: Train load BV 2000 $\psi\gamma = 0.8$
Wind load $\psi\gamma = 0.6$

The butt weld between the upper flange and the web and the fillet weld between the web and the vertical stiffener, will be checked for fatigue. Also the holes in the upper flange will be confirmed. The fatigue strength is checked according to section 6:5 in BSK 99.

2.7.1 Fatigue strength of butt weld between upper flange and web

2.7.1.1 Stresses due to axle pressure

When the wheels is acting on the plates 180 mm x 460 mm, which are resting on the upper flange, there will be a change in stress distribution vertically on the butt welds.



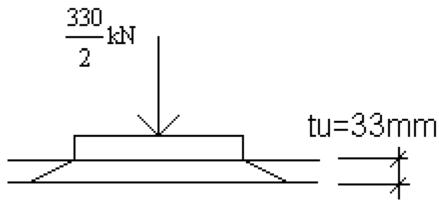
$\kappa := \frac{2}{3}$ $n := 10 \cdot 10^6$ 10 million cycles

Detail 22 in BSK 99 (page 182) gives for weld class WB

C_{perp} = 71 MPa (Perpendicular)

BSK 99, Table 6:524

frd = frk / (1,1x1,2) = $\frac{66.1}{1.1 \cdot 1.2} = 50.076$ MPa $f_{rd.perp} := 50.1$ MPa



Load distribution 1:2,5 gives:

thickness web $t_w := 12$ mm

The compression surface will be tw*(180+2*2,5*33) = 12*345 mm^2

The dynamic respons is a global response, therefore:

$L_{best} := 18$ $D_f := 1 + \frac{4}{(8 + L_{best})}$ $D_f = 1.154$

330 kN represent two weels

Load combination factor

Wheel pressure = $F_p := 330 \cdot 0.5 \cdot D_f \cdot 0.8$ $F_p = 152.308$ kN

Stress width = $\sigma_{rd.perp} := \frac{\left(F_p \cdot 10^3\right)}{12 \cdot 345}$ $\sigma_{rd.perp} = 36.789$ MPa

$\sigma_{rd.perp} = 36.789$ MPa $<$ $f_{rd.perp} := 50.1$ MPa OK!

2.7.1.2 Stress range due to bending

$n_n := 2 \cdot 10^6$ and $\kappa := \frac{2}{3}$ gives $C_{parallel} = 100$ MPa

The characteristic fatigue resistance f_{rk} is calculated by interpolation from table 6:524 in BSK 99.

$f_{rk} := 148.74$ MPa

$f_{rd.para} := \frac{f_{rk}}{1.2 \cdot 1.1}$ $f_{rd.para} = 112.682$ MPa

σrd,para = ΔM / W $_{buttweld}$

$W = \frac{I}{e} = \frac{0.02838}{1.225 + 0.045 - 0.766} = 0.056 \quad m^3$ e = Distance from GC to the weld

From Matlab calculation

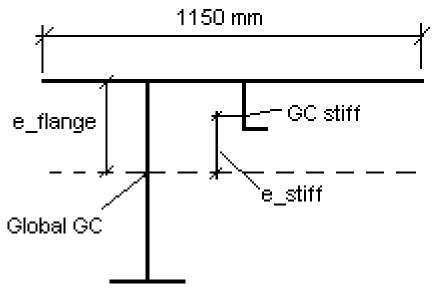
$\Delta M := 3287 + 31 \qquad \Delta M = 3318 \quad kNm$ (max moment variation in mid section)

$\sigma_{Rd.para} := \frac{\Delta M \cdot 10^{-3}}{0.0563} \qquad \sigma_{Rd.para} = 58.934$ MPa

$\sigma_{Rd.para} = 58.934$ MPa < $f_{rd.para} := 112.682$ MPa OK!

2.7.1.3 Stress range due to shear

$\tau_{rd} := \Delta V \cdot \frac{(A_{flange} \cdot e_{flange} + A_{stiff} \cdot e_{stiff})}{I \cdot t_w}$



Over the support we get from Matlab calculation

$\Delta V := 785.3 + 4 \qquad \Delta V = 789.3 \quad kN$

$e_{flange} := 504 + \frac{33}{2} \qquad e_{flange} = 520.5$ mm

$e_{stiff} := 504 - 185 \qquad e_{stiff} = 319 \quad mm$

$A_{flange} := 1150 \cdot 33 \qquad A_{flange} = 38000 \quad mm^2$

$A_{stiff} := 3950 \quad mm^2$

$I = 0.02838 \quad m^4$

$t_w := 12 \quad mm$

$\tau_{Rd.para} := \frac{\left[\Delta V \cdot 10^{-3} \cdot (38000 \cdot 521 + 3950 \cdot 319) \cdot 10^{-9}\right]}{0.02838 \cdot 0.012}$

$\tau_{Rd.para} = 48.805$ MPa

$f_{rvd} := 0.6 \cdot f_{rd.para}$ $f_{rvd} = 67.609$ MPa

$\tau_{Rd.para} = 48.805$ MPa $<$ $f_{rvd} = 67.609$ MPa OK!

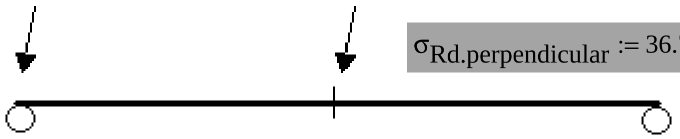
2.7.1.4 Compilation of stresses for the butt weld

The weld will be checked for the worst case, with maximum stress due to bending, maximum stress due to shear and maximum stress due to wheel pressure.

$\tau_{Rd.para} = 48.805$ MPa $\sigma_{Rd.para} = 58.934$ MPa

$\sigma_{Rd.perpendicular} := 36.79$ MPa

(Whole upper flange)



BSK 99, 6:512c

$$\text{UTN} = \sqrt{\frac{\tau_{Rd.para}^{\;2}}{f_{rvd}^{\;2}} + \frac{\sigma_{Rd.para}^{\;2}}{f_{rd.para}^{\;2}} + \frac{\sigma_{Rd.perp}^{\;2}}{f_{rd.perp}^{\;2}}} \le 1.10$$

$$\sqrt{\left(\frac{48.81}{67.609}\right)^2 + \left(\frac{58.93}{112.682}\right)^2 + \left(\frac{36.79}{50.6}\right)^2} = 1.15 < 1.10$$ Not OK!

The fatigue capacity of the weld is not satisfied for this max-max-max case. Therefore some sections will be chosen and checked for their actual stresses in these sections.

Since the number of design loading cycles has increased from 10^6, in BV BRO edition 4, to 2*10^6, in BV BRO edition 7, the fatigue resistance has decreased.

	x	ΔV	$\tau_{Rd.para}$	ΔM	$\sigma_{Rd.para}$	Force	$\sigma_{rd.perp}$	UTN
Max M	3	433,9	26,8	1891,4	33,6	152,3	36,8	0,886
Max V	3	562,7	34,8	1886,6	33,5	152,3	36,8	0,945
Max M	6	317,2	19,6	2988,8	53,1	152,3	36,8	0,919
Max V	6	364,2	22,5	2950,0	52,4	152,3	36,8	0,931
Max M	9	3,5	0,2	3318,4	58,9	152,3	36,8	0,902
Max V	9	138,4	8,6	3177,7	56,4	152,3	36,8	0,898
								OK!

The actual response for the sections are calculated, and the interaction-check is fulfilled

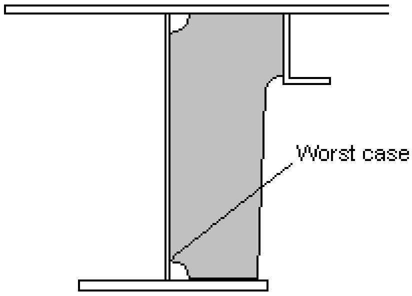
2.7.2 Fatigue strength of fillet weld between web and stiffener

In the lower flange, the fillet weld between the stiffener and the web is the worst case, in bending, detail 44, page 186 BSK 99

Weld class WB C = 71 MPa

2.7.2.1 Stress range due to bending

The characteristic fatigue resistance f_{rk} is calculated by interpolation from table 6:524 in BSK 99.



$n := 2 \cdot 10^6$ $\quad f_{rk} := 105.48 \text{ MPa}$

$\kappa := \frac{2}{3}$

$f_{rd.perpen.} := \frac{f_{rk}}{(1.1 \cdot 1.2)}$ $\quad f_{rd.perpen.} = 79.909$ MPa

$I3 := 0.02838 \quad m^4$

$e3 := 766 - 45 \quad e3 = 721 \quad mm$

e3 is the distance between the weld and the global gravity center.

$W_s := \frac{I3}{e3 \cdot 10^{-3}} \quad W_s = 0.039 \quad m^3$

$\Delta M := 3287 + 31 \quad \Delta M = 3318 \quad kNm$ (max moment variation in mid section)

$\sigma_{rd.perpen.} := \frac{\Delta M \cdot 10^{-3}}{W_s}$

$\sigma_{rd.perpen.} = 84.295$ MPa > $f_{rd.perpen.} = 79.909$ MPa NOT OK!

2.7.2.2 Stress range due to shear

$f_{rvd} := 0.6 \cdot f_{rd.perpen.}$ $\quad f_{rvd} = 47.945$ MPa

$\Delta V = 789.3 \quad kN$ (Max shear force close to the support)

$$\tau_{rd.par} := \frac{\left[\Delta V \cdot 10^{-6} \cdot 630 \cdot 45 \cdot \left(766 - \frac{45}{2}\right) \cdot 10^{-6}\right]}{0.028382 \cdot 0.005}$$

$\tau_{rd.par} = 58.622$ MPa

$\tau_{rd.par} = 58.622$ MPa > $f_{rvd} = 47.945$ MPa NOT OK!

The fillet weld between the stiffener and the web does not make the requirements for shear capacity and bending set out by BSK.

2.7.3 Fatigue strength of holes in upper flange

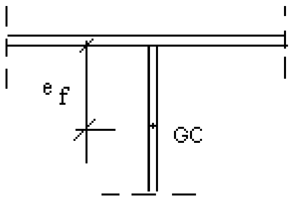
Detail 8, Appenix 3, BSK 99, Distance to the edge >3d gives

$C_{par} := 80$ $\quad f_{rk.3} := 119.23$ MPa

$f_{rdpar1} := \dfrac{f_{rk.3}}{(1.2 \cdot 1.1)}$ $\quad f_{rdpar1} = 90.326$ MPa

$\Delta M = 3318$ kNm $\quad x1 := 8.4$ m

$e_f := 1225 + 45 - 766 + \dfrac{33}{2}$ $\quad e_f = 520.5$ mm



$I := 0.02838$

$W1 := \dfrac{I}{e_f \cdot 10^{-3}}$ $\quad W1 = 0.055$ m^3

$\sigma_{rdpar1} := \dfrac{\Delta M \cdot 10^{-3}}{W1}$ $\quad \sigma_{rdpar1} = 60.853$ MPa

$\sigma_{rdpar1} = 60.853$ MPa $<$ $f_{rdpar1} = 90.326$ MPa OK!

Appendix B

Calculation according to Eurocode

Contents

Appendix B

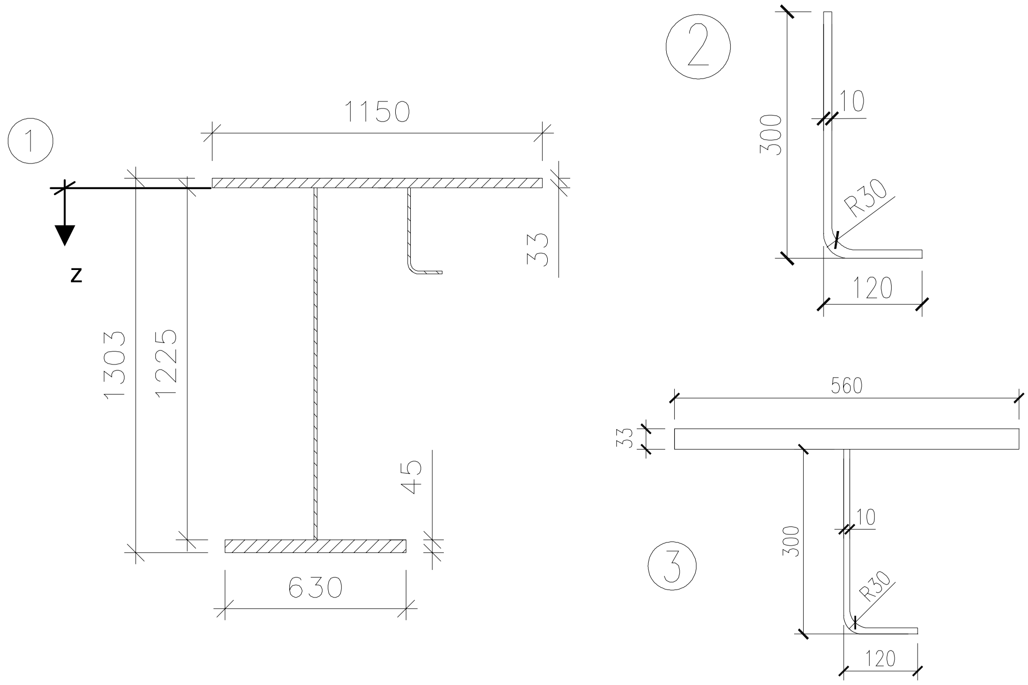
1. Loads and actions on the bridge

According to Eurocode

Documents used for receiving the loads are:

EN 1991-1-1	General actions- Densities, self-weight, imposed loads for buildings
prEN 1991-2	Traffic loads on bridges
ENV 1991-2-4	Wind actions
ENV 1993-2:1997	Steel bridges

The main beam is calculated as simply supported with half the bridge contributing



1.1 Cross-sectional constants

Table 1.1 Cross-sectional constants

Section nr	Section name	Web		Upper flange		Lower flange		Centre of gravity
		t	h	t	b	t	b	z
		mm	mm	mm	mm	mm	mm	mm
1	Main beam + stiff	12	1225	33	1150	45	630	524
2	Stiffener	10	290	0	0	10	105	185
3	Stiff + upper flange	10	290	33	560	10	105	19

Section nr	Section name		Area	Weight	Ix	W_{upper}	W_{lower}
			m^2	kg	m^4	m^3	m^3
1	Main beam + stiff		0,085	680	0,028	0,05099	0,03802
2	Stiffener		0,004	32	4E-05	0,0002	0,00033
3	Stiff + upper flange		0,0224	179	2E-04	0,0033	0,00061

Distance from the centre of gravity to lower edge of the beam 1225+45-504=766 mm

The shear centre is located 300mm over the top of the rail.

