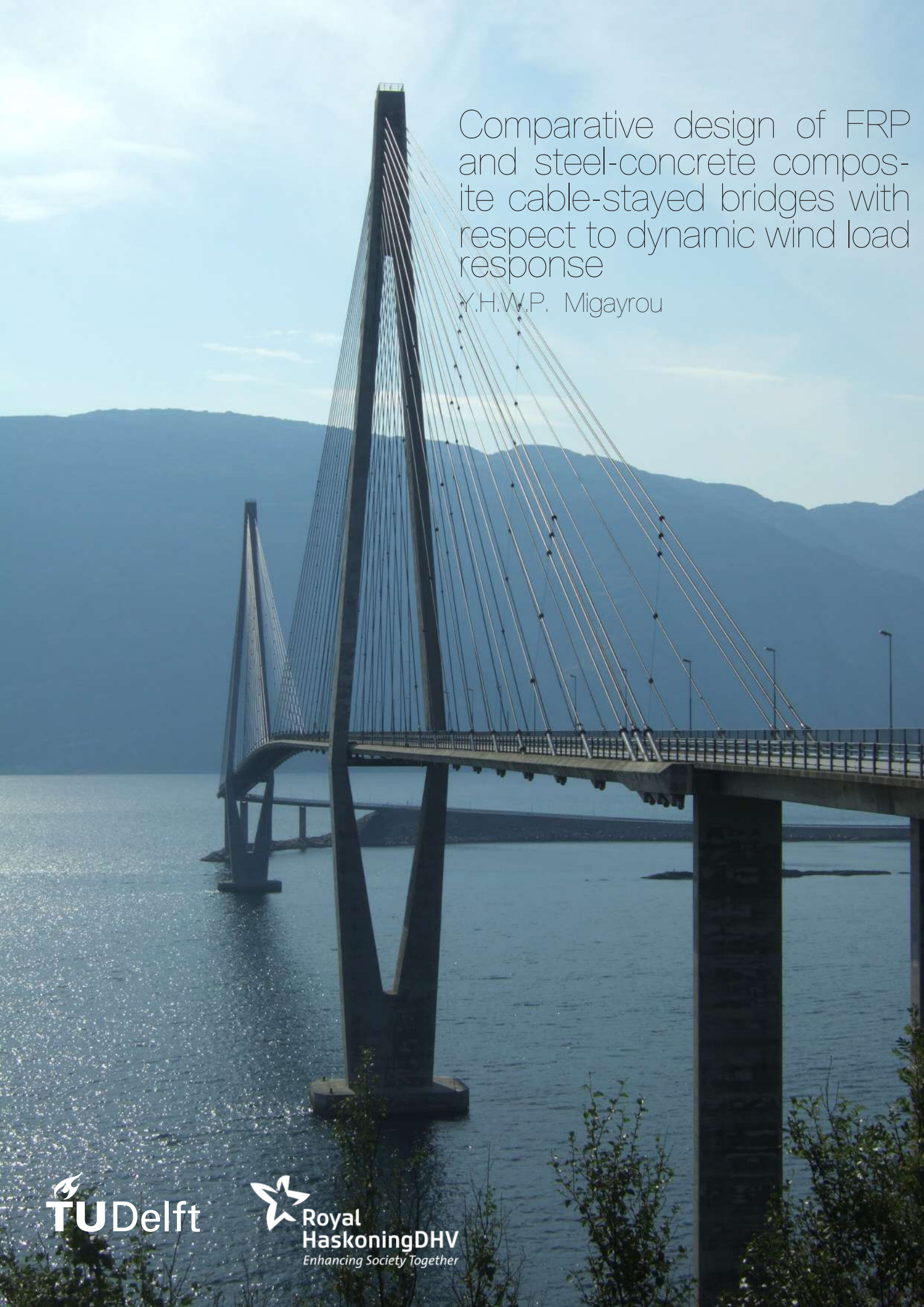




Comparative design of FRP and steel-concrete composite cable-stayed bridges with respect to dynamic wind load response

Y.H.W.P. Migayrou

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by

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Preface

This thesis is written as the final part of the Master of Science in Structural Engineering at the faculty of Civil Engineering and Geosciences at the Delft University of Technology. It has been performed in collaboration with Royal HaskoningDHV, in Rotterdam. I would like to express my gratitude for the opportunity they gave me, as well as the guidance and assistance that came with it.

First of all I would like to thank my entire graduation committee for their involvement in this research. Thank you Ton Boeters for our weekly meetings, your insight and problem-solving mindset which helped me throughout the entire project. Thank you Marko Pavlovic for your constructive feedback and advice. Thank you Milan Veljkovic for keeping an eye on the bigger picture, and the valuable feedback that came along. Thank you Max Hendriks for your advice on a proper completion of this thesis. I would also like to thank Liesbeth Tromp who gave me advice on all FRP related questions, as well as useful feedback on the entire thesis.

Last but not least I would like to thank my family for supporting me when needed. Special thanks to Tanya who listened to me and supported me during the entire project.

*Y.H.W.P. Migayou
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Abstract

For a housing project in Sirdal, Norway, a bridge is required in order to reach the construction site. Royal HaskoningDHV is interested in the possibilities that FRP composites might offer in terms of bridge design for this situation. Added to this, the fact that construction is mainly possible from one side of the river lead to the idea of an FRP cable-stayed bridge (the deck is made from FRP composite, however the cables and the pylon are respectively made from steel and concrete). The main span is 64 meters, which is large for an FRP bridge. Combined with its lightweight, low stiffness and structural damping, the structural response to wind excitation might be an issue. Next to designing an FRP cable-stayed bridge, this thesis therefore focuses in a second part on the dynamic behaviour to wind excitation of the bridge. To have a point of comparison with a more 'traditional' cable-stayed bridge, in parallel to the FRP bridge, a steel-concrete alternative has been designed and analyzed.

In the first part of the thesis, a literature study has been performed in order to identify relevant topics with respect to FRP composites and cable-stayed bridges. It became clear that the possibility of an FRP cable-stayed bridge had barely been researched. A few relevant topics with respect to wind dynamics have also been given attention.

In the second part of the thesis, initial sizing of the two cable-stayed bridges has been performed. This has first been done analytically in order to determine initial dimensions of the structure. For the FRP cable-stayed bridge, the lack of literature means that the optimal cable anchor distance has to be determined. This has been done with a parameter study, which ended with the conclusion that an optimal cable anchor distance is 8 meters. The critical check for the FRP bridge is the deflection in the main span. This means the design is stiffness based, unlike the steel-concrete bridge where the design is based on a bending moment strength check. The cable anchor distance for the steel-concrete bridge is found to be 12.8 meters which is confirmed by literature. Dynamic wind checks according to EN1991-1-4, annex E, have been performed and indicated that the FRP bridge is sensitive to flutter. This resulted in minimum values for the deck height in order for the deck to be safe against flutter.

After obtaining these preliminary designs from hand calculations, these have been modelled in Sofistik. ULS and SLS checks have been performed. Two main issues have been identified, namely cable pretension loss and local force introduction at the cable-deck connection. These were not included in the analytic calculations. The designs have been improved to obtain the preliminary designs ready to be analyzed under dynamic wind loads. As a point of comparison, the production costs of the bridges have been investigated. The FRP bridge is 31 % more expensive than the steel-concrete bridge, from unit price estimates. If the weights per m^2 are compared, the FRP bridge is equal to $\frac{1}{4}$ of the weight of the steel-concrete composite bridge. Life cycle costs have not been investigated.

In the third part of the thesis, the important parameters for wind analysis have been determined: Rayleigh damping, the wind load model and the wind coefficients of the decks. Rayleigh damping has been defined with the help of a modal analysis of the bridges. The wind load model has been defined with knowledge from the literature study and Sofistik. The wind coefficients have been defined based on the results from the CFD simulations. Sofistik includes this functionality with the Dolfyn module. Once these parameters had been defined, the wind analysis could be performed.

First of all, the first order natural frequencies for vertical bending and torsion of the FRP bridge are higher than the frequencies of the steel-concrete bridge. If compared to the Von Karman wind spectrum, the steel-concrete bridge can be expected to be more sensitive to wind excitation. The wind analysis results showed that indeed the vertical displacements of

the steel-concrete bridge are larger (12 mm vs 8 mm for the FRP bridge). However, due to its low damping, the accelerations of the FRP bridge are far higher than for the steel-concrete bridge (resp. 1.30 and 0.72 m/s^2). The natural frequencies for vertical bending and torsion of the FRP bridge are quite close ($f_b = 2.88$ Hz and $f_t = 3.227$ Hz), indicating a certain sensitivity to flutter. According to annex E of EN1991-1-4, the bridge is safe nevertheless. The dynamic wind analysis results only show the effects of buffeting. Motion-induced vibrations are not included as flutter derivatives could not be computed.

Finally, to push the comparison further, long span versions of the two bridges have been designed (main span of 200 meters length). The analytic calculations used for the design of the short span bridges could not be used: the effect of cable pretension loss due to axial compression of the deck and horizontal pylon deflections was too high. The traffic loader from Sofistik also showed larger deflections than the ones computed analytically. The analytic method has a span limit for it being valid. After several iterations in Sofistik, the dimensions for the long span bridges have nevertheless been determined. The dynamic analysis results showed that the FRP bridge has become far more sensitive to turbulence-induced wind excitation (buffeting), compared to the steel-concrete bridge (vertical deflection amplitude: relative increase of 96 % for FRP, compared to a relative decrease of 6 % for steel-concrete). This issue is covered by the design of the bridges, and is not critical for the design. It might be more critical for longer spans. The main issue is the sensitivity to flutter of the FRP bridges: both the short and long span bridges have ϵ values around 1.11. The design of the decks are in both cases based on the flutter requirement from EN1991-1-4. This is not the case for the steel-concrete bridge, where the ϵ values are resp. 2.35 and 1.72.

Material costs for the FRP cable-stayed bridge are 30 % higher both for short and long span if compared to the steel-concrete composite bridge. Construction and maintenance costs have not been included. The maintenance costs being lower for the FRP bridge, these might make a difference in the total LCC costs.

As a conclusion, short span FRP cable-stayed bridges are feasible from a structural and economic point of view, with a stiffness-based design. The deck dimensions are determined based on the flutter requirement from EN1991-1-4. Long span FRP cable-stayed bridges show additional issues that need to be dealt with. If compared to the steel-concrete bridge, the production costs difference is still around 30 %. This comes mostly due to the deck: its dimensions are determined based on the flutter requirement from EN1991-1-4, and therefore because of a relatively high torsional stiffness requirement. Increasing the torsional stiffness (and vertical bending stiffness) is important to make a flutter safe design, and for even larger spans, this has to be done with complex and most likely expensive deck designs. The material and construction costs will be far higher. The low maintenance costs will most likely not make big enough of a difference to make the FRP cable-stayed bridge economically more interesting than the steel-concrete composite bridge. Based on the results in this report, the use of FRP is deemed possible for short spans, but is not recommended for long spans (200 meters for instance). To give an accurate answer, extensive LCC analyses would be required.

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Introduction

1.1. Background information

The use of FRP composites in bridge design is continuously expanding in the Netherlands considering the first FRP bridge in our country is from 1997, in Harlingen (figure 1.1). Since then, the use of this innovative material has developed a lot with the construction of several pedestrian (and bicycle) or road traffic bridges, such as the movable bridge Nelson Mandela (figure 1.2). FRP composites are lightweight and have very high strength. This strength to weight ratio is therefore very convenient for building lightweight, slender structures. Other advantages such as low maintenance costs or rapid construction can be cited. Nevertheless, there are still a lot of ongoing research topics for this material, some being:

- Vibrations of the structure and the level of comfort caused by these
- Buckling of FRP composite members, and the entire structure
- Fatigue
- Creep

In recent years, more and more companies discovered all the possibilities FRP composites have to offer, aesthetically and structurally speaking. Feasibility studies concerning for instance long span FRP bridges have been done, such as a 3700 meter long suspension bridge for a fjord in Norway [33]. One of the important topics was the issue of the cables. For such a long span, the weight of steel cables is too large, therefore aramid cables were considered. With a total cable weight reduction of 75%, dynamic behavior becomes crucial. Nevertheless according to this study, the use of aramid cables was proven feasible.

This brings us to thinking which new possibilities could be explored next: Royal HaskoningDHV is interested in the possibility of applying FRP to other bridge types. For a site in



Figure 1.1: First FRP (pedestrian) bridge in Harlingen [28]



Figure 1.2: Nelson Mandela Bridge in Alkmaar [28]

Norway, one of the potentially interesting concepts generated was an FRP cable-stayed bridge. This site is located near Stavanger, in Sirdal, where a housing project in the mountains is planned (see figure 1.3). One of the important issues is the lack of infrastructure to reach the site: a bridge followed by a tunnel are needed, indicated by the red circle in figure 1.3. Due to the geographical complexity of the site, a bridge type that allows construction from one side is preferred. The idea is to build a cable-stayed bridge from FRP open to road traffic, with a main span of 64 meters. Considering only little research has been done on that subject, a lot of knowledge can be gained to further improve and increase the use of FRP composites in bridge design.



Figure 1.3: Map of housing project in Sirdal (source: Teknaconsult)

The environment in Norway can be tough, with a lot of wind. The structure will be very lightweight compared to a more traditional material such as concrete or steel. Considering the low temperatures, wind and rain, the use of this innovative material brings many benefits. Only little maintenance is required, due to a high durability and high chemical resistance.

1.2. Problem definition

FRP composites exist for a long time already but their use in infrastructure is relatively recent, meaning a lot of research is still being done [24]. When it comes to the field of bridges, alternatives to traditional materials, such as steel, are sought due to corrosion, fatigue, maintenance problems: FRP composites is one of these alternatives. The stiffness of FRP is lower than that of steel, but it has a far higher strength. Although this presents some advantages, combined with the low self-weight of the material, the dynamic behavior of FRP bridges can become governing [37].

Royal HaskoningDHV is interested in the design of an FRP cable-stayed bridge and its dynamic response to wind loads. The location for this bridge is given, and other alternatives might be suitable too, which can be investigated through a conceptual study. To have a point of comparison for the cable-stayed bridge design, it is desirable to also design a bridge with traditional materials, in this case a steel-concrete composite bridge. Through careful analysis of the design parameters for the chosen concepts, the goal would be to obtain feasible preliminary designs. Comparing these designs would identify the issues of the FRP bridge, and show the effect of material choice. Due to the sensitivity to dynamic wind excitation, it is desirable to perform a dynamic wind analysis of the obtained designs. Ultimately, the wind analysis is to be carried out on long-span alternatives of these bridges to see what the differences are when a common span for a cable-stayed bridge is used (in this thesis, 200 meters).

Besides a preliminary design for the selected location, this study will provide more insight into the general design of FRP cable-stayed bridges, but also their dynamic behavior. The

problem statement can be reduced to:

As a consequence of a lack of experience, a study of the design parameters for an innovative FRP cable-stayed bridge is necessary to obtain a structurally safe and functional preliminary design, which has to be further assessed and improved through a dynamic analysis and a comparison to a bridge concept with traditional materials.

1.3. Research objective and questions

The research objective defines the general purpose of this thesis. The sub-questions will guide us to complete the research objective.

1.3.1. Research objective

The objective of this thesis is to contribute to the knowledge of FRP bridge design, through the design of a cable-stayed bridge and the analysis of its dynamic behavior.

The main research question that has been formulated is as follows:

How can fiber-reinforced polymer composites be used as a construction material to design a cable-stayed bridge, which is able to withstand traffic loads and dynamic wind load excitation, while still being technically and economically feasible, and how does the FRP solution compare to a more "traditional" bridge design?

1.3.2. Sub-questions

To answer the main research question, several sub-questions are formulated below, each focusing on a certain aspect. The conclusions drawn from these will form an overall answer to the problem.

Literature study

- What makes FRP composites suitable for bridge construction? What are the important characteristics of FRP bridges?
- What are the important characteristics of cable-stayed bridges?
- How can a dynamic wind load be modelled?
- What are the dynamic effects of wind actions on cable-stayed bridges?
- What are the design requirements for bridges?

Preliminary design

- How can structurally safe and functional preliminary designs of the FRP and steel-concrete composite cable-stayed bridge concepts be achieved?
- What are the main design parameters for an FRP cable-stayed bridge, and how are these linked?
- What are the differences in structural behaviour between the obtained designs of the FRP and steel-concrete composite bridges?
- Is the FRP cable-stayed bridge economically feasible and competitive compared to the steel-concrete composite cable-stayed bridge?

Advanced analysis

- What important parameters need to be defined prior to a dynamic wind analysis of a cable-stayed bridge, and how are they defined?
- What are the responses of the bridges to dynamic wind loads? How do they compare?

- How does a long span version of these cable-stayed bridges behave, more specifically the FRP one? How does it compare to the short span version?

1.4. Project scope and outline

The thesis committee is composed of:

- Prof. Dr. M. Veljkovic, from Delft University of Technology (chair)
- Dr. M. Pavlovic, from Delft University of Technology
- Dr. ir. M.A.N. Hendriks, from Delft University of Technology
- Ir. A.G. Boeters, from Royal HaskoningDHV

First of all, the requirements and boundary conditions linked to the project site are defined in chapter 2.

Part I consists of an extensive literature study about selected topics relevant to the design and analysis of an FRP cable-stayed bridge. In chapters 3 and 4, cable-stayed bridges, and FRP composites are introduced. An overview of certain important aspects of FRP such as its structural properties, or applications in bridge design, are given. Chapter 5 is relevant for part III as it addresses bridge dynamics, with a focus on wind excitation.

In part II, the objective is to obtain preliminary bridge designs. In chapters 6 and 7, the two cable-stayed bridges are given initial dimensions through analytic calculations, one of them being an FRP cable-stayed bridge and the other a steel-concrete composite cable-stayed bridge. Afterwards, these are modelled in Sofistik and the structural integrity is checked. After the necessary adjustments, preliminary designs are obtained.

Finally, in part III, an advanced analysis of the bridges is performed. In chapter 8 the wind loading and other parameters are defined before performing a dynamic wind analysis of the bridges in chapter 9. In chapter 10, the dynamic wind analysis is extended to long span alternatives of the two bridges to obtain a point of comparison.

The outline of this master thesis is illustrated in the flowchart below (see figure 1.4).

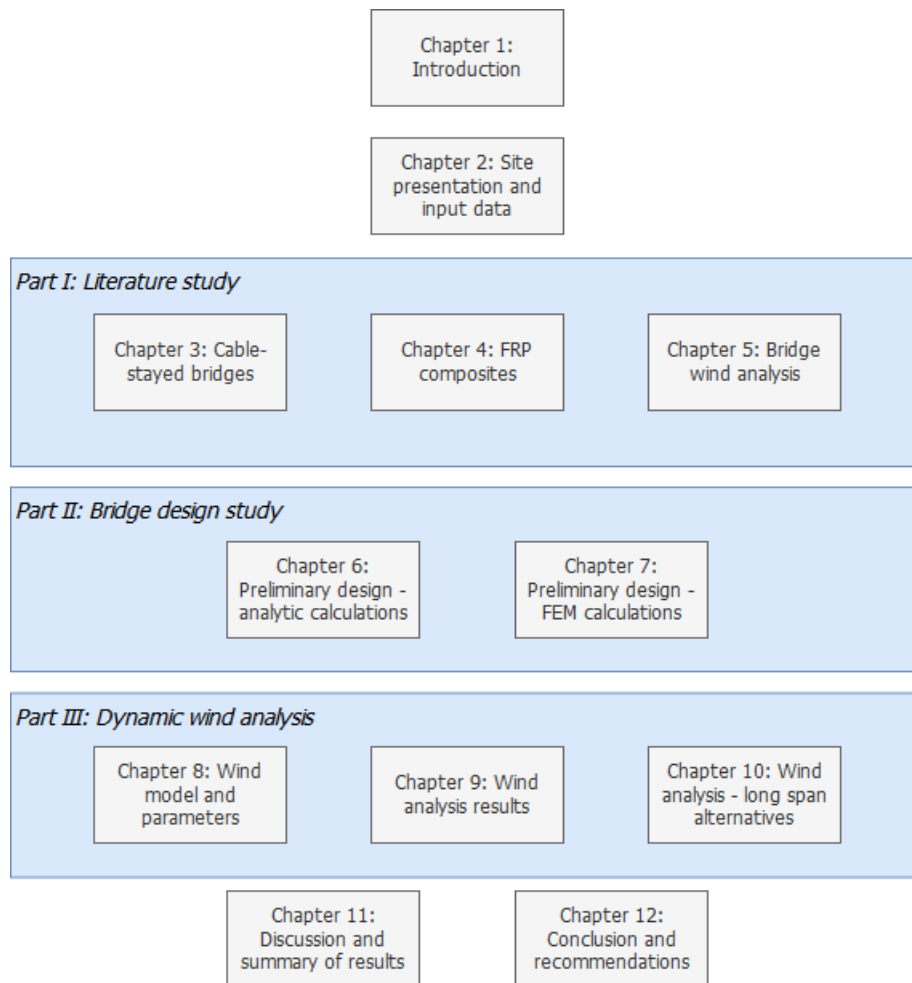


Figure 1.4: Thesis outline

1.5. Method

In this section, the general approach adopted in this thesis is detailed. This thesis has two main parts:

- A preliminary design for an FRP cable-stayed bridge, and a steel-concrete composite cable-stayed bridge. In this part the focus lies on how to obtain a cost-effective preliminary design for an FRP cable-stayed bridge. To have a point of comparison, in parallel, a steel-concrete composite cable-stayed bridge is designed.
- A dynamic wind analysis of these bridges. This part is the main focus of this thesis. From this analysis, the dynamic response of both bridges can be compared, and give more information on whether or not an FRP cable-stayed bridge is structurally and economically feasible.

All modelling will be done in SOFiSTiK, which is a FE software specialized in all kinds of structural applications, including bridges and dynamic analysis.

Bridge design

When designing a bridge, an important aspect are the costs linked to it. To be able to compare the bridges, the costs are estimated at every design step. However only material costs are considered: taxes, life cycle costs are left aside. To obtain the two preliminary designs, several steps are defined. These are as follows (see figure 1.5).

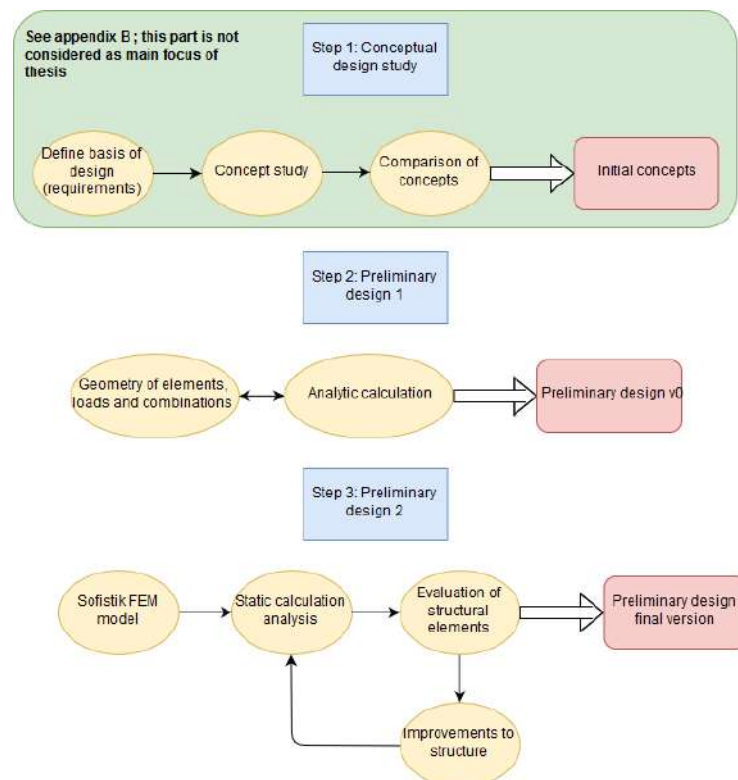


Figure 1.5: Design procedure

Step 1 consists of the conceptual design study. With the definition of requirements specific to the site, location and bridge, different concepts can be developed and compared from an economic point of view through a multi-criteria analysis. Finally, the best scoring concepts can potentially be chosen. This step has only shortly been treated and can be found in

appendix B. It mostly serves to show that other bridge types can be considered, but the goal of this thesis is not necessarily to find the most suitable bridge for this location.

Step 2 starts off with the assumption of the geometry and dimensions of the elements of the FRP and the steel-concrete composite cable-stayed bridges. With iterative, analytic calculations, the main design parameters are determined and are sized accordingly to obtain cost-effective designs. The costs only include production. This step is especially important for the FRP cable-stayed bridge, because there is no literature available on the cost-optimal dimensions for a preliminary design.

Finally in step 3, Sofistik models of the resulting bridges from step 2 are made, analyzed, evaluated and improved. These models are used as a validation for the analytic calculations. These will also be used in the dynamic wind analysis.

Dynamic wind analysis

The dynamic wind analysis procedure is given below in figure 1.6. Sofistik has several modules which together allow to perform a dynamic wind analysis:

- The CFD module Dolfyn is necessary to obtain wind coefficients of the bridge deck. It simulates the wind flow around the deck and computes the wind coefficients accordingly. This analysis is performed separately and the resulting wind coefficients are used as input parameters for the deck in the FEM model.
- The module Sofiloat allows the definition of transient wind loads in all 3 directions. Together with the modules Dyna and Dynr, which respectively perform the dynamic wind analysis and the processing of the analysis, a complete wind analysis can be performed. Only buffeting (turbulence-induced vibrations) is considered.

Once the analyses are performed, the structure can be evaluated, improved if necessary and analyzed again. After sufficient iterations, a dynamically safe preliminary bridge design can be obtained.

This iterative process will highlight which issues arise for the bridges and their dynamic response to wind loads (buffeting).

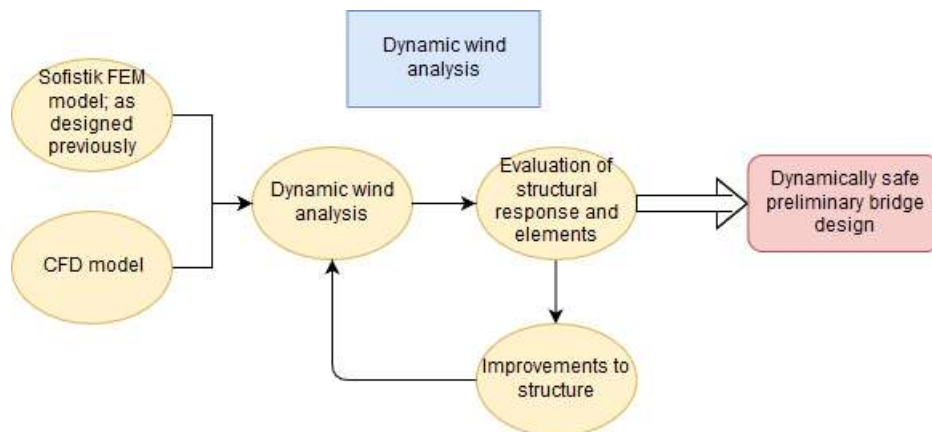


Figure 1.6: Wind analysis procedure

Requirements and boundary conditions

To be able to define the basis of design, it is important to gather all necessary information. It consists of three main parts. The first part is the input data, which gives all requirements with respect to the use, bridge and the site. This results in a set of boundary conditions. The second part is the engineer's knowledge, which will for a part be reflected by the literature study in this thesis. The third part are the design requirements, which are given in appendix A. [15]

2.1. Requirements for use

Use of the bridge

The bridge is to be built in Sirdal, in Norway, to connect a new housing area to the local road Fv468. More general information is given in section 1.1. The only existing village nearby is Tjørhom, which is very small (see figure 2.1). It is therefore unlikely that many pedestrians will walk on the bridge; mainly cars will use it.



Figure 2.1: Surroundings of new housing area in Sirdal

During the lifetime of the bridge, the traffic on the bridge will for the greatest part only consist of cars, but it's important to also consider the construction phase. Indeed the bridge is the starting point for the construction of the housing area, as well as the construction of the tunnel needed to reach the construction site. Therefore during construction of the entire project, heavy trucks and construction machines should be able to cross the bridge. This

means the bridge will require good transverse load distribution to account for these heavy loads. During normal use, the limit speed will be 50 km/h.

Durability

According to table 2.1 of EN1990 [10], the design working life of the bridge should be 100 years, during which the structure should fulfill its function from a serviceability and structural point of view. Considering an FRP cable-stayed bridge is innovative, the structure should remain easily accessible for inspection and maintenance. The durability of FRP structures (ageing effects) is included in calculations through the use of material and conversion factors as explained in appendix A.

2.2. Requirements specific to the bridge

The alignment of the bridge has been defined as illustrated in figures 2.2 and 2.3. Nevertheless, this is not fully fixed. In the side view, it would be possible to make the deck horizontal, without inclination. Among other things, this would be beneficial for the clearance height below the deck as the left abutment could be made higher. This will be further detailed in the next section. It mostly depends on the type of abutment used and how these are placed. For the sake of simplicity, the deck will be considered horizontal.

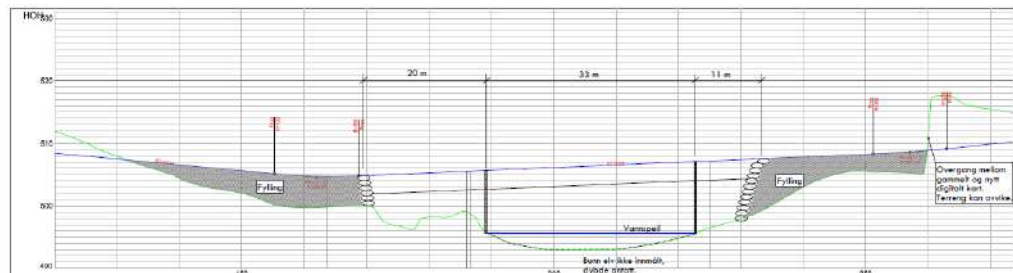


Figure 2.2: Side view of bridge alignment (source: Teknaconsult)

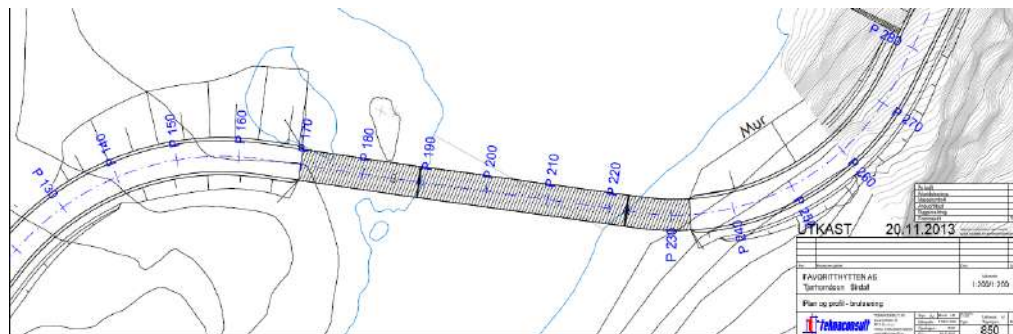


Figure 2.3: Top view of bridge alignment (source: Teknaconsult)

The bridge should preferably not need any additional piers. This is due to the high water levels from the river that runs below. This will be discussed in the next section. The bridge minimum width is defined by the presence of two traffic lanes and the space for parapets and guard rails. The minimum width for these elements is defined as follows:

- Traffic lanes: 3.5 m (x2)
- Vehicle parapet, security distances and guard rail: 1.5 m (x2)

This results in a minimum width of the bridge equal to 10 meters for the transverse section (see figure 2.4). The span is defined in figure 2.2 as 64 meters. The bridge will not be skewed,

and will not have curvatures. The top of the deck will be at a height of 12 meters above average water level, see figure 2.5. The top of the abutments is dependent on the height of the deck.

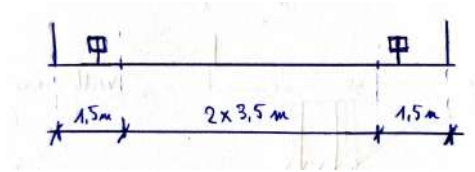


Figure 2.4: Definition of minimal bridge width

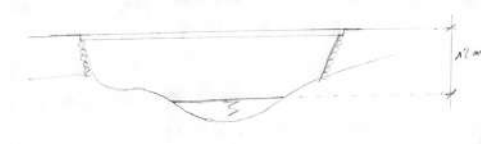


Figure 2.5: Height of top of the deck above water level

An important requirement concerning the bridge is its vibrational response to wind excitation. If the main construction material is for example concrete, the high mass and inherent damping make the construction already less prone to these issues. In the case of an FRP bridge, this is not the case and proper attention needs to be given to these issues. The comfort limit for pedestrian bridges will be used for this bridge, being 0.7 m/s^2 . A limit for the deflections is also defined, according to the Bruprosjektering [30]: $L/350$ where L is the span length. In the conceptual design phase it is important to look at the geometry of the concepts and the material, as this gives already an indication on the aerodynamic behaviour of the structure.

Furthermore, the aesthetics of the bridge are important but this will remain outside of the scope of this thesis considering the goal is to study certain structural aspects of an FRP cable-stayed bridge.

Last but not least, economic considerations are crucial when it comes to making design decisions. Costs will be looked at, and considered in the preliminary design. Only simple estimates will be made based on the amount or volume of material used. The unit values used in this thesis have been provided by Royal HaskoningDHV.

2.3. Requirements specific to the site

Concerning the site where the bridge is to be built, it is in a valley (see figure 2.6), which is decisive for the main wind direction. It is assumed in this study that the main wind direction will be transverse to the bridge.

Furthermore, it is important to consider the available space and the topology of the surroundings. As can be seen in figure 2.7, there is only little place available next to the road for building the bridge. Due to the presence of rocks and steep terrain, it will be important to think of possible erection methods of the bridge. It is furthermore important to mention that the bridge will have to be built from one side only. The use of temporary supports should be avoided because of the costs and time involved. Pontoons can be used if necessary.



Figure 2.6: Bridge construction site



Figure 2.7: Bridge construction site

As mentioned earlier, the bridge will cross a narrow water stream which during spring can have high water levels. For this reason, there should be no piers placed in the water and the bridge should span in one go for about 64 meters. Additionally the water level during that period can be such that if the bridge deck is not placed high enough, water as well as debris floating in it might hit the bridge. The construction period of the bridge should not take place during this season. Rapid construction is therefore an additional requirement (8-10 months duration).

Of course when building a structure, the geotechnical aspects also play an important role, but these will be left aside in the scope of this thesis. It will be assumed that the foundations have sufficient strength to carry the bridge.

2.4. Summary of requirements

Several requirements have been given so far. Taking them all explicitly into account would require a large amount of time for the design. To give a good overview and clearly draw the boundary conditions for the conceptual bridge design, a summarizing list of the chosen and most important requirements is given below. The other requirements have not been taken into account in the design and were therefore not explicitly given explanation.

- Use of the bridge: the main users will be car traffic (LM1 traffic load considered). The bridge should fulfill serviceability and ultimate limit states according to Eurocode (and CUR96).
- Durability: will indirectly be taken into account. One of the reasons for applying FRP is the fact it requires little maintenance if properly designed. The designs will be kept simple to avoid having complex details. This reduces the need for maintenance and potential weak spots.
- Geometry of the bridge: Span of 64 meters, minimum width of 10 meters, and height of top of the deck with respect to water level of 12 meters. The construction height of the deck should be kept to a minimum, and the aerodynamic behaviour should be kept in mind when defining the shape of the deck. There should be no intermediate piers. The clearance height is not defined precisely but should be kept to a minimum.
- The deflection limit is $L/350$ and the acceleration limit is 0.7 m/s^2 .
- Construction of the bridge: Construction from one side is considered as best option. Rapid construction required, as it should be done before the melting season: within 8-10 months.
- Cost-effectiveness: Will be considered with simple estimates of the bridge construction costs in the preliminary design, with the help of unit prices that consider production. Helps comparison of the two bridges.

These requirements are the basis for the conceptual design where different bridge types were considered for this specific location. The conceptual design can be found in appendix B.

I

Literature study

Cable-stayed bridges and other bridge types

In this chapter, the main characteristics of cable-stayed bridges will be identified. This will highlight which important design aspects need to be considered for the FRP bridge design.

3.1. Structural system and components

3.1.1. Structural behavior and cable systems

Cable-stayed bridges are used for spans from 100m up to 1000m, which can be considered as the economic span range. As a comparison, from figure 3.2, it can be seen what the economic span ranges are for other bridge types [32].

Cable-stayed bridges consist of 4 main structural components, being the deck, the cable system, the pylons and the anchor blocks. The loads are transferred from the deck to the stays, through compression and bending. The stays, in tension, then transfer the loads to the pylons. Finally through compression the loads are transferred to the foundations [3](figure 3.1).

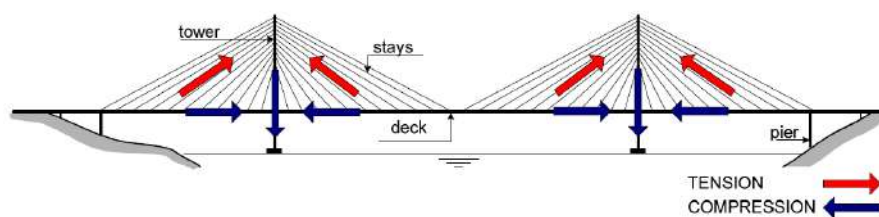


Figure 3.1: Forces in cable-stayed bridge [3]

First of all it is important to mention the different types of cable-stayed bridges, which are defined by the cable system. The stiffened deck is supported by stays, which are anchored in the pylon. The stays can be arranged in 3 different ways (figure 3.4):

- The fan system has all stays anchored at the top of the pylon, radiating to the deck. The detailing of the anchorages can be complicated due to the amount of stays, meaning a true fan system is often only applied for a limited amount of stays. It can be added that nowadays easy replacement of stays is a requirement, which can be difficult for fan systems. Nevertheless this system is structurally speaking the most beneficial as the vertical forces from the stays are transferred with a minimum amount of steel to

the pylon. It also reduces the compression in the deck, and the bending moment in the pylon [2].

- The harp system has a parallel distribution of the stays over the height of the pylon. This system is aesthetically speaking more interesting, due to the reduction of visual intersections between stays. Nevertheless large bending moments are produced in the pylons due to the anchorages being placed over the height of the pylon, and the compression forces in the deck are larger. The cables are also less effective due to a more horizontal angle. A greater height of the pylons is often required too [36].
- The semi-fan system comes close to the fan system. The cable anchorages at the top of the pylon are more spread out, allowing for easier detailing. It has most of the advantages of the fan system, with a more aesthetically appealing design.

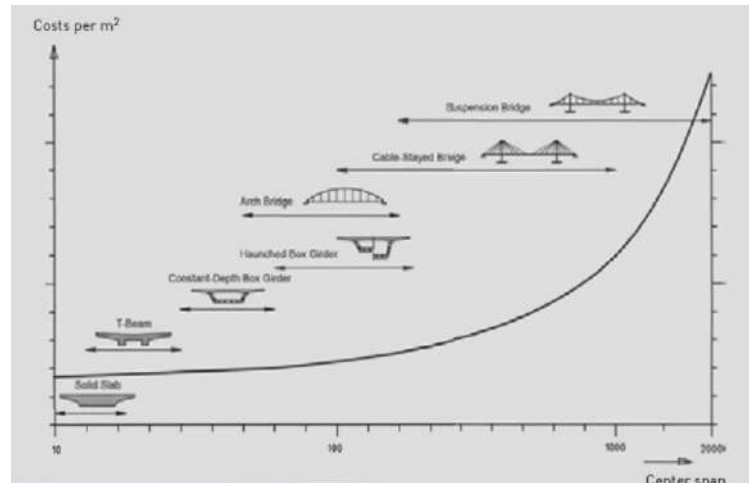


Figure 3.2: Comparison of bridge types based on economic main span range [32]

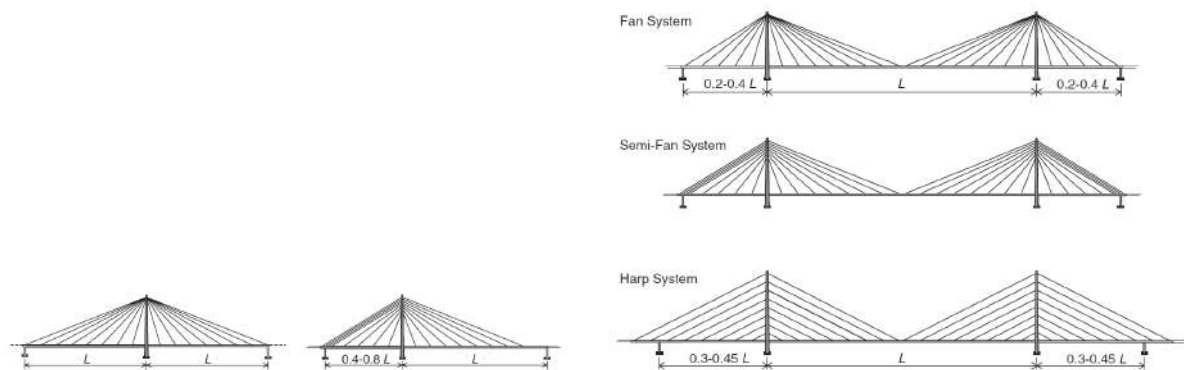


Figure 3.3: Two-span cable-stayed bridge arrangements [13]

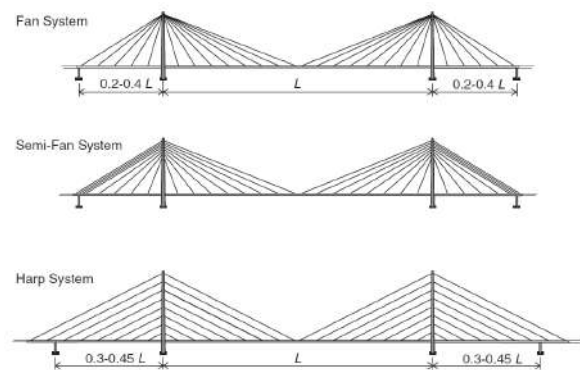


Figure 3.4: Cable-stayed bridge systems [13]

Most often three-span bridges are applied, with a rather large main span and shorter side spans. However, two-span cable bridges are also applied on a regular basis, with either a symmetrical cable arrangement, or an asymmetrical one with a larger main span (figure 3.3). This arrangement is more suitable for short spans when there are no limitations with regard to the pylon height or cable lengths.

So far the longitudinal cable arrangement was discussed. In transverse direction, 3 basic cable arrangements are often applied (figure 3.5):

- The single plane arrangement where all stays are aligned on the center of the cross-section. It has mostly aesthetic advantages. Structurally speaking it offers no rotational

restraint, meaning the deck will need sufficient torsional stiffness to make up for it.

- The double plane arrangement is rather popular, as it creates a rather high torsional stiffness of the bridge. It's often applied for long spans.
- The A-frame arrangement is often applied for very long spans (over 600m). Similar to the double plane arrangement, it creates a lot of torsional restraint for the deck, as well as considerable lateral restraint, due to the inclined stays and the frame action from the pylons [2].

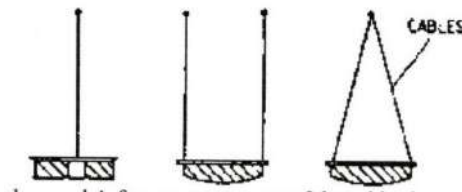


Figure 3.5: Transverse cable arrangements

3.1.2. Cables and anchoring

The stay cables can be considered to be the key component of cable-stayed bridges as they define the behaviour of the bridge at all stages. Three main (steel) cable systems are applied in modern design, which are the locked coil ropes, the parallel wire cables and the parallel strand cables, also illustrated in figure 3.6 [13].

- The locked coil ropes have in their core round wires, with a certain helix, and on the outer part interlocked wires with a Z-shape. They are entirely prefabricated, have good corrosion resistance and easy maintenance due to the compact configuration of the wires. Some disadvantages are the reduced stiffness, the creep sensitivity, reduced tensile and fatigue strength.
- Parallel wire cables are a bundle of straight 7 mm diameter wires. They can be post-tensioned and are entirely prefabricated. They have high stiffness, no creep, high tensile and fatigue strength [13]. Parallel wire cables were developed after locked coil ropes to solve the issues these had.
- Parallel strand cables are based on the same principle as parallel wire cables, except the individual wires are replaced by 7-wire strands. This type of cable has the advantage of being cost-effective, can be assembled on site. Individual strands can also be replaced.

Another modern alternative is the use of carbon cables. Due to their very high longitudinal strength and low weight, carbon cables have many advantages but remain very expensive and mostly of interest for long spans where cable-weight reduction is important.

Characteristics	Modern locked coil rope	Parallel wire cable	Parallel strand cable
			
$E \cdot 10^{-6}$ [N/mm ²]	0.170	0.205	0.195
ϵ_u [N/mm ²]	1470	1670	1870
$\Delta\sigma$ [N/mm ²]	150	200	200

Figure 3.6: Comparison of stay cables [32]

The cables have to be anchored at the tower and at the beam to transfer forces into the structure. Different anchoring systems exist, depending on the type of cable, the cable ar-

rangement, the material or the specific needs for the bridge. A few examples are given in figures 3.7, 3.8 and 3.9.

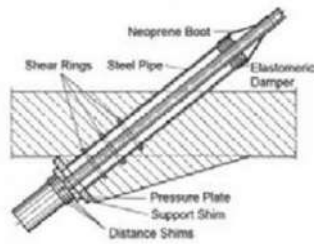


Figure 3.7: Cable anchorage in concrete deck, with shims [32]

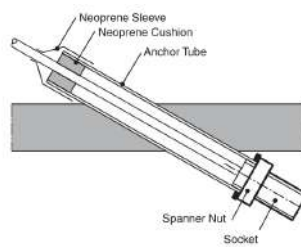


Figure 3.8: Anchorage of cable with spanner nut [13]



Figure 3.9: Open sockets, with turnbuckle [32]

In figure 3.10, the possible anchoring systems are also well illustrated and give a good overview of possibilities for force transfer.

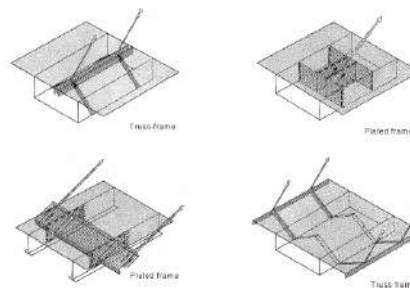


Figure 3.10: Examples of cable-deck connections [25]

Last but not least, a cable-stayed bridge can be earth-anchored or self-anchored. The former one means the backing stay cable is directly anchored to the foundation, which means that there is no horizontal force transmission to the deck. The latter one means that the backing stay cable is anchored to the deck, thus transmitting a large horizontal force. Which one is applied often depends on the soil conditions.

3.1.3. Deck systems

Action of the deck

In cable-stayed bridges, the compression in the deck is quite high, therefore a high axial stiffness is required, and buckling can become critical. Flexural stiffness in vertical direction is important in the transfer of vertical loads. It can be divided into 3 actions [13], where action 1 is the ability to carry the load locally between cable anchor points. The second action is the ability to carry for a part the load between bridge supports, not just anchor points, thus referring to the overall stiffness behaviour. Finally, the deck should be able to distribute concentrated forces, which is important for multi-cable systems (figure 3.11).

Concerning the flexural stiffness in transverse direction, self-anchored systems transfer the entire load through lateral bending, whereas earth-anchored systems have an interaction between the deck and the cable system for the load transmission [13]. Generally it is advantageous to make the deck continuous, resulting in a moment diagram as in (c) from figure 3.12. As a point of comparison, the other two illustrated systems for the lateral direction have configurations which are less advantageous.

Finally the deck of a cable-stayed bridge requires a good torsional support for eccentric loads, as well as wind loads. If the deck is supported by only one cable plane, it will have to have

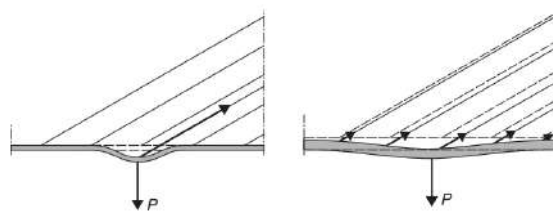


Figure 3.11: Distribution of concentrated force in cable-stayed bridge deck [13]

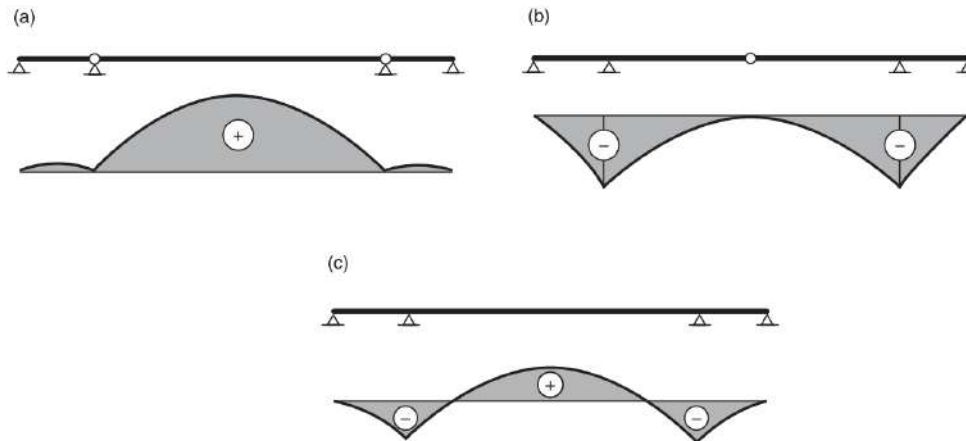


Figure 3.12: Moments in deck in lateral direction [13]

a high torsional stiffness (for example a box girder). With two cable planes, the deck can have a much lower torsional stiffness, though sometimes it can be beneficial to still have a cross-section with high torsional stiffness for a more even force distribution [13].

Supports and expansion joints

The support conditions of the deck, as well as the position of expansion joints has an influence on the force distribution in the structure. The structure can be self-anchored thus having a high compressive normal force. If the system is earth-anchored, tensile normal forces will occur in the deck. As an example three deck configurations are given in figure 3.13, with the corresponding normal force distribution. The presence of expansion joints at the tower, combined with the earth anchor supports, means there are only tensile normal forces in the deck (system A). System B is a self-anchored system which clearly needs most material for the deck. Nevertheless it must be remembered that systems A and C require a heavy abutment [25]. These are only a few examples and other support conditions are possible. This depends on the design of the expansion joint, the bending moments in the pylon, the stiffness of the deck supports, construction limitations, bending moments caused by horizontal reaction force.

Cross-section of the deck

As was just discussed, 2 important design criteria for the choice of the cross-section of the deck are the buckling resistance, and torsional stiffness. Other criteria also have their importance such as wind area and aerodynamic shape. Several systems exist and have been applied: precast or cast-in-situ concrete elements, steel box girders, steel plate girders.

Steel decks are often applied for their lightweight. This advantage is beneficial when large spans are required, or poor soil conditions are present, or also for rapid construction. Concrete decks are often applied for shorter spans. Up to 300 meters, they have proven to be

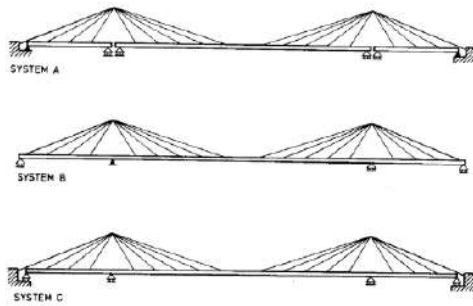


Figure 3.13: Examples of deck configuration [25]

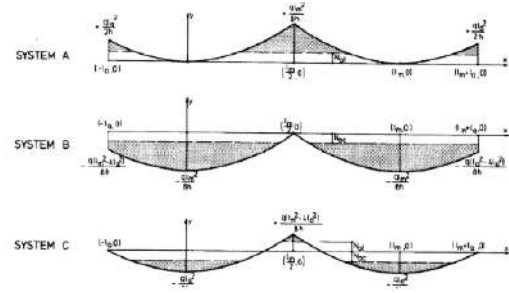


Figure 3.14: Normal force distribution for the different configurations [25]

economically more viable [25]. For greater spans, the high self-weight of concrete is such that many cables are required, as well as excessive dimensions of the deck. However their large self-weight makes it suitable for railway bridges due to their large self-weight and high damping, which is beneficial from a dynamic point of view. For intermediate spans, composite decks are often applied: the interaction of steel and concrete is beneficial as the concrete takes most of the compressive forces and the steel the bending moments. A few examples of cross sections will be shown in the figures below.

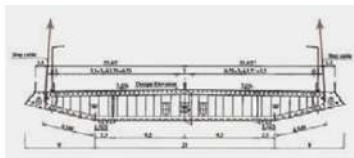


Figure 3.15: Steel box girder [32]

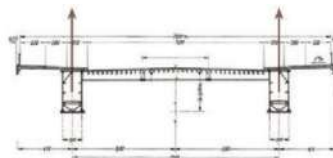


Figure 3.16: Steel open cross section [32]

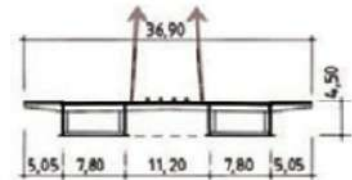


Figure 3.17: Separate steel box girders [32]



Figure 3.18: Concrete box girder [32]

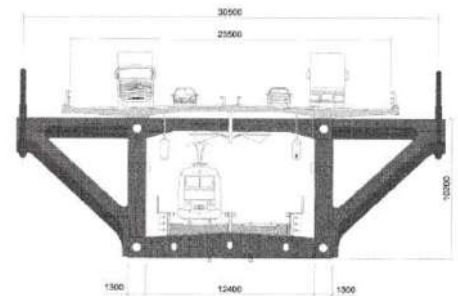


Figure 3.19: Truss beam cross section of the Öresund bridge [32]

3.1.4. Pylon

As has already been briefly discussed earlier, the configuration of the pylons greatly depends on the cable arrangement (one plane, double plane..), as well as the aesthetic choices of the architect. See figure 3.20.

The material for the pylon depends on several parameters, but most of the time due to the high compression forces, concrete is chosen. Steel pylons are mostly applied when high pylons are required, or when the soil conditions are very bad, thus requiring a light-weight structure.

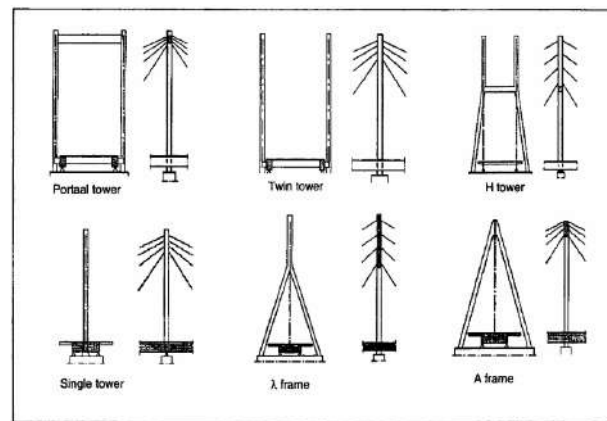


Figure 3.20: Pylon configurations [25]

3.2. Key design parameters of a cable-stayed bridge

In this section, some key design aspects of a cable-stayed bridge will be discussed.

The three main parts that make up the structural system of a cable-stayed bridge are the cables, the pylon and the deck. The overall stiffness of the system is for the greatest part dependent on the stiffness of the stay cables. Nevertheless, the stiffness of the cables is reduced by cable sag:

- In figure 3.21, the formula from Ernst [6] gives this stiffness reduction. The steel stress is very important as it occurs as the third power. This means that a high cable stress is required to obtain a high stiffness.
- Cable sag is the geometrical non-linearity cable-stayed bridges are most sensitive for and should in many cases be taken into account in the design. This has been proven by [12]. According to the ESDEP course [7], the cable-sag effect does not have to be considered when the horizontal projection of the stay cables is inferior to 150 meters. Other non-linearities are the beam-column effect and large displacement effects.

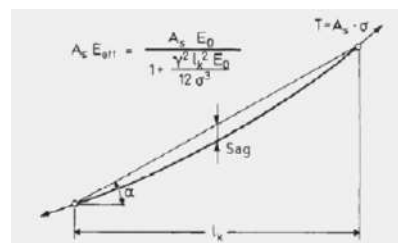


Figure 3.21: Effective stiffness of stay cables [32]

When designing the deck for global dimensions, the cables supporting it can be considered to be vertical spring elements. An approximate calculation of the bending moments in the deck can be obtained by modelling this deck as a beam on elastic foundation. The bedding factor c can be determined from the vertical displacement from a unit load divided by the cable distance [32]. To optimize the bending moments in the deck, the cables need to be prestressed individually. The level of prestressing is determined by the situation of zero displacement due to dead load. This method has been developed by [35]. See as example the comparison with and without prestressing of the cables in figures 3.22 and 3.23. Several other cable optimization methods have been developed, which are discussed in [14]. Analytically the cables can be optimized using the theory of Troitsky.

In order to have an idea of the overall geometry of the bridge, a few design rules can be used [25]. For asymmetric bridges, the ratio pylon height above deck and length of main span lies

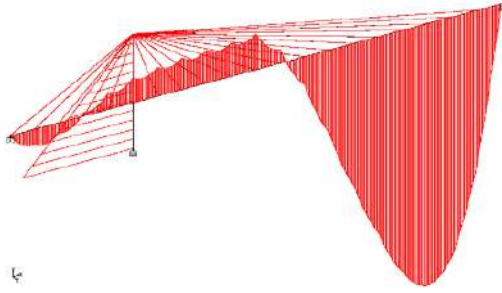


Figure 3.22: Bending moments in deck due to dead load, without prestressing [25]

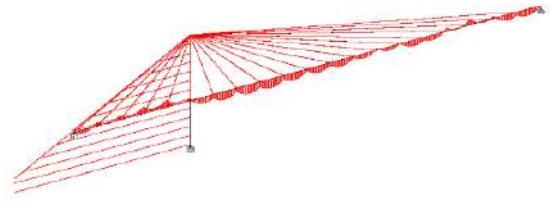


Figure 3.23: Optimized moments caused by self-weight, by prestressing of the cables [25]

in between 0.35 and 0.45, and the minimum angle of the connection between cable and deck is 25 degrees. The average distance between the cable anchors for steel bridges is 16 meters, whereas for concrete bridges it is 8 meters. Concerning the ratio between the length of the side and main span, it should be around 0.3 for steel bridges, and between 0.3 and 0.45 for concrete bridges. The optimal ratio between pylon height and main span length is given in figure 3.24, from an economic point of view.

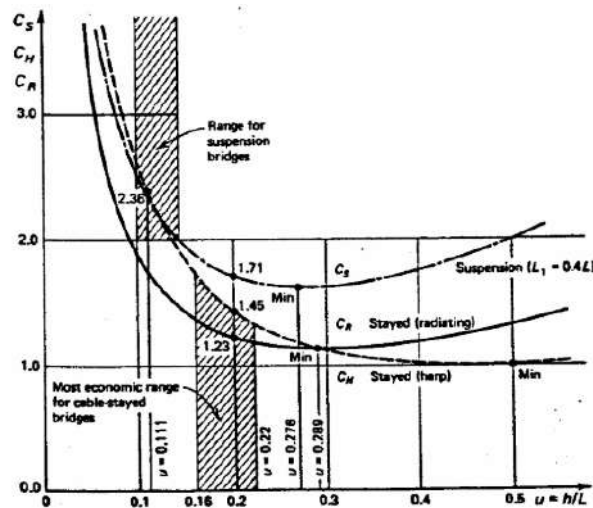


Figure 3.24: Optimal ratio between pylon height and main span length, from an economic point of view [25]

Finally, according to the N400 Bruprosjektering [30] which gives guidance concerning the design of bridges in Norway, cable-stayed bridges may be analyzed according to the first order elasticity theory if a cable stiffness reduction is applied due to cable sag. However 2nd order effects need to be taken into account for the analysis of the global stability of the pylon and girder.

FRP composites

4.1. Structural properties of FRP

Fiber reinforced polymer composite material has two main components being the resin and the fibers. Its properties depend on those of its constituents, but also on the geometry, the ratio of fibers to resin, the adhesion between the two constituents or the orientation of the fibers [21].

Fibers have a very high ultimate tensile strength in longitudinal direction, thus providing the necessary stiffness and strength to the final product. In transverse direction, the strength of the UD laminates is far lower. This means the strength and stiffness of the FRP material is dependent on how the fibers are oriented. The most commonly used fibers are glass and carbon fibers, the latter being the most expensive. This is due to the better properties of carbon fibers, having higher ultimate tensile strength and stiffness, as well as better fatigue resistance. The resin acts as a matrix between the fibers, transferring the loads and determining the geometry of the structure. It also gives a certain resistance against compression to the FRP and prevents buckling of the fibres. Several types of resin are applied, such as polyester, vinylester or epoxy. The latter is the most expensive, and shrinks less. In figure 4.1 an example of the stress-strain curve of both materials and the final FRP product is given.

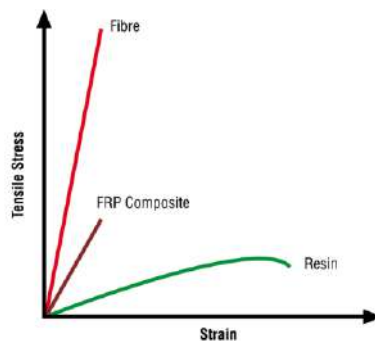


Figure 4.1: Schematic representation of the stress-strain curve of FRP and its constituents [22]

A few properties of fibers and resins are given in the tables 4.1 and 4.2.

FRP elements are usually made up of laminates, which themselves consist of plies. The plies each have fibers oriented in a certain direction (figure 4.2). Often laminates are designed with a symmetrical structure, meaning that on each side of the middle of the laminate the same plies are placed at the same distance. This prevents undesirable mechanical coupling. Uni-Directional (UD) plies have all their fibers oriented in the same direction and they are

Property	Glass	Carbon	Aramid
Stiffness [GPa]	70-80	160-440	60-180
Tensile strength [MPa]	2400	2000-5300	3100-3600
Strain at failure [%]	2.6	1-1.5	1.7
Density [kg/m ³]	2500-2600	1800-2000	1540

Table 4.1: Properties of most commonly used fibers [21]

Property	Polyester	Vinylester	Epoxy
Stiffness [GPa]	2.4-4.6	3-3.5	3.5
Tensile strength [MPa]	40-85	50-80	60-80
Strain at failure [%]	1.2-4.5	5	3-5
Density [kg/m ³]	1150-1250	1150-1250	1150-1200
Drying shrinkage [%]	6-8	5-7	<2

Table 4.2: Properties of most commonly used resins [21]

almost always applied in infrastructure. The rule of mixture can be applied to plies in order to have a rather accurate estimate of the stiffness of this ply in fiber direction (see equation 4.1) [19].

$$E = \beta * E_f * V_f + E_m * V_m \quad (4.1)$$

where β is the fibre efficiency factor (=1.0 for UD reinforcement), E_f and V_f respectively the E modulus and volume fraction of fibre, E_m and V_m respectively the E modulus and matrix volume fraction.

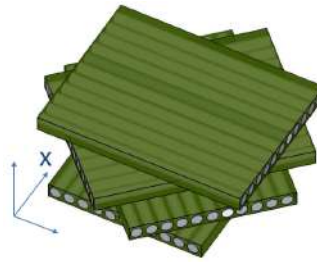


Figure 4.2: Example of laminate structure [21]

Depending on the amount of fibres, their orientation and properties, the properties of the laminate can be obtained. This is done with the classical laminate theory. It gives insight into the strength and stiffness of the laminate, but also the coupling effects between the plies due to the fiber orientation. Nevertheless, performing these calculations by hand requires a lot of time and effort, and most often software (for example eLamX2), based on this theory, is used to find the laminate properties. The theory also has its limitations [21]:

- The ply properties are generalized to the entire ply and it is not possible to determine what happens on a microscopic level in the ply.
- If delamination occurs, the theory is not valid anymore.
- It is a linear elastic analysis, thus non-linear behavior of the plies is not taken into account.
- The laminate has a constant thickness.
- If holes or other discontinuities are present, the theory is not valid.
- The plies are considered thin compared to the laminate.

In the recently published CUR-Aanbeveling 96 [26], the laminate properties are also derived from the classical laminate theory. A simplified strain criteria is used for determining the

tensile, compressive as well as shear strength in ULS in the plane of the laminate. An axial strain limit of 1.2% is used and for shear a limit of 1.6%. An example for two GFRP laminates is given in table 4.3.

Strength properties	Quasi-isotropic laminate 0°(25%) / 90°(25%) / +45°(25%) / -45°(25%)	Anisotropic laminate 0°(55%) / 90°(15%) / +45°(15%) / -45°(15%)
$f_{xt,Rk}$ [N/mm ²]	223	310
$f_{xc,Rk}$ [N/mm ²]	223	310
$f_{yt,Rk}$ [N/mm ²]	223	191
$f_{yc,Rk}$ [N/mm ²]	223	191
$\tau_{xy,Rk}$ [N/mm ²]	112	90

Table 4.3: Strength properties for two GFRP laminates ($V_f=50\%$), under one directional stress, or pure shear stress [26]

In FRP structural elements, core materials such as honeycomb structures or foam are often applied. Most of them are applied to either stiffen the panel, increase the shear rigidity, or give the panel a desired shape. Typical foams are PUR, PVC or PIM. Their properties are given in CUR96 [26].

4.2. Failure mechanisms

According to [21], FRP composites have the following failure mechanisms:

- Splitting. Cracks can occur in the material, parallel to the fibres, and through the entire thickness of one or several plies. It can occur due to tensile action in transverse direction.
- Delamination is a crack formation between two plies. It is one of the most seen failure mechanisms. It occurs when the shear force between plies is too high.
- Buckling (Euler buckling) can occur for slender elements, under compression loads, same as for any other material. Nevertheless with FRP there can also be buckling failure at the material level: fibres and plies. If one of these elements buckles, it often results in delamination.
- Fatigue failure can occur for FRP. The S-N curve is flatter for FRP than for steel. This means that at a similar stress level, the number of cycles is greater. The fatigue life of FRP can therefore be very long if the fibres are properly oriented with respect to the loads and critical parts (connections) are studied in detail.
- Impact damage. As was mentioned earlier, FRP has a linear elastic behaviour, without any ductility. After an impact, the material may be damaged but this is not necessarily visible.
- Creep. Creep is often not a large issue for FRP but can still occur.

The CUR-Aanbeveling 96 [26] indicates that an FRP structure should be checked on strength, stability, deformations, vibrations and fatigue. In ULS, the stresses in the structure need to be checked to assess the strength. This can be done on either ply-, laminate- or cross-section level. For plates and shells, only on ply- and laminate-levels can stresses be checked due to varying stresses in the cross-section. Furthermore for the stability in ULS, buckling should be checked, and finally fatigue too. In SLS, the deformations and vibrations of the structure should be checked. The stiffness of the structure is critical for these checks.

4.3. FRP applications to bridge design

FRP bridges exist for several years now. This section won't give an overview of all FRP applications in this domain, but will focus on relevant applications for this thesis, being cable-stayed bridges, and corresponding components.

4.3.1. FRP cable-stayed bridge examples

A few FRP cable-stayed bridges and bridge concepts have already been built or analyzed. The first FRP cable-stayed bridge to be built is the Aberfeldy footbridge in Scotland (see figure 4.3) [27], in 1992. It is a three-span cable-stayed bridge, with a fan cable-arrangement. The deck, pylon and beams are all made with ACCS components, which are FRP building panels connected together (see figure 4.5). The deck arrangement is given in figure 4.4. The primary beams have been placed at the anchor points in the deck. A few issues were found concerning the bridge.

- The bridge has a high strength-to-weight ratio. This means that stiffness will dominate the design. The deflections were found to be slightly too high due to a rather low stiffness of the cables.
- The frequency limit of 5 Hz due to pedestrian excitation was not reached: the natural frequency was equal to 1.5 Hz even with additional ballast load. A frictional joint was installed between the handrail and upright to improve the dynamic behaviour through damping, but due to lack of rigidity this wasn't effective. Other studies were done [4], concluding the bridge behaviour was acceptable though lively. The damping ratio's remained low even with the improvements, around 0.75%.
- Surface weathering due to improper detailing of the parapets.



Figure 4.3: Aberfeldy footbridge [27]

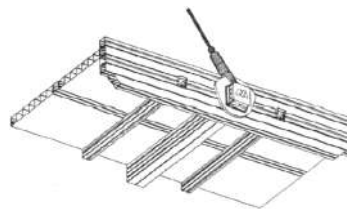


Figure 4.4: Deck arrangement of Aberfeldy bridge [27]

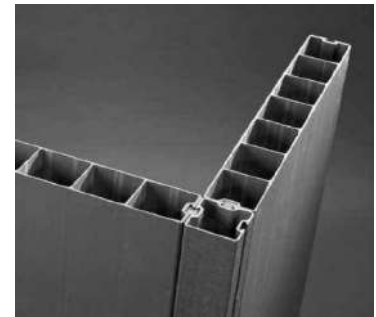


Figure 4.5: ACCS system [27]

In the article from M. Khalifa [18], a design for an all-FRP pedestrian cable-stayed bridge is analyzed. The bridge is illustrated in figures 4.6 and 4.7. Two deck designs were studied (see figures 4.8 and 4.9). For the pultruded GFRP box girder, side parts can be added for the aerodynamic characteristics. Both configurations are viable. The deck ends were taken as hinged to the abutments to avoid too high compression forces in the decks. For the towers, a diamond shape was chosen for aesthetics, high torsional stiffness, lateral stability and ease of handling cable anchorages. The 2-plane fan cable arrangement was chosen because of the easier cable anchorages and torsional stability it offers. The bridge was analyzed static linearly, static non-linearly, and dynamically. The overall factor of safety of 5 was adopted, which of course is rather high and indicates a design far from optimal. Nevertheless the authors concluded the bridge was feasible, though careful attention needs to be given to the dynamics of the bridge. They studied four forms of wind-induced vibrations, namely vortex shedding, galloping, flutter and buffeting.

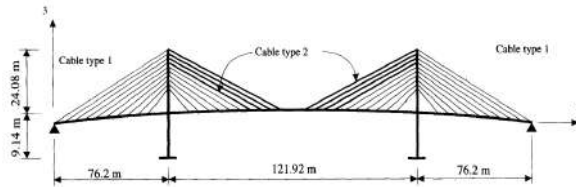


Figure 4.6: FRP cable stayed bridge design [18]

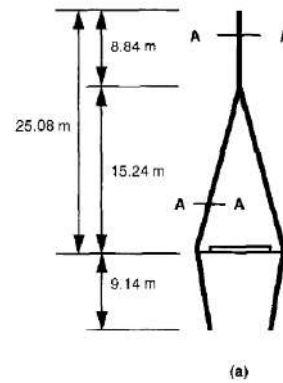


Figure 4.7: Filament wound GFRP tower [18]

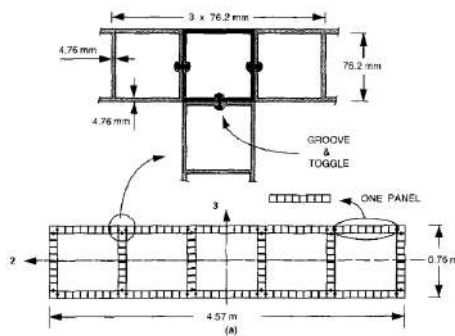


Figure 4.8: Pultruded GFRP box girder alternative [18]

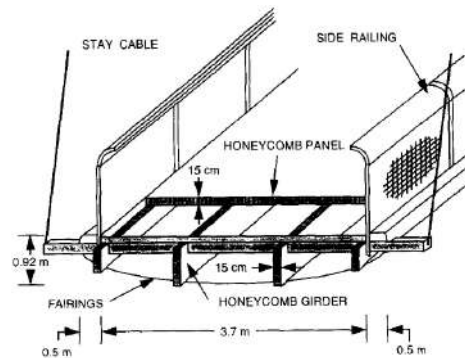


Figure 4.9: Honeycomb deck alternative [18]

4.3.2. FRP bridge deck systems

Several FRP bridge deck systems exist, based on the type of bridge and its intended use. Some of the most typical systems will be explained in this section.

Two main technologies are being used to produce FRP elements. The first one is pultrusion, which allows the production of profiles and sections with constant thickness. The other technique is VARTM (or vacuum infusion) where the plate thickness is variable and complex freeform shapes can be obtained. Several companies produce their own FRP structural elements (beams, decks): Fibercore Europe (Dutch company), Fiberline (Danish company) or Strongwell (American company).

Many FRP deck slabs are produced by pultrusion and supported by steel or FRP girders. The deck slab can be a sandwich panel, or a pultruded panel such as the fiberline plank (figure 4.10) or the fiberline ASSET (figure 4.11). The former is suitable for pedestrian and cycle bridges, whereas the latter can be used for highway bridges. One issue, especially when the girders are from steel, is the connection between the slab and the girders: often a combination of adhesives and mechanical fasteners is the best option [19]. One of the largest beams available on the market, designed for heavy-traffic use, would be the Strongwell's EXTREN DWB (see figure 4.12). Another Strongwell system is for example the COMPOSOLITE system (or ACCS system) seen previously in figures 4.5 and 4.8, which allows for easy on-site assembly of pultruded panels to create sections.

Other bridge decks are entirely made in one piece with vacuum infusion, allowing for freedom of shape and efficient use of material. An example is the Dragonfly bridge in Harderwijk (see figure 4.13). It is a pedestrian and bicycle bridge, also capable of carrying heavy emergency

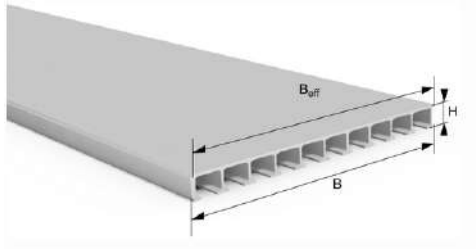


Figure 4.10: Fiberline plank [11]

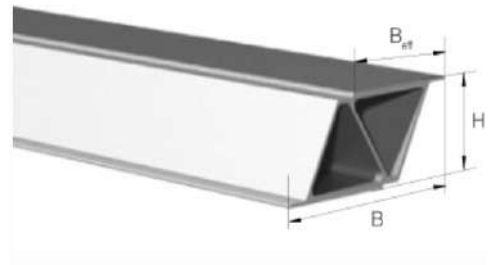


Figure 4.11: Fiberline ASSET bridge deck system [11]



Figure 4.12: Strongwell's EXTREN DWB [31]

vehicles, with a span of 22 meters. The architectural and preliminary design was done by RHDHV. Often these types of decks can be modelled as load-carrying shells. Depending on the span, loads and shape of the deck, transverse and longitudinal stiffeners might be added in the deck. Another VARTM FRP bridge is the Apatech footbridge in Moscow (see figure 4.14, with a span of 22 meters. The bridge was produced with one mould, which can be reused to produce other footbridges of lengths between 15 and 30 meters. In figure 4.15, the longitudinal and cross-section of the bridge can be seen.



Figure 4.13: Dragonfly bridge in Harderwijk [17]



Figure 4.14: Apatech footbridge in Moscow [1]

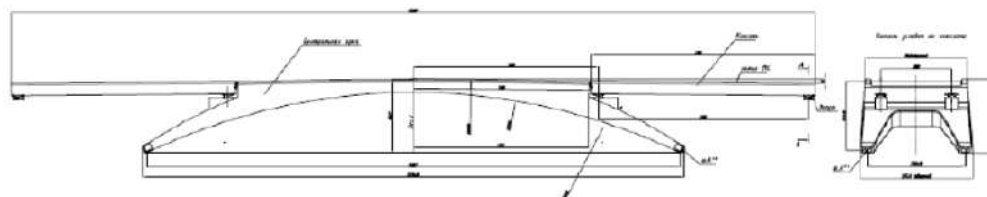


Figure 4.15: Apatech footbridge longitudinal- and cross-section [1]

5

Bridge wind dynamics

Many early cable-stayed bridges were destroyed by wind effects. This was due to a lack of understanding of the involved dynamic phenomena. In this chapter, some general information about wind engineering will first be given, before discussing wind-induced vibrations which can cause failure and discomfort.

5.1. Wind engineering

This section will give an introduction to wind engineering and how to model the wind loads. The behaviour of wind around a structure will also be highlighted.

When assessing the safety of a structure against wind excitation, regulations require that during the lifetime of the bridge the structure retains its structural integrity, as well as functionality. To check this during the design phase, only a time frame of 10 minutes is actually used in the Eurocode. The structure should be exposed to all resonances during this time period. During these 10 minutes, the wind speeds have two main components: the mean wind speed (time constant, spatially variable), and the fluctuating, turbulent wind speed (time and spatially variable).

The mean wind velocity profile has to be modelled for engineering purposes. It is defined by a logarithmic profile as illustrated in figure 5.1. There is friction occurring between the earth's surface and the air flow, due to the terrain roughness. This determines the wind speed in the boundary layer, and causes lower wind speeds at low altitude. Two main methods for determining the wind profile exist. The Logarithmic law is used by the Eurocode, and is of biggest interest here. It defines the wind profile with the following expressions 5.1 and 5.2. The roughness length indicates the height above ground at which the wind speed is zero. In the EN1991-1-4 [9], the logarithmic profile is used for the mean wind velocity up to 200 meters above ground. It also defines 4 terrain categories, with corresponding parameters. [5]

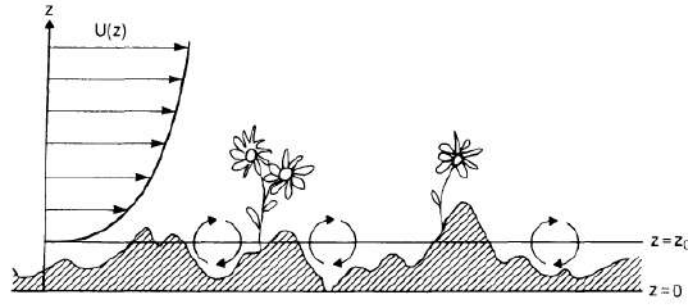
$$U(z) = u_* * \frac{1}{\kappa} * \ln \frac{z}{z_0} \quad (5.1)$$

where: κ is the von Kármán's constant (approximately 0.4), z_0 the roughness length and u_* the friction velocity:

$$u_* = \sqrt{\frac{\tau_0}{\rho}} \quad (5.2)$$

where: τ_0 is the shear stress at ground surface and ρ the air density.

When designing a structure, an extreme value should be used for the mean wind speed. The extreme average wind speed can be obtained through statistical analysis of recorded wind

Figure 5.1: Wind profile, with illustration of roughness length z_0 [5]

speed data. In the EN1991-1-4 [9], a period of 10 minutes is used for determining the extreme mean wind speed during a year.

The fluctuating wind speed (or turbulence) is dependent on time and space (see figure 5.2), and can be considered to be a stochastic process. It is influenced by the surface roughness, as it increases the rougher the terrain gets. It has three components: u , v and w , respectively in longitudinal, transverse, and vertical direction. Turbulence intensity is defined as the ratio between the standard deviation and the mean wind speed (equation 5.3).

$$I_u(z) = \frac{\sigma_u(z)}{U(z)} \quad (5.3)$$

where: $\sigma_u(z)$ is the standard deviation of the turbulence component u , $U(z)$ the mean wind velocity.

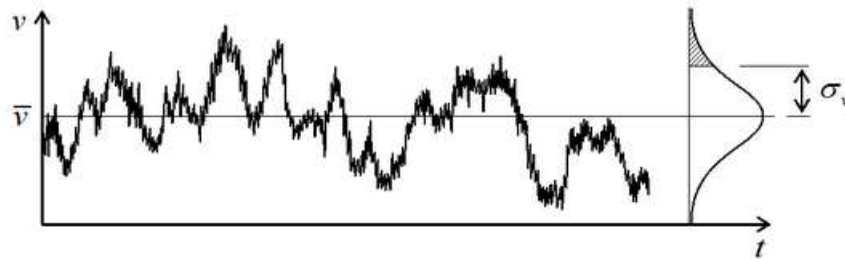


Figure 5.2: Example of turbulence [34]

The fluctuating wind velocity is described by using a variance spectrum of the wind velocity. Considering the spectra varies based on the time and place, we can not speak of a single spectrum. A reduced spectrum is defined in equation 5.4. Some commonly used reduced spectra are the Davenport, Harris or Simiu spectra, which are respectively defined in equations 5.5, 5.6 and 5.7 [34], and illustrated in figure 5.3.

$$F_D(f) = \frac{f * S_{vv}(f)}{\sigma_v^2} \quad (5.4)$$

where: σ_v is the standard deviation of the wind speed, S_{vv} is the variance spectrum, f the frequency.

$$F_D = \frac{2}{3} \frac{x^2}{(1 + x^2)^{4/3}} \quad (5.5)$$

$$F_D = \frac{3}{5} \frac{x}{(2 + x^2)^{5/6}} \quad (5.6)$$

$$F_D = \frac{2}{3} \frac{x}{(1 + x)^{5/3}} \quad (5.7)$$

where: $x = \frac{fL}{v(10)}$ is a dimensionless frequency, L a characteristic length and $v(10)$ the mean wind speed at 10m height.

In Sofistik, the Von Karman spectrum is used as it most accurately represents reality. It is defined as follows:

$$F_D = \frac{4X + 3021X^3}{(1 + 283X^2)^{\frac{11}{6}}} \quad (5.8)$$

where $X = \frac{fL}{V}$, and V is the mean wind speed at the considered height.

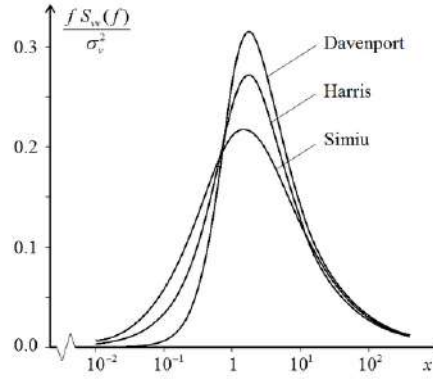


Figure 5.3: Turbulence spectra [34]

Last but not least, it is important to mention the coherence. Wind velocity is dependent on time, but also on space. The wind velocity at two points is not necessarily equal, but nevertheless related. The coherence will give the physical relation between the spectra of both processes. This is explained in more detail in [5].

Sofistik incorporates all these aspects if the parameters are properly defined.

The modelling of wind has been briefly discussed so far, which leads to the next part which is the behaviour of wind near a structure. This aspect influences the response of the structure. The wind load on an object is influenced by two phenomena being viscous friction and dynamic pressure. The viscous friction is defined in equation 5.9.

$$\tau = \eta \frac{dv}{dz} \quad (5.9)$$

where: τ is the shear stress, η the dynamic viscosity, v the air velocity and z the normal on the surface parallel to the flow direction.

The dynamic pressure can be found with Bernoulli's equation (equation 5.10). The influence of gravity is negligible for air (ρgz), thus reducing the equation to (5.11) which corresponds to the increase of pressure due to stagnating wind.

$$\frac{1}{2}\rho v^2 + \rho gz + p = \text{constant} \quad (5.10)$$

$$q = \frac{1}{2}\rho v^2 \quad (5.11)$$

Which phenomena dominates is determined by the Reynolds number (Re). It is defined as the ratio between inertia forces and viscous forces in a flow, see equation 5.12.

$$Re = \frac{F_i}{F_v} = \frac{\rho v^2 L^2}{\eta v L} = \frac{\rho v L}{\eta} = \frac{v L}{\nu} \quad (5.12)$$

where: L is the characteristic length of the object, ρ the density of air, ν the kinematic viscosity ($\nu = \frac{\eta}{\rho}$).

The Reynolds number gives a measure for the way air flows around an object (see figure 5.4). When looking at bridges, most often bluff bodies are encountered as is depicted in figure 5.5. The air flow is separated at the leading edge corners. A thin region of high shear and vorticity is created, which is separated from the outer layers. This is called a free shear layer. For bluff bodies with long "after-bodies", the shear layer may reattach to the surface, though vortices will still form and roll on the surface [16].

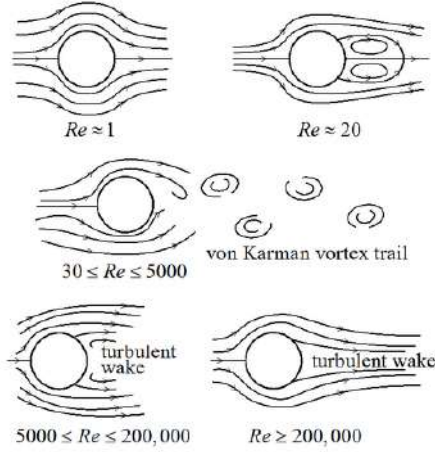


Figure 5.4: Air flow around objects depending on Re [34]

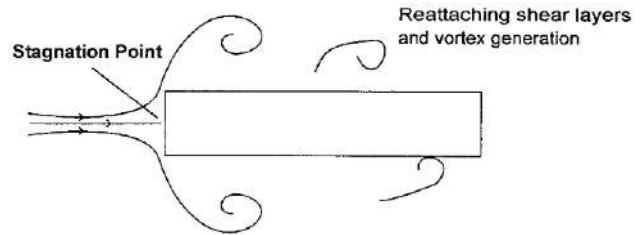


Figure 5.5: Flow around bluff body [16]

Last but not least, it is important to mention the aerodynamic forces and moments. These are entirely determined by the wind pressure and shear stress distribution over the body surface. The aerodynamic force and moment are the resultant of the integration of the wind pressure and shear stress over the body surface. The aerodynamic force has two components, being the drag and lift forces. The drag force acts parallel to the air flow, and the lift force perpendicular to the flow. The aerodynamic moment depends on where the moment equilibrium is taken. The simplified equations representing these aerodynamic forces and moment are:

$$F_D = \frac{1}{2} C_D \rho u^2 A_{ref} \quad (5.13)$$

$$F_L = \frac{1}{2} C_L \rho u^2 A_{ref} \quad (5.14)$$

$$M = \frac{1}{2} C_M \rho u^2 A_{ref} l_{ref} \quad (5.15)$$

where C_D , C_L and C_M are the aerodynamic coefficients for drag, lift and moment.

Typically, the aerodynamic wind coefficients are determined from either wind tunnel tests or CFD simulations. Sofistik can perform the latter. This can be used to determine the coefficients for the designed decks.

5.2. Wind-induced vibrations

Wind-induced vibrations in bridges can be classified in three main categories: turbulence-induced, vortex-induced and motion-induced. These excitation mechanisms can occur at the same time and amplify each other, though they tend to occur each at certain wind velocities (see figure 5.6). All three will be explained in the following sections.

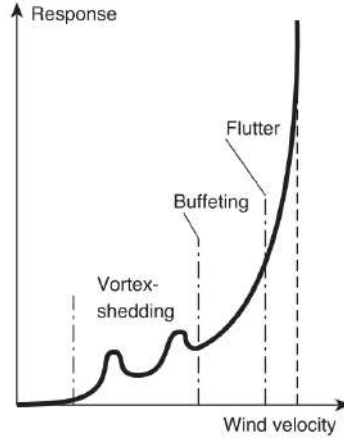


Figure 5.6: Dynamic response of a bridge, depending on wind velocities [13]

5.2.1. Turbulence-induced vibrations

Turbulence-induced vibrations are important for the calculation of section forces. These result from the static wind pressure, as well as the aerodynamic coefficient. Both vary with time and therefore induce dynamic excitation. This is also called buffeting, and often determines the size of structural elements in the design stage [32]. It is not a self-exciting mechanism, and is purely defined by the time and space fluctuations of the wind.

The buffeting loading of a bridge can be determined as the summation of the three components defined as follows [32]:

- The static component q_0 , resulting from the mean wind pressure, as defined in equation 5.11.
- The time- and space-dependent fluctuations of the wind pressures q_q , also called the quasi-static or background component.
- The velocity fluctuations near the resonance points of the structure: q_r , or resonant modal response.

This results in the following expression:

$$q = q_0 + \sqrt{q_q^2 + q_r^2} \quad (5.16)$$

where q_q and q_r need to be summed up through the square root-sum-of-square. This is due to the fact that these both are stochastic processes.

The buffeting forces can usually be modelled as quasi-static forces, as described in the previous section. The shape and angle of attack are important for the aerodynamic coefficients [13]. This results in the following equations:

$$F_D = \frac{1}{2} C_D(\alpha) \rho u^2 A_{ref} \quad (5.17)$$

$$F_L = \frac{1}{2} C_L(\alpha) \rho u^2 A_{ref} \quad (5.18)$$

$$M = \frac{1}{2} C_M(\alpha) \rho u^2 A_{ref} l_{ref} \quad (5.19)$$

where α is the angle of attack.

5.2.2. Vortex-induced vibrations

Vortex-induced vibrations occur when the deck (or pylon) is subjected to a transverse wind flow whose boundary layer separates. This leads to the creation of vortices being carried downstream. These vortices may shed alternately, and form a von Karman vortex trail, which is illustrated in figure 5.4. This finally excites the structure transversely to the flow direction. If these oscillations have frequencies close to the natural frequencies of the bridge, resonance may occur, leading to the so-called lock-in phenomenon. The critical vortex-shedding velocity is [13]:

$$U_{cr} = \frac{f_i D}{St} \quad (5.20)$$

where: D is the characteristic dimension of the structure (depth of girder), St is the Strouhal number (depends on the Reynolds number, needs to be defined with wind tunnel tests), and f_i the vortex shedding frequency.

This critical velocity is not a definite measure for when the bridge will oscillate, as it only indicates when the frequencies of the vortices and of a particular eigenmode coincide. For oscillation of the bridge to occur, other conditions need to be met: low structural damping, turbulence intensities typically below 5-8% (see equation 5.3). For girders, the wind axis needs to be close to perpendicular to the girder axis. A last condition is that the critical wind velocity is high enough to have enough energy to excite the bridge, but not too high either, causing disruption of the creation of vortices. Overall, vortex-induced vibrations haven't been critical for cable-stayed bridges [16].

5.2.3. Motion-induced vibrations

Motion-induced vibrations occur at high wind speeds due to the dominance of self-excited aerodynamic forces. Several types exist and the main ones will be explained here.

Flutter is characterized by a coupling of the vertical and torsional oscillations, which occurs when the corresponding frequencies are equal to one another. The following two coupled simplified equations of motion describe the behaviour of a suspended bluff body (see figure 5.7) [16]:

$$\ddot{z} + 2\eta_z \omega_z \dot{z} + \omega_z^2 z = \frac{F_z(t)}{m} + H_1 \dot{z} + H_2 \dot{\theta} + H_3 \theta \quad (5.21)$$

$$\ddot{\theta} + 2\eta_\theta \omega_\theta \dot{\theta} + \omega_\theta^2 \theta = \frac{M(t)}{I} + A_1 \dot{z} + A_2 \dot{\theta} + A_3 \theta \quad (5.22)$$

The aeroelastic derivatives, or flutter derivatives (A_i and H_i) determine the sensitivity of the bridge deck to flutter, based on the sign and magnitude of the derivatives. $F_z(t)$ and $M(t)$ are due to other mechanisms acting on the static body. ω_z and ω_θ are the undamped circular frequencies for vertical motion and rotation.

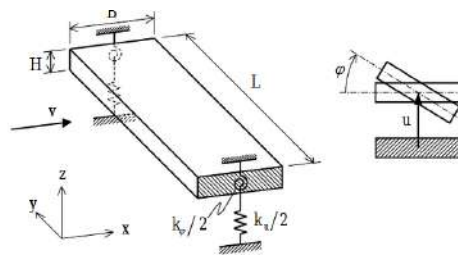


Figure 5.7: Suspended bluff body [29]

A few examples of types of motion-induced vibrations based on the flutter derivatives are given in table 5.1.

Name	Conditions	Type of motion	Type of section
Galloping	$H_1 > 0$	translational	Square section
'Stall' flutter	$A_2 > 0$	rotational	Rectangle, H-section
'Classical' flutter	$H_2 > 0, A_1 > 0$	coupled	Flat plate, airfoil

Table 5.1: Examples of types of aerodynamic instabilities [16]

When designing a bridge, usually the flutter derivatives aren't considered yet, and the design is based on the the critical flutter speed. The flutter derivatives have to be determined from wind tunnel tests or CFD models, and are required for extensive dynamic analyses. The critical flutter speed can be determined experimentally or with empirical formulae. It should be greater than the design wind speed at the location. Selberg's formula [16] is often applied:

$$U_F = 0.44d \sqrt{(\omega_T^2 - \omega_V^2) \frac{\sqrt{\nu}}{\mu}} \quad (5.23)$$

where: $\nu = 8(\frac{r}{d})^2$, $\mu = \frac{\pi \rho_a d^2}{4m}$, r the radius of gyration of the cross section, ω_T and ω_V the circular frequencies in the first torsional and first vertical bending modes.

Galloping is an oscillating motion occurring perpendicular, in vertical direction, to the wind direction. The load maintains itself by the motion. Only rectangular cross-section are usually affected. Galloping is oftentimes not the biggest issue for cable-stayed bridges, but a short description will be given nevertheless [29]. We consider the single mass spring system in figure 5.8, affected by steady wind with velocity v , perpendicular to the beam. The force in the direction of the vertical motion is:

$$F_z = \frac{1}{2} \rho B L v^2 C_z(\alpha) \quad (5.24)$$

with:

$$C_z(\alpha) = \frac{1}{\cos \alpha^2} (C_L(\alpha) \cos \alpha + C_D(\alpha) \sin \alpha) \approx C_L(\alpha) + \alpha C_D(\alpha) \quad (5.25)$$

In figure 5.9, C_z is given as a function of α . By taking $-\dot{u} = \alpha v$ (considering α small), we obtain:

$$F_z = \frac{1}{2} B L \rho v^2 \left(\frac{dC_z}{d\alpha} \right) \alpha = -\frac{1}{2} A \rho v \dot{u} \left(\frac{dC_z}{d\alpha} \right) \quad (5.26)$$

If $(\frac{dC_z}{d\alpha}) < 0$, a vertical force F_z will occur, thus amplifying the motion. This is when galloping will occur.

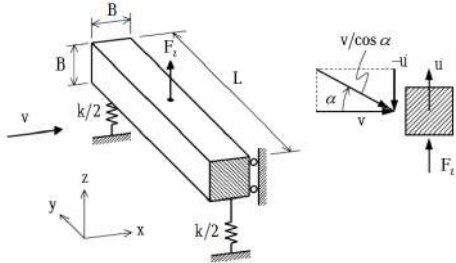


Figure 5.8: Galloping of a single mass-spring system, with steady wind [29]

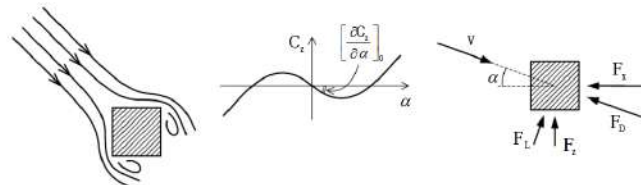


Figure 5.9: C_z as a function of α [29]

It must be mentioned that these motion-induced vibrations will not be included in the wind analyses in Sofistik. The reason lies in the complexity of determining flutter derivatives which

was not possible within the scope of this thesis. However, EN1991-1-4 gives guidelines in annex E to perform simple checks on sensitivity of the bridge for these type of vibrations. These checks will be performed.

II

Bridge design study

6

Preliminary design; Initial sizing of cable-stayed bridges

In this chapter, initial dimensions of the FRP and the steel-concrete cable stayed bridge concepts will be sought and obtained through analytic calculations. These dimensions are chosen by optimizing for costs. Considering no information is available on the optimal cable disposition for GFRP cable-stayed bridges, a short parameter study was done.

6.1. Concept 1: FRP cable-stayed bridge

6.1.1. Design approach: initial sizing through analytic calculation

Main design parameters and assumptions

To design an FRP cable-stayed bridge, the main design parameters have to be identified first and their influence has to be analyzed. In this study, the emphasis lies on the design of the deck, and therefore on the design parameters that affect that element.

When designing a cable-stayed bridge at an early design stage, several parameters can be varied, as has been mentioned in chapter 3. The main design element here is the FRP deck. Two main layouts were taken: a box girder and a sandwich plate alternative. These are illustrated in figure 6.1. The box girder deck has a top flange which consists of a sandwich panel spanning in transverse direction. This allows for a larger web spacing. The sandwich plate deck requires more webs to properly carry vertical loads. For each of these alternatives, the following two main design parameters were identified and varied to obtain the most cost-effective design (see figure 6.2):

- The distance between cable anchor points, which can also be translated to the number of cables. In this case, the deck was divided in segments of identical length based on the number of cables.
- The height of the deck.

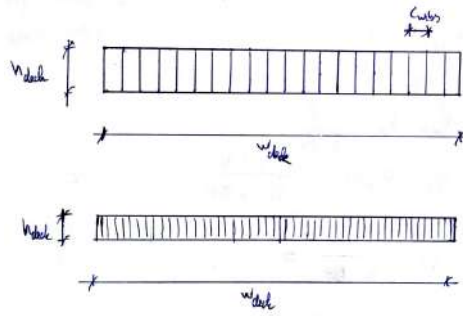


Figure 6.1: Two FRP deck layouts: box girder, and sandwich panel

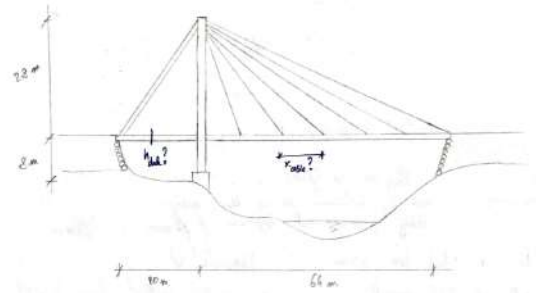


Figure 6.2: General bridge layout

Before starting the analytic design calculations, a few assumptions were made:

- A portal frame pylon is applied, which allows the application of two cable planes. This increases the torsional stiffness of the deck system.
- One pair of backing stay cables is applied in the 20 meter long side span.
- A longitudinal fan cable arrangement is applied. According to [2], structurally speaking, it is the most efficient system as the vertical forces from the cables are transferred with a minimum amount of steel to the pylon.
- Concerning the supports of the deck at tower level, it is assumed that the main and the side span are both simply supported.
- A few initial dimensions were defined, for a part, with the help of existing rules of thumb for cable-stayed bridge design. Some were already mentioned earlier in the conceptual design:
 - The pylon height was defined with the ratio pylon height / length of main span being equal to 0.35 - 0.45, which results in a chosen pylon height of 28m. This is applicable in this situation as it is related to the efficiency of the cables at carrying vertical forces: it results in a minimum angle at the cable-deck connection of 25 degrees.
 - The side span length was chosen according to the ratio of side span length and main span length $\frac{l_{ss}}{l_{ms}} = 0.3$, which results in a side span length of 20m.
 - The box girder deck has web spacings of 500mm and a width of 10m. The flange thickness increases linearly from 20 to 50 mm as the deck height increases. The web thickness increases linearly from 10 to 40 mm as the deck height increases.
 - The sandwich panel deck has web spacings of 100mm and a width of 10m. The flange thickness increases linearly from 15 to 35 mm as the deck height increases. The web thickness increases linearly from 8 to 20 mm as the deck height increases.

Definition of materials, loads and analyses

The materials that have been used are given in table 6.1. The properties of the anisotropic GFRP are given in table 6.2. Other material properties are given in table 6.3.

Materials	
Deck	Anisotropic GFRP ($V_f=50\%$), as defined in CUR96
Parallel strand cables	7mm steel wire strands, Y1860
Pylon	Concrete C40/50

Table 6.1: Applied materials

Stiffness properties	Anisotropic GFRP (Vf = 50%)
E_x [GPa]	25,8
E_y [GPa]	15,9
G_{xy} [GPa]	5,6
ν_{xy}	0,32

Strength properties	Anisotropic GFRP (Vf = 50%)
f_x [MPa]	310
f_y [MPa]	191
τ_{xy} [MPa]	90

Table 6.2: Anisotropic GFRP material properties

	Concrete C40/50
E_c [GPa]	35
$f_{c,k}$ [MPa]	40
f_{ctm} [MPa]	3,5
Reinforcement	B500

	Y1860
E_{eff} [GPa]	190
f_s [MPa]	837 (=0,45*1860)
Δf_s [MPa]	200

Table 6.3: Other material properties

For the analytic calculations, the loads for one cable plane are given in table 6.4. Only self-weight (dead load) and traffic loads (live loads) are considered at this stage. The dead load includes an estimate of the self-weight of the deck and the self-weight of additional installations. For traffic loads, Load model 1 from EN1991-2 is used (see figure 6.3).

Loads for one cable plane			
Dead load	Self-weight deck + additional weight from installations	UDL	12 kN/m
Live load	Traffic load model 1	UDL	14,33 kN/m
		CL	470 kN

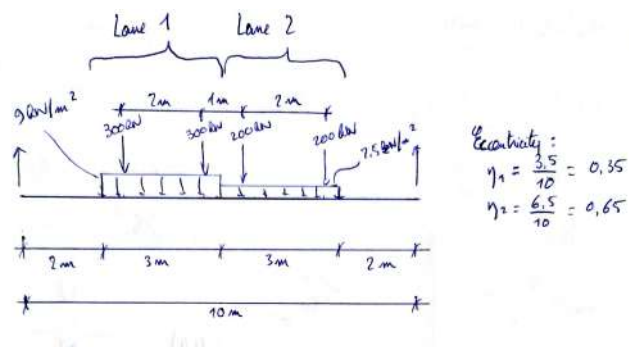


Table 6.4: Load definition for one cable plane

Figure 6.3: Load model 1 definition

Load factors have been determined according to table A2.4 from the Norwegian National Annex of EN1990 [10]. Material factors and conversion factors for FRP have been determined according to CUR96 [26]. These are given in appendix C.1.

Concerning the analyses performed, a design approach similar to the one applied in [32] is used. In these preliminary design calculations, the bridge is modelled in different ways, based on the element that is being analyzed. All calculations are performed for one cable plane, meaning half the bridge is considered.

- The bridge is modelled as an articulated system for the computation of normal forces in the deck, in the pylon, as well as the maximum forces in the cables (see figure 6.4). This allows for sizing of the backing stay cables and the cables in the main span through a strength check.
- A first attempt at sizing the deck can be done through a simple check on stresses occurring in the deck due to bending. The bending moments are computed as follows:
 - The bridge is modelled as a rigidly supported beam for the computation of bending moments due to dead load (see figure 6.5).
 - The bridge is modelled as a beam on elastic foundation for the computation of bending moments due to live load (see figure 6.6).

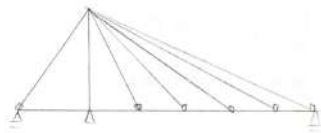


Figure 6.4: Articulated system



Figure 6.5: Rigidly supported beam, modelling the main span

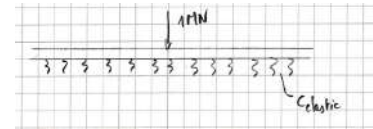


Figure 6.6: Elastically supported beam, modelling the main span

However a stress check due to bending moments in the deck was not critical and did not allow proper sizing of the deck. Unlike cable stayed-bridges made from "traditional" materials, the dead load of an FRP bridge is very low, thus causing very little bending moments. Another reason would be the strength-stiffness ratio, causing less load distribution across the cables (see also figure 3.11). Under live loads, the deck therefore tends to deform more than with traditional materials.

In order to size the deck, it was therefore found that the deflection of the deck, under live loads, was critical: in other words, the stiffness is decisive. The deflection has to be checked globally (entire main span), and locally (between two cable anchor points). The criteria that is used is taken from the N400 Bruprosjektering [30] and states the following: the maximum deflection should not exceed $\frac{L}{350}$, where L is the length of the considered span (global: main span, local: c.t.c. distance cable anchors). The deflections are computed as follow:

- To compute the global deflection, the deck is analyzed as a beam on elastic foundation, with a constant bedding factor. In reality, this bedding factor is not constant along the span. It depends on the number of cables and the dimensions of the deck and cables. The chosen bedding factor was computed for the most critical cable location for deflection (around $\frac{3}{4} * l$ of the main span), in order to obtain an approximation as close as possible to reality. This location was determined with the use of Sofistik: the module traffic loader can run loads at different locations on a beam to determine the maximum deflection. This was done for several cable anchor distances: the results are given in appendix C.7.
- To compute the local deflection, the deck is modelled as a continuous beam on supports. The loads are applied between two supports, over the elastic length computed earlier for the beam on elastic foundation. This was done in Sofistik with simple models, as it was not possible by hand. Extrapolation from the Sofistik result allowed the computation of local deflections for every deck dimensions of the corresponding cable anchor distance.

The critical check for determining the cost-optimal number of cables and deck height is therefore the deflection of the deck in the main span. To be more accurate, the global + local deflection was actually decisive. The approach is schematized in figure 6.7. This optimization process is done in Mathcad. The computed deflections have been checked against deflections computed with simple Sofistik models and showed good correlation. Often a slight increase in deflection can be noted due to the transition from an elastic foundation to discrete springs. All of these calculations are given in appendix C.1: only the Mathcad file for the chosen bridge is given. All other (not included) files contain the same calculations with a minor change being the number of cables.

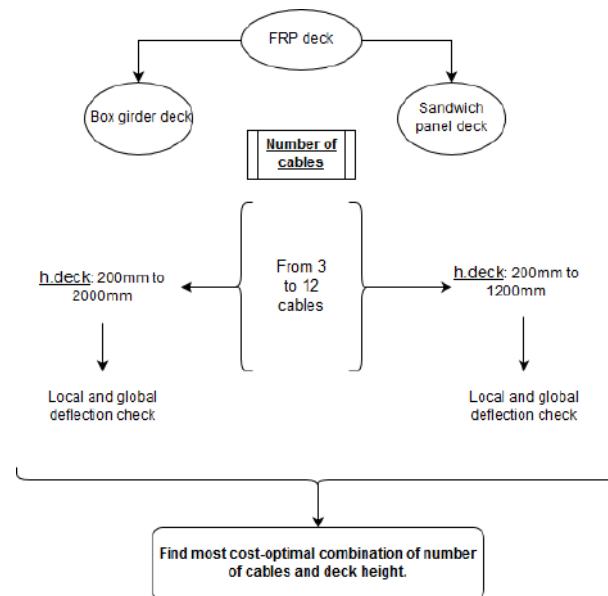


Figure 6.7: Diagram initial design of FRP deck

6.1.2. Results of analytic calculation

Cost-optimal values for main design parameters

Before presenting the results, it is important to mention how the costs have been obtained. For every chosen number of cables, the required deck dimensions, pylon dimensions and cable dimensions are computed. These are translated to costs according to the following unit prices (given by Royal HaskoningDHV, only includes production costs):

- FRP deck: 12 euros/kg
- Cables: 15 euros/kg
- Pylon: 700 euros/m³

This resulted in the graphs from figures 6.8 and 6.9, respectively for the box girder and sandwich panel alternatives. The overall costs of the sandwich panel alternative are quite superior to the box girder. Therefore the study for the box girder was further refined. From figure 6.8, the most economic bridge has 12 cables per cable plane with a corresponding deck height of 270mm (the 10 cables alternative has almost the same price). However, a few dynamic wind checks have been performed and the deck did not meet the flutter requirement defined in Annex E from EN1991-1-4. After a few iterations, the required deck height to meet this dynamic requirement was found to be at least 390 mm. When looking at figure 6.12, which represents the minimum required deck height based on the number of cables, it can be seen that a deck height of 390 mm corresponds almost to a bridge with 7 cables. Therefore this will be the chosen bridge design: 7 cables with a corresponding deck height of 430 mm.

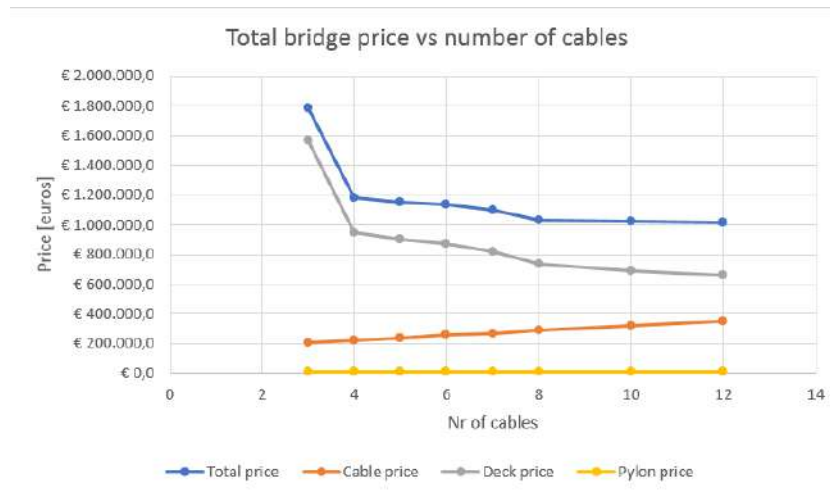


Figure 6.8: Minimum total bridge price optimum, for cable anchor distance ; Box girder deck

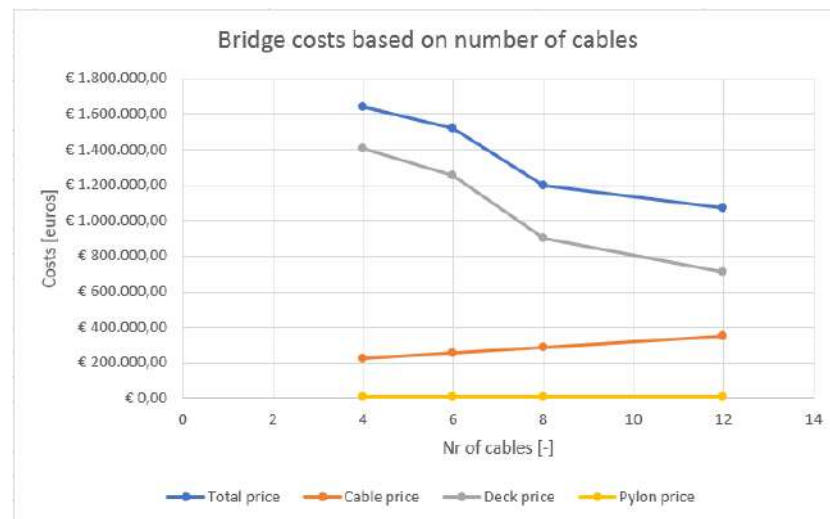


Figure 6.9: Minimum total bridge price optimum, for cable anchor distance ; Sandwich panel deck

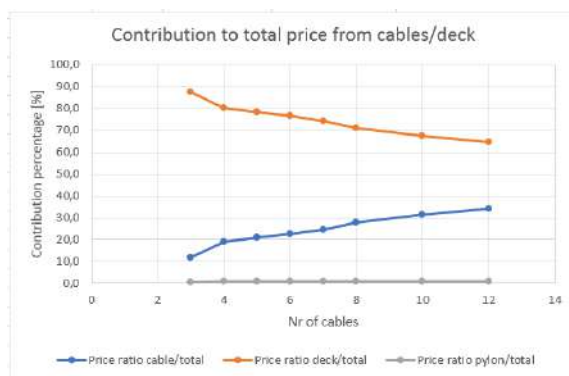


Figure 6.10: Price percentage of bridge elements ; Box girder deck

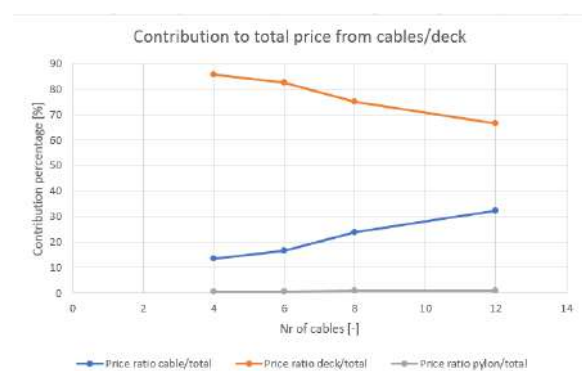


Figure 6.11: Price percentage of bridge elements ; Sandwich panel deck

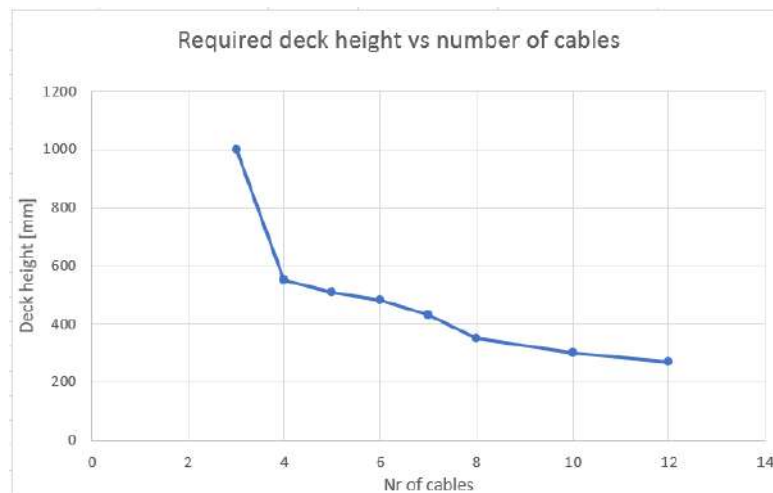


Figure 6.12: Required deck height for box girder deck, based on number of cables

It can be noticed that the pylon barely contributes to the total price variations as can be seen in figures 6.10 and 6.11. The costs contribution of the deck and cables both vary almost linearly for both deck layouts: as the number of cables increases (thus the costs), the deck dimensions become smaller and the total deck costs decrease. This is quite logical of course, but certain aspects have not been taken into account in this comparison. Just to give an example, the number of cable-deck connections increases when more cables are applied. The costs of these details can be consequent, but this is an aspect that is left aside within this thesis. It would require a separate study.

As a conclusion to this parameter study, the chosen deck system is the box girder deck. The values for the main parameters are as follows:

- Deck height: 430 mm.
- Number of cables per cable plane and corresponding cable anchor distance: 7 cables, 8 meters

The price of this system is **1.097 million euros**, without taxes, based on unit prices given by Royal HaskoningDHV. The corresponding price per square meter is 1305 euros. It is only an estimate of the costs, and serves as a point of comparison.

Strength and aerodynamic checks of chosen box girder deck system

The performed checks were done for the following bridge dimensions, see table 6.5 and figure 6.13. The bridge layout is the same as given previously. With a cable spacing of 8 meters, the number of cables per cable plane (in the main span) is 7, with a fan cable arrangement. The cable dimensions are given in table 6.6.

Deck dimensions	
Deck height	430mm
Web spacing	500mm
Flange thickness	24mm
Web thickness	14mm

Table 6.5: Dimensions of stay cables and deck, for 7 pairs of cables; box girder deck

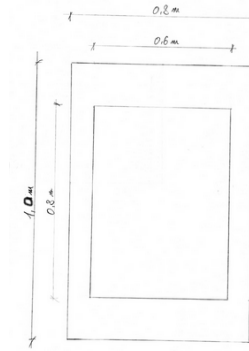


Figure 6.13: Dimensions of pylon, for 7 pairs of cables; box girder deck

Cables	Distance [m]	Angle [degrees]	Cable length [m]	A req [mm ²]	Strands req [-]	Actual strands [-]	A actual [mm ²]	Diameter [mm]	
1	8	74,1	29,1	1142	8	8	1200	39,08820095	1*8 strands
2	16	60,3	32,2	1264	9	12	1900	47,87307365	1*12 strands
3	24	49,4	36,9	1446	10	12	1800	47,87307365	1*12 strands
4	32	41,2	42,5	1667	12	12	1800	47,87307365	1*12 strands
5	40	35,0	48,8	1914	13	14	2100	51,70882946	2*7 strands
6	48	30,3	55,6	2179	15	16	2400	55,27906392	4*4 strands
7	56	26,6	62,6	2455	17	19	2850	60,23896333	1*19 strands
Backing stay cable	20	54,5	34,4	16200	108	108	16200	143,6192209	12*9 strands

Table 6.6: Cable dimensions; box girder deck

The strength unity checks are given in table 6.7. Further details can be found in appendix C.2. As explained earlier, the dimensions of the deck are based on deflections in SLS. Its dimensions were computed for a unity check of 1.0.

Bridge element	Performed calculation	Unity check	Note
Backing stay cable	Stress check	-	Required amount of strands 102. Provided amount of strands 108.
Pylon	Stress check due to combined bending and normal forces	0,884	-
Deck	Stress check due to bending forces	0,229	-
	Stress check due to normal forces	0,082	-

Table 6.7: Strength checks of bridge elements

A few aerodynamic checks have been performed, as defined in EN1991-1-4, annex E [9]. The first bending and torsional natural frequencies have been computed according to [32] and are as follows:

- First bending natural frequency: $f_b = 2.859 \frac{1}{s}$
- First torsional natural frequency: $f_t = 5.392 \frac{1}{s}$

The checks have been summarized in table 6.8. The computations can be found in appendix C.3. The average wind speed for the location (Sirdal, Norway) is found to be $v_m = 19.477 \frac{m}{s}$, from the Norwegian National Annex, EN1991-1-4 [8]. It is worth mentioning that an additional check on flutter safety according to [32] is given in equation 6.1. This criterion is not satisfied however ($\epsilon = 1.886$), whereas it is safe according to the Eurocode formula for the critical flutter wind speed.

$$\epsilon = \frac{f_t}{f_b} > 2 \quad (6.1)$$

Aerodynamic checks	Critical wind speed	Check
Vortex shedding	-	Can't be checked
Galloping	28,552 m/s	$> 1,25 * v_m = 24,3 \text{ m/s}$. Safe
Flutter	44,04 m/s	$> 2,0 * v_m = 38,95 \text{ m/s}$. Safe

Table 6.8: Aerodynamic checks of bridge deck according to EN1991-1-4, Annex E

6.2. Concept 2: Steel-concrete composite cable-stayed bridge

6.2.1. Design approach: initial sizing through analytic calculation

Main design parameters and assumptions

Obtaining initial dimensions for the steel-concrete composite cable-stayed bridge does not require a study into main design parameters. From literature [25], the optimal cable anchor distance lies around 12 meters. Therefore the following cable layouts will be studied and the most cost-optimal alternative will be chosen:

- 2 cables: c.t.c distance cable anchor points of 21.5m
- 3 cables: c.t.c distance cable anchor points of 16m
- 4 cables: c.t.c distance cable anchor points of 12.8m
- 5 cables: c.t.c distance cable anchor points of 10.7m

The steel-concrete composite cable-stayed bridge will have the same layout as the FRP one, to allow proper comparison of their costs. The main difference is the deck. Here the deck will be a plate girder deck as illustrated in figure 6.14.

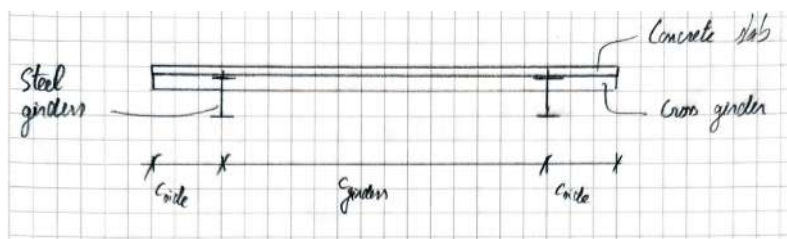


Figure 6.14: Steel-concrete plate girder deck

The assumptions made concerning the analytic design calculations are the same as for the FRP cable-stayed bridge. The only difference concerns the deck dimensions:

- The centre-to-centre distance between girders is 7 meters. The cantilevered parts of the deck have a length of 1.5 meters.
- The steel-concrete composite deck is assumed to have full interaction.

As was done previously, for every cable anchor distance, a Mathcad file was created. In the appendix C.4, only the file for the chosen cable anchor distance was added.

Definition of materials, loads and analyses

The materials that have been used are given in table 6.9. The concrete deck has an environmental class XC4, which corresponds to a nominal concrete cover of 40 mm. The properties of the concrete and the steel for the cables are given in table 6.3. The properties of steel S355 are given in table 6.10 Further properties are given in appendix C.4.

Materials	
Deck	Concrete C40/50 ; B500
Girders	Steel S355
Parallel strand cables	7 mm steel wire strands, Y1860
Pylon	Concrete C40/50 ; B500

Table 6.9: Materials steel-concrete composite cable-stayed bridge concept

Steel S355	
E [GPa]	210
f _y [MPa]	355

Table 6.10: Steel S355 properties

The loads are defined in table 6.11. The traffic load model 1 has been defined previously.

Loads for one cable plane			
Dead load	Self-weight deck + additional weight from installations	CL	27,4 kN/m
Live load	Traffic load model 1	UDL	14,325 kN/m
		CL	470 kN

Table 6.11: Load definition for one cable plane

The load and material factors have been determined according to the Norwegian NA of EN1990 [10].

The performed analyses have already been defined previously: the bridge is modelled according to the element that is being analyzed (see figures 6.4, 6.5 and 6.6) and all calculations are performed for one cable plane. Unlike the FRP cable-stayed bridge, the deck can be sized for bending moments due to dead and live loads. A few additional checks have been performed to determine the concrete deck height for instance (shear check). Only linear elastic analyses have been used.

