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Departamento de Engenharia Civil

Anna Dominika Kur

Projeto estrutural de uma ponte pedonal metálica

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Dissertação apresentada à Universidade de Aveiro para cumprimento dos requisitos necessários à obtenção do grau de Mestre em Engenharia Civil, realizada sob a orientação científica do Doutor Nuno Filipe Ferreira Soares Borges Lopes, Professor Auxiliar do Departamento de Engenharia Civil e Doutor Paulo Jorge de Melo Matias Faria de Vila Real, Professor catedrático do Departamento de Engenharia Civil.

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Departamento de Engenharia Civil

Anna Dominika Kur

Structural Design of a Steel Footbridge

Thesis is submitted to the University of Aveiro to fulfill the necessary requirements for the degree of Master of Civil Engineering, made under the scientific supervision of Dr. Nuno Filipe Ferreira Soares Borges Lopes, Assistant Professor of the Department of Civil Engineering and Dr. Paulo Jorge de Melo Matias Faria de Vila Real, Professor of the Department of Civil Engineering.

I dedicated this work to my parents, my sister and my brother.

Jury

President

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Thanks

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I have been extremely lucky to have a supervisor who cared so much about my work, and who responded to my questions so promptly.

I would also like to thank also to Dr. Paulo Jorge de Melo Matias Faria de Vila Real, Professor of the Department of Civil Engineering for valuable time, all advice and supporting.

Thanks.

Keywords: Steel construction, footbridge, design

Abstract:

This thesis presents the modeling and structural designing of a steel footbridge located in Poland. All work is based on the European Norms (Eurocodes, especially EN 1993 - Eurocode 3: Design of steel structures).

This work includes the theoretical part, contains the definition of basic concepts, shows the types of pedestrian bridges and presents some of the most interesting examples of existing footbridges.

It is presented the modeling and collection of loads acting on a footbridge. Static calculations were performed with the help of Autodesk Robot Structural Analysis 2011, using the finite element method.

This design included the choice of cross-section of the dirernet structural elements and safety verification of the conections, among all others necessary design requirements.

Summarizing, this study shows the structural design of the footbridge located in Lodz, Poland.

palavras-chave

construção metálica, pontes pedonais, projeto

resumo

Esta tese apresenta a modelação e projeto estrutural de uma ponte pedonal de aço localizada na Polónia. Todo o trabalho é baseado nas normas europeias (Eurocódigos, especialmente a EN 1993 - Eurocódigo 3: Projeto de estruturas em aço).

Este trabalho inclui a parte teórica, contém a definição de conceitos básicos, mostra tipos de pontes pedonais existentes e apresenta alguns dos exemplos mais interessantes.

Apresenta-se a modelação e cálculo das cargas que atuam sobre uma ponte pedonal.
Foram realizados aplicando o programa Autodesk Robot Structural Analysis 2011 cálculos estáticos, usando o método de elementos finitos.

Este dimensionamento incluiu a escolha de secção transversal dos diferentes elementos estruturais e verificação da segurança das ligações, entre todos os outros requisitos necessários para o projeto.

Resumindo, este estudo mostra o projeto estrutural de uma ponte pedonal localizada em Lodz, na Polónia.

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1. Scope and introduction

1.1. General considerations

Steel structural elements provide virtually unlimited freedom to shape architectural form and allows the use of large precast thus shortening construction time. Leading to wide range of different structural solutions.

A footbridge [1] of this thesis is a bridge which is designed with pedestrian traffic. Some footbridges also accommodate other users such as horses and bicycles, while keeping motor vehicles such as cars off the bridge. This type of bridge is usually narrow and lightweight.

For safety, most footbridges are equipped with safety rails, which provide grips for people, who may feel unsteady on their feet, and they are also designed to prevent people from falling of the bridge. In cases where a footbridge passes over other forms of traffic, the bridge may be fully enclosed for additional safety, ensuring that things cannot be dropped from the bridge to hit people or vehicles below.

Enclosed footbridges are sometimes known as skyways, especially when they pass between buildings; the skyway design is used in some cold climates to allow people to move between buildings in complexes such as hospitals without having to go outdoors.

Footbridges are also used inside, and to provide access on the exterior of buildings, in which case they are known as catwalks.

These are many other considerations to account for in the design of footbridges, name of them will be detailed in this document.

The aesthetic impact of these constructions in the built environment is typically high, leading to a number of possible solutions that can vary in function of the final propose, landscape or other construction in the area..

1.2. Objectives

This document describes the theoretical background with the basic necessary concepts for the design of footbridges. Also presented some of the interesting, existing pedestrian bridges.

The model of this study was designed in the program Autodesk Robot Structural Analysis 2011, which consists of a truss girder a support element and skeleton of the elevator. All elements are designed as round tubes of steel S235. The computational model consists of groups of bars with different cross sections, and different lengths of elements. This division allows a more economical construction and lower consumption of steel.

For construction have influenced many types of loads, such as:

- steady load (weight of construction),
- service load (traffic load- people)
- climatic load (wind, snow, temperature).

Considering the effect of the above described loads, the bar elements of the construction and the connections (both a connection of foundation and the connections between the bars) in accordance with applicable European standards.

This thesis presents the complete design of a steel footbridge structure.

1.3. Case study

The case study of the thesis is a steel pedestrian bridge with a span of about 14m and a width of about 3m. Footbridge designed only for pedestrian and cycling traffic. It will allow for safe the passage from one side of the street to the other. The proposed facility is located in Poland, near Lodz.

1.4. Footbridge types

There are six basic types of steel footbridges [*2*]:

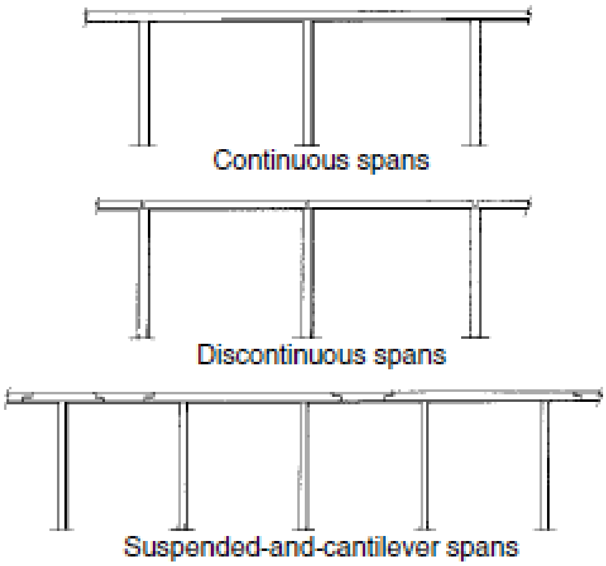
- Girder footbridges,
- Rigid frame footbridges
- Arches footbridges
- Cable-stayed footbridges
- Suspension footbridges
- Truss footbridges

1.4.1. Girder footbridges

A girder bridge is probably the most common and most basic bridge. This type of bridge, in general, is a bridge built of girders placed on bridge abutments and foundation piers. In turn, a bridge deck is built on top of the girders in order to carry traffic.

A girder bridge only requires a rigid horizontal structure, also known as a beam, and two supports at each end to rest upon it. These components are what allow the downward weight of the bridge and any traffic to travel across it.

The size and height of the beam are controlled by the distance that the beam can span. By increasing the height of the beam, the beam has more material and thus more easily dissipate any tension exerted on the foundation.

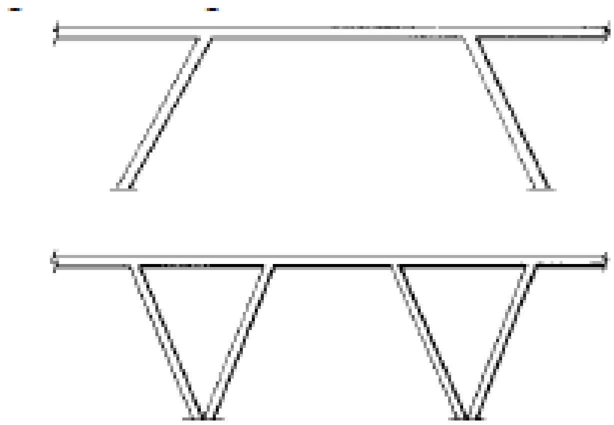


Picture 1. Examples of girder bridges (pictures from [2])

1.4.2. Rigid frame footbridges

Rigid frame (Rahmen) bridges is kind of bridge where the piers or towers are connected to the girders. Rigid-frame bridges are a bridge type made up of groups of rigidly connected members on which bending moment, axial force, and shear force work at the same time.

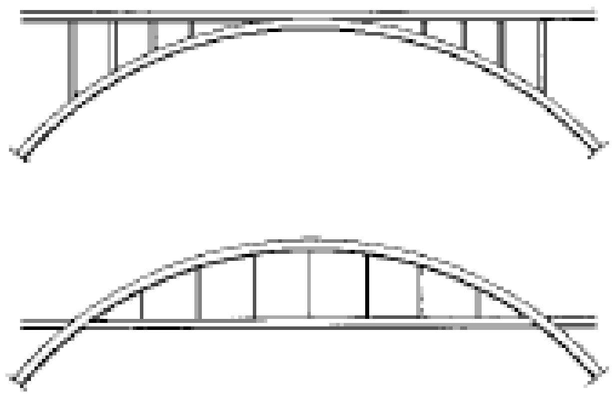
The styles used almost exclusively are the pi-shaped frame (are used frequently as the piers and supports for inner city highways. The frame supports a raised highway and at the same time allows traffic to run directly under the bridge), the batter post frame (well suited for river and valley crossings because piers tilted at an angle can straddle the crossing more effectively without requiring the construction of foundations in the middle of the river or piers in deep parts of a valley), and the V shaped frame (make effective use of foundations, provides two supports to the girder, reducing the number of foundations and creating a less cluttered profile).



Picture 2. Examples of rigid frame bridges (pictures from [2])

1.4.3. Arches footbridges

Arches are the second oldest bridge type and a classic structure (after girders) and are good choices for crossing valleys and rivers since the arch doesn't require piers in the center.
The basic principle of arch bridge is its curved design, which does not push load forces straight down, but instead they are conveyed along the curve of the arch to the supports on each end. These supports (called abutments) carry the load of entire bridge and are responsible for holding the arch in the precise position unmoving position. Conveying of forces across the arch is done via central keystone on the top of the arch. Its weight pushes the surrounding rocks down and outward, making entire structure very rigid and strong.



Picture 3. Examples of arches bridges (pictures from [2]).

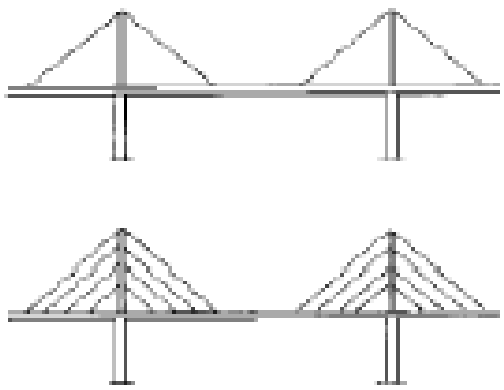
1.4.4. Cable-stayed footbridges

A cable-stayed bridge has one or more towers (or pylons), from which cables support the bridge deck.

In the cable-stayed bridge, the towers are the primary load-bearing structures which transmit the bridge loads to the ground. A cantilever approach is often used to support the bridge deck near the towers, but lengths further from them are supported by cables running directly to the towers. All static horizontal forces of the cable-stayed bridge are balanced so that the supporting towers does not tend to tilt or slide, only needs to resist horizontal forces from the live loads.

In cable-stayed bridges the main longitudinal girders are supported by a few or many ties in the vertical or near-vertical plane, which are hung from one or more tall towers and are usually anchored at the bottom to the girders.

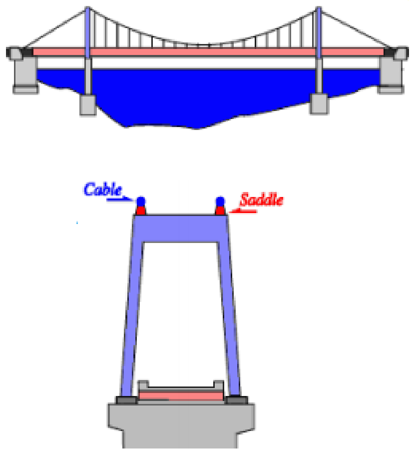
Steel cables are extremely strong but very flexible (only a few cables are strong enough to support the entire bridge, their flexibility makes them weak to a force we rarely consider: the wind).



Picture 4. Examples of cable-stayed bridges (pictures from [2]).

1.4.5. Suspension footbridges

The suspension bridge allows for the longest span. This is a type of bridge in which the deck (the load-bearing portion) is hung below suspension cables on vertical suspenders. Cables are suspended between towers, vertical suspender cables that carry the weight of the deck below, upon which traffic crosses. This arrangement allows the deck to be level or to arc upward for additional clearance. The suspension cables must be anchored at each end of the bridge, because any load applied to the bridge is transformed into a tension in these main cables. The main cables continue beyond the pillars to deck-level supports, and further continue to connections with anchors in the ground. The roadway is supported by vertical suspender cables or rods, called hangers.

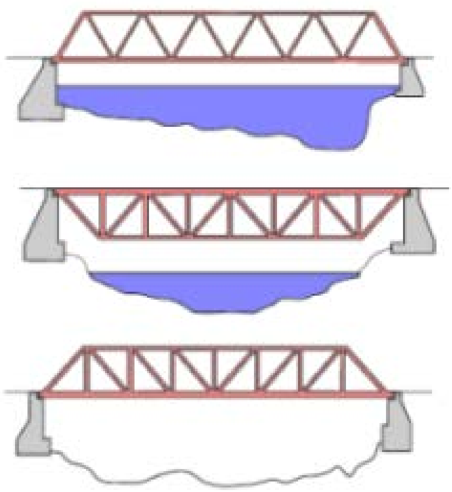


Picture 5. Examples of cable-stayed bridges (pictures from [3]).

1.4.6. Truss footbridges

The truss is a simple skeletal structure. A truss is a triangulated framework of elements that act primarily in tension and compression. It is a light-weight yet very stiff form of construction.

Trusses may be used as girders below the deck level, or as through girders with the deck at the bottom chord level. Such through-truss girders minimize the effective construction depth, and the length of approach embankments. Hence, they are particularly suited to footbridges and railway bridges.



Picture 6. Examples of truss bridges (pictures from [3]).

1.5. Examples of footbridges

In this section are shown some of the most interesting examples of the different type of bridges in the world.

1.5.1. South Wharf Foot Bridge

The Seafarers Bridge is a footbridge over the Yarra River between Docklands and South Wharf in Melbourne, Victoria, in Australia. It was built in December 2010. This is a typical example of a cable-stayed bridge, made of steel.

The bridge main span is supported by steel ties connected to elliptical arches, with three arches on the north side and four arches on the south side. The longest span of 75 meters.

Picture 7. South Wharf Footbridge (pictures from [4] and [5]).

1.5.2. Footbridge in La Roche Sur Yon

The footbridge involves the using lateral beams composed of a diagonal mesh of small plate strips that are riveted together. It was built in February 2010.

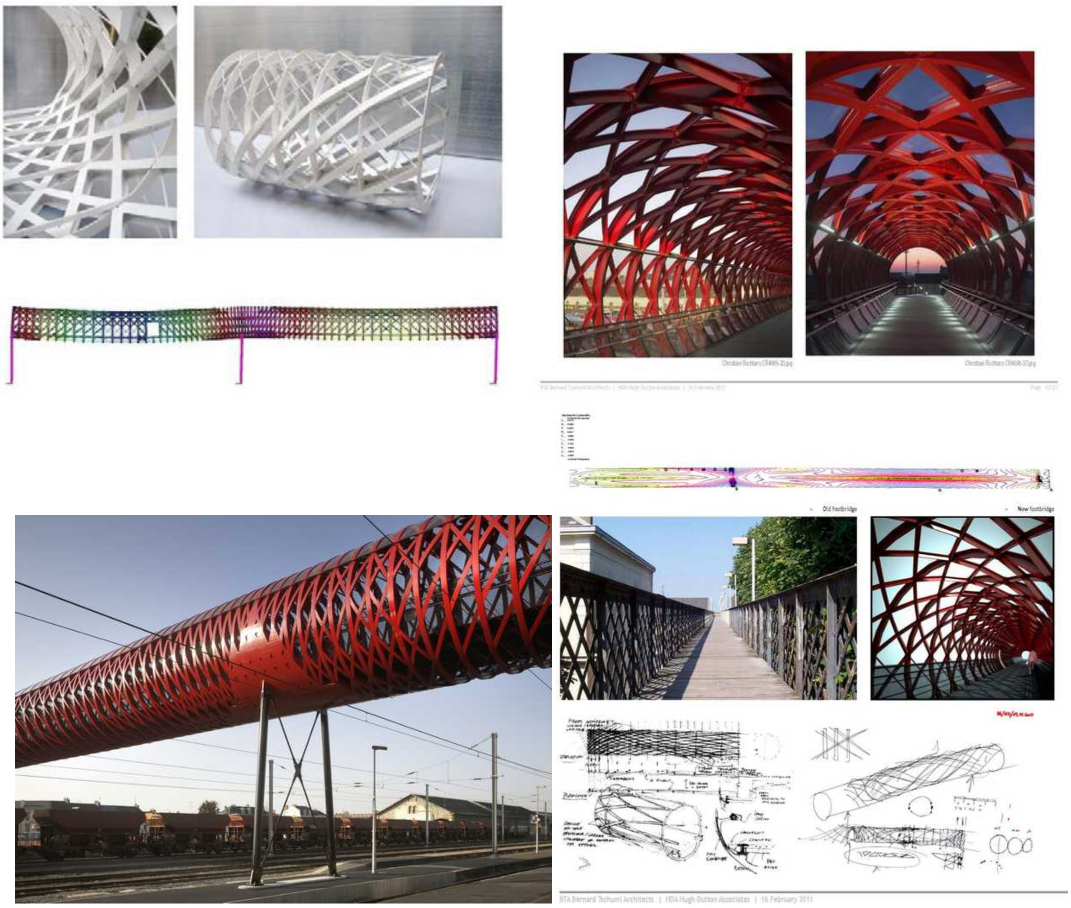
The design of the bridge is made of a diagonal mesh, in a tubular. Footbridges over railways require lateral protection for safety of both the users and the trains below. The complete volume provides a single structural solution that possesses the necessary inertia to span between the available support points as well as provide support for the required protective screens and a canopy cover.

Length of the footbridge is: 67 meters (35 32 spans). Used: 201 square meters of concrete deck, 160 tons of total weight, of which 130 tons of steel i.e. 1,9 tons per linear meter, 76 circular diaphragms, 2100 compressive diagonals, 600 tensile rods, 4300 meters of profiles, about 10 km of welding, 1800 conical nodes.

The footbridge is an example of a bridge truss. The main component the carrier is grid.

The triangulated mesh of the Roche sur Yon main structural tube is articulated to distinguish between the tensile and compression forces by using simple tie rods for the tensile members. The ties have no compressive capacity and express therefore the tensile zones.

The compressive members are in ‘T’ or ‘H’ sections corresponding to the magnitude of forces in them. The section sizes of the members vary as a function of the loading to optimize the steel mass and further express the forces in the system. Mid-span, the lower chords are tensile, while the upper members are compressed. The inverse is true at the support points, where the bending moments are inverted. The shear forces in tubular truss are generally greater at the support points and tending more and more vertical the closer one approaches the supports. The pattern of triangulation of the truss follows this change in direction of forces.



Picture 8. Footbridge La Roche sur Yon, in France (pictures from [6])

1.5.3. Merchant Square footbridge in Grand Union Canal, England

Totally another example of crossing the bridge is footbridge in Merchant Square, Grand Union Canal. It will be built in 2014.

The 3m wide cantilevered Merchant Square footbridge spans 20m across the Grand Union Canal and is raised using hydraulic jacks. Footbridge was designed with five fabricated steel beams. The beams forming the deck open in sequence, with the first rising to an angle of 80 degrees and the last achieving the required clearance over the canal of 2.5m tall by 5.5m wide at mid channel. The bridge balustrades are formed from twin rows of inclined stainless steel rods, overlapping to form a robust yet filigree and highly transparent structure.

Picture 9. Merchant Square footbridge in Grand Union Canal, England (pictures from [7]).

1.5.4. Footbridge over the River Segre in Lleida, Spain

The footbridge is only a link between the two banks of the river Segre where it crosses the city of Lleida. It was built in 2000. The crossing of the entire width of the river is solved with two single pillars and no other support. Central span is 83m

Two steel box beams arranged on different levels, stiffened by means of transversal beams made from the same material and a concrete slab, permit understanding the surface as a structural suite with a Z-section. The slab overhangs the lower beam by 1, 15 meters and reduces the visual effect of the two-meter edge.

Picture 10. Footbridge Over the River Segre in Lleida (pictures from [8]).

1.5.5. Arganzuela Footbridge in Madrid, Spain

A series of bridges over the river will allow passage from one side of the park to the other. Designed to link the neighborhoods on the right and left banks of the river, the Arganzuela Footbridge will be the longest of all the built bridges. The bridge will be for both pedestrians and cyclists. It was built in 2010.

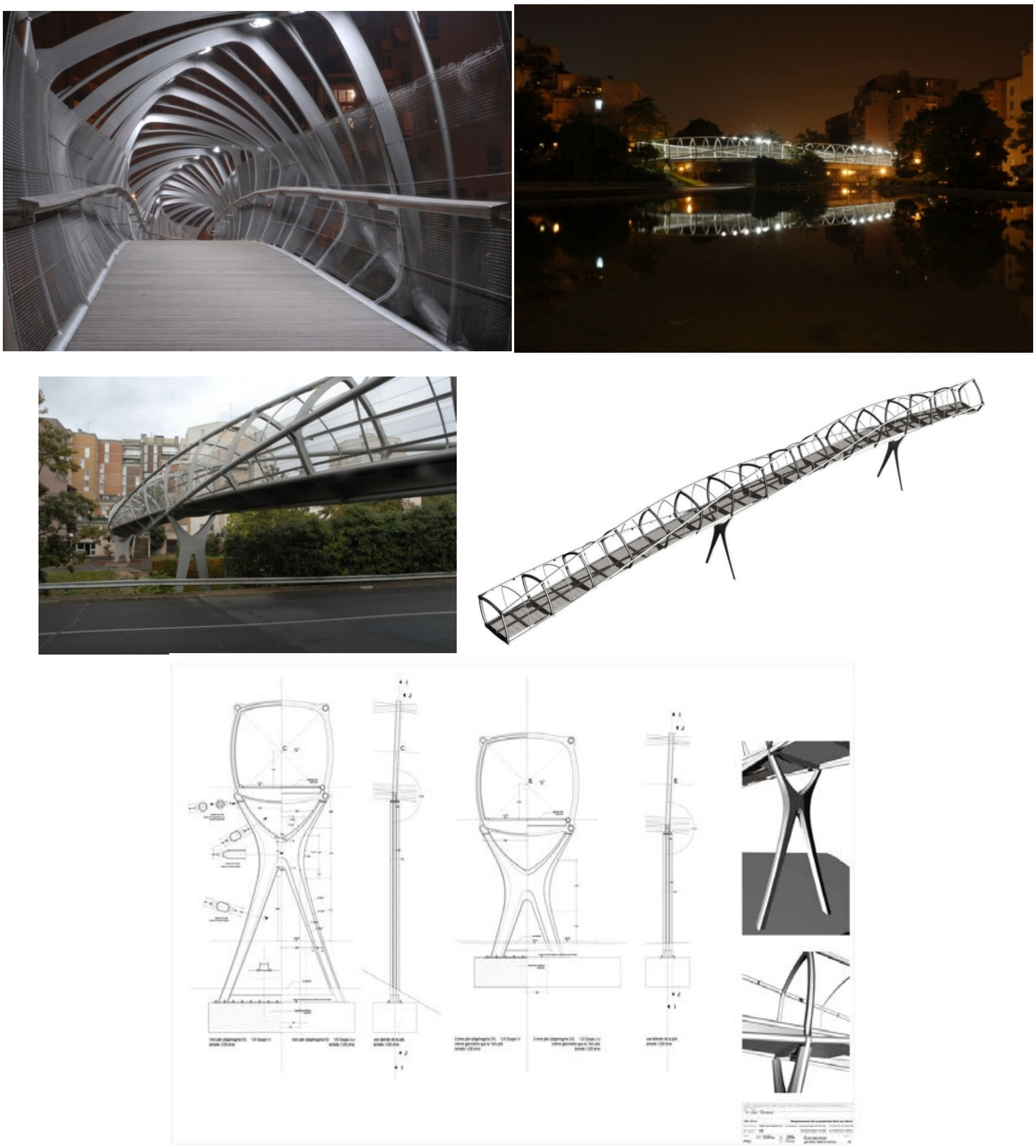
Built area is: footbridge 150 m (section 1) 128 m (section 2) lengths, 5 to 12 m width. Cone like in structure, the bridge has two interlocking metal spirals, wrapped by a metallic ribbon. Spaced wooden slats make up the floor of the bridge.

Picture 11. Arganzuela Footbridge in Madrid (pictures from [9]).

1.5.6. Pedestrian footbridge in Evry, France

Bridge in Evry connected safe, fun and transparent, span is 70m. It was built in 2008. This footbridge combined architectural intentions and technical considerations and creates a steel structure forming a volume which encompasses pedestrians.

The load-bearing structure consists of an array of round tubes echoing the rotation of DNA across the entire span of the footbridge. Each tube makes a quarter turn per section, successively becoming part of the load-bearing structure beneath the decking, the load-bearing structure beneath the anti-vandal mesh on the sides, and an aesthetic element integrating the lights above the pedestrians.



Picture 12. Pedestrian footbridge in Evry, France (pictures from [10]).

1.5.7. Footbridge in Avenida Presidente Vargas, Brazil

Footbridge is located on Presidente Vargas Avenue, Gamboa, downtown Rio de Janeiro. It was built in 2010.

In attempt to connect Gamboa with the major corporate buildings on the opposite side, a 207 meter-long covered footbridge was designed to allow both pedestrian crossing and bus-metro integration. Supported by only three long-span steel arches with entwined bases, which two of are 90 meter-long and one 40 meter-long.

The footbridge is the only access to the station .It connects both sides of the avenue and also gives access to passengers from bus corridors located on the central reservations bellow, which are equipped with staircases, escalators and lifts for disabled people.

Picture 13. Footbridge in Avenida Presidente Vargas (pictures from [11]).

1.5.8. Helix Bridge, Marina Bay, Singapore

The Helix connected seamlessly with the pedestrian promenade at the Bayfrontand Marina Promenade, and is part of a 3.5km long waterfront promenade that loops around the Bay. The 280m bridge comprises five spans (three internal spans, each is 65m and two approach spans, each is 43m). It was built in 2010.

The resulting bridge comprises two delicate helix structures that act together as a tubular truss to resist the design loads. This approach was inspired by the form of the curved DNA structure. The helix tubes only touch each other in one position, under the bridge deck. The two spiraling parts are held apart by a series of light struts and rods, as well as stiffening rings, to form a rigid structure. This arrangement is strong and ideal for the curved form. The stainless bridge is met by concrete abutments at either side.

The 280 m bridge is made up of three 65 m spans and two 45 m end spans. If the steel were stretched out straight from end to end, it would measure 2.25 km in length. The major and minor helices, which spiral in opposite directions, have an overall diameter of 10.8 m and 9.4 m respectively, about 3-storeys high. The outer helix is formed from six tubes (273 mm in diameter) which are set equidistant from one another.

The inner helix consists of five tubes, also 273 mm in diameter. Over the river, the bridge is supported by unusually light tapered stainless steel columns, which are filled with concrete. The columns form inverted tripod shapes which support the bridge above each of the pilecaps. The bridge weighs around 1700 tons in total.

This incredibly intricate and lightweight bridge uses five times less steel than a conventional box girder bridge. Each steel member is optimized for strength and only a few different section sizes are used.

Picture 14. Helix Bridge, Marina Bay, Singapore (pictures from [12], [13]; and [14]).

1.5.9. Calatrava footbridge in Bilbao, Spain

The Zubizuri (Basque for "white bridge"), also called the Campo Volantin Bridge or Puente del Campo Volantin, is a tied arch footbridge across the Nervion River in Bilbao, linking the Campo Volantin on right bank and Uribitarte on left bank of the river. It was built in 1997. This is a suspension bridge. The design consists of an inclined structural steel arch linking two platforms, with access ramps and stairways on both banks.

Picture 15. Calatrava footbridge in Bilbao, (pictures courtesy of Prof. Dr. Nuno Lopes)

1.5.10. Ponte Pedonal Circular, Aveiro, Portugal

Ponte Pedonal Circular is a circular footbridge in Aveiro, which joins the three banks of canals. The footbridge has a diameter of 26 meters with 2 meters wide walkway. The bridge is used by pedestrians and cyclists and also has a ramp for persons with limited mobility.

The footbridge can be characterized by a circular tray suspended by metal rods from its inner edge to the mast tilted. The mast is supported by metal bars tied with mass of concrete. The solution for indirect foundations of the mast and the massive mooring allowed the almost complete elimination of horizontal forces transmitted to the massive geotechnical.

Picture 16. Circular footbridge, Aveiro, Portugal

1.5.11. Footbridge in the University of Aveiro, Aveiro, Portugal

The footbridge connects canteen on campus of the University of Aveiro. It is possible to navigate the footbridge by bike and is designed for movement disabled people - structure has two driveways.

Engineering design is an excellent example of a truss bridge. In the design reset to transverse forces and bending moments and twisting in the bars. Nonzero only are axial forces. The steel bridge has a plurality of support that is the foundation and moves the internal forces to the ground.

This example of the bridge characterize by a high stiffness of the structure.

Picture 17. Footbridge in the University of Aveiro, Aveiro, Portugal

1.6. Design rules

Calculation and design of the footbridge will be made in accordance with:

- EN 1990:2002 [15] (basic design rules);
- EN 1991:2003 [16] (actions on structures);
- EN 1993:2005 [17] (design of steel general rules);

1.7. Action on footbridges:

The footbridges are following loads [*18*]:

- Self-weight of the structure and other permanent action;
- Snow loads;
- Wind action on footbridges;

- Thermal actions;
- Action during execution;
- Traffic loads on footbridges;

1.8. Vibrations

During the design of footbridges special attention should be paid to vibration. Footbridges are characterized by low weight and increasing efficiency in the material, which in turn causes in a high ratio of load. As a result of this trend, many footbridges have become more susceptible to vibrations when subjected to dynamic loads. The dynamic forces can cause dangerous amplitudes of the vibration. Vibrations of footbridges may occur in vertical and horizontal directions. The vibrations cause the risk of resonance. [19]

If the vibration behavior due to expected pedestrian traffic is checked with dynamic calculations and satisfies the requirement comfort, any type of footbridge can be designed and constructed [20], [21].

1.9. Concluding remarks

The footbridges are complex engineering structures. Emphasize that exist a lot of different types of footbridges.

The footbridge will be designed in steel S235. It is a versatile and effective material that provides efficient and sustainable solutions and at the same time cheap. [*22*]

For construction works multiple workloads: dead, exploitation, should also take into account the effect of vibration.

2. Pre-design

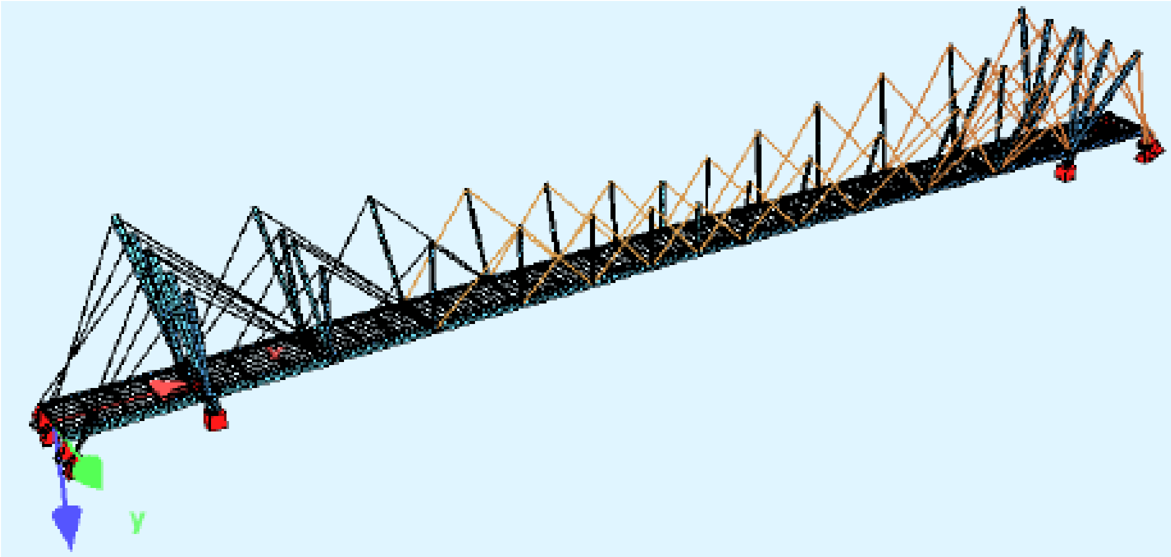
In this part of this work I have been considered a variety of design solutions-discussed construction of the footbridge. All examples solutions satisfy the initial conditions-both appropriate, the geometric and material.

Possibilities considered (there are not all possible solutions, because there are very many different combinations of solution):

2.1. The proposal I

Another example is considered footbridge type tensegrity structure. This is an extremely interesting solution due to the superstructure, in which there is mutual stabilization of compression and tension elements.

The following is a sketch of the solution:



Picture 19. Tensegrity structure footbridge

Span of structures is 14m, and the width is equal to 3m (like ramps width). The construction is placed at a height of 4 m above the ground level.

The mechanical stability of the structure provides an initial compression of the structure.

All components are connected by only the ends of the pins. The rigid rod elements are joined together by means of slender elements (taut rope, thin rods, etc.). Structural components work as trusses-only compressive and tensile. The main structural component of this solution is the truss, is the most visible of prefabricated pair on both sides of the platform. The footbridge's trusses are reversed in such a way as to create the suspended element. Each truss is composed of poles of which are extreme pylons secured into the foundation and the center of the platform is connected to the posts. Applied masts (part of the truss) are different height, diameter and slope.

The primary disadvantages of this type of structure should be the need to perform complex analysis due to large deformations, susceptibility to the effects of dynamic loads and the need for analysis of the various phases of assembly. It is the appearance of the hassle of complicated, multi-phase assembly and costly and often laborious in execution details (nodes) and difficult program compression.

Considered footbridge with the ability to navigate through the pedestrians, cyclists and adapted for use for people with disabilities. For the applied properly profiled ramps.

2.2. The proposal II

Another example is considered footbridge suspended from the bridge to the arc.

It is a very attractive solution, for reasons of attractiveness of architectural forms, and the possibility of using the platform of a small construction height.

The sketch below shows the structure of the arc:

Picture 20. Arch structure footbridge

Span of structures is 14m, and the width is equal to 3m. The design is placed at a height of 4.5 m above ground level. (Ramps provide this height).

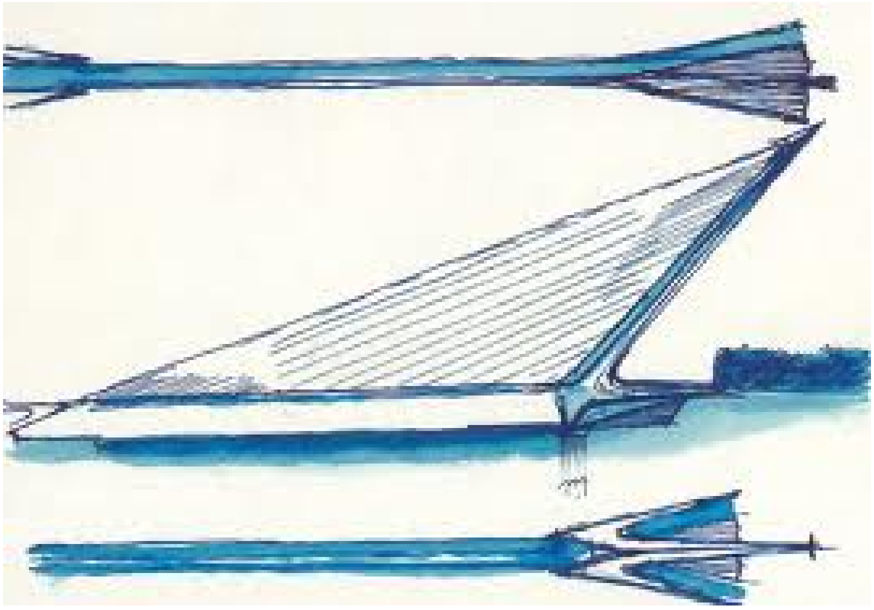
Used hangers support each side of the platform. Arc works like a bent beam loaded by horizontal forces. The most preferred solution is to use in construction of the box section. The arc should be using rods with high strength steel anchor block beachhead. The platform can be suspended by rope strands with heads spade and a combination of active plating placed under a bridge. It must be anchored in the deck with the use of coaxial cables boxes (to minimize the size of the anchorage). Steel, fixed arch consists of a steel grate combined with a reinforced concrete slab. The bow tie is not used, because the whole force of expansion is transferred to massive reinforced concrete abutments. Smooth transition arch-support is achieved by pulling over a bridge abutment blocks and by reducing its cross-section upwards. Props must be set up on stilts.

This construction requires substantial foundations due to the large horizontal forces (forces of expansion). It is their major drawback. The foundation must also be placed deep in the ground.

Properly profiled ramps allow bicycle traffic and make pedestrian bridge is designed for usage by persons with disabilities.

2.3. The proposal III

During the selection of the structural model proposed footbridge also considered a suspended footbridge. Suspended structures meet architectural value, and economical for large spans. The following is a sketch of this solution:



Picture 21. Suspended structure footbridge

In the proposed embodiment of the span structure is 14.5 m, while the width is equal to 3m. The design is placed at a height of 4.5 m above ground level. The mast structure has a height of 10m (measured from ground level to the highest point).

Loads of footbridges are transferred to the pylon directly by steel shrouds. Used as a single pylon mast. It was decided to apply the suspension harp, one-sided, without the use of any extractor. This gives the dynamism of the structure. In the present facility pylon bending forces are balanced only by the weight of the mast tilted from the vertical. In order to ensure the safety of the structure (formed large bending moments) decided to thicken the lower part of the pylon. A suspension cables are anchored to a large eccentricity (in relation to the axis of the pylon). Created a very large torsional moments.

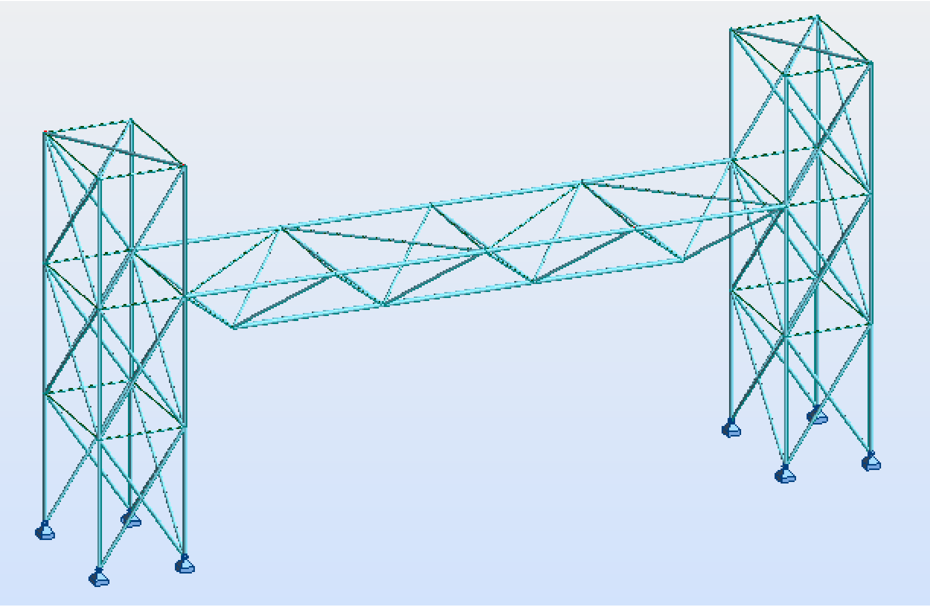
Due to the large range of design tasks suspended footbridge is not the best solution.

Access is assured for the disabled and cyclists.

2.4. The proposal IV

The last case under consideration is a single-span steel bridge built in the lattice structure of the tubular elements of round section.

The following is a sketch of the solution:



Picture 21. Truss structure footbridge

The proposed solve span structure is 14m, and the width is equal to 3m. The design is placed at a height of 4.5 m above ground level.

Truss support element in cross-section is in the shape of a triangle equilateral. The design consists of a lattice girder footbridge connected by welding at the nodes. Span mounted by a screw joint span. Columns supporting structures made of circular steel sections. The supports are designed with circle cross-section tubes, which are a supporting structure for the stairs. The platform is made of plate steel-concrete composite.

For effective use of the individual elements of the grid parameters should be used for the side sections of pipe, columns and diagonals with possibly different thicknesses of the same external dimensions of the components of that type.

Considered footbridge with the ability to navigate through the pedestrians, cyclists and adapted for use for people with disabilities by using a suitably profiled ramp

2.5. Selection designed construction

Due to the environmental conditions and the inspiration footbridge at the University of Aveiro was chosen for modeling truss footbridge. An important advantage of the chosen design is the high rigidity, safety and the ability to mount the bracket.

Truss design provides own high stiffness, another advantage is zeroing to the moments and shear forces in structural elements.

The truss bridges are widely used in terms of structures exposed to high dynamic actions and is not needed where a large width of the crossing.

Because of the large number of cyclists and the promotion of healthy lifestyles decided to allow cyclists to move the structure design. Due to the care of people with disabilities was decided to create two elevators.

3. Definition of the actions

The loads acting on structures have been inflicted in accordance with EN 1991-1 Eurocode 1: Actions on structures.

3.1. Data

The table attached to this work as Annex 2 presents a summary of all defined burden in this chapter.

The above table contains the number of the case load, its characteristics, the list of bars which have been given load, the place of the load on the bars and the selected workloads.

The calculation presented in this chapter of work has been done on the basis of adopted data, which are as follows:

Dimensions the designed structure:

Width:	B=3m
Overall length (with lift):	L=18m
Width of elevator:	$B_e = 3m$
Length of elevator:	$L_e = 2m$
Height of elevator:	$H_e = 9m$
The length of the truss:	$L_t = L - 2L_w = 14m$
The width of the truss:	$B_t = 3m$
The height of the truss:	$H_t = 1{,}5m$

Note: Dimensions of the structure are given in axes

The most important parameters adopted:

Acceleration due to gravity:	$g = 9{,}81 \frac{N}{kg}$
Weight of steel:	$\gamma_s = 7850 \frac{kg}{m^3}$
Height above ground:	$z = 9{,}0[m]$

3.2. Self weight of structural

3.2.1. Self weight of structural steelwork

Load value was adopted in accordance with EN 1991-1 Eurocode 1: Actions on structures - Part 1-1: General actions - densities, self-weight, Imposed loads for buildings.

Self weight of structural steelwork elements in program [a] is set automatically. Have been given the following features the load:

Label:	1:STA1
Nature:	dead

3.2.2. The balustrade

Load value was adopted in accordance with EN 1991-1 Eurocode 1: Actions on structures - Part 1-1: General actions - densities, self-weight, Imposed loads for buildings.

For safety reasons, provided for installing steel balustrade with a height of 1.1 m.

Parameters balustrade:

Span length:	2000mm
Material:	Steel S235JR
Spacing of posts:	1000-2000mm
Weight h-1, 1m (flat 100x12 and 50x10):	48kg/linear meter

Note: Manufacturer's catalog [23].

Railing used on both sides of the footbridge.

The appearance of used balustrade:

Picture 22. The selected balustrade [23]

Balustrade load acting on truss structure:

$$\gamma_b = m_b \cdot g = 48 \left[\frac{kg}{m}\right] \cdot 9{,}81 \left[\frac{N}{kg}\right] = 0{,}471 \left[\frac{kN}{m}\right]$$

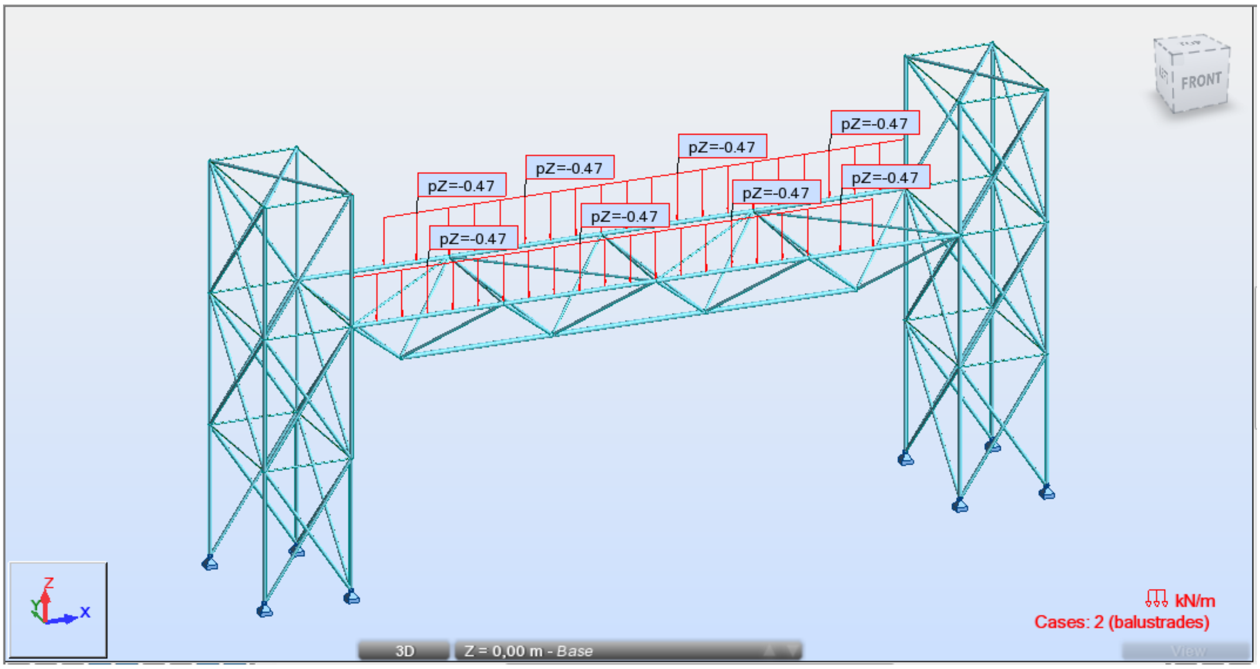
Have been given the following features the load:

Label: 2: balustrades

Nature: dead

Load weight balustrade should be modeled as linear load acting on the upper chord of the truss.

The load specified from a way shown in the figure below:



Picture 23. Screenshot of the program [a]- loads: balustrades

3.2.3. The pavement

Load value was adopted in accordance with EN 1991-1 Eurocode 1: Actions on structures - Part 1-1: General actions - densities, self-weight, Imposed loads for buildings.

For aesthetic reasons pavement structure is made from African hardwood Azobe (Bongossi), all parameters are shown in [24]. It is weather resistant and very durable.

Used boards with dimensions 30x125x3000mm, maintains a small spacing between boards for maintaining the bridge clean.

Density (self weight of wood): $\rho_w = 1200 \left[\frac{kg}{m^3}\right]$

Self weight of wood planks:

$$\gamma_w = \rho_w \cdot g \cdot 30mm = 1200 \left[\frac{kg}{m^3}\right] \cdot 9{,}81 \left[\frac{N}{kg}\right] \cdot 30[mm] = 0{,}353 \left[\frac{kN}{m}\right]$$

Wood should be attached to the structure by means of screws, so increased surface load of 25%:

$$\gamma_{ws} = 1{,}25[-] \cdot \gamma_w = 1{,}25 \cdot 0{,}353 \left[\frac{kN}{m}\right] = 0{,}441 \left[\frac{kN}{m^2}\right]$$

Have been given the following features the load:

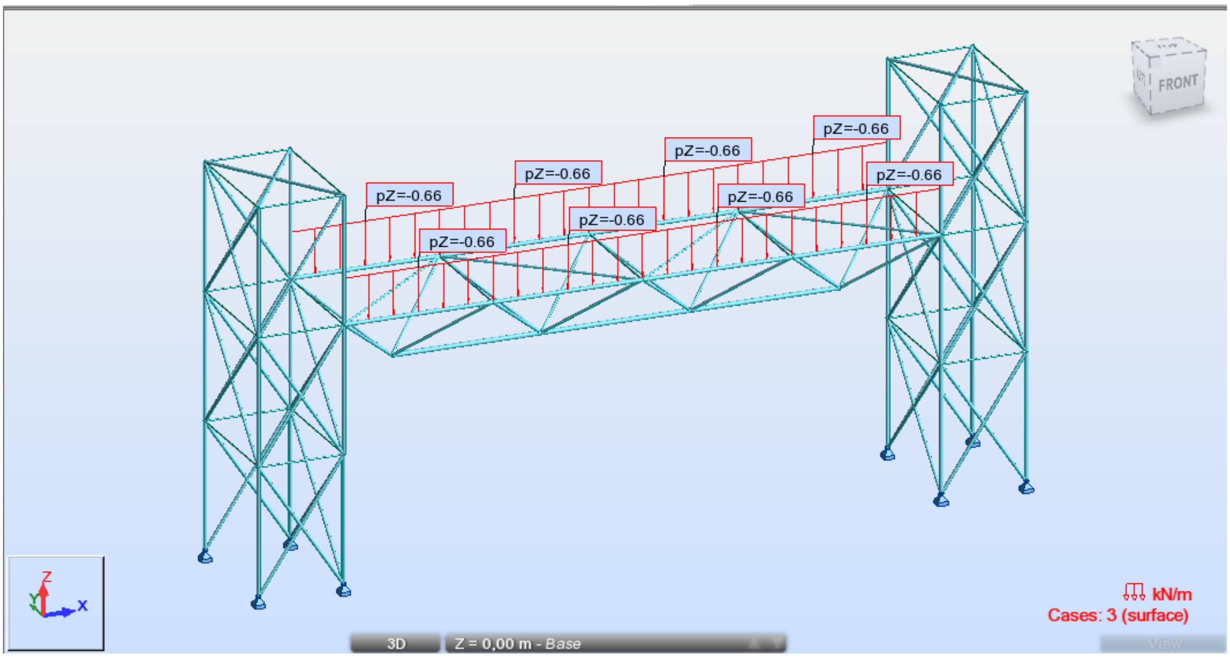
Label: 3: surface

Nature: dead

Weight should be distributed on the surface upper chord of the truss as linear loads the following values:

$$Q_{surf} = \gamma_{ws} \cdot \frac{3[m]}{2} = 0{,}441 \left[\frac{kN}{m^2}\right] \cdot \frac{3[m]}{2} = 0{,}662 \left[\frac{kN}{m}\right]$$

The load specified from a way shown in the figure below:



Picture 24. Screenshot of the program [a]- loads: surface

3.2.4. Ramp for bicycles and disabled

According to the Decree of the Minister of Infrastructure dated 12 April 2002 on the technical conditions to be met by buildings and their location, OJ No 2002, No. 75, item. 690 paragraph 70 maximum slope of the ramp for bikes and people with disabilities, ramps above 0.5 m, located on the outside of the building is 6%.

Below shows the calculation of the minimum course length of the ramp:

The conversion of the slope in percentage to radian:

$$\theta = \frac{6[\%] \cdot 0,5 \cdot \pi}{100[\%]} = 0,094[rad]$$

At the pedestrian overpass at a height of 6m and the required length of the ramp slope is:

$$\frac{6[\mathrm{m}]}{\tan(\theta)} = 63,473[m]$$

For economic reasons and limited space, it was decided to give up the construction of driveways. The movement of disabled people and cyclists will be carried out using the elevator.

3.2.5. Stairs

Load value was adopted in accordance with EN 1991-1 Eurocode 1: Actions on structures - Part 1-1: General actions - densities, self-weight, Imposed loads for buildings.

System designed steel stairs with rails on both sides. Number of steps: 35, step width 29cm, height of 16.67 cm. Weight stairs is carried by poles on which they are based. The construction of the bridge carries the weight of only the last 3 steps along with a railing.

Picture 25. Steel stairs

Every 10 steps starting from the bottom find a landing single width 1.5 m The entire structure contains three landings, under which there are poles carrying the weight of the structure according to (Regulation of the Minister of Infrastructure dated 12 April 2002 on the technical conditions to be met by buildings and their location (Journal of Laws of 2002, No. 75, item. 690)).

Steps consist of flat steel measuring 29 x 3cm and length equal to 2m. The thickness is 3mm flat; the thickness of one stair tread is 30mm. According to the catalog of the manufacturer one step weighs 6kg.

Picture 26. Steel steps

Load the stairs acting on the grid:

Weight of steps:

$$\gamma_{sta} = 3 \cdot m_{sch} \cdot \frac{g}{2[m]} = 3 \cdot 6[kg] \cdot \frac{9{,}81\left[\frac{N}{kg}\right]}{2[m]} = 0{,}088\left[\frac{kN}{m}\right]$$

Weight of balustrade:

$$\gamma_{bs} = m_b \cdot g \cdot \frac{m}{3 \cdot 29[cm]} = 48\left[\frac{kg}{m}\right] \cdot 9{,}81\left[\frac{N}{kg}\right] \cdot \frac{[m]}{3 \cdot 29[cm]} = 0{,}541[kN/m]$$

On the grid is transferred to the following load of stairs and barriers:

$$\gamma_{sb} = \gamma_{sta} + \gamma_{bs} = 0{,}088\left[\frac{kN}{m}\right] + 0{,}541\left[\frac{kN}{m}\right] = 0{,}629\left[\frac{kN}{m}\right]$$

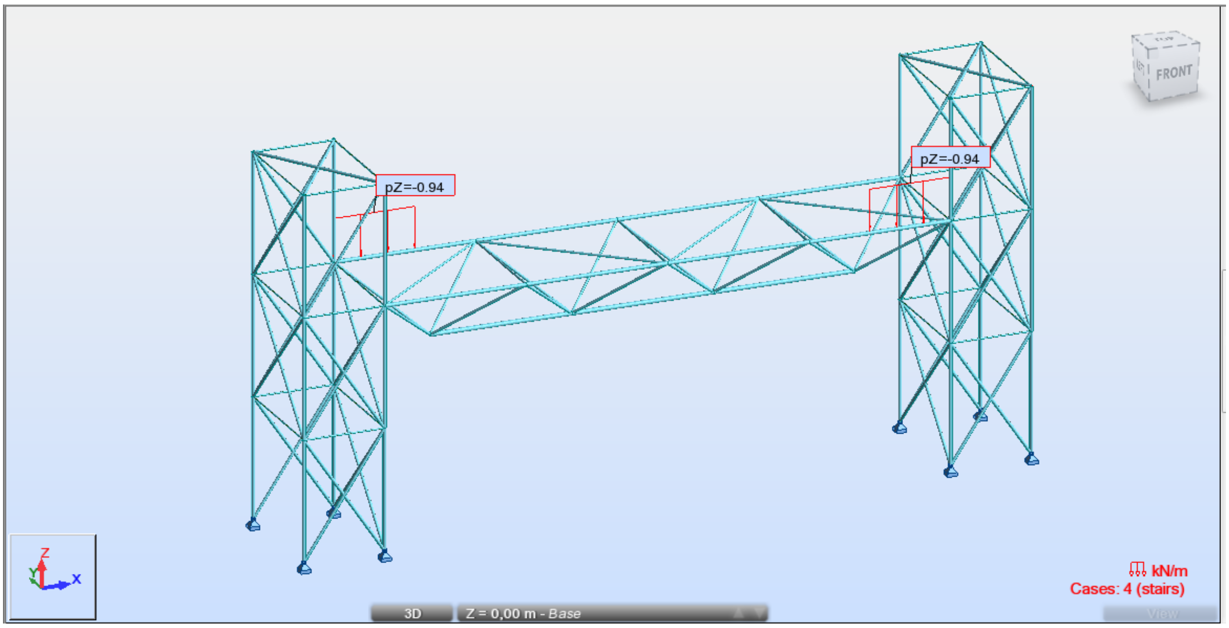
This load must be increased by 50% due to the bolt and the connecting steel components stairs.

$$\gamma_{st} = 1{,}5(\gamma_{sta} + \gamma_{bs}) = 1{,}5\left(0{,}088\left[\frac{kN}{m}\right] + 0{,}541\left[\frac{kN}{m}\right]\right) = 0{,}944\left[\frac{kN}{m}\right]$$

Have been given the following features the load:

Label:	4: stairs
Nature:	dead

The load specified from a way shown in the figure below:



Picture 27. Screenshot of the program [a]- loads: stairs

3.2.6. Elevator

Weight lift is moved by a frame of the device.

3.3. Snow

Load value was adopted in accordance with EN 1991-1-3 Eurocode 1 - Actions on structures - Part 1-3: General actions - Snow loads. All references in this section refer to this standard.

Analysis of snow load is carried out as for the flat roof.

Łódź is located in II Zone snow load (picture NB.1).

The characteristic value of snow load on tile ground for this zone is:

$$s_k = 0{,}9 \left[\frac{kN}{m^2}\right] \text{ (table NB.1)}$$

The thermal coefficient or the structure located on the outside is:

$$C_t = 1{,}0 \; [-] \text{ (according point 5.2.(8))}$$

The exposure coefficient for normal land:

$$C_e = 1{,}0 \; [-] \text{ (according point 5.2.(7))}$$

Shape factor the roof is (flat roof):

$\mu_1 = 0{,}8[-]$ (table 5.2) $\mu_2 = 0{,}8[-]$ (table 5.2)

Snow loads on roofs shall be determined for the persistent / transient design situations as follows:

$$s = \mu_1 \cdot C_e \cdot C_t \cdot s_k = 0{,}8[-] \cdot 1{,}0[-] \cdot 1{,}0[-] \cdot 0{,}9 \left[\frac{kN}{m^2}\right] = 0{,}72 \left[\frac{kN}{m^2}\right] \text{ (formula 5.1)}$$

Have been given the following features the load:

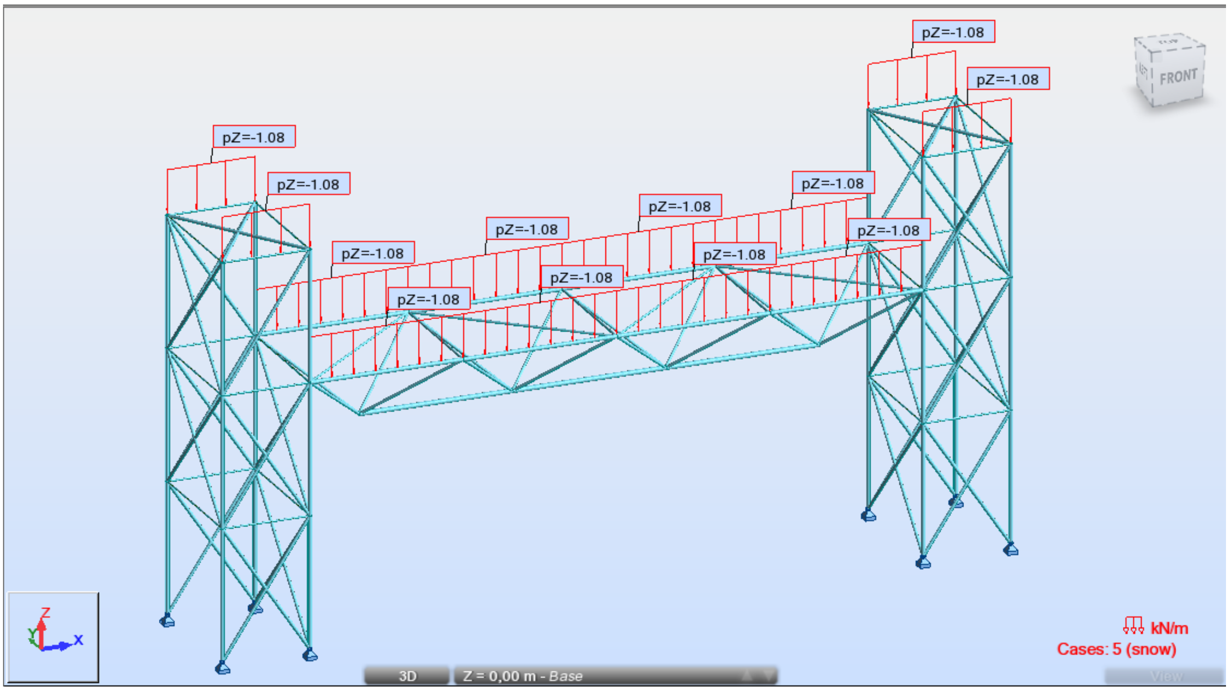
Label: 5: snow

Nature: snow

Snow load should be distributed on the surface upper chord of the truss as a linear load a value equal to:

$$Q_s = s \cdot \frac{3[m]}{2} = 0{,}72 \left[\frac{kN}{m^2}\right] \cdot \frac{3[m]}{2} = 1{,}08 \left[\frac{kN}{m}\right]$$

The load specified from a way shown in the figure below:



Picture 28. Screenshot of the program [a]- loads: snow

3.3.1. The accidental design situations

Snow loads on roofs shall be determined for the accidental design situations where exceptional snow load is the accidental action as described below:

The coefficient for exceptional snow loads:

$$C_{esl} = 2{,}0\ [-] \text{ (according NOTE point 4.3)}$$

The design value of exceptional snow load on the ground for the given location:

$$s_{Ad} = C_{esl} \cdot s_k = 1{,}8 \left[\frac{kN}{m^2}\right] \text{ (formula 4.1)}$$

Snow loads:

$$s = \mu_1 \cdot C_e \cdot C_t \cdot s_{Ad} = 0{,}8[-] \cdot 1{,}0[-] \cdot 1{,}0[-] \cdot 1{,}8 \left[\frac{kN}{m^2}\right] = 1{,}44 \left[\frac{kN}{m^2}\right] \text{(formula 5.2)}$$

There are no snowdrifts.

Have been given the following features the load:

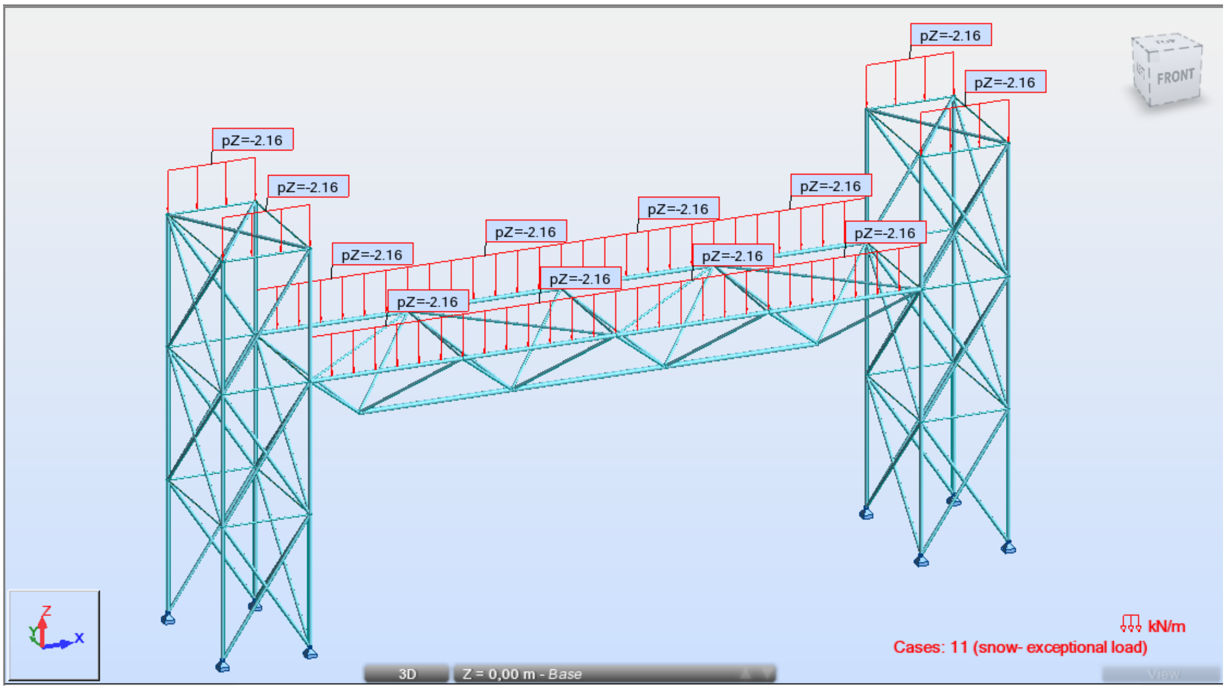
Label: 11: snow- exceptional load

Nature: snow

Snow load should be distributed on the surface upper chord of the truss as a linear load a value equal to:

$$Q_s = s \cdot \frac{3[m]}{2} = 1{,}44 \left[\frac{kN}{m^2}\right] \cdot \frac{3[m]}{2} = 2{,}16 \left[\frac{kN}{m}\right]$$

The load specified from a way shown in the figure below:



Picture 29. Screenshot of the program [a]- loads: snow- exceptional load

3.4. Wind

Load value was adopted in accordance with EN 1991-1-4 Eurocode 1: Actions on structures — General actions — Part 1-4: Wind actions.

Łódź is located in the I zone of the wind load (picture NA.1).

The level of the projected area of less than 300 m above sea level (Lodz 161,8-278,5 m above sea level).

The fundamental value of the basic wind velocity is:

$$v_{b,0} = 22{,}0 \left[\frac{m}{s}\right] \text{(table NA.1)}$$

The basic velocity pressure:

$$q_{b,0} = 0{,}3\ \left[\frac{kN}{m^2}\right] \text{ (table NA.1)}$$

The directional factor:

$$c_{dir} = 1{,}0\ [-] \text{ (according 4.2. NOTE 2)}$$

The season factor:

$$season = 1{,}0\ [-] \text{ (according 4.2. NOTE 3)}$$

The basic wind velocity:

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} = 1{,}0 \cdot 1{,}0 \cdot 22{,}0\ \left[\frac{m}{s}\right] = 22{,}0\ \left[\frac{m}{s}\right] \text{ (formula 4.1)}$$

The orography factor:

$$c_0(z) = 1{,}0[-] \text{ (according 4.3. NOTE 1)}$$

Terrain categories and terrain parameters were defined as the Terrain II category:

Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights (table 4.1.)