1.2 Permanent loads

Loads are calculated on half the bridge

1.2.1 Self-weight

The value of the density, in EN 1991-1-1, table A.4, and section 4.1, is slightly higher than Bro 2004, therefore the value of the self-weight is increased by a factor 77.75/77= 1,01

mean value: (77+78,5)/2= 77,75 kN/m^3

Main beam in weight/m			687	kg/m
For load combination in ULS the load factor = 1.05 according to EN 1990:2002 table A2.4(A).				
Load factor	1,05			
Gravity constant	10 m/s^2		**7,21**	**kN/m**
Cross beams		1.4*44.4/6	10,36	
Stiffener		15*8*0.28*1.225/3	13,72	
Stiffener		15*8*0.2*1.225/6	4,9	
Cantilever		10*8*0.45*1.8/6	10,8	
UPE 180		19.7*3	59,1	
Iron bars walkway		35*1.4	49	
Railing		60*2	120	
Screws and nuts		2*30*8*0.18*0.46/0.6	66,24	
Cables			50	
			388	kg/m
Load factor	1,05		**4,07**	**kN/m**

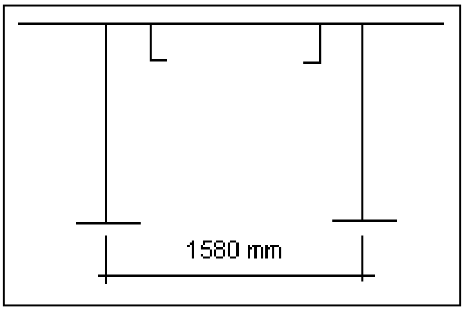
The values of the self-weight are calculated for half the bridge

1.3 Variable actions

1.3.1 Traffic load

According to the technical report:
At design of the main beams, it is assumed that the rail can be displaced by maximum 20mm.
e = 20 mm

Distance between the webs = 1,58m



If the rail is displaced in the transverse direction, the load will slightly increase on one beam while it decreases on the other beam.

Increase of load, caused by rail displacement
The Rail displacement factor is F=(1580/2+20)/1580 **F = 0,513**

The bridge is placed in a straight line with a span length of L = 18m

The dynamic contribution is calculated according to prEN 1991-2 (6.4.5.2)

$$\Phi_2 = \frac{1{,}44}{\sqrt{L_\Phi} - 0{,}2} + 0{,}82 \qquad L_\Phi = 18m$$

Dyn = $\Phi_2 =$ **1,176**

Load factor (LF), Load-combination ULS **LF = 1,45**

Design load factor in load-combination ULS, including rail displacement factor, load factor and dynamic factor

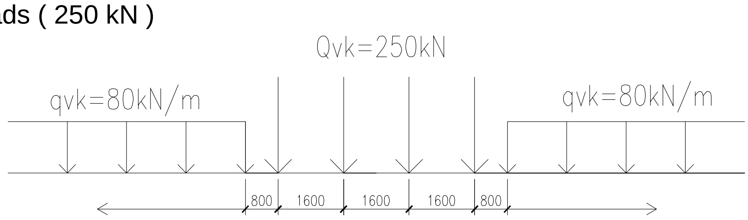
F_{dim} = F*LF*Dyn = 0,8743

In prEN 1991-2, there are two different load models to consider.

Load Model 71 (prEN 1991-2, 6.3.2)
Load Model SW/2 (prEN 1991-2, 6.3.3)

Load model 71

The characteristic values of Load Model 71 consists of, two uniformly distributed loads (80 kN/m) and four axis loads (250 kN)



These characteristic values are multiplied by a factor alpha = 1.33.
When multiplied with the factor alpha the characteristic value becomes the classified value.

alpha = 1,33

	Characteristic		Classified		
P =	250 kN	alpha*P =	332,5 kN		4 pcs
q =	80 kN/m	alpha*q =	106,4 kN/m		

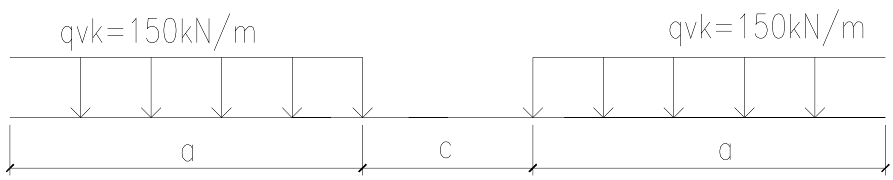
Design load in load combination ULS

$P_{dim} = P*F_{dim} = 332,5*F_{dim}$ $P_{dim} =$ 290,7 kN

$q_{dim} = q*F_{dim} = 106,4*F_{dim}$ $q_{dim} =$ 93,0 kN/m

Load Model SW/2

Load Model SW/2 consists of two uniformly distributed loads (150 kN/m)



a = 25 m
c = 7 m

The worst case for this bridge is one uniformly distributed load over the entire length of the bridge

Load Model 71 result in a greater response due to the axle loads.
Therefore the bridge will be designed according to load LM 71.

1.3.2 Derailment load

According to prEN 1991-2 (6.7.1) it assumes that derailment load LM 71, including alpha factor, shall be displaced in the transverse direction, that in this case is defined by the derailment protection to 300 mm

This is an accident case with load factor = 0,7
which result in the design force:

$P_{dim} = 1,33*250*0,7*(1580/2+300)/1580*D$ $P_{dim} =$ 188,9 kN

The derailment load will not be the design load for the main beam.

1.3.3 Nosing force

According to prEN 1991-2 (6.5.2), the bridge shall be designed for a single nosing force, with the characteristic value of 100 kN, which is acting at the top of the rail.

The nosing force shall be multiplied by the alpha value of 1.33.

P_{nosing} = 1.33*100 = **133 kN** classified value

Due to the fact that the shear centre is located slightly above the upper edge of the rail, a load in the transverse direction will give a small contribution to the vertical load.
This load will not be used for design of the main beam,
but will be used for design of local parts

1.3.4 Wind load

According to ENV 1991-2-4 (10.11.2) the wind load on the bridge structure should be set to 6 kN/m^2.

Characteristic wind load on bridge **6,0 kN/m^2**

The wind load acting on the train was calculated using ENV 1991-2-4

The external pressure

$$W_e = q_{ref} \cdot C_e(z) \cdot C_{pe} \tag{5.1}$$

The reference mean wind velocity pressure

$$q_{ref} = \frac{\rho}{2} \cdot V_{ref}^2 \qquad \rho = 1.25 kg/m^3 \tag{7.1}$$

Air density

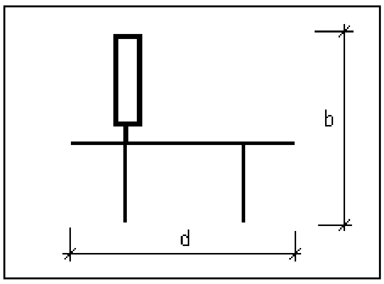
Reference wind velocity for Gothenburg, $V_{ref,0}$ = 25 m/s (Figure A9 in ENV 1991-2-4)

V_{ref} = C_{DIR}*C_{TEM}*C_{ALT}*$V_{ref,0}$ (7.2)

C_{DIR} = C_{TEM} = C_{ALT} = 1.0 => V_{ref} = 25 m/s (7.2)

q_{ref} = 1.25/2*25^2 = 0,39 kN/m^2

Force coefficient for bridge deck is calculated for the bridge with traffic



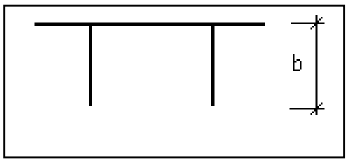
(Figure 10.11.2 in ENV 1991-2-4)

$$C_f = C_{pe} = C_{f,0} \cdot \psi_\lambda$$

$$\left.\begin{array}{l}\dfrac{d}{b} = \dfrac{2300}{1525+4000} = 0.416 \\ \text{curve b}\end{array}\right\} \Rightarrow \quad C_{f,0} = 2{,}4$$

(Figure 10.11.2 in ENV 1991-2-4)

The slenderness reduction factor
is calculated for the height of the bridge cross-section



(Figure 10.11.2 in ENV 1991-2-4)

$$\left.\begin{array}{l}\lambda = \dfrac{L}{b} = \dfrac{18000}{1525} = 11.8 \\[2ex] \varphi = \dfrac{A}{A_C} = \dfrac{L\cdot b}{L\cdot b} = 1\end{array}\right\} \Rightarrow \quad \psi_\lambda = 0{,}71$$

(Figure 10.14.2 in ENV 1991-2-4)

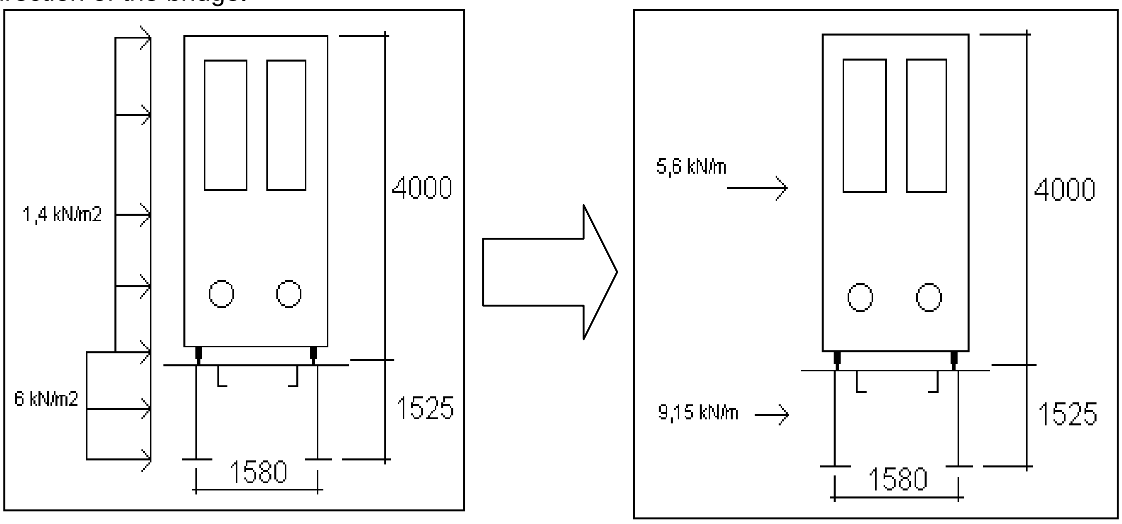
$$C_f = C_{f,0}\cdot\psi_\lambda = 2.4\cdot 0.71$$

$$\left.\begin{array}{l}z = 7\text{m} \\ \text{Terrain category 2}\end{array}\right\} \Rightarrow \quad \text{Ce(z)} = \quad 2{,}1$$

(Figure 8.3 in ENV 1991-2-4)

Characteristic wind load on train **We = 0.39*2.4*0.71*2.1=** **1,4kN/m^2**

The load is transformed from an area based value to a distributed load acting in the longitudinal direction of the bridge.



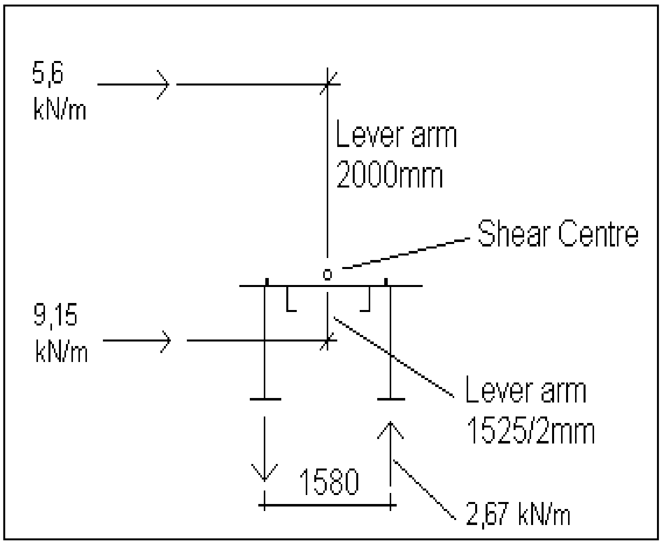
height of bridge	1,525	m
height of train	4	m

Distributed loads in longitudinal direction

Characteristic wind load on the bridge	1.525*6 =	**9,15**	**kN/m**
Characteristic wind load on the train	4*1.4 =	**5,6**	**kN/m**

1.3.4.1 Vertical load increase due to wind.

Calculation on the safe side. Assume the shear centre is located in line with the top edge of the rail.



