CRACOW UNIVERSITY OF TECHNOLOGY
Faculty of Civil Engineering
Institute of Building Materials and Structures (L-1)
Chair of Bridges and Tunnel Structures (L-16)

IGOR SKIBA

N° of student's book: 97740
Degree course: Civil Engineering
Full time 1st cycle studies

Conceptual Design of an Alternative Footbridge over "Rudawa" River

Koncepcyjny projekt alternatywnej kładki nad rzeką Rudawa

BACHELOR OF SCIENCE

Grade:	Date:	Supervisor's signature:

Supervisor:
Dr inż. Marek Pańtak

Cracow, January 2015

I would like to express my deep gratitude to my supervisor dr inż. Marek Pańtak for his time, patience and kindness during preparation of this thesis.
Author

ABSTRACT

The main purpose of the work was to elaborate a study project about a steel arch footbridge. The work includes the general information about footbridges, their classification due to material and construction types. Conceptual design and static model were made. Selective elements were chosen for necessary calculations.

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1. INTRODUCTION

1.1. SUBJECT AND AIM OF THESIS

The aim of present diploma work was to design an alternative footbridge for the current existing one over a “Rudawa” river in Cracow, Poland. The work has a purpose of creating a different solution of a bridge passage for pedestrians, bicyclists and service vehicles. On the basis of performed measurements on the already existing structure, a new footbridge was designed. The thesis has also a purpose to broaden horizons about bridge engineering and gives the basic skills in designing a footbridge.

1.2. SCOPE OF WORK

The scope of work has been divided into a study part and a design one. The study part starts from general information about footbridges and basic history of bridges. Section covers three major types of footbridges in terms of construction materials as well as the basic types of arch footbridges with examples. In design part, essential measurements were performed of already existing structure and a footbridge which was my inspiration all the way during preparation of this thesis. Due to that, a conceptual design has been made and so the selective parts of static analysis has been done.

I. STUDY PART

2. FOOTBRIDGES

2.1 GENERAL INFORMATION ABOUT FOOTBRIDGES

Throughout history, footbridges were strongly linked to bridges itself. The differences between those two terms started to vary at the end of the 18th century when irrepressible development of industrialization together with mobility and traffic intensity enforced distinction in passage for the vehicle flow and for the pedestrian one. The division remained and nowadays it is hard to imagine town's landscape without these structures especially in highly populated, urbanized cities and metropolises.

Why do we need footbridges?

Constant improvement of human life incites pursuit for better and more convenient life. And footbridges are one of the ways to achieve that because they fulfill convention they were invented for – namely they are used to shorten the route from one place to another as well as to cover a distance without which the way would be impassable.

Besides the obvious application of footbridges, they are also used as a mean of pedestrians' safety. There are countless dangers such as hazardous roads, road junctions and railways which are gradually being eliminated thanks to footbridges and underground passages which enable safer transition.

Figure no. 2.1. Footbridge over A4 highway in Katowice, Poland [M1]

Footbridges may satisfy esthetic values combined with expression of modernity, technological advancement in civil engineering and beauty of construction itself. Footbridge is relatively cheap comparing to for example bridge. Nevertheless, the higher eagerness for complexity and originality of one's footbridge leads to significant increment of costs needed to achieve such high expectations.

Table 2.1 Selective construction costs of footbridges. [B1]

Name	Localization	Year of Construction	Price [€/m^2]
Vranow-Brücke	Vranow-Lake, CZ	1993	1000
Pasarela del Malecón	Murcia, E	1997	2500
Punt da Suransans	Via Mala, CH	1999	5000
Passerelle Solférino	Paris, F	1999	5500
Katzbuckel-brücke	Duisburg, D	1998	7000
Miho Museum Bridge	Kyoto, J	1997	17000
Millennium Bridge	London, GB	2000	17000
Millennium Bridge	Gateshead, GB	2001	32000
Footbridge in Wrocław - Leśnica	Wrocław - Leśnica, PL	1999	1600
Footbridge "Krzywy kij"	A4 Highway, PL	2000	1230
Footbridge "Łuk Erosa"	A4 Highway, PL	2000	1580

Easiness of forming unique, beautiful shapes of a footbridge may come in handy in becoming so called "landmark". Landmark is a characteristic point, a signature of a city in terms of engineering accomplishments. Wonderful footbridges please the eye and embellish the infrastructure of a city.

Figure no. 2.2 Gateshead Millennium Bridge, UK [M2]


Figure no. 2.3. Seafarers Bridge, Australia [M3]

2.2 INTRODUCTION TO ARCH STRUCTURES

The arch bridge is one of the most popular type of a bridge. To fully understand its phenomenon why it is still so vastly in use, we have to go back to our ancestors' time. Allegedly, the oldest existing arch bridge was created by Mycenaean civilization in 1300 BC in Greece which is still used by the local inhabitants.

But it was the Romans who fully realized and appreciated the arches in a bridge construction. The shape of Roman arch bridges was generally semicircular. Load forces were carried through the curve of the arch meeting with the ground support which provided lightweight structure as well as prevention from being swept away during floods while additionally costs of construction were minimalized because of the lower demand for materials. The circular arch form allowed creating spans much longer than stone beams. That is why it is not surprising why Romans favored arch bridges and created more than 1000 stone ones in Europe, Asia and North Africa.

Figure no. 2.4. Pont du Gard in Roman Gaul, France [M4]

As the time went on, during medieval times in Europe, progress of arch bridges was slightly improved due to narrower piers, thinner arch barrels and lower span-rise ratios. These solutions enabled increase of spans up to 70 meters. Medieval bridges are known for its pointed arch which decreased horizontal thrust at the abutments.

The purposes of a bridge back then, was not only to create a passage but also it was a place for miscellaneous shops, chapels and even could be used as a defense of nearby cities.

Figure no. 2.5. Pont Valentré, France [M3]

Not many ingenious ideas and techniques were introduced in bridge construction until 18^{th} century. Then a new age of science and engineering began. Architects and engineers of that time started to use cast iron which was an enormous step in terms of a bridge engineering. Because of iron, a new fundamental design system was introduced – a truss system which revolutionized civil engineering and bridge construction.

Figure no. 2.6. Ironbridge, UK [M5]

In modern times, many new building materials have been popularized in bridge construction. In addition to timber, stone, iron and masonry units which were used in past engineering history, new materials such as concrete (reinforced and pre-stressed), steel and combination of those materials occurred.

The steel arch bridges seem to have a similar action in terms of structure to the old masonry arch bridges. The arch exerts horizontal thrusts on the foundations and works mainly in compression. The deck could be supported on struts or might be suspended on hangers.

Figure no. 2.7. Te Rewa Rewa Bridge, New Zealand [M6]

2.3 MATERIALS USED IN CONSTRUCTION OF FOOTBRIDGES

Nowadays, three major types of footbridges in terms of material may be distinguished.

2.3.1 WOODEN FOOTBRIDGES

Wooden footbridges are commonly built in environmental areas such as: forests, parks, national parks, nature reserves etc. Timber does not interfere with nature and perfectly compose to natural landscapes. Furthermore, wooden bridges are receiving general attention because they are environmentally friendly comparing to other bridges.

Figure no. 2.8. Oakstead Pedestrian Bridge, Land O'Lakes, Florida, USA [M7]

Nonetheless, there are some hazards accompanied by usage of wood. For not sheltered timber bridge surface, there is a danger of slipping, especially when wood planks are parallel to the longitudinal direction of a footbridge. One way to eliminate that problem is to apply vertical planks. Because of hygroscopic nature of wood the moisture expansion should be also taken into account and moisture control should be performed periodically. What is also worth mentioning, wood is organic matter, it means it is the main source of nutrition for animals, plants and insects. To solve that problem fumigants should be implied. Frequent maintenance is the key to a beautiful wooden footbridge.

2.3.2 CONCRETE FOOTBRIDGES

Concrete footbridges are eagerly built in urban areas because of its durability and strength in comparison to other types of materials such as steel or wood. Concrete can withstand natural disasters such as earthquakes and hurricanes. Moreover, it has the ability to resist extreme fluctuations of temperature as well as corrosive chemicals. It is not likely to rot, corrode or decay as other building materials. Because of its flexibility properties, diverse shapes may be achieved.

Figure no. 2.9. Footbridge at the Muehlheimer Hafen, Cologne, Germany [M8]

Walking, jumping and running are a daily bread for footbridges which means that hence vibrations are inseparable, unceasing phenomenon. Concrete damps it more effectively than steel and also ensures low maintenance, affordability and is extremely energy efficient in production.

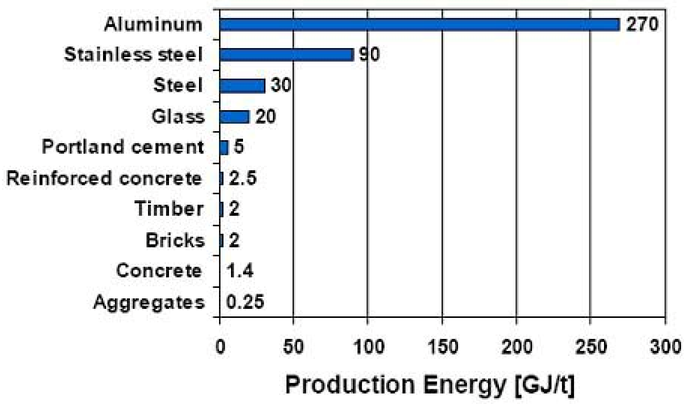


Figure no. 2.10. Production of energy [M9]

2.3.3 STEEL FOOTBRIDGES

Steel among all the other building materials has the highest application in terms of versatility. While designing a steel footbridge, the only limitation a designer can encounter is his imagination. Because of the prefabrication of components, the speed of steel footbridge construction is significantly increased whereby the disruption of the public is minimized. Steel, as well as concrete is also highly resistant to natural disasters such as earthquakes. In 1994 Northridge, an earthquake demolished almost all the surrounding transportation infrastructure, however 96% of the existing steel bridges were completely untouched, even though they were designed according to 1940s technology. Steel footbridges are cost-effective which means that due to the light weight of steel, costs may be reduced because of smaller abutments, fast installation and lighter equipment. Steel bridges have proved to have a life span higher than 100 years. As J.A Waddell, internationally renowned bridge designer, 1921, said:

'The life of a metal bridge that is scientifically designed, honestly and carefully built, and not seriously overloaded, if properly maintained, is indefinitely long'

But if a footbridge happened to reach the end of its useful life, the girders could be cut and returned to the steelworks for recycling as steel is a sustainable material.

Figure no. 2.11. Celtic Gateway Footbridge, UK [M4]

2.4 TYPES OF ARCH FOOTBRIDGES

2.4.1 DECK ARCH FOOTBRIDGES

Deck arch footbridge is an arch footbridge in which the bridge deck is located above the crown of the arch – in other words the deck is supported by arch. Weight and loads are transferred into a horizontal thrust restrained by the abutments at each side. The area between the deck and the arches is called spandrel. It may be divided into:

- Closed-spandrel footbridge – when the spandrel area is solid. This type of deck arch footbridge was used in the distant past when masonry and stone arch were popular.

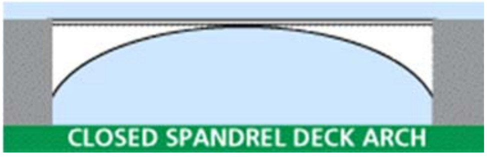


Figure no. 2.12. Scheme of closed spandrel deck arch [M10]

- Open-spandrel footbridge – when the spandrel area is assembled by columns, typical in modern bridge engineering.

Figure no. 2.13. Scheme of open spandrel deck arch [M10]

- Spandrel-braced footbridge – when the spandrel area is assembled by columns, typical in modern bridge engineering.

Figure no. 2.14. Scheme of spandrel-braced arch [M10]

A footbridge located in Rietveld Park in Nesselande can be classified as a deck arch footbridge because of its spandrel braced arch type. The total length equals to 93m while every span exceeds 20m. Because of its modular design, it was possible to create 3 types:

- Type 1: l=20-95m, b=4.2m
- Type 2: l=16-20m, b=3.6-4.2m
- Type 3: l=12-16m, b=1.2-2.5m

Figure no. 2.15. Footbridge in Rotterdam [M11]

Figure no. 2.16. Footbridge in Rotterdam. [M12]

The Ripshorst Footbridge is definitely holding everyone's attention because of its extraordinary shape of the arch for not only the arch is curved in elevation, but also curved in plan. The length of middle-span of a deck is about 78m, while the total length equals 130m. The width of a deck is 3m. The arch diameter ranges from 370mm up to 550m. The strut diameter ranges from 80mm to 220mm.

Figure no. 2.17. Ripshorst Footbridge, Germany [M13]

Figure no. 2.18. Ripshorst Footbridge, Germany [M2]

2.4.2 TIED ARCH FOOTBRIDGES

A tied arch footbridge is an arch bridge in which the arches are composed with the bridge deck. The bridge deck ties the arch and is under a tension because it carries the horizontal force as a tie member. The arch thrust is taken by a tie member and thanks to that the substructure is responsible for taking only a vertical load. Nevertheless, it is needed for one support to be restrained in the longitudinal direction to carry the forces resulting from wind, braking, and acceleration. The tied arch footbridges are usually used when there is no good foundation material or it is economically unfavorable to reach the bedrock.

Figure no. 2.19. GFRP Lleida Pedestrian Bridge, Spain. [M4]

The Lleida Bridge is a pedestrian bridge crossing a highway and a railway line. The span equals 38m while width equals 3m. The footbridge rises 6.2m above ground level. It is worth to mention that this footbridge is entirely made of GFRP (Glass Fiber Reinforced Polymer) joined together using stainless steel fixings. Craning the whole structure happened really fast, namely it lasted 3 hours.

A flagship example of modernity, innovation and even futurism is certainly Zubizuri (meaning white bridge) also called Campo Volantin Footbridge. It was made by recognizable architect and civil engineer famous for combining the knowledge and skills of his two faculties to show engineering as an art. White structure has a parabolic steel inclined arch with the pedestrian floor made of glass. The length equals 75m while the height equals 15m.

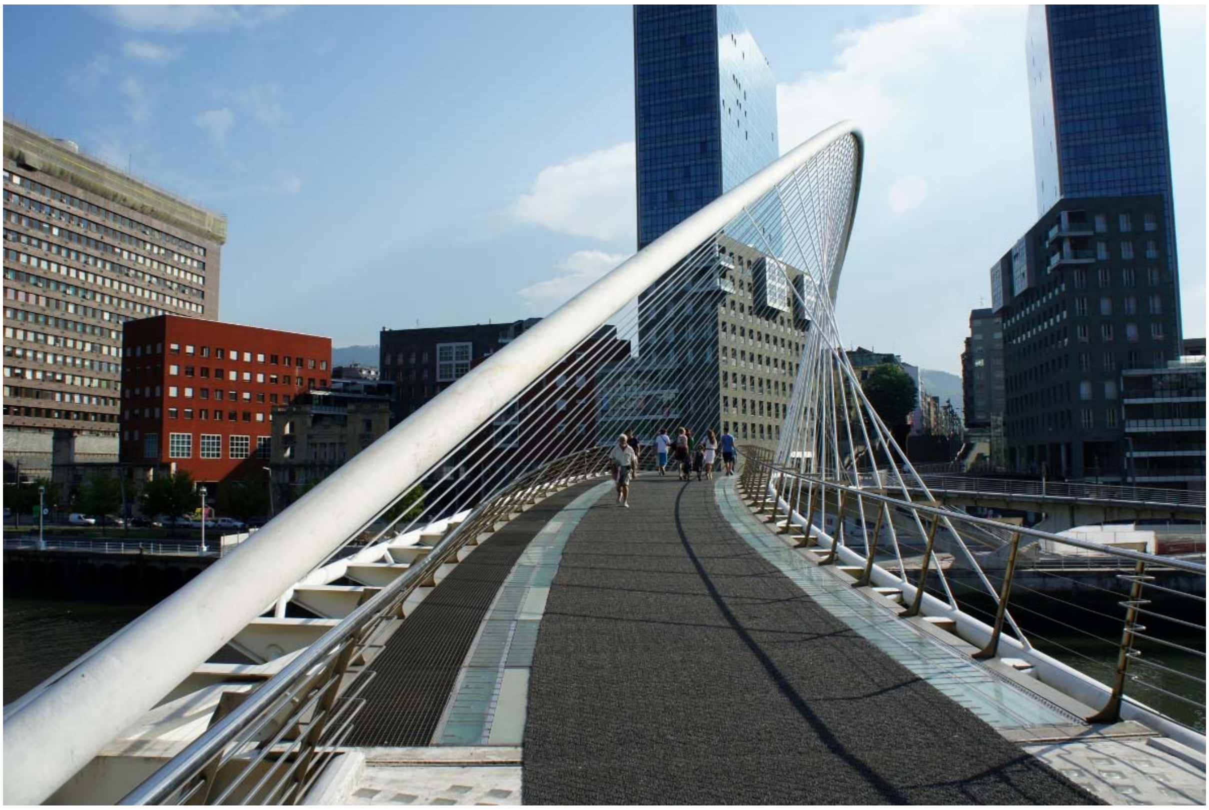


Figure no. 2.20. Zubizuri, Spain [M14]

2.4.3 THROUGH ARCH FOOTBRIDGES

The through arch footbridge is defined by a base of an arch structure being below the deck and rising above the deck, in other words a deck is suspended on the arch. These types of footbridges are usually used when the span of a deck is long. Approximately, for semi-circular arches the height of a footbridge is half of the length of the span. This kind of system gives a possibility to reduce the absolute height of a structure. However, a reliable foundations need to be provided, and even bigger ones when there is a need of flattening an arch because then a thrust is increased.

The humbler bay arch bridge is a perfect example of old-fashioned beautiful-looking pedestrians' and bicycles' footbridge constructed in 1994 in Toronto. The length equals130m whereas the longest span is 100m. The peak height rises above 21m. Forty four stainless-steel hangers, each 50mm, are fastened into high-strength steel pipes of 1200mm diameter. Concrete-filled caissons were used as foundations and went down 30m into the ground up to the bedrock.

Figure no. 2.21 Humber Bay Arch Bridge, Canada [M4]

Another fine example of a through arch footbridge is Mcloughlin Boulevard Pedestrian Bridge built in 2006. It has a length of 92m and main span of 73m with 450mm diameter pipes. A ribbon deck suspended on two inclined inward arches was assembled from precast segments and composite deck slab. The bearing tendons were post-tensioned during the erection of the deck and are formed from strands which are protected by cast-in-place slab. Edge pipes complete the hanger system which contains small tension rod that resists the lateral force.

Figure no. 2.22. McLoughlin Boulevard Pedestrian Bridge, USA [M15]

II. DESIGN PART

3. GENERAL INFORMATION

3.1 PURPOSE AND APPLICATION OF THE STRUCTURE

The structure which is supposed to be designed is a footbridge, which function is to allow pedestrians and cyclists to pass a “Rudawa” River in Cracow.

3.2 SUBJECT OF STUDIES

Subject of studies is the arch footbridge, which will be replaced for the current truss footbridge situated in Cracow.

3.3 LOCALIZATION

The new object will be located across the “Rudawa” River in Wola Justowska district in Cracow city.