6.2.2. Results of analytic calculation

Cost-optimal alternative

Before giving the result, the costs have been computed for the entire bridge. For the pylon and cables, their total weight was computed and multiplied by a unit price. The price of the deck was computed with its total surface area. The unit prices are as follows (includes only production costs):

- Steel girders: 5 euros/kg
- Concrete slab: 300 euros/ m^3
- Cables: 15 euros/kg
- Pylon: 700 euros/ m^3

Figure 6.15 shows the total bridge costs depending on the cable anchor distance. The cost-optimal number of cables is 4, which corresponds to a c.t.c cable distance of 12.8m. The excel sheet containing the results of this cost analysis can be found in appendix C.5.

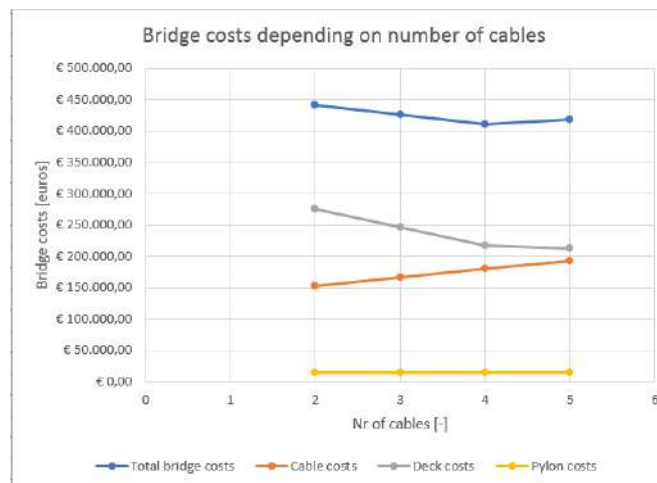


Figure 6.15: Total bridge costs depending on cable anchor distance

The corresponding total price is **411316 euros**, based on unit prices provided by Royal HaskoningDHV. The fact that the cost-optimal c.t.c distance between cables is 12.8m shows good concordance with the values from literature. This validates the results from this preliminary design.

Strength and aerodynamic checks of chosen alternative

The chosen alternative has the following dimensions, see table 6.12, 6.13 and figure 6.16. The cable spacing is 12.8 meters (4 cables per cable plane, in main span) and the bridge layout is still the same as previously given.

Deck system	Chosen dimensions
Steel girder height	650 mm
Flange thickness (top and bottom)	20 mm
Flange width (top and bottom)	250 mm
Web thickness	15 mm
Concrete deck height	350 mm

Table 6.12: Dimensions of deck, for cable spacing of 12.8m

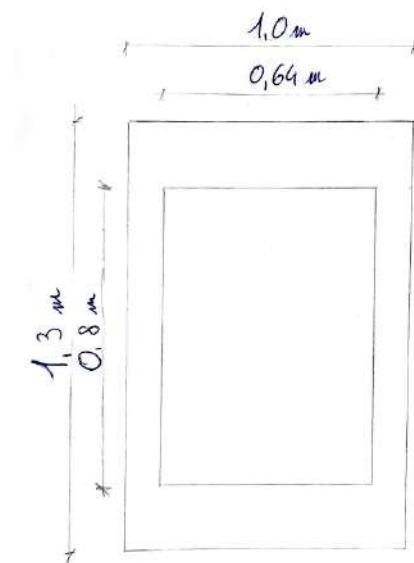


Figure 6.16: Dimensions of pylon, for cable spacing of 12.8m

Cables	Distance [m]	Angle [degrees]	Cable length [m]	A req [mm ²]	Strands req [-]	Actual strands [-]	A actual [mm ²]	Diameter [mm]	
1	12,8	65,43	30,79	2172	15	16	2400,00	55,28	4*4 strands
2	25,6	47,56	37,94	2676	18	19	2850,00	60,24	1*19 strands
3	38,4	36,10	47,52	3352	23	24	3600,00	67,70	2*12 strands
4	51,2	28,67	58,36	4117	28	28	4200,00	73,13	3*7 strands
Backing stay cable	20	54,46	34,41	-	-	108	16200,00	143,62	12*9 strands

Table 6.13: Dimensions of cables, for cable spacing of 12.8m

The strength unity checks are given in table 6.14. Further details can be found in appendix C.4. The dimensions of the bridge of the bridge have been computed based on the critical unity check being approximately between 0.85 and 0.95. The partial safety factors according to EN1990 have been applied. A quick check on deflections in SLS also indicated a unity check of 0.516 (=96mm/183mm).

Bridge element	Performed calculation	Unity check	Note
Backing stay cable	Stress check	-	See previous table
Main span stay cable	Stress check	-	See previous table
Pylon	Stress check due to bending and normal forces	0,886	-
Concrete deck	Shear check	0,98	-
	Bending stress check	0,492	-
	Compressive forces check	0,149	-
Steel girder	Bending stress check	0,94	-
	Buckling safety check	0,186	-

Table 6.14: Strength checks of bridge elements

Concerning the aerodynamic checks, the first bending and torsional natural frequencies have been computed according to [32]. These are as follows:

- First bending natural frequency: $f_b = 1.253 \frac{1}{s}$
- First torsional natural frequency: $f_t = 2.391 \frac{1}{s}$

The criterion on flutter ($\epsilon = \frac{f_t}{f_b} > 2$) is not satisfied, considering ϵ is equal to 1.908. The other criteria as defined in EN1991-1-4 have been checked too, see results in table 6.15. However the safety for vortex shedding has not been performed considering the Eurocode did not give indications on how to perform the check for this cross-section.

Aerodynamic checks	Critical wind speed	Check
Vortex shedding	-	Could not be checked
Galloping	145,985 m/s	$> 1,25 * v_{\text{m}} = 24,3 \text{ m/s}$. Safe
Flutter	93,96 m/s	$> 2 * v_{\text{m}} = 38,96 \text{ m/s}$. Safe

Table 6.15: Aerodynamic checks of bridge deck according to EN1991-1-4, Annex E

The check on flutter with the the formula from EN1991-1-4 is OK. The ϵ check is not satisfied: it only indicates as a quick and simple check whether flutter might or might not be an issue. In the EN1991-1-4 it is used as a criteria to determine whether the section is at risk. If it is, then the more detailed formula from table 6.15 should be applied.

6.3. Short discussion

For both bridges, initial dimensions of the main elements have been found with a few analytic calculations. The GFRP bridge required more calculations to find out what the optimal combination of dimensions was, with a critical check based on deflections. The stress checks have also been performed and were found to have low unity checks (0.25 for bending moments for example). As previously stated, this is due to the low stiffness and low self-weight

of the FRP deck. The steel concrete composite bridge was dimensioned based on a bending moment check for the deck. It is interesting to compare the EI and the mass per unit length of the two chosen decks, see comparison below. From the $\frac{EI}{m}$ ratio, the FRP deck has a higher stiffness per unit mass, which indicates that the system is relatively efficient. However if only the stiffnesses from both decks are compared, the steel-concrete deck has a far higher value, which is to be expected, with total costs inferior to the FRP deck.

- FRP deck:
 - $EI = 6.233 * 10^5 \text{ kNm}^2$
 - $m = 7.999 \frac{\text{kN}}{\text{m}}$
 - $\frac{EI}{m} = 7.793 * 10^4 \text{ m}^3$
- Steel-concrete composite deck:
 - $EI = 2.162 * 10^6 \text{ kNm}^2$
 - $m = 87.007 \frac{\text{kN}}{\text{m}}$
 - $\frac{EI}{m} = 2.485 * 10^4 \text{ m}^3$

Though no exact limiting value is given, it is desirable to keep the clearance height below the deck as large as possible. The needed deck height for the FRP alternative is 430 mm, whereas the composite alternative requires a deck height of 1000 mm. Now, if we consider the costs based on unit prices provided by Royal HaskoningDHV, the FRP alternative requires 1.097 million euros, compared to 0.41 million euros for the composite one. The FRP bridge is 2.5 times more expensive. So far, both bridges have their advantages, cost-wise or dimension-wise. The aerodynamic checks were in both cases positive but do need further attention. Especially the FRP bridge where the deck is on the verge of being safe for flutter.

In the next chapter, the chosen bridges with their dimensions will be given further detail, and analyzed with a Sofistik FEM model.

7

Preliminary design; static calculation with FEM

In this chapter, the bridges and their corresponding dimensions, obtained in chapter 6, are modelled in Sofistik. The different loads and how these have been computed are also detailed. The approach and the results will be given in the following sections, whereas the detailed calculations are in appendix D. Once both bridges are designed accordingly, the costs and total weights are estimated once more to give a general point of comparison.

7.1. Concept 1: FRP cable-stayed bridge

The first section will give more details about the geometry of the modelled system, with as starting point the results from the analytic calculation. Afterwards the load cases and combinations are given, and finally the results. Based on the results, required improvements are discussed in order to obtain a final design.

7.1.1. Geometry and Sofistik model

The global geometry of the bridge is now fixed and is given in figure 7.1. The bridge has two cable planes, therefore also 2 pylons which are connected with a transversal connector beneath the deck (at a height of 7.57 meters from the bottom of the pylon). The side span also requires cables. Therefore 2 pairs of cables were added at respectively 6 and 13 meters from the pylon. These were given the same dimensions as cables 1 and 2 as they are at a similar distance from the pylon.

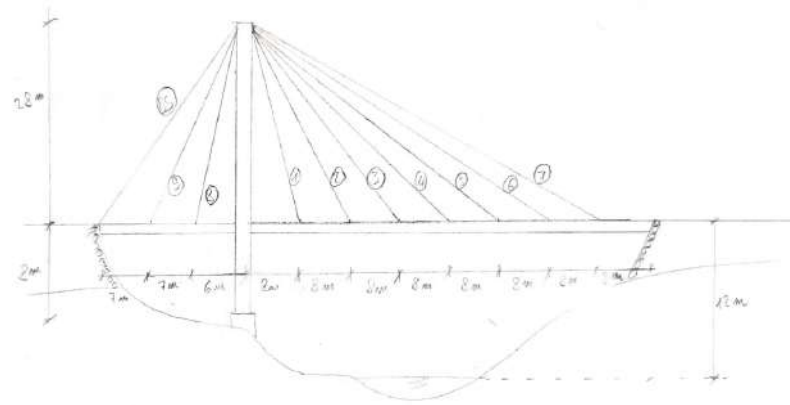


Figure 7.1: Global geometry FRP cable stayed bridge

The GFRP deck is illustrated in figure 7.2. Additionally it has transverse stiffeners placed every 4 meters. These are of course provided at the cable anchor points as well. The connection between the cables and the deck is done through a rectangular hollow cross beam. Its dimensions are determined within Sofistik based on the cable prestressing forces and the loads from the deck. At the cable-deck connections, the deck is fixed to these cross-beams. It is also illustrated in figure 7.2. At the connection between deck and backing stay cable, the cross girder has larger dimensions to carry the large forces transmitted by the cables.

In figure 7.3, a simplified top view of the deck is given. This is simply to illustrate how, from a constructive point of view, the stiffeners in the deck can be arranged. The edge stiffeners are continuous, whereas the inner longitudinal webs are discontinuous (only a few are actually illustrated here). The transverse stiffeners can therefore be continuous from edge to edge. Avoiding discontinuities in these elements reduces the weak, potential failure spots from gluing. This has no further effect on the Sofistik model.

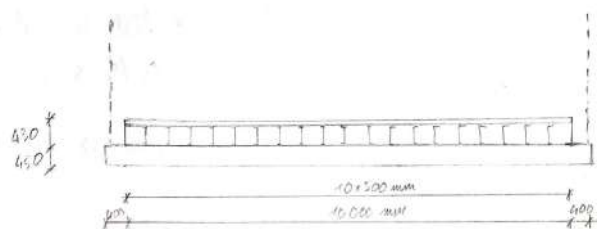


Figure 7.2: Dimensions of GFRP deck

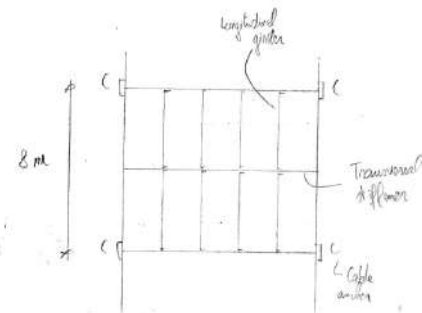


Figure 7.3: Schematic top view of deck

Finally, the top plate of the deck is assumed to be a sandwich panel spanning in transverse direction, whereas it is modelled as one plate in Sofistik. The fibres are mainly oriented in longitudinal direction, giving the strong properties to the plate in that direction. This has no further effect on the calculations and will be kept like this for the moment. The dimensions of the sandwich panel can easily be determined when necessary, considering the required stiffness is known. The dimensions of all deck elements are given in tables 7.1 and 7.2.

Deck		
Length main span [m]	64	
Length side span [m]	20	
Height [mm]	430	
Width [mm]	10000	
	t [mm]	Spacing [mm]
Edge plates	28	-
Longitudinal webs	14	500
Upper flange	24	-
Lower flange	24	-
Transversal stiffeners	28	4000

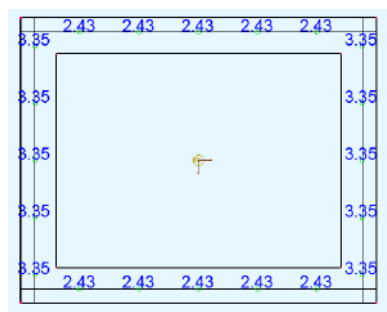
Table 7.1: Dimensions of GFRP deck

Cross girders	
Length [m]	10.8
Height [mm]	420
Width [mm]	420
Thickness [mm]	30

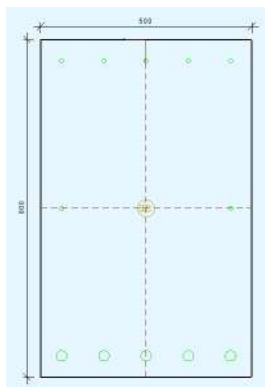
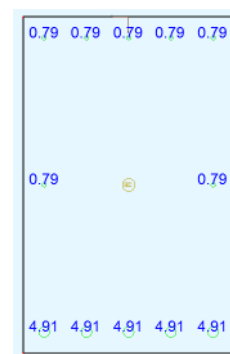
Table 7.2: Dimensions of FRP cross girders

Cross girder backing stay cable	
Length [m]	10.8
Height [mm]	540
Width [mm]	600
Thickness webs [mm]	22
Thickness flanges [mm]	18

The dimensions of the pylon have been given previously. In figure 7.4, the minimum amount of required longitudinal reinforcement is also given, in cm^2 . This has been computed according to EN1992-1-1 (equation 9.1N). The actual required amount of longitudinal reinforcement will be computed by Sofistik.

Figure 7.4: Minimum required longitudinal reinforcement in pylon, in cm^2

The two pylons are connected by a transversal connector. The dimensions and minimum longitudinal reinforcement are given in figures 7.5 and 7.6.

Figure 7.5: Dimensions of transversal connector:
800*500mmFigure 7.6: Minimum longitudinal reinforcement
transversal connector in cm^2

The dimensions of the cables are given in table 7.3 below. These have been computed based on the maximum utilization level for strength, in the previous section.

The model entered in Sofistik is given in figure 7.7. The side span is simply supported in y- and z-direction. At the pylon, the deck is supported in x-, y- and z-direction, whereas the pylons are rigidly supported. The main span is simply supported in y- and z-direction. All deck elements have been modelled as plates. The pylons and transversal connector are beam elements, as well as the cross girders. The cables are each modelled as one single

Cables	Distance [m]	Angle [degrees]	Cable length [m]	A req [mm ²]	Strands req [-]	Actual strands [-]	A actual [mm ²]	Diameter [mm]	
1	8	74,1	29,1	1142	8	8	1200	39,08820095	f*8 strands
2	16	60,3	32,2	1264	9	12	1800	47,87307365	f*12 strands
3	24	49,4	36,9	1446	10	12	1800	47,87307365	f*12 strands
4	32	41,2	42,5	1667	12	12	1800	47,87307365	f*12 strands
5	40	35,0	48,8	1914	13	14	2100	51,70882946	2*7 strands
6	48	30,3	55,6	2179	15	16	2400	55,27906392	4*4 strands
7	56	26,6	62,6	2455	17	19	2850	60,23896333	f*19 strands
8	6	77,9	28,6			8	1200	39,08820095	f*8 strands
9	13	65,1	30,9			12	1800	47,87307365	f*12 strands
Backing stay cable	20	54,5	34,4	16200	108	108	16200	143,6192209	12*9 strands

Table 7.3: Cable dimensions for FRP cable-stayed bridge

cable element. The backing stay cables are self-anchored, in the deck. The other cables are connected through a rigid element to the cross girders.



Figure 7.7: Sofistik FRP bridge model

7.1.2. Load cases and combinations

In table 7.4, a summary of the load cases defined in Sofistik is given. Further details are given in the following subsections.

Permanent loads					
	Action	LC			
Dead load: self-weight	G_1	1			
Additional dead load	G_2	2			
Cable prestress	P	20			

Variable loads		
Wind		
Wind X+	W	70
Wind X-	W	71
Wind Y+	W	72
Wind Y-	W	73
Wind Z+	W	74
Wind Z-	W	75

Traffic: LM1 tandem system and UDL					
Maximum my mx p px	GR_T	601	Maximum my mx p px	GR_U	681
Minimum my mx p px	GR_T	602	Minimum my mx p px	GR_U	682
Maximum vz my t py	GR_T	603	Maximum vz my t py	GR_U	683
Minimum vz my t py	GR_T	604	Minimum vz my t py	GR_U	684
Maximum mt nx m pz	GR_T	605	Maximum mt nx m pz	GR_U	685
Minimum mt nx m pz	GR_T	606	Minimum mt nx m pz	GR_U	686
Maximum n ny ptx ux	GR_T	607	Maximum n ny ptx ux	GR_U	687
Minimum n ny ptx ux	GR_T	608	Minimum n ny ptx ux	GR_U	688
Maximum mz vx pty uy	GR_T	609	Maximum mz vx pty uy	GR_U	689
Minimum mz vx pty uy	GR_T	610	Minimum mz vx pty uy	GR_U	690
Maximum vy vy ptz uz	GR_T	611	Maximum vy vy ptz uz	GR_U	691
Minimum vy vy ptz uz	GR_T	612	Minimum vy vy ptz uz	GR_U	692

Table 7.4: Load cases as defined in Sofistik

Dead load and additional dead load

The dead load due to selfweight of the structural elements is automatically computed by Sofistik. The additional dead load consists of a wearing layer made of epoxy with a thickness of $t=12\text{mm}$, as well as 4 guard/security railings. This is summarized in table 7.5.

Non-structural	Width [m]	L [m]	A [m ²]	t [m]	q [kN/m ²]	q [kN/m]
Wearing layer 12m	10	84	840	0,012	0,276	-
Railings	0,5	336	-	-	-	0,6

Table 7.5: Additional dead load as defined in Sofistik

Cable prestressing

The cables have to be prestressed in order to meet the dead load condition: zero vertical displacement at the cable anchor points and zero horizontal displacement at the top of the pylon. This gives an optimal bending moment distribution in the deck and zero bending moments in the pylon, due to dead load. The computation was performed with Sofistik, which allows a precise calculation of the prestressing forces in all 10 pairs of cables. The resulting forces are given in table 7.6.

Cable	Prestressing force [kN]
1	90,1
2	107,5
3	115,1
4	124,6
5	145,2
6	162
7	187,9
8	82,3
9	75,6
Backing stay cable	1469

Table 7.6: Cable prestressing forces

Wind loads

The wind loads have been computed according to EN1991-1-4 and corresponding national annex NS. The fundamental wind velocity $v_{b,0}$ was taken as 24 m/s. Further information about the calculation can be found in appendix D.1. The wind loads are given in table 7.7 and 7.8. The wind loads have been combined in order to obtain intermediate load combinations, as defined in EN1991-1-4. These are given in table 7.9.

	Reference height [m]	Wind load			
		x-direction	y-direction	z-direction	
Girder	12,385	0,87	0,35	0,32	kN/m2
Pylon	12	1,49	0,95	-	kN/m
	24	1,88	1,20	-	kN/m
	36	2,13	1,36	-	kN/m
Transverse connector	10,48	-	0,85	0,42	kN/m

Table 7.7: Wind loads on FRP bridge

Cables		1	2	3	4	5	6	7	8	9	BS cable
Reference area	[m ² /m]	0,04	0,05	0,05	0,05	0,05	0,06	0,06	0,04	0,05	0,14
Force Fw bottom cables	[kN/m]	0,03	0,04	0,04	0,04	0,04	0,04	0,05	0,03	0,04	0,11
Force Fw top cables	[kN/m]	0,04	0,05	0,05	0,05	0,06	0,06	0,07	0,04	0,05	0,16

Table 7.8: Wind loads on FRP bridge cables

Wind	Combination	800	801	802	803	804	805	806	807
Wind X+	70	1	1			0,4		0,4	
Wind X-	71			1	1		0,4		0,4
Wind Y+	72					1	1		
Wind Y-	73							1	1
Wind Z+	74	1		1					
Wind Z-	75		1		1				

Table 7.9: Wind load combinations

Traffic loads

The traffic loads were computed according to EN1991-2, load model 1. The definition of the lanes and the loads has been done previously in section 6.1.1. To obtain the most critical positions of the tandem systems and the UDL, Sofistik computed the force, moment, stress and displacement envelopes when placing the loads at different spots along the length of the bridge. These envelopes are given by load cases 601-612 and 681-692.

Design load combinations

The load combinations for design are defined according to EN1990:NS. Four main combinations have been defined:

- Serviceability limit state, characteristic combination:

$$G_{k,j,sup} + G_{k,j,inf} + P_k + Q_{k,1} + \phi_{0,i} * Q_{k,i} \quad (7.1)$$

- Serviceability limit state, frequent combination:

$$G_{k,j,sup} + G_{k,j,inf} + P_k + \phi_{1,1} Q_{k,1} + \phi_{2,i} * Q_{k,i} \quad (7.2)$$

- Serviceability limit state, quasi-permanent combination:

$$G_{k,j,sup} + G_{k,j,inf} + P_k + \phi_{2,i} * Q_{k,i} \quad (7.3)$$

- Ultimate limit state, fundamental combination:

$$\gamma_{G,j} * G_{k,j} + \gamma_P * P_k + \gamma_{Q,1} * Q_{k,1} + \gamma_{Q,i} * \phi_{0,i} * Q_{k,i} \quad (7.4)$$

In Sofistik, these have respectively been denoted as combinations 1 to 4. According to EN1990 (A.2.4.2), the SLS frequent combination should be used when assessing deformations of road bridges. The stresses will be checked with the ULS fundamental combination. An overview of the actions and their corresponding factors is presented below in table 7.10. With Sofistik, for each design combination, several design loadcases are obtained, giving envelopes for moments, forces, stresses and deformations in the structure.

type	part	sup	Designation	$\gamma-u$	$\gamma-f$	$\gamma-a$	ψ_0	ψ_1	ψ_2	ψ_{inf}
G_1	G	perm	dead load g1	1.35	1.00	1.00	1.00	1.00	1.00	1.00
G_2	G	perm	ADL	1.35	1.00	1.00	1.00	1.00	1.00	1.00
P	P	perm	prestressing	1.00	1.00	1.00	1.00	1.00	1.00	1.00
W	Q	excl	wind loading	1.50	0.00	1.00	0.60	0.20	0.00	0.60
GR_T	Q_1	excl	gr1a LM1	1.35	0.00	1.00	0.75	0.75	0.00	0.80
GR_U	Q_1	excl	gr1a LM1	1.35	0.00	1.00	0.40	0.40	0.00	0.80

Table 7.10: Overview of actions and combination factors

7.1.3. Results of FEM calculation

A static calculation of the structure has been performed with Sofistik, with the load combinations defined earlier. In the following sections, only the critical results are presented, which are also summarized in appendix D.2.

FRP deck

The FRP deck is verified in ULS with an elastic analysis. The maximum stresses occurring in the deck have been checked against the limit stresses as defined in CUR 96 (see equation 7.5). The conversion and material factors are also given in tables 7.11 and 7.12.

$$X_d = \frac{\eta_c * X}{\gamma_M} \quad (7.5)$$

Material factor	
GFRP	
γ_{M1}	1,15
γ_{M2}	1,2
γ_M	1,38

Table 7.11: Material factor FRP

Conversion factor		
Temperature	η_{ct}	0,9
Moisture	η_{cm}	0,9
Creep	η_{cv}	0,7
Fatigue	η_{cf}	0,9
Strength checks	$\eta_{ct} * \eta_{cm} * \eta_{cv}$	0,567
Comfort (vibrations)	$\eta_{ct} * \eta_{cm} * \eta_{cf}$	0,729
Deflection checks - long term	$\eta_{ct} * \eta_{cm} * \eta_{cv} * \eta_{cf}$	0,5103
Deflection checks - short term	$\eta_{ct} * \eta_{cm} * \eta_{cf}$	0,729

Table 7.12: Conversion factors FRP

The resulting design stresses for strength checks are:

- $\sigma_1 = 127.4 \text{ MPa}$
- $\sigma_2 = 78.5 \text{ MPa}$
- $\tau_{12} = 37.0 \text{ MPa}$

The critical stresses in the deck are given in table 7.13, per element, and are based on envelopes of maximum stresses. Several elements have too high unity checks. These are in every case due to peak values, mostly due to the concentrated modelling of load introductions. The most critical element is the bottom flange, with a unity check of 1.90. In figure 7.8, the peak values for longitudinal stresses in the bottom flange are illustrated. Peak

values occur at the left support, in the side span, where the backing stay cables introduce forces in the deck. This is also the reason for higher stresses in the edge stiffeners. The high stresses in the transverse stiffeners are due to the deck-cable connections in the main span (further explanations in section 7.1.4).

	Critical stress	Value [MPa]	Unity check
Bottom flange	nxy	70.3	1.90
Top flange	nxx	-72.8	0.57
Edge stiffeners	nxx	-199.4	1.57
Longitudinal stiffeners	nxy	32.15	0.87
Transversal stiffeners	nxy	-46.5	1.26

Table 7.13: Critical stresses in GFRP deck

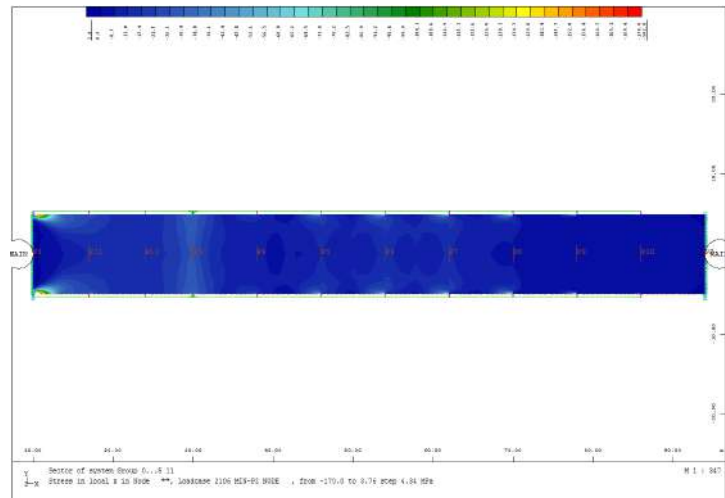


Figure 7.8: Longitudinal stresses in bottom flange of deck

A quick check on local buckling of the longitudinal stiffeners was also performed. This was done with a separate model in Sofistik. A wheel load, or 150 kN, as defined in EN1991-2 was taken and placed over a width of 400 mm on the stiffener. An imperfection of 2 mm was taken as defined in appendix H3 of CUR96 [26]. The resulting critical stress is 69.7 MPa (see figure 7.9). Considering most of the fibres are oriented in longitudinal deck direction, this stress was checked with a design stress of 70.6 MPa (includes material and conversion factor for stability). The resulting unity check is 0.99.

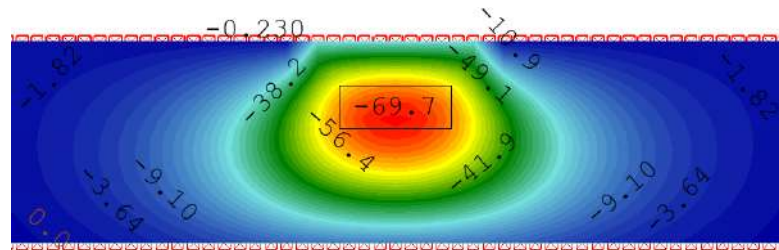


Figure 7.9: Critical buckling stress from wheel load in longitudinal stiffener, in MPa

The cross girders in the main span have been designed to have a high stiffness, in order to reduce local deflections of the deck in transverse direction. At the moment the deflection is 21.6 mm for the most critically loaded girder, which is acceptable if the deflection is to be limited to $L/350$ ($=10800/350=30.9\text{mm}$). The unity strength checks are quite low because of this stiffness requirement (see table 7.14). Concerning the cross girder for the backing stay cable, the unity checks for forces in local Z-direction are too high, especially the shear force V_z . With a maximum horizontal deflection of 24.2 mm, it satisfies the stiffness requirement of $L/350$ ($=30.9\text{mm}$).

Strength check

Cross girder backing stay cable						
Max My	978,3	[kNm]	M_Rd,c,y	1015	[kNm]	UC 0,96
Max Mz	1362	[kNm]	M_Rd,c,z	1315	[kNm]	UC 1,04
Max Vy	654,2	[kN]	V_Rd,y	715,53	[kN]	UC 0,91
Max Vz	1234	[kN]	V_Rd,z	774,69	[kN]	UC 1,72

Cross girders main span						
Max My	226,8	[kNm]	M_Rd,c,y	900	[kNm]	UC 0,25
Max Mz	430,9	[kNm]	M_Rd,c,z	900	[kNm]	UC 0,48
Max Vy	340,2	[kN]	V_Rd,y	2016	[kN]	UC 0,17
Max Vz	181,8	[kN]	V_Rd,z	2016	[kN]	UC 0,09

Table 7.14: Strength checks of cross girders FRP deck

The maximum deflection in the main span is 295.7 mm for the SLS frequent combination, which includes the conversion factor for short term deflections. The limit defined by Bruprosjektering [30] is $\frac{L}{350} = \frac{64000}{350} = 182.9\text{mm}$. This results in a unity check of 1.62 (see also section 7.1.4 for optimization). In the side span, the maximum deflection is 81.0 mm. The limit is equal $\frac{20000}{350} = 57.1\text{mm}$. The unity check is equal to 1.42. In figure 7.10, the deflections can be visualized. These results include local effects (transversely, and longitudinally between stays), and have in both cases been measured in the middle of the deck.

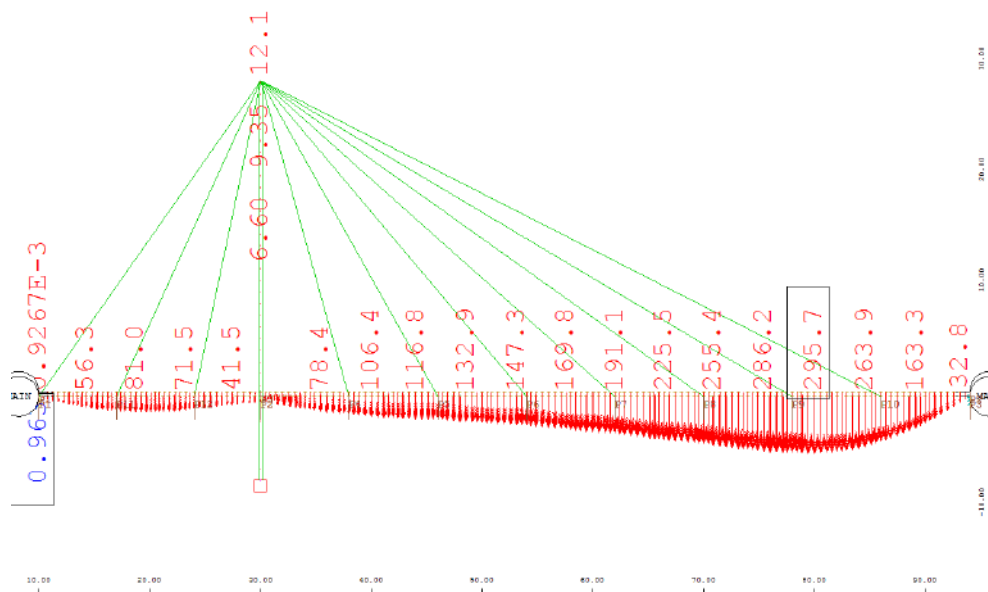


Figure 7.10: Maximum deflections GFRP deck, SLS frequent load combination, in mm

Pylon and transversal connector

The design of the pylon and transversal connector has been done by the Sofistik design module both in ULS (fundamental combination) and in SLS (frequent combination). The module computed the required amount of reinforcement in both elements. This resulted in the following required amounts of longitudinal and shear reinforcement (in cm^2), see figures 7.11 and 7.12. The pylon only requires minimal shear reinforcement ($1.12 \text{ cm}^2/\text{m}$) whereas the transversal connector requires on average $26.7 \text{ cm}^2/\text{m}$.

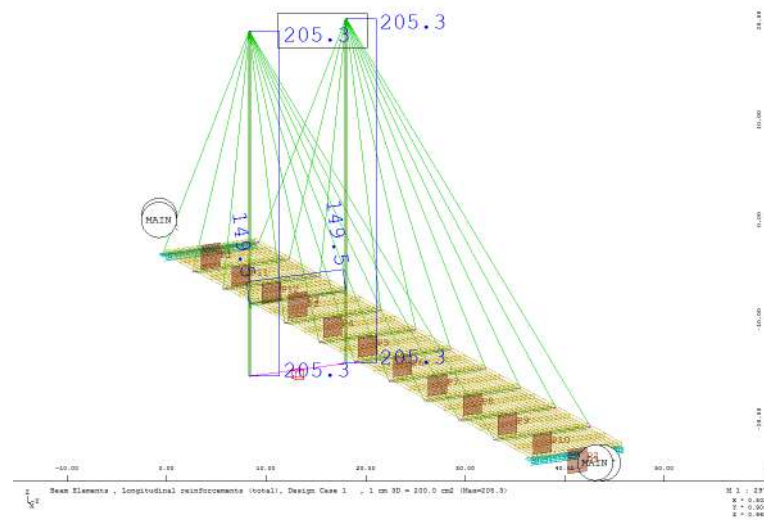


Figure 7.11: Required longitudinal reinforcement in pylon and connector in cm^2

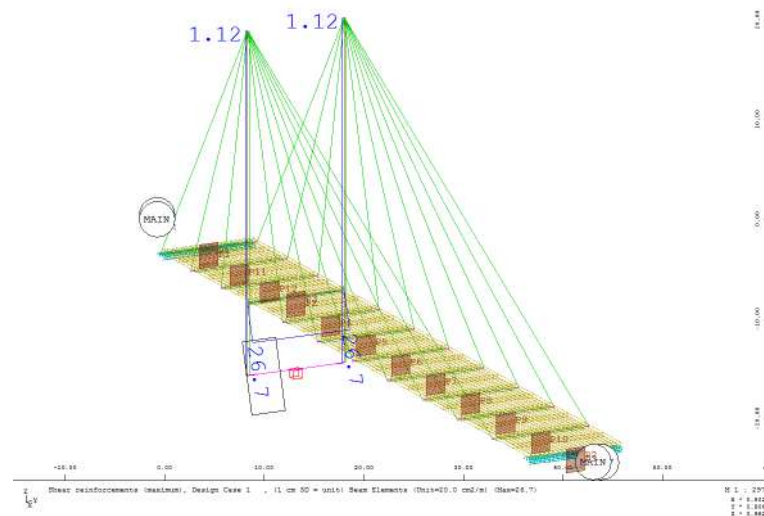


Figure 7.12: Required shear reinforcement in pylon and connector in cm^2

The maximum horizontal deflection of the pylon occurs in y direction (see figures 7.14 and 7.13) and is equal to 121.9 mm. In x-direction, the pylon deflects 82.6 mm maximum. The maximum vertical deflection of the connector is equal to 27.4 mm. With an allowable deflection of $\frac{10000}{350} = 28.6$, the check is ok.

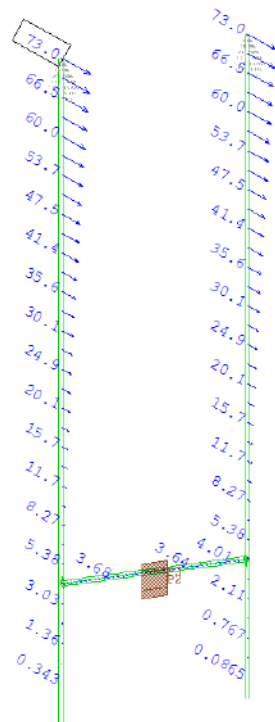


Figure 7.13: Maximum horizontal deflection of pylon in x direction, in mm

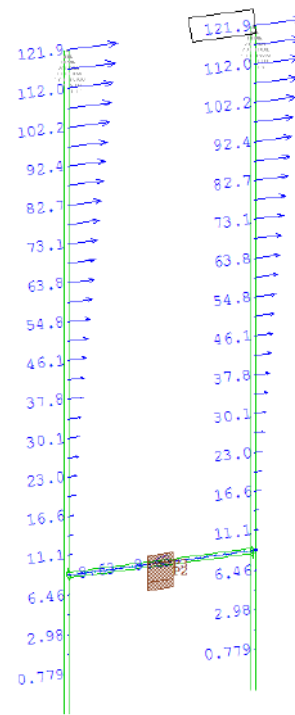


Figure 7.14: Maximum horizontal deflection of pylon in y direction, in mm

Cables

The cables have been checked for maximum stresses in ULS. The stress checks are given in table 7.15. The tensile design stress for the cables is $f_s = 837 \text{ MPa}$.

Cable	max N [kN]	A actual [mm ²]	max stress [Mpa]	UC stress max
1	95,2	1200	79,3	0,09
2	124,6	1800	69,2	0,08
3	143,6	1800	79,8	0,10
4	168,3	1800	93,5	0,11
5	206,7	2100	98,4	0,12
6	230,3	2400	96,0	0,11
7	194,6	2850	68,3	0,08
8	75,9	1200	63,3	0,08
9	75,7	1800	42,1	0,05
Backing stay cable	1382	16200	85,3	0,10

Table 7.15: Stresses in cables in ULS, including unity checks

7.1.4. Improvements and final design

The required improvements are as follows:

- The reduction of peak values in the deck due to the introduction of local forces.
- The cross girder for the backing stay cable is overloaded and needs to be further optimized.
- The final and main issue is the deflection in the main span, and side span. These are too large by an important amount.

Considering the large deflections of the deck, this was dealt with first. The reason for this difference with the hand calculation results is the horizontal deflection of the pylon and the cross girder to which the backing stay cables are connected. The deck also tends to undergo axial compression. This causes loosening of the cables and therefore a general loss of stiffness of the bridge. These effects were not incorporated in the analytic calculations: the deflection calculation was based on the extension of the critically loaded stay, and backing stay cables, but did not include deflection of the pylon. Additionally, the system was 2D and therefore did not consider transverse deflections.

To remedy the issue of the cables loosening due to the deflection of the pylon, the latter was given larger dimensions. Concerning the deflection of the cross girder for the backing stay cable, several solutions exist, each with its advantages and drawbacks:

- A massive concrete cross girder. It is rather cost-efficient and can easily achieve the required stiffness, but an issue would be the connection with the FRP deck.
- A steel, rectangular hollow section. Would require large dimensions to obtain required stiffness.
- Placing FRP beams to transfer the forces into the deck, with diagonals. Probably rather complex, and expensive.

Defining which solution is optimal would require a separate study. For now, the choice was made to apply a massive concrete girder, which is placed in the abutments. Furthermore, the backing stay cable was given larger dimensions. All these changes are summarized in tables 7.16.

Pylon		
Outer depth	1300	[mm]
Outer width	800	[mm]
Inner depth	700	[mm]
Inner width	500	[mm]

Backing stay cable		
Metallic cross-sectional area	81900	[mm ²]
Diameter	323	[mm]
Nr of strands	6*91 strands	

Cross girder BS cable		
Material	Concrete C40/50; B500	
Dimensions	1500*1500 mm	

Table 7.16: Intermediate improvements to FRP cable-stayed bridge

These changes resulted in the following total deflections. In figure 7.15, only the deflections of the edges of the deck are shown and only local deflections between the stays are shown. This result should be almost equal to the hand calculations, and this is the case (hand calculation: 183 mm ; FEM: 194.7 mm). The 11.7 mm difference is due to the pylon still slightly deflecting horizontally, which cannot be completely suppressed. In figure 7.16, the maximum deflection of the entire deck is shown, including all local deflections (longitudinal and transverse). These still exceed the SLS limit of $L/350$, both in side span and main span.

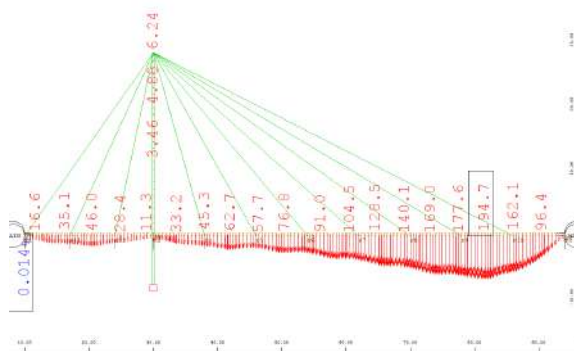


Figure 7.15: Deflection of FRP deck ; edges only, with intermediate improvements

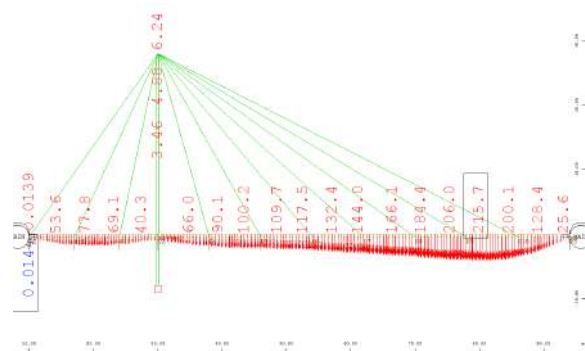


Figure 7.16: Deflection of entire FRP deck, with intermediate improvements

The local deflections from figure 7.16 were checked:

- In transverse direction: maximum local deflection is 26.1 mm. The limit is $10000/350 = 28.6$ mm. OK. However, in the side span, the maximum local deflection in transverse direction was 40.3 mm.
- In longitudinal direction, between 2 cables: maximum local deflection has been checked previously and satisfied the requirement.

The deck dimensions will be slightly increased in the side span in order to reduce local deflections. In the main span, the deflections will be reduced by increasing the cable dimensions.

The final dimensions of the deck are given below in table 7.17, and the final dimensions of the cables in table 7.18. The final dimensions of the pylon are as shown in table 7.16, as well as the final dimensions of the cross girder for the backing stay cable. The dimensions of the other cross girders have not been changed.

Deck	
Height [mm]	430
Width [mm]	10000
Longitudinal web spacing [mm]	5000
Edge stiffeners thickness [mm]	28
<i>Main span</i>	
Flange thickness [mm]	24
Web thickness [mm]	14
Transversal stiffener thickness [mm]	32
Transversal stiffener spacing [mm]	4000
<i>Side span</i>	
Flange thickness [mm]	32
Web thickness [mm]	14
Transversal stiffener [mm]	32
Transversal stiffener spacing [mm]	~2000-2300

Table 7.17: Final dimensions of FRP deck

Cables	Distance [m]	Actual strands [-]	A actual [mm ²]	Final strands [-]	A final [mm ²]	Diameter [mm]
1	8	8	1200	14	2100	51,7
2	16	12	1800	21	3150	63,3
3	24	12	1800	21	3150	63,3
4	32	12	1800	21	3150	63,3
5	40	14	2100	25	3750	69,1
6	48	16	2400	28	4200	73,1
7	56	19	2850	34	5100	80,6
8	6	8	1200	14	2100	51,7
9	13	12	1800	21	3150	63,3
Backing stay cable	20	546	81900	546	81900	322,9

Table 7.18: Final dimensions of cables for FRP bridge

The final maximum deflection in the main span was found to be 176.8 mm, and in the side span 56.7 mm, which both satisfy the deflection limit. The stresses in all elements have been checked again and were found to be acceptable too. The concrete cross girder was checked with the Sofistik module (see figures 7.17 and 7.18). All other checks are summarized in appendix D.3. A detailed printout of this final calculation can be found in appendix D.4.

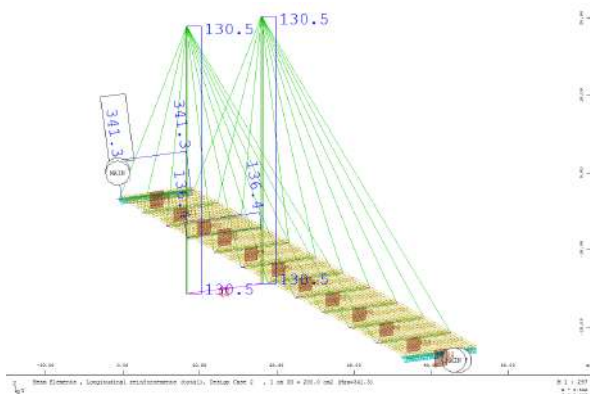


Figure 7.17: Required longitudinal reinforcement in pylon and connector in cm^2 , after design improvements

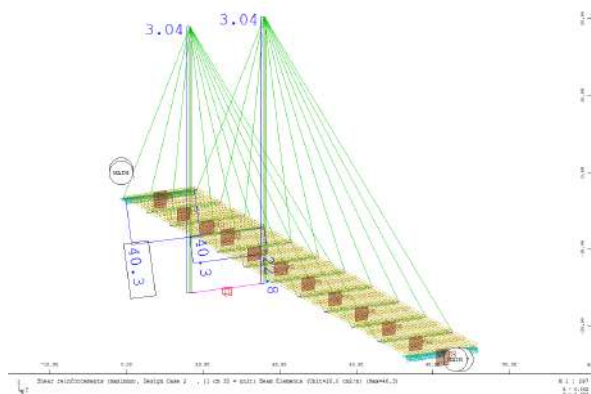


Figure 7.18: Required shear reinforcement in pylon and connector in cm^2 , after design improvements

Last but not least, a check on local buckling of the lower flange of the deck was also performed. The highest compressive stress from the final Sofistik model was taken, spread over the width between two longitudinal webs (a load of 988.8 kN/m over a width of 500 mm). The resulting critical buckling stress is equal to 68.2 MPa, see also figure 7.19. To obtain this critical stress, several plate lengths have been tested, with the length for which the stress is critical equal to 1400 mm. This finally resulted in a unity check of 0.59 ($= \frac{68.2}{114.6}$).

7.2. Concept 2: Steel-concrete composite cable-stayed bridge

The steel concrete composite cable-stayed bridge will be considered the same way as the FRP bridge. Considering several aspects are the same, only important differences in the geometry, model and loads will be treated.

7.2.1. Geometry and Sofistik model

The global geometry of this bridge is given in figure 7.20. An additional cable was added in the side span to reduce stresses and deflections. It was given the same dimensions as the first cable in the main span.

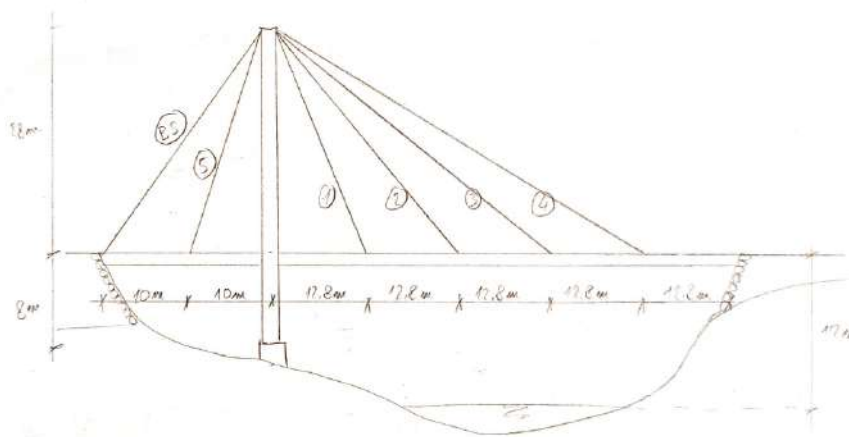


Figure 7.20: Global geometry of steel-concrete composite bridge

The steel concrete composite deck is illustrated in figure 6.14. The Sofistik cross section of the girder is given in figure 7.21. The concrete deck lies on two of these girders and has a thickness of 350 mm. Cross girders have been placed every 4.23 meters (5 meters in side span). These span between the two longitudinal steel girders, and are simple I beams (more information in table 7.19). At the cable anchor points, the cross girders extend outwards, to allow a proper connection with the cables, as illustrated in figure 7.24. Due to the high (vertical and horizontal) forces introduced by the cables, these cross girders have rectangular hollow cross sections. The actual cable-cross girder connection is modelled as a rigid link/element in Sofistik. Finally, the backing stay cable introduces very large forces (high prestressing forces+live loads from main span) into the deck at the side span. In order to avoid loosening of the backing stay cable and to meet ULS requirements, the cross girder at the side span support is a large concrete beam. The dimensions are given in table 7.19.

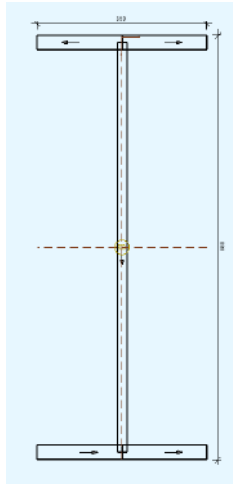


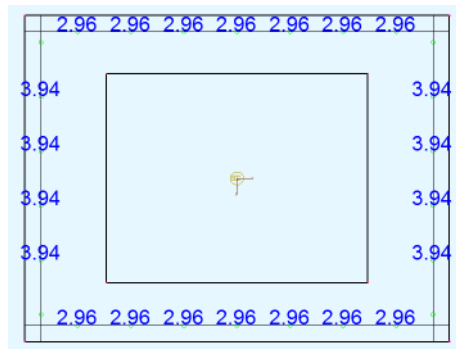
Figure 7.21: Cross-section of the steel girder

Deck	Dimensions	
Length main span [m]	64	
Length side span [m]	20	
Height concrete slab [mm]	350	
Height steel girder [mm]	850	
Width deck [mm]	10000	
etc distance girders [mm]	7000	
Minimum longitudinal reinforcement [mm ²]	10764	For entire slab width
Top and bottom flange thickness[mm]	40	
Web thickness [mm]	20	
Top and bottom flange width[mm]	350	
Cross girders main	IPE 300	
Cross girders side span support	Concrete beam 1500*1500mm	
Cross girders at cable anchors	Rectangular hollow section, 850*700mm	

Table 7.19: Composite deck dimensions

In Sofistik the deck system can be considered as a hybrid system. The slab is modelled as a plate element, with a certain thickness. It is placed on the longitudinal (and cross-) girders, which are modelled as beam elements. It is assumed that the slab and girders have full interaction.

The dimensions of the pylon have been given previously. The minimum reinforcement (in cm^2) has also been defined in the Sofistik, as illustrated in figure 7.22. The actual required amount of reinforcement will be computed by Sofistik. The two pylons are also connected by a transversal connector, which is a concrete beam of 1100*1100mm. Finally, the dimensions of the cables are given in table 7.20. It is important to mention that, similarly to the FRP bridge, the dimensions of the backing stay cable have been increased. This was done to make sure the horizontal deflections of the pylon are limited. By doing so, the cables in the main span loosen less, therefore lose less stiffness and hence the forces in the deck system are reduced.

Figure 7.22: Minimum required longitudinal reinforcement in pylon, in cm^2

Cables	Distance [m]	Angle [degrees]	Cable length [m]	A req [mm ²]	Strands req [-]	Actual strands [-]	A actual [mm ²]	Diameter [mm]	
1	12,8	65,43	30,79	2172	15	16	2400,00	55,28	4*4 strands
2	25,6	47,56	37,94	2676	18	19	2850,00	60,24	1*19 strands
3	38,4	38,10	47,52	3352	23	24	3600,00	67,70	2*12 strands
4	51,2	28,67	58,36	4117	28	28	4200,00	73,13	3*7 strands
5	10	70,35	29,73			16	2400,00	55,28	4*4 strands
Backing stay cable	20	54,46	34,41			546	81900,00	322,92	6*91 strands

Table 7.20: Cable dimensions for composite cable-stayed bridge

The entire Sofistik model is shown in figures 7.23 and 7.24. The deck supports are the same as for the FRP model (Roller at side span edge, pinned at pylon, and roller at main span edge). The pylons are rigidly supported. The cables are modelled as single cable elements and the backing stay cables are self-anchored. The other cables are connected through a rigid element to the deck.

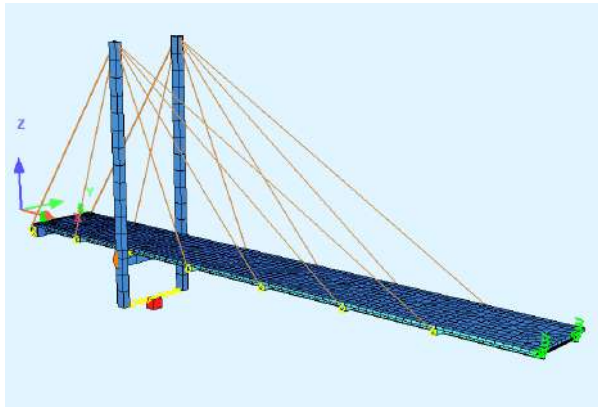


Figure 7.23: Sofistik composite bridge model



Figure 7.24: Sofistik composite bridge model, bottom view

7.2.2. Load cases and combinations

The load cases are identical to the ones for the FRP bridge. For reference, see figure 7.4.

Dead load and additional dead load

The dead load due to self-weight is automatically computed by Sofistik. The additional dead load consists of an asphalt layer with a thickness of 150 mm, as well as 4 guard railings. See figure 7.21.

Non-structural	Width [m]	L [m]	A [m ²]	t [m]	q [kN/m ²]	q [kN/m]
Asphalt		10	84	840	0,15	3,75
Railings		0,5	336	-	-	0,6

Table 7.21: Additional dead load as defined in Sofistik

Cable prestressing

As previously explained, the cables need prestressing to meet the dead load condition in order to obtain an optimal bending moment distribution in the deck due to dead load. This calculation was done with Sofistik. The resulting prestressing forces are given in table 7.22.

Cable	Prestressing force [kN]
1	1308
2	1397
3	1750
4	2278
5	1061
Backing stay cable	15462

Table 7.22: Cable prestressing forces

Wind loads

The wind loads have been computed in a similar fashion as with the FRP model. Further information can be found in appendix D.5. The wind loads are summarized in tables 7.23 and 7.24. The intermediate combinations are the same as for the FRP bridge (see table 7.9).

	Reference height [m]	Wind load			
		x-direction	y-direction	z-direction	
Girder	12,435	1,00	0,40	0,40	kN/m ²
Pylon	12	1,89	1,20	-	kN/m
	24	2,40	1,52	-	kN/m
	36	2,71	1,72	-	kN/m
Transverse	10,9	-	1,19	0,93	kN/m

Table 7.23: Wind loads on composite bridge

Cables		1	2	3	4	5	BS cable
Reference area	[m ² /m]	0,060	0,062	0,073	0,077	0,060	0,323
Force Fw bottom cables	[kN/m]	0,047	0,049	0,057	0,060	0,047	0,254
Force Fw top cables	[kN/m]	0,068	0,070	0,083	0,087	0,068	0,365

Table 7.24: Wind loads on composite bridge cables

Traffic loads

The traffic loads have been computed in exactly the same manner as for the FRP bridge. Reference is made to section 7.1.2.

Design load combinations

Same definition as for the FRP bridge. Reference is made to section 7.1.2.

7.2.3. Results of FEM calculation

A static calculation of the structure has been performed in Sofistik. All exact results are available in appendix D.7, in an extended Sofistik printout. Only the critical results are presented in the sections below. These are also summarized in appendix D.6.

Steel-concrete composite deck

The composite deck is designed in ULS with an elastic analysis. The concrete slab and the steel girders have been looked at separately.

The steel girders have been checked by the module in Sofistik. The results are given in table 7.25. All sections have been designed to satisfy the ULS requirements. The dimensions of the longitudinal steel girder are larger than the ones obtained for the analytic calculations. This is due to the combination of stresses from bending moments and normal forces in the girder. This was not taken into account in the analytic calculation, where the dimensions of the girder were only based on the maximum bending moment M_y : it may be noticed that in Mathcad this maximum bending moment in the deck (girder+slab) is 2528 kNm, and approximately 2106 kNm in Sofistik for the girder alone, showing only a minor difference, which is due to the slab. The difference in dimensions is therefore not due to an error in bending moments. At the same location, a normal force of 6550 kN was found, which explains why the dimensions of the girder had to be increased.

		N	Vy	Vz	My	Mz	Mtp	Mts	Mb	Ncr	SCL	Total
		$\sigma-x$	$\sigma+y$	τ	$\sigma-v$	$\sigma-s$	$\sigma-dyn$	As-l	As-v	crack	c/t	
Section	9	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	(3)	0.591
IPE 300		0.591	0.562	0.162	0.591	-	-	-	-	-	0.277	
Section	10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	(3) ¹	0.990
Steel girder		0.759	0.989	0.264	0.990	-	-	-	-	-	0.501	
Section	12	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	(1)	0.722
Cross girders for cabl		0.499	0.714	0.370	0.722	-	-	-	-	-	0.191	
Section	14	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	(1)	0.915
Cross girders for cabl		0.909	0.909	0.397	0.915	-	-	-	-	-	0.327	
Total		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	(3) ¹	0.990
		0.909	0.989	0.397	0.990	-	-	-	-	-	0.501	

¹ Section of class 4 has small stresses allowing to be treated as class 3 (see EN 1993-1-1 5.5.2 (9))

N	normal force	t	shear stress
Vy, Vz	shear force	$\sigma-v$	von Mises stress
My, Mz	bending	$\sigma-s$	stress in reinforcements
Mtp, Mts	torsion (p)primary and (s)secondary	$\sigma-dyn$	stress range
Mb	warping moment	As-l	longitudinal reinforcements
Ncr	flexural buckling	As-v	transverse reinforcements
SCL	cross-section class	crack	crack width
$\sigma-x$	longitud. compressive stress	c/t	stress dependant utilisation level (see AQB Manual 2.3.2)
$\sigma+y$	longitud. tensile stress	Total	most unfavorable utilisation for all checks

Table 7.25: Maximum utilisation steel girders and cross girders

The concrete slab has been checked with a simple stress check. In appendix D.7, in the figures showing the forces, the peak values have been neglected. These are singularities. The maximum forces are summarized in table 7.26, and the unity checks are all OK. The highest longitudinal forces in the slab are shown in figure 7.25.

Concrete slab	Force [kN/m]	Unity check [-]	
nxx	-5299	0.56775	At side span support
	1262	0.876388889	At side span support
nyy	-2748	0.294428571	At side span support
	950	0.659722222	At side span support
Shear	103.4	0.355186095	Slab/girder

Table 7.26: Forces in concrete slab

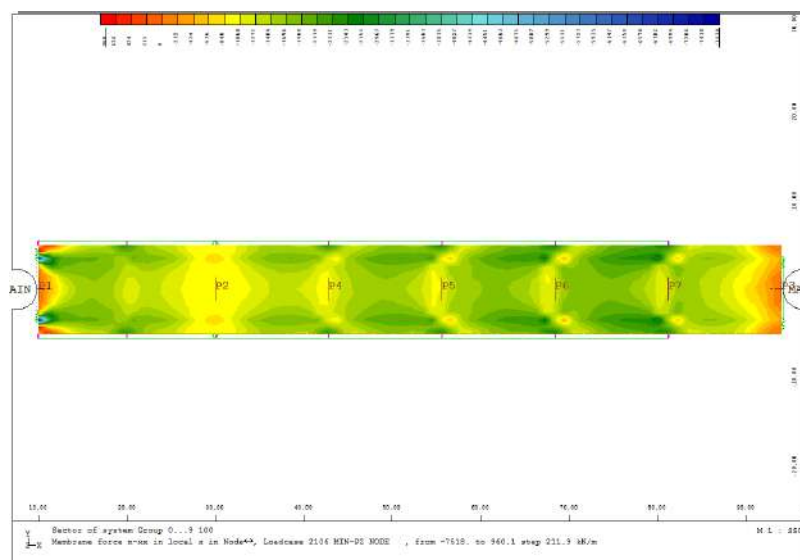


Figure 7.25: Maximum longitudinal force in slab

The cross girder for the backing stay cable is a concrete beam of 1500*1500 mm, and was checked and validated separately in SLS and ULS by the Sofistik modules. The choice for a massive concrete girder was made to avoid loosening of the backing stay cables and thus the cables in the main span. As with the FRP bridge, other solutions are possible, but not

considered in the context of this study. The required amount of longitudinal reinforcement was computed and is given in figure 7.26.

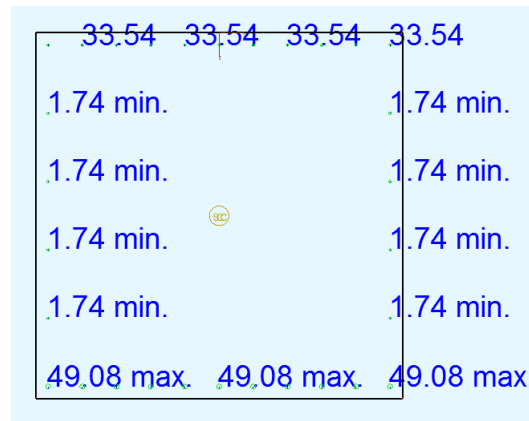


Figure 7.26: Maximum required amount of longitudinal reinforcement in cross girder for backing stay cable, in cm^2

Concerning the deflections of the deck (see figure 7.27), in the main span, the highest value is 82.2 mm. The limit value defined by Bruprosjektering [30] is $\frac{L}{350} = 182.9mm$. This results in a unity check of 0.45. The deflection in the side span is equal to 15.7 mm. The unity check is equal to 0.28. The deck is safe concerning deflections.

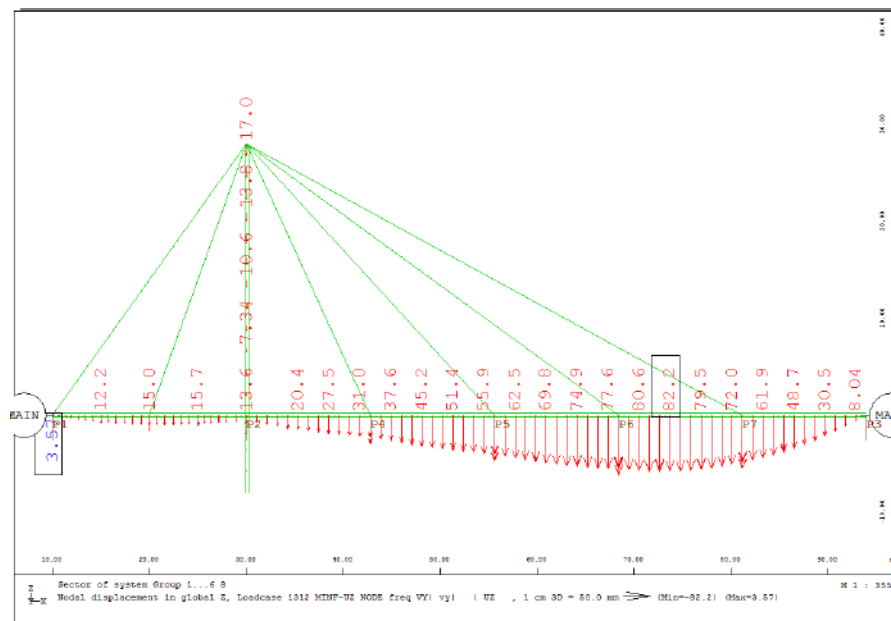


Figure 7.27: Maximum deflections steel-concrete composite deck, SLS frequent combination

Pylon and transversal connector

The pylon and the transversal connector have been designed by the Sofistik module, both in ULS and in SLS (frequent combination). The module computed the required amount of reinforcement in both elements. The results are presented in figures 7.28 and 7.29.

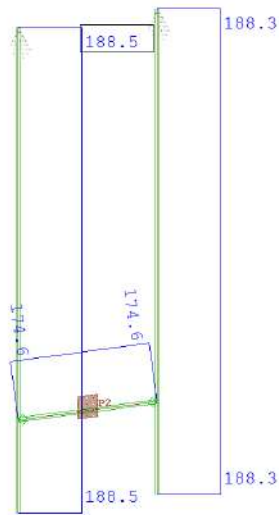


Figure 7.28: Required longitudinal reinforcement in pylon and transversal connector in cm^2

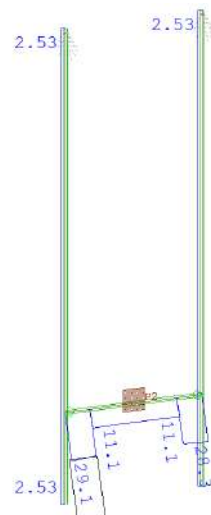


Figure 7.29: Required shear reinforcement in transversal connector, in cm^2

The maximum deflection of the pylon occurs in y-direction and is equal to 46.5 mm. In x-direction, the deflection is 13.3 mm. The maximum deflection of the connector is equal to 12.5 mm, which satisfies the limit value of $L/350 = 28.6$ mm.

Cables

The cables have been checked for maximum stresses in ULS. These are summarized in table 7.30. The tensile design stress is 837 MPa.

Cable	max N [kN]	A actual [mm ²]	max stress [Mpa]	UC stress max
1	1405	2400	585	0,70
2	1770	2850	621	0,74
3	2164	3600	601	0,72
4	2387	4200	568	0,68
5	899,7	2400	375	0,45
Backing stay cable	9138	81900	112	0,13

Figure 7.30: Stresses in cables in ULS, including unity checks

7.3. Short discussion

Two preliminary designs have been obtained from the Sofistik calculations. Both present differences with the designs from the analytic calculations from chapter 6 as some dimensions had to be increased. The reasons for these differences can be summarized with the following two points:

- The horizontal deflection from the pylon reduces the stiffness from the cables in the main span. Axial compression of the deck also has some influence. These effects increase the deflections in the main span, as well as the forces. To reduce these deflections from the pylon, the dimensions of the pylon and the backing stay cable had to be increased. As a consequence of the large forces from the backing stay cable, the corresponding cross girder was given very large dimensions (for both bridges a 1500*1500 mm concrete beam).
- The local deflections in transverse direction, which had an effect on the dimensions of the FRP bridge. The analytic calculations were based on a 2D system, where these transverse deflections were not considered.

Finally the production costs have once more been computed based on the new dimensions, see tables 7.27 and 7.28. The unit prices given in the previous section have been used. The costs of the bridges have increased due to the necessary improvements (addition of cross girders, increased dimensions of cables etc). The price difference between the bridges has not changed however: it is still approximately 0.6-0.7 million euros, which is for 2/3 due to the deck (includes cross girders), and 1/3 due to the cables. The costs per m^2 , given below, are based on the total bridge costs.

- FRP bridge: 3157 euros/ m^2
- Steel-concrete composite bridge: 2402 euros/ m^2

FRP bridge	Weight [kg]	Volume [m ³]	Total price [euros]
FRP deck	78695		€ 944.340,00
Cross girders	6078		€ 72.936,00
Cross girder BS cable		22,5	€ 6.750,00
Cables	106001		€ 1.590.015,00
Pylon+connector		54	€ 37.800,00
		Total	€ 2.651.841,00

Table 7.27: Costs of preliminary FRP bridge design

Steel concrete bridge	Weight [kg]	Volume [m ³]	Total price [euros]
Steel girders	59346		€ 296.730,00
Cross girders	52810		€ 264.050,00
Cross girder BS cable		22,5	€ 6.750,00
Concrete slab		294	€ 88.200,00
Cables	87515		€ 1.312.725,00
Pylon+connector		70,046	€ 49.032,20
		Total	€ 2.017.487,20

Table 7.28: Costs of preliminary steel concrete composite bridge design

The weights per m^2 , given below, indicate that the steel concrete bridge is 75% heavier. In this calculation, the deck, cables and cross girders have been included.

- FRP bridge: 294 kg/ m^2
- Steel-concrete composite bridge: 1180 kg/ m^2

The weights per meter length and the corresponding EI are as indicated below, if only the decks are considered (no cables, no cross girders). These parameters can give an indication on the efficiency of the system, especially if compared with the costs. The results for the FRP deck are similar to the ones from section 6.3, except for the increase in mass due to

the addition of transverse stiffeners. The results for the steel-concrete composite deck are different because the dimensions of the steel girder have been increased to account for the increased bending moments from cable pretension loss in the main span. Therefore the stiffness from the steel-concrete deck rises; the mass too, but only slightly considering most of it comes from the concrete slab. The final preliminary designs therefore show that the steel-concrete composite deck is slightly more efficient, if the ratio $\frac{EI}{m}$ is considered. It also clearly highlights why stiffness is so critical in the design of an FRP cable-stayed bridge.

- FRP deck:

- $EI = 6.298 * 10^5 \text{ kNm}^2$

- $m = 936.85 \frac{\text{kg}}{\text{m}} (9.187 \frac{\text{kN}}{\text{m}})$

- $\frac{EI}{m} = 6.855 * 10^4 \text{ m}^3$

- Steel-concrete composite deck:

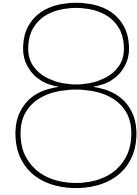
- $EI = 8.595 * 10^6 \text{ kNm}^2$

- $m = 9450.20 \frac{\text{kg}}{\text{m}} (92.675 \frac{\text{kN}}{\text{m}})$

- $\frac{EI}{m} = 9.275 * 10^4 \text{ m}^3$

III

Dynamic wind analysis



Wind models and parameters

Before performing a dynamic wind analysis, several parameters need to be defined first. These are summarized as follows:

- The wind model needs to be defined, in order to have representative wind loads on the structure.
- Damping of the structure needs to be defined. In Sofistik, this is done with Rayleigh damping. This requires a modal analysis of the structure to find the frequency range within which the most important natural frequencies are damped.
- The aerodynamic wind coefficients of the deck have to be determined with the use of CFD, which can be done with a module in Sofistik.

Once these parameters are defined in Sofistik, the bridge can be analyzed. The dynamic response has to be assessed and based on the results, improvements can be made to ensure structural safety. The focus lies on the dynamic response of the decks.

8.1. Sofistik bridge models for wind analysis

In this section, the Sofistik FEM models that is used for the wind analysis is highlighted. A few arrangements and simplifications had to be made, compared to the models from the preliminary design.

The Sofistik FEM models from the preliminary design had decks which consisted of a combination of plate and beam elements. This allowed to design the bridges in more detail, and highlighted a few issues, mainly the local introduction of cable forces in the decks. This latter issue was very critical for the FRP bridge. However, in order to run a wind analysis, high computational capacity is required. By reducing the decks to single beam elements, the computation time is highly reduced. Also, Sofistik only allows the definition of wind coefficients for beam elements. Some properties of both decks, modelled as beam elements, are given below in table 8.1. These have automatically been computed by Sofistik.

Bridge deck beam element properties	FRP bridge	Steel-concrete composite bridge
Sectional area A [m ²]	6,25E-01	7,02E-01
Bending moment of inertia I _y [m ⁴]	2,44E-02	4,09E-02
Bending moment of inertia I _z [m ⁴]	5,46E+00	6,20E+00
Torsional moment of inertia I _t [m ⁴]	8,62E-02	2,60E-02

Table 8.1: Properties of FRP and steel-concrete decks, modelled as beam elements in Sofistik

For both bridges, the transverse elements, such as cross girders or transverse stiffeners,

have not been modelled due to the simplification of the bridge models. The additional loads of these elements were added to the models. The cable-deck connection still consists of a rigid element, only now it is connected directly to the deck, not to the cross girders.

FRP cable-stayed bridge

After running the wind analysis on the FRP bridge, it seemed that the pylon was not stiff enough: with lateral wind, the excessive deformations of the pylon also caused excessive deformations in the deck. It dominated the response of the deck. However, as a check on the accuracy of the Sofistik model, the first natural frequencies for torsion and bending were computed both by hand (same method as in appendix C.3 and C.6) and with the software (see figures 8.1 and 8.2). The results are presented below. For bending, the natural frequencies are almost equal. For torsion, they are slightly different. The minor differences probably lie in the strong influence of the pylon in the Sofistik model, which can not properly be accounted for in the hand calculations. This is especially true for the natural frequency for torsion where the Sofistik model reveals a large influence of the pylons. Nevertheless, the bridge can hereby be considered as accurately modelled in Sofistik.

- First bending natural frequency:
 - $f_b = 2.858$ Hz from Sofistik
 - $f_b = 2.823$ Hz from hand calculations
- First torsional natural frequency:
 - $f_t = 5.309$ Hz from Sofistik
 - $f_t = 5.920$ Hz from hand calculations

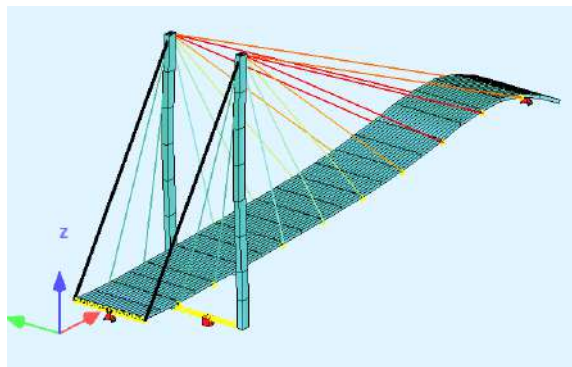


Figure 8.1: Vibrational mode 5 of FRP bridge ; $f_n = 2.858$ Hz

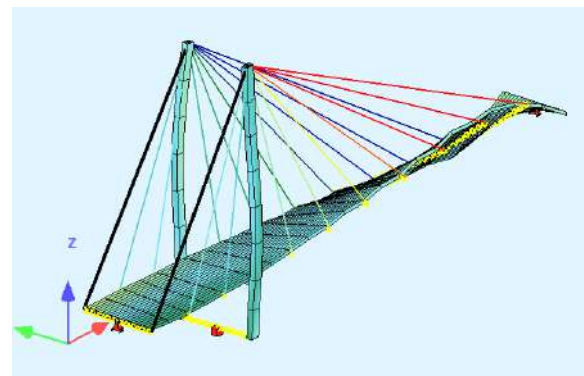


Figure 8.2: Vibrational mode 11 of FRP bridge ; $f_n = 5.309$ Hz

In order to keep the study focused on the response of the deck only, the pylon was given a much higher stiffness. Other options were explored, such as simply placing a pinned support at the top end of the cables. However this resulted in completely different natural frequencies of the structure, which would not come close to the original ones. The effect of the backing stay cable would also be neglected: it would be pinned on both ends.

The pylon is still from concrete, with dimensions of 3.0*2.4m. The model that is used for the further wind analysis is illustrated in figure 8.3.

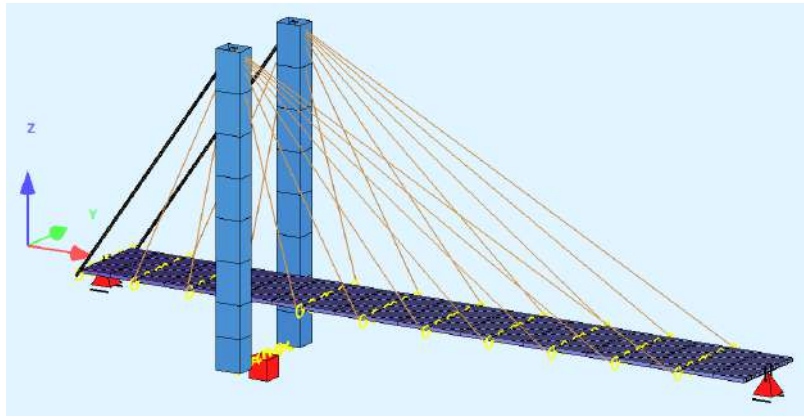
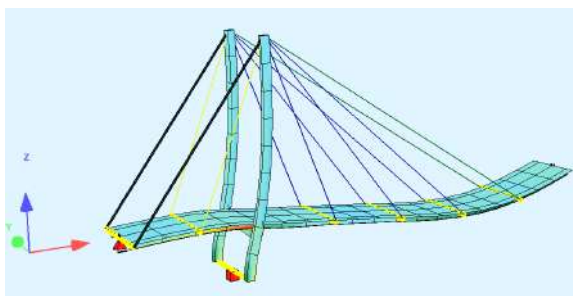
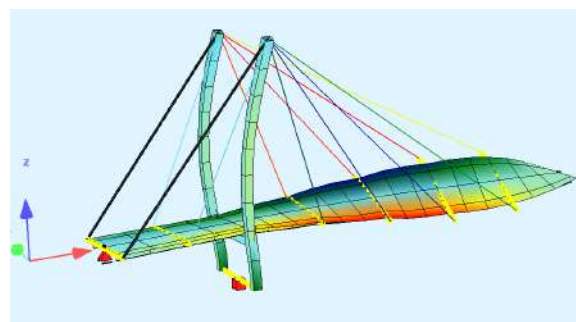


Figure 8.3: Sofistik model of FRP cable-stayed bridge, for wind analysis

Steel-concrete cable-stayed bridge

A first wind analysis was also run for the steel-concrete cable-stayed bridge. The lateral deflections of the pylon, which were a concern previously, are still an issue though more limited. The dimensions of these pylons are already larger than for the FRP bridge, and the high weight of the deck limits the influence of the lateral deformations of the pylon on the deck. Nevertheless, in order to keep the comparison between the 2 bridges focused on the deck, it was chosen to model the pylons for the steel-concrete cable-stayed bridge with the same dimensions as for the FRP bridge. Before presenting the final model, a quick comparison of the natural frequencies computed by Sofistik and by hand are shown, for the original model. The results, given below, are quite similar (see also figures 8.4 and 8.5). The differences can be explained by the same reasons as the FRP bridge.

- First bending natural frequency:
 - $f_b = 1.096$ Hz from Sofistik
 - $f_b = 1.144$ Hz from hand calculations
- First torsional natural frequency:
 - $f_t = 2.201$ Hz from Sofistik
 - $f_t = 2.364$ Hz from hand calculations

Figure 8.4: Vibrational mode 3 of steel-concrete composite bridge ; $f_n = 1.096$ HzFigure 8.5: Vibrational mode 6 of steel-concrete composite bridge ; $f_n = 2.201$ Hz

The final model for the wind analysis of the steel-concrete composite bridge is shown in figure 8.6. The pylons have the same dimensions as the FRP bridge (3.0*2.4 m).

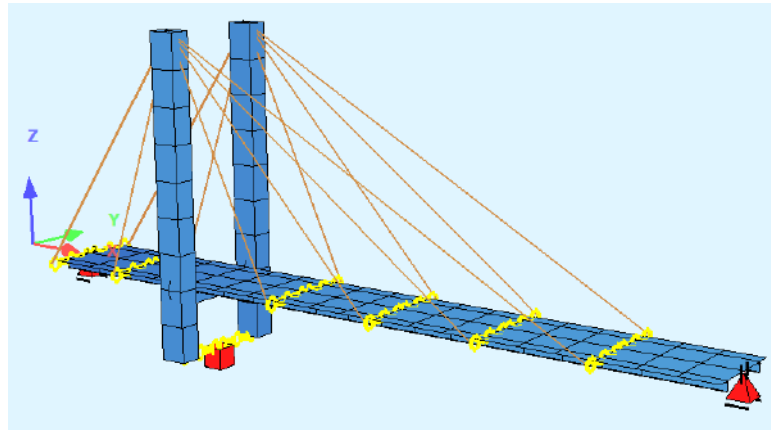


Figure 8.6: Sofistik model of steel-concrete composite bridge, for wind analysis

8.2. Wind load model

In order to perform a transient wind analysis, a wind velocity time history is required. According to Eurocode, a time frame of 10 minutes is sufficient. Sofistik has included the automatic generation of such a time history within its module Sofiloat. The two main wind velocity components have to be properly defined in order to run the module: the mean wind speed and the turbulent wind speed. The wind definition as given in the paragraphs below is the same for both the FRP and the steel-concrete composite bridge.

The mean wind speed is defined through a wind speed profile along the height. In Sofistik, this can be done by defining the Norwegian NS of the EN1991-1-4. Based on the design code recommendations, the software generates a wind profile that suits the input parameters (wind direction, ground height, roughness class). To obtain a dynamic wind generation, a power spectrum of the turbulence components has to be defined. The Karman spectrum has been used in this situation, which is recommended by Sofistik as it best describes reality in common practice. It includes by default appropriate parameter values for the coherence. With this spectrum and the input parameters from NS:EN1991-1-4, a random time history of the wind velocity can be obtained, in lateral, longitudinal and vertical directions. The main wind parameters are given in table 8.2 and the Karman spectrum is illustrated in figure 8.7. On the abscissa, X represents the reduced frequency: $X = \frac{fL}{U}$, where L is the effective wave length, and U the wind speed. When defining this spectrum in Sofistik, it is important to properly define the maximum frequency. It should be such that it covers the most important frequencies of the structure, in this case the 1st order vertical bending and torsion natural frequencies. The highest natural frequency of importance of the bridges are 3.227 Hz (FRP bridge) and 2.793 Hz (steel-concrete bridge). The maximum frequency was therefore set at 3.3 Hz.

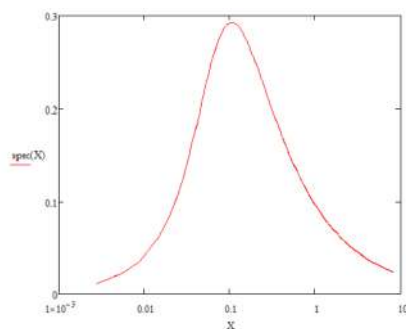


Figure 8.7: Von Karman spectrum as defined in Sofistik for both cable-stayed bridges

Wind parameters from Sofistik	According to NS:EN 1991-1-4	
Terrain category	III	
Atmospheric wind velocity	59.88 [m/s]	
Mean wind speed	24 [m/s]	(c_{dir} , c_{season} , c_{alt} , c_{prob} included)
Deck reference height	12.385 [m]	
Roughness z_0	0.3 [m]	
Turbulence wind spectrum	Karman	(in 3 directions)

Table 8.2: Wind parameters as defined in Sofistik

The main wind direction has to be defined as well. It has been defined as perpendicular to the longitudinal bridge direction for both considered bridges. The bridge is located in a valley, which leads to the main wind direction being perpendicular to the deck. The corresponding wind speeds, turbulences and effective wave lengths are given in figure 8.8.

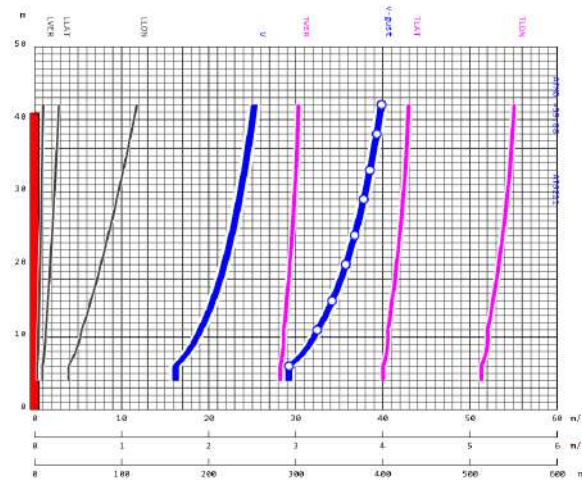


Figure 8.8: Wind speeds, turbulence and effective wave lengths as defined in Sofistik

Considering the generation of such a wind velocity time history is random, it will be different every time it is generated again, for every node in the structure. The wind speeds between the nodes are linked by the coherence functions. An example of such a time history for a point in the FRP deck is given in figures 8.9, 8.10 and 8.11. It may be noticed that the mean wind speed in lateral direction is equal to 19.20 m/s. This is almost equal to the mean wind speed computed for the static wind load in chapter 7 (mean wind speed deck: 19.23 m/s). The mean wind speed in vertical and longitudinal directions is equal to zero because of the wind direction: the wind blows laterally with a reference wind speed of 24 m/s, which means that in the other directions only turbulent wind will have an effect.

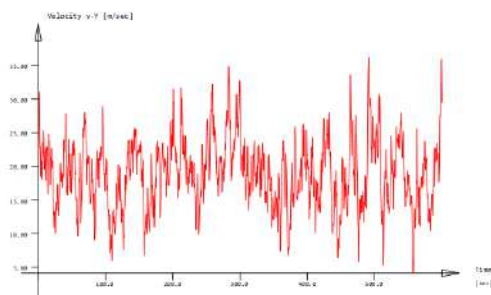


Figure 8.9: Wind velocity time history for a node in the FRP deck, in lateral direction



Figure 8.10: Wind velocity time history for a node in the FRP deck, in longitudinal direction

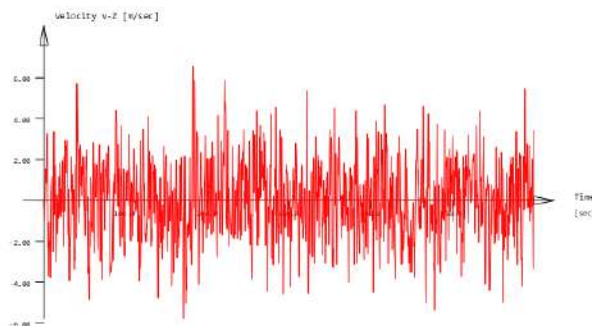


Figure 8.11: Wind velocity time history for a node in the FRP deck, in vertical direction

8.3. Modal analysis

A modal analysis is performed to identify the important natural frequencies. This will also allow the definition of a frequency range to define Rayleigh damping. For both bridges, the deck is modelled as a beam element in Sofistik. Elements such as the cross girders or the transverse stiffeners have been modelled as additional masses.

FRP cable-stayed bridge

The results for the FRP cable-stayed bridge are presented below in table 8.3, for the 7 first modes. The figures corresponding to the modes can be found in appendix E.1.

Mode nr	fn [Hz]	Description
1	1,079	Pylons symmetric lateral bending
2	1,131	Pylons symmetric longitudinal bending
3	1,31	Pylons anti-symmetric lateral bending
4	2,88	Deck 1st order vertical bending
5	3,227	Deck 1st order torsion, pylons anti-symmetric longitudinal bending
6	3,464	Deck lateral bending
7	4,558	Deck 2nd order vertical bending

Table 8.3: Natural frequencies of FRP bridge

The frequencies that are of most interest are the first order vertical bending frequency and the first order torsional frequency of the deck. These can give an indication on the sensitivity to dynamic wind excitation of the bridge. In this case, these correspond to the modes 4 and 5 (resp $f_n = 2.88$ Hz and $f_n = 3.227$ Hz) and are indicated in figures 8.13 and 8.12. The previously given requirement of $\epsilon = \frac{f_t}{f_b} > 2$, which indicates safety against flutter, is not met.

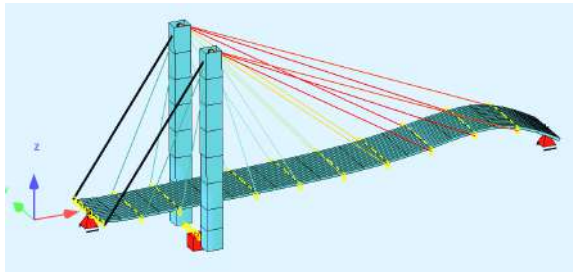


Figure 8.12: Vibrational mode 4 of FRP bridge ; $f_n = 2.880$ Hz

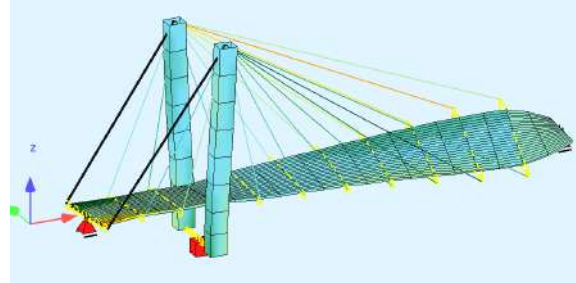


Figure 8.13: Vibrational mode 5 of FRP bridge ; $f_n = 3.227$ Hz

Finally, in appendix E.1, the modal mass participations and eigenvalues of the 100 first modes are given. The frequency range for Rayleigh damping is taken as starting at the first important natural frequency, being the 1st vertical bending mode of the deck ($f_n = 2.880$ Hz). The last value of the range is taken as the natural frequency for the 2nd order torsion mode of the deck: $f_n = 8.174$ Hz (mode 16). This properly applies damping to the most important natural frequencies of the deck.

Rayleigh damping (or modal damping) depends on 2 important terms, one proportional to the mass (the α term) and one proportional to the stiffness (the β term). These terms can be computed with the following formulas:

$$\alpha = \xi * 4 * \pi * \frac{f_1 * f_2}{f_1 + f_2} \quad (8.1)$$

$$\beta = \xi * \frac{1}{\pi * (f_1 + f_2)} \quad (8.2)$$

where: ξ corresponds to the required modal damping; f_1 and f_2 to the frequency range.

The required modal damping values for the FRP bridge are as follows:

- FRP composite: 1% damping as defined by CUR96 [26]
- Reinforced concrete: 2% damping as defined by the Sofistik manual
- Steel cables: 0.3% damping as defined by [23]

This results in the following Rayleigh damping values:

- FRP composite: $\alpha = 0.267569$ and $\beta = 0.000288$
- Reinforced concrete: $\alpha = 0.535138$ and $\beta = 0.000576$
- Steel cables: $\alpha = 0.080271$ and $\beta = 0.000086$

Steel-concrete cable-stayed bridge

The results for the steel-concrete cable-stayed bridge are presented in table 8.4. The figures corresponding to the modes can be found in appendix E.2.

Mode nr	fn [Hz]	Description
1	1,101	Pylons symmetric lateral bending
2	1,187	Deck 1st order vertical bending
3	1,323	Pylons anti-symmetric lateral bending
4	1,835	Deck 2nd order vertical bending, pylons symmetric longitudinal bending
5	2,051	Deck 2nd order vertical bending
6	2,793	Deck 1st order torsion

Table 8.4: Natural frequencies of steel concrete composite bridge

The first bending natural frequency is equal to 1.187 Hz, and the first torsional natural frequency is 2.793 Hz. These are illustrated in figures 8.14 and 8.15. The flutter requirement is satisfied ($\epsilon = 2.35$).

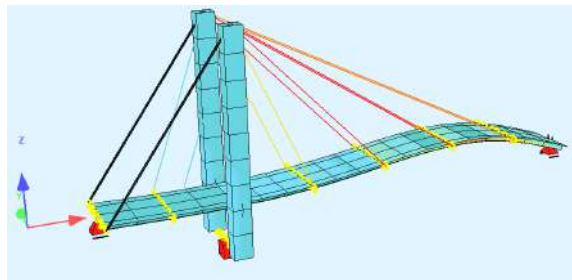


Figure 8.14: Vibrational mode 2 of steel-concrete bridge ; $f_n = 1.187\text{Hz}$

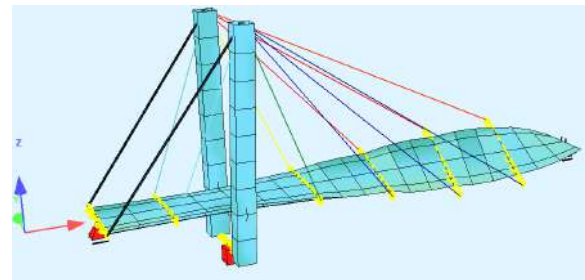


Figure 8.15: Vibrational mode 6 of steel-concrete bridge ; $f_n = 2.793\text{ Hz}$

The modal mass participation has been computed for the 60 first modes of the structure (see appendix E.2). The frequency range for Rayleigh damping is defined as starting at a frequency of 1.187 Hz. This range is defined as ending at a natural frequency corresponding to the 2nd order torsion frequency: $f_n = 3.990\text{ Hz}$ (mode 10). This frequency range covers the most important modes.

The required modal damping values for the steel-concrete composite bridge are as follows:

- Composite deck system: 1.5% damping
- Reinforced concrete (pylon): 2% damping

- Steel cables: 0.3% damping

This results in the following Rayleigh damping values:

- Composite deck system: $\alpha = 0.172411$ and $\beta = 0.000922$
- Reinforced concrete: $\alpha = 0.229881$ and $\beta = 0.001230$
- Steel cables: $\alpha = 0.034482$ and $\beta = 0.000184$

8.4. Cross-sectional wind coefficients

To perform a dynamic transient wind analysis of a bridge, the wind coefficients of the deck have to be defined. These have to be determined with the use of CFD software, or wind tunnel testing. Sofistik has a CFD module, called Dolfyn, which was used in the present study. For a range of inclination angles, the drag, lift and moment coefficients of the previously designed FRP and steel-concrete decks have been computed.

The important parameters were defined as follows:

- Air density: $\rho = 1.25 \text{ kg/m}^3$
- Wind speed: $U = 19.23 \text{ m/s}$. Taken as the mean wind velocity, computed according to EN1991-1-4. Includes reduction factors for roughness
- Turbulence model: $k - \epsilon$ model
- Initial turbulent intensity: $I_v = 0.27 \%$. Computed according to EN1991-1-4.
- Length scale: $L = 53.925 \text{ m}$. Computed according to EN1991-1-4.
- Range of inclination angles: from -10 up to $+10$ degrees.
- Size of CFD simulation box: according to the Sofistik manual, the distance of the section to the boundaries above and below should be such that the section does not block more than 1 or 2 % of the incoming flow. The length of the area before the section should be approximately 2 times the deck width, and the length of the area after the section 15 times the deck width. These requirements ensure a proper modelling of the wind, and avoid disturbances in the wind pressures acting on the deck.
- The mesh size has a large influence on the resulting wind velocities, and the pressures exerted on the deck. For both decks, a mesh sensitivity study was performed. For the FRP deck, a mesh size of 1.0 m far away from the section is defined. Close to the section, the mesh is refined to 0.1 m. For the steel-concrete deck, a mesh size of 3.0 m far away from the section is defined. Close to the section, the mesh is refined to 0.05m. The sensitivity studies are given in appendix E.3.

FRP cable-stayed bridge

The wind coefficients for the FRP deck are given in figure 8.16. The drag coefficient is almost constant across all angles, whereas the lift and moment coefficients vary linearly. The drag coefficient was computed with the deck height as reference, the lift coefficient with the deck width, and the moment coefficient with both. An example of the wind velocities around the deck, for an angle of 0 degrees, and the above parameters, is given in figure 8.17.

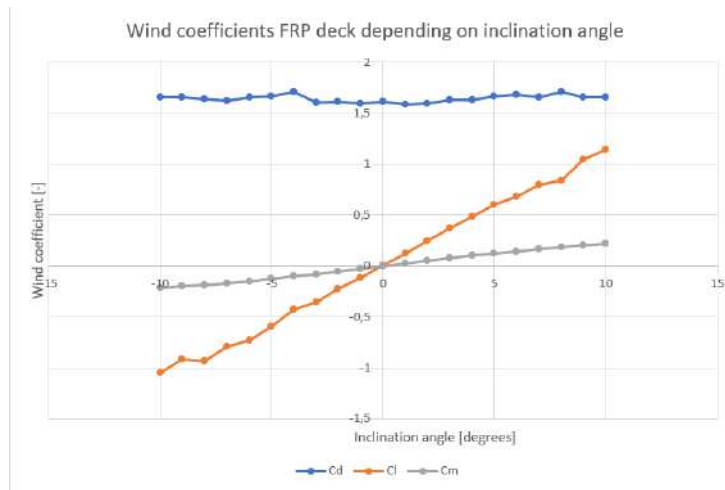


Figure 8.16: Wind coefficients for drag, lift and moments, for the FRP deck

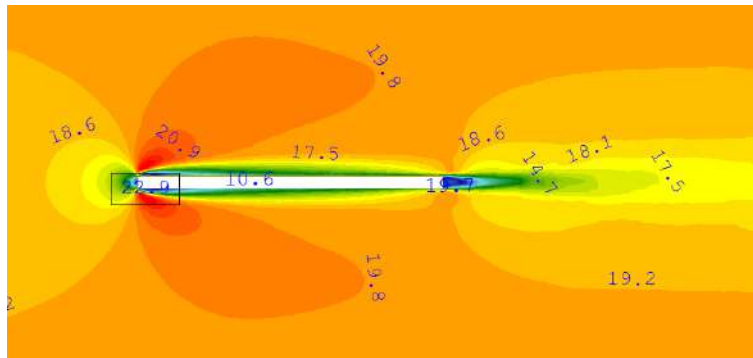


Figure 8.17: Wind velocities around FRP deck, for inclination angle of 0 degrees; units: [m/s]

A check-up comparison was made with the computation of static wind loads in EN1991-1-4. The lateral wind force per unit length acting on a bridge element is defined as follows in the Eurocode:

$$F_w = \frac{1}{2} * \rho * v_b^2 * C * A_{ref,x} \quad (8.3)$$

where:

$$C = c_e * c_{f,x}$$

ρ is the density of air, v_b is the basic wind speed, c_e is the exposure factor, $c_{f,x}$ is the force coefficient and $A_{ref,x}$ is the reference area.

For a basic wind speed of 24.0 m/s, a total force F_w of 30.7 kN is found to act laterally on the bridge. This corresponds to a distributed load of 0.366 kN/m along the length of the deck, or a distributed area load of 0.850 kN/m². In Sofistik, the obtained wind pressures acting on the deck are given in figure 8.18 (parameters as defined earlier), with an inclination angle of 0 degree. The maximum wind pressure on the incoming wind side is 0.324 kN/m, which translates to an area load of 0.75 kN/m² if we assume this maximum wind pressure would act on the entire edge.

The force coefficient $c_{f,x}$ from the Eurocode used in the above calculation is equal to 1.3, and the exposure factor is 1.817. This results in a total drag coefficient $C = 2.36$. The drag coefficient computed in Sofistik is equal to 1.61. The difference is relatively large due to the simplifications from the Eurocode. Another quick check is the pressure distribution between the wind- and leeward edge: in the Eurocode it is defined as 0.8 and 0.5 respectively. The CFD module gives an average pressure of 0.173 kN/m on the windward edge and 0.075 kN/m

on the leeward edge. In the Eurocode, a pressure on the windward edge equal to 0.173 kN/m, would correspond to 0.108 kN/m. Again the Eurocode results are slightly more conservative. With the CFD module, the wind pressure can be computed more accurately as well as the corresponding drag factor.

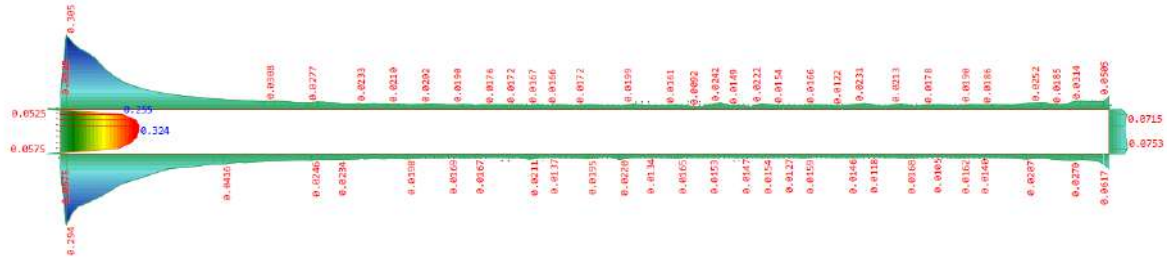


Figure 8.18: Wind pressures acting on bridge deck, inclination angle = 0 degree, units: kN/m

Steel-concrete cable-stayed bridge

The wind coefficients for the steel-concrete deck are given in figure 8.19. The drag coefficient was computed with the total deck height as reference, the lift coefficient with the deck width and the moment coefficient with both. The drag coefficient becomes lesser for negative inclination angles. This can partly be explained by the fact that the steel girder on the windward edge is less exposed, which reduces the total drag. Conversely, for positive inclination angles, the drag factor increases as the same steel girder is more exposed to wind. An example of the wind velocities around the section for inclination angle 0 degrees is given in figure 8.20.

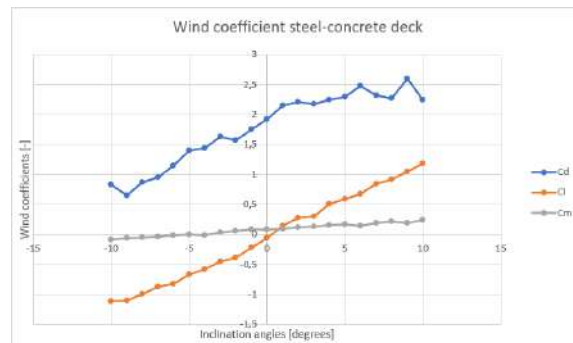


Figure 8.19: Wind coefficients for drag, lift and moments, for the steel-concrete deck

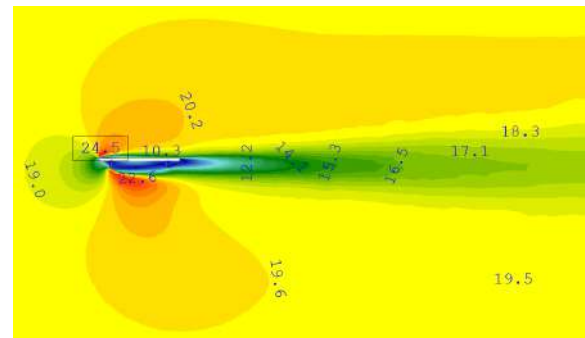


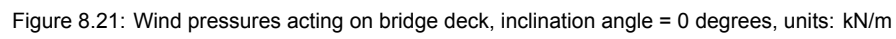
Figure 8.20: Wind velocities around steel-concrete deck, for inclination angle of 0 degrees; units: [m/s]

The drag coefficient $c_{f,x}$ from the Eurocode is equal to 1.3 for this cross-section. The corresponding exposure factor c_e is equal to 1.795. This results in a total drag coefficient $C = 2.33$. The drag coefficient in Sofistik is equal to 1.92. The difference between these values can be explained by the same reasons as with the FRP deck.

Another quick check can be performed. The Dutch National Annex of EN1991-1-4 states that the pressure on the 2nd girder (on the leeward edge) should be equal to 2/3 that of the 1st girder. With a quick look at figure 8.21, this is not the case here. However, this can partly be explained by the Reynolds number, which in this case is almost equal to $14E+6$:

$$Re = \frac{vL}{\nu} = \frac{19.23 * 10}{1.373 * 10^{-5}} = 13.98 * 10^6 \quad (8.4)$$

If figure 5.4 is considered, the situation where $Re > 200000$ is present here. The fluid flows past the body without reattaching. This explains why the 2nd girder is barely affected by the wind.



8.5. Sensitivity analysis for time step integration

In order to obtain accurate structural responses from the bridge, it is important to properly define the increment size for the time step integration. This has been done for both the FRP bridge and the steel-concrete composite bridge. The increment size was varied from 0.5 seconds to 0.01 seconds (0.5, 0.2, 0.1, 0.05, 0.02 and 0.01 seconds). The bridge models consisted of the deck only, supported by springs instead of cables. The deck was loaded during 10 minutes (600 seconds) by a random wind loading as defined in section 8.2. The deflections at a critical location in the deck were measured for every run with a different increment size. The complete results are presented in appendix E.4, and are summarized below:

- For the FRP bridge, the results become stable from an increment size of 0.05 sec on. The peaks are slightly higher for the step size 0.05 sec than for the 0.02 sec step size. The results for step sizes 0.02 and 0.01 sec are identical. Step size 0.02 sec is chosen (see figure 8.22), being as accurate as 0.01 sec with reduced computation time.
- Concerning the steel-concrete composite bridge, the results become similar from an increment size of 0.05 sec on as well. The results are identical for step sizes 0.05 sec and 0.02 sec. It was therefore chosen to further compute with step size 0.05 sec (see figure 8.23).

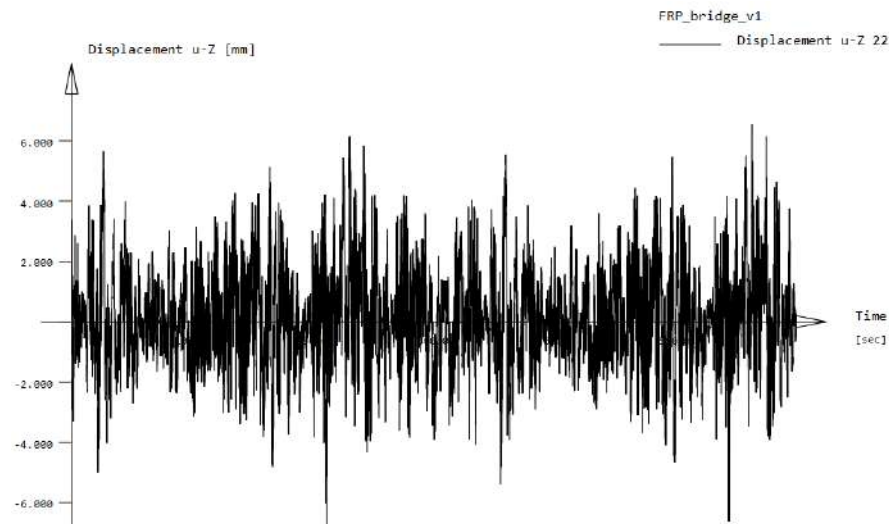


Figure 8.22: Deflection in FRP deck due to 10 min wind load, with a step size of 0.02 sec



Figure 8.23: Deflection in steel concrete composite deck due to 10 min wind load, with a step size of 0.05 sec

Wind analysis: results and improvements

In this chapter, the results from the dynamic wind analyses will be presented and discussed.

9.1. FRP cable-stayed bridge

The dynamic wind analysis was carried out for a linear elastic system, which most importantly means that the cables behave as linear elastic springs (no cable sagging). The Sofistik results printout is available in appendix E.5.

A check on the stresses in the deck has been performed, in ULS. All live loads have been included: traffic and dynamic wind loads. The unity checks were satisfactory and are given in table 9.1. The maximum dynamic wind forces were included in the calculation: for every node, the maximum wind force from the time history is taken. This is illustrated in figure 9.1, where the maximum normal forces from the transient wind analysis are given. The utilisation levels are lower than previously computed in chapter 7. This is because the local introduction of forces in the deck is not taken into account. The deck is considered as one beam element.

Maximum Utilisation Level

	N	Vy	Vz	My	Mz	Mtp	Mts	Mb	Ncr	SCL	Total
	$\sigma-x$	$\sigma+x$	τ	$\sigma-v$	$\sigma-s$	$\sigma-dyn$	As-l	As-v	crack	c/t	
Section 5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	-	0.380
Deck main span	0.298	0.282	0.380	0.000	-	-	-	-	-	-	
Section 12	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	-	0.372
Deck side span	0.158	0.144	0.372	0.000	-	-	-	-	-	-	
Total	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	-	0.380
	0.298	0.282	0.380	0.000	-	-	-	-	-	-	
N	normal force		τ	shear stress							
Vy,Vz	shear force		$\sigma-v$	von Mises stress							
My,Mz	bending		$\sigma-s$	stress in reinforcements							
Mtp,Mts	torsion (p)primary and (s)secondary		$\sigma-dyn$	stress range							
Mb	warping moment		As-l	longitudinal reinforcements							
Ncr	flexural buckling		As-v	transverse reinforcements							
SCL	cross-section class		crack	crack width							
$\sigma-x$	longitud. compressive stress		c/t	stress dependant utilisation level (see AQB Manual 2.3.2)							
$\sigma+x$	longitud. tensile stress		Total	most unfavorable utilisation for all checks							

Table 9.1: Utilisation levels of stresses in ULS, in FRP deck

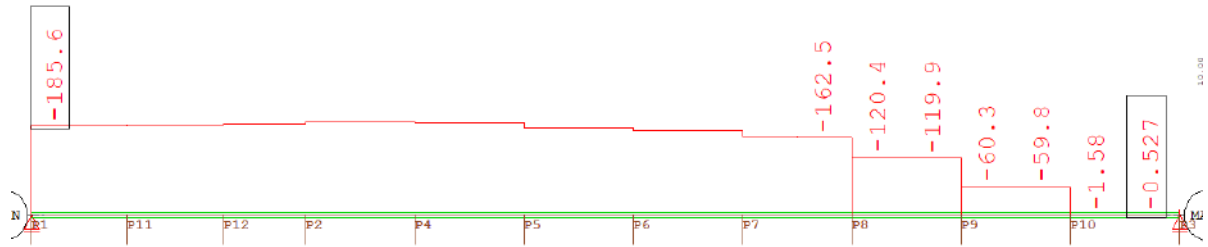


Figure 9.1: Maximum compressive normal forces in FRP deck due to wind, in ULS, in kN

The maximum vertical deflections occurring in the deck have been checked once more, in SLS frequent combination. These are given in figure 9.2. However, with this combination, the deflection from wind is not included due to the combination factors. The deflection is only due to traffic loads and permanent loads. It fulfills the limit of $L/350 = 183$ mm.

The vertical deflections from the wind analysis for a time period of 10 min, at the same critical location of figure 9.2 are given in figure 9.3. The maximum amplitude is approximately 8 mm. Flutter type vibrations (torsional divergence, classical flutter or torsional galloping) will not arise since flutter derivatives were not defined. Computing these require complex CFD computations, coupled with structural dynamic analyses, and could not be completed as part of this thesis. However, Sofistik does include effects such as aerodynamic damping or bending galloping excitation into their simulations if wind coefficients are properly defined. In this wind analysis run, these vibrations do not seem to occur. A quick check for flutter according to EN1991-1-4, annex E was performed. The criteria defining whether flutter should be checked are all met and the formula which therefore should be applied is:

$$v_{div} = \left(\frac{2k_{\theta}}{\rho_{air} d^2 \frac{dc_M}{d\theta}} \right)^{\frac{1}{2}} \quad (9.1)$$

where: k_{θ} is the torsional stiffness and $\frac{dc_M}{d\theta}$ the rate of change of the aerodynamic moment coefficient with respect to rotation about the torsional centre (in annex E from EN1991-1-4, a graph indicates how to compute this value) [9].

The resulting critical flutter velocity is 39.55 m/s. It is higher than 2 times the mean wind speed ($2 \cdot 19.20 = 38.40$ m/s), and can therefore be considered as safe.

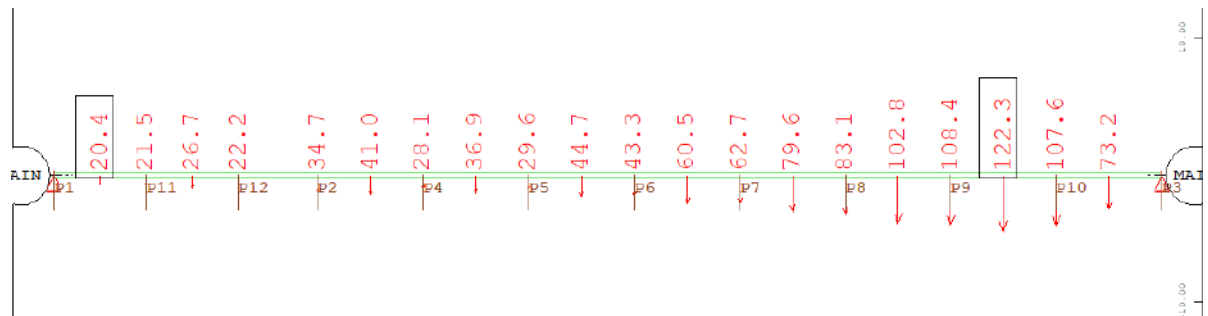


Figure 9.2: Maximum vertical deflections of FRP deck, in SLS frequent combination, in mm



Figure 9.3: Vertical deflections of FRP deck, for a time period of 10 min, in mm

The maximum lateral deflections of the deck due to dynamic wind loading are rather low, as can be observed in figure 9.4. In figure 9.5, the lateral deflections due to static wind loading are also illustrated as a comparison. The latter are slightly higher, but the conclusion is still that the deck barely deflects laterally. This is partly due to the width of the deck (10 meters), which combined with the limited span (64 meters), creates a relatively stiff lateral system.

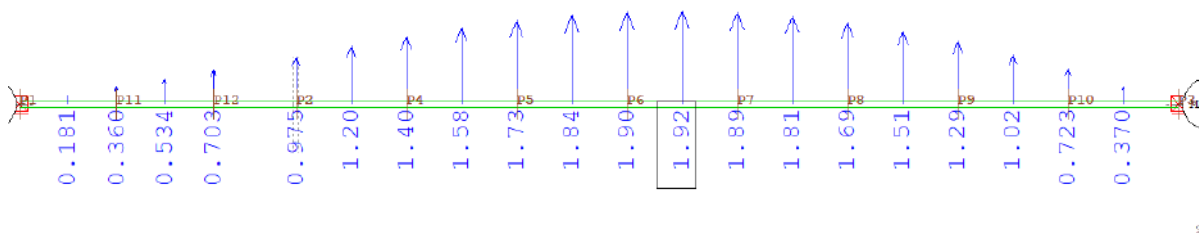


Figure 9.4: Maximum lateral deflection of FRP deck due to dynamic wind loading, in mm

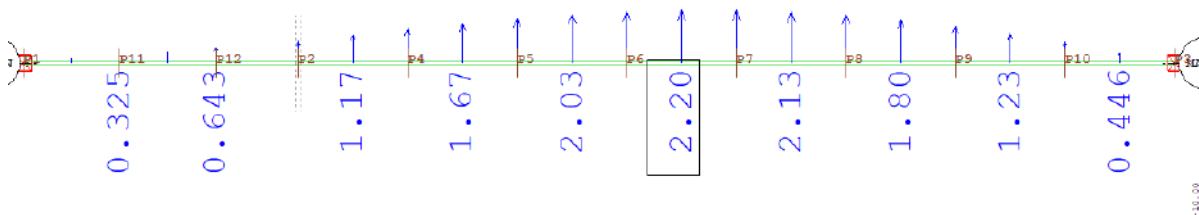


Figure 9.5: Lateral deflection of FRP deck due to static wind loading, in mm

The maximum acceleration defined in the Eurocode for pedestrian comfort is 0.7 m/s^2 for vertical vibrations. The measured vertical accelerations of the deck at the most critical location are given in figure 9.6. The maximum amplitude is 1.30 m/s^2 , which is almost double the limit defined by the Eurocode. This is discussed in section 9.3.

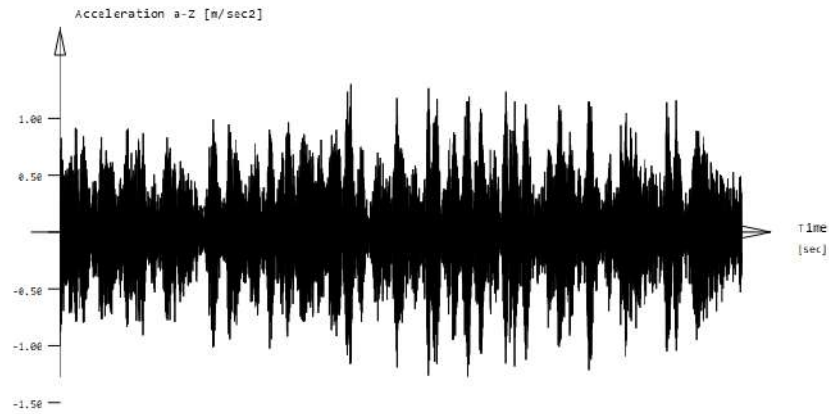


Figure 9.6: Vertical accelerations in FRP deck at critical location (at around 2/3 of main span), in m/s^2

Finally, the response spectrum of the vertical displacement of the deck is given in figure 9.7. The deck is prone for resonance in vertical direction if excited by forces with a period of 0.38 sec. This corresponds to the first natural frequency of the bridge ($f_n = 2.88$ Hz). The damping is defined as the material damping for FRP (1%).

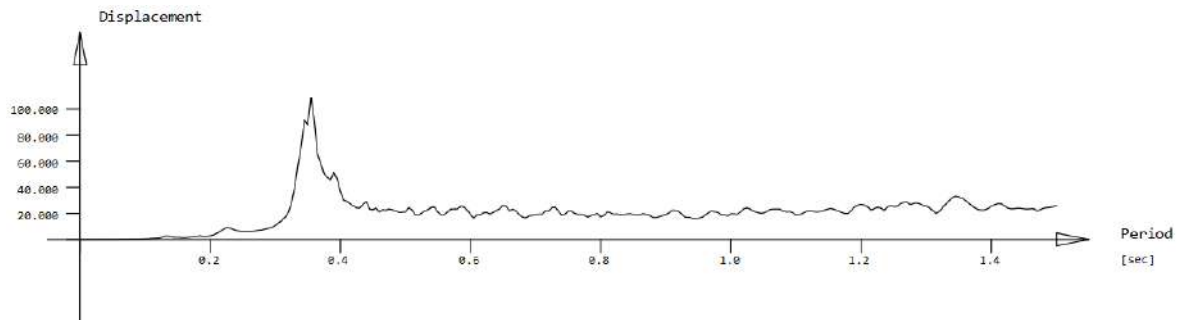


Figure 9.7: Response spectrum of FRP deck, for vertical displacements

9.2. Steel-concrete composite cable-stayed bridge

Similarly to the FRP bridge, a dynamic wind analysis for a linear elastic system was carried out. The Sofistik results printout is available in appendix E.6.

First of all, a quick check of the stresses was performed, in ULS. The checks have been done with the Sofistik modules and were satisfactory both for the steel girders and for the concrete slab. The utilisation levels are summarized in tables 9.2 and 9.3. The utilisation level for the slab is just acceptable, but it must be reminded that the analysis was linear elastic. The slab has more capacity if a plastic analysis were to be carried out. Finally, the stresses have been compared to the results from the preliminary design and showed good concordance.

Maximum Utilisation Level

	N	Vy	Vz	My	Mz	Mtp	Mts	Mb	Ncr	SCL	Total
	σ -x	σ +x	τ	σ -v	σ -s	σ -dyn	As-l	As-v	crack	c/t	
Section 4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	-	0.846
composite deck	0.569	0.846	0.413	0.846	0.170	-	-	-	-	-	
Total	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	-	0.846
	0.569	0.846	0.413	0.846	0.170	-	-	-	-	-	

N	normal force	τ	shear stress
Vy,Vz	shear force	σ -v	von Mises stress
My,Mz	bending	σ -s	stress in reinforcements
Mtp,Mts	torsion (p)rimary and (s)econdary	σ -dyn	stress range
Mb	warping moment	As-l	longitudinal reinforcements
Ncr	flexural buckling	As-v	transverse reinforcements
SCL	cross-section class	crack	crack width
σ -x	longitud. compressive stress	c/t	stress dependant utilisation level (see AQB Manual 2.3.2)
σ +x	longitud. tensile stress	Total	most unfavorable utilisation for all checks

Table 9.2: Utilisation level of stresses in ULS, in longitudinal steel girders

Maximum Utilisation Level

	N	Vy	Vz	My	Mz	Mtp	Mts	Mb	Ncr	SCL	Total
	σ -x	σ +x	τ	σ -v	σ -s	σ -dyn	As-l	As-v	crack	c/t	
Section 4	0.000	0.000	0.671	0.000	0.000	0.766	0.000	0.000	-	-	1.003
composite deck	0.000	0.000	0.000	1.003	-	-	0.841	1.000	-	-	
Total	0.000	0.000	0.671	0.000	0.000	0.766	0.000	0.000	-	-	1.003
	0.000	0.000	0.000	1.003	-	-	0.841	1.000	-	-	

Table 9.3: Utilisation level of stresses in ULS, in concrete slab

The maximum vertical deflections occurring in SLS have been checked once more and are illustrated in figure 9.8. These are again due only to traffic and permanent loads, because of the combination factors. The limit of $L/350 = 183$ mm is fulfilled by far.

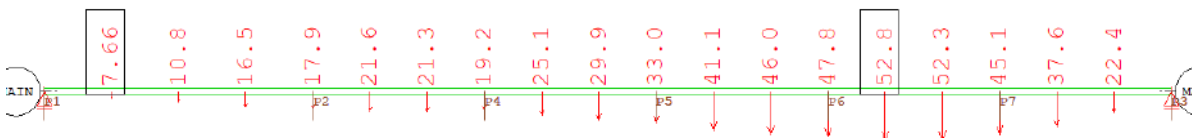


Figure 9.8: Maximum vertical deflections of steel-concrete composite deck

The dynamic response of the composite deck due to wind loading will be highlighted now. In figure 9.9, the vertical deflections of the steel-concrete composite deck are given, for a time period of 10 min. The maximum amplitude is approximately 12 mm. Flutter is most probably not an issue considering $\epsilon = \frac{f_t}{f_b} = 2.35 > 2.0$, which is confirmed by the EN1991-1-4 (one of the criteria for the deck to possibly be sensitive to flutter was not met).

The lateral deflections, which are illustrated in figure 9.11, remain very small with a maximum deflection of 1.03 mm. This peak is given in figure 9.10. In figure 9.12, the lateral deflections from static wind loading is given as a comparison. These come very close to the results from dynamic loading. A quick hand calculation also confirmed the results, with a deflection of 1.125 mm.

