Budapest University of Technology and Economics
Faculty of Civil Engineering

Design of an open-air stage roof structure

Diploma Work

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MŰEGYETEM 1782

DIPLOMA WORK

Name:	Tran Hong Tho	Neptun code:	HO833D
Training:	M.Sc. Degree Program in Civil Engineering	Semester:	2023/24/1
Specialization:	Specialization in Structures	Registry nr:	MSc-SP-T-026-23/24/1
Subject:	Design of an open-air stage roof structure		

Project description:

The topic of the thesis is the design of a double layer truss roof structure of an open-air stage in Budapest. The structure should have negative gaussian curvature. The topology should follow an optimal geometry. Total span of the structure is about ~20 m.

Tasks to be carried out:

1. Prepare a literature review on curved roof structures, analysing the possible structural variants focusing on double layer truss system, include existing examples and analytical solutions.
2. Propose the type of network systems to be studied in detail based on topological considerations and analytical solutions.
3. Optimise the topology using parametric design principles.
4. Build up a validated numerical model which can be used for detailed design.
5. Using the numerical model complete the global structural design of the truss roof structure having the favourable network system, with the level of detail assigned by the consultants.
6. Propose the construction technology of the designed structure.
7. Prepare a technical description of the designed structure.

A diploma diary has to be made during the preparation which has to be joined to the thesis at the end of the semester. The level and specification of the task and details on the design are to be registered in the diary by the supervisors.

Supervisor:		
Dr. Péter Hegyi, assistant professor	BME Dept. of Structural Engineering	100 %

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Date:	4.9.2023
Deadline for the project:	8.12.2023

Dr. Dunai László Chair of the Specialization	Dr. Rózsa Szabolcs Dean

BUDAPESTI MŰSZAKI ÉS GAZDASÁGTUDOMÁNYI EGYETEM ÉPÍTŐMÉRNÖKI KAR DÉKÁN 2.

Student Declaration about the Originality of the Thesis and Self-activity

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I also declare that I did not mislead my supervisor nor the professor issuing the diploma assignment concerning the originality of my work.

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ABSTRACT

This thesis serves as a design project with the primary objective of building a roof structure for an open-air stage. The goal is to construct a stage that can accommodate at least 1,500 concertgoers concurrently, taking into consideration safety, comfort, and visibility, while also meeting the structural design specifications. One of the primary goals is to design a suitable structure that eliminates the need for internal columns, thereby attaining an unobstructed view through the structure's appropriate shape.

The pre-design process for the hyperbolic shape structure involves the modification of parameters in Rhino/Grasshopper, taking into consideration stage requirements, aesthetic perspective of the point, and production cost. A substantial theoretical burden will be exerted on the structure whose shape possesses a negative Gaussian curvature; this will enable the shape optimization process to continue.

After constructing the 3D model in the finite element analysis (FEA) software RFEM, linear analysis is conducted to evaluate the structural performance under self-weight, wind, snow, temperature, and seismic loading. The ULS and SLS checks were conducted, and the optimal model was identified through iterative modifications of the cross sections of the members, with member utilization below 100%. Subsequently, a number of connections are meticulously designed at designated locations to guarantee the structure's overall stability, strength, and safety. Numerous factors must be taken into account during the connection design process, such as load transfer, material compatibility, structural integrity, and construction viability.

Afterwards, in the final phase, a construction solution is suggested and technical description are prepared to ensure that this structure satisfies various criteria, including design feasibility, cost implications, structural integrity, time effectiveness, adaptability, innovation, and risk mitigation.

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1. INTRODUCTION

1.1. Concept of Structural Designer

Structural designers represent a modern generation of engineers who serve as vital intermediaries between traditional structural engineers and architects. Their role entails developing the most optimal design solutions based on specific project requirements, which are contingent upon various factors. Therefore, it is accurate to assert that they shape the very core of a project. This places a significant burden of responsibility on the shoulders of the designer.

A proficient structural designer endeavors to encompass a multitude of factors to create a comprehensive design. This comprehensive approach considers aesthetics, structural integrity, and construction feasibility. The integration of all these facets profoundly influences the design, a fact that becomes evident in this specific project. Naturally, this approach presents numerous challenges that can complicate the design process. Consequently, it is imperative for a structural designer to possess intuition, foresight, and a profound knowledge of structural principles to adeptly navigate these challenges.

The primary duty of the structural designer is to guarantee that the structure fulfills all its intended functions before embarking on the design process. The design process serves as the ultimate determinant of the "structural life" of any given structure. This impels a designer to scrutinize the behavior of the structure during its operational phase. Past incidents, such as the collapse of certain space truss structures due to escalating nonlinear parameters, underscore the critical importance of this evaluation. A notable example is the collapse of a double layer truss in Hartford USA, 1978.

Today, beside the fundamental knowledge that provides a strong foundation for designing free-form structures, advanced design techniques like finite element software, computational design, etc. are available for structural engineers to make designs more accurate. Coupled with a number of modeling software options, they constitute indispensable tools for the contemporary structural designer.

1.2. The beauty of hyperboloid and hyper space truss

Space truss structures captivate attention due to their remarkable strength-to-weight characteristics. These structures can cover a vast area with notably minimal material consumption. A primary distinction between trusses and frames is that space frames necessitate connections resistant to bending to ensure structural stability. In contrast, space trusses, built from tubular components and connected at pin-jointed nodes, achieve stability through the configuration of their members and boundary girders/support . The beauty of space truss structure lies in their versatility, allowing for a range of configurations, from simple flat or double-curved forms. The design emphasis thus shifts towards generating as many consistent facets as possible within the structural surfaces.

In geometry, a hyperboloid of revolution, also known as a circular hyperboloid, is the three-dimensional surface produced by the rotation of a hyperbola around one of its principal axes. It can be described and represented using various parameterization methods and equations shown below:

$$\frac{x^2}{a^2}+\frac{y^2}{b^2}-\frac{z^2}{c^2}=1 \quad \text{or} \quad \frac{x^2}{a^2}+\frac{y^2}{b^2}-\frac{z^2}{c^2}=-1$$

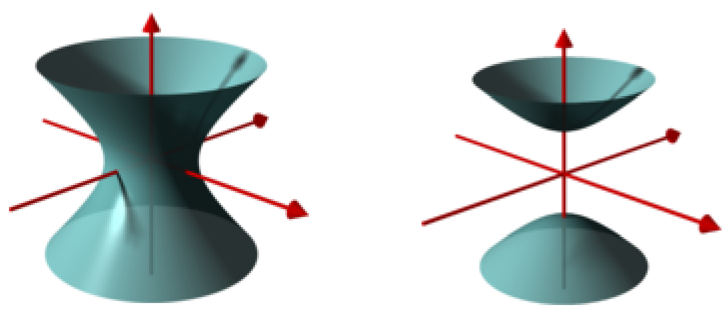


Figure 1. The figures of hyperboloid of one sheet and hyperboloid of two sheet respectively (" Hyperboloid").

There are two types of hyperboloids. In the first scenario (+1 on the right-hand side of the equation), it is a one-sheet hyperboloid, also referred to as a hyperbolic hyperboloid. This surface is connected and exhibits negative Gaussian curvature at each point. Consequently, in the vicinity of each point, the intersection of the hyperboloid and its tangent plane comprises two branches of curves with distinct tangents at that point. In the case of the one-sheet hyperboloid, these branches of curves are lines, making it a doubly ruled surface. In the second scenario (−1 on the right-hand side of the equation), it is a two-sheet hyperboloid, also known as an elliptic hyperboloid. This surface has two connected components and demonstrates positive Gaussian curvature at every point. It is convex, meaning that the tangent plane at each point intersects the surface only at that point.

Apart from hyperboloid, The hyperbolic paraboloid is a three-dimensional surface that extends infinitely and exhibits cross-sectional shapes resembling hyperbolas and parabolas. It can be described and represented using various parameterization methods and equations shown below:

$$\mathbf{z = x^2 - y^2} \quad \text{or} \quad \mathbf{y = xy.}$$

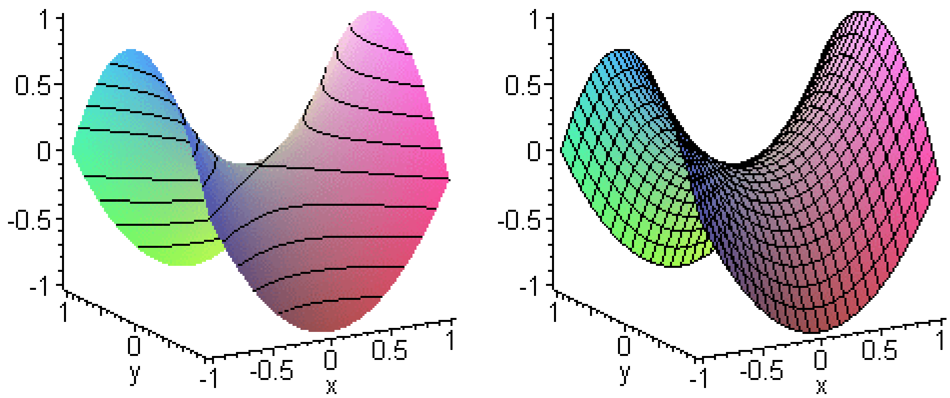


Figure 2. The graphs of hyperbolic paraboloid ("Hyperbolic Paraboloids").

We employ the term "**hypar**" to refer to a shape resembling a hyperbolic paraboloid, or to be more precise, a partial hyperbolic paraboloid that has been extracted from the complete infinite surface. The geometry, including their initial curvature, coupled with the boundary conditions and the type of applied loads, determines how they transmit loads or, in the event of loads surpassing their capacity, how they experience failure.

1.3. Objectives

This thesis essentially constitutes a design project aimed at creating an open-air stage roof structure. The objective is to develop a stage a minimum of 1500 concert attendees simultaneously. The proposed structure is intended to be constructed using a steel space truss spanning a 30-meter-wide stage. This presents a complex overall design process, necessitating the establishment of a set of objectives to ensure the efficiency and structural integrity of the design. These objectives are outlined as follows:

- **Aesthetic Efficiency**: Given that this stage is intended for concerts and outdoor events, having an aesthetically pleasing roof structure is advantageous. It has the potential to become an iconic and pride-inducing structure for the city or town. Therefore, designing a structure that is not only functional but also aesthetically appealing is of paramount importance. An elegantly designed structure holds a unique advantage in capturing the attention of people.
- **Structural Efficiency**: There exist numerous structural solutions for various design criteria. However, it is incumbent upon the designer to create a structurally sound design that efficiently transfers loads to the ground in the most effective manner possible.
- **Constructional Efficiency**: The design should prioritize efficiency in terms of construction. This entails optimising the layout, dimensions, and geometry of the space truss to minimize material waste and reduce costs without compromising structural integrity. Additionally, the choice of construction methods can impact costs significantly. Innovative construction techniques, such as prefabrication or modular assembly, may decrease labor and time expenses, resulting in cost savings. Efficient construction scheduling and sequencing can also influence the economics of the project.

It is important to acknowledge that meeting the defined criteria above is somewhat subjective. This subjectivity arises because aspects like the beauty of a structure can be interpreted differently by various individuals. What may be aesthetically pleasing to one person may not hold the same appeal for another. Moreover, factors like constructional efficiency can be contingent on financial considerations. Nonetheless, the ultimate aim of this design is to strike a balance between all three objectives.

1.4. Design approach

A specific design approach is established to meet the aforementioned objectives. Design, being an iterative process, can be quite demanding if not adequately planned in advance. The overall sequence of the design strategy employed in this project is outlined as follows:

- **Comprehensive Study of Hypar Space Truss Design:** Acquiring and studying all relevant theory pertaining to the design of hypar space truss structures is the initial step. This includes an examination of various hyper surfaces, their structural behavior, and potential failure modes.
- **Historical Review of the Steel Space Truss Industry:** A thorough exploration of the history of the steel space truss industry is conducted, along with an examination of key figures and notable structures. Over the last decade, several space truss designers have gained prominence, and an effort is made to understand their methodologies. These designers have an impressive portfolio of functioning space truss structures, some of which share characteristics with the anticipated space truss design for this project.
- **Analysis of Existing Concert Stages:** Currently operational concert stages are scrutinized to gain insights into the expected loads on the roof. Any special loads that require consideration are identified in this phase.
- **Establishment of Pre-Design Considerations:** Based on the information gathered in the first three steps, pre-design considerations are defined. These considerations essentially set the design boundaries or constraints within which the designer can operate freely. This step marks the commencement of the actual design process.
- **Creation of Design Framework:** Once all relevant factors have been taken into account, the framework for the main design process is formulated. This procedure is rigorously adhered to throughout the design endeavor.
- **Evaluation of Structural Behavior:** Finally, the behavior of the designed hyper space truss structure is analyzed and documented under various load combinations to ensure its proper functioning.

The initial three steps encompass a comprehensive literature review conducted prior to starting the actual design of the open-air stage roof. They serve as the foundation for establishing a robust design process, which is subsequently applied to arrive at the final proposal. This design process is elucidated in detail within the scope of this thesis.

2. EXISTING KNOWLEDGE RELEVANT TO HYPAR SPACE TRUSS STRUCTURES.

2.1. Classification and Application of Space Structures

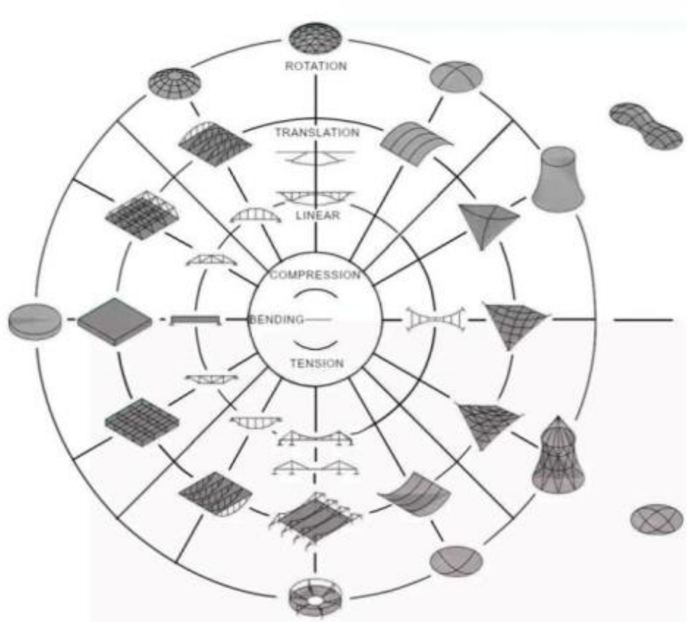


Figure 3. Classification of Space Structures (Design, Analysis and Construction of space Structure, the Mero Legacy).

Structures are typically categorized based on their predominant load-bearing characteristics, which involve either compression, tension, bending, or combinations of these forces. Figure 2 arranges various structural systems into four sectors, each corresponding to a specific load-bearing behavior: compression is represented above, tension below, and combined compression, tension, and bending are placed on the left and right sectors for classification.

2.2. Example of Structures on hypar surfaces

2.2.1. The first hypar structure

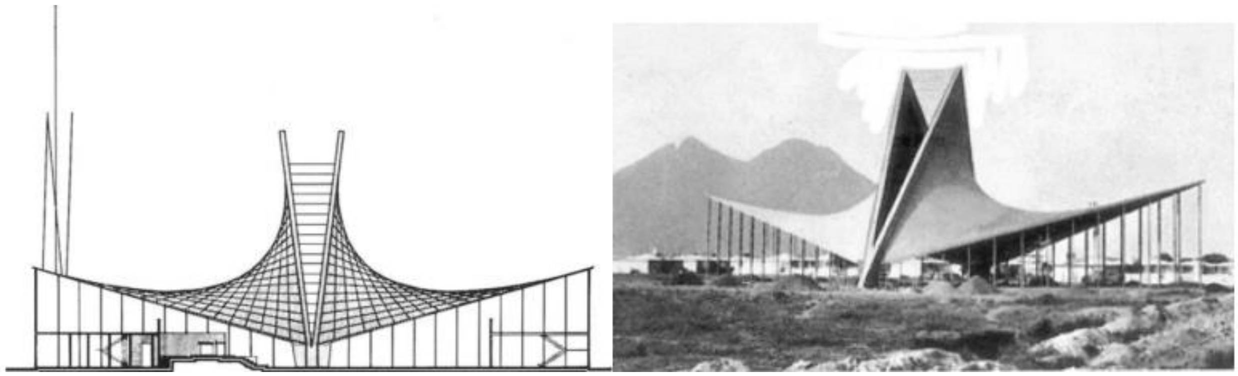


Figure 4. Church San Obrero in Nuevo Leon/Mexico, designed by Felix Candela (Design, Analysis and Construction of space Structure, the Mero Legacy) .

The initial hyperbolic paraboloidal (hypar) structures date back to the concrete shells designed by Felix Candela (1910-1997). Candela capitalized on the scaffolding aspect of hypar structures by utilizing the property that a hypar surface can be formed using straight lines. Figure 3 illustrates a design sketch and the concrete shell structure of a hypar from 1959.

Later, Felix Candela had a chance to design a stadium roof for the Islamic University in Riyadh, which required an impressive span of approximately 150 meters. Candela provided valuable assistance to the architects at TYPSA in Madrid, and unsurprisingly, he suggested a hypar structure for the roof, as depicted in Figure 4.

Drawing from his extensive experience with remarkably thin shell structures, Candela envisioned a single-layer reticulated shell design. However, this presented significant challenges, as it proved difficult to achieve the stability of such a large-span hypar shell with a single-layer truss structure that was both practical and stable. Eventually, Candela agreed to a less dense double-layer space truss solution, which met the stability requirements while maintaining a lower weight compared to a concrete shell with a thickness of only 4 cm.

Figure 5. Stadium roof for the Islamic University in Riyadh/SA, Arch.TYPSA, Eng. Mero (Design, Analysis and Construction of space Structure, the Mero Legacy).

2.2.2. Markham Moor Scorer Building

Figure 6. Markham Moor Petrol Station (Markham Moor Scorer Building).

The hyperbolic paraboloid featured structures, which was created by Lincoln based architect Sam Scorer, originally functioned as a petrol station. Later, The building was added beneath the petro station. These hypers (hyperbolic paraboloid structures) were experimented aiming to give the impression of hovering but also demonstrate engineering efficiency, as mere 75mm thick concrete roof was applied. The cantilever canopy using a shell concrete structure forms a continuous plane where two inverted parabolas positioned at right angles to each other. As the result, the canopy functions as two systems of arches with one set of arches under tension and the other under compression.

2.2.3. Royal Carpet Factory

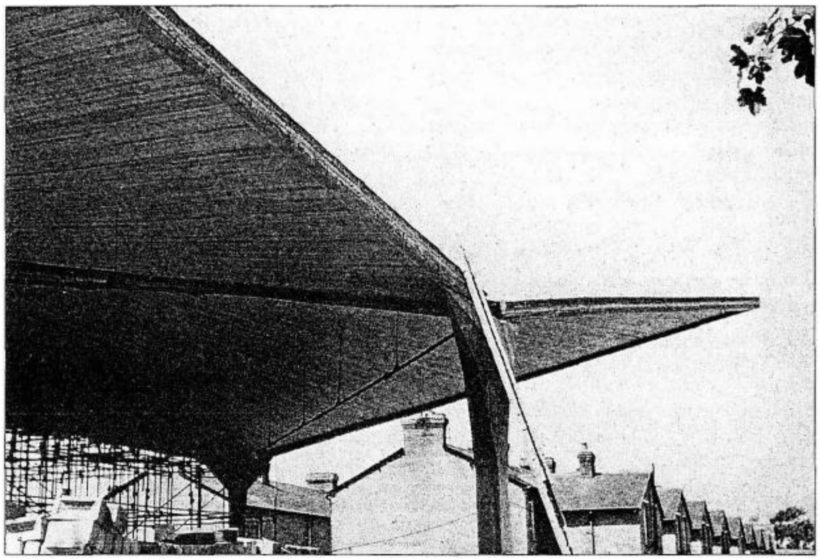


Figure 2. Royal Carpet Factory, Wilton (1957) during erection (The design and construction of timber hyperbolic paraboloid shell roofs in Britain).

A shell roof enveloped a structure using a slender covering material that drew its strength and rigidity from its unique form. Consequently, a mere 3-inch (7.62 cm) thick timber membrane, shaped in a doubly curved hp, and reinforced along its edges, could span over a 60-foot square area without requiring any internal supports. Typically, this membrane consisted of layers of boards, and to reinforce the edges, laminated timber elements bonded with glue were commonly employed. The outward forces at the supports were typically countered by steel ties or, on occasion, reinforced concrete buttresses. Building roofs were created using either a single cohesive unit or by amalgamating multiple units.

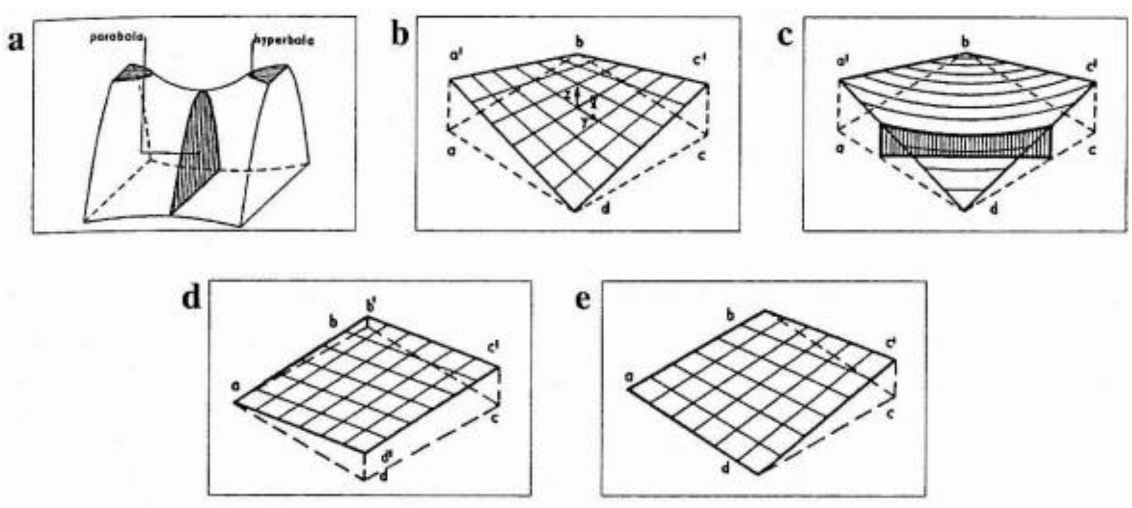


Figure 3. Different shapes of Hyperbolic paraboloid (Design, Analysis and Construction of space Structure, the Mero Legacy)

The hyperbolic paraboloid is a three-dimensional surface that falls within the category of surfaces known as conicoids. More familiar examples of conicoids include the sphere (like a tennis ball), ellipsoid (such as a rugby ball), hyperboloid of revolution (like a cooling tower), and the cone. A hyperbolic paraboloid has a distinctive saddle shape, and when it's cut, it reveals hyperbolas and parabolas.

Despite its intricate appearance, constructing a hyperbolic paraboloid is relatively straightforward. In Figure 3b, there are four straight lines that define the square abcd. To start, lift points a and c to new positions a' and c'. Initially, place a' and c' at the same elevation above the plane abcd. Then, divide a'd and c'b into an equal number of segments (let's say six) and connect corresponding points. Similarly, do this with a'b and c'd. The surface formed by these straight lines constitutes a portion of a hyperbolic paraboloid.

Despite the presence of straight lines on its surface, this structure is a doubly curved surface known as a "ruled surface." As depicted in Figure 3b, all vertical cross-sections of the surface that run parallel to its edges are straight lines, while those parallel to the diagonals take on the shape of parabolas, as illustrated in Figure 3c.

Mathematically, the surface can be described by the equation $z = kxy$, with the axes indicated in Figure 3b. Here, "k" represents a constant that quantifies the slope of the square's sides. For instance, when "k" is small, the surface is relatively shallow, but when "k" is large, the edges have a steeper incline. The value of "k" holds significance as it determines the height rise of the shell. In the case of a symmetrical shell, the "rise" is defined as half the difference in elevation between the highest and lowest corners.

To simplify matters in the previous explanation, we positioned a' and c' in Figure 3b at the same elevation above the plane abcd. However, in the general scenario, all points are situated at varying heights, as shown in Figure 3d, while the surface still maintains its hyperbolic paraboloid shape. An intriguing special circumstance arises when only one point (let's say c) is raised or lowered, as depicted in Figure 3e.

With the geometry now established, the next step is to calculate the stresses within the structure when it's subjected to loads. This calculation is essential for determining the appropriate size of the structural members. While a comprehensive analysis can be highly complex, there exists a simplified model for understanding how the load is transmitted to the ground.

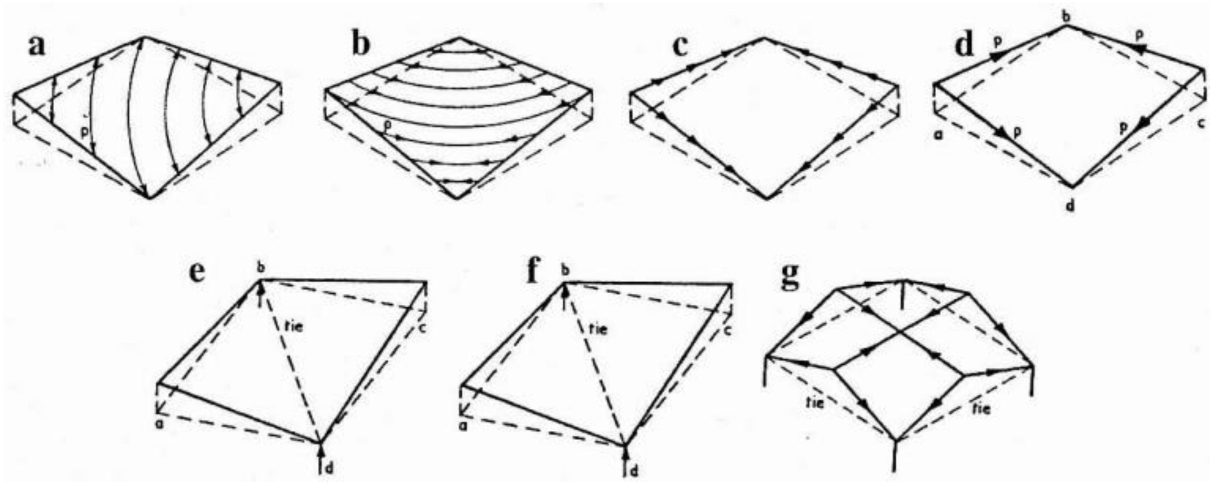


Figure 4. Structural action of the hyperbolic paraboloid (Design, Analysis and Construction of space Structure, the Mero Legacy)

This shell structure can be conceptualized as a system comprising intersecting "arches" (as seen in Figure 4a) and "suspension cables" (illustrated in Figure 4b), with half of the load being supported by the "arches" and the remaining half by the "suspension cables."

Therefore, the surface experiences direct compression along directions parallel to the "arches" and direct tension along directions parallel to the "cables." Because the sections parallel to both diagonals result in the same parabolic shape, the force at any given point along the edge, due to the "arch," equals the force applied by the "cable." Additionally, these forces act at equal angles to the edge but in opposite directions – one inward and the other outward. As a result, there is no component acting perpendicular to the edge. Consequently, this dual system of forces can be broken down into a series of shear forces along the edge, as depicted in Figure 4c. These shear forces can be combined into a single force per edge, as shown in Figure 4d, necessitating the inclusion of an edge stiffening element, typically referred to as an "edge beam," to carry this force.

The method used to transmit these edge forces to the ground depends on whether a single unit (as seen in Figure 4f) is employed or if multiple units are grouped together (as illustrated in Figure 4g).

For a single unit, only two supports are needed when there is an uniformly distributed load. If the roof is upheld by vertical columns at points b and d, as shown in Figure 4d, then the edge beams experience compression, and the forces represented as P at the supports can be separated into a downward vertical force, denoted as V, and an outward horizontal force, termed H, as illustrated in Figure 4e. The vertical force is directly channeled downward through the column to the ground. Typically, the outward horizontal force is countered using one of two methods: points b and d can be connected (as demonstrated in Figure 4f), or the column can be engineered as a buttress to withstand the bending stresses generated by the horizontal force acting at its top. When dealing with multiple units, it is often feasible for one unit to counteract the forces exerted by its neighboring unit. In Figure 4g, the forces along the ridge members balance each other out, provided that the units are evenly loaded. In such cases, it is only necessary to restrain the four corners by employing ties around the perimeter.

2.3. The general concept of space truss structures formation

The Platonic Solids, which are simply five regular convex polyhedra, are shown in Table 1. The hexahedron group is made up of the tetrahedron, octahedron, and hexahedron, while the DI-group is made up of the icosahedron and the dodecahedron.

The design of planar, terraced, and folded-plate space trusses, which rely on metrics based on 2 and 3, is based on the hexahedron group. On curved carrier surfaces, however, it can also provide the topological foundation for space structures. The design of geodesic domes, whose metrics are related to the Golden Section, is in turn based on the DI-group.

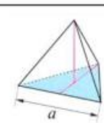
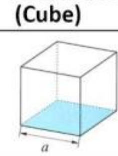
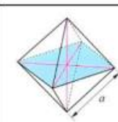
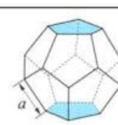
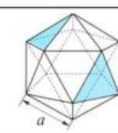
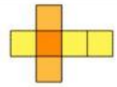
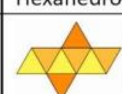
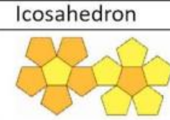
	Tetrahedron	**Hexahedron (Cube)**	**Octahedron**	**Dodecahedron**	**Icosahedron**
					
Side faces	4 equilateral triangles	6 squares	8 equilateral triangles	12 regular pentagons	20 equilateral triangles
Vertices	4	8	6	20	12
Edges	6	12	12	30	30
dual of	Tetrahedron	Octahedron	Hexahedron	Icosahedron	Dodecahedron
Net					
Number of different nets	2	11	11	43380	43380
Number of edges at a vertex	3	3	4	3	5
Number of vertices of a face	3	4	3	5	3

Table 1. The Platonic Solids (Design, Analysis and Construction of space Structure, the Mero Legacy)

The number of alternative planar developments from which a solid can be constructed is indicated by the "number of different nets" in Table 1.

The spatial blocks tetrahedron, half-octahedron, and half-cuboctahedron (variants 1 and 2) can be used to create the most common node-bar space structures, with tetrahedron and half-octahedron being the most used. The metrics of the previously described space units are shown in Figs. 5 to 8.

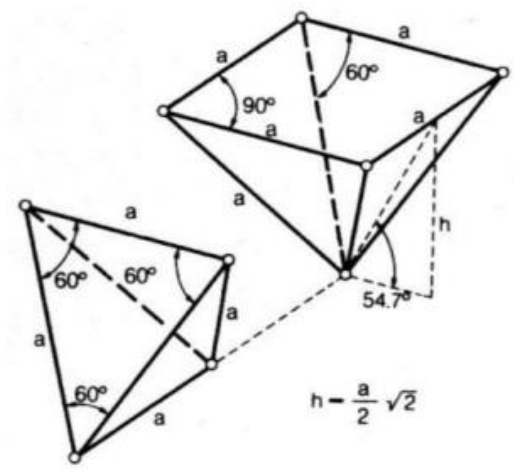


Figure 7. Half-octahedron and tetrahedron (½ O + T) (Design, Analysis and Construction of space Structure, the Mero Legacy)

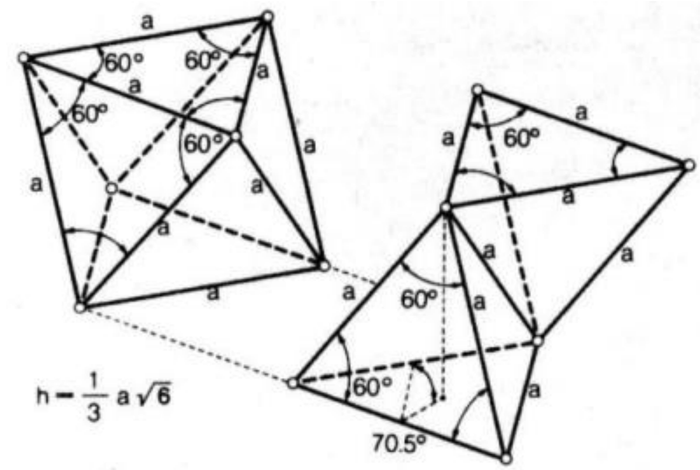


Figure 8. Octahedron and tetrahedron (O + T) (Design, Analysis and Construction of space Structure, the Mero Legacy)

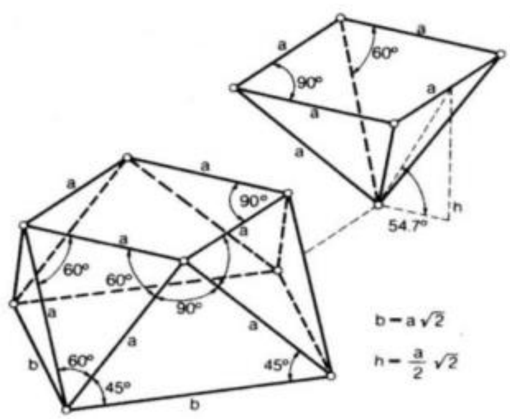


Figure 9. Half-cuboctahedron_1 and half-octahedron (½ O + ½ CO) (Design, Analysis and Construction of space Structure, the Mero Legacy)

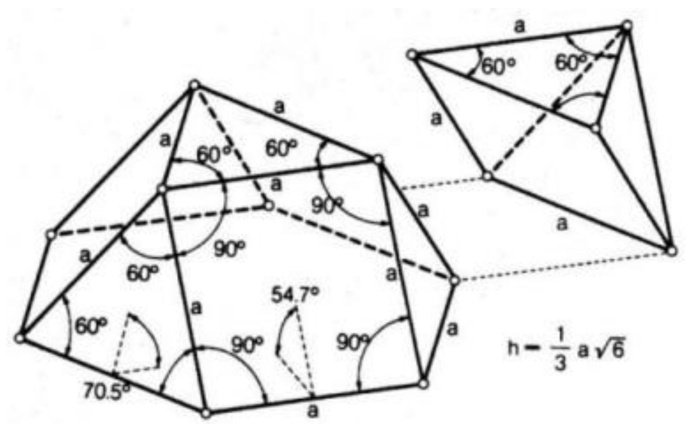


Figure 10. Half-cuboctahedron_2 and half-octahedron (½ O + ½ CO) (Design, Analysis and Construction of space Structure, the Mero Legacy)

The examples of planar, terraced, and curved space structures that can be constructed using the fundamental elements mentioned above are shown in the following sections.

Here are some examples on planar carrier and *curved carrier* surfaces:

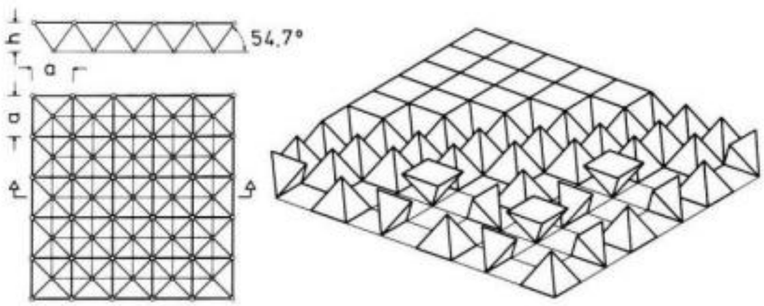


Figure 11. Two-layered, square-on- offset -square grid (Design, Analysis and Construction of space Structure, the Mero Legacy)

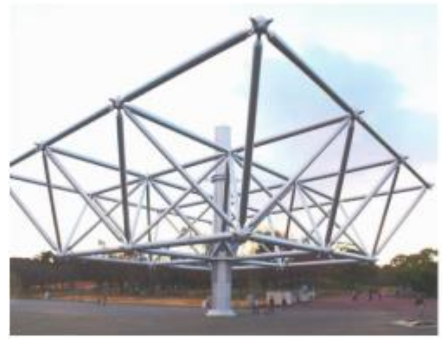


Figure 12. Remains of the giant space truss for the Expo 1970 (Design, Analysis and Construction of space Structure, the Mero Legacy)

Figure 13. Tokuyama multipurpose hall Taiyo Kogyo TM-System (Design, Analysis and Construction of space Structure, the Mero Legacy)

Figure 14. Cultural Center in Baku, Mero-System (Design, Analysis and Construction of space Structure, the Mero Legacy)

2.4. Space Structures Classification

Translation, stretching or contracting, rotation, and their combinations are the primary operations for the geometric production of surfaces. Freeform surfaces, which are typically created using NURBS techniques, and specific procedures, executed using specialized programs, as in the case of geodesic domes, round out the range.

Fig. 14 provides an overview of surface generation options. Even though freeform and translation surfaces appear to be opposites of one another, practical design frequently aims to approximate freeform surfaces with translation surfaces to produce planar quadrangular meshes, particularly for glazed projects. In this thesis, we mainly focus on the translation surface , especially with hyperbolic paraboloid and Nurb-surfaces.

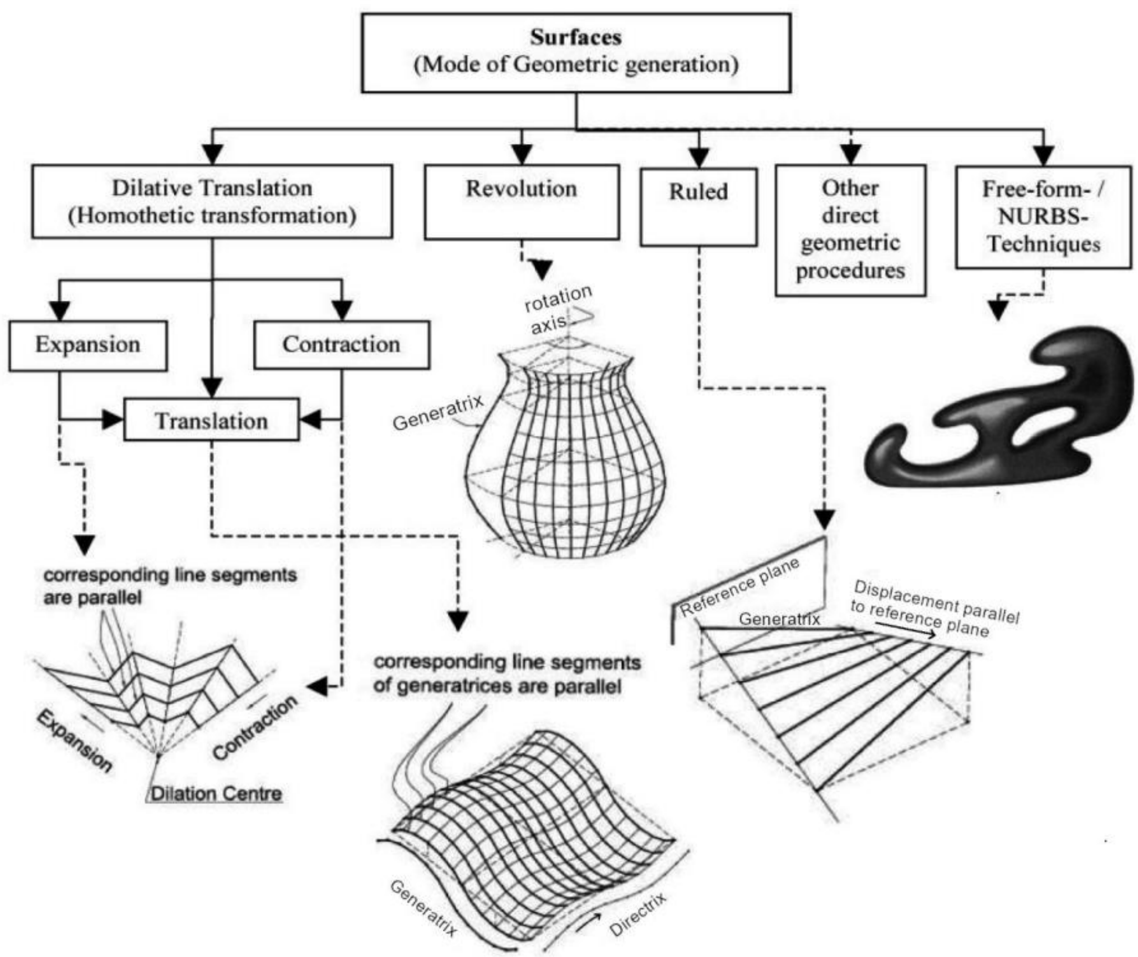


Figure 15. Surface genealogy according to generation mode (Design, Analysis and Construction of space Structure, the Mero Legacy)

2.4.1. Structural Nets on Translation Surfaces

As seen on the left of fig. 15, a translation surface is created by shifting a curve called the generatrix parallel to itself along a curve called the directrix. The resulting net has flat meshes or panels. Additionally, the generatrix can be affinely expanded or contracted at specific translations. Only if the generatrix stays parallel to itself will the meshes in this case remain planar. On the right image of Fig. 15, when generatrices are additionally rotated, the case is depicted with a slight diversion.

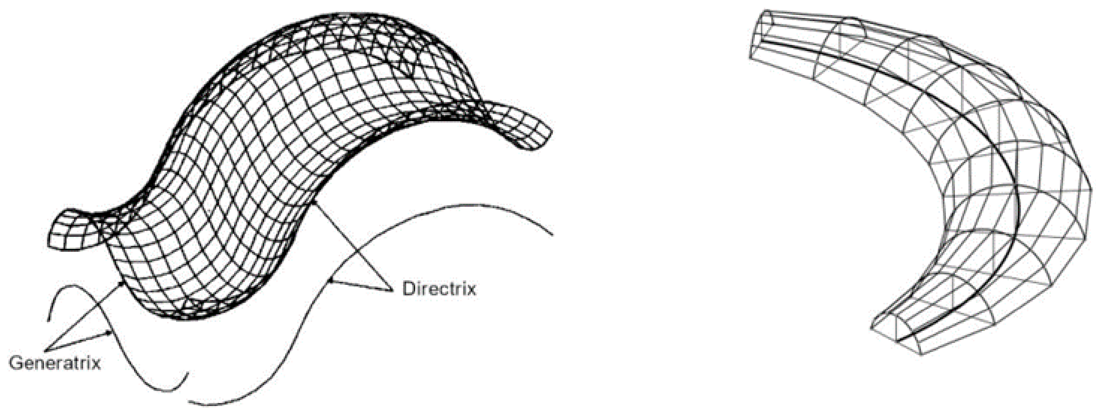


Figure 16. Translation surfaces and the generatrices additionally with +/- stretching and rotations (Design, Analysis and Construction of space Structure, the Mero Legacy)

In order to make the structural grid along the curved rails path fitted gradually inside the prescribed boundary, rotations are used in addition to translations with stretching of the generatrix in the geometric design of the glazed ceiling for the major railway station in Berlin (fig. 16). In this structure, the rotations' insignificant magnitude ensures that the glass meshes remain nearly flat.

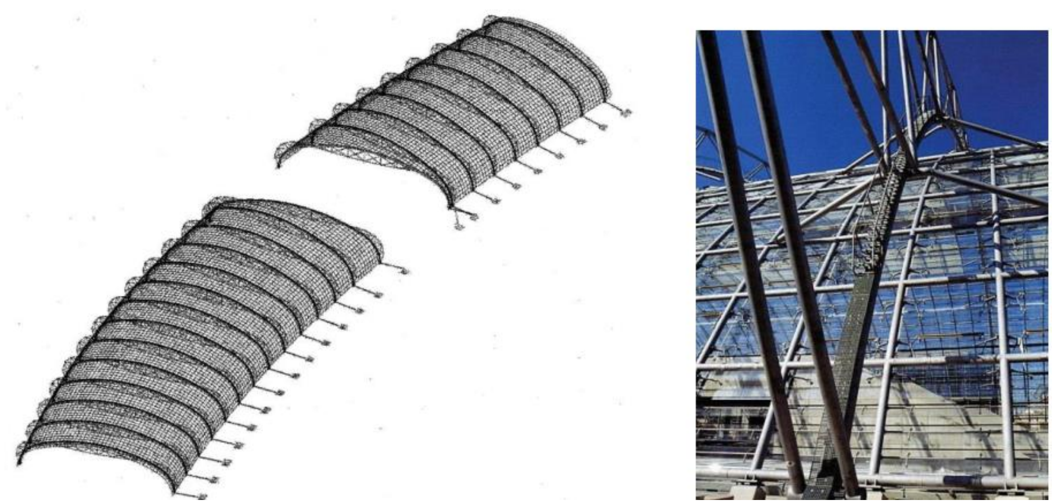


Figure 17. the major railway station in Berlin (Design, Analysis and Construction of space Structure, the Mero Legacy)

2.4.2. Translation Surface with Hyperbolic Paraboloid

The glazed roof of the Leipzig Industry Palace is formed as a hyperbolic paraboloid on a horizontal boundary with an elongated trapezium shape. A polyline arch is translated along the opposing long edges of the trapezium-shaped boundary to create the grid (fig. 17).

Figure 18. Example Industry Palace, Leipzig, sbp, 1994 (Design, Analysis and Construction of space Structure, the Mero Legacy)

Creation of structural grid

The Industry Palace in Leipzig serves as a model for the fundamental generation approach (fig. 18)

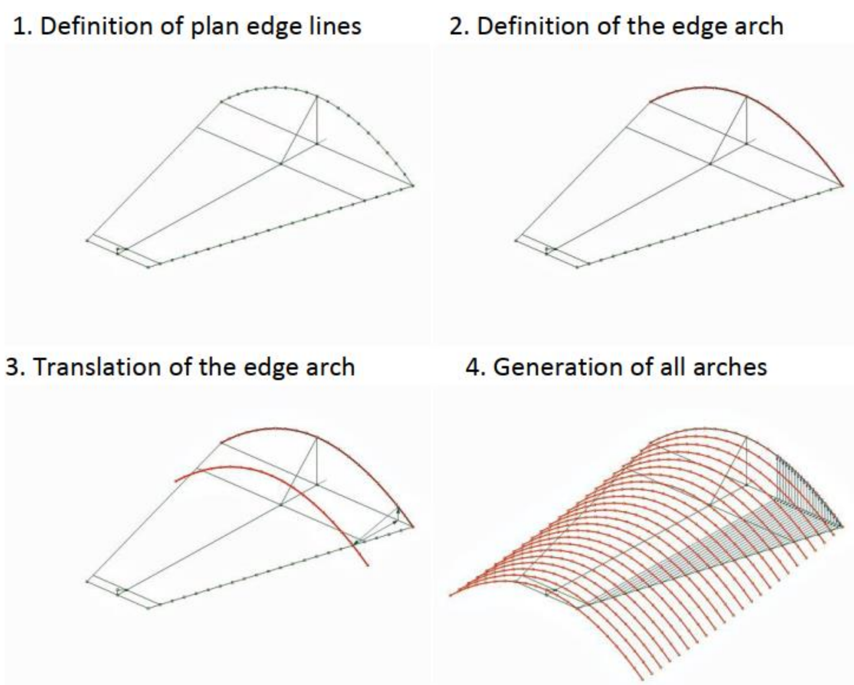


Figure 19. The structural grid generation for the Industry Palace Leipzig with Rhino (Design, Analysis and Construction of space Structure, the Mero Legacy)

2.4.3. NURBS-Surface

In order to distinguish free form surfaces from other types, such as the traditional analytical surfaces that can be specified by a comparatively simple formula of analytic geometry, or surfaces that are numerically obtained by simulating a form-giving agent like gravity on a flexible hanging membrane, form designers and modelers generally refer to free form surfaces as NURBS-Surfaces.

A specific type of mathematical representation of free form curves and surfaces is known as "non-uniform rational B-Splines," or NURBS. In the field sof architecture, product design, structural engineering, and construction design, the form designer utilizes NURBS method based computer application custom-designed for modeling and design free form shape. As an illustration, by considering the widely used application Rhinoceros and its numerous add-ons and plug-ins which are primarily designed to model free forms and networks algorithmically and parametrically and to present and analyze those models geometrically. A grid of curves extending in the surface's u- and v- directions, which correspond to the surface's intrinsic coordinate directions in its two-dimensional space, can be used to represent a NURBS-surface. An example of how a NURBS surface, its iso-curves, and its control-points framework are often represented is shown in Fig. 19. But for practical purposes, a NURBS-surface can alternatively be described as a collection of arbitrary curves that represent various portions of the desired surface.

The resulting shape of a NURBS-surface is produced by the built-in NURBS surface generator in the context of a NURBS-design computer application like Rhinoceros (Rhino).

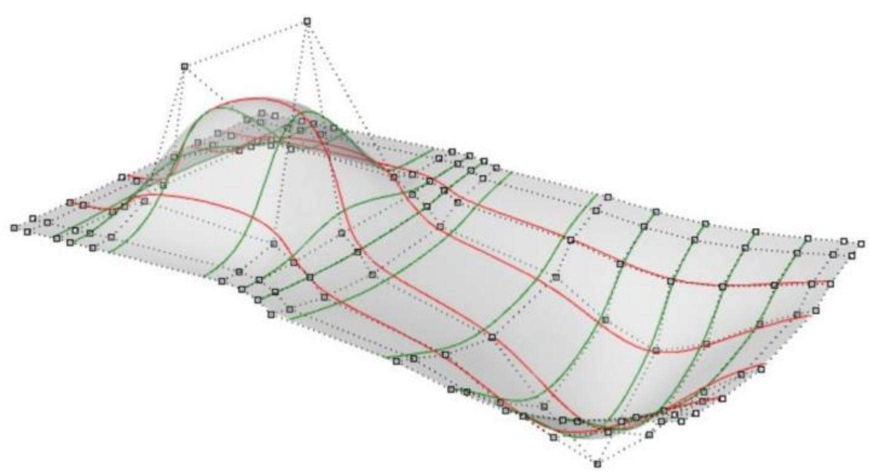


Figure 20. The control points structure of a NURBS-surface, with red iso-curves in the u- direction and green iso-curves in the v-direction. (Design, Analysis and Construction of space Structure, the Mero Legacy)

The following processing steps are generally necessary for the development of a structural network:

1. Definition of boundary and sections curves as either free form curves (splines) or simple analytical curves (arc, parabola, ellipse, etc) (fig. 20)

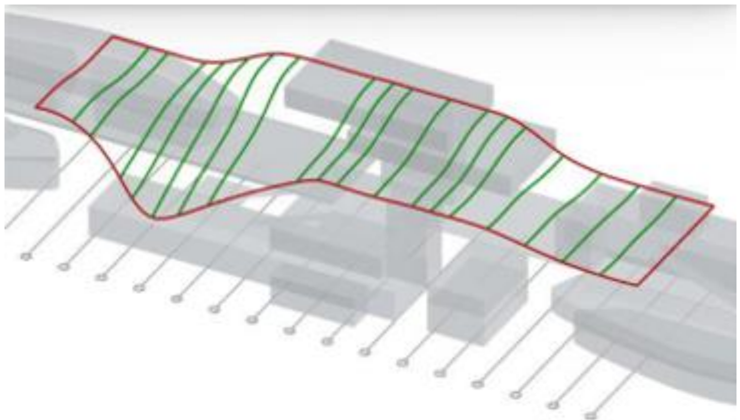


Figure 21. Definition of boundary and section curves. (Design, Analysis and Construction of space Structure, the Mero Legacy)

3. Producing the free form by first generating a macro-grid using the boundary and section curves, then producing distinct patches inside the grid meshes. As shown in fig. 21, these patches join to create a single surface while making sure satisfaction of continuity, curvature, and tangential criteria at the shared inter-boundaries.

Numerous NURBS programs offer simpler customized methods like the "loft"-function, which may create an uninterrupted free form surface from a collection of discrete free form curves.

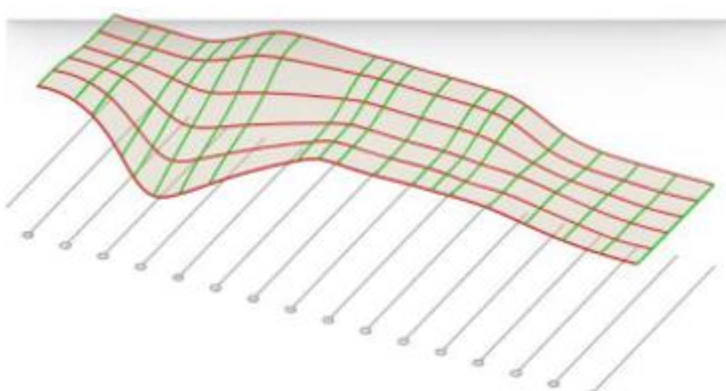


Figure 22. Macro-net (Design, Analysis and Construction of space Structure, the Mero Legacy)

3. There are different ways to approach the net creation on a free form surface. A straightforward method is to connect the intersection points of a grid of extracted u- and v-isocurves in the required net density.

The net generation process is depicted in several phases in figures. 20 through 22.

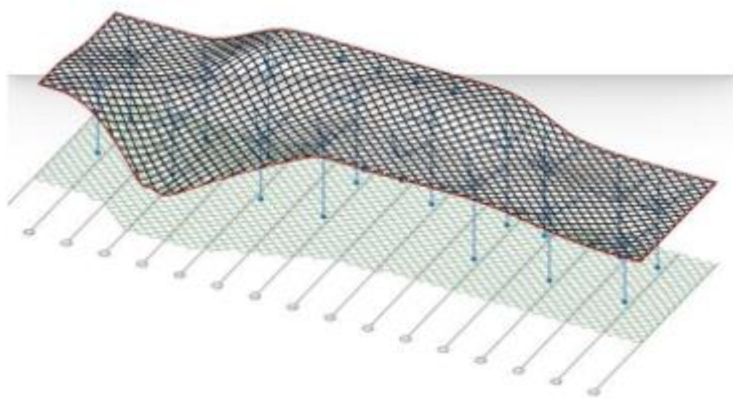


Figure 23. Projection of a diagonal net (Design, Analysis and Construction of space Structure, the Mero Legacy)

For instance, the majority of the structural grid for the glazed roof of the New Milan Fair was built using a parallel projection technique. Here, the architect provided free-form surface which is used for projection of a planar horizontal diagonal grid (Massimiliano Fuksas).

The roof is approximately 1200 meters long and 30 meters wide. In Fig. 23, a section of the building is depicted. There are roughly 46300 m2 of glass surfaces (fig. 24).

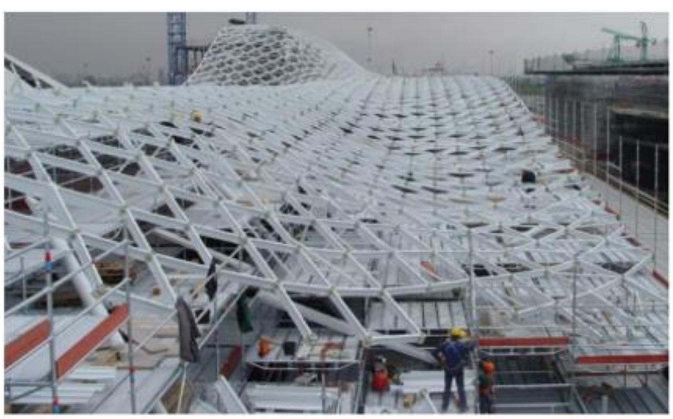


Figure 24. Structural grid during assembly (Design, Analysis and Construction of space Structure, the Mero Legacy)

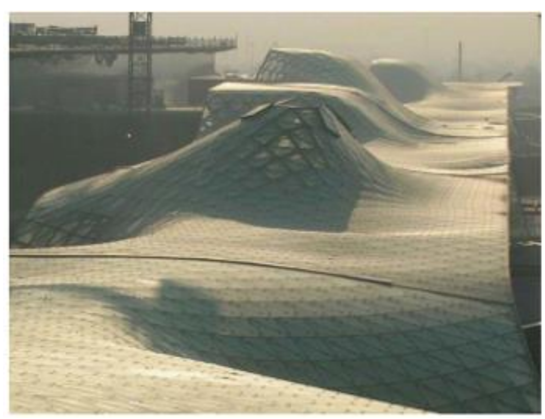


Figure 25. the glass cladding on the structural grid (Design, Analysis and Construction of space Structure, the Mero Legacy)

3. PRE-DESIGN CONSIDERATIONS

Before embarking on the actual design process, there is a multitude of considerations to address. These encompass a wide range of factors, spanning from client requirements to the insurance of structural integrity and optimal functionality. Typically. structural designers have many choices to address these requirements. Consequently, it becomes essential to identify the principal requirements to facilitate the development of an adequate design. These conditions can be envisioned as design constraints, serving as the bounds within which the designer can get to be creative in the design. The examination of prior designs, particularly those pertaining to hypar space truss structures, aids in establishing prerequisites for the roof structure design.

3.1. Stage requirements

Figure 26. Stage view in Margaret island open-air theatre (Source: Google image)

The primary design requirement pertains to the dimensions of the roof, which are contingent on the size of the stage it is intended to cover. In this project, the stage must accommodate a concert with over 10 attendees and a substantial amount of equipment concurrently. While precise calculations are elaborated upon later in this thesis, it can be intuitively stated that this space truss will encompass a considerable span, exceeding 20 meters. It's also important to note that space trusses are highly sensitive to imperfections and tend to buckle quite easily, even before reaching their buckling load limit. Designers must take this into account throughout the design process. These imperfections not only pose challenges during construction but may also introduce additional compressive forces, which can undermine the stability of the structure.