Area ratio depends on the amount surface roughness:

$$z_{0II} = 0{,}05[m] \text{ (table 4.1.)} \qquad z_0 = 0{,}05[m] \text{ (table 4.1.)}$$

The terrain factor depending on the roughness length z_0 :

$$k_r = 0{,}19\left(\frac{z_0}{z_{0II}}\right)^{0{,}07} = 0{,}19\left(\frac{0{,}05[m]}{0{,}05[m]}\right)^{0{,}07} = 0{,}19[-] \text{ (formula 4.5.)}$$

Because:

$$z_{min} < z < z_{max}$$

Where:

z_{min} - the minimum height defined in Table 4.1

z_{max} - 200 m (according 4.3.2)

$$2[m] < 9[m] < 200[m]$$

According 4.3.2. the roughness factor is equal to:

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) = k_r \cdot \ln\left(\frac{z}{z_0}\right) = 0{,}19 \cdot \ln\left(\frac{9[m]}{0{,}05[m]}\right) = 0{,}987[-] \text{ (formula 4.4.)}$$

The wind velocity:

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b = 0{,}987[-] \cdot 1{,}0[-] \cdot 22{,}0\ \left[\frac{m}{s}\right] = 21{,}707\left[\frac{m}{s}\right] \text{ (formula 4.3.)}$$

The turbulence factor:

$$k_1 = 1{,}0[-] \text{ (according 4.4.(1))}$$

The standard deviation of the turbulence:

$$\sigma_v = k_r \cdot v_b \cdot k_1 = 0{,}19[-] \cdot 22\left[\frac{m}{s}\right] \cdot 1{,}0[-] = 4{,}18\left[\frac{m}{s}\right] \text{ (formula 4.6)}$$

The turbulence intensity: (for $z_{min} < z < z_{max}$):

$$I_{vz} = \frac{k_1}{c_o(z)\cdot\ln\left(\frac{z}{z_0}\right)} = \frac{1{,}0[-]}{1{,}0[-])\cdot\ln\left(\frac{9[m]}{0{,}05[m]}\right)} = 0{,}193[-] \text{ (formula 4.7)}$$

Air density:

$$\rho = 1{,}25\left[\frac{kg}{m^3}\right] \text{ (according E1.5.3. NOTE 1)}$$

The peak velocity pressure:

$$q_p(z) = [1 + 7I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b = [1 + 7 \cdot 0{,}193[-]] \cdot \frac{1}{2} \cdot 1{,}25\left[\frac{kg}{m^3}\right] \cdot \left(21{,}707\left[\frac{m}{s}\right]\right)^2 = 691{,}443[Pa] \text{ (formula 4.8)}$$

The basic velocity pressure:

$$q_b = 0{,}5 \cdot \rho \cdot v^2{}_b = 0{,}5 \cdot 1{,}25\left[\frac{kg}{m^3}\right] \cdot \left(22\left[\frac{m}{s}\right]\right)^2 = 302{,}5[Pa] \text{ (formula 4.10)}$$

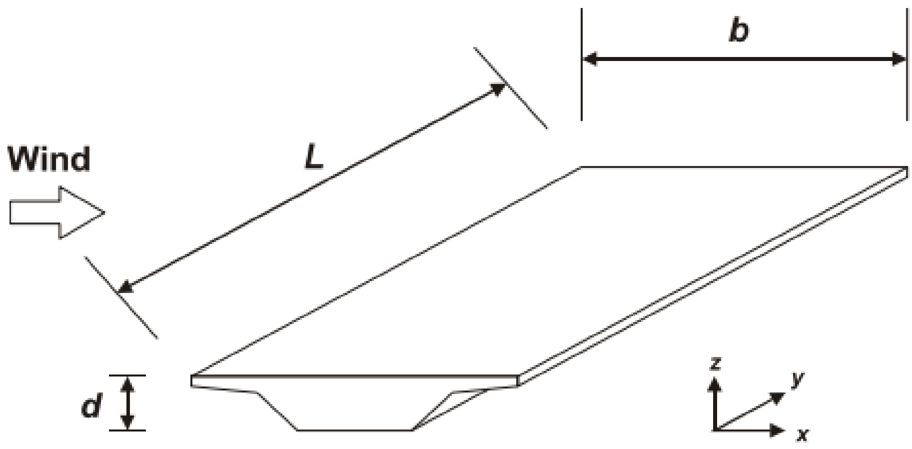
The exposure factor:

$$c_{ez} = \frac{q_p(z)}{q_b} = \frac{691{,}443[Pa]}{302{,}5[Pa]} = 2{,}286[-] \text{ (formula 4.9)}$$

The structural factor:

$c_s = 1{,}0[-]$ (according 8.2. NOTE 2) $c_d = 1{,}0[-]$ (according 8.2. NOTE 2)

Below shows the directions of wind on the structure:



Picture 30. Directions of wind actions on bridges

3.4.1. Direction Y – truss

The force coefficient without free-end flow:

$$c_{fy,0} = 1{,}3[-] \text{ (according 8.3.1 NOTE 2)}$$

Force coefficients for wind actions on bridge decks in the x-direction

$$c_{f,y} = c_{fy,0} = 1{,}3[-] \text{ (formula 8.1)}$$

The wind load factor:

$$c_y = c_{ez} \cdot c_{f,y} = 2{,}286[-] \cdot 1{,}3[-] = 2{,}971[-] \text{ (according 8.3.2)}$$

The reference area:

$$A_{refy} = 30[mm] \cdot 3{,}5[m] \cdot 4 + 40[mm] \cdot 3{,}5[m] \cdot 3 + 20[mm] \cdot 2{,}305[m] \cdot 8 + 14 \cdot 1{,}1[m] \cdot 12[mm] \cdot 7 = 2{,}502[m^2]$$

Force in x-direction:

$$F_{wy} = 0{,}5 \cdot \rho \cdot v^2{}_b \cdot c_x \cdot A_{refx} = 0{,}5 \cdot 1{,}25 \left[\frac{kg}{m^3}\right] \cdot \left(22 \left[\frac{m}{s}\right]\right)^2 \cdot 2{,}971[-] \cdot 2{,}502[m^2] = 2{,}249[kN] \text{ (formula 8.2)}$$

Have been given the following features the load:

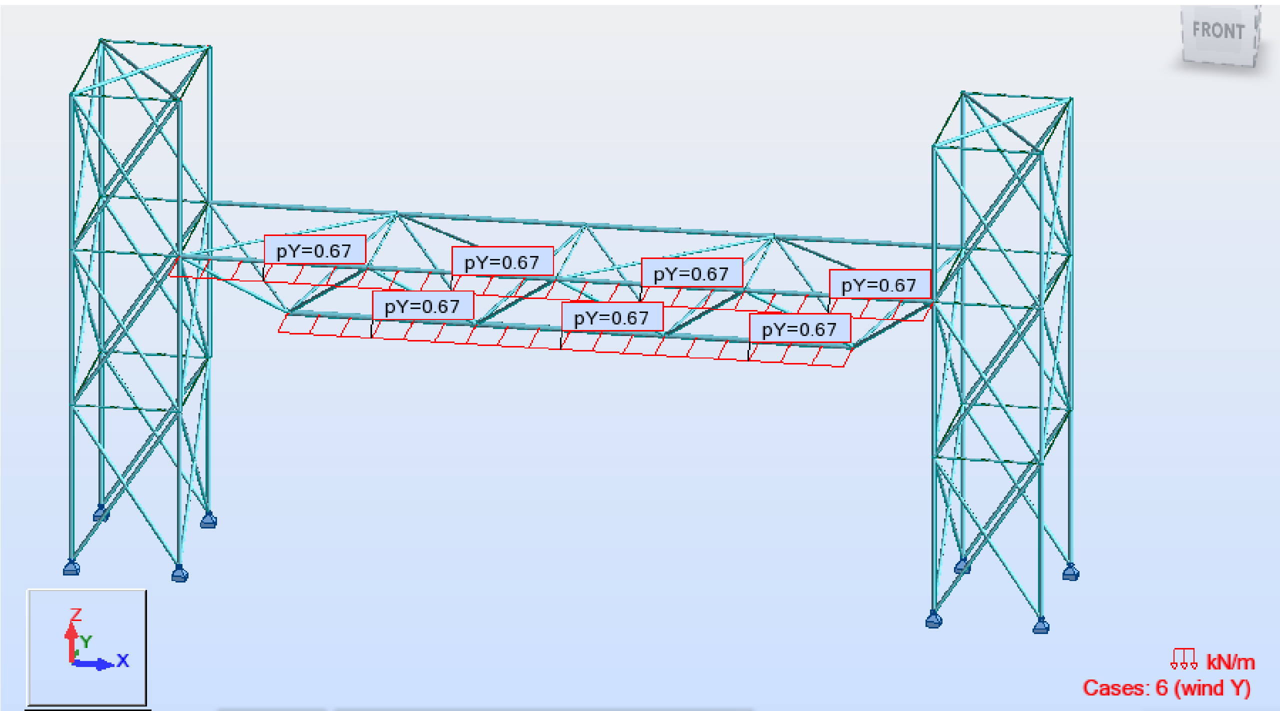
Label: 6: wind Y

Nature: wind

Wind load should be distributed on the surface upper chord of the truss as a linear load a value equal to:

$$Q_{wy} = \frac{F_y \cdot \frac{1{,}5[m]}{2}}{A_{refy}} = \frac{2{,}249[kN] \cdot \frac{1{,}5[m]}{2}}{2{,}502[m^2]} = 0{,}674 \left[\frac{kN}{m}\right]$$

The load specified from a way shown in the figure below:



Picture 31. Screenshot of the program [a]- loads: wind Y

3.4.2. Direction Z- truss

The force coefficient:

$$c_{f,z} = 0{,}9[-] \text{ (according 8.3.3 NOTE 1)}$$

$$c_z = 2{,}286[-] \cdot 0{,}9[-] = 2{,}057[-] \text{ (according 8.3.3)}$$

The reference area:

$$A_{refz} = 1[m^2]$$

Force in z-direction:

$$F_{wz} = 0{,}5 \cdot \rho \cdot v^2{}_b \cdot c_z \cdot A_{refz} = 0{,}5 \cdot 1{,}25 \left[\frac{kg}{m^3}\right] \cdot \left(22 \left[\frac{m}{s}\right]\right)^2 \cdot 2{,}057[-] \cdot 1{,}0[m^2] =$$

$$0{,}622[kN] \text{ (formula 8.2)}$$

Have been given the following features the load:

Label: 8: wind Z

Nature: wind

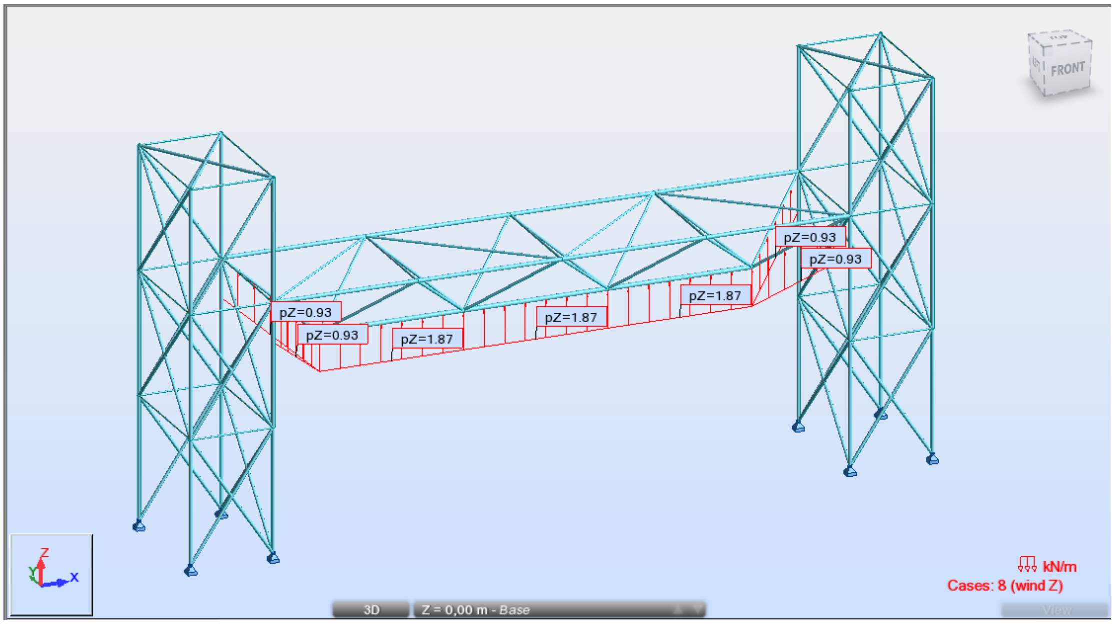
Wind load should be distributed on the surface upper chord of the truss as a linear load a value equal to:

$$Q_{wz} = \frac{F_x \cdot 3[m]}{A_{refz}} = \frac{0{,}622[kN] \cdot 3[m]}{1[m^2]} = 1{,}867 \left[\frac{kN}{m}\right]$$

External element:

$$Q_{wze} = \frac{Q_{wz}}{2} = \frac{1{,}867 \left[\frac{kN}{m}\right]}{2} = 0{,}9335 \left[\frac{kN}{m}\right]$$

The load specified from a way shown in the figure below:



Picture 32. Screenshot of the program [a]- loads: wind Z

3.4.3. Direction Y- elevator (external)

The impact of working in the Y direction, treated as the wind acting on the building (construction elevator is part of the built-up). Specifically for the vertical wall.

Dimension construction elevator: (where b- crosswind direction)

$b_y = 3[m]$ $\qquad$ $h = 9[m]$

The eccentricity of a force or edge distance:

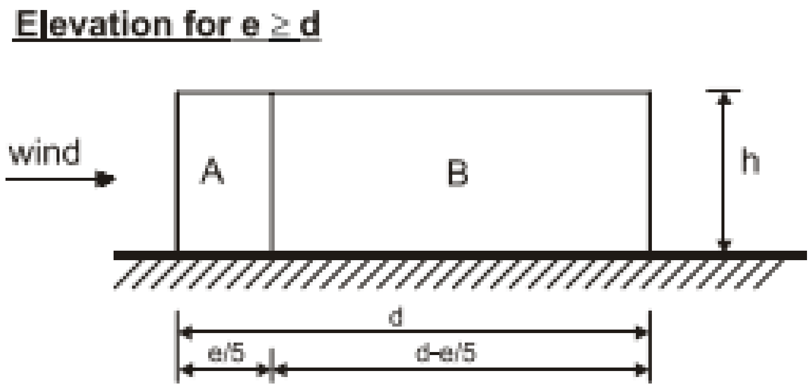
$$e_y = \min(b_y, 2h) = \min(3[m], 18[m]) = 3[m] \text{ (picture 7.5)}$$

Depth of the structure:

$$d_y = 2[m]$$

Because:

$$\frac{h}{d_y} = \frac{9[m]}{2[m]} = 4,5$$



Picture 33. Considered the situation-drawing comp

Zone:

A:

$\frac{e_y}{5} = \frac{3[m]}{5} = 0,6[m]$

B:

$d_y - \frac{e_y}{5} = 2,0[m] - \frac{3[m]}{5} = 1,4[m]$

The values of the external pressure:

$c_{pey\,10A} = -1,2[-]$ (table 7.1) $\qquad$ $c_{pey\,10b} = -0,8$ (table 7.1)

The wind pressure acting on the external surfaces:

$$w_{AeX} = q_{pz} c_{pey\,10A} = 691,443[Pa] \cdot (-1,2[-]) = -0,83 \left[\frac{kN}{m^2}\right] \text{ (formula 5.1)}$$

$$w_{BeX} = q_{pz} c_{pey\,10B} = 691,443[Pa] \cdot (-0,8[-]) = -0,55 \left[\frac{kN}{m^2}\right] \text{ (formula 5.1)}$$

Have been given the following features the load:

Label: $\qquad$ 12: wind Y-lift external

Nature: $\qquad$ wind

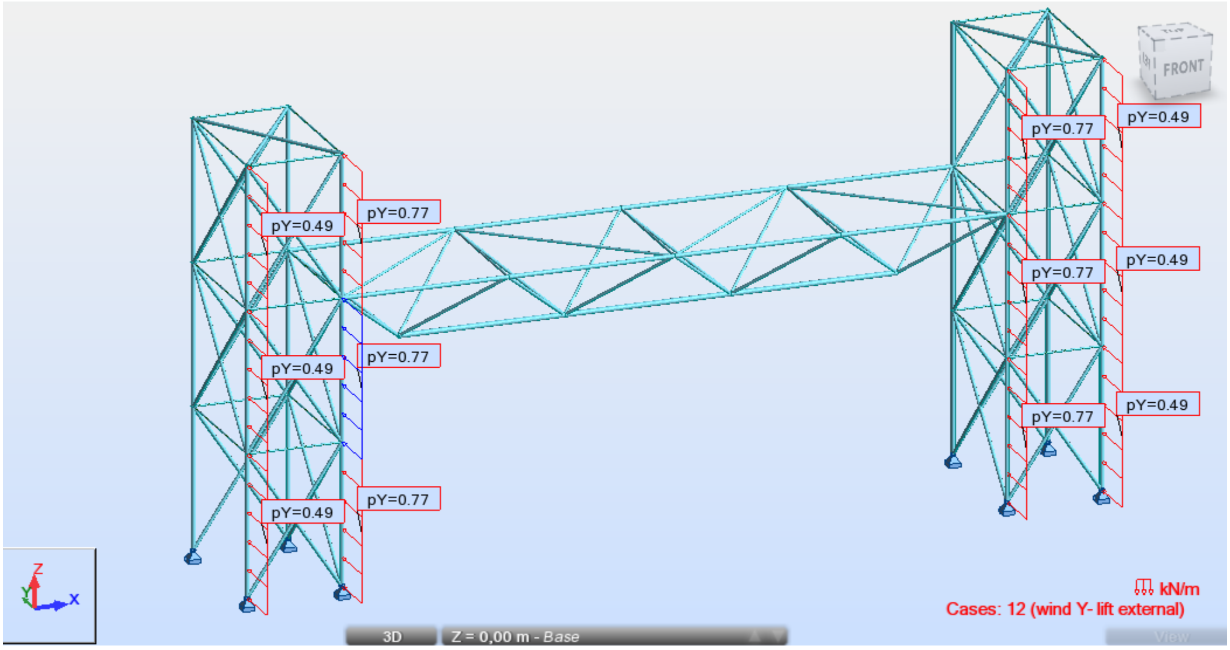
Wind load should be distributed as a linear load a value equal to:

$$Q_{wY} = w_{AeX} \cdot \left(\frac{y}{5}\right) = -0{,}83 \left[\frac{kN}{m^2}\right] \cdot \left(\frac{3[m]}{5}\right) = 0{,}498 \left[\frac{kN}{m}\right]$$

External element:

$$Q_{wY} = w_{Bex} \cdot \left(d_y - \frac{e_y}{5}\right) = -0{,}55 \left[\frac{kN}{m^2}\right] \cdot \left(2{,}0[m] - \frac{3[m]}{5}\right) = 0{,}770 \left[\frac{kN}{m}\right]$$

The load specified from a way shown in the figure below:



Picture 34. Screenshot of the program [a]- loads: wind Y- lift external

3.4.4. Direction Y- elevator (internal)

Internal pressure coefficient:

$c_{piy\,101} = 0{,}2[-]$ $\qquad$ $c_{piy\,102} = -0{,}3[-]$ (according 7.2.9. (6) NOTE 2)

Wind pressure acting on the inter surface of the structure:

$$w_{iX} = q_{pz} c_{piy\,101} = 691{,}443[Pa] \cdot 0{,}2[-] = 0{,}138 \left[\frac{kN}{m^2}\right] \text{ (formula 5.1)}$$

$$w_{iX} = q_{pz} c_{piy\,101} = 691{,}443[Pa] \cdot (-0{,}3[-]) = -0{,}207 \left[\frac{kN}{m^2}\right] \text{ (formula 5.1)}$$

Value assumed as more unfavorable. Internal and external pressures shall be considered to act at the same time. (according 7.2.9. (1))

Have been given the following features the load:

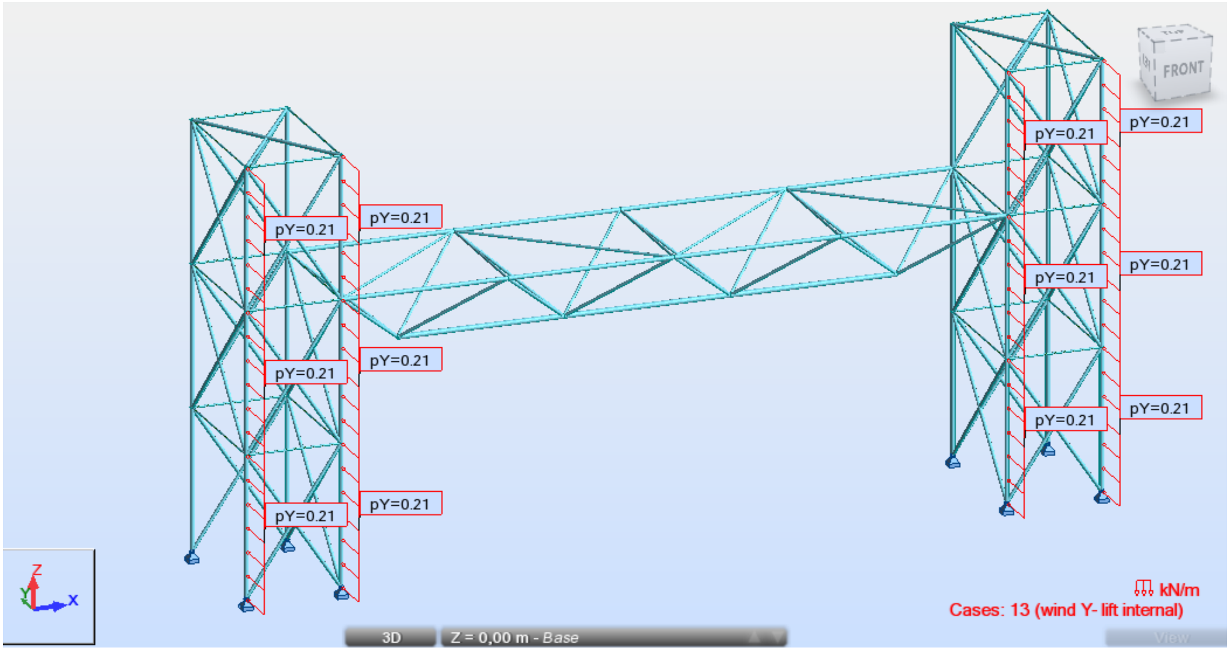
Label: 13: wind Y-lift internal

Nature: wind

Wind load should be distributed as a linear load a value equal to:

$$Q_{wY} = w_{ix} \cdot \left(\frac{2[m]}{2}\right) = -0{,}207 \left[\frac{kN}{m^2}\right] \cdot \left(\frac{2[m]}{2}\right) = -0{,}207 \left[\frac{kN}{m}\right]$$

The load specified from a way shown in the figure below:



Picture 35. Screenshot of the program [a]- loads: wind Y- lift internal

3.4.5. Direction X- elevator (external)

The impact of working in the X direction, treated as the wind acting on the building (construction elevator is part of the built-up). Specifically for the vertical wall.

Dimension construction elevator: (where b- crosswind direction)

$b_x = 2[m]$ $\qquad\qquad$ $h = 9[m]$

The eccentricity of a force or edge distance:

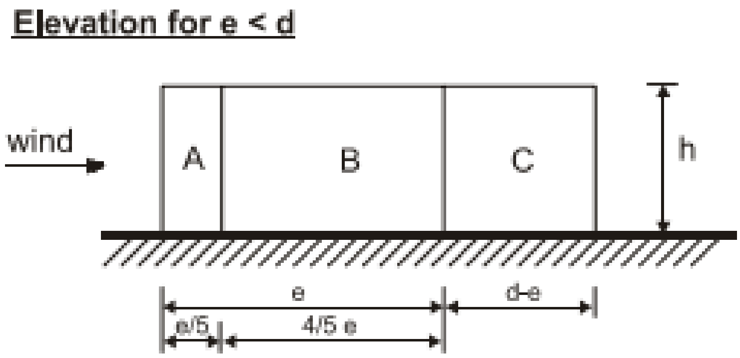
$$e_x = \min(b_x, 2h) = \min(2[m], 18[m]) = 2[m] \text{ (picture 7.5)}$$

Depth of the structure:

$$d_x = 3[m]$$

Because:

$$\frac{h}{d_x} = \frac{9[m]}{3[m]} = 3$$



Picture 36. Considered the situation-drawing comp

Zone:

A: $\frac{e_x}{5} = \frac{3[m]}{5} = 0,6[m]$

B: $4 \cdot \frac{e_x}{5} = 4 \cdot \frac{3[m]}{5} = 1,6[m]$

C: $d_x - e_x = 3[m] - 2[m] = 1[m]$

The values of the external pressure:

$c_{pex\,10A} = -1,2[-]$ (table 7.1)

$c_{pex\,10b} = -0,8$ (table 7.1)

$c_{pex\,10C} = -0,5[-]$ (table 7.1)

The wind pressure acting on the external surfaces:

$$w_{Aey} = q_{pz} c_{pex\,10A} = 691,443[Pa] \cdot (-1,2[-]) = -0,83\left[\frac{kN}{m^2}\right] \text{ (formula 5.1)}$$

$$w_{Bey} = q_{pz} c_{pex\,10B} = 691,443[Pa] \cdot (-0,8[-]) = -0,55\left[\frac{kN}{m^2}\right] \text{ (formula 5.1)}$$

$$w_{Cey} = q_{pz} c_{pex\,10C} = 691,443[Pa] \cdot (-0,5[-]) = -0,35\left[\frac{kN}{m^2}\right] \text{ (formula 5.1)}$$

Have been given the following features the load:

Label: 7: wind X-exter

Nature: wind

Wind load should be distributed as a linear load a value equal to: (length of rod 0- 0,6m):

$$Q_{wx} = w_{AeX} \cdot \left(\frac{3[m]}{2}\right) = -0,83\left[\frac{kN}{m^2}\right] \cdot \left(\frac{3[m]}{2}\right) = -1,245\left[\frac{kN}{m}\right]$$

$$Q_{wx} = w_{AeX} \cdot (3[m]) = -0,83\left[\frac{kN}{m^2}\right] \cdot (3[m]) = -2,490\left[\frac{kN}{m}\right]$$

$$Q_{wx} = w_{AeX} \cdot (4,5[m]) = -0,83\left[\frac{kN}{m^2}\right] \cdot (4,5[m]) = -3,735\left[\frac{kN}{m}\right]$$

Wind load should be distributed as a linear load a value equal to: (length of rod 0,6-2,0m):

$$Q_{wx} = w_{BeX} \cdot \left(\frac{3[m]}{2}\right) = -0,55\left[\frac{kN}{m^2}\right] \cdot \left(\frac{3[m]}{2}\right) = -0,825\left[\frac{kN}{m}\right]$$

$$Q_{wx} = w_{BeX} \cdot (3[m]) = -0,55\left[\frac{kN}{m^2}\right] \cdot (3[m]) = -1,650\left[\frac{kN}{m}\right]$$

$$Q_{wx} = w_{BeX} \cdot (4{,}5[m]) = -0{,}55 \left[\frac{kN}{m^2}\right] \cdot (4{,}5[m]) = -2{,}475 \left[\frac{kN}{m}\right]$$

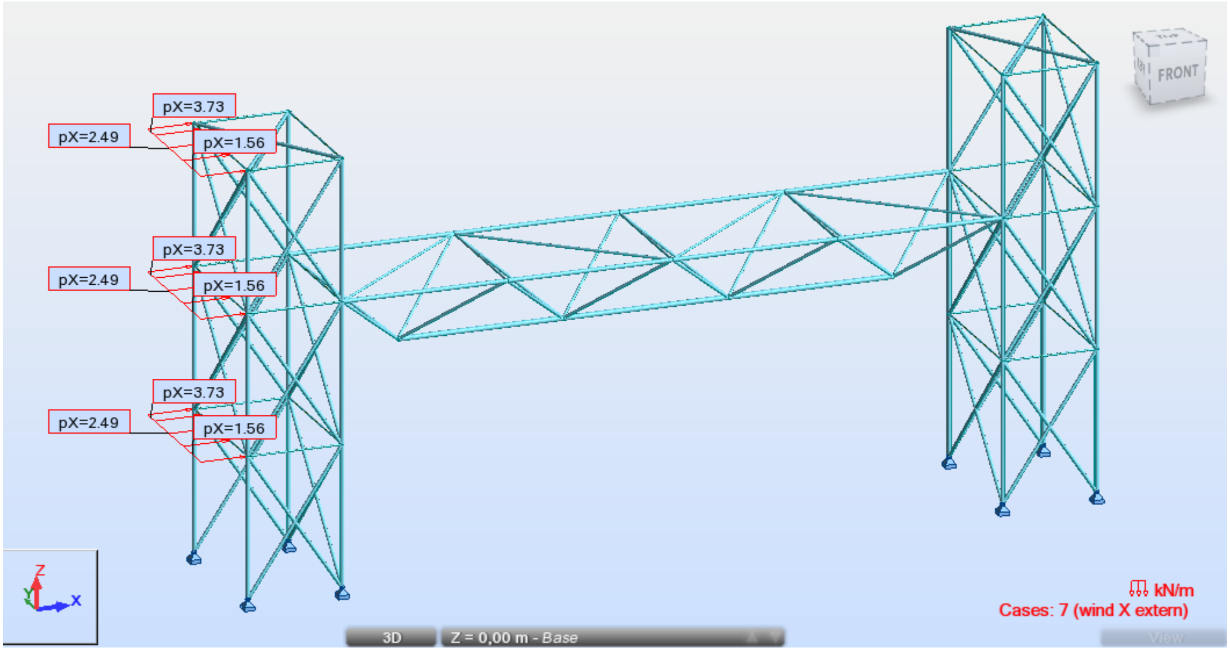
Wind load should be distributed as a linear load a value equal to: (length of rod 2,0-3,0m):

$$Q_{wx} = w_{CeX} \cdot \left(\frac{3[m]}{2}\right) = -0{,}35 \left[\frac{kN}{m^2}\right] \cdot \left(\frac{3[m]}{2}\right) = -0{,}525 \left[\frac{kN}{m}\right]$$

$$Q_{wx} = w_{CeX} \cdot (3[m]) = -0{,}35 \left[\frac{kN}{m^2}\right] \cdot (3[m]) = -1{,}050 \left[\frac{kN}{m}\right]$$

$$Q_{wx} = w_{CeX} \cdot (4{,}5[m]) = -0{,}35 \left[\frac{kN}{m^2}\right] \cdot (4{,}5[m]) = -1{,}575 \left[\frac{kN}{m}\right]$$

The load specified from a way shown in the figure below:



Picture 37. Screenshot of the program [a]- loads: wind X- external

3.4.6. Direction X- elevator (internal)

Internal pressure coefficient:

$c_{pix\,101} = 0{,}2[-]$ (according 7.2.9. (6) NOTE 2)

$c_{pix\,102} = -0{,}3[-]$ (according 7.2.9. (6) NOTE 2)

Wind pressure acting on the inter surface of the structure:

$$w_{iy} = q_{pz} c_{pix\,101} = 691{,}443[Pa] \cdot 0{,}2[-] = 0{,}138 \left[\frac{kN}{m^2}\right] \text{ (formula 5.1)}$$

$$w_{iy} = q_{pz} c_{pix\,101} = 691{,}443[Pa] \cdot (-0{,}3[-]) = -0{,}207 \left[\frac{kN}{m^2}\right] \text{ (formula 5.1)}$$

Value assumed of more unfavorable. Internal and external pressures shall be considered to act at the same time. (according 7.2.9. (1))

Have been given the following features the load:

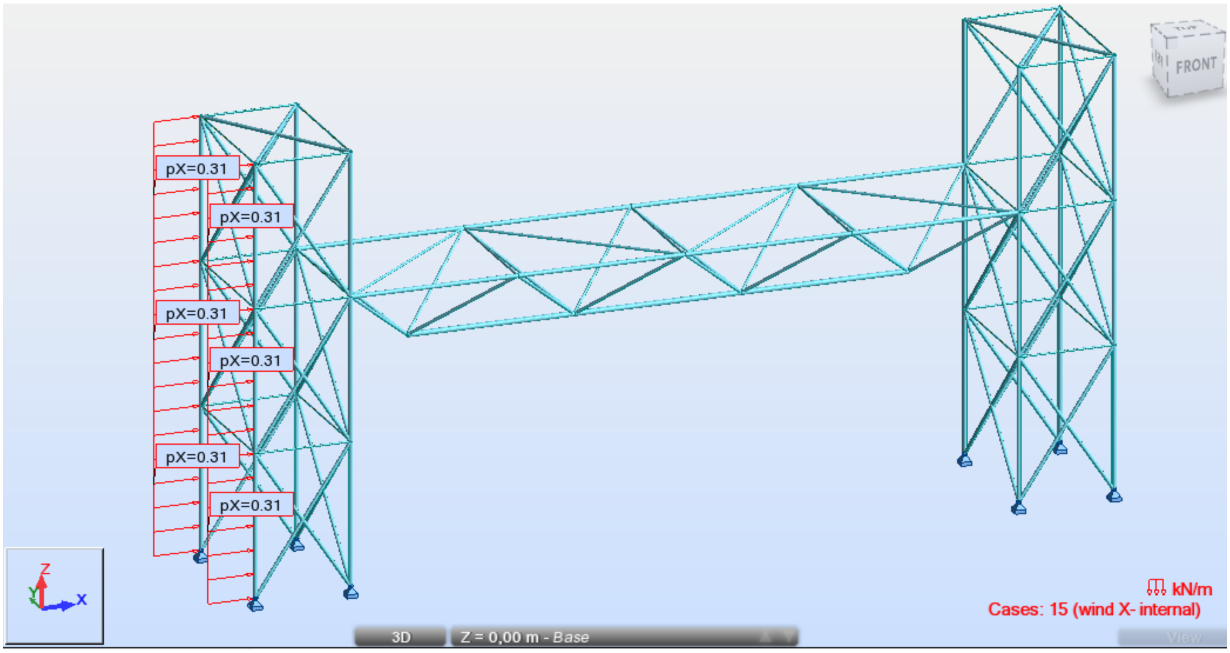
Label: 15: wind X- internal

Nature: wind

Wind load should be distributed as a linear load a value equal to:

$$Q_{wx} = w_{iy} \cdot \left(\frac{3[m]}{2}\right) = -0{,}207 \left[\frac{kN}{m^2}\right] \cdot \left(\frac{3[m]}{2}\right) = -0{,}311 \left[\frac{kN}{m}\right]$$

The load specified from a way shown in the figure below:



Picture 38. Screenshot of the program [a]- loads: wind X- internal

3.5. Exploitation load

Load value was adopted in accordance with EN 1991-2 Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges.

3.5.1. People

For road bridges supporting footways or cycle tracks, a uniformly distributed load is equal to:

$$q_{fk} = 5{,}0 \left[\frac{kN}{m^2}\right] \text{ (according 5.3.2.1.(1) NOTE)}$$

Have been given the following features the load:

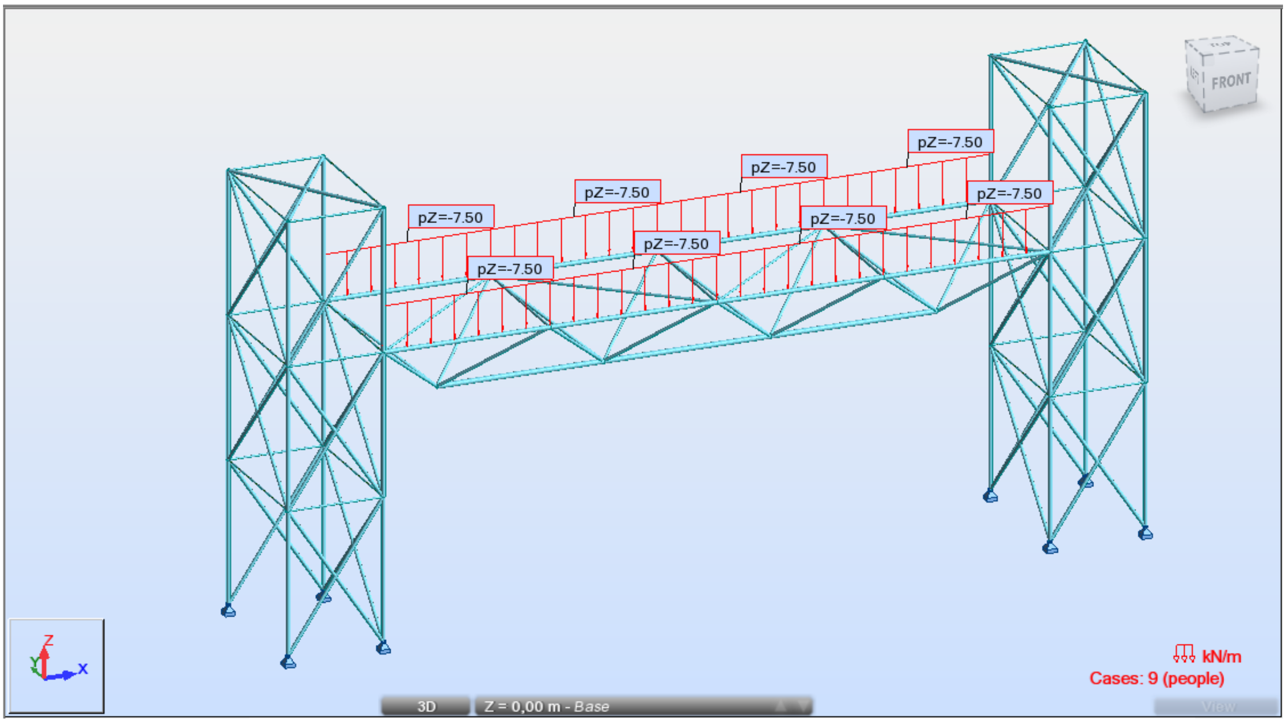
Label: 9: people

Nature: exploitation

Load should be distributed on the surface upper chord of the truss as a linear load a value equal to:

$$Q_{pep} = q_{fk} \cdot \frac{3[m]}{2} = 5{,}0\left[\frac{kN}{m^2}\right] \cdot \left(\frac{3[m]}{2}\right) = 7{,}5\left[\frac{kN}{m}\right]$$

The load specified from a way shown in the figure below:



Picture 39. Screenshot of the program [a]- loads: people

3.5.2. Concentrated load

The characteristic value of the concentrated load is equal:

$$Q_{fwk} = 10{,}0[kN] \text{ (according 5.3.2.2.(1))}$$

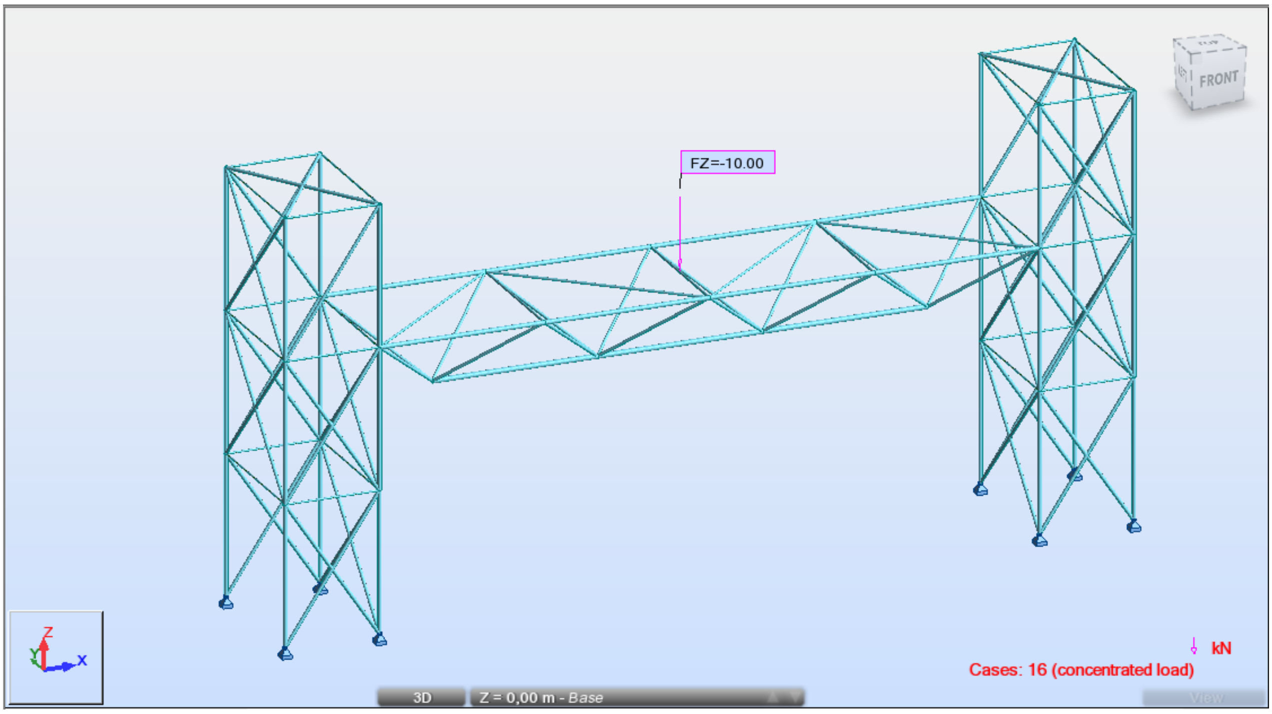
Have been given the following features the load:

Label: 16: concentrated load

Nature: exploitation

The characteristic value of the concentrated load Q_{fwk} should be taken equal to 10kN acting on a square surface of sides 0,10 m.

The load specified from a way shown in the figure below:



Picture 40. Screenshot of the program [a]- loads: concentrated load

3.5.3. Horizontal forces

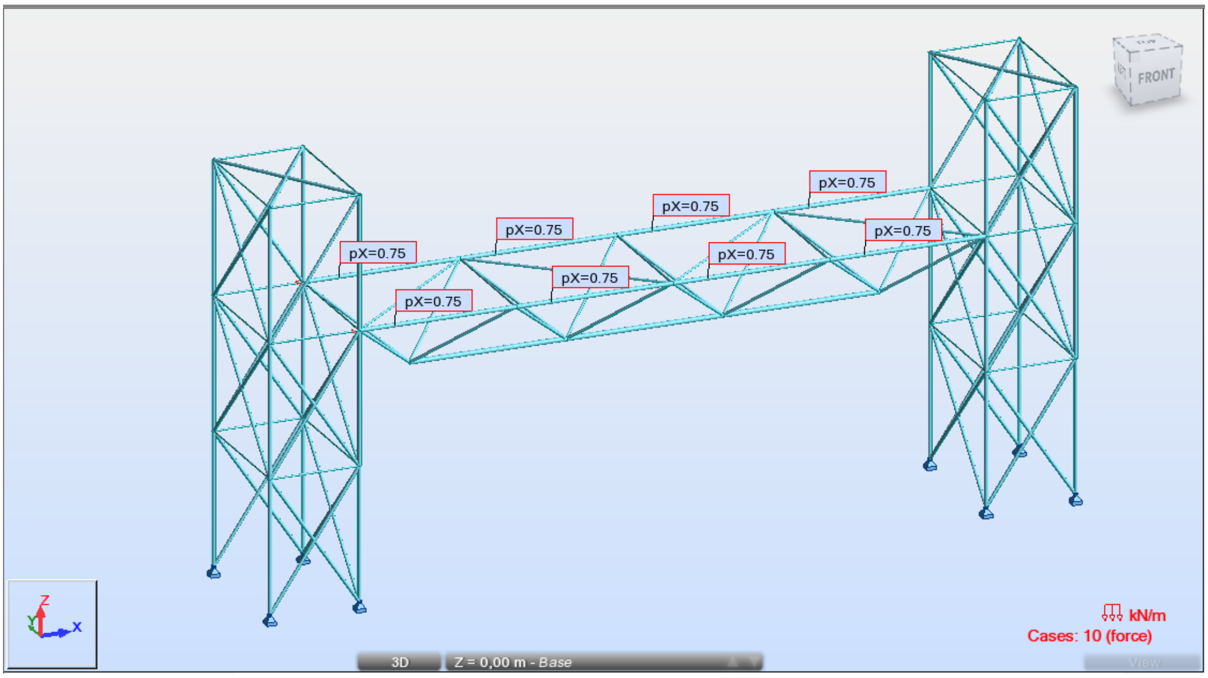
Horizontal force is equal to 10 percent of the total load corresponding to the uniformly distributed load:

$$Q_{lfk} = 0{,}1 \cdot q_{fk} \cdot 3[m] = 0{,}1 \cdot 5\left[\frac{kN}{m^2}\right] \cdot 3[m] = 1{,}5\left[\frac{kN}{m}\right] \text{ (according 5.4(2))}$$

Have been given the following features the load:

Label:	10: force
Nature:	exploitation

The load specified from a way shown in the figure below:



Picture 41. Screenshot of the program [a]- loads: horizontal force

Never work with the load centered Q_{fwk}

3.5.4. Force of the impact of road vehicles under the bridge

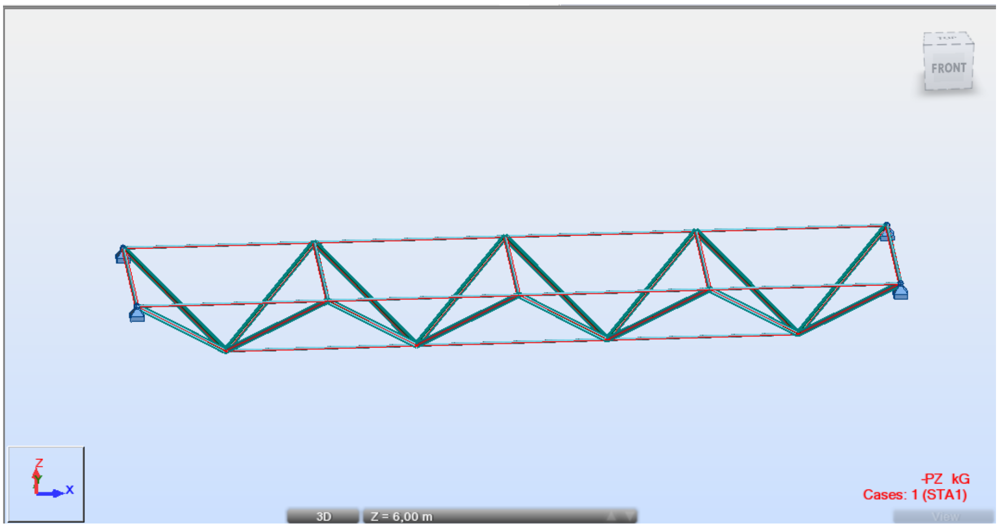
Pillars of the footbridge will be protected from impact of vehicles traveling under the bridge using appropriate barriers.

3.5.5. Vibrations

After entering the data into the program: loads and combinations (Chapter 4.8. of this work) began to analyze the vibration of the structure.

Due to the high complexity of the structure, during this part of the calculation modeled only the carrier element structure exposed to vibration-truss.

The structure looks as shown below:



Picture 42. Screenshot of the program [a]- vibration- analysis model

The program [a] is possible to analyze the vibrations only for load cases (but not with their combination).

When performing calculations in the modal analysis module matrix created for program [a] is not positive definite - the critical load may have been exceeded. The results are presented in the table attached to this work as Annex 3.

In Annex 3, N/A means that matrix created by the program, necessary to measure the frequency value is not positive definite (further calculations for these cases are not executed by the program [a]) - the critical load may have been exceeded for this case (so the worst case is temperature cooling, temperature heating and temperature difference components). These cases highlighted in orange in the table above.

In some cases, exceeded authorized and regulated frequency. These cases highlighted in red in the table above (almost all cases without wind in direction Z, force and extensional snow). The worst cases is load by people- frequency is equal 2,25[Hz]. In this case footbridge could fall into vibrations.

According 5.7.(2) (NOTE): Effects of pedestrian traffic on a footbridge depend on various factors as, for example, the number and location of people likely to be simultaneously on the bridge, and also on external circumstances, more or less linked to the location of the bridge. In the absence of significant response of the bridge, a pedestrian normally walking exerts on it the following simultaneous periodic forces:

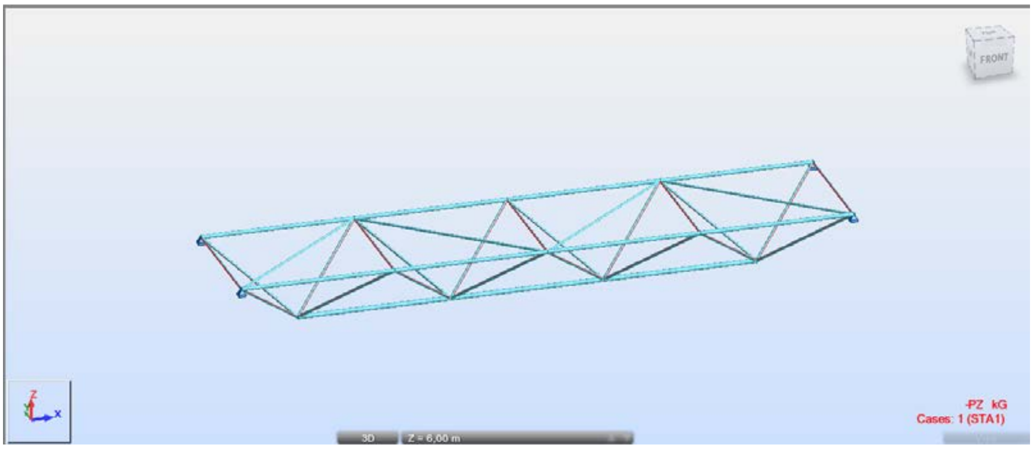
- in the vertical direction, with a frequency range of between 1 and 3 Hz, and
- in the horizontal direction, with a frequency range of between 0,5 and 1,5 Hz.

- Groups of joggers may cross a footbridge with a frequency of 3 Hz.

Accordingly, the record of the frequency of the bridge is undesirable in the range of 1-3 Hz (vertical direction) and 0.5-1.5 (horizontal direction).

The proposed design conditions are not met. Vibration can cause resonance and the destruction of the structure.

With the above-mentioned reasons, it was decided to add to the static structure of additional structural bars to prevent vibrations. Model looks like this:



Picture 43. Screenshot of the program [a]- vibration- new analysis model

An analysis of the modal structure. The results are presented in the table attached to this work as Annex 4.

After adding additional bars vibration structures are not dangerous. The program is able to calculate the frequency for each of the load cases and the normalized frequency is not exceeded. In addition to this case, now we keep a large supply capacity.

Additional structural modeling bars allowed protecting the structure against damage caused by resonance. Additional rods ensure safety of the construction in this area.

3.6. Ice

According to EN 1993-3-1 load icing and the standard ISO 12494—2005 load of ice should not be considered on this footbridge.

3.7. Temperature

Load value was adopted in accordance with EN 1991-1-5 Eurocode 1: Actions on structures - Part 1-5: General actions – Thermal actions.

The initial temperature when structural element is restrained:

$$T_0 = 8^o C \text{ (according NB 1.1)}$$

Coefficients for calculation of maximum (minimum) shade air temperature with an annual probability of being exceeded, p, other than 0,02:

$k_1 = 0{,}818[-]$ $k_2 = 0{,}047[-]$

$k_3 = 0{,}502[-]$ $k_4 = -0{,}128[-]$ (according NB 1.2)

The maximum shade air temperature with an annual probability of being exceeded of 0,02 (equivalent to a mean return period of 50 years):

$$T_{max} = 40[^oC] \text{ (picture NB 2)}$$

The level location of the object:

$$H_{nmp} = 220{,}15[m]$$

The maximum temperature at the level location of the object:

$$T_{max}(H) = -0{,}0053 \cdot \frac{1^oC}{m} \cdot H + T_{max} = -0{,}0053 \cdot \frac{1^oC}{m} \cdot 220{,}15[m] + 40[^oC] = 38{,}833[^oC]$$

(according NB 1.3)

The minimum shade air temperature with an annual probability of being exceeded of 0,02 (equivalent to a mean return period of 50 years):

$$T_{min} = -30[^oC] \text{ (picture NB.3)}$$

The minimum temperature at the level location of the object:

$$T_{max}(H) = -0{,}0035 \cdot \frac{1^oC}{m} \cdot H + T_{min} = -0{,}0035 \cdot \frac{1^oC}{m} \cdot 220{,}15[m] + (-30[^oC]) =$$

$$-30{,}771[^oC] \text{ (according NB 1.3)}$$

The considered structure is a steel bridge, classified as type I.

Further calculations were made according to Chapter 4.2. - changes in temperature in bridges.

3.7.1. Uniform temperature component

The minimum uniform bridge temperature component:

$$T_{e,min} = -42[^oC] \text{ (picture 6.1)}$$

The maximum uniform bridge temperature component:

$$T_{e,max} = 56[^oC] \text{ (picture 6.1)}$$

According 6.1.3.1.(4) NOTE2 for steel truss the maximum values given for type 1 may be reduced by 3°C, so:

$T_{e,min} = -39[^oC]$ $T_{e,max} = 53[^oC]$

The maximum contraction range of uniform bridge temperature component:

$$\Delta T_{Ncon} = T_0 - T_{e,min} = 8[^{oC}] - (-39[^oC]) = 47[^oC] \text{ (formula 6.1)}$$

Have been given the following features the load:

Label: 18: temperature 2- cooling

Nature: temperature

Load a melting has been asked to design bars.

The maximum expansion range of uniform bridge temperature component:

$$\Delta T_{Nexp} = T_0 - T_{e,max} = 8[^oC] - 53[^oC] = 45[^oC] \text{ (formula 6.2)}$$

Have been given the following features the load:

Label: 17: temperature 1- heating

Nature: temperature

Load a melting has been asked to design bars.

The overall range of the uniform bridge temperature component is:

$$\Delta T_N = T_{emax} - T_{e,min} = 53[^{oC}] - (-39[^oC]) = 92[^oC] \text{ (according 6.1.3.4.(3) NOTE1)}$$

3.7.2. Temperature difference components

Over a prescribed time period heating and cooling of a bridge deck's upper surface will result in a maximum heating (top surface warmer) and a maximum cooling (bottom surface warmer) temperature variation.

Vertical linear component (Approach 1) :

The linear temperature difference component (heating):

$$\Delta T_{Mheat} = 18[^oC] \text{ (table 6.1)}$$

The linear temperature difference component (cooling):

$$\Delta T_{Mcool} = 13[^oC] \text{ (table 6.1)}$$

Vertical linear component:

$$\Delta T_M = \Delta T_{Mheat} - \Delta T_{Mcool} = 18[^oC] - 13[^oC] = 5[^oC]$$

Force acts in the vertical direction. Assumed the recommended value of the horizontal temperature difference between the outside edges of the bridge equal to 5 degrees Celsius.

Have been given the following features the load:

Label: 14: temp

Nature: temperature

Load a melting has been asked to design bars.

3.8. Combination

The combinations specified by standards were asked as automatic in program [a] according to EN1990: 2002. Defined simplified automatic combination.

When the combination are determining program [a] remember their mutual dependencies. Introduced the following assumptions when creating a combination of:

- Snow in accidental design situation and snow in normal situation always mutually exclusive;
- Horizontal force and concentrated load always mutually exclusive;
- Wind in direction X, Y and Z always mutually exclusive;
- Self weight (self weight of structural steelwork, balustrade, surface and stairs) always act together;
- Wind in the X direction acting on the grid and an elevator (both internal and external) always together;
- Wind in the Y direction acting on the elevator (both internal and external) always together;

Tables with the combinations of loads is presented in as Annex 5 attached to this work.

The program [a] defined combinations of both ULS and SLS. In the second column are presented accepted load factors and load combinations taken into account (in the following way: the number of load x load factor).

Parameters of the coefficients are consistent with those in the PN-EN 1990.

4. Modeling

Static calculations will be performed in Autodesk Robot Structural Analysis Professional 2011. Program [a] using the finite element method in the static calculations of the structure.

4.1.1. Finite-element method

Modeling the structure was decided the use of the finite-element method. Finite Element Method (FEM) is a mathematical tool increasingly used for engineering calculations that lets converted a set of differential equations describing the behavior or characteristics of a continuous medium on the system of nonlinear equations. [25]

4.1.2. Static model

Before performing work program [a] design standards established in accordance with point 1.8. this work. A defined location of the footbridge (due to inter alia climatic factors) as Lodz, Poland. Cross-sections of steel bars will be determined in accordance with the Catalogue Polish Steel Profiles 2007 (due to the location of an object, they are the easiest to obtain).

Construction of the proposed footbridge was modeled with a single bar elements made of steel S235. Individual elements of the bar are connected in the respective nodes.

Round cross sections of the bars have their own high rigidity. As a result, even small cross sections are able to move to very large internal forces. Thus, by using tubular sections reduce the consumption of steel. Thereby reduce the cost of construction, transport and installation of components become easier. Aesthetic appearance is really important.

Closed circular cross-sectional shape, and their symmetry provides advantageous mechanical properties: for instance not being subjected to lateral-torsional buckling.

Round tubular elements have a very favorable aerodynamic shape, which makes the wind flows around the sections, and its effect on the structure is not as large as in the case of other shapes structural elements.

No sharp edge of the section provides a very easy applying impregnate protective system against corrosion.

Was decided to apply rod elements made of CHS (Circular Hollow Sections) due to the following factors:

- the aesthetic factors,
- the high stiffness,
- the tubular elements not subject to lateral-torsional buckling according to EN 1993,
- the less steel consumption (sections are hollow),
- the favorable aerodynamic shape (especially due to the wind),
- regular shape allows the precise and easy to perform corrosion protection,
- easy assembly and transportation.

Thus was decided to cope with the following disadvantages of cross section:

- price,
- difficulty to performance of connection.

The connection of foundation was modeled as hinge support with blocked moved toward UX, UY and UZ. The base are transferred the internal forces of the column without causing significant moments that may adversely affect the structure or the elements as a whole. It also has sufficient capacity to for trading under calculation load.

The characteristics of the bar elements are defined by the user as follows:

- buckling

Buckling coefficient equal to 1.0 was adopted in the direction Y, and Z. This value is equal to the coefficient of buckling for pinned member (according to the geometry and characteristics of the project). Buckling curve according to EN 1993-1 has been defined as a curve c for cold formed elements;

- program during the calculation takes into account the impact of flexural-torsional buckling;
- program during the calculation does not account for lateral buckling (the tubular elements not subject to lateral buckling according to EN 1993);

4.1.3. Groups of bars

For reasons of constructional elements are divided into 9 groups of bars having the same cross sections, and parameters.

The following table summarizes the main features of bars depending on the group:

The group number	Name of the group	Element numbers	The cross-section [mm]	Designation on the drawings
1.	The upper chard truss	23-32	$\emptyset 88,9x6,3$	1
2.	The lower chard truss	38-40	$\emptyset 108,0x8,0$	2
3.	The truss post	34-36	$\emptyset 51,0x4,0$	3
4.	The extreme post of truss	23,37	$\emptyset 54,0x4,0$	4
5.	The middle counterbrace of truss	43-54	$\emptyset 60,3x5,6$	5
6.	The extreme counterbrace of truss	41-42,55-56	$\emptyset 60,3x5,6$	6
7.	Elevator- post	1-6, 11-13, 17-19, 126-131, 135-137, 141-143	$\emptyset 88,9x5,0$	7
8.	Elevator- horizontal bar	7, 14-16, 20-24, 37, 132-134, 138-140, 144-147, 238, 240-243	$\emptyset 38,0x4,0$	8.1, 8.2
9.	Elevator- counterbrace	57-61, 63-72, 75-80, 148-169	$\emptyset 70,0x5,0$	9.1, 9.2, 9.3, 9.4

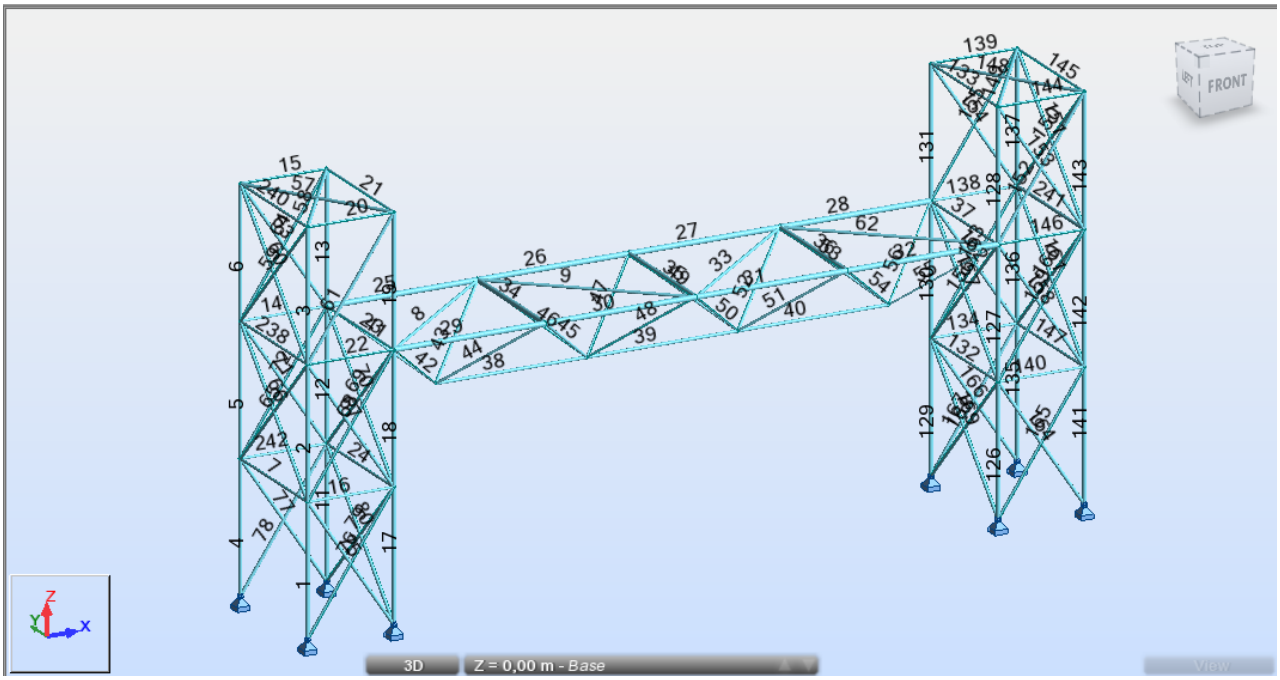
4.2. Analysis

For structural reasons described in Section 3.5.5. this paper changed the static model by adding an additional group bars. Below are the current appearance of constructing:.

The following table summarizes the main features of bars depending on the group:

The group number	Name of the group	Element numbers	The cross-section [mm]	Designation on the drawings
1.	The upper chard truss	23-32	$\emptyset 88,9x6,3$	1
2.	The lower chard truss	38-40	$\emptyset 108,0x8,0$	2
3.	The truss post	34-36	$\emptyset 51,0x4,0$	3
4.	The extreme post of truss	23,37	$\emptyset 54,0x4,0$	4
5.	The middle counterbrace of truss	43-54	$\emptyset 60,3x5,6$	5
6.	The extreme counterbrace of truss	41-42,55-56	$\emptyset 60,3x5,6$	6
7.	Elevator- post	1-6, 11-13, 17-19, 126-131, 135-137, 141-143	$\emptyset 88,9x5,0$	7
8.	Elevator- horizontal bar	7, 14-16, 20-24, 37, 132-134, 138-140, 144-147, 238, 240-243	$\emptyset 38,0x4,0$	8.1, 8.2
9.	Elevator- counterbrace	57-61, 63-72, 75-80, 148-169	$\emptyset 70,0x5,0$	9.1, 9.2, 9.3, 9.4
10.	Vibration	8-9, 33, 62	$\emptyset 51,0x4,0$	10

Below is a pictorial drawing of construction along with the numbers of bars:



Picture 44. Screenshot of the program [a]- element numbers

Calculations were performed for analysis: static-linear.

Annex 6 attached to this paper presents the table with the values of the internal forces of the bars, as the case load.

Due to the very large number of combinations and elements in the table decided to post only the most unfavorable values of normal force. This value of this force is crucial. Other intangible forces: shear force and moments are negligibly small.

5. Design

5.1. Elements

In this chapter, all appeal relate to EN 1993-1: Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings.

Due to the very small value of cutting forces and moments their impact are omitted. Decisive influence on the dimensioning of the structure is the normal force.

The following is a course dimensioning bars for each of the reference group, the calculations were made for the element with the greatest normal force.

Other bars and bars of the group should be identical, we will check the bar of calculation the worst situation.

Value obtained from the internal forces of the program [a], which are presented in the Annex 6. Takes into account the most powerful force for code combinations.

The table below presents for each group of bars, the largest value of the normal force:

Group number	Bar number	Combination	Value of normal force [kN]
1	26	56	$108{,}02$
2	39	65	$-196{,}67$
3	35	65	$-22{,}83$
4	23	37	$25{,}06$
5	54	63	$57{,}22$
6	55	63	$-59{,}12$
7	11	48	$142{,}72[kN]$
8	23	52	$4{,}06[kN]$
9	166	102	$60{,}10[kN]$
10	8	121	$-8{,}93$

5.1.1. Data

Basic data used in the calculation:

The yield strength:

$$f_y = 235 \left[\frac{N}{mm^2}\right] \text{ (table 3.1)}$$

The ultimate strength:

$$f_u = 360 \left[\frac{N}{mm^2}\right] \text{ (table 3.1)}$$

The partial factor for resistance of cross-sections whatever the class:

$$\gamma_{M0} = 1{,}0[-] \text{ (according 6.1.(1) NOTE 2)}$$

The pal1ial factor for resistance of members to instability assessed by member checks:

$$\gamma_{M1} = 1{,}0[-] \text{ (according 6.1.(1) NOTE 2)}$$

The modulus of elasticity:

$$E = 210000{,}0 \left[\frac{N}{mm^2}\right] \text{ (according 3.2.6(1))}$$

5.1.2. The element with the greatest strength of normal force - Group II

The following is a calculation procedure is necessary to dimension the element. Calculations were made for the item with the highest normal force.

The calculations were performed for other groups of elements have been incorporated into this study as Annex 1.

The largest force occurs in the normal rod of group II, which is characterized by the following parameters:

Number most strenuous rod:	39
Number combinations:	65
Design value of the normal force:	$N_{Ed} = -196,67[kN]$
Rod length:	$L_{cr} = 3,5[m]$
Adopted cross-section:	$\emptyset 108x8$
	d=108,0[mm]
	t=8,0[mm]

Cross-sectional area:

$$A = \pi(0,5d)^2 - \pi(0,5d-t)^2 = \pi(0,5 \cdot 108[mm])^2 - \pi(0,5 \cdot 108[mm] - 8[mm])^2 = 25,13[cm^2]$$

Second moment of area:

$$I = \pi \cdot \frac{d^4-(d-2t)^4}{64} = \pi \cdot \frac{(108[mm])^4-(108[mm]-2\cdot 8[mm])^4}{64} = 316,17[cm^4]$$

Cross-section requirements for plastic global analysis:

Factor:

$$\varepsilon_s = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{f_y}} = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{235\left[\frac{N}{mm^2}\right]}} = 1[-] \text{ (table 5.2)}$$

The ratio of cross-sectional diameter to its thickness:

$$\frac{d}{t} = \frac{108[mm]}{8[mm]} = 13,5[-] \text{ (table 5.2)}$$

Section in bending and/or compression:

$$\frac{d}{t} = 13,5[-] \leq 50\varepsilon_s^2 = 50 \cdot 1[-]^2 = 50[-] \text{ (table 5.2)}$$

So the section is a Class 1 (table 5.2).

If axial compression section is class 1, the in bending and compression is also a Class 1.

The design resistance of the bar in compression

The design resistance to normal forces of the cross-section for uniform compression (for class 1 cross-sections):

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{25,13[cm^2]\cdot 235\left[\frac{N}{mm^2}\right]}{1,0[-]} = 590,619[kN] \text{ (formula 6.10)}$$

Condition section capacity in compression:

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{196{,}97[kN]}{590{,}619[kN]} = 0{,}33 < 1{,}0 \text{ (formula 6.9)}$$

Condition fulfilled.

Buckling resistance of members

Buckling curves:

c (table 6.2)

Imperfection factors for buckling curves:

$$\propto = 0{,}49[-] \text{ (table 6.1)}$$

The elastic critical force for the relevant buckling mode based on the gross cross sectional properties:

$$N_{cr} = \pi^2 \cdot E \cdot \frac{I}{L_{cr}^2} = \pi^2 \cdot 210000 \left[\frac{N}{mm^2}\right] \cdot \frac{316{,}17[cm^4]}{(3{,}5[m])^2} = 534{,}938[kN]$$

The parameter of appropriate non-dimensional slenderness:

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{25{,}13[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{534{,}938[kN]}} = 1{,}051[-] \text{ (according 6.3.1.2 (1))}$$

$$\Phi = 0{,}5\left[1 + \propto \left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] = 0{,}5[1 + 0{,}49[-]\ (1{,}051[-] - 0{,}2) + (1{,}051[-])^2] = 1{,}260[-] \text{ (according 6.3.1.2 (1))}$$

The reduction factor for relevant buckling mode :

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1{,}260[-] + \sqrt{(1{,}260[-])^2 - (1{,}051[-])^2}} = 0{,}511[-] \le 1{,}0[-] \text{ (formula 6.49)}$$

The design buckling resistance of a compression member:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0{,}511[-] \cdot 25{,}13[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 301{,}845[kN] \text{ (formula 6.47)}$$

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{196{,}97[kN]]}{301{,}845[kN]} = 0{,}652[-] \le 1{,}0[-] \text{ (formula 6.46)}$$

Condition fulfilled.

Cross-section chosen properly.

5.2. Connections

All references in this section refer to EN 1993-1-8: Eurocode 3: Design of steel structures - Part 1-8: Design of joints.

