Pedestrian bridge made of recycled plastic

Master thesis

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Content

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Abstract

The aim of the design was to use an innovative material for construction of 12 meters long and 3 meters wide pedestrian bridge. Steel reinforced recycled plastic beams were used. First, three alternative designs were analyzed with use of FrameMaster finite element program. Based on the deflection behavior under the loading from service vehicle and possible production of all three designed bridges, the truss bridge design has been chosen as a best design.

Further, truss bridge was modeled and analyzed in SCIA Engineering finite element program. One model with assumption of perfectly fixed connections of truss elements and two models with truss composed of top and bottom chords constructed as continuous beams and diagonals connected with pin joints have been created. The modeled variants of truss bridge were subjected to five load combinations based on Eurocode provisions. The worst load combination was load from crowd. All three models fulfilled the deflection requirements of deflections in vertical direction. The model with fixed connection was showing good results but the production and assembly of the truss with fixed connections would be too complicated. The model with pin connection was chosen for further analysis.

The load bearing truss had too high horizontal displacement of the top chord and therefore the 3D truss model was developed. The 3D truss was composed of one top chord and two bottom chords. The number of diagonals was doubled. Special pin connection had to be developed for the construction of 3D truss bridge. The design of the pin connection would have to be developed in more detailed calculation which did not fit into the time frame of this project.

The 3D truss pedestrian bridge 12 meters long and 3 meters wide is possible to construct under the assumption of possible production of the pin joint.

Key words: recycled plastic material, composite, recycled polypropylene, recycled polyethylene, pedestrian bridge, truss

Introduction

Recycling can be defined as using of old materials to fabricate new products. This process of the recycling is one of the key factors leading to a sustainable environment. Human behaviour ignores this fact and keeps producing more and more waste that is very difficult to decompose by natural biological processes. Particularly, plastic materials usually require up to hundreds of years to disintegrate. Therefore, it is necessary to look for some new methods in which any plastic materials could be recycled and used again.

This master thesis is done in the Royal Lankhorst Euronete Company, department Lankhorst Engineered Products. The company has very long history and was founded in 1803. Their main business is the production of ropes, which originally were made of natural materials. In the beginning of 20^{th} century synthetic materials started to be used. Later around 1970^{th} the company had too much plastic waste, therefore the idea of recycling was born and the first recycled plastic products were made. Later the department of Recycled Plastic Products and Engineered Products were established. Nowadays, Royal Lankhorst Euronete Engineered Products is a leader in recycled plastic products and production. They have developed their own mixture of recycled plastic material (called KLP = Kuntsthof Lankhorst Products) and they developed a method to produce recycled plastic beams with steel reinforcement.

The Royal Lankhorst Euronete is building pedestrian bridges in the Netherlands over a decade. The production of recycled plastic structural elements is very limited because of the specialised production process (high temperature and pressure) and the available machine capacities. Therefore, the maximum span of built pedestrian bridges is 8.5 m up to now.

This master thesis focuses on a design of a pedestrian bridge of 12 m span – or longer if possible – made of steel reinforced KLP. The bridge can be built as an easily variable system for different lengths of spans and widths. Mainly KLP and structural elements that can be produced in Royal Lankhorst Euronete factory have been used. Also, the bridge exerts only vertical forces onto its foundation (the horizontal forces are carried by the bridge itself). The bridge can be assembled in the factory and later transported to the desired place. Considered loads on the bridge are: pedestrians, cyclists and service vehicle.

1 Recycled plastic material KLP

1.1 General description

The innovative material used in this project is KLP (Kunststof Lankhorst Product). It is a mixture of recycled plastics, i.e. polypropylene and polyethylene.

KLP is generally used in construction in the same way as wood but it has several advantages compared to wood. It does not need any painting and its life span is much longer. However, KLP possesses some properties that are different from wood. Creep and shrinkage are different. For example it is well known that wood shrinks differently in the direction of grain and in the tangential and radial direction to the grain. KLP material shrinks in all directions in the same manner. Another different property of KLP compared to wood is that KLP does not absorb water. It is water resistant and it does not change its volume with changes of the surrounding humidity (1).

KLP has been already used in many varieties of products (Figure 1) i.e. facades, fences, poles, garden furniture, pedestrian bridges (maximum span 8.5 m) etc.

Figure 1: Some examples of products made of recycled plastic material KLP (2)

1.2 Production

The production of recycled plastic material starts with separation of the polypropylene (PP) and polyethylene (PE) waste from the common plastic waste. There are two well used types of PE. These are High-density Polyethylene (HDPE) with Plastic Identification Code 2 (Figure 2) and Low-density polyethylene (LDPE) with Plastic Identification Code 4. The most common products of HDPE are for example bottle caps, foldable chairs and tables, water pipes, refillable bottles, puck board and many others (3). The most common products of LDPE are plastic bags, various containers, six pack rings, squeezable bottles (honey, mustard, etc.), flexible containers lids and many others (4).

The plastic products from PP have Plastic Identification Code 5 and the most common products are yoghurt containers, microwaveable disposable take-away containers, disposable cups and plates and many others (5).

Figure 2: Plastic identification codes for High-density PE, Low-density PE and PP

After separation, the waste is cleaned and chopped into 4 – 8 mm small pieces or flakes or it is grounded and melted into drops of 4 – 5 mm (Figure 3). These small pieces are transported into the extruder where under heat (190° - 230°C) and pressure (70 bars) all pieces are melted and pressed into a mould. Then the filled mould is put into a water bath to cool down. The time necessary for the mould and product to cool down is dependent on the size of the product. After cooling the product can be taken out from the mould.

The size of the mould is limited by the size of machinery available for production. In Royal Lankhorst Euronete factory the machinery (operated manually) is normally able to handle moulds of about 1500 kg, cranes are able to carry load up to 2000 kg.

For the production of KLP beams with steel reinforcement automatic production is used and the weight of the mould can go up to 2500 kg. The maximum length of the beam possible to produce in the Royal Lankhorst Euronete factory is 5.4 m.

Figure 3: Chopped plastic waste and drops of recycled plastic material ready for production

1.3 Properties

The PP and PE plastic waste usually contains a lot of additives and different colours. Because of that the recycled product has dark grey colour varying with the current composition of the plastic waste. Therefore, black carbon is added in order to assure a standard black colour. Other colours are possible by separating the same colour PP and PE plastic waste, but it is more expensive.

The black colour also enables a very good protection from UV-light. Normally transparent plastic material is sensitive to the UV light which damages the bond in the polymer molecule and causes the degradation of the material. In the case of the black colour the UV-light has influence only on the first tenths of millimetres of the surface and there is hardly any effect on the mechanical properties (1).

The KLP can be used in many structures, where wood is used, but in general it is less stiff than wood. Plastic expands under higher temperature and shrinks under colder temperature. Hence designed structures should allow the material to expand and shrink (1).

The KLP mechanical properties are given in the Table 1. The material KLP is the common mixture of PE and PP. KLP - PE is a basic mixture consisting mainly of recycled PE, KLP - PP is a mixture of recycled PP and the KLP-V is a material reinforced with glass fibres.

Table 1: Mechanical properties of KLP

	Standard	**Unit**	**KLP**	**KLP - PE**	**KLP - PP**	**KLP - V**
Density	ISO 1183	kg/m^3	800	820	850	870
Tensile Strength	ISO 527	MPa	15	9	19.5	24
Young's Modulus	ISO 527	MPa	850	250	1200	1500
Strain at Yielding	ISO 527	%	6	20	7	3
Strain at Breaking	ISO 527	%	6	>250	50	4.5
Flexural Strength	ISO 178	MPa	28	14	25	30
Flexural Modulus	ISO 178	MPa	1000	250	1200	2000
Creep Modulus 10 yrs	-	MPa	250	50	-	-
Linear Thermal Expansion	ISO 11359	$x10^{-4}/℃$	1-1.5	~200	120	70
Melting temperature	ISO 11357	℃	140-160	110 - 130	165	130 - 170

1.4 Construction

Drilling, sawing, milling and planing like is used to process wood can be used for KLP. There is only one warning not to choose too high cutting speed to prevent the material from melting.

Also the connection technology, which is used for wooden structures, is used for KLP structures. Nailing is possible but practice shows that pre-drilling and screwing are better. Chipboard screws are the most suitable and it is advised to use stainless steel screws. Connections by bolts are very good and an often used connection method.

Welding is possible by melting, although, it is necessary to use special extrusion welding device. A good weld can reach up to 30-50% of the strength of the original material. But generally welding is not recommended (1).

Also gluing is not satisfactory because most of glues are hydrophilic and recycled plastic material is hydrophobic. It means that gluing agent and material do not form a sufficiently strong bond. Therefore, gluing is not recommended.

Another type of connection is a clip connection. The production of plastic products is very dependent on shape and size of the mould. Therefore, complex shapes of plastic products are possible to produce with 2% accuracy. Example of a clip connection is presented in Figure 4.

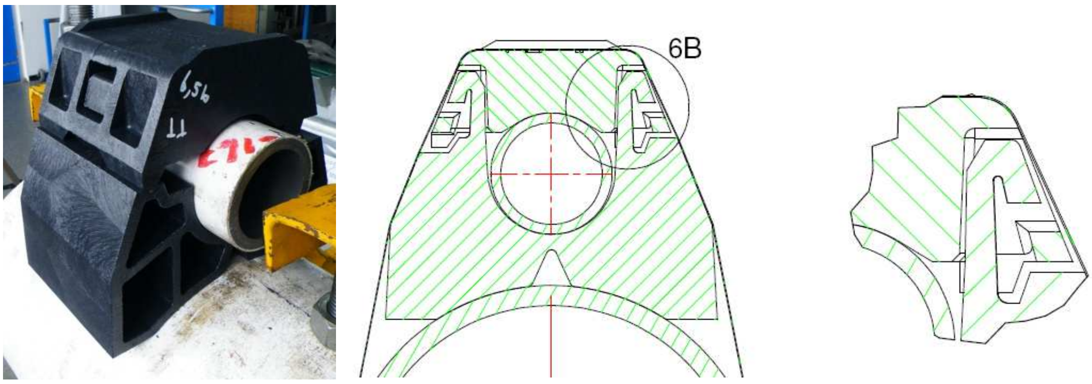

Figure 4: Example of a clip connection of a KLP product

1.5 KLP – S with steel reinforcement

The Royal Lankhorst Euronete Company has developed a system for making plastic products with steel reinforcement. This means that the steel reinforcement bars are fully submerged into the plastic material. A cross-section of a steel reinforced beam is presented in Figure 5. The steel bars are standard ribbed reinforcement produced for reinforced concrete. The reinforcing bars are of steel type S435 and three diameters of 8, 12 and 16 mm are used.

As explained in the Section 1.2, in order to make a plastic product, the material has to be heated and then pressed into the mould. In the case of beams with steel reinforcement the reinforcement is first placed into the mould and then plastic is injected. Under the required pressure the steel bars would be immediately pushed into the corners of the mould and the plastic material would not be able to cover the steel reinforcement bars. Therefore, the mould includes a piston, which keeps the steel bars in their position (Figure 6). In 2005, the Royal Lankhorst Euronete Company has patented the production methodology of recycled plastic beams with steel reinforcement (Figure 6).

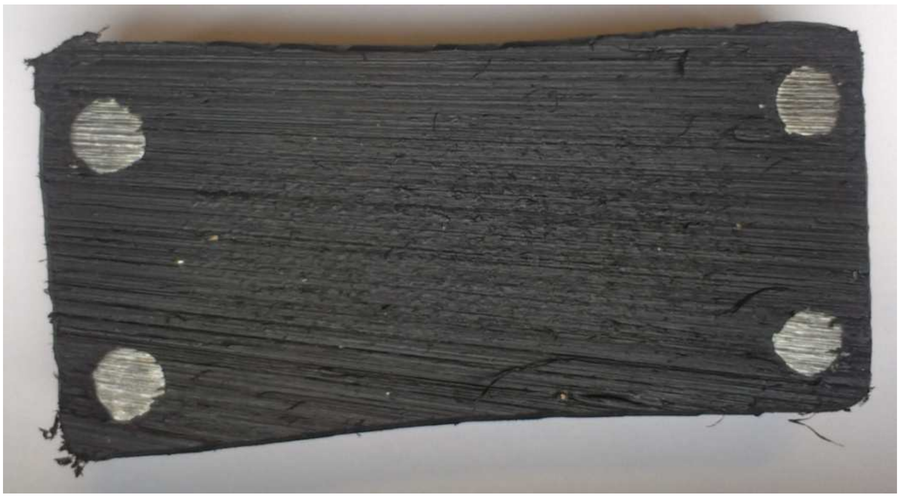

Figure 5: Cross-section of a KLP-S beam

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(54) **Method and apparatus for manufacturing an elongate reinforced plastic construction part, and reinforced plastic construction part manufactured with such an apparatus**

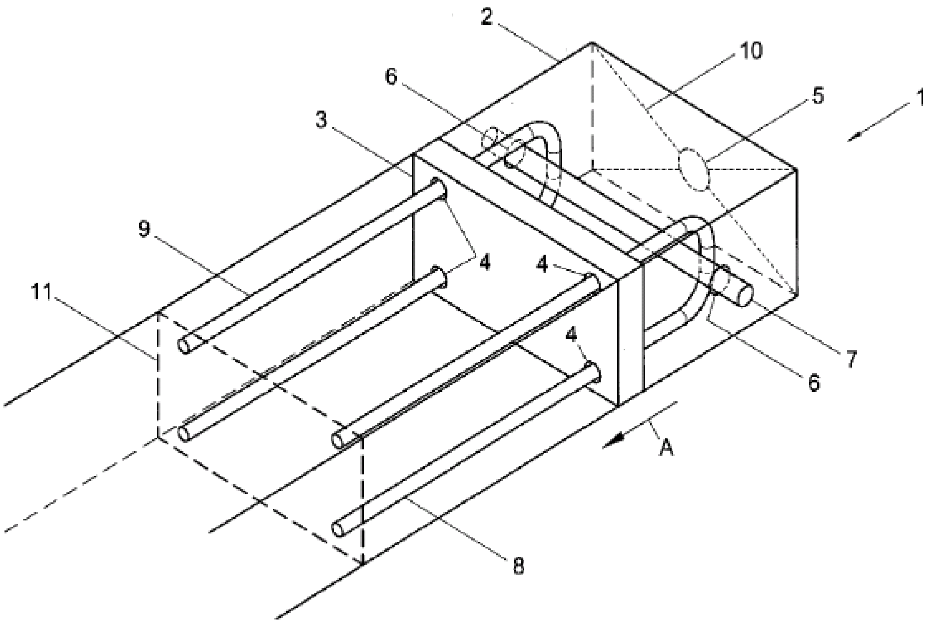


Figure 6: The method of the production of the steel reinforced plastic beams(6)

With the steel reinforcement, it was possible to improve the mechanical properties of KLP beams and other products. The mechanical properties of KLP-S beams are presented in the Table 2. The standard KLP-S beams are of a rectangular cross-section with variable height and width (150x70 mm, 160x80 mm and 180x80 mm). The maximum length is 5 m and the bar diameters are 8, 12 and 16 mm.

Table 2: Mechanical properties of standard KLP-S beams (KLP with steel reinforcement)

	Standard	Unit	150x70 S8	150x70 S12	160x80 S8	160x80 S12	160x80 S16	180x80 S8	180x80 S12	180x80 S16
Density	DIN 53479	kg/m^3	990	1170	960	1100	1290	950	1070	1240
Flexural strength	NEN-EN 408	MPa	21.1	44.3	16.6	35.1	58.5	15.1	32.2	54.2
Elongation at flexural strength	NEN-EN 408	MPa	18.0	37.7	14.1	29.8	49.8	12.9	27.4	46.1
Flexural modulus	NEN-EN 408	MPa	9150	19050	7250	15250	25450	6700	14200	23850
Elongation at break	NEN-EN 408	%	NB	NB	NB	NB	NB	NB	NB	NB
Creep modulus 10+ years	DIN 53444	MPa	4550	9500	3600	7600	12700	3350	7100	11900
Young's modulus	-	MPa	4560	9940	3540	7650	13420	3190	6840	11960
Tensile strength	-	MPa	10.3	23.1	7.9	17.7	31.4	7.0	15.7	27.9
Max. Shear stress	-	MPa	14.9	23.9	13.2	20.1	29.7	12.6	18.7	27.2
Linear Thermal Expansion coef.		x10-4/°C	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1

2 Existing bridges made of recycled plastic material

2.1 Introduction

Worldwide, there is a tendency to use recycled plastic materials in structures where structural components are in contact with water (like bridges). It is because plastics are water (and many solvents) resistant materials.

From the year 2009 there were already several heavy load bridges made of recycled plastic material in the United States and in the United Kingdom. For the construction very massive beams were used and the clear spans of the heavy load bridges were 4 and 9 m.

Royal Lankhorst Euronete has built almost hundred pedestrian bridges mainly in the Netherlands. As was explained in the first chapter the size of the structural elements is limited by the production and therefore, the maximum span of built pedestrian bridges was 8.5 m up to now.

Although, world production has proven it was possible to build even heavy load (tank, train etc.) bridges from recycled plastic material, it is still challenging to design a pedestrian bridge of 12 m span from structural elements, which are possible to be produced in the Royal Lankhorst Euronete factory capabilities.

2.2 Heavy load bridges

The first road bridge, made of recycled thermoplastic composite material, that can carry tank traffic (load about 70 tons) was built in 2009 at Fort Bragg, N.C. and the first railway bridge (load about 130 tons) was built one year later, 2010 at Fort Eustis, Virginia in the USA. These bridges were made for the U.S. Army under the guidance of engineers from U.S. Army Corps of Engineers, Engineer Research and Development Center (ERDC), and U.S. Army Construction Engineering Research Laboratory (CERL, Champaign, Ill.). The recycled content was compounded and bridge components were extruded under the direction of Axion International Inc. using the technology developed at Rutgers University, New Jersey, USA. (7)

Figure 7: Bridges made of recycled plastic material – a road bridge which can carry load of 70 tons (left) and a railway bridge that can carry load of 130 tons (right); maximum clear span of both bridges was 4 m (7).

2.2.1 General description

The structure of the heavy load bridge is designed using the methodology typical for wooden structures – girder and cross beam construction. All structural elements (solid round pilings, I-beams, deck boards and railing) are extruded from a compound of recycled, post-consumer HDPE (everything that remains after the commonly recycled polyethylene terephthalate – PET) and recycled, post-industrial fibre-reinforced PP (automobile bumpers). The bridge components were produced by the company Axion International Inc. which has an exclusive license to use a patented process developed at the AMIPP Advanced Polymer Center at Rutgers University, New Jersey. The Rutgers process is immiscible polymer blending with control of glass fibres direction (8). The orientation of fibres reduces the total amount of glass fibres. By using the Rutgers process, the final mixture with 11% fibres by weight has strength of a conventionally extruded product with 34% randomly oriented fibres. The method also results in highly oriented, more uniform alignment of polymer molecular chains (7).

2.2.2 Structural design

The design of the bridge can be seen in Figure 8. The piles of 305 mm diameter support a cross-beam, an I-beam, with vertical compression members that can carry the load from girders (Figure 9). The girders are placed over the cross-beam side by side (Figure 10). I-beams are composed of two T-shape beams; 457 mm wide and 305 mm high. These T-beams are turned by 180 degrees to each other and they are bolted together in order to form an I-beam (Figure 11). The maximum clear span is about 4 m (7).

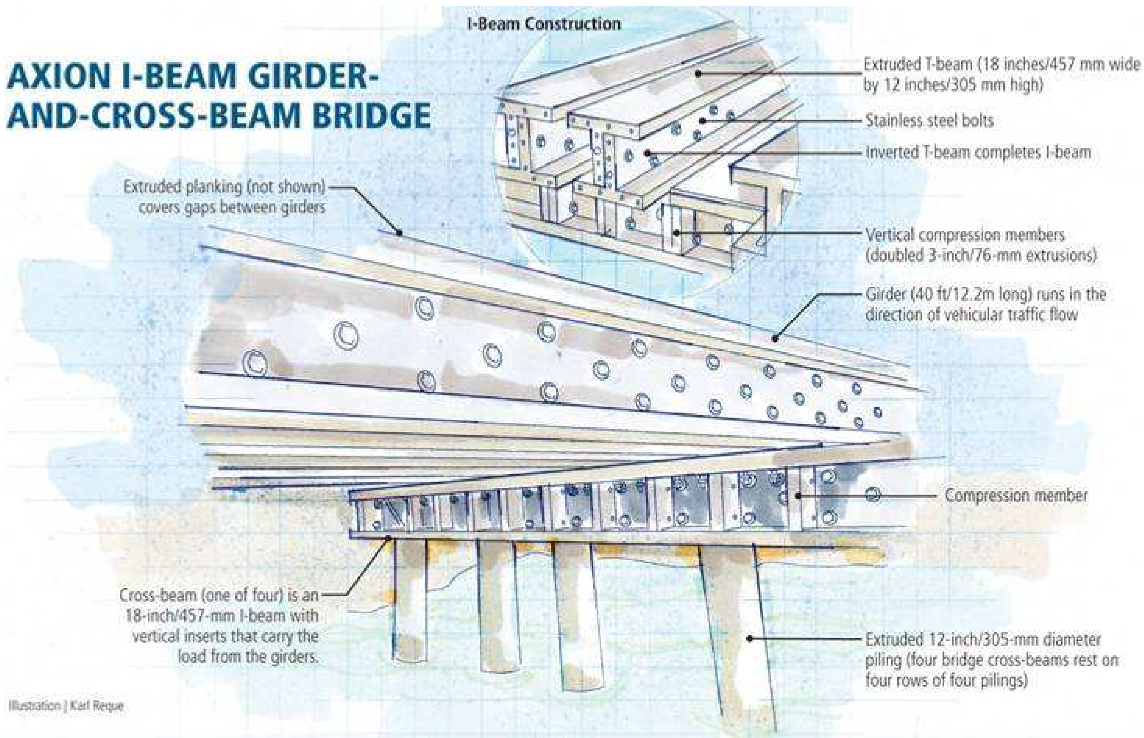


Figure 8: The structure composition of tank and railway bridge (7)

The modulus of elasticity of this recycled thermoplastic composite material is 1720 MPa and ultimate flexural/tensile/compressive strength is about 24 MPa (8) (which is same as KLP, see Table 1). In order to minimize deflection and creep of the structure the bridge was design to withstand allowable flexural, tensile and compressive stress of 4.2 MPa and the working stress of less than 15% of the ultimate strength. Such a conservative design should assure no creep in 25 years.