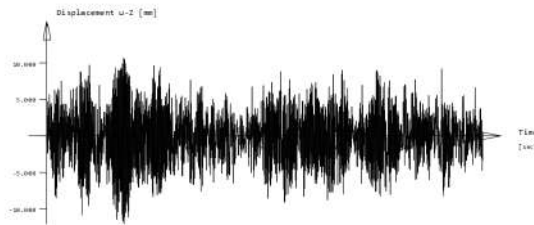


Figure 9.9: Vertical deflections of steel-concrete composite deck at most critical location, for a time period of 10 min, in mm

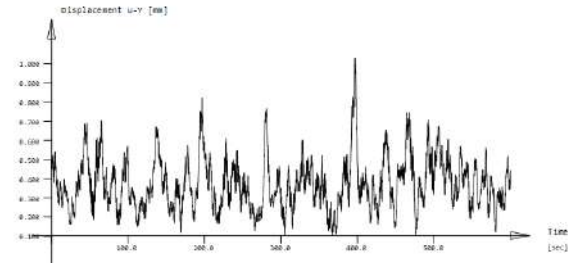


Figure 9.10: Lateral deflections of steel-concrete composite deck at most critical location, for a time period of 10 min, in mm

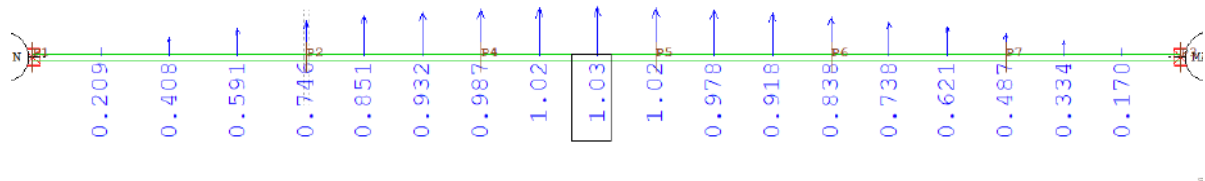


Figure 9.11: Maximum lateral deflections of steel-concrete composite deck due to dynamic wind loading, in mm

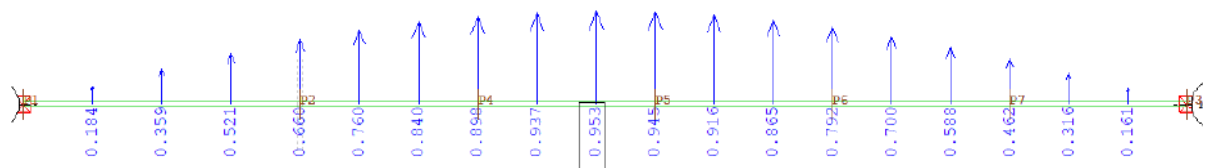


Figure 9.12: Lateral deflections of steel-concrete composite deck due to static wind loading, in mm

The accelerations of the deck at the most critical location in the deck, are given in figure 9.13. The maximum amplitude is 0.72 m/s^2 . The limit for pedestrian comfort is 0.7 m/s^2 according to EN1990. This requirement can be considered to be met.

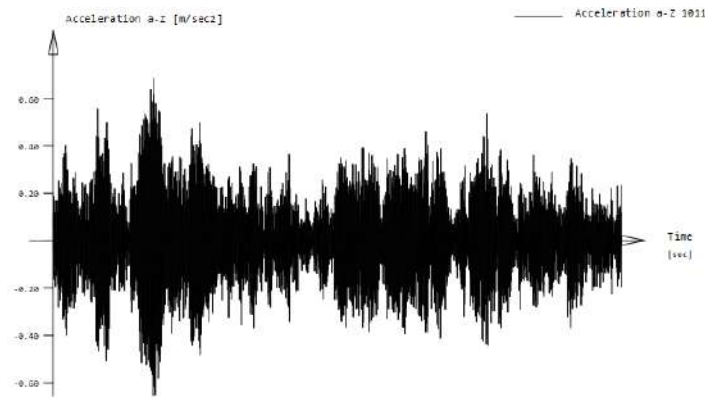


Figure 9.13: Vertical accelerations of steel concrete composite deck at most critical location, for a time period of 10 min, in m/s^2

Finally, the response spectrum of the vertical displacement of the deck is given in figure 9.14. The deck is prone for resonance in vertical direction if excited by forces with a period of approximately 0.88 sec, which corresponds to the first natural frequency for bending ($f_n = 1.187$ Hz).

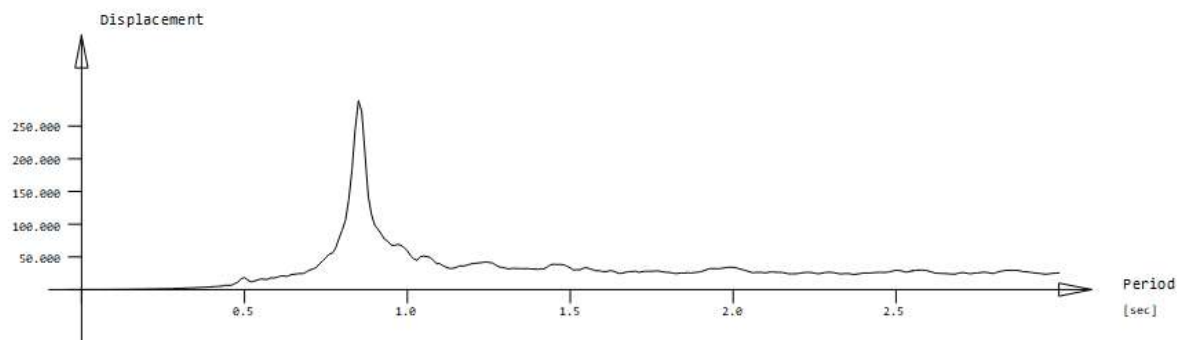


Figure 9.14: Response spectrum of composite deck, for vertical displacements

9.3. Short discussion

The results obtained for both cable-stayed bridges show that the designs are structurally safe for dynamic wind loads. However a few points are worth mentioning and discussing:

- The vertical deflections from the steel-concrete deck have higher amplitudes than the FRP deck (maximum amplitude 12 mm for steel-concrete deck and 8 mm for FRP deck). This is not what would be expected at first. However, if the structural factors $C_s C_d$ from both bridges are compared, it is noticed that the C_d term is slightly higher for the steel-concrete deck. This factor accounts for the influence of structural vibrations due to turbulence, which means that the steel-concrete deck is more sensitive to turbulence than the FRP deck. The C_d factors are given below, determined according to EN1991-1-4:
 - FRP deck: $C_s = 0.736$ and $C_d = 1.065$
 - Steel-concrete deck: $C_s = 0.733$ and $C_d = 1.122$

Another reason for the steel-concrete bridge to be more sensitive to wind excitation can be found in the wind load spectrum, in this case the von Karman spectrum. In figure 9.15, the spectrum from figure 8.7 is illustrated, zoomed on the reduced natural frequencies ($X = \frac{fL}{V}$) of the structure (1st order vertical bending and torsion). From this it can be seen that the vertical bending frequency of the steel-concrete bridge lies closer to the maximum of the wind spectrum. The steel-concrete bridge will tend to be more excited by the wind.

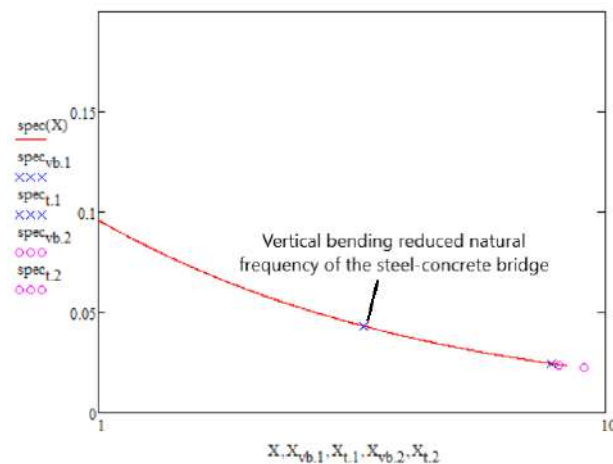


Figure 9.15: Von Karman spectrum wind load, zoomed on natural frequencies of the bridges; purple = FRP bridge ; blue = steel-concrete composite bridge

- These results are slightly simplified considering the pylons were given a very large stiffness to make sure only the behaviour of the deck could be analyzed. Further optimization is necessary to obtain a realistic design.
- The FRP deck exhibits slightly too high vertical accelerations for pedestrian comfort. However it may be argued that these would only occur for very high wind velocities, and in such a scenario, the bridge would most certainly not be used by a pedestrian. With a few iterations, it was found that for a mean wind speed of 14.64 m/s, the FRP deck satisfies the requirement for pedestrian comfort with a maximum acceleration of 0.68 m/s^2 .
- At this stage, no further design improvements concerning the decks are required in order to improve their dynamic behaviour for wind loads. Adding elements, or making more complex cross-sections would only increase the costs.

The two bridges that were designed and analyzed so far have a short span if compared to other cable-stayed bridges. The wind excitation remains limited. The typical main span range for this type of bridges is usually 100-1000 meters. Wind-induced vibrations usually become critical for the higher values of this span range. The next step is therefore to study how long span alternatives would behave.

10

Wind analysis: application to long span alternatives

As a final case study, the same bridges were taken and given a main span of 200 meters. By increasing the main span, the effect of wind loading will be increased which allows for a more in-depth comparison of the bridges.

10.1. Long span bridge models

Before presenting the models and their dimensions, it is important to mention the following:

- The cables have all been fixed at the top of the pylon, meaning the bridges have fan cable systems. Considering the amount of cables, this is not realistic and a semi-harp system would be a more feasible solution. For the sake of simplicity, the fan system was kept nevertheless.
- The main span has a length of 200 meters, which means that cable sagging should be accounted for. It has not been done here, as the main goal is to obtain models whose results can easily be compared to the previous ones.

10.1.1. FRP cable-stayed bridge

A first try to determine the deck dimensions for this 200 meter long FRP cable-stayed bridge, was done with the existing mathcad calculation sheets. The global dimensions for the bridge have been taken as indicated below. The main span was given a length of 200 meters and the other global dimensions were changed as to keep the same dimension ratios:

- Cable anchor distance: 8m
- Main span length: 200m
- Side span length: 64m
- Total number of pairs of cables: 32
- Pylon dimensions: 9.0*6.0m
- Cable dimensions: based on strength checks in Mathcad

The deck dimensions were first taken as being equal to the dimensions computed earlier (deck height of 430 mm). This was done to check the accuracy of the mathcad calculations compared to Sofistik results. In the Sofistik model, one of the main issues was the cables that would tend to loosen a lot due to pylon deflection, as well as axial compression of deck.

This is one of the effects which is not accounted for in the mathcad calculations. If in the Sofistik model the top of the pylon is pinned and axial compression prevented with additional longitudinal supports, the results between Sofistik and Mathcad are rather similar:

- Maximum vertical deflection in Mathcad due to resp. UDL and TS for deck height 430mm (elastic length: 11m): 112mm and 357mm
- Maximum vertical deflection in Sofistik due to resp. UDL and TS for deck height 430mm over same elastic length: 118mm and 375mm (see also figures 10.1 and 10.2)

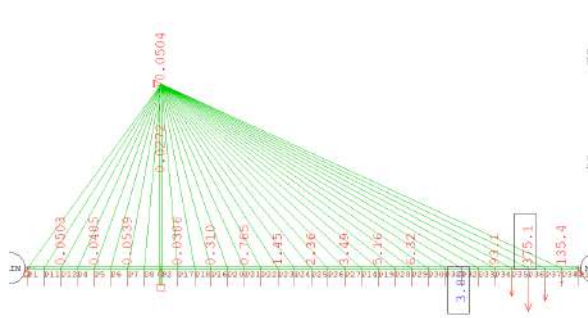


Figure 10.1: Maximum vertical deflections from TS as defined in Mathcad ; without cable pretension loss ; units: [mm]

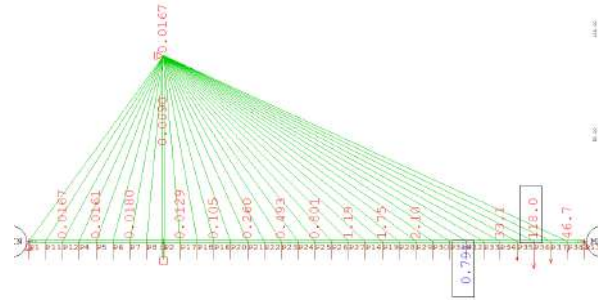


Figure 10.2: Maximum vertical deflections from UDL as defined in Mathcad ; without cable pretension loss ; units: [mm]

However the final conclusion was that the mathcad calculation sheet does not allow proper dimensioning of the deck: first of all the effect of cable pretension loss can not be neglected for this span length. In Sofistik, if cable pretension loss is included, the deflections are as indicated below. The difference with the previous results is significant.

- Vertical deflection Sofistik due to resp. UDL and TS for deck height 430mm over same elastic length, with cable pretension loss: 152.2 and 478.6mm (see also figures 10.3 and 10.4)

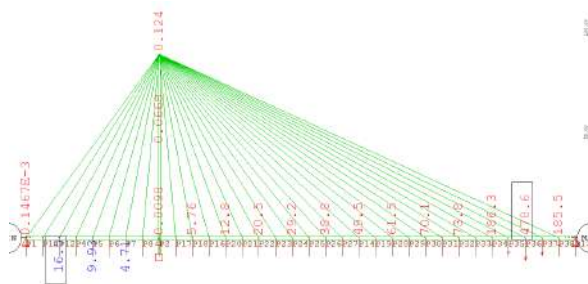


Figure 10.3: Maximum vertical deflections from TS as defined in Mathcad ; with cable pretension loss ; units: [mm]

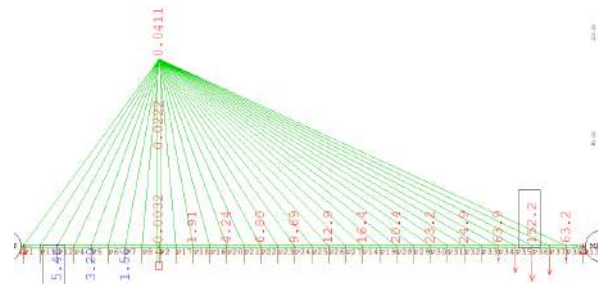


Figure 10.4: Maximum vertical deflections from UDL as defined in Mathcad ; with cable pretension loss ; units: [mm]

Additionally, the results from Mathcad can not be applied for a bridge with such a span because the maximum, critical deflection can not be computed by simply assuming the deck is a beam on elastic foundation. When using the traffic loader in Sofistik, the maximum deflections were found to be as indicated in the figures 10.5 and 10.6 (for TS and UDL), far higher than the previously computed values.

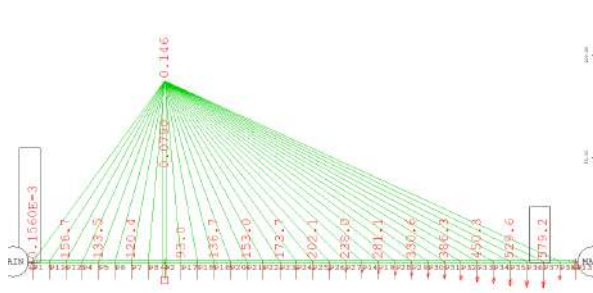


Figure 10.5: Maximum vertical deflections envelope from TS with influence line method from Sofistik traffic loader ; with cable pretension loss ; units: [mm]

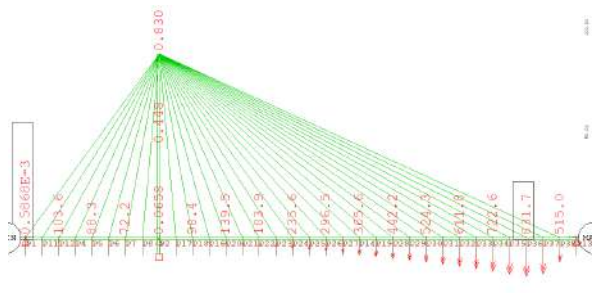


Figure 10.6: Maximum vertical deflections from UDL with influence line method from Sofistik traffic loader ; with cable pretension loss ; units: [mm]

It was chosen to further design the bridge iteratively in Sofistik. This has been done by adjusting the deck and cable dimensions, while avoiding over-dimensioning the deck, to obtain realistic wind results in the next section. As a check, the safety against flutter was computed according to EN1991-1-4 (same calculation as in section 9.1). It was found that a minimum deck height of 1000 mm is required. The cable dimensions were taken based on hand calculations (strength requirement). The pylon however has been over-dimensioned as was done in the previous chapter: this avoids excessive cable pretension loss and excessive lateral deformations due to wind. The issue of axial compression which also increases cable pretension loss, has been reduced considering the deck dimensions have increased. The bridge has been checked in SLS and ULS, with a critical deflection unity check of 0.98 (=560.6mm/571.0mm). The envelope for the stress unity checks in the FRP deck are shown in figure 10.7. The final model is illustrated in figure 10.8 and in table 10.1 the deck dimensions are given.

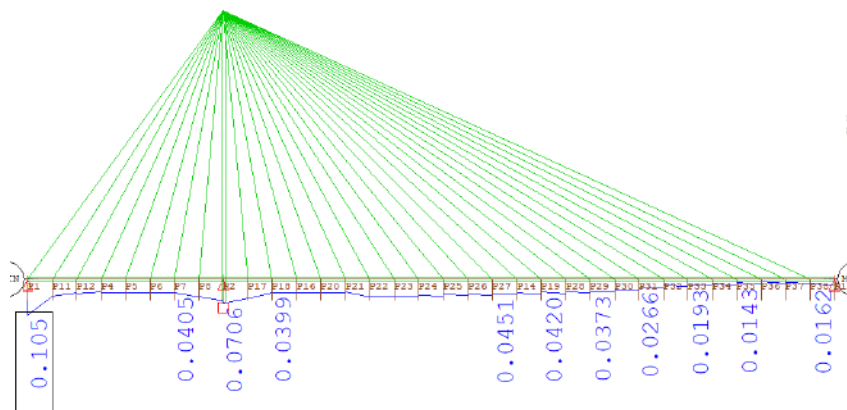


Figure 10.7: Envelope of stress unity checks in FRP deck

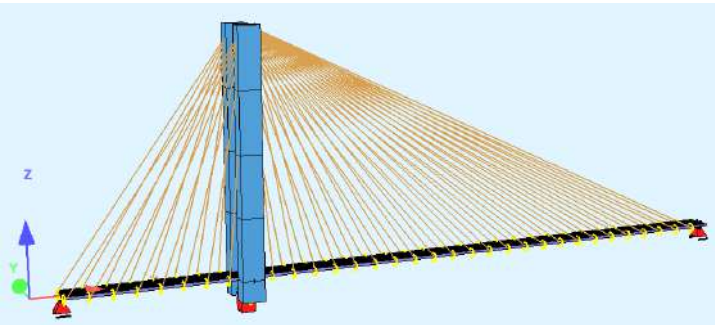


Figure 10.8: FRP long-span cable-stayed bridge FEM model

Deck	
Height [mm]	1000
Width [mm]	10000
Longitudinal web spacing [mm]	500
Flange thickness [mm]	42
Web thickness [mm]	24

Table 10.1: FRP deck dimensions

The pylon was given dimensions of 9.0*6.0m. The cables have been dimensioned on strength:

- The smallest cable has a diameter of 37 mm and the largest 58 mm.
- The backing stay cable was not dimensioned on strength. Its dimensions were increased to reduce horizontal deflections of the pylons. It has a diameter of 400 mm.

The wind coefficients of the FRP deck have been computed again and are given in figure 10.9.

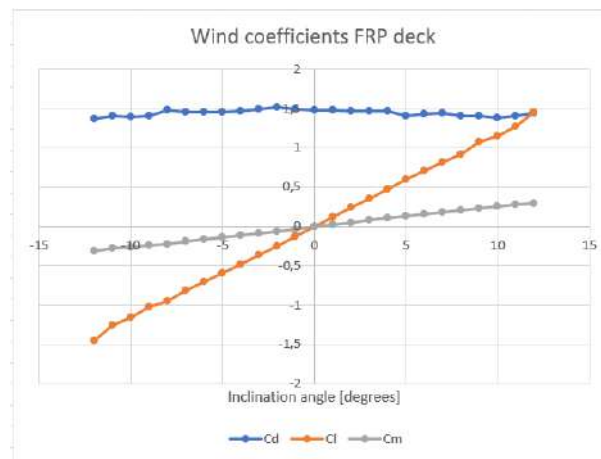


Figure 10.9: FRP deck wind coefficients

Last but not least, a modal analysis of the bridge has been performed to obtain the natural frequencies. The 7 first modes are summarized in table 10.2.

Mode nr	fn [Hz]	Description
1	0.387	Pylons symmetric lateral bending
2	0.396	Pylons anti-symmetric lateral bending
3	0.521	Deck lateral bending
4	0.775	Deck 1st order vertical bending
5	0.861	Deck 1st torsion, pylons anti-symmetric longitudinal bending
6	1.138	Deck 1st order vertical bending
7	1.434	Deck 2nd order vertical bending

Table 10.2: Natural frequencies for the 7 first modes of the FRP cable-stayed bridge

The 2 important modes are the first order vertical bending and first order torsion ones. These correspond respectively to modes 4 and 5 (see also figures 10.10 and 10.11):

- First order vertical bending natural frequency: $f_n = 0.775$ Hz
- First order torsion natural frequency: $f_n = 0.861$ Hz

These two frequencies lie really close. This means coupling of vertical and torsional motion may very well occur at high wind speeds, or in other words flutter may occur. A check from EN1991-1-4 was performed as explained earlier, resulting in the bridge being safe nevertheless.

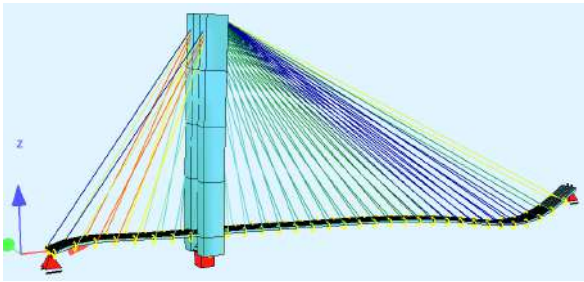


Figure 10.10: 1st order vertical bending mode

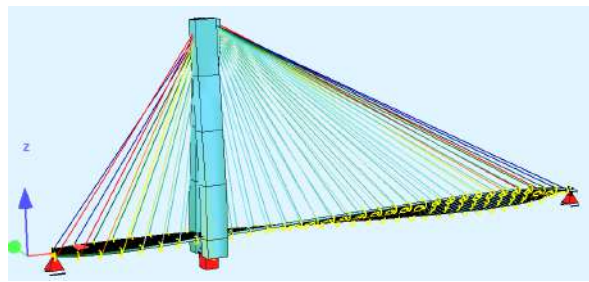


Figure 10.11: 1st order torsion mode

In order to define the Rayleigh damping, a frequency range starting at 0.521 Hz and ending at 3.190 Hz (3rd order torsion of deck) was defined.

10.1.2. Steel-concrete composite cable-stayed bridge

The steel-concrete composite bridge deck was dimensioned based on strength checks (combination of bending moments and normal forces in deck). A first calculation was performed with the existing mathcad calculation sheet, and compared with results from Sofistik. This was done for the following system:

- Cable anchor distance: 12m
- Main span length: 200m
- Side span length: 64m
- Total number of pairs of cables: 21
- Pylon dimensions: 9.0*6.0m
- Cable dimensions: based on strength checks in Mathcad

The dimensions of the previously designed deck were kept for the comparison between Sofistik and Mathcad results. The bending moments at the most critical location were computed for both. In Sofistik, the loads from Mathcad were placed at this critical location (180 meters from pylon). The results below do not include safety factors.

- Maximum bending moment in Mathcad for one cable plane: 5295 kNm. For the full deck width (2 cable planes): 10590 kNm.
- Maximum bending moment in Sofistik for two cable planes: 14038 kNm.

The difference is relatively large between the Mathcad and the Sofistik results. This might partly be due to the loss of pretension of the cables. To this can be added that this is not the maximum bending moment in the deck. When running the traffic loader module in Sofistik, a maximum bending moment of 34819 kNm in the main span is found. See also figure 10.12. This result does include combination factors and safety factors according to EN1990, but even without it is nevertheless considerably higher than the hand calculation value.

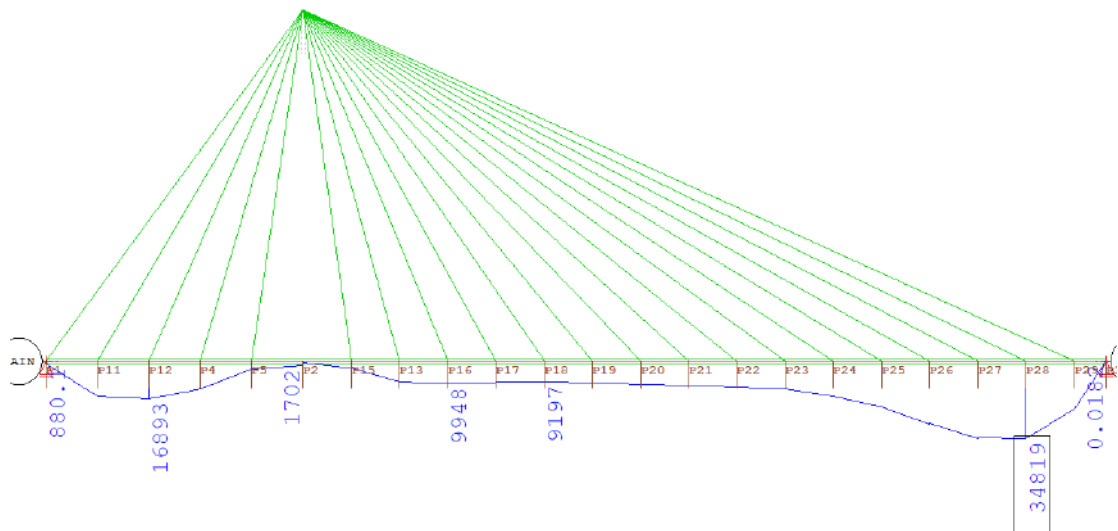


Figure 10.12: Maximum bending moment in main span of steel-concrete composite bridge

The bridge will be designed iteratively in Sofistik. The pylon has already been given large dimensions, and mostly the cables and the deck are adapted based on the strength requirements. After several iterations, the obtained design is as indicated in figure 10.13 and table 10.3. The critical strength check in the deck was found to be at the pylon level, due to the relatively large hogging bending moment combined with large normal forces. The unity checks

ended up being as indicated in figure 10.14 (in ULS). The flutter calculation from EN1991-1-4 resulted in the deck being safe.

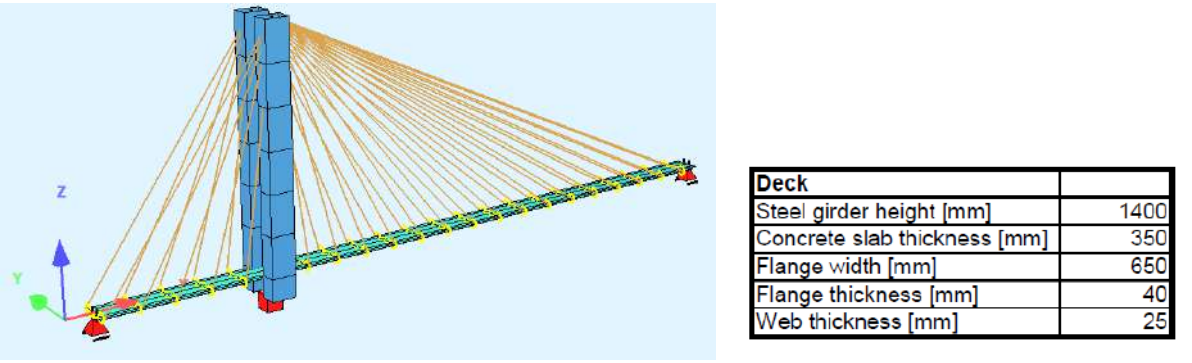


Figure 10.13: Steel-concrete composite long-span cable-stayed bridge FEM model

Table 10.3: Steel-concrete composite deck dimensions

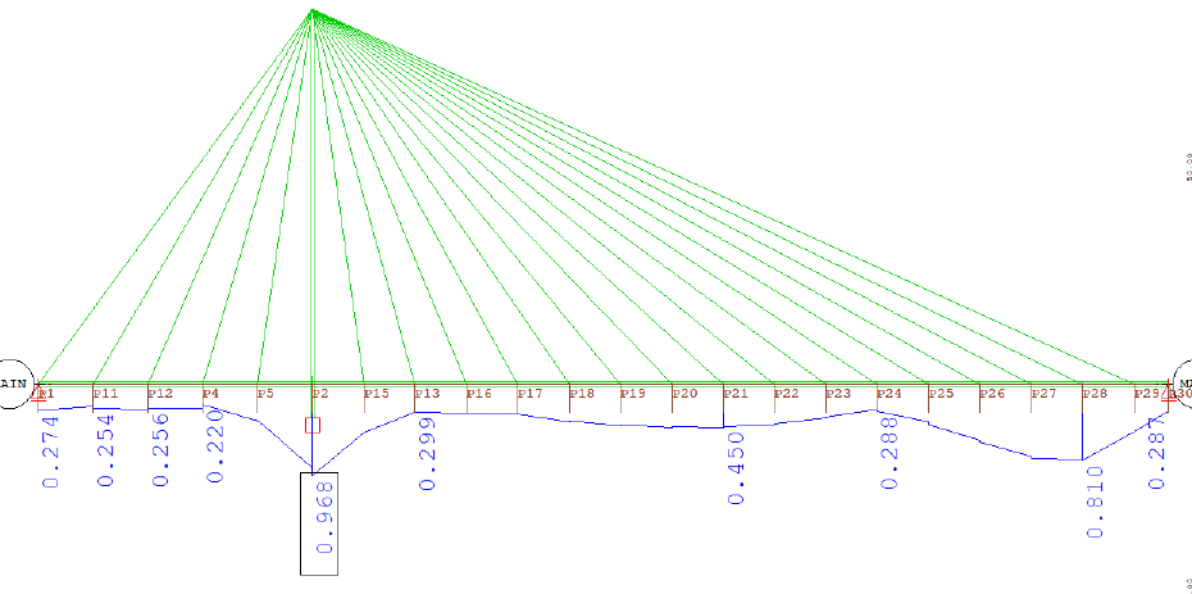


Figure 10.14: Strength unity checks in steel-concrete composite deck

The dimensions of the cables are as follows:

- The smallest cable has a diameter of 50 mm and the largest 77 mm.
- The backing stay cable has a diameter of 400 mm to avoid horizontal deflections of the pylon.

The wind coefficients of the deck have been computed again and are given in figure 10.15.

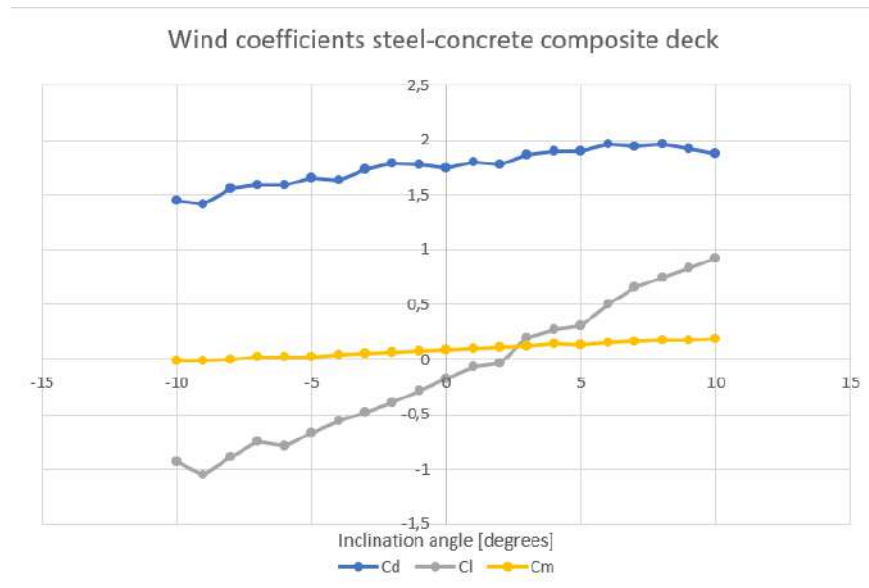


Figure 10.15: Steel-concrete composite deck wind coefficients

The modal analysis resulted in the natural frequencies as indicated in table 10.4.

Mode nr	f_n [Hz]	Description
1	0.387	Pylons symmetric lateral bending
2	0.398	Pylons anti-symmetric lateral bending
3	0.523	Deck 1st order vertical bending
4	0.524	Deck lateral bending
5	0.77	Deck 2nd order vertical bending
6	0.899	Deck 1st torsion, pylons anti-symmetric longitudinal bending
7	0.951	Deck 3rd order vertical bending

Table 10.4: Natural frequencies for the 7 first modes of the steel-concrete composite cable-stayed bridge

The first order vertical bending and first order torsional natural frequencies correspond to modes 3 and 6 (see also figures 10.16 and 10.17):

- First order vertical bending natural frequency: $f_n = 0.523$ Hz
- First order torsion natural frequency: $f_n = 0.899$ Hz

The 1st order torsion natural frequency is smaller than twice the 1st order vertical bending frequency. According to EN1991-1-4 (annex E), the structure has been checked for flutter safety. It satisfies the requirement.

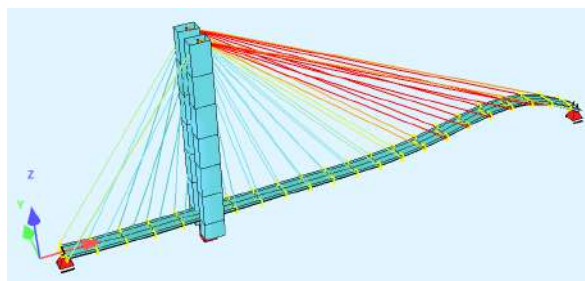


Figure 10.16: 1st order vertical bending mode

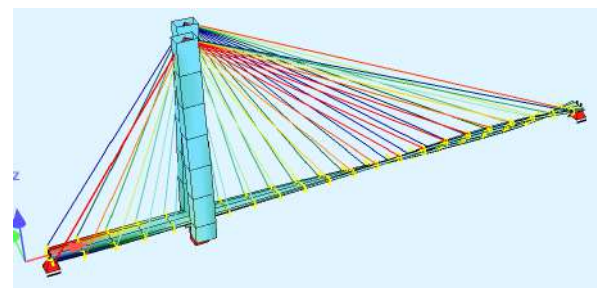


Figure 10.17: 1st order torsion mode

In order to define the Rayleigh damping, a frequency range starting at 0.523 Hz and ending at 1.866 Hz (2nd order torsion of deck) was defined.

10.2. Wind analysis results

The important results of both bridges are presented below, printouts can be found in appendix F.

10.2.1. FRP cable-stayed bridge

The dynamic response of the FRP deck is more pronounced which can be seen in the vertical deflections time response at the most critical location in the deck (at 168m from the pylon). This is illustrated in figure 10.18. The maximum amplitude is 49.12 mm, compared to 8 mm for the short span bridge.

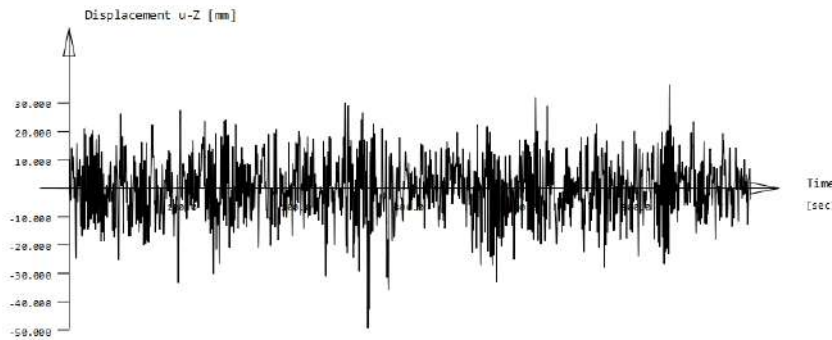


Figure 10.18: Vertical deflections in FRP deck for a time period of 10 min, at critical location, in mm

The highest vertical accelerations were measured at the same location as the maximum vertical deflection. This response time history is illustrated in figure 10.19. The highest amplitude is $0.61 \frac{m}{s^2}$, which fulfills the Eurocode requirement of $0.7 \frac{m}{s^2}$ for pedestrian comfort. The maximum amplitude of the short span bridge was $1.30 \frac{m}{s^2}$.

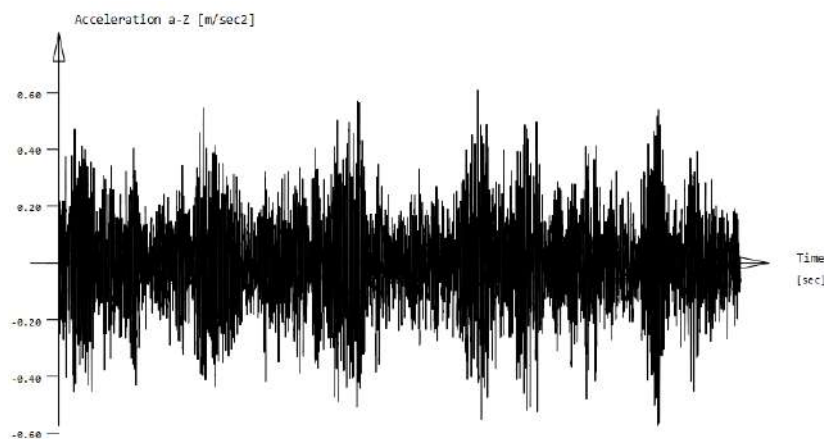


Figure 10.19: Vertical accelerations in FRP deck for a time period of 10 min, at critical location, in $\frac{m}{s^2}$

The highest lateral deflections were measured at 112m from the pylon. The corresponding time history is illustrated in figure 10.20, and has a maximum amplitude of 80.4mm. Compared to the 1.92mm lateral deflection for the short span bridge, there is a relatively large increase.

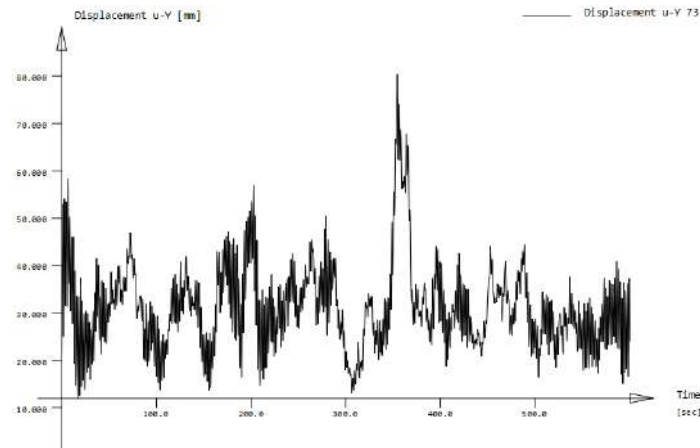


Figure 10.20: Lateral deflections in FRP deck for a time period of 10 min, at critical location, in mm

10.2.2. Steel-concrete composite bridge

The dynamic response of the steel-concrete composite bridge has increased, but relatively not as much as the FRP bridge. The vertical deflections were highest at 156m from the pylon. The corresponding time history is illustrated in figure 10.21, with a maximum amplitude of 35.2mm. The shorter span bridge had maximum deflection of 12 mm.

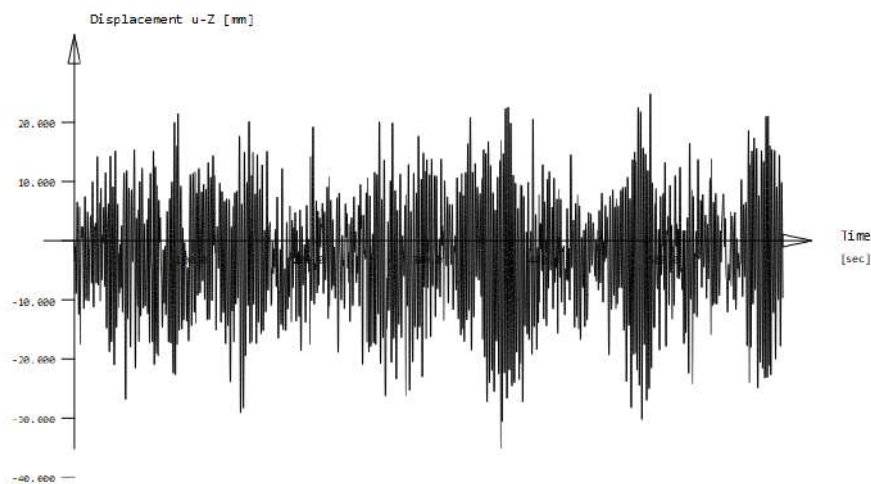


Figure 10.21: Vertical deflections in steel-concrete composite deck for a time period of 10 min, at critical location, in mm

The vertical accelerations of the deck were highest at the same location, and are illustrated for that case in figure 10.22. The maximum amplitude is $0.38 \frac{m}{s^2}$, which satisfies the Eurocode requirement for pedestrian comfort. Compared to the shorter span bridge, the amplitude is halved ($0.72 \frac{m}{s^2}$ for short span bridge).

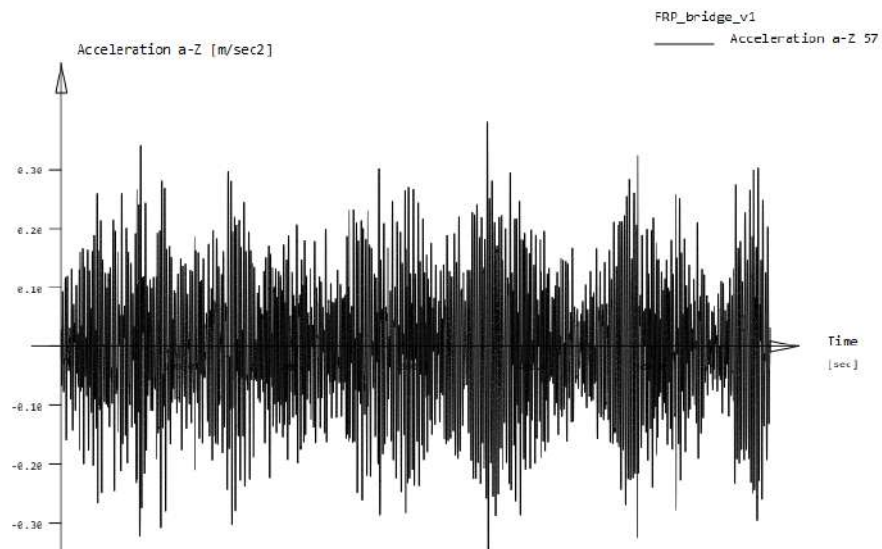


Figure 10.22: Vertical accelerations in steel-concrete composite deck for a time period of 10 min, at critical location, in $\frac{m}{s^2}$

Finally, the highest lateral deflections of the deck were found at a distance of 108m from the pylon (see figure 10.23). The maximum amplitude is 20.78mm, which compared to the short span bridge is far higher (for short span bridge 1.03mm).

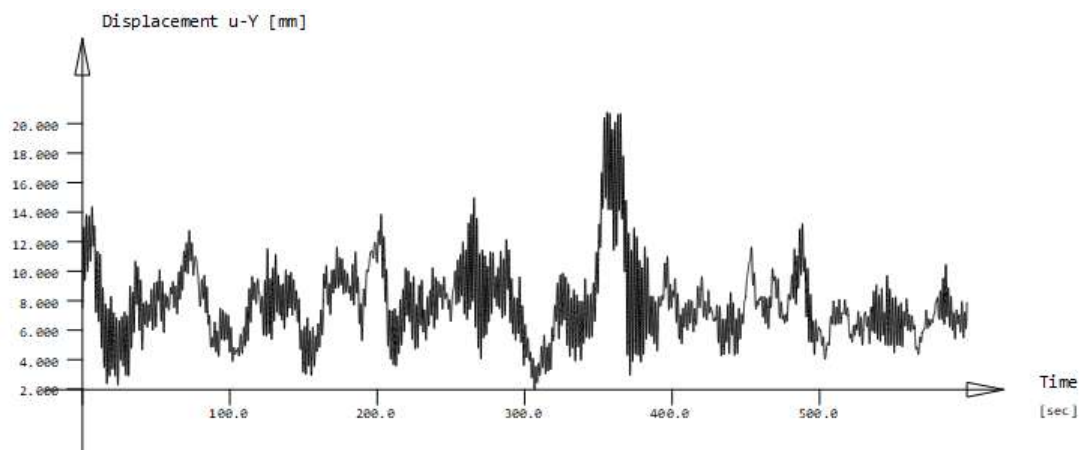


Figure 10.23: Lateral deflections in steel-concrete composite deck for a time period of 10 min, at critical location, in mm

10.3. Short discussion

What is of most interest here is how the increase in span affects the dynamic response both for the FRP deck and the steel-concrete deck. First of all, if the Von Karman wind load spectrum is considered once more (see figure 10.24), it can be noticed that the steel-concrete bridge still has the lowest vertical bending frequency even though the natural frequencies of the FRP bridge have relatively decreased far more than the steel-concrete bridge if compared to the results from figure 9.15. However, unlike previously, the steel-concrete bridge does not have the largest vertical deflections: the FRP deck has higher accelerations and higher vertical deflections. This can be explained by the damping of both structures, which is not accounted for in the computation of the natural frequencies. In figure 10.25, the Rayleigh damping as defined for both decks is illustrated. Clearly the FRP deck has lesser damping, which in turn results in larger wind excitation.

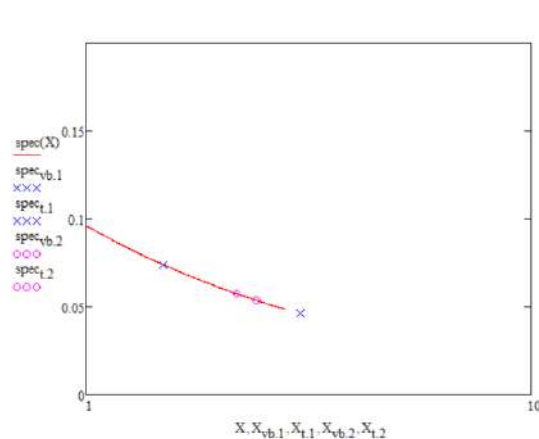


Figure 10.24: Von Karman spectrum wind load; zoomed in on natural frequencies of long-span bridges ; purple = FRP bridge ; blue = steel-concrete bridge

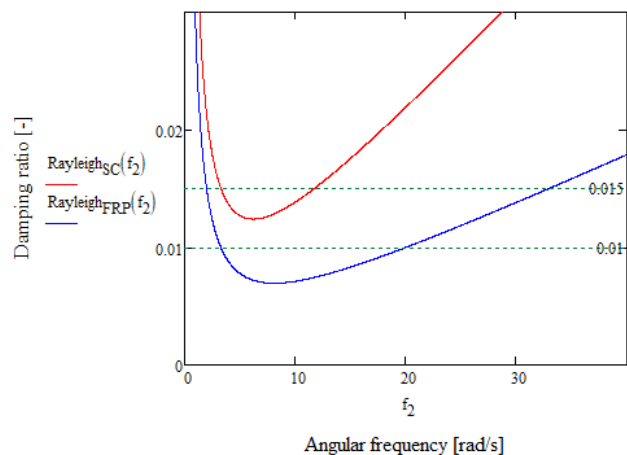


Figure 10.25: Rayleigh damping for both decks ; red = steel-concrete bridge ; blue = FRP bridge

In table 10.5, a comparison of the results of the short and the long span bridges is given based on the relative change in amplitude of the results. The results shown in the table are the corresponding amplitudes divided by the length of the main span (for the short span: 64m, and for the long span: 200m). The last column corresponds to the percent change between short and long span. From this, it can be seen that the amplitudes of the vertical deflections have, relatively to the main span, increased far more for the FRP bridge, which is also the case for the lateral deflections. The relative amplitudes for the vertical accelerations have for both bridges decreased by a lot. The sensitivity to wind excitation (buffeting) increases rapidly for the FRP bridge if the deflections are considered.

FRP

Relative amplitude	Short span [x/L]	Long span [x/L]	Relative change [%]
Vertical deflections [mm]	0,125	0,246	96
Vertical accelerations [m/s ²]	0,020	0,003	-85
Lateral deflections [mm]	0,030	0,402	1240

Steel concrete

Relative amplitude	Short span [x/L]	Long span [x/L]	Relative change [%]
Vertical deflections [mm]	0,188	0,176	-6
Vertical accelerations [m/s ²]	0,011	0,002	-83
Lateral deflections [mm]	0,016	0,1039	546

Table 10.5: Comparison of wind analysis results for short and long span bridges

Furthermore, a few noticeable aspects are worth mentioning:

- The lateral deflections of the FRP bridge are far larger than the lateral deflections of the steel-concrete bridge (maximum amplitudes of resp. 80.4mm and 20.78mm). The diaphragm effect from the short span is lost, and this has a large effect on the FRP bridge. To this can be added that the FRP bridge is very slender, with a deck height of 1000mm, and lightweight ; which is the opposite of the steel-concrete bridge.
- Remarkably the accelerations of both decks have decreased compared to the original designs, which is not what would be expected at first. However the mass per unit length has increased for both bridges, thus increasing damping, which is especially the case for the FRP bridge. This will reduce the accelerations. See the comparison of masses per unit length below in figure 10.26.

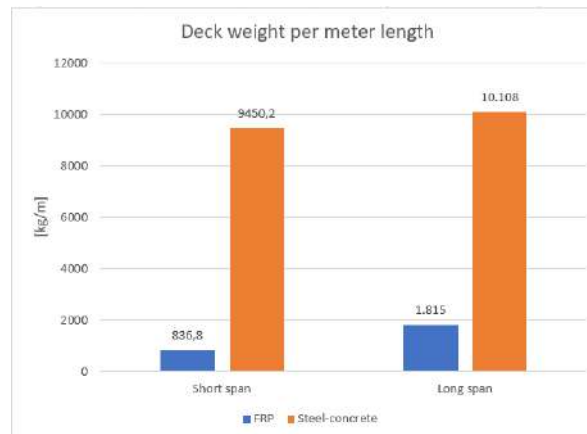


Figure 10.26: Weight per meter length of both decks

- Cable pretension loss is quite important in the design of the cable-stayed bridges, especially for the FRP bridge. For this FRP bridge, stiffness is crucial and a loss of stiffness has therefore a large effect on the design of the different elements. Axial compression should be reduced, and horizontal pylon deflections should be kept small. This effect was one of the main reasons for the analytic calculations not being applicable for the design of long span cable-stayed bridges. Some solutions might be considered in order to reduce this effect, such as applying A-frame pylons in longitudinal directions. This would reduce horizontal deflections of the pylon.

As was done previously, it is also interesting to consider costs of these long span bridges. Based on the same unit prices used so far, the costs have been computed and are shown in table 10.6. The total production cost of the FRP bridge is clearly superior to the steel-concrete bridge. This does not include the pylon due to the simplifications that were made earlier, nor transverse structural elements (cross girders). If only the costs of the decks are considered and compared to the costs for the short span decks, the costs for the FRP deck increase more rapidly than for the steel-concrete deck (see table 10.7). This is without taking maintenance into account: on the long term, the FRP deck will draw more benefits due to the low maintenance requirements. It is also worth mentioning that the relative weight increase of the decks from short to long span is far higher for the FRP deck (see table 10.8). This is for a great part due to the fact that the deck needs a minimum deck height for flutter safety. Even then, stiffness was rather critical considering the SLS deflection unity check was 0.98 (=560.6mm/571.0mm). This shows that for FRP cable-stayed bridges, the fact that the design is stiffness-based leads to a rapid increase in deck dimensions. With an expensive material such as FRP, this also leads to a rapid cost increase. This can also be seen in table 10.9 where the relative stiffness of the FRP deck from short to long span increases by 232 %, compared to 30 % for the steel-concrete composite deck. This is a good illustration of the fact that the design of the FRP deck is governed by stiffness.

FRP bridge	Weight [kg]	Volume [m ³]	Total price [euros]
FRP deck	479104		€ 5.749.248,00
Cables	528279		€ 7.924.185,00
		Total	€ 13.673.433,00

Steel concrete bridge	Weight [kg]	Volume [m ³]	Total price [euros]
Steel girders	358525		€ 1.792.625,00
Concrete slab	2310000	924	€ 277.200,00
Cables	558288		€ 8.374.320,00
		Total	€ 10.444.145,00

Table 10.6: Total production costs of long span bridges

	Short span [million euros]	Long span [million euros]	% increase
FRP	0,944	5,749	509,00
Steel-concrete	0,385	2,069	437,40

Table 10.7: Relative deck cost increase

	Short span [kg/m]	Long span [kg/m]	% increase
FRP	836,8	1814,8	116,87
Steel-concrete	9450,2	10108	6,96

Table 10.8: Relative deck weight increase

EI/L	Short span [kNm]	Long span [kNm]	Relative change [%]
FRP	9841	32635	232
Steel concrete	134297	174950	30

Table 10.9: Relative deck stiffness increase; units: kNm

This section may be concluded with the fact that the sensitivity to wind excitation increases more rapidly for increasing span length for the FRP bridge than for the steel-concrete bridge. Especially the check on flutter was critical for determining the FRP deck dimensions. For even longer spans, this will most likely be even more the case: the 1st order natural frequencies for vertical bending and torsion will remain close to one another meaning the bridge will be flutter sensitive. However this is for a great part due to the design of the pylons. For large span lengths, applying A-frame pylons as the one in figure 3.20 would increase the torsional stiffness of the system because the anti-symmetric longitudinal bending of the pylons would be prohibited. The first order torsional natural frequency would increase, thus reducing the risk of flutter.

IV

Conclusion

Discussion and summary of results

In chapter 1, the main research question and related sub-questions have been formulated in order to reach the objective of this thesis: contributing to the knowledge of FRP bridge design, through the design of a cable-stayed bridge and the analysis of its dynamic behaviour. This section will start with answers to the individual sub-questions, before concluding with the main research question. The sub-questions related to the literature study will not be treated here. These have been answered extensively in the corresponding chapters and have been used as a background for the rest of the thesis.

Sub-questions

Preliminary design

- *How can structurally safe and functional preliminary designs of the FRP and steel-concrete composite cable-stayed bridge concepts be achieved?*

The preliminary designs of both bridges have been obtained with two main steps:

- Analytic computations to determine general dimensions of the structures. This included a short parameter study to determine the optimal cable anchor distance for an FRP cable-stayed bridge; as well as the optimal corresponding deck layout.
- FEM models in Sofistik, with checks in ULS and SLS. Compared to the analytic calculations, two issues were highlighted: pretension loss of the cables due to horizontal pylon deflections causing a stiffness reduction of the entire structure, and the introduction of cable forces in the deck.

After the necessary changes, final preliminary models were obtained.

- *What are the main design parameters for an FRP cable-stayed bridge, and how are these linked?*

The main design parameters for an FRP cable-stayed bridge are:

- The distance between cable anchor points. As a result of the parameter study, the optimal cable anchor distance was found to be 8 meters.
- The height of the deck.

The design of the FRP cable-stayed bridge is stiffness-based. The deflections in the main span determine the required initial dimensions of the deck, whereas a concrete or steel deck is dimensioned on strength. It was noticed that for increasing cable anchor distance, the deck height had to be increased to counter both local and global deflections. The calculation of the global deflection of the deck was based on it being modelled as a beam on elastic foundation. The elastic bedding factor c reduced rapidly for decreasing

cable anchor distance, which resulted in a rapid increase in deck dimensions. For reference see figure 6.12. Additionally a minimum deck height has been computed based on the flutter check from EN1991-1-4, annex E.

- *What are the differences in structural behaviour between the obtained designs of the FRP and steel-concrete composite bridges?*

From analytic calculations, it was found that the optimal cable anchor distance for the steel-concrete composite bridge was 12.8 meters, which was confirmed by literature. The corresponding deck is dimensioned based on strength checks. Wind excitation is not critical in the design. The optimal cable anchor distance for the FRP bridge was 8 meters. However due to its low stiffness, deflections are critical instead of strength checks. Flutter checks gave a lower bound value for the deck height. The cables are also not fully utilized, unlike the steel-concrete composite bridge cables.

A comparison of the $\frac{EI}{m}$ ratios of the final preliminary designs show that the FRP deck can achieve a relatively high stiffness per unit mass with limited deck height (430 mm). With a factor 10 higher mass per unit length (936.85 kg/m vs 9450.2 kg/m), the steel-concrete deck is slightly more efficient if only the $\frac{EI}{m}$ ratio is considered. It does however require a total deck height of 1200 mm.

- *Is the FRP cable-stayed bridge economically feasible and competitive compared to the steel-concrete composite cable-stayed bridge?*

The total production costs of the preliminary bridge designs, based on unit prices provided by Royal HaskoningDHV, were as follows:

- FRP cable-stayed bridge: 2.65 million euros
- Steel-concrete cable-stayed bridge: 2.02 million euros

The FRP bridge is 0.63 million euros more expensive according to these estimates. From the static calculations and at this design stage, the FRP bridge seems structurally feasible and economically viable. The additional necessary investment will result in a lighter structure (294 kg/m^2 vs 1180 kg/m^2 for the steel-concrete bridge), with an aesthetically pleasing thinner deck. The foundation costs, which were not considered in the context of this thesis, will be reduced for the FRP bridge, which may for a part reduce the cost gap with the steel-concrete bridge. To this may be added that maintenance, which is an important reason for applying FRP, was not included in the computation of the costs. The steel-concrete bridge will have higher maintenance costs during its lifetime, compared to the FRP bridge. A life-cycle cost analysis is required to properly take these costs into account.

Advanced analysis

- *What important parameters need to be defined prior to a dynamic wind analysis of a cable-stayed bridge, and how are they defined?*

The most important parameters can be listed as follows:

- Damping of the structure. This was done by defining the Rayleigh damping of the structural elements that make up the model. The frequency range of the damped natural frequencies was defined with the help of a modal analysis.
- The aerodynamic wind coefficients for every deck have been defined through CFD wind simulations. This was done with the Dolfyn module in Sofistik. The resulting wind coefficient results were slightly lower than the Eurocode values. This can be explained by the simplifications from Eurocode, resulting in conservative values. The definition of these wind coefficients allowed proper modelling of buffeting.
- *What are the responses of the bridges to dynamic wind loads? How do they compare?*
The steel-concrete composite bridge deck exhibits higher vertical deflection amplitudes than the FRP deck, due to dynamic wind loads. This is not what would be expected at first, but looking into the natural frequencies of the bridges, it can be noticed that the vertical bending frequency of the steel-concrete bridge is far lower than the one of

the FRP bridge. This means the steel-concrete deck is more prone to wind excitation. However, the FRP deck has higher accelerations than the steel-concrete deck (1.30 vs 0.72 m/s^2). The Rayleigh damping of the FRP deck (1% critical damping) is lower than for the steel-concrete deck (1.5% critical damping). Accelerations are therefore more likely to have higher amplitudes, even though the vertical bending natural frequency of the FRP deck is not as low as the steel-concrete deck one.

Motion-induced vibrations couldn't be measured in Sofistik, but according to hand calculations, the bridges were both safe. The flutter check for the FRP bridge was on the verge of safety however and is therefore far more sensitive to these vibrations than the steel-concrete alternative.

- *How does a long span version of these cable-stayed bridges behave, more specifically the FRP one? How does it compare to the short span version?*

The bridges with a 200 meter long main span exhibited behaviours which were more in line with what would be expected. The FRP bridge has the largest vertical deflections and accelerations (resp. 49.12 mm and 0.61 m/s^2), compared to the steel-concrete composite bridge (resp. 35.2 mm and 0.38 m/s^2). The difference between the two long-span bridges is mostly due to the slenderness and light-weight of the FRP bridge. Although the 1st order natural frequency for vertical bending of the steel-concrete composite bridge is still the lowest (making it more prone for wind excitation), the difference with the natural frequencies from the FRP bridge is lesser: the FRP bridge is more sensitive to the wind excitation than previously with the short span. The relative low damping and low mass of the FRP bridge will consequently cause a larger dynamic response.

Compared to the short span bridges, the sensitivity to wind excitation has increased far more rapidly for the FRP bridge. This can especially be seen from the dynamic response to buffeting (turbulence-induced vibrations). For the FRP bridge a relative increase of 96 % in the amplitude of vertical deflections occurs between the short and long span, which for the steel-concrete composite bridge is a relative reduction of 6 %. Similarly lateral deflections relatively increased more than twice as rapidly for the FRP deck than for the steel-concrete composite deck from short to long span (FRP: +1240 % ; steel-concrete: +546 %). The FRP bridge is still very sensitive to flutter considering $\epsilon = 0.861/0.775 = 1.11 < 2.0$. This is due to the low torsional stiffness of the deck and the low self-weight. The design of the pylons also has a great influence on the torsional stiffness of the system and thus the torsional natural frequency.

Main research question

How can fiber-reinforced polymer composites be used as a construction material to design a cable-stayed bridge, which is able to withstand traffic loads and dynamic wind load excitation, while still being technically and economically feasible, and how does the FRP solution compare to a more "traditional" bridge design?

From the preliminary design and the dynamic wind analysis, an FRP cable-stayed bridge seems feasible both technically and economically, for short spans. The general approach for designing 'traditional' cable-stayed bridges at a preliminary design stage is not suitable due to the high strength and the relatively low stiffness of the material. An alternative sizing method was setup:

- From this thesis, the optimal cable anchor distance is 8 meters. The location for maximum deflection should be estimated.
- At this location, the dimensions of the corresponding cable and backing stay cable can be computed as explained in [32].
- The deck should then be modelled as a beam on an elastic foundation. The bedding factor can be estimated from the deflection due to a unit point load (computed as the extension from the local cable and the backing stay cable), and the cable anchor distance.

- With this system, the global deflection due to traffic loads LM1, which is critical, can be computed. The UDL should be applied over the elastic length of a beam on elastic foundation. The local deflection between cable anchors can not be neglected and should be checked individually as well as combined with the global deflection. The local deflection can be computed based on a continuous beam or with a simple FEM model. The deflections should be checked against the SLS deflection requirement.
- It is important to perform checks on dynamic wind excitation as defined in annex E from EN1991-1-4. Especially the flutter safety check might result in minimum required deck dimensions.

This initial design method showed its limitations when applied to the 200 meter span cable-stayed bridge. Proper results can be found analytically, which do match with results from FEM models. However these analytic calculations do not take an important aspect into account: cable pretension loss. This effect causes a loss of stiffness of the total structure and is due to horizontal pylon deflections and axial compression of the deck. This effect was present for the short span bridge, but it was more limited, making the above method applicable. For the long span bridge, this effect was increased and played an important role in the deflections of the bridge, and therefore the sizing of the elements. It could not be neglected and the calculations to obtain initial sizing of the long span bridges had therefore to be made in Sofistik through iterations. The cable pretension loss effect was the greatest for the FRP cable-stayed bridge, thus making this type of bridge less efficient. In order to reduce this cable pretension loss effect, some solutions can be thought of:

- Applying an A-frame pylon which reduces horizontal pylon deflections and thus cable pretension loss.
- Increasing the pylon height, which in turn increases the angles of the cable-deck connections. This makes the cables more efficient and increases the stiffness of the entire structure. Additionally, the deck will undergo lesser axial compression, thus reducing cable pretension loss.

Dynamically speaking, an FRP cable-stayed bridge exhibits large responses (high accelerations), but these can be controlled with proper design. A short span FRP cable-stayed bridge exhibits rather high accelerations due to wind, where the steel-concrete composite bridge had far lower accelerations. This is assumed to not be a large issue as these accelerations (1.30 m/s^2 when the pedestrian comfort limit is 0.7) occur for high wind speeds, during which pedestrians most certainly do not use the bridge. With a few iterations, it was found that the comfort limit requirement was met for a mean wind speed of 14.64 m/s (the actual mean wind speed is 19.4 m/s). Flutter was also decisive for the dimensions of the deck, with an ϵ value of 1.12. When increasing the length of the main span (thus all of the dimensions of the bridge), motion-induced vibrations were still critical for the FRP bridge ($\epsilon = 0.861/0.775 = 1.11$), which also had to be considered explicitly in the deck design. The steel-concrete composite long span cable-stayed bridge also has an ϵ value inferior to 2 ($\epsilon = 0.899/0.523 = 1.72$), but is not as much at risk (checks based on EN1991-1-4, annex E):

- Flutter check FRP: critical flutter speed of 38.569 m/s, larger than $2v_m = 38.4 \text{ m/s}$.
- Flutter check steel-concrete: critical flutter speed of 61.743 m/s, larger than $2v_m = 38.4 \text{ m/s}$.

The torsional natural frequency is for a great part dependent on the design of the pylons. The mode shape of the torsional natural frequency corresponds to an anti-symmetric horizontal bending displacement of the pylons. This effect can be avoided by applying A-framed pylons, thus increasing the value of the first order torsional natural frequency. Increasing the torsional stiffness of the deck would be another solution for increasing the torsional natural frequency. Some possibilities to do so are as follows:

- Instead of applying a rectangular shape, applying rounded edges would increase the torsional stiffness.

- Adding more material on the edges, and/or more longitudinal stiffeners towards the edges.
- Adding transverse stiffeners. Its in-plane shear capacity would increase the torsional stiffness of the cross-section.

The costs of the different bridge designs discussed in this thesis were only for production of the materials. Actual construction and maintenance costs were not included as this can not simply be estimated with unit prices. From the production costs only, and if only the costs for cables and deck are considered, the FRP cable-stayed bridge is both for the short and long span approximately 30 % more expensive than the steel-concrete composite bridge. This is also summarized in table 11.1. Costs for the substructure are not included but can be assumed to be cheaper for the FRP bridge due to its lightweight structure.

The construction and maintenance costs are hard to estimate but can be discussed qualitatively. The difference between the two type of bridges in this thesis when it comes to construction lies mainly in the deck:

- Even though the FRP deck is very lightweight, it has to be brought to site in pieces and assembled in-situ. Large cranes and machinery are required.
- The steel-concrete composite deck also has to be assembled in pieces; the concrete slab can be brought to the construction site as precast elements.
- The construction of the steel-concrete composite bridge is probably less expensive because of the easy assembly of the bridge deck elements. The machinery required in both cases is rather similar and won't make a large difference cost-wise. Assembly of the FRP deck elements is complex; these have to be glued and/or mechanically attached under the right conditions, making the whole process more expensive.

If only maintenance costs were considered, the FRP cable-stayed bridge would have the advantage. This material requires little maintenance if the design of the structure has been done properly. Steel structures however need regular inspection and maintenance in order to avoid damage from corrosion for example. Repainting of the structure is one of the maintenance routines that is required. This type of maintenance can be quite expensive, especially for large spans.

To properly account for these life-cycle costs, a LCC analysis is required. It is hard at this stage to clearly state what the outcome would be. What can be said is that the maintenance costs for the FRP bridge are lower than for the steel-concrete composite bridge. This cost difference increases for increasing span: for a certain span length, it might be possible that this maintenance cost difference makes the FRP bridge more interesting than the steel-concrete composite alternative when the total life-cycle costs are considered.

Production costs	FRP	Steel-concrete	% difference
Short span	€ 2.614.041	€ 1.968.455	32,80%
Long span	€ 13.673.433	€ 10.444.145	30,92%

Table 11.1: Production costs comparison ; only deck and cables

To conclude this discussion, the main issue resulting from this thesis, in which the focus lies on the dynamic response to wind excitation, is the fact that the FRP cable-stayed bridge is very sensitive to flutter both for short and long spans. This sensitivity is such that it defines the dimensions of the FRP deck. For short spans, the required torsional stiffness can easily be reached to make a flutter safe, and therefore also a structurally safe, design. However, for longer spans, stiffness-related design issues arise (cable pretension loss), which also become critical for the design. In combination with an increasing sensitivity to motion-induced vibrations, the design of long-span FRP cable-stayed bridges becomes complex: obtaining a stiff deck system with a high torsional stiffness will require a more complex deck shape than the

one considered in this thesis. The total production and construction costs will be far higher than for a steel-concrete composite cable-stayed bridge; this is already illustrated by the production costs alone, from table 11.1. The maintenance costs are the missing factor. These can make the FRP variant economically interesting, but are not easily estimated. Therefore no clear conclusion can be drawn on the economic feasibility of long span FRP cable-stayed bridges.

Conclusion and recommendations

12.1. Main conclusions

The main conclusions of this master thesis are as follows:

- Short span FRP cable-stayed bridges (in this thesis, main span of 64 meters) can be designed analytically, with a stiffness-based design, and a check on the safety for flutter excitation according to EN1991-1-4. The latter defines the required deck dimensions. A cable anchor distance of 8 meters is optimal.
- Long span FRP cable-stayed bridges can not be designed analytically due to the cable pretension loss effect from horizontal pylon deflections and axial compression of the deck. The design has to be done through iterations with a FEM model. The required deck dimensions are also defined by the flutter requirement from EN1991-1-4.
- From the dynamic wind analyses performed in this thesis, it may be concluded that buffeting is not an issue for the spans that have been investigated (64m and 200m). However, for increasing span lengths, the sensitivity for buffeting of the FRP bridge grows far more rapidly than for the steel-concrete bridge (relative increase of 96 % for vertical deflection amplitudes for the FRP bridge, compared to a relative reduction of 6 % for the steel-concrete composite bridge). The main issue that is highlighted throughout the project is the sensitivity for flutter of the FRP cable-stayed bridge. Both the short and long span bridges have ϵ values around 1.11, indicating that flutter really is a risk. As a means of comparison, the steel-concrete composite bridge has ϵ values of respectively 2.35 and 1.72. The deck dimensions for the FRP bridge were therefore also determined based on the flutter requirement of EN1991-1-4. The torsional stiffness of the FRP deck, but also of the entire cable-stayed bridge are of great importance in order to obtain a design that is safe for flutter. The pylons have a great influence on the total torsional stiffness and should be designed accordingly.
- Material costs for the FRP cable-stayed bridge are 30 % higher both for short and long span if compared to the steel-concrete composite bridge. This does not include construction nor maintenance costs. Due to complex detailing, construction costs of the FRP bridge are probably much higher than for the steel-concrete bridge. Maintenance costs for the FRP bridge are however much lower. A LCC analysis is required to determine these costs more exactly.
- The combination of several issues that arise when designing FRP cable-stayed bridges make this system inefficient and expensive when compared with a steel-concrete alternative. To obtain a stiff deck system for even longer spans (high torsional stiffness is required for flutter, as well as high vertical bending stiffness due to stiffness-based design), complex deck shapes are required which will increase the total production and

construction costs far beyond the costs for a steel-concrete bridge. Maintenance costs might make a difference depending on the span length, but this is once more something to be investigated based on LCC analyses.

12.2. Recommendations

Recommendations for further research on the subject of this thesis are summarized below:

- Motion-induced vibrations of the FRP deck have not been investigated in-depth due to the complexity of determining flutter derivatives. For short and large spans, it was shown that these vibrations play a crucial role in the structural safety of the bridge. Vibrations of this type, in FRP decks, should be further investigated. To do this properly, the effect of pylons should be incorporated. The pylons have a great influence on the torsional natural frequency: applying an A-frame will increase the torsional stiffness of the system, as well as the first order torsional natural frequency.
- An important advantage of FRP compared to steel is the low maintenance. The costs computed in this thesis only included production costs. Taking costs for maintenance (as well as construction costs) into account through a LCC analysis would give more insight in the economic advantages of FRP when applied to a cable-stayed bridge. It would be of interest to determine if there is a certain span range for which applying FRP would be structurally and economically more interesting than steel and/or concrete.

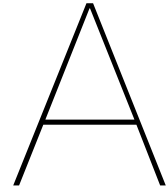
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V

Appendix



Design input for bridges

A.1. Design requirements

When designing a (cable-stayed) bridge, many parameters, demands and requirements need to be taken into account. In figure A.1, all demands when designing a bridge are listed. The most important ones for this thesis will be treated in the following sections.

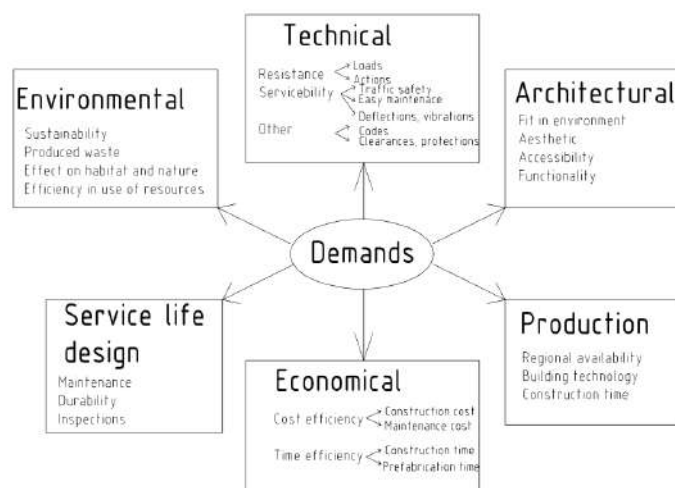


Figure A.1: Demands for the design of bridges [20]

According to the EN1990 [10], a structure should be designed and executed to withstand all actions that might occur during its intended working life, and fulfill the specified requirements regarding serviceability of the structure. Considering a structure can not be expected to fulfill this for all circumstances or situations, EN1990 states that the structure should have an appropriate degree of reliability, taking economic aspects into account. These levels of reliability can be different depending on whether the structural resistance or serviceability of the structure is considered. To achieve the required level of reliability, a combination of measures should be used [10]. Some of them are:

- Protective measures
- Design calculations should include partial factors, and representative values of action
- Measures concerning quality management
- Quality control during execution

When looking at serviceability of a bridge, the basis of design should include information concerning the intended service life, the requirements concerning functionality, comfort, appearance, and assumptions behind calculations. As for structural safety, some of the considerations are the hazard scenarios, measures to be taken to guarantee structural safety, soil conditions, calculation assumptions (modelling of the structure) and accepted risks. [15].

A.2. Actions on the bridge

According to EN1990 [10], actions can be classified according to their variation in time:

- Permanent actions (G), such as self-weight of the structure or fixed equipment
- Variable actions (Q), such as traffic or wind load
- Accidental actions (A), such as impacts

An action is represented by its characteristic value F_k . Variable actions are represented by [10]:

- The combination value $\phi_0 Q_k$, used for the verification of ULS and irreversible SLS.
- The frequent value $\phi_1 Q_k$, used for the verification of ULS involving accidental actions and reversible SLS.
- The quasi-permanent value $\phi_2 Q_k$, used for the verification of ULS involving accidental actions and reversible SLS. It is also used for the calculation of long-term effects.

A.2.1. Permanent actions: Self-weight and additional dead load

The self-weight of the structure is composed of the mass of all individual permanent structural elements. FRP composites will be used for the design of the bridge in this thesis. The material properties of FRP have not been standardized in the Eurocode. Nevertheless the densities can be found in the documentation of the products used, or derived with the rule of mixtures:

$$\rho = V_f * \rho_f + V_m * \rho_m \quad (\text{A.1})$$

where: V_f is the fibre fraction volume, ρ_f the density of the fibres, V_m the matrix fraction volume and ρ_m the density of the matrix.

Additional dead load comes from non-structural elements (for example: ballast, railings) and should also be taken into account in the design.

A.2.2. Variable actions: Traffic loads

The traffic loads are defined in EN1991-2. First of all, the actual loads on a bridge depend on the categories of vehicle, and more specifically on the actual traffic on the bridge. This traffic varies from bridge to bridge and depends on the composition of the traffic, its density, its conditions, the extreme vehicle weights, and axle loads. These parameters are taken into account through the use of load models, which will be shortly introduced later.

The carriageway of the bridge should be divided into notional lanes. The number and width of the lanes is defined in table A.1. An example of a division of the carriageway is given in figure A.2. Usually, the most loaded lane is number 1. Further definition of the notional lanes is given in EN1991-2 [10].

There are in total 4 load models. An exact description will not be given here, the author refers to EN1991-2 for that. Nevertheless each load traffic model represents a certain situation [10]:

- Load model 1 (LM1): Concentrated and UDL's. Covers most of the effects of traffic of lorries and cars. This model should be used for general and local verifications.
- Load model 2 (LM2): A single axle load on specific tyre contact areas which covers the local effects of the normal traffic on short structural members.

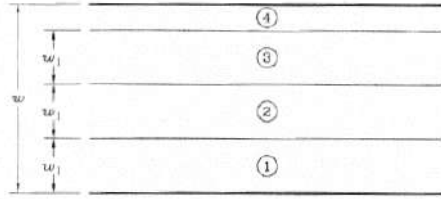


Figure A.2: Notional lanes [10]

Carriageway width w	Number of notional lanes	Width of a notional lane w_l	Width of the remaining area
$w < 5,4$ m	$n_l = 1$	3 m	$w - 3$ m
$5,4 \text{ m} \leq w < 6$ m	$n_l = 2$	$\frac{w}{2}$	0
$6 \text{ m} \leq w$	$n_l = \text{Int}\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_l$

NOTE For example, for a carriageway width equal to 11m, $n_l = \text{Int}\left(\frac{11}{3}\right) = 3$, and the width of the remaining area is $11 - 3 \times 3 = 2$ m.

Table A.1: Notional lanes [10]

- Load model 3 (LM3): Used for representing special vehicles, with a set of assemblies of axle loads. For general and local verifications.
- Load model 4 (LM4): A crowd loading, for general verifications only.

Depending on the type and use of the bridge, load models 1, 2 and 3 should always be applied in the design of the bridge. Load model 4 is only used for certain transient design situations. Horizontal braking and acceleration loads should also be considered in the design. A multi-component action can therefore occur in the bridge, meaning group of loads need to be considered as occurring at the same time. This is defined in table 4.4a of EN1990.

A.2.3. Wind loads

In this section, the wind loads will be defined when a dynamic analysis is not required. According to EN1991-4 [9], simplified static loads can be applied in that case. The dynamic wind analysis is discussed in chapter 5 on bridge wind dynamics.

The wind load acting on a bridge deck can act in three directions (along the deck, transverse to the deck, and in vertical direction). The wind-induced forces along those directions are as follows:

- In transverse direction: $F_w = \frac{1}{2} \rho v_b^2 C_x A_{ref,x}$, where ρ is the air density, v_b the basic wind speed, C the wind load factor (defined as $C = c_e c_{f,x}$ in EN1991-4), and $A_{ref,x}$ the reference area.
- In vertical direction, similar to the transverse direction loading: $F_z = \frac{1}{2} \rho v_b^2 C_z A_{ref,z}$
- In longitudinal direction, the force may be taken as 25% of the force in transverse direction for plated bridges, and for truss bridges as 50%.

To determine the factors and further definition of the different terms, reference is made to the EN1991-4 [9], section 8.

A.3. Material properties and design values

When designing with FRP, the design value of a material property is defined according to CUR96 [26] as:

$$X_d = \frac{\eta_c * X_k}{\gamma_M} \quad (\text{A.2})$$

where X_k is the characteristic value of a material property, η_c is the conversion factor, and γ_M is the material factor.

The material factor γ_M has to be used for checks of FRP structures in ULS. It is defined as:

$$\gamma_M = \gamma_{M1} * \gamma_{M2} \quad (\text{A.3})$$

where γ_{M1} is the partial material factor which takes into account geometrical imperfections and uncertainties about the obtention of the properties. γ_{M2} is the partial material factor

which takes into account uncertainties about strength properties and which is dependent on the spread of material properties. The values for both are defined in section 2.4.4.3. of CUR96.

The conversion factor η_c accounts for several effects that may affect the properties of FRP. It is defined as:

$$\eta_c = \eta_{ct} * \eta_{cm} * \eta_{cv} * \eta_{cf} \quad (\text{A.4})$$

where: η_{ct} is the conversion factor for temperature effects, η_{cm} is the conversion factor for moisture effects, η_{cv} is the conversion factor for creep and, η_{cf} is the conversion factor for fatigue.

Based on the checks that are performed, not all conversion factors have to be multiplied with one another. This is defined in table A.2.

Conversiefactor	Toetsaspect						
	Sterkte (UGT)	Stabiliteit (UGT)	Vermoeding (UGT)	Vervorming onder quasi-blijvende belasting (BGT)	Vervorming onder kortdurende belasting (BGT) ¹⁾	Comfort (trillingen) (BGT) ²⁾	Schade (BGT) ³⁾
η_{ct}	✓	✓	✓	✓	✓	✓	✓
η_{cm}	✓	✓	✓	✓	✓	✓	✓
η_{cv} ³⁾	✓	✓ ⁴⁾		✓			✓
η_{cf} ⁵⁾		✓		✓	✓	✓	✓

Table A.2: Table of conversion factors, based on the performed check [26]

All values for the conversion factors are given in section 2.4.5 of CUR96 [26].

A.4. Limit state design

The Eurocode [10] states that a distinction should be made between the ultimate limit state, in which the structural safety is assessed, and the serviceability limit state, in which the functionality of the structure is assessed.

A.4.1. Ultimate limit state

The ultimate limit state of a structure concerns its stability, strength and fatigue failure. The limit state for stability (loss of static equilibrium) is verified by [10]:

$$E_{d,dst} \leq E_{d,stab} \quad (\text{A.5})$$

where: $E_{d,dst}$ is the design value of the effect of destabilizing actions, and $E_{d,stab}$ the design value of the effect of stabilizing actions.

When considering an FRP bridge, the following stability failure modes should be considered, as defined in CUR96 [26]:

- Buckling
- Lateral-torsional buckling
- Flexural-torsional buckling
- Local buckling

The limit state for strength is verified by [10]:

$$E_d \leq R_d \quad (\text{A.6})$$

where: E_d is the design value of effects of actions (internal forces, moments) and R_d is the design value of the corresponding action.

The following corresponding failure modes should be considered [26]:

- For plates and shells, the effects of normal and shear stresses, inter-laminar shear, compression and tensile stresses should be checked.
- For bar elements, the effects of axial compression and tensile stresses, bending moments, shear and torsion should be checked.

Failure due to fatigue is due to repeated cyclic loading of the structure, leading to progressive failure and degradation of the material. This is especially crucial at constructive details. The limit state for fatigue is based on the rule of Miner and is verified by [10]:

$$D = \sum_i^M \frac{n_i}{N_i} \leq 1 \quad (\text{A.7})$$

where: n_i is the number of repeated loading cycles of a certain magnitude, and N_i is the number of repeated loading cycles of a certain magnitude up to failure.

Fatigue only has to be taken into account for structures for which the magnitude of cyclic loading varies and the expected number of loading cycles is greater than 5000; or for which the absolute maximum cyclic loading is greater than 40% of the failure load. For FRP traffic bridges, fatigue load model 4b of EN1991-1-2 should be applied according to CUR96.

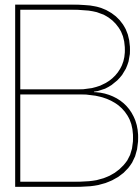
A.4.2. Serviceability limit state

The serviceability limit state concerns a check on functional aspects such as deflections and vibrations of the structure. These checks are often critical in the design of FRP structures, due to the relatively low stiffness of the material.

The limits on deflections are not defined in the CUR 96. Therefore the limit from the Bruprosjektering will be taken [30]. For any load combination, the deformation should be limited to $L/350$ where L is the span length.

Vibrations of bridges can have several origins. In the case of the to-be-designed bridge, wind excitation is of major importance and is discussed in more detail in chapter 5. EN1991-1-4 states that for standard bridges with spans below 40 meters, a dynamic wind analysis is not required. Nevertheless, in this thesis, this is not the case, considering the main construction material is FRP, and the span is 64 meters. A dynamic analysis will be performed in the final part of this thesis.

When performing a dynamic analysis, it is important to check the different wind-induced oscillations, which can be for example vortex-shedding, flutter, galloping, or buffeting. Some indications on when a bridge might be sensitive to one of these behaviours are given in Annex E of EN1991-1-4. Issues that might arise from these oscillations are fatigue, resonance (which in term can lead to failure of the structure), or discomfort for users.



Conceptual design

First of all, existing bridge types will be discussed and judged on whether they would be suitable for the location. The main design drivers are summarized as follows:

- Span of bridge, without intermediate supports
- Maintenance
- Way of installation
- Wind response

After this general discussion, some feasible concepts of the remaining bridge type(s) will be compared based on their cost-effectiveness. This will only be done from a qualitative point of view.

B.1. Bridge types

In the following section, the bridge types are given, as well as the reason why or why not these would be suitable for the location in Norway.

- Plate girder bridges have lengths going from a few meters up to 125 meters, which requires intermediate piers. In this situation, intermediate piers aren't allowed and a simply supported plate girder bridge would be needed. However the dimensions and the weight of the required structure would be excessive to reach a span of 64 meters. This type of bridge is therefore considered as unsuitable.
- This type of bridge comes across the same issue as plate girder bridges: to reach long spans, in this case up to 300 meters, intermediate piers are often required. However, box girder bridges can reach a longer span while being simply supported. Nevertheless, box girder bridges are most certainly not cost-effective when compared to truss, arch or even cable-stayed bridges. Therefore they will not be studied further.
- The required span of 64 meters is very suitable for truss bridges. This bridge type has a better material use than plate or box girder bridges, and can therefore achieve large cross-sections where material is placed only where necessary. Ideally, only compressive or tensile forces would occur, which in practice is not always the case. The main issue however is the way of installation: bringing in the bridge as one piece might be possible but is a complex situation. Other construction methods might be possible. Considered as suitable for now.
- This range of spans is very suitable for the application of arch bridges. The construction of this bridge can for instance start with the arches built up in segments, and afterwards the placement of the deck. The construction method is probably more cost-effective

than the truss bridge, though still slightly complex. Arch bridges are very suitable for this location when only considering the span. However, the same reasoning as with the truss bridge applies here: the construction of this bridge probably requires intermediate supports or complex installations. Considered as suitable for now.

- The economic span range for cable-stayed bridges starts at approximately 100 meters. However, this bridge type can be applied for shorter spans, as is the case here, although it might not be as cost-effective as other alternatives when building with traditional materials. It has the most suitable construction method for this location.
- Suspension bridges are built when long spans are required, and this usually starts at around 300 meters. The required span in this situation is far from that. To this can be added that the access to one side of the river is limited, nor is there enough place to build a side span over there. The anchoring of the cables on that side is also not possible as a steep mountain side is close to the river.

The three bridge types that will be considered next are the arch, truss and cable-stayed bridge. A few concepts will be explored for each of these bridge types and compared in the following section.

B.2. Bridge concepts

B.2.1. Cable-stayed bridge concepts

When looking for concepts of cable-stayed bridges, many parameters can be varied. Some of the most important ones are given below. The concepts presented hereafter are based upon variations of these parameters. Every time, arguments are given whether or not, the choice for a parameter is suitable for the site and its requirements.

- Materials used
- Longitudinal and transverse cable arrangement
- Cable anchoring (earth-anchored or self-anchored system)
- Connection between pylon and deck (lateral, vertical, torsional restraints) and supports
- Pylon
- Deck system

A few assumptions were made when making the concepts. From literature [25], a few rules of thumb can be used for the general dimensions: $\frac{h_{pyl}}{l_{ms}} = 0.35-0.45$ and $\frac{l_{ss}}{l_{ms}} = 0.3$. Furthermore, as previously explained, no piers nor pylons should be built in the main span of 64 meters. Therefore the filling on the left side of the span won't be placed over a length of 20 meters. This empty space will be the side span of the cable-stayed bridge. The pylon will be placed at the edge of where the filling should have been: this is shown in the figures below. The pylon and cables will respectively, for every concept, be made of concrete and steel. Pylons made of steel are very expensive and only applied when the pylon needs to be high. Only the material of the deck will be varied.

Concerning the construction of the bridge, the method is similar for all concepts: the pylon will be built first. Afterwards, the side span and backing stay cable will be placed. Finally the main span will be placed by cantilevering sections one at a time. The main difference between the construction of the concepts will lay in the size of the elements and the required machines: concrete elements are for instance far heavier than FRP elements.

- Concrete cable-stayed bridge concept

The parameters for this concept have been chosen as described below.

- Longitudinal cable arrangement: fan arrangement, with one pair of backing stay cables (see figure B.1). From literature, the optimal cable anchor distance is approximately 8 meters, which in this situation translates to 7 pairs of cables in the main span. Transverse cable arrangement: A-frame arrangement (see figure B.2). The fan arrangement is possible in this situation due to the short span and therefore little amount of cables. With a large amount of cables, the connection between pylon and cables would become too complex. The transverse A-frame arrangement ensures a higher torsional stiffness of the deck. This is beneficial for long span bridges, but probably not necessary in this situation.
- The bridge is self-anchored. The compressive forces are therefore high at pylon level, but this is not an issue for concrete. It also avoids the installation of large anchor blocks.
- The supports of the deck are illustrated in figure B.3. At pylon level, the deck is only supported in lateral direction. Having no vertical support at pylon level greatly reduces the bending moments. However additional cables might be necessary in the side span.
- The pylon has an A-frame arrangement, which is related to the cable arrangement previously discussed.
- The deck is a concrete box girder, which means the deck has rather high torsional stiffness. This deck system and the high damping of concrete means that the bridge is probably not too sensitive to wind excitation.

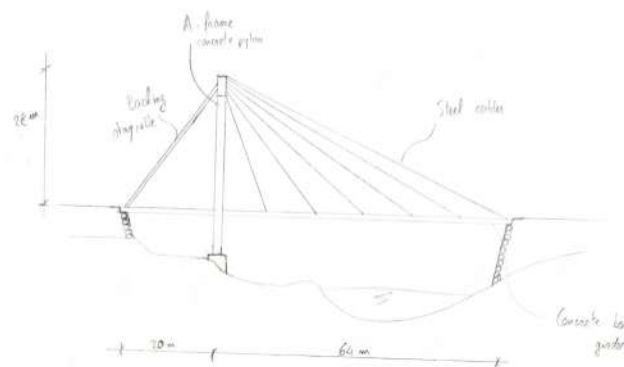


Figure B.1: Side view of concept A cable-stayed bridge

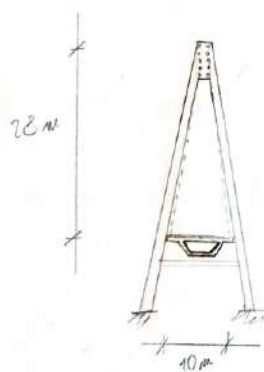


Figure B.2: Transverse view of concept A

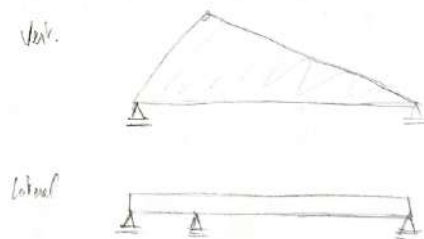


Figure B.3: Vertical and lateral supports of concept A bridge

- Steel-concrete composite cable-stayed bridge concept

The parameters for this concept are as follows:

- Longitudinal cable arrangement: semi-fan arrangement, with one pair of backing stay cables (see figure B.4). From literature, for composite systems, the ideal cable anchor distance is around 13m (total of 4 pairs of cable in main span). Transverse cable arrangement: λ -frame arrangement (see figure B.5). The semi-fan arrangement is rather similar to the fan arrangement, structurally speaking, with slightly easier pylon-cable connections. The λ -frame arrangement increases the torsional stiffness of the deck system, which in itself has very little.
- The bridge is self-anchored. The compressive forces are therefore high at pylon level, but this is not an issue: the concrete carries most of the compressive forces.
- The supports of the deck are the same as in figure B.3. Similarly to the previous concept, cables in the main span might be required.
- The pylon has a λ -frame arrangement, which is related to the cable arrangement previously discussed.
- The deck is a steel-concrete plate girder system, which means the deck has a low torsional stiffness. This deck system in itself would be probably be sensitive to wind excitation due to its bluff shape and low torsional stiffness. However, the inclined cable system makes up for it and provides the required stiffness.

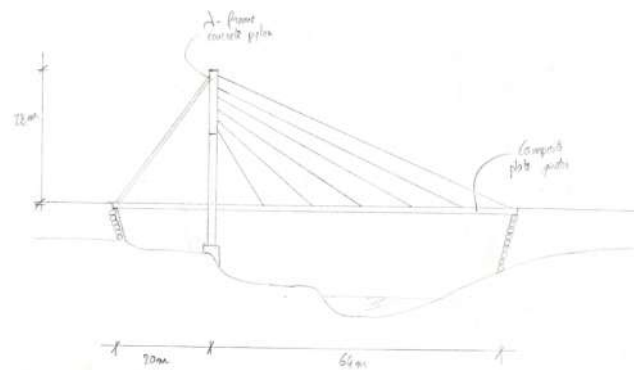


Figure B.4: Side view of concept B cable-stayed bridge

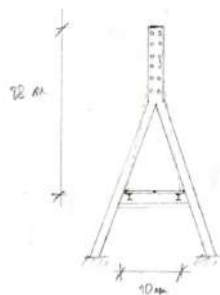


Figure B.5: Transverse view of concept B

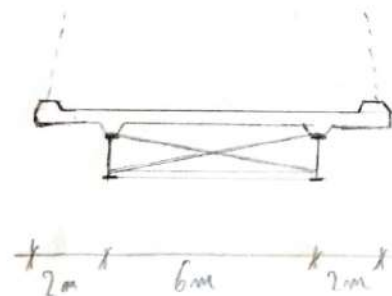


Figure B.6: Deck system of concept B

- Cable-stayed bridge concept with orthotropic steel deck

The parameters for this concept are as follows:

- Longitudinal cable arrangement: harp arrangement (see figure B.7). From literature, for steel decks, the ideal cable anchor distance is around 16m (total of 3 pairs of cable in main span). Transverse cable arrangement: single cable plane arrangement (see figure B.8). The harp system is not very advantageous structurally speaking but allows easier

detailing of the connections and is more aesthetically pleasing. Probably not the best arrangement for a cable stayed bridge in this site. The single cable plane means that the deck should have a high torsional stiffness of itself. The cross-section from figure B.9 should suffice, considering the span.

- The bridge is self-anchored. The compressive forces are therefore high at pylon level. Buckling of the deck might become critical.
- The deck should be pinned at pylon level, with some torsional restraint. At the side supports, the deck is on rollers.
- A single pylon is applied.
- The deck is a orthotropic steel deck, with a high torsional stiffness. Considering the span, and the high inherent stiffness of the cross section, wind excitation should not be an issue if properly designed.

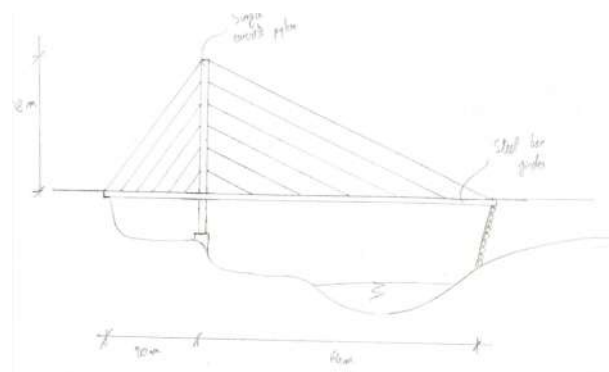


Figure B.7: Side view of concept C cable-stayed bridge

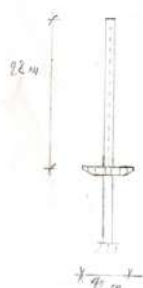


Figure B.8: Transverse view of concept C

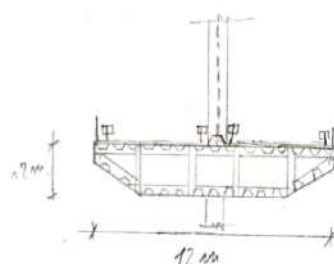


Figure B.9: Deck system of concept C

• FRP cable-stayed bridge concepts

The parameters for this concept are as follows:

- Longitudinal cable arrangement: semi-fan arrangement, with one pair of backing stay cables (see figure B.10). It is not known what the optimal number of cables would be. Transverse cable arrangement: two plane cable arrangement (see figure B.11). The semi-fan arrangement is rather similar to the fan arrangement, structurally speaking, with slightly easier pylon-cable connections. The two plane cable arrangement increases the torsional stiffness of the deck system.
- The bridge is self-anchored. The compressive forces are therefore high at pylon level: buckling might be an issue.

- The deck is pinned at pylon level, and has also torsional restraint at that place. At the abutments, the supports are rollers.
- Two pylons are provided, connected with a transversal connector.
- Two main FRP deck systems can be applied, as illustrated in figures B.12 and B.13. The first option consists of box girders, and a sandwich plate on top. The latter is one large sandwich panel. Considering stiffness is often an issue when designing with FRP, the latter option is probably not optimal due to its low torsional stiffness. To this can be added that damping of FRP is low (around 1%). Dynamic behaviour of the deck can quickly become critical if not designed properly.

Compared to previous concepts, quick erection of the structure is possible, without using heavy (and expensive) machinery.

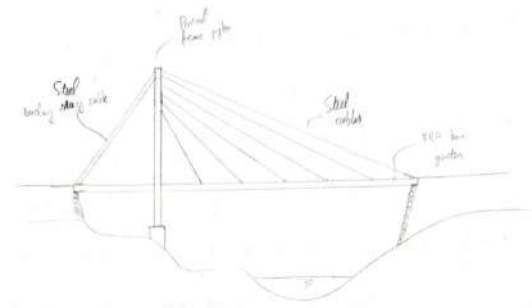


Figure B.10: Side view of FRP cable-stayed bridge concept

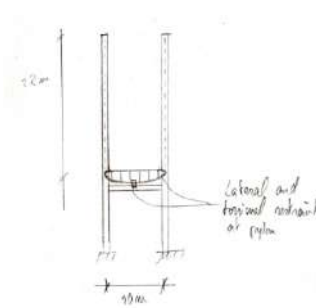


Figure B.11: Transverse view of FRP cable-stayed bridge concept

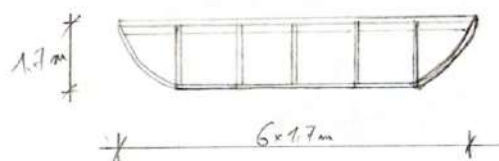


Figure B.12: FRP deck system option 1



Figure B.13: FRP deck system option 2

B.2.2. Truss bridge concepts

A few initial decisions and assumptions were made for the concepts presented below. The bridge will have to be a through or semi-through truss bridge due to the clearance requirement for flooding. From literature [7], the span-to-height ratio can be taken as 10 for the through bridge and 15 for the semi-through bridge, in the case of road bridges. The diagonals have an angle to the horizontal plane varying between 50 and 60 degrees, as recommended.

When it comes to the construction of the structure, it will be considered possible to build from two sides. The concepts will be brought in three segments to the location and lifted to their place. It is assumed the water will be accessible. Two temporary supports will have to be provided. Placing the whole bridge at once is considered as not possible due to lack of space, and too large dimensions.

• Through truss bridge concepts

This type of bridge efficiently carries the loads as material is placed where necessary. Ideally, only compressive forces occur but this is often not completely the case. Several parameters can be varied to obtain an ideal truss bridge: the truss layout (number of trusses, orientation), materials. Two examples are given below to highlight this. Concept 1 (figure B.14) is a

Warren through steel truss bridge, with a composite deck. This is one of the most applied truss layouts. It is simple and rather effective. Concept 2 (figure B.17) is a Pratt through truss bridge, with an FRP deck. Considering the span and the fact that there is no height limitation, applying the through truss concept allows for an efficient material use. With a proper design, a rather lightweight structure can be achieved.

The construction method of this type of bridge was given earlier. The approximate height was estimated to be 6.4 meters, and the length of the segments would be approximately 21 meters. These are very large segments to bring to site. Transport will be costly, as well as erection, considering large cranes will be required. The concrete deck of concept 1 would be cast after the segments are placed. Concept 2 would be entirely prefabricated.

Durability-wise, the applied materials have an important influence. However, some things can be said which does not concern the materials. Through truss bridges have many connections, which usually are bolted or welded. Both require proper execution and are critical in the design. If a durable through truss bridge is to be achieved, the connections will need to be looked at carefully. Inspection can be done rather easily, and should be done on regular intervals.

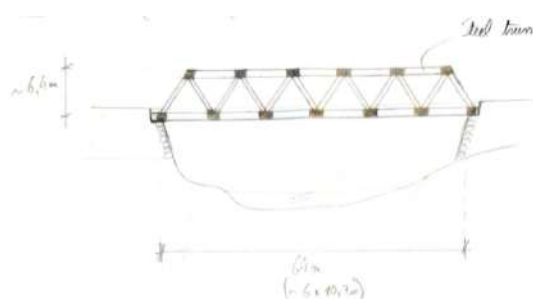


Figure B.14: Side view of Warren through truss bridge, concept 1

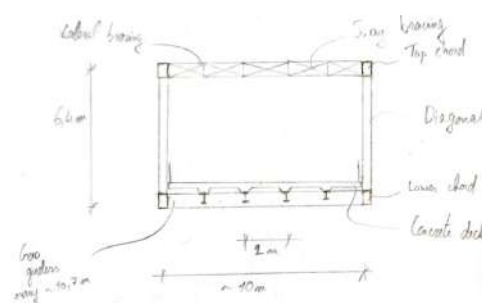


Figure B.15: Cross-sectional view of Warren through truss bridge

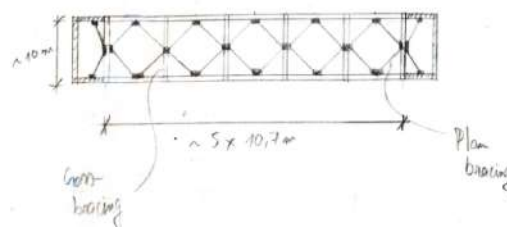


Figure B.16: Top view of Warren through truss bridge

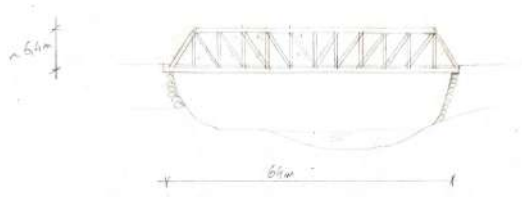


Figure B.17: Side view of Pratt through truss bridge, concept 2

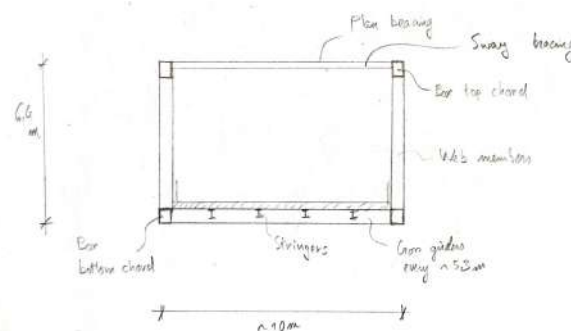


Figure B.18: Cross-sectional view of Pratt through truss bridge

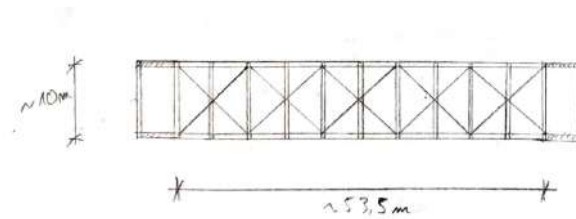


Figure B.19: Top view of Pratt through truss bridge

- Semi-through truss bridge concepts

The main difference with through truss bridges, is the fact that there is no upper flange structure with semi-through truss bridges. This means the torsional rigidity provided by the closed frame in through truss bridges, needs to be provided through a U-frame in this case (see figure B.21). Same as previously, several parameters can be varied which can result in many possible designs. Two examples are given below. Concept 3 is a Warren semi-through truss bridge, with an FRP sandwich panel deck (see figure B.20). Concept 4 is a modified Warren semi-through truss bridge, with an orthotropic steel deck. Compared to the through truss bridge, additional material will be required to reach proper rigidity. The buckling behaviour of the structure will also be critical for the design. The required height might be such that a plan bracing between top chords is still required to prevent buckling. In that case, the structure would no longer be a semi-through system.

The construction method is the same as the through truss bridge. The only difference lies in the approximate total height. For semi-through truss bridges, this would be approximately 4.3 meters, thus reducing by a non negligible amount the size of the prefabricated segments that need to be transported. Nevertheless, in the end, the construction costs are probably the same as for through truss bridges. For both concepts, the deck is already fixed on the transported segments.

When it comes to durability, the same can be said as for through truss bridges. Maybe slightly less connections will be present, due to the lack of a top flange, but this is not very impactful in the end.

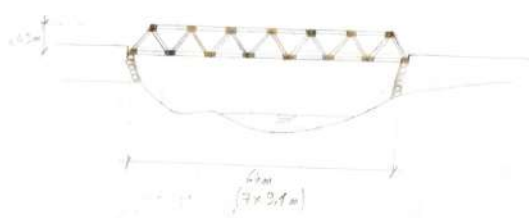


Figure B.20: Side view of Warren semi-through truss bridge, concept 3

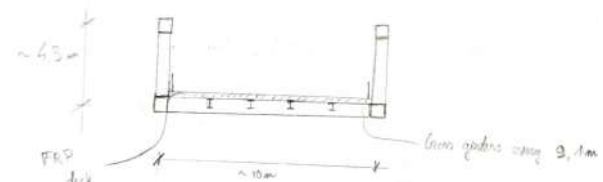


Figure B.21: Cross-sectional view of Warren semi-through truss bridge

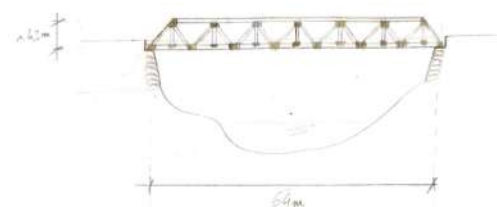


Figure B.22: Side view of Modified Warren semi-through truss bridge, concept 4

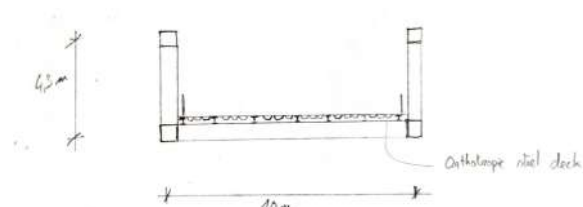


Figure B.23: Cross-sectional view of Warren semi-through truss bridge

B.2.3. Arch bridge concepts

The bridge concepts presented below will all be through arch bridges, in order to keep the highest possible clearance below the deck. Only the most applied arch bridge types will be considered, being the basic arch bridge with horizontal load transmission to the foundations, and the tied arch bridge. The rise of the arch can be estimated from literature. According to the ESDEP lecture [7], the rise-to-span ratio should be between $1/5$ and $1/6$.

The construction method of arch bridges can be as follows:

- Placing the deck in segments first. Temporary supports will be required. Afterwards erection of the arches as segments, as well as the hangers.
- Temporary support tower, supporting the arch bridge with cables, as illustrated in figure B.24. All elements are placed incrementally. Construction would take place from both sides of the span.
- For the basic arch bridge, where the horizontal thrust is transmitted to the foundations, the arches could be built first. If properly anchored to the foundations, the arches will stand by themselves. Afterwards the hangers can be placed, and the deck hung.

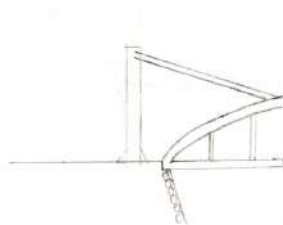


Figure B.24: Temporary support tower for arch bridge

- Basic arch bridge, with horizontal thrust into foundations

An example of this type of bridge is given in figures B.25 and B.26. The arches are made from steel, as is almost always done in modern designs. The arch is for the greatest part under compression (although it also carries bending moments in this situation) and the horizontal loads at its ends are transmitted to the foundations/rocks. This concept could very well be built in this situation because of the presence of rocks on the sides. In other countries where the soil is soft (for example the Netherlands), this concept would not be feasible. The hangers are simple steel cables and the deck is a composite steel-concrete system, where the concrete would be cast-in-situ. For this concept, the arch is the main load-bearing element in the structure and will therefore have large dimensions. The deck can be rather slender and lightweight.

The construction methods given earlier could all be applied here. However, the most cost-effective method would probably be to place the arch first, in three segments (two on the sides at first, and lift the third one in the middle). Once the arch is completed, the hangers can be placed, as well as the deck. This method requires only little temporary supports.

From a durability point of view, the maintenance for such a bridge depends for a great part on the materials applied. For this example, steel and concrete were applied, which most certainly would require only little maintenance if the steel was properly coated and the concrete properly executed. Inspection is rather easy.

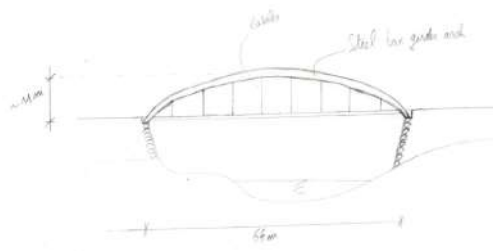


Figure B.25: Side view of the basic arch bridge concept

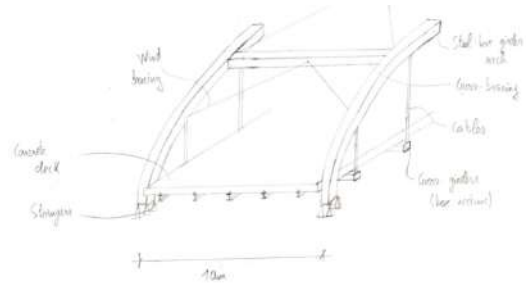


Figure B.26: 3D view of concept A bridge

- Tied arch bridge, with stiffening girder

Tied arch bridges do not transfer horizontal loads from the arch to the supports. These are carried by the stiffening girder (or the deck). That is why, as opposed to the basic arch bridge, the main load-bearing element here is the stiffening girder, which carries bending moments and axial forces. The steel arch is mainly subjected to compressive forces and is therefore more slender. In figures B.27 and B.28, an example is given. In this case, an FRP deck is applied, which is quite beneficial in this case as it will only have to transfer the load to the cross girders.

When it comes to construction of this concept, only 2 of the 3 given methods are applicable. The most cost-effective alternative is most certainly the one where the plate girders and cross girders are placed first on temporary supports. Afterwards the arches are built and finally the hangers are connected to the deck. The FRP deck can be placed as last as segments and fixed to the cross girders.

Same as with the basic arch bridge, the durability of this bridge is for a great part dependent on the materials applied. In this case, the deck is made from FRP. This requires little maintenance. If the steel has been properly protected, the whole structure can be considered to need little maintenance.

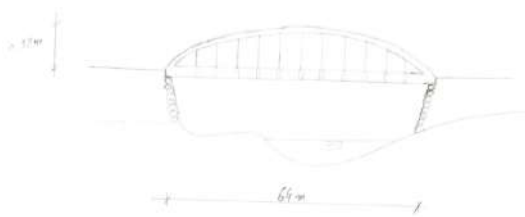


Figure B.27: Side view of the tied arch bridge, with stiffening girder, concept

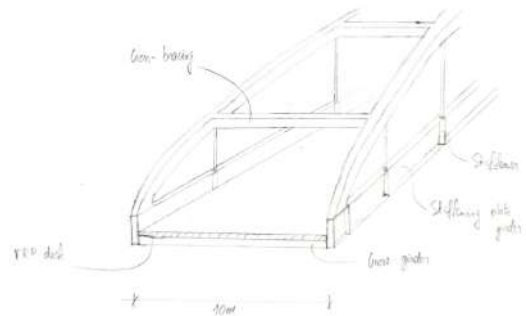


Figure B.28: 3D view of tied arch bridge

B.2.4. Concept comparison and discussion

In order to compare the concepts, a short multi-criteria analysis will be done assessing qualitatively the cost efficiency. Three main criteria will be used, as presented hereafter. A short list of questions is given that indicates what was reflected upon when grading the criteria.

- Materials: What materials are used? Are they used efficiently?
- Construction method: What method is applied? Is heavy/expensive machinery required? Specific measures required to get access to the construction site? Transport and production of elements (cast-in-situ, prefab)? Are temporary supports required?

- Maintenance and durability: The structure has to be designed to have a lifetime of 100 years, with minimal maintenance. What measures are required therefore and are these expensive? Is inspection of the structure easy, and how often should this be done?

The criteria will be graded from -2 to 2 (-2, -1, 0, 1, 2), where -2 indicates that for instance the material use is not cost-effective, or that the construction method is very expensive for that concept. Weight factors were given to the criteria: materials and construction method were considered slightly more important than the other criteria and were therefore given a weight factor of 2 instead of 1. The results are given in the following table B.1.

Criteria	Weight factor	Truss bridge concepts		Arch bridge concepts		Cable-stayed bridge concepts		
		Through truss bridge	Semi-through truss bridge	Basic arch bridge	Tied arch bridge	Concrete	Steel-concrete	Steel FRP
Materials	2	2	1	2	1	1	1	-1
Construction method	2	-1	0	1	0	1	2	1
Maintenance	1	0	0	1	1	1	1	-1
TOTAL	-	2	2	4	3	5	7	6

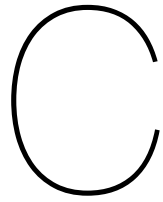
Table B.1: Multi-criteria analysis of concepts

The best scoring concepts are the FRP and the steel-concrete composite cable-stayed bridge. Applying one of these concepts would probably lead to an economical bridge design. The main interest in this thesis is to compare an FRP cable-stayed bridge to another bridge concept with traditional materials. The FRP cable-stayed bridge concept detailed above did score rather well and will form a good basis for a further design. The other bridge concept will be the steel-concrete cable-stayed bridge.

Bridge aesthetics and form-finding were not taken into account in the making of the concepts. The main goal was to look at the materials and the structural system, and the costs that were related to it.

The next step is to further develop the chosen concepts by defining initial dimensions of the different structural elements and connections through simple hand calculations. These first design attempts will be modelled in Sofistik for further analysis.

It must be reminded that this conceptual design study was only performed to get a rough idea of the possibilities for this construction site. The results from the multi-criteria analysis are subjective and are open for debate.



