CZECH TECHNICAL
UNIVERSITY
IN PRAGUE

FACULTY
OF CIVIL
ENGINEERING

MASTER‘S THESIS

2018

KATEŘINA
SOUKUPOVÁ

ČESKÉ VYSOKÉ UČENÍ TECHNICKÉ V PRAZE
Fakulta stavební
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ZADÁNÍ DIPLOMOVÉ PRÁCE

I. OSOBNÍ A STUDIJNÍ ÚDAJE

Příjmení: Soukupová Jméno: Kateřina Osobní číslo: 410831

Zadávající katedra: Katedra ocelových a dřevěných konstrukcí

Studijní program: Stavební inženýrství

Studijní obor: Konstrukce a dopravní stavby

II. ÚDAJE K DIPLOMOVÉ PRÁCI

Název diplomové práce: Posouzení a zatížitelnost železničního mostu v Praze Motole

Název diplomové práce anglicky: The assesment and the load capacity of the bridge in Prague - Motol

Pokyny pro vypracování:
Provedení prohlídky mostní konstrukce a posouzení jejího stavu.
Dále pak vytvoření prostorového výpočetního modelu a stanovení zatížitelnosti mostu.
Posouzení výsledků podrobného výpočtu a zjednodušeného ručního posouzení.
Zpracování přehledných výkresovů mostu.

Seznam doporučené literatury:
METODICKÝ POKYN PRO URČOVÁNÍ ZATÍŽITELNOSTI ŽELEZNIČNÍCH MOSTNÍCH OBJEKTŮ. 2015.

Jméno vedoucího diplomové práce: doc. Ing. Pavel Ryjáček, Ph.D.

Datum zadání diplomové práce: 9.10.2017 Termín odevzdání diplomové práce: 7.1.2018
Údaj uveďte v souladu s datem v časovém plánu příslušného ak. roku

Podpis vedoucího prace Podpis vedoucího katedry

III. PŘEVZETÍ ZADÁNÍ

Beru na vědomí, že jsem povinen vypracovat diplomovou práci samostatně, bez cizí pomoci, s výjimkou poskytnutých konzultací. Seznam použité literatury, jiných pramenů a jmen konzultantů je nutné uvést v diplomové práci a při citování postupovat v souladu s metodickou příručkou ČVUT „Jak psát vysokoškolské závěrečné práce" a metodickým pokynem ČVUT „O dodržování etických principů při přípravě vysokoškolských závěrečných prací".

10.10.2017
Datum převzetí zadání Podpis studenta(ky)

Statutory declaration

I declare that I have developed and wrote the enclosed master's thesis completely by myself and that I have listed all the information sources used in accordance with the Methodological Guideline of the Ethics of Higher Education Final Works. I also agree with the possible publication of the results of the thesis.

In Prague, 4th January 2018

…………………………………………

signature of the author

Acknowledgement

I would first like to thank my thesis advisor doc. Ing. Pavel Ryjáček, Ph.D. for valuable advice and expert comments when consulting my master's thesis.

I also have to express my deep gratitude to my family, especially my parents, and Eduard Karmazín for providing me with unfailing support and continuous encouragement throughout my years of study and through the process of writing this thesis. This accomplishment would not have been possible without them.

CZECH TECHNICAL UNIVERSITY IN PRAGUE

Faculty of Civil Engineering
Department of Steel and Timber Structures

The assessment and the load capacity of the bridge in Prague - Motol

Posouzení a zatížitelnost železničního mostu v Praze Motole

Master's thesis

Department: Department of Steel and Timber Structures
Study program: Structural and Transportation Engineering

Thesis supervisor at CTU in Prague: doc. Ing. Pavel Ryjáček, Ph.D.
Thesis supervisor at TUM in Munich: Dr.-Ing. Joseph Ndogmo

Kateřina Soukupová

Prague 2018

CZECH TECHNICAL UNIVERSITY IN PRAGUE
Faculty of Civil Engineering
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The assessment and the load capacity of the bridge in Prague - Motol

Master's thesis

A.0. Title sheets

Kateřina Soukupová

Prague 2018

Abstract

The topic of this master's thesis is an assessment of a steel railway bridge at stationing 12,478 km of the existing line Smíchov-Hostivice in Prague. The bridge consists of a box girder with an orthotropic bridge deck and a track ballast. It is a simply supported beam with a span of 42,8 meters carried by four cast-iron bearings on massive concrete abutments. In the area of the bridge the axis of the rail runs in a shape of a spiral. The road under the bridge is horizontally curved. The axes are in angle 35°. First part of the thesis is an inspection and diagnostics of the bridge. Consequently, all parts of steel structure are calculated and load capacity in assessed.

Key words
steel structure
railway bridge
orthotropic bridge deck
load capacity
box girder
buckling and shear lag

Abstrakt

Předmětem této diplomové práce je posouzení ocelového železničního mostu ve staničení 12,478 km stávající trati Smíchov-Hostivice v Praze. Most je tvořen komorovým nosníkem s ortotropní mostovkou a kolejovým ložem. Jedná se o prostě podepřený nosník o rozpětí 42,8m podepřený čtyřmi ocelolitinovými ložisky na masivních betonových opěrách. V místě přemostění je osa železniční trati v přechodnici. Silniční komunikace pod mostem leží v oblouku. Úhel os v místě křížení je 35°. První částí diplomové práce je prohlídka mostní konstrukce a posouzení jejího stavu. Následně jsou posouzeny všechny části ocelové konstrukce a je stanovena zatížitelnost.

Klíčová slova
ocelová konstrukce
železniční most
ortotropní mostovka
zatížitelnost
komorový most
boulení a smykové ochabnutí

Attachment list:

Part A.0: Title sheets
Part A.1: Technical report of bridge inspection
Part A.2: Detailed static analysis

Part B: Preliminary static analysis

Part C: Drawing documentation

CZECH TECHNICAL UNIVERSITY IN PRAGUE
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The assessment and the load capacity of the bridge in Prague - Motol

Master's thesis

A.1. Technical report of bridge inspection

Kateřina Soukupová

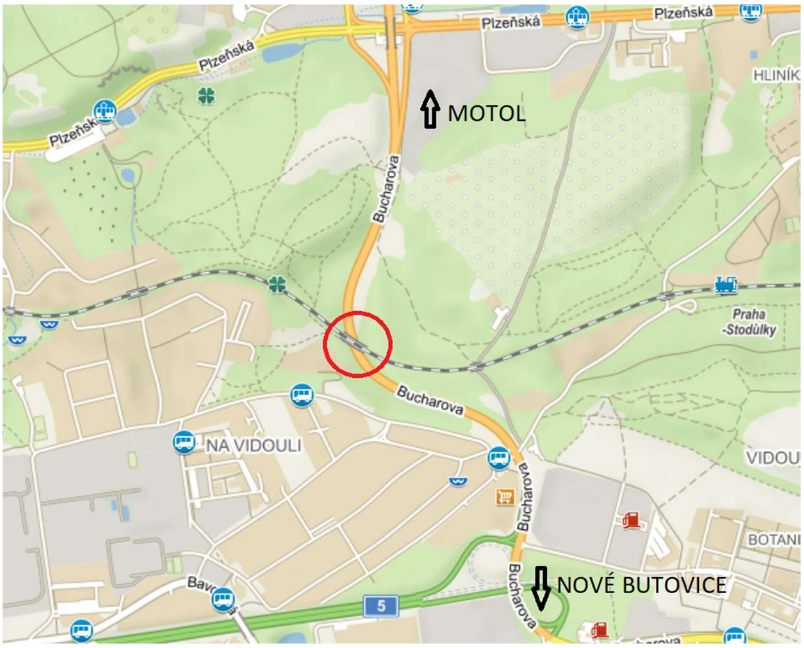
Prague 2018

Contents

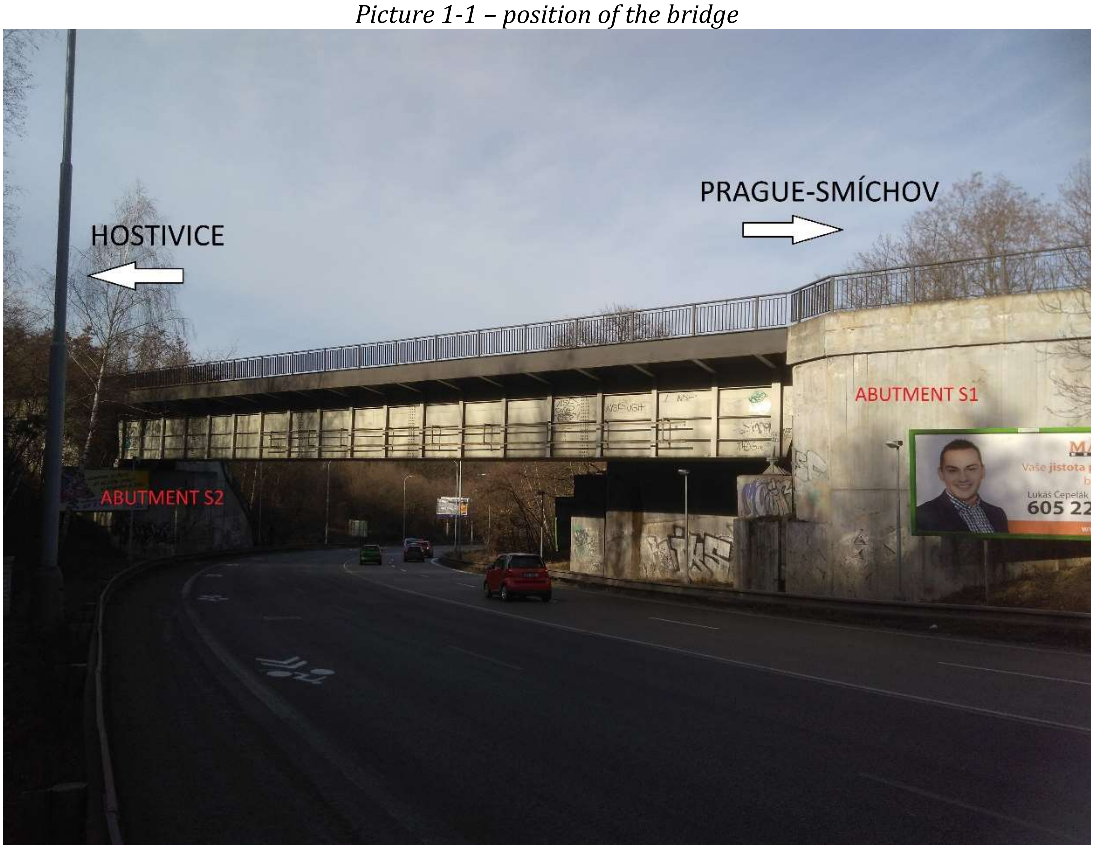
1 Introduction

The topic of my master's thesis is an assessment of a railway bridge on a track between Prague-Smíchov and Hostivice (km 12,478). The bridge was built in year 1980 and it takes place over road JZM Motol in deep cutting. Anticorrosive coating was renewed last time in year 1983. The axis of a single track runs in spiral of curve with radius R=366m. Positive vertical alignment in direction of stationing is 11,2‰.

Under construction goes road in left curve with radius r=200m and vertical alignment is 4,55‰, it rises in direction of stationing. An angle of tangents in place of crossing point is 35°22'. Transverse slope of road is 4,5%. A width road is 16,84m. Head clearance under the bridge is 5,09m.

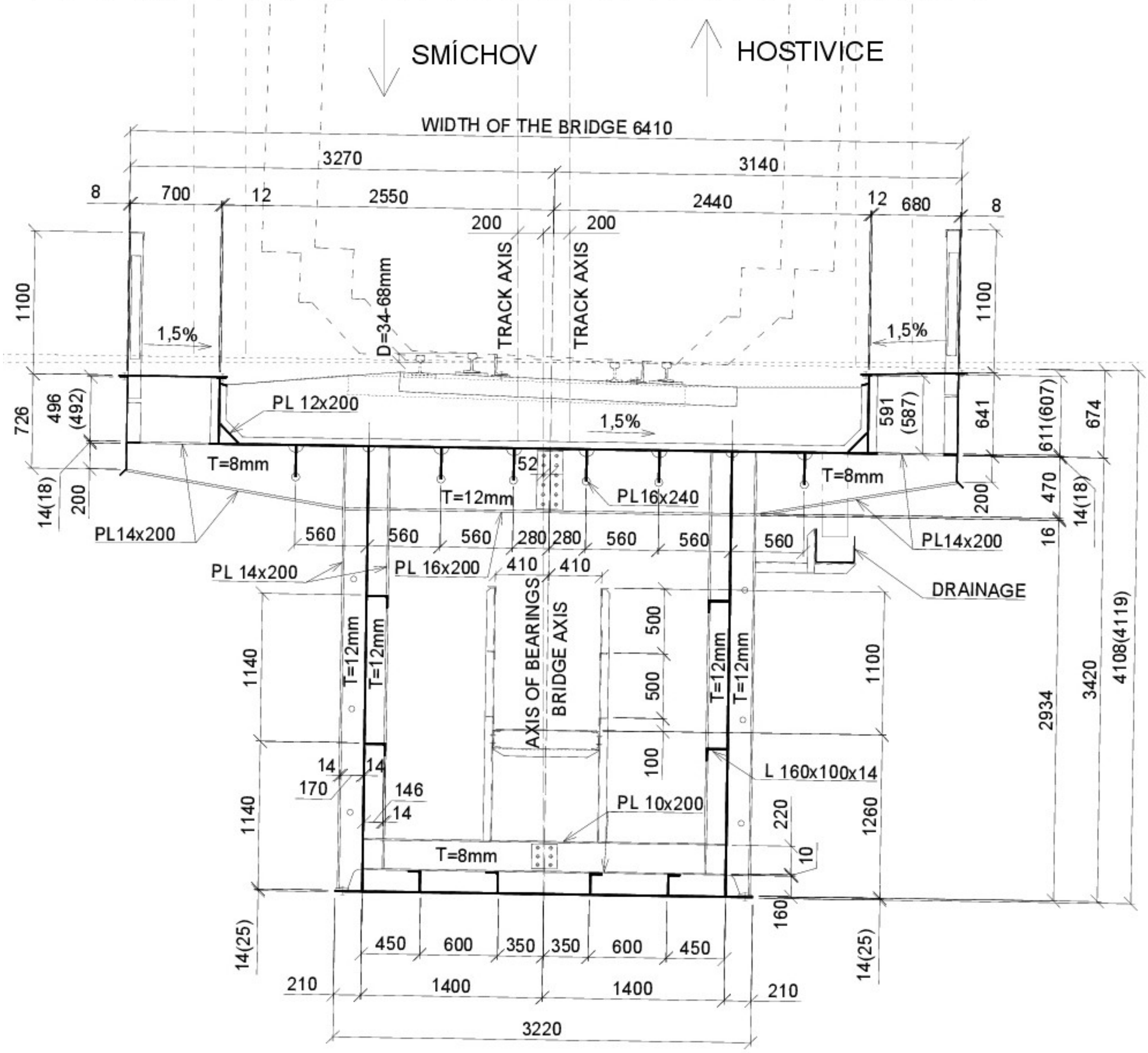


Picture 1-1 – position of the bridge



Picture 1-2 - view of the bridge from the direction of Nové Butovice

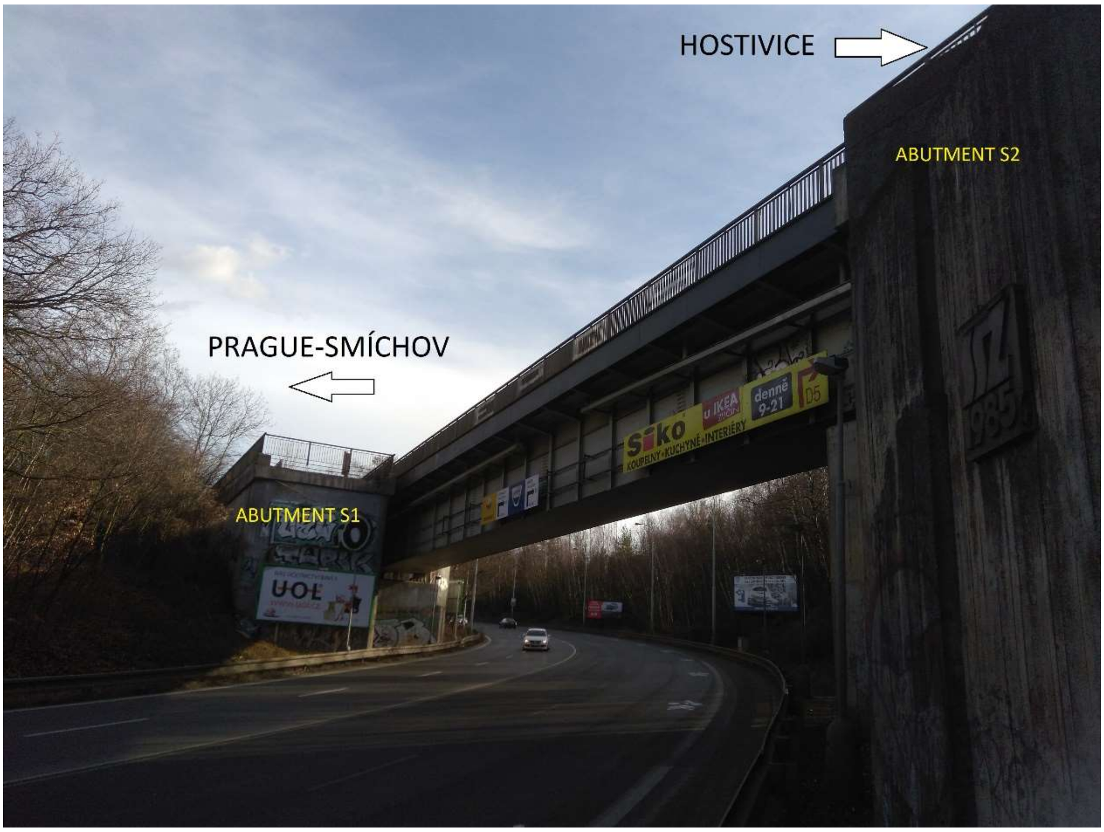
EXEMPLARY CROSS SECTION M1:50



MATERIALS:
LOAD BEARING STEEL STRUCTURE (EXCLUDING UPPER LONGITUDINAL STIFFENERS):
STEEL 37 (S235)
UPPER LONGITUDINAL STIFFENERS: STEEL 51 (S355)
BEARINGS: CAST IRON
SCREW JOINTS: HIGHT STRENGTH BOLTS 10.9

SURFACE WORKING:
OUTSIDE SURFACE:
METALIZATION IN COMPOSITION 70μZn and 150μAl
SYNTHETIC SURFACE COLOR S2013
INSIDE SURFACE:
TWO LAYERS OF COLOR O2005
THREE LAYERS OF COLOR S2302-Plumbinex

STRUCTURE OF TRACK BALLAST:
TRACK BALLAST (FRACTION 16/32, DRUMED GRAVEL) ... MINIMAL THICKNESS 400mm
PROTECTIVE CEMENT SCREED WITH WIRE INSERT ... THICKNESS 50mm
INSULATING INSERT STICKED WITH GLUE... THICKNESS 10mm
PLATE OF BRIDGE DECK



Picture 1-3 - view of the bridge from the direction of Motol

2 Description of construction

A steel box girder bridge is single span. A monorail lies on continuous ballast bed. Span of bridge is 42,8m. The construction has an upper orthotropic bridge deck. The longitudinal stiffeners have open cross section and are placed in distance of 0,56m. Cross girders are 2,4m distant.

Hight of webs of main box girder is 3420mm and their axial distance is 2800m. Flanges and webs are stiffened by cross girders and longitudinal stiffeners. Shape of superstructure is S49. Continuous rail lies on timber sleepers.

Basic parameters of construction:

- Established name: Bucharova
- Coordinates of the middle of construction: 50°03´39.880“N, 14°20´01.9474“E
- Length of bridge: 89,6m
- Length of steel construction: 44,8m
- Span of bridge: 42,8m
- Width of bridge: 6,41m
- Electrification: none
- Critical running speed: 70km/h
- Track class load with associated speed: C3-70
- Angle of crossing: 35°22’
- Skew: left
- Number of rails: 1
- Thickness of ballast bed: 400mm

Construction is welded. For screwed connection of webs of main box girder and upper cross girders are used high strength steel screws. Bridge joints are perpendicular to rail axis.

On support S1 (direction of Prague-Smíchov) is a pair of rocker type expansion bearings and on support S2 (direction Hostivice) are placed fixed bearings.

Picture 2-1 - bolted joints of box girder and cross girder

The condition of the steel structure is good, without any discovered defects. The inside of the chamber beam is dry and without corrosion. The stiffeners and the inspection footbridge are also in order. The bridge bearings on the S1 support are slightly clogged, without corrosion. Concrete under the bearings is cracked and under the left bearing on the side from the back wall is it missing. The bearings on the S2 support are polluted and the rollers are not lubricated but functional. The distance between the main beam and the back wall is 70 mm at the beginning and 115 mm at the end. There are no defects found out by a visual inspection of welds and high strength bolts. The behaviour of the structure is quiet when a train is passing. Inside the box girder is a large amount of waste.

Picture 2-2 – inspection walkway inside a bridge

Picture 2-3 - fixed bearing (type II-P-5)

Picture 2-4 - expansion bearing (type II-V-5)

3 Substructure

Substructure of the bridge consists of massive concrete abutments without surface working (beam seat, back wall and abutment wall) and longitudinal joint. Wing walls are on surface protected by concrete plaster.

Abutment S1: There are traces of running water on the back wall, with vertical shrinkage cracks with a maximum width of 0,2mm. The beam seat is partly covered with waste and irregularly cracked. The concrete under the bearings is in good condition. On the abutment wall, there is a horizontal crack 1 mm wide, the masonry is polluted by illegal graffiti. Surface is irregularly cracked, water leaks through.

On the left wing wall, at the lower part is an area of about 1,0 m^2, where the masonry is fallen off, and an area of about 2,0m^2 is drained and there is a risk of further falling. At the beginning are two vertical cracks with a width up to 1 mm on the entire wing wall height, at the bottom leak a binder. On the cornice, there is a few horizontal and vertical cracks with leakage of the binder (creation of crust). The right wing wall also has horizontal and vertical cracks.

Abutment S2: Places with poorly compacted concrete are visible on the back wall, there are traces of running water and vertical shrinkage cracks with a maximum width of 0,2mm. The beam seat is in good condition. At the top is a horizontal crack 1mm wide.

At the end of the left wing wall the surface is fallen off in areas of 2x0,70m^2. Vertical cracks are full height. On the cornice, there is a few of horizontal and vertical cracks with binder leakage. The right wing wall also has a vertical crack, the concrete is degraded there, the reinforcement cover is not sufficient and the reinforcement corrodes.

Picture 3-1 – bulging wing walls – abutment S1 on the left side and S2 on the right side

Picture 3-2 - water leakage and vertical crack on abutment S2

Picture 3-3 Left: signs of poorly compacted concrete, right: water leakage by expansion joints

4 Superstructure

The track is on the bridge in a right curve with a cant of the track 34-68mm. Rails are welded and their type is S49 with a flat ribbed plate. Sleepers are made of oak timber. There are no rail joints. Between the track ballast and steel structure is protective cement screed of thickness 5cm. There are guard rails with dimension L160x100x14mm. Their distance from running surface of rails is 180-190mm.

The clamping screws of the rail fasteners are not tightened. The sleepers are cracked longitudinally, but otherwise their condition is good. The surface coating of the guard rails is damaged from 10% of the area (classification Ri 4). There is one screw missing in every joint. The track ballast is without defects.

Picture 4-1 - details of superstructure of the bridge

5 Equipment of bridge

5.1 Drainage

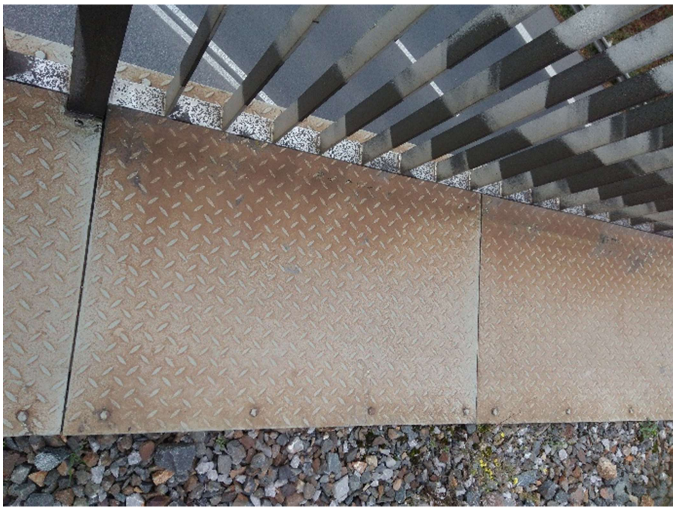
The orthotropic bridge deck is drained by a transverse slope 1,5% and square longitudinal draining channels fastened vertical cross girders. The water is lead along abutments to canalisation.

Drainage of a bridge deck is functional, drainage of expansion joints is damaged and the water leaks through.

Picture 5-1 - details of drainage

5.2 Floors on maintenance walkways

The floors are made of 5mm thick chequer plate attached with screws. The plate is in a good condition (Ri0), some screws are missing and some of them are corroded.



Picture 5-2 - detail of maintenance walkway

5.3 Railing

Railing is made of welded steel. It has vertical filling. Hight of the railing is 1,1m on both sides. First and last vertical profile of railing have safety coating. Length of railing is 90m without dilatation. Vertical profiles are above wing walls embedded into concrete cornice and above steel structure are they welded to ends of upper cross girders.

On the left side is the fourth post from the end of the bridge pushed out of concrete. Outside of the load-bearing structure, some welds of railing are cracked. The railing in the area of steel structure is solid.

5.4 Maintenance equipment

On support S2 (direction Hostivice) is placed a steel ladder. Anchor bolts of ladder are partially torn from the masonry of abutment. Entrance in the box girder is unsecured from both sides. There is a steel walkway inside. At the beginning and the end of walkway are the ladders for descending to the bottom of the box beam.

Picture 5-3 - maintenance ladders

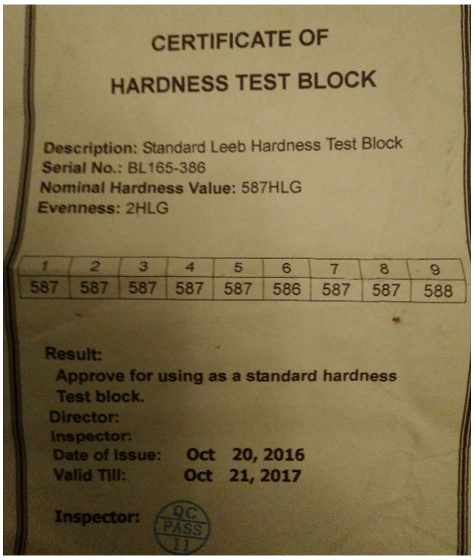
5.5 Other equipment

On the outside of the web of the box girder are fixed handles, on which are fastened steel structures for advertising banners.

6 Verification of used materials

6.1 Used equipment and measurement method

A portable digital KT-C hardness tester with probe D was used to measure hardness of a materials. It allows to measure steel hardness by the Leeb method and it converts it into different scales, also to the strength of steel in MPa.



CERTIFICATE OF
HARDNESS TEST BLOCK

Description: Standard Leeb Hardness Test Block
Serial No.: BL165-386
Nominal Hardness Value: 587HLG
Evenness: 2HLG

1	2	3	4	5	6	7	8	9
587	587	587	587	587	586	587	587	588

Result:
Approve for using as a standard hardness Test block.
Director:
Inspector:
Date of Issue: Oct 20, 2016
Valid Till: Oct 21, 2017
Inspector: QC PASS 11

Picture 6-1 - hardness tester KT-C

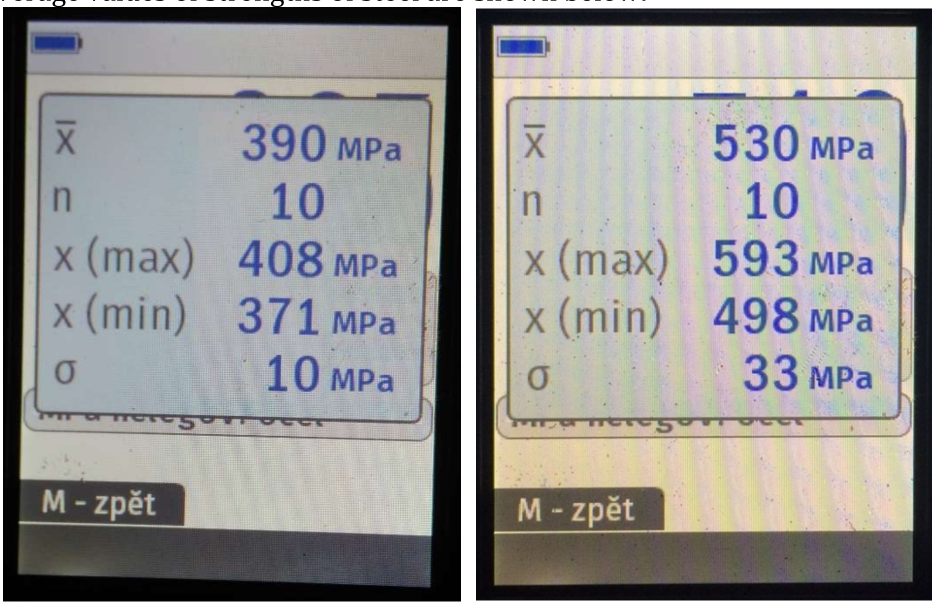
For measurement, it is necessary to remove the coating layer. Therefore, the test place was always polished with an angular grinder for smooth glossy metal, as it is seen the next pictures.

Most of the construction should be made of steel S235 (360MPa tensile strength), only the longitudinal stiffeners of the orthotropic bridge deck should be made of steel S355 (tensile strength 510MPa). Since the materials are listed in the documentation of the bridge, just 10 measurements are done for each material and the average value is calculated from them.

Picture 6-2 – grinded steel surface

6.2 Results of measurement

Average values of strengths of steel are shown below:



Picture 6-3 - average values of strength – flange of box girder (left), longitudinal stiffener (right)

It can be stated, that the longitudinal stiffeners are actually made of steel with a tensile strength of 510MPa and the rest of the structure has a tensile strength 360MPa. The average results are higher than standard values. Deviations are not too big and the lower values are probably caused by imperfect abrasion of the coating.

7 Conclusion

The steel part of bridge construction is in good condition and the quality of materials is verified. Defects of concrete abutments are described above.

All parts of steel structure of the bridge are assessed and the results are introduced in the attachment A.2 Detailed static analysis. The load capacity of main box girder, longitudinal stiffener and cross girder is also calculated in B. Preliminary static analysis, it is compared with results of detailed analysis and evaluated as resembling.

The detailed static analysis consists of list of loads, material characteristics, description of computational model and assessment of all parts of steel structure. Load capacity of main box girder is verified in midspan, cross section above a support and cross section in place, where thickness of flanges changes. Assessments of cross girders, longitudinal stiffeners, stiffeners inside box girder, the plate of bridge deck and joints of these parts are included. In closing is verified load capacity of bearings and a combined response of structure and track.

During the calculation was found out, that the rigidness of longitudinal stiffeners does not meet the requirements of standard EN 1993-1-5. For this reason, a nonlinear calculation is made and it leads to conclusion, that the torsion stiffness of the stiffeners does not increase the buckling of the panels.

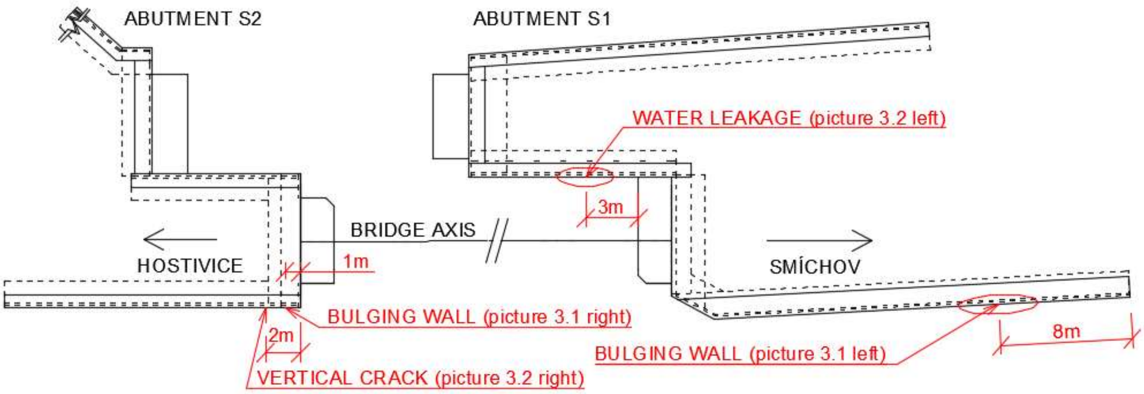
Construction and its components are assessed for ultimate limit state, serviceability limit state and fatigue. All parts are satisfactory for traffic load LM71. Stress in upper fibre of main box girder in midspan is determinative for load capacity of the bridge. The value is z_{LM71}=1,051 and it meets the requirement for line category of the bridge C3 with associated speed 70km/h.

7.1 Recommended repairs and maintenance

Load-bearing part of steel construction including the bearings is without corrosion weakening and the corrosion protection is in good condition (Ri 0). Operational cleaning is recommended. Surrounding of the bearings and the interior of the box girder are contaminated by waste.

Rails, timber sleepers and track ballast are also in good condition. Corrosion protection of guarding rail is damaged (Ri 4). It is necessary to renew the coating. The degree of corrosion aggression is S5-I, because the bridge runs across a road on which vehicles with spreading agents can be operated. According to regulation ČD S5/4, the protective coating system OSN15 is designed. Previously, the surface must be cleaned, required surface preparation is St3. It is recommended to tighten the screws of the rail fasteners and add the missing L-profile bolts.

Drainage along the bridge is fine. It is advisable to replace the drainage of expansion joints of the bridge in order to prevent the water from flowing down the back walls. There are missing screws of maintenance walkways, which should be added. Anchor bolts of ladder, which are torn from the masonry of abutment should be fixed.



Picture 7-1 – defects of concrete abutments

With regard to concrete supports, it would be advisable to remediate wet masonry and bulging parts, whereupon the repair of the covering concrete wall should be carried out by inserting a new fitting and concrete. A vertical crack in which corrodes the reinforcement should be repaired. In this case, remediation of the reinforced concrete structure is done by method called reprofilation using special cement mortars. It is performed in following ways: mechanical removal of degraded parts of concrete, removal of reinforcement rust and its subsequent passivation, refinement of the concrete structure to the original shape using reprofiled mortars.

List of used literature

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List of pictures

CZECH TECHNICAL UNIVERSITY IN PRAGUE

Faculty of Civil Engineering

Department of Steel and Timber Structures

The assessment and the load capacity of the bridge in Prague - Motol

Master's thesis

A.2. Detailed static analysis

Kateřina Soukupová

Prague 2018

Contents

1 Action on structure

1.1 Permanent loads

1.1.1 Self-weight of the structure

Self-weight of the structure is autogenerated by the program SCIA Engineer for the first method of calculation, which is integration of stress on section of the bridge to get the values of internal forces.

For the second method, where a thickness of plates is changed in case of counting with effective areas of sections, is the self-weight of every part placed in the program manually according to the real weight of all beams and plates.

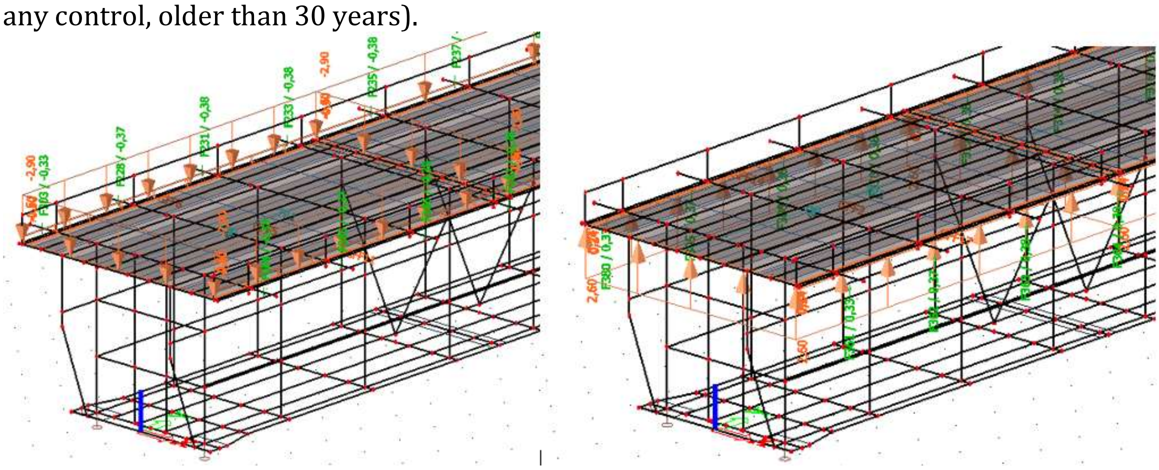
1.1.2 Other permanent loads

- protective cement screed with liner and insulation, total thickness 5 cm
$$g_{k2} = 0{,}05 * 24 = 1{,}20 kN/m^2$$
- cable trays and cables
$$g_{k3} = 2\text{x}0{,}80 kN/m$$
- track ballast, thickness 400mm
$$g_{k4} = 0{,}4 * 20 = 8{,}00 kN/m^2$$
- rails and fasteners
$$g_{k5} = 1{,}80 kN/m$$
- external railing, non-load bearing plate on side
$$g_{k6} = 0{,}70 + 0{,}9 = 1{,}6 kN/m$$
- inner maintenance walkway
$$g_{k7} = 1{,}25 kN/m$$

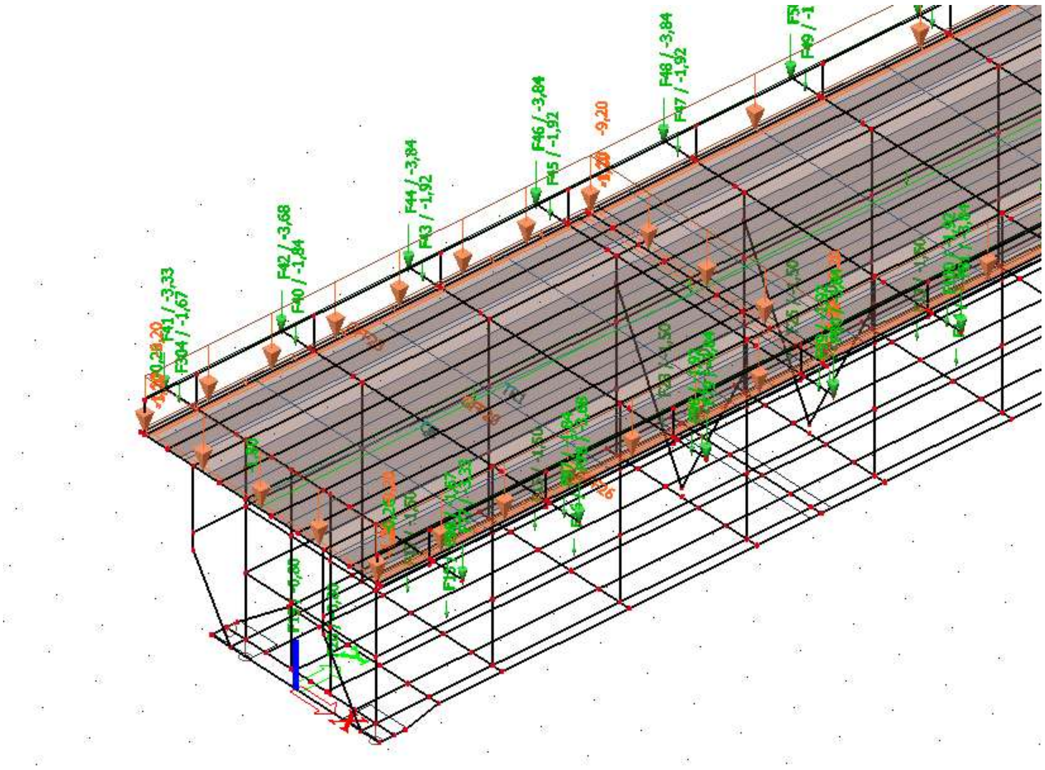
For permanent load are considered these deviation:

$$g_{k2,\text{max}} = 1{,}2 * 1{,}4 = 1{,}68 kN/m^2$$
$$g_{k2,\text{min}} = 1{,}2 * 0{,}8 = 0{,}96 kN/m^2$$
$$g_{k3,\text{max}} = 0{,}8 * 1{,}2 = 0{,}96 kN/m$$
$$g_{k3,\text{max}} = 0{,}8 * 0{,}8 = 0{,}64 kN/m$$
$$g_{k4,\text{max}} = 8{,}00 * 1{,}3 = 10{,}40 kN/m^2$$
$$g_{k4,\text{min}} = 8{,}00 * 0{,}7 = 5{,}60 kN/m^2$$

Partial factor for permanent loads: $\gamma_{\text{FG}} = 1{,}25$ (coefficient for steel member, without any control, older than 30 years).



Picture 1.1-1 - maximal and minimal deviation of other permanent loads



Picture 1.1-2 – other permanent loads

1.2 Variable loads

1.2.1 Traffic loads

For traffic loads is used the load model LM71 and load classification factor α=1,00. The load is placed in the most unfavourable position for each element, alleviating effects are being neglected. For maximal effect of the wind is used the load model "unloaded train", which consists of load with a characteristic value of 10kN/m.

Partial factor for traffic loads is $\gamma_{\mathrm{Q,LM71}} = 1{,}30$ (coefficient for steel member, without any control, older than 30 years).

1.2.2 Distribution of traffic loads

Eccentricity of the vertical load in case of the shape of the rails is added to the model of the bridge as a polyline path of the loads.

The lateral displacement of vertical loads is considered. Ratio of wheel loads is taken 1,25:1. In longitudinal direction are point forces of load model LM71 distributed on three neighbouring rail supports in ratio 1:2:1. The distribution of loads beneath sleepers is in slope of 4:1 in longitudinal and transverse direction.

Dimension of the sleepers is 260x150mm and their length is 2600mm. Their axial distance is 600mm. The distance between the wheels is assumed 1500 mm.

- longitudinal distributional width

$$b_{p1} = 260 + \frac{1}{4} * 330 * 2 = 425\text{mm}$$

- transverse distributional width

$$b_{p2} = 2600 + \frac{1}{4} * 330 * 2 = 2765\text{mm}$$

- distributed point force of load model LM71

$$Q_{vk} = 125/0{,}425 = 294{,}12\text{kN/m}$$

- uniformly distributed load of model LM71

$$q_{vk} = 80 * 0{,}6/0{,}425 = 112{,}94\text{kN/m}$$

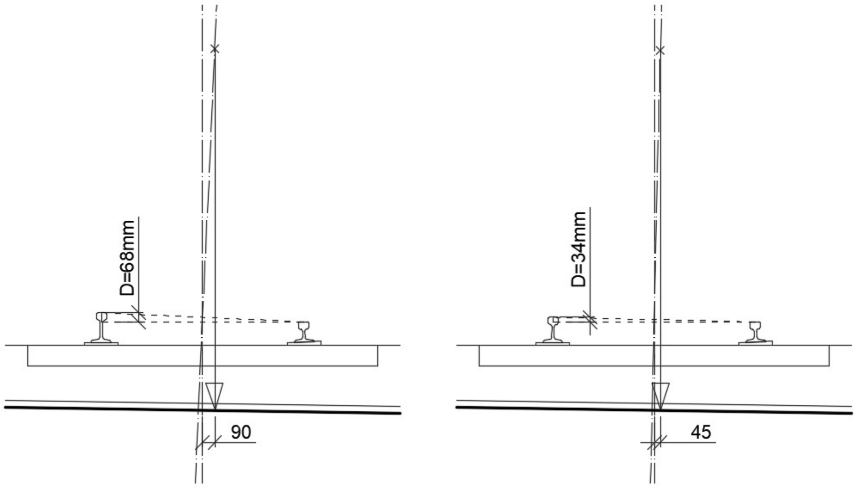
Assessment of an eccentricity of traffic loads:

- influence of eccentricity of wheel loads

$$E_1 = \frac{\mathrm{r}}{18} = \frac{1500}{18} = 83\mathrm{mm}$$

- influence of tilting the track

$$E_2 = -45 \sim -90\mathrm{mm}$$



Picture 1.2-1 - eccentricity caused by the tilting of the track

- maximal outside eccentricity

$$E_L = -45 + 83 = +38\mathrm{mm}$$

- maximal inside eccentricity

$$E_R = -90 - 83 = -173\mathrm{mm}$$

- unit vertical impulse

$$i_z = -\frac{1}{b_{r2}} = -\frac{1}{2{,}765} = -0{,}362 kN/m^2$$

- unit bending moment

$$M_{1L} = \frac{1 * E_L}{1000} = \frac{1 * 38}{1000} = 0{,}038 kNm/m$$

$$M_{1R} = \frac{1 * E_R}{1000} = \frac{1 * 173}{1000} = 0{,}173 kNm/m$$

- distributed unit load – outside eccentricity

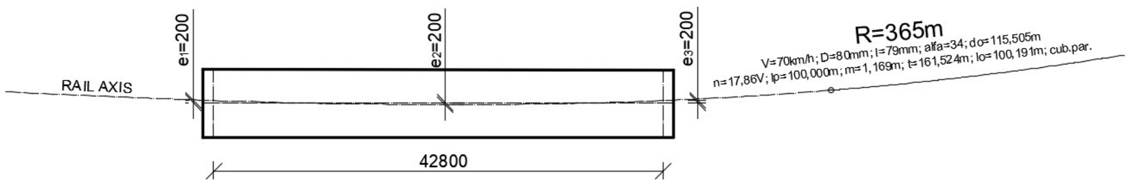
$$q_{L,L} = i_z + 6 * \frac{M_{1L}}{b_{p2}^2} = -0{,}362 - 6 * \frac{0{,}038}{2{,}765^2} = -0{,}392 kN/m^2$$

$$q_{R,L} = i_z - 6 * \frac{M_{1L}}{b_{p2}^2} = -0{,}362 + 6 * \frac{0{,}038}{2{,}765^2} = -0{,}332 kN/m^2$$

- distributed unit load – inside eccentricity

$$q_{R,L} = i_z + 6 * \frac{M_{1R}}{b_{p2}^2} = -0{,}362 + 6 * \frac{0{,}173}{2{,}765^2} = -0{,}226 kN/m^2$$

$$q_{R,R} = i_z - 6 * \frac{M_{1R}}{b_{p2}^2} = -0{,}362 - 6 * \frac{0{,}173}{2{,}765^2} = -0{,}498 kN/m^2$$



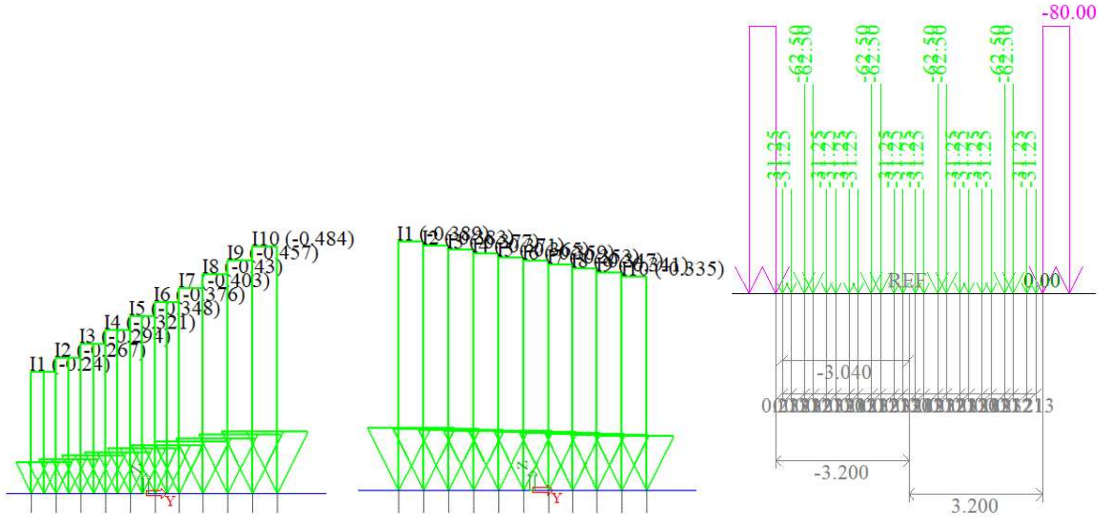
Picture 1.2-2 – plan view of the bridge – position of a rail axis and an axis of the bridge

To set the transverse load distribution to the program is the distributional width divided to ten parts. The distance x_1 and x_2 is measured from the axis of the rail. It is also counted with 100mm possible eccentricity of the position of the track.

	from	to	from	to	average loads	
	x_{1L} [mm]	x_{2L} [mm]	x_{1P} [mm]	x_{2P} [mm]	q_L [kN/m]	q_P [kN/m]
1	-1483	-1206	-1283	-1006	-0,389	-0,240
2	-1206	-930	-1006	-730	-0,383	-0,267
3	-930	-653	-730	-453	-0,377	-0,294
4	-653	-377	-453	-177	-0,371	-0,321
5	-377	-100	-177	100	-0,365	-0,348
6	-100	177	100	377	-0,359	-0,376
7	177	453	377	653	-0,353	-0,403
8	453	730	653	930	-0,347	-0,430
9	730	1006	930	1206	-0,341	-0,457
10	1006	1283	1206	1483	-0,335	-0,484

Table 1 - transversal distribution of traffic load

In longitudinal direction is every point force of load model LM71 distributed to six forces according to the rules of distribution (distributed on 3 sleepers in ratio 1:2:1, in every longitudinal distributional width are located two point forces).



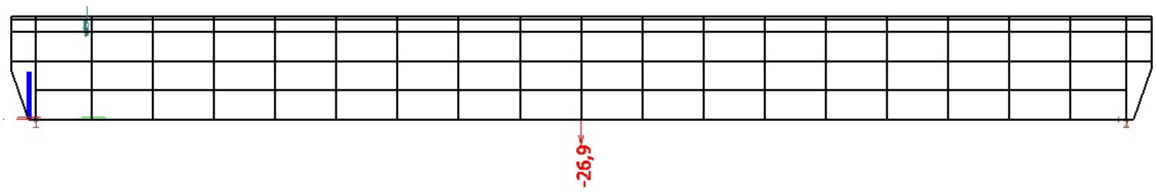
Picture 1.2-3 – transversal and longitudinal distribution of unit traffic load

1.2.3 Dynamic analysis

According EN 1991-2 is the dynamic analysis not required, because the line speed is under 200km/h and first natural bending frequency n_0 is within limits.

$$23{,}58 * L^{-0{,}592} = 23{,}58 * 42{,}8^{-0{,}592} = 2{,}551 \leq n_0 = \frac{17{,}75}{\sqrt{\delta_0}} = \frac{17{,}75}{\sqrt{26{,}9}} = 3{,}422 \leq$$

$$\leq 94{,}76 * L^{-0{,}748} = 94{,}76 * 42{,}8^{-0{,}748} = 5{,}706$$



Picture 1.2-4 - deflection at mid span due to permanent load

1.2.4 Dynamic factors

Dynamic factor for track with standard maintenance:

$$\phi_3 = \frac{2,16}{\sqrt{L_\phi} - 0,2} + 0,73 = \frac{2,16}{\sqrt{42,8} - 0,2} + 0,73 = 1,071$$

Deck plate (both directions), continuous longitudinal ribs:

$$L_\phi = 3 * 2,4 = 7,200m$$

$$\phi_3 = \frac{2,16}{\sqrt{7,200} - 0,2} + 0,73 = 1,600$$

Cross girders:

$$L_\phi = 2 * 2,8 = 5,6m$$

$$\phi_3 = \frac{2,16}{\sqrt{5,600} - 0,2} + 0,73 = 1,727$$

End cross girders:

$$L_\phi = 3,600m$$

$$\phi_3 = \frac{2,16}{\sqrt{3,600} - 0,2} + 0,73 = 2,000$$

1.2.5 Centrifugal forces

For the calculation is used a line speed 70 km/h. Reduction factor is f =1 for speed under 120km/h. The force is horizontal and perpendicular to the track axis at a height of 1,8m above the rail. It is combined with the appropriate vertical load. The radius of curvature is at one end of the bridge R_1 = 886,762m and the second R_2 = 434,726m. The mean radius r = 660,744m is considered. Load classification factor is α=1,00 and dynamic factor is not used for centrifugal forces. Partial factor for centrifugal forces is $\gamma_{Q,LM71} = 1,30$.

$$Q_{tk} = \frac{V^2}{127 * r} * (f * Q_{vk}) = \frac{70^2}{127 * 660,744} * (1 * 250) = 14,60kN$$

$$q_{tk} = \frac{V^2}{127 * r} * (f * q_{vk}) = \frac{70^2}{127 * 660,744} * (1 * 80) = 4,67kN/m$$

The centrifugal force is given into program as vertical load, that is counted as a distribution of a bending moment caused by eccentrically placed centrifugal force.

- a unit horizontal force, a unit bending moment

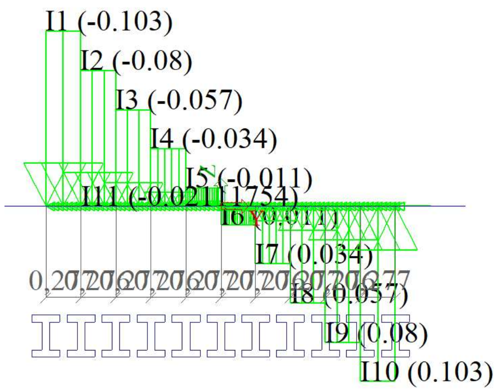
$$\mathrm{k} = \frac{V^2 * f}{127 * r} = \frac{70^2 * 1}{127 * 660,744} = 0,05839$$

- unit bending moment

$$M_1 = \mathrm{k} * \mathrm{r} = 0,05839 * 2,5 = 0,146kNm/m$$

- distributed unit load

$$q_1 = \pm 6 * \frac{M_1}{b_p^2} = \pm 6 * \frac{0,146}{2,765^2} = \pm 0,115kN/m^2$$



Picture 1.2-5 - transversal distribution of centrifugal forces

	from	to	average loads
	x₁ [mm]	x₂ [mm]	q₁ [kN/m]
1	-1383	-1106	-0,103
2	-1106	-830	-0,080
3	-830	-553	-0,057
4	-553	-277	-0,034
5	-277	0	-0,011
6	0	277	0,011
7	277	553	0,034
8	553	830	0,057
9	830	1106	0,080
10	1106	1383	0,103

Table 2 - transversal distribution of centrifugal forces

1.2.6 Nosing force

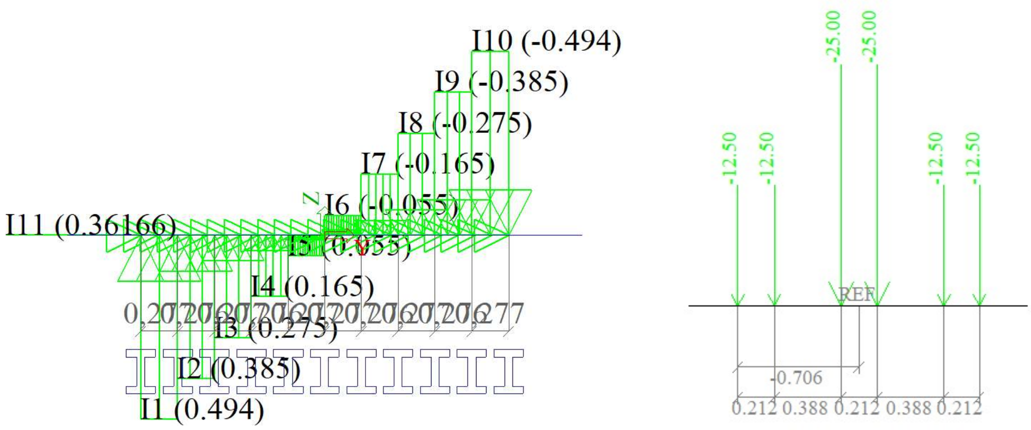
As nosing force is considered a concentrated force Q_{sk}=100kN acting horizontally, at the top of the rails, perpendicular to the centre-line of track. Load classification factor is α=1,00 and dynamic factor is not used. Partial factor for centrifugal forces is $\gamma_{Q,Qs} = 1{,}30$. Nosing force is always combined with the vertical traffic load.

- distributed unit load

$$q_1 = \pm 6 * \frac{M_1}{b_p^2} = \pm 6 * \frac{1{,}0 * 0{,}7}{2{,}765^2} = \pm 0{,}549 kN/m^2$$

	from	to	average loads
	x₁ [mm]	x₂ [mm]	q₁ [kN/m]
1	-1383	-1106	-0,494
2	-1106	-830	-0,385
3	-830	-553	-0,275
4	-553	-277	-0,165
5	-277	0	-0,055
6	0	277	0,055
7	277	553	0,165
8	553	830	0,275
9	830	1106	0,385
10	1106	1383	0,494

Table 3 - transversal distribution of nosing force



Picture 1.2-6 – transversal and longitudinal distribution of nosing force

It is distributed on three neighbouring rail supports in ratio 1:2:1, in every longitudinal distributional width are located two point forces.

1.2.7 Actions due to traction and braking

This load acts at the top of the rails in the longitudinal direction. It is uniformly distributed over the corresponding influence length. Load classification factor is α=1,00 and dynamic factor is not used. Partial factor for centrifugal forces is $\gamma_{\mathrm{Q,Qab}} = 1{,}30$. The force is always combined with the vertical traffic load.

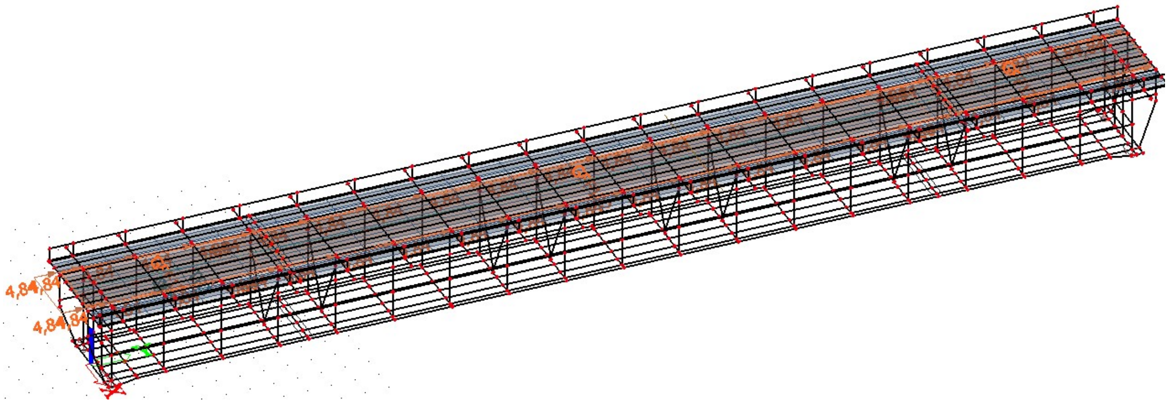
Traction force:

$$Q_{lak} = 33 * L_{ab} = 33 * 42{,}8 = 1412{,}4kN \geq 1000kN \rightarrow Q_{lak} = 1000kN$$

Braking force:

$$Q_{lbk} = 20 * L_{ab} = 20 * 42{,}8 = 856kN \leq 6000kN$$

For a continuous track on the bridge is considered, that 60% of the traction force is transmitted by the bearings and the load-bearing steel structure.



Picture 1.2-7 - acions due to traction

1.2.8 Actions for non-public footpaths

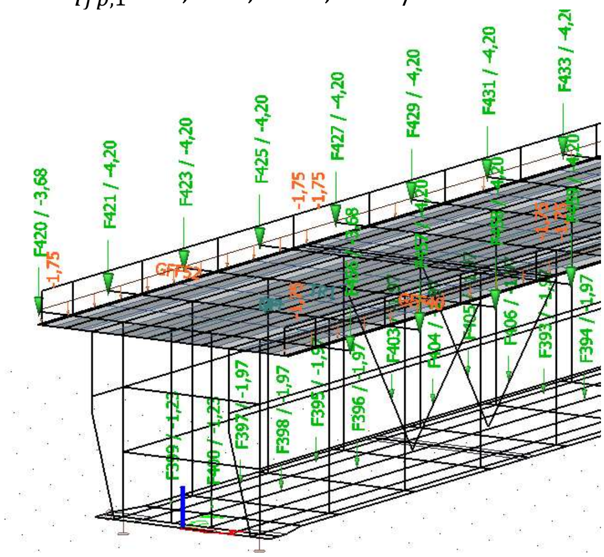
Partial factor for loads of footpaths is $\gamma_{\mathrm{Q,fp}} = 1{,}50$.

Load of inner maintenance walkway:

$$q_{fp,1} = 2{,}0 * 0{,}8 = 1{,}64kN/m$$

Load of upper non-public footpaths:

$$q_{fp,1} = 5{,}0 * 0{,}7 = 3{,}50kN/m$$



Picture 1.2-8 - actions for non-public footpaths

1.2.9 Wind actions

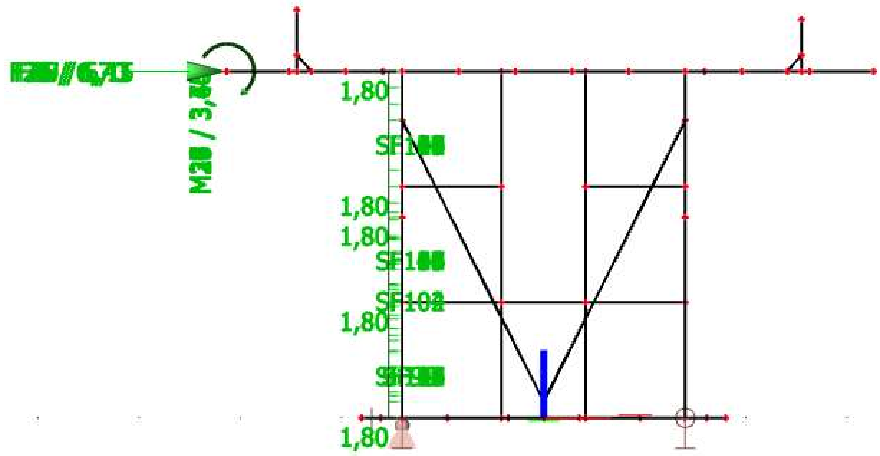
The height of the bridge construction is 4,100m, the height of an open parapet is considered to be 0,6m and the train height for the calculation is 4m. Partial factor for wind actions is $\gamma_{\mathrm{Q,w}} = 1{,}35$.