Moment calculation around Shear Centre results in

(5.6*2-9.15*1.525/2)/1.58 = 2,67 kN/m

Design vertical load due to wind, in load combination ULS

Load factor	1,5	EN 1990:2002 table A2.4(A).
Combination factor	0,75	EN 1990:2002 table A2.4(A).

$q_{wind,dim}$ = 1.5*0.75*2.67 = 3,0 kN/m (vertical load on one beam due to wind)

1.3.5 Horizontal force

According to prEN 1991-2:2002 the greatest characteristic value of the brake- and traction force is

P_{dim} = 33 kN/m (Traction force)
L 18 m

This value should be multiplied by the factor alpha from chapter 1.3.1.

P_{max} = 18*33*1.33 790 kN Classified value

1.4 Fatigue load

The fatigue load is calculated using ENV 1993-2:1997

The check have been made with the, Simplified fatigue load model for railway bridges. (9.2.3)

The loads used are characteristic values of train load LM 71, from section 1.3.1, including dynamic factor and damage equivalence factor

Axle loads	250kN
Uniformly distributed	80kN/m

Dynamic factor is taken from prEN 1991-2:2002

The value of the dynamic factor is calculated by consideration of the vehicle speed and the span length of the structural member.

$$\Phi_2 = 1 + \frac{1}{2}(\varphi' + \frac{1}{2}\varphi'') \tag{D.1}$$

$$\varphi' = \frac{K}{1 - K + K^4} \tag{D.2}$$

$$K = \frac{v}{160} \quad \text{For } L \leq 20\text{m} \tag{D.3}$$

v is maximum permitted vehicle speed [m/s]

$$v = 70km/h = \frac{70}{3.6} = 19.4m/s$$

$$K = \frac{19.4}{160} = 0.12125$$

$$\varphi'' = 0.56e^{-\frac{L^2}{100}} \qquad L = 18m \quad \text{Span length} \tag{D.5}$$

$$\varphi' = \frac{0.12125}{1 - 0.12125 + 0.12125^4} = 0.1379$$

$$\varphi'' = 0.56e^{-\frac{18^2}{100}} = 0.02193$$

The dynamic factor for fatigue is

$$\Phi_2 = 1 + \frac{1}{2}(0.1379 + \frac{1}{2}0.02193) = 1.0744$$

Damage equivalence factor for railway bridges is taken from ENV 1993-2:1997 section 9.5.3

$\lambda = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4$ but $\lambda \leq \lambda_{max}$ $\lambda_{max} = 1.4$

λ_1 Depends on span length

λ_2 Takes into account the traffic volume

λ_3 Takes into account the design life of the bridge

λ_4 Takes into account the number of tracks

λ_1	= 0.72	(Table 9.5, 25t Mix)
λ_2	= 1.00	(Table 9.6, $25*10^6$ton/track/year)
λ_3	= 1.04	(Table 9.7, 120 years)
λ_4	= 1.00	(Table 9.8, One track)

$\lambda = 0.72 \cdot 1.00 \cdot 1.04 \cdot 1.00 = 0.7488$ $\lambda < \lambda_{max} = 1.4$ OK !

Design loads for fatigue

Train axle load	250*1.0744*0.7488 =	**201,1**	**kN**
Uniformly distributed load	80*1.0744*0.7488 =	**64,4**	**kN/m**

2 Design of the main beam, EC 3

Documents used in the design

EN 1990:2002/prA1:2004	Application for Bridges
prEN1991-2:2002 (E)	Traffic loads on bridges
ENV 1993-1-1:1992	General rules and rules for buildings
prEN 1993-1-1:2004	General rules and rules for buildings
ENV 1993-2:1997	Steel bridges
SS-ENV 1993-2	Steel bridges, NAD
prEN 1993-1-8:2002	Design of joints
prEN 1993-1-9:2002	Fatigue strength of steel structures
SS EN 1993-1-9	Fatigue, NAD

2.1 Stresses over the cross-section, ULS

The first check is the stresses over the cross-section, the bridge is simply supported, so the largest response will appear in the middle of the beam. The bending moment will be calculated for each load, and then added together.

The formula of the bending moment is $M := q\cdot\frac{L^2}{8}$ where L = 18 m

All the loads in kN/m are from section 1.

2.1.1 Self-weight

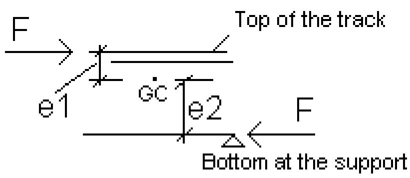
Self-weigth $M_{self} := \frac{18^2}{8}\cdot(7.21 + 4.074)$ $M_{self} = 457.002$ kNm

2.1.2 Wind load

Wind load $M_{wind} := \frac{18^2}{8}\cdot 3.00$ $M_{wind} = 121.5$ kNm

2.1.3 Brake- and acceleration force

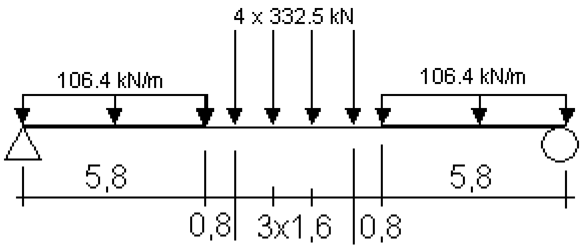
The brake- and acceleration force is acting in top of the track. The horizontal reaction force at the support acts in the bottom of the beam cross-section. The center of gravity is located closer to the top, therefore, the total moment of this force is negative, which is a favorable response, and is not added to the calculations.



e1 < e2 => Negative moment

2.1.4 Train load

The design train load is LM 71:



Moment in mid span

Point loads $M_p := 332.5(2\cdot 6.6 + 1.6)$ $M_p = 4921$ kNm

Uniform $M_u := 106.4\frac{5.8^2}{2}$ $M_u = 1789.648$ kNm

Total moment in mid span $M_{train} := 0.8743(M_p + M_u)$

Fdim

$M_{train} = 5867.12$ kNm

2.1.5 The total moment

$M_{tot1} := M_{self} + M_{wind} + M_{train}$ $M_{tot1} = 6445.622$ kNm

$M_{tot} := \frac{M_{tot1}}{1000}$ $M_{tot} = 6.446$ MNm

2.1.6 Stresses over the cross-section

f_y The nominal value of the yield strength is taken from Table 3.1 (ENV 1993-1-1), according to the technical report.

γ_M From section 5.1.1 in NAD(S)/SS-ENV 1993-2.

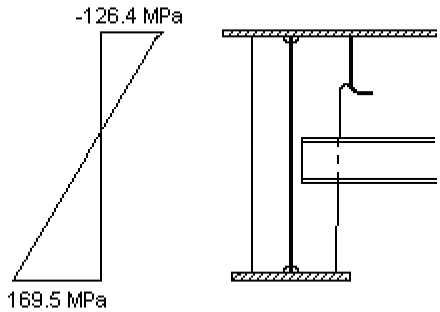
$f_y := 355$ MPa $\gamma_{M0} := 1.0$

$f_{yd} := \frac{f_y}{\gamma_{M0}}$ $f_{yd} = 355$ MPa

$W_{u1} := 0.05099$ m^3

$W_{l1} := 0.03802$ m^3

(Table 1.1 cross-sectional constants)



This give the following stresses.

$$\sigma_{tf} := \frac{M_{tot}}{-W_{u1}} \qquad \sigma_{tf} = -126.41 \quad \text{MPa}$$

$$\sigma_{bf} := \frac{M_{tot}}{W_{l1}} \qquad \sigma_{bf} = 169.532 \quad \text{MPa}$$

$$\left|\sigma_{tf}\right| = 126.41 \text{ MPa} \quad < \quad f_{yd} = 355 \text{ MPa} \quad \text{OK!}$$ (Compression)

$$\sigma_{bf} = 169.532 \text{ MPa} \quad < \quad f_{yd} = 355 \text{ MPa} \quad \text{OK!}$$ (Tension)

2.2 The longitudinal stiffeners with the effective flange, ULS

First the average stress over the cross-section is checked. Secondly the flexural buckling is confirmed. Then the stresses due to eccentricity of the nosing- and axle force are checked. Finally the flange of the longitudinal stiffener is verified for lateral buckling.

2.2.1 Average stress in longitudinal stiffener with effective flange

top flange $560 \cdot 33 \ \text{mm}^2$ (Table 1.1 cross-sectional constants)

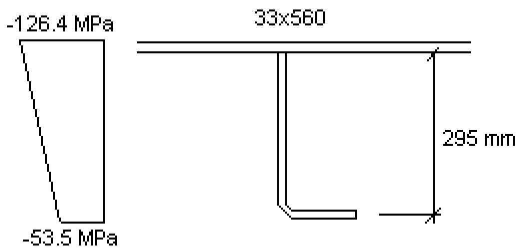
$\sigma_{tf} = -126.41$ MPa (compression)

The stress is linear over the cross section,

$$1303\text{mm} - \frac{45\text{mm}}{2} - \frac{33\text{mm}}{2} = 1.264\text{m}$$ (Distance between center of flanges)

$$\sigma_{ratio} := \frac{-(\sigma_{tf} - \sigma_{bf})}{1.264} \qquad \sigma_{ratio} = 234.131$$ MPa / meter

Stress in bottom flange of stiffener $$\sigma_{bs} := \sigma_{tf} + \left(\frac{0.033}{2} + 0.295\right) \cdot \sigma_{ratio}$$



$\sigma_{bs} = -53.478$ MPa (Compression)

Average stress in the longitudinal stiffener

$$\sigma_{average} := \frac{\sigma_{tf} + \sigma_{bs}}{2}$$

$$\sigma_{average} = -89.944 \text{ MPa}$$

Total compression force in longitudinal stiffener

$P_{tot} := 33 \cdot 560 \cdot \sigma_{tf} + 285 \cdot 10 \cdot \sigma_{average} + 110 \cdot 10 \cdot \sigma_{bs}$

$P_{tot} = -2651212.57$ N

Average stress in the local cross-section

$\sigma_{b.Sd} := \dfrac{P_{tot}}{(33 \cdot 560 + 10 \cdot 395)}$ $f_y := 355$ MPa $\gamma_{M0} := 1.0$

$\sigma_{b.Sd} = -118.199$ MPa $f_{yd} := \dfrac{f_y}{\gamma_{M0}}$ $f_{yd} = 355$ MPa

$|\sigma_{b.Sd}| = 118.199$ MPa $<$ $f_{yd} = 355$ MPa OK!

2.2.2 Buckling resistance of members

Check of the buckling resistance, section 6.3.1 in prEN1993-1-1:2004.

The longitudinal stiffeners are stiffened every third meter by the vertical stiffeners.

The modulus of elasticity is taken from section 3.2.6 in prEN1993-1-1:2004.

$f_y := 355$ MPa $E := 210000$ MPa $\gamma_{M1} := 1.0$

$f_{yd} := \dfrac{f_y}{\gamma_{M1}}$ $f_{yd} = 355$ MPa $L_{cr} := 3$ m

$A_3 := 0.0224$ $I_3 := 0.00017$ (Table 1.1 cross-sectional constants)

Stress capacity

$\sigma_{b.Rd} := \chi \cdot f_{yd}$

Actual stress

$\sigma_{Ed} := \sigma_{b.Sd}$ $\sigma_{Ed} = -118.199$ MPa

$\iota_1 := \sqrt{\dfrac{I_3}{A_3}}$ $\lambda := \dfrac{L_{cr}}{\iota_1}$ $\iota_1 = 0.087$

$\lambda_1 := \pi \cdot \sqrt{\dfrac{E}{f_y}}$ $\lambda_1 = 76.409$ $\lambda_s := \left(\dfrac{\lambda}{\lambda_1}\right)$

Buckling curve c $\alpha := 0.49$

$$\phi := 0.5\left[1 + \alpha\cdot(\lambda_s - 0.2) + \lambda_s^{\,2}\right] \qquad \phi = 0.663$$

$$\chi := \frac{1}{\phi + \sqrt{\phi^2 - \lambda_s^{\,2}}} \qquad \chi = 0.87$$

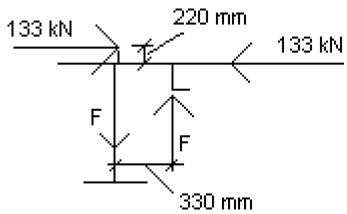
$$\sigma_{b.Rd} := \chi\cdot f_{yd} \qquad \sigma_{b.Rd} = 308.908 \text{ MPa}$$

$$\left|\sigma_{Ed}\right| = 118.199 \text{ MPa} < \sigma_{b.Rd} = 308.908 \text{ MPa} \qquad \text{OK!}$$

2.2.3 Forces caused by eccentricity of the nosing force.

The nosing force acts at the top edge of the rail, i.e. 220 mm over the upper flange. This will give a moment that is taken by two forces, one in the web and one in the longitudinal stiffener.