Analytic calculations cable-stayed bridge alternatives

C.1. Initial sizing FRP box girder deck; 7 cables

Input parameters

Deck

$$t_{fl} := 50\text{mm}$$

$$w_{deck} := 10\text{m}$$

$$c_{webs} := 500\text{mm}$$

$$t_{web} := 20\text{mm}$$

$$n_{webs} := \frac{w_{deck}}{c_{webs}} + 1 = 21$$

$$n_t := 5$$

$$h_{core}(h_{deck}) := h_{deck} - 2 \cdot t_{fl}$$

$$t := n_t \cdot t_{web}$$

$$I_y(h_{deck}) := 0.5 \cdot \left[\left[\frac{1}{12} \cdot w_{deck} \cdot t_{fl}^3 + w_{deck} \cdot t_{fl} \cdot \left(\frac{h_{deck}}{2} \right)^2 \right] \cdot 2 + n_{webs} \cdot \left(\frac{1}{12} \cdot t_{web} \cdot h_{deck}^3 \right) \right]$$

$$a_t := w_{deck}$$

$$A_{deck}(h_{deck}) := 0.5 \cdot (2 \cdot t_{fl} \cdot w_{deck} + n_{webs} \cdot t_{web} \cdot h_{deck})$$

$$I_t(h_{deck}) := \frac{2 \cdot t \cdot t_{fl} \cdot (a_t - t)^2 \cdot (h_{deck} - t_{fl})^2}{a_t \cdot t + h_{deck} \cdot t_{fl} - t^2 - t_{fl}^2}$$

$$I_z(h_{deck}) := 2 \cdot \left(\frac{1}{12} \cdot t_{fl} \cdot w_{deck}^3 \right) + 21 \cdot \left(\frac{1}{12} \cdot h_{deck} \cdot t_{web}^3 \right) + (2h_{deck} \cdot t_{web}) \cdot \sum_{j=0}^9 \left(\frac{w_{deck}}{2} - c_{webs} \cdot j \right)^2$$

$$I_p(h_{deck}) := 2I_y(h_{deck}) + I_z(h_{deck})$$

Pylon

$$w_{out,pylon} := 0.8\text{m}$$

$$a_{out,pylon} := 1.15\text{m}$$

$$w_{in,pylon} := 0.6\text{m}$$

$$a_{in,pylon} := 0.8\text{m}$$

$$I_{y,pylon} := \frac{1}{12} \cdot w_{out,pylon} \cdot a_{out,pylon}^3 - \frac{1}{12} \cdot w_{in,pylon} \cdot a_{in,pylon}^3 = 7.579 \times 10^{10} \cdot \text{mm}^4$$

$$A_{pylon} := w_{out,pylon} \cdot a_{out,pylon} - w_{in,pylon} \cdot a_{in,pylon} = 0.44\text{m}^2$$

Geometry bridge

$$l_{ms} := 64\text{m}$$

$$h_{pylon} := 28\text{m}$$

$$l_{ss} := 20\text{m}$$

$$SB := \text{atan}\left(\frac{h_{pylon}}{l_{ss}}\right) = 54.462 \cdot \text{deg}$$

$$nr_{cables} := 7$$

$$h_{pylon.bot} := 8\text{m}$$

$$x_{cable,theoretical} := \frac{l_{ms}}{nr_{cables} + 1} = 8\text{m}$$

$$x_{cable} := 8\text{m}$$

Materials

Anisotropic GFRP, $V_f=50\%$

$$E_1 := 25.8 \text{ GPa} \quad \sigma_1 := 310 \text{ MPa} \quad \text{fibres} := 2570 \frac{\text{kg}}{\text{m}^3}$$

$$E_2 := 15.9 \text{ GPa} \quad \sigma_2 := 191 \text{ MPa} \quad \text{resin} := 120 \frac{\text{kg}}{\text{m}^3}$$

$$G_{12} := 5.6 \text{ GPa} \quad \sigma_{12} := 90 \text{ MPa} \quad \text{total} := 0.5 \cdot \text{fibres} + 0.5 \cdot \text{resin} = 1.345 \times 10^3 \frac{\text{kg}}{\text{m}^3}$$

Steel cables: Parallel strand cables

Steel 1660/1860

$$E_{\text{eff}} := 1.9 \cdot 10^5 \frac{\text{N}}{\text{mm}^2} \quad \text{steel} := 78.5 \frac{\text{kN}}{\text{m}^3}$$

$$f_s := 0.45 \cdot 1860 \frac{\text{N}}{\text{mm}^2} = 837 \cdot \frac{\text{N}}{\text{mm}^2}$$

$$f_s := 200 \frac{\text{N}}{\text{mm}^2}$$

Strands $\Phi 15$: $A_s := 150 \text{ mm}^2$

Concrete pylon C40/50

$$E_{\text{concrete}} := 35 \text{ GPa} \quad \text{concrete} := 24 \frac{\text{kN}}{\text{m}^3}$$

$$f_{c,k} := 40 \text{ MPa}$$

$$f_{ctm} := 3.5 \text{ MPa}$$

Material factors FRP

$$M_1 := 1.15$$

$$M_2 := 1.2$$

Voor sterkte check

$$M_{\text{strength}} := M_1 \cdot M_2 = 1.38$$

Conversion factors FRP

Temperature

$$c_t := 0.9 \quad \text{for strength and deflection}$$

Moisture

$$c_m := 0.9 \quad \text{for strength and deflection. It is supposed that dry and wet conditions alternate.}$$

Creep

$$c_v := 0.7 \quad \text{for strength calculation. As calculated in CUR96, for anisotropic GFRP laminate.}$$

Fatigue

$$c_f := 0.9 \quad \text{for deflection.}$$

Strength

$$c_{\text{strength}} := c_t \cdot c_m \cdot c_v = 0.567$$

Deflection

$$c_{def} := c_t \cdot c_m \cdot c_f = 0.729$$

Loads for one cable plane

$$DL \quad q_{DL} := 12 \frac{kN}{m} \quad \text{Estimate of dead load deck. Height h unknown}$$

$$LL \quad q_{LL} := 14.325 \frac{kN}{m} \quad \text{Taken from EN1991-2. LM1}$$

$$Q_{LL} := 470 kN \quad \text{The live loads for one cable plane are computed with the eccentricities.}$$

Load factors for ULS

$$G := 1.35 \quad \text{for permanent loads}$$

$$Q := 1.35 \quad \text{for variable loads, in this case road traffic.}$$

Calculation of cables, articulated system

Backstay cable

$$l_{backstay} := n_{cables} \cdot x_{cable} = 56 m$$

Dead load

$$V_{DL} := l_{backstay} \cdot G \cdot q_{DL} \cdot \frac{30}{20} = 1.361 \times 10^3 \cdot kN \quad \text{Anchor force, from moment equilibrium about pylon}$$

$$S_{DL} := \frac{V_{DL}}{\sin(\angle SB)} = 1.672 \times 10^3 \cdot kN \quad \text{Force in backstay cable}$$

Live load

$$\max V_{LL} := l_{ms} \cdot Q \cdot q_{LL} \cdot \frac{l_{ms}}{2 \cdot l_{ss}} + Q \cdot Q_{LL} \cdot \frac{l_{ms}}{2 \cdot l_{ss}} = 2.995 \times 10^3 \cdot kN$$

$$\min V_{LL} := -l_{ss} \cdot Q \cdot q_{LL} \cdot \frac{l_{ss}}{2 \cdot l_{ss}} - Q \cdot Q_{LL} \cdot \frac{l_{ss}}{2 \cdot l_{ss}} = -510.637 \cdot kN$$

$$\max S_{LL} := \frac{\max V_{LL}}{\sin(\angle SB)} = 3.681 \times 10^3 \cdot kN \quad \text{Max force in backstay cable from live load}$$

$$\min S_{LL} := \frac{\min V_{LL}}{\sin(\angle SB)} = -627.524 \cdot kN \quad \text{Min force in backstay cable from live load}$$

Sizing of cable

$$A_{tension} := \frac{S_{DL} + \max S_{LL}}{f_s} = 6.396 \times 10^3 \cdot mm^2$$

$$A_{fatigue} := \frac{|\max S_{LL} + \min S_{LL}|}{f_s} = 1.527 \times 10^4 \cdot mm^2 \quad \text{Governing}$$

$$n_{req} := \frac{A_{fatigue}}{A_s} = 102 \quad \text{Chosen nr of strands 12*9}$$

$$n_{actual} := 12 \cdot 9 = 108$$

$$A_{actual.SB} := n_{actual} \cdot A_s = 1.62 \times 10^4 \cdot mm^2$$

Stay cable at critical location

$$d_{S6} := 48\text{m}$$

Distance of critical stay cable to pylon

$$S_6 := \text{atan}\left(\frac{h_{\text{pylon}}}{d_{S6}}\right) = 30.256 \cdot \text{deg}$$

Angle to horizontal plane

$$S_{DL,S6} := \frac{x_{\text{cable}} \cdot Q \cdot q_{DL}}{\sin(S_6)} = 257.209 \cdot \text{kN}$$

Force in cable from DL

$$\max S_{LL,S6} := \frac{x_{\text{cable}} \cdot G \cdot q_{LL} + G \cdot Q_{LL}}{\sin(S_6)} = 1.566 \times 10^3 \cdot \text{kN}$$

Max force in cable from LL

$$\max S_{F,S6} := \max S_{LL,S6} = 1.566 \times 10^3 \cdot \text{kN}$$

Max force in cable for fatigue

Sizing of cable

$$A_{\text{req},S6} := \frac{S_{DL,S6} + \max S_{LL,S6}}{f_s} = 2.179 \times 10^3 \cdot \text{mm}^2$$

$$n_{\text{req},S6} := \frac{A_{\text{req},S6}}{A_s} = 14.524$$

15 strands required. Chosen 4*4 strands.

$$n_{\text{actual},S6} := 16$$

$$A_{\text{actual},S6} := n_{\text{actual},S6} \cdot A_s = 2.4 \times 10^3 \cdot \text{mm}^2$$

Calculation of beam bending moments

Moments due to dead load, rigidly supported system

$$\max M_{\text{rigid},DL} := \frac{G \cdot q_{DL} \cdot x_{\text{cable}}^2}{24} = 43.2 \cdot \text{kN} \cdot \text{m}$$

Moments due to live load, beam on elastic foundation

Determination of elastic length L_{elastic}

A load of 1 MN is applied at the critical anchor point. The deflection due to this load will be determined. From this, the elastic length can be determined, to allow the system to be translated to a beam on an elastic foundation

$$l_{SB} := \frac{l_{ss}}{\cos(S_B)} = 34.409 \text{m}$$

Length of backstay cable

$$l_{S6} := \frac{d_{S6}}{\cos(S_6)} = 55.57 \text{m}$$

Length of stay cable at critical location

$$F := 1\text{MN}$$

Applied unit force

$$S_{SB} := \frac{d_{S6}}{l_{ss}} \cdot \frac{1}{\sin(S_B)} = 2.949 \cdot \frac{\text{MN}}{\text{MN}}$$

Force in backstay cable due to point load F

$$S_{S6} := \frac{1}{\sin(S_6)} = 1.985 \cdot \frac{\text{MN}}{\text{MN}}$$

Force in stay cable 6 due to point load F

$$F := S_{SB}^2 \cdot \frac{l_{SB}^2}{E_{\text{eff}} \cdot A_{\text{actual},SB}} + S_{S6}^2 \cdot \frac{l_{S6}^2}{E_{\text{eff}} \cdot A_{\text{actual},S6}} = 0.577 \cdot \frac{\text{m}}{\text{MN}}$$

Deflection due to unit force

$$c_{\text{elastic}} := \frac{1}{x_{\text{cable}} \cdot F} = 0.217 \cdot \frac{\text{MN}}{\text{m}^2}$$

Elastic support

$$L_{\text{elastic}}(h_{\text{deck}}) := \sqrt[4]{\frac{4 \cdot E_1 \cdot I_y(h_{\text{deck}})}{c_{\text{elastic}}}}$$

Elastic length

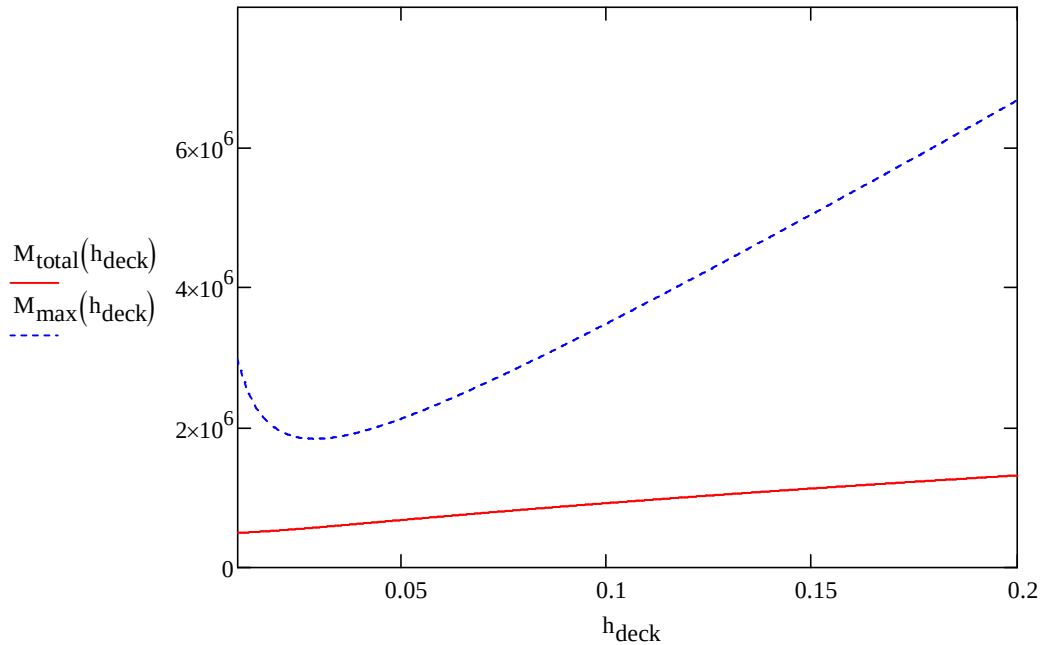
$$M_{\text{LL}}(h_{\text{deck}}) := 0.161 \cdot Q \cdot q_{\text{LL}} \cdot (L_{\text{elastic}}(h_{\text{deck}}))^2 + \frac{1}{4} \cdot Q \cdot Q_{\text{LL}} \cdot L_{\text{elastic}}(h_{\text{deck}})$$

Moment from live load

$$M_{\text{max}}(h_{\text{deck}}) := c_{\text{strength}} \cdot \frac{1}{M_{\text{strength}}} \cdot \frac{I_y(h_{\text{deck}})}{0.5 \cdot h_{\text{deck}}}$$

Maximum allowed moment

$$M_{\text{total}}(h_{\text{deck}}) := M_{\text{LL}}(h_{\text{deck}}) + \max M_{\text{rigid.DL}}$$



Normal forces in beam, per cable plane

From UDL:

$$N_{\text{UDL}} := \frac{G \cdot q_{\text{DL}} + Q \cdot q_{\text{LL}}}{2 \cdot h_{\text{pylon}}} \cdot l_{\text{ms}}^2 = 2.599 \times 10^3 \cdot \text{kN}$$

From concentrated load:

$$N_{\text{CL}} := Q \cdot Q_{\text{LL}} \cdot \frac{1}{\tan\left(\frac{\pi}{6}\right)} = 1.088 \times 10^3 \cdot \text{kN}$$

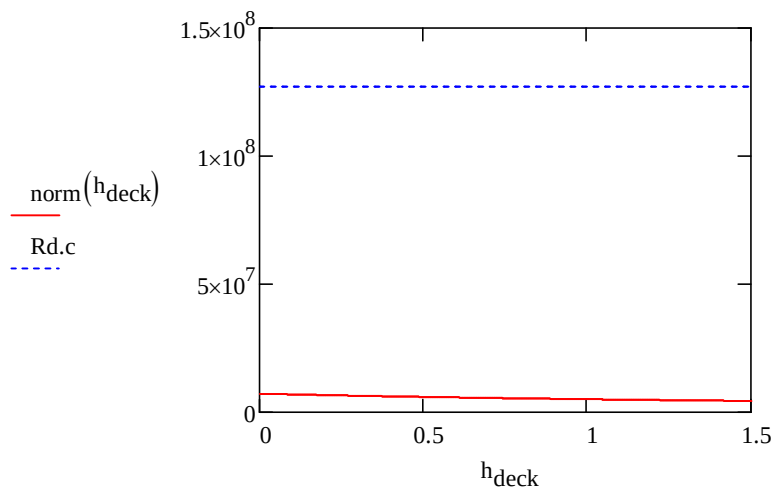
Total normal forces in beam, per cable plane:

$$N_{\text{total}} := N_{\text{UDL}} + N_{\text{CL}} = 3.687 \times 10^3 \cdot \text{kN}$$

Unity check:

$$\text{norm}(h_{\text{deck}}) := \frac{N_{\text{total}}}{A_{\text{deck}}(h_{\text{deck}})}$$

$$R_{d,c} := \frac{c_{\text{strength}} \cdot 1}{M_{\text{strength}}} = 127.37 \cdot \text{MPa}$$



Pylon dimensioning

Normal force in pylon

$$N_{\text{pylon}} := \left(G \cdot q_{\text{DL}} + Q \cdot q_{\text{LL}} \right) \cdot (l_{\text{ss}} + l_{\text{ms}}) \cdot \frac{0.5 \cdot (l_{\text{ss}} + l_{\text{ms}})}{l_{\text{ss}}} + Q \cdot q_{\text{LL}} \cdot \frac{\left(\frac{l_{\text{ms}}}{2} + l_{\text{ss}} \right)}{l_{\text{ss}}} = 7.919 \times 10^3 \cdot \text{kN}$$

Sum of moments about left support. Total normal force in pylon from DL and LL.

Bending moments in pylon, in longitudinal direction

$$S_{\text{B,LL}} := \frac{Q \cdot q_{\text{LL}} \cdot l_{\text{ms}} \cdot \frac{l_{\text{ms}} \cdot 0.5}{l_{\text{ss}}} + Q \cdot q_{\text{LL}} \cdot \frac{0.5 \cdot l_{\text{ms}}}{l_{\text{ss}}}}{\sin(\angle_{\text{SB}}) \cdot 1 \text{ MN}} = 3.681 \cdot \frac{\text{MN}}{\text{MN}}$$

Backstay cable force, from LL

$$S_{\text{B,unit}} := \frac{1 \text{ MN}}{\cos(\angle_{\text{SB}}) \cdot 1 \text{ MN}} = 1.72 \cdot \frac{\text{MN}}{\text{MN}}$$

Backstay cable force from horizontal unit point load, applied at the top of the pylon

$$\text{pylon} := S_{\text{B,LL}} \cdot S_{\text{B,unit}} \cdot \frac{l_{\text{SB}} \cdot 1 \text{ MN}}{E_{\text{eff}} \cdot A_{\text{actual,SB}}} = 70.801 \cdot \text{mm}$$

Deflection at pylon top due to backing stay cable elongation.

$$M_{\text{tower}} := \text{pylon} \cdot \frac{3 \cdot E_{\text{concrete}} \cdot I_{\text{y,pylon}}}{(h_{\text{pylon}} + h_{\text{pylon.bot}})^2} = 434.755 \cdot \text{kN} \cdot \text{m}$$

$$\max_{\text{pylon.bot}} := -\frac{N_{\text{pylon}}}{A_{\text{pylon}}} - \frac{M_{\text{tower}} \cdot 0.5 \cdot a_{\text{out,pylon}}}{I_{\text{y,pylon}}} = -21.295 \cdot \text{MPa}$$

$$\min_{\text{pylon.bot}} := -\frac{N_{\text{pylon}}}{A_{\text{pylon}}} + \frac{M_{\text{tower}} \cdot 0.5 \cdot a_{\text{out,pylon}}}{I_{\text{y,pylon}}} = -14.699 \cdot \text{MPa}$$

$$\text{UC} := \frac{\left| \max_{\text{pylon.bot}} \right|}{\frac{f_{\text{c,k}}}{1.5}} = 0.799$$

Deflection of deck

The strength calculation did not allow proper dimensioning of the deck. Deflection is most probably governing. The deck will be modelled as an infinite beam on elastic foundation. The global deflection will be computed, for the elastic length computed below. Only live loads will be applied. Deflections due to dead load are counteracted by post-tensioning of the stay cables.

$$E_{1,\text{defl}} := c_{\text{def}} \cdot E_1 = 18.808 \cdot \text{GPa}$$

$$G_{12,\text{defl}} := c_{\text{def}} \cdot G_{12} = 4.082 \cdot \text{GPa}$$

$$i := 0..180$$

$$E_{2,defl} := c_{def} \cdot E_2 = 11.591 \cdot \text{GPa}$$

$$h_{deck.5_i,0} := (200 + i \cdot 10) \text{mm}$$

The height of the deck varies from 200 to 2000mm.

$$t_{fl1}(h_{deck}) := 20 \text{mm} + 30 \text{mm} \cdot \frac{(h_{deck} - 200 \text{mm})}{1800 \text{mm}}$$

flange thickness varies between 20 and 50 mm

$$t_{web.5}(h_{deck}) := 10 \text{mm} + 30 \text{mm} \cdot \frac{h_{deck} - 200 \text{mm}}{2000 \text{mm} - 200 \text{mm}}$$

web thickness varies between 10 and 40mm

$$I_{y2}(h_{deck}) := 0.5 \cdot \left[\left[\frac{1}{12} \cdot w_{deck} \cdot t_{fl1}(h_{deck})^3 + w_{deck} \cdot \left[t_{fl1}(h_{deck}) \cdot \left(\frac{h_{deck}}{2} \right)^2 \right] \right] \cdot 2 + n_{webs} \cdot \left(\frac{1}{12} \cdot t_{web.5}(h_{deck}) \cdot h_{deck}^3 \right) \right]$$

$$w_F := S_{SB}^2 \cdot \frac{l_{SB}}{E_{eff} \cdot A_{actual.SB}} + S_{S6}^2 \cdot \frac{l_{S6}}{E_{eff} \cdot A_{actual.S6}}$$

Deflection due to unit force

$$c_{elastic} := \frac{1}{x_{cable} \cdot F} = 0.217 \cdot \frac{N}{\text{mm}^2}$$

Elastic support

$$L_{elastic}(h_{deck}) := \sqrt[4]{\frac{4 \cdot E_{1,defl} \cdot I_{y2}(h_{deck})}{c_{elastic}}}$$

Elastic length

$$(h_{deck}) := \sqrt[4]{\frac{c_{elastic}}{4 \cdot E_{1,defl} \cdot I_{y2}(h_{deck})}}$$

$$w_{Q,LL}(h_{deck}) := \frac{Q_{LL}}{8 \cdot E_{1,defl} \cdot I_{y2}(h_{deck}) \cdot (h_{deck})^3}$$

Deflection due to point load Q_{LL}

$$w_{q,LL}(h_{deck}) := \frac{q_{LL}}{2 \cdot c_{elastic}} \cdot \left(2 - 2 \cdot \cos \left((h_{deck}) \cdot \frac{L_{elastic}(h_{deck})}{2} \right) \cdot e^{-(h_{deck}) \cdot \frac{L_{elastic}(h_{deck})}{2}} \right)$$

Deflection due to UDL q_{LL}

$$w_{LL,total}(h_{deck}) := w_{Q,LL}(h_{deck}) + w_{q,LL}(h_{deck})$$

Total deflection, including conversion factor

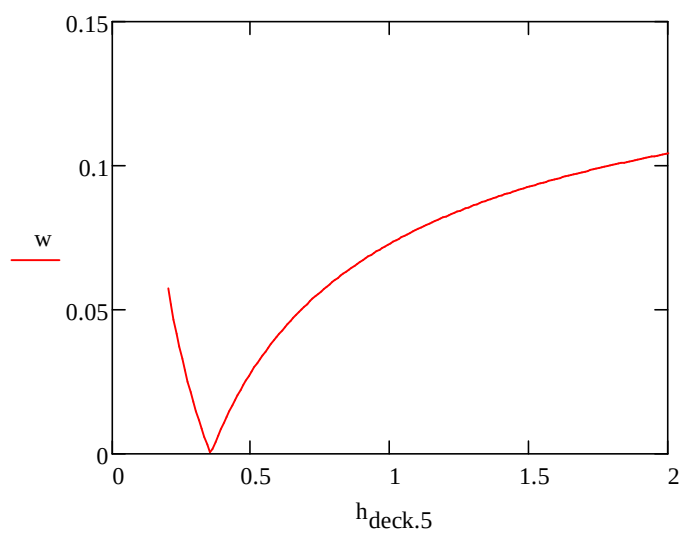
$$w_{max} := \frac{l_{ms}}{350} = 0.183 \text{m}$$

Maximum deflection, as defined in Bruprosjektering

$$w_i := |w_{max} - w_{LL,total}(h_{deck.5_i})|$$

$$h_{S,opt,global} := \text{lookup}(\min(w), w, h_{deck.5}) = (0.35) \text{m}$$

Optimal deck height, for global deflection check



$$d_{\text{cables}} := \begin{pmatrix} 8 \\ 16 \\ 24 \\ 32 \\ 40 \\ 48 \\ 56 \end{pmatrix} \text{m}$$

$$\text{cables} := \text{atan}\left(\frac{h_{\text{pylon}}}{d_{\text{cables}}}\right) \qquad l_{\text{total}} := \frac{d_{\text{cables}}}{\cos(\text{cables})}$$

$$S_{\text{cables}} := \frac{1}{\sin(\text{cables})}$$

$$A_{\text{cables}} := \begin{bmatrix} \frac{(S_{\text{cables}_0})^2}{S_{S6}^2} \cdot \frac{l_{\text{total}_0}}{l_{S6}} \cdot A_{\text{actual.S6}} \\ \frac{(S_{\text{cables}_1})^2}{S_{S6}^2} \cdot \frac{l_{\text{total}_1}}{l_{S6}} \cdot A_{\text{actual.S6}} \\ \frac{(S_{\text{cables}_2})^2}{S_{S6}^2} \cdot \frac{l_{\text{total}_2}}{l_{S6}} \cdot A_{\text{actual.S6}} \\ \frac{(S_{\text{cables}_3})^2}{S_{S6}^2} \cdot \frac{l_{\text{total}_3}}{l_{S6}} \cdot A_{\text{actual.S6}} \\ \frac{(S_{\text{cables}_4})^2}{S_{S6}^2} \cdot \frac{l_{\text{total}_4}}{l_{S6}} \cdot A_{\text{actual.S6}} \\ \frac{(S_{\text{cables}_5})^2}{S_{S6}^2} \cdot \frac{l_{\text{total}_5}}{l_{S6}} \cdot A_{\text{actual.S6}} \\ \frac{(S_{\text{cables}_6})^2}{S_{S6}^2} \cdot \frac{l_{\text{total}_6}}{l_{S6}} \cdot A_{\text{actual.S6}} \end{bmatrix} = \begin{pmatrix} 345.374 \\ 469.078 \\ 701.461 \\ 1.075 \times 10^3 \\ 1.628 \times 10^3 \\ 2.4 \times 10^3 \\ 3.433 \times 10^3 \end{pmatrix} \cdot \text{mm}^2$$

$$D_{\text{cables}} := \sqrt{\frac{4 \cdot A_{\text{cables}}}{\pi}} = \begin{pmatrix} 20.97 \\ 24.439 \\ 29.885 \\ 37 \\ 45.528 \\ 55.279 \\ 66.11 \end{pmatrix} \cdot \text{mm}$$

To check the results, a simple Sofistik model was made, with cable and deck dimensions as computed above. The load was placed at the critical cable. The result is as follows:

$$w_{\text{global.FEM}} := 225.4 \text{ mm} \quad \text{Global deflection, including local deflection}$$

$$w_{\text{loc.FEM.CL}} := 24.1 \text{ mm} \quad \text{Local deflection due to CL}$$

$$w_{\text{loc.FEM.UDL}} := 3.36 \text{ mm} \quad \text{Local deflection due to UDL}$$

$$w_{\text{loc.FEM}} := w_{\text{loc.FEM.CL}} + w_{\text{loc.FEM.UDL}} = 0.027 \text{ m} \quad \text{Total local deflection}$$

The computed deflection in this sheet is:

$$w_{\text{LL.total}}(h_{\text{S.opt.global}}) = 0.184 \text{ m}$$

$$w_{\text{LL.total}}(h_{\text{S.opt.global}}) + w_{\text{loc.FEM}} = 0.211 \text{ m}$$

Comes close to $w_{\text{global.FEM}}$. Difference is mostly due to discretization.

Local deflection check:

$$w_{\text{max.loc}} := \frac{x_{\text{cable}}}{350} = 0.023 \text{ m}$$

$$A_{\text{CL}} := \frac{w_{\text{loc.FEM.CL}}}{\frac{Q_{\text{LL}} \cdot x_{\text{cable}}^3}{E_{1.\text{defl}} \cdot I_{y2}(h_{\text{S.opt.global}})}} = (0.014)$$

$$B_{UDL} := \frac{w_{loc.FEM.UDL}}{\frac{q_{LL} \cdot x_{cable}^4}{E_{1.defl} \cdot I_{y2}(h_{S.opt.global})}} = (7.937 \times 10^{-3})$$

$$w_{loc}(h_{deck}) := B_{UDL} \cdot \frac{q_{LL} \cdot x_{cable}^4}{E_{1.defl} \cdot I_{y2}(h_{deck})} + A_{CL} \cdot \frac{Q_{LL} \cdot x_{cable}^3}{E_{1.defl} \cdot I_{y2}(h_{deck})}$$

$$w_{loc_i} := \left| w_{max.loc} - w_{loc}(h_{deck.5_i}) \right|$$

$$h_{S.opt.loc} := \text{lookup}\left(\min\left(w_{loc}, w_{loc}, h_{deck.5}\right) = (0.38) \text{ m}\right)$$

The final required deck thickness is computed by adding up the local and global deflections with the formulas from above.

$$w_{final}(h_{deck}) := w_{loc}(h_{deck}) + w_{LL.total}(h_{deck})$$

$$w_{final_i} := \left| w_{max} - w_{final}(h_{deck.5_i}) \right|$$

$$h_{opt.global.final} := \text{lookup}\left(\min\left(w_{final}, w_{final}, h_{deck.5}\right) = 0.43 \text{ m}\right)$$

The global deflection is critical. The obtained dimensions are as follows:

$$h_{S.opt_0} := \max(h_{S.opt.loc}, h_{opt.global.final}) = 0.43 \text{ m}$$

$$t_{fl1}(h_{S.opt}) = (23.833) \cdot \text{mm}$$

$$t_{web.5}(h_{S.opt}) = (13.833) \cdot \text{mm}$$

Final cables

$$S_{cables} := \text{atan}\left(\frac{h_{pylon}}{d_{cables}}\right)$$

$$S_{DL.cables} := \frac{x_{cable} \cdot Q \cdot q_{DL}}{\sin(S_{cables})}$$

$$\max S_{LL.cables} := \frac{x_{cable} \cdot G \cdot q_{LL} + G \cdot Q_{LL}}{\sin(S_{cables})}$$

$$A_{req.cables} := \frac{S_{DL.cables} + \max S_{LL.cables}}{f_s} = \begin{pmatrix} 1.142 \times 10^3 \\ 1.264 \times 10^3 \\ 1.446 \times 10^3 \\ 1.667 \times 10^3 \\ 1.914 \times 10^3 \\ 2.179 \times 10^3 \\ 2.455 \times 10^3 \end{pmatrix} \cdot \text{mm}^2$$

Required cables according to strength check

Computation of costs

$$A_{web.5} := \overrightarrow{(t_{web.5}(h_{deck.5}) \cdot h_{deck.5})}$$

$$A_{deck.5} := 2 \cdot t_{fl1}(h_{deck.5}) \cdot w_{deck} + n_{webs} \cdot A_{web.5}$$

$$A_{cable.5} := 2 \cdot A_{req.cables} \quad \text{Cable surface area for one pair of cables}$$

Cable lengths calculation and total volume comparison

$$V_{intermediate} := \overrightarrow{(A_{cable.5} \cdot l_{total})}$$

$$V_{deck.5} := A_{deck.5} \cdot (l_{ms} + l_{ss}) \quad \text{Total deck volume}$$

$$V_{cables.5} := \sum V_{intermediate} = 1.133 \cdot m^3 \quad \text{Total cable volume}$$

$$V_{pylon} := A_{pylon} \cdot (h_{pylon} + h_{pylon.bot}) = 15.84 \cdot m^3$$

Costs comparison

$$\text{FRP deck: } 12 \text{ euros/kg} \quad UP_{FRP} := 12 \frac{\text{€}}{\text{kg}} \quad \text{Pylon :} \quad UP_{pylon} := 700 \frac{\text{€}}{m^3}$$

$$\text{Cables: } 15 \text{ euros/kg} \quad UP_{cables} := 15 \frac{\text{€}}{\text{kg}}$$

$$P_{deck.5} := UP_{FRP} \cdot V_{deck.5} \cdot \text{total}$$

$$P_{pylon_0} := V_{pylon} \cdot UP_{pylon} = 1.109 \times 10^4 \text{ €}$$

$$P_{backstay} := 2l_{SB} \cdot A_{actual.SB} \cdot \frac{\text{steel}}{g} \cdot UP_{cables} = 1.339 \times 10^5 \text{ €}$$

$$P_{cables.5_0} := UP_{cables} \cdot V_{cables.5} \cdot \frac{\text{steel}}{g} + P_{backstay} = 2.699 \times 10^5 \text{ €} \quad \text{Minimum cable price}$$

$$P_{total.5} := P_{deck.5} + P_{cables.5_0} + P_{pylon_0}$$

The results are as follow:

$$P_{deck.opt} := \text{lookup}(h_{S.opt_0}, h_{deck.5}, P_{deck.5}) = (8.156 \times 10^5) \text{ €} \quad \text{Minimum deck price}$$

$$P_{opt} := \text{lookup}(h_{S.opt_0}, h_{deck.5}, P_{total.5}) = (1.097 \times 10^6) \text{ €} \quad \text{Minimum total price}$$

$$h_{S.opt} = (0.43) \text{ m} \quad t_{fl1}(h_{S.opt}) = (23.833) \cdot \text{mm} \quad t_{web.5}(h_{S.opt}) = (13.833) \cdot \text{mm}$$

C.2. Strength checks of chosen FRP box girder deck

Input parameters

Deck

$$\begin{aligned}t_{fl} &:= 24\text{mm} & w_{deck} &:= 10\text{m} & c_{webs} &:= 500\text{mm} & n_t &:= 5 \\t_{web} &:= 14\text{mm} & n_{webs} &:= \frac{w_{deck}}{c_{webs}} + 1 = 21 & t &:= n_t \cdot t_{web} \\h_{S,opt} &:= 430\text{mm} & a_t &:= w_{deck} \\h_{core}(h_{deck}) &:= h_{deck} - 2 \cdot t_{fl} \\I_y(h_{deck}) &:= 0.5 \cdot \left[\left[\frac{1}{12} \cdot w_{deck} \cdot t_{fl}^3 + w_{deck} \cdot t_{fl} \cdot \left(\frac{h_{deck}}{2} \right)^2 \right] \cdot 2 + n_{webs} \cdot \left(\frac{1}{12} \cdot t_{web} \cdot h_{deck}^3 \right) \right] \\A_{deck}(h_{deck}) &:= 0.5 \cdot (2 \cdot t_{fl} \cdot w_{deck} + n_{webs} \cdot t_{web} \cdot h_{deck}) \\I_t(h_{deck}) &:= \frac{2 \cdot t \cdot t_{fl} \cdot (a_t - t)^2 \cdot (h_{deck} - t_{fl})^2}{a_t \cdot t + h_{deck} \cdot t_{fl} - t^2 - t_{fl}^2} \\I_z(h_{deck}) &:= 2 \cdot \left(\frac{1}{12} \cdot t_{fl} \cdot w_{deck}^3 \right) + 21 \cdot \left(\frac{1}{12} \cdot h_{deck} \cdot t_{web}^3 \right) + (2h_{deck} \cdot t_{web}) \cdot \sum_{j=0}^9 \left(\frac{w_{deck}}{2} - c_{webs} \cdot j \right)^2 \\I_p(h_{deck}) &:= 2I_y(h_{deck}) + I_z(h_{deck})\end{aligned}$$

Pylon

$$\begin{aligned}w_{out,pylon} &:= 0.8\text{m} \\a_{out,pylon} &:= 1.0\text{m} \\w_{in,pylon} &:= 0.6\text{m} \\a_{in,pylon} &:= 0.8\text{m} \\I_{y,pylon} &:= \frac{1}{12} \cdot w_{out,pylon} \cdot a_{out,pylon}^3 - \frac{1}{12} \cdot w_{in,pylon} \cdot a_{in,pylon}^3 = 4.107 \times 10^{10} \cdot \text{mm}^4 \\A_{pylon} &:= w_{out,pylon} \cdot a_{out,pylon} - w_{in,pylon} \cdot a_{in,pylon} = 0.32\text{m}^2\end{aligned}$$

Geometry bridge

$$\begin{aligned}l_{ms} &:= 64\text{m} & h_{pylon} &:= 28\text{m} \\l_{ss} &:= 20\text{m} & SB &:= \text{atan}\left(\frac{h_{pylon}}{l_{ss}}\right) = 54.462 \cdot \text{deg} \\nr_{cables} &:= 7 & h_{pylon.bot} &:= 8\text{m} \\x_{cable,theoretical} &:= \frac{l_{ms}}{nr_{cables} + 1} = 8\text{m} \\x_{cable} &:= 8\text{m}\end{aligned}$$

Materials

Anisotropic GFRP, $V_f=50\%$

$$\begin{aligned}
 E_1 &:= 25.8 \text{ GPa} & \sigma_1 &:= 310 \text{ MPa} & \text{fibres} &:= 2570 \frac{\text{kg}}{\text{m}^3} \\
 E_2 &:= 15.9 \text{ GPa} & \sigma_2 &:= 191 \text{ MPa} & \text{resin} &:= 120 \frac{\text{kg}}{\text{m}^3} \\
 G_{12} &:= 5.6 \text{ GPa} & \sigma_{12} &:= 90 \text{ MPa} & \text{total} &:= 0.5 \cdot \text{fibres} + 0.5 \cdot \text{resin} = 1.345 \times 10^3 \frac{\text{kg}}{\text{m}^3}
 \end{aligned}$$

Steel cables: Parallel strand cables

Steel 1660/1860

$$\begin{aligned}
 E_{\text{eff}} &:= 1.9 \cdot 10^5 \frac{\text{N}}{\text{mm}^2} & \text{steel} &:= 78.5 \frac{\text{kN}}{\text{m}^3} \\
 f_s &:= 0.45 \cdot 1860 \frac{\text{N}}{\text{mm}^2} = 837 \cdot \frac{\text{N}}{\text{mm}^2} \\
 f_s &:= 200 \frac{\text{N}}{\text{mm}^2}
 \end{aligned}$$

$$\text{Strands } \Phi 15: \quad A_s := 150 \text{ mm}^2$$

Concrete pylon C40/50

$$\begin{aligned}
 E_{\text{concrete}} &:= 35 \text{ GPa} & \text{concrete} &:= 24 \frac{\text{kN}}{\text{m}^3} \\
 f_{c,k} &:= 40 \text{ MPa} \\
 f_{\text{ctm}} &:= 3.5 \text{ MPa}
 \end{aligned}$$

Material factors FRP

$$M_1 := 1.15$$

$$M_2 := 1.2$$

Voor sterkte check

$$M.\text{strength} := M_1 \cdot M_2 = 1.38$$

Conversion factors FRP

Temperature

$$c_t := 0.9 \quad \text{for strength and deflection}$$

Moisture

$$c_m := 0.9 \quad \text{for strength and deflection. It is supposed that dry and wet conditions alternate.}$$

Creep

$$c_v := 0.7 \quad \text{for strength calculation. As calculated in CUR96, for anisotropic GFRP laminate.}$$

Fatigue

$$c_f := 0.9 \quad \text{for deflection.}$$

Strength

$$c.\text{strength} := c_t \cdot c_m \cdot c_v = 0.567$$

Deflection

$$c_{def} := c_t \cdot c_m \cdot c_f = 0.729$$

Loads for one cable plane

$$DL \quad q_{DL} := A_{deck} \cdot (h_{S,opt}) \cdot \rho_{total} \cdot g + 0.276 \cdot 5 \frac{kN}{m} + 1.2 \frac{kN}{m} = 6.579 \cdot \frac{kN}{m}$$

Includes wearing layer and railings

$$LL \quad q_{LL} := 14.325 \frac{kN}{m}$$

Taken from EN1991-2. LM1

$$Q_{LL} := 470 kN$$

The live loads for one cable plane are computed with the eccentricities.

Load factors for ULS

$$G := 1.35 \quad \text{for permanent loads}$$

$$Q := 1.35 \quad \text{for variable loads, in this case road traffic.}$$

Calculation of cables, articulated system

Backstay cable

$$l_{backstay} := n_{rcables} \cdot x_{cable} = 56 m$$

Dead load

$$V_{DL} := l_{backstay} \cdot G \cdot q_{DL} \cdot \frac{30}{20} = 746.095 \cdot kN$$

Anchor force, from moment equilibrium about pylon

$$S_{DL} := \frac{V_{DL}}{\sin(\angle SB)} = 916.879 \cdot kN$$

Force in backstay cable

Live load

$$\max V_{LL} := l_{ms} \cdot Q \cdot q_{LL} \cdot \frac{l_{ms}}{2 \cdot l_{ss}} + Q \cdot Q_{LL} \cdot \frac{l_{ms}}{2 \cdot l_{ss}} = 2.995 \times 10^3 \cdot kN$$

$$\min V_{LL} := -l_{ss} \cdot Q \cdot q_{LL} \cdot \frac{l_{ss}}{2 \cdot l_{ss}} - Q \cdot Q_{LL} \cdot \frac{l_{ss}}{2 \cdot l_{ss}} = -510.637 \cdot kN$$

$$\max S_{LL} := \frac{\max V_{LL}}{\sin(\angle SB)} = 3.681 \times 10^3 \cdot kN$$

Max force in backstay cable from live load

$$\min S_{LL} := \frac{\min V_{LL}}{\sin(\angle SB)} = -627.524 \cdot kN$$

Min force in backstay cable from live load

Sizing of cable

$$A_{tension} := \frac{S_{DL} + \max S_{LL}}{f_s} = 5.493 \times 10^3 \cdot mm^2$$

$$A_{fatigue} := \frac{|\max S_{LL} + \min S_{LL}|}{f_s} = 1.527 \times 10^4 \cdot mm^2$$

Governing

$$n_{req} := \frac{A_{fatigue}}{A_s} = 102$$

Chosen nr of strands 12*9

$$n_{actual} := 12 \cdot 9 = 108$$

$$A_{actual,SB} := n_{actual} \cdot A_s = 1.62 \times 10^4 \cdot mm^2$$

Stay cable at critical location

$$d_{S6} := 48\text{m}$$

Distance of critical stay cable to pylon

$$S_6 := \text{atan}\left(\frac{h_{\text{pylon}}}{d_{S6}}\right) = 30.256 \cdot \text{deg}$$

Angle to horizontal plane

$$S_{DL,S6} := \frac{x_{\text{cable}} \cdot Q \cdot q_{DL}}{\sin(S_6)} = 141.022 \cdot \text{kN}$$

Force in cable from DL

$$\max S_{LL,S6} := \frac{x_{\text{cable}} \cdot G \cdot q_{LL} + G \cdot Q_{LL}}{\sin(S_6)} = 1.566 \times 10^3 \cdot \text{kN}$$

Max force in cable from LL

$$\max S_{F,S6} := \max S_{LL,S6} = 1.566 \times 10^3 \cdot \text{kN}$$

Max force in cable for fatigue

Sizing of cable

$$A_{\text{req},S6} := \frac{S_{DL,S6} + \max S_{LL,S6}}{f_s} = 2.04 \times 10^3 \cdot \text{mm}^2$$

$$n_{\text{req},S6} := \frac{A_{\text{req},S6}}{A_s} = 13.599$$

14 strands required. Chosen 4*4 strands.

$$n_{\text{actual},S6} := 16$$

$$A_{\text{actual},S6} := n_{\text{actual},S6} \cdot A_s = 2.4 \times 10^3 \cdot \text{mm}^2$$

Calculation of beam bending moments

Moments due to dead load, rigidly supported system

$$\max M_{\text{rigid},DL} := \frac{G \cdot q_{DL} \cdot x_{\text{cable}}^2}{24} = 23.686 \cdot \text{kN} \cdot \text{m}$$

Moments due to live load, beam on elastic foundation

Determination of elastic length L_{elastic}

A load of 1 MN is applied at the critical anchor point. The deflection due to this load will be determined. From this, the elastic length can be determined, to allow the system to be translated to a beam on an elastic foundation

$$l_{SB} := \frac{l_{ss}}{\cos(S_B)} = 34.409 \text{m}$$

Length of backstay cable

$$l_{S6} := \frac{d_{S6}}{\cos(S_6)} = 55.57 \text{m}$$

Length of stay cable nr 6

$$F := 1\text{MN}$$

Applied unit force

$$S_{SB} := \frac{d_{S6}}{l_{ss}} \cdot \frac{1}{\sin(S_B)} = 2.949 \cdot \frac{\text{MN}}{\text{MN}}$$

Force in backstay cable due to point load F

$$S_{S6} := \frac{1}{\sin(S_6)} = 1.985 \cdot \frac{\text{MN}}{\text{MN}}$$

Force in stay cable 6 due to point load F

$$F := S_{SB}^2 \cdot \frac{l_{SB}}{E_{\text{eff}} \cdot A_{\text{actual},SB}} + S_{S6}^2 \cdot \frac{l_{S6}}{E_{\text{eff}} \cdot A_{\text{actual},S6}} = 0.577 \cdot \frac{\text{m}}{\text{MN}}$$

Deflection due to unit force

$$c_{\text{elastic}} := \frac{1}{x_{\text{cable}} \cdot F} = 0.217 \cdot \frac{\text{MN}}{\text{m}^2}$$

Elastic support

$$L_{\text{elastic}}(h_{\text{deck}}) := \sqrt[4]{\frac{4 \cdot E_1 \cdot I_y(h_{\text{deck}})}{c_{\text{elastic}}}}$$

Elastic length

$$M_{\text{LL}}(h_{\text{deck}}) := 0.161 \cdot Q \cdot q_{\text{LL}} \cdot (L_{\text{elastic}}(h_{\text{deck}}))^2 + \frac{1}{4} \cdot Q \cdot Q_{\text{LL}} \cdot L_{\text{elastic}}(h_{\text{deck}}) \quad \text{Moment from live load}$$

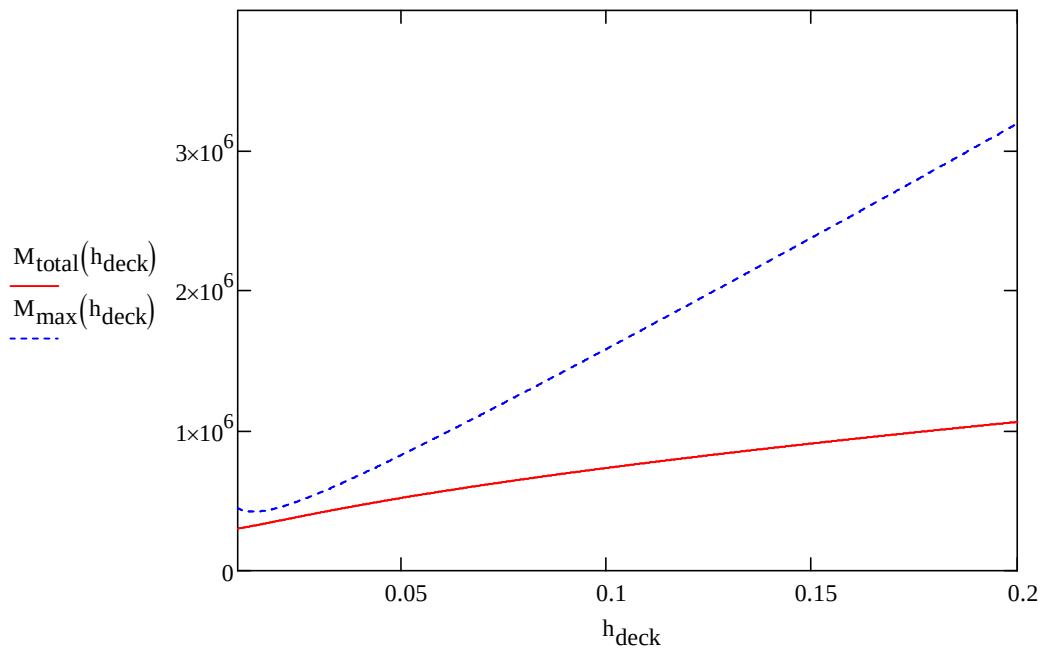
$$M_{\text{max}}(h_{\text{deck}}) := c_{\text{strength}} \cdot \frac{1}{M_{\text{strength}}} \cdot \frac{I_y(h_{\text{deck}})}{0.5 \cdot h_{\text{deck}}} \quad \text{Maximum allowed moment}$$

$$M_{\text{total}}(h_{\text{deck}}) := M_{\text{LL}}(h_{\text{deck}}) + \max M_{\text{rigid.DL}}$$

$$M_{\text{max}}(h_{\text{S.opt}}) = 7.156 \times 10^3 \cdot \text{kN} \cdot \text{m}$$

$$M_{\text{total}}(h_{\text{S.opt}}) = 1.642 \times 10^3 \cdot \text{kN} \cdot \text{m}$$

$$UC_{\text{moments}} := \frac{M_{\text{total}}(h_{\text{S.opt}})}{M_{\text{max}}(h_{\text{S.opt}})} = 0.229$$



Normal forces in beam, per cable plane

From UDL:

$$N_{\text{UDL}} := \frac{G \cdot q_{\text{DL}} + Q \cdot q_{\text{LL}}}{2 \cdot h_{\text{pylon}}} \cdot l_{\text{ms}}^2 = 2.064 \times 10^3 \cdot \text{kN}$$

From concentrated load:

$$N_{\text{CL}} := Q \cdot Q_{\text{LL}} \cdot \frac{1}{\tan\left(\frac{\pi}{6}\right)} = 1.088 \times 10^3 \cdot \text{kN}$$

Total normal forces in beam, per cable plane:

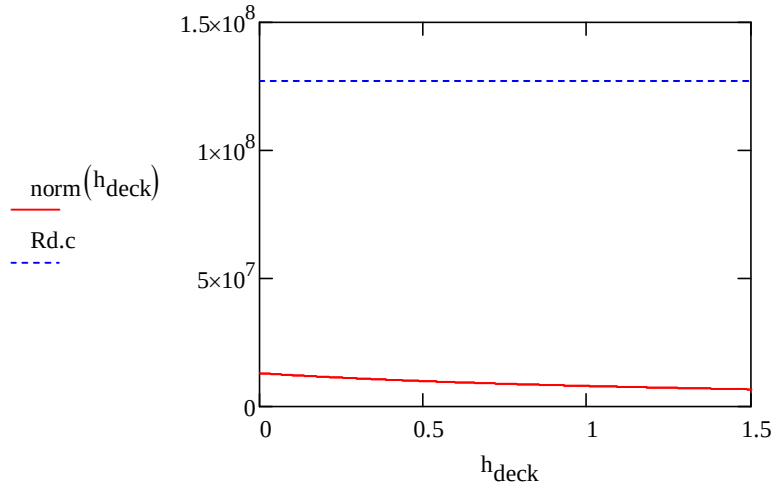
$$N_{\text{total}} := N_{\text{UDL}} + N_{\text{CL}} = 3.152 \times 10^3 \cdot \text{kN}$$

Unity check:

$$\text{norm}(h_{\text{deck}}) := \frac{N_{\text{total}}}{A_{\text{deck}}(h_{\text{deck}})}$$

$$\text{Rd.c} := \frac{\text{c.strength} \cdot 1}{\text{M.strength}} = 127.37 \cdot \text{MPa}$$

$$\text{UC}_{\text{normalforce}} := \frac{\text{norm}(h_{\text{S.opt}})}{\text{Rd.c}} = 0.082$$



Pylon dimensioning

Normal force in pylon

$$N_{\text{pylon}} := \left(G \cdot q_{\text{DL}} + Q \cdot q_{\text{LL}} \right) \cdot (l_{\text{ss}} + l_{\text{ms}}) \cdot \frac{0.5 \cdot (l_{\text{ss}} + l_{\text{ms}})}{l_{\text{ss}}} + Q \cdot Q_{\text{LL}} \cdot \left(\frac{l_{\text{ms}}}{2} + l_{\text{ss}} \right) = 6.628 \times 10^3 \cdot \text{kN}$$

Sum of moments about left support. Total normal force in pylon from DL and LL.

Bending moments in pylon, in longitudinal direction

$$S_{\text{B.LL}} := \frac{Q \cdot q_{\text{LL}} \cdot l_{\text{ms}} \cdot \frac{l_{\text{ms}} \cdot 0.5}{l_{\text{ss}}} + Q \cdot Q_{\text{LL}} \cdot \frac{0.5 \cdot l_{\text{ms}}}{l_{\text{ss}}}}{\sin(\text{SB}) \cdot 1 \text{MN}} = 3.681 \cdot \frac{\text{MN}}{\text{MN}} \quad \text{Backstay cable force, from LL}$$

$$S_{\text{B.unit}} := \frac{1 \text{MN}}{\cos(\text{SB}) \cdot 1 \text{MN}} = 1.72 \cdot \frac{\text{MN}}{\text{MN}}$$

Backstay cable force from horizontal unit point load, applied at the top of the pylon

$$\text{pylon} := S_{\text{B.LL}} \cdot S_{\text{B.unit}} \cdot \frac{l_{\text{SB}} \cdot 1 \text{MN}}{E_{\text{eff}} \cdot A_{\text{actual.SB}}} = 70.801 \cdot \text{mm}$$

Deflection at pylon top due to backing stay cable elongation.

$$M_{\text{tower}} := \text{pylon} \cdot \frac{3 \cdot E_{\text{concrete}} \cdot I_{\text{y.pylon}}}{(h_{\text{pylon}} + h_{\text{pylon.bot}})^2} = 235.566 \cdot \text{kN} \cdot \text{m}$$

$$\max_{\text{pylon.bot}} := -\frac{N_{\text{pylon}}}{A_{\text{pylon}}} - \frac{M_{\text{tower}} \cdot 0.5 \cdot a_{\text{out.pylon}}}{I_{\text{y.pylon}}} = -23.58 \cdot \text{MPa}$$

$$\min_{\text{pylon.bot}} := -\frac{N_{\text{pylon}}}{A_{\text{pylon}}} + \frac{M_{\text{tower}} \cdot 0.5 \cdot a_{\text{out.pylon}}}{I_{\text{y.pylon}}} = -17.844 \cdot \text{MPa}$$

$$\text{UC} := \frac{|\max_{\text{pylon.bot}}|}{\frac{f_{\text{c.k}}}{1.5}} = 0.884$$

Design of cross girders

The cross girders are rectangular hollow sections made from FRP. The second cross girder calculation concerns the backing stay cross girder. The design moments are calculated in Sofistik.

$$h_{cg} := 540\text{mm}$$

$$w_{cg} := 600\text{mm}$$

$$t_{cg,web} := 22\text{mm}$$

$$t_{cg,fl} := 18\text{mm}$$

$$I_{cg,y} := \frac{1}{12} \cdot w_{cg} \cdot t_{cg,fl}^3 \cdot 2 + 2 \cdot w_{cg} \cdot t_{cg,fl} \cdot \left(\frac{h_{cg}}{2} \right)^2 + 2 \cdot \frac{1}{12} \cdot t_{cg,web} \cdot h_{cg}^3 = 2.153 \times 10^{-3} \text{ m}^4$$

$$I_{cg,z} := \frac{1}{12} \cdot h_{cg} \cdot t_{cg,web}^3 \cdot 2 + 2 \cdot h_{cg} \cdot t_{cg,web} \cdot \left(\frac{w_{cg}}{2} \right)^2 + 2 \cdot \frac{1}{12} \cdot t_{cg,fl} \cdot w_{cg}^3 = 2.787 \times 10^{-3} \text{ m}^4$$

$$M_{Rd,y} := c.strength \cdot \frac{1}{M.strength} \cdot \frac{I_{cg,y}}{0.5 \cdot h_{cg}} = 1.015 \times 10^3 \cdot \text{kN} \cdot \text{m}$$

$$M_{Rd,z} := c.strength \cdot \frac{1}{M.strength} \cdot \frac{I_{cg,z}}{0.5 \cdot h_{cg}} = 1.315 \times 10^3 \cdot \text{kN} \cdot \text{m}$$

$$h_{cg} := 420\text{mm}$$

$$w_{cg} := 420\text{mm}$$

$$t_{cg,web} := 30\text{mm}$$

$$t_{cg,fl} := 30\text{mm}$$

$$I_{cg,y} := \frac{1}{12} \cdot w_{cg} \cdot t_{cg,fl}^3 \cdot 2 + 2 \cdot w_{cg} \cdot t_{cg,fl} \cdot \left(\frac{h_{cg}}{2} \right)^2 + 2 \cdot \frac{1}{12} \cdot t_{cg,web} \cdot h_{cg}^3 = 1.484 \times 10^{-3} \text{ m}^4$$

$$I_{cg,z} := \frac{1}{12} \cdot h_{cg} \cdot t_{cg,web}^3 \cdot 2 + 2 \cdot h_{cg} \cdot t_{cg,web} \cdot \left(\frac{w_{cg}}{2} \right)^2 + 2 \cdot \frac{1}{12} \cdot t_{cg,fl} \cdot w_{cg}^3 = 1.484 \times 10^{-3} \text{ m}^4$$

$$M_{Rd,y} := c.strength \cdot \frac{1}{M.strength} \cdot \frac{I_{cg,y}}{0.5 \cdot h_{cg}} = 899.866 \cdot \text{kN} \cdot \text{m}$$

$$M_{Rd,z} := c.strength \cdot \frac{1}{M.strength} \cdot \frac{I_{cg,z}}{0.5 \cdot h_{cg}} = 899.866 \cdot \text{kN} \cdot \text{m}$$

C.3. Aerodynamic checks of chosen FRP box girder deck



Dynamic checks of chosen FRP box girder deck

Computation of first bending and torsional frequencies of deck. Afterwards checks according to Eurocode EN1991-1-4 Annex E for galloping, vortex shedding and flutter.

Wind data

The bridge is to be built in Sirdal, Norway. The wind data is taken from the NA of EN1991-1-4.

$$v_{b,0} := 24 \frac{\text{m}}{\text{s}}$$

$$c_{\text{dir}} := 1.0$$

$$c_{\text{season}} := 1.0$$

$$c_{\text{alt}} := 1.0$$

$$c_{\text{prob}} := 1.0$$

$$v_b := c_{\text{dir}} \cdot c_{\text{season}} \cdot c_{\text{alt}} \cdot c_{\text{prob}} \cdot v_{b,0} = 24 \frac{\text{m}}{\text{s}}$$

The terrain category is taken as III. This results in the following values :

$$k_T := 0.22$$

$$z := 12\text{m}$$

Height of deck above water level

$$z_0 := 0.3\text{m}$$

$$c_T := k_T \cdot \ln\left(\frac{z}{z_0}\right) = 0.812$$

$$c_0 := 1.0$$

$$v_m := c_T \cdot c_0 \cdot v_b = 19.477 \frac{\text{m}}{\text{s}}$$

Frequency computation, according to Svensson book

$$f_b := \frac{1}{2} \cdot \sqrt{\frac{c_{\text{elastic}}}{\frac{q_{\text{DL}}}{g}}} = 2.859 \frac{1}{\text{s}}$$

First bending eigenfrequency of deck

$$m_{\text{deck}} := 2 \cdot \frac{q_{\text{DL}}}{g} = 1.342 \times 10^3 \frac{\text{kg}}{\text{m}}$$

$$t_{\text{web}} := n_t \cdot (t_{\text{web}})$$

$$A_{\text{deck,actual}}(h_{\text{deck}}) := 0.5 \left[2 \cdot (t_{\text{fl}}) \cdot w_{\text{deck}} + n_{\text{webs}} \cdot t_{\text{web}} \cdot h_{\text{deck}} \right]$$

$$I_{y,\text{actual}}(h_{\text{deck}}) := 0.5 \cdot \left[\left[\frac{1}{12} \cdot w_{\text{deck}} \cdot t_{\text{fl}}^3 + w_{\text{deck}} \cdot t_{\text{fl}} \cdot \left(\frac{h_{\text{deck}}}{2} \right)^2 \right] \cdot 2 + n_{\text{webs}} \cdot \left(\frac{1}{12} \cdot t_{\text{web}} \cdot h_{\text{deck}}^3 \right) \right]$$

$$I_{z,actual}(h_{deck}) := 2 \cdot \left[\frac{1}{12} \cdot (t_{fl}) \cdot w_{deck}^3 \right] + 21 \cdot \left(\frac{1}{12} \cdot h_{deck} \cdot t_{web}^3 \right) + [2h_{deck} \cdot (t_{web})] \cdot \sum_{j=0}^9 \left(\frac{w_{deck}}{2} - c_{webs} \cdot j \right)^2$$

$$I_{t,actual}(h_{deck}) := \frac{2 \cdot t \cdot t_{fl} \cdot (a_t - t)^2 \cdot [h_{deck} - (t_{fl})]^2}{a_t \cdot t + h_{deck} \cdot (t_{fl}) - t^2 - t_{fl}^2}$$

$$b_s := 11m$$

$$i_y := \sqrt{\frac{I_{z,actual}(h_{S,opt})}{2 A_{deck,actual}(h_{S,opt})}} = 2.917 m$$

$$f_t := \frac{b_s}{2 \cdot i_y} \cdot f_b = 5.392 \frac{1}{s}$$

First torsional eigenfrequency of deck

Vortex shedding

Ratio of largest to smallest crosswind dimension of structure, both taken in plane perpendicular to the wind, exceeds 6. The critical wind velocity should be computed.

$$\frac{w_{deck}}{h_{S,opt}} = 23.256$$

The safety against vortex shedding cannot be checked. The Strouhal number can not be computed for this cross section with the guidelines given in annex E from EN1991-1-4.

Galloping

Structural damping taken according to CUR96, table 7.1:

$$s := 0.01$$

$$\rho_{air} := 1.25 \frac{kg}{m^3}$$

Scruton number:

$$Sc := \frac{2 \cdot s \cdot m_{deck}}{\rho_{air} \cdot h_{S,opt}^2} = 116.111$$

Factor of galloping instability not given in table E.7 from EN1991-1-4. Therefore upper bound value taken.

$$a_G := 10$$

$$v_{crit,gp} := \frac{2 \cdot Sc}{a_G} \cdot \min(f_t, f_b) \cdot h_{S,opt} = 28.552 \frac{m}{s}$$

$$1.25 \cdot v_m = 24.347 \frac{m}{s}$$

Safe. The critical wind velocity for galloping is larger than 1.25 times the mean wind velocity.

Flutter and divergence

According to EN1991-1-4, all 3 criteria are met. Therefore a velocity check should be performed.

$$f_t = 5.392 \frac{1}{s} \quad f_b = 2.859 \frac{1}{s}$$

$$I_{t,actual}(h_{S,opt}) = 0.077 \text{ m}^4 \qquad G_{12,defl} := G_{12} \cdot c_{,def}$$

$$dcd \; := \left[-6.3 \cdot \left(\frac{h_{S,opt}}{w_{deck}} \right)^2 - 0.38 \cdot \left(\frac{h_{S,opt}}{w_{deck}} \right) + 1.6 \right] = 1.572$$

$$k \; := \frac{^2}{4} \cdot \frac{G_{12,defl} \cdot I_{t,actual}(h_{S,opt})}{l_{ms}^2} = 1.905 \times 10^5 \frac{1}{m} \cdot N \cdot \frac{m}{rad}$$

$$v_{div} := \sqrt{\frac{2 \cdot k}{air \cdot w_{deck}^2 \cdot dcd}} = 44.038 \frac{m}{s} \qquad \qquad \qquad > \qquad \qquad \qquad 2 \cdot v_m = 38.955 \frac{m}{s}$$

Just
safe

According to

Sve

$\overset{mm}{\lambda} := \frac{f_t}{f_b} = 1.886$

Not
safe.

C.4. Initial sizing steel concrete composite cable-stayed bridge; 4 cables

Input parameters

$$n_{\text{cables}} := 4$$

$$x_{\text{cable.th}} := \frac{64\text{m}}{n_{\text{cables}} + 1} = 12.8\text{m}$$

$$x_{\text{cable}} := 12.8\text{m}$$

Materials

Reinforced concrete C40/50, B500

$$E_{\text{cm}} := 35\text{GPa} \quad \rho_{\text{concrete}} := 24 \frac{\text{kN}}{\text{m}^3}$$

$$f_{\text{c.k}} := 40\text{MPa}$$

$$f_{\text{ctm}} := 3.5\text{MPa}$$

With long term
effects:

$$f_{\text{s.k}} := 500\text{MPa}$$

$$E_{\text{c}} := \frac{E_{\text{cm}}}{2} = 1.75 \times 10^{10} \text{ Pa}$$

$$\tau_{\text{R.d}} := 0.42\text{MPa}$$

$$G_{\text{shear}} := 14675\text{MPa}$$

$$c_{\text{nom}} := 35\text{mm}$$

Steel S355

$$E_{\text{steel}} := 210\text{GPa} \quad \rho_{\text{steel}} := 78.5 \frac{\text{kN}}{\text{m}^3}$$

$$f_{\text{y}} := 355\text{MPa}$$

Steel cables: Parallel strand cables

Steel

$$E_{\text{eff}} := 1.9 \cdot 10^5 \frac{\text{N}}{\text{mm}^2}$$

$$f_{\text{s}} := 0.45 \cdot 1860 \frac{\text{N}}{\text{mm}^2} = 837 \cdot \frac{\text{N}}{\text{mm}^2}$$

$$\Delta f_{\text{s}} := 200 \frac{\text{N}}{\text{mm}^2}$$

$$\text{Strands } \Phi 15: \quad A_{\text{s}} := 150\text{mm}^2$$

Concrete pylon C40/50: Same as deck

Deck Full interaction
 assumed

$$w_{\text{deck}} := 10\text{m}$$

$$c_{\text{girders}} := 7\text{m} \quad c_{\text{side}} := \frac{w_{\text{deck}} - c_{\text{girders}}}{2} = 1.5\text{m}$$

$$b_{\text{eff}} := 2 \cdot \frac{0.5 \cdot x_{\text{cable}}}{8} = 1.6\text{m}$$

$$h_{\text{c}} := 350\text{mm}$$

$$n := \frac{E_{\text{steel}}}{E_{\text{c}}} = 12$$

$$A_c := h_c \cdot w_{\text{deck}} = 3.5 \text{ m}^2$$

$$A_{c,\text{eff}} := h_c \cdot b_{\text{eff}} = 0.56 \text{ m}^2$$

$$h_{\text{girder}} := 650 \text{ mm}$$

Minimum reinforcement, as defined in EN1994.2 ; 7.4.2

$$t_{\text{web}} := 15 \text{ mm}$$

$$k_s := 0.9$$

$$\sigma_s := 200 \text{ MPa}$$

$$t_{\text{fl.top}} := 20 \text{ mm}$$

$$k := 0.8$$

$$A_{\text{ct}} := A_{c,\text{eff}}$$

$$t_{\text{fl.bot}} := 20 \text{ mm}$$

$$k_c := 0.751$$

$$w_{\text{fl.top}} := 250 \text{ mm}$$

$$A_b := k_s \cdot k \cdot k_c \cdot \frac{f_{\text{ctm}} \cdot A_{\text{ct}}}{\sigma_s} = 5.299 \times 10^3 \cdot \text{mm}^2$$

$$w_{\text{fl.bot}} := 250 \text{ mm}$$

Pure tension member

$$h_{\text{web}} := h_{\text{girder}} - t_{\text{fl.bot}} - t_{\text{fl.top}} = 0.61 \text{ m}$$

$$\frac{A_b}{2 \cdot b_{\text{eff}}} = 16.56 \frac{1}{\text{m}} \cdot \text{cm}^2$$

Steel girder classification

$$\varepsilon := \left(\sqrt{\frac{235}{355}} \right) = 0.814$$

Web under bending	$\frac{h_{\text{web}}}{t_{\text{web}}} = 40.667$	<	$72 \cdot \varepsilon = 58.58$	Class 1

Flange under compression	$\frac{\frac{w_{\text{fl.bot}}}{2} - \frac{t_{\text{web}}}{2}}{t_{\text{fl.bot}}} = 5.875$	<	$9 \cdot \varepsilon = 7.323$	Class 1

$$A_{\text{girder}} := w_{\text{fl.top}} \cdot t_{\text{fl.top}} + w_{\text{fl.bot}} \cdot t_{\text{fl.bot}} + t_{\text{web}} \cdot h_{\text{web}} = 1.915 \times 10^4 \cdot \text{mm}^2$$

$$A_{\text{eq}} := A_{\text{girder}} + \frac{A_{c,\text{eff}}}{n} = 0.066 \text{ m}^2$$

$$z_{\text{NA.girder}} := \frac{w_{\text{fl.bot}} \cdot t_{\text{fl.bot}} \cdot \left(h_{\text{girder}} - \frac{t_{\text{fl.bot}}}{2} \right) + h_{\text{web}} \cdot t_{\text{web}} \cdot \left(\frac{h_{\text{web}}}{2} + t_{\text{fl.top}} \right) + w_{\text{fl.top}} \cdot t_{\text{fl.top}} \cdot \frac{t_{\text{fl.top}}}{2}}{A_{\text{girder}}} = 0.325 \text{ m}$$

Neutral axis position for steel girder only, from top fibre level

$$z_{\text{steel.top}} := z_{\text{NA.girder}} + h_c = 0.675 \text{ m}$$

Position neutral axis steel girder to slab top fibre level

$$z_{\text{concrete.top}} := \frac{h_c}{2} = 0.175 \text{ m}$$

Position neutral axis concrete slab to top fibre level

$$S_{\text{girder}} := A_{\text{girder}} \cdot z_{\text{steel.top}} = 0.013 \cdot \text{m}^3$$

First moment of inertia of girder to top fibre level

$$S_{\text{concrete}} := A_{c,\text{eff}} \cdot z_{\text{concrete.top}} = 0.098 \cdot \text{m}^3$$

First moment of inertia of concrete slab to top fibre level

$$S_{\text{eq}} := S_{\text{girder}} + \frac{S_{\text{concrete}}}{n} = 0.021 \cdot \text{m}^3$$

First moment of inertia of equivalent all steel section

$$z_{\text{NA.eq}} := \frac{S_{\text{eq}}}{A_{\text{eq}}} = 0.32 \text{ m}$$

Position neutral axis for equivalent all steel section, from top fibre level

$$z_{\text{NA.eq.steel}} := z_{\text{steel.top}} - z_{\text{NA.eq}} = 0.355 \text{ m}$$

Position neutral axis for eq all steel section, for steel

$$z_{NA,eq.concrete} := z_{NA,eq} - \frac{h_c}{2} = 0.145 \text{ m}$$

girder
Position neutral for eq all steel section, for concrete slab

$$I_{girder} := \frac{1}{12} \cdot w_{fl.bot} \cdot t_{fl.bot}^3 + \left(h_{girder} - z_{NA,girder} - \frac{t_{fl.bot}}{2} \right)^2 \cdot w_{fl.bot} \cdot t_{fl.bot} + \frac{1}{12} \cdot t_{web} \cdot h_{web}^3 + \left(z_{NA,girder} - \frac{h_{web}}{2} - t_{fl.top} \right)^2 \cdot t_{web}$$

Moment of inertia steel
girder (wont fit)

$$I_{concrete} := \frac{1}{12} \cdot b_{eff} \cdot h_c^3 = 5.717 \times 10^{-3} \text{ m}^4$$

Moment of inertia concrete
slab

$$I_{eq.girder} := I_{girder} + A_{girder} \cdot z_{NA,eq.steel}^2 = 3.683 \times 10^{-3} \text{ m}^4$$

Moment of inertia steel girder in equivalent steel
section

$$I_{eq.concrete} := I_{concrete} + A_{c,eff} \cdot z_{NA,eq.concrete}^2 = 0.018 \text{ m}^4$$

Moment of inertia concrete slab in equivalent steel
section

$$I_{eq.total} := I_{eq.girder} + \frac{I_{eq.concrete}}{n} = 5.147 \times 10^{-3} \text{ m}^4$$

Moment of inertia of equivalent steel
section.

Pylon

$$w_{out.pylon} := 1 \text{ m}$$

$$a_{out.pylon} := 1.3 \text{ m}$$

$$w_{in.pylon} := 0.64 \text{ m}$$

$$a_{in.pylon} := 0.8 \text{ m}$$

$$I_{y.pylon} := \frac{1}{12} \cdot w_{out.pylon} \cdot a_{out.pylon}^3 - \frac{1}{12} \cdot w_{in.pylon} \cdot a_{in.pylon}^3 = 1.558 \times 10^{11} \cdot \text{mm}^4$$

$$A_{pylon} := w_{out.pylon} \cdot a_{out.pylon} - w_{in.pylon} \cdot a_{in.pylon} = 0.788 \text{ m}^2$$

Geometry bridge

$$l_{ms} := 64 \text{ m}$$

$$h_{pylon} := 28 \text{ m}$$

$$l_{ss} := 20 \text{ m}$$

$$\alpha_{SB} := \text{atan}\left(\frac{h_{pylon}}{l_{ss}}\right) = 54.462 \cdot \text{deg}$$

$$l_{cables} := x_{cable} \cdot n_{cables} = 51.2 \text{ m}$$

$$h_{pylon.bot} := 8 \text{ m}$$

Loads for one cable plane

$$\text{DL} \quad q_{selfweight} := 0.5 \left(\rho_{concrete} \cdot A_c + \rho_{steel} \cdot A_{girder} \cdot 2 \right) = 43.503 \cdot \frac{\text{kN}}{\text{m}}$$

$$\rho_{additional} := 25 \frac{\text{kN}}{\text{m}^3} \quad t_{additional} := 0.015 \text{ m}$$

$$q_{additional} := \rho_{additional} \cdot t_{additional} \cdot 4.5 \text{ m} = 1.688 \cdot \frac{\text{kN}}{\text{m}}$$

$$q_{DL} := q_{selfweight} + q_{additional} = 45.191 \cdot \frac{\text{kN}}{\text{m}}$$

$$\text{LL} \quad q_{LL} := 14.325 \frac{\text{kN}}{\text{m}}$$

Taken from EN1991-2.
LM1

$$Q_{LL} := 470 \text{ kN}$$

Load factors

$\gamma_G := 1.35$	for permanent loads
$\gamma_Q := 1.35$	for variable loads, in this case road traffic.

Calculation of cables, articulated system

Backstay cable

$l_{\text{backstay}} := 60\text{m}$ Length of main span carried by backstay cable

Dead load

$$V_{DL} := l_{\text{backstay}} \cdot \gamma_G \cdot q_{DL} \cdot \frac{30}{20} = 5.491 \times 10^3 \cdot \text{kN} \quad \text{Anchor force, from moment equilibrium about pylon}$$

$$S_{DL} := \frac{V_{DL}}{\sin(\alpha_{SB})} = 6.748 \times 10^3 \cdot \text{kN} \quad \text{Force in backstay cable}$$

Live load

$$\max V_{LL} := l_{ms} \cdot \gamma_Q \cdot q_{LL} \cdot \frac{l_{ms}}{2 \cdot l_{ss}} + \gamma_Q \cdot Q_{LL} \cdot \frac{l_{ms}}{2 \cdot l_{ss}} = 2.995 \times 10^3 \cdot \text{kN}$$

$$\min V_{LL} := -l_{ss} \cdot \gamma_Q \cdot q_{LL} \cdot \frac{l_{ss}}{2 \cdot l_{ss}} - \gamma_Q \cdot Q_{LL} \cdot \frac{l_{ss}}{2 \cdot l_{ss}} = -510.637 \cdot \text{kN}$$

$$\max S_{LL} := \frac{\max V_{LL}}{\sin(\alpha_{SB})} = 3.681 \times 10^3 \cdot \text{kN} \quad \text{Max force in backstay cable from live load}$$

$$\min S_{LL} := \frac{\min V_{LL}}{\sin(\alpha_{SB})} = -627.524 \cdot \text{kN} \quad \text{Min force in backstay cable from live load}$$

Sizing of

$$A_{\text{tension}} := \frac{S_{DL} + \max S_{LL}}{f_s} = 1.246 \times 10^4 \cdot \text{mm}^2$$

$$A_{\text{fatigue}} := \frac{|\max S_{LL} + \min S_{LL}|}{\Delta f_s} = 1.527 \times 10^4 \cdot \text{mm}^2 \quad \text{Governing}$$

$$n_{\text{req}} := \frac{A_{\text{fatigue}}}{A_s} = 102 \quad \text{Chosen nr of strands } 12 \times 9$$

$$n_{\text{actual}} := 12 \cdot 9 = 108$$

$$A_{\text{actual.SB}} := n_{\text{actual}} \cdot A_s = 1.62 \times 10^4 \cdot \text{mm}^2$$

Stay cable at critical location

$$d_{S6} := 38.4\text{m} \quad \text{Distance to pylon of stay cable at critical location}$$

$$\alpha_{S6} := \text{atan}\left(\frac{h_{\text{pylon}}}{d_{S6}}\right) = 36.098 \cdot \text{deg} \quad \text{Angle to horizontal plane}$$

$$S_{DL.S6} := \frac{x_{\text{cable}} \cdot \gamma_G \cdot q_{DL}}{\sin(\alpha_{S6})} = 1.325 \times 10^3 \cdot \text{kN} \quad \text{Force in cable from DL}$$

$$\max S_{LL.S6} := \frac{x_{cable} \cdot \gamma_Q \cdot q_{LL} + \gamma_Q \cdot Q_{LL}}{\sin(\alpha_{S6})} = 1.497 \times 10^3 \cdot \text{kN} \quad \text{Max force in cable from LL}$$

$$\max \Delta S_{F.S6} := \max S_{LL.S6} = 1.497 \times 10^3 \cdot \text{kN} \quad \text{Max force in cable for fatigue}$$

Sizing of

$$A_{req.S6} := \frac{S_{DL.S6} + \max S_{LL.S6}}{f_s} = 3.372 \times 10^3 \cdot \text{mm}^2$$

$$n_{req.S6} := \frac{A_{req.S6}}{A_s} = 22.481 \quad \text{23 strands required. Chosen 2*12 strands.}$$

$$n_{actual.S6} := 24$$

$$A_{actual.S6} := n_{actual.S6} \cdot A_s = 3.6 \times 10^3 \cdot \text{mm}^2$$

Calculation of beam bending moments

Moments due to dead load, rigidly supported

system

$$\max M_{rigid.DL} := \frac{\gamma_G \cdot q_{DL} \cdot x_{cable}^2}{24} = 416.478 \cdot \text{kN} \cdot \text{m}$$

Moments due to live load, beam on elastic foundation

Determination of elastic length $L_{elastic}$

A load of 1 MN is applied at the critical cable anchor point. The deflection due to this load will be determined. From this, the elastic length can be determined.

$$l_{SB} := \frac{l_{ss}}{\cos(\alpha_{SB})} = 34.409 \text{ m} \quad \text{Length of backstay cable}$$

$$l_{S6} := \frac{d_{S6}}{\cos(\alpha_{S6})} = 47.524 \text{ m} \quad \text{Length of stay cable at critical location}$$

$$F_{\delta} := 1 \text{ MN} \quad \text{Applied unit force}$$

$$S_{SB} := \frac{d_{S6}}{l_{ss}} \cdot \frac{1}{\sin(\alpha_{SB})} = 2.359 \cdot \frac{\text{MN}}{\text{MN}} \quad \text{Force in backstay cable due to point load F}$$

$$S_{S6} := \frac{1}{\sin(\alpha_{S6})} = 1.697 \cdot \frac{\text{MN}}{\text{MN}} \quad \text{Force in stay cable 6 due to point load F}$$

$$\delta_F := S_{SB}^2 \cdot \frac{l_{SB}^2}{E_{eff} \cdot A_{actual.SB}} + S_{S6}^2 \cdot \frac{l_{S6}^2}{E_{eff} \cdot A_{actual.S6}} = 0.262 \cdot \frac{\text{m}}{\text{MN}} \quad \text{Deflection due to unit force}$$

$$c_{elastic} := \frac{1}{x_{cable} \cdot \delta_F} = 0.298 \cdot \frac{\text{MN}}{\text{m}^2} \quad \text{Elastic support}$$

$$L_{elastic} := \sqrt[4]{\frac{4 \cdot E_{steel} \cdot I_{eq.total}}{c_{elastic}}} = 10.978 \text{ m} \quad \text{Elastic length}$$

$$M_{LL} := 0.161 \cdot \gamma_Q \cdot q_{LL} \cdot L_{elastic}^2 + \frac{1}{4} \cdot \gamma_Q \cdot Q_{LL} \cdot L_{elastic} = 2.117 \times 10^3 \cdot \text{kN} \cdot \text{m} \quad \text{Moment from live load}$$

$$M_{\text{total}} := M_{\text{LL}} + \max M_{\text{rigid.DL}} = 2.533 \times 10^3 \cdot \text{kN} \cdot \text{m}$$

Total moment from DL and LL

Elastic
solution

$$\sigma_{\text{bottom}} := \frac{M_{\text{total}}}{I_{\text{eq.total}}} \cdot (h_{\text{girder}} + h_{\text{c}} - z_{\text{NA.eq}}) = 334.397 \cdot \text{MPa}$$

$$UC_{\text{bot.elastic}} := \frac{\sigma_{\text{bottom}}}{\frac{f_y}{1.0}} = 0.942$$

$$\sigma_{\text{top}} := \frac{1}{n} \cdot \frac{M_{\text{total}}}{I_{\text{eq.total}}} \cdot (-z_{\text{NA.eq}}) = -13.143 \cdot \text{MPa}$$

$$UC_{\text{top.elastic}} := \frac{|\sigma_{\text{top}}|}{\frac{f_{\text{c,k}}}{1.5}} = 0.493$$

Normal forces in beam, per cable plane

From UDL:

$$N_{\text{UDL}} := \frac{\gamma_G \cdot q_{\text{DL}} + \gamma_Q \cdot q_{\text{LL}}}{2 \cdot h_{\text{pylon}}} \cdot l_{\text{ms}}^2 = 5.877 \times 10^3 \cdot \text{kN}$$

From concentrated load:

$$N_{\text{CL}} := \gamma_Q \cdot Q_{\text{LL}} \cdot \frac{1}{\tan(\alpha_{\text{S6}})} = 870.171 \cdot \text{kN}$$

Total normal forces in beam, per cable plane:

$$N_{\text{total}} := N_{\text{UDL}} + N_{\text{CL}} = 6.747 \times 10^3 \cdot \text{kN}$$

Occurs at
pylon

$$\sigma_{\text{comp}} := \frac{N_{\text{total}}}{0.5 \cdot A_{\text{c}}} = 3.855 \cdot \text{MPa}$$

$$UC_{\text{comp}} := \frac{\sigma_{\text{comp}}}{\frac{f_{\text{c,k}}}{1.5}} = 0.145$$

Safety against buckling:

$$P_k := 2 \cdot \sqrt{E_{\text{steel}} \cdot I_{\text{eq.total}} \cdot c_{\text{elastic}}} = 3.588 \times 10^4 \cdot \text{kN}$$

Taken from Svensson book. For a beam on elastic supports.

$$UC_{\text{buckling}} := \frac{N_{\text{total}}}{P_k} = 0.188$$

Pylon dimensioning

Normal force in pylon

$$N_{\text{pylon}} := (\gamma_G \cdot q_{\text{DL}} + \gamma_Q \cdot q_{\text{LL}}) \cdot (l_{\text{ss}} + l_{\text{ms}}) \cdot \frac{0.5 \cdot (l_{\text{ss}} + l_{\text{ms}})}{l_{\text{ss}}} + \gamma_Q \cdot Q_{\text{LL}} \cdot \left(\frac{\frac{l_{\text{ms}}}{2} + l_{\text{ss}}}{l_{\text{ss}}} \right) = 1.582 \times 10^4 \cdot \text{kN}$$

Sum of moments about left support. Total normal force in pylon from DL and LL.

Bending moments in pylon, in longitudinal direction

$$S_{B.LL} := \frac{\gamma_Q \cdot q_{LL} \cdot l_{ms} \cdot \frac{l_{ms} \cdot 0.5}{l_{ss}} + \gamma_Q \cdot Q_{LL} \cdot \frac{0.5 \cdot l_{ms}}{l_{ss}}}{\sin(\alpha_{SB}) \cdot 1 \text{ MN}} = 3.681 \cdot \frac{\text{MN}}{\text{MN}}$$

Backstay cable force, from LL

$$S_{B.unit} := \frac{1 \text{ MN}}{\cos(\alpha_{SB}) \cdot 1 \text{ MN}} = 1.72 \cdot \frac{\text{MN}}{\text{MN}}$$

Backstay cable force from horizontal unit point load, applied at the top of the pylon

$$\delta_{pylon} := S_{B.LL} \cdot S_{B.unit} \cdot \frac{l_{SB} \cdot 1 \text{ MN}}{E_{eff} \cdot A_{actual.SB}} = 70.801 \cdot \text{mm}$$

Deflection at pylon top due to backing stay cable elongation.

$$M_{tower} := \delta_{pylon} \cdot \frac{3 \cdot E_{cm} \cdot I_{y.pylon}}{(h_{pylon} + h_{pylon.bot})^2} = 893.563 \cdot \text{kN} \cdot \text{m}$$

$$\max \sigma_{pylon.bot} := -\frac{N_{pylon}}{A_{pylon}} - \frac{M_{tower} \cdot 0.5 \cdot a_{out.pylon}}{I_{y.pylon}} = -23.808 \cdot \text{MPa}$$

$$\min \sigma_{pylon.bot} := -\frac{N_{pylon}}{A_{pylon}} + \frac{M_{tower} \cdot 0.5 \cdot a_{out.pylon}}{I_{y.pylon}} = -16.351 \cdot \text{MPa}$$

$$UC := \frac{|\max \sigma_{pylon.bot}|}{\frac{f_{c.k}}{1.5}} = 0.893$$

Calculation of concrete deck, shear check. EN 1992-1 ; 6.2.2

$$Q_{LL.loc} := 192 \frac{\text{kN}}{\text{m}}$$

$$q_{concrete.DL} := 2 \text{ m} \cdot h_c \cdot \rho_{concrete} + \rho_{additional} \cdot t_{additional} \cdot 2 \text{ m} = 17.55 \frac{1}{\text{m}} \cdot \text{kN}$$

$$V_{E.d} := \gamma_G \cdot q_{concrete.DL} \cdot \frac{1 \text{ m}}{1 \text{ m}} + \gamma_Q \cdot \frac{(Q_{LL.loc} \cdot 1 \text{ m})}{1 \text{ m}} = 282.892 \frac{1}{\text{m}} \cdot \text{kN}$$

Shear force per unit length

$$b_w := 1 \text{ m}$$

$$C_{Rd.c} := \frac{0.18}{1.5} = 0.12$$

$$k := 1 + \sqrt{\frac{200 \text{ mm}}{h_c}} = 1.756$$

Should be smaller or equal than 2.

$$\rho_l := \frac{A_b}{b_w \cdot h_c} = 0.015$$

Should be smaller or equal to 0.02

$$V_{Rd.c} := C_{Rd.c} \cdot k \cdot \left(100 \cdot \rho_l \cdot \frac{f_{c.k}}{\text{MPa}} \right)^{\frac{1}{3}} \cdot \frac{b_w \cdot h_c}{\text{mm}^2} \cdot \text{N} = 289.614 \cdot \text{kN}$$

Sufficient

$$v_{min} := 0.035 \cdot k^{\frac{3}{2}} \cdot \left(\frac{f_{c.k}}{\text{MPa}} \right)^{0.5} = 0.515$$

$$V_{Rd.c.min} := v_{min} \cdot \frac{b_w \cdot h_c}{\text{mm}^2} \cdot \text{N} = 180.271 \cdot \text{kN}$$

Deflection of deck

$$\beta := \sqrt[4]{\frac{c_{\text{elastic}}}{4 \cdot E_{\text{steel}} \cdot I_{\text{eq.total}}}}$$

$$w_{Q,LL} := \frac{Q_{LL}}{8 \cdot E_{\text{steel}} \cdot I_{\text{eq.total}} \cdot \beta^3}$$

$$w_{q,LL} := \frac{q_{LL}}{2 \cdot c_{\text{elastic}}} \cdot \left(2 - 2 \cdot \cos\left(\beta \cdot \frac{L_{\text{elastic}}}{2}\right) \cdot e^{-\beta \cdot \frac{L_{\text{elastic}}}{2}} \right)$$

$$w_{LL,\text{total}} := w_{Q,LL} + w_{q,LL} = 0.094 \text{ m}$$

$$w_{\text{max}} := \frac{l_{\text{ms}}}{350} = 0.183 \text{ m}$$

$$UC_{\text{defl}} := \frac{w_{LL,\text{total}}}{w_{\text{max}}} = 0.516$$

Cost estimation of bridge

Cables: total weight of cables
required

$$d_{\text{cables}} := \begin{pmatrix} 12.8 \\ 25.6 \\ 38.4 \\ 51.2 \end{pmatrix} \text{ m}$$

$$\alpha_{S,\text{cables}} := \text{atan}\left(\frac{h_{\text{pylon}}}{d_{\text{cables}}}\right)$$

$$S_{DL,\text{cables}} := \frac{x_{\text{cable}} \cdot \gamma_Q \cdot q_{DL}}{\sin(\alpha_{S,\text{cables}})}$$

$$l_{\text{total}} := \frac{d_{\text{cables}}}{\cos(\alpha_{S,\text{cables}})}$$

$$\max S_{LL,\text{cables}} := \frac{x_{\text{cable}} \cdot \gamma_G \cdot q_{LL} + \gamma_G \cdot Q_{LL}}{\sin(\alpha_{S,\text{cables}})}$$

$$A_{\text{req.cables}} := \frac{S_{DL,\text{cables}} + \max S_{LL,\text{cables}}}{f_s} = \begin{pmatrix} 2.185 \times 10^3 \\ 2.692 \times 10^3 \\ 3.372 \times 10^3 \\ 4.141 \times 10^3 \end{pmatrix} \cdot \text{mm}^2 \quad \text{Required cables according to strength check}$$

$$A_{\text{cable.ms}} := 2 \cdot A_{\text{req.cables}}$$

$$A_{\text{cable.ss}} := 2 \cdot A_{\text{actual.SB}} = 0.032 \text{ m}^2$$

$$V_{\text{intermediate}} := \overrightarrow{(A_{\text{cable.ms}} \cdot l_{\text{total}})}$$

$$V_{\text{cable.ms}} := \sum V_{\text{intermediate}} = 1.143 \cdot \text{m}^3$$

Total cable volume main span

$$V_{\text{cable.ss}} := A_{\text{cable.ss}} \cdot l_{\text{SB}} = 1.115 \cdot \text{m}^3$$

Total cable volume side span

$$W_{\text{cable}} := \left[(V_{\text{cable.ms}} + V_{\text{cable.ss}}) \cdot \frac{\rho_{\text{steel}}}{g} \right] = (1.807 \times 10^4) \text{ kg}$$

Total cable weight

Deck :

$$V_{\text{girder}} := 2 \cdot A_{\text{girder}} \cdot (l_{\text{ms}} + l_{\text{ss}}) = 3.217 \cdot \text{m}^3$$

$$W_{\text{girder}_0} := V_{\text{girder}} \cdot \frac{\rho_{\text{steel}}}{g} = 2.575 \times 10^4 \text{ kg}$$

$$V_{\text{concrete}_0} := A_{\text{c}} \cdot (l_{\text{ms}} + l_{\text{ss}}) = 294 \cdot \text{m}^3$$

Pylon

:

$$V_{\text{pylon}_0} := A_{\text{pylon}} \cdot (h_{\text{pylon}} + h_{\text{pylon.bot}}) = 28.368 \cdot \text{m}^3$$

Total volume pylon

$$W_{\text{cable.excel}} := \text{WRITEEXCEL} \left(\frac{W_{\text{cable}}}{\text{kg}}, "Comparisoncompositescsb.xlsx", "comparison!V3" \right)$$

$$W_{\text{girder.excel}} := \text{WRITEEXCEL} \left(\frac{W_{\text{girder}}}{\text{kg}}, "Comparisoncompositescsb.xlsx", "comparison!W3" \right)$$

$$V_{\text{concrete.excel}} := \text{WRITEEXCEL} \left(\frac{V_{\text{concrete}}}{\text{m}^3}, "Comparisoncompositescsb.xlsx", "comparison!V5" \right)$$

$$V_{\text{pylon.excel}} := \text{WRITEEXCEL} \left(\frac{V_{\text{pylon}}}{\text{m}^3}, "Comparisoncompositescsb.xlsx", "comparison!X3" \right)$$

C.5. Summary of results analytic calculations steel concrete composite deck

nr cables **2** (21,5m)

Cable weight [kg]	Girder weight [kg]	Volume pylon [m3]
15.191,99	37318,24833	28,37
Concrete slab volume [m3]		
294		

	Price per unit [euros/unit]	Price [euros]
Cables	10,00	€ 151.919,89
Girder	5,00	€ 186.591,24
Concrete slab	300	€ 88.200,00
Pylon	500,00	€ 14.184,00
Total price		€ 440.895,14

Deck total € 274.791,24

nr cables **3** (16m)

Cable weight [kg]	Girder weight [kg]	Volume pylon [m3]
16.689,90	31468,36076	28,37
Concrete slab volume [m3]		
294		

	Price per unit [euros/unit]	Price [euros]
Cables	10,00	€ 166.899,03
Girder	5,00	€ 157.341,80
Concrete slab	300	€ 88.200,00
Pylon	500,00	€ 14.184,00
Total price		€ 426.624,84

Deck total € 245.541,80

nr cables **4** (12,8m)

Cable weight [kg]	Girder weight [kg]	Volume pylon [m3]
18.016,74	25752,95335	28,37
Concrete slab volume [m3]		
294		

	Price per unit [euros/unit]	Price [euros]
Cables	10,00	€ 180.167,43
Girder	5,00	€ 128.764,77
Concrete slab	300	€ 88.200,00
Pylon	500,00	€ 14.184,00
Total price		€ 411.316,19

Deck total € 216.964,77

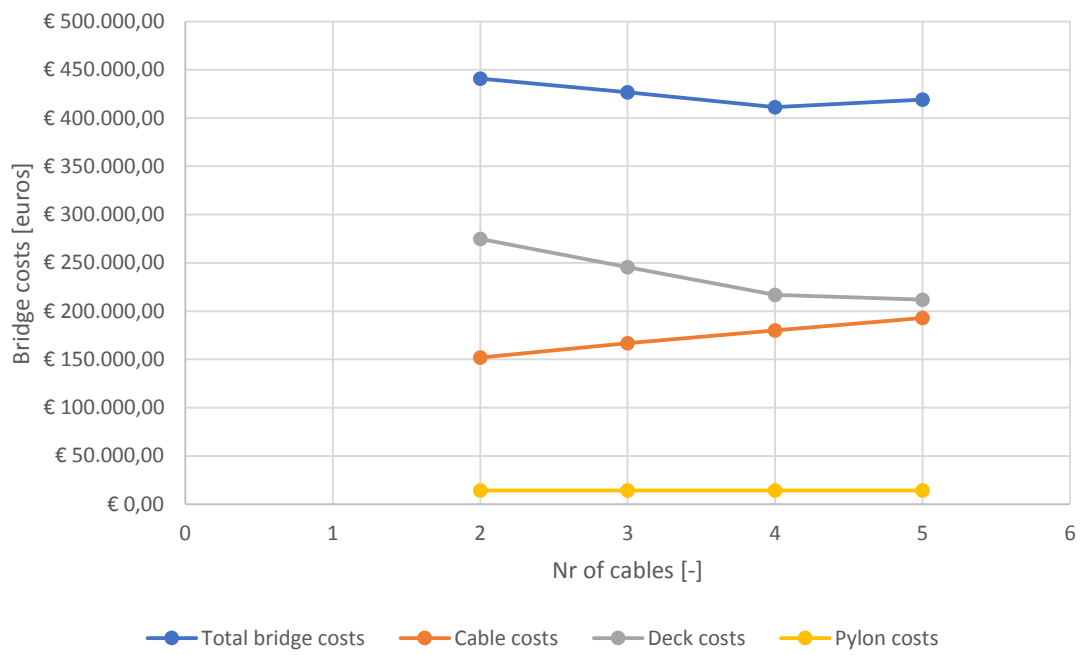
nr cables **5** (10,7m)

Cable weight [kg]	Girder weight [kg]	Volume pylon [m3]
19.290,19	24744,35205	28,37
Concrete slab volume [m3]		
294		

	Price per unit [euros/unit]	Price [euros]
Cables	10,00	€ 192.901,91
Girder	5,00	€ 123.721,76
Concrete slab	300	€ 88.200,00
Pylon	500,00	€ 14.184,00
Total price		€ 419.007,67

Deck total € 211.921,76

Bridge costs depending on number of cables



C.6. Aerodynamic checks of steel concrete composite deck

**Dynamic checks of chosen steel-concrete plate girder deck**

Computation of first bending and torsional frequencies of deck. Afterwards checks according to Eurocode EN1991-1-4 Annex E for galloping, vortex shedding and flutter.

Wind data

The bridge is to be built in Sirdal, Norway. The wind data is taken from the NA of EN1991-1-4.

$$v_{b,0} := 24 \frac{\text{m}}{\text{s}}$$

$$c_{\text{dir}} := 1.0$$

$$c_{\text{season}} := 1.0$$

$$c_{\text{alt}} := 1.0$$

$$c_{\text{prob}} := 1.0$$

$$v_b := c_{\text{dir}} \cdot c_{\text{season}} \cdot c_{\text{alt}} \cdot c_{\text{prob}} \cdot v_{b,0} = 24 \frac{\text{m}}{\text{s}}$$

The terrain category is taken as III. This results in the following values :

$$k_r := 0.22$$

$$z := 12\text{m}$$

Height of deck above water level

$$z_0 := 0.3\text{m}$$

$$c_r := k_r \cdot \ln\left(\frac{z}{z_0}\right) = 0.812$$

$$c_0 := 1.0$$

$$v_m := c_r \cdot c_0 \cdot v_b = 19.477 \frac{\text{m}}{\text{s}}$$

Frequency computation, according to Svensson book

$$F_{\text{www}} := 1\text{MN}$$

$$S_{\text{backstay}} := \frac{\frac{l_{\text{ms}}^2 \cdot q_{\text{DL}}}{2} + \frac{q_{\text{DL}} \cdot l_{\text{ss}}^2}{2}}{l_{\text{ss}} \cdot \sin(\angle \text{SB}) \cdot F} = 6.164 \times 10^{-3} \frac{1}{\text{N}} \cdot \text{kN}$$

$$S_{\text{midspan}} := \frac{x_{\text{cable}} \cdot q_{\text{DL}}}{\sin(\angle \text{S6}) \cdot F} = 9.696 \times 10^{-4} \frac{1}{\text{N}} \cdot \text{kN}$$

$$\text{DL} := S_{\text{backstay}} \cdot S_{\text{SB}} \cdot \frac{l_{\text{SB}}}{E_{\text{eff}} \cdot A_{\text{actual.SB}}} + S_{\text{midspan}} \cdot S_{\text{S6}} \cdot \frac{l_{\text{S6}}}{E_{\text{eff}} \cdot A_{\text{actual.S6}}} = 2.769 \times 10^{-4} \frac{1}{\text{N}} \cdot \text{mm}$$

$$c_{\text{elastic.DL}} := \frac{1}{x_{\text{cable}} \cdot \text{DL}} = 0.282 \cdot \frac{\text{MN}}{\text{m}^2}$$

$$f_b := \frac{1}{2} \cdot \sqrt{\frac{c_{\text{elastic.DL}}}{\frac{q_{\text{DL}}}{g}}} = 1.253 \frac{1}{s}$$

First bending eigenfrequency of deck

$$I_z := \frac{1}{12} \cdot h_c \cdot w_{\text{deck}}^3 + 2 \cdot \left[\frac{1}{12} \cdot h_{\text{girder}} \cdot t_{\text{web}}^3 + h_{\text{girder}} \cdot t_{\text{web}} \cdot \left(\frac{c_{\text{girders}}}{2} \right)^2 \right] = 29.406 \text{ m}^4$$

$$b_s := 11 \text{ m}$$

$$i_y := \sqrt{\frac{I_z}{A_c + 2 \cdot A_{\text{girder}}}} = 2.883 \text{ m}$$

$$f_t := \frac{b_s}{2 \cdot i_y} \cdot f_b = 2.391 \frac{1}{s}$$

$$\text{flutter} := \frac{f_t}{f_b} = 1.908 \quad \text{If larger than 2, then safe for flutter.}$$

Vortex shedding

Ratio of largest to smallest crosswind dimension of structure, both taken in plane perpendicular to the wind, exceeds 6. The critical wind velocity should be computed.

Strouhal number cant be computed according to EN1991-1-4 for this cross-section. $r_{\text{shedding}} := \frac{w_{\text{deck}}}{(h_c + h_{\text{girder}})} = 10$

Galloping

Structural damping taken according to EN1991-1-4, table F2:

$$s := 0.04$$

$$\text{air} := 1.25 \frac{\text{kg}}{\text{m}^3}$$

$$m_{\text{deck}} := \frac{q_{\text{DL}} \cdot 2}{g} = 9.102 \times 10^3 \frac{\text{kg}}{\text{m}}$$

Scruton number:

$$Sc := \frac{2 \cdot s \cdot m_{\text{deck}}}{\text{air} \cdot (h_{\text{girder}} + h_c)^2} = 582.505$$

Factor of galloping instability not given in table E.7 from EN1991-1-4. Therefore upper bound value taken.

$$a_G := 10$$

$$v_{\text{crit.gp}} := \frac{2 \cdot Sc}{a_G} \cdot f_b \cdot (h_{\text{girder}} + h_c) = 145.985 \frac{\text{m}}{\text{s}}$$

$$1.25 \cdot v_m = 24.347 \frac{\text{m}}{\text{s}}$$

Safe. The critical wind velocity for galloping is larger than 1.25 times the mean wind velocity.

Flutter and divergence

According to EN1991-1-4, the 3 criteria were met, considering the torsional natural frequency is less than 2 times the lowest translational natural frequency. A velocity check is required.

$$I_t := 1.87 \cdot 10^{-2} \text{ m}^4$$

$$G_{CS} := 73426 \text{ MPa}$$

>>>> Computed in Sofistik

$$f_t = 2.391 \frac{1}{s}$$

$$2 \cdot f_b = 2.506 \frac{1}{s}$$

$$dcd := \left[-6.3 \cdot \left(\frac{h_c + h_{gird}}{w_{deck}} \right)^2 - 0.38 \cdot \left(\frac{h_c + h_{gird}}{w_{deck}} \right) + 1.6 \right] = 1.499$$

$$k := \frac{2}{4} \cdot \frac{G_{CS} \cdot I_t}{l_{ms}^2} = 8.271 \times 10^5 \frac{1}{m} \cdot N \cdot \frac{m}{rad}$$

Taken from article about revision EN1991-1-4

$$v_{div} := \sqrt{\frac{2 \cdot k}{air \cdot w_{deck}^2 \cdot dcd}} = 93.96 \frac{m}{s}$$

>>> 2*vm

$$2 \cdot v_m = 38.955 \frac{m}{s}$$

Safe. Assumed that at one end of the main span torsion is prevented. At the other end, simply supported.

According to Svensson:

$$\frac{f_t}{f_b} = 1.908$$

If larger than 2, then safe for flutter.

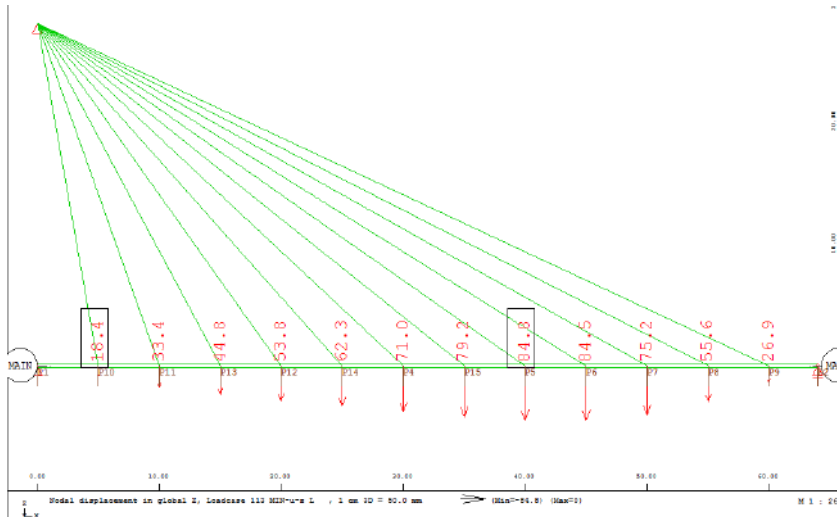
The two checks do not give similar conclusions.

C.7. Location of maximum deflection in deck

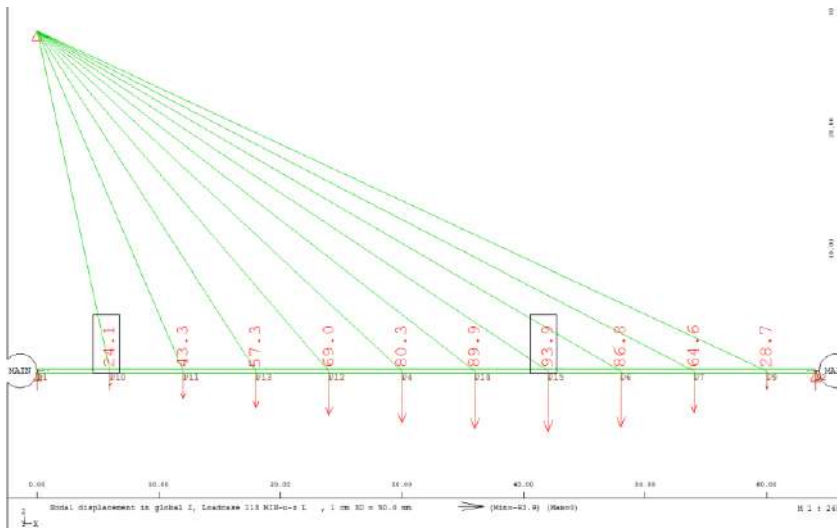
Location of maximum deflection in deck, for several cable anchor distances

In all cases, the deck has the same dimensions. The dimensions of the cable increase as the distance to the pylon increases.

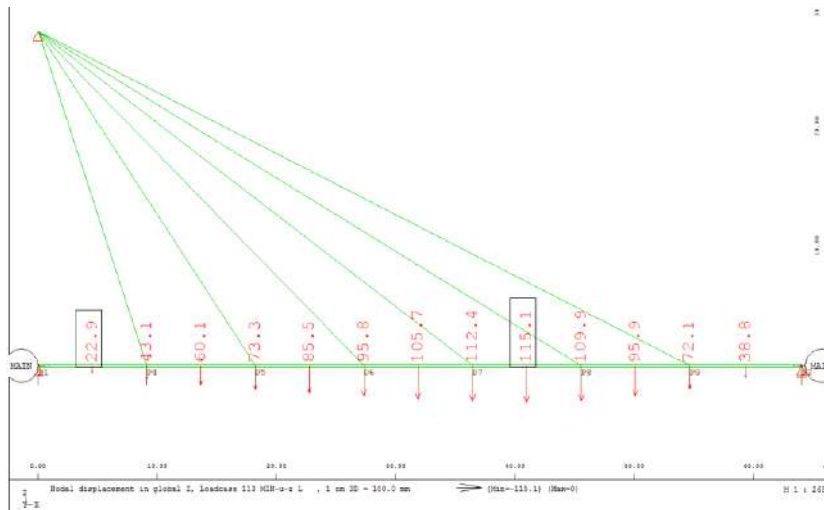
- 12 cables: maximum deflection occurs at 40 meters distance from the pylon.



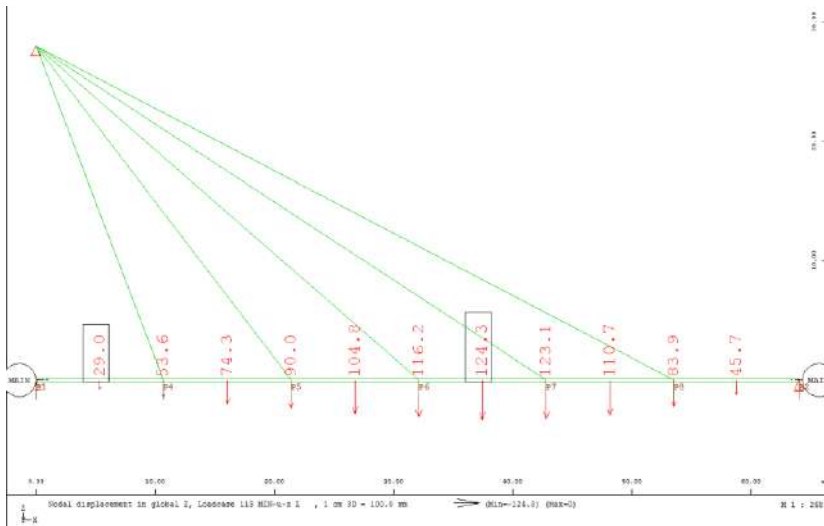
- 10 cables: maximum deflection occurs at 42 meters distance from the pylon



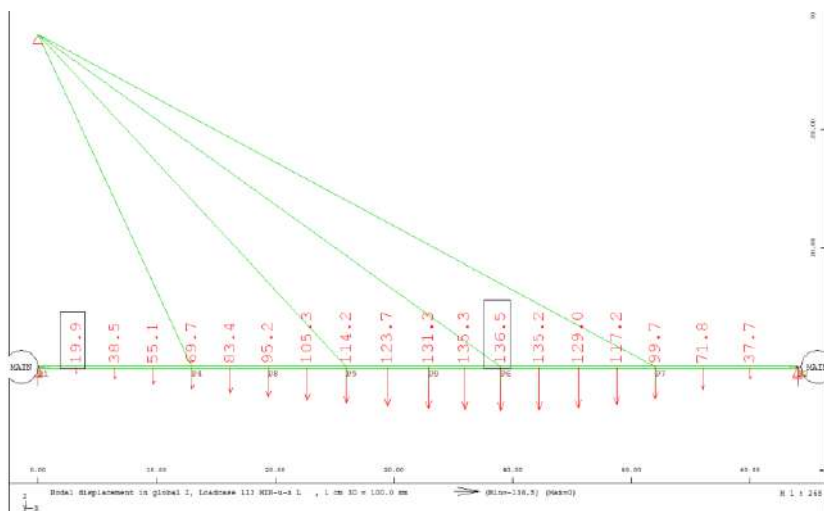
- 6 cables: maximum deflection occurs at 41 meters distance from the pylon

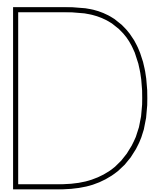


- 5 cables: maximum deflection occurs at 38 meters distance from the pylon



- 4 cables: maximum deflection occurs at 39 meters distance from the pylon.