- density of air
  $\rho = 1{,}25 kg/m^3$
- the reference height under terrain
  z=7,086m
- terrain factor
  $$k_r = 0{,}19 * \left(\frac{z_0}{z_{0II}}\right)^{0{,}7} = 0{,}19 * \left(\frac{0{,}05}{0{,}05}\right)^{0{,}7} = 0{,}19$$
- roughness coefficient (terrain category II)
  $$c_r(z) = k_r * ln\frac{z}{z_0} = 0{,}19 * ln\frac{7{,}086}{0{,}05} = 0{,}9412$$
- wind area 2 … $v_{b,0}$=25m/s
- directional factor und season factor
  $c_{dir} = c_{season} = 1{,}0$
- basic wind velocity
  $v_b = c_{dir} * c_{season} * v_{b,0} = 1{,}0 * 1{,}0 * 25 = 25m/s$
- orography factor
  $c_o(z) = 1{,}0$
- turbulence factor
  $k_I = 1{,}0$
- mean wind velocity
  $v_m(z) = c_r(z) * c_o(z) * v_b = 0{,}9412 * 1{,}0 * 25 = 23{,}53m/s$
- turbulence intensity
  $$I_v(z) = \frac{k_I}{c_0(z) * \ln\left(\frac{z}{z_0}\right)} = \frac{1{,}0}{1{,}0 * \ln\left(\frac{7{,}086}{0{,}05}\right)} = 0{,}202$$
- peak velocity pressure
  $$q_{wk} = (1 + 7 * I_v(z)) * \frac{1}{2} * \rho * v_m{}^2 = (1 + 7 * 0{,}202) * \frac{1}{2} * 1{,}25 * 23{,}53^2 * 10^{-3} = = 0{,}835kN/m^2$$
- wind actions without train
  $$\frac{b}{d_{tot}} = \frac{6410}{4700} = 1{,}364 \rightarrow c_{f,x} = 2{,}155$$
  $q_{wk1} = c_{f,x} * q_{wk} = 2{,}155 * 0{,}835 = 1{,}800kN/m^2$
  Wind on railing causes a bending moment:
  $m_{wk1} = 1{,}800 * 0{,}6 * (0{,}3 + 0{,}68) = 1{,}058kNm/m$
- wind actions on the bridge and train
  $$\frac{b}{d_{tot}} = \frac{6410}{8100} = 0{,}791 \rightarrow c_{f,x} = 2{,}341$$
  $q_{wk2} = c_{f,x} * q_{wk} = 2{,}341 * 0{,}835 = 1{,}956kN/m^2$

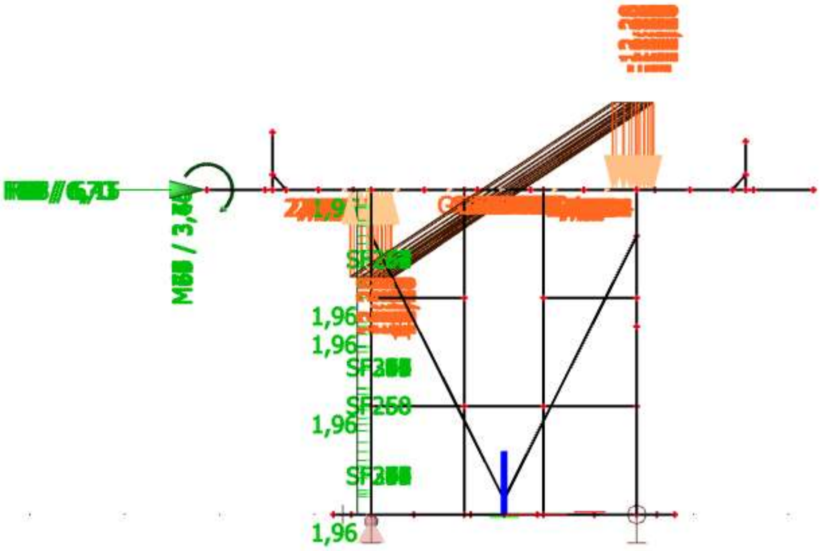
Wind on a train causes a bending moment below. It is distributed in the distributional width under sleepers.

$$m_{wk1} = 1{,}956 * 4{,}0 * 2{,}0 = 15{,}648kNm/m$$

- distributed unit load
  $$q_1 = \pm 6 * \frac{M_1}{b_p^2} = \pm 6 * \frac{15{,}648}{2{,}765^2} = \pm 12{,}281kN/m^2$$



Picture 1.2-9 - wind actions without a train



Picture 1.2-10 - wind actions with a train

wind actions in vertical direction … $c_{f,x} = 0{,}9$

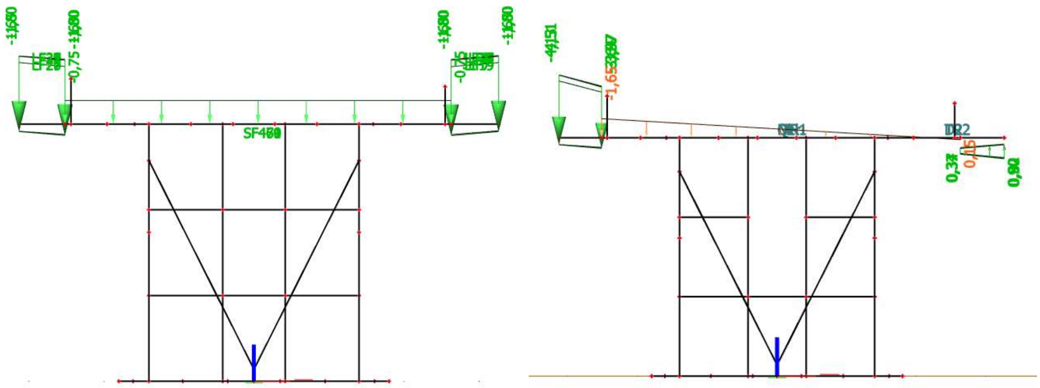
$q_{wk1} = c_{f,x} * q_{wk} = 0{,}9 * 0{,}835 = 0{,}752 kN/m^2$

$e_y = 0{,}25 * b_{tot} = 0{,}25 * 6{,}41 = 1{,}603 m$

- distributed unit load

$$q_1 = -0{,}752 \pm 6 * \frac{0{,}752 * 6{,}41 * 1{,}603}{6{,}41^2}$$

$$q_{1R} = -1{,}880 kN/m^2, q_{1L} = +0{,}376 kN/m^2$$



Picture 1.2-11 – vertical wind actions

1.2.10 Thermal actions

The steel parts of the bridge are grouped as Type 1. Thermal actions consist of a uniform temperature component and a temperature difference component. Partial factor for thermal actions is $\gamma_{F,t} = 1{,}5$.

Uniform temperature component:

- T_{max}=40°C … maximal shade air temperature
- T_{min}= -32°C … minimal shade air temperature
- T_0=10°C … initial bridge temperature
- maximal uniform bridge temperature component
$$T_{e,max} = T_{max} + 16 = 40 + 16 = 56°C$$
- minimal uniform bridge temperature component
$$T_{e,min} = T_{min} - 3 = -32 - 3 = -35°C$$
- maximum expansion range of the uniform bridge temperature component
$$\Delta T_{N,exp} = T_{e,max} - T_0 = 56 - 10 = 46°C$$
- maximum contraction range of the uniform bridge temperature component
$$\Delta T_{N,con} = T_0 - T_{e,min} = 10 - (-35) = 45°C$$

Temperature difference linear component:

- $\Delta T_{m,heat}$=18*0,6=11°C - temperature difference for top warmer than bottom
- $\Delta T_{m,cool}$=13*1,4=18°C - temperature difference for bottom warmer than top

Simultaneity of both components:

$\Delta T_{m,cool} + \omega_n * \Delta T_{N,exp} = 18 + 0{,}35 * 46 = 34{,}1°C$

$\omega_m * \Delta T_{m,cool} + \Delta T_{N,exp} = 0{,}75 * 18 + 46 = \mathbf{59{,}5}°C$

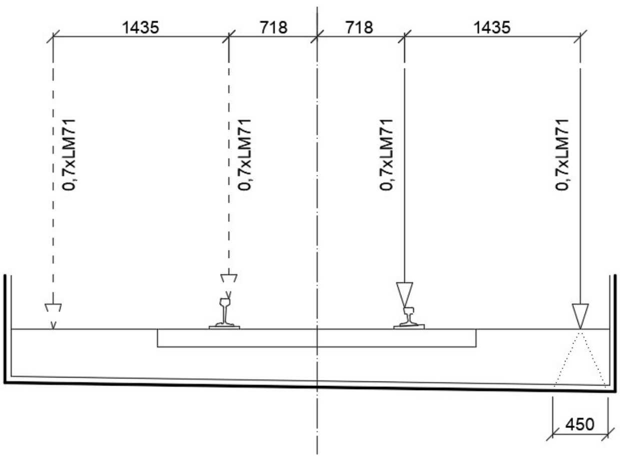
$\Delta T_{m,cool} + \omega_n * \Delta T_{N,con} = 18 + 0{,}35 * 45 = 33{,}8°C$

$\omega_m * \Delta T_{m,cool} + \Delta T_{N,con} = 0{,}75 * 18 + 45 = \mathbf{58{,}5}°C$

- For expansion is considered a temperature +59,5°C, for contraction -58,5°C.

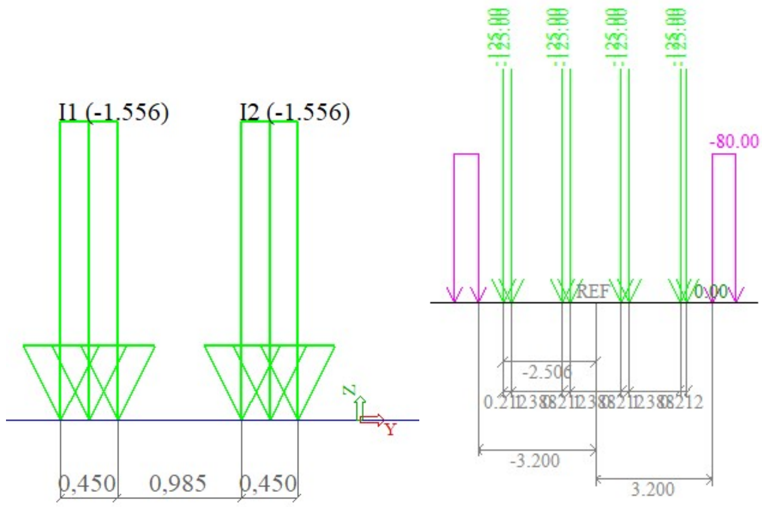
1.2.11 Derailment actions

There are two design situations considered. First design situation includes derailment of railway vehicles, which remain in the track area on both wheel tracks. Second design situation includes a vehicle balanced on the edge of the bridge, load model LM71 is taken as a uniformly distributed load of maximal length 20m.

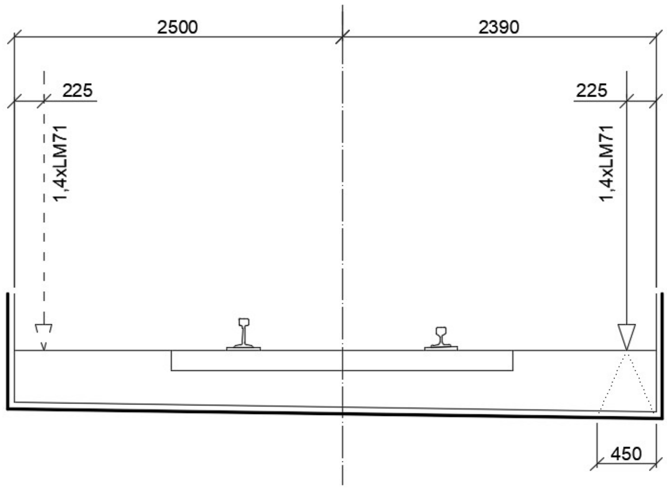


Picture 1.2-12 - Design situation I

- distributed unit load
$$q_1 = -\frac{0{,}7}{0{,}45} = -1{,}556 kN/m^2$$



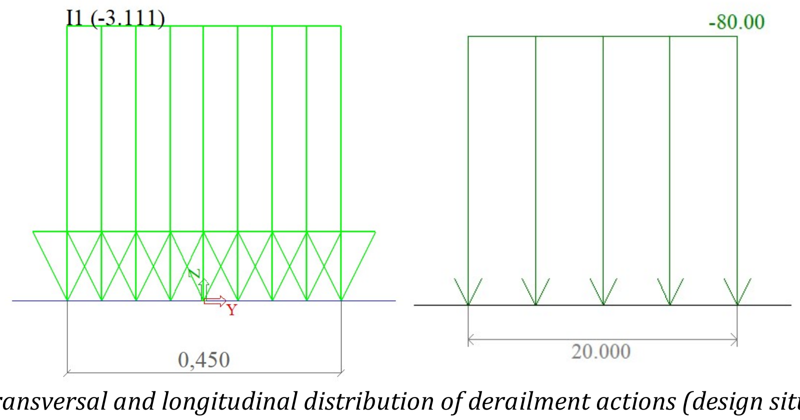
Picture 1.2-13 - transversal and longitudinal distribution of derailment actions (design situation I)



Picture 1.2-14- Design situation II

- distributed unit load

$$q_1 = -\frac{1{,}4}{0{,}45} = -3{,}111 kN/m^2$$



Picture 1.2-15 - transversal and longitudinal distribution of derailment actions (design situation II)

1.3 Combination of actions

Groups of loads for rail traffic (the values inside brackets are used for alleviating effects):

- Gr11 (vertical maximum + longitudinal maximum)

$$1 * LM71 + 1(0) * braking + 0{,}5(0) * centrifugal\ force + 0{,}5(0) * nosing\ force$$

- Gr12 (vertical maximum + transverse maximum)

$$1 * LM71 + 0{,}5(0) * braking + 1(0) * centrifugal\ force + 1(0) * nosing\ force$$

- Gr13 (longitudinal maximum)

$$1(0{,}5) * LM71 + 1 * braking + 0{,}5(0) * centrifugal\ force + 0{,}5(0) * nosing\ force$$

- Gr14 (lateral maximum)

$$1(0{,}5) * LM71 + 0{,}5(0) * braking + 1 * centrifugal\ force + 1 * nosing\ force$$

- Gr15 (lateral stability with "unloaded train")

$$1 * unloaded\ train + 1(0) * centrifugal\ force + 1(0) * nosing\ force$$

It is counted with the design situations 6.10a and 6.10b according to EN 1990 for ultimate limit states:

$$\gamma_{G,j,sup} * G_{k,j,\mathrm{sup}}\left(\gamma_{j,inf} * G_{k,j,\mathrm{inf}}\right) + \sum \gamma_{Q,i} * \psi_{0,i} * Q_{k,i} \quad (6.10a)$$

$$\xi * \gamma_{G,j,sup} * G_{k,j,\mathrm{sup}}\left(\xi * \gamma_{j,inf} * G_{k,j,\mathrm{inf}}\right) + \gamma_{Q,1} * Q_{k,1} + \sum \gamma_{Q,i} * \psi_{0,i} * Q_{k,i} \quad (6.10b)$$

Accidental combination of actions:

$$G_{k,j,\mathrm{sup}}\left(G_{k,j,\mathrm{inf}}\right) + A_d + \psi_{1,1} * Q_{k,1} + \sum \psi_{2,i} * Q_{k,i}$$

Serviceability limit state:

$$G_{k,j,\mathrm{sup}}\left(G_{k,j,\mathrm{inf}}\right) + Q_{k,1} + \sum \psi_{0,i} * Q_{k,i} \quad (characteristic)$$

$$G_{k,j,\mathrm{sup}}\left(G_{k,j,\mathrm{inf}}\right) + \psi_{1,1} * Q_{k,1} + \sum \psi_{2,i} * Q_{k,i} \quad (frequent)$$

$$G_{k,j,\mathrm{sup}}\left(G_{k,j,\mathrm{inf}}\right) + \sum \psi_{2,i} * Q_{k,i} \quad (quasi - permanent)$$

2 Material

All parts of the construction except of longitudinal stiffeners of upper plate are made of steel type 37. The yield and ultimate stress depend on the thickness of material.

$$t \leq 25mm \ldots\ f_y = 235MPa, f_u = 360MPa$$

$$t \geq 25mm \ldots\ f_y = 215MPa, f_u = 360MPa$$

The longitudinal stiffeners of upper plate are made of steel type 52. The yield and ultimate stress:

$$t \leq 50mm \ldots\ f_y = 355MPa, f_u = 510MPa$$

Particular partial factors:

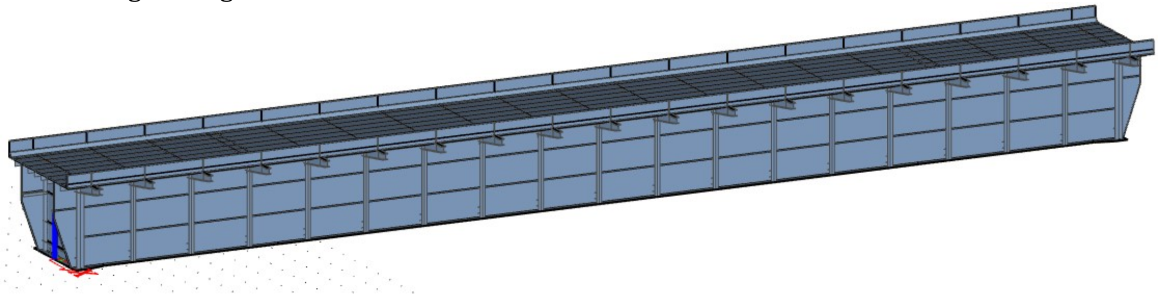
γ_{M0}=1,0

γ_{M1}=1,1

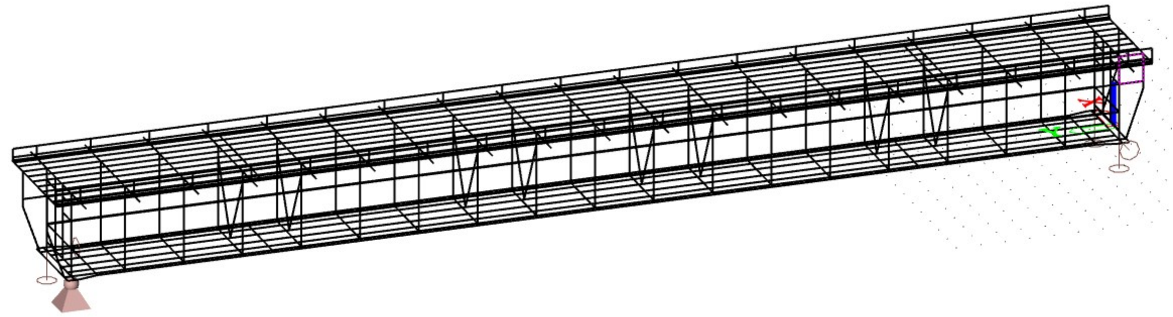
γ_{M2}=1,25

3 Computational model

Computational model was created in program Scia Engineer 17.01.1030. The bridge is modelled as a simple beam. There are four supports, on one side they are hinged and on other side sliding in longitudinal direction.

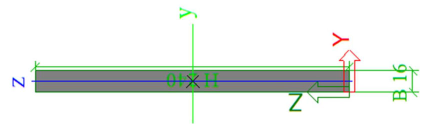


Picture 3 - 1 – rendered 3D model



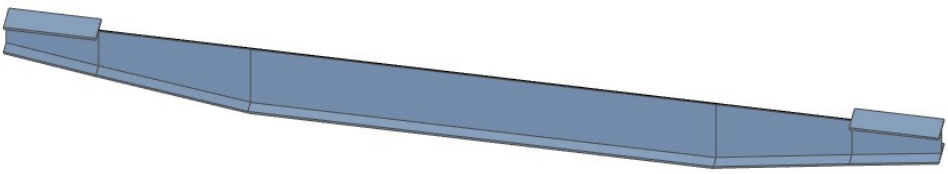
Picture 3 - 2 - 3D model

Flanges and webs of box girder are 2D members. Longitudinal stiffeners and cross girders are input as plate ribs. Longitudinal stiffeners of upper flange have open cross section P240x16.

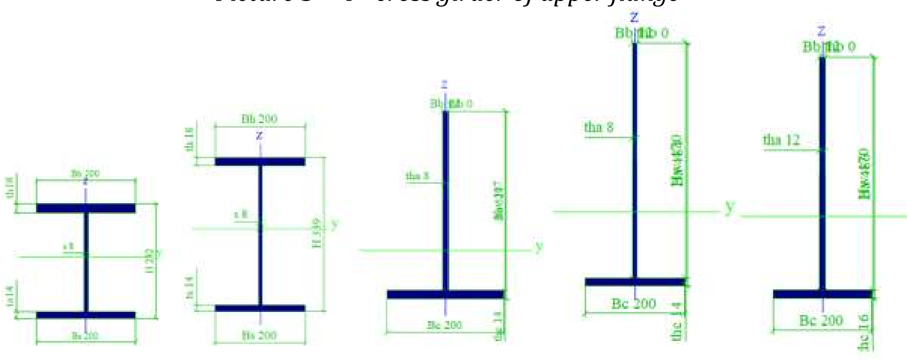


Picture 3 - 3 - longitudinal stiffener of upper flange

Cross girders of upper flange have arbitrary profile according to drawing documentation.

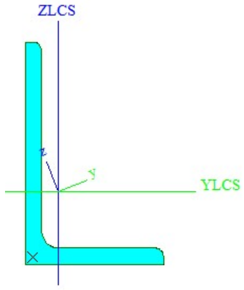


Picture 3 - 4 - cross girder of upper flange



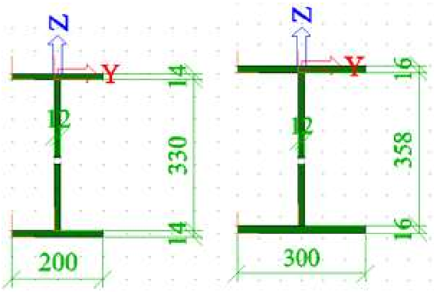
Picture 3 - 5 - cross sections of cross girder of upper flange

Longitudinal stiffeners of web and lower flange has a cross section L160x100x14.



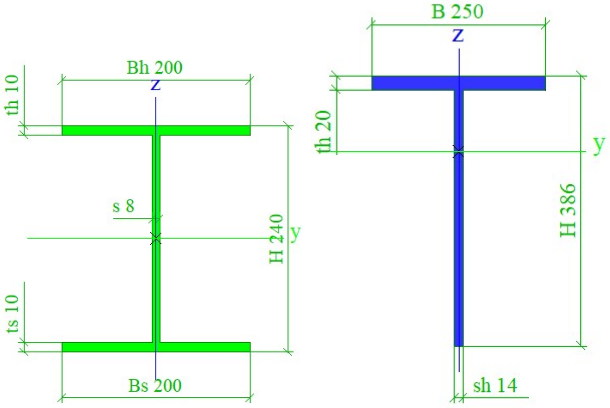
Picture 3 - 6 - longitudinal stiffener of webs and lower flange

Cross girders of web have following cross sections. End transverse stiffeners above supports is more massive.

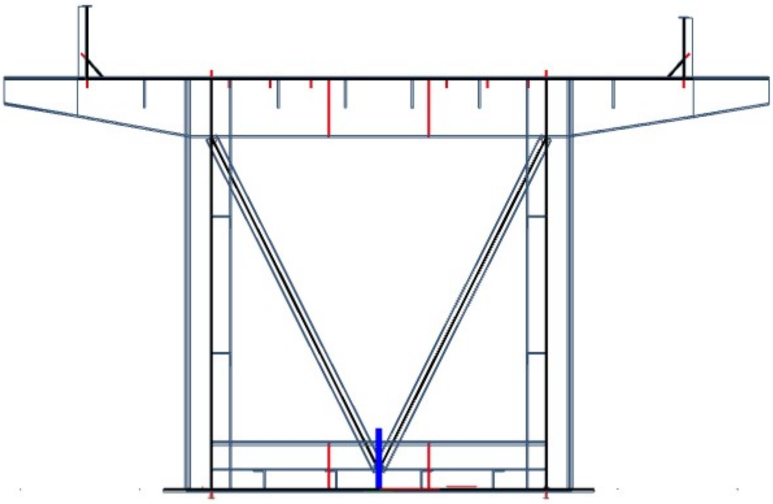


Picture 3 - 7 - intermediate transverse stiffener (left) and end transverse stiffeners (right) - web

Cross girders of lower flange have following cross sections. End cross stiffeners above supports has different cross sections than the intermediate ones.

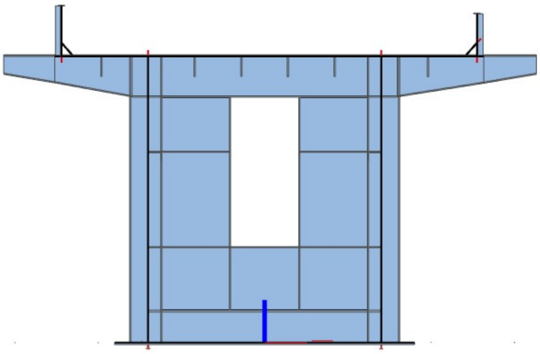


Picture 3 - 8 - intermediate cross girder (left) and end cross girder (right) – lower flange



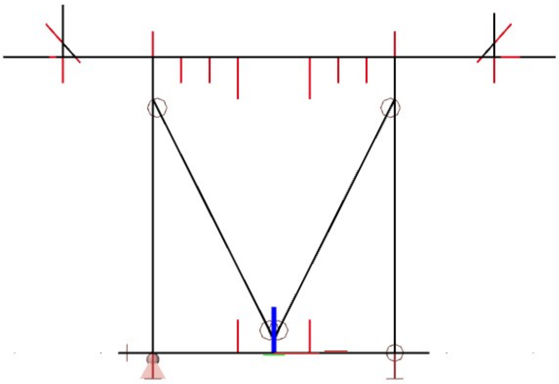
Picture 3 - 9 - position of stiffeners

Above supports is modelled diaphragm. It consists of 2D member and stiffeners according to drawing documentation



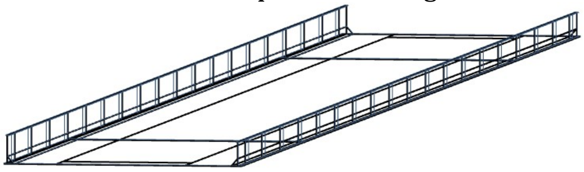
Picture 3 - 10 - diaphragm above support

There are inner stiffeners inside box girder, which are entered as beams with hinges on both ends. Eccentricity of connection is taken into consideration with rigid arms. Cross section of these stiffeners is SHS100x100x8.



Picture 3 - 11 - inner stiffeners in box girder

Maintenance walkways are considered in model just as action on structure. As a last part of load bearing structure are modelled plates framing the track ballast and its stiffeners.



Picture 3 - 12 - plates framing the track ballast

Mash of 2D members is generated according to type of calculation. Mostly is used an average size of element 10cm.

Picture 3 - 13 - generated mash

4 Static analysis

4.1 Box girder

4.1.1 Effective cross section – compressive force, midspan

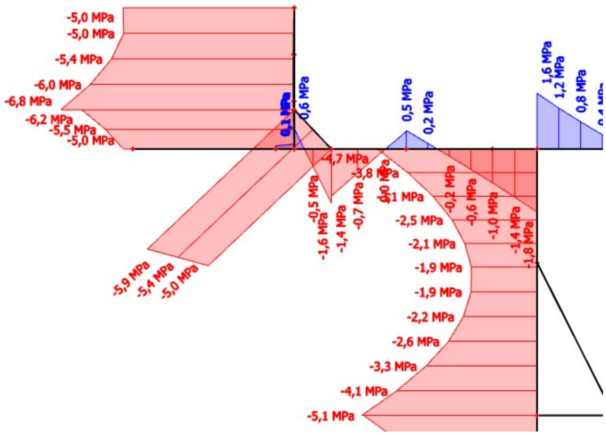
Generally:

For the three ways of stress distribution (normal force, bending moment M_y and M_z) are calculated effective cross sections in a midspan and in stationing L=7,8m, where a thickness of flanges changes.

It is considered a local buckling for every subpanel, a shear lag of flanges and a global buckling. For one or two stiffeners in a compression zone is calculation simplified by a fictitious isolated strut supported on an elastic foundation (annex A.2, EN 1993-1-5). If there are more stiffeners in a compression zone, elastic critical plate buckling is calculated by formula in annex A.1, EN 1993-1-5.

A vertical plate framing a track ballast is stiffened on a top by plate 80x8mm. Below is verified, whether it is possible to consider this plate as an internal compression element (not outstanding), although the upper stiffener is subtle. For this part of construction is made non-linear calculation with eccentricity b/200=591/200=3,0mm. On a picture bellow is a development of stress caused by wind.

internal element			outstanding element		
ψ=	1,00		ψ=	1,00	
k_σ=	4,000		k_σ=	0,430	
b=	450	mm	b=	450	mm
t=	12	mm	t=	12	mm
f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000	
λ_P=	0,660		λ_P=	2,014	
ρ=	1,000		ρ=	0,450	
b_{eff}=	**450**	**mm**	***b_{eff}=***	**203**	**mm**

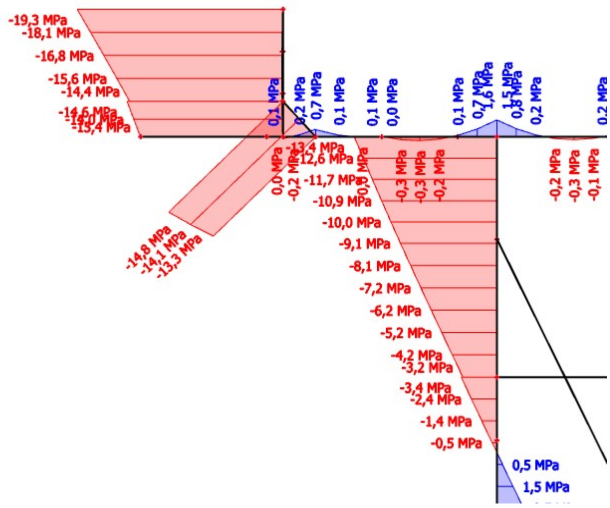


Picture 4.1-1 – stress in plate framing track ballast – horizontal loads

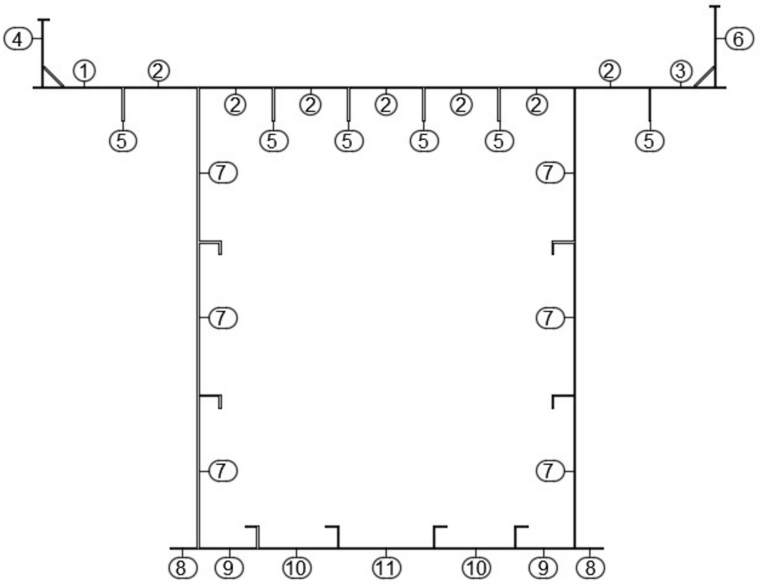
For outstanding element is supposed, that the stress is concentrated in length 203mm. It does not fit the real behaviour:

6,8MPa*0,203*12=16,6kN <5,6MPa*0,45*12=30,2kN

Supposing the plate behaviour as an internal element, it does not buckle and stress should be uniformly distributed. In a non-linear result of calculation there is a peak of stress for horizontal load. The resultant of the stress on this vertical plate is placed 288mm above the upper flange of box girder, in case of uniformly distributed stress it should be 296mm. There is no stress peak caused by vertical load, which is dominant. Because of that we can neglect the difference of 8mm. For buckling of this plate is used formula for plate supported on both sides.



Picture 4.1-2 – stress in plate framing track ballast – horizontal loads



Picture 4.1-3 - numbers of subpanels

Local buckling:

subpanel no. 1			subpanel no. 2			subpanel no. 3			subpanel no. 4		
ψ=	1,00		ψ=	1,00		ψ=	1,00		ψ=	1,00	
k_σ=	4,000		k_σ=	4,000		k_σ=	4,000		k_σ=	4,000	
b=	441	mm	b=	544	mm	b=	331	mm	b=	351	mm
t=	18	mm	t=	18	mm	t=	18	mm	t=	12	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,431		λ_P=	0,532		λ_P=	0,324		λ_P=	0,515	
ρ=	1,000		ρ=	1,000		ρ=	1,000		ρ=	1,000	
b_{eff}=	***441***	***mm***	***b_{eff}=***	***544***	***mm***	***b_{eff}=***	***331***	***mm***	***b_{eff}=***	**351**	**mm**
b_{e1}=	221	mm	b_{e1}=	272	mm	b_{e1}=	166	mm	b_{e1}=	176	mm

subpanel no. 5			subpanel no. 6			subpanel no. 7			subpanel no. 8		
ψ=	1,00		ψ=	1,00		ψ=	1,00		ψ=	1,00	
k_σ=	0,430		k_σ=	4,000		k_σ=	4,000		k_σ=	0,430	
b=	240	mm	b=	446	mm	b=	1140	mm	b=	203	mm
t=	16	mm	t=	12	mm	t=	14	mm	t=	25	mm
f_y=	355	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	0,814		ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,990		λ_P=	0,654		λ_P=	1,434		λ_P=	0,436	
ρ=	0,818		ρ=	1,000		ρ=	0,590		ρ=	1,000	
b_{eff}=	**196**	**mm**	***b_{eff}=***	***446***	***mm***	***b_{eff}=***	***673***	***mm***	***b_{eff}=***	**203**	**mm**
			b_{e1}=	223	mm	b_{e1}=	337	mm			

subpanel no. 9			subpanel no. 10			subpanel no. 11		
ψ=	1,00		ψ=	1,00		ψ=	1,00	
k_σ=	4,000		k_σ=	4,000		k_σ=	4,000	
b=	431	mm	b=	588	mm	b=	700	mm
t=	25	mm	t=	25	mm	t=	25	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,304		λ_P=	0,414		λ_P=	0,493	
ρ=	1,000		ρ=	1,000		ρ=	1,000	
b_{eff}=	***431***	***mm***	***b_{eff}=***	***588***	***mm***	***b_{eff}=***	***700***	***mm***
b_{e1}=	216	mm	b_{e1}=	294	mm	b_{e1}=	350	mm

Global buckling:

A thickness of the inner part of upper flange is modified according to formula $A_{c,eff} = \rho_c * A_{c,eff,loc}$. The reduced thickness of plate is 15,6mm.

Plate effect - inner part			Column effect - inner part		
$A_{c,eff,loc}$=	52925	mm^2	$\sigma_{cr,sl}$=	1692,8	MPa
A_c=	55680	mm^2	ψ_1=	1	
$\beta_{a,c}$=	0,951		ψ_2=	1	
t=	18	mm	b_1=	544	mm
b=	2786	mm	b_2=	544	mm
σ_E=	7,931	MPa	$b_{1,inf}$=	272	mm
a=	2388	mm	$b_{2,inf}$=	272	mm
α=	0,857		$b_{1,eff}$=	544	mm
I_{sl}=	270037619	mm^4	$b_{2,eff}$=	544	mm
I_p=	1487908	mm^4	$b_{1,inf,eff}$=	272	mm
γ=	181,5		$b_{2,inf,eff}$=	272	mm
$\sum A_{sl}$=	15360	mm^2	$I_{sl,1}$=	64832595	mm^4
A_p=	50148	mm^2	$A_{sl,1}$=	13920	mm^2
δ=	0,306		a=	2388	mm
ψ=	1		b_c=	544	mm
			b_{sl1}=	544	mm
k_σ=	191,2		$\sigma_{cr,c}$=	1692,8	MPa
$\sigma_{cr,p}$=	1516,4	MPa	$A_{sl1,eff}$=	13222	mm^2
λ_p=	0,384		$\beta_{a,c}$=	0,950	
ρ=	**1,000**		λ_c=	0,363	
			i=	68,2	mm
			e=	93,4	mm
Interaction - inner part			α_e=	0,613	
ξ=	0		Φ=	0,616	
ρ_c=	**0,898**		**χ_c=**	**0,898**	

Overhanging part does not buckle itself ($\sigma_{com,Ed}$=148,5MPa<ρ_C*f_y/γ_{M1}=0,928*235/1,1=198,3MPa). But there is a stress peak caused by buckling of inner panel, so the thickness is modified in the same way.

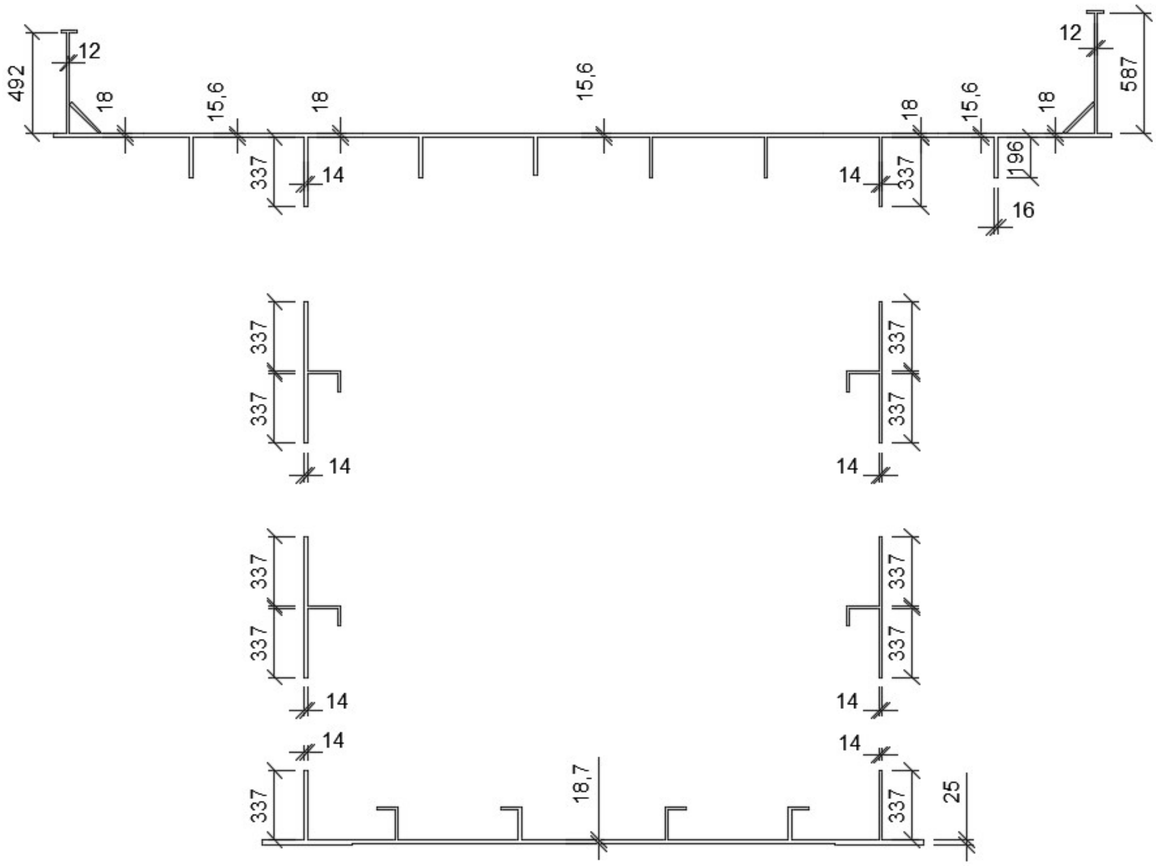
Plate effect - stiffener 5			Column effect		
I_{sl}=	63563131	mm^4	$\sigma_{cr,sl}$=	1778,1	MPa
b_1=	455	mm	ψ_1=	1	
b_2=	560	mm	ψ_2=	1	
b=	1015	mm	b_1=	441	mm
t=	18	mm	b_2=	544	mm
a_c=	3957	mm	$b_{1,inf}$=	220,5	mm
a=	2388	mm	$b_{2,inf}$=	272	mm
$A_{c,eff,loc}$=	12295	mm^2	$b_{1,eff}$=	441	mm
A_c=	12993	mm^2	$b_{2,eff}$=	544	mm
$\beta_{a,c}$=	0,946		$b_{1,inf,eff}$=	220,5	mm
$A_{sl,1}$=	12993	mm^2	$b_{2,inf,eff}$=	272,0	mm
$\sigma_{cr,sl}$=	2012,0	MPa	$I_{sl,1}$=	63563131	mm^4
ν=	0,3		$A_{sl,1}$=	12993	mm^2
b_c=	544	mm	a=	2388	mm
b_{sl1}=	544	mm	b_c=	544	mm
			b_{sl1}=	544	mm
$\sigma_{cr,p}$=	2012,0	MPa	$\sigma_{cr,c}$=	1778,1	MPa
λ_p=	0,332		$A_{sl1,eff}$=	12295	mm^2
ψ=	1,00		$\beta_{a,c}$=	0,946	
ρ=	**1,000**		λ_c=	0,354	
			i=	69,9	mm
			e=	93,0	mm
Interaction - outside part			α_e=	0,610	
ξ=	0,132		Φ=	0,609	
ρ_c=	**0,928**		**χ_c=**	**0,904**	

A thickness of web is not modified, because of validity of formula $\sigma_{com,Ed}$=62,5MPa< <ρ_C*f_y/γ_{M1}=0,846*235/1,1=180,7MPa.

Plate effect - one stiffener			**Plate effect - both stiffeners**			**Column effect - web**		
I_{sl}=	45557133	mm^4	I_{sl}=	91114265,9	mm^4	$\sigma_{cr,sl}$=	845,3	MPa
b_1=	1140	mm	b_1=	1710	mm	ψ_1=	1,00	
b_2=	1140	mm	b_2=	1710	mm	ψ_2=	1,00	
b=	2280	mm	b=	3420	mm	b_1=	1140	mm
t=	14	mm	t=	14	mm	b_2=	1140	mm
a_c=	8109	mm	a_c=	13070	mm	$b_{1,inf}$=	570	mm
a=	2388	mm	a=	2388	mm	$b_{2,inf}$=	570	mm
$A_{c,eff,loc}$=	26105	mm^2	$A_{c,eff,loc}$=	26105	mm^2	$b_{1,eff}$=	673	mm
A_c=	39176	mm^2	A_c=	39176	mm^2	$b_{2,eff}$=	673	mm
$\beta_{a,c}$=	0,666		$\beta_{a,c}$=	0,666		$b_{1,inf,eff}$=	337	mm
$\sigma_{cr,sl}$=	851,6	MPa	$\sigma_{cr,sl}$=	846,2	MPa	$b_{2,inf,eff}$=	337	mm
ν=	0,3		ν=	0,3		$I_{sl,1}$=	45557133	mm^4
b_c=	2280	mm	b_c=	3420	mm	$A_{sl,1}$=	19588	mm^2
b_{sl1}=	2280	mm	b_{sl1}=	3420	mm	a=	2388	mm
$\sigma_{cr,p1}$=	851,6	MPa	$\sigma_{cr,p2}$=	846,2	MPa	b_c=	3420	mm
λ_{p1}=	0,429		λ_{p2}=	0,430		b_{sl1}=	3420	mm
ψ=	1,00		ψ=	1,00		$\sigma_{cr,c}$=	845,3	MPa
ρ1=	**1,000**		ρ2=	**1,000**		$A_{sl1,eff}$=	13052	mm^2
						$\beta_{a,c}$=	0,666	
$\sigma_{cr,p}$=	846,2	MPa				λ_c=	0,430	
ρ=	**1,000**					i=	48,2	mm
						e=	95,4	mm
Interaction - web						α_e=	0,668	
ξ=	0,001					Φ=	0,670	
ρ_c=	**0,846**					**χ_c=**	**0,846**	

A thickness of lower flange is modified according to formula $A_{c,eff} = \rho_c * A_{c,eff,loc}$. The reduced thickness of plate is 18,7mm.

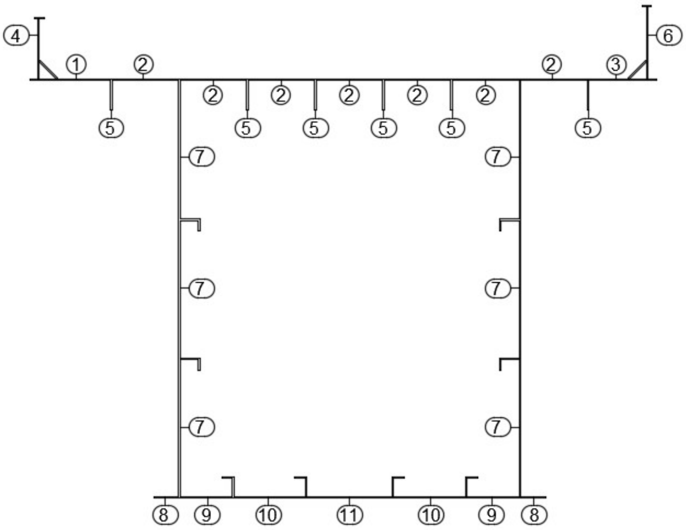
Plate effect - inner part			**Column effect - inner part**		
$A_{c,eff,loc}$=	72715	mm^2	$\sigma_{cr,sl}$=	913,1	MPa
A_c=	72715	mm^2	ψ_1=	1	
$\beta_{a,c}$=	1,000		ψ_2=	1	
t=	25	mm	b_1=	700	mm
b=	2786	mm	b_2=	588	mm
σ_E=	15,299	MPa	$b_{1,inf}$=	350	mm
a=	2388	mm	$b_{2,inf}$=	294	mm
α=	0,857		$b_{1,eff}$=	700	mm
I_{sl}=	201424136	mm^4	$b_{2,eff}$=	588	mm
I_p=	3986378,2	mm^4	$b_{1,inf,eff}$=	350	mm
γ=	50,528		$b_{2,inf,eff}$=	294	mm
$\sum A_{sl}$=	13840	mm^2	$I_{sl,1}$=	49892689	mm^4
A_p=	69650	mm^2	$A_{sl,1}$=	19860	mm^2
δ=	0,199		a=	2388	mm
ψ=	1		b_c=	544	mm
			b_{sl1}=	544	mm
k_σ=	59,7		$\sigma_{cr,c}$=	913,1	MPa
$\sigma_{cr,p}$=	912,7	MPa	$A_{sl1,eff}$=	19860	mm^2
λ_p=	0,507		$\beta_{a,c}$=	1,000	
ρ=	**1,000**		λ_c=	0,507	
			i=	50,1	mm
			e=	97,9	mm
Interaction - inner part			α_e=	0,666	
ξ=	0,000		Φ=	0,731	
ρ_c=	**0,795**		**χ_c=**	**0,795**	



Picture 4.1-4 – effective cross section – compressive force, midspan

$A_{eff} = 0{,}27548m^2 (whole\ cross\ section)$
$A_{eff,sl} = 0{,}01882m^3 (A_{eff,sl} \ldots\ yeald\ strength\ 355MPa)$

4.1.2 Effective cross section – compressive force, L=7,8m



Picture 4.1-5 - numbers of subpanels

Local buckling:

subpanel no. 1			subpanel no. 2			subpanel no. 3			subpanel no. 4		
ψ=	1,00		ψ=	1,00		ψ=	1,00		ψ=	1,00	
k_σ=	4,000		k_σ=	4,000		k_σ=	4,000		k_σ=	4,000	
b=	441	mm	b=	544	mm	b=	331	mm	b=	355	mm
t=	14	mm	t=	14	mm	t=	14	mm	t=	12	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,555		λ_P=	0,684		λ_P=	0,416		λ_P=	0,521	
ρ=	1,000		ρ=	0,992		ρ=	1,000		ρ=	1,000	
b_{eff}=	***441***	***mm***	***b_{eff}=***	***539***	***mm***	***b_{eff}=***	***331***	***mm***	***b_{eff}=***	***355***	***mm***
b_{e1}=	221	mm	b_{e1}=	270	mm	b_{e1}=	166	mm	b_{e1}=	178	mm

subpanel no. 5			subpanel no. 6			subpanel no. 7			subpanel no. 8		
ψ=	1,00		ψ=	1,00		ψ=	1,00		ψ=	1,00	
k_σ=	0,430		k_σ=	4,000		k_σ=	4,000		k_σ=	0,430	
b=	240	mm	b=	450	mm	b=	1140	mm	b=	203	mm
t=	16	mm	t=	12	mm	t=	14	mm	t=	14	mm
f_y=	355	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	0,814		ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,990		λ_P=	0,660		λ_P=	1,434		λ_P=	0,779	
ρ=	0,818		ρ=	1,000		ρ=	0,590		ρ=	0,974	
b_{eff}=	**196**	**mm**	**b_{eff}=**	**450**	**mm**	***b_{eff}=***	***673***	***mm***	**b_{eff}=**	**198**	**mm**
			b_{e1}=	225	mm	b_{e1}=	337	mm			

subpanel no. 9			subpanel no. 10			subpanel no. 11		
ψ=	1,00		ψ=	1,00		ψ=	1,00	
k_σ=	4,000		k_σ=	4,000		k_σ=	4,000	
b=	437	mm	b=	588	mm	b=	700	mm
t=	14	mm	t=	14	mm	t=	14	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,550		λ_P=	0,739		λ_P=	0,880	
ρ=	1,000		ρ=	0,950		ρ=	0,852	
b_{eff}=	***431***	***mm***	***b_{eff}=***	***559***	***mm***	***b_{eff}=***	***596***	***mm***
b_{e1}=	216	mm	b_{e1}=	279	mm	b_{e1}=	298	mm

Global buckling:

A thickness of the inner part of upper flange is modified according to formula $A_{c,eff} = \rho_c * A_{c,eff,loc}$. The reduced thickness of plate is 12,3mm.

Plate effect - inner part			**Column effect - inner part**		
$A_{c,eff,loc}$=	43704	mm²	$\sigma_{cr,sl}$=	1871,2	MPa
A_c=	46720	mm²	ψ_1=	1	
$\beta_{a,c}$=	0,935		ψ_2=	1	
t=	14	mm	b_1=	544	mm
b=	2786	mm	b_2=	544	mm
σ_E=	4,798	MPa	$b_{1,inf}$=	272	mm
a=	2388	mm	$b_{2,inf}$=	272	mm
α=	0,857		$b_{1,eff}$=	539	mm
I_{sl}=	252109659,7	mm⁴	$b_{2,eff}$=	539	mm
I_p=	700072	mm⁴	$b_{1,inf,eff}$=	270	mm
γ=	360,120		$b_{2,inf,eff}$=	270	mm
$\sum A_{sl}$=	15360	mm²	$I_{sl,1}$=	60132189	mm⁴
A_p=	39004	mm²	$A_{sl,1}$=	11680	mm²
δ=	0,394		a=	2388	mm
ψ=	1		b_c=	544	mm
			b_{sl1}=	544	mm
k_σ=	353,6		$\sigma_{cr,c}$=	1871,2	MPa
$\sigma_{cr,p}$=	1696,7	MPa	$A_{sl1,eff}$=	10919	mm²
λ_p=	0,360		$\beta_{a,c}$=	0,935	
ρ=	**1,000**		λ_c=	0,343	
			i=	71,8	mm
			e=	85,3	mm
Interaction - inner part			α_e=	0,597	
ξ=	0		Φ=	0,601	
ρ_c=	**0,913**		**χ_c=**	**0,913**	

Overhanging part does not buckle itself ($\sigma_{com,Ed}$=86,0MPa<ρ_C*f_y/γ_{M1}=0,928*235/1,1=198,3MPa). But there is a stress peak caused by buckling of inner panel, so the thickness is modified in the same way.

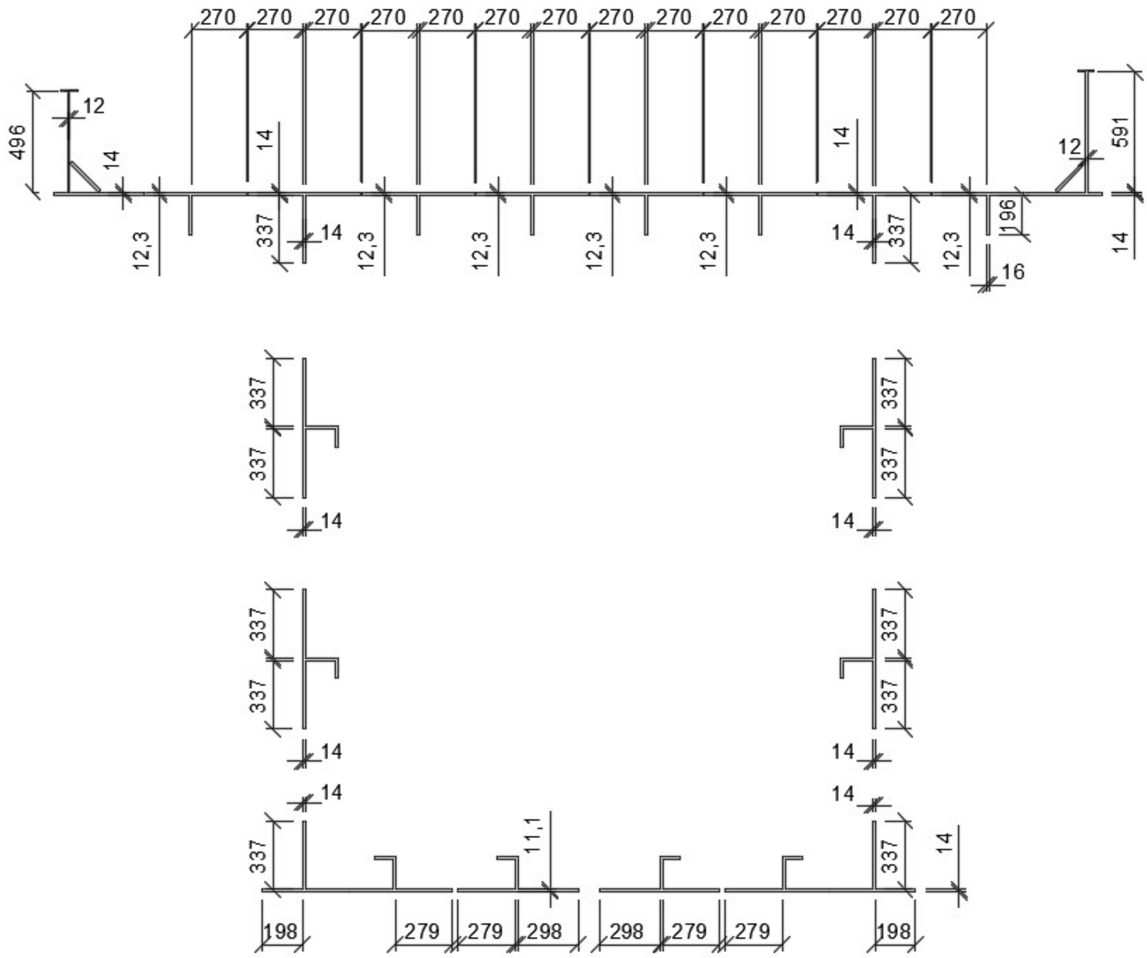
Plate effect - stiffener 5			**Column effect**		
I_{sl}=	58780762	mm⁴	$\sigma_{cr,sl}$=	1949,5	MPa
b_1=	455	mm	ψ_1=	1	
b_2=	560	mm	ψ_2=	1	
b=	1015	mm	b_1=	441	mm
t=	14	mm	b_2=	544	mm
a_c=	4685	mm	$b_{1,inf}$=	220,5	mm
a=	2388	mm	$b_{2,inf}$=	272	mm
$A_{c,eff,loc}$=	10230	mm²	$b_{1,eff}$=	441	mm
A_c=	10959	mm²	$b_{2,eff}$=	539	mm
$\beta_{a,c}$=	0,933		$b_{1,inf,eff}$=	220,5	mm
$A_{sl,1}$=	10959	mm²	$b_{2,inf,eff}$=	269,7	mm
$\sigma_{cr,sl}$=	2079,9	MPa	$I_{sl,1}$=	58780762,1	mm⁴
ν=	0,3		$A_{sl,1}$=	10959	mm²
b_c=	544	mm	a=	2388	mm
b_{sl1}=	544	mm	b_c=	544	mm
			b_{sl1}=	544	mm
$\sigma_{cr,p}$=	2079,9	MPa	$\sigma_{cr,c}$=	1949,5	MPa
λ_p=	0,325		$A_{sl1,eff}$=	10230	mm²
ψ=	1,00		$\beta_{a,c}$=	0,933	
ρ=	**0,993**		λ_c=	0,335	
			i=	73,2	mm
			e=	83,2	mm
Interaction - outside part			α_e=	0,592	
ξ=	0,067		Φ=	0,596	
ρ_c=	**0,928**		**χ_c=**	**0,918**	

A thicknes of web is not modified, because of validity of formula $\sigma_{com,Ed}$=46,0MPa< <ρ_C*f_y/γ_{M1}=0,846*235/1,1=180,7MPa.

Plate effect - one stiffener			**Plate effect - both stiffeners**			**Column effect - web**		
I_{sl}=	45557133	mm^4	I_{sl}=	91114266	mm^4	$\sigma_{cr,sl}$=	845,3	MPa
b_1=	1140	mm	b_1=	1710	mm	ψ_1=	1,00	
b_2=	1140	mm	b_2=	1710	mm	ψ_2=	1,00	
b=	2280	mm	b=	3420	mm	b_1=	1140	mm
t=	14	mm	t=	14	mm	b_2=	1140	mm
a_c=	8109	mm	a_c=	13070	mm	$b_{1,inf}$=	570	mm
a=	2388	mm	a=	2388	mm	$b_{2,inf}$=	570	mm
$A_{c,eff,loc}$=	26105	mm^2	$A_{c,eff,loc}$=	26105	mm^2	$b_{1,eff}$=	673	mm
A_c=	39176	mm^2	A_c=	39176	mm^2	$b_{2,eff}$=	673	mm
$\beta_{a,c}$=	0,666		$\beta_{a,c}$=	0,666		$b_{1,inf,eff}$=	337	mm
$\sigma_{cr,sl}$=	851,6	MPa	$\sigma_{cr,sl}$=	846,2	MPa	$b_{2,inf,eff}$=	337	mm
ν=	0,3		ν=	0,3		$I_{sl,1}$=	45557133	mm^4
b_c=	2280	mm	b_c=	3420	mm	$A_{sl,1}$=	19588	mm^2
b_{sl1}=	2280	mm	b_{sl1}=	3420	mm	a=	2388	mm
$\sigma_{cr,p1}$=	851,6	MPa	$\sigma_{cr,p2}$=	846,2	MPa	b_c=	3420	mm
λ_{p1}=	0,429		λ_{p2}=	0,430		b_{sl1}=	3420	mm
ψ=	1,00		ψ=	1,00		$\sigma_{cr,c}$=	845,3	MPa
ρ1=	**1,000**		ρ2=	**1,000**		$A_{sl1,eff}$=	13052	mm^2
						$\beta_{a,c}$=	0,666	
$\sigma_{cr,p}$=	846,2	MPa				λ_c=	0,430	
ρ=	**1,000**					i=	48,2	mm
						e=	95,4	mm
Interaction - web						α_e=	0,668	
ξ=	0,001					Φ=	0,670	
ρ_c=	**0,846**					**χ_c=**	**0,846**	

A thickness of lower flange is modified according to formula $A_{c,eff} = \rho_c * A_{c,eff,loc}$. The reduced thickness of plate is 11,1mm.