$F_{nosing} := 133$ kN



$\gamma_{Q.1} := 1.45$ (Load factor)

$$F := \gamma_{Q.1}\cdot\frac{(F_{nosing}\cdot 220)}{330} \qquad F = 128.567 \text{ kN}$$

2.2.4 Forces caused by eccentricity of the axle force

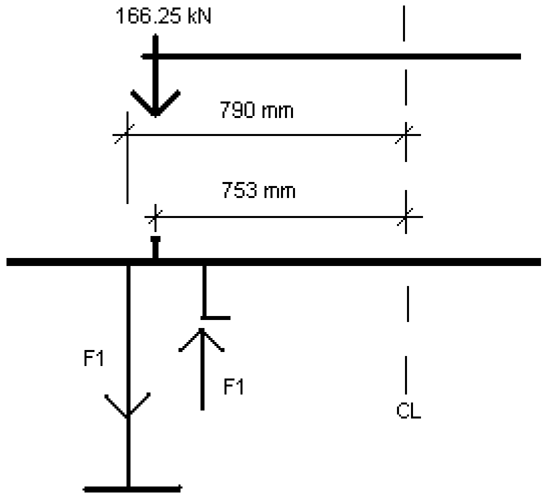
The rail is located 753 mm from centre line (CL) of the bridge and the webs are located 790 mm from CL of the bridge. This will give a moment that is taken by two forces, one in the web and one in the longitudinal stiffener. Also add the displacment excentricity of 20 mm

This gives the eccentricity of:

$790 - 753 + 20 = 57$ mm

Calculate for one axle force.

$\dfrac{332.5}{2} = 166.25$ kN



Dynamic factor

$L_{\phi loc} := 4.5$ Length of the local member, Table 6.2 in prEN 1991-2:2002(E)

$D := \dfrac{1.44}{\sqrt{L_{\phi loc}} - 0.2} + 0.82 \qquad D = 1.569$

$\gamma_{Q.2} := 1.45$ (Load factor)

$\psi_0 := 0.80$ (Other variable load)

Calculating the bending moment

$M := 0.057 \cdot 166.25 \cdot 1.569 \cdot 1.45 \cdot 0.80 \qquad M = 17.247$ kNm

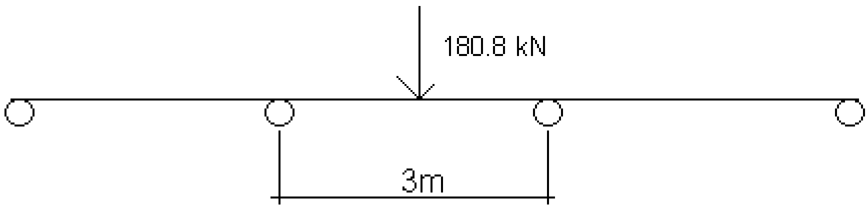
This gives a vertical force of

$F1 := \dfrac{M}{0.33} \qquad F1 = 52.264$ kN

2.2.5 Stresses in longitudinal stiffener with effective flange, caused by axle- and nosing force

Calculate the longitudinal stiffener as a continuous beam with support every third meter

The total force $F_{tot} := F + F1$ $F_{tot} = 180.831$ kN



Calculation of the moment

Table value on safe side from elementary case

M = Ftot*L*0.18

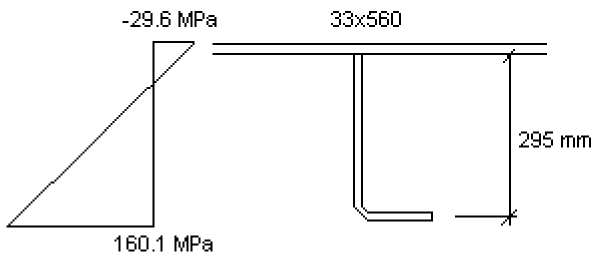
$M := F_{tot} \cdot 3 \cdot 0.18$ $M = 97.649$ kNm

Stresses in the longitudinal stiffener

$W_{l3} := 0.00061 m^3$ $W_{u3} := 0.0033\ m^3$ (Table 1.1 cross-sectional constants)

Lower edge $\sigma_{stiff} := \dfrac{M \cdot 10^{-3}}{W_{l3}}$ $\sigma_{stiff} = 160.08$ MPa (tension)

Upper edge $\sigma_{flange} := \dfrac{M \cdot 10^{-3}}{-W_{u3}}$ $\sigma_{flange} = -29.59$ MPa (compression)



$\left|\sigma_{flange}\right| = 29.59$ MPa < $f_{yd} = 355$ MPa OK!

$\sigma_{stiff} = 160.08$ MPa < $f_{yd} = 355$ MPa OK!

2.2.6 Lateral buckling in the flange of the longitudinal stiffener

The flange of the longitudinal stiffener will be checked with one third of the stiffener's web contributing. The axle- and nosing force will not be included, because they give tension stresses in the flange, section 6.3.1 in prEN 1993-1-1:2004.

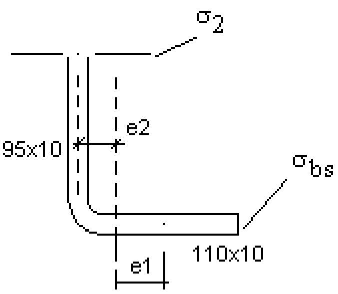
$$N_{b.Rd} := \chi_2 \cdot A_2 \cdot \frac{f_y}{\gamma_{M1}}$$

$\sigma_{bs} = -53.478$ MPa

$\sigma_{tf} = -126.41$ MPa

$\sigma_{ratio} = 234.131$ MPa/m

$\sigma_2 := \sigma_{bs} - \sigma_{ratio} \cdot 0.095$ $\qquad \sigma_2 = -75.72$ MPa



$A_2 := 95 \cdot 10 + 110 \cdot 10$ $\qquad A_2 = 2050$ mm^2

$e_2 := \frac{(110 \cdot 10 \cdot 50)}{2050}$ $\qquad e_2 := 26.829$ mm

$e_1 := \frac{110}{2} - e_2 - 5$ $\qquad e_1 = 23.171$ mm

$I_2 := e_2^{\,2} \cdot 95 \cdot 10 + e_1^{\,2} \cdot 110 \cdot 10 + \frac{\left(10 \cdot 110^3\right)}{12}$ $\qquad I_2 = 2383556.911$ mm^4

$L_{cr2} := 3000$

$\iota_2 := \sqrt{\frac{I_2}{A_2}}$ $\qquad \iota_2 = 34.099$ $\qquad \lambda_2 := \frac{L_{cr2}}{\iota_2}$

$\lambda_1 := \pi \cdot \sqrt{\frac{E}{f_y}}$ $\qquad \lambda_1 = 76.409$ $\qquad \lambda_{s2} := \frac{\lambda_2}{\lambda_1}$ $\qquad \lambda_{s2} = 1.151$

Buckling curve c $\qquad \alpha = 0.49$

$\phi_2 := 0.5\left[1 + \alpha \cdot \left(\lambda_{s2} - 0.2\right) + \lambda_{s2}^{\,2}\right]$ $\qquad \phi_2 = 1.396$

$\chi_2 := \frac{1}{\phi_2 + \sqrt{\phi_2^{\,2} - \lambda_{s2}^{\,2}}}$ $\qquad \chi_2 = 0.458$ but, must be smaller or equal to 1.

$N_{b.Rd} := \chi_2 \cdot A_2 \cdot 10^{-3} \cdot \frac{f_y}{\gamma_{M1}}$ $\qquad N_{b.Rd} = 333.017$ kN

Average stress in the lower third of the longitudinal stiffener

$\sigma_{aver} := \frac{(\sigma_{bs} + \sigma_2)}{2}$ $\sigma_{aver} = -64.599$ MPa

$N_{Ed} := (\sigma_{bs} \cdot 110 \cdot 10 + \sigma_{aver} \cdot 95 \cdot 10) \cdot 10^{-3}$ $N_{Ed} = -120.194$ kN

$|N_{Ed}| = 120.194$ MPa $<$ $N_{b.Rd} = 333.017$ MPa OK!

2.3 Buckling of the plate between the longitudinal stiffeners, ULS

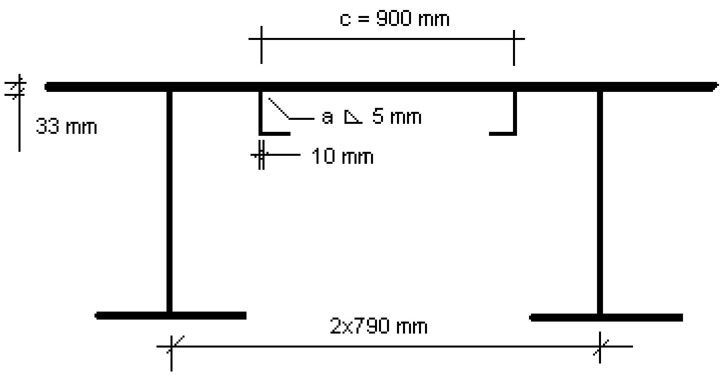
Stresses in the longitudinal plate.

$\sigma_{flange} = -29.59$ MPa Stress from nosing- and axle force

$\sigma_{tf} = -126.41$ MPa Stress from global bending

Total stress $\sigma_{total} := \sigma_{flange} + \sigma_{tf}$

$\sigma_{total} = -156$ MPa



The free length of the flange

$2 \cdot 790 - 2 \cdot 330 - 10 - 2 \cdot 5 = 900$ mm $c := 900$ mm $t_f := 33$

From Table 5.3.1 in prEN 1993-1-1:2004

$\frac{c}{t_f} = 27.273$

$42 \cdot \sqrt{\frac{235}{355}} = 34.172$

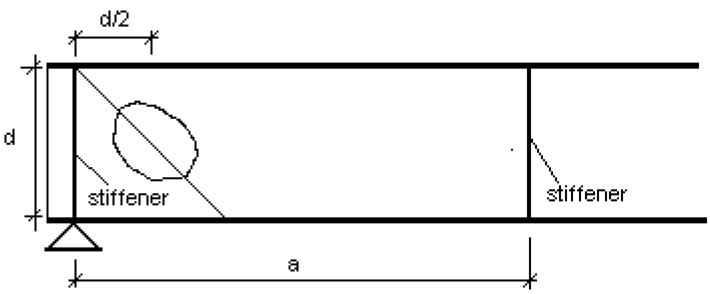
$\frac{c}{t_f} < 42 \cdot \varepsilon$ OK! Class 3

$f_y = 355$ $\gamma_{M1} = 1$

$|\sigma_{total}| = 156$ MPa $<$ $\frac{f_y}{\gamma_{M1}} = 355$ MPa OK! No risk of buckling!

2.4 Shear buckling of the web, ULS

The magnitude of the shear force is taken from the Matlab calculation. The buckling is checked for the highest shear force at x = d/2 = 0.6m from the support.



$V_{Ed} := 1.449$ MN (Max shear force 0.6 m from the support, From Matlab calculation)

Check of shear buckling resistance with simple post-critical method.
According to ENV 1993-1-1:1992 section 5.6.3.

$V_{be.Rd} := d \cdot t_w \cdot \frac{\tau_{be}}{\gamma_{M1}}$ $\quad a := 3000$ mm $\quad d := 1225$ mm $\quad d_1 := d - 2 \cdot \sqrt{2} \cdot 5$

$d_1 = 1210.858$ mm $\quad t_w := 12$ mm $\quad f_y = 355$ MPa $\quad f_{yw} := f_y \quad \gamma_{M1} = 1$

$\frac{a}{d} = 2.449$ => $k_\tau := 5.34 + \frac{4}{\left(\frac{a}{d_1}\right)^2}$ $\quad k_\tau = 5.992$

$\varepsilon_1 := \sqrt{\frac{235}{f_y}}$ $\quad \lambda_w := \frac{\left(\frac{d_1}{t_w}\right)}{37.4 \varepsilon_1 \cdot \sqrt{k_\tau}}$ $\quad \lambda_w = 1.355$

when $\lambda w \geq 1.2$ => $\tau_{be} := \frac{0.9 \cdot f_{yw}}{\lambda_w \cdot \sqrt{3}}$ $\quad \tau_{be} = 136.164$ MPa

$V_{be.Rd} := 1.225 \, 0.012 \frac{\tau_{be}}{1.0}$ $\quad V_{be.Rd} = 2.002$ MN

$V_{Ed} = 1.449$ MN $\quad < \quad V_{be.Rd} = 2.002$ MN $\quad$ OK!

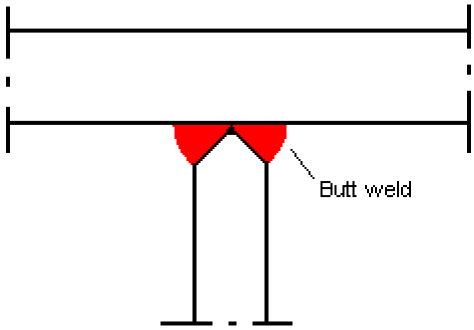
2.5 Welds between the web and the flanges, ULS

The welds, both between the upper flange and the web, and between the lower flange and the web will be checked for the highest Shear Force. This is a static analysis in the ultimate limit state. The checks are carried out with Eurocode prEN 1993-1-8:2002, section 4.5, 4.7.

2.5.1 Butt welds between web and upper flange

Butt welded connection against upper flange

Calculate weld as if weld has the same strength as the material



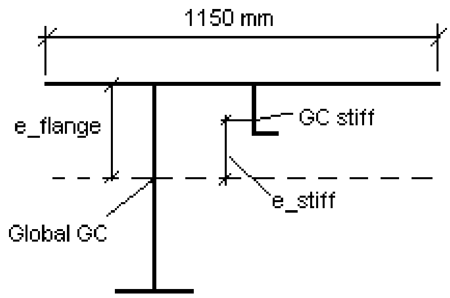
S 355N, $f_u := 490$ MPa

$\beta_w := 0.9$

$\gamma_{M2} := 1.2$ NAD SS EN 1993-2

$$f_{vw.d} := \frac{\left(\frac{f_u}{\sqrt{3}}\right)}{\beta_w \cdot \gamma_{M2}}$$ $f_{vw.d} = 261.946$ MPa

The highest value of the Shear force is received over the support.