3.2. Aesthetic view of point

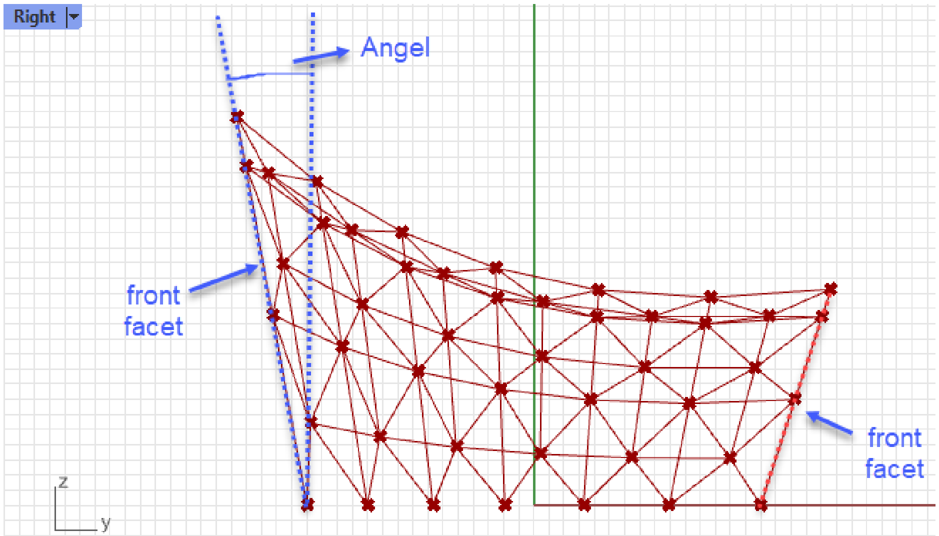


Figure 27. Right view of structure in Rhino

A structure's aesthetics are a fundamental consideration in the design process. The aim is to create a structure that appears pure, uncluttered, preferably uncomplicated, and imparts a sense of stability. To begin with, it is advisable to increase the angle of the front facet of the roof, which can range from 5 degrees to 25 degrees. This adjustment offers two advantages: Firstly, it will enhance the structure's visual appeal and secondly, the angle acts as an overhang, providing shade and shielding a portion of the stage from direct sunlight.

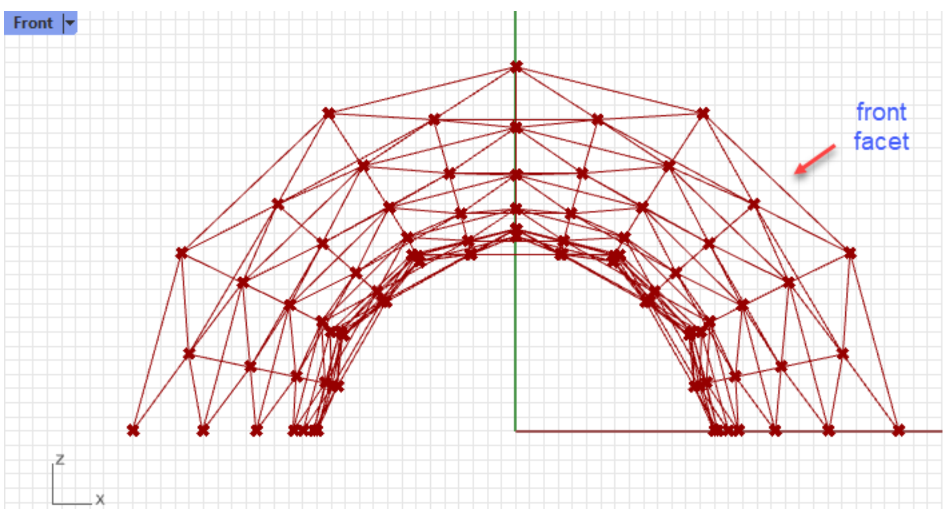


Figure 28. Front view of structure in Rhino

In addition, to maintain the desired negative Gaussian curvature of the shape, the front facet cross-section should be proportionately larger than the middle cross-section where the stage will be located. This not only enhances the aesthetics but also allows for a greater number of concertgoers to enjoy the performance without visual distortion.

3.3. Cost of production

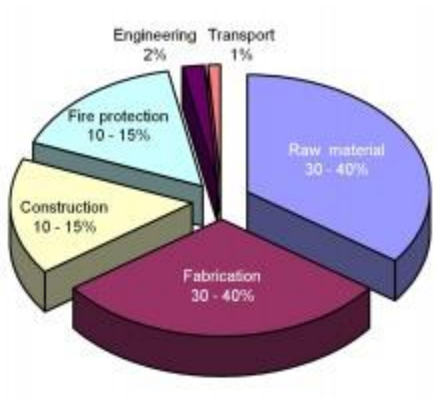


Figure 29. Breakdown of costs of steel structure (Cost planning through design stages)

The development of the steel structure design can be subdivided into two main components: the steel members, which serve as the primary supports carrying the loads, and the connections and fittings, including stiffeners and joints responsible for transferring forces between structural elements. During the detailed design phases, cost estimations for the structural steelwork entail a breakdown into distinct constituents, specifically the steel members and the considerations for connections and fire protection. Once the steel members have been dimensioned and selected, their individual lengths are measured and multiplied by the corresponding weight (expressed in kg/m) to determine the total weight. To ascertain the cost of the structure, each component is assigned a rate per tonne and subsequently summed.

As illustrated in the above figure, the structure's cost encompasses several elements, including raw materials, fabrication, engineering, transportation, fire protection, and construction. The figure illustrates that raw material costs typically constitute approximately 30%-40% of the total cost, while fabrication costs also account for a similar 30%-40%. Consequently, it is of paramount importance to economize on these factors, given their substantial contribution to the overall cost of a steel structure. This entails reducing the volume of steel used and the number of joints, which can help mitigate expenses.

4. MODELING OF THE STRUCTURE

The modeling of the structure is done by using rhinoceros and grasshopper plus a plug-in like LunchBox to generate the space truss. The steps are shown below:

- Step 1: Principle arches development
- Step 2: Nurbs-surface generation using loft function
- Step 3: Space truss creation using LunchBox

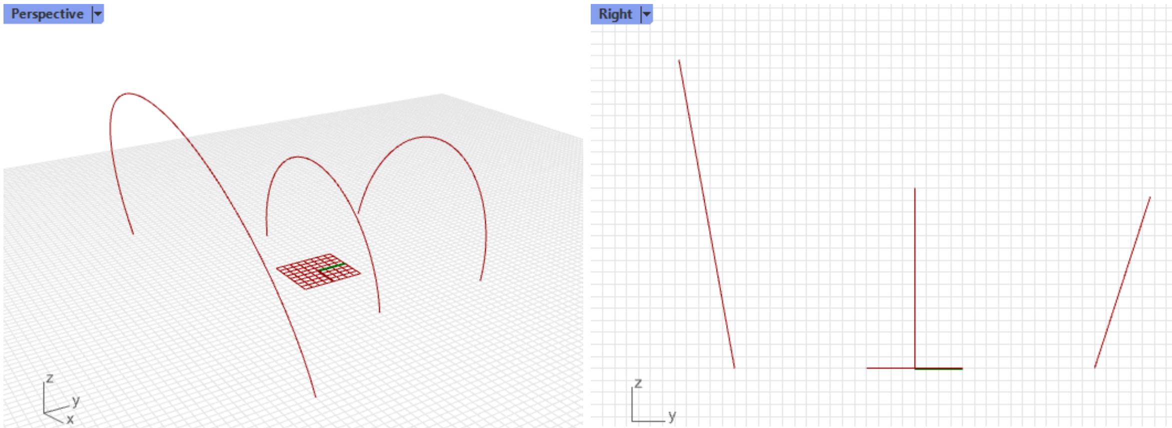


Figure 30. Principle arches in perspective and side views

Initially, three primary arches are established, positioned at the front, middle, and rear of the structure. These arches serve as key elements in delineating the boundaries and section curves required for the subsequent generation of NURBS surfaces in step 2. The distance between the front and back arches is 15 meters. While the middle arch size is fixed at 30 meters in diameter, the front and back arches can be flexibly modified in terms of their dimensions, the type of symmetrical arch employed (such as round, segmental, horseshoe, or parabolic), and the angle at which the arches are inclined from the vertical axis as observed in the side view.

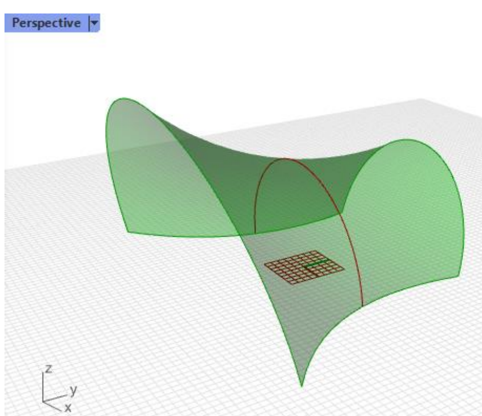


Figure 31. Nurbs surface generation

Secondly, the NURBS surface is created using the soft function within Grasshopper, following a structured sequence of the provided arches. Further information regarding the generation of NURBS surfaces is elaborated upon in section 2.4.3: NURBS Surface.

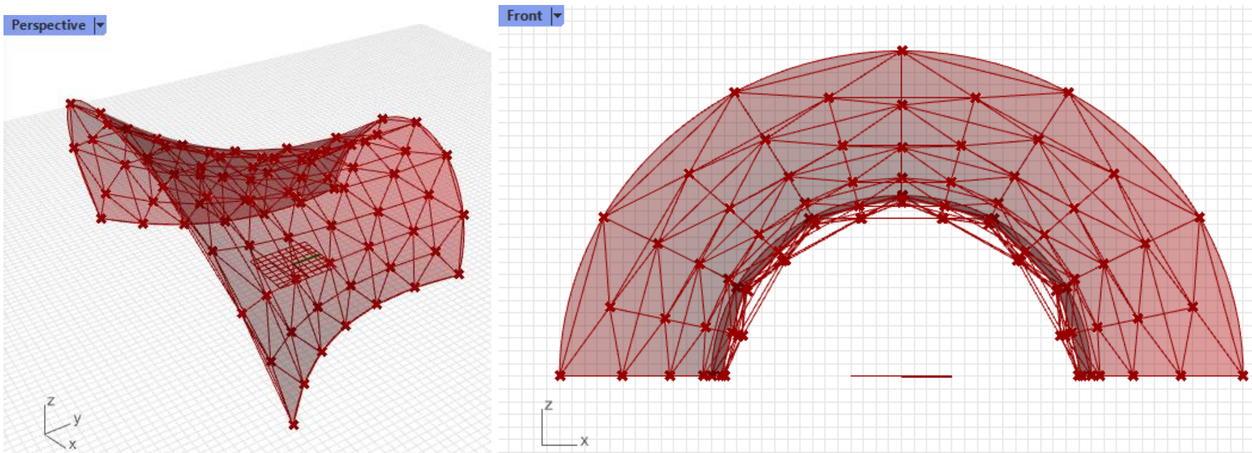


Figure 32. Space truss generation

Thirdly, the generation of the space truss is carried out utilizing the LunchBox plug-in. Within this plug-in, the space truss function is employed to construct a space truss framework on a designated surface, which serves as the roof surface. This space truss structure is built using spatial blocks half-octahedron and tetrahedron in the figure 6. During this phase, adjustments can be made freely, guided by the provided divisions in both the lateral and horizontal directions, as well as the truss depth, as depicted in the accompanying figure below. But in this thesis, truss depth is set at 2 meter.

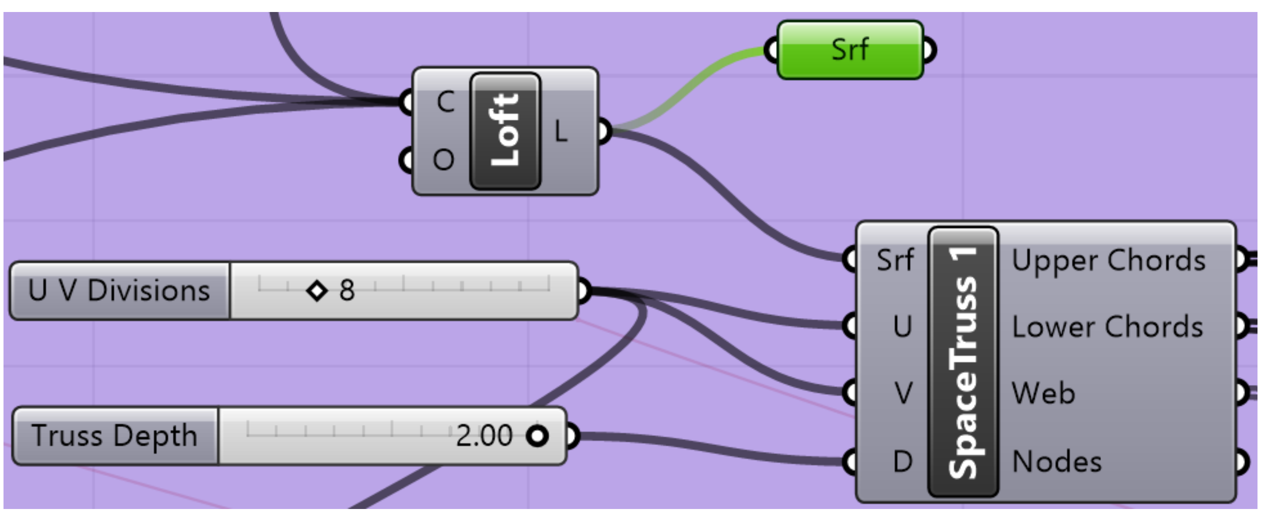


Figure 33. Space truss function in LunchBox plug-in

5. Lay-out Optimisation

The layout optimization process is executed within Rhinoceros Grasshopper utilizing the Galapagos plug-in. Galapagos offers two categories of optimization algorithms: evolutionary and annealing. For the purpose of this thesis, the evolutionary solver is employed to optimize the layout due to the small number of given degree of freedom. This particular solver method systematically seeks out a favorable result and subsequently refines it by introducing minor adjustments to the parameters.

5.1. Optimisation parameters

As outlined in Section 3: Pre-Design Considerations, achieving an efficient and ideal structural form necessitates the consideration of three key factors: the initial client requirements pertaining to the stage, the aesthetic perspective, and construction cost. Each of these factors entails a specific parameter serving as a variable within the optimisation process. The inner joints is subjected to equally concentrated loads with a total magnitude of 500 kN to simulate a heavy structure. In the case of the initial client requirements and the aesthetic perspective, the chosen variables involve the cross-sectional configuration of the front facet and its angle measured relative to the vertical axis. Conversely, for the purpose of minimizing construction costs, adjustments are made to the mesh division of space.

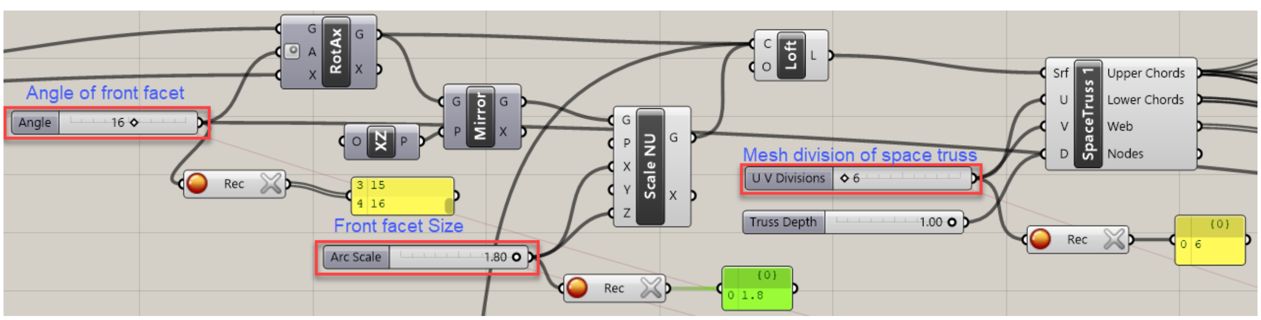


Figure 34. Space truss function in LunchBox plug-in

The diagram depicted above displays adjustable sliders pertaining to three variables involved in the optimisation process: the angle of the front facet, the size of the front facet, and the division of the space truss mesh. Regarding the angle of the front facet, this parameter spans a range from 5 to 25 degrees. A minimum angle of 5 degrees is adhered to during the design process, particularly from an aesthetic standpoint.

The size of the front facet is subject to scaling adjustments, with a scale factor falling within the specified range of 1.5 to 1.8 in both the x and z directions. This signifies that a front facet with a minimum diameter of 45 meters will be provided, allowing for the arrangement of an extended row of seats to accommodate concertgoers who wish to enjoy the performance. Concerning the mesh division, the range varies between 6 and 8, with the specific condition that only even numbers within this range are considered.

5.2. Optimisation criteria

Based on the requirements outlined in Section 3: Pre-Design Considerations, the optimization process will focus on Cost of Production criteria:

Regarding the cost of production, the aim is to minimize it while maintaining a balance with the structural bearing capacity factor. The initial cost estimate of the structure is calculated using the following formula:

Cost of Structure = Mass x Price per Kilogram + Number of Joints x Price per Joint

To estimate the cost of fabrication and raw materials, a preliminary connection was designed using connection design software Ideastatica. The cost of connection production is automatically calculated based on the cost per unit weight configured in the program settings. The default cost settings, which are provided below, are considered suitable for the estimation.

Steel

Steel grade	Total weight [kg]	Unit cost [€/kg]	Cost [€]
S 450	131.64	2.00	263.28

Bolts

Bolt assembly	Total weight [kg]	Unit cost [€/kg]	Cost [€]
M22 8.8	13.98	5.00	69.88

Welds

Weld type	Throat thickness [mm]	Leg size [mm]	Total weight [kg]	Unit cost [€/kg]	Cost [€]
Double fillet	7.5	10.6	0.53	40.00	21.20
Fillet rear	4.0	5.7	0.61	40.00	24.36

Hole drilling

Bolt assembly cost [€]	Percentage of bolt assembly cost [%]	Cost [€]
69.88	30.0	20.96

Cost summary

Cost estimation summary	Cost [€]
Total estimated cost	399.68

Figure 35. Connection cost estimation in Ideastatica

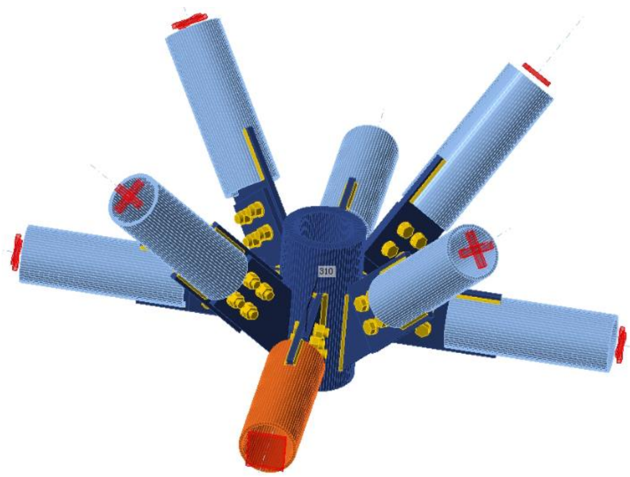


Figure 36. Preliminary joint design in Ideastatica

Regarding the load-bearing capacity of structural elements, the objective is to minimize the stress levels derived from the internal forces within these elements. Throughout the optimization process, all members' cross-section are optimised using Karamba3d plugin to meet required ultilisation of .steel members after performing cross section resistance checks, stability analyses, and serviceability limit state design checks.

5.3. Result

Upon completing the optimization process, the outcomes are as follows: the ideal configuration is characterized by an angle of 5 degrees, a front facet size scaled to 1.5, and a mesh division of the space truss set at 8. This configuration yields the lowest total cost, measuring 116923.1. The member experiencing the highest ultilisation registers at 0.846111, and the mass of structure amounts to 19641.03 euros. A detailed breakdown of these results is presented below:

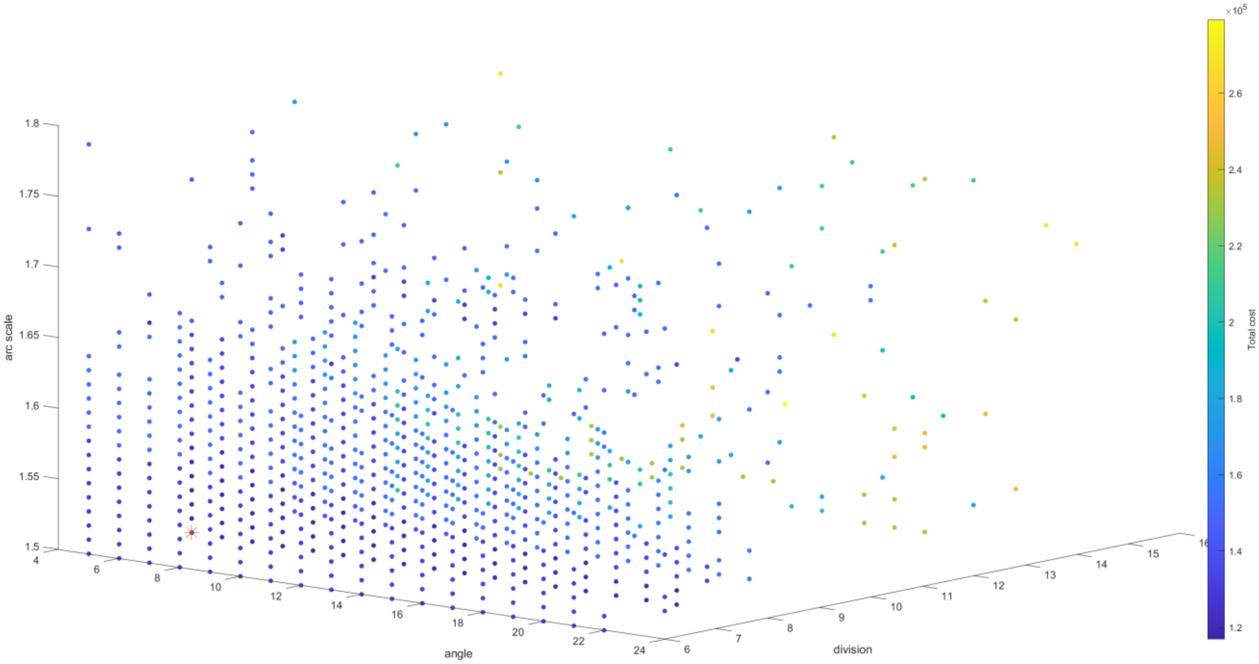


*Figure 37. Optimisation result (the most optimal result is marked *)*

6. Detail analysis

6.1. Purpose and scope

The primary aim of the design is the calculation of structure. Designing the foundation does not belong to the scope, only the steel design of structure.
This report contains the structural analysis of the structure following conditions:
a) Ultimate Limit States (ULS)
b) Serviceability Limit States (SLS)
The analysis shows that the structure is sufficient to carry the loads in Ultimate limit states. The overall load-carrying capacity of all steel elements and connections is sufficient in accordance with Eurocode and Hungarian National Annex.
The displacements and deformations in Serviceability limit states are within the prescribed limitations. The maximum deflections and deformations of the structural elements are found to fulfil the requirements of the design code.

6.2. . Analysis tools

The structural analyses have primarily been performed by using Dlubal RFEM 6.02 FEM software. The basic load calculations and detailed design have been generated automatically in RFEM. RWIND 2 CFD analysis software, IDEA Statica CBFEM structural design software and MathCAD are ultilised for wind load generation, connection design and manual calculation during the design.

6.3. . System of units

The System International (SI) is used for all structural design and the distance and measurements of structures are also added to drawings in SI units.

Quantity	Unit
Angle	degree [deg]
Length	millimetre [mm]
Mass	kilogram [kg]
Force	kilo Newton [kN]
Time	Second [s]
Temperature	degree Celsius (°C)

Table 2. Units

6.4. . Coordinate system

A global coordinate system is chosen for the computer model. The coordinate system is a rectangular coordinate system (X, Y, Z) where X and Y are horizontal axis and Z is vertical to upward axis.

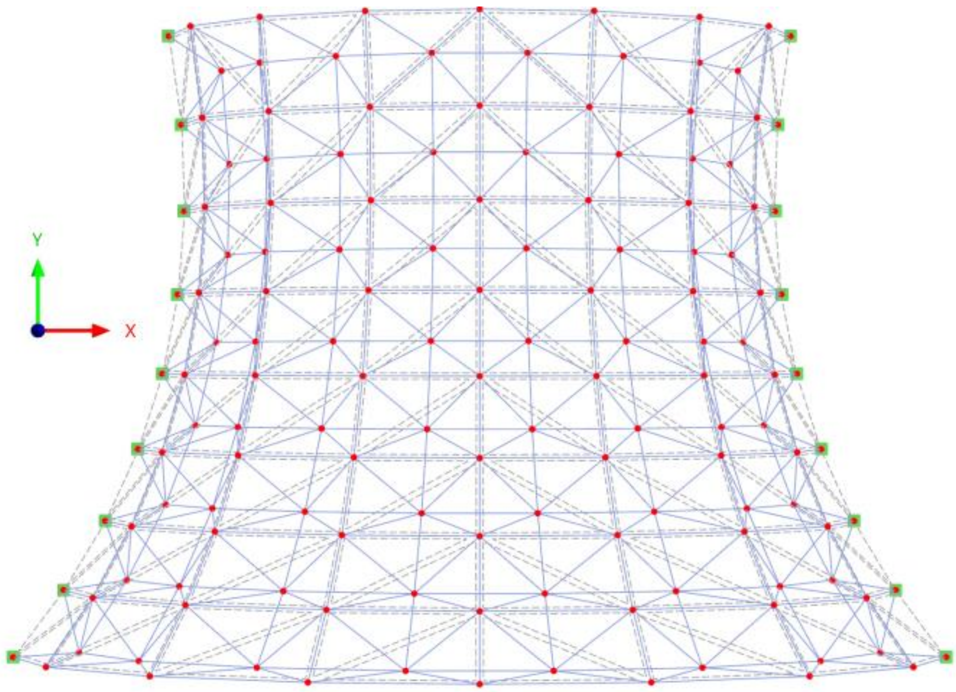


Figure 38. Global coordinate system of the calculation models

The local coordinate system is associated with each member. The X-axis is directed in frame direction, while the Z-axis is directed upwards.

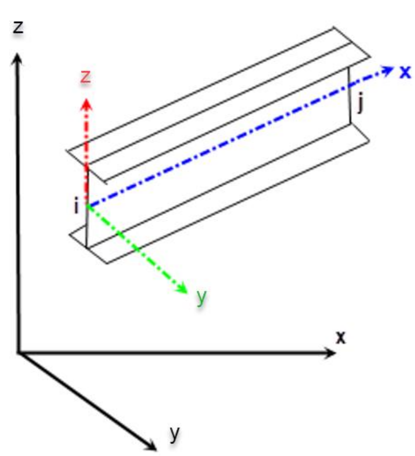


Figure 39. Members local coordinate system (Source: Google image)

6.5. Analysis and code check parameters

For deriving the internal forces, generally, a first-order elastic calculation has been performed, since sway buckling mode is not relevant for the structure, the second order effects are negligible for the structure.

RFEM structural analysis software performs capacity checks of members in accordance with Eurocode standard and Hungarian National Annex. For different stability examinations the members were given buckling length and relevant parameters according to their unsupported lengths, boundary conditions and actual structural behaviour, taken into consideration.

6.6. Material properties

Steel grade S355

Hot-rolled products of non-alloy structural steel were used for all steel members. Mechanical properties are according to the ref. /4/:

- *fy= 355 N/mm2 (MPa) - Yield stress*
- *fu= 490 N/mm2 (MPa) - Tensile stress*
- *E= 210 000 N/mm2 (MPa) - Young's modulus of elasticity*
- *ν= 0.3 - Poisson's ratio*
- *ρ= 7850 (kg/m3) - Mass density*

6.7. Material safety factors

Material factors in accordance with the Eurocode standard

- *for profiles, plates and full penetration welds* $\gamma_{My} = 1.10$; $\gamma_{Mu} = 1.50$
  for the EN code settings:
  $\gamma_{M0} = 1.00$;
  $\gamma_{M1} = 1.00$
  $\gamma_{M2} = 1.25$ *for bolts and fillet welds*

6.8. Wind loads generation in Rwind 2

6.8.1. Overview

In the thesis, wind loads applied on the structure are generated through Computational Fluid Dynamics (CFD) wind analysis using the Rwind 2 software. Rwind 2 is an excellent tool designed for generating wind-induced loads on various structures. It operates as a standalone program, utilized externally to establish load cases and wind loads for models in RFEM 6 and RSTAB 9. Employing a numerical CFD model, RWIND 2 conducts a fluid-mechanics simulation to depict the flow around objects in a wind tunnel. The outcome of this simulation process yields specific wind loads tailored for implementation in RFEM or RSTAB.

A three-dimensional mesh comprising finite volumes is utilized for the simulation process. RWIND facilitates automatic meshing, and the overall mesh density, along with local mesh refinement near the model, can be easily adjusted using a few parameters. To generate a finite volume mesh for Computational Fluid Dynamics (CFD), the model must adhere to topological correctness. In RWIND 2, model boundaries are defined by triangles. The term 'topologically correct' implies that these triangles must form a closed triangular mesh—each mesh edge has precisely two adjacent triangles, and the triangles should not intersect or touch each other, except for common edges and vortices. This requirement is achieved through the implementation of a simplified model, represented by a specialized mesh that 'shrink-wraps' the original model. This mesh maintains topological correctness and serves as a model boundary for generating a three-dimensional finite volume mesh for CFD calculations.

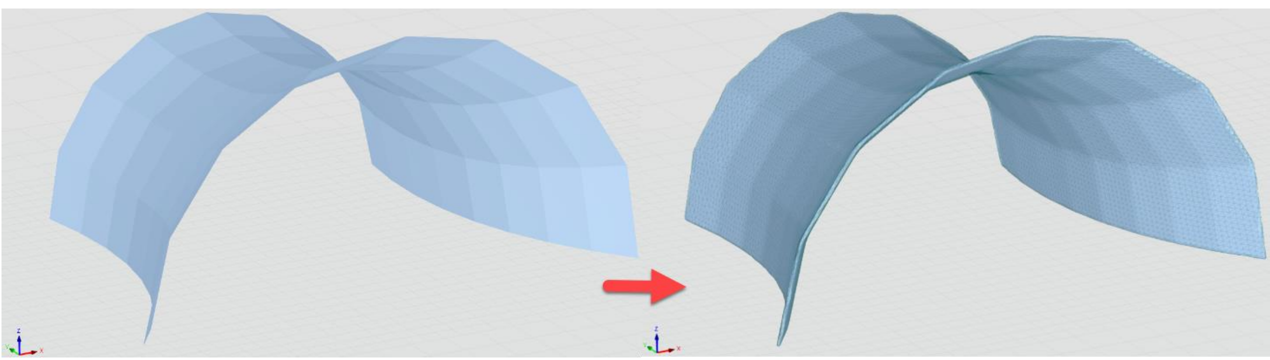


Figure 40. Transformation of the structure surface to shrink-wrapping model

To compute the airflow and surface pressure on the model, a numerical solver for incompressible turbulent flow using the finite volume method is employed. The outcomes are subsequently extrapolated onto the model. Presently, RWIND utilizes the OpenFOAM® software package, which, based on numerous tests, has demonstrated excellent results and is extensively utilized as a tool for Computational Fluid Dynamics (CFD) simulations.

The simulation outcomes encompass pressure and velocity fields surrounding the model, streamlines, surface pressure, and member forces. These findings depict the results of a stationary analysis and are visually represented through color maps (isobands) or isolines directly on the model. Additionally, the streamlines can be showcased in an animated view, facilitating a comprehensive assessment of the impacts of laminar and turbulent flow.

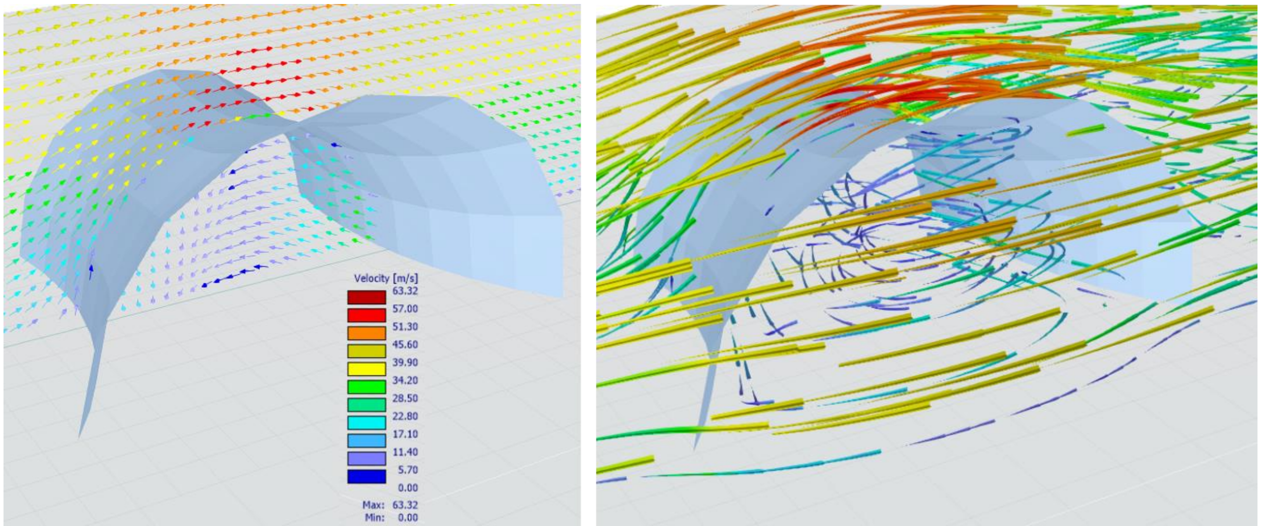


Figure 41. Wind Velocity Vector and Streamline (Wind flows in +X direction)

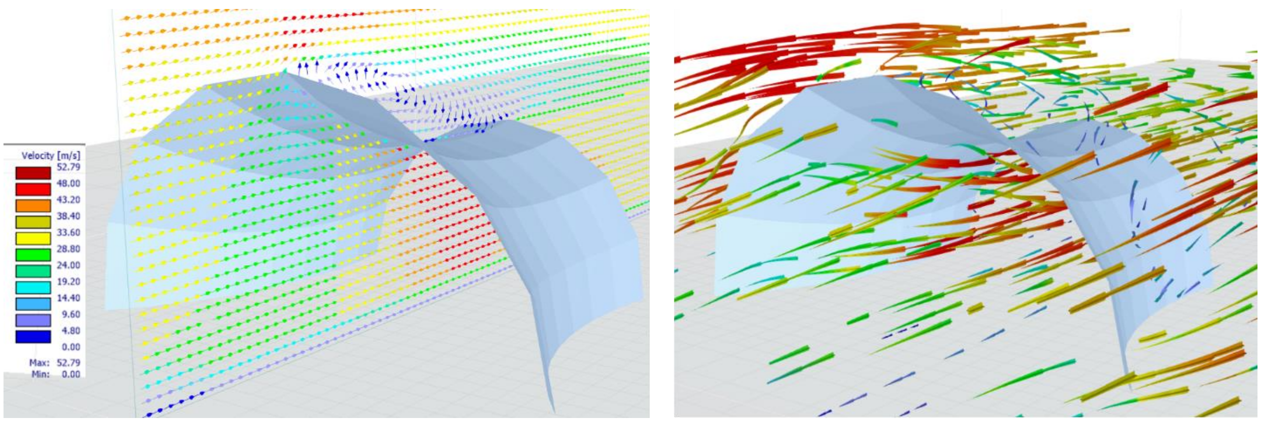


Figure 42. Wind Velocity Vector and Streamline (Wind flows in +Y direction)

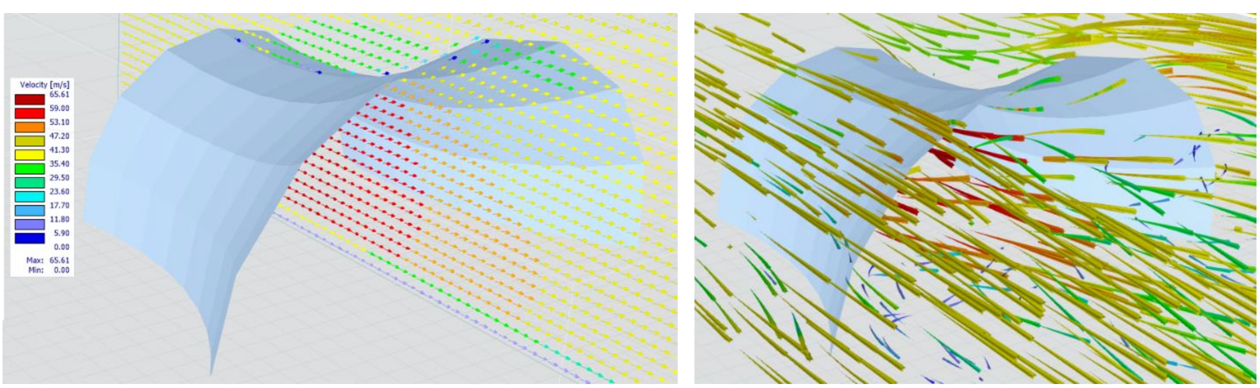


Figure 43. Wind Velocity Vector and Streamline (Wind flows in -Y direction)

As a crucial outcome of the simulation, loads are generated for the RFEM/RSTAB model. These loads are then exported to the corresponding load cases, where they are utilized as

Finite Element (FE) nodal loads or member loads. The following extrapolation process shows how the results on the computational mesh in RWind are transferred to the surface of the original model, often referred to as the 'Original Mesh'.

1. Identifying the Boundary Triangles

The extrapolation procedure commences by identifying the boundary triangles of the mesh situated on the model surface. We search for triangles whose centroid is sufficiently close to the original mesh. Throughout this process, specific criteria are considered, such as ensuring that triangles do not overlap..

2. Extrapolation of Pressure on the Original Model Mesh

Subsequently, for every boundary triangle of the original mesh, we identify an appropriate point on the computational mesh and determine the corresponding pressure value at that location. To locate a point on the computational mesh, we initially establish the normal vector 'n' at the center of the triangle 'S'. Next, we pinpoint a resulting point at the intersection of the normal vector 'n' with the triangle of the computational mesh, as illustrated in the image below. If the resulting point cannot be determined using this method, we resort to selecting the nearest available point in the surrounding area.

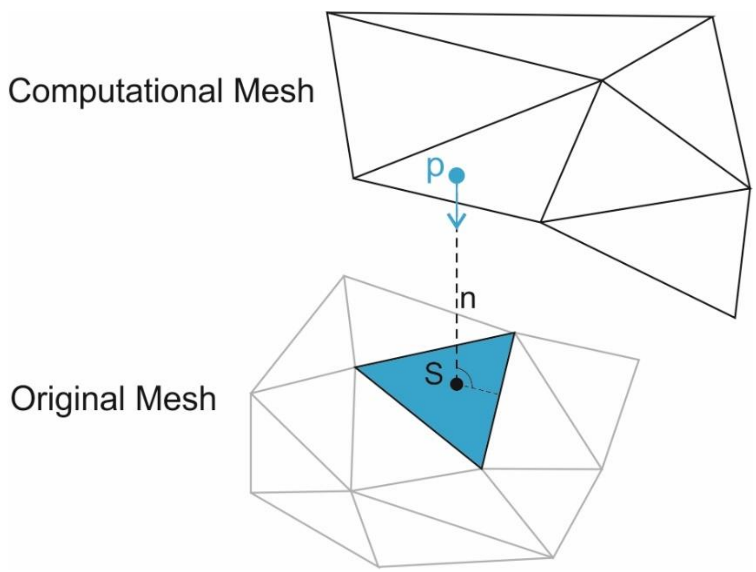


Figure 44. Extrapolation between the computational and original mesh (Online Manuals)

If a suitable point is successfully identified on the computational mesh, the associated pressure value at that point is utilized and extrapolated to encompass the entire triangle of the original mesh, as depicted in the provided image. This process yields the pressure values within each triangle of the original model mesh. On the model's surface, these values are non-zero (positive values indicate pressure, while negative values denote suction), whereas the pressure is uniformly zero across the remainder of the model. Consequently, it becomes feasible to compute the forces exerted at the centers of the triangles, arising from the pressure induced by the surface of these triangles. The direction of these forces aligns with the normal vector to the respective triangle.

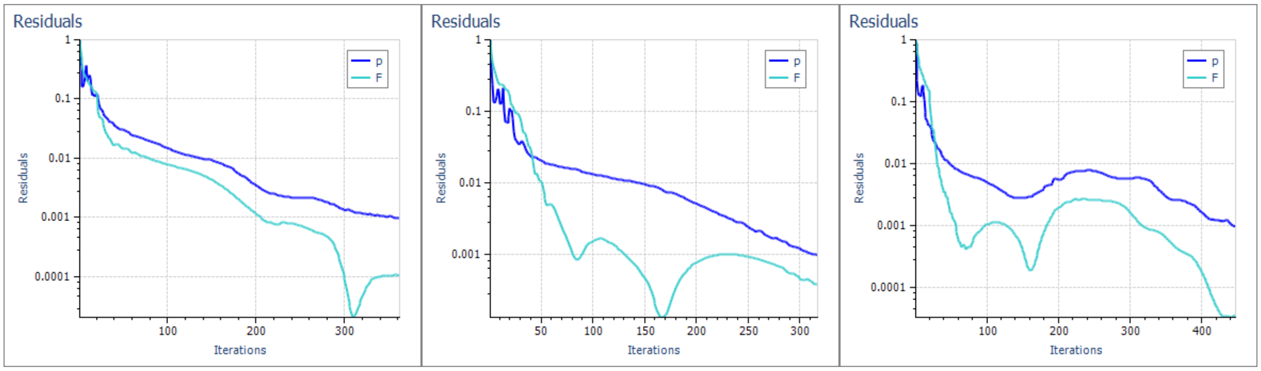


Figure 45. Residual during simulation process (in case of wind direction in +X, +Y and -Y direction respectively from left to right)

The chart depicts the evolution of the residual quantity applied throughout the iterative simulation process. Commencing with an initial residual quantity value, the simulation iteratively refines the residuals, aiming to diminish the imbalance in the finite volume. As the residuals decrease, the solution becomes more precise. In RWIND 2, the numerical solution is deemed accurate when the residual quantity falls below 0.001 within 500 iterations.

6.8.2. Simulation parameters

The wind profiles and parameter set up in the wind simulation is shown below based on Eurocode standard EN 1991 and Hungarian National Annex MSZ 2016-09:

- Simulation type: Steady flow
- Fundamental wind velocity: 23.6 m/s
- Kinematic viscosity: 1.5e-05 m^2/s
- Density: 1.25 kg/m^3
- Finite volume mesh density: 10%

Height z [m]	Wind Velocity v [m/s]	Turbulence Intensity I [%]
0.000	29.29	21.71
6.449	37.40	15.46
12.898	40.35	13.96
19.346	42.06	13.21
25.795	43.28	12.73
32.244	44.22	12.38
38.693	44.98	12.11
45.141	45.63	11.88
51.590	46.19	11.70
58.039	46.68	11.54
64.488	47.12	11.40
70.936	47.52	11.28
77.385	47.88	11.17
83.834	48.22	11.07
90.283	48.53	10.98
96.732	48.81	10.90

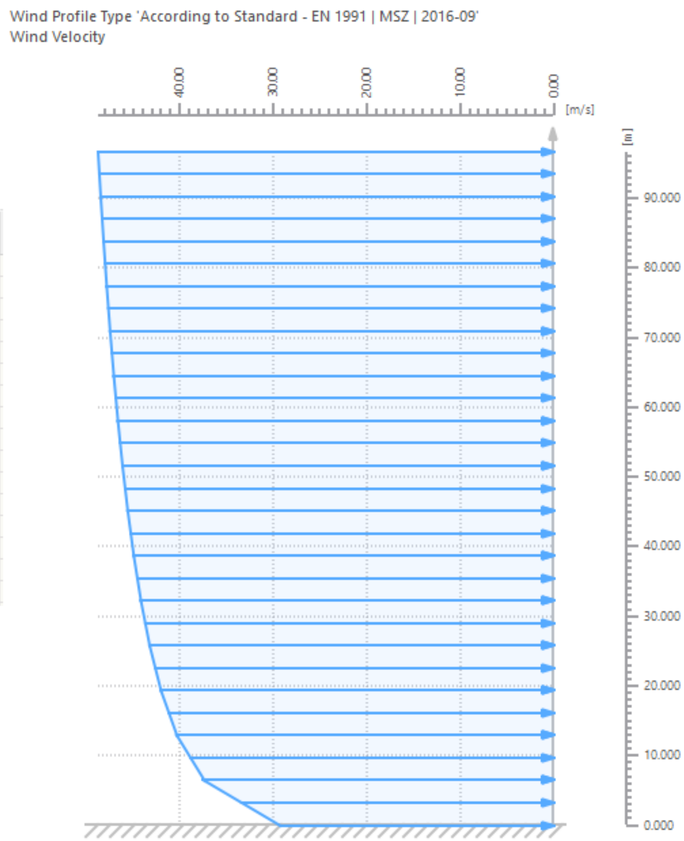


Figure 46. Wind profile in RWind 2

6.9. Analysis of structure

The analysis shows that all structures are sufficient to carry the loads in the Ultimate Limit States. The resistance of all the steel elements (beams, columns) is sufficient in accordance with the design code. The maximum deflections and displacements of the examined structures fit with the prescribed serviceability limits in Service Limits States.

6.9.1. Structure Geometry

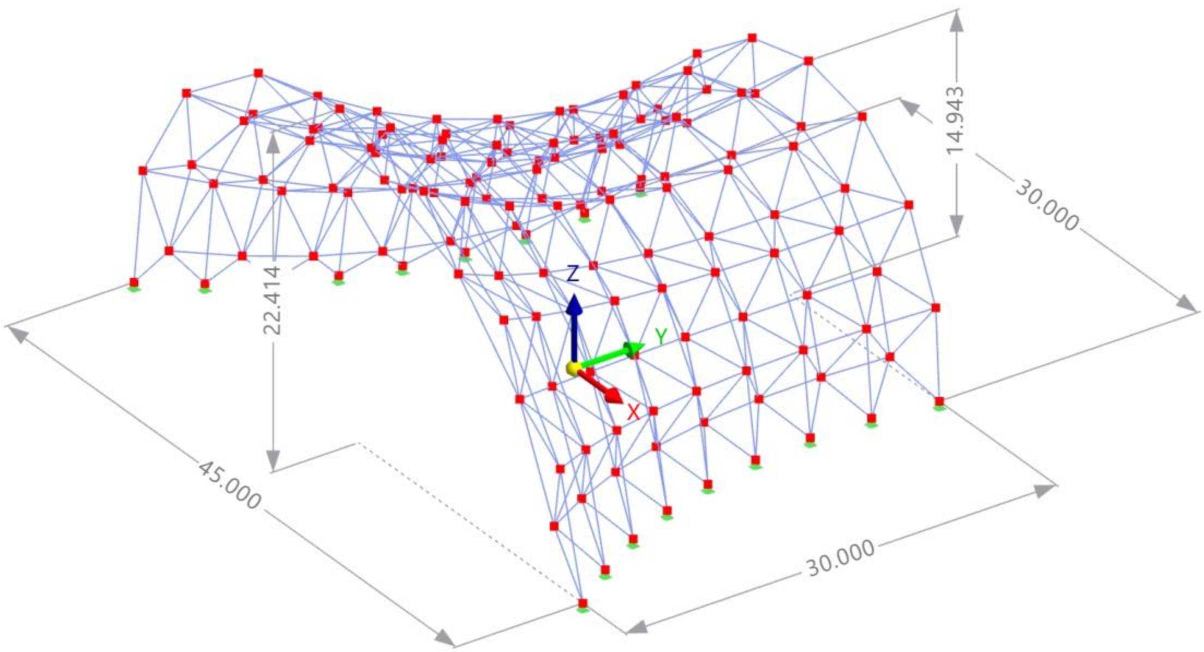


Figure 47. Static model geometry from Rfem 6 and Global coordinate system

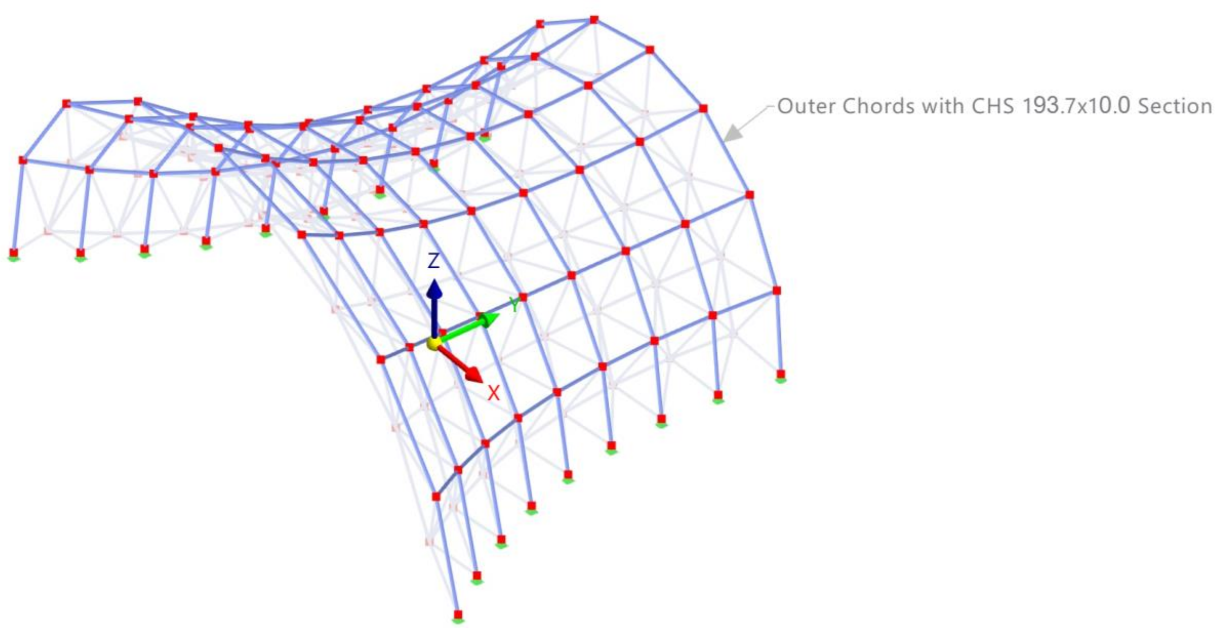


Figure 48. Outer chords of the structure with CHS 177.8x88 section

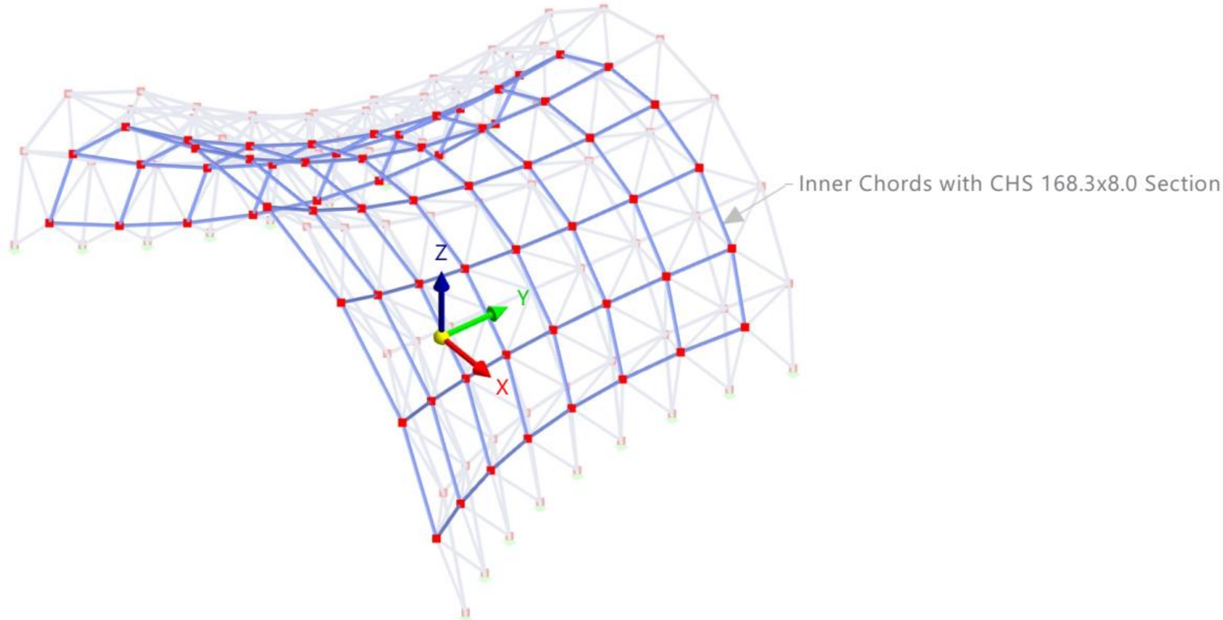


Figure 49. Inner chords of the structure with CHS 168.3x8.0 section

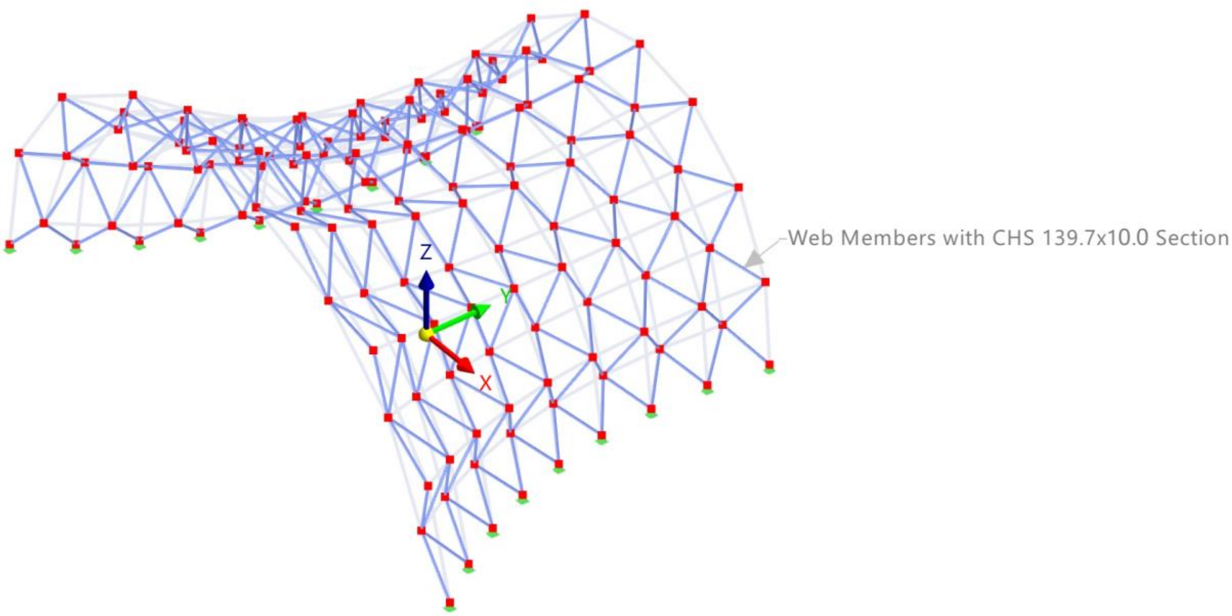


Figure 50. Web members of the structure with CHS 139.7x7.1 section

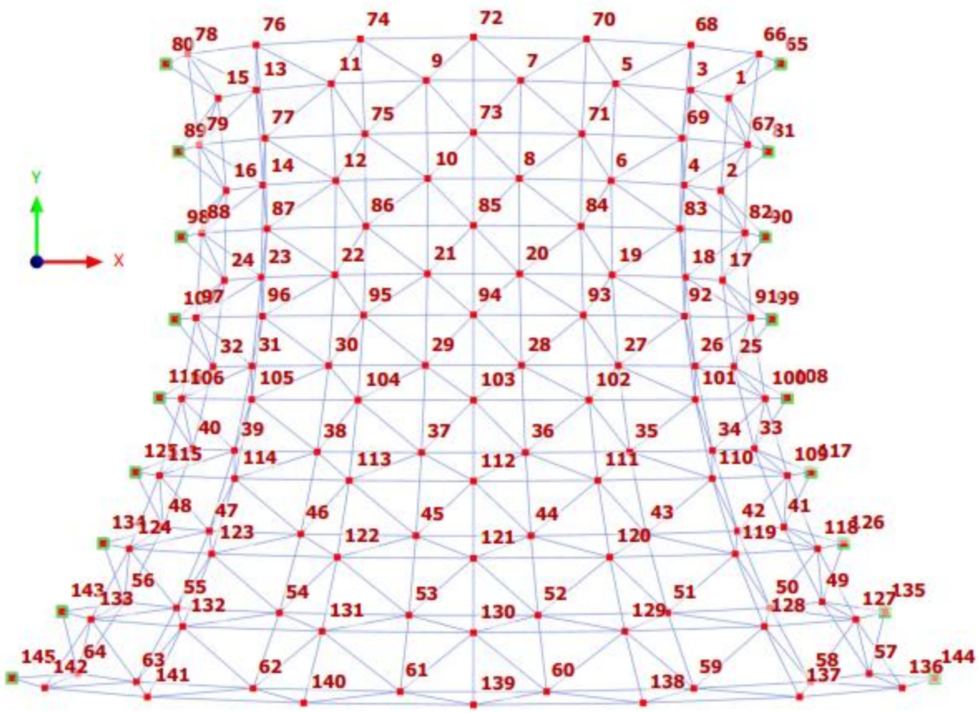


Figure 51. Node numbers

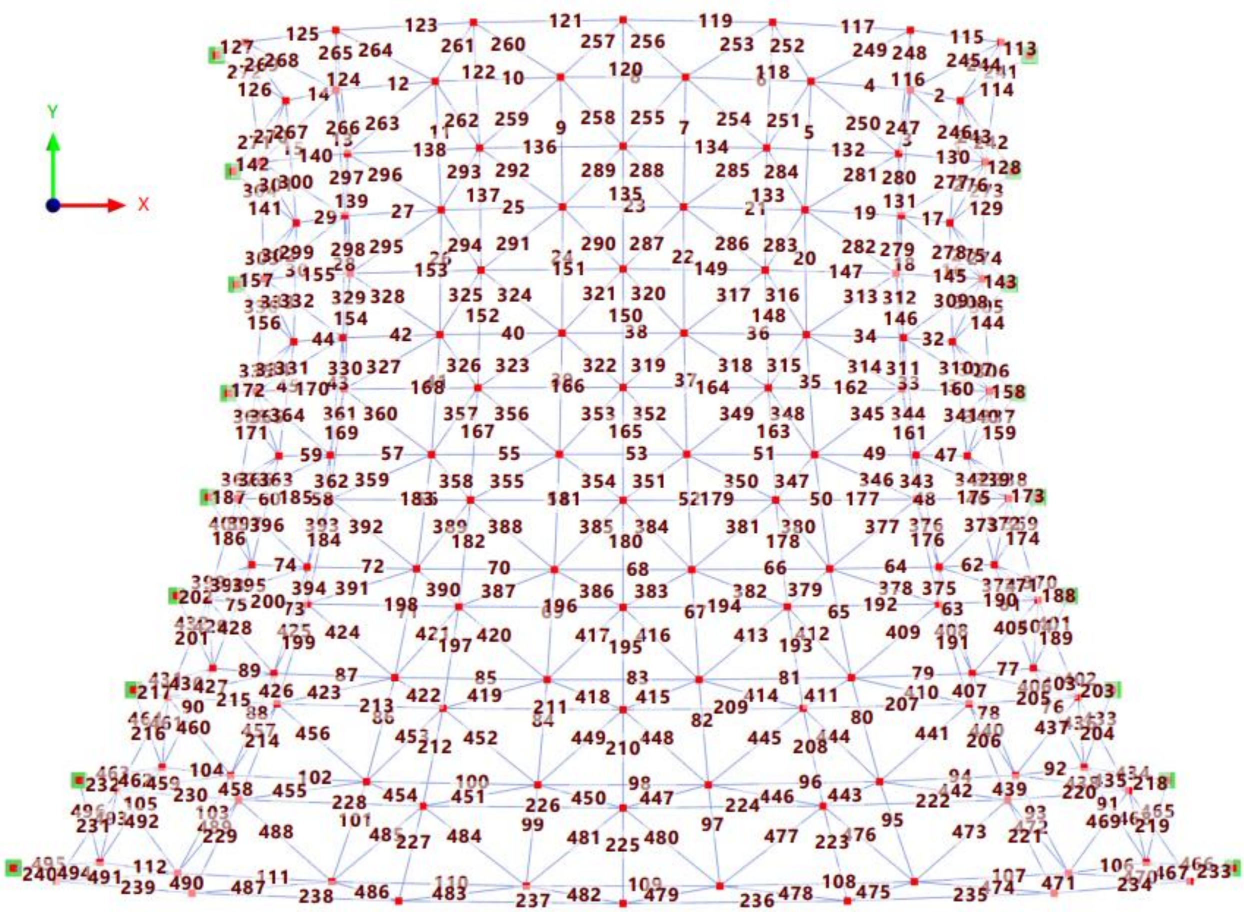


Figure 52. Member numbering

6.9.2. Boundary conditions

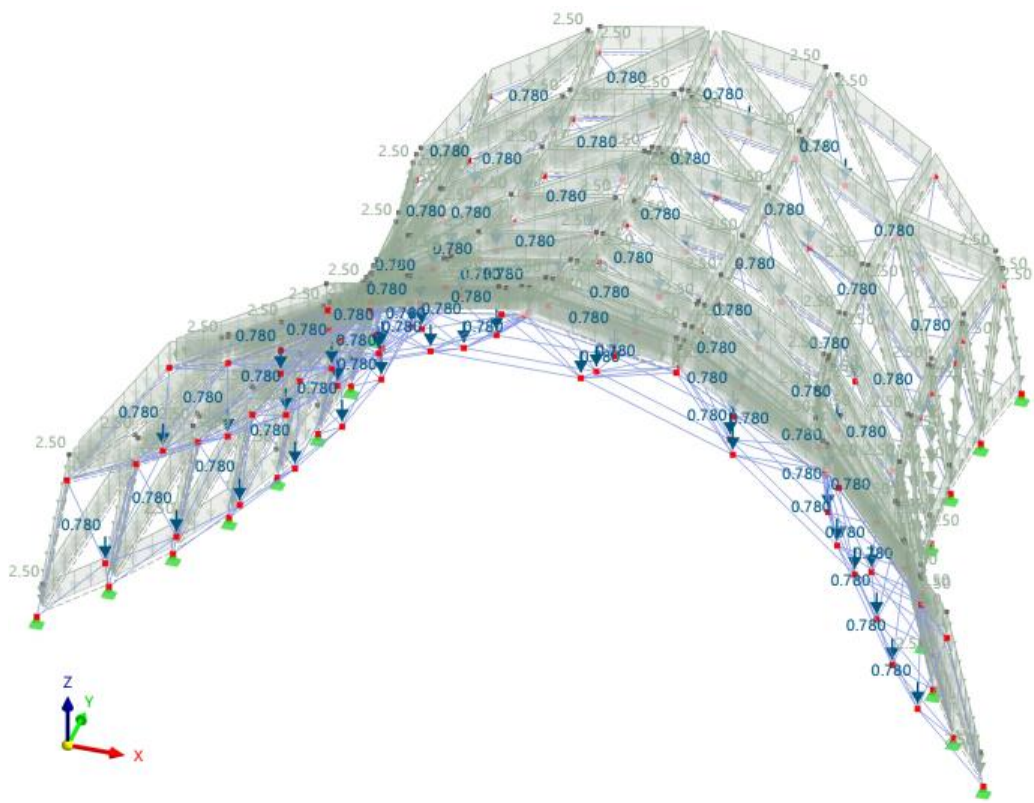
- Members of structure have truss behavior with hinges arranged at the member ends that transfer no moments.