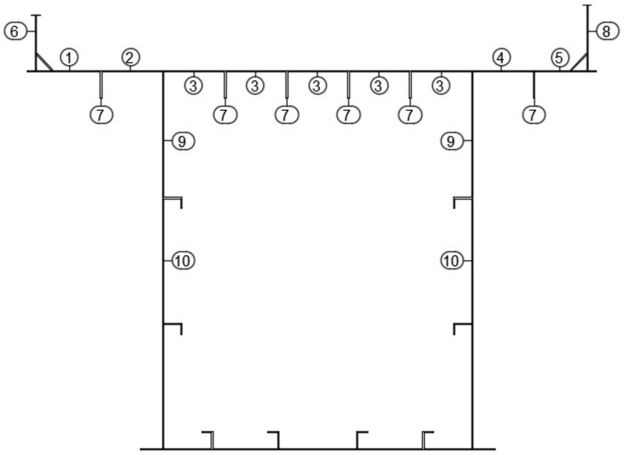
Plate effect - inner part			**Column effect - inner part**		
$A_{c,eff,loc}$=	44538	mm^2	$\sigma_{cr,sl}$=	1183,1	MPa
A_c=	46810	mm^2	ψ_1=	1	
$\beta_{a,c}$=	0,951		ψ_2=	1	
t=	14	mm	b_1=	700	mm
b=	2786	mm	b_2=	588	mm
σ_E=	4,798	MPa	$b_{1,inf}$=	350	mm
a=	2388	mm	$b_{2,inf}$=	294	mm
α=	0,857		$b_{1,eff}$=	596	mm
I_{sl}=	166743759	mm^4	$b_{2,eff}$=	559	mm
I_p=	700072	mm^4	$b_{1,inf,eff}$=	298	mm
γ=	238,181		$b_{2,inf,eff}$=	279	mm
$\sum A_{sl}$=	13840	mm^2	$I_{sl,1}$=	41157793	mm^4
A_p=	39004	mm^2	$A_{sl,1}$=	12644	mm^2
δ=	0,355		a=	2388	mm
ψ=	1		b_c=	544	mm
			b_{sl1}=	544	mm
k_σ=	241,3		$\sigma_{cr,c}$=	1183,1	MPa
$\sigma_{cr,p}$=	1157,7	MPa	$A_{sl1,eff}$=	11714	mm^2
λ_p=	0,439		$\beta_{a,c}$=	0,926	
ρ=	**1,000**		λ_c=	0,429	
			i=	57,1	mm
			e=	82,6	mm
Interaction - inner part			α_e=	0,620	
ξ=	0		Φ=	0,663	
ρ_c=	**0,856**		**χ_c=**	**0,856**	



Picture 4.1-6 – effective cross section – compressive force, L=7,8m

$A_{eff} = 0{,}22703m^2$ $(whole\ cross\ section)$,
$A_{eff,sl} = 0{,}01882m^3(A_{eff,sl} \ldots\ yeald\ strength\ 355MPa)$

4.1.3 Effective cross section – bending moment M_y, midspan



Picture 4.1-7 - numbers of subpanels

Upper flange - buckling and shear lag

subpanel no. 1			subpanel no. 2			subpanel no. 3			subpanel no. 4			subpanel no. 5		
ψ=	1		ψ=	1		ψ=	1		ψ=	1		ψ=	1	
k_σ=	4,000		k_σ=	4,000		k_σ=	4,000		k_σ=	4,000		k_σ=	4,000	
b=	441	mm	b=	545	mm	b=	544	mm	b=	545	mm	b=	331	mm
t=	18	mm	t=	18	mm	t=	18	mm	t=	18	mm	t=	18	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,431		λ_P=	0,533		λ_P=	0,532		λ_P=	0,533		λ_P=	0,324	
ρ=	1,000		ρ=	1,000		ρ=	1,000		ρ=	1,000		ρ=	1,000	
b_{eff}=	***441***	***mm***	***b_{eff}=***	***545***	***mm***	***b_{eff}=***	***544***	***mm***	***b_{eff}=***	***545***	***mm***	***b_{eff}=***	***331***	***mm***
			α0=	1,000		α0=	1,000					α0=	1,000	
			b0=	1862	mm	b0=	1400	mm				b0=	1732	mm
			Le=	42800	mm	Le=	42800	mm				Le=	42800	mm
			κ=	0,044		κ=	0,033					κ=	0,040	
			β=	0,988		β=	0,993					β=	0,990	
			teff=	**17,8**	**mm**	**teff=**	**17,9**	**mm**				**teff=**	**17,8**	**mm**
b_{e1}=	221	mm	b_{e1}=	273	mm	b_{e1}=	272	mm	b_{e1}=	273	mm	b_{e1}=	166	mm

Web - buckling

subpanel no. 6			subpanel no. 7			subpanel no. 8			subpanel no. 9			subpanel no. 10		
ψ=	0,76		ψ=	0,85		ψ=	0,73		ψ=	0,27		ψ=	-1,75	
k_σ=	4,529		k_σ=	0,487		k_σ=	4,614		k_σ=	6,236		k_σ=	45,343	
b=	351	mm	b=	240	mm	b=	446	mm	b=	1140	mm	b=	1126	mm
t=	12	mm	t=	16	mm	t=	12	mm	t=	14	mm	b_c=	412	mm
f_y=	235	MPa	f_y=	355	MPa	f_y=	235	MPa	f_y=	235	MPa	t=	14	mm
ε=	1,000		ε=	0,814		ε=	1,000		ε=	1,000		f_y=	235	MPa
λ_P=	0,484		λ_P=	0,930		λ_P=	0,609		λ_P=	1,148		ε=	1,000	
ρ=	1,000		ρ=	0,858		ρ=	1,000		ρ=	0,735		λ_P=	0,421	
b_{eff}=	***351***	**mm**	***b_{eff}=***	**206**	**mm**	***b_{eff}=***	***446***	**mm**	***b_{eff}=***	**838**	**mm**	ρ=	1,000	
b_{e1}=	166	mm				b_{e1}=	209	mm	b_{e1}=	354	mm	***b_{eff}=***	**412**	**mm**
b_{e2}=	185	mm				b_{e2}=	237	mm	b_{e2}=	484	mm	b_{e1}=	165	mm
												b_{e2}=	247	mm

Lower flange - shear lag

t=	25	mm
α0=	1,000	
b0=	1400	mm
Le=	42800	mm
κ=	0,033	
β=	0,993	
teff=	**24,8**	**mm**

Global buckling:

A thickness of the inner part of upper flange is modified according to formula $A_{c,eff} = \rho_c * A_{c,eff,loc}$. The reduced thickness of plate is 15,4mm.

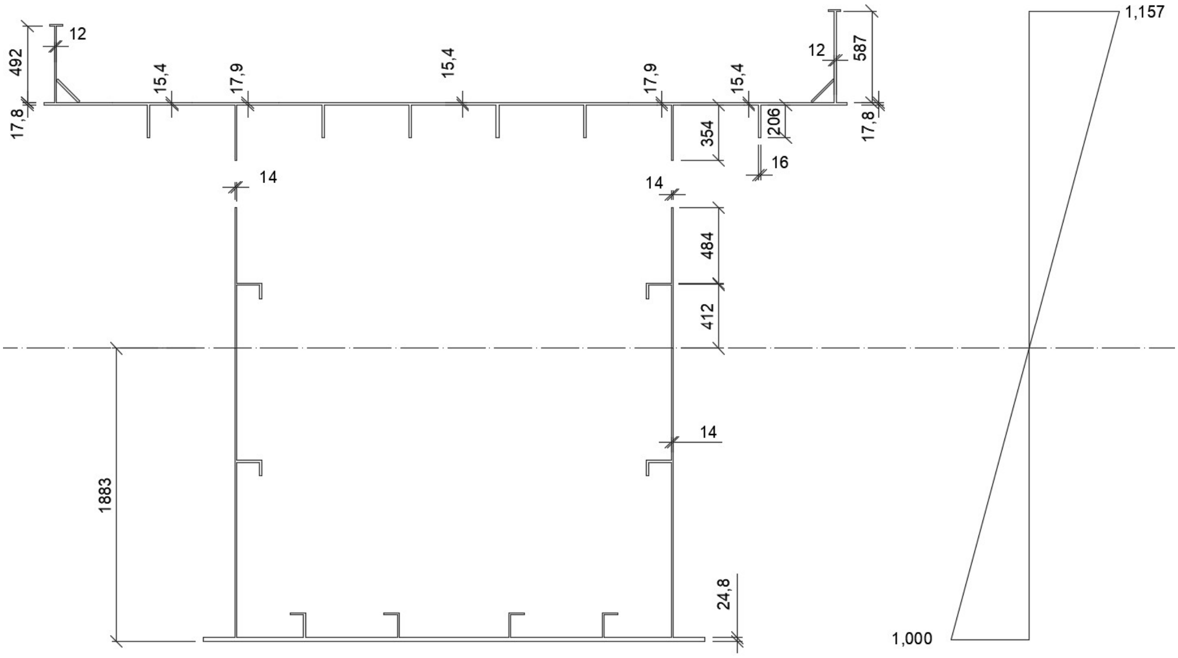
Plate effect - inner part			**Column effect - inner part**		
$A_{c,eff,loc}$=	53223	mm²	$\sigma_{cr,sl}$=	1692,8	MPa
A_c=	55680	mm²	ψ_1=	1	
$\beta_{a,c}$=	0,956		ψ_2=	1	
t=	18	mm	b_1=	544	mm
b=	2786	mm	b_2=	544	mm
σ_E=	7,931	MPa	$b_{1,inf}$=	272	mm
a=	2388	mm	$b_{2,inf}$=	272	mm
α=	0,857		$b_{1,eff}$=	544	mm
I_{sl}=	270754550	mm⁴	$b_{2,eff}$=	544	mm
I_p=	1487908	mm⁴	$b_{1,inf,eff}$=	272	mm
γ=	181,970		$b_{2,inf,eff}$=	272	mm
$\sum A_{sl}$=	15360	mm²	$I_{sl,1}$=	64832595	mm⁴
A_p=	50148	mm²	$A_{sl,1}$=	13920	mm²
δ=	0,306		a=	2388	mm
ψ=	1		b_c=	544	mm
			b_{sl1}=	544	mm
k_σ=	191,7		$\sigma_{cr,c}$=	1692,8	MPa
$\sigma_{cr,p}$=	1520,4	MPa	$A_{sl1,eff}$=	13306	mm²
λ_p=	0,384		$\beta_{a,c}$=	0,956	
ρ=	**1,000**		λ_c=	0,364	
			i=	68,2	mm
			e=	93,5	mm
Interaction - inner part			α_e=	0,613	
ξ=	0		Φ=	0,617	
ρ_c=	**0,897**		**χ_c=**	**0,897**	

Overhanging part does not buckle itself ($\sigma_{com,Ed}$=148,5MPa<ρ_C*f_y/γ_{M1}=0,927*235/1,1=198,0MPa). But there is a stress peak caused by buckling of inner panel, so the thickness is modified in the same way.

Plate effect - stiffener 7			**Column effect**		
I_{sl}=	63707243	mm⁴	$\sigma_{cr,sl}$=	1780,9	MPa
b_1=	455	mm	ψ_1=	1	
b_2=	560	mm	ψ_2=	1	
b=	1015	mm	b_1=	441	mm
t=	18	mm	b_2=	545	mm
a_c=	3959	mm	$b_{1,inf}$=	220,5	mm
a=	2388	mm	$b_{2,inf}$=	272,5	mm
$A_{c,eff,loc}$=	12347	mm²	$b_{1,eff}$=	441	mm
A_c=	13002	mm²	$b_{2,eff}$=	545	mm
$\beta_{a,c}$=	0,950		$b_{1,inf,eff}$=	220,5	mm
$A_{sl,1}$=	13002	mm²	$b_{2,inf,eff}$=	272,5	mm
$\sigma_{cr,sl}$=	2014,6	MPa	$I_{sl,1}$=	63707243	mm⁴
ν=	0,3		$A_{sl,1}$=	13002	mm²
b_c=	544	mm	a=	2388	mm
b_{sl1}=	544	mm	b_c=	544	mm
			b_{sl1}=	544	mm
$\sigma_{cr,p}$=	2014,6	MPa	$\sigma_{cr,c}$=	1780,9	MPa
λ_p=	0,333		$A_{sl1,eff}$=	12456	mm²
ψ=	1,00		$\beta_{a,c}$=	0,958	
ρ=	**1,000**		λ_c=	0,356	
			i=	70,0	mm
			e=	91,0	mm
Interaction - outside part			α_e=	0,607	
ξ=	0,131		Φ=	0,610	
ρ_c=	**0,927**		**χ_c=**	**0,904**	

A thickness of web is not modified, because of validity of formula $\sigma_{com,Ed}$=62,5MPa< <ρ_C*f_y/γ_{M1}=1,000*235/1,1=235,0MPa.

Plate effect - web			**Column effect - web**		
I_{sl}=	63508030	mm^4	$\sigma_{cr,sl}$=	1523,2	MPa
b_1=	1140	mm	ψ_1=	0,27	
b_2=	2280	mm	ψ_2=	-1,75	
b=	3420	mm	b_1=	1140	mm
t=	14	mm	b_2=	412	mm
a_c=	11259	mm	$b_{1,inf}$=	658	mm
a=	2388	mm	$b_{2,inf}$=	165	mm
$A_{c,eff,loc}$=	16197	mm^2	$b_{1,eff}$=	838	mm
A_c=	18643	mm^2	$b_{2,eff}$=	412	mm
$\beta_{a,c}$=	0,869		$b_{1,inf,eff}$=	484	mm
$\sigma_{cr,sl}$=	1526,3	MPa	$b_{2,inf,eff}$=	165	mm
ν=	0,3		$I_{sl,1}$=	63508030	mm^4
b_c=	1565	mm	$A_{sl,1}$=	15154	mm^2
b_{sl1}=	419	mm	a=	2388	mm
			b_c=	1565	mm
			b_{sl1}=	419	mm
$\sigma_{cr,p}$=	5700,7	MPa	$\sigma_{cr,c}$=	5689,3	MPa
λ_p=	0,189		$A_{sl1,eff}$=	12708	mm^2
ψ=	0,00		$\beta_{a,c}$=	0,839	
ρ=	**1,000**		λ_c=	0,186	
			i=	64,7	mm
			e=	111,3	mm
Interaction - web			α_e=	0,645	
ξ=	0,002		Φ=	0,513	
ρ_c=	**1,000**		**χ_c=**	**1,000**	

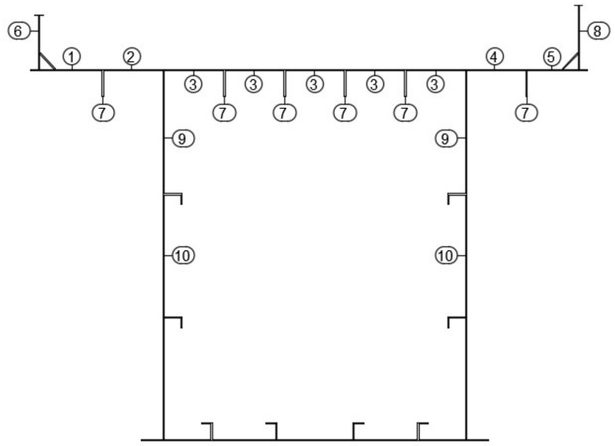


Picture 4.1-8 - Effective cross section – bending moment My, midspan

$w_{y,el,eff} = 0{,}33629m^3$
$e_y = -0{,}065m$

These values are calculated for the upper fibres of the beam, which are determinative for assessment.

4.1.4 Effective cross section – bending moment M_y, L=7,8m



Picture 4.1-9 - numbers of subpanels

Upper flange - buckling and shear lag

subpanel no. 1			subpanel no. 2			subpanel no. 3			subpanel no. 4			subpanel no. 5		
ψ=	1		ψ=	1		ψ=	1		ψ=	1		ψ=	1	
k_σ=	4,000		k_σ=	4,000		k_σ=	4,000		k_σ=	4,000		k_σ=	4,000	
b=	441	mm	b=	545	mm	b=	544	mm	b=	545	mm	b=	331	mm
t=	14	mm	t=	14	mm	t=	14	mm	t=	14	mm	t=	14	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,555		λ_P=	0,685		λ_P=	0,684		λ_P=	0,685		λ_P=	0,416	
ρ=	1,000		ρ=	0,991		ρ=	0,992		ρ=	0,991		ρ=	1,000	
b_{eff}=	***441***	***mm***	***b_{eff}=***	***540***	***mm***	***b_{eff}=***	***539***	***mm***	***b_{eff}=***	***540***	***mm***	***b_{eff}=***	***331***	***mm***
			α0=	0,995		α0=	0,996					α0=	1,000	
			b0=	1862	mm	b0=	1400	mm				b0=	1732	mm
			Le=	42800	mm	Le=	42800	mm				Le=	42800	mm
			κ=	0,043		κ=	0,033					κ=	0,040	
			β=	0,988		β=	0,993					β=	0,990	
			teff=	**13,8**	**mm**	**teff=**	**13,9**	**mm**				**teff=**	**13,9**	**mm**
b_{e1}=	221	mm	b_{e1}=	270	mm	b_{e1}=	270	mm	b_{e1}=	270	mm	b_{e1}=	166	mm

Web - buckling

subpanel no. 6			subpanel no. 7			subpanel no. 8			subpanel no. 9			subpanel no. 10		
ψ=	0,74		ψ=	0,83		ψ=	0,71		ψ=	0,19		ψ=	-3,00	
k_σ=	4,572		k_σ=	0,493		k_σ=	4,662		k_σ=	6,590		k_σ=	95,680	
b=	355	mm	b=	240	mm	b=	450	mm	b=	1140	mm	b=	1126	mm
t=	12	mm	t=	16	mm	t=	12	mm	t=	14	mm	b_c=	273	mm
f_y=	235	MPa	f_y=	355	MPa	f_y=	235	MPa	f_y=	235	MPa	t=	14	mm
ε=	1,000		ε=	0,814		ε=	1,000		ε=	1,000		f_y=	235	MPa
λ_P=	0,487		λ_P=	0,924		λ_P=	0,612		λ_P=	1,117		ε=	1,000	
ρ=	1,000		ρ=	0,862		ρ=	1,000		ρ=	0,754		λ_P=	0,290	
b_{eff}=	***355***	***mm***	***b_{eff}=***	***207***	***mm***	***b_{eff}=***	***450***	***mm***	***b_{eff}=***	***860***	***mm***	ρ=	1,000	
b_{e1}=	167	mm				b_{e1}=	210	mm	b_{e1}=	358	mm	***b_{eff}=***	***273***	***mm***
b_{e2}=	188	mm				b_{e2}=	240	mm	b_{e2}=	502	mm	b_{e1}=	109	mm
												b_{e2}=	164	mm

Lower flange - shear lag

t=	14	mm
α0=	1,000	
b0=	1400	mm
Le=	42800	mm
κ=	0,033	
β=	0,993	
teff=	**13,9**	**mm**

Global buckling:

A thickness of the inner part of upper flange is modified according to formula $A_{c,eff} = \rho_c * A_{c,eff,loc}$. The reduced thickness of plate is 12,1mm.

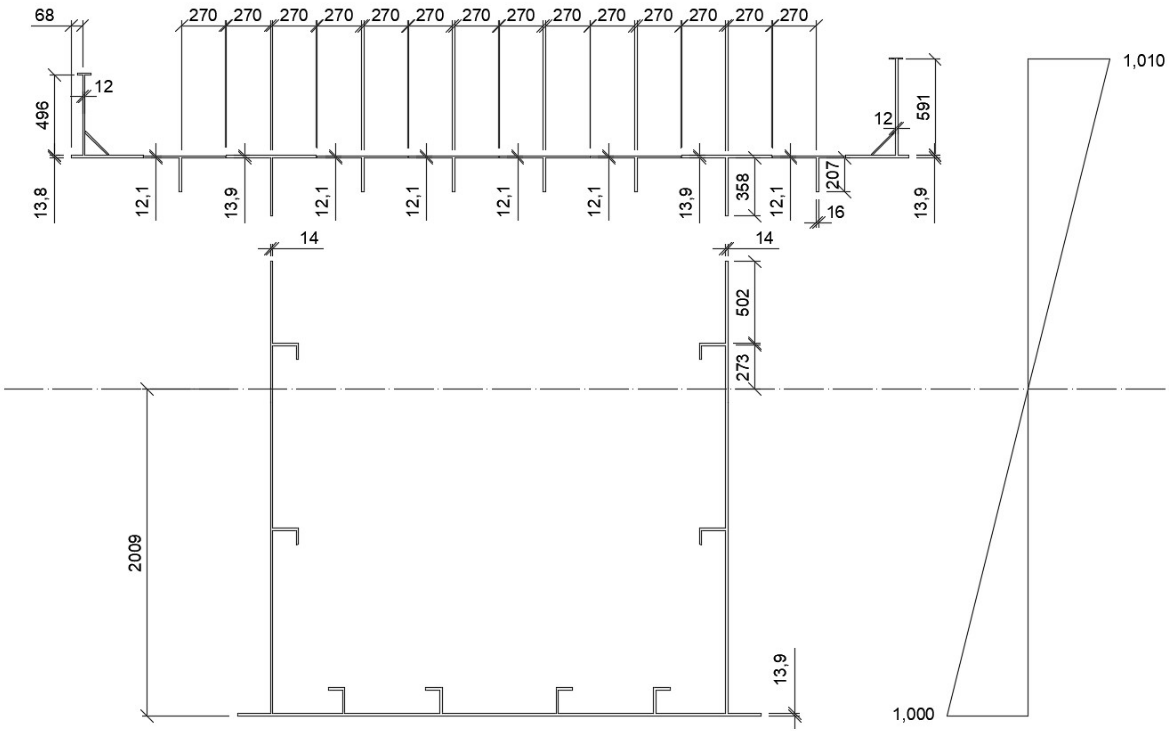
Plate effect - inner part			Column effect - inner part		
$A_{c,eff,loc}$=	44135	mm^2	$\sigma_{cr,sl}$=	1871,2	MPa
A_c=	46720	mm^2	ψ_1=	1	
$\beta_{a,c}$=	0,945		ψ_2=	1	
t=	14	mm	b_1=	544	mm
b=	2786	mm	b_2=	544	mm
σ_E=	4,798	MPa	$b_{1,inf}$=	272	mm
a=	2388	mm	$b_{2,inf}$=	272	mm
α=	0,857		$b_{1,eff}$=	539	mm
I_{sl}=	263482493	mm^4	$b_{2,eff}$=	539	mm
I_p=	700072	mm^4	$b_{1,inf,eff}$=	270	mm
γ=	376,365		$b_{2,inf,eff}$=	270	mm
$\sum A_{sl}$=	15360	mm^2	$I_{sl,1}$=	60132189	mm^4
A_p=	39004	mm^2	$A_{sl,1}$=	11680	mm^2
δ=	0,394		a=	2388	mm
ψ=	1		b_c=	544	mm
			b_{sl1}=	544	mm
k_σ=	369,5		$\sigma_{cr,c}$=	1871,2	MPa
$\sigma_{cr,p}$=	1772,8	MPa	$A_{sl1,eff}$=	11034	mm^2
λ_p=	0,354		$\beta_{a,c}$=	0,945	
ρ=	**1,000**		λ_c=	0,344	
			i=	71,8	mm
			e=	85,3	mm
Interaction - inner part			α_e=	0,597	
ξ=	0		Φ=	0,602	
ρ_c=	**0,912**		**χ_c=**	**0,912**	

Overhanging part does not buckle itself ($\sigma_{com,Ed}$=86,0MPa<ρ_C*f_y/γ_{M1}=0,928*235/1,1=198,3MPa). But there is a stress peak caused by buckling of inner panel, so the thickness is modified in the same way.

Plate effect - stiffener 7			Column effect		
I_{sl}=	58795187	mm^4	$\sigma_{cr,sl}$=	1948,7	MPa
b_1=	455	mm	ψ_1=	1	
b_2=	560	mm	ψ_2=	1	
b=	1015	mm	b_1=	441	mm
t=	14	mm	b_2=	545	mm
a_c=	4685	mm	$b_{1,inf}$=	220,5	mm
a=	2388	mm	$b_{2,inf}$=	272,5	mm
$A_{c,eff,loc}$=	10316	mm^2	$b_{1,eff}$=	441	mm
A_c=	10966	mm^2	$b_{2,eff}$=	540	mm
$\beta_{a,c}$=	0,941		$b_{1,inf,eff}$=	220,5	mm
$A_{sl,1}$=	10966	mm^2	$b_{2,inf,eff}$=	270,0	mm
$\sigma_{cr,sl}$=	2079,1	MPa	$I_{sl,1}$=	58795187	mm^4
ν=	0,3		$A_{sl,1}$=	10966	mm^2
b_c=	544	mm	a=	2388	mm
b_{sl1}=	544	mm	b_c=	544	mm
			b_{sl1}=	544	mm
$\sigma_{cr,p}$=	2079,1	MPa	$\sigma_{cr,c}$=	1948,7	MPa
λ_p=	0,326		$A_{sl1,eff}$=	10316	mm^2
ψ=	1,00		$\beta_{a,c}$=	0,941	
ρ=	**0,998**		λ_c=	0,337	
			i=	73,2	mm
			e=	81,3	mm
Interaction - outside part			α_e=	0,590	
ξ=	0,067		Φ=	0,597	
ρ_c=	**0,928**		**χ_c=**	**0,917**	

A thickness of web is not modified, because of validity of formula $\sigma_{com,Ed}$=46,0MPa< <ρ_C*f_y/γ_{M1}=1,000*235/1,1=213,6MPa.

Plate effect - web			**Column effect - web**		
I_{sl}=	62711855	mm^4	$\sigma_{cr,sl}$=	1574,7	MPa
b_1=	1140	mm	ψ_1=	0,19	
b_2=	2280	mm	ψ_2=	-3,00	
b=	3420	mm	b_1=	1140	mm
t=	14	mm	b_2=	273	mm
a_c=	11224	mm	$b_{1,inf}$=	666	mm
a=	2388	mm	$b_{2,inf}$=	109	mm
$A_{c,eff,loc}$=	14480	mm^2	$b_{1,eff}$=	860	mm
A_c=	16768	mm^2	$b_{2,eff}$=	273	mm
$\beta_{a,c}$=	0,864		$b_{1,inf,eff}$=	502	mm
$\sigma_{cr,sl}$=	1577,9	MPa	$b_{2,inf,eff}$=	165	mm
ν=	0,3		$I_{sl,1}$=	62711855	mm^4
b_c=	1426	mm	$A_{sl,1}$=	14475	mm^2
b_{sl1}=	280	mm	a=	2388	mm
			b_c=	1426	mm
			b_{sl1}=	280	mm
$\sigma_{cr,p}$=	8036,0	MPa	$\sigma_{cr,c}$=	8019,7	MPa
λ_p=	0,159		$A_{sl1,eff}$=	12965	mm^2
ψ=	0,00		$\beta_{a,c}$=	0,896	
ρ=	**1,000**		λ_c=	0,162	
			i=	65,8	mm
			e=	109,3	mm
Interaction - web			α_e=	0,639	
ξ=	0,002		Φ=	0,501	
ρ_c=	**1,000**		**χ_c=**	**1,000**	

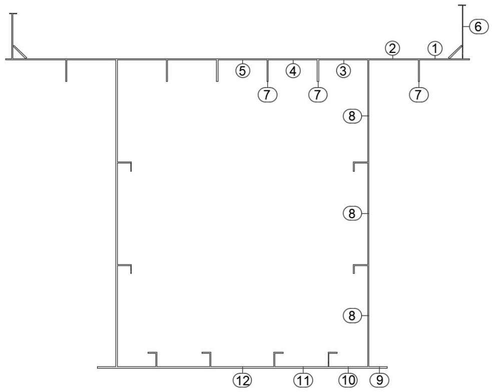


Picture 4.1-10 - Effective cross section – bending moment My, L=7,8m

$w_{y,el,eff} = 0,27261m^3, e_y = -0,062m$

These values are calculated for the upper fibres of the beam, which are determinative for assessment.

4.1.5 Effective cross section – bending moment M_z, midspan



Picture 4.1-11 - numbers of subpanels

Buckling - subpanels

subpanel no. 1			subpanel no. 2			subpanel no. 3			subpanel no. 4		
ψ=	0,81		ψ=	0,72		ψ=	0,63		ψ=	0,41	
k_σ=	4,410		k_σ=	4,621		k_σ=	4,880		k_σ=	5,635	
b=	331	mm	b=	545	mm	b=	545	mm	b=	544	mm
t=	18	mm	t=	18	mm	t=	18	mm	t=	18	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,308		λ_P=	0,496		λ_P=	0,483		λ_P=	0,448	
ρ=	1,000		ρ=	1,000		ρ=	1,000		ρ=	1,000	
b_{eff}=	***331***	***mm***	***b_{eff}=***	***545***	***mm***	***b_{eff}=***	***545***	***mm***	***b_{eff}=***	***544***	***mm***

subpanel no. 5			subpanel no. 6			subpanel no. 7			subpanel no. 8		
ψ=	-0,47		ψ=	1,00		ψ=	1,00		ψ=	1,00	
k_σ=	12,904		k_σ=	4,000		k_σ=	0,430		k_σ=	4,000	
b=	544	mm	b=	446	mm	b=	240	mm	b=	1140	mm
b_c=	373	mm	t=	12	mm	t=	16	mm	t=	14	mm
t=	18	mm	f_y=	235	MPa	f_y=	355	MPa	f_y=	235	MPa
f_y=	235	MPa	ε=	1,000		ε=	0,814		ε=	1,000	
ε=	1,000		λ_P=	0,654		λ_P=	0,990		λ_P=	1,434	
λ_P=	0,296		ρ=	1,000		ρ=	0,818		ρ=	0,590	
ρ=	1,000		***b_{eff}=***	**446**	**mm**	***b_{eff}=***	**196**	**mm**	***b_{eff}=***	***673***	***mm***
b_{eff}=	**373**	**mm**	b_{e1}=	178	mm				α0=	0,768	
b_{e1}=	149	mm	b_{e2}=	268	mm				b0=	1710	mm
b_{e2}=	224	mm							Le=	42800	mm
									κ=	0,031	
									β=	0,994	
									teff=	**13,9**	**mm**
									b_{e1}=	337	mm

subpanel no. 9			subpanel no. 10			subpanel no. 11			subpanel no. 12		
ψ=	0,87		ψ=	0,70		ψ=	0,43		ψ=	-0,55	
k_σ=	0,440		k_σ=	4,675		k_σ=	5,543		k_σ=	14,269	
b=	203	mm	b=	431	mm	b=	588	mm	b=	700	mm
t=	25	mm	t=	25	mm	t=	25	mm	b_c=	453	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	t=	25	mm
ε=	1,000		ε=	1,000		ε=	1,000		f_y=	235	MPa
λ_P=	0,431		λ_P=	0,281		λ_P=	0,352		ε=	1,000	
ρ=	1,000		ρ=	1,000		ρ=	1,000		λ_P=	0,261	
b_{eff}=	**203**	**mm**	***b_{eff}=***	***431***	***mm***	***b_{eff}=***	***588***	***mm***	ρ=	1,000	
			b_{e1}=	201	mm	b_{e1}=	257	mm	***b_{eff}=***	**453**	**mm**
			b_{e2}=	230	mm	b_{e2}=	331	mm	b_{e1}=	181	mm
									b_{e2}=	272	mm

Global buckling:

A thickness of the inner part of upper flange is not modified, because of validity of formula $\sigma_{com,Ed}$=161,8MPa<ρ_C*f_y/γ_{M1}=0,947*235/1,1=202,3MPa.

Plate effect - stiff. 7L			**Plate effect - stiff. 7R**		**Plate effect - both stiff.**		**Column effect - inner part**		
I_{sl}=	64700312	mm^4	62685500	mm^4	127385812	mm^4	$\sigma_{cr,sl}$=	1715,2	MPa
b_1=	560	mm	560	mm	840	mm	ψ_1=	0,63	
b_2=	560	mm	1680	mm	1960	mm	ψ_2=	0,41	
b=	1120	mm	2240	mm	2800	mm	b_1=	545	mm
t=	18	mm	18	mm	18	mm	b_2=	544	mm
a_c=	4302	mm	6216	mm	9285	mm	$b_{1,inf}$=	296	mm
a=	2388	mm	2388	mm	2388	mm	$b_{2,inf}$=	237	mm
$A_{sl,eff,loc}$=	24212	mm^2	24212	mm^2	24212	mm^2	$b_{1,eff}$=	545	mm
$A_{sl,1}$=	13710	mm^2	12346	mm^2	26056	mm^2	$b_{2,eff}$=	544	mm
A_{sl}=	26056	mm^3	26056	mm^3	26056,1	mm^3	$b_{1,inf,eff}$=	296	mm
$\beta_{a,c}$=	0,929		0,929		0,929		$b_{2,inf,eff}$=	237	mm
$\sigma_{cr,sl}$=	1876,7	MPa	1885,3	MPa	1784,6	MPa	$I_{sl,1}$=	64700312	mm^4
ν=	0,3		0,3		0,3		$A_{sl,1}$=	13710	mm^2
b_c=	2800	mm	2800	mm	2800	mm	a=	2388	mm
b_{sl1}=	2240	mm	1680	mm	1960	mm	b_c=	1493	mm
$\sigma_{cr,p1}$=	2345,8	MPa	3142,1	MPa	2549,4	MPa	b_{sl1}=	933	mm
λ_{p1}=	0,305		0,264		0,293		$\sigma_{cr,c}$=	2744,3	MPa
ψ=	0		0		0		$A_{sl1,eff}$=	13013	mm^2
ρ_1=	1,000		1,000		1,000		$\beta_{a,c}$=	0,949	
							λ_c=	0,285	
$\sigma_{cr,p}$=	2345,8	MPa					i=	68,7	mm
ρ=	**1,000**						e=	92,7	mm
Interaction - inner part							α_e=	0,611	
ξ=	0,000						Φ=	0,567	
ρ_c=	**0,947**						χ_c=	**0,947**	

A thickness of the outside part of upper flange is not modified, because of validity of formula $\sigma_{com,Ed}$=148,5MPa<ρ_C*f_y/γ_{M1}=0,964*235/1,1=205,9MPa.

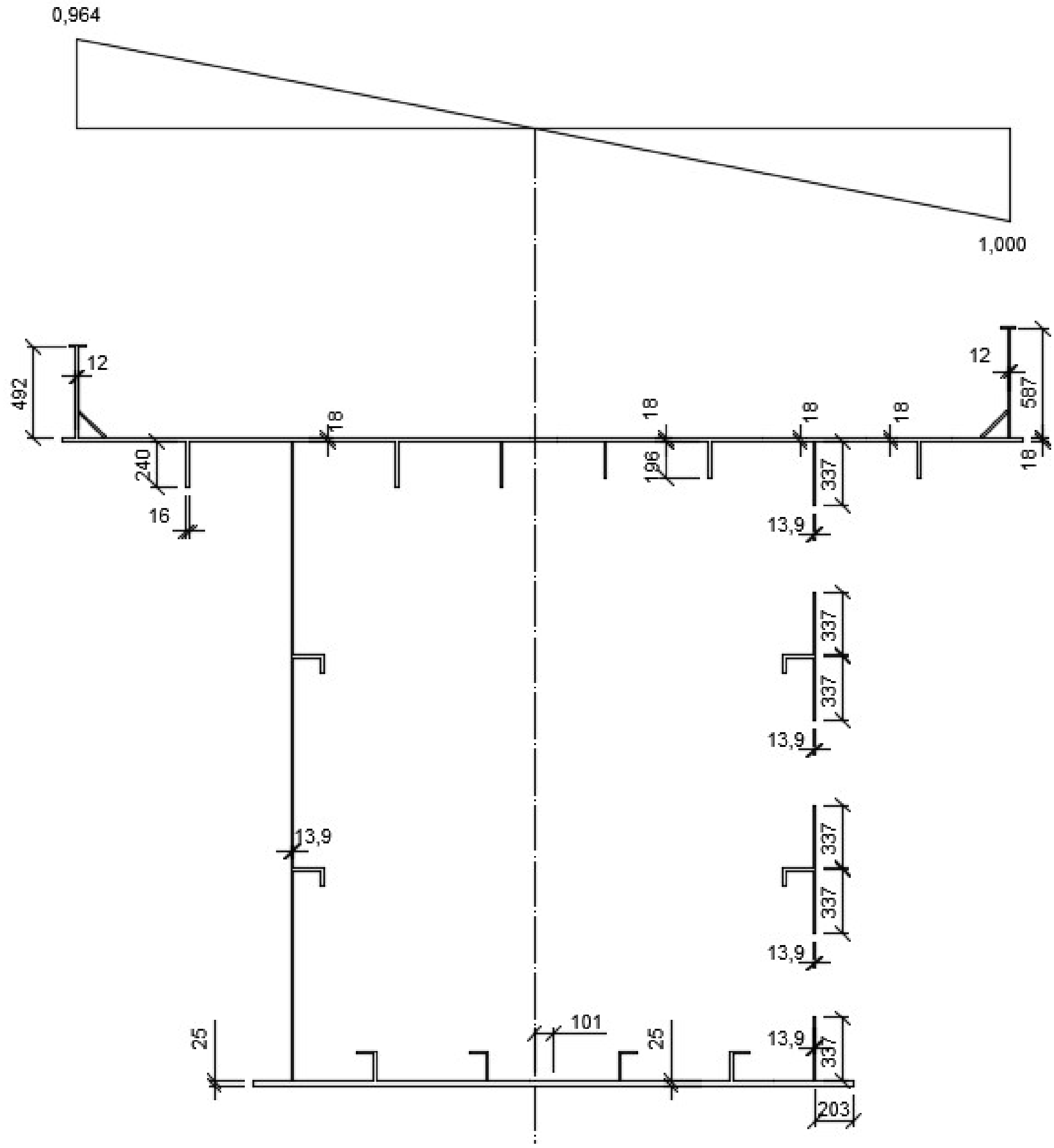
Plate effect - stiffener 7			**Column effect**		
I_{sl}=	61807899	mm^4	$\sigma_{cr,sl}$=	1898,7	MPa
b_1=	345	mm	ψ_1=	0,81	
b_2=	560	mm	ψ_2=	0,72	
b=	905	mm	b_1=	331	mm
t=	18	mm	b_2=	545	mm
a_c=	3521	mm	$b_{1,inf}$=	173	mm
a=	2388	mm	$b_{2,inf}$=	255	mm
$A_{c,eff,loc}$=	11134	mm^2	$b_{1,eff}$=	331	mm
A_c=	11832	mm^2	$b_{2,eff}$=	545	mm
$\beta_{a,c}$=	0,941		$b_{1,inf,eff}$=	173	mm
$A_{sl,1}$=	11832	mm^2	$b_{2,inf,eff}$=	255	mm
$\sigma_{cr,sl}$=	2297,0	MPa	$I_{sl,1}$=	61807899	mm^4
ν=	0,3		$A_{sl,1}$=	11832	mm^2
b_c=	2613	mm	a=	2388	mm
b_{sl1}=	1944	mm	b_c=	2613	mm
			b_{sl1}=	1944	mm
$\sigma_{cr,p}$=	3087,5	MPa	$\sigma_{cr,c}$=	2552,1	MPa
λ_p=	0,268		$A_{sl1,eff}$=	11134	mm^2
ψ=	0,59		$\beta_{a,c}$=	0,941	
ρ=	**1,000**		λ_c=	0,294	
			i=	72,3	mm
			e=	87,1	mm
Interaction - oustside part			α_e=	0,598	
ξ=	0,210		Φ=	0,572	
ρ_c=	**0,964**		**χ_c=**	**0,942**	

A thickness of web is not modified, because of validity of formula $\sigma_{com,Ed}$=62,5MPa< <ρ_C*f_y/γ_{M1}=0,847*235/1,1=181,0MPa.

Plate effect - one stiffener			Plate effect - both stiffeners		Column effect - web		
I_{sl}=	45557133	mm^4	91114266	mm^4	$\sigma_{cr,sl}$=	845,3	MPa
b_1=	1140	mm	1710	mm	ψ_1=	1,00	
b_2=	1140	mm	1710	mm	ψ_2=	1,00	
b=	2280	mm	3420	mm	b_1=	1140	mm
t=	14	mm	14	mm	b_2=	1140	mm
a_c=	8109	mm	13070	mm	$b_{1,inf}$=	570	mm
a=	2388	mm	2388	mm	$b_{2,inf}$=	570	mm
$A_{c,eff,loc}$=	25990	mm^2	25990	mm^2	$b_{1,eff}$=	673	mm
A_c=	39176	mm^2	39176	mm^2	$b_{2,eff}$=	673	mm
$\beta_{a,c}$=	0,663		0,663		$b_{1,inf,eff}$=	337	mm
$\sigma_{cr,sl}$=	851,6	MPa	846,2	MPa	$b_{2,inf,eff}$=	337	mm
ν=	0,3		0,3		$I_{sl,1}$=	45557133	mm^4
b_c=	2280	mm	3420	mm	$A_{sl,1}$=	19588	mm^2
b_{sl1}=	2280	mm	3420	mm	a=	2388	mm
$\sigma_{cr,p1}$=	851,6	MPa	846,2	MPa	b_c=	3420	mm
λ_{p1}=	0,428		0,429		b_{sl1}=	3420	mm
ψ=	1,00		1,00		$\sigma_{cr,c}$=	845,3	MPa
ρ_1=	**1,000**		**1,000**		$A_{sl1,eff}$=	12995	mm^2
					$\beta_{a,c}$=	0,663	
$\sigma_{cr,p}$=	846,2	MPa			λ_c=	0,429	
ρ=	**1,000**				i=	48,2	mm
					e=	95,4	mm
Interaction - web					α_e=	0,668	
ξ=	0,001				Φ=	0,669	
ρ_c=	**0,847**				**χ_c=**	**0,846**	

A thickness lower flange is not modified, because of validity of formula $\sigma_{com,Ed}$=10,4MPa< <ρ_C*f_y/γ_{M1}=0,875*235/1,1=186,9MPa. (10,4MPa is caused by horizontal load)

Plate effect - stiff. 7L			Plate effect - stiff. 7R		Plate effect - both stiff.		Column effect - inner part		
I_{sl}=	47617378	mm^4	48032792	mm^4	95650171	mm^4	$\sigma_{cr,sl}$=	1085,0	MPa
b_1=	444	mm	600	mm	744	mm	ψ_1=	0,70	
b_2=	600	mm	1756	mm	2056	mm	ψ_2=	0,43	
b=	1044	mm	2356	mm	2800	mm	b_1=	431	mm
t=	25	mm	25	mm	25	mm	b_2=	588	mm
a_c=	2921	mm	4750	mm	6512	mm	$b_{1,inf}$=	230	mm
a=	2388	mm	2388	mm	2388	mm	$b_{2,inf}$=	257	mm
$A_{sl,eff,loc}$=	32605	mm^2	32605	mm^2	32605	mm^2	$b_{1,eff}$=	431	mm
$A_{sl,1}$=	15951	mm^2	16554	mm^2	32505	mm^2	$b_{2,eff}$=	588	mm
A_{sl}=	32505	mm^3	32505	mm^3	32505	mm^3	$b_{1,inf,eff}$=	230	mm
$\beta_{a,c}$=	1,003		1,003		1,003		$b_{2,inf,eff}$=	257	mm
$\sigma_{cr,sl}$=	1565,3	MPa	1121,4	MPa	1088,7	MPa	$I_{sl,1}$=	47617378	mm^4
ν=	0,3		0,3		0,3		$A_{sl,1}$=	15951	mm^2
b_c=	2800	mm	2800	mm	2800	mm	a=	2388	mm
b_{sl1}=	2356	mm	1756	mm	2056	mm	b_c=	1493	mm
$\sigma_{cr,p1}$=	1860,3	MPa	1788,0	MPa	1482,6	MPa	b_{sl1}=	1059	mm
λ_{p1}=	0,356		0,363		0,399		$\sigma_{cr,c}$=	1530,5	MPa
ψ=	0		0		0		$A_{sl1,eff}$=	15951	mm^2
ρ_1=	1,000		1,000		1,000		$\beta_{a,c}$=	1,000	
							λ_c=	0,392	
$\sigma_{cr,p}$=	1482,6	MPa					i=	54,6	mm
ρ=	**1,000**						e=	93,8	mm
Interaction - inner part							α_e=	0,645	
ξ=	0,000						Φ=	0,639	
ρ_c=	**0,875**						**χ_c=**	**0,875**	

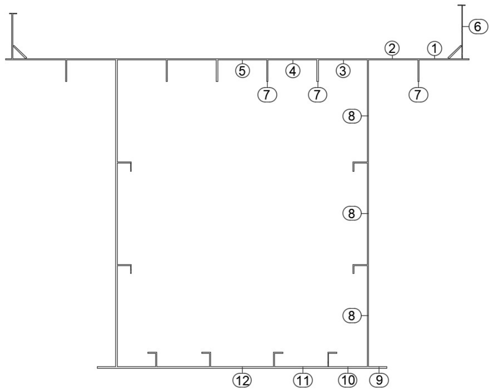


Picture 4.1-12 - Effective cross section – bending moment Mz, midspan

$w_{z,el,eff} = 0{,}23221m^3$
$e_z = -0{,}092m$

These values are calculated for the upper fibres of the beam, which are determinative for assessment.

4.1.6 Effective cross section – bending moment M_z,L=7,8m



Picture 4.1-13 - numbers of subpanels

Buckling - subpanels

subpanel no. 1			subpanel no. 2			subpanel no. 3			subpanel no. 4		
ψ=	0,81		ψ=	0,73		ψ=	0,63		ψ=	0,42	
k_σ=	4,407		k_σ=	4,615		k_σ=	4,868		k_σ=	5,592	
b=	331	mm	b=	545	mm	b=	545	mm	b=	544	mm
t=	14	mm	t=	14	mm	t=	14	mm	t=	14	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,397		λ_P=	0,638		λ_P=	0,621		λ_P=	0,579	
ρ=	1,000		ρ=	1,000		ρ=	1,000		ρ=	1,000	
b_{eff}=	***331***	***mm***	**b_{eff}=**	***545***	***mm***	**b_{eff}=**	***545***	***mm***	**b_{eff}=**	***544***	***mm***
b_{e1}=	158	mm	b_{e1}=	255	mm	b_{e1}=	249	mm	b_{e1}=	237	mm
b_{e2}=	173	mm	b_{e2}=	290	mm	b_{e2}=	296	mm	b_{e2}=	307	mm

subpanel no. 5			subpanel no. 6			subpanel no. 7			subpanel no. 8		
ψ=	-0,40		ψ=	1,00		ψ=	1,00		ψ=	1,00	
k_σ=	11,924		k_σ=	4,000		k_σ=	0,430		k_σ=	4,000	
b=	544	mm	b=	450	mm	b=	240	mm	b=	1140	mm
b_c=	391	mm	t=	12	mm	t=	16	mm	t=	14	mm
t=	14	mm	f_y=	235	MPa	f_y=	355	MPa	f_y=	235	MPa
f_y=	235	MPa	ε=	1,000		ε=	0,814		ε=	1,000	
ε=	1,000		λ_P=	0,660		λ_P=	0,990		λ_P=	1,434	
λ_P=	0,396		ρ=	1,000		ρ=	0,818		ρ=	0,590	
ρ=	1,000		**b_{eff}=**	**450**	**mm**	**b_{eff}=**	**196**	**mm**	**b_{eff}=**	***673***	***mm***
b_{eff}=	**391**	**mm**	b_{e1}=	180	mm				$\alpha 0$=	0,768	
b_{e1}=	157	mm	b_{e2}=	270	mm				b0=	1710	mm
b_{e2}=	235	mm							Le=	42800	mm
									κ=	0,031	
									β=	0,994	
									teff=	**13,9**	**mm**
									b_{e1}=	337	mm

subpanel no. 9			subpanel no. 10			subpanel no. 11			subpanel no. 12		
ψ=	0,87		ψ=	0,71		ψ=	0,44		ψ=	-0,49	
k_σ=	0,440		k_σ=	4,666		k_σ=	5,508		k_σ=	13,286	
b=	203	mm	b=	431	mm	b=	588	mm	b=	700	mm
t=	14	mm	t=	14	mm	t=	14	mm	b_c=	471	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	t=	14	mm
ε=	1,000		ε=	1,000		ε=	1,000		f_y=	235	MPa
λ_P=	0,770		λ_P=	0,502		λ_P=	0,630		ε=	1,000	
ρ=	0,982		ρ=	1,000		ρ=	1,000		λ_P=	0,483	
b_{eff}=	**199**	**mm**	**b_{eff}=**	***431***	***mm***	**b_{eff}=**	***588***	***mm***	ρ=	1,000	
			b_{e1}=	201	mm	b_{e1}=	257	mm	**b_{eff}=**	**471**	**mm**
			b_{e2}=	230	mm	b_{e2}=	331	mm	b_{e1}=	188	mm
									b_{e2}=	283	mm

Global buckling:

A thickness of the inner part of upper flange is not modified, because of validity of formula $\sigma_{com,Ed}$=82,7MPa<ρ_C*f_y/γ_{M1}=0,956*235/1,1=204,2MPa.

Plate effect - stiff. 7L			**Plate effect - stiff. 7R**		**Plate effect - both stiff.**		**Column effect - inner part**		
I_{sl}=	59849305	mm^4	60449455	mm^4	120298760	mm^4	$\sigma_{cr,sl}$=	1888,0	MPa
b_1=	560	mm	560	mm	840	mm	ψ_1=	0,63	
b_2=	560	mm	1680	mm	1960	mm	ψ_2=	0,42	
b=	1120	mm	2240	mm	2800	mm	b_1=	545	mm
t=	14	mm	14	mm	14	mm	b_2=	544	mm
a_c=	5094	mm	7437	mm	11052	mm	$b_{1,inf}$=	295	mm
a=	2388	mm	2388	mm	2388	mm	$b_{2,inf}$=	237	mm
$A_{sl,eff,loc}$=	20258	mm^2	20258	mm^2	20258	mm^2	$b_{1,eff}$=	545	mm
$A_{sl,1}$=	11521	mm^2	10548	mm^2	22070	mm^2	$b_{2,eff}$=	544	mm
A_{sl}=	22070	mm^3	22070	mm^3	22069,8	mm^3	$b_{1,inf,eff}$=	295	mm
$\beta_{a,c}$=	0,918		0,918		0,918		$b_{2,inf,eff}$=	237	mm
$\sigma_{cr,sl}$=	1978,4	MPa	2104,8	MPa	1985,4	MPa	$I_{sl,1}$=	59849305	mm^4
ν=	0,3		0,3		0,3		$A_{sl,1}$=	11521	mm^2
b_c=	2800	mm	2800	mm	2800	mm	a=	2388	mm
b_{sl1}=	2240	mm	1680	mm	1960	mm	b_c=	1511	mm
$\sigma_{cr,p1}$=	2473,0	MPa	3508,0	MPa	2836,3	MPa	b_{sl1}=	951	mm
λ_{p1}=	0,295		0,248		0,276		$\sigma_{cr,c}$=	2999,4	MPa
ψ=	0		0		0		$A_{sl1,eff}$=	10824	mm^2
ρ_1=	1,000		1,000		1,000		$\beta_{a,c}$=	0,939	
							λ_c=	0,271	
$\sigma_{cr,p}$=	2473,0	MPa					i=	72,1	mm
ρ=	**1,000**						e=	84,6	mm
Interaction - inner part							α_e=	0,596	
ξ=	0,000						Φ=	0,558	
ρ_c=	**0,956**						**χ_c=**	**0,956**	

A thickness of the outside part of upper flange is not modified, because of validity of formula $\sigma_{com,Ed}$=86,0MPa<ρ_C*f_y/γ_{M1}=0,961*235/1,1=205,3MPa.

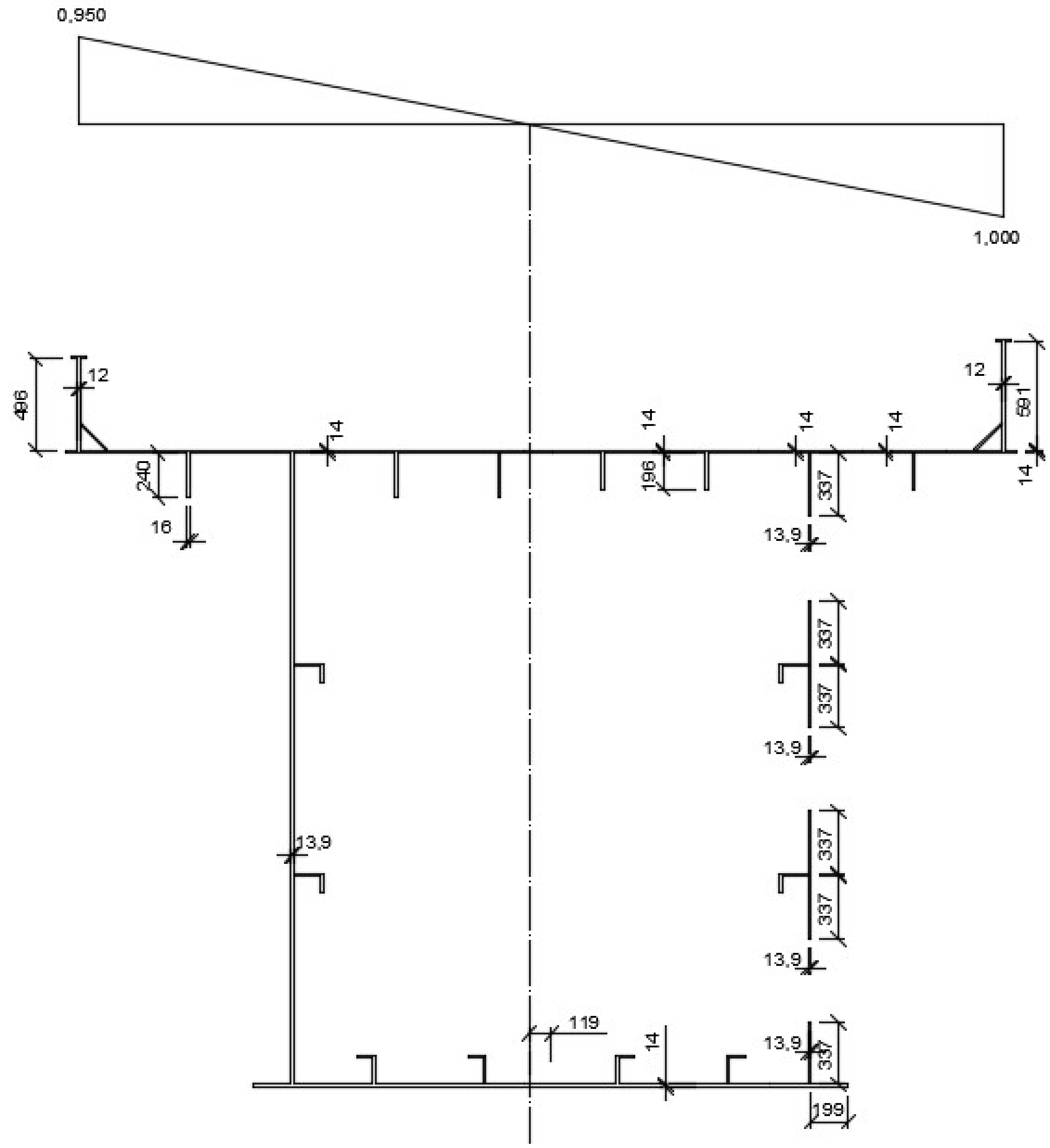
Plate effect - stiffener 7			**Column effect**		
I_{sl}=	56819684	mm^4	$\sigma_{cr,sl}$=	2053,4	MPa
b_1=	345	mm	ψ_1=	0,81	
b_2=	560	mm	ψ_2=	0,73	
b=	905	mm	b_1=	331	mm
t=	14	mm	b_2=	545	mm
a_c=	4163	mm	$b_{1,inf}$=	173	mm
a=	2388	mm	$b_{2,inf}$=	255	mm
$A_{c,eff,loc}$=	9359	mm^2	$b_{1,eff}$=	331	mm
A_c=	10057	mm^2	$b_{2,eff}$=	545	mm
$\beta_{a,c}$=	0,931		$b_{1,inf,eff}$=	173	mm
$A_{sl,1}$=	10057	mm^2	$b_{2,inf,eff}$=	255	mm
$\sigma_{cr,sl}$=	2273,9	MPa	$I_{sl,1}$=	56819684	mm^4
ν=	0,3		$A_{sl,1}$=	10057	mm^2
b_c=	2620	mm	a=	2388	mm
b_{sl1}=	1951	mm	b_c=	2620	mm
			b_{sl1}=	1951	mm
$\sigma_{cr,p}$=	3053,7	MPa	$\sigma_{cr,c}$=	2757,5	MPa
λ_p=	0,268		$A_{sl1,eff}$=	9359	mm^2
ψ=	0,59		$\beta_{a,c}$=	0,931	
ρ=	**1,000**		λ_c=	0,282	
			i=	75,2	mm
			e=	78,5	mm
Interaction - outside part			α_e=	0,584	
ξ=	0,107		Φ=	0,563	
ρ_c=	**0,961**		**χ_c=**	**0,951**	

A thickness of web is not modified, because of validity of formula $\sigma_{com,Ed}$=46,0MPa< <ρ_C*f_y/γ_{M1}=0,847*235/1,1=181,0MPa.

Plate effect - one stiffener			**Plate effect - both stiffeners**		**Column effect - web**		
I_{sl}=	45557133	mm^4	91114266	mm^4	$\sigma_{cr,sl}$=	845,3	MPa
b_1=	1140	mm	1710	mm	ψ_1=	1,00	
b_2=	1140	mm	1710	mm	ψ_2=	1,00	
b=	2280	mm	3420	mm	b_1=	1140	mm
t=	14	mm	14	mm	b_2=	1140	mm
a_c=	8109	mm	13070	mm	$b_{1,inf}$=	570	mm
a=	2388	mm	2388	mm	$b_{2,inf}$=	570	mm
$A_{c,eff,loc}$=	25990	mm^2	25990	mm^2	$b_{1,eff}$=	673	mm
A_c=	39176	mm^2	39176	mm^2	$b_{2,eff}$=	673	mm
$\beta_{a,c}$=	0,663		0,663		$b_{1,inf,eff}$=	337	mm
$\sigma_{cr,sl}$=	851,6	MPa	846,2	MPa	$b_{2,inf,eff}$=	337	mm
ν=	0,3		0,3		$I_{sl,1}$=	45557133	mm^4
b_c=	2280	mm	3420	mm	$A_{sl,1}$=	19588	mm^2
b_{sl1}=	2280	mm	3420	mm	a=	2388	mm
$\sigma_{cr,p1}$=	851,6	MPa	846,2	MPa	b_c=	3420	mm
λ_{p1}=	0,428		0,429		b_{sl1}=	3420	mm
ψ=	1,00		1,00		$\sigma_{cr,c}$=	845,3	MPa
ρ_1=	1,000		1,000		$A_{sl1,eff}$=	12995	mm^2
					$\beta_{a,c}$=	0,663	
$\sigma_{cr,p}$=	846,2	MPa			λ_c=	0,429	
ρ=	**1,000**				i=	48,2	mm
					e=	95,4	mm
Interaction - web					α_e=	0,668	
ξ=	0,001				Φ=	0,669	
ρ_c=	**0,847**				**χ_c=**	**0,846**	

A thickness lower flange is not modified, because of validity of formula $\sigma_{com,Ed}$=10,4MPa< <ρ_C*f_y/γ_{M1}=0,906*235/1,1=193,6MPa. (10,4MPa is caused by horizontal load)

Plate effect - stiff. 7L			**Plate effect - stiff. 7R**		**Plate effect - both stiff.**		**Column effect - inner part**		
I_{sl}=	38637664	mm^4	41460457	mm^4	80098121	mm^4	$\sigma_{cr,sl}$=	1338,9	MPa
b_1=	444	mm	600	mm	744	mm	ψ_1=	0,71	
b_2=	600	mm	1756	mm	2056	mm	ψ_2=	0,44	
b=	1044	mm	2356	mm	2800	mm	b_1=	431	mm
t=	14	mm	14	mm	14	mm	b_2=	588	mm
a_c=	4283	mm	7073	mm	9623	mm	$b_{1,inf}$=	230	mm
a=	2388	mm	2388	mm	2388	mm	$b_{2,inf}$=	258	mm
$A_{sl,eff,loc}$=	21404	mm^2	21404	mm^2	21404	mm^2	$b_{1,eff}$=	431	mm
$A_{sl,1}$=	10488	mm^2	10888	mm^2	21376	mm^2	$b_{2,eff}$=	588	mm
A_{sl}=	21376	mm^3	21376	mm^3	21375,9	mm^3	$b_{1,inf,eff}$=	230	mm
$\beta_{a,c}$=	1,001		1,001		1,001		$b_{2,inf,eff}$=	258	mm
$\sigma_{cr,sl}$=	1467,2	MPa	1401,9	MPa	1367,0	MPa	$I_{sl,1}$=	38637664	mm^4
ν=	0,3		0,3		0,3		$A_{sl,1}$=	10488	mm^2
b_c=	2800	mm	2800	mm	2800	mm	a=	2388	mm
b_{sl1}=	2356	mm	1756	mm	2056	mm	b_c=	1511	mm
$\sigma_{cr,p1}$=	1743,7	MPa	2235,3	MPa	1861,7	MPa	b_{sl1}=	1077	mm
λ_{p1}=	0,367		0,324		0,356		$\sigma_{cr,c}$=	1879,0	MPa
ψ=	0		0		0		$A_{sl1,eff}$=	10460	mm^2
ρ_1=	1,000		1,000		1,000		$\beta_{a,c}$=	0,997	
							λ_c=	0,353	
$\sigma_{cr,p}$=	1743,7	MPa					i=	60,7	mm
ρ=	**1,000**						e=	76,2	mm
Interaction - inner part							α_e=	0,603	
ξ=	0,000						Φ=	0,609	
ρ_c=	**0,906**						**χ_c=**	**0,906**	



Picture 4.1-14 - Effective cross section – bending moment Mz, L=7,8m

$w_{z,el,eff} = 0,20130m^3$
$\mathrm{e_z} = -0,116\mathrm{m}$

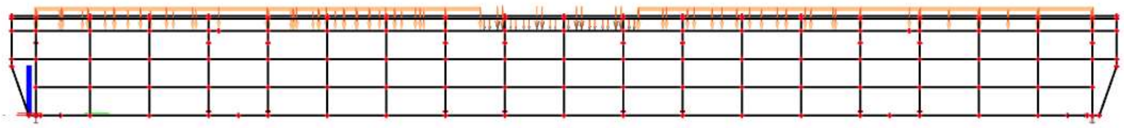
These values are calculated for the upper fibres of the beam, which are determinative for assessment.