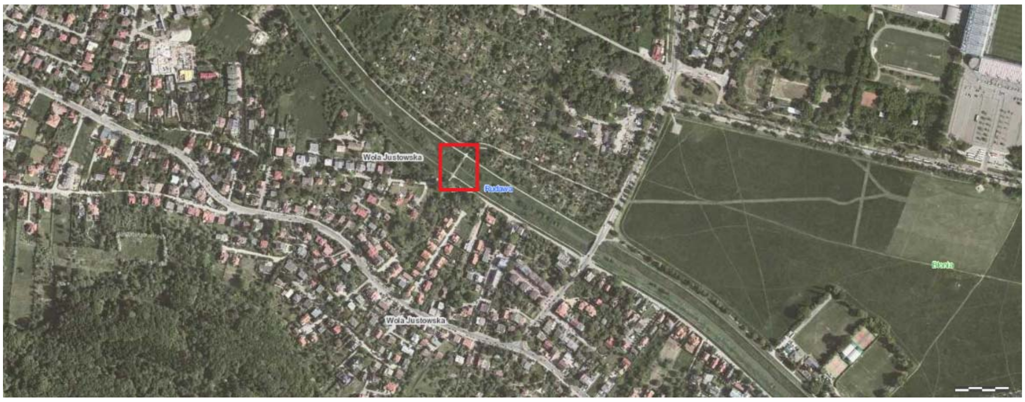


Figure no. 3.1. Localization [M16]

Figure no. 3.2. Localization [M16]

3.4 THE DESCRIPTION OF CURRENTLY EXISTING FOOTBRIDGE

The currently existing footbridge has truss type system of diagonal and perpendicular members based on stones abutments. The deck surface is made of wooden planks placed horizontally to the longitudinal direction of a structure. It was created during Austrian Partition in 18^{th} century.

Figure no. 3.3. Side view of the footbridge

Figure no. 3.4. Front view of the footbridge

Basic dimensions:

- Length: 20m
- Width: 3.30m
- Truss spacing: 3.25m

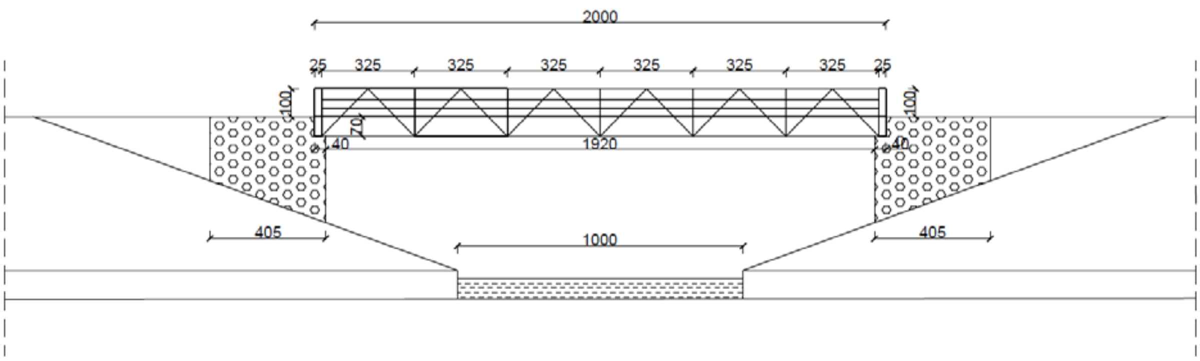


Figure no. 3.5. Basic scheme of side view of the footbridge

Figure no. 3.6. Barrier of the footbridge

Figure no. 3.7. Bottom view of the deck

3.5 INSPIRATION

The footbridge which I was inspired by was a footbridge located in Cracow city, district Ruczaj, over a “Wilga” River whose pattern I decided to follow.

Figure no. 3.8. Side view of the footbridge

Figure no. 3.9. Front view of the footbridge

This footbridge gave me an insight, what are the basic dimensions of particular elements. As my alternative footbridge had really similar dimensions, I was able to assume dimensions elements.

Basic dimensions:

- Length: 28m
- Width: 3.20m
- Arch diameter: 275mm
- Arch elevation: 3.5m
- Girder height: 400mm
- Hanger diameter: 30mm

Figure no. 3.10. Hanger-arch connection

Figure no. 3.11. Hanger-girder fixing

3.6 TECHNICAL DESCRIPTION OF THE STRUCTURE

The designed footbridge is a steel arch footbridge with the deck of 20m theoretical span. The deck consists of side girders suspended on bearing arches, transverse beams and longitudinal beams of steel slab deck. The side girders represent the ties of steel deck slab. The side girders are I beams HEB 400. The bearing arches are designed from circular pipes of 244.5mm diameter and 10mm thickness. The rise of the arch equals 5.0m. Transverse beams are I beams HEB 200. Transverse beams are joined with longitudinal beams which are made of I beams IPE 120. On longitudinal beams a steel plate is located of 12mm thickness constituting slab deck. The deck is suspended on bearing arches by hangers of 20mm diameter. The bracings are made of circular pipes of 193.7mm diameter and 10mm thickness. The design of abutments were not performed in this work, they were assumed as currently existing ones.

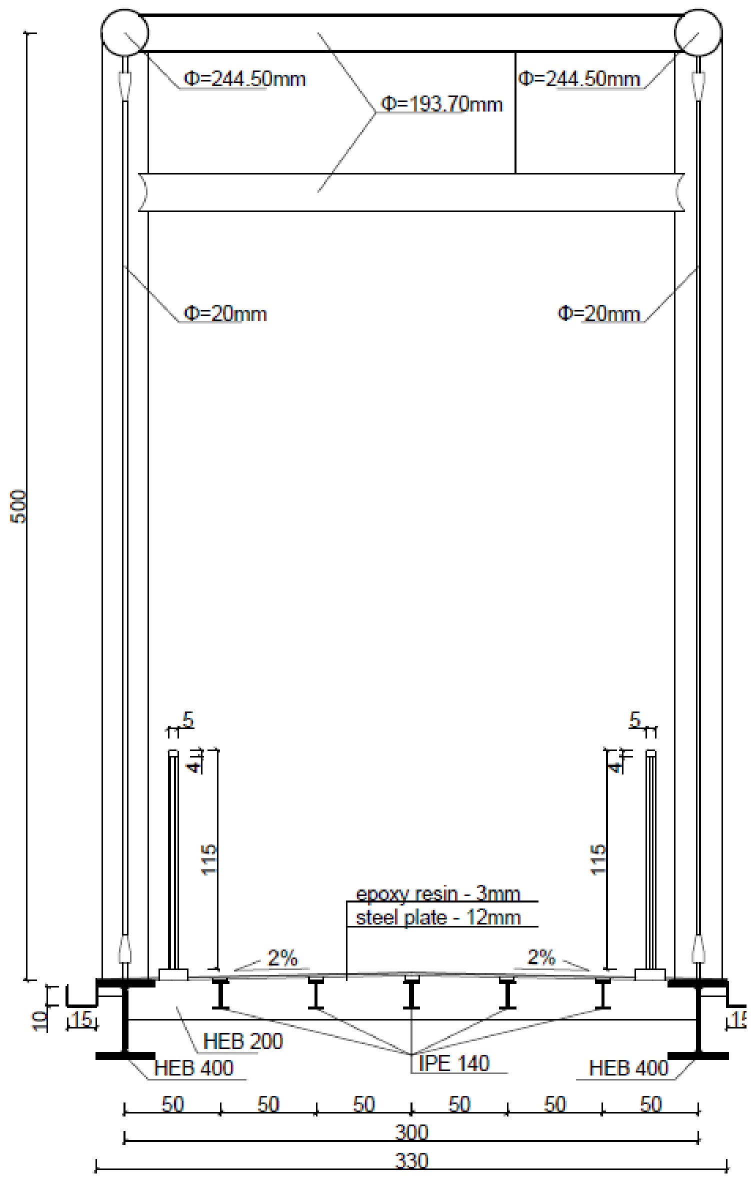


Figure no.3.12. Cross-section of the footbridge

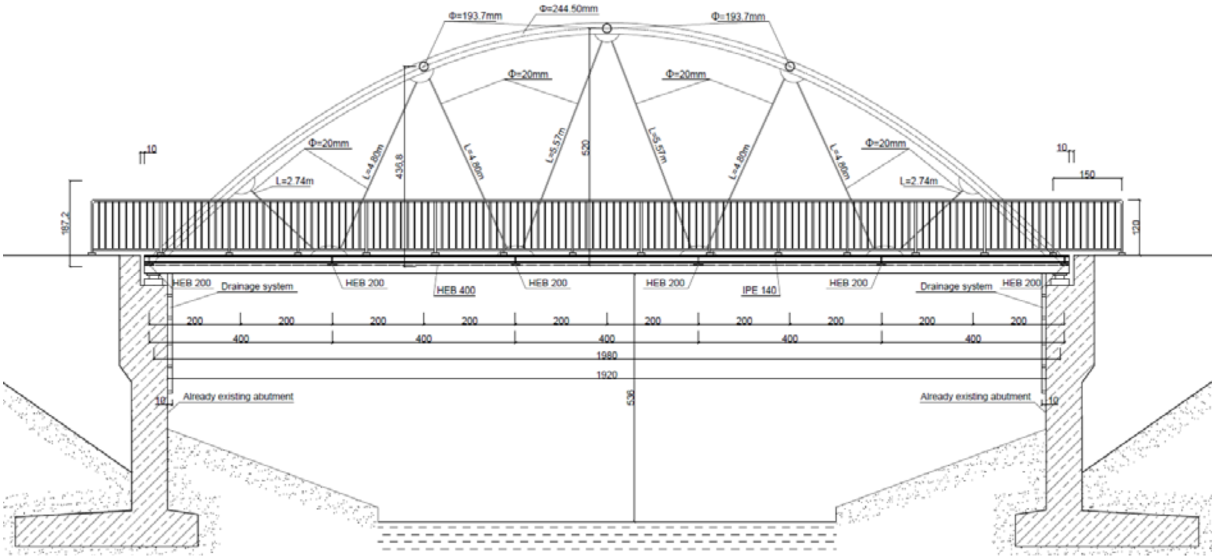


Figure no.3.13. Longitudinal section of the footbridge

Surface layer has been assumed as the epoxy resin layer of 3mm thickness. A modular dilatations equal to 100mm have been added at the end of the footbridge. A drainage system has been included with slope of 2% and gutter diameter of 100mm. Elastomeric bearings have been chosen.

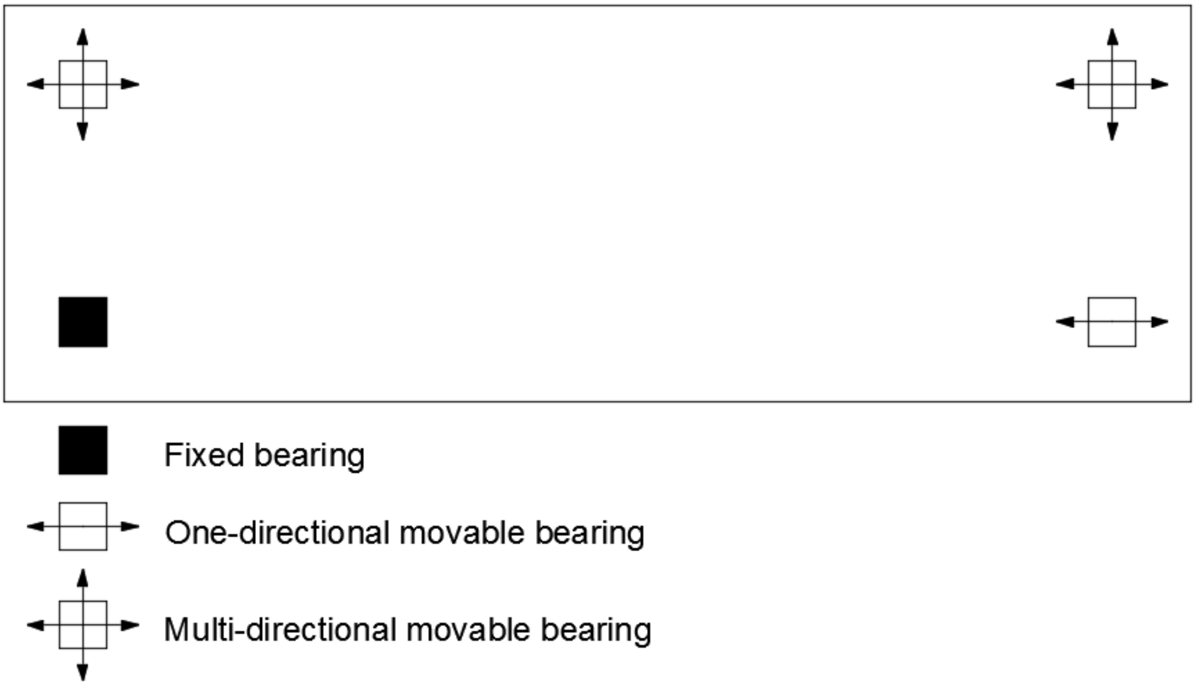


Figure no.3.14. Bearing scheme

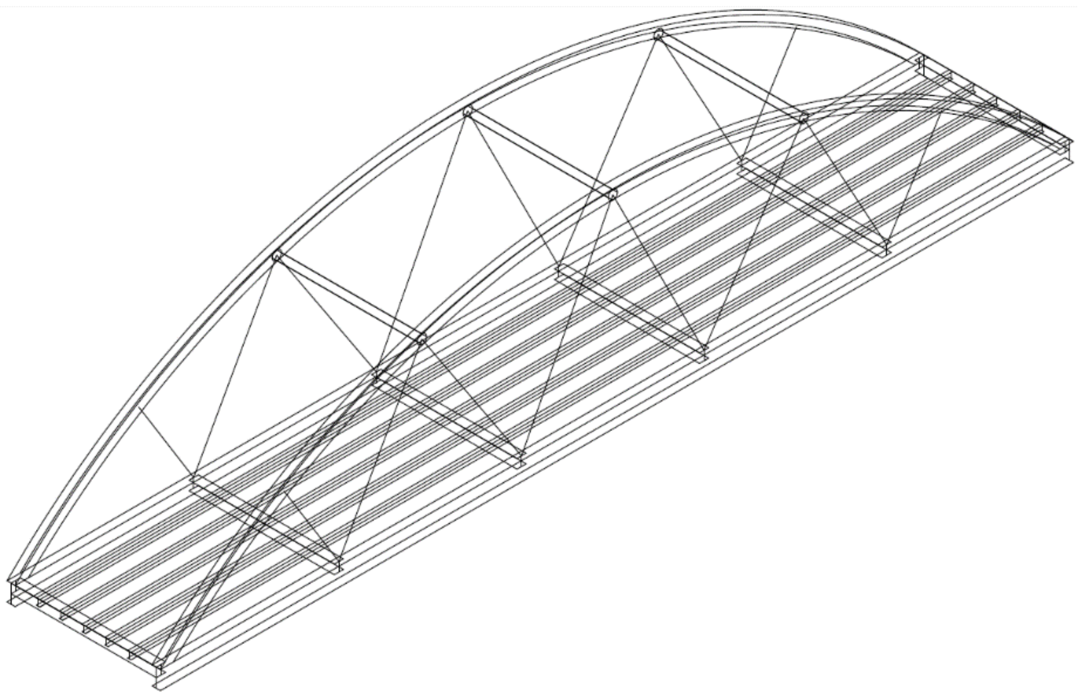
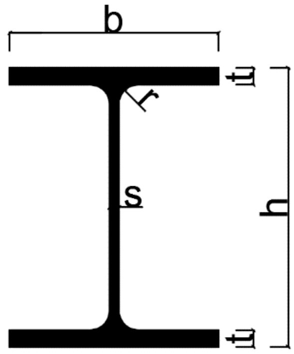


Figure no. 3.15. Conceptual form of the structure

3.7 DESCRIPTION OF STRUCTURAL ELEMENTS

3.7.1 MAIN GIRDERS

Table no. 3.1. HEB 400 section specifications.

Name	Symbol	Value	Unit	Scheme
height	h	400	mm	
width	b	300	mm	
web thickness	s	13.5	mm	
flange thickness	t	24	mm	
radius	r	27	mm	
area	A	197.8	cm^2	
weight	m	158	kg/m	
y-y moment of inertia	ly	57680	cm^4	
y-y plastic coefficient	Wpl.y	3232	cm^3	
z-z moment of inertia	lz	10820	cm^4	
z-z plastic coefficient	Wpl.z	1104	cm^3	

Basic information:

- number of elements: 2
- section of element: HEB 400
- material of element: Steel S355
- length of element: 20m
- distance to each other: 3m

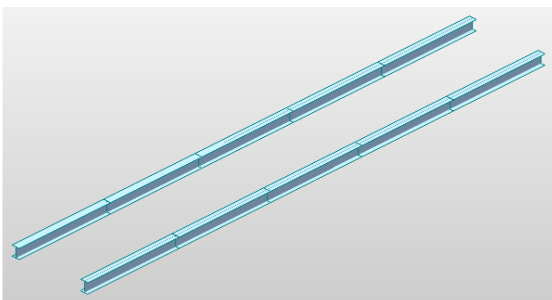
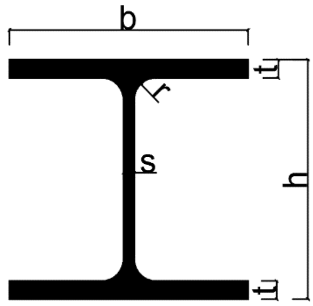


Figure no. 3.16. Girders visualization

3.7.2 TRANSVERSE BEAMS

Table no. 3.2. HEB 200 section specifications.

Name	Symbol	Value	Unit	Scheme
height	h	200	mm	
width	b	200	mm	
web thickness	s	9	mm	
flange thickness	t	15	mm	
radius	r	18	mm	
area	A	78.1	cm^2	
weight	m	62.5	kg/m	
y-y moment of inertia	ly	5696	cm^4	
y-y plastic coefficient	Wpl.y	642.5	cm^3	
z-z moment of inertia	lz	2003	cm^4	
z-z plastic coefficient	Wpl.z	304.8	cm^3	

Basic information:

- number of elements: 6
- section of element: HEB 200
- material of element: Steel S355
- length of element: 3m
- distance to each other: 4m

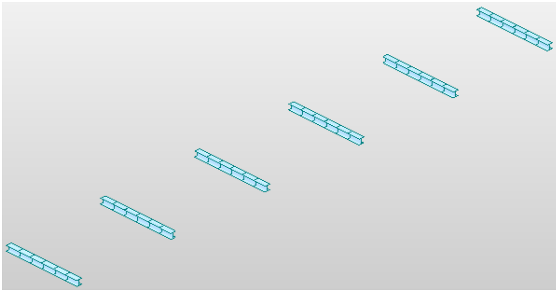
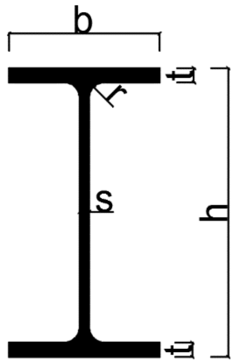


Figure no. 3.17. Transverse beams visualization

3.7.3 LONGITUDINAL BEAMS

Table no. 3.3. HEB 400 section specifications.