$V_{max} := 1.532$ MN (train, wind, self)

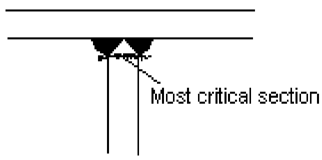
$A_{flange} := 1.150\,0.033$ $A_{flange} = 0.038$ m^2

$e_{flange} := 0.524 + \frac{0.033}{2}$ $e_{flange} = 0.54$ m

$A_{stiff} := 0.395\,0.010$ $A_{stiff} = 0.004$ m^2

$e_{stiff} := 0.524 - 0.185$ $e_{stiff} = 0.339$ m

$S_1 := A_{flange} \cdot e_{flange} + A_{stiff} \cdot e_{stiff}$ $S_1 = 0.022$ m^3



$I := 0.02838$ m^4

$b := 0.012$ m

The most critical section is the web, since it is most narrow.

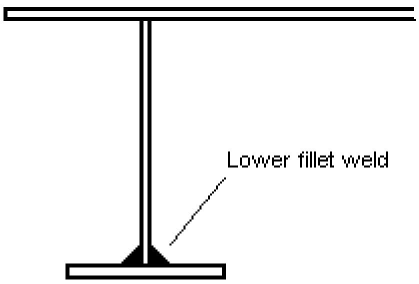
$$\tau_{Sd.para} := \frac{V_{max} \cdot S_1}{I \cdot b}$$ $\tau_{Sd.para} = 98.296$ MPa

$\tau_{Sd.para} = 98.296$ MPa $<$ $f_{vw.d} = 261.946$ MPa OK!

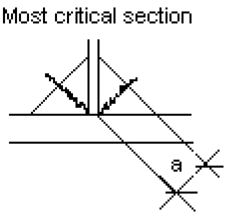
2.5.2 Fillet weld between web and lower flange

ENV 1993-1-1:1992, section 6.6.5.3.

$S_s := 630 \cdot 45 \cdot \left(766 - \frac{45}{2}\right)$ $S_s = 21078225$ mm^3



Thickness of weld, a = 5 mm



$b_s := 0.01$ m

The highest value of the shear force is received over the support.

$V_{max} = 1.532$ MN (from Matlab calculation)

$$\tau_{S.para} := \frac{V_{max} \cdot S_s \cdot 10^{-9}}{I \cdot b_s}$$ $\tau_{S.para} = 113.784$ MPa

$f_u := 490$ MPa

$\beta_w := 0.9$

$\gamma_{M2} := 1.2$

$$f_{vw.d} := \frac{\left(\frac{f_u}{\sqrt{3}}\right)}{\beta_w \cdot \gamma_{M2}}$$ $f_{vw.d} = 261.946$ MPa

$\tau_{S.para} = 113.784$ MPa $<$ $f_{vw.d} = 261.946$ MPa OK!

2.6 Max allowed displacement, SLS

According to EN 1990:2002/prA1:2004, A2.4.4.2.3, the max vertical displacement is limited to L / 600. The maximum displacement should be calculated using the characteristic value of load model 71.

For this bridge $L := 18000$

$$p_{max} := \frac{L}{600} \qquad p_{max} = 30 \ \text{mm}$$

With characteristic value of LM 71, the Matlab calculation gives:

$p := 20.0 \ \text{mm}$ (In mid section)

$p = 20 \ \text{mm} \quad < \quad p_{max} = 30 \ \text{mm} \quad$ OK!

2.7 Fatigue

When checking the bridge for fatigue strength the self-weigth is excluded. This is because the largest response variation in stresses is considered, and since the self-weigth is constant it will not contribute to this variation.

The simplified fatigue load model for railway bridges will be used according to section 9.2.3 in ENV 1993-2:1997.

Load used: LM 71

The butt weld between the web and upper flange and the fillet weld between the vertical stiffener and the web will be checked for fatigue strength. Also the holes in the upper flange will be confirmed.

Dynamic factor for fatigue

The dynamic factor for fatigue is calculated according to Annex D, section D.1 in prEN 1991-2:2002 (E).

$$\Phi_2 := 1 + \frac{\left(\zeta' + \frac{\zeta''}{2}\right)}{2}$$

$\nu := \frac{70}{3.6}$ $\quad \nu = 19.444 \ \frac{m}{s}$ $\quad \nu$ is the maximum permitted vehicle speed

$$K := \frac{\nu}{160}$$

$L_\Phi := 18$ m Table 6.2 case 5.1 in prEN1991-2:2002 (E).

$$\zeta' := \frac{K}{1 - K + K^4}$$

$\zeta'' := 0.5 \cdot e^{-\frac{\left(L_\Phi\right)^2}{100}}$ $\quad \zeta'' = 0.02$

$$\Phi_2 := 1 + \frac{\left(\zeta' + \frac{\zeta''}{2}\right)}{2}$$

$$\Phi_2 = 1.074$$

General design method

Annex D section D.2 in prEN 1991-2:2002 (E)

$$\gamma_{Ff} \cdot \lambda_f \cdot \Phi_2 \cdot \Delta\sigma_{71.i} < \frac{\Delta\sigma_{R.i}}{\gamma_{Mf.i}}$$

Damage equivalence factors λ

From section 9.5.3 in ENV1993-2:1997.

$\lambda_f := \lambda_{1f} \cdot \lambda_{2f} \cdot \lambda_{3f} \cdot \lambda_{4f}$

λ_{1f} Is taken from table 9.5 in ENV1993-2:1997, according to NAD SS-ENV 1993-2.
25 t Mix, L=18 m
Interpolation gives

$\lambda_{1f} := 0.72$

λ_{2f} Is taken from table 9.6 in ENV1993-2:1997. The traffic load is chosen to 25*10^6 tonnes a year.

$\lambda_{2f} := 1.00$

λ_{3f} Is taken from table 9.7 in ENV1993-2:1997 in terms of design life to 120 years according to the technical report.

$\lambda_{3f} := 1.04$

λ_{4f} Is chosen to 1.00 according to NAD SS EN 1993-2 section 9.5.2(6). The bridge has a single track.

$\lambda_{4f} := 1.00$

$\lambda_f < \lambda_{max}$ $\lambda_{max} := 1.4$ section 9.5.3(9) in ENV1993-2:1997.

$\lambda_f := \lambda_{1f} \cdot \lambda_{2f} \cdot \lambda_{3f} \cdot \lambda_{4f}$

$\lambda_f = 0.749$

Partial factors for fatigue

$\gamma_{Ff} := 1.00$ Section 9.3(1) in ENV1993-2:1997.

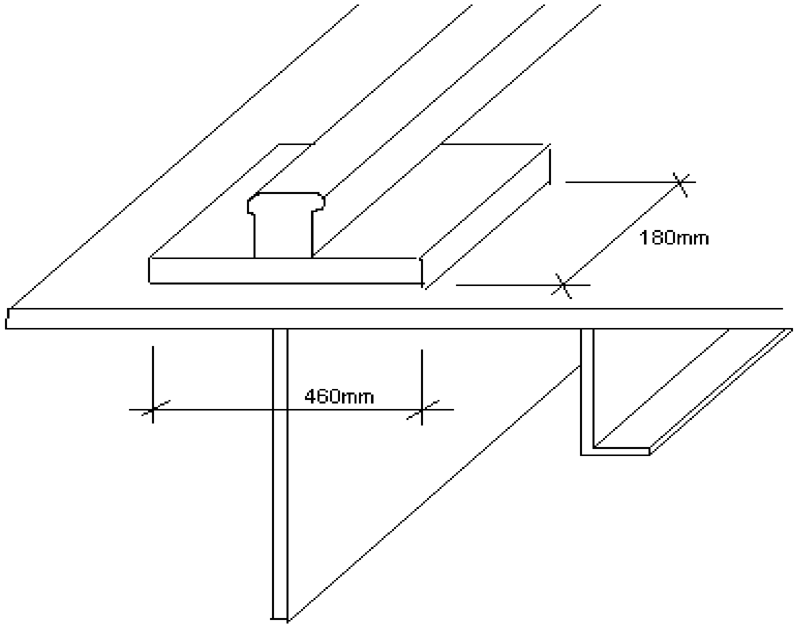
$\gamma_{Mf} := 1.35$ Table 3.1 in prEN 1993-1-9 : 2002.
And NAD SS EN 1993-1-9.
For the structural system.

$\gamma_{Mfh} := 1.15$ Table 3.1 in prEN 1993-1-9 : 2002.
And NAD SS EN 1993-1-9.
For the holes in upper flange.

2.7.1 Fatigue strength of butt weld between upper flange and web

2.7.1.1 Stresses due to axle pressure

When the wheels is acting on the plates 180 mm x 460 mm, which are resting on the upper flange, there will be a change in stress distribution vertically on the butt welds.



$n := 10 \cdot 10^6$ 10 million cycles for a correct comparison

Detail category 71 prEN 1993-1-9:2002, Toe failure in full penetration butt welds

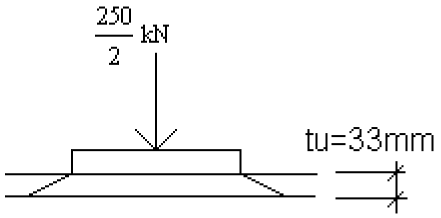
$C_{perp} := 71 \;\; \text{MPa}$ For 2 million cycles

For 10 million cycles the capacity becomes, prEN 1993-1-9:2002, Figure 7.1

$$\Delta\sigma_{R.1} := C_{perp} \cdot \left(2 \cdot \frac{10^6}{10 \cdot 10^6}\right)^{\frac{1}{3}}$$ $\Delta\sigma_{R.1} = 41.521 \;\; \text{MPa}$ (Perpendicular)

$\gamma_{Mf} = 1.35$

$$\frac{\Delta\sigma_{R.1}}{\gamma_{Mf}} = 30.756 \;\; \text{MPa}$$



Load distribution 1:2.5 gives:

thickness web $t_w := 12$ mm

The compression surface will be tw*(180+2*2,5*33) = 12*345 mm^2

The dynamic contribution has to be added to the load:

$\Phi_2 = 1.074$ $\lambda_f = 0.749$

250 kN represent two wheels

Wheel load $F_{71} := 250 \cdot 0.5 \cdot \Phi_2 \cdot \lambda_f$ $F_{71} = 100.531$ kN

Stress width $\Delta\sigma_{71.1} := \dfrac{\left(F_{71} \cdot 10^3\right)}{12 \cdot 345}$ $\Delta\sigma_{71.1} = 24.283$ MPa

$\gamma_{Ff} \cdot \Delta\sigma_{71.1} = 24.283$ MPa < $\dfrac{\Delta\sigma_{R.1}}{\gamma_{Mf}} = 30.756$ MPa OK!

2.7.1.2 Stress range due to bending

In Eurocode, the stresses acting in the parallel direction of the weld is not considered.

2.7.1.3 Stress range due to shear

$n_n := 2 \cdot 10^6$ cycles For comparison

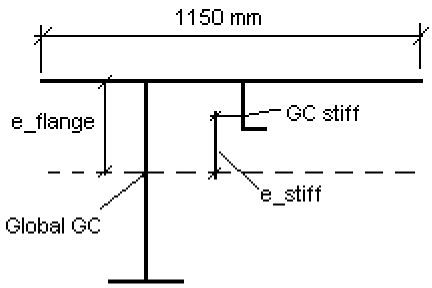
Detail catagory 112, Butt weld carried out from both sides

$C_{para.2} := 112$ MPa $\gamma_{Mf} = 1.35$

$\Delta\tau_{R.2} := \dfrac{C_{para.2}}{\sqrt{3}}$ $\Delta\tau_{R.2} = 64.663$ MPa section 1.4 d) in prEN 1993-1-9:2002

$\dfrac{\Delta\tau_{R.2}}{\gamma_{Mf}} = 47.899$ MPa

$$\Delta\tau_{71.2} := \Delta V \cdot \frac{\left(A_{flange} \cdot e_{flange} + A_{stiff} \cdot e_{stiff}\right)}{I \cdot t_w}$$



Over the support we get from Matlab calculation

$\Delta V := 498 + 3.3$ $\qquad \Delta V = 501.3$ kN

$e_{flange} := 504 + \frac{33}{2}$ $\qquad e_{flange} = 520.5$ mm

$e_{stiff} := 504 - 185$ $\qquad e_{stiff} = 319$ mm

$A_{flange} := 1150 \cdot 33$ $\qquad A_{flange} = 38000$ mm^2

$A_{stiff} := 3950$ mm^2

$I = 0.02838$ m^4

$t_w := 12$ mm

$$\Delta\tau_{71.2} := \frac{\left[\Delta V \cdot 10^{-3} \cdot (38000 \cdot 521 + 3950 \cdot 319) \cdot 10^{-9}\right]}{0.02838 \cdot 0.012} \qquad \Delta\tau_{71.2} = 30.997 \text{ MPa}$$

$\gamma_{Ff} \cdot \Delta\tau_{71.2} = 30.997$ MPa $<$ $\frac{\Delta\tau_{R.2}}{\gamma_{Mf}} = 47.899$ MPa OK!

2.7.1.4 Compilation of stresses for the butt weld

The weld will be checked for the worst case, maximum stress due to shear and maximum stress due to wheel pressure.