4.1.7 Cross section in midspan

A determinative combination of load (formula 6.10b, gr12/gr14):

Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	maintenance load	1,20
	temperature max linear	0,90
	traction force	0,65
	wind Z-	1,01
	wind L with train	1,01
	LM71 - L - maximal M	1,39
	centrifugal force - maximal M	1,30
	nosing force - L - maximal M	1,30

Position of traffic load causing the critical bending moment (the middle point of traffic load is situated in stationing 21,48m)



Picture 4.1-15 - critical position of traffic load

Inner forces found out by integration of stress on cross section:

N_{Ed}	$\pm$192,4	kN
$V_{y,Ed}$	14,5	kN
$V_{z,Ed}$	47,4	kN
$M_{y,Ed}$	60864,5	kNm
$M_{z,Ed}$	6092,6	kNm

Resistance to bending moments:

$$M_{y,el,Rd} = w_{y,el,eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}33629 * \frac{235000}{1{,}1} = 71843{,}8kNm$$

$$M_{z,el,Rd} = w_{z,el,eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}23221 * \frac{235000}{1{,}1} = 49608{,}5kNm$$

- Flexural buckling:

  Buckling length:

  $L_{cr,y} = 42800mm \ldots span\ of\ a\ simple\ beam$

  Non-dimensional slenderness:

  $$\lambda_1 = \pi * \sqrt{\frac{E}{f_y}} = \pi * \sqrt{\frac{210000}{235}} = 93{,}913$$

  $$\lambda' = \frac{L_{cr}}{i_y} * \frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda_1} = \frac{42800}{1429} * \frac{\sqrt{\frac{0{,}22703}{0{,}28265}}}{93{,}913} = 0{,}286$$

  Reduction factor:

  $$\varnothing = 0{,}5 * [1 + \alpha * (\lambda' - 0{,}2) + \lambda'^2] = 0{,}5 * [1 + 0{,}34 * (0{,}286 - 0{,}2) + 0{,}286^2] = 0{,}556$$

  $$\chi_y = \frac{1}{\varnothing + \sqrt{\varnothing^2 - \lambda'^2}} = \frac{1}{0{,}556 + \sqrt{0{,}556^2 - 0{,}286^2}} = 0{,}968$$

- Torsional-flexural buckling:

Buckling length:

$L_{cr,\omega} = 42800mm \ldots span\ of\ a\ simple\ beam$

Non-dimensional slenderness:

$$\lambda_1 = \pi * \sqrt{\frac{E}{f_y}} = \pi * \sqrt{\frac{210000}{235}} = 93{,}913$$

$$\lambda_\omega = \sqrt{\frac{I_P}{\frac{I_\omega}{L_{cr,\omega}^2} + \frac{I_t}{25}}} = \sqrt{\frac{1{,}14773}{\frac{0{,}07471}{42{,}8^2} + \frac{0{,}41847}{25}}} = 8{,}270$$

$$I_P = I_y + I_z + A * a^2 = 0{,}57758 + 0{,}56958 + 0{,}28265 * 0{,}045^2 = 1{,}14773m^4$$

$$i_P = \sqrt{\frac{I_P}{A}} = \sqrt{\frac{1{,}14773}{0{,}28265}} * 10^3 = 2015{,}0mm$$

$$I_t = \frac{4 * A_k^2}{\oint_s \frac{ds}{t(s)}} = \frac{4 * (3{,}434 * 2{,}814)^2}{2 * \frac{3434}{14} + 2 * \frac{2814}{14}} = 0{,}41847$$

$$\lambda_z = \frac{L_{cr}}{i_z} = \frac{42800}{1420} = 30{,}141$$

$$\lambda_{z\omega} = \sqrt{{\lambda_z}^2 + \propto * {\lambda_\omega}^2} = \sqrt{30{,}141^2 + 0{,}000477 * 8{,}270^2} = 30{,}142$$

$$\propto = \left(\frac{a_z}{i_P}\right)^2 = \left(\frac{44}{2015}\right)^2 = 0{,}000477$$

$$\lambda'_{z\omega} = \lambda_{z\omega} * \frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda_1} = 30{,}142 * \frac{\sqrt{\frac{0{,}22703}{0{,}28265}}}{93{,}913} = 0{,}288$$

Reduction factor:

$$\varnothing = 0{,}5 * \left[1 + \alpha * (\lambda' - 0{,}2) + \lambda'^2\right] = 0{,}5 * [1 + 0{,}34 * (0{,}288 - 0{,}2) + 0{,}288^2] = = 0{,}556$$

$$\chi_{z\omega} = \frac{1}{\varnothing + \sqrt{\varnothing^2 - \lambda'^2}} = \frac{1}{0{,}556 + \sqrt{0{,}556^2 - 0{,}288^2}} = 0{,}969 \rightarrow determinative$$

Resistance to normal forces *(including an addition of longitudinal flanges, their yield strength is 120MPa higher):*

$$N_{Rd} = \chi * A_{eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}969 * (0{,}22703 * \frac{235000}{1{,}1} + 0{,}01882 * \frac{120000}{1{,}1}) = 48987{,}8kN$$

$$\frac{N_{Ed,max}}{N_{Rd}} = \frac{787{,}3}{48987{,}8} = 0{,}016 < 0{,}040 \ \rightarrow N_{Rd} = A_{eff} * \frac{f_{yk}}{\gamma_{M1}} =$$

$$= 0{,}27548 * \frac{235000}{1{,}1} + 0{,}01882 * \frac{120000}{1{,}1} = 60905{,}6kN$$

Buckling resistance is calculated supposing the minimal area of cross section (thickness of flanges 14mm). The maximal normal force on a beam 787,3kN (combination formula 6.10b, gr13, a cross section above the support) is still smaller than 4% of buckling resistance, so the buckling effects can be ignored for every cross section of the box girder.

Combination of compression and biaxial bending (upper fibre of a beam):

$$\eta_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_{y,N}}{M_{y,el,Rd}} + \frac{M_{z,Ed} + N_{Ed} * e_{z,N}}{M_{z,el,Rd}} =$$

$$= \frac{192{,}4}{60905{,}6} + \frac{60864{,}5 + 192{,}4 * 0{,}065}{71843{,}8} + \frac{6092{,}6 + 192{,}4 * 0{,}092}{49608{,}5} = 0{,}974$$

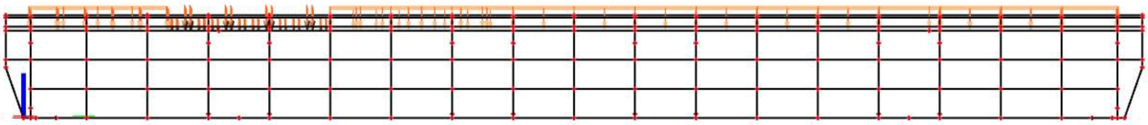
<u>SATISFACTORY (97,4%)</u>

4.1.8 Cross section in stationing L=7,8m

A determinative combination of load (formula 6.10b, gr12/gr14):

Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	maintenance load	1,20
	temperature max linear	0,90
	traction force	0,65
	wind Z-	1,01
	wind L with train	1,01
	LM71 - L - maximal M	1,39
	centrifugal force - maximal M	1,30
	nosing force - L - maximal M	1,30

Position of traffic load causing the critical bending moment (the middle point of traffic load is situated in stationing 7,88m)



Picture 4.1-16 - critical position of traffic load

Inner forces found out by integration of stress on cross section:

N_{Ed}	-318,4	kN
$V_{y,Ed}$	301,2	kN
$V_{z,Ed}$	3653,8	kN
$M_{y,Ed}$	36512,7	kNm
$M_{z,Ed}$	3628,0	kNm

Resistance to bending moments*:*

$$M_{y,el,Rd} = w_{y,el,eff} * \frac{f_{yk}}{\gamma_{M1}} = 0,27261 * \frac{235000}{1,1} = 58239,4kNm$$

$$M_{z,el,Rd} = w_{z,el,eff} * \frac{f_{yk}}{\gamma_{M1}} = 0,20130 * \frac{235000}{1,1} = 43005,0kNm$$

Resistance to normal forces *(including an addition of longitudinal flanges, their yield strength is 120MPa higher)*:

$$N_{Rd} = A_{eff} * \frac{f_{yk}}{\gamma_{M1}} = 0,22703 * \frac{235000}{1,1} + 0,01882 * \frac{120000}{1,1} = 50555,0kN$$

Combination of compression and biaxial bending (upper fibre of a beam):

$$\eta_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_{y,N}}{M_{y,el,Rd}} + \frac{M_{z,Ed} + N_{Ed} * e_{z,N}}{M_{z,el,Rd}} =$$

$$= \frac{318,4}{50555,0} + \frac{36512,7 + 318,4 * 0,062}{58239,4} + \frac{3628,0 + 318,4 * 0,116}{43005,0} = 0,719$$

SATISFACTORY (71,9%)

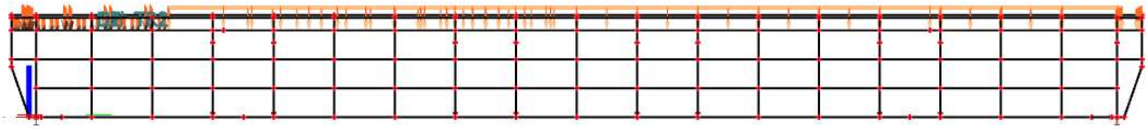
4.1.9 Cross section in stationing L=0,0m

<u>Maximal shear force:</u>

A determinative combination of load for shear force (formula 6.10b, gr12/gr14):

Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	maintenance load	1,20
	temperature maximal	0,90
	wind Z-, left	1,01
	wind L with train	1,01
	LM71 - L - maximal V	1,39
	centrifugal force - maximal V	1,30
	nosing force - L - maximal V	1,30

Position of traffic load causing the critical shear force (the middle point of traffic load is situated in stationing 3,04m)



Picture 4.1-17 – critical position of traffic load LM71

Inner forces found out by integration of stress on cross section:

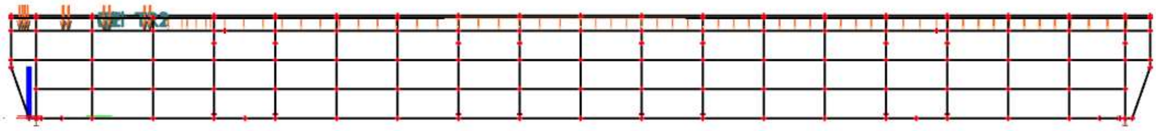
$V_{y,Ed}$	537,8	kN
$V_{z,Ed}$	5784,9	kN
$M_{x,Ed}$	2268,1	kNm

<u>Maximal torsional moment:</u>

A determinative combination of load for maximal torsional moment (formula 6.11b):

Linear - ultimate	self-weight	1,00
	other permanent load average	1,00
	other permanent load maximal	1,00
	temperature maximal	0,50
	wind Z-, left	0,50
	wind L with train	0,50
	derailment 1 - max V	1,00

Position of traffic load causing the critical shear force and torsional moment (the middle point of traffic load is situated in stationing 2,89m)



Picture 4.1-18 – critical position of traffic load – derailment, design situation I

Inner forces found out by integration of stress on cross section:

$V_{y,Ed}$	148,5	kN
$V_{z,Ed}$	4973,5	kN
$M_{x,Ed}$	4631,6	kNm

$$\psi L = L * \sqrt{\frac{GI_t}{EI_\omega}} = 42,8 * \sqrt{\frac{81 * 0,41847}{210 * 0,07471}} = 62,9 > 10 \rightarrow internal\ St.Venant\ torsion$$

Shear buckling for a whole web (for a=2200mm):

$$\alpha = \frac{a}{h_w} = \frac{2200}{3420} = 0,643$$

$$k_\tau = 4,1 + \frac{6,3 + 0,18 * \frac{I_{sl}}{t^3 * h_w}}{\alpha^2} + 2,2 * \sqrt[3]{\frac{I_{sl}}{t^3 * h_w}} =$$

$$= 4,1 + \frac{6,3 + 0,18 * \frac{2*33348536}{14^3*3420}}{0,643^2} + 2,2 * \sqrt[3]{\frac{2 * 33348536}{14^3 * 3420}} = 26,662$$

$$\sigma_E = 190000 * \left(\frac{t}{b}\right)^2 = 190000 * \left(\frac{14}{3420}\right)^2 = 3,184 MPa$$

$$\tau_{cr} = k_\tau * \sigma_E = 26,662 * 3,184 = 84,891 MPa$$

$$\lambda_w = 0,76 * \sqrt{\frac{f_{yw}}{\tau_{cr}}} = 0,76 * \sqrt{\frac{235}{84,891}} = 1,264\ (determinative)$$

Shear buckling for a subpanel (for a=2200mm):

$$\alpha = \frac{a}{h_w} = \frac{2200}{1140} = 1,930$$

$$k_\tau = 5,34 + 4 * \left(\frac{h_w}{a}\right)^2 = 5,34 + 4 * \left(\frac{1140}{2200}\right)^2 = 6,414$$

$$\sigma_E = 190000 * \left(\frac{t}{b}\right)^2 = 190000 * \left(\frac{14}{1140}\right)^2 = 28,655 MPa$$

$$\tau_{cr} = k_\tau * \sigma_E = 6,243 * 28,655 = 183,795 MPa$$

$$\lambda_w = 0,76 * \sqrt{\frac{f_{yw}}{\tau_{cr}}} = 0,76 * \sqrt{\frac{235}{183,795}} = 0,859$$

Shear resistance:

$$\chi_w = \frac{0,83}{\lambda_w} = \frac{0,83}{1,264} = 0,657$$

A left web is more loaded, because traffic load is not uniformly distributed, V_{Ed1}=3150,8kN:

$$V_{bw,Rd} = \frac{\chi_w * f_{yw} * h_w * t}{\sqrt{3} * \gamma_{M1}} = \frac{0,657 * 235 * 3420 * 14}{\sqrt{3} * 1,1 * 1000} = 3880,0\text{kN}$$

$$\tau_{t,Ed} = \frac{M_{x,d}}{2 * A_k * t_{min}} = \frac{2268,1}{2 * (3,434 * 2,814) * 14} = 8,383 MPa$$

$$V_{bw,T,Rd} = \left[1 - \frac{\tau_{t,Ed}}{(f_y/\sqrt{3})/\gamma_{M0}}\right] * V_{bw,Rd} = \left[1 - \frac{8,383}{(235/\sqrt{3})/1,0}\right] * 3880 = 3640,3 kN >$$
$$> 3150,8\text{kN}$$

<u>SATISFACTORY (86,5%)</u>

Resistance to maximal torsional moment, V_{Ed1}=3287,9kN (shear force in one web):

$$\tau_{t,Ed} = \frac{M_{x,d}}{2 * A_k * t_{min}} = \frac{4631,6}{2 * (3,434 * 2,814) * 14} = 17,118 MPa$$

$$\frac{T_{Ed}}{T_{Rd}} = \frac{\tau_{t,Ed}}{(f_y/\sqrt{3})/\gamma_{M0}} = \frac{17,118}{235/\sqrt{3}/1,0} = 0,126$$

$$V_{bw,T,Rd} = [1 - T_{Ed}/T_{Rd}] * V_{bw,Rd} = [1 - 0,126] * 3880,0 = 3390,5 kN > 3287,9\text{kN}$$

<u>SATISFACTORY (97,0%)</u>

4.1.10 Combination of share force and bending moment

Assessment is done in place of $V_{Ed} = \frac{V_{bw,T,Rd}}{2}$, the cross section is found by iteration for following combination of loads, which causes maximal bending moment:

Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	maintenance load	1,20
	temperature max linear	0,90
	traction force	1,30
	wind Z-	1,01
	wind L with train	1,01
	LM71 - L - maximal M	1,39
	centrifugal force - maximal M	1,30
	nosing force - L - maximal M	1,30

Shear buckling for a whole web (for a=2400mm):

$$\alpha = \frac{a}{h_w} = \frac{2400}{3420} = 0{,}702$$

$$k_\tau = 4{,}1 + \frac{6{,}3 + 0{,}18 * \frac{I_{sl}}{t^3 * h_w}}{\alpha^2} + 2{,}2 * \sqrt[3]{\frac{I_{sl}}{t^3 * h_w}} =$$

$$= 4{,}1 + \frac{6{,}3 + 0{,}18 * \frac{2*33348536}{14^3*3420}}{0{,}702^2} + 2{,}2 * \sqrt[3]{\frac{2 * 33348536}{14^3 * 3420}} = 23{,}710$$

$$\sigma_E = 190000 * \left(\frac{t}{b}\right)^2 = 190000 * \left(\frac{14}{3420}\right)^2 = 3{,}184MPa$$

$$\tau_{cr} = k_\tau * \sigma_E = 23{,}710 * 3{,}184 = 75{,}493MPa$$

$$\lambda_w = 0{,}76 * \sqrt{\frac{f_{yw}}{\tau_{cr}}} = 0{,}76 * \sqrt{\frac{235}{75{,}493}} = 1{,}341\ (determinative)$$

Shear buckling for a subpanel (for a=2400mm):

$$\alpha = \frac{a}{h_w} = \frac{2400}{1140} = 2{,}105$$

$$k_\tau = 5{,}34 + 4 * \left(\frac{h_w}{a}\right)^2 = 5{,}34 + 4 * \left(\frac{1140}{2400}\right)^2 = 6{,}243$$

$$\sigma_E = 190000 * \left(\frac{t}{b}\right)^2 = 190000 * \left(\frac{14}{1140}\right)^2 = 28{,}655MPa$$

$$\tau_{cr} = k_\tau * \sigma_E = 6{,}243 * 28{,}655 = 178{,}879MPa$$

$$\lambda_w = 0{,}76 * \sqrt{\frac{f_{yw}}{\tau_{cr}}} = 0{,}76 * \sqrt{\frac{235}{178{,}879}} = 0{,}871$$

Shear resistance:

- For $V_{Ed} = \frac{V_{bw,T,Rd}}{2} = 3520{,}3$kN (stationing 8,75m)

N_{Ed}	-310,3	kN
$M_{x,Ed}$	1234,8	kNm
$M_{y,Ed}$	38746,4	kNm

$$\chi_w = \frac{0{,}83}{\lambda_w} = \frac{0{,}83}{1{,}341} = 0{,}619$$

$$V_{bw,Rd} = \frac{2 * \chi_w * f_{yw} * h_w * t}{\sqrt{3} * \gamma_{M1}} = \frac{2 * 0{,}619 * 235 * 3420 * 14}{\sqrt{3} * 1{,}1 * 1000} = 7311{,}1\text{kN}$$

$$\tau_{t,Ed} = \frac{M_{x,d}}{2 * A_k * t_{min}} = \frac{1234{,}8}{2 * (3{,}434 * 2{,}814) * 14} = 4{,}564MPa$$

$$V_{bw,T,Rd} = \left[1 - \frac{\tau_{t,Ed}}{(f_y/\sqrt{3})/\gamma_{M0}}\right] * V_{bw,Rd} = \left[1 - \frac{4{,}564}{(235/\sqrt{3})/1{,}0}\right] * 7311{,}1 = 7040{,}6kN$$

A [m²]	3,2838e-01	
Ay [m²], Az [m²]	1,9094e-01	1,1766e-01
Iy [m⁴], Iz [m⁴]	7,4082e-01	6,3506e-01
Wel.y [m³], Wel.z [m³]	3,4615e-01	2,4598e-01
Wpl.y [m³], Wpl.z [m³]	4,5826e-01	4,0787e-01
Iw [m⁶], It [m⁴]	0,0000e+00	2,9792e-04
dy [mm], dz [mm]	0	0
cY.UCS [mm], cZ.UCS [mm]	-10	1910
α [deg]	1,98	
IYZLCS [m⁴]	-3,6447e-03	
Mpl.y.+ [Nm], Mpl.y.- [Nm]	1,08e+08	1,08e+08
Mpl.z.+ [Nm], Mpl.z.- [Nm]	9,58e+07	9,58e+07
AL [m²/m], AD [m²/m]	2,1695e+01	4,0177e+01

Picture

A [m²]	2,1883e-01	
Ay [m²], Az [m²]	1,8405e-01	5,5046e-02
Iy [m⁴], Iz [m⁴]	6,3765e-01	4,2459e-01
Wel.y [m³], Wel.z [m³]	3,1484e-01	1,6379e-01
Wpl.y [m³], Wpl.z [m³]	3,2733e-01	2,5620e-01
Iw [m⁶], It [m⁴]	0,0000e+00	7,4830e-05
dy [mm], dz [mm]	0	0
cY.UCS [mm], cZ.UCS [mm]	-15	2000
α [deg]	0,90	
IYZLCS [m⁴]	-3,3491e-03	
Mpl.y.+ [Nm], Mpl.y.- [Nm]	7,69e+07	7,69e+07
Mpl.z.+ [Nm], Mpl.z.- [Nm]	6,02e+07	6,02e+07
AL [m²/m], AD [m²/m]	2,4100e+01	2,4583e+01

Picture

Picture 4.1-19 – bending moment resistance

$$\mu_1 = \frac{M_{Ed}}{M_{el,N,Rd}} = \frac{38746{,}4 + 310{,}3 * 0{,}065}{73667{,}7 * (1 - \frac{310{,}3}{0{,}32838*235000/1{,}1})} = 0{,}529 < \frac{M_{f,N,Rd}}{M_{el,N,Rd}} =$$

$$= \frac{66187{,}8 * (1 - \frac{310{,}3}{0{,}21883*235000/1{,}1})}{73667{,}7 * (1 - \frac{310{,}3}{0{,}32838*235000/1{,}1})} = 0{,}896$$

- Another critical cross section is the stationing L=7,8m (change of thickness of flanges):

N_{Ed}	-318,4	kN
$M_{x,Ed}$	1327,5	kNm
$M_{y,Ed}$	36512,7	kNm

A [m²]	2,7417e-01	
Ay [m²], Az [m²]	2,1673e-01	2,2179e-01
Iy [m⁴], Iz [m⁴]	5,6303e-01	5,5990e-01
Wel.y [m³], Wel.z [m³]	1,8920e-01	1,7659e-01
Wpl.y [m³], Wpl.z [m³]	3,3039e-01	3,3549e-01
Iw [m⁶], It [m⁴]	0,0000e+00	1,7789e-04
dy [mm], dz [mm]	0	0
cY.UCS [mm], cZ.UCS [mm]	-8	2035
α [deg]	31,07	
IYZLCS [m⁴]	-1,3811e-03	
Mpl.y.+ [Nm], Mpl.y.- [Nm]	7,76e+07	7,76e+07
Mpl.z.+ [Nm], Mpl.z.- [Nm]	7,88e+07	7,88e+07
AL [m²/m], AD [m²/m]	4,0285e+01	4,0285e+01

Picture

A [m²]	1,6457e-01	
Ay [m²], Az [m²]	1,2908e-01	1,4585e-01
Iy [m⁴], Iz [m⁴]	4,4633e-01	3,4999e-01
Wel.y [m³], Wel.z [m³]	1,9749e-01	1,3427e-01
Wpl.y [m³], Wpl.z [m³]	2,0664e-01	2,0241e-01
Iw [m⁶], It [m⁴]	0,0000e+00	3,2329e-05
dy [mm], dz [mm]	0	0
cY.UCS [mm], cZ.UCS [mm]	-12	2244
α [deg]	0,56	
IYZLCS [m⁴]	-9,3733e-04	
Mpl.y.+ [Nm], Mpl.y.- [Nm]	4,86e+07	4,86e+07
Mpl.z.+ [Nm], Mpl.z.- [Nm]	4,76e+07	4,76e+07
AL [m²/m], AD [m²/m]	2,4691e+01	2,4691e+01

Picture

Picture 4.1-20 – bending moment resistance

$$\eta_1 = \frac{M_{Ed}}{M_{el,N,Rd}} = \frac{36512{,}7 + 318{,}4 * 0{,}062}{59369{,}4 * (1 - \frac{318{,}4}{0{,}27417*235000/1{,}1})} = 0{,}619 < \frac{M_{f,N,Rd}}{M_{el,N,Rd}} =$$

$$= \frac{42491{,}2 * (1 - \frac{318{,}4}{0{,}16457*235000/1{,}1})}{59369{,}4 * (1 - \frac{318{,}4}{0{,}27417*235000/1{,}1})} = 0{,}713$$

Shear force and bending moment do not interact.

4.1.11 Resistance to transverse forces

<u>Subpanel:</u>

Effective loaded length:

$$m_1 = \frac{f_{yf} * b_f}{f_{yw} * t_w} = \frac{235 * 434}{235 * 14} = 31{,}000$$

$$m_2 = 0{,}02 * \left(\frac{h_w}{t_f}\right)^2 = 0{,}02 * \left(\frac{1140}{14}\right)^2 = 132{,}612$$

$$l_y = s_s + 2 * t_f * \left(1 + \sqrt{m_1 + m_2}\right) = 425 + 2 * 14 * \left(1 + \sqrt{31 + 132{,}612}\right) = 811{,}2mm$$

Reduction factor for effective length:

$$k_F = 6 + 2 * \left[\frac{h_w}{a}\right]^2 = 6 + 2 * \left[\frac{1140}{2400}\right]^2 = 6{,}451$$

$$F_{cr} = 0{,}9 * k_F * E * \frac{t_w{}^3}{h_w} = 0{,}9 * 6{,}451 * 210 * \frac{14^3}{1140} = 2934{,}7kN$$

$$\lambda_f' = \sqrt{\frac{l_y * t_w * f_{yw}}{F_{cr}}} = \sqrt{\frac{811{,}2 * 14 * 235}{2934{,}7 * 10^3}} = 0{,}954$$

$$\chi_F = \frac{0{,}5}{\lambda_f{}'} = \frac{0{,}5}{0{,}954} = 0{,}524$$

Resistance to local buckling:

$$L_{eff} = \chi_F * l_y = 0{,}524 * 811{,}2 = 425{,}3mm$$

$$F_{Rd} = \frac{f_{yw} * L_{eff} * t_w}{\gamma_{M1}} = \frac{235 * 425{,}3 * 14}{1{,}1} * 10^{-3} = 1272{,}1kN$$

<u>Whole web:</u>

Effective loaded length:

$$m_1 = \frac{f_{yf} * b_f}{f_{yw} * t_w} = \frac{235 * 434}{235 * 14} = 31$$

$$m_2 = 0{,}02 * \left(\frac{h_w}{t_f}\right)^2 = 0{,}02 * \left(\frac{3420}{14}\right)^2 = 1193{,}5$$

$$l_y = s_s + 2 * t_f * \left(1 + \sqrt{m_1 + m_2}\right) = 425 + 2 * 14 * \left(1 + \sqrt{31 + 1193{,}5}\right) = 1432{,}8mm$$

Reduction factor for effective length:

$$k_F = 6 + 2 * \left[\frac{h_w}{a}\right]^2 = 6 + 2 * \left[\frac{3420}{2400}\right]^2 = 10{,}061$$

$$F_{cr} = 0{,}9 * k_F * E * \frac{t_w{}^3}{h_w} = 0{,}9 * 10{,}061 * 210 * \frac{14^3}{3420} = 1525{,}7kN$$

$$\lambda_f' = \sqrt{\frac{l_y * t_w * f_{yw}}{F_{cr}}} = \sqrt{\frac{1432{,}8 * 14 * 235}{1525{,}7 * 10^3}} = 1{,}758$$

$$\chi_F = \frac{0{,}5}{\lambda_f{}'} = \frac{0{,}5}{1{,}758} = 0{,}284$$

Resistance to local buckling:

$$L_{eff} = \chi_F * l_y = 0{,}284 * 1432{,}8 = 407{,}6mm$$

$$F_{Rd} = \frac{f_{yw} * L_{eff} * t_w}{\gamma_{M1}} = \frac{235 * 407{,}6 * 14}{1{,}1} * 10^{-3} = 1219{,}0kN \rightarrow determinative$$

$$F_{Ed} = \frac{0{,}332 + 0{,}498}{2} * \left(\frac{2{,}765}{2} + 0{,}300\right) * 125 * 1{,}3 * 1{,}071 + (0{,}115 + 0{,}549) * \frac{2{,}765}{4} * 1{,}3 =$$

$$= 122{,}1kN \; (\ldots wheel\ force\ with\ maximal\ eccenriccity, cenrifugal\ and\ nosing\ force)$$

$$\eta_2 = \frac{F_{Ed}}{F_{Rd}} = \frac{122{,}1}{1219{,}0} = 0{,}100 \leq 1$$

Interaction between transverse force, bending moment and axial force (midspan):

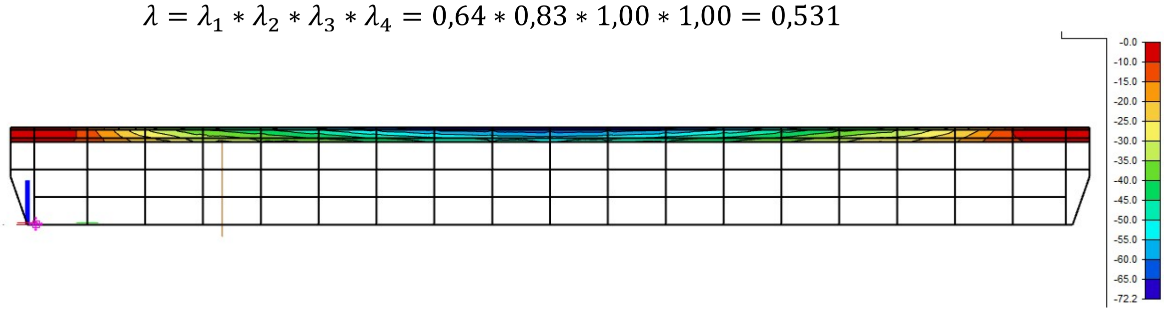
$$\eta_2 + 0{,}8 * \mu_1 = 0{,}100 + 0{,}8 * 0{,}697 = 0{,}658 \leq 1{,}4 \rightarrow not\ determinative\ influence$$

4.1.12 Fatigue

Upper fibre of box girder (EN 1993-1-9, table 8.1, detail 1):

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$$\lambda_1 = 0{,}64\ (for\ span\ L = 42{,}8m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 0{,}64 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}531$$



Picture 4.1-21 - maximal stress caused by traffic load

$$\Phi_3 * \Delta\sigma_{71} = 1{,}071 * 72{,}2 = 77{,}326MPa$$

$$1{,}0 * 0{,}531 * 77{,}326 = 41{,}060MPa \leq \frac{160}{1{,}15} = 139{,}130MPa$$

SATISFACTORY (29,5%)

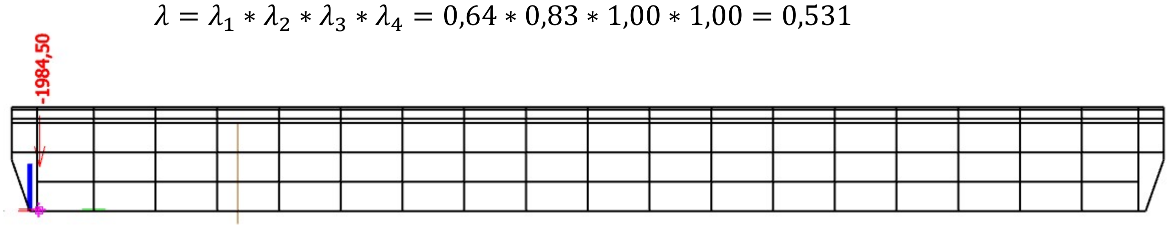
Load capacity:

$$z_{\text{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71}} = \frac{139{,}130}{41{,}060} = \mathbf{3,387} \rightarrow not\ determinative$$

Upper fibre of web (EN 1993-1-9, table 8.1, detail 6):

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\tau_{71} \leq \frac{\Delta\tau_C}{\gamma_{Mf}}$$

$$\lambda_1 = 0{,}64\ (for\ span\ L = 42{,}8m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 0{,}64 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}531$$



Picture 4.1-22 - maximal shear force caused by traffic load

$$\Phi_3 * \Delta\tau = \Phi_3 * \frac{V_{Ed,1} * S_f}{t * I_y} = 1{,}071 * \frac{1948{,}5 * 0{,}15266}{14 * 0{,}55558} = 40{,}958MPa$$

$$1{,}0 * 0{,}531 * 40{,}958 = 21{,}749MPa \leq \frac{100}{1{,}15} = 86{,}957MPa$$

SATISFACTORY (25,0%)

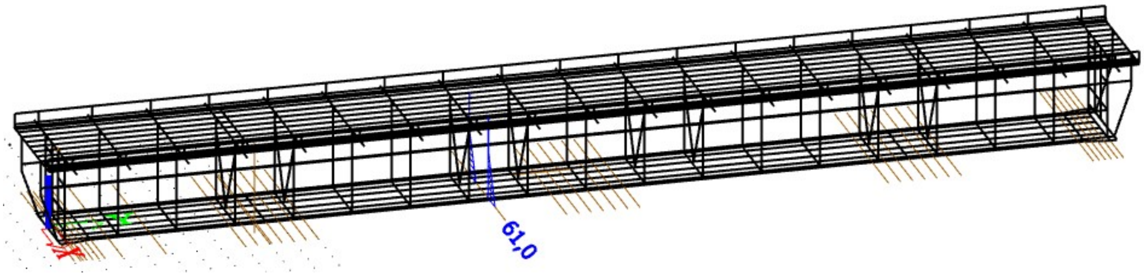
Load capacity:

$$z_{\text{LM71}} = \frac{\Delta\tau_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\tau_{71}} = \frac{86{,}957}{21{,}749} = \mathbf{3,997} \rightarrow not\ determinative$$

<u>Bolted joint of web - double covered symmetrical connection with preloaded high strength bolts (EN 1993-1-9, table 8.1, detail 8):</u>

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$\lambda_1 = 0{,}64\ (for\ span\ L = 42{,}8m)$
$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$
$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$
$\lambda_4 = 1{,}00\ (monorail)$
$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 0{,}64 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}531$



Picture 4.1-23 - maximal stress caused by traffic load

$$\Phi_3 * \Delta\sigma_{71} = 1{,}071 * 61{,}0 = 65{,}331 MPa$$

$$1{,}0 * 0{,}531 * 65{,}331 = 34{,}691 MPa \leq \frac{112}{1{,}15} = 97{,}391 MPa$$

<u>SATISFACTORY (35,6%)</u>

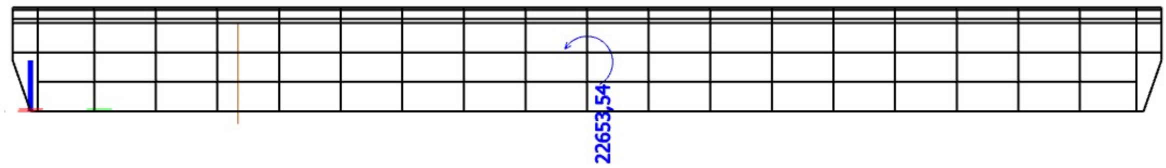
Load capacity:

$$z_{\text{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71}} = \frac{97{,}391}{34{,}691} = \mathbf{2{,}807} \rightarrow not\ determinative$$

<u>Welds of web and flanges - automatic or fully mechanized fillet weld carried out form both sides (EN 1993-1-9, table 8.2, detail 3), welds of longitudinal stiffener and lower flange (EN 1993-1-9, table 8.4, detail 1):</u>

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$\lambda_1 = 0{,}64\ (for\ span\ L = 42{,}8m)$
$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$
$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$
$\lambda_4 = 1{,}00\ (monorail)$
$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 0{,}64 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}531$



Picture 4.1-24 - maximal bending moment caused by traffic load

Stress in weld (lower welds, critical detail category is 56 - stiffeners)

$$\Phi_3 * \Delta\sigma_{71} = 1{,}071 * \frac{M_{Ed}}{w_y} = 1{,}071 * \frac{22653{,}5}{0{,}39366} * 10^{-3} = 61{,}632 MPa$$

$$1{,}0 * 0{,}531 * 61{,}632 = 32{,}727 MPa \leq \frac{56}{1{,}15} = 48{,}696 MPa$$

<u>SATISFACTORY (67,2%)</u>

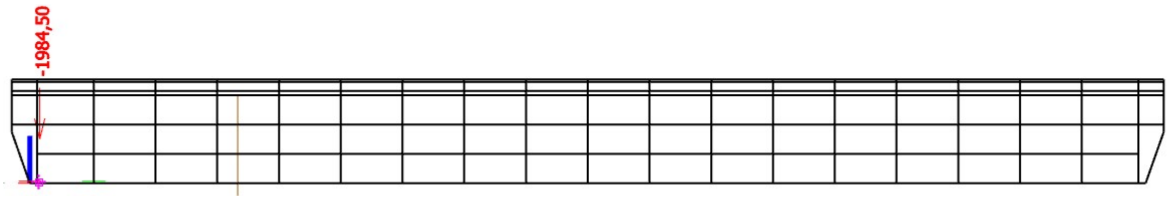
Load capacity:

$$z_{\text{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71}} = \frac{48{,}696}{32{,}727} = \mathbf{1{,}488} \rightarrow not\ determinative$$

Welds of web and flanges - continuous fillet weld transmitting a shear flow (EN 1993-1-9, table 8.5, detail 8):

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\tau_{71} \leq \frac{\Delta\tau_C}{\gamma_{Mf}}$$

$$\lambda_1 = 0{,}64\ (for\ span\ L = 42{,}8m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 0{,}64 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}531$$



Picture 4.1-25 - maximal shear force caused by traffic load

Stress in weld (upper welds are deciding)

$$\Phi_2 * \Delta\tau = \Phi_3 * \frac{V_{Ed,1} * S_f}{2 * a * I_y} = 1{,}071 * \frac{1948{,}5 * 0{,}15266}{2 * 6 * 0{,}55558} = 47{,}785MPa$$

$$1{,}0 * 0{,}531 * 47{,}785 = 25{,}374MPa \leq \frac{80}{1{,}15} = 69{,}565MPa$$

SATISFACTORY (36,5%)

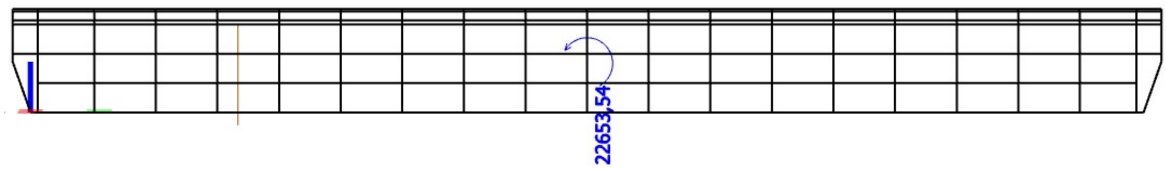
Load capacity:

$$z_{\mathrm{LM71}} = \frac{\Delta\tau_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\tau_{71}} = \frac{69{,}565}{25{,}374} = \mathbf{2{,}742} \rightarrow not\ determinative$$

Vertical weld of plate framing the track ballast and its stiffener – vertical stiffeners welded to beam or plate girder (EN 1993-1-9, table 8.4, detail 7):

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$$\lambda_1 = 0{,}64\ (for\ span\ L = 42{,}8m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 0{,}64 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}531$$



Picture 4.1-26 - maximal bending moment caused by traffic load

$$\Phi_3 * \Delta\sigma_{71} = \Phi_3 * \frac{M_{Ed}}{w_y} = 1{,}071 * \frac{22653{,}5}{0{,}33629} * 10^{-3} = 72{,}146MPa$$

$$1{,}0 * 0{,}531 * 72{,}146 = 38{,}310MPa \leq \frac{80}{1{,}15} = 69{,}565MPa$$

SATISFACTORY (55,1%) – *for transverse welds of vertical stiffeners of webs is a stress lower, so it is not necessary to assess*

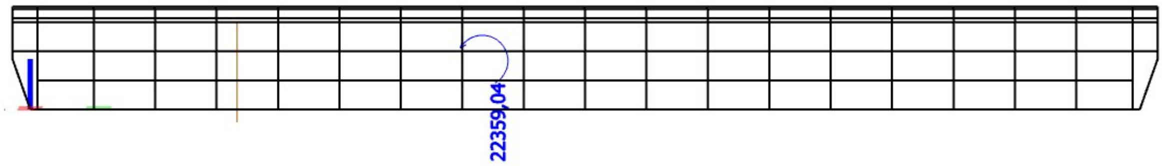
Load capacity:

$$z_{\mathrm{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71}} = \frac{69{,}565}{33{,}902} = \mathbf{1{,}816} \rightarrow not\ determinative$$

Constructional welds of lower flange – longitudinal butt weld, not grinded (EN 1993-1-9, table 8.2, detail 10) and transverse splices, grounded flash to flange surface (EN 1993-1-9, table 8.3, detail 1):

$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$ (all welds are in detail category 112)

$\lambda_1 = 0{,}64\ (for\ span\ L = 42{,}8m)$
$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$
$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$
$\lambda_4 = 1{,}00\ (monorail)$
$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 0{,}64 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}531$



Picture 4.1-27 - maximal bending moment caused by traffic load

$$\Phi_3 * \Delta\sigma_{71} = \Phi_3 * \frac{M_{Ed}}{w_y} = 1{,}071 * \frac{22359{,}0}{0{,}38844} * 10^{-3} = 61{,}648MPa$$

$$1{,}0 * 0{,}531 * 61{,}648 = 32{,}735MPa \leq \frac{112}{1{,}15} = 97{,}391MPa$$

SATISFACTORY (33,6%)

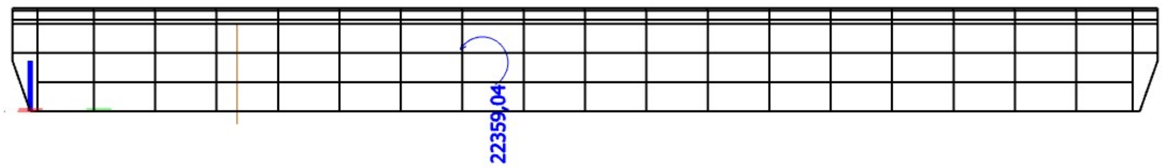
Load capacity:

$$z_{\text{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71}} = \frac{97{,}391}{32{,}735} = \mathbf{2{,}975} \rightarrow not\ determinative$$

Constructional welds of upper flange – longitudinal butt weld, not grinded (EN 1993-1-9, table 8.2, detail 10) and transverse splices, not grounded flush (EN 1993-1-9, table 8.3, detail 9):

$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$ (transverse splices have determinative detail category 80)

$\lambda_1 = 0{,}64\ (for\ span\ L = 42{,}8m)$
$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$
$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$
$\lambda_4 = 1{,}00\ (monorail)$
$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 0{,}64 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}531$



Picture 4.1-28 - maximal bending moment caused by traffic load

$$\Phi_3 * \Delta\sigma_{71} = \Phi_3 * \frac{M_{Ed}}{w_y} = 1{,}071 * \frac{22359{,}0}{0{,}46352} * 10^{-3} = 51{,}663MPa$$

$$1{,}0 * 0{,}531 * 51{,}663 = 27{,}433MPa \leq \frac{80}{1{,}15} = 69{,}565MPa$$

SATISFACTORY (39,4%)

Load capacity:

$$z_{\text{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff}*\lambda*\Phi_3*\Delta\sigma_{71}} = \frac{69{,}565}{27{,}433} = \mathbf{2{,}536} \rightarrow not\ determinative$$

4.1.13 Loading capacity

Midspan:

N_{Ed}	192,4	kN	$N_{Ed,LM71}$	189,3	kN	$N_{Ed,rs}$	3,1	kN
$M_{y,Ed}$	60864,5	kNm	$M_{y,Ed,LM71}$	32060,7	kNm	$M_{y,Ed,rs}$	28803,8	kNm
$M_{z,Ed}$	6092,6	kNm	$M_{z,Ed,LM71}$	3362,9	kNm	$M_{z,Ed,rs}$	2729,7	kNm

$$\eta_{1,rs} = \frac{N_{Ed,rs}}{N_{Rd}} + \frac{M_{y,Ed,rs} + N_{Ed,rs} * e_{y,N}}{M_{y,el,Rd}} + \frac{M_{z,Ed,rs} + N_{Ed,rs} * e_{z,N}}{M_{z,el,Rd}} = \frac{3,1}{60905,6} + \frac{28803,8 + 3,1 * 0,065}{71843,8} + \frac{2729,9 + 3,1 * 0,092}{49608,5} = 0,456$$

$$\eta_{1,LM71} = \frac{N_{Ed,LM71}}{N_{Rd}} + \frac{M_{y,Ed,LM71} + N_{Ed,LM71} * e_{y,N}}{M_{y,el,Rd}} + \frac{M_{z,Ed,LM71} + N_{Ed,LM71} * e_{z,N}}{M_{z,el,Rd}} = \frac{189,3}{60905,6} + \frac{32060,7 + 189,3 * 0,065}{71843,8} + \frac{3362,9 + 189,3 * 0,092}{49608,5} = 0,518$$

$$z_{LM71} = \frac{1 - \eta_{1,rs}}{\eta_{1,LM71}} = \frac{1 - 0,456}{0,518} = \mathbf{1,051}$$

Stationing L=7,8m (change of thickness of flanges):

N_{Ed}	318,4	kN	$N_{Ed,LM71}$	306,3	kN	$N_{Ed,rs}$	12,1	kN
$M_{y,Ed}$	36512,7	kNm	$M_{y,Ed,LM71}$	18667,6	kNm	$M_{y,Ed,rs}$	17845,0	kNm
$M_{z,Ed}$	3628,0	kNm	$M_{z,Ed,LM71}$	1898,8	kNm	$M_{z,Ed,rs}$	1729,2	kNm

$$\eta_{1,rs} = \frac{N_{Ed,rs}}{N_{Rd}} + \frac{M_{y,Ed,rs} + N_{Ed,rs} * e_{y,N}}{M_{y,el,Rd}} + \frac{M_{z,Ed,rs} + N_{Ed,rs} * e_{z,N}}{M_{z,el,Rd}} = \frac{12,1}{50555,0} + \frac{17845,0 + 12,1 * 0,062}{58239,4} + \frac{1829,2 + 12,1 * 0,116}{43005,0} = 0,347$$

$$\eta_{1,LM71} = \frac{N_{Ed,LM71}}{N_{Rd}} + \frac{M_{y,Ed,LM71} + N_{Ed,LM71} * e_{y,N}}{M_{y,el,Rd}} + \frac{M_{z,Ed,LM71} + N_{Ed,LM71} * e_{z,N}}{M_{z,el,Rd}} = \frac{306,3}{50555,0} + \frac{18667,6 + 306,3 * 0,062}{58239,4} + \frac{1798,8 + 306,3 * 0,116}{43005,0} = 0,372$$

$$z_{LM71} = \frac{1 - \eta_{1,rs}}{\eta_{1,LM71}} = \frac{1 - 0,347}{0,372} = 1,756$$

$$M_{Ed} = 1,756 * 18667,6 + 17845,0 = 50625,3\text{kNm}$$

$$\eta_1 = \frac{M_{Ed}}{M_{el,N,Rd}} = \frac{50625,3}{59046,7} = 0,857 > \frac{M_{f,N,Rd}}{M_{el,N,Rd}} = 0,713$$

$$\rightarrow \textit{interaction of bending and shear force}$$

$V_{z,Ed}$	3653,8	kN	$V_{z,Ed,LM71}$	1968,9	kN	$V_{z,Ed,rs}$	1684,9	kN
$M_{x,Ed}$	1478,3	kNm	$M_{x,Ed,LM71}$	867,7	kNm	$M_{x,Ed,rs}$	610,7	kNm

$$\tau_{t,Ed} = \frac{M_{x,d}}{2 * A_k * t_{min}}$$

$$\frac{T_{Ed}}{T_{Rd}} = \frac{\tau_{t,Ed}}{(f_y/\sqrt{3})/\gamma_{M1}}$$

$$V_{bw,T,Rd} = [1 - T_{Ed}/T_{Rd}] * V_{bw,Rd}$$

Because a shear resistance depends on value of torsional moment, load capacity is found iteratively in excel table below. For assessment is used formula for interaction of shear force and bending moment (EN 1993-1-5):

$$\eta_1 + \left(1 - \frac{M_{f,N,Rd}}{M_{el,N,Rd}}\right) * (2 * \eta_3 - 1)^2 \leq 1{,}0$$

$$\eta_1 = \frac{\mathrm{N_{Ed}}}{\mathrm{N_{Rd}}} + \frac{\mathrm{M_{y,Ed}} + \mathrm{N_{Ed}} * \mathrm{e_{y,N}}}{\mathrm{M_{y,el,Rd}}} + \frac{\mathrm{M_{z,Ed}} + \mathrm{N_{Ed}} * \mathrm{e_{z,N}}}{\mathrm{M_{z,el,Rd}}}$$

$$\eta_3 = \frac{V_{Ed}}{V_{bw,T,Rd}}$$

$$M_{f,N,Rd} = M_{f,Rd} * \left(1 - \frac{N_{Ed}}{A_f * \frac{f_y}{\gamma_{M1}}}\right) = 42491{,}2 * (1 - \frac{N_{Ed}}{0{,}16457 * 235000/1{,}1}) kNm$$

$$M_{el,N,Rd} = M_{el,Rd} * \left(1 - \frac{N_{Ed}}{A * \frac{f_y}{\gamma_{M1}}}\right) = 59369{,}4 * (1 - \frac{N_{Ed}}{0{,}27417 * 235000/1{,}1}) kNm$$

Inner forces are for loading capacity expressed:

$$X_{Ed} = z_{LM71} * X_{Ed,LM71} + X_{Ed,rs}$$

z_{LM71}=	**1,658**				
N_{Ed}	520,0	kN	η_1=	0,964	
$M_{y,Ed}$	48795,9	kNm	T_{Ed}/T_{Rd}=	0,061	
$M_{z,Ed}$	4877,4	kNm	$V_{bw,Rd}$=	7279,7	kN
$V_{z,Ed}$	4949,3	kN	η_3=	0,680	>0,5
$M_{x,Ed}$	2049,2	kNm	$k=1-M_{f,Rd}/M_{pl,Rd}$=	0,284	
			$\eta_1+k*(2\eta_3-1)^2$=	1,000	

Stationing L=0,0m

$V_{z,Ed,1}$	3150,8	kN	$V_{z,Ed,LM71,1}$	1633,0	kN	$V_{z,Ed,rs,1}$	1517,8	kN
$M_{x,Ed}$	2268,1	kNm	$M_{x,Ed,LM71}$	1331,2	kNm	$M_{x,Ed,rs}$	936,9	kNm

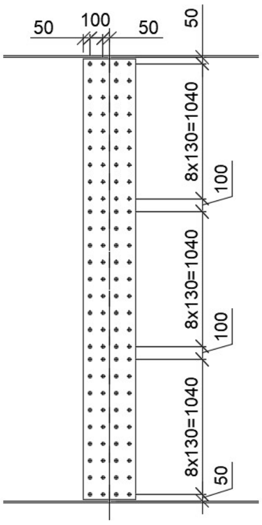
$$\eta_{3,\mathrm{rs}} = \frac{\mathrm{V_{Ed,rs,1}}}{V_{bw,T,Rd}} = \frac{1517{,}8}{3640{,}3} = 0{,}417$$

$$\eta_{3,\mathrm{LM71}} = \frac{\mathrm{V_{Ed,LM71,1}}}{V_{bw,T,Rd}} = \frac{1633{,}0}{3640{,}3} = 0{,}449$$

$$z_{\mathrm{LM71}} = \frac{1 - \eta_{3,\mathrm{rs}}}{\eta_{3,\mathrm{LM71}}} = \frac{1 - 0{,}417}{0{,}449} = \mathbf{1{,}298}$$

4.1.14 Joints

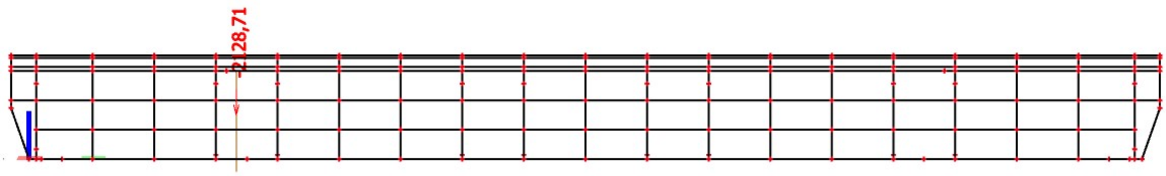
4.1.14.1 Bolted joint of webs



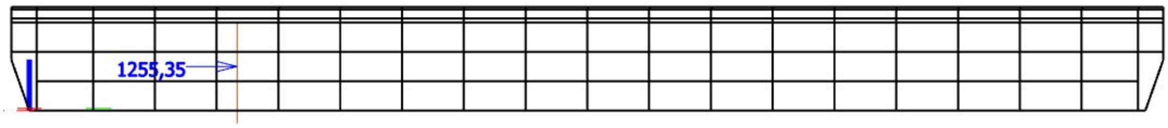
Picture 4.1-29 - spacings of bolts on web

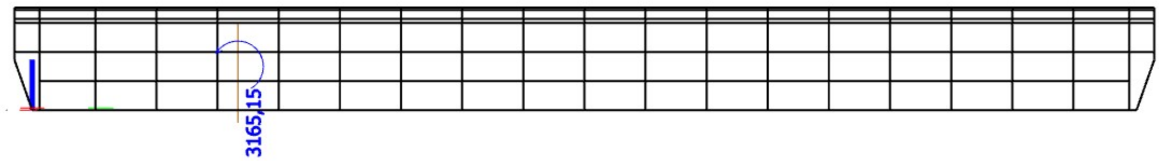
<u>Stationing L=7,8m:</u>

It is verified, if the bolted joint can bear max. shear force and bending moment of web:



$z_{LM71} * V_{Ed}$ =1,051*2128,7=2237,3kN





$$z_{LM71} * M_{Ed} = 1{,}051 * \left[3165{,}2 + 1255{,}35 * \left(1{,}995 - \frac{3{,}42}{2}\right)\right] = 3702{,}4\text{kNm}$$

For joint are used bolts M24 class 10.9 organised in twice two rows with spacings: e_1=50mm, p_1=130mm, e_2=50mm, p_2=100mm, 27 bolts in each row.

Vertical force component for one bolt:

$$V_v = \frac{V_{Ed}}{54} = \frac{2237{,}3}{54} = 41{,}431kN$$

Horizontal force component for one bolt (counted from bellow):

e_T= 1995 mm

č.	e[mm]	F_H[kN]	M[kN]
1	1945	**251,544**	489,3
2	1815	234,7	426,0
3	1685	217,9	367,2
4	1555	201,1	312,7
5	1425	184,3	262,6
6	1295	167,5	216,9
7	1165	150,7	175,5
8	1035	133,9	138,5

9	905	117,0	105,9
10	805	104,1	83,8
11	675	87,3	58,9
12	545	70,5	38,4
13	415	53,7	22,3
14	285	36,9	10,5
15	155	20,0	3,1
16	25	3,2	0,1
17	105	13,6	1,4
18	235	30,4	7,1
19	335	43,3	14,5
20	465	60,1	28,0
21	595	77,0	45,8
22	725	93,8	68,0
23	855	110,6	94,5
24	985	127,4	125,5
25	1115	144,2	160,8
26	1245	161,0	200,5
27	1375	177,8	244,5
∑			**3702,4**

$$V_h = \frac{F_H}{2} = \frac{251,544}{2} = 125,772kN$$

Design force:

$$V_{Ed,1} = \sqrt{{V_v}^2 + {V_h}^2} = \sqrt{41,431^2 + 125,772^2} = 132,420kN$$

Shear resistance per shear plane:

$$F_{v,Rd} = \frac{\alpha_V * A_s * f_{ub} * i}{\gamma_{M2}} = \frac{0,5 * 353 * 1000 * 2}{1,25} * 10^{-3} = 282,400kN$$

Bearing resistance:

$$F_{b,Rd} = \frac{\alpha_b * k_1 * d * t * f_u}{\gamma_{M2}} = \frac{0,641 * 2,5 * 24 * 14 * 360}{1,25} * 10^{-3} = 155,077kN$$

$$\alpha_b = min\left(\frac{e_1}{3 * d_0}, \frac{p_1}{3 * d_0} - 0,25, \frac{f_{ub}}{f_u}, 1\right) = min\left(\frac{50}{3 * 26}, \frac{130}{3 * 26} - 0,25, \frac{1000}{490}, 1\right) = 0,641$$

$$k_1 = \min\left(2,8 * \frac{e_2}{d_0} - 1,7; 2,5\right) = \min\left(2,8 * \frac{50}{26} - 1,7; 2,5\right) = 2,5$$

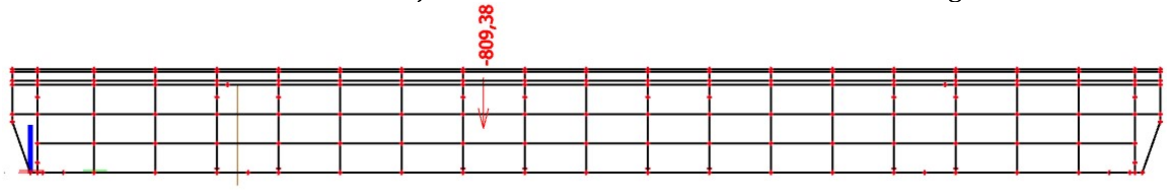
Critical load capacity.

$F_{b,Rd} = 155,077kN > V_{Ed,1} = 132,420kN$

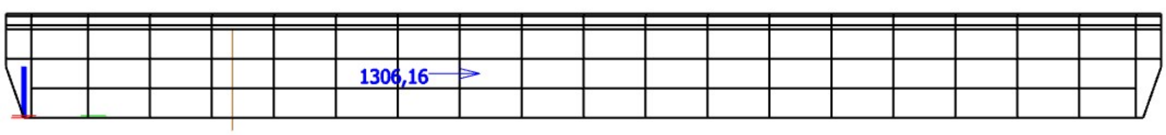
SATISFACTORY

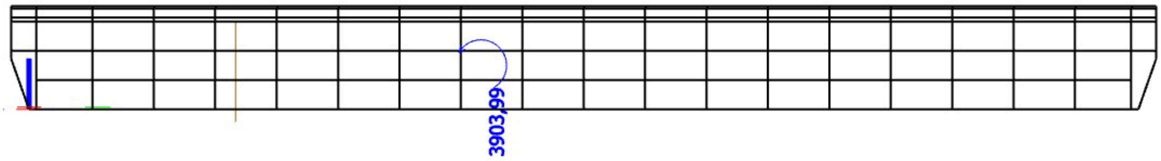
Stationing L=17,2m:

It is verified, if the bolted joint can bear max. shear force and bending moment of web:



$$z_{LM71} * V_{Ed} = 1,051 * 809,4 = 850,7\text{kN}$$





$$z_{LM71} * M_{Ed} = 1{,}051 * \left[3904{,}0 + 1306{,}2 * \left(1{,}858 - \frac{3{,}42}{2}\right)\right] = 4306{,}3\text{kNm}$$

For joint are used bolts M24 class 10.9 organised in twice two rows with spacings: e_1=50mm, p_1=130mm, e_2=50mm, p_2=100mm, 27 bolts in each row.

Vertical force component for one bolt:

$$V_v = \frac{V_{Ed}}{54} = \frac{850{,}7}{54} = 15{,}754kN$$

Horizontal force component for one bolt (counted from bellow):

e_T= 1858 mm

č.	e[mm]	F_H[kN]	M[kN]
1	1808	**288,083**	520,9
2	1678	267,4	448,6
3	1548	246,7	381,8
4	1418	225,9	320,4
5	1288	205,2	264,3
6	1158	184,5	213,7
7	1028	163,8	168,4
8	898	143,1	128,5
9	768	122,4	94,0
10	668	106,4	71,1
11	538	85,7	46,1
12	408	65,0	26,5
13	278	44,3	12,3
14	148	23,6	3,5
15	18	2,9	0,1
16	112	17,8	2,0
17	242	38,6	9,3
18	372	59,3	22,0
19	472	75,2	35,5
20	602	95,9	57,7
21	732	116,6	85,4
22	862	137,3	118,4
23	992	158,1	156,8
24	1122	178,8	200,6
25	1252	199,5	249,8
26	1382	220,2	304,3
27	1512	240,9	364,3
Σ			**4306,3**

$$V_h = \frac{F_H}{2} = \frac{288{,}083}{2} = 144{,}042kN$$

Design force:

$$V_{Ed,1} = \sqrt{{V_v}^2 + {V_h}^2} = \sqrt{15{,}754^2 + 144{,}042^2} = 144{,}900kN$$

Shear resistance per shear plane:

$$F_{v,Rd} = \frac{\alpha_V * A_s * f_{ub} * i}{\gamma_{M2}} = \frac{0{,}5 * 353 * 1000 * 2}{1{,}25} * 10^{-3} = 282{,}400kN$$

Bearing resistance:

$$F_{b,Rd} = \frac{\alpha_b * k_1 * d * t * f_u}{\gamma_{M2}} = \frac{0{,}641 * 2{,}5 * 24 * 14 * 360}{1{,}25} * 10^{-3} = 155{,}077kN$$

$$\alpha_b = min\left(\frac{e_1}{3*d_0}, \frac{p_1}{3*d_0} - 0{,}25, \frac{f_{ub}}{f_u}, 1\right) = min\left(\frac{50}{3*26}, \frac{130}{3*26} - 0{,}25, \frac{1000}{490}, 1\right) = 0{,}641$$

$$k_1 = \min\left(2{,}8*\frac{e_2}{d_0} - 1{,}7; 2{,}5\right) = \min\left(2{,}8*\frac{50}{26} - 1{,}7; 2{,}5\right) = 2{,}5$$

Critical load capacity.

$F_{b,Rd} = 155{,}077kN > V_{Ed,1} = 144{,}900kN$

SATISFACTORY

Verification of the carrying capacity of the weakened part of the web:

$\chi_w = 0{,}657$

$$V_{bw,Rd} = \frac{\chi_w * f_{yw} * h_w * t}{\sqrt{3} * \gamma_{M1}} = \frac{0{,}657 * 235 * (3420 - 27 * 26) * 14}{\sqrt{3} * 1{,}1 * 1000} = 3083{,}6\text{kN} \geq$$
$$\geq z_{LM71} * V_{Ed} = 1{,}051 * 1976{,}7 = 2077{,}51\text{kN}$$

$$N_{net,Rd} = A_{net} * f_y = \left(50 + \frac{130}{2} - 26\right) * 14 * 235 * 10^{-3} = 292{,}810\text{kN} \geq V_{Ed,1} =$$
$$= 144{,}900kN$$

SATISFACTORY

Verification of slip-resistance:

Slip resistance (coefficients k_P=0,63 and k_S=0,45 according to original documentation):

$F_{p,C} = k_P * A_s * f_{ub} = 0{,}63 * 353 * 1000 * 10^{-3} = 222{,}390kN$

$$F_{s,Rd} = \frac{k_s * \mu * i}{\gamma_{M3}} * F_{p,C} = \frac{1{,}0 * 0{,}45 * 2}{1{,}25} * 222{,}390 = 160{,}121kN$$

Critical load capacity:

$F_{s,Rd} = 160{,}121kN > V_{Ed,1} = 144{,}900kN$

SATISFACTORY

4.1.14.2 Welded joint of webs

For connection of web and flanges of box girder is used double sided fillet weld a=6mm.