Name	Symbol	Value	Unit	Scheme
height	h	140	mm	
width	b	73	mm	
web thickness	s	4.7	mm	
flange thickness	t	6.9	mm	
radius	r	7	mm	
area	A	16.4	cm^2	
weight	m	13.1	kg/m	
y-y moment of inertia	ly	541	cm^4	
y-y plastic coefficient	Wpl.y	88.3	cm^3	
z-z moment of inertia	lz	44.9	cm^4	
z-z plastic coefficient	Wpl.z	19.3	cm^3	

Basic information:

- number of elements: 5
- section of element: IPE 140
- material of element: Steel S355
- length of element: 20m
- distance to each other: 0.5m

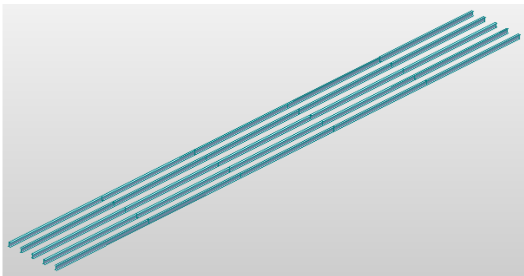
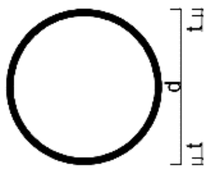


Figure no. 3.18. Longitudinal beams visualization

3.7.4 ARCHES

Table no. 3.4. Arch section specifications.

Name	Symbol	Value	Unit	Scheme
diameter	d	244.5	mm	
outer diameter	t	10	mm	
area	A	73.67	cm^2	
moment of inertia	I	5073.15	cm^4	
section modulus	W	414.98	cm^3	
plastic section modulus	Wpl	550.24	cm^3	

Basic information:

- number of elements: 2
- section of an element: Tube
- material of an element: Steel S355
- length of an element: 23.17m
- distance to each other: 3m
- rise of the arch: 5.2m

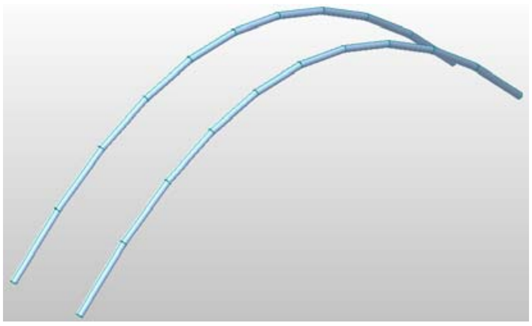


Figure no. 3.19. Arches visualization

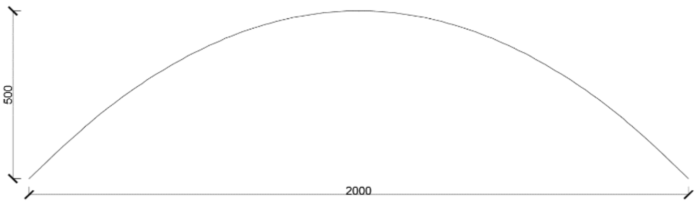
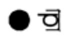


Figure no. 3.20. Dimensions of the arch.

3.7.5 HANGERS

Table no. 3.5. Hanger section specification.

Name	Symbol	Value	Unit	Scheme
diameter	d	20	mm	
area	A	3.14	cm^2	

Basic information:

- number of elements: 16
- section of an element: Hanger 20mm
- material of an element: Steel S355
- length of elements: 52.44m

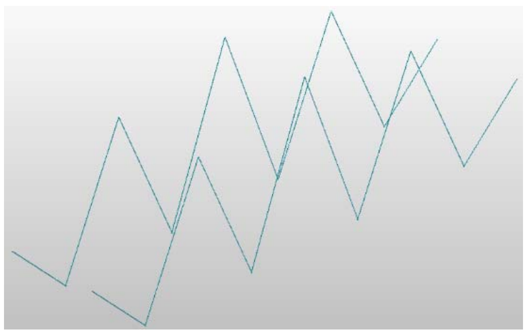


Figure no. 3.21. Bracings visualization

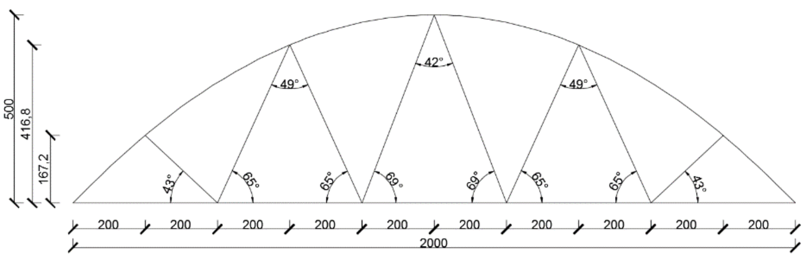
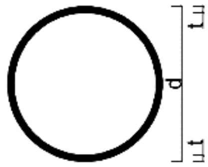


Figure no. 3.22. Dimensions of hangers.

3.7.6 BRACINGS

Table no. 3.6. Bracings section specifications.

Name	Symbol	Value	Unit	Scheme
diameter	d	193.7	mm	
outer diameter	t	10	mm	
area	A	57.71	cm^2	
moment of inertia	I	2441.59	cm^4	
section modulus	W	252.10	cm^3	
plastic section modulus	Wpl	337.79	cm^3	

Basic information:

- number of elements: 3
- section of an element: Tube
- material of an element: Steel S355
- length: 3m

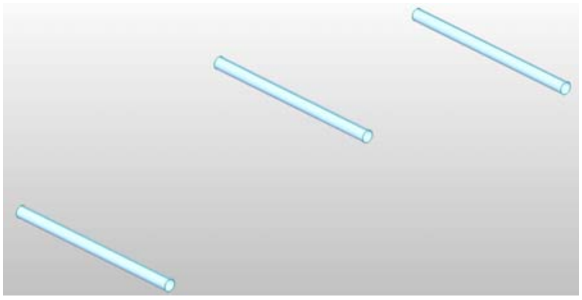


Figure no. 3.23. Bracings visualization

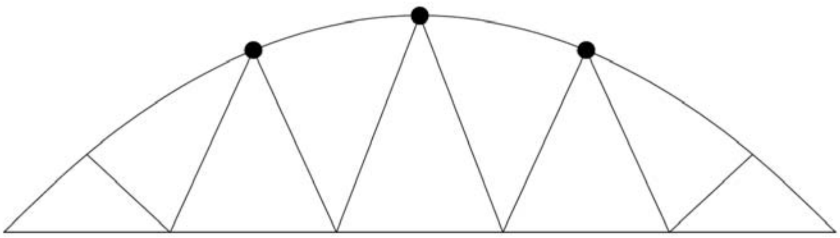


Figure no. 3.24. Placement of bracings.

3.7.7 SLAB

Table no. 3.7. Slab section specifications.

Name	Symbol	Value	Unit	Scheme
height	h	120	mm	

Basic information:

- number of elements: 1
- section of an element: Plate
- material of an element: Steel S355
- length of an element: 20m
- width of an element: 3m

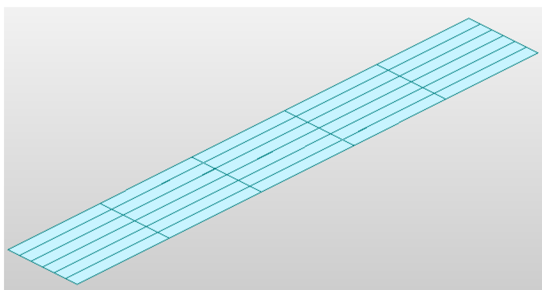


Figure no. 3.25. Slab visualization

3.8 DECK

An orthotropic deck has been chosen.

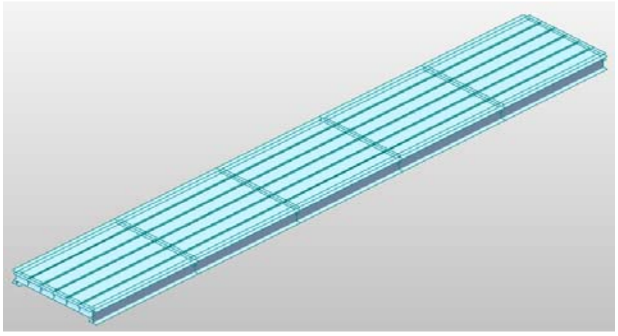


Figure no. 3.26. Top side deck view

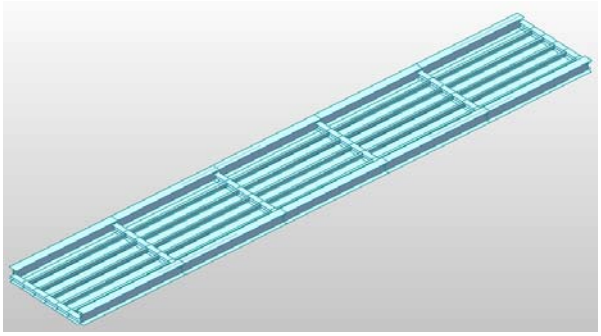


Figure no. 3.27. Bottom side deck view

There was a danger that deck might not be correctly made because of the fact that centers of gravity of main girders, transverse beams, longitudinal beams and plate were different, meaning that these centers of gravity were levitating without any connections. It was decided to create a fictitious connectors to be sure that the every element of the deck is working with other elements of the deck. Fictitious connectors were made of HEB400 section and a user defined S355 steel material. Values of modulus of elasticity and weight density were changed to be nearly 0 in order for the fictitious connectors to not interfere with structure.

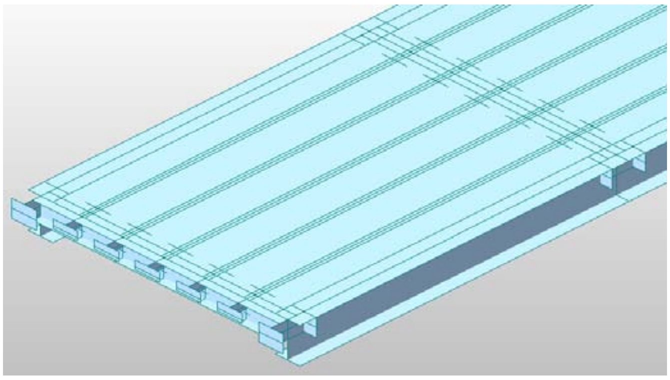


Figure no. 3.28. Fictitious connectors

Final model visualization.

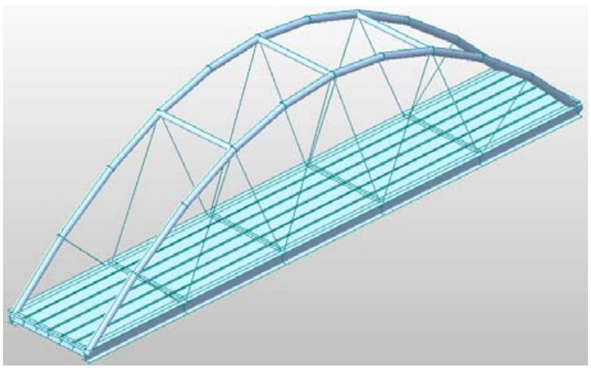


Figure no. 3.29. Footbridge model

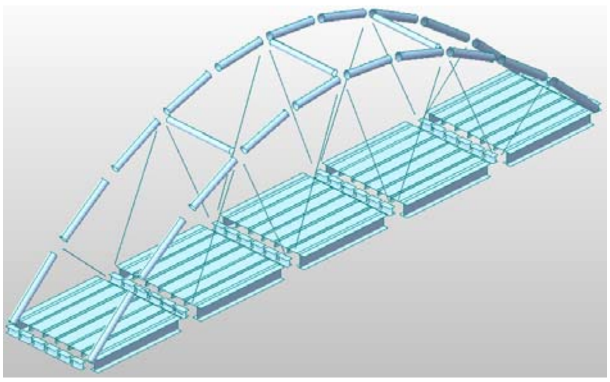


Figure no. 3.30. Separated elements of model

4. LOAD MODELS

4.1 DEAD LOAD OF STRUCTURAL ELEMENTS

There was no need to calculate dead load of individual structural elements because Midas Civil 2014 software has an option to assign self-weight to all elements at once.

4.2 DEAD LOAD OF NON-STRUCTURAL ELEMENTS

Barriers were chosen to ensure the safety for pedestrians as well as for cyclists. Total height equals to 1.2m, spacing of posts to 1.5 and total span to 3m.

$$g_{b.ch} := 8.3\ \frac{kg}{m} \cdot g = 0.081\ \frac{kN}{m}$$

$$g_{b.d} := g_{b.ch} \cdot 1.35 = 0.11\ \frac{kN}{m}$$

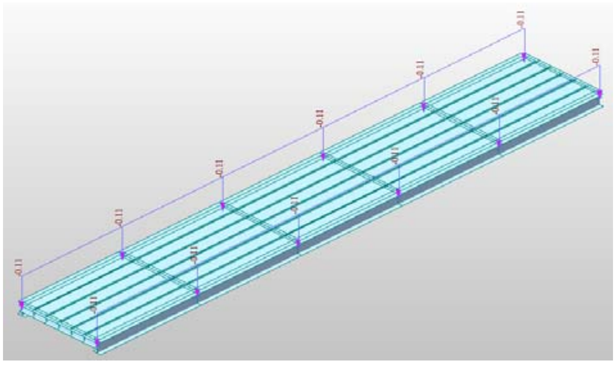


Figure no. 4.1. Barrier load model on deck

4.3 CROWD LOAD

Crowd load may be defined by the National Annex or for the particular project.

I method

$$q_{fk.ch} := 5\ \frac{kN}{m^2}$$

$$q_{fk.d} := 1.35 \cdot q_{fk.ch} = 6.75\ \frac{kN}{m^2}$$

But when it is not specified, the recommended value equals:

II method

$L := 20$ total length of footbridge

$$q_{fk.ch} := 2.0 + \frac{120}{L+30} = 4.4$$

$q_{fk.ch} := 4.5\ \frac{kN}{m^2}$ assumed characteristic value of crowd load

$$q_{fk.d} := 1.35 \cdot q_{fk.ch} = 6.075\ \frac{kN}{m^2}$$

4.4 SERVICE VEHICLE LOAD

When there is no permanent obstacle that prevents a vehicle from entering into the footbridge, there is a need of considering a service vehicle.

According to PN 1991-2-2002, two-axle load of group of 80kN and 40kN have been assumed with wheel base of 3m and a track of 1.3m with a square areas of side equal to 0.2m

$$Q_{SV1} := 80\ kN$$

$$Q_{SV2} := 40\ kN$$

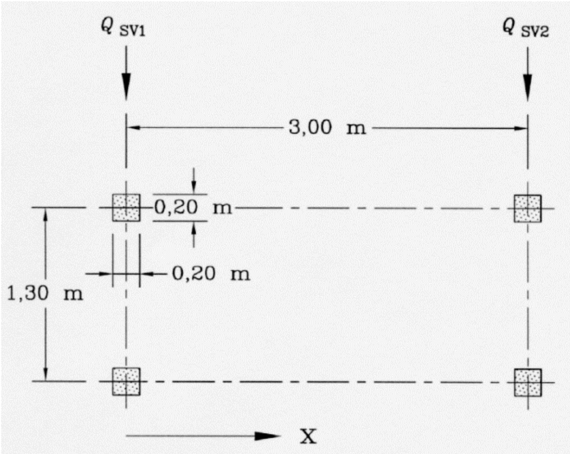


Figure no. 4.2. Vehicle load model [E2]

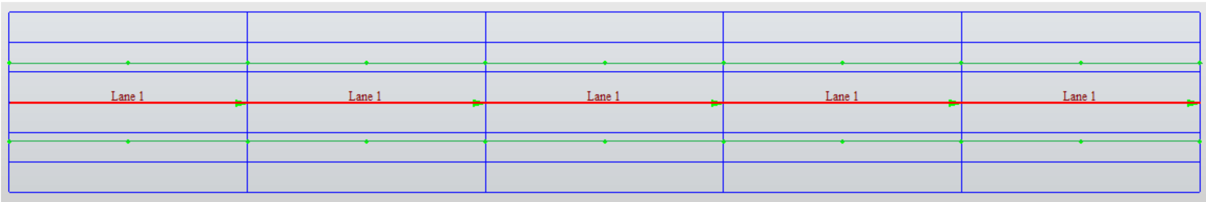


Figure no. 4.3. Lane top view of the deck

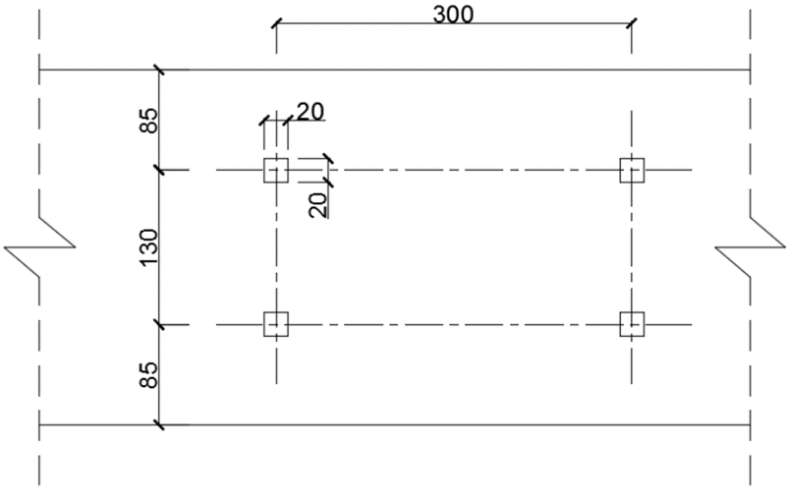


Figure no. 4.4. Vehicle load model on designing footbridge

5. CALCULATIONS

5.1 GROUPS OF LOADS

Load type		Vertical forces		Horizontal forces
Load system		Uniformly distributed load	Service vehicle	
Groups of loads	gr1	q_{fk}	0	Q_{flk}
	gr2	0	Q_{serv}	Q_{flk}

Figure no. 5.1. Definition of groups of loads [E2]

Uniformly distributed load is load due to crowd. Horizontal force is assumed to 0 because wind has not been taken into consideration.

5.2 BENDING MOMENT DIAGRAM DUE TO SELF-WEIGHT

In order to establish the worst case scenario of loadings for particular elements, a bending moment diagram due to self-weight was generated using a Midas Civil 2014 software. Selective elements which are bended the most, will be chosen to evaluate resistances of these elements.

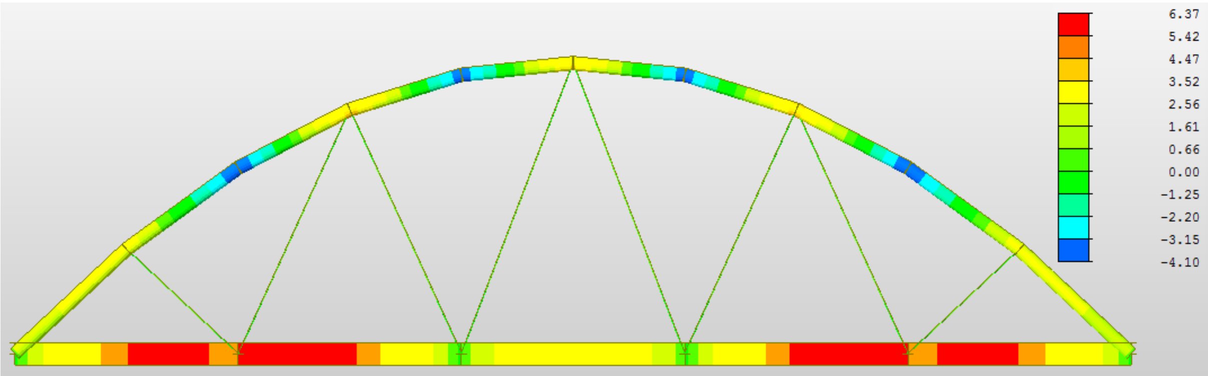


Figure no. 5.2. Diagram of bending moment due to self-weight

It was assumed that influence line analysis will be performed for arch beam, hanger and girder. For the slab, longitudinal beam and transverse beam, the worst case scenario will be assigned the same as in the girder analysis of influence lines.