Over next two years bridges will be monitored using a series of deflection and strain gauges. Based on initial testing, bridge service life has been estimated to over 50 years, with the expectation of minimal maintenance. In comparison to a conventionally treated timber bridge design, carrying the same load, the cost of the thermoplastic composite bridge was lower (7).

Figure 9: ***Left:*** **Piles are also made of recycled plastic material;** ***Right:*** **Cross beams with vertical compression members (7).**

Figure 10: ***Left:*** **The girders are placed over the cross beams;** ***Right:*** **The bridge deck is composed of I-beams laid side by side next to each other.**

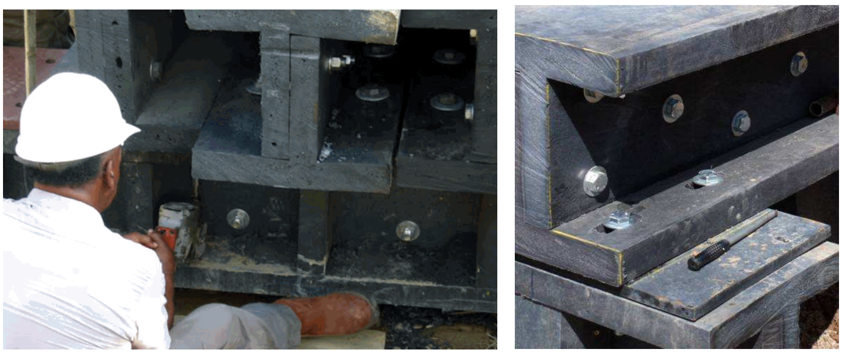


Figure 11: ***Left:*** **I–beams profile formed from two T-shape beams;** ***Right:*** **There can be seen a larger bolt hole which allows the material to expand and shrink.**

Figure 12: The railway bridge with a maximum clear span of 4 m.

2.2.3 Recycled plastic bridges in the United Kingdom

The first recycled thermoplastic road bridge built outside the United States was constructed in 2011 over the river Tweed, at Easter Dawyck in Peeblesshire in the United Kingdom. The bridge was designed to carry mainly agricultural vehicles and machinery of up to 44 tonnes (9). A total length of the bridge is 30 m and clear span is 9 m (Figure 13).

The project was conceived by Vertech Ltd. and it was supported by the Welsh Assembly Government. But the collaboration with partners from Dawyck Estates, specialist bridge designer Cass Hayward LLP, Cardiff University's School of Engineering, Rutgers University's AMIPP Department, Polywood Inc. and Axion International Inc. has made this project possible.

The structural design of the bridge seems almost identical to the bridge design of the road and railway bridges built in USA (Figure 7 and Figure 8). Considering that the bridge was built in collaboration with Axion International Inc and the Rutgers process technology was used, it can be assumed that the bridge design differs only with respect of European design provisions valid in the UK.

There will be more bridges built in UK's rural area. Bridges from recycled thermoplastic material promise decreasing of annual maintenance costs and large reduction of waste material sent into the landfill (9).

Figure 13: The installation of the 9 m clear span bridge in the UK.

2.2.4 Conclusion

From the available information about physical properties of the material used for production of bridges in the USA and in the UK, it is obvious that the recycled plastic material KLP has comparable material properties. The production of the structural elements from a composite material is highly limited by the size of the production, size of the moulds and extrusion machinery.

If the Royal Lankhorst Euronete would have been equipped to produce large profile beams (like Axion International Inc.), a similar bridge could be built as well.

Although it has been proved that recycled plastic material can be used in construction also for heavy loads, it is still challenging to design a pedestrian bridge with 12 m of a clear span (in construction of road and railway bridge the maximum clear spans were 4 m and 9 m).

2.3 KLP pedestrian bridges

2.3.1 KLP pedestrian bridge without steel reinforcement

Over a decade Royal Lankhorst Euronete has produced almost hundred of pedestrian bridges made of KLP material. In Figure 14 a 10-years old KLP pedestrian bridge is presented. In this bridge design extra steel beams and ties were used to reach a larger span (Figure 15). Clear span of this bridge is 4.5 m. At distance of 1.5 meter from the support a steel beam connected to railing by steel ties is placed. These steel beams support KLP girders, which were able to support only 1.5 m of a clear span.

The KLP bridges hardly need maintenance. It might be necessary to clean the surface of the bridge. If the bridge is placed in the shade, a wet surface on the bridge attracts algae and moss (Figure 16) and then the deck can become slippery.

Figure 14: The 10-years old KLP Bridge located in Sneek, The Netherlands, Europe. There were used extra steel beams and steel ties connected to railing in order to obtain 4.5 meter of clear span.

Figure 15: *Left:* the steel beam which is supporting KLP girders. *Right:* the detail of the connection of the steel tie and the railing of the bridge.

Figure 16: Left: KLP girder from the bottom view of the bridge. It is mostly in a shade and therefore it is wet which creates good growing conditions for algae and moss. Right: The moss is growing between the deck planks.

2.3.2 KLP pedestrian bridge with steel reinforcement

Over the years of the work with the recycled and also virgin plastics materials Royal Lankhorst Euronete has developed a method for placing steel reinforcement bars into the KLP beams. A 6-years old bridge with the steel reinforced beams can be seen in Figure 17. The beams with steel reinforcement are able to support 5 m of a clear span. The total span of the bridge is about 10 m and the beams are supported in the middle by the concrete piles and girder (Figure 18). The development in the production of KLP deck planks, which are now produced with stripe notches, can be seen in Figure 18 . The stripe notches are used in order to increase non-skid property of KLP. Figure 19 illustrates details of the connection of the railing. A bridge with two concrete supports is presented in Figure 20.

Figure 17: The 6-years old KLP Bridge located in Sneek, The Netherlands, Europe. In this bridge the clear span is 5 m and deck is supported by KLP beams with steel reinforcement.

Figure 18: Left: the total length of the bridge is 10 m and the bridge is supported in the middle by concrete piles and a concrete girder. Right: Detail of the deck planks with stripe notches which increase non-skid property of KLP.

Figure 19: The details of the connection of the railing.

Figure 20: About 5-years old KLP Bridge located in Sneek, The Netherlands, Europe. The clear span is 4.5 m.

2.3.3 KLP arch bridge

Together with the development of steel reinforced elements also new possibilities came available in bridge design. An KLP arch bridge is shown in Figure 21. The steel reinforcement with KLP material is very a unique combination. The arch is formed from 6 steel reinforced elements connected together and then bolted into an arch. The construction process of the arch bridge is an interesting procedure and therefore, it is described illustratively in the following sections.

Figure 21: A placing of the KLP arch bridge located in Mepple, The Netherlands, Europe (2009).

2.3.3.1 Construction of the KLP arch bridge

The standard span of the arch bridge is 8.5 m, the width of the bridge is 3 m and the height of the arch and railing is about 1 meter.

The most important part of the bridge is the arch which is composed from six KLP-S beams. The arrangement of the beams in the arch is depicted in Figure 22.

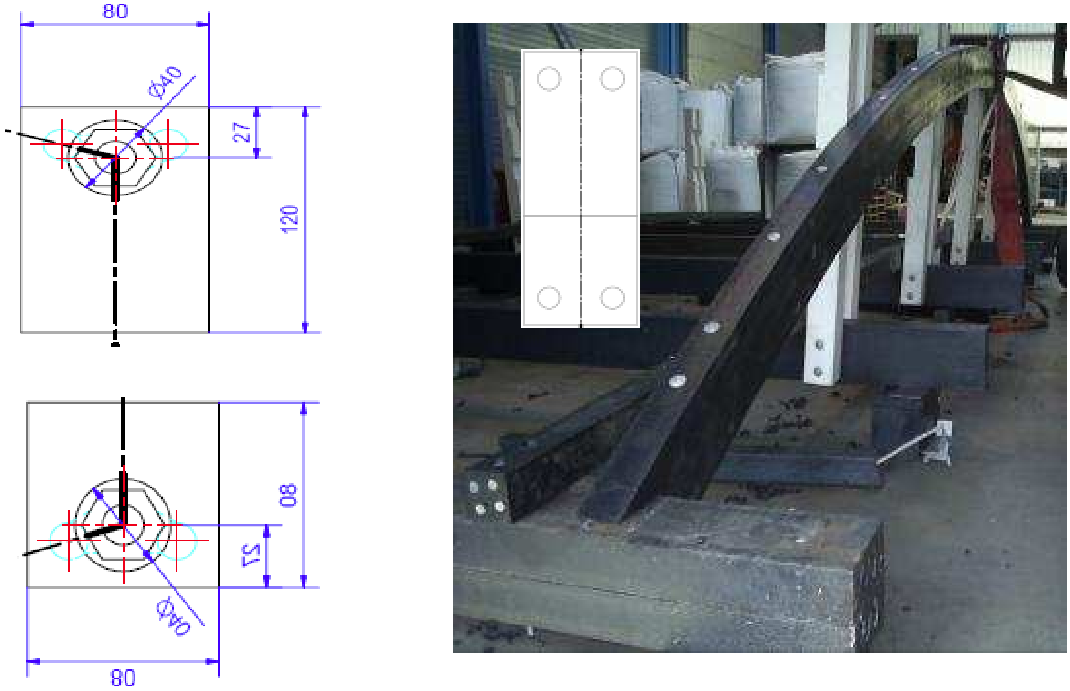


Figure 22: The arrangement of the arch

The cross-section of the arch is composed of two beams with different dimensions. The larger beam is 120 mm high and 80 mm wide. The smaller beam is square cross-section of 80 x 80 mm. Both beams have only two reinforcement steel bars which are placed closer to one side.

The beam of non symmetrical cross-section comes out of the production in a slightly curved shape. The cause of this curvature is shrinkage of the KLP material, which is about 3%. During production, directly after the injection of KLP into the mould and when the cooling process starts, the KLP material shrinks on one side of the non symmetrical cross-section, but on other side the KLP material cannot shrink, because there is a steel reinforcement bar, which restrains the shrinkage (Figure 23).

The standard length of the produced beams is 3 m. The reinforcement in the beams is ended with a large nut (diameter 40 mm). Through this nut the beams can be connected to form longer elements. Three beams are necessary to be connected in order to obtain the required arch for 8.5 m length of the bridge (Figure 23).

Figure 23: Left: the curved beams with two reinforcement bars. Right: the beams connected together.

The long beam of smaller cross-section is turned in the direction that the reinforcement would be in the bottom of the arch cross-section and then it is squeezed into the curve of the larger cross-section beam (Figure 24).

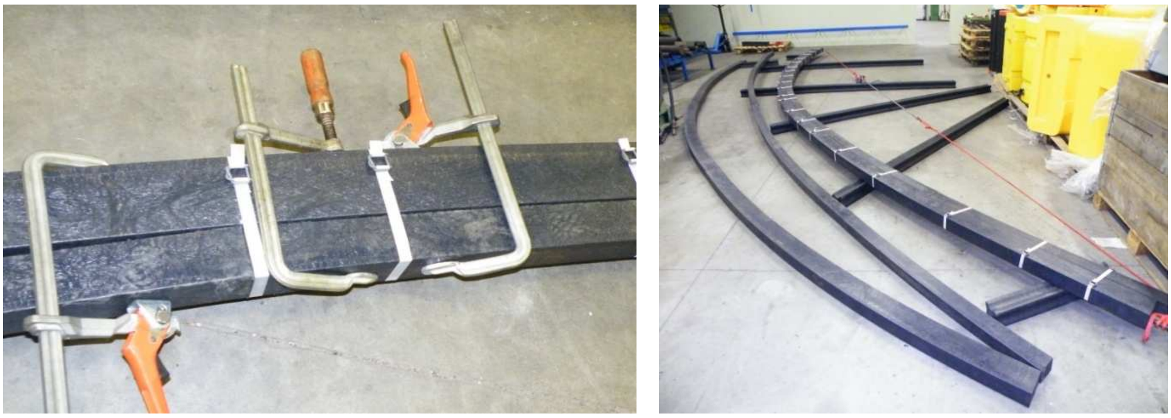


Figure 24: Left: the smaller and larger beams squeezed together with tools and fixed with temporary plastic strips. Right: long elements prepared to be put together and the large and the small beams fully fixed with temporary plastic strips.

The temporarily fixed arch is curved until the required height of the arch is obtained (Figure 25). Then the beams are drilled through and bolted together (Figure 26). Once the beams are bolted the plastic strips can be removed and the arch is formed (Figure 27).

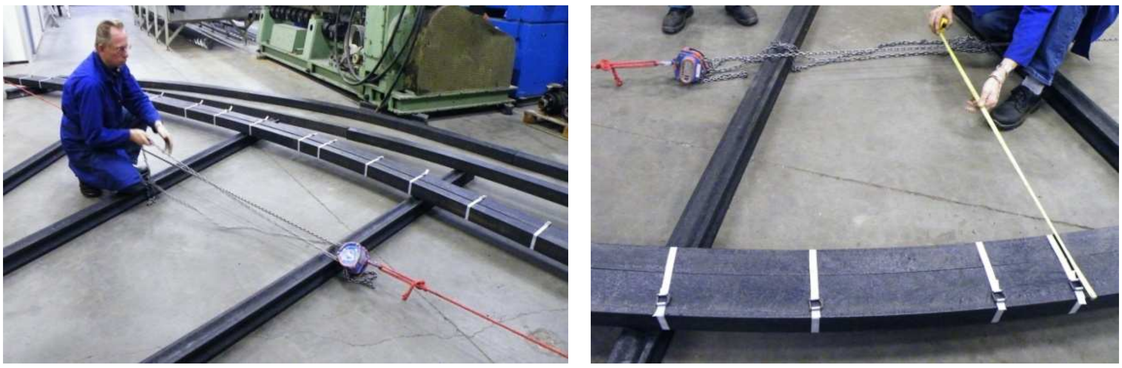


Figure 25: The beams fixed together are curved into an arch of a required height.

Figure 26: The curved beams are drilled and bolted together.

Figure 27: Left: Fully connected beams forming the arch. Right: the arch ready to be placed into its position on the bridge.

Figure 28: Left: placing of the arch into its position. Right: the end of the arch is bolted through the end transversal beam.

Figure 29: Left: the connection of arch to the railing poles. Right: The arches placed on both sides of the bridge.

When the arches are ready, the transversal beams with railing poles can be built up and the arches can be fixed on both sides of the bridge (Figure 28 and Figure 29). After placing of arches into their positions, the construction of the deck can start.

The deck is composed of longitudinal beams supported every 1.5 m by transversal beams. The KLP deck planks are fixed to the top of the longitudinal beams and in order to fulfill requirement of deck deflection the deck planks have to be supported every 0.5 m. Therefore for the bridge of 3 m width about 7 longitudinal beams are necessary.

The longitudinal beams are composed of KLP steel reinforced beams of 80 x 80 mm square cross-section with 4 steel reinforcement bars (Figure 30). The steel reinfrocement is finished with a nut at the ends of beams (like in the case of beams used for the arch). Throught nuts the beams are conected and the reinforcement works continuously over the ful length of the bridge.

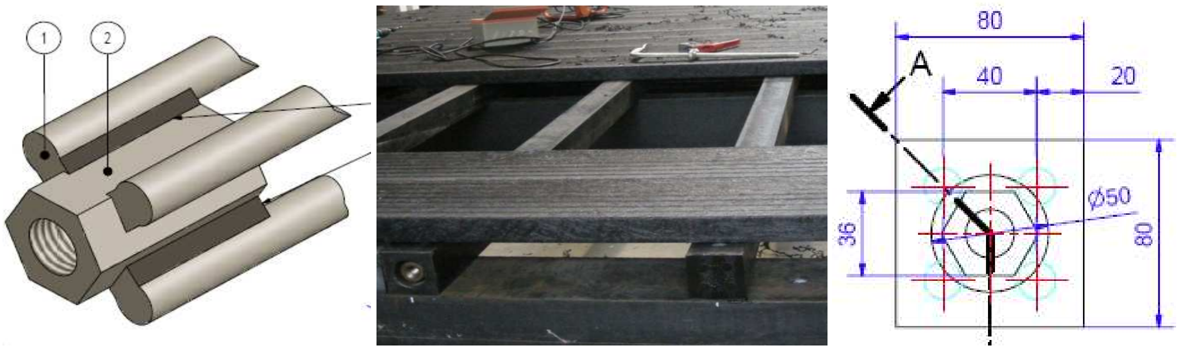


Figure 30: Left: The connection of the steel reinforcement to the end nut. Middle: The deck construction. Right: The dimensions of the longitudinal beams.

A simple thread is used for the connection of the beams (Figure 31). First the thread is fully screwed into the nut of one beam. The other beam is screwed onto the free part of the thread until the beams meet very closely (Figure 32) and then with the help of a special tools the beams are tightened even more (Figure 33). The longitudinal beams are laied on the transversal beams and then over longitudinal beams the deck planks are placed (Figure 34). As a last step the railing beams are connected on the top of the railing piles and the bridge is finished (Figure 21).

Figure 31: The KLP steel reinforced beams with reinforcement finished by nuts at the ends and the thread used for the connection of the beams.

Figure 32: Connecting of the beams by rotating the whole 3 m long beam.

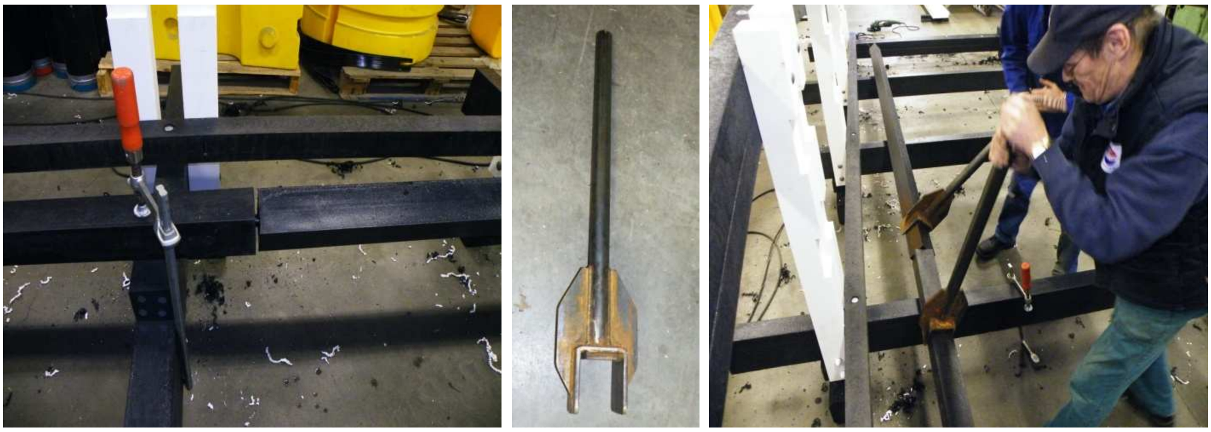


Figure 33: the beams are connected very tightly with the help of tools specially made for this purpose.

Figure 34: The deck construction.

3 Structural design of the pedestrian bridge

At first, three bridge designs (deck arch bridge, tied arch and truss) have been suggested and deflections under the worst loading combination were calculated with the help of the FrameMaster program. Then the best design was chosen and this design was further developed in chapter 4. The choice was based on the calculated deflections, basic behaviour of the structures, a construction procedures and advantages and disadvantages of each design.

3.1 General information

3.1.1 Dimensions

In most of the European countries the maximum length of an element transported on the road without special arrangements is 12 m (10). Therefore, the 12 m length of the bridge is chosen for 3 basic designs. The width of the bridge is 3 m.

3.1.2 Load

The bridge is going to be used by pedestrians and cyclist, but an unintended vehicle can pass the bridge (no arrangements against entering the car on the bridge will be available).

Based on the Eurocode provisions NEN-EN 1991-2 Eurocode 1: Action on structures – Part 2: Traffic loads on bridges (11) and National Annex to NEN-EN 1991-2+C1/NB (12), following loads are recommended for the design of a pedestrian bridge:

- Self-weight
- Three models, mutually exclusive
    - Uniformly distributed load q_{fk} = 5 kN/m^2 (from pedestrians, NEN-EN 1991-2+C1/NB, 5.3.2.1)
    - Concentrated load Q_{fwk}: is not considered because of possible presence of service vehicle (NEN-EN 1991-2, 5.3.2.2(3))
    - Loads representing service vehicles, Q_{serv} (NEN-EN 1991-2+C1/NB, 5.3.2.3,):
        - Two axle loads Q_{sv1} = 25 kN and Q_{sv2} = 25 kN
        - Two axles with wheel base of 3 m
        - A wheel-centre to wheel-centre of 1.75 m
        - Square contact area of side 0.25 m (Figure 35)

Figure 35 is taken from NEN-EN 1991-2 Eurocode 1: Action on structures – Part 2: Traffic loads on bridges (11) and National Annex to NEN-EN 1991-2+C1/NB (12) specifies other load per axles, different distance of wheel base and contact area.

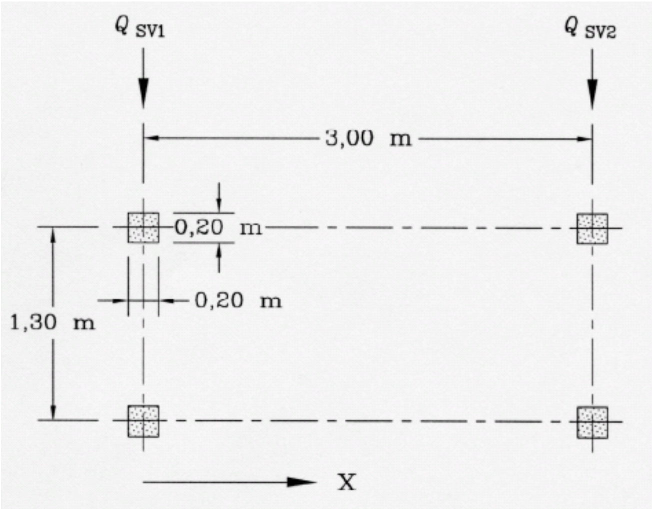


Figure 35: The dimensions of the wheels and their position according NEN-EN 1991-2. The National Annex specifies a wheel-centre to a wheel-centre of 1.75 m and a contact square area of 0,25 m.

The tied arch and truss bridge (described in Section 3.2.2 and 3.2.3) have the load bearing structures on sides of the bridge. The deck arch bridge has arches placed below the deck in a distance of 0.5 m (described in section 3.2.1). This creates a large difference in distribution of load. For the truss and tied arch bridge the uniformly distributed load has greater effect on bridge deflection. Each truss or tied arch is carrying half of the deck (1.5 m) of uniformly distributed load. In deck arch bridge the load bearing arches are carrying only 0.5 m of uniformly distributed load and the vehicle can be positioned directly on the arch. Therefore the behavior under the load from service vehicle is chosen as decisive for the comparison of all three designs.