Cross section above support (upper welds are deciding):

$$\tau_{II,1} = \frac{V_{Ed,1} * S_f}{2 * a * I_y} = \frac{3150{,}8 * 0{,}15266}{2 * 6 * 0{,}55558} = 76{,}52MPa$$

$\tau_{\perp,1} = \sigma_{\perp,1} = 0MPa$

$$\sqrt{\sigma_{\perp,1}{}^2 + 3 * (\tau_{\perp,1}{}^2 + \tau_{II,1}{}^2)} = \sqrt{0 + 3 * (0^2 + 76{,}52^2)} = 132{,}54MPa < \frac{f_u}{\beta_w * \gamma_{M2}} =$$
$$= \frac{360}{0{,}8 * 1{,}25} = 360{,}0MPa$$

SATISFACTORY (36,8%)

Cross section in midspan (lower welds are deciding):

$$\sigma_{II,1} = \frac{M_{Ed}}{w_y} = \frac{60864{,}5}{0{,}39366} * 10^{-3} = 154{,}612MPa$$

$\tau_{\perp,1} = \sigma_{\perp,1} = \tau_{II,1} = 0MPa$

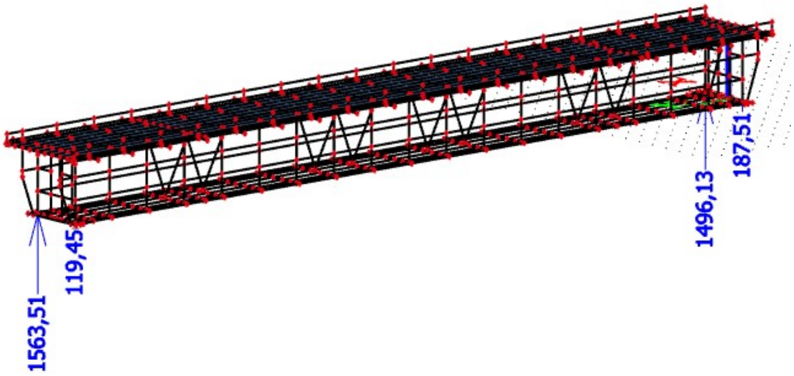
$$\sqrt{\sigma_{II,1}{}^2 + 3 * (\tau_{\perp,1}{}^2 + \tau_{II,1}{}^2)} = \sqrt{154{,}612^2 + 3 * (0^2 + 0^2)} = 154{,}612MPa < \frac{f_u}{\beta_w * \gamma_{M2}}$$
$$= \frac{360}{0{,}8 * 1{,}25} = 360{,}0MPa$$

SATISFACTORY (43,5%)

4.1.15 Stability of construction

Combination of loads for verification of stability of bridge position (gr15):

self-weight	0,95
other permanent load average	0,95
other permanent load minimal	0,95
wind L with train	1,35
unloaded train	0,95
centrifugal force - unloaded train	1,30

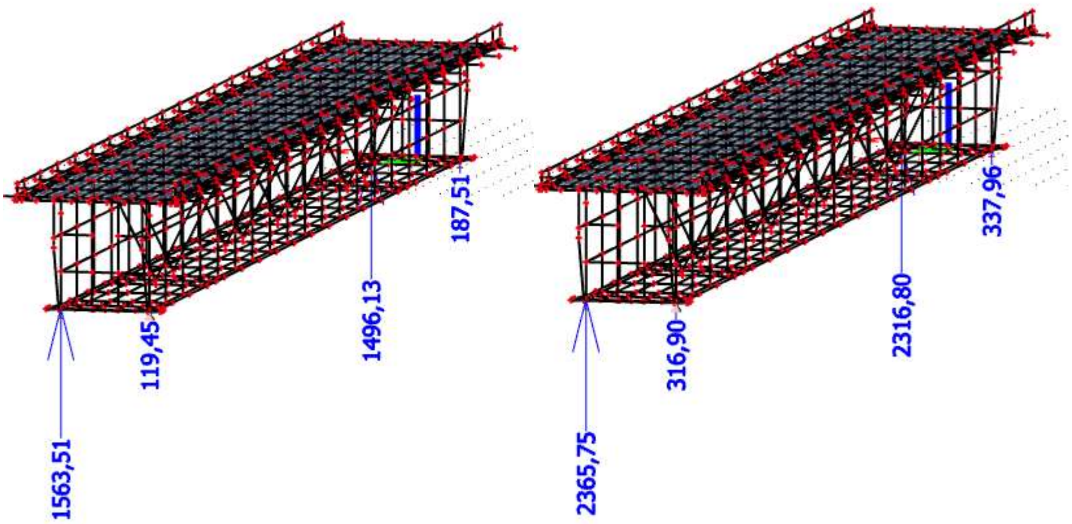


Picture 4.1-30 - support reactions – unloaded train

Combination of loads (derailment) for verification of stability of bridge position:

self-weight	1,00
other permanent load average	1,00
other permanent load minimal	1,00
wind L with train	0,50
derailment 1 - max M	1,00

self-weight	1,00
other permanent load average	1,00
other permanent load minimal	1,00
wind L	0,50
derailment 2 - max M	1,00



Picture 4.1-31 – support reactions - derailment, design situation I and II

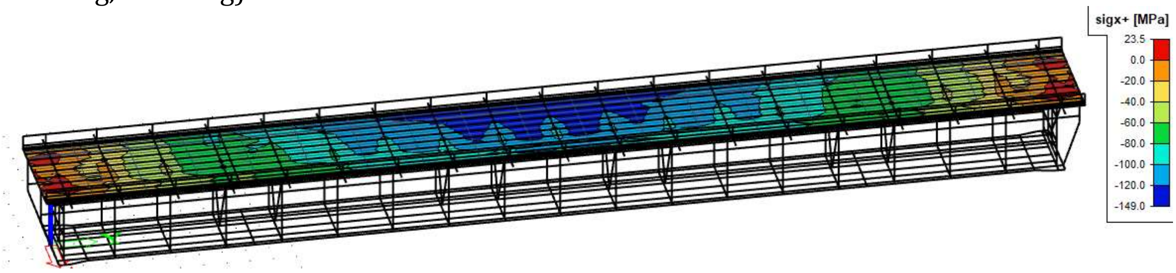
All support reactions are compressive, so the formula $M_{stab} \geq M_{destab}$ is valid and the construction is stable.

4.1.16 Comparative assessment of stress in a box girder

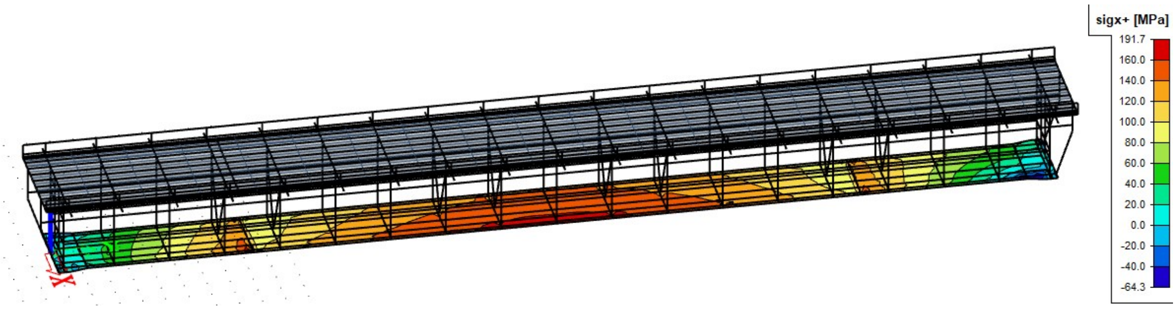
The stress for the determinative combination is drawn on following pictures:

Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	maintenance load	1,20
	temperature max linear	0,90
	traction force	0,65
	wind Z-	1,01
	wind L with train	1,01
	LM71 - L - maximal M	1,39
	centrifugal force - maximal M	1,30
	nosing force - L - maximal M	1,30

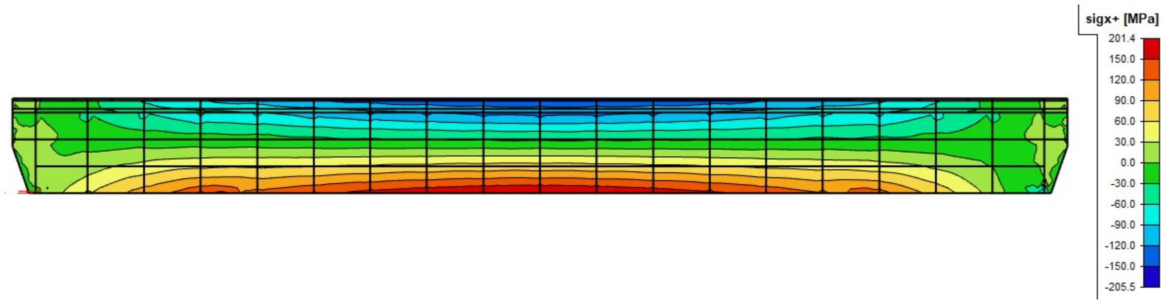
The thickness of plates is changed according to dimension of effective cross sections. For every subpanel is used an average thickness (including weakening by local and global buckling, shear lag).



Picture 4.1-32 - stress in box girder - upper flange



Picture 4.1-33 - stress in box girder - lower flange



Picture 4.1-34 - stress in box girder – web and vertical plates

$$\sigma_{max} = 205{,}5MPa \leq f_{yd} = \frac{235}{1{,}1} = 213{,}6MPa$$

The maximal normal stress in midspan reach 96,2% of yield strength. This result is very similar as in chapter 4.1.7 (97,4%), so the calculation is considered valid. The small difference of results is caused by fact, that the effective cross sections for compressive normal force and bending moment M_z are conservative. For the normal force is considered all cross section in compressive zone and for bending moment M_z is compressed whole web, so values of A_{eff} and $w_{z,el,eff}$ are slightly lower than it would be for real stress diagram.

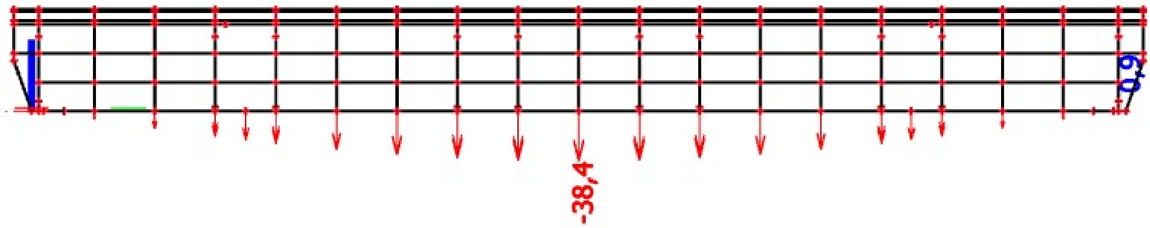
4.1.17 Serviceability limit state

Web breathing:

$$\frac{b}{t} = \frac{1140}{14} = 81{,}4 \leq 55 + 3{,}3 * L = 196{,}2 \rightarrow web\ breathing\ can\ be\ neglected$$

Vertical deformation:

Deformation caused by permanent load are eliminated by precamber.



Picture 4.1-35 - vertical deformation caused by variable loads

$$w_{max} = 38{,}4mm < \frac{L}{600} = \frac{42800}{600} = 71{,}3mm$$

SATISFACTORY (53,8%)

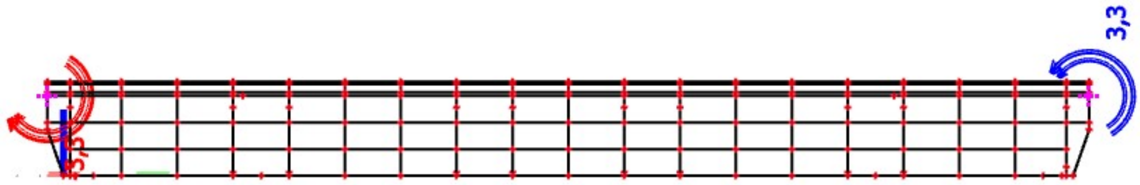
Load capacity:

$$\eta_{\mathrm{rs}} = \frac{w_{max,rs}}{w_{lim}} = \frac{4{,}3}{71{,}3} = 0{,}060$$

$$\eta_{\mathrm{LM71}} = \frac{w_{max,LM71}}{w_{lim}} = \frac{34{,}1}{71{,}3} = 0{,}478$$

$$z_{\mathrm{LM71}} = \frac{1 - \eta_{\mathrm{rs}}}{\eta_{\mathrm{LM71}}} = \frac{1 - 0{,}060}{0{,}478} = \mathbf{1{,}967}$$

Rotation at the end of deck:



Picture 4.1-36 - rotation caused by variable loads

$$\theta_{max} = 3{,}3mrad < \theta_{lim} = 5mrad$$

SATISFACTORY (62,0%)

Load capacity:

$$\eta_{\mathrm{rs}} = \frac{\theta_{max,rs}}{\theta_{lim}} = \frac{0{,}3}{5{,}0} = 0{,}060$$

$$\eta_{\mathrm{LM71}} = \frac{\theta_{max,LM71}}{\theta_{lim}} = \frac{3{,}0}{5{,}0} = 0{,}600$$

$$z_{\mathrm{LM71}} = \frac{1 - \eta_{\mathrm{rs}}}{\eta_{\mathrm{LM71}}} = \frac{1 - 0{,}060}{0{,}600} = \mathbf{1{,}567}$$

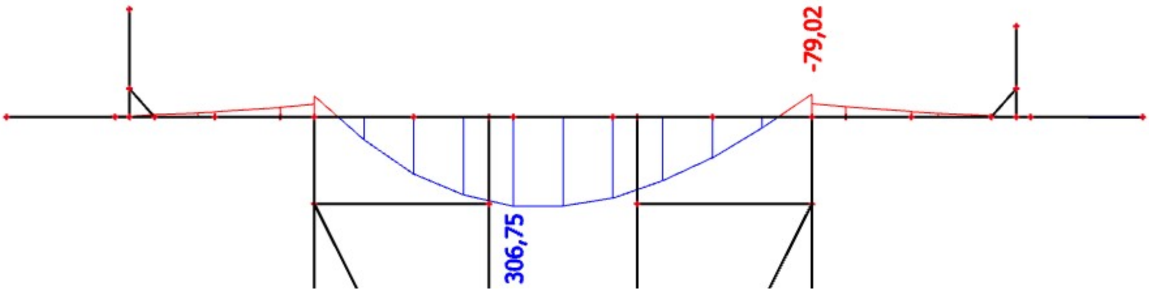
Serviceability limit state is not determinative for load capacity.

4.2 Cross girder

4.2.1 Midspan

Combination of loads causing maximal bending moment (gr12, formula 10.b)

Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	temperature max linear	0,90
	wind R with train	1,01
	CG,LM71,Qr-maxM	2,25
	CG,CF-maxM1	1,30
	CG,NFr-maxM	1,30

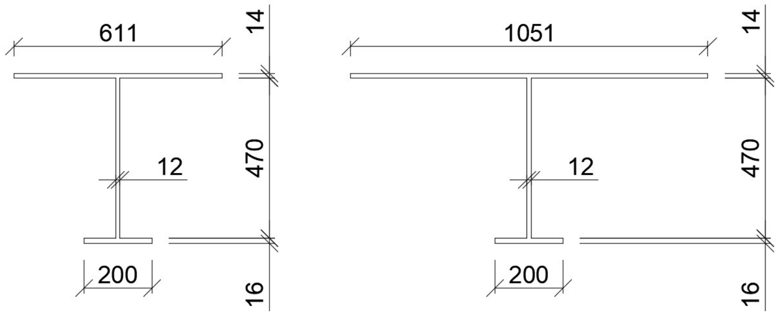


Picture 4.2-1 - bending moment diagram

- calculation of dimension of an effective cross section including a buckling and a shear lag

ψ=	1	
k_σ=	4,000	
b=	2400	mm
t=	14	mm
f_y=	235	MPa
ε=	1,000	
λ_P=	3,018	
ρ=	0,307	
b_{eff}=	***737***	***mm***
α_0=	0,554	
b_0=	1200	mm
$L_{e,1}$=	1960	mm
$L_{e,2}$=	2240	mm
κ_1=	0,339	
κ_2=	0,536	
sagging bending		
β^κ=	0,829	
beff=	**611**	**mm**
hogging bending		
β^κ=	0,438	
beff=	**1051**	**mm**

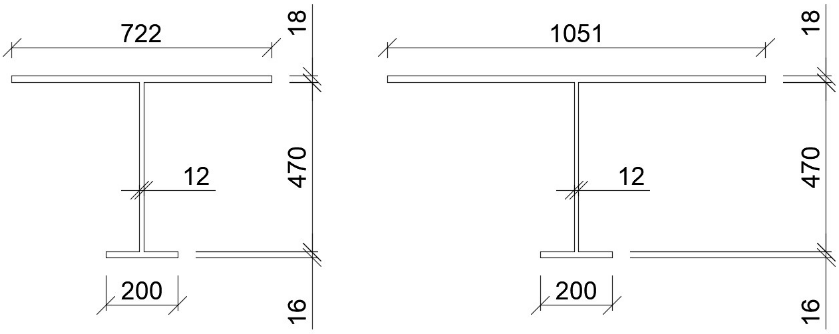
ψ=	1	
k_σ=	4,000	
b=	2400	mm
t=	18	mm
f_y=	235	MPa
ε=	1,000	
λ_P=	2,347	
ρ=	0,386	
b_{eff}=	***927***	***mm***
α_0=	0,621	
b_0=	1200	mm
$L_{e,1}$=	1960	mm
$L_{e,2}$=	2240	mm
κ_1=	0,380	
κ_2=	0,536	
sagging bending		
β^κ=	0,779	
beff=	**722**	**mm**
hogging bending		
β^κ=	0,438	
beff=	**1051**	**mm**



Picture 4.2-2 – eff. cross section in areas of positive and negative bending moment, flange thickness 14mm

$w_{y,eff,14+} = 0{,}0021454m^3 \rightarrow determinative$

$w_{y,eff,14-} = 0{,}0022373m^3 \rightarrow determinative$



Picture 4.2-3 – eff. cross section in areas of positive and negative bending moment, flange thickness 18mm

$w_{y,eff,18+} = 0{,}0022312m^3$

$w_{y,eff,18-} = 0{,}0022834m^3$

$$N_{Rd} = A * \frac{f_{yk}}{\gamma_{M1}} = 0{,}017394 * \frac{235}{1{,}1} * 1000 = 3716{,}0kN$$

$$\mathrm{M_{y,el,Rd}} = \mathrm{w_{y,eff}} * \frac{\mathrm{f_{yk}}}{\gamma_{M1}} = 0{,}0021454 * \frac{235}{1{,}1} * 1000 = 458{,}3\mathrm{kNm}$$

Lower fibre of cross girder (bending moment M_z is negligible):

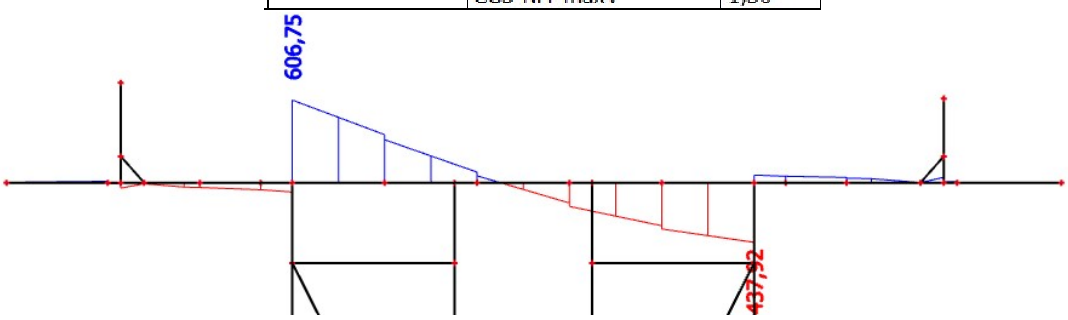
$$\mu_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_y}{M_{y,el,Rd}} = \frac{28{,}0}{3716{,}0} + \frac{306{,}8 - 28{,}0 * 0{,}099}{458{,}3} = 0{,}671$$

SATISFACTORY (67,1%)

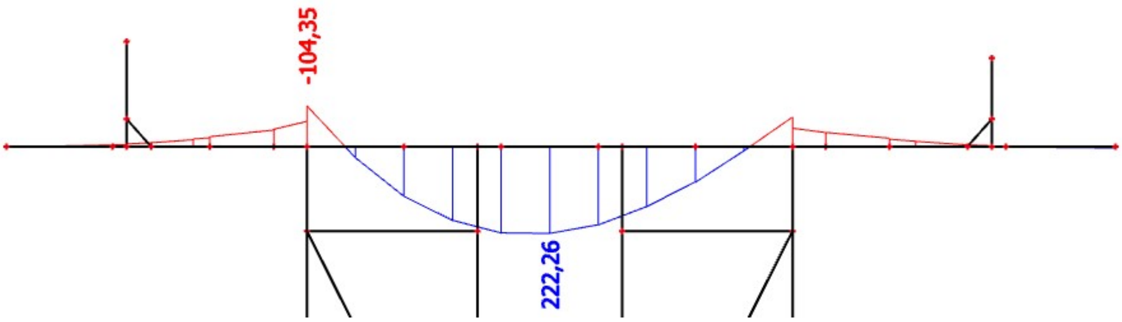
4.2.2 Cross section above support

Combination of loads causing maximal shear force (gr12, formula 10.b):

Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	wind Z-, right	1,01
	wind R with train	1,01
	CG3-LM71,Qr-maxV	2,25
	CG3-NFr-maxV	1,30



Picture 4.2-4 - shear force diagram



Picture 4.2-5 – bending moment diagram

E=	210000	MPa
G=	81000	MPa
I_t=	1505101,33	mm^4
I_y=	825932001	mm^4
I_z=	1365159273	mm^4
I_w=	2,4124E+12	mm^6
k_w=	1,00	
k_y=	2,00	
k_z=	1,00	
C_1=	1,85	
C_2=	0,46	
C_3=	1,00	
L=	1671	mm
z_g=	10	mm
z_s=	121	mm
z_j=	65	mm
ζ_g=	0,897	
ζ_j=	5,925	
κ_{wt}=	3,833	
μ_{cr}=	22,756	
M_{cr}=	**252924,3**	**kNm**
α_{LT}=	0,76	
w_y=	2237316	mm^3
λ_{LT}=	0,046	
ϕ_{LT}=	0,442	
χ_{LT}=	**1,133**	

Cross section does not buckle in bending.

Shear resistance above a support:

$$V_{Rd} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{235 * 430 * 12}{\sqrt{3} * 1,0 * 1000} = 700,1kN > 606,8\text{kN}$$

<u>SATISFACTORY (86,7%)</u>

- For $V_{Ed} = 545,2\text{kN} \dots M_{Ed} = 104,4\text{kNm}$

$$M_{el,Rd} = \text{w}_{\text{y,eff}} * \frac{\text{f}_{\text{yk}}}{\gamma_{M1}} = 0,0022373 * \frac{235}{1,1} * 1000 = 478,0\text{kNm}$$

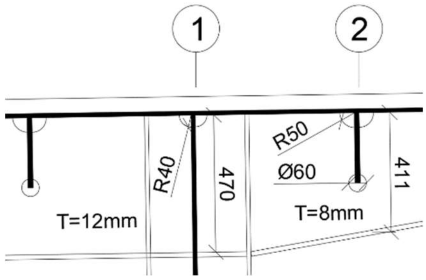
$$M_{f,Rd} = \text{w}_{\text{y,eff,f}} * \frac{\text{f}_{\text{yk}}}{\gamma_{M1}} = 0,0015222 * \frac{235}{1,1} * 1000 = 325,2\text{kNm}$$

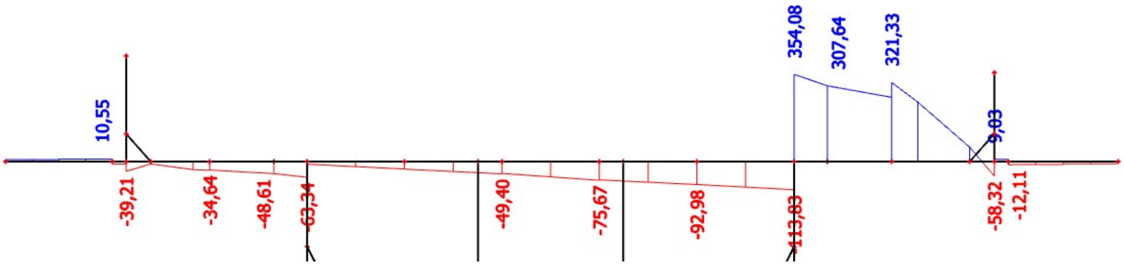
$$\mu_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_y}{M_{y,el,Rd}} = \frac{49,9}{5032,0} + \frac{104,4 - 49,9 * 0,055}{478,0} = 0,223 \ll \frac{M_{f,Rd}}{M_{y,el,Rd}} =$$

$$= \frac{325,2 * (1 - \frac{49,9}{0,017914*235000/1,1})}{478,0 * (1 - \frac{49,9}{0,023554*235000/1,1})} = 0,678 \ldots \textit{interaction does not assess}$$

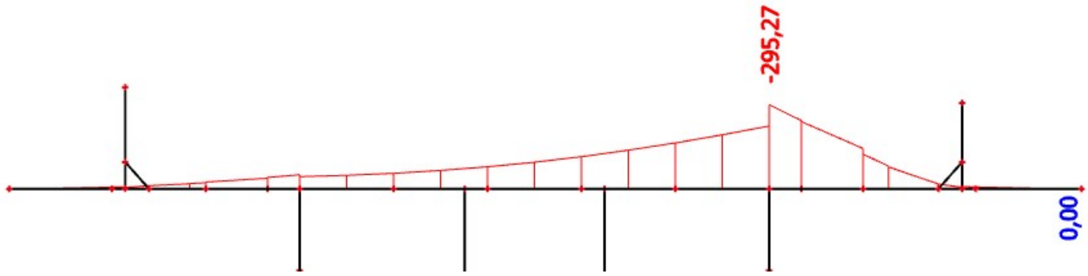
Combination of loads causing maximal shear force and bending moment (derailment – design situation II):

Linear - ultimate	self-weight	1,00
	other permanent load average	1,00
	other permanent load maximal	1,00
	maintenance load	1,00
	wind Z-, left	0,50
	CG,D2-maxV	1,00





Picture 4.2-6 - shear force diagram



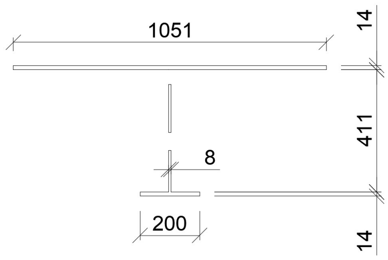
Picture 4.2-7 – bending moment diagram

$$V_{Rd,1} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{235 * 430 * 12}{\sqrt{3} * 1,0 * 1000} = 700,1kN > 354,1\text{kN}$$

$$\frac{b}{t} = \frac{470}{8} = 58,8 \leq \frac{72}{\mu} * \varepsilon = \frac{72}{1,2} * 1,0 = 60 \rightarrow \textit{web does not buckle in shear}$$

$$V_{Rd,2} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{235 * 301 * 8}{\sqrt{3} * 1,0 * 1000} = 326,7kN > 321,3\text{kN}$$

<u>Interaction of shear force and bending moment in cross section 2-2:</u>



Picture 4.2-8 – effective cross section for M_y in section 2-2

$$N_{Rd} = A * \frac{f_{yk}}{\gamma_{M1}} = 0{,}021728 * \frac{235}{1{,}1} * 1000 = 4641{,}9kN$$

$$\mathrm{M_{y,el,Rd}} = \mathrm{w_{y,eff}} * \frac{\mathrm{f_{yk}}}{\gamma_{M1}} = 0{,}0015000 * \frac{235}{1{,}1} * 10^3 * (1 - \frac{9{,}21}{0{,}001922 * 235000/1{,}1}) = 313{,}3\mathrm{kNm}$$

$$\mu_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_y}{M_{y,el,Rd}} = \frac{9{,}8}{4641{,}9} + \frac{141{,}7 - 9{,}8 * 0{,}040}{320{,}5} = 0{,}443$$

$$\eta_3 = \frac{V_{Ed}}{V_{Rd}} = \frac{321{,}3}{326{,}7} = 0{,}983$$

$$\mathrm{M_{f,Rd}} = \mathrm{w_{y,eff}} * \frac{\mathrm{f_{yk}}}{\gamma_{M1}} = 0{,}0011679 * \frac{235}{1{,}1} * 10^3 * (1 - \frac{9{,}21}{0{,}0017514 * 235000/1{,}1}) = 243{,}4\mathrm{kNm}$$

$$\eta_1 + \left(1 - \frac{M_{f,Rd}}{M_{el,Rd}}\right) * (2 * \eta_3 - 1)^2 = 0{,}443 + \left(1 - \frac{243{,}4}{313{,}3}\right) * (2 * 0{,}983 - 1)^2 = 0{,}651 \leq 1{,}0$$

SATISFACTORY (98,3% - shear resistance)

4.2.3 Load capacity

For shear resistance above web (load capacity is not determined for derailment, critical combination of loads includes following variable loads: horizontal wind, vertical wind with right eccentricity, vertical load of LM71 with maximal right eccentricity and dynamic factor $\phi_3 = 1{,}727$, nosing force)

Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	wind Z-, right	1,01
	wind R with train	1,01
	CG3-LM71,Qr-maxV	2,25
	CG3-NFr-maxV	1,30

$V_{z,Ed}$	606,8 kN	$V_{z,Ed,LM71}$	535,1 kN	$V_{z,Ed,rs}$	71,7 kN
$M_{y,Ed}$	104,4 kNm	$M_{y,Ed,LM71}$	63,1 kNm	$M_{y,Ed,rs}$	41,3 kNm

$$\eta_{3,\mathrm{LM71}} = \frac{\mathrm{V_{Ed,LM71}}}{V_{Rd}} = \frac{535{,}1}{700{,}1} = 0{,}764$$

$$\eta_{3,\mathrm{rs}} = \frac{\mathrm{V_{Ed,rs}}}{V_{Rd}} = \frac{71{,}7}{700{,}1} = 0{,}102$$

$$z_{\mathrm{LM71}} = \frac{1 - \eta_{3,\mathrm{rs}}}{\eta_{3,\mathrm{LM71}}} = \frac{1 - 0{,}102}{0{,}764} = \mathbf{1{,}175}$$

$$N_{Ed} = 23{,}1 * 1{,}175 + 26{,}8 = 53{,}9kNm$$

$$M_{Ed} = 63{,}1 * 1{,}175 + 41{,}3 = 115{,}4kNm$$

$$\mu_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_y}{M_{y,el,Rd}} = \frac{53{,}9}{5032{,}0} + \frac{115{,}4 - 53{,}9 * 0{,}055}{478{,}0} = 0{,}246 \ll \frac{M_{f,Rd}}{M_{y,el,Rd}} =$$

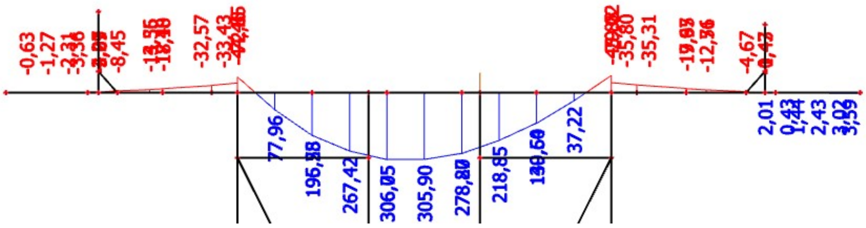
$$= \frac{325{,}2 * (1 - \frac{49{,}9}{0{,}017914 * 235000/1{,}1})}{478{,}0 * (1 - \frac{49{,}9}{0{,}023554 * 235000/1{,}1})} = 0{,}678 \ldots \textit{interaction does not assess}$$

4.2.4 Bolted joint of web

Thickness of flange 18mm:

It is verified, if the bolted joint can bear maximal stress caused by bending of web:

$$\sigma_{Ed} = z_{LM71} * \frac{M_{Ed}}{w_{y,eff}} = 1{,}175 * \frac{305{,}9}{0{,}0022312} * 10^{-3} = 161{,}094MPa$$



Picture 4.2-9 – maximal bending moment for cross girder of flanges thickness 18mm

$$M_{w,Ed} = w_{y,w} * \sigma_{Ed} =$$

$$= \frac{\frac{1}{12} * 0{,}012 * 0{,}47^3 + 0{,}012 * 0{,}47 * \left(\frac{0{,}47}{2} - 0{,}345\right)^2}{0{,}361} * 161{,}094 * 10^3$$

$$= 76{,}784kNm$$

For joint are used bolts M20 class 10.9 organised in two rows (one on each side) with spacings: e_1=60mm, p_1=70mm, e_2=50mm, p_2=100mm, 6 bolts in each row.

Horizontal force component for one bolt:

e_T= 345 mm

č.	e[mm]	F_H[kN]	M[kN]
1	285	**138,2**	39,4
2	215	104,3	22,4
3	145	70,3	10,2
4	75	36,4	2,7
5	5	2,4	0,0
6	65	31,5	2,0
∑			**76,784**

$V_h = 138{,}197kN$

Design force:

$$V_{Ed,1} = \sqrt{{V_v}^2 + {V_h}^2} = \sqrt{0^2 + 138{,}197^2} = 138{,}197kN$$

Shear resistance per shear plane:

$$F_{v,Rd} = \frac{\alpha_V * A_s * f_{ub} * i}{\gamma_{M2}} = \frac{0{,}5 * 245 * 1000 * 2}{1{,}25} * 10^{-3} = 196{,}000kN$$

Bearing resistance:

$$F_{b,Rd} = \frac{\alpha_b * k_1 * d * t * f_u}{\gamma_{M2}} = \frac{0{,}909 * 2{,}5 * 20 * 12 * 360}{1{,}25} * 10^{-3} = 163{,}636kN$$

$$\alpha_b = min\left(\frac{e_1}{3 * d_0}, \frac{p_1}{3 * d_0} - 0{,}25, \frac{f_{ub}}{f_u}, 1\right) = min\left(\frac{60}{3 * 22}, \frac{70}{3 * 22} - 0{,}25, \frac{1000}{490}, 1\right)$$

$$= 0{,}909$$

$$k_1 = \min\left(2{,}8 * \frac{e_2}{d_0} - 1{,}7; 2{,}5\right) = \min\left(2{,}8 * \frac{50}{22} - 1{,}7; 2{,}5\right) = 2{,}5$$

Critical load capacity:

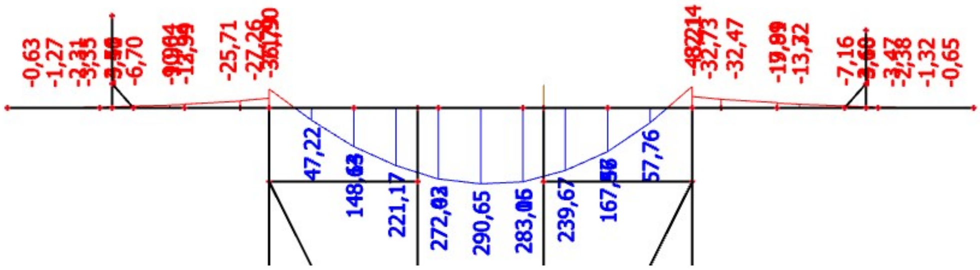
$F_{b,Rd} = 163{,}636kN > V_{Ed,1} = 138{,}197kN$

SATISFACTORY

<u>Thickness of flange 14mm:</u>

It is verified, if the bolted joint can bear maximal stress caused by bending of web:

$$\sigma_{Ed} = z_{LM71} * \frac{M_{Ed}}{w_{y,eff}} = 1{,}175 * \frac{290{,}7}{0{,}0021454} * 10^{-3} = 159{,}212 MPa$$



Picture 4.2-10 – maximal bending moment for cross girder of flanges thickness 18mm

$$M_{w,Ed} = w_{y,w} * \sigma_{Ed} =$$

$$= \frac{\frac{1}{12} * 0{,}012 * 0{,}47^3 + 0{,}012 * 0{,}47 * \left(\frac{0{,}47}{2} - 0{,}309\right)^2}{0{,}325} * 159{,}212 * 10^3$$

$$= 65{,}991 kNm$$

For joint are used bolts M20 class 10.9 organised in two rows (one on each side) with spacings: e_1=60mm, p_1=70mm, e_2=50mm, p_2=100mm, 6 bolts in each row.

Horizontal force component for one bolt:

e_T= 309 mm

č.	e[mm]	F_H[kN]	M[kN]
1	249	**138,5**	34,5
2	179	99,6	17,8
3	109	60,6	6,6
4	39	21,7	0,8
5	31	17,2	0,5
6	101	56,2	5,7
∑			**65,991**

$V_h = 138{,}541 kN$

Design force:

$$V_{Ed,1} = \sqrt{V_v^2 + V_h^2} = \sqrt{0^2 + 138{,}197^2} = 138{,}197 kN$$

Shear resistance per shear plane:

$$F_{v,Rd} = \frac{\alpha_V * A_s * f_{ub} * i}{\gamma_{M2}} = \frac{0{,}5 * 245 * 1000 * 2}{1{,}25} * 10^{-3} = 196{,}000 kN$$

Bearing resistance:

$$F_{b,Rd} = \frac{\alpha_b * k_1 * d * t * f_u}{\gamma_{M2}} = \frac{0{,}909 * 2{,}5 * 20 * 12 * 360}{1{,}25} * 10^{-3} = 163{,}636 kN$$

$$\alpha_b = min\left(\frac{e_1}{3 * d_0}, \frac{p_1}{3 * d_0} - 0{,}25, \frac{f_{ub}}{f_u}, 1\right) = min\left(\frac{60}{3 * 22}, \frac{70}{3 * 22} - 0{,}25, \frac{1000}{490}, 1\right)$$

$$= 0{,}909$$

$$k_1 = \min\left(2{,}8 * \frac{e_2}{d_0} - 1{,}7; 2{,}5\right) = \min\left(2{,}8 * \frac{50}{22} - 1{,}7; 2{,}5\right) = 2{,}5$$

Critical load capacity:

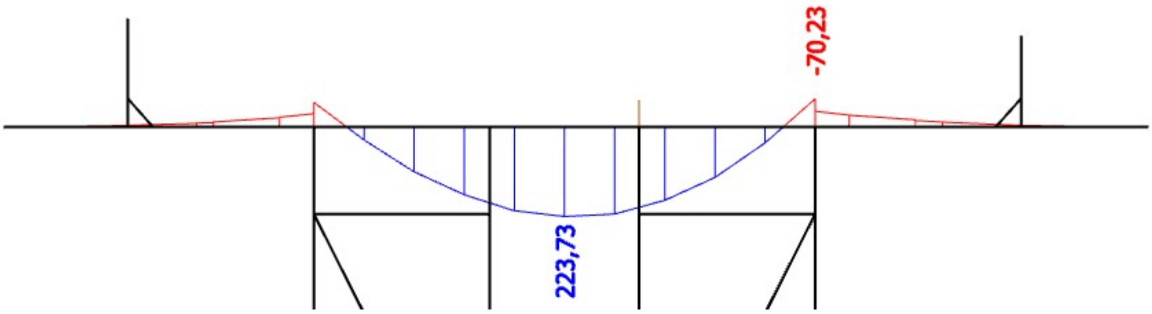
$F_{b,Rd} = 163{,}636 kN > V_{Ed,1} = 138{,}541 kN$

<u>SATISFACTORY</u>

Serviceability limit state – verification of slip-resistance:

It is verified, if the bolted joint (category B) can bear maximal stress caused by bending of web without slip:

$$\sigma_{Ed} = z_{LM71} * \frac{M_{Ed}}{w_{y,eff}} = 1{,}175 * \frac{223{,}7}{0{,}0021454} * 10^{-3} = 122{,}517MPa$$



Picture 4.2-11 – maximal bending moment for cross girder, serviceability limit state

$$M_{w,Ed} = w_{y,w} * \sigma_{Ed} =$$

$$= \frac{\frac{1}{12} * 0{,}012 * 0{,}47^3 + 0{,}012 * 0{,}47 * \left(\frac{0{,}47}{2} - 0{,}309\right)^2}{0{,}325} * 122{,}517 * 10^3$$

$$= 50{,}781kNm$$

Horizontal force component for one bolt:

e_T=	309	mm	
č.	e[mm]	F_H[kN]	M[kN]
1	249	**106,609**	26,5
2	179	76,6	13,7
3	109	46,7	5,1
4	39	16,7	0,7
5	31	13,3	0,4
6	101	43,2	4,4
Σ			**50,781**

$V_h = 106{,}609kN$

Design force:

$$V_{Ed,1} = \sqrt{{V_v}^2 + {V_h}^2} = \sqrt{0^2 + 106{,}609^2} = 106{,}609kN$$

Slip resistance (coefficients k_P=0,63 and k_S=0,45 according to original documentation):

$$F_{p,C} = k_P * A_s * f_{ub} = 0{,}63 * 245 * 1000 * 10^{-3} = 154{,}350kN$$

$$F_{s,Rd} = \frac{k_s * \mu * i}{\gamma_{M3}} * F_{p,C} = \frac{1{,}0 * 0{,}45 * 2}{1{,}25} * 154{,}350 = 111{,}132kN > V_{Ed,1} = 106{,}609kN$$

The joint is slip resistant only for serviceability limit state. Ultimate limit state may result in slippage. So bellow is verified bending resistance of cross girder (assessment in chapter 4.2.1), where cross section consists of flanges, the web is neglected:

$$N_{Rd} = A * \frac{f_{yk}}{\gamma_{M1}} = 0{,}011754 * \frac{235}{1{,}1} * 1000 = 2511{,}1kN$$

$$\mathrm{M_{y,el,Rd}} = \mathrm{w_{y,eff}} * \frac{\mathrm{f_{yk}}}{\gamma_{M1}} = 0{,}0016862 * \frac{235}{1{,}1} * 1000 = 360{,}2\mathrm{kNm}$$

$$\mu_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_y}{M_{y,el,Rd}} = \frac{28{,}0}{3716{,}0} + \frac{306{,}8 - 28{,}0 * 0{,}099}{360{,}2} = 0{,}852$$

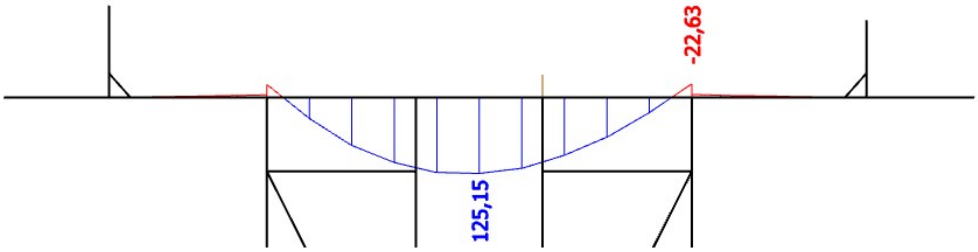
SATISFACTORY (85,2%) – not determinative (percentage of use of cross girder 86,7%)

4.2.5 Fatigue

Lower fibre of cross girder (EN 1993-1-9, table 8.1, detail 1):

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$$\lambda_1 = 1{,}03\ (for\ span\ L = 2 * 2{,}8 = 5{,}6m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 1{,}03 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}855$$



Picture 4.2-12 - maximal bending moment caused by traffic load

$$\Phi_3 * \Delta\sigma_{71} = \Phi_3 * \frac{M_{Ed}}{w_{y,eff}} = 1{,}727 * \frac{125{,}2}{0{,}0021454} * 10^{-3} = 100{,}783 MPa$$

$$1{,}0 * 0{,}855 * 100{,}783 = 86{,}170 MPa \leq \frac{160}{1{,}15} = 139{,}130 MPa$$

SATISFACTORY (61,9%)

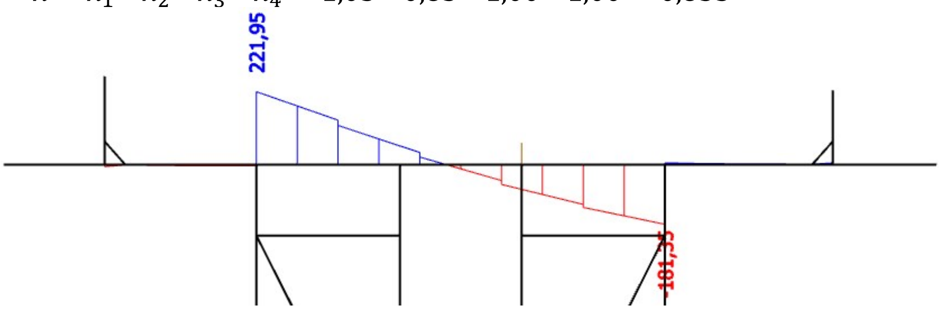
Load capacity:

$$z_{\mathrm{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71}} = \frac{139{,}130}{86{,}170} = \mathbf{1{,}615} \rightarrow not\ determinative$$

Upper fibre of web (EN 1993-1-9, table 8.1, detail 6), welds of web and flanges - continuous fillet weld transmitting a shear flow (EN 1993-1-9, table 8.2, detail 8):

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\tau_{71} \leq \frac{\Delta\tau_C}{\gamma_{Mf}}$$

$$\lambda_1 = 1{,}03\ (for\ span\ L = 2 * 2{,}8 = 5{,}6m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 1{,}03 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}855$$



Picture 4.2-13 - maximal shear force caused by traffic load

$$\Phi_3 * \Delta\tau = \Phi_3 * \frac{V_{Ed,1} * S_f}{t * I_y} = 1{,}727 * \frac{222{,}0 * 0{,}0016245}{12 * 0{,}00080459} = 64{,}507 MPa$$

$$1{,}0 * 0{,}855 * 64{,}507 = 55{,}154 MPa \leq \frac{80}{1{,}15} = 69{,}565 MPa$$

SATISFACTORY (79,3%)

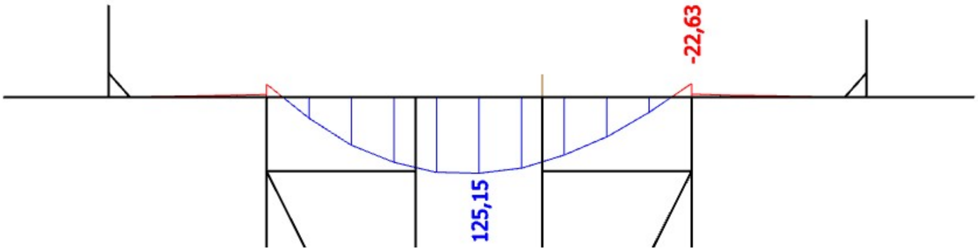
Load capacity:

$$z_{\mathrm{LM71}} = \frac{\Delta\tau_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\tau_{71}} = \frac{69{,}565}{55{,}154} = \mathbf{1{,}261} \rightarrow not\ determinative$$

<u>Bolted joint of web - double covered symmetrical connection with preloaded high strength bolts (EN 1993-1-9, table 8.1, detail 8), welds of web and flanges - automatic or fully mechanized fillet welds carried out from both sides, containing stop/start position (EN 1993-1-9, table 8.2, detail 3) – both have detail category 112</u>

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$$\lambda_1 = 1{,}03\ (for\ span\ L = 2 * 2{,}8 = 5{,}6m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 1{,}03 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}855$$



Picture 4.2-14 - maximal bending moment caused by traffic load

$$\Phi_3 * \Delta\sigma_{71} = \Phi_3 * \frac{M_{Ed}}{w_{y,eff}} = 1{,}727 * \frac{125{,}2}{0{,}0021454} * \frac{345}{361} * 10^{-3} = 96{,}316MPa$$

$$1{,}0 * 0{,}855 * 96{,}316 = 82{,}350MPa \leq \frac{112}{1{,}15} = 97{,}391MPa$$

<u>SATISFACTORY (84,6%)</u>

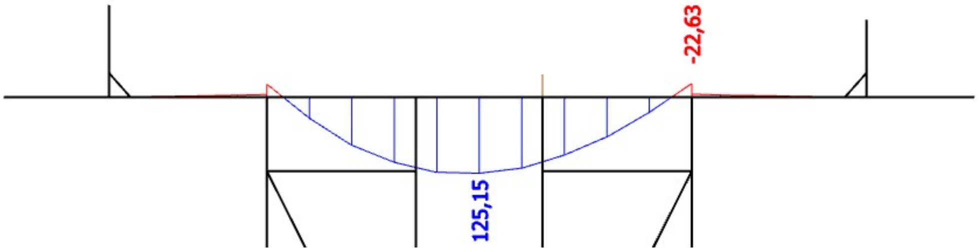
Load capacity:

$$z_{\text{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71}} = \frac{97{,}391}{82{,}350} = \mathbf{1{,}183} \rightarrow not\ determinative$$

<u>Constructional welds of lower flange – transverse splices, grounded flush to plate surface parallel to direction of the arrow (EN 1993-1-9, table 8.3, detail 1):</u>

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$$\lambda_1 = 1{,}03\ (for\ span\ L = 2 * 2{,}8 = 5{,}6m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 1{,}03 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}855$$



Picture 4.2-15 - maximal bending moment caused by traffic load

$$\Phi_3 * \Delta\sigma_{71} = \Phi_3 * \frac{M_{Ed}}{w_{y,eff}} = 1{,}727 * \frac{125{,}2}{0{,}0021454} * 10^{-3} = 100{,}783MPa$$

$$1{,}0 * 0{,}855 * 100{,}783 = 86{,}170MPa \leq \frac{112}{1{,}15} = 97{,}391MPa$$

<u>SATISFACTORY (88,5%)</u>

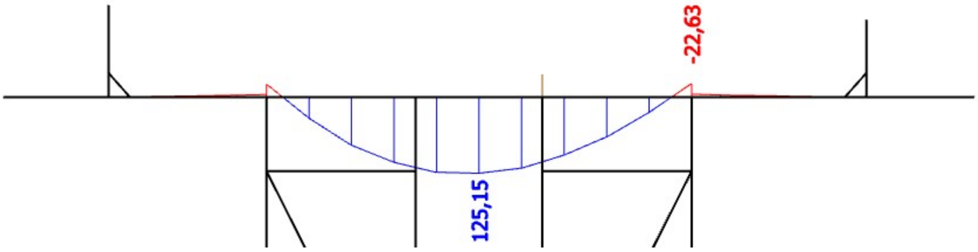
Load capacity:

$$z_{\text{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71}} = \frac{97{,}391}{86{,}170} = \mathbf{1{,}130} \rightarrow \boldsymbol{determinative}$$

Constructional welds of upper flange – transverse splices, not grounded flush (EN 1993-1-9, table 8.3, detail 9):

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$$\lambda_1 = 1{,}03 \; (for\ span\ L = 2 * 2{,}8 = 5{,}6m)$$
$$\lambda_2 = 0{,}83 \; (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00 \; (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00 \; (monorail)$$
$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 1{,}03 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}855$$



Picture 4.2-16 - maximal bending moment caused by traffic load

$$\Phi_3 * \Delta\sigma_{71} = \Phi_3 * \frac{M_{Ed}}{w_{y,eff}} = 1{,}727 * \frac{125{,}2}{0{,}0039881} * 10^{-3} = 54{,}217MPa$$

$$1{,}0 * 0{,}855 * 54{,}217 = 46{,}356MPa \leq \frac{80}{1{,}15} = 97{,}391MPa$$

SATISFACTORY (66,6%)

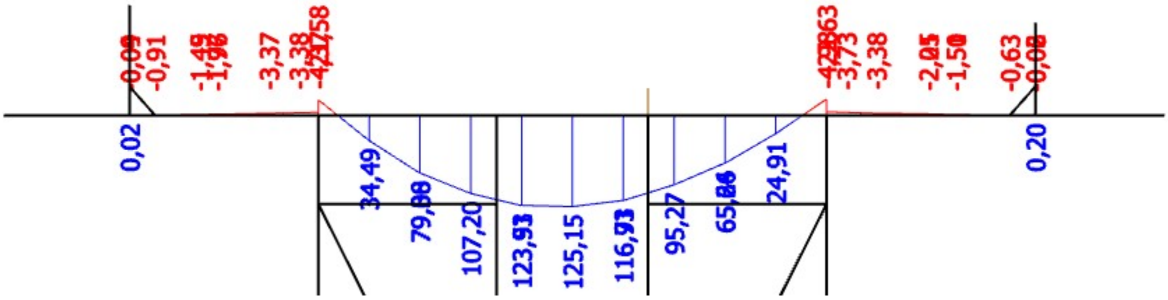
Load capacity:

$$z_{\mathrm{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71}} = \frac{69{,}565}{46{,}356} = \mathbf{1{,}501} \rightarrow not\ determinative$$

Upper weld of web and flange – longitudinal filled weld with a cope hole with radius 50mm (EN 1993-1-9, table 8.2, detail 9):

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$$\lambda_1 = 1{,}03 \; (for\ span\ L = 2 * 2{,}8 = 5{,}6m)$$
$$\lambda_2 = 0{,}83 \; (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00 \; (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00 \; (monorail)$$
$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 1{,}03 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}855$$



Picture 4.2-17 - maximal bending moment caused by traffic load

$$\Phi_3 * \Delta\sigma_{71} = \Phi_3 * \frac{M_{Ed}}{w_{y,eff}} = 1{,}727 * \frac{123{,}9}{0{,}0043348} * 10^{-3} = 49{,}362MPa$$

$$1{,}0 * 0{,}855 * 46{,}881 = 42{,}205MPa \leq \frac{71}{1{,}15} = 61{,}736MPa$$

SATISFACTORY (68,4%)

Load capacity:

$$z_{\mathrm{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71}} = \frac{61{,}736}{42{,}205} = \mathbf{1{,}463} \rightarrow not\ determinative$$

Connection of longitudinal stringer to cross girder (EN 1993-1-9, table 8.9, detail 2):

$$\gamma_{Ff} * \lambda * \Phi_3 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

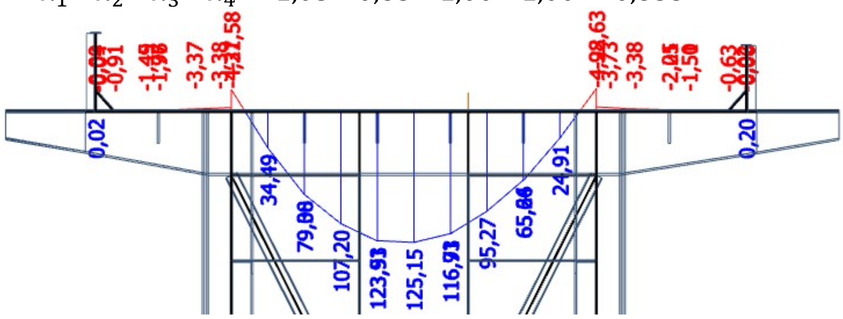
$$\lambda_1 = 1{,}03\ (for\ span\ L = 2 * 2{,}8 = 5{,}6m)$$

$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$

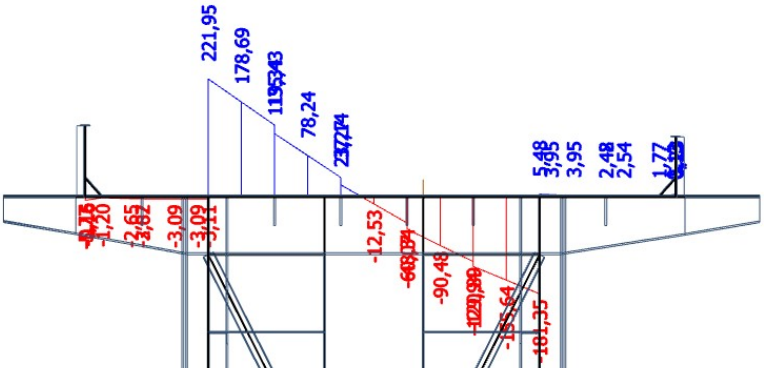
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$

$$\lambda_4 = 1{,}00\ (monorail)$$

$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 1{,}03 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}855$$



Picture 4.2-18 - maximal bending moment caused by traffic load



Picture 4.2-19 - maximal shear force caused by traffic load

Stringer 1 (maximal normal stress is under a cope hole by the upper flange):

$$\Delta\sigma = \frac{\Delta M_s}{w_{net,s}} = \frac{79{,}9}{0{,}0060888} * 10^{-3} = 13{,}122MPa$$

$$\Delta\tau = \frac{\Delta V_s}{A_{w,net,s}} = \frac{135{,}4}{4320} * 10^3 = 31{,}343MPa$$

$$\Phi_3 * \Delta\sigma_{71,eq} = \Phi_3 * \frac{1}{2} * \left(\Delta\sigma + \sqrt{\Delta\sigma^2 + \Delta\tau^2}\right) =$$

$$= 1{,}727 * \frac{1}{2} * \left(13{,}122 + \sqrt{13{,}122^2 + 31{,}343^2}\right) = 40{,}672MPa$$

Stringer 2:

$$\Delta\sigma = \frac{\Delta M_s}{w_{net,s}} = \frac{123{,}9}{0{,}0060888} * 10^{-3} = 20{,}349MPa$$

$$\Delta\tau = \frac{\Delta V_s}{A_{w,net,s}} = \frac{37{,}1}{4320} * 10^3 = 10{,}848MPa$$

$$\Phi_3 * \Delta\sigma_{71,eq} = \Phi_3 * \frac{1}{2} * \left(\Delta\sigma + \sqrt{\Delta\sigma^2 + \Delta\tau^2}\right) =$$

$$= 1{,}727 * \frac{1}{2} * \left(20{,}349 + \sqrt{20{,}349^2 + 10{,}848^2}\right) = 37{,}483MPa$$

$$1{,}0 * 0{,}855 * 40{,}672 = 34{,}774MPa \leq \frac{56}{1{,}15} = 48{,}696MPa$$

SATISFACTORY (71,4%)

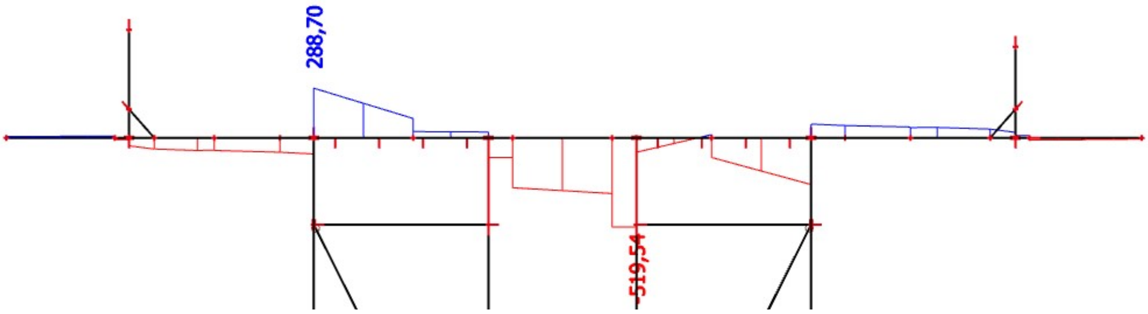
Load capacity:

$$z_{\mathrm{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff}*\lambda*\Phi_3*\Delta\sigma_{71}} = \frac{48{,}696}{34{,}774} = \mathbf{1{,}400} \rightarrow not\ determinative$$

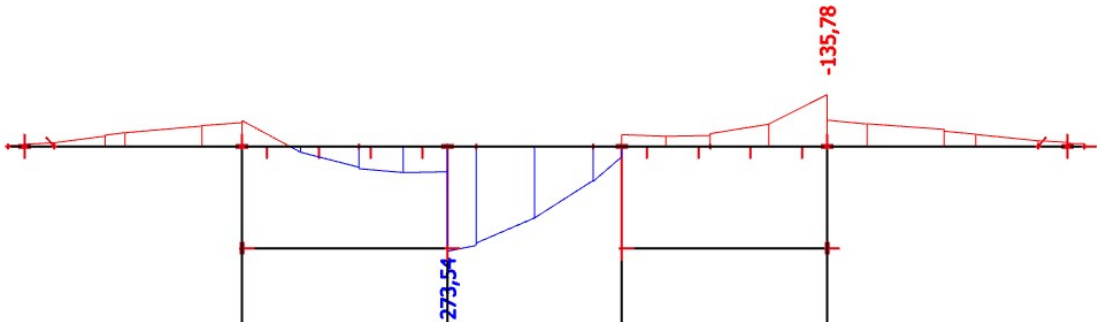
4.3 End cross girder

Combination of loads causing maximal shear force (gr12, formula 10.b):

Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	maintenance load	1,20
	temperature maximal	0,90
	traction force	0,65
	wind R	1,01
	wind Z-, right	1,01
	ECG-LM71,Qr-maxV	2,60
	ECG-NFr-maxV	1,30



Picture 4.3-1 - shear force diagram



Picture 4.3-2 – bending moment diagram

Shear resistance in place of maximal shear force:

$$V_{Rd,1} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{235 * 470 * 14}{\sqrt{3} * 1,0 * 1000} = 892,8kN > 519,5\text{kN}$$

SATISFACTORY (58,2%)

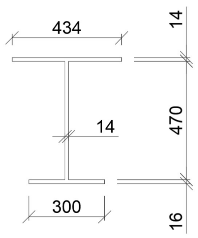
Shear resistance in place of longitudinal stiffener:

$$V_{Rd,2} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{235 * 360 * 14}{\sqrt{3} * 1,0 * 1000} = 683,8kN > 519,5\text{kN}$$

SATISFACTORY (76,0%)

- Flexural buckling – vertical plane:

$$\lambda' = \frac{L_{cr,z}}{i_y} * \frac{1}{\lambda_1} = \frac{983}{208} * \frac{1}{93,913} = 0,050 \leq 0,200 \rightarrow cross\ sectional\ check$$

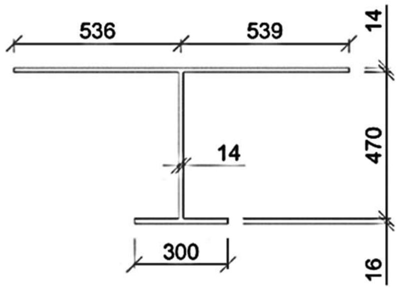


Picture 4.3-3 - effective cross section for compression

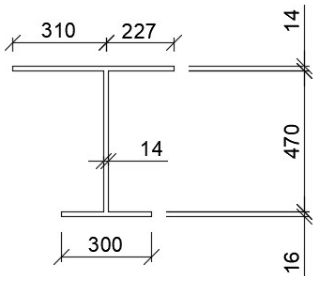
$$N_{Rd} = A_{eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}016896 * \frac{235}{1{,}1} * 1000 = 3609{,}6kN$$

ψ=	1	
k_σ=	4,000	
b=	2200	mm
t=	14	mm
f_y=	235	MPa
ε=	1,000	
λ_P=	2,767	
ρ=	0,333	
b_{eff}=	***732***	***mm***
α_0=	0,577	
b_0=	1100	mm
$L_{e,1}$=	1960	mm
$L_{e,2}$=	2240	mm
κ_1=	0,324	
κ_2=	0,491	
sagging bending		
β^κ=	0,847	
beff=	**620**	**mm**
hogging bending		
β^κ=	0,487	
beff=	**1071**	**mm**