5.3 ARCH CALCULATIONS

5.3.1 ARCH INFLUENCE LINES

Influences lines due to bending moments in the arch.

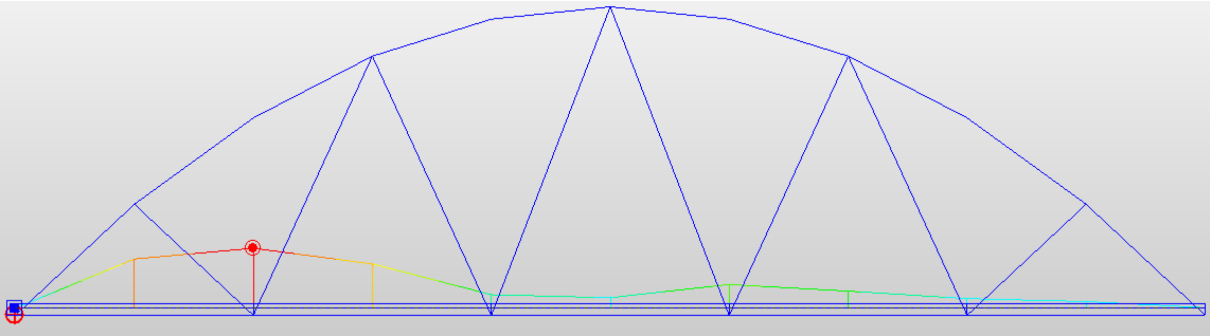


Figure no. 5.3. Arch influence line at the beginning of the arch

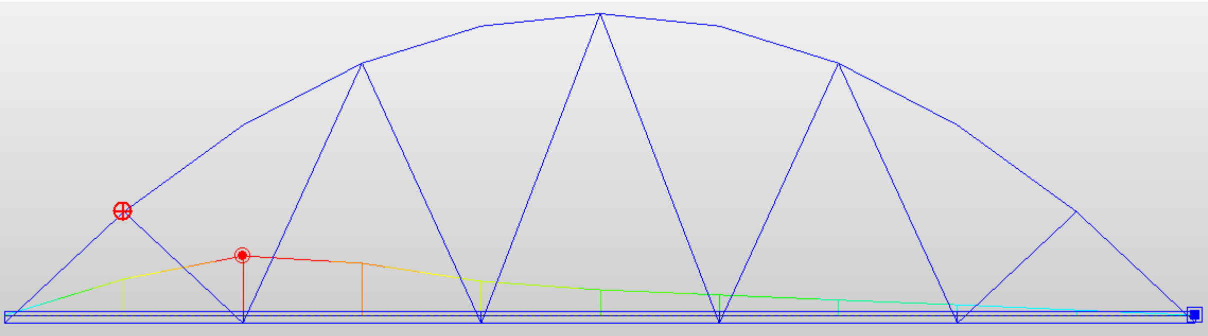


Figure no. 5.4. Arch influence line in the 1/10 of the length of the span

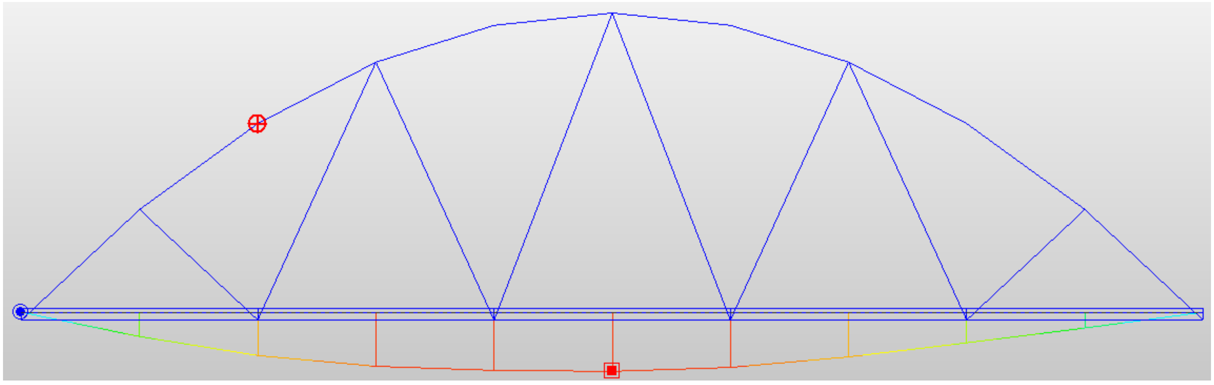


Figure no. 5.5. Arch influence line in the 1/5 of the length of the span

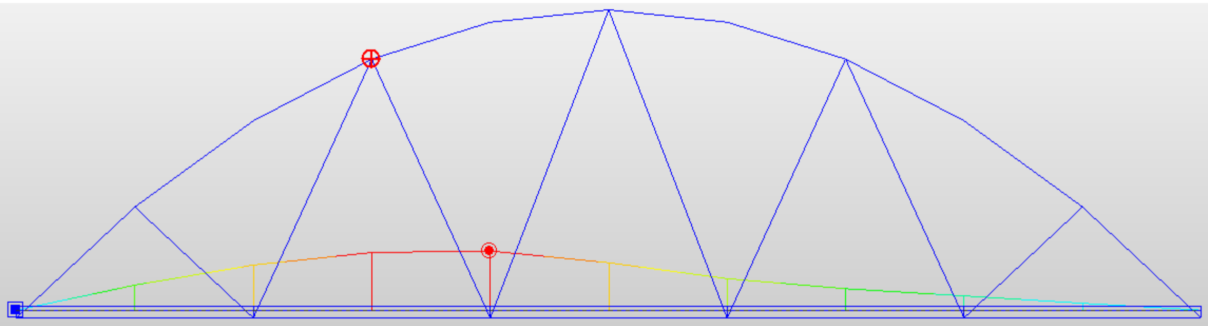


Figure no. 5.6. Arch influence line in the 3/10 of the length of the span

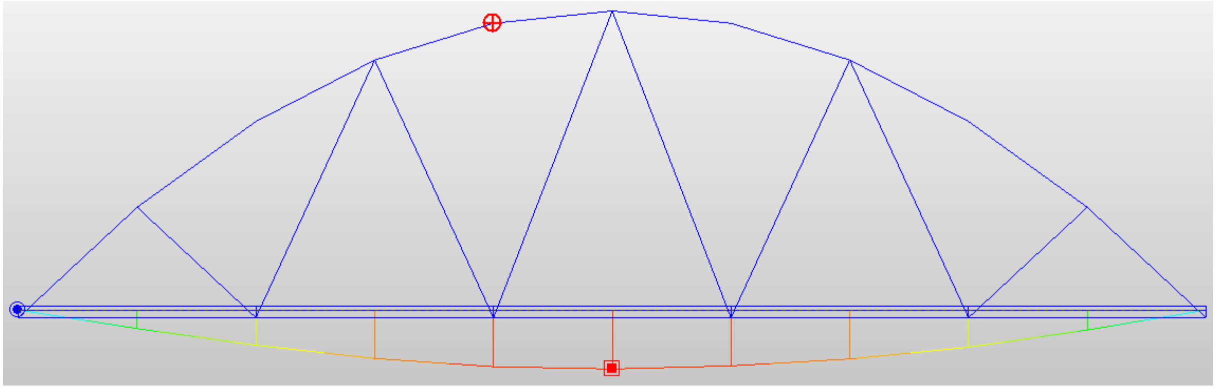


Figure no. 5.7. Arch influence line in the 2/5 of the length of the span

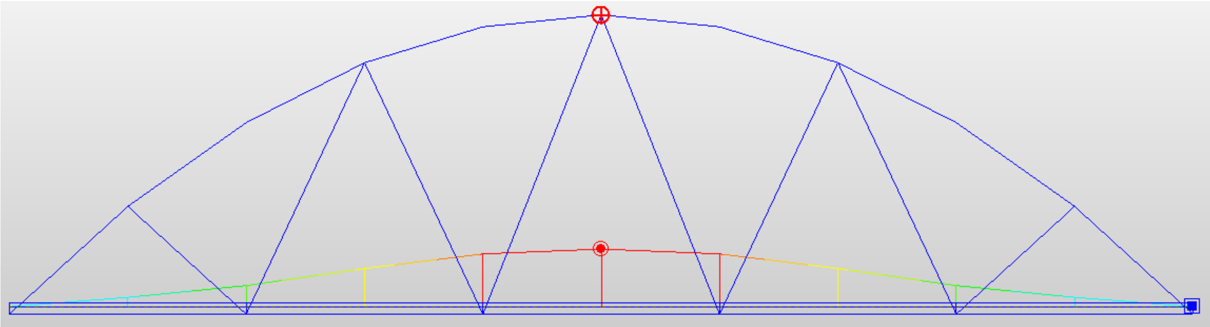


Figure no. 5.8. Arch influence line in the 1/2 of the length of the span

Influence lines due to longitudinal forces in the arch.

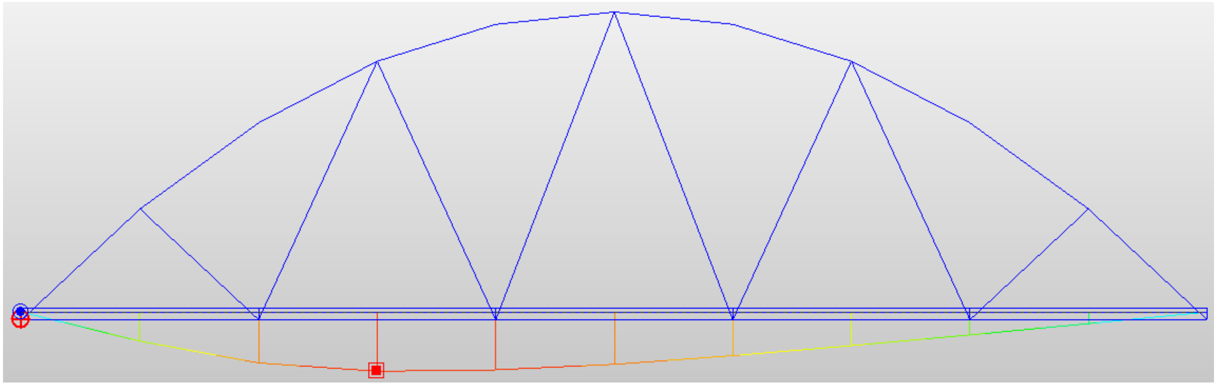


Figure no. 5.9. Arch influence line at the beginning of the arch.

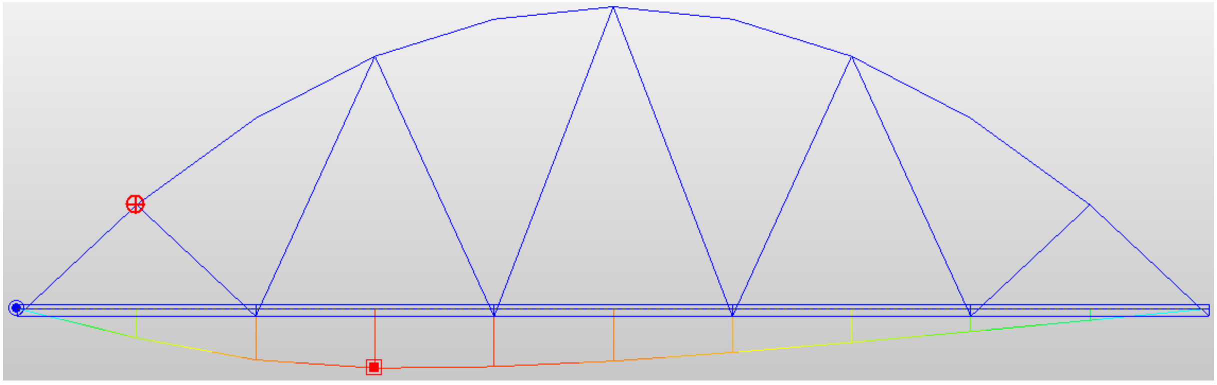


Figure no. 5.10. Arch influence line in the 1/5 of the length of the span

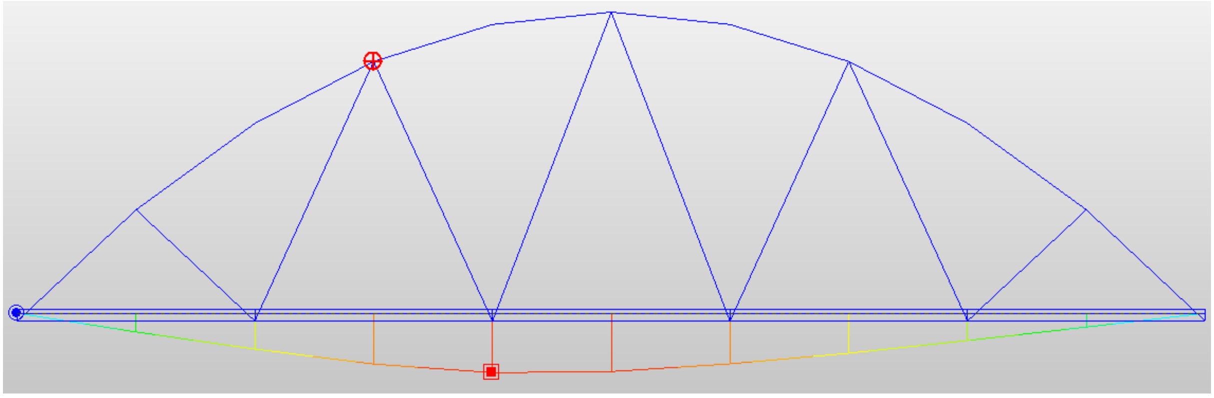


Figure no. 5.11. Arch influence line in the 3/10 of the length of the span. Chosen to calculation

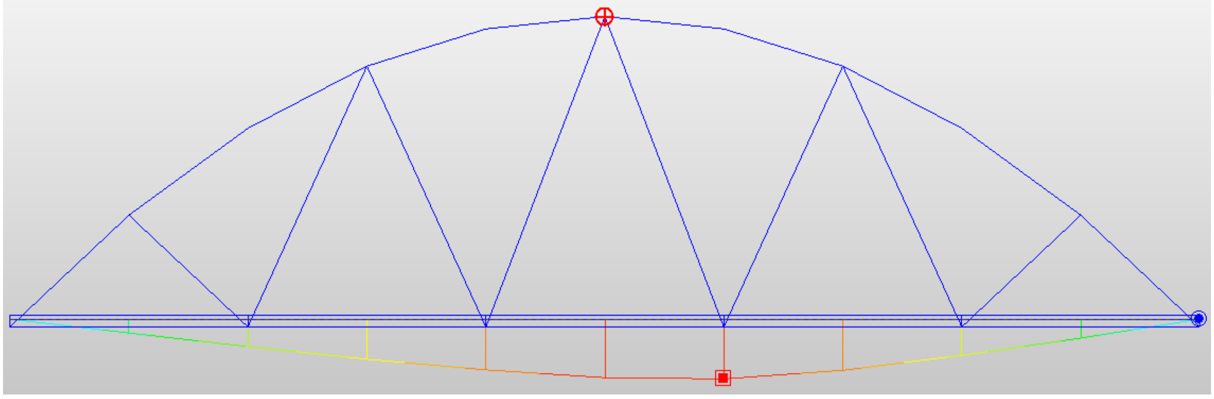


Figure no. 5.12. Arch influence line in the 1/2 of the length of the span

5.3.2 MAXIMAL FORCES DIAGRAMS

According to arch influence line in the Figure no. 5.11, the worst load case scenario has been prepared.

Two combinations were taken into consideration:

- 1st combination - dead load + crowd load
- 2nd combination - dead load + service vehicle load

Forces in 2nd combination were greatest hence were taken for further calculations.

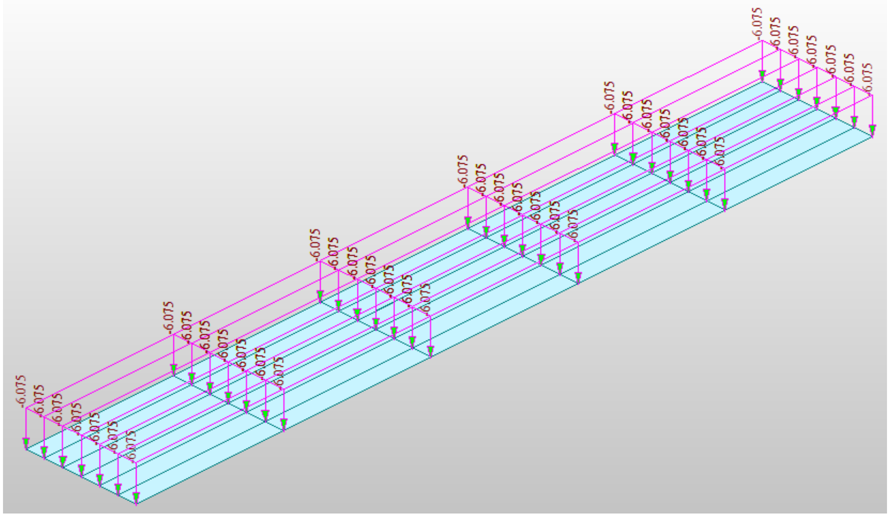


Figure no. 5.13 Crowd load model

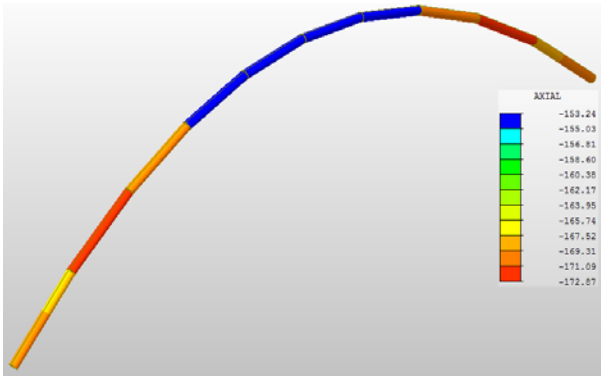


Figure no. 5.14. Diagram of axial forces

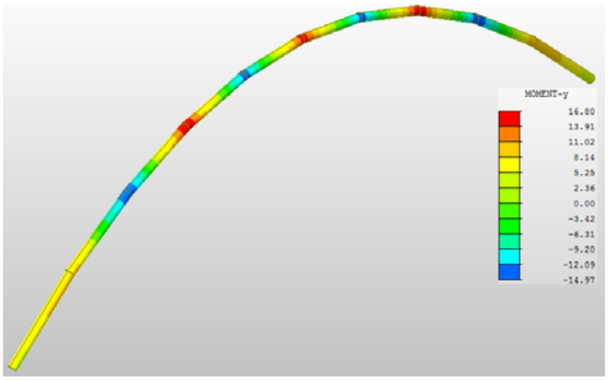


Figure no. 5.15. Diagram of bending moments.