Support No.	Nodes No.	Coordinate System	Translation Spring [kN/m] $C_{u,X}$	Translation Spring [kN/m] $C_{u,Y}$	Translation Spring [kN/m] $C_{u,Z}$	Rotation Spring [kNm/rad] $C_{\varphi,X}$	Rotation Spring [kNm/rad] $C_{\varphi,Y}$	Rotation Spring [kNm/rad] $C_{\varphi,Z}$
2	☑☑☑ ☐☐☑							
	65,80,81,89,90,98,99,107,108,116,117,125,126,134,135,143-145	1 - Global XYZ	☒	☒	☒	☐	☐	☒

Figure 53. Boundary condition of supports

- Translation in X, Y and Z direction and rotation about Z direction in Global coordination system are restrained for those nodes listed in the figure above.

6.9.3. Loads



-

- *Figure 54. Load case 1- Permanent actions*

Permanent actions:

- Selfweight of structural members
- Weight of UHPC claddings: 2.5kN/m²
- Suspended loads from equipments on the inner nodes: 5 ton distributed on 64 nodes means 0.78kN per node.

Snow loads: (for the persistent/ Transient design situations)

$s_k := 1.25 \ \frac{kN}{m^2}$ Characteristic value of snow load on the ground

$C_e := 1$ Exposure coefficient

$C_t := 1$ Thermal coefficient

$\mu_3 := 2$

Case I: $s := 0.8 \cdot C_e \cdot C_t \cdot s_k = 1 \ \frac{kN}{m^2}$

Case II: Distributed loads magnitude:

$S1 := \frac{1.}{2} \cdot 0.5 \cdot \mu_3 \cdot C_e \cdot C_t \cdot s_k = 0.625 \ \frac{kN}{m^2}$

$S2 := \frac{1.}{2} \cdot \mu_3 \cdot C_e \cdot C_t \cdot s_k = 1.25 \ \frac{kN}{m^2}$

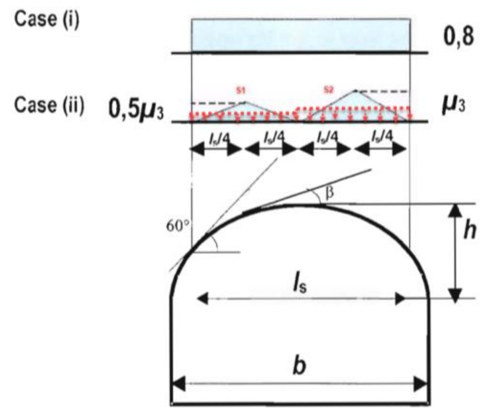


Figure 55. Snow load calculation in Mathcad and it's shape coefficients for cylindrical roof (EN 1991-1-3:2003)

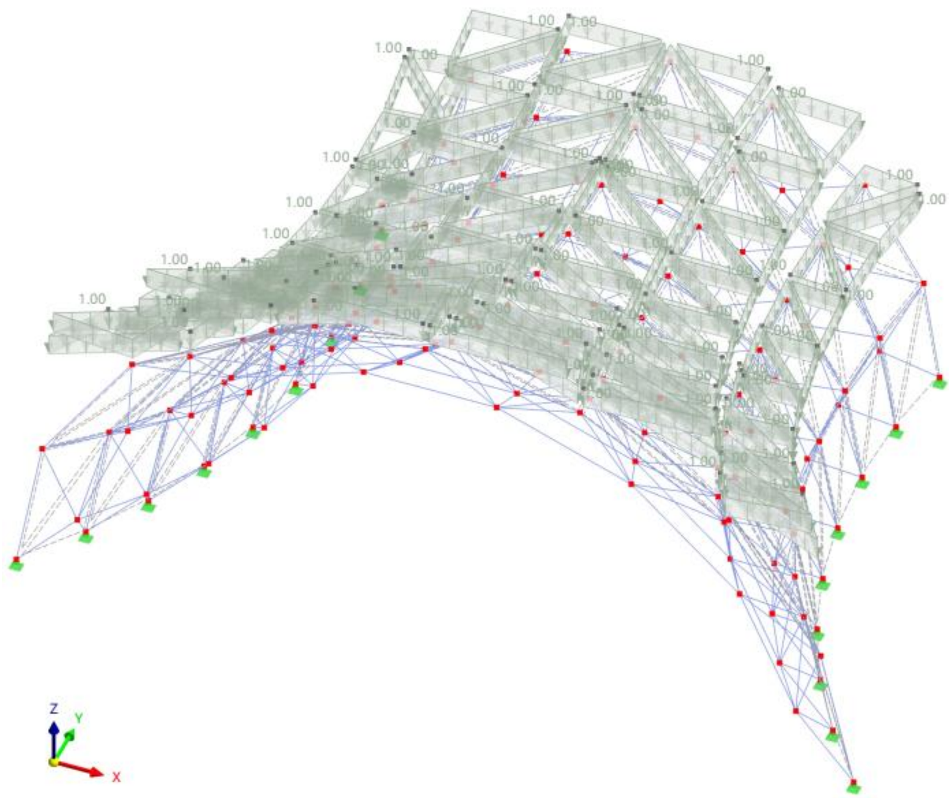


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Figure 56. Load case 2 - Snow loads 1 – H<= 1000m

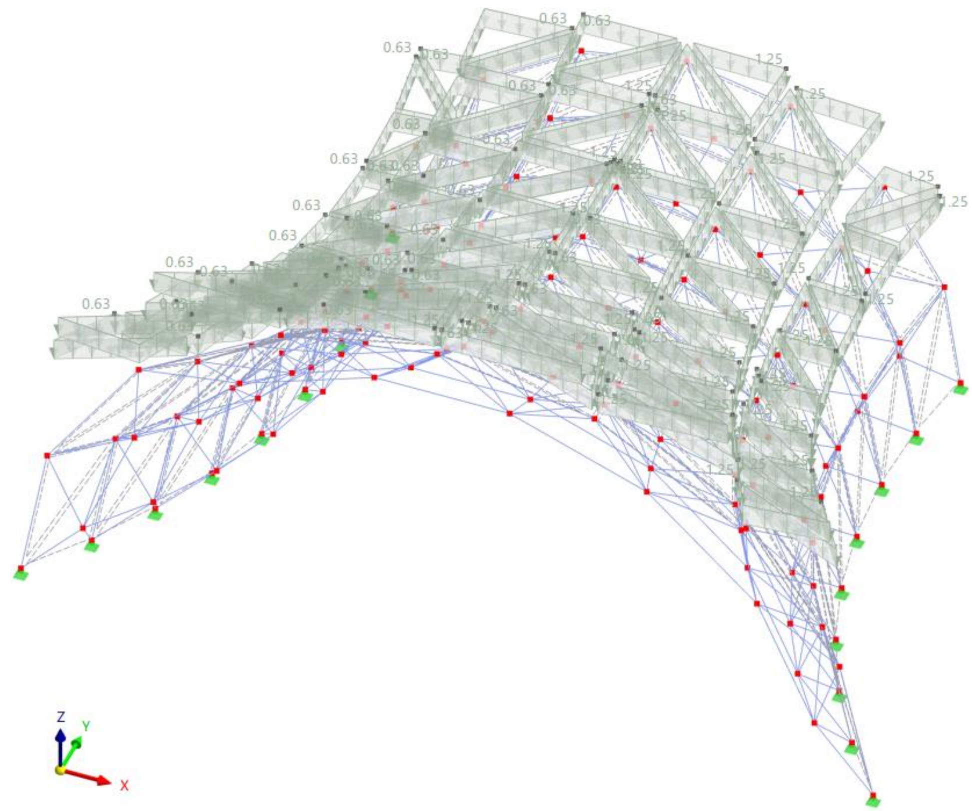


Figure 57. Load case 3 - Snow loads 2 – H<= 1000m

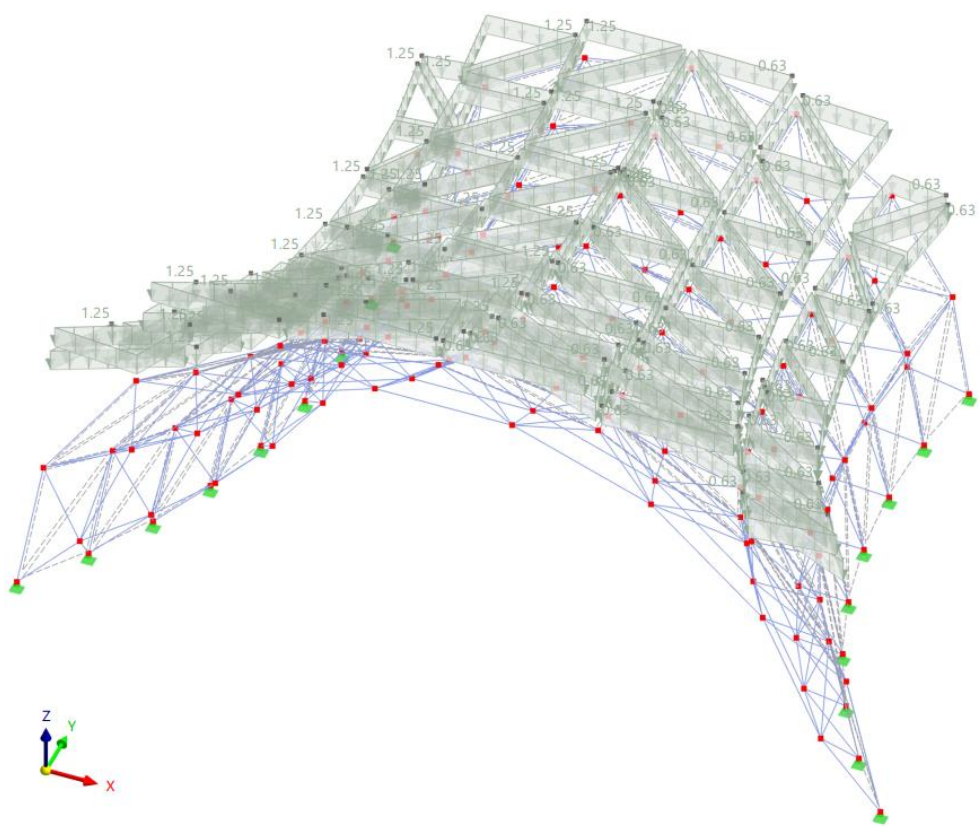


Figure 58. Load case 4 - Snow loads 3 – H<= 1000m

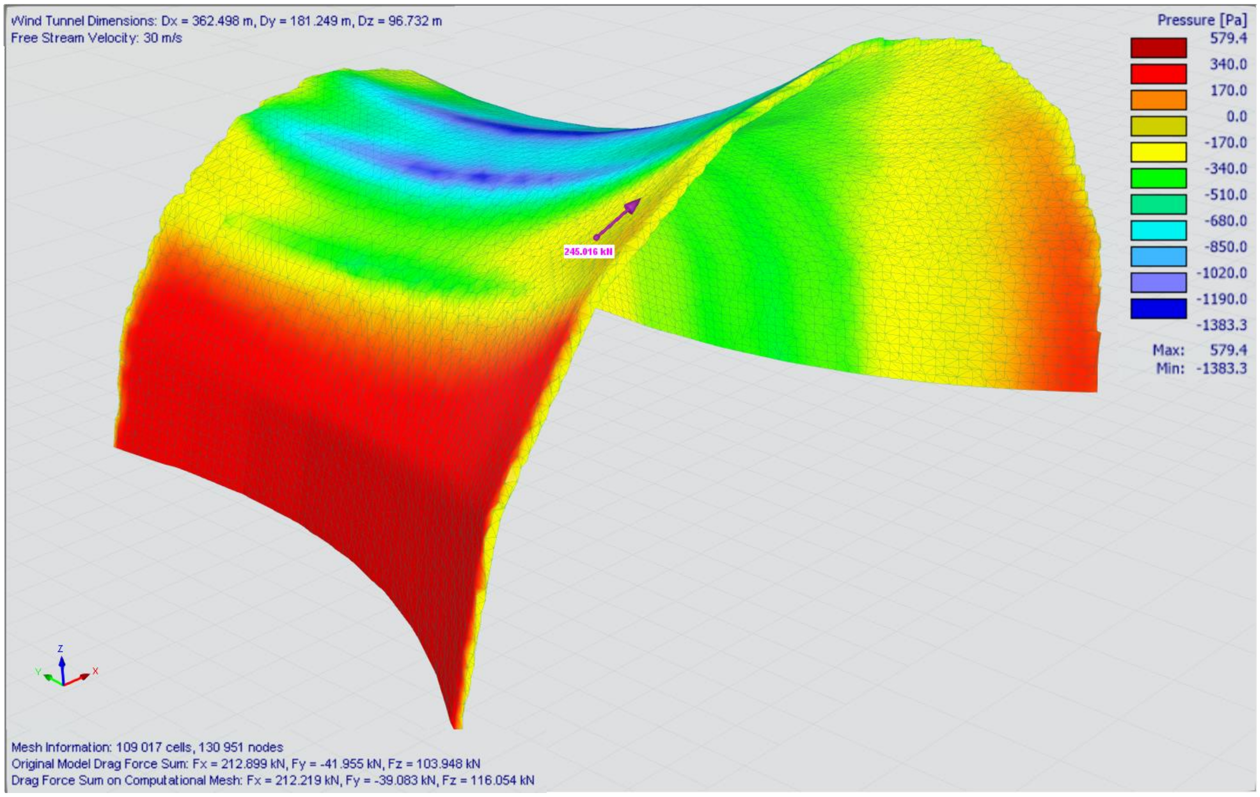


Figure 59. Load case 5 – Wind load in+X direction in Rwind 2 and in +X direction in RFem 6 (Surface pressure in the computational mesh)

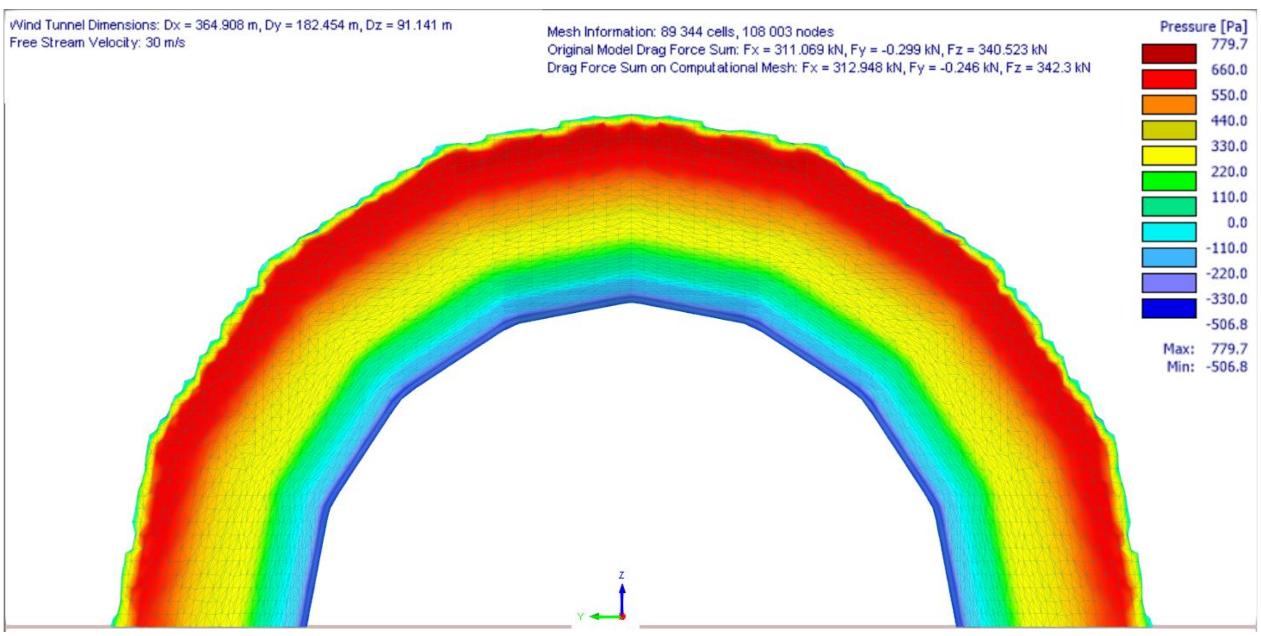


Figure 60. Load case 6 – Wind load in+X direction in Rwind 2 but in +Y direction in RFem 6 (Surface pressure in the computational mesh)- Front view

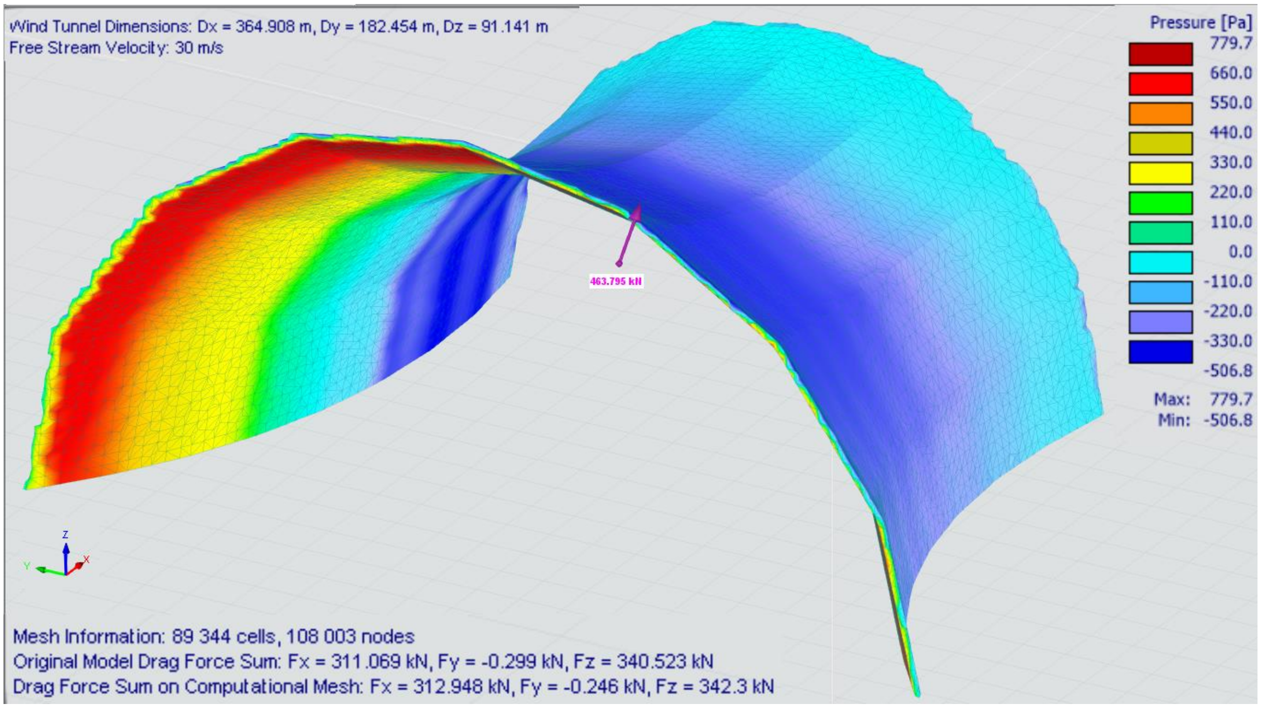


Figure 61. Load case 6 – Wind load in+X direction in Rwind 2 but in +Y direction in RFem 6 (Surface pressure in the computational mesh)- Perspective view

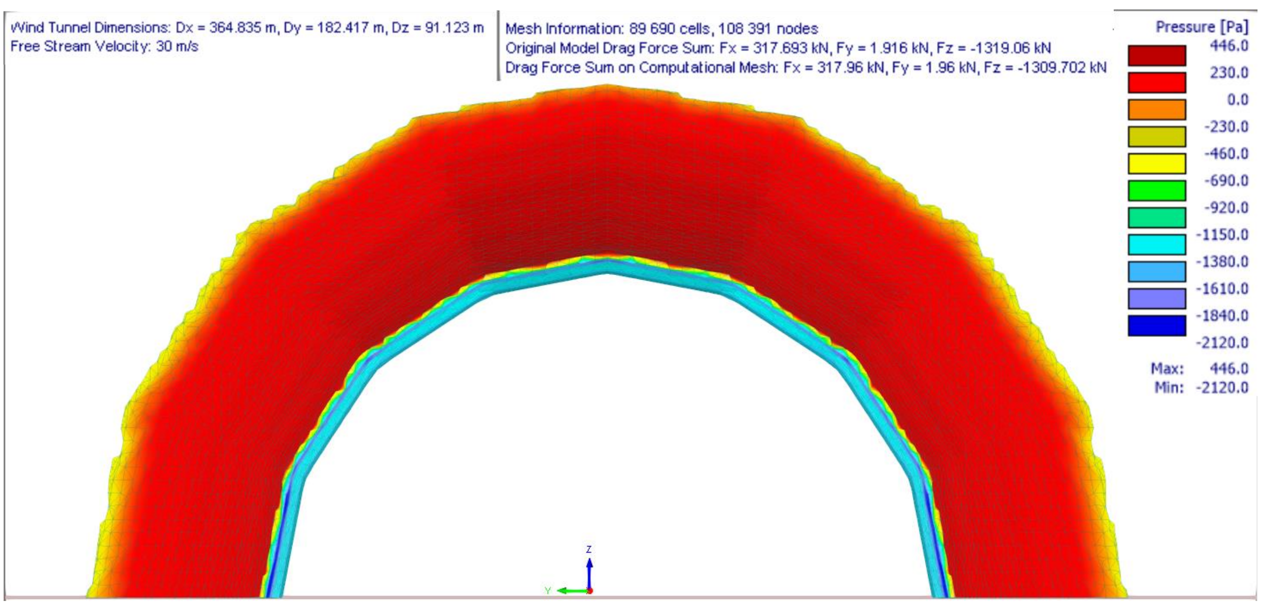


Figure 62. Load case 7 – Wind load in+X direction in Rwind 2 but in -Y direction in RFem 6 (Surface pressure in the computational mesh)- Back view

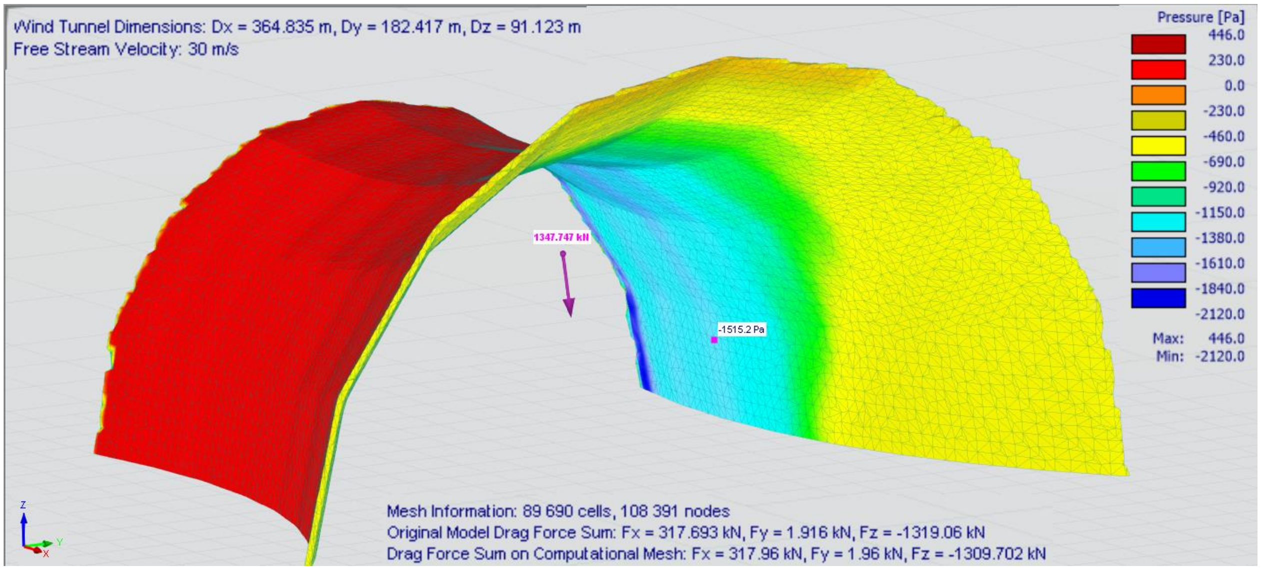


Figure 63. Load case 7 – Wind load in+X direction in Rwind 2 but in -Y direction in RFem 6 (Surface pressure in the computational mesh)- Perspective view

Temperature loads:

- Temperature load is applied according to MSZ EN 1991-1-5:2005:

Operating temperature	Structural calculation
Minimum T_{min}	-15
Maximum T_{max}	53
Reference T_{ref}	10
$\Delta T1 = T_{max}-T_{ref}$	43
$\Delta T2 = T_{min}-T_{ref}$	-25

Table 3. Temperature load

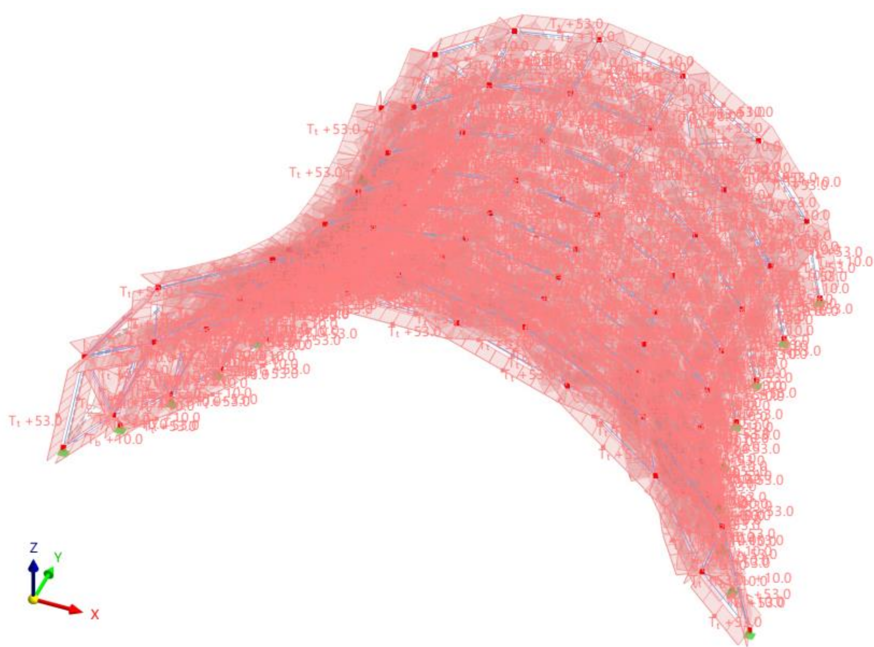


Figure 64. Load case 8 – Temperature loads (T=+53°C, T_{base}=10°C)

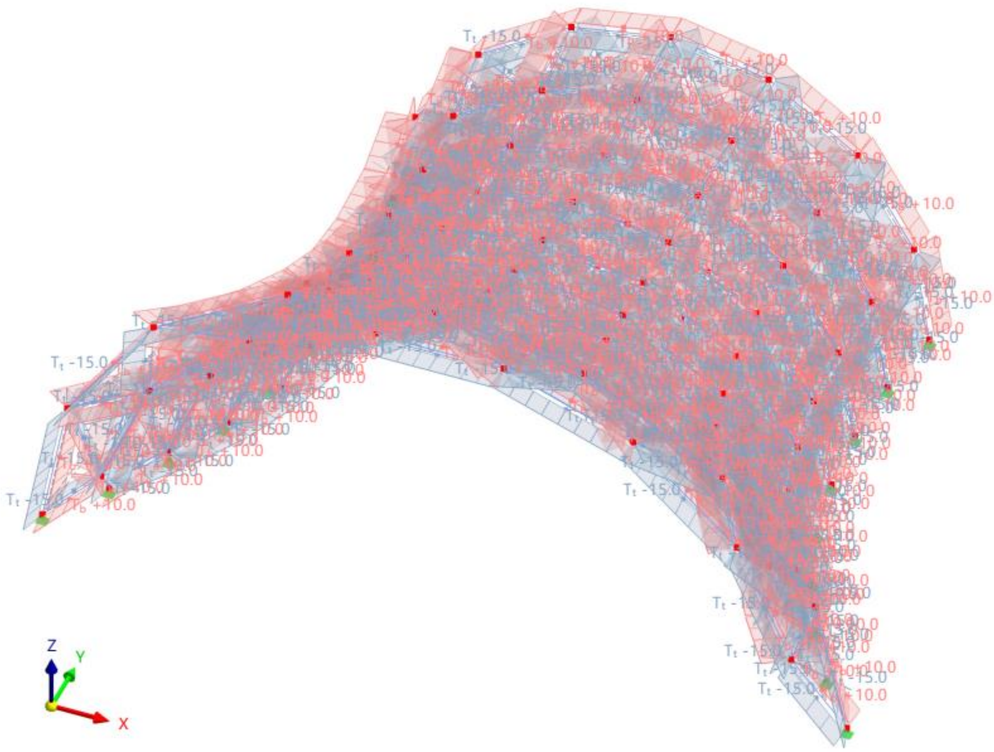


Figure 65. Load case 9 – Temperature loads (T=-15°C, T_{base}=10°C)

6.9.4. Load factors

The permanent and variable loads' factors are defined as follows:

	Type	g_G	g_Q
a	Permanent actions only	1.35	-
b	Combination of permanent and variable actions	1.35	1.5

Table 4. load factors

The detail of load case combinations are shown in the Appendix A: Load Case Combinations (LCC)

6.9.5. Modal analysis

6.9.5.1. Modal analysis settings

- Modal analysis is conducted with acting masses in X-direction and Y direction.
- Number of modes: 45

MODAL ANALYSIS SETTINGS - NEGLECT MASSES

MA No.	Object Type	List	Components in Direction u_X	u_Y	u_Z	Components About Axis φ_X	φ_Y	φ_Z	Comment
3	Node with support	65,80,81,89,90,98,99,107,108,116,117,125,126,134,135,143-145	☒	☒	☒	☒	☒	☒	

Table 5. Modal analysis settings – Neglect masses

Masses at the support node will be neglected in the modal analysis because the pinned support does not have translation in three direction. As the result, the total effective modal mass in X and Y direction can reach 90%.

6.9.5.2. Modal analysis results

1.1 **NATURAL FREQUENCIES** **Modal Analysis**

Mode No.	Eigenvalue λ [1/s²]	Angular Frequency ω [rad/s]	Natural Frequency f [Hz]	Natural Period T [s]
	AE LC10 - Modal Analysis			
1	181.716	13.480	2.145	0.466
2	393.168	19.828	3.156	0.317
3	580.561	24.095	3.835	0.261
4	1011.759	31.808	5.062	0.198
5	1496.263	38.682	6.156	0.162
6	1565.854	39.571	6.298	0.159
7	1581.692	39.770	6.330	0.158
8	1602.691	40.034	6.372	0.157
9	1749.314	41.825	6.657	0.150
10	1994.313	44.658	7.107	0.141
11	1999.848	44.720	7.117	0.141
12	2024.598	44.996	7.161	0.140
13	2122.422	46.070	7.332	0.136
14	2175.366	46.641	7.423	0.135
15	2185.212	46.746	7.440	0.134
16	2221.379	47.132	7.501	0.133
17	2271.800	47.663	7.586	0.132
18	2288.168	47.835	7.613	0.131
19	2298.309	47.941	7.630	0.131
20	2338.610	48.359	7.697	0.130
21	2339.164	48.365	7.698	0.130
22	2355.941	48.538	7.725	0.129
23	2356.827	48.547	7.727	0.129
24	2433.039	49.326	7.850	0.127
25	2441.668	49.413	7.864	0.127
26	2499.632	49.996	7.957	0.126
27	2518.192	50.182	7.987	0.125
28	2543.245	50.431	8.026	0.125
29	2572.453	50.719	8.072	0.124
30	2744.402	52.387	8.338	0.120
31	2807.155	52.983	8.432	0.119
32	3190.332	56.483	8.990	0.111
33	3285.972	57.323	9.123	0.110
34	3306.892	57.506	9.152	0.109
35	3384.313	58.175	9.259	0.108
36	3400.364	58.313	9.281	0.108
37	3444.247	58.688	9.340	0.107
38	3447.501	58.715	9.345	0.107
39	3495.625	59.124	9.410	0.106
40	3543.393	59.526	9.474	0.106
41	3585.799	59.882	9.530	0.105
42	3589.670	59.914	9.536	0.105
43	3638.165	60.317	9.600	0.104
44	3714.198	60.944	9.700	0.103
45	3792.825	61.586	9.802	0.102

Table 6. Natural Frequencies

In the first model shape, the value of natural frequencies are following:

- Eigenvalue: 181.716 $1/s^2$
- Angular frequency: 13.48 rad/s
- Natural frequency: 2.145 Hz
- Natural period: 0.466 s

1.1 **EFFECTIVE MODAL MASSES** **Modal Analysis**

Mode	Modal Mass	Transl. Eff. Modal Mass [kg]			Rotat. Eff. Modal Mass [kgm²]			Transl. Eff. Modal Mass Factor [--]			Rotat. Eff. Modal Mass Factor [--]		
No.	M_i [kg]	m_{eX}	m_{eY}	m_{eZ}	$m_{e\varphi X}$	$m_{e\varphi Y}$	$m_{e\varphi Z}$	f_{meX}	f_{meY}	f_{meZ}	$f_{m\varphi X}$	$f_{m\varphi Y}$	$f_{m\varphi Z}$
	LC10 - Modal Analysis												
1	117595.9	300911.0	0.0	0.0	0.00	610010.00	30290100.00	0.639	0.000	0.000	0.000	0.060	0.300
2	127778.7	149281.0	0.0	0.0	0.00	327642.00	54330800.00	0.317	0.000	0.000	0.000	0.032	0.537
3	46744.3	0.0	327686.0	0.0	2321560.00	0.00	0.00	0.000	0.696	0.000	0.229	0.000	0.000
4	8392.5	0.0	83764.3	0.0	816234.00	0.00	0.00	0.000	0.178	0.000	0.080	0.000	0.000
5	1179.7	723.0	0.0	0.0	0.00	1.42	833551.00	0.002	0.000	0.000	0.000	0.000	0.008
6	853.8	19.4	0.0	0.0	0.00	5888.05	135189.00	0.000	0.000	0.000	0.000	0.001	0.001
7	422.5	0.0	1104.9	0.0	28177.50	0.00	0.00	0.000	0.002	0.000	0.003	0.000	0.000
8	435.4	0.0	2266.0	0.0	43846.50	0.00	0.00	0.000	0.005	0.000	0.004	0.000	0.000
9	1660.4	906.5	0.0	0.0	0.00	15628.60	619119.00	0.002	0.000	0.000	0.000	0.002	0.006
10	370.4	169.3	0.0	0.0	0.00	24984.10	324911.00	0.000	0.000	0.000	0.000	0.002	0.003
11	256.5	0.0	312.0	0.0	6050.80	0.00	0.00	0.000	0.001	0.000	0.001	0.000	0.000
12	901.1	46.7	0.0	0.0	0.00	141482.00	1254860.00	0.000	0.000	0.000	0.000	0.014	0.012
13	475.4	0.0	8.9	0.0	3716.19	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000
14	419.0	0.0	3.3	0.0	5271.80	0.00	0.00	0.000	0.000	0.000	0.001	0.000	0.000
15	567.3	41.6	0.0	0.0	0.00	4795.78	288705.00	0.000	0.000	0.000	0.000	0.000	0.003
16	810.3	208.1	0.0	0.0	0.00	99525.20	150246.00	0.000	0.000	0.000	0.000	0.010	0.001
17	384.8	0.0	241.0	0.0	9351.89	0.00	0.00	0.000	0.001	0.000	0.001	0.000	0.000
18	556.3	42.2	0.0	0.0	0.00	4808.64	13646.40	0.000	0.000	0.000	0.000	0.000	0.000
19	314.7	0.0	1497.3	0.0	91798.30	0.00	0.00	0.000	0.003	0.000	0.009	0.000	0.000
20	278.4	0.0	12.0	0.0	644.30	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000
21	504.7	1.4	0.0	0.0	0.00	1417.36	22344.60	0.000	0.000	0.000	0.000	0.000	0.000
22	282.2	0.0	163.7	0.0	1238.27	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000
23	381.6	21.9	0.0	0.0	0.00	14678.40	117482.00	0.000	0.000	0.000	0.000	0.001	0.001
24	1604.7	0.0	1007.0	0.0	56603.60	0.00	0.00	0.000	0.002	0.000	0.006	0.000	0.000
25	1660.6	33.8	0.0	0.0	0.00	22074.10	193273.00	0.000	0.000	0.000	0.000	0.002	0.002
26	374.6	0.0	338.0	0.0	20661.70	0.00	0.00	0.000	0.001	0.000	0.002	0.000	0.000
27	567.1	93.1	0.0	0.0	0.00	35834.40	120476.00	0.000	0.000	0.000	0.000	0.004	0.001
28	488.0	0.0	18.2	0.0	4131.15	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000
29	877.3	72.7	0.0	0.0	0.00	91983.50	303830.00	0.000	0.000	0.000	0.000	0.009	0.003
30	217.2	0.0	436.6	0.0	21091.00	0.00	0.00	0.000	0.001	0.000	0.002	0.000	0.000
31	261.5	1.1	0.0	0.0	0.00	261.15	139187.00	0.000	0.000	0.000	0.000	0.000	0.001
32	115.7	0.0	41.5	0.0	5813.15	0.00	0.00	0.000	0.000	0.000	0.001	0.000	0.000
33	263.9	298.1	0.0	0.0	0.00	96958.40	597819.00	0.001	0.000	0.000	0.000	0.010	0.006
34	227.1	0.0	4.7	0.0	3.57	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000
35	1049.0	836.7	0.0	0.0	0.00	160233.00	1881130.00	0.002	0.000	0.000	0.000	0.016	0.019
36	459.1	0.0	2.4	0.0	1897.77	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000
37	400.2	145.9	0.0	0.0	0.00	1823.15	53322.20	0.000	0.000	0.000	0.000	0.000	0.001
38	433.9	0.0	535.5	0.0	89987.60	0.00	0.00	0.000	0.001	0.000	0.009	0.000	0.000
39	593.5	325.7	0.0	0.0	0.00	185904.00	823634.00	0.001	0.000	0.000	0.000	0.018	0.008
40	599.4	0.0	1867.6	0.0	49582.80	0.00	0.00	0.000	0.004	0.000	0.005	0.000	0.000
41	342.1	194.3	0.0	0.0	0.00	60535.90	25526.60	0.000	0.000	0.000	0.000	0.006	0.000
42	414.5	0.0	1253.2	0.0	71903.70	0.00	0.00	0.000	0.003	0.000	0.007	0.000	0.000
43	320.4	115.8	0.0	0.0	0.00	200406.00	354963.00	0.000	0.000	0.000	0.000	0.020	0.004
44	1307.3	0.0	1374.6	0.0	11760.00	0.00	0.00	0.000	0.003	0.000	0.001	0.000	0.000
45	593.8	0.0	2675.3	0.0	223348.00	0.00	0.00	0.000	0.006	0.000	0.022	0.000	0.000
Σ	324736.5	454490.0	426614.0	0.0	3884680.00	2106870.00	92874100.00	0.965	0.906	0.000	0.382	0.207	0.919
Σ_M		470769.0	470769.0	0.0	10159800.00	10159800.00	1.01e+08						
%		96.54 %	90.62 %		38.24 %	20.74 %	91.86 %						

Table 7. Effective Modal Masses

- Total translational effective modal mass in x direction accounts for 96,54 % of total mass of structure while 90.62% is the proportion of total translational effective modal mass in y direction.

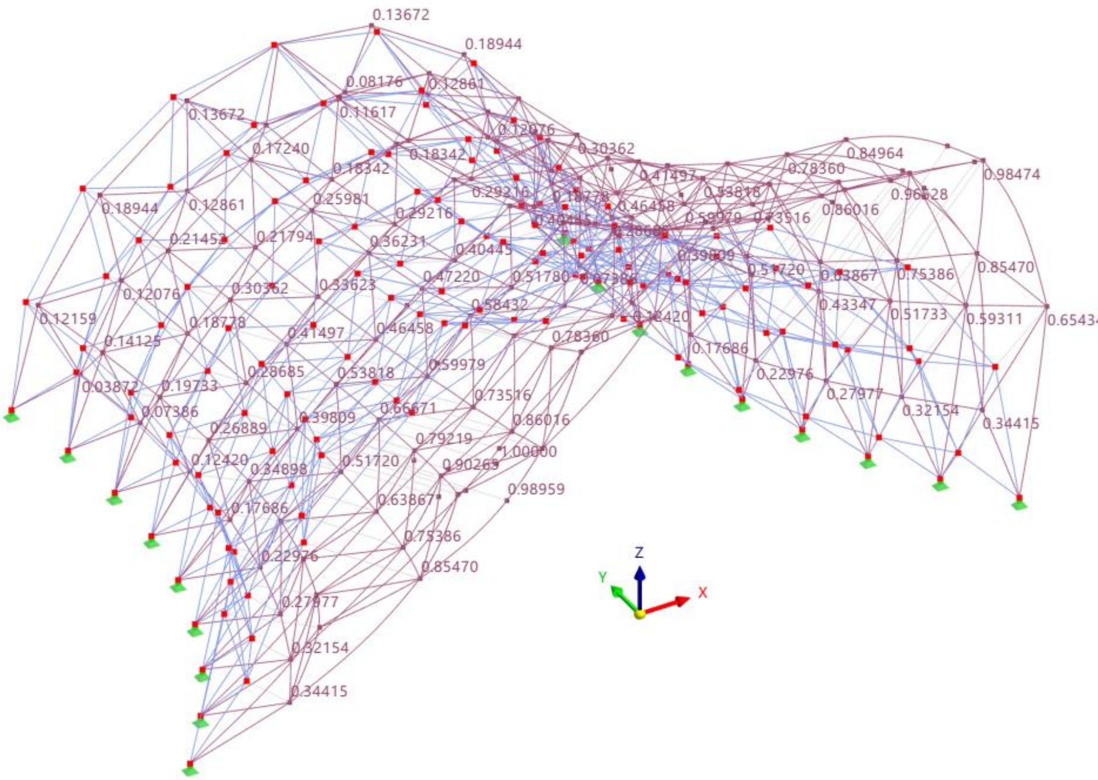


Figure 66. Mode shape 1

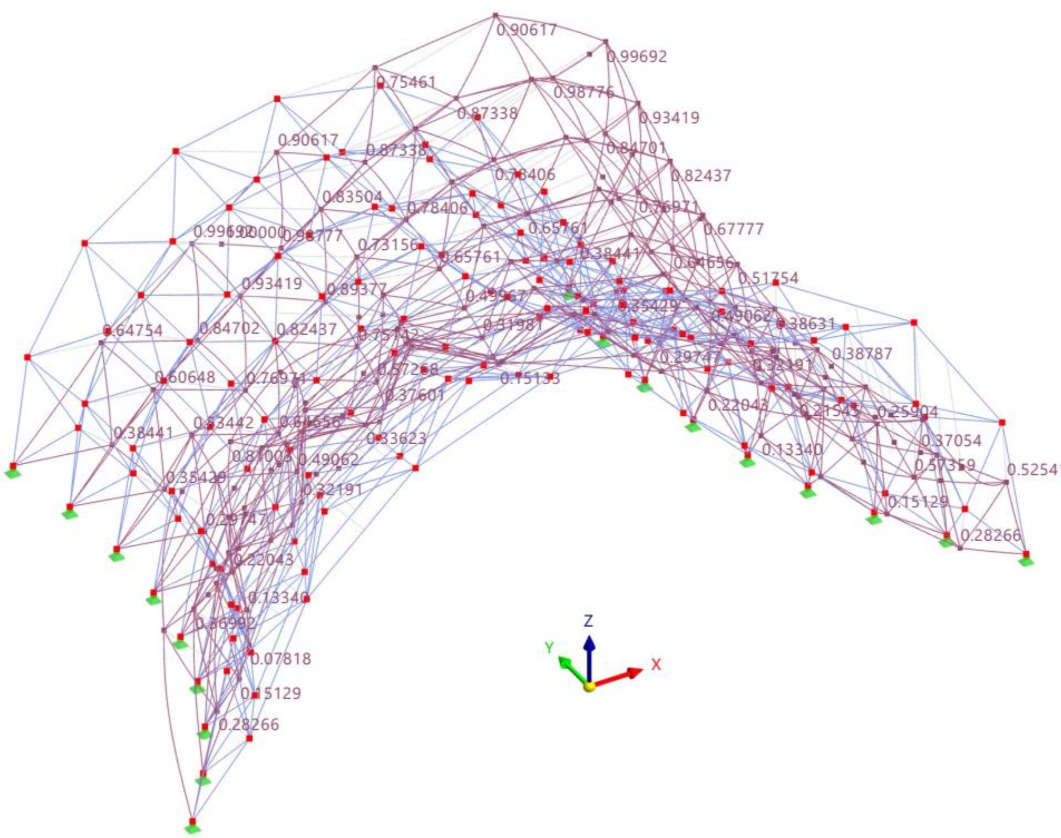


Figure 67. Mode shape 2

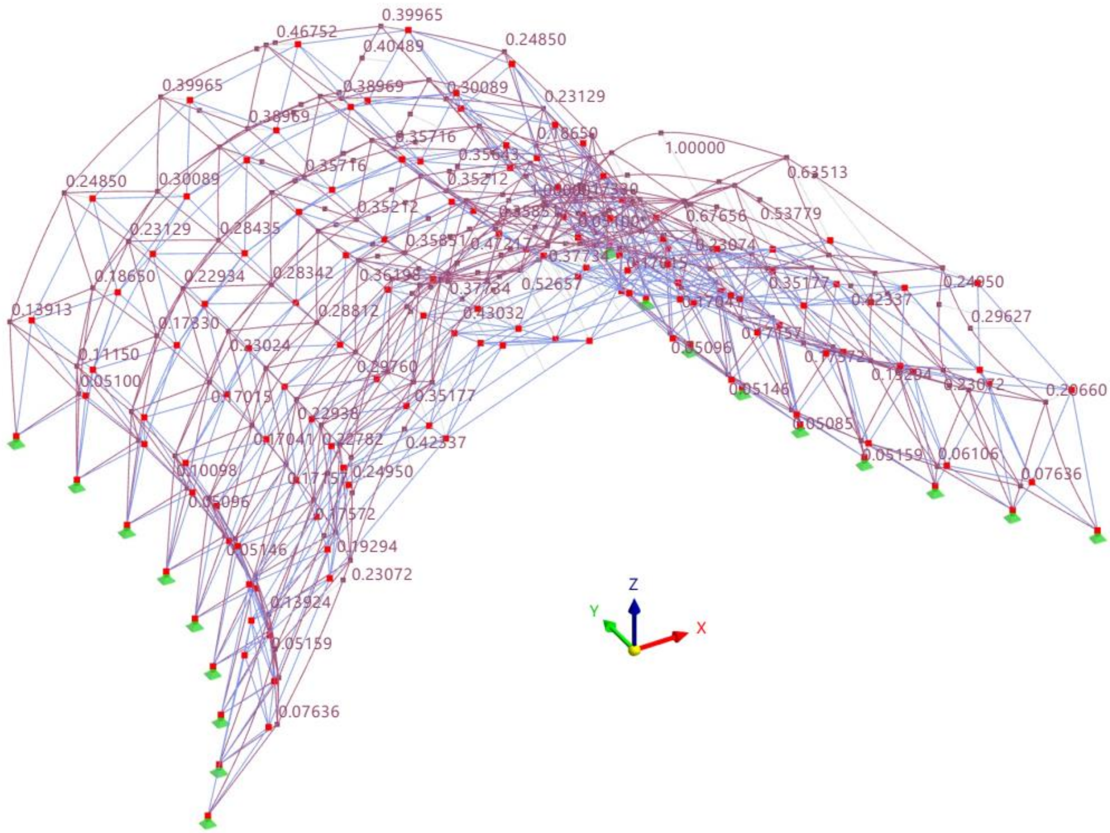


Figure 68. Mode shape 3

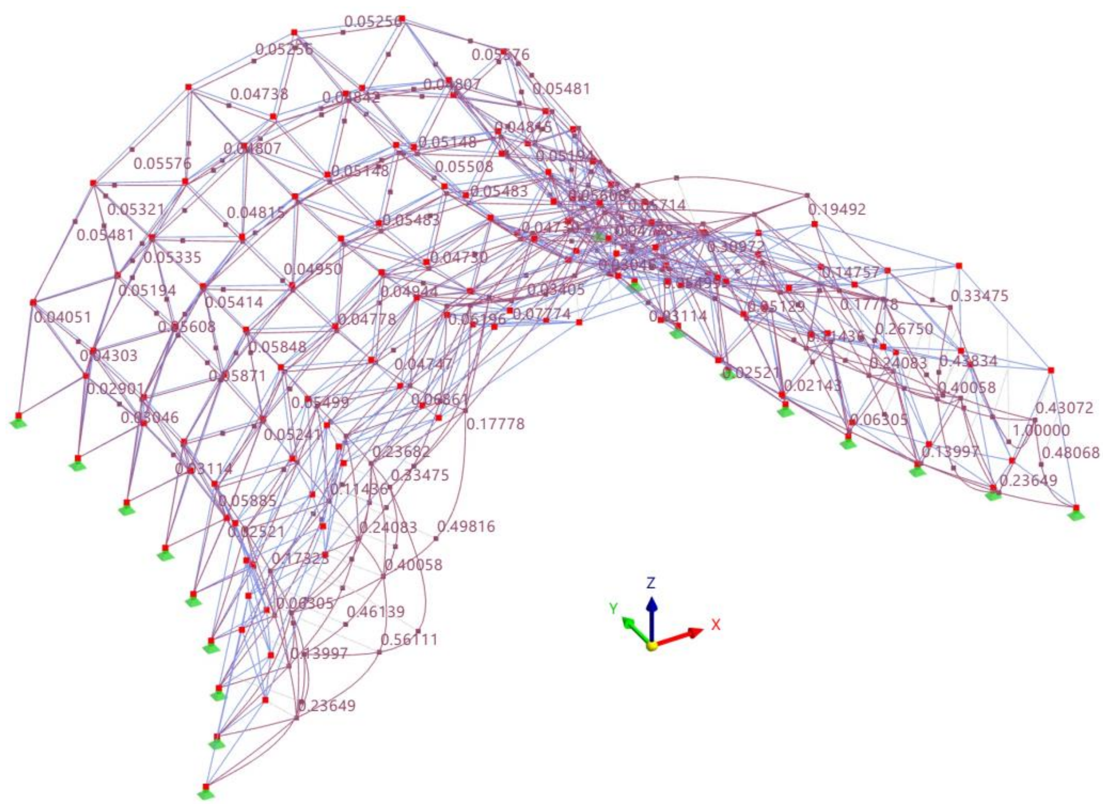


Figure 69. Mode shape 4

6.9.6. Earthquake Loads Settings

6.9.6.1. Response Spectra

Response spectra is defined using EN 1998-1 and MSZ 2013-07 standards. It's Parameters are given in the table below:

According to standard - EN 1998 -1|MSZ|2013-07	
Spectrum shape	Design Spectrum
Spectrum direction	Horizontal
Ground Type	C
Earthquake action	
Seismic zone	4
Importance class	Class II
Design ground acceleration	$a_g = 1.37$
Factors	
Behavior factor q	1.5
Limit value β	0.2

Table 8. Response Spectra Parameters

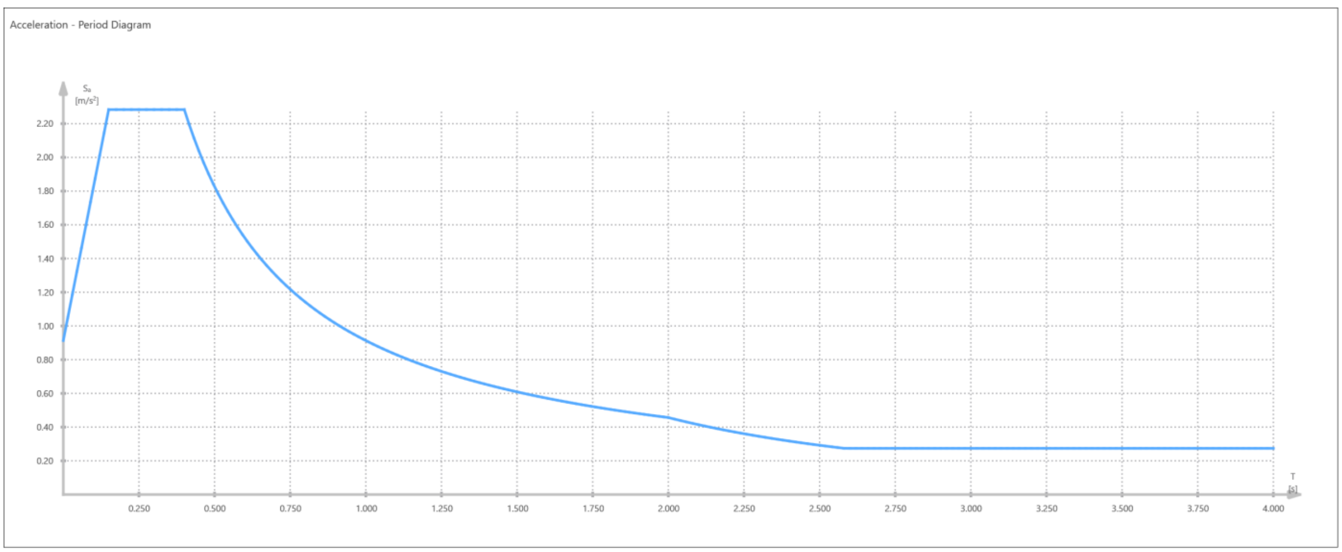


Figure 70. Acceleration – Period Diagram

6.9.6.2. Spectral Analysis Setting

- Equivalent linear combination is used
- According to EN 1998-1 4.3.3.5.1, SRSS Scacle sum 100/30 rule combination of directional components shown in figure below is used for periodic reponses.