3.2 3 basic designs

The KLP with steel reinforcement is a very unique combination of materials and also the KLP material itself is a rather weak material, hence the beams with steel reinforcement are used in all three designs. It is important for the reinforcement to be connected over the all load bearing structure in order to be able withstand required loads. The connections, which are already successfully used in construction of Royal Lankhorst Euronete's pedestrian bridges, are also applied in the 3 basic designs.

3.2.1 Deck arch bridge

The load bearing structure of the deck arch bridge is composed of curved arches and steel ropes ties, which are placed below the deck of the bridge. The main load bearing structures are arches constructed in a same manner as in the KLP arch bridge. The arch is composed of two long beams with different cross-sectional area (Figure 36), curved and bolted together. The assembly of the arch is described in the Section 2.3.3.1.

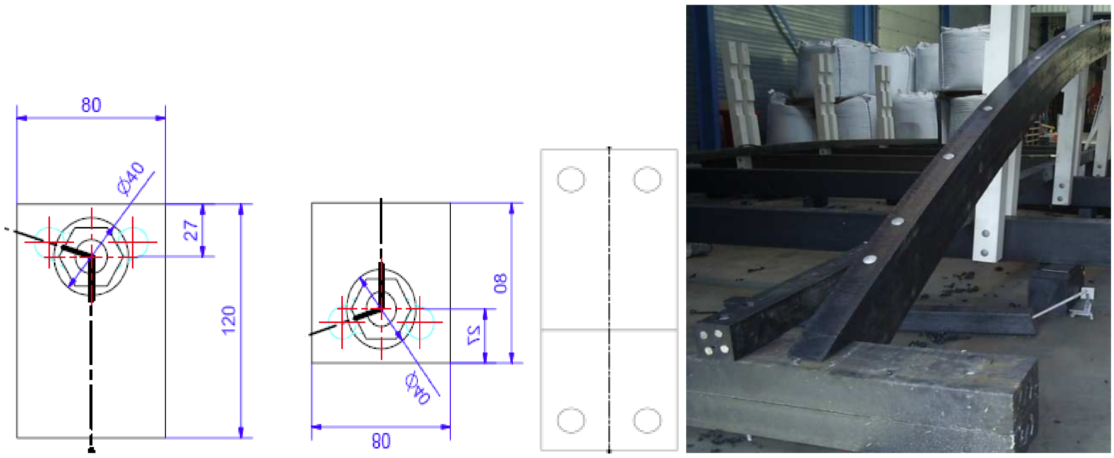


Figure 36: The arch composition – two beams with different cross-sectionalal area non-symmetrically reinforced with steel reinforcement.

The load is acting directly on the top of the arches. The bridge should be accessible to bikes and wheel chair, therefore arches have large radius and their height is only 1 meter (Figure 37).

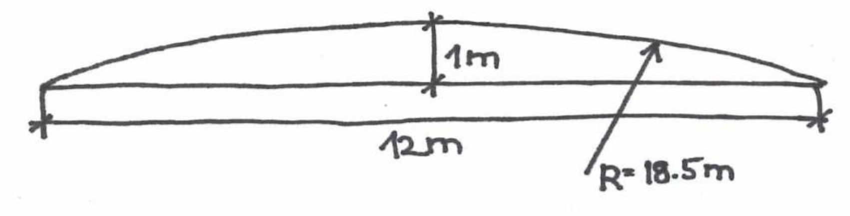


Figure 37: Sketch of the deck arch with basic dimensions.

3.2.1.1 Connections

The rope ties are holding the horizontal forces of the arch. The arches are connected to the transversal end beams and on these end beams the ties are fastened. Ties could be fastened with steel strap which would go along the perimeter of the end beam, where ties would be connected through nuts. Another option is that ties could go through the end beams and they could be fixed with a nut at the end as presented in Figure 38.

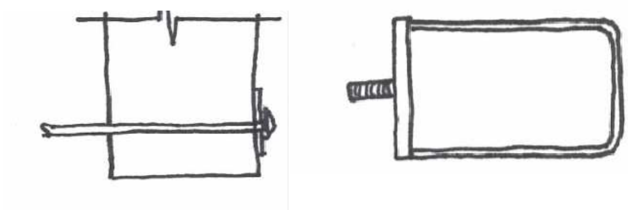


Figure 38: Sketch of the connection of the ties to the end beam.

3.2.1.2 Deck

The deck planks are laid directly on the top of the arches. The deck planks need to be supported every 0.5 m (in order to maintain required deflection) and therefore the bridge is composed of 7 arches. Sketch of the deck composition is depicted in Figure 39.

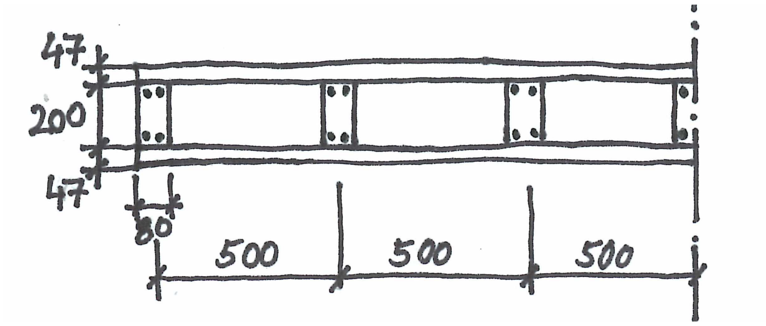


Figure 39: Sketch of the cross-section of the tied-arch bridge.

3.2.1.3 Load

The load from the bridge deck is acting directly on the arches. Each arch is carrying only 0.5 m of the uniformly distributed load. The load from the service vehicle can be considered as a point load and this load is having greater effect on behaviour of the bridge.

The axle loads of service vehicle are both Q_{sv} = 25 kN. The axle load recalculated per wheels is 12.5 kN.

The worst load for the behaviour of the bridge is when the vehicle is acting with one axle in the middle of the bridge span (Figure 40). This load position is used in FrameMaster calculation.

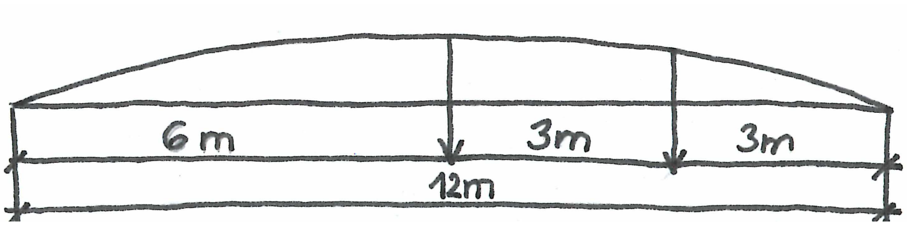


Figure 40: Sketch of the vehicle acting with one of the axles in the middle of the deck arch.

3.2.1.4 Calculation

In the FrameMaster program calculation input data for two different elements have to be determined. The deck arch is composed of the arch and tie.

The arch

- Cross-sectional area of the arch: A = 200*80 = 16000 mm^2
- Tensile strength and compression strength: f_{tk} = f_{ck} = 25.1 MPa (in reality compression strength might be higher and it is conservative to assume that compression strength is same as tensile strength)
- Young's modulus of elasticity for given cross-sectional area: E = 10500 MPa

- It is not really known yet how the KLP material behaves together with steel reinforcement. Therefore, it is assumed that the entire load is carried by the steel and KLP does not have any contribution. The Young's modulus of elasticity is calculated from relation: $(EI)_{cross} = (EI)_{reinforcement}$. Where the product of Young's modulus and moment of inertia of steel should be equal to product of Young's modulus and moment of inertia of the cross-section. The Young's modulus of steel reinforcement is known. The moment of inertia of steel bars and moment inertia of the cross-section are calculated from the given dimensions (Figure 49). The Young's modulus of the cross-section is then obtained: $E_{cross} = (EI)_{reinforcement} / I_{cross}$

The ties

- Diameter: 20
- Young's modulus: 210 Gpa
- Yield strength: f_{yk} = 355 MPa

The deck arch is simply supported and it is loaded by two point loads of 12.5 kN. One point load is acting in the middle of the bridge span – 6 m, and the other load is acting in a distance of the wheel base from the first load (wheel base is 3 m, see Section 3.1.2).

The deformed deck arch is presented in Figure 41, Figure 42 and Figure 43. The internal forces of the deck arch elements are represented by percentage of usage.

The calculation shows the maximum deflection is 55.6 mm, which does not fulfil the condition: $\delta < \frac{l}{250} = \frac{12000}{250} = 48$ mm.

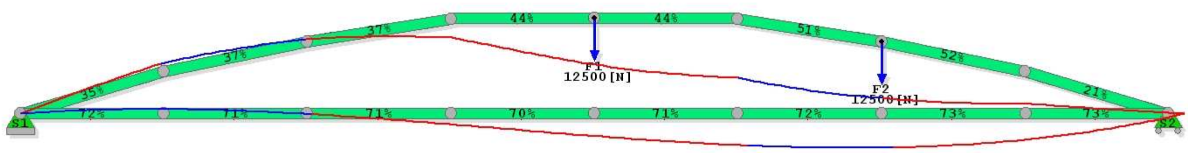


Figure 41: The results of the deck arch calculation. The maximum deflection was 52.6 mm. The internal forces in the members are represented by percentage to which the member is used under given load. The self weight of the bridge is not considered.

In the calculation of deck arch bridge, when self weight was applied, the result showed very large displacement 1250 mm of the ties (Figure 42). Therefore additional hangers had to be added to the model (Figure 43).

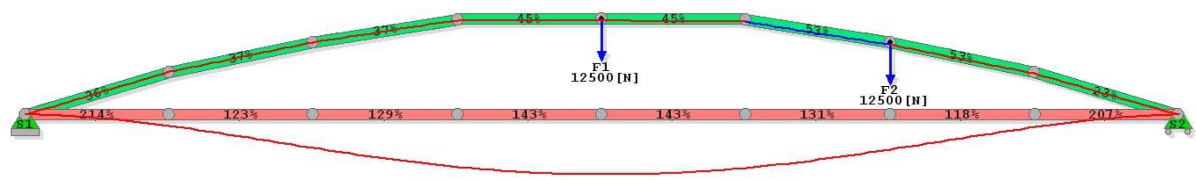


Figure 42: The results of the deck arch calculation when applied self weight. The deflection of the ties was 1250 mm.

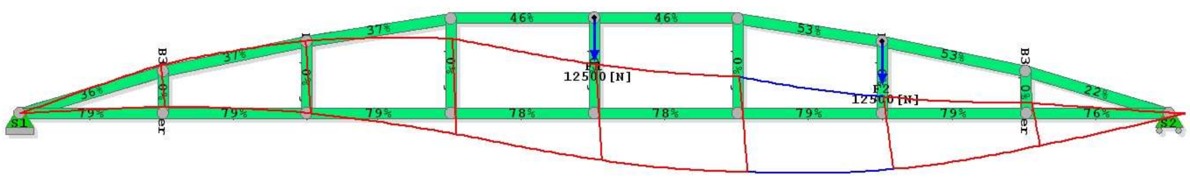


Figure 43: The results of the deck arch calculation when applied self weight and additional hangers. The deflection of the bridge was 55.6 mm.

3.2.2 Tied-arch bridge

The tied-arch bridge design is based on the design of already produced KLP arch bridge. The main load bearing structures are arches constructed in a same manner as in the KLP arch bridge. The arch is composed of two long beams with different cross-sectional area (Figure 36), curved and bolted together. The assembly of the arch is described in the Section 2.3.3.1.

In order to obtain larger (12 m) span the arch has smaller radius and the height of the arch is 2.5 m (which should be sufficiently high above the human height). The deck is connected to the arch through hangers in 1.5 m distance. The basic dimensions of the tied-arch bridge are presented in Figure 44.

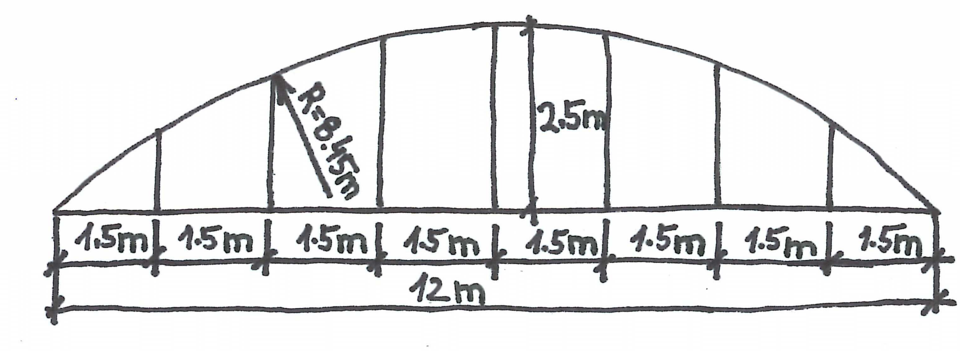


Figure 44: Sketch of the tied-arch with basic dimensions.

3.2.2.1 Connections

The hangers of the tied-arch bridge are steel bars or ropes. On the top the hangers could be connected to the arch through bolts which are already part of the arch (the bolts which hold two beams together and keep the curvature of the arch). Another option for connection is that there could be steel strap going along a perimeter of the arch's cross-section. Simple sketches of such joints are shown in Figure 45.

On the bottom the hangers could go through the transversal beams. On the bottom side of the transversal beams hangers would be fixed with nut and underlay plate, which would help to avoid stress concentration (Figure 45).

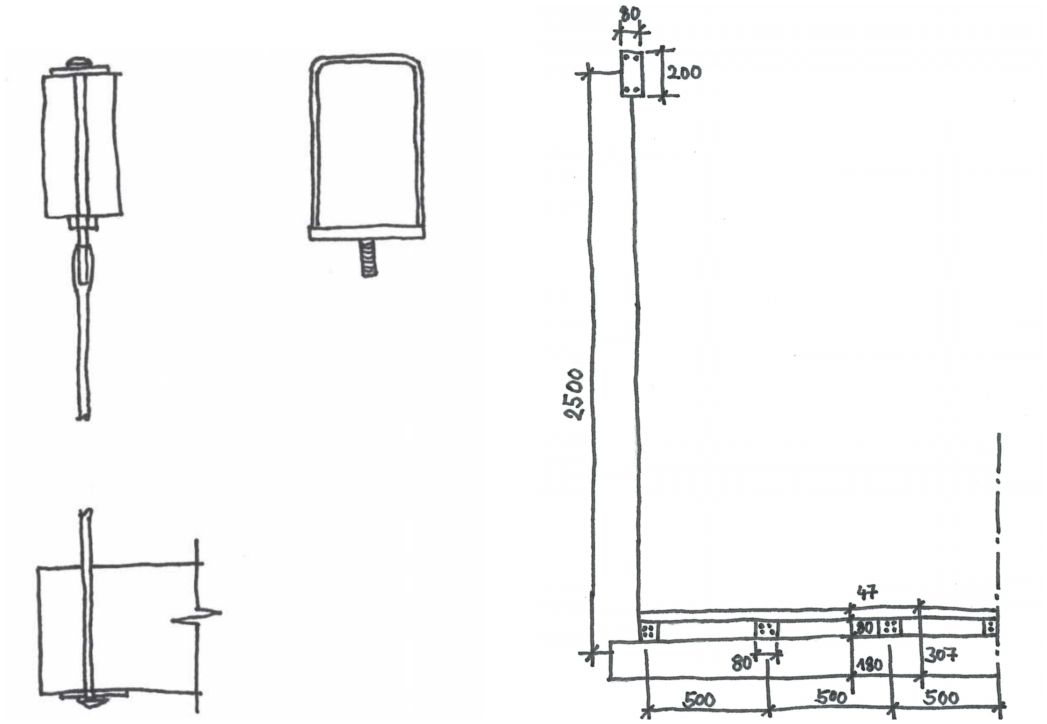


Figure 45: ***Left:*** **Sketch of the connection of hangers.** ***Right:*** **Sketch of the cross-section of the tied-arch bridge.**

3.2.2.2 Deck

The deck composition is same as in the KLP arch bridge (Figure 34). The transversal beams are connected through hangers to the arch (in KLP arch bridge it is realised through railing poles). On the transversal beams the longitudinal beams are placed in a distance of 0.5 m and the deck planks come on the top of the longitudinal beams. A sketch of the cross-section of the tied-arch bridge is depicted in Figure 45.

3.2.2.3 Load

The load from the bridge deck is acting on the arch through hangers and the transversal beams. Therefore the load acting on the tied-arch bridge can be considered as point loads.

The axle loads of service vehicle are both Q_{sv} = 25 kN. The axle load recalculated per wheels is 12.5 kN.

In the case of the tied-arch bridge the load bearing structures (tied-arches) are on the sides of the bridge. The bridge is 3 m wide and vehicle can be closer to one side of the bridge and this side would carry higher load then the other side. The 50 % increased load is considered for the case of the vehicle close to one side. Hence, it gives 18.75 kN.

The worst load for the behaviour of the bridge is when the vehicle is acting with one axle in the middle of the bridge span (Figure 46). This load position is used in FrameMaster calculation.

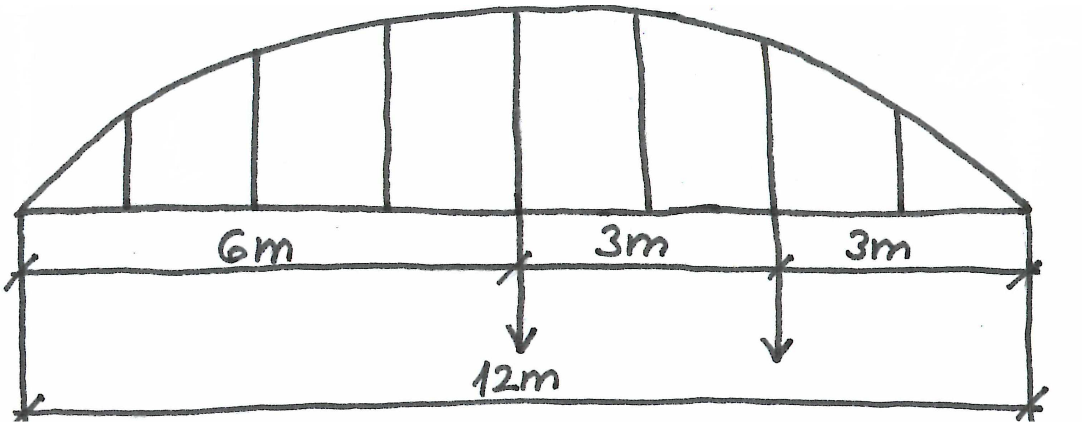


Figure 46: Sketch of the vehicle acting with one of the axles in the middle of the tied-arch.

3.2.2.4 Calculation

In the FrameMaster program calculation input data for three different elements have to be determined. The tied-arch is composed of the arch, hangers and the longitudinal beams of the deck.

The arch

- Cross-sectional area of the arch: A = 200*80 = 16000 mm^2
- Tensile strength and compression strength: $f_{tk} = f_{ck} = 25.1$ MPa (in reality compression strength might be higher and it is conservative to assume that compression strength is same as tensile strength)
- Young's modulus of elasticity for given cross-sectional area: E = 10500 MPa
    - It is not really known yet how the KLP material behaves together with steel reinforcement. Therefore, it is assumed that the entire load is carried by the steel and KLP does not have any contribution. The Young's modulus of elasticity is calculated from relation: $(EI)_{cross} = (EI)_{reinforcement}$. Where the product of Young's modulus and moment of inertia of steel should be equal to product of Young's modulus and moment of inertia of the cross-section. The Young's modulus of steel reinforcement is known. The moment of inertia of steel bars and moment inertia of the cross-section are calculated from the given dimensions (Figure 31). The Young's modulus of the cross-section is then obtained: $E_{cross} = (EI)_{reinforcement} / I_{cross}$

The longitudinal beams

- There are 7 longitudinal beams over the deck cross-section. The calculation is based on the assumption that half of the longitudinal beams is working for each arch. Therefore larger cross-sectional area of the longitudinal beams is used in the calculation: A = 3*80*80 = 19200 mm^2
- Tensile strength and compression strength: f_{tk} = f_{ck} = 35.3 MPa (in reality compression strength might be higher and it is conservative to assume that compression strength is same as tensile strength)
- Young's modulus of elasticity for given cross-sectional area: E = 25100 MPa (calculated in a same manner as the stiffness of the arch)

The hangers

- Diameter: 10 mm
- Young's modulus: 210 Gpa
- Yield strength: f_{yk} = 355 MPa

The tied-deck arch is simply supported and it is loaded by two point loads of 18.75 kN (The FrameMaster program automatically rounds this value to 18.8 kN). One point load is acting in the middle of the bridge span – 6 m, and the other load is acting in a distance of the wheel base from the first load (wheel base is 3 m, see Section 3.1.2).

The deformed tied-arch is presented in Figure 47. The internal forces of the tied-arch elements are represented by percentage of usage.

The calculation shows the maximum deflection is 72.5 mm, which does not fulfil the condition: $\delta < \frac{l}{250} = \frac{12000}{250} = 48$ mm.

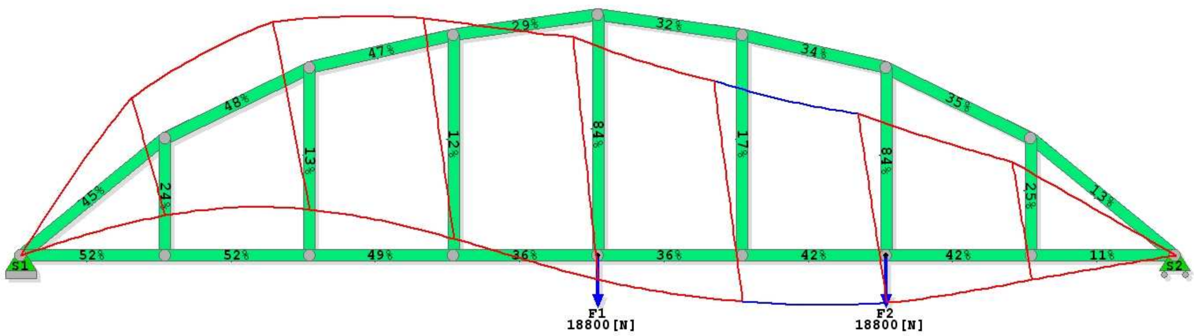


Figure 47: The results of the tied-arch calculation. The maximum deflection was 72.5 mm. The internal forces in the members are represented by percentage to which the member is used under given load.