Table no. 5.1. Arch maximal forces.

Arch			
No.	Load case	Fx [kN]	My [kNm]
Combination 2	Dead Load + Vehicle Load	172.87	16.80

5.3.3 ARCH BEARING CAPACITY

5.3.3.1 DETERMINATION OF THE CLASS OF CROSS-SECTION

According to [E4] (6.3.3):

$\gamma_s := 78.5\ \dfrac{\mathrm{kN}}{\mathrm{m}^3}$ - weight of steel

$f_y := 355\ \mathrm{MPa}$ - yield strength for S355

$f_u := 510\ \mathrm{MPa}$ - ultimate strength for 355

$E_s := 210\ \mathrm{GPa}$ - longitudinal young modulus

$G_s := 810\ \mathrm{GPa}$ - transversial young modulus

$A := 7367\ \mathrm{mm}^2$ - cross-section area

$I := 5073.15\ \mathrm{cm}^4$ - moment of inertia

$W := 414.98\ \mathrm{cm}^3$ - section modulus

$W_{pl} := 550.24\ \mathrm{cm}^3$ - plastic section modulus

$d := 244.5\ \mathrm{mm}$ - diameter of the arch

$t := 10\ \mathrm{mm}$ - thickness of the arch

$\varepsilon := \sqrt{\dfrac{235\ \text{MPa}}{f_y}} = 0.814$ according to PN-EN-1993-1-1 (table 5.2)

$\dfrac{d}{t} \le 50\ \varepsilon^2 = 1$

Cross-section has been identified as **Class 1.**

5.3.3.2 BENDING WITH AXIAL COMPRESSION

$N_{Ed} := 172.87\ \text{kN}$ - design value of the compression force

$M_{y.Ed} := 16.80\ \text{kN} \cdot \text{m}$ - design value of bending moment about y-y axis

$\delta := 5\ \text{cm}$ - maximal member deflection along the member, value taken from Midas Civil 2014 software

$L := 23.17\ \text{m}$ - length of the arch

$s := \dfrac{L}{2} = 11.585\ \text{m}$ - half of the length of the arch

$f := 5.\ 2\ \text{m}$ - height of the arch (rise)

$l := 20\ \text{m}$ - length of arch projection

$\dfrac{f}{l} = 0.5 \qquad \dfrac{f}{l} > 0.1$

$\beta := 0.56$ according to [E4] Table D.4 for parabolic form

$N_{cr} := \left(\dfrac{\pi}{\beta \cdot s}\right)^2 \cdot E_s \cdot I = 2498.208\ \text{kN}$ - elastic critical force according to [E4] D3.1

$C_{mi.0} := 1 + \left(\dfrac{\pi^2\ E_s \cdot I \cdot \text{abs}(\delta)}{L^2 \cdot \text{abs}(M_{y.Ed})} - 1\right) \dfrac{N_{Ed}}{N_{cr}} = 0.971$ - equivalent moment factor

$\lambda := \sqrt{\dfrac{A \cdot f_y}{N_{cr}}} = 1.023$ - for class 1,2 or 3 cross sections

$\alpha := 0.21$ - imperfection factor for hot finished hollow sections

$\Phi := 0.5\ \left(1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right) = 1.11$

$\chi := \dfrac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}} = 0.649 \qquad \chi \le 1 = 1$ - reduction factor

$\gamma_{M0} := 1 \qquad \gamma_{M1} := 1.1$

$$N_{c.Rd} := \frac{A \cdot f_y}{\gamma_{M0}} = 2615.285\ \mathrm{kN}$$

$$N_{Rk} := N_{c.Rd}$$

$$M_{c.Rd} := \frac{W_{pl} \cdot f_y}{\gamma_{M0}} = 195.335\ \mathrm{kN \cdot m}$$

$$M_{Rk} := M_{c.Rd}$$

$\Delta M_{y.Ed} := 0$ - moment due to the shift of the centroidal axis

$$\frac{N_{Ed}}{\frac{\chi \cdot N_{Rk}}{\gamma_{M1}}} + \frac{C_{mi.0} \cdot \left(M_{y.Ed} + \Delta M_{y.Ed}\right)}{\frac{M_{Rk}}{\gamma_{M1}}} = 20.384\%$$

$$\frac{N_{Ed}}{\frac{\chi \cdot N_{Rk}}{\gamma_{M1}}} + \frac{C_{mi.0} \cdot \left(M_{y.Ed} + \Delta M_{y.Ed}\right)}{\frac{M_{Rk}}{\gamma_{M1}}} \le 0.9 = 1$$

condition satisfied

5.4 HANGER CALCULATIONS

5.4.1 HANGER INFLUENCE LINES

Influence lines due to longitudinal forces in selective hangers.

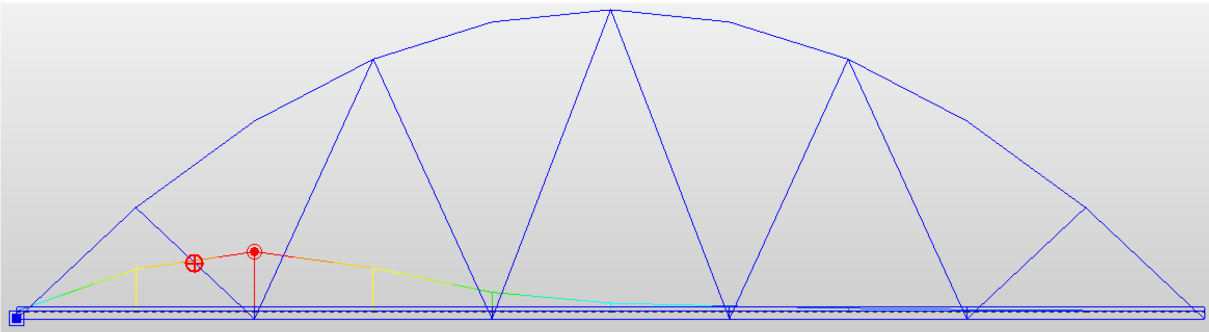


Figure no. 5.16. First hanger influence line

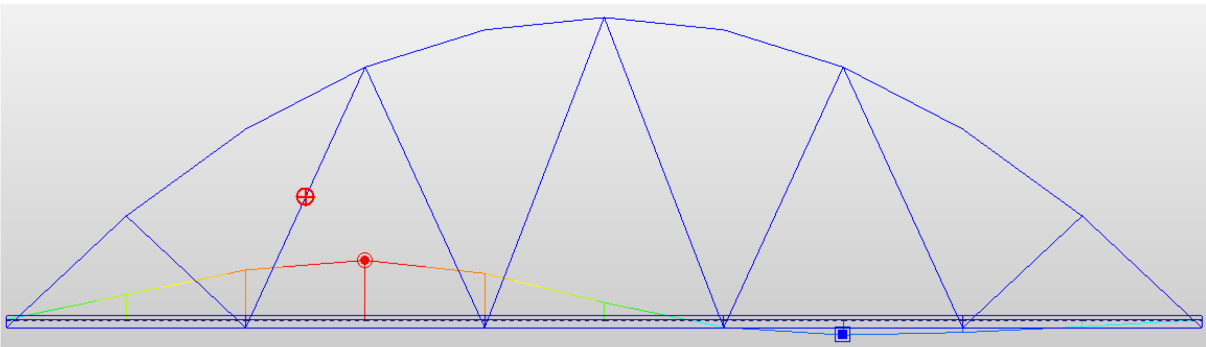


Figure no. 5.17. Second hanger influence line. Chosen to calculation

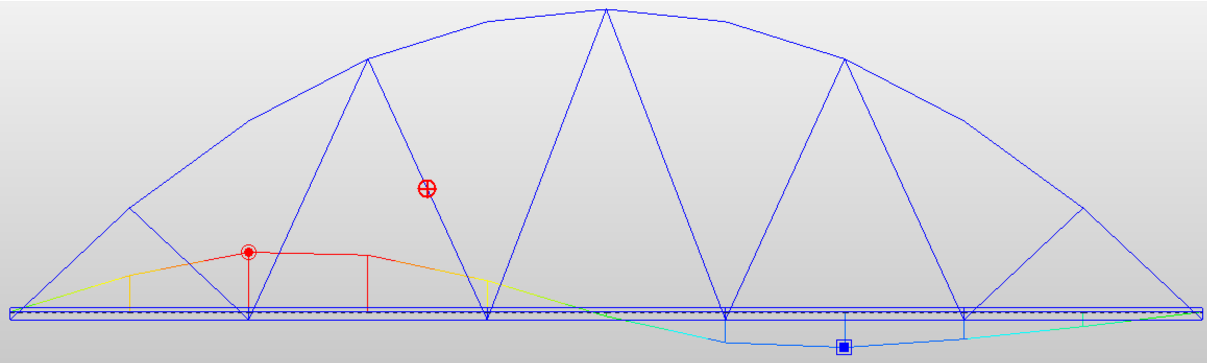


Figure no. 5.18. Third hanger influence line

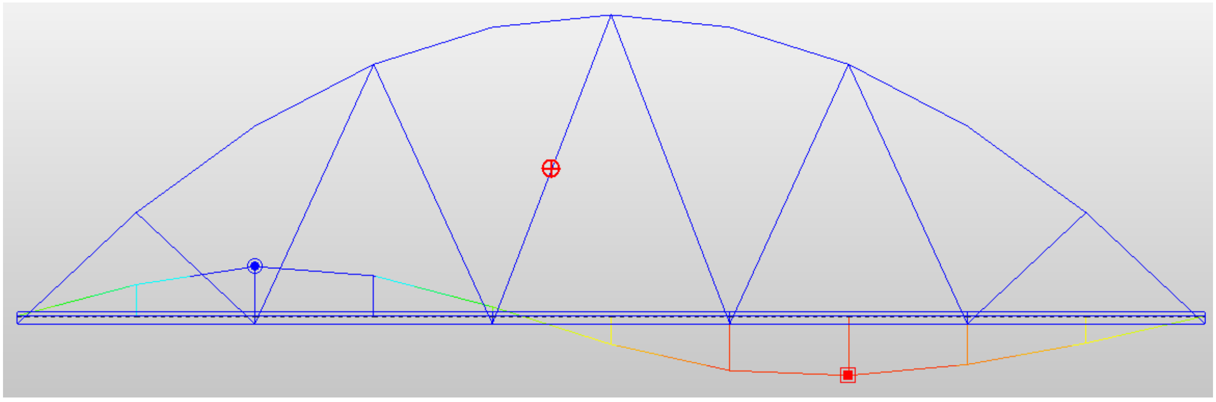


Figure no. 5.19. Fourth hanger influence line

5.4.2 MAXIMAL FORCES DIAGRAMS

According to hanger influence line in the Figure no. 5.17, the worst load case scenario has been prepared.

Three combinations were taken into consideration:

- 1st combination - dead load + crowd load 1
- 2nd combination – dead load + crowd load 2
- 3rd combination - dead load + service vehicle load

Forces in 3rd combination were greatest hence were taken for further calculations.

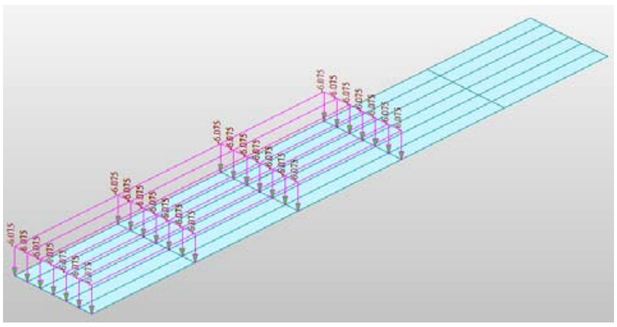


Figure no. 5.20. Crowd load model 1

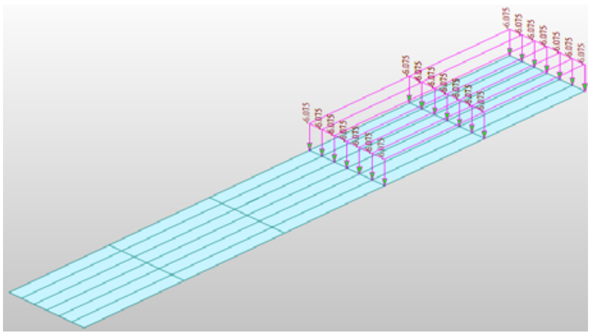


Figure no. 5.21. Crowd load model 2

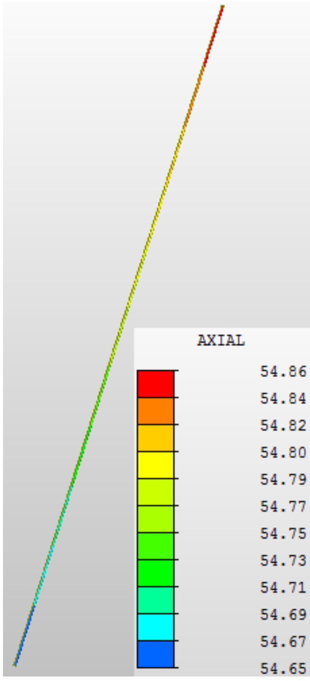


Figure no. 5.22. Diagram of axial forces

Table no. 5.2. Hanger maximal forces.

Hanger		
No.	Load case	Fx [kN]
Combination 3	Dead Load + Vehicle Load	54.86

5.4.3 HANGER BEARING CAPACITY

5.4.3.1 TENSION

According to [E3] (6.2.3):

$f_y := 355\ \text{MPa}$ - yield strength for S355

$\gamma_{M0} := 1$

$A := 3.14\ \text{cm}^2$ - area of cross-section

$N_{t.pl.Rd} := A \cdot \dfrac{f_y}{\gamma_{M0}} = 111.47\ \text{kN}$ - design plastic resistance of the gross cross-section

$N_{t.Ed} := 54.86\ \text{kN}$

$\dfrac{N_{t.Ed}}{N_{t.pl.Rd}} = 49.215\%$ **condition satisfied**

5.5 GIRDER CALCULATIONS

5.5.1 GIRDER INFLUENCE LINES

Influences lines due to bending moments in the girder.

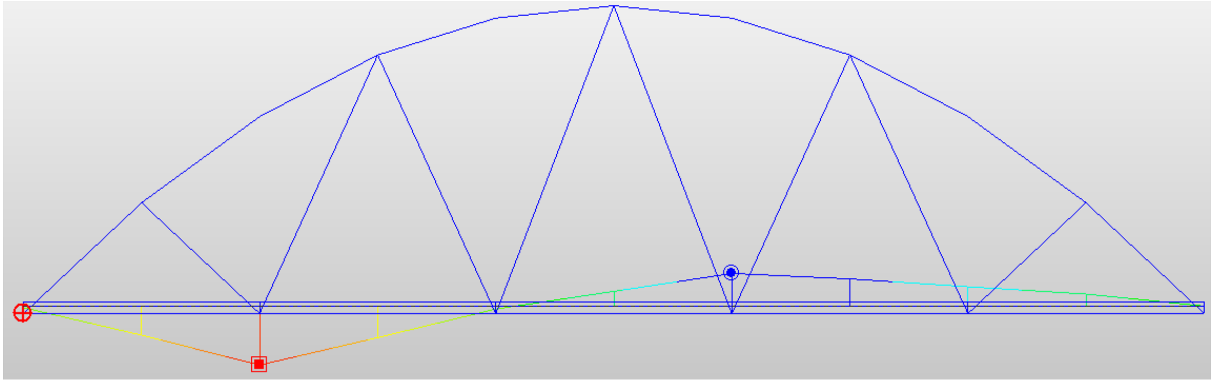


Figure no. 5.23. Girder influence line at the beginning of the span

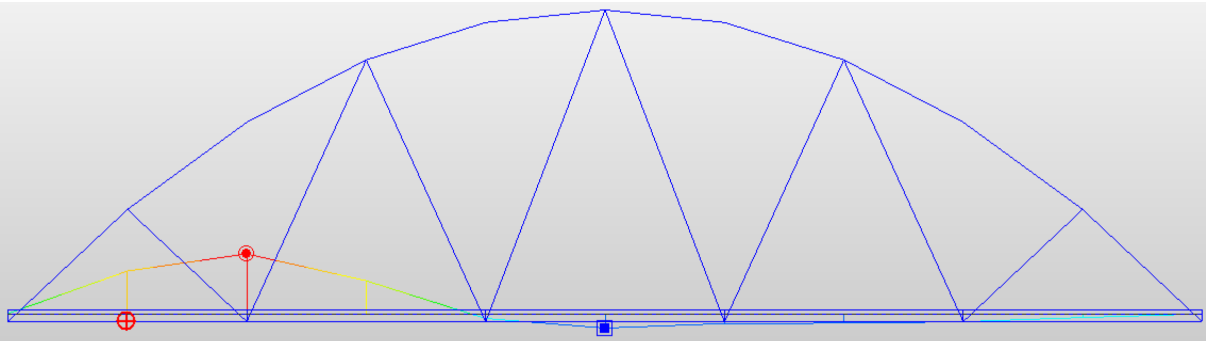


Figure no. 5.24. Girder influence line in the 1/10 of the length of the span

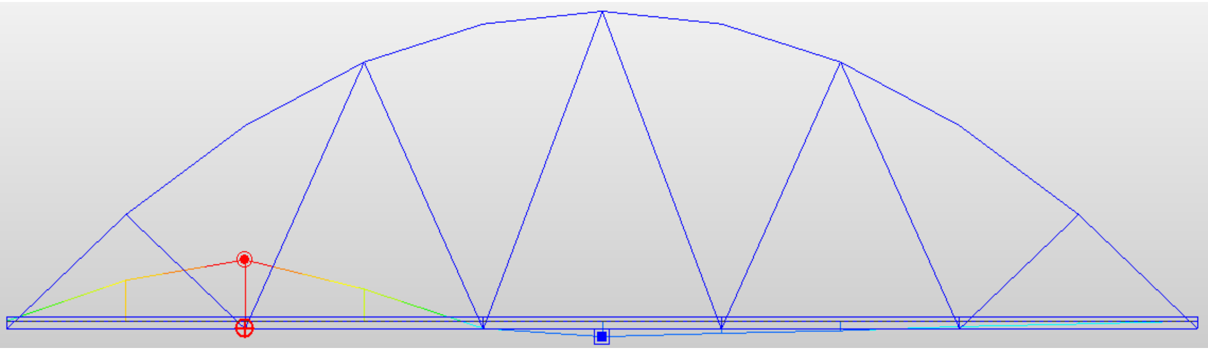


Figure no. 5.25. Girder influence line in the 1/5 of the length of the span - chosen to calculations