$\Delta\tau_{71.2} = 30.997$ MPa

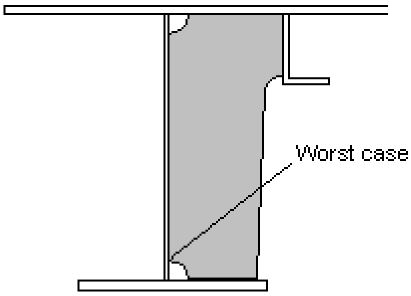
$\Delta\sigma_{71.1} = 24.283$ MPa (Whole upper flange)

In case of combined stress, section 8 (2) in prEN 1993-1-9:2002.

$$\left(\gamma_{Ff} \cdot \frac{\Delta\sigma_{71.1}}{\frac{\Delta\sigma_{R.1}}{\gamma_{Mf}}}\right)^3 + \left(\gamma_{Ff} \cdot \frac{\Delta\tau_{71.2}}{\frac{\Delta\tau_{R.2}}{\gamma_{Mf}}}\right)^5 = 0.606 \quad \leq \quad 1.0 \qquad \text{OK!}$$

2.7.2 Fatigue strength of fillet weld between web and stiffener

2.7.2.1 Stress range due to bending



Detail catagory 80, l<50mm, 7) Vertical stiffeners welded to a beam or plate girder

$C_3 := 80 \quad \text{MPa}$ $\qquad \gamma_{Mf} = 1.35$

$n := 2 \cdot 10^6$ (For comparison)

$\Delta\sigma_{R.3} := C_3$ $\qquad \Delta\sigma_{R.3} = 80 \quad \text{MPa}$

$$\frac{\Delta\sigma_{R.3}}{\gamma_{Mf}} = 59.259 \quad \text{MPa}$$

$I3 := 0.02838 \quad m^4$

$e3 := 766 - 45$ $\qquad e3 = 721 \quad \text{mm}$

e3 is the distance between the weld and the global gravity center.

$$W_s := \frac{I3}{e3 \cdot 10^{-3}} \qquad W_s = 0.039 \quad m^3$$

$\Delta M := 2085 + 34$ $\qquad \Delta M = 2119 \quad \text{kNm}$ (max moment variation in mid section)

$$\Delta\sigma_{71.3} := \frac{\Delta M \cdot 10^{-3}}{W_s} \qquad \Delta\sigma_{71.3} = 53.834 \quad \text{MPa}$$ (perpendicular)

$$\gamma_{Ff} \cdot \Delta\sigma_{71.3} = 53.834 \quad \text{MPa} \quad < \quad \frac{\Delta\sigma_{R.3}}{\gamma_{Mf}} = 59.259 \quad \text{MPa}$$ **OK!**

2.7.2.2 Stress range due to shear

$n_n := 2 \cdot 10^6 \quad \text{cycles}$ For comparison

Detail category 80

$C_3 = 80 \quad \text{MPa}$

$$\Delta\tau_{R.3} := \frac{C_3}{\sqrt{3}} \qquad \Delta\tau_{R.3} = 46.188 \quad \text{MPa}$$ section 1.4 d) in prEN 1993-1-9:2002

$$\frac{\Delta\tau_{R.3}}{\gamma_{Mf}} = 34.213 \quad \text{MPa}$$

$\Delta V = 501.3 \quad kN$ (Max shear force close to the support)

$$\Delta\tau_{71.3} := \frac{\left[\Delta V \cdot 10^{-3} \cdot 630 \cdot 45 \cdot \left(766 - \frac{45}{2}\right) \cdot 10^{-9}\right]}{0.028382 \cdot 0.005} \qquad \Delta\tau_{71.3} = 37.232 \quad MPa$$

$$\gamma_{Ff} \cdot \Delta\tau_{71.3} = 37.232 \quad MPa \quad > \quad \frac{\Delta\tau_{R.3}}{\gamma_{Mf}} = 34.213 \quad MPa$$

NOT OK!

The fillet weld between the stiffener and the web <u>does not</u> make the requirements for shear capacity, set out by Eurocode

2.7.3 Fatigue strength of holes in upper flange

Detail category 90 in prEN 1993-1-9:2002, Table 8.1, Structural element with holes subjected to bending and axial forces

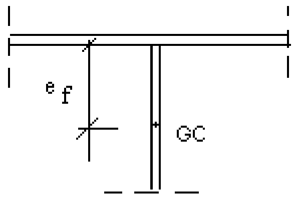
$C_4 := 90 \quad MPa \qquad \gamma_{Mfh} = 1.15$

$\Delta\sigma_{R.4} := 90 \quad MPa$

$$\frac{\Delta\sigma_{R.4}}{\gamma_{Mfh}} = 78.261 \quad MPa$$

$\Delta M = 2119 \quad kNm$

$$e_f := 1225 + 45 - 766 + \frac{32}{2} \qquad e_f = 520 \quad mm$$



$I := 0.02838$

$$W1 := \frac{I}{e_f \cdot 10^{-3}} \qquad W1 = 0.055 \quad m^3$$

$$\Delta\sigma_{71.4} := \frac{\Delta M \cdot 10^{-3}}{W1} \qquad \Delta\sigma_{71.4} = 38.826 \quad MPa$$

$$\gamma_{Ff} \cdot \Delta\sigma_{71.4} = 38.826 \quad MPa \quad < \quad \frac{\Delta\sigma_{R.4}}{\gamma_{Mfh}} = 78.261 \quad MPa$$

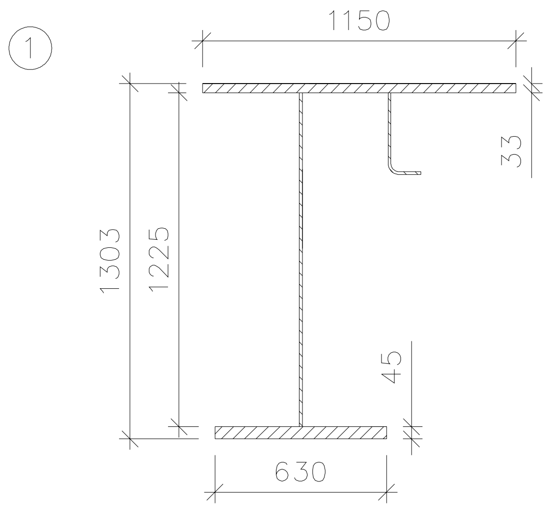
OK!

Appendix C

Translated version of original calculation

Translated version of original Calculation

The main beam is calculated as simply supported with half the bridge contributing



Cross-sectional constants

Section nr	Section name	Web		Upper flange		Lower flange		Centre of gravity
		t	h	t	b	t	b	
		mm	mm	mm	mm	mm	mm	mm
1	Main beam + stiff	12	1225	33	1150	45	630	524
2	Stiffener	10	290	0	0	10	105	185
3	Stiff + upper flange	10	290	33	560	10	105	19

Section nr	Section name	Area	Weight	Ix	W_{upper}	W_{lower}
		m2	kg	m4	m3	m3
1	Main beam + stiff	0,085	680	0,028	0,05099	0,03802
2	Stiffener	0,004	32	4E-05	0,0002	0,00033
3	Stiff + upper flange	0,0224	179	2E-04	0,0033	0,00061

Distance from centre of gravity to lower edge of beam 1225+45-504=766

3. Actions

Actions are calculated on half the bridge

3.1 Selfweight

Main beam in weight/m		680	kg/m
For load combination IV loadfactor = 1.05			
Load factor	1,05		
Gravity constant	10 m/s^2	7,14	kN/m

Cross beams	1.4*44.4/6	10,36	
Stiffener	15*8*0.28*1.225/3	13,72	
Stiffener	15*8*0.2*1.225/6	4,9	
Cantilever	10*8*0.45*1.8/6	10,8	
UPE 180	19.7*3	59,1	
Iron bar walkway	35*1.4	49	
Railing	60*2	120	
Screws and nuts	2*30*8*0.18*0.46/0.6	66,24	
Cables		50	
		384,12	kg/m
	Load comb IV* 1.05 =	4,03	kN/m

3.2 Traffic load

The bridge is designed according to TRAINLOAD LM 2000, Temporary load directions for railway bridges 1998-03-16

At design of the main beams, it is assumed that the rail can be displaced by: 20 mm

Distance between the webs = 1,58 m

Increase of load, caused by rail displacement F = (1.58/2+0.02)/1.58 F = 0,513

The bridge is placed on a straight line with a span length of L = 18m L = 18 m

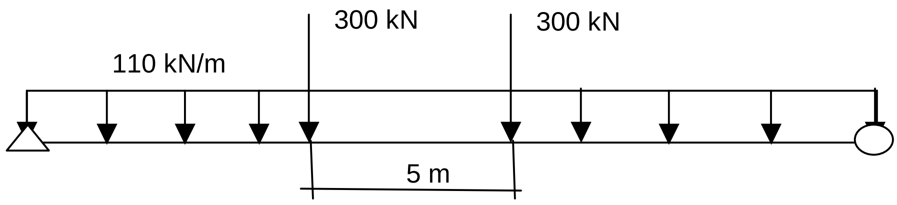
Dynamic contribution, $D = 1+4/(8+L_{best})$, where L_{best} = 18m D = 1,154

Load factor, Load combination IV =	1,4	LM 2000
Load factor, Load combination IV =	1,2	SW/2
Load factor, fatigue	0,8	Fatigue

Design load in load combination IV Fdim*P Fdim = F*D*LF = 0,828 LM 2000

Frdim (Fatigue) = 0,473 Fatigue

The traffic load consists of one uniformly distributed load (110 kN/m)
and two axle loads (300 kN) with cc = 5m



P =	300kN	2pcs
q =	110kN/m	

Design load in load combination IV $P_{dim} = P*F_{dim} = 300*F_{dim}$ $P_{dim} =$ 248,44kN

$q_{dim} =$ 91,10kN/m

For fatigue, the same load is used, but with the load factor 0,8 $P_{rdim} =$ 141,97kN

$q_{rdim} =$ 52,05kN/m

3.3 Traffic load SW/2

The bridge should also be able to take trainload SW/2, static load, with the load factor 1,2
This will not be used in design because it gives a smaller value then LM 2000

Train load SW/2 is a uniformly distributed load 150kN/m

Total load LM 2000 (110*18+300*2)*1,4*1,154 = 4167,7kN

Total load SW/2 150*18*1.2*1.176 = 3810,2kN

The difference will become even bigger if moment and shear force is calculated.

3.2 Derailment load

According to BVBRO 221.36 it assumes that derailment load LM 2000 shall be
displaced in the transverse direction by a measure that in this case is
determent by the derailment protection to 300mm

This is an accident case with load factor = 0,8
witch gives the design force:

$P_{dim} = 300*0.8*(1580/2+300)/1580*D$ $P_{dim} =$ 191,0kN

The derailment force will not be the design force for the main beam,
but it will be used for local design

3.3 Nosing force

According to BVBRO 221.2223, the bridge shall be designed for a single nosing force (80 kN) which is acting at the top of the rail.

Due to the fact that the shear centre is located slightly above the upper edge of the rail, a load in the transverse direction will give a small contribution to the vertical load.
This load will not be used for design of the main beam,
but will be used for design in the transverse direction and for design of local parts

3.4 Walkway load

According to BVBRO 221.223 the walkway is not in use for pedestrians,
therefore this load (3.0 kN/m^2) is not considered for the main beam

3.5 Wind load

According to BVBRO 221.27		Characteristic load	1,8	kN/m2
height of bridge	1,525 m			
height of train	4 m	reduction factor =	0,6	
Characteristic load	Load on bridge	1.525*1,8 =	2,745	kN/m
Characteristic load	Load on train	4*1.8*0.6 =	4,32	kN/m

Calculation on the safe side. Assume the shear centre is located in line with the top edge of the rail.

Vertical load increase on beam with no wind directly on it.

(4.32*2-2.75*1.525/2)/1.58 = 4,14 kN/m

In load combination IV with trainload, this is calculated with a load factor = 0,6

qwind,dim = 0,6*4,1436 = 2,486 kN/m

3.6 Horizontal force

On the safe side the main beam is designed for a longitudinal horizontal force
in combination with the traffic load

Pmax = 1200
Pmin = 0

The force is acting 50 mm below the lower flange with gives

Mmax = 1200*0,6 = 720 kNm

Design of main bridge structure

Bending Moment
The largest magnitude of the bending moment, is in the middle of the simply supported beam.

$$\text{Self} - \text{weight} \qquad \frac{18^2}{8}\cdot(7.14+4.03) = 452.385 \quad \frac{\text{kN}}{\text{m}}$$

$$\text{Wind} - \text{load} \qquad \frac{18^2}{8}\cdot 2.48 = 100.44 \qquad \frac{\text{kN}}{\text{m}}$$

Break- and Accalerationforce
Since, the total moment of this force is negative, it is not added to the calculations.