$$E_{Ed} = 1,0 \cdot E_{EdX} \oplus 0,3 \cdot E_{EdY}$$
$$E_{Ed} = 0,3 \cdot E_{EdX} \oplus 1,0 \cdot E_{EdY}$$

Figure 71. 100/30 rule (E_{Ed}: mode shapes)

6.9.7. Result

This chapter presents the results of the global analysis for on-site conditions. The results presented in this report are limited to the following:

a) Code check / utilization factors – Ultimate Limit States

b) Typical deflections – Serviceability Limit States

c) Connection check

6.9.7.1. Nodal support internal forces

For ULS (STR/GEO) -Permanent and transient – Load combinations:

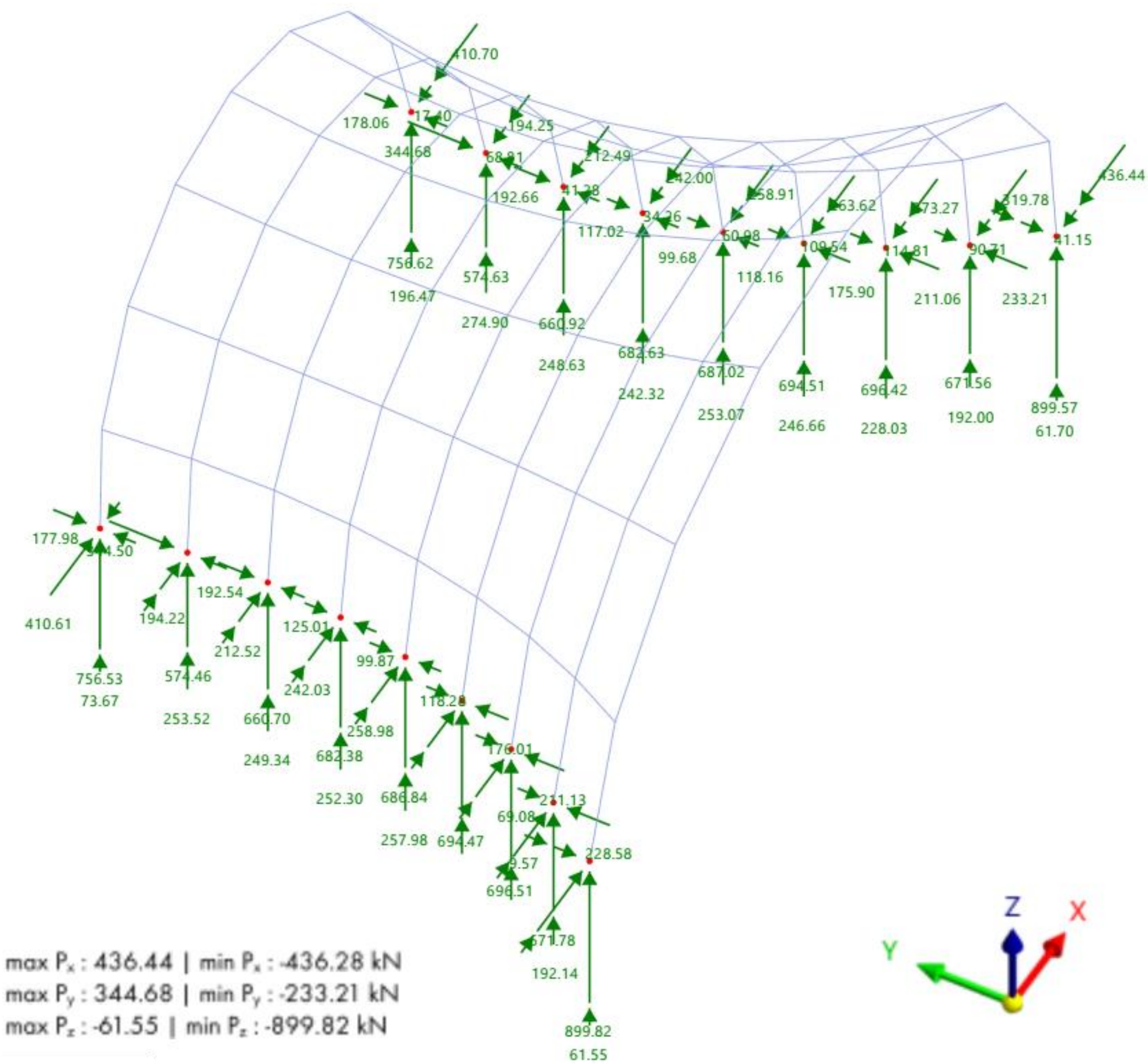


Figure 72. Nodal support internal forces for Permanent and transient – Load combinations

Node No.		Support Forces [kN]			Support Moments [kNm]			Corresponding Loading
		P_x	P_y	P_z	M_x	M_y	M_z	
	Total max/min values with corresponding values							
144	P_x	436.44	3.92	-899.6	0	0	0	CO134
145	P_x	-436.3	3.93	-899.8	0	0	0	CO125
81	P_y	113.17	344.68	-349.9	0	0	0	CO131
135	P_y	282.3	-233.2	-392.2	0	0	0	CO128
145	P_z	-41.01	5.74	-61.55	0	0	0	CO94
145	P_z	-436.3	3.93	-899.8	0	0	0	CO125
65	M_x	17.4	-10.91	-233.2	0	0	0	CO1
65	M_x	17.4	-10.91	-233.2	0	0	0	CO1
65	M_y	17.4	-10.91	-233.2	0	0	0	CO1
65	M_y	17.4	-10.91	-233.2	0	0	0	CO1
65	M_z	17.4	-10.91	-233.2	0	0	0	CO1
65	M_z	17.4	-10.91	-233.2	0	0	0	CO1

Table 9. Nodal support internal forces for Permanent and transient – Load combinations

For ULS (STR/GEO) – Seismic - Load combinations:

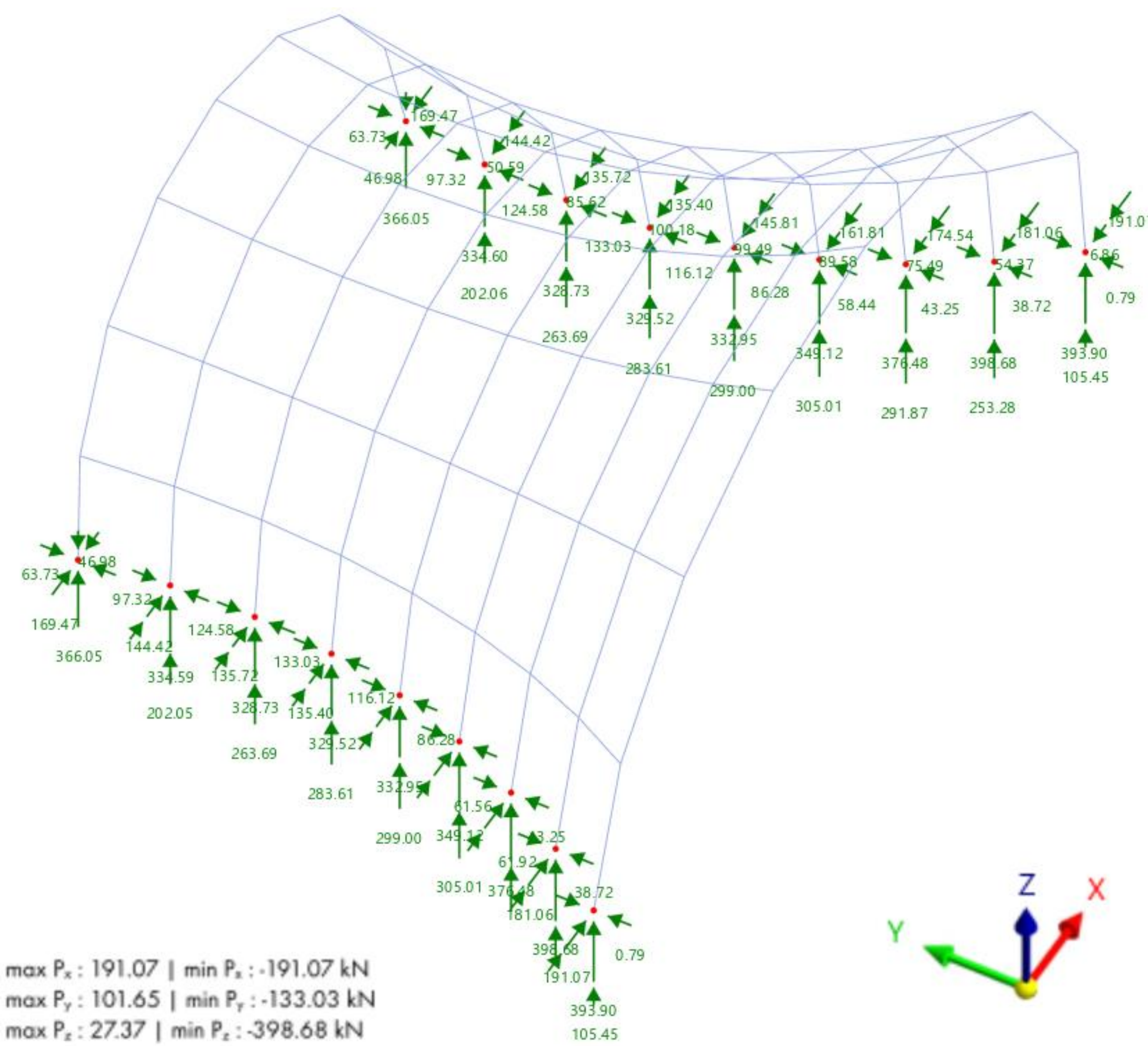


Figure 73. Nodal support internal forces for Seismic – Load combinations

Node No.		Support Forces [kN]			Support Moments [kNm]			Corresponding Loading
		P_x	P_y	P_z	M_x	M_y	M_z	
	Total max/min values with corresponding values							
144	P_x	191.07	1.65	-383.8	0	0	0	LC1 + LC11 (2.145 Hz)
145	P_x	-191.1	1.65	-383.8	0	0	0	LC1 + LC11 (2.145 Hz)
98	P_y	-128.7	101.65	-327.6	0	0	0	LC1 + LC11 (2.145 Hz)
90	P_y	92.67	-133	-264.8	0	0	0	LC1 + LC11 (2.145 Hz)
65	P_z	-44.88	-27.05	27.37	0	0	0	LC1 + LC11 (2.145 Hz)
135	P_z	167.16	-6.66	-398.7	0	0	0	LC1 + LC11 (2.145 Hz)
65	M_x	-46.98	-29.88	23.53	0	0	0	LC1 + LC11 (2.145 Hz)
65	M_x	-46.98	-29.88	23.53	0	0	0	LC1 + LC11 (2.145 Hz)
65	M_y	-46.98	-29.88	23.53	0	0	0	LC1 + LC11 (2.145 Hz)
65	M_y	-46.98	-29.88	23.53	0	0	0	LC1 + LC11 (2.145 Hz)
65	M_z	-46.98	-29.88	23.53	0	0	0	LC1 + LC11 (2.145 Hz)
65	M_z	-46.98	-29.88	23.53	0	0	0	LC1 + LC11 (2.145 Hz)

Table 10. Nodal support internal forces for Seismic – Load combinations

6.9.7.2. Beam internal max/min forces

For ULS (STR/GEO) -Permanent and transient – Load combinations:

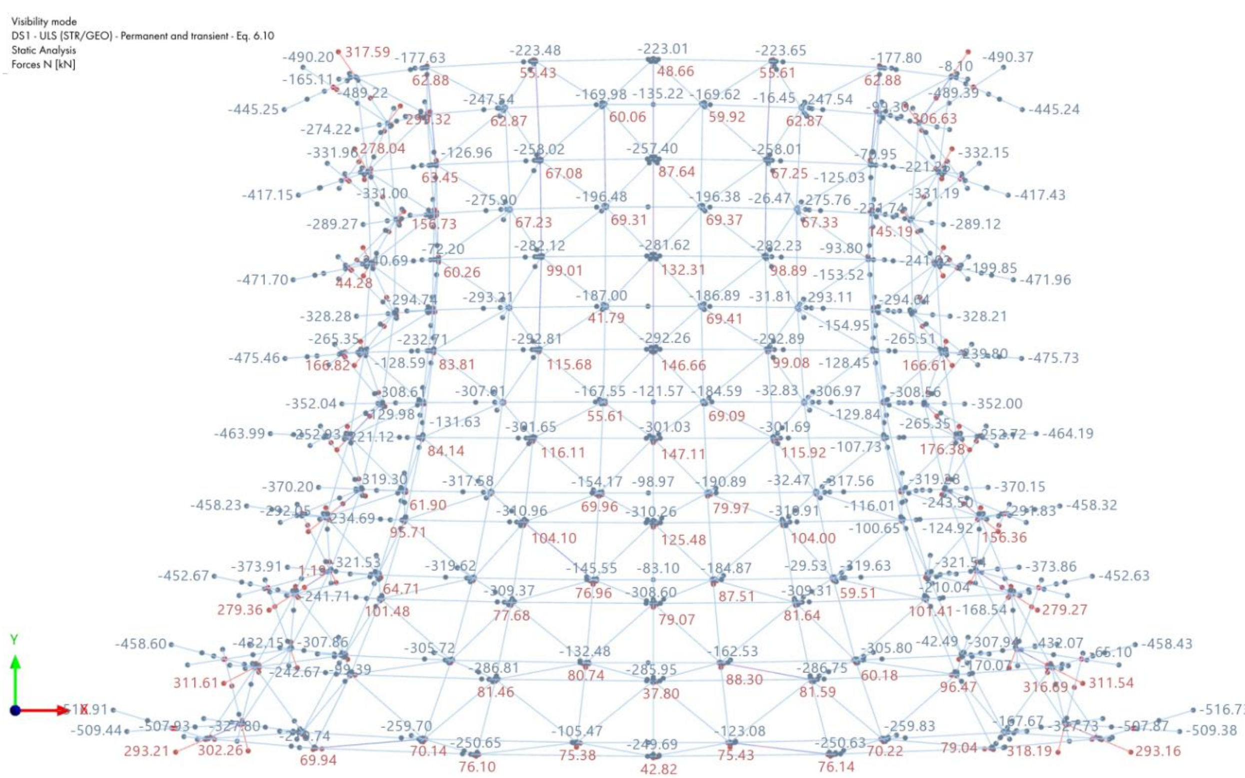


Figure 74. Axial forces for Permanent and transient – Load combinations

Member No.	Node No.	Location x [m]		Forces [kN] N	Vy	Vz	Moments [kNm] MT	My	Mz	Corresponding Loading
	Total max/min values with corresponding values									
465	57	4.688	N	318.19	0	-0.67	0	0	0	CO128
240	145	8.414	N	-516.9	0	-0.5	0	0	0	CO116
1	1	0.000	Vy	14.37	0	0.97	0	0	0	CO1
1	1	0.000	Vy	14.37	0	0.97	0	0	0	CO1
236	138	0.000	Vz	-158.1	0	2.53	0	0	0	CO1
236	139	8.431	Vz	-157.1	0	-2.53	0	0	0	CO1
1	1	0.000	MT	14.37	0	0.97	0	0	0	CO1
1	1	0.000	MT	14.37	0	0.97	0	0	0	CO1
236		4.215	My	-157.6	0	0	0	5.34	0	CO1
1	1	0.000	My	14.37	0	0.97	0	0	0	CO1
195		3.576	Mz	28.2	0	-0.87	0	0.52	0	CO100
191		3.507	Mz	-45.48	0	-0.88	0	0.52	0	CO98

Table 11. Beam internal max/min forces for Permanent and transient – Load combinations

For ULS (STR/GEO) – Seismic - Load combinations:

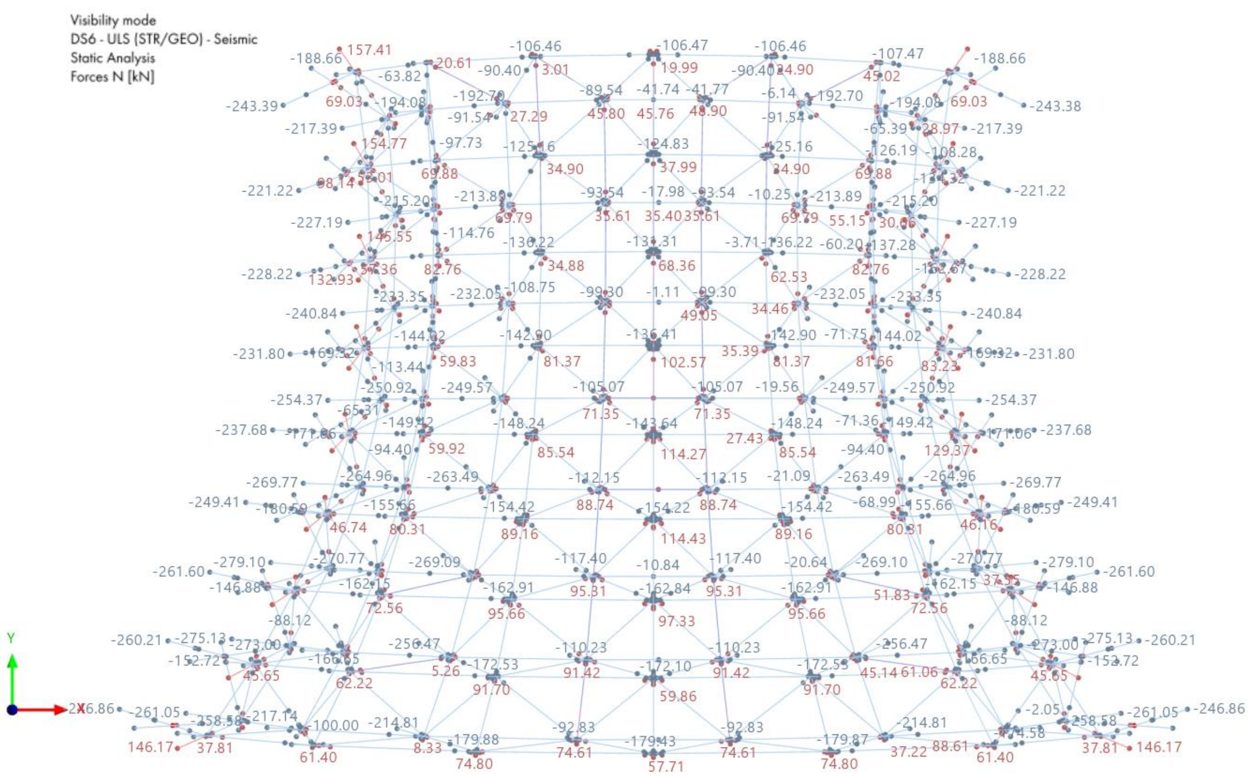


Figure 75. Nodal support internal forces for Seismic – Load combinations

Member No.	Node No.	Location x [m]		Forces [kN] N	V_y	V_z	Moments [kNm] M_T	M_y	M_z	Corresponding Loading
	Total max/min values with corresponding values									
244	66	0	N	157.41	-0.01	0.44	0	0	0	LC1 + LC11 (2.145 Hz)
77	41	0		-279.1	0.02	0.26	0	0	0	LC1 + LC11 (2.145 Hz)
236		7.935	Vy	-175.9	1.03	-1.67	0	0.88	0.51	LC1 + LC11 (2.145 Hz)
236		7.935		-56.99	-1.03	-1.64	0	0.87	-0.51	LC1 + LC11 (2.145 Hz)
235	137	0	Vz	-171.6	-0.35	2.06	0	0	0	LC1 + LC11 (2.145 Hz)
238	141	9.12		-171.6	0.35	-2.06	0	0	0	LC1 + LC11 (2.145 Hz)
1	1	0	MT	10.65	0	0.72	0	0	0	LC1 + LC11 (2.145 Hz)
1	1	0		10.65	0	0.72	0	0	0	LC1 + LC11 (2.145 Hz)
235		4.56	My	-171.3	0.03	-0.01	0	4.72	0.81	LC1 + LC11 (2.145 Hz)
233		4.95		-133.5	-0.01	-0.05	0	-0.38	-0.55	LC1 + LC11 (2.145 Hz)
234		4.796	Mz	-100.1	0	-0.07	0	4.08	2.67	LC1 + LC11 (2.145 Hz)
234		4.796		35.77	0	-0.05	0	1.1	-2.67	LC1 + LC11 (2.145 Hz)

Table 12. Beam internal max/min forces for Seismic – Load combinations

6.9.7.3. Von Mises stresses

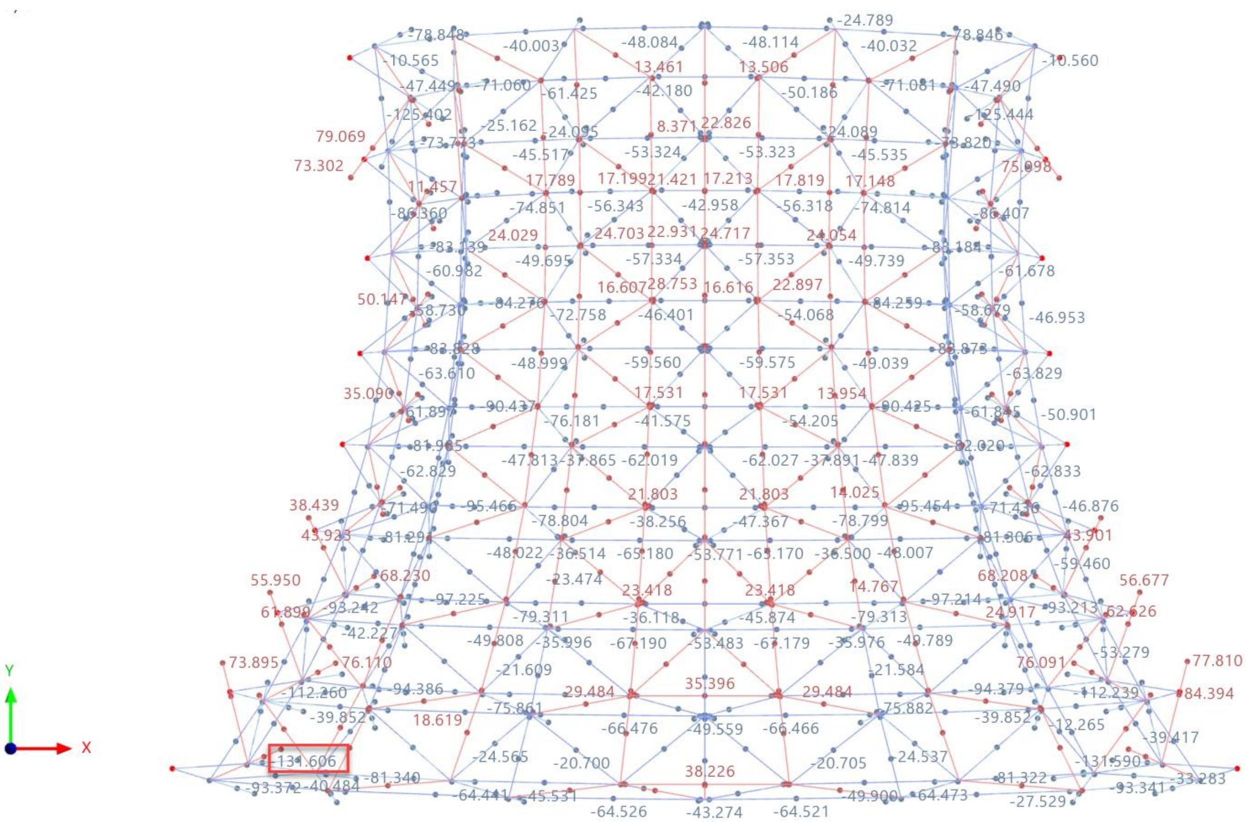


Figure 76. Von Mises stresses of members

The maximum stress 131.606 MPa is below allowable strength fy/1.0=355 MPa.

6.9.7.4. Ultilisation degrees of the members and deflections

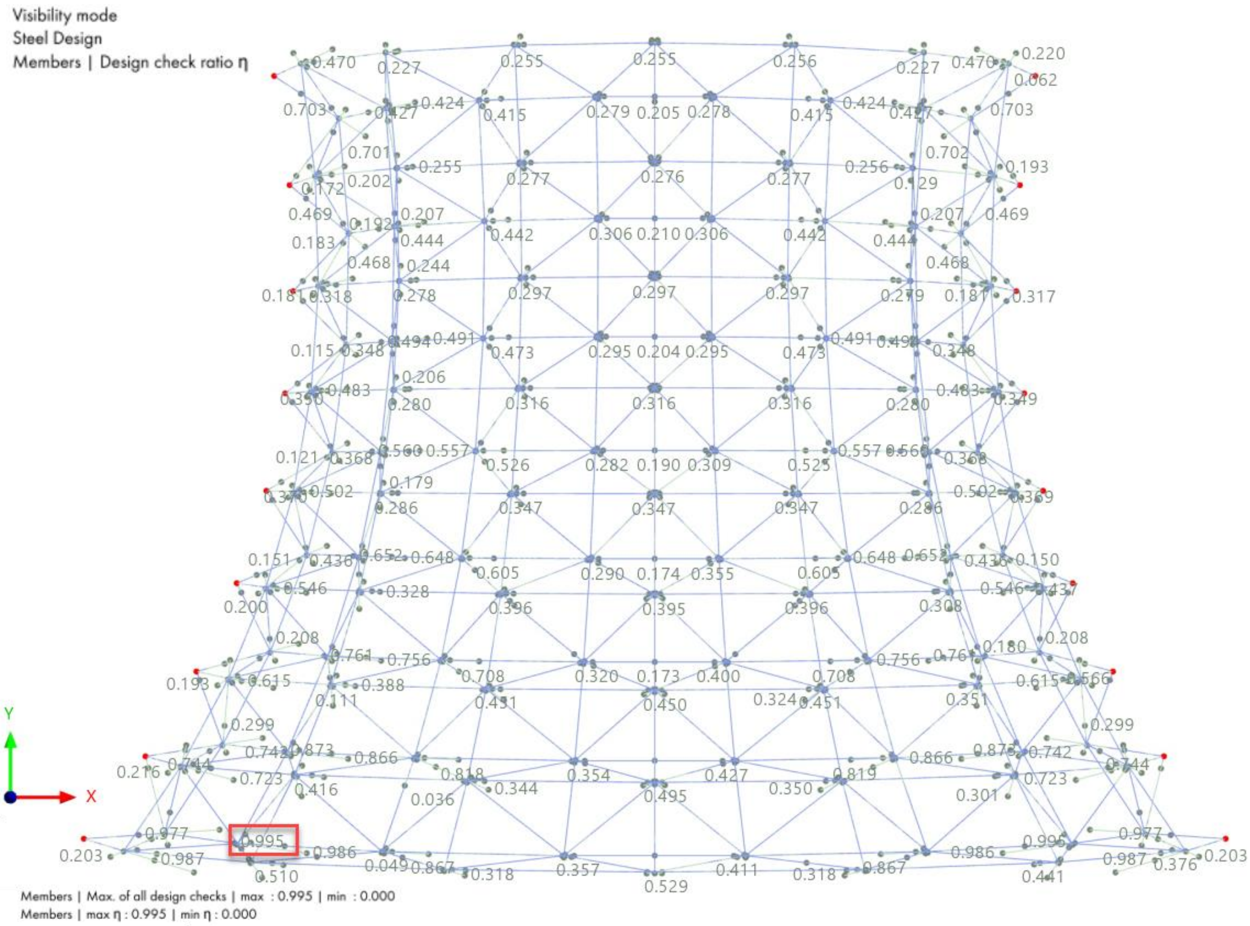


Figure 77. Ultilisations of members

Design Situation	Member No.	Location x [m]	Stress Point No.	Loading No.	Design Check Ratio η [--]	Design Check Type	Description
DS1	ULS (STR/GEO) - Permanent and transient - Eq. 6.10						
DS1	248	0.000		CO80	0.000	SP0100.00	Section Proof | Negligible internal forces
DS1	465	4.688		CO128	0.220	SP1100.00	Section Proof | Tension acc. to EN 1993-1-1, 6.2.3
DS1	495	0.000		CO125	0.353	SP1200.00	Section Proof | Compression acc. to EN 1993-1-1, 6.2.4
DS1	236	0.000		CO1	0.003	SP3100.02	Section Proof | Shear in z-axis acc. to EN 1993-1-1, 6.2.6(2) | Plastic design
DS1	236	4.215		CO1	0.044	SP4100.03	Section Proof | Bending about y-axis acc. to EN 1993-1-1, 6.2.5 | Plastic design
DS1	495	2.369		CO125	0.183	SP6500.02	Section Proof | Bending about y-axis, axial force and shear acc. to EN 1993-1-1, 6.2.9.1 and 6.2.10 | Plastic design
DS1	240	8.414		CO116	0.972	ST1100.00	Stability | Flexural buckling about principal y-axis acc. to EN 1993-1-1, 6.3.1
DS1	240	8.414		CO116	0.972	ST1300.00	Stability | Flexural buckling about principal z-axis acc. to EN 1993-1-1, 6.3.1
DS1	112	7.714		CO68	0.995	ST3100.00	Stability | Bending and buckling about principal axes acc. to EN 1993-1-1, 6.3.3

Design Situation	Member No.	Location x [m]	Stress Point No.	Loading No.	Design Check Ratio η [--]	Design Check Type	Description
DS4	SLS - Quasi-permanent						
DS4	1	0.000		CO146	0.000	SE0100.00	Serviceability | Negligible deflections
DS4	238	4.560		CO146	0.146	SE1100.00	Serviceability | Deflections in z-direction
DS6	ULS (STR/GEO) - Seismic						
DS6	248	0.000		RC1	0.000	SP0100.00	Section Proof | Negligible internal forces
DS6	244	0.000		RC1	0.109	SP1100.00	Section Proof | Tension acc. to EN 1993-1-1, 6.2.3
DS6	77	0.000		RC1	0.195	SP1200.00	Section Proof | Compression acc. to EN 1993-1-1, 6.2.4
DS6	235	0.000		RC1	0.003	SP3100.02	Section Proof | Shear in z-axis acc. to EN 1993-1-1, 6.2.6(2) | Plastic design
DS6	234	8.633		RC1	0.001	SP3200.02	Section Proof | Shear in y-axis acc. to EN 1993-1-1, 6.2.6(2) | Plastic design
DS6	237	0.000		RC1	0.003	SP3300.02	Section Proof | Resulting shear acc. to EN 1993-1-1, 6.2.6(2) | Plastic design
DS6	238	4.560		RC1	0.039	SP4100.03	Section Proof | Bending about y-axis acc. to EN 1993-1-1, 6.2.5 | Plastic design
DS6	234	4.796		RC1	0.022	SP5100.03	Section Proof | Bending about z-axis acc. to EN 1993-1-1, 6.2.5 | Plastic design
DS6	234	4.317		RC1	0.002	SP6500.01	Section Proof | Biaxial bending, axial force and shear acc. to EN 1993-1-1, 6.2.9.1 and 6.2.10 | Plastic design
DS6	87	3.154		RC1	0.071	SP6500.02	Section Proof | Bending about y-axis, axial force and shear acc. to EN 1993-1-1, 6.2.9.1 and 6.2.10 | Plastic design
DS6	106	4.821		RC1	0.050	SP6500.03	Section Proof | Bending about z-axis, axial force and shear acc. to EN 1993-1-1, 6.2.9.1 and 6.2.10 | Plastic design
DS6	100	3.404		RC1	0.001	SP6500.04	Section Proof | Biaxial bending and shear acc. to EN 1993-1-1, 6.2.9.1 and 6.2.10 | Plastic design
DS6	106	0.000		RC1	0.763	ST1100.00	Stability | Flexural buckling about principal y-axis acc. to EN 1993-1-1, 6.3.1
DS6	106	0.000		RC1	0.763	ST1300.00	Stability | Flexural buckling about principal z-axis acc. to EN 1993-1-1, 6.3.1
DS6	106	0.000		RC1	0.807	ST3100.00	Stability | Bending and buckling about principal axes acc. to EN 1993-1-1, 6.3.3

Table 13. Governing ultilisations of structure's members

The highest deflection: 12.2 mm is below allowable limit L/200 =150 mm.

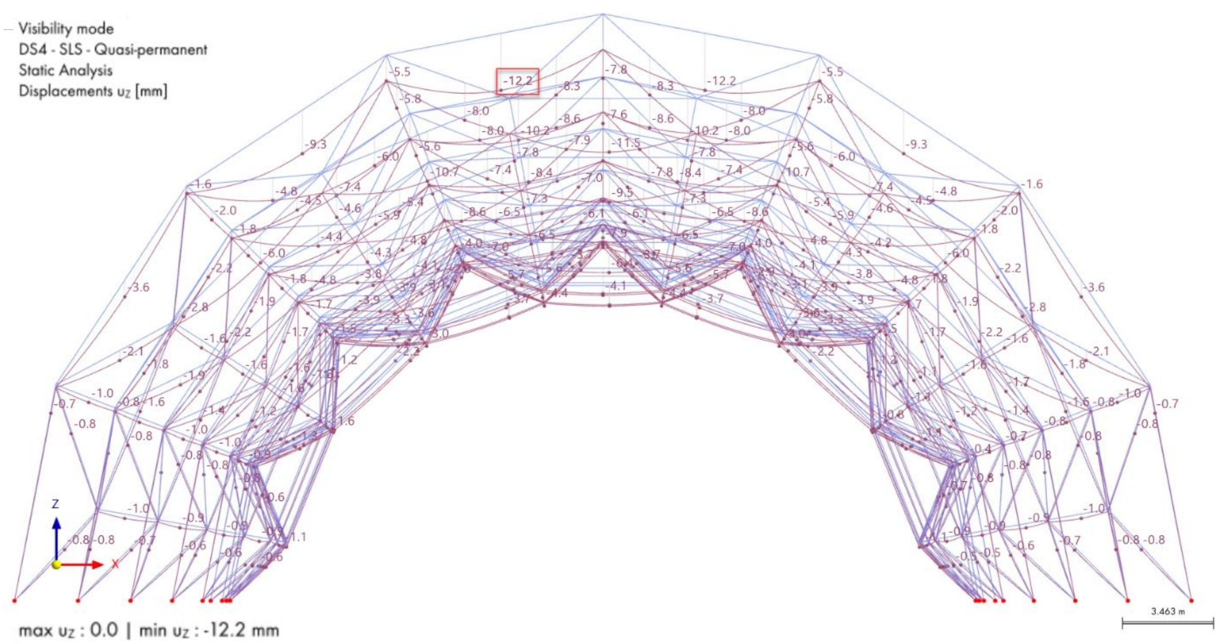


Figure 78. Deflection checks

The detail of steel design of member number 112 is attached in appendix A.

6.9.7.5. Check of connections

See below for the location of the joints. The joints were checked with forces from their most critical load combination.

The connections are checked with a separate CBFEM Analysis program, IDEAStatica and Hilti PROFIS Engineering. The connection forces have come from the member FEM model.

The detail design concept includes the following:

- building a detailed sub-model for the bolts and welds and the connected plates with the profiles
- the examination of selected connection type from the structure: one joint from upper layer, one joint from lower layer and two base joints.
- Summary of connection design is attached below while the detail information about connection design is attached in appendix A, such as the detail connection check of joints and bases and shear lug .
- In the joints of outer chords, notional member was created in order to transmit external loads from snow, wind loads…. into the joint.
- In the base, shear lug is applied to resist high shear force.

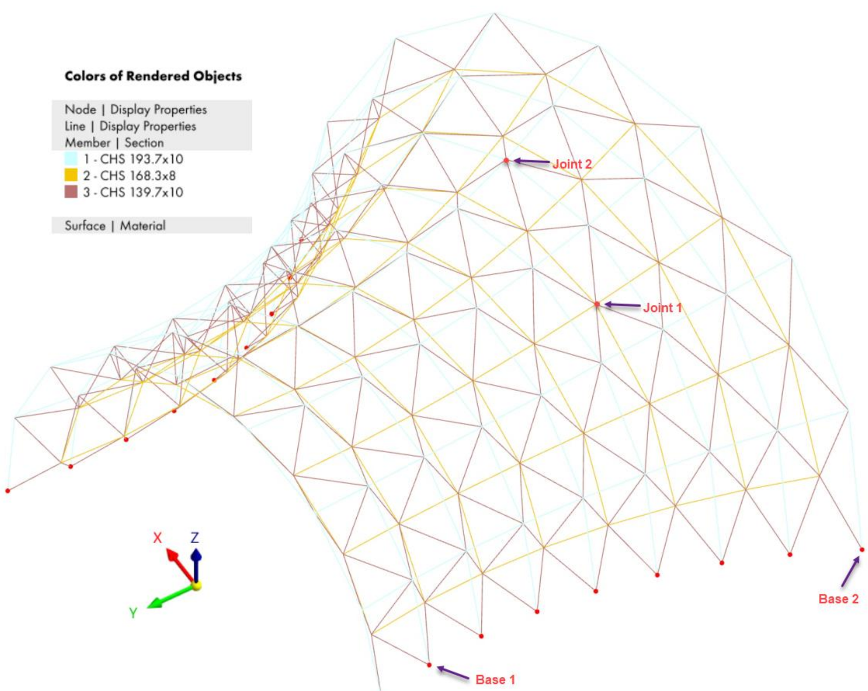


Figure 79. Connection joints

6.9.7.5.1. Connection design Parameters

The bolts of connections are made 8.8 quality, including the foundation adhesive anchor. The steel quality of endplates, baseplates, gusset plates, and stiffeners and cross sections are generally S355JR.

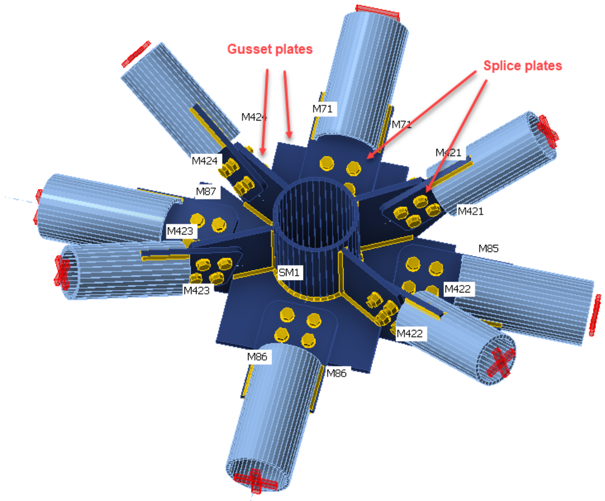
Steel connection calculations are made with IDEA Statica FEM software based on EN1993-1-8 and EN 1992-4. Detailed calculation results are included in the Appendices. Code Settings are shown below:

Item	Value	Unit	Reference
Safety factor γ_{M0}	1.00	-	EN 1993-1-1: 6.1
Safety factor γ_{M1}	1.00	-	EN 1993-1-1: 6.1
Safety factor γ_{M2}	1.25	-	EN 1993-1-1: 6.1
Safety factor γ_{M3}	1.25	-	EN 1993-1-8: 2.2
Safety factor γ_{C}	1.50	-	EN 1992-1-1: 2.4.2.4
Safety factor γ_{Inst}	1.20	-	EN 1992-4: Table 4.1
Joint coefficient βj	0.67	-	EN 1993-1-8: 6.2.5
Effective area - influence of mesh size	0.10	-	
Friction coefficient - concrete	0.25	-	EN 1993-1-8
Friction coefficient in slip-resistance	0.30	-	EN 1993-1-8 tab 3.7
Limit plastic strain	0.05	-	EN 1993-1-5
Detailing	No		
Distance between bolts [d]	2.20	-	EN 1993-1-8: tab 3.3
Distance between bolts and edge [d]	1.20	-	EN 1993-1-8: tab 3.3
Concrete breakout resistance check	Both		EN 1992-4: 7.2.1.4 and 7.2.2.5
Use calculated αb in bearing check.	Yes		EN 1993-1-8: tab 3.4
Cracked concrete	Yes		EN 1992-4
Local deformation check	No		CIDECT DG 1, 3 - 1.1
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints
Braced system	Yes		EN 1993-1-8: 5.2.2.5

6.9.7.5.2. Check of Joint 1

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]
M71	6 - CHS168.3/8.0	-101.3	16.3	32.2	0	0	0
M85	6 - CHS168.3/8.0	179.2	-21.8	27.0	0	0	0
M86	6 - CHS168.3/8.0	-105.2	21.4	33.9	0	0	0
M87	6 - CHS168.3/8.0	178.1	-45.0	26.8	0	0	0
M421	32 - CHS139.7/10.0	-132.0	-34.9	46.5	0	0	0
M422	32 - CHS139.7/10.0	147.5	-61.8	8.9	0	0	0
M423	32 - CHS139.7/10.0	12.5	1.2	-35.5	0	0	0
M424	32 - CHS139.7/10.0	-40.1	15.9	-3.9	0	0	0



Material

Steel	S 355 (EN)
Bolts	M20 8.8

Load effects (forces in equilibrium)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	M71 / Begin	-18.2	0.0	-0.9	0.0	0.0	0.0
	M85 / Begin	135.2	0.0	-1.2	0.0	0.0	0.0
	M86 / End	24.7	0.0	0.9	0.0	0.0	0.0
	M87 / End	-319.6	0.0	1.0	0.0	0.0	0.0
	M421 / Begin	53.5	0.0	-0.8	0.0	0.0	0.0
	M422 / Begin	158.6	0.0	-0.5	0.0	0.0	0.0
	M423 / Begin	-1.9	0.0	-1.0	0.0	0.0	0.0
	M424 / Begin	-7.1	0.0	-0.9	0.0	0.0	0.0

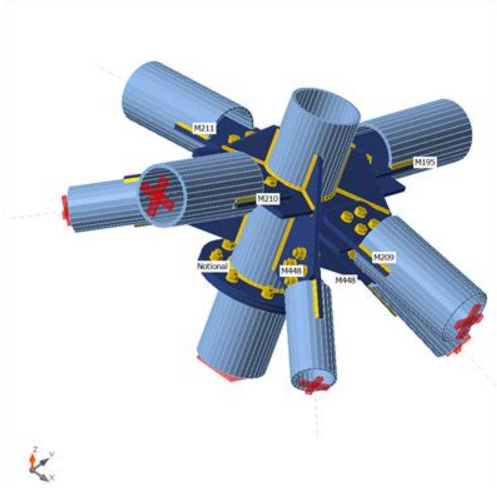
Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.1 < 5.0%	OK
Bolts	97.0 < 100%	OK
Welds	98.3 < 100%	OK
Buckling	7.81	
GMNA	Calculated	

6.9.7.5.3. Check of Joint 2

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]
M195	53 - CHS193.7/10.0	-90.0	22.6	0.0	0	0	0
M209	53 - CHS193.7/10.0	-179.6	10.8	27.0	0	0	0
M210	53 - CHS193.7/10.0	-90.0	28.6	0.0	0	0	0
M211	53 - CHS193.7/10.0	179.6	-10.8	27.4	0	0	0
M415	52 - CHS139.7/10.0	21.5	-44.3	-29.5	0	0	0
M418	52 - CHS139.7/10.0	158.5	-44.3	29.0	0	0	0
M448	52 - CHS139.7/10.0	-41.4	-14.7	-23.5	0	0	0
M449	52 - CHS139.7/10.0	-138.6	-14.7	23.2	0	0	0
Notional	28 - CHS219.1/10.0	90.0	63.0	0.0	250	0	0



Material

Steel	S 355 (EN)
Bolts	M20 8.8

Load effects (forces in equilibrium)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	M195 / Begin	-72.8	0.0	-1.2	0.0	0.0	0.0
	M209 / Begin	298.0	0.0	-2.0	0.0	0.0	0.0
	M210 / End	21.7	0.0	1.1	0.0	0.0	0.0
	M211 / End	-308.6	0.0	2.0	0.0	0.0	0.0
	M415 / End	-70.2	0.0	0.7	0.0	0.0	0.0
	M418 / End	-52.5	0.0	0.7	0.0	0.0	0.0
	M448 / End	-17.7	0.0	0.9	0.0	0.0	0.0
	M449 / End	-19.5	0.0	0.9	0.0	0.0	0.0
	Notional / Begin	-189.0	-0.7	-55.0	0.0	0.0	0.0

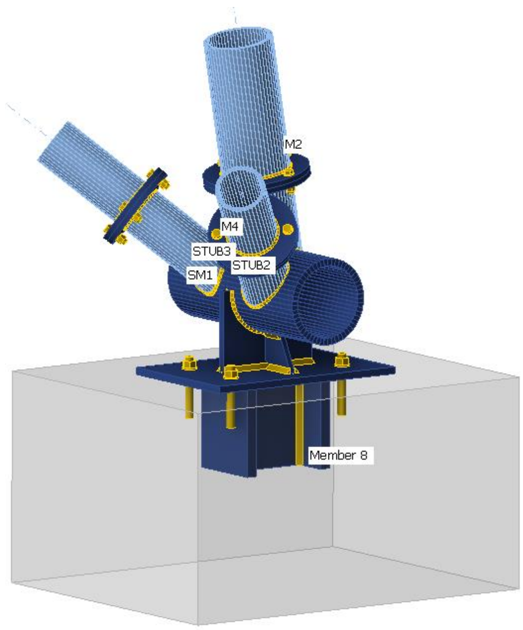
Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.7 < 5.0%	OK
Bolts	98.8 < 100%	OK
Welds	98.2 < 100%	OK
Buckling	13.63	
GMNA	Calculated	

6.9.7.5.4. Check of Base 1

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]
M2	33 - CHS193.7/10.0	-78.2	79.0	0.0	0	0	0
M4	34 - CHS139.7/10.0	-44.0	35.2	6.0	0	0	0
M5	34 - CHS139.7/10.0	-135.2	37.1	0.0	0	0	0



Material

Steel	S 355 (EN)
Bolts	M16 8.8, M24 8.8

Foundation block

CB 1		
Dimensions	900 x 1100	mm
Depth	600	mm
Anchor	M24 8.8	
Anchoring length	120	mm
Shear force transfer	Shear lug	
Cross-section of shear lug	Iw280x220	
Length of shear lug	265	mm

Load effects (forces in equilibrium)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	M2 / End	-417.1	0.0	0.0	0.0	0.0	0.0
	M4 / End	55.1	0.0	0.0	0.0	0.0	0.0
	M5 / End	-156.1	0.0	0.0	0.0	0.0	0.0
LE2	M2 / End	-248.2	0.0	0.0	0.0	0.0	0.0
	M4 / End	213.5	0.0	0.0	0.0	0.0	0.0
	M5 / End	-249.8	0.0	0.0	0.0	0.0	0.0

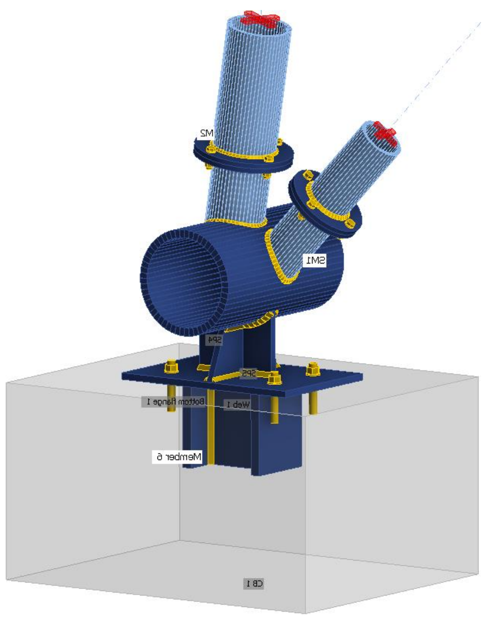
Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	1.3 < 5.0%	OK
Bolts	82.4 < 100%	OK
Anchors	81.2 < 100%	OK
Welds	98.9 < 100%	OK
Concrete block	97.5 < 100%	OK
Shear	17.4 < 100%	OK
Buckling	39.78	

6.9.7.5.5. Check of Base 2

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]
M2	33 - CHS193.7/10.0	-73.1	78.7	1.0	0	0	0
M5	34 - CHS139.7/10.0	-93.8	47.5	0.0	0	0	0



Material

Steel	S 355 (EN)
Bolts	M16 8.8, M24 8.8

Foundation block

CB 1		
Dimensions	900 x 1100	mm
Depth	600	mm
Anchor	M24 8.8	
Anchoring length	100	mm
Shear force transfer	Shear lug	
Cross-section of shear lug	Iw280x220	
Length of shear lug	265	mm

Load effects (forces in equilibrium)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	M2 / End	-516.7	0.0	0.0	0.0	0.0	0.0
	M5 / End	-504.4	0.0	0.0	0.0	0.0	0.0

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	1.1 < 5.0%	OK
Bolts	22.2 < 100%	OK
Anchors	68.9 < 100%	OK
Welds	99.6 < 100%	OK
Concrete block	98.5 < 100%	OK
Shear	48.3 < 100%	OK
Buckling	18.81	

7. THE CONSTRUCTION TECHNOLOGY OF THE DESIGNED STRUCTURE

This chapter outlines the cantilever method employed for installing the hyper space truss structure. This method allows constructors to connect components without constraints in the existing structure, facilitated by the high precision achieved in workshop fabrication.

First step, the position of bases will be ensured using optical Levels, tape measure according to general arrangement plane. Subsequently, the bases of structure will be constructed before the construction of super structure will take place.

Moving on to the second step, each joint is prefabricated in the manufacturing process and transported to the construction site. The structure is divided into five modules, and each module is further divided into three assemblies. Each assembly is then assembled in the field before erection. Module construction initiates from both sides and is assembled at the midpoint.

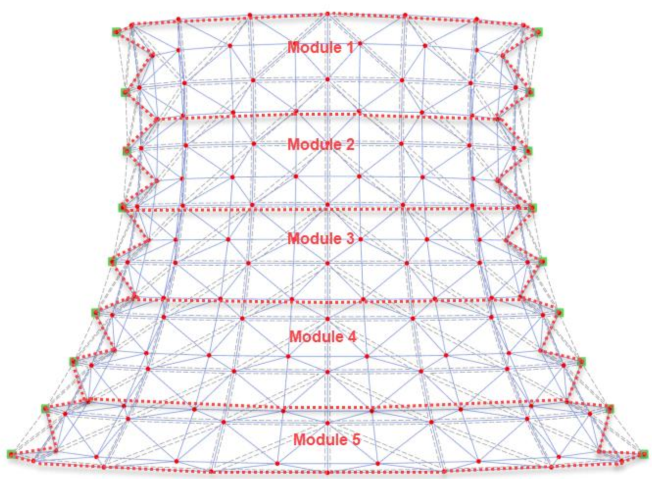


Figure 80. Construction modules

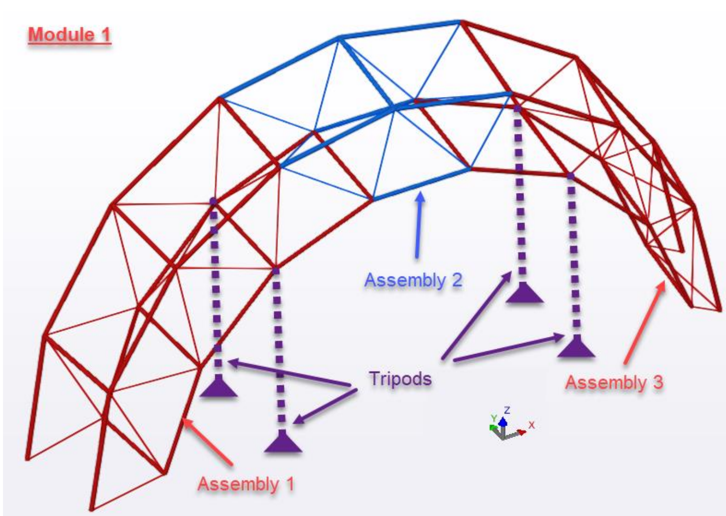


Figure 81. Assemblies of Module 1

The erection of structure is carried out using Meva fixed scaffolding tower and tower cranes. When the assembly is lifted using cranes to the installation position, two workers at each end of the assembly standing on the scaffold will be needed to adjust the assembly to ensure it's proper position. Then the assemblies 1 and 3 will be attached to the pre-assembled „tripods" before being attached to assembly 2 using bolt-tightening machine shown in the figure above. Several pre-assembled „tripods", which play auxiliary supports of structure during construction, will be placed at the free edge of module to prevent the deflection of the assembly due to it's selfweight. These tripods can be made of truss or HEM cross-section depending on acting concentrated loads from partial self-weight of structure. An example of tripod is shown in the figure below. Once the final module assembly reachs it's final position at the middle and connects to the tripod and the other assembly, the module's bearing is fixed.

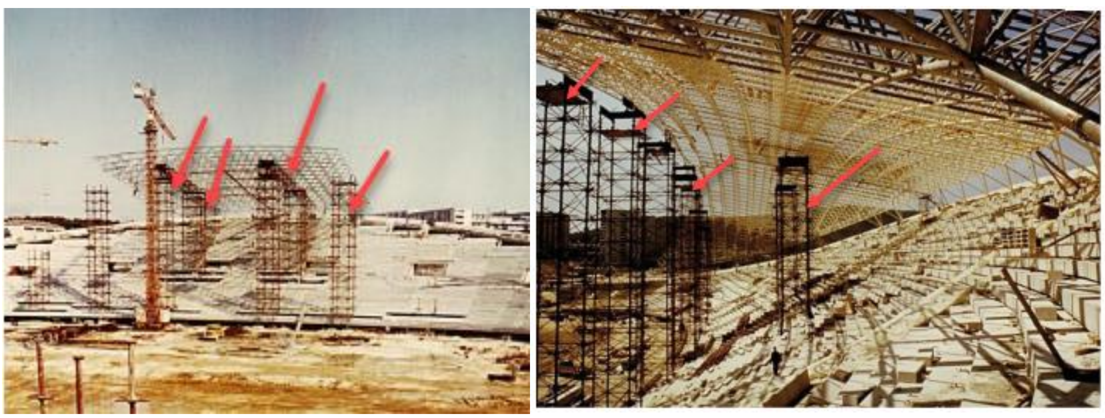


Figure 82. Pre-assembled tripod examples

Subsequently, every assemblies 1 and 2 of next modules are installed through symmetrical cantilevering of pre-assembled "tripods" in the long direction of the structure. Upon completing the installation, the structure is settled down by first releasing the internal auxiliary supports at the middle of the roof and then removing the auxiliary supports at the inner edge symmetrically, starting from the center of the structure.

8. TECHNICAL DESCRIPTION

In this part, seven technical drawings are made which include arrangement drawing, and detail drawings of four designed connections in the 6.9.7.5 section.

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10. APPENDIX A

Detail of steel design of member number 112

Detail Connection Check of Joint 1

Detail Connection Check of Joint 2

Detail Connection Check of Base 1

Detail Connection Check of Base 2

Detail check of anchor bolt and footing in Hilti

Detail of steel design of member number 112

Section Classification					
Classification	Automatically	Class 1			
Subpanel No. 1 | Class 1					
Support of part	Type	Pipe			
Stress at start	σ_A	82.097	N/mm²	> 0	Compression
Stress at end	σ_B	82.097	N/mm²	> 0	Compression
Outer diameter of circular tubular sections	d	168.3	mm		Tab. 5.2
Thickness of section part	t	8.0	mm		Tab. 5.2
Yield strength	$f_{y,d}$	355.000	N/mm²		3.2.1
Material coefficient	ε	0.814	--		Tab. 5.2
d/t-limit for class 1	λ_1	33.099	--		Tab. 5.2
d/t-limit for class 2	λ_2	46.338	--		Tab. 5.2
d/t-limit for class 3	λ_3	59.577	--		Tab. 5.2
d/t ratio	d/t	21.038	--	$\leq \lambda_1$	Tab. 5.2
Class of section part		Class 1			Tab. 5.2

Design Check Values					
Design compression force	$N_{c,Ed}$	330.85	kN		
Design bending moment (maximum on segment)	$M_{y,Ed}$	1.19	kNm		
Depth of section	h	168.3	mm		1.6(1), Fig. 1.1
Width of section	b	168.3	mm		1.6(1), Fig. 1.1
Sectional area	A	4030.000	mm²		
Moment of inertia	I_y	1297.00	cm⁴		
Moment of inertia	I_z	1297.00	cm⁴		
Plastic section modulus	$W_{pl,y}$	206.00	cm³		
Modulus of elasticity	E	210000.000	N/mm²		
Yield strength	f_y	355.000	N/mm²		3.2.1(1)
Partial factor	γ_{M0}	1.00	--		6.1(1)
Partial factor	γ_{M1}	1.00	--		6.1(1)
Characteristic value of resistance to compression	N_{Rk}	1430.650	kN		6.3.3, Tab. 6.7
Characteristic value of resistance to bending moments	$M_{y,Rk}$	73.13	kNm		6.3.3, Tab. 6.7
Buckling curve	BC_y	c			6.3.1.2, Tab. 6.2
Imperfection factor	α_y	0.490	--		6.3.1.2, Tab. 6.1
Buckling length	$L_{cr,y}$	7.714	m		1.5.6
Elastic critical force	$N_{cr,y}$	451.75	kN		6.3.1.2(1)
Non-dimensional slenderness	$\bar{\lambda}_y$	1.780	--		6.3.1.3(1)
Value to determine reduction factor χ	Φ_y	2.470	--		6.3.1.2(1)
Reduction factor for buckling	χ_y	0.24	--		6.3.1.2(1), Eq. 6.49
Buckling curve	BC_z	c			6.3.1.2, Tab. 6.2
Imperfection factor	α_z	0.490	--		6.3.1.2, Tab. 6.1
Buckling length	$L_{cr,z}$	7.714	m		1.5.6
Elastic critical force	$N_{cr,z}$	451.75	kN		6.3.1.2(1)
Non-dimensional slenderness	$\bar{\lambda}_z$	1.780	--		6.3.1.3(1)
Value to determine reduction factor χ	Φ_z	2.470	--		6.3.1.2(1)
Reduction factor	χ_z	0.24	--		6.3.1.2(1), Eq. 6.49
Ratio of end moments	Ψ_y	0.000	--		
Hogging moment	$M_{h,y}$	0.00	kNm		Tab. B.3
Sagging moment	$M_{s,y}$	1.19	kNm		Tab. B.3
Factor	$\alpha_{h,y}$	0.000	--		Tab. B.3
Equivalent uniform moment factor	C_{my}	0.950	--		Tab. B.3
Interaction factor	k_{yy}	1.685	--		
Interaction factor	k_{zy}	1.011	--		6.3.3(4)
Design component for N	$\eta_{N\ 6.61}$	0.968	--		6.3.3(4), Eq. 6.61
Design component for M_y	$\eta_{My\ 6.61}$	0.027	--		6.3.3(4), Eq. 6.61
Design ratio	$\eta_{6.61}$	0.995	--		6.3.3(4), Eq. 6.61
Design component for N	$\eta_{N\ 6.62}$	0.968	--		6.3.3(4), Eq. 6.62
Design component for M_y	$\eta_{My\ 6.62}$	0.016	--		6.3.3(4), Eq. 6.62
Design ratio	$\eta_{6.62}$	0.984	--		6.3.3(4), Eq. 6.62
Design check ratio	η	**0.995**	--	**≤ 1** ✔	EN 1993-1-1, 6.3.3(4), Eq. 6.61,...