5.2.1. The connection in the node 28

Theoretical base

Today there are many different types of connections used in metal structures can be distinguished:

- pin-type connection (rivets, ordinary screws, screws medium machining, pins),
- slip-resistant connections (compressed with high strength screws),
- butt joints (compressed with high strength screws),
- thermal connection (welded),
- non-structural connections (pins inserted, self-tapping screws, blind rivets binding, bonding). [26]

In this document designed welded connections, bolted connections (category A in accordance with EN 1993-1-8) and the nodes that connection a large number of bars used in the alternative steel plates.

Welded connections

Welded joints to be made of a combination of bars by welding.

Welding is the joining of metals, comprising bringing heat to the joint flanks causing the transition to liquid state and the interconnection in this condition. When cooled down, creates a uniform interface.

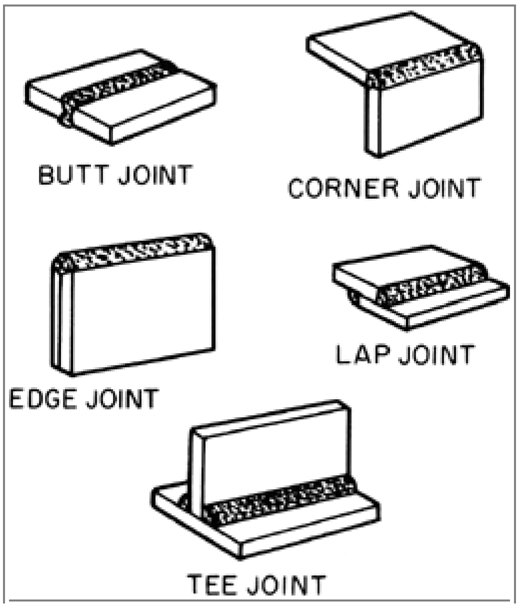
Advantages of welded joints:

- The ease and speed of execution,
- Simple design: no additional elements, low weight,
- The possibility of full automation,

Disadvantages of welded joints:

- Additional stress and strain,
- Problems with welding some materials
- Necessary qualified staff,
- Need specialized equipment.

There are different connections of welded structures: butt joints, T-joints, lap joints, and corner joints, like on the picture:



Picture 45. Types of welded joint [26]

The material has mechanical properties of the weld (yield strength and tensile strength, elongation at break) at least the same as the base materials.

The joints are designed according to the simple method. Way to design according to this method was presented in part calculation of this subsection. [26], [27]

Bolted connection

Bolted joints is a fastening method using a threaded pin or rod with a head at one end (bolt), designed to be inserted through holes in assembled parts and secured by a mated nut, that is tightened by applying torque.

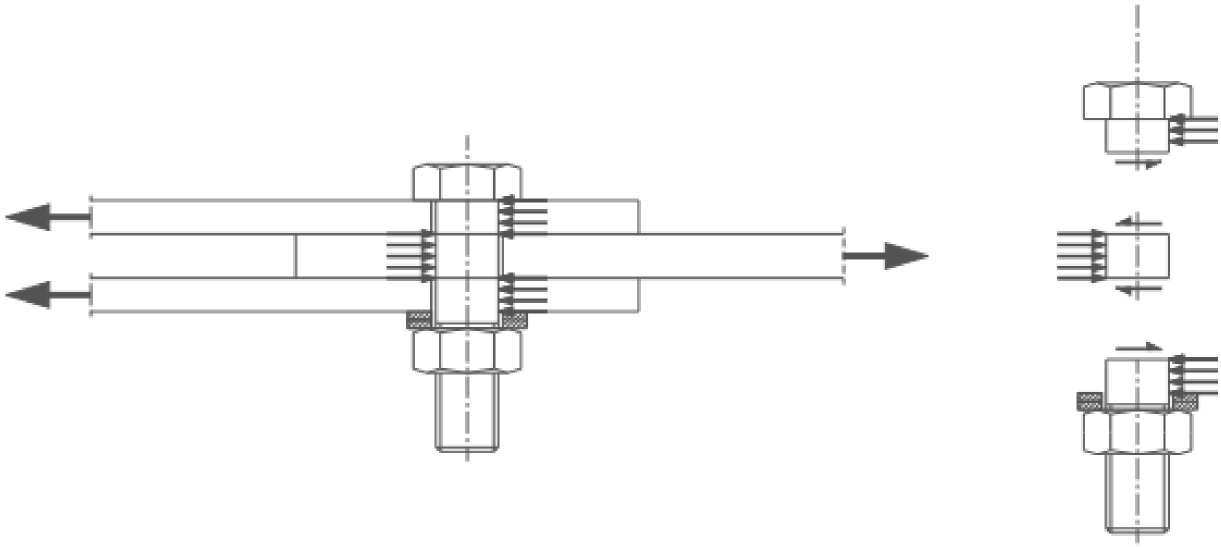
There are different types of bolted connections. They can be categorized based on the type of loading:

- Tension member connection and splice (the bolts to forces that tend to shear the shank),
- Beam end simple connection (the bolts to forces that tend to shear the shank),
- Hanger connection (the hanger connection puts the bolts in tension);

This study will use the connection category A according to EN 1993-1-8.

In category A: Bearing type category bolts from class 4.6 up to and including class 10.9 should be used. No preloading and special provisions for contact surfaces are required. The design ultimate shear load should not exceed the design shear resistance, nor the design bearing resistance.

The following is a connection category A:



Picture 46. Bolted connection: category A[26]

In category A: Bearing type, we should check:

- Shear resistance,
- Bearing resistance. [26], [27]

Way to design connections are shown in Annex 7.

Data

Basic data used in the calculation:

The partial safe factor of resistance to joints in hollow section lattice girder:

$$\gamma_{M5} = 1{,}0[-] \text{ (table 2.1)}$$

Partial safety factors for joints:

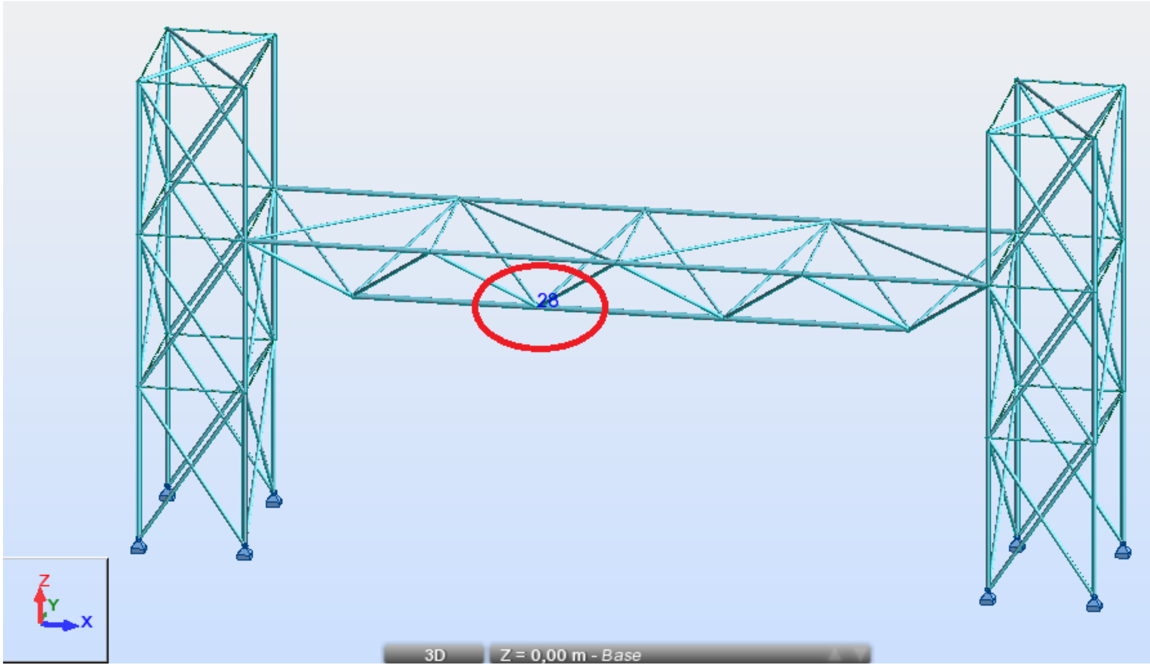
$$\gamma_{M2} = 1{,}25[-] \text{ (according 2.2.(2) NOTE)}$$

Correlation factor for fillet welds :

$$\beta_w = 0{,}8[-] \text{ (table 4.1)}$$

The connection in the node 28

The following is a schematic the calculations made when designing the node number 28, which was presented in the figure below:

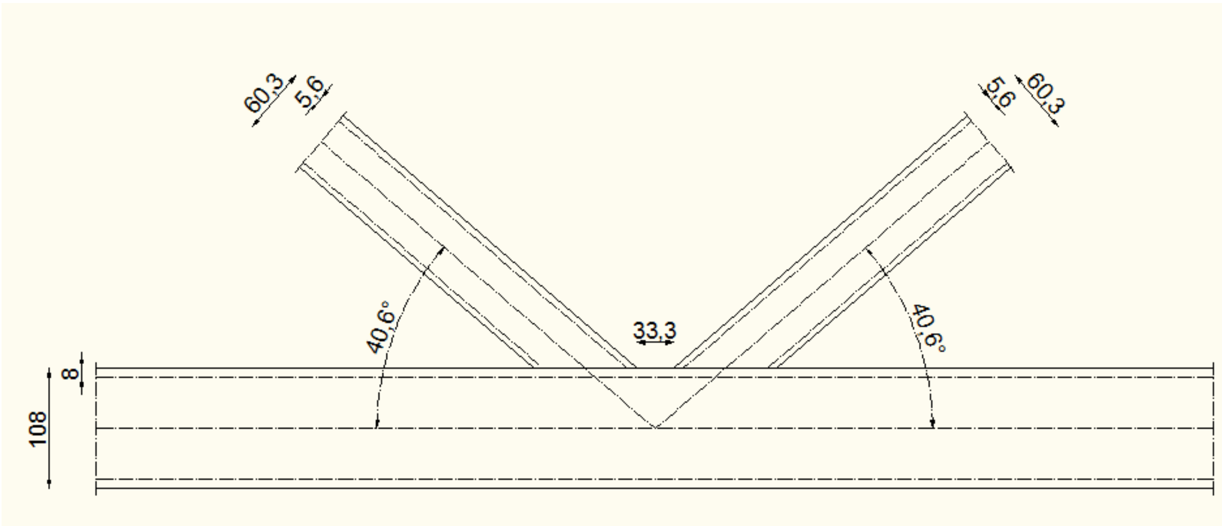


Picture 47. Screenshot of the program [a]- node 28

This node connects the rods numbered 38, 39, 45, 46, 47 and 48.

This node was presented on the drawings 05- Steel Footbridge- node 28, 29, which was attached to this work.

A node is defined as K type according to figure 7.1.



Picture 48. Screenshot of the program [b]- geometry of node 28

Geometry of the connection:

$d_1 = 60{,}3[mm]$ $d_2 = 60{,}3[mm]$ $d_0 = 108{,}0[mm]$

$t_1 = 5{,}6[mm]$ $t_2 = 5{,}6[mm]$ $t_0 = 8{,}0[mm]$

The angles of inclination of diagonals:

$\theta_1 = 40{,}6[°]$ $\theta_2 = 40{,}6[°]$

Design value of the normal force (the maximum value of the normal force in the diagonals - diagonal number 45, in which there is the normal force equal to -24.72kN (Annex 6), is the value for the combination of the number 63 (Annex 5)):

$$N_{Ed} = 24{,}72[kN]$$

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_{1,2}}{d_0} = \frac{60{,}3[mm]}{108{,}0[mm]} = 0{,}56[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{108{,}0[mm]}{8{,}0[mm]} = 13{,}5[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{108{,}0[mm]}{8{,}0[mm]} = 13{,}5[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_{1,2}}{t_{1,2}} = \frac{60{,}3[mm]}{5{,}6[mm]} = 10{,}8[-] \leq 50{,}0$

Condition fulfilled.

Gap:

$$g = 33{,}3[mm] \geq t_1 + t_2 = 5{,}6[mm] + 5{,}6[mm] = 11{,}2[mm]$$

g determined in accordance with the geometry of the connections and figure 1.3.

Condition fulfilled.

According 7.4.1.(2) for joints within the range of validity given in Table 7.1, only chord face failure and punching shear need be considered. The design resistance of a connection should be taken as the minimum value for these two criteria.

Chord face failure:

Factors:

$k_p = 1{,}0[-]$ $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{108{,}0[mm]}{2 \cdot 8{,}0[mm]} = 6{,}75[-]$(table 7.2)

$$k_g = \gamma^{0,2} \cdot \left(1 + \frac{0{,}024 \cdot \gamma^{1,2}}{1 + \exp(0{,}5 \cdot \frac{g}{t_0} - 1{,}33)}\right) = (6{,}75[-])^{0,2} \cdot \left(1 + \frac{0{,}024 \cdot (6{,}75[-])^{1,2}}{1 + \exp(0{,}5 \cdot \frac{33{,}3[mm]}{8{,}0[mm]} - 1{,}33)}\right) = 1{,}545[-]$$

(table 7.2)

Chord face failure:

$$N_{t,Rd}=\frac{\frac{k_g\cdot k_p\cdot f_{y0}\cdot t_0^2}{\sin\ \theta_{1,2}}\left(1{,}8+10{,}2\frac{d_{1,2}}{d_0}\right)}{\gamma_{M5}}=\frac{\frac{1{,}545[-]\cdot 1{,}0[-]\cdot 235\left[\frac{N}{mm^2}\right]\cdot(8[mm\ ])^2}{\sin\ 40{,}6[°]}\left(1{,}8+10{,}2\frac{60{,}3[mm\ ]}{108{,}0[mm\ ]}\right)}{1{,}0}=267{,}571[kN]$$

(table 7.2)

Punching shear failure for K gap joints:

$$d_{1,2}=60{,}3[mm]\leq d_o-2t_0=108{,}0[mm]-2\cdot 8{,}0[mm]=92[mm] \text{ (table 7.2)}$$

$$N_{1,2,Rd}=\frac{f_{y0}}{\sqrt{3}}\cdot t_0\cdot\pi\cdot d_{1,2}\cdot\frac{\frac{1+\sin\ \theta_{1,2}}{2\sin^2\theta_{1,2}}}{\gamma_{M5}}=\frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}}\cdot 8{,}0[mm]\cdot\pi\cdot 60{,}3[mm]\cdot\frac{\frac{1+\sin\ 40{,}6[°]}{2\sin^2 40{,}6[°]}}{1{,}0}=$$

$$400{,}739[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1,2}=\min\left(N_{1,2Rd},N_{t,Rd}\right)=\min(400{,}739[kN];267{,}571[kN])=267{,}571[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd}=\mu\cdot N_{Rd1,2}=0{,}9\cdot 267{,}571[kN]=240{,}814[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}}=\frac{24{,}72[kN]}{240{,}814[kN]}=0{,}10[-]<1{,}0[-]$$

Condition fulfilled.

Welded connections

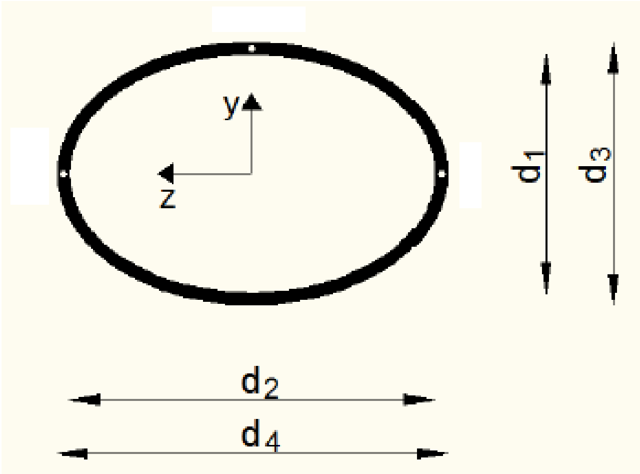
Simplified method for design resistance of fillet weld

The bars are joined by a fillet weld. Was assumed weld thickness equal to 3 mm.

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_z=N_{Ed}\cos\theta_{1,2}=24{,}72[kN]\cdot\cos(40{,}6[^o])=18{,}769[kN]$$

$$N_x=N_{Ed}\sin\theta_{1,2}=24{,}72[kN]\cdot\sin(40{,}6[^o])=16{,}087[kN]$$



Picture 49. Geometry of the quad weld (diagonal number 45)

Where dimensions are:

The smaller the diameter of the inner ellipse:

$$d_1 = 60{,}3[mm]$$

The larger diameter of the inner ellipse:

$$d_2 = \frac{d_1[mm]}{\sin(\theta)} = \frac{60{,}3[mm]}{\sin(40{,}6[^o])} = 92{,}66[mm]$$

The smaller diameter of the external ellipse:

$$d_3 = d_1 + 2a = 60{,}3[mm] + 2 \cdot 3[mm] = 66{,}3[mm]$$

The larger diameter of external ellipse:

$$d_4 = d_2 + 2a = 92{,}66[mm] + 2 \cdot 3[mm] = 98{,}66[mm]$$

Surface area:

$$A_w = \frac{\pi}{4}(d_3 \cdot d_4 - d_1 \cdot d_2) = \frac{\pi}{4}(66{,}3[mm] \cdot 98{,}66[mm] - 60{,}3[mm] \cdot 92{,}66[mm])$$
$$= 7{,}491[cm^2]$$

Normal stresses of tension force:

$$\tau_H = \frac{N_z}{A_w} = \frac{18{,}769[kN]}{7{,}491[cm^2]} = 2{,}505\left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_V = \frac{N_x}{A_w} = \frac{16{,}087[kN]}{7{,}491[cm^2]} = 2{,}148\left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau = \sqrt{\tau_V^2 + \tau_H^2} = \sqrt{\left(2{,}148\left[\frac{kN}{cm^2}\right]\right)^2 + \left(2{,}505\left[\frac{kN}{cm^2}\right]\right)^2} = 3{,}300\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} = \frac{36\left[\frac{kN}{cm^2}\right]/\sqrt{3}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

Condition:

$$\tau = 3{,}300\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

The dimensioning other connections are attached to the document as Annex 7.

5.2.2. Foundation

Theoretical base

There are two basic types of connections foundation:

- simple base joints,
- fixed base joints;

In this project a column is nominally pin – connected to a foundation that is designed assuming that the base moment is zero.

Usual practice to attach flexible type of column base to the concrete foundation by two anchor bolts symmetrically placed about the web on the column's major axis. Anchor bolts provide resistance to any uplift forces which arise in the column.

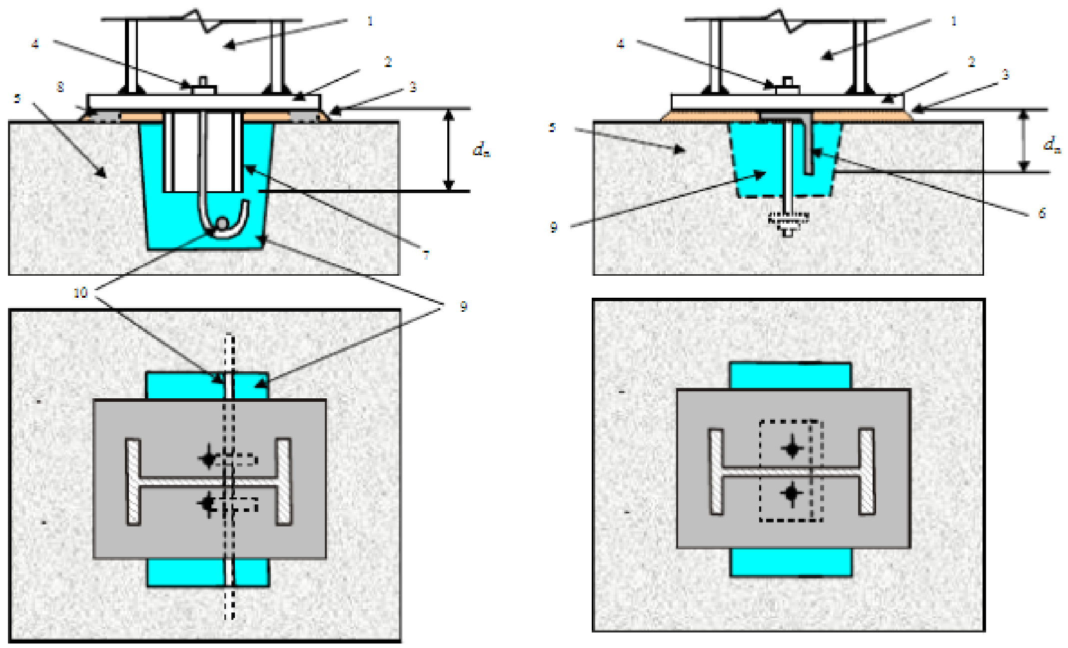
The basic design approach is to ensure that the bearing stresses under the base plate neither exceed the design bearing strength of the foundation joint material nor lead to excessive bending of the base plate. [28]

In this document designed of shear nibs for column bases.

The horizontal force transferred by the shear nib welded to the underside of the base plate. The steel plate is placed in a foundation pocket of sufficient depth and size. The pocket is filled with non-shrink concrete after the column and the anchor bolts are positioned.

A shear nib consists of a short length of steel profile welded to the underside of the base plate. Once the concrete is poured into the reserve hole for the anchor bolts and the column grouted in its final position, the nib is placed in the foundation. The shear force acting on the column base can be communicated to the foundation by the nib acting horizontally leading to compression over the vertical surface of the nib against the concrete foundation.

The figure below shows typical column bases with shear nibs [29]:



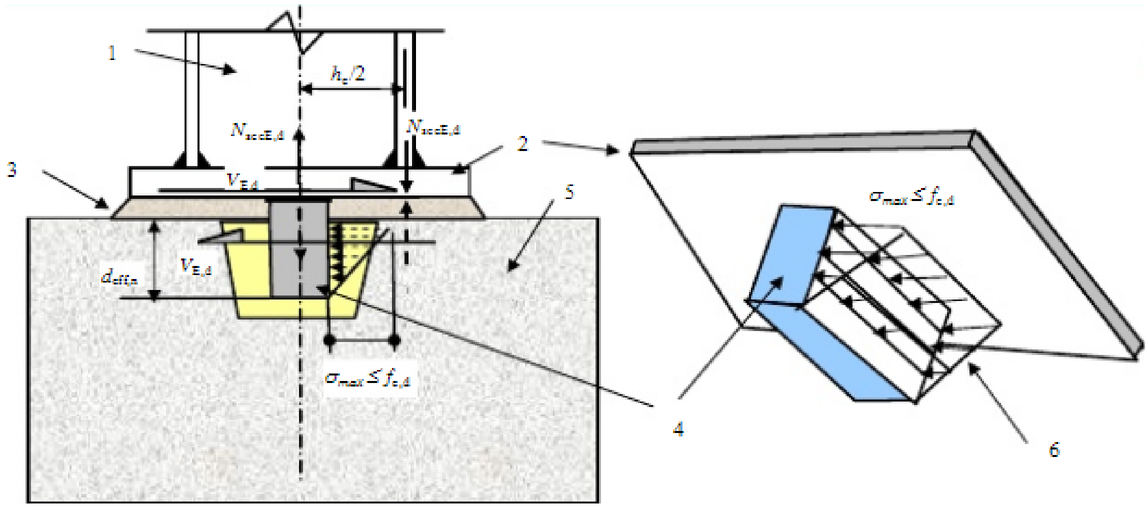
Key :

1. I section column
2. Base plate
3. Joint space to be filled with grout
4. Anchor bolt
5. Concrete foundation
6. Angle section shear nib
7. I section shear nib
8. Steel positioning/levelling plate
9. Pocket reservation to be filled with non shrink concrete or grout after column positioning
10. Foundation reinforcing bar

Picture 50. Typical column bases with shear nibs [29]

Design model

Shear nib model showing the forces and stresses induced: distribution of compressive stresses over shear nib and secondary forces is shown on the picture [29]:



Key :

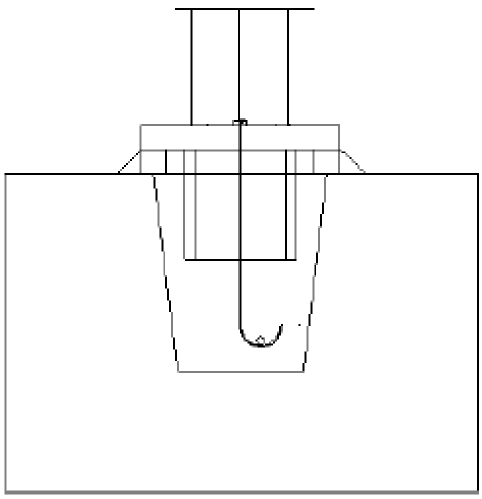
1. I section column
2. Base plate
3. Joint material (grout)
4. Nib
5. Concrete foundation
6. Triangular distribution of pressure on the nib

Picture 51. Shear nib model showing the forces and stresses induced: distribution of compressive stresses over shear nib and secondary forces [29]

Design

The basis of the design elements of round tubes performed as articulated foot steel with against an abutment, which carrying of horizontal force.

All joins of foundation in the construction were in the same way. For the calculation the value obtained from the reaction of the program [a], for the most strenuous node foundation, assuming the most unfavorable combination of loads.



Picture 52. Sketch showing connection foundation

It was assumed that the total horizontal force is transferred to the foundation by the anvil-omitted part caused by friction between the base plate and the foundation and the participation of anchor bolts.

Internal forces: $N_{Ed} = 171{,}62[kN]$ $V_{Ed} = 30{,}70[kN]$

The tubular element base structure:

$d = 88{,}9[mm]$ $t = 5[mm]$

Base plate:

$h_p = 410{,}00[mm]$ $b_p = 410{,}00[mm]$ $t_p = 20{,}00[mm]$

Dimension of a spread footing:

$d_f = 900{,}00[mm]$ $b_f = 900{,}00[mm]$ $h_f = 600{,}00[mm]$

Adopted concrete class C30/37

Characteristic compressive cylinder strength of concrete at 28 days:

$$f_{ck} = 30{,}000\left[\frac{N}{mm^2}\right]$$ (according EN 1992-1-1 table 3.1.)

The coefficient:

$$\propto_{cc} = 1{,}00[-]$$ (according EN 1992-1-1 3.1.6. (1) NOTE)

$$\gamma_c = 1{,}4[-]$$ (according EN 1992-1-1 Table NA.2)

The value of the design compressive strength:

$$f_{cd} = \propto_{cc} \cdot \frac{f_{ck}}{\gamma_c} = 1{,}00 \cdot \frac{30{,}000\left[\frac{N}{mm^2}\right]}{1{,}4[-]} = 21{,}429\left[\frac{N}{mm^2}\right]$$ (according EN 1992-1-1 formula 3.15)

Assume the joint material (grout) layer to be 30 mm thick.

Determination of the dimensions of the abutment member from steel HEB:

HEB 140- dimension:

$b_n = 140{,}00[mm]$ $h_n = 140{,}00[mm]$ $z_n = 12{,}00[mm]$

$r_n = 12{,}00[mm]$ $A_n = 26{,}00[cm^2]$

Conditions:

$$min\, b_n = 140{,}00[mm] \geq \max\left(90; \frac{V_{Ed}}{30 f_{cd}}\right) mm = \geq \max\left(90; \frac{30{,}70[kN]}{30 \cdot 21{,}429\left[\frac{N}{mm^2}\right]}\right)$$

$$= \max(90; 47{,}755)$$

$$max b_n = 140{,}00[mm] \leq \mathrm{b_p} - 2\mathrm{t_{fc}} = 410{,}00[mm] - 2 \cdot 5[mm] = 400[mm]$$

Estimate the minimum required depth of angle nib:

$$\min d_{effnn} = 160[mm] \geq \max\left(60; \frac{2V_{Ed}}{b_n \cdot f_{cd}}\right) mm$$

$$= \max\left(60[mm]; \frac{2 \cdot 230{,}70[kN]}{140{,}00[mm] \cdot 21{,}429\left[\frac{N}{mm^2}\right]}\right)$$

$$= \max(60[mm]; 153{,}795[mm]) = 153{,}795[mm]$$

Check the maximum practical limits on the nib depth:

$$\min d_{effnn} + 30[mm] = 160[mm] + 30[mm] = 190[mm]$$

$$\leq \min(0{,}8d_f; h_n) = \min(0{,}8 \cdot 160[mm]; 140[mm])$$

$$= \min(128[mm]; 140[mm]) = 128[m]$$

Condition fulfilled.

Estimate the secondary tensile force in the vertical angle leg:

$$N_{sexEd} = V_{Ed} \cdot \left(\frac{d_{effn}}{3} + 30\right) \cdot \frac{2}{d} = 30{,}70[kN] \cdot \left(\frac{160[mm]}{3} + 30\right) \cdot \frac{2}{88{,}9[mm]}$$

$$= 57{,}555[kN]$$

Check the leg thickness under combined shear and tension using the Von Mises criteria:

$$t_a = 12{,}00[mm] \geq \sqrt{\left(\frac{N_{sexEd}}{b_n \cdot f_{yn}}\right)^2 + 3\left(\frac{V_{Ed}}{b_n \cdot f_{yn}}\right)^2} = \frac{V_{Ed}}{f_{yn} \cdot b_n}\sqrt{\left(\frac{2 \cdot \frac{d_{effn}}{3} + 30}{d}\right)^2 + 3}$$

$$= \frac{30{,}70[kN]}{235\left[\frac{N}{mm^2}\right] \cdot 140{,}00[mm]}\sqrt{\left(\frac{2 \cdot \frac{160[mm]}{3} + 30}{88{,}9[mm]}\right)^2 + 3} = 2{,}161[mm]$$

The minimum required weld sizes are then:

Web double fillet welds :

$$\propto_v = 3[mm] \geq \frac{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2} \cdot V_{Ed}}{f_u(2h_n + b_n)} = \frac{\sqrt{3} \cdot 0{,}8[-] \cdot 1{,}25[-] \cdot 30{,}70[kN]}{235\left[\frac{N}{mm^2}\right](2 \cdot 140{,}00[mm] + 140{,}00[mm])}$$

$$= 0{,}539[mm]$$

Flange double fillet welds :

$$\propto_v = 3[mm] \geq \frac{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2} \cdot N_{secEd}}{f_u(2b_{fc} - t_{wc})} = \frac{\sqrt{3} \cdot 0{,}8[-] \cdot 1{,}25[-] \cdot 57{,}555[kN]}{235\left[\frac{N}{mm^2}\right](2 \cdot 140{,}00[mm] - 12{,}00[mm])}$$

$$= 1{,}583[mm]$$

Check of the local resistance of the column web:

The column web is subjected to the concentrated secondary tensile force N_{secEd} . The following local resistance check is made:

$$N_{secEd} = 57{,}555[kN] \leq \frac{(t_a \cdot b_{eff})f_{yc}}{\gamma_{M0}} = \frac{(12{,}0[mm] \cdot 64{,}426[mm])235\left[\frac{N}{mm^2}\right]}{1{,}00[-]}$$
$$= 181{,}681[kN]$$

The force is assumed to be distributed over the following effective width in the column web:

$$b_{eff} = t_a + 2t_p + 5(\sqrt{2}\, a_{wc}) = 12[mm] + 2 \cdot 5[mm] + 5(\sqrt{2} \cdot 6[mm]) = 64{,}426[mm]$$

The recess in the foundation:

$$\frac{V_{Ed}}{A} \leq f_{cd}$$
$$\frac{V_{Ed}}{f_{cd}} \leq A$$
$$\frac{V_{Ed}}{f_{cd}} = \frac{30{,}70[kN]}{21,429\left[\frac{N}{mm^2}\right]} = 14{,}326[cm^2] \leq A = 14[cm] \cdot 14[cm] = 196[cm^2]$$

The design bearing strength

$$A_{c0} = h_p \cdot b_p = 410{,}00[mm] \cdot 410{,}00[mm] = 1681{,}0[cm^2]$$

The maximum calculation area load distribution:

$$A_{c1} = d_f \cdot b_f = 900{,}00[mm] \cdot 900{,}00[mm] = 8100{,}0[cm^2]$$

Ratio: $\propto = \sqrt{\frac{A_{c1}}{A_{c0}}} = \sqrt{\frac{8100{,}0[cm^2]}{1681{,}0[cm^2]}} = 2{,}195[-]$

So, the foundation joint material coefficient:

$$\beta_j = \frac{2}{3}[-] \text{ (according 1993-1-8 6.2.5.(7)}$$

The design bearing strength of the joint:

$$f_{jd} = \propto \cdot \beta_j \cdot f_{cd} = 2{,}195[-] \cdot \frac{2}{3}[-] \cdot 21,429\left[\frac{N}{mm^2}\right] = 31{,}358\left[\frac{N}{mm^2}\right] \text{ (formula 6.6)}$$

The additional bearing width:

$$c = t_p\sqrt{\frac{f_{yp}}{3 \cdot f_{jd} \cdot \gamma_{M0}}} = 20{,}00[mm]\sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{3 \cdot 31{,}358\left[\frac{N}{mm^2}\right] \cdot 1{,}0[-]}} = 31{,}610[mm] \text{ (formula 6.5)}$$

Forend with a small overhang:

$A_0 = 410{,}00[mm] \cdot 410{,}00[mm] = 1681[cm^2] > 0{,}95 \cdot d^2 = 0{,}95 \cdot (88{,}9[mm])^2 = 75{,}08[cm^2]$

The design resistance of the column base plate:

$$N_{Rd} = f_{jd}\big((2d+2t)(c+2t)+(d-2c-2t)(2c+t)\big)$$

$$= 31{,}358\left[\frac{N}{mm^2}\right]\big((2\cdot 88{,}9[mm]+2\cdot 5{,}0[mm])(31{,}610[mm]+2\cdot 5{,}0[mm]) + (88{,}9[mm]-2\cdot 31{,}610[mm]-2\cdot 5{,}0[mm])(2\cdot 31{,}610[mm]+5{,}0[mm])\big) = 278{,}586[kN] \quad > \quad N_{jEd} = 171{,}62[kN]$$

Condition fulfilled.

Check the dimensions of the sheet metal:

$$b_p = 410{,}00[mm] \quad > \quad d+2\cdot t = 88{,}9[mm]+2\cdot 5{,}0[mm] = 98{,}9[mm]$$

$$h_p = 410{,}00[mm] \quad > \quad d+2\cdot t = 88{,}9[mm]+2\cdot 5{,}0[mm] = 98{,}9[mm]$$

$$t_p = 20{,}00[mm] \quad \geq \quad \frac{c}{\left(\frac{f_{yp}}{3\cdot f_{jd}\cdot\gamma_{M0}}\right)^{0,5}} = \frac{31{,}610[mm]}{\left(\frac{235\left[\frac{N}{mm^2}\right]}{3\cdot 31{,}358\left[\frac{N}{mm^2}\right]\cdot 1{,}0[-]}\right)^{0,5}} = 20{,}00[mm]$$

Sheet metal was sized correctly.

The term resistance of the column base to compression:

$$e_b = \frac{b_f-d-2t}{2} = \frac{900{,}00[mm]-88{,}9[mm]-2\cdot 5{,}0mm]}{2} = 400{,}55[mm]$$

$$e_h = \frac{h_f-d-2t}{2} = \frac{600{,}00[mm]-88{,}9[mm]-2\cdot 5{,}0mm]}{2} = 250{,}55[mm]$$

Ratio:

$$\propto = \min\left(1+2\cdot\frac{e_h}{h_p};1+2\cdot\frac{e_b}{b_p};3{,}1+\frac{d_f}{max(h_p;b_p)}\right)$$

$$= \min\left(1+2\cdot\frac{250{,}55[mm]}{410{,}00[mm]};1+2\cdot\frac{400{,}55[mm]}{410{,}00[mm]};3{,}1+\frac{900{,}00[mm]}{max(410{,}00[mm];410{,}00[mm])}\right) = \min(2{,}222;2{,}954;5{,}295) = 2{,}222[-]$$

So, the foundation joint material coefficient:

$$\beta_j = \frac{2}{3}[-] \text{ (according 1993-1-8 6.2.5.(7)}$$

The design bearing strength of the joint:

$$f_{jd} = \propto\cdot\beta_j\cdot f_{cd} = 2{,}222[-]\cdot\frac{2}{3}[-]\cdot 21{,}429\left[\frac{N}{mm^2}\right] = 46{,}718\left[\frac{N}{mm^2}\right] \text{ (formula 6.6)}$$

The design resistance of the column base plate:

$$N_{j,Rd} = f_{jd}\big((2d+2t)(c+2t)+(d-2c-2t)(2c+t)\big)$$

$$= 46{,}718\left[\frac{N}{mm^2}\right]\big((2\cdot 88{,}9[mm]+2\cdot 5{,}0[mm])\ (31{,}610[mm]+2\cdot 5{,}0[mm]) + (88{,}9[mm]-2\cdot 31{,}610[mm]-2\cdot 5{,}0[mm])(2\cdot 31{,}610[mm]+5{,}0[mm])\big) = 415{,}045[kN] \quad > \quad N_{jEd} = 171{,}62[kN]$$

Condition fulfilled.

Resistance due to strength pullout

In the foundation occurs snatching force equal to 30,70[kN]. Connection dimensioned to pull-out force only because of the foundation bolts. Check the destruction of concrete by splitting and breaking concrete cone is not the subject of this study.

The load caused by pullout force shall be transferred through the foundation bolts M20 class 8.8.

Tension resistance:

Factor: $k_2 = 0{,}90[-]$ (table 3.4)

The design tension resistance per bolt:

$$F_{t,Rd} = \frac{k_2\cdot f_{ub}\cdot A_s}{\gamma_{M2}} = \frac{0{,}90\ [-]\cdot 800\left[\frac{N}{mm^2}\right]\cdot \pi\cdot\frac{(10[\mathrm{mm}])^2}{4}}{1{,}25[-]} = 45{,}239[kN] \text{ (table 3.4)}$$

Resistance: $\frac{V_{Ed}}{F_{t,Rd}} = \frac{30{,}70[kN]}{45{,}239[kN]} = 0{,}68[-]$

Condition fulfilled for only one screw. If the individual screw satisfies tension capacity, it is not necessary to check two screws.

Resistance due to the pull-out force foundation screws:

$$N_{Rk,s} = A_s\cdot f_{ub} = \pi\cdot\frac{(10[\mathrm{mm}])^2}{4}\cdot 800\left[\frac{N}{mm^2}\right] = 62{,}832[\mathrm{kN}]$$

Resistance: $\frac{V_{Ed}}{\mathrm{N_{Rk,s}}} = \frac{30{,}70[kN]}{62{,}832[kN]} = 0{,}49[-]$

Condition fulfilled for only one screw. If the individual screw satisfies tension capacity, it is not necessary to check two screws.

6. Final considerations

Designing a footbridge, even if it has a small span, is a complex and laborious process. The design must meet all conditions of strength, be adapted for public use and be resistant to the effects of weather, climate and external conditions. The structure must also be adapted to the movement of people with disabilities.

Truss structures are characterized by having very small internal moments in comparison to the axial forces, in theirs bars. During operation of the load perpendicular to the upper chord (the most common case in the load combinations) the lower chord of the grid is tensioned, and the upper compression.

When designing the structure, it was necessary to take into account many types of loads. The designed truss work dead load, in the form of:

- structural weight
- the burden of the railing
- the weight of the stairs
- weight surface.

Also included is the impact of climatic conditions, the Polish conditions, by giving define the following climatic loads:

- snow (in two different situation)
- wind (acting in three planes (XZ, YZ, XY), sucking wind and wind pressure)
- temperature

The footbridge will be earmarked to pedestrians and people with disabilities, it is to consider of traffic load:

- horizontal force,
- concentrated load
- pedestrian traffic load.

A decisive impact on the lightweight steel supporting constructions is the temperature. In a climate of Central European temperature differences are significant, thus causing contraction and expansion of the material. Thereby, the temperature load is one of the dominant. This has been a major consideration.

In addition, design of light steel structures exposed to dynamic load requires consideration of the effect of resonance. Under the influence of the dynamic load, the structure vibrates, which causes discomfort to users and may cause destruction of property. The Footbridge analyzed in this work on the first proposed geometry achieved natural frequency within a frequency range which could cause vibration. The risk of structural damage as a result of resonance was very large. This problem was solved by introducing additional rods design-enhanced structural rigidity which decreased susceptibility to dynamic loads and ensured a safe natural frequency.

Due to the large number of types of loads, very combinations were obtained and considered.

Due to the complexity of the geometry modeling and the bar profiles were divided into 10 groups. For each group of rod, for the worst case of combination performed dimensioning due to the: compression and buckling.

A major challenge in the construction of circular profiles are connection nodes. Areas where in several planes, different rods connect with various sections and angles of inclination to the main bar are particularly interesting. A variety of solutions can be used: welded joints, bolt joints, using steel plates. In some of the designed connections used all of these types. Some of them will be complicated in execution, but also provides low consumption of steel.

Were used the projected flexible foundation is subjected to horizontal and the vertical force. Due to this fact it was designed not only for a combination of column and slab, but also designed to shear nib. This solution enables the transfer of both acting forces.

The design on the ultimate limit state ensures the safety of the structure. Equally important, the design structures to the serviceability limit state, which prevents excessive deflection of the structure.

Allowed deflection of the structure compared to the value from program [a]:

$$w_{max} = \frac{L}{250} = \frac{14000[mm]}{250} = 40[mm] \quad \geq \quad 35{,}1[mm]$$

Terms of ULS and SLS status have been met.

The steel structure weighs about 4 tons, it proves that steel has a high strength/weight ratio. Thus, the dead weight of steel structures is relatively small. Construction of a pedestrian steel bridges is in comparison with concrete ones very light.

7. Reference:

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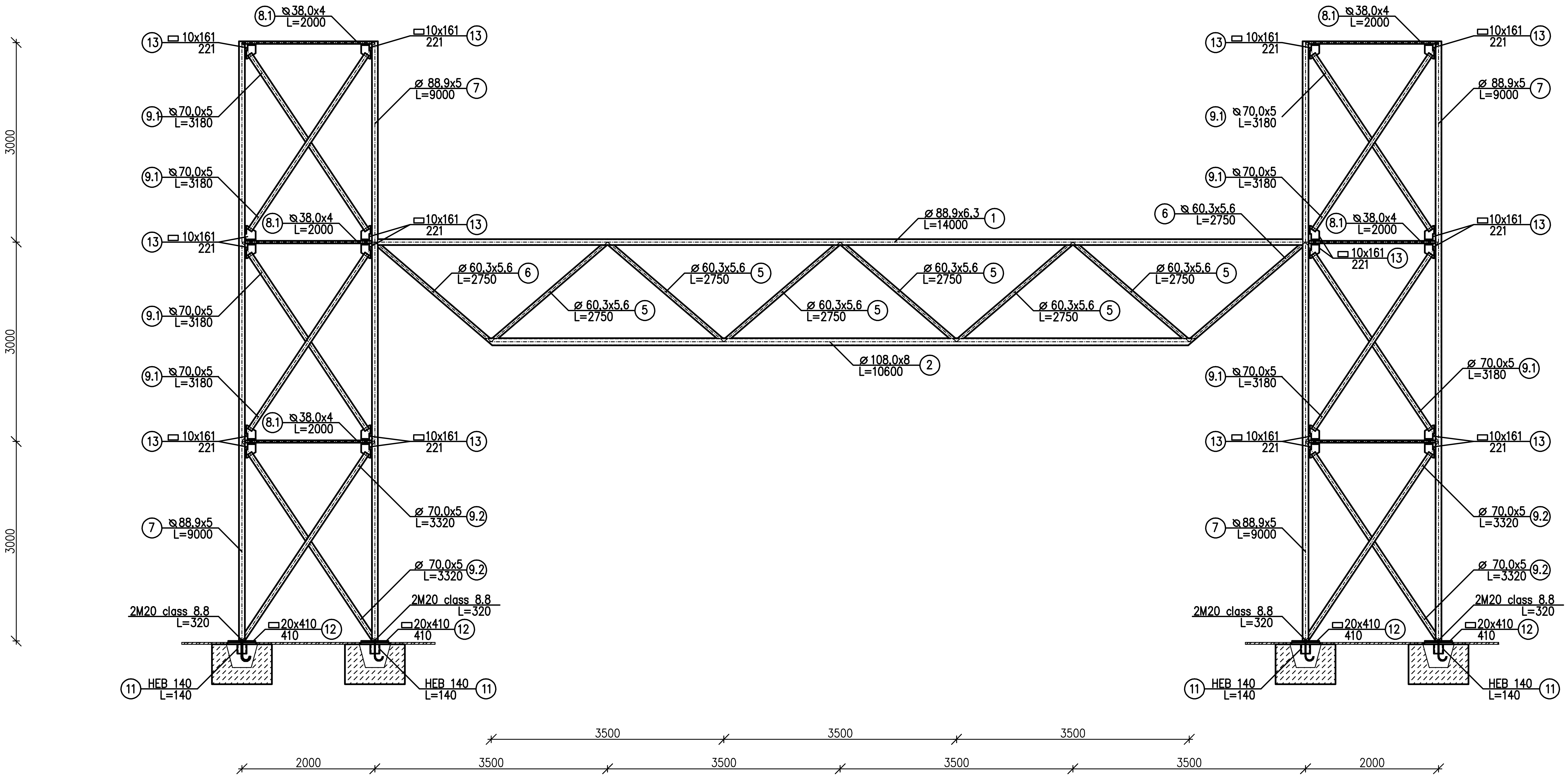
Programs:

[a] Autodesk Robot Structural Analysis Professional 2011,

[b] AutoCad 2009

Steel Footbridge– plane XZ

scale: 1:50



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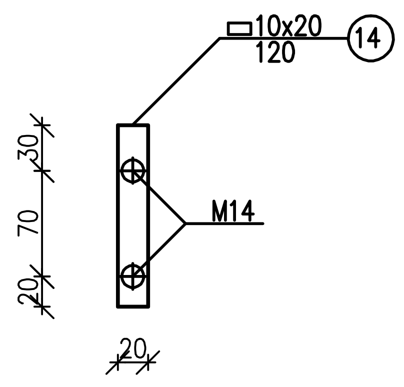

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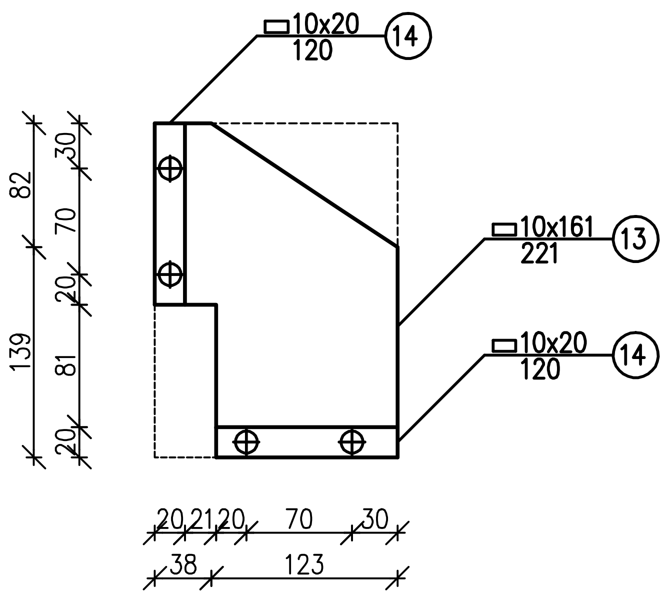


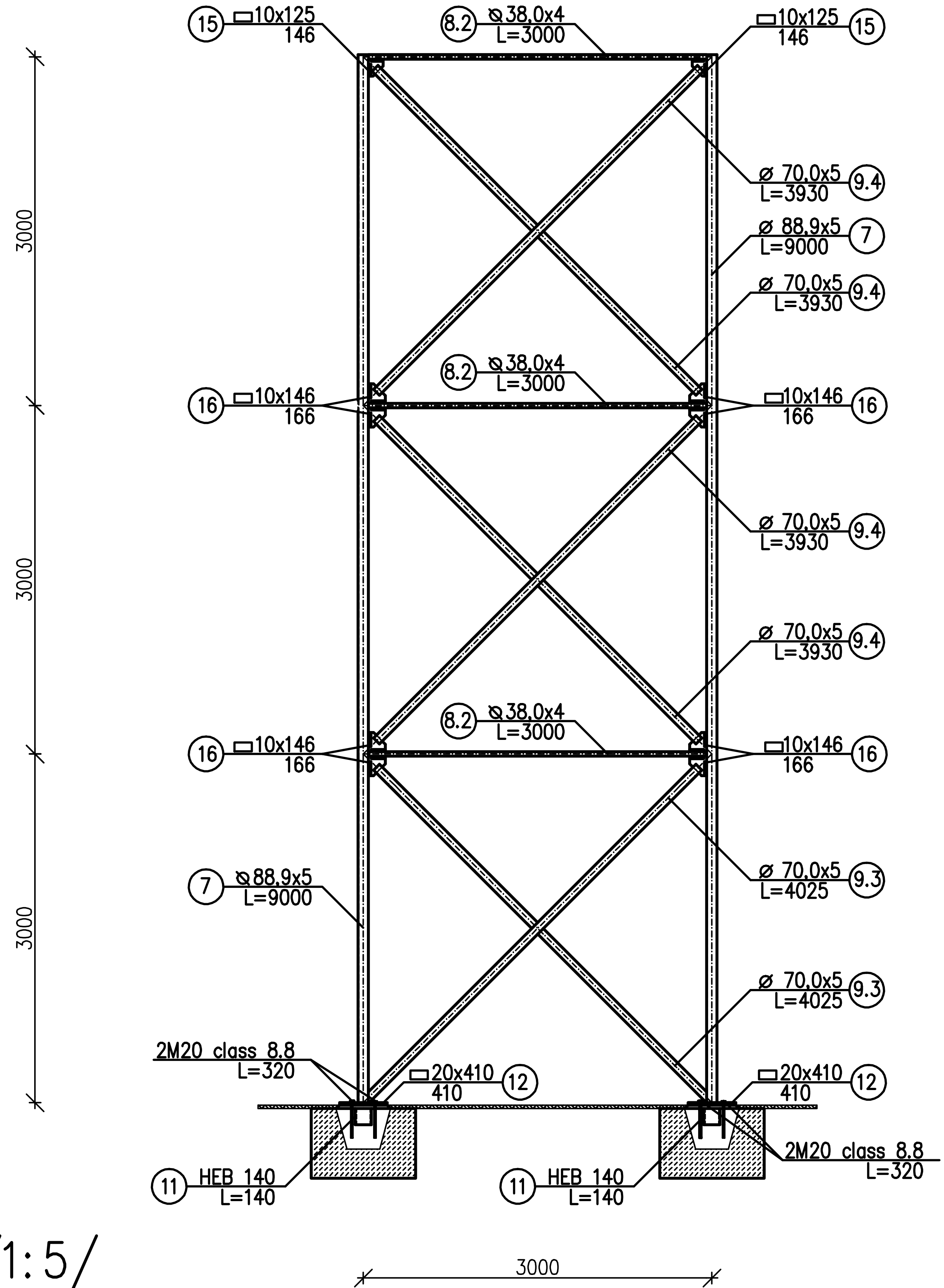
steel S235

NOTE: The weld are shown in the detail drawings.

Subject:	Drawing number:
Steel Footbridge– plane XZ University of Aveiro	01
	Scale: 1:50
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014
	Drawn by: Anna Kur

Steel Footbridge– plane YZ

scale: 1:50



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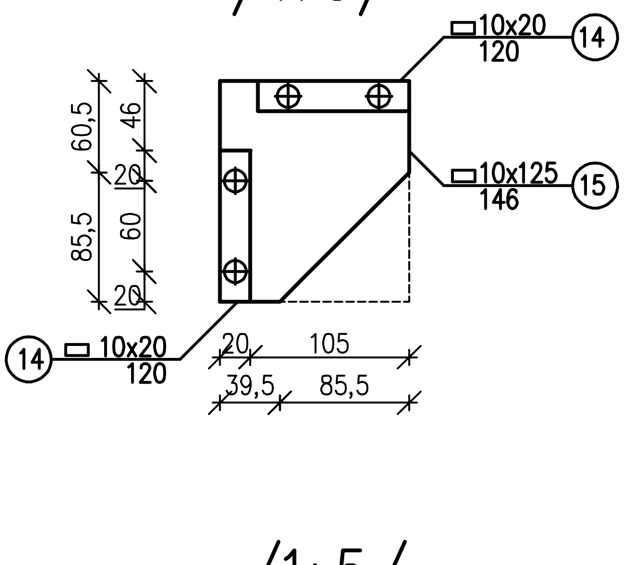

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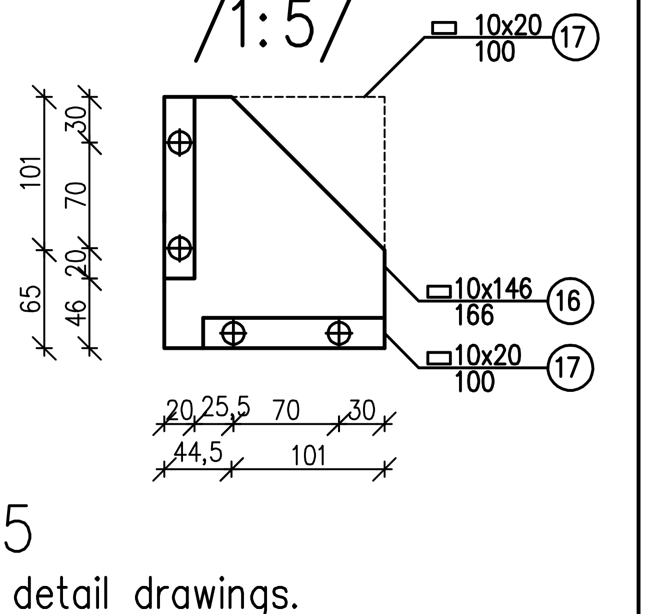

steel S235

NOTE: The weld are shown in the detail drawings.

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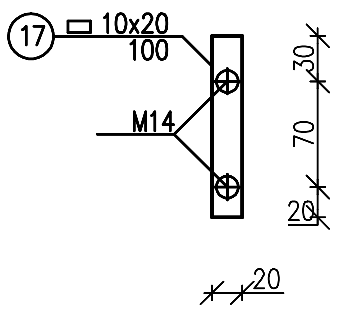
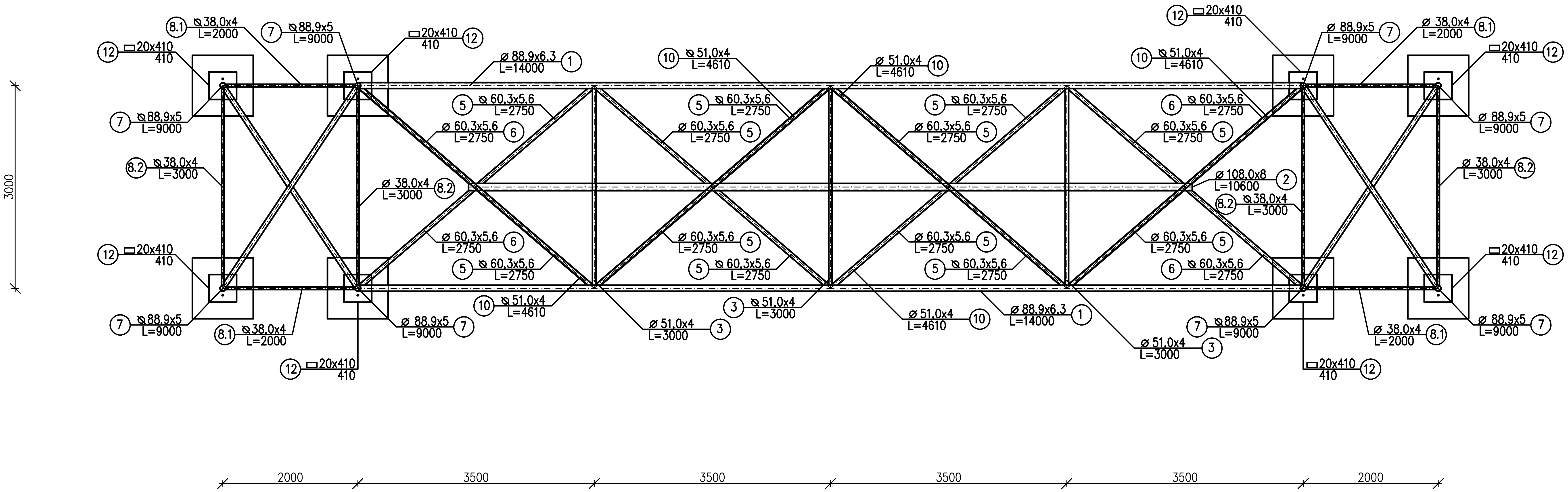


Subject: Steel Footbridge– plane YZ University of Aveiro	**Drawing number:** 02 **Scale:** 1:50
Project: Master thesis:: Structural Design of a Steel Footbridge	**Date:** 30.06.2014 **Drawn by:** Anna Kur

Steel Footbridge– plane XY

scale: 1:50



steel S235

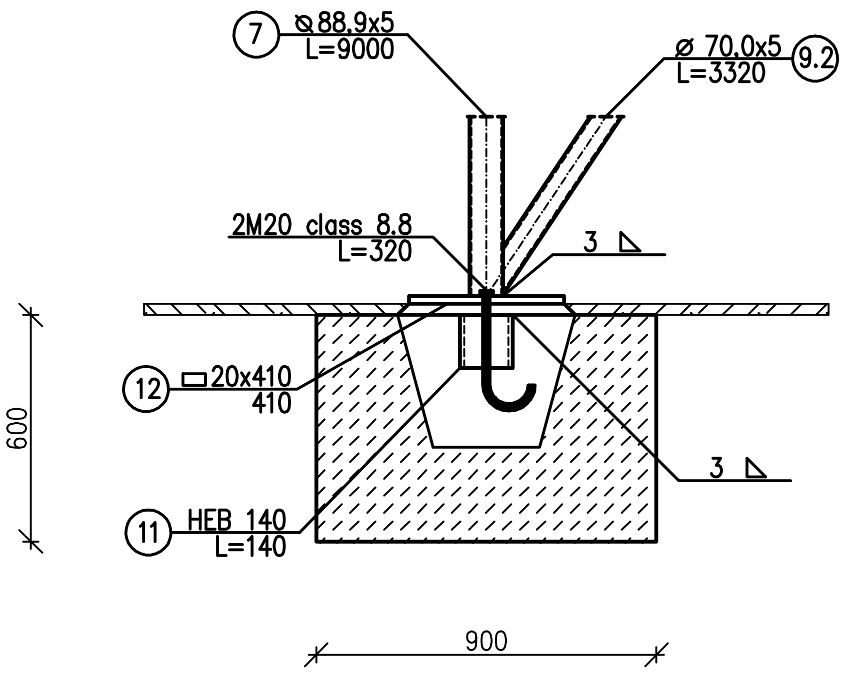
NOTE: The weld are shown in the detail drawings.