ψ=	1	
k_σ=	0,430	
b=	1000	mm
t=	14	mm
f_y=	235	MPa
ε=	1,000	
λ_P=	3,835	
ρ=	0,248	
b_{eff}=	***248***	***mm***
α_0=	0,498	
b_0=	1000	mm
$L_{e,1}$=	1960	mm
$L_{e,2}$=	2240	mm
κ_1=	0,254	
κ_2=	0,446	
sagging bending		
β^κ=	0,916	
beff=	**227**	**mm**
hogging bending		
β^κ=	0,539	
beff=	**539**	**mm**



Picture 4.3-5 - effective cross section for hogging bending moment M_y



Picture 4.3-4 - effective cross section for sagging bending moment M_y

$$M_{y,el,Rd,+} = w_{y,eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}0030489 * \frac{235}{1{,}1} * 1000 = 651{,}4kNm$$

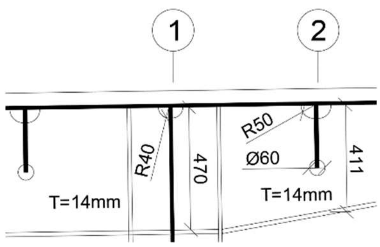
$$M_{y,el,Rd,+} = w_{y,eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}0028804 * \frac{235}{1{,}1} * 1000 = 615{,}4kNm$$

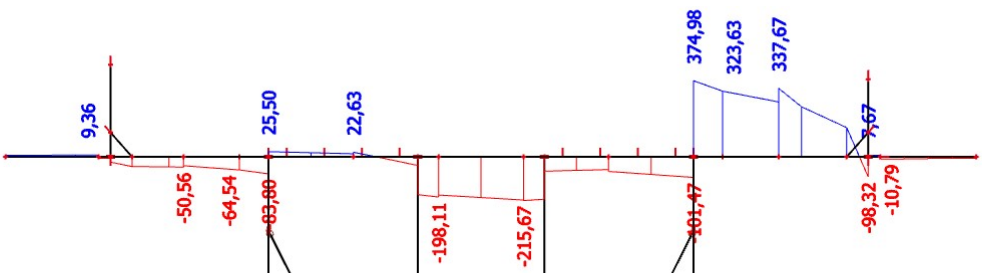
Lower fibre of the end cross girder above web:

$$\mu_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_y}{M_{y,el,Rd}} = -\frac{98{,}3}{3609{,}6} + \frac{273{,}5 + 98{,}3 * 0{,}111}{651{,}4} = 0{,}409$$

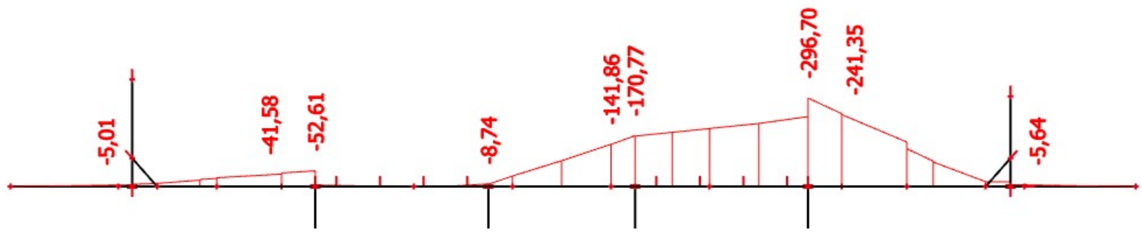
Combination of loads causing maximal shear force and bending moment (derailment – design situation II):

Linear - ultimate	self-weight	1,00
	other permanent load average	1,00
	other permanent load maximal	1,00
	maintenance load	1,00
	wind Z-, left	0,50
	CG,D2-maxV	1,00





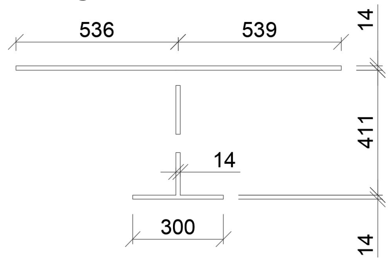
Picture 4.3-6 - shear force diagram



Picture 4.3-7 – bending moment diagram

$$V_{Rd,1} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{235 * 430 * 14}{\sqrt{3} * 1{,}0 * 1000} = 816{,}8kN > 375{,}0\text{kN}$$
$$V_{Rd,2} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{235 * 301 * 14}{\sqrt{3} * 1{,}0 * 1000} = 571{,}7kN > 337{,}7\text{kN}$$

Interaction of shear force and bending moment in cross section 2-2:



Picture 4.3-8 – effective cross section for M_y in section 2-2

$$N_{Rd} = A * \frac{f_{yk}}{\gamma_{M1}} = 0{,}023464 * \frac{235}{1{,}1} * 1000 = 5012{,}8kN$$
$$\text{M}_{\text{y,el,Rd}} = \text{w}_{\text{y,eff}} * \frac{\text{f}_{\text{yk}}}{\gamma_{M1}} = 0{,}0022864 * \frac{235}{1{,}1} * 10^3 * (1 - \frac{61{,}3}{0{,}02346 * 235000/1{,}1}) = 482{,}5\text{kNm}$$

$$\mu_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_y}{M_{y,el,Rd}} = -\frac{61{,}3}{5012{,}8} + \frac{141{,}7 + 61{,}3 * 0{,}038}{488{,}5} = 0{,}283$$
$$\eta_3 = \frac{V_{Ed}}{V_{Rd}} = \frac{337{,}7}{571{,}7} = 0{,}591$$
$$\text{M}_{\text{f,Rd}} = \text{w}_{\text{y,eff}} * \frac{\text{f}_{\text{yk}}}{\gamma_{M1}} = 0{,}0017473 * \frac{235}{1{,}1} * 10^3 * (1 - \frac{61{,}3}{0{,}01925 * 235000/1{,}1}) = 367{,}7\text{kNm}$$

$$\eta_1 + \left(1 - \frac{M_{f,Rd}}{M_{el,Rd}}\right) * (2 * \eta_3 - 1)^2 = 0{,}283 + \left(1 - \frac{367{,}7}{482{,}5}\right) * (2 * 0{,}591 - 1)^2 = 0{,}290 \leq 1{,}0$$

SATISFACTORY (59,1% - shear resistance)

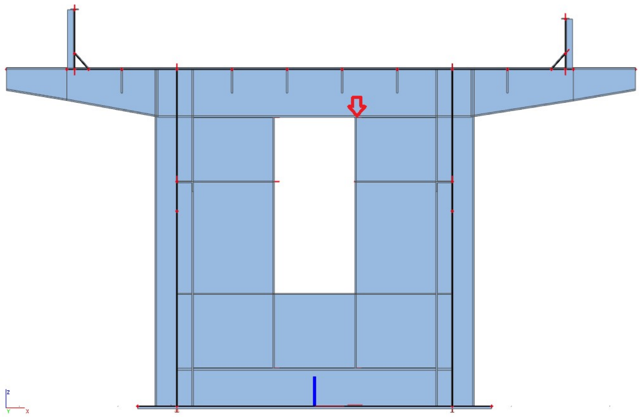
4.3.1 Load capacity

Cross in place of longitudinal stiffener:

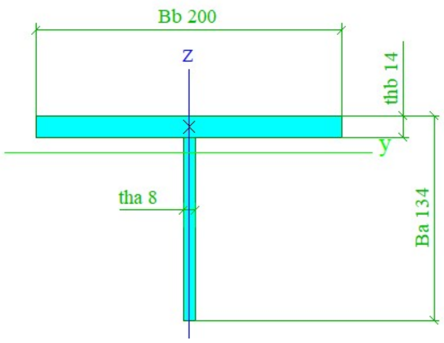
$V_{z,Ed}$	519,5 kN	$V_{z,Ed,LM71}$	337,7 kN	$V_{z,Ed,rs}$	182,2 kN

$$\eta_{3,\text{rs}} = \frac{\text{V}_{\text{Ed,rs}}}{V_{bw,T,Rd}} = \frac{182{,}2}{683{,}8} = 0{,}266$$
$$\eta_{3,\text{LM71}} = \frac{\text{V}_{\text{Ed,LM71}}}{V_{bw,T,Rd}} = \frac{337{,}7}{683{,}8} = 0{,}494$$
$$z_{\text{LM71}} = \frac{1 - \eta_{3,\text{rs}}}{\eta_{3,\text{LM71}}} = \frac{1 - 0{,}266}{0{,}494} = \mathbf{1{,}486}$$

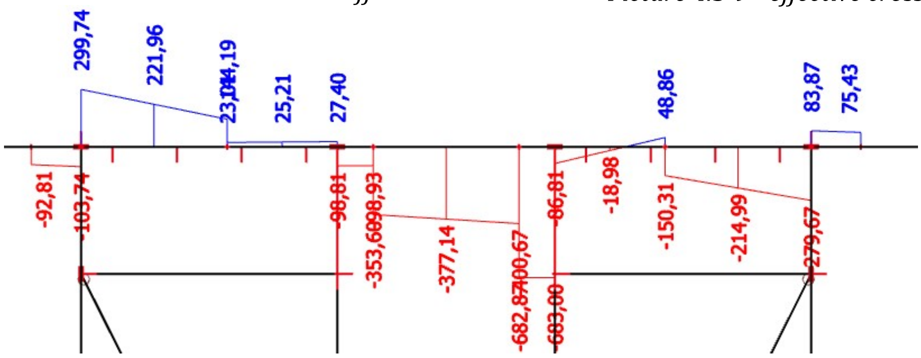
The end cross girder is supported by vertical stiffener. Shear resistance of this stiffener is verified bellow. It should carry the difference of shear force ΔV_{Ed}=597,0kN. Effective cross section consists of full area of stiffener and adjacent part of plate (length 15εt).



Picture 4.3-10 - critical loaded stiffener



Picture 4.3-9 - effective cross section



Picture 4.3-11 - shear force diagram of end cross beam for load capacity z_{LM71}=1,486

- Flexural buckling:

  Buckling length:

  $L_{cr,y} = 1140mm$

  Non-dimensional slenderness:

$$\lambda_1 = \pi * \sqrt{\frac{E}{f_y}} = \pi * \sqrt{\frac{210000}{235}} = 93,913$$

$$\lambda' = \frac{L_{cr,z}}{i_y} * \frac{1}{\lambda_1} = \frac{1140}{34} * \frac{1}{93,913} = 0,357$$

  Reduction factor:

$$\varnothing = 0,5 * \left[1 + \alpha * (\lambda' - 0,2) + \lambda'^2\right] = 0,5 * [1 + 0,49 * (0,357 - 0,2) + 0,357^2] = = 0,602$$

$$\chi_y = \frac{1}{\varnothing + \sqrt{\varnothing^2 - \lambda'^2}} = \frac{1}{0,602 + \sqrt{0,602^2 - 0,357^2}} = 0,920$$

- Torsional-flexural buckling:

  Buckling length:

  $L_{cr,\omega} = 0,75 * 2548 = 1911mm$

$$\lambda_\omega = \sqrt{\frac{I_P}{\frac{I_\omega}{L_{cr,\omega}^2} + \frac{I_t}{25}}} = \sqrt{\frac{1,48320 * 10^7}{\frac{0}{1911^2} + \frac{2,0461*10^5}{25}}} = 42,57$$

$$I_P = I_y + I_z + A * a^2 = 4,4069 * 10^6 + 9,3385 * 10^6 + 3760 * 17^2 = = 1,48320 * 10^7 mm^4$$

$$i_P = \sqrt{\frac{I_P}{A}} = \sqrt{\frac{1,48320 * 10^7}{3760}} = 62,8mm$$

$$I_t = 2{,}0461 * 10^5 mm^4$$

$$\lambda_z = \frac{L_{cr}}{i_z} = \frac{1911}{50} = 38{,}22$$

$$\lambda_{z\omega} = \kappa_z * \sqrt{\lambda_z{}^2 + \lambda_\omega{}^2} = 0{,}797 * \sqrt{42{,}57^2 + 38{,}22^2} = 45{,}60$$

$$\kappa_z = \sqrt{\frac{1 + (a_z/i_p)}{2}} = \sqrt{\frac{1 + (17/62{,}8)}{2}} = 0{,}797$$

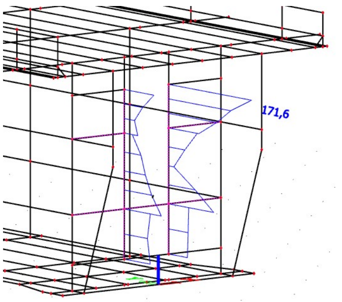
$$\lambda'_{z\omega} = \frac{\lambda_{z\omega}}{\lambda_1} = \frac{45{,}60}{93{,}913} = 0{,}486$$

Reduction factor:

$$\varnothing = 0{,}5 * \left[1 + \alpha * (\lambda' - 0{,}2) + \lambda'^2\right] = 0{,}5 * [1 + 0{,}49 * (0{,}486 - 0{,}2) + 0{,}486^2] =$$
$$= 0{,}688$$

$$\chi_{z\omega} = \frac{1}{\varnothing + \sqrt{\varnothing^2 - \lambda'^2}} = \frac{1}{0{,}688 + \sqrt{0{,}688^2 - 0{,}486^2}} = 0{,}851 \rightarrow determinative$$

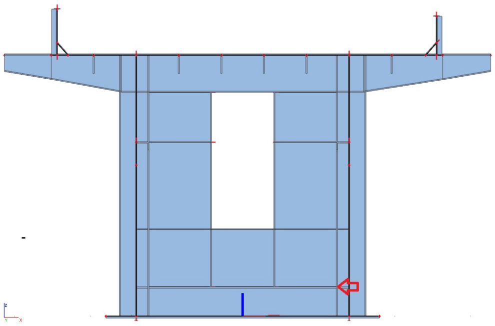
$$N_{Rd} = \chi * A * \frac{f_{yk}}{\gamma_{M1}} = 0{,}851 * 3760 * \frac{235}{1{,}1} * 10^{-3} = 683{,}6kN \geq 597{,}0kN$$

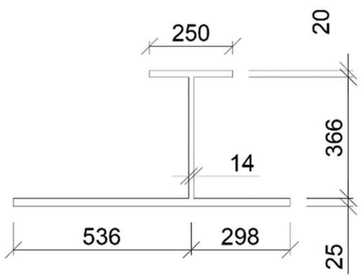


Picture 4.3-12 - stress in stiffener under end cross girder according to software (maximal: 171,6MPa)

Vertical stiffener can carry the difference of shear force in end cross girder.

Maximal shear force in lower cross girder: V_{Ed}=393,1kN. Other inner forces are neglectable (M_{Ed}=42,3kNm).





Picture 4.3-13 - effective cross section of lower cross girder

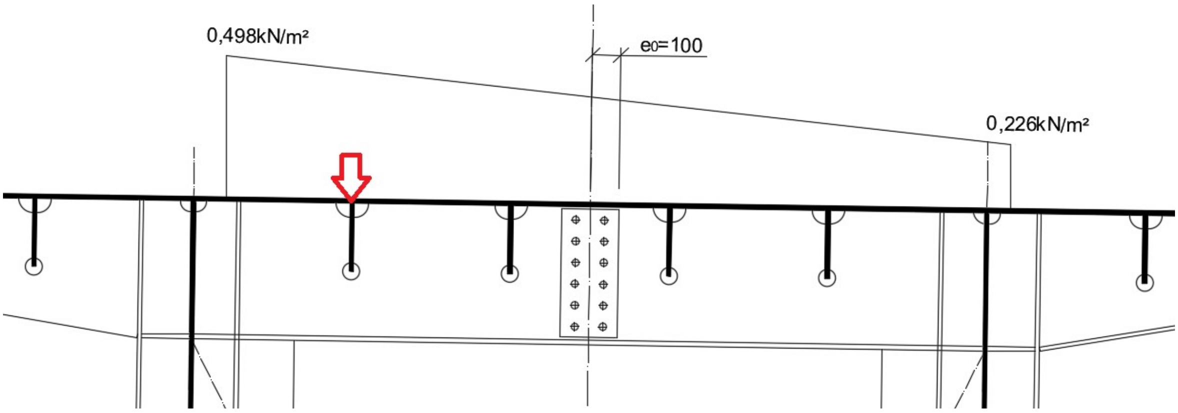
$$V_{Rd} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{235 * 366 * 14}{\sqrt{3} * 1{,}0 * 1000} = 698{,}2kN > 393{,}1\text{kN}$$

The load capacity Z_{LM71}=1,486 is valid.

Stress in all parts of the end cross girder is smaller than in intermediate cross girders. Therefore, there is no need to verify bolted joint of a web and a fatigue.

4.4 Longitudinal stiffener

For maximal load of longitudinal stiffener is considered traffic load with maximal eccentricity (chapter 1.2.2). It is also counted with 100mm possible eccentricity of the position of the track according to following picture:



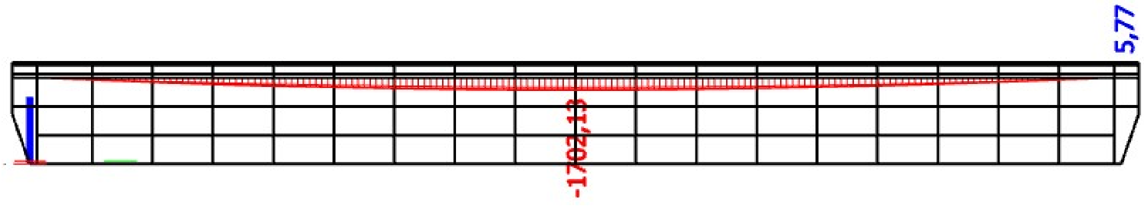
Picture 4.4-1 – critical traffic load of longitudinal stiffener

Critical combination of loads for longitudinal stiffener (gr12, formula 10.b, dynamic factor for global effects is considered 1,071, for local effects 1,600):

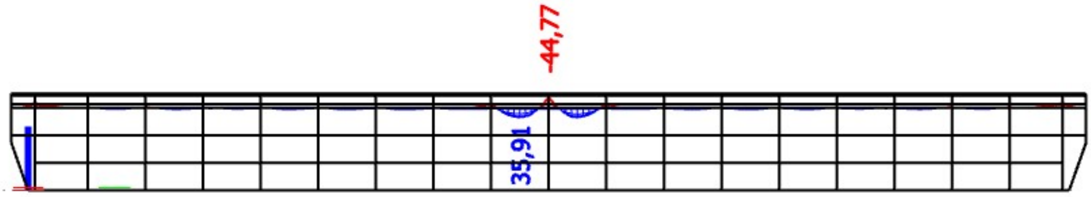
	γ	ψ_0	ξ
permanent load	1,25	-	0,85
LM71 -for normal force ϕ=1,071 - for bending moment ϕ=1,600	1,30	-	
centrifugal force	0,00	-	(favourable)
nosing force (right)	1,30	-	
traction force	0,65	-	
maintenance load	1,50	0,8	
wind horizontal (right)	1,35	0,75	
wind vertical	1,35	0,75	
linear temperature minimal	1,50	0,6	

4.4.1 Cross section in midspan

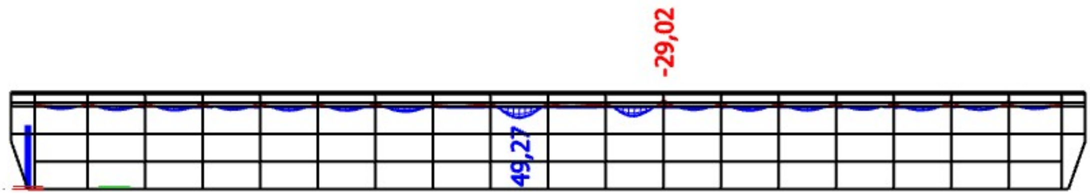
For every load is found its critical position on a bridge and the alleviating effects are neglected. Inner forces diagrams:



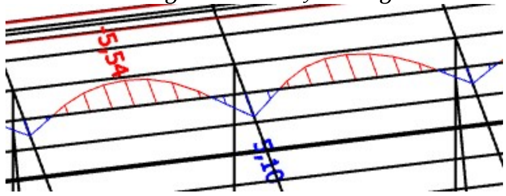
Picture 4.4-2 – maximal normal force in longitudinal stiffener, midspan



Picture 4.4-3 – minimal bending moment M_y in longitudinal stiffener, midspan

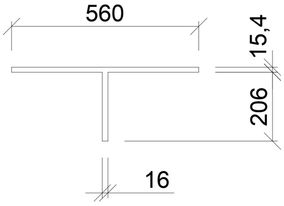


Picture 4.4-4 – maximal bending moment M_y in longitudinal stiffener, midspan



Picture 4.4-5 - bending moment M_z in longitudinal stiffener, midspan

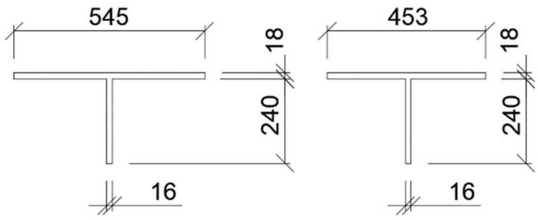
Effective cross section of longitudinal stiffener for normal force is calculated in chapter 4.1.3:



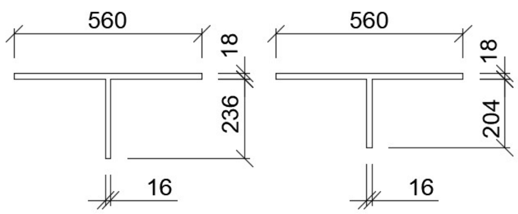
Effective cross section for bending moments:

ψ=	1	
$k_σ$=	4,000	
b=	560	mm
t=	18	mm
f_y=	235	MPa
ε=	1,000	
$λ_P$=	0,548	
ρ=	1,000	
b_{eff}=	***560***	***mm***
$α_0$=	1,000	
b_0=	280	mm
$L_{e,1}$=	1680	mm
$L_{e,2}$=	1200	mm
$κ_1$=	0,167	
$κ_2$=	0,233	
sagging bending		
$β^κ$=	0,973	
beff=	**545**	**mm**
hogging bending		
$β^κ$=	0,809	
beff=	**453**	**mm**

ψ=	0	
$k_σ$=	7,810	
b=	560	mm
t=	18	mm
f_y=	235	MPa
ε=	1,000	
$λ_P$=	0,392	
ρ=	1,000	
b_{eff}=	***560***	***mm***
$α_0$=	1,000	
b_0=	240	mm
$L_{e,1}$=	1680	mm
$L_{e,2}$=	1200	mm
$κ_1$=	0,143	
$κ_2$=	0,200	
sagging bending		
$β^κ$=	0,983	
beff=	**236**	**mm**
hogging bending		
$β^κ$=	0,850	
beff=	**204**	**mm**



Picture 4.4-7 – effective cross section for M_y



Picture 4.4-6 – effective cross section for M_z

Stress in lower fibre: *(interaction coefficients k_{ij} are not used, for T-cross section of longitudinal stiffener there would be according to standard k_{ij}<1, but it is valid for beams, in calculation below is the stiffener for compression taken as a part of whole cross girder)*

$$\sigma_x = \frac{N_{Ed}}{A_{eff}} + \frac{M_{y,Ed}}{w_{y,el,eff}} = \frac{1702,1}{11920} * 10^3 + \frac{44,77}{0,00029897} * 10^{-3} = 292,5MPa < \frac{f_{yk}}{\gamma_{M1}} = \frac{355}{1,1} = = 322,7MPa$$

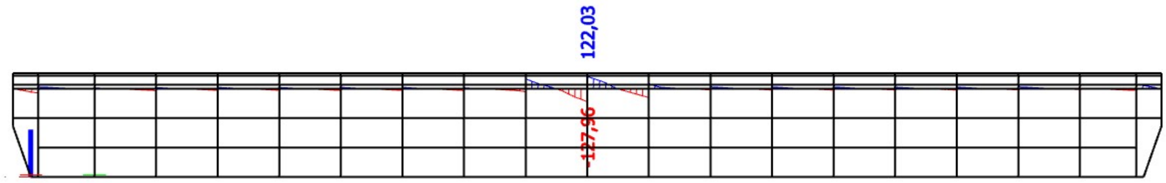
SATISFACTORY (90,6%)

Stress in upper fibre:

$$\sigma_x = \frac{N_{Ed}}{A_{eff}} + \frac{M_{y,Ed}}{w_{y,el,eff}} + \frac{M_{z,Ed}}{w_{z,el,eff}} = \frac{1690,8}{11920} * 10^3 + \frac{49,27}{0,0014360} * 10^{-3} + \frac{5,54}{0,00094109} * 10^{-3} =$$

$$= 182,0MPa < \frac{f_{yk}}{\gamma_{M1}} = \frac{235}{1,1} = 213,6MPa$$

SATISFACTORY (85,2%)



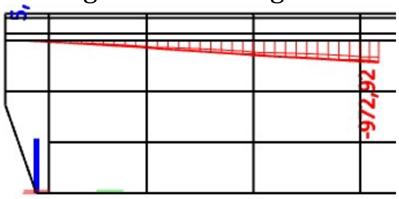
Picture 4.4-8 – maximal shear force in longitudinal stiffener, midspan

$$V_{Rd} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{355 * 240 * 16}{\sqrt{3} * 1,0 * 1000} = 787,0kN > 128,0kN$$

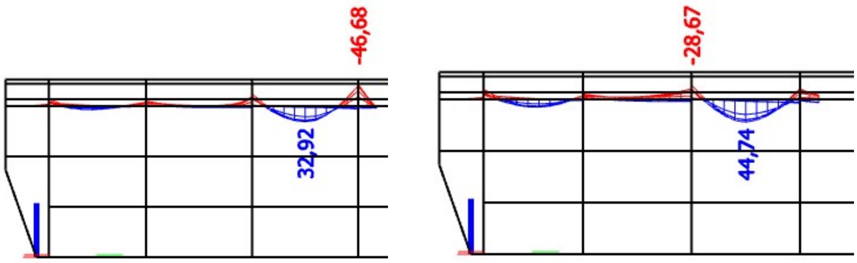
SATISFACTORY (16,3% - no interaction of shear force and bending moment)

4.4.2 Cross section in stationing L=7,4m

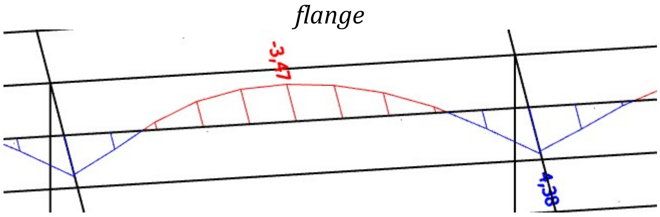
In that place changes thickness of upper flanges. For every load is found its critical position on a bridge and the alleviating effects are neglected. Inner forces diagrams:



Picture 4.4-9 – maximal normal force in longitudinal stiffener, change of thickness of upper flange

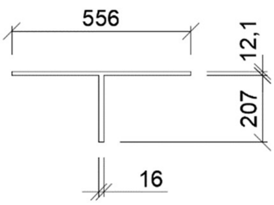


Picture 4.4-10 – extremes of bending moment M_y in longitudinal stiffener, change of thickness of upper flange



Picture 4.4-11 - bending moment M_z in longitudinal stiffener, change of thickness of upper flange

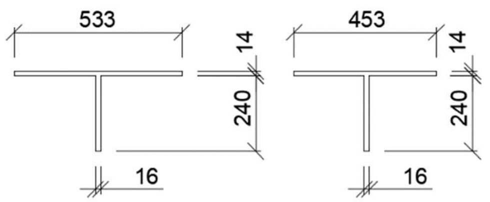
Effective cross section of longitudinal stiffener for normal force is calculated in chapter 4.1.4:



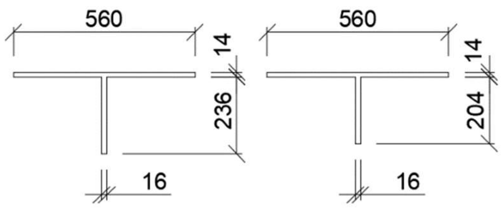
Effective cross section for bending moments:

ψ=	1	
k_σ=	4,000	
b=	560	mm
t=	14	mm
f_y=	235	MPa
ε=	1,000	
λ_P=	0,704	
ρ=	0,976	
b_{eff}=	***547***	***mm***
α_0=	0,988	
b_0=	280	mm
$L_{e,1}$=	1680	mm
$L_{e,2}$=	1200	mm
κ_1=	0,165	
κ_2=	0,233	
sagging bending		
β^κ=	0,974	
beff=	**533**	**mm**
hogging bending		
β^κ=	0,809	
beff=	**453**	**mm**

ψ=	0	
k_σ=	7,810	
b=	560	mm
t=	14	mm
f_y=	235	MPa
ε=	1,000	
λ_P=	0,504	
ρ=	1,000	
b_{eff}=	***560***	***mm***
α_0=	1,000	
b_0=	240	mm
$L_{e,1}$=	1680	mm
$L_{e,2}$=	1200	mm
κ_1=	0,143	
κ_2=	0,200	
sagging bending		
β^κ=	0,983	
beff=	**236**	**mm**
hogging bending		
β^κ=	0,850	
beff=	**204**	**mm**



Picture 4.4-12 – effective cross section for M_y



Picture 4.4-13 – effective cross section for M_z

Stress in lower fibre:

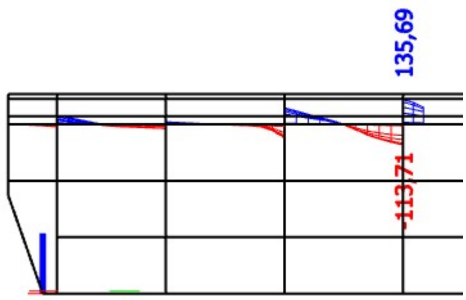
$$\sigma_x = \frac{N_{Ed}}{A_{eff}} + \frac{M_{y,Ed}}{w_{y,el,eff}} = \frac{972,9}{10040} * 10^3 + \frac{46,68}{0,00028685} * 10^{-3} = 259,6MPa < \frac{f_{yk}}{\gamma_{M1}} = \frac{355}{1,1} = = 322,7MPa$$

SATISFACTORY (80,5%)

Stress in upper fibre:

$$\sigma_x = \frac{N_{Ed}}{A_{eff}} + \frac{M_{y,Ed}}{w_{y,el,eff}} + \frac{M_{z,Ed}}{w_{z,el,eff}} = \frac{813,8}{10040} * 10^3 + \frac{44,74}{0,0011889} * 10^{-3} + \frac{3,47}{0,00073202} * 10^{-3} = = 123,4MPa < \frac{f_{yk}}{\gamma_{M1}} = \frac{235}{1,1} = 213,6MPa$$

SATISFACTORY (57,8%)



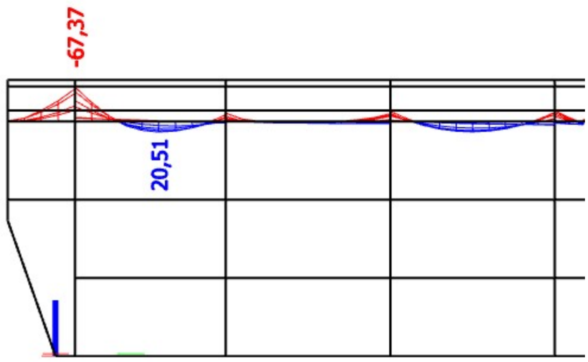
Picture 4.4-14 – maximal shear force in longitudinal stiffener, change of thickness of upper flange

$$V_{Rd} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{355 * 240 * 16}{\sqrt{3} * 1,0 * 1000} = 787,0kN > 135,7\text{kN}$$

SATISFACTORY (17,2% - no interaction of shear force and bending moment)

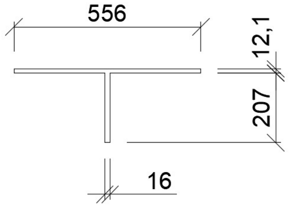
4.4.3 Cross section above support

Because of solid end girder, there is maximal bending moment in cross section above support. For every load is found its critical position on a bridge and the alleviating effects are neglected. Inner forces diagrams:



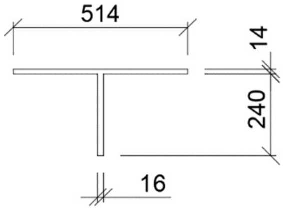
Picture 4.4-15 – extremes of bending moment M_y in longitudinal stiffener, above support

Effective cross section of longitudinal stiffener for normal force is calculated in chapter 4.1.4:



Effective cross section for bending moments:

α_0=	0,988	
b_0=	280	mm
$L_{e,2}$=	2000	mm
κ_2=	0,140	
hogging bending		
β^{κ}=	0,917	
beff=	**514**	**mm**

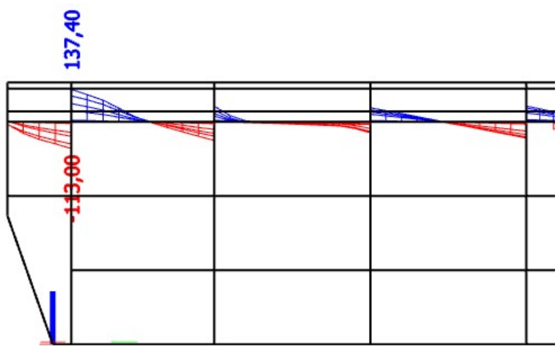


Picture 4.4-16 – effective cross section for M_y

Stress in lower fibre:

$$\sigma_x = \frac{M_{y,Ed}}{w_{y,el,eff}} = \frac{67,37}{0,00029059} * 10^{-3} = 231,8MPa < \frac{f_{yk}}{\gamma_{M1}} = \frac{355}{1,1} = 322,7MPa$$

<u>SATISFACTORY (71,8%)</u>



Picture 4.4-17 – maximal shear force in longitudinal stiffener, above support

$$V_{Rd} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{355 * 240 * 16}{\sqrt{3} * 1,0 * 1000} = 787,0kN > 137,4\text{kN}$$

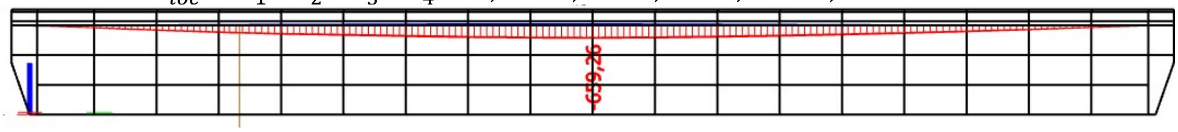
<u>SATISFACTORY (17,5% - no interaction of shear force and bending moment)</u>

4.4.4 Fatigue

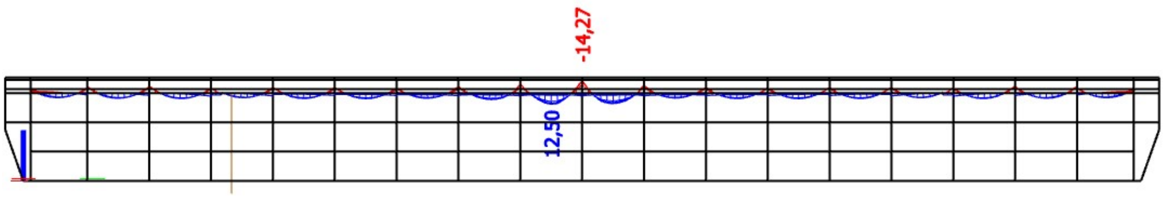
Lower fibre of longitudinal stiffener in midspan (EN 1993-1-9, table 8.1, detail 1):

$$\gamma_{Ff} * \lambda * \Phi_2 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$$\lambda_1 = 1{,}03\ (for\ span\ L = 2 * 2{,}4 = 4{,}8m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda_{loc} = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 1{,}03 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}855$$



Picture 4.4-18 – maximal normal force caused by traffic load in longitudinal stiffener, midspan



Picture 4.4-19 – minimal bending moment M_y caused by traffic load in longitudinal stiffener, midspan

$$\sigma_{glob} = \frac{N_{Ed}}{A_{eff}} = \frac{659{,}3}{11920} * 10^3 = 55{,}310MPa$$

$$\sigma_{loc} = \frac{M_{y,Ed}}{w_{y,el,eff}} = \frac{14{,}27}{0{,}00029897} * 10^{-3} = 47{,}731MPa$$

$$\lambda * \Phi_2 * \Delta\sigma_{71} = \lambda_{loc} * \Phi_{loc} * \Delta\sigma_{loc} + \lambda_{glob} * \Phi_{glob} * \Delta\sigma_{glob} =$$
$$= 0{,}855 * 1{,}600 * 47{,}731 + 0{,}531 * 1{,}071 * 55{,}310 =$$
$$= 96{,}750MPa$$

$$1{,}0 * 96{,}750 = 96{,}750MPa \leq \frac{160}{1{,}15} = 139{,}130MPa$$

SATISFACTORY (69,5%)

Load capacity:

$$z_{\text{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_2 * \Delta\sigma_{71}} = \frac{139{,}130}{96{,}750} = \mathbf{1{,}438} \rightarrow not\ determinative$$

Upper fibre of web in midspan, butt weld – longitudinal attachment (EN 1993-1-9, table 8.4, detail 1):

$$\gamma_{Ff} * \lambda * \Phi_2 * \Delta\sigma_{71} \leq \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

$$\lambda_1 = 1{,}03\ (for\ span\ L = 2 * 2{,}4 = 4{,}8m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda_{loc} = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 1{,}03 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}855$$

$$\sigma_{glob} = \frac{N_{Ed}}{A_{eff}} = \frac{651{,}21}{11920} * 10^3 = 54{,}632MPa$$

$$\sigma_{loc} = \frac{M_{y,Ed}}{w_{y,el,eff}} = \frac{12{,}50}{0{,}0013930} * 10^{-3} = 8{,}973MPa$$

$$\lambda * \Phi_2 * \Delta\sigma_{71} = \lambda_{loc} * \Phi_{loc} * \Delta\sigma_{loc} + \lambda_{glob} * \Phi_{glob} * \Delta\sigma_{glob} =$$
$$= 0{,}855 * 1{,}600 * 8{,}973 + 0{,}531 * 1{,}071 * 54{,}632 = 43{,}345MP$$

$$1{,}0 * 43{,}345 = 43{,}345MPa \leq \frac{56}{1{,}15} = 48{,}696MPa$$

SATISFACTORY (89,0%)

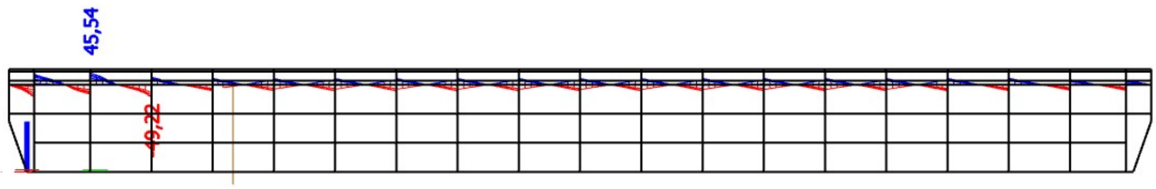
Load capacity:

$$z_{\mathrm{LM71}} = \frac{\Delta\sigma_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_2 * \Delta\sigma_{71}} = \frac{48{,}696}{43{,}345} = \mathbf{1,123} \rightarrow \boldsymbol{determinative}$$

Upper fibre of web (EN 1993-1-9, table 8.1, detail 6), welds of web and flanges - continuous fillet weld transmitting a shear flow (EN 1993-1-9, table 8.2, detail 8):

$$\gamma_{Ff} * \lambda * \Phi_2 * \Delta\tau_{71} \leq \frac{\Delta\tau_C}{\gamma_{Mf}}$$

$$\lambda_1 = 1{,}03\ (for\ span\ L = 2 * 2{,}4 = 4{,}8m)$$
$$\lambda_2 = 0{,}83\ (traffic\ per\ year\ 10^6 t)$$
$$\lambda_3 = 1{,}00\ (design\ life\ 100\ years)$$
$$\lambda_4 = 1{,}00\ (monorail)$$
$$\lambda_{loc} = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4 = 1{,}03 * 0{,}83 * 1{,}00 * 1{,}00 = 0{,}855$$



Picture 4.4-20 - maximal shear force caused by traffic load

$$\Phi_2 * \Delta\tau = \Phi_2 * \frac{V_{Ed,1} * S_f}{t * I_y} = 1{,}600 * \frac{49{,}22 * 0{,}000268632}{16 * 0{,}000059446} = 22{,}242MPa$$

$$1{,}0 * 0{,}855 * 22{,}242 = 19{,}017MPa \leq \frac{80}{1{,}15} = 69{,}565MPa$$

SATISFACTORY (27,3%)

Load capacity:

$$z_{\mathrm{LM71}} = \frac{\Delta\tau_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_2 * \Delta\tau_{71}} = \frac{69{,}565}{19{,}017} = \mathbf{3,659} \rightarrow not\ determinative$$

Load carrying joint of longitudinal stiffener – partial penetration joint (EN 1993-1-9, table 8.5, details 1,3):

$$\gamma_{Ff} * \lambda * \Phi_2 * \Delta\tau_{71} \leq \frac{\Delta\tau_C}{\gamma_{Mf}}$$

$$\Delta\tau_{loc,w} = \frac{V_{Ed}}{4 * a * l} = \frac{49{,}22}{4 * 5 * 160} * 10^3 = 15{,}381MPa$$

$$\lambda_{loc} * \Phi_{loc} * \Delta\tau_{loc} = 0{,}855 * 1{,}600 * 15{,}381 = 21{,}042MPa$$

$$1{,}0 * 21{,}042 = 21{,}042MPa \leq \frac{80}{1{,}15} = 69{,}565MPa$$

SATISFACTORY (30,2%)

Load capacity:

$$z_{\mathrm{LM71}} = \frac{\Delta\tau_C/\gamma_{Mf}}{\gamma_{Ff} * \lambda * \Phi_2 * \Delta\tau_{71}} = \frac{69{,}565}{21{,}042} = \mathbf{3,306} \rightarrow not\ determinative$$

4.4.5 Load capacity

Lower fibres:

N_{Ed}	1702,1 kN	$N_{Ed,LM71}$	882,1 kN	$N_{Ed,rs}$	820,1 kN
$M_{y,Ed}$	44,77 kNm	$M_{y,Ed,LM71}$	40,79 kNm	$M_{y,Ed,rs}$	3,98 kNm

$$\eta_{1,\mathrm{rs}} = \frac{N_{Ed,rs}}{A_{eff} * f_y} + \frac{M_{y,Ed,rs}}{w_{y,el,eff} * f_y} = \frac{820{,}1 * 10^3}{11920 * 355/1{,}1} + \frac{3{,}98 * 10^{-3}}{0{,}00029897 * 355/1{,}1} = 0{,}254$$

$$\eta_{1,\mathrm{LM71}} = \frac{N_{Ed,LM71}}{A_{eff} * f_y} + \frac{M_{y,Ed,LM71}}{w_{y,el,eff} * f_y} = \frac{882{,}1 * 10^3}{11920 * 355/1{,}1} + \frac{40{,}79 * 10^{-3}}{0{,}00029897 * 355/1{,}1} = 0{,}652$$

$$z_{\mathrm{LM71}} = \frac{1 - \eta_{1,\mathrm{rs}}}{\eta_{1,\mathrm{LM71}}} = \frac{1 - 0{,}254}{0{,}652} = \mathbf{1{,}144}$$

$$V_{Ed} = 1{,}144 * 112{,}8 + 15{,}1 = 144{,}1kN$$

$$\mu_2 = \frac{V_{Ed}}{V_{Rd}} = \frac{144{,}1}{787{,}0} = 0{,}183 < 0{,}5$$

Load capacity is valid.

Lower fibres:

N_{Ed}	1690,8 kN	$N_{Ed,LM71}$	875,0 kN	$N_{Ed,rs}$	815,8 kN
$M_{y,Ed}$	49,27 kNm	$M_{y,Ed,LM71}$	43,77 kNm	$M_{y,Ed,rs}$	5,50 kNm
$M_{z,Ed}$	5,54 kNm	$M_{z,Ed,LM71}$	3,83 kNm	$M_{z,Ed,rs}$	1,71 kNm

$$\eta_{1,\mathrm{rs}} = \frac{N_{Ed,rs}}{A_{eff} * f_y} + \frac{M_{y,Ed,rs}}{w_{y,el,eff} * f_y} + \frac{M_{z,Ed,rs}}{w_{z,el,eff} * f_y} =$$

$$= \frac{815{,}8 * 10^3}{11920 * \frac{235}{1{,}1}} + \frac{5{,}50 * 10^{-3}}{0{,}0014360 * \frac{235}{1{,}1}} + \frac{1{,}71 * 10^{-3}}{0{,}00094109 * \frac{235}{1{,}1}} = 0{,}347$$

$$\eta_{1,\mathrm{LM71}} = \frac{N_{Ed,LM71}}{A_{eff} * f_y} + \frac{M_{y,Ed,LM71}}{w_{y,el,eff} * f_y} + \frac{M_{z,Ed,LM71}}{w_{z,el,eff} * f_y} =$$

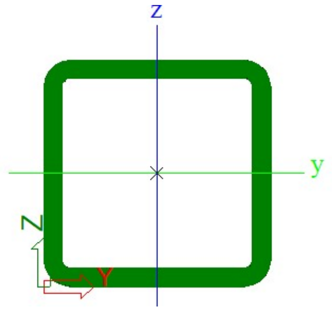
$$= \frac{875{,}0 * 10^3}{11920 * 235/1{,}1} + \frac{43{,}77 * 10^{-3}}{0{,}0014360 * 235/1{,}1} + \frac{3{,}83 * 10^{-3}}{0{,}00094109 * 235/1{,}1} = 0{,}505$$

$$z_{\mathrm{LM71}} = \frac{1 - \eta_{1,\mathrm{rs}}}{\eta_{1,\mathrm{LM71}}} = \frac{1 - 0{,}347}{0{,}505} = \mathbf{1{,}293}$$

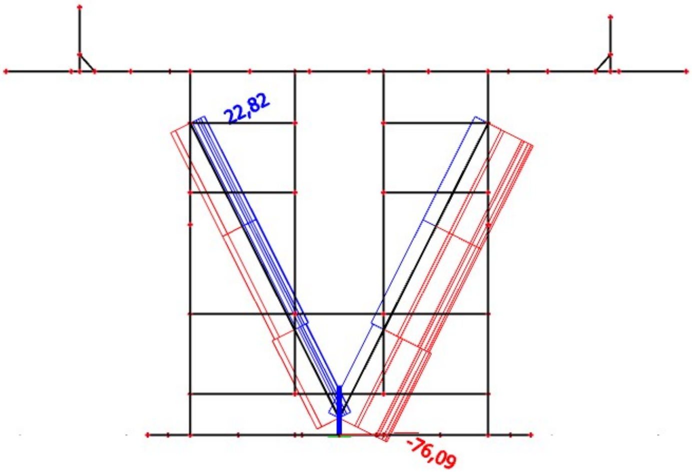
4.5 Stiffener inside box girder

Combination of loads causing maximal shear force (gr12, formula 10.b):

Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	maintenance load	1,20
	temperature minimal	0,90
	traction force	0,65
	wind L	1,01
	wind Z-, left	1,01
	stiffener-maxN-LM71,Ql	1,39
	stiffener-maxN-CF	1,30
	stiffener-maxN-NFl	1,30



Picture 4.5-1 – cross section (SHS 100x8)



Picture 4.5-2 – maximal normal force

- Flexural buckling:

Buckling length:

$L_{cr,y} = 3108mm$

Non-dimensional slenderness:

$$\lambda_1 = \pi * \sqrt{\frac{E}{f_y}} = \pi * \sqrt{\frac{210000}{235}} = 93{,}913$$

$$\lambda' = \frac{L_{cr,z}}{i_y} * \frac{1}{\lambda_1} = \frac{3108}{37} * \frac{1}{93{,}913} = 0{,}894$$

Reduction factor:

$$\varnothing = 0{,}5 * \left[1 + \alpha * (\lambda' - 0{,}2) + \lambda'^2\right] = 0{,}5 * [1 + 0{,}21 * (0{,}894 - 0{,}2) + 0{,}894^2] = 0{,}973$$

$$\chi_y = \frac{1}{\varnothing + \sqrt{\varnothing^2 - \lambda'^2}} = \frac{1}{0{,}973 + \sqrt{0{,}973^2 - 0{,}894^2}} = 0{,}737$$

$$\mathrm{N_{Rd}} = \chi * \mathrm{A} * \frac{\mathrm{f_{yk}}}{\gamma_{M1}} = 0{,}737 * 2880 * \frac{235}{1{,}1} * 10^{-3} = 453{,}4kN \geq 76{,}1kN$$

SATISFACTORY (16,8%)

4.5.1 Load capacity

N_{Ed}	76,1 kN	$N_{Ed,LM71}$	53,3 kN	$N_{Ed,rs}$	22,8 kN

$$\eta_{1,\mathrm{rs}} = \frac{N_{Ed,rs}}{N_{Rd}} = \frac{22{,}8}{453{,}4} = 0{,}050$$

$$\eta_{1,\mathrm{LM71}} = \frac{N_{Ed,LM71}}{N_{Rd}} = \frac{53{,}3}{453{,}4} = 0{,}118$$

$$z_{\mathrm{LM71}} = \frac{1-\eta_{1,\mathrm{rs}}}{\eta_{1,\mathrm{LM71}}} = \frac{1-0{,}050}{0{,}118} = \mathbf{8{,}051}$$

4.6 Bridge deck

Combination of loads causing maximal shear force (gr12, formula 10.b):

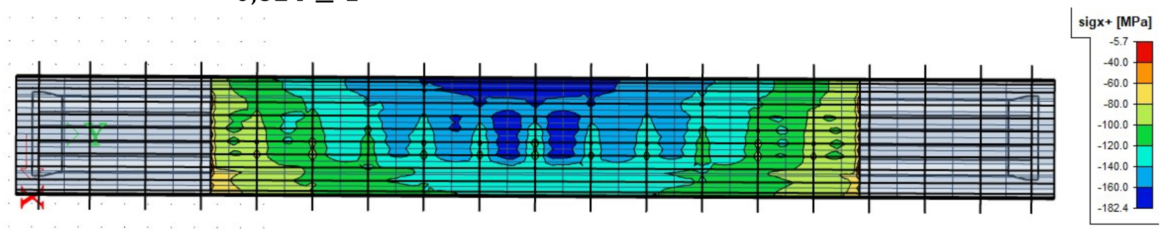
Linear - ultimate	self-weight	1,06
	other permanent load average	1,06
	other permanent load maximal	1,06
	maintenance load	1,20
	temperature max linear	0,90
	traction force	0,65
	wind Z-	1,01
	wind L with train	1,01
	LM71 - L - maximal M	2,08
	centrifugal force - maximal M	1,30
	nosing force - L - maximal M	1,30

Thickness of plates is modified according to effective cross sections calculated in previous captures, so local and global buckling effect is included. For upper flange is used orthotropy for different directions.

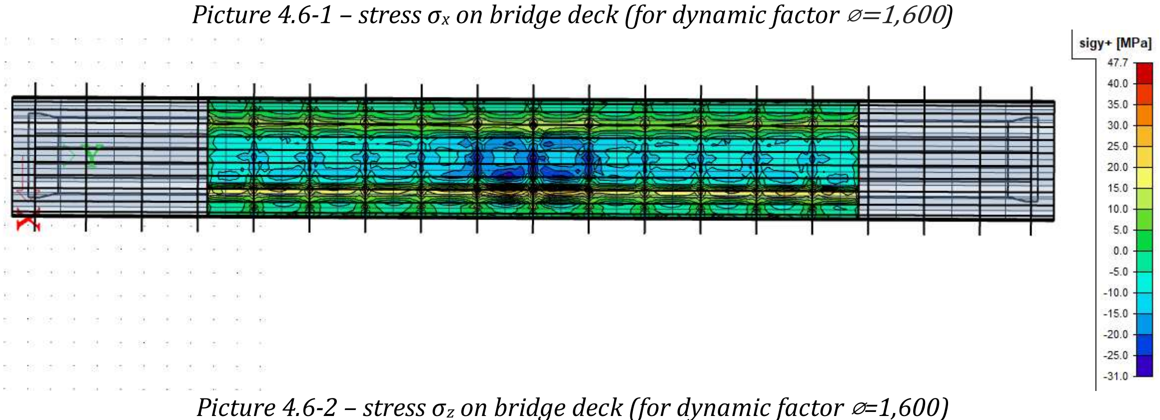
$$\rho = \left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M1}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M1}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{\frac{f_y}{\gamma_{M1}}}\right) * \left(\frac{\sigma_{z,Ed}}{\frac{f_y}{\gamma_{M1}}}\right) + 3 * \left(\frac{\tau_{Ed}}{f_y/\gamma_{M1}}\right)^2 =$$

$$= \left(\frac{-173,6}{235/1,1}\right)^2 + \left(\frac{34,8}{235/1,1}\right)^2 - \left(\frac{-173,6}{235/1,1}\right) * \left(\frac{34,8}{235/1,1}\right) + 3 * \left(\frac{8,3}{235/1,1}\right)^2 =$$

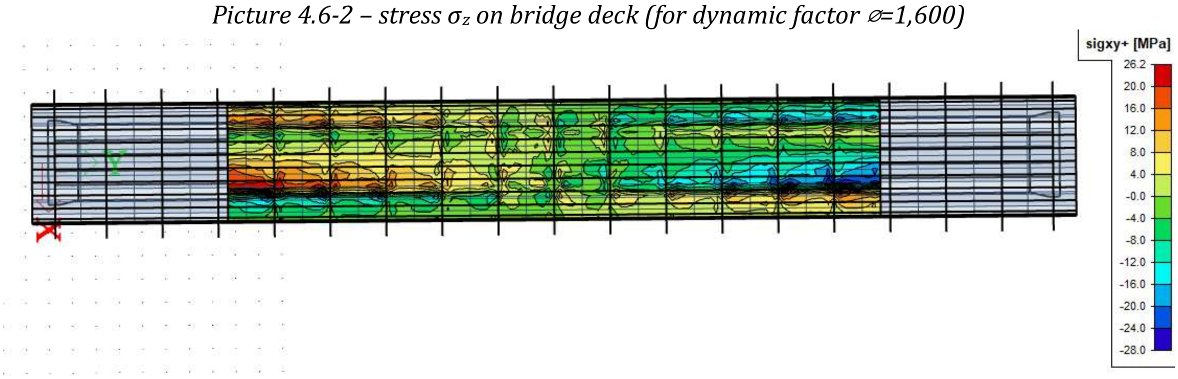
$$= 0,824 \leq 1$$



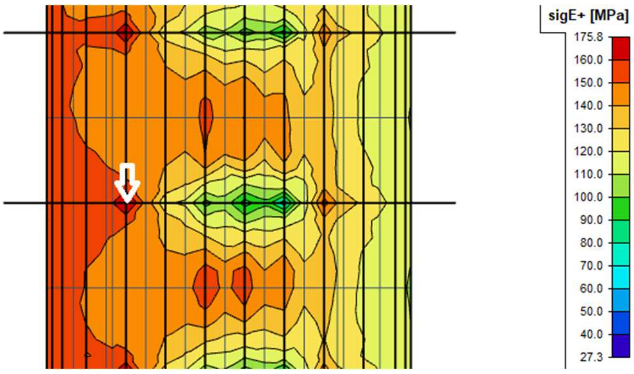
Picture 4.6-1 – stress σ_x on bridge deck (for dynamic factor ⌀=1,600)



Picture 4.6-2 – stress σ_z on bridge deck (for dynamic factor ⌀=1,600)

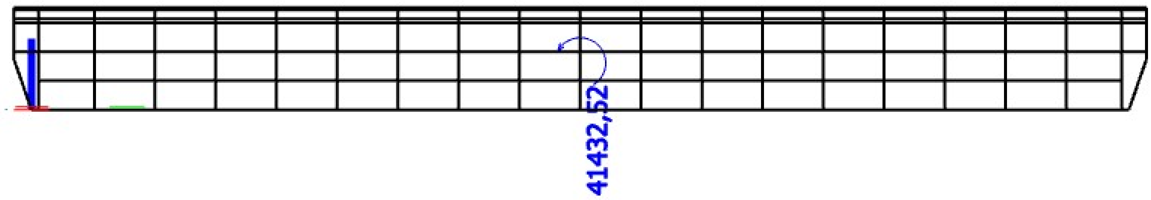


Picture 4.6-3 – stress τ on bridge deck (for dynamic factor ⌀=1,600)



Picture 4.6-4 – maximal stress on bridge deck (for dynamic factor ⌀=1,600)

These results are calculated for dynamic factor ⌀=1,600, but correctly should be for global influence (normal stress caused by bending of a main box girder) used dynamic factor ⌀=1,071. It is taken in consideration bellow:



Picture 4.6-5 - bending moment caused by traffic load LM71 (for dynamic factor ⌀=1,600)

Modified normal stress in direction x:

$$\sigma_{x,Ed,M} = \frac{M_{Ed}}{I_y} * z_f = \frac{41432,5*10^{-3}}{0,73143} * 1,580 = 89,5MPa$$

$$\sigma_{x,Ed} = -173,6 + \frac{(1,600 - 1,071)}{1,600} * 89,5 = -144,0MPa$$

Assessment of a plate of bridge deck

$$\rho = \left(\frac{\sigma_{x,Ed}}{f_y/\gamma_{M1}}\right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y/\gamma_{M1}}\right)^2 - \left(\frac{\sigma_{x,Ed}}{\frac{f_y}{\gamma_{M1}}}\right) * \left(\frac{\sigma_{z,Ed}}{\frac{f_y}{\gamma_{M1}}}\right) + 3 * \left(\frac{\tau_{Ed}}{f_y/\gamma_{M1}}\right)^2 =$$

$$= \left(\frac{-144,0}{235/1,1}\right)^2 + \left(\frac{34,8}{235/1,1}\right)^2 - \left(\frac{-144,0}{235/1,1}\right) * \left(\frac{34,8}{235/1,1}\right) + 3 * \left(\frac{8,3}{235/1,1}\right)^2 =$$

$$= 0,595 \leq 1$$

<u>SATISFACTORY</u>

4.6.1 Load capacity

Load capacity of bridge deck is find iteratively using a table below:

	σ_x[MPa]	σ_z[MPa]	τ[MPa]
μ_{LM71}	-79,1	25,6	6,9
μ_{rs}	-64,9	9,2	1,4

z_{LM71}= 1,513

	σ_x[MPa]	σ_z[MPa]	τ[MPa]	ρ
$z_{LM71}*\sigma$	-184,6	47,9	11,8	1,000

4.7 Verification of rigid stiffeners

4.7.1 Upper cross girder

- critical buckling stress – plate type behaviour

$$\alpha = \frac{a}{b} = \frac{2800}{42800} = 0{,}065 \leq \sqrt[4]{\gamma} = \sqrt[4]{886{,}7} = 5{,}457$$

$$\delta = \frac{\sum A_{sl}}{A_p} = 0{,}205$$

$$\gamma = \frac{I_{sl}}{I_p} = 886{,}7$$

$$\psi = 1$$

$$k_\sigma = \frac{2*((1+\alpha^2)^2+\gamma-1)}{\alpha^2*(\psi+1)*(1+\delta)} = \frac{2*((1+0{,}065^2)^2+886{,}7-1)}{0{,}065^2*(1+1)*(1+0{,}205)} = 174167{,}5$$

$$\sigma_E = 190000*\left(\frac{t}{b}\right)^2 = 190000*\left(\frac{18}{42800}\right)^2 = 0{,}034MPa$$

$$\sigma_{cr,p} = k_\sigma * \sigma_E = 174167{,}5*0{,}034 = 5853{,}0MPa$$

- critical buckling stress – column type behaviour

$$\sigma_{cr,c} = \frac{\pi^2*E*I_{sl,1}}{A_{sl,1}*a^2} = \frac{\pi^2*210000*1{,}0343*10^9}{52040*2800^2} = 5254{,}3MPa$$

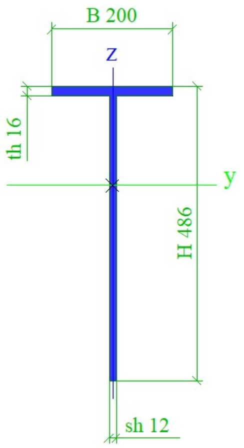
Cross girder is verified using N_{Ed} calculated for cross section in midspan and minimal load capacity z_{LM71}=1,051:

$$w_0 = \frac{s}{300} = \frac{2400}{300} = 8mm$$

$$\sigma_m = \frac{\sigma_{cr,c}}{\sigma_{cr,p}}*\frac{N_{Ed}}{b}*\left(\frac{1}{a_1}+\frac{1}{a_2}\right) = \frac{5254{,}3}{5853{,}0}*\frac{12682{,}2}{2800}*\left(\frac{1}{2400}+\frac{1}{2400}\right)*10^3 = 3{,}388MPa$$

$$u = \frac{\pi^2*E*e_{max}}{\frac{f_y*300*b}{\gamma_{M1}}} = \frac{\pi^2*210000*323}{\frac{235*300*2800}{1{,}1}} = 3{,}73050$$

$$I_{st} = \frac{\sigma_m}{E}*\left(\frac{b}{\pi}\right)^4*\left(1+w_0*\frac{300}{b}*u\right) = \frac{3{,}388}{210000}*\left(\frac{2800}{\pi}\right)^4*\left(1+8*\frac{300}{2800}*3{,}73050\right) = 4{,}27369*10^7mm^4$$



Picture 4.7-1 - cross girder

$I_{CG} = 2{,}24450*10^8mm^4 > 4{,}27369*10^7mm^4$

<u>SATISFACTORY – cross girder provides rigid support for longitudinal stiffeners</u>

Verification of resistance to torsional buckling:

$$I_y = \frac{1}{12} * (200 * 16^3 + 12 * 470^3) + 200 * 16 * 478^2 + 470 * 12 * 235^2 = 1{,}14651 * 10^9 mm^4$$

$$I_z = \frac{1}{12} * (16 * 200^3 + 470 * 12^3) = 1{,}07343 * 10^7 mm^4$$

$$\frac{I_t}{I_p} = \frac{\frac{1}{3} * (470 * 12^3 + 200 * 16^3)}{1{,}14651 * 10^9 + 1{,}07343 * 10^7} = \frac{543786{,}7}{1{,}15724 * 10^9} = 0{,}000470 > 5{,}3 * \frac{f_y}{E} = 5{,}3 * \frac{235}{210000} = 0{,}00593$$

NOT SATISFACTORY

$$L_{cr,\omega} = 0{,}75 * 2400 = 1800mm$$

$$\sigma_{cr} = \frac{1}{I_P} * \left(GI_t + \frac{\pi^2 * EI_w}{a^2}\right) = \frac{1}{1{,}15724 * 10^9} * \left(81000 * 543786{,}7 + \frac{\pi^2 * 210000 * 2{,}51942 * 10^{12}}{1800^2}\right) = 1430{,}7MPa > 6 * f_y = 6 * 235 = 1410MPa$$

SATISFACTORY (fulfilment of one condition is sufficient)

4.7.2 Upper longitudinal stiffener

Verification of resistance to torsional buckling:

$$I_y = \frac{1}{12} * 16 * 240^3 + 240 * 16 * 120^2 = 7{,}37280 * 10^7 mm^4$$

$$I_z = \frac{1}{12} * 240 * 16^3 = 81920mm^4$$

$$\frac{I_t}{I_p} = \frac{\frac{1}{3} * 240 * 16^3}{7{,}37280 * 10^7 + 81920} = \frac{327680}{7{,}37810 * 10^7} = 0{,}00444 > 5{,}3 * \frac{f_y}{E} = 5{,}3 * \frac{355}{210000} = 0{,}00896$$

NOT SATISFACTORY

$$\sigma_{cr} = \frac{1}{I_P} * \left(GI_t + \frac{\pi^2 * EI_w}{a^2}\right) = \frac{1}{7{,}37810 * 10^7} * (81000 * 327680 + 0) = 359{,}7MPa > 6 * f_y = 6 * 355 = 2130MPa$$

NOT SATISFACTORY

Lateral torsional buckling is verified by non-linear analysis in chapter 4.7.6.