3.2.3 Truss bridge

The truss bridge is composed of two trusses placed instead of the railing on both sides of the bridge. The idea is to have one length of steel reinforced beam, which could be placed into the chords of the truss and also into diagonals. For 12 m long bridge 1.5 m long beam element is the most suitable (Figure 48). The beams of the cross-section 80 x 80 mm with 4 steel bars reinforcement finished with nut, which are used for the longitudinal beams in the KLP arch bridge (Figure 31 and Figure 49), are chosen as a “start" truss elements.

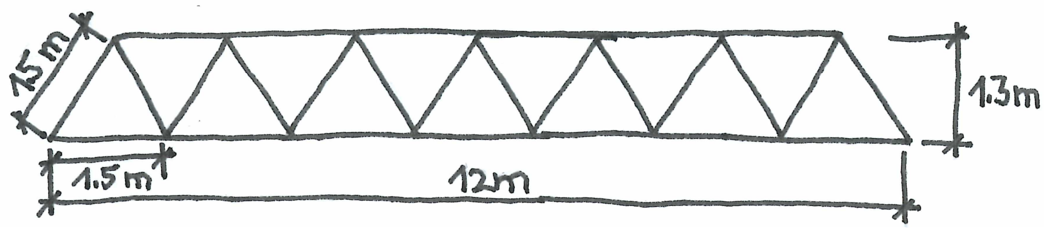


Figure 48: a sketch of a truss beam with basic dimensions – equilateral composition

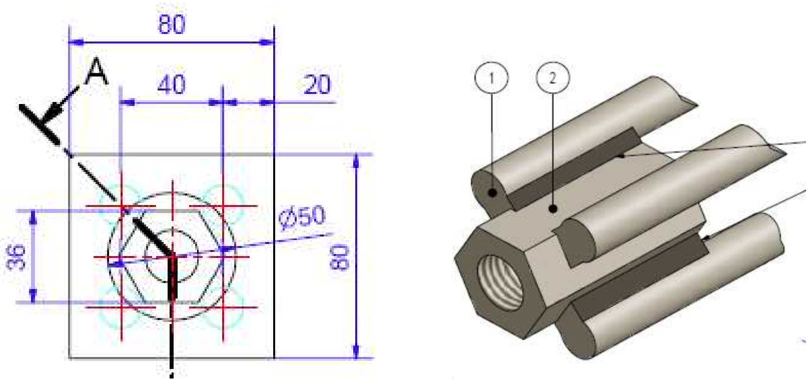


Figure 49: The cross-section and steel reinforcement of the steel reinforced beam used for truss elements.

3.2.3.1 Connections

For the connections of the truss elements a special joint has to be produced. First idea of such a joint is presented in Figure 50. It is composed of two steel rods diameter 20 mm finished by a tread at both ends of the rods. One rod is bent into an angle of 60 degrees and welded to the other rod, which is straight. Then chords and diagonal beams can be connected to the joint through the large nuts. The problem is how the truss would be assembled and where would be left and right thread. In the first calculation a fixed connection has been assumed.

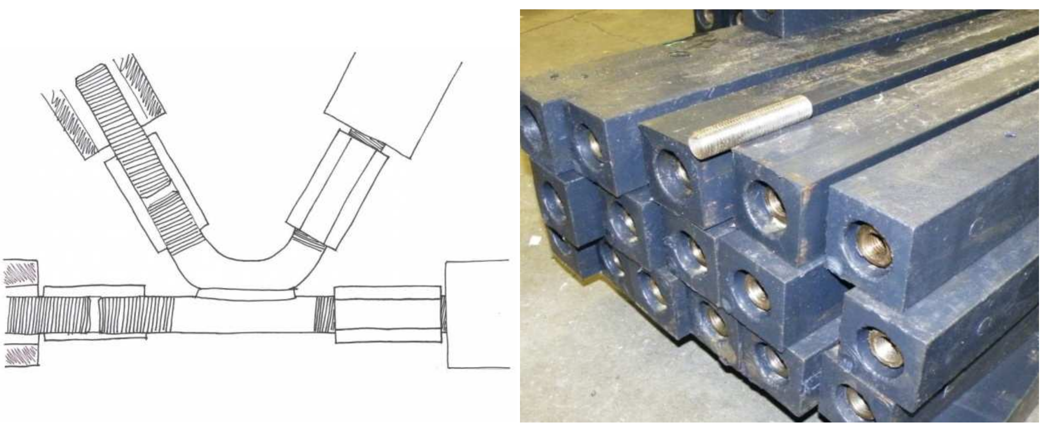


Figure 50: Sketch of a joint for truss

3.2.3.2 Deck

The deck composition is same as in the KLP arch bridge (Figure 34 and Figure 51). Transversal beams are laid on the bottom chord of the truss. On the transversal beams the longitudinal beams are placed at a distance of 0.5 m and the deck planks come on the top of the longitudinal beams.

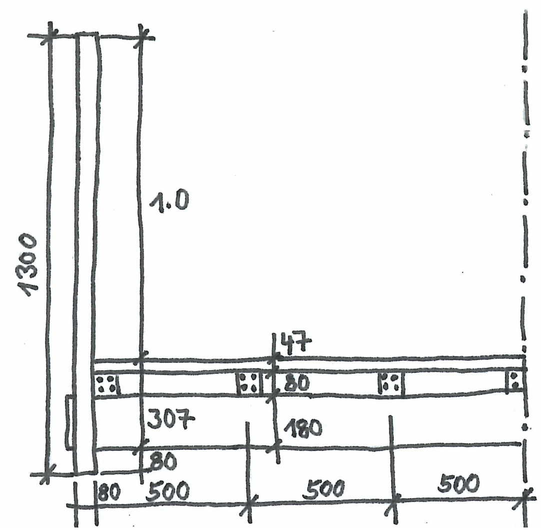


Figure 51: Sketch of a cross-section of the truss bridge.

3.2.3.3 Load

The load from the bridge deck is acting on the truss through the transversal beams. Therefore the load acting on the truss can be considered as point load.

The axle loads of service vehicle are both Q_{sv} = 25 kN. The axle load should be recalculated per wheels, which is 12.5 kN.

In the case of the truss bridge the load bearing structures (trusses) are on the sides of the bridge. The bridge is 3 m wide and the vehicle can be closer to one side of the bridge. This side would carry higher load then the other side. The 50 % increased load is considered for the case of the vehicle close to one side. Hence, it gives 18.75 kN.

The worst load for the behaviour of the bridge is when the vehicle is acting with one axle in the middle of the bridge span (Figure 52). This load position is used in FrameMaster calculation.

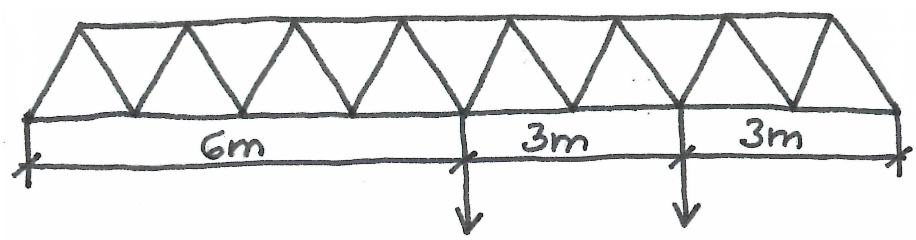


Figure 52: Sketch of the vehicle acting with one of the axles in the middle of the truss.

3.2.3.4 Calculation

In the FrameMaster program calculation few input data has to be determined.

- Cross-sectional area of the truss element: A = 80*80 = 6400 mm^2
- Tensile strength and compression strength: f_{tk} = f_{ck} = 35.3 MPa (in reality compression strength might be higher and it is conservative to assume that compression strength is same as tensile strength)
- Young's modulus of elasticity for given cross-sectional area: E = 25100 MPa
    - It is not really known yet how the KLP material behaves together with steel reinforcement. Therefore, it is assumed that the entire load is carried by the steel and KLP does not have any contribution. The Young's modulus of elasticity is calculated from relation: $(EI)_{cross} = (EI)_{reinforcement}$. Where the product of Young's modulus and moment of inertia of steel should be equal to product of Young's modulus and moment of inertia of the cross-section. The Young's modulus of steel reinforcement is known. The moment of inertia of steel bars and moment inertia of the cross-section are calculated from the given dimensions (Figure 49). The Young's modulus of the cross-section is then obtained: $E_{cross} = (EI)_{reinforcement} / I_{cross}$

The truss is simply supported and it is loaded by two point loads of 18.75 kN (The FrameMaster program automatically rounds this value to 18.8 kN). One point load is acting in the middle of the bridge span – 6 m, and the other load is acting in a distance of the wheel base from the first load (wheel base is 3 m, see Section 3.1.2).

The model and deformed truss are presented in Figure 53. The internal forces of the truss elements are represented by percentage of usage.

The calculation shows the maximum deflection is 9.97 mm, which fulfils the condition: $\delta < \frac{l}{250} = \frac{12000}{250} = 48$ mm.

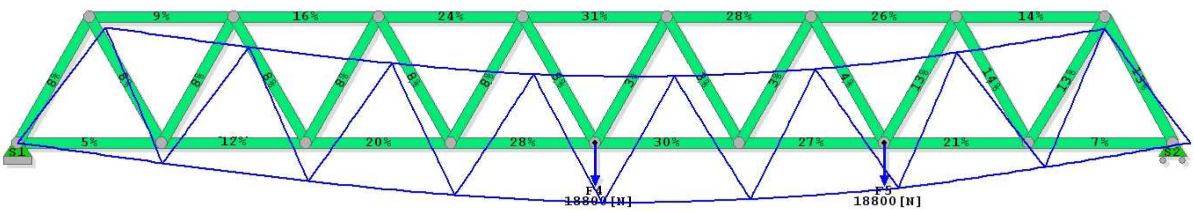


Figure 53: The results of the truss calculation. The maximum deflection was 9.97 mm. The internal forces in the members are represented by percentage to which the member is used under given load.

3.3 Conclusion

The results, advantages and disadvantages of all three designs are summarized in the Table 3.

The deck arch bridge deflection of 55.6 mm does not fulfil allowed deflection of 48 mm and the composition of the steel ties would require too much steel material. Although, the height of the deck arch is only one meter, passing of the bridge by bike or the wheel chair could be uncomfortable for users.

Also the tied-arch bridge deflection of 72.5 mm does not fulfil the requirements of allowable deflection. One of the requirements for the bridge design is that the bridge should be possible to assemble in the factory and then transport it to a desired place. The height of the tied-arch bridge is 2.5 m in a first calculation and it is the most probable it would be necessary to increase the height of the arch, when provided with detailed calculation. The height of the tied-arch could be problematic for the transport of the bridge.

The truss design showed the best behaviour. The deflection was only 9.97 mm. Once the mould of required length of the truss element and joints would be made the production and assembly of the truss could be rather simple. Therefore the truss bridge design was chosen for the further calculation.

Table 3: Summary of 3 basic designs

Bridge type	Advantages	Disadvantages	Deflection
Deck arch	Known production and assembly of the arch, easy for variation of the bridge span and width, integrated design	Production and protection of joints and steel hangers	56 mm
Tied-arch	Known production and assembly of the arch, easy for variation of the bridge span	Might be complicated for transport, production and protection of joints and steel hangers	73 mm
Truss	Easy to assemble, easy in variation of the bridge span, truss can be used for railing	Change of the length of the beam mould or production of a new mould, production and protection of joints	10 mm

4 Truss

4.1 Truss bridge description

The design of the truss bridge with lower deck has been chosen for further elaboration. The FEM program SCIA Engineering (version 2009.0) has been used for detailed calculation. The model of the truss bridge can be seen in Figure 54.

The whole structure is carried by two trusses on sides, which also serve as a railing. The truss is composed of 1.5 m long elements of 80 x 80 mm with four steel reinforcing bars ø16. Trusses support transversal beams. Two transversal beams 160 x 80 mm with four steel bars ø16 are placed close to each connection of the truss, where the diagonals meet with bottom chord. On the transversal beams the longitudinal beams of a cross-section 80 x 80 mm with four steel bars ø16 are placed and on top of the longitudinal beams the deck planks are laid. Trusses are supported by end beams which are composed of two beams of cross-section 180 x 80 mm, each beam with four steel bars ø16. The ends of the longitudinal beams are also connected to these large end beams.

The whole composition of the bridge can be seen in Figure 55.

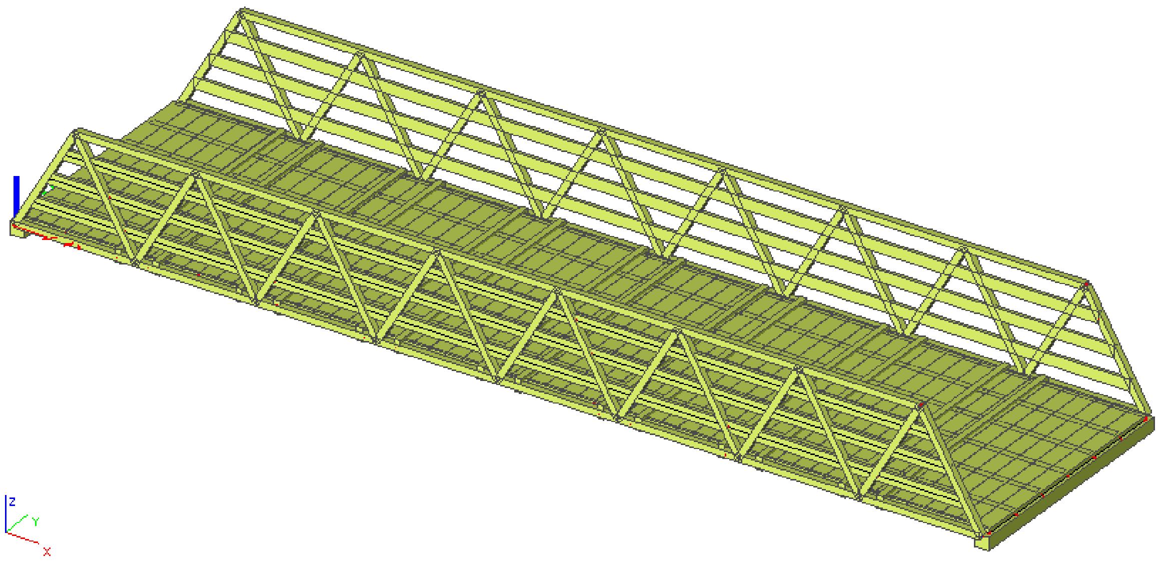


Figure 54: The truss bridge modelled in SCIA Engineering FEM program.

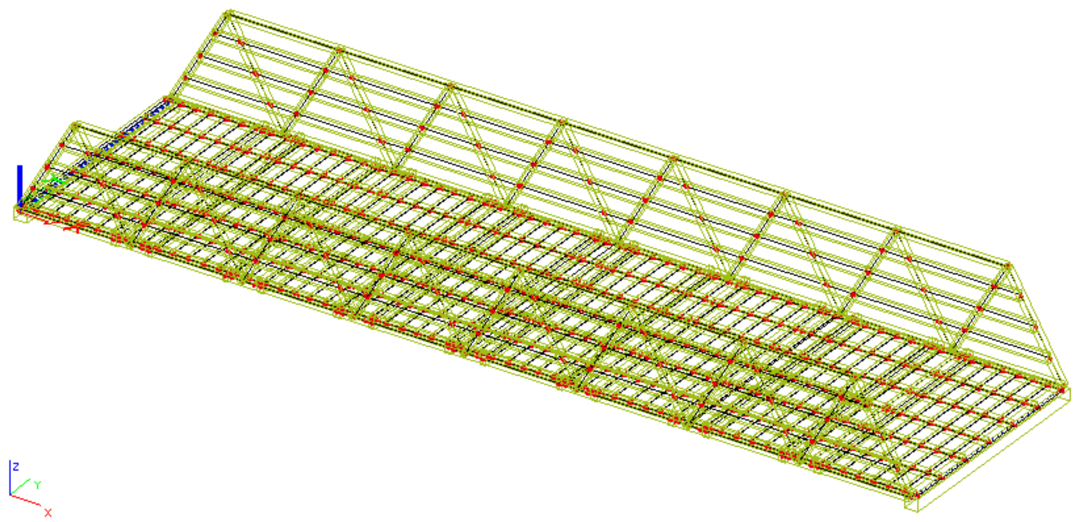


Figure 55: The skeleton of truss bridge modelled in SCIA Engineering FEM program.

4.2 Models description

Three different models were calculated in SCIA Engineering.

The SCIA model consists of 1D elements (truss, beams) and 2D shell elements (deck planks). It is assumed that the joints in the deck grid or truss are ideally fixed or ideally pinned. No additional stiffness in rotation or translation of the joint has been assumed.

The dimensions and input data for SCIA models are presented in Table 4.

Table 4: Input data used in SCIA Engineering models.

Element	Young's modulus [MPa]	Tensile and compression strength [MPa]	Cross-section [mm^2]	Area [mm^2]	Max. normal force [kN]
Truss	29300	64.5	80 x 80	6400	413
Longitudinal beam	29300	64.5	80 x 80	6400	413
Transversal beam	24000	49.0	160 x 80	12600	617
End beam	12100	30.0	2 x 80 x 180	28800	864
Deck plank	2000	33.3	47 x 200	9400	313

First analyzed model is with fixed truss connections. The idea of a joint for truss bridge depicted in Figure 50 was discussed with the company specialist on steel works and production of moulds in Royal Lankhorst Euronete. The joint would be possible to produce but special attention has to be paid to positions of left and right threads on joint and on the truss elements. It was also pointed out that the assembly could be rather complicated since diagonal and chord elements would need to be screwed onto the joints simultaneously on top and bottom side.

Therefore a different joint, which would enable easier assembly of a truss, was created. Also it would be convenient, if the length of the bridge could be even more variable. With the welded joint (Figure 50) there would be fixed angle of 60 degrees and therefore only elements of same length could be connected together, which for 1.5 m long elements gives lengths of 6, 7.5, 9, 10.5, 12 m etc.

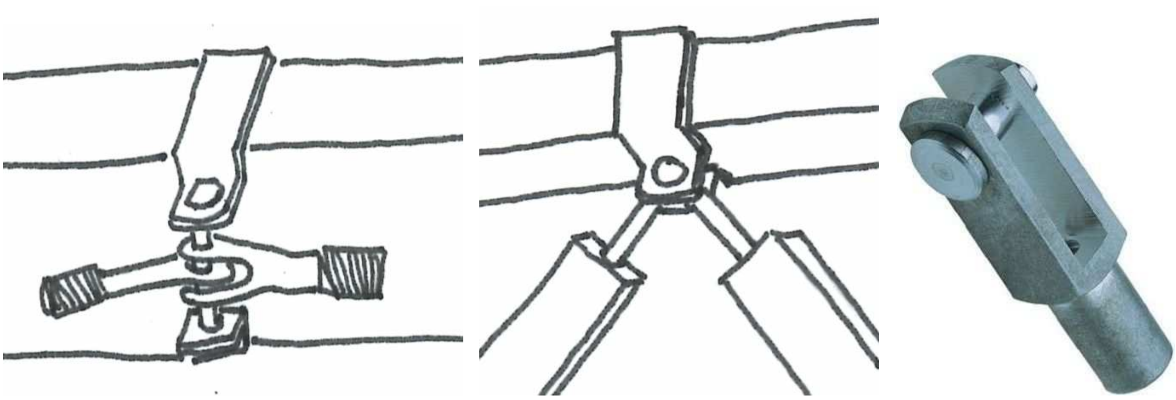


Figure 56: Sketch of a pin joint of the truss

A sketch of a pin joint is presented in Figure 56. For this type of a joint the top and bottom chord of the truss are continuous beams. The steel strap with holes at the ends is placed around the chord beam. Steel hooks or fork type joints ended with thread (presented in Figure 56) are screwed into diagonal elements. The diagonal elements are then connected with a bolt through the steel strap.

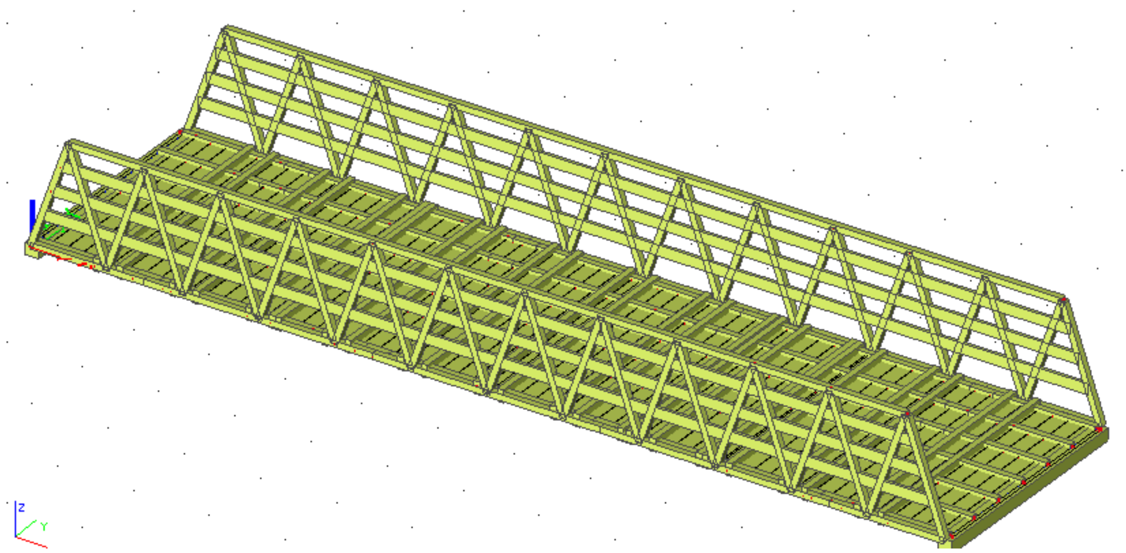


Figure 57: The model with diagonals connected with pin joints at 1 meter distance.

The pin joint enables change of the position of diagonals with respect to obtain desired length of the bridge. Therefore two models are modelled in a way that top and bottom chords are continuous beams and diagonals are connected to them with pin connections. Last model is modelled with diagonals connected with pin connection and length 1.5 m as well, but the distance between connections of diagonals is only 1 m, as it can be seen in Figure 57. The detail of a pin joint used in SCIA models can be seen in Figure 58. For the pin joint only the rotation in the plane of the truss was allowed. The list and variation of models is presented in Table 5.