Subject: Steel Footbridge– plane XY University of Aveiro	Drawing number: 03
	Scale: 1:50
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014
	Drawn by: Anna Kur

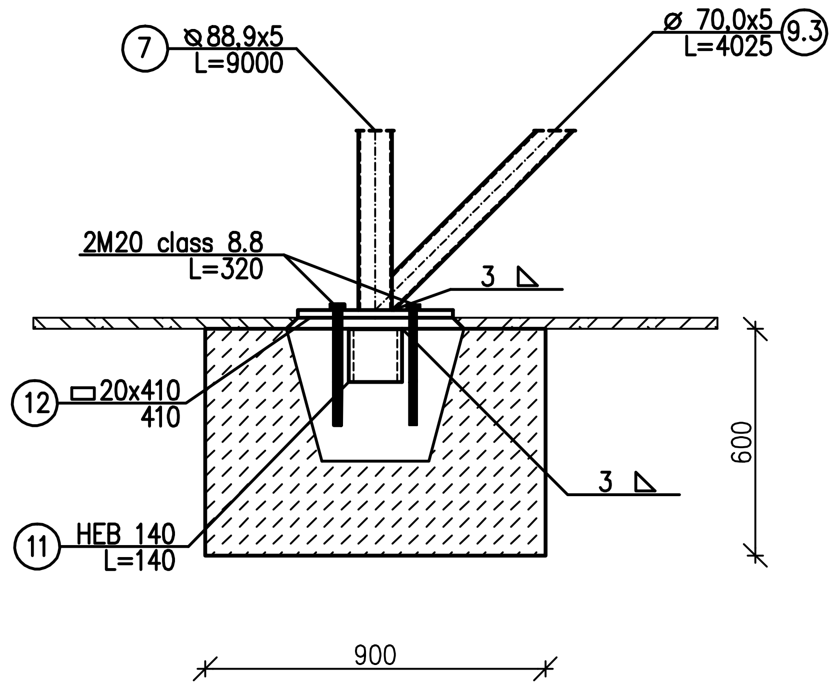
Steel Footbridge– foundation

scale: 1:20

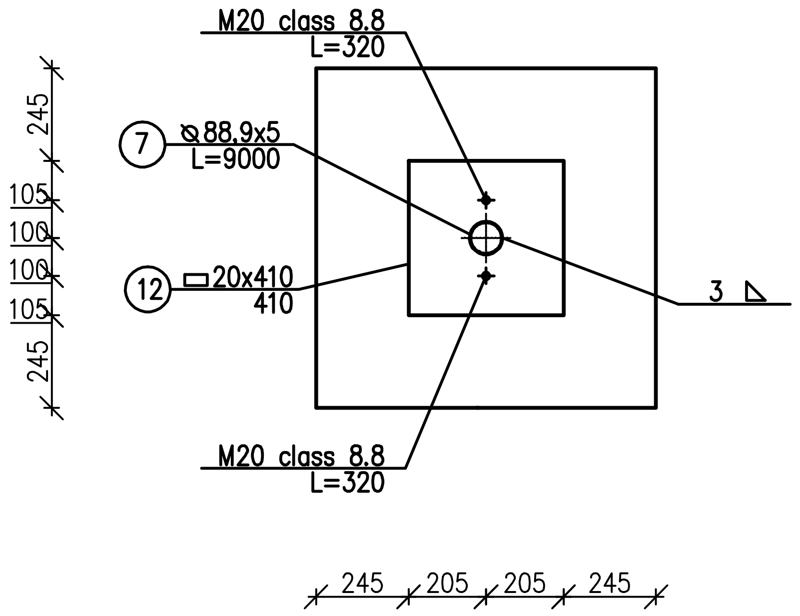
Plane XZ



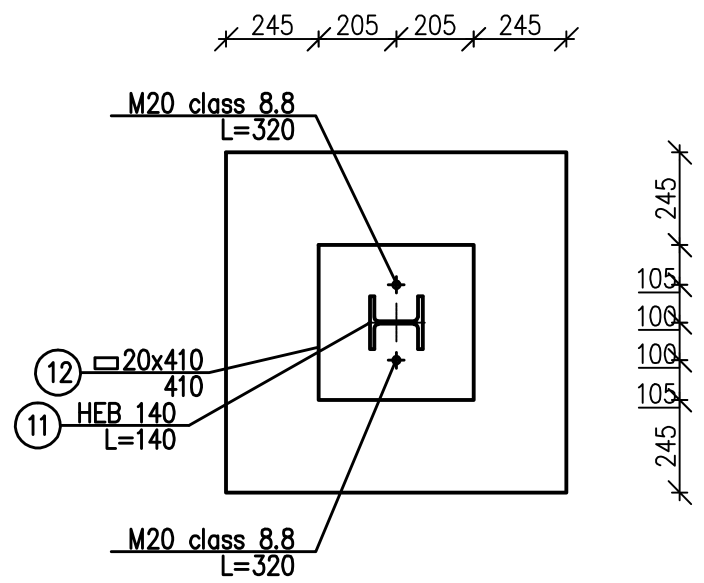
Plane YZ



Plane XY



Plane XZ– nib



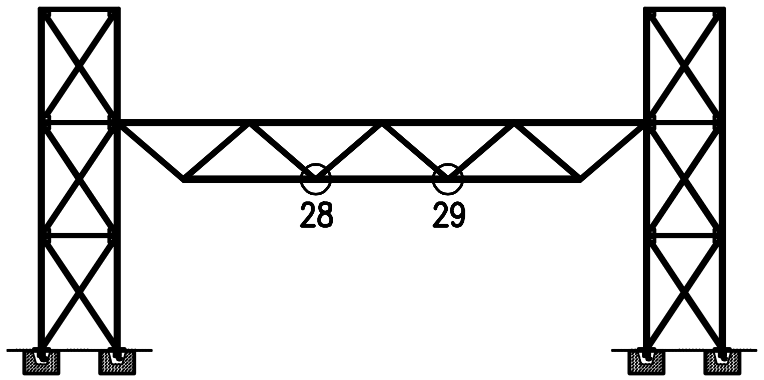
Concrete C30/37

steel S235

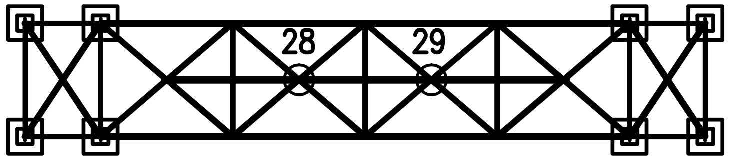
Subject: Steel Footbridge– plane YZ University of Aveiro	**Drawing number:** 04 **Scale:** 1:20
Project: Master thesis:: Structural Design of a Steel Footbridge	**Date:** 30.06.2014 **Drawn by:** Anna Kur

Steel Footbridge– node 28, 29

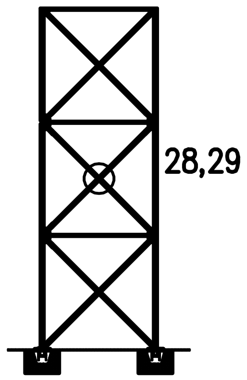
Location of the node– plane XZ
scale: 1:200



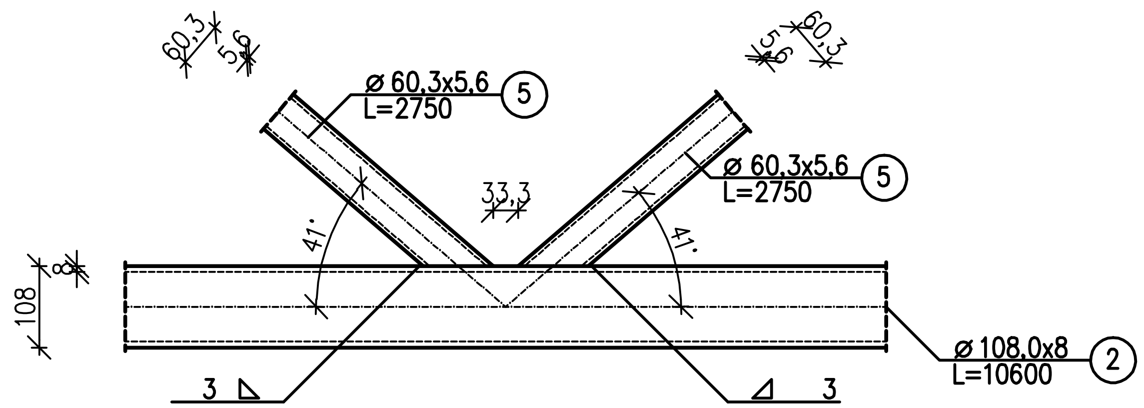
Location of the node– plane XY
scale: 1:200



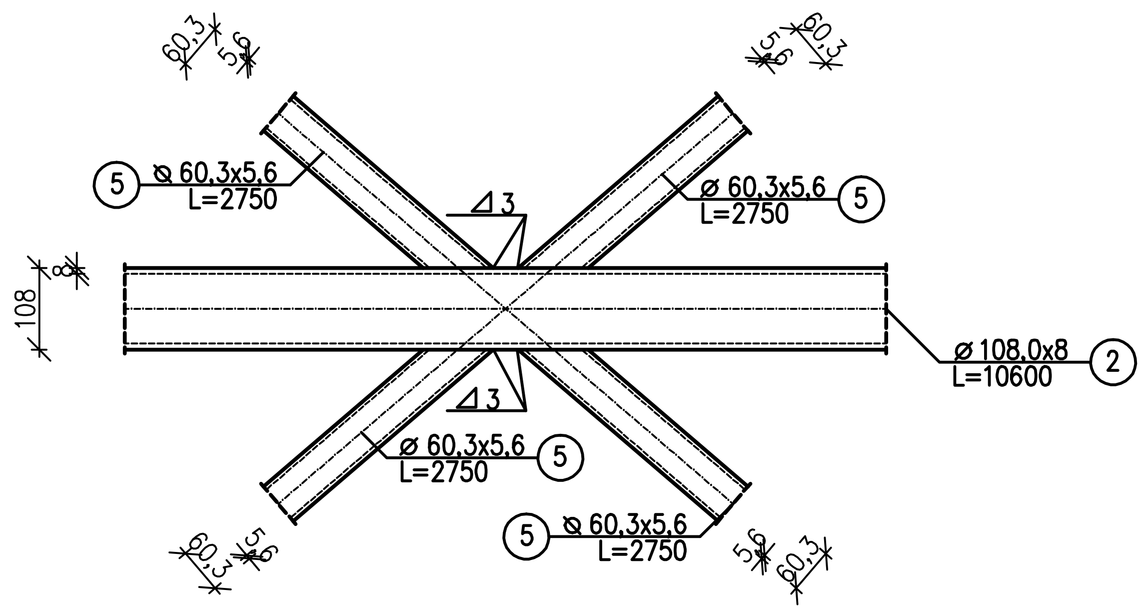
Location of the node– plane YZ
scale: 1:200



Plane XZ



Plane XY

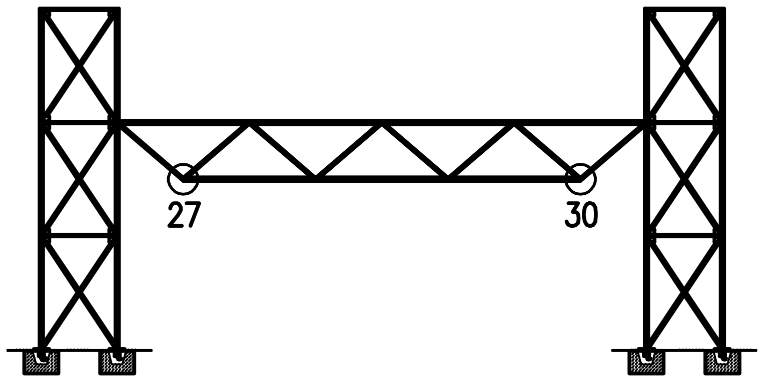


steel S235

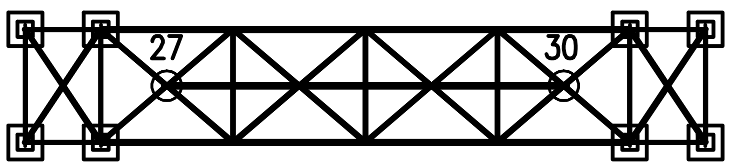
Subject: Steel Footbridge– node 28, 29 University of Aveiro	Drawing number: 05
	Scale: 1:10
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014
	Drawn by: Anna Kur

Steel Footbridge– node 27, 30

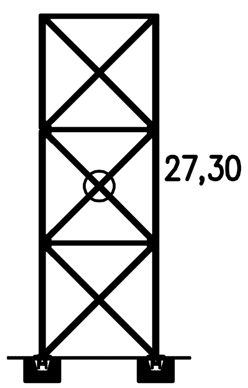
Location of the node– plane XZ
scale: 1:200



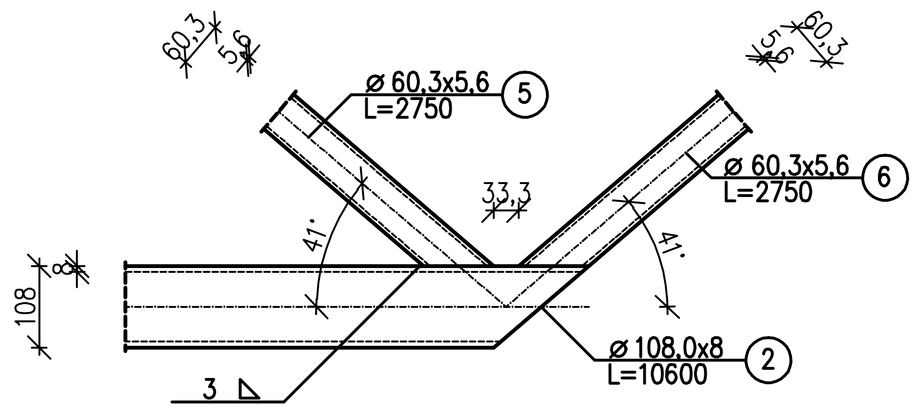
Location of the node– plane XY
scale: 1:200



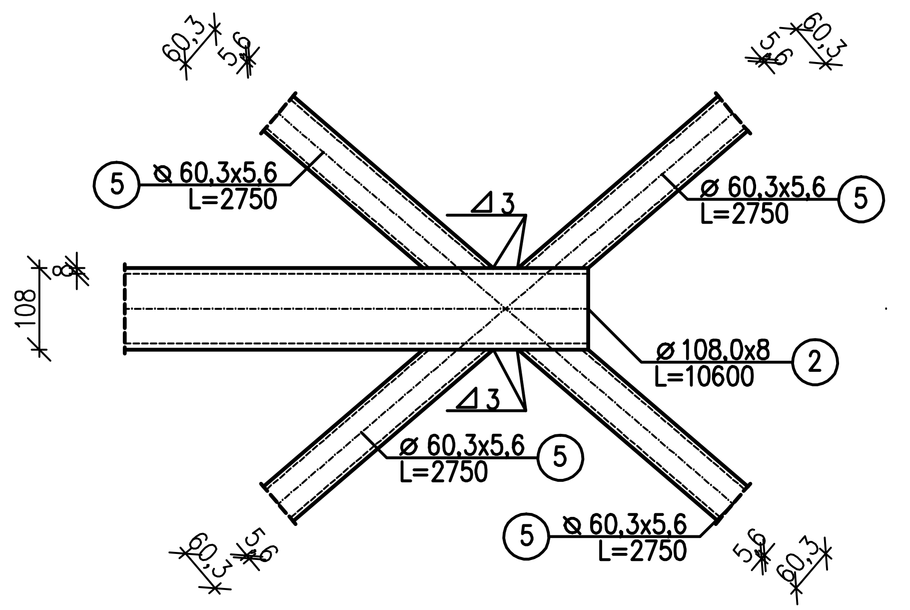
Location of the node– plane YZ
scale: 1:200



Plane XZ



Plane XY

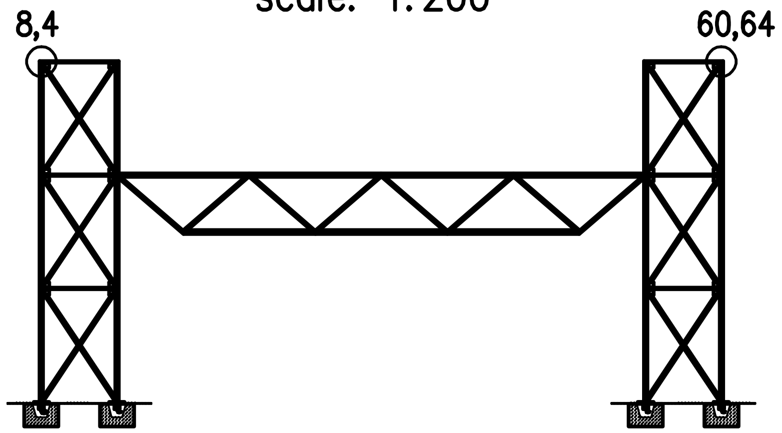


steel S235

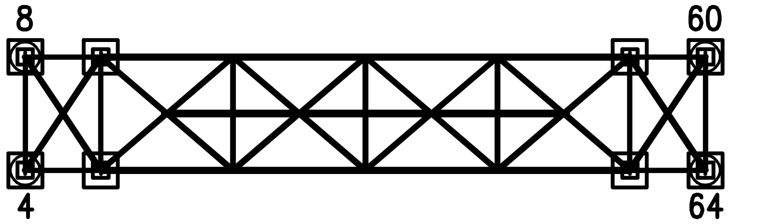
Subject: Steel Footbridge– node 27, 30 University of Aveiro	Drawing number: 06
	Scale: 1:10
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014
	Drawn by: Anna Kur

Steel Footbridge– node 4, 8, 60, 64

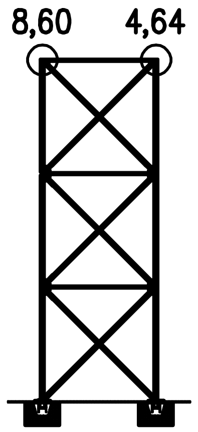
Location of the node– plane XZ
scale: 1:200



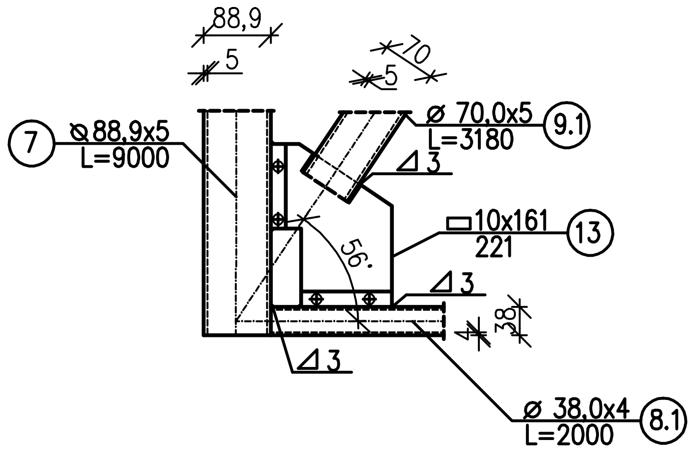
Location of the node– plane XY
scale: 1:200



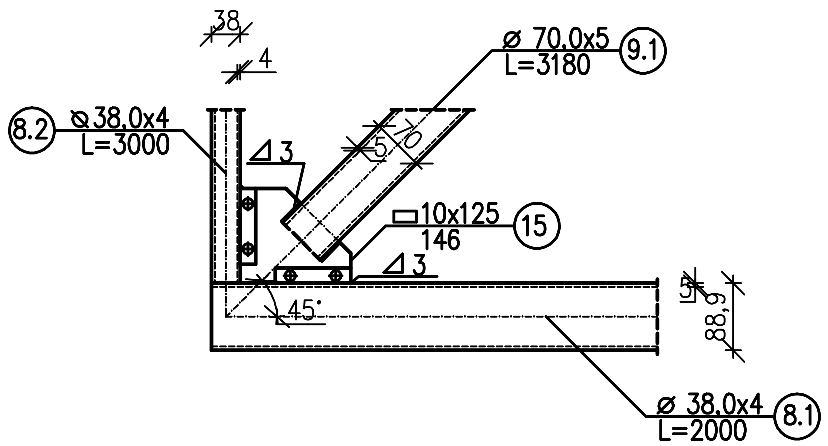
Location of the node– plane YZ
scale: 1:200



Plane XZ



Plane YZ

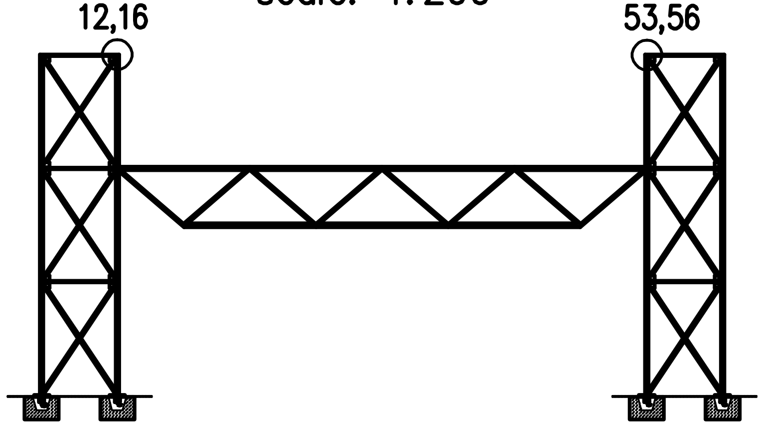


steel S235

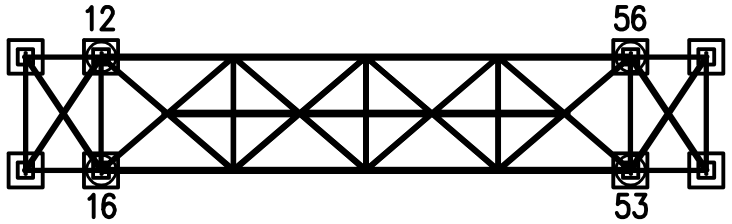
Subject:	Drawing number: 07
Steel Footbridge– node 4, 8, 60, 64 University of Aveiro	Scale: 1:10
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014 Drawn by: Anna Kur

Steel Footbridge– node 12, 16, 53, 56

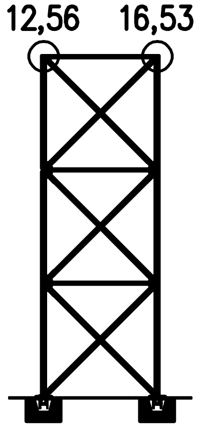
Location of the node– plane XZ
scale: 1:200



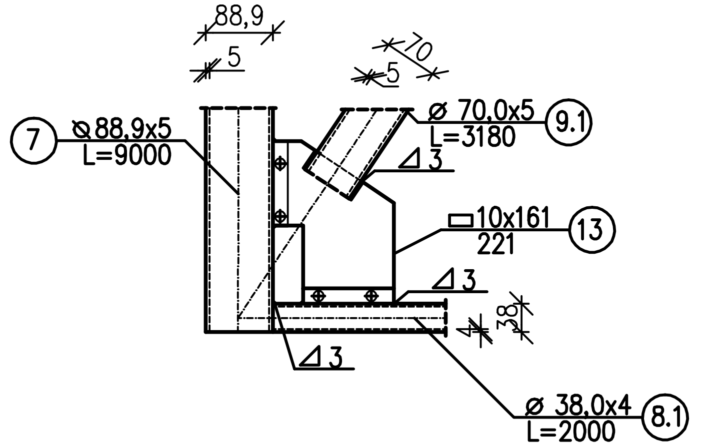
Location of the node– plane XY
scale: 1:200



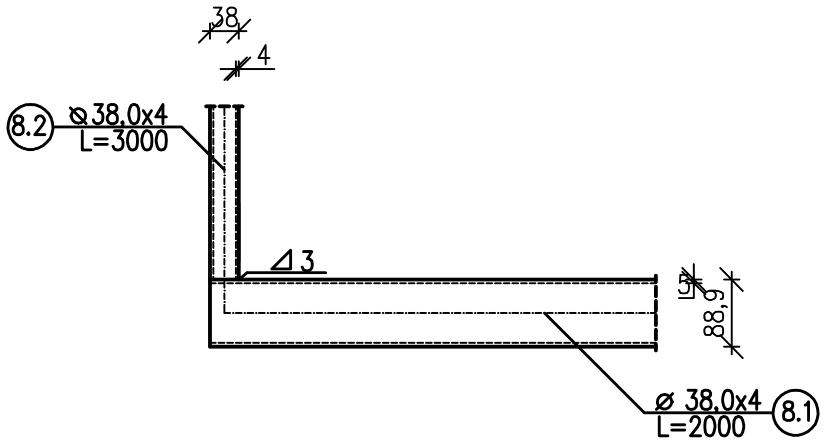
Location of the node– plane YZ
scale: 1:200



Plane XZ



Plane YZ



steel S235

Subject:
Steel Footbridge– node 12, 16, 53, 56
University of Aveiro

Drawing number: 08

Scale: 1:10

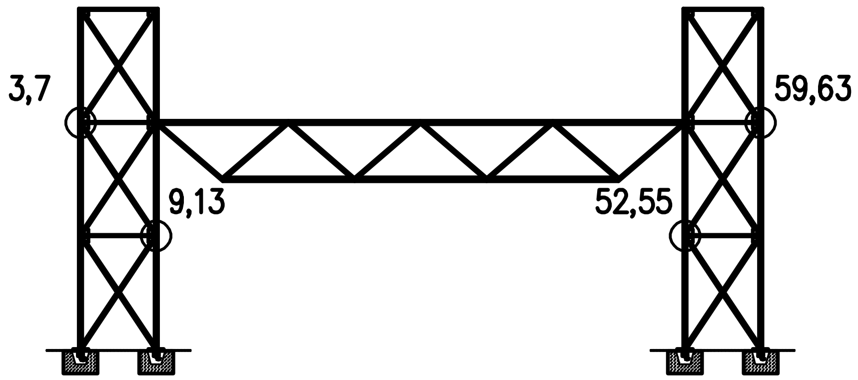
Project:
Master thesis::
Structural Design of a Steel Footbridge

Date: 30.06.2014

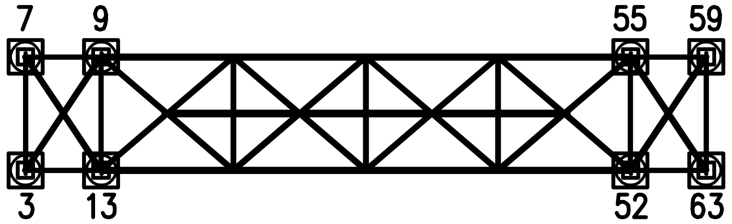
Drawn by: Anna Kur

Steel Footbridge– node 3, 7, 9, 13, 52, 55, 59, 63

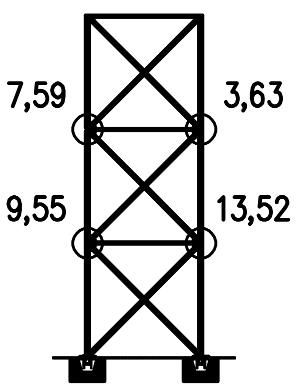
Location of the node– plane XZ
scale: 1:200



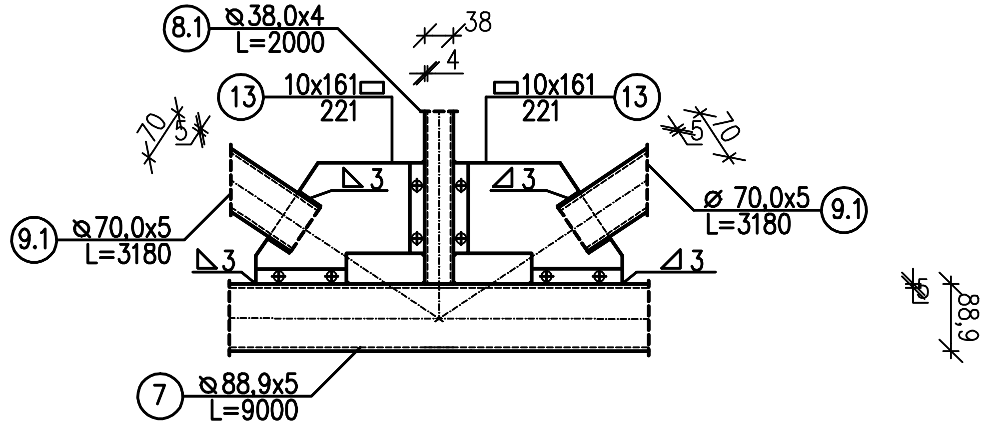
Location of the node– plane XY
scale: 1:200



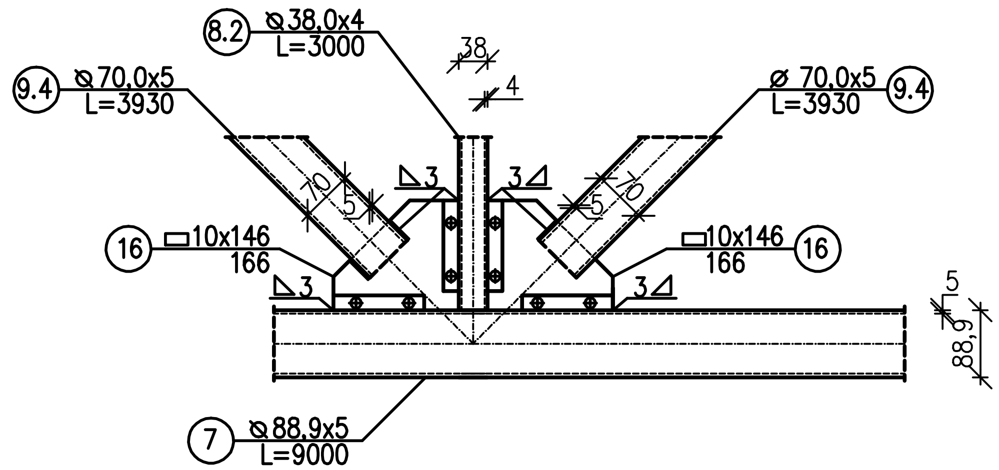
Location of the node– plane YZ
scale: 1:200



Plane YZ



Plane YZ

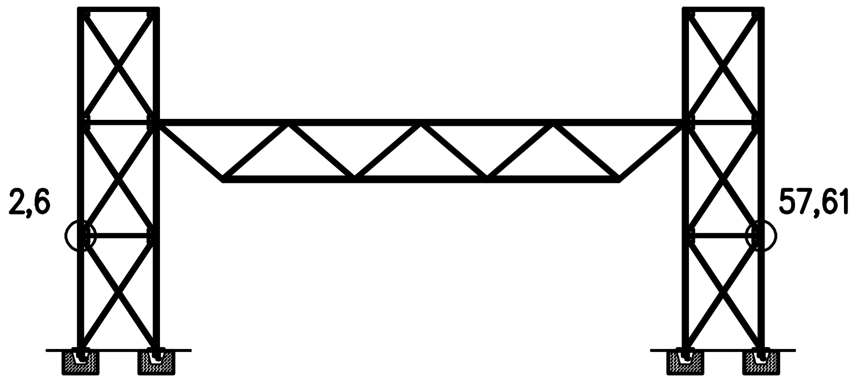


steel S235

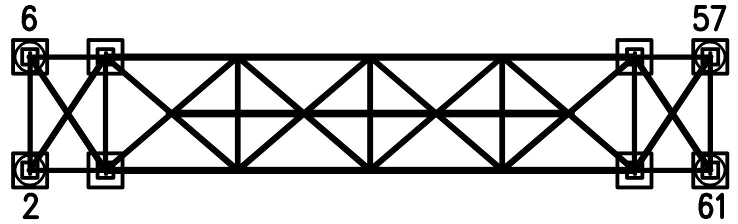
Subject: Steel Footbridge– node 3, 7, 9, 13, 52, 55, 59, 63 University of Aveiro	Drawing number: 09
	Scale: 1:10
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014
	Drawn by: Anna Kur

Steel Footbridge– node 2, 6, 57, 61

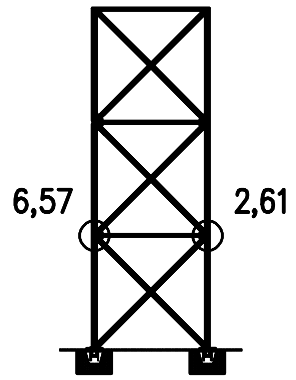
Location of the node– plane XZ
scale: 1:200



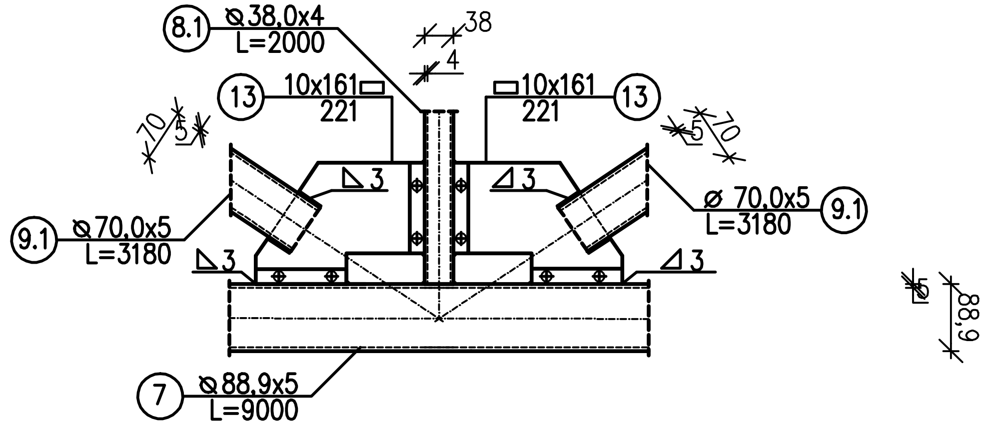
Location of the node– plane XY
scale: 1:200



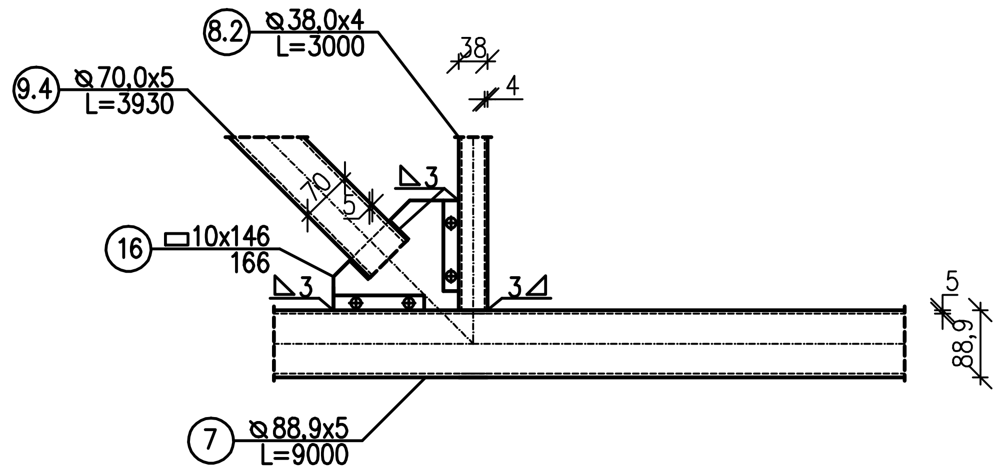
Location of the node– plane YZ
scale: 1:200



Plane YZ



Plane YZ

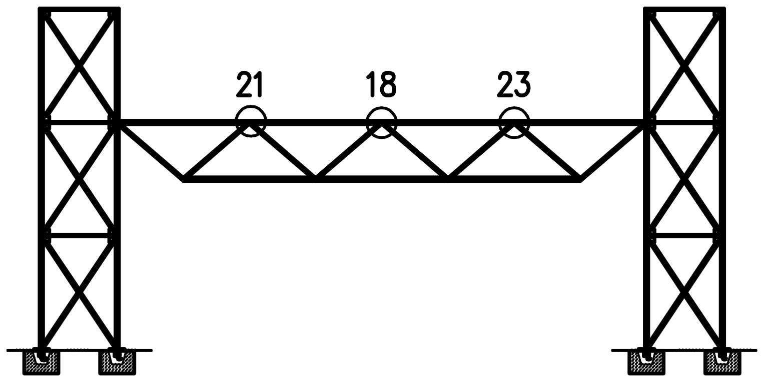


steel S235

Subject:	Drawing number:
Steel Footbridge– node 2, 6, 57, 61 University of Aveiro	10
	Scale: 1:10
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014
	Drawn by: Anna Kur

Steel Footbridge– node 18, 21, 23

Location of the node– plane XZ
scale: 1:200



Location of the node– plane XY
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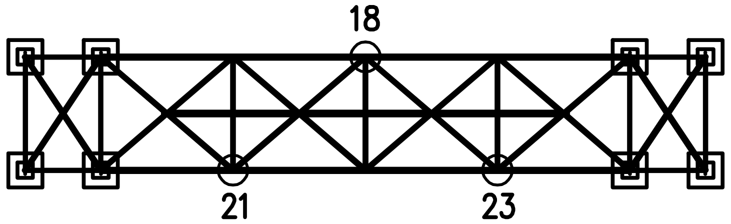
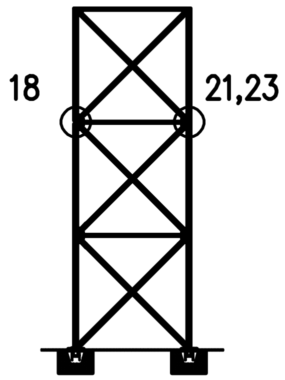
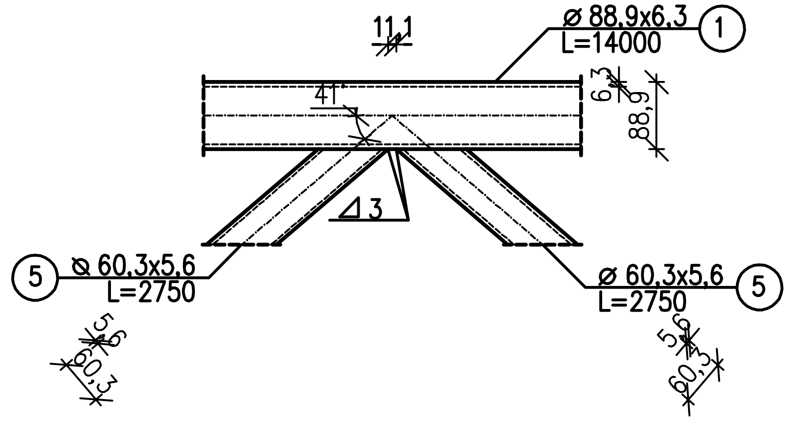


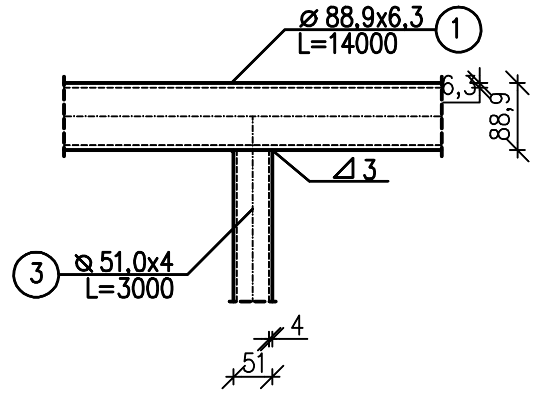
Location of the node– plane YZ
scale: 1:200



Plane XZ



Plane XY

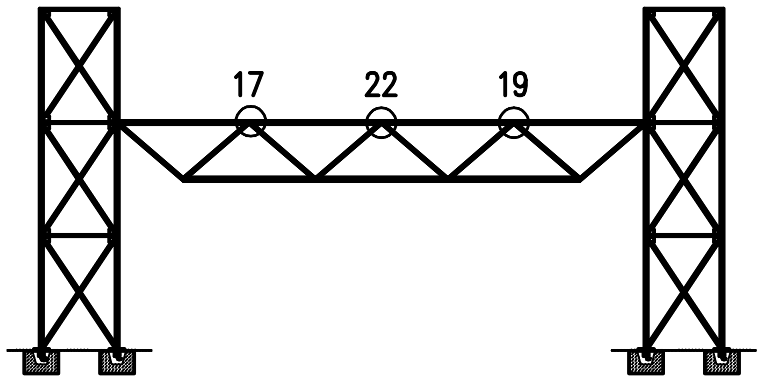


steel S235

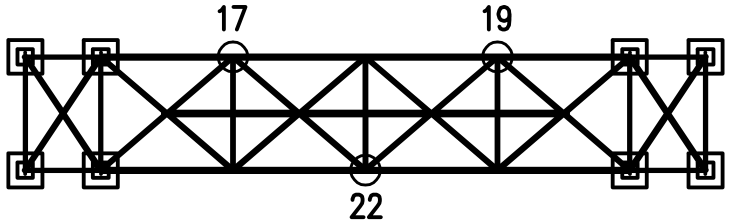
Subject:	Drawing number:
Steel Footbridge– node 18, 21, 23 University of Aveiro	11
	Scale: 1:10
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014
	Drawn by: Anna Kur

Steel Footbridge– node 17, 19, 22

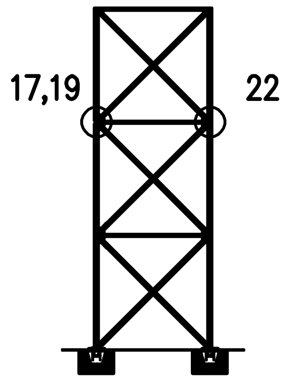
Location of the node– plane XZ
scale: 1:200



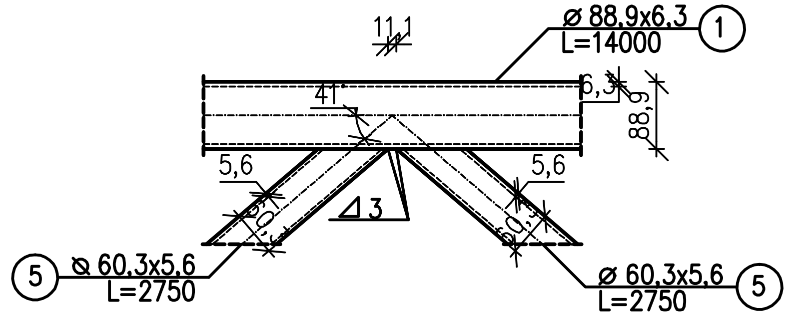
Location of the node– plane XY
scale: 1:200



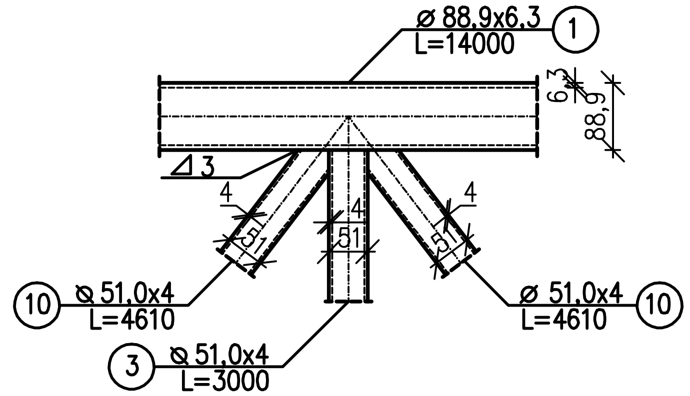
Location of the node– plane YZ
scale: 1:200



Plane XZ



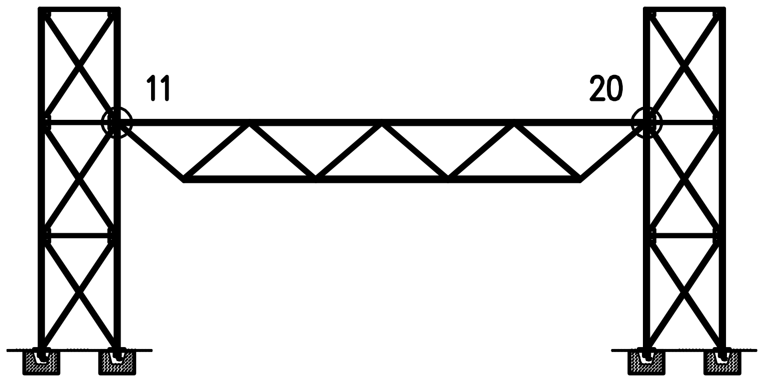
Plane XY



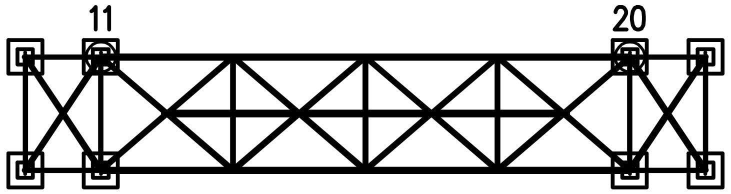
steel S235

Subject: Steel Footbridge– node 17, 19, 22 University of Aveiro	Drawing number: 12
	Scale: 1:12
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014
	Drawn by: Anna Kur

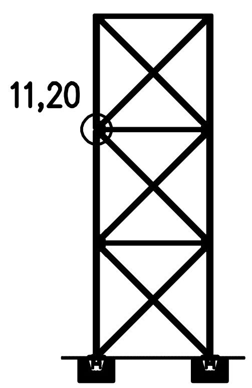
Location of the node– plane XZ
scale: 1:200



Location of the node– plane XY
scale: 1:200

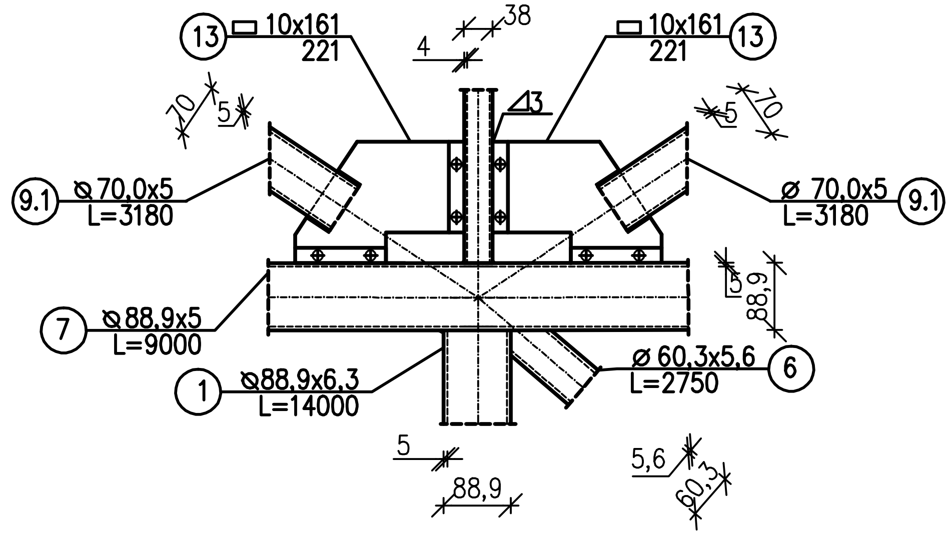


Location of the node– plane YZ
scale: 1:200

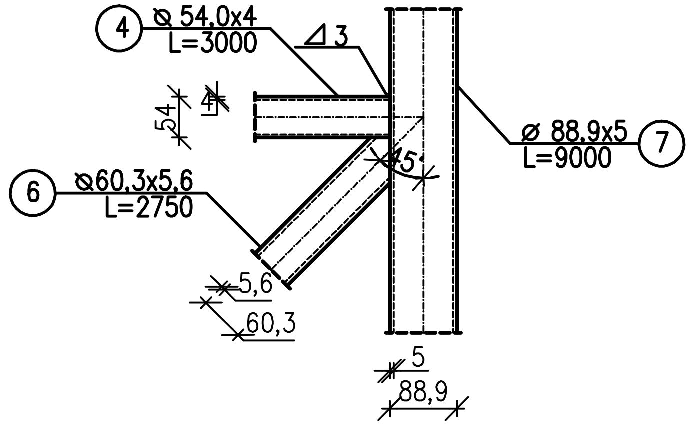


Steel Footbridge– node 11, 20

Plane XZ



Plane XY

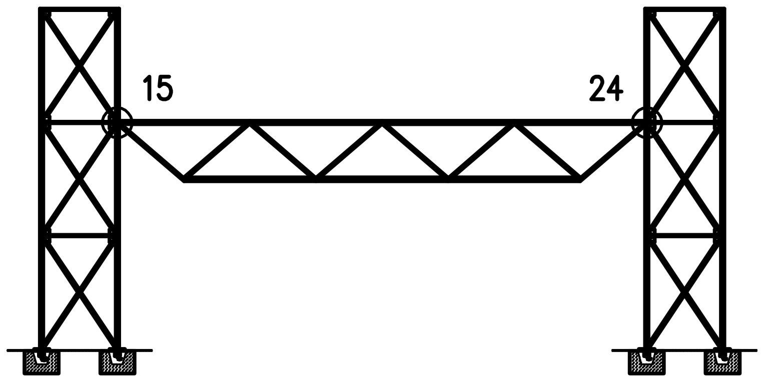


steel S235

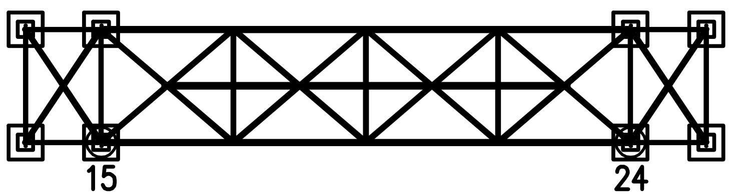
Subject:	Drawing number: 13
Steel Footbridge– node 11, 20 University of Aveiro	Scale: 1:10
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014
	Drawn by: Anna Kur

Steel Footbridge– node 15, 24

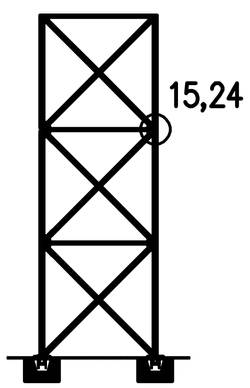
Location of the node– plane XZ
scale: 1:200



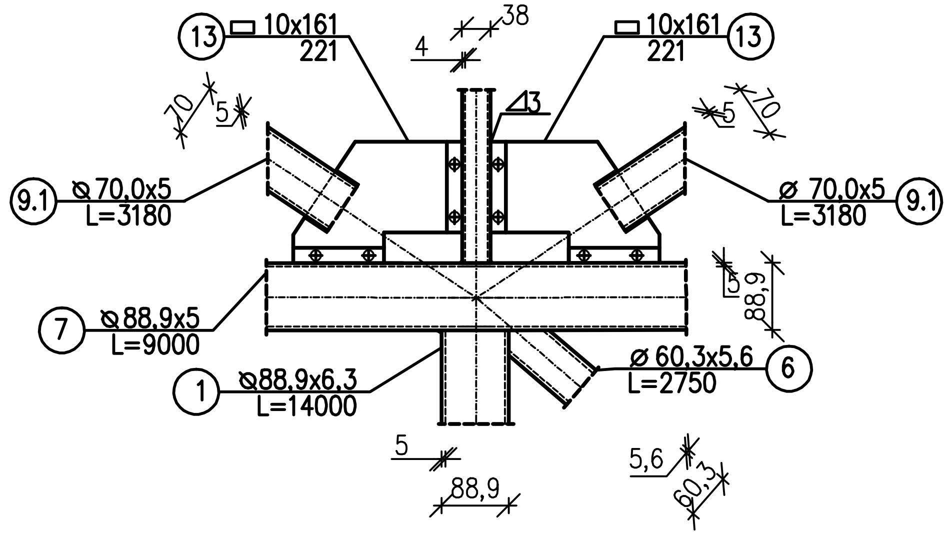
Location of the node– plane XY
scale: 1:200



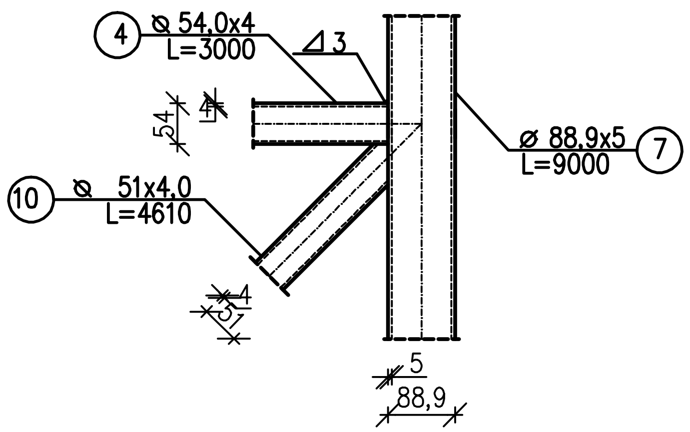
Location of the node– plane YZ
scale: 1:200



Plane XZ



Plane XY



steel S235

Subject: Steel Footbridge– node 15, 24 University of Aveiro	**Drawing number:** 14
	Scale: 1:10
Project: **Master thesis::** Structural Design of a Steel Footbridge	**Date:** 30.06.2014
	Drawn by: Anna Kur

Steel Footbridge– Bill of materials

Group	Cross section	Weight [kg/m]	Lenght [m]	Number of elements	Weight [t]
1	fi 88,9 x 6,3	12,830	14	2	0,359
2	fi 108 x 8	19,730	10,6	1	0,209
3	fi 51 x 4	4,640	3	3	0,042
4	fi 54 x 4	4,930	3	2	0,030
5	fi 60,3 x 5,6	7,550	2,75	12	0,249
6	fi 60,3 x 5,6	7,550	2,75	4	0,083
7	fi 88,9 x 5	10,350	9	8	0,745
8.1	fi 38 x 4	3,350	2	12	0,080
8.2	fi 38 x 4	3,350	3	10	0,101
9.1	fi 70 x 5	8,010	3,18	8	0,204
9.2	fi 70 x 5	8,010	3,32	16	0,425
9.3	fi 70 x 5	8,010	3,93	16	0,504
9.4	fi 70 x 5	8,010	4,025	8	0,258
10	fi 51 x 4	4,640	4,61	4	0,086
11	HEB 140	33,700	0,14	8	0,038
12	20 x 410 x 410	26,392	-	8	0,211
13	10 x 161 x 221	2,793	-	40	0,112
14	10 x 20 x 120	0,188	-	96	0,018
15	10 x 125 x 146	1,547	-	8	0,012
16	10 x 146 x 166	1,903	-	32	0,061
17	10 x 20 x 100	0,157	-	64	0,010
				Total weight [t]	3,836

steel S235

Subject:	Drawing number:
Steel Footbridge– Bill of materials University of Aveiro	15
	Scale:
Project: Master thesis:: Structural Design of a Steel Footbridge	Date: 30.06.2014
	Drawn by: Anna Kur

Table of Contents:

Design

Elements

All calculations were performed according to the data contained in section 5.1.1. this work.

Group I

Number most strenuous rod: 26

Number combinations: 56

Design value of the normal force: $N_{Ed} = 108{,}02[kN]$

Rod length: $L_{cr} = 3{,}5[m]$

Adopted cross-section: $\emptyset 88{,}9\ x\ 6{,}3$

d=88,9 [mm]

t=6,3 [mm]

Cross-sectional area:

$$A = \pi(0{,}5d)^2 - \pi(0{,}5d - t)^2 = \pi(0{,}5 \cdot 88{,}9[mm])^2 - \pi(0{,}5 \cdot 88{,}9[mm] - 6{,}3[mm])^2 = 16{,}35[cm^2]$$

Second moment of area:

$$I = \pi \cdot \frac{d^4 - (d-2t)^4}{64} = \pi \cdot \frac{(88{,}9[mm])^4 - (88{,}9[mm] - 2 \cdot 6{,}3[mm])^4}{64} = 140{,}236[cm^4]$$

Cross-section requirements for plastic global analysis:

Factor:

$$\varepsilon_s = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{f_y}} = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{235\left[\frac{N}{mm^2}\right]}} = 1[-] \text{ (table 5.2)}$$

The ratio of cross-sectional diameter to its thickness:

$$\frac{d}{t} = \frac{88{,}9[mm]}{6{,}3[mm]} = 14{,}11[-] \text{ (table 5.2)}$$

Section in bending and/or compression:

$$\frac{d}{t} = 14{,}11[-] \le 50\varepsilon_s^2 = 50 \cdot 1[-]^2 = 50[-] \text{ (table 5.2)}$$

So the section is a Class 1 (table 5.2).

If axial compression section is class 1, the in bending and compression is also a Class 1.

The design resistance of the bar in compression

The design resistance to normal forces of the cross-section for uniform compression (for class 1 cross-sections):

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{16{,}35[cm^2]\cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 384{,}225[kN] \text{ (formula 6.10)}$$

Condition section capacity in compression:

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{108{,}02[kN]}{384{,}225[kN]} = 0{,}28 < 1{,}0 \text{ (formula 6.9)}$$

Condition fulfilled.

Buckling resistance of members

Buckling curves:

c (table 6.2)

Imperfection factors for buckling curves:

$$\propto = 0{,}49[-] \text{ (table 6.1)}$$

The elastic critical force for the relevant buckling mode based on the gross cross sectional properties:

$$N_{cr} = \pi^2 \cdot E \cdot \frac{I}{L_{cr}^2} = \pi^2 \cdot 210000\left[\frac{N}{mm^2}\right] \cdot \frac{140{,}236[cm^4]}{(3{,}5[m])^2} = 237{,}270[kN]$$

The parameter of appropriate non-dimensional slenderness:

$$\bar{\lambda} = \sqrt{\frac{A\cdot f_y}{N_{cr}}} = \sqrt{\frac{16{,}35[cm^2]\cdot 235\left[\frac{N}{mm^2}\right]}{237{,}270[kN]}} = 1{,}272[-] \text{ (according 6.3.1.2 (1))}$$

$$\Phi = 0{,}5\left[1+\propto\left(\bar{\lambda}-0{,}2\right)+\bar{\lambda}^2\right] = 0{,}5[1+0{,}49[-](1{,}272[-]-0{,}2)+(1{,}272[-])^2] = 1{,}572[-] \text{ (according 6.3.1.2 (1))}$$

The reduction factor for relevant buckling mode:

$$\chi = \frac{1}{\Phi+\sqrt{\Phi^2-\bar{\lambda}^2}} = \frac{1}{1{,}572[-]+\sqrt{(1{,}572[-])^2-(1{,}272[-])^2}} = 0{,}401[-] \leq 1{,}0[-] \text{ (formula 6.49)}$$

The design buckling resistance of a compression member:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0{,}401[-] \cdot 16{,}35[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 154{,}074[kN] \text{ (formula 6.47)}$$

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{108{,}02[kN]}{154{,}074[kN]} = 0{,}701[-] \leq 1{,}0[-] \text{ (formula 6.46)}$$

Condition fulfilled.

Cross-section chosen properly.

Group III

Number most strenuous rod:	35
Number combinations:	63
Design value of the normal force:	$N_{Ed} = -22{,}83[kN]$
Rod length:	$L_{cr} = 3{,}0[m]$
Adopted cross-section:	$\emptyset 51x4$
	d=51,0[mm]
	t=4,0[mm]

Cross-sectional area:

$$A = \pi(0{,}5d)^2 - \pi(0{,}5d - t)^2 = \pi(0{,}5 \cdot 51[mm])^2 - \pi(0{,}5 \cdot 51[mm] - 4[mm])^2 = 5{,}91[cm^2]$$

Second moment of area:

$$I = \pi \cdot \frac{d^4 - (d-2t)^4}{64} = \pi \cdot \frac{(51[mm])^4 - (51[mm] - 2 \cdot 4[mm])^4}{64} = 16{,}427[cm^4]$$

Cross-section requirements for plastic global analysis.

Factor:

$$\varepsilon_s = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{f_y}} = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{235\left[\frac{N}{mm^2}\right]}} = 1[-] \text{ (table 5.2)}$$

The ratio of cross-sectional diameter to its thickness:

$$\frac{d}{t}=\frac{51[mm]}{4[mm]}=12{,}75[-] \text{ (table 5.2)}$$

Section in bending and/or compression:

$$\frac{d}{t}=12{,}75[-]\leq 50\varepsilon_s^2=50\cdot 1[-]^2=50[-] \text{ (table 5.2)}$$

So the section is a Class 1 (table 5.2).

If axial compression section is class 1, the in bending and compression is also a Class 1.

The design resistance of the bar in compression

The design resistance to normal forces of the cross-section for uniform compression (for class 1 cross-sections):

$$N_{c,Rd}=\frac{Af_y}{\gamma_{M0}}=\frac{5{,}91[cm^2]\cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]}=138{,}80[kN] \text{ (formula 6.10)}$$

Condition section capacity in compression:

$$\frac{N_{Ed}}{N_{c,Rd}}=\frac{22{,}83[kN]}{138{,}80[kN]}=0{,}16<1{,}0 \text{ (formula 6.9)}$$

Condition fulfilled.

Buckling resistance of members

Buckling curves:

c (table 6.2)

Imperfection factors for buckling curves:

$$\propto=0{,}49[-] \text{ (table 6.1)}$$

The elastic critical force for the relevant buckling mode based on the gross cross sectional properties:

$$N_{cr}=\pi^2\cdot E\cdot\frac{I}{L_{cr}^2}=\pi^2\cdot 210000\left[\frac{N}{mm^2}\right]\cdot\frac{16{,}427[cm^4]}{(3{,}0[m])^2}=37{,}83[kN]$$

The parameter of appropriate non-dimensional slenderness:

$$\bar{\lambda}=\sqrt{\frac{A\cdot f_y}{N_{cr}}}=\sqrt{\frac{5{,}91[cm^2]\cdot 235\left[\frac{N}{mm^2}\right]}{37{,}83[kN]}}=1{,}915[-] \text{ (according 6.3.1.2 (1))}$$

$$\Phi = 0{,}5\left[1 + \propto \left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] = 0{,}5[1 + 0{,}49[-]\ (1{,}915[-] - 0{,}2) + (1{,}915[-])^2] = 2{,}755[-]\ \text{(according 6.3.1.2 (1))}$$

The reduction factor for relevant buckling mode :

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{2{,}755[-] + \sqrt{(2{,}755[-])^2 - (1{,}915[-])^2}} = 0{,}211[-] \leq 1{,}0[-]\ \text{(formula 6.49)}$$

The design buckling resistance of a compression member:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0{,}211[-] \cdot 5{,}91[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 29{,}31[kN]\ \text{(formula 6.47)}$$

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{22{,}83[kN]]}{29{,}31[kN]} = 0{,}779[-] \leq 1{,}0[-]\ \text{(formula 6.46)}$$

Condition fulfilled.

Cross-section chosen properly.

Group IV

Number most strenuous rod: 23

Number combinations: 37

Design value of the normal force: $N_{Ed} = 25{,}06[kN]$

Rod length: $L_{cr} = 3{,}0[m]$

Adopted cross-section: $\emptyset 54x4$

d=54,0[mm]

t=4,0[mm]

Cross-sectional area:

$$A = \pi(0{,}5d)^2 - \pi(0{,}5d - t)^2 = \pi(0{,}5 \cdot 54[mm])^2 - \pi(0{,}5 \cdot 54[mm] - 4[mm])^2 = 6{,}28[cm^2]$$

Second moment of area:

$$I = \pi \cdot \frac{d^4 - (d - 2t)^4}{64} = \pi \cdot \frac{(54[mm])^4 - (54[mm] - 2 \cdot 4[mm])^4}{64} = 19{,}761[cm^4]$$

Cross-section requirements for plastic global analysis.

Factor:

$$\varepsilon_s = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{f_y}} = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{235\left[\frac{N}{mm^2}\right]}} = 1[-] \text{ (table 5.2)}$$

The ratio of cross-sectional diameter to its thickness:

$$\frac{d}{t} = \frac{54[mm]}{4[mm]} = 13{,}5[-] \text{ (table 5.2)}$$

Section in bending and/or compression:

$$\frac{d}{t} = 13{,}5[-] \leq 50\varepsilon_s^2 = 50 \cdot 1[-]^2 = 50[-] \text{ (table 5.2)}$$

So the section is a Class 1 (table 5.2).

If axial compression section is class 1, the in bending and compression is also a Class 1.

The design resistance of the bar in compression

The design resistance to normal forces of the cross-section for uniform compression (for class 1 cross-sections):

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{6{,}28[cm^2]\cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 147{,}65[kN] \text{ (formula 6.10)}$$

Condition section capacity in compression:

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{25{,}06[kN]}{147{,}65[kN]} = 0{,}17 < 1{,}0 \text{ (formula 6.9)}$$

Condition fulfilled.

Buckling resistance of members

Buckling curves:

c (table 6.2)

Imperfection factors for buckling curves:

$$\propto = 0{,}49[-] \text{ (table 6.1)}$$

The elastic critical force for the relevant buckling mode based on the gross cross sectional properties:

$$N_{cr} = \pi^2 \cdot E \cdot \frac{I}{L_{cr}^2} = \pi^2 \cdot 210000\left[\frac{N}{mm^2}\right] \cdot \frac{19,761\left[cm^4\right]}{(3,0[m])^2} = 45,51[kN]$$

The parameter of appropriate non-dimensional slenderness:

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{6,28[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{45,51[kN]}} = 1,801[-] \text{ (according 6.3.1.2 (1))}$$

$$\Phi = 0,5\left[1 + \propto \left(\bar{\lambda} - 0,2\right) + \bar{\lambda}^2\right] = 0,5[1 + 0,49[-]\ (1,801[-] - 0,2) + (1,801[-])^2] = 2,515[-] \text{ (according 6.3.1.2 (1))}$$

The reduction factor for relevant buckling mode:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{2,515[-] + \sqrt{(2,515[-])^2 - (1,801[-])^2}} = 0,234[-] \leq 1,0[-] \text{ (formula 6.49)}$$

The design buckling resistance of a compression member:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0,234[-] \cdot 6,28\left[cm^2\right] \cdot 235\left[\frac{N}{mm^2}\right]}{1,0[-]} = 34,58[kN] \text{ (formula 6.47)}$$

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{26,01[kN]]}{34,58[kN]} = 0,752[-] \leq 1,0[-] \text{ (formula 6.46)}$$

Condition fulfilled.

Cross-section chosen properly.

Group V

Number most strenuous rod:	54
Number combinations:	63
Design value of the normal force:	$N_{Ed} = 57,22[kN]$
Rod length:	$L_{cr} = 2,75[m]$
Adopted cross-section:	$\emptyset 60,3x5,6$
	d=60,3[mm]
	t=5,6[mm]

Cross-sectional area:

$$A = \pi(0{,}5d)^2 - \pi(0{,}5d - t)^2 = \pi(0{,}5 \cdot 60{,}3[mm])^2 - \pi(0{,}5 \cdot 60{,}3[mm] - 5{,}6[mm])^2 = 9{,}62[cm^2]$$

Second moment of area:

$$I = \pi \cdot \frac{d^4 - (d-2t)^4}{64} = \pi \cdot \frac{(60{,}3[mm])^4 - (60{,}3[mm] - 2 \cdot 5{,}6[mm])^4}{64} = 36{,}369[cm^4]$$

Cross-section requirements for plastic global analysis.

Factor:

$$\varepsilon_s = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{f_y}} = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{235\left[\frac{N}{mm^2}\right]}} = 1[-] \text{ (table 5.2)}$$

The ratio of cross-sectional diameter to its thickness:

$$\frac{d}{t} = \frac{60{,}3[mm]}{5{,}6[mm]} = 10{,}77[-] \text{ (table 5.2)}$$

Section in bending and/or compression:

$$\frac{d}{t} = 10{,}77[-] \leq 50\varepsilon_s^2 = 50 \cdot 1[-]^2 = 50[-] \text{ (table 5.2)}$$

So the section is a Class 1 (table 5.2).

If axial compression section is class 1, the in bending and compression is also a Class 1.

The design resistance of the bar in compression

The design resistance to normal forces of the cross-section for uniform compression (for class 1 cross-sections):

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{9{,}62[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 226{,}15[kN] \text{ (formula 6.10)}$$

Condition section capacity in compression:

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{57{,}22[kN]}{226{,}15[kN]} = 0{,}25 < 1{,}0 \text{ (formula 6.9)}$$

Condition fulfilled.

Buckling resistance of members

Buckling curves:

c (table 6.2)

Imperfection factors for buckling curves:

$\propto = 0{,}49[-]$ (table 6.1)

The elastic critical force for the relevant buckling mode based on the gross cross sectional properties:

$$N_{cr} = \pi^2 \cdot E \cdot \frac{I}{L_{cr}^2} = \pi^2 \cdot 210000\left[\frac{N}{mm^2}\right] \cdot \frac{19{,}761[cm^4]}{(2{,}75[m])^2} = 99{,}68[kN]$$

The parameter of appropriate non-dimensional slenderness:

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{9{,}62[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{99{,}68[kN]}} = 1{,}506[-] \text{ (according 6.3.1.2 (1))}$$

$$\Phi = 0{,}5\left[1 + \propto\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] = 0{,}5[1 + 0{,}49[-]\,(1{,}506[-] - 0{,}2) + (1{,}506[-])^2] = 1{,}954[-] \text{ (according 6.3.1.2 (1))}$$

The reduction factor for relevant buckling mode:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1{,}954[-] + \sqrt{(1{,}954[-])^2 - (1{,}506[-])^2}} = 0{,}312[-] \le 1{,}0[-] \text{ (formula 6.49)}$$

The design buckling resistance of a compression member:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0{,}312[-] \cdot 9{,}62[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 70{,}67[kN] \text{ (formula 6.47)}$$

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{57{,}22[kN]]}{70{,}67[kN]} = 0{,}810[-] \le 1{,}0[-] \text{ (formula 6.46)}$$

Condition fulfilled.

Cross-section chosen properly.

Group VI

Number most strenuous rod: 55

Number combinations: 63

Design value of the normal force: $N_{Ed} = -59{,}12[kN]$

Rod length: $L_{cr} = 2{,}75[m]$

Adopted cross-section: $\emptyset 60{,}3x5{,}6$

d=60,3[mm]

t=5,6[mm]

Cross-sectional area:

$$A = \pi(0{,}5d)^2 - \pi(0{,}5d - t)^2 = \pi(0{,}5 \cdot 60{,}3[mm])^2 - \pi(0{,}5 \cdot 60{,}3[mm] - 5{,}6[mm])^2 = 9{,}62[cm^2]$$

Second moment of area:

$$I = \pi \cdot \frac{d^4 - (d-2t)^4}{64} = \pi \cdot \frac{(60{,}3[mm])^4 - (60{,}3[mm] - 2 \cdot 5{,}6[mm])^4}{64} = 36{,}369[cm^4]$$

Cross-section requirements for plastic global analysis.

Factor:

$$\varepsilon_s = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{f_y}} = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{235\left[\frac{N}{mm^2}\right]}} = 1[-] \text{ (table 5.2)}$$

The ratio of cross-sectional diameter to its thickness:

$$\frac{d}{t} = \frac{60{,}3[mm]}{5{,}6[mm]} = 10{,}77[-] \text{ (table 5.2)}$$

Section in bending and/or compression:

$$\frac{d}{t} = 10{,}77[-] \leq 50\varepsilon_s^2 = 50 \cdot 1[-]^2 = 50[-] \text{ (table 5.2)}$$

So the section is a Class 1 (table 5.2).

If axial compression section is class 1, the in bending and compression is also a Class 1.

The design resistance of the bar in compression

The design resistance to normal forces of the cross-section for uniform compression (for class 1 cross-sections):

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{9{,}62[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 226{,}15[kN] \text{ (formula 6.10)}$$

Condition section capacity in compression:

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{59{,}12[kN]}{226{,}15[kN]} = 0{,}26 < 1{,}0 \text{ (formula 6.9)}$$

Condition fulfilled.

Buckling resistance of members

Buckling curves:

c (table 6.2)

Imperfection factors for buckling curves:

$$\propto = 0{,}49[-] \text{ (table 6.1)}$$

The elastic critical force for the relevant buckling mode based on the gross cross sectional properties:

$$N_{cr} = \pi^2 \cdot E \cdot \frac{I}{L_{cr}^2} = \pi^2 \cdot 210000 \left[\frac{N}{mm^2}\right] \cdot \frac{19{,}761[cm^4]}{(2{,}75[m])^2} = 99{,}68[kN]$$

The parameter of appropriate non-dimensional slenderness:

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{9{,}62[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{99{,}68[kN]}} = 1{,}506[-] \text{ (according 6.3.1.2 (1))}$$

$$\Phi = 0{,}5\left[1 + \propto(-0{,}2) + \bar{\lambda}^2\right] = 0{,}5[1 + 0{,}49[-]\,(1{,}506[-] - 0{,}2) + (1{,}506[-])^2] = 1{,}954[-]$$
(according 6.3.1.2 (1))

The reduction factor for relevant buckling mode :

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1{,}954[-] + \sqrt{(1{,}954[-])^2 - (1{,}506[-])^2}} = 0{,}312[-] \leq 1{,}0[-] \text{ (formula 6.49)}$$

The design buckling resistance of a compression member:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0{,}312[-] \cdot 9{,}62[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 70{,}67[kN] \text{ (formula 6.47)}$$

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{59{,}12[kN]]}{70{,}67[kN]} = 0{,}836[-] \leq 1{,}0[-] \text{ (formula 6.46)}$$

Condition fulfilled.

Cross-section chosen properly.

Group VII

Number most strenuous rod: 11

Number combinations: 48

Design value of the normal force: $N_{Ed} = 142{,}72[kN]$

Rod length: $L_{cr} = 3{,}0[m]$

Adopted cross-section: $\emptyset 88{,}9x5$

d=88,9[mm]

t=5,0[mm]

Cross-sectional area:

$$A = \pi(0{,}5d)^2 - \pi(0{,}5d - t)^2 = \pi(0{,}5 \cdot 88{,}9[mm])^2 - \pi(0{,}5 \cdot 88{,}9[mm] - 5{,}0[mm])^2 = 13{,}18[cm^2]$$

Second moment of area:

$$I = \pi \cdot \frac{d^4 - (d-2t)^4}{64} = \pi \cdot \frac{(88{,}9[mm])^4 - (88{,}9[mm] - 2 \cdot 5{,}0[mm])^4}{64} = 116{,}37[cm^4]$$

Cross-section requirements for plastic global analysis.

Factor:

$$\varepsilon_s = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{f_y}} = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{235\left[\frac{N}{mm^2}\right]}} = 1[-] \text{ (table 5.2)}$$

The ratio of cross-sectional diameter to its thickness:

$$\frac{d}{t} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \text{ (table 5.2)}$$

Section in bending and/or compression:

$$\frac{d}{t} = 17{,}78[-] \leq 50\varepsilon_s^2 = 50 \cdot 1[-]^2 = 50[-] \text{ (table 5.2)}$$

So the section is a Class 1 (table 5.2).

If axial compression section is class 1, the in bending and compression is also a Class 1.

The design resistance of the bar in compression

The design resistance to normal forces of the cross-section for uniform compression (for class 1 cross-sections):

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{13{,}18[cm^2]\cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 309{,}71[kN] \text{ (formula 6.10)}$$

Condition section capacity in compression:

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{142{,}72[kN]}{309{,}71[kN]} = 0{,}46 < 1{,}0 \text{ (formula 6.9)}$$

Condition fulfilled.

Buckling resistance of members

Buckling curves:

c (table 6.2)

Imperfection factors for buckling curves:

$$\propto = 0{,}49[-] \text{ (table 6.1)}$$

The elastic critical force for the relevant buckling mode based on the gross cross sectional properties:

$$N_{cr} = \pi^2 \cdot E \cdot \frac{I}{L_{cr}^2} = \pi^2 \cdot 210000\left[\frac{N}{mm^2}\right] \cdot \frac{116{,}37[cm^4]}{(3{,}0[m])^2} = 267{,}99[kN]$$

The parameter of appropriate non-dimensional slenderness:

$$\bar{\lambda} = \sqrt{\frac{A\cdot f_y}{N_{cr}}} = \sqrt{\frac{13{,}18[cm^2]\cdot 235\left[\frac{N}{mm^2}\right]}{267{,}99[kN]}} = 1{,}075[-] \text{ (according 6.3.1.2 (1))}$$

$$\Phi = 0{,}5\left[1+\propto\left(\bar{\lambda}-0{,}2\right)+\bar{\lambda}^2\right] = 0{,}5[1+0{,}49[-]\,(1{,}075[-]-0{,}2)+(1{,}075[-])^2] = 1{,}292[-] \text{ (according 6.3.1.2 (1))}$$

The reduction factor for relevant buckling mode :

$$\chi = \frac{1}{\Phi+\sqrt{\Phi^2-\bar{\lambda}^2}} = \frac{1}{1{,}292[-]+\sqrt{(1{,}292[-])^2-(1{,}075[-])^2}} = 0{,}498[-] \leq 1{,}0[-] \text{ (formula 6.49)}$$

The design buckling resistance of a compression member:

$$N_{b,Rd} = \frac{\chi\cdot A\cdot f_y}{\gamma_{M1}} = \frac{0{,}498[-]\cdot 13{,}18[cm^2]\cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 154{,}14[kN] \text{ (formula 6.47)}$$

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{142{,}72[kN]]}{154{,}14[kN]} = 0{,}926[-] \leq 1{,}0[-] \text{ (formula 6.46)}$$

Condition fulfilled.

Cross-section chosen properly.

Group VIII

Number most strenuous rod:	23
Number combinations:	52
Design value of the normal force:	$N_{Ed} = 24{,}06[kN]$
Rod length:	$L_{cr} = 2{,}0[m]$
Adopted cross-section:	$\emptyset 38{,}0x4$
	d=38,0[mm]
	t=4,0[mm]

Cross-sectional area:

$$A = \pi(0{,}5d)^2 - \pi(0{,}5d - t)^2 = \pi(0{,}5 \cdot 38{,}0[mm])^2 - \pi(0{,}5 \cdot 38{,}0[mm] - 4{,}0[mm])^2 = 4{,}27[cm^2]$$

Second moment of area:

$$I = \pi \cdot \frac{d^4-(d-2t)^4}{64} = \pi \cdot \frac{(38{,}0[mm])^4-(38{,}0[mm]-2\cdot 4{,}0[mm])^4}{64} = 6{,}259[cm^4]$$

Cross-section requirements for plastic global analysis.