Static FEM analysis - additional documents

D.1. Wind load definition FRP bridge

Geometrie

Type	plaatbrug of kokerbrug
totale lengte	84 m
breedte	10 m
vrije hoogte onderkant brug	11,57 m
constructiehoogte brug (d)	0,43 m
hellende loef en leizijde (α)	0 gr
helling brugdek (β)	0 gr
Verkeers-type	wegverkeer

Coördinatenstelsel

x	breedte
y	lengte
z	hoogte

NB - reductie agv α vervalt
NB - toeslag dwarshelling moet meegenomen worden
Verkeersband 2 meter

Referentie oppervlakken

wind in X-richting

$A_{ref,x}$

Afscheidingstype	open brugleuning en open veiligheidskering
dicht of open kering	Aan beide kanten
leuning / schermhoogte (d_1)	1,2 m
d_{tot} - zonder verkeer	1,63 m
d_{tot} - met verkeer	2,43 m

$A_{ref,x}$ - zonder verkeer	136,92 m ²
$A_{ref,x}$ - met verkeer	204,12 m ²

wind in Z-richting

$A_{ref,z}$

$A_{ref,z}$	840 m ²

Windbelastingen

Referentiehoogte (Ze)	12,385 m
Ontwerplevensduur =	100 jaar
C_{prob} =	1,05

Windgebied = III

$V_{b,0}$ =	24 m/s
V_b =	24,0 m/s

V^*b = 24,1 m/s

For Norway

From Norwegian NA

Cdir	1	Terrain roughness	
Cseason	1	Terrain categ III	
Calt	1	z_0	0,3 m
Cprob	1	z_{min}	8 m
		z_{max}	200 m
		$z_{0,II}$	0,05 m
Z_0 =	0,3		

H4.3.1 Variatie met hoogte

referentiehoogte (Ze) = 12,385 1,00

(4.3)

referentiehoogte (Ze) =	12,385	$V_{m(z)}$	
gereduceerde windsnelheid, verkeersband	12,385	19,23 m/s	
		19,27 m/s	

H4.4

Windturbulentie

referentiehoogte (Ze) = 12,385 $I_{w(z)}$ = 0,2688

H4.5

Extreme stuwdruk

	$q_p(z)$	
referentiehoogte (Ze) =	0,67	kN/m ²
gereduceerde stuwdruk, verkeersband	0,67	kN/m ²

$C_e(z)$	$C_s C_d$
1,850	0,89 alleen een waarde groter dan
1,850	0,89 1,0 wordt verrekend als toeslag

Krachtcoëfficiënten

wind in X-richting

	b/d_{tot}	$C_{fc,0}$	toeslag dwarshelling	lijnfunctie
zonder verkeer	b/d_{tot}	6,13	1,30	Bouwstadium, open brugleuning e.d.
met verkeer	b/d_{tot}	4,12	1,30	Bouwstadium, open brugleuning e.d.

wind in Z-richting

zonder verkeer	b/d_{tot}	6,13
hoek α van de wind met hor		0 gr
$\theta = \alpha + \beta$		0 gr
figuur NB.8 C_{fz} =	0,49	let op ook de negatieve waarde gebruiken voor wind in tegengestelde richting

Belasting door wind

wind in X-richting

representatieve belasting	per m1	Totaal	Prep
zonder verkeer	F_w = 1,4 kN/m1	118,6 kN	0,87 kN/m ²
met verkeer	F^*w = 2,1 kN/m1	177,5 kN	0,87 kN/m ²

wind in Y-richting

= 40% van windkrachten in de x richting. (in de x-richting treed dan gelijktijdig ook dezelfde kracht als in de y-richting op)

representatieve belasting	per m1	Totaal	Prep
zonder verkeer	F_w = 0,6 kN/m1	47,4 kN	0,35 kN/m ²
met verkeer	F^*w = 0,8 kN/m1	71,0 kN	0,35 kN/m ²

wind in Z-richting

Als belasting in Z-richting ongunstig werkt treed deze gelijktijdig met de x-richting op

representatieve belasting	per m1	Totaal	Prep
zonder verkeer	F_w = 3,2 kN/m1	272,5 kN	0,32 kN/m ²
met $e=b/4$	M_w = 8,1 kNm/m1	681,4 kNm/m1	
met verkeer	F^*w = 3,3 kN/m1	273,7 kN	0,33 kN/m ²
	M^*w = 8,1 kNm/m1	684,3 kNm/m1	

Belasting door wind**wind in X-richting**

representatieve belasting

zonder verkeer

met verkeer

Totaal

	X-richting	Y-richting	Z-richting	
Fw =	118,6			kN
F*w =	177,5			kN

Verdeelde belasting

X-richting	Y-richting	Z-richting	
0,87			kN/m ²
0,87			kN/m ²

wind in Y-richting

representatieve belasting

zonder verkeer

met verkeer

= 40% van windkrachten in de x richting. (in de x-richting treed dan gelijktijdig ook dezelfde kracht als in de y-richting op)

Fw =	47,4	47,4		kN
F*w =	71,0	71,0		kN

0,35	0,35		kN/m ²
0,35	0,35		kN/m ²

wind in Z-richting

representatieve belasting

zonder verkeer

met verkeer

Als belasting in Z-richting ongunstig werkt treed deze gelijktijdig met de x-richting op

Fw =	118,6		272,5	kN
Mw =			681,4	kNm
F*w =	177,5		273,7	kN
M*w =			684,3	kNm

0,87		0,32	kN/m ²
0,87		0,33	kN/m ²

Pylon

Geometry

Height

Width

Depth

36 [m]

1 [m]

Inner width

0,8 [m]

A

0,32 [m^2]

Inner depth

0,6 [m]

Ac

0,8 [m^2]

Transversal connector

height

0,8 [m]

width

0,5 [m]

Length

10 [m]

Pylon

Direction

x

y

d/b

0,8

1,25

cf0

2,4

1,9

 Ψ_r

1

1

Effective slenderness λ

31,68

39,6 0,88*/b

Solidity ratio A/Ac

0,4

0,4

End-effect factor Ψ_λ

0,94

0,95

Force coefficient cf

2,256

1,805

Transversal connector

Direction

y

z

d/b

0,625

1,6

cf0

2,36

1,85

 Ψ_r

1

1

Effective slenderness

12,5

12,5

Solidity ratio A/Ac

1

1

End-effect factor Ψ_λ

0,72

0,72

Force coefficient cf

1,6992

1,332

Pylon

Reference height

zb

[m]

Turbulence intensity

lv

[-]

Terrain roughness factor

Cr(z)

[-]

Mean wind velocity

Traffic

vm(z)*

[m/s]

No traffic

vm(z)

[m/s]

Peak velocity pressure at

Traffic

qp(ze)*

[kN/m^2]

reference height zb

No traffic

qp(ze)

[kN/m^2]

Strips

12

24

36

0,271085031 0,228205 0,20887764

0,79454528 0,943842 1,03117465

19,06908672 22,6522 24,7481915

0,658532966 0,833001 0,94249774

Direction

x

y

Reference area

1

0,8 [m^2/m]

Force Fw

ze

x

y

12 1,48565 0,950921603 [kN/m]

24 1,87925 1,202853348 [kN/m]

36 2,126275 1,360966736 [kN/m]

Transversal connector

Reference height

zb

[m]

10,48

Turbulence intensity

lv

[-]

0,281417326

Terrain roughness factor

Cr(z)

[-]

0,765373386

Mean wind velocity

Traffic

vm(z)*

[m/s]

No traffic

vm(z)

[m/s]

18,36896126

Peak velocity pressure at

Traffic

qp(ze)*

[kN/m^2]

reference height zb

No traffic

qp(ze)

[kN/m^2]

0,626316931

Direction

y

z

Reference area

0,8

0,5 [m^2/m]

Force Fw

0,85139

0,417127 [kN/m]

Cables

Geometry

Cable anchor distance

Diameter

Length

[m]

1

2

3

4

5

6

7

BS cable

[m]

8

16

24

32

40

48

56

20

[m]

0,039

0,048

0,048

0,048

0,052

0,055

0,06

0,1436

[m]

29,1

32,2

36,9

42,5

48,8

55,6

62,6

34,4

cf0

1,2 For stranded cables, (7.9.2)

End-effect factor Ψ_λ

1

Force coefficient cf

1,2

Top of cables

Bottom of cables

Reference height

zb

[m]

36

11,785

Turbulence intensity

lv

[-]

0,208877645

0,27242

Terrain roughness factor

Cr(z)

[-]

1,031174646

0,790651

Mean wind velocity

Traffic

vm(z)*

[m/s]

No traffic

vm(z)

[m/s]

24,74819151

18,97563

Peak velocity pressure at

Traffic

qp(ze)*

[kN/m^2]

reference height zb

No traffic

qp(ze)

[kN/m^2]

0,942497739

0,654197

Cables		1	2	3	4	5	6	7	BS cable
Reference area	[m^2/m]	0,039	0,048	0,048	0,048	0,052	0,055	0,06	0,1436
Force Fw bottom cables	[kN/m]	0,030616	0,037681755	0,037681755	0,037682	0,0408219	0,04317701	0,0471022	0,11273125
Force Fw top cables	[kN/m]	0,044109	0,05428787	0,05428787	0,054288	0,05881186	0,06220485	0,0678598	0,16241121

D.2. Results first FRP Sofistik model - summary

FRP deck

Material factor	
GFRP	
γ_{M1}	1,15
γ_{M2}	1,2
γ_M	1,38

Conversion factor		
Temperature	η_{ct}	0,9
Moisture	η_{cm}	0,9
Creep	η_{cv}	0,7
Fatigue	η_{cf}	0,9
Strength checks	$\eta_{ct} \cdot \eta_{cm} \cdot \eta_{cv} \cdot \eta_{cf}$	0,567
Comfort (vibrations)	$\eta_{ct} \cdot \eta_{cm}$	0,729
Deflection checks -	$\eta_{ct} \cdot \eta_{cm}$	0,5103
Deflection checks -	$\eta_{ct} \cdot \eta_{cm}$	0,729

Anisotropic GFRP, $V_f=50\%$						
E1 [Gpa]	E2 [Gpa]	G12 [Gpa]	σ_1 [Mpa]	σ_2 [Mpa]	τ_{12} [Mpa]	Density [kN/m ³]
25,8	15,9	5,6	310	191	90	13,2

Stress checks (glob): ULS - design combinations

With factors:

σ_1 [Mpa]	σ_2 [Mpa]	τ_{12} [Mpa]
127,370	78,476	36,978

Parts		n_{xx} [Mpa]	n_{yy} [Mpa]	n_{xy} [Mpa]
Bottom flange	min	-169,9	-30	-65
	max	90,4	33,8	70,3
Top flange	min	-72,8	-37,1	-20,6
	max	18,2	5,56	20,58
Edge stiffeners	min	-199,4	-49,8	-20
	max	72,9	12,5	21,55
Longitudinal stiffene	min	-75,8	-38	-30,3
	max	58,6	14,8	32,15
Transversal stiffene	min	-48,6	-22	-46,5
	max	40,4	23,1	46,5

	Unity check	n_{xx} [Mpa]	n_{yy} [Mpa]	n_{xy} [Mpa]
Bottom flange	min	1,334	0,382	1,758
	max	0,710	0,431	1,901
Top flange	min	0,572	0,473	0,557
	max	0,143	0,071	0,557
Edge stiffeners	min	1,566	0,635	0,541
	max	0,572	0,159	0,583
Longitudinal stiffene	min	0,595	0,484	0,819
	max	0,460	0,189	0,869
Transversal stiffene	min	0,382	0,280	1,257
	max	0,317	0,294	1,257

Unity check critical	-	1,565523127	0,634588216	1,90111699

Deflections

SLS - freq combination according to EN1990 (A.2.4.2)

Main span

Global deflection in : 295,7 [mm] Limit defined by B 182,8571429 [mm]

Total deflection 295,7 [mm]

Unity check global d **1,6171094**

Side span

Global deflection in : 81 [mm] Limit defined by B 57,14285714 [mm]

Unity check **1,4175****Cross girders**

Total deflection 21,6 [mm] Limit 30,85714286 [mm]

(main span)

UC 0,7

Strength check

Cross girder backing stay cable					
Max My	978,3 [kNm]	M _{Rd,c,y}	1015 [kNm]	UC	0,96
Max Mz	1362 [kNm]	M _{Rd,c,z}	1315 [kNm]	UC	1,04
Max Vy	654,2 [kN]	V _{Rd,y}	715,53 [kN]	UC	0,91
Max Vz	1234 [kN]	V _{Rd,z}	774,69 [kN]	UC	1,72

Cross girders main span					
Max My	226,8 [kNm]	M _{Rd,c,y}	900 [kNm]	UC	0,25

Max Mz	430,9	[kNm]	M_Rd,c,z	900	[kNm]	UC	0,48
Max Vy	340,2	[kN]	V.Rd.y	2016	[kN]	UC	0,17
Max Vz	181,8	[kN]	V.Rd.z	2016	[kN]	UC	0,09

Pylon and transversal connector

Concrete C40/50				
Ec [Mpa]	Density [kN	fck [Mpa]	fctm [Mpa]	yM
35000	25	40	3,5	1,5

Deflections pylon SLS freq

x y
82,6 121,6 mm

Deflections transversal connector

z Max defl [mm]
27,4 mm 28,57142857 **0,959 OK**

Cables

Steel cables ; Y1860				
E_eff [Mpa]	fs [Mpa]	Δfs [Mpa]	As [mm^2]	Density [kN/m^3]
19000	837	200	150	78,5

Stress checks

Cable	max N [kN]	A actual [mm^2]	max stress [Mpa]	UC stress max
1	95,2	1200	79,3	0,09
2	124,6	1800	69,2	0,08
3	143,6	1800	79,8	0,10
4	168,3	1800	93,5	0,11
5	206,7	2100	98,4	0,12
6	230,3	2400	96,0	0,11
7	194,6	2850	68,3	0,08
8	75,9	1200	63,3	0,08
9	75,7	1800	42,1	0,05
Backing stay cable	1382	16200	85,3	0,10

D.3. Results final FRP Sofistik model - summary

FRP deck

Material factor	
GFRP	
γ_{M1}	1,15
γ_{M2}	1,2
γ_M	1,38

Conversion factor		
Temperature	η_{ct}	0,9
Moisture	η_{cm}	0,9
Creep	η_{cv}	0,7
Fatigue	η_{cf}	0,9
Strength checks	$\eta_{ct}\eta_{cm}\eta_{cv}$	0,567
Comfort (vibrations)	$\eta_{ct}\eta_{cm}\eta_{cv}$	0,729
Deflection checks - long term	$\eta_{ct}\eta_{cm}\eta_{cv}$	0,5103
Deflection checks - short term	$\eta_{ct}\eta_{cm}\eta_{cv}$	0,729

Anisotropic GFRP, Vf=50%						
E1 [Gpa]	E2 [Gpa]	G12 [Gpa]	σ_1 [Mpa]	σ_2 [Mpa]	τ_{12} [Mpa]	Density [kN/m^3]
25,8	15,9	5,6	310	191	90	13,2

Stress checks (global)

ULS - design combinations

With factor

σ_1 [Mpa]	σ_2 [Mpa]	τ_{12} [Mpa]
127,370	78,476	36,978

Parts		n_{xx} [Mpa]	n_{yy} [Mpa]	n_{xy} [Mpa]
Bottom flange	min	-71,9	-14	-28,2
	max	81,9	32,7	28,2
Top flange	min	-60,2	-37,06	-21,1
	max	14,4	3,11	21,1
Edge stiffeners	min	-92,2	-48,7	-17,9
	max	54,6	12	11,8
Longitudinal stiffeners	min	-54,5	-25,3	-29,3
	max	46,8	10,54	32,4
Transversal stiffeners	min	-51,2	-21,3	-36,5
	max	41,2	4,03	36,5

	Unity checks	n_{xx} [Mpa]	n_{yy} [Mpa]	n_{xy} [Mpa]
Bottom flange	min	0,564	0,178	0,763
	max	0,643	0,417	0,763
Top flange	min	0,473	0,472	0,571
	max	0,113	0,040	0,571
Edge stiffeners	min	0,724	0,621	0,484
	max	0,429	0,153	0,319
Longitudinal stiffeners	min	0,428	0,322	0,792
	max	0,367	0,134	0,876
Transversal stiffeners	min	0,402	0,271	0,987
	max	0,323	0,051	0,987

Unity check critical part	-	0,723877795	0,620571207	0,987066432

Deflections

SLS - freq combination according to EN1990 (A.2.4.2)

Main span

Global deflection in Z dir [mm] 176,8 [mm] Limit defined by Bru 182,8571429 [mm]

Total deflection 176,8 [mm]

Unity check global deflection **0,966875**

Side span

Global deflection in Z dir [mm] 56,7 [mm] Limit defined by Bru 57,14285714 [mm]

Unity check **0,99225**

Cross girders

Total deflection (main span) 21,6 [mm] Limit 30,85714286 [mm]

UC 0,7

Strength check

Cross girder backing stay cable > Sofistik

Cross girders main span						
Max My	285,4	[kNm]	M_Rd,c,y	900	[kNm]	UC 0,32
Max Mz	467,2	[kNm]	M_Rd,c,z	900	[kNm]	UC 0,52
Max Vy	365,9	[kN]	V_Rd,y	2016	[kN]	UC 0,18
Max Vz	182,6	[kN]	V_Rd,z	2016	[kN]	UC 0,09

Pylon and transversal connector

Concrete C40/50				
Ec [Mpa]	Density [kN/m	fck [Mpa]	fctm [Mpa]	γM
35000	25	40	3,5	1,5

Deflections pylon
x SLS freq
y
20 71,6 mm

Deflections transversal connector
z

25,1 mm

Max defl [mm]
28,57142857

0,8785 OK

Cables

Steel cables ; Y1860				
E_eff [Mpa]	fs [Mpa]	Δfs [Mpa]	As [mm^2]	Density [kN/m^3]
19000	837	200	150	78,5

Stress checks

Cable	max N [kN]	A actual [mm^2]	max stress [Mpa]	UC stress max
1	100,7	2100	48,0	0,06
2	129,8	3150	41,2	0,05
3	148,3	3150	47,1	0,06
4	172,6	3150	54,8	0,07
5	214,7	3750	57,3	0,07
6	250,6	4200	59,7	0,07
7	253,5	5100	49,7	0,06
8	90	2100	42,9	0,05
9	86,5	3150	27,5	0,03
Backing stay cable	1519	81900	18,5	0,02

D.4. Results final FRP Sofistik model - extended Sofistik printout

FRP_bridge_v1

Materials

Default design code is EuroNorm EN 1993-2:2006 Steel Structures (Norge) V 2018

Structure: B (Road bridges)

Snow load zone : 1

Materials

Mat	Classification
1	C 40/50 (EN 1992)
2	Y 1860 A (EN 1992)
3	GFK EP
4	B 500 B (EN 1992)

Mat 1 C 40/50 (EN 1992)

Young's modulus	E	35220	[N/mm2]	Safetyfactor		1.50	[-]
Poisson's ratio	μ	0.20	[-]	Strength	fc	34.00	[MPa]
Shear modulus	G	14675	[N/mm2]	Nominal strength	fck	40.00	[MPa]
Compression modulus	K	19567	[N/mm2]	Tensile strength	fctm	3.51	[MPa]
Nominal Weight	γ	25.0	[kN/m3]	Tensile strength	fctk,05	2.46	[MPa]
Mean density	ρ	2400.0	[kg/m3]	Tensile strength	fctk,95	4.56	[MPa]
Elongation coefficient	α	1.00E-05	[1/K]	Bond strength	fbd	3.68	[MPa]
				Service strength	fc	48.00	[MPa]
				Fatigue strength	fcd,fat	19.04	[MPa]
				Tensile strength	fctd	1.64	[MPa]
				Tensile failure energy	Gf	0.15	[N/mm]

Mat 2 Y 1860 A (EN 1992)

Young's modulus	E	195000	[N/mm2]	Safetyfactor		1.10	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	1600.00	[MPa]
Shear modulus	G	75000	[N/mm2]	Compressive yield	fyc	1600.00	[MPa]
Compression modulus	K	162500	[N/mm2]	Tensile strength	ft	1860.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	1860.00	[MPa]
Mean density	ρ	7850.0	[kg/m3]	Ultimate strain		60.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.		0.75	[-]
max. thickness	t-max	18.00	[mm]	EN 1992 bond coeff.	k1	1.60	[-]
Relaxation	EN-1992	Class	2	Hardening modulus	Eh	0.00	[MPa]
Relaxation	$\rho(1000h)$	2.50	[%]	Proportional limit	fp	1600.00	[MPa]
				Dynamic allowance	σ -dyn	160.87	[MPa]

FRP_bridge_v1

Materials

Mat 3 GFK EP

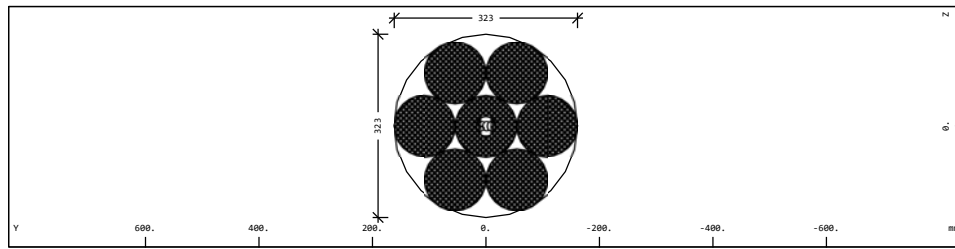
Young's modulus	E	25800	[N/mm2]	Safetyfactor		1.38	[-]
Poisson's ratio	μ	0.32	[-]	Bending strength	fm,0	550.00	[MPa]
Shear modulus	G	5600	[N/mm2]	Bending strength	fm,90	550.00	[MPa]
Compression modulus	K	333	[N/mm2]	Tensile strength	ft,0	310.00	[MPa]
Nominal Weight	γ	13.4	[kN/m3]	Tensile strength	ft,90	191.00	[MPa]
Mean density	ρ	1345.0	[kg/m3]	Compress. strength	fc,0	310.00	[MPa]
Elongation coefficient	α	0.00E+00	[1/K]	Compress. strength	fc,90	191.00	[MPa]
Young's modulus	E-90	15900	[N/mm2]	Shear strength	fv	90.00	[MPa]
Shear modulus	G-90	5000	[N/mm2]	Shear strength	fv,T	300.00	[MPa]
Poisson's ratio	μ -90	0.00	[-]	Shear strength	fv,p1	300.00	[MPa]
Permanent combination	kmod	1.00	[-]				
Long term combination		1.00	[-]				
Middle term combination		1.00	[-]				
Short term combination		1.00	[-]				
Very short term combination		1.00	[-]				

Mat 4 B 500 B (EN 1992)

Young's modulus	E	200000	[N/mm2]	Safetyfactor		1.15	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	500.00	[MPa]
Shear modulus	G	76923	[N/mm2]	Compressive yield	fyc	500.00	[MPa]
Compression modulus	K	166667	[N/mm2]	Tensile strength	ft	540.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	540.00	[MPa]
Mean density	ρ	7850.0	[kg/m3]	Ultimate strain		50.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.		1.00	[-]
max. thickness	t-max	32.00	[mm]	EN 1992 bond coeff.	k1	0.80	[-]
				Hardening modulus	Eh	0.00	[MPa]
				Proportional limit	fp	500.00	[MPa]
				Dynamic allowance	σ -dyn	152.17	[MPa]

FRP_bridge_v1

Cross section No. 1 - Cable 323 1X7



Cross section No. 1 - Cable 323 1X7

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α[°]
1	2	8.1898E-02		5.340E-04	0.0	0.0	195000	1043.3	
		0.000E+00		5.340E-04	0.0	0.0	75000	(CABLE)	
= Cable 323 1X7									
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid		
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre		
A[m2]	sectional area				E[N/mm2]		Young's modulus		
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length		
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia								
I-1[m4],I-2[m4],α[°]	principal moments of inertia and angle of the principal axes								
MRf	reinforcement material number								
It[m4]	torsional moment of inertia								
G[N/mm2]	Shear modulus								

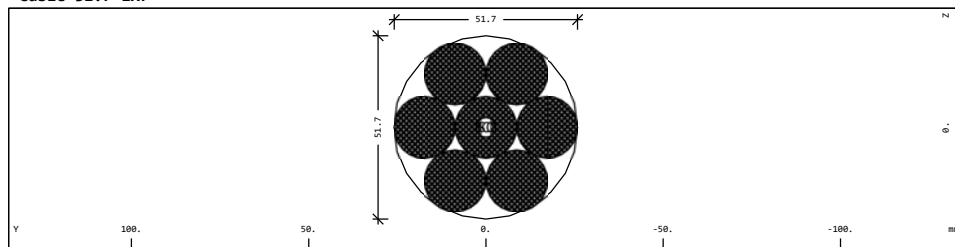
Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	87323.4						0.00	0.00	0.0	0.0	
D ¹	52923.3						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force				Mb[kNm2]		warping moment				
Vy[kN],Vz[kN]	shear force				My[kNm],Mz[kNm]		bending moment				
Mt[kNm]	primary torsional moment				y[mm],z[mm]		ordinate of plastic centre				
Mt2[kNm]	secondary torsional moment				BUCK		buckling curve (LTB, y-y, z-z)				

FRP_bridge_v1

Cross Sections

Cross section No. 2 - Cable 51.7 1X7



Cross section No. 2 - Cable 51.7 1X7

Static properties of cross section

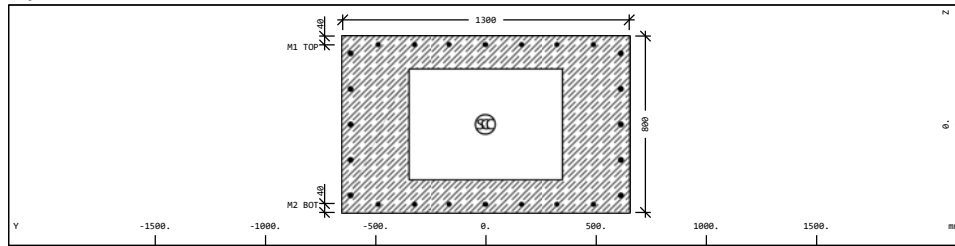
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α[°]
2	2	2.0982E-03		3.505E-07	0.0	0.0	195000	26.7	
		0.000E+00		3.505E-07	0.0	0.0	75000	(CABLE)	
= Cable 51.7 1X7									
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid		
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre		
A[m2]	sectional area				E[N/mm2]		Young's modulus		
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length		
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia								
I-1[m4],I-2[m4],α[°]	principal moments of inertia and angle of the principal axes								
MRf	reinforcement material number								
It[m4]	torsional moment of inertia								
G[N/mm2]	Shear modulus								

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	2237.2						0.00	0.00	0.0	0.0	
D ¹	1355.9						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force				Mb[kNm2]		warping moment				
Vy[kN],Vz[kN]	shear force				My[kNm],Mz[kNm]		bending moment				
Mt[kNm]	primary torsional moment				y[mm],z[mm]		ordinate of plastic centre				
Mt2[kNm]	secondary torsional moment				BUCK		buckling curve (LTB, y-y, z-z)				

FRP_bridge_v1
Cross Sections

Cross section No. 3 - Pylon



Cross section No. 3 - Pylon

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α[°]
3	1	6.9000E-01	3.516E-01	4.818E-02	0.0	0.0	35220	1725.0	1.322E-01
	4	1.164E-01	3.480E-01	1.322E-01	0.0	0.0	14675	(BEAM)	4.818E-02

= Pylon

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m2]	sectional area	E[N/mm2]	Young's modulus
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia		
I-1[m4],I-2[m4],α[°]	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m4]	torsional moment of inertia		
G[N/mm2]	Shear modulus		

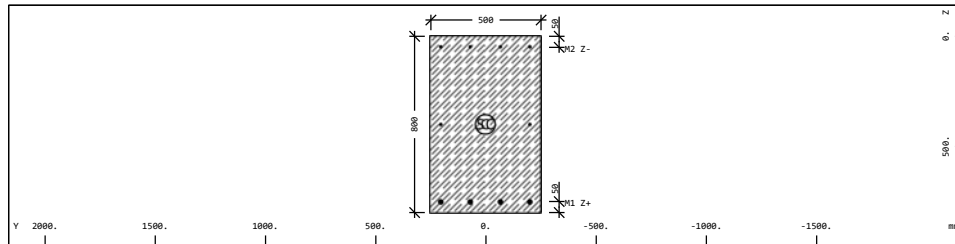
Reinforcement global values

Layer	Mref	Mat	As	As-min	As-max	D	yr	zr	L-tors	N-p	My-p	Mz-p
			[cm2]	[cm2]	[cm2]	[mm]	[mm]	[mm]	[mm]	[kN]	[kNm]	[kNm]
M1 TOP	1	4	16.56	16.56		12	0.0	-360.0	1.7			
M2 BOT	1	4	16.56	16.56		12	0.0	360.0	1.7			
M3 L	1	4	16.74	16.74		12	-610.0	0.0	1.7			
M4 R	1	4	16.74	16.74		12	610.0	0.0	1.7			

Layer	layer of reinforcement	D	bar diameter
Mref	embedding reference material	yr,zr	ordinate of elastic centroid
Mat	material number	L-tors	torsional effective length
As	reinforcement area	N-p	prestress normal force
As-min	minimum reinforcement area	My-p,Mz-p	prestress bending moment
As-max	maximum reinforcement area		

FRP_bridge_v1
Cross Sections

Cross section No. 4 - B/H = 500 / 800 mm



Cross section No. 4 - B/H = 500 / 800 mm

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α[°]
4	1	4.0000E-01	3.333E-01	2.133E-02	0.0	0.0	35220	1000.0	
	4	2.037E-02	3.333E-01	8.333E-03	400.0	400.0	14675	(BEAM)	

= B/H = 500 / 800 mm

= B/H = 500 / 800 mm

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m2]	sectional area	E[N/mm2]	Young's modulus
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia		
I-1[m4],I-2[m4],α[°]	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m4]	torsional moment of inertia		
G[N/mm2]	Shear modulus		

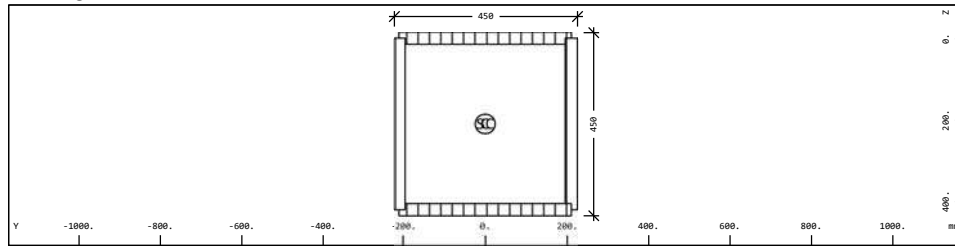
Reinforcement global values

Layer	Mref	Mat	As	As-min	As-max	D	yr	zr	L-tors	N-p	My-p	Mz-p
			[cm2]	[cm2]	[cm2]	[mm]	[mm]	[mm]	[mm]	[kN]	[kNm]	[kNm]
M1 Z+	1	4	12.56	12.56		20	0.0	750.0	400.0			
M2 Z-	1	4	3.16	3.16		10	0.0	50.0	400.0			
M3 Y+-	1	4	1.58	1.58	1.58	10	0.0	400.0	1400.0			

Layer	layer of reinforcement	D	bar diameter
Mref	embedding reference material	yr,zr	ordinate of elastic centroid
Mat	material number	L-tors	torsional effective length
As	reinforcement area	N-p	prestress normal force
As-min	minimum reinforcement area	My-p,Mz-p	prestress bending moment
As-max	maximum reinforcement area		

FRP_bridge_v1
Cross Sections

Cross section No. 5 - Cross girder



Cross section No. 5 - Cross girder

Static properties of cross section

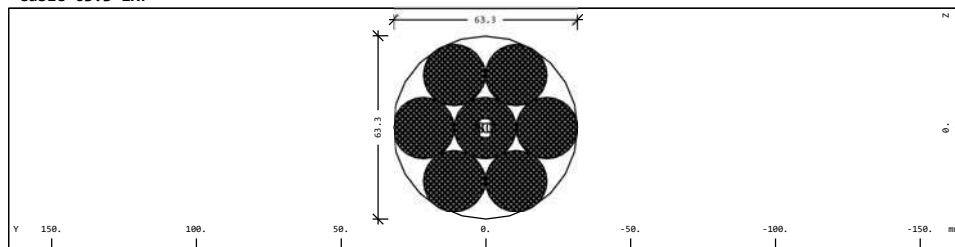
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$
5	3	5.0400E-02	2.100E-02	1.482E-03	0.0	0.0	25800	67.5	
		2.238E-03	2.100E-02	1.482E-03	210.0	210.0	5600	(BEAM)	
= Cross girder									
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid		
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre		
A[m2]	sectional area				E[N/mm2]		Young's modulus		
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length		
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia								
I-1[m4],I-2[m4], $\alpha[^\circ]$	principal moments of inertia and angle of the principal axes								
MRf	reinforcement material number								
It[m4]	torsional moment of inertia								
G[N/mm2]	Shear modulus								

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
E ¹	15624.0	2016.00	2016.00	839.16			2187.36	2187.36	0.0	210.0	-, b, c
F ¹	11321.7	1460.87	1460.87	608.09			1585.04	1585.04	0.0	210.0	
¹ E/F = elastic characteristic/design values (maximum strain reaches yield limit)											
N[kN]	normal force		Mb[kNm2]		warping moment						
Vy[kN],Vz[kN]	shear force		My[kNm],Mz[kNm]		bending moment						
Mt[kNm]	primary torsional moment		y[mm],z[mm]		ordinate of plastic centre						
Mt2[kNm]	secondary torsional moment		BUCK		buckling curve (LTb, y-y, z-z)						

FRP_bridge_v1
Cross Sections

Cross section No. 6 - Cable 63.3 1X7



Cross section No. 6 - Cable 63.3 1X7

Static properties of cross section

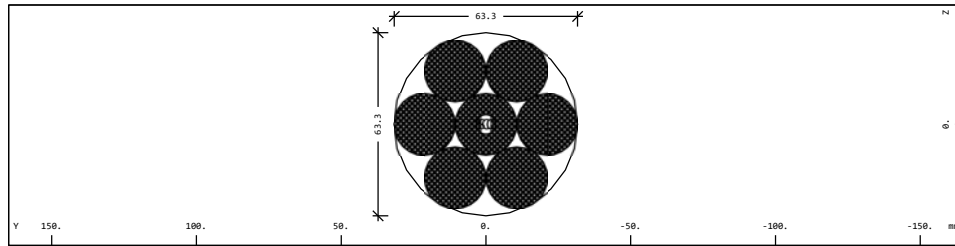
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$
6	2	3.1454E-03		7.877E-07	0.0	0.0	195000	40.1	
		0.000E+00		7.877E-07	0.0	0.0	75000	(CABLE)	
= Cable 63.3 1X7									
SNo	section number		yc[mm],zc[mm]		ordinate of elastic centroid				
Mat	material number		ysc[mm],zsc[mm]		ordinate of shear centre				
A[m2]	sectional area		E[N/mm2]		Young's modulus				
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area		g[kg/m]		weight per length				
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia								
I-1[m4],I-2[m4], $\alpha[^\circ]$	principal moments of inertia and angle of the principal axes								
MRf	reinforcement material number								
It[m4]	torsional moment of inertia								
G[N/mpg2]	Shear modulus								

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	3353.8						0.00	0.00	0.0	0.0	
D ¹	2032.6						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force		Mb[kNm2]		warping moment						
Vy[kN],Vz[kN]	shear force		My[kNm],Mz[kNm]		bending moment						
Mt[kNm]	primary torsional moment		y[mm],z[mm]		ordinate of plastic centre						
Mt2[kNm]	secondary torsional moment		BUCK		buckling curve (LTb, y-y, z-z)						

FRP_bridge_v1
Cross Sections

Cross section No. 7 - Cable 63.3 1X7



Static properties of cross section

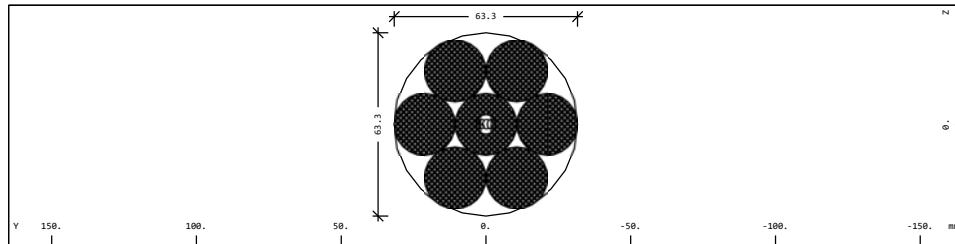
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α[°]
7	2	3.1454E-03		7.877E-07	0.0	0.0	195000	40.1	
		0.000E+00		7.877E-07	0.0	0.0	75000	(CABLE)	
= Cable 63.3 1X7									
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid		
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre		
A[m2]	sectional area				E[N/mm2]		Young's modulus		
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length		
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia								
I-1[m4],I-2[m4],α[°]	principal moments of inertia and angle of the principal axes								
MRf	reinforcement material number								
It[m4]	torsional moment of inertia								
G[N/mm2]	Shear modulus								

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	3353.8						0.00	0.00	0.0	0.0	
D ¹	2032.6						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force				Mb[kNm2]		warping moment				
Vy[kN],Vz[kN]	shear force				My[kNm],Mz[kNm]		bending moment				
Mt[kNm]	primary torsional moment				y[mm],z[mm]		ordinate of plastic centre				
Mt2[kNm]	secondary torsional moment				BUCK		buckling curve (LTB, y-y, z-z)				

FRP_bridge_v1
Cross Sections

Cross section No. 8 - Cable 63.3 1X7



Static properties of cross section

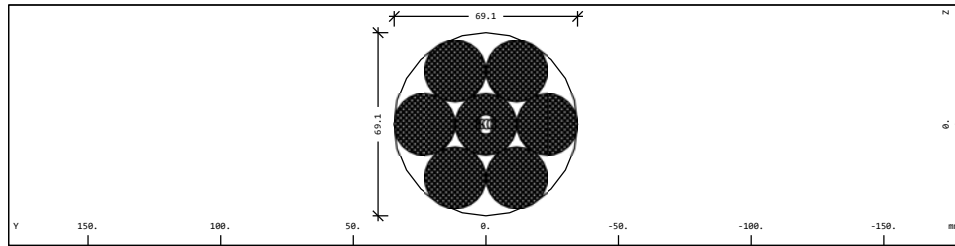
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α[°]
8	2	3.1454E-03		7.877E-07	0.0	0.0	195000	40.1	
		0.000E+00		7.877E-07	0.0	0.0	75000	(CABLE)	
= Cable 63.3 1X7									
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid		
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre		
A[m2]	sectional area				E[N/mm2]		Young's modulus		
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length		
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia								
I-1[m4],I-2[m4],α[°]	principal moments of inertia and angle of the principal axes								
MRf	reinforcement material number								
It[m4]	torsional moment of inertia								
G[N/mm2]	Shear modulus								

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	3353.8						0.00	0.00	0.0	0.0	
D ¹	2032.6						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force				Mb[kNm2]		warping moment				
Vy[kN],Vz[kN]	shear force				My[kNm],Mz[kNm]		bending moment				
Mt[kNm]	primary torsional moment				y[mm],z[mm]		ordinate of plastic centre				
Mt2[kNm]	secondary torsional moment				BUCK		buckling curve (LTB, y-y, z-z)				

FRP_bridge_v1
Cross Sections

Cross section No. 9 - Cable 69.1 1X7



Cross section No. 9 - Cable 69.1 1X7

Static properties of cross section

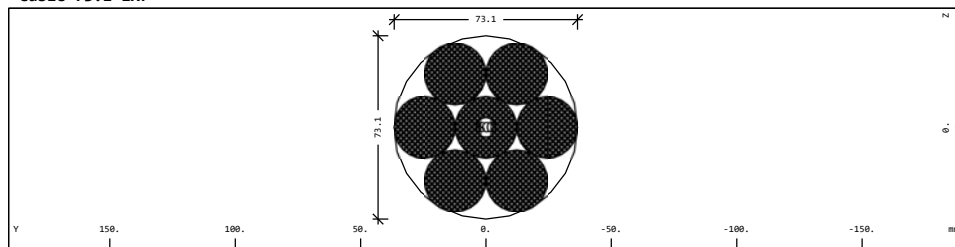
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]					
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]					
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$					
9	2	3.7482E-03		1.119E-06	0.0	0.0	195000	47.7						
		0.000E+00		1.119E-06	0.0	0.0	75000	(CABLE)						
= Cable 69.1 1X7														
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid							
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre							
A[m2]	sectional area				E[N/mm2]		Young's modulus							
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length							
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia													
I-1[m4],I-2[m4], $\alpha[^\circ]$	principal moments of inertia and angle of the principal axes													
MRf	reinforcement material number													
It[m4]	torsional moment of inertia													
G[N/mm2]	Shear modulus													

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	3996.5						0.00	0.00	0.0	0.0	
D ¹	2422.1						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force				Mb[kNm2]		warping moment				
Vy[kN],Vz[kN]	shear force				My[kNm],Mz[kNm]		bending moment				
Mt[kNm]	primary torsional moment				y[mm],z[mm]		ordinate of plastic centre				
Mt2[kNm]	secondary torsional moment				BUCK		buckling curve (LTB, y-y, z-z)				

FRP_bridge_v1
Cross Sections

Cross section No. 10 - Cable 73.1 1X7



Cross section No. 10 - Cable 73.1 1X7

Static properties of cross section

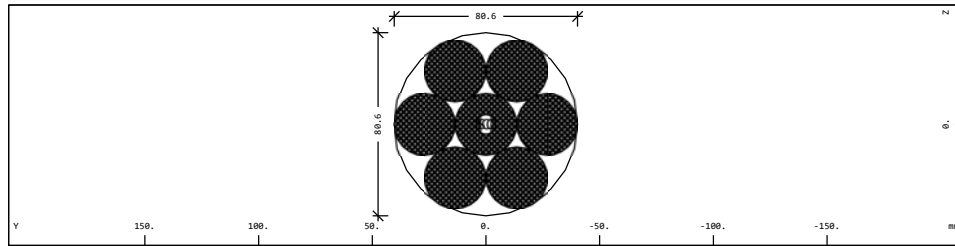
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]					
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]					
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$					
10	2	4.1947E-03		1.401E-06	0.0	0.0	195000	53.4						
		0.000E+00		1.401E-06	0.0	0.0	75000	(CABLE)						
= Cable 73.1 1X7														
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid							
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre							
A[m2]	sectional area				E[N/mm2]		Young's modulus							
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length							
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia													
I-1[m4],I-2[m4], $\alpha[^\circ]$	principal moments of inertia and angle of the principal axes													
MRf	reinforcement material number													
It[m4]	torsional moment of inertia													
G[N/mpg2]	Shear modulus													

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	4472.6						0.00	0.00	0.0	0.0	
D ¹	2710.7						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force				Mb[kNm2]		warping moment				
Vy[kN],Vz[kN]	shear force				My[kNm],Mz[kNm]		bending moment				
Mt[kNm]	primary torsional moment				y[mm],z[mm]		ordinate of plastic centre				
Mt2[kNm]	secondary torsional moment				BUCK		buckling curve (LTB, y-y, z-z)				

FRP_bridge_v1
Cross Sections

Cross section No. 11 - Cable 80.6 1X7



Cross section No. 11 - Cable 80.6 1X7

Static properties of cross section

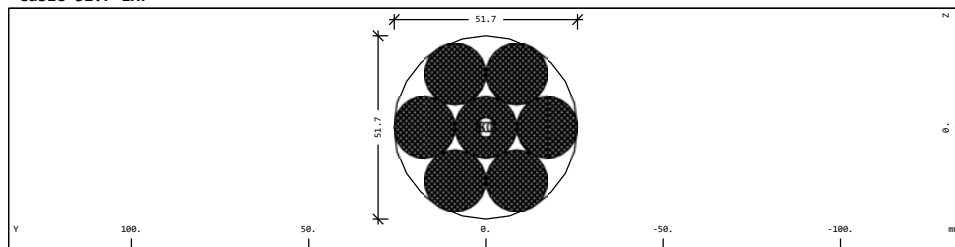
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]					
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]					
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$					
11	2	5.0996E-03		2.071E-06	0.0	0.0	195000	65.0						
		0.000E+00		2.071E-06	0.0	0.0	75000	(CABLE)						
= Cable 80.6 1X7														
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid							
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre							
A[m2]	sectional area				E[N/mm2]		Young's modulus							
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length							
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia													
I-1[m4],I-2[m4], $\alpha[^\circ]$	principal moments of inertia and angle of the principal axes													
MRf	reinforcement material number													
It[m4]	torsional moment of inertia													
G[N/mm2]	Shear modulus													

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	5437.5						0.00	0.00	0.0	0.0	
D ¹	3295.4						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force				Mb[kNm2]		warping moment				
Vy[kN],Vz[kN]	shear force				My[kNm],Mz[kNm]		bending moment				
Mt[kNm]	primary torsional moment				y[mm],z[mm]		ordinate of plastic centre				
Mt2[kNm]	secondary torsional moment				BUCK		buckling curve (LTB, y-y, z-z)				

FRP_bridge_v1
Cross Sections

Cross section No. 13 - Cable 51.7 1X7



Cross section No. 13 - Cable 51.7 1X7

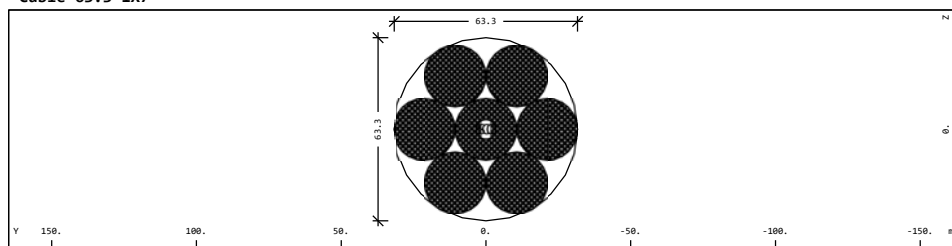
Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]					
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]					
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$					
13	2	2.0982E-03		3.505E-07	0.0	0.0	195000	26.7						
		0.000E+00		3.505E-07	0.0	0.0	75000	(CABLE)						
= Cable 51.7 1X7														
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid							
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre							
A[m2]	sectional area				E[N/mm2]		Young's modulus							
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length							
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia													
I-1[m4],I-2[m4], $\alpha[^\circ]$	principal moments of inertia and angle of the principal axes													
MRf	reinforcement material number													
It[m4]	torsional moment of inertia													
G[N/mpg2]	Shear modulus													

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	2237.2						0.00	0.00	0.0	0.0	
D ¹	1355.9						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force				Mb[kNm2]		warping moment				
Vy[kN],Vz[kN]	shear force				My[kNm],Mz[kNm]		bending moment				
Mt[kNm]	primary torsional moment				y[mm],z[mm]		ordinate of plastic centre				
Mt2[kNm]	secondary torsional moment				BUCK		buckling curve (LTB, y-y, z-z)				

Cross section No. 14 - Cable 63.3 1X7



Cross section No. 14 - Cable 63.3 1X7

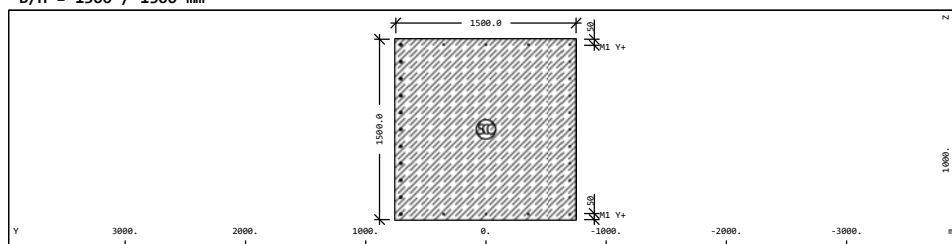
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α [°]
14	2	3.1454E-03		7.877E-07	0.0	0.0	195000	40.1	
		0.000E+00		7.877E-07	0.0	0.0	75000	(CABLE)	

= Cable 63.3 IX7			
SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m2]	sectional area	E[N/mm2]	Young's modulus
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia		
I-1[m4],I-2[m4],a["]	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m4]	torsional moment of inertia		
G[N/mm2]	Sbarg_modulus		

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	3353.8						0.00	0.00	0.0	0.0	
D ¹	2032.6						0.00	0.00	0.0	0.0	

¹ C/D = plastic characteristic/design values (ultimate bearing capacity)		
N[kN]	normal force	Mb[kNm2]
Vy[kN], Vz[kN]	shear force	My[kNm], Mz[kNm]
Mt[kNm]	primary torsional moment	y[mm], z[mm]
Mt2[kNm]	secondary torsional moment	BUCK
		buckling curve (LTB, v-v, z-z)

Cross section No. 15 - B/H = 1500 / 1500 mm

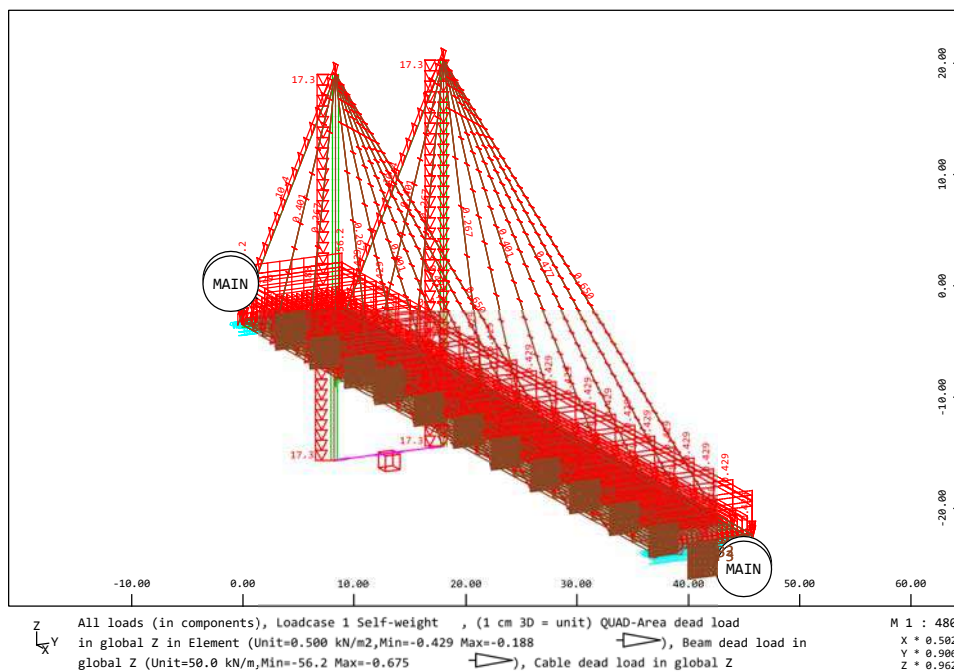


Cross section No. 15 - B/H = 1500 / 1500 mm

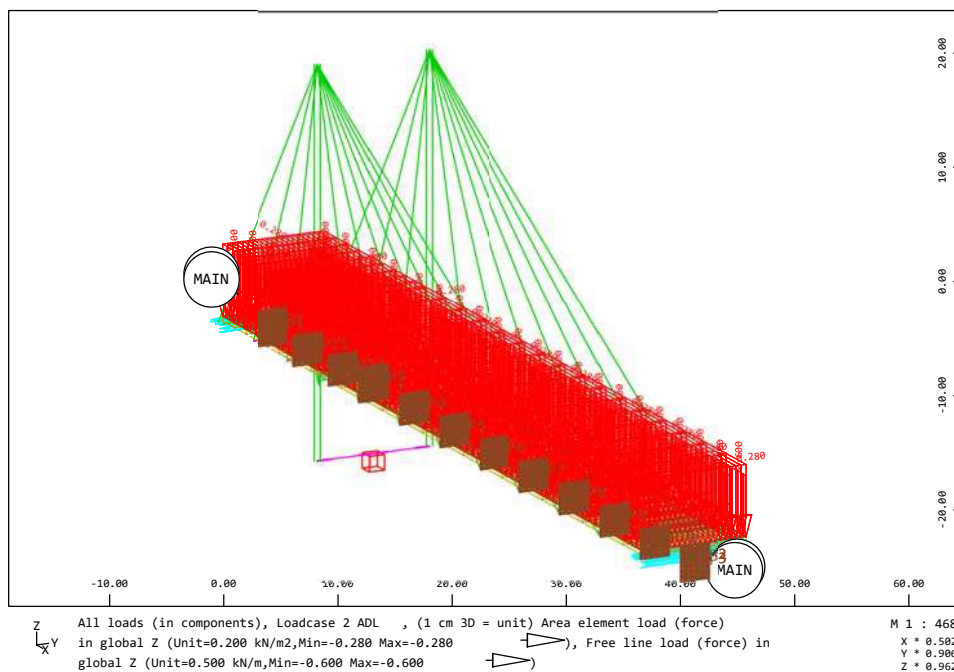
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α [°]
15	1	2.2500E+00	1.875E+00	4.219E-01	0.0	0.0	35220	5625.0	
	4	7.117E-01	1.875E+00	4.219E-01	750.0	750.0	14675	(BFAM)	

[illegible][illegible]

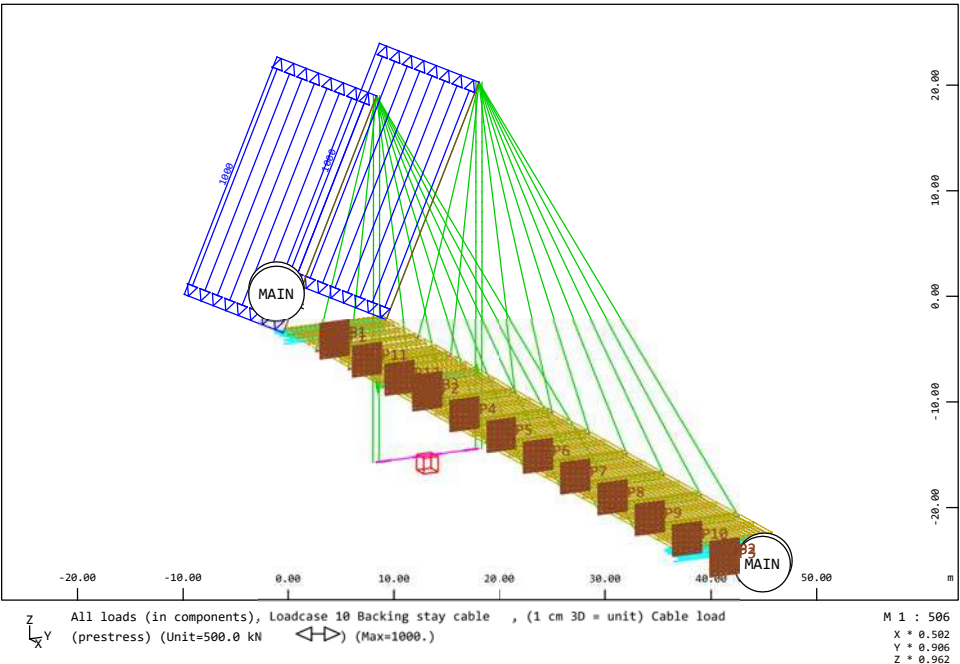
FRP_bridge_v1
Graphical Output



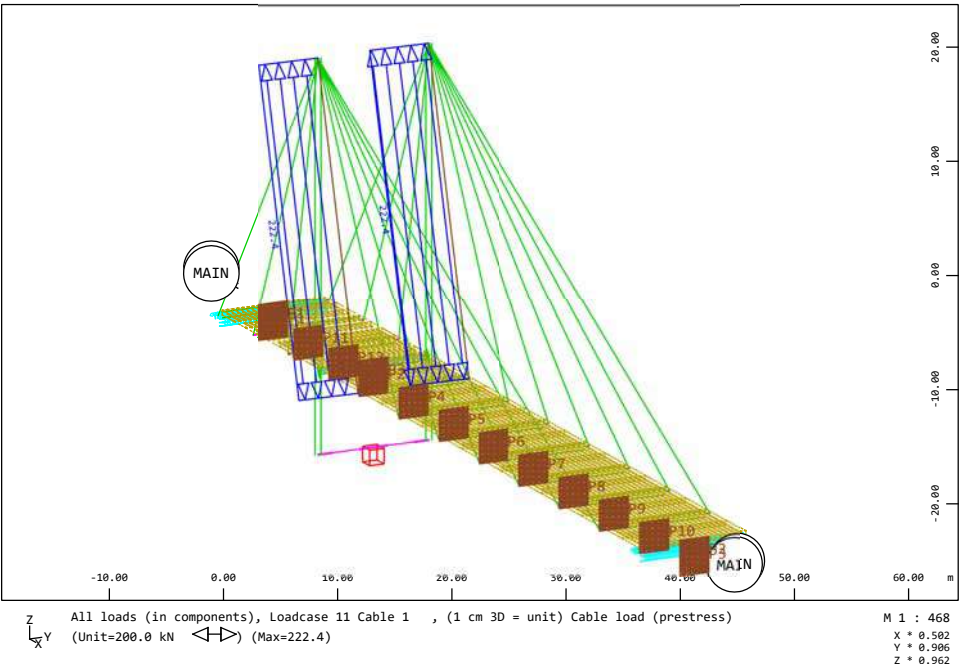
FRP_bridge_v1
Graphical Output



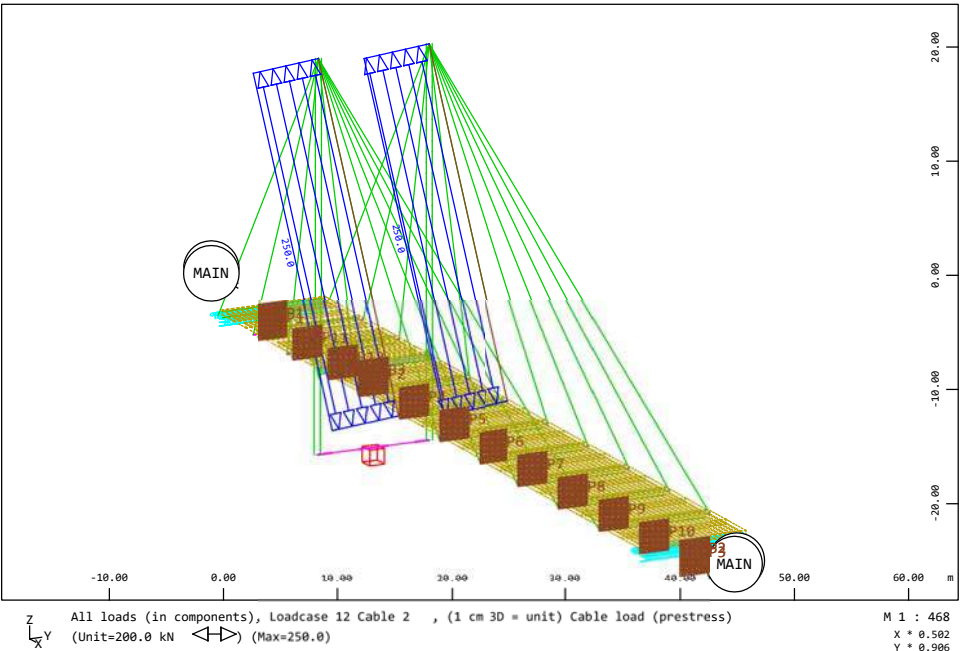
FRP_bridge_v1
Graphical Output



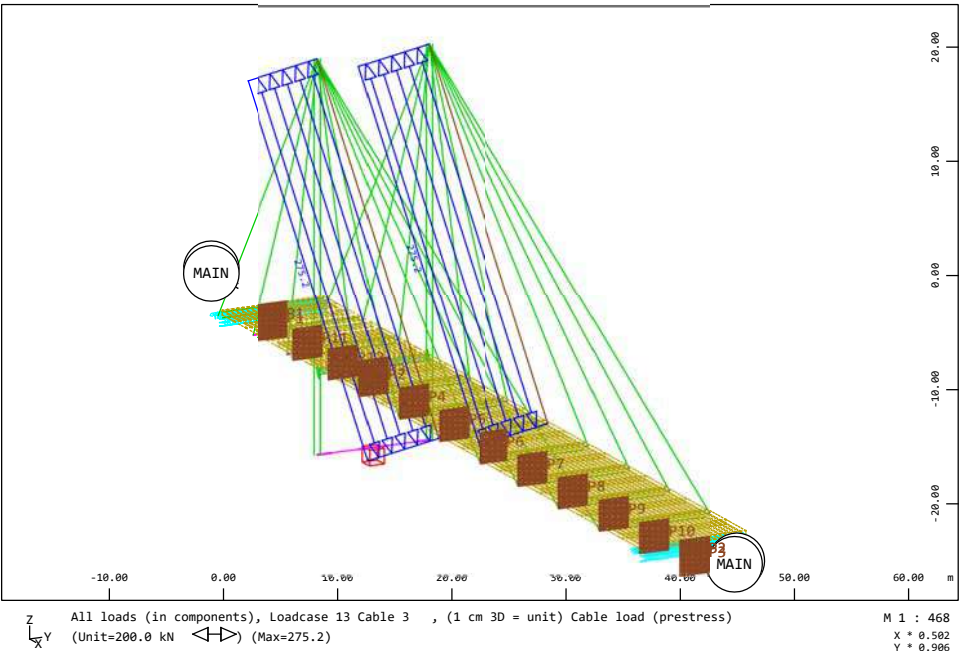
FRP_bridge_v1
Graphical Output



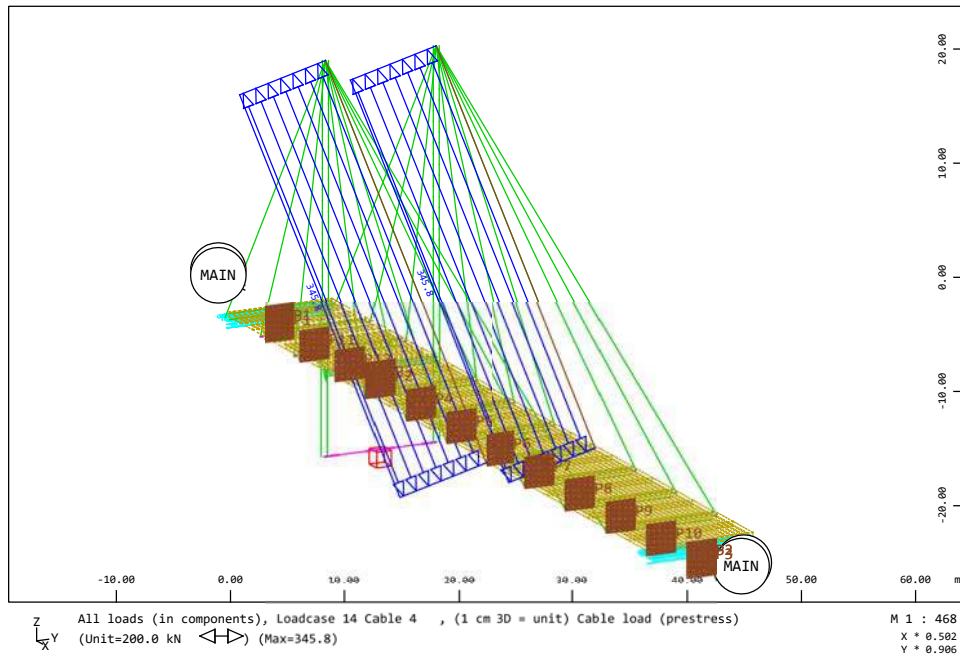
FRP_bridge_v1
Graphical Output



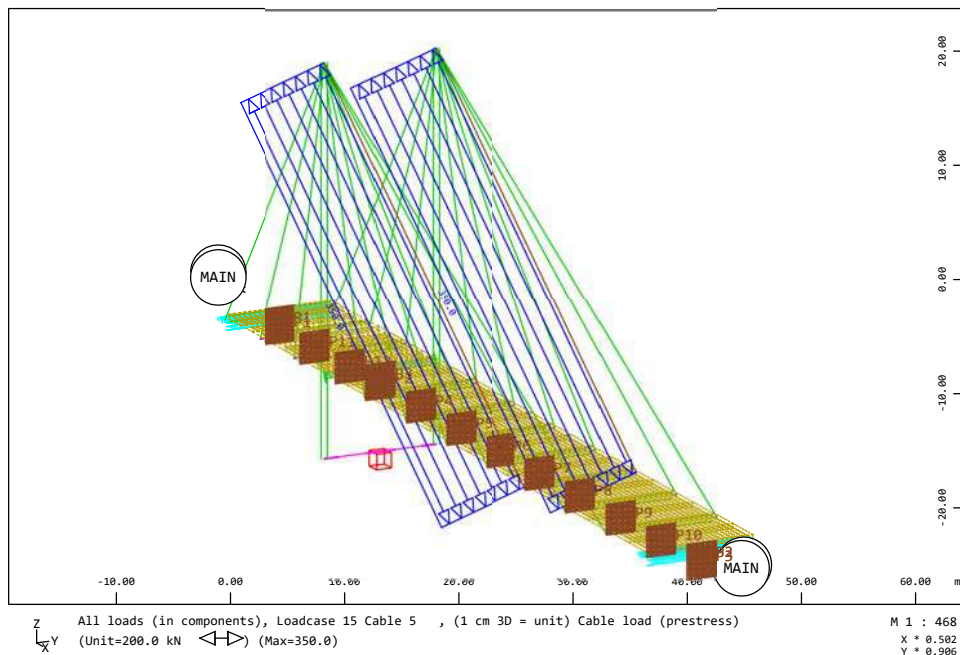
FRP_bridge_v1
Graphical Output



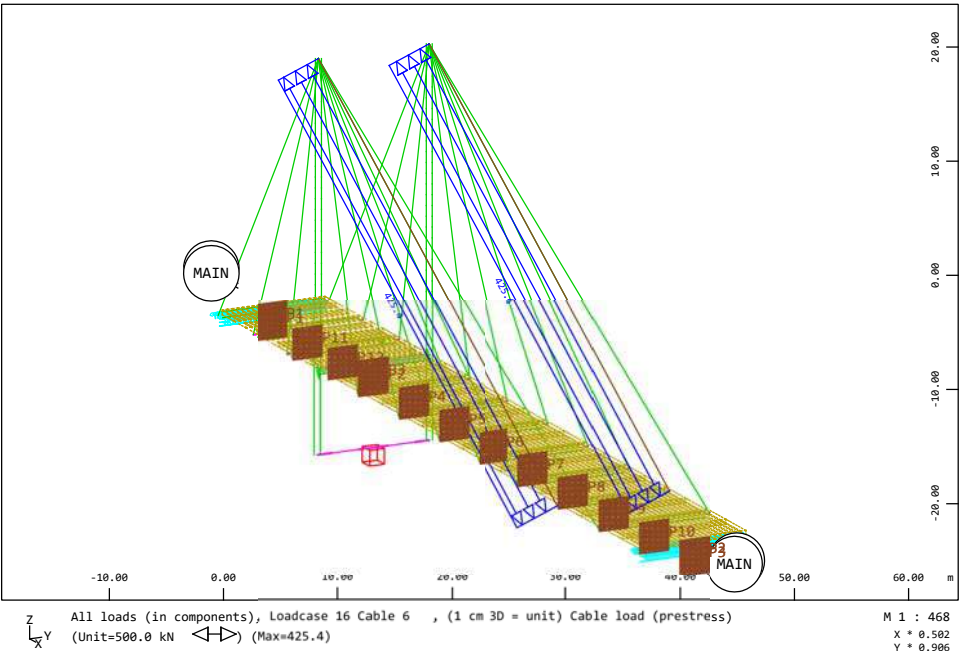
FRP_bridge_v1
Graphical Output



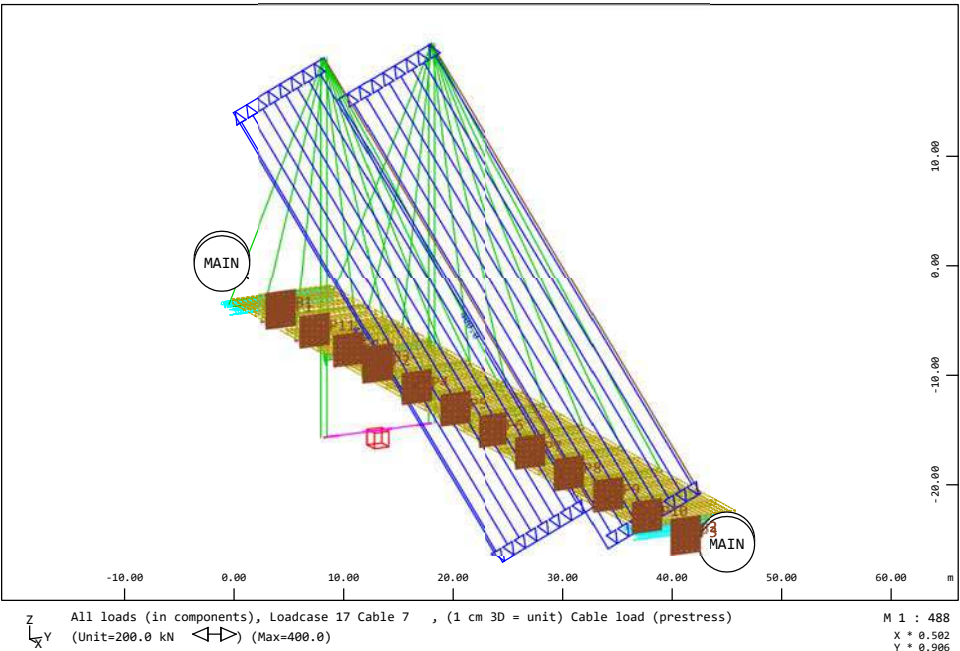
FRP_bridge_v1
Graphical Output



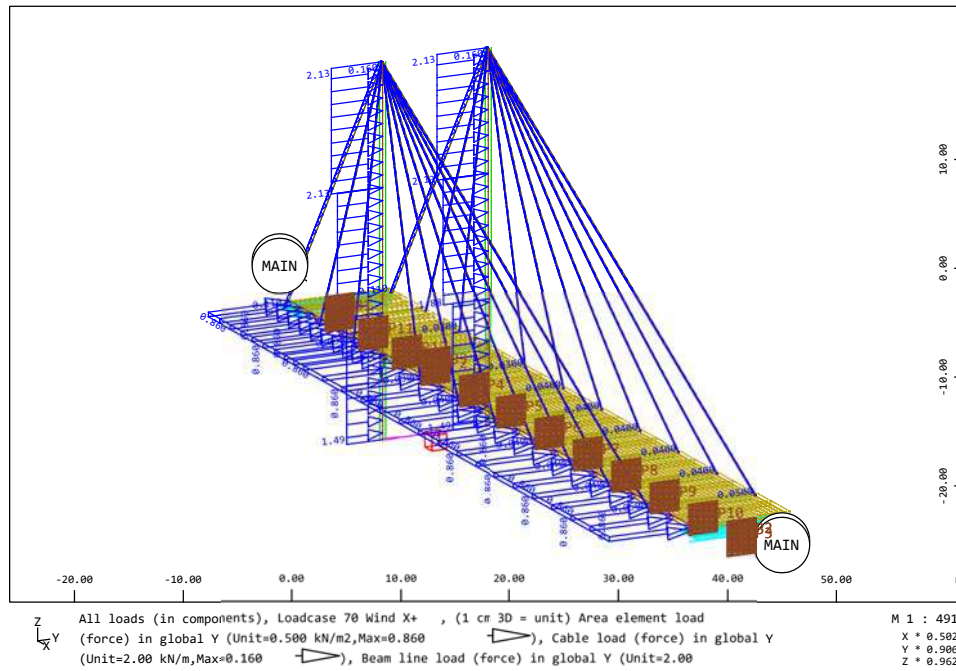
FRP_bridge_v1
Graphical Output



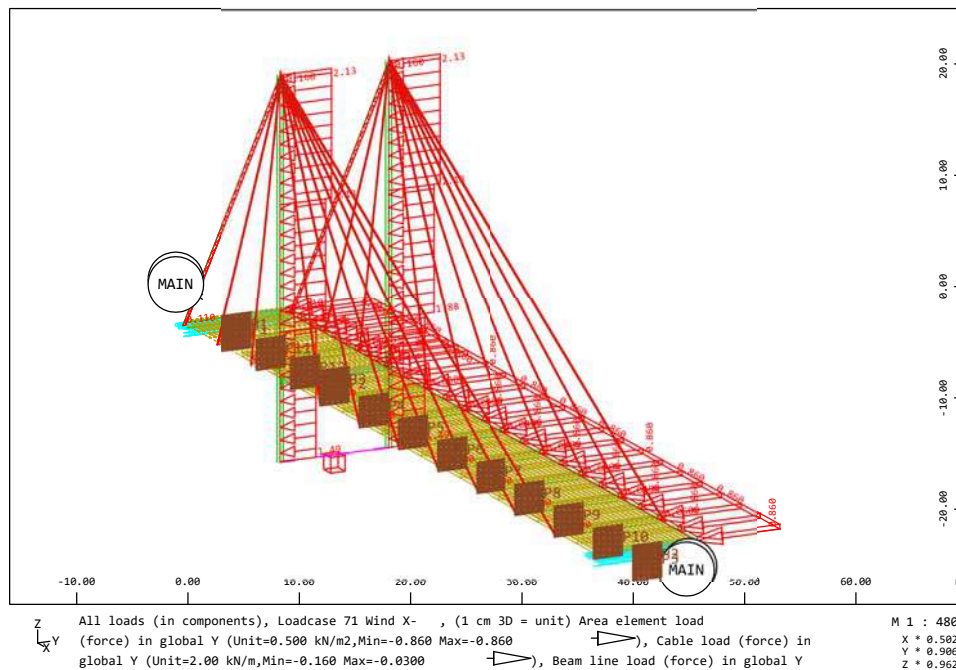
FRP_bridge_v1
Graphical Output



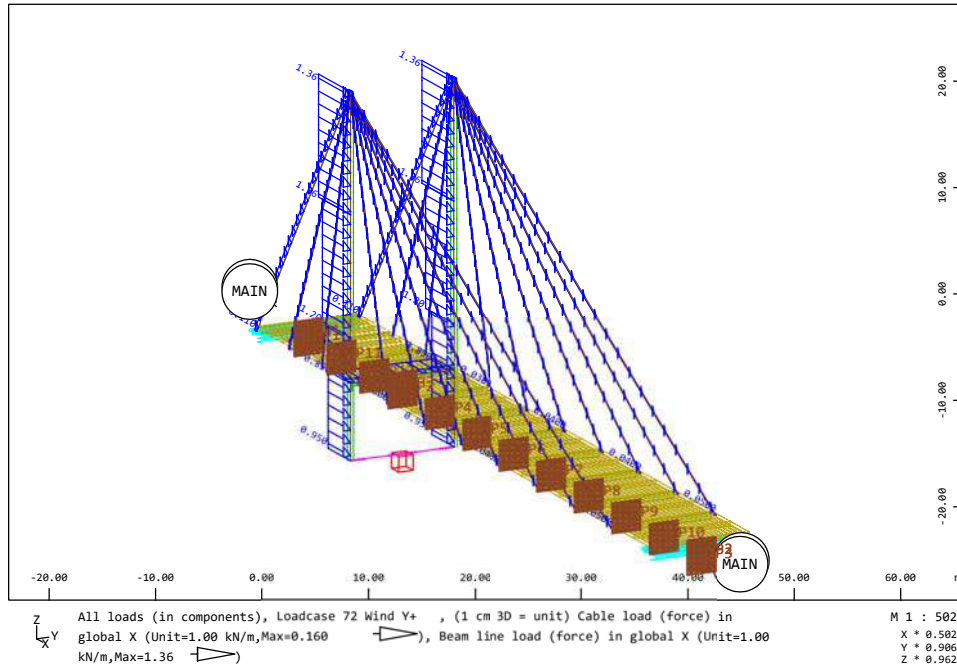
FRP_bridge_v1
Graphical Output



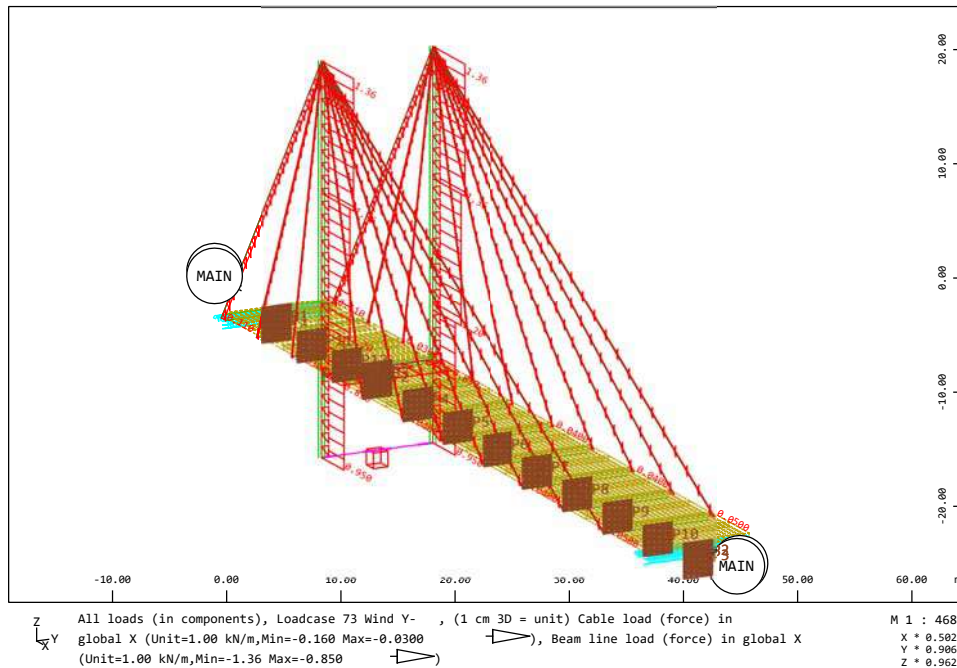
FRP_bridge_v1
Graphical Output



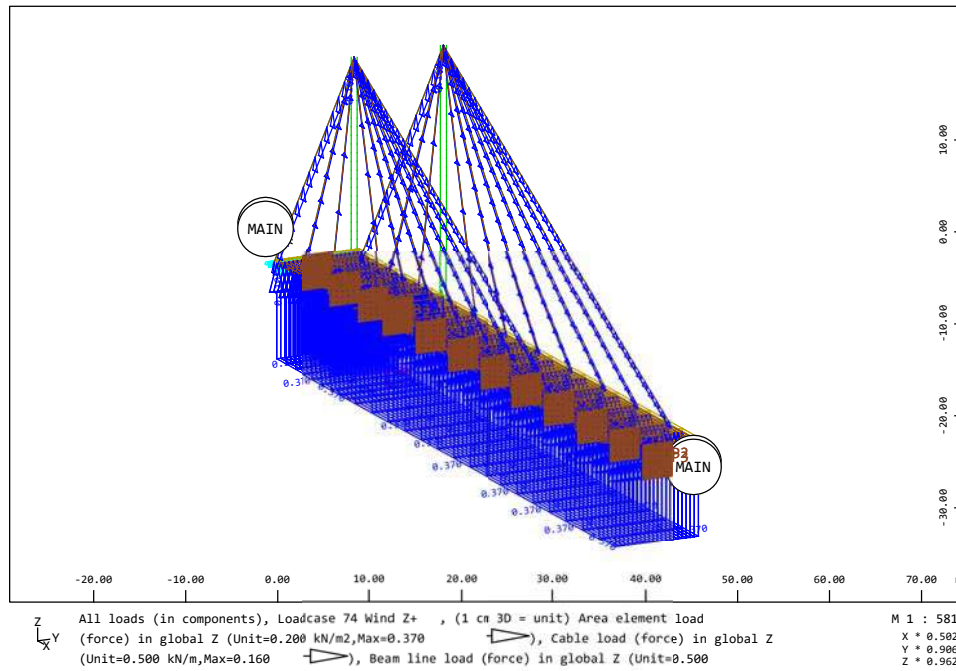
FRP_bridge_v1
Graphical Output



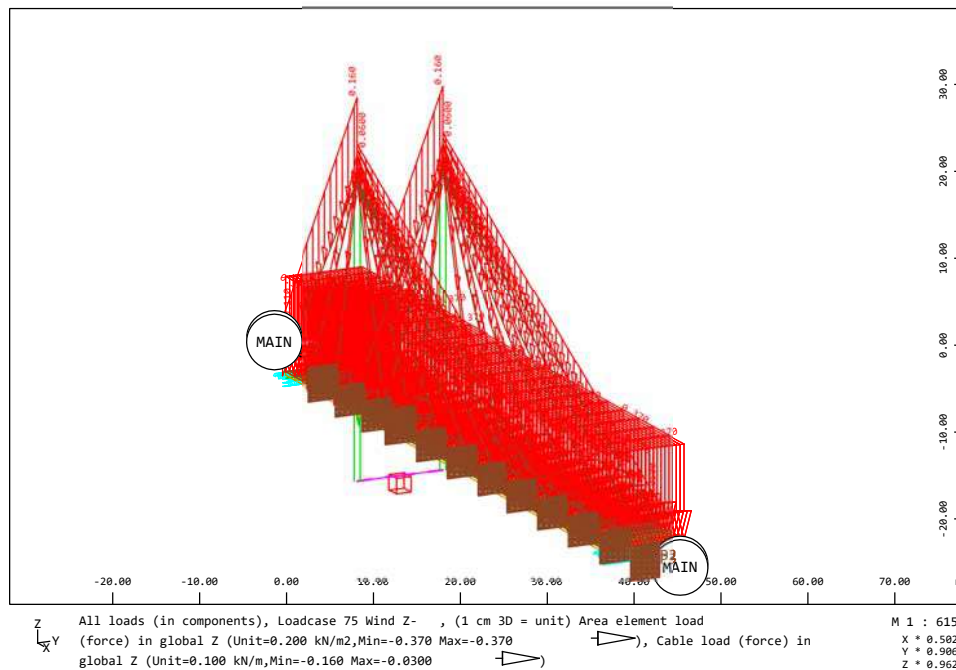
FRP_bridge_v1
Graphical Output



FRP_bridge_v1
Graphical Output



FRP_bridge_v1
Graphical Output



FRP_bridge_v1
Load models

Actions

type	part	sup	Designation	γ -u	γ -f	γ -a	ψ_0	ψ_1	ψ_2	ψ_{inf}
G_1	G	perm	dead load g1	1.35	1.00	1.00	1.00	1.00	1.00	1.00
G_2	G	perm	ADL	1.35	1.00	1.00	1.00	1.00	1.00	1.00
P	P	perm	prestressing	1.00	1.00	1.00	1.00	1.00	1.00	1.00
W	Q	excl	wind loading	1.50	0.00	1.00	0.60	0.20	0.00	0.60
GR_T	Q_1	excl	gria LM1	1.35	0.00	1.00	0.75	0.75	0.00	0.80
GR_U	Q_1	excl	gria LM1	1.35	0.00	1.00	0.40	0.40	0.00	0.80
			Reliability factor	Kfi	1.000					
			Reduction factor	xsi	0.890					
type	action	γ -u, γ -f, γ -a		partial safety factors for unfavourable/favourable/accidental						
part	partition of the action	$\psi_0,\psi_1,\psi_2,\psi_{inf}$		combination coefficients						
sup	superposition type									

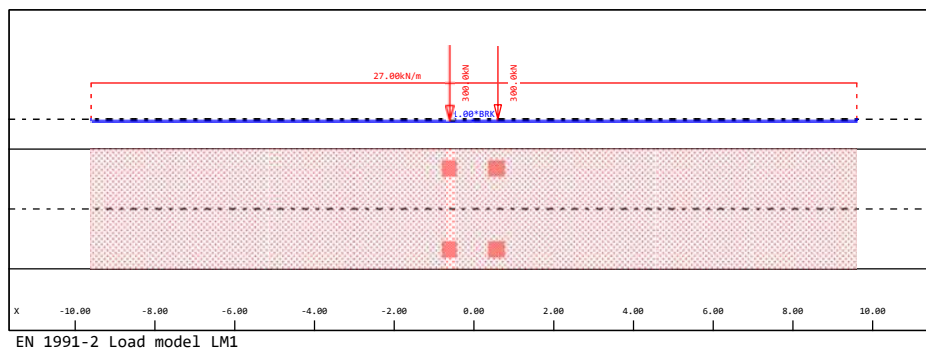
Load Train 100

LM1 300 EN 1991-2 Load model LM1

Load elements of Load Train	Load value	Remark
Load Train	300.000 [-]	
Axle load	300.0 [kN]	
Traffic Lane	9.00 [kN/m2]	
Residual Area	2.50 [kN/m2]	
Wind Load Height	3.500 [m]	
Brake load	900.0 [kN]	2.70 * L < 900 [kN]
Total factor	1.000 [-]	
Width of loading	3.000 [m]	
Fact.centrifugal	1.000 [-]	
Loading travels in both directions		

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FRP_bridge_v1
Load models



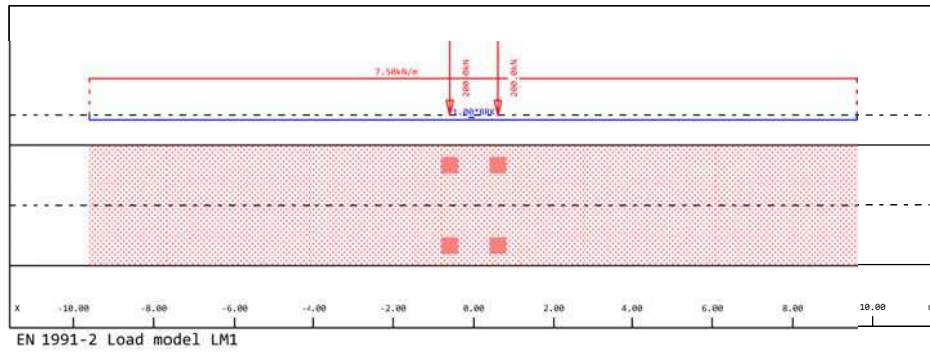
Load Train 101

LM1 200 EN 1991-2 Load model LM1

Load elements of Load Train	Load value	Remark
Load Train	200.000 [-]	
Axle load	200.0 [kN]	
Traffic Lane	2.50 [kN/m2]	
Residual Area	2.50 [kN/m2]	
Wind Load Height	3.500 [m]	
Brake load	900.0 [kN]	
Total factor	1.000 [-]	
Width of loading	3.000 [m]	
Fact.centrifugal	1.000 [-]	
Loading travels in both directions		

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FRP_bridge_v1
Load models



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FRP_bridge_v1
Superpositions

Superposition according to EuroNorm EN 1993-2:2006 Steel Structures

Combination rule Number 1

SLS characteristic combination

Superposition according to manual MAXIMA formula 2.4

$$E_{d,rare} = E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus Q_{k,1} \oplus \sum_{i \geq 1} \psi_{0,i} \cdot Q_{k,i} \right\}$$

Resulting Load Cases type SLS characteristic combination

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		γ_u	γ_f	γ_a	ψ_0	ψ_1	ψ_2	ψ_{1inf}			
G_1	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00	1.00	PERM	dead load g1
	1										Self-weight
G_2	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00	1.00	PERM	ADL
	2										ADL
GR_T	Q_1	1.35	0.00	1.00	0.75	0.75	0.00	0.80			gr1a LM1
	601								1.00	A42	MAX-PX NODE GR_T: my mx p px
	602								1.00	A42	MIN-PX NODE GR_T: my mx p px
	603								1.00	A42	MAX-PY NODE GR_T: vz my t py
	604								1.00	A42	MIN-PY NODE GR_T: vz my t py
	605								1.00	A42	MAX-PZ NODE GR_T: mt nx m pz
	606								1.00	A42	MIN-PZ NODE GR_T: mt nx m pz
	607								1.00	A42	MAX-UX NODE GR_T: n ny ptx ux
	608								1.00	A42	MIN-UX NODE GR_T: n ny ptx ux
	609								1.00	A42	MAX-UY NODE GR_T: mz vx pty uy
	610								1.00	A42	MIN-UY NODE GR_T: mz vx pty uy
	611								1.00	A42	MAX-UZ NODE GR_T: vy vy ptz uz
	612								1.00	A42	MIN-UZ NODE GR_T: vy vy ptz uz
	613								1.00	A42	MAX-MXY QUAD GR_T: mxy
	614								1.00	A42	MIN-MXY QUAD GR_T: mxy
	615								1.00	A42	MAX-NXY QUAD GR_T: nxy
	616								1.00	A42	MIN-NXY QUAD GR_T: nxy

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FRP_bridge_v1
Superpositions

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		γ-u	γ-f	γ-a	ψ ₀	ψ ₁	ψ ₂	ψ ₁ inf			
GR_U	Q_1	1.35	0.00	1.00	0.40	0.40	0.00	0.80			gr1a LM1
	681								1.00	A25	MAX-PX NODE GR_U: my mx p px
	682								1.00	A25	MIN-PX NODE GR_U: my mx p px
	683								1.00	A25	MAX-PY NODE GR_U: vz my t py
	684								1.00	A25	MIN-PY NODE GR_U: vz my t py
	685								1.00	A25	MAX-PZ NODE GR_U: mt nx m pz
	686								1.00	A25	MIN-PZ NODE GR_U: mt nx m pz
	687								1.00	A25	MAX-UX NODE GR_U: n ny ptx ux
	688								1.00	A25	MIN-UX NODE GR_U: n ny ptx ux
	689								1.00	A25	MAX-UY NODE GR_U: mz vx pty uy
	690								1.00	A25	MIN-UY NODE GR_U: mz vx pty uy
	691								1.00	A25	MAX-UZ NODE GR_U: vy vy ptz uz
	692								1.00	A25	MIN-UZ NODE GR_U: vy vy ptz uz
	693								1.00	A25	MAX-MXY QUAD GR_U: mxy
	694								1.00	A25	MIN-MXY QUAD GR_U: mxy
	695								1.00	A25	MAX-NXY QUAD GR_U: nxy
	696								1.00	A25	MIN-NXY QUAD GR_U: nxy
P	P	1.00	1.00	1.00	1.00	1.00	1.00	1.00			prestressing
	20								1.00	PERM	Prestress cables
W	Q	1.50	0.00	1.00	0.60	0.20	0.00	0.60			wind loading
	800								1.00	A10	Wind X+ Z+
	801								1.00	A10	Wind X+ Z-
	802								1.00	A10	Wind X- Z+
	803								1.00	A10	Wind X- Z-
	804								1.00	A10	Wind Y+ 0.4*X+
	805								1.00	A10	Wind Y+ 0.4*X-
	806								1.00	A10	Wind Y- 0.4*X+
	807								1.00	A10	Wind Y- 0.4*X-
Act	action									Fact	factor for load case
Part	partition of the action									Type	type of the load case
γ-u, γ-f, γ-a	partial safety factors for unfavourable/favourable/accidental									PERM	permanent load grouped in actions
ψ ₀ , ψ ₁ , ψ ₂ , ψ ₁ inf	combination coefficients									A	exclusive load
LC	number of the load case										

Combination rule Number 2

SLS frequent combination

Superposition according to manual MAXIMA formula 2.5

FRP_bridge_v1
Superpositions

$$E_{d,frequ} = E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \right\}$$

Resulting Load Cases type SLS frequent combination

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		γ-u	γ-f	γ-a	ψ ₀	ψ ₁	ψ ₂	ψ ₁ inf			
G_1	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00			dead load g1
	1								1.00	PERM	Self-weight
G_2	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00			ADL
	2								1.00	PERM	ADL
GR_T	Q_1	1.35	0.00	1.00	0.75	0.75	0.00	0.80			gr1a LM1
	601								1.00	A42	MAX-PX NODE GR_T: my mx p px
	602								1.00	A42	MIN-PX NODE GR_T: my mx p px
	603								1.00	A42	MAX-PY NODE GR_T: vz my t py
	604								1.00	A42	MIN-PY NODE GR_T: vz my t py
	605								1.00	A42	MAX-PZ NODE GR_T: mt nx m pz
	606								1.00	A42	MIN-PZ NODE GR_T: mt nx m pz
	607								1.00	A42	MAX-UX NODE GR_T: n ny ptx ux
	608								1.00	A42	MIN-UX NODE GR_T: n ny ptx ux
	609								1.00	A42	MAX-UY NODE GR_T: mz vx pty uy
	610								1.00	A42	MIN-UY NODE GR_T: mz vx pty uy
	611								1.00	A42	MAX-UZ NODE GR_T: vy vy ptz uz
	612								1.00	A42	MIN-UZ NODE GR_T: vy vy ptz uz
	613								1.00	A42	MAX-MXY QUAD GR_T: mxy
	614								1.00	A42	MIN-MXY QUAD GR_T: mxy
	615								1.00	A42	MAX-NXY QUAD GR_T: nxy
	616								1.00	A42	MIN-NXY QUAD GR_T: nxy

FRP_bridge_v1
Superpositions

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		γ-u	γ-f	γ-a	ψ ₀	ψ ₁	ψ ₂	ψ ₁ inf			
GR_U	Q_1	1.35	0.00	1.00	0.40	0.40	0.00	0.80			gr1a LM1
	681								1.00	A25	MAX-PX NODE GR_U: my mx p px
	682								1.00	A25	MIN-PX NODE GR_U: my mx p px
	683								1.00	A25	MAX-PY NODE GR_U: vz my t py
	684								1.00	A25	MIN-PY NODE GR_U: vz my t py
	685								1.00	A25	MAX-PZ NODE GR_U: mt nx m pz
	686								1.00	A25	MIN-PZ NODE GR_U: mt nx m pz
	687								1.00	A25	MAX-UX NODE GR_U: n ny ptx ux
	688								1.00	A25	MIN-UX NODE GR_U: n ny ptx ux
	689								1.00	A25	MAX-UY NODE GR_U: mz vx pty uy
	690								1.00	A25	MIN-UY NODE GR_U: mz vx pty uy
	691								1.00	A25	MAX-UZ NODE GR_U: vy vy ptz uz
	692								1.00	A25	MIN-UZ NODE GR_U: vy vy ptz uz
	693								1.00	A25	MAX-MXY QUAD GR_U: mxy
	694								1.00	A25	MIN-MXY QUAD GR_U: mxy
	695								1.00	A25	MAX-NXY QUAD GR_U: nxy
	696								1.00	A25	MIN-NXY QUAD GR_U: nxy
P	P	1.00	1.00	1.00	1.00	1.00	1.00	1.00			prestressing
	20								1.00	PERM	Prestress cables
W	Q	1.50	0.00	1.00	0.60	0.20	0.00	0.60			wind loading
	800								1.00	A10	Wind X+ Z+
	801								1.00	A10	Wind X+ Z-
	802								1.00	A10	Wind X- Z+
	803								1.00	A10	Wind X- Z-
	804								1.00	A10	Wind Y+ 0.4*X+
	805								1.00	A10	Wind Y+ 0.4*X-
	806								1.00	A10	Wind Y- 0.4*X+
	807								1.00	A10	Wind Y- 0.4*X-
Act	action									Fact	factor for load case
Part	partition of the action									Type	type of the load case
γ-u, γ-f, γ-a	partial safety factors for unfavourable/favourable/accidental									PERM	permanent load grouped in actions
ψ ₀ , ψ ₁ , ψ ₂ , ψ ₁ inf	combination coefficients									A	exclusive load
LC	number of the load case										

Combination rule Number 3

SLS quasi-permanent combination

Superposition according to manual MAXIMA formula 2.7

FRP_bridge_v1
Superpositions

$$E_{d,perm} = E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \right\}$$

Resulting Load Cases type SLS quasi-permanent combination

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		γ-u	γ-f	γ-a	ψ ₀	ψ ₁	ψ ₂	ψ ₁ inf			
G_1	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00			dead load g1
	1								1.00	PERM	Self-weight
G_2	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00			ADL
	2								1.00	PERM	ADL
GR_T	Q_1	1.35	0.00	1.00	0.75	0.75	0.00	0.80			gr1a LM1
	601								1.00	A42	MAX-PX NODE GR_T: my mx p px
	602								1.00	A42	MIN-PX NODE GR_T: my mx p px
	603								1.00	A42	MAX-PY NODE GR_T: vz my t py
	604								1.00	A42	MIN-PY NODE GR_T: vz my t py
	605								1.00	A42	MAX-PZ NODE GR_T: mt nx m pz
	606								1.00	A42	MIN-PZ NODE GR_T: mt nx m pz
	607								1.00	A42	MAX-UX NODE GR_T: n ny ptx ux
	608								1.00	A42	MIN-UX NODE GR_T: n ny ptx ux
	609								1.00	A42	MAX-UY NODE GR_T: mz vx pty uy
	610								1.00	A42	MIN-UY NODE GR_T: mz vx pty uy
	611								1.00	A42	MAX-UZ NODE GR_T: vy vy ptz uz
	612								1.00	A42	MIN-UZ NODE GR_T: vy vy ptz uz
	613								1.00	A42	MAX-MXY QUAD GR_T: mxy
	614								1.00	A42	MIN-MXY QUAD GR_T: mxy
	615								1.00	A42	MAX-NXY QUAD GR_T: nxy
	616								1.00	A42	MIN-NXY QUAD GR_T: nxy

FRP_bridge_v1
Superpositions

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		$\gamma-u$	$\gamma-f$	$\gamma-a$	ψ_0	ψ_1	ψ_2	ψ_{1inf}			
GR_U	Q_1	1.35	0.00	1.00	0.40	0.40	0.00	0.80			gr1a LM1
	681								1.00	A25	MAX-PX NODE GR_U: my mx p px
	682								1.00	A25	MIN-PX NODE GR_U: my mx p px
	683								1.00	A25	MAX-PY NODE GR_U: vz my t py
	684								1.00	A25	MIN-PY NODE GR_U: vz my t py
	685								1.00	A25	MAX-PZ NODE GR_U: mt nx m pz
	686								1.00	A25	MIN-PZ NODE GR_U: mt nx m pz
	687								1.00	A25	MAX-UX NODE GR_U: n ny ptx ux
	688								1.00	A25	MIN-UX NODE GR_U: n ny ptx ux
	689								1.00	A25	MAX-UY NODE GR_U: mz vx pty uy
	690								1.00	A25	MIN-UY NODE GR_U: mz vx pty uy
	691								1.00	A25	MAX-UZ NODE GR_U: vy vy ptz uz
	692								1.00	A25	MIN-UZ NODE GR_U: vy vy ptz uz
	693								1.00	A25	MAX-MXY QUAD GR_U: mxy
	694								1.00	A25	MIN-MXY QUAD GR_U: mxy
	695								1.00	A25	MAX-NXY QUAD GR_U: nxy
	696								1.00	A25	MIN-NXY QUAD GR_U: nxy
P	P	1.00	1.00	1.00	1.00	1.00	1.00	1.00			prestressing
	20								1.00	PERM	Prestress cables
W	Q	1.50	0.00	1.00	0.60	0.20	0.00	0.60			wind loading
	800								1.00	A10	Wind X+ Z+
	801								1.00	A10	Wind X+ Z-
	802								1.00	A10	Wind X- Z+
	803								1.00	A10	Wind X- Z-
	804								1.00	A10	Wind Y+ 0.4*X+
	805								1.00	A10	Wind Y+ 0.4*X-
	806								1.00	A10	Wind Y- 0.4*X+
	807								1.00	A10	Wind Y- 0.4*X-
Act	action									Fact	factor for load case
Part	partition of the action									Type	type of the load case
$\gamma-u, \gamma-f, \gamma-a$	partial safety factors for unfavourable/favourable/accidental									PERM	permanent load grouped in actions
$\psi_0, \psi_1, \psi_2, \psi_{1inf}$	combination coefficients									A	exclusive load
LC	number of the load case										

Combination rule Number 4

ULS fundamental combination

Superposition according to manual MAXIMA formula 2.1

FRP_bridge_v1
Superpositions

$$E_d = E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus \gamma_P \cdot P_k \oplus \gamma_{Q,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \right\}$$

Resulting Load Cases type ULS fundamental combination

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		$\gamma-u$	$\gamma-f$	$\gamma-a$	ψ_0	ψ_1	ψ_2	ψ_{1inf}			
G_1	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00			dead load g1
	1								1.00	PERM	Self-weight
G_2	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00			ADL
	2								1.00	PERM	ADL
GR_T	Q_1	1.35	0.00	1.00	0.75	0.75	0.00	0.80			gr1a LM1
	601								1.00	A42	MAX-PX NODE GR_T: my mx p px
	602								1.00	A42	MIN-PX NODE GR_T: my mx p px
	603								1.00	A42	MAX-PY NODE GR_T: vz my t py
	604								1.00	A42	MIN-PY NODE GR_T: vz my t py
	605								1.00	A42	MAX-PZ NODE GR_T: mt nx m pz
	606								1.00	A42	MIN-PZ NODE GR_T: mt nx m pz
	607								1.00	A42	MAX-UX NODE GR_T: n ny ptx ux
	608								1.00	A42	MIN-UX NODE GR_T: n ny ptx ux
	609								1.00	A42	MAX-UY NODE GR_T: mz vx pty uy
	610								1.00	A42	MIN-UY NODE GR_T: mz vx pty uy
	611								1.00	A42	MAX-UZ NODE GR_T: vy vy ptz uz
	612								1.00	A42	MIN-UZ NODE GR_T: vy vy ptz uz
	613								1.00	A42	MAX-MXY QUAD GR_T: mxy
	614								1.00	A42	MIN-MXY QUAD GR_T: mxy
	615								1.00	A42	MAX-NXY QUAD GR_T: nxy
	616								1.00	A42	MIN-NXY QUAD GR_T: nxy

FRP_bridge_v1
Superpositions

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		$\gamma-u$	$\gamma-f$	$\gamma-a$	ψ_0	ψ_1	ψ_2	ψ_{inf}			
GR_U	Q_1	1.35	0.00	1.00	0.40	0.40	0.00	0.80			gr1a LM1
	681								1.00	A25	MAX-PX NODE GR_U: my mx p px
	682								1.00	A25	MIN-PX NODE GR_U: my mx p px
	683								1.00	A25	MAX-PY NODE GR_U: vz my t py
	684								1.00	A25	MIN-PY NODE GR_U: vz my t py
	685								1.00	A25	MAX-PZ NODE GR_U: mt nx m pz
	686								1.00	A25	MIN-PZ NODE GR_U: mt nx m pz
	687								1.00	A25	MAX-UX NODE GR_U: n ny ptx ux
	688								1.00	A25	MIN-UX NODE GR_U: n ny ptx ux
	689								1.00	A25	MAX-UY NODE GR_U: mz vx pty uy
	690								1.00	A25	MIN-UY NODE GR_U: mz vx pty uy
	691								1.00	A25	MAX-UZ NODE GR_U: vy vy ptz uz
	692								1.00	A25	MIN-UZ NODE GR_U: vy vy ptz uz
	693								1.00	A25	MAX-MXY QUAD GR_U: mxy
	694								1.00	A25	MIN-MXY QUAD GR_U: mxy
	695								1.00	A25	MAX-NXY QUAD GR_U: nxy
	696								1.00	A25	MIN-NXY QUAD GR_U: nxy
P	P	1.00	1.00	1.00	1.00	1.00	1.00	1.00			prestressing
	20								1.00	PERM	Prestress cables
W	Q	1.50	0.00	1.00	0.60	0.20	0.00	0.60			wind loading
	800								1.00	A10	Wind X+ Z+
	801								1.00	A10	Wind X+ Z-
	802								1.00	A10	Wind X- Z+
	803								1.00	A10	Wind X- Z-
	804								1.00	A10	Wind Y+ 0.4*X+
	805								1.00	A10	Wind Y+ 0.4*X-
	806								1.00	A10	Wind Y- 0.4*X+
	807								1.00	A10	Wind Y- 0.4*X-
Act	action									Fact	factor for load case
Part	partition of the action									Type	type of the load case
$\gamma-u, \gamma-f, \gamma-a$	partial safety factors for unfavourable/favourable/accidental									PERM	permanent load grouped in actions
$\psi_0, \psi_1, \psi_2, \psi_{inf}$	combination coefficients									A	exclusive load
LC	number of the load case										

FRP_bridge_v1
ULS design beams

Default design code is EuroNorm EN 1993-2:2006 Steel Structures (Norge) V 2018
Structure: B (Road bridges)

Materials

Mat	Classification
1	C 40/50 (EN 1992)
2	Y 1860 A (EN 1992)
3	GFK EP
4	B 500 B (EN 1992)

Selected Beam Elements

Selection	NoA	NoE	x[m]	Type
BEAM	all elements			
NoA, NoE	range of element numbers			
x[m]	x-ordinate of beam section or station on axis			
Type	element type			

Reinforcement will be accounted for sectional values as defined in AQUA
Reinforcements saved as Design case No. 1

Considered Load Cases

LC	ACT	REF	CS	Designation
2101	(D)			MAX-PX NODE
2102	(D)			MIN-PX NODE
2103	(D)			MAX-PY NODE
2104	(D)			MIN-PY NODE
2105	(D)			MAX-PZ NODE
2106	(D)			MIN-PZ NODE
2107	(D)			MAX-UX NODE design N nyy U
2108	(D)			MIN-UX NODE design N nyy U
2109	(D)			MAX-UY NODE design MZ vx U
2110	(D)			MIN-UY NODE design MZ vx U
2111	(D)			MAX-UZ NODE design VY vy U
2112	(D)			MIN-UZ NODE design VY vy U
LC	load case			
ACT	action			
REF	reference point for forces and moments			
CS	section the load case is acting on			

FRP_bridge_v1
ULS design beams

Longitudinal Reinforcements - Design case No. 1

Beam	x[m]	SNo	p [o/o]	As1 [cm2]	vm [m]	As1-0 [cm2]	As1-1 [cm2]	As1-2 [cm2]	As1-3 [cm2]	As1-4 [cm2]	As1-5 [cm2]
Maxim. values	3	1.89	130.53	1.416			48.13T	48.13T	17.12T	17.14T	
Maxim. values	4	3.41	136.45	0.900			106.80T	22.65T	7.00T		
Maxim. values	15	1.52	341.31	1.798			127.62T	90.20T	123.49T		

p geometric part of reinforcements
As1 total longitudinal reinforcement
vm shift rule of longitudinal reinforcement (0.0 if already included by normal force)
As1-0,As1-1,As1-2,As1-3,As1-4,As1-5 longitudinal reinforcement per layer

Shear Reinforcements per Cutted Part of Section - Design case No. 1

Beam	x[m]	SNo	As1-Mt [cm2/m]	As/s [cm2/m]	As/s-1 [cm2/m]
Maxim. values	15	41.83		3.04	
Maxim. values	15	41.83		22.80	
Maxim. values	15	41.83		40.27	

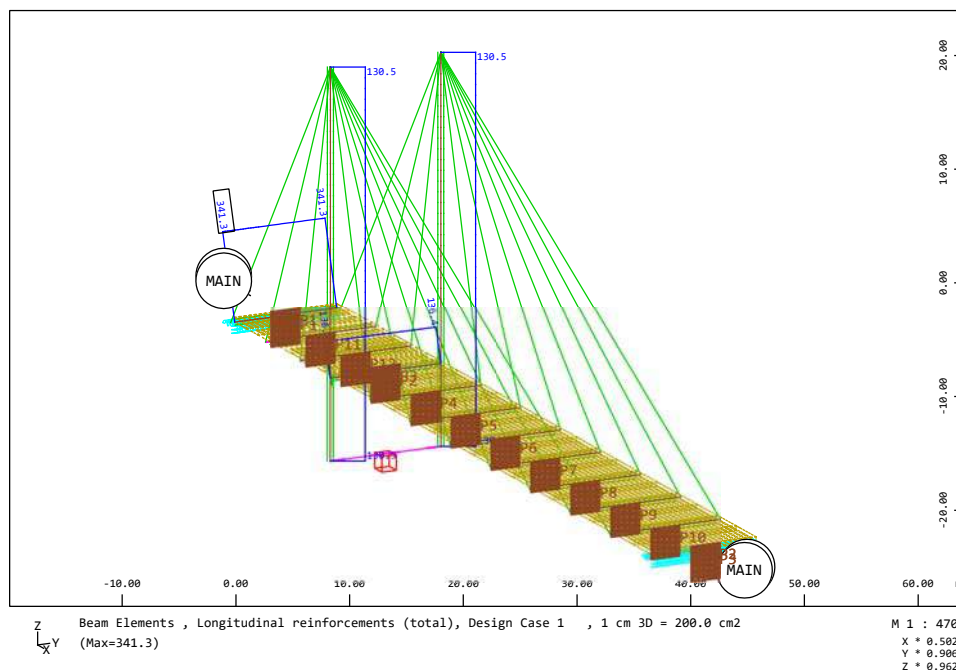
As1-Mt nominal longitudinal reinforcement per circumference of equivalent section due to torsion
As/s area of transverse reinforcements
As/s-1 total transverse reinforcement per layer and cutted element

Maximum Utilisation Level

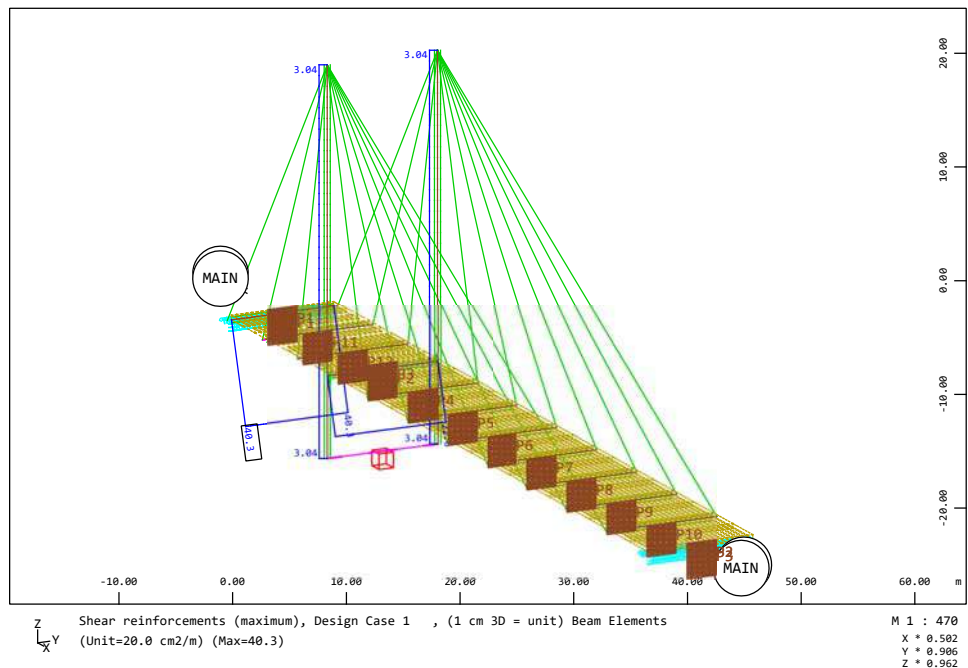
	N	Vy	Vz	My	Mz	Mtp	Mts	Mb	Ncr	SCL	Total
	$\sigma-x$	$\sigma+x$	τ	$\sigma-v$	$\sigma-s$	$\sigma-dyn$	As-l	As-v	crack	c/t	
Section 3	0.000	0.000	0.049	0.000	0.000	0.030	0.000	0.000	-	-	1.000
Pylon	0.000	0.000	0.000	0.066	-	-	1.000	0.210	-	-	
Section 4	0.000	0.000	0.800	0.000	0.000	0.111	0.000	0.000	-	-	1.000
B/H = 500 / 800 mm	0.000	0.000	0.000	0.833	-	-	1.000	1.000	-	-	
Section 5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	-	0.414
Cross girder	0.000	0.000	0.000	0.000	-	-	-	-	-	-	
Section 15	0.000	0.000	0.476	0.000	0.000	0.522	0.000	0.000	-	-	1.000
B/H = 1500 / 1500 mm	0.000	0.000	0.000	0.998	-	-	1.000	1.000	-	-	
Total	0.000	0.000	0.800	0.000	0.000	0.522	0.000	0.000	-	-	1.000
	0.000	0.000	0.000	0.998	-	-	1.000	1.000	-	-	

N normal force
Vy,Vz shear force
My,Mz bending
Mtp,Mts torsion (p)rimary and (s)econdary
Mb warping moment
Ncr flexural buckling
SCL cross-section class
 $\sigma-x$ longitud. compressive stress
 $\sigma+x$ longitud. tensile stress
 τ shear stress
 $\sigma-v$ von Mises stress
 $\sigma-s$ stress in reinforcements
 $\sigma-dyn$ stress range
As-l longitudinal reinforcements
As-v transverse reinforcements
crack crack width
c/t stress dependant utilisation level (see AQB Manual 2.3.2)
Total most unfavorable utilisation for all checks

FRP_bridge_v1
Graphical Output



FRP_bridge_v1
Graphical Output



FRP_bridge_v1
SLS design beams

Default design code is EuroNorm EN 1993-2:2006 Steel Structures (Norge) V 2018
Structure: B (Road bridges)

Materials

Mat	Classification
1	C 40/50 (EN 1992)
2	Y 1860 A (EN 1992)
3	GFK EP
4	B 500 B (EN 1992)

Selected Beam Elements

Selection	NoA	NoE	x[m]	Type
BEAM	all elements			
NoA,NoE range of element numbers				
x[m] x-ordinate of beam section or station on axis				
Type element type				

Reinforcement will be accounted for sectional values as defined in AQUA
Reinforcements saved as Design case No. 2
Reinforcements superposed with existing Design case No. 1

Considered Load Cases

LC	ACT	REF	CS	Designation
1301	(F)			MAXF-PX NODE
1302	(F)			MINF-PX NODE
1303	(F)			MAXF-PY NODE
1304	(F)			MINF-PY NODE
1305	(F)			MAXF-PZ NODE
1306	(F)			MINF-PZ NODE
1307	(F)			MAXF-UX NODE freq N nyy UX
1308	(F)			MINF-UX NODE freq N nyy UX
1309	(F)			MAXF-UY NODE freq MZ vx UY
1310	(F)			MINF-UY NODE freq MZ vx UY
1311	(F)			MAXF-UZ NODE freq VY vy UZ
1312	(F)			MINF-UZ NODE freq VY vy UZ
LC load case REF reference point for forces and moments				
ACT action CS section the load case is acting on				

Maximum Utilisation Level

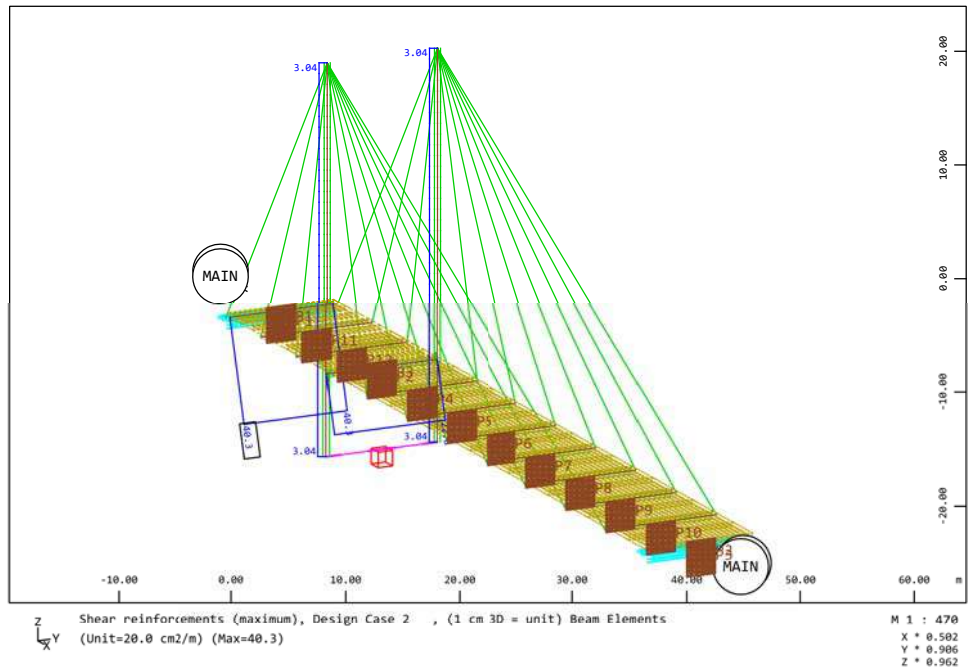
N	normal force	T	shear stress
Vy,Vz	shear force	$\sigma-v$	von Mises stress
My,Mz	bending	$\sigma-s$	stress in reinforcements
Mtp,Mts	torsion (p)primary and (s)econdary	$\sigma-dyn$	stress range
Mb	warping moment	As-l	longitudinal reinforcements
Ncr	flexural buckling	As-v	transverse reinforcements
SCl	cross-section class	crack	crack width
$\sigma-x$	longitud. compressive stress	c/t	stress dependant utilisation level (see AQB Manual 2.3.2)
$\sigma-x$	longitud. tensile stress	Total	most unfavorable utilisation for all checks

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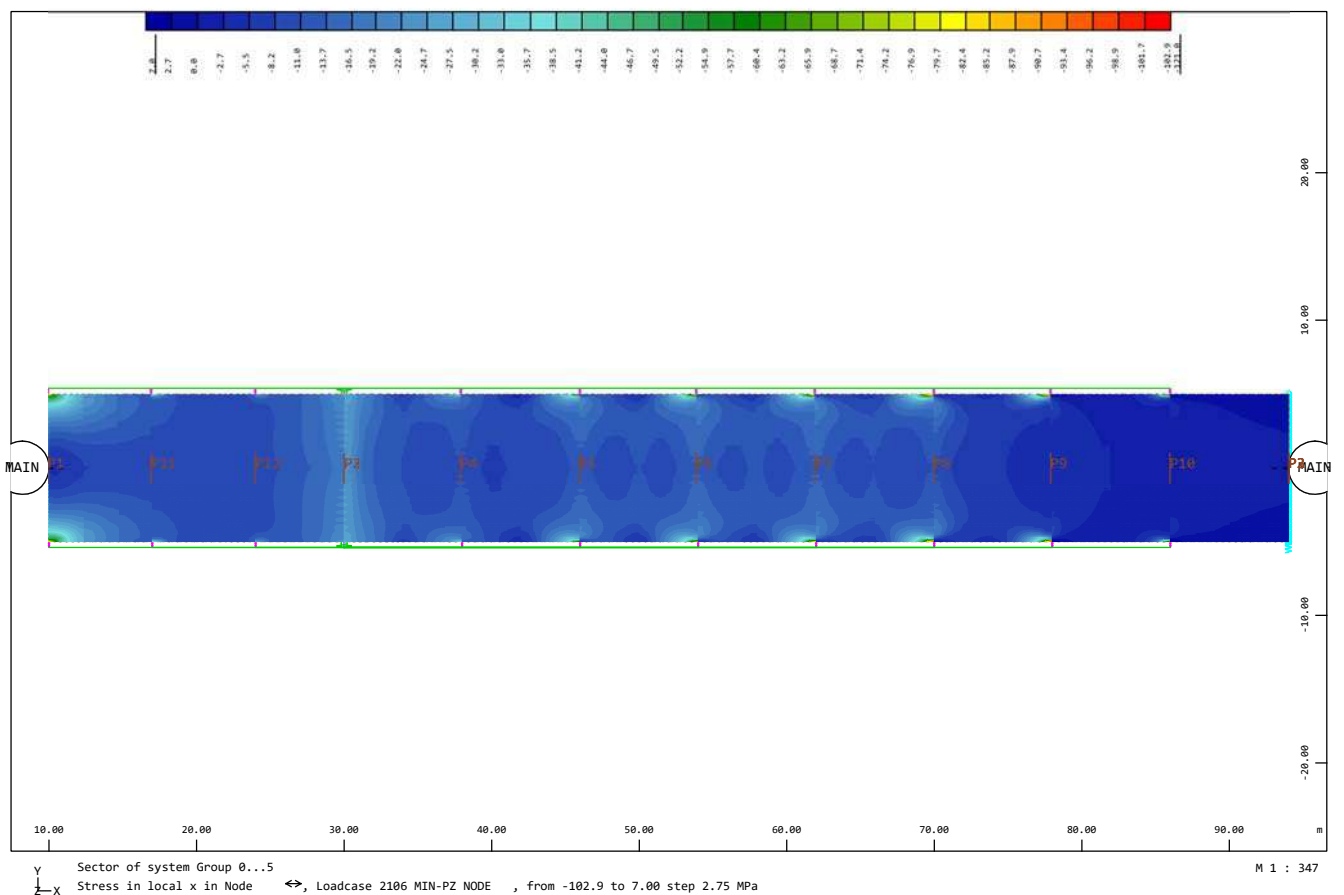
Graphical Output