4.7.3 Longitudinal stiffeners of web and lower flange

Verification of resistance to torsional buckling:

$$I_P = 8{,}917 * 10^6 + 2{,}695 * 10^6 + 3460 * (106^2 + 24^2) = 5{,}248 * 10^7 mm^4$$

$$I_t = \frac{1}{3} * (160 + 100) * 14^3 = 237813mm^4$$

$$\frac{I_t}{I_p} = \frac{237813}{5{,}248 * 10^7} = 0{,}00453 > 5{,}3 * \frac{f_y}{E} = 5{,}3 * \frac{235}{210000} = 0{,}00593$$

NOT SATISFACTORY

$$\sigma_{cr} = \frac{1}{I_P} * \left(GI_t + \frac{\pi^2 * EI_w}{a^2}\right) = \frac{1}{5{,}248 * 10^7} * \left(81000 * 237813 + \frac{\pi^2 * 210000 * 1{,}19467 * 10^{11}}{2400^2}\right) = 1186{,}2MPa > 6 * f_y = 6 * 355 = 2130MPa$$

NOT SATISFACTORY

Longitudinal stiffeners L160x100x14 are in most cases in tensile zone or the compression is not significant (σ_{com}=60,6MPa<<f_y/γ_{M1}=235/1,1=213,6MPa), so we can suppose, that they do not buckle in torsion.

4.7.4 End transverse stiffener of web

Verification of resistance to torsional buckling:

$I_P = 3{,}8944 * 10^8 + 7{,}2082 * 10^7 + 20492 * 27^2 = 4{,}76461 * 10^8 mm^4$

$I_t = 8{,}9089 * 10^5 mm^4$

$$\frac{I_t}{I_p} = \frac{8{,}9089 * 10^5}{4{,}76461 * 10^8} = 0{,}00189 > 5{,}3 * \frac{f_y}{E} = 5{,}3 * \frac{235}{210000} = 0{,}00593$$

NOT SATISFACTORY

$L_{cr,\omega} = 0{,}75 * (3420 - 400 - 486) = 1900mm$

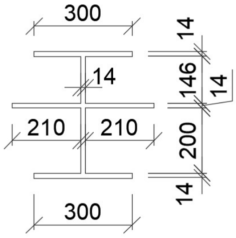
$$\sigma_{cr} = \frac{1}{I_P} * \left(GI_t + \frac{\pi^2 * EI_w}{L_{cr,\omega}{}^2}\right) =$$

$$= \frac{1}{4{,}76461 * 10^8} * \left(81000 * 8{,}9089 * 10^5 + \frac{\pi^2 * 210000 * 2{,}24897 * 10^{12}}{1900^2}\right) =$$

$$= 2861{,}4MPa > 6 * f_y = 6 * 235 = 1410MPa$$

SATISFACTORY (fulfilment of one condition is sufficient)

Stiffener is assessed for normal force N_{Ed}=3265,6kN (combination for maximal shear force in box girder, traffic load multiplied by load capacity z_{LM71}=1,051):



- Flexural buckling:

Buckling length:

$L_{cr,z} = 0{,}75 * 2534 = 1900mm$

Non-dimensional slenderness:

$$\lambda_1 = \pi * \sqrt{\frac{E}{f_y}} = \pi * \sqrt{\frac{210000}{235}} = 93{,}913$$

$$\lambda' = \frac{L_{cr,z}}{i_y} * \frac{1}{\lambda_1} = \frac{1900}{138} * \frac{1}{93{,}913} = 0{,}147 \le 0{,}200 \rightarrow cross\ sectional\ check$$

$$\mathrm{N_{Rd}} = \mathrm{A} * \frac{\mathrm{f_{yk}}}{\gamma_{M1}} = 20492 * \frac{235}{1{,}1} * 10^{-3} = 4377{,}8kN$$

$$\mathrm{M_{y,el,Rd}} = \mathrm{w_{y,eff}} * \frac{\mathrm{f_{yk}}}{\gamma_{M1}} = 0{,}0019971 * \frac{235000}{1{,}1} = 426{,}7kNm$$

$$k_{yy} = c_{my} * \left(1 + 0{,}6 * \lambda' * \frac{N_{Ed}}{N_{Rd}}\right) = 1{,}0 * \left(1 + 0{,}6 * 0{,}147 * \frac{3284{,}7}{4377{,}8}\right) = 1{,}066$$

$$\frac{N_{Ed}}{N_{Rd}} + k_{yy} * \frac{N_{Ed} * e_y}{M_{y,el,Rd}} = \frac{3265{,}6}{4377{,}8} + 1{,}066 * \frac{3265{,}6 * 0{,}027}{426{,}7} = 0{,}966 \le 1$$

SATISFACTORY

4.7.5 Transverse stiffener of web

Verification of resistance to torsional buckling:

$I_P = 2{,}0170 * 10^8 + 1{,}8714 * 10^7 + 15816 * 12^2 = 2{,}22692 * 10^8 mm^4$

$I_t = 5{,}6401 * 10^5 mm^4$

$$\frac{I_t}{I_p} = \frac{5{,}6401 * 10^5}{2{,}22692 * 10^8} = 0{,}00253 > 5{,}3 * \frac{f_y}{E} = 5{,}3 * \frac{235}{210000} = 0{,}00593$$

NOT SATISFACTORY

$$\sigma_{cr} = \frac{1}{I_P} * \left(GI_t + \frac{\pi^2 * EI_w}{L_{cr,\omega}{}^2} \right) =$$

$$= \frac{1}{2,22692 * 10^8} * \left(81000 * 5,6401 * 10^5 + \frac{\pi^2 * 210000 * 5,54922 * 10^{11}}{1900^2} \right) =$$

$$= 1635,8MPa > 6 * f_y = 6 * 235 = 1410MPa$$

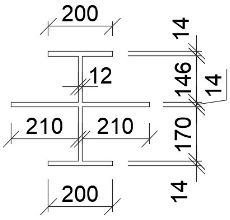
SATISFACTORY (fulfilment of one condition is sufficient)

Minimal second moment of area for intermediate stiffeners:

$$\frac{a}{h_w} = \frac{2200}{3420} = 0,643 < \sqrt{2} \rightarrow I_{st} = 2,0170 * 10^8 mm^4 \geq 1,5 * h_w^3 * \frac{t^3}{a^2} = 1,5 * 3420^3 * \frac{14^3}{2200^2}$$

$$= 3,4018 * 10^7 mm^4$$

SATISFACTORY



Buckling length:

$$L_{cr,z} = 0,75 * 2534 = 1900mm$$

Non-dimensional slenderness:

$$\lambda' = \frac{L_{cr,z}}{i_y} * \frac{1}{\lambda_1} = \frac{1900}{116} * \frac{1}{93,913} = 0,174 \leq 0,200 \rightarrow cross\ sectional\ check$$

$$\mathrm{N_{Rd}} = \mathrm{A} * \frac{\mathrm{f_{yk}}}{\gamma_{M1}} = 15816 * \frac{235}{1,1} * 10^{-3} = 3378,9kN$$

$$\mathrm{M_{y,el,Rd}} = \mathrm{w_{y,eff}} * \frac{\mathrm{f_{yk}}}{\gamma_{M1}} = 0,0011416 * \frac{235000}{1,1} = 243,9kNm$$

Stiffener is assessed for normal force calculated below (combination for maximal shear force in box girder, traffic load multiplied by load capacity z_{LM71}=1,051, V_{Ed}=3053,3kN is considered in distance 0,5h_w from support):

$$\alpha = \frac{a}{h_w} = \frac{2400}{3420} = 0,702$$

$$k_\tau = 4,00 + 5,34 * \left(\frac{h_w}{a}\right)^2 = 4,00 + 5,34 * \left(\frac{3420}{2400}\right)^2 = 14,843$$

$$\sigma_E = 190000 * \left(\frac{t}{b}\right)^2 = 190000 * \left(\frac{14}{3420}\right)^2 = 3,184MPa$$

$$\tau_{cr} = k_\tau * \sigma_E = 14,843 * 3,184 = 47,258MPa$$

$$\lambda_w = 0,76 * \sqrt{\frac{f_{yw}}{\tau_{cr}}} = 0,76 * \sqrt{\frac{235}{47,258}} = 1,695$$

$$N_{Ed} = V_{Ed} - \frac{1}{\lambda'^2_w} * \frac{f_{yw} * h_w * t}{\sqrt{3} * \gamma_{M1}} = 3053,3 - \frac{1}{1,695^2} * \frac{235 * 3420 * 14}{\sqrt{3} * 1,1 * 1000} = 997,7kN$$

$$k_{yy} = c_{my} * \left(1 + 0,6 * \lambda' * \frac{N_{Ed}}{N_{Rd}}\right) = 1,0 * \left(1 + 0,6 * 0,184 * \frac{997,7}{3378,9}\right) = 1,032$$

$$\frac{N_{Ed}}{N_{Rd}} + k_{yy} * \frac{N_{Ed} * e_y}{M_{y,el,Rd}} = \frac{997,7}{3378,9} + 1,032 * \frac{997,7 * 0,012}{243,9} = 0,346 \leq 1$$

SATISFACTORY

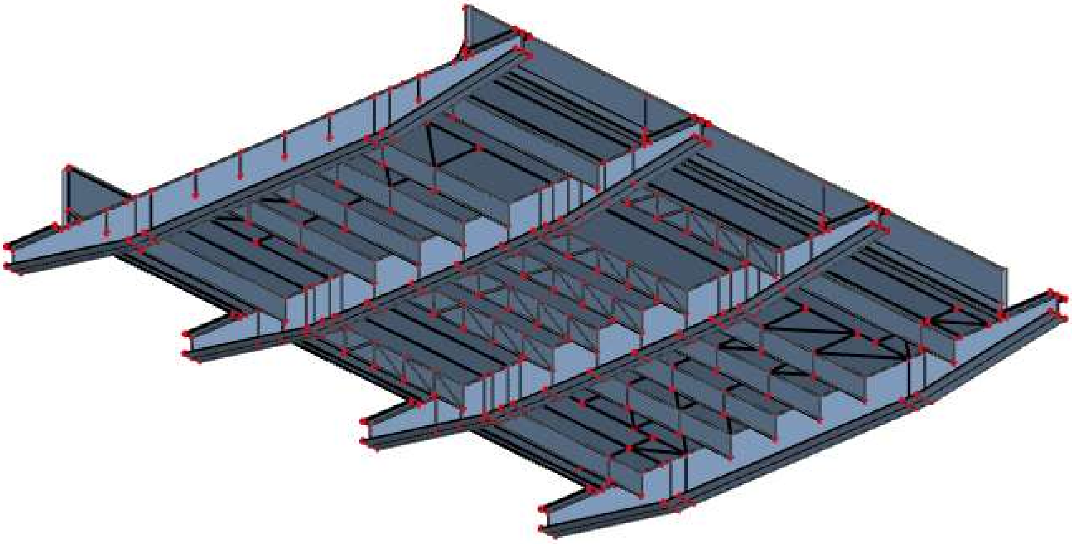
4.7.6 Non -linear analysis of behaviour of longitudinal stiffener in torsion

Longitudinal stiffeners of upper flange do not meet requirements for resistance to torsional buckling according to standard EN 1993-1-5. In this case it is necessary to verify a correctness of global buckling calculation in previous captures using non-linear calculation. There are considered following imperfections:

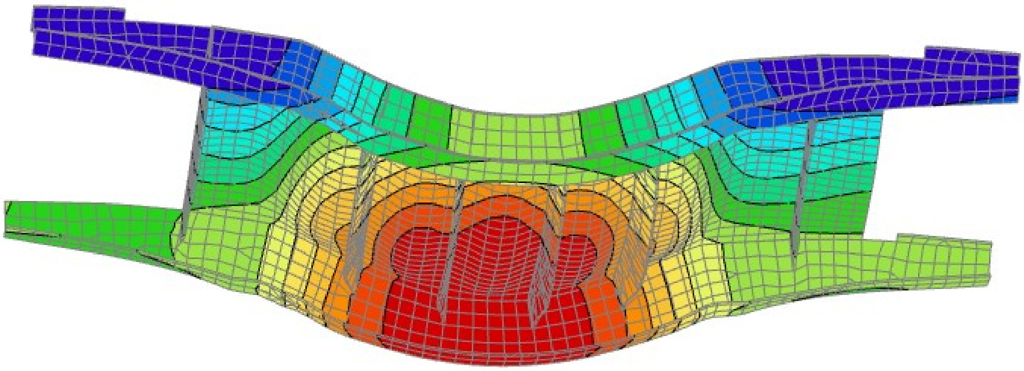
$$Inner\ panel\ of\ upper\ flange: e_0 = \frac{b}{200} = \frac{2800}{200} = 14mm\ (bow\ in\ shape\ of\ buckling)$$

$$Outside\ panel\ of\ upper\ flange: e_0 = \frac{b}{200} = \frac{810}{200} = 4mm\ (bow\ in\ shape\ of\ buckling)$$

$$Longitudinal\ stiffener: e_0 = \frac{1}{50} * h = \frac{1}{50} * 240 = 4{,}8mm\ (bow\ twist)$$



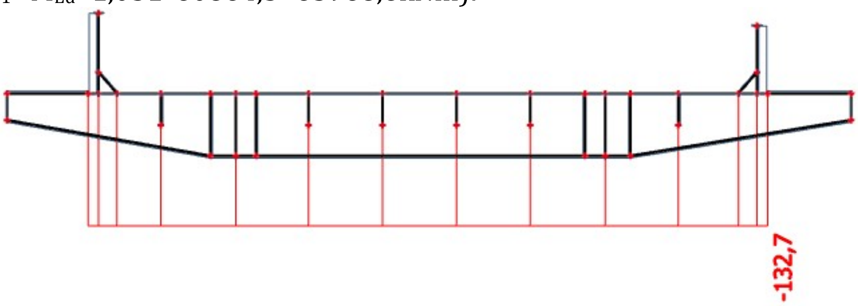
Picture 4.7-2 – part of model with imperfections for non-linear analysis



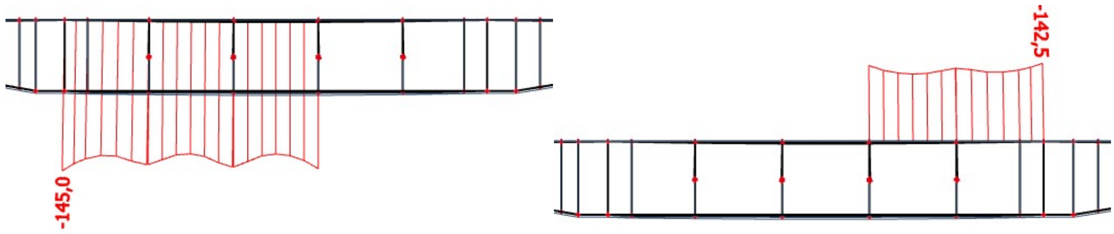
Picture 4.7-3 - shape of bending for implementation of imperfections

<u>Flange (thickness 18mm):</u>

Stress calculated by linear and non-linear calculation for the same bending moment ($M_{Ed,max}$=z_{LM71}* M_{Ed}=1,051*60864,5=63968,6kNm):



Picture 4.7-4 - uniformly distributed stress calculated by linear method



Picture 4.7-5 - stress diagram calculated by non-linear method – inner part of upper flange

In chapter 4.1.3 is considered buckling coefficient for flange ρ_C=0,897 for inner part of flange, which should be equal the ratio of stresses for linear and non-linear behaviour.

Comparison with non-linear analysis results:

$$\frac{\sigma_I}{\sigma_{II}} = \frac{132,7}{145,0} = 0,915 > \rho_c = 0,897$$

$$\rightarrow torsional\ deviation\ does\ not\ increase\ bukcling\ effect$$

<u>Longitudinal stiffener:</u>

In chapter 4.1.3 is considered buckling coefficient for longitudinal stiffener (including local and global buckling) ρ=0,897*0,858=0,770, which should be equal the ratio of stresses for linear and non-linear behaviour.

Picture 4.7-6 - average stress, linear behaviour (left) and stress diagram for non-linear behaviour (right)

$$\frac{\sigma_I}{\sigma_{II}} = \frac{118,2}{131,3} = 0,900 > \rho = 0,770$$

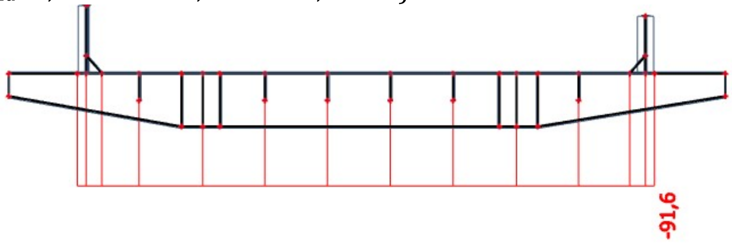
Comparison of effective area whole upper flange with longitudinal stiffeners:

- Effective area calculated in chapter 4.1.3:
  $A_{eff,1} = 103692mm^2$
- Effective area – non-linear calculation:
  $A_{eff,2} = 0,915 * (18 * 5150) + 0,900 * (6 * 16 * 240) = 105578mm^2$
- $A_{eff,1} < A_{eff,2} \rightarrow effective\ cross\ section\ is\ not\ necessary\ to\ modify$

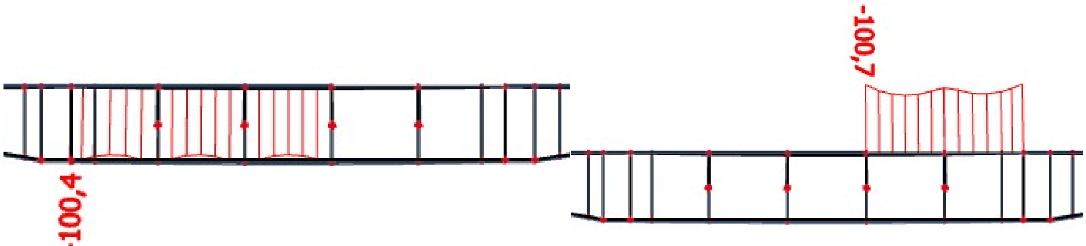
It is proved that torsional resistance of longitudinal stiffeners is sufficient and torsional buckling does not increase buckling effect. So calculations in capture 4.1.3 are considered valid.

<u>Flange (thickness 14mm):</u>

Stress calculated by linear and non-linear calculation for the same bending moment ($M_{Ed,max}$=z_{LM71}* M_{Ed}=1,051*36512,7=38374,8kNm):



Picture 4.7-7 - uniformly distributed stress calculated by linear method



Picture 4.7-8 - stress diagram calculated by non-linear method – inner part of upper flange

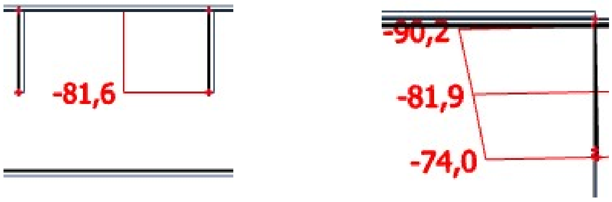
In chapter 4.1.4 is considered buckling coefficient for flange (including local and global buckling) ρ=0,912*0,992=0,904 for inner part of flange, which should be equal the ratio of stresses for linear and non-linear behaviour.

Comparison with non-linear analysis results:

$$\frac{\sigma_I}{\sigma_{II}} = \frac{91,6}{100,7} = 0,910 > \rho = 0,904 \rightarrow torsional\ deviation\ increase\ bukcling\ effect$$

<u>Longitudinal stiffener:</u>

In chapter 4.1.4 is considered buckling coefficient for longitudinal stiffener (including local and global buckling) ρ=0,912*0,862=0,786, which should be equal the ratio of stresses for linear and non-linear behaviour.



Picture 4.7-9 - average stress, linear behaviour (left) and stress diagram for non-linear behaviour (right)

$$\frac{\sigma_I}{\sigma_{II}} = \frac{81,6}{90,2} = 0,905 > \rho = 0,786$$

Comparison of effective area whole upper flange with longitudinal stiffeners:

- Effective area calculated in chapter 4.1.4:
  $A_{eff,1} = 85224mm^2$
- Effective area – non-linear calculation:
  $A_{eff,2} = 0,910 * (14 * 5150) + 0,905 * (6 * 16 * 240) = 86462mm^2$
- $A_{eff,1} < A_{eff,2} \rightarrow effective\ cross\ section\ is\ not\ necessary\ to\ modify$

It is proved that torsional resistance of longitudinal stiffeners is sufficient and torsional buckling does not increase buckling effect. So calculations in capture 4.1 are considered valid.

4.8 Comparison of preliminary a detailed assessment

<u>Percentage of use:</u>

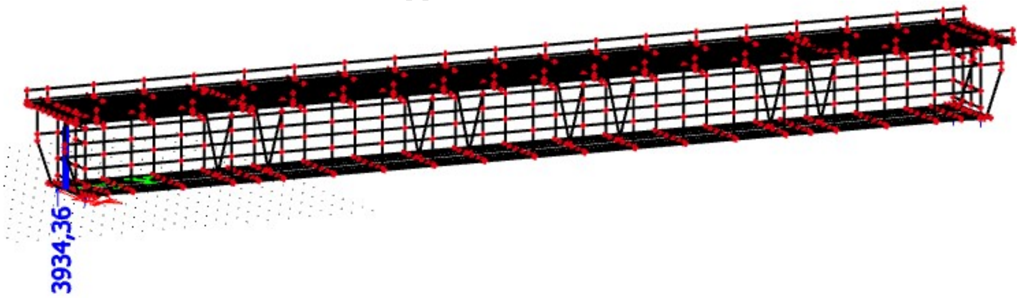
[%]	preliminary	detailed
Box girder - midspan	96,2	97,4
Box girder - L=7,8m	70,4	71,9
Cross girder	85,7	86,7

The results from preliminary assessment are verified. Percentage of use are in detailed assessment slightly higher, because for preliminary assessment were some loads neglected and there was not calculated with transverse distribution of load. For detailed assessment are also used different effective cross section for three ways of stress distribution.

4.9 Bearings and a combined response of structure and track

On support S1 is a pair of expansion bearings (type II-V-5) and on support S2 are placed fixed bearing (type II-P-5).

Maximal vertical reaction in support:



$F_{D,z} = 3934{,}4kN \leq 5000kN$

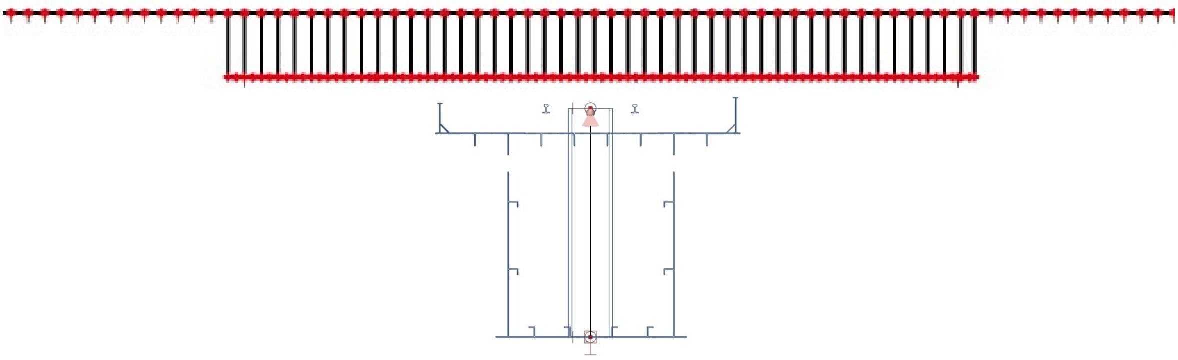
SATISFACTORY

Maximal transverse reaction in support:

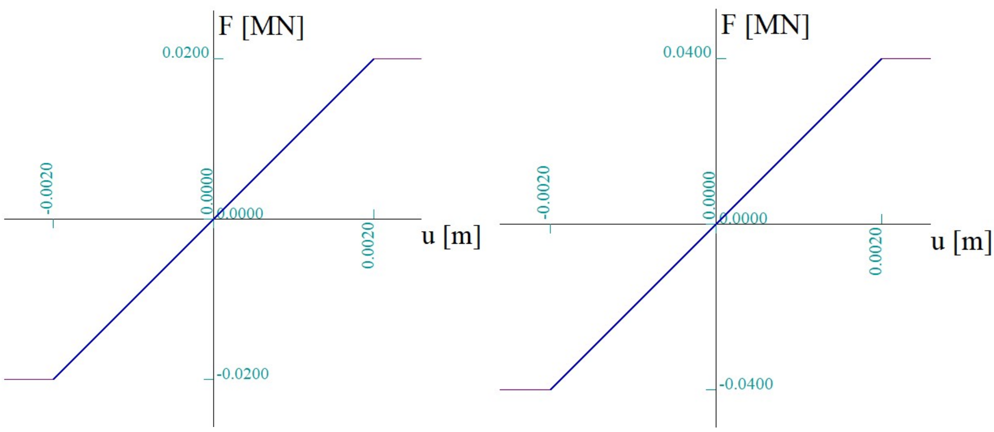
$F_{D,y} = 310{,}9kN \leq 400kN$

SATISFACTORY

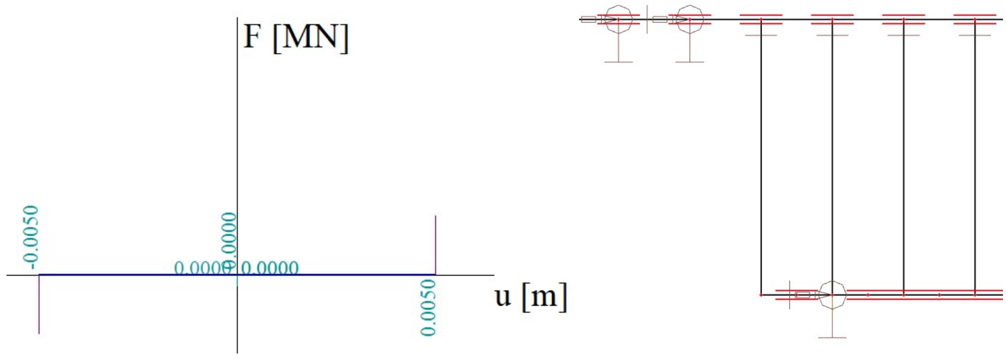
For longitudinal reaction is created a model, which takes into an account a combined response of structure and track for variable load. Constructions of bridge and rail is connected using imaginary beams of infinite stiffness. Rigidity of the connection is considered as shown below. Fixed bearing has longitudinal tolerance ±5mm, it is also taken in consideration using non-linear support in longitudinal direction. There is different resistance of track in ballast during summer and winter, when the ballast is frozen. However, a combination of force caused by deflection, warming and traction force is decisive.



Picture 4.9-1 - model for combined response of structure and track

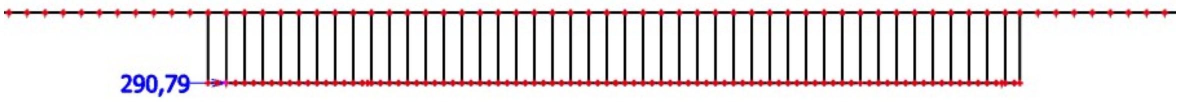


Picture 4.9-2 – stiffness of non-linear support in long. direction – not-loaded rail (left), loaded rail (right)

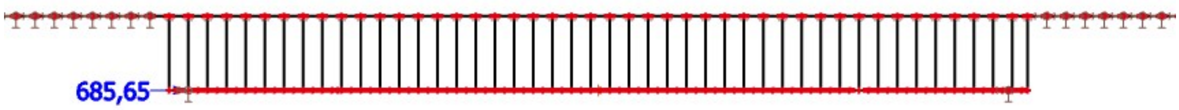


Picture 4.9-3 – stiffness of fixed bearing (left) and detail of non-linear supports (right)

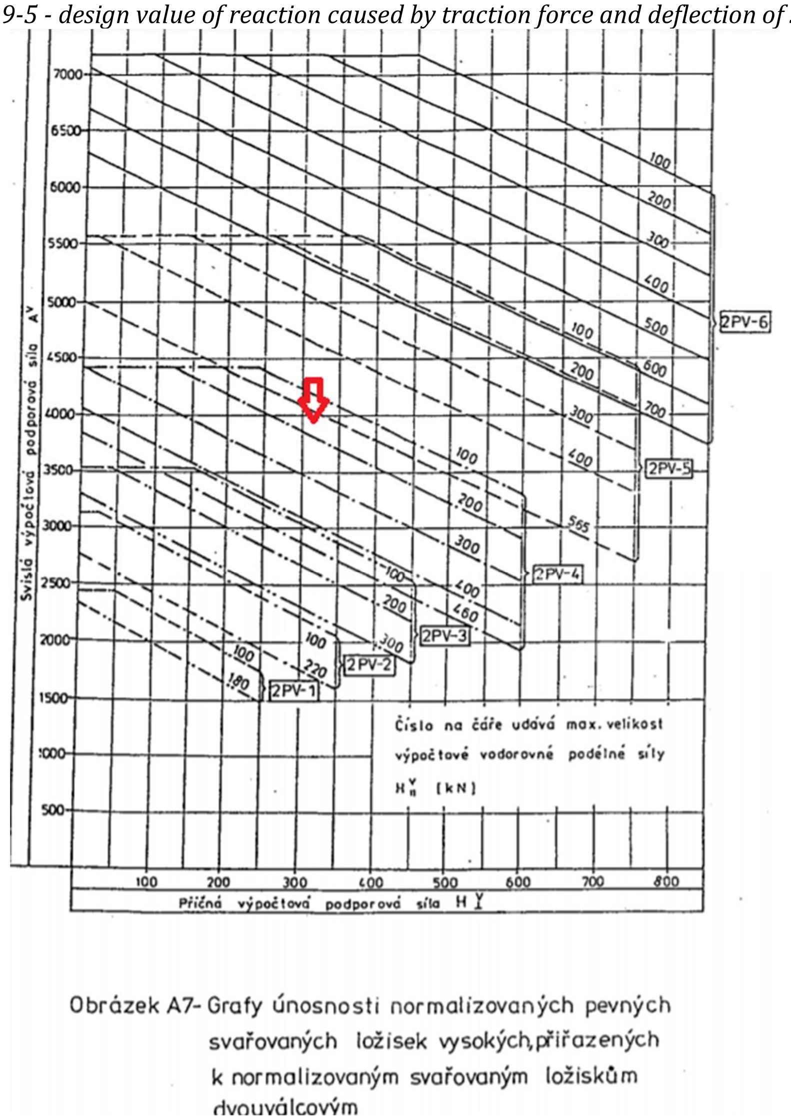
Maximal longitudinal reaction in support:



Picture 4.9-4 - design value of reaction caused by temperature load



Picture 4.9-5 - design value of reaction caused by traction force and deflection of structure

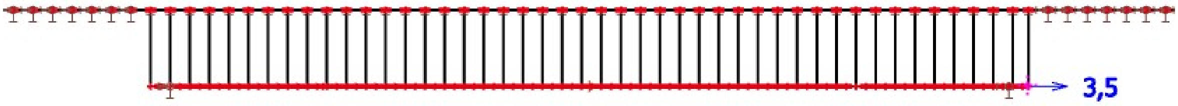


Picture 4.9-6 - design resistance of bearings in longitudinal direction

$$F_{D,x} = \frac{1}{2} * (290{,}79 + 685{,}65) = 488{,}22kN < 565kN$$

The longitudinal reaction in support of a bridge meets the requirements to load capacity of fixed bearing (type II-P-5).

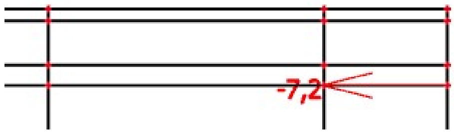
Verification of limit deformation:



Picture 4.9-7 - longitudinal deformation caused by traction force

$w_x = 3{,}5mm < w_{x,lim} = 5{,}0mm$

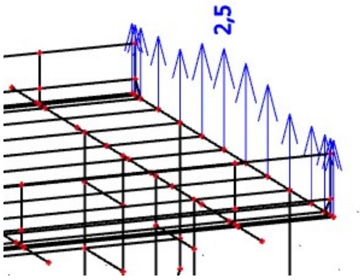
SATISFACTORY



Picture 4.9-8 – long. deformation caused by deflection from traffic load (side with expansion bearing)

$w_x = 7{,}2mm < w_{x,lim} = 10{,}0mm$

SATISFACTORY



Picture 4.9-9 – longitudinal deformation caused by deflection from traffic load (side with fixed bearing)

$w_x = 2{,}5mm < w_{x,lim} = 3{,}0mm$

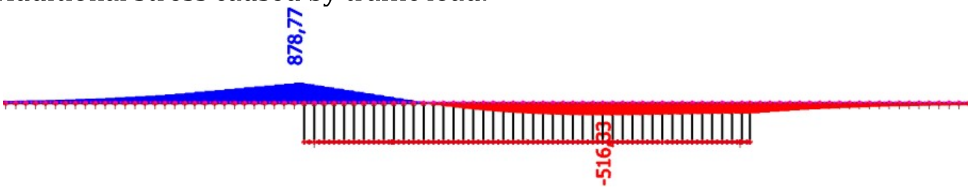
SATISFACTORY

Verification of permissible additional rail stress:

Additional stress caused by temperature:

$$\Delta\sigma_{max} = 19{,}298MPa, \Delta\sigma_{min} = 28{,}063MPa$$

Additional stress caused by traffic load:



Picture 4.9-10 - maximal additional normal force caused by traffic load

$$\Delta\sigma_{max} = 62{,}908MPa,\ \Delta\sigma_{min} = 40{,}435MPa$$

Total additional stress:

$\Delta\sigma_{max} = 19{,}298 + 62{,}908 = 82{,}206MPa \leq 92{,}0MPa$

$\Delta\sigma_{min} = 28{,}063 + 40{,}435 = 68{,}498MPa \leq 72{,}0MPa$

SATISFACTORY

Load capacity (additional stresses are determinative)

$$\eta_{1,\mathrm{rs}} = \frac{\Delta\sigma_{max,rs}}{\Delta\sigma_{lim}} = \frac{28{,}063}{72{,}0} = 0{,}390$$

$$\eta_{1,\mathrm{LM7}} = \frac{\Delta\sigma_{max,LM71}}{\Delta\sigma_{lim}} = \frac{40{,}435}{72{,}0} = 0{,}562$$

$$z_{\mathrm{LM71}} = \frac{1 - \eta_{1,\mathrm{rs}}}{\eta_{1,\mathrm{LM71}}} = \frac{1 - 0{,}390}{0{,}562} = \mathbf{1{,}087}$$

4.10 Summary of load capacity

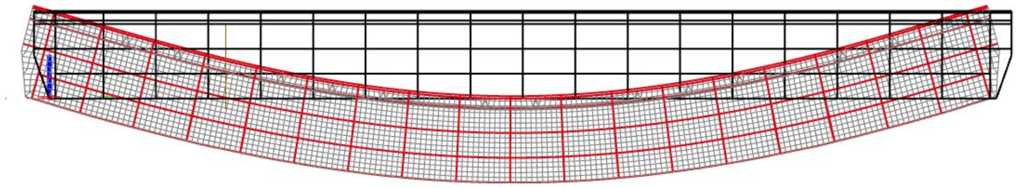
In a table below are summarized results of all parts of construction.

	ULS	fatigue (if determinative)
Box girder - midspan	**1,051**	-
Box girder - L=7,8m	1,658	-
Box girder - above support	1,298	-
Intermediate cross girder	1,175	1,130
End cross girder	1,486	-
Longitudinal stiffener	1,144	1,123
Stiffener inside box girder	8,051	-
Bridge deck	1,513	-
Bearings, combined response of structure and track	1,087	-
Serviceability limit state	1,567	

4.11 Compatibility of the interface between vehicle and infrastructure

The bridge is classified by line category C3-70. The construction is assessed for corresponding traffic load according to standard EN 15528.

Dynamic factor for real train:



Picture 4.11-1 - first natural bending frequency n_0=3,35

$$K = \frac{v}{2 * L_{\Phi} * n_0} = \frac{70/3,6}{2 * 42,8 * 3,35} = 0,06781$$

$$\varphi' = \frac{K}{1 - K + K^4} = \frac{0,06781}{1 - 0,06781 + 0,06781^4} = 0,07274$$

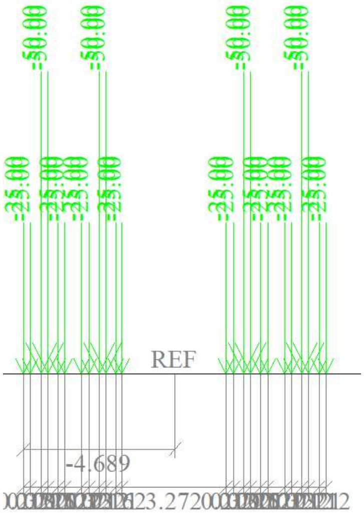
$$\alpha = \frac{v}{22} = \frac{70/3,6}{22} = 0,884$$

$$\varphi'' = \frac{\alpha}{100} * \left[56 * e^{-\left(\frac{L_{\Phi}}{10}\right)^2} + 50 * \left(\frac{L_{\Phi} * n_0}{80} - 1\right) * e^{-\left(\frac{L_{\Phi}}{20}\right)^2}\right] =$$
$$= \frac{0,884}{100} * \left[56 * e^{-\left(\frac{42,8}{10}\right)^2} + 50 * \left(\frac{42,8 * 3,35}{80} - 1\right) * e^{-\left(\frac{42,8}{20}\right)^2}\right] =$$
$$= 0,00359$$

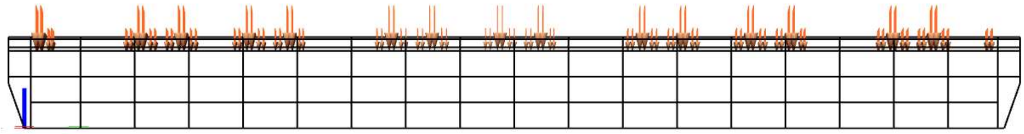
$$\Phi_{T1} = 1 + \varphi' + \varphi'' = 1 + 0,07274 + 0,00359 = 1,07633$$

$$\psi = \frac{\Phi_{T1}}{\Phi_3} = \frac{1{,}07633}{1{,}07100} = 1{,}005$$

For traffic load is supposed to be distributed in longitudinal direction on three neighbouring rail supports in ratio 1:2:1. It consists of four axle forces of value 200kN in length 11,1m, alternative continuous load is 72kN/m.



Picture 4.11-2 – traffic load for line category C3



Picture 4.11-3 - critical position of traffic load for maximal bending moment

Inner forces in midspan caused by traffic load LM71 load for line category C3:

$N_{Ed,LM71}$	189,3	kN	$N_{Ed,T}$	189,3	kN
$M_{y,Ed,LM71}$	29935,3	kNm	$M_{y,Ed,T}$	21160,7	kNm
$M_{z,Ed,LM71}$	3362,9	kNm	$M_{z,Ed,T}$	2656,3	kNm

$$\eta_{1,\mathrm{LM71}} = \frac{\mathrm{N_{Ed,LM71}}}{\mathrm{N_{Rd}}} + \frac{\mathrm{M_{y,Ed,LM71}} + \mathrm{N_{Ed,LM71}} * \mathrm{e_{y,N}}}{\mathrm{M_{y,el,Rd}}} + \frac{\mathrm{M_{z,Ed,LM71}} + \mathrm{N_{Ed,LM71}} * \mathrm{e_{z,N}}}{\mathrm{M_{z,el,Rd}}} = \frac{189{,}3}{60905{,}6} + \frac{29935{,}3 + 189{,}3 * 0{,}065}{71843{,}8} + \frac{3362{,}9 + 189{,}3 * 0{,}092}{49608{,}5} = 0{,}48809$$

$$\eta_{1,\mathrm{T}} = \frac{\mathrm{N_{Ed,T}}}{\mathrm{N_{Rd}}} + \frac{\mathrm{M_{y,Ed,T}} + \mathrm{N_{Ed,T}} * \mathrm{e_{y,N}}}{\mathrm{M_{y,el,Rd}}} + \frac{\mathrm{M_{z,Ed,T}} + \mathrm{N_{Ed,T}} * \mathrm{e_{z,N}}}{\mathrm{M_{z,el,Rd}}} = \frac{189{,}3}{60905{,}6} + \frac{21160{,}7 + 189{,}3 * 0{,}065}{71843{,}8} + \frac{2656{,}3 + 189{,}3 * 0{,}092}{49608{,}5} = 0{,}35171$$

$$\lambda_{LM71} = \frac{\eta_{1,\mathrm{T}}}{\eta_{1,\mathrm{LM71}}} = \frac{0{,}35171}{0{,}48809} = 0{,}721$$

$$Z_{LM71} = \mathbf{1{,}051} \geq \psi * \lambda_{LM71} = 1{,}005 * 0{,}721 = \mathbf{0{,}725}$$

<u>SATISFACTORY</u>

The calculation meets the requirement for line category C3.

5 Conclusion

Below is introduced list of load capacity according to railway regulation [17].

A. identification of the bridge:			
track section	0711 - Prague, Smíchov - Hostivice		
definition section	Prague, Stodůlky - Prage, Zličín		
B. identification of the part of bridge:			
load bearing steel structure			
C. additional information			
category of load capacity: C			
3D computational model in Scia Engineer program			
Geometry of track:			
	beginning	middle	end
radius [m]	434,726	660,744	886,762
cant [mm]	68	51	34
eccentricity [mm]	200	-200	200

No.	Element	Detail	Stress	type	L_P	Φ_i	L_Φ	$\gamma_{F,LM71}$	see side	note	z_{LM71}
1	main girder, midspan	upper fibres	normal	N+M	42,8	1,071	42,8	1,30	42,53	**ULS, deciding detail**	**1,051**
2	main girder, stationing L=7,8m	upper fibres	normal +shear	N+V+M	42,8	1,071	42,8	1,30	43,54	ULS	**1,658**
3	main girder, above supports	web	shear	V	42,8	1,071	42,8	1,30	45,54	ULS	**1,298**
4	main girder, midspan	upper fibres	normal	N+M	42,8	1,071	42,8	1,30	49	fatigue	**3,387**
5	main girder, above supports	upper fibre of web	shear flow	V	42,8	1,071	42,8	1,30	49	fatigue	**3,997**
6	main girder, web	bolted joint	normal	M	42,8	1,071	42,8	1,30	50	fatigue	**2,807**
7	main girder	welds of longitudinal stiffeners	normal	M	42,8	1,071	42,8	1,30	50	fatigue	**1,488**
8	main girder	welds of web and flanges	shear flow	V	42,8	1,071	42,8	1,30	51	fatigue	**2,742**
9	main girder	vertical weld in upper part	normal	M	42,8	1,071	42,8	1,30	51	fatigue	**1,816**
10	main girder	const. welds of lower flange	normal	M	42,8	1,071	42,8	1,30	52	fatigue	**2,975**
11	main girder	const. welds of upper flange	normal	M	42,8	1,071	42,8	1,30	52	fatigue	**2,536**
12	main girder, midspan	vertical deformation	deformation	w	42,8	1,071	42,8	1,00	61	SLS	**1,967**
13	main girder, above supports	roation	rotation	Φ	42,8	1,071	42,8	1,00	61	SLS	**1,567**
14	cross girder	above web of main girder	shear	V	2,8	1,727	5,6	1,30	64,66	ULS	**1,175**

15	end cross girder	above web of main girder	shear	V	2,8	2,000	3,6	1,30	74,76	ULS	**1,486**
16	cross girder	upper fibres, midspan	normal	M	2,8	1,727	5,6	1,30	70	fatigue	**1,615**
17	cross girder	upper fibre of web	shear flow	V	2,8	1,727	5,6	1,30	70	fatigue	**1,261**
18	cross girder	bolted joint	normal	M	2,8	1,727	5,6	1,30	71	fatigue	**1,183**
19	cross girder	const. welds of lower flange	normal	M	2,8	1,727	5,6	1,30	71	fatigue	**1,130**
20	cross girder	const. welds of upper flange	normal	M	2,8	1,727	5,6	1,30	72	fatigue	**1,501**
21	cross girder	longitudinal welds	normal	M	2,8	1,727	5,6	1,30	72	fatigue	**1,463**
22	cross girder	connection of long. stringer	normal	V+M	2,8	1,727	5,6	1,30	73	fatigue	**1,400**
23	longitudinal stiffener	upper fibre	normal	N+M	2,4	1,600	7,2	1,30	80,86	ULS	**1,144**
24	longitudinal stiffener	lower fibre	normal	N+M	2,4	1,600	7,2	1,30	81,86	ULS	**1,293**
25	longitudinal stiffener	lower fibres, midspan	normal	N+M	2,4	1,600	7,2	1,30	84	fatigue	**1,438**
26	longitudinal stiffener	longitudal welds	normal	N+M	2,4	1,600	7,2	1,30	85	fatigue	**1,123**
27	longitudinal stiffener	upper fibre of web	shear flow	V	2,4	1,600	7,2	1,30	85	fatigue	**3,659**
28	longitudinal stiffener	load bearing weld	shear	V	2,4	1,600	7,2	1,30	85	fatigue	**3,306**
29	stiffener inside box girder	buckling	normal	N	3,108	1,071	42,8	1,30	87	ULS	**8,051**
30	bridge deck plate	see picture 4.6-4	maximal stress	N+M	2,4	1,600	7,2	1,30	89	ULS	**1,513**
31	track - additional stress	combined response	normal	N	42,8	1,071	42,8	1,30	99	SLS	**1,087**

Type: N=normal force, M=bending moment, V=shear force, w=deflection, Φ=rotation.

Critical detail for load capacity is upper fibre of main girder in midspan, where load capacity is calculated for normal stress and its value is 1,051.

All parts of steel structure of a bridge are suitable for the LM71 traffic load and load category C3-70.

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List of pictures

CZECH TECHNICAL UNIVERSITY IN PRAGUE
Faculty of Civil Engineering
Department of Steel and Timber Structures

The assessment and the load capacity of the bridge in Prague - Motol

Master's thesis

B. Preliminary static analysis

Kateřina Soukupová

Prague 2018

Contents

1 Introduction

This attachment of thesis was written as part of the subject diploma seminar (134DISE). The results of this preliminary static analysis are compared to detailed analysis in A.2, chapter 4.8.

2 Action on structure

2.1 Permanent loads

2.1.1 Self-weight of the structure

The self-weight is calculated from the dimensions shown in the drawings.

- self-weight of steel structure (including box girder, inner maintenance walkway, continuous longitudinal ribs)

$$g_{k1} = 29{,}26kN/\text{m}$$

- self-weight of cross girders and stiffeners placed in distances 2,4m

$$F_{k1} = 9{,}92kN$$

2.1.2 Other permanent loads

- protective cement screed with liner and insulation, total thickness 5 cm

$$g_{k2} = 5{,}6 * 0{,}05 * 24 = 6{,}72kN/m$$

- cable trays and cables

$$g_{k3} = 1{,}60kN/m$$

- track ballast, thickness 400mm

$$g_{k4} = 5{,}0 * 0{,}4 * 20 = 40{,}00kN/m$$

- rails and fasteners

$$g_{k5} = 1{,}80kN/m$$

- railing

$$g_{k5} = 1{,}00kN/m$$

Total permanent loads:

$$g_{k,\text{g}} = 29{,}26 + 6{,}72 + 1{,}6 + 40 + 1{,}8 + 4 * 1{,}00 + 9{,}92/2{,}4 = 87{,}51kN/m$$

For permanent load are considered these deviation:

$$g_{k2,\text{max}} = 6{,}72 * 1{,}4 = 9{,}41kN/m$$
$$g_{k2,\text{min}} = 6{,}72 * 0{,}8 = 5{,}38kN/m$$
$$g_{k3,\text{max}} = 1{,}60 * 1{,}2 = 1{,}92kN/m$$
$$g_{k3,\text{max}} = 1{,}60 * 0{,}8 = 1{,}28kN/m$$
$$g_{k4,\text{max}} = 40{,}00 * 1{,}3 = 52{,}00kN/m$$
$$g_{k4,\text{min}} = 40{,}00 * 0{,}7 = 28{,}00kN/m$$
$$g_{k,\text{g,max}} = 87{,}51 + 2{,}67 + 0{,}32 + 12{,}00 = 102{,}50kN/m$$
$$g_{k,\text{g,min}} = 87{,}51 - 1{,}34 - 0{,}32 - 12{,}00 = 72{,}52kN/m$$

Partial factor for permanent loads: $\gamma_{\text{FG}} = 1{,}25$ (coefficient for steel member, without any control, older than 30 years).

2.2 Variable loads

2.2.1 Traffic loads

For traffic loads is used the load model LM71 and load classification factor α=1,00. The load is placed in the most unfavourable position for each element, alleviating effects are being neglected.

Partial factor for traffic loads is $\gamma_{\text{Q,LM71}} = 1{,}30$ (coefficient for steel member, without any control, older than 30 years).

2.2.2 Dynamic factors

Dynamic factor for track with standard maintenance:

$$\phi_3 = \frac{2,16}{\sqrt{L_\phi} - 0,2} + 0,73 = \frac{2,16}{\sqrt{42,8} - 0,2} + 0,73 = 1,071$$

Cross girders:

$$L_\phi = 2 * 2,800 = 5,600m$$

$$\phi_3 = \frac{2,16}{\sqrt{5,600} - 0,2} + 0,73 = 1,727$$

2.2.3 Centrifugal forces

For the calculation is used a line speed 70 km/h. The force is horizontal and perpendicular to the track axis at a height of 1,8 m above the rail. It is combined with the appropriate vertical load. The radius of curvature is at one end of the bridge R_1 = 886,762m and the second R_2 = 434,726m. The mean radius r = 660,744m is considered. Load classification factor is α=1,00 and dynamic factor is not used for centrifugal forces. Partial factor for centrifugal forces is $\gamma_{\mathrm{Q,LM71}} = 1{,}30$.

$$Q_{tk} = \frac{V^2}{127 * r} * (f * Q_{vk}) = \frac{70^2}{127 * 660,744} * (1 * 250) = 14,60kN$$

$$q_{tk} = \frac{V^2}{127 * r} * (f * q_{vk}) = \frac{70^2}{127 * 660,744} * (1 * 80) = 4,67kN/m$$

2.2.4 Nosing force

As nosing force is considered a concentrated force Q_{sk}=100kN acting horizontally, at the top of the rails, perpendicular to the centre-line of track.

2.2.5 Actions due to traction and braking

This load act at the top of the rails in the longitudinal direction. It is uniformly distributed over the corresponding influence length.

Traction force:

$$Q_{lak} = 33 * L_{ab} = 33 * 42,8 = 1412,4kN \geq 1000kN \rightarrow Q_{lak} = 1000kN$$

Braking force:

$$Q_{lbk} = 20 * L_{ab} = 20 * 42,8 = 856kN \leq 6000kN$$

For a continuous track on the bridge is considered, that 60% of the traction force is transmitted by the bearings and the load-bearing steel structure.

2.2.6 Actions for non-public footpaths

Partial factor for loads of footpaths is $\gamma_{\mathrm{Q,fp}} = 1{,}50$.

Load of inner maintenance walkway:

$$q_{fp,1} = 2,0 * 0,8 = 1,64kN/m$$

Load of upper non-public footpaths:

$$q_{fp,1} = 5,0 * 0,7 = 3,50kN/m$$

2.2.7 Wind actions

The height of the bridge construction is 4,100m, the height of an open parapet is considered 0,6m high and the train height for the calculation is 4m. Partial factor for wind actions is $\gamma_{\mathrm{Q,w}} = 1{,}35$.

- density of air
  $\rho = 1,25kg/m^3$
- the reference height under terrain
  z=7,086m

- terrain factor
  $k_r = 0{,}19 * \left(\frac{z_0}{z_{0II}}\right)^{0{,}7} = 0{,}19 * \left(\frac{0{,}05}{0{,}05}\right)^{0{,}7} = 0{,}19$
- roughness coefficient (terrain category II)
  $c_r(z) = k_r * ln\frac{z}{z_0} = 0{,}19 * ln\frac{7{,}086}{0{,}05} = 0{,}9412$
- wind area 2 … $v_{b,0}$=25m/s
- directional factor und season factor
  $c_{dir} = c_{season} = 1{,}0$
- basic wind velocity
  $v_b = c_{dir} * c_{season} * v_{b,0} = 1{,}0 * 1{,}0 * 25 = 25m/s$
- orography factor
  $c_o(z) = 1{,}0$
- turbulence factor
  $k_I = 1{,}0$
- mean wind velocity
  $v_m(z) = c_r(z) * c_o(z) * v_b = 0{,}9412 * 1{,}0 * 25 = 23{,}53m/s$
- turbulence intensity
  $I_v(z) = \frac{k_I}{c_0(z) * \ln\left(\frac{z}{z_0}\right)} = \frac{1{,}0}{1{,}0 * \ln\left(\frac{7{,}086}{0{,}05}\right)} = 0{,}202$
- peak velocity pressure
  $q_{wk} = (1 + 7 * I_v(z)) * \frac{1}{2} * \rho * {v_m}^2 == (1 + 7 * 0{,}202) * \frac{1}{2} * 1{,}25 * 23{,}53^2 * 10^{-3} = 0{,}835kN/m^2$
- wind actions without train
  $\frac{b}{d_{tot}} = \frac{6410}{4700} = 1{,}364 \rightarrow c_{f,x} = 2{,}155$
  $q_{wk1} = c_{f,x} * q_{wk} = 2{,}155 * 0{,}835 * 4{,}7 = 8{,}457kN/m$
- wind actions on the bridge and train
  $\frac{b}{d_{to}} = \frac{6410}{8100} = 0{,}791 \rightarrow c_{f,x} = 2{,}341$
  $q_{wk} = c_{f,x} * q_{wk} = 2{,}341 * 0{,}835 * 8{,}1 = 15{,}833kN/m$

2.2.8 Thermal actions

The steel parts of the bridge are grouped as Type 1. Thermal actions consist of a uniform temperature component and a temperature difference component. Partial factor for thermal actions is $\gamma_{\mathrm{F,t}} = 1{,}5$.

Uniform temperature component:

- T_{max}=40°C … maximal shade air temperature
- T_{min}= -32°C … minimal shade air temperature
- T_0=10°C … initial bridge temperature
- maximal uniform bridge temperature component
  $$T_{e,max} = T_{max} + 16 = 40 + 16 = 56℃$$
- minimal uniform bridge temperature component
  $$T_{e,min} = T_{min} - 3 = -32 - 3 = -35℃$$
- maximum expansion range of the uniform bridge temperature component
  $$\Delta T_{N,exp} = T_{e,max} - T_0 = 56 - 10 = 46℃$$
- maximum contraction range of the uniform bridge temperature component
  $$\Delta T_{N,con} = T_0 - T_{e,min} = 10 - (-35) = 45℃$$

Temperature difference linear component:

- $\Delta T_{m,heat}$=18*0,6=11°C - temperature difference for top warmer than bottom
- $\Delta T_{m,cool}$=13*1,4=18°C - temperature difference for bottom warmer than top

Simultaneity of both components:

$\Delta T_{m,cool} + \omega_n * \Delta T_{N,exp} = 18 + 0{,}35 * 46 = 34{,}1°C$

$\omega_m * \Delta T_{m,cool} + \Delta T_{N,exp} = 0{,}75 * 18 + 46 = \mathbf{59{,}5}°C$

$\Delta T_{m,cool} + \omega_n * \Delta T_{N,con} = 18 + 0{,}35 * 45 = 33{,}8°C$

$\omega_m * \Delta T_{m,cool} + \Delta T_{N,con} = 0{,}75 * 18 + 45 = \mathbf{58{,}5}°C$

- For expansion is considered a temperature +59,5°C, for contraction -58,5°C.

2.3 Combination of actions

- For a global assessment of the girder are used following combination of actions. The main variable loads is traffic.

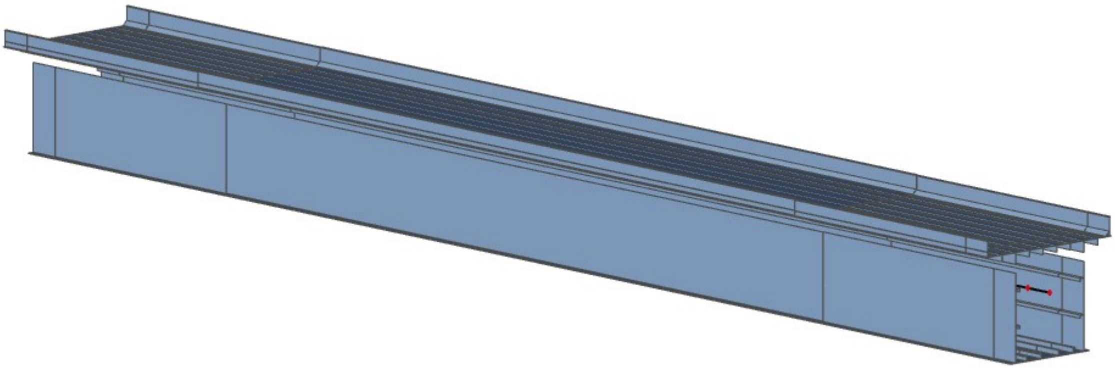
Name	Description	Type	Load cases	Coeff. [-]
10a		Envelope - ultimate	permanent load	1,06
			traction force	1,30
			nosing force	1,30
			maintanance load	1,20
			wind with train	1,01
			teperature expansion	0,90
			teperature contraction	0,90
			UL-LM71-Min Vz	1,39
			UL-LM71-Min My	1,39
			UL-LM71-Max Vz	1,39
			UL-LM71-Max My	1,39
			UL1-LM71-Min Vz	1,39
			UL1-LM71-Min My	1,39
			UL1-LM71-Max Vz	1,39
			UL1-LM71-Max My	1,39
10b		Envelope - ultimate	permanent load	1,25
			traction force	1,04
			nosing force	1,04
			maintanance load	1,20
			wind with train	1,01
			teperature expansion	0,90
			teperature contraction	0,90
			UL-LM71-Min Vz	1,11
			UL-LM71-Min My	1,11
			UL-LM71-Max Vz	1,11
			UL-LM71-Max My	1,11
			UL1-LM71-Min Vz	1,11
			UL1-LM71-Min My	1,11
			UL1-LM71-Max Vz	1,11
			UL1-LM71-Max My	1,11

- Combination for assessment of cross girder

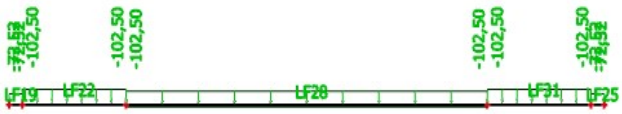
10a		Envelope - ultimate	self-weight autogenerated	1,06
			self-weight plate	1,06
			other permanent load	1,06
			maintanance load	1,20
			LM71	2,40
			temperature expansion	0,90
			temperature contraction	0,90
			nosing force	1,30
			centrifugal force	1,30
10b		Envelope - ultimate	self-weight autogenerated	1,25
			self-weight plate	1,25
			other permanent load	1,25
			maintanance load	1,20
			LM71	1,80
			temperature expansion	0,90
			temperature contraction	0,90
			nosing force	1,04
			centrifugal force	1,04

3 Computational model

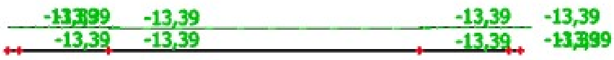
For preliminary assessment is the construction simulated as a simple beam. Permanent load, maintenance load, traction force and wind actions are placed as continuous uniformly distributed load with appropriate eccentricities. Temperature expansion and contraction is considered according to the calculation.