Factor:

$$\varepsilon_s = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{f_y}} = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{235\left[\frac{N}{mm^2}\right]}} = 1[-] \text{ (table 5.2)}$$

The ratio of cross-sectional diameter to its thickness:

$$\frac{d}{t} = \frac{38{,}0[mm]}{4{,}0[mm]} = 9{,}5[-] \text{ (table 5.2)}$$

Section in bending and/or compression:

$$\frac{d}{t} = 9{,}5[-] \leq 50\varepsilon_s^2 = 50 \cdot 1[-]^2 = 50[-] \text{ (table 5.2)}$$

So the section is a Class 1 (table 5.2).

If axial compression section is class 1, the in bending and compression is also a Class 1.

The design resistance of the bar in compression

The design resistance to normal forces of the cross-section for uniform compression (for class 1 cross-sections):

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{4{,}27[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 100{,}41[kN] \text{ (formula 6.10)}$$

Condition section capacity in compression:

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{24{,}06[kN]}{100{,}41[kN]} = 0{,}24 < 1{,}0 \text{ (formula 6.9)}$$

Condition fulfilled.

Buckling resistance of members

Buckling curves:

c (table 6.2)

Imperfection factors for buckling curves:

$$\propto = 0{,}49[-] \text{ (table 6.1)}$$

The elastic critical force for the relevant buckling mode based on the gross cross sectional properties:

$$N_{cr} = \pi^2 \cdot E \cdot \frac{I}{L_{cr}^2} = \pi^2 \cdot 210000\left[\frac{N}{mm^2}\right] \cdot \frac{6{,}259[cm^4]}{(3{,}0[m])^2} = 32{,}43[kN]$$

The parameter of appropriate non-dimensional slenderness:

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{4{,}27[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{32{,}43[kN]}} = 1{,}759[-] \text{ (according 6.3.1.2 (1))}$$

$$\Phi = 0{,}5\left[1 + \propto \left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] = 0{,}5[1 + 0{,}49[-]\,(1{,}759[-] - 0{,}2) + (1{,}759[-])^2] = 2{,}430[-] \text{ (according 6.3.1.2 (1))}$$

The reduction factor for relevant buckling mode :

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{2{,}430[-] + \sqrt{(2{,}430[-])^2 - (1{,}759[-])^2}} = 0{,}243[-] \leq 1{,}0[-] \text{ (formula 6.49)}$$

The design buckling resistance of a compression member:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0{,}243[-] \cdot 4{,}27[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 24{,}45[kN] \text{ (formula 6.47)}$$

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{24{,}06[kN]]}{24{,}45[kN]} = 0{,}984[-] \leq 1{,}0[-] \text{ (formula 6.46)}$$

Condition fulfilled.

Cross-section chosen properly.

Group IX

Number most strenuous rod:	166
Number combinations:	102
Design value of the normal force:	$N_{Ed} = 60{,}10[kN]$
Rod length:	$L_{cr} = 3{,}61[m]$
Adopted cross-section:	$\emptyset 70{,}0x5$
	d=70,0[mm]
	t=5,0[mm]

Cross-sectional area:

$$A = \pi(0{,}5d)^2 - \pi(0{,}5d - t)^2 = \pi(0{,}5 \cdot 70{,}0[mm])^2 - \pi(0{,}5 \cdot 70{,}0[mm] - 5{,}0[mm])^2 = 10{,}21[cm^2]$$

Second moment of area:

$$I = \pi \cdot \frac{d^4 - (d-2t)^4}{64} = \pi \cdot \frac{(70{,}0[mm])^4 - (70{,}0[mm] - 2 \cdot 5{,}0[mm])^4}{64} = 54{,}24[cm^4]$$

Cross-section requirements for plastic global analysis.

Factor:

$$\varepsilon_s = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{f_y}} = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{235\left[\frac{N}{mm^2}\right]}} = 1[-] \text{ (table 5.2)}$$

The ratio of cross-sectional diameter to its thickness:

$$\frac{d}{t} = \frac{70,0[mm]}{5,0[mm]} = 14,0[-] \text{ (table 5.2)}$$

Section in bending and/or compression:

$$\frac{d}{t} = 14,0[-] \leq 50\varepsilon_s^2 = 50 \cdot 1[-]^2 = 50[-] \text{ (table 5.2)}$$

So the section is a Class 1 (table 5.2).

If axial compression section is class 1, the in bending and compression is also a Class 1.

The design resistance of the bar in compression

The design resistance to normal forces of the cross-section for uniform compression (for class 1 cross-sections):

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{10,21[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1,0[-]} = 239,94[kN] \text{ (formula 6.10)}$$

Condition section capacity in compression:

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{60,10[kN]}{239,94[kN]} = 0,25 < 1,0 \text{ (formula 6.9)}$$

Condition fulfilled.

Buckling resistance of members

Buckling curves:

c (table 6.2)

Imperfection factors for buckling curves:

$$\propto = 0,49[-] \text{ (table 6.1)}$$

The elastic critical force for the relevant buckling mode based on the gross cross sectional properties:

$$N_{cr} = \pi^2 \cdot E \cdot \frac{I}{L_{cr}^2} = \pi^2 \cdot 210000 \left[\frac{N}{mm^2}\right] \cdot \frac{54{,}24[cm^4]}{(3{,}61[m])^2} = 86{,}26[kN]$$

The parameter of appropriate non-dimensional slenderness:

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{10{,}21[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{86{,}26[kN]}} = 1{,}668[-] \text{ (according 6.3.1.2 (1))}$$

$$\Phi = 0{,}5\left[1 + \propto \left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] = 0{,}5[1 + 0{,}49[-]\ (1{,}668[-] - 0{,}2) + (1{,}668[-])^2] = 2{,}250[-] \text{ (according 6.3.1.2 (1))}$$

The reduction factor for relevant buckling mode :

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{2{,}250[-] + \sqrt{(2{,}250[-])^2 - (1{,}668[-])^2}} = 0{,}266[-] \leq 1{,}0[-] \text{ (formula 6.49)}$$

The design buckling resistance of a compression member:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0{,}266[-] \cdot 10{,}21[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 63{,}80[kN] \text{ (formula 6.47)}$$

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{60{,}10[kN]]}{63{,}80[kN]} = 0{,}942[-] \leq 1{,}0[-] \text{ (formula 6.46)}$$

Condition fulfilled.

Cross-section chosen properly.

Group X

Number most strenuous rod:	8
Number combinations:	121
Design value of the normal force:	$N_{Ed} = -8{,}93[kN]$
Rod length:	$L_{cr} = 4{,}61[m]$
Adopted cross-section:	$\emptyset 51x4$
	d=51,0[mm]
	t=4,0[mm]

Cross-sectional area:

$$A = \pi(0{,}5d)^2 - \pi(0{,}5d - t)^2 = \pi(0{,}5 \cdot 51[mm])^2 - \pi(0{,}5 \cdot 51[mm] - 4[mm])^2 = 5{,}91[cm^2]$$

Second moment of area:

$$I = \pi \cdot \frac{d^4 - (d-2t)^4}{64} = \pi \cdot \frac{(51[mm])^4 - (51[mm] - 2 \cdot 4[mm])^4}{64} = 16{,}427[cm^4]$$

Cross-section requirements for plastic global analysis.

Factor:

$$\varepsilon_s = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{f_y}} = \sqrt{\frac{235\left[\frac{N}{mm^2}\right]}{235\left[\frac{N}{mm^2}\right]}} = 1[-] \text{ (table 5.2)}$$

The ratio of cross-sectional diameter to its thickness:

$$\frac{d}{t} = \frac{51[mm]}{4[mm]} = 12{,}75[-] \text{ (table 5.2)}$$

Section in bending and/or compression:

$$\frac{d}{t} = 12{,}75[-] \leq 50\varepsilon_s^2 = 50 \cdot 1[-]^2 = 50[-] \text{ (table 5.2)}$$

So the section is a Class 1 (table 5.2).

If axial compression section is class 1, the in bending and compression is also a Class 1.

The design resistance of the bar in compression

The design resistance to normal forces of the cross-section for uniform compression (for class 1 cross-sections):

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{5{,}91[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 138{,}80[kN] \text{ (formula 6.10)}$$

Condition section capacity in compression:

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{8{,}93[kN]}{138{,}80[kN]} = 0{,}06 < 1{,}0 \text{ (formula 6.9)}$$

Condition fulfilled.

Buckling resistance of members

Buckling curves:

c (table 6.2)

Imperfection factors for buckling curves:

$$\propto = 0{,}49[-] \text{ (table 6.1)}$$

The elastic critical force for the relevant buckling mode based on the gross cross sectional properties:

$$N_{cr} = \pi^2 \cdot E \cdot \frac{I}{L_{cr}^2} = \pi^2 \cdot 210000 \left[\frac{N}{mm^2}\right] \cdot \frac{16{,}427[cm^4]}{(4{,}61[m])^2} = 16{,}02[kN]$$

The parameter of appropriate non-dimensional slenderness:

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{5{,}91[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{16{,}02[kN]}} = 2{,}943[-] \text{ (according 6.3.1.2 (1))}$$

$$\Phi = 0{,}5\left[1 + \propto \left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right] = 0{,}5[1 + 0{,}49[-]\ (2{,}943[-] - 0{,}2) + (2{,}943[-])^2] = 5{,}504[-] \text{ (according 6.3.1.2 (1))}$$

The reduction factor for relevant buckling mode :

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{5{,}504[-] + \sqrt{(5{,}504[-])^2 - (2{,}943[-])^2}} = 0{,}098[-] \leq 1{,}0[-] \text{ (formula 6.49)}$$

The design buckling resistance of a compression member:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = \frac{0{,}098[-] \cdot 5{,}91[cm^2] \cdot 235\left[\frac{N}{mm^2}\right]}{1{,}0[-]} = 13{,}67[kN] \text{ (formula 6.47)}$$

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{8{,}93[kN]]}{13{,}67[kN]} = 0{,}653[-] \leq 1{,}0[-] \text{ (formula 6.46)}$$

Condition fulfilled.

Cross-section chosen properly.

Case	Load type	List of bars	Load (start) [x / l]	Load (end) [x / l]	Force					
1:STA1	self-weight	1to9 11to72 75to80 126to169 238 240to242	Whole structure		-Z	Factor=1,00	MEMO:			
2:balustrades	trapezoidal load (2p)	25	X1=2,00	X2=3,50	PX1=0,0	PY1=0,0	PZ1=-0,47	PX2=0,0	PY2=0,0	PZ2=-0,47
2:balustrades	trapezoidal load (2p)	32	X1=0,0	X2=1,50	PX1=0,0	PY1=0,0	PZ1=-0,47	PX2=0,0	PY2=0,0	PZ2=-0,47
2:balustrades	uniform load	26to31	PX=0,0	not project.	PY=0,0	PZ=-0,47	global	absolute	AL=0,0	BE=0,0
3:surface	uniform load	25to32	PX=0,0	not project.	PY=0,0	PZ=-0,66	global	absolute	AL=0,0	BE=0,0
4:stairs	trapezoidal load (2p)	25	X1=0,0	X2=2,00	PX1=0,0	PY1=0,0	PZ1=-0,94	PX2=0,0	PY2=0,0	PZ2=-0,94
4:stairs	trapezoidal load (2p)	32	X1=1,50	X2=3,50	PX1=0,0	PY1=0,0	PZ1=-0,94	PX2=0,0	PY2=0,0	PZ2=-0,94
5:snow	uniform load	15 20 25to32 139 144	PX=0,0	not project.	PY=0,0	PZ=-1,08	global	absolute	AL=0,0	BE=0,0
6:wind X	uniform load	29to32 38to40	PX=0,0	not project.	PY=0,67	PZ=0,0	global	absolute	AL=0,0	BE=0,0
7:wind Y extern	trapezoidal load (2p)	7 238 240	X1=0,0	X2=0,40	PX1=3,73	PY1=0,0	PZ1=0,0	PX2=3,73	PY2=0,0	PZ2=0,0
7:wind Y extern	trapezoidal load (2p)	7 238 240	X1=0,40	X2=2,00	PX1=2,49	PY1=0,0	PZ1=0,0	PX2=2,49	PY2=0,0	PZ2=0,0
7:wind Y extern	trapezoidal load (2p)	7 238 240	X1=2,00	X2=3,00	PX1=1,56	PY1=0,0	PZ1=0,0	PX2=1,56	PY2=0,0	PZ2=0,0
8:wind Z	uniform load	38to40	PX=0,0	not project.	PY=0,0	PZ=1,87	global	absolut	AL=0,	BE=0,0

								e	0	
8:wind Z	uniform load	41 42 55 56	PX=0,0	not project.	PY= 0,0	PZ= 0,93	glob al	abs olut e	AL =0, 0	BE= 0,0
9:people	uniform load	25to32	PX=0,0	not project.	PY= 0,0	PZ=- 7,50	glob al	abs olut e	AL =0, 0	BE= 0,0
10:force	uniform load	25to32	PX=0,75	not project.	PY= 0,0	PZ= 0,0	glob al	abs olut e	AL =0, 0	BE= 0,0
11:snow-exceptional load	uniform load	15 20 25to32 139 144	PX=0,0	not project.	PY= 0,0	PZ=- 2,16	glob al	abs olut e	AL =0, 0	BE= 0,0
12:wind X-lift external	uniform load	1to3 141to143	PX=0,0	not project.	PY= 0,49	PZ= 0,0	glob al	abs olut e	AL =0, 0	BE= 0,0
12:wind X-lift external	uniform load	17to19 126to128	PX=0,0	not project.	PY= 0,77	PZ= 0,0	glob al	abs olut e	AL =0, 0	BE= 0,0
13:wind X-lift internal	uniform load	1to3 17to19 126to128 141to143	PX=0,0	not project.	PY= 0,21	PZ= 0,0	glob al	abs olut e	AL =0, 0	BE= 0,0
14:TEMP	thermal load	1to7 11to32 34to61 63to72 75to80 126to169 238 240to242	TX=0,0		TY= 5,00	TZ= 0,0	ME MO:			
15:wind Y-internal	uniform load	1to6	PX=0,31	not project.	PY= 0,0	PZ= 0,0	glob al	abs olut e	AL =0, 0	BE= 0,0
16:concentra ted load	bar force	35	FX=0,0	CY=0,0	FY= 0,0	FZ=- 10,0 0	CX =0,0	CZ =0, 0	X= 0,5 0	glob al
17:TEMP1-heating	thermal load	1to7 11to32 34to61 63to72 75to80 126to169 238 240to242	TX=53,00		TY= 0,0	TZ= 0,0	ME MO:			
17:TEMP1-heating	thermal load		TX=45,00		TY= 0,0	TZ= 0,0	ME MO:			

18:TEMP2-cooling	thermal load	1to7 11to32 34to61 63to72 75to80 126to169 238 240to242	TX=-39,00		TY= 0,0	TZ= 0,0	ME MO:			
18:TEMP2-cooling	thermal load		TX=-47,00		TY= 0,0	TZ= 0,0	ME MO:			

Case	Mode	Frequency (Hz)	Period (sec)	Total mass UX (kg)	Total mass UY (kg)	Total mass UZ (kg)
1	1	2,63	0,38	789,71	789,71	789,71
1	2	6,04	0,17	789,71	789,71	789,71
1	3	10,01	0,1	789,71	789,71	789,71
1	4	22,97	0,04	789,71	789,71	789,71
1	5	27,29	0,04	789,71	789,71	789,71
1	6	28,59	0,03	789,71	789,71	789,71
1	7	29,46	0,03	789,71	789,71	789,71
1	8	30,26	0,03	789,71	789,71	789,71
1	9	30,48	0,03	789,71	789,71	789,71
1	10	30,6	0,03	789,71	789,71	789,71
2	1	2,61	0,38	789,71	789,71	789,71
2	2	6,03	0,17	789,71	789,71	789,71
2	3	10	0,1	789,71	789,71	789,71
2	4	22,99	0,04	789,71	789,71	789,71
2	5	27,38	0,04	789,71	789,71	789,71
2	6	28,69	0,03	789,71	789,71	789,71
2	7	29,47	0,03	789,71	789,71	789,71
2	8	30,27	0,03	789,71	789,71	789,71
2	9	30,49	0,03	789,71	789,71	789,71
2	10	30,62	0,03	789,71	789,71	789,71
3	1	2,6	0,38	789,71	789,71	789,71
3	2	6,03	0,17	789,71	789,71	789,71
3	3	9,98	0,1	789,71	789,71	789,71
3	4	22,99	0,04	789,71	789,71	789,71
3	5	27,43	0,04	789,71	789,71	789,71
3	6	28,74	0,03	789,71	789,71	789,71
3	7	29,5	0,03	789,71	789,71	789,71
3	8	30,3	0,03	789,71	789,71	789,71
3	9	30,52	0,03	789,71	789,71	789,71
3	10	30,65	0,03	789,71	789,71	789,71
4	1	2,63	0,38	789,71	789,71	789,71

4	2	6,04	0,17	789,71	789,71	789,71
4	3	10,03	0,1	789,71	789,71	789,71
4	4	22,96	0,04	789,71	789,71	789,71
4	5	27,23	0,04	789,71	789,71	789,71
4	6	28,52	0,04	789,71	789,71	789,71
4	7	29,42	0,03	789,71	789,71	789,71
4	8	30,22	0,03	789,71	789,71	789,71
4	9	30,43	0,03	789,71	789,71	789,71
4	10	30,56	0,03	789,71	789,71	789,71
5	1	2,59	0,39	789,71	789,71	789,71
5	2	6,01	0,17	789,71	789,71	789,71
5	3	9,95	0,1	789,71	789,71	789,71
5	4	23,01	0,04	789,71	789,71	789,71
5	5	27,56	0,04	789,71	789,71	789,71
5	6	28,86	0,03	789,71	789,71	789,71
5	7	29,55	0,03	789,71	789,71	789,71
5	8	30,35	0,03	789,71	789,71	789,71
5	9	30,58	0,03	789,71	789,71	789,71
5	10	30,71	0,03	789,71	789,71	789,71
6	1	2,6	0,38	789,71	789,71	789,71
6	2	6,03	0,17	789,71	789,71	789,71
6	3	10,02	0,1	789,71	789,71	789,71
6	4	22,95	0,04	789,71	789,71	789,71
6	5	27,14	0,04	789,71	789,71	789,71
6	6	28,43	0,04	789,71	789,71	789,71
6	7	29,41	0,03	789,71	789,71	789,71
6	8	30,21	0,03	789,71	789,71	789,71
6	9	30,39	0,03	789,71	789,71	789,71
6	10	30,58	0,03	789,71	789,71	789,71
7	1	2,63	0,38	789,71	789,71	789,71
7	2	6,05	0,17	789,71	789,71	789,71
7	3	10,03	0,1	789,71	789,71	789,71
7	4	22,97	0,04	789,71	789,71	789,71

7	5	27,24	0,04	789,71	789,71	789,71
7	6	28,53	0,04	789,71	789,71	789,71
7	7	29,42	0,03	789,71	789,71	789,71
7	8	30,22	0,03	789,71	789,71	789,71
7	9	30,43	0,03	789,71	789,71	789,71
7	10	30,55	0,03	789,71	789,71	789,71
8	1	2,58	0,39	789,71	789,71	789,71
8	2	6,04	0,17	789,71	789,71	789,71
8	3	10,07	0,1	789,71	789,71	789,71
8	4	22,95	0,04	789,71	789,71	789,71
8	5	27,23	0,04	789,71	789,71	789,71
8	6	28,54	0,04	789,71	789,71	789,71
8	7	29,28	0,03	789,71	789,71	789,71
8	8	30,11	0,03	789,71	789,71	789,71
8	9	30,28	0,03	789,71	789,71	789,71
8	10	30,43	0,03	789,71	789,71	789,71
9	1	2,25	0,45	789,71	789,71	789,71
9	2	5,78	0,17	789,71	789,71	789,71
9	3	9,48	0,11	789,71	789,71	789,71
9	4	23,22	0,04	789,71	789,71	789,71
9	5	29,37	0,03	789,71	789,71	789,71
9	6	30,3	0,03	789,71	789,71	789,71
9	7	30,62	0,03	789,71	789,71	789,71
9	8	31,14	0,03	789,71	789,71	789,71
9	9	31,46	0,03	789,71	789,71	789,71
9	10	31,74	0,03	789,71	789,71	789,71
10	1	2,63	0,38	789,71	789,71	789,71
10	2	6,05	0,17	789,71	789,71	789,71
10	3	10,03	0,1	789,71	789,71	789,71
10	4	22,96	0,04	789,71	789,71	789,71
10	5	27,24	0,04	789,71	789,71	789,71
10	6	28,53	0,04	789,71	789,71	789,71
10	7	29,4	0,03	789,71	789,71	789,71

10	8	30,21	0,03	789,71	789,71	789,71
10	9	30,39	0,03	789,71	789,71	789,71
10	10	30,61	0,03	789,71	789,71	789,71
11	1	2,54	0,39	789,71	789,71	789,71
11	2	5,98	0,17	789,71	789,71	789,71
11	3	9,87	0,1	789,71	789,71	789,71
11	4	23,06	0,04	789,71	789,71	789,71
11	5	27,87	0,04	789,71	789,71	789,71
11	6	29,18	0,03	789,71	789,71	789,71
11	7	29,68	0,03	789,71	789,71	789,71
11	8	30,48	0,03	789,71	789,71	789,71
11	9	30,73	0,03	789,71	789,71	789,71
11	10	30,87	0,03	789,71	789,71	789,71
12	1	2,63	0,38	789,71	789,71	789,71
12	2	6,05	0,17	789,71	789,71	789,71
12	3	10,03	0,1	789,71	789,71	789,71
12	4	22,97	0,04	789,71	789,71	789,71
12	5	27,24	0,04	789,71	789,71	789,71
12	6	28,53	0,04	789,71	789,71	789,71
12	7	29,42	0,03	789,71	789,71	789,71
12	8	30,22	0,03	789,71	789,71	789,71
12	9	30,43	0,03	789,71	789,71	789,71
12	10	30,55	0,03	789,71	789,71	789,71
13	1	2,63	0,38	789,71	789,71	789,71
13	2	6,05	0,17	789,71	789,71	789,71
13	3	10,03	0,1	789,71	789,71	789,71
13	4	22,97	0,04	789,71	789,71	789,71
13	5	27,24	0,04	789,71	789,71	789,71
13	6	28,53	0,04	789,71	789,71	789,71
13	7	29,42	0,03	789,71	789,71	789,71
13	8	30,22	0,03	789,71	789,71	789,71
13	9	30,43	0,03	789,71	789,71	789,71
13	10	30,55	0,03	789,71	789,71	789,71

14	10	N/A	N/A	N/A	N/A	N/A
14	9	N/A	N/A	N/A	N/A	N/A
14	8	N/A	N/A	N/A	N/A	N/A
14	7	N/A	N/A	N/A	N/A	N/A
14	6	N/A	N/A	N/A	N/A	N/A
14	5	N/A	N/A	N/A	N/A	N/A
14	4	N/A	N/A	N/A	N/A	N/A
14	3	N/A	N/A	N/A	N/A	N/A
14	2	N/A	N/A	N/A	N/A	N/A
14	1	N/A	N/A	N/A	N/A	N/A
15	1	2,63	0,38	789,71	789,71	789,71
15	2	6,05	0,17	789,71	789,71	789,71
15	3	10,03	0,1	789,71	789,71	789,71
15	4	22,97	0,04	789,71	789,71	789,71
15	5	27,24	0,04	789,71	789,71	789,71
15	6	28,53	0,04	789,71	789,71	789,71
15	7	29,42	0,03	789,71	789,71	789,71
15	8	30,22	0,03	789,71	789,71	789,71
15	9	30,43	0,03	789,71	789,71	789,71
15	10	30,55	0,03	789,71	789,71	789,71
16	1	2,6	0,38	789,71	789,71	789,71
16	2	6,05	0,17	789,71	789,71	789,71
16	3	9,98	0,1	789,71	789,71	789,71
16	4	23	0,04	789,71	789,71	789,71
16	5	27,51	0,04	789,71	789,71	789,71
16	6	28,82	0,03	789,71	789,71	789,71
16	7	29,51	0,03	789,71	789,71	789,71
16	8	30,31	0,03	789,71	789,71	789,71
16	9	30,52	0,03	789,71	789,71	789,71
16	10	30,66	0,03	789,71	789,71	789,71
17	1	N/A	N/A	N/A	N/A	N/A
17	2	N/A	N/A	N/A	N/A	N/A
17	10	N/A	N/A	N/A	N/A	N/A

17	9	N/A	N/A	N/A	N/A	N/A
17	8	N/A	N/A	N/A	N/A	N/A
17	7	N/A	N/A	N/A	N/A	N/A
17	6	N/A	N/A	N/A	N/A	N/A
17	5	N/A	N/A	N/A	N/A	N/A
17	4	N/A	N/A	N/A	N/A	N/A
17	3	N/A	N/A	N/A	N/A	N/A
18	10	N/A	N/A	N/A	N/A	N/A
18	9	N/A	N/A	N/A	N/A	N/A
18	8	N/A	N/A	N/A	N/A	N/A
18	7	N/A	N/A	N/A	N/A	N/A
18	6	N/A	N/A	N/A	N/A	N/A
18	5	N/A	N/A	N/A	N/A	N/A
18	4	N/A	N/A	N/A	N/A	N/A
18	3	N/A	N/A	N/A	N/A	N/A
18	2	N/A	N/A	N/A	N/A	N/A
18	1	N/A	N/A	N/A	N/A	N/A

Case	Mode	Frequency (Hz)	Period (sec)	Total mass UX (kg)	Total mass UY (kg)	Total mass UZ (kg)
1	1	17,52	0,06	853,85	853,85	853,85
1	2	18,67	0,05	853,85	853,85	853,85
1	3	20,23	0,05	853,85	853,85	853,85
1	4	20,41	0,05	853,85	853,85	853,85
1	1	20,52	0,05	853,85	853,85	853,85
1	1	20,53	0,05	853,85	853,85	853,85
1	1	20,53	0,05	853,85	853,85	853,85
1	1	20,53	0,05	853,85	853,85	853,85
1	1	20,53	0,05	853,85	853,85	853,85
1	1	20,53	0,05	853,85	853,85	853,85
2	1	20,53	0,05	853,85	853,85	853,85
2	1	20,53	0,05	853,85	853,85	853,85
2	1	20,53	0,05	853,85	853,85	853,85
2	1	20,54	0,05	853,85	853,85	853,85
2	1	20,54	0,05	853,85	853,85	853,85
2	1	20,54	0,05	853,85	853,85	853,85
2	1	20,54	0,05	853,85	853,85	853,85
2	1	20,54	0,05	853,85	853,85	853,85
2	1	20,54	0,05	853,85	853,85	853,85
2	1	20,55	0,05	853,85	853,85	853,85
3	1	21,7	0,05	853,85	853,85	853,85
3	5	22,04	0,05	853,85	853,85	853,85
3	2	22,4	0,04	853,85	853,85	853,85
3	2	22,4	0,04	853,85	853,85	853,85
3	2	22,41	0,04	853,85	853,85	853,85
3	2	22,41	0,04	853,85	853,85	853,85
3	2	22,41	0,04	853,85	853,85	853,85
3	2	22,41	0,04	853,85	853,85	853,85
3	2	22,41	0,04	853,85	853,85	853,85
3	2	22,41	0,04	853,85	853,85	853,85
4	2	22,42	0,04	853,85	853,85	853,85

4	2	22,42	0,04	853,85	853,85	853,85
4	2	22,43	0,04	853,85	853,85	853,85
4	2	22,43	0,04	853,85	853,85	853,85
4	2	22,44	0,04	853,85	853,85	853,85
4	2	22,44	0,04	853,85	853,85	853,85
4	2	22,47	0,04	853,85	853,85	853,85
4	6	22,56	0,04	853,85	853,85	853,85
4	2	22,59	0,04	853,85	853,85	853,85
4	7	23,13	0,04	853,85	853,85	853,85
5	2	23,14	0,04	853,85	853,85	853,85
5	8	24,04	0,04	853,85	853,85	853,85
5	3	26,91	0,04	853,85	853,85	853,85
5	3	27,03	0,04	853,85	853,85	853,85
5	3	27,06	0,04	853,85	853,85	853,85
5	3	27,06	0,04	853,85	853,85	853,85
5	3	27,06	0,04	853,85	853,85	853,85
5	3	27,06	0,04	853,85	853,85	853,85
5	3	27,06	0,04	853,85	853,85	853,85
5	3	27,06	0,04	853,85	853,85	853,85
6	3	27,06	0,04	853,85	853,85	853,85
6	3	27,12	0,04	853,85	853,85	853,85
6	3	27,19	0,04	853,85	853,85	853,85
6	3	27,23	0,04	853,85	853,85	853,85
6	3	27,3	0,04	853,85	853,85	853,85
6	3	27,34	0,04	853,85	853,85	853,85
6	3	27,58	0,04	853,85	853,85	853,85
6	4	28	0,04	853,85	853,85	853,85
6	4	28,01	0,04	853,85	853,85	853,85
6	4	28,03	0,04	853,85	853,85	853,85
7	4	28,08	0,04	853,85	853,85	853,85
7	4	28,08	0,04	853,85	853,85	853,85
7	4	28,08	0,04	853,85	853,85	853,85
7	4	28,08	0,04	853,85	853,85	853,85

7	4	28,08	0,04	853,85	853,85	853,85
7	4	28,08	0,04	853,85	853,85	853,85
7	4	28,12	0,04	853,85	853,85	853,85
7	4	28,16	0,04	853,85	853,85	853,85
7	4	28,19	0,04	853,85	853,85	853,85
7	4	28,22	0,04	853,85	853,85	853,85
8	4	28,26	0,04	853,85	853,85	853,85
8	5	28,4	0,04	853,85	853,85	853,85
8	3	28,43	0,04	853,85	853,85	853,85
8	4	28,45	0,04	853,85	853,85	853,85
8	5	28,53	0,04	853,85	853,85	853,85
8	5	28,55	0,04	853,85	853,85	853,85
8	5	28,56	0,04	853,85	853,85	853,85
8	5	28,56	0,04	853,85	853,85	853,85
8	5	28,56	0,04	853,85	853,85	853,85
8	5	28,56	0,04	853,85	853,85	853,85
9	5	28,56	0,04	853,85	853,85	853,85
9	5	28,59	0,03	853,85	853,85	853,85
9	5	28,61	0,03	853,85	853,85	853,85
9	5	28,68	0,03	853,85	853,85	853,85
9	5	28,72	0,03	853,85	853,85	853,85
9	5	28,78	0,03	853,85	853,85	853,85
9	5	28,81	0,03	853,85	853,85	853,85
9	6	28,84	0,03	853,85	853,85	853,85
9	6	28,93	0,03	853,85	853,85	853,85
9	6	28,93	0,03	853,85	853,85	853,85
10	6	28,93	0,03	853,85	853,85	853,85
10	6	28,93	0,03	853,85	853,85	853,85
10	6	28,93	0,03	853,85	853,85	853,85
10	6	28,93	0,03	853,85	853,85	853,85
10	6	28,93	0,03	853,85	853,85	853,85
10	6	28,94	0,03	853,85	853,85	853,85
10	6	28,97	0,03	853,85	853,85	853,85

10	5	29,01	0,03	853,85	853,85	853,85
10	6	29,01	0,03	853,85	853,85	853,85
10	6	29,04	0,03	853,85	853,85	853,85
11	6	29,09	0,03	853,85	853,85	853,85
11	6	29,13	0,03	853,85	853,85	853,85
11	6	29,38	0,03	853,85	853,85	853,85
11	4	29,52	0,03	853,85	853,85	853,85
11	5	29,84	0,03	853,85	853,85	853,85
11	7	30,32	0,03	853,85	853,85	853,85
11	7	30,4	0,03	853,85	853,85	853,85
11	7	30,42	0,03	853,85	853,85	853,85
11	7	30,45	0,03	853,85	853,85	853,85
11	7	30,45	0,03	853,85	853,85	853,85
12	7	30,45	0,03	853,85	853,85	853,85
12	7	30,45	0,03	853,85	853,85	853,85
12	7	30,45	0,03	853,85	853,85	853,85
12	7	30,46	0,03	853,85	853,85	853,85
12	8	30,49	0,03	853,85	853,85	853,85
12	7	30,5	0,03	853,85	853,85	853,85
12	7	30,52	0,03	853,85	853,85	853,85
12	7	30,55	0,03	853,85	853,85	853,85
12	7	30,56	0,03	853,85	853,85	853,85
12	7	30,61	0,03	853,85	853,85	853,85
13	8	30,64	0,03	853,85	853,85	853,85
13	8	30,64	0,03	853,85	853,85	853,85
13	8	30,64	0,03	853,85	853,85	853,85
13	8	30,64	0,03	853,85	853,85	853,85
13	8	30,64	0,03	853,85	853,85	853,85
13	8	30,65	0,03	853,85	853,85	853,85
13	8	30,69	0,03	853,85	853,85	853,85
13	8	30,69	0,03	853,85	853,85	853,85
13	8	30,69	0,03	853,85	853,85	853,85
13	8	30,7	0,03	853,85	853,85	853,85

14	8	30,73	0,03	853,85	853,85	853,85
14	8	30,74	0,03	853,85	853,85	853,85
14	6	30,76	0,03	853,85	853,85	853,85
14	7	30,77	0,03	853,85	853,85	853,85
14	8	30,79	0,03	853,85	853,85	853,85
14	8	30,94	0,03	853,85	853,85	853,85
14	7	31,62	0,03	853,85	853,85	853,85
14	8	31,66	0,03	853,85	853,85	853,85
14	3	32,22	0,03	853,85	853,85	853,85
14	4	34	0,03	853,85	853,85	853,85
15	5	34,18	0,03	853,85	853,85	853,85
15	6	35,1	0,03	853,85	853,85	853,85
15	7	35,95	0,03	853,85	853,85	853,85
15	8	36,38	0,03	853,85	853,85	853,85
15	9	37,96	0,03	853,85	853,85	853,85
15	9	39,87	0,03	853,85	853,85	853,85
15	9	39,88	0,03	853,85	853,85	853,85
15	9	39,88	0,03	853,85	853,85	853,85
15	9	39,88	0,03	853,85	853,85	853,85
15	9	39,88	0,03	853,85	853,85	853,85
16	9	39,88	0,03	853,85	853,85	853,85
16	9	39,88	0,03	853,85	853,85	853,85
16	9	39,89	0,03	853,85	853,85	853,85
16	9	39,89	0,03	853,85	853,85	853,85
16	9	39,89	0,03	853,85	853,85	853,85
16	9	39,89	0,03	853,85	853,85	853,85
16	9	39,89	0,03	853,85	853,85	853,85
16	9	39,9	0,03	853,85	853,85	853,85
16	9	39,9	0,03	853,85	853,85	853,85
16	9	39,91	0,03	853,85	853,85	853,85
17	9	39,98	0,03	853,85	853,85	853,85
17	10	41,05	0,02	853,85	853,85	853,85
17	10	41,39	0,02	853,85	853,85	853,85

17	10	41,4	0,02	853,85	853,85	853,85
17	10	41,4	0,02	853,85	853,85	853,85
17	10	41,4	0,02	853,85	853,85	853,85
17	10	41,4	0,02	853,85	853,85	853,85
17	10	41,4	0,02	853,85	853,85	853,85
17	10	41,4	0,02	853,85	853,85	853,85
17	10	41,4	0,02	853,85	853,85	853,85
18	10	41,4	0,02	853,85	853,85	853,85
18	10	41,41	0,02	853,85	853,85	853,85
18	10	41,41	0,02	853,85	853,85	853,85
18	10	41,41	0,02	853,85	853,85	853,85
18	10	41,41	0,02	853,85	853,85	853,85
18	10	41,42	0,02	853,85	853,85	853,85
18	10	41,43	0,02	853,85	853,85	853,85
18	10	41,49	0,02	853,85	853,85	853,85
18	9	41,83	0,02	853,85	853,85	853,85
18	10	42,38	0,02	853,85	853,85	853,85

Combinations	Name	Analysis type	Combination nature
25 (C)	SGN/1=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+11*0.75+17*1.50	Linear Combination	ULS
26 (C)	SGN/2=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+10*1.05+15*0.90+18	Linear Combination	ULS
27 (C)	SGN/3=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+11*0.75+17*1.50	Linear Combination	ULS
28 (C)	SGN/4=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+10*1.05+15*0.90+18	Linear Combination	ULS
29 (C)	SGN/5=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+11*1.50+15*0.90+17	Linear Combination	ULS
30 (C)	SGN/6=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+10*1.05+12*1.50+13	Linear Combination	ULS
31 (C)	SGN/7=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+11*0.75+12*0.90+13	Linear Combination	ULS
32 (C)	SGN/8=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+10*1.05+11*1.50+18	Linear Combination	ULS
33 (C)	SGN/9=1*1.00+2*1.00+3*1.00+4*1.00+7*1.50+15*1.50+17*0.90	Linear Combination	ULS
34 (C)	SGN/10=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+12*0.90+13*0.90	Linear Combination	ULS
35 (C)	SGN/11=1*1.35+2*1.35+3*1.35+4*1.35+10*1.05+11*0.75+18*1.50	Linear Combination	ULS
36 (C)	SGN/12=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+11*0.75+16*1.05	Linear Combination	ULS
37 (C)	SGN/13=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+10*1.05+18*1.50	Linear Combination	ULS
38 (C)	SGN/14=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+10*1.05+11*0.75	Linear Combination	ULS
39 (C)	SGN/15=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*0.90 + 17*1.50	Linear Combination	ULS
40 (C)	SGN/16=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+10*1.05+11*1.50	Linear Combination	ULS
41 (C)	SGN/17=1*1.35+2*1.35+3*1.35+4*1.35+7*1.50+10*1.05+11*0.75	Linear Combination	ULS
42 (C)	SGN/18=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 17*1.50	Linear	ULS

		Combination	
43 (C)	SGN/19=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+15*0.90+17*1.50	Linear Combination	ULS
44 (C)	SGN/20=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+10*1.05+11*0.75	Linear Combination	ULS
45 (C)	SGN/21=1*1.00+2*1.00+3*1.00+4*1.00+7*1.50+10*1.05+15*1.50	Linear Combination	ULS
46 (C)	SGN/22=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+10*1.05+12*0.90	Linear Combination	ULS
47 (C)	SGN/23=1*1.35+2*1.35+3*1.35+4*1.35+11*0.75+16*1.05+17*1.50	Linear Combination	ULS
48 (C)	SGN/24=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+10*1.05+11*0.75	Linear Combination	ULS
49 (C)	SGN/25=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+12*1.50+13*1.50	Linear Combination	ULS
50 (C)	SGN/26=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+10*1.05+11*0.75	Linear Combination	ULS
51 (C)	SGN/27=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+15*0.90+17*1.50	Linear Combination	ULS
52 (C)	SGN/28=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+11*1.50+12*0.90	Linear Combination	ULS
53 (C)	SGN/29=1*1.00+2*1.00+3*1.00+4*1.00+8*1.50+10*1.05+18*0.90	Linear Combination	ULS
54 (C)	SGN/30=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*1.50 + 18*0.90	Linear Combination	ULS
55 (C)	SGN/31=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+10*1.05+11*1.50	Linear Combination	ULS
56 (C)	SGN/32=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+11*1.50+16*1.05	Linear Combination	ULS
57 (C)	SGN/33=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*0.90 + 18*1.50	Linear Combination	ULS
58 (C)	SGN/34=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+10*1.05+11*0.75	Linear Combination	ULS
59 (C)	SGN/35=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+11*1.50+16*1.05	Linear Combination	ULS
60 (C)	SGN/36=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+12*1.50+13*1.50	Linear Combination	ULS

61 (C)	SGN/37=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+11*1.50+18*0.90	Linear Combination	ULS
62 (C)	SGN/38=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*1.50 + 17*0.90	Linear Combination	ULS
63 (C)	SGN/39=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+11*1.50+12*0.90	Linear Combination	ULS
64 (C)	SGN/40=1*1.35+2*1.35+3*1.35+4*1.35+11*1.50+16*1.05+17*0.90	Linear Combination	ULS
65 (C)	SGN/41=1*1.35+2*1.35+3*1.35+4*1.35+11*1.50+16*1.05+18*0.90	Linear Combination	ULS
66 (C)	SGN/42=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+11*1.50+16*1.05	Linear Combination	ULS
67 (C)	SGN/43=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+10*1.50+18*0.90	Linear Combination	ULS
68 (C)	SGN/44=1*1.35+2*1.35+3*1.35+4*1.35+7*1.50+16*1.05+15*1.50	Linear Combination	ULS
69 (C)	SGN/45=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+10*1.05+11*0.75	Linear Combination	ULS
70 (C)	SGN/46=1*1.35+2*1.35+3*1.35+4*1.35+7*1.50+15*1.50+17*0.90	Linear Combination	ULS
71 (C)	SGN/47=1*1.00+2*1.00+3*1.00+4*1.00+10*1.05+11*1.50+18*0.90	Linear Combination	ULS
72 (C)	SGN/48=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+10*1.05+11*0.75	Linear Combination	ULS
73 (C)	SGN/49=1*1.00+2*1.00+3*1.00+4*1.00+10*1.05+11*0.75+18*1.50	Linear Combination	ULS
74 (C)	SGN/50=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+12*0.90+13*0.90	Linear Combination	ULS
75 (C)	SGN/51=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+11*0.75+12*1.50	Linear Combination	ULS
76 (C)	SGN/52=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+11*0.75+12*1.50	Linear Combination	ULS
77 (C)	SGN/53=1*1.35+2*1.35+3*1.35+4*1.35+7*1.50+10*1.05+15*1.50	Linear Combination	ULS
78 (C)	SGN/54=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+12*0.90+13*0.90	Linear Combination	ULS
79 (C)	SGN/55=1*1.35+2*1.35+3*1.35+4*1.35+7*1.50+11*0	Linear	ULS

	.75+15*1.50	Combination	
80 (C)	SGN/56=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+11*0.75+12*1.50	Linear Combination	ULS
81 (C)	SGN/57=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+11*0.75+12*0.90	Linear Combination	ULS
82 (C)	SGN/58=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+10*1.05+18*1.50	Linear Combination	ULS
83 (C)	SGN/59=1*1.35+2*1.35+3*1.35+4*1.35+11*0.75+16*1.05+18*1.50	Linear Combination	ULS
84 (C)	SGN/60=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+10*1.05+12*0.90	Linear Combination	ULS
85 (C)	SGN/61=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+10*1.05+17*1.50	Linear Combination	ULS
86 (C)	SGN/62=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+10*1.05+11*0.75	Linear Combination	ULS
87 (C)	SGN/63=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+12*1.50+13*1.50	Linear Combination	ULS
88 (C)	SGN/64=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+11*0.75+12*0.90	Linear Combination	ULS
89 (C)	SGN/65=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+10*1.05+15*0.90	Linear Combination	ULS
90 (C)	SGN/66=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+11*0.75+12*0.90	Linear Combination	ULS
91 (C)	SGN/67=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 16*1.05 + 18*1.50	Linear Combination	ULS
92 (C)	SGN/68=1*1.00+2*1.00+3*1.00+4*1.00+11*0.75+16*1.05+18*1.50	Linear Combination	ULS
93 (C)	SGN/69=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+10*1.05+12*0.90	Linear Combination	ULS
94 (C)	SGN/70=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+12*0.90+13*0.90	Linear Combination	ULS
95 (C)	SGN/71=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+11*0.75+12*0.90	Linear Combination	ULS
96 (C)	SGN/72=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+10*1.05+15*0.90	Linear Combination	ULS
97 (C)	SGN/73=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+12*1.50+13*1.50	Linear Combination	ULS

98 (C)	SGN/74=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+10*1.05+11*1.50	Linear Combination	ULS
99 (C)	SGN/75=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+12*0.90+13*0.90	Linear Combination	ULS
100 (C)	SGN/76=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+10*1.05+11*0.75	Linear Combination	ULS
101 (C)	SGN/77=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+10*1.05+11*0.75	Linear Combination	ULS
102 (C)	SGN/78=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+10*1.05+11*0.75	Linear Combination	ULS
103 (C)	SGN/79=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 16*1.05 + 18*1.50	Linear Combination	ULS
104 (C)	SGN/80=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+10*1.05+12*1.50	Linear Combination	ULS
105 (C)	SGN/81=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+11*0.75+12*1.50	Linear Combination	ULS
106 (C)	SGN/82=1*1.00+2*1.00+3*1.00+4*1.00+11*1.50+16*1.05+17*0.90	Linear Combination	ULS
107 (C)	SGN/83=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+10*1.05+12*1.50	Linear Combination	ULS
108 (C)	SGN/84=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 11*0.75 + 17*1.50	Linear Combination	ULS
109 (C)	SGN/85=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+10*1.05+11*0.75	Linear Combination	ULS
110 (C)	SGN/86=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 8*0.90 + 17*1.50	Linear Combination	ULS
111 (C)	SGN/87=1*1.35+2*1.35+3*1.35+4*1.35+7*1.50+10*1.05+15*1.50	Linear Combination	ULS
112 (C)	SGN/88=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00	Linear Combination	ULS
113 (C)	SGN/89=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+11*0.75+16*1.05	Linear Combination	ULS
114 (C)	SGN/90=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+11*0.75+12*1.50	Linear Combination	ULS
115 (C)	SGN/91=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+11*0.75+12*1.50	Linear Combination	ULS
116 (C)	SGN/92=1*1.00+2*1.00+3*1.00+4*1.00+7*1.50+10*1	Linear	ULS

	.05+11*0.75	Combination	
117 (C)	SGN/93=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 11*0.75 + 18*1.50	Linear Combination	ULS
118 (C)	SGN/94=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+11*0.75+18*1.50	Linear Combination	ULS
119 (C)	SGN/95=1*1.00+2*1.00+3*1.00+4*1.00+7*1.50+16*1.05+15*1.50	Linear Combination	ULS
120 (C)	SGN/96=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+11*0.75+16*1.05	Linear Combination	ULS
121 (C)	SGN/97=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+10*1.05+12*1.50	Linear Combination	ULS
122 (C)	SGN/98=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+11*0.75+15*0.90	Linear Combination	ULS
123 (C)	SGN/99=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35	Linear Combination	ULS
124 (C)	SGN/100=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+11*0.75+15*0.90	Linear Combination	ULS
125 (C)	SGN/101=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 16*1.05 + 17*1.50	Linear Combination	ULS
126 (C)	SGN/102=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+10*1.05+11*0.75	Linear Combination	ULS
127 (C)	SGN/103=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+12*1.50+13*1.50	Linear Combination	ULS
128 (C)	SGN/104=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+11*0.75+12*0.90	Linear Combination	ULS
129 (C)	SGN/105=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+15*0.90+18*1.50	Linear Combination	ULS
130 (C)	SGN/106=1*1.00+2*1.00+3*1.00+4*1.00+7*1.50+11*0.75+15*1.50	Linear Combination	ULS
131 (C)	SGN/107=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+10*1.05+11*0.75	Linear Combination	ULS
132 (C)	SGN/108=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 10*1.05 + 18*1.50	Linear Combination	ULS
133 (C)	SGN/109=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+12*1.50+13*1.50	Linear Combination	ULS
134 (C)	SGN/110=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 11*0.75 + 18*1.50	Linear Combination	ULS

135 (C)	SGN/111=1*1.00+2*1.00+3*1.00+4*1.00+7*1.50+10*1.05+15*1.50	Linear Combination	ULS
136 (C)	SGN/112=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 11*0.75 + 17*1.50	Linear Combination	ULS
137 (C)	SGN/113=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+11*0.75+16*1.05	Linear Combination	ULS
138 (C)	SGN/114=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 10*1.05 + 17*1.50	Linear Combination	ULS
139 (C)	SGN/115=1*1.00+2*1.00+3*1.00+4*1.00+7*1.50+11*0.75+16*1.05	Linear Combination	ULS
140 (C)	SGN/116=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+11*0.75+16*1.05	Linear Combination	ULS
141 (C)	SGN/117=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+16*1.05+15*0.90	Linear Combination	ULS
142 (C)	SGN/118=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+11*0.75+12*1.50	Linear Combination	ULS
143 (C)	SGN/119=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+16*1.05+15*0.90	Linear Combination	ULS
144 (C)	SGN/120=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+16*1.05+17*1.50	Linear Combination	ULS
145 (C)	SGN/121=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 18*1.50	Linear Combination	ULS
146 (C)	SGN/122=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+11*0.75+12*0.90	Linear Combination	ULS
147 (C)	SGN/123=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+12*1.50+13*1.50	Linear Combination	ULS
148 (C)	SGN/124=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+10*1.05+11*0.75	Linear Combination	ULS
149 (C)	SGN/125=1*1.00+2*1.00+3*1.00+4*1.00+7*1.50+11*0.75+15*1.50	Linear Combination	ULS
150 (C)	SGN/126=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+16*1.05+15*0.90	Linear Combination	ULS
151 (C)	SGN/127=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+11*1.50+18*0.90	Linear Combination	ULS
152 (C)	SGN/128=1*1.35+2*1.35+3*1.35+4*1.35+10*1.50+11*0.75+18*0.90	Linear Combination	ULS
153 (C)	SGN/129=1*1.00+2*1.00+3*1.00+4*1.00+8*1.50+10*	Linear	ULS

	1.05+11*0.75	Combination	
154 (C)	SGN/130=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+12*0.90+13*0.90	Linear Combination	ULS
155 (C)	SGN/131=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+11*1.50+12*0.90	Linear Combination	ULS
156 (C)	SGN/132=1*1.35+2*1.35+3*1.35+4*1.35+8*1.50+10*1.05+17*0.90	Linear Combination	ULS
157 (C)	SGN/133=1*1.00+2*1.00+3*1.00+4*1.00+6*1.50+11*0.75+12*1.50	Linear Combination	ULS
158 (C)	SGN/134=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+10*1.05+17*1.50	Linear Combination	ULS
159 (C)	SGN/135=1*1.00+2*1.00+3*1.00+4*1.00+8*1.50+10*1.05+17*0.90	Linear Combination	ULS
160 (C)	SGN/136=1*1.35+2*1.35+3*1.35+4*1.35+8*1.50+16*1.05+17*0.90	Linear Combination	ULS
161 (C)	SGN/137=1*1.35+2*1.35+3*1.35+4*1.35+7*1.50+10*1.05+11*0.75	Linear Combination	ULS
162 (C)	SGN/138=1*1.35+2*1.35+3*1.35+4*1.35+8*1.50+16*1.05+18*0.90	Linear Combination	ULS
163 (C)	SGN/139=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 10*1.05 + 18*1.50	Linear Combination	ULS
164 (C)	SGN/140=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 10*1.05 + 17*1.50	Linear Combination	ULS
165 (C)	SGN/141=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 11*1.50 + 18*0.90	Linear Combination	ULS
166 (C)	SGN/142=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+16*1.05+18*1.50	Linear Combination	ULS
167 (C)	SGN/143=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+11*0.75+15*0.90	Linear Combination	ULS
168 (C)	SGN/144=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+15*0.90+18*1.50	Linear Combination	ULS
169 (C)	SGN/145=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+10*1.05+11*1.50	Linear Combination	ULS
170 (C)	SGN/146=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+11*0.75+12*0.90	Linear Combination	ULS
171 (C)	SGN/147=1*1.35+2*1.35+3*1.35+4*1.35+8*1.50+10*1.05+11*0.75	Linear Combination	ULS

172 (C)	SGN/148=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+11*1.50+16*1.05	Linear Combination	ULS
173 (C)	SGN/149=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 11*1.50 + 17*0.90	Linear Combination	ULS
174 (C)	SGN/150=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 8*0.90 + 18*1.50	Linear Combination	ULS
175 (C)	SGN/151=1*1.00+2*1.00+3*1.00+4*1.00+8*1.50+16*1.05+17*0.90	Linear Combination	ULS
176 (C)	SGN/152=1*1.35+2*1.35+3*1.35+4*1.35+10*1.05+11*1.50+18*0.90	Linear Combination	ULS
177 (C)	SGN/153=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+12*0.90+13*0.90	Linear Combination	ULS
178 (C)	SGN/154=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+11*0.75+18*1.50	Linear Combination	ULS
179 (C)	SGN/155=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+10*1.50+15*0.90	Linear Combination	ULS
180 (C)	SGN/156=1*1.00+2*1.00+3*1.00+4*1.00+7*0.90+11*1.50+16*1.05	Linear Combination	ULS
181 (C)	SGN/157=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+10*1.05+11*1.50	Linear Combination	ULS
182 (C)	SGN/158=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+16*1.05+18*1.50	Linear Combination	ULS
183 (C)	SGN/159=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+11*1.50+17*0.90	Linear Combination	ULS
184 (C)	SGN/160=1*1.35+2*1.35+3*1.35+4*1.35+6*1.50+12*1.50+13*1.50	Linear Combination	ULS
185 (C)	SGN/161=1*1.35+2*1.35+3*1.35+4*1.35+7*1.50+16*1.05+15*1.50	Linear Combination	ULS
186 (C)	SGN/162=1*1.35+2*1.35+3*1.35+4*1.35+8*1.50+11*0.75+16*1.05	Linear Combination	ULS
187 (C)	SGN/163=1*1.35+2*1.35+3*1.35+4*1.35+7*1.50+11*0.75+16*1.05	Linear Combination	ULS
188 (C)	SGN/164=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+11*0.75+16*1.05	Linear Combination	ULS
189 (C)	SGN/165=1*1.35+2*1.35+3*1.35+4*1.35+10*1.05+11*0.75+17*1.50	Linear Combination	ULS
190 (C)	SGN/166=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+10*	Linear	ULS

	1.50+11*0.75	Combination	
191 (C)	SGN/167=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+10*1.50+17*0.90	Linear Combination	ULS
192 (C)	SGN/168=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 18*1.50	Linear Combination	ULS
193 (C)	SGN/169=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+11*0.75+16*1.05	Linear Combination	ULS
194 (C)	SGN/170=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+10*1.05+12*0.90	Linear Combination	ULS
195 (C)	SGN/171=1*1.00+2*1.00+3*1.00+4*1.00+8*0.90+10*1.50+11*0.75	Linear Combination	ULS
196 (C)	SGN/172=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+10*1.05+11*0.75	Linear Combination	ULS
197 (C)	SGN/173=1*1.35+2*1.35+3*1.35+4*1.35+10*1.05+11*1.50+17*0.90	Linear Combination	ULS
198 (C)	SGN/174=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 16*1.05 + 17*1.50	Linear Combination	ULS
199 (C)	SGN/175=1*1.35+2*1.35+3*1.35+4*1.35+6*0.90+12*0.90+13*0.90	Linear Combination	ULS
200 (C)	SGN/176=1*1.35+2*1.35+3*1.35+4*1.35+7*0.90+16*1.05+15*0.90	Linear Combination	ULS
201 (C)	SGN/177=1*1.00+2*1.00+3*1.00+4*1.00+7*1.50+16*1.05+15*1.50	Linear Combination	ULS
202 (C)	SGN/178=1*1.35 + 2*1.35 + 3*1.35 + 4*1.35 + 10*1.50 + 18*0.90	Linear Combination	ULS
203 (C)	SGN/179=1*1.00+2*1.00+3*1.00+4*1.00+6*0.90+10*1.05+11*1.50	Linear Combination	ULS
204 (C)	SGN/180=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+16*1.50+18*0.90	Linear Combination	ULS
205 (C)	SGN/181=1*1.35+2*1.35+3*1.35+4*1.35+8*0.90+16*1.05+17*1.50	Linear Combination	ULS
206 (C)	SGU/1=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+11*0.50+17*1.00	Linear Combination	SLS
207 (C)	SGU/2=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+10*0.70+15*0.60+18	Linear Combination	SLS
208 (C)	SGU/3=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+11*1.00+15*0.60+17	Linear Combination	SLS

209 (C)	SGU/4=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+10*0.70+12*1.00+13	Linear Combination	SLS
210 (C)	SGU/5=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+11*0.50+12*0.60+13	Linear Combination	SLS
211 (C)	SGU/6=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+10*0.70+11*1.00+18	Linear Combination	SLS
212 (C)	SGU/7=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+15*1.00+17*0.60	Linear Combination	SLS
213 (C)	SGU/8=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+12*0.60+13*0.60+17	Linear Combination	SLS
214 (C)	SGU/9=1*1.00+2*1.00+3*1.00+4*1.00+10*0.70+11*0.50+18*1.00	Linear Combination	SLS
215 (C)	SGU/10=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+11*0.50+16*0.70	Linear Combination	SLS
216 (C)	SGU/11=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+10*0.70+18*1.00	Linear Combination	SLS
217 (C)	SGU/12=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+10*0.70+11*0.50	Linear Combination	SLS
218 (C)	SGU/13=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*0.60 + 17*1.00	Linear Combination	SLS
219 (C)	SGU/14=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+10*0.70+11*1.00	Linear Combination	SLS
220 (C)	SGU/15=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+10*0.70+11*0.50	Linear Combination	SLS
221 (C)	SGU/16=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 17*1.00	Linear Combination	SLS
222 (C)	SGU/17=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+15*0.60+17*1.00	Linear Combination	SLS
223 (C)	SGU/18=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+10*0.70+11*0.50	Linear Combination	SLS
224 (C)	SGU/19=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+10*0.70+15*1.00	Linear Combination	SLS
225 (C)	SGU/20=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+10*0.70+12*0.60	Linear Combination	SLS
226 (C)	SGU/21=1*1.00+2*1.00+3*1.00+4*1.00+11*0.50+16*0.70+17*1.00	Linear Combination	SLS
227 (C)	SGU/22=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+10*0	Linear	SLS

	.70+11*0.50	Combination	
228 (C)	SGU/23=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+12*1.00+13*1.00	Linear Combination	SLS
229 (C)	SGU/24=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+10*0.70+11*0.50	Linear Combination	SLS
230 (C)	SGU/25=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+11*1.00+12*0.60	Linear Combination	SLS
231 (C)	SGU/26=1*1.00+2*1.00+3*1.00+4*1.00+8*1.00+10*0.70+18*0.60	Linear Combination	SLS
232 (C)	SGU/27=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*1.00 + 18*0.60	Linear Combination	SLS
233 (C)	SGU/28=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+10*0.70+11*1.00	Linear Combination	SLS
234 (C)	SGU/29=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+11*1.00+16*0.70	Linear Combination	SLS
235 (C)	SGU/30=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*0.60 + 18*1.00	Linear Combination	SLS
236 (C)	SGU/31=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+10*0.70+11*0.50	Linear Combination	SLS
237 (C)	SGU/32=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+11*1.00+16*0.70	Linear Combination	SLS
238 (C)	SGU/33=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+12*1.00+13*1.00	Linear Combination	SLS
239 (C)	SGU/34=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+11*1.00+18*0.60	Linear Combination	SLS
240 (C)	SGU/35=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*1.00 + 17*0.60	Linear Combination	SLS
241 (C)	SGU/36=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+11*1.00+12*0.60	Linear Combination	SLS
242 (C)	SGU/37=1*1.00+2*1.00+3*1.00+4*1.00+11*1.00+16*0.70+17*0.60	Linear Combination	SLS
243 (C)	SGU/38=1*1.00+2*1.00+3*1.00+4*1.00+11*1.00+16*0.70+18*0.60	Linear Combination	SLS
244 (C)	SGU/39=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+11*1.00+16*0.70	Linear Combination	SLS
245 (C)	SGU/40=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+10*1.00+18*0.60	Linear Combination	SLS

246 (C)	SGU/41=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+16*0.70+15*1.00	Linear Combination	SLS
247 (C)	SGU/42=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+10*0.70+11*0.50	Linear Combination	SLS
248 (C)	SGU/43=1*1.00+2*1.00+3*1.00+4*1.00+10*0.70+11*1.00+18*0.60	Linear Combination	SLS
249 (C)	SGU/44=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+11*0.50+12*1.00	Linear Combination	SLS
250 (C)	SGU/45=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+12*0.60+13*0.60	Linear Combination	SLS
251 (C)	SGU/46=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+11*0.50+15*1.00	Linear Combination	SLS
252 (C)	SGU/47=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+11*0.50+12*1.00	Linear Combination	SLS
253 (C)	SGU/48=1*1.00+2*1.00+3*1.00+4*1.00+11*0.50+16*0.70+18*1.00	Linear Combination	SLS
254 (C)	SGU/49=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+10*0.70+12*0.60	Linear Combination	SLS
255 (C)	SGU/50=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+10*0.70+17*1.00	Linear Combination	SLS
256 (C)	SGU/51=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+12*1.00+13*1.00	Linear Combination	SLS
257 (C)	SGU/52=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+11*0.50+12*0.60	Linear Combination	SLS
258 (C)	SGU/53=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+10*0.70+15*0.60	Linear Combination	SLS
259 (C)	SGU/54=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+11*0.50+12*0.60	Linear Combination	SLS
260 (C)	SGU/55=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 16*0.70 + 18*1.00	Linear Combination	SLS
261 (C)	SGU/56=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+12*0.60+13*0.60	Linear Combination	SLS
262 (C)	SGU/57=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+12*1.00+13*1.00	Linear Combination	SLS
263 (C)	SGU/58=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+10*0.70+11*0.50	Linear Combination	SLS
264 (C)	SGU/59=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+10*0	Linear	SLS

	.70+12*1.00	Combination	
265 (C)	SGU/60=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+11*0.50+12*1.00	Linear Combination	SLS
266 (C)	SGU/61=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 11*0.50 + 17*1.00	Linear Combination	SLS
267 (C)	SGU/62=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+10*0.70+11*0.50	Linear Combination	SLS
268 (C)	SGU/63=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+10*0.70+15*1.00	Linear Combination	SLS
269 (C)	SGU/64=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 16*0.50	Linear Combination	SLS
270 (C)	SGU/65=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+11*0.50+16*0.70	Linear Combination	SLS
271 (C)	SGU/66=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*0.20	Linear Combination	SLS
272 (C)	SGU/67=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+10*0.70+11*0.50	Linear Combination	SLS
273 (C)	SGU/68=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 11*0.50 + 18*1.00	Linear Combination	SLS
274 (C)	SGU/69=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+11*0.50+18*1.00	Linear Combination	SLS
275 (C)	SGU/70=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+16*0.70+15*1.00	Linear Combination	SLS
276 (C)	SGU/71=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+11*0.50+15*0.60	Linear Combination	SLS
277 (C)	SGU/72=1*1.00+2*1.00+3*1.00+4*1.00+6*0.20+10*0.30+12*0.20	Linear Combination	SLS
278 (C)	SGU/73=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+11*0.50+15*0.60	Linear Combination	SLS
279 (C)	SGU/74=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 11*0.20 + 16*0.30	Linear Combination	SLS
280 (C)	SGU/75=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 16*0.70 + 17*1.00	Linear Combination	SLS
281 (C)	SGU/76=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+11*0.50+12*0.60	Linear Combination	SLS
282 (C)	SGU/77=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+15*0.60+18*1.00	Linear Combination	SLS

283 (C)	SGU/78=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+11*0.50+15*1.00	Linear Combination	SLS
284 (C)	SGU/79=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 10*0.70 + 18*1.00	Linear Combination	SLS
285 (C)	SGU/80=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 10*0.50	Linear Combination	SLS
286 (C)	SGU/81=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 10*0.30 + 11*0.20	Linear Combination	SLS
287 (C)	SGU/82=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+11*0.50+16*0.70	Linear Combination	SLS
288 (C)	SGU/83=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 10*0.70 + 17*1.00	Linear Combination	SLS
289 (C)	SGU/84=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+11*0.50+16*0.70	Linear Combination	SLS
290 (C)	SGU/85=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+16*0.70+15*0.60	Linear Combination	SLS
291 (C)	SGU/86=1*1.00+2*1.00+3*1.00+4*1.00+6*1.00+11*0.50+12*1.00	Linear Combination	SLS
292 (C)	SGU/87=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+16*0.70+15*0.60	Linear Combination	SLS
293 (C)	SGU/88=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+16*0.70+17*1.00	Linear Combination	SLS
294 (C)	SGU/89=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 18*1.00	Linear Combination	SLS
295 (C)	SGU/90=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 7*0.20 + 15*0.20	Linear Combination	SLS
296 (C)	SGU/91=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 11*0.20	Linear Combination	SLS
297 (C)	SGU/92=1*1.00+2*1.00+3*1.00+4*1.00+8*1.00+10*0.70+11*0.50	Linear Combination	SLS
298 (C)	SGU/93=1*1.00+2*1.00+3*1.00+4*1.00+8*1.00+10*0.70+17*0.60	Linear Combination	SLS
299 (C)	SGU/94=1*1.00+2*1.00+3*1.00+4*1.00+7*0.20+10*0.30+15*0.20	Linear Combination	SLS
300 (C)	SGU/95=1*1.00+2*1.00+3*1.00+4*1.00+8*1.00+16*0.70+17*0.60	Linear Combination	SLS
301 (C)	SGU/96=1*1.00+2*1.00+3*1.00+4*1.00+8*1.00+16*0	Linear	SLS

	.70+18*0.60	Combination	
302 (C)	SGU/97=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 11*1.00 + 18*0.60	Linear Combination	SLS
303 (C)	SGU/98=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+16*0.70+18*1.00	Linear Combination	SLS
304 (C)	SGU/99=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+10*0.70+11*1.00	Linear Combination	SLS
305 (C)	SGU/100=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+11*0.50+12*0.60	Linear Combination	SLS
306 (C)	SGU/101=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+11*1.00+16*0.70	Linear Combination	SLS
307 (C)	SGU/102=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 11*1.00 + 17*0.60	Linear Combination	SLS
308 (C)	SGU/103=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+12*0.60+13*0.60	Linear Combination	SLS
309 (C)	SGU/104=1*1.00+2*1.00+3*1.00+4*1.00+7*0.60+10*1.00+15*0.60	Linear Combination	SLS
310 (C)	SGU/105=1*1.00+2*1.00+3*1.00+4*1.00+6*0.60+10*0.70+11*1.00	Linear Combination	SLS
311 (C)	SGU/106=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+11*1.00+17*0.60	Linear Combination	SLS
312 (C)	SGU/107=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*0.20 + 16*0.30	Linear Combination	SLS
313 (C)	SGU/108=1*1.00+2*1.00+3*1.00+4*1.00+8*1.00+11*0.50+16*0.70	Linear Combination	SLS
314 (C)	SGU/109=1*1.00+2*1.00+3*1.00+4*1.00+7*1.00+11*0.50+16*0.70	Linear Combination	SLS
315 (C)	SGU/110=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+11*0.50+16*0.70	Linear Combination	SLS
316 (C)	SGU/111=1*1.00+2*1.00+3*1.00+4*1.00+10*0.70+11*0.50+17*1.00	Linear Combination	SLS
317 (C)	SGU/112=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+10*1.00+11*0.50	Linear Combination	SLS
318 (C)	SGU/113=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+10*1.00+17*0.60	Linear Combination	SLS
319 (C)	SGU/114=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+10*1.00+11*0.50	Linear Combination	SLS

320 (C)	SGU/115=1*1.00+2*1.00+3*1.00+4*1.00+10*0.70+11*1.00+17*0.60	Linear Combination	SLS
321 (C)	SGU/116=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00	Linear Combination	SLS
322 (C)	SGU/117=1*1.00+2*1.00+3*1.00+4*1.00+7*0.20+16*0.30+15*0.20	Linear Combination	SLS
323 (C)	SGU/118=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 10*1.00 + 18*0.60	Linear Combination	SLS
324 (C)	SGU/119=1*1.00+2*1.00+3*1.00+4*1.00+6*0.20+12*0.20+13*0.20	Linear Combination	SLS
325 (C)	SGU/120=1*1.00+2*1.00+3*1.00+4*1.00+8*0.60+16*1.00+18*0.60	Linear Combination	SLS
326 (C)	SGU/121=1*1.00 + 2*1.00 + 3*1.00 + 4*1.00 + 8*0.20 + 10*0.30	Linear Combination	SLS