Design Check ST3100 | EN 1993 | MSZ | 2015-11

Stability
Bending and buckling about principal axes acc. to EN 1993-1-1, 6.3.3

$$N_{cr,y} = (\pi)^2 \cdot E \cdot \frac{I_y}{(L_{cr,y})^2}$$ 6.3.1.2(1)

$$= (\pi)^2 \cdot 210000.000\,N/mm^2 \cdot \frac{1297.00\,cm^4}{(7.714\,m)^2}$$

$$= 451.75\,kN$$

$$\bar{\lambda}_y = \sqrt{\frac{A \cdot f_y}{N_{cr,y}}}$$ 6.3.1.3(1)

$$= \sqrt{\frac{4030.000\,mm^2 \cdot 355.000\,N/mm^2}{451.75\,kN}}$$

$$= 1.780$$

$$\Phi_y = 0.5 \cdot \left[1 + \alpha_y \cdot \left(\bar{\lambda}_y - 0.2\right) + \left(\bar{\lambda}_y\right)^2\right]$$ 6.3.1.2(1)

$$= 0.5 \cdot \left[1 + 0.490 \cdot (1.780 - 0.2) + (1.780)^2\right]$$

$$= 2.470$$

$$\chi_y = \frac{1}{\Phi_y + \sqrt{(\Phi_y)^2 - \left(\bar{\lambda}_y\right)^2}}$$ 6.3.1.2(1), Eq. 6.49

$$= \frac{1}{2.470 + \sqrt{(2.470)^2 - (1.780)^2}}$$

$$= 0.24$$

$$N_{cr,z} = (\pi)^2 \cdot E \cdot \frac{I_z}{(L_{cr,z})^2}$$ 6.3.1.2(1)

$$= (\pi)^2 \cdot 210000.000\,N/mm^2 \cdot \frac{1297.00\,cm^4}{(7.714\,m)^2}$$

$$= 451.75\,kN$$

$$\bar{\lambda}_z = \sqrt{\frac{A \cdot f_y}{N_{cr,z}}}$$ 6.3.1.3(1)

$$= \sqrt{\frac{4030.000\,mm^2 \cdot 355.000\,N/mm^2}{451.75\,kN}}$$

$$= 1.780$$

$$\Phi_z = 0.5 \cdot \left[1 + \alpha_z \cdot \left(\bar{\lambda}_z - 0.2\right) + \left(\bar{\lambda}_z\right)^2 \right]$$

$$= 0.5 \cdot \left[1 + 0.490 \cdot (1.780 - 0.2) + (1.780)^2 \right]$$

$$= 2.470$$

6.3.1.2(1)

$$\chi_z = \frac{1}{\Phi_z + \sqrt{(\Phi_z)^2 - \left(\bar{\lambda}_z\right)^2}}$$

$$= \frac{1}{2.470 + \sqrt{(2.470)^2 - (1.780)^2}}$$

$$= 0.24$$

6.3.1.2(1), Eq. 6.49

$$N_{Rk} = A \cdot f_y$$

$$= 4030.000\,\mathrm{mm}^2 \cdot 355.000\,\mathrm{N/mm}^2$$

$$= 1430.650\,\mathrm{kN}$$

6.3.3, Tab. 6.7

$$M_{y,Rk} = W_{pl,y} \cdot f_y$$

$$= 206.00\,\mathrm{cm}^3 \cdot 355.000\,\mathrm{N/mm}^2$$

$$= 73.13\,\mathrm{kNm}$$

6.3.3, Tab. 6.7

$$\alpha_{h,y} = \frac{M_{h,y}}{M_{s,y}}$$

$$= \frac{0.00\,\mathrm{kNm}}{1.19\,\mathrm{kNm}}$$

$$= 0.000$$

Tab. B.3

$$C_{my} = 0.95 + 0.05 \cdot \alpha_{h,y}$$

$$= 0.95 + 0.05 \cdot 0.000$$

$$= 0.950$$

Tab. B.3

$$k_{yy} = C_{my} \cdot \left(1 + \left(\bar{\lambda}_y - 0.2\right) \cdot \frac{N_{c,Ed}}{\chi_y \cdot \frac{N_{Rk}}{\gamma_{M1}}} \right)$$

$$= 0.950 \cdot \left(1 + (1.780 - 0.2) \cdot \frac{330.85\,\mathrm{kN}}{0.24 \cdot \frac{1430.650\,\mathrm{kN}}{1.00}} \right)$$

$$= 2.402$$

$$k_{yy} = \min\left(k_{yy},\ C_{my} \cdot \left(1 + 0.8 \cdot \frac{N_{c,Ed}}{\chi_y \cdot \frac{N_{Rk}}{\gamma_{M1}}} \right) \right)$$

$$= \min\left(2.402,\ 0.950 \cdot \left(1 + 0.8 \cdot \frac{330.85\,\mathrm{kN}}{0.24 \cdot \frac{1430.650\,\mathrm{kN}}{1.00}} \right) \right)$$

$$= 1.685$$

$$k_{zy} = 0.6 \cdot C_{my} \cdot \left(1 + \left(\bar{\lambda}_y - 0.2\right) \cdot \frac{N_{c,Ed}}{\chi_y \cdot \frac{N_{Rk}}{\gamma_{M1}}}\right)$$

$$= 0.6 \cdot 0.950 \cdot \left(1 + (1.780 - 0.2) \cdot \frac{330.85\ \mathrm{kN}}{0.24 \cdot \frac{1430.650\ \mathrm{kN}}{1.00}}\right)$$

$$= 1.441$$

6.3.3(4)

$$k_{zy} = \min\left(k_{zy},\ 0.6 \cdot C_{my} \cdot \left(1 + 0.8 \cdot \frac{N_{c,Ed}}{\chi_y \cdot \frac{N_{Rk}}{\gamma_{M1}}}\right)\right)$$

$$= \min\left(1.441,\ 0.6 \cdot 0.950 \cdot \left(1 + 0.8 \cdot \frac{330.85\ \mathrm{kN}}{0.24 \cdot \frac{1430.650\ \mathrm{kN}}{1.00}}\right)\right)$$

$$= 1.011$$

6.3.3(4)

$$\eta_{N6.61} = \frac{N_{c,Ed}}{\chi_y \cdot \frac{N_{Rk}}{\gamma_{M1}}}$$

$$= \frac{330.85\ \mathrm{kN}}{0.24 \cdot \frac{1430.650\ \mathrm{kN}}{1.00}}$$

$$= 0.968$$

6.3.3(4), Eq. 6.61

$$\eta_{My6.61} = k_{yy} \cdot \frac{\left|M_{y,Ed}\right|}{\frac{M_{y,Rk}}{\gamma_{M1}}}$$

$$= 1.685 \cdot \frac{|1.19\ \mathrm{kNm}|}{\frac{73.13\ \mathrm{kNm}}{1.00}}$$

$$= 0.027$$

6.3.3(4), Eq. 6.61

$$\eta_{6.61} = \eta_{N6.61} + \eta_{My6.61}$$

$$= 0.968 + 0.027$$

$$= 0.995$$

6.3.3(4), Eq. 6.61

$$\eta_{N6.62} = \frac{N_{c,Ed}}{\chi_z \cdot \frac{N_{Rk}}{\gamma_{M1}}}$$

$$= \frac{330.85\ \mathrm{kN}}{0.24 \cdot \frac{1430.650\ \mathrm{kN}}{1.00}}$$

$$= 0.968$$

6.3.3(4), Eq. 6.62

$$\eta_{My6.62} = k_{zy} \cdot \frac{\left|M_{y,Ed}\right|}{\frac{M_{y,Rk}}{\gamma_{M1}}}$$

$$= 1.011 \cdot \frac{|1.19\ \mathrm{kNm}|}{\frac{73.13\ \mathrm{kNm}}{1.00}}$$

$$= 0.016$$

6.3.3(4), Eq. 6.62

$\eta_{6.62} = \eta_{N6.62} + \eta_{My6.62}$ — 6.3.3(4), Eq. 6.62

$= 0.968 + 0.016$

$= 0.984$

$\eta = \max(\eta_{6.61}, \eta_{6.62})$ — 6.3.3(4), Eq. 6.61, 6.62

$= \max(0.995, 0.984)$

$= 0.995$

$\boxed{\eta = 0.995 \le 1}$ ✔

$N_{cr,y}$	Elastic critical force
E	Modulus of elasticity
I_y	Moment of inertia
$L_{cr,y}$	Buckling length
$\bar{\lambda}_y$	Non-dimensional slenderness
A	Sectional area
f_y	Yield strength
Φ_y	Value to determine reduction factor χ
α_y	Imperfection factor
χ_y	Reduction factor for buckling
$N_{cr,z}$	Elastic critical force
I_z	Moment of inertia
$L_{cr,z}$	Buckling length
$\bar{\lambda}_z$	Non-dimensional slenderness
Φ_z	Value to determine reduction factor χ
α_z	Imperfection factor
χ_z	Reduction factor
N_{Rk}	Characteristic value of resistance to compression
$M_{y,Rk}$	Characteristic value of resistance to bending moments
$W_{pl,y}$	Plastic section modulus
$\alpha_{h,y}$	Factor
$M_{h,y}$	Hogging moment
$M_{s,y}$	Sagging moment
C_{my}	Equivalent uniform moment factor
k_{yy}	Interaction factor
$N_{c,Ed}$	Design compression force
γ_{M1}	Partial factor
k_{zy}	Interaction factor
$\eta_{N\ 6.61}$	Design component for N
$\eta_{My\ 6.61}$	Design component for M_y
$M_{y,Ed}$	Design bending moment (maximum on segment)
$\eta_{6.61}$	Design ratio
$\eta_{N\ 6.62}$	Design component for N
$\eta_{My\ 6.62}$	Design component for M_y
$\eta_{6.62}$	Design ratio

Detail Connection Check of Joint 1

Geometry

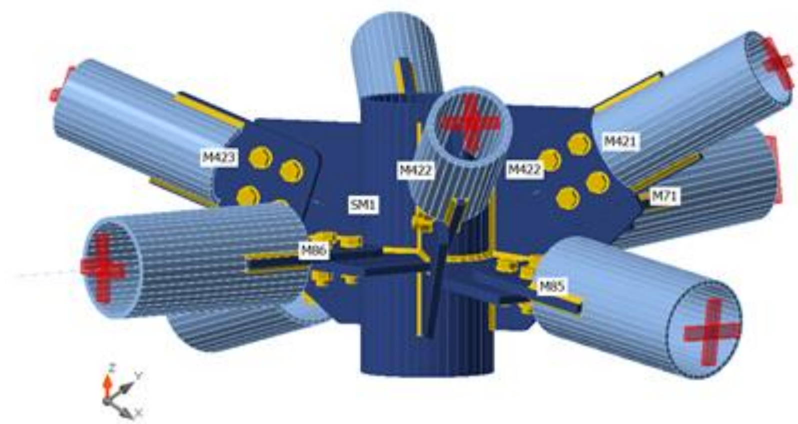
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]
M71	6 - CHS168.3/8.0	-101.3	16.3	32.2	0	0	0
M85	6 - CHS168.3/8.0	179.2	-21.8	27.0	0	0	0
M86	6 - CHS168.3/8.0	-105.2	21.4	33.9	0	0	0
M87	6 - CHS168.3/8.0	178.1	-45.0	26.8	0	0	0
M421	32 - CHS139.7/10.0	-132.0	-34.9	46.5	0	0	0
M422	32 - CHS139.7/10.0	147.5	-61.8	8.9	0	0	0
M423	32 - CHS139.7/10.0	12.5	1.2	-35.5	0	0	0
M424	32 - CHS139.7/10.0	-40.1	15.9	-3.9	0	0	0

Supports and forces

Name	Support	Forces in	X [mm]
M71 / end	N-Vy-Vz-Mx-My-Mz	Position	0
M85 / end	Mx-My-Mz	Position	0
M86 / end	Mx-My-Mz	Position	0
M87 / end	Mx-My-Mz	Position	0
M421 / end	Mx-My-Mz	Position	0
M422 / end	Mx-My-Mz	Position	0
M423 / end	Mx-My-Mz	Position	0
M424 / end	Mx-My-Mz	Position	0

Cross-sections

Name	Material
6 - CHS168.3/8.0	S 355
32 - CHS139.7/10.0	S 355
28 - CHS219.1/10.0	S 355



Bolts

Name	Bolt assembly	Diameter [mm]	f_u [MPa]	Gross area [mm^2]
M20 8.8	M20 8.8	20	800.0	314

Load effects (forces in equilibrium)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	M71 / Begin	-18.2	0.0	-0.9	0.0	0.0	0.0
	M85 / Begin	135.2	0.0	-1.2	0.0	0.0	0.0
	M86 / End	24.7	0.0	0.9	0.0	0.0	0.0
	M87 / End	-319.6	0.0	1.0	0.0	0.0	0.0
	M421 / Begin	53.5	0.0	-0.8	0.0	0.0	0.0
	M422 / Begin	158.6	0.0	-0.5	0.0	0.0	0.0
	M423 / Begin	-1.9	0.0	-1.0	0.0	0.0	0.0
	M424 / Begin	-7.1	0.0	-0.9	0.0	0.0	0.0

Unbalanced forces

Name	X [kN]	Y [kN]	Z [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	-0.8	-0.6	4.4	1.1	2.6	-0.4

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.1 < 5.0%	OK
Bolts	97.0 < 100%	OK
Welds	98.3 < 100%	OK
Buckling	7.81	
GMNA	Calculated	

Plates

Name	t_p [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [MPa]	Status
M71	8.0	LE1	117.0	0.0	0.0	OK
M85	8.0	LE1	223.6	0.0	0.0	OK
M86	8.0	LE1	30.0	0.0	0.0	OK
M87	8.0	LE1	355.2	0.1	0.0	OK
M421	10.0	LE1	78.2	0.0	0.0	OK
M422	10.0	LE1	202.2	0.0	0.0	OK
M423	10.0	LE1	16.3	0.0	0.0	OK
M424	10.0	LE1	12.8	0.0	0.0	OK
SM1	10.0	LE1	269.4	0.0	0.0	OK
CPL1a	15.0	LE1	221.5	0.0	1.6	OK
CPL1b	15.0	LE1	23.0	0.0	1.6	OK
SP1	15.0	LE1	355.2	0.1	39.8	OK
CPL4	15.0	LE1	355.3	0.1	39.5	OK
CPL5a	15.0	LE1	248.1	0.0	1.6	OK
CPL5b	15.0	LE1	30.4	0.0	5.2	OK
CPL6a	15.0	LE1	200.4	0.0	26.8	OK
CPL6b	15.0	LE1	282.1	0.0	23.3	OK
SP2	15.0	LE1	170.7	0.0	2.1	OK
CPL7	15.0	LE1	34.9	0.0	2.6	OK
CPL8a	15.0	LE1	82.0	0.0	7.6	OK
CPL8b	15.0	LE1	90.2	0.0	6.1	OK
SP3	15.0	LE1	230.6	0.0	15.0	OK
CPL9	15.0	LE1	148.5	0.0	16.5	OK
SP4	15.0	LE1	158.7	0.0	3.5	OK
CPL10	15.0	LE1	104.6	0.0	7.1	OK

Design data

Material	f_y [MPa]	ε_{lim} [%]
S 355	355.0	5.0

Symbol explanation

t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain

$\sigma_{c,Ed}$ Contact stress
f_y Yield strength
ε_{lim} Limit of plastic strain

Detailed result for CPL4

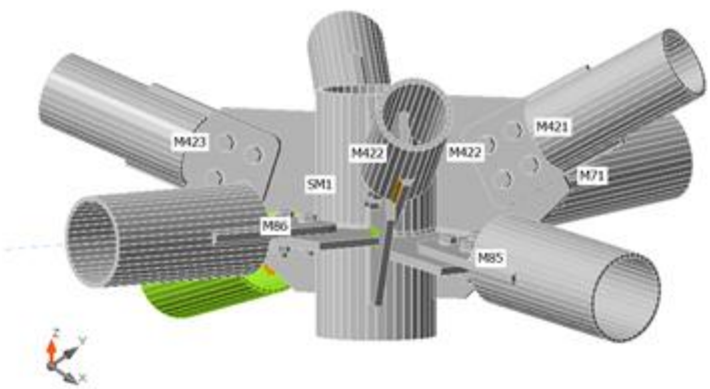
Design values used in the analysis

$f_{yd} = \frac{f_{yk}}{\gamma_{M0}} =$ 355.0 MPa

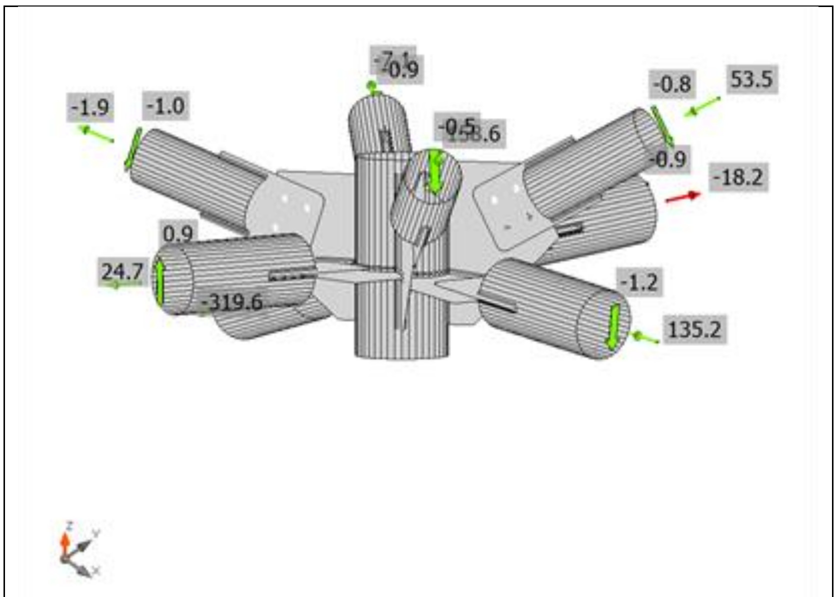
Where:

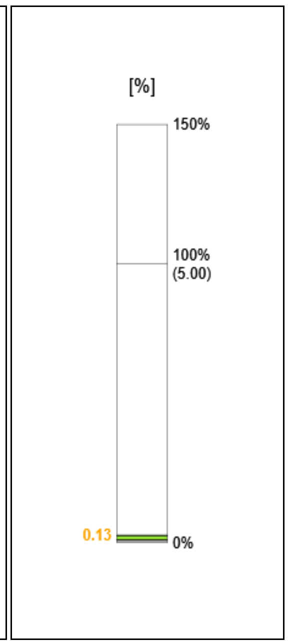
$f_{yk} =$ 355.0 MPa – characteristic yield strength

$\gamma_{M0} =$ 1.00 – partial safety factor for steel material EN 1993-1-1 – 6.1

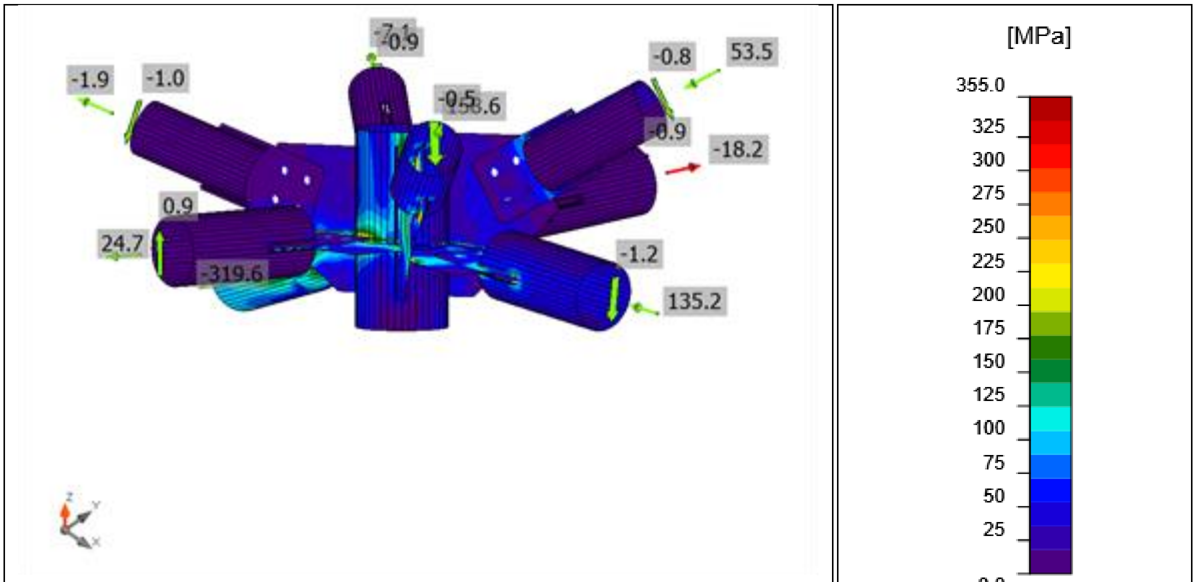


Overall check, LE1



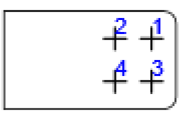
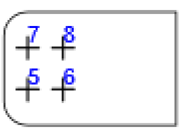
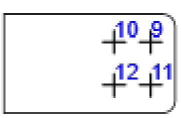
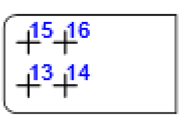
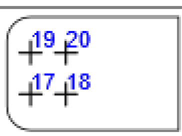
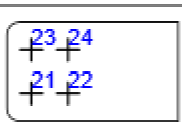
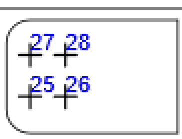


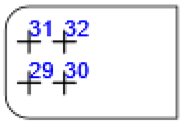
Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	$F_{t,Ed}$ [kN]	$F_{v,Ed}$ [kN]	$F_{b,Rd}$ [kN]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	M20 8.8 - 1	LE1	1.7	4.8	178.2	1.2	5.2	6.0	OK
	B2	M20 8.8 - 1	LE1	0.3	5.2	169.6	0.2	5.5	5.6	OK
	B3	M20 8.8 - 1	LE1	0.2	2.3	200.5	0.2	2.4	2.6	OK
	B4	M20 8.8 - 1	LE1	0.5	2.4	249.1	0.4	2.5	2.8	OK
	B5	M20 8.8 - 1	LE1	21.3	81.1	193.8	15.1	86.2	97.0	OK
	B6	M20 8.8 - 1	LE1	24.0	79.7	193.8	17.0	84.7	96.9	OK
	B7	M20 8.8 - 1	LE1	19.7	79.6	193.8	13.9	84.6	94.6	OK
	B8	M20 8.8 - 1	LE1	22.6	79.1	193.8	16.0	84.1	95.5	OK
	B9	M20 8.8 - 1	LE1	1.0	2.9	179.4	0.7	3.0	3.5	OK
	B10	M20 8.8 - 1	LE1	1.6	3.0	181.8	1.1	3.1	4.0	OK
	B11	M20 8.8 - 1	LE1	2.6	1.6	225.2	1.8	1.7	3.1	OK
	B12	M20 8.8 - 1	LE1	0.8	2.4	293.5	0.6	2.6	3.0	OK
	B13	M20 8.8 - 1	LE1	6.9	39.8	193.8	4.9	42.3	45.8	OK
	B14	M20 8.8 - 1	LE1	9.7	40.4	193.8	6.9	43.0	47.9	OK
	B15	M20 8.8 - 1	LE1	10.4	39.1	193.8	7.4	41.6	46.8	OK
	B16	M20 8.8 - 1	LE1	14.0	39.2	193.8	9.9	41.7	48.7	OK
	B17	M20 8.8 - 1	LE1	1.6	5.0	178.2	1.1	5.4	6.2	OK
	B18	M20 8.8 - 1	LE1	1.0	5.3	180.5	0.7	5.6	6.1	OK
	B19	M20 8.8 - 1	LE1	2.9	7.2	178.2	2.0	7.7	9.1	OK
	B20	M20 8.8 - 1	LE1	0.8	7.2	176.3	0.6	7.7	8.1	OK
	B21	M20 8.8 - 1	LE1	2.1	16.6	193.8	1.5	17.6	18.6	OK
	B22	M20 8.8 - 1	LE1	3.3	16.7	193.8	2.3	17.8	19.4	OK
	B23	M20 8.8 - 1	LE1	3.2	10.6	193.8	2.3	11.3	12.9	OK
	B24	M20 8.8 - 1	LE1	4.8	10.5	193.8	3.4	11.2	13.6	OK
	B25	M20 8.8 - 1	LE1	5.3	33.4	193.8	3.8	35.5	38.2	OK
	B26	M20 8.8 - 1	LE1	4.9	33.0	193.8	3.5	35.1	37.6	OK
	B27	M20 8.8 - 1	LE1	7.5	34.8	193.8	5.3	37.0	40.8	OK
	B28	M20 8.8 - 1	LE1	8.3	34.0	193.8	5.9	36.2	40.4	OK

	B29	M20 8.8 - 1	LE1	3.2	5.6	178.2	2.3	5.9	7.6	OK
	B30	M20 8.8 - 1	LE1	2.1	5.7	179.5	1.5	6.1	7.2	OK
	B31	M20 8.8 - 1	LE1	2.2	4.5	178.2	1.5	4.8	5.9	OK
	B32	M20 8.8 - 1	LE1	2.2	4.4	177.1	1.6	4.7	5.8	OK

Design data

Grade	$F_{t,Rd}$ [kN]	$B_{p,Rd}$ [kN]	$F_{v,Rd}$ [kN]
M20 8.8 - 1	141.1	352.1	94.1

Symbol explanation

$F_{t,Ed}$	Tension force
$F_{v,Ed}$	Resultant of bolt shear forces Vy and Vz in shear planes
$F_{b,Rd}$	Plate bearing resistance EN 1993-1-8 – Tab. 3.4
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Interaction of tension and shear EN 1993-1-8 – Tab. 3.4
$F_{t,Rd}$	Bolt tension resistance EN 1993-1-8 – Tab. 3.4
$B_{p,Rd}$	Punching shear resistance EN 1993-1-8 – Tab. 3.4
$F_{v,Rd}$	Bolt shear resistance EN 1993-1-8 – Tab. 3.4

Detailed result for B5

Tension resistance check (EN 1993-1-8 – Table 3.4)

$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = 141.1 \text{ kN} \geq F_{t,Ed} = 21.3 \text{ kN}$$

Where:

$k_2 = 0.90$ – Factor

$f_{ub} = 800.0$ MPa – Ultimate tensile strength of the bolt

$A_s = 245$ mm^2 – Tensile stress area of the bolt

$\gamma_{M2} = 1.25$ – Safety factor

Punching resistance check (EN 1993-1-8 – Table 3.4)

$$B_{p,Rd} = \frac{0.6\, \pi\, d_m\, t_p f_u}{\gamma_{M2}} = 352.1 \text{ kN} \geq F_{t,Ed} = 21.3 \text{ kN}$$

Where:

$d_m = 32$ mm – The mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller

$t_p = 15$ mm – Plate thickness

$f_u = 490.0$ MPa – Ultimate strength

$\gamma_{M2} = 1.25$ – Safety factor

Shear resistance check (EN 1993-1-8 – Table 3.4)

$$F_{v,Rd} = \frac{\beta_p\, \alpha_v f_{ub} A}{\gamma_{M2}} = 94.1 \text{ kN} \geq F_{v,Ed} = 81.1 \text{ kN}$$

Where:

$\beta_p = 1.00$ – Reduction factor for packing

$\alpha_v = 0.60$ – Reduction factor for shear stress

$f_{ub} = 800.0$ MPa – Ultimate tensile strength of the bolt

$A = 245$ mm^2 – Tensile stress area of the bolt

$\gamma_{M2} = 1.25$ – Safety factor

Bearing resistance check (EN 1993-1-8 – Table 3.4)

$F_{b,Rd} = \frac{k_1\ \alpha_b f_u\ dt}{\gamma_{M2}} =$ 193.8 kN ≥ $F_{b,Ed} =$ 81.1 kN

Where:

$k_1 = \min(2.8\frac{e_2}{d_0} - 1.7, 1.4\frac{p_2}{d_0} - 1.7, 2.5) = 2.50$ – Factor for edge distance and bolt spacing perpendicular to the direction of load transfer

$\alpha_b = \min(\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u}, 1) = 0.66$ – Factor for end distance and bolt spacing in direction of load transfer

$e_2 = 60$ mm – Distance to the plate edge perpendicular to the shear force

$p_2 = 70$ mm – Distance between bolts perpendicular to the shear force

$d_0 = 22$ mm – Bolt hole diameter

$e_1 = 250$ mm – Distance to the plate edge in the direction of the shear force

$p_1 = 60$ mm – Distance between bolts in the direction of the shear force

$f_{ub} = 800.0$ MPa – Ultimate tensile strength of the bolt

$f_u = 490.0$ MPa – Ultimate strength of the plate

$d = 20$ mm – Nominal diameter of the fastener

$t = 15$ mm – Thickness of the plate

$\gamma_{M2} = 1.25$ – Safety factor

Utilization in tension

$\frac{F_{t,Ed}}{\min(F_{t,Rd}\,;\,B_{p,Rd})} =$ 0.15 ≤ 1.0

Where:

$F_{t,Ed} = 21.3$ kN – Tensile force

$F_{t,Rd} = 141.1$ kN – Tension resistance

$B_{p,Rd} = 352.1$ kN – Punching resistance

Utilization in shear

$\max(\frac{F_{v,Ed}}{F_{v,Rd}}\,;\,\frac{F_{b,Ed}}{F_{b,Rd}}) =$ 0.86 ≤ 1.0

Where:

$F_{v,Ed} = 81.1$ kN – Shear force (in decisive shear plane)

$F_{v,Rd} = 94.1$ kN – Shear resistance

$F_{b,Ed} = 81.1$ kN – Bearing force (for decisive plate)

$F_{b,Rd}$ = 193.8 kN – Bearing resistance

Interaction of tension and shear (EN 1993-1-8 – Table 3.4)

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4\,F_{t,Rd}} = \quad 0.97 \quad \leq \quad 1.0$$

Where:

$F_{v,Ed}$ = 81.1 kN – Shear force (in decisive shear plane)

$F_{v,Rd}$ = 94.1 kN – Shear resistance

$F_{t,Ed}$ = 21.3 kN – Tensile force

$F_{t,Rd}$ = 141.1 kN – Tension resistance

Welds

Item	Edge	T_w [mm]	L [mm]	Loads	$\sigma_{w,Ed}$ [MPa]	ε_{Pl} [%]	$\sigma_{\perp}$ [MPa]	$\tau_{\perp}$ [MPa]	$\tau_{\|\|}$ [MPa]	Ut [%]	Ut_c [%]	Status
SM1-arc 30	CPL1a	◢5.0◣	348	LE1	130.4	0.0	34.3	-45.5	56.6	29.9	14.9	OK
		◢5.0◣	348	LE1	111.3	0.0	-48.0	39.9	42.0	25.6	18.5	OK
CPL1b	M424-arc 8	◢5.0	130	LE1	30.1	0.0	7.5	-6.2	-15.6	6.9	6.9	OK
CPL1b	M424-arc 9	◢5.0	130	LE1	21.0	0.0	-1.2	1.0	12.0	4.8	4.8	OK
CPL1b	M424-arc 24	◢5.0	130	LE1	17.6	0.0	-2.8	3.2	9.5	4.0	4.0	OK
CPL1b	M424-arc 25	◢5.0	130	LE1	8.5	0.0	3.9	-2.1	3.8	2.0	0.0	OK
CPL4	M87-arc 19	◢5.0	130	LE1	427.0	0.1	116.0	36.4	-234.5	98.0	72.8	OK
CPL4	M87-arc 39	◢5.0	130	LE1	428.3	0.8	-192.1	133.5	-176.1	98.3	93.2	OK
CPL1a	SP1	◢5.0◣	159	LE1	427.5	0.4	-142.4	-130.7	-192.5	98.1	72.8	OK
		◢5.0◣	159	LE1	306.4	0.0	193.8	-58.0	-124.1	70.3	51.3	OK
SM1-arc 36	SP1	◢5.0◣	16	LE1	68.6	0.0	-1.4	-13.9	-37.1	15.8	15.8	OK
		◢5.0◣	16	LE1	95.7	0.0	-8.1	-7.5	54.6	22.0	22.0	OK
CPL5a	SP2	◢5.0◣	110	LE1	210.6	0.0	-35.5	-46.7	110.4	48.4	31.4	OK
		◢5.0◣	110	LE1	333.8	0.0	-106.4	93.3	-157.1	76.6	60.0	OK
CPL6a	SP3	◢5.0◣	125	LE1	357.4	0.0	-214.7	-160.8	36.8	82.1	42.2	OK
		◢5.0◣	124	LE1	121.2	0.0	-22.3	13.5	67.5	27.8	21.7	OK
CPL8a	SP3	◢5.0◣	147	LE1	253.3	0.0	-137.1	-96.2	76.6	58.1	48.1	OK
		◢5.0◣	147	LE1	152.0	0.0	-33.7	24.6	-82.0	34.9	29.5	OK
SM1-arc 35	SP1	◢5.0◣	16	LE1	98.9	0.0	0.9	-1.9	-57.1	22.7	22.7	OK

		◢ 5.0 ◣	16	LE1	120.9	0.0	-23.6	15.7	66.6	27.8	27.8	OK
SM1-arc 34	SP1	◢ 5.0 ◣	16	LE1	27.2	0.0	10.1	1.7	-14.5	6.2	6.2	OK
		◢ 5.0 ◣	16	LE1	26.7	0.0	-22.4	6.3	5.5	6.3	6.3	OK
SM1-arc 33	SP1	◢ 5.0 ◣	17	LE1	25.8	0.0	8.2	2.0	14.0	5.9	5.9	OK
		◢ 5.0 ◣	17	LE1	69.6	0.0	-17.1	5.4	-38.5	16.0	16.0	OK
SM1-arc 32	SP1	◢ 5.0 ◣	17	LE1	87.0	0.0	2.7	-0.9	50.2	20.0	20.0	OK
		◢ 5.0 ◣	17	LE1	138.5	0.0	-15.4	8.6	-79.0	31.8	31.8	OK
SM1-arc 31	SP1	◢ 5.0 ◣	15	LE1	67.8	0.0	4.1	12.0	37.2	15.6	15.6	OK
		◢ 5.0 ◣	15	LE1	98.0	0.0	17.5	-7.1	-55.2	22.5	22.5	OK
SM1-arc 37	CPL5a	◢ 5.0 ◣	328	LE1	164.1	0.0	-54.0	18.5	87.5	37.7	19.4	OK
		◢ 5.0 ◣	328	LE1	71.5	0.0	37.2	23.6	-26.1	16.4	9.8	OK
CPL5b	M423-arc 8	◢ 5.0	130	LE1	8.3	0.0	0.6	1.8	-4.4	1.9	0.0	OK
CPL5b	M423-arc 9	◢ 5.0	130	LE1	14.6	0.0	5.9	-2.9	7.2	3.4	3.4	OK
CPL5b	M423-arc 24	◢ 5.0	130	LE1	37.3	0.0	-11.7	6.0	19.5	8.6	8.6	OK
CPL5b	M423-arc 25	◢ 5.0	130	LE1	24.8	0.0	9.5	-3.8	12.7	5.7	5.7	OK
CPL5a	SP1	◢ 5.0 ◣	148	LE1	427.3	0.3	-198.5	-172.0	134.8	98.1	78.7	OK
		◢ 5.0 ◣	148	LE1	238.8	0.0	-49.6	38.5	-129.2	54.8	52.9	OK
SM1-arc 10	CPL6a	◢ 5.0 ◣	349	LE1	171.1	0.0	-84.3	-79.6	32.5	39.3	26.7	OK
		◢ 5.0 ◣	348	LE1	56.7	0.0	-14.4	9.8	-30.1	13.0	12.3	OK
CPL6b	M422-arc 8	◢ 5.0	130	LE1	426.9	0.0	-124.6	90.7	217.6	98.0	82.4	OK
CPL6b	M422-arc 9	◢ 5.0	130	LE1	233.3	0.0	52.6	11.0	-130.8	53.6	40.0	OK
CPL6b	M422-arc 24	◢ 5.0	130	LE1	221.2	0.0	53.9	10.8	123.4	50.8	40.2	OK
CPL6b	M422-arc 25	◢ 5.0	130	LE1	427.1	0.2	-140.2	92.6	-213.7	98.1	68.4	OK
CPL6a	SP2	◢ 5.0 ◣	137	LE1	188.8	0.0	2.4	18.1	-107.5	43.3	35.8	OK
		◢ 5.0 ◣	136	LE1	274.0	0.0	-69.1	69.1	136.6	62.9	50.6	OK
SM1-arc 10	SP2	◢ 5.0 ◣	6	LE1	124.6	0.0	3.3	14.8	70.4	28.6	28.6	OK
		◢ 5.0 ◣	6	LE1	124.9	0.0	7.9	4.0	-71.9	28.7	28.7	OK

SM1-arc 9	SP2	◢ 5.0 ◣	16	LE1	216.0	0.0	-2.2	1.3	124.7	49.6	48.8	OK
		◢ 5.0 ◣	16	LE1	219.4	0.0	13.7	-9.8	-126.0	50.4	49.5	OK
SM1-arc 8	SP2	◢ 5.0 ◣	16	LE1	179.3	0.0	-24.6	-20.0	100.6	41.2	40.6	OK
		◢ 5.0 ◣	16	LE1	171.2	0.0	2.1	3.3	-98.8	39.3	38.8	OK
SM1-arc 7	SP2	◢ 5.0 ◣	16	LE1	168.2	0.0	-47.5	-41.1	83.6	38.6	38.1	OK
		◢ 5.0 ◣	16	LE1	140.3	0.0	-19.5	26.7	-75.7	32.2	32.1	OK
SM1-arc 6	SP2	◢ 5.0 ◣	16	LE1	156.8	0.0	-60.9	-53.9	63.7	36.0	35.6	OK
		◢ 5.0 ◣	16	LE1	118.3	0.0	-33.9	41.6	-50.5	27.2	27.2	OK
SM1-arc 5	SP2	◢ 5.0 ◣	16	LE1	143.7	0.0	-68.6	-62.2	38.0	33.0	32.8	OK
		◢ 5.0 ◣	16	LE1	104.7	0.0	-43.0	50.0	-23.2	24.0	24.0	OK
SM1-arc 4	SP2	◢ 5.0 ◣	16	LE1	134.0	0.0	-70.8	-65.0	9.5	30.8	30.8	OK
		◢ 5.0 ◣	16	LE1	111.7	0.0	-51.1	57.2	3.3	25.6	25.6	OK
SM1-arc 3	SP2	◢ 5.0 ◣	16	LE1	135.4	0.0	-69.5	-65.0	-16.6	31.1	31.1	OK
		◢ 5.0 ◣	16	LE1	126.0	0.0	-55.0	60.0	26.2	28.9	28.9	OK
SM1-arc 2	SP2	◢ 5.0 ◣	16	LE1	143.2	0.0	-65.9	-62.4	-38.7	32.9	32.7	OK
		◢ 5.0 ◣	16	LE1	137.2	0.0	-53.4	57.2	45.3	31.5	31.5	OK
SM1-arc 1	SP2	◢ 5.0 ◣	16	LE1	151.0	0.0	-59.5	-56.3	-57.0	34.7	34.4	OK
		◢ 5.0 ◣	16	LE1	146.5	0.0	-47.1	50.4	62.2	33.6	33.4	OK
SM1-arc 40	SP2	◢ 5.0 ◣	16	LE1	158.8	0.0	-53.9	-48.6	-71.3	36.5	36.0	OK
		◢ 5.0 ◣	16	LE1	153.2	0.0	-32.0	37.5	78.0	35.2	34.8	OK
SM1-arc 39	SP2	◢ 5.0 ◣	16	LE1	155.7	0.0	-41.1	-32.1	-80.6	35.8	35.3	OK
		◢ 5.0 ◣	16	LE1	166.8	0.0	-11.6	20.8	93.8	38.3	37.8	OK
SM1-arc 38	SP2	◢ 5.0 ◣	16	LE1	131.0	0.0	-35.8	-7.5	-72.4	30.1	30.1	OK
		◢ 5.0 ◣	16	LE1	184.6	0.0	14.2	14.0	105.4	42.4	41.8	OK
SM1-arc 37	SP2	◢ 5.0 ◣	7	LE1	99.0	0.0	-60.8	27.9	-35.4	22.7	22.7	OK
		◢ 5.0 ◣	7	LE1	161.2	0.0	19.8	67.7	62.8	37.0	36.5	OK
CPL7	M86-arc 1	◢ 5.0	129	LE1	49.7	0.0	20.6	-13.0	-22.6	11.4	11.4	OK

CPL7	M86-arc 20	◢ 5.0	129	LE1	35.6	0.0	-10.9	-2.8	19.4	8.2	8.2	OK
CPL7	M86-arc 21	◢ 5.0	129	LE1	34.7	0.0	16.2	-7.7	-16.0	8.0	8.0	OK
CPL7	M86-arc 40	◢ 5.0	129	LE1	39.8	0.0	-9.9	0.2	22.2	9.1	9.1	OK
SM1-arc 19	CPL8a	◢ 5.0 ◣	368	LE1	77.1	0.0	-30.1	-25.8	-31.9	17.7	13.8	OK
		◢ 5.0 ◣	368	LE1	79.8	0.0	28.4	21.3	-37.5	18.3	11.5	OK
CPL8b	M421-arc 8	◢ 5.0	130	LE1	98.8	0.0	19.3	2.7	55.9	22.7	16.6	OK
CPL8b	M421-arc 9	◢ 5.0	130	LE1	159.6	0.0	-42.5	32.9	-82.5	36.7	24.7	OK
CPL8b	M421-arc 24	◢ 5.0	130	LE1	162.3	0.0	-50.8	31.9	83.1	37.3	30.6	OK
CPL8b	M421-arc 25	◢ 5.0	130	LE1	71.3	0.0	8.0	-10.4	39.5	16.4	14.7	OK
CPL9	M85-arc 1	◢ 5.0	129	LE1	209.8	0.0	50.6	6.3	-117.4	48.2	37.3	OK
CPL9	M85-arc 20	◢ 5.0	129	LE1	212.2	0.0	52.6	10.1	118.3	48.7	32.0	OK
CPL9	M85-arc 21	◢ 5.0	129	LE1	262.3	0.0	-97.0	63.4	-125.7	60.2	50.0	OK
CPL9	M85-arc 40	◢ 5.0	129	LE1	204.5	0.0	49.7	-0.2	114.5	46.9	41.2	OK
SM1-arc 18	SP3	◢ 5.0 ◣	16	LE1	74.4	0.0	-4.0	4.0	-42.7	17.1	17.1	OK
		◢ 5.0 ◣	16	LE1	84.7	0.0	10.0	0.3	48.6	19.5	19.5	OK
SM1-arc 17	SP3	◢ 5.0 ◣	16	LE1	115.3	0.0	-7.6	-7.3	-66.0	26.5	26.5	OK
		◢ 5.0 ◣	16	LE1	127.1	0.0	-6.3	7.0	73.0	29.2	29.2	OK
SM1-arc 16	SP3	◢ 5.0 ◣	16	LE1	87.1	0.0	1.0	-1.2	-50.3	20.0	20.0	OK
		◢ 5.0 ◣	16	LE1	91.6	0.0	-3.7	0.6	52.8	21.0	21.0	OK
SM1-arc 15	SP3	◢ 5.0 ◣	16	LE1	71.3	0.0	9.7	4.9	-40.5	16.4	16.4	OK
		◢ 5.0 ◣	16	LE1	61.4	0.0	-1.7	-5.4	35.0	14.1	14.1	OK
SM1-arc 14	SP3	◢ 5.0 ◣	16	LE1	57.2	0.0	17.3	11.0	-29.5	13.1	13.1	OK
		◢ 5.0 ◣	16	LE1	31.2	0.0	-0.5	-9.3	15.4	7.2	7.2	OK
SM1-arc 13	SP3	◢ 5.0 ◣	16	LE1	49.7	0.0	23.6	15.0	-20.3	11.4	11.4	OK
		◢ 5.0 ◣	16	LE1	19.5	0.0	-2.5	-11.1	-1.7	4.5	4.5	OK
SM1-arc 12	SP3	◢ 5.0 ◣	16	LE1	45.1	0.0	25.0	16.3	-14.3	10.3	10.3	OK
		◢ 5.0 ◣	16	LE1	31.0	0.0	-2.8	-10.9	-14.0	7.1	7.1	OK

SM1-arc 11	SP3	◢ 5.0 ◣	12	LE1	37.2	0.0	29.7	3.1	-12.6	8.5	8.5	OK
		◢ 5.0 ◣	12	LE1	52.3	0.0	-4.5	-29.6	-5.3	12.0	12.0	OK
CPL8a	SP4	◢ 5.0 ◣	103	LE1	142.0	0.0	0.9	-1.9	82.0	32.6	30.1	OK
		◢ 5.0 ◣	103	LE1	135.2	0.0	-15.8	14.8	-76.1	31.1	29.4	OK
CPL1a	SP4	◢ 5.0 ◣	105	LE1	213.9	0.0	8.3	-15.9	-122.4	49.1	39.9	OK
		◢ 5.0 ◣	105	LE1	296.7	0.0	-119.8	110.6	111.1	68.1	55.9	OK
SM1-arc 30	SP4	◢ 5.0 ◣	4	LE1	145.6	0.0	-26.1	30.8	76.8	33.4	33.2	OK
		◢ 5.0 ◣	4	LE1	92.7	0.0	16.8	33.2	-40.8	21.3	21.3	OK
SM1-arc 29	SP4	◢ 5.0 ◣	16	LE1	141.7	0.0	-16.1	5.8	81.1	32.5	32.4	OK
		◢ 5.0 ◣	16	LE1	124.3	0.0	3.5	15.2	-70.1	28.5	28.5	OK
SM1-arc 28	SP4	◢ 5.0 ◣	16	LE1	149.6	0.0	-7.1	-8.2	85.9	34.4	34.1	OK
		◢ 5.0 ◣	16	LE1	113.3	0.0	-25.3	26.2	-58.2	26.0	26.0	OK
SM1-arc 27	SP4	◢ 5.0 ◣	16	LE1	130.4	0.0	-17.7	-22.8	71.0	29.9	29.9	OK
		◢ 5.0 ◣	16	LE1	123.5	0.0	-50.6	49.8	-41.8	28.4	28.4	OK
SM1-arc 26	SP4	◢ 5.0 ◣	16	LE1	106.2	0.0	-20.9	-32.1	50.8	24.4	24.4	OK
		◢ 5.0 ◣	16	LE1	139.8	0.0	-67.9	63.6	-30.6	32.1	32.0	OK
SM1-arc 25	SP4	◢ 5.0 ◣	16	LE1	87.3	0.0	-26.1	-38.3	29.2	20.0	20.0	OK
		◢ 5.0 ◣	16	LE1	145.9	0.0	-74.5	69.7	-19.7	33.5	33.3	OK
SM1-arc 24	SP4	◢ 5.0 ◣	16	LE1	77.8	0.0	-29.3	-40.9	7.7	17.9	17.9	OK
		◢ 5.0 ◣	16	LE1	142.0	0.0	-74.0	69.4	-8.6	32.6	32.5	OK
SM1-arc 23	SP4	◢ 5.0 ◣	16	LE1	78.4	0.0	-30.2	-40.0	-12.1	18.0	18.0	OK
		◢ 5.0 ◣	16	LE1	128.7	0.0	-67.2	63.3	3.4	29.5	29.5	OK
SM1-arc 22	SP4	◢ 5.0 ◣	16	LE1	82.4	0.0	-28.1	-33.9	-29.1	18.9	18.9	OK
		◢ 5.0 ◣	16	LE1	106.8	0.0	-53.1	51.0	16.2	24.5	24.5	OK
SM1-arc 21	SP4	◢ 5.0 ◣	16	LE1	87.1	0.0	-16.7	-21.0	-44.7	20.0	20.0	OK
		◢ 5.0 ◣	16	LE1	88.0	0.0	-33.7	31.7	34.6	20.2	20.2	OK
SM1-arc 20	SP4	◢ 5.0 ◣	16	LE1	64.1	0.0	-14.2	-8.3	-35.1	14.7	14.7	OK

		◢ 5.0 ◣	16	LE1	67.6	0.0	-12.7	18.1	33.8	15.5	15.5	OK
SM1-arc 19	SP4	◢ 5.0 ◣	2	LE1	43.8	0.0	-18.1	-3.2	-22.8	10.1	10.1	OK
		◢ 5.0 ◣	2	LE1	25.3	0.0	-1.4	14.2	3.1	5.8	5.8	OK
CPL10	M71-arc 1	◢ 5.0	129	LE1	34.6	0.0	-6.7	10.9	-16.3	8.0	7.8	OK
CPL10	M71-arc 20	◢ 5.0	129	LE1	109.6	0.0	-40.7	26.2	52.6	25.2	21.9	OK
CPL10	M71-arc 21	◢ 5.0	129	LE1	148.9	0.0	58.4	-35.9	70.5	34.2	22.4	OK
CPL10	M71-arc 40	◢ 5.0	129	LE1	75.7	0.0	25.1	-20.4	-35.8	17.4	15.6	OK
CPL4	M87	◢ 5.0	259	LE1	428.2	0.8	-199.7	140.2	167.9	98.3	76.2	OK

Design data

Material	f_u [MPa]	β_w [-]	$\sigma_{w,Rd}$ [MPa]	0.9 σ [MPa]
S 355	490.0	0.90	435.6	352.8

Symbol explanation

T_w	Throat thickness a
L	Length
$\sigma_{w,Ed}$	Equivalent stress
ε_{Pl}	Strain
$\sigma_\perp$	Perpendicular stress
$\tau_\perp$	Shear stress perpendicular to weld axis
$\tau_{\|\|}$	Shear stress parallel to weld axis
Ut	Utilization
Ut_c	Weld capacity utilization
f_u	Ultimate strength of weld
β_w	Correlation factor EN 1993-1-8 – Tab. 4.1
$\sigma_{w,Rd}$	Equivalent stress resistance
0.9 σ	Perpendicular stress resistance: 0.9*fu/γM2
◢	Fillet weld

Detailed result for CPL4 / M87-arc 39

Weld resistance check (EN 1993-1-8 – Cl. 4.5.3.2)

$\sigma_{w,Rd} = f_u/(\beta_w \gamma_{M2}) =$ 435.6 MPa ≥ $\sigma_{w,Ed} = [\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)]^{0.5} =$ 428.3 MPa

$\sigma_{\perp,Rd} = 0.9 f_u / \gamma_{M2} =$ 352.8 MPa ≥ $|\sigma_\perp| =$ 192.1 MPa

where:

$f_u =$ 490.0 MPa – Ultimate strength

$\beta_w =$ 0.90 – Correlation factor EN 1993-1-8 – Tab. 4.1

γ_{M2} = 1.25 – Safety factor

Stress utilization

$U_t = \max(\frac{\sigma_{w,Ed}}{\sigma_{w,Rd}}; \frac{|\sigma_\perp|}{\sigma_{\perp,Rd}}) =$ 0.98 ≤ 1.0

Where:

$\sigma_{w,Ed}$ = 428.3 MPa – Maximum normal stress transverse to the axis of the weld

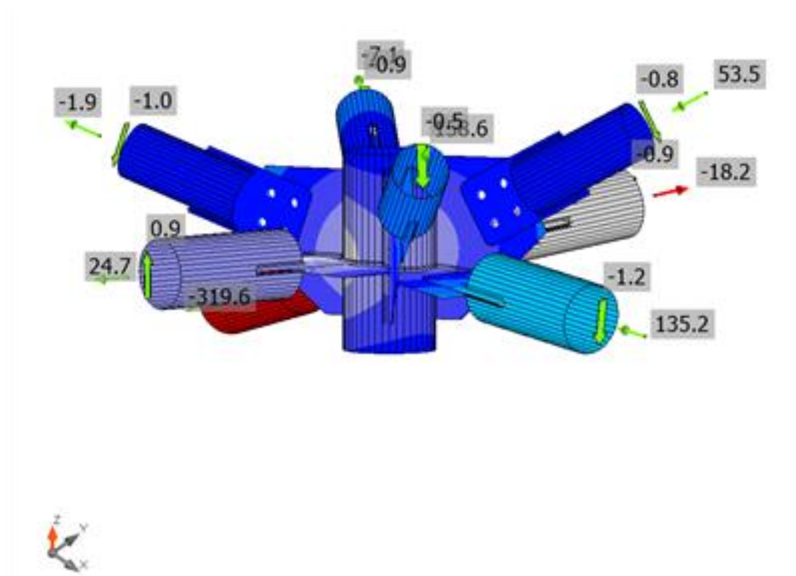
$\sigma_{w,Rd}$ = 435.6 MPa – Equivalent stress resistance

$\sigma_\perp$ = -192.1 MPa – Normal stress perpendicular to the throat

$\sigma_{\perp,Rd}$ = 352.8 MPa – Perpendicular stress resistance

Buckling

Loads	Shape	Factor [-]
LE1	1	7.81
	2	15.82
	3	18.28
	4	25.08
	5	32.31
	6	34.58



First buckling mode shape, LE1

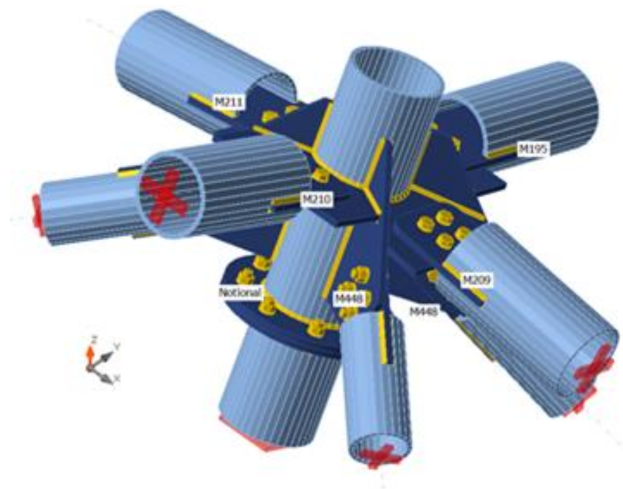
Detail Connection Check of Joint 2

Geometry

Name	Cross-section	β – Direction [°]	γ – Pitch [°]	α - Rotation[°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]
M195	53 - CHS193.7/10.0	-90.0	22.6	0.0	0	0	0
M209	53 - CHS193.7/10.0	-179.6	10.8	27.0	0	0	0
M210	53 - CHS193.7/10.0	-90.0	28.6	0.0	0	0	0
M211	53 - CHS193.7/10.0	179.6	-10.8	27.4	0	0	0
M415	52 - CHS139.7/10.0	21.5	-44.3	-29.5	0	0	0
M418	52 - CHS139.7/10.0	158.5	-44.3	29.0	0	0	0
M448	52 - CHS139.7/10.0	-41.4	-14.7	-23.5	0	0	0
M449	52 - CHS139.7/10.0	-138.6	-14.7	23.2	0	0	0
Notional	28 - CHS219.1/10.0	90.0	63.0	0.0	250	0	0

Supports and forces

Name	Support	Forces in	X [mm]
M195 / end	Mx-My-Mz	Position	0
M209 / end	Mx-My-Mz	Position	0
M210 / end	Mx-My-Mz	Position	0
M211 / end	Mx-My-Mz	Position	0
M415 / end	Mx-My-Mz	Position	0
M418 / end	Mx-My-Mz	Position	0
M448 / end	Mx-My-Mz	Position	0
M449 / end	Mx-My-Mz	Position	0
Notional / end	N-Vy-Vz-Mx-My-Mz	Position	250



Cross-sections

Name	Material
53 - CHS193.7/10.0	S 355
52 - CHS139.7/10.0	S 355
28 - CHS219.1/10.0	S 355

Bolts

Name	Bolt assembly	Diameter [mm]	f_u [MPa]	Gross area [mm^2]
M20 8.8	M20 8.8	20	800.0	314

Load effects (forces in equilibrium)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	M195 / Begin	-72.8	0.0	-1.2	0.0	0.0	0.0
	M209 / Begin	298.0	0.0	-2.0	0.0	0.0	0.0
	M210 / End	21.7	0.0	1.1	0.0	0.0	0.0
	M211 / End	-308.6	0.0	2.0	0.0	0.0	0.0
	M415 / End	-70.2	0.0	0.7	0.0	0.0	0.0
	M418 / End	-52.5	0.0	0.7	0.0	0.0	0.0
	M448 / End	-17.7	0.0	0.9	0.0	0.0	0.0
	M449 / End	-19.5	0.0	0.9	0.0	0.0	0.0
	Notional / Begin	-189.0	-0.7	-55.0	0.0	0.0	0.0

Unbalanced forces

Name	X [kN]	Y [kN]	Z [kN]	Mx [kNm]	My [kNm]	Mz[ ]
LE1	0.0	-0.5	0.0	0.0	0.2	0.1

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.7 < 5.0%	OK
Bolts	98.8 < 100%	OK
Welds	98.2 < 100%	OK
Buckling	13.63	
GMNA	Calculated	

Plates

Name	t_p [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [MPa]	Status
M195	10.0	LE1	47.9	0.0	0.0	OK
M209	10.0	LE1	250.4	0.0	0.0	OK
M210	10.0	LE1	39.9	0.0	0.0	OK
M211	10.0	LE1	324.2	0.7	0.0	OK
M415	10.0	LE1	105.0	0.0	0.0	OK
M418	10.0	LE1	93.0	0.0	0.0	OK
M448	10.0	LE1	94.6	0.0	0.0	OK
M449	10.0	LE1	96.3	0.0	0.0	OK
Notional	10.0	LE1	96.6	0.0	0.0	OK
STUB1	10.0	LE1	240.4	0.0	0.0	OK
SP1	15.0	LE1	179.9	0.0	9.0	OK
CPL5	15.0	LE1	92.6	0.0	9.0	OK
SP2	15.0	LE1	142.1	0.0	6.6	OK
CPL6	15.0	LE1	109.5	0.0	6.6	OK
SP3	15.0	LE1	322.9	0.1	36.7	OK
CPL7	15.0	LE1	322.8	0.0	37.7	OK
SP4	15.0	LE1	322.9	0.1	32.0	OK
CPL8	15.0	LE1	322.8	0.0	29.3	OK
STUB1-EPa	15.0	LE1	278.3	0.0	36.6	OK
STUB1-EPb	15.0	LE1	284.4	0.0	36.6	OK
CPL9a	15.0	LE1	178.8	0.0	11.3	OK
CPL9b	15.0	LE1	160.9	0.0	7.6	OK
CPL10a	15.0	LE1	139.5	0.0	8.0	OK
CPL10b	15.0	LE1	170.5	0.0	13.0	OK
CPL11a	15.0	LE1	128.1	0.0	7.1	OK
CPL11b	15.0	LE1	158.4	0.0	9.0	OK
CPL12a	15.0	LE1	166.3	0.0	11.8	OK
CPL12b	15.0	LE1	164.8	0.0	7.5	OK