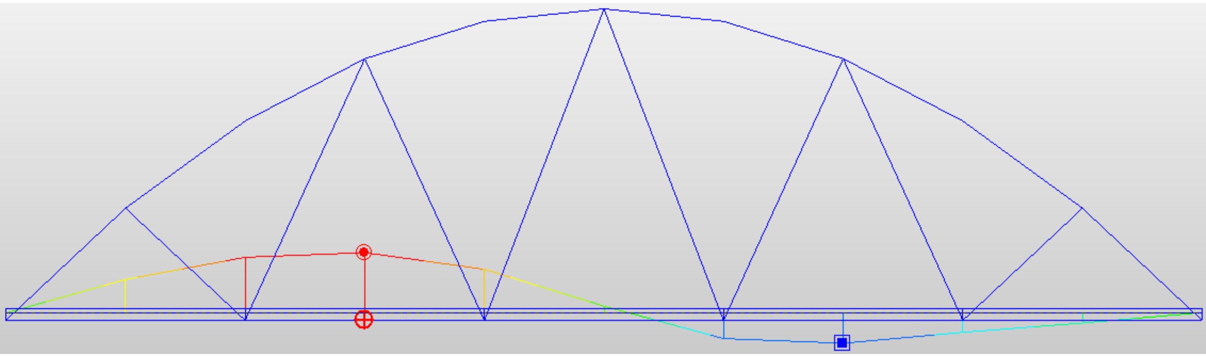


Figure no. 5.26. Girder influence line in the 3/10 of the length of the span

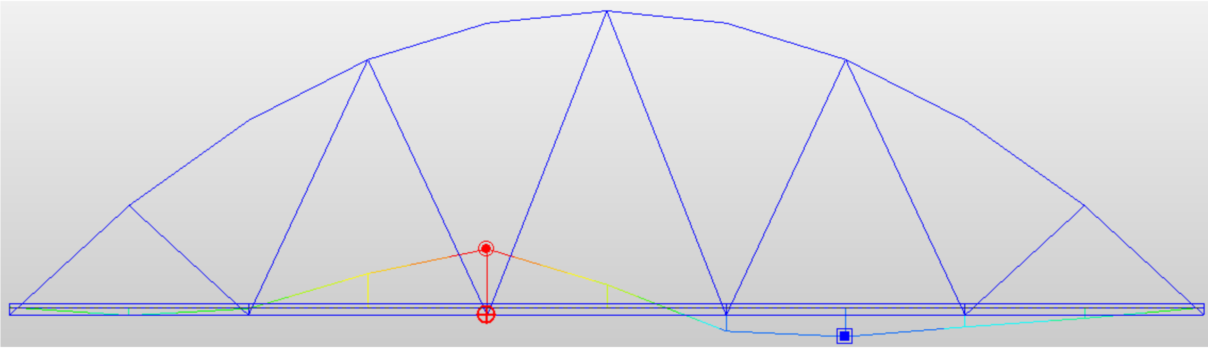


Figure no. 5.27. Girder influence line in the 4/10 of the length of the span

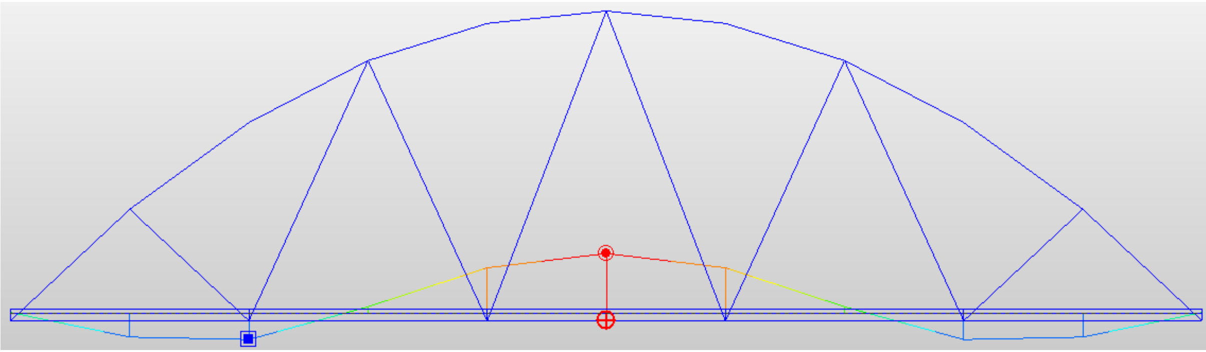


Figure no. 5.28. Girder influence line in the 1/2 of the length of the span

Influences lines due to longitudinal forces in the girder.

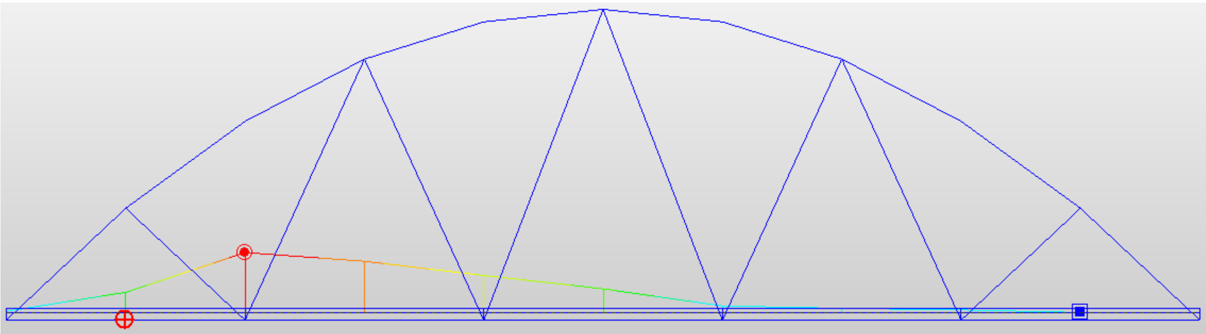


Figure no. 5.29. Girder influence line in the 1/10 of the length of the span

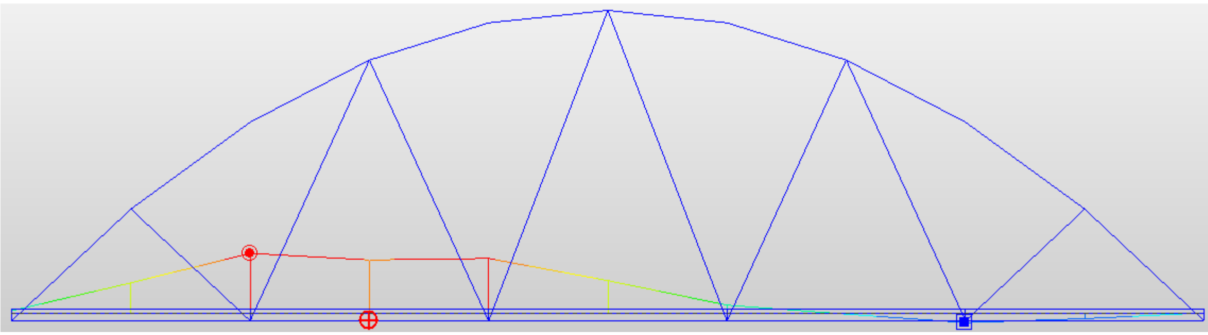


Figure no. 5.30. Girder influence line in the 3/10 of the length of the span

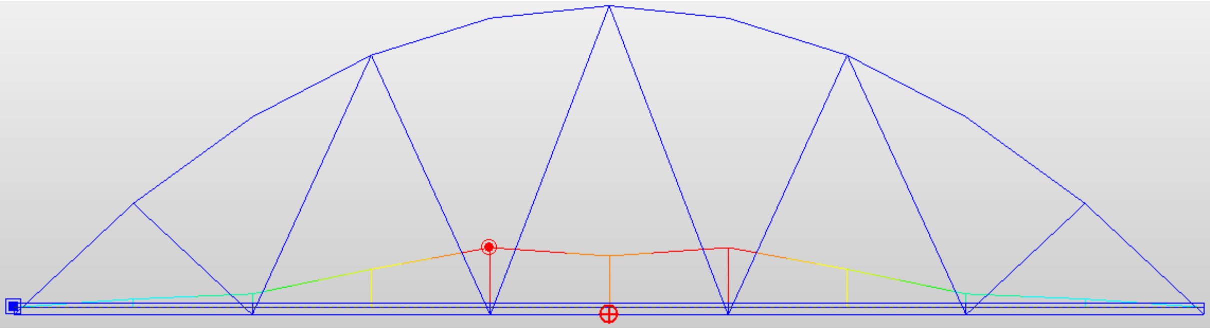


Figure no. 5.31. Girder influence line in the 1/2 of the length of the span

5.5.2 MAXIMAL FORCES DIAGRAMS

According to girder influence line in the Figure no. 5.25, the worst load case scenario has been prepared.

Three combinations were taken into consideration:

- 1st combination - dead load + crowd load 1
- 2nd combination – dead load + crowd load 2
- 3rd combination - dead load + service vehicle load

Forces in 3rd combination were greatest hence were taken for further calculations.

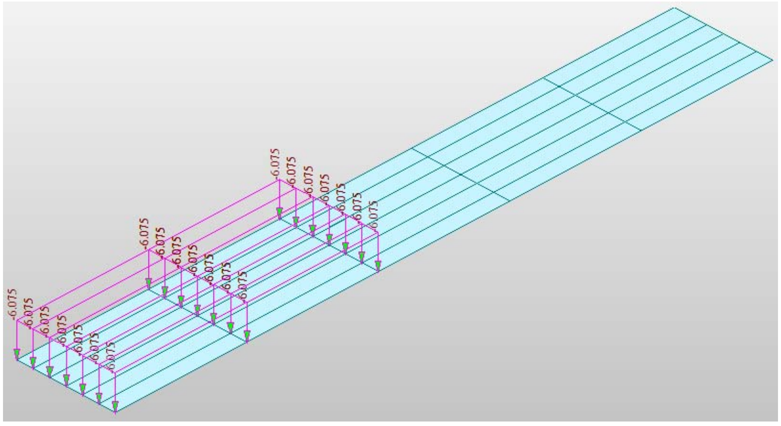


Figure no. 5.32. Crowd load model 1

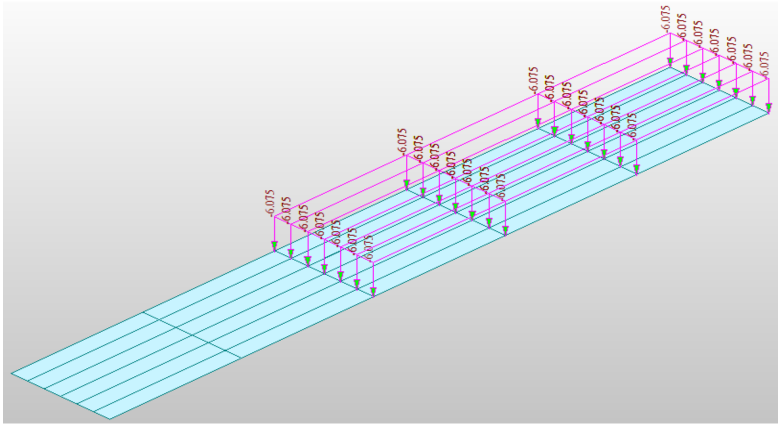


Figure no. 5.33. Crowd load model 2

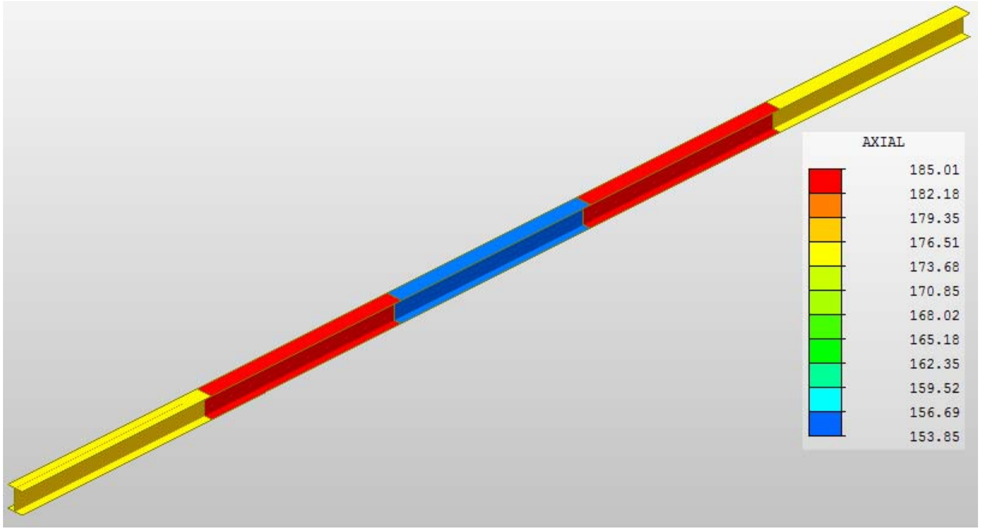


Figure no. 5.34. Diagram of axial forces

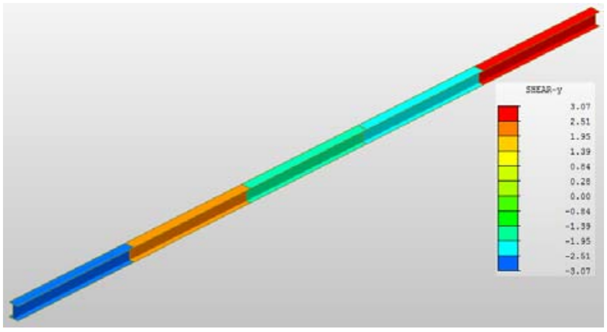


Figure no. 5.35. Diagram of shear forces about y

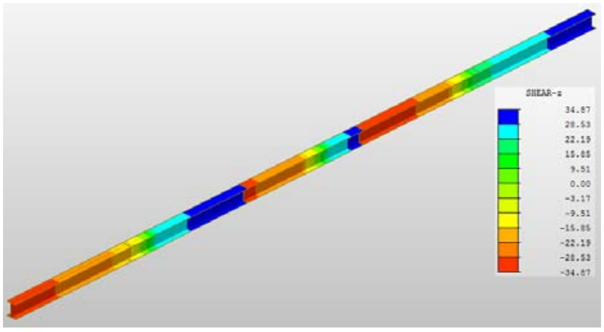


Figure no. 5.36. Diagram of shear forces about z

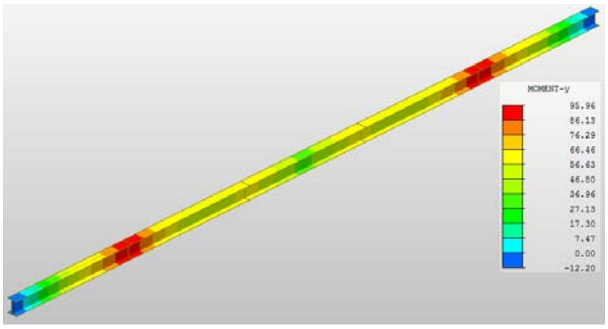


Figure no. 5.37. Diagram of bending moments about y

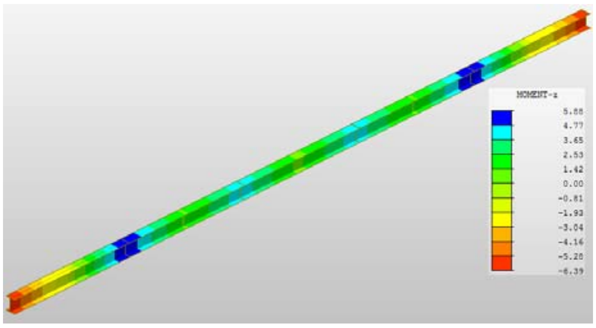


Figure no. 5.38. Diagram of bending moments about z

Table no. 5.3. Girder maximal forces.

Longitudinal Beam						
No.	Load case	Fx [kN]	Fy [kN]	Fz [kN]	My [kNm]	Mz [kNm]
Combination 3	Dead Load + Vehicle Load	185.01	3.07	34.87	95.96	6.39

5.5.3 GIRDER BEARING CAPACITIES

5.5.3.1 DETERMINATION OF THE CLASS OF CROSS-SECTION

$h := 400\ \text{mm}$	- total height
$b := 300\ \text{mm}$	- width
$t_w := 13.5\ \text{mm}$	- web thickness
$t_f := 24\ \text{mm}$	- flange thickness
$r := 27\ \text{mm}$	- radius
$A := 197.8\ \text{cm}^2$	- area of cross-section
$W_{pl.y} := 3232\ \text{cm}^3$	- y-y plastic coefficient
$W_{pl.z} := 1104\ \text{cm}^3$	- z-z plastic coefficient

$c := h - 2 \cdot t_f - 2 \cdot r = 298\ \text{mm}$ according to Table 5.2, PN-EN 1993-1-1

$f_y := 355\ \text{MPa}$

$\varepsilon := \sqrt{235\ \frac{\text{MPa}}{f_y}} = 0.814$ according to Table 5.2, PN-EN 1993-1-1

$\alpha := 0.5$

$\gamma_{M0} := 1$

$\frac{c}{t_w} \le \frac{36\ \varepsilon}{\alpha} = 1$ Cross-section has been defined as **Class 1.**

5.5.3.2 TENSION

According to [E3] (6.2.3):

$N_{t.Rd} := \frac{A \cdot f_y}{\gamma_{M0}} = 7021.9\ \text{kN}$ - design plastic resistance

$N_{Ed} := 185.01\ \text{kN}$

$\frac{N_{Ed}}{N_{t.Rd}} = 2.635\%$ **contidition satisfied**

5.5.3.3 SHEARING

According to [E3] (6.2.3)

Shear about y axis

$A_{v.y} := A - 2 \cdot b \cdot t_f + \left(t_w + 2 \cdot r\right) \cdot t_f = 70\ \text{cm}^2$ - shear area for rolled I and H sections

$V_{pl.y.Rd} := \dfrac{A_{v.y}}{\gamma_{M0}} \cdot \dfrac{f_y}{\sqrt{3}} = 1434.715\ \text{kN}$ - design plastic shear resistance

$V_{y.Ed} := 3.07\ \text{kN}$

$\dfrac{V_{y.Ed}}{V_{pl.y.Rd}} = 0.214\%$ **contidition satisfied**

Shear about z axis

$A_{v.z} := A - 2 \cdot b \cdot t_f + \left(t_w + 2 \cdot r\right) \cdot t_f = 70\ \text{cm}^2$ - shear area for rolled I and H sections

$V_{pl.z.Rd} := \dfrac{A_{v.z}}{\gamma_{M0}} \cdot \dfrac{f_y}{\sqrt{3}} = 1434.715\ \text{kN}$ - design plastic shear resistance

$V_{z.Ed} := 34.87\ \text{kN}$

$\dfrac{V_{z.Ed}}{V_{pl.z.Rd}} = 2.43\%$ **contidition satisfied**

5.5.3.4 BENDING

According to [E3] (6.2.5)

Bending about y axis

$M_{pl.y.Rd} := \dfrac{W_{pl.y} \cdot f_y}{\gamma_{M0}} = 1147.36\ \text{kN} \cdot \text{m}$ - design resistance for bending about y axis for cross section 1 or 2

$M_{y.Ed} := 95.96\ \text{kN} \cdot \text{m}$

$\dfrac{M_{y.Ed}}{M_{pl.y.Rd}} = 8.364\%$ **contidition satisfied**

Bending about z axis

$M_{pl.z.Rd} := \frac{W_{pl.z} \cdot f_y}{\gamma_{M0}} = 391.92\ kN \cdot m$ — - design resistance for bending about y axis for cross section 1 or 2

$M_{z.Ed} := 6.39\ kN \cdot m$

$\frac{M_{z.Ed}}{M_{pl.z.Rd}} = 1.63\%$ — **contidition satisfied**

5.5.3.5 BENDING AND AXIAL FORCE

According to [E3] (6.2.9) (4)

$n := \frac{N_{Ed}}{N_{t.Rd}} = 0.026$

$a := \frac{A - 2\ b \cdot t_f}{A} = 0.272$ $\qquad a \leq 0.5 = 1$

$M_{Ny.Rd} := M_{pl.y.Rd} \cdot \frac{1-n}{1-0.5 \cdot a} = 1292.968\ kN \cdot m$ — - reduced design plastic resistance moment about y axis

$M_{Nz.Rd} := M_{pl.z.Rd} \cdot \frac{1-n}{1-0.5 \cdot a} = 441.657\ kN \cdot m$ — - reduced design plastic resistance moment about z axis

$\alpha := 2$ — for I and H sections

$\beta := 5 \cdot n = 0.132$ — for I and H sections

$\left(\frac{M_{y.Ed}}{M_{Ny.Rd}}\right)^{\alpha} + \left(\frac{M_{z.Ed}}{M_{Nz.Rd}}\right)^{\beta} = 57.785\%$ — **contidition satisfied**

$\frac{N_{Ed}}{N_{t.Rd}} + \frac{M_{y.Ed}}{M_{pl.y.Rd}} + \frac{M_{z.Ed}}{M_{pl.z.Rd}} = 12.629\%$ — **contidition satisfied**

5.6 PLATE CALCULATIONS

5.6.1 MAXIMAL FORCES DIAGRAMS

The worst case scenario has been chosen the same as in the girder case. It was assumed that there are 2 most unfavorable combinations for plate. First combination comprises dead load and crowd load situated on the whole deck, and second one involves dead load and service vehicle load.