The bridge in the SCIA model is simply supported with line supports for 1D member. One line support is fixed in vertical z-direction and the other line support is fixed in the vertical z-direction and horizontal y-direction as depicted in Figure 59.

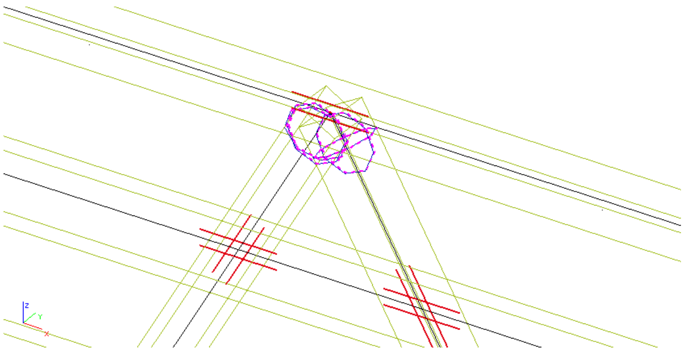


Figure 58: Detail of a pin joint in the SCIA models

Table 5: Variation of models

Model	Connection	Length of chord elements
I	Fixed	1.5 m
II	Pin	1.5 m
III	Pin	1.0 m

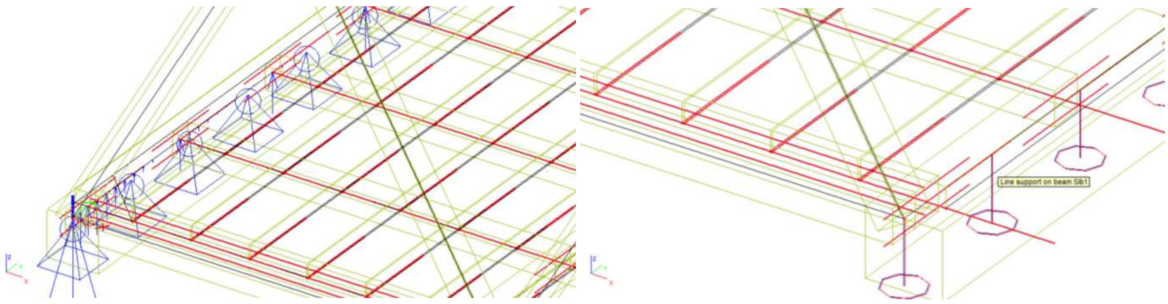


Figure 59: Simply supported bridge in the SCIA models – line supports

4.3 Load combinations

Five different load combinations are analyzed for each model. The loads acting simultaneously have to be combined with combination factors based on Table NB. 17 of National Annex NEN-EN 1990+A1+A1/C2/NB (13).

Load combinations consist:

- CO1 - uniformly distributed load
  - Vertical load: uniformly distributed load 5kN/m^2 (Figure 60)
  - Horizontal load: 10% of vertical load = 0.5 kN/m^2, active over the whole bridge in the longitudinal direction (NEN-EN 1991-2+C1/NB, 5.4; Figure 61)
  - Wind load: 0.3 x 1 kN/m^2(National Annex to NEN-EN 1991-4+A1+C1/NB, Table NB.5 (14))
  - Line load: 3 kN/m vertical and horizontal on the top of the railing (NEN-EN 1991-2+C1/NB, 4.8)
- CO2 - service vehicle in the middle (Figure 62)
  - Service vehicle according Dutch national annex NEN-EN 1991-2+C1/NB, 5.3.2.2:
    - Two axles load Q_{sv1} = 25 kN and Q_{sv2} = 25 kN
    - Two axles with wheel base of 3 m
    - A wheel-centre to wheel-centre of 1.75 m
    - Square contact area of side 0.25 m
  - Vertical load: service vehicle in the middle of the length and width of the bridge
  - Horizontal load: 30% of vertical load = 7.5 kN (NEN-EN 1991-2+C1/NB, 5.4)
  - wind load: 0.3 x 1 kN/m^2(National Annex to NEN-EN 1991-4+A1+C1/NB, Table NB.5 (14))
- CO3 - service vehicle on the side (Figure 63)
  - Service vehicle according Dutch national annex NEN-EN 1991-2+C1/NB, 5.3.2.2:
    - Two axles load Q_{sv1} = 25 kN and Q_{sv2} = 25 kN
    - Two axles with wheel base of 3 m
    - A wheel-centre to wheel-centre of 1.75 m
    - Square contact area of side 0.25 m
  - Vertical load: service vehicle in the middle of the length of the bridge and as close as possible to the truss on side
  - Horizontal load: 30% of vertical load = 7.5 kN (NEN-EN 1991-2+C1/NB, 5.4)
  - wind load: 0.3 x 1 kN/m^2(National Annex to NEN-EN 1991-4+A1+C1/NB, Table NB.5 (14))
- CO4 - only wind (Figure 65)
  - Wind load: 1 kN/m^2(National Annex to NEN-EN 1991-4+A1+C1/NB, Table NB.5 (14))
  - Vertical load: 0.4 x uniformly distributed load 5 kN/m^2 = 2 kN/m^2
  - Horizontal load: 0.4 x 10% of vertical load = 0.2 kN/m^2, active over the whole bridge deck in the longitudinal direction (NEN-EN 1991-2+C1/NB, 5.4)
- CO5 - load on railing only (Figure 64)
  - line load: 3 kN/m vertical and horizontal on the top of the railing (NEN-EN 1991-2+C1/NB, 4.8)

Table NB.10 – 5.1 of National annex NEN-EN 1991-2+C1/NB specifies that in the case of unintended vehicle on a pedestrian bridge the load from the vehicle should be taken together with reduced uniformly distributed load from pedestrians (reduction factor 0.8). The reduced uniformly distributed load should be considered in the 5 m distance from the front and back of the vehicle. The worst position of a vehicle for the behavior of the bridge is when the vehicle is in the middle of the bridge. For 12 meters long bridge it is when one axle is acting in the distance 4.5 m and other is acting in 7.5 m from the end of the bridge. In this case the 5 meter limitation is covered for the whole bridge and therefore no uniformly distributed load has to be considered together with vehicle. The load combinations are summarized in Table 6.

Table 6: Load combinations.

CO1	Uniformly distributed load
CO2	Vehicle in the middle of the bridge
CO3	Vehicle on the side of the bridge
CO4	Wind
CO5	Load on railing

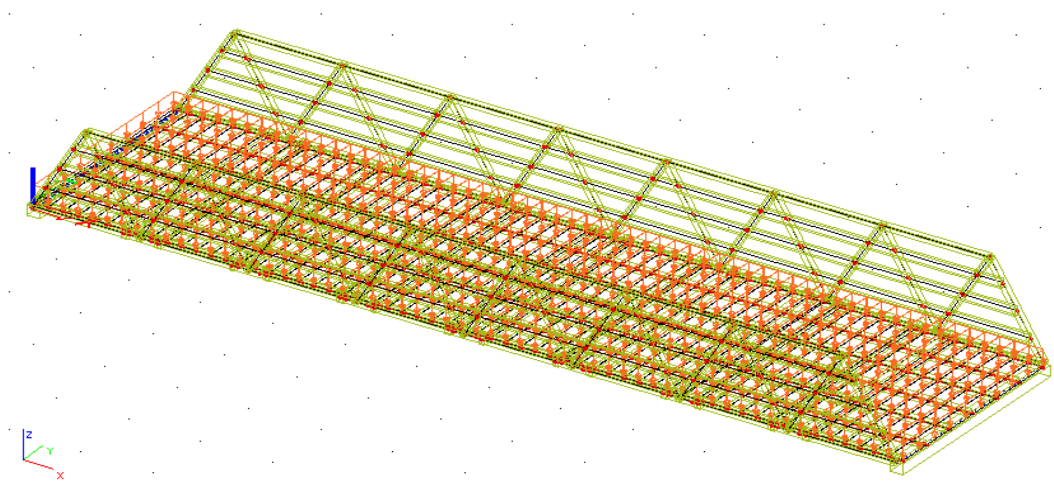


Figure 60: The load combination CO1 – uniformly distributed vertical load 5 kN/m^2.

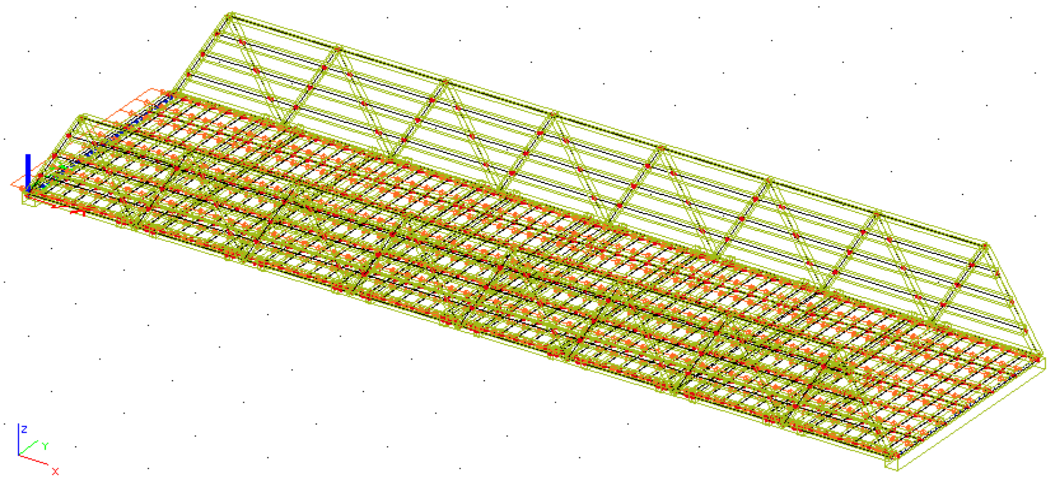


Figure 61: The load combination CO1 – uniformly distributed horizontal load 0.5 kN/m^2.

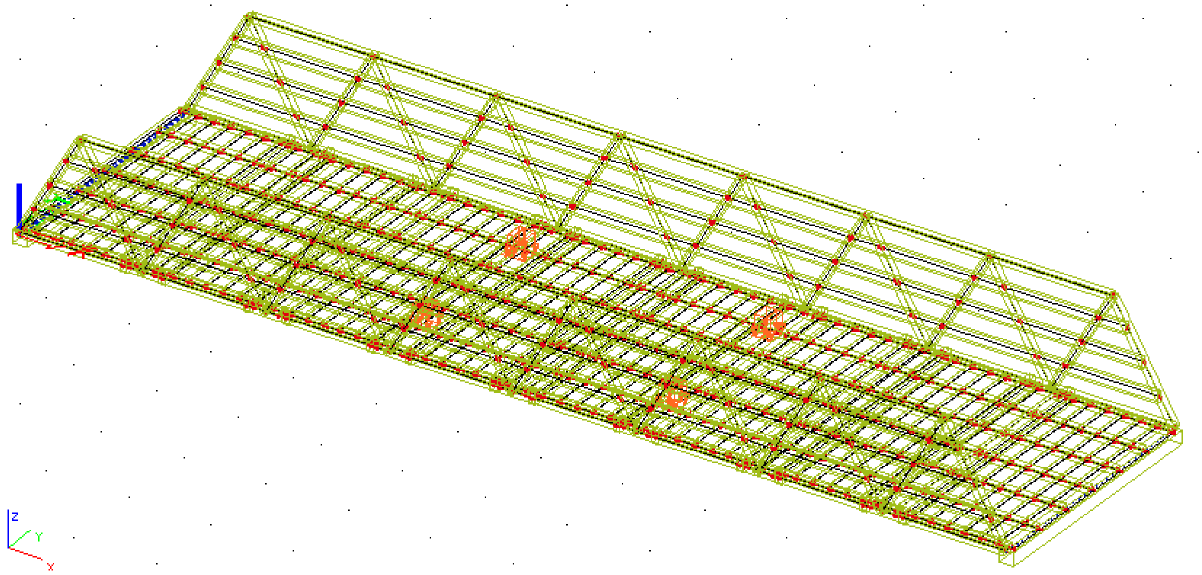


Figure 62: The load combination CO2 – service vehicle located in the middle of the bridge length and in the middle of the bridge width.

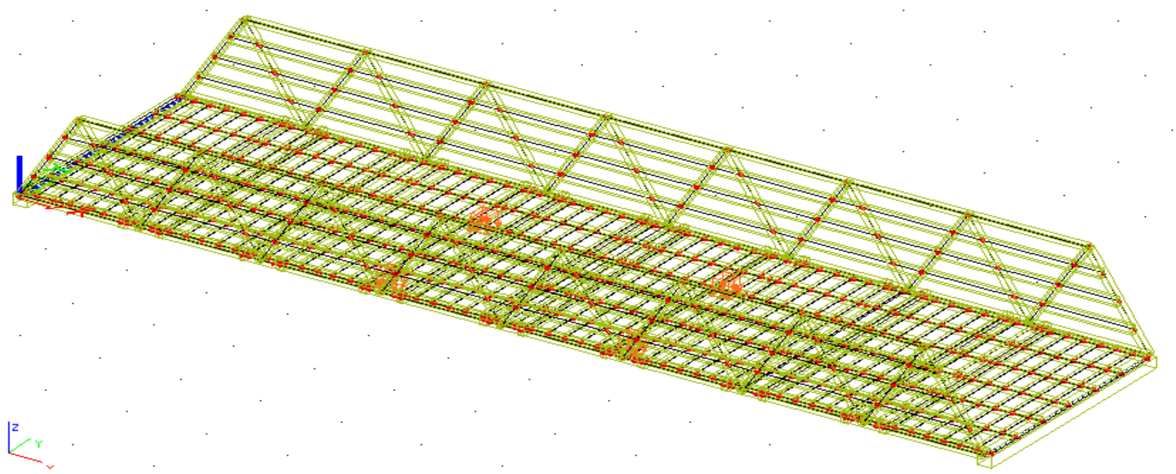


Figure 63: The load combination CO3 – service vehicle located on the side.

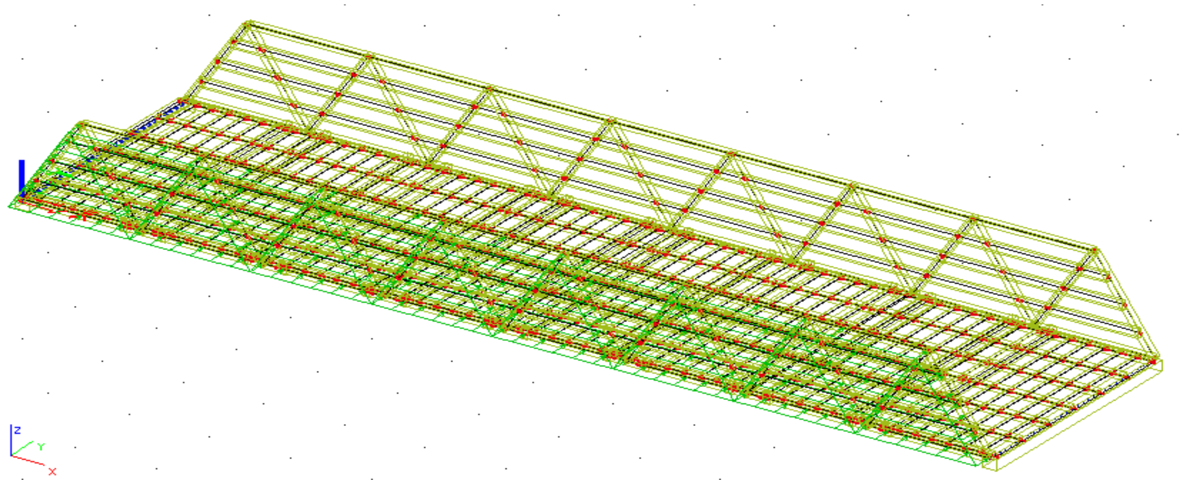


Figure 64: The load combination CO4 – wind load 1 kN/m^2.

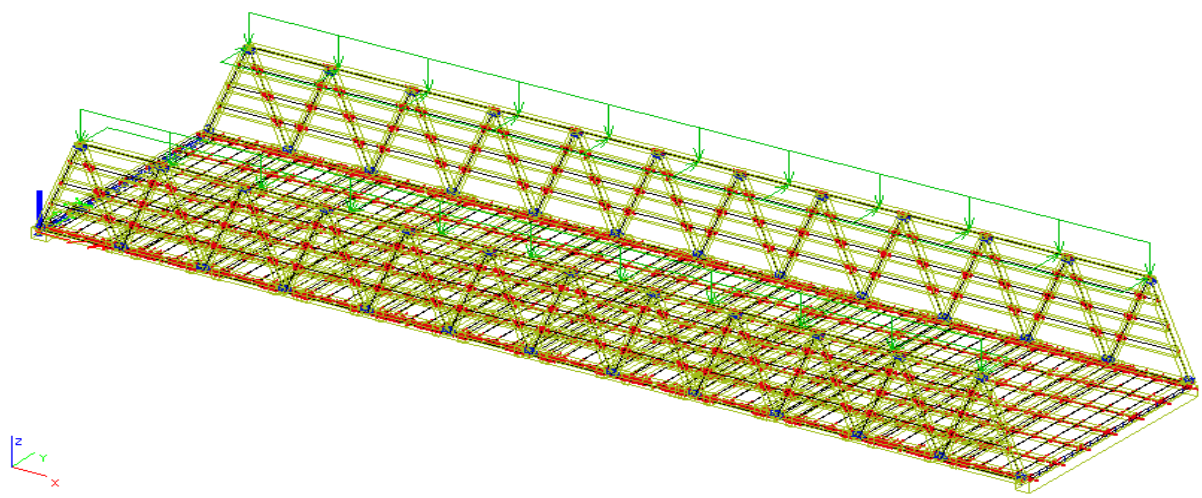


Figure 65: The load combination CO5 – line load on the top of the railing 3 kN/m.

4.4 Calculation

The Serviceability Limit State and Ultimate Limit State (15) are calculated for all three models with five load combinations. For the Serviceability Limit State calculation load factors are equal to 1 and for Ultimate Limit State load factors are 1.35 as stated in the Dutch national annex NEN-EN 1990+A1+A1/C2/NB, Table NB. 11 – A2.4(A).

$$\sum Y_{G,j} G_{k,j} + Y_{Q,1} \Psi_{0,1} Q_{k,1} + \sum Y_{Q,i} \Psi_{0,i} Q_{k,i}$$

4.4.1 Serviceability Limit State

The resulting deflections for different parts of the bridge and for all load combinations are presented in Table 7.

Maximal deflections in the vertical direction u_z are noted for Bottom chord and Deck part (Figure 68). The "Deck" part represents the whole composition of the deck – transversal beams, longitudinal beams and deck planks. In SCIA the truss, transversal and longitudinal beams are modelled as 1D elements to which the cross-sectional properties are prescribed. Therefore it is not possible in SCIA models put longitudinal beams on top of the transversal beams (as would be possible in the case of 3D solid elements) and deflections for individual parts of the deck cannot be displayed.

The allowable deflection for the truss is according the relation: $\delta < \frac{l}{250} = \frac{12000}{250} = 48$ mm.

The allowable deflection for the deck is according the relation: $\delta < \frac{l}{250} = \frac{3000}{250} = 12$ mm.

As it can be seen in the Table 7 the required vertical deflection u_z is met for all models and all combinations of the truss bridge.

The Model I. is model with fixed connections. The Model II. is identical to the Model I. only difference is that the diagonals are connected with pin connections to the top and bottom chords, which are continuous beams. In the Table 7 it can be seen that deflections of the deck and of the bottom chord of these two models almost do not differ. The pin joint is better with respect to the production and assembly of the bridge and therefore based on the SLS the Model II. is considered as better design.

The results of the Model III. show low deflection of the bottom chord and of the deck, but the number of transversals beam has increased rapidly, because the diagonals are connected to the chord in the distance of 1 meter (Figure 57). There are 22 transversal beams in the Model III., which is of 8 transversal beams more than in the Model II.

Deflection u_y represents the deflection in horizontal direction (Figure 69). Under the given load combinations the top chord is displaced in horizontal direction up to 50 mm. In SCIA model the rotation only in the plane of the truss was used for definition of the pin joints. In reality the steel joints (as sketched in Figure 56) will be very slender and some rotation in other than plane of the truss will be possible. Hence SCIA models show that the truss is not very stable in the horizontal direction. The Model II. appears to be the most economic and suitable design and therefore Model II. is further developed into 3D truss design which could restrain horizontal deflections (Section 4.5).

Table 7: Serviceability Limit State - deflection

SLS Service Limite State		Maximal deflection u_z [mm]		Maximal deflection u_y [mm]	
		Bottom chord	**Deck**	**Top chord**	**Bottom chord**
Model I	CO1	15	3	36	0.1 – 0.5
	CO2	3	4	6	
	CO3	4	3	4	
	CO4	4	5	10	
	CO5	5	1	51	
Model II	CO1	15	3	40	0.1 – 0.5
	CO2	3	7	6	
	CO3	4	4	4	
	CO4	4	5	11	
	CO5	5	1	55	
Model III	CO1	14	1	28	0.1 – 0.5
	CO2	2	3	4	
	CO3	3	2	3	
	CO4	4	3	8	
	CO5	5	1	38	

There are presented deflections of the truss bridge for various load combinations in Figure 66, Figure 67, Figure 68 and Figure 69 (these figures are only illustrative, the values may not correspond with values in the Table 7).

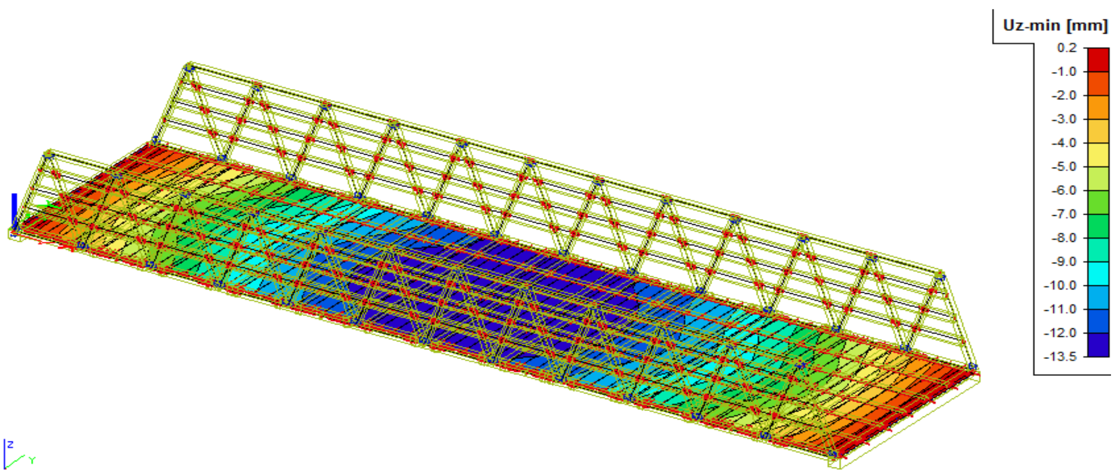


Figure 66: The deflection of a bridge under the load combination CO1 – uniformly distributed load.