Design data

Material	f_y [MPa]	ε_{lim} [%]
S 355	355.0	5.0

Symbol explanation

t_p Plate thickness
σ_{Ed} Equivalent stress
ε_{Pl} Plastic strain
$\sigma_{c,Ed}$ Contact stress
f_y Yield strength
ε_{lim} Limit of plastic strain

Detailed result for M211

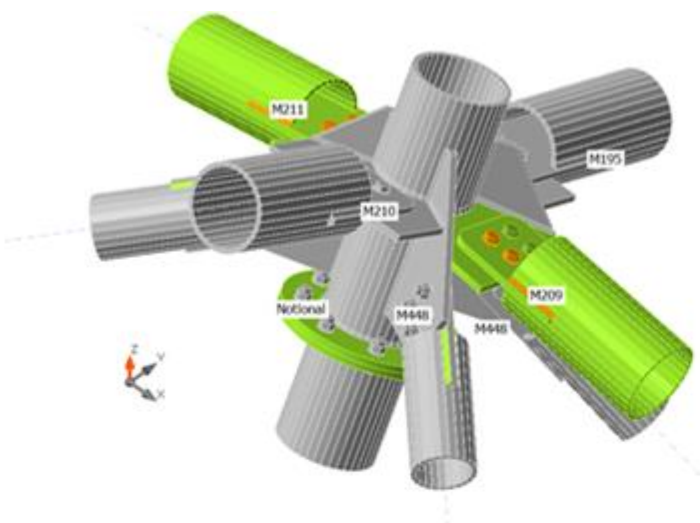
Design values used in the analysis

$f_{yd} = \frac{f_{yk}}{\gamma_{M0}} =$ 322.7 MPa

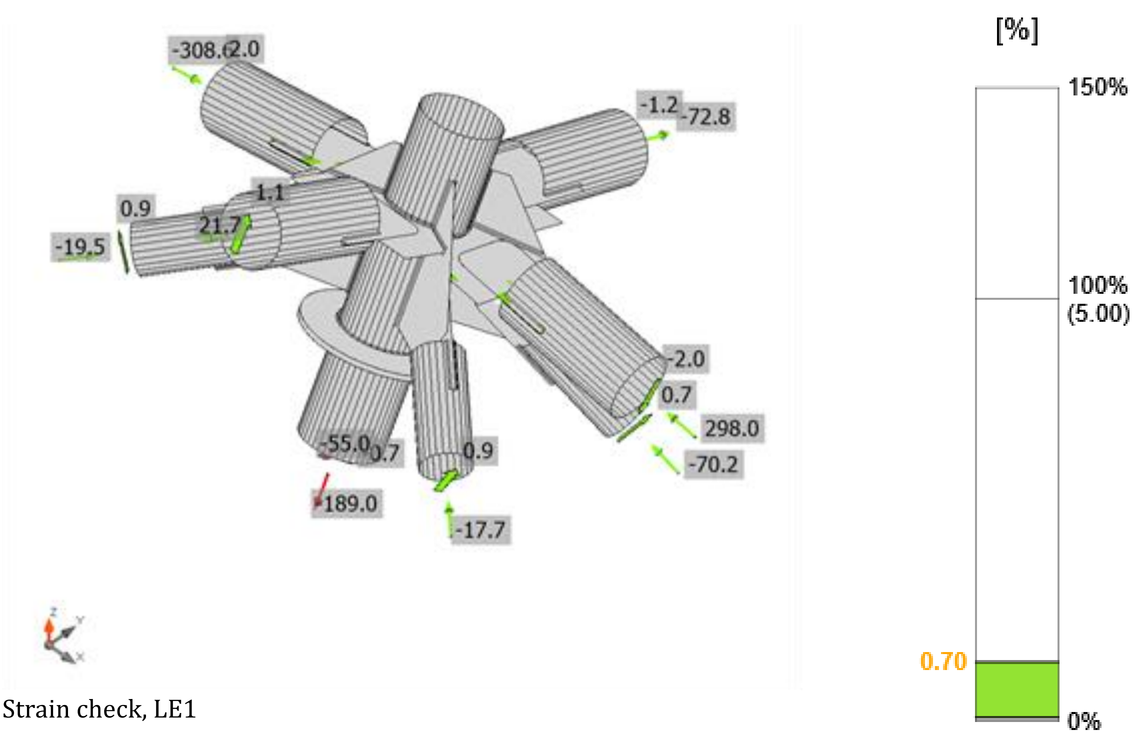
Where:

f_{yk} = 355.0 MPa – characteristic yield strength

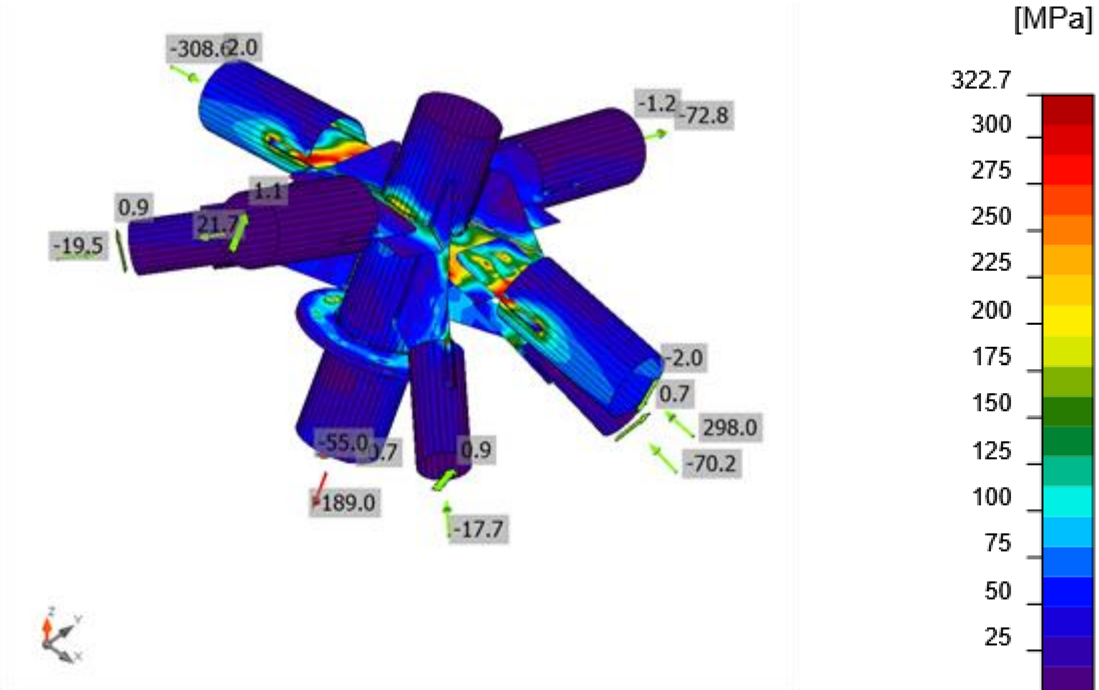
γ_{M0} = 1.10 – partial safety factor for steel material EN 1993-1-1 – 6.1



Overall check, LE1

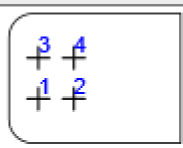
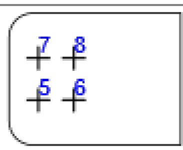
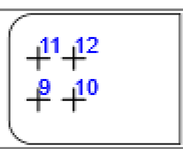
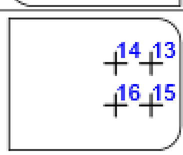
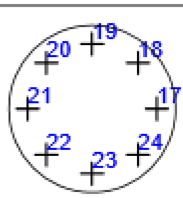


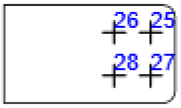
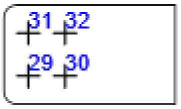
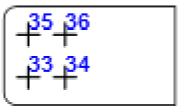
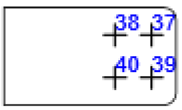
Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	$F_{t,Ed}$ [kN]	$F_{v,Ed}$ [kN]	$F_{b,Rd}$ [kN]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	M20 8.8 - 1	LE1	9.7	19.4	179.4	7.4	22.2	27.5	OK
	B2	M20 8.8 - 1	LE1	0.6	17.8	144.4	0.5	20.4	20.7	OK
	B3	M20 8.8 - 1	LE1	9.2	18.7	179.4	7.0	21.4	26.4	OK
	B4	M20 8.8 - 1	LE1	0.7	17.1	144.4	0.5	19.6	20.0	OK
	B5	M20 8.8 - 1	LE1	1.4	4.8	179.4	1.1	5.5	6.3	OK
	B6	M20 8.8 - 1	LE1	6.4	6.2	144.4	4.9	7.1	10.6	OK
	B7	M20 8.8 - 1	LE1	1.3	4.7	179.4	1.0	5.4	6.1	OK
	B8	M20 8.8 - 1	LE1	6.8	6.1	144.4	5.2	7.0	10.7	OK
	B9	M20 8.8 - 1	LE1	9.1	81.7	179.4	6.9	93.8	98.7	OK
	B10	M20 8.8 - 1	LE1	9.0	81.8	179.4	6.9	93.9	98.8	OK
	B11	M20 8.8 - 1	LE1	20.5	75.1	179.4	15.7	86.2	97.4	OK
	B12	M20 8.8 - 1	LE1	29.0	70.2	179.4	22.2	80.6	96.5	OK
	B13	M20 8.8 - 1	LE1	20.5	74.9	179.4	15.7	85.9	97.2	OK
	B14	M20 8.8 - 1	LE1	27.9	70.6	179.4	21.3	81.0	96.2	OK
	B15	M20 8.8 - 1	LE1	8.5	76.4	179.4	6.5	87.7	92.4	OK
	B16	M20 8.8 - 1	LE1	9.2	76.4	179.4	7.0	87.7	92.7	OK
	B17	M20 8.8 - 1	LE1	52.0	7.0	272.2	39.8	8.0	36.4	OK
	B18	M20 8.8 - 1	LE1	33.3	6.7	190.0	25.5	7.7	25.9	OK
	B19	M20 8.8 - 1	LE1	25.2	6.5	185.4	19.3	7.5	21.3	OK
	B20	M20 8.8 - 1	LE1	30.0	6.9	191.5	23.0	7.9	24.3	OK
	B21	M20 8.8 - 1	LE1	46.1	7.2	272.2	35.3	8.2	33.4	OK
	B22	M20 8.8 - 1	LE1	62.1	7.3	192.3	47.6	8.3	42.3	OK

	B23	M20 8.8 - 1	LE1	65.3	6.9	185.4	50.0	7.9	43.6	OK
	B24	M20 8.8 - 1	LE1	64.8	7.1	193.6	49.6	8.1	43.5	OK
	B25	M20 8.8 - 1	LE1	7.2	13.9	207.0	5.5	15.9	19.9	OK
	B26	M20 8.8 - 1	LE1	9.7	14.8	272.2	7.4	17.0	22.3	OK
	B27	M20 8.8 - 1	LE1	0.7	8.5	203.8	0.5	9.8	10.2	OK
	B28	M20 8.8 - 1	LE1	1.9	8.4	157.3	1.4	9.7	10.7	OK
	B29	M20 8.8 - 1	LE1	2.9	12.6	272.2	2.2	14.5	16.1	OK
	B30	M20 8.8 - 1	LE1	2.1	12.3	196.8	1.6	14.1	15.2	OK
	B31	M20 8.8 - 1	LE1	7.3	27.0	179.4	5.6	31.0	35.0	OK
	B32	M20 8.8 - 1	LE1	10.5	26.5	179.4	8.0	30.5	36.2	OK
	B33	M20 8.8 - 1	LE1	1.6	9.7	272.2	1.2	11.2	12.0	OK
	B34	M20 8.8 - 1	LE1	1.6	9.7	185.6	1.2	11.1	12.0	OK
	B35	M20 8.8 - 1	LE1	7.2	24.1	233.0	5.5	27.7	31.6	OK
	B36	M20 8.8 - 1	LE1	10.1	23.7	179.4	7.7	27.2	32.7	OK
	B37	M20 8.8 - 1	LE1	7.7	13.3	209.2	5.9	15.2	19.4	OK
	B38	M20 8.8 - 1	LE1	10.3	14.2	272.2	7.9	16.3	21.9	OK
	B39	M20 8.8 - 1	LE1	0.7	7.5	187.0	0.5	8.6	9.0	OK
	B40	M20 8.8 - 1	LE1	1.9	7.5	157.3	1.5	8.6	9.7	OK

Design data

Grade	$F_{t,Rd}$ [kN]	$B_{p,Rd}$ [kN]	$F_{v,Rd}$ [kN]
M20 8.8 - 1	130.7	326.0	87.1

Symbol explanation

$F_{t,Ed}$	Tension force
$F_{v,Ed}$	Resultant of bolt shear forces Vy and Vz in shear planes
$F_{b,Rd}$	Plate bearing resistance EN 1993-1-8 – Tab. 3.4
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Interaction of tension and shear EN 1993-1-8 – Tab. 3.4
$F_{t,Rd}$	Bolt tension resistance EN 1993-1-8 – Tab. 3.4
$B_{p,Rd}$	Punching shear resistance EN 1993-1-8 – Tab. 3.4
$F_{v,Rd}$	Bolt shear resistance EN 1993-1-8 – Tab. 3.4

Detailed result for B10

Tension resistance check (EN 1993-1-8 – Table 3.4)

$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} = 130.7 \text{ kN} \geq F_{t,Ed} = 9.0 \text{ kN}$$

Where:

$k_2 = 0.90$ – Factor

$f_{ub} = 800.0$ MPa – Ultimate tensile strength of the bolt

$A_s = 245$ mm^2 – Tensile stress area of the bolt

$\gamma_{M2} = 1.35$ – Safety factor

Punching resistance check (EN 1993-1-8 – Table 3.4)

$$B_{p,Rd} = \frac{0.6\,\pi\, d_m\, t_p f_u}{\gamma_{M2}} = 326.0 \text{ kN} \geq F_{t,Ed} = 9.0 \text{ kN}$$

Where:

d_m = 32 mm – The mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller

t_p = 15 mm – Plate thickness

f_u = 490.0 MPa – Ultimate strength

γ_{M2} = 1.35 – Safety factor

Shear resistance check (EN 1993-1-8 – Table 3.4)

$F_{v,Rd} = \frac{\beta_p\ \alpha_v f_{ub} A}{\gamma_{M2}} =$ 87.1 kN ≥ $F_{v,Ed} =$ 81.8 kN

Where:

β_p = 1.00 – Reduction factor for packing

α_v = 0.60 – Reduction factor for shear stress

f_{ub} = 800.0 MPa – Ultimate tensile strength of the bolt

A = 245 mm^2 – Tensile stress area of the bolt

γ_{M2} = 1.35 – Safety factor

Bearing resistance check (EN 1993-1-8 – Table 3.4)

$F_{b,Rd} = \frac{k_1\ \alpha_b f_u\ dt}{\gamma_{M2}} =$ 179.4 kN ≥ $F_{b,Ed} =$ 81.8 kN

Where:

$k_1 = \min(2.8\frac{e_2}{d_0} - 1.7, 1.4\frac{p_2}{d_0} - 1.7, 2.5) =$ 2.50 – Factor for edge distance and bolt spacing perpendicular to the direction of load transfer

$\alpha_b = \min(\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u}, 1) =$ 0.66 – Factor for end distance and bolt spacing in direction of load transfer

e_2 = 123 mm – Distance to the plate edge perpendicular to the shear force

p_2 = 70 mm – Distance between bolts perpendicular to the shear force

d_0 = 22 mm – Bolt hole diameter

e_1 = 139 mm – Distance to the plate edge in the direction of the shear force

p_1 = 60 mm – Distance between bolts in the direction of the shear force

f_{ub} = 800.0 MPa – Ultimate tensile strength of the bolt

f_u = 490.0 MPa – Ultimate strength of the plate

d = 20 mm – Nominal diameter of the fastener

t = 15 mm – Thickness of the plate

γ_{M2} = 1.35 – Safety factor

Utilization in tension

$\frac{F_{t,Ed}}{\min(F_{t,Rd}\ ;\ B_{p,Rd})} =$ 0.07 ≤ 1.0

Where:

$F_{t,Ed}$ = 9.0 kN – Tensile force

$F_{t,Rd}$ = 130.7 kN – Tension resistance

$B_{p,Rd}$ = 326.0 kN – Punching resistance

Utilization in shear

$\max(\frac{F_{v,Ed}}{F_{v,Rd}}; \frac{F_{b,Ed}}{F_{b,Rd}}) =$ 0.94 ≤ 1.0

Where:

$F_{v,Ed}$ = 81.8 kN – Shear force (in decisive shear plane)

$F_{v,Rd}$ = 87.1 kN – Shear resistance

$F_{b,Ed}$ = 81.8 kN – Bearing force (for decisive plate)

$F_{b,Rd}$ = 179.4 kN – Bearing resistance

Interaction of tension and shear (EN 1993-1-8 – Table 3.4)

$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4\,F_{t,Rd}} =$ 0.99 ≤ 1.0

Where:

$F_{v,Ed}$ = 81.8 kN – Shear force (in decisive shear plane)

$F_{v,Rd}$ = 87.1 kN – Shear resistance

$F_{t,Ed}$ = 9.0 kN – Tensile force

$F_{t,Rd}$ = 130.7 kN – Tension resistance

Welds

Item	Edge	T_w [mm]	L [mm]	Loads	$\sigma_{w,Ed}$ [MPa]	ε_{Pl} [%]	$\sigma_{\perp}$ [MPa]	$\tau_{\perp}$ [MPa]	$\tau_{\|\|}$ [MPa]	Ut [%]	Ut_c [%]	Status
CPL5	M195-arc 1	◢ 5.0	130	LE1	108.2	0.0	36.8	-30.0	50.5	26.8	22.8	OK
CPL5	M195-arc 16	◢ 5.0	130	LE1	100.4	0.0	33.5	-28.0	-46.9	24.9	22.3	OK
CPL5	M195-arc 17	◢ 5.0	130	LE1	92.5	0.0	-16.9	2.1	52.5	22.9	20.2	OK
CPL5	M195-arc 32	◢ 5.0	130	LE1	94.6	0.0	-17.4	2.0	-53.6	23.5	18.7	OK
CPL6	M210-arc 1	◢ 5.0	130	LE1	42.8	0.0	-9.0	14.6	19.2	10.6	10.4	OK
CPL6	M210-arc 16	◢ 5.0	130	LE1	53.3	0.0	-14.0	17.5	-23.9	13.2	12.7	OK
CPL6	M210-arc 17	◢ 5.0	130	LE1	103.0	0.0	33.5	-29.7	-47.7	25.5	22.1	OK
CPL6	M210-arc 32	◢ 5.0	130	LE1	91.3	0.0	28.3	-26.6	42.5	22.6	15.2	OK
CPL7	M211-arc 16	◢ 5.0	130	LE1	395.3	0.0	101.7	5.2	220.5	98.0	87.0	OK
CPL7	M211-arc 16	◢ 5.0	130	LE1	395.3	0.1	96.1	27.4	-219.7	98.0	75.7	OK
CPL7	M211-arc 32	◢ 5.0	130	LE1	395.4	0.1	116.4	41.7	214.1	98.0	67.6	OK
CPL7	M211-arc 32	◢ 5.0	130	LE1	395.6	0.3	-174.3	123.1	-164.0	98.1	84.1	OK

CPL8	M209-arc 1	◢ 5.0	130	LE1	395.3	0.0	115.2	28.7	-216.4	98.0	71.4	OK
CPL8	M209-arc 16	◢ 5.0	130	LE1	395.2	0.0	85.4	21.3	221.8	98.0	70.7	OK
CPL8	M209-arc 17	◢ 5.0	130	LE1	395.3	0.0	-132.3	109.0	-185.4	98.0	83.4	OK
CPL8	M209-arc 32	◢ 5.0	130	LE1	396.1	0.5	-167.0	109.6	176.1	98.2	88.2	OK
STUB1-arc 16	CPL9a	◢ 5.0 ◣	398	LE1	135.9	0.0	45.5	-33.4	65.9	33.7	25.2	OK
		◢ 5.0 ◣	398	LE1	102.6	0.0	3.3	-5.5	-58.9	25.4	21.6	OK
CPL9b	M448-arc 6	◢ 5.0	130	LE1	127.7	0.0	46.6	-23.7	64.4	31.7	20.8	OK
CPL9b	M448-arc 7	◢ 5.0	130	LE1	259.8	0.0	-78.3	50.3	133.9	64.4	53.8	OK
CPL9b	M448-arc 18	◢ 5.0	130	LE1	66.8	0.0	31.8	-7.9	33.0	16.6	12.7	OK
CPL9b	M448-arc 19	◢ 5.0	130	LE1	32.6	0.0	-30.8	0.4	6.2	9.4	8.5	OK
STUB1-arc 26	CPL10a	◢ 5.0 ◣	398	LE1	84.7	0.0	-67.6	-29.5	1.0	22.0	11.6	OK
		◢ 5.0 ◣	399	LE1	164.8	0.0	-96.2	71.8	28.4	40.9	28.9	OK
CPL10b	M415-arc 6	◢ 5.0	130	LE1	38.5	0.0	18.7	0.2	-19.4	9.5	9.5	OK
CPL10b	M415-arc 7	◢ 5.0	130	LE1	53.3	0.0	-37.9	9.0	19.6	13.2	13.2	OK
CPL10b	M415-arc 18	◢ 5.0	130	LE1	140.5	0.0	28.0	-3.5	-79.4	34.8	26.0	OK
CPL10b	M415-arc 19	◢ 5.0	130	LE1	300.2	0.0	-90.0	59.2	154.4	74.4	65.3	OK
STUB1-arc 35	CPL11a	◢ 5.0 ◣	399	LE1	145.7	0.0	43.7	-48.2	64.2	36.1	22.3	OK
		◢ 5.0 ◣	398	LE1	80.2	0.0	-71.0	7.1	20.3	21.7	10.0	OK
CPL11b	M418-arc 6	◢ 5.0	130	LE1	41.0	0.0	-33.1	5.8	-12.8	10.2	9.6	OK
CPL11b	M418-arc 7	◢ 5.0	130	LE1	16.1	0.0	6.3	1.1	-8.5	4.2	4.2	OK
CPL11b	M418-arc 18	◢ 5.0	130	LE1	279.2	0.0	-82.3	54.7	-144.0	69.2	40.2	OK
CPL11b	M418-arc 19	◢ 5.0	130	LE1	122.2	0.0	22.8	-3.8	69.2	30.3	21.6	OK
STUB1-arc 5	CPL12a	◢ 5.0 ◣	398	LE1	94.0	0.0	-44.2	-1.6	47.8	23.3	19.4	OK
		◢ 5.0 ◣	398	LE1	147.8	0.0	53.4	26.7	-75.0	36.7	21.5	OK
CPL12b	M449-arc 6	◢ 5.0	130	LE1	267.8	0.0	-82.1	52.5	-137.5	66.4	36.6	OK
CPL12b	M449-arc 7	◢ 5.0	130	LE1	138.2	0.0	49.7	-25.6	-69.9	34.3	27.2	OK
CPL12b	M449-arc 18	◢ 5.0	130	LE1	40.3	0.0	-34.6	2.6	-11.7	10.6	9.3	OK

CPL12b	M449-arc 19	◢ 5.0	130	LE1	64.2	0.0	32.3	-7.0	-31.2	15.9	14.8	OK
CPL10a	SP1	◢ 5.0 ◣	144	LE1	192.2	0.0	15.5	14.4	109.7	47.7	39.5	OK
		◢ 5.0 ◣	144	LE1	247.1	0.0	-119.0	109.3	-60.7	61.3	51.9	OK
CPL11a	SP1	◢ 5.0 ◣	144	LE1	218.1	0.0	13.3	9.4	-125.3	54.1	44.1	OK
		◢ 5.0 ◣	144	LE1	210.2	0.0	-97.9	99.1	41.3	52.1	48.7	OK
STUB1-arc 35	SP1	◢ 5.0 ◣	4	LE1	124.6	0.0	-29.8	-1.3	69.8	30.9	30.9	OK
		◢ 5.0 ◣	4	LE1	102.2	0.0	-1.6	27.1	-52.5	25.4	25.4	OK
STUB1-arc 34	SP1	◢ 5.0 ◣	16	LE1	125.1	0.0	-21.0	-9.1	70.6	31.0	31.0	OK
		◢ 5.0 ◣	16	LE1	129.6	0.0	-16.5	27.5	-68.9	32.1	32.1	OK
STUB1-arc 33	SP1	◢ 5.0 ◣	16	LE1	138.6	0.0	-15.5	-15.5	78.0	34.4	34.1	OK
		◢ 5.0 ◣	16	LE1	143.9	0.0	-40.3	42.0	-67.8	35.7	35.3	OK
STUB1-arc 32	SP1	◢ 5.0 ◣	16	LE1	104.4	0.0	-26.4	-27.9	51.2	25.9	25.9	OK
		◢ 5.0 ◣	16	LE1	142.6	0.0	-60.0	61.1	-42.9	35.4	35.0	OK
STUB1-arc 31	SP1	◢ 5.0 ◣	16	LE1	68.6	0.0	-28.8	-31.8	16.8	17.0	17.0	OK
		◢ 5.0 ◣	16	LE1	138.8	0.0	-67.9	68.1	-15.8	34.4	34.1	OK
STUB1-arc 30	SP1	◢ 5.0 ◣	16	LE1	72.4	0.0	-29.5	-32.0	-20.8	17.9	17.9	OK
		◢ 5.0 ◣	16	LE1	135.4	0.0	-66.1	66.6	15.0	33.6	33.4	OK
STUB1-arc 29	SP1	◢ 5.0 ◣	16	LE1	109.1	0.0	-27.8	-26.7	-54.8	27.1	27.1	OK
		◢ 5.0 ◣	16	LE1	133.4	0.0	-53.2	56.2	42.8	33.1	32.9	OK
STUB1-arc 28	SP1	◢ 5.0 ◣	16	LE1	142.5	0.0	-18.3	-15.7	-80.1	35.3	35.0	OK
		◢ 5.0 ◣	16	LE1	138.2	0.0	-30.8	34.1	69.9	34.3	34.0	OK
STUB1-arc 27	SP1	◢ 5.0 ◣	16	LE1	135.5	0.0	-24.9	-3.6	-76.8	33.6	33.4	OK
		◢ 5.0 ◣	16	LE1	138.9	0.0	-5.4	24.3	76.4	34.5	34.2	OK
STUB1-arc 26	SP1	◢ 5.0 ◣	4	LE1	139.9	0.0	-30.7	21.9	-75.7	34.7	34.4	OK
		◢ 5.0 ◣	4	LE1	100.6	0.0	9.5	38.1	43.6	25.0	25.0	OK
CPL12a	SP2	◢ 5.0 ◣	142	LE1	209.8	0.0	-21.8	-11.9	-119.9	52.0	40.3	OK
		◢ 5.0 ◣	142	LE1	154.4	0.0	-0.9	11.6	88.4	38.3	30.5	OK

CPL9a	SP2	◢ 5.0 ◣	118	LE1	208.6	0.0	-22.1	-12.3	119.1	51.7	46.4	OK
		◢ 5.0 ◣	118	LE1	191.9	0.0	-53.1	61.6	-86.8	47.6	38.1	OK
STUB1-arc 5	SP2	◢ 5.0 ◣	2	LE1	120.5	0.0	13.0	-53.1	44.3	29.9	29.9	OK
		◢ 5.0 ◣	2	LE1	124.2	0.0	-49.0	-19.7	-62.9	30.8	30.8	OK
STUB1-arc 6	SP2	◢ 5.0 ◣	16	LE1	127.5	0.0	7.3	-17.2	71.4	31.6	31.6	OK
		◢ 5.0 ◣	16	LE1	92.1	0.0	-36.0	10.1	-47.9	22.8	22.8	OK
STUB1-arc 7	SP2	◢ 5.0 ◣	16	LE1	139.1	0.0	-7.1	-7.2	79.9	34.5	34.2	OK
		◢ 5.0 ◣	16	LE1	118.8	0.0	-34.4	33.8	-56.3	29.5	29.5	OK
STUB1-arc 8	SP2	◢ 5.0 ◣	16	LE1	109.1	0.0	-17.2	-21.4	58.4	27.1	27.1	OK
		◢ 5.0 ◣	16	LE1	129.2	0.0	-58.2	52.9	-40.5	32.0	32.0	OK
STUB1-arc 9	SP2	◢ 5.0 ◣	16	LE1	84.3	0.0	-24.2	-26.9	38.1	20.9	20.9	OK
		◢ 5.0 ◣	16	LE1	134.8	0.0	-66.6	62.9	-24.9	33.4	33.2	OK
STUB1-arc 10	SP2	◢ 5.0 ◣	16	LE1	59.9	0.0	-24.7	-28.5	13.4	14.8	14.8	OK
		◢ 5.0 ◣	16	LE1	138.6	0.0	-72.5	67.7	-8.3	34.4	34.1	OK
STUB1-arc 11	SP2	◢ 5.0 ◣	16	LE1	59.0	0.0	-24.8	-28.2	-12.7	14.6	14.6	OK
		◢ 5.0 ◣	16	LE1	140.3	0.0	-73.0	68.5	9.2	34.8	34.5	OK
STUB1-arc 12	SP2	◢ 5.0 ◣	16	LE1	82.8	0.0	-23.5	-26.0	-37.8	20.5	20.5	OK
		◢ 5.0 ◣	16	LE1	136.4	0.0	-66.9	63.4	26.2	33.8	33.6	OK
STUB1-arc 13	SP2	◢ 5.0 ◣	16	LE1	109.9	0.0	-16.8	-20.9	-59.1	27.2	27.2	OK
		◢ 5.0 ◣	16	LE1	130.2	0.0	-57.4	52.3	42.6	32.3	32.2	OK
STUB1-arc 14	SP2	◢ 5.0 ◣	16	LE1	140.3	0.0	-5.3	-5.8	-80.7	34.8	34.5	OK
		◢ 5.0 ◣	16	LE1	121.3	0.0	-33.8	32.9	58.6	30.1	30.1	OK
STUB1-arc 15	SP2	◢ 5.0 ◣	16	LE1	133.5	0.0	6.6	-15.4	-75.4	33.1	32.9	OK
		◢ 5.0 ◣	16	LE1	99.7	0.0	-33.0	9.8	53.4	24.7	24.7	OK
STUB1-arc 16	SP2	◢ 5.0 ◣	2	LE1	125.8	0.0	9.2	-53.1	-49.3	31.2	31.2	OK
		◢ 5.0 ◣	2	LE1	132.7	0.0	-48.4	-16.3	69.5	32.9	32.8	OK
CPL11a	SP3	◢ 7.0 ◣	184	LE1	175.3	0.0	-3.0	-16.5	-99.8	43.5	38.4	OK

		◢ 7.0 ◣	184	LE1	166.4	0.0	-29.2	10.6	94.0	41.3	33.4	OK
CPL12a	SP3	◢ 5.0 ◣	159	LE1	373.3	0.0	-205.6	-179.4	12.7	92.6	75.4	OK
		◢ 5.0 ◣	159	LE1	229.1	0.0	-31.5	20.2	-129.4	56.8	51.7	OK
STUB1-arc 3	SP3	◢ 5.0 ◣	16	LE1	247.4	0.0	-8.0	-20.3	-141.3	61.4	59.5	OK
		◢ 5.0 ◣	16	LE1	219.6	0.0	5.2	-16.7	125.7	54.5	53.5	OK
STUB1-arc 2	SP3	◢ 5.0 ◣	16	LE1	193.3	0.0	-8.0	-24.1	-108.9	47.9	47.2	OK
		◢ 5.0 ◣	16	LE1	174.3	0.0	21.3	-34.8	93.6	43.2	42.6	OK
STUB1-arc 1	SP3	◢ 5.0 ◣	16	LE1	114.4	0.0	-7.6	-27.5	-59.9	28.4	28.4	OK
		◢ 5.0 ◣	16	LE1	131.4	0.0	29.6	-46.3	57.6	32.6	32.5	OK
STUB1-arc 40	SP3	◢ 5.0 ◣	16	LE1	50.0	0.0	-6.8	-27.9	6.6	12.4	12.4	OK
		◢ 5.0 ◣	16	LE1	87.6	0.0	28.8	-47.3	6.4	21.7	21.7	OK
STUB1-arc 39	SP3	◢ 5.0 ◣	17	LE1	129.3	0.0	-2.9	-21.3	71.5	32.1	32.0	OK
		◢ 5.0 ◣	17	LE1	110.1	0.0	20.2	-37.9	-49.7	27.3	27.3	OK
STUB1-arc 38	SP3	◢ 5.0 ◣	17	LE1	202.3	0.0	-5.1	-17.9	115.4	50.2	49.3	OK
		◢ 5.0 ◣	17	LE1	171.0	0.0	9.1	-21.4	-96.2	42.4	41.8	OK
STUB1-arc 37	SP3	◢ 5.0 ◣	17	LE1	253.6	0.0	-5.2	-13.4	145.8	62.9	60.8	OK
		◢ 5.0 ◣	17	LE1	231.2	0.0	2.1	-10.4	-133.1	57.3	56.2	OK
STUB1-arc 36	SP3	◢ 5.0 ◣	14	LE1	173.9	0.0	-7.7	18.3	98.6	43.1	42.5	OK
		◢ 5.0 ◣	14	LE1	164.8	0.0	23.7	6.4	-93.9	40.9	40.3	OK
STUB1-arc 4	SP3	◢ 5.0 ◣	16	LE1	156.1	0.0	-5.6	-0.8	-90.1	38.7	38.2	OK
		◢ 5.0 ◣	16	LE1	163.3	0.0	15.5	-6.4	93.6	40.5	39.9	OK
CPL9a	SP4	◢ 5.0 ◣	162	LE1	342.9	0.0	-173.3	-165.4	-42.7	85.0	64.5	OK
		◢ 5.0 ◣	162	LE1	236.9	0.0	-28.8	18.7	134.5	58.7	52.5	OK
CPL10a	SP4	◢ 5.0 ◣	182	LE1	217.6	0.0	-5.1	-28.9	122.3	54.0	48.7	OK
		◢ 5.0 ◣	182	LE1	217.9	0.0	-49.1	34.2	-117.7	54.0	43.2	OK
STUB1-arc 25	SP4	◢ 5.0 ◣	14	LE1	133.3	0.0	-4.7	19.6	-74.4	33.0	32.9	OK
		◢ 5.0 ◣	14	LE1	146.3	0.0	26.8	2.6	83.0	36.3	35.8	OK

STUB1-arc 24	SP4	◢ 5.0 ◣	17	LE1	199.3	0.0	-1.6	-5.8	-114.9	49.4	48.6	OK
		◢ 5.0 ◣	17	LE1	203.5	0.0	-1.5	-3.1	117.4	50.5	49.6	OK
STUB1-arc 23	SP4	◢ 5.0 ◣	16	LE1	151.3	0.0	4.2	-4.9	-87.2	37.5	37.0	OK
		◢ 5.0 ◣	16	LE1	145.5	0.0	3.1	-13.4	82.9	36.1	35.6	OK
STUB1-arc 22	SP4	◢ 5.0 ◣	16	LE1	93.1	0.0	8.6	-5.4	-53.2	23.1	23.1	OK
		◢ 5.0 ◣	16	LE1	81.9	0.0	8.7	-24.8	40.0	20.3	20.3	OK
STUB1-arc 21	SP4	◢ 5.0 ◣	16	LE1	17.0	0.0	6.5	-8.2	-3.9	4.2	4.2	OK
		◢ 5.0 ◣	16	LE1	55.2	0.0	13.2	-29.1	-10.6	13.7	13.7	OK
STUB1-arc 20	SP4	◢ 5.0 ◣	16	LE1	83.8	0.0	5.6	-9.2	47.4	20.8	20.8	OK
		◢ 5.0 ◣	16	LE1	110.1	0.0	12.1	-28.2	-56.5	27.3	27.3	OK
STUB1-arc 19	SP4	◢ 5.0 ◣	16	LE1	149.7	0.0	2.0	-11.1	85.7	37.1	36.6	OK
		◢ 5.0 ◣	16	LE1	159.1	0.0	7.3	-21.6	-89.2	39.5	38.9	OK
STUB1-arc 18	SP4	◢ 5.0 ◣	16	LE1	203.6	0.0	-2.1	-11.6	117.0	50.5	49.7	OK
		◢ 5.0 ◣	16	LE1	208.3	0.0	-5.5	-5.6	-120.1	51.7	50.8	OK
STUB1-arc 17	SP4	◢ 5.0 ◣	16	LE1	135.5	0.0	3.1	0.2	78.2	33.6	33.4	OK
		◢ 5.0 ◣	16	LE1	147.9	0.0	11.6	-13.9	-84.0	36.7	36.2	OK
STUB1-EPa	Notional	◢ 5.0	656	LE1	210.4	0.0	40.5	-13.5	-118.5	52.2	32.6	OK
STUB1-EPb	STUB1	◢ 5.0	656	LE1	204.2	0.0	121.5	-94.8	1.3	50.6	34.6	OK

Design data

Material	f_u [MPa]	β_w [-]	$\sigma_{w,Rd}$ [MPa]	0.9 σ [MPa]
S 355	490.0	0.90	403.3	326.7

Symbol explanation

T_w	Throat thickness a
L	Length
$\sigma_{w,Ed}$	Equivalent stress
ε_{Pl}	Strain
$\sigma_\perp$	Perpendicular stress
$\tau_\perp$	Shear stress perpendicular to weld axis
$\tau_{\|\|}$	Shear stress parallel to weld axis
Ut	Utilization
Ut_c	Weld capacity utilization
f_u	Ultimate strength of weld

β_w Correlation factor EN 1993-1-8 – Tab. 4.1
$\sigma_{w,Rd}$ Equivalent stress resistance
0.9 σ Perpendicular stress resistance: 0.9*fu/γM2
◢ Fillet weld

Detailed result for CPL8 / M209-arc 32

Weld resistance check (EN 1993-1-8 – Cl. 4.5.3.2)

$\sigma_{w,Rd} = f_u/(\beta_w \gamma_{M2}) =$ 403.3 MPa ≥ $\sigma_{w,Ed} = [\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)]^{0.5} =$ 396.1 MPa

$\sigma_{\perp,Rd} = 0.9 f_u / \gamma_{M2} =$ 326.7 MPa ≥ $|\sigma_\perp| =$ 167.0 MPa

where:

$f_u =$ 490.0 MPa – Ultimate strength

$\beta_w =$ 0.90 – Correlation factor EN 1993-1-8 – Tab. 4.1

$\gamma_{M2} =$ 1.35 – Safety factor

Stress utilization

$U_t = \max(\frac{\sigma_{w,Ed}}{\sigma_{w,Rd}} ; \frac{|\sigma_\perp|}{\sigma_{\perp,Rd}}) =$ 0.98 ≤ 1.0

Where:

$\sigma_{w,Ed} =$ 396.1 MPa – Maximum normal stress transverse to the axis of the weld

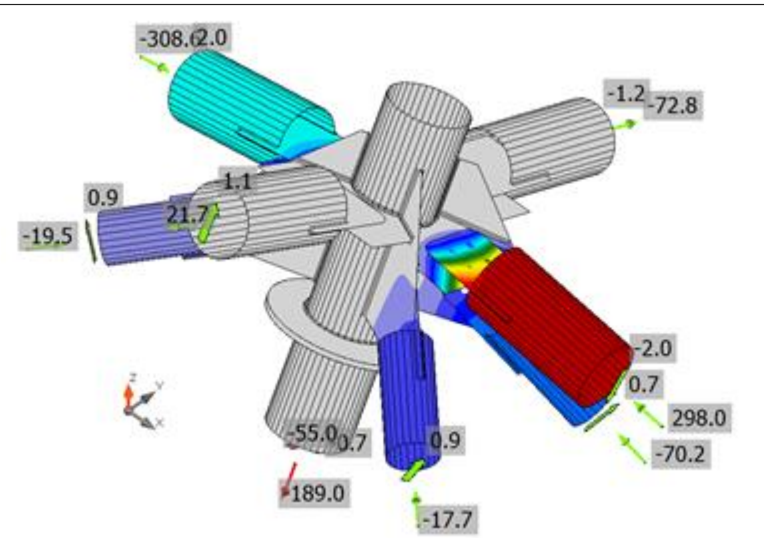
$\sigma_{w,Rd} =$ 403.3 MPa – Equivalent stress resistance

$\sigma_\perp =$ -167.0 MPa – Normal stress perpendicular to the throat

$\sigma_{\perp,Rd} =$ 326.7 MPa – Perpendicular stress resistance

Buckling

Loads	Shape	Factor [-]
LE1	1	13.63
	2	14.74
	3	37.34
	4	38.58
	5	40.60
	6	51.16



First buckling mode shape, LE1

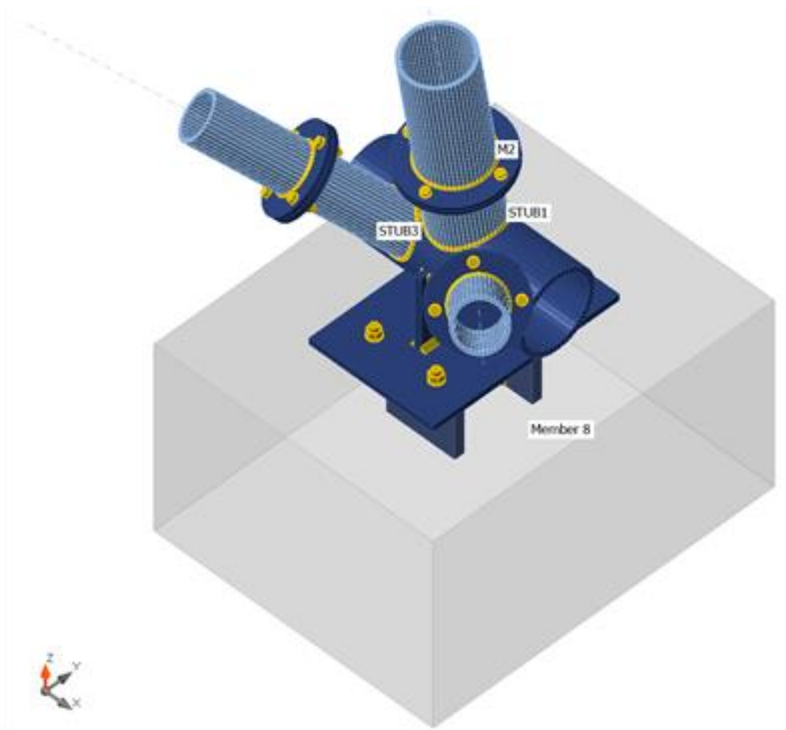
Detail Connection Check of Base 1

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]
M2	33 - CHS193.7/10.0	-78.2	79.0	0.0	0	0	0
M4	34 - CHS139.7/10.0	-44.0	35.2	6.0	0	0	0
M5	34 - CHS139.7/10.0	-135.2	37.1	0.0	0	0	0

Supports and forces

Name	Support	Forces in	X [mm]
M2 / end		Node	0
M4 / end		Node	0
M5 / end		Node	0



Cross-sections

Name	Material
33 - CHS193.7/10.0	S 355
34 - CHS139.7/10.0	S 355
17 - CHS244.5/20.0	S 355
30 - Iw280x220	S 355

Anchors / Bolts

Name	Bolt assembly	Diameter [mm]	f_u [MPa]	Gross area [mm²]
M16 8.8	M16 8.8	16	800.0	201
M24 8.8	M24 8.8	24	800.0	452

Load effects (forces in equilibrium)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	M2 / End	-417.1	0.0	0.0	0.0	0.0	0.0
	M4 / End	55.1	0.0	0.0	0.0	0.0	0.0
	M5 / End	-156.1	0.0	0.0	0.0	0.0	0.0
LE2	M2 / End	-248.2	0.0	0.0	0.0	0.0	0.0
	M4 / End	213.5	0.0	0.0	0.0	0.0	0.0
	M5 / End	-249.8	0.0	0.0	0.0	0.0	0.0

Unbalanced forces

Name	X [kN]	Y [kN]	Z [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	104.5	134.4	-471.8	0.0	0.0	0.0
LE2	257.2	65.6	-271.3	0.0	0.0	0.0

Foundation block

Item	Value	Unit
CB 1		
Dimensions	900 x 1100	mm
Depth	600	mm
Anchor	M24 8.8	
Anchoring length	120	mm
Shear force transfer	Shear lug	
Cross-section of shear lug	Iw280x220	
Length of shear lug	265	mm

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	1.3 < 5.0%	OK
Bolts	82.4 < 100%	OK
Anchors	81.2 < 100%	OK
Welds	98.9 < 100%	OK
Concrete block	97.5 < 100%	OK
Shear	17.4 < 100%	OK
Buckling	39.78	

Plates

Name	t_p [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [MPa]	Status
M2	10.0	LE1	79.0	0.0	0.0	OK
M4	10.0	LE2	141.6	0.0	0.0	OK
M5	10.0	LE2	67.4	0.0	0.0	OK
SM1	20.0	LE2	354.9	0.0	0.0	OK
STUB1	10.0	LE1	157.7	0.0	0.0	OK
STUB2	10.0	LE2	144.8	0.0	0.0	OK
STUB3	10.0	LE1	98.2	0.0	0.0	OK
Member 8-tfl 1	30.0	LE2	330.2	0.0	0.0	OK
Member 8-bfl 1	30.0	LE2	246.3	0.0	0.0	OK
Member 8-w 1	20.0	LE2	108.1	0.0	0.0	OK
STUB1-EPa	15.0	LE1	13.7	0.0	23.9	OK
STUB1-EPb	15.0	LE1	16.7	0.0	21.5	OK
STUB2-EPa	15.0	LE2	228.6	0.0	127.3	OK
STUB2-EPb	15.0	LE2	228.9	0.0	127.3	OK
STUB3-EPa	15.0	LE2	11.7	0.0	17.6	OK
STUB3-EPb	15.0	LE2	12.2	0.0	17.7	OK
SP3	15.0	LE2	355.3	0.1	0.0	OK
SP4	15.0	LE2	357.8	1.3	0.0	OK
SP5	20.0	LE2	299.7	0.0	0.0	OK
SP6	20.0	LE2	355.4	0.2	0.0	OK

Design data

Material	f_y [MPa]	ε_{lim} [%]
S 355	355.0	5.0

Symbol explanation

t_p	Plate thickness
σ_{Ed}	Equivalent stress
ε_{Pl}	Plastic strain
$\sigma_{c,Ed}$	Contact stress
f_y	Yield strength
ε_{lim}	Limit of plastic strain

Detailed result for SP4

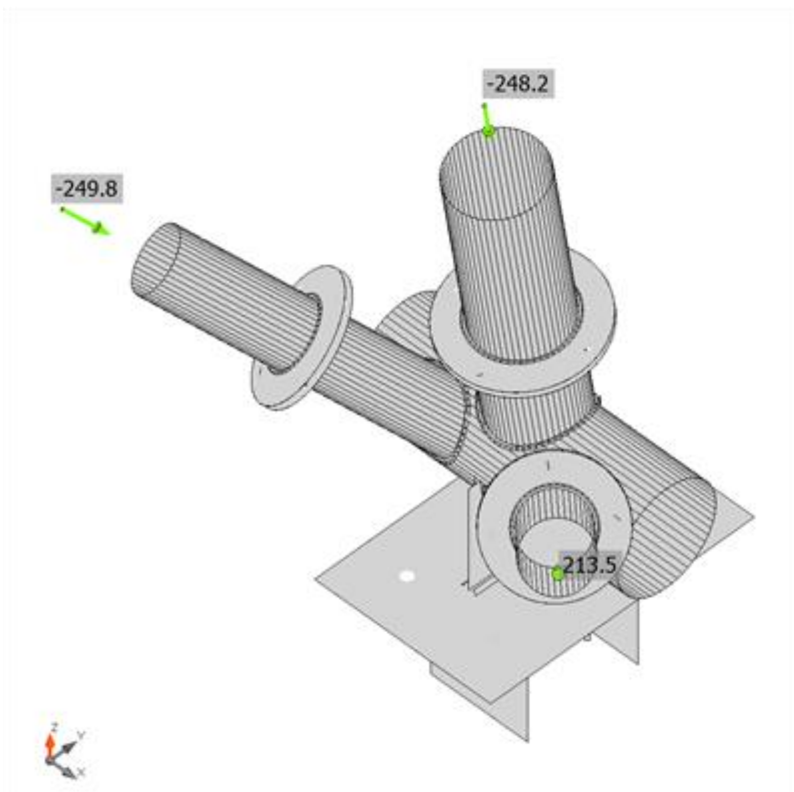
Design values used in the analysis

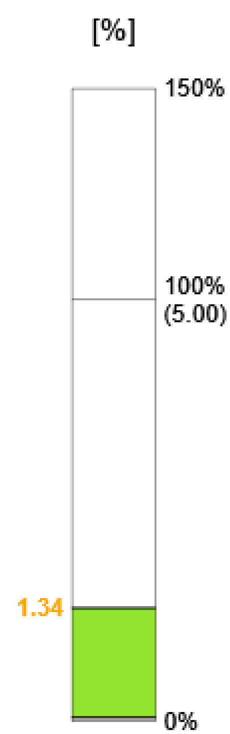
$f_{yd} = \frac{f_{yk}}{\gamma_{M0}} =$ 355.0 MPa

Where:

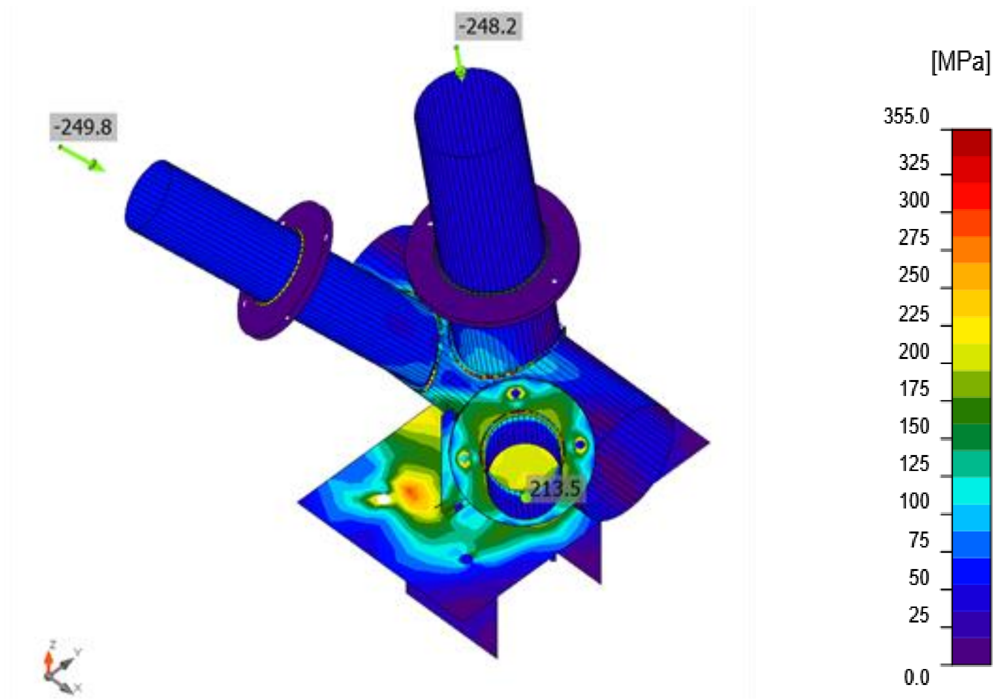
$f_{yk} =$ 355.0 MPa – characteristic yield strength

$\gamma_{M0} =$ 1.00 – partial safety factor for steel material EN 1993-1-1 – 6.1



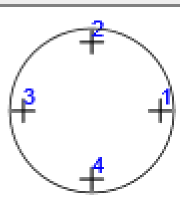
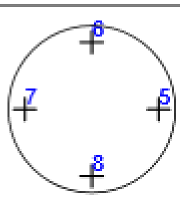
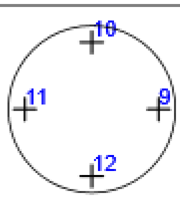


Strain check, LE2



Equivalent stress, LE2

Bolts

Shape	Item	Grade	Loads	$F_{t,Ed}$ [kN]	$F_{v,Ed}$ [kN]	$F_{b,Rd}$ [kN]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	M16 8.8 - 1	LE1	0.1	0.1	108.8	0.1	0.2	0.2	OK
	B2	M16 8.8 - 1	LE1	0.2	0.1	108.8	0.2	0.2	0.3	OK
	B3	M16 8.8 - 1	LE1	0.2	0.1	104.8	0.2	0.2	0.3	OK
	B4	M16 8.8 - 1	LE1	0.3	0.1	101.3	0.3	0.2	0.4	OK
	B5	M16 8.8 - 1	LE2	74.4	0.0	97.7	82.3	0.1	58.8	OK
	B6	M16 8.8 - 1	LE2	74.5	0.0	150.9	82.4	0.0	58.9	OK
	B7	M16 8.8 - 1	LE2	74.5	0.0	95.2	82.3	0.0	58.8	OK
	B8	M16 8.8 - 1	LE2	74.5	0.0	95.1	82.4	0.0	58.9	OK
	B9	M16 8.8 - 1	LE2	0.3	0.0	128.4	0.3	0.0	0.3	OK
	B10	M16 8.8 - 1	LE2	0.3	0.0	145.9	0.3	0.0	0.2	OK
	B11	M16 8.8 - 1	LE2	0.3	0.0	144.5	0.3	0.0	0.3	OK
	B12	M16 8.8 - 1	LE2	0.3	0.0	149.3	0.3	0.0	0.2	OK

Design data

Grade	$F_{t,Rd}$ [kN]	$B_{p,Rd}$ [kN]	$F_{v,Rd}$ [kN]
M16 8.8 - 1	90.4	281.2	60.3

Symbol explanation

$F_{t,Ed}$ Tension force

$F_{v,Ed}$ Resultant of bolt shear forces Vy and Vz in shear planes
$F_{b,Rd}$ Plate bearing resistance EN 1993-1-8 – Tab. 3.4
Ut_t Utilization in tension
Ut_s Utilization in shear
Ut_{ts} Interaction of tension and shear EN 1993-1-8 – Tab. 3.4
$F_{t,Rd}$ Bolt tension resistance EN 1993-1-8 – Tab. 3.4
$B_{p,Rd}$ Punching shear resistance EN 1993-1-8 – Tab. 3.4
$F_{v,Rd}$ Bolt shear resistance EN 1993-1-8 – Tab. 3.4

Detailed result for B8

Tension resistance check (EN 1993-1-8 – Table 3.4)

$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} =$ 90.4 kN ≥ $F_{t,Ed} =$ 74.5 kN

Where:

$k_2 = 0.90$ – Factor
$f_{ub} = 800.0$ MPa – Ultimate tensile strength of the bolt
$A_s = 157\ mm^2$ – Tensile stress area of the bolt
$\gamma_{M2} = 1.25$ – Safety factor

Punching resistance check (EN 1993-1-8 – Table 3.4)

$B_{p,Rd} = \frac{0.6\ \pi\ d_m\ t_p f_u}{\gamma_{M2}} =$ 281.2 kN ≥ $F_{t,Ed} =$ 74.5 kN

Where:

$d_m = 25$ mm – The mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller
$t_p = 15$ mm – Plate thickness
$f_u = 490.0$ MPa – Ultimate strength
$\gamma_{M2} = 1.25$ – Safety factor

Shear resistance check (EN 1993-1-8 – Table 3.4)

$F_{v,Rd} = \frac{\beta_p\ \alpha_v f_{ub} A}{\gamma_{M2}} =$ 60.3 kN ≥ $F_{v,Ed} =$ 0.0 kN

Where:

$\beta_p = 1.00$ – Reduction factor for packing
$\alpha_v = 0.60$ – Reduction factor for shear stress
$f_{ub} = 800.0$ MPa – Ultimate tensile strength of the bolt
$A = 157\ mm^2$ – Tensile stress area of the bolt
$\gamma_{M2} = 1.25$ – Safety factor

Bearing resistance check (EN 1993-1-8 – Table 3.4)

$F_{b,Rd} = \frac{k_1\ \alpha_b f_u\ dt}{\gamma_{M2}} =$ 95.1 kN ≥ $F_{b,Ed} =$ 0.0 kN

Where:

$k_1 = \min(2.8\frac{e_2}{d_0} - 1.7, 1.4\frac{p_2}{d_0} - 1.7, 2.5) = 2.19$ – Factor for edge distance and bolt spacing perpendicular to the direction of load transfer

$\alpha_b = \min(\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u}, 1) = 0.46$ – Factor for end distance and bolt spacing in direction of load transfer

$e_2 = 25$ mm – Distance to the plate edge perpendicular to the shear force

$p_2 = \infty$ mm – Distance between bolts perpendicular to the shear force

$d_0 = 18$ mm – Bolt hole diameter

$e_1 = 25$ mm – Distance to the plate edge in the direction of the shear force

$p_1 = \infty$ mm – Distance between bolts in the direction of the shear force

$f_{ub} = 800.0$ MPa – Ultimate tensile strength of the bolt

$f_u = 490.0$ MPa – Ultimate strength of the plate

$d = 16$ mm – Nominal diameter of the fastener

$t = 15$ mm – Thickness of the plate

$\gamma_{M2} = 1.25$ – Safety factor

Utilization in tension

$\frac{F_{t,Ed}}{\min(F_{t,Rd}\,;\,B_{p,Rd})} = 0.82 \leq 1.0$

Where:

$F_{t,Ed} = 74.5$ kN – Tensile force

$F_{t,Rd} = 90.4$ kN – Tension resistance

$B_{p,Rd} = 281.2$ kN – Punching resistance

Utilization in shear

$\max(\frac{F_{v,Ed}}{F_{v,Rd}}; \frac{F_{b,Ed}}{F_{b,Rd}}) = 0.00 \leq 1.0$

Where:

$F_{v,Ed} = 0.0$ kN – Shear force (in decisive shear plane)

$F_{v,Rd} = 60.3$ kN – Shear resistance

$F_{b,Ed} = 0.0$ kN – Bearing force (for decisive plate)

$F_{b,Rd} = 95.1$ kN – Bearing resistance

Interaction of tension and shear (EN 1993-1-8 – Table 3.4)

$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4\,F_{t,Rd}} = 0.59 \leq 1.0$

Where:

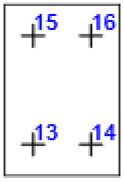
$F_{v,Ed} = 0.0$ kN – Shear force (in decisive shear plane)

$F_{v,Rd} = 60.3$ kN – Shear resistance

$F_{t,Ed} = 74.5$ kN – Tensile force

$F_{t,Rd} = 90.4$ kN – Tension resistance

Anchors

Shape	Item	Loads	N_{Ed} [kN]	$V_{Rd,cp}$ [kN]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	A13	LE2	130.0	115.0	81.2	0.3	66.0	OK
	A14	LE1	6.7	115.0	4.2	0.1	0.2	OK
	A15	LE2	105.5	115.0	65.9	0.4	43.5	OK
	A16	LE2	0.0	115.0	0.0	0.4	0.0	OK

Design data

Grade	$N_{Rd,s}$ [kN]	$V_{Rd,s}$ [kN]
M24 8.8 - 2	160.0	113.0

Symbol explanation

N_{Ed}	Tension force
$V_{Rd,cp}$	Design resistance in case of concrete pryout failure - EN 1992-4 – 7.2.2.4
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
$N_{Rd,s}$	Design tensile resistance of a fastener in case of steel failure - EN 1992-4 – 7.2.1.3
$V_{Rd,s}$	Design shear resistance of a fastener in case of steel failure - EN 1992-4 – 7.2.2.3.1

Detailed result for A13

Following checks of anchors loaded in tension are not provided and will be checked using Hilti PROFIS Engineering software:

- Pull-out failure of fastener (for post-installed mechanical anchors) - EN 1992-4 – 7.2.1.5
- Combined pull-out and concrete failure (for post-installed bonded anchors) - EN 1992-4 – 7.2.1.6
- Concrete splitting failure - EN 1992-4 – 7.2.1.7

Concrete blow-out failure is provided only for anchors with washer plates.
Anchor tensile resistance (EN 1992-4 – 7.2.1.3)

$$N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}} = \quad 160.0 \quad \text{kN} \quad \geq \quad N_{Ed} = \quad 130.0 \quad \text{kN}$$

$$N_{Rk,s} = c \cdot A_s \cdot f_{uk} = \quad 240.0 \quad \text{kN}$$

Where:

$c = 0.85$	– reduction factor for cut thread
$A_s = 353 \text{ mm}^2$	– tensile stress area
$f_{uk} = 800.0 \text{ MPa}$	– minimum tensile strength of the bolt
$\gamma_{Ms} = 1.50$	– safety factor for steel

$$\gamma_{Ms} = 1.2 \cdot \frac{f_{uk}}{f_{yk}} \geq 1.4$$

, where:
$f_{yk} =$
640.0 MPa – minimum yield strength of the bolt

Shear resistance (EN 1992-4 – 7.2.2.3.1)

$V_{Rd,s} =$ 113.0 kN ≥ $V_{Ed} =$ 0.4 kN

$V_{Rk,s} = k_7 \cdot V^0_{Rk,s} =$ 141.2 kN

Where:

$k_7 = 1.00$ – coefficient for anchor steel ductility

$$k_7 = \begin{cases} 0.8, & A < 0.08 \\ 1.0, & A \geq 0.08 \end{cases}$$

, where:
$A =$
0.12 – bolt grade elongation at rupture

$V^0_{Rk,s} = 141.2$ kN – the characteristic shear strength

$V^0_{Rk,s} = k_6 \cdot A_s \cdot f_{uk}$
, where:
$k_6 =$
0.50 – coefficient for anchor resistance in shear
$A_s =$
353 mm^2 – tensile stress area
$f_{uk} =$
800.0 MPa – specified ultimate strength of anchor steel

$\gamma_{Ms} = 1.25$ – safety factor for steel

Interaction of tensile and shear forces in steel (EN 1992-4 – Table 7.3)

$(\frac{N_{Ed}}{N_{Rd,s}})^2 +$ 0.66 ≤ 1.0

Where:

$N_{Ed} = 130.0$ kN – design tension force

$N_{Rd,s} = 160.0$ kN – fastener tensile strength

$V_{Ed} = 0.4$ kN – design shear force

$V_{Rd,s} = 113.0$ kN – fastener shear strength

Interaction of tensile and shear forces in concrete (EN 1992-4 – Table 7.3)

$(\frac{N_{Ed}}{N_{Rd,i}})^{1.5} +$ 0.00 ≤ 1.0

Where:

$\frac{N_{Ed}}{N_{Rd,i}}$ – the largest utilization value for tension failure modes

$\frac{V_{Ed}}{V_{Rd,i}}$ – the largest utilization value for shear failure modes

$\frac{N_{Ed,g}}{N_{Rd,c}} = 0\%$ – concrete breakout failure of anchor in tension

$\frac{N_{Ed}}{N_{Rd,p}} = 0\%$ – concrete pullout failure

$\frac{N_{Ed}}{N_{Rd,cb}} = 0\%$ – concrete blowout failure

$\frac{V_{Ed}}{V_{Rd,c}} = 0\%$ – concrete edge failure

$\frac{V_{Ed}}{V_{Rd,cb}} = 0\%$ – concrete pryout failure

Supplementary reinforcement (EN 1992-4 – 7.2.1.9)
Supplementary reinforcement should resist force of 239.4 kN in tension.