FRP_bridge_v1
 Graphical Output

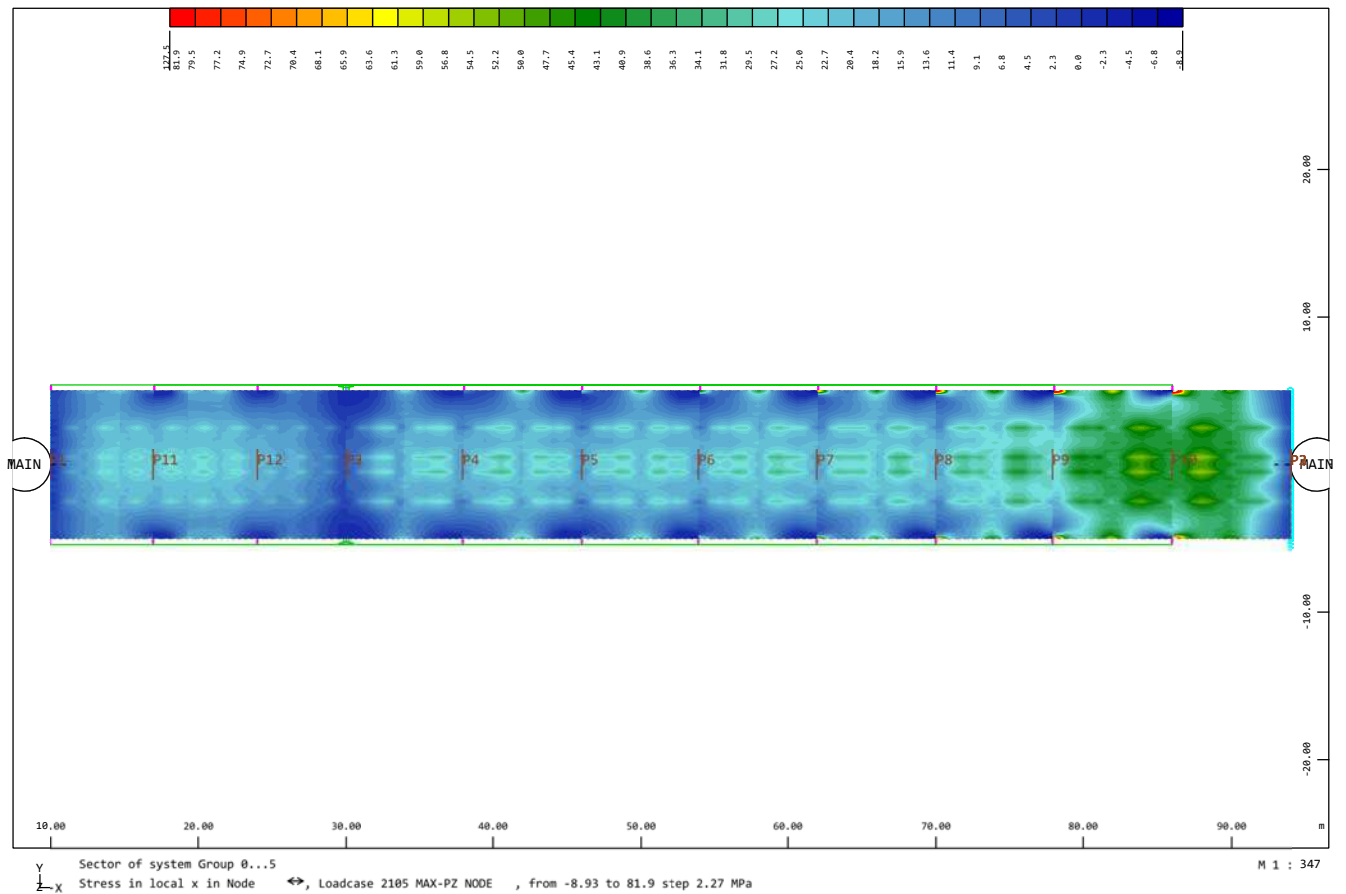


FRP_bridge_v1



FRP_bridge_v1

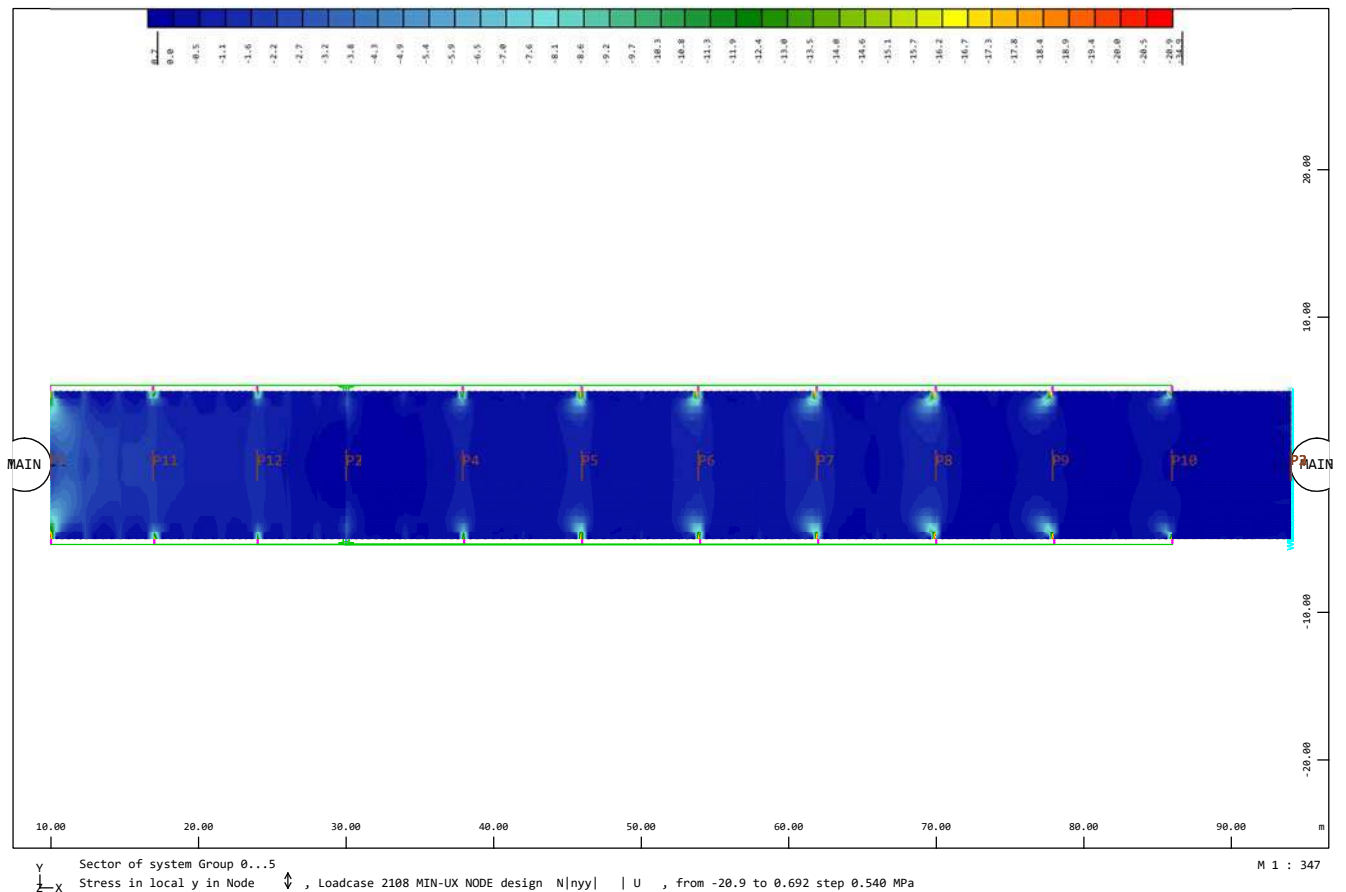
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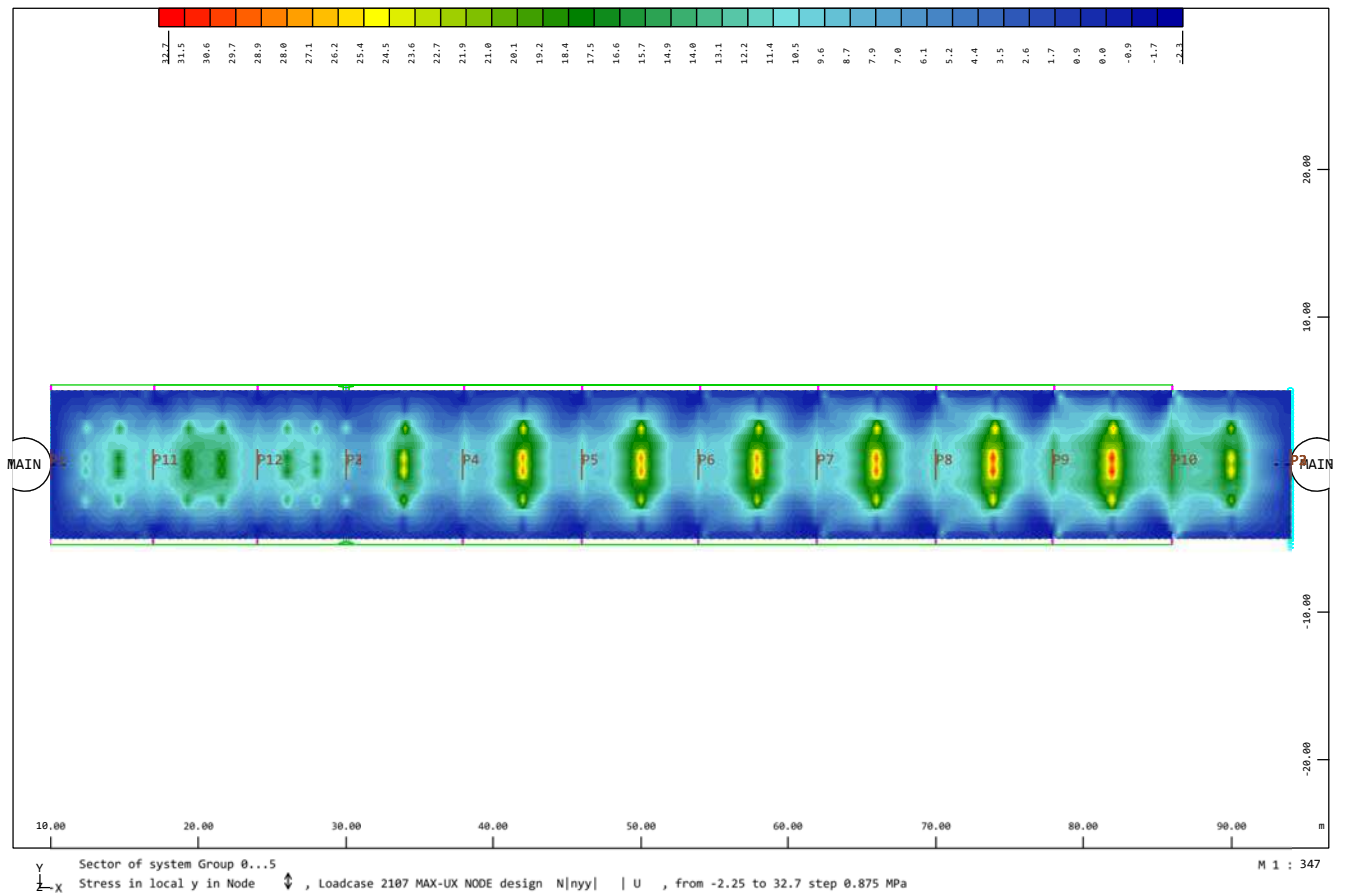
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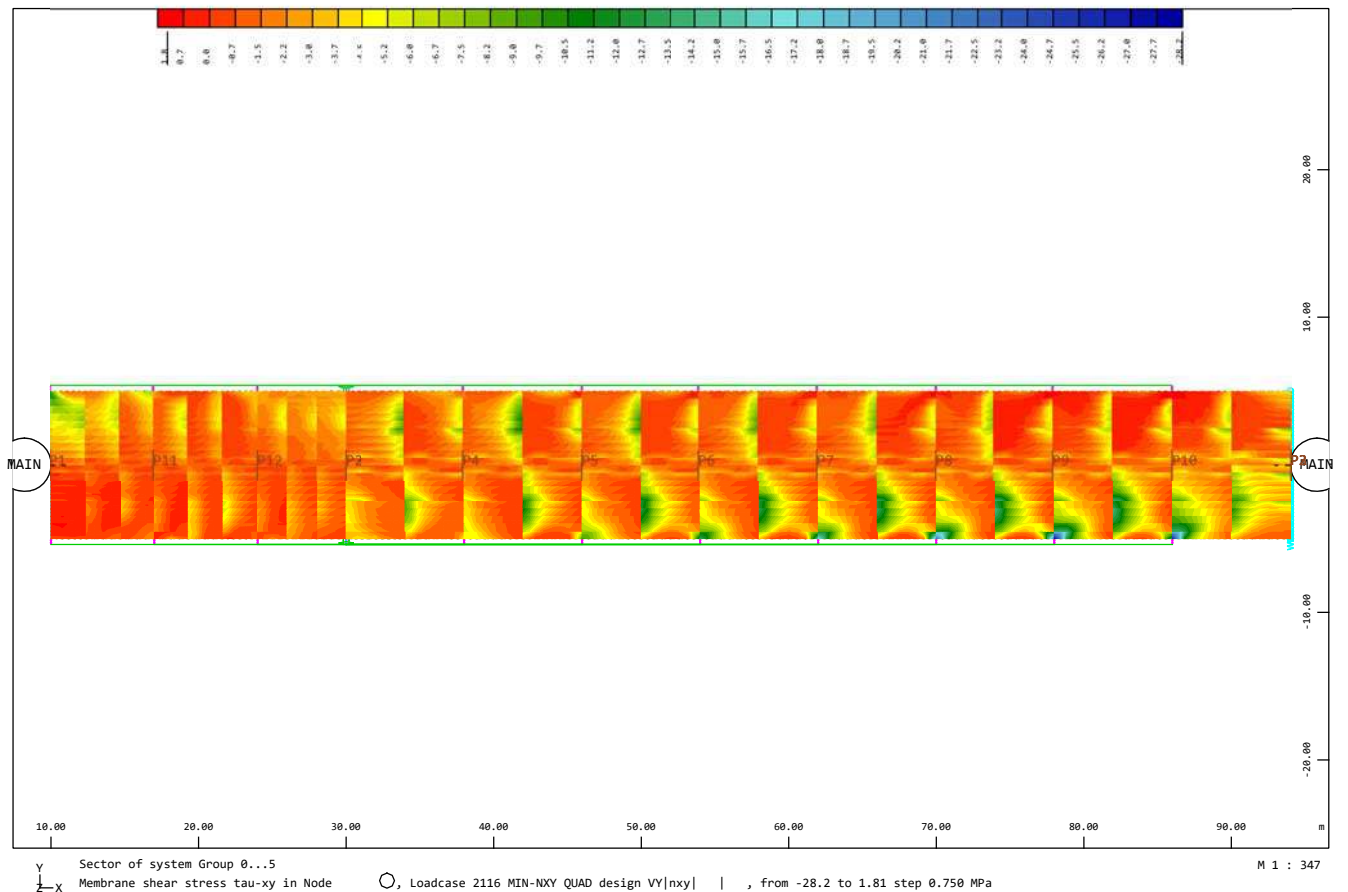
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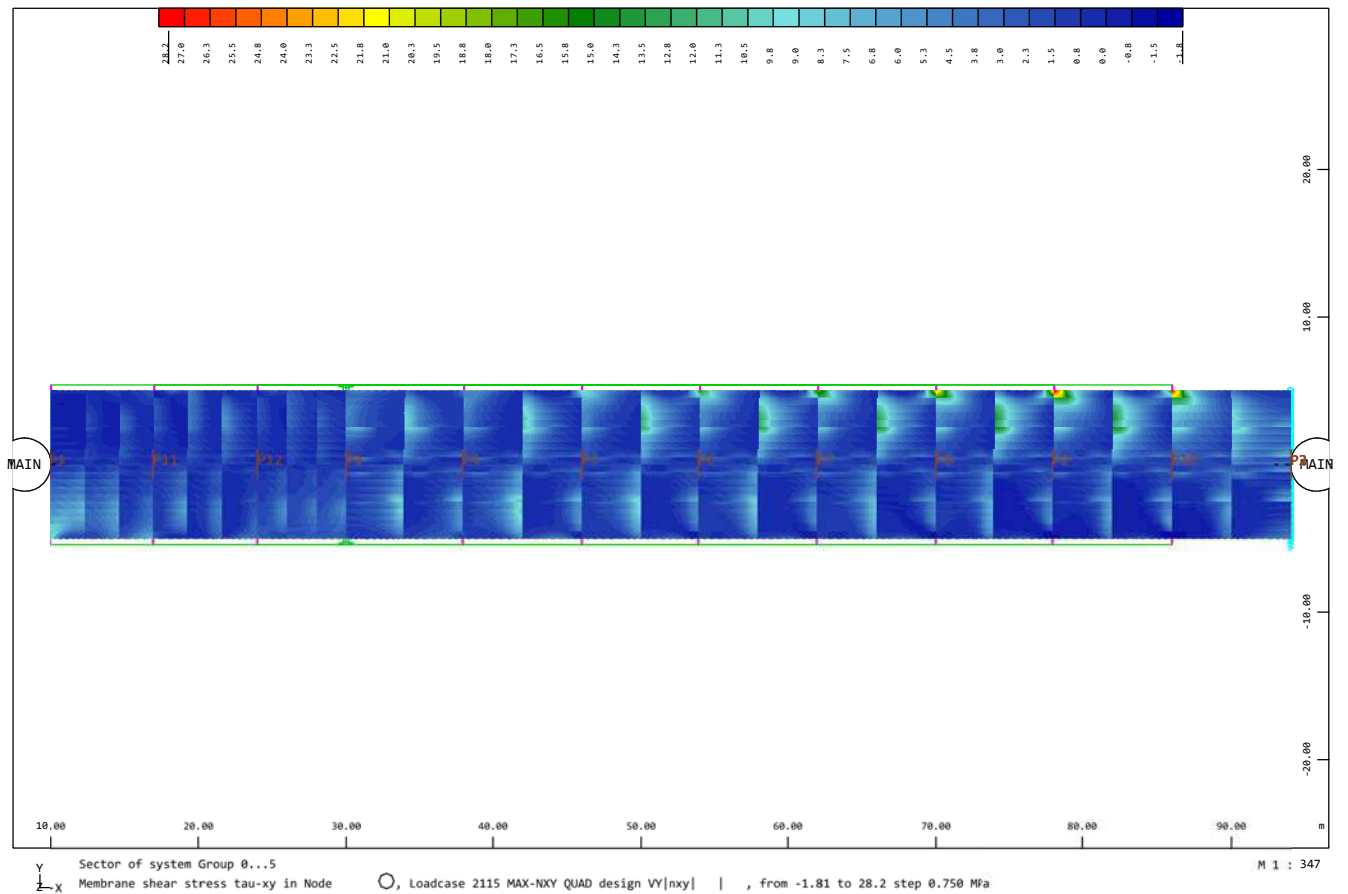
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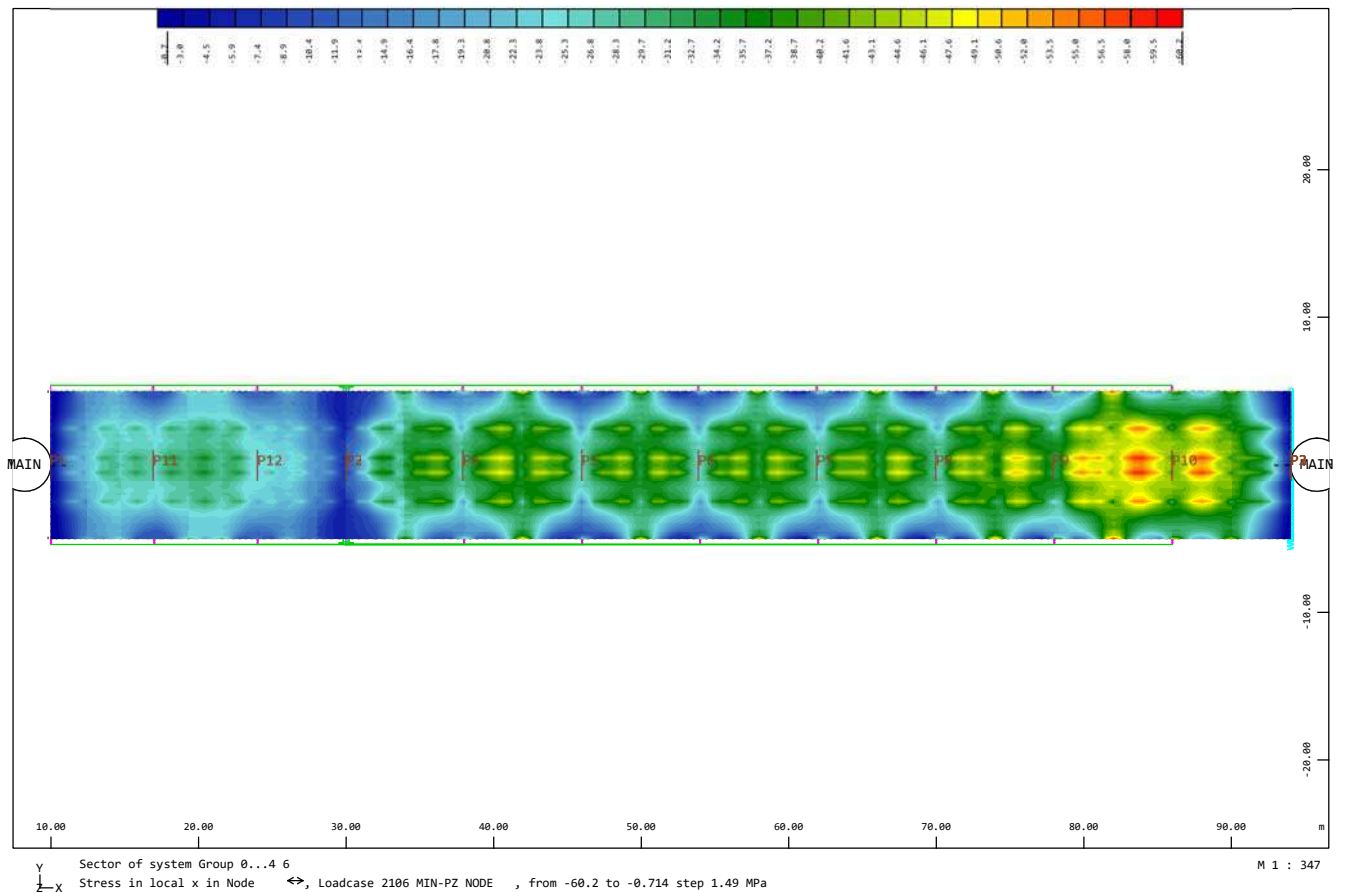
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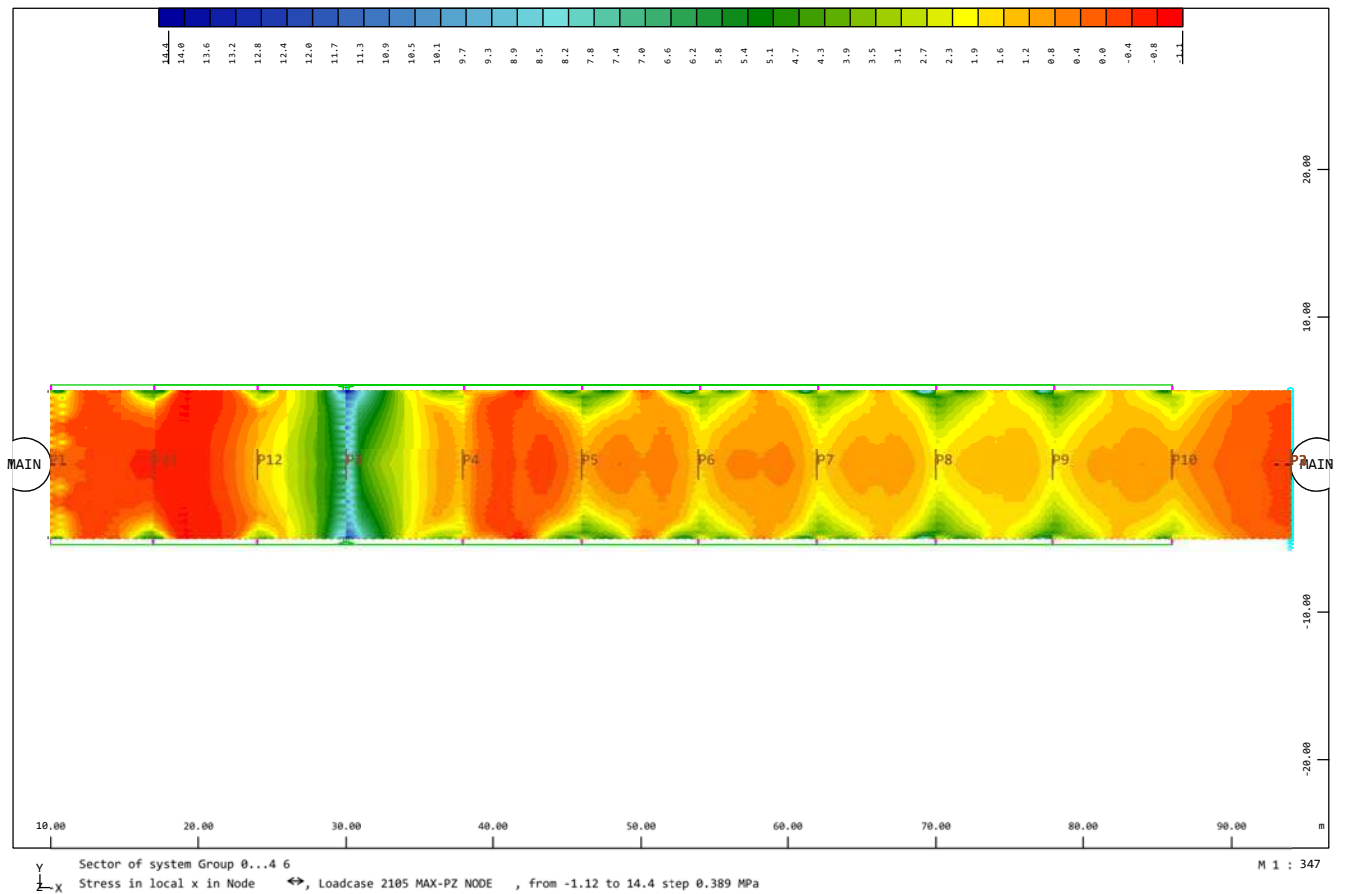
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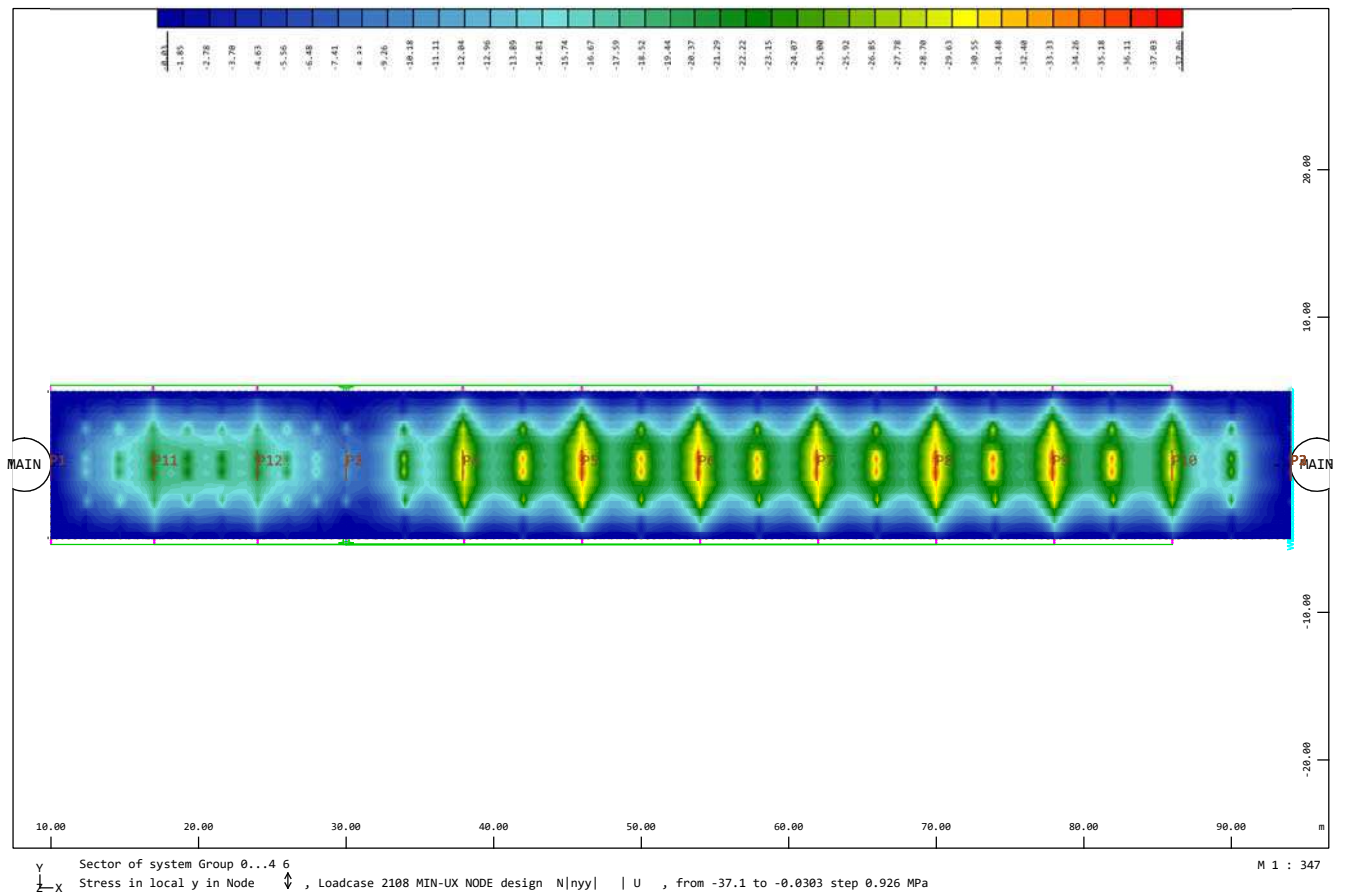
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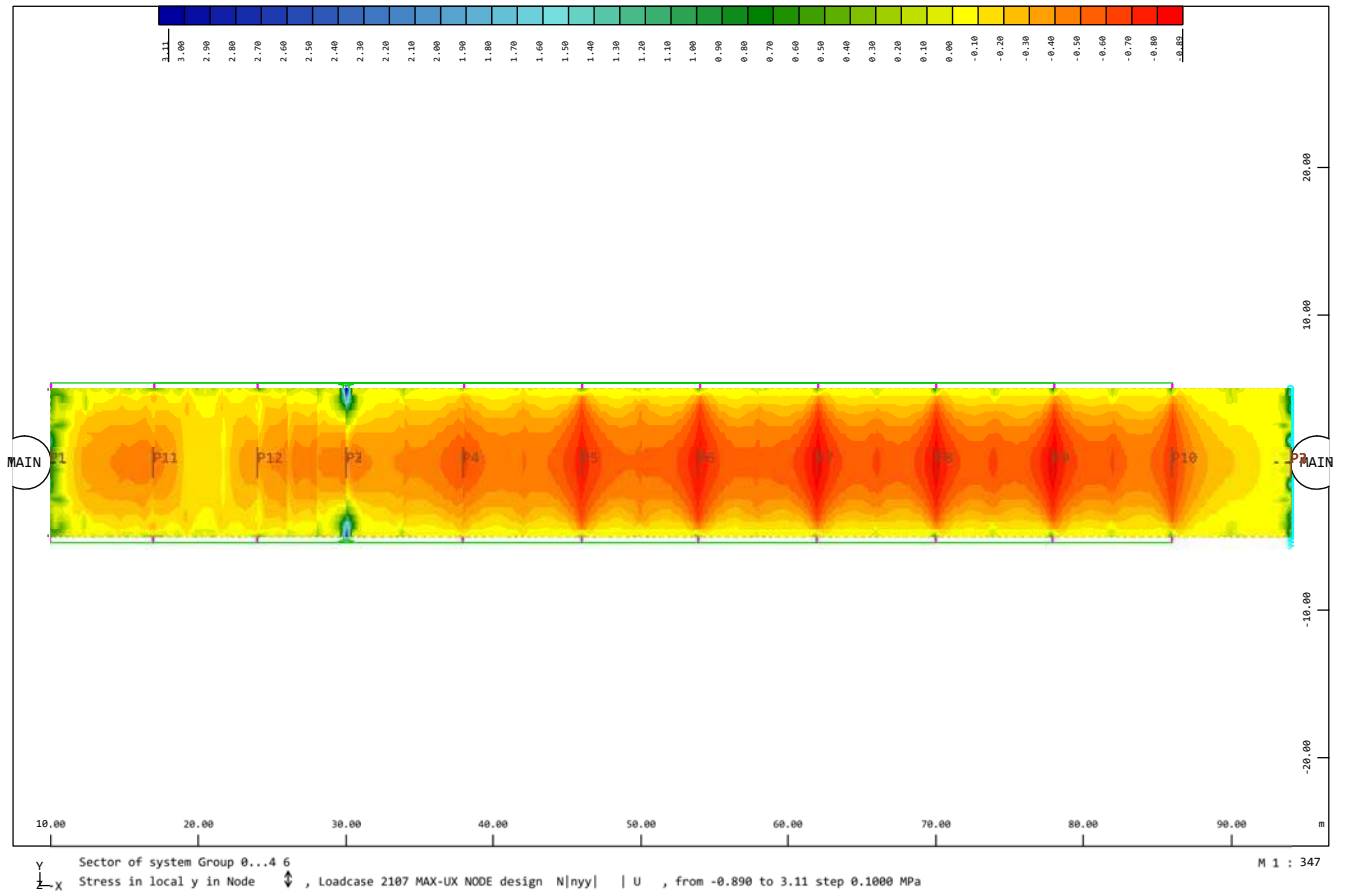
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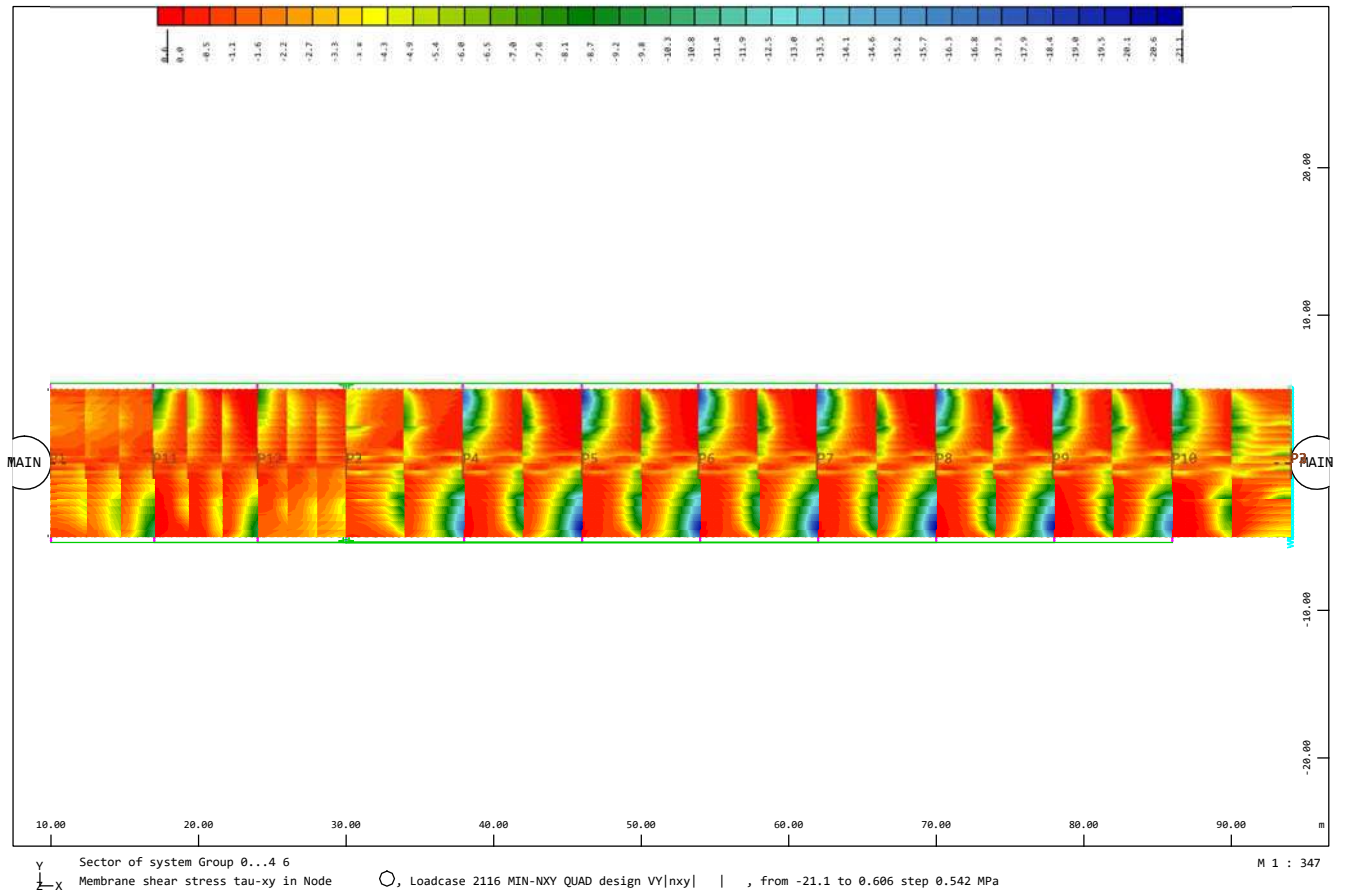
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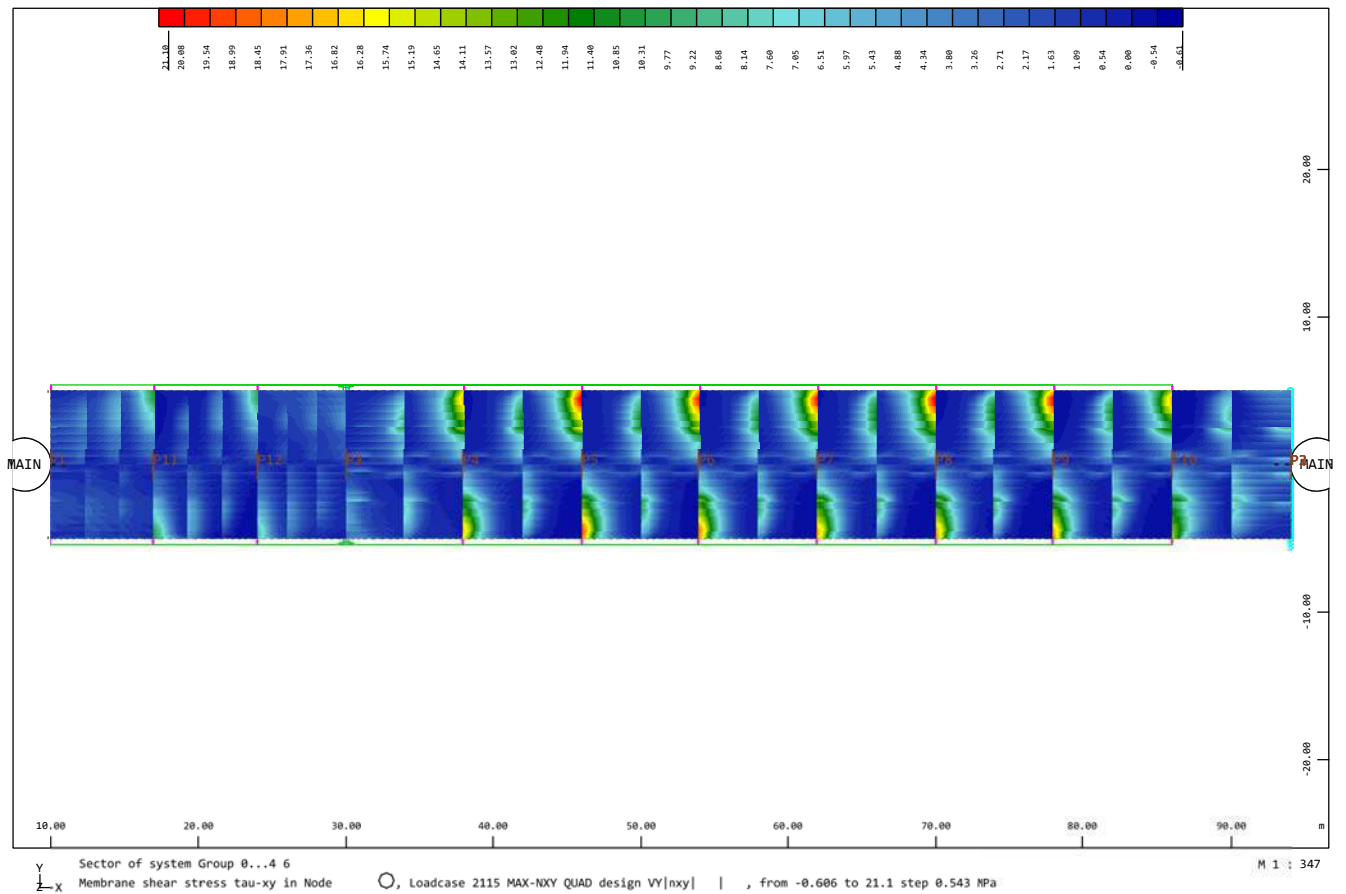


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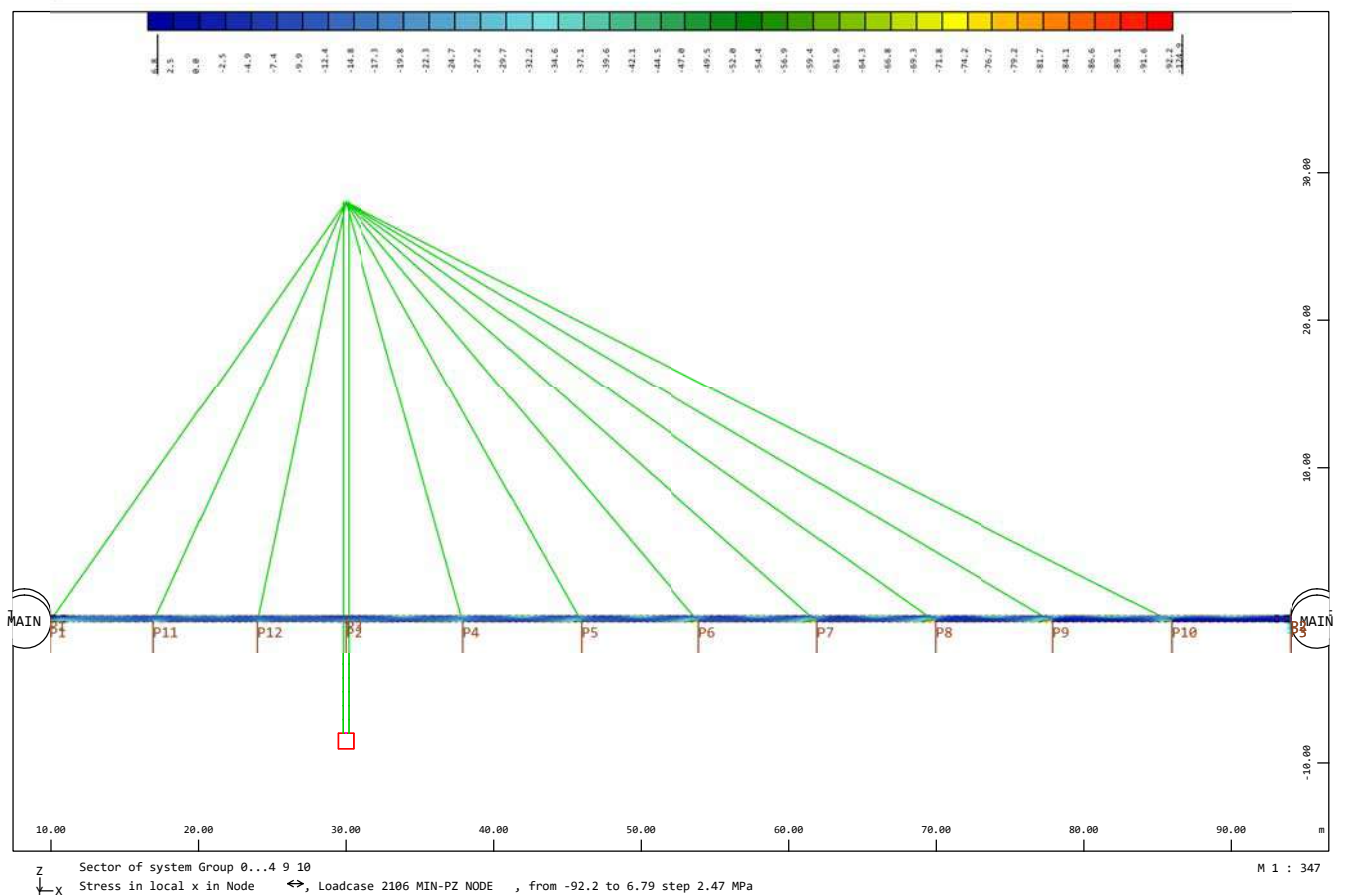
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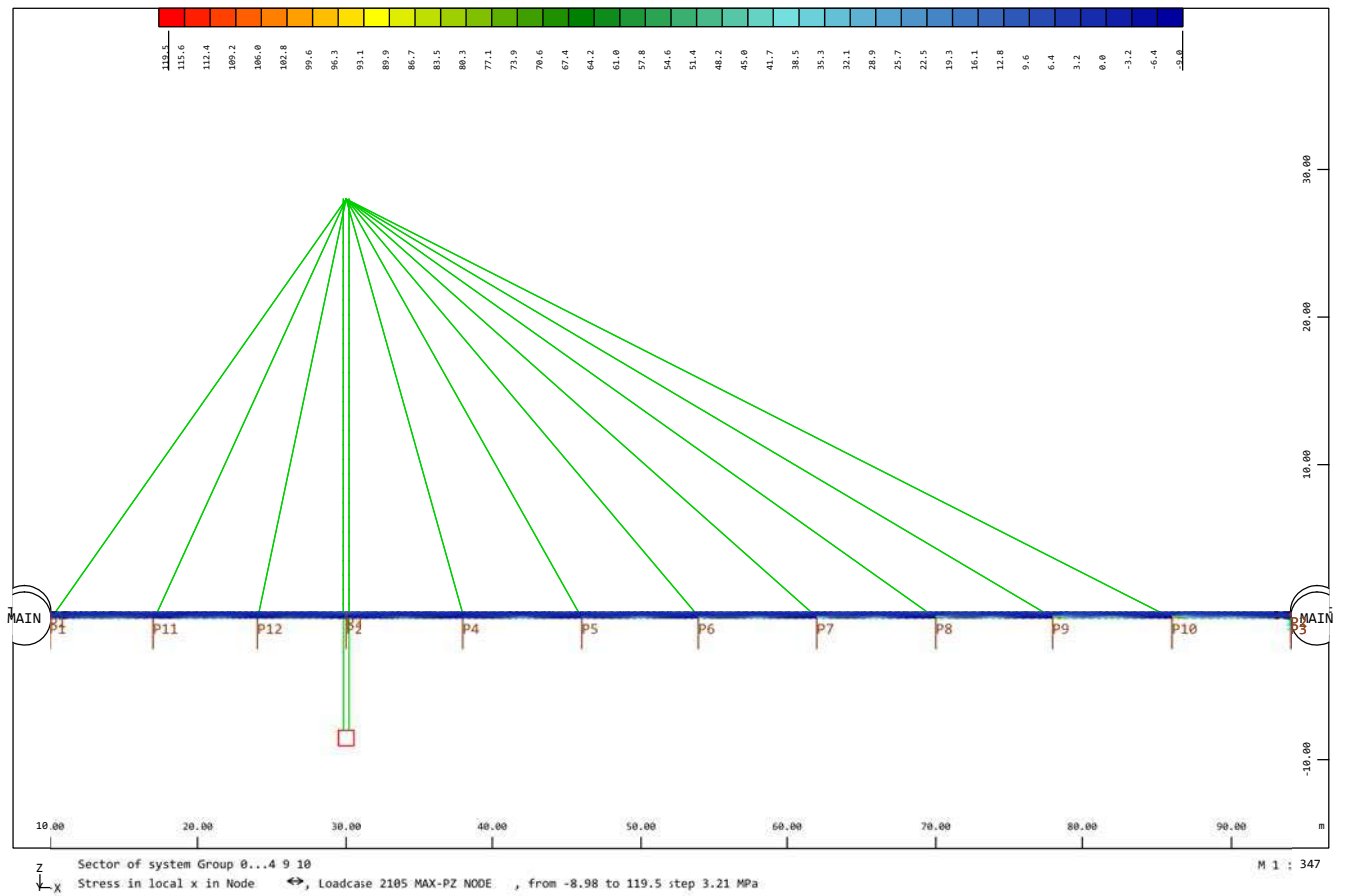


FRP_bridge_v1



FRP_bridge_v1

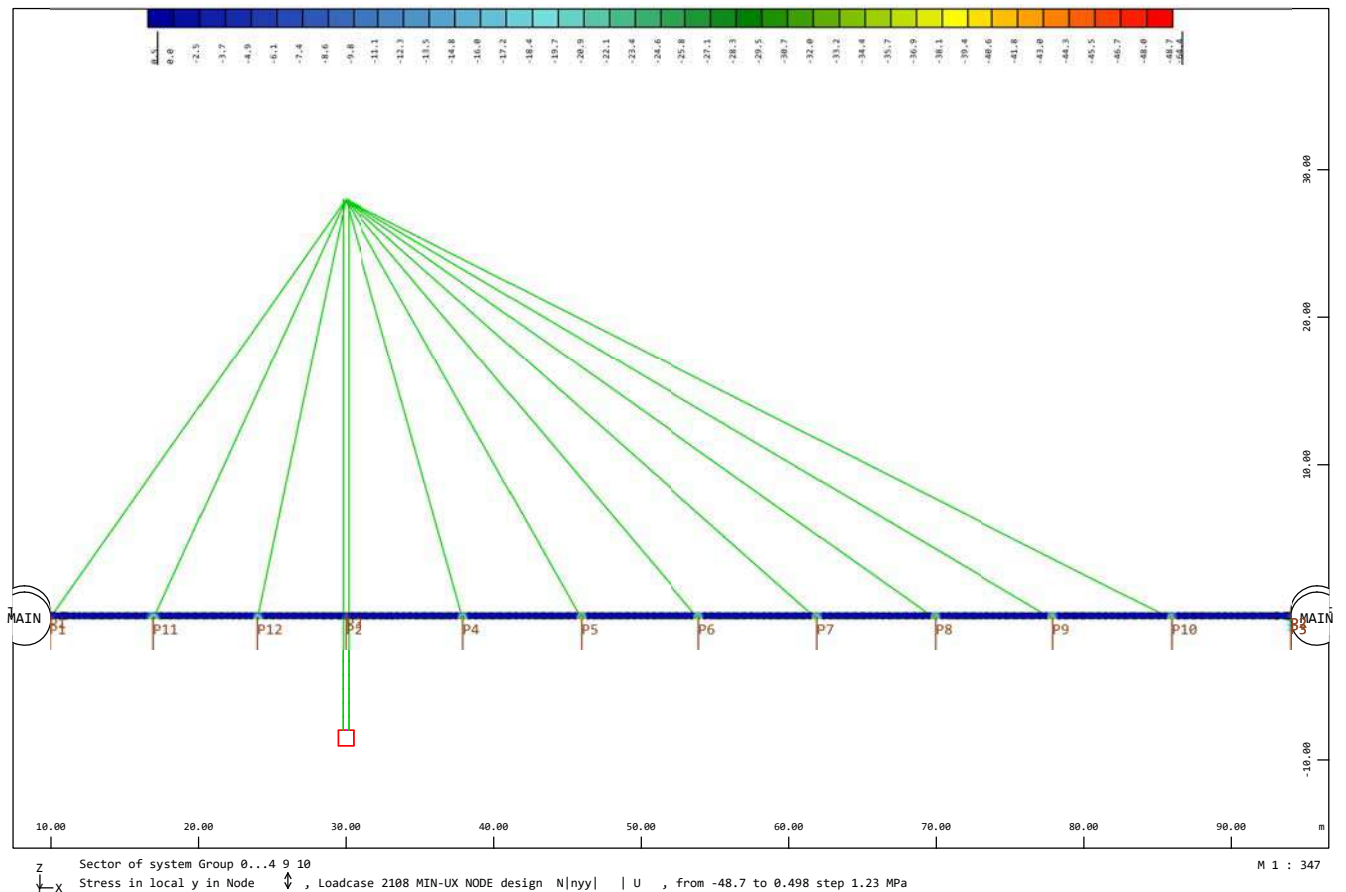
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reduced scale factor 0.000

FRP_bridge_v1

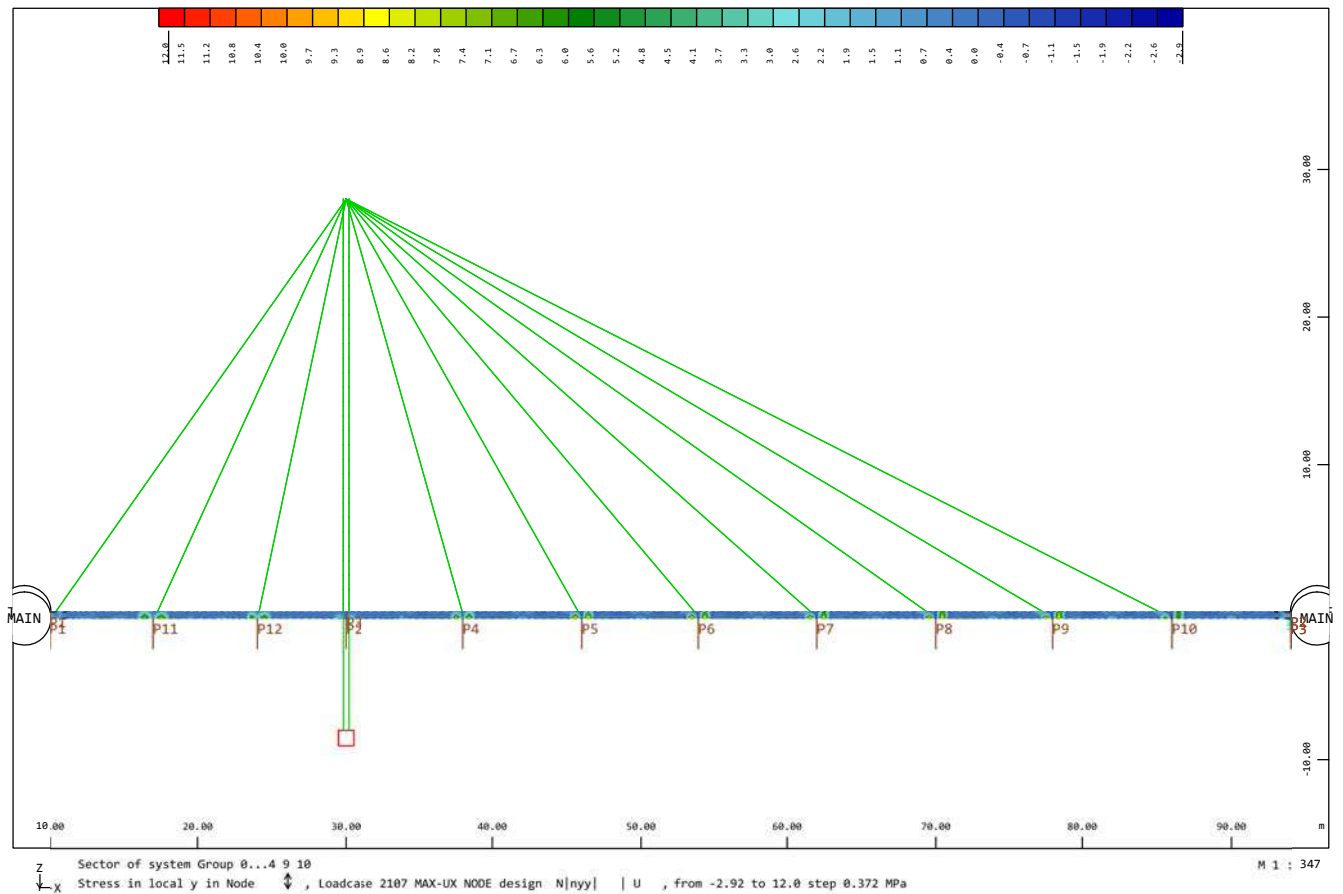
SOFiSTiK AG - www.sofistik.de



reduced scale factor 0.000

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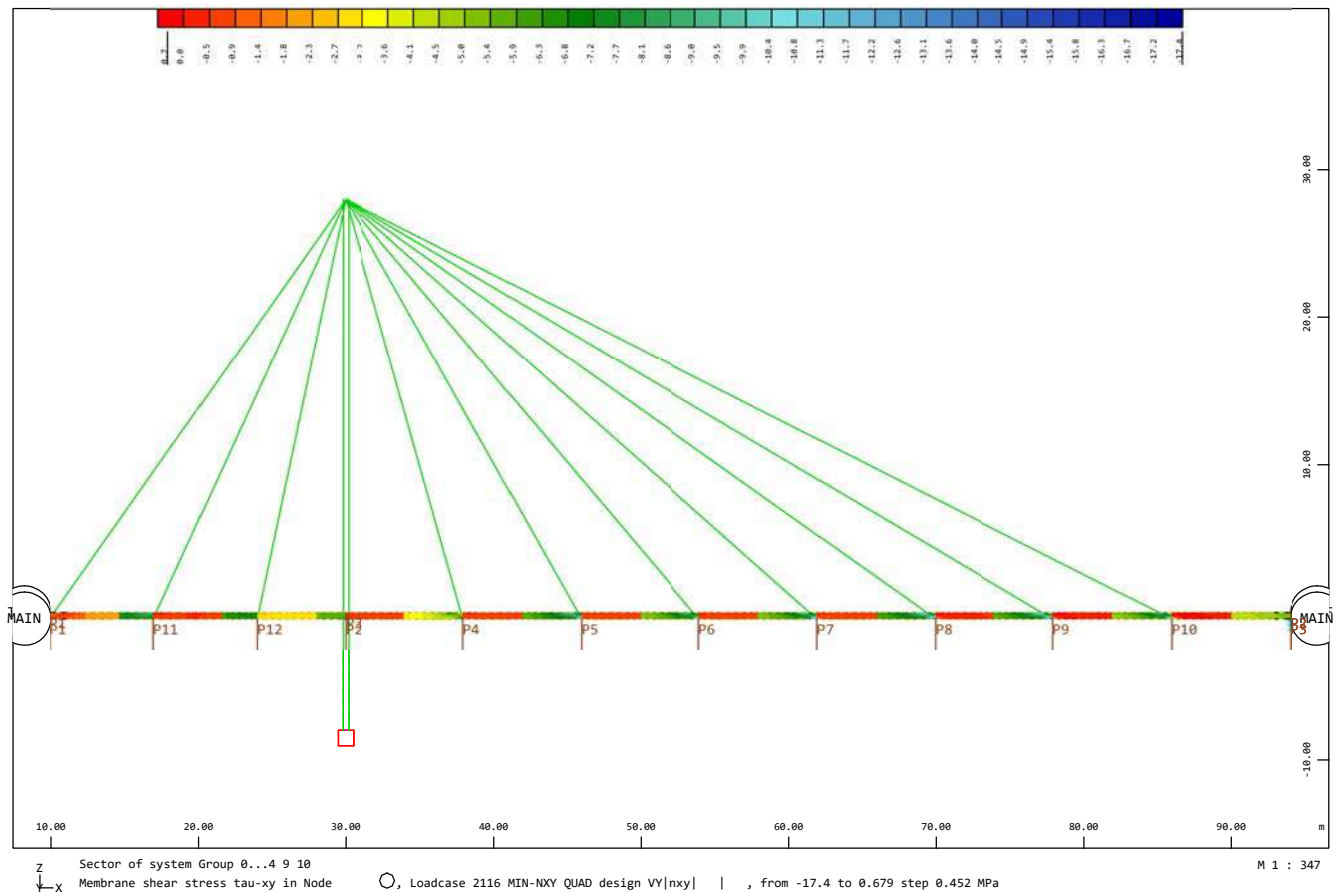
SOFiSTiK AG - www.sofistik.de



reduced scale factor 0.000

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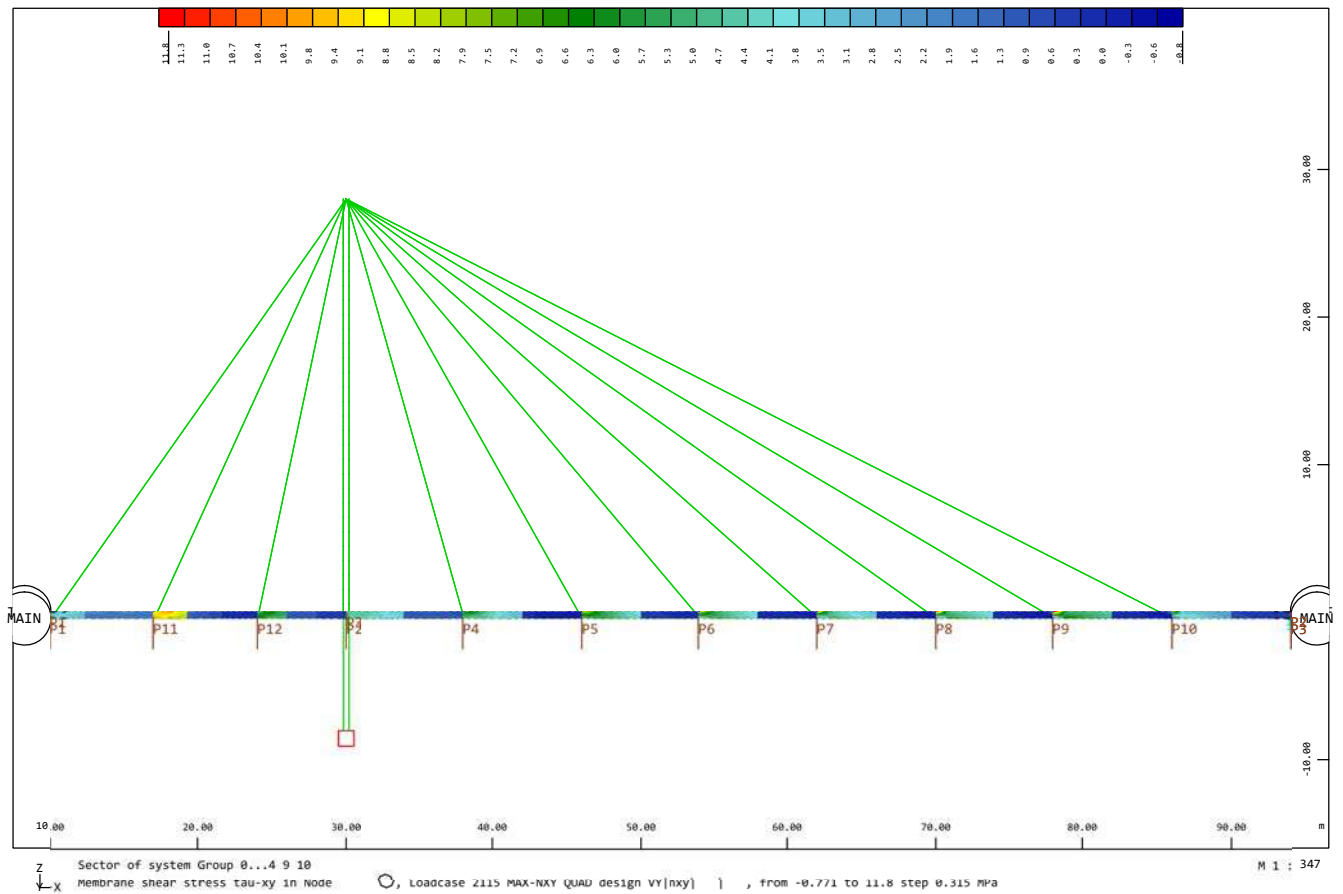
SOFiSTiK AG - www.sofistik.de



reduced scale factor 0.000

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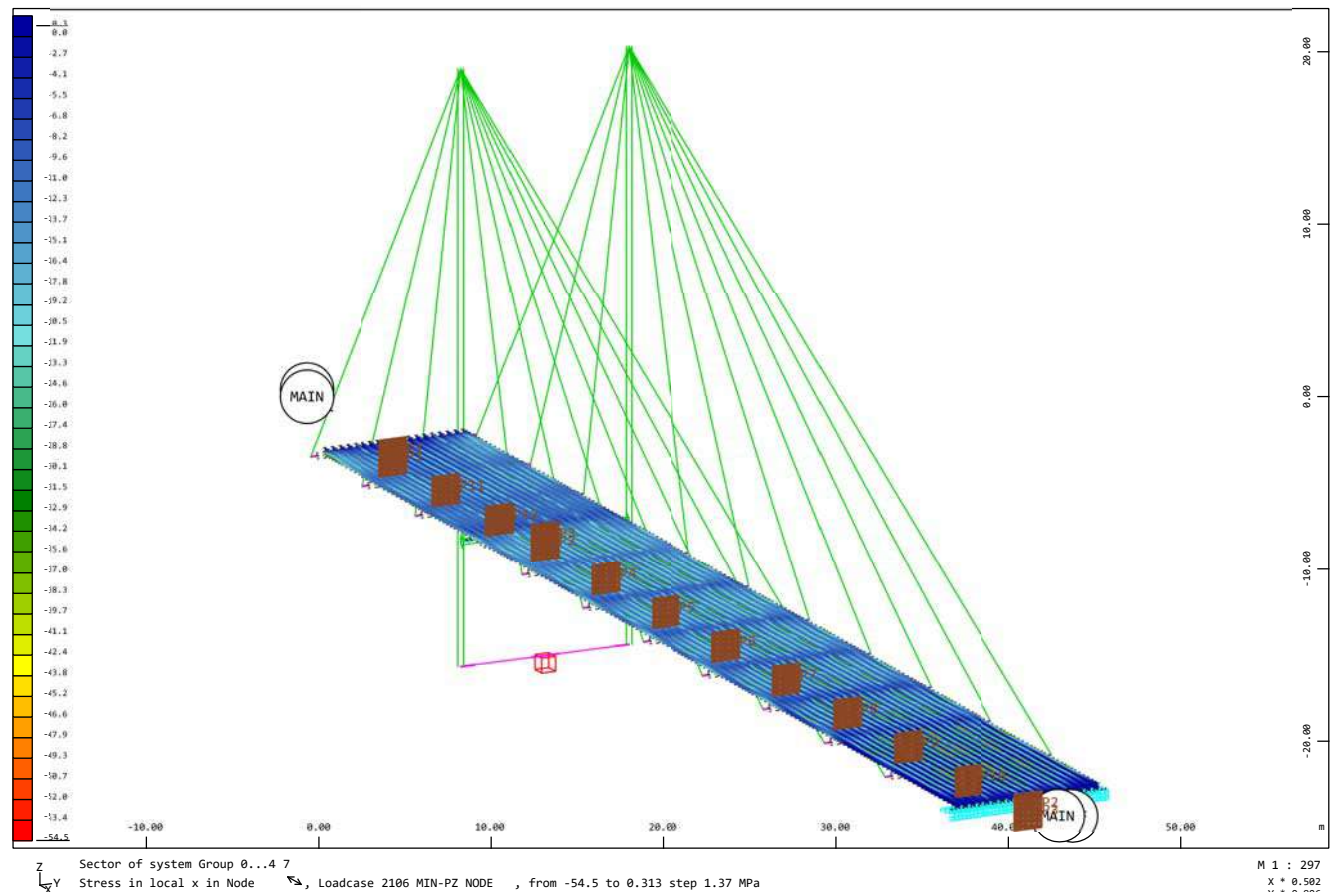
SOFiSTiK AG - www.sofistik.de



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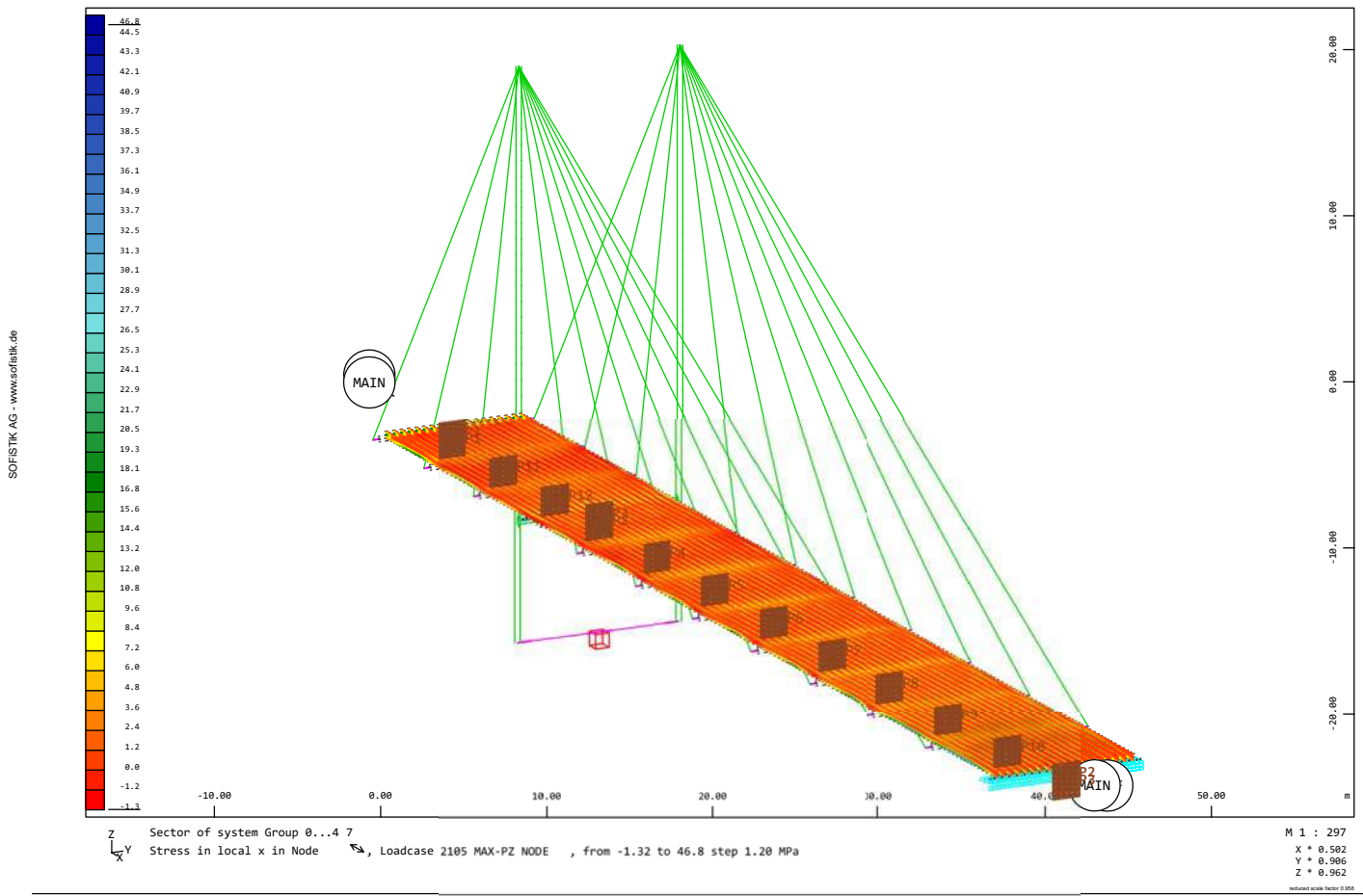
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SOFiSTiK AG - www.sofistik.de

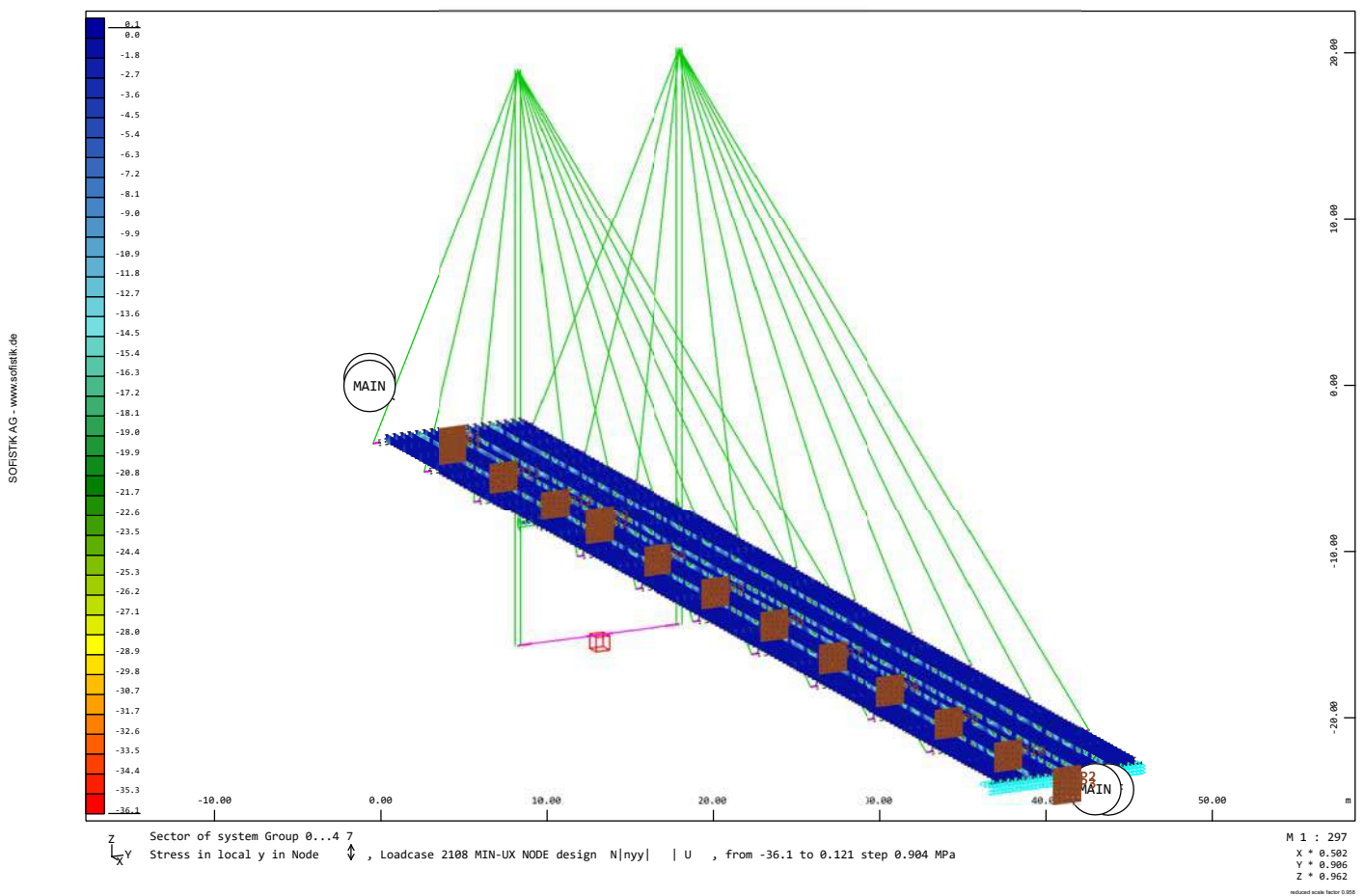


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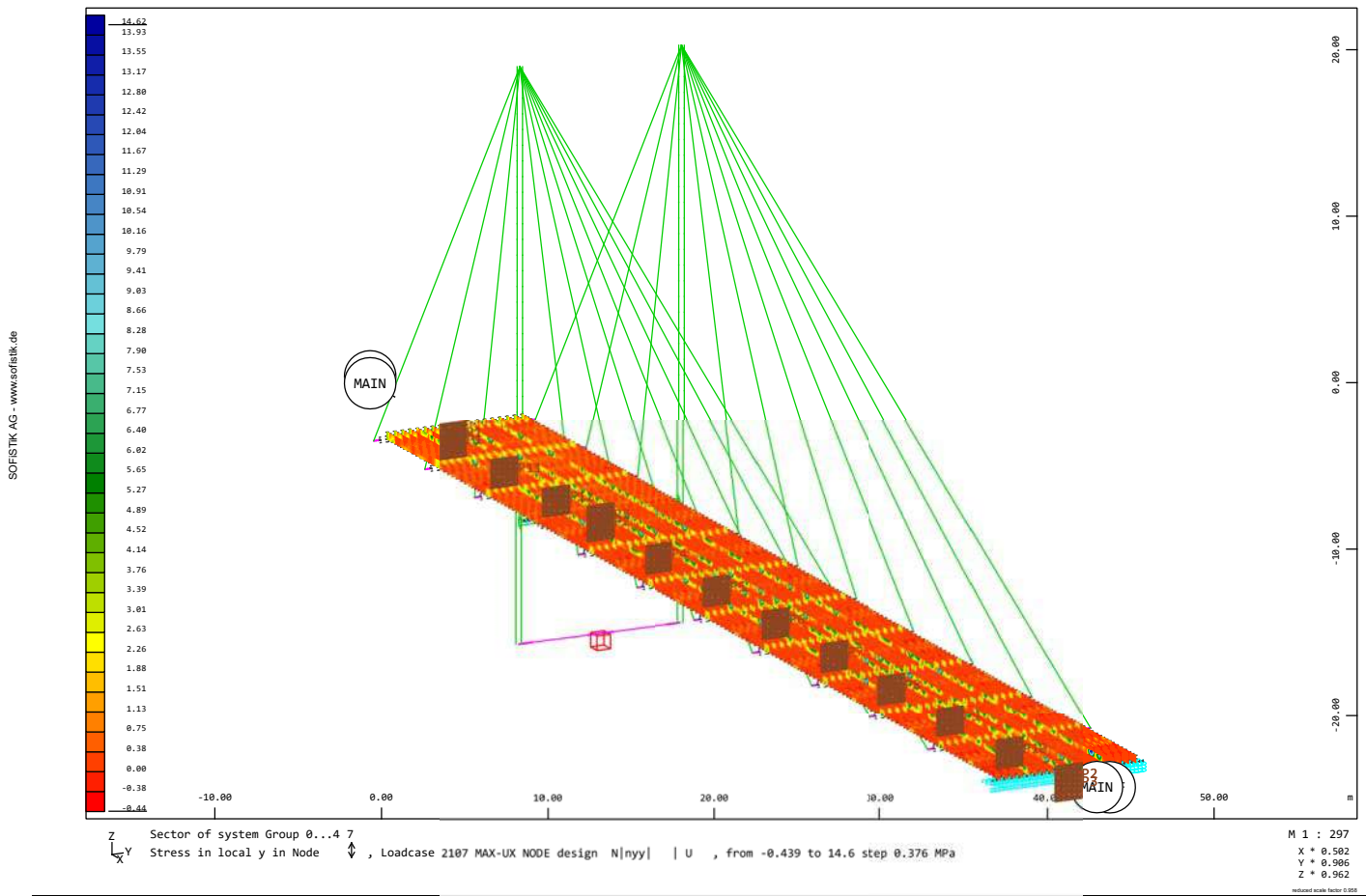
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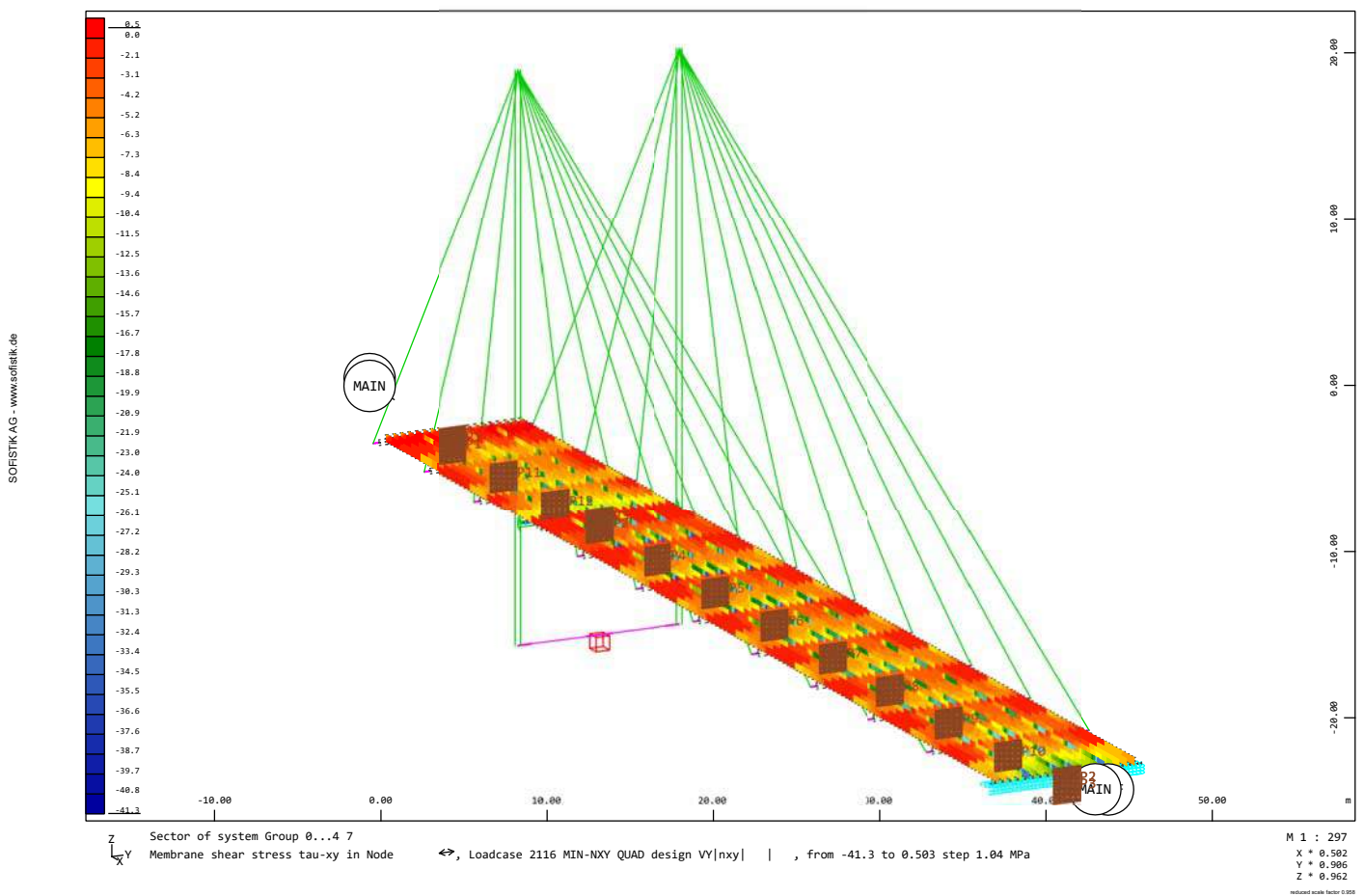
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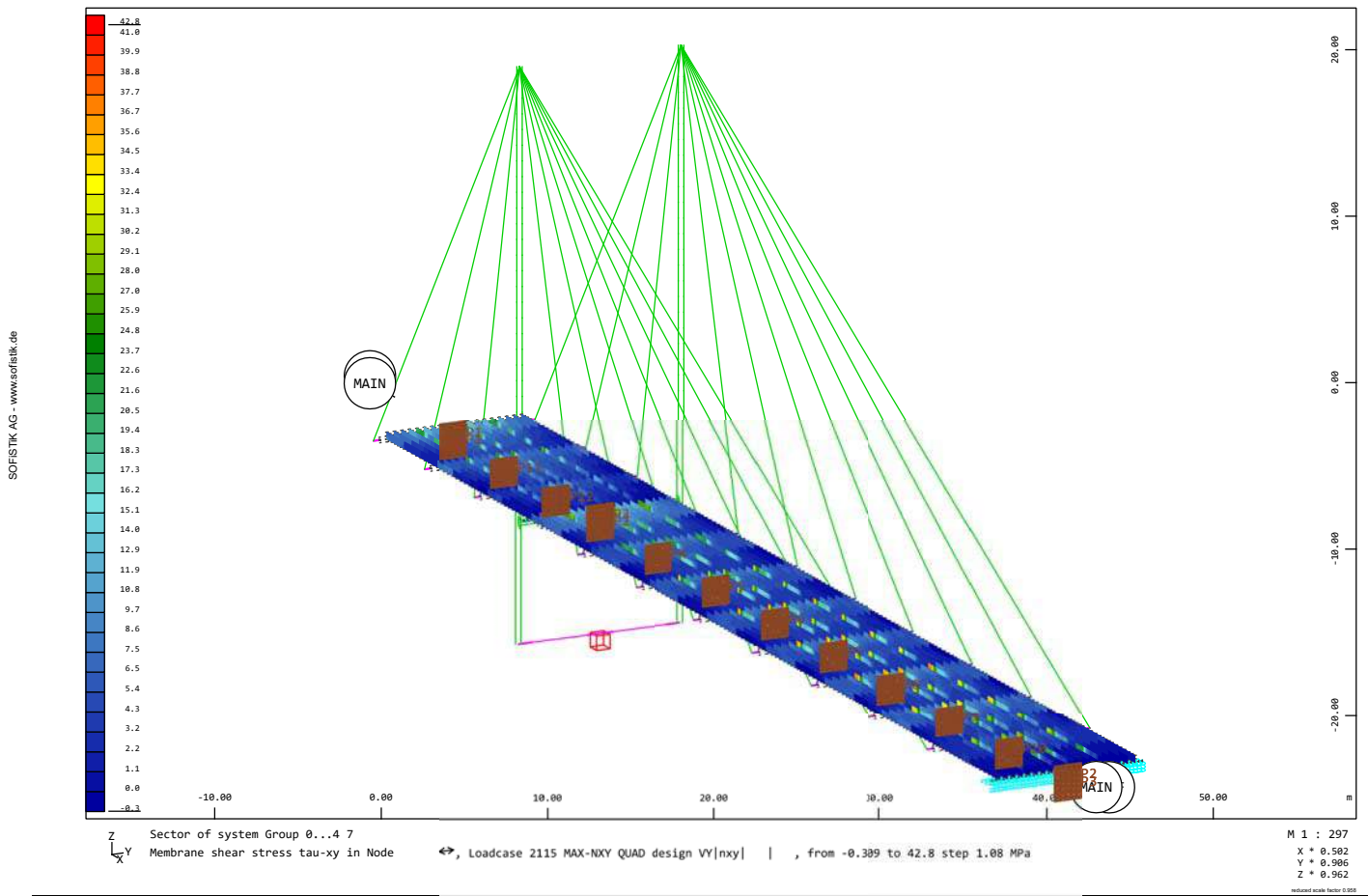
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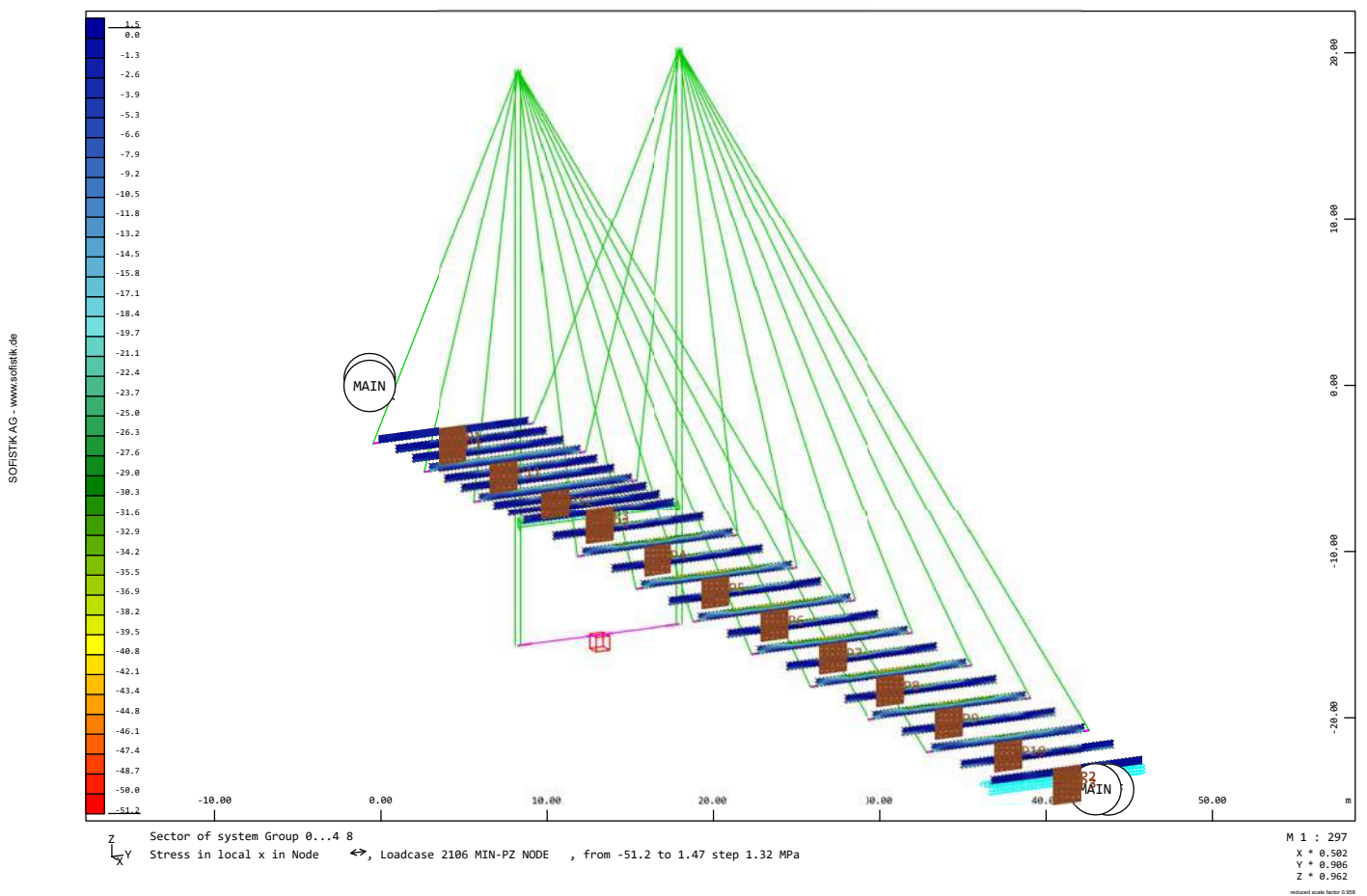
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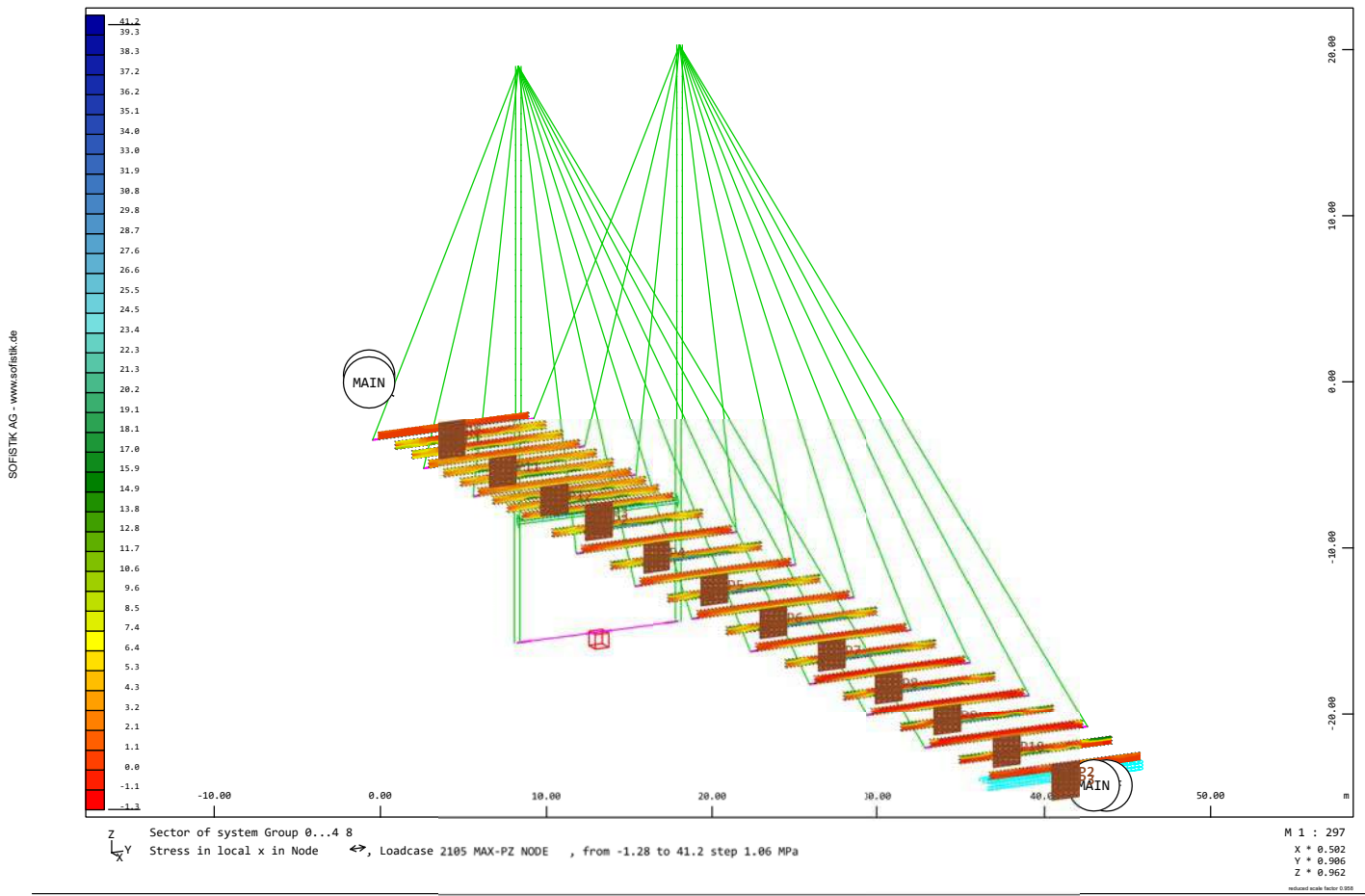
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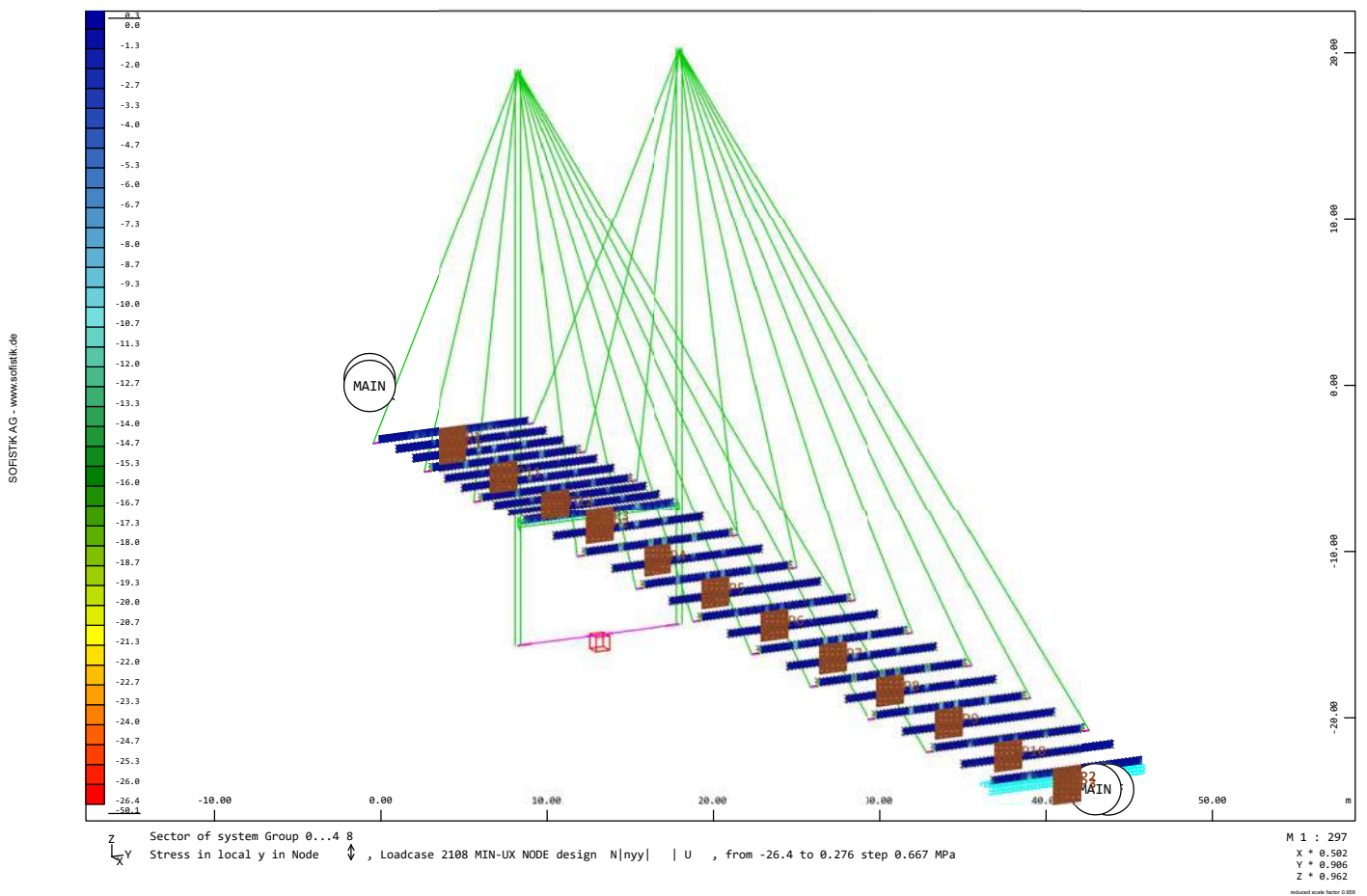
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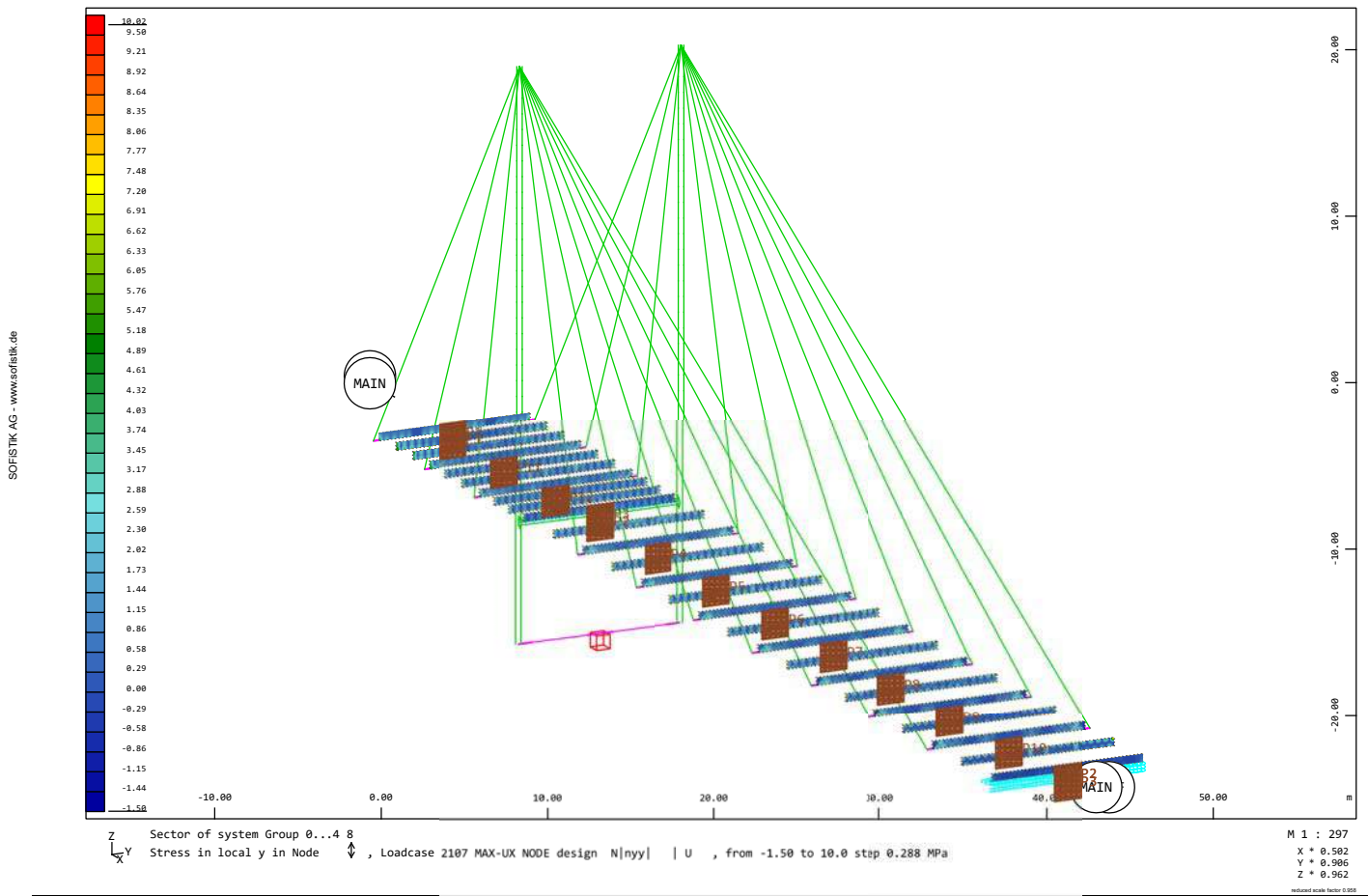
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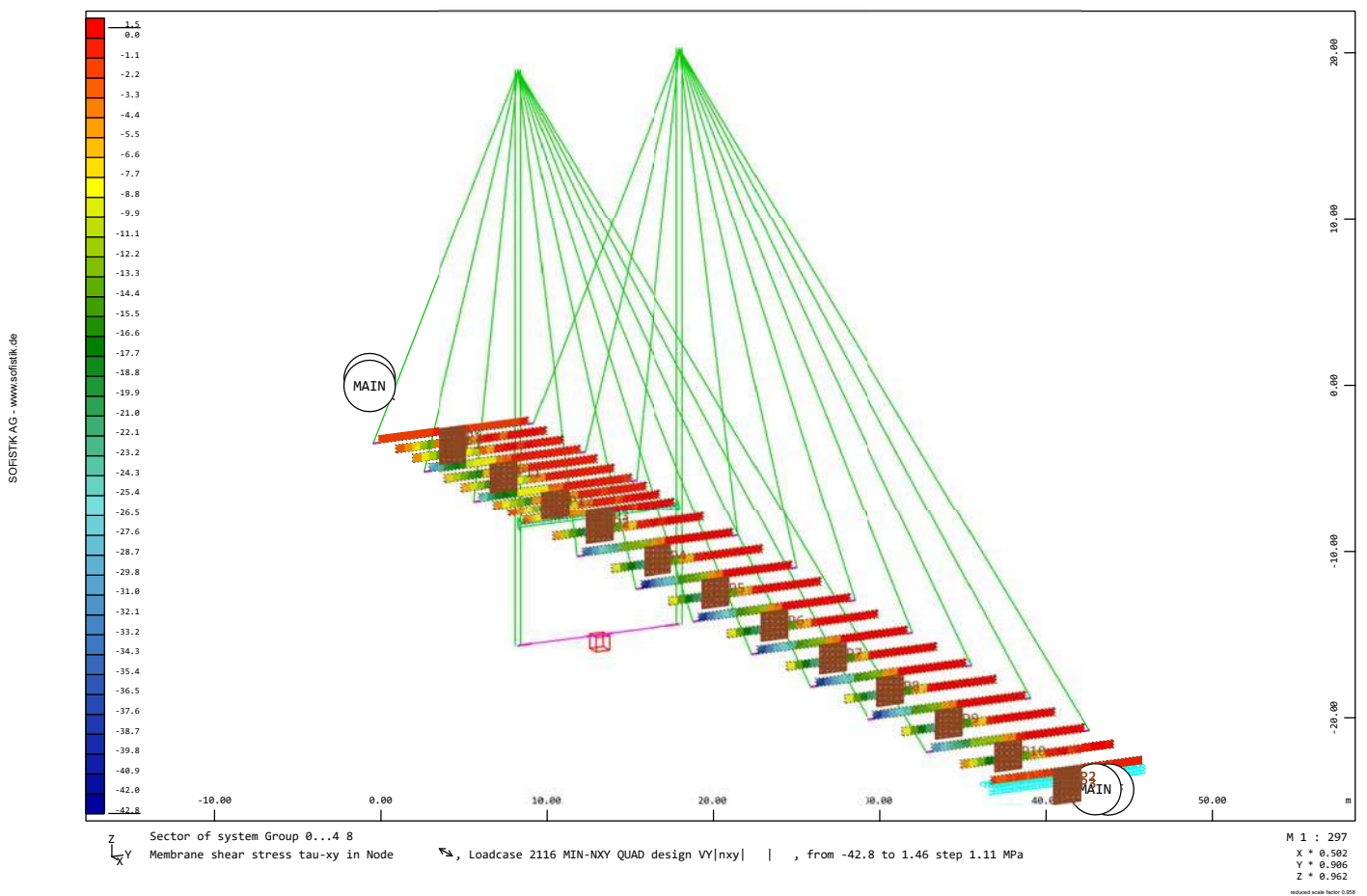
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FRP_bridge_v1

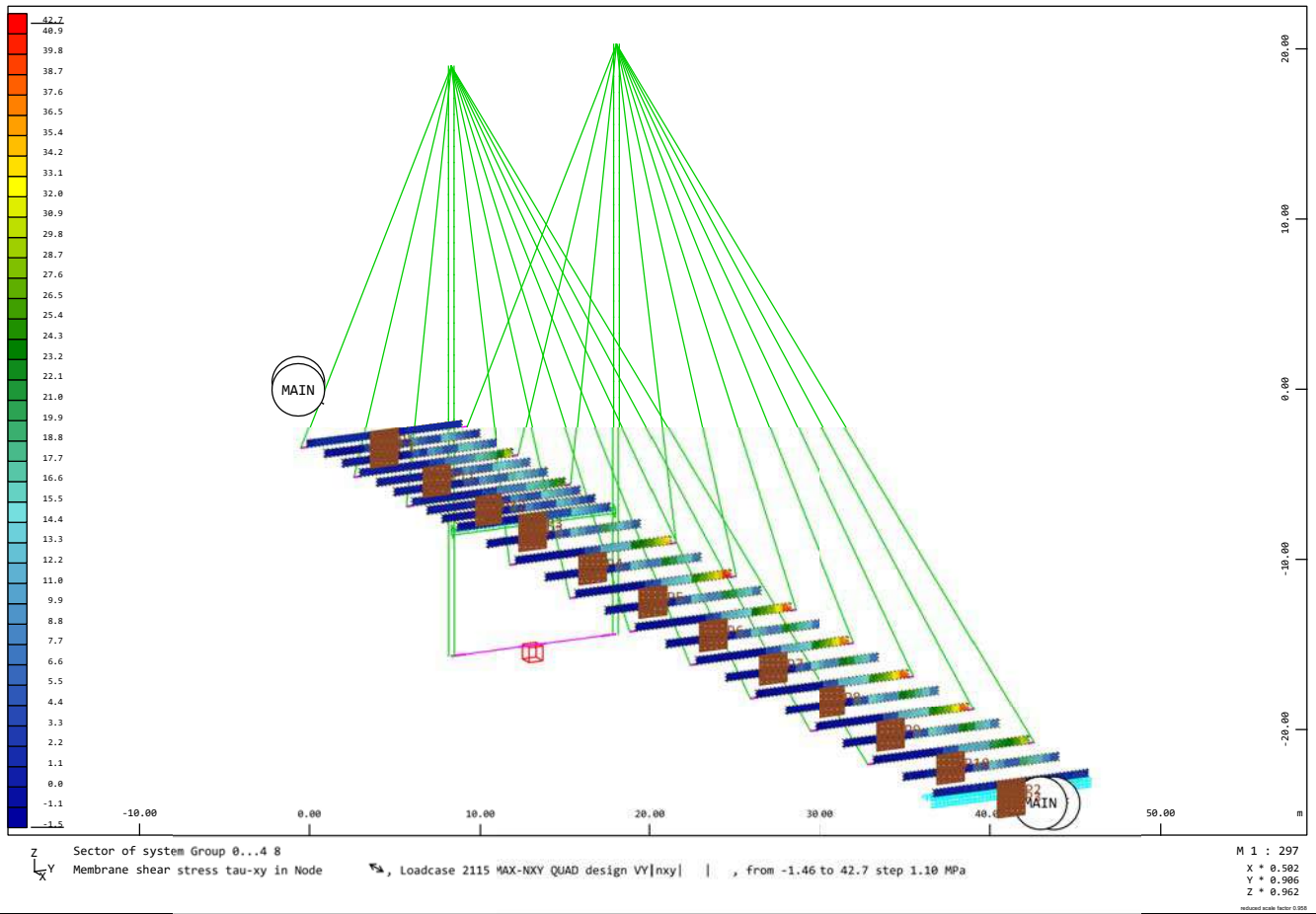


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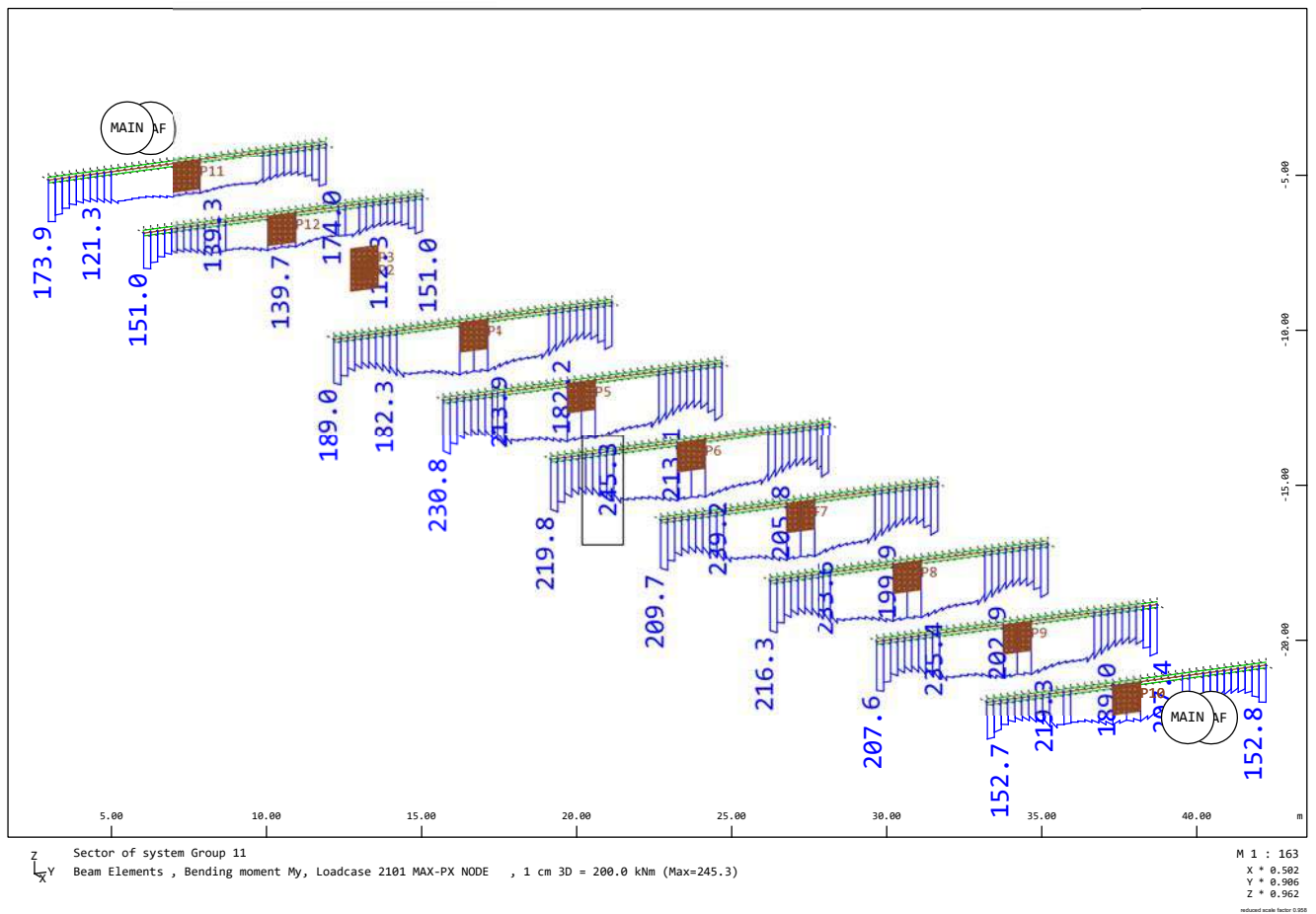
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SOFiSTiK AG - www.sofistik.de

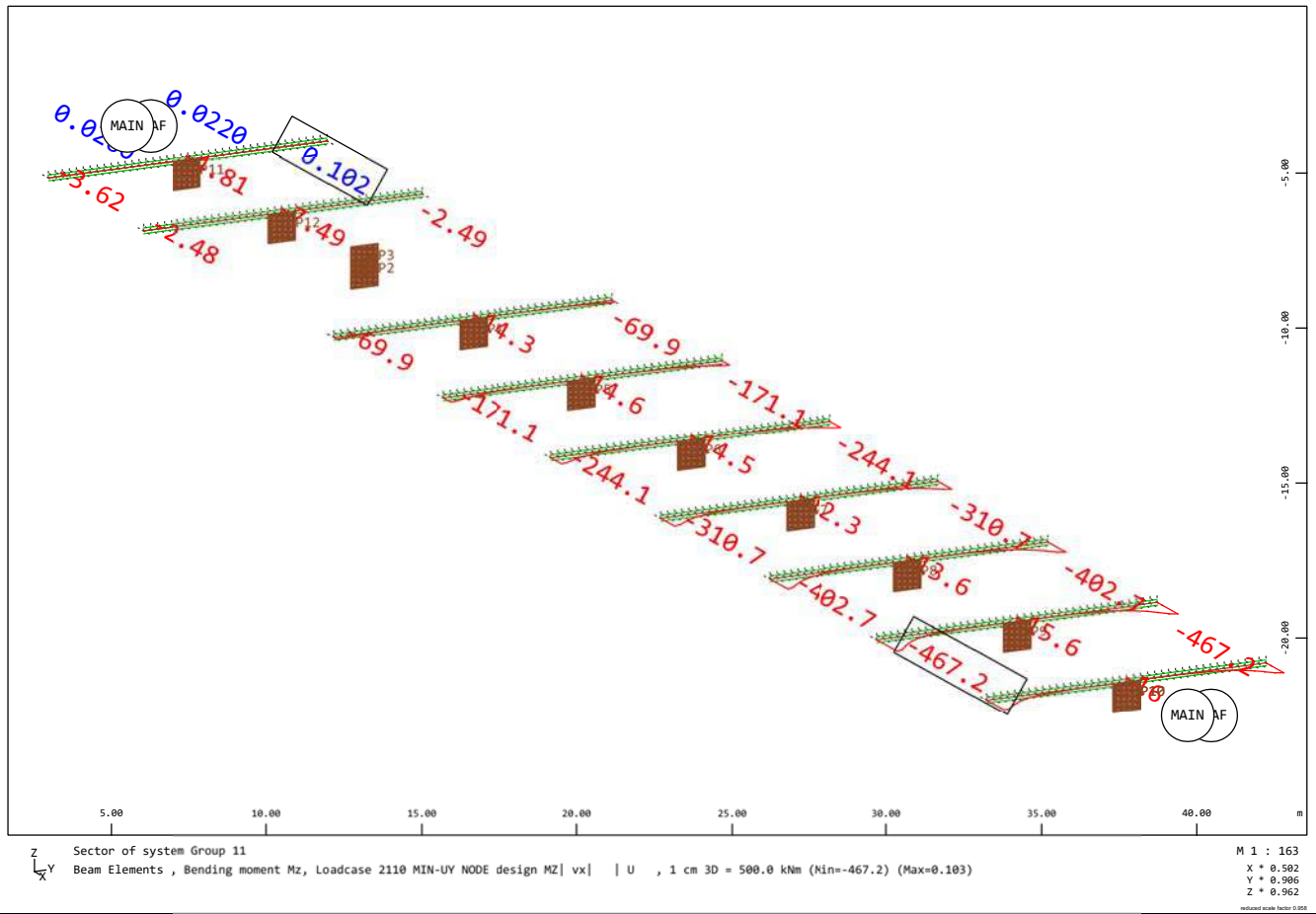


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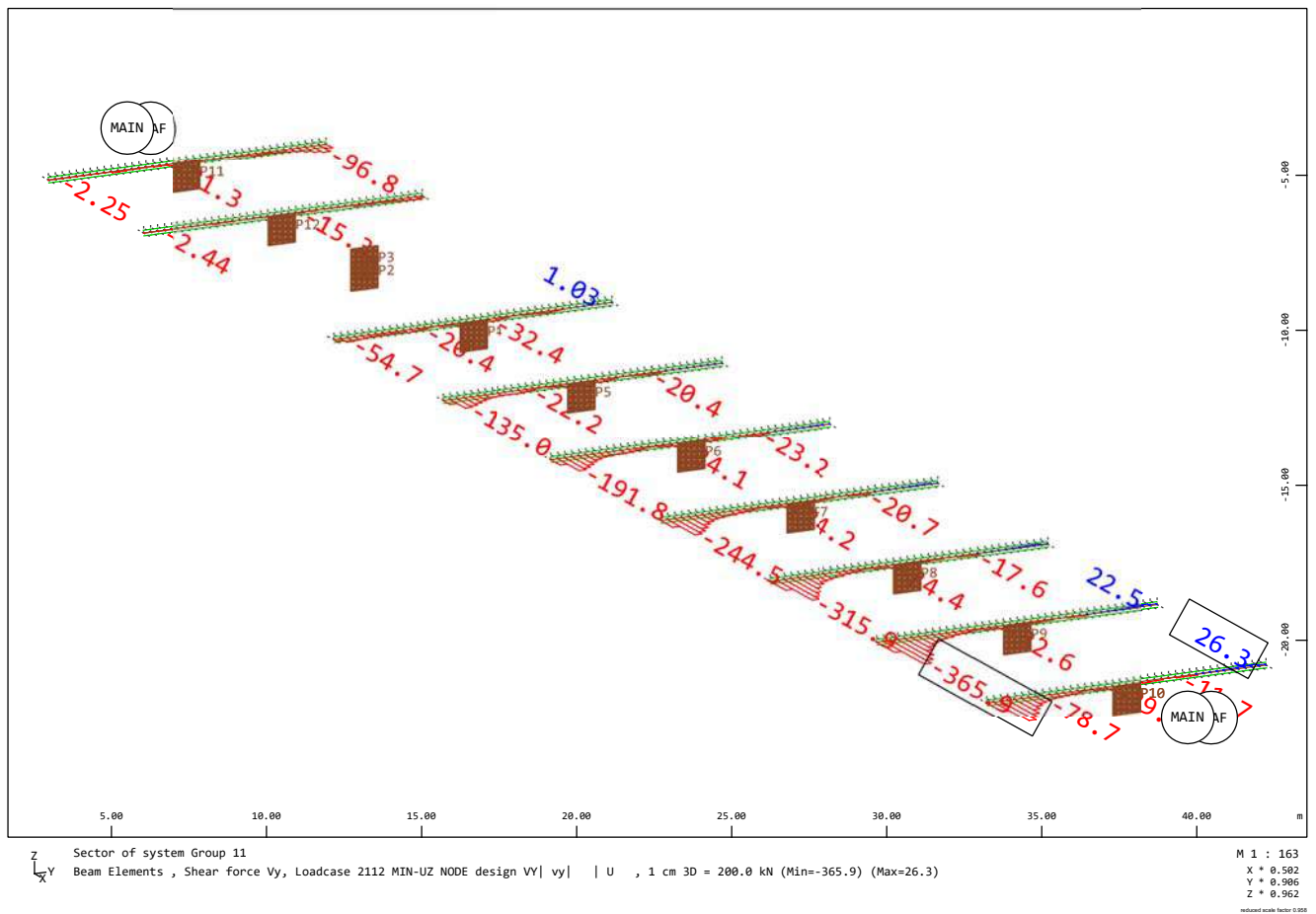
SOFiSTiK AG - www.sofistik.de



FRP_bridge_v1

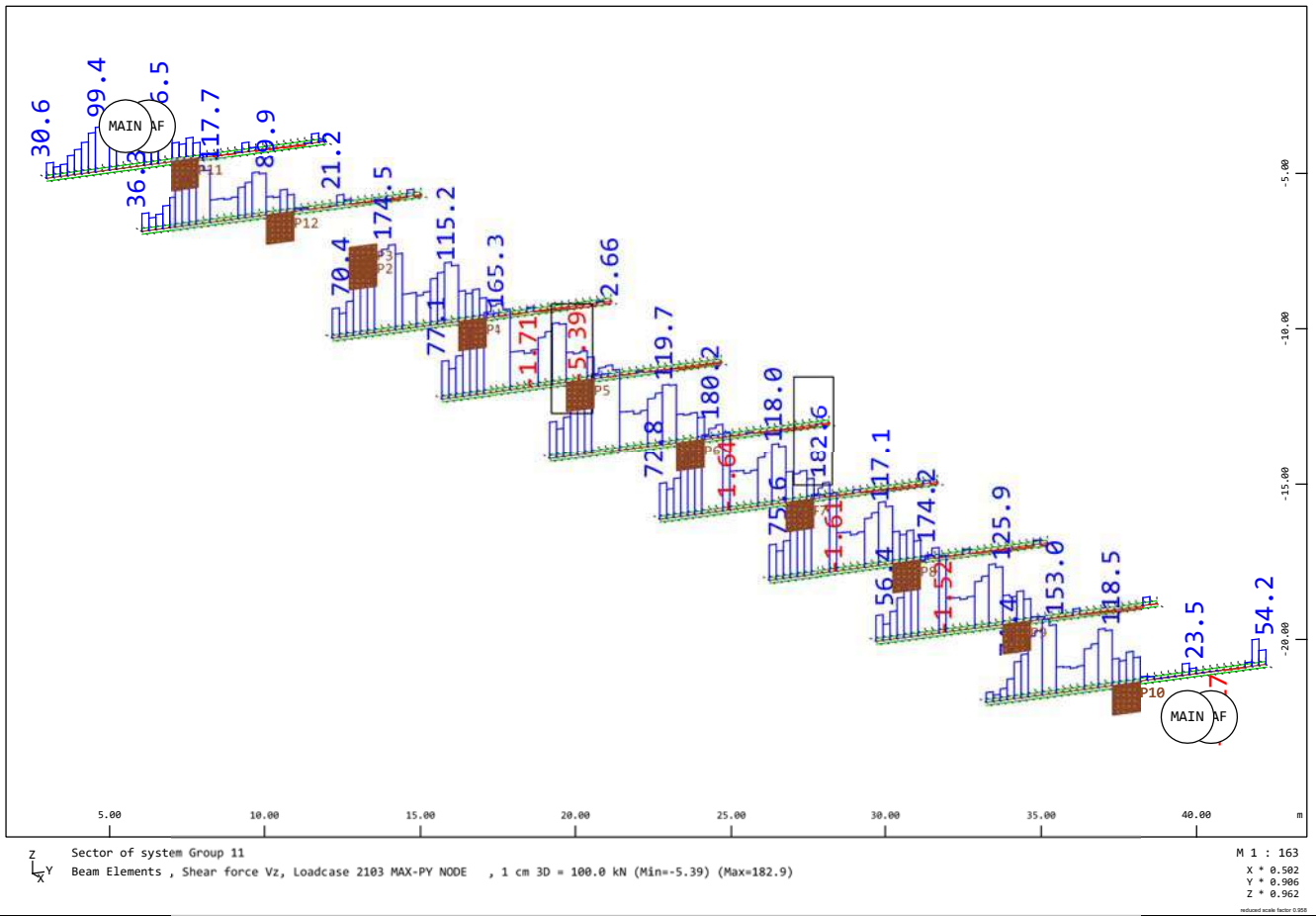


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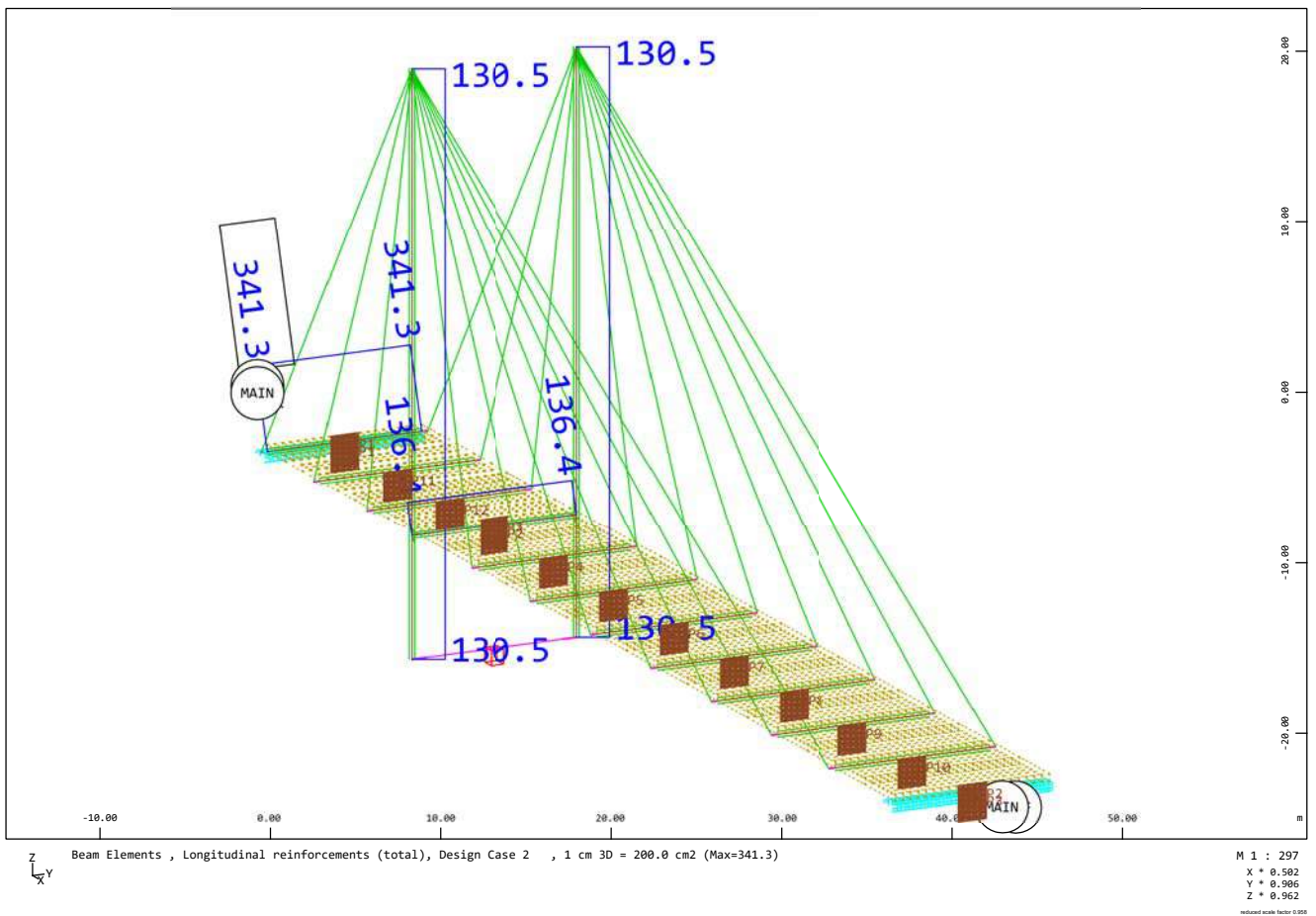
FRP_bridge_v1

SOFiSTiK AG - www.sofistik.de



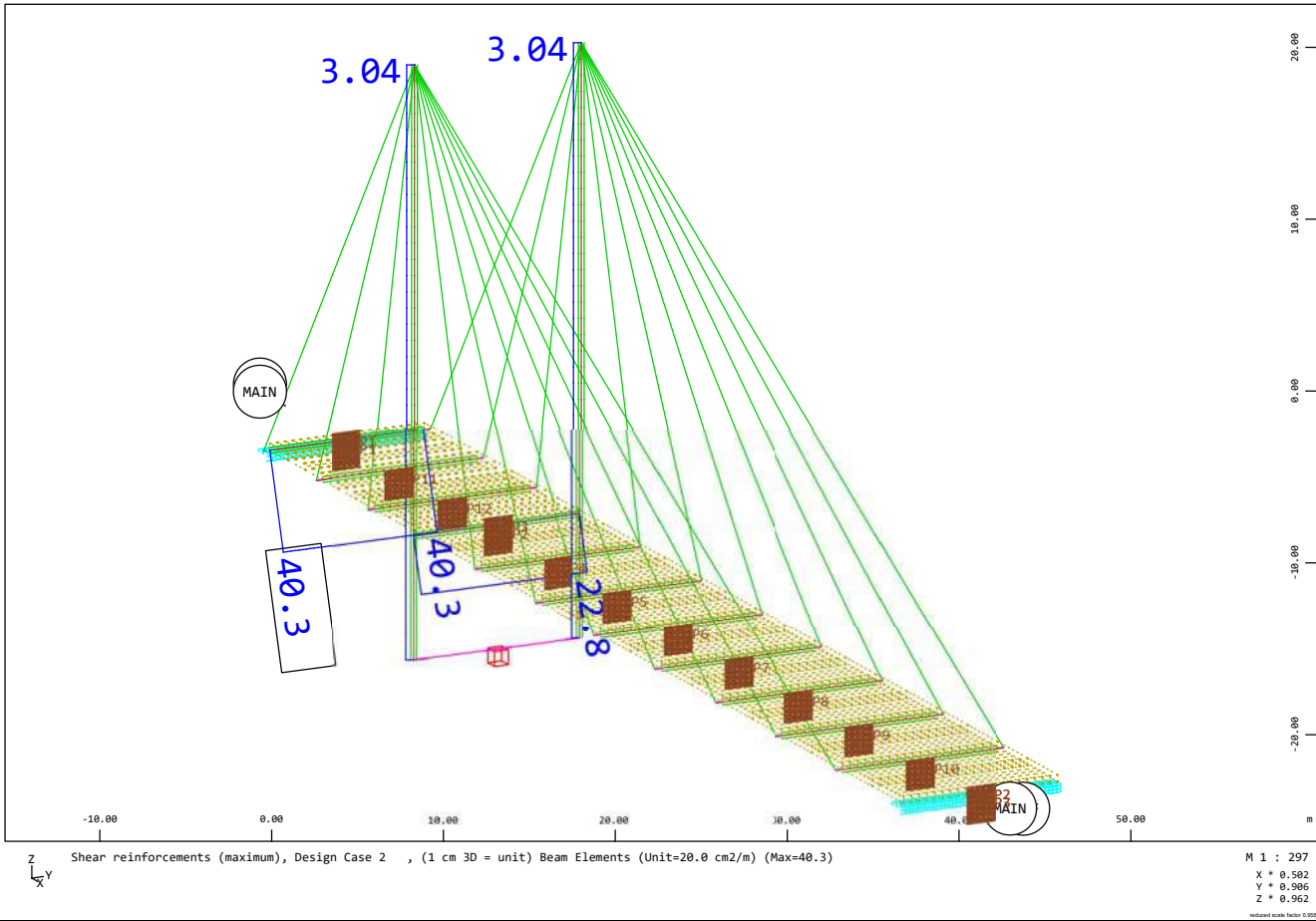
FRP_bridge_v1

SOFiSTiK AG - www.sofistik.de



FRP_bridge_v1

SOFiSTiK AG - www.sofistik.de



D.5. Wind load definition steel concrete composite bridge

Geometrie

Type	dek met langsligger:
totale lengte	84 m
breedte	10 m
vrije hoogte onderkant brug	10,95 m
constructiehoogte brug (d)	1,77 m
hellende loef en leizijde (α)	0 gr
helling brugdek (β)	0 gr
Verkeerstype	wegverkeer

Coördinatenstelsel

x	breedte
y	lengte
z	hoogte
NB - reductie agv α vervalt	
NB -toeslag dwarshelling moet meegenomen worden	
Verkeersband	2 meter

equivalente constructiehoogte overnemen vanuit tabel 2 (cel G25)

Referentie oppervlakken

wind in X-richting

Afscheidingstype	open brugleuning en open veiligheidsk
dicht of open kering	Aan beide kanten
leuning / schermhoogte (d_1)	1,2 m
d_{tot} - zonder verkeer	2,97 m
d_{tot} - met verkeer	3,77 m

$A_{ref,x}$ - zonder verkeer 249,48 m²

$A_{ref,x}$ - met verkeer 316,68 m²

wind in Z-richting

$A_{ref,z}$ 840 m²

Windbelastingen

Referentiehoogte (Z_e)	12,435 m
Ontwerplevensduur =	100 jaar
C _{prob} =	1,05

Windgebied = III

$V_{b,0}$ = 24 m/s

V_b = 24,0 m/s

V^*b = 24,1 m/s

For Norway

From Norwegian NA

C _{dir}	1	Terrain roughness	
C _{season}	1	Terrain ca III	
C _{alt}	1	z_0	0,3 m
C _{prob}	1	z_{min}	8 m
		z_{max}	200 m
		$z_{0,II}$	0,05 m
		z_0	0,3

H4.3.1

Variatie met hoogte

referentiehoogte (Z_e) = 12,4 1,00

(4.3)

referentiehoogte (Z_e) = 12,4 19,25 m/s
gereduceerde windsnelheid, verkeersband 12,4 19,30 m/s

H4.4

Windturbulentie

$I_z(z)$ =
referentiehoogte (Z_e) = 12,4 0,2685

H4.5

Extreme stuwdruk

	$q_p(z)$	$C_e(z)$	$C_s C_d$
referentiehoogte (Z_e) =	0,67 kN/m ²	1,853	0,89 alleen een waarde groter dan
reducede stuwdruk, verkeersband	0,67 kN/m ²	1,853	0,89 1,0 wordt verrekend als toeslag

Krachtcoëfficiënten

wind in X-richting

	b/d_{tot}	$C_{fx,0}$	toeslag dwarshelling	lijnfunctie
zonder verkeer	b/d_{tot}	3,37 1,50	1,50	Bouwstadium, open brugleuning e.d.
met verkeer	b/d_{tot}	2,65 1,72	1,72	Bouwstadium, open brugleuning e.d.

wind in Z-richting

zonder verkeer	b/d_{tot}	3,37	
hoek α van de wind met ho		0 gr	
$\theta = \alpha + \beta$		0 gr	
figuur NB.8 C_{fz} =	0,61		let op ook de negatieve waarde gebruiken voor wind in tegengestelde richting

Belasting door wind

wind in X-richting

representatieve belasting	per m1	Totaal	Prep
zonder verkeer	F_w = 3,0 kN/m1	249,5 kN	1,00 kN/m ²
met verkeer	F^*w = 4,4 kN/m1	365,7 kN	1,15 kN/m ²

wind in Y-richting

= 40% van windkrachten in de x richting. (in de x-richting treed dan gelijktijdig ook dezelfde kracht als in de y-richting op)

representatieve belasting	per m1	Totaal	Prep
zonder verkeer	F_w = 1,2 kN/m1	99,8 kN	0,40 kN/m ²
met verkeer	F^*w = 1,7 kN/m1	146,3 kN	0,46 kN/m ²

wind in Z-richting

Als belasting in Z-richting ongunstig werkt treed deze gelijktijdig met de x-richting op

representatieve belasting	per m1	Totaal	Prep
zonder verkeer	F_w = 4,0 kN/m1	339,4 kN	0,40 kN/m ²
met $e=b/4$	M_w = 10,1 kNm/m1	848,5 kNm/m1	
met verkeer	F^*w = 4,1 kN/m1	340,9 kN	0,41 kN/m ²
	M^*w = 10,1 kNm/m1	852,2 kNm/m1	

Belasting door wind

wind in X-richting

representatieve belasting

Totaal	Verdeelde belasting		
X-richting	Y-richting	Z-richting	X-richting
			Y-richting
			Z-richting

zonder verkeer	Fw =	249,5			kN	1,00			kN/m ²
met verkeer	F*w =	365,7			kN	1,15			kN/m ²

wind in Y-richting = 40% van windkrachten in de x richting. (in de x-richting treed dan gelijktijdig ook dezelfde kracht als in de y-richting op)
representatieve belasting

zonder verkeer	Fw =	99,8	99,8		kN	0,40	0,40		kN/m ²
met verkeer	F*w =	146,3	146,3		kN	0,46	0,46		kN/m ²

wind in Z-richting Als belasting in Z-richting ongunstig werkt treed deze gelijktijdig met de x-richting op
representatieve belasting

zonder verkeer	Fw =	249,5		339,4	kN	1,00		0,40	kN/m ²
	Mw =			848,5	kNm				
met verkeer	F*w =	365,7		340,9	kN	1,15		0,41	kN/m ²
	M*w =			852,2	kNm				

Pylon

Geometry

Height	36 [m]					
Width	1,3 [m]	Inner width	0,8 [m]	A	0,788 [m ²]	
Depth	1 [m]	Inner depth	0,64 [m]	Ac	1,3 [m ²]	

Transversal connector	height	1,1 [m]
	width	1,1 [m]
	Length	10 [m]

Pylon

Direction	x	y		Direction	y	z
d/b	0,7692	1,3		d/b	1	1
cf0	2,38	1,94		cf0	2,36	1,85
Ψr	1	1		Ψr	1	1
Effective slenderness λ	24,369	31,68	0,88*l/b	Effective slendernes	9,09	9,09
Solidity ratio A/Ac	0,6062	0,60615		Solidity ratio A/Ac	1	1
End-effect factor Ψλ	0,93	0,94		End-effect factor Ψλ	0,72	0,72
Force coefficient cf	2,2134	1,8236		Force coefficient cf	1,7	1,33

Transversal connector

Direction	y	z
d/b	1	1
cf0	2,36	1,85
Ψr	1	1
Effective slendernes	9,09	9,09
Solidity ratio A/Ac	1	1
End-effect factor Ψλ	0,72	0,72
Force coefficient cf	1,7	1,33

Pylon

Reference height	zb	[m]	12	24	36
Turbulence intensity	Iv	[-]	0,271085031	0,23	0,21
Terrain roughness factor	Cr(z)	[-]	0,79454528	0,94	1,03
Mean wind velocity	Traffic vm(z)*	[m/s]			
	No traffic vm(z)	[m/s]	19,06908672	22,7	24,7
Peak velocity pressure at	Traffic qp(ze)*	[kN/m ²]			
reference height zb	No traffic qp(ze)	[kN/m ²]	0,658532966	0,83	0,94

Direction	x	y
Reference area	1,3	1 [m ² /m]
	ze	x y
Force Fw	12	1,89488 1,2009007 [kN/m]
	24	2,39689 1,5190605 [kN/m]
	36	2,71196 1,7187389 [kN/m]

Verification for loads computed by Sofistik
Good accordance

Transversal connector

Reference height	zb	[m]	10,9
Turbulence intensity	Iv	[-]	0,278339436
Terrain roughness factor	Cr(z)	[-]	0,773836918
Mean wind velocity	Traffic vm(z)*	[m/s]	
	No traffic vm(z)	[m/s]	18,57208603
Peak velocity pressure at	Traffic qp(ze)*	[kN/m ²]	
reference height zb	No traffic qp(ze)	[kN/m ²]	0,635600553

Direction	y	z
Reference area	1,1	1,1 [m ² /m]
Force Fw	1,188	0,93128 [kN/m]

Cables

Geometry		1	2	3	4	5	BS cable
Cable anchor distance	[m]	13	26	39	52	10	20
Diameter	[m]	0,0602	0,0618	0,0731	0,08	0,06	0,323
Length	[m]	30,87	38,21	48,01	59,1	29,7	34,41

cf0	1,2	For stranded cables, (7.9.2)
End-effect factor Ψλ	1	
Force coefficient cf	1,2	

Top of cables

Reference height	zb	[m]	36	11,8
Turbulence intensity	Iv	[-]	0,208877645	0,27
Terrain roughness factor	Cr(z)	[-]	1,031174646	0,79
Mean wind velocity	Traffic vm(z)*	[m/s]		
	No traffic vm(z)	[m/s]	24,74819151	19
Peak velocity pressure at	Traffic qp(ze)*	[kN/m ²]		
reference height zb	No traffic qp(ze)	[kN/m ²]	0,942497739	0,66

Cables		1	2	3	4	5	BS cable
Reference area	[m ² /m]	0,060	0,062	0,073	0,077	0,060	0,323
Force Fw bottom cables	[kN/m]	0,047	0,049	0,057	0,060	0,047	0,254
Force Fw top cables	[kN/m]	0,068	0,070	0,083	0,087	0,068	0,365

D.6. Results steel-concrete composite bridge - summary

Composite deck

Material factors

Concrete C40/50

γ_M 1,5

B500

γ_M 1,15

Steel S355

γ_M 1

height concrete

350 [mm]

Total A.s.x per m

33,12 [cm²/m]

Total A.s.y per m

33,12 [cm²/m]

Concrete C40/50

E_c [Mpa]

35000

Density [kN/m³]

25

f_{ck} [Mpa]

f_{ctm} [Mpa]

40

3,5

Reinforcement steel B500

f_y [Mpa]

500

Structural steel S355

E [Gpa]

210

f_y [Mpa]

355

Density [kN/m³]

78,5

Design values slab

Slab is assumed to be under full compression or full tension when checked.