Train load

$$M := 0.828\left[110\cdot\frac{18^2}{8} + 300\cdot\left(9-\frac{5}{2}\right)\right] \qquad M = 5.303\times 10^3 \ \text{kNm}$$

Total moment in mid section

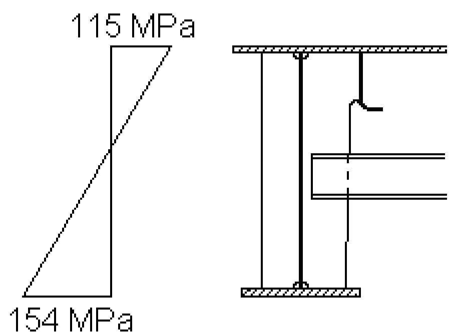
$$\text{Mtot} := 452 + 100 + 5303 \qquad \text{Mtot} = 5.855\times 10^3 \quad \text{kNm}$$

The total moment

This give the following stresses.

$$\sigma\text{tf} \qquad \frac{5.868}{0.05099} = 115.081 \quad \text{Mpa}$$

$$\sigma\text{bf} \qquad \frac{5.868}{0.03802} = 154.34 \quad \text{MPa}$$



Check of longditudinal stiffeners

top flange $560\cdot 33 \quad \text{mm}^2$

$$\sigma\text{tf} := 115.081 \quad \text{MPa}$$

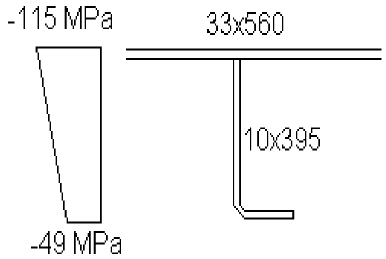
The stress is linear over the cross section,

$$1303mm - \frac{45mm}{2} - \frac{33mm}{2} = 1.264m$$

$$\frac{(115 + 154)}{1.264} = 212.816$$ MPa / meter

Stress in bottom flange

$$\sigma bottom := -115 + \left(\frac{33}{2} + 295\right) \cdot 0.212816$$



$$\sigma bottom = -48.708$$ MPa Compression

Average stress in the longitudinal stiffener

$$\frac{-(115 + 49)}{2} = -82$$ MPa

Total compression force in longitudinal stiffener

$$33 \cdot 560 \cdot 115 + 285 \cdot 10 \cdot 82 + 110 \cdot 10 \cdot 49 = 2.413 \times 10^{6}$$ N

The longditudinal stiffeners are stiffened every third meter.
Calculation of the flexural buckling.

$$I3 := 0.00017$$

$$A3 := 0.0224$$

$$i := \left(\frac{I3}{A3}\right)^{0.5} \qquad i = 0.087 \text{ m}$$

$$Lc := 3 \text{ m}$$

$$\lambda c := \frac{Lc}{\pi \cdot i} \cdot \sqrt{\frac{345}{210000}} \qquad \lambda c = 0.444$$

$$\omega c := 0.86$$ Buckling curve C

Allowable Stress:

$$0.86 \frac{345}{1.2} = 247.25 \text{ Mpa}$$

Average stress in the cross-section

$$2.413\cdot 10^{6}\cdot\frac{1}{\left(33.560\cdot 10^{-6}+10.395\cdot 10^{-6}\right)}=1.076\times 10^{8}$$ Pa

$247.25 > 107$ MPa OK!

Stresses caused of the Nosing Force.

The nosing force acts approximately 220mm over the gravity center of the upper flange

This is balanced by two forces, one in the major web and one in the longitudinal stiffener

$\text{Force} := \frac{(80\cdot 220)}{330}$ $\text{Force} = 53.333$ kN

The rail is located with c/c 1507mm which means 753mm from CL Bridge
The webs are located 790mm from CL Bridge
Add the excentricity of 20mm

This gives the eccentricity of: $790 - 753 + 20 = 57$ mm

Calculate for one axle force $\frac{300}{2} = 150$ kN

$\text{Lbest} := 0$ On the safe side

$D := 1 + \frac{4}{(8+0)}$ $D = 1.5$

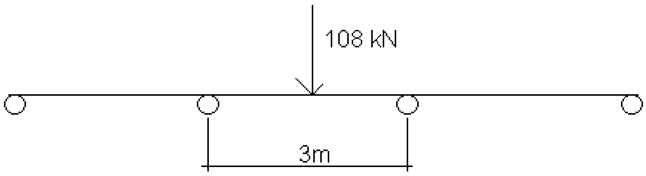
Load factor = 1,4

Moment : $0.057\cdot 150\cdot 1.5\cdot 1.4 = 17.955$ kNm

This gives a vertical force of $\frac{18}{0.33} = 54.545$ kN

The total force $55 + 53 = 108$ kN

Calculate the longitudinal stiffener as a continuous beam with support every third meter



Approximate calculation of the M = P*L*0.18

$M := 108 \cdot 3 \cdot 0.18 \qquad M = 58.32 \ \text{kNm}$

Stresses in the longitudinal stiffener

$Wl := 0.00061 \ m^3 \qquad Wu := 0.0033 \ m^3$

Lower edge $\sigma stiff := \frac{M}{Wl}$ $\sigma stiff = 9.561 \times 10^4$ (tension) OK !!

Upper edge $\sigma flange := \frac{M}{Wu}$ $\sigma flange = 1.767 \times 10^4$ (compression)

Buckling of the plate between the stiffeners

Stresses in the longitudinal plate due to eccentricity and nosing force,

ca 95 Mpa

Global bending 115 Mpa

$E_k := 210000$

$f_{yk} := 345$

Total stress 210 MPa

The free length of the flange

$2 \cdot 780 - 2 \cdot 330 - 10 - 2 \cdot 5 = 880$ 900mm

$$\beta_{fel} := 1.14 \sqrt{\frac{E_k}{f_{yk}}} \qquad \beta_{fel} = 28.126$$

$$\beta_f := \frac{900}{33} \qquad \beta_f = 27.273$$

$Bfel > Bf$ and the the top flange is not fully used, $\frac{345}{1.2} > 210$

Also calculate lateral bucklig in the stiffened flange, and add one third of the stiffeners webb, not including the nosing force and the eccenticity.

$\sigma := -71$ MPa

$$-\left[\frac{(115+49\cdot 2)}{3}\right] = -71 \quad \text{MPa}$$

$A2 := 2050$ mm2

$$205\cdot 10 = 2.05\times 10^{3} \quad \text{mm}^{2}$$

$e2 := \frac{(110\cdot 10\cdot 50)}{2050}$ $\qquad$ $e2 := 26.829$ mm

$I2 := 26.829^{2}\cdot 95\cdot 10 + 23^{2}\cdot 110\cdot 10 + \frac{\left(10\cdot 110^{3}\right)}{12}$ $\qquad$ $I2 = 2.375\times 10^{6}$ mm^{4}

$i2 := \sqrt{\frac{I2}{A2}}$ $\qquad$ $i2 = 34.036$ mm

$Lc := 3000$ mm

$\lambda_c := \frac{Lc}{\pi\cdot i2}\cdot\sqrt{\frac{f_{yk}}{E_k}}$ $\qquad$ $\lambda_c = 1.137$

$\omega c := 0.44$ $\qquad$ Buckling curve C

$N_{Rd} := 0.44\cdot\frac{345}{1.2}\cdot 2050\cdot 10^{-3}$ $\qquad$ $N_{Rd} = 259.325$ kN

$\left(49+\frac{71}{2}\right)$

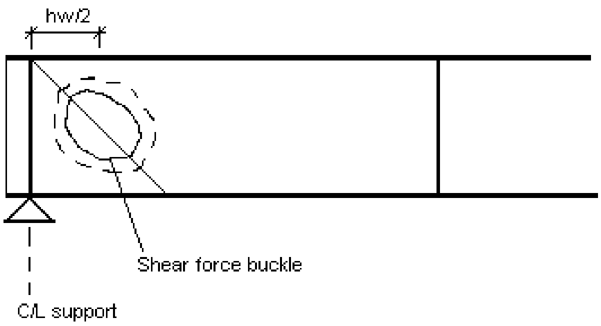
$N_{Sd} = 49\cdot 110\cdot 10 + 60\cdot 95\cdot 10$ $\qquad$ $N_{Sd} := 117$ kN

$N_{Rd} > N_{Sd}$ OK!

The elastic hinch that the web offer the flange, has been ignored.

Sher Force

The magnitude of the Shear Force is taken from the system calculation. The buckling length is set to half the beam height, starting at the support.



The system calculation, chapter 7, gives.
by one tenth of element 2,

$x := 0 + \frac{6.0}{10}$ $\quad x = 0.6 \quad m \quad => \quad Vsd := 1.29\ MPa$

$hw := 1.225$
$tw := 12$

vertical web stiffeners every third meter

K 18:26 gives

$$\kappa_{\tau} = 5.34 + \frac{4}{\left(\frac{3}{1.225}\right)^2} = 6.01$$

$$\lambda_w = \frac{0.81}{\sqrt{6.01}} \cdot \frac{1225}{12} \cdot \sqrt{\frac{355}{210000}} = 1.387$$

$$\omega_v = \frac{0.5}{1.39} = 0.36$$

$$V_{Rd} = 0.36 \cdot \frac{355}{1.2} \cdot 1225 \cdot 12 = 1.566 \times 10^6 \quad N \qquad Vrd := 1.57 \quad MN$$

$Vrd > Vsd$ $\quad$ OK!

Butt welds between web and upper flange

Butt welded connection against upper flange

Calculate weld as if weld has the same
strength as the material

S 355, $f_{uk} := 490$ MPa $\gamma_n := 1.2$

$\gamma_m := 1.2$

$f_{wd} := 0.6 \cdot 0.9 \cdot \frac{490}{1.2 \cdot 1.2}$ $f_{wd} = 183.75$ MPa (BSK 99, eq 6:32a)

For upper butt weld

$\tau_{Sd.paralell} = \frac{(V \cdot S)}{I \cdot b} = 0.0615 V$

$x = 0$ m

$V := 1370$ kN

$I := 0.02838$ m^4

$b := 0.012$ m

$S := A_{flange} \cdot e_{flange} + A_{stiff} \cdot e_{stiff}$

$S := 38000 \cdot 521 + 3950 \cdot 319$ $S = 2.106 \times 10^7$ mm^3

$\tau_{Sd} := 85$ MPa << $f_{wd} = 183.75$ MPa OK!!

Max allowed Displacement

According to BVH BRO the max displacement is limitid to L / 800

For this bridge $L := 18m$

$$\frac{L}{800} = 0.023m$$

This is valid in load combination V:C with $\psi\gamma$ = 1,0

This is checked in the computer calculation and gives a displacement of 22,4mm. This is very close to the allowed value, however, when considering the wind-load it was assumed that the shear center was located in line with the upper edge of the rail, but it is acctualy located 300mm above the upper edge of the rail.
This make the calculation on the safe side.

4.5 Fatigue

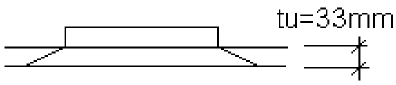
The bridge is designed for LM 2000, with the load factor 0,8

which gives $\frac{0.8}{1.4} = 0.571$ % of the load from load combination IV

Fatigue from axle pressure

When the wheels is acting on the plates 180mm x 460mm witch are resting on the upper flange, there will be a change in stress distribution veritically on the butt weld

Load distribution 1:2,5 gives:



$tw := 12 \quad mm$

The compression surface will be tw*(180+2*2,5*33) = 12*345 mm

Two weels  D

Wheel pressure = $300 \cdot 0.5 \cdot 1.154 \cdot 0.8 = 138.48 \quad kN$

Stress width = $\frac{138}{12 \cdot 345} = 0.033 \quad MPa \quad = \sigma_{rd,parallel}$

Fatigue parameter according to BV BRO 221.2218

$\kappa := \frac{2}{3}$

Wheel pressure : $n := 1 \cdot 10^7$ cycels

Detail 22 in BSK 99 (page 182) gives

$C_{parallel}$ = 100 MPa

$C_{perendicular}$ = 71 MPa

$n = 1 \times 10^7$ and $\kappa := \frac{2}{3}$ gives

frd = frk / (1,1x1,2) = $\frac{66.8}{1.1 \cdot 1.2} = 50.606 \quad MPa$

The change of stresses from bending gives (with 1x10^6 cycles)

Distance from lower GC to lower edge of beam

σrd,parallel = ΔM / W$_{buttweld}$

$$W = \frac{I}{e} = \frac{0.02838}{1.225 + 0.045 - 0.766} = 0.056 \quad m^3$$

From chapter 7 in system calculation

$\Delta M := 3031 + 31$ $\quad\quad$ $\Delta M = 3.062 \times 10^{3}$ kNm

$$\sigma_{rd.parallel} := \frac{\left(3062 \cdot 10^{3}\right)}{0.0563} \qquad \sigma_{rd.parallel} = 5.439 \times 10^{7} = 54 \ \text{MPa}$$

$F_{rd.parallel} := 138$ MPa

$$\left(\frac{54}{71.2}\right)^{2} + \left(\frac{33}{50.6}\right)^{2} = 0.58 + 0.42 = 1.0$$

From the shear stresses the Fatigue width is recived

$$\tau_{rd} := \Delta V \cdot \frac{\left(A_{flange} \cdot e_{flange} + A_{stiff} \cdot e_{stiff}\right)}{I \cdot t_{w}}$$

At beam end we get from (chapter 7, EC 2,0)

$\Delta V := 712 + 4$ $\quad\quad$ $\Delta V = 716$ kN

From chapter 2 we get

$e_{flange} := 504 + \frac{33}{2}$ $\quad\quad$ $e_{flange} = 520.5$ mm

$e_{stiff} := 504 - 185$ $\quad\quad$ $e_{stiff} = 319$ mm

$A_{flange} := 1150 \cdot 33$ $\quad\quad$ $A_{flange} = 38000 \ \text{mm}^{2}$

$A_{stiff} := 3950 \ \text{mm}^{2}$

$I = 0.02838 \ \text{m}^{4}$

$t_{w} := 12$ mm

$$\tau_{Rd.parallel} := \frac{\left[716 \cdot 10^{3} \cdot (38000 \cdot 521 + 3950 \cdot 319) \cdot 10^{-9}\right]}{0.02838 \cdot 0.012}$$