Bar	Case	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
1	48 (C)	-88,11	0,06	-0,52	0,19	-0,34	-0,06
2	26 (C)	-30,33	0,04	-0,41	-0,01	-0,23	-0,06
3	29 (C)	4,13	0,11	0,08	0,07	0,09	0,14
4	48 (C)	-88,65	-0,02	-0,51	-0,21	-0,31	-0,05
5	26 (C)	-31,49	-0,08	-0,4	0,04	-0,23	0,12
6	33 (C)	-7,17	0,02	-0,7	0,05	-0,32	-0,1
7	26 (C)	5,05	-3,71	0,05	0	-0,02	-1,6
8	121 (C)	-8,95	-0,05	0,11	-0,01	-0,03	-0,18
9	69 (C)	2,61	0,01	0,15	0,01	-0,17	0,02
11	48 (C)	142,59	-0,04	-0,01	0,01	0,06	0,06
12	40 (C)	53,63	-0,02	0,06	-0,01	-0,16	-0,04
13	40 (C)	7,69	-0,12	0,34	-0,09	-0,28	-0,11
14	35 (C)	-18,41	0	0,04	0	-0,01	0
15	48 (C)	2,99	0,16	1,66	-0,01	-0,53	0,13
16	47 (C)	-13,8	0	0,05	0	-0,02	0
17	48 (C)	140,42	0,04	-0,01	-0,01	0,05	-0,07
18	40 (C)	51,74	0,04	0,01	0	-0,11	0,04
19	41 (C)	10,13	0,35	0,15	0,16	-0,08	0,53
20	41 (C)	4,3	-0,27	1,66	0	-0,53	-0,21
21	45 (C)	-0,95	0	0,06	0	-0,03	-0,01
22	38 (C)	-22,3	0,25	0,07	0,01	-0,04	0,24
23	52 (C)	24,06	-0,09	0,02	0	0,03	-0,12
24	55 (C)	-19,81	0	0,07	0	-0,03	0
25	36 (C)	62,05	0	6,68	-0,09	0	0
26	56 (C)	108,02	0	8,6	0	0	0
27	56 (C)	107,49	0	8,6	0	0	0
28	58 (C)	63,24	0	-5,77	0,09	0	0
29	36 (C)	59,53	0	5,77	0	0	0
30	56 (C)	106,33	0	8,6	0,03	0	0
31	56 (C)	106,6	0	8,6	-0,04	0	0
32	58 (C)	63,18	0	-6,68	-0,01	0	0
33	75 (C)	2,89	-0,01	0,1	-0,01	0	-0,01
34	29 (C)	-17,58	0	-0,09	0	0	0
35	52 (C)	-22,83	0	-5,34	0	0	0

36	29 (C)	-17,94	0	0,09	0	0	0
37	52 (C)	24,01	0,09	0,11	0	-0,1	0,13
38	52 (C)	-141,71	-1,06	0,46	0,03	0	0
39	59 (C)	-196,97	0	0,46	0	0	0
40	52 (C)	-141,71	-1,06	0,46	-0,03	0	0
41	64 (C)	-56,42	0	0,12	0,02	0	0
42	63 (C)	-58,84	0	-0,12	-0,01	0	0
43	64 (C)	55,5	0	0,12	-0,02	0	0
44	63 (C)	56,94	0	0,12	0,01	0	0
45	63 (C)	-24,72	0	0,12	-0,01	0	0
46	65 (C)	-22,12	0	-0,12	0	0	0
47	63 (C)	20,8	0	-0,12	0,01	0	0
48	59 (C)	21,68	0	-0,12	0,01	0	0
49	56 (C)	21,42	0	-0,12	0	0	0
50	65 (C)	21,06	0	0,12	-0,01	0	0
51	63 (C)	-24,33	0	-0,12	0,01	0	0
52	56 (C)	-22,76	0	-0,12	0	0	0
53	56 (C)	55,47	0	-0,12	0,02	0	0
54	63 (C)	57,22	0	0,12	-0,01	0	0
55	63 (C)	-59,12	0	-0,12	0,01	0	0
56	56 (C)	-56,39	0	-0,12	-0,02	0	0
57	68 (C)	1,15	0,49	0,24	0	-0,2	1,31
58	121 (C)	0,95	-0,16	-0,12	0,16	0,16	-0,18
59	38 (C)	2,29	-0,08	-0,12	-0,03	-0,06	0,26
60	35 (C)	1,92	-0,03	-0,17	-0,01	-0,11	0,04
61	41 (C)	-9,4	-0,33	-0,04	-0,17	0,11	0,44
62	49 (C)	-8,25	0,06	0,13	0,01	-0,13	0,06
63	79 (C)	10,24	0,29	-0,14	-0,24	-0,1	-0,34
64	77 (C)	-6,73	0,33	0,11	0,18	-0,05	0,38
65	48 (C)	-6,7	0,04	0,17	0,21	-0,12	-0,18
66	48 (C)	-6,58	0,02	0,16	-0,2	-0,1	0,29
67	48 (C)	40,46	-0,2	-0,09	0,13	-0,01	0,25
68	46 (C)	-40,54	0,49	0,08	-0,03	-0,07	0,93
69	80 (C)	34,22	-0,36	-0,3	0,02	-0,3	0,78
70	59 (C)	19,08	0,02	-0,17	-0,01	-0,12	-0,07

71	48 (C)	40,32	0,2	-0,09	-0,15	-0,02	-0,23
72	31 (C)	36,27	0,44	-0,1	-0,01	-0,02	-0,77
75	48 (C)	30,16	-0,18	-0,13	0,14	-0,07	0,21
76	26 (C)	-55,23	-0,09	0,12	-0,08	-0,1	-0,11
77	48 (C)	30,74	0,19	-0,12	-0,16	-0,06	-0,2
78	26 (C)	-55,08	0,09	-0,12	0,11	-0,11	-0,11
79	86 (C)	33,68	-0,33	-0,21	-0,02	-0,26	0,67
80	87 (C)	-24,45	-0,31	0,25	0	-0,24	-0,71
126	83 (C)	121,36	0,05	0,01	0	0,05	0,11
127	65 (C)	44,72	0,04	-0,03	0,01	0,11	0,06
128	65 (C)	5,33	0,08	-0,33	-0,01	0,26	0,13
129	88 (C)	133,41	0,06	0,1	-0,02	-0,12	0,13
130	63 (C)	46,37	0,34	-0,09	-0,08	0,16	0,3
131	63 (C)	5,65	-0,87	-0,46	0,22	0,48	-1,56
132	64 (C)	-15,83	0	0,07	0	-0,03	0
133	86 (C)	0,8	0,04	0,2	0	-0,25	0,06
134	102 (C)	-16,48	-0,19	0,05	0,01	-0,01	-0,21
135	89 (C)	79,9	0,03	0,04	-0,01	0,07	0
136	58 (C)	26,11	-0,03	0,04	0	-0,08	-0,03
137	65 (C)	4,45	-0,08	0,33	0,02	-0,31	-0,11
138	83 (C)	-16,4	0	0,05	0	-0,02	0
139	84 (C)	-2,13	0,06	0,04	-0,03	-0,01	0,06
140	58 (C)	-14,31	0	0,04	0	0	0
141	96 (C)	80,4	-0,03	0,05	0,01	0,09	0
142	58 (C)	26,18	0,03	0,04	0	-0,07	0,03
143	63 (C)	4,88	-0,86	0,34	0,11	-0,37	-0,32
144	89 (C)	-2,08	0	0,03	0	-0,01	0
145	89 (C)	1,36	0	0,05	0	-0,02	0
146	88 (C)	-18,12	-0,23	0,02	-0,01	0	-0,23
147	96 (C)	-4,04	0	0,07	0	-0,03	0
148	97 (C)	1,13	0,25	0,55	-0,14	-0,99	0,46
149	69 (C)	-0,34	0,23	-0,17	-0,04	0,67	0,41
150	88 (C)	2,37	0,11	-0,16	0,02	-0,11	-0,3
151	88 (C)	2,05	0,16	-0,13	0,04	-0,05	-0,36
152	99 (C)	-4,59	-0,26	-0,07	0,03	-0,01	0,6

153	58 (C)	4,52	-0,01	-0,18	0	-0,15	0,03
154	58 (C)	4,51	0,01	-0,17	0	-0,15	-0,03
155	102 (C)	5,06	-0,28	-0,14	0,05	-0,07	0,39
156	80 (C)	34,38	0,38	-0,32	-0,03	-0,32	-0,79
157	65 (C)	19,08	-0,01	-0,16	0,01	-0,1	0,03
158	58 (C)	42,05	0	-0,09	-0,01	-0,03	0
159	88 (C)	34,7	-0,35	-0,1	0,02	-0,03	0,62
160	93 (C)	6,81	0,23	-0,26	0,05	-0,36	-0,53
161	96 (C)	5,9	0,01	-0,17	0	-0,13	-0,02
162	58 (C)	41,57	0	-0,1	0	-0,04	0
163	89 (C)	-33,57	-0,01	0,07	0	-0,03	-0,02
164	58 (C)	57,71	-0,02	-0,13	-0,01	-0,09	0,01
165	83 (C)	20,17	0,01	-0,13	-0,01	-0,07	-0,02
166	102 (C)	59,01	-0,04	-0,13	0,16	-0,11	0,26
167	88 (C)	21,62	-0,19	0,18	-0,03	-0,17	-0,43
168	105 (C)	31,32	0,32	-0,2	0,01	-0,25	-0,65
169	30 (C)	-27,34	0,3	0,25	0	-0,24	0,68
238	26 (C)	4,06	-3,71	0,05	0	-0,02	-1,61
240	48 (C)	-1,27	-3,73	0,07	0	-0,04	-1,63
241	58 (C)	-3,2	0	0,07	0	-0,03	0
242	90 (C)	-16,6	0,2	0,04	-0,01	-0,01	0,18
243	48 (C)	-1,29	-2,73	0,07	0	-0,02	-1,24

Table of Contents:

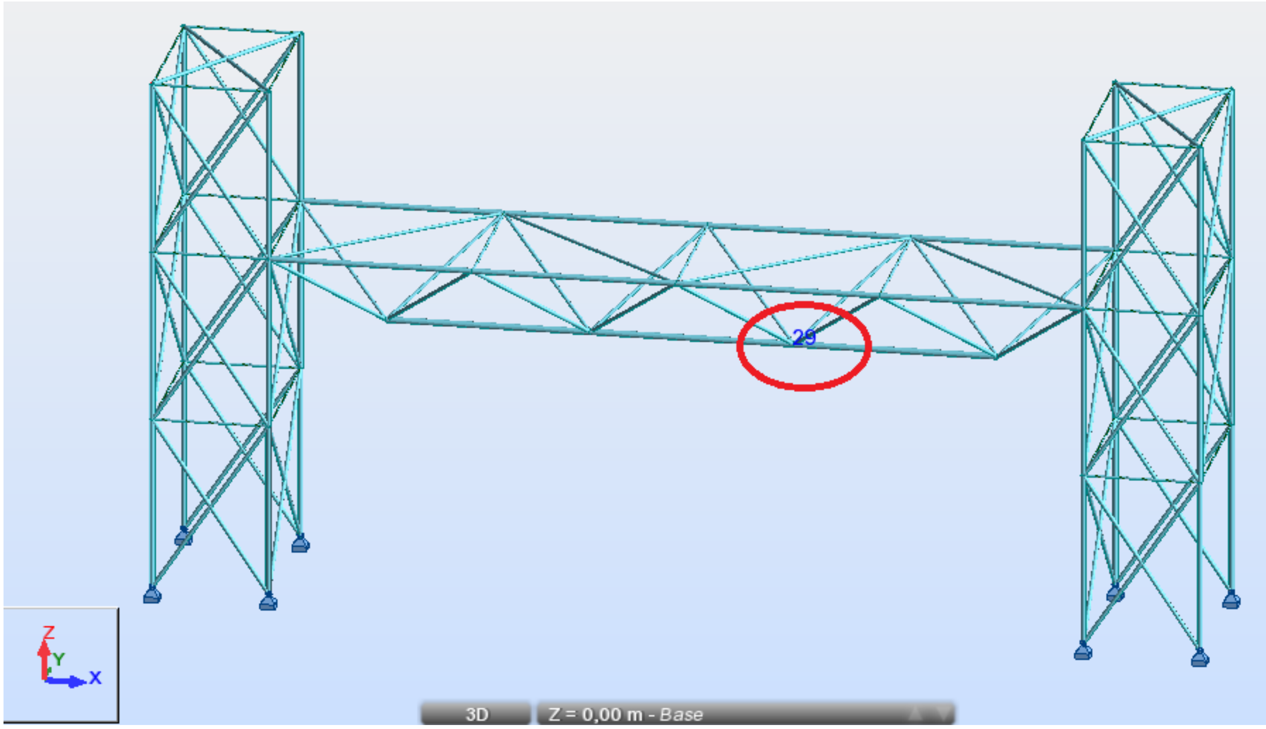
Design

Connections

All calculations were performed according to the data contained in section 5.2. this work.

Node 29

The following is a schematic the calculations made when designing the node number 29, which was presented in the figure below:



Screenshot of the program [a]- node 29

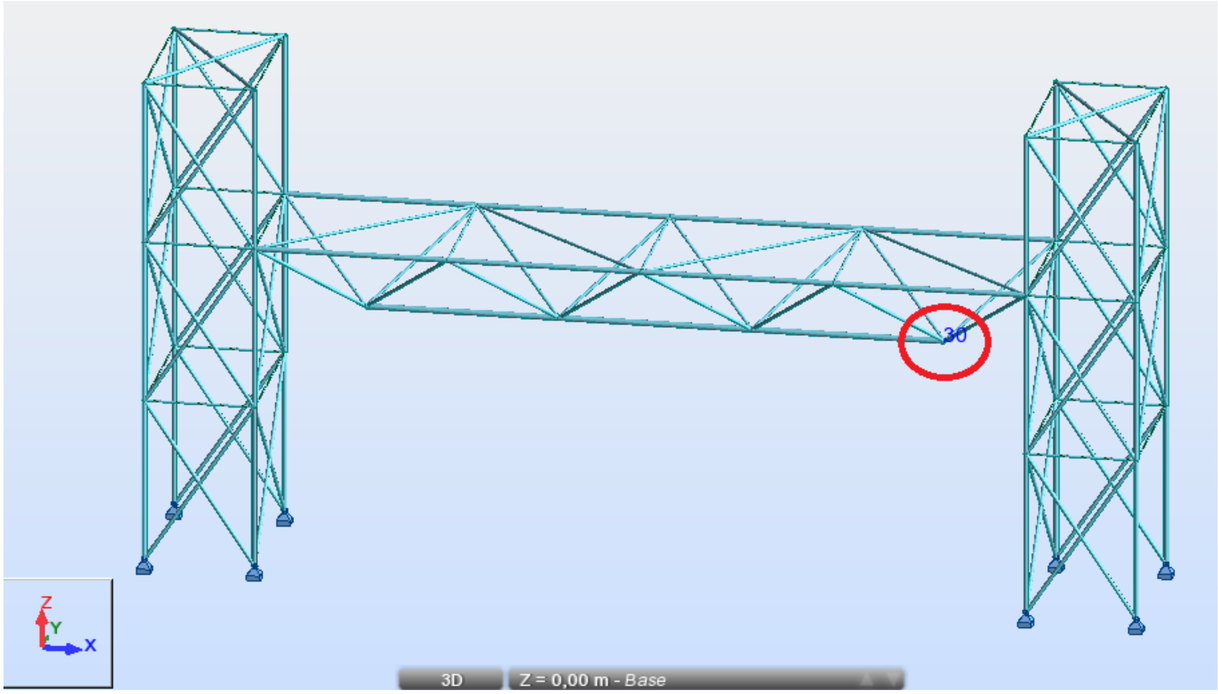
This node connects the rods numbered 39, 40, 49, 50, 51 and 52.

Geometry of the connection like connection in node 28 and force is smaller than in node 28 ($N_{Ed} = 24{,}33[kN]$). Thus, the connection can be designed in the same way.

This node was presented on the drawings 05- Steel Footbridge- node 28, 29, which was attached to this work.

Node 30

The following is a schematic the calculations made when designing the node number 30, which was presented in the figure below:

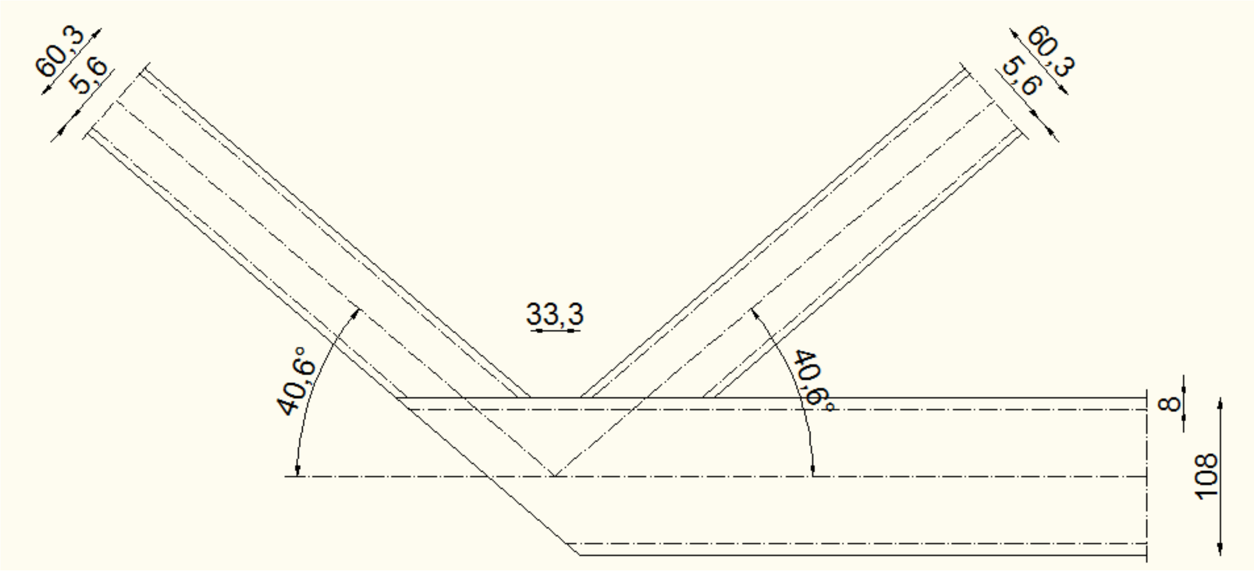


Screenshot of the program [a]- node 30

This node connects the rods numbered 54, 56, 53, 55 and 40.

This node was presented on the drawings 06- Steel Footbridge- node 27, 30, which was attached to this work.

A node is defined as K type according to figure 7.1.



Screenshot of the program [b]- geometry of node 30

Geometry of the connection:

$d_1 = 60{,}3[\mathrm{mm}]$ $d_2 = 60{,}3[\mathrm{mm}]$ $d_0 = 108{,}0[\mathrm{mm}]$
$t_1 = 5{,}6[\mathrm{mm}]$ $t_2 = 5{,}6[\mathrm{mm}]$ $t_0 = 8{,}0[\mathrm{mm}]$
The angles of inclination of diagonals:

$\theta_1 = 40{,}6[°]$ $\theta_2 = 40{,}6[°]$

Design value of the normal force: $N_{Ed} = -59{,}12[\mathrm{kN}]$

Node K

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \le \frac{d_{1,2}}{d_0} = \frac{60{,}3[\mathrm{mm}]}{108{,}0[\mathrm{mm}]} = 0{,}56[-] \le 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \le \frac{d_0}{t_0} = \frac{108{,}0[\mathrm{mm}]}{8{,}0[\mathrm{mm}]} = 13{,}5[-] \le 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \le \frac{d_0}{t_0} = \frac{108{,}0[\mathrm{mm}]}{8{,}0[\mathrm{mm}]} = 13{,}5[-] \le 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_{1,2}}{t_{1,2}} = \frac{60{,}3[\mathrm{mm}]}{5{,}6[\mathrm{mm}]} = 10{,}8[-] \le 50{,}0$

Condition fulfilled.

Gap: $g = 33{,}3[\mathrm{mm}] \quad \ge \quad t_1 + t_2 = 5{,}6[\mathrm{mm}] + 5{,}6[\mathrm{mm}] = 11{,}2[\mathrm{mm}]$

g determined in accordance with the geometry of the connections and figure 1.3.

Condition fulfilled.

Chord face failure:

Factors:

$k_p = 1{,}0[-]$ $\gamma = \frac{d_0}{2\cdot t_0} = \frac{108{,}0[mm]}{2\cdot 8{,}0[mm]} = 6{,}75[-]$(table 7.2)

$$k_g = \gamma^{0{,}2}\cdot\left(1+\frac{0{,}024\cdot\gamma^{1{,}2}}{1+\exp(0{,}5\cdot\frac{g}{t_0}-1{,}33)}\right) = (6{,}75[-])^{0{,}2}\cdot\left(1+\frac{0{,}024\cdot(6{,}75[-])^{1{,}2}}{1+\exp(0{,}5\cdot\frac{33{,}3[mm]}{8{,}0[mm]}-1{,}33)}\right) = 1{,}545[-]$$

(table 7.2)

Chord face failure:

$$N_{t,Rd} = \frac{k_g\cdot k_p\cdot f_{y0}\cdot t_0^2}{\sin\theta_{1,2}}(1{,}8+10{,}2\frac{d_{1,2}}{d_0})/\gamma_{M5} = \frac{1{,}545[-]\cdot 1{,}0[-]\cdot 235\left[\frac{kN}{mm^2}\right]\cdot(8[mm])^2}{\sin 40{,}6[°]}(1{,}8+10{,}2\frac{60{,}3[mm]}{108{,}0[mm]})/1{,}0 = 267{,}571[kN]$$ (table 7.2)

Punching shear failure for K gap joints:

$$d_{1,2} = 60{,}3[mm] \le d_o - 2t_0 = 108{,}0[mm] - 2\cdot 8{,}0[mm] = 92[mm]$$ (table 7.2)

$$N_{1,2,Rd} = \frac{f_{y0}}{\sqrt{3}}\cdot t_0\cdot\pi\cdot d_{1,2}\cdot\frac{\frac{1+\sin\theta_{1,2}}{2\sin^2\theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}}\cdot 8{,}0[mm]\cdot\pi\cdot 60{,}3[mm]\cdot\frac{\frac{1+\sin 40{,}6[°]}{2\sin^2 40{,}6[°]}}{1{,}0} = 400{,}739[kN]$$ (table 7.2)

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1,2} = min(N_{1,2Rd}, N_{t,Rd}) = min(400{,}739[kN]; 267{,}571[kN]) = 267{,}571[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu\cdot N_{Rd1,2} = 0{,}9\cdot 267{,}571[kN] = 240{,}814[kN]$$ (table 7.7)

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{59{,}12[kN]}{240{,}814[kN]} = 0{,}25[-] < 1{,}0[-]$$

Condition fulfilled.

Welded connections

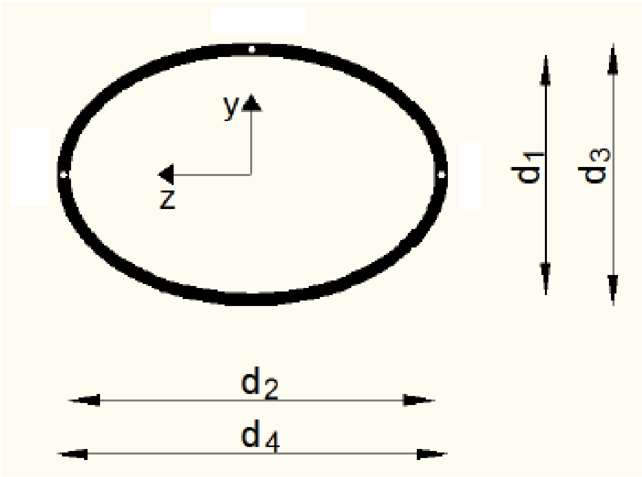
Simplified method for design resistance of fillet weld

Adopted fillet welds of thickness equal to: $a = 3[mm]$

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_z = N_{Ed}\cos\theta_{1,2} = 59{,}12[kN]\cdot\cos(40{,}6[^o]) = 44{,}888[kN]$$

$$N_x = N_{Ed}\sin\theta_{1,2} = 59{,}12[kN]\cdot\sin(40{,}6[^o]) = 38{,}474[kN]$$



Geometry of the quad weld (diagonal number 45)

Where dimensions are:

The smaller the diameter of the inner ellipse:

$$d_1 = 60{,}3[mm]$$

The larger diameter of the inner ellipse:

$$d_2 = \frac{d_1[mm]}{\sin(\theta)} = \frac{60{,}3[mm]}{\sin(40{,}6[^o])} = 92{,}66[mm]$$

The smaller diameter of the external ellipse:

$$d_3 = d_1 + 2a = 60{,}3[mm] + 2 \cdot 3[mm] = 66{,}3[mm]$$

The larger diameter of external ellipse:

$$d_4 = d_2 + 2a = 92{,}66[mm] + 2 \cdot 3[mm] = 98{,}66[mm]$$

Surface area:

$$A_w = \frac{\pi}{4}(d_3 \cdot d_4 - d_1 \cdot d_2) = \frac{\pi}{4}(66{,}3[mm] \cdot 98{,}66[mm] - 60{,}3[mm] \cdot 92{,}66[mm]) = 7{,}491[cm^2]$$

Normal stresses of tension force:

$$\tau_H = \frac{N_z}{A_w} = \frac{44{,}888[kN]}{7{,}491[cm^2]} = 5{,}992\left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_V = \frac{N_x}{A_w} = \frac{38{,}474[kN]}{7{,}491[cm^2]} = 5{,}136\left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau = \sqrt{\tau_V^2 + \tau_H^2} = \sqrt{\left(5{,}136\left[\frac{kN}{cm^2}\right]\right)^2 + \left(5{,}992\left[\frac{kN}{cm^2}\right]\right)^2} = 7{,}892\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} = \frac{36\left[\frac{kN}{cm^2}\right]/\sqrt{3}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

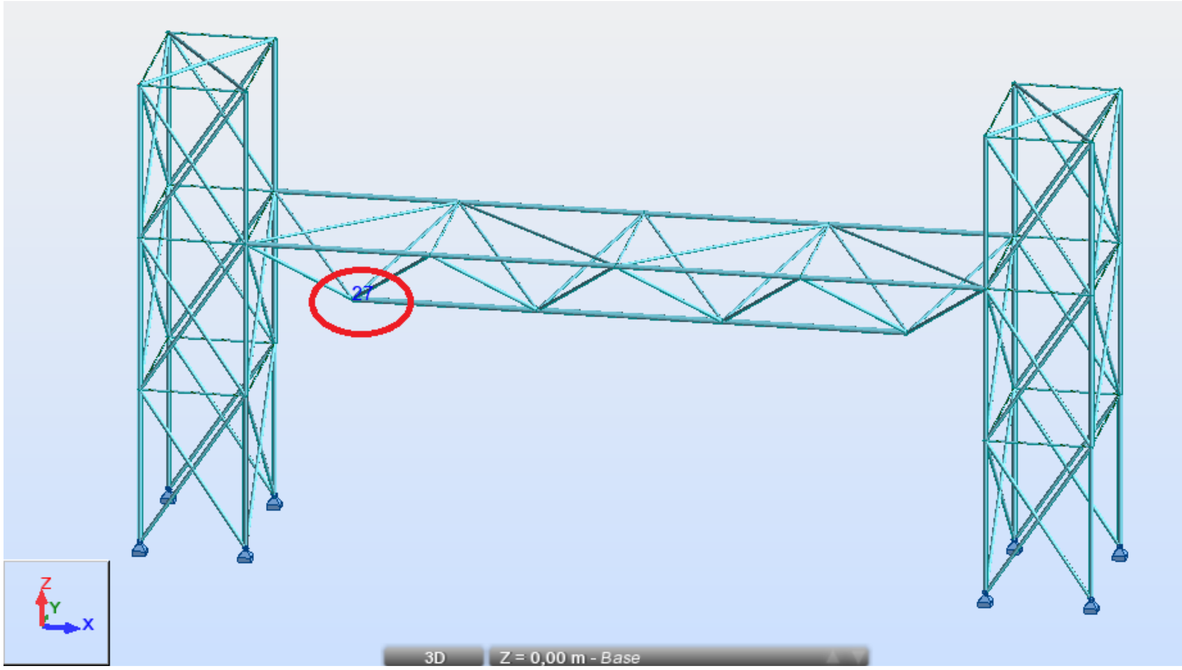
Condition:

$$\tau = 7{,}892\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

Node 27

The following is a schematic the calculations made when designing the node number 27, which was presented in the figure below:



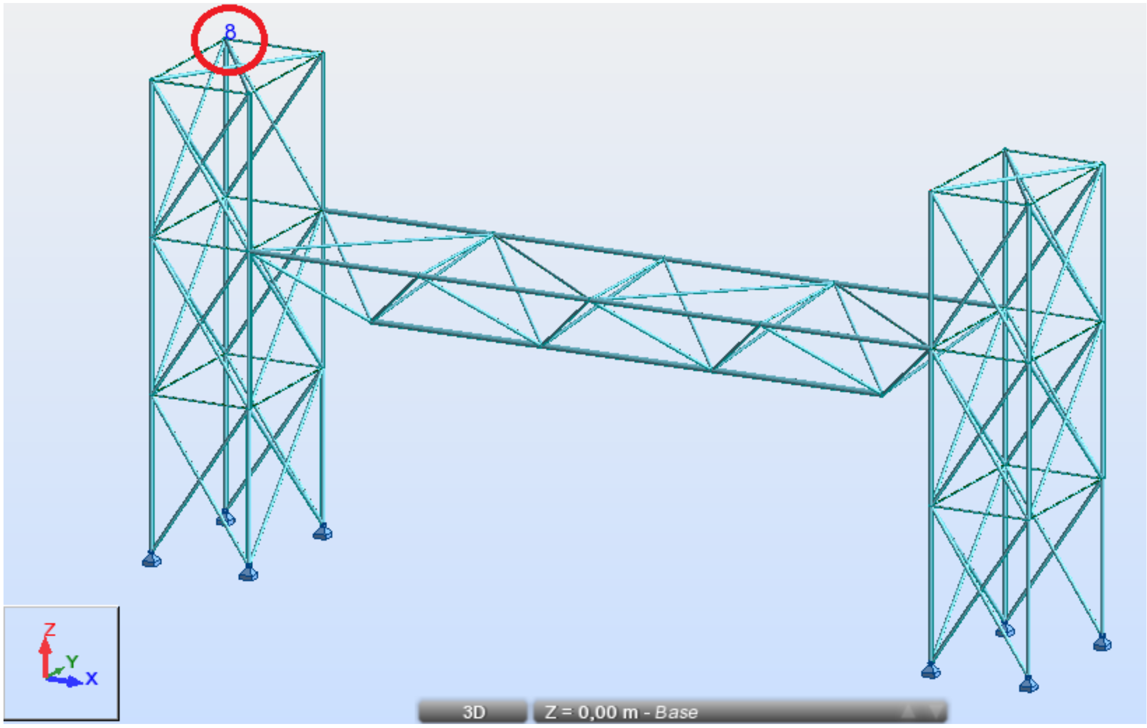
Screenshot of the program [a]- node 27

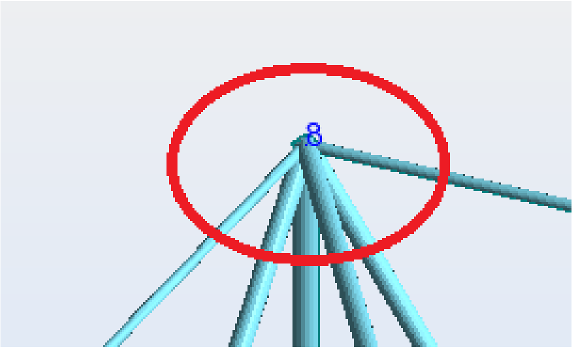
Geometry of the connection like connection in node 30 and force is smaller than in node 30 ($N_{Ed} = 58{,}84[kN]$). Thus, the connection can be designed in the same way.

This node was presented on the drawings 06- Steel Footbridge- node 27, 30, which was attached to this work.

Node 8

The following is a schematic the calculations made when designing the node number 8, which was presented in the figure below:





Screenshot of the program [a]- node 8

This node connects the rods numbered 6, 60, 240, 57, 63 and 15.

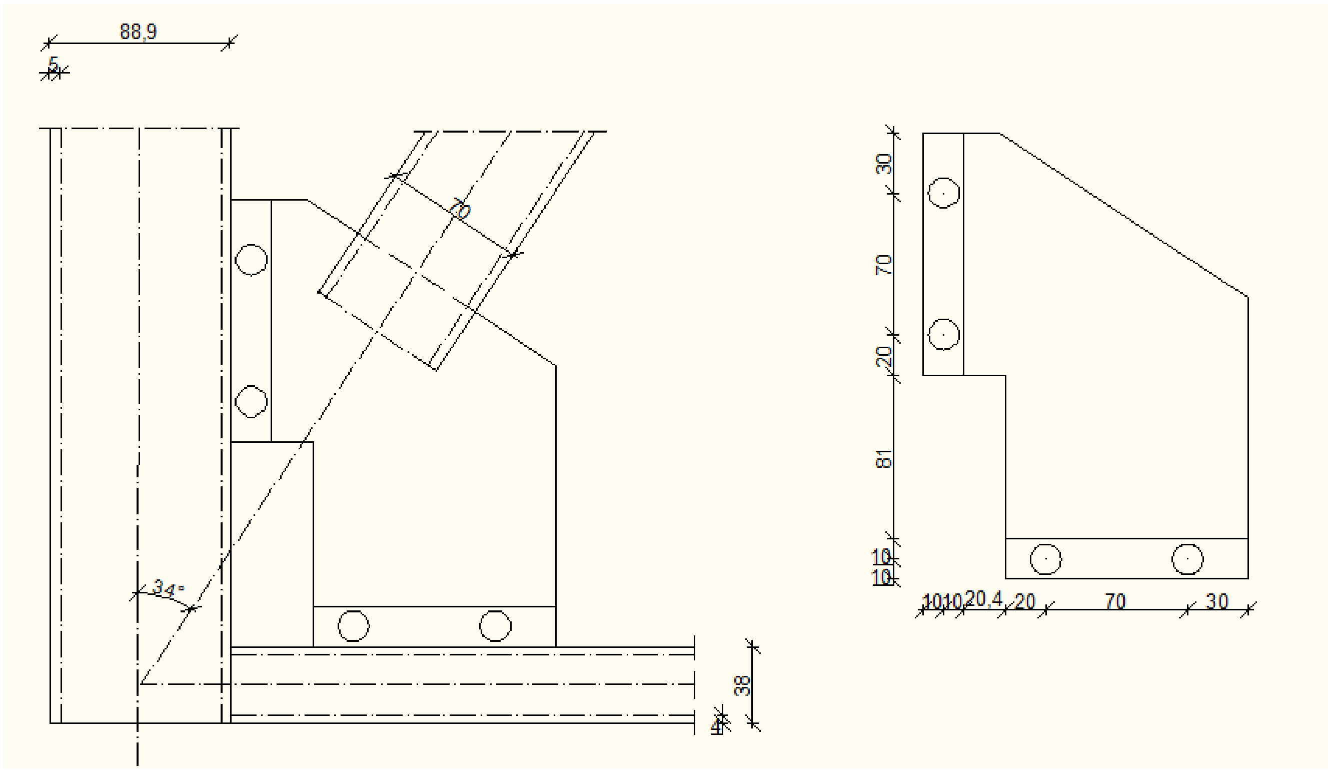
This node was presented on the drawings 07- Steel Footbridge- node 4, 8, 60, 64, which was attached to this work.

A node is defined as YT type according to figure 7.1, and in plane YZ and XZ have different geometry. So at first shows calculated for plane YZ like for node Y and then like for node T, then was performed the same calculation but like for plane XZ, and for plane ZY.

PLANE ZX

Due to the complexity of the construction of the node (in its place combines up to six bars) decided to use steel plates.

The geometry of the screw connection meets all the requirements specified in table 3.3.



Screenshot of the program [b]- geometry of node 8 XZ with geometry of plate

Forces:

$N_{Ed,6} = 7{,}17[kN]$ $N_{Ed,60} = 1{,}94[kN]$ $N_{Ed,240} = -1{,}27[kN]$

$N_{Ed,57} = 1{,}15[kN]$ $N_{Ed,63} = 10{,}24[kN]$ $N_{Ed,15} = 2{,}99[kN]$

Geometry of the connection:

$d_1 = 70{,}0[mm]$ $d_2 = 38{,}0[mm]$ $d_0 = 88{,}9[mm]$

$t_1 = 5{,}0[mm]$ $t_2 = 4{,}0[mm]$ $t_0 = 5{,}0[mm]$

The angles of inclination of diagonals:

$\theta_1 = 34{,}0[°]$ $\theta_2 = 90{,}0[°]$

Geometry of the plates:

Cross section:

$A_0 = 263{,}290[cm^2]$ $A_1 = 24{,}0[cm^2]$ $A_2 = 24{,}0[cm^2]$

Thickness:

$t_0 = 10{,}0[mm]$ $t_1 = 10{,}0[mm]$ $t_2 = 10{,}0[mm]$

The bolt's parameters:

The nominal bolt diameter: $d = 14{,}0[mm]$

The hole diameter for a bolt: $d_0 = 15{,}0[mm]$

The nominal values of the yield strength for bolts: $f_{yb} = 480\left[\frac{N}{mm^2}\right]$ (table 3.1)

The nominal values of the ultimate tensile strength for bolts: $f_{ub} = 800\left[\frac{N}{mm^2}\right]$ (table 3.1)

The tensile stress area of the bolt or of the anchor bolt: $A_s = 115[mm^2]$

Resistance of steel plate (Big plate):

In the case of elements asymmetrically connected in the nodes via mechanical fasteners category A design tension resistance defines itself as resistance boundary:

The reduction factor (linear interpolation): $\beta_2 = 0{,}58[-]$ (table 3.8)

The net area of a cross section:

$$A_{net\,0} = min\left(A_0 - \left(nd_0 - \left(\frac{s_1^2}{4p_1} + \frac{s_2^2}{4 \cdot p_2}\right)\right)t_0, A_0 - d_0 t_0\right) = min\left(263{,}290[cm^2] - \left(4 \cdot 15{,}0[mm] - \left(\frac{(70[mm])^2}{4 \cdot 111[mm]} + \frac{(26[mm])^2}{4 \cdot 70[mm]}\right)\right) \cdot 10{,}0[mm]; 263{,}290[cm^2] - 15{,}0[mm] \cdot 10{,}0[mm]\right) = min(258{,}635[cm^2];\ 261{,}790[cm^2]) = 258{,}635[cm^2]$$

(according to EN 1991-1-1 6.2.2.2.(4))

The design tension resistance (according EN 1993-1-1 5.2.3.(2) b)):

$$N_{u,Rd0} = \frac{\beta_2 A_{net\,0} f_u}{\gamma_{M2}} = \frac{0{,}70[-] \cdot 258{,}635[cm^2] \cdot 360\left[\frac{N}{mm^2}\right]}{1{,}25[-]} = 5214{,}082[kN]$$ (formula 4.7 EN 1991-1-1)

The design plastic resistance of the gross cross-section:

$$N_{pl,Rd0} = \frac{A_0\, f_u}{\gamma_{M0}} = \frac{263{,}290[cm^2]\cdot 360\left[\frac{N}{mm^2}\right]}{1{,}00[-]} = 9478{,}440\ [kN]$$ (formula 4.6EN 1991-1-1)

The design value of the resistance to tension forces:

$$N_{t,Rd0} = min\left(N_{u,Rd0}, N_{pl,Rd0}\right) = min\ \ (5214{,}082[kN], 9478{,}440\ [kN]) = 5214{,}082[kN]$$

The Lug connects angle members and their fasteners to a gusset or other supporting part and should be designed to transmit a force 1,2 times the force in the out stand of the angle connected. (according 3.10.4 (1)).

The condition resistance of section:

$$\frac{1{,}2\cdot N_{Ed,63}}{N_{t,Rd0}} = \frac{1{,}2\cdot 10{,}24[kN]}{5214{,}082\ [kN]} = \ 0{,}002\ <\ \ 1{,}0$$

Condition fulfilled.

Resistance of steel plate (smaller plate):

The net area of a cross section:

$$A_{net\,1,2} = A_{1,2} - d_0 t_{1,2} = 24{,}00[cm^2] - 15{,}0[mm]\cdot 10{,}0[mm] = 22{,}50[cm^2]$$

(according to EN 1991-1-1 6.2.2.2.(4))

The design tension resistance (according EN 1993-1-1 5.2.3.(2) b)):

$$N_{u,Rd1,2} = \frac{\beta_2 A_{net\ 1,2} f_u}{\gamma_{M2}} = \frac{0{,}70[-]\ \cdot 22{,}50[cm^2]\cdot 360\left[\frac{N}{mm^2}\right]}{1{,}25[-]} = 453{,}600\ [kN]$$ (formula 4.7 EN 1991-1-1)

The design plastic resistance of the gross cross-section:

$$N_{pl,Rd1,2} = \frac{A_{1,2}\, f_u}{\gamma_{M0}} = \frac{24{,}00[cm^2]\cdot 360\left[\frac{N}{mm^2}\right]}{1{,}00[-]} = 864{,}00\ [\mathrm{k}N]$$ (formula 4.6EN 1991-1-1)

The design value of the resistance to tension forces:

$$N_{t,Rd1,2} = min\left(N_{u,Rd1,2}, N_{pl,Rd1,2}\right) = min\ \ (453{,}600\ \ [kN], 864{,}00\ [kN]) = 453{,}600\ \ [kN]$$

The Lug connects angle members and their fasteners to a gusset or other supporting part and should be designed to transmit a force 1,2 times the force in the out stand of the angle connected. (according 3.10.4 (1)).

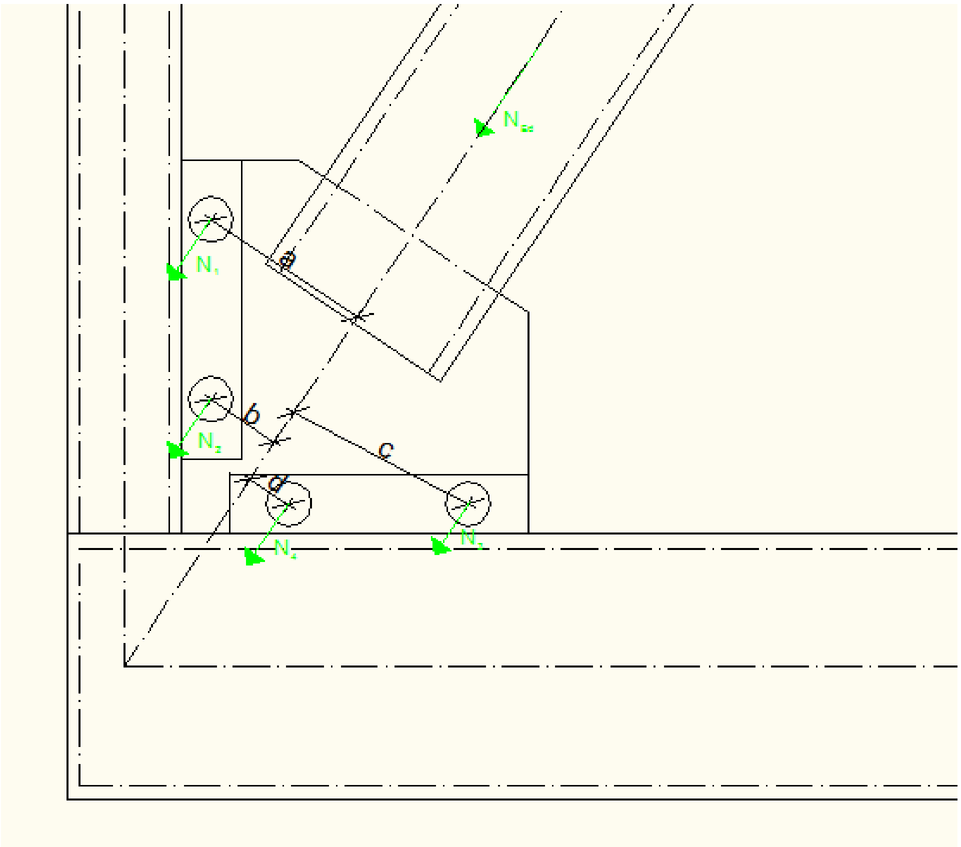
The condition resistance of section:

$$\frac{1{,}2\cdot N_{Ed,63}}{N_{t,Rd1,2}} = \frac{1{,}2\cdot 10{,}24[kN]}{453{,}600\ \ [kN]} = \ 0{,}027 <\ \ 1{,}0$$

Condition fulfilled.

The steel plate was designed correctly.

Design resistance of individual fasteners



Forces in bolts

$$N_1 = N_2 = N_3 = N_4 = \frac{10{,}24[kN]}{4} = 2{,}56[kN]$$

Shear resistance per shear plane:

Shear plane passes through the threaded portion, so:

Factor: $\propto_v = 0{,}60[-]$ (table 3.4)

The design shear resistance per bolt (for one bolt):

$$F_{vRd} = \propto_v \cdot f_{ub} \cdot \frac{A_s}{\gamma_{M2}} = 0{,}60[-] \cdot 800\left[\frac{N}{mm^2}\right] \cdot \frac{115[mm^2]}{1{,}25[-]} = 44{,}16[kN] \text{ (table 3.4)}$$

The bearing resistance:

The factor:

$$\alpha_b = min\left(1{,}0; \frac{f_{ub}}{f_u}; \frac{e_1}{3d_0}; \frac{p_1}{3d_0} + \frac{1}{4}\right) = min\left(1{,}0; \frac{800\left[\frac{N}{mm^2}\right]}{360\left[\frac{N}{mm^2}\right]}; \frac{30[mm]}{3 \cdot 15[mm]}; \frac{70[mm]}{3 \cdot 15[mm]} + \frac{1}{4}\right) =$$
$$min(1{,}0;\ 2{,}222;\ 0{,}667; 1{,}806) = 0{,}667\ [-] \text{ (table 3.4)}$$

The factor:

$$k_1 = min\left(2{,}5; 2{,}8 \cdot \frac{e_2}{d_0} - 1{,}7; 1{,}4 \cdot \frac{p_2}{d_0} - 1{,}7\right) = min\left(2{,}5; 2{,}8 \cdot \frac{20[mm]}{15[mm]} - 1{,}7; 1{,}4 \cdot \frac{111[mm]}{15[mm]} - 1{,}7\right) = min(2{,}5; 2{,}033; 8{,}660) = 2{,}033\ [-]$$ (table 3.4)

The design bearing resistance per bolt:

$$F_{bRd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot \frac{t_0}{\gamma_{M2}} = 2{,}033[-] \cdot 0{,}667\ [-] \cdot 360\left[\frac{kN}{mm^2}\right] \cdot 14[mm] \cdot \frac{10[mm]}{1{,}25[-]} = 54{,}674[kN]$$ (table 3.4)

Tension resistance:

Factor: $k_2 = 0{,}90[-]$ (table 3.4)

The design tension resistance per bolt:

$$F_{tRd} = \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0{,}90\ [-] \cdot 800\left[\frac{N}{mm^2}\right] \cdot 115\left[mm^2\right]}{1{,}25[-]} = 66{,}24[kN]$$ (table 3.4)

Condition of resistance connection:

$$N_{jRd} = min(N_{tRd}, F_{vRd}, F_{bRd}, F_{tRd})$$

$$N_{0Rd} = min(309{,}024\ [kN]; 44{,}16[kN]\ ; 54{,}674[kN]; 66{,}24[kN]) = 44{,}16[kN]$$

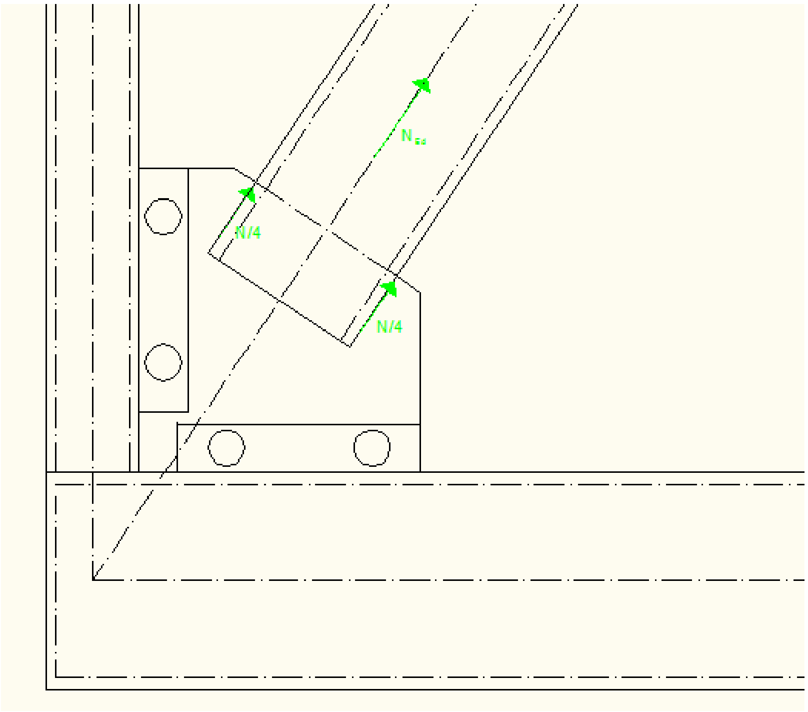
The condition resistance of section (compared to the biggest force in bolt):

$$\frac{1{,}2 \cdot N_{Ed,63}}{N_{0Rd}} = \frac{1{,}2 \cdot 2{,}56[\mathrm{kN}]}{44{,}16\ [kN]} = 0{,}07 < 1{,}0$$

Condition fulfilled.

Welded connections

Connection between diagonal rod and steel plate



Forces in weld

Adopted fillet welds of thickness equal to: $a = 3[mm]$

Simplified method for design resistance of fillet weld

Forces in weld:

$$N_1 = N_2 = \frac{10{,}24[kN]}{4} = 2{,}56[kN]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = \frac{\frac{360\left[\frac{N}{mm^2}\right]}{\sqrt{3}}}{0{,}8[-]\cdot 1{,}25[-]} = 207{,}846\left[\frac{N}{mm^2}\right] \text{ (formula 4.4)}$$

Independent of the orientation of the weld throat plane to the applied force, the design resistance per unit length should be determined from:

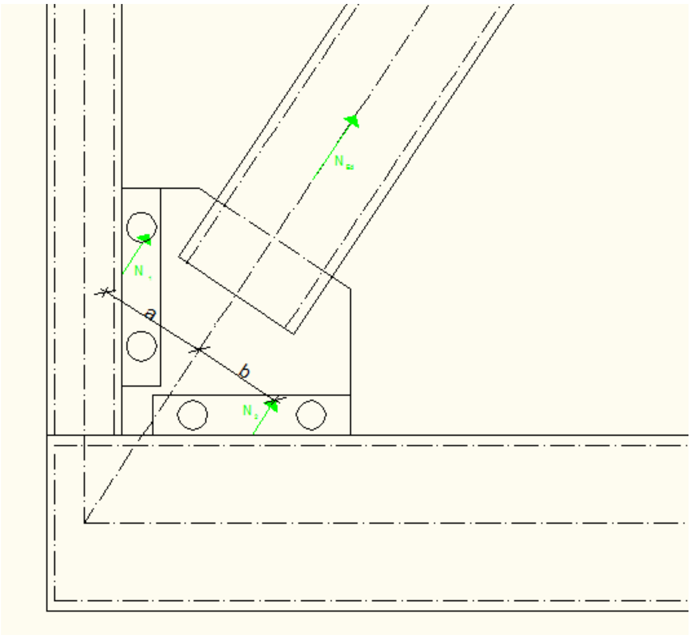
$F_{w,Rd} = l_s \cdot f_{vw,d} \cdot a = 35[mm] \cdot 207{,}846\left[\frac{N}{mm^2}\right] \cdot 3[mm] = 21{,}824[kN]$ (formula 4.3)

Condition:

$$F_{w,Ed} = 2{,}56[kN] \quad \leq \quad F_{w,Rd} = 21{,}824\,[kN]$$

Condition fulfilled.

Welded between orthogonal bars and steel plates



Forces in weld

Forces:

$$\mathrm{N}_1 = \frac{N_{Ed} \cdot b}{a+b} = \frac{10{,}24[kN] \cdot 46[mm]}{50{,}9[mm] + 46[mm]} = 4{,}861[kN]$$

$$\mathrm{N}_2 = \frac{N_{Ed} \cdot a}{a+b} = \frac{10{,}24[kN] \cdot 50{,}9[mm]}{50{,}9[mm] + 46[mm]} = 5{,}379[kN]$$

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_{z1} = N_1 \cos\theta_{1,2} = 4{,}861[kN] \cdot \cos(34[^o]) = 4{,}030[kN]$$

$$N_{x1} = N_1 \sin\theta_{1,2} = 4{,}861[kN] \cdot \sin(34[^o]) = 2{,}718[kN]$$

$$N_{z2} = N_2 \cos\theta_{1,2} = 5{,}379[kN] \cdot \cos(34[^o]) = 4{,}459[kN]$$

$$N_{x2} = N_2 \sin\theta_{1,2} = 5{,}379[kN] \cdot \sin(34[^o]) = 3{,}008[kN]$$

Surface area:

$$A_w = 120[mm] \cdot 3[mm] = 3{,}6[cm^2]$$

Normal stresses of tension force:

$$\tau_{H1} = \frac{N_{z1}}{A_w} = \frac{4{,}030[kN]}{3{,}6[cm^2]} = 1{,}119\left[\frac{kN}{cm^2}\right]$$

$$\tau_{H2} = \frac{N_{z2}}{A_w} = \frac{4{,}459[kN]}{3{,}6[cm^2]} = 1{,}239\left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_{V1} = \frac{N_{x1}}{A_w} = \frac{2{,}718[kN]}{3{,}6[cm^2]} = 0{,}755\left[\frac{kN}{cm^2}\right]$$

$$\tau_{V2} = \frac{N_{x2}}{A_w} = \frac{3{,}008[kN]}{3{,}6[cm^2]} = 0{,}836\left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau_1 = \sqrt{\tau_{V1}^2 + \tau_{H1}^2} = \sqrt{\left(1{,}119\left[\frac{kN}{cm^2}\right]\right)^2 + \left(0{,}755\left[\frac{kN}{cm^2}\right]\right)^2} = 1{,}350\left[\frac{kN}{cm^2}\right]$$

$$\tau_2 = \sqrt{\tau_{V2}^2 + \tau_{H2}^2} = \sqrt{\left(1{,}239\left[\frac{kN}{cm^2}\right]\right)^2 + \left(0{,}836\left[\frac{kN}{cm^2}\right]\right)^2} = 1{,}495\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = \frac{\frac{36\left[\frac{kN}{cm^2}\right]}{\sqrt{3}}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

Condition:

$$\tau_1 = 1{,}350\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

$$\tau_2 = 1{,}495\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

Welded between orthogonal bars

Simplified method for design resistance of fillet weld

Forces:

$$F_{\sigma\perp} = F_{\tau\perp} = \frac{N_{Ed,15}}{\sqrt{2}} = \frac{2{,}99[kN]}{\sqrt{2}} = 2{,}114[kN]$$
$$F_{\sigma II} = 0[kN]$$

Stresses:

$$\sigma_\perp = \frac{F_{\sigma\perp}}{A_w} = \frac{2{,}114[kN]}{\pi \cdot ((19[mm])^2 - (19[mm] - 3[mm])^2)} = 0{,}641\left[\frac{kN}{cm^2}\right]$$

$$\tau_{\perp} = \frac{\mathrm{F}_{\sigma\mathrm{II}}}{\mathrm{A}_{\mathrm{w}}} = \frac{2{,}114[kN]}{\pi \cdot ((19[mm])^2 - (19[mm] - 3[mm])^2)} = 0{,}641 \left[\frac{kN}{cm^2}\right]$$

$$\tau_{\mathrm{II}} = 0$$

Stress resultant:

$$\sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\mathrm{II}}^2)} = \sqrt{\left(0{,}641 \left[\frac{kN}{cm^2}\right]\right)^2 + 3\left(\left(0{,}641 \left[\frac{kN}{cm^2}\right]\right)^2 + 0\right)} = 1{,}282 \left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = \frac{\frac{36\left[\frac{kN}{cm^2}\right]}{\sqrt{3}}}{0{,}8[-] \cdot 1{,}25[-]} = 20{,}785 \left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

Condition:

$$\tau = 1{,}282 \quad \leq \quad f_{vw,d} = 20{,}785 \left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

The connection has been sized correctly. Weld thickness of 3 mm is sufficient.

NODE Y planes XZ

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_1}{d_0} = \frac{70{,}0[mm]}{88{,}9[mm]} = 0{,}79[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_1}{t_1} = \frac{70,0[mm]}{5,0[mm]} = 14,0[-] \leq 50,0$

Condition fulfilled.
Chord face failure:

$k_p = 1,0[-]$ $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88,9[mm]}{2 \cdot 5,0[mm]} = 8,89[-]$(table 7.2)

The ratio of the mean diameter or width of the brace members (like for Y node):

$$\beta = \frac{d_1}{d_0} = \frac{70,0[mm]}{88,9[mm]} = 0,787[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_1}(2,8+14,2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8,89[-])^{0,2} \cdot 1,0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (5,0[mm])^2}{\sin(34[^o])}(2,8+14,2 \cdot (0,787[-])^2)}{1,0} = 188,581[kN]$$
(table 7.2)

Punching shear failure for T gap joints:

$$d_1 = 70,0[mm] \leq d_o - 2t_0 = 88,9[mm] - 2 \cdot 5,0[mm] = 78,9[mm] \text{ (table 7.2)}$$
$$N_{1,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{\frac{1+\sin\theta_{1,2}}{2\sin^2\theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 5,0[mm] \cdot \pi \cdot 70,0[mm] \cdot \frac{\frac{1+\sin(34[°])}{2\sin^2(34[°])}}{1,0} =$$
$$321,939[kN] \text{ (table 7.2)}$$
According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1} = min(N_{1Rd}, N_{t,Rd}) = min(188,581[kN]; 321,939[kN]) = 188,581[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd1,2} = 0,9 \cdot 188,581[kN] = 169,723[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{10,24[kN]}{169,723[kN]} = 0,06[-] < 1,0[-]$$

Condition fulfilled.

NODE T XZ

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0,2 \leq \frac{d_2}{d_0} = \frac{38,0[mm]}{88,9[mm]} = 0,43[-] \leq 1,0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88,9[mm]}{5,0[mm]} = 17,78[-] \leq 50,0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88,9[mm]}{5,0[mm]} = 17,78[-] \leq 50,0$

Condition fulfilled.

Braces:

Tension: $\frac{d_2}{t_2} = \frac{38,0[mm]}{5,0[mm]} = 7,6[-] \leq 50,0$

Condition fulfilled.

Chord face failure:

Factors:

$k_p = 1,0[-]$ $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88,9[mm]}{2 \cdot 5,0[mm]} = 8,89[-]$(table 7.2)

The ratio of the mean diameter or width of the brace members (like for T node):

$$\beta = \frac{d_2}{d_0} = \frac{38,0[mm]}{88,9[mm]} = 0,427[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_3}(2,8+14,2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8,89[-])^{0,2} \cdot 1,0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (5,0[mm])^2}{\sin(90[^o])}(2,8+14,2 \cdot (0,427[-])^2)}{1,0} = 49,012[kN]$$

(table 7.2)

Punching shear failure for T gap joints:

$$d_2 = 38,0[mm] \leq d_o - 2t_0 = 88,9[mm] - 2 \cdot 5,0[mm] = 78,9[mm] \text{ (table 7.2)}$$

$$N_{2,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_2 \cdot \frac{\frac{1+\sin\theta_{1,2}}{2\sin^2\theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 5,0[mm] \cdot \pi \cdot 38,0[mm] \cdot \frac{\frac{1+\sin(90[°])}{2\sin^2(90[°])}}{1,0} = 80,986[kN]$$

(table 7.2)

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd2} = min\left(N_{2Rd}, N_{t,Rd}\right) = min(80{,}986[kN]; 49{,}012[kN]) = 49{,}012[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd1,2} = 0{,}9 \cdot 49{,}012[kN] = 44{,}111[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{2{,}99[kN]}{44{,}111[kN]} = 0{,}07[-] < 1{,}0[-]$$

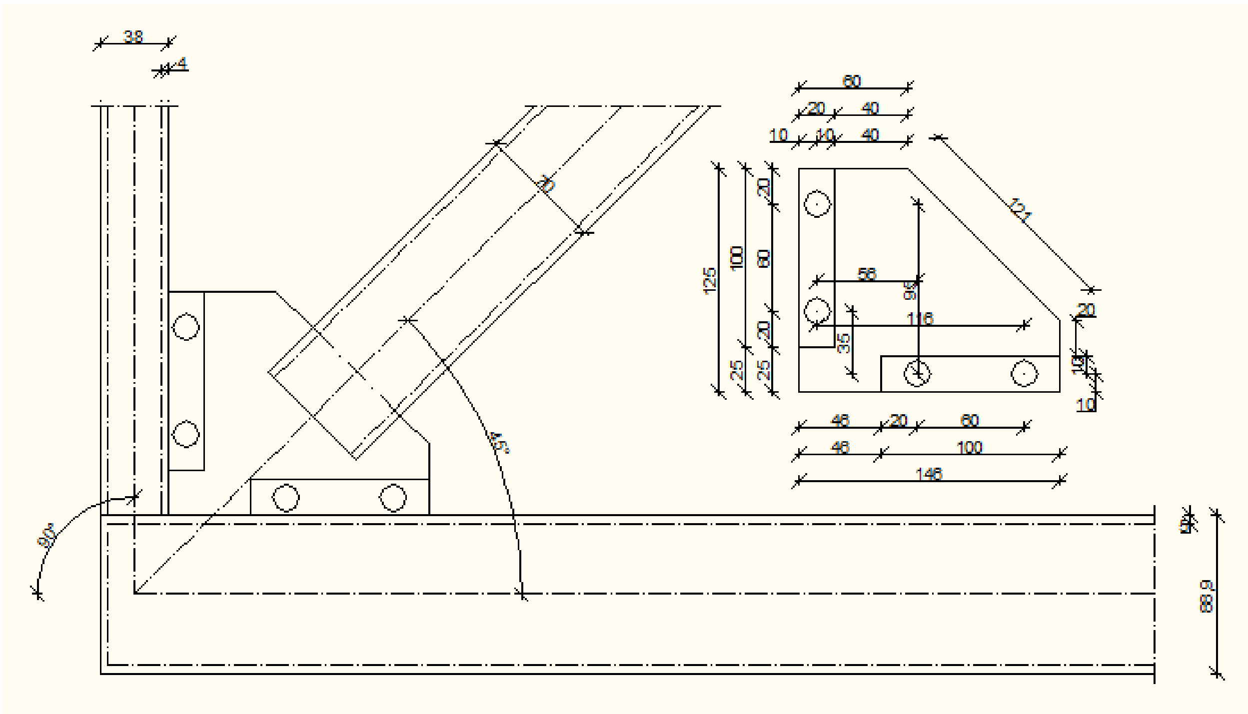
Condition fulfilled.

Plane YZ

Steel plate is welded to the rod number 60, with numbers rods 240 and 6 are connected by means of steel overlays. Steel caps are fixed to the steel plate by means of bolts M14 class 8.8.

The bolted connection is category A, (in this category bolts from class 4.6 up to and including class 10.9 should be used. No preloading and special provisions for contact surfaces are required. The design: shear, bearing resistance). (according 3.4.1. (1) a)).

The geometry of the screw connection meets all the requirements specified in table 3.3.



Screenshot of the program [b]- geometry of node 8 YZ with geometry of plate

Geometry of the connection:

$d_1 = 70{,}0[mm]$ $d_2 = 38{,}0[mm]$ $d_0 = 88{,}9[mm]$

$t_1 = 5{,}0[mm]$ $t_2 = 4{,}0[mm]$ $t_0 = 5{,}0[mm]$

The angles of inclination of diagonals:

$\theta_1 = 45{,}0[°]$ $\theta_2 = 90{,}0[°]$

Geometry of the plates:

Cross section:

$A_0 = 145{,}881[cm^2]$ $A_1 = 20{,}0[cm^2]$ $A_2 = 20{,}0[cm^2]$

Thickness:

$t_0 = 10{,}0[mm]$ $t_1 = 10{,}0[mm]$ $t_2 = 10{,}0[mm]$

Resistance of steel plate (Big plate):

In the case of elements asymmetrically connected in the nodes via mechanical fasteners category A design tension resistance defines itself as resistance boundary:

The reduction factor (linear interpolation): $\beta_2 = 0{,}58[-]$ (table 3.8)

The net area of a cross section:

$$A_{net\,0} = min\left(A_0 - \left(nd_0 - \left(\frac{s_1^2}{4p_1} + \frac{s_2^2}{4\cdot p_2}\right)\right)t_0, A_0 - d_0t_0\right) = min\left(145{,}881[cm^2] - \left(4\cdot 15{,}0[mm] - \left(\frac{(60[mm])^2}{4\cdot 35[mm]} + \frac{(56[mm])^2}{4\cdot 95[mm]}\right)\right)\cdot 10{,}0[mm];\ 145{,}881[cm^2] - 15{,}0[mm]\cdot 10{,}0[mm]\right) = min(143{,}278[cm^2]; 144{,}381[cm^2]) = 143{,}278[cm^2]$$

(according to EN 1991-1-1 6.2.2.2.(4))

The design tension resistance (according EN 1993-1-1 5.2.3.(2) b)):

$$N_{u,Rd0} = \frac{\beta_2 A_{net\,0} f_u}{\gamma_{M2}} = \frac{0{,}58[-]\cdot 143{,}278[cm^2]\cdot 360\left[\frac{N}{mm^2}\right]}{1{,}25[-]} = 2393{,}316[kN]$$ (formula 4.7 EN 1991-1-1)

The design plastic resistance of the gross cross-section:

$$N_{pl,Rd0} = \frac{A_0\, f_u}{\gamma_{M0}} = \frac{145{,}881[cm^2]\cdot 360\left[\frac{N}{mm^2}\right]}{1{,}00[-]} = 52517{,}160\ [kN]$$ (formula 4.6EN 1991-1-1)

The design value of the resistance to tension forces:

$$N_{t,Rd0} = min(N_{u,Rd0}, N_{pl,Rd0}) = min\ (2393{,}316\ [kN], 52517{,}160\ [\ [kN]) = 2393{,}316[kN]$$

The Lug connects angle members and their fasteners to a gusset or other supporting part and should be designed to transmit a force 1,2 times the force in the out stand of the angle connected. (according 3.10.4 (1)).

The condition resistance of section:

$$\frac{1{,}2 \cdot N_{Ed,60}}{N_{t,Rd0}} = \frac{1{,}2 \cdot 1{,}94[kN]}{2393{,}316\ [kN]} = \ 0{,}001 \ < \ 1{,}0$$

Condition fulfilled.

Resistance of steel plate (smaller plate):

The net area of a cross section:

$$A_{net\,1,2} = A_{1,2} - d_0 t_{1,2} = 20{,}00[cm^2] - 15{,}0[mm] \cdot 10{,}0[mm] = 18{,}50[cm^2]$$

(according to EN 1991-1-1 6.2.2.2.(4))

The design tension resistance (according EN 1993-1-1 5.2.3.(2) b)):

$$N_{u,Rd1,2} = \frac{\beta_2 A_{net\ 1,2} f_u}{\gamma_{M2}} = \frac{0{,}70[-]\ \cdot 18{,}50[cm^2] \cdot 360\left[\frac{N}{mm^2}\right]}{1{,}25[-]} = 372{,}960\ [kN]$$ (formula 4.7 EN 1991-1-1)

The design plastic resistance of the gross cross-section:

$$N_{pl,Rd1,2} = \frac{A_{1,2}\, f_u}{\gamma_{M0}} = \frac{20{,}00[cm^2] \cdot 360\left[\frac{N}{mm^2}\right]}{1{,}00[-]} = 720{,}200\ [\mathrm{k}N]$$ (formula 4.6EN 1991-1-1)

The design value of the resistance to tension forces:

$$N_{t,Rd1,2} = min(N_{u,Rd1,2}, N_{pl,Rd1,2}) = min\ (372{,}960\ [kN], 720{,}200\ [kN]) = 372{,}960\ \ [kN]$$

The Lug connects angle members and their fasteners to a gusset or other supporting part and should be designed to transmit a force 1,2 times the force in the out stand of the angle connected. (according 3.10.4 (1)).

The condition resistance of section:

$$\frac{1{,}2 \cdot N_{Ed,60}}{N_{t,Rd1,2}} = \frac{1{,}2 \cdot 1{,}94[kN}{372{,}960\ \ [kN]} = \ 0{,}06 < \ \ 1{,}0$$

Condition fulfilled.