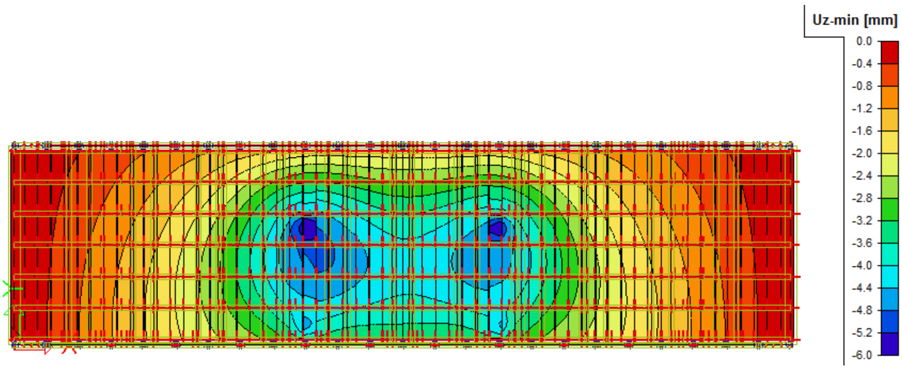


Figure 67: The deflection of a bridge under the load combination CO3 – vehicle on side.

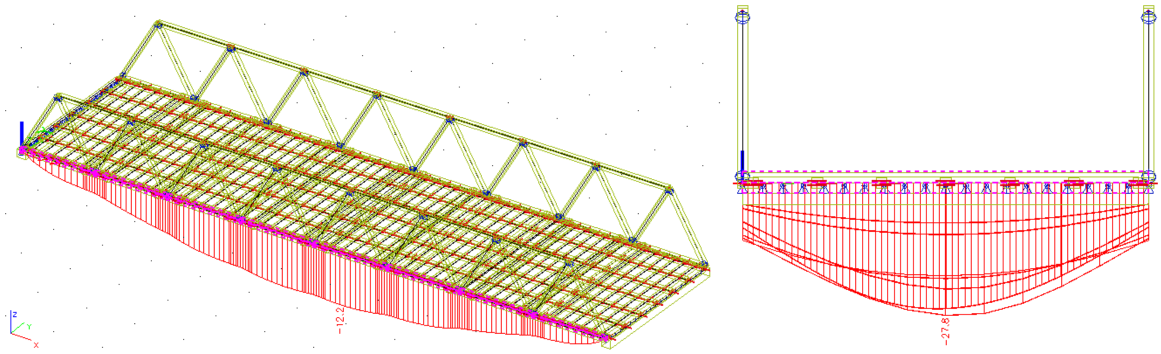


Figure 68: The deflection only in z-direction of the truss and cross-sectional view of the deck deflection – load combination CO1 - crowed

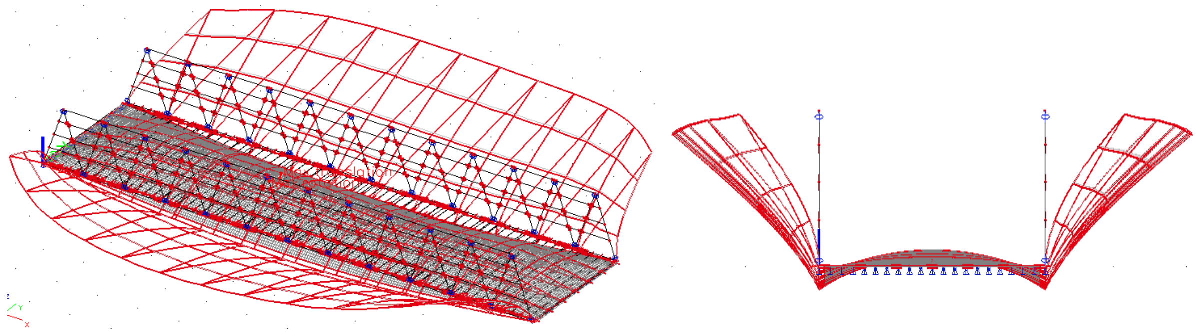


Figure 69: The magnified horizontal displacement of trusses – CO5.

4.4.2 Ultimate Limit State

In the Serviceability Limit State calculation was found that all three models fulfil the deflection requirements. Therefore all models are calculated in the ULS calculation and are presented in Table 9. In the ULS calculation the moments and normal forces of elements are checked.

4.4.2.1 Normal forces check

The largest normal force is the compression force of -210.91 kN in the top chord in the Model II. load combination CO1 (Table 9). Maximal normal force, which can be carried by truss element and based on the simple Hook's law calculation (F=σA), is 412.8 kN in compression and tension (see Table 4). All truss elements are able to withstand tension forces but in the case of compression forces the buckling could occur.

Table 8: The bridge elements properties, maximal normal forces and stresses introduced by moments.

Element	Tensile and compression strength [MPa]	Cross-section [mm^2]	Max. Element normal force [kN]	Section modulus [mm^3]	Stress caused by moment [MPa]
Truss	64.5	80 x 80	412	85333	21.5
Longitudinal beam	64.5	80 x 80	412	85333	13.0
Transversal beam	49.0	160 x 80	617	341333	14.5
End beam	30.0	2 x 80 x 180	864	768000	-
Deck plank	33.3	47 x 200	313	73633	15.0

4.4.2.2 Moment check

Moments introduce stresses in the bottom and top fibre of the structural elements. This stress caused by moment has to stay within the element properties.

The stress introduced by a moment can be calculated by dividing a moment by section modulus of a cross-section: $\sigma = \frac{M}{W}$.

Section modulus W of the rectangular cross-section is: $W = \frac{bh^2}{6}$.

The largest moments are highlighted with different colour in the Table 9. The calculated stresses are summarized in Table 8 and it is obvious that stresses introduced by moments are within the element capacities.

There are presented positions of compression, tension forces and moments of the truss and deck in Figure 70, Figure 71, Figure 72 and Figure 73 (these figures are only illustrative, the values may not correspond with values in the Table 9).

Table 9: Ultimate Limit State – moments and normal forces.

ULS Ultimate Limit State		Maximal Moment [kNm]				Maximal Normal forces in truss [kN]		
		Bottom chord	Transversal beam	Longitudinal beam	Deck planks [kNm/m]	Top chord	Diagonal	Bottom chord
Model I	CO1	1.8	4.9	1.1	0.3	-210	-97	86
	CO2	0.8	2.5	0.8	5.1*	-64	-26	55
	CO3	0.9	3.0	0.9	3.9*	-75	-29	31
	CO4	0.8	3.1	0.6	0.2*	-81	-38	31
	CO5	0.5	3.8	0.3	0.2*	-99	-47	45
Model II	CO1	1.5	4.9	0.9	0.3	-210	-97	85
	CO2	0.8	2.5	0.8	5.1*	-64	-26	25
	CO3	0.9	3.1	0.9	3.9*	-74	-29	30
	CO4	0.7	3.1	0.5	0.2*	-81	-38	31
	CO5	0.3	3.8	0.2	0.2*	-100	-47	45
Model III	CO1	0.9	3.2	0.8	0.3	-198	-95	74
	CO2	0.5	1.8	1.1	5.5*	-63	-26	23
	CO3	0.7	2.2	1.1	4.5*	-72	-29	28
	CO4	0.5	2.1	0.5	0.1*	-78	-38	27
	CO5	0.2	2.8	0.2	0.2*	-99	-48	39

* Local stresses under the wheel load

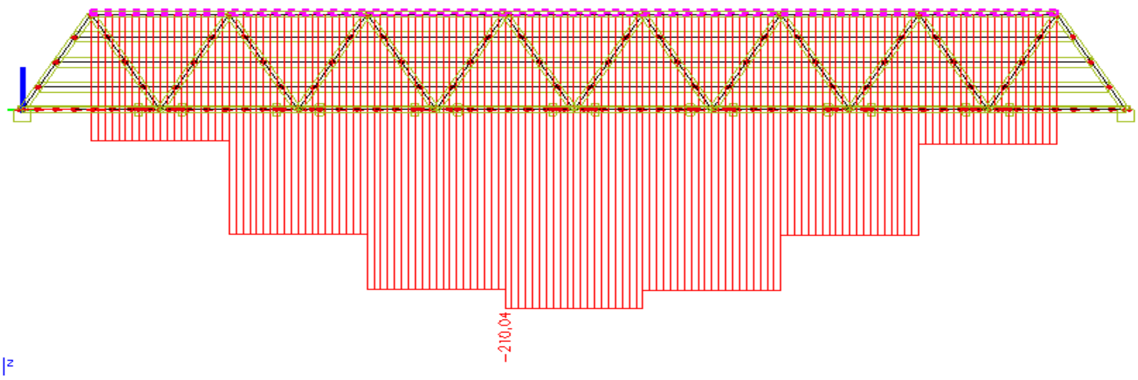


Figure 70: The compression forces in the top chord.

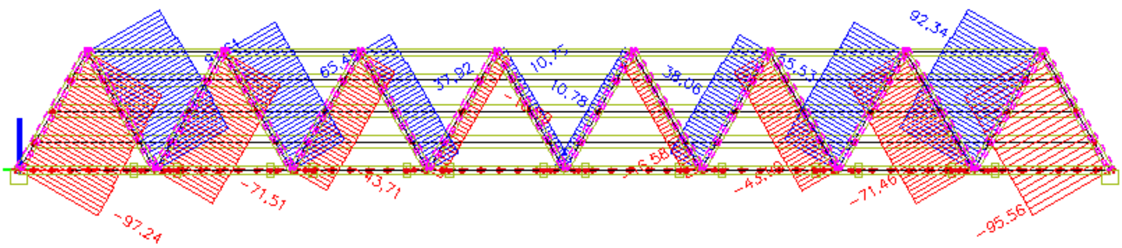


Figure 71: The normal forces in the diagonals.

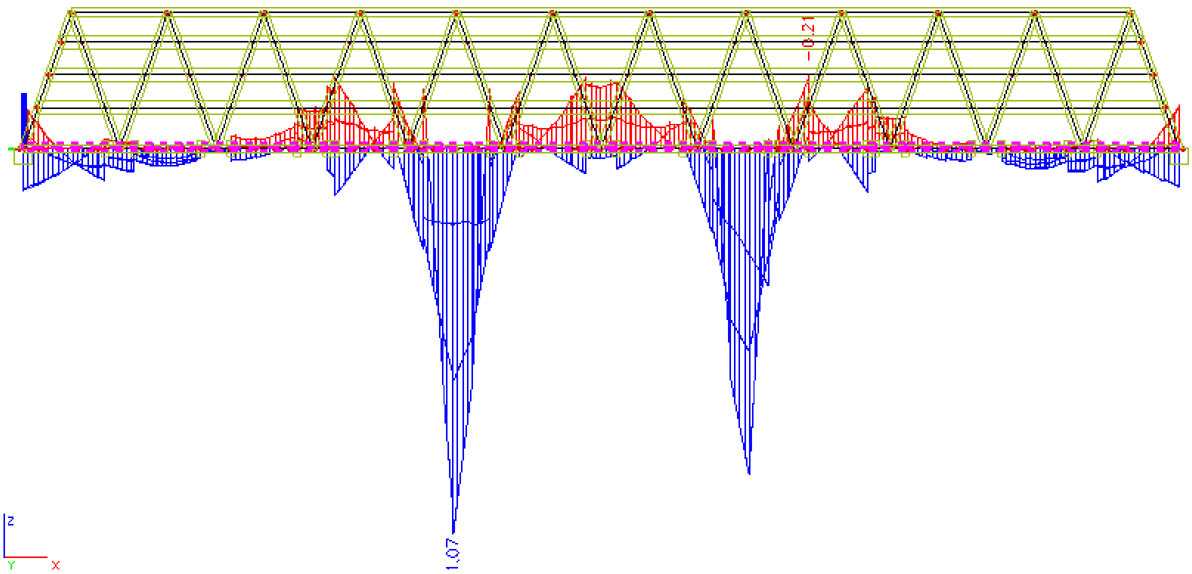


Figure 72: The moments in the longitudinal beams, load combination CO2 – vehicle in the middle.

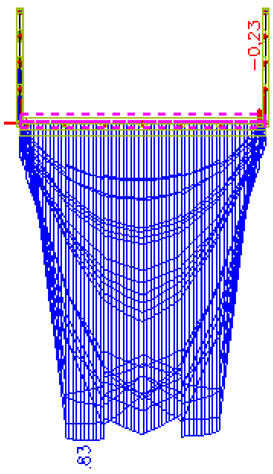


Figure 73: Moment distribution of the transversal beams under an axle load, load combination CO2 – vehicle in the middle.

4.5 3D truss

4.5.1 Description

The load bearing structure of a new design contains 3D trusses on sides of the bridge. There are two bottom chords for one truss. The transversal beams of the bridge are prolonged 0.5 m on each side. One bottom chord is placed at the end of the transversal beam and the other is placed in a distance of 0.42 m from the outer bottom chord. The length of the diagonal elements is kept 1.5 m and the number of diagonals is doubled. A half of the diagonals is connected to one bottom chord and other to the second bottom chord. Four diagonals meet in one joint at top chord and two diagonals meet at bottom chord. The SCIA model of bridge with 3D trusses is presented in Figure 74 and a skeleton of model is shown in Figure 75.

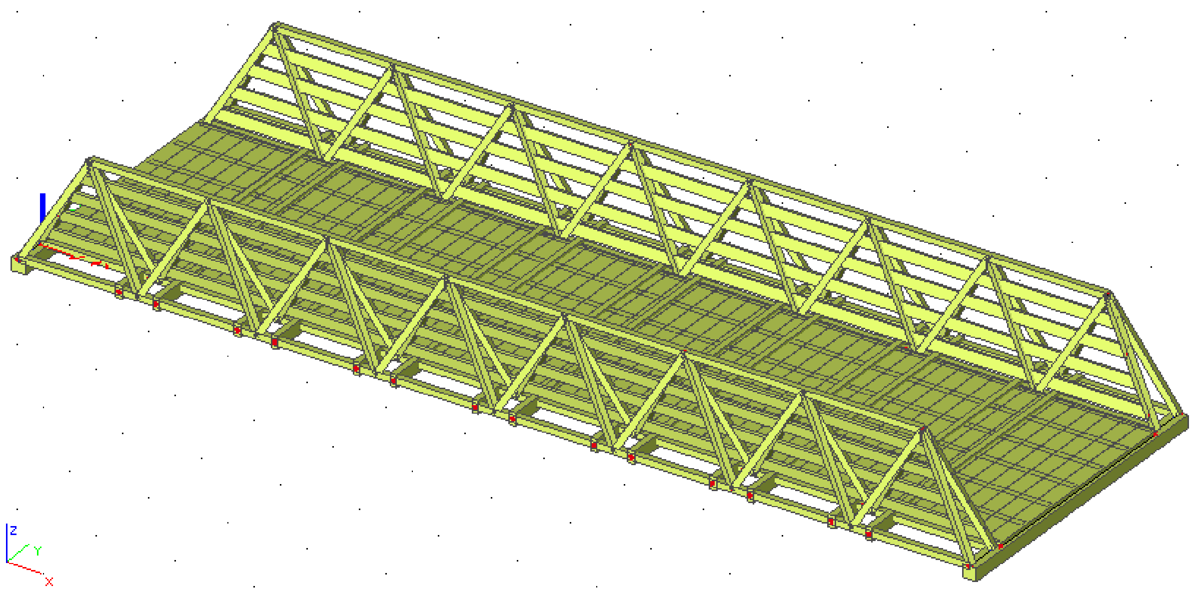


Figure 74: SCIA model of a bridge with 3D truss.

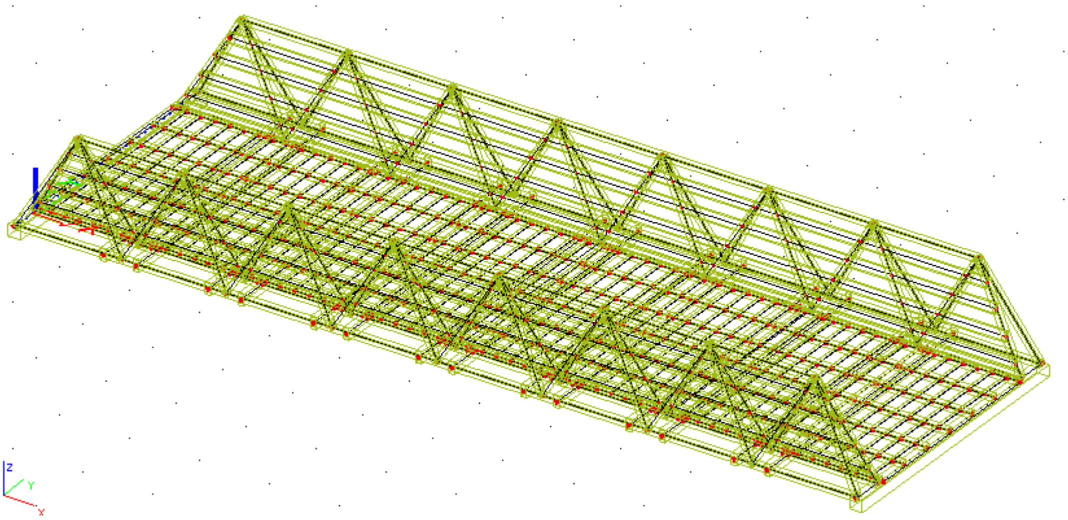


Figure 75: The skeleton of 3D truss bridge modelled in SCIA.

4.5.2 Calculation

The bridge with 3D trusses was calculated for SLS and ULS with same load combination as used for calculation of the simple truss (Section 4.3).

The vertical and horizontal deflections are summarized in Table 10. The requirements for deflection of the truss and deck are met and horizontal deflection of top chord is improved (from 54.8 mm horizontal deflection to 12.9 mm, see also Table 7).

Table 10: Serviceability Limit State of 3D truss - deflections

SLS Service Limite State		Maximal deflection u_z [mm]		Maximal deflection u_y [mm]	
		Bottom chord	Deck	Top chord	Bottom chord
3D truss	CO1	13	5	13	4
	CO2	3	3	4	1
	CO3	3	3	3	1
	CO4	4	3	7	2
	CO5	6*	2	13	5

*outer bottom chord

In the Table 11 the results from ULS calculations are presented. The tensile normal forces calculated in elements can be carried by the all designed cross-sections (Table 11 and Table 12). The buckling resistance of top chord has to be checked for the compression force of 207.95 kN. The buckling resistance calculation is provided in the Section 4.6.

Also stresses introduce by moments have to be checked again. The check calculation is summarized in Table 12.

There are presented various deflections of the 3D truss bridge in Figure 76, Figure 77, Figure 78, Figure 79, Figure 80, Figure 81 and Figure 82 (these figures are only illustrative, the values may not correspond with values in the Table 10).

Table 11: Ultimate Limit State of 3D truss – moments and normal forces

ULS Ultimate Limit State		Maximal Moment [kNm]				Maximal Normal forces in truss [kN]		
		Bottom chord	Transversal beam	Longitudinal beam	Deck planks [kNm/m]	Top chord	Diagonal	Bottom chord
3D truss	CO1	2.2	4.8	0.9	0.3	-207	-84	63
	CO2	1.0	2.2	0.8	5.7*	-69	-33	25
	CO3	1.5	2.6	0.8	4.5*	-78	-33	28
	CO4	0.9	2.8	0.5	0.2*	-86	-54	31
	CO5	1.0	2.5	0.1	0.2*	-96	-60	50

* Local moments under the wheel load

Table 12: The bridge elements properties, maximal normal forces and stresses introduced by moments in 3D truss

Element	Tensile and compression strength [MPa]	Cross-section [mm^2]	Max. Element normal force [kN]	Section modulus [mm^3]	Stress caused by moment [MPa]
Truss	64.5	80 x 80	413	85333	25.5
Longitudinal beam	64.5	80 x 80	413	85333	10.2
Transversal beam	49.0	160 x 80	617	341333	14.1
End beam	30.0	2 x 80 x 180	864	768000	-
Deck plank	33.3	47 x 200	313	73633	15.4

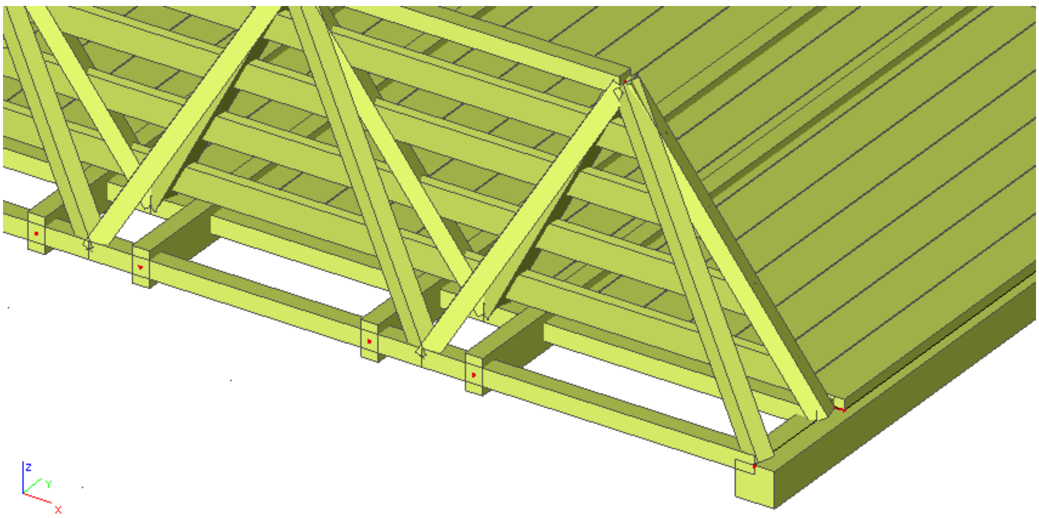


Figure 76: Detail of the 3D truss.