$\tau_{Rd.parallel} = 4.427 \times 10^{7} = 44$ MPa

$f_{rvd} := 0.6 \cdot f_{rd.parallel} = 0.6 \cdot 138 = 82.8$ MPa

Compilation Fatigue

Although we have taken the worse section for shear force width,
we take care of this for max bending stress

$\tau_{Rd.parallel} = 44$ MPa $\sigma_{Rd.parallel} := 54$ MPa

$\sigma_{Rd.perpendicular} := 33$ MPa (Whole upper flange)

BSK 99, 6:512c

$$\sqrt{\frac{\tau_{Rd.para}^{2}}{f_{rvd}^{2}} + \frac{\sigma_{Rd.para}^{2}}{f_{rd.para}^{2}} + \frac{\sigma_{Rd.perp}^{2}}{f_{rd.perp}^{2}}} \le 1.10$$

$$\sqrt{\left(\frac{44}{83}\right)^2 + \left(\frac{54}{138}\right)^2 + \left(\frac{33}{50.6}\right)^2} = 0.927 \quad < \quad 1.10 \qquad \text{OK!}$$

Fatigue in the lower flange
In the lower flange the stiffener is the worst case, in bending
Def. 44 BSK 99, Wb,C=71

$n := 10^6$

$\chi := \frac{2}{3}$

$f_{rdpar} := \frac{129}{(1.2 \cdot 1.1)}$ $\qquad f_{rdpar} = 97.727$ MPa

$I3 := 0.02838$ m^4

$e3 := 766 - 45$ $\qquad e3 = 721$ mm

$W_s := \frac{I3}{e3}$ $\qquad W_s = 3.936 \times 10^{-5}$ m^3

$deltaM := 3031 + 31$ $\qquad deltaM = 3.062 \times 10^3$ Nm

$$\sigma_{rdpar} := \frac{deltaM}{W_s} \qquad \sigma_{rdpar} = 7.779\times 10^7 \quad Pa$$

$$f_{rdpar} > \sigma_{rdpar} \qquad (0.0251 \cdot deltaM) = 76.856$$

For shear the capacity is:

$$f_{rvd} := 0.6 \cdot 98 \qquad f_{rvd} = 58.8 \quad MPa$$

$$deltaV := 716 \quad kN$$

$$\tau_{rdpar} := \frac{\left[716 \cdot 630 \cdot 45 \cdot \left(766 - \frac{45}{2}\right) \cdot 10^{-6}\right]}{0.028382 \cdot 0.005} \qquad \tau_{rdpar} = 5.318\times 10^7 \quad Pa$$

$$f_{rvd} > \tau_{rdpar} \qquad (0.074 \cdot deltaV) = 52.984$$

Combine the worst bending stresses with the worst shear stresses, on the safe side

$$UTN := \left[\left(\frac{\tau_{rdpar}}{f_{rvd}}\right)^2 + \left(\frac{\sigma_{rdpar}}{f_{rdpar}}\right)^2\right]^{\frac{1}{2}} \qquad UTN = 1.205\times 10^6$$

$$UTN > 1.10$$

Check at 2.4, 3.6, 4.8, 6.0 m

x	deltaV	\t.rdpar	deltaM	\s.rdpar	UTN
2,4	570	42	1481	37	0,81
3,6	510	38	2040	51	0,83
4,8	457	34	2477	62	0,88
6,0	465	34	2794	70	0,92
7,2	449	33	2990	75	0,95
8,4	440	33	3068	77	0,96
9,0	439	33	3062	77	0,96

Fatigue in holes in upper flange

Detal 8, Distance to the edge >3d gives

$C_{par} := 80$

$f_{rdpar1} := \frac{146}{(1.2 \cdot 1.1)}$ $\quad f_{rdpar1} = 110.606$ MPa

$delta1M := 3068 \ \frac{kN}{m}$ $\quad x1 := 8.4$ m

$e_f := 1225 + 45 - 766 + \frac{32}{2}$ $\quad e_f = 520$ mm

$I := 0.02838$

$W1 := \frac{I}{e_f}$ $\quad W1 = 5.458 \times 10^{-5}$ $\quad$ GC upper flange

$\sigma_{rdpar1} := \frac{delta1M}{W1}$ $\quad \sigma_{rdpar1} = 5.621 \times 10^{7}$ MPa

$\sigma_{rdpar1} < f_{rdpar1}$

The lower fillet weld

$S_s := 630 \cdot 45 \cdot \left(766 - \frac{45}{2}\right)$ $\quad S_s = 2.108 \times 10^{7}$ mm^3

$b_s := 0.01$ m $\quad S_{s1} := 21$

$\tau_{sdpar12} := \frac{1370 \cdot S_{s1}}{I \cdot b_s}$ $\quad \tau_{sdpar12} = 1.014 \times 10^{8}$ MPa

$\tau_{sdpar12} < 183$ MPa

Appendix D - Results from Matlab calculation

Response in ULS

Max moment in mid span, ULS

BV BRO				Eurocode		
x	M(x)	V(x)		x	M(x)	V(x)
[m]	[MNm]	[MN]		[m]	[MNm]	[MN]
0	0	1,198			0,000	1,250
1	1,145	1,095		1	1,194	1,143
2	2,212	0,984		2	2,325	1,028
3	3,122	0,884		3	3,266	0,928
4	3,935	0,783		4	4,156	0,813
5	4,695	0,675		5	4,903	0,712
6	5,303	0,788		6	5,568	0,624
7	5,775	0,300		7	6,075	0,312
8	6,073	0,286		8	6,383	0,303
9	6,129	0		9	6,446	0,000

Max shear force at support, ULS

BV BRO				Eurocode		
x	M(x)	V(x)		x	M(x)	V(x)
[m]	[MNm]	[MN]		[m]	[MNm]	[MN]
0	0,000	1,458		0	0	1,532
1	1,146	1,170		1	1,230	1,226
2	2,125	1,883		2	2,351	0,922
3	3,102	1,870		3	3,254	0,907
4	3,770	0,583		4	3,937	0,603
5	4,280	0,297		5	4,481	0,298
6	4,554	0,248		6	4,738	0,248
7	4,758	0,143		7	4,941	0,140
8	4,852	0,038		8	5,038	0,033
9	4,827	-0,067		9	5,000	-0,074

Max shear force x = 0.6m, ULS

BV BRO				Eurocode		
x	M(x)	V(x)		x	M(x)	V(x)
[m]	[MNm]	[MN]		[m]	[MNm]	[MN]
0	0	1,384		0	0	1,455
0,6	0,829	1,378		0,6	0,872	1,449
1	1,253	1,099		1	1,362	1,152
2	2,406	1,086		2	2,525	1,138
3	3,223	0,799		3	3,378	0,833
4	3,939	0,512		4	4,124	0,529
5	4,504	0,497		5	4,698	0,513
6	4,791	0,211		6	5,007	0,208
7	4,951	0,125		7	5,155	0,120
8	5,021	0,020		8	5,219	0,013
9	4,989	-0,085		9	5,179	-0,095

Max displacement, SLS

Vertical displacement, SLS				
BV BRO			Eurocode	
x	p		x	p
[m]	[mm]		[m]	[mm]
0	0,0		0	0,0
1	3,9		1	3,5
2	7,6		2	6,9
3	11,1		3	10,0
4	14,3		4	12,9
5	17,0		5	15,3
6	19,2		6	17,4
7	20,9		7	18,8
8	21,9		8	19,7
9	22,4		9	20,0

Fatigue, BV BRO

For fatigue, the highest moment with belonging shear force, and vice versa, had to be calculated in some sections.
This vas one by moving the train load BV 2000 along the bridge.

At support		
$V_{max}(0)$	785,3	kN
M(0)	0	kNm

Mid span		
$M_{max}(9)$	3287	kN
V(9)	0	kNm

BV BRO				
x = 3m			x = 6m	
M(3m)	V(3m)		M(6m)	V(6m)
[MNm]	[MN]		[MNm]	[MN]
1,7387	0,4107		2,5849	0,0910
1,8213	0,4577		2,8084	0,1379
1,8393	0,4304		2,8695	0,2668
1,8345	0,5593		2,8764	0,2395
1,8067	0,5320		2,8917	0,2122
1,7862	0,5151		2,9153	0,1849
1,7668	0,5086		2,9472	0,3138
1,7474	0,5021		2,9250	0,2865
1,7279	0,4956		2,9111	0,2592
1,7085	0,4892		2,9056	0,2319
1,6891	0,4827		2,9084	0,3607
1,6696	0,4762		2,8570	0,3334
1,6502	0,4697		2,8140	0,3061

Max moment x =3m		
$M_{max}(3m)$	1839,3	kNm
V(3m)	430,4	kN

Max moment x = 6m		
$M_{max}(6m)$	2947,2	kNm
V(6m)	313,8	kN

Max shear force x = 3m		
$V_{max}(3m)$	559,3	kNm
M(3m)	1834,5	kN

Max shear force x = 6m		
$V_{max}(6m)$	360,7	kNm
M(6m)	2908,4	kN

Max Negative moment with belonging shear force was calculated for one axle load on the edge of the 0.4m cantilever at the end of the bridge

M(3)	-52,055	kNm
M(6)	-41,644	kNm
M(9)	-31,233	kNm

V(3)	-3,47	kN
V(6)	-3,47	kN
V(9)	-3,47	kN

Max stress range for each section

Max delta M		
Delta M(0)	0,0	kNm
Delta M(3)	#REF!	kNm
Delta M(6)	#REF!	kNm
Delta M(9)	#REF!	kNm

Belonging delta V		
Delta V(0)	#REF!	kN
Delta V(3)	#REF!	kN
Delta V(6)	#REF!	kN
Delta V(9)	#REF!	kN

Max delta V		
Delta V(0)	#REF!	kN
Delta V(3)	#REF!	kN
Delta V(6)	#REF!	kN
Delta V(9)	#REF!	kN

Belonging delta M		
Delta M(0)	0,0	kNm
Delta M(3)	#REF!	kNm
Delta M(6)	#REF!	kNm
Delta M(9)	3177,7	kNm

Fatigue, Eurocode

Max moment		
x	M(x)	V(x)
[m]	[kNm]	[MN]
0	0	398
1	381	365
2	743	330
3	1045	299
4	1316	267
5	1573	233
6	1793	206
7	1950	103
8	2065	103
9	2085	0

Max shear force		
x	M(x)	V(x)
[m]	[kNm]	[kN]
0	0	498
1	384	395
2	755	292
3	1057	292
4	1255	189
5	1426	86
6	1499	73
7	1558	40
8	1582	7
9	1569	-26

Negative moment and shear force

Min moment		
M(9)	-34,0	kNm

Min shear force		
V(0)	-3,3	kN

Max variation of moment and shear

Moment variation		
Delta M(9)	2119,0	kNm

Shear force variation		
Delta V(0)	501	kN

Appendix E - Photos

Bridge over Kvillebäcken

Kvillebäcken

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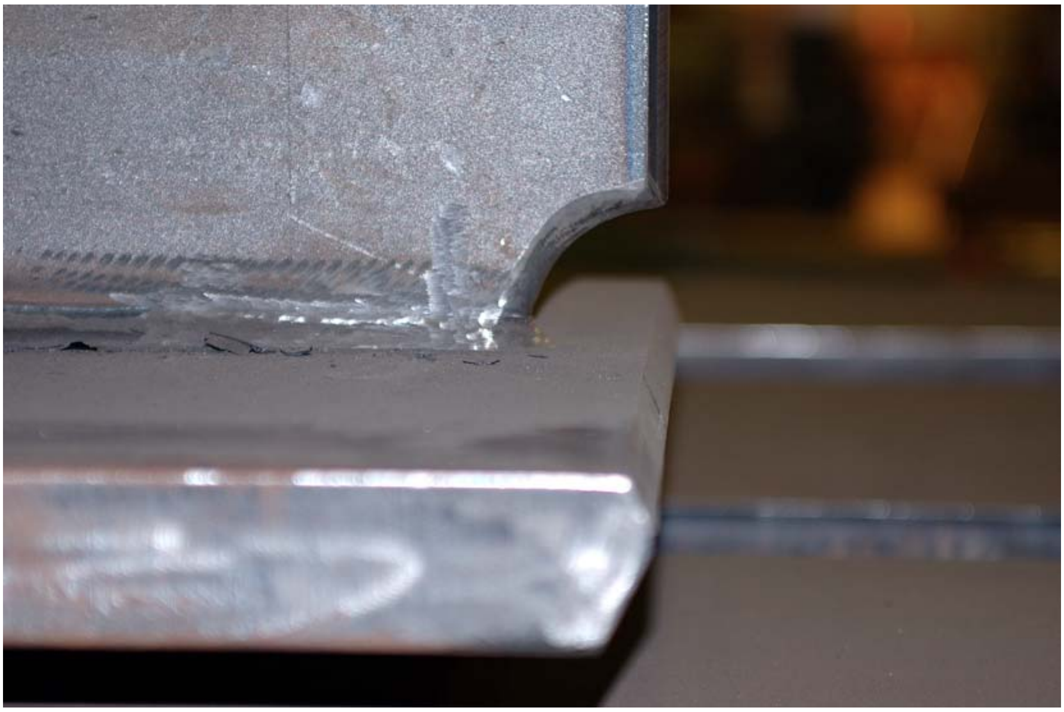

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