The steel plate was designed correctly.

Design resistance of individual fasteners

$$\mathrm{N}_1 = \mathrm{N}_2 = \mathrm{N}_3 = \mathrm{N}_4 = \frac{1{,}94[\mathrm{kN}]}{4} = 0{,}485[\mathrm{kN}]$$

Shear resistance per shear plane:

Shear plane passes through the threaded portion, so:

Factor: $\propto_v = 0{,}60[-]$ (table 3.4)

The design shear resistance per bolt (for one bolt):

$$F_{vRd} = \propto_v \cdot f_{ub} \cdot \frac{A_s}{\gamma_{M2}} = 0{,}60[-] \cdot 800 \left[\frac{N}{mm^2}\right] \cdot \frac{115[mm^2]}{1{,}25[-]} = 44{,}16[kN] \text{ (table 3.4)}$$

The bearing resistance:

The factor:

$$\alpha_b = min\left(1{,}0; \frac{f_{ub}}{f_u}; \frac{e_1}{3d_0}; \frac{p_1}{3d_0} + \frac{1}{4}\right) = min\left(1{,}0; \frac{800\left[\frac{N}{mm^2}\right]}{360\left[\frac{N}{mm^2}\right]}; \frac{20[mm]}{3 \cdot 15[mm]}; \frac{60[mm]}{3 \cdot 15[mm]} + \frac{1}{4}\right) = min(1{,}0;\ 2{,}222;\ 0{,}444; 1{,}583) = 0{,}444\ [-] \text{ (table 3.4)}$$

The factor:

$$k_1 = min\left(2{,}5; 2{,}8 \cdot \frac{e_2}{d_0} - 1{,}7; 1{,}4 \cdot \frac{p_2}{d_0} - 1{,}7\right) = min\left(2{,}5; 2{,}8 \cdot \frac{20[mm]}{15[mm]} - 1{,}7; 1{,}4 \cdot \frac{56[mm]}{15[mm]} - 1{,}7\right) = min(2{,}5; 2{,}033; 3{,}527) = 2{,}033\ [-] \text{ (table 3.4)}$$

The design bearing resistance per bolt:

$$F_{bRd} = k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot \frac{t_0}{\gamma_{M2}} = 2{,}033[-] \cdot 0{,}444\ [-] \cdot 360 \left[\frac{kN}{mm^2}\right] \cdot 14[mm] \cdot \frac{10[mm]}{1{,}25[-]} = 36{,}395[kN] \text{ (table 3.4)}$$

Tension resistance:

Factor: $k_2 = 0{,}90[-]$ (table 3.4)

The design tension resistance per bolt:

$$F_{tRd} = \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0{,}90\ [-] \cdot 800\left[\frac{N}{mm^2}\right] \cdot 115[mm^2]}{1{,}25[-]} = 66{,}24[kN] \text{ (table 3.4)}$$

Condition of resistance connection:

$$N_{jRd} = min(N_{tRd}, F_{vRd}, F_{bRd}, F_{tRd})$$

$$N_{0Rd} = min(372{,}960\ [kN]; 44{,}16[kN]; 36{,}395[kN]; 66{,}24[kN]) = 44{,}16[kN]$$

The condition resistance of section (compared to the biggest force in bolt):

$$\frac{1{,}2 \cdot N_{Ed,63}}{N_{0Rd}} = \frac{1{,}2 \cdot 0{,}485[\mathrm{kN}]}{44{,}16\ [kN]} = \ 0{,}01 < \quad 1{,}0$$

Condition fulfilled.

Welded connections

Connection between smaller plate and bars

Because internal force in bar 60 is smaller than in plane XZ, and the geometry is the same, adopted the same welded connections.

NODE Y planes YZ

Design value of the normal force (the maximum value of the normal force in the diagonals in the XZ plane - diagonal number 60, in which there is the greatest normal force equal to -1,94kN (Annex 6), is the value for the combination of the number 35 (Annex 5)):

$$N_{Ed} \ = 1{,}94[kN]$$

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_1}{d_0} = \frac{70{,}0[mm]}{88{,}9[mm]} = 0{,}79[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_1}{t_1} = \frac{70{,}0[mm]}{5{,}0[mm]} = 14{,}0[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

$$k_p = 1{,}0[-] \qquad \gamma = \frac{d_0}{2\cdot t_0} = \frac{88{,}9[mm]}{2\cdot 5{,}0[mm]} = 8{,}89[-] \text{ (table 7.2)}$$

The ratio of the mean diameter or width of the brace members (like for Y node):

$$\beta = \frac{d_1}{d_0} = \frac{70{,}0[mm]}{88{,}9[mm]} = 0{,}787[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_1}(2{,}8+14{,}2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8{,}89[-])^{0,2}\cdot 1{,}0[-]\cdot 235\left[\frac{N}{mm^2}\right]\cdot(5{,}0[mm])^2}{\sin(45[^o])}(2{,}8+14{,}2\cdot(0{,}787[-])^2)}{1{,}0} = 236{,}361[kN]$$

(table 7.2)

Punching shear failure for T gap joints:

$$d_1 = 70{,}0[mm] \le d_o - 2t_0 = 88{,}9[mm] - 2\cdot 5{,}0[mm] = 78{,}9[mm] \text{ (table 7.2)}$$

$$N_{1,Rd} = \frac{f_{y0}}{\sqrt{3}}\cdot t_0\cdot\pi\cdot d_1\cdot\frac{\frac{1+\sin\theta_{1,2}}{2\sin^2\theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}}\cdot 5{,}0[mm]\cdot\pi\cdot 70{,}0[mm]\cdot\frac{\frac{1+\sin(45[°])}{2\sin^2(45[°])}}{1{,}0} = 254{,}675[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1} = min(N_{1Rd}, N_{t,Rd}) = min(254{,}675[kN]; 236{,}361[kN]) = 236{,}361[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu\cdot N_{Rd1,2} = 0{,}9\cdot 236{,}361[kN] = 212{,}725[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{1{,}94[kN]}{212{,}725[kN]} = 0{,}01[-] < 1{,}0[-]$$

Condition fulfilled.

NODE T YZ

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \le \frac{d_2}{d_0} = \frac{38{,}0[mm]}{88{,}9[mm]} = 0{,}43[-] \le 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \le \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \le 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_2}{t_2} = \frac{38{,}0[mm]}{5{,}0[mm]} = 7{,}6[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

Factors:

$k_p = 1{,}0[-]$ $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 5{,}0[mm]} = 8{,}89[-]$(table 7.2)

The ratio of the mean diameter or width of the brace members (like for T node):

$$\beta = \frac{d_2}{d_0} = \frac{38{,}0[mm]}{88{,}9[mm]} = 0{,}427[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin \theta_3}(2{,}8+14{,}2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8{,}89[-])^{0,2} \cdot 1{,}0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (5{,}0[mm])^2}{\sin(90[^o])}(2{,}8+14{,}2 \cdot (0{,}427[-])^2)}{1{,}0} = 49{,}012[kN]$$

(table 7.2)

Punching shear failure for T gap joints:

$$d_2 = 38{,}0[mm] \leq d_o - 2t_0 = 88{,}9[mm] - 2 \cdot 5{,}0[mm] = 78{,}9[mm] \text{ (table 7.2)}$$

$$N_{2,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_2 \cdot \frac{\frac{1+\sin \theta_{1,2}}{2 \sin^2 \theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 5{,}0[mm] \cdot \pi \cdot 38{,}0[mm] \cdot \frac{\frac{1+\sin(90[°])}{2 \sin^2(90[°])}}{1{,}0} = 80{,}986[kN]$$

(table 7.2)

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd2} = min(N_{2Rd}, N_{t,Rd}) = min(80{,}986[kN]; 49{,}012[kN]) = 49{,}012[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd1,2} = 0{,}9 \cdot 49{,}012[kN] = 44{,}111[kN] \text{ (table 7.7)}$$

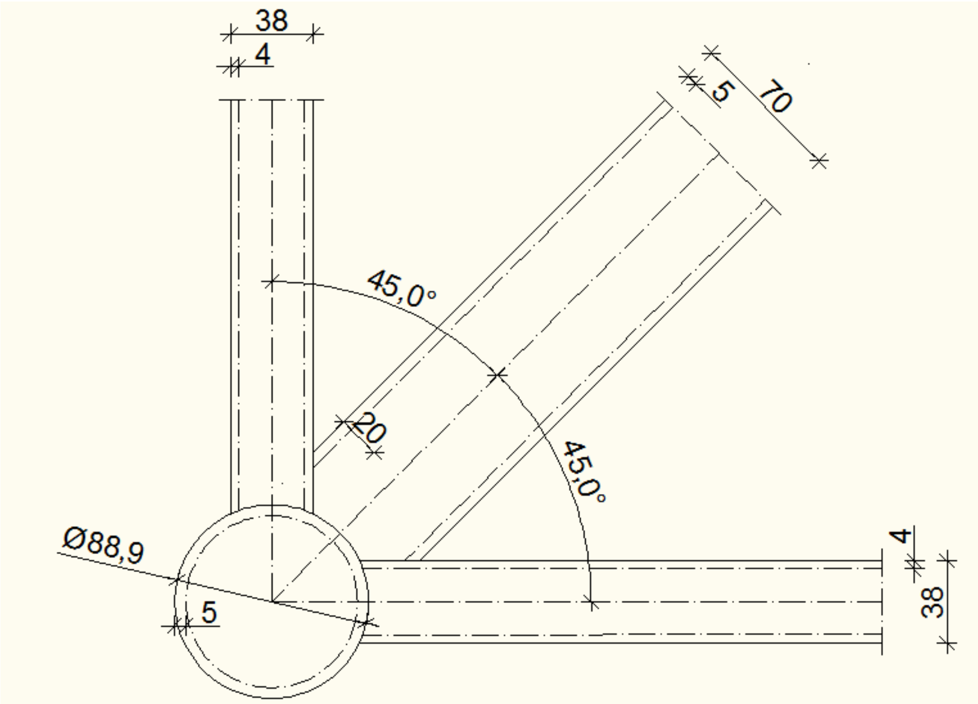
The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{1{,}94[kN]}{44{,}111[kN]} = 0{,}03[-] < 1{,}0[-]$$

Condition fulfilled.

Plane XY

The node 8, the XY direction of the main the bar to connect 3 rods- with numbers 15, 57 and 240. Because the bar with the number 57 is the smallest internal force, node can be designed as shown below:



Screenshot of the program [b]- geometry of node 8 in plane XY

The connection rod 240 and with numbers 15 is identical to the XZ and YZ planes. Therefore only shown how the dimension the bar with the number 57 for this node.

Node Y Plane XY

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_1}{d_0} = \frac{70{,}0[mm]}{88{,}9[mm]} = 0{,}79[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_1}{t_1} = \frac{70,0[mm]}{5,0[mm]} = 14,0[-] \leq 50,0$

Condition fulfilled.

Chord face failure:

Factors:

$k_p = 1,0[-]$ (table 7.2) $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88,9[mm]}{2 \cdot 5,0[mm]} = 8,89[-]$

The ratio of the mean diameter or width of the brace members (like for T node):

$$\beta = \frac{d_1}{d_0} = \frac{70,0[mm]}{88,9[mm]} = 0,787[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{sin\theta_3}(2,8+14,2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8,89[-])^{0,2} \cdot 1,0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (5,0[mm])^2}{sin(90[^o])}(2,8+14,2 \cdot (0,787[-])^2)}{1,0} = 105,453[kN]$$

(table 7.2)

Punching shear failure for T gap joints:

$$d_1 = 70,0[mm] \leq d_o - 2t_0 = 88,9[mm] - 2 \cdot 5,0[mm] = 78,9[mm] \text{ (table 7.2)}$$

$$N_{1,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{\frac{1+sin\ \theta_{1,2}}{2\ sin^2\ \theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 5,0[mm] \cdot \pi \cdot 70,0[mm] \cdot \frac{\frac{1+sin\ (90[°])}{2\ sin^2(90[°])}}{1,0} =$$

$$149,185[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1} = min(N_{1Rd}, N_{t,Rd}) = min(149,185[kN]; 105,453[kN]) = 105,453[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd1,2} = 0,9 \cdot 105,453[kN] = 94,908[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{1,15[kN]}{94,908[kN]} = 0,01[-] < 1,0[-]$$

Welded between orthogonal bars

Simplified method for design resistance of fillet weld

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_z = N_{Ed} \cos\theta_{1,2} = 2,99[kN] \cdot \cos(45[^o]) = 2,114[kN]$$

$$N_x = N_{Ed}\ \sin\theta_{1,2} = 2{,}99[kN]\cdot\sin(45[^o]) = 2{,}114[kN]$$

Surface area (from program [b]):

$$A_w = 7{,}875[cm^2]$$

Normal stresses of tension force:

$$\tau_H = \frac{N_z}{A_w} = \frac{2{,}114[kN]}{7{,}875[cm^2]} = 0{,}268\left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_V = \frac{N_x}{A_w} = \frac{2{,}114[kN]}{7{,}875[cm^2]} = 0{,}268\left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau = \sqrt{\tau_V^2 + \tau_H^2} = \sqrt{\left(0{,}268\left[\frac{kN}{cm^2}\right]\right)^2 + \left(0{,}268\left[\frac{kN}{cm^2}\right]\right)^2} = 0{,}379\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w\cdot\gamma_{M2}} = \frac{36\left[\frac{kN}{cm^2}\right]/\sqrt{3}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

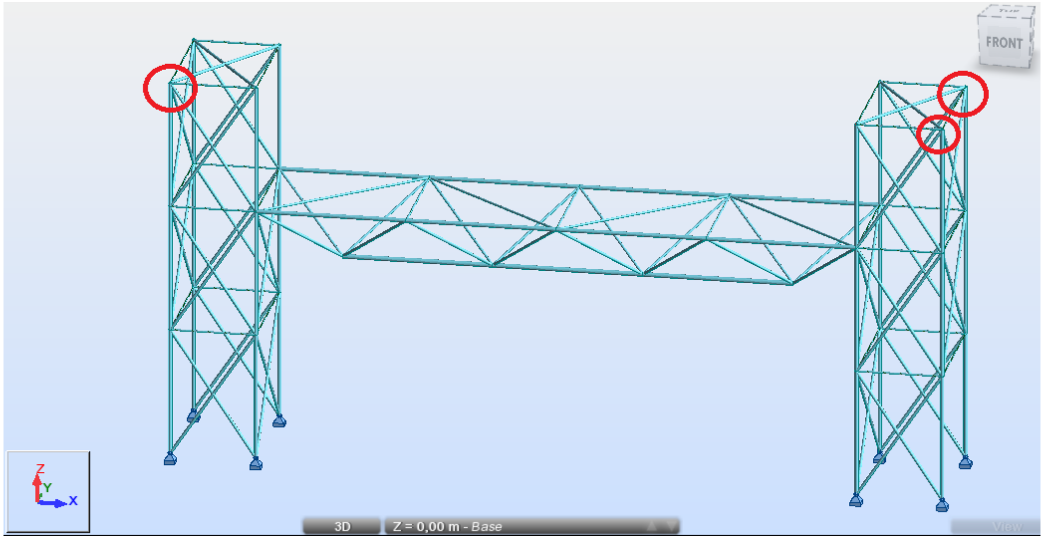
Condition:

$$\tau = 0{,}379\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

Node 4, 60, 64

The following are a schematic the calculations made when designing the node number 4, 60, 64 which were presented in the figure below:



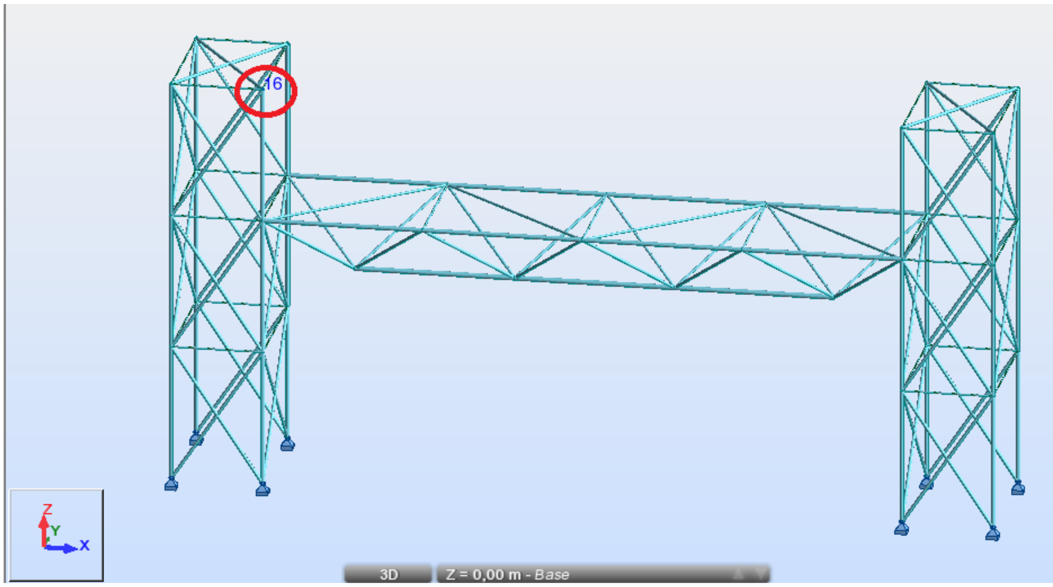
Screenshot of the program [a]- nodes 4, 60, 64.

Geometry of the connections are the same like connection in node 8 and force is smaller in node 4, 60 and 64 is smaller than in node 8. Thus, the connection can be designed in the same way like in node 8.

This nodes was presented on the drawings 07- Steel Footbridge- node 4, 8, 60, 64, which was attached to this work.

Node 16

The following is a schematic the calculations made when designing the node number 16, which was presented in the figure below:



Screenshot of the program [a]- node 16

This node connects the rods numbered 19, 21, 57, 20 and 61.

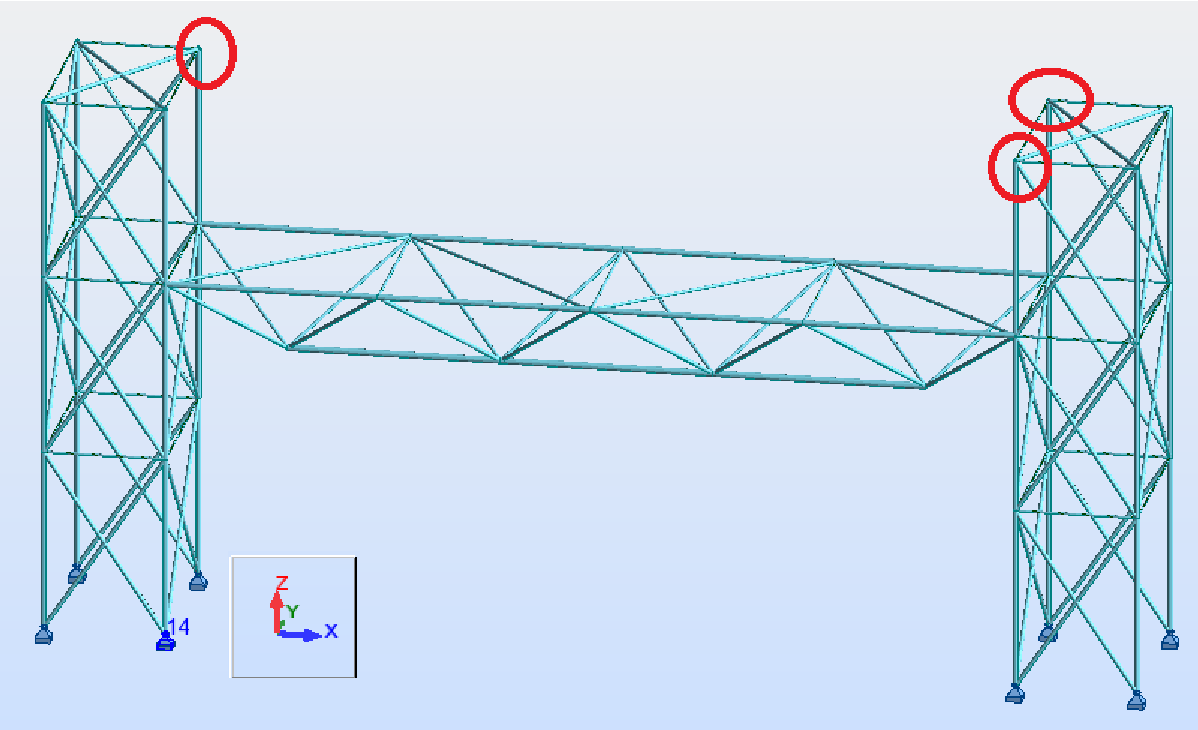
This node was presented on the drawings 08- Steel Footbridge- node 12, 16, 53, 56, which was attached to this work.

This node is very similar to the node number 8. Differs only the lack of diagonal the bar in the YZ plane. The remaining geometry node is the same as the node number 8. The normal force acting on bar is 10,13kN (this is the value that is smaller than the node 8). Therefore, the node can be performed in the same manner as said eighth node.

It is recommended to design the steel plate in the XZ plane fixed for the help of two straps to the bars with numbers 19 and 20 - using two M14 bolts class 8.8. Other bars and rods are joined by a fillet weld thickness of 3mm.

Node 12, 53, 56

The following are a schematic the calculations made when designing the node number 12, 53, 56 which were presented in the figure below:



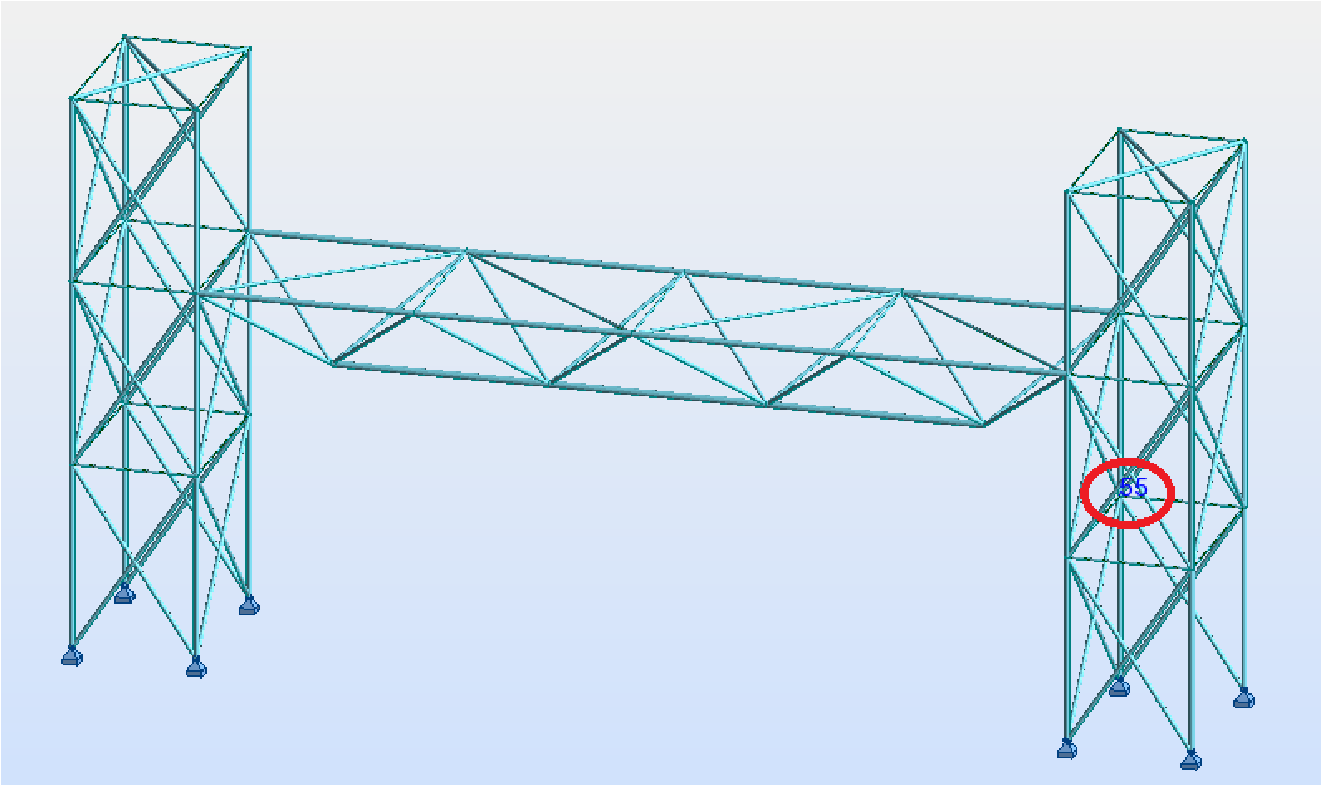
Screenshot of the program [a]- nodes 12, 53, 56

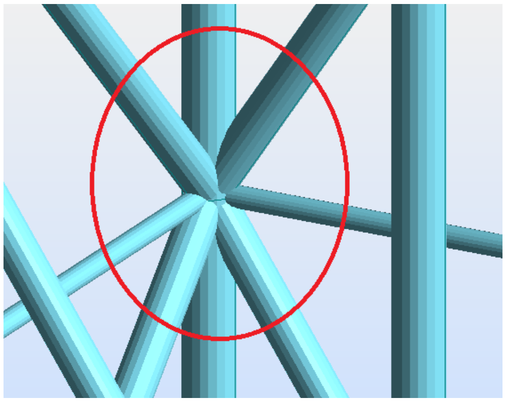
Geometry of the connections are the same like connection in node 16 and force is smaller in node 12, 53, 56 is smaller than in node 16. Thus, the connection can be designed in the same way like in node 16.

This nodes was presented on the drawings 08- Steel Footbridge- node 12, 16, 53, 56, which was attached to this work.

Node 55

The following is a schematic the calculations made when designing the node number 55, which was presented in the figure below:





Screenshot of the program [a]- node 55

This node connects the rods numbered 129, 130, 156, 132, 169, 166, 134 and 163.

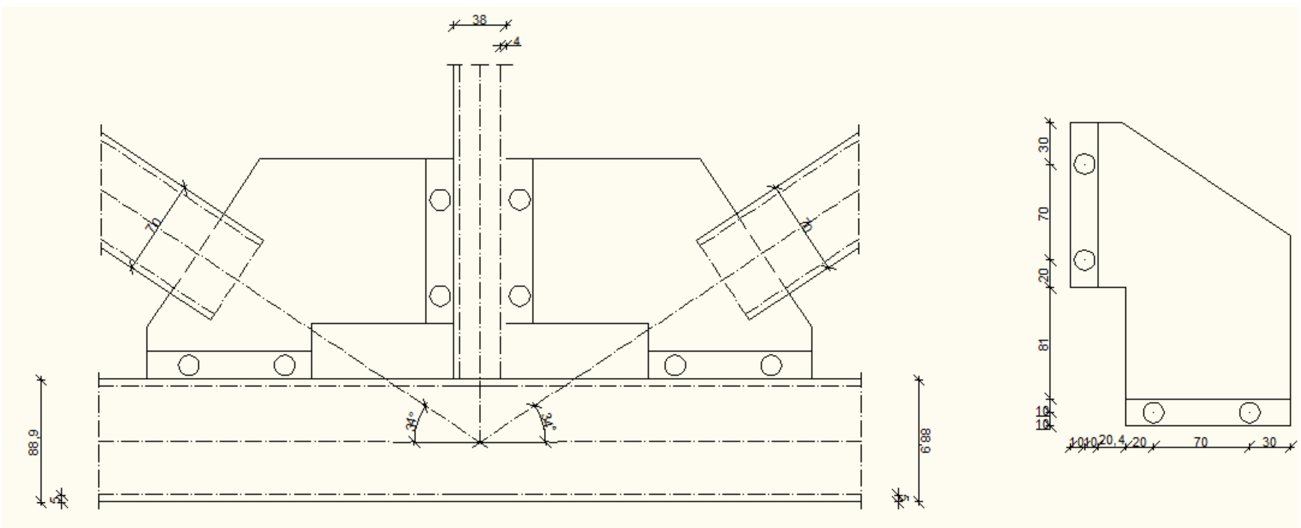
A node is defined as KT type according to figure 7.1, and in plane YZ and XZ have different geometry. So at first shows calculated for plane YZ like for node Y and then like for node K, then was performed the same calculation but like for plane XZ.

PLANE ZX

Due to the complexity of the construction of the node (in its place combines up to eight bars) decided to use steel plates.

Steel plate is welded to the rod number 163 and 166, with numbers rods 129, 130, 134 are connected by means of steel overlays. Steel caps are fixed to the steel plate by means of bolts M14 class 8.8.

The bolted connection is category A. The geometry of the screw connection meets all the requirements specified in table 3.3.



Screenshot of the program [b]- geometry of node 55 XZ with geometry of plate

Forces:

$N_{Ed,129} = 133{,}41[kN]$ $N_{Ed,130} = 46{,}37[kN]$ $N_{Ed,156} = 34{,}38[kN]$

$N_{Ed,132} = -15{,}83[kN]$ $N_{Ed,169} = -27{,}34[kN]$ $N_{Ed,166} = 59{,}01[kN]$

$N_{Ed,134} = -16{,}48[kN]$ $N_{Ed,163} = -33{,}57[kN]$

Geometry of the connection:

$d_1 = 70{,}0[mm]$ $d_2 = 38{,}0[mm]$ $d_0 = 88{,}9[mm]$

$t_1 = 5{,}0[mm]$ $t_2 = 4{,}0[mm]$ $t_0 = 5{,}0[mm]$

The angles of inclination of diagonals:

$\theta_1 = 34{,}0[°]$ $\theta_2 = 90{,}0[°]$

Geometry of the plates:

Cross section:

$A_0 = 263{,}290[cm^2]$ $A_1 = 24{,}0[cm^2]$ $A_2 = 24{,}0[cm^2]$

Thickness:

$t_0 = 10{,}0[mm]$ $t_1 = 10{,}0[mm]$ $t_2 = 10{,}0[mm]$

Resistance of steel plate (Big plate):

Like for node 8.

The condition resistance of section:

$$\frac{1{,}2 \cdot N_{Ed,166}}{N_{t,Rd0}} = \frac{1{,}2 \cdot 59{,}01[kN]}{5214{,}082\ [kN]} = 0{,}014 < 1{,}0$$

Condition fulfilled.

Resistance of steel plate (smaller plate):

Like for node 8.

The condition resistance of section:

$$\frac{1{,}2 \cdot N_{Ed,166}}{N_{t,Rd1,2}} = \frac{1{,}2 \cdot 59{,}01[kN]}{453{,}600\ [kN]} = 0{,}156 < 1{,}0$$

Condition fulfilled.

The steel plate was designed correctly.

Design resistance of individual fasteners

Like for node 8.

$$\mathrm{N}_1 = \mathrm{N}_2 = \mathrm{N}_3 = \mathrm{N}_4 = \frac{59{,}01[\mathrm{kN}]}{4} = 14{,}75[\mathrm{kN}]$$

The condition resistance of section (compared to the biggest force in bolt):

$$\frac{1{,}2 \cdot N_{Ed}}{N_{0Rd}} = \frac{1{,}2 \cdot 14{,}75[\mathrm{kN}]}{44{,}16\ [kN]} = 0{,}41 < 1{,}0$$

Welded connections

Welded between orthogonal bars and steel plates

$$\mathrm{N}_1 = \frac{N_{Ed} \cdot b}{a + b} = \frac{59{,}01[\mathrm{kN}] \cdot 65{,}2[mm]}{100{,}2[mm] + 65{,}2[mm]} = 23{,}261[kN]$$

$$N_2 = \frac{N_{Ed} \cdot a}{a+b} = \frac{59{,}01[\mathrm{kN}] \cdot 100{,}2[mm]}{100{,}2[mm] + 65{,}2[mm]} = 35{,}748[kN]$$

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_{z1} = N_1 \cos\theta_{1,2} = 23{,}261[kN] \cdot \cos(34[^o]) = 19{,}284[kN]$$

$$N_{x1} = N_1 \sin\theta_{1,2} = 23{,}261[kN] \cdot \sin(34[^o]) = 13{,}007[kN]$$

$$N_{z2} = N_2 \cos\theta_{1,2} = 35{,}748[kN] \cdot \cos(34[^o]) = 29{,}636[kN]$$

$$N_{x2} = N_2 \sin\theta_{1,2} = 35{,}748[kN] \cdot \sin(34[^o]) = 19{,}990[kN]$$

Surface area:

$$A_w = 120[mm] \cdot 3[mm] = 3{,}6[cm^2]$$

Normal stresses of tension force:

$$\tau_{H1} = \frac{N_{z1}}{A_w} = \frac{19{,}284[kN]}{3{,}6[cm^2]} = 5{,}357 \left[\frac{kN}{cm^2}\right]$$

$$\tau_{H2} = \frac{N_{z2}}{A_w} = \frac{29{,}636[kN]}{3{,}6[cm^2]} = 8{,}232 \left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_{V1} = \frac{N_{x1}}{A_w} = \frac{13{,}007[kN]}{3{,}6[cm^2]} = 3{,}613 \left[\frac{kN}{cm^2}\right]$$

$$\tau_{V2} = \frac{N_{x2}}{A_w} = \frac{19{,}990[kN]}{3{,}6[cm^2]} = 5{,}553 \left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau_1 = \sqrt{\tau_{V1}^2 + \tau_{H1}^2} = \sqrt{\left(5{,}357\left[\frac{kN}{cm^2}\right]\right)^2 + \left(8{,}232\left[\frac{kN}{cm^2}\right]\right)^2} = 9{,}821\left[\frac{kN}{cm^2}\right]$$

$$\tau_2 = \sqrt{\tau_{V2}^2 + \tau_{H2}^2} = \sqrt{\left(3{,}613\left[\frac{kN}{cm^2}\right]\right)^2 + \left(5{,}553\left[\frac{kN}{cm^2}\right]\right)^2} = 6{,}625\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = \frac{\frac{36\left[\frac{kN}{cm^2}\right]}{\sqrt{3}}}{0{,}8[-] \cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

Condition:

$$\tau_1 = 9{,}821\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

$$\tau_2 = 6{,}625\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Conditions fulfilled.

Connection between diagonal rod and steel plate

Adopted fillet welds of thickness equal to: $a = 3[mm]$

Simplified method for design resistance of fillet weld

Forces in weld:

$$N_1 = N_2 = \frac{59{,}01[kN]}{4} = 14{,}75[kN]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = \frac{\frac{360\left[\frac{N}{mm^2}\right]}{\sqrt{3}}}{0{,}8[-]\cdot 1{,}25[-]} = 207{,}846\left[\frac{N}{mm^2}\right] \quad \text{(formula 4.4)}$$

Independent of the orientation of the weld throat plane to the applied force, the design resistance per unit length should be determined from:

$F_{w,Rd} = l_s \cdot f_{vw,d} \cdot a = 35[mm] \cdot 207{,}846\left[\frac{N}{mm^2}\right] \cdot 3[mm] = 21{,}824[kN]$ (formula 4.3)

Condition:

$$F_{w,Ed} = 14{,}75[kN] \quad \leq \quad F_{w,Rd} = 21{,}824\,[kN]$$

Condition fulfilled.

Welded between orthogonal bars

Forces:

$$F_{\sigma\perp} = F_{\tau\perp} = \frac{N_{Ed,134}}{\sqrt{2}} = \frac{16{,}48[kN]}{\sqrt{2}} = 11{,}653[kN]$$

$$F_{\sigma II} = 0[kN]$$

Stresses:

$$\sigma_\perp = \frac{F_{\sigma\perp}}{A_w} = \frac{11{,}653[kN]}{\pi \cdot ((19[mm])^2 - (19[mm] - 3[mm])^2)} = 3{,}533\left[\frac{kN}{cm^2}\right]$$

$$\tau_\perp = \frac{F_{\sigma II}}{A_w} = \frac{11{,}653[kN]}{\pi \cdot ((19[mm])^2 - (19[mm] - 3[mm])^2)} = 3{,}533\left[\frac{kN}{cm^2}\right]$$

$$\tau_{II} = 0$$

Stress resultant:

$$\sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_{II}^2)} = \sqrt{\left(3{,}533\left[\frac{kN}{cm^2}\right]\right)^2 + 3\left(\left(3{,}533\left[\frac{kN}{cm^2}\right]\right)^2 + 0\right)} = 7{,}066\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = \frac{\frac{36\left[\frac{kN}{cm^2}\right]}{\sqrt{3}}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

Condition:

$$\tau = 7{,}066\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

NODE K planes YZ

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_{1,2}}{d_0} = \frac{70{,}0[mm]}{88{,}9[mm]} = 0{,}79[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_{1,2}}{t_{1,2}} = \frac{70{,}0[mm]}{5{,}0[mm]} = 14[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

Factors:

$$k_p = 1{,}0[-] \qquad \gamma = \frac{d_0}{2\cdot t_0} = \frac{88{,}9[mm]}{2\cdot 5{,}0[mm]} = 8{,}89[-] \text{(table 7.2)}$$

$$k_g = \gamma^{0,2}\cdot\left(1+\frac{0{,}024\cdot\gamma^{1,2}}{1+exp(0{,}5\cdot\frac{g}{t_0}-1{,}33)}\right) = (8{,}89[-])^{0,2}\cdot\left(1+\frac{0{,}024\cdot(8{,}89[-])^{1,2}}{1+exp(0{,}5\cdot\frac{-31{,}4[mm]}{5{,}0[mm]}-1{,}33)}\right) = 2{,}054[-]$$

(table 7.2)

Chord face failure:

$$N_{t,Rd} = \frac{k_g\cdot k_p\cdot f_{y0}\cdot t_0^2}{sin\,\theta_{1,2}}(1{,}8+10{,}2\frac{d_{1,2}}{d_0})/\gamma_{M5} = \frac{2{,}054[-]\cdot 1{,}0[-]\cdot 235\left[\frac{N}{mm^2}\right]\cdot(5[mm])^2}{sin(34)[°]}(1{,}8+10{,}2\frac{70{,}0[mm]}{88{,}9[mm]})/1{,}0 = 212{,}161[kN] \text{ (table 7.2)}$$

Punching shear failure for K gap joints:

$$d_{1,2} = 70{,}0[mm] \le d_o - 2t_0 = 88{,}9[mm] - 2\cdot 5{,}0[mm] = 78{,}9[mm] \text{ (table 7.2)}$$

$$N_{1,2,Rd} = \frac{f_{y0}}{\sqrt{3}}\cdot t_0\cdot\pi\cdot d_{1,2}\cdot\frac{\frac{1+sin\,\theta_{1,2}}{2\,sin^2\theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}}\cdot 5{,}0[mm]\cdot\pi\cdot 70{,}0[mm]\cdot\frac{\frac{1+sin\,34[°]}{2\,sin^2 34[°]}}{1{,}0} = 321{,}939[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1,2} = min(N_{1,2Rd}, N_{t,Rd}) = min(321{,}939[kN]; 212{,}161[kN]) = 212{,}161[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu\cdot N_{Rd1,2} = 0{,}9\cdot 212{,}161[kN] = 190{,}945[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{59{,}01[kN]}{190{,}945[kN]} = 0{,}31[-] < 1{,}0[-]$$

Condition fulfilled.

NODE T XZ

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \le \frac{d_2}{d_0} = \frac{38{,}0[mm]}{88{,}9[mm]} = 0{,}43[-] \le 1{,}0$

Condition fulfilled.

Chords:

Tension: $$10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$$

Condition fulfilled.

Compression (class 1): $$10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$$

Condition fulfilled.

Braces:

Tension: $$\frac{d_2}{t_2} = \frac{38{,}0[mm]}{5{,}0[mm]} = 7{,}6[-] \leq 50{,}0$$

Condition fulfilled.

Chord face failure:

Factors:

$$k_p = 1{,}0[-] \qquad \gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 5{,}0[mm]} = 8{,}89[-] \text{(table 7.2)}$$

The ratio of the mean diameter or width of the brace members (like for T node):

$$\beta = \frac{d_2}{d_0} = \frac{38{,}0[mm]}{88{,}9[mm]} = 0{,}427[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin \theta_3}(2{,}8 + 14{,}2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8{,}89[-])^{0,2} \cdot 1{,}0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (5{,}0[mm])^2}{\sin(90[^o])}(2{,}8 + 14{,}2 \cdot (0{,}427[-])^2)}{1{,}0} = 49{,}012[kN]$$

(table 7.2)

Punching shear failure for T gap joints:

$$d_2 = 38{,}0[mm] \leq d_o - 2t_0 = 88{,}9[mm] - 2 \cdot 5{,}0[mm] = 78{,}9[mm] \text{ (table 7.2)}$$

$$N_{2,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_2 \cdot \frac{\frac{1 + \sin \theta_{1,2}}{2 \sin^2 \theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 5{,}0[mm] \cdot \pi \cdot 38{,}0[mm] \cdot \frac{\frac{1 + \sin(90[°])}{2 \sin^2(90[°])}}{1{,}0} = 80{,}986[kN]$$

(table 7.2)

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd2} = min(N_{2Rd}, N_{t,Rd}) = min(80{,}986[kN]; 49{,}012[kN]) = 49{,}012[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd1,2} = 0{,}9 \cdot 49{,}012[kN] = 44{,}111[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

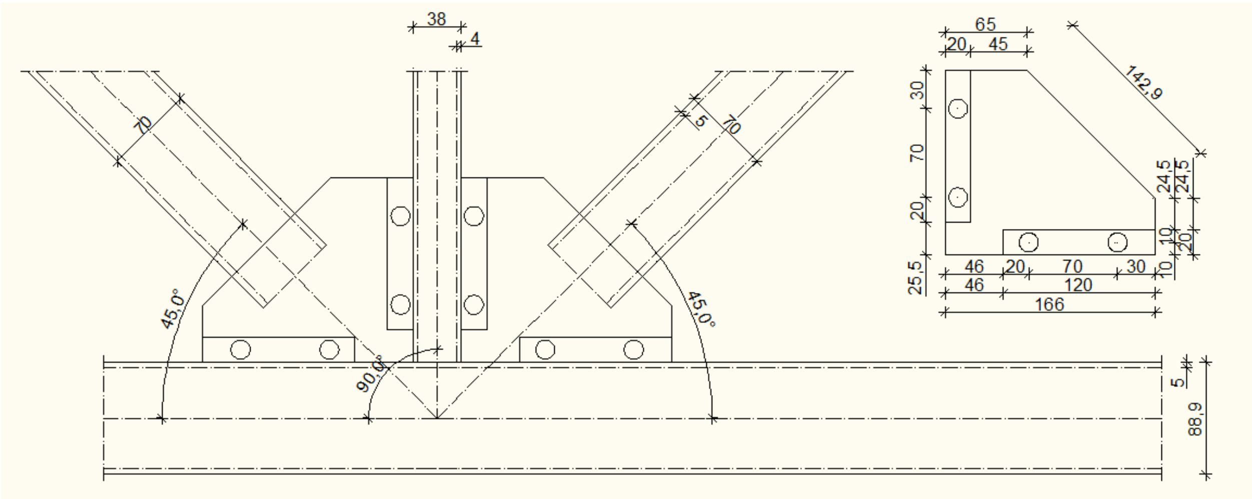
$$\frac{N_{Ed}}{N_{Rd}} = \frac{-16{,}48[kN]}{44{,}111[kN]} = 0{,}37[-] < 1{,}0[-]$$

Condition fulfilled.

Plane YZ

Steel plate is welded to the rod number 156 and 169, with numbers rods 129,130 and 132 are connected by means of steel overlays. Steel caps are fixed to the steel plate by means of bolts M14 class 8.8.

The geometry of the screw connection meets all the requirements specified in table 3.3.



Screenshot of the program [b]- geometry of node 55 YZ with geometry of plate

Geometry of the connection:

$d_1 = 70{,}0[mm]$	$d_2 = 38{,}0[mm]$	$d_0 = 88{,}9[mm]$
$t_1 = 5{,}0[mm]$	$t_2 = 4{,}0[mm]$	$t_0 = 5{,}0[mm]$

The angles of inclination of diagonals:

$\theta_1 = 45{,}0[°]$ $\theta_2 = 90{,}0[°]$

Geometry of the plates:

Cross section:

$A_0 = 190{,}526[cm^2]$	$A_1 = 24{,}0[cm^2]$	$A_2 = 24{,}0[cm^2]$

Thickness:

$t_0 = 10{,}0[mm]$ $t_1 = 10{,}0[mm]$ $t_2 = 10{,}0[mm]$

Resistance of steel plate (Big plate):

In the case of elements asymmetrically connected in the nodes via mechanical fasteners category A design tension resistance defines itself as resistance boundary:

The reduction factor (linear interpolation): $\beta_2 = 0{,}70[-]$ (table 3.8)

The net area of a cross section:

$$A_{net\,0} = min\left(A_0 - \left(nd_0 - \left(\frac{s_1^2}{4p_1} + \frac{s_2^2}{4 \cdot p_2}\right)\right)t_0, A_0 - d_0 t_0\right) = min\left(190{,}526[cm^2] - \left(4 \cdot 15{,}0[mm] - \left(\frac{(70[mm])^2}{4 \cdot 35[mm]} + \frac{(26[mm])^2}{4 \cdot 70[mm]}\right)\right) \cdot 10{,}0[mm]; 190{,}526[cm^2] - 15{,}0[mm] \cdot 10{,}0[mm]\right) = min(188{,}267[cm^2];\ 189{,}026[cm^2]) = 188{,}267[cm^2]$$

(according to EN 1991-1-1 6.2.2.2.(4))

The design tension resistance (according EN 1993-1-1 5.2.3.(2) b)):

$$N_{u,Rd0} = \frac{\beta_2 A_{net\,0} f_u}{\gamma_{M2}} = \frac{0{,}70[-] \cdot 188{,}267[cm^2] \cdot 360\left[\frac{N}{mm^2}\right]}{1{,}25[-]} = 3795{,}463[kN]$$ (formula 4.7 EN 1991-1-1)

The design plastic resistance of the gross cross-section:

$$N_{pl,Rd0} = \frac{A_0\, f_u}{\gamma_{M0}} = \frac{190{,}526[cm^2] \cdot 360\left[\frac{N}{mm^2}\right]}{1{,}00[-]} = 6858{,}936\ [kN]$$ (formula 4.6EN 1991-1-1)

The design value of the resistance to tension forces:

$$N_{t,Rd0} = min(N_{u,Rd0}, N_{pl,Rd0}) = min(3795{,}463[kN], 6858{,}936\ [kN]) = 3795{,}463[kN]$$

The Lug connects angle members and their fasteners to a gusset or other supporting part and should be designed to transmit a force 1,2 times the force in the out stand of the angle connected. (according 3.10.4 (1)).

The condition resistance of section:

$$\frac{1{,}2 \cdot N_{Ed}}{N_{t,Rd0}} = \frac{1{,}2 \cdot 34{,}38[kN]}{3795{,}463\ [kN]} = 0{,}011 < 1{,}0$$

Condition fulfilled.

Resistance of steel plate:

Like for plane XZ.

$$\mathrm{N_1} = \mathrm{N_2} = \mathrm{N_3} = \mathrm{N_4} = \frac{34{,}38[\mathrm{kN}]}{4} = 8{,}60[\mathrm{kN}]$$

The condition resistance of section:

$$\frac{1{,}2 \cdot N_{Ed,153}}{N_{0Rd}} = \frac{1{,}2 \cdot 8{,}60[\mathrm{kN}]}{44{,}16\ [kN]} = 0{,}23 < \quad 1{,}0$$

Condition fulfilled.

The steel plate was designed correctly.

Welded connections

Welded between orthogonal bars and steel plates

$$\mathrm{N_1} = \frac{N_{Ed} \cdot b}{a+b} = \frac{34{,}38[\mathrm{kN}] \cdot 57{,}6[mm]}{78{,}6[mm] + 57{,}6[mm]} = 14{,}540[kN]$$

$$\mathrm{N_2} = \frac{N_{Ed} \cdot a}{a+b} = \frac{59{,}01[\mathrm{kN}] \cdot 78{,}6[mm]}{78{,}6[mm] + 57{,}6[mm]} = 19{,}840[kN]$$

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_{z1} = N_1 \cos\theta_{1,2} = 14{,}540[kN] \cdot \cos(45[^o]) = 10{,}281[kN]$$

$$N_{x1} = N_1 \sin\theta_{1,2} = 14{,}540[kN] \cdot \sin(45[^o]) = 10{,}281[kN]$$

$$N_{z2} = N_2 \cos\theta_{1,2} = 19{,}840[kN] \cdot \cos(45[^o]) = 14{,}029[kN]$$

$$N_{x2} = N_2 \sin\theta_{1,2} = 19{,}840[kN] \cdot \sin(45[^o]) = 14{,}029[kN]$$

Surface area:

$$A_w = 120[mm] \cdot 3[mm] = 3{,}6[cm^2]$$

Normal stresses of tension force:

$$\tau_{H1} = \frac{N_{z1}}{A_w} = \frac{10{,}281[kN]}{3{,}6[cm^2]} = 2{,}856\left[\frac{kN}{cm^2}\right]$$

$$\tau_{H2} = \frac{N_{z2}}{A_w} = \frac{14{,}029[kN]}{3{,}6[cm^2]} = 3{,}897\left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_{V1} = \frac{N_{x1}}{A_w} = \frac{10{,}281[kN]}{3{,}6[cm^2]} = 2{,}856\left[\frac{kN}{cm^2}\right]$$

$$\tau_{V2} = \frac{N_{x2}}{A_w} = \frac{14{,}029[kN]}{3{,}6[cm^2]} = 3{,}897\left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau_1 = \sqrt{\tau_{V1}^2 + \tau_{H1}^2} = \sqrt{\left(2{,}856\left[\frac{kN}{cm^2}\right]\right)^2 + \left(2{,}856\left[\frac{kN}{cm^2}\right]\right)^2} = 4{,}039\left[\frac{kN}{cm^2}\right]$$

$$\tau_2 = \sqrt{\tau_{V2}^2 + \tau_{H2}^2} = \sqrt{\left(3{,}897\left[\frac{kN}{cm^2}\right]\right)^2 + \left(3{,}897\left[\frac{kN}{cm^2}\right]\right)^2} = 5{,}511\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = \frac{\frac{36\left[\frac{kN}{cm^2}\right]}{\sqrt{3}}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

Condition:

$$\tau_1 = 4{,}039\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

$$\tau_2 = 5{,}511\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

Connection between diagonal rod and steel plate

Adopted fillet welds of thickness equal to: $a = 3[mm]$

Simplified method for design resistance of fillet weld

Forces in weld:

$$N_1 = N_2 = \frac{34{,}38[kN]}{4} = 8{,}595[kN]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} = \frac{360\left[\frac{N}{mm^2}\right]/\sqrt{3}}{0{,}8[-]\cdot 1{,}25[-]} = 207{,}846\left[\frac{N}{mm^2}\right] \text{ (formula 4.4)}$$

Independent of the orientation of the weld throat plane to the applied force, the design resistance per unit length should be determined from:

$F_{w,Rd} = l_s \cdot f_{vw,d} \cdot a = 35[mm] \cdot 207{,}846\left[\frac{N}{mm^2}\right] \cdot 3[mm] = 21{,}824[kN]$ (formula 4.3)

Condition:

$$F_{w,Ed} = 8{,}595[kN] \quad \leq \quad F_{w,Rd} = 21{,}824\ [kN]$$

Condition fulfilled.

Welded between orthogonal bars

Forces:

$$\mathrm{F}_{\sigma\perp} = F_{\tau\perp} = \frac{N_{Ed,156}}{\sqrt{2}} = \frac{34{,}38[kN]}{\sqrt{2}} = 24{,}310[kN]$$

$$\mathrm{F}_{\sigma\mathrm{II}} = 0[kN]$$

Stresses:

$$\sigma_{\perp} = \frac{\mathrm{F}_{\sigma\perp}}{\mathrm{A_w}} = \frac{24{,}310[kN]}{\pi\cdot((19[mm])^2 - (19[mm] - 3[mm])^2)} = 7{,}370\left[\frac{kN}{cm^2}\right]$$

$$\tau_{\perp} = \frac{\mathrm{F}_{\sigma\mathrm{II}}}{\mathrm{A_w}} = \frac{24{,}310[kN]}{\pi\cdot((19[mm])^2 - (19[mm] - 3[mm])^2)} = 7{,}370\left[\frac{kN}{cm^2}\right]$$

$$\tau_{\mathrm{II}} = 0$$

Stress resultant:

$$\sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\mathrm{II}}^2)} = \sqrt{\left(7{,}370\left[\frac{kN}{cm^2}\right]\right)^2 + 3\left(\left(7{,}370\left[\frac{kN}{cm^2}\right]\right)^2 + 0\right)} = 14{,}74\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w\cdot\gamma_{M2}} = \frac{\frac{36\left[\frac{kN}{cm^2}\right]}{\sqrt{3}}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{(formula 4.4)}$$

Condition:

$$\tau = 14{,}74\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

NODE K planes YZ

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_{1,2}}{d_0} = \frac{70{,}0[mm]}{88{,}9[mm]} = 0{,}79[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_{1,2}}{t_{1,2}} = \frac{70{,}0[mm]}{5{,}0[mm]} = 14[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

Factors:

$k_p = 1{,}0[-]$ $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 5{,}0[mm]} = 8{,}89[-]$(table 7.2)

$$k_g = \gamma^{0,2} \cdot \left(1 + \frac{0{,}024 \cdot \gamma^{1,2}}{1 + exp(0{,}5 \cdot \frac{g}{t_0} - 1{,}33)}\right) = (8{,}89[-])^{0,2} \cdot \left(1 + \frac{0{,}024 \cdot (8{,}89[-])^{1,2}}{1 + exp(0{,}5 \cdot \frac{-24[mm]}{5{,}0[mm]} - 1{,}33)}\right) = 2{,}047[-]$$

(table 7.2)

Chord face failure:

$$N_{t,Rd} = \frac{k_g \cdot k_p \cdot f_{y0} \cdot t_0^2}{sin\ \theta_{1,2}} (1{,}8 + 10{,}2 \frac{d_{1,2}}{d_0}) / \gamma_{M5} = \frac{2{,}047[-] \cdot 1{,}0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (5[mm])^2}{sin(45)[°]} (1{,}8 + 10{,}2 \frac{70{,}0[mm]}{88{,}9[mm]}) / 1{,}0 = 167{,}209[kN]$$ (table 7.2)

Punching shear failure for K gap joints:

$d_{1,2} = 70{,}0[mm] \leq d_o - 2t_0 = 88{,}9[mm] - 2 \cdot 5{,}0[mm] = 78{,}9[mm]$ (table 7.2)

$$N_{1,2,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_{1,2} \cdot \frac{\frac{1 + sin\ \theta_{1,2}}{2\ sin^2 \theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 5{,}0[mm] \cdot \pi \cdot 70{,}0[mm] \cdot \frac{\frac{1 + sin\ 45[°]}{2\ sin^2 45[°]}}{1{,}0} = 254{,}675[kN]$$ (table 7.2)

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd\,1,2} = \min(N_{1,2Rd}, N_{t,Rd}) = \min(254{,}675[kN]; 167{,}209[kN]) = 167{,}209[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd\,1,2} = 0{,}9 \cdot 167{,}209[kN] = 150{,}488[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{34{,}38[kN]}{150{,}488[kN]} = 0{,}24[-] < 1{,}0[-]$$

Condition fulfilled.

NODE T YZ

Like in plane XZ

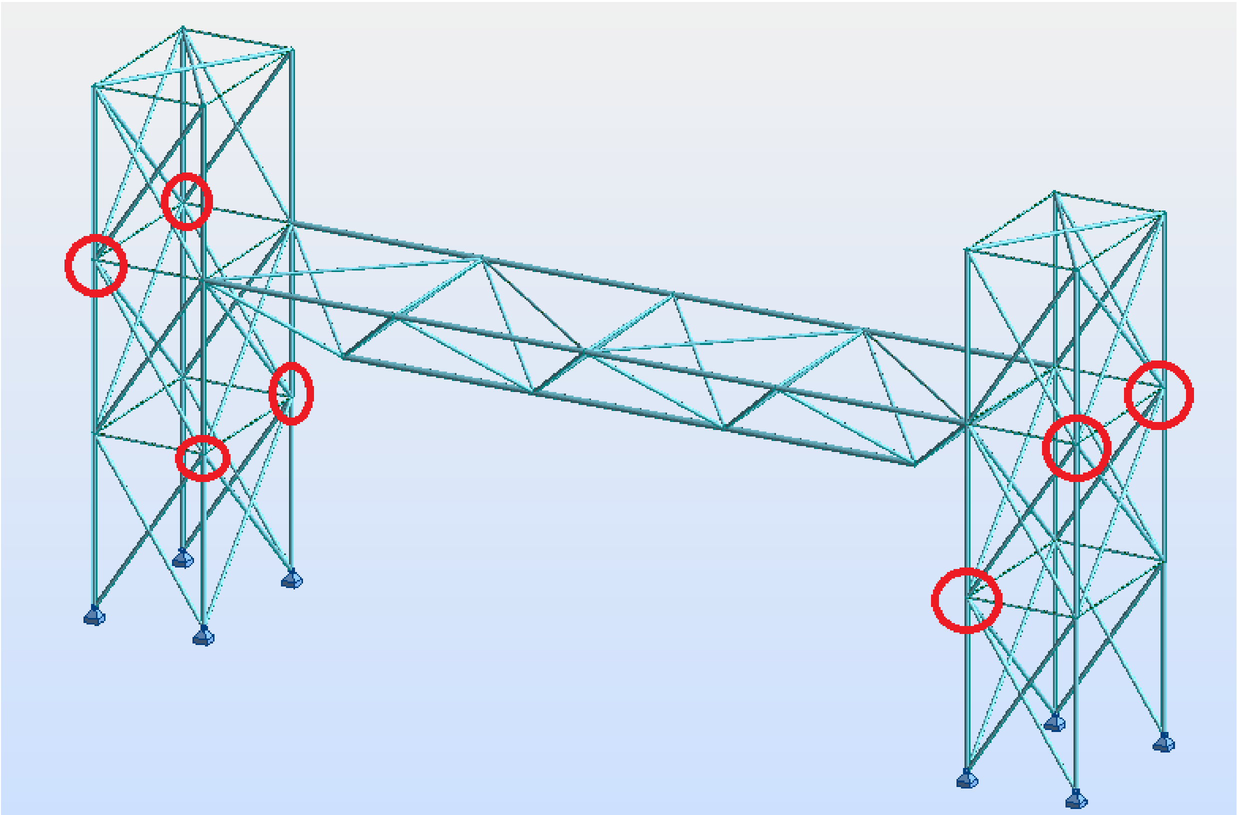
The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{15{,}83[kN]}{44{,}111[kN]} = 0{,}36[-] < 1{,}0[-]$$

Condition fulfilled.

Node 52, 63, 59, 13, 9, 7, 3

The following are a schematic the calculations made when designing the node number 52, 63, 59, 13, 9, 7, 3 which were presented in the figure below:



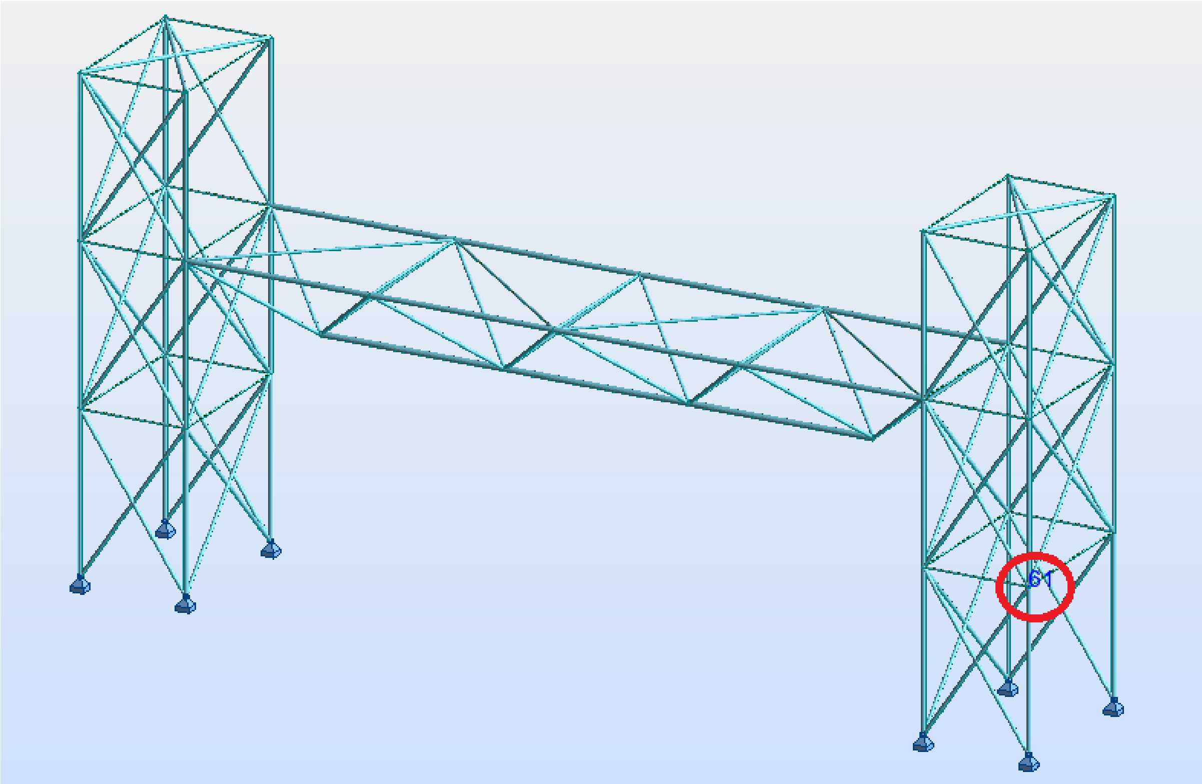
Screenshot of the program [a]- nodes 52, 63, 59, 13, 9, 7, 3

Geometry of the connections are the same like connection in node 55 and force is smaller in node 52, 63, 59, 13, 9, 7, 3 is smaller than in node 55. Thus, the connection can be designed in the same way like in node 55.

This nodes was presented on the drawings 09- Steel Footbridge- node 3, 7, 9, 13, 52, 55, 59, 63, which was attached to this work.

Node 61

The following is a schematic the calculations made when designing the node number 61, which was presented in the figure below:



Screenshot of the program [a]- node 61

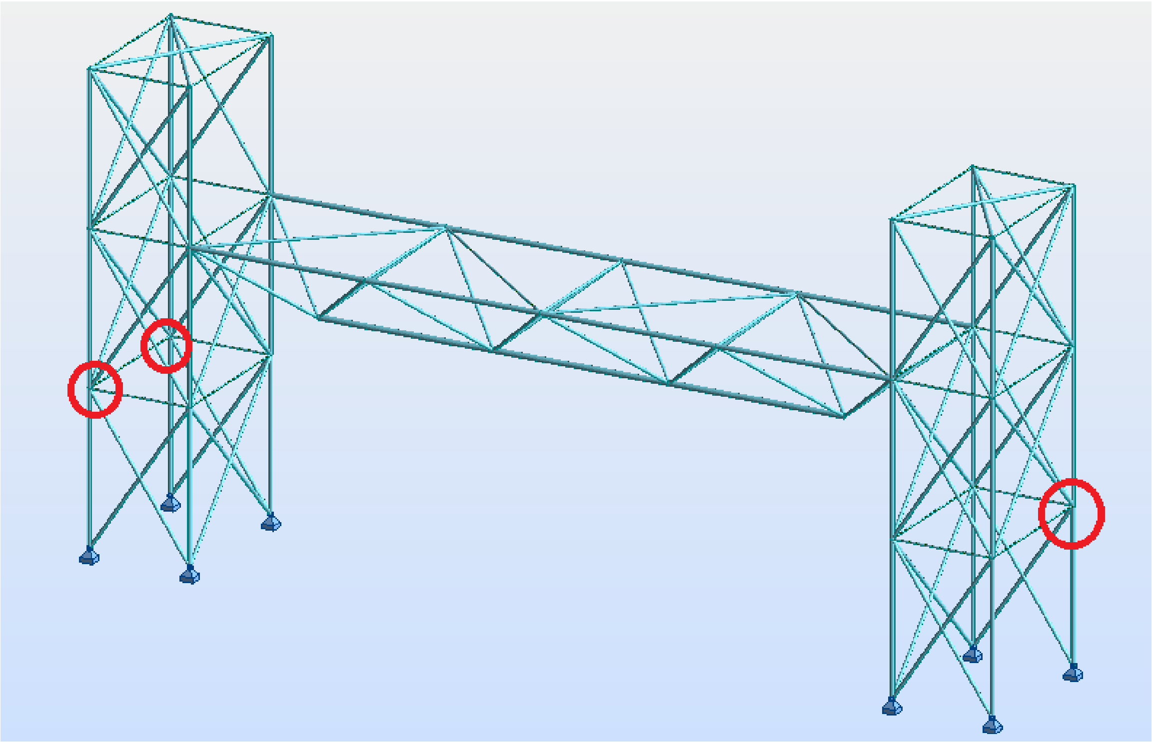
This node is very similar to the node number 55. Differs is only the lack of the diagonal bar in the YZ plane. The remaining geometry node is the same as the node number 55. The normal force acting on bar is 42,05kN (this is the value that is smaller than the node 55). Therefore, the node can be performed in the same manner as said eighth node.

It is recommended to design the steel plate in the XZ plane and YZ plane, used screw M14 class 8.8.. Other bars and rods are joined by a fillet weld thickness of 3mm.

This nodes was presented on the drawings 10- Steel Footbridge- node 2, 6, 57, 61, which was attached to this work.

Node 2, 6, 57

The following are a schematic the calculations made when designing the node number 2, 6, 57 which were presented in the figure below:



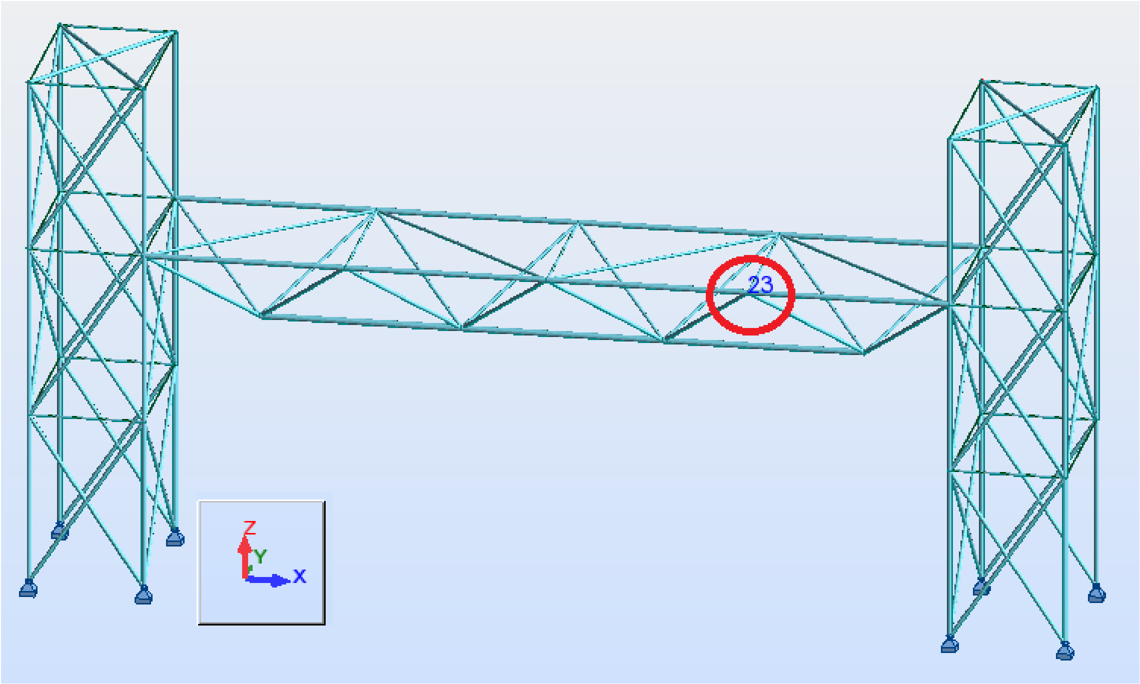
Screenshot of the program [a]- nodes 2, 6, 57.

Geometry of the connections are the same like connection in node 61 and force is smaller in node 2, 6, 57 is smaller than in node 61. Thus, the connection can be designed in the same way like in node 61.

This nodes was presented on the drawings 10- Steel Footbridge- node 2, 6, 57, 61, which was attached to this work.