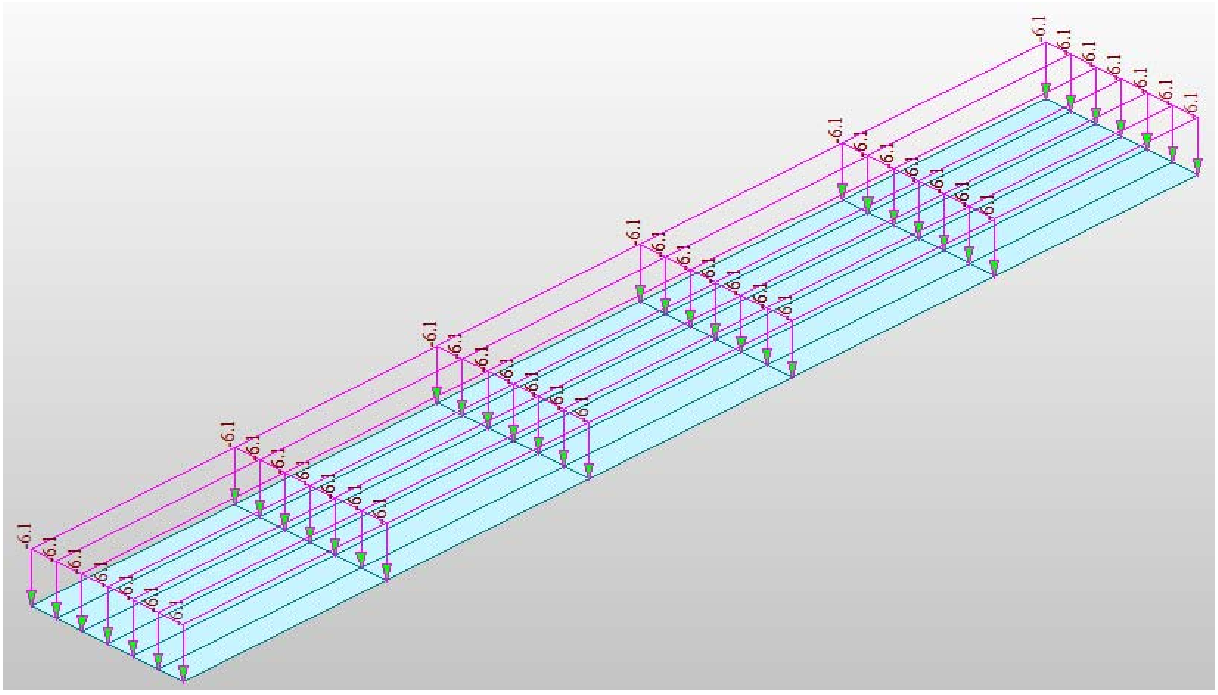


Figure no. 5.39. Crowd load model

1st combination - dead load + crowd load.

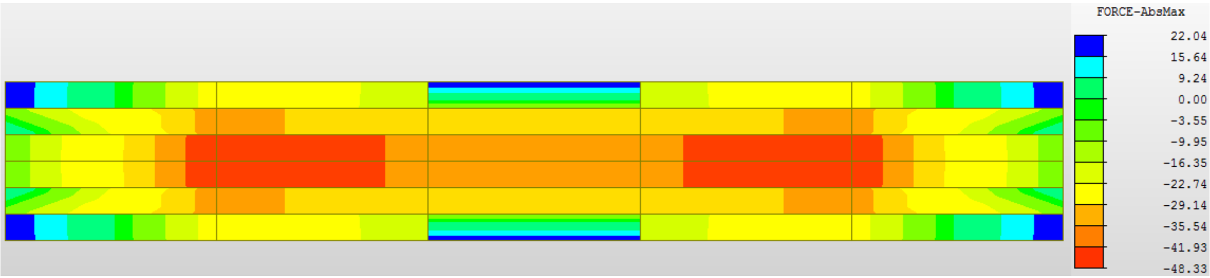


Figure no. 5.40. Diagram of maximal and minimal longitudinal forces in the slab due to 1st combination

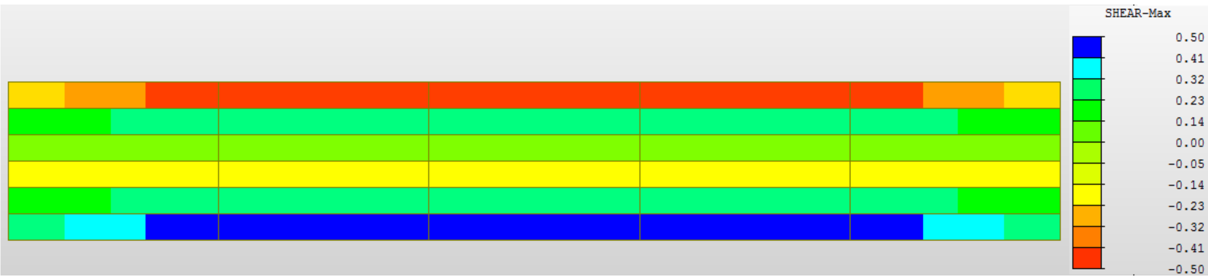


Figure no. 5.41. Diagram of maximal and minimal shear forces in the slab due to 1st combination

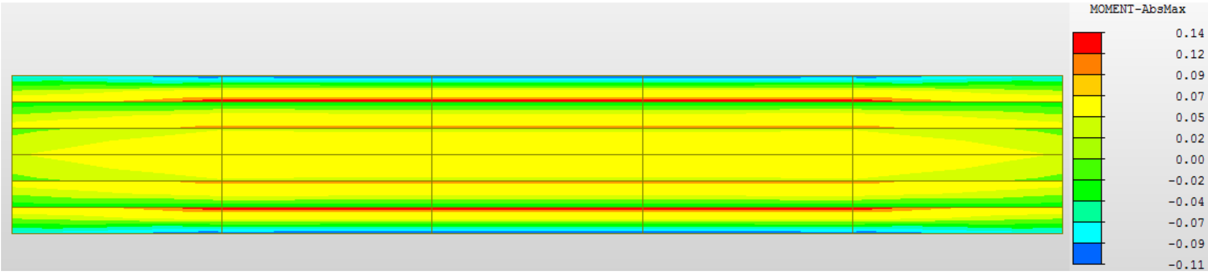


Figure no. 5.42. Diagram of maximal and minimal bending moments in the slab due to 1st combination

2nd combination - dead load + service vehicle load.

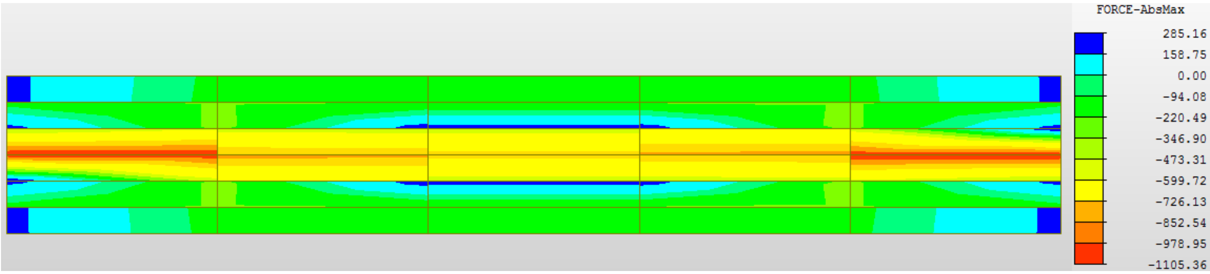


Figure no. 5.43. Diagram of maximal and minimal longitudinal forces in the slab due to 2nd combination

Figure no. 5.44. Diagram of maximal and minimal shear forces in the slab due to 2nd combination

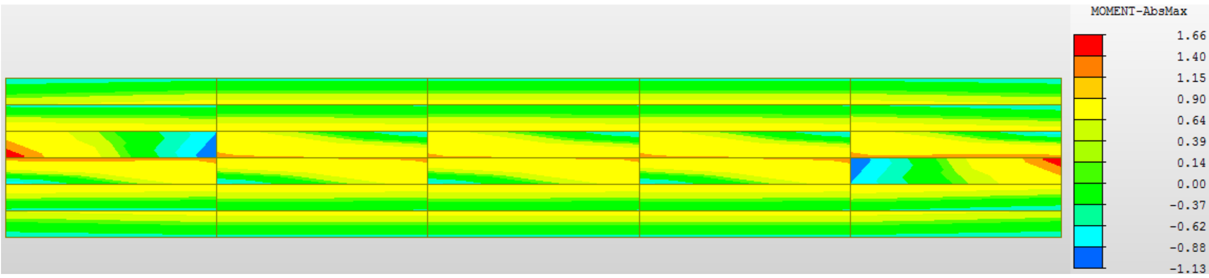


Figure no. 5.45. Diagram of maximal and minimal bending moments in the slab due to 2nd combination

Following values have been taken in further calculations.

Table no. 5.4. Plate maximal forces

Kind of internal force	Maximal value [kN], [kNm]
Longitudinal forces	1105.36
Shear forces	2.05
Bending moments	1.66

5.6.2 PLATE BEARING CAPACITIES

5.6.2.1 DETERMINATION OF THE CLASS OF CROSS-SECTION

$f_y := 335 \text{ MPa}$

$\varepsilon := \sqrt{\dfrac{235 \text{ MPa}}{f_y}} = 0.838$

$b := 50 \text{ cm}$ - distance between ribs

$t := 12 \text{ mm}$ - thickness of the plate

$\alpha := 0.5$

$$Class_{Pl} := \begin{cases} \text{if } \dfrac{b}{t} \le \dfrac{36 \cdot \varepsilon}{\alpha} & \text{"Class 1"} \\ \text{if } \dfrac{36 \cdot \varepsilon}{\alpha} < \dfrac{b}{t} \le \dfrac{41.5 \cdot \varepsilon}{\alpha} & \text{"Class 2"} \end{cases} = \text{"Class 1"}$$

Cross section has been defined as **Class 1.**

5.6.2.2 GEOMETRICAL CHARACTERISTICS OF CROSS-SECTION

$b := 1 \text{ m}$ - width of one running meter

$A := t \cdot b = 120 \text{ cm}^2$ - area of cross section for one running meter

$z_t := t \cdot 0.5 = 6 \text{ mm}$ - distance between the center of gravity to top fibres

$z_b := z_t - t = -6 \text{ mm}$ - distance between the center of gravity to bottom fibres

$I := \dfrac{b \cdot t^3}{12} = 14.4 \text{ cm}^4$ - moment of inertia of the plate

$i := \sqrt{\dfrac{I}{A}} = 3.464 \text{ mm}$ - radius of inertia

$W_t := \dfrac{I}{z_t} = 24 \text{ cm}^3$ - plastic coefficient for top fibres

$W_b := \dfrac{I}{z_b} = -24 \text{ cm}^3$ - plastic coefficient for bottom fibres

$W_{pl} := 2 \cdot (0.5 \cdot t) \cdot b \cdot 0.25 \cdot t = 36 \text{ cm}^3$ - plastic coefficient

5.6.2.3 TENSION

$\gamma_{M0} := 1.1$ resistance of Class 1,2 or 3 cross-section

$N_{t.pl.Rd} := \frac{A \cdot f_y}{\gamma_{M0}} = 3654.545\ \text{kN}$ - design plastic resistance of the gross section

$N_{t.Ed} := 1105.36\ \text{kN}$

$\frac{N_{t.Ed}}{N_{t.pl.Rd}} = 30.246\%$ **condition satisfied**

5.6.2.4 BENDING

$M_{pl.Rd} := \frac{W_{pl} \cdot f_y}{\gamma_{M0}} = 10.964\ \text{kN} \cdot \text{m}$ - design plastic resistance moment of the gross section

$M_{Ed} := 2.05\ \text{kN} \cdot \text{m}$

$\frac{M_{Ed}}{M_{pl.Rd}} = 18.698\%$ **condition satisfied**

5.6.2.5 SHEARING

$A_{v.z} := A = 120\ \text{cm}^2$ shear area for plates

$V_{pl.z.Rd} := \frac{A_{v.z} \cdot \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = 2109.953\ \text{kN}$ design plastic shear resistance

$V_{z.Ed} := 1.66\ \text{kN}$

$\frac{V_{z.Ed}}{V_{pl.z.Rd}} = 0.079\%$ **condition satisfied**

5.6.2.6 BENDING AND AXIAL FORCE

$M_{N.Rd} := M_{pl.Rd} \cdot \left(1 - \left(\frac{N_{t.Ed}}{N_{t.pl.Rd}}\right)^2\right) = 9.961\ \text{kN} \cdot \text{m}$ design plastic resistance moment

$M_{Ed} = 2.05\ \text{kN} \cdot \text{m}$

$\frac{M_{Ed}}{M_{N.Rd}} = 20.581\%$ **condition satisfied**

5.7 LONGITUDINAL BEAM CALCULATIONS

The third longitudinal beam has been selected. The worst case scenario has been chosen the same as in the girder case. The same procedure of calculations as in the girder case.

5.7.1 MAXIMAL FORCES DIAGRAMS

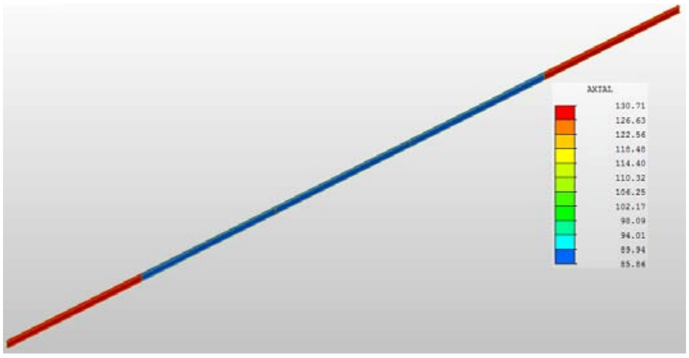


Figure no. 5.46. Diagram of axial forces

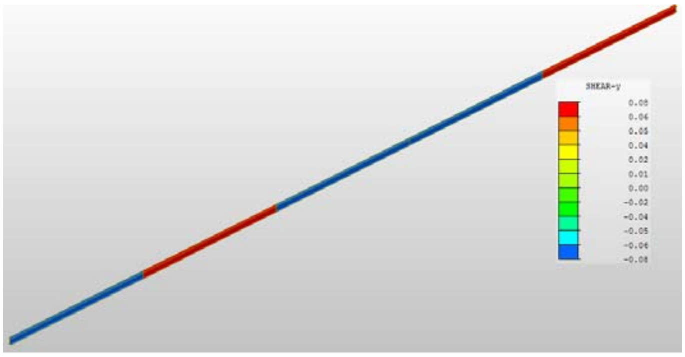


Figure no. 5.47. Diagram of shear forces about y axis

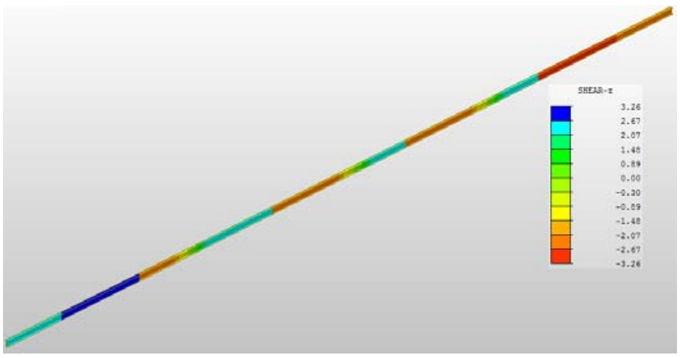


Figure no. 5.48. Diagram of shear forces about z axis

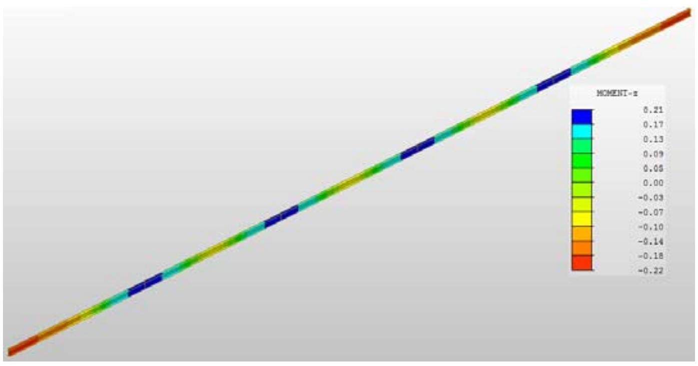


Figure no. 5.49. Diagram of bending moments about y axis

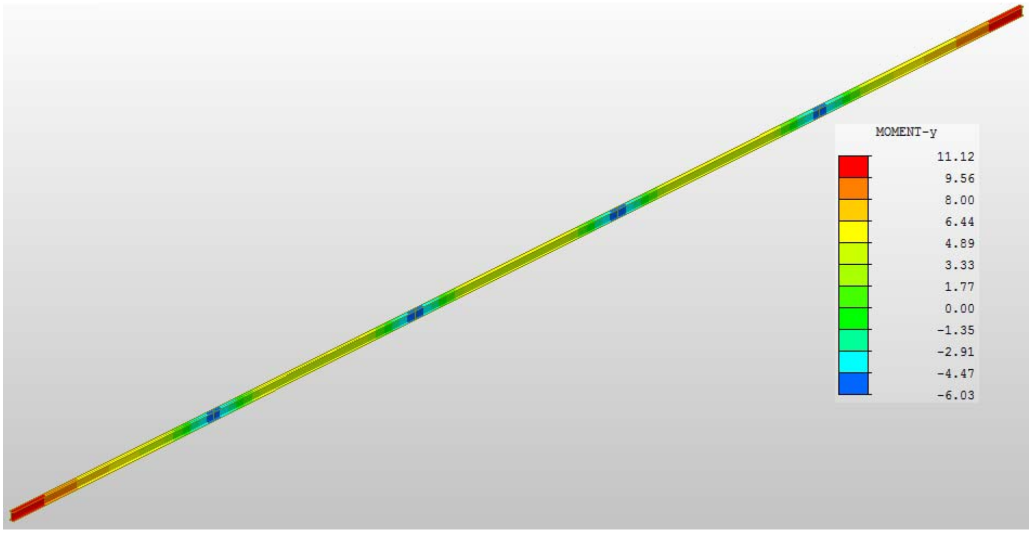


Figure no. 5.50. Diagram of bending moments about z axis

Table no. 5.5. Longitudinal beam maximal forces

Longitudinal Beam						
No.	Load case	Fx [kN]	Fy [kN]	Fz [kN]	My [kNm]	Mz [kNm]
Combination 3	Dead Load + Vehicle Load	130.71	0.08	3.26	11.12	0.22