Welds

Item	Edge	Material	T_w [mm]	L [mm]	Loads	$\sigma_{w,Ed}$ [MPa]	ε_{Pl} [%]	$\sigma_{\perp}$ [MPa]	$\tau_{\perp}$ [MPa]	$\tau_{\|\|}$ [MPa]	Ut [%]	Ut_c [%]	Status
SM1-arc 42	SP3	S 355	◢10.0◣	68	LE2	120.3	0.0	72.5	53.3	15.2	27.6	22.3	OK
		S 355	◢10.0◣	68	LE2	201.0	0.0	162.7	-64.9	-20.9	46.2	40.5	OK
SM1-arc 42	SP4	S 355	◢10.0◣	68	LE2	342.3	0.0	-158.9	-139.1	106.3	78.6	63.8	OK
		S 355	◢10.0◣	68	LE2	379.9	0.0	-140.5	163.2	-122.0	87.2	78.9	OK
SP5	SP4	S 355	◢10.0◣	99	LE2	426.9	0.0	-154.2	-146.3	177.2	98.0	70.4	OK
		S 355	◢10.0◣	99	LE2	426.9	0.0	-133.2	140.9	-187.1	98.0	70.2	OK
SP5	SP3	S 355	◢5.0◣	99	LE2	427.2	0.2	203.9	190.0	104.2	98.1	76.1	OK
		S 355	◢5.0◣	99	LE2	331.1	0.0	105.1	-132.6	-123.5	76.0	50.6	OK
Member 8-tfl 1	Member 8-w 1	S 355	◢10.0◣	265	LE2	57.7	0.0	0.0	-33.2	-2.5	13.2	13.2	OK
		S 355	◢10.0◣	264	LE2	98.1	0.0	-62.6	35.9	24.9	22.5	11.6	OK
Member 8-bfl 1	Member 8-w 1	S 275	◢10.0◣	264	LE1	81.0	0.0	24.9	-19.0	40.3	20.0	10.1	OK
		S 275	◢10.0◣	264	LE2	78.8	0.0	-54.3	-26.7	-19.3	19.5	13.4	OK
SP5	Member 8-tfl 1	S 355	◢10.0◣	220	LE2	426.9	0.0	-276.7	-141.0	123.9	98.0	78.7	OK
		S 355	◢10.0◣	220	LE2	267.3	0.0	95.9	-106.9	-96.5	61.4	47.0	OK

SP5	Member 8-bfl 1	S 355	◢ 10.0 ◣	220	LE2	427.2	0.2	142.3	159.3	169.5	98.1	85.1	OK
		S 355	◢ 10.0 ◣	220	LE2	427.7	0.5	174.3	-164.5	-154.2	98.2	89.1	OK
SP5	Member 8-w 1	S 355	◢ 10.0 ◣	220	LE2	98.6	0.0	-55.1	-24.9	40.1	22.6	19.4	OK
		S 355	◢ 10.0 ◣	219	LE2	85.1	0.0	26.7	-13.7	-44.6	19.5	12.4	OK
SM1-arc 28	SP6	S 355	◢ 5.0 ◣	4	LE1	427.0	0.1	129.6	118.3	202.9	98.0	89.4	OK
		S 355	◢ 5.0 ◣	4	LE1	426.9	0.1	111.7	-125.7	-202.0	98.0	89.3	OK
SM1-arc 29	SP6	S 355	◢ 5.0 ◣	13	LE1	112.3	0.0	34.1	25.6	56.2	25.8	25.8	OK
		S 355	◢ 5.0 ◣	13	LE1	60.2	0.0	12.2	-20.8	-27.0	13.8	13.8	OK
SM1-arc 30	SP6	S 355	◢ 5.0 ◣	13	LE1	93.9	0.0	-43.4	-46.1	13.6	21.6	21.6	OK
		S 355	◢ 5.0 ◣	13	LE2	124.1	0.0	-45.1	40.2	53.3	28.5	28.5	OK
SM1-arc 31	SP6	S 355	◢ 5.0 ◣	13	LE1	109.5	0.0	-50.9	-53.8	-15.5	25.1	25.1	OK
		S 355	◢ 5.0 ◣	13	LE1	177.9	0.0	-71.0	68.1	65.1	40.8	40.3	OK
SM1-arc 32	SP6	S 355	◢ 5.0 ◣	13	LE1	138.2	0.0	-51.4	-53.9	-50.8	31.7	31.7	OK
		S 355	◢ 5.0 ◣	13	LE1	240.1	0.0	-78.9	76.4	106.4	55.1	54.2	OK
SM1-arc 33	SP6	S 355	◢ 5.0 ◣	13	LE1	177.6	0.0	-52.5	-56.7	-79.9	40.8	40.2	OK
		S 355	◢ 5.0 ◣	13	LE1	296.2	0.0	-92.1	88.1	136.6	68.0	64.9	OK
SM1-arc 34	SP6	S 355	◢ 5.0 ◣	13	LE1	203.1	0.0	-48.0	-54.5	-100.1	46.6	45.9	OK
		S 355	◢ 5.0 ◣	13	LE1	326.7	0.0	-100.8	94.5	152.5	75.0	70.1	OK

SM1-arc 35	SP6	S 355	◢ 5.0 ◣	13	LE1	220.7	0.0	-35.0	-44.6	-117.6	50.7	49.8	OK
		S 355	◢ 5.0 ◣	13	LE1	336.8	0.0	-101.9	92.4	160.6	77.3	71.9	OK
SM1-arc 36	SP6	S 355	◢ 5.0 ◣	13	LE1	240.6	0.0	-21.6	-33.6	-134.2	55.2	54.3	OK
		S 355	◢ 5.0 ◣	13	LE1	332.5	0.0	-95.4	83.5	163.8	76.3	71.1	OK
SM1-arc 37	SP6	S 355	◢ 5.0 ◣	13	LE1	262.4	0.0	-6.5	-20.2	-150.1	60.2	58.6	OK
		S 355	◢ 5.0 ◣	13	LE1	318.0	0.0	-79.2	65.6	165.3	73.0	68.6	OK
SM1-arc 38	SP6	S 355	◢ 5.0 ◣	13	LE1	288.9	0.0	6.7	-5.9	-166.6	66.3	63.5	OK
		S 355	◢ 5.0 ◣	13	LE1	305.0	0.0	-54.9	42.2	168.0	70.0	66.4	OK
SM1-arc 39	SP6	S 355	◢ 5.0 ◣	13	LE1	314.0	0.0	15.7	6.7	-180.9	72.1	67.9	OK
		S 355	◢ 5.0 ◣	13	LE1	295.8	0.0	-23.3	14.3	169.7	67.9	64.8	OK
SM1-arc 40	SP6	S 355	◢ 5.0 ◣	13	LE1	332.6	0.0	17.4	17.6	-190.9	76.4	71.1	OK
		S 355	◢ 5.0 ◣	13	LE1	294.8	0.0	18.5	-18.3	168.9	67.7	64.6	OK
SM1-arc 41	SP6	S 355	◢ 5.0 ◣	13	LE1	341.9	0.0	18.1	30.1	-194.8	78.5	72.8	OK
		S 355	◢ 5.0 ◣	13	LE2	317.1	0.0	171.3	-131.3	80.5	72.8	68.4	OK
SM1-arc 42	SP6	S 355	◢ 5.0 ◣	13	LE1	368.7	0.0	-0.4	48.7	-207.2	84.6	78.0	OK
		S 355	◢ 5.0 ◣	13	LE2	427.0	0.1	271.2	-182.6	53.9	98.0	89.4	OK
SM1-arc 43	SP6	S 355	◢ 5.0 ◣	13	LE2	272.8	0.0	-107.2	55.2	-133.9	62.6	60.6	OK
		S 355	◢ 5.0 ◣	13	LE2	427.2	0.2	254.5	-177.1	-88.9	98.1	89.5	OK

SM1-arc 44	SP6	S 355	◢ 5.0 ◣	13	LE2	276.8	0.0	-123.9	-10.0	-142.6	63.6	61.3	OK
		S 355	◢ 5.0 ◣	13	LE2	427.2	0.2	236.6	-177.0	-104.0	98.1	89.5	OK
SM1-arc 45	SP6	S 355	◢ 5.0 ◣	13	LE2	279.2	0.0	-160.6	-57.2	-118.8	64.1	61.7	OK
		S 355	◢ 5.0 ◣	13	LE2	427.0	0.1	237.3	-170.5	-113.8	98.0	89.4	OK
SM1-arc 46	SP6	S 355	◢ 5.0 ◣	13	LE2	281.1	0.0	-182.2	-95.7	-78.1	64.5	62.1	OK
		S 355	◢ 5.0 ◣	13	LE2	426.9	0.0	239.0	-165.9	-119.1	98.0	89.3	OK
SM1-arc 47	SP6	S 355	◢ 5.0 ◣	13	LE2	297.5	0.0	-198.8	-125.1	-26.2	68.3	65.1	OK
		S 355	◢ 5.0 ◣	13	LE2	407.0	0.0	231.1	-157.2	-112.7	93.4	85.4	OK
SM1-arc 48	SP6	S 355	◢ 5.0 ◣	13	LE1	399.3	0.0	-155.6	-120.2	175.0	91.7	83.9	OK
		S 355	◢ 5.0 ◣	13	LE1	362.2	0.0	27.7	8.0	-208.3	83.2	76.8	OK
SM1-arc 49	SP6	S 355	◢ 5.0 ◣	13	LE1	427.3	0.2	-134.2	-109.8	206.8	98.1	89.5	OK
		S 355	◢ 5.0 ◣	13	LE1	427.1	0.2	-5.9	36.6	-243.8	98.1	89.4	OK
SM1-arc 50	SP6	S 355	◢ 5.0 ◣	13	LE1	427.9	0.6	-120.5	-105.1	212.5	98.2	89.9	OK
		S 355	◢ 5.0 ◣	13	LE1	427.6	0.5	-30.4	49.7	-241.2	98.2	89.7	OK
SM1-arc 51	SP6	S 355	◢ 5.0 ◣	13	LE1	428.5	1.0	-109.3	-100.6	217.1	98.4	89.5	OK
		S 355	◢ 5.0 ◣	13	LE1	428.2	0.8	-50.7	61.3	-237.7	98.3	89.9	OK
SM1-arc 52	SP6	S 355	◢ 5.0 ◣	13	LE1	429.1	1.3	-102.5	-98.8	219.4	98.5	89.0	OK
		S 355	◢ 5.0 ◣	13	LE1	428.8	1.1	-65.2	69.8	-234.5	98.4	89.3	OK

SM1-arc 53	SP6	S 355	◢ 5.0 ◣	13	LE1	429.6	1.6	-97.4	-96.8	221.3	98.6	89.1	OK
		S 355	◢ 5.0 ◣	13	LE1	429.2	1.4	-72.9	73.8	-232.8	98.5	89.1	OK
SM1-arc 54	SP6	S 355	◢ 5.0 ◣	13	LE1	429.8	1.7	-93.6	-95.5	222.6	98.7	89.2	OK
		S 355	◢ 5.0 ◣	13	LE1	429.5	1.5	-79.0	77.2	-231.2	98.6	89.1	OK
SM1-arc 55	SP6	S 355	◢ 5.0 ◣	13	LE1	430.0	1.8	-95.8	-101.2	219.9	98.7	89.3	OK
		S 355	◢ 5.0 ◣	13	LE1	429.7	1.6	-91.0	85.2	-227.0	98.6	89.1	OK
SM1-arc 56	SP6	S 355	◢ 5.0 ◣	13	LE1	430.3	2.0	-100.0	-112.0	214.1	98.8	89.4	OK
		S 355	◢ 5.0 ◣	13	LE1	429.8	1.7	-108.2	94.8	-220.7	98.7	89.3	OK
SM1-arc 1	SP6	S 355	◢ 5.0 ◣	4	LE1	430.7	2.2	-96.5	-114.6	213.5	98.9	89.7	OK
		S 355	◢ 5.0 ◣	4	LE1	430.2	1.9	-115.3	94.8	-219.7	98.8	89.4	OK
SP5	SP6	S 355	◢ 10.0 ◣	299	LE1	408.2	0.0	-155.2	-166.1	-141.1	93.7	68.1	OK
		S 355	◢ 10.0 ◣	298	LE1	427.2	0.2	-186.3	180.5	129.1	98.1	73.8	OK
STUB1-EPa	M2	S 355	◢ 4.0	577	LE1	255.9	0.0	-136.0	125.1	2.5	58.7	58.6	OK
STUB1-EPb	STUB1	S 355	◢ 4.0	577	LE1	263.3	0.0	-136.4	129.9	-6.0	60.5	57.9	OK
SM1-arc 22	STUB1	S 355	◢ 4.0	617	LE1	427.0	0.1	-341.1	144.6	-32.6	98.0	48.9	OK
STUB2-EPa	M4	S 355	◢ 4.0	407	LE2	349.0	0.0	250.3	-135.4	37.1	80.1	78.4	OK
STUB2-EPb	STUB2	S 355	◢ 4.0	407	LE2	345.7	0.0	250.2	-133.2	34.9	79.4	78.0	OK
SM1-arc 21	STUB2	S 355	◢ 4.0	484	LE2	286.2	0.0	195.7	-110.7	-47.8	65.7	32.0	OK
STUB3-EPa	M5	S 355	◢ 4.0	407	LE2	217.1	0.0	-117.0	105.6	1.2	49.8	49.8	OK
STUB3-EPb	STUB3	S 355	◢ 4.0	407	LE2	216.9	0.0	-118.0	105.1	0.6	49.8	49.8	OK
SM1-arc 20	STUB3	S 355	◢ 4.0	476	LE2	295.7	0.0	-98.3	51.5	-152.6	67.9	56.3	OK

Design data

Material	f_u [MPa]	β_w [-]	$\sigma_{w,Rd}$ [MPa]	0.9 σ [MPa]
S 355	490.0	0.90	435.6	352.8
S 275	430.0	0.85	404.7	309.6

Symbol explanation

T_w	Throat thickness a
L	Length
$\sigma_{w,Ed}$	Equivalent stress
ε_{Pl}	Strain
$\sigma_\perp$	Perpendicular stress
$\tau_\perp$	Shear stress perpendicular to weld axis
$\tau_{\|\|}$	Shear stress parallel to weld axis
Ut	Utilization
Ut_c	Weld capacity utilization
f_u	Ultimate strength of weld
β_w	Correlation factor EN 1993-1-8 – Tab. 4.1
$\sigma_{w,Rd}$	Equivalent stress resistance
0.9 σ	Perpendicular stress resistance: 0.9*fu/γM2
◢	Fillet weld

Detailed result for SM1-arc 1 / SP6

Weld resistance check (EN 1993-1-8 – Cl. 4.5.3.2)

$\sigma_{w,Rd} = f_u/(\beta_w \gamma_{M2}) =$ 435.6 MPa ≥ $\sigma_{w,Ed} = [\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)]^{0.5} =$ 430.7 MPa

$\sigma_{\perp,Rd} = 0.9 f_u / \gamma_{M2} =$ 352.8 MPa ≥ $|\sigma_\perp| =$ 96.5 MPa

where:

$f_u =$ 490.0 MPa – Ultimate strength

$\beta_w =$ 0.90 – Correlation factor EN 1993-1-8 – Tab. 4.1

$\gamma_{M2} =$ 1.25 – Safety factor

Stress utilization

$U_t = \max(\frac{\sigma_{w,Ed}}{\sigma_{w,Rd}} ; \frac{|\sigma_\perp|}{\sigma_{\perp,Rd}}) =$ 0.99 ≤ 1.0

Where:

$\sigma_{w,Ed} =$ 430.7 MPa – Maximum normal stress transverse to the axis of the weld

$\sigma_{w,Rd} =$ 435.6 MPa – Equivalent stress resistance

$\sigma_\perp =$ -96.5 MPa – Normal stress perpendicular to the throat

$\sigma_{\perp,Rd} =$ 352.8 MPa – Perpendicular stress resistance

Concrete block

Item	Loads	c [mm]	A_{eff} [mm2]	σ [MPa]	k_j [-]	f_{jd} [MPa]	Ut [%]	Status
CB 1	LE2	34	13928	39.2	3.00	40.2	97.5	OK

Symbol explanation

c	Bearing width
A_{eff}	Effective area
σ	Average stress in concrete
k_j	Concentration factor
f_{jd}	The ultimate bearing strength of the concrete block
Ut	Utilization

Detailed result for CB 1

Concrete block compressive resistance check (EN 1993-1-8 – 6.2.5)

$f_{jd} =$ 40.2 MPa ≥ $\sigma =$ 39.2 MPa

Where:

f_{jd} – concrete block design bearing strength:

$f_{jd} = \alpha_{cc} \beta_j k_j \frac{f_{ck}}{\gamma_c}$

, where:

$\alpha_{cc} =$

1.00 – long term effects on compressive strength factor

$\beta_j =$

0.67 – grout quality factor

$k_j =$

3.00 – concentration factor

$f_{ck} =$

30.0 MPa – characteristic resistance of concrete in compression

$\gamma_c =$

1.50 – safety factor for concrete

σ – average compressive stress in concrete under base plate

$\sigma = \frac{N}{A_{eff}}$

, where:

$N =$

545.9 kN – compressive normal force acting on concrete block

$A_{eff} =$

13928 mm^2 – effective area on which normal force is distributed

Shear in contact plane

Name	Loads	V_y [kN]	V_z [kN]	$V_{Rd,y}$ [kN]	$V_{Rd,z}$ [kN]	$V_{c,Rd}$ [kN]	U_t [%]	Status
SP5	LE2	255.6	65.3	2705.5	901.8	1519.2	17.4	OK

Symbol explanation

V_y	Shear force in base plate Vy
V_z	Shear force in base plate Vz
$V_{Rd,y}$	Shear resistance
$V_{Rd,z}$	Shear resistance
$V_{c,Rd}$	Concrete bearing resistance
U_t	Utilization

Detailed result for SP5

Shear lug steel resistance (EN 1993-1-1 – 6.2.6)

$V_{Rd,y} = A_{vx} \frac{f_y}{3^{0.5}\gamma_{M0}} =$ 2705.5 kN

$V_{Rd,z} = A_{vy} \frac{f_y}{3^{0.5}\gamma_{M0}} =$ 901.8 kN

Where:

$A_{vy} =$ 13200 mm^2	– shear area Ay of shear lug cross-section
$A_{vz} =$ 4400 mm^2	– shear area Az of shear lug cross-section
$f_y =$ 355.0 MPa	– anchor yield strength
$\gamma_{M0} =$ 1.00	– safety factor

Concrete bearing resistance (EN 1992-1-1 – 6.5.4)

$V_{c,Rd} = A\sigma_{Rd,max} =$ 1519.2 kN

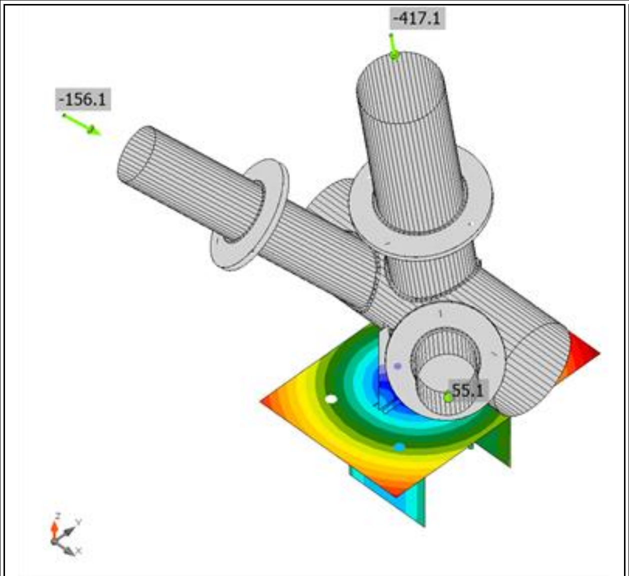
Where:

$A = lb =$ 86320 mm^2	– projected area of the shear lug excluding the portion above concrete
$l =$ 265 mm	– length of the shear lug excluding the portion above concrete
$b =$ 326 mm	– projected width of the shear lug in the direction of shear load
$\sigma_{Rd,max} = k_1 v' f_{ck}/\gamma_c =$ 17.6 MPa	– maximum stress which can be applied at the edges of the node
$k_1 =$ 1.00	– factor - EN 1992-1-1 - Equation (6.60)
$v' = 1 - f_{ck}/250 =$ 0.88	– factor - EN 1992-1-1 - Equation (6.57N)
$f_{ck} =$ 30.0 MPa	– characteristic resistance of concrete in compression
$\gamma_c =$ 1.50	– safety factor

Utilization in shear

$U_t =$ 0.17 ≤ 1

Buckling



First buckling mode shape, LE1

Loads	Shape	Factor [-]
LE1	1	39.78
	2	55.62
	3	88.01
	4	95.58
	5	113.13
	6	122.43
LE2	1	51.62
	2	57.72
	3	84.00
	4	97.61
	5	118.04
	6	134.19

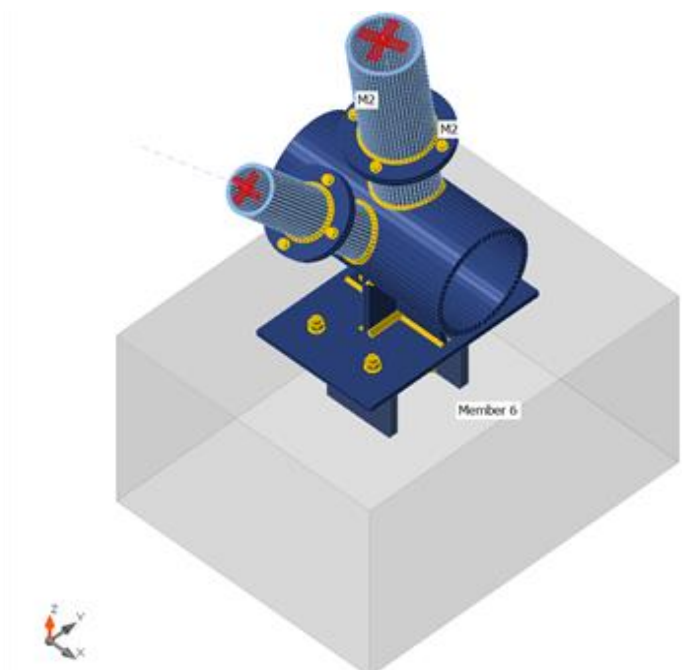
Detail Connection Check of Base 2

Geometry

Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]
M2	33 - CHS193.7/10.0	-73.1	78.7	1.0	0	0	0
M5	34 - CHS139.7/10.0	-93.8	47.5	0.0	0	0	0

Supports and forces

Name	Support	Forces in	X [mm]
M2 / end	Mx-My-Mz	Node	0
M5 / end	Mx-My-Mz	Node	0



Cross-sections

Name	Material
33 - CHS193.7/10.0	S 355
34 - CHS139.7/10.0	S 355
35 - CHS323.9/25.0	S 355
27 - CHS177.8,16	S 355
30 - Iw280x220	S 355

Anchors / Bolts

Name	Bolt assembly	Diameter [mm]	f_u [MPa]	Gross area [mm^2]
M16 8.8	M16 8.8	16	800.0	201
M24 8.8	M24 8.8	24	800.0	452

Load effects (forces in equilibrium)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	M2 / End	-516.7	0.0	0.0	0.0	0.0	0.0
	M5 / End	-504.4	0.0	0.0	0.0	0.0	0.0

Unbalanced forces

Name	X [kN]	Y [kN]	Z [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	-6.8	436.9	-878.5	0.0	0.0	0.0

Foundation block

Item	Value	Unit
CB 1		
Dimensions	900 x 1100	mm
Depth	600	mm
Anchor	M24 8.8	
Anchoring length	100	mm
Shear force transfer	Shear lug	
Cross-section of shear lug	Iw280x220	
Length of shear lug	265	mm

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	1.1 < 5.0%	OK
Bolts	22.2 < 100%	OK
Anchors	68.9 < 100%	OK
Welds	99.6 < 100%	OK
Concrete block	98.5 < 100%	OK
Shear	48.3 < 100%	OK
Buckling	18.81	

Plates

Name	t_p [mm]	Loads	σ_{Ed} [MPa]	ε_{Pl} [%]	$\sigma_{c,Ed}$ [MPa]	Status
M2	10.0	LE1	169.5	0.0	0.0	OK
M5	10.0	LE1	283.9	0.0	0.0	OK
SM1	25.0	LE1	355.2	0.1	0.0	OK
STUB1	16.0	LE1	145.8	0.0	0.0	OK
STUB3	10.0	LE1	282.6	0.0	0.0	OK
Member 6-tfl 1	30.0	LE1	255.1	0.0	0.0	OK
Member 6-bfl 1	30.0	LE1	291.0	0.0	0.0	OK
Member 6-w 1	20.0	LE1	259.2	0.0	0.0	OK
STUB1-EPa	15.0	LE1	129.5	0.0	39.5	OK
STUB1-EPb	15.0	LE1	126.4	0.0	37.9	OK
STUB3-EPa	15.0	LE1	85.8	0.0	64.0	OK
STUB3-EPb	15.0	LE1	85.7	0.0	65.1	OK
SP3	15.0	LE1	328.8	0.0	0.0	OK

SP4	20.0	LE1	302.8	0.0	0.0	OK
SP5	20.0	LE1	317.0	0.0	0.0	OK
SP6	20.0	LE1	357.4	1.1	0.0	OK

Design data

Material	f_y [MPa]	ε_{lim} [%]
S 355	355.0	5.0

Symbol explanation

t_p Plate thickness
σ_{Ed} Equivalent stress
ε_{Pl} Plastic strain
$\sigma_{c,Ed}$ Contact stress
f_y Yield strength
ε_{lim} Limit of plastic strain

Detailed result for SP6

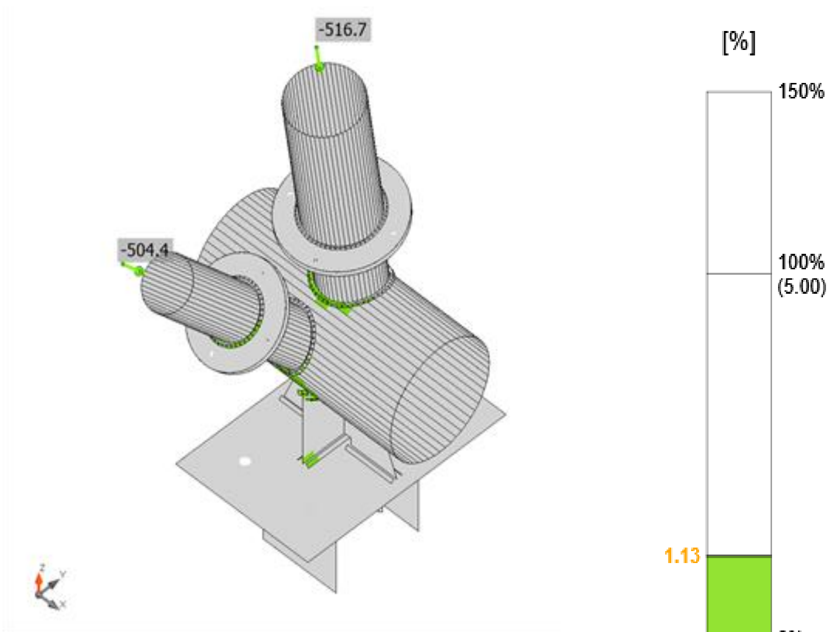
Design values used in the analysis

$f_{yd} = \frac{f_{yk}}{\gamma_{M0}} =$ 355.0 MPa

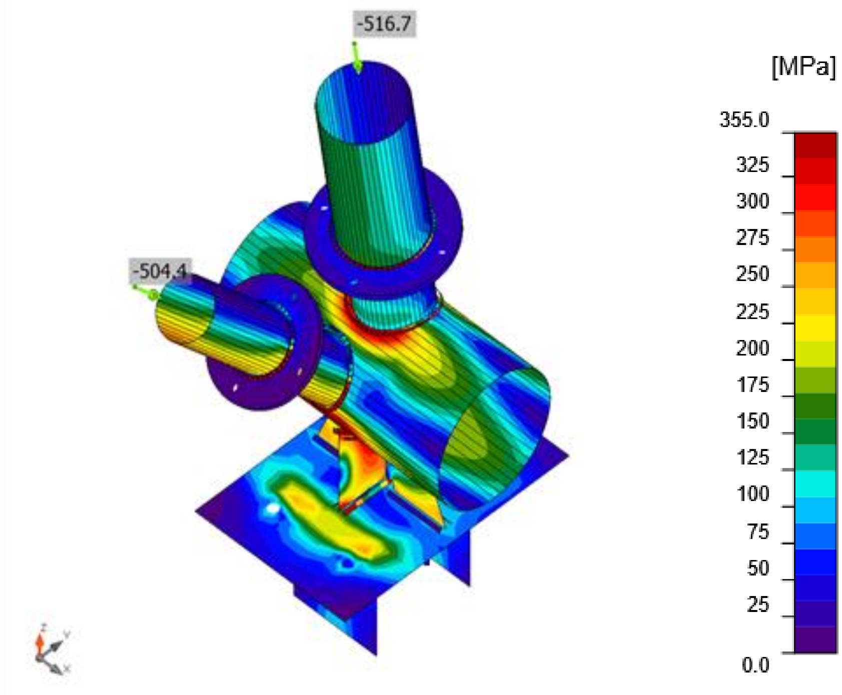
Where:

f_{yk} = 355.0 MPa – characteristic yield strength

γ_{M0} = 1.00 – partial safety factor for steel material EN 1993-1-1 – 6.1

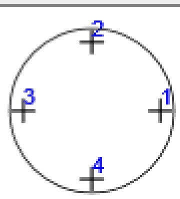
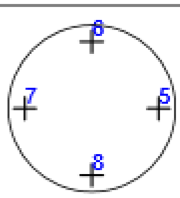


Strain check, LE1



Equivalent stress, LE1

Bolts

Shape	Item	Grade	Loads	$F_{t,Ed}$ [kN]	$F_{v,Ed}$ [kN]	$F_{b,Rd}$ [kN]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	M16 8.8 - 1	LE1	0.5	2.4	95.1	0.5	3.9	4.3	OK
	B2	M16 8.8 - 1	LE1	0.2	2.5	95.8	0.2	4.2	4.4	OK
	B3	M16 8.8 - 1	LE1	0.6	2.8	95.4	0.7	4.6	5.2	OK
	B4	M16 8.8 - 1	LE1	0.9	2.9	95.1	0.9	4.8	5.5	OK
	B5	M16 8.8 - 1	LE1	0.0	0.9	151.1	0.0	1.4	1.4	OK
	B6	M16 8.8 - 1	LE1	20.1	0.9	152.0	22.2	1.5	17.4	OK
	B7	M16 8.8 - 1	LE1	0.0	0.9	150.2	0.0	1.4	1.4	OK
	B8	M16 8.8 - 1	LE1	1.1	0.9	152.3	1.2	1.5	2.4	OK

Design data

Grade	$F_{t,Rd}$ [kN]	$B_{p,Rd}$ [kN]	$F_{v,Rd}$ [kN]
M16 8.8 - 1	90.4	281.2	60.3

Symbol explanation

$F_{t,Ed}$ Tension force
$F_{v,Ed}$ Resultant of bolt shear forces Vy and Vz in shear planes
$F_{b,Rd}$ Plate bearing resistance EN 1993-1-8 – Tab. 3.4
Ut_t Utilization in tension
Ut_s Utilization in shear

Ut_{ts} Interaction of tension and shear EN 1993-1-8 – Tab. 3.4
$F_{t,Rd}$ Bolt tension resistance EN 1993-1-8 – Tab. 3.4
$B_{p,Rd}$ Punching shear resistance EN 1993-1-8 – Tab. 3.4
$F_{v,Rd}$ Bolt shear resistance EN 1993-1-8 – Tab. 3.4

Detailed result for B6

Tension resistance check (EN 1993-1-8 – Table 3.4)

$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} =$ 90.4 kN ≥ $F_{t,Ed} =$ 20.1 kN

Where:

$k_2 = 0.90$ – Factor

$f_{ub} = 800.0\ \text{MPa}$ – Ultimate tensile strength of the bolt

$A_s = 157\ \text{mm}^2$ – Tensile stress area of the bolt

$\gamma_{M2} = 1.25$ – Safety factor

Punching resistance check (EN 1993-1-8 – Table 3.4)

$B_{p,Rd} = \frac{0.6\ \pi\ d_m\ t_p f_u}{\gamma_{M2}} =$ 281.2 kN ≥ $F_{t,Ed} =$ 20.1 kN

Where:

$d_m = 25\ \text{mm}$ – The mean of the across points and across flats dimensions of the bolt head or the nut, whichever is smaller

$t_p = 15\ \text{mm}$ – Plate thickness

$f_u = 490.0\ \text{MPa}$ – Ultimate strength

$\gamma_{M2} = 1.25$ – Safety factor

Shear resistance check (EN 1993-1-8 – Table 3.4)

$F_{v,Rd} = \frac{\beta_p\ \alpha_v f_{ub} A}{\gamma_{M2}} =$ 60.3 kN ≥ $F_{v,Ed} =$ 0.9 kN

Where:

$\beta_p = 1.00$ – Reduction factor for packing

$\alpha_v = 0.60$ – Reduction factor for shear stress

$f_{ub} = 800.0\ \text{MPa}$ – Ultimate tensile strength of the bolt

$A = 157\ \text{mm}^2$ – Tensile stress area of the bolt

$\gamma_{M2} = 1.25$ – Safety factor

Bearing resistance check (EN 1993-1-8 – Table 3.4)

$F_{b,Rd} = \frac{k_1\ \alpha_b f_u\ dt}{\gamma_{M2}} =$ 152.0 kN ≥ $F_{b,Ed} =$ 0.9 kN

Where:

$k_1 = \min(2.8\frac{e_2}{d_0} - 1.7, 1.4\frac{p_2}{d_0} - 1.7, 2.5) = 2.19$ – Factor for edge distance and bolt spacing perpendicular to the direction of load transfer

$\alpha_b = \min(\frac{e_1}{3d_0}, \frac{p_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u}, 1) = 0.74$ – Factor for end distance and bolt spacing in direction of load transfer

$e_2 = 25\ \text{mm}$ – Distance to the plate edge perpendicular to the shear force

$p_2 =$ 200 mm – Distance between bolts perpendicular to the shear force

$d_0 =$ 18 mm – Bolt hole diameter

$e_1 =$ 40 mm – Distance to the plate edge in the direction of the shear force

$p_1 =$ ∞ mm – Distance between bolts in the direction of the shear force

$f_{ub} =$ 800.0 MPa – Ultimate tensile strength of the bolt

$f_u =$ 490.0 MPa – Ultimate strength of the plate

$d =$ 16 mm – Nominal diameter of the fastener

$t =$ 15 mm – Thickness of the plate

$\gamma_{M2} =$ 1.25 – Safety factor

Utilization in tension

$\frac{F_{t,Ed}}{\min(F_{t,Rd};B_{p,Rd})} =$ 0.22 ≤ 1.0

Where:

$F_{t,Ed} =$ 20.1 kN – Tensile force

$F_{t,Rd} =$ 90.4 kN – Tension resistance

$B_{p,Rd} =$ 281.2 kN – Punching resistance

Utilization in shear

$\max(\frac{F_{v,Ed}}{F_{v,Rd}};\frac{F_{b,Ed}}{F_{b,Rd}}) =$ 0.02 ≤ 1.0

Where:

$F_{v,Ed} =$ 0.9 kN – Shear force (in decisive shear plane)

$F_{v,Rd} =$ 60.3 kN – Shear resistance

$F_{b,Ed} =$ 0.9 kN – Bearing force (for decisive plate)

$F_{b,Rd} =$ 152.0 kN – Bearing resistance

Interaction of tension and shear (EN 1993-1-8 – Table 3.4)

$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4\,F_{t,Rd}} =$ 0.17 ≤ 1.0

Where:

$F_{v,Ed} =$ 0.9 kN – Shear force (in decisive shear plane)

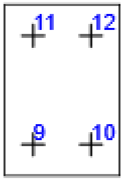
$F_{v,Rd} =$ 60.3 kN – Shear resistance

$F_{t,Ed} =$ 20.1 kN – Tensile force

$F_{t,Rd} =$ 90.4 kN – Tension resistance

Anchors

Shape	Item	Loads	N_{Ed}	$V_{Rd,cp}$	Ut_t	Ut_s	Ut_{ts}	Status

			[kN]	[kN]	[%]	[%]	[%]	
	A9	LE1	104.9	93.7	65.5	0.2	42.9	OK
	A10	LE1	110.2	93.7	68.9	0.2	47.4	OK
	A11	LE1	0.0	93.7	0.0	0.2	0.0	OK
	A12	LE1	0.0	93.7	0.0	0.2	0.0	OK

Design data

Grade	$N_{Rd,s}$ [kN]	$V_{Rd,s}$ [kN]
M24 8.8 - 2	160.0	113.0

Symbol explanation

N_{Ed}	Tension force
$V_{Rd,cp}$	Design resistance in case of concrete pryout failure - EN 1992-4 – 7.2.2.4
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
$N_{Rd,s}$	Design tensile resistance of a fastener in case of steel failure - EN 1992-4 – 7.2.1.3
$V_{Rd,s}$	Design shear resistance of a fastener in case of steel failure - EN 1992-4 – 7.2.2.3.1

Detailed result for A10

Following checks of anchors loaded in tension are not provided and will be checked using Hilti PROFIS Engineering:

- Pull-out failure of fastener (for post-installed mechanical anchors) - EN 1992-4 – 7.2.1.5
- Combined pull-out and concrete failure (for post-installed bonded anchors) - EN 1992-4 – 7.2.1.6
- Concrete splitting failure - EN 1992-4 – 7.2.1.7

Concrete blow-out failure is provided only for anchors with washer plates.

Anchor tensile resistance (EN 1992-4 – 7.2.1.3)

$$N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{Ms}} = \quad 160.0 \quad \text{kN} \quad \geq \quad N_{Ed} = \quad 110.2 \quad \text{kN}$$

$$N_{Rk,s} = c \cdot A_s \cdot f_{uk} = \quad 240.0 \quad \text{kN}$$

Where:

$c = 0.85$	– reduction factor for cut thread
$A_s = 353 \text{ mm}^2$	– tensile stress area
$f_{uk} = 800.0 \text{ MPa}$	– minimum tensile strength of the bolt
$\gamma_{Ms} = 1.50$	– safety factor for steel

$$\gamma_{Ms} = 1.2 \cdot \frac{f_{uk}}{f_{yk}} \geq 1.4$$

, where:

f_{yk} =
640.0 MPa – minimum yield strength of the bolt

Shear resistance (EN 1992-4 – 7.2.2.3.1)

$V_{Rd,s}$ = 113.0 kN ≥ V_{Ed} = 0.3 kN

$V_{Rk,s} = k_7 \cdot V^0_{Rk,s}$ = 141.2 kN

Where:

$k_7 = 1.00$ – coefficient for anchor steel ductility

$$k_7 = \begin{cases} 0.8, & A < 0.08 \\ 1.0, & A \geq 0.08 \end{cases}$$

, where:
A =
0.12 – bolt grade elongation at rupture

$V^0_{Rk,s}$ = 141.2 kN – the characteristic shear strength

$V^0_{Rk,s} = k_6 \cdot A_s \cdot f_{uk}$

, where:
k_6 =
0.50 – coefficient for anchor resistance in shear
A_s =
353 mm² – tensile stress area
f_{uk} =
800.0 MPa – specified ultimate strength of anchor steel

γ_{Ms} = 1.25 – safety factor for steel

Interaction of tensile and shear forces in steel (EN 1992-4 – Table 7.3)

$(\frac{N_{Ed}}{N_{Rd,s}})^2 +$ 0.47 ≤ 1.0

Where:

N_{Ed} = 110.2 kN – design tension force

$N_{Rd,s}$ = 160.0 kN – fastener tensile strength

V_{Ed} = 0.3 kN – design shear force

$V_{Rd,s}$ = 113.0 kN – fastener shear strength

Interaction of tensile and shear forces in concrete (EN 1992-4 – Table 7.3)

$(\frac{N_{Ed}}{N_{Rd,i}})^{1.5} +$ 0.00 ≤ 1.0

Where:

$\frac{N_{Ed}}{N_{Rd,i}}$ – the largest utilization value for tension failure modes

$\frac{V_{Ed}}{V_{Rd,i}}$ – the largest utilization value for shear failure modes

$\frac{N_{Ed,g}}{N_{Rd,c}}$ = 0% – concrete breakout failure of anchor in tension

$\frac{N_{Ed}}{N_{Rd,p}}$ = 0% – concrete pullout failure

$\frac{N_{Ed}}{N_{Rd,cb}}$ = 0% – concrete blowout failure

$\frac{V_{Ed}}{V_{Rd,c}} = 0\%$ – concrete edge failure

$\frac{V_{Ed}}{V_{Rd,cb}} = 0\%$ – concrete pryout failure

Supplementary reinforcement (EN 1992-4 – 7.2.1.9)
Supplementary reinforcement should resist force of 215.1 kN in tension.

Welds

Item	Edge	Material	T_w [mm]	L [mm]	Loads	$\sigma_{w,Ed}$ [MPa]	ε_{Pl} [%]	$\sigma_{\perp}$ [MPa]	$\tau_{\perp}$ [MPa]	$\tau_{\parallel}$ [MPa]	Ut [%]	Ut_c [%]	Status
SM1-arc 42	SP3	S 355	5.0	93	LE1	427.5	0.4	-225.9	-208.6	20.2	98.1	85.5	OK
		S 355	5.0	93	LE1	291.2	0.0	-77.5	157.1	-39.9	66.9	55.1	OK
SM1-arc 42	SP4	S 355	10.0	93	LE1	306.5	0.0	-187.0	-138.5	-21.9	70.4	55.6	OK
		S 355	10.0	93	LE1	140.3	0.0	-17.8	77.6	20.9	32.2	26.6	OK
SP5	SP4	S 355	10.0	139	LE1	38.3	0.0	23.0	6.8	-16.3	8.8	7.5	OK
		S 355	10.0	139	LE1	162.8	0.0	-82.2	80.2	12.5	37.4	33.1	OK
SP5	SP3	S 355	10.0	139	LE1	55.2	0.0	-22.8	-28.3	-6.4	12.7	9.8	OK
		S 355	10.0	139	LE1	138.4	0.0	-65.8	60.4	36.0	31.8	26.3	OK
Member 6-tfl 1	Member 6-w 1	S 355	10.0	264	LE1	151.4	0.0	-53.5	-52.6	-62.6	34.8	18.7	OK
		S 355	10.0	264	LE1	151.4	0.0	-52.4	53.3	62.4	34.8	17.7	OK
Member 6-bfl 1	Member 6-w 1	S 275	10.0	264	LE1	280.5	0.0	46.5	-46.4	152.8	69.3	34.1	OK
		S 275	10.0	264	LE1	282.3	0.0	46.1	46.2	-154.0	69.8	34.5	OK
SP5	Member 6-tfl 1	S 355	10.0	220	LE1	370.6	0.0	-208.1	-141.2	106.8	85.1	74.5	OK
		S 355	10.0	219	LE1	194.5	0.0	127.3	-83.2	-16.9	44.7	29.2	OK

SP5	Member 6-bfl 1	S 355	◢ 10.0 ◣	220	LE1	337.1	0.0	63.3	117.6	-150.7	77.4	71.1	OK
		S 355	◢ 10.0 ◣	219	LE1	427.6	0.5	205.0	-188.2	107.3	98.2	96.7	OK
SP5	Member 6-w 1	S 355	◢ 10.0 ◣	219	LE1	269.3	0.0	-124.7	-122.6	62.9	61.8	52.1	OK
		S 355	◢ 10.0 ◣	219	LE1	267.2	0.0	-120.6	122.7	-62.4	61.4	51.3	OK
SM1-arc 36	SP6	S 355	◢ 10.0 ◣	17	LE1	427.1	0.1	-144.6	-146.1	-180.2	98.1	89.4	OK
		S 355	◢ 10.0 ◣	17	LE1	427.1	0.1	-146.2	144.7	180.9	98.1	89.4	OK
SM1-arc 37	SP6	S 355	◢ 10.0 ◣	17	LE1	427.0	0.1	-159.2	-159.4	-164.1	98.0	89.4	OK
		S 355	◢ 10.0 ◣	17	LE1	427.0	0.1	-159.0	158.7	164.8	98.0	89.4	OK
SM1-arc 38	SP6	S 355	◢ 10.0 ◣	17	LE1	399.7	0.0	-99.9	-99.4	-200.1	91.8	84.0	OK
		S 355	◢ 10.0 ◣	17	LE1	402.2	0.0	-99.5	100.0	201.5	92.3	84.5	OK
SM1-arc 39	SP6	S 355	◢ 10.0 ◣	17	LE1	379.0	0.0	-35.7	-35.0	-215.0	87.0	80.0	OK
		S 355	◢ 10.0 ◣	17	LE1	379.6	0.0	-35.0	35.7	215.3	87.2	80.2	OK
SM1-arc 40	SP6	S 355	◢ 10.0 ◣	17	LE1	417.2	0.0	11.5	12.3	-240.4	95.8	87.3	OK
		S 355	◢ 10.0 ◣	17	LE1	415.2	0.0	13.1	-12.3	239.3	95.3	86.9	OK
SM1-arc 41	SP6	S 355	◢ 10.0 ◣	17	LE1	426.9	0.0	40.9	37.9	-242.4	98.0	89.2	OK
		S 355	◢ 10.0 ◣	17	LE1	426.9	0.0	36.6	-39.6	242.3	98.0	89.2	OK
SM1-arc 42	SP6	S 355	◢ 10.0 ◣	17	LE1	426.9	0.0	50.5	54.0	-238.7	98.0	89.2	OK
		S 355	◢ 10.0 ◣	17	LE1	426.9	0.0	55.9	-52.4	238.6	98.0	89.2	OK

SM1-arc 43	SP6	S 355	◢ 10.0 ◣	17	LE1	300.2	0.0	40.8	43.9	-166.0	68.9	65.6	OK
		S 355	◢ 10.0 ◣	17	LE1	291.3	0.0	45.4	-42.2	160.7	66.9	64.0	OK
SM1-arc 44	SP6	S 355	◢ 10.0 ◣	17	LE1	219.7	0.0	16.9	13.2	-125.8	50.4	49.6	OK
		S 355	◢ 10.0 ◣	17	LE1	212.3	0.0	11.2	-14.9	121.5	48.7	47.9	OK
SM1-arc 45	SP6	S 355	◢ 10.0 ◣	17	LE1	117.3	0.0	-0.3	-0.8	-67.7	26.9	26.9	OK
		S 355	◢ 10.0 ◣	17	LE1	113.9	0.0	-1.3	0.8	65.7	26.1	26.1	OK
SM1-arc 46	SP6	S 355	◢ 10.0 ◣	17	LE1	22.6	0.0	-9.8	-10.2	-5.8	5.2	5.2	OK
		S 355	◢ 10.0 ◣	17	LE1	23.0	0.0	-11.2	10.8	4.0	5.3	5.3	OK
SM1-arc 47	SP6	S 355	◢ 10.0 ◣	17	LE1	106.2	0.0	-24.4	-24.2	54.5	24.4	24.4	OK
		S 355	◢ 10.0 ◣	17	LE1	109.3	0.0	-24.6	24.7	-56.3	25.1	25.1	OK
SM1-arc 48	SP6	S 355	◢ 10.0 ◣	17	LE1	283.2	0.0	-86.6	-86.6	129.4	65.0	62.5	OK
		S 355	◢ 10.0 ◣	17	LE1	286.6	0.0	-87.3	87.3	-131.3	65.8	63.1	OK
SM1-arc 49	SP6	S 355	◢ 10.0 ◣	17	LE1	426.9	0.0	-130.0	-130.2	195.3	98.0	89.2	OK
		S 355	◢ 10.0 ◣	17	LE1	426.9	0.0	-129.9	129.6	-195.7	98.0	89.2	OK
SM1-arc 50	SP6	S 355	◢ 10.0 ◣	17	LE1	427.1	0.2	-125.9	-126.0	199.1	98.1	89.4	OK
		S 355	◢ 10.0 ◣	17	LE1	427.1	0.2	-125.8	125.6	-199.4	98.1	89.4	OK
SM1-arc 51	SP6	S 355	◢ 10.0 ◣	17	LE1	427.1	0.1	-115.7	-115.7	207.2	98.1	89.4	OK
		S 355	◢ 10.0 ◣	17	LE1	427.1	0.1	-115.6	115.5	-207.4	98.1	89.4	OK

SM1-arc 52	SP6	S 355	◢ 10.0 ◣	17	LE1	426.9	0.0	-102.0	-102.0	216.5	98.0	89.3	OK
		S 355	◢ 10.0 ◣	17	LE1	426.9	0.0	-101.8	101.8	-216.6	98.0	89.3	OK
SM1-arc 53	SP6	S 355	◢ 10.0 ◣	10	LE1	247.6	0.0	-47.3	-48.4	131.7	56.9	55.8	OK
		S 355	◢ 10.0 ◣	10	LE1	248.1	0.0	-48.3	47.2	-132.3	57.0	55.9	OK
SP5	SP6	S 355	◢ 10.0 ◣	263	LE1	433.8	4.0	-213.5	-214.0	-41.7	99.6	99.6	OK
		S 355	◢ 10.0 ◣	263	LE1	433.7	4.0	-214.0	213.6	42.9	99.6	99.6	OK
STUB1-EPa	M2	S 355	◢ 5.0	577	LE1	414.9	0.0	-198.5	209.6	16.9	95.3	60.0	OK
STUB1-EPb	STUB1	S 355	◢ 5.0	508	LE1	427.9	0.6	-264.1	194.4	-0.6	98.2	63.1	OK
SM1-arc 18	STUB1	S 355	◢ 5.0	530	LE1	432.1	3.0	-287.7	186.0	-6.6	99.2	94.1	OK
STUB3-EPa	M5	S 355	◢ 7.0	407	LE1	431.5	2.7	-224.0	212.9	-1.5	99.1	99.1	OK
STUB3-EPb	STUB3	S 355	◢ 7.0	407	LE1	431.3	2.6	-224.8	212.5	-0.8	99.0	99.0	OK
SM1-arc 27	STUB3	S 355	◢ 7.0	436	LE1	427.0	0.1	-179.3	115.0	-191.9	98.0	70.0	OK

Design data

Material	f_u [MPa]	β_w [-]	$\sigma_{w,Rd}$ [MPa]	0.9 σ [MPa]
S 355	490.0	0.90	435.6	352.8
S 275	430.0	0.85	404.7	309.6

Symbol explanation

T_w	Throat thickness a
L	Length
$\sigma_{w,Ed}$	Equivalent stress
ε_{Pl}	Strain
$\sigma_{\perp}$	Perpendicular stress
$\tau_{\perp}$	Shear stress perpendicular to weld axis
$\tau_{\|\|}$	Shear stress parallel to weld axis
Ut	Utilization
Ut_c	Weld capacity utilization
f_u	Ultimate strength of weld
β_w	Correlation factor EN 1993-1-8 – Tab. 4.1
$\sigma_{w,Rd}$	Equivalent stress resistance
0.9 σ	Perpendicular stress resistance: 0.9*fu/γM2
◢	Fillet weld

Detailed result for SP5 / SP6

Weld resistance check (EN 1993-1-8 – Cl. 4.5.3.2)

$\sigma_{w,Rd} = f_u/(\beta_w \gamma_{M2}) =$ 435.6 MPa ≥ $\sigma_{w,Ed} = [\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)]^{0.5} =$ 433.8 MPa

$\sigma_{\perp,Rd} = 0.9 f_u / \gamma_{M2} =$ 352.8 MPa ≥ $|\sigma_\perp| =$ 214.2 MPa

where:

- $f_u =$ 490.0 MPa – Ultimate strength
- $\beta_w =$ 0.90 – Correlation factor EN 1993-1-8 – Tab. 4.1
- $\gamma_{M2} =$ 1.25 – Safety factor

Stress utilization

$U_t = \max(\frac{\sigma_{w,Ed}}{\sigma_{w,Rd}} ; \frac{|\sigma_\perp|}{\sigma_{\perp,Rd}}) =$ 1.00 ≤ 1.0

Where:

- $\sigma_{w,Ed} =$ 433.8 MPa – Maximum normal stress transverse to the axis of the weld
- $\sigma_{w,Rd} =$ 435.6 MPa – Equivalent stress resistance
- $\sigma_\perp =$ -214.2 MPa – Normal stress perpendicular to the throat
- $\sigma_{\perp,Rd} =$ 352.8 MPa – Perpendicular stress resistance

Concrete block

Item	Loads	c [mm]	A_{eff} [mm2]	σ [MPa]	k_j [-]	f_{jd} [MPa]	Ut [%]	Status
CB 1	LE1	34	28380	39.6	3.00	40.2	98.5	OK

Symbol explanation

c	Bearing width
A_{eff}	Effective area
σ	Average stress in concrete
k_j	Concentration factor
f_{jd}	The ultimate bearing strength of the concrete block
Ut	Utilization