Compression

$N_{Rd,c}$ -9333,333333 [kN/m]

Tension

$N_{Rd,t,x}$ 1440 [kN/m]

$N_{Rd,t,y}$ 1440 [kN/m]

$V_{Rd,c}$ 291,115 [kN/m]

Stress checks

ULS - design combinations

Steel girders checked with Sofistik module

Concrete slab	Force [kN/m]	Unity check [-]	
nxx	-5299	0,56775	At side span support
	1262	0,876388889	At side span support
nyy	-2748	0,294428571	At side span support
	950	0,659722222	At side span support
Shear	103,4	0,355186095	Slab/girder

Deflections

SLS - freq combination

Main span

Global deflection in Z-dir

82,2 [mm]

Limit:

182,8571429 [mm]

UC

0,44953125

Side span

Global deflection in Z-dir

15,7 [mm]

Limit:

57,14285714 [mm]

UC

0,27475

Pylon and transversal connector

Deflections pylon

SLS- freq combination

x-direction

y-direction

13,3

46,5 [mm]

ULS checks performed by Sofistik

Cables

Steel cables ; Y1860

E_{eff} [Mpa]

19000

f_s [Mpa]

837

Δf_s [Mpa]

200

A_s [mm²]

150

Density [kN/m³]

78,5

Stress checks

Cable	max N [kN]	A actual [mm ²]	max stress [Mpa]	UC stress max
1	1405	2400	585	0,70
2	1770	2850	621	0,74
3	2164	3600	601	0,72
4	2387	4200	568	0,68
5	899,7	2400	375	0,45
Backing stay cable	9138	81900	112	0,13

D.7. Results steel-concrete composite bridge - extended Sofistik printout

Composite model v2

Materials

Default design code is EuroNorm 1994-2:2005 Composite Structures (Norge) V 2018

Structure: B (Road bridges)

Snow load zone : 1

Materials

Mat	Classification
1	C 40/50 (EN 1992)
2	Y 1860 A (EN 1992)
4	B 500 B (EN 1992)
5	S 355 (EN 1993)

Mat 1 C 40/50 (EN 1992)

Young's modulus	E	35220	[N/mm2]	Safetyfactor		1.50	[-]
Poisson's ratio	μ	0.20	[-]	Strength	fc	34.00	[MPa]
Shear modulus	G	14675	[N/mm2]	Nominal strength	fck	40.00	[MPa]
Compression modulus	K	19567	[N/mm2]	Tensile strength	fctm	3.51	[MPa]
Nominal Weight	γ	25.0	[kN/m3]	Tensile strength	fctk,05	2.46	[MPa]
Mean density	ρ	2400.0	[kg/m3]	Tensile strength	fctk,95	4.56	[MPa]
Elongation coefficient	α	1.00E-05	[1/K]	Bond strength	fbd	3.68	[MPa]
				Service strength	fc	48.00	[MPa]
				Fatigue strength	fcd,fat	19.04	[MPa]
				Tensile strength	fctd	1.64	[MPa]
				Tensile failure energy	Gf	0.15	[N/mm]

Mat 2 Y 1860 A (EN 1992)

Young's modulus	E	195000	[N/mm2]	Safetyfactor		1.10	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	1600.00	[MPa]
Shear modulus	G	75000	[N/mm2]	Compressive yield	fyc	1600.00	[MPa]
Compression modulus	K	162500	[N/mm2]	Tensile strength	ft	1860.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	1860.00	[MPa]
Mean density	ρ	7850.0	[kg/m3]	Ultimate strain		60.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.		0.75	[-]
max. thickness	t-max	18.00	[mm]	EN 1992 bond coeff.	k1	1.60	[-]
Relaxation	EN-1992	Class 2		Hardening modulus	Eh	0.00	[MPa]
Relaxation	$\rho(1000h)$	2.50	[%]	Proportional limit	fp	1600.00	[MPa]
				Dynamic allowance	σ -dyn	160.87	[MPa]

Composite model v2

Materials

Mat 4 B 500 B (EN 1992)

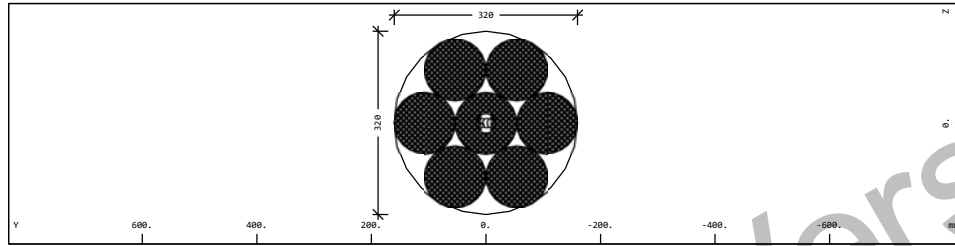
Young's modulus	E	200000	[N/mm2]	Safetyfactor		1.15	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	500.00	[MPa]
Shear modulus	G	76923	[N/mm2]	Compressive yield	fyc	500.00	[MPa]
Compression modulus	K	166667	[N/mm2]	Tensile strength	ft	540.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	540.00	[MPa]
Mean density	ρ	7850.0	[kg/m3]	Ultimate strain		50.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.		1.00	[-]
max. thickness	t-max	32.00	[mm]	EN 1992 bond coeff.	k1	0.80	[-]
				Hardening modulus	Eh	0.00	[MPa]
				Proportional limit	fp	500.00	[MPa]
				Dynamic allowance	σ -dyn	152.17	[MPa]

Mat 5 S 355 (EN 1993)

Young's modulus	E	210000	[N/mm2]	Safetyfactor		1.10	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	355.00	[MPa]
Shear modulus	G	80769	[N/mm2]	Compressive yield	fyc	355.00	[MPa]
Compression modulus	K	175000	[N/mm2]	Tensile strength	ft	490.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	490.00	[MPa]
Mean density	ρ	7850.0	[kg/m3]	Ultimate strain		100.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.		0.00	[-]
max. thickness	t-max	40.00	[mm]	EN 1992 bond coeff.	k1	0.00	[-]
				Hardening modulus	Eh	0.00	[MPa]
				Proportional limit	fp	355.00	[MPa]
				Dynamic allowance	σ -dyn	0.00	[MPa]

Composite model v2

Cross section No. 1 - Cable 320 1X7



Cross section No. 1 - Cable 320 1X7

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α [°]
1	2	8.0384E-02		5.145E-04	0.0	0.0	195000	1024.0	
		0.000E+00		5.145E-04	0.0	0.0	75000	(CABLE)	
= Cable 320 1X7									
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid		
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre		
A[m2]	sectional area				E[N/mm2]		Young's modulus		
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length		
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia								
I-1[m4],I-2[m4], α [°]	principal moments of inertia and angle of the principal axes								
MRf	reinforcement material number								
It[m4]	torsional moment of inertia								
G[N/mm2]	Shear modulus								

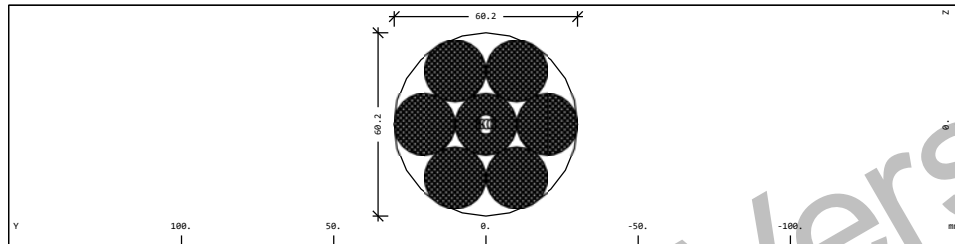
Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	85708.8						0.00	0.00	0.0	0.0	
D ¹	51944.7						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force				Mb[kNm2]		warping moment				
Vy[kN],Vz[kN]	shear force				My[kNm],Mz[kNm]		bending moment				
Mt[kNm]	primary torsional moment				y[mm],z[mm]		ordinate of plastic centre				
Mt2[kNm]	secondary torsional moment				BUCK		buckling curve (LTB, y-y, z-z)				

Composite model v2

Cross Sections

Cross section No. 2 - Cable 60.2 1X7



Cross section No. 2 - Cable 60.2 1X7

Static properties of cross section

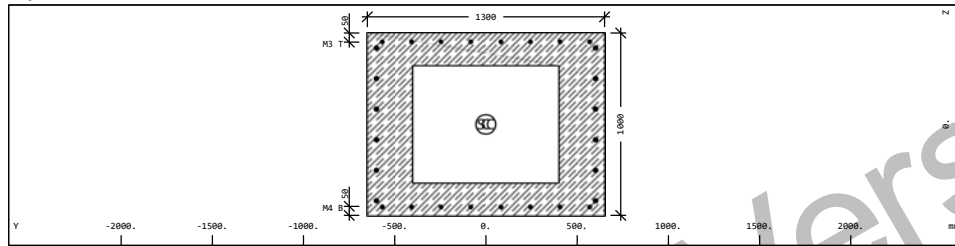
SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α [°]
2	2	2.8449E-03		6.444E-07	0.0	0.0	195000	36.2	
		0.000E+00		6.444E-07	0.0	0.0	75000	(CABLE)	
= Cable 60.2 1X7									
SNo	section number				yc[mm],zc[mm]		ordinate of elastic centroid		
Mat	material number				ysc[mm],zsc[mm]		ordinate of shear centre		
A[m2]	sectional area				E[N/mm2]		Young's modulus		
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area				g[kg/m]		weight per length		
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia								
I-1[m4],I-2[m4], α [°]	principal moments of inertia and angle of the principal axes								
MRf	reinforcement material number								
It[m4]	torsional moment of inertia								
G[N/mm2]	Shear modulus								

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	3033.3						0.00	0.00	0.0	0.0	
D ¹	1838.4						0.00	0.00	0.0	0.0	
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)											
N[kN]	normal force				Mb[kNm2]		warping moment				
Vy[kN],Vz[kN]	shear force				My[kNm],Mz[kNm]		bending moment				
Mt[kNm]	primary torsional moment				y[mm],z[mm]		ordinate of plastic centre				
Mt2[kNm]	secondary torsional moment				BUCK		buckling curve (LTB, y-y, z-z)				

Composite model v2
Cross Sections

Cross section No. 3 - Pylon



Cross section No. 3 - Pylon

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$
3	1	7.8800E-01	4.058E-01	9.086E-02	0.0	0.0	35220	1970.0	1.558E-01
	4	1.899E-01	3.845E-01	1.558E-01	0.0	0.0	14675	(BEAM)	9.086E-02

= Pylon

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m2]	sectional area	E[N/mm2]	Young's modulus
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia		
I-1[m4],I-2[m4], $\alpha[^\circ]$	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m4]	torsional moment of inertia		
G[N/mm2]	Shear modulus		

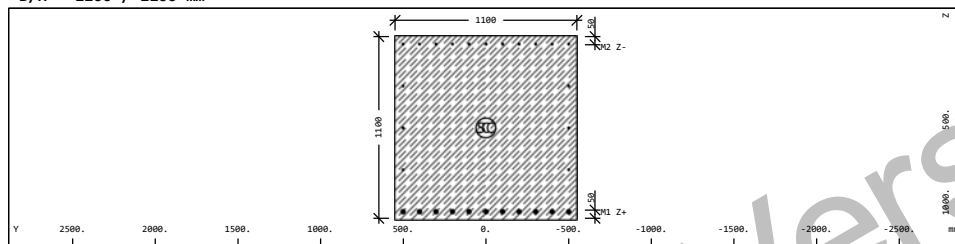
Reinforcement global values

Layer	Mref	Mat	As	As-min	As-max	D	yr	zr	L-tors	N-p	My-p	Mz-p
			[cm2]	[cm2]	[cm2]	[mm]	[mm]	[mm]	[mm]	[kN]	[kNm]	[kNm]
M1	R	1	4	23.66	23.66	12	600.0	0.0	1175.2			
M2	L	1	4	23.66	23.66	12	-600.0	0.0	1175.2			
M3	T	1	4	23.66	23.66	12	0.0	-450.0	1175.2			
M4	B	1	4	23.66	23.66	12	0.0	450.0	1175.2			

Layer	layer of reinforcement	D	bar diameter
Mref	embedding reference material	yr,zr	ordinate of elastic centroid
Mat	material number	L-tors	torsional effective length
As	reinforcement area	N-p	prestress normal force
As-min	minimum reinforcement area	My-p,Mz-p	prestress bending moment
As-max	maximum reinforcement area		

Composite model v2
Cross Sections

Cross section No. 5 - B/H = 1100 / 1100 mm



Cross section No. 5 - B/H = 1100 / 1100 mm

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$
5	1	1.2100E+00	1.008E+00	1.220E-01	0.0	0.0	35220	3025.0	1.008E+00
	4	2.058E-01	1.008E+00	1.220E-01	550.0	550.0	14675	(BEAM)	1.008E+00

= B/H = 1100 / 1100 mm

= B/H = 1100 / 1100 mm

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m2]	sectional area	E[N/mm2]	Young's modulus
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia		
I-1[m4],I-2[m4], $\alpha[^\circ]$	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m4]	torsional moment of inertia		
G[N/mm2]	Shear modulus		

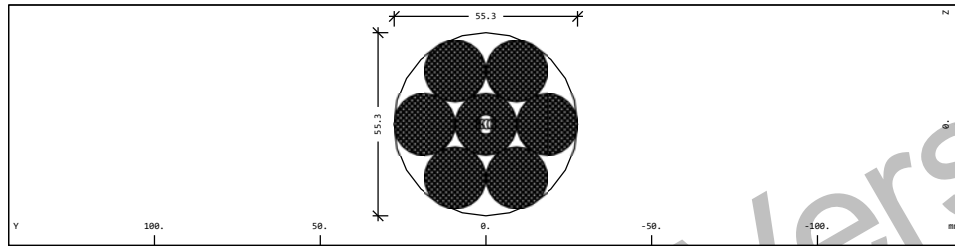
Reinforcement global values

Layer	Mref	Mat	As	As-min	As-max	D	yr	zr	L-tors	N-p	My-p	Mz-p
			[cm2]	[cm2]	[cm2]	[mm]	[mm]	[mm]	[mm]	[kN]	[kNm]	[kNm]
M1	Z+	1	4	54.01	54.01	25	0.0	1050.0	1000.0			
M2	Z-	1	4	8.69	8.69	10	0.0	50.0	1000.0			
M3	Y+-	1	4	4.74	4.74	10	0.0	550.0	2000.0			

Layer	layer of reinforcement	D	bar diameter
Mref	embedding reference material	yr,zr	ordinate of elastic centroid
Mat	material number	L-tors	torsional effective length
As	reinforcement area	N-p	prestress normal force
As-min	minimum reinforcement area	My-p,Mz-p	prestress bending moment
As-max	maximum reinforcement area		

Composite model v2
Cross Sections

Cross section No. 6 - Cable 55.3 1X7



Cross section No. 6 - Cable 55.3 1X7

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α [°]
6	2	2.4006E-03		4.588E-07	0.0	0.0	195000	30.6	
		0.000E+00		4.588E-07	0.0	0.0	75000	(CABLE)	

= Cable 55.3 1X7

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m2]	sectional area	E[N/mm2]	Young's modulus
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia		
I-1[m4],I-2[m4], α [°]	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m4]	torsional moment of inertia		
G[N/mm2]	Shear modulus		

Design forces and moments

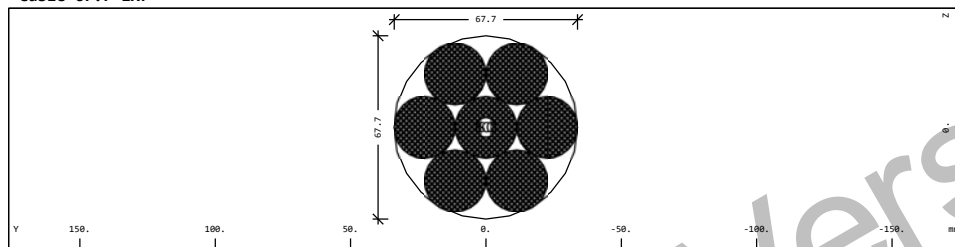
	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	2559.6						0.00	0.00	0.0	0.0	
D ¹	1551.3						0.00	0.00	0.0	0.0	

¹ C/D = plastic characteristic/design values (ultimate bearing capacity)

N[kN]	normal force	Mb[kNm2]	warping moment
Vy[kN],Vz[kN]	shear force	My[kNm],Mz[kNm]	bending moment
Mt[kNm]	primary torsional moment	y[mm],z[mm]	ordinate of plastic centre
Mt2[kNm]	secondary torsional moment	BUCK	buckling curve (LTB, y-y, z-z)

Composite model v2
Cross Sections

Cross section No. 7 - Cable 67.7 1X7



Cross section No. 7 - Cable 67.7 1X7

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α [°]
7	2	3.5979E-03		1.031E-06	0.0	0.0	195000	45.8	
		0.000E+00		1.031E-06	0.0	0.0	75000	(CABLE)	

= Cable 67.7 1X7

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m2]	sectional area	E[N/mm2]	Young's modulus
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia		
I-1[m4],I-2[m4], α [°]	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m4]	torsional moment of inertia		
G[N/mm2]	Shear modulus		

Design forces and moments

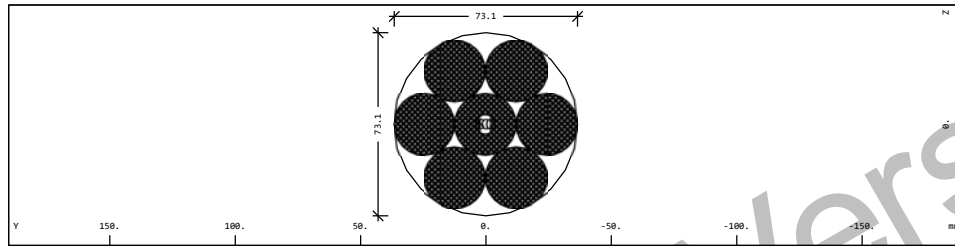
	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	3836.2						0.00	0.00	0.0	0.0	
D ¹	2325.0						0.00	0.00	0.0	0.0	

¹ C/D = plastic characteristic/design values (ultimate bearing capacity)

N[kN]	normal force	Mb[kNm2]	warping moment
Vy[kN],Vz[kN]	shear force	My[kNm],Mz[kNm]	bending moment
Mt[kNm]	primary torsional moment	y[mm],z[mm]	ordinate of plastic centre
Mt2[kNm]	secondary torsional moment	BUCK	buckling curve (LTB, y-y, z-z)

Composite model v2
Cross Sections

Cross section No. 8 - Cable 73.1 1X7



Cross section No. 8 - Cable 73.1 1X7

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α [°]
8	2	4.1947E-03		1.401E-06	0.0	0.0	195000	53.4	
		0.000E+00		1.401E-06	0.0	0.0	75000	(CABLE)	

= Cable 73.1 1X7

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m2]	sectional area	E[N/mm2]	Young's modulus
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia		
I-1[m4],I-2[m4], α [°]	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m4]	torsional moment of inertia		
G[N/mm2]	Shear modulus		

Design forces and moments

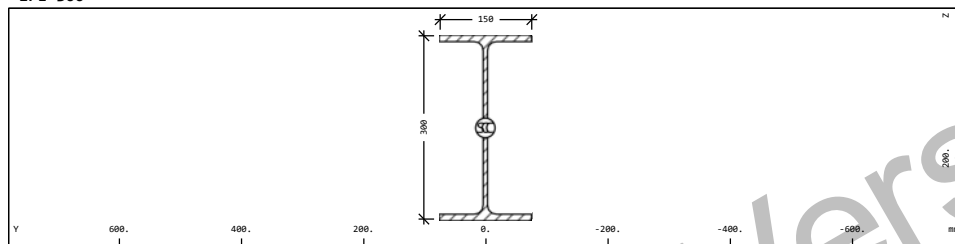
	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	4472.6						0.00	0.00	0.0	0.0	
D ¹	2710.7						0.00	0.00	0.0	0.0	

¹ C/D = plastic characteristic/design values (ultimate bearing capacity)

N[kN]	normal force	Mb[kNm2]	warping moment
Vy[kN],Vz[kN]	shear force	My[kNm],Mz[kNm]	bending moment
Mt[kNm]	primary torsional moment	y[mm],z[mm]	ordinate of plastic centre
Mt2[kNm]	secondary torsional moment	BUCK	buckling curve (LTB, y-y, z-z)

Composite model v2
Cross Sections

Cross section No. 9 - IPE 300



Cross section No. 9 - IPE 300

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					α [°]
9	5	5.3812E-03	2.940E-03	8.356E-05	0.0	0.0	210000	42.2	
		1.976E-07	2.077E-03	6.038E-06	150.0	150.0	80769	(BEAM)	

= IPE 300

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m2]	sectional area	E[N/mm2]	Young's modulus
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia		
I-1[m4],I-2[m4], α [°]	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m4]	torsional moment of inertia		
G[N/mm2]	Shear modulus		

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	1910.3	657.92	526.37	5.61	95.17	6.18	223.06	44.46	0.0	150.0	-, a, b
E ²	1910.3	407.79	387.04	2.22	58.57	3.95	197.75	28.58	0.0	150.0	
D ¹	1736.7	598.11	478.52	5.10	86.52	5.62	202.78	40.42	0.0	150.0	
F ²	1736.7	370.72	351.85	2.02	53.24	3.59	179.77	25.98	0.0	150.0	

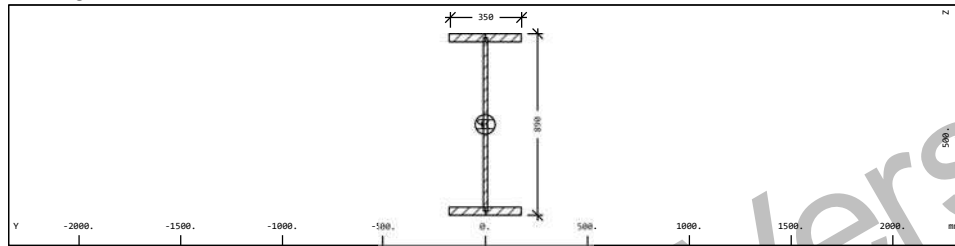
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)² E/F = elastic characteristic/design values (maximum strain reaches yield limit)

N[kN]	normal force	Mb[kNm2]	warping moment
Vy[kN],Vz[kN]	shear force	My[kNm],Mz[kNm]	bending moment
Mt[kNm]	primary torsional moment	y[mm],z[mm]	ordinate of plastic centre
Mt2[kNm]	secondary torsional moment	BUCK	buckling curve (LTB, y-y, z-z)

Composite model v2

Cross Sections

Cross section No. 10 - Steel girder



Cross section No. 10 - Steel girder

Static properties of cross section

SNo	Mat	A[m ²]	Ay[m ²]	Iy[m ⁴]	yc[mm]	ysc[mm]	E[N/mm ²]	g[kg/m]	I-1[m ⁴]
	MRf	It[m ⁴]	Az[m ²]	Iz[m ⁴]	zc[mm]	zsc[mm]	G[N/mm ²]		I-2[m ⁴]
			Ayz[m ²]	Iyz[m ⁴]					α[°]
10	5	4.5000E-02	2.333E-02	6.081E-03	0.0	0.0	210000	353.2	
		1.720E-05	1.651E-02	2.858E-04	435.0	435.0	80769	(BEAM)	

= Steel girder

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m ²]	sectional area	E[N/mm ²]	Young's modulus
Ay[m ²],Az[m ²],Ayz[m ²]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m ⁴],Iz[m ⁴],Iyz[m ⁴]	bending moment of inertia		
I-1[m ⁴],I-2[m ⁴],α[°]	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m ⁴]	torsional moment of inertia		
G[N/mm ²]	Shear modulus		

Design forces and moments

	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	15975.0	5738.86	3484.31	149.62	2439.02	369.64	5506.94	899.92	0.0	435.0	-, b, c
E ²	15975.0	3825.91	3213.84	88.13	1626.01	246.43	5079.46	579.83	0.0	435.0	
D ¹	14522.7	5217.15	3167.55	136.02	2217.29	336.04	5006.31	818.11	0.0	435.0	
F ²	14522.7	3478.10	2921.67	80.12	1478.19	224.03	4617.69	527.12	0.0	435.0	

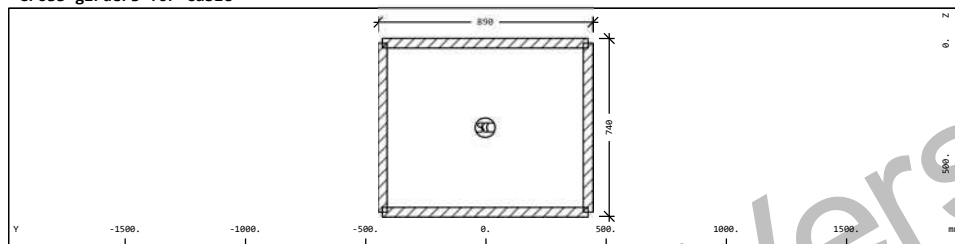
¹ C/D = plastic characteristic/design values (ultimate bearing capacity)² E/F = elastic characteristic/design values (maximum strain reaches yield limit)

N[kN]	normal force	Mb[kNm2]	warping moment
Vy[kN],Vz[kN]	shear force	My[kNm],Mz[kNm]	bending moment
Mt[kNm]	primary torsional moment	y[mm],z[mm]	ordinate of plastic centre
Mt2[kNm]	secondary torsional moment	BUCK	buckling curve (LTB, y-y, z-z)

Composite model v2

Cross Sections

Cross section No. 12 - Cross girders for cable



Cross section No. 12 - Cross girders for cable

Static properties of cross section

SNo	Mat	A[m ²]	Ay[m ²]	Iy[m ⁴]	yc[mm]	ysc[mm]	E[N/mm ²]	g[kg/m]	I-1[m ⁴]
	MRf	It[m ⁴]	Az[m ²]	Iz[m ⁴]	zc[mm]	zsc[mm]	G[N/mm ²]		I-2[m ⁴]
			Ayz[m ²]	Iyz[m ⁴]					α[°]
12	5	1.2400E-01	5.884E-02	1.062E-02	0.0	0.0	210000	973.4	1.421E-02
		1.834E-02	4.450E-02	1.421E-02	350.0	350.0	80769	(BEAM)	1.062E-02
				-2.260E-10					90.00

= Cross girders for cable

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m ²]	sectional area	E[N/mm ²]	Young's modulus
Ay[m ²],Az[m ²],Ayz[m ²]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m ⁴],Iz[m ⁴],Iyz[m ⁴]	bending moment of inertia		
I-1[m ⁴],I-2[m ⁴],α[°]	principal moments of inertia and angle of the principal axes		
MRf	reinforcement material number		
It[m ⁴]	torsional moment of inertia		
G[N/mm ²]	Shear modulus		

Design forces and moments

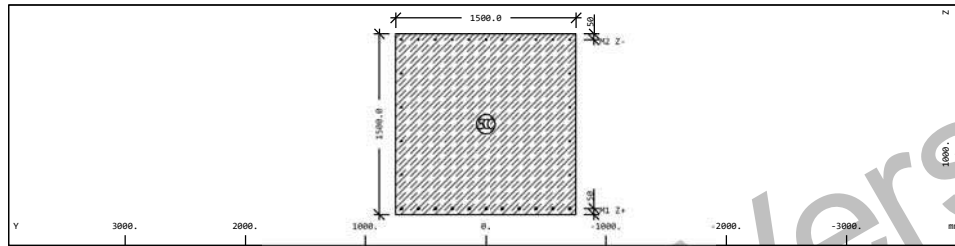
	N[kN]	Vy[kN]	Vz[kN]	Mt[kNm]	Mt2[kNm]	Mb[kNm2]	My[kNm]	Mz[kNm]	y[mm]	z[mm]	BUCK
C ¹	44020.0	13937.24	11477.72	10122.58	0.00	3273.99	11928.00	13578.75	0.0	350.0	-, b, c
E ²	44020.0	12182.18	10361.83	8867.38	609.75	211.22	10768.33	11868.83	0.0	350.0	
D ¹	40018.2	12670.21	10434.29	9202.34		2976.35	10843.64	12344.32	0.0	350.0	
F ²	40018.2	11074.71	9419.85	8061.25	554.32	192.02	9789.40	10789.85	0.0	350.0	

¹ C/D = plastic characteristic/design values (ultimate bearing capacity)² E/F = elastic characteristic/design values (maximum strain reaches yield limit)

N[kN]	normal force	Mb[kNm2]	warping moment
Vy[kN],Vz[kN]	shear force	My[kNm],Mz[kNm]	bending moment
Mt[kNm]	primary torsional moment	y[mm],z[mm]	ordinate of plastic centre
Mt2[kNm]	secondary torsional moment	BUCK	buckling curve (LTB, y-y, z-z)

Composite model v2
Cross Sections

Cross section No. 16 - B/H = 1500 / 1500 mm



Cross section No. 16 - B/H = 1500 / 1500 mm

Static properties of cross section

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	Mrf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$
16	1	2.250E+00	1.875E+00	4.219E-01	0.0	0.0	35220	5625.0	
	4	7.117E-01	1.875E+00	4.219E-01	750.0	750.0	14675	(BEAM)	

= B/H = 1500 / 1500 mm

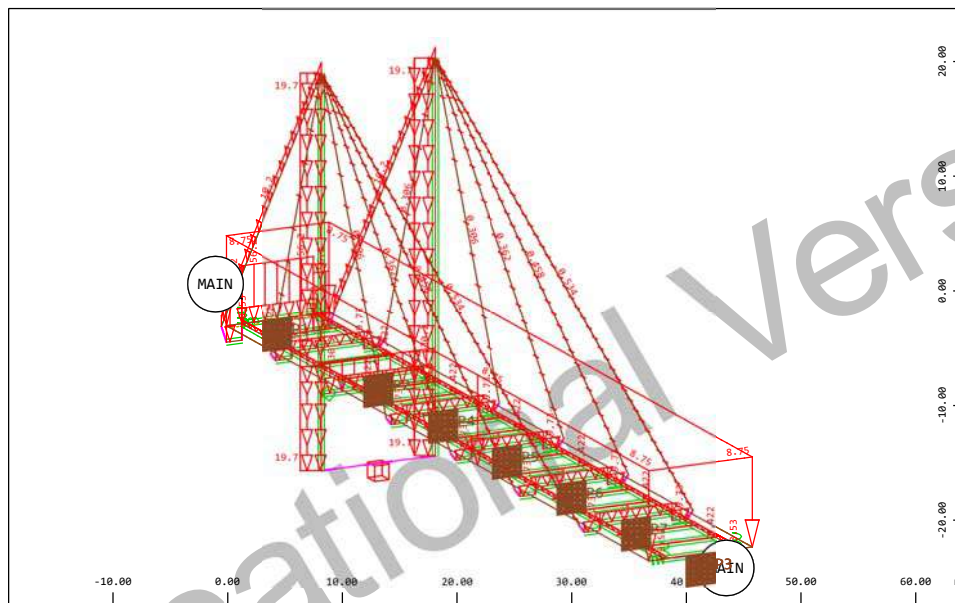
= B/H = 1500 / 1500 mm

SNo	section number	yc[mm],zc[mm]	ordinate of elastic centroid
Mat	material number	ysc[mm],zsc[mm]	ordinate of shear centre
A[m2]	sectional area	E[N/mm2]	Young's modulus
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area	g[kg/m]	weight per length
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia		
I-1[m4],I-2[m4], $\alpha[^\circ]$	principal moments of inertia and angle of the principal axes		
Mrf	reinforcement material number		
It[m4]	torsional moment of inertia		
G[N/mm2]	Shear modulus		

Reinforcement global values

Layer	Mref	Mat	As	As-min	As-max	D	yr	zr	L-tors	N-p	My-p	Mz-p
			[cm2]	[cm2]	[cm2]	[mm]	[mm]	[mm]	[mm]	[kN]	[kNm]	[kNm]
M1	Z+	1	4	34.54	34.54	20	0.0	1450.0	1400.0			
M2	Z-	1	4	8.69	8.69	10	0.0	50.0	1400.0			
M3	Y+-	1	4	6.32	6.32	10	0.0	750.0	2800.0			

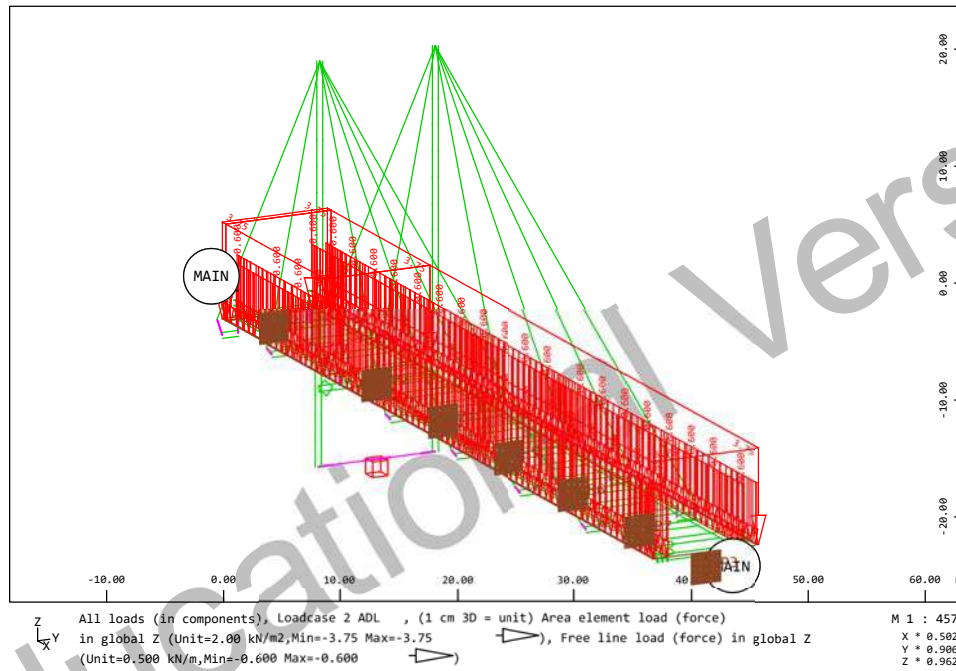
Layer	layer of reinforcement	D	bar diameter
Mref	embedding reference material	yr,zr	ordinate of elastic centroid
Mat	material number	L-tors	torsional effective length
As	reinforcement area	N-p	prestress normal force
As-min	minimum reinforcement area	My-p,Mz-p	prestress bending moment
As-max	maximum reinforcement area		

Composite model v2
Graphical Output

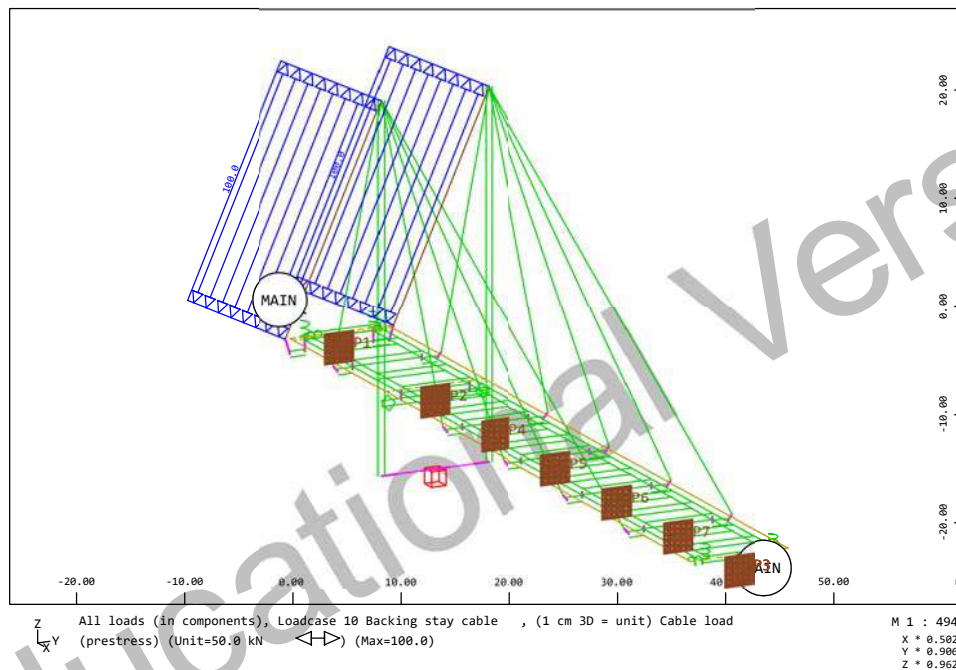
All loads (in components), Loadcase 1 Self-weight, (1 cm 3D = unit) QUAD-Area dead load
in global Z in Element (Unit=5.00 kN/m2, Min=-8.75 Max=-8.75) Beam dead load in
global Z (Unit=50.0 kN/m, Min=-56.2 Max=-0.422) Cable dead load in global Z

M 1 : 466
X * 0.502
Y * 0.906
Z * 0.962

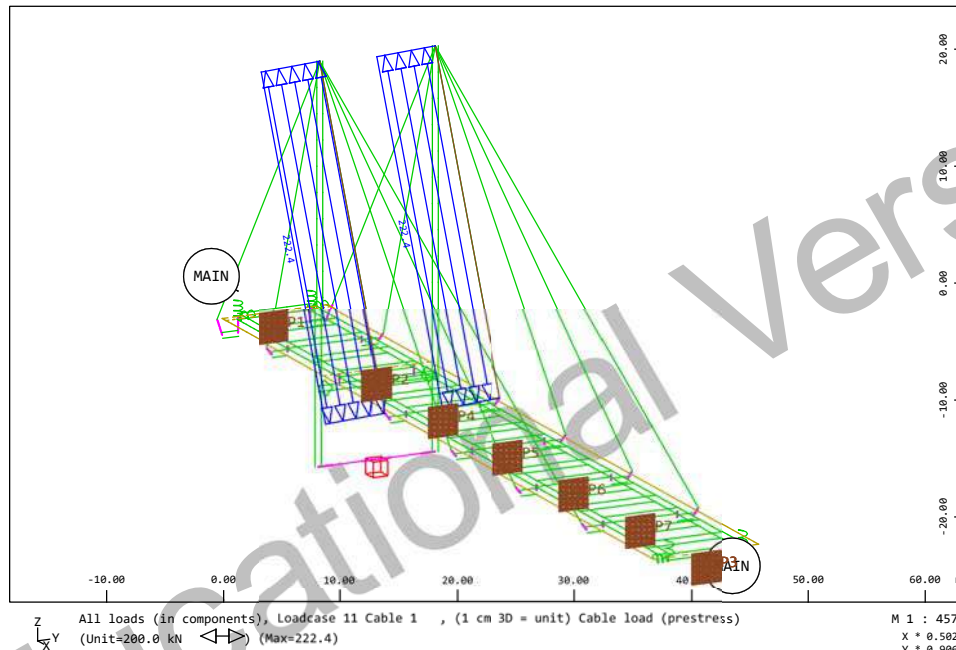
Composite model v2
Graphical Output



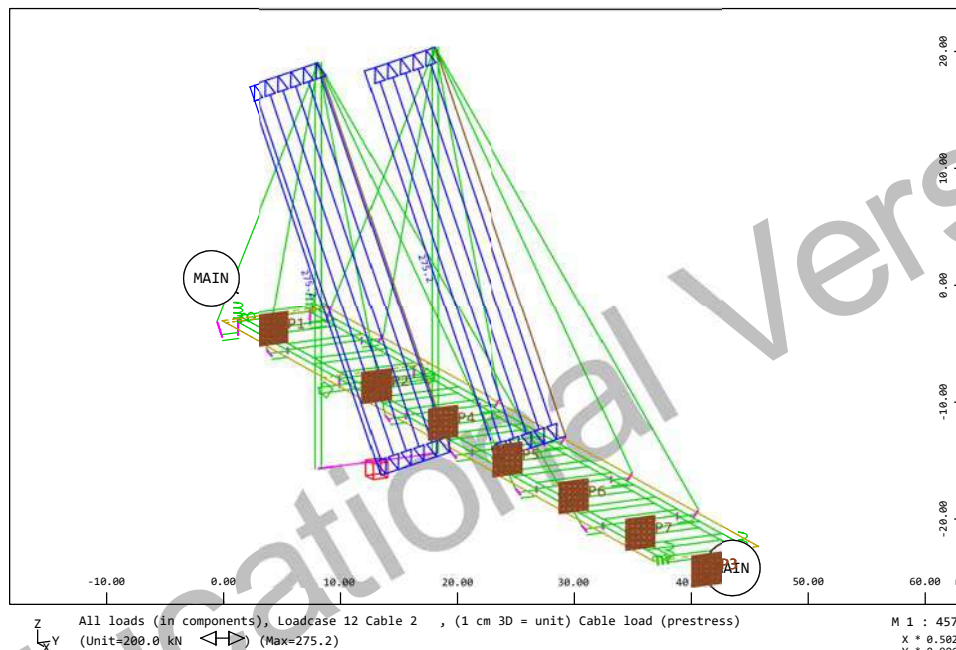
Composite model v2
Graphical Output



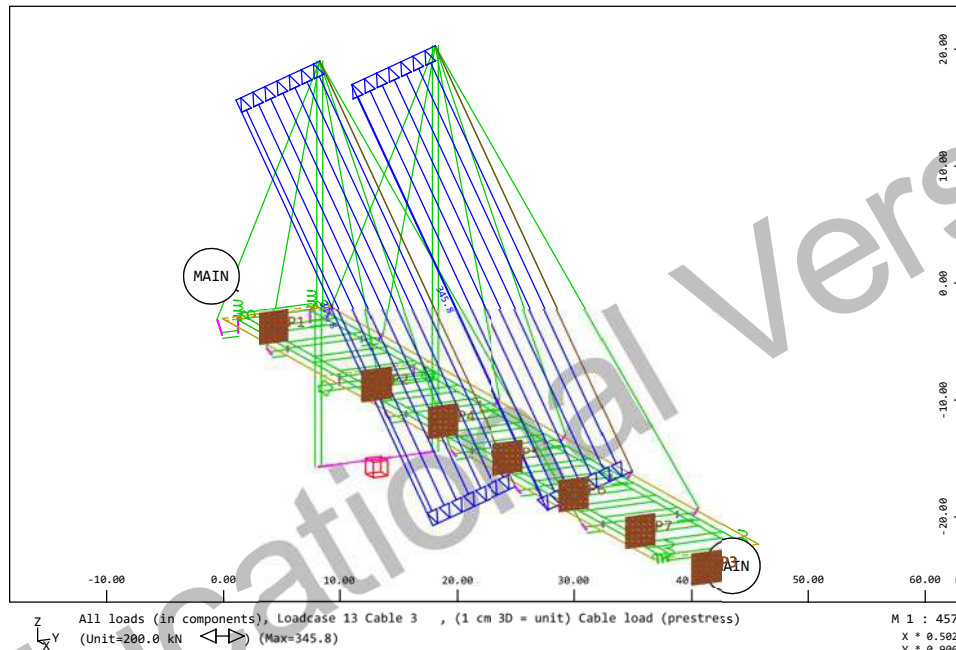
Composite model v2
Graphical Output



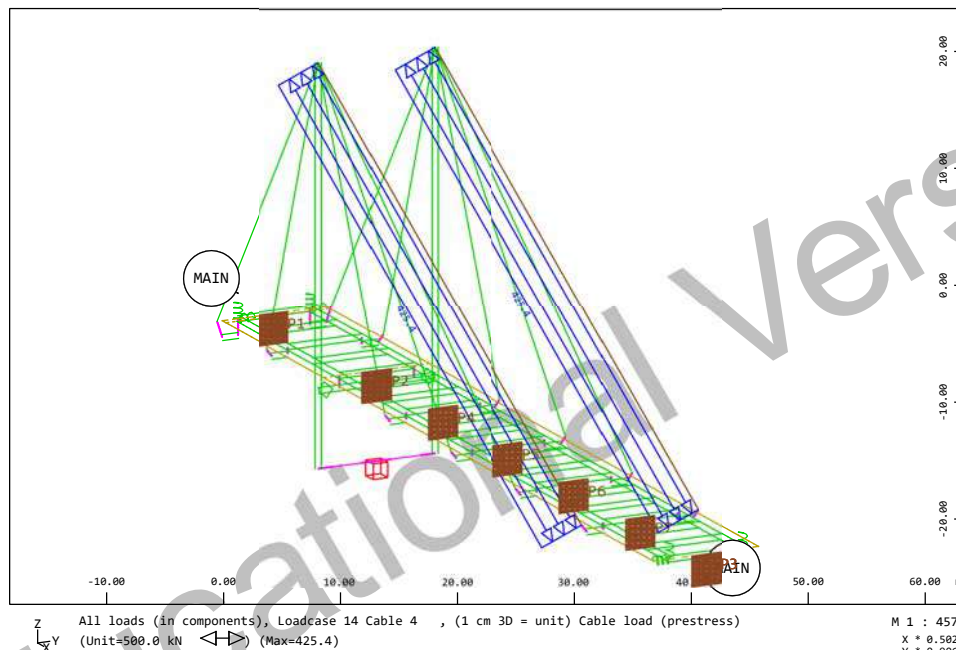
Composite model v2
Graphical Output



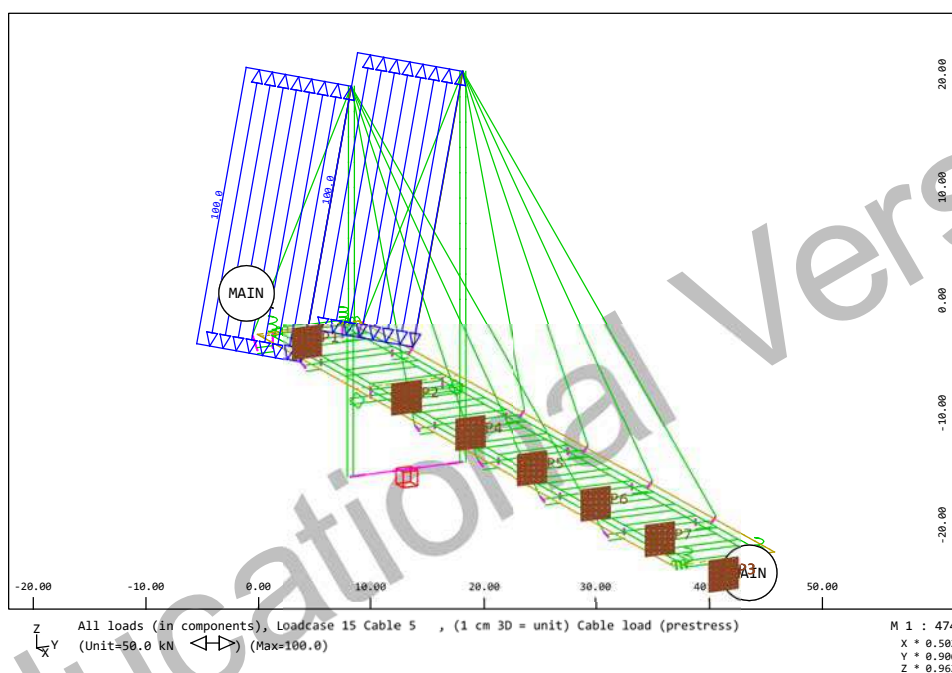
Composite model v2
Graphical Output



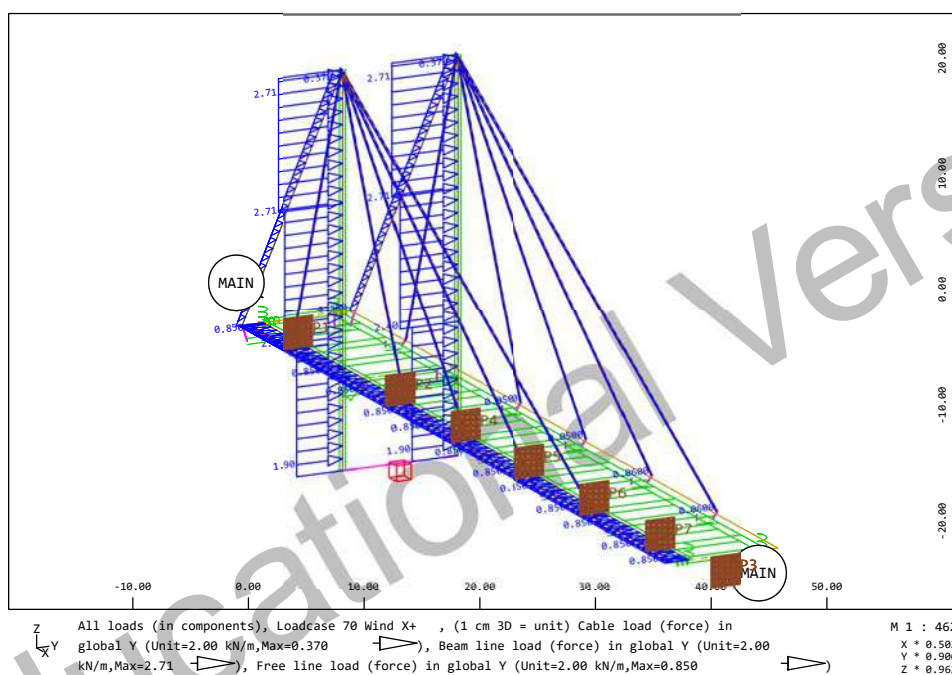
Composite model v2
Graphical Output



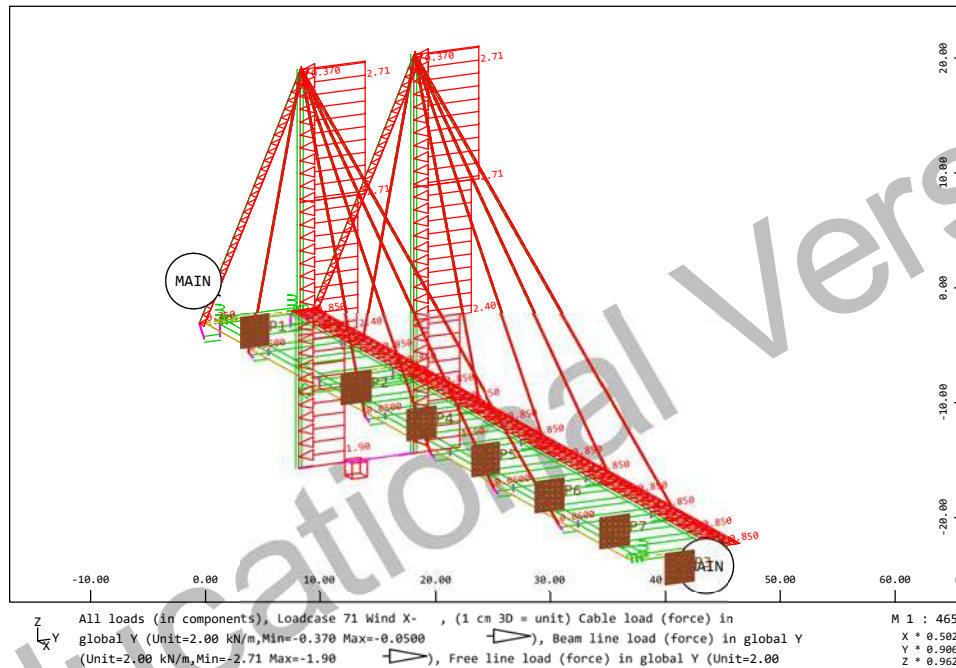
SOFiSTIK AG - www.sofistik.de



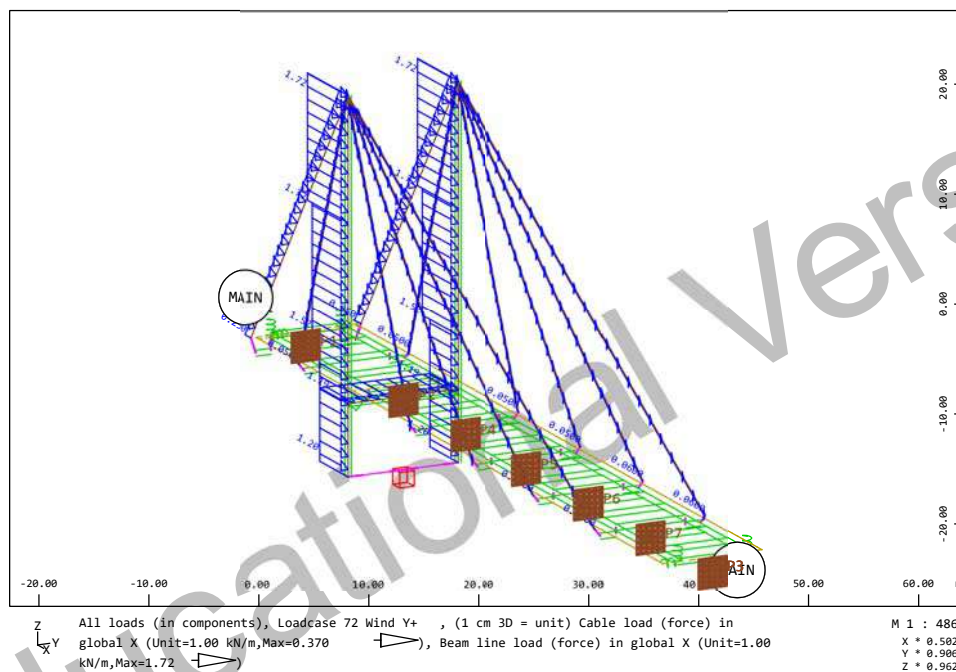
SOFiSTIK AG - www.sofistik.de

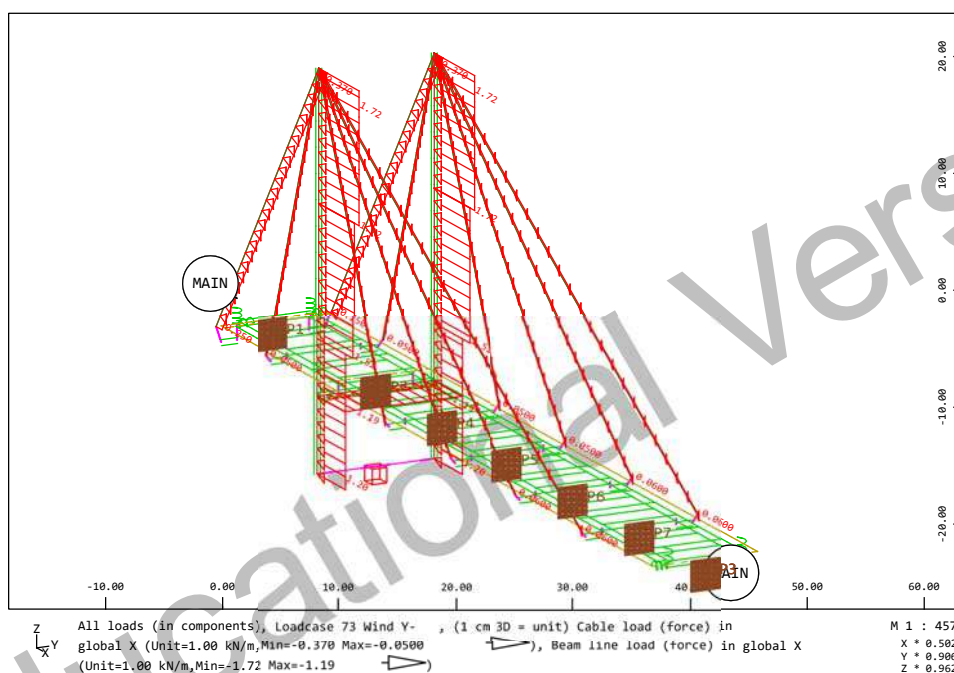


Composite model v2
Graphical Output

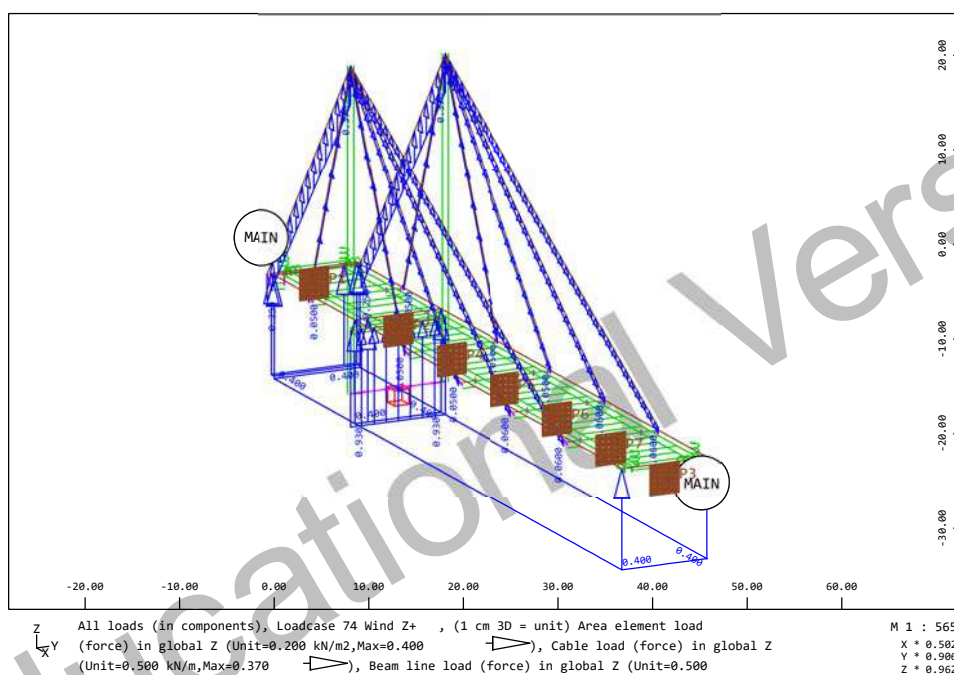


Composite model v2
Graphical Output

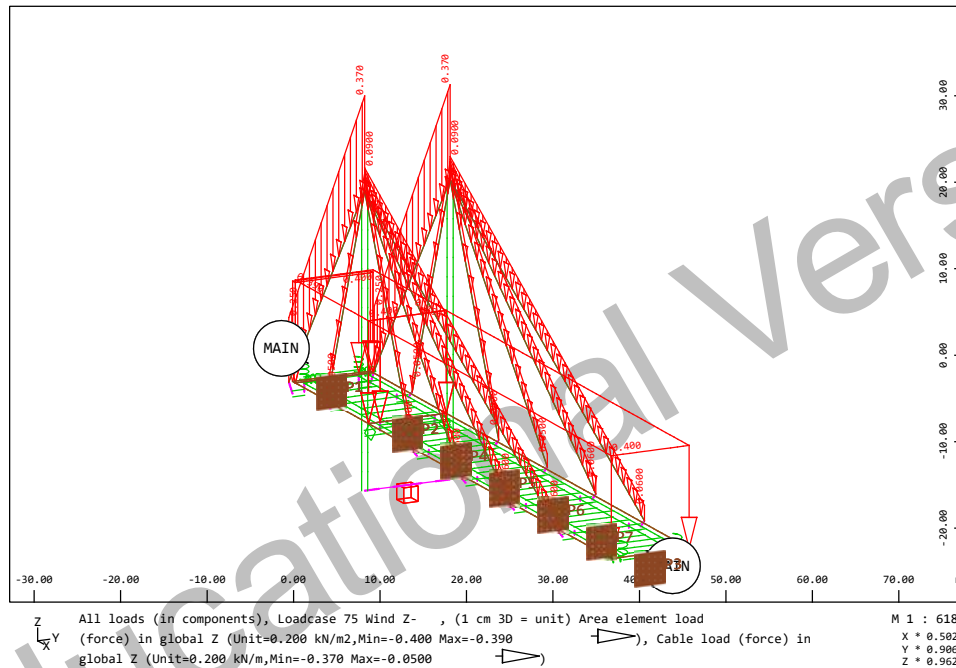


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Composite model v2
Graphical Output



Composite model v2
Load models

Actions

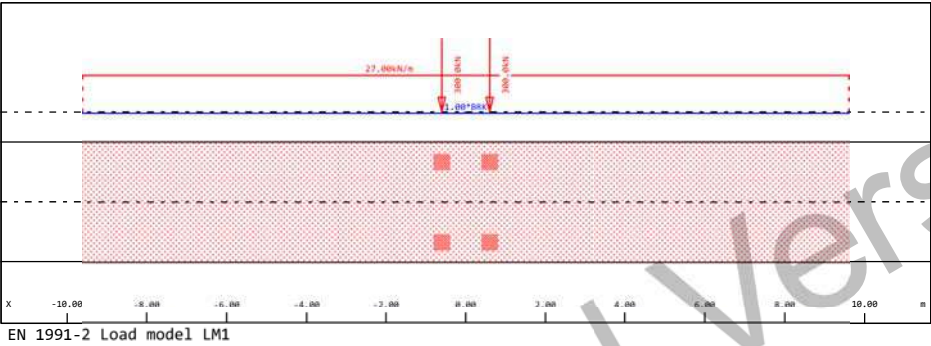
type	part	sup	Designation	γ-u	γ-f	γ-a	ψ ₀	ψ ₁	ψ ₂	ψ _{inf}
G_1	G	perm	dead load g1	1.35	1.00	1.00	1.00	1.00	1.00	1.00
G_2	G	perm	ADL	1.35	1.00	1.00	1.00	1.00	1.00	1.00
P	P	perm	prestressing	1.00	1.00	1.00	1.00	1.00	1.00	1.00
W	Q	excl	wind loading	1.50	0.00	1.00	0.60	0.20	0.00	0.60
GR_T	Q_1	excl	gria LM1	1.35	0.00	1.00	0.75	0.75	0.00	0.80
GR_U	Q_1	excl	gria LM1	1.35	0.00	1.00	0.40	0.40	0.00	0.80
			Reliability factor	Kfi	1.000					
			Reduction factor	xsi	0.890					
type	action	γ-u,γ-f,γ-a		partial safety factors for unfavourable/favourable/accidental						
part	partition of the action	ψ ₀ ,ψ ₁ ,ψ ₂ ,ψ _{inf}		combination coefficients						
sup	superposition type									

Load Train 100

LM1 300 EN 1991-2 Load model LM1

Load elements of Load Train	Load value	Remark
Load Train	300.000 [-]	
Axle load	300.0 [kN]	
Traffic Lane	9.00 [kN/m ²]	
Residual Area	2.50 [kN/m ²]	
Wind Load Height	3.500 [m]	
Brake load	900.0 [kN]	2.70 * L < 900 [kN]
Total factor	1.000 [-]	
Width of loading	3.000 [m]	
Fact.centrifugal	1.000 [-]	
Loading travels in both directions		

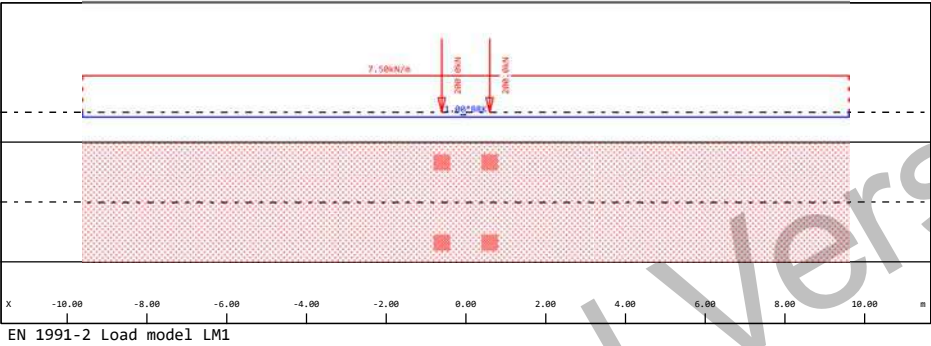
Composite model v2
Load models



Load Train 101
LM1 200 EN 1991-2 Load model LM1

Load elements of Load Train	Load value	Remark
Load Train	200.000 [-]	
Axle load	200.0 [kN]	
Traffic Lane	2.50 [kN/m2]	
Residual Area	2.50 [kN/m2]	
Wind Load Height	3.500 [m]	
Brake load	900.0 [kN]	
Total factor	1.000 [-]	
Width of loading	3.000 [m]	
Fact.centrifugal	1.000 [-]	
Loading travels in both directions		

Composite model v2
Load models



Superposition according to EuroNorm 1994-2:2005 Composite Structures

$$E_{d,rare} = E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus Q_{k,1} \oplus \sum_{i \geq 1} \psi_{0,i} \cdot Q_{k,i} \right\}$$

Resulting Load Cases type SLS characteristic combination

Load Case selection and Actions

Superposition Factors									Fact	Type	Designation			
Act	Part LC	γ_u	γ_f	γ_a	ψ_0	ψ_1	ψ_2	ψ_{inf}						
G_1	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00			dead load g1			
	1								1.00	PERM	Self-weight			
G_2	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00			ADL			
	2								1.00	PERM	ADL			
GR_T	Q_1	1.35	0.00	1.00	0.75	0.75	0.00	0.80			gr1a LM1			
	601								1.00	A42	MAX-PX	NODE GR_T:	my mx	p px
	602								1.00	A42	MIN-PX	NODE GR_T:	my mx	p px
	603								1.00	A42	MAX-PY	NODE GR_T:	vz my	t py
	604								1.00	A42	MIN-PY	NODE GR_T:	vz my	t py
	605								1.00	A42	MAX-PZ	NODE GR_T:	mt nx	m pz
	606								1.00	A42	MIN-PZ	NODE GR_T:	mt nx	m pz
	607								1.00	A42	MAX-UX	NODE GR_T:	n ny	ptx ux
	608								1.00	A42	MIN-UX	NODE GR_T:	n ny	ptx ux
	609								1.00	A42	MAX-UY	NODE GR_T:	mz vx	pty uy
	610								1.00	A42	MIN-UY	NODE GR_T:	mz vx	pty uy
	611								1.00	A42	MAX-UZ	NODE GR_T:	vy vy	ptz uz
	612								1.00	A42	MIN-UZ	NODE GR_T:	vy vy	ptz uz
	613								1.00	A42	MAX-MXY	QUAD GR_T:	mxy	
	614								1.00	A42	MIN-MXY	QUAD GR_T:	mxy	
	615								1.00	A42	MAX-NXY	QUAD GR_T:	nxy	
	616								1.00	A42	MIN-NXY	QUAD GR_T:	nxy	

Load Case selection and Actions

Act	Part	Superposition Factors							Fact	Type	Designation
		γ_u	γ_f	γ_a	ψ_0	ψ_1	ψ_2	ψ_{inf}			
GR_U	Q_1	1.35	0.00	1.00	0.40	0.40	0.00	0.80			gr1a LM1
	681								1.00	A25	MAX-PX NODE GR_U: my mx p px
	682								1.00	A25	MIN-PX NODE GR_U: my mx p px
	683								1.00	A25	MAX-PY NODE GR_U: vz my t py
	684								1.00	A25	MIN-PY NODE GR_U: vz my t py
	685								1.00	A25	MAX-PZ NODE GR_U: mt nx m pz
	686								1.00	A25	MIN-PZ NODE GR_U: mt nx m pz
	687								1.00	A25	MAX-UX NODE GR_U: n ny ptx ux
	688								1.00	A25	MIN-UX NODE GR_U: n ny ptx ux
	689								1.00	A25	MAX-UY NODE GR_U: mz vx pty uy
	690								1.00	A25	MIN-UY NODE GR_U: mz vx pty uy
	691								1.00	A25	MAX-UZ NODE GR_U: vy vy ptz uz
	692								1.00	A25	MIN-UZ NODE GR_U: vy vy ptz uz
	693								1.00	A25	MAX-MXY QUAD GR_U: mxy
	694								1.00	A25	MIN-MXY QUAD GR_U: mxy
	695								1.00	A25	MAX-NXY QUAD GR_U: nxy
	696								1.00	A25	MIN-NXY QUAD GR_U: nxy
P	P	1.00	1.00	1.00	1.00	1.00	1.00	1.00			prestressing
	16								1.00	PERM	Prestress cables
W	Q	1.50	0.00	1.00	0.60	0.20	0.00	0.60			wind loading
	800								1.00	A10	Wind X+
	801								1.00	A10	Wind X-
	802								1.00	A10	Wind Y+ 0.4*X+
	803								1.00	A10	Wind Y+ 0.4*X-
	804								1.00	A10	Wind Y- 0.4*X+
	805								1.00	A10	Wind Y- 0.4*X-
	806								1.00	A10	Wind Z+
	807								1.00	A10	Wind Z-
	808								1.00	A10	Wind load in x+ direction for Z
	809								1.00	A10	Wind load in x- direction for Z
Act	action								Fact	factor for load case	
Part	partition of the action								Type	type of the load case	
$\gamma_u, \gamma_f, \gamma_a$	partial safety factors for unfavourable/favourable/accidental								PERM	permanent load grouped in actions	
$\psi_0, \psi_1, \psi_2, \psi_{inf}$	combination coefficients								A	exclusive load	
LC	number of the load case										

Composite model v2
Superpositions

Combination rule Number 2

SLS frequent combination

Superposition according to manual MAXIMA formula 2.5

$$E_{d,frequ} = E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \psi_{2,i} \cdot Q_{k,i} \right\}$$

Resulting Load Cases type SLS frequent combination

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		γ -u	γ -f	γ -a	ψ_0	ψ_1	ψ_2	ψ_{1inf}			
G_1	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00	1.00	PERM	dead load g1
	1										Self-weight
G_2	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00	1.00	PERM	ADL
	2										ADL
GR_T	Q_1	1.35	0.00	1.00	0.75	0.75	0.00	0.80			gr1a LM1
	601								1.00	A42	MAX-PX NODE GR_T: my mx p px
	602								1.00	A42	MIN-PX NODE GR_T: my mx p px
	603								1.00	A42	MAX-PY NODE GR_T: vz my t py
	604								1.00	A42	MIN-PY NODE GR_T: vz my t py
	605								1.00	A42	MAX-PZ NODE GR_T: mt nx m pz
	606								1.00	A42	MIN-PZ NODE GR_T: mt nx m pz
	607								1.00	A42	MAX-UX NODE GR_T: n ny ptx ux
	608								1.00	A42	MIN-UX NODE GR_T: n ny ptx ux
	609								1.00	A42	MAX-UY NODE GR_T: mz vx pty uy
	610								1.00	A42	MIN-UY NODE GR_T: mz vx pty uy
	611								1.00	A42	MAX-UZ NODE GR_T: vy vy ptz uz
	612								1.00	A42	MIN-UZ NODE GR_T: vy vy ptz uz
	613								1.00	A42	MAX-MXY QUAD GR_T: mxy
	614								1.00	A42	MIN-MXY QUAD GR_T: mxy
	615								1.00	A42	MAX-NXY QUAD GR_T: nxy
	616								1.00	A42	MIN-NXY QUAD GR_T: nxy

Composite model v2
Superpositions

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		γ -u	γ -f	γ -a	ψ_0	ψ_1	ψ_2	ψ_{1inf}			
GR_U	Q_1	1.35	0.00	1.00	0.40	0.40	0.00	0.80			gr1a LM1
	681								1.00	A25	MAX-PX NODE GR_U: my mx p px
	682								1.00	A25	MIN-PX NODE GR_U: my mx p px
	683								1.00	A25	MAX-PY NODE GR_U: vz my t py
	684								1.00	A25	MIN-PY NODE GR_U: vz my t py
	685								1.00	A25	MAX-PZ NODE GR_U: mt nx m pz
	686								1.00	A25	MIN-PZ NODE GR_U: mt nx m pz
	687								1.00	A25	MAX-UX NODE GR_U: n ny ptx ux
	688								1.00	A25	MIN-UX NODE GR_U: n ny ptx ux
	689								1.00	A25	MAX-UY NODE GR_U: mz vx pty uy
	690								1.00	A25	MIN-UY NODE GR_U: mz vx pty uy
	691								1.00	A25	MAX-UZ NODE GR_U: vy vy ptz uz
	692								1.00	A25	MIN-UZ NODE GR_U: vy vy ptz uz
	693								1.00	A25	MAX-MXY QUAD GR_U: mxy
	694								1.00	A25	MIN-MXY QUAD GR_U: mxy
	695								1.00	A25	MAX-NXY QUAD GR_U: nxy
	696								1.00	A25	MIN-NXY QUAD GR_U: nxy
P	P	1.00	1.00	1.00	1.00	1.00	1.00	1.00			prestressing
	16								1.00	PERM	Prestress cables
W	Q	1.50	0.00	1.00	0.60	0.20	0.00	0.60			wind loading
	800								1.00	A10	Wind X+
	801								1.00	A10	Wind X-
	802								1.00	A10	Wind Y+ 0.4*X+
	803								1.00	A10	Wind Y+ 0.4*X-
	804								1.00	A10	Wind Y- 0.4*X+
	805								1.00	A10	Wind Y- 0.4*X-
	806								1.00	A10	Wind Z+
	807								1.00	A10	Wind Z-
	808								1.00	A10	Wind load in x+ direction for Z
	809								1.00	A10	Wind load in x- direction for Z
Act	action								Fact	factor for load case	
Part	partition of the action								Type	type of the load case	
γ -u, γ -f, γ -a	partial safety factors for unfavourable/favourable/accidental								PERM	permanent load grouped in actions	
$\psi_0,\psi_1,\psi_2,\psi_{1inf}$	combination coefficients								A	exclusive load	
LC	number of the load case										

Combination rule Number 3
SLS quasi-permanent combination
Superposition according to manual MAXIMA formula 2.7

$$E_{d,perm} = E \left\{ \sum_{j \geq 1} G_{k,j} \oplus P_k \oplus \sum_{i \geq 1} \psi_{2,i} \cdot Q_{k,i} \right\}$$

Resulting Load Cases type SLS quasi-permanent combination

Load Case selection and Actions

Superposition Factors									Fact	Type	Designation
Act	Part LC	γ_u	γ_f	γ_a	ψ_0	ψ_1	ψ_2	ψ_{1inf}			
G_1	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00			dead load g1
	1								1.00	PERM	Self-weight
G_2	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00			ADL
	2								1.00	PERM	ADL
GR_T	Q_1	1.35	0.00	1.00	0.75	0.75	0.00	0.80			gr1a LM1
	601								1.00	A42	MAX-PX NODE GR_T: my mx p px
	602								1.00	A42	MIN-PX NODE GR_T: my mx p px
	603								1.00	A42	MAX-PY NODE GR_T: vz my t py
	604								1.00	A42	MIN-PY NODE GR_T: vz my t py
	605								1.00	A42	MAX-PZ NODE GR_T: mt nx m pz
	606								1.00	A42	MIN-PZ NODE GR_T: mt nx m pz
	607								1.00	A42	MAX-UX NODE GR_T: n ny ptx ux
	608								1.00	A42	MIN-UX NODE GR_T: n ny ptx ux
	609								1.00	A42	MAX-UY NODE GR_T: mz vx pty uy
	610								1.00	A42	MIN-UY NODE GR_T: mz vx pty uy
	611								1.00	A42	MAX-UZ NODE GR_T: vy vy ptz uz
	612								1.00	A42	MIN-UZ NODE GR_T: vy vy ptz uz
	613								1.00	A42	MAX-MXY QUAD GR_T: mxy
	614								1.00	A42	MIN-MXY QUAD GR_T: mxy
	615								1.00	A42	MAX-NXY QUAD GR_T: nxy
	616								1.00	A42	MIN-NXY QUAD GR_T: nxy

Load Case selection and Actions

Load Case Selection and Actions											
		Superposition Factors									
Act	Part	γ_u	γ_f	γ_a	ψ_0	ψ_1	ψ_2	ψ_{1inf}	Fact	Type	Designation
	LC										
GR_U	Q_1	1.35	0.00	1.00	0.40	0.40	0.00	0.80			gria LM1
	681								1.00	A25	MAX-PX NODE GR_U: my mx p px
	682								1.00	A25	MIN-PX NODE GR_U: my mx p px
	683								1.00	A25	MAX-PY NODE GR_U: vz my t py
	684								1.00	A25	MIN-PY NODE GR_U: vz my t py
	685								1.00	A25	MAX-PZ NODE GR_U: mt nx m pz
	686								1.00	A25	MIN-PZ NODE GR_U: mt nx m pz
	687								1.00	A25	MAX-UX NODE GR_U: n ny ptx ux
	688								1.00	A25	MIN-UX NODE GR_U: n ny ptx ux
	689								1.00	A25	MAX-UY NODE GR_U: mz vx pty uy
	690								1.00	A25	MIN-UY NODE GR_U: mz vx pty uy
	691								1.00	A25	MAX-UZ NODE GR_U: vy vy ptz uz
	692								1.00	A25	MIN-UZ NODE GR_U: vy vy ptz uz
	693								1.00	A25	MAX-MXY QUAD GR_U: mxy
	694								1.00	A25	MIN-MXY QUAD GR_U: mxy
	695								1.00	A25	MAX-NXY QUAD GR_U: nxy
	696								1.00	A25	MIN-NXY QUAD GR_U: nxy
P	P	1.00	1.00	1.00	1.00	1.00	1.00	1.00			prestressing
	16								1.00	PERM	Prestress cables
W	Q	1.50	0.00	1.00	0.60	0.20	0.00	0.60			wind loading
	800								1.00	A10	Wind X+
	801								1.00	A10	Wind X-
	802								1.00	A10	Wind Y+ 0.4*X+
	803								1.00	A10	Wind Y+ 0.4*X-
	804								1.00	A10	Wind Y- 0.4*X+
	805								1.00	A10	Wind Y- 0.4*X-
	806								1.00	A10	Wind Z+
	807								1.00	A10	Wind Z-
	808								1.00	A10	Wind load in x+ direction for Z
	809								1.00	A10	Wind load in x- direction for Z
Act	action								Fact	factor for load case	
Part	partition of the action								Type	type of the load case	
$\gamma_u, \gamma_f, \gamma_a$	partial safety factors for unfavourable/favourable/accidental								PERM	permanent load grouped in actions	
$\psi_0, \psi_1, \psi_2, \psi_{1inf}$	combination coefficients								A	exclusive load	
LC	number of the load case										

Composite model v2
Superpositions

Combination rule Number 4

ULS fundamental combination

Superposition according to manual MAXIMA formula 2.1

$$E_d = E \left\{ \sum_{j \geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus \gamma_P \cdot P_k \oplus \gamma_{Q,1} \cdot Q_{k,1} \oplus \sum_{i > 1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i} \right\}$$

Resulting Load Cases type ULS fundamental combination

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		γ_u	γ_f	γ_a	ψ_0	ψ_1	ψ_2	ψ_{1inf}			
G_1	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00	1.00	PERM	dead load g1
	1										Self-weight
G_2	G	1.35	1.00	1.00	1.00	1.00	1.00	1.00	1.00	PERM	ADL
	2										ADL
GR_T	Q_1	1.35	0.00	1.00	0.75	0.75	0.00	0.80			gr1a LM1
	601								1.00	A42	MAX-PX NODE GR_T: my mx p px
	602								1.00	A42	MIN-PX NODE GR_T: my mx p px
	603								1.00	A42	MAX-PY NODE GR_T: vz my t py
	604								1.00	A42	MIN-PY NODE GR_T: vz my t py
	605								1.00	A42	MAX-PZ NODE GR_T: mt nx m pz
	606								1.00	A42	MIN-PZ NODE GR_T: mt nx m pz
	607								1.00	A42	MAX-UX NODE GR_T: n ny ptx ux
	608								1.00	A42	MIN-UX NODE GR_T: n ny ptx ux
	609								1.00	A42	MAX-UY NODE GR_T: mz vx pty uy
	610								1.00	A42	MIN-UY NODE GR_T: mz vx pty uy
	611								1.00	A42	MAX-UZ NODE GR_T: vy vy ptz uz
	612								1.00	A42	MIN-UZ NODE GR_T: vy vy ptz uz
	613								1.00	A42	MAX-MXY QUAD GR_T: mxy
	614								1.00	A42	MIN-MXY QUAD GR_T: mxy
	615								1.00	A42	MAX-NXY QUAD GR_T: nxy
	616								1.00	A42	MIN-NXY QUAD GR_T: nxy

Composite model v2
Superpositions

Load Case selection and Actions

Act	Part LC	Superposition Factors							Fact	Type	Designation
		γ_u	γ_f	γ_a	ψ_0	ψ_1	ψ_2	ψ_{1inf}			
GR_U	Q_1	1.35	0.00	1.00	0.40	0.40	0.00	0.80			gr1a LM1
	681								1.00	A25	MAX-PX NODE GR_U: my mx p px
	682								1.00	A25	MIN-PX NODE GR_U: my mx p px
	683								1.00	A25	MAX-PY NODE GR_U: vz myy t py
	684								1.00	A25	MIN-PY NODE GR_U: vz myy t py
	685								1.00	A25	MAX-PZ NODE GR_U: mt nxx m pz
	686								1.00	A25	MIN-PZ NODE GR_U: mt nxx m pz
	687								1.00	A25	MAX-UX NODE GR_U: n nyy ptx ux
	688								1.00	A25	MIN-UX NODE GR_U: n nyy ptx ux
	689								1.00	A25	MAX-UY NODE GR_U: mz vx pty uy
	690								1.00	A25	MIN-UY NODE GR_U: mz vx pty uy
	691								1.00	A25	MAX-UZ NODE GR_U: vy vy ptz uz
	692								1.00	A25	MIN-UZ NODE GR_U: vy vy ptz uz
	693								1.00	A25	MAX-MXY QUAD GR_U: mxy
	694								1.00	A25	MIN-MXY QUAD GR_U: mxy
	695								1.00	A25	MAX-NXY QUAD GR_U: nxy
	696								1.00	A25	MIN-NXY QUAD GR_U: nxy
P	P	1.00	1.00	1.00	1.00	1.00	1.00	1.00			prestressing
	16								1.00	PERM	Prestress cables
W	Q	1.50	0.00	1.00	0.60	0.20	0.00	0.60			wind loading
	800								1.00	A10	Wind X+
	801								1.00	A10	Wind X-
	802								1.00	A10	Wind Y+ 0.4*X+
	803								1.00	A10	Wind Y+ 0.4*X-
	804								1.00	A10	Wind Y- 0.4*X+
	805								1.00	A10	Wind Y- 0.4*X-
	806								1.00	A10	Wind Z+
	807								1.00	A10	Wind Z-
	808								1.00	A10	Wind load in x+ direction for Z
	809								1.00	A10	Wind load in x- direction for Z
Act	action								Fact	factor for load case	
Part	partition of the action								Type	type of the load case	
$\gamma_u, \gamma_f, \gamma_a$	partial safety factors for unfavourable/favourable/accidental								PERM	permanent load grouped in actions	
$\psi_0, \psi_1, \psi_2, \psi_{1inf}$	combination coefficients								A	exclusive load	
LC	number of the load case										

Composite model v2
Steel - Resistance of Cross Sections

Default design code is EuroNorm 1994-2:2005 Composite Structures (Norge) V 2018
Structure: B (Road bridges)

Materials

Mat	Classification
1	C 40/50 (EN 1992)
2	Y 1860 A (EN 1992)
4	B 500 B (EN 1992)
5	S 355 (EN 1993)

Selected Beam Elements

Selection	NoA	NoE	x[m]	Type
Grp 5	50001			
Grp 6	60001			
Grp 8	80001			
Grp 11	110001			
NoA, NoE range of element numbers				
x[m] x-ordinate of beam section or station on axis				
Type element type				

Reinforcement will be accounted for sectional values as defined in AQUA
Reinforcements saved as Design case No. 1

Considered Load Cases

LC	ACT	REF	CS	Designation	$\gamma-u$	$\gamma-f$	ψ_0	ψ_1	ψ_2	ψ_{1inf}	SUP
2101	(D)			MAX-PX NODE							
2102	(D)			MIN-PX NODE							
2103	(D)			MAX-PY NODE							
2104	(D)			MIN-PY NODE							
2105	(D)			MAX-PZ NODE							
2106	(D)			MIN-PZ NODE							
2107	(D)			MAX-UX NODE design N nyy							
2108	(D)			MIN-UX NODE design N nyy							
2109	(D)			MAX-UY NODE design MZ vx							
2110	(D)			MIN-UY NODE design MZ vx							
2111	(D)			MAX-UZ NODE design VY vy							
2112	(D)			MIN-UZ NODE design VY vy							
LC load case					CS section the load case is acting on						
ACT action					SUP action type, group and superposition category						
REF reference point for forces and moments											

Composite model v2
Steel - Resistance of Cross Sections

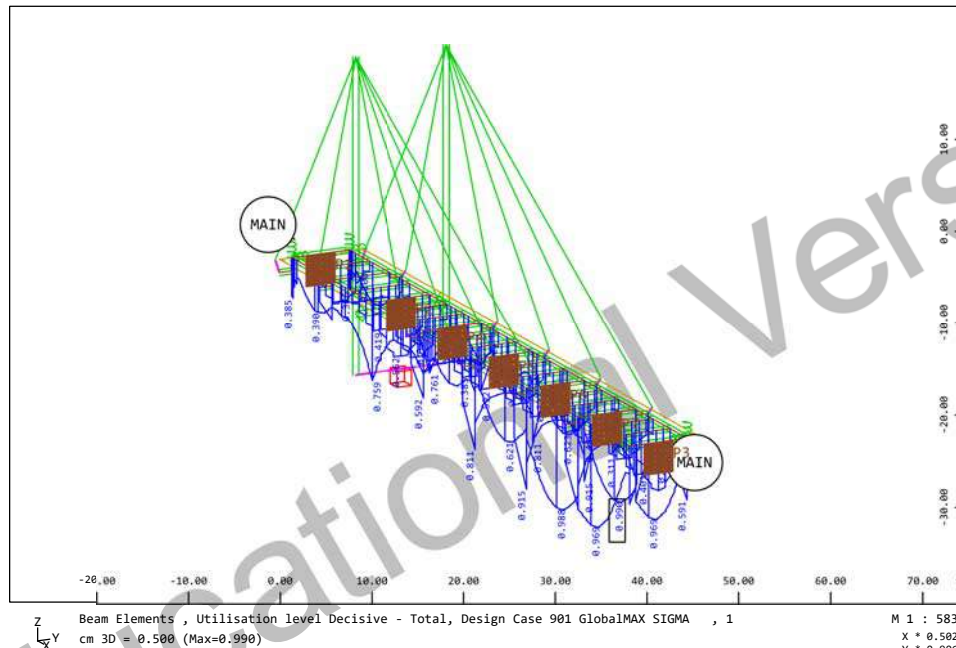
Maximum Utilisation Level

	N	Vy	Vz	My	Mz	Mtp	Mts	Mb	Ncr	SCL	Total
	$\sigma-x$	$\sigma+x$	τ	$\sigma-v$	$\sigma-s$	$\sigma-dyn$	As-l	As-v	crack	c/t	
Section 9	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	(3)	0.591
IPE 300	0.591	0.562	0.162	0.591	-	-	-	-	-	0.277	
Section 10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	(3) ¹	0.990
Steel girder	0.759	0.989	0.264	0.990	-	-	-	-	-	0.501	
Section 12	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	(1)	0.722
Cross girders for cabl	0.499	0.714	0.370	0.722	-	-	-	-	-	0.191	
Section 14	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	(1)	0.915
Cross girders for cabl	0.909	0.909	0.397	0.915	-	-	-	-	-	0.327	
Total	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	(3) ¹	0.990
	0.909	0.989	0.397	0.990	-	-	-	-	-	0.501	

¹ Section of class 4 has small stresses allowing to be treated as class 3 (see EN 1993-1-1 5.5.2 (9))

N	normal force	τ	shear stress
Vy, Vz	shear force	$\sigma-v$	von Mises stress
My, Mz	bending	$\sigma-s$	stress in reinforcements
Mtp, Mts	torsion (p)primary and (s)econdary	$\sigma-dyn$	stress range
Mb	warping moment	As-l	longitudinal reinforcements
Ncr	flexural buckling	As-v	transverse reinforcements
SCL	cross-section class	crack	crack width
$\sigma-x$	longitud. compressive stress	c/t	stress dependant utilisation level (see AQB Manual 2.3.2)
$\sigma+x$	longitud. tensile stress	Total	most unfavorable utilisation for all checks

Composite model v2
Graphical Output



Composite model v2
ULS design beams

Default design code is EuroNorm 1994-2:2005 Composite Structures (Norge) V 2018
Structure: B (Road bridges)

Materials

Mat	Classification
1	C 40/50 (EN 1992)
2	Y 1860 A (EN 1992)
4	B 500 B (EN 1992)
5	S 355 (EN 1993)

Selected Beam Elements

Selection	NoA	NoE	x[m]	Type
Grp 1	10001			
Grp 4	40001			
Grp 11	110001			

NoA, NoE range of element numbers
x[m] x-ordinate of beam section or station on axis
Type element type

Reinforcement will be accounted for sectional values as defined in AQUA
Reinforcements saved as Design case No. 1

Considered Load Cases

LC	ACT	REF	CS	Designation
2101	(D)			MAX-PX NODE
2102	(D)			MIN-PX NODE
2103	(D)			MAX-PY NODE
2104	(D)			MIN-PY NODE
2105	(D)			MAX-PZ NODE
2106	(D)			MIN-PZ NODE
2107	(D)			MAX-UX NODE design N nyy U
2108	(D)			MIN-UX NODE design N nyy U
2109	(D)			MAX-UY NODE design MZ vx U
2110	(D)			MIN-UY NODE design MZ vx U
2111	(D)			MAX-UZ NODE design VY vy U
2112	(D)			MIN-UZ NODE design VY vy U

LC load case REF reference point for forces and moments
ACT action CS section the load case is acting on

Composite model v2
ULS design beams

Longitudinal Reinforcements - Design case No. 1

Beam	x[m]	SNo	p [o/o]	Asl [cm2]	vm [m]	Asl-0 [cm2]	Asl-1 [cm2]	Asl-2 [cm2]	Asl-3 [cm2]	Asl-4 [cm2]	Asl-5 [cm2]
10001	0.000	3	2.39	188.29	1.561		29.75T	29.75T	64.32T	64.48T	
10017	0.000	3	2.39	188.47	1.237		29.70T	29.70T	64.52T	64.54T	
40001	0.000	5	1.44	174.59			136.48	28.23	9.88		
110001	0.000	16	4.10	922.74	1.557		539.87T	368.92T	13.95T		
110004	0.000	17	5.69	1280.55	1.711		967.51T	166.32T	146.72T		
110005	0.000	17	5.69	1280.57	1.557		967.52T	166.34T	146.71T		

Note: Layer includes reinforcements for torsion if followed by T

Note: Layer has only compression reinforcements if followed by a quote

p geometric part of reinforcements
Asl total longitudinal reinforcement
vm shift rule of longitudinal reinforcement (0.0 if already included by normal force)
Asl-0,Asl-1,Asl-2,Asl-3,Asl-4,Asl-5 longitudinal reinforcement per layer

Shear Reinforcements per Cutted Part of Section - Design case No. 1

Beam	x[m]	SNo	Asl-Mt [cm2/m]	As/s [cm2/m]	As/s-1 [cm2/m]
10001	0.000	3	0.65		2.53
10002	0.000	3	0.65		2.53
10003	0.000	3	0.65		2.53
10004	0.000	3	0.65		2.53
10005	0.000	3	0.65		2.53
10006	0.000	3	0.65		2.53
10007	0.000	3	0.65		2.53
10008	0.000	3	0.65		2.53
10009	0.000	3	0.65		2.53
10010	0.000	3	0.65		2.53
10011	0.000	3	0.65		2.53
10012	0.000	3	0.65		2.53
10013	0.000	3	0.65		2.53
10014	0.000	3	0.65		2.53
10015	0.000	3	0.65		2.53
10016	0.000	3	0.65		2.53
10017	0.000	3	0.65		2.53
10018	0.000	3	0.65		2.53
10019	0.000	3	0.65		2.53
10020	0.000	3	0.65		2.53
10021	0.000	3	0.65		2.53
10022	0.000	3	0.65		2.53

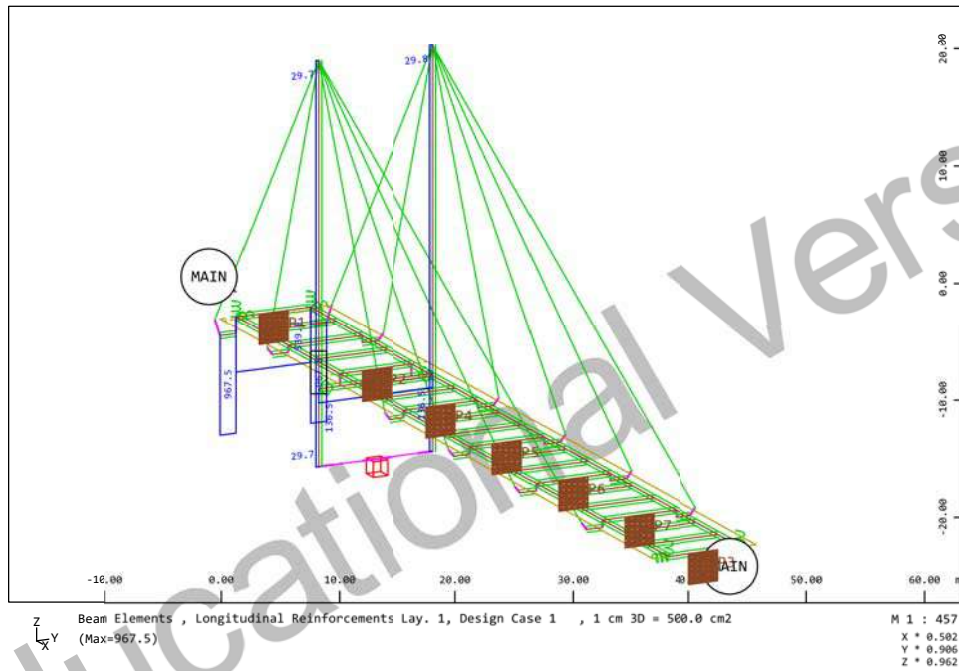
Composite model v2
ULS design beams

Shear Reinforcements per Cutted Part of Section - Design case No. 1

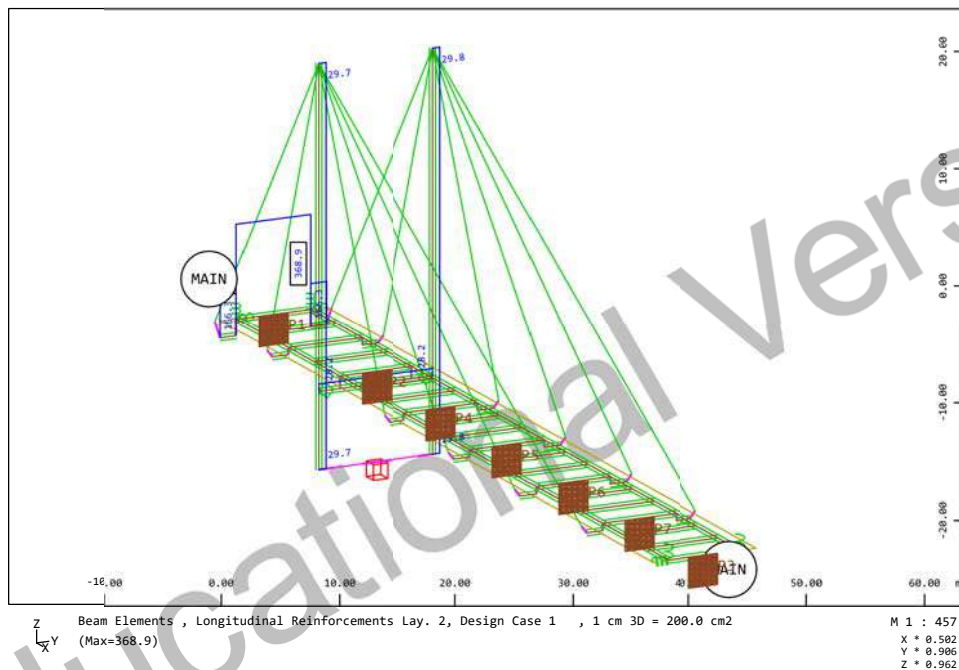
Beam	x[m]	SNo	Asl-Mt [cm2/m]	As/s [cm2/m]	As/s-1 [cm2/m]
10023	0.000	3	0.65		2.53
10024	0.000	3	0.65		2.53
10025	0.000	3	0.65		2.53
10026	0.000	3	0.65		2.53
10027	0.000	3	0.65		2.53
10028	0.000	3	0.65		2.53
10029	0.000	3	0.65		2.53
10030	0.000	3	0.65		2.53
10031	0.000	3	0.65		2.53
10032	0.000	3	0.65		2.53
40001	0.000	5	2.65		28.27
40001	2.000	5	2.65		27.80
40002	0.000	5	2.65		11.13
40002	2.333	5	2.65		11.13
40003	0.000	5	2.65		11.13
40003	2.333	5	2.65		11.13
40004	0.000	5	2.65		11.13
40004	2.333	5	2.65		11.13
40005	0.000	5	2.65		27.80
40005	2.000	5	2.65		29.09
110001	0.000	16	3.54		15.18
110001	2.333	16	3.54		15.18
110002	0.000	16	3.54		15.18
110002	2.333	16	3.54		15.18
110003	0.000	16	3.54		15.18
110003	2.333	16	3.54		15.18
110004	0.000	17	50.71		illegal
110004	1.500	17	50.71		illegal
110005	0.000	17	50.70		illegal
110005	1.500	17	50.70		illegal

Asl-Mt nominal longitudinal reinforcement per circumference of
equivalent section due to torsion
As/s area of transverse reinforcements
As/s-1 total transverse reinforcement per layer and cutted element

Composite model v2
Graphical Output



Composite model v2
Graphical Output



Composite model v2
SLS design beams

Default design code is EuroNorm 1994-2:2005 Composite Structures (Norge) V 2018
Structure: B (Road bridges)

Materials

Mat	Classification
1	C 40/50 (EN 1992)
2	Y 1860 A (EN 1992)
4	B 500 B (EN 1992)
5	S 355 (EN 1993)

Selected Beam Elements

Selection	NoA	NoE	x[m]	Type
Grp 1	10001			
Grp 4	40001			
Grp 11	110001			
NoA, NoE range of element numbers				
x[m] x-ordinate of beam section or station on axis				
Type element type				

Reinforcement will be accounted for sectional values as defined in AQUA
Reinforcements saved as Design case No. 2
Reinforcements superposed with existing Design case No. 1

Considered Load Cases

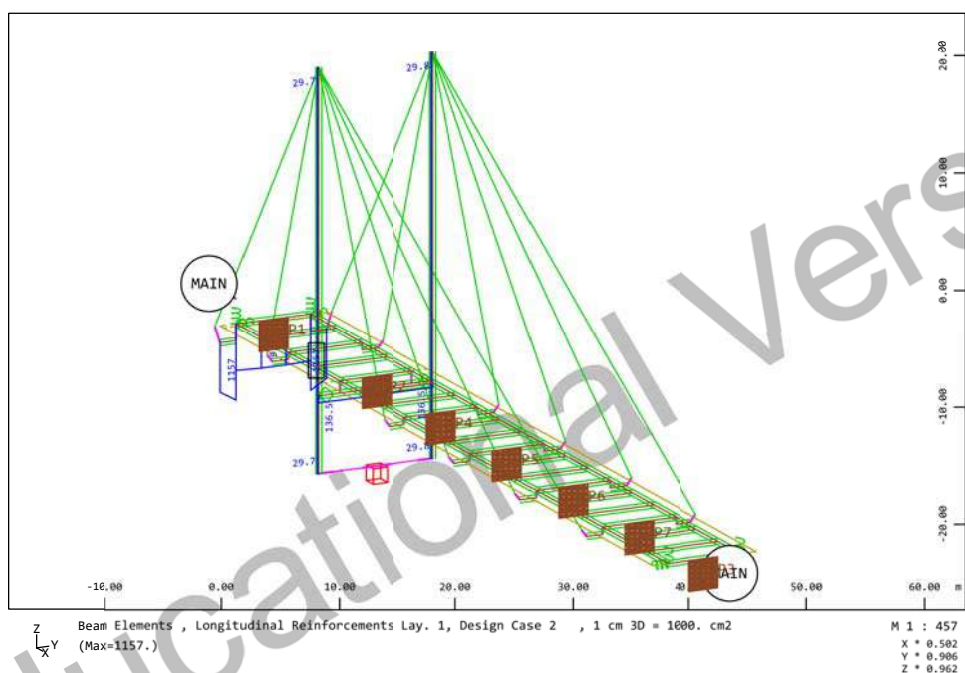
LC	ACT	REF	CS	Designation
1401	(P)			MAXP-PX NODE
1402	(P)			MINP-PX NODE
1403	(P)			MAXP-PY NODE
1404	(P)			MINP-PY NODE
1405	(P)			MAXP-PZ NODE
1406	(P)			MINP-PZ NODE
1407	(P)			MAXP-UX NODE perm N nyy UX
1408	(P)			MINP-UX NODE perm N nyy UX
1409	(P)			MAXP-UY NODE perm MZ vx UY
1410	(P)			MINP-UY NODE perm MZ vx UY
1411	(P)			MAXP-UZ NODE perm VY vy UZ
1412	(P)			MINP-UZ NODE perm VY vy UZ
LC load case				
REF reference point for forces and moments				
ACT action				
CS section the load case is acting on				

Composite model v2
SLS design beams

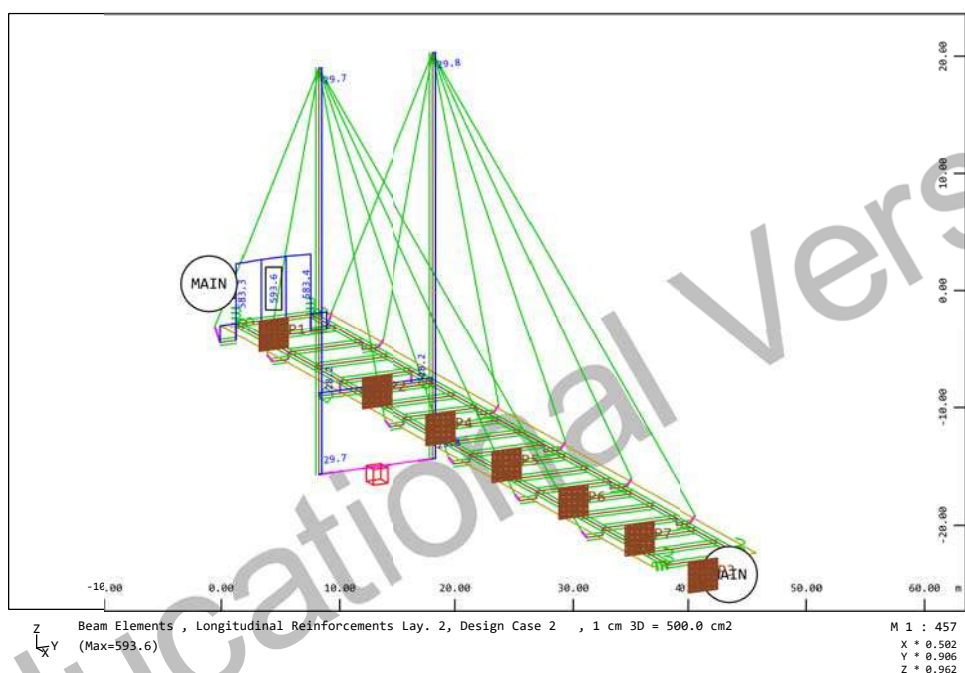
Maximum Utilisation Level

	N	Vy	Vz	My	Mz	Mtp	Mts	Mb	Ncr	SCL	Total
	$\sigma-x$	$\sigma+x$	τ	$\sigma-v$	$\sigma-s$	$\sigma-dyn$	As-l	As-v	crack	c/t	
Section 5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	-	0.486
B/H = 1100 / 1100 mm	0.000	0.000	0.000	0.000	-	-	-	-	0.486	-	-
Section 16	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	-	1.000
B/H = 1500 / 1500 mm	0.000	0.000	0.000	0.000	-	-	-	-	1.000	-	-
Section 17	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	-	1.000
B/H = 1500 / 1500 mm	0.000	0.000	0.000	0.000	-	-	-	-	1.000	-	-
Total	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-	-	1.000
	0.000	0.000	0.000	0.000	-	-	-	-	1.000	-	-
N	normal force			τ	shear stress						
Vy, Vz	shear force			$\sigma-v$	von Mises stress						
My, Mz	bending			$\sigma-s$	stress in reinforcements						
Mtp, Mts	torsion (p)primary and (s)econdary			$\sigma-dyn$	stress range						
Mb	warping moment			As-l	longitudinal reinforcements						
Ncr	flexural buckling			As-v	transverse reinforcements						
SCL	cross-section class			crack	crack width						
$\sigma-x$	longitud. compressive stress			c/t	stress dependant utilisation level (see AQB Manual 2.3.2)						
$\sigma+x$	longitud. tensile stress			Total	most unfavorable utilisation for all checks						

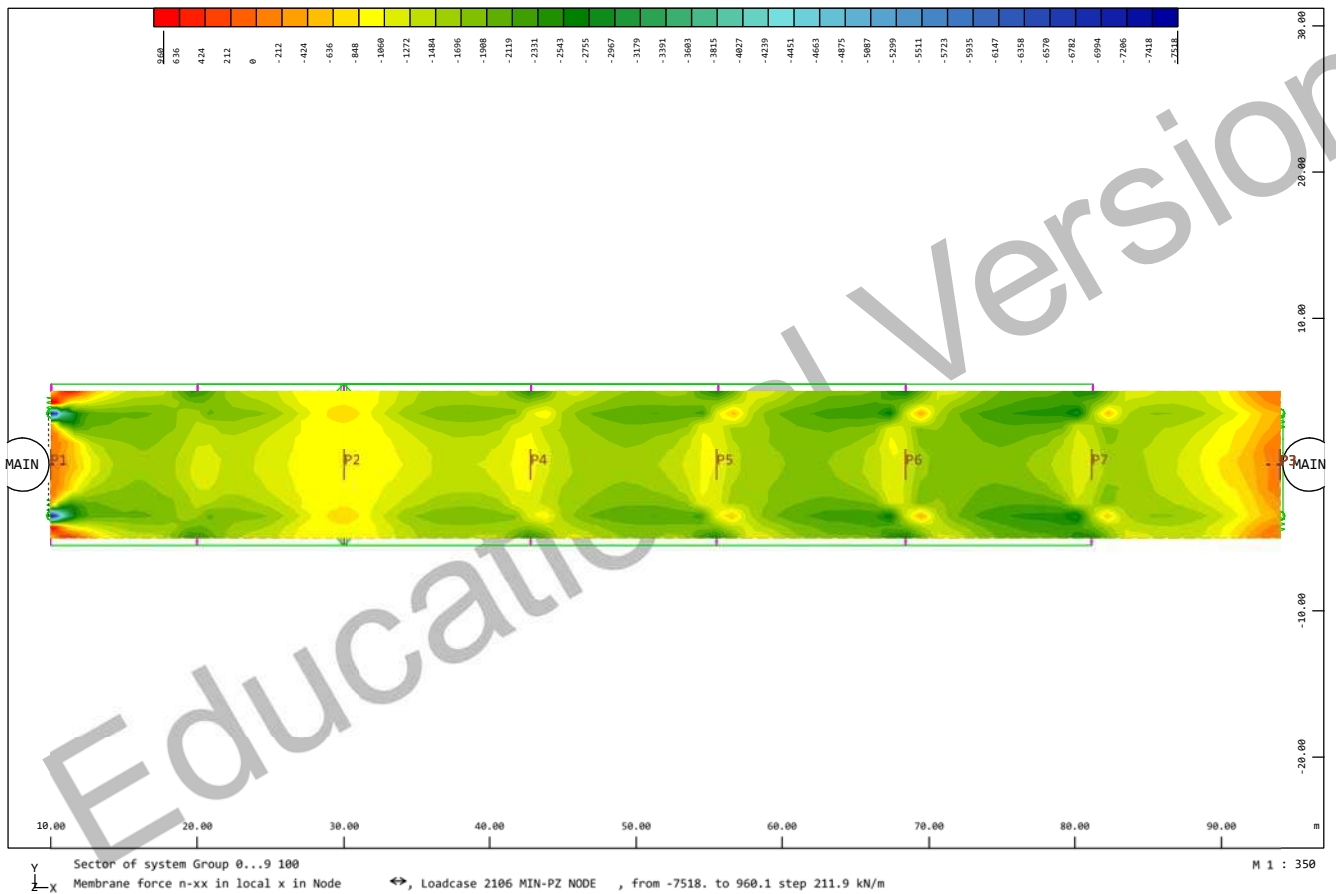
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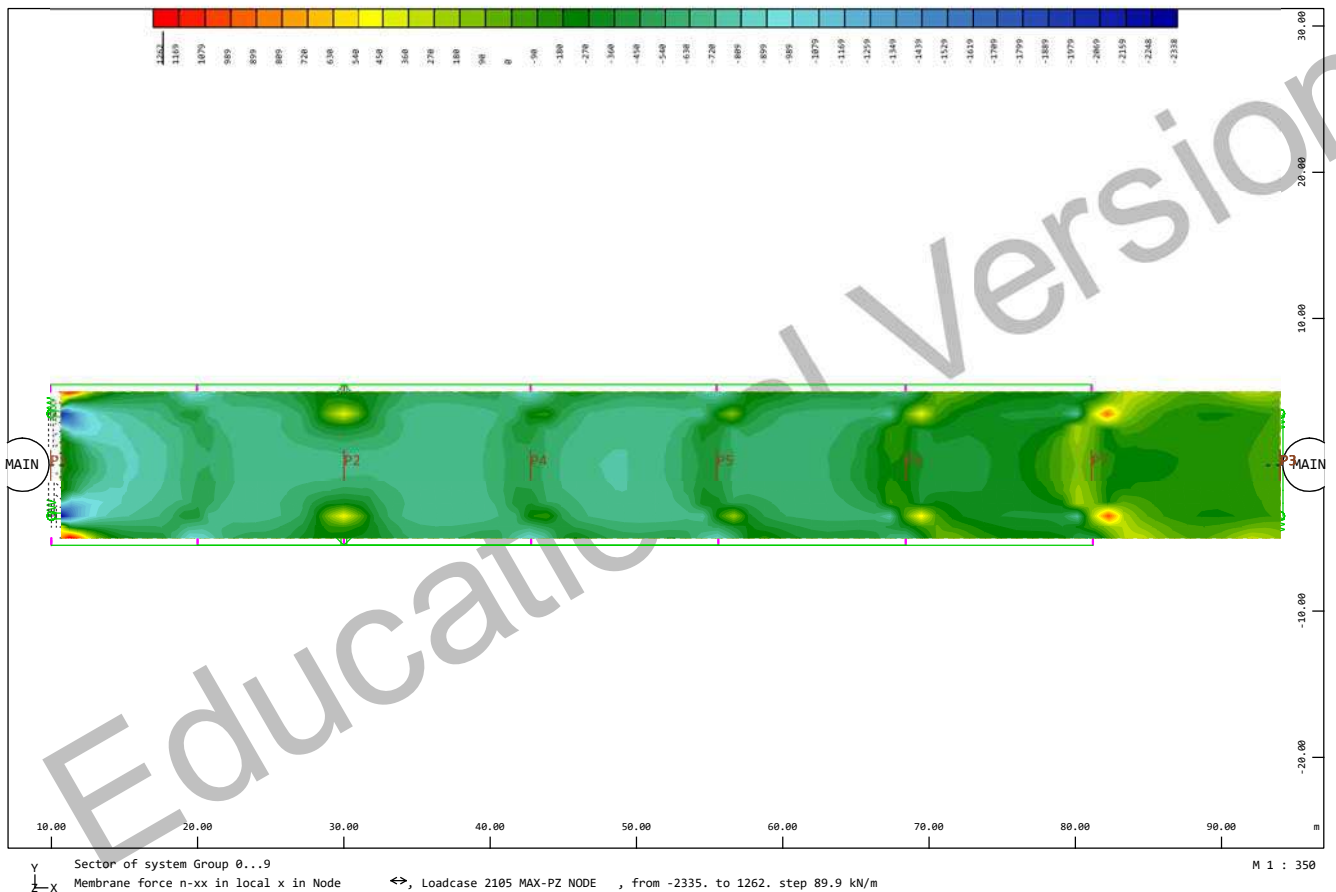
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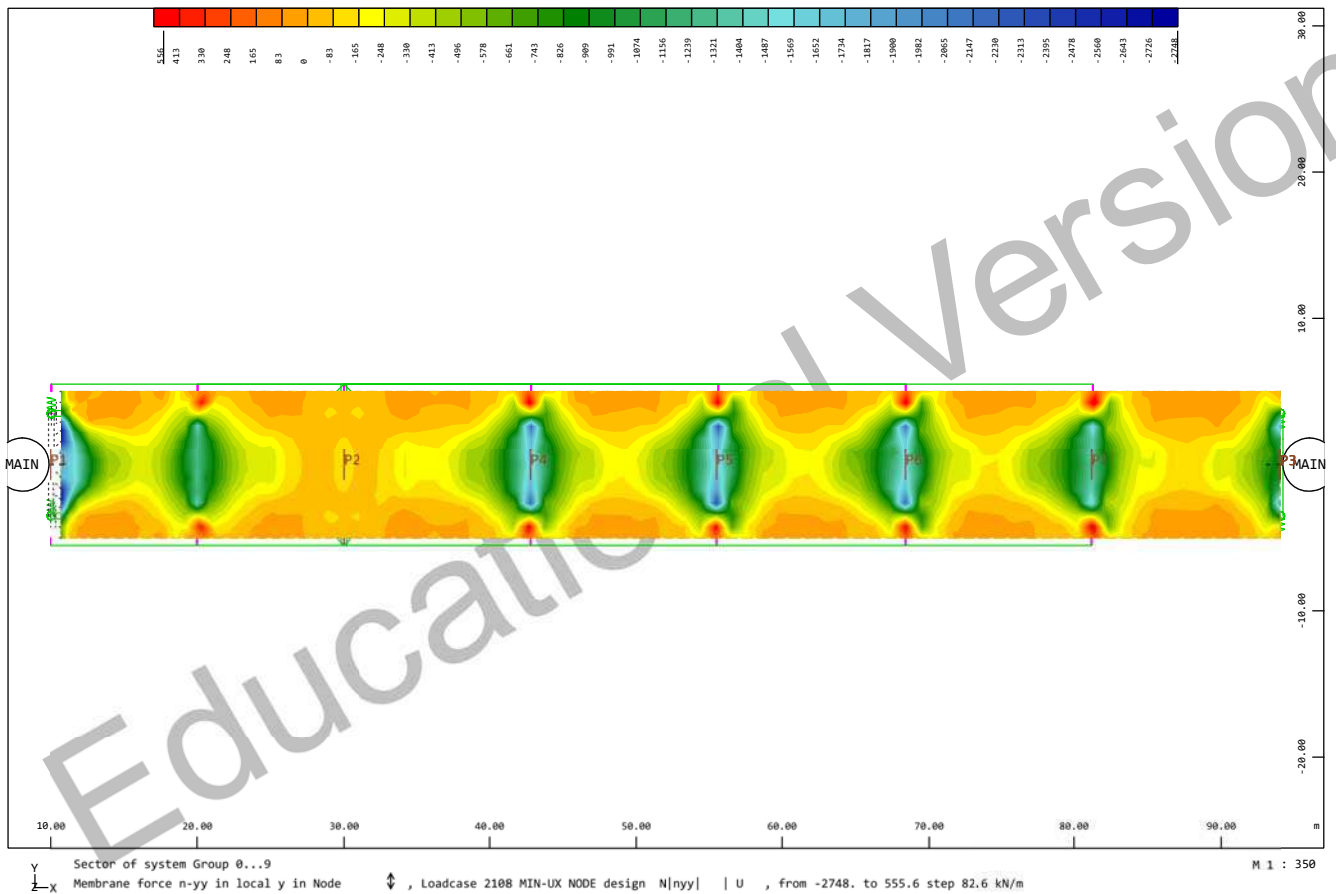
Composite model v2