5.7.2 LONGITUDINAL BEAM BEARING CAPACITIES

5.7.2.1 DETERMINATION OF THE CLASS OF CROSS-SECTION

$h := 140\ \text{mm}$ - total height

$b := 73\ \text{mm}$ - width

$t_w := 4.7\ \text{mm}$ - web thickness

$t_f := 6.9\ \text{mm}$ - flange thickness

$r := 7\ \text{mm}$ - radius

$A := 16.4\ \text{cm}^2$ - area of cross-section

$W_{pl.y} := 88.3\ \text{cm}^3$ - y-y plastic coefficient

$W_{pl.z} := 19.3\ \text{cm}^3$ - z-z plastic coefficient

$c := h - 2 \cdot t_f - 2 \cdot r = 112.2 \text{ mm}$ according to Table 5.2, PN-EN 1993-1-1

$f_y := 355 \text{ MPa}$

$\varepsilon := \sqrt{235 \dfrac{\text{MPa}}{f_y}} = 0.814$ according to Table 5.2, PN-EN 1993-1-1

$\alpha := 0.5$

$\gamma_{M0} := 1$

$\dfrac{c}{t_w} \le \dfrac{36\ \varepsilon}{\alpha} = 1$ Cross-section has been defined as **Class 1.**

5.7.2.2 TENSION

$N_{t.Rd} := \dfrac{A \cdot f_y}{\gamma_{M0}} = 582.2 \text{ kN}$ - design plastic resistance

$N_{Ed} := 130.71 \text{ kN}$

$\dfrac{N_{Ed}}{N_{t.Rd}} = 22.451\%$ **contidition satisfied**

5.7.2.3 SHEARING

Shear about y axis

$A_{v.y} := A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f = 7.616 \text{ cm}^2$ - shear area for rolled I and H sections

$V_{pl.y.Rd} := \dfrac{A_{v.y}}{\gamma_{M0}} \cdot \dfrac{f_y}{\sqrt{3}} = 156.103 \text{ kN}$ - design plastic shear resistance

$V_{y.Ed} := 0.08 \text{ kN}$

$\dfrac{V_{y.Ed}}{V_{pl.y.Rd}} = 0.051\%$ **contidition satisfied**

Shear about z axis

$A_{v.z} := A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f = 7.616 \text{ cm}^2$ - shear area for rolled I and H sections

$V_{pl.z.Rd} := \frac{A_{v.z}}{\gamma_{M0}} \cdot \frac{f_y}{\sqrt{3}} = 156.103 \text{ kN}$ - design plastic shear resistance

$V_{z.Ed} := 3.26 \text{ kN}$

$\frac{V_{z.Ed}}{V_{pl.z.Rd}} = 2.088\%$ **contidition satisfied**

5.7.2.4 BENDING

Bending about y axis

$M_{pl.y.Rd} := \frac{W_{pl.y} \cdot f_y}{\gamma_{M0}} = 31.347 \text{ kN} \cdot \text{m}$ - design resistance for bending about y axis for cross section 1 or 2

$M_{y.Ed} := 11.12 \text{ kN} \cdot \text{m}$

$\frac{M_{y.Ed}}{M_{pl.y.Rd}} = 35.474\%$ **contidition satisfied**

Bending about z axis

$M_{pl.z.Rd} := \frac{W_{pl.z} \cdot f_y}{\gamma_{M0}} = 6.852 \text{ kN} \cdot \text{m}$ - design resistance for bending about y axis for cross section 1 or 2

$M_{z.Ed} := 0.22 \text{ kN} \cdot \text{m}$

$\frac{M_{z.Ed}}{M_{pl.z.Rd}} = 3.211\%$ **contidition satisfied**

5.7.2.5 BENDING AND AXIAL FORCE

$$n := \frac{N_{Ed}}{N_{t.Rd}} = 0.225$$

$$a := \frac{A - 2\, b \cdot t_f}{A} = 0.386 \qquad a \le 0.5 = 1$$

$$M_{Ny.Rd} := M_{pl.y.Rd} \cdot \frac{1-n}{1-0.5 \cdot a} = 30.118\ \text{kN} \cdot \text{m}$$ - reduced design plastic resistance moment about y axis

$$M_{Nz.Rd} := M_{pl.z.Rd} \cdot \frac{1-n}{1-0.5 \cdot a} = 6.583\ \text{kN} \cdot \text{m}$$ - reduced design plastic resistance moment about z axis

$\alpha := 2$ for I and H sections

$\beta := 5 \cdot n = 1.123$ for I and H sections

$$\left(\frac{M_{y.Ed}}{M_{Ny.Rd}}\right)^{\alpha} + \left(\frac{M_{z.Ed}}{M_{Nz.Rd}}\right)^{\beta} = 15.836\%$$ **contidition satisfied**

$$\frac{N_{Ed}}{N_{t.Rd}} + \frac{M_{y.Ed}}{M_{pl.y.Rd}} + \frac{M_{z.Ed}}{M_{pl.z.Rd}} = 61.136\%$$ **contidition satisfied**

5.8 TRANSVERSE BEAM CALCULATIONS

5.8.1 MAXIMAL FORCES DIAGRAMS

The third transverse beam has been selectedThe worst case scenario has been chosen the same as in the girder.

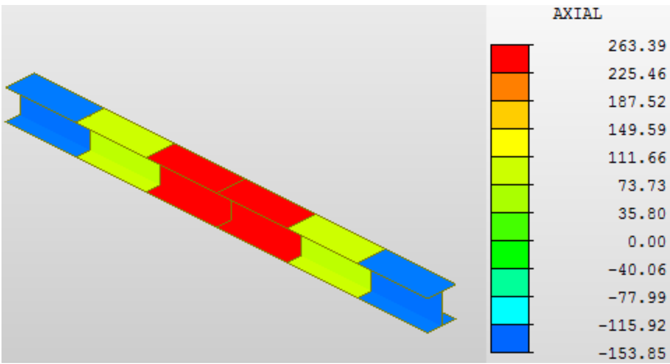


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Figure no. 5.51. Diagram of axial forces

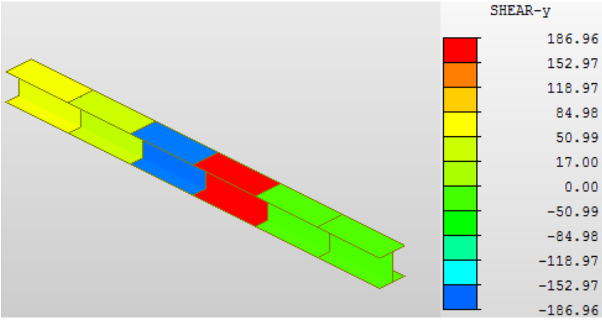

Figure no. 5.52. Diagram of shear forces about y axis

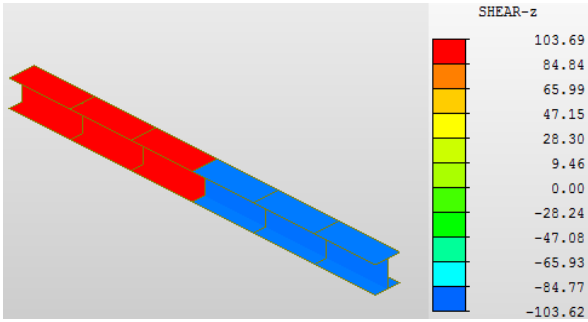

Figure no. 5.53. Diagram of shear forces about z axis

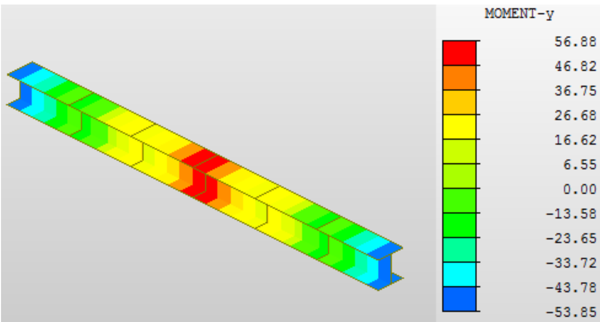

Figure no. 5.54. Diagram of bending moments about y

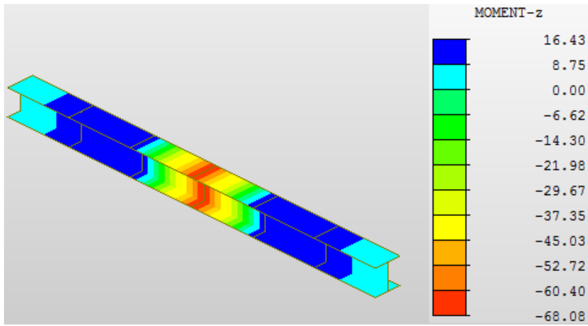

Figure no. 5.55. Diagram of bending moments about z

Table no. 5.6. Transverse beam maximal forces

Transverse Beam						
No.	Load case	Fx[kN]	Fy [kN]	Fz [kN]	My [kNm]	Mz [kNm]
Combination 3	Dead Load + Vehicle Load	263.39	186.96	103.69	56.88	68.08

5.8.2 TRANSVERSE BEAM BEARING CAPACITIES

5.8.2.1 DETERMINATION OF THE CLASS OF CROSS-SECTION

$h := 200\ \mathrm{mm}$ - total height

$b := 200\ \mathrm{mm}$ - width

$t_w := 9\ \mathrm{mm}$ - web thickness

$t_f := 15\ \mathrm{mm}$ - flange thickness

$r := 18\ \mathrm{mm}$ - radius

$A := 78.1\ \mathrm{cm}^2$ - area of cross-section

$W_{pl.y} := 642.5\ \mathrm{cm}^3$ - y-y plastic coefficient

$W_{pl.z} := 304.8\ \mathrm{cm}^3$ - z-z plastic coefficient

$c := h - 2 \cdot t_f - 2 \cdot r = 134\ \mathrm{mm}$ according to Table 5.2, PN-EN 1993-1-1

$f_y := 355\ \mathrm{MPa}$ - yield strength for S355

$$\varepsilon := \sqrt{235\ \frac{\text{MPa}}{f_y}} = 0.814$$

$$\alpha := 0.5$$

$$\gamma_{M0} := 1$$

$$\frac{c}{t_w} \le \frac{36\ \varepsilon}{\alpha} = 1$$

Cross-section has been indefined as **Class 1.**

5.8.2.2 TENSION

$$N_{t.Rd} := \frac{A \cdot f_y}{\gamma_{M0}} = 2772.55\ \text{kN}$$

- design plastic resistance

$$N_{Ed} := 263.39\ \text{kN}$$

$$\frac{N_{Ed}}{N_{t.Rd}} = 9.5\%$$

contidition satisfied

5.8.2.3 SHEARING

Shear about y axis

$$A_{v.y} := A - 2 \cdot b \cdot t_f + \left(t_w + 2 \cdot r\right) \cdot t_f = 24.85\ \text{cm}^2$$

- shear area for rolled I and H sections

$$V_{pl.y.Rd} := \frac{A_{v.y}}{\gamma_{M0}} \cdot \frac{f_y}{\sqrt{3}} = 509.324\ \text{kN}$$

- design plastic shear resistance

$$V_{y.Ed} := 186.69\ \text{kN}$$

$$\frac{V_{y.Ed}}{V_{pl.y.Rd}} = 36.654\%$$

contidition satisfied

Shear about z axis

$A_{v.z} := A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f = 24.85 \ \text{cm}^2$ - shear area for rolled I and H sections

$V_{pl.z.Rd} := \frac{A_{v.z}}{\gamma_{M0}} \cdot \frac{f_y}{\sqrt{3}} = 509.324 \ \text{kN}$ - design plastic shear resistance

$V_{z.Ed} := 103.69 \ \text{kN}$

$\frac{V_{z.Ed}}{V_{pl.z.Rd}} = 20.358\%$ **contidition satisfied**

5.8.2.4 BENDING

Bending about y axis

$M_{pl.y.Rd} := \frac{W_{pl.y} \cdot f_y}{\gamma_{M0}} = 228.088 \ \text{kN} \cdot \text{m}$ - design resistance for bending about y axis for cross section 1 or 2

$M_{y.Ed} := 56.88 \ \text{kN} \cdot \text{m}$

$\frac{M_{y.Ed}}{M_{pl.y.Rd}} = 24.938\%$ **contidition satisfied**

Bending about z axis

$M_{pl.z.Rd} := \frac{W_{pl.z} \cdot f_y}{\gamma_{M0}} = 108.204 \ \text{kN} \cdot \text{m}$ - design resistance for bending about y axis for cross section 1 or 2

$M_{z.Ed} := 68.08 \ \text{kN} \cdot \text{m}$

$\frac{M_{z.Ed}}{M_{pl.z.Rd}} = 62.918\%$ **contidition satisfied**

5.8.2.5 BENDING AND AXIAL

Bending and axial force according to PN-EN-1993-1-1 (6.2.9) (4)

$$n := \frac{N_{Ed}}{N_{t.Rd}} = 0.095$$

$$a := \frac{A - 2\ b \cdot t_f}{A} = 0.232 \qquad a \le 0.5 = 1$$

$$M_{Ny.Rd} := M_{pl.y.Rd} \cdot \frac{1-n}{1-0.5 \cdot a} = 233.474\ \text{kN} \cdot \text{m}$$

- reduced design plastic resistance moment about y axis

$$M_{Nz.Rd} := M_{pl.z.Rd} \cdot \frac{1-n}{1-0.5 \cdot a} = 110.759\ \text{kN} \cdot \text{m}$$

- reduced design plastic resistance moment about z axis

$\alpha := 2$ for I and H sections

$\beta := 5 \cdot n = 0.475$ for I and H sections

$$\left(\frac{M_{y.Ed}}{M_{Ny.Rd}}\right)^{\alpha} + \left(\frac{M_{z.Ed}}{M_{Nz.Rd}}\right)^{\beta} = 85.296\%$$

contidition satisfied

$$\frac{N_{Ed}}{N_{t.Rd}} + \frac{M_{y.Ed}}{M_{pl.y.Rd}} + \frac{M_{z.Ed}}{M_{pl.z.Rd}} = 97.356\%$$

contidition satisfied

6. RESULSTS, CONCLUSIONS AND FINAL REMARKS

The purpose of this work was to design an alternative footbridge for the currently existing one over a “Rudawa” river in Cracow. Designed footbridge is an arch structure with a through deck. Main girders are suspended on bearing arches and work as ties for these arches. Following sections have been assumed:

- girder – HEB 400,
- transverse beam – HEB 200,
- longitudinal beam – IPE 140,
- arch – circular tube of 244.5mm and 10mm thickness,
- bracing – circular tube of 193.7mm and 10mm thickness,
- hanger – 20mm diameter,
- slab – 120mm.

Project covers calculations for main structural elements, namely:

- arch,
- hanger,
- girder,
- plate,
- longitudinal beam,
- transverse beam.

In order to establish maximal values of forces, the analysis of influence lines were performed for arch, hanger and girder. There was no need to perform analysis of influence lines for plate, longitudinal beam and transverse beam because it was assumed that the worst case scenario of loading combination will be the same as in the girder case. Results are presented in the table 6.1 and 6.2. On the basis of performed calculations, it can be concluded that designed footbridge fulfills requirements for SLS and ULS according to Eurocodes.

Table no. 6.1. Final results for arch, hanger and girder

<table>
<tr><th></th><th>Force Value</th><th>Resistance</th><th>Bearing Capacity</th></tr>
<tr><td>Arch</td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">Bending and Compression Force</td><td>Nx = 172,87</td><td>NRk = 2615,285</td><td rowspan="2">20.38%</td></tr>
<tr><td>My = 16,8</td><td>MRk = 195,335</td></tr>
<tr><td>Hanger</td><td></td><td></td><td></td></tr>
<tr><td>Tension</td><td>54,86</td><td>111,47</td><td>49,22%</td></tr>
<tr><td>Girder</td><td></td><td></td><td></td></tr>
<tr><td>Tension</td><td>185,01</td><td>7021,9</td><td>2,63%</td></tr>
<tr><td>Shear about y</td><td>3,07</td><td>1434,715</td><td>0,21%</td></tr>
<tr><td>Shear about z</td><td>34,87</td><td>1434,715</td><td>2,43%</td></tr>
<tr><td>Bending about y</td><td>95,96</td><td>1147,36</td><td>8,36%</td></tr>
<tr><td>Bending about z</td><td>6,39</td><td>391,92</td><td>1,63%</td></tr>
<tr><td rowspan="3">Bending and axial force</td><td>Nx = 185,01</td><td>-</td><td rowspan="3">57,79%</td></tr>
<tr><td>My = 95,96</td><td>Mn.y.Rd = 1292,968</td></tr>
<tr><td>Mz = 6,39</td><td>Mn.z.Rd = 441,657</td></tr>
</table>

Table no. 6.2. Final results for plate, longitudinal beam and transverse beam

	Force Value [kN], [kNm]	Resistance [kN], [kNm]	Bearing Capacity
Plate			
Tension	1105,36	3654,545	30,25%
Bending	2,05	10,964	18,70%
Shear	1,66	2109,953	0,08%
Bending and Axial Force	2,05	9,961	20,58%
Longitudinal Beam			
Tension	130,71	582,2	22,45%
Shear about y	0,08	156,103	0,05%
Shear about z	3,26	156,103	2,09%
Bending about y	11,12	31,347	35,47%
Bending about z	0,22	6,852	3,21%
Bending and axial force	Nx = 130,71	-	15,84%
	My = 11,12	Mn.y.Rd = 30,118	
	Mz = 0,22	Mn.z.Rd = 6,583	
Transverse Beam			
Tension	263,39	2772,55	9,50%
Shear about y	186,69	509,324	36,65%
Shear about z	103,69	509,324	20,36%
Bending about y	56,88	228,088	24,94%
Bending about z	68,08	108,204	62,92%
Bending and axial force	Nx = 263,39	-	85,30%
	My = 56,88	Mn.y.Rd = 233,474	
	Mz = 68,08	Mn.z.Rd = 110,759	

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Standards:
[E1] PN-EN-1990-2002 – Basis of structural design.
[E2] PN-EN 1991-2-2002 – Traffic loads on bridges.
[E3] PN-EN 1993-1-1-2001 – Design of steel structures. General structural rules.
[E4] PN-EN 1993-2-2006 – Design of steel structures. Steel bridges.

Multimedia:
[M1] http://katowice.gazeta.pl
[M2] https://www.flickr.com
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Figures:
[K1] Top View – 1:50
[K2] Longitudinal View – 1:50
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