Detailed result for CB 1

Concrete block compressive resistance check (EN 1993-1-8 – 6.2.5)

$f_{jd} =$ 40.2 MPa ≥ $\sigma =$ 39.6 MPa

Where:

f_{jd} – concrete block design bearing strength:

$f_{jd} = \alpha_{cc} \beta_j k_j \frac{f_{ck}}{\gamma_c}$

, where:

$\alpha_{cc} =$

1.00 – long term effects on compressive strength factor

$\beta_j =$

0.67 – grout quality factor
k_j =
3.00 – concentration factor
f_{ck} =
30.0 MPa – characteristic resistance of concrete in compression
γ_c =
1.50 – safety factor for concrete
σ – average compressive stress in concrete under base plate
$\sigma = \frac{N}{A_{eff}}$
, where:
N =
1123.9 kN – compressive normal force acting on concrete block
A_{eff} =
28380 mm² – effective area on which normal force is distributed

Shear in contact plane

Name	Loads	V_y [kN]	V_z [kN]	$V_{Rd,y}$ [kN]	$V_{Rd,z}$ [kN]	$V_{c,Rd}$ [kN]	U_t [%]	Status
SP5	LE1	-6.8	435.8	2705.5	901.8	1046.4	48.3	OK

Symbol explanation

V_y	Shear force in base plate Vy
V_z	Shear force in base plate Vz
$V_{Rd,y}$	Shear resistance
$V_{Rd,z}$	Shear resistance
$V_{c,Rd}$	Concrete bearing resistance
U_t	Utilization

Detailed result for SP5

Shear lug steel resistance (EN 1993-1-1 – 6.2.6)

$V_{Rd,y} = A_{vx} \frac{f_y}{3^{0.5}\gamma_{M0}} =$ 2705.5 kN

$V_{Rd,z} = A_{vy} \frac{f_y}{3^{0.5}\gamma_{M0}} =$ 901.8 kN

Where:

A_{vy} = 13200 mm² – shear area Ay of shear lug cross-section

A_{vz} = 4400 mm² – shear area Az of shear lug cross-section

f_y = 355.0 MPa – anchor yield strength

γ_{M0} = 1.00 – safety factor

Concrete bearing resistance (EN 1992-1-1 – 6.5.4)

$V_{c,Rd} = A\sigma_{Rd,max}$ = 1046.4 kN

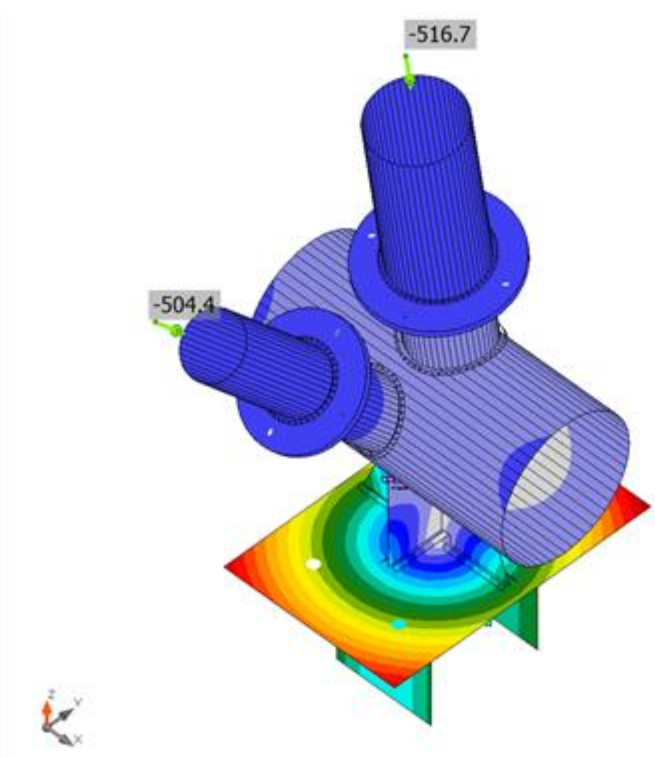
Where:

$A = lb = 59452\ \mathrm{mm}^2$	– projected area of the shear lug excluding the portion above concrete
$l = 265\ \mathrm{mm}$	– length of the shear lug excluding the portion above concrete
$b = 224\ \mathrm{mm}$	– projected width of the shear lug in the direction of shear load
$\sigma_{Rd,max} = k_1 v' f_{ck}/\gamma_c = 17.6\ \mathrm{MPa}$	– maximum stress which can be applied at the edges of the node
$k_1 = 1.00$	– factor - EN 1992-1-1 - Equation (6.60)
$v' = 1 - f_{ck}/250 = 0.88$	– factor - EN 1992-1-1 - Equation (6.57N)
$f_{ck} = 30.0\ \mathrm{MPa}$	– characteristic resistance of concrete in compression
$\gamma_c = 1.50$	– safety factor

Utilization in shear

U_t = 0.48 ≤ 1

Buckling



First buckling mode shape, LE1

Loads	Shape	Factor [-]
LE1	1	18.81
	2	22.84
	3	23.95
	4	26.67
	5	43.04
	6	45.23

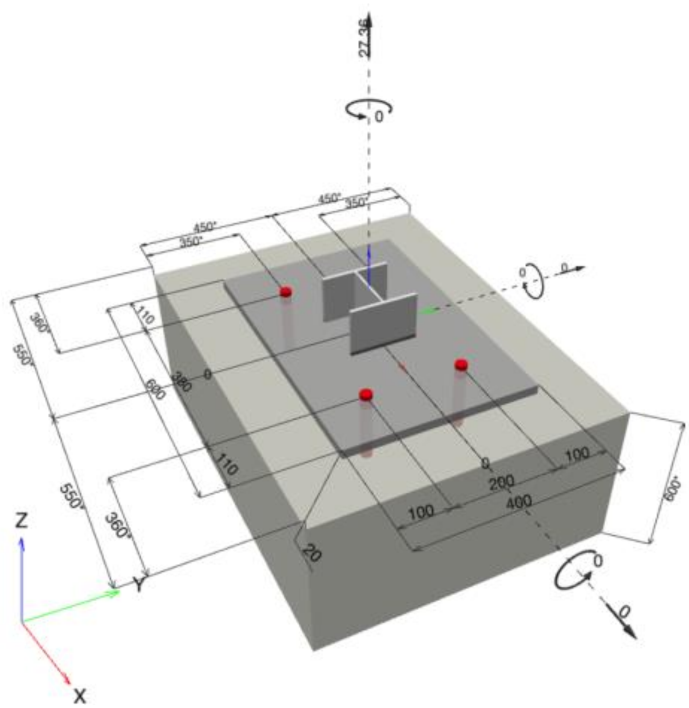
Detail check of anchor bolt and footing in Hilti

1.1 Input data

Anchor type and diameter:	**Headed fastener 5.8 M24**
Effective embendment depth:	h_{ef} = 120.0 mm
Material:	5.8
Evaluation Service Report:	-
Issued I Valid:	- | -
Proof:	Design Method EN 1992-4, CastInPlace
Stand-off installation:	e_b = 0.0 mm (no stand-off); t = 20.0 mm
Anchor plate[CBFEM] :	l_x x l_y x t = 600.0 mm x 400.0 mm x 20.0 mm;
Profile:	IPBi/HEA, IPBI 140 / HE 140 A; (L x W x T x FT) = 133.0 mm x 140.0 mm x 5.5 mm x 8.5 mm
Base material:	cracked concrete, C30/37, $f_{c,cyl}$ = 30.00 N/mm^2; h = 600.0 mm, User-defined partial material safety factor γ_c = 1.500
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)
	no longitudinal edge reinforcement Reinforcement to control splitting acc. to EN 1992-4, 7.2.1.7 (2) b) 2) present

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [mm] & Loading [kN, kNm]



1.1.1 Load combination

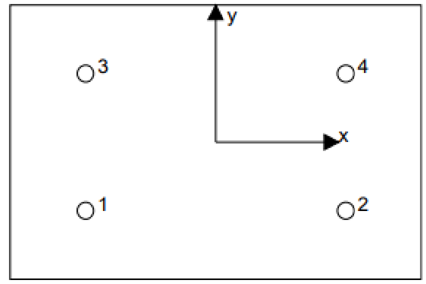
Case	Description	Forces [kN] / Moments [kNm]	Seismic	Fire	Max. Util. Anchor [%
1	Combination 1	$N = 27.360$; $V_x = 0.000$; $V_y = 0.000$; $M_x = 0.000$; $M_y = 0.000$; $M_z = 0.000$;	no	no	35

1.2 Load case/Resulting anchor forces

Anchor reactions [kN]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	11.499	0.045	0.042	0.015
2	11.491	0.045	-0.042	0.015
3	11.488	0.045	0.042	-0.015
4	11.504	0.045	-0.042	-0.015



resulting tension force in (x/y)=(0.0/0.0): 0.000 [kN]
resulting compression force in (x/y)=(0.0/0.0): 0.000 [kN]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load EN 1992-4, Section 7.2.1

	Load [kN]	Capacity [kN]	Utilization β_N [%]	Status
Steel Strength*	11.504	156.828	8	OK
Pullout Strength*	11.504	84.823	14	OK
Concrete blowout failure in direction **	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	22.995	66.432	35	OK
Splitting failure**	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

1.3.1 Steel Strength

$N_{Ed} \leq N_{Rd,s} = \frac{N_{Rk,s}}{\gamma_{M,s}}$ EN 1992-4, Table 7.1

$N_{Rk,s}$ [kN]	$\gamma_{M,s}$	$N_{Rd,s}$ [kN]	N_{Ed} [kN]
235.242	1.500	156.828	11.504

1.3.2 Pullout Strength

$N_{Ed} \leq N_{Rd,p} = \frac{N_{Rk,p}}{\gamma_{M,p}}$ EN 1992-4, Table7.1

$N_{Rk,p} = k_2 \cdot A_h \cdot f_{ck}$ EN 1992-4, Eq. (7.11)

k_2	A_h [mm^2]	f_{ck} [N/mm^2]
7.500	565	30.00

$N_{Rk,p}$ [kN]	$\gamma_{M,p}$	$N_{Rd,p}$ [kN]	N_{Ed} [kN]
127.235	1.500	84.823	11.504

1.3.3 Concrete Breakout Failure

$N_{Ed} \le N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{M,c}}$	EN 1992-4, Table 7.1
$N_{Rk,c} = N^0_{Rk,c} \cdot \frac{A_{c,N}}{A^0_{c,N}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec1,N} \cdot \psi_{ec2,N} \cdot \psi_{M,N}$	EN 1992-4, Eq. (7.1)
$N^0_{Rk,c} = k_1 \cdot \sqrt{f_{ck}} \cdot h_{ef}^{1,5}$	EN 1992-4, Eq. (7.2)
$A^0_{c,N} = s_{cr,N} \cdot s_{cr,N}$	EN 1992-4, Eq. (7.3)
$\psi_{s,N} = 0.7 + 0.3 \cdot \frac{c}{c_{cr,N}} \le 1.00$	EN 1992-4, Eq. (7.4)
$\psi_{ec1,N} = \frac{1}{1 + \left(\frac{2 \cdot e_{N,1}}{s_{cr,N}}\right)} \le 1.00$	EN 1992-4, Eq. (7.6)
$\psi_{ec2,N} = \frac{1}{1 + \left(\frac{2 \cdot e_{N,2}}{s_{cr,N}}\right)} \le 1.00$	EN 1992-4, Eq. (7.6)
$\psi_{M,N} = 1$	EN 1992-4, Eq. (7.7)

$A_{c,N}$ [mm^2]	$A^0_{c,N}$ [mm^2]	$c_{cr,N}$ [mm]	$s_{cr,N}$ [mm]	$f_{c,cyl}$ [N/mm^2]
201,600	129,600	180.0	360.0	30.00

$e_{c1,N}$ [mm]	$\psi_{ec1,N}$	$e_{c2,N}$ [mm]	$\psi_{ec2,N}$	$\psi_{s,N}$	$\psi_{re,N}$
0.0	1.000	0.1	1.000	1.000	1.000

z [mm]	$\psi_{M,N}$	k_1	$N^0_{Rk,c}$ [kN]	$\gamma_{M,c}$	$N_{Rd,c}$ [kN]	N_{Ed} [kN]
0.8	1.000	8.900	64.080	1.500	66.432	22.995

Group anchor ID
2, 4

1.4 Shear load EN 1992-4, Section 7.2.2

	Load [kN]	Capacity [kN]	Utilization β_V [%]	Status
Steel Strength (without lever arm)*	0.045	94.097	1	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	0.084	132.845	1	OK
Concrete edge failure in direction x+**	0.089	52.282	1	OK

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength (without lever arm)

$V_{Ed} \le V_{Rd,s} = \frac{V_{Rk,s}}{\gamma_{M,s}}$	EN 1992-4, Table 7.2
$V_{Rk,s} = k_7 \cdot V^0_{Rk,s}$	EN 1992-4, Eq. (7.35)

$V^0_{Rk,s}$ [kN]	k_7	$V_{Rk,s}$ [kN]	$\gamma_{M,s}$	$V_{Rd,s}$ [kN]	V_{Ed} [kN]
117.621	1.000	117.621	1.250	94.097	0.045

1.4.2 Pryout Strength

$V_{Ed} \le V_{Rd,cp} = \frac{V_{Rk,cp}}{\gamma_{M,c,p}}$	EN 1992-4, Table 7.2
$V_{Rk,cp} = k_8 \cdot N_{Rk,c}$	EN 1992-4, Eq. (7.39a)
$N_{Rk,c} = N^0_{Rk,c} \cdot \frac{A_{c,N}}{A^0_{c,N}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec1,N} \cdot \psi_{ec2,N} \cdot \psi_{M,N}$	EN 1992-4, Eq. (7.1)

$N^0_{Rk,c}$ $= k_1 \cdot \sqrt{f_{ck}} \cdot h_{ef}^{1,5}$ EN 1992-4, Eq. (7.2)

$A^0_{c,N}$ $= s_{cr,N} \cdot s_{cr,N}$ EN 1992-4, Eq. (7.3)

$\psi_{s,N}$ $= 0.7 + 0.3 \cdot \frac{c}{c_{cr,N}} \leq 1.00$ EN 1992-4, Eq. (7.4)

$\psi_{ec1,N}$ $= \frac{1}{1 + \left(\frac{2 \cdot e_{V,1}}{s_{cr,N}}\right)} \leq 1.00$ EN 1992-4, Eq. (7.6)

$\psi_{ec2,N}$ $= \frac{1}{1 + \left(\frac{2 \cdot e_{V,2}}{s_{cr,N}}\right)} \leq 1.00$ EN 1992-4, Eq. (7.6)

$\psi_{M,N}$ $= 1$ EN 1992-4, Eq. (7.7)

$A_{c,N}$ [mm^2]	$A^0_{c,N}$ [mm^2]	$c_{cr,N}$ [mm]	$s_{cr,N}$ [mm]	k_8	$f_{c,cyl}$ [N/mm^2]
201,600	129,600	180.0	360.0	2.000	30.00

$e_{c1,V}$ [mm]	$\psi_{ec1,N}$	$e_{c2,V}$ [mm]	$\psi_{ec2,N}$	$\psi_{s,N}$	$\psi_{re,N}$	$\psi_{M,N}$
0.0	1.000	0.1	1.000	1.000	1.000	1.000

k_1	$N^0_{Rk,c}$ [kN]	$\gamma_{M,c,p}$	$V_{Rd,cp}$ [kN]	V_{Ed} [kN]
8.900	64.080	1.500	132.845	0.084

Group anchor ID
1, 3

1.4.3 Concrete edge failure in direction x+

$V_{Ed} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{M,c}}$ EN 1992-4, Table 7.2

$V_{Rk,c}$ $= k_T \cdot V^0_{Rk,c} \cdot \frac{A_{c,V}}{A^0_{c,V}} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ EN 1992-4, Eq. (7.40)

$V^0_{Rk,c}$ $= k_9 \cdot d_{nom}^{\alpha} \cdot l_f^{\beta} \cdot \sqrt{f_{ck}} \cdot c_1^{1,5}$ EN 1992-4, Eq. (7.41)

α $= 0.1 \cdot \left(\frac{l_f}{c_1}\right)^{0,5}$ EN 1992-4, Eq. (7.42)

β $= 0.1 \cdot \left(\frac{d_{nom}}{c_1}\right)^{0,2}$ EN 1992-4, Eq. (7.43)

$A^0_{c,V}$ $= 4.5 \cdot c_1^2$ EN 1992-4, Eq. (7.44)

$\psi_{s,V}$ $= 0.7 + 0.3 \cdot \frac{c_2}{1.5 \cdot c_1} \leq 1.00$ EN 1992-4, Eq. (7.45)

$\psi_{h,V}$ $= \left(\frac{1.5 \cdot c_1}{h}\right)^{0,5} \geq 1.00$ EN 1992-4, Eq. (7.46)

$\psi_{ec,V}$ $= \frac{1}{1 + \left(\frac{2 \cdot e_V}{3 \cdot c_1}\right)} \leq 1.00$ EN 1992-4, Eq. (7.47)

$\psi_{\alpha,V}$ $= \sqrt{\frac{1}{(\cos \alpha_V)^2 + (0.5 \cdot \sin \alpha_V)^2}} \geq 1.00$ EN 1992-4, Eq. (7.48)

l_f [mm]	d_{nom} [mm]	k_9	α	β	$f_{c,cyl}$ [N/mm^2]
120.0	24.00	1.700	0.058	0.058	30.00

c_1 [mm]	$A_{c,V}$ [mm^2]	$A^0_{c,V}$ [mm^2]
360.0	486,000	583,200

$\psi_{s,V}$	$\psi_{h,V}$	α_V [°]	$\psi_{\alpha,V}$	$e_{c,V}$ [mm]	$\psi_{ec,V}$	$\psi_{re,V}$
0.894	1.000	19.02	1.042	0.1	1.000	1.000

$V^0_{Rk,c}$ [kN]	k_T	$\gamma_{M,c}$	$V_{Rd,c}$ [kN]	V_{Ed} [kN]
100.953	1.0	1.500	52.282	0.089

1.5 Combined tension and shear loads (EN 1992-4, Section 7.2.3)

Steel failure

β_N	β_V	α	Utilization $\beta_{N,V}$ [%]	Status
0.073	0.000	2.000	1	OK

$\beta_N^{\alpha} + \beta_V^{\alpha} \leq 1.0$

Concrete failure

β_N	β_V	α	Utilization $\beta_{N,V}$ [%]	Status
0.346	0.002	1.500	21	OK

$\beta_N^{\alpha} + \beta_V^{\alpha} \leq 1.0$

1.7 Installation data

Anchor plate, steel: S 235; E = 210,000.00 N/mm^2; f_{yk} = 235.00 N/mm^2
Profile: IPBi/HEA, IPBI 140 / HE 140 A; (L x W x T x FT) = 133.0 mm x 140.0 mm x 5.5 mm x 8.5 mm

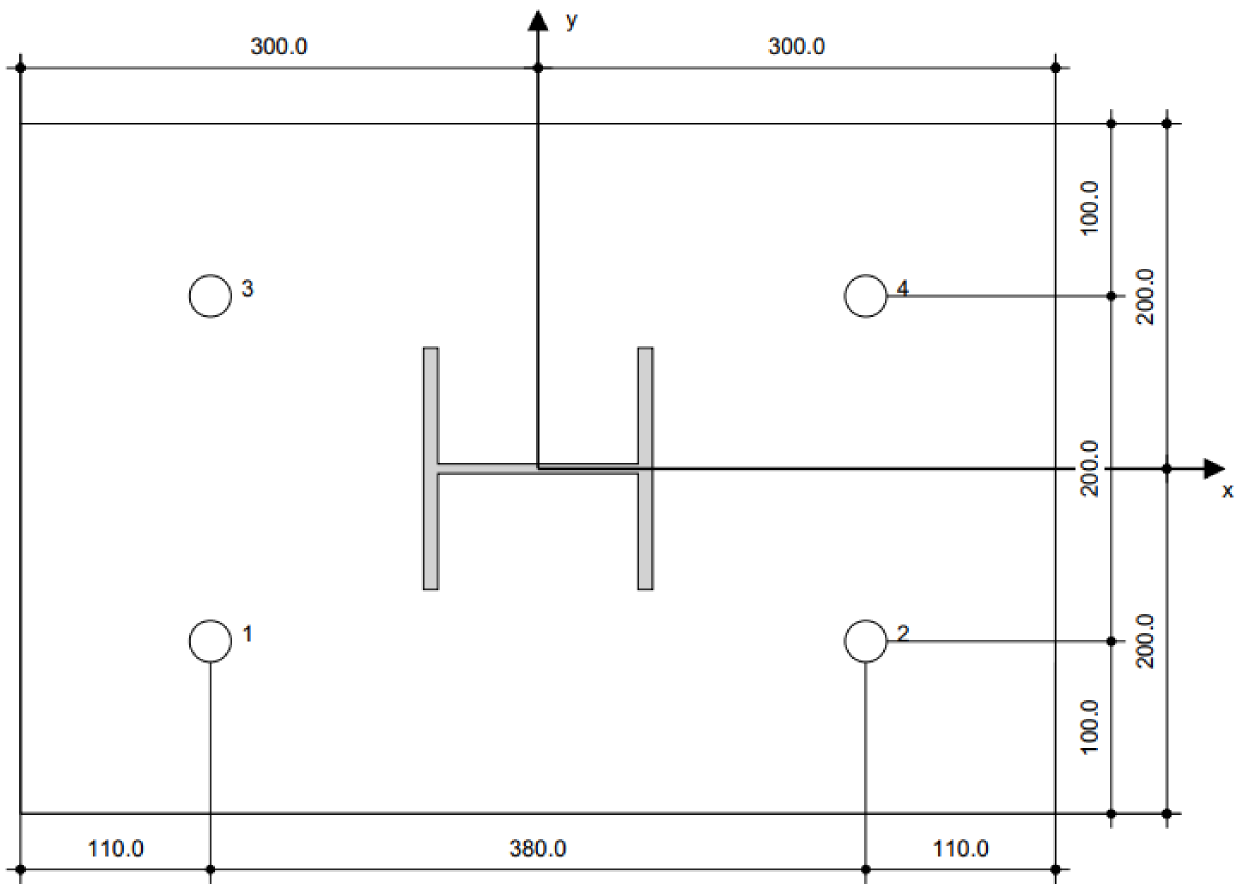
Hole diameter in the fixture: d_f = 24.0 mm

Plate thickness (input): 20.0 mm

Anchor type and diameter: Headed fastener 5.8 M24

Item number: not available

Minimum thickness of the base material: 0.0 mm



Load Case Combinations (LCC)

The load case combinations for the stress analysis and steel design of the structure follow limit states below :

a) Ultimate limit states ULS (STR/GEO)

b) Quasi-Permanent SLS

c) Ultimate limit states ULS (STR/GEO) – Seismic

LCC No.	Type of	LC1	LC2	LC3	LC4	LC5	LC6	LC7	LC8	LC9	LC11
1	DS1 - ULS (STR/GEO)	1.35									
2	DS1 - ULS (STR/GEO)	1.35	1.5								
3	DS1 - ULS (STR/GEO)	1.35		1.5							
4	DS1 - ULS (STR/GEO)	1.35			1.5						
5	DS1 - ULS (STR/GEO)	1.35	1.5			0.9					
6	DS1 - ULS (STR/GEO)	1.35	1.5				0.9				
7	DS1 - ULS (STR/GEO)	1.35	1.5					0.9			
8	DS1 - ULS (STR/GEO)	1.35		1.5		0.9					
9	DS1 - ULS (STR/GEO)	1.35		1.5			0.9				
10	DS1 - ULS (STR/GEO)	1.35		1.5				0.9			
11	DS1 - ULS (STR/GEO)	1.35			1.5	0.9					
12	DS1 - ULS (STR/GEO)	1.35			1.5		0.9				
13	DS1 - ULS (STR/GEO)	1.35			1.5			0.9			
14	DS1 - ULS (STR/GEO)	1.35	1.5			0.9			0.9		
15	DS1 - ULS (STR/GEO)	1.35	1.5			0.9			0.9	0.9	
16	DS1 - ULS (STR/GEO)	1.35	1.5			0.9				0.9	
17	DS1 - ULS (STR/GEO)	1.35	1.5				0.9		0.9		
18	DS1 - ULS (STR/GEO)	1.35	1.5				0.9		0.9	0.9	
19	DS1 - ULS (STR/GEO)	1.35	1.5				0.9			0.9	
20	DS1 - ULS (STR/GEO)	1.35	1.5					0.9	0.9		
21	DS1 - ULS (STR/GEO)	1.35	1.5					0.9	0.9	0.9	
22	DS1 - ULS (STR/GEO)	1.35	1.5					0.9		0.9	
23	DS1 - ULS (STR/GEO)	1.35		1.5		0.9			0.9		
24	DS1 - ULS (STR/GEO)	1.35		1.5		0.9			0.9	0.9	
25	DS1 - ULS (STR/GEO)	1.35		1.5		0.9				0.9	
26	DS1 - ULS (STR/GEO)	1.35		1.5			0.9		0.9		
27	DS1 - ULS (STR/GEO)	1.35		1.5			0.9		0.9	0.9	
28	DS1 - ULS (STR/GEO)	1.35		1.5			0.9			0.9	
29	DS1 - ULS (STR/GEO)	1.35		1.5				0.9	0.9		
30	DS1 - ULS (STR/GEO)	1.35		1.5				0.9	0.9	0.9	
31	DS1 - ULS (STR/GEO)	1.35		1.5				0.9		0.9	
32	DS1 - ULS (STR/GEO)	1.35			1.5	0.9			0.9		
33	DS1 - ULS (STR/GEO)	1.35			1.5	0.9			0.9	0.9	
34	DS1 - ULS (STR/GEO)	1.35			1.5	0.9				0.9	
35	DS1 - ULS (STR/GEO)	1.35			1.5		0.9		0.9		
36	DS1 - ULS (STR/GEO)	1.35			1.5		0.9		0.9	0.9	
37	DS1 - ULS (STR/GEO)	1.35			1.5		0.9			0.9	
38	DS1 - ULS (STR/GEO)	1.35			1.5			0.9	0.9		
39	DS1 - ULS (STR/GEO)	1.35			1.5			0.9	0.9	0.9	
40	DS1 - ULS (STR/GEO)	1.35			1.5			0.9		0.9	
41	DS1 - ULS (STR/GEO)	1.35	1.5						0.9		
42	DS1 - ULS (STR/GEO)	1.35	1.5						0.9	0.9	
43	DS1 - ULS (STR/GEO)	1.35	1.5							0.9	
44	DS1 - ULS (STR/GEO)	1.35		1.5					0.9		
45	DS1 - ULS (STR/GEO)	1.35		1.5					0.9	0.9	
46	DS1 - ULS (STR/GEO)	1.35		1.5						0.9	
47	DS1 - ULS (STR/GEO)	1.35			1.5				0.9		
48	DS1 - ULS (STR/GEO)	1.35			1.5				0.9	0.9	
49	DS1 - ULS (STR/GEO)	1.35			1.5					0.9	
50	DS1 - ULS (STR/GEO)	1.35				1.5					
51	DS1 - ULS (STR/GEO)	1.35					1.5				

52	DS1 - ULS (STR/GEO)	1.35						1.5			
53	DS1 - ULS (STR/GEO)	1.35	0.75			1.5					
54	DS1 - ULS (STR/GEO)	1.35	0.75				1.5				
55	DS1 - ULS (STR/GEO)	1.35	0.75					1.5			
56	DS1 - ULS (STR/GEO)	1.35		0.75		1.5					
57	DS1 - ULS (STR/GEO)	1.35		0.75			1.5				
58	DS1 - ULS (STR/GEO)	1.35		0.75				1.5			
59	DS1 - ULS (STR/GEO)	1.35			0.75	1.5					
60	DS1 - ULS (STR/GEO)	1.35			0.75		1.5				
61	DS1 - ULS (STR/GEO)	1.35			0.75			1.5			
62	DS1 - ULS (STR/GEO)	1.35	0.75			1.5			0.9		
63	DS1 - ULS (STR/GEO)	1.35	0.75			1.5			0.9	0.9	
64	DS1 - ULS (STR/GEO)	1.35	0.75			1.5				0.9	
65	DS1 - ULS (STR/GEO)	1.35	0.75				1.5		0.9		
66	DS1 - ULS (STR/GEO)	1.35	0.75				1.5		0.9	0.9	
67	DS1 - ULS (STR/GEO)	1.35	0.75				1.5			0.9	
68	DS1 - ULS (STR/GEO)	1.35	0.75					1.5	0.9		
69	DS1 - ULS (STR/GEO)	1.35	0.75					1.5	0.9	0.9	
70	DS1 - ULS (STR/GEO)	1.35	0.75					1.5		0.9	
71	DS1 - ULS (STR/GEO)	1.35		0.75		1.5			0.9		
72	DS1 - ULS (STR/GEO)	1.35		0.75		1.5			0.9	0.9	
73	DS1 - ULS (STR/GEO)	1.35		0.75		1.5				0.9	
74	DS1 - ULS (STR/GEO)	1.35		0.75			1.5		0.9		
75	DS1 - ULS (STR/GEO)	1.35		0.75			1.5		0.9	0.9	
76	DS1 - ULS (STR/GEO)	1.35		0.75			1.5			0.9	
77	DS1 - ULS (STR/GEO)	1.35		0.75				1.5	0.9		
78	DS1 - ULS (STR/GEO)	1.35		0.75				1.5	0.9	0.9	
79	DS1 - ULS (STR/GEO)	1.35		0.75				1.5		0.9	
80	DS1 - ULS (STR/GEO)	1.35			0.75	1.5			0.9		
81	DS1 - ULS (STR/GEO)	1.35			0.75	1.5			0.9	0.9	
82	DS1 - ULS (STR/GEO)	1.35			0.75	1.5				0.9	
83	DS1 - ULS (STR/GEO)	1.35			0.75		1.5		0.9		
84	DS1 - ULS (STR/GEO)	1.35			0.75		1.5		0.9	0.9	
85	DS1 - ULS (STR/GEO)	1.35			0.75		1.5			0.9	
86	DS1 - ULS (STR/GEO)	1.35			0.75			1.5	0.9		
87	DS1 - ULS (STR/GEO)	1.35			0.75			1.5	0.9	0.9	
88	DS1 - ULS (STR/GEO)	1.35			0.75			1.5		0.9	
89	DS1 - ULS (STR/GEO)	1.35				1.5			0.9		
90	DS1 - ULS (STR/GEO)	1.35				1.5			0.9	0.9	
91	DS1 - ULS (STR/GEO)	1.35				1.5				0.9	
92	DS1 - ULS (STR/GEO)	1.35					1.5		0.9		
93	DS1 - ULS (STR/GEO)	1.35					1.5		0.9	0.9	
94	DS1 - ULS (STR/GEO)	1.35					1.5			0.9	
95	DS1 - ULS (STR/GEO)	1.35						1.5	0.9		
96	DS1 - ULS (STR/GEO)	1.35						1.5	0.9	0.9	
97	DS1 - ULS (STR/GEO)	1.35						1.5		0.9	
98	DS1 - ULS (STR/GEO)	1.35							1.5		
99	DS1 - ULS (STR/GEO)	1.35							1.5	1.5	
100	DS1 - ULS (STR/GEO)	1.35								1.5	
101	DS1 - ULS (STR/GEO)	1.35	0.75						1.5		
102	DS1 - ULS (STR/GEO)	1.35	0.75						1.5	1.5	
103	DS1 - ULS (STR/GEO)	1.35	0.75							1.5	
104	DS1 - ULS (STR/GEO)	1.35		0.75					1.5		
105	DS1 - ULS (STR/GEO)	1.35		0.75					1.5	1.5	
106	DS1 - ULS (STR/GEO)	1.35		0.75						1.5	
107	DS1 - ULS (STR/GEO)	1.35			0.75				1.5		
108	DS1 - ULS (STR/GEO)	1.35			0.75				1.5	1.5	
109	DS1 - ULS (STR/GEO)	1.35			0.75					1.5	
110	DS1 - ULS (STR/GEO)	1.35	0.75			0.9			1.5		
111	DS1 - ULS (STR/GEO)	1.35	0.75			0.9			1.5	1.5	
112	DS1 - ULS (STR/GEO)	1.35	0.75			0.9				1.5	
113	DS1 - ULS (STR/GEO)	1.35	0.75				0.9		1.5		
114	DS1 - ULS (STR/GEO)	1.35	0.75				0.9		1.5	1.5	
115	DS1 - ULS (STR/GEO)	1.35	0.75				0.9			1.5	
116	DS1 - ULS (STR/GEO)	1.35	0.75					0.9	1.5		

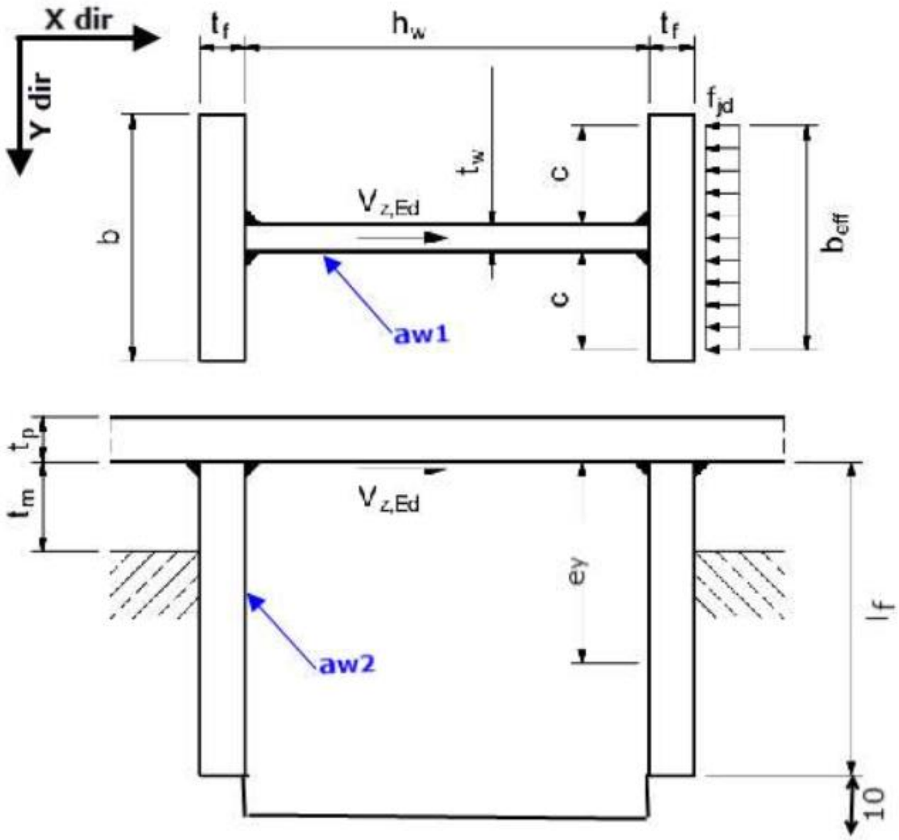
117	DS1 - ULS (STR/GEO)	1.35	0.75					0.9	1.5	1.5	
118	DS1 - ULS (STR/GEO)	1.35	0.75					0.9		1.5	
119	DS1 - ULS (STR/GEO)	1.35		0.75		0.9			1.5		
120	DS1 - ULS (STR/GEO)	1.35		0.75		0.9			1.5	1.5	
121	DS1 - ULS (STR/GEO)	1.35		0.75		0.9				1.5	
122	DS1 - ULS (STR/GEO)	1.35		0.75			0.9		1.5		
123	DS1 - ULS (STR/GEO)	1.35		0.75			0.9		1.5	1.5	
124	DS1 - ULS (STR/GEO)	1.35		0.75			0.9			1.5	
125	DS1 - ULS (STR/GEO)	1.35		0.75				0.9	1.5		
126	DS1 - ULS (STR/GEO)	1.35		0.75				0.9	1.5	1.5	
127	DS1 - ULS (STR/GEO)	1.35		0.75				0.9		1.5	
128	DS1 - ULS (STR/GEO)	1.35			0.75	0.9			1.5		
129	DS1 - ULS (STR/GEO)	1.35			0.75	0.9			1.5	1.5	
130	DS1 - ULS (STR/GEO)	1.35			0.75	0.9				1.5	
131	DS1 - ULS (STR/GEO)	1.35			0.75		0.9		1.5		
132	DS1 - ULS (STR/GEO)	1.35			0.75		0.9		1.5	1.5	
133	DS1 - ULS (STR/GEO)	1.35			0.75		0.9			1.5	
134	DS1 - ULS (STR/GEO)	1.35			0.75			0.9	1.5		
135	DS1 - ULS (STR/GEO)	1.35			0.75			0.9	1.5	1.5	
136	DS1 - ULS (STR/GEO)	1.35			0.75			0.9		1.5	
137	DS1 - ULS (STR/GEO)	1.35				0.9			1.5		
138	DS1 - ULS (STR/GEO)	1.35				0.9			1.5	1.5	
139	DS1 - ULS (STR/GEO)	1.35				0.9				1.5	
140	DS1 - ULS (STR/GEO)	1.35					0.9		1.5		
141	DS1 - ULS (STR/GEO)	1.35					0.9		1.5	1.5	
142	DS1 - ULS (STR/GEO)	1.35					0.9			1.5	
143	DS1 - ULS (STR/GEO)	1.35						0.9	1.5		
144	DS1 - ULS (STR/GEO)	1.35						0.9	1.5	1.5	
145	DS1 - ULS (STR/GEO)	1.35						0.9		1.5	
146	DS4 - SLS	1									
147	DS6 - ULS - Seismic	1									1

Table 14. Load combinations

Detail check of Base shear lug

Description : Calculates the strength of the shear key of the support base plate for all direction acting forces.
Based on: Beispiele zur Bemessung von Stahltragwerken nach DIN EN 1993 Eurocode 3
© 2012 Wilhelm Ernst & Sohn, Verlag für Architektur und technische Wissenschaften GmbH & Co. KG, Rotherstr. 21, 10245 Berlin, Germany - ISBN 978-3-433-02961-9 - Beispiel 2.7

Force paralell to web:



$h_w := 220mm$	$t_w := 20mm$	Web dimensions [mm]
$b := 220mm$	$t_f := 30mm$	Flange dimensions [mm]
$t_p := 20mm$		Thickness of base plate [mm]
$t_m := 0mm$		Thickness of grouting [mm]
$l_f := 265mm$	$h := h_w + 2 \cdot t_f = 280 \cdot mm$	Height of shear key [mm]

$V_{z.Ed} := 436.27kN$ Max. horizontal load at **X-dir.** [kN]

$V_{y.Ed} := 344.7kN$ Max. horizontal load at **Y-dir**. [kN]

Comb. of horizontal loads when acting at the same time

$a_{w1} := 10mm \quad < \quad \min\left(\frac{t_w}{2}, t_p \cdot 0.7\right) = 10.0 \cdot mm$ (2 sided fillet weld)

$a_{w2} := 10mm \quad < \quad \min\left(\frac{t_w}{2}, t_f \cdot 0.7\right) = 10.0 \cdot mm$ (2 sided fillet weld)

$a_{w3} := 10mm$ < $\min\left(\frac{t_f}{2}, t_p \cdot 0.7\right) = 14.0 \cdot mm$ (2 sided fillet weld)

$f_y := 335 \cdot \frac{N}{mm^2}$ $\varepsilon_s := \sqrt{\frac{235}{f_y} \cdot MPa} = 0.838$ Yield stress [N/mm²]

$f_u := 470 \frac{N}{mm^2}$ Ultimate tensile stress [N/mm²]

$f_{ck} := 30 \cdot \frac{N}{mm^2}$ Characteristic compressive cylinder strength of concrete after 28 days (f.c1 = f'c) [N/mm²]

$\gamma_{M0} := 1.05$ $\gamma_{M1} := 1.05$ $\gamma_{M2} := 1.25$ Safety factors of steel

$\gamma_c := 1.50$ Safety factor of concrete

$$f_{cd} := \frac{f_{ck}}{\gamma_c} = 20.00 \cdot \frac{N}{mm^2}$$

$\beta_j := \frac{2}{3}$ $A_{c1\_c0} := \sqrt{3}$ $\beta_w := 0.9$ Parameters of design

Horizontal load in Z-dir.:

Capacity of the concrete behind the flange:

$$f_{jd} := \beta_j \cdot f_{cd} \cdot A_{c1\_c0} = 2.31 \cdot \frac{kN}{cm^2}$$

$$c_c := t_f \cdot \sqrt{\frac{f_y}{3 \cdot f_{jd} \cdot \gamma_{M0}}} = 64.4 \cdot mm$$

$$b_{eff} := \min\left(2 \cdot c_c + t_w, b\right) = 148.8 \cdot mm$$

$$\boxed{F_{c.Rd} := \left(l_f - t_m\right) \cdot b_{eff} \cdot f_{jd} = 910.4 \cdot kN}$$

Capacity of the web:

$\eta := 1.20$

$\frac{h_w}{t_w} = 11.0$ < $72 \cdot \frac{\varepsilon_s}{\eta} = 50.253$ so It's no local web buckling

$A_v := \eta \cdot h_w \cdot t_w = 52.8 \cdot cm^2$

$$\boxed{V_{web.Rd} := \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = 972.6 \cdot kN}$$

Capacity of web-baseplate welding:

$l_{w1} := h_w - 2a_{w2} = 200 \cdot mm$

$$f_{vw.d} := \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} = 24.12 \cdot \frac{kN}{cm^2}$$

$$\boxed{V_{w.1.Rd} := 2 \cdot a_{w1} \cdot l_{w1} \cdot f_{vw.d} = 964.8 \cdot kN}$$

Capacity of web-flange welding:

$$I_y := \frac{h^3 \cdot b - h_w^{\,3} \cdot (b - t_w)}{12} = 22499 \cdot cm^4$$

$a_f := h - t_f = 250 \cdot mm$

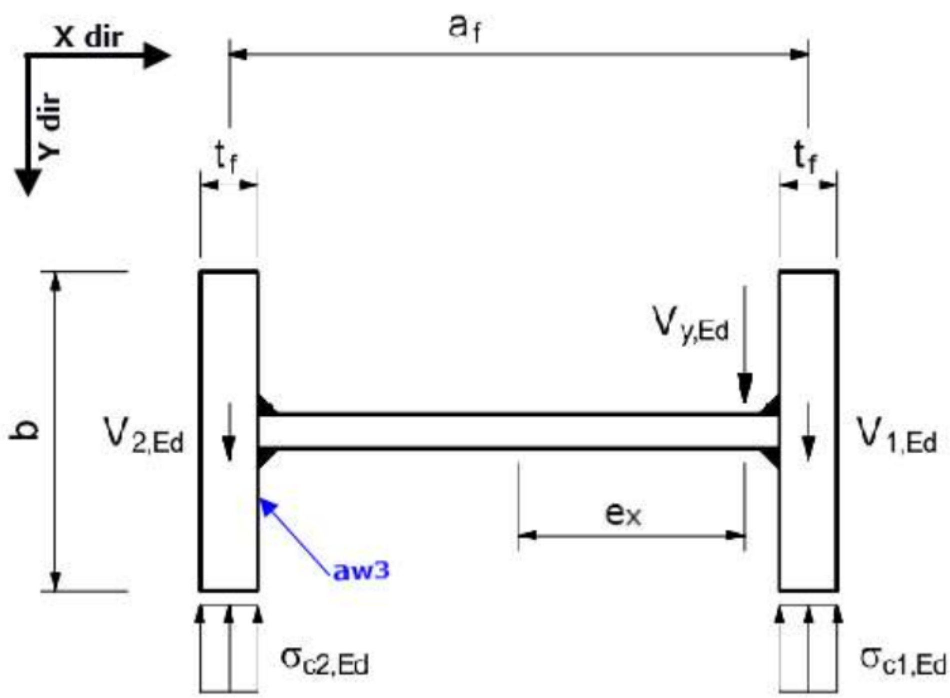
$S_{yf} := 0.5 \cdot a_f \cdot b \cdot t_f = 825 \cdot cm^3$

$$\boxed{V_{w.2.Rd} := 2 \cdot a_{w2} \cdot f_{vw.d} \cdot \frac{I_y}{S_{yf}} = 1.3 \times 10^3 \cdot kN}$$

$$\boxed{V_{z.Rd} := \min\left(F_{c.Rd}, V_{web.Rd}, V_{w.1.Rd}, V_{w.2.Rd}\right) = 910.4 \cdot kN}$$

$\frac{V_{z.Ed}}{V_{z.Rd}} = 0.48$ < 1.0 OK!

Force perpendicular to web:



Horizontal load in Y-direction:

Torsion: $T_{x.Ed} := 0kN \cdot m$

Eccentricity: $e_x := \dfrac{T_{x.Ed}}{V_{y.Ed}} = 0 \cdot mm$ horizontal

$e_y := (l_f + t_m) \cdot 0.5 = 132.5 \cdot mm$ vertical

$$\boxed{V_{1.Ed} := V_{y.Ed} \cdot \left(\frac{1}{2} + \frac{e_x}{a_f}\right) = 172.3 \cdot kN}$$

Maximum concrete pressure:

$$\boxed{F_{c1.Rd} := (l_f - t_m) \cdot t_f \cdot f_{jd} = 183.6 \cdot kN}$$

Capacity of the flange:

$A_{v.f} := b \cdot t_f = 66 \cdot cm^2$

$$\boxed{V_{f1.Rd} := \frac{A_{v.f} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = 1215.7 \cdot kN}$$

$$\boxed{V_{y1.Rd} := \min(F_{c1.Rd}, V_{f1.Rd}) = 183.6 \cdot kN}$$

$\dfrac{V_{1.Ed}}{V_{y1.Rd}} = 0.94$ < 1.0 OK!

Capacity of a flange-baseplate welding:

$$l_{w3} := \frac{1}{2}\left(2\cdot b - t_w - 2\cdot a_{w2}\right) = 200\cdot mm$$

$$F_{T.w.Ed} := \frac{V_{z.Ed}}{l_{w3}}\cdot\frac{e_y}{a_f} + \frac{6\cdot V_{1.Ed}}{l_{w3}^{\,2}}\cdot e_y = 45.82\cdot\frac{kN}{cm}$$

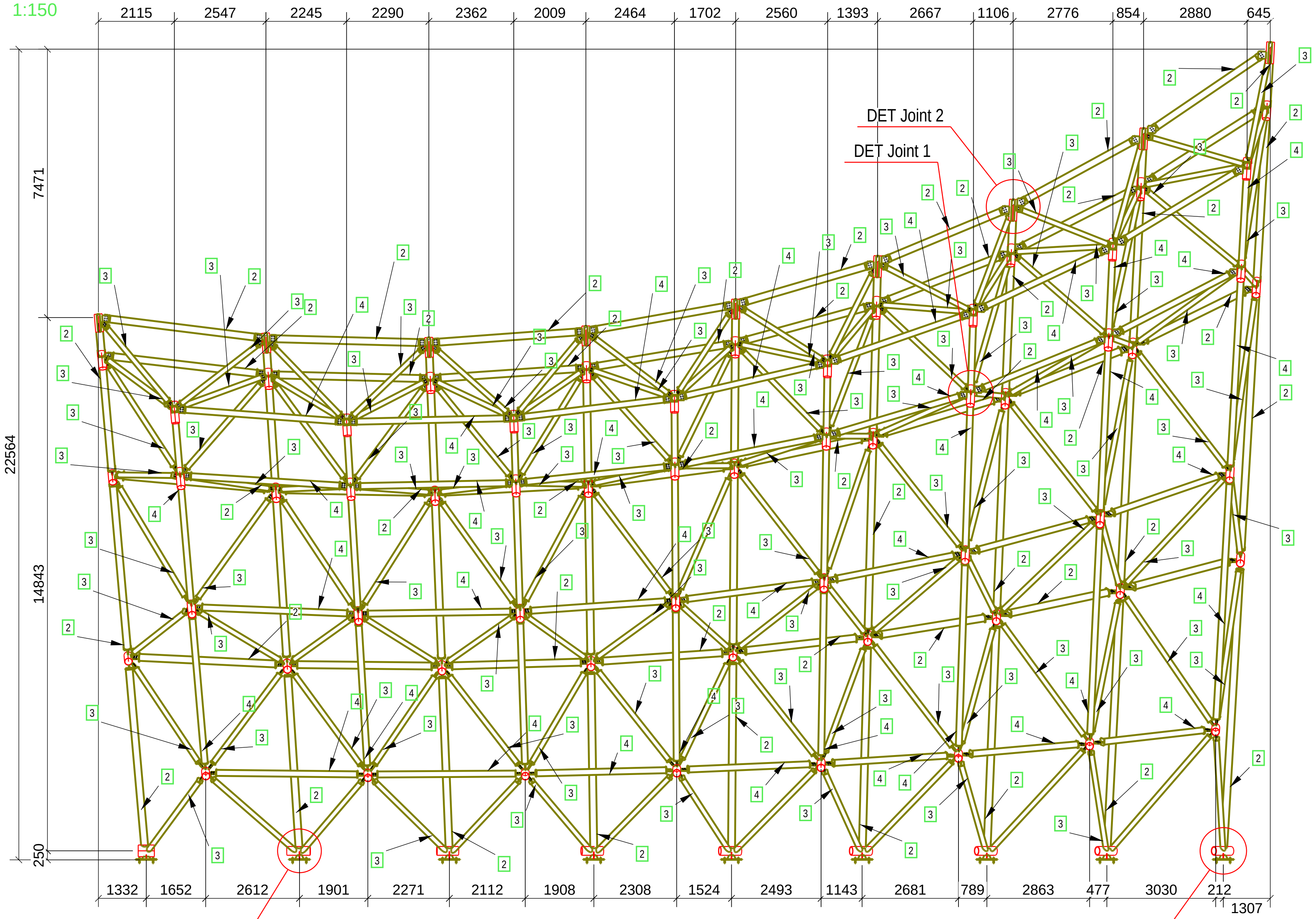
$$F_{II.w.Ed} := \frac{V_{1.Ed}}{l_{w3}} = 8.62\cdot\frac{kN}{cm}$$

$$\boxed{F_{w3.Ed} := \frac{1}{2}\cdot\sqrt{F_{T.w.Ed}^{\,2} + F_{II.w.Ed}^{\,2}} = 23.31\cdot\frac{kN}{cm}}$$

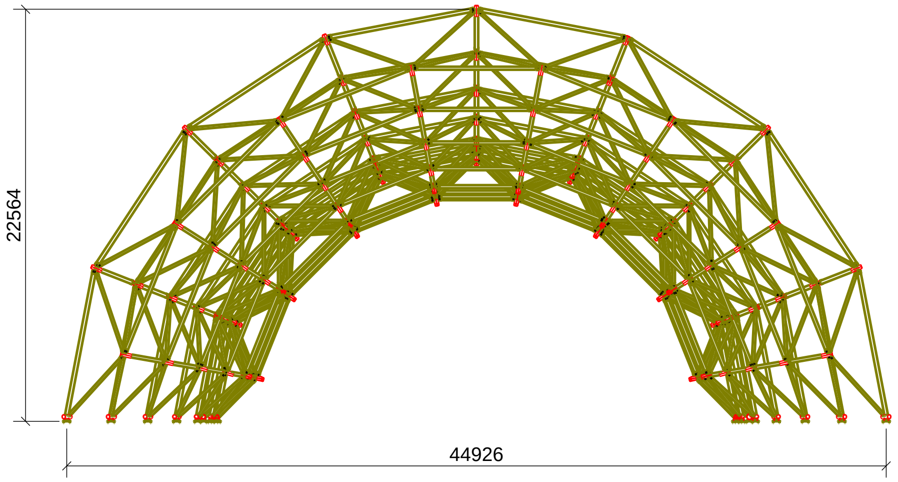
$$\boxed{F_{w3.Rd} := f_{vw.d}\cdot a_{w3} = 24.12\cdot\frac{kN}{cm}}$$

$$\frac{F_{w3.Ed}}{F_{w3.Rd}} = 0.97 \quad < \quad 1.0$$ OK!

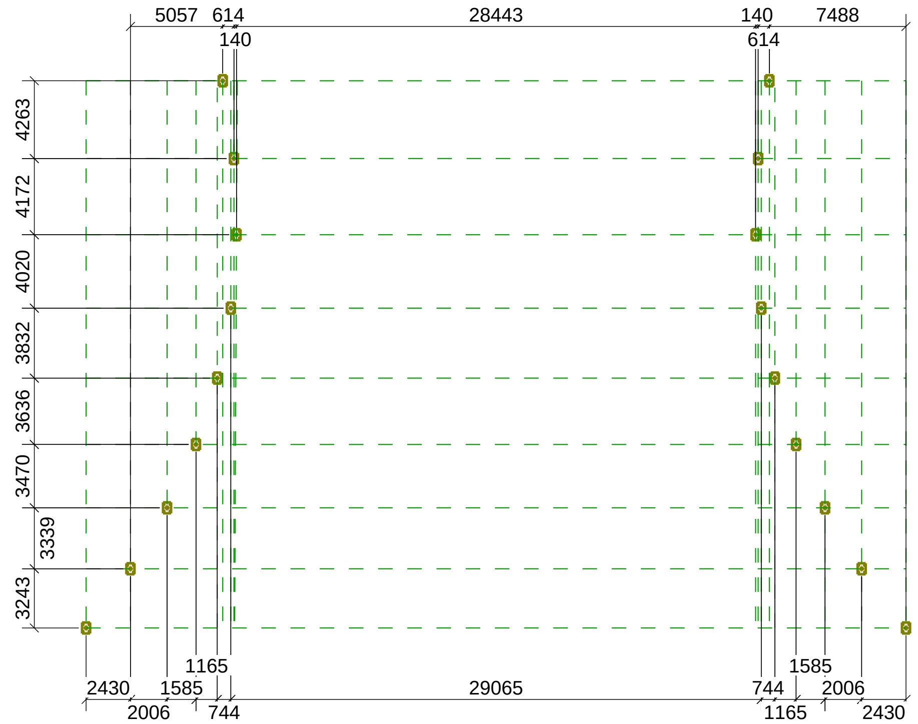
Side View
1:150



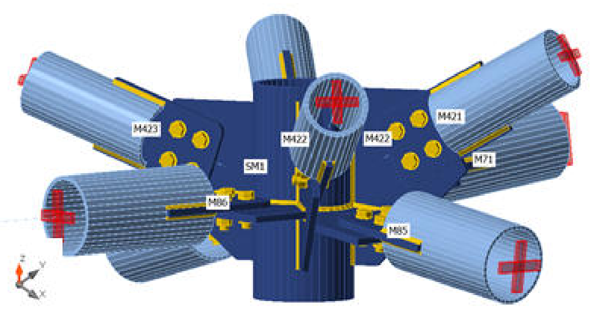
Front View
1:400



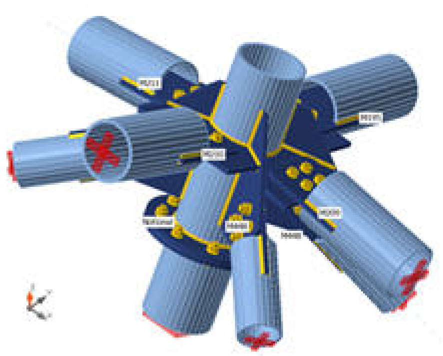
- 0.15 Ground Base
1:400



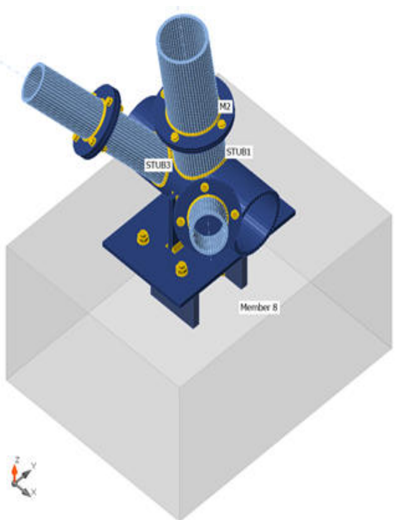
DET Joint 1



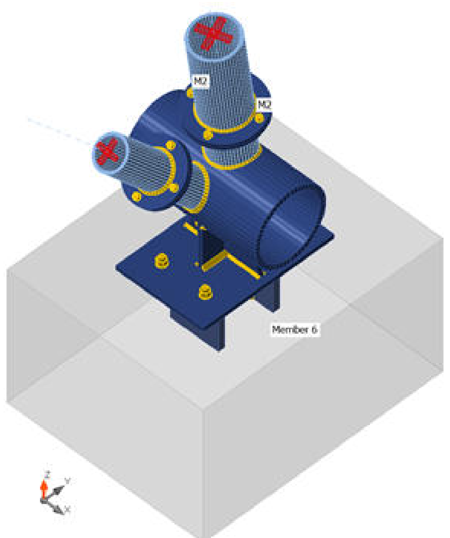
DET Joint 2



DET Base 1



DET Base 2

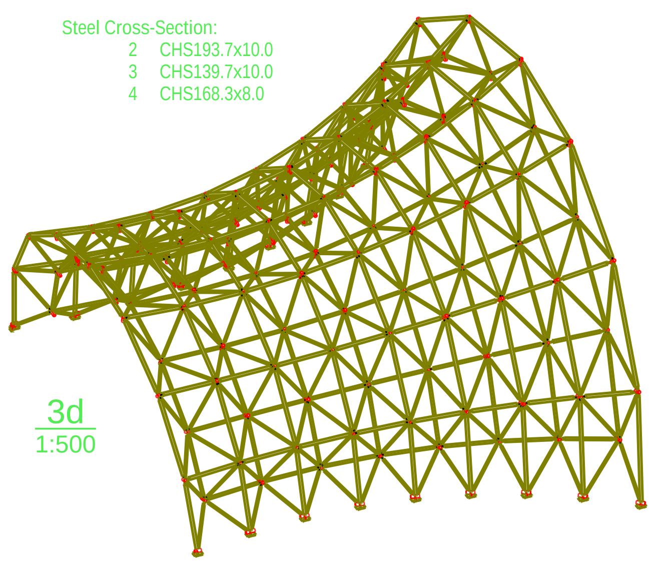


Notes:

Ground Leverl: - 150.0 mm

Steel Material: S355

Steel Cross-Section:
- 2 CHS193.7x10.0
- 3 CHS139.7x10.0
- 4 CHS168.3x8.0



3d
1:500

Project		Project Number	1
Design of an open-air stage roof structure			
Drawing Title	General Arrangement Drawing		
Drawn by	T.H Tho	Scale @A4	1:150
Designed by	T.H Tho	Date	27.11.2023
Checked by	Dr. Péter Hegyi	Drawing Number	1
Approved by		Revision	

Tekla Structures