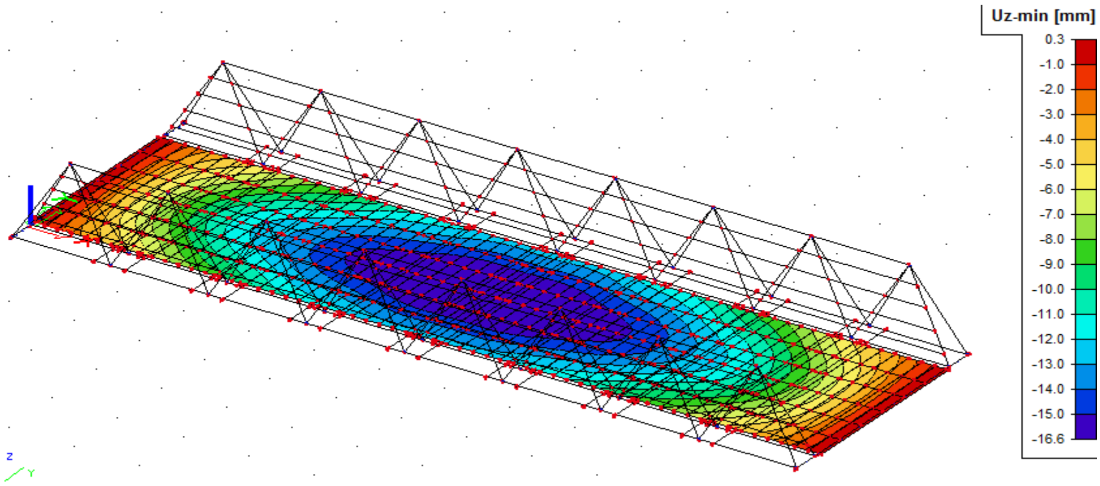


Figure 77: Vertical deflection of a 3D truss bridge load combination CO1 – uniformly distributed load.

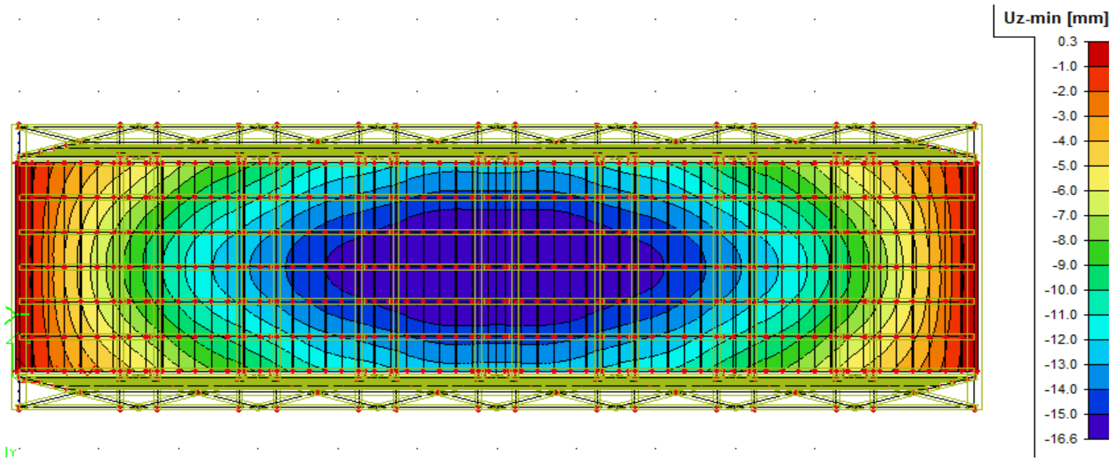


Figure 78: Vertical deflection of a 3D truss bridge load combination CO1 – uniformly distributed load.

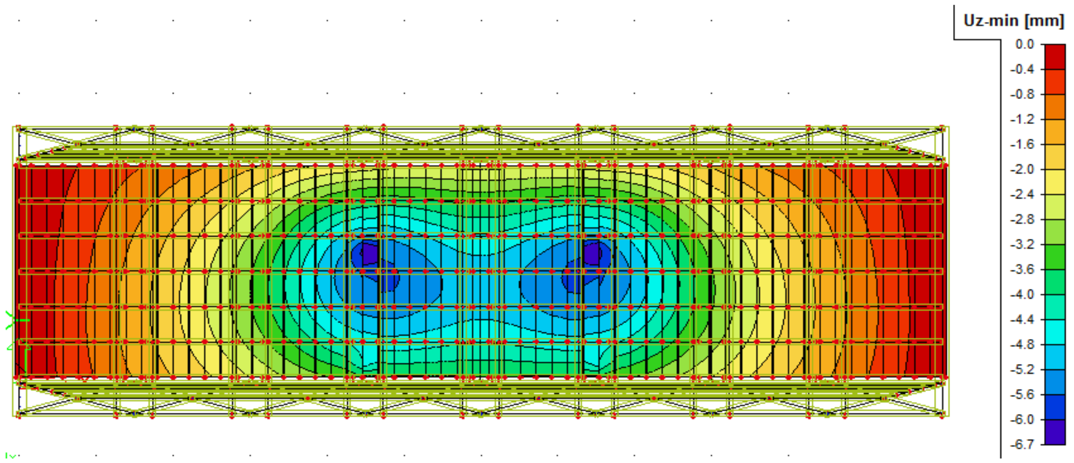


Figure 79: Vertical deflection of a 3D truss bridge load combination CO3 – vehicle on side.

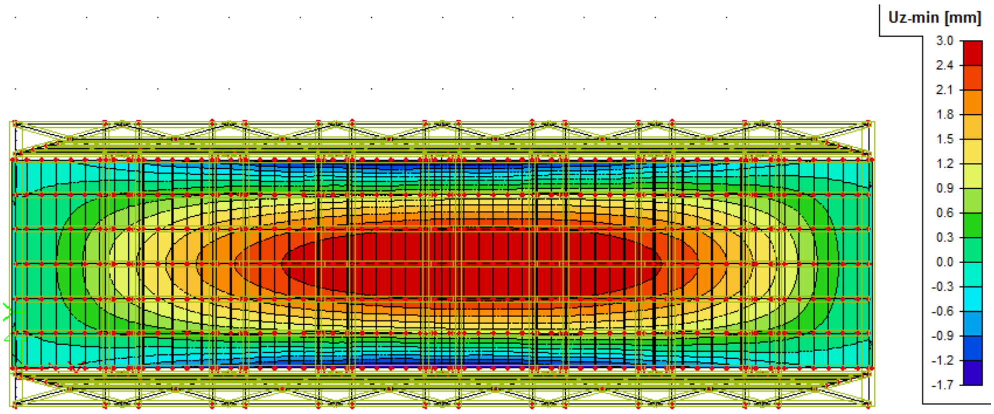


Figure 80: Vertical deflection of a 3D truss bridge load combination CO5 – load on railing.

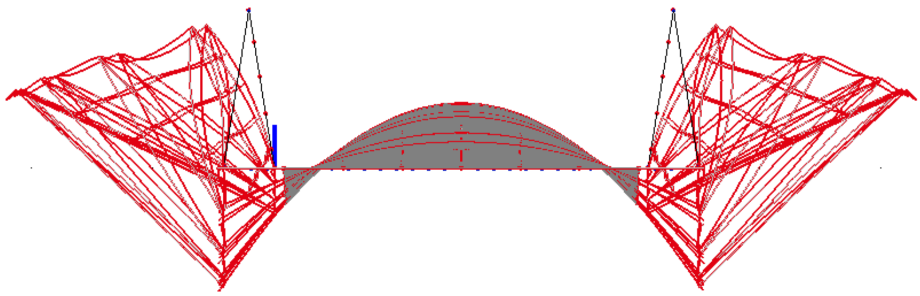


Figure 81: Magnified deformed 3D truss bridge CO5 – load on railings.

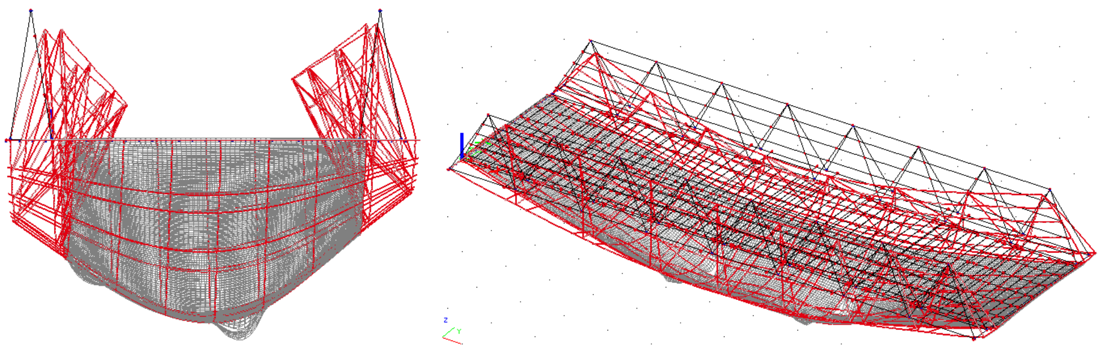


Figure 82: Magnified deformed 3D truss bridge cross-section load combination CO3 – vehicle on side.

4.6 Buckling

Truss elements are subjected to compression or tension. The compression elements have to be checked for sufficient buckling resistance. The buckling properties of the KLP material with steel reinforcement are not known yet, therefore, the buckling resistance calculation is based on an assumption that the buckling resistance is provided only by the steel reinforcement and that the KLP material almost does not have any contribution to the resistance.

4.6.1 Buckling resistance of cross-section 80 x 80 mm

The buckling resistance of a member is calculated according NEN-EN 1993-1-1 Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings (16), Section 6.3 Buckling resistance of members.

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

Where N_{Ed} is the design value of the compression force and $N_{b,Rd}$ is the design buckling resistance of the compression member.

The maximal compression force (207.95 kN, see Figure 70 and Table 11) calculated in FEM program is in the middle top chord element of the truss.

The design buckling resistance of the compression member is defined:

$$N_{b,Rd} = \frac{\chi A f_y}{\Upsilon_{M1}}$$

where $\boldsymbol{\chi}$ is the reduction factor for the relevant buckling mode; f_y is the yield strength and Υ_{M1} is partial factor for resistance of members to instability assessed by member checks. It has been assumed that only steel reinforcement is active in buckling resistance. Partial factor of steel reinforcement is specified in National Annex to NEN-EN 1992-1-1 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings, Table 2.1N (17):

$$\Upsilon_{M1} = 1.15$$

The steel type S435 is used in truss elements, therefore:

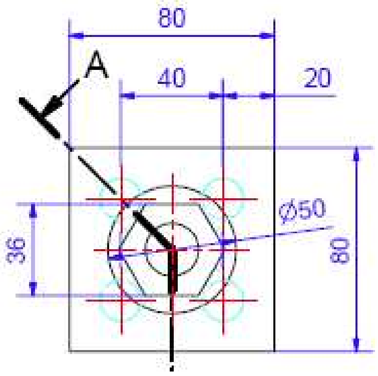
$$f_y = 435 \text{ MPa}$$

One steel reinforcement bar can be considered as solid section. There are four steel bars of ø16 in the truss element with spacing of 40 mm (Figure 83). Based on the presence of the KLP material, the assumption that all four steel bars behave as one solid section, is made. However, the cross-sectional area and moment of inertia are considered only for steel reinforcement bars.

$$A = 4\pi r^2 = 804 \text{ mm}^2$$

$$I = 4\left(\frac{\pi d^4}{64} + \pi\left(\frac{d}{2}\right)^2 a^2\right) = 4\left(\frac{\pi \times 16^4}{64} + \pi\left(\frac{16}{2}\right)^2 \times 20^2\right) = 334567\ \mathrm{mm}^4$$

where **a** is a distance of the steel reinforcement bar centre to the neutral axis of the cross-section.



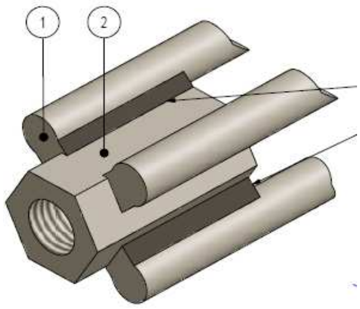


Figure 83: The steel reinforcement dimensions

The reduction factor **χ** can be defined according a Figure 6.4 Buckling curves, NEN-EN 1993-1-1 Eurocode 3 (Figure 84).

The buckling curve is determined from Table 6.2: Selection of buckling curve for a cross-section, NEN-EN 1993-1-1 Eurocode 3 (16). The buckling curve **c** should be used for solid sections.

The non-dimensional slenderness $\bar{\lambda}$ is calculated with:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{804 \times 435}{308191}} = 1.07$$

where N_{cr} is the elastic critical force for the relevant buckling mode. The elastic critical force is calculated:

$$N_{cr} = \frac{\pi^2 EI}{(Kl)^2} = \frac{\pi^2 \times 210 \times 10^3 \times 334567}{(1 \times 1500)^2} = 308191\ N$$

where *l* is the length of the truss element and K is for coefficient for the buckling mode. The compression truss member is considered as pinned element, therefore K=1.

Then the reduction factor is (Figure 84):

$$\chi = 0.54$$

and the design buckling resistance of the compression member is:

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} = \frac{0.54 \times 804 \times 435}{1.15} = 164276\ \mathrm{N}$$

The compression member verification against buckling is as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{207950}{164276} = 1.27 \nleq 1.0$$

The requirement for the buckling resistance of 80 x 80 mm cross-section is not met.

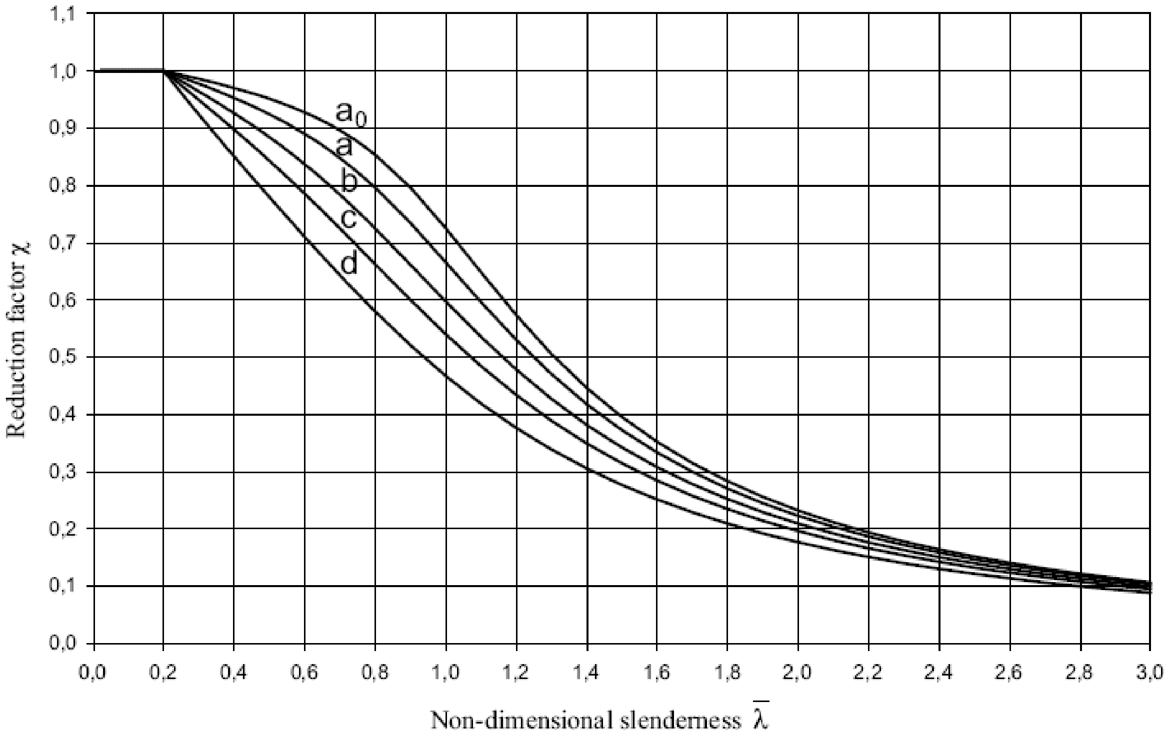


Figure 84: Buckling curves (16).

4.6.2 New position of steel reinforcement in the cross-section 80 x 80 mm

It is possible to produce KLP steel beams with smaller plastic cover around steel reinforcement than is used in the cross-section 80 x 80 mm. Common cover for KLP beams with steel reinforcement is 8 mm. In considered cross-section the cover is 12 mm, because the steel bars are welded to the nut, which then defines the position of the steel reinforcement. If the steel reinforcement bars for the considered cross-section of 80 x 80 mm would be placed only 4 mm closer to the surface of the element, then the moment of inertia and the critical force increases, which results in decrease of non-dimensional slenderness and increase of reduction factor from 0.54 to 0.61 and the compression member verification against buckling is:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{207950 \times 1.15}{0.61 \times 804 \times 435} = 1.12 \nleq 1.0$$

$$I = 4\left(\frac{\pi \times 16^4}{64} + \pi\left(\frac{16}{2}\right)^2 \times 24^2\right) = 476115\ \mathrm{mm}^4;\ N_{cr} = \frac{\pi^2 \times 210 \times 10^3 \times 476115}{(1 \times 1500)^2} = \ 438580\ N$$

$$\bar{\lambda} = \sqrt{\frac{804 \times 435}{438580}} = 0.89$$

The verification requirement against buckling still is not met, but the KLP material contribution was not used for the buckling resistance. The presented compression force 207.95 kN is from the load combination CO1, which is a load combination of uniformly distributed load = fully crowded bridge with people standing even on the railing. It is very conservative combination. It could be safe to assume that the 10% of the buckling resistance could be carried by combination of KLP material with steel reinforcement.

4.6.3 Correct position of steel reifnrocement bars – conservative approach

If the requirements of the buckling resistance should be met, when considered only the contribution from steel reinforcement, the dimensions of the cross-section would have to be increased only to 90 x 90 mm with cover of the steel reinforcement taken as 8 mm. Total area of steel reinforcement stays same but the distance from the neutral axis is larger and therefore, the moment of inertia is larger:

$$\mathrm{A} = 4\pi \mathrm{r}^2 = 804 \mathrm{~mm}^2$$

$$\mathrm{I} = 4\left(\frac{\pi \mathrm{d}^4}{64} + \pi\left(\frac{\mathrm{d}}{2}\right)^2 \mathrm{a}^2\right) = 4\left(\frac{\pi \times 16^4}{64} + \pi\left(\frac{16}{2}\right)^2 \times 29^2\right) = 689240 \mathrm{~mm}^4$$

The elastic critical force N_{cr} for the relevant buckling mode also becomes larger, non-dimensional slenderness decreases and reduction factor increases:

$$N_{cr} = \frac{\pi^2 \times 210 \times 10^3 \times 689240}{(0.7 \times 1500)^2} = 634903\ N$$

$$\bar{\lambda} = \sqrt{\frac{804 \times 435}{634903}} = 0.74$$

$$\chi = 0.7$$

and the requirement for the buckling resistance are met:

$$\frac{\mathrm{N_{Ed}}}{\mathrm{N_{b,Rd}}} = \frac{207950 \times 1.15}{0.7 \times 804 \times 435} = 0.98 < 1.0$$

In the cross-section 80 x 80 mm the reinforcement bars are welded directly to the nut. This defines the position of the steel reinforcement and distance of the steel bars from the neutral axis. When the reinforcement is necessary to be placed farther from the neutral axis, new way of connection of steel bars to the nut has to be created.

4.6.4 Buckling resistance of diagonals

The idea was to use the same length of KLP-S beams for all elements of the truss, but during the design process this idea has been changed. The top chord is continuous beam to which diagonals of 1.5 m length are connected with pin joint. The diagonals can be connected in any distance. Therefore, the top beam can be constructed of 3 m long beams (which is a optimal length for production) and based on the buckling resistance calculation new cross-section is recommended and new mould to produce this beam has to be developed. For the diagonals new mould has to be produced anyway, because the length of 1.5 m is not standard for the Royal Lankhorst Euronete production. The maximal compression forces in top chord and diagonals are different. Compression force in top chord is two times higher than compression force in diagonal. Therefore, there is no reason to keep the dimensions of diagonals same as dimension of top chord and new cross-section for diagonals can be designed.

The maximal force in diagonals calculated in FEM model is 84 kN (Table 11). Considering the buckling resistance of the cross-section of 80 x 80 mm and 12 mm cover on the steel reinforcement (Section 4.6.1) under the compression force of 84 kN, the verification of buckling resistance is:

$$\frac{\mathrm{N_{Ed}}}{\mathrm{N_{b,Rd}}} = \frac{84000 \times 1.15}{0.54 \times 804 \times 435} = 0.51 < 1.0$$

The buckling resistance of diagonals under the load of 84 kN is very high and the cross-section could be optimized.

The easiest to change in the cross-section is the diameter of the steel reinforcement. In the KLP-S beams production a diameter of 12 mm is normally used. The buckling resistance of the same cross-section but with reinforcement bars of 12 mm diameter is:

$$\mathrm{A} = 4\pi \mathrm{r}^2 = 452\ \mathrm{mm}^2$$

$$\mathrm{I} = 4\left(\frac{\pi \mathrm{d}^4}{64} + \pi\left(\frac{\mathrm{d}}{2}\right)^2 \mathrm{a}^2\right) = 4\left(\frac{\pi \times 12^4}{64} + \pi\left(\frac{12}{2}\right)^2 \times 22^2\right) = 223028\ \mathrm{mm}^4$$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{452 \times 435}{205445}} = 0.98$$

$$N_{cr} = \frac{\pi^2 EI}{(Kl)^2} = \frac{\pi^2 \times 210 \times 10^3 \times 223028}{(1 \times 1500)^2} = 205445\ N$$

$$\chi = 0.55$$

$$\frac{\mathrm{N_{Ed}}}{\mathrm{N_{b,Rd}}} = \frac{84000 \times 1.15}{0.55 \times 452 \times 435} = 0.89 < 1.0$$

The cross-section with 12 mm diameter of steel reinforcement is more economic design.

4.6.5 Conclusion

There is no available information on buckling behavior of the KLP-S beams. The assumption that only steel reinforcement is contributing to the buckling resistance is very conservative. In order to design economical and efficient pedestrian bridge the further research on buckling behavior of KLP-S beams is recommended.

4.7 Pin joint

The pedestrian bridge was calculated with an assumption of pin joint between top/bottom chord and diagonals. The pin joint could be easy for the assembly and also could allow the more variability of the bridge span length.