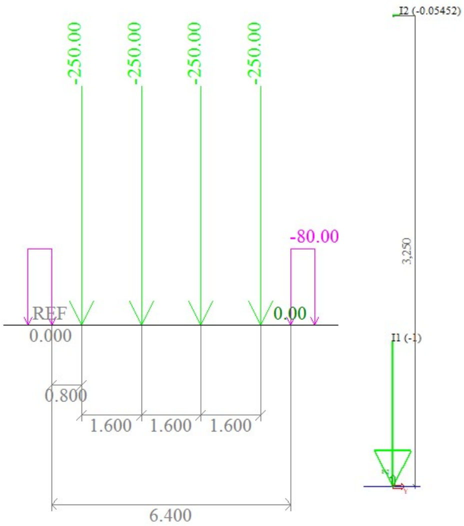
Picture 2.3-1 – 2D model of a box girder with effective cross sections



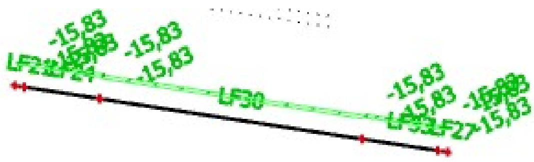
Picture 2.3-2 - permanent load



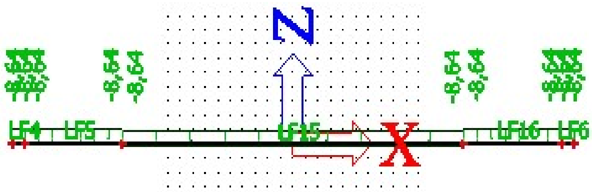
Picture 2.3-3 - traction force



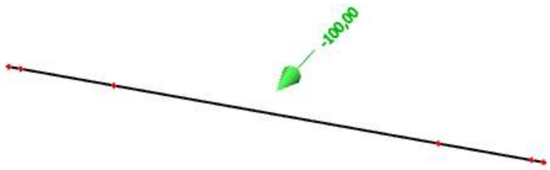
Picture 2.3-4 - traffic load



Picture 2.3-5 - wind actions

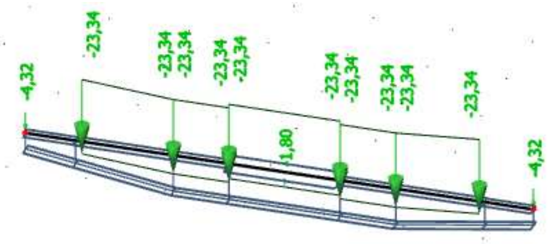


Picture 2.3-6 - maintenance load

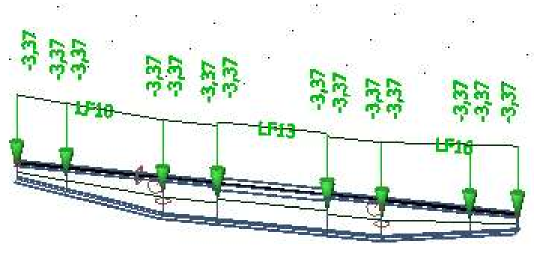


Picture 2.3-7 – nosing force in midspan

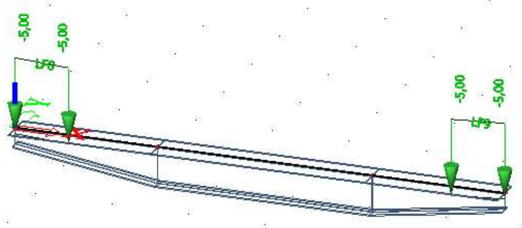
The cross girder is in preliminary assessment modelled as a simple beam with overhanging endings. It is loaded as bellow:



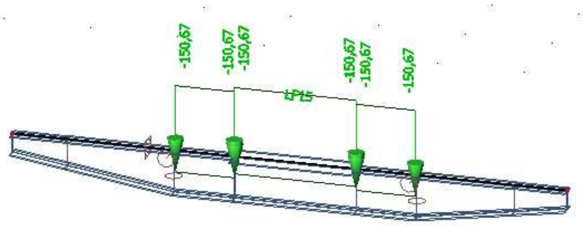
Picture 2.3-8 - other permanent load



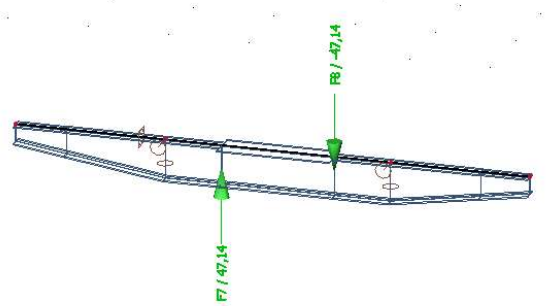
Picture 2.3-9 - self-weight - plate of bridge deck



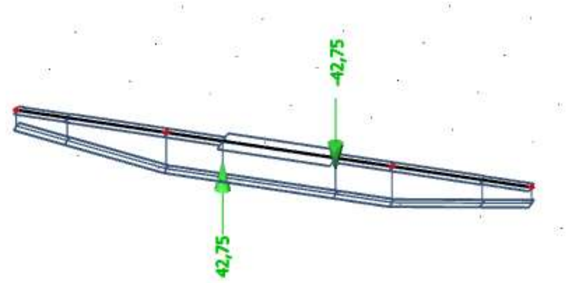
Picture 2.3-10 - maintenance load



Picture 2.3-11 - traffic load LM71



Picture 2.3-13 - nosing force



Picture 2.3-12 - centrifugal force

Forces caused by nosing force:

$$F_{k,\mathrm{NF}} = 100 * 0{,}66/1{,}4 = 47{,}14\mathrm{kN}$$

The uniformly distributed traffic load is calculated below, a supposed value of reaction for one wheel force is 208,3kN (according to static tables).

$$g_{k,\mathrm{LM71}} = 208{,}3 * 2/2{,}765 = 150{,}67\mathrm{kN/m}$$

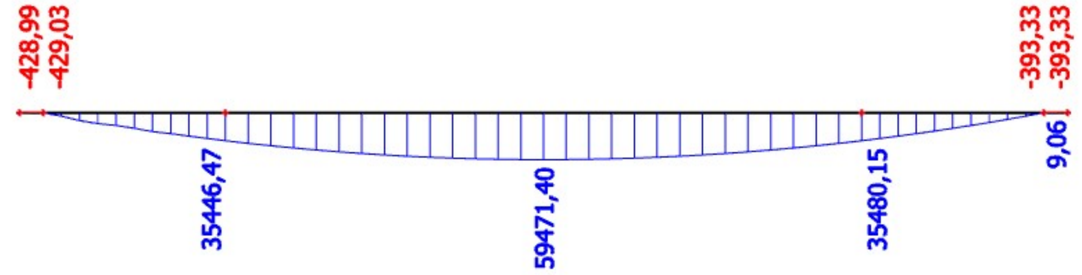
Forces caused by centrifugal force:

$$F_{k,\mathrm{CF}} = (208{,}3 * 2 * 0{,}058393) * 2{,}46/1{,}4 = 42{,}75\mathrm{kN}$$

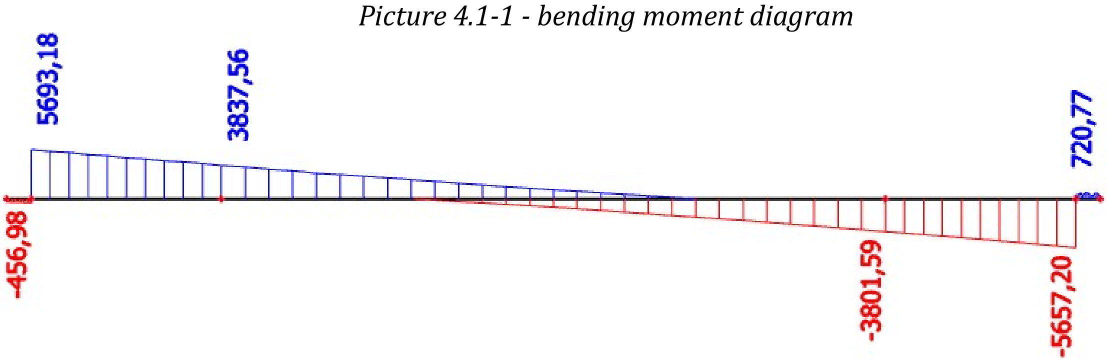
4 Preliminary static analysis

4.1 Main box girder

4.1.1 Inner forces



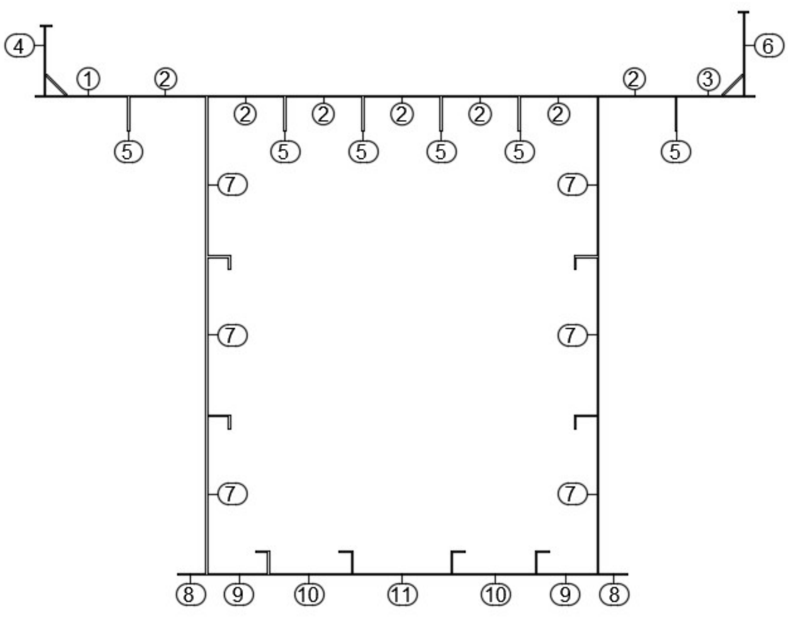
Picture 4.1-1 - bending moment diagram



Picture 4.1-2 - shear force diagram

4.1.2 Mid-span cross section

Dimensions of individual parts of cross sections are found out in an iterative manner. These dimensions depend on a real stress diagram in computational model. It takes into an account the local and global buckling of compression areas of the cross section and the shear lag of the flanges.



Picture 4.1-3 - numbers of subpanels

4.1.2.1 Local buckling and shear lag

- subpanel no. 1

Ratio of compressive and tensile stress:

$\psi = 1$

Buckling factor:

$k_\sigma = 4$

Plate slenderness for buckling:

$$\lambda_P = \frac{\frac{b}{t}}{28{,}4 * \varepsilon * \sqrt{k_\sigma}} = \frac{\frac{441}{18}}{28{,}4 * 1{,}0 * \sqrt{4}} = 0{,}431$$

Reduction factor:

$\rho = 1{,}000$

Effective width:

$b_{eff} = \rho * b = 1{,}000 * 441 = 441mm$

Upper flange - buckling and shear lag

subpanel no. 1			subpanel no. 2			subpanel no. 3			subpanel no. 4			subpanel no. 5		
ψ=	1		ψ=	1		ψ=	1		ψ=	1		ψ=	1	
k_σ=	4,000		k_σ=	4,000		k_σ=	4,000		k_σ=	4,000		k_σ=	4,000	
b=	441	mm	b=	545	mm	b=	544	mm	b=	545	mm	b=	331	mm
t=	18	mm	t=	18	mm	t=	18	mm	t=	18	mm	t=	18	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,431		λ_P=	0,533		λ_P=	0,532		λ_P=	0,533		λ_P=	0,324	
ρ=	1,000		ρ=	1,000		ρ=	1,000		ρ=	1,000		ρ=	1,000	
b_{eff}=	***441***	***mm***	***b_{eff}=***	***545***	***mm***	***b_{eff}=***	***544***	***mm***	***b_{eff}=***	***545***	***mm***	***b_{eff}=***	***331***	***mm***
			α0=	1,000		α0=	1,000					α0=	1,000	
			b0=	1862	mm	b0=	1400	mm				b0=	1732	mm
			Le=	42800	mm	Le=	42800	mm				Le=	42800	mm
			κ=	0,044		κ=	0,033					κ=	0,040	
			β=	0,988		β=	0,993					β=	0,990	
			teff=	**17,8**	**mm**	**teff=**	**17,9**	**mm**				**teff=**	**17,8**	**mm**
b_{e1}=	221	mm	b_{e1}=	273	mm	b_{e1}=	272	mm	b_{e1}=	273	mm	b_{e1}=	166	mm

Web - buckling

subpanel no. 6			subpanel no. 7			subpanel no. 8			subpanel no. 9			subpanel no. 10		
ψ=	0,75		ψ=	0,88		ψ=	0,77		ψ=	0,25		ψ=	-1,72	
k_σ=	4,556		k_σ=	0,474		k_σ=	4,505		k_σ=	6,308		k_σ=	44,242	
b=	351	mm	b=	240	mm	b=	446	mm	b=	1140	mm	b=	1126	mm
t=	12	mm	t=	16	mm	t=	12	mm	t=	14	mm	b_c=	419	mm
f_y=	235	MPa	f_y=	355	MPa	f_y=	235	MPa	f_y=	235	MPa	t=	14	mm
ε=	1,000		ε=	0,814		ε=	1,000		ε=	1,000		f_y=	235	MPa
λ_P=	0,483		λ_P=	0,943		λ_P=	0,617		λ_P=	1,142		ε=	1,000	
ρ=	1,000		ρ=	0,849		ρ=	1,000		ρ=	0,739		λ_P=	0,426	
b_{eff}=	***351***	**mm**	***b_{eff}=***	**204**	**mm**	***b_{eff}=***	***446***	**mm**	***b_{eff}=***	**842**	**mm**	ρ=	1,000	
b_{e1}=	165	mm				b_{e1}=	211	mm	b_{e1}=	355	mm	***b_{eff}=***	**419**	**mm**
b_{e2}=	186	mm				b_{e2}=	235	mm	b_{e2}=	488	mm	b_{e1}=	168	mm
												b_{e2}=	251	mm

Lower flange - shear lag

t=	25	mm
α0=	1,000	
b0=	1400	mm
Le=	42800	mm
κ=	0,033	
β=	0,993	
teff=	**24,8**	**mm**

4.1.2.2 Global buckling

- global buckling– inner part of box girder – plate type behaviour

$\alpha = \frac{a}{b} = \frac{2388}{2786} = 0{,}857 \leq \sqrt[4]{\gamma} = \sqrt[4]{181{,}970} = 3{,}673$

$\delta = \frac{\sum A_{sl}}{A_p} = \frac{15360}{50148} = 0{,}306$

$\gamma = \frac{I_{sl}}{I_p} = \frac{270754550}{1487908} = 181{,}970$

$\psi = 1$

$k_\sigma = \frac{2 * ((1 + \alpha^2)^2 + \gamma - 1)}{\alpha^2 * (\psi + 1) * (1 + \delta)} = \frac{2 * ((1 + 0{,}857^2)^2 + 181{,}970 - 1)}{0{,}857^2 * (1 + 1) * (1 + 0{,}306)} = 191{,}7$

$\sigma_E = 190000 * \left(\frac{t}{b}\right)^2 = 190000 * \left(\frac{18}{2786}\right)^2 = 7{,}931 MPa$

$\sigma_{cr,p} = k_\sigma * \sigma_E = 191{,}7 * 7{,}931 = 1520{,}4 MPa$

$\beta_{a,c} = \frac{A_{c,eff,loc}}{A_c} = \frac{53086}{55680} = 0{,}953$

$\lambda_P = \sqrt{\frac{\beta_{a,c} * f_y}{\sigma_{cr,p}}} = \sqrt{\frac{0{,}953 * 235}{1520{,}4}} = 0{,}384$

$\rho = 1{,}000$

- global buckling– inner part of box girder – column type behaviour

$\sigma_{cr,sl} = \sigma_{cr,c} = \frac{\pi^2 * E * I_{sl,1}}{A_{sl,1} * a^2} = \frac{\pi^2 * 210000 * 64832595}{13920 * 2388^2} = 1692{,}8 MPa$

$\lambda_c = \sqrt{\frac{\beta_{a,c} * f_y}{\sigma_{cr,c}}} = \sqrt{\frac{0{,}953 * 235}{1692{,}8}} = 0{,}364$

$\alpha_e = \alpha + \frac{0{,}09}{i/e} = 0{,}49 + \frac{0{,}09}{68{,}2/93{,}5} = 0{,}613$

$\emptyset = 0{,}5 * (1 + \alpha_e * (\lambda_c - 0{,}2) + {\lambda_c}^2) = 0{,}5 * (1 + 0{,}613 * (0{,}364 - 0{,}2) + 0{,}364^2) = 0{,}616$

$\chi_c = \frac{1}{\emptyset + \sqrt{\emptyset^2 - {\lambda_c}^2}} = \frac{1}{0{,}616 + \sqrt{0{,}616^2 - 0{,}364^2}} = 0{,}898$

- global buckling– inner part of box girder – interaction

$\xi = 0$

$\rho_c = (\rho - \chi_c) * \xi * (2 - \xi) + \chi_c = (1{,}000 - 0{,}898) * 0 * (2 - 0) + 0{,}898 = \mathbf{0{,}898}$

A thickness of the flange is modified according to formula $A_{c,eff} = \rho_c * A_{c,eff,loc}$. So the reduced thickness of plate is 15,4mm.

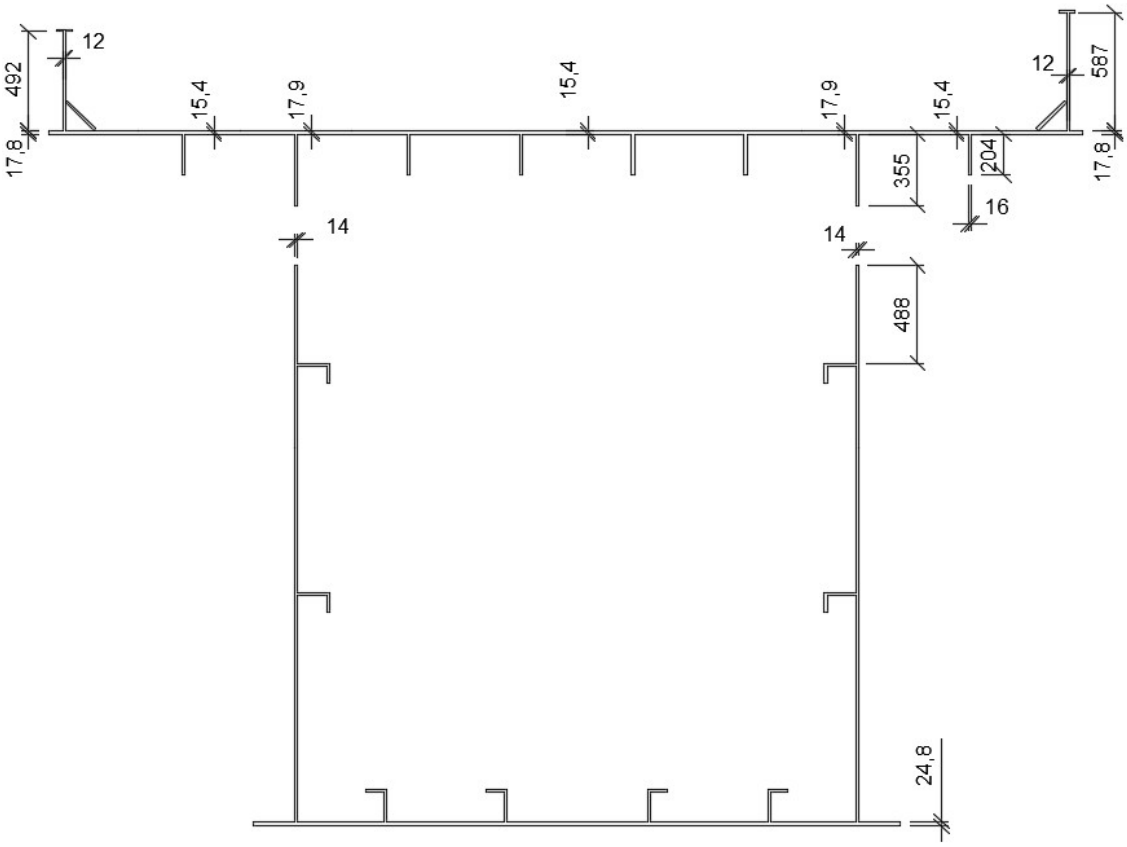
- global buckling – does not buckle itself because of validity of formula $\sigma_{com,Ed}$=144,5MPa< <ρ_C*f_y/γ_{M1}=0,928*235/1,1=198,3MPa. But there is a stress peak caused by buckling of neighbouring panel, so the thickness is modified in the same way.

Plate effect - stiffener 7			**Column effect**		
I_{sl}=	63707243	mm^4	$\sigma_{cr,sl}$=	1780,9	MPa
b_1=	455	mm	ψ_1=	1	
b_2=	560	mm	ψ_2=	1	
b=	1015	mm	b_1=	441	mm
t=	18	mm	b_2=	545	mm
a_c=	3959	mm	$b_{1,inf}$=	220,5	mm
a=	2388	mm	$b_{2,inf}$=	272,5	mm
$A_{c,eff,loc}$=	12312	mm^2	$b_{1,eff}$=	441	mm
A_c=	13002	mm^2	$b_{2,eff}$=	545	mm
$\beta_{a,c}$=	0,947		$b_{1,inf,eff}$=	220,5	mm
$A_{sl,1}$=	13002	mm^2	$b_{2,inf,eff}$=	272,5	mm
$\sigma_{cr,sl}$=	2014,6	MPa	$I_{sl,1}$=	63707243	mm^4
ν=	0,3		$A_{sl,1}$=	13002	mm^2
b_c=	544	mm	a=	2388	mm
b_{sl1}=	544	mm	b_c=	544	mm
			b_{sl1}=	544	mm
$\sigma_{cr,p}$=	2014,6	MPa	$\sigma_{cr,c}$=	1780,9	MPa
λ_p=	0,332		$A_{sl1,eff}$=	12422	mm^2
ψ=	1,00		$\beta_{a,c}$=	0,955	
ρ=	**1,000**		λ_c=	0,355	
			i=	70,0	mm
			e=	91,0	mm
Interaction - outside part			α_e=	0,607	
ξ=	0,131		Φ=	0,610	
ρ_c=	**0,928**		**χ_c=**	**0,904**	

- global buckling of web – web does not buckle

Plate effect - web			**Column effect - web**		
I_{sl}=	63572809	mm^4	$\sigma_{cr,sl}$=	1518,7	MPa
b_1=	1140	mm	ψ_1=	0,25	
b_2=	2280	mm	ψ_2=	-1,72	
b=	3420	mm	b_1=	1140	mm
t=	14	mm	b_2=	419	mm
a_c=	11262	mm	$b_{1,inf}$=	660	mm
a=	2388	mm	$b_{2,inf}$=	168	mm
$A_{c,eff,loc}$=	16348	mm^2	$b_{1,eff}$=	842	mm
A_c=	18762	mm^2	$b_{2,eff}$=	419	mm
$\beta_{a,c}$=	0,871		$b_{1,inf,eff}$=	488	mm
$\sigma_{cr,sl}$=	1521,7	MPa	$b_{2,inf,eff}$=	168	mm
ν=	0,3		$I_{sl,1}$=	63572809	mm^4
b_c=	1572	mm	$A_{sl,1}$=	15214	mm^2
b_{sl1}=	426	mm	a=	2388	mm
			b_c=	1572	mm
			b_{sl1}=	426	mm
$\sigma_{cr,p}$=	5615,4	MPa	$\sigma_{cr,c}$=	5604,2	MPa
λ_p=	0,191		$A_{sl1,eff}$=	12801	mm^2
ψ=	0,00		$\beta_{a,c}$=	0,841	
ρ=	**1,000**		λ_c=	0,188	
			i=	64,6	mm
			e=	111,4	mm
Interaction - web			α_e=	0,645	
ξ=	0,002		Φ=	0,514	
ρ_c=	**1,000**		**χ_c=**	**1,000**	

4.1.2.3 Normal stress



Picture 4.1-4 – effective cross section in midspan

$$N_{Rd} = A_{eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}31987 * \frac{235}{1{,}1} * 1000 = 68335{,}9kNm$$

$$M_{y,el,Rd} = w_{y,el,eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}33619 * \frac{235}{1{,}1} * 1000 = 71822{,}4kNm$$

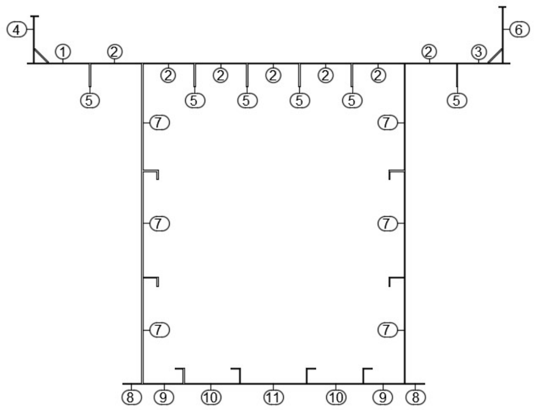
$$M_{z,el,Rd} = w_{z,el,eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}24287 * \frac{235}{1{,}1} * 1000 = 51885{,}9kNm$$

$$\eta_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_{y,N}}{M_{y,el,Rd}} + \frac{M_{z,Ed} + N_{Ed} * e_{z,N}}{M_{z,el,Rd}} =$$

$$= \frac{190{,}4}{68335{,}9} + \frac{59471{,}4 + 190{,}4 * 0{,}065}{71822{,}4} + \frac{6801{,}0 + 190{,}4 * 0{,}001}{51885{,}9} = 0{,}962$$

<u>SATISFACTORY (96,2%)</u>

4.1.3 Cross section in place of thickeness of flanges



Picture 4.1-5 - numbers of subpanels

4.1.3.1 Local buckling and shear lag

Upper flange - buckling and shear lag

subpanel no. 1			subpanel no. 2			subpanel no. 3			subpanel no. 4			subpanel no. 5		
ψ=	1		ψ=	1		ψ=	1		ψ=	1		ψ=	1	
k_σ=	4,000		k_σ=	4,000		k_σ=	4,000		k_σ=	4,000		k_σ=	4,000	
b=	441	mm	b=	545	mm	b=	544	mm	b=	545	mm	b=	331	mm
t=	14	mm	t=	14	mm	t=	14	mm	t=	14	mm	t=	14	mm
f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa	f_y=	235	MPa
ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000		ε=	1,000	
λ_P=	0,555		λ_P=	0,685		λ_P=	0,684		λ_P=	0,685		λ_P=	0,416	
ρ=	1,000		ρ=	0,991		ρ=	0,992		ρ=	0,991		ρ=	1,000	
b_{eff}=	***441***	***mm***	***b_{eff}=***	***540***	***mm***	***b_{eff}=***	***539***	***mm***	***b_{eff}=***	***540***	***mm***	***b_{eff}=***	***331***	***mm***
			α0=	0,995		α0=	0,996					α0=	1,000	
			b0=	1862	mm	b0=	1400	mm				b0=	1732	mm
			Le=	42800	mm	Le=	42800	mm				Le=	42800	mm
			κ=	0,043		κ=	0,033					κ=	0,040	
			β=	0,988		β=	0,993					β=	0,990	
			teff=	**13,8**	**mm**	**teff=**	**13,9**	**mm**				**teff=**	**13,9**	**mm**
b_{e1}=	221	mm	b_{e1}=	270	mm	b_{e1}=	270	mm	b_{e1}=	270	mm	b_{e1}=	166	mm

Web - buckling

subpanel no. 6			subpanel no. 7			subpanel no. 8			subpanel no. 9			subpanel no. 10		
ψ=	0,74		ψ=	0,86		ψ=	0,74		ψ=	0,25		ψ=	-2,60	
k_σ=	4,581		k_σ=	0,482		k_σ=	4,581		k_σ=	6,308		k_σ=	77,501	
b=	355	mm	b=	240	mm	b=	450	mm	b=	1140	mm	b=	1126	mm
t=	12	mm	t=	16	mm	t=	12	mm	t=	14	mm	b_c=	273	mm
f_y=	235	MPa	f_y=	355	MPa	f_y=	235	MPa	f_y=	235	MPa	t=	14	mm
ε=	1,000		ε=	0,814		ε=	1,000		ε=	1,000		f_y=	235	MPa
λ_P=	0,487		λ_P=	0,935		λ_P=	0,617		λ_P=	1,142		ε=	1,000	
ρ=	1,000		ρ=	0,854		ρ=	1,000		ρ=	0,739		λ_P=	0,322	
b_{eff}=	***355***	**mm**	***b_{eff}=***	**205**	**mm**	***b_{eff}=***	***450***	**mm**	***b_{eff}=***	**842**	**mm**	ρ=	1,000	
b_{e1}=	167	mm				b_{e1}=	211	mm	b_{e1}=	355	mm	***b_{eff}=***	**273**	**mm**
b_{e2}=	188	mm				b_{e2}=	239	mm	b_{e2}=	488	mm	b_{e1}=	109	mm
												b_{e2}=	164	mm

Lower flange - shear lag

t=	14	mm
α0=	1,000	
b0=	1400	mm
Le=	42800	mm
κ=	0,033	
β=	0,993	
teff=	**13,9**	**mm**

4.1.3.2 Global buckling and shear lag

- global buckling - inner part of box girder

Plate effect - inner part			**Column effect - inner part**		
$A_{c,eff,loc}$=	44018	mm^2	$\sigma_{cr,sl}$=	1871,2	MPa
A_c=	46720	mm^2	ψ_1=	1	
$\beta_{a,c}$=	0,942		ψ_2=	1	
t=	14	mm	b_1=	544	mm
b=	2786	mm	b_2=	544	mm
σ_E=	4,798	MPa	$b_{1,inf}$=	272	mm
a=	2388	mm	$b_{2,inf}$=	272	mm
α=	0,857		$b_{1,eff}$=	539	mm
I_{sl}=	263482493	mm^4	$b_{2,eff}$=	539	mm
I_p=	700072	mm^4	$b_{1,inf,eff}$=	270	mm
γ=	376,365		$b_{2,inf,eff}$=	270	mm
$\sum A_{sl}$=	15360	mm^2	$I_{sl,1}$=	60132189	mm^4
A_p=	39004	mm^2	$A_{sl,1}$=	11680	mm^2
δ=	0,394		a=	2388	mm
ψ=	1		b_c=	544	mm
			b_{sl1}=	544	mm
k_σ=	369,5		$\sigma_{cr,c}$=	1871,2	MPa
$\sigma_{cr,p}$=	1772,8	MPa	$A_{sl1,eff}$=	11004	mm^2
λ_p=	0,353		$\beta_{a,c}$=	0,942	
ρ=	**1,000**		λ_c=	0,344	
			i=	71,8	mm
			e=	85,3	mm
Interaction - inner part			α_e=	0,597	
ξ=	0		Φ=	0,602	
ρ_c=	**0,912**		**χ_c=**	**0,912**	

A thickness of the flange is modified according to formula $A_{c,eff} = \rho_c * A_{c,eff,loc}$. So the reduced thickness of plate is 12,2mm.

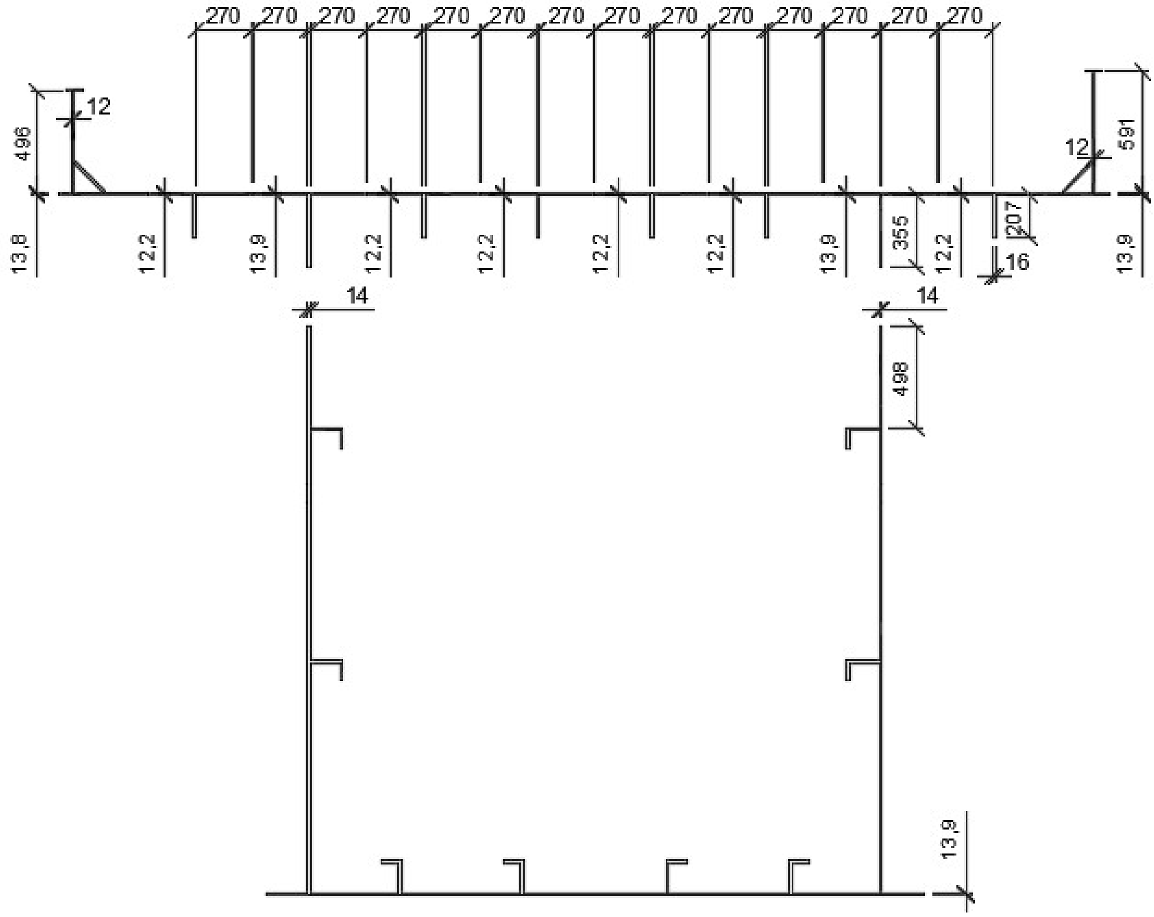
- global buckling – does not buckle itself because of validity of formula $\sigma_{com,Ed}$=102,4MPa< <ρ_C*f_y/γ_{M1}=0,928*235/1,1=198,3MPa. But there is a stress peak caused by buckling of neighbouring panel, so the thickness is modified in the same way

Plate effect - stiffener 7			**Column effect**		
I_{sl}=	58795187	mm^4	$\sigma_{cr,sl}$=	1948,7	MPa
b_1=	455	mm	ψ_1=	1	
b_2=	560	mm	ψ_2=	1	
b=	1015	mm	b_1=	441	mm
t=	14	mm	b_2=	545	mm
a_c=	4685	mm	$b_{1,inf}$=	220,5	mm
a=	2388	mm	$b_{2,inf}$=	272,5	mm
$A_{c,eff,loc}$=	10287	mm^2	$b_{1,eff}$=	441	mm
A_c=	10966	mm^2	$b_{2,eff}$=	540	mm
$\beta_{a,c}$=	0,938		$b_{1,inf,eff}$=	220,5	mm
$A_{sl,1}$=	10966	mm^2	$b_{2,inf,eff}$=	270,0	mm
$\sigma_{cr,sl}$=	2079,1	MPa	$I_{sl,1}$=	58795187	mm^4
ν=	0,3		$A_{sl,1}$=	10966	mm^2
b_c=	544	mm	a=	2388	mm
b_{sl1}=	544	mm	b_c=	544	mm
			b_{sl1}=	544	mm
$\sigma_{cr,p}$=	2079,1	MPa	$\sigma_{cr,c}$=	1948,7	MPa
λ_p=	0,326		$A_{sl1,eff}$=	10287	mm^2
ψ=	1,00		$\beta_{a,c}$=	0,938	
ρ=	**0,996**		λ_c=	0,336	
			i=	73,2	mm
			e=	81,3	mm
Interaction - outside part			α_e=	0,590	
ξ=	0,067		Φ=	0,597	
ρ_c=	**0,928**		**χ_c=**	**0,918**	

- global buckling of web – web does not buckle

Plate effect - web			**Column effect - web**		
I_{sl}=	62619608	mm^4	$\sigma_{cr,sl}$=	1580,9	MPa
b_1=	1140	mm	ψ_1=	0,25	
b_2=	2280	mm	ψ_2=	-2,60	
b=	3420	mm	b_1=	1140	mm
t=	14	mm	b_2=	273	mm
a_c=	11220	mm	$b_{1,inf}$=	660	mm
a=	2388	mm	$b_{2,inf}$=	109	mm
$A_{c,eff,loc}$=	14276	mm^2	$b_{1,eff}$=	842	mm
A_c=	16690	mm^2	$b_{2,eff}$=	273	mm
$\beta_{a,c}$=	0,855		$b_{1,inf,eff}$=	488	mm
$\sigma_{cr,sl}$=	1584,1	MPa	$b_{2,inf,eff}$=	168	mm
ν=	0,3		$I_{sl,1}$=	62619608	mm^4
b_c=	1426	mm	$A_{sl,1}$=	14397	mm^2
b_{sl1}=	280	mm	a=	2388	mm
			b_c=	1426	mm
			b_{sl1}=	280	mm
$\sigma_{cr,p}$=	8067,5	MPa	$\sigma_{cr,c}$=	8051,1	MPa
λ_p=	0,158		$A_{sl1,eff}$=	12801	mm^2
ψ=	0,00		$\beta_{a,c}$=	0,889	
ρ=	**1,000**		λ_c=	0,161	
			i=	66,0	mm
			e=	109,2	mm
Interaction - web			α_e=	0,639	
ξ=	0,002		Φ=	0,501	
ρ_c=	**1,000**		**χ_c=**	**1,000**	

4.1.3.3 Normal stress



Picture 4.1-6 - effective cross section, stress diagram

$$N_{Rd} = A_{eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}26633 * \frac{235}{1{,}1} * 1000 = 56897{,}8kNm$$

$$M_{y,el,Rd} = w_{y,el,eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}27276 * \frac{235}{1{,}1} * 1000 = 58270{,}9kNm$$

$$M_{z,el,Rd} = w_{z,el,eff} * \frac{f_{yk}}{\gamma_{M1}} = 0{,}21416 * \frac{235}{1{,}1} * 1000 = 45751{,}8kNm$$

$$\eta_1 = \frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed} + N_{Ed} * e_{y,N}}{M_{y,el,Rd}} + \frac{M_{z,Ed} + N_{Ed} * e_{z,N}}{M_{z,el,Rd}} =$$

$$= \frac{335{,}1}{56897{,}8} + \frac{35480{,}2 + 335{,}1 * 0{,}062}{58270{,}9} + \frac{4053{,}2 + 335{,}1 * 0{,}005}{45751{,}8} = 0{,}704$$

SATISFACTORY (70,4%)

4.1.4 Design shear resistance

Shear buckling for a whole web (for a=2200mm):

$$\alpha = \frac{a}{h_w} = \frac{2200}{3420} = 0{,}643$$

$$k_\tau = 4{,}1 + \frac{6{,}3 + 0{,}18 * \frac{I_{sl}}{t^3 * h_w}}{\alpha^2} + 2{,}2 * \sqrt[3]{\frac{I_{sl}}{t^3 * h_w}} =$$

$$= 4{,}1 + \frac{6{,}3 + 0{,}18 * \frac{2*33348536}{14^3*3420}}{0{,}643^2} + 2{,}2 * \sqrt[3]{\frac{2 * 33348536}{14^3 * 3420}} = 26{,}662$$

$$\sigma_E = 190000 * \left(\frac{t}{b}\right)^2 = 190000 * \left(\frac{14}{3420}\right)^2 = 3{,}184MPa$$

$$\tau_{cr} = k_\tau * \sigma_E = 26{,}662 * 3{,}184 = 84{,}891MPa$$

$$\lambda_w = 0{,}76 * \sqrt{\frac{f_{yw}}{\tau_{cr}}} = 0{,}76 * \sqrt{\frac{235}{84{,}891}} = 1{,}264\ (determinative)$$

Shear buckling for a subpanel (for a=2200mm):

$$\alpha = \frac{a}{h_w} = \frac{2200}{1140} = 1{,}930$$

$$k_\tau = 5{,}34 + 4 * \left(\frac{h_w}{a}\right)^2 = 5{,}34 + 4 * \left(\frac{1140}{2200}\right)^2 = 6{,}414$$

$$\sigma_E = 190000 * \left(\frac{t}{b}\right)^2 = 190000 * \left(\frac{14}{1140}\right)^2 = 28{,}655MPa$$

$$\tau_{cr} = k_\tau * \sigma_E = 6{,}243 * 28{,}655 = 183{,}795MPa$$

$$\lambda_w = 0{,}76 * \sqrt{\frac{f_{yw}}{\tau_{cr}}} = 0{,}76 * \sqrt{\frac{235}{183{,}795}} = 0{,}859$$

Shear resistance:

$$\chi_w = \frac{0{,}83}{\lambda_w} = \frac{0{,}83}{1{,}264} = 0{,}657$$

$$V_{bw,Rd} = \frac{2 * \chi_w * f_{yw} * h_w * t}{\sqrt{3} * \gamma_{M1}} = \frac{2 * 0{,}657 * 235 * 3420 * 14}{\sqrt{3} * 1{,}1 * 1000} = 7760{,}0\text{kN} > 5693{,}2\text{kN}$$

<u>SATISFACTORY (73,3%)</u>

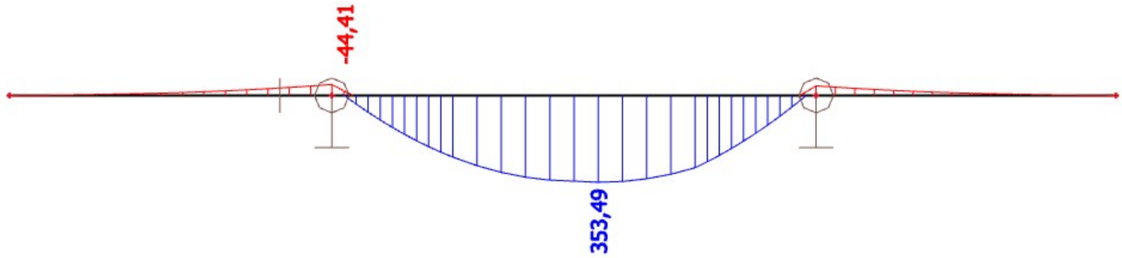
4.1.4.1 Combination of shear force and bending moment

- For $V_{Ed} = \frac{V_{bw,Rd}}{2} = \frac{7760{,}0}{2} = 3880{,}0\text{kN} \dots M_{Ed} = 35491{,}6\text{kNm}$

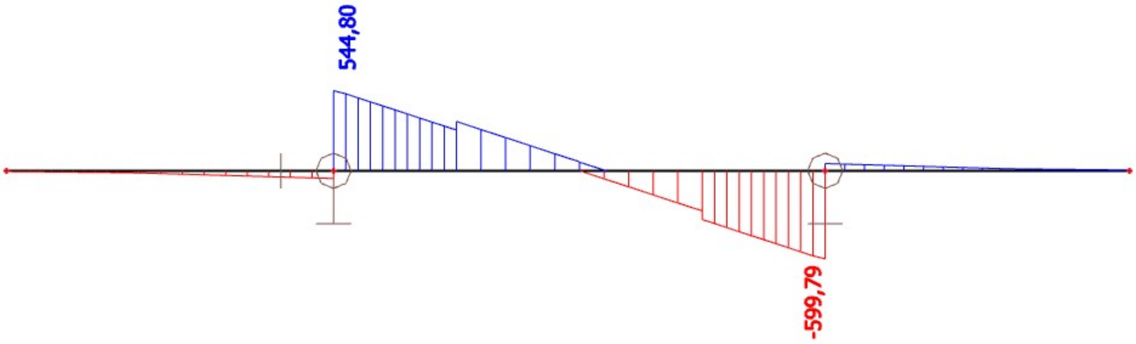
$$\mu_1 = \frac{M_{Ed}}{M_{el,Rd}} = \frac{35491{,}6}{59037{,}5} = 0{,}601 < \frac{M_{f,Rd}}{M_{el,Rd}} = \frac{42483{,}8}{59037{,}5} = 0{,}720 \dots interaction\ does\ not\ assess$$

4.2 Cross girder

4.2.1 Inner forces



Picture 4.2-1 - bending moment diagram

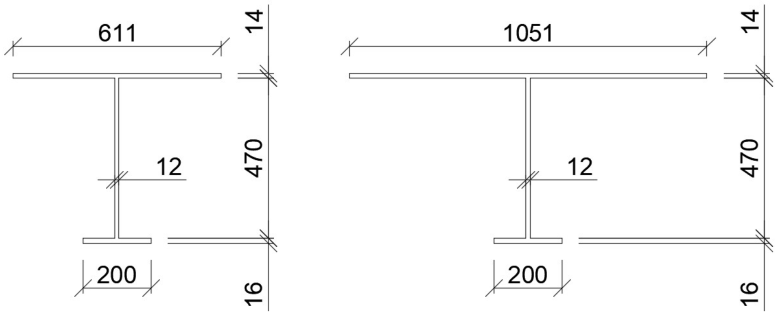


Picture 4.2-2 - shear force diagram

4.2.2 Bending strength

- calculation of dimension of an effective cross section including a buckling and a shear lag

ψ=	1	
$k_σ$=	4,000	
b=	2400	mm
t=	14	mm
f_y=	235	MPa
ε=	1,000	
$λ_P$=	3,018	
ρ=	0,307	
b_{eff}=	***737***	***mm***
$α_0$=	0,554	
b_0=	1200	mm
$L_{e,1}$=	1960	mm
$L_{e,2}$=	2240	mm
$κ_1$=	0,339	
$κ_2$=	0,536	
sagging bending		
$β^κ$=	0,829	
beff=	**611**	**mm**
hogging bending		
$β^κ$=	0,438	
beff=	**1051**	**mm**



Picture 4.2-3 – eff. cross section in areas of positive and negative bending moment

$w_{y,eff,14+} = 0{,}0021454m^3$
$w_{y,eff,14-} = 0{,}0022373m^3$

$$\mathrm{M_{y,el,Rd}} = \mathrm{w_{y,el}} * \frac{\mathrm{f_{yk}}}{\gamma_{M1}} = 0{,}0021454 * \frac{235}{1{,}1} * 1000 = 458{,}3\mathrm{kNm} > 353{,}5\mathrm{kNm}$$

SATISFACTORY (77,1%)

4.2.3 Design shear resistence

$$V_{pl,Rd} = \frac{f_y * h_w * t}{\sqrt{3} * \gamma_{M0}} = \frac{235 * 430 * 12}{\sqrt{3} * 1{,}0 * 1000} = 700{,}1kN > 599{,}8\mathrm{kN}$$

SATISFACTORY (85,7%)

4.2.3.1 Combination of shear force and bending moment

- For $V_{Ed} = 599{,}8\mathrm{kN} \ldots M_{Ed} = 38{,}8\mathrm{kNm}$

$$\mu_1 = \frac{M_{Ed}}{M_{el,Rd}} = \frac{38{,}8}{478{,}0} = 0{,}081 < \frac{M_{f,Rd}}{M_{el,Rd}} = \frac{325{,}2}{478{,}0} = 0{,}680 \ldots interaction\ does\ not\ assess$$

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List of pictures

CZECH TECHNICAL UNIVERSITY IN PRAGUE

Faculty of Civil Engineering

Department of Steel and Timber Structures

The assessment and the load capacity of the bridge in Prague - Motol

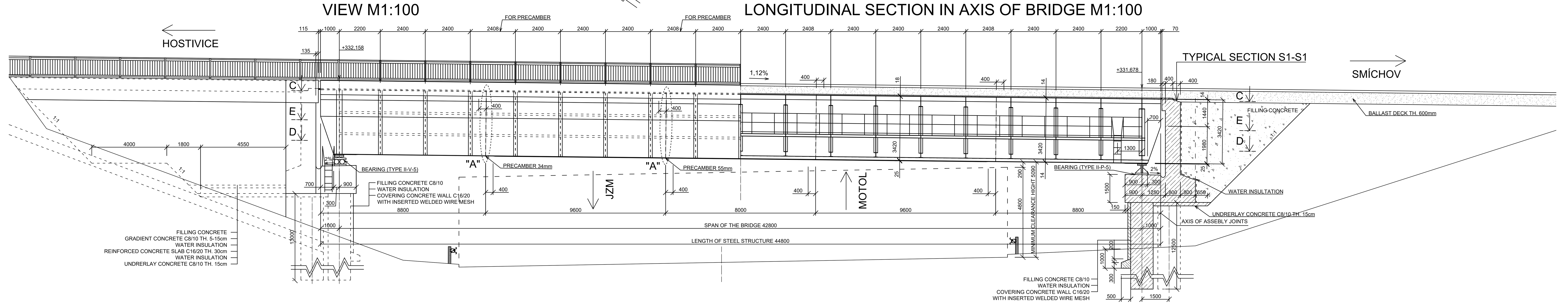
Master's thesis

C. Drawing documentation

Kateřina Soukupová

Prague 2018

List of drawings:



MATERIALS:
LOAD BEARING STEEL STRUCTURE (EXCLUDING UPPER LONGITUDINAL STIFFENERS):
STEEL 37 (S235)
UPPER LONGITUDINAL STIFFENERS: STEEL 51 (S355)
BEARINGS: CAST IRON
SCREW JOINTS: HIGHT STRENGTH BOLTS 10.9

SURFACE WORKING:
OUTSIDE SURFACE:
METALIZATION IN COMPOSITION 70μZn and 150μAl
SYNTHETIC SURFACE COLOR S2013
INSIDE SURFACE:
TWO LAYERS OF COLOR O2005
THREE LAYERS OF COLOR S2302-Plumbinex

STRUCTURE OF TRACK BALLAST:
TRACK BALLAST (FRACTION 16/32, DRUMED GRAVEL) ... MINIMAL THICKNESS 400mm
PROTECTIVE CEMENT SCREED WITH WIRE INSERT ... THICKNESS 50mm
INSULATING INSERT STICKED WITH GLUE... THICKNESS 10mm
PLATE OF BRIDGE DECK

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Faculty, university:	Faculty of Civil Engineering, CTU in Prague
Thesis supervisor:	doc. Ing. Pavel RYJÁČEK, Ph.D.
Drawn by:	Kateřina Soukupová
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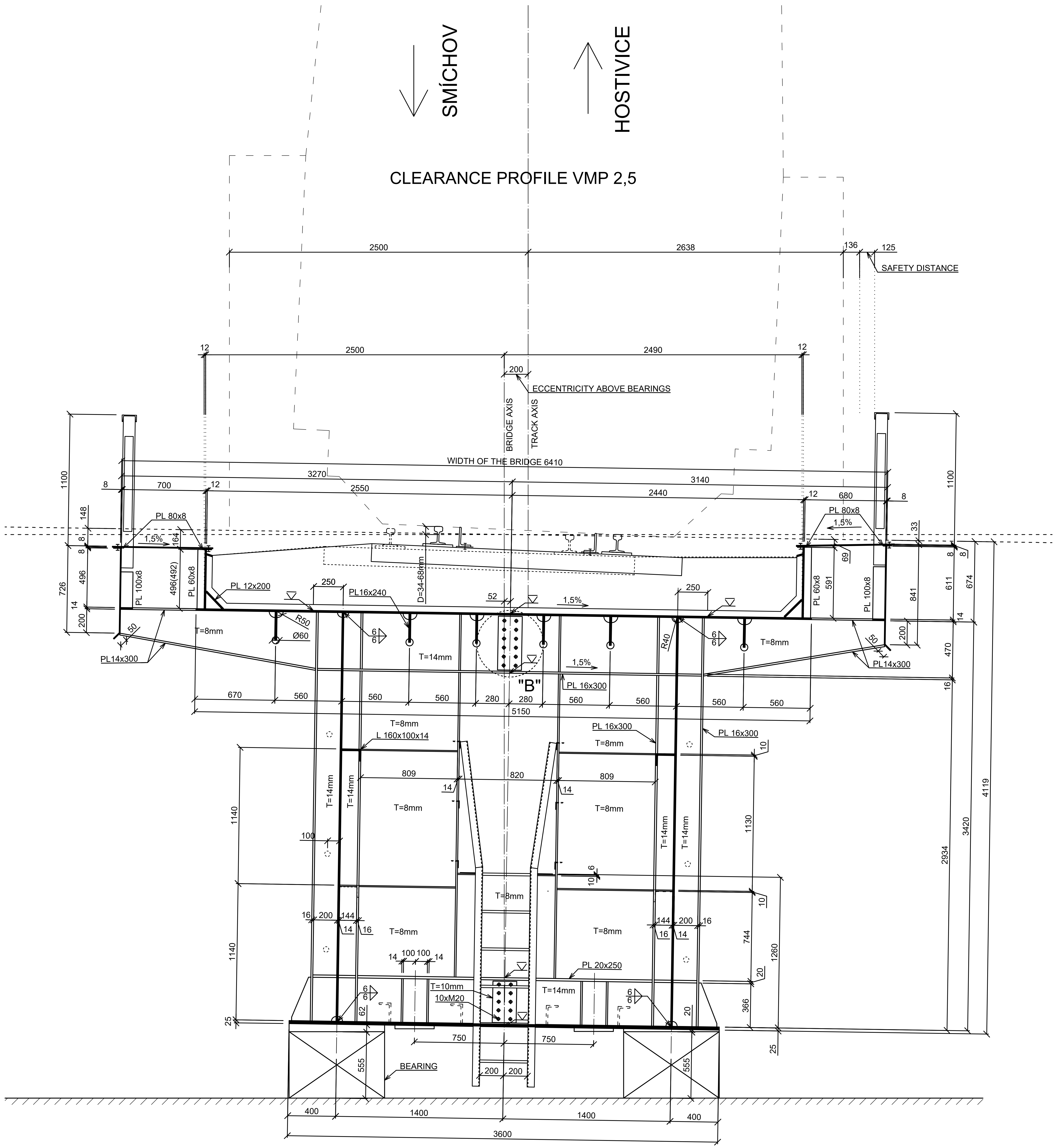
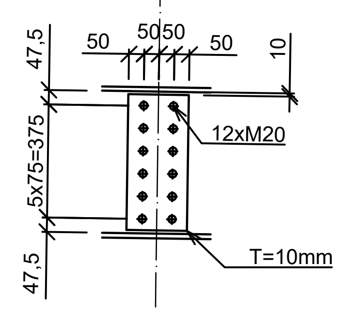
Drawing:	PLAN VIEW AND LONGITUDINAL CROSS SECTIONS
Academic year:	2017/2018
Scale:	1:100
Attachment No.:	C.1
Size:	10xA4
Other scales:	1:50
Date:	01/2018

CROSS SECTION 6-6 (1-1 mirror-inverted) 1:25



DETAIL "B" 1:25



MATERIALS:
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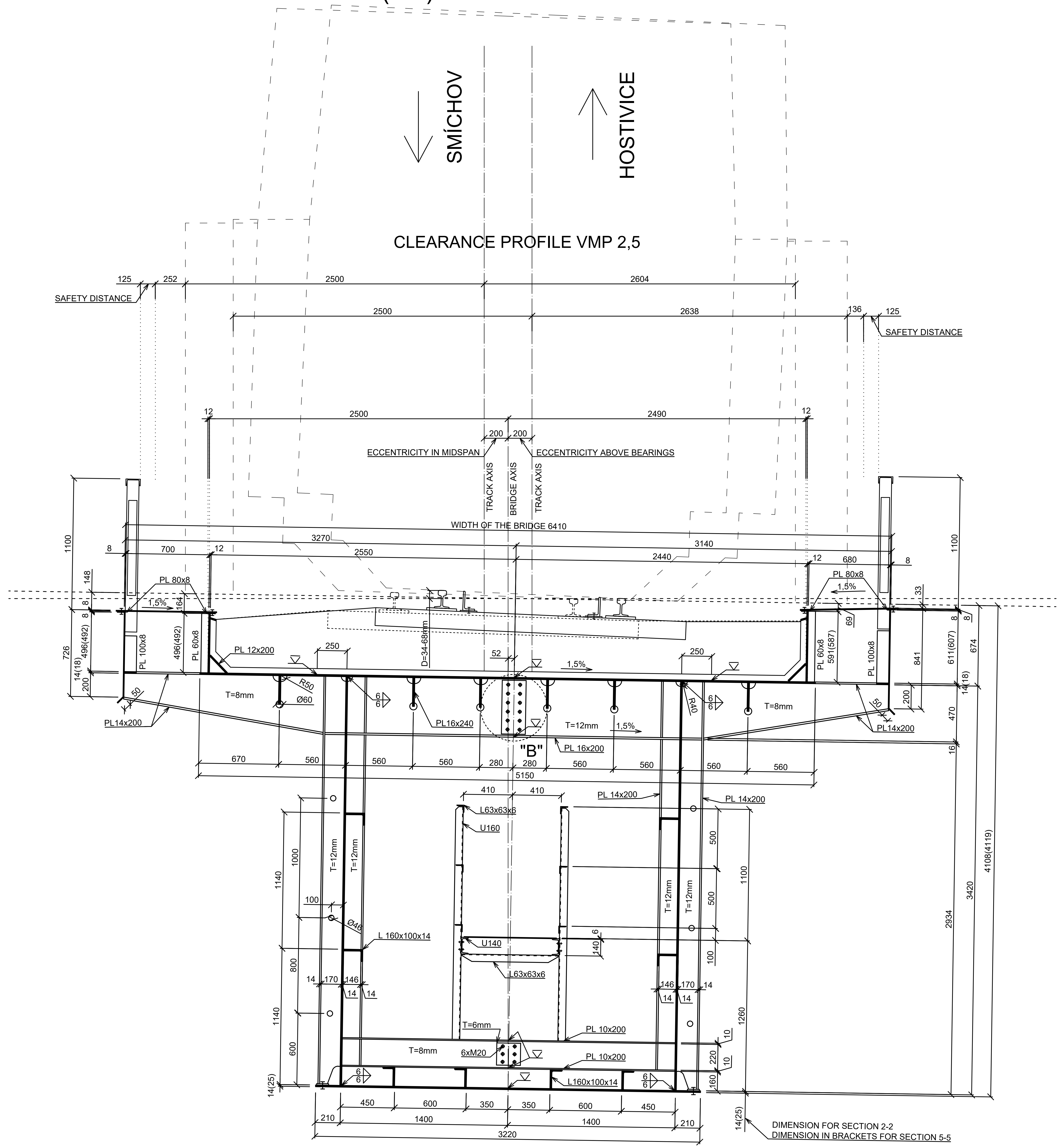

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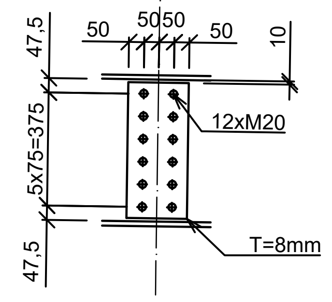

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Drawing:	CROSS SECTION 2-2, 5-5			Size:	A2
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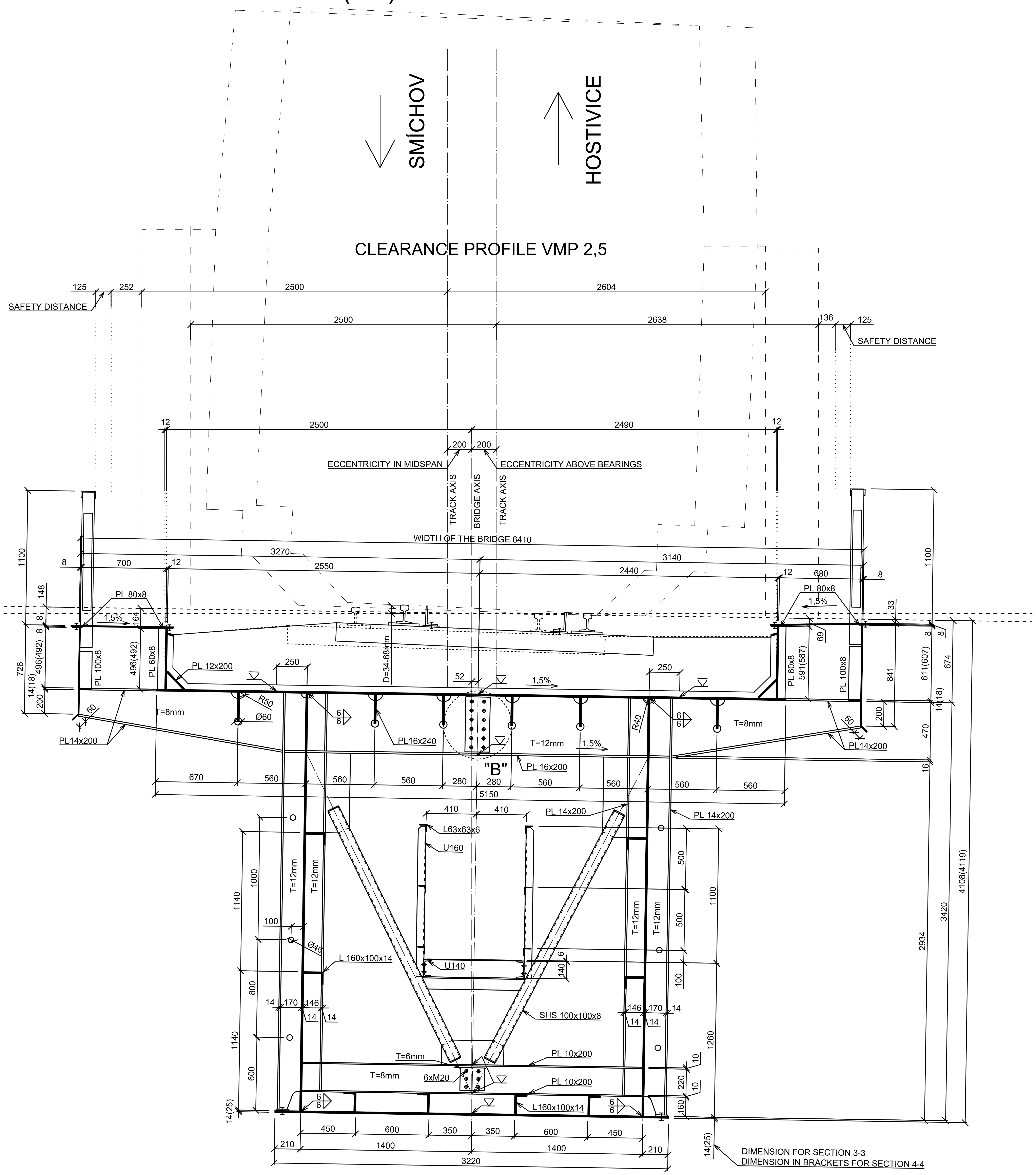

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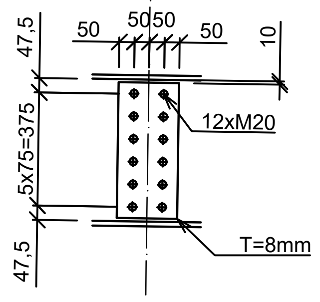

MATERIALS:
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Drawing:	CROSS SECTION 3-3, 4-4

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