Node 23

The following is a schematic the calculations made when designing the node number 23, which was presented in the figure below:



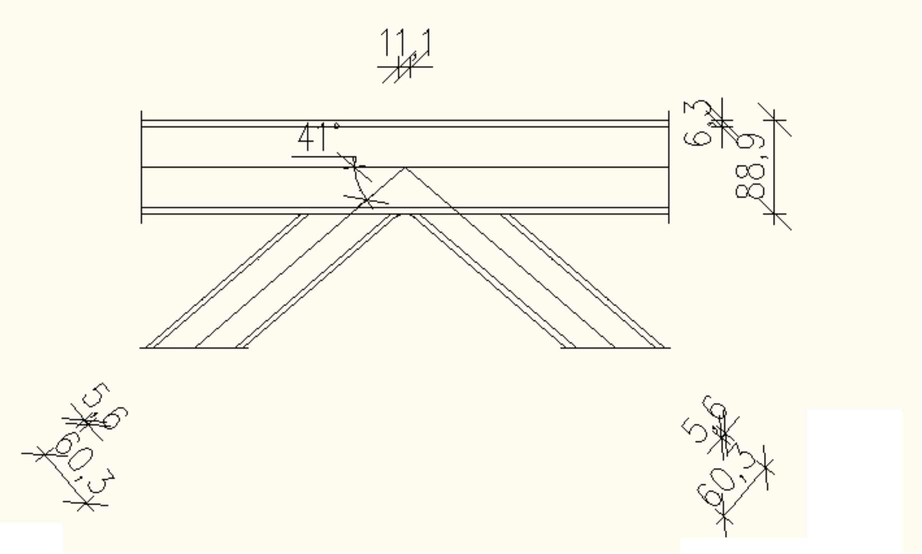
Screenshot of the program [a]- node 23

This node connects the rods numbered 31, 32, 51, 54, 36.

A node is defined as K type in a plane ZX and like type T in a plane YX according to figure 7.1. When the calculations are dimensioned first node K, then T.

This node was presented on the drawings 11- Steel Footbridge- node 18, 21, 23, which was attached to this work.

Plane ZX – node K



Screenshot of the program [b]- geometry of node 23

Geometry of the connection:

$d_1 = 60{,}3[mm]$ $d_2 = 60{,}3[mm]$ $d_0 = 88{,}9[mm]$
$t_1 = 5{,}6[mm]$ $t_2 = 5{,}6[mm]$ $t_0 = 6{,}3[mm]$
The angles of inclination of diagonals:

$\theta_1 = 40{,}6[°]$ $\theta_2 = 40{,}6[°]$
Design value of the normal force: $N_{Ed} = 57{,}22[kN]$

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_{1,2}}{d_0} = \frac{60{,}3[mm]}{88{,}9[mm]} = 0{,}68[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{6{,}3[mm]} = 14{,}11[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{6{,}3[mm]} = 14{,}11[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_{1,2}}{t_{1,2}} = \frac{60{,}3[mm]}{5{,}6[mm]} = 10{,}77[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

Factors:

$k_p = 1{,}0[-]$ $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 6{,}3[mm]} = 7{,}06[-]$ (table 7.2)

$$k_g = \gamma^{0,2} \cdot \left(1 + \frac{0{,}024 \cdot \gamma^{1,2}}{1+\exp(0{,}5 \cdot \frac{g}{t_0} - 1{,}33)}\right) = (7{,}06[-])^{0,2} \cdot \left(1 + \frac{0{,}024 \cdot (7{,}06[-])^{1,2}}{1+\exp(0{,}5 \cdot \frac{11{,}1[mm]}{6{,}3[mm]} - 1{,}33)}\right) = 1{,}704[-]$$

(table 7.2)

Chord face failure:

$$N_{t,Rd} = \frac{\frac{k_g \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin\ \theta_{1,2}}\left(1{,}8+10{,}2\frac{d_{1,2}}{d_0}\right)}{\gamma_{M5}} = \frac{\frac{1{,}704\,[-]\cdot 1{,}0[-]\cdot 235\left[\frac{N}{mm^2}\right]\cdot(6{,}3[mm\ ])^2}{\sin\ 40{,}6[°]}\left(1{,}8+10{,}2\frac{60{,}3[mm\ ]}{88{,}9[mm\ ]}\right)}{1{,}0} = 212{,}928[kN]$$

(table 7.2)

Punching shear failure for K gap joints:

$$d_{1,2} = 60{,}3[mm] \leq d_o - 2t_0 = 88{,}9[mm] - 2 \cdot 6{,}3[mm] = 76{,}30[mm] \text{ (table 7.2)}$$

$$N_{1,2,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_{1,2} \cdot \frac{\frac{1+\sin\ \theta_{1,2}}{2\sin^2\theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 6{,}3[mm] \cdot \pi \cdot 60{,}3[mm] \cdot \frac{\frac{1+\sin\ 40{,}6[°]}{2\sin^2 40{,}6[°]}}{1{,}0} = 315{,}582[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1,2} = \min\left(N_{1,2Rd}, N_{t,Rd}\right) = \min(315{,}582[kN]; 212{,}928[kN]) = 212{,}928[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd1,2} = 0{,}9 \cdot 212{,}928[kN] = 191{,}635[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{57{,}22[kN]}{191{,}635[kN]} = 0{,}30[-] < 1{,}0[-]$$

Condition fulfilled.

Welded connections

Calculations weld was performed directional method.

Adopted fillet welds of thickness equal to: $a = 3[mm]$

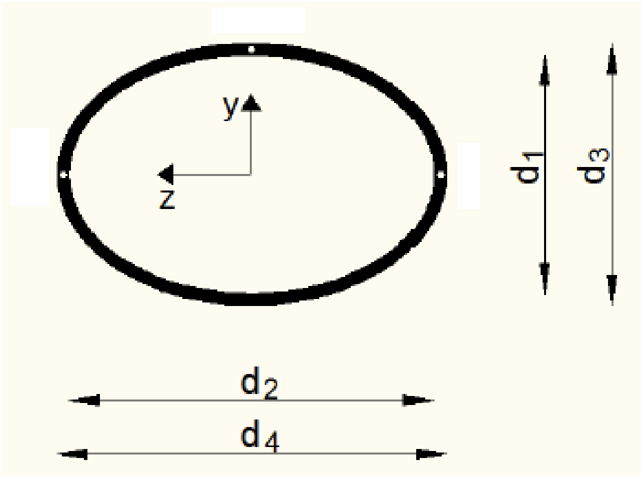
Simplified method for design resistance of fillet weld

The bars are joined by a fillet weld. Was assumed weld thickness equal to 3 mm.

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_z = N_{Ed}\cos\theta_{1,2} = 57{,}22[kN] \cdot \cos(40{,}6[^o]) = 43{,}446[kN]$$

$$N_x = N_{Ed}\sin\theta_{1,2} = 57{,}22[kN] \cdot \sin(40{,}6[^o]) = 37{,}237[kN]$$



Picture 51. Geometry of the quad weld (diagonal number 45)

Where dimensions are:

The smaller the diameter of the inner ellipse:

$$d_1 = 60{,}3[mm]$$

The larger diameter of the inner ellipse:

$$d_2 = \frac{d_1[mm]}{\sin(\theta)} = \frac{60{,}3[mm]}{\sin(40{,}6[^o])} = 92{,}66[mm]$$

The smaller diameter of the external ellipse:

$$d_3 = d_1 + 2a = 60{,}3[mm] + 2 \cdot 3[mm] = 66{,}3[mm]$$

The larger diameter of external ellipse:

$$d_4 = d_2 + 2a = 92{,}66[mm] + 2 \cdot 3[mm] = 98{,}66[mm]$$

Surface area:

$$A_w = \frac{\pi}{4}(d_3 \cdot d_4 - d_1 \cdot d_2) = \frac{\pi}{4}(66{,}3[mm] \cdot 98{,}66[mm] - 60{,}3[mm] \cdot 92{,}66[mm]) = 7{,}491[cm^2]$$

Normal stresses of tension force:

$$\tau_H = \frac{N_z}{A_w} = \frac{43{,}446[kN]}{7{,}491[cm^2]} = 5{,}800\left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_V = \frac{N_x}{A_w} = \frac{37{,}237[kN]}{7{,}491[cm^2]} = 4{,}971\left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau = \sqrt{\tau_V^2 + \tau_H^2} = \sqrt{\left(4{,}971\left[\frac{kN}{cm^2}\right]\right)^2 + \left(5{,}800\left[\frac{kN}{cm^2}\right]\right)^2} = 7{,}639\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

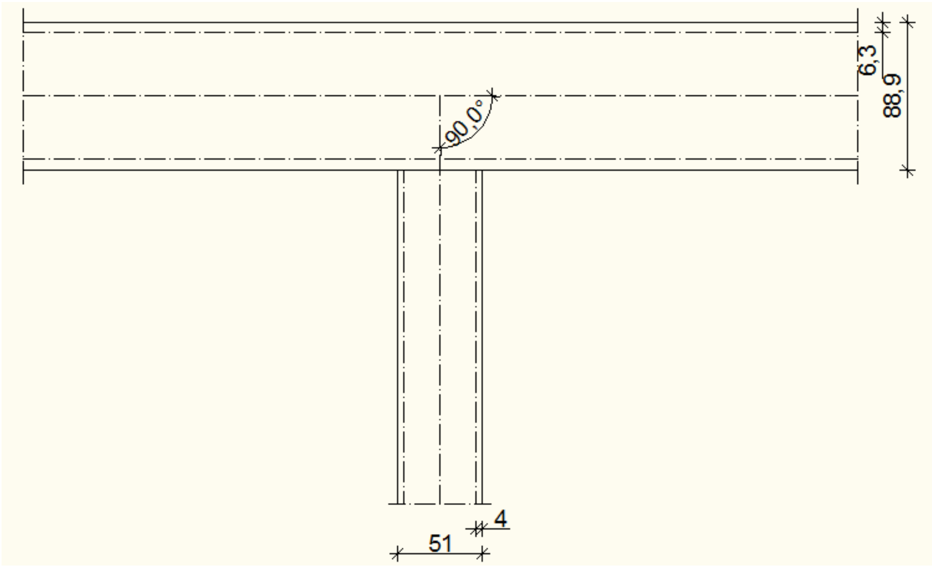
$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} = \frac{36\left[\frac{kN}{cm^2}\right]/\sqrt{3}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

Condition:

$$\tau = 7{,}639\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

Plane ZY – node T



Screenshot of the program [b]- geometry of node 23

Geometry of the connection:

$d_1 = 51{,}0[mm]$ $d_0 = 88{,}9[mm]$ $t_1 = 4{,}0[mm]$ $t_0 = 6{,}3[mm]$

The angles of inclination of diagonals: $\theta_1 = 90{,}0[°]$

Design value of the normal force: $N_{Ed} = -17{,}94[kN]$

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_1}{d_0} = \frac{51{,}0[mm]}{88{,}9[mm]} = 0{,}57[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{6{,}3[mm]} = 14{,}11[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{6{,}3[mm]} = 14{,}11[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_1}{t_1} = \frac{51{,}0[mm]}{4{,}0[mm]} = 12{,}75[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

Factors:

$k_p = 1{,}0[-]$ (table 7.2) $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 6{,}3[mm]} = 7{,}056[-]$

The ratio of the mean diameter or width of the brace members (like for T node):

$$\beta = \frac{d_1}{d_0} = \frac{51{,}0[mm]}{88{,}9[mm]} = 0{,}574[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_3}(2{,}8+14{,}2\beta^2)}{\gamma_{M5}} = \frac{\frac{(7{,}056[-])^{0,2} \cdot 1{,}0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (6{,}3[mm])^2}{\sin(90[^o])}(2{,}8+14{,}2 \cdot (0{,}574[-])^2)}{1{,}0} = 103{,}105[kN] \text{ (table 7.2)}$$

Punching shear failure for T gap joints:

$$d_1 = 51{,}0[mm] \leq d_o - 2t_0 = 88{,}9[mm] - 2 \cdot 6{,}3[mm] = 76{,}3[mm] \text{ (table 7.2)}$$

$$N_{1,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{\frac{1+\sin\theta_{1,2}}{2\sin^2\theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 6{,}3[mm] \cdot \pi \cdot 51{,}0[mm] \cdot \frac{\frac{1+\sin(90[°])}{2\sin^2(90[°])}}{1{,}0} = 136{,}952[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1} = min\left(N_{1Rd}, N_{t,Rd}\right) = min(136{,}952[kN]; 103{,}105[kN]) = 103{,}105[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd1,2} = 0{,}9 \cdot 103{,}105[kN] = 92{,}795[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{17{,}94[kN]}{92{,}795[kN]} = 0{,}19[-] < 1{,}0[-]$$

Condition fulfilled.

Welded connections

Forces:

$$\mathrm{F}_{\sigma\perp} = F_{\tau\perp} = \frac{N_{Ed,15}}{\sqrt{2}} = \frac{17{,}94[kN]}{\sqrt{2}} = 12{,}685[kN]$$

$$\mathrm{F}_{\sigma\mathrm{II}} = 0[kN]$$

Stresses:

$$\sigma_\perp = \frac{\mathrm{F}_{\sigma\perp}}{\mathrm{A_w}} = \frac{12{,}685[kN]}{\pi \cdot ((25{,}5[mm])^2 - (25{,}5[mm] - 3[mm])^2)} = 2{,}804\left[\frac{kN}{cm^2}\right]$$

$$\tau_\perp = \frac{\mathrm{F}_{\sigma\mathrm{II}}}{\mathrm{A_w}} = \frac{12{,}685[kN]}{\pi \cdot ((25{,}5[mm])^2 - (25{,}5[mm] - 3[mm])^2)} = 2{,}804\left[\frac{kN}{cm^2}\right]$$

$$\tau_{\mathrm{II}} = 0$$

Stress resultant:

$$\sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_{\mathrm{II}}^2)} = \sqrt{\left(2{,}804\left[\frac{kN}{cm^2}\right]\right)^2 + 3\left(\left(2{,}804\left[\frac{kN}{cm^2}\right]\right)^2 + 0\right)} = 5{,}608\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = \frac{\frac{36\left[\frac{kN}{cm^2}\right]}{\sqrt{3}}}{0{,}8[-] \cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

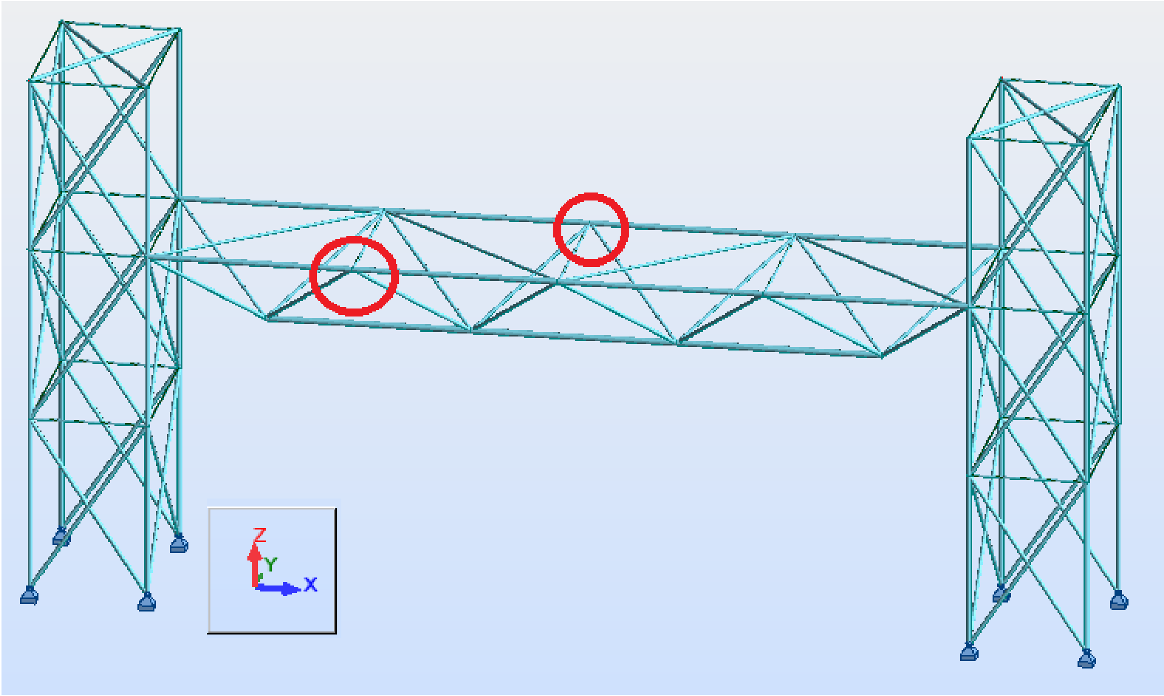
Condition:

$$\tau = 5{,}608\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

Node 18, 21

The following are a schematic the calculations made when designing the node number 18, 21 which were presented in the figure below:



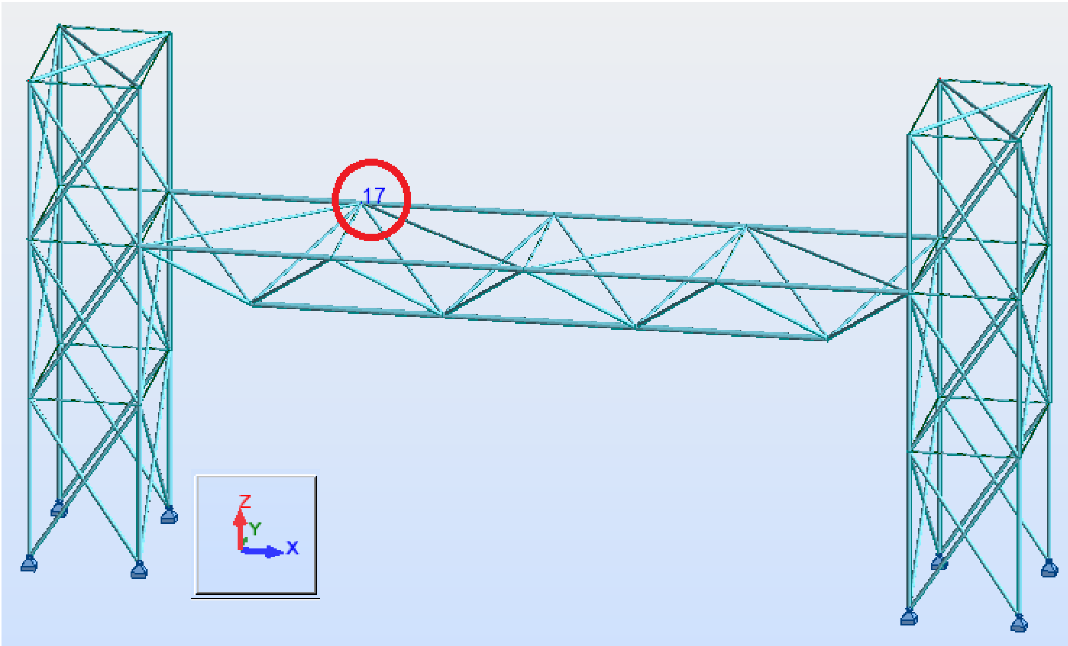
Screenshot of the program [a]- nodes 18, 21

Geometry of the connections are the same like connection in node 23 and force is smaller in node 18 and 21 is smaller than in node 23. Thus, the connection can be designed in the same way like in node 23.

This node was presented on the drawings 11- Steel Footbridge- node 18, 21, 23, which was attached to this work.

Node 17

The following is a schematic the calculations made when designing the node number 17, which was presented in the figure below:



Screenshot of the program [a]- node 17

This node connects the rods numbered 25, 26, 8, 43, 34, 46, 9.

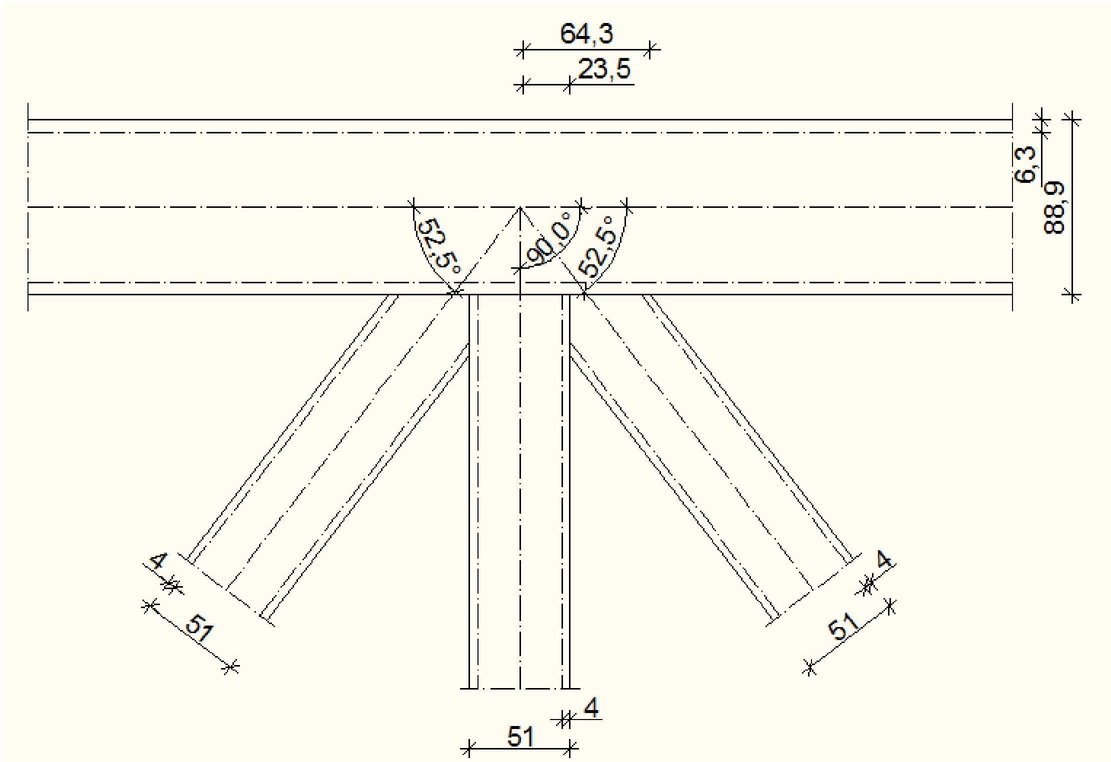
A node is defined as K type in a plane ZX and like type KT in a plane YX according to figure 7.1. When the calculations are dimensioned first node K, then T.

This node was presented on the drawings 12- Steel Footbridge- node 17, 19, 22, which was attached to this work.

Plane ZX – node K

Because the geometry of the node in this plane is the same as for the node 23, and the internal forces are smaller, in this plane must be designed in a way that the node identical to the node 23.

Plane XY – node K



Screenshot of the program [b]- geometry of node 17

Geometry of the connection:

$d_1 = 51{,}0[mm]$ $d_2 = 51{,}0[mm]$ $d_3 = 51{,}0[mm]$ $d_0 = 88{,}9[mm]$
$d_1 = 4{,}0[mm]$ $d_2 = 4{,}0[mm]$ $d_2 = 4{,}0[mm]$ $d_0 = 6{,}3[mm]$
The angles of inclination of diagonals:

$\theta_1 = 52{,}5[°]$ $\theta_2 = 52{,}5[°]$ $\theta_2 = 90{,}0\ [°]$
Node K

Design value of the normal force: $N_{Ed} = 55{,}50[kN]$

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_{1,2}}{d_0} = \frac{51{,}0[mm]}{88{,}9[mm]} = 0{,}57[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $$10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{6{,}3[mm]} = 14{,}11[-] \leq 50{,}0$$

Condition fulfilled.

Compression (class 1): $$10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{6{,}3[mm]} = 14{,}11[-] \leq 50{,}0$$

Condition fulfilled.

Braces:

Tension: $$\frac{d_{1,2}}{t_{1,2}} = \frac{51{,}0[mm]}{4{,}0[mm]} = 12{,}75[-] \leq 50{,}0$$

Overlap: $$\lambda_{ov} = \frac{q}{p} \cdot 100[\%] = \frac{23{,}5[mm]}{64{,}3[mm]} \cdot 100[\%] = 36{,}58[\%] > 25[\%]$$

q, p determined in accordance with the geometry of the connections and figure 1.3.

Condition fulfilled.

Chord face failure:

Factors:

$$k_p = 1{,}0[-] \qquad \gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 6{,}3[mm]} = 7{,}056[-] \text{ (table 7.2)}$$

$$k_g = \gamma^{0,2} \cdot \left(1 + \frac{0{,}024 \cdot \gamma^{1,2}}{1 + \exp(0{,}5 \cdot \frac{g}{t_0} - 1{,}33)}\right) = (7{,}056[-])^{0,2} \cdot \left(1 + \frac{0{,}024 \cdot (7{,}056[-])^{1,2}}{1 + \exp(0{,}5 \cdot \frac{-21{,}2[mm]}{6{,}3[mm]} - 1{,}33)}\right) = 1{,}833[-]$$

(table 7.2)

Chord face failure:

$$N_{t,Rd} = \frac{\frac{k_g \cdot k_p \cdot f_{y0} \cdot t_0^2}{\sin\theta_{1,2}}\left(1{,}8 + 10{,}2\frac{d_{1,2}}{d_0}\right)}{\gamma_{M5}} = \frac{\frac{1{,}833[-] \cdot 1{,}0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (6{,}3[mm])^2}{\sin(52{,}5)[°]}\left(1{,}8 + 10{,}2\frac{51{,}0[mm]}{88{,}9[mm]}\right)}{1{,}0} = 164{,}889\ [kN]$$

(table 7.2)

Punching shear failure for K gap joints:

$$d_{1,2} = 51{,}0[mm] \leq d_o - 2t_0 = 88{,}9[mm] - 2 \cdot 6{,}3[mm] = 76{,}3[mm] \text{ (table 7.2)}$$

$$N_{1,2,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_{1,2} \cdot \frac{\frac{1 + \sin\theta_{1,2}}{2\sin^2\theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 6{,}3[mm] \cdot \pi \cdot 51{,}0[mm] \cdot \frac{\frac{1 + \sin(52{,}5)[°]}{2\sin^2(52{,}5)[°]}}{1{,}0} =$$

$$195{,}106[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1,2} = \min(N_{1,2Rd}, N_{t,Rd}) = \min(195{,}106[kN]; 164{,}889[kN]) = 164{,}889[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd1,2} = 0{,}9 \cdot 164{,}889[kN] = 148{,}400[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{17{,}58[kN]}{148{,}400[kN]} = 0{,}12[-] < 1{,}0[-]$$

Condition fulfilled.

Welded connections

Simplified method for design resistance of fillet weld

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_z = N_{Ed} \cos\theta_{1,2} = 17{,}58[kN] \cdot \cos(52{,}5[^o]) = 10{,}702[kN]$$

$$N_x = N_{Ed} \sin\theta_{1,2} = 17{,}58[kN] \cdot \sin(52{,}5[^o]) = 13{,}947[kN]$$

Surface area (read from the program [b]):

$$A_w = 5{,}715[cm^2]$$

Normal stresses of tension force:

$$\tau_H = \frac{N_z}{A_w} = \frac{10{,}702[kN]}{5{,}715[cm^2]} = 1{,}874\left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_V = \frac{N_x}{A_w} = \frac{13{,}947[kN]}{5{,}715[cm^2]} = 2{,}440\left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau = \sqrt{\tau_V^2 + \tau_H^2} = \sqrt{\left(2{,}440\left[\frac{kN}{cm^2}\right]\right)^2 + \left(1{,}874\left[\frac{kN}{cm^2}\right]\right)^2} = 3{,}077\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} = \frac{36\left[\frac{kN}{cm^2}\right]/\sqrt{3}}{0{,}8[-] \cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

Condition:

$$\tau = 3{,}077\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

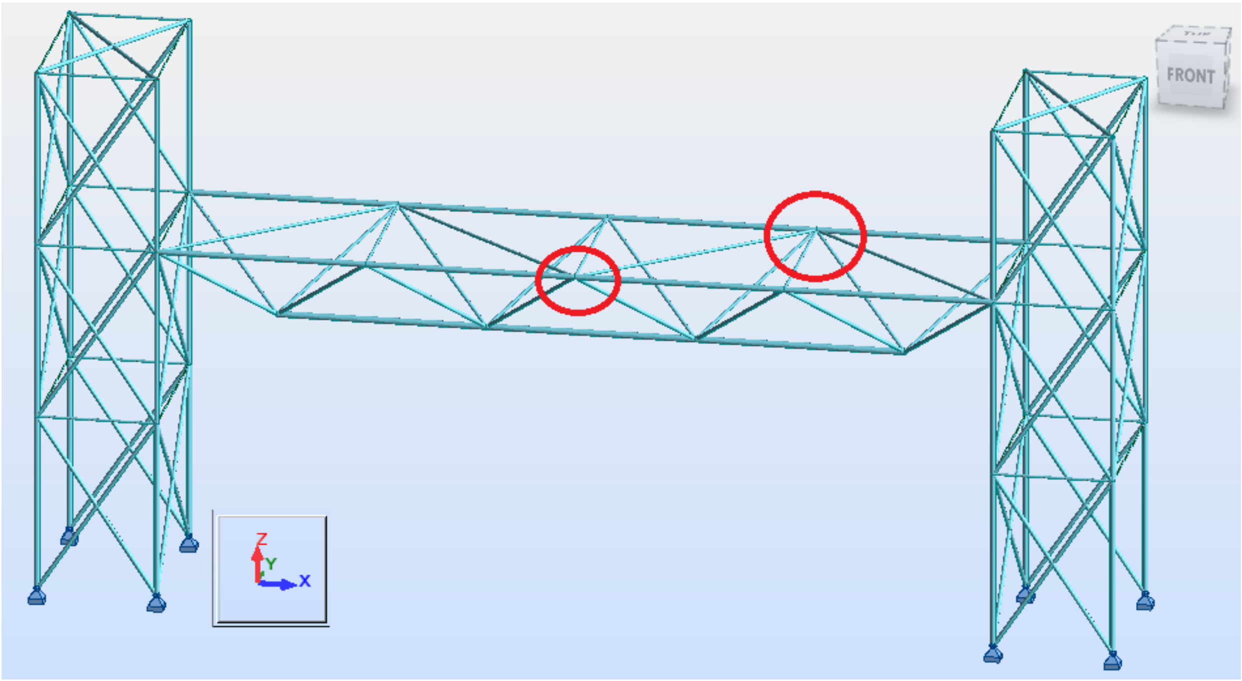
Condition fulfilled.

Plane ZY – node T

The node T has the same geometry as the node T, in the node 23, and the internal forces are smaller. Thus, node T should be designed in the same way like node 23.

Node 19, 22

The following are a schematic the calculations made when designing the node number 19, 22 which were presented in the figure below:



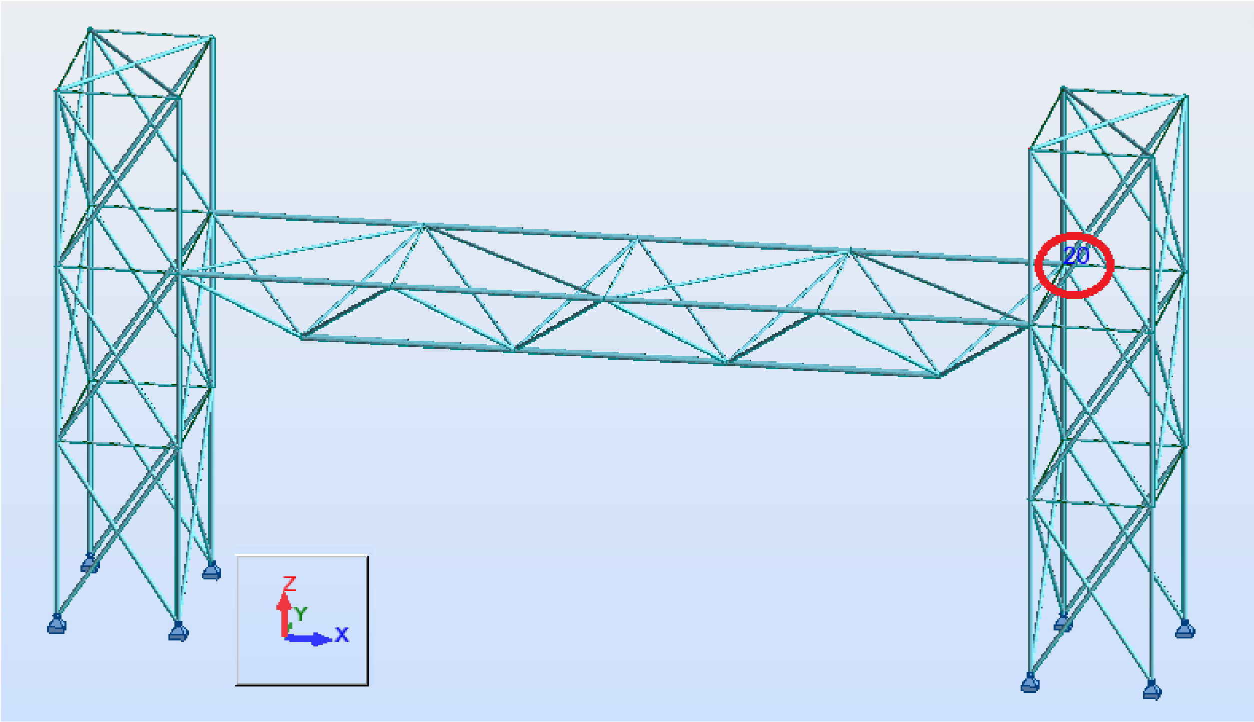
Screenshot of the program [a]- nodes 19,22

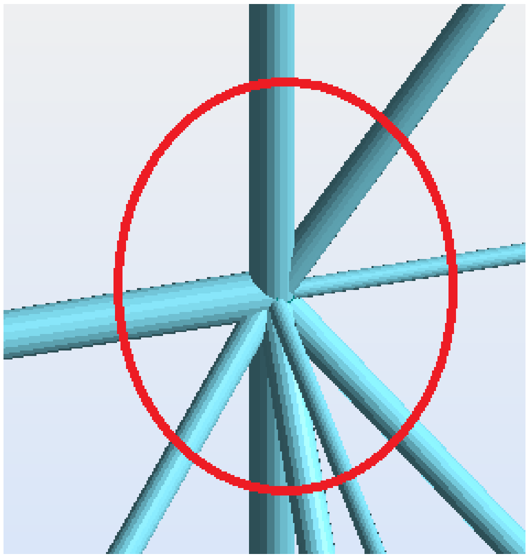
Geometry of the connections are the same like connection in node 17 and force is smaller in node 19 and 22 is smaller than in node 17. Thus, the connection can be designed in the same way like in node 17.

This node was presented on the drawings 12- Steel Footbridge- node 17, 19, 22, which was attached to this work.

Node 20

The following is a schematic the calculations made when designing the node number 20, which was presented in the figure below:





Screenshot of the program [a]- node 20

This node connects the rods numbered 130, 131, 28, 56, 37, 157, 162, 138 and 155.

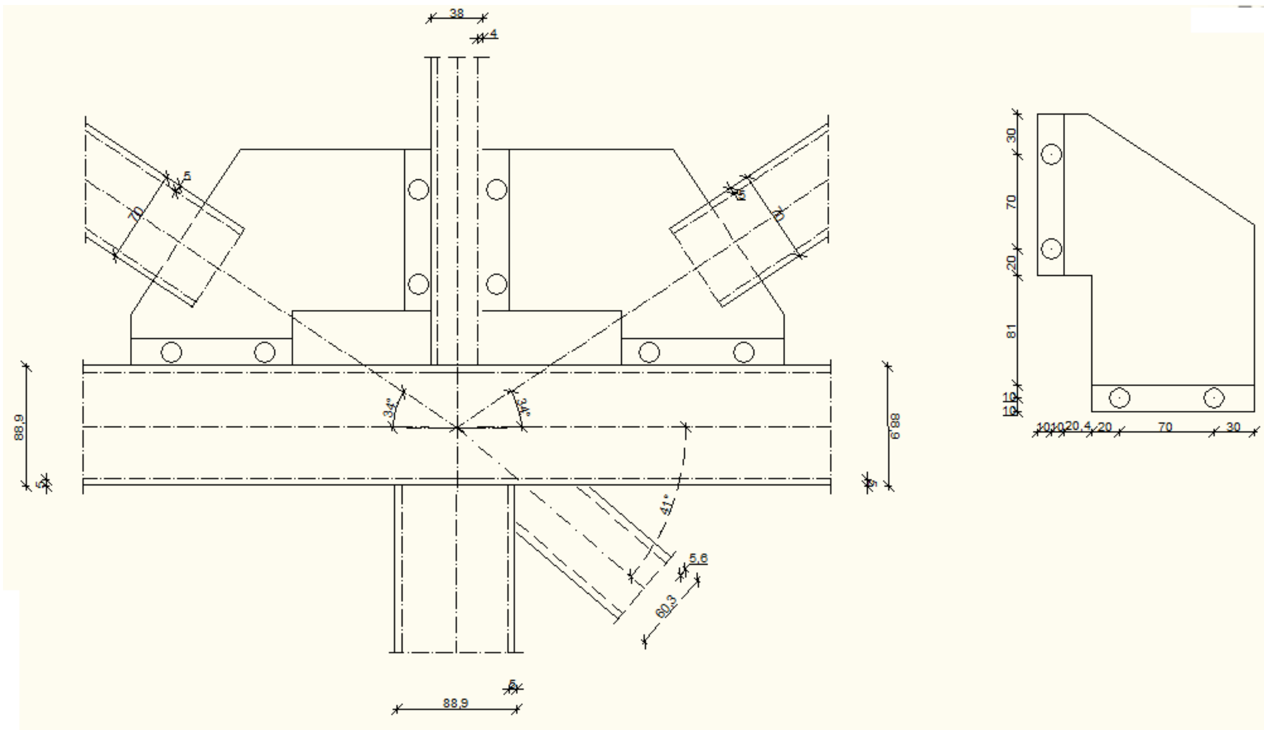
A node is defined in plane XZ and YZ have different geometry. In plane ZX is KTY and in plane ZY is type TY according to table 7.1.

This node was presented on the drawings 13- Steel Footbridge- node 11, 20, which was attached to this work.

PLANE ZX

Steel plate is welded to the bar number 130 and 131, with numbers rods 135, 155, 162 are connected by means of steel overlays. Steel caps are fixed to the steel plate by means of bolts M14 class 8.8.

The bolted connection is category A. The geometry of the screw connection meets all the requirements specified in table 3.3.



Screenshot of the program [b]- geometry of node 20 XZ with geometry of plate

Forces:

$N_{Ed,130} = 46{,}37[kN]$ $\quad N_{Ed,131} = 5{,}65[kN]$ $\quad N_{Ed,28} = 63{,}24[kN]$
$N_{Ed,56} = -56{,}39[kN]$ $\quad N_{Ed,37} = 25{,}01[kN]$ $\quad N_{Ed,157} = 19{,}08[kN]$
$N_{Ed,162} = 41{,}57[kN]$ $\quad N_{Ed,138} = -16{,}40[kN]$ $\quad N_{Ed,155} = 5{,}06[kN]$

Geometry of the connection:

$d_1 = 70{,}0[mm]$ $\quad d_2 = 70{,}0[mm]$ $\quad d_0 = 88{,}9[mm]$
$t_1 = 5{,}0[mm]$ $\quad t_2 = 5{,}0[mm]$ $\quad t_0 = 5{,}0[mm]$
$d_{\mathrm{T}} = 88{,}9[mm]$ $\quad \mathrm{t}_{\mathrm{T}} = 6{,}3[mm]$ $\quad d_{\mathrm{Y}} = 60{,}3[mm]$
$\mathrm{t}_{\mathrm{Y}} = 5{,}6[mm]$

The angles of inclination of diagonals:

$\theta_1 = 34[°]$ $\quad \theta_2 = 34[°]$ $\quad \theta_{\mathrm{T}} = 90{,}0[°]$ $\quad \theta_{\mathrm{Y}} = 41[°]$

Geometry of the plates:

Like in node 55.

Node K XZ

The geometry of the node K is the same as the geometry of the node 55, a smaller internal forces (41.457kN). Therefore, node K design in the same way as node 55

NODE T XZ

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_T}{d_0} = \frac{88{,}9[mm]}{88{,}9[mm]} = 1{,}0[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_T}{t_T} = \frac{88{,}9[mm]}{6{,}3[mm]} = 14{,}11[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

Factors:

$k_p = 1{,}0[-]$ $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 5{,}0[mm]} = 8{,}89[-]$(table 7.2)

The ratio of the mean diameter or width of the brace members (like for T node):

$$\beta = \frac{d_T}{d_0} = \frac{88{,}9[mm]}{88{,}9[mm]} = 1{,}000[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_3}(2,8+14,2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8,89[-])^{0,2} \cdot 1,0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (5,0[mm])^2}{\sin(90[^o])}(2,8+14,2 \cdot (1,0[-])^2)}{1,0} = 154,610[kN]$$

(table 7.2)

Punching shear failure for T gap joints:

$$d_T = 88,9[mm] > d_o - 2t_0 = 88,9[mm] - 2 \cdot 5,0[mm] = 78,9[mm] \text{ (table 7.2)}$$

The destruction of the belt does not occur.

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{63,24[kN]}{154,610[kN]} = 0,41[-] < 1,0[-]$$

Condition fulfilled.

Welded connections

Forces:

$$F_{\sigma\perp} = F_{\tau\perp} = \frac{N_{Ed}}{\sqrt{2}} = \frac{63,24[kN]}{\sqrt{2}} = 44,717[kN]$$

$$F_{\sigma II} = 0[kN]$$

Stresses:

$$\sigma_\perp = \frac{F_{\sigma\perp}}{A_w} = \frac{44,717[kN]}{\frac{\pi}{4} \cdot ((88,9[mm])^2 - (88,9[mm] - 2 \cdot 6[mm])^2)} = 5,523\left[\frac{kN}{cm^2}\right]$$

$$\tau_\perp = \frac{F_{\sigma II}}{A_w} = \frac{44,717[kN]}{\frac{\pi}{4} \cdot ((88,9[mm])^2 - (88,9[mm] - 2 \cdot 6[mm])^2)} = 5,523\left[\frac{kN}{cm^2}\right]$$

$$\tau_{II} = 0$$

Stress resultant:

$$\sqrt{\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_{II}^2)} = \sqrt{\left(5,523\left[\frac{kN}{cm^2}\right]\right)^2 + 3\left(\left(5,523\left[\frac{kN}{cm^2}\right]\right)^2 + 0\right)} = 11,046\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = \frac{\frac{36\left[\frac{kN}{cm^2}\right]}{\sqrt{3}}}{0,8[-] \cdot 1,25[-]} = 20,785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

Condition:

$$\tau = 11{,}046 \left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785 \left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

NODE Y planes XZ

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_Y}{d_0} = \frac{60{,}3[mm]}{88{,}9[mm]} = 0{,}68[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_Y}{t_Y} = \frac{60{,}3[mm]}{5{,}6[mm]} = 10{,}77[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

$k_p = 1{,}0[-]$ $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 5{,}0[mm]} = 8{,}89[-]$(table 7.2)

The ratio of the mean diameter or width of the brace members (like for Y node):

$$\beta = \frac{d_Y}{d_0} = \frac{60{,}3[mm]}{88{,}9[mm]} = 0{,}678[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_Y}(2,8+14,2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8,89[-])^{0,2}\cdot 1,0[-]\cdot 235\left[\frac{N}{mm^2}\right]\cdot(5,0[mm])^2}{\sin(41[^o])}(2,8+14,2\cdot(0,678[-])^2)}{1,0} = 129,304[kN]$$

(table 7.2)

Punching shear failure for T gap joints:

$$d_1 = 60,3[mm] \leq d_o - 2t_0 = 88,9[mm] - 2\cdot 5,0[mm] = 78,9[mm] \text{ (table 7.2)}$$

$$N_{Y,Rd} = \frac{f_{y0}}{\sqrt{3}}\cdot t_0\cdot\pi\cdot d_Y\cdot\frac{\frac{1+\sin\theta_Y}{2\sin^2\theta_Y}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}}\cdot 5,0[mm]\cdot\pi\cdot 60,3[mm]\cdot\frac{\frac{1+\sin(41[°])}{2\sin^2(41[°])}}{1,0} =$$

$$1852,220[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{RdY} = min(N_{YRd}, N_{t,Rd}) = min(1852,220[kN]; 129,304[kN]) = 129,304[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu\cdot N_{RdY} = 0,9\cdot 129,304[kN] = 116,374[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{56,39[kN]}{116,374[kN]} = 0,48[-] < 1,0[-]$$

Condition fulfilled.

Welded connections

Adopted fillet welds of thickness equal to: $a = 3[mm]$

Simplified method for design resistance of fillet weld

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_z = N_{Ed}\cos\theta_{1,2} = 56,39[kN]\cdot\cos(41[^o]) = 42,558[kN]$$

$$N_x = N_{Ed}\sin\theta_{1,2} = 56,39[kN]\cdot\sin(41[^o]) = 36,995[kN]$$

Surface area (from the program [b]):

$$A_w = 7,251[cm^2]$$

Normal stresses of tension force:

$$\tau_H = \frac{N_z}{A_w} = \frac{42,558[kN]}{7,251[cm^2]} = 5,869\left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_V = \frac{N_x}{A_w} = \frac{36{,}995[kN]}{7{,}251[cm^2]} = 5{,}102\left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau = \sqrt{\tau_V^2 + \tau_H^2} = \sqrt{\left(5{,}102\left[\frac{kN}{cm^2}\right]\right)^2 + \left(5{,}869\left[\frac{kN}{cm^2}\right]\right)^2} = 7{,}777\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} = \frac{36\left[\frac{kN}{cm^2}\right]/\sqrt{3}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

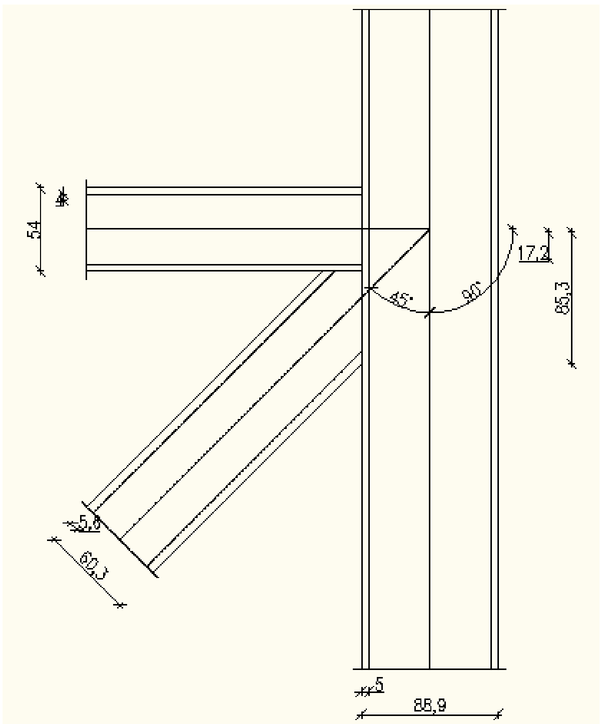
Condition:

$$\tau = 7{,}777\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

Plane YZ

The geometry of the screw connection meets all the requirements specified in table 3.3.



Screenshot of the program [b]- geometry of node 20 YZ

Geometry of the connection:

$d_1 = 60{,}3[mm]$ $d_2 = 54{,}0[mm]$ $d_0 = 88{,}9[mm]$
$t_1 = 5{,}6[mm]$ $t_2 = 4{,}0[mm]$ $t_0 = 5{,}0[mm]$
The angles of inclination of diagonals:

$\theta_1 = 45{,}0[°]$ $\theta_2 = 90{,}0[°]$

NODE Y planes YZ

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_1}{d_0} = \frac{60{,}3[mm]}{88{,}9[mm]} = 0{,}68[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_1}{t_1} = \frac{60{,}3[mm]}{5{,}6[mm]} = 10{,}77[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

$k_p = 1{,}0[-]$ $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 5{,}0[mm]} = 8{,}89[-]$(table 7.2)

The ratio of the mean diameter or width of the brace members (like for Y node):

$$\beta = \frac{d_1}{d_0} = \frac{60{,}3[mm]}{88{,}9[mm]} = 0{,}678[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_1}(2{,}8+14{,}2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8{,}89[-])^{0,2}\cdot 1{,}0[-]\cdot 235\left[\frac{N}{mm^2}\right]\cdot(5{,}0[mm])^2}{\sin(45[^o])}(2{,}8+14{,}2\cdot(0{,}678[-])^2)}{1{,}0} = 119{,}969[kN]$$

(table 7.2)

Punching shear failure for T gap joints:

$$d_1 = 60{,}3[mm] \le d_o - 2t_0 = 88{,}9[mm] - 2\cdot 5{,}0[mm] = 78{,}9[mm] \text{ (table 7.2)}$$

$$N_{1,Rd} = \frac{f_{y0}}{\sqrt{3}}\cdot t_0\cdot\pi\cdot d_1\cdot\frac{\frac{1+\sin\theta_{1,2}}{2\sin^2\theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}}\cdot 5{,}0[mm]\cdot\pi\cdot 60{,}3[mm]\cdot\frac{\frac{1+\sin(45[^\circ])}{2\sin^2(45[^\circ])}}{1{,}0} =$$

$$219{,}384[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1} = min(N_{1Rd}, N_{t,Rd}) = min(219{,}384[kN]; 119{,}969[kN]) = 119{,}969[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu\cdot N_{Rd1,2} = 0{,}9\cdot 119{,}969[kN] = 107{,}972[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{56{,}39[kN]}{107{,}972[kN]} = 0{,}52[-] < 1{,}0[-]$$

Condition fulfilled.

Welded connections

Adopted fillet welds of thickness equal to: $a = 3[mm]$

Simplified method for design resistance of fillet weld

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_z = N_{Ed}\cos\theta_{1,2} = 56{,}39[kN]\cdot\cos(45[^o]) = 39{,}874[kN]$$

$$N_x = N_{Ed}\sin\theta_{1,2} = 56{,}39[kN]\cdot\sin(45[^o]) = 39{,}874[kN]$$

Surface area (from program [b]):

$$A_w = 7{,}251[cm^2]$$

Normal stresses of tension force:

$$\tau_H = \frac{N_z}{A_w} = \frac{39{,}874[kN]}{7{,}251[cm^2]} = 5{,}499\left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_V = \frac{N_x}{A_w} = \frac{39{,}874[kN]}{7{,}251[cm^2]} = 5{,}499\left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau = \sqrt{\tau_V^2 + \tau_H^2} = \sqrt{\left(5{,}499\left[\frac{kN}{cm^2}\right]\right)^2 + \left(5{,}499\left[\frac{kN}{cm^2}\right]\right)^2} = 7{,}777\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} = \frac{36\left[\frac{kN}{cm^2}\right]/\sqrt{3}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

Condition:

$$\tau = 7{,}777\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

NODE T YZ

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_2}{d_0} = \frac{54{,}0[mm]}{88{,}9[mm]} = 0{,}61[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_2}{t_2} = \frac{54{,}0[mm]}{4{,}0[mm]} = 13{,}5[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

Factors:

$$k_p = 1{,}0[-] \qquad \gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 5{,}0[mm]} = 8{,}89[-] \text{(table 7.2)}$$

The ratio of the mean diameter or width of the brace members (like for T node):

$$\beta = \frac{d_2}{d_0} = \frac{54{,}0[mm]}{88{,}9[mm]} = 0{,}607[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin \theta_3}(2{,}8+14{,}2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8{,}89[-])^{0,2} \cdot 1{,}0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (5{,}0[mm])^2}{\sin(90[^o])}(2{,}8+14{,}2 \cdot (0{,}607[-])^2)}{1{,}0} = 73{,}048[kN]$$

(table 7.2)

Punching shear failure for T gap joints:

$$d_2 = 54{,}0[mm] \le d_o - 2t_0 = 88{,}9[mm] - 2 \cdot 5{,}0[mm] = 78{,}9[mm] \text{ (table 7.2)}$$

$$N_{2,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_2 \cdot \frac{\frac{1+\sin \theta_{1,2}}{2 \sin^2 \theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 5{,}0[mm] \cdot \pi \cdot 54{,}0[mm] \cdot \frac{\frac{1+\sin(90[°])}{2 \sin^2(90[°])}}{1{,}0} = 115{,}086[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd2} = min(N_{2Rd}, N_{t,Rd}) = min(115{,}086[kN]; 73{,}048[kN]) = 49{,}012[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd1,2} = 0{,}9 \cdot 73{,}048[kN] = 65{,}743[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{25{,}01[kN]}{65{,}743[kN]} = 0{,}38[-] < 1{,}0[-]$$

Condition fulfilled.

Welded connections

Forces:

$$\mathrm{F}_{\sigma\perp} = F_{\tau\perp} = \frac{N_{Ed}}{\sqrt{2}} = \frac{25{,}01[kN]}{\sqrt{2}} = 17{,}685[kN]$$

$$\mathrm{F}_{\sigma\mathrm{II}} = 0[kN]$$

Stresses:

$$\sigma_\perp = \frac{\mathrm{F}_{\sigma\perp}}{\mathrm{A_w}} = \frac{17{,}685[kN]}{\pi \cdot ((27[mm])^2 - (27[mm] - 3[mm])^2)} = 3{,}679\left[\frac{kN}{cm^2}\right]$$

$$\tau_{\perp} = \frac{F_{\sigma II}}{A_w} = \frac{17{,}685[kN]}{\pi \cdot ((27[mm])^2 - (27[mm] - 3[mm])^2)} = 3{,}679\left[\frac{kN}{cm^2}\right]$$

$$\tau_{II} = 0$$

Stress resultant:

$$\sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{II}^2)} = \sqrt{\left(3{,}679\left[\frac{kN}{cm^2}\right]\right)^2 + 3\left(\left(3{,}679\left[\frac{kN}{cm^2}\right]\right)^2 + 0\right)} = 7{,}358\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{\frac{f_u}{\sqrt{3}}}{\beta_w \cdot \gamma_{M2}} = \frac{\frac{36\left[\frac{kN}{cm^2}\right]}{\sqrt{3}}}{0{,}8[-]\cdot 1{,}25[-]} = 20{,}785\left[\frac{kN}{cm^2}\right] \text{(formula 4.4)}$$

Condition:

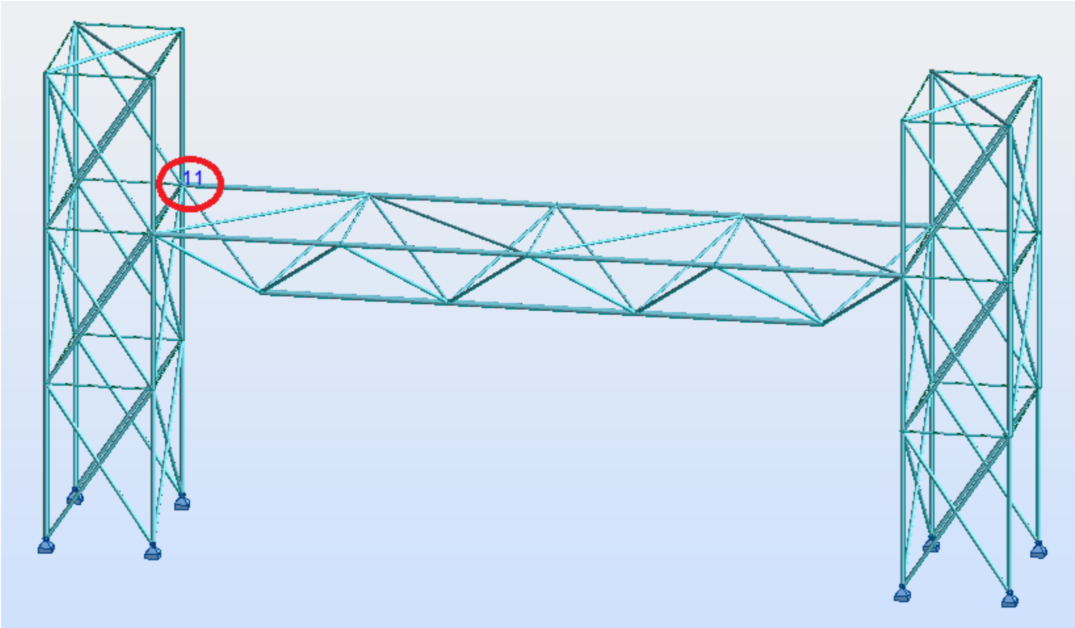
$$\tau = 7{,}358\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20{,}785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

The connection has been sized correctly. Weld thickness of 3 mm is sufficient.

Node 11

The following are a schematic the calculations made when designing the node number 11 which were presented in the figure below:



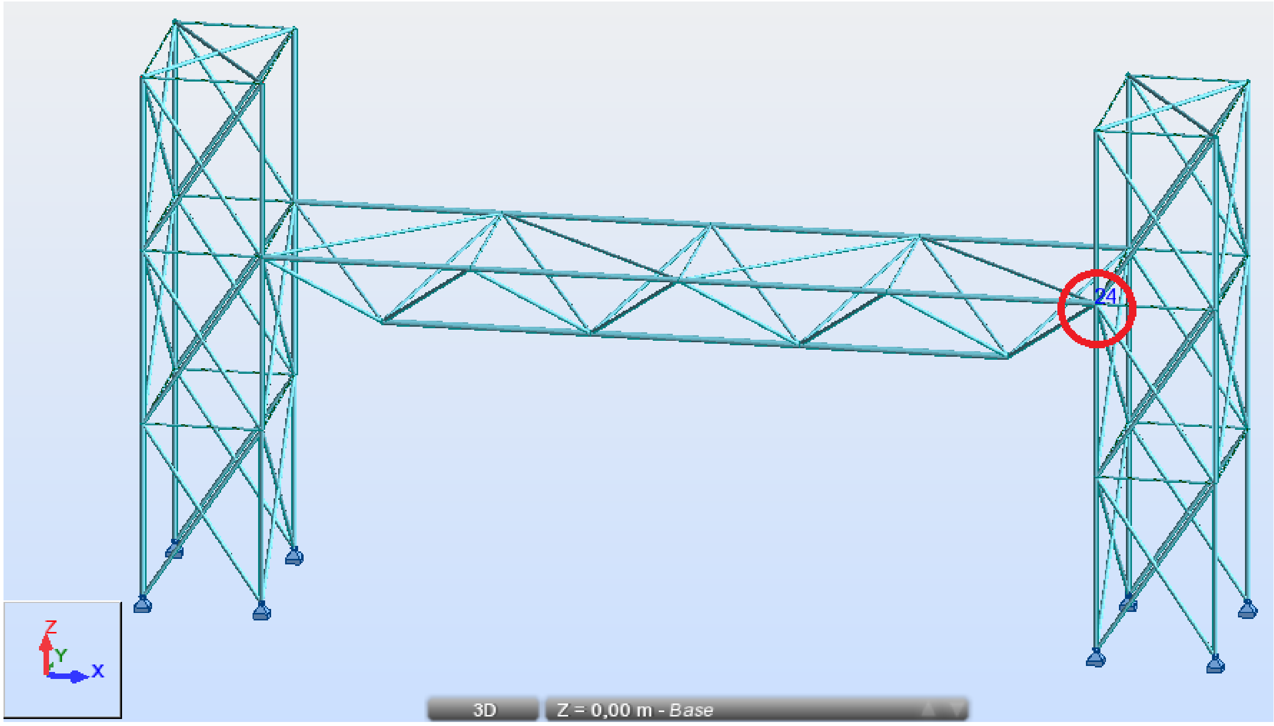
Screenshot of the program [a]- nodes 11

Geometry of the connections are the same like connection in node 20 and force is smaller in node 11 is smaller than in node 20. Thus, the connection can be designed in the same way like in node 20.

This node was presented on the drawings 13- Steel Footbridge- node 11, 20, which was attached to this work.

Node 24

The following is a schematic the calculations made when designing the node number 24, which was presented in the figure below:



Screenshot of the program [a]- node 24

This node connects the rods numbered 128, 127, 62, 32, 55, 37, 152, 146, 158 and 156.

This node is very similar to the node number 20. Differs only is the extra diagonal bar in the YX plane. The remaining geometry node is the same as the node number 20. The normal forces acting on bars are smaller. Therefore, the node can be performed in the same way as in node 20 in plane XZ and YZ.

This node was presented on the drawings 14- Steel Footbridge- node 15, 24, which was attached to this work.

NODE Y planes YX

Checking the conditions specified in Table 7.1:

Both conditions should be compressive and tensile (for some combinations of compression occurs, and the other tension).

Diameter ratio: $0{,}2 \leq \frac{d_1}{d_0} = \frac{51{,}0[mm]}{88{,}9[mm]} = 0{,}57[-] \leq 1{,}0$

Condition fulfilled.

Chords:

Tension: $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Compression (class 1): $10 \leq \frac{d_0}{t_0} = \frac{88{,}9[mm]}{5{,}0[mm]} = 17{,}78[-] \leq 50{,}0$

Condition fulfilled.

Braces:

Tension: $\frac{d_1}{t_1} = \frac{51{,}0[mm]}{4{,}0[mm]} = 12{,}75[-] \leq 50{,}0$

Condition fulfilled.

Chord face failure:

$k_p = 1{,}0[-]$ $\gamma = \frac{d_0}{2 \cdot t_0} = \frac{88{,}9[mm]}{2 \cdot 5{,}0[mm]} = 8{,}89[-]$(table 7.2)

The ratio of the mean diameter or width of the brace members (like for Y node):

$$\beta = \frac{d_1}{d_0} = \frac{51{,}0[mm]}{88{,}9[mm]} = 0{,}574[-] \text{ (according 1.5. (6))}$$

Chord face failure:

$$N_{t,Rd} = \frac{\frac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_1}(2{,}8+14{,}2\beta^2)}{\gamma_{M5}} = \frac{\frac{(8{,}89[-])^{0,2} \cdot 1{,}0[-] \cdot 235\left[\frac{N}{mm^2}\right] \cdot (5{,}0[mm])^2}{\sin(45[^o])}(2{,}8+14{,}2 \cdot (0{,}574[-])^2)}{1{,}0} = 96{,}188[kN]$$

(table 7.2)

Punching shear failure for T gap joints:

$$d_1 = 51{,}0[mm] \leq d_o - 2t_0 = 88{,}9[mm] - 2 \cdot 5{,}0[mm] = 78{,}9[mm] \text{ (table 7.2)}$$

$$N_{1,Rd} = \frac{f_{y0}}{\sqrt{3}} \cdot t_0 \cdot \pi \cdot d_1 \cdot \frac{\frac{1+\sin\theta_{1,2}}{2\sin^2\theta_{1,2}}}{\gamma_{M5}} = \frac{235\left[\frac{N}{mm^2}\right]}{\sqrt{3}} \cdot 5{,}0[mm] \cdot \pi \cdot 51{,}0[mm] \cdot \frac{\frac{1+\sin(45[°])}{2\sin^2(45[°])}}{1{,}0} =$$

$$185{,}549[kN] \text{ (table 7.2)}$$

According 7.4.1.(2) the design resistance of a connection:

$$N_{Rd1} = min(N_{1Rd}, N_{t,Rd}) = min(185,549[kN]; 96,188[kN]) = 96,188[kN]$$

Node is a spatial factor should therefore be taken into account:

$$N_{Rd} = \mu \cdot N_{Rd1,2} = 0,9 \cdot 96,188[kN] = 86,569[kN] \text{ (table 7.7)}$$

The condition carrying capacity:

$$\frac{N_{Ed}}{N_{Rd}} = \frac{8,25[kN]}{86,569[kN]} = 0,10[-] < 1,0[-]$$

Condition fulfilled.

Welded connections

The force acting on the bar is divided into force orthogonal (N_z) and parallel (N_x) to the surface of the weld:

$$N_z = N_{Ed} \cos\theta_{1,2} = 9,25[kN] \cdot \cos(45[^o]) = 6,541[kN]$$

$$N_x = N_{Ed} \sin\theta_{1,2} = 9,25[kN] \cdot \sin(45[^o]) = 6,541[kN]$$

Surface area (from program [b]):

$$A_w = 6,341[cm^2]$$

Normal stresses of tension force:

$$\tau_H = \frac{N_z}{A_w} = \frac{6,541[kN]}{6,341[cm^2]} = 1,031\left[\frac{kN}{cm^2}\right]$$

Shear stress on the strength:

$$\tau_V = \frac{N_x}{A_w} = \frac{6,541[kN]}{6,341[cm^2]} = 1,031\left[\frac{kN}{cm^2}\right]$$

Stress resultant:

$$\tau = \sqrt{\tau_V^2 + \tau_H^2} = \sqrt{\left(1,031\left[\frac{kN}{cm^2}\right]\right)^2 + \left(1,031\left[\frac{kN}{cm^2}\right]\right)^2} = 1,458\left[\frac{kN}{cm^2}\right]$$

The design shear strength of the weld:

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} = \frac{36\left[\frac{kN}{cm^2}\right]/\sqrt{3}}{0,8[-] \cdot 1,25[-]} = 20,785\left[\frac{kN}{cm^2}\right] \text{ (formula 4.4)}$$

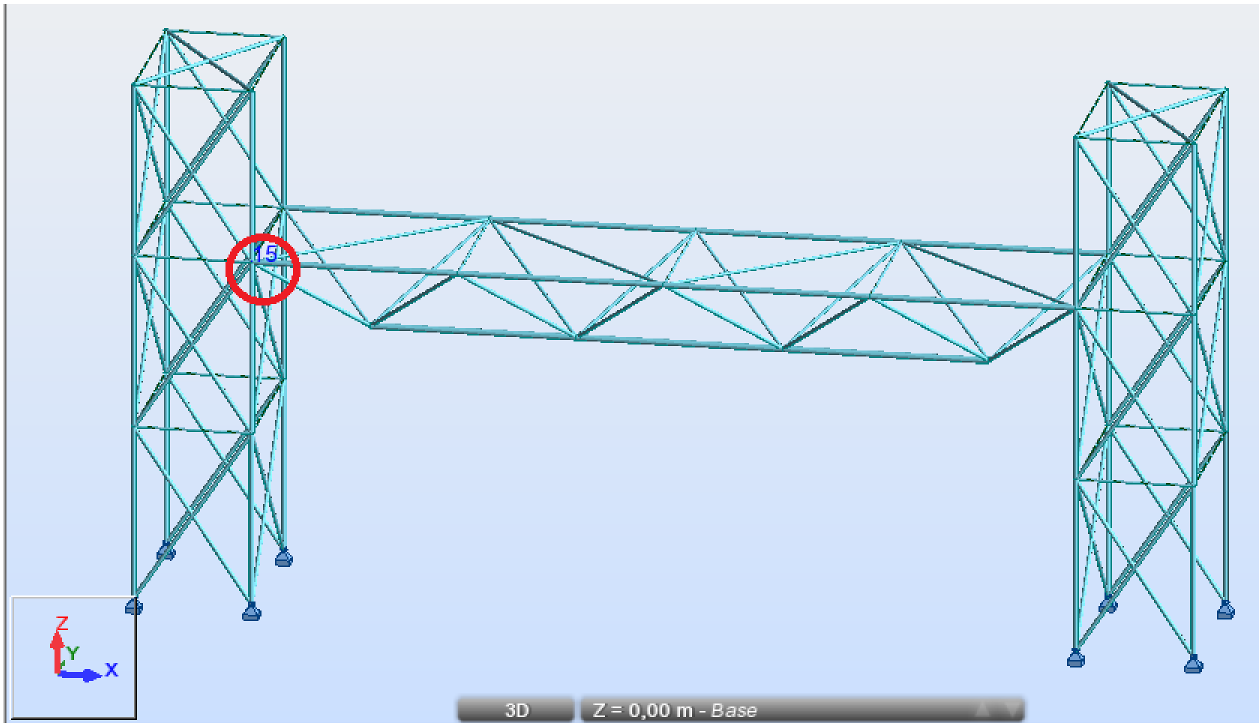
Condition:

$$\tau = 1,458\left[\frac{kN}{cm^2}\right] \quad \leq \quad f_{vw,d} = 20,785\left[\frac{kN}{cm^2}\right]$$

Condition fulfilled.

Node 15

The following are a schematic the calculations made when designing the node number 15 which were presented in the figure below:



Screenshot of the program [a]- nodes 15

Geometry of the connections are the same like connection in node 24 and force is smaller in node 15 is smaller than in node 24. Thus, the connection can be designed in the same way like in node 24.

This node was presented on the drawings 14- Steel Footbridge- node 15, 24, which was attached to this work.