The first idea how the pin joint could be arranged is that a steel strip would go around the ¾ of the chord perimeter. Ends of the steel strip are connected with bolt which is also connecting the diagonals. The steel reinforcement bars in the diagonal beam element is finished with nut through which the diagonal are connected with eye and fork bolts. The further description of the joint is written in the following sections.

There are three different types of joint necessary for the construction of the 3D truss (Figure 85). Joint 1 is the joint on the top chord, where four diagonals meet. Joint 2 is a joint on the bottom chord, where two diagonals meet, and Joint 3 is end joint, where only one diagonal is connected to the bottom chord.

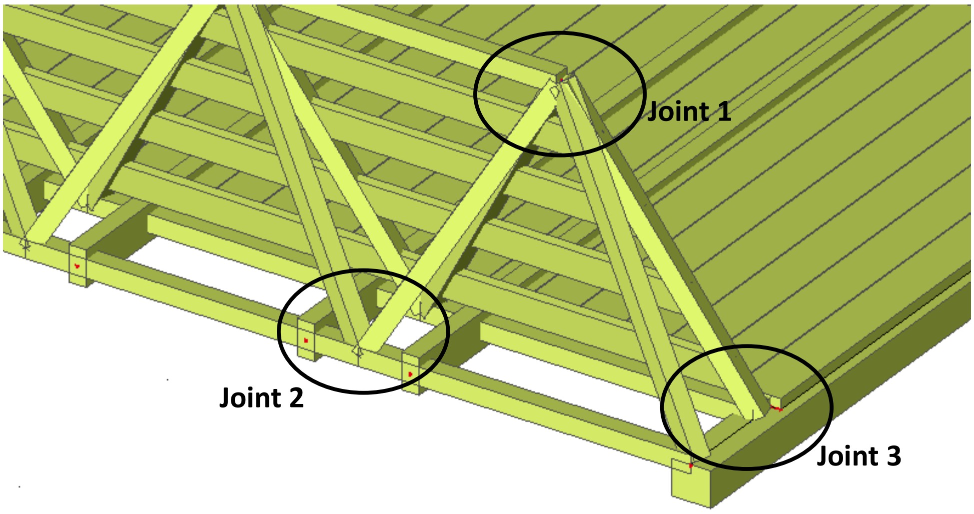


Figure 85: Three types of joint in the 3D truss

The largest forces in the truss diagonals are at the ends of the 3D truss as is shown in Figure 86. The forces are acting into the joint under angles and for the design of the joint the diagonal forces are decomposed to vertical and horizontal parts. The forces on the joint are not symmetrical. The inner diagonals are subjected to larger compression or tension forces.

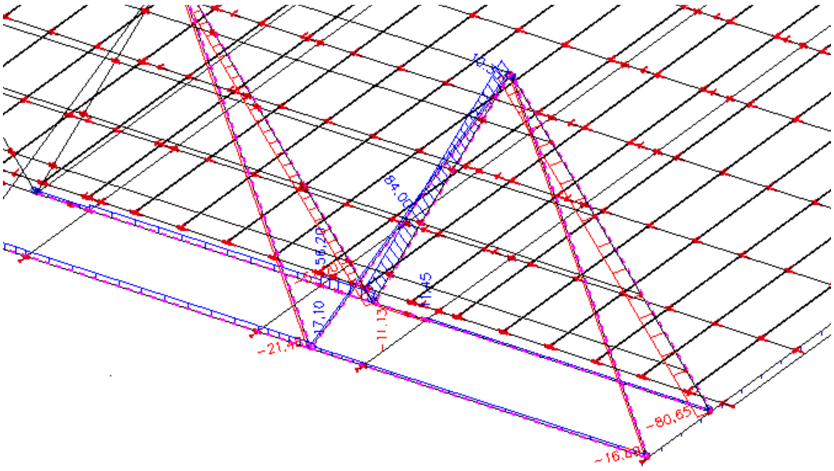


Figure 86: Compression and tension forces in the diagonals at the end of the bridge under the load combination CO1 - crowed Bottom joints CO1.

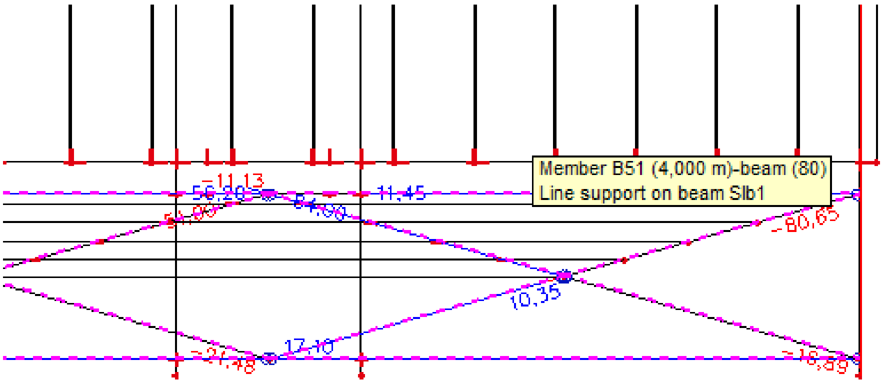


Figure 87: Top view on the compression and tension forces in the diagonals at the end of the bridge under the load combination CO1 - crowed

First, the calculation is concentrated on design of the steel strip. The vertical forces in the joint are presented for each joint in a cross-sectional sketch and the horizontal forces are presented in a sketch for each joint as a bottom view of the joint (Figure 88, Figure 89, Figure 90). To calculate forces in the steel strip in the vertical and horizontal composition of the joint the bolt is considered as a simply supported beam, where the steel strips are supports. Then, forces in the steel strip are calculated with force and moment equilibriums. The forces and calculated reactions in the steel strips are summarized in Table 13.

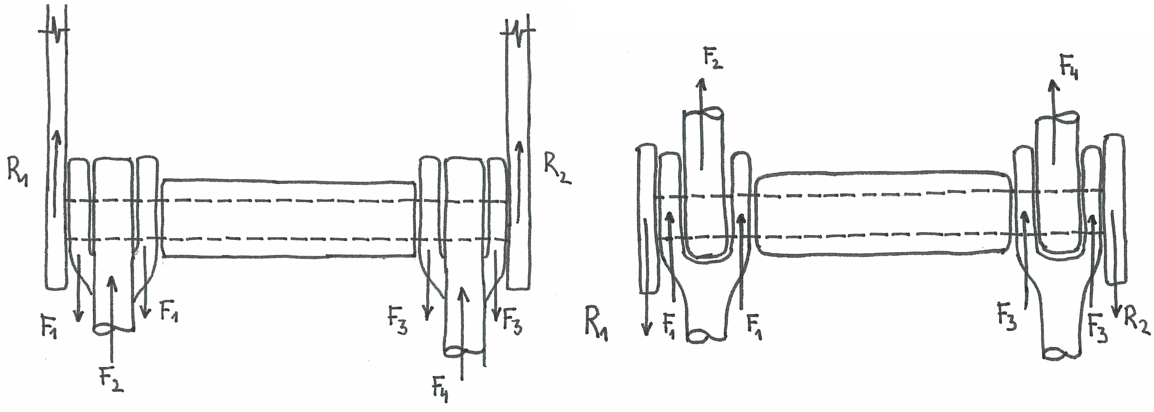


Figure 88: Sketch of Joint 1 – vertical forces (left) and horizontal forces (right)

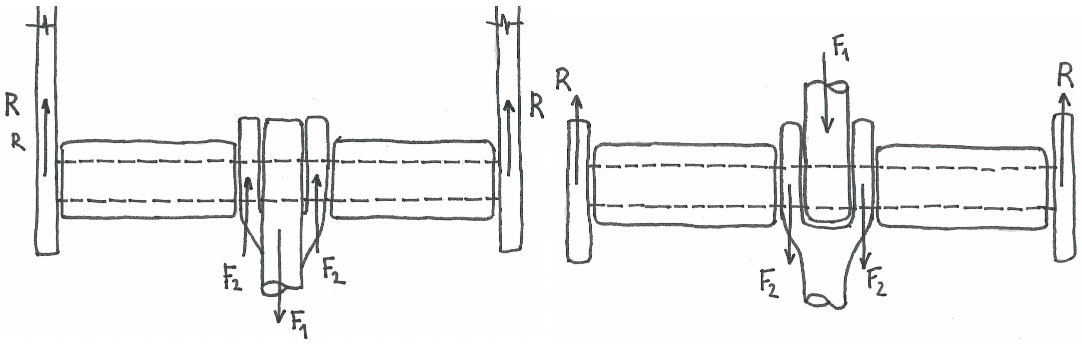


Figure 89: Sketch of Joint 2 – vertical forces (left) and horizontal forces (right)

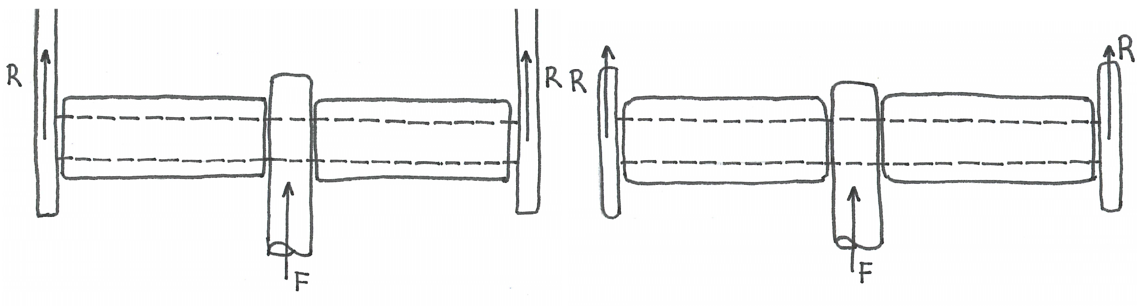
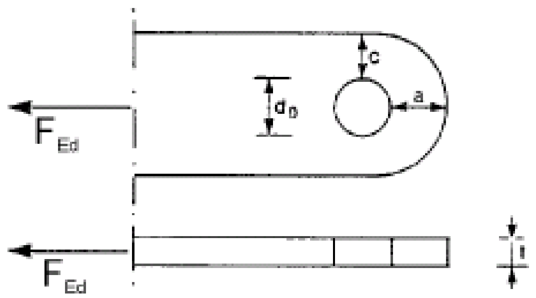


Figure 90: Sketch of Joint 3 – vertical forces (left) and horizontal forces (right)

Table 13: Vertical and horizontal forces in the joints

Vertical			Horizontal		
Joint 1	F1	37 kN	Joint 1	F1	19 kN
	F2	72 kN		F2	37 kN
	F3	7.5 kN		F3	4 kN
	F4	15 kN		F4	8 kN
	R1	1.7 kN		R1	65 kN
	R2	0.35 kN		R2	25 kN
Joint 2	F1	73.5 kN	Joint 2	F1	39.5 kN
	F2	23.5 kN		F2	12.5 kN
	R	13.25 kN		R	32.25 kN
Joint 3	F	71 kN	Joint 3	F	38 kN
	R	35.5 kN		R	19 kN

The largest forces on the steel strip are in the horizontal direction of the Joint 1. The size of the steel strip is calculated according the Eurocode 3: Design of steel structures – Part 1 – 8: Design of joints, Table 3.9 Type A(18) which is shown in Figure 91. For the force of 65 kN and the hole for the bolt of for example 18 mm, steel S335 and thickness 5 mm, the steel strip would have to be about 80 mm wide.



$$a \geq \frac{F_{Ed}\gamma_{M0}}{2\,t\,f_y} + \frac{2\,d_0}{3} \;:\; c \geq \frac{F_{Ed}\gamma_{M0}}{2\,t\,f_y} + \frac{d_0}{3}$$

Figure 91: Geometrical requirements for pin ended members according Eurocode 3: Design of steel structures – Part 1 – 8: Design of steel joints, Table 3.9, Type A (18)

The size of the steel strip is not very big problem but problem are the horizontal forces. The total horizontal forces in the Joint 1 are 90 kN. There are almost no friction forces between steel and KLP and therefore the steel strip would slide away. It is necessary to somehow resist these large horizontal forces.

An option could be to create a beam with two different cross-sectional areas, where the steel strip would be placed between to larger cross-section and kept that way in its position. Sketch of the beam with different cross-sectional areas is shown in Figure 92. This beam would not work either because the shear strength of the KLP is about 6 MPa and the horizontal forces in the steel strip creates stress of about 54 MPa on the larger cross-section. As a result the steel strip would cut the large cross-section as a knife slice through butter.

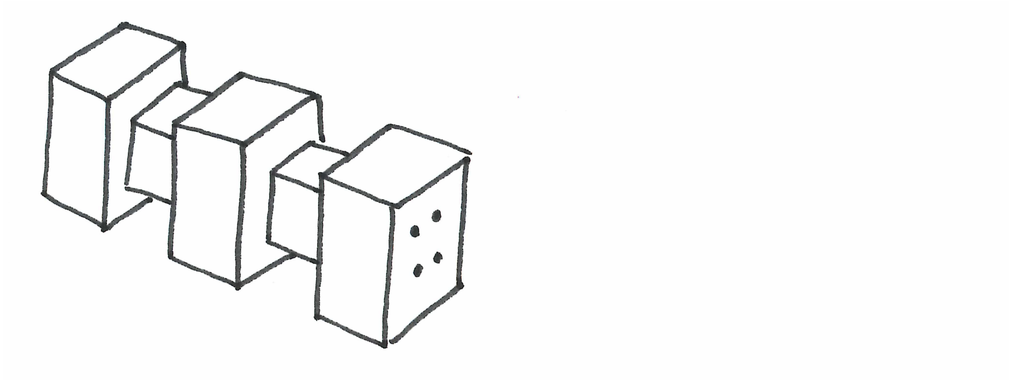


Figure 92: Sketch of the beam with different cross-sectional area

It would be convenient if the large horizontal forces could be somehow transferred directly onto the steel reinforcement. After the discussion with the specialists from Engineered Products department it was suggested, that it is possible to weld steel tubes onto the reinforcement. These tubes would be fully submerged into KLP as steel reinforcement is. It is possible to locate, where the tubes are, and drill through them. Then the steel U-shape connection could be put through the beam. This U-shape would be in the steel tubes through which the horizontal forces would be transferred directly to the steel reinforcement. U-shape could be finished with thread at each end and the eye hook with nut could be screwed onto U-shape one it is pulled through the beam. Through the eyes of U-shape the bolt and eye bolt and fork bolts of diagonals could be connected. An example assembly of such a joint is presented in Figure 93, Figure 94, Figure 95 and Figure 96. The design of such a joint would require detailed elaboration, which is not within the time capacities of this master thesis (see also chapter 6 Further research).

Figure 93: An example of the joint where the forces would be directly transferred into the steel reinforcement. Steel tubes are fully submerged into recycled plastic. The position of tube is possible to record and later drill through them. Then the U-shape profile can be put through and eye bolts can be connected to it.

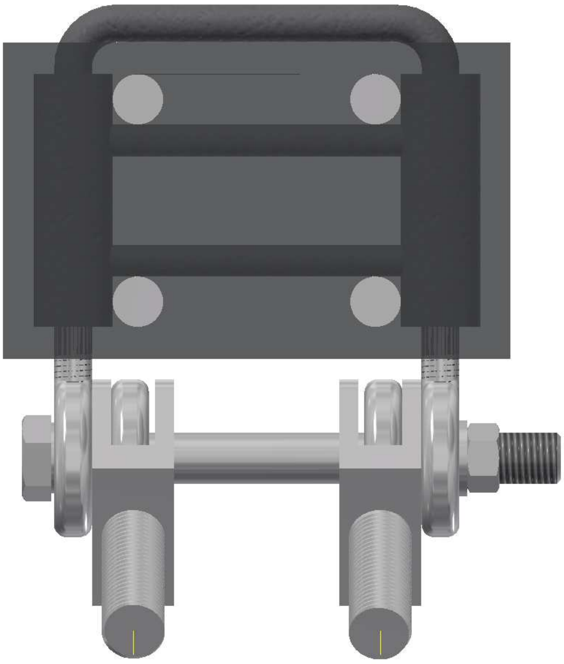


Figure 94: Cross-sectional view

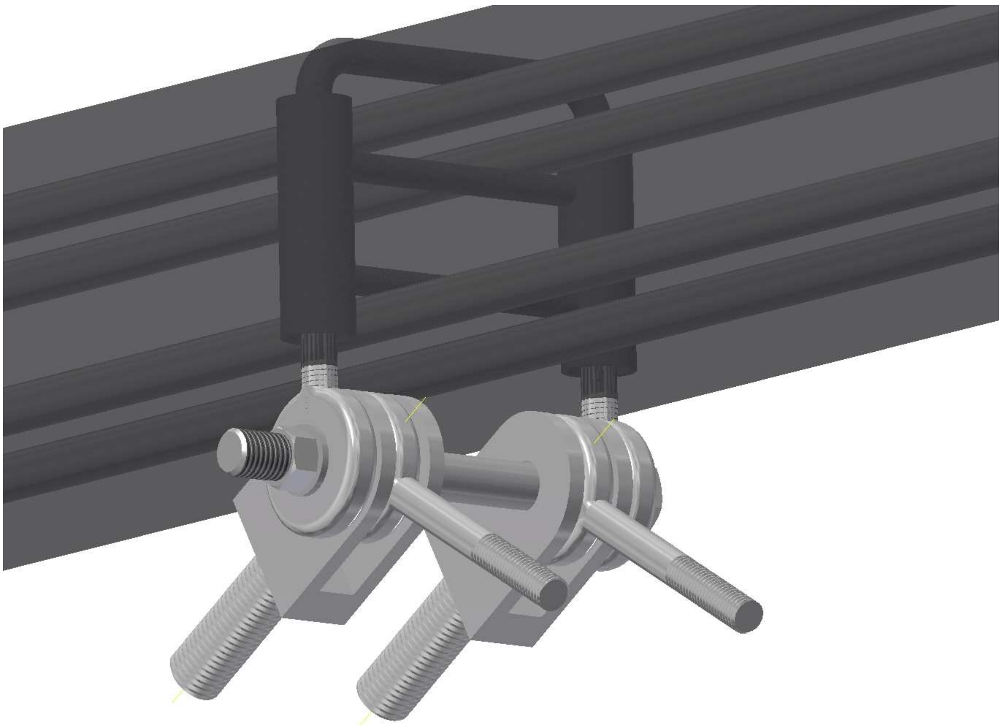


Figure 95: Diagonals of the truss are connected through eye and fork bolt

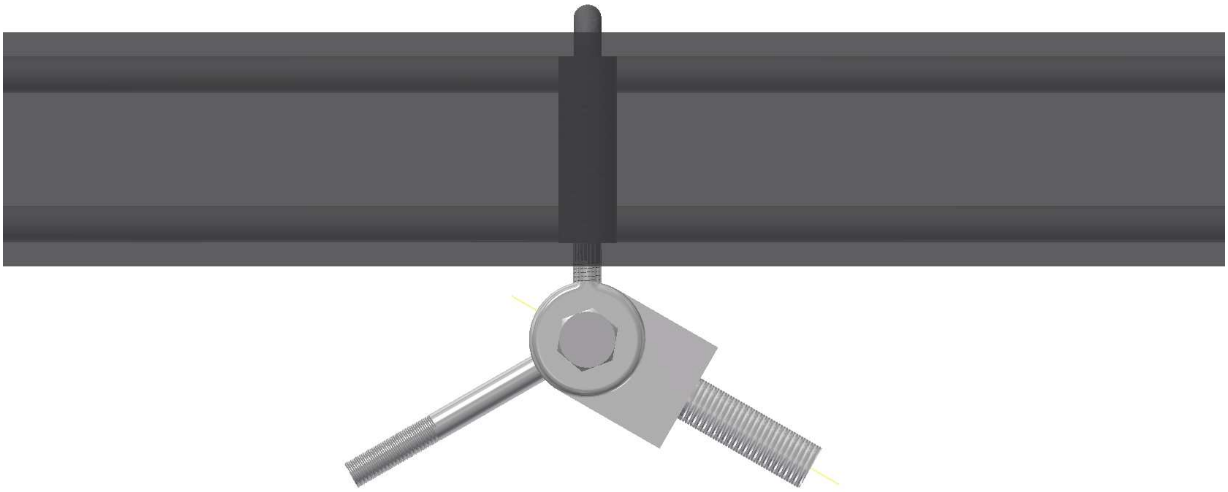


Figure 96: Side view of the joint

5 Conclusions

The structural design of the pedestrian bridge with the span of 12 meters was achieved. The structural design was calculated according Eurocode and available information about structural properties of the recycled plastic material with steel reinforcement.

The detailed design and calculation of joints would show, if it is possible to build the bridge or if it is necessary to decrease the span or width of the bridge.

Also further research on several subjects would help to make design more economical and efficient if desired. The suggestions for further research are presented in following chapter 6.

6 Further research

6.1 KLP together with steel reinforcement

KLP material with steel reinforcement is very interesting combination. It is not known yet, how these to materials behave together and how each of the materials contributes to the structural properties. Several bending tests on specific sizes of the beams were done, but there are no general rules how to define structural properties of any cross-section. It would be interesting to try to find out, if some of the already existing design rules for steel, timber or concrete could comply for KLP with steel reinforcement or it these rules could be combined.

6.2 Buckling

Another interesting research topic is a buckling resistance of the KLP with steel reinforcement. Most of the products made of KLP-S are beams and in general these elements are very slender, therefore the buckling resistance capacity can be decisive for the whole structure, where the KLP-S beams are used. KLP itself is rather weak material and steel reinforcement improves the structural properties of the KLP-S beams. With regard to the buckling resistance the KLP material could have a large contribution to the buckling resistance. Compression test on KLP-S beams and columns could be done to obtain buckling curves, which provide reduction factors for the buckling resistance. With the proper information about buckling behavior of the KLP-S beams more efficient structures could be designed.

6.3 Connections

In the construction practice with KLP the methodology for timber structures is used. It would be interesting to test several connections of KLP based on the Timber structures design rules and see if the KLP behaves in comparable manner or if it totally differs. Also it would be interesting to define some design rules according the connections could be and should be done.

Another type of the connections is the connection in KLP-S products. Mostly these connections are done through the connection of steel reinforcement, but the behavior of the elements with steel reinforcement connected through KLP material should differ from the connection of elements made of only KLP.

6.4 Variation of the truss bridge

After the detailed design and calculation of the joints for 3D truss bridge, it could be further investigated how variable is this design. For example, it would be interesting to see if it is possible to make longer span of the bridge, when the costumer would require smaller width of the bridge. Also it would be interesting to try to make calculation in excel or some other program for the variation of the width and span of the 3D truss bridge.

7 Resources

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