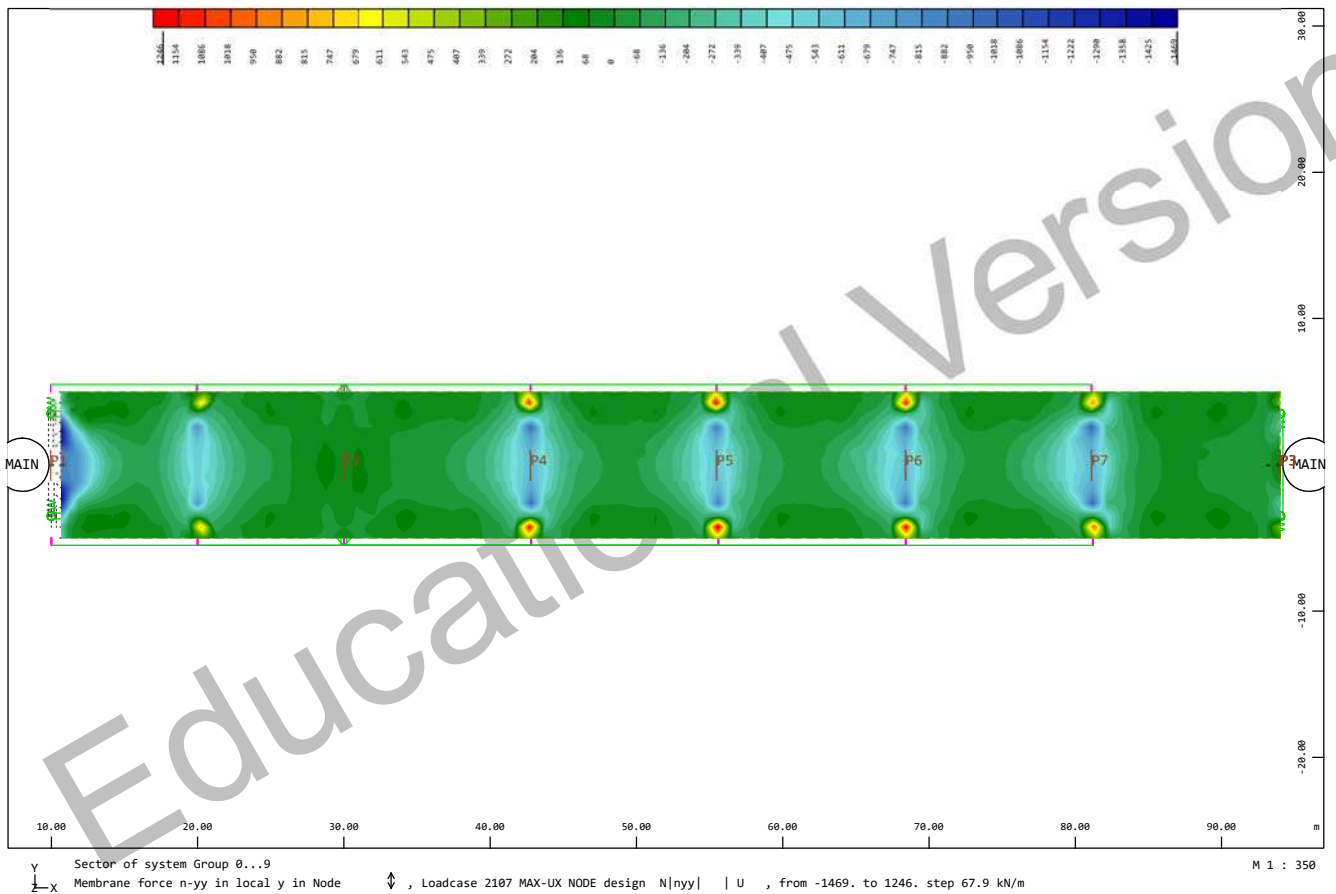
Composite model v2



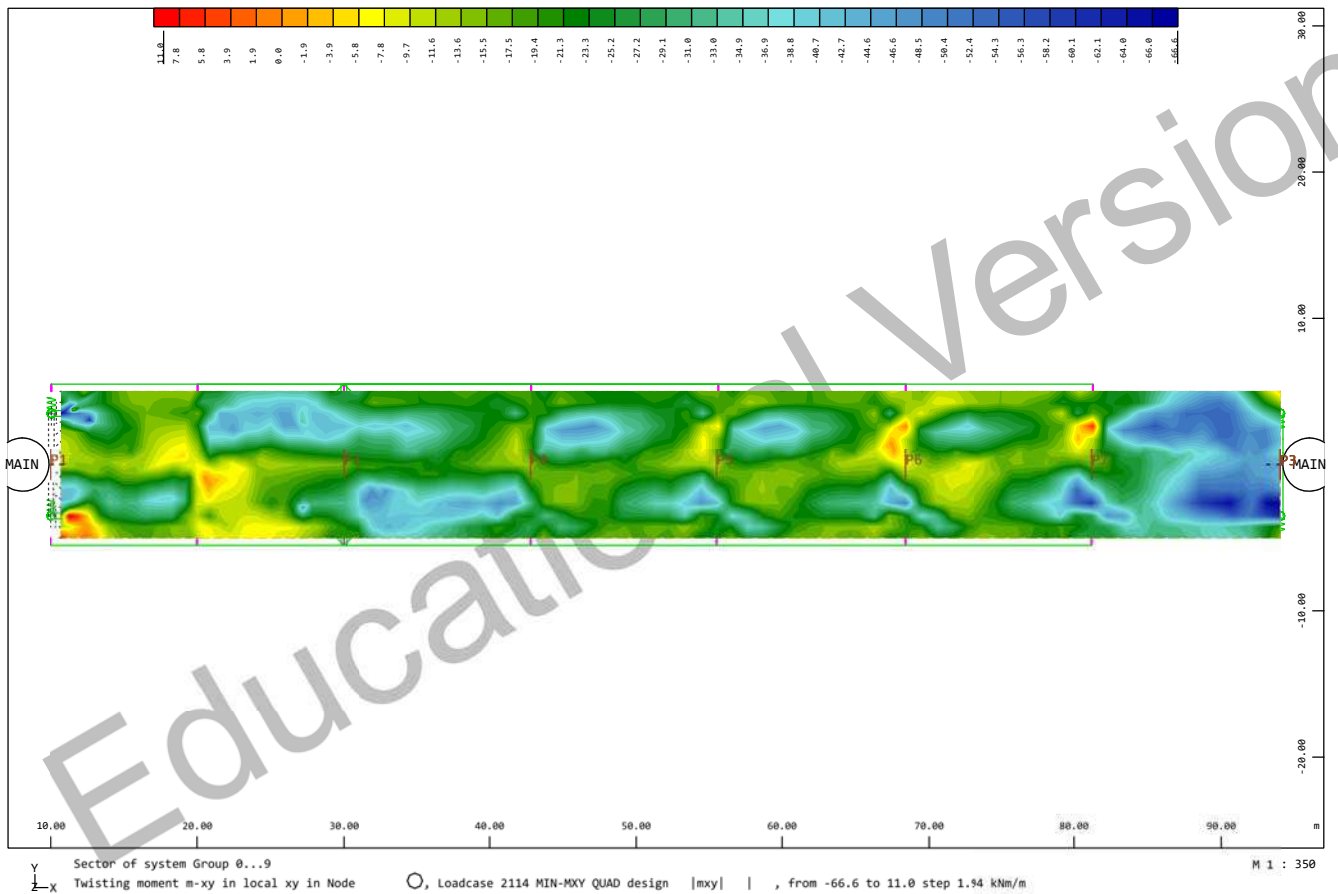
Composite model v2



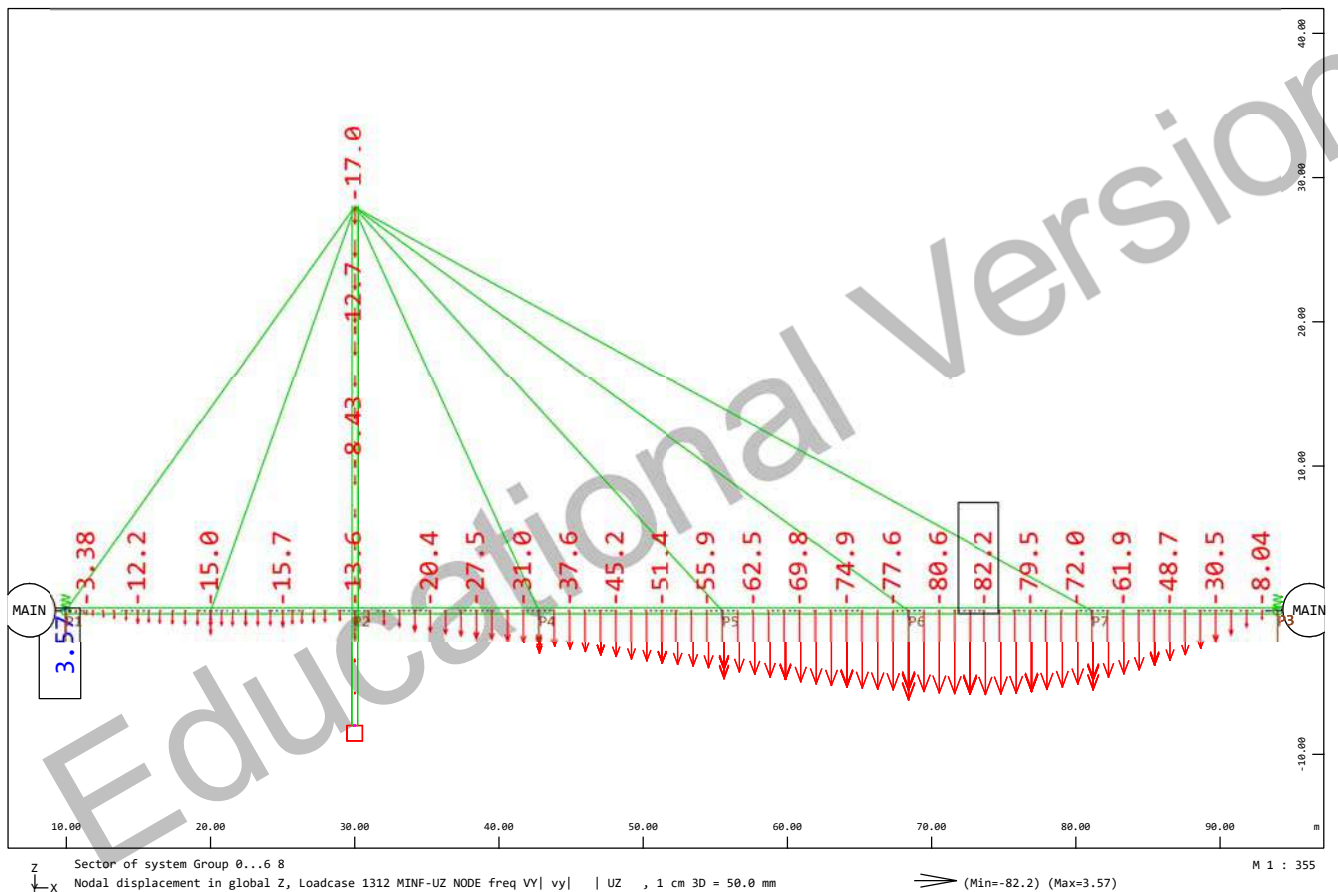
Composite model v2



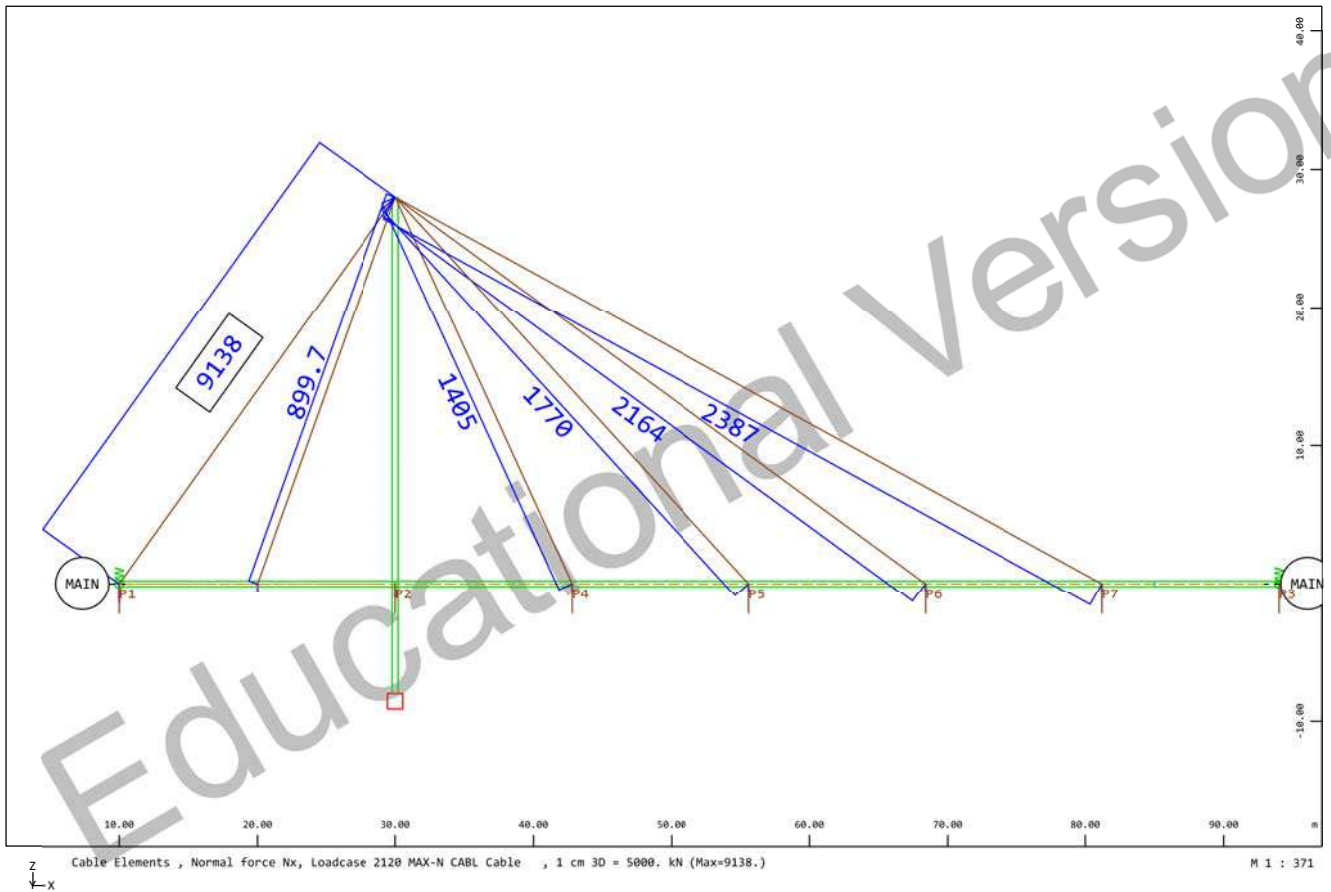
Composite model v2



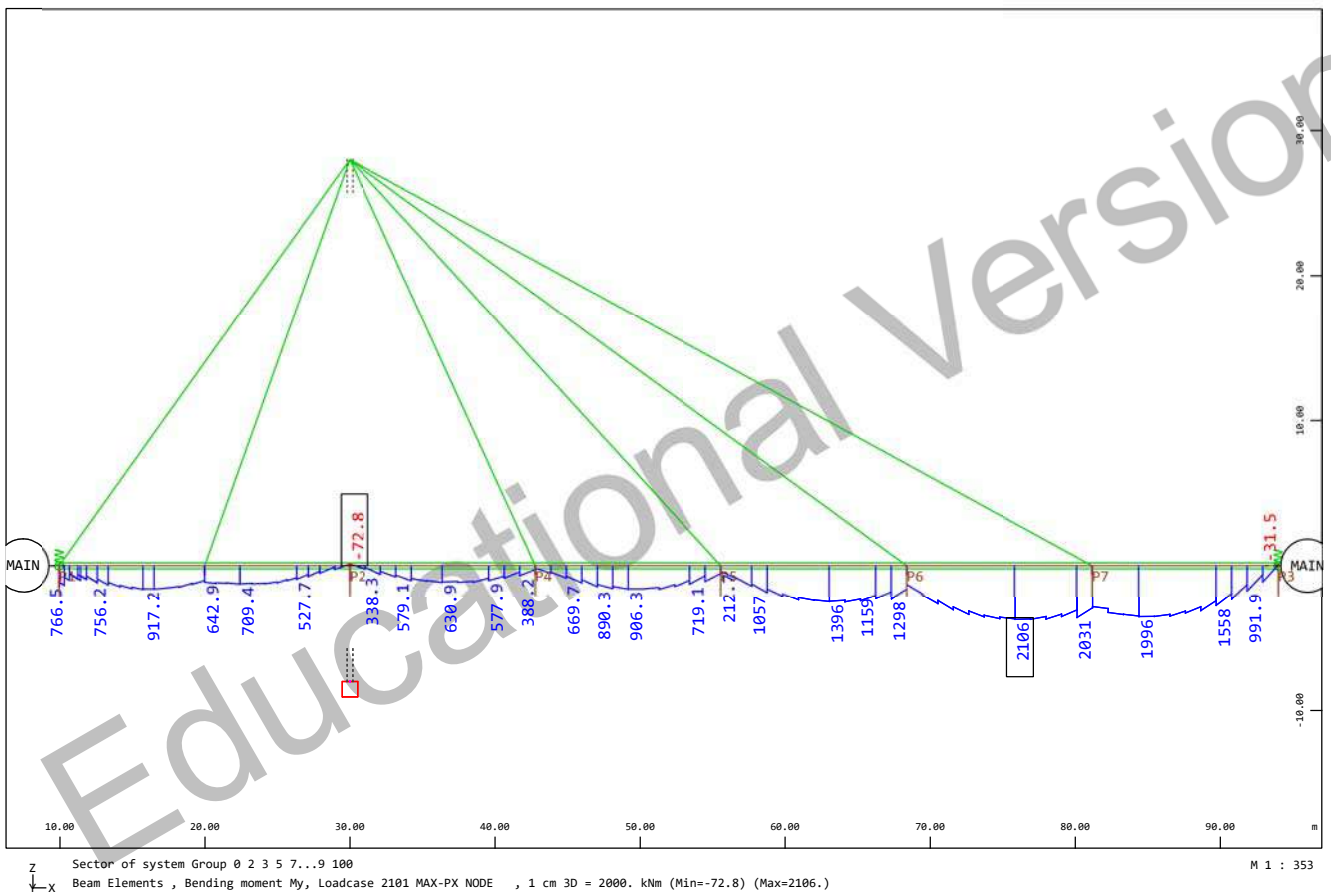
Composite model v2



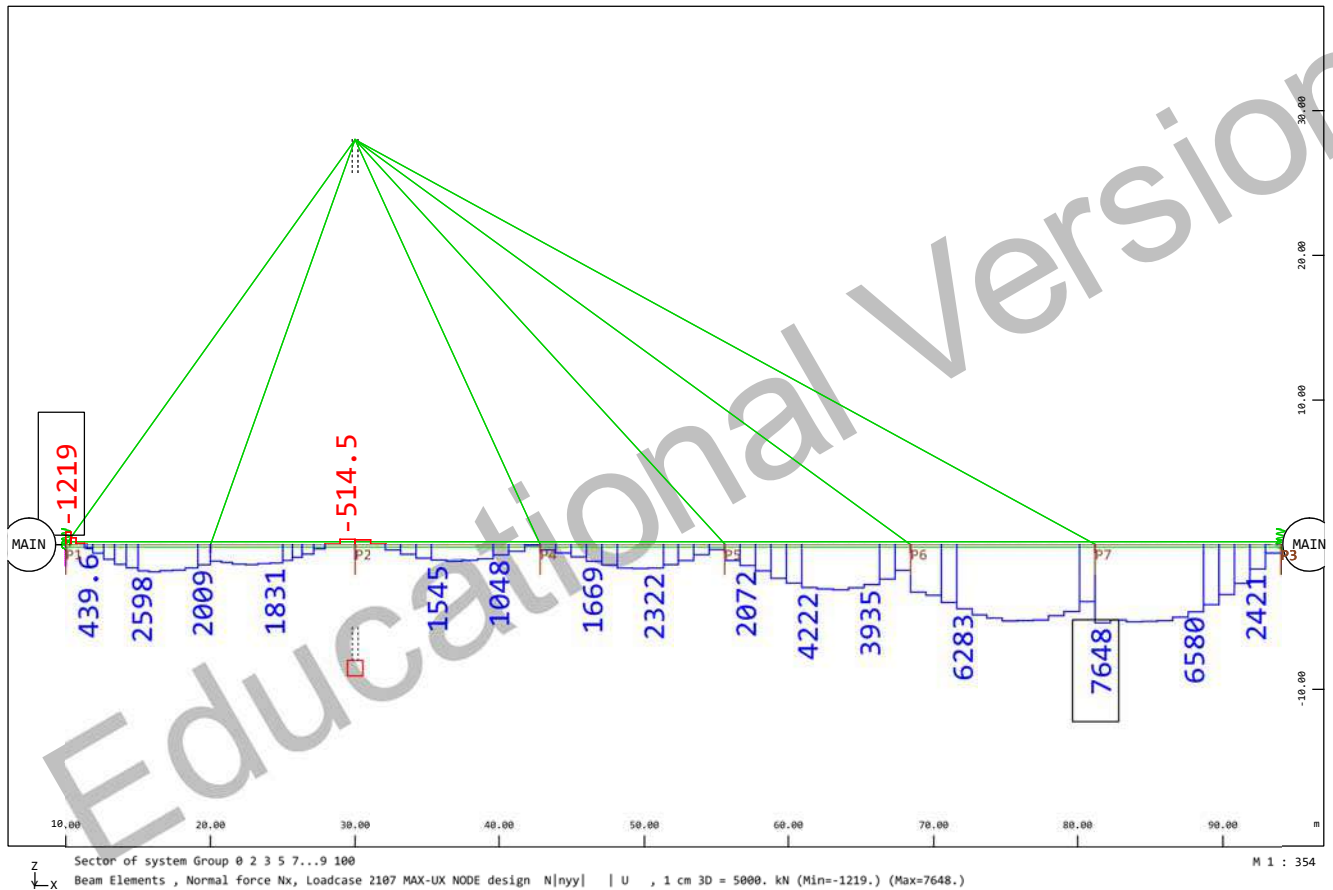
Composite model v2



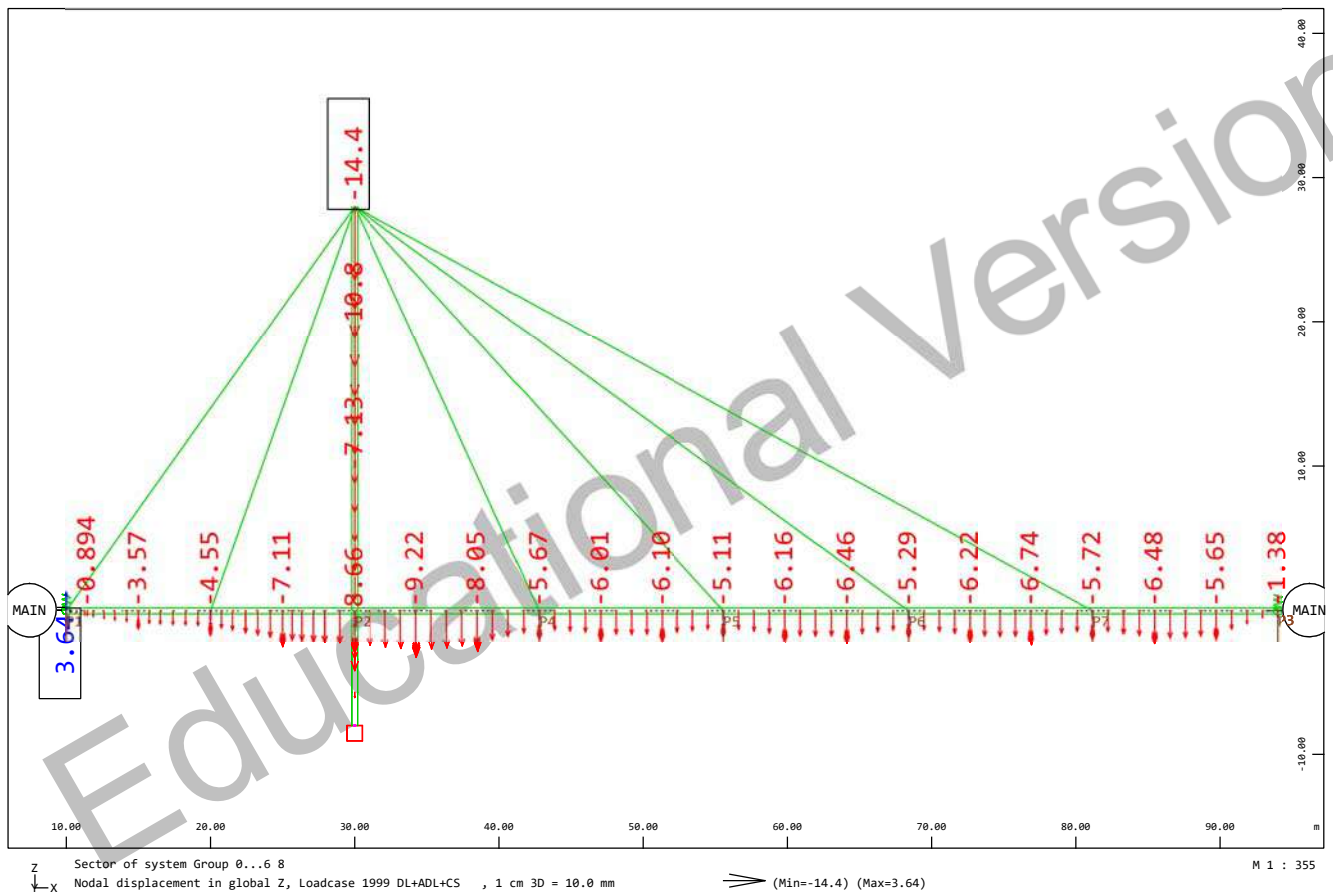
Composite model v2



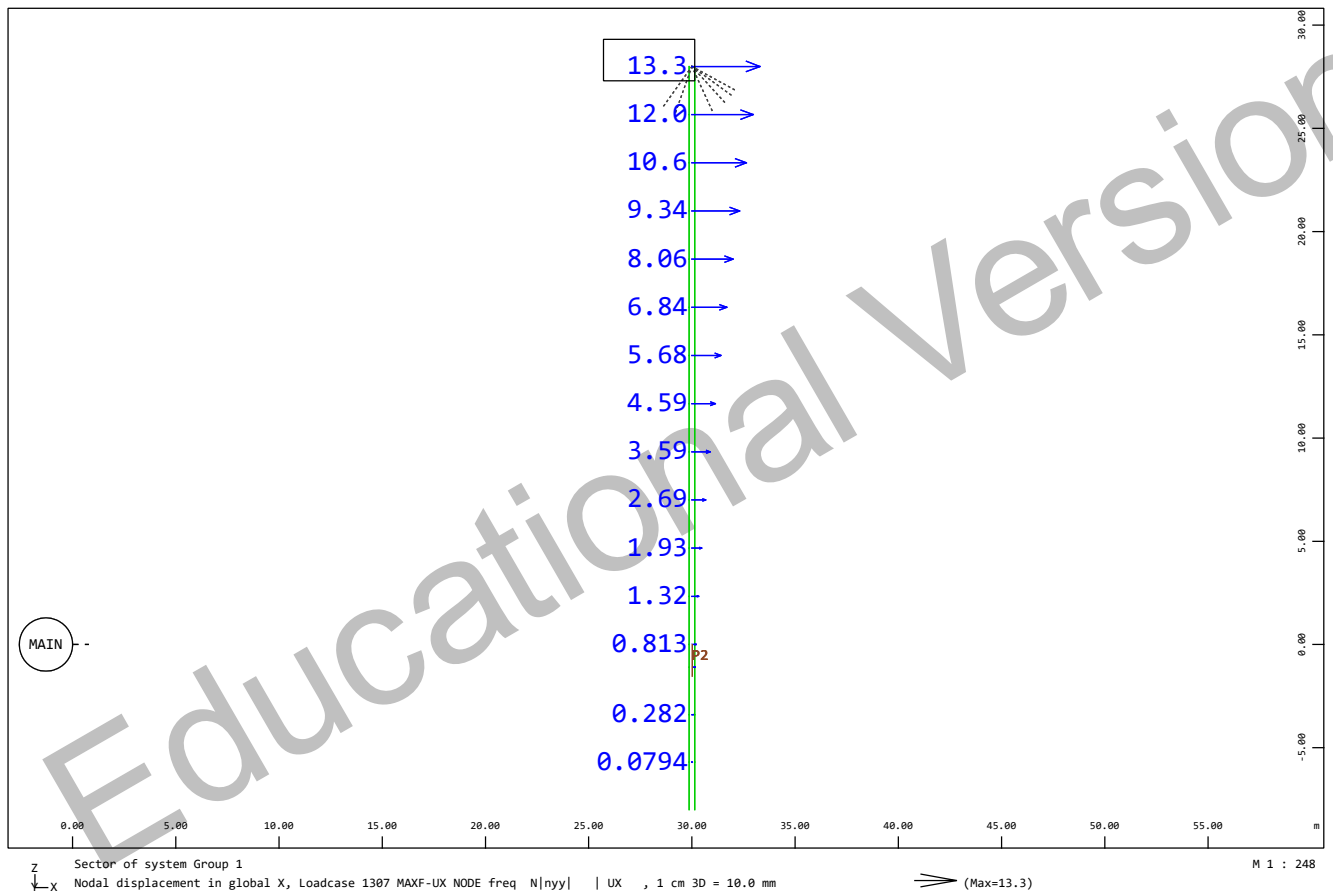
Composite model v2



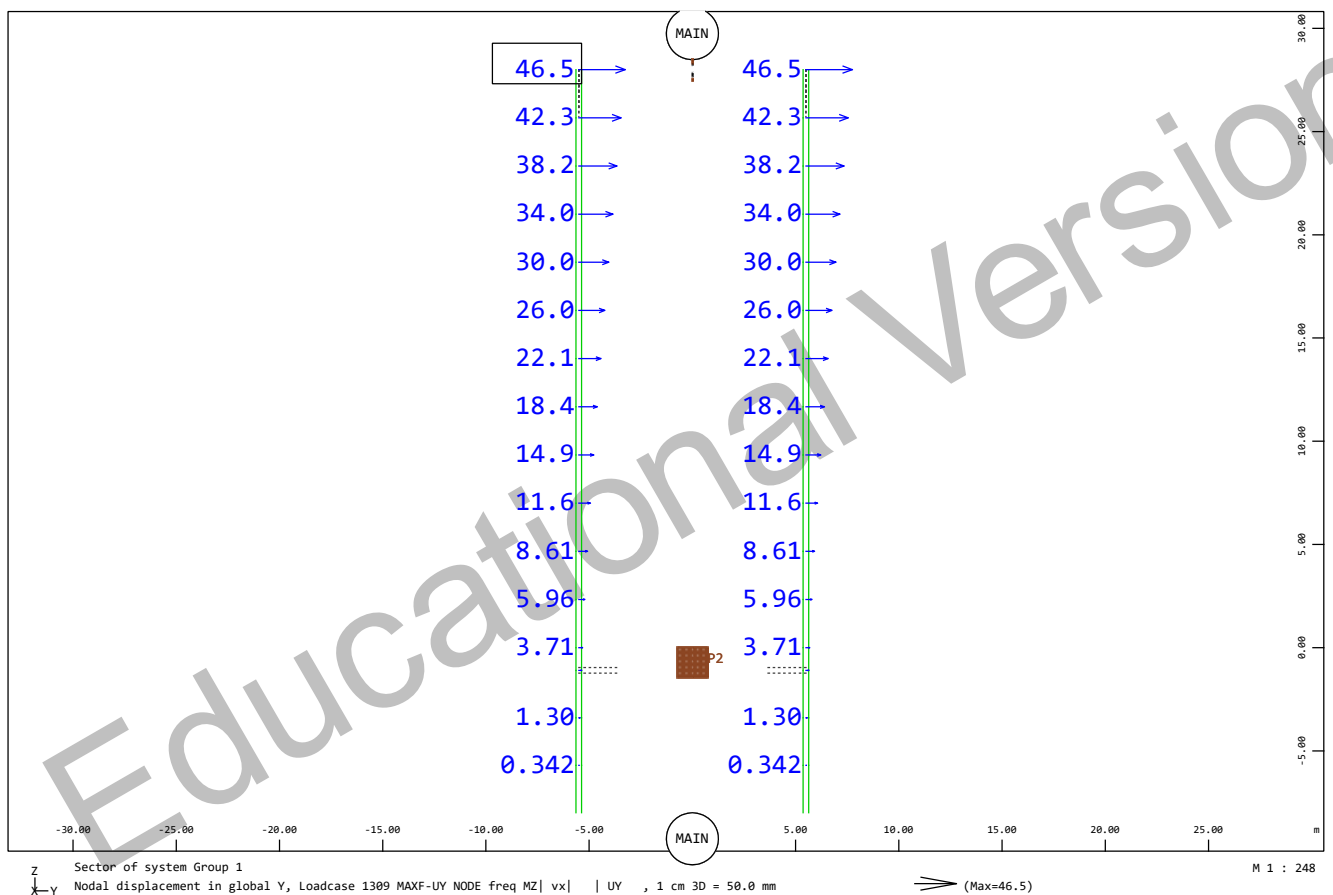
Composite model v2



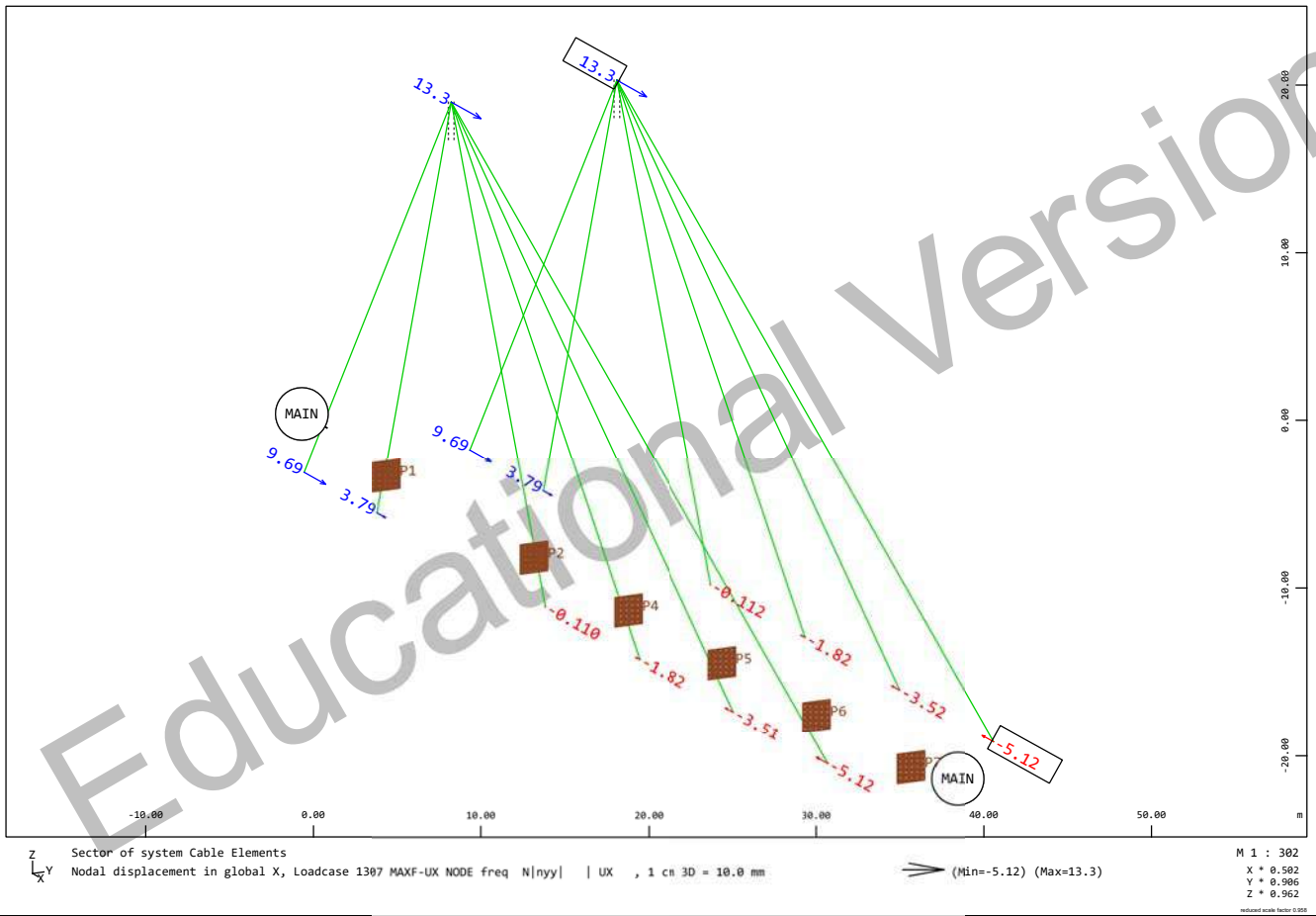
Composite model v2



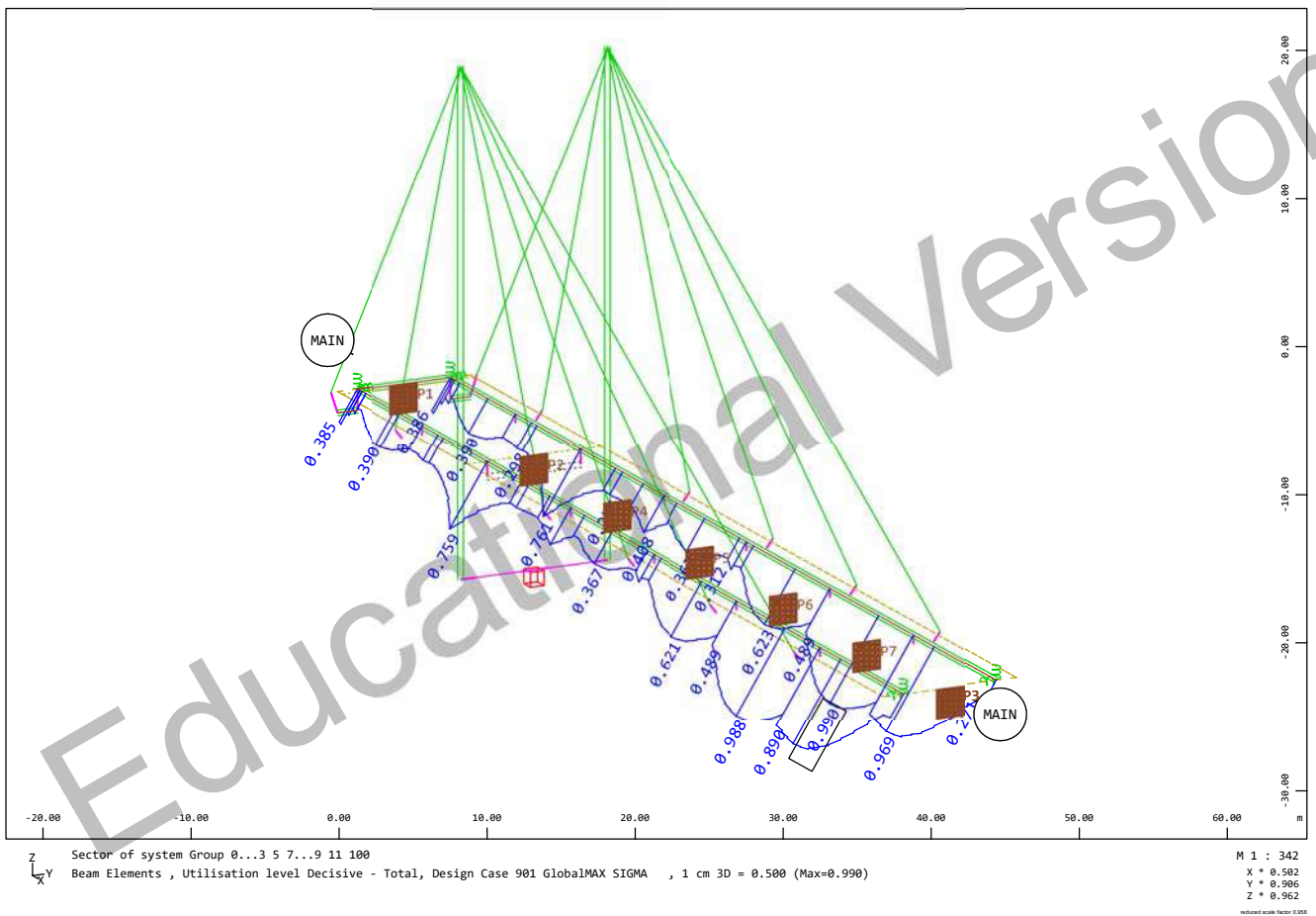
Composite model v2



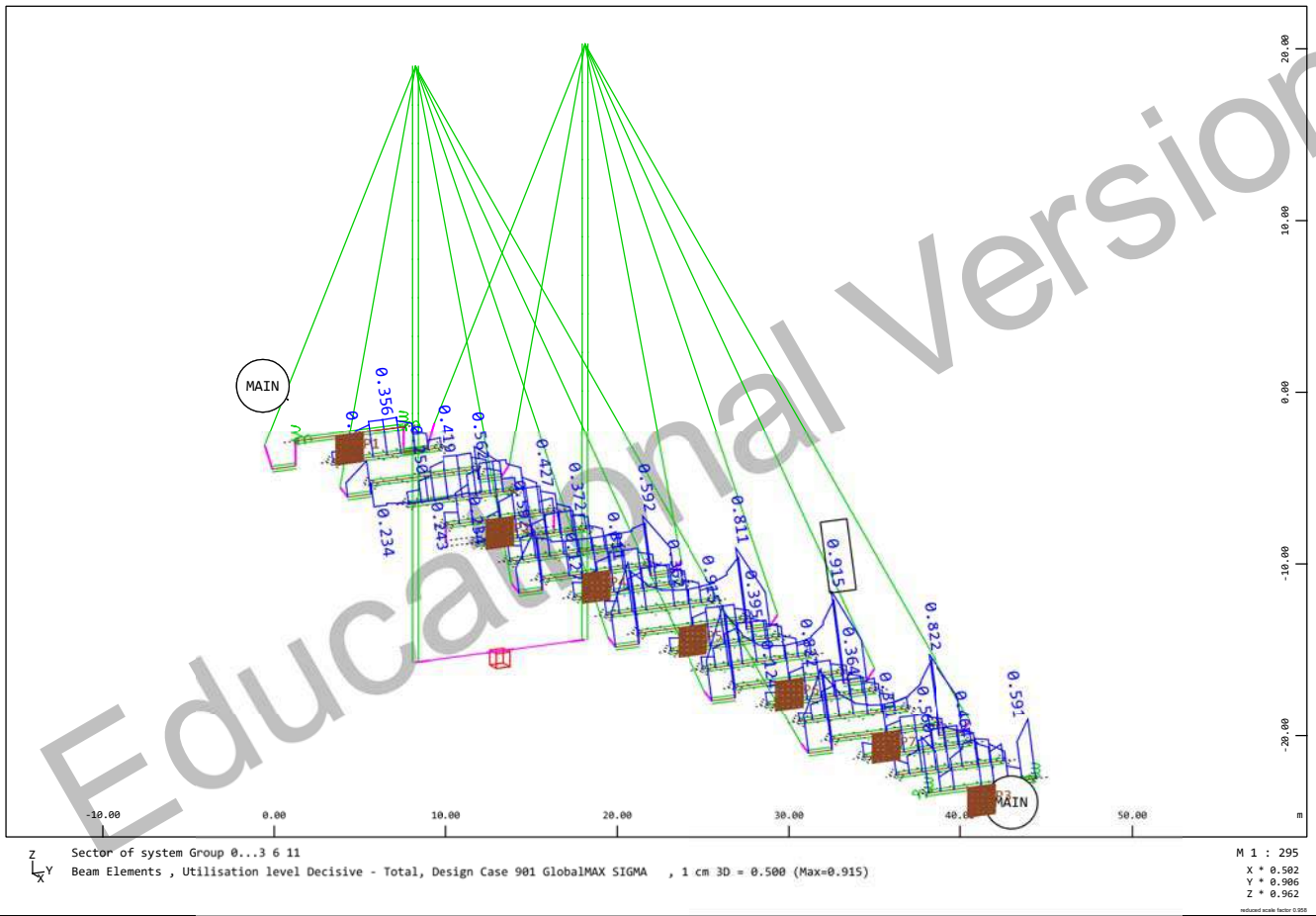
Composite model v2



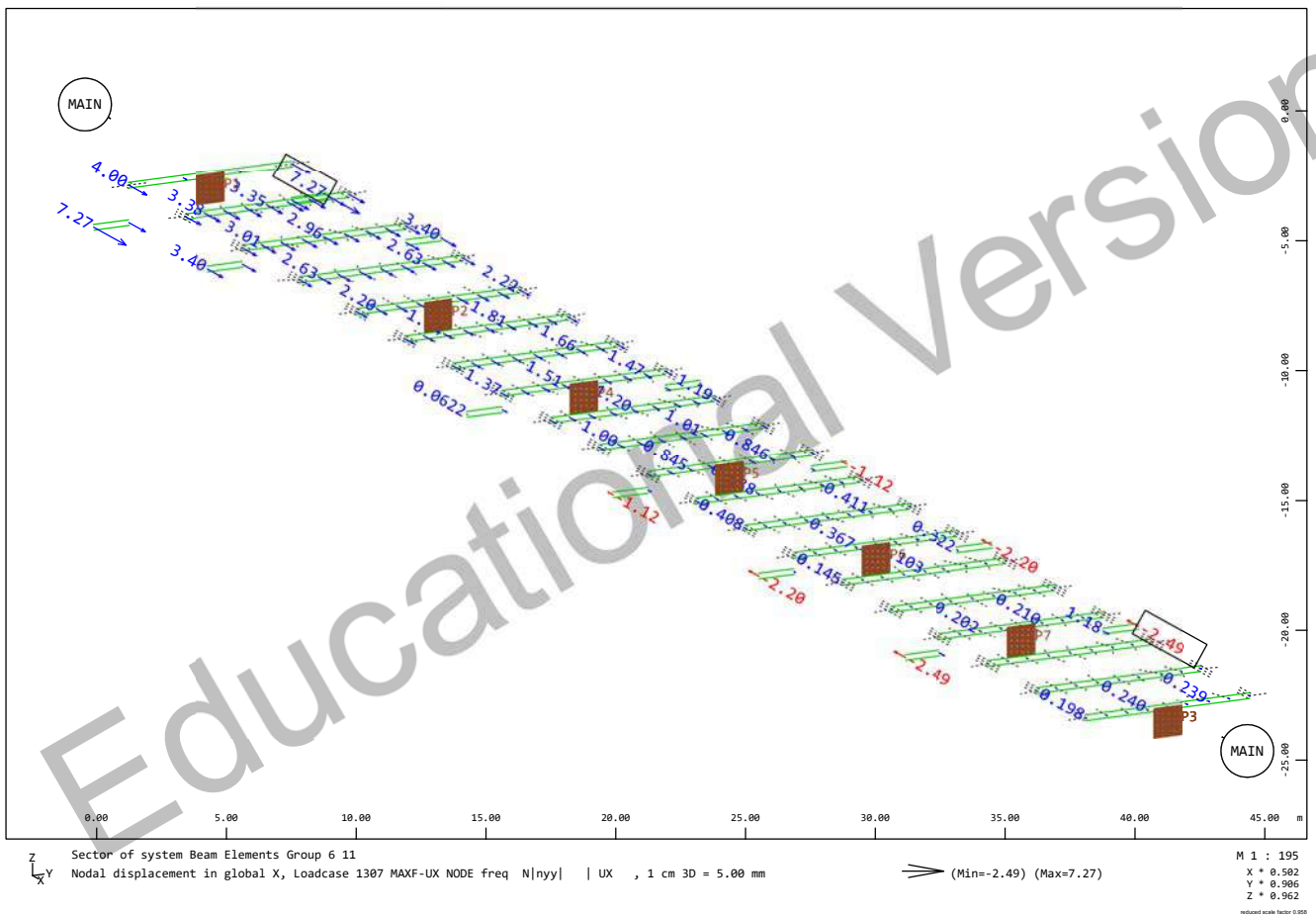
Composite model v2



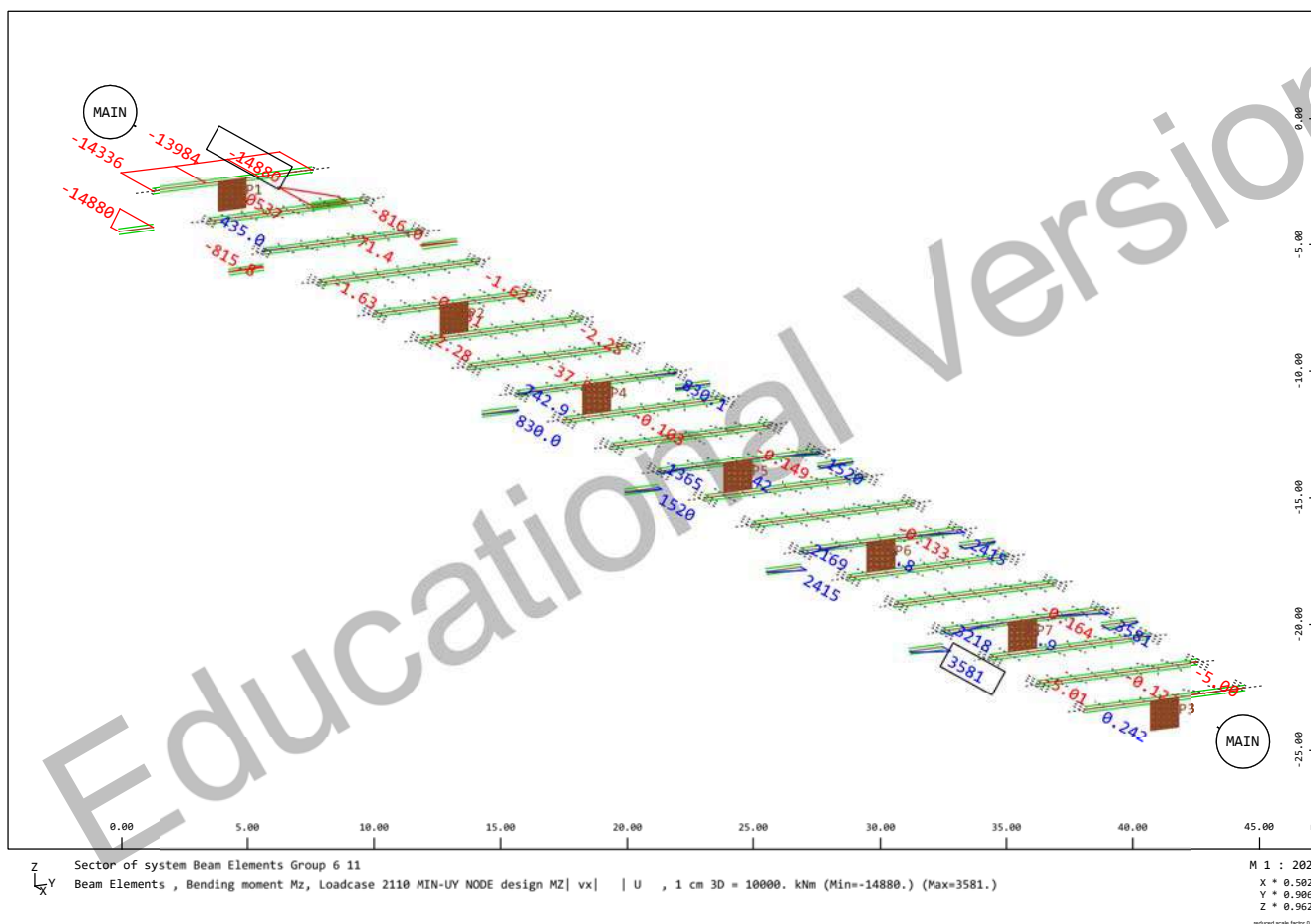
Composite model v2



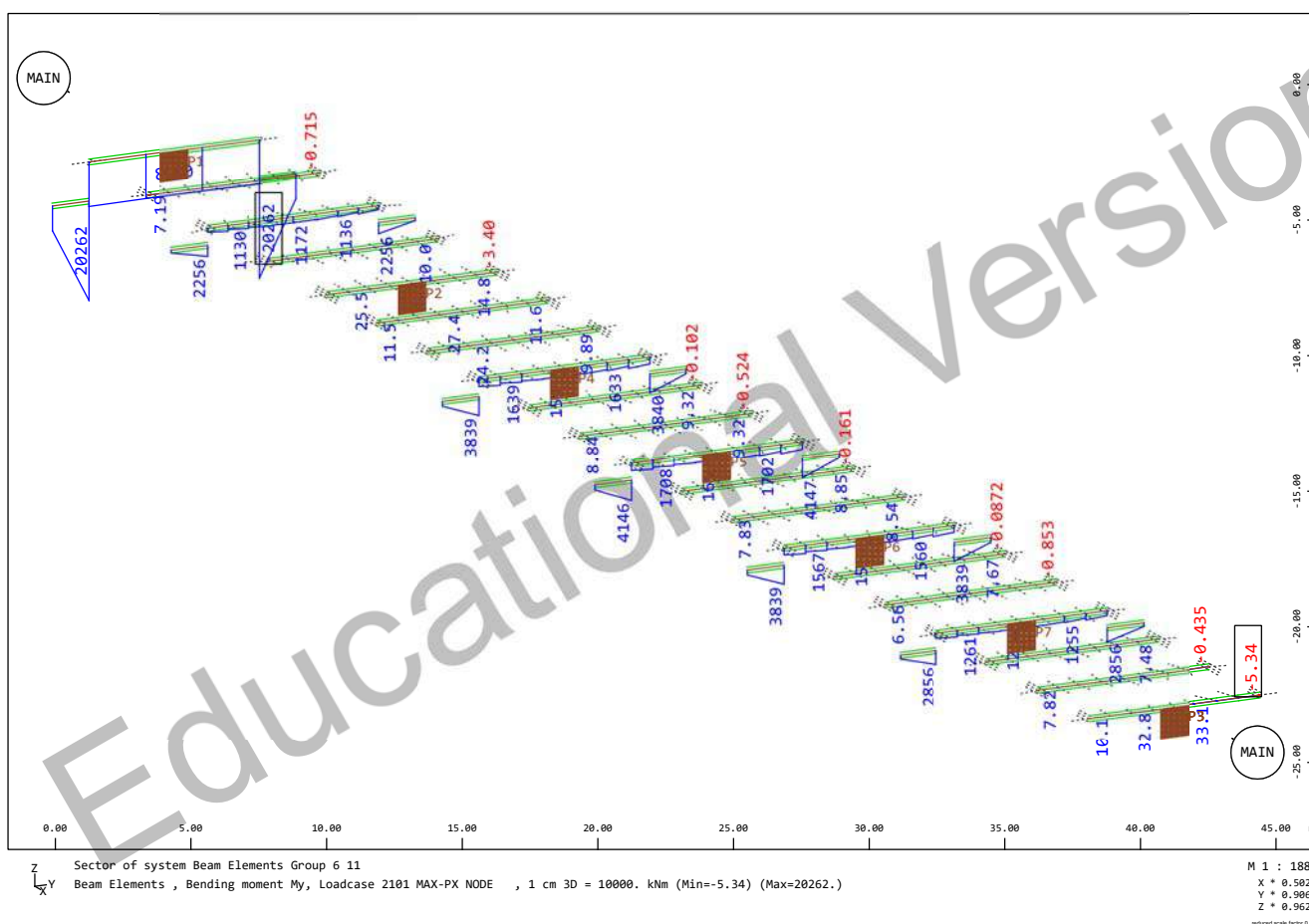
Composite model v2



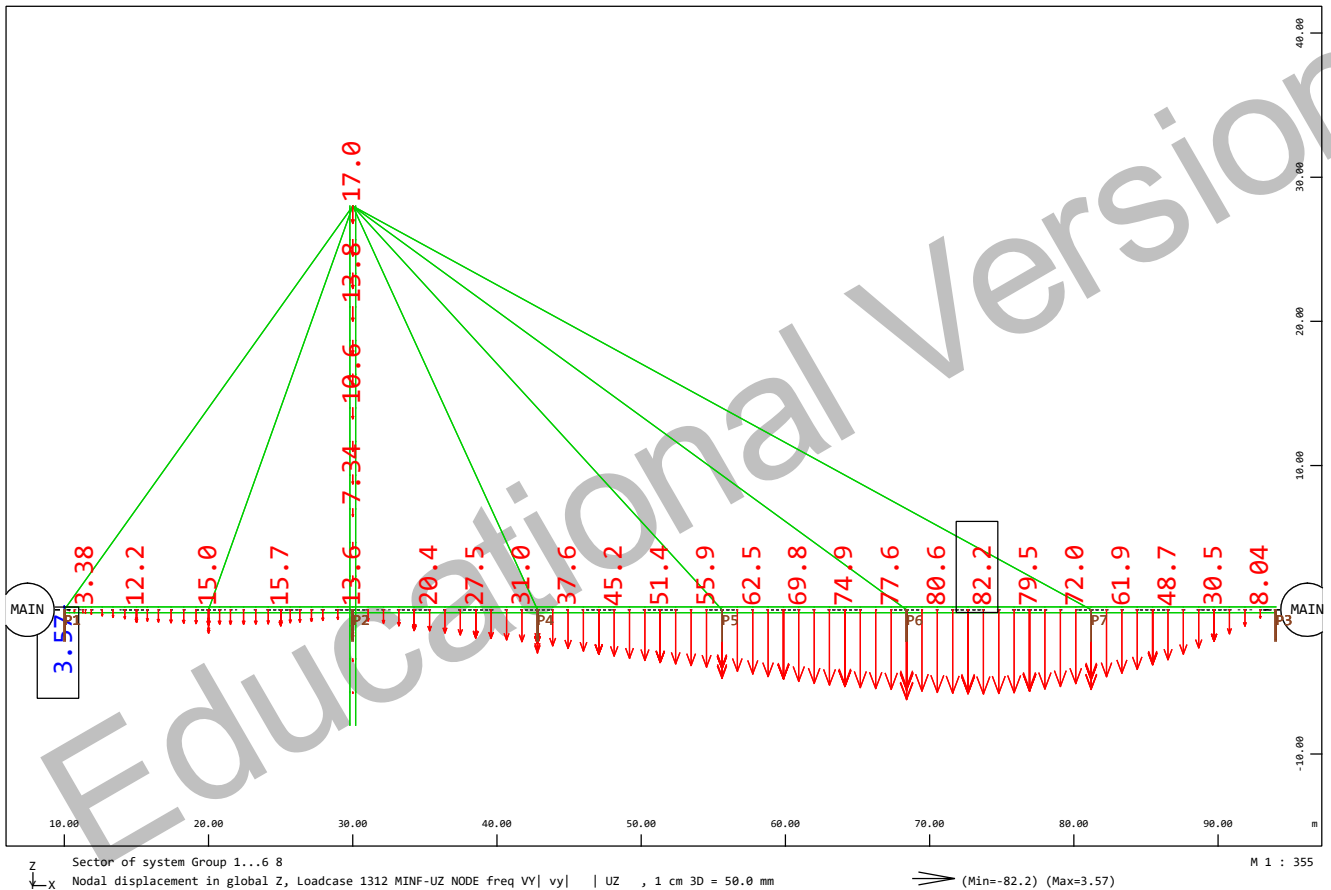
Composite model v2



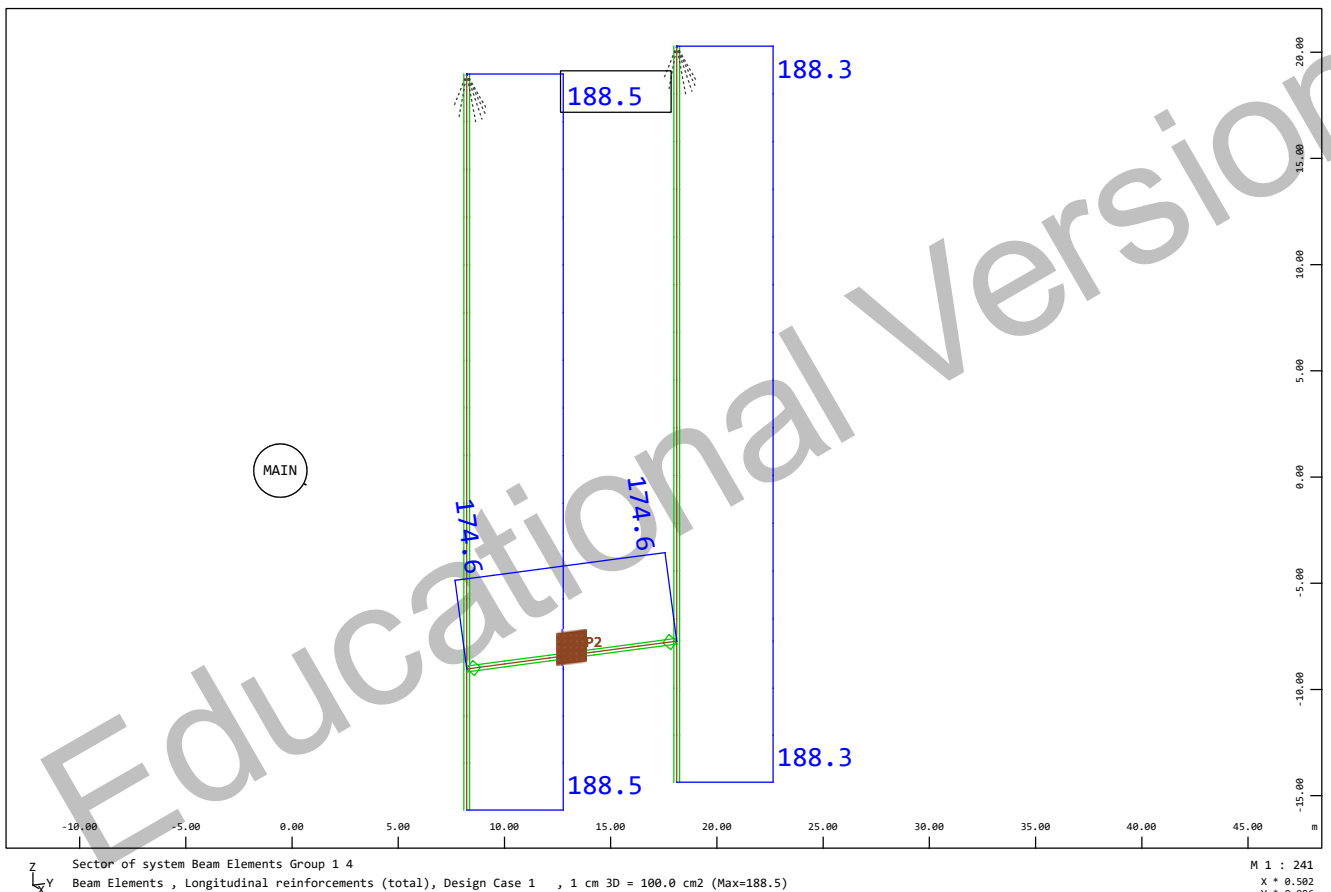
Composite model v2



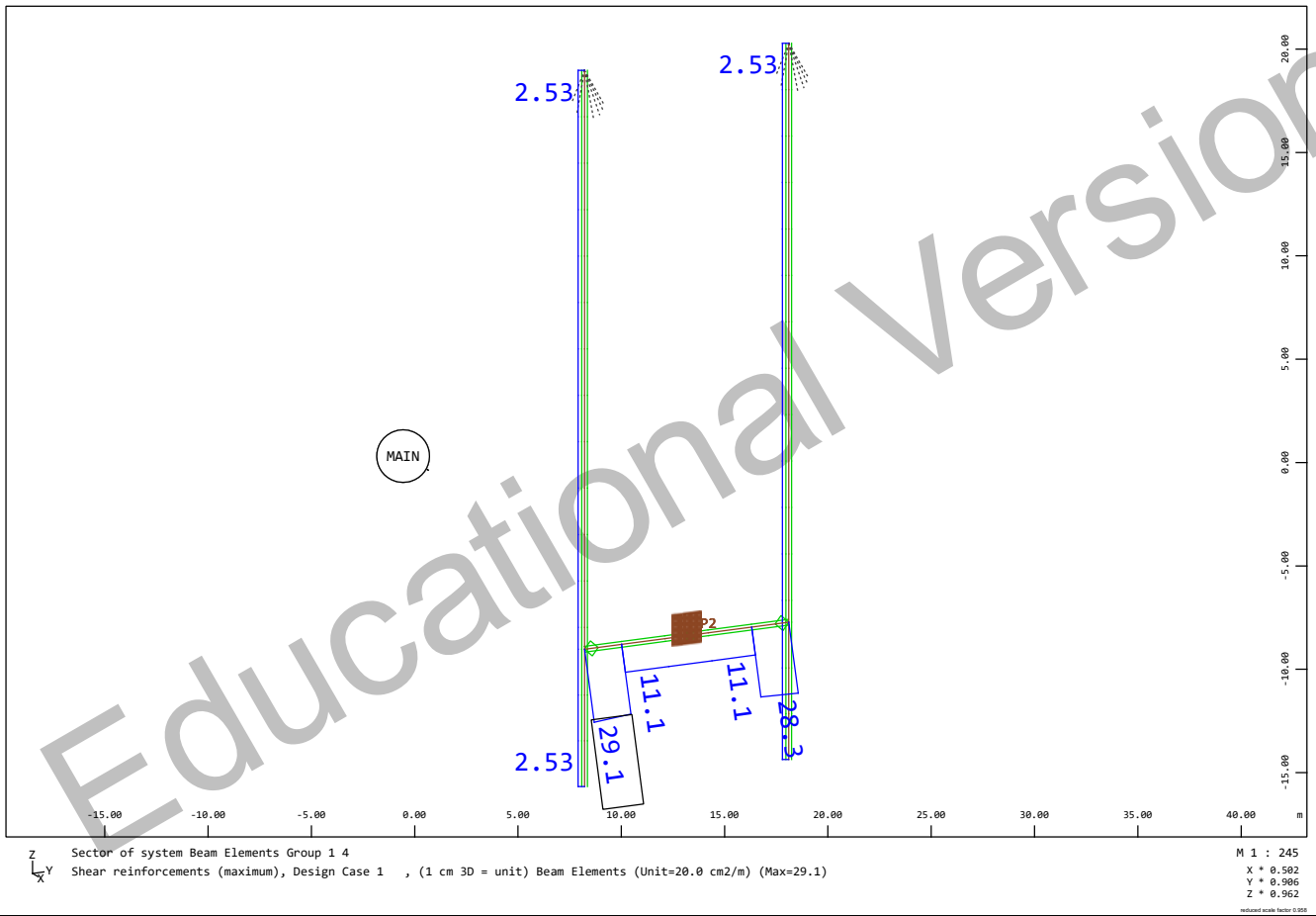
Composite model v2



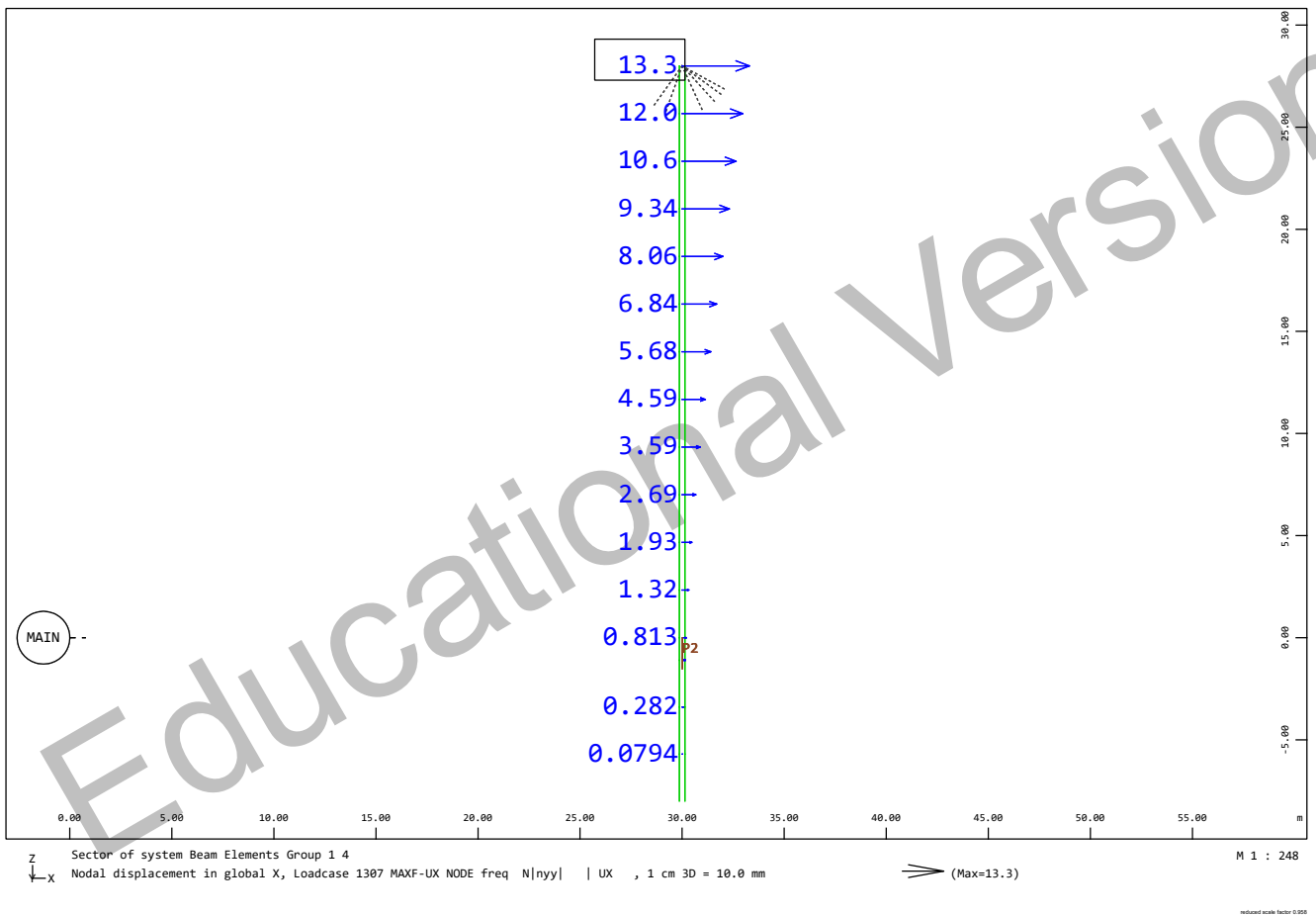
Composite model v2



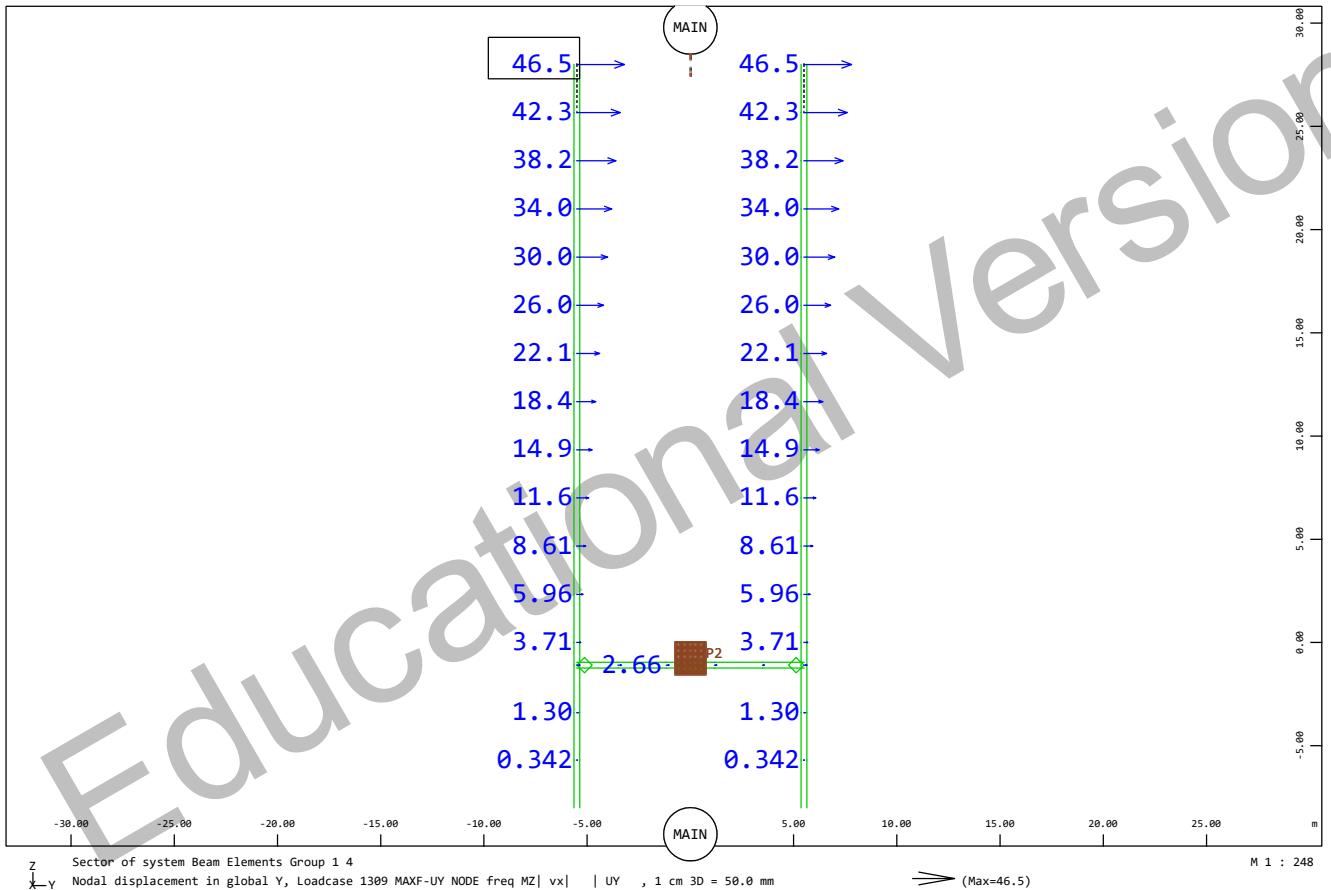
Composite model v2



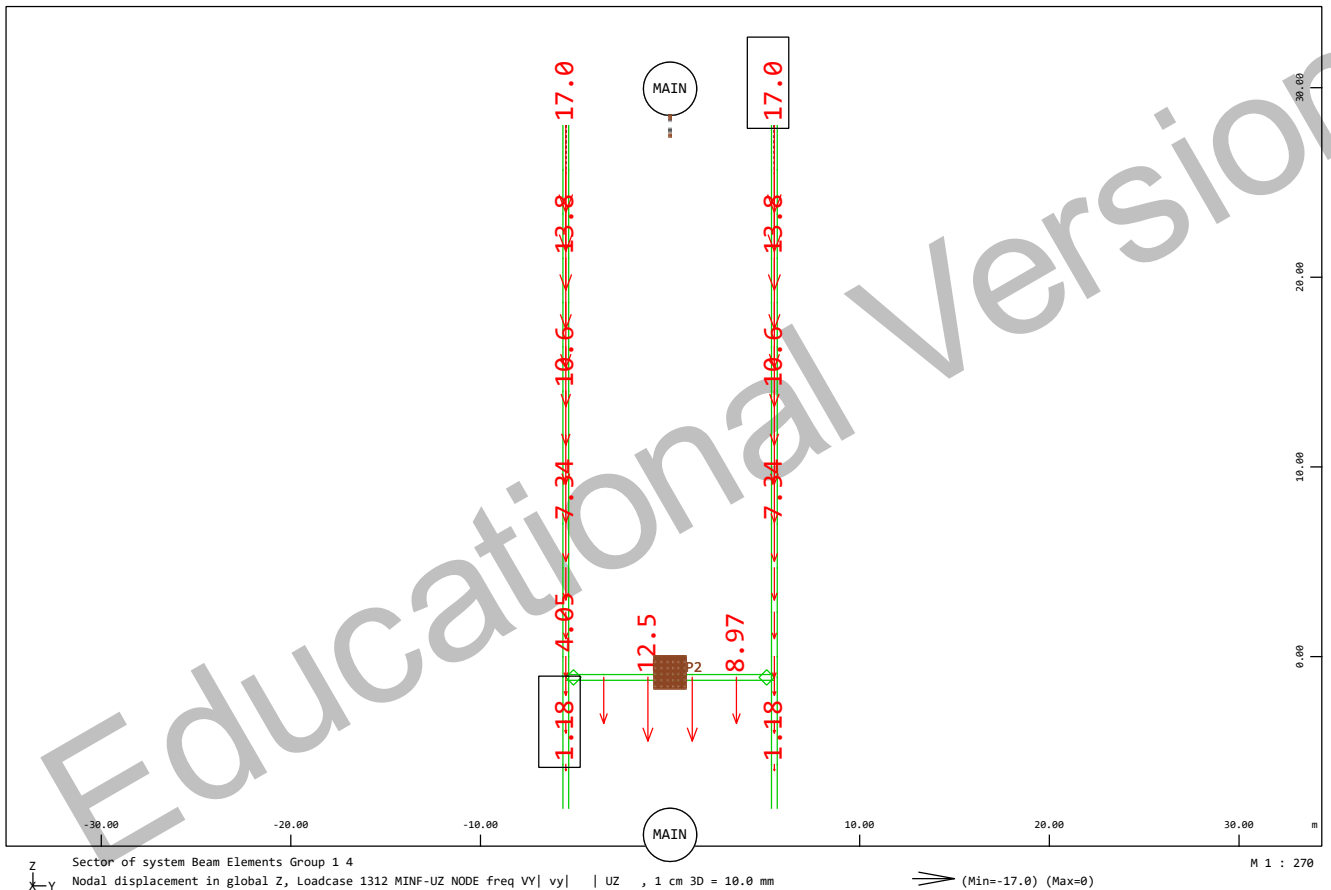
Composite model v2

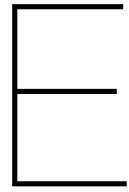


Composite model v2



Composite model v2





Dynamic wind analysis - additional documents

E.1. Modal analysis of FRP bridge

FRP_bridge_v1

System- and Control Information

Control Information

Primary load case 1999 +Opt:add_F +Opt:add_GS
 Number of unknowns 226
 unknowns per node 6
 Number eigenvalues 100

Groups

Grp	Option	CS	Factor	Rayleigh-A [1/sec]	Rayleigh-B [sec]	ξ [o/o]	Wind
0	FULL		1.000	0.000000	0.000000	0.00	
1	FULL		1.000	0.000000	0.000000	0.00	
2	FULL		1.000	0.000000	0.000000	0.00	
3	FULL		1.000	0.000000	0.000000	0.00	
4	FULL		1.000	0.000000	0.000000	0.00	
5	FULL		1.000	0.000000	0.000000	0.00	
CS construction stage							
Factor Factor on stiffness				ξ stiffness proportional damping ratio			
Rayleigh-A mass proportional damping ratio				Wind modal damping ratio			
				options for dynamic wind loading			

Beam Elements

Finite beam elements without intermediate sections
 Shear deformations accounted for with nonconforming SOFiSTiK-Timoshenko beam

Primary load case 1999

Sum of masses and mass moments of inertia

Node	TM			RM			RMB [tm2]
	X[t]	Y[t]	Z[t]	X[tm2]	Y[tm2]	Z[tm2]	
total ¹	1541.972	1541.972	1541.972	8.442E+02	3.820E+02	2.211E+03	-
	S[m] ²			RM(S) ³			
	30.855	0.000	8.887	2.322E+05	7.489E-03	-1.23E+04	
				7.489E-03	3.792E+05	1.598E-03	
				-1.23E+04	1.598E-03	2.504E+05	
active ¹	1541.972	1475.498	1475.498	8.133E+02	3.820E+02	2.211E+03	-
	S[m] ²			RM(S) ³			
	30.855	0.000	9.307	2.262E+05	7.489E-03	-1.28E+04	
				7.489E-03	3.995E+05	1.745E-03	
				-1.28E+04	1.745E-03	2.824E+05	
¹ sum of the total and the active nodal masses							
² coordinates of the center of gravity							
³ 3x3 rotational mass matrix at the center of gravity							
TM translational masses in X-, Y- and Z-direction							
RM rotational masses about X-, Y- and Z-axis							
RMB warping mass							

Processing

Parameter of System of Equations

Number of unknowns 226 (Direct sparse Gauss-Solver)
 Total entries 1849
 Total entries after fill in 2984
 Mass matrix 926 (consistent), incl. rotational masses

Control Information

Using Vectoriteration	
Iteration vectors	102
Iterations	77

Eigenvalues

No.	LC	λ [rad2/sec2]	error [-]	ω [rad/sec]	f [Hz]	T [sec]	ξ [%]	X[%]	Meff Y[%]	Z[%]	participation		
1	500	4.5985E+01	0.0E+00	6.781	1.079	0.927	0.000	0.0	55.9	0.0	0.0	61.4	0.0
2	501	5.0483E+01	0.0E+00	7.105	1.131	0.884	0.000	65.3	0.0	0.0	69.1	0.0	0.0
3	502	6.7717E+01	0.0E+00	8.229	1.310	0.764	0.000	0.0	0.0	0.0	0.0	0.0	0.0

FRP_bridge_v1

Eigenvalues

Eigenvalues

No.	LC	λ [rad2/sec2]	error [-]	ω [rad/sec]	f [Hz]	T [sec]	ξ [%]	X[%]	Meff Y[%]	Z[%]	participation X[%] Y[%] Z[%]		
4	503	3.2748E+02	0.0E+00	18.097	2.880	0.347	0.000	0.5	0.0	4.4	0.5	0.0	5.0
5	504	4.1119E+02	0.0E+00	20.278	3.227	0.310	0.000	0.0	0.3	0.0	0.0	0.4	0.0
6	505	4.7371E+02	0.0E+00	21.765	3.464	0.289	0.000	0.0	4.2	0.0	0.0	4.9	0.0
7	506	8.2025E+02	0.0E+00	28.640	4.558	0.219	0.000	0.2	0.0	0.2	0.2	0.0	0.3
8	507	1.2538E+03	0.0E+00	35.409	5.636	0.177	0.000	1.2	0.0	0.3	1.2	0.0	0.3
9	508	1.6365E+03	0.0E+00	40.453	6.438	0.155	0.000	5.2	0.0	0.3	5.4	0.0	0.4
10	509	1.6812E+03	0.0E+00	41.003	6.526	0.153	0.000	3.5	0.0	3.6	3.6	0.0	4.1
11	510	1.8073E+03	0.0E+00	42.512	6.766	0.148	0.000	0.0	14.9	0.0	0.0	18.2	0.0
12	511	1.8676E+03	0.0E+00	43.215	6.878	0.145	0.000	0.0	0.2	0.0	0.0	0.3	0.0
13	512	2.0089E+03	6.1E-15	44.821	7.133	0.140	0.000	3.0	0.0	0.4	3.2	0.0	0.5
14	513	2.3975E+03	0.0E+00	48.964	7.793	0.128	0.000	1.0	0.0	0.6	1.0	0.0	0.7
15	514	2.5875E+03	0.0E+00	50.867	8.096	0.124	0.000	0.0	0.0	0.0	0.0	0.0	0.0
16	515	2.6376E+03	0.0E+00	51.357	8.174	0.122	0.000	0.0	0.1	0.0	0.0	0.1	0.0
17	516	2.7819E+03	0.0E+00	52.743	8.394	0.119	0.000	0.0	0.0	1.2	0.0	0.0	1.3
18	517	3.3472E+03	4.5E-15	57.855	9.208	0.109	0.000	0.0	0.0	0.0	0.0	0.0	0.0
19	518	3.7524E+03	0.0E+00	61.257	9.749	0.103	0.000	0.0	0.1	0.0	0.0	0.1	0.0
20	519	3.8069E+03	0.0E+00	61.700	9.820	0.102	0.000	0.1	0.0	0.0	0.1	0.0	0.0
21	520	4.3792E+03	0.0E+00	66.175	10.532	0.095	0.000	0.0	0.0	0.0	0.0	0.0	0.0
22	521	4.9606E+03	0.0E+00	70.431	11.209	0.089	0.000	3.8	0.0	0.1	3.9	0.0	0.1
23	522	5.3566E+03	0.0E+00	73.189	11.648	0.086	0.000	0.0	0.0	0.0	0.0	0.0	0.0
24	523	5.8067E+03	0.0E+00	76.201	12.128	0.082	0.000	0.5	0.0	0.1	0.6	0.0	0.0
25	524	6.2481E+03	0.0E+00	79.045	12.580	0.079	0.000	0.0	0.0	0.0	0.0	0.0	0.0
26	525	6.8994E+03	0.0E+00	83.063	13.220	0.076	0.000	0.0	0.0	0.0	0.0	0.0	0.0
27	526	7.0297E+03	0.0E+00	83.843	13.344	0.075	0.000	0.2	0.0	0.1	0.2	0.0	0.1
28	527	7.4021E+03	5.0E-15	86.035	13.693	0.073	0.000	0.5	0.0	0.5	0.5	0.0	0.6
29	528	7.9680E+03	0.0E+00	89.263	14.207	0.070	0.000	0.0	0.0	0.0	0.0	0.0	0.0
30	529	8.4136E+03	0.0E+00	91.726	14.599	0.068	0.000	0.0	0.0	0.0	0.0	0.0	0.0
31	530	8.8743E+03	5.1E-15	94.204	14.993	0.067	0.000	0.0	0.0	0.0	0.0	0.0	0.0
32	531	9.0809E+03	2.9E-14	95.294	15.166	0.066	0.000	0.0	0.0	0.0	0.0	0.0	0.0
33	532	9.0809E+03	2.7E-14	95.294	15.166	0.066	0.000	0.0	0.0	0.0	0.0	0.0	0.0
34	533	9.6975E+03	1.0E-14	98.476	15.673	0.064	0.000	0.0	0.0	0.0	0.0	0.0	0.0
35	534	9.8447E+03	4.8E-15	99.221	15.791	0.063	0.000	0.0	0.0	0.0	0.0	0.0	0.0
36	535	1.1418E+04	4.9E-15	106.853	17.006	0.059	0.000	0.0	0.6	0.0	0.0	0.6	0.0
37	536	1.1492E+04	1.6E-14	107.198	17.061	0.059	0.000	0.0	0.0	0.0	0.0	0.0	0.0
38	537	1.1773E+04	1.4E-14	108.504	17.269	0.058	0.000	0.0	3.2	0.0	0.0	3.3	0.0
39	538	1.2722E+04	0.0E+00	112.790	17.951	0.056	0.000	0.0	0.0	0.0	0.0	0.0	0.0
40	539	1.3418E+04	4.6E-14	115.835	18.436	0.054	0.000	0.0	0.8	0.0	0.0	0.8	0.0
41	540	1.3698E+04	1.7E-14	117.039	18.627	0.054	0.000	0.0	2.3	0.0	0.0	2.3	0.0
42	541	1.6083E+04	4.1E-14	126.819	20.184	0.050	0.000	0.0	1.0	0.0	0.0	0.9	0.0
43	542	1.6331E+04	1.7E-14	127.793	20.339	0.049	0.000	0.0	0.3	0.0	0.0	0.3	0.0
44	543	1.7713E+04	8.2E-14	133.089	21.182	0.047	0.000	0.0	0.0	0.0	0.0	0.0	0.0
45	544	1.9372E+04	1.4E-14	139.182	22.152	0.045	0.000	0.0	0.0	0.0	0.0	0.0	0.0
46	545	2.0596E+04	2.7E-14	143.514	22.841	0.044	0.000	0.0	0.0	0.0	0.0	0.0	0.0
47	546	2.1424E+04	1.4E-13	146.370	23.295	0.043	0.000	0.0	0.0	0.0	0.0	0.0	0.0
48	547	2.1806E+04	3.1E-14	147.668	23.502	0.043	0.000	0.6	0.0	0.0	0.6	0.0	0.0
49	548	2.2541E+04	2.4E-13	150.137	23.895	0.042	0.000	4.5	0.0	2.5	4.4	0.0	2.7
50	549	2.4227E+04	1.3E-13	155.652	24.773	0.040	0.000	0.0	0.1	0.0	0.0	0.1	0.0
51	550	2.5039E+04	3.2E-14	158.237	25.184	0.040	0.000	0.0	0.0	0.9	0.0	0.0	1.0
52	551	2.5565E+04	6.7E-13	159.892	25.448	0.039	0.000	0.0	0.1	0.0	0.0	0.1	0.0
53	552	2.6566E+04	2.1E-13	162.992	25.941	0.039	0.000	0.0	0.0	52.0	0.0	0.0	57.9
54	553	2.7196E+04	3.7E-13	164.911	26.246	0.038	0.000	0.0	0.0	0.0	0.0	0.0	0.0
55	554	2.8699E+04	2.8E-13	169.409	26.962	0.037	0.000	0.0	0.0	0.0	0.0	0.0	0.0
56	555	2.9023E+04	2.2E-13	170.362	27.114	0.037	0.000	0.2	0.0	11.3	0.2	0.0	12.4
57	556	3.1150E+04	2.7E-13	176.494	28.090	0.036	0.000	0.0	0.0	0.0	0.0	0.0	0.0
58	557	3.4438E+04	8.5E-13	185.575	29.535	0.034	0.000	0.0	0.0	0.0	0.0	0.0	0.0
59	558	3.7180E+04	1.9E-12	192.821	30.688	0.033	0.000	0.0	0.0	0.0	0.0	0.0	0.0
60	559	3.7610E+04	1.1E-12	193.933	30.865	0.032	0.000	0.0	0.0	0.1	0.0	0.0	0.1

FRP_bridge_v1

Eigenvalues

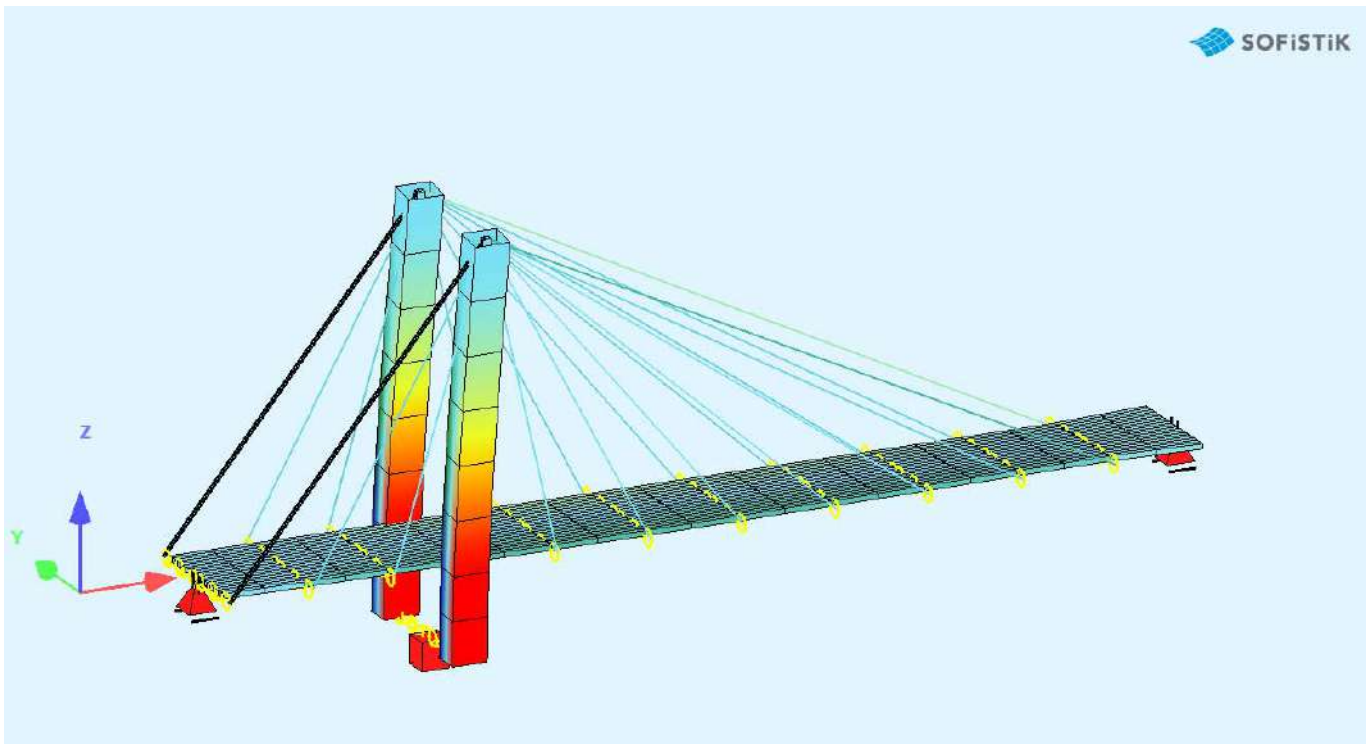
Eigenvalues

No.	LC	λ [rad2/sec2]	error [-]	ω [rad/sec]	f [Hz]	T [sec]	ξ [%]	X[%]	Meff Y[%]	Z[%]	participation		
											X[%]	Y[%]	Z[%]
61	560	4.0054E+04	2.0E-12	200.134	31.852	0.031	0.000	0.0	0.2	0.0	0.0	0.2	0.0
62	561	4.3427E+04	1.1E-13	208.392	33.167	0.030	0.000	0.0	0.0	0.0	0.0	0.0	0.0
63	562	4.3785E+04	5.7E-13	209.248	33.303	0.030	0.000	0.0	0.0	0.0	0.0	0.0	0.0
64	563	4.8803E+04	9.4E-13	220.914	35.160	0.028	0.000	0.0	0.0	0.1	0.0	0.0	0.1
65	564	5.2482E+04	6.1E-12	229.089	36.461	0.027	0.000	0.0	0.9	0.0	0.0	1.0	0.0
66	565	5.5991E+04	7.3E-12	236.624	37.660	0.027	0.000	0.0	1.5	0.0	0.0	1.7	0.0
67	566	5.6498E+04	3.1E-12	237.692	37.830	0.026	0.000	0.0	0.0	0.0	0.0	0.0	0.0
68	567	6.4021E+04	1.1E-11	253.024	40.270	0.025	0.000	0.1	0.0	0.0	0.1	0.0	0.0
69	568	6.7260E+04	4.3E-12	259.345	41.276	0.024	0.000	0.0	0.0	0.0	0.0	0.0	0.0
70	569	7.3107E+04	2.0E-12	270.383	43.033	0.023	0.000	0.0	0.1	0.0	0.0	0.1	0.0
71	570	8.2068E+04	6.7E-12	286.475	45.594	0.022	0.000	3.0	0.0	0.0	2.7	0.0	0.0
72	571	8.2963E+04	2.9E-11	288.033	45.842	0.022	0.000	0.0	0.0	0.0	0.0	0.0	0.0
73	572	8.3979E+04	3.5E-11	289.792	46.122	0.022	0.000	0.0	0.0	0.0	0.0	0.0	0.0
74	573	8.3979E+04	1.0E-10	289.792	46.122	0.022	0.000	0.0	0.0	0.0	0.0	0.0	0.0
75	574	1.0070E+05	1.1E-10	317.329	50.505	0.020	0.000	0.0	0.0	0.0	0.0	0.0	0.0
76	575	1.0470E+05	1.9E-12	323.574	51.498	0.019	0.000	0.0	0.1	0.0	0.0	0.1	0.0
77	576	1.1682E+05	8.4E-11	341.789	54.397	0.018	0.000	0.0	0.0	0.0	0.0	0.0	0.0
78	577	1.2141E+05	7.3E-11	348.445	55.457	0.018	0.000	0.0	0.0	0.0	0.0	0.0	0.0
79	578	1.5390E+05	1.2E-10	392.304	62.437	0.016	0.000	0.0	1.7	0.0	0.0	1.2	0.0
80	579	1.6400E+05	2.5E-10	404.969	64.453	0.016	0.000	0.0	0.0	0.0	0.0	0.0	0.0
81	580	1.6841E+05	2.8E-10	410.374	65.313	0.015	0.000	0.0	0.3	0.0	0.0	0.2	0.0
82	581	1.7332E+05	3.4E-11	416.320	66.259	0.015	0.000	0.0	0.0	0.0	0.0	0.0	0.0
83	582	1.8974E+05	7.2E-10	435.586	69.326	0.014	0.000	0.0	0.0	0.0	0.0	0.0	0.0
84	583	2.0141E+05	3.2E-10	448.790	71.427	0.014	0.000	0.1	0.0	0.0	0.0	0.0	0.0
85	584	2.1554E+05	1.8E-09	464.259	73.889	0.014	0.000	1.6	0.0	0.0	1.2	0.0	0.0
86	585	2.1883E+05	2.1E-10	467.788	74.451	0.013	0.000	0.0	0.0	0.0	0.0	0.0	0.0
87	586	2.2242E+05	1.6E-09	471.610	75.059	0.013	0.000	0.0	0.1	0.0	0.0	0.0	0.0
88	587	2.3524E+05	2.0E-10	485.015	77.193	0.013	0.000	0.0	0.0	6.6	0.0	0.0	7.8
89	588	2.3557E+05	6.7E-09	485.351	77.246	0.013	0.000	0.0	0.0	0.0	0.0	0.0	0.0
90	589	2.3601E+05	5.8E-09	485.804	77.318	0.013	0.000	0.0	0.0	0.0	0.0	0.0	0.0
91	590	2.4519E+05	2.9E-09	495.164	78.808	0.013	0.000	0.0	0.0	0.0	0.0	0.0	0.0
92	591	2.4519E+05	2.0E-09	495.164	78.808	0.013	0.000	0.0	0.0	0.0	0.0	0.0	0.0
93	592	2.5364E+05	2.5E-10	503.623	80.154	0.012	0.000	0.1	0.0	0.0	0.0	0.0	0.0
94	593	2.6159E+05	3.5E-09	511.456	81.401	0.012	0.000	0.0	0.0	0.0	0.0	0.0	0.0
95	594	2.9043E+05	5.4E-10	538.918	85.771	0.012	0.000	0.0	0.0	0.0	0.0	0.0	0.0
96	595	3.0039E+05	1.4E-09	548.080	87.230	0.011	0.000	0.0	0.0	0.0	0.0	0.0	0.0
97	596	3.0933E+05	2.4E-09	556.174	88.518	0.011	0.000	0.0	0.0	0.0	0.0	0.0	0.0
98	597	3.3660E+05	3.0E-08	580.175	92.338	0.011	0.000	0.0	0.0	0.0	0.0	0.0	0.0
99	598	3.3924E+05	3.8E-09	582.446	92.699	0.011	0.000	0.0	0.0	0.0	0.0	0.0	0.0
100	599	3.5528E+05	8.9E-07	596.050	94.864	0.011	0.000	0.0	0.7	0.0	0.0	0.6	0.0
101		3.5596E+05	2.8E-08	596.623	94.956	0.011	$\Sigma(\%)^1$	95.2	90.2	85.9	98.9	99.3	95.8
102		3.6403E+05	4.1E-06	603.351	96.026	0.010							

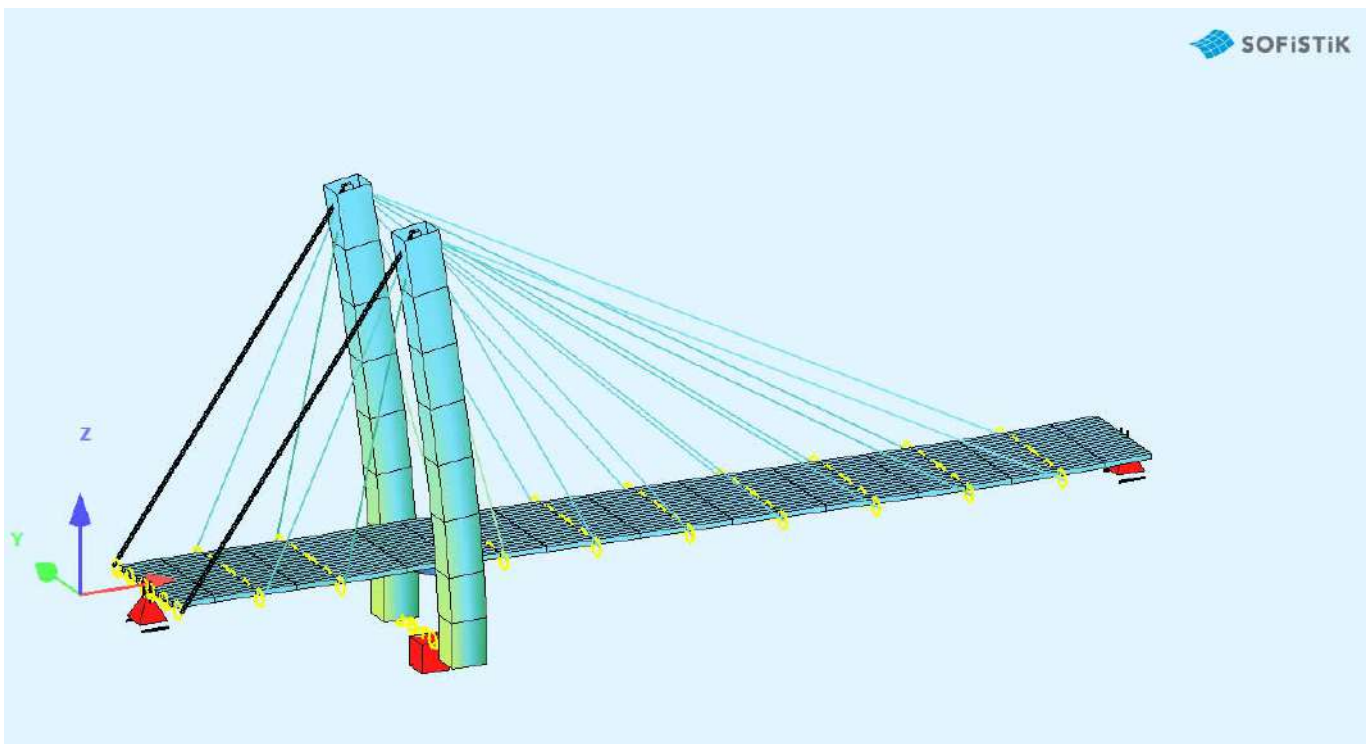
¹ Total effective mass / participation factors of activated masses in X-, Y- and Z-direction.

No.	eigenmode number	f	eigenfrequency
LC	load case	T	eigenperiod
λ	eigenvalue	ξ	modal damping ratio
error	relative eigenvalue error	Meff	effective modal mass in X-, Y- and Z-direction
ω	circular eigenfrequency	participation	participation factors in X-, Y- and Z-direction

FRP_bridge_v1
Eigenvalues

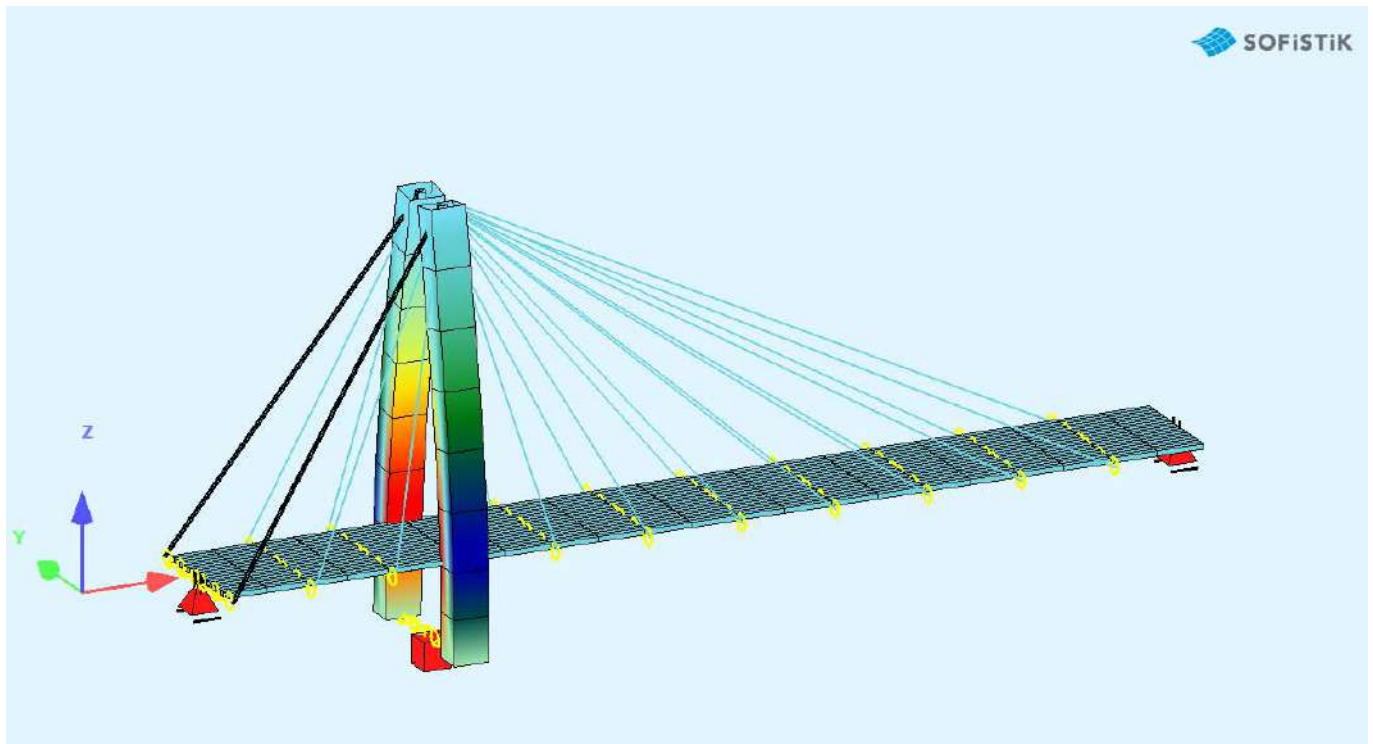


Mode 1: $f_n = 1.079$ Hz

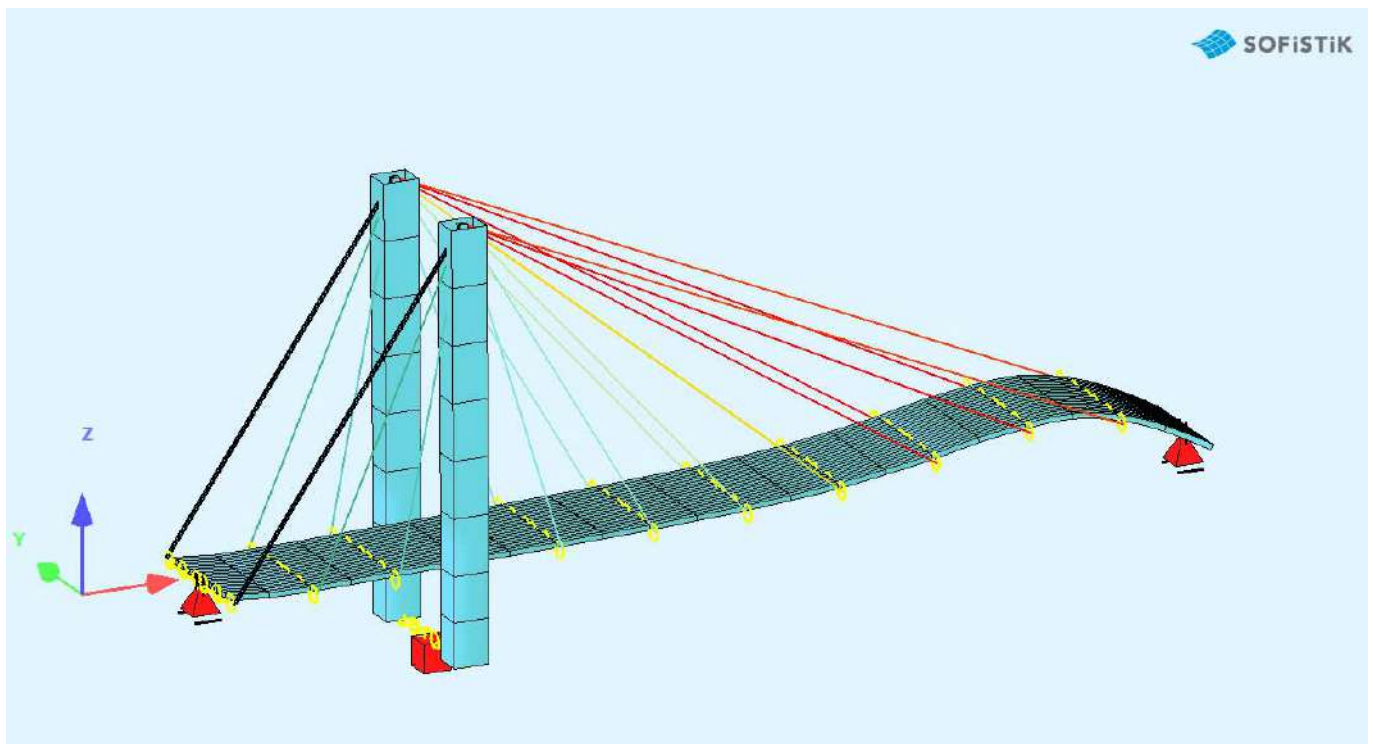


Mode 2: $f_n = 1.131$ Hz

FRP_bridge_v1
Eigenvalues

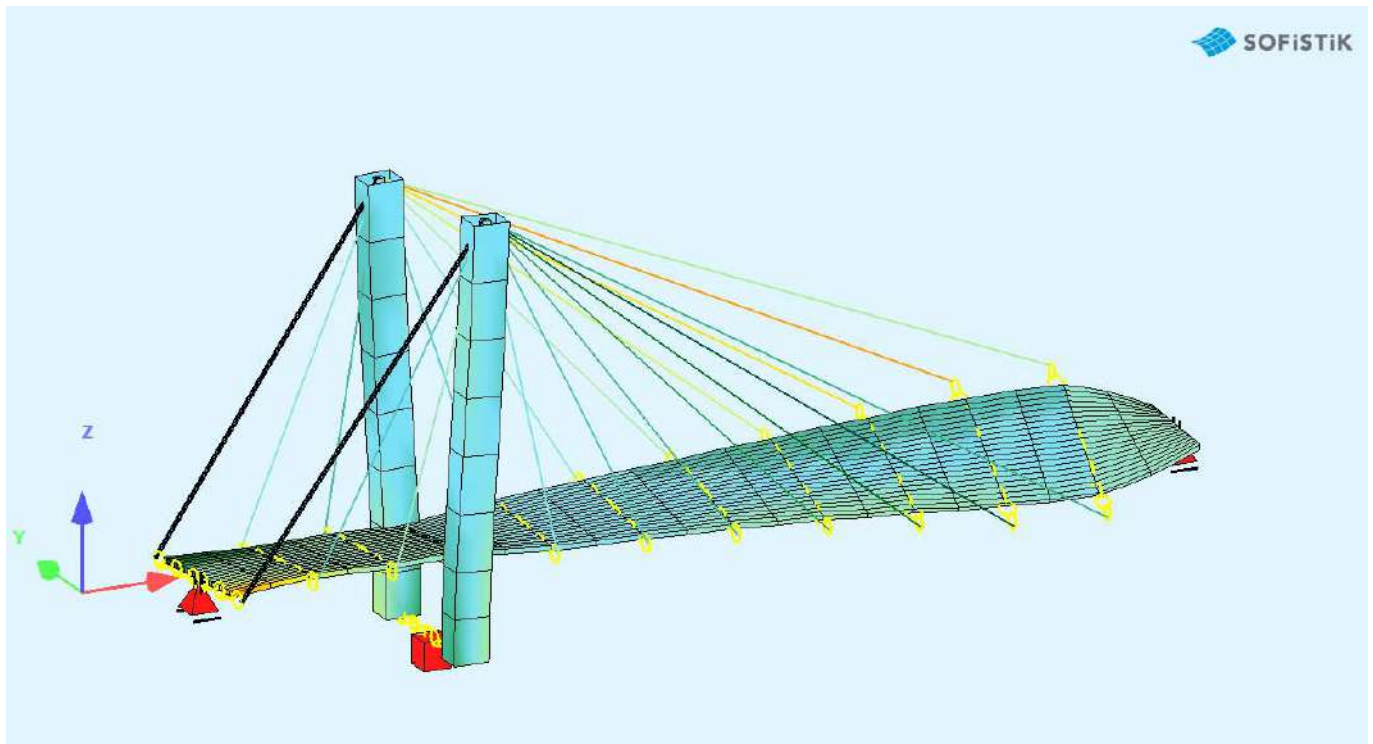


Mode 3: $f_n = 1.310$ Hz



Mode 4: $f_n = 2.880$ Hz

FRP_bridge_v1
Eigenvalues

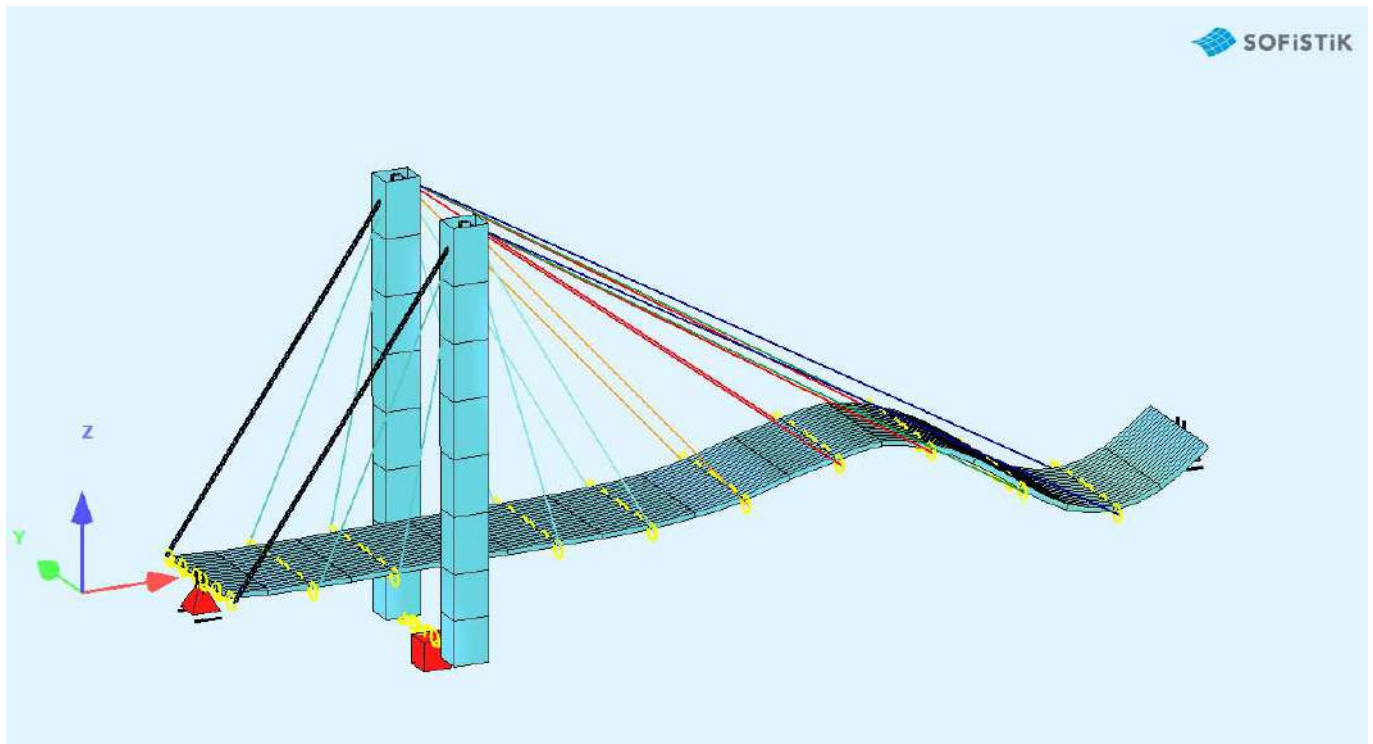


Mode 5: $f_n = 3.227$ Hz



Mode 6: $f_n = 3.464$ Hz

FRP_bridge_v1
Eigenvalues



Mode 7: $f_n = 4.558$ Hz

E.2. Modal analysis of steel-concrete composite bridge

Composite model v2

System- and Control Information

Control Information

Primary load case 1999 +Opt:add_F +Opt:add_GS
 Number of unknowns 224
 unknowns per node 6
 Number eigenvalues 60

Groups

Grp	Option	CS	Factor	Rayleigh-A [1/sec]	Rayleigh-B [sec]	ξ [o/o]	Wind
0	FULL		1.000	0.000000	0.000000	0.00	
1	FULL		1.000	0.000000	0.000000	0.00	
2	FULL		1.000	0.000000	0.000000	0.00	
3	FULL		1.000	0.000000	0.000000	0.00	
4	FULL		1.000	0.000000	0.000000	0.00	
5	FULL		1.000	0.000000	0.000000	0.00	
CS construction stage Factor Factor on stiffness Rayleigh-A mass proportional damping ratio Rayleigh-B stiffness proportional damping ratio ξ modal damping ratio Wind options for dynamic wind loading							

Beam Elements

Finite beam elements without intermediate sections
 Shear deformations accounted for with nonconforming SOFiSTiK-Timoshenko beam

Primary load case 1999

Sum of masses and mass moments of inertia

Node	TM			RM			RMB
	X[t]	Y[t]	Z[t]	X[tm2]	Y[tm2]	Z[tm2]	[tm2]
total ¹	2605.761	2605.762	2605.762	7.183E+03	5.460E+02	7.770E+03	-
	S[m] ²			RM(S) ³			
	39.431	0.000	5.202	2.750E+05	7.595E-03	-1.28E+05	
				7.595E-03	1.316E+06	-2.06E-03	
				-1.28E+05	-2.06E-03	1.144E+06	
active ¹	2605.761	2482.738	2482.738	6.803E+03	5.460E+02	7.770E+03	-
	S[m] ²			RM(S) ³			
	39.431	0.000	5.460	2.715E+05	7.595E-03	-1.28E+05	
				7.595E-03	1.237E+06	-1.97E-03	
				-1.28E+05	-1.97E-03	1.072E+06	
¹ sum of the total and the active nodal masses ² coordinates of the center of gravity ³ 3x3 rotational mass matrix at the center of gravity TM translational masses in X-, Y- and Z-direction RM rotational masses about X-, Y- and Z-axis RMB warping mass							

Processing

Parameter of System of Equations

Number of unknowns 224 (Direct sparse Gauss-Solver)
 Total entries 1621
 Total entries after fill in 2614
 Mass matrix 871 (consistent), incl. rotational masses

Control Information

Using Vectoriteration	
Iteration vectors	62
Iterations	19

Eigenvalues

No.	LC	λ [rad2/sec2]	error [-]	ω [rad/sec]	f [Hz]	T [sec]	ξ [%]	X[%]	Meff Y[%]	Z[%]	participation X[%]	Y[%]	Z[%]
1	500	4.7884E+01	0.0E+00	6.920	1.101	0.908	0.000	0.0	37.1	0.0	0.0	39.7	0.0
2	501	5.5630E+01	0.0E+00	7.459	1.187	0.842	0.000	0.7	0.0	24.3	0.7	0.0	27.0
3	502	6.9113E+01	0.0E+00	8.313	1.323	0.756	0.000	0.0	0.0	0.0	0.0	0.0	0.0

Composite model v2

Eigenvalues

Eigenvalues

No.	LC	λ [rad2/sec2]	error [-]	ω [rad/sec]	f [Hz]	T [sec]	ξ [%]	X[%]	Meff Y[%]	Z[%]	participation		
											X[%]	Y[%]	Z[%]
4	503	1.3294E+02	0.0E+00	11.530	1.835	0.545	0.000	47.4	0.0	0.5	49.0	0.0	0.6
5	504	1.6601E+02	0.0E+00	12.884	2.051	0.488	0.000	32.8	0.0	2.4	33.9	0.0	2.7
6	505	3.0787E+02	0.0E+00	17.546	2.793	0.358	0.000	0.0	1.9	0.0	0.0	2.1	0.0
7	506	3.5481E+02	0.0E+00	18.837	2.998	0.334	0.000	0.6	0.0	11.5	0.6	0.0	12.5
8	507	5.0786E+02	0.0E+00	22.536	3.587	0.279	0.000	1.5	0.0	5.3	1.5	0.0	5.7
9	508	5.4220E+02	0.0E+00	23.285	3.706	0.270	0.000	0.0	29.6	0.0	0.0	33.1	0.0
10	509	6.2834E+02	0.0E+00	25.067	3.990	0.251	0.000	0.0	0.4	0.0	0.0	0.5	0.0
11	510	8.0185E+02	0.0E+00	28.317	4.507	0.222	0.000	0.0	0.1	0.0	0.0	0.2	0.0
12	511	8.8548E+02	0.0E+00	29.757	4.736	0.211	0.000	1.0	0.0	0.4	1.1	0.0	0.4
13	512	9.5046E+02	0.0E+00	30.830	4.907	0.204	0.000	0.0	0.5	0.0	0.0	0.6	0.0
14	513	1.0510E+03	0.0E+00	32.420	5.160	0.194	0.000	2.0	0.0	0.1	2.1	0.0	0.1
15	514	1.3305E+03	0.0E+00	36.476	5.805	0.172	0.000	0.0	0.5	0.0	0.0	0.6	0.0
16	515	1.5688E+03	0.0E+00	39.608	6.304	0.159	0.000	0.0	0.8	0.0	0.0	0.9	0.0
17	516	1.6690E+03	0.0E+00	40.853	6.502	0.154	0.000	0.0	0.0	0.0	0.0	0.0	0.0
18	517	1.7205E+03	0.0E+00	41.479	6.602	0.151	0.000	0.0	0.0	0.2	0.0	0.0	0.2
19	518	1.8801E+03	0.0E+00	43.360	6.901	0.145	0.000	0.0	5.9	0.0	0.0	7.1	0.0
20	519	2.7477E+03	0.0E+00	52.418	8.343	0.120	0.000	0.0	0.0	0.0	0.0	0.0	0.0
21	520	2.8594E+03	0.0E+00	53.474	8.511	0.118	0.000	0.0	0.0	0.0	0.0	0.0	0.0
22	521	2.9360E+03	0.0E+00	54.185	8.624	0.116	0.000	0.0	0.0	0.2	0.0	0.0	0.2
23	522	2.9607E+03	0.0E+00	54.412	8.660	0.115	0.000	0.0	0.0	0.0	0.0	0.0	0.0
24	523	3.7210E+03	0.0E+00	61.000	9.708	0.103	0.000	0.0	0.0	0.0	0.0	0.0	0.0
25	524	4.0094E+03	0.0E+00	63.320	10.078	0.099	0.000	5.0	0.0	0.0	5.1	0.0	0.0
26	525	4.0506E+03	0.0E+00	63.644	10.129	0.099	0.000	0.0	0.0	0.0	0.0	0.0	0.0
27	526	4.3548E+03	0.0E+00	65.991	10.503	0.095	0.000	0.0	0.3	0.0	0.0	0.3	0.0
28	527	4.7032E+03	0.0E+00	68.580	10.915	0.092	0.000	0.0	0.0	0.0	0.0	0.0	0.0
29	528	5.3848E+03	0.0E+00	73.381	11.679	0.086	0.000	0.0	0.0	0.0	0.0	0.0	0.0
30	529	6.9013E+03	0.0E+00	83.074	13.222	0.076	0.000	0.0	0.1	0.0	0.0	0.0	0.0
31	530	7.1760E+03	0.0E+00	84.711	13.482	0.074	0.000	0.0	0.0	0.4	0.0	0.0	0.3
32	531	8.3120E+03	0.0E+00	91.170	14.510	0.069	0.000	0.0	0.0	0.0	0.0	0.0	0.0
33	532	9.6840E+03	0.0E+00	98.407	15.662	0.064	0.000	0.0	0.0	0.0	0.0	0.0	0.0
34	533	9.7308E+03	0.0E+00	98.645	15.700	0.064	0.000	0.0	0.0	0.0	0.0	0.0	0.0
35	534	1.0108E+04	0.0E+00	100.538	16.001	0.062	0.000	0.0	0.9	0.0	0.0	0.9	0.0
36	535	1.0354E+04	0.0E+00	101.756	16.195	0.062	0.000	0.0	0.0	0.0	0.0	0.0	0.0
37	536	1.0407E+04	0.0E+00	102.016	16.236	0.062	0.000	0.0	0.1	0.0	0.0	0.1	0.0
38	537	1.1434E+04	0.0E+00	106.931	17.019	0.059	0.000	0.0	6.6	0.0	0.0	6.7	0.0
39	538	1.2522E+04	0.0E+00	111.903	17.810	0.056	0.000	0.0	0.4	0.0	0.0	0.4	0.0
40	539	1.3309E+04	0.0E+00	115.363	18.361	0.054	0.000	0.2	0.0	0.1	0.2	0.0	0.1
41	540	1.4215E+04	5.2E-15	119.227	18.976	0.053	0.000	0.0	0.2	0.0	0.0	0.2	0.0
42	541	1.4338E+04	0.0E+00	119.741	19.057	0.052	0.000	0.0	0.0	0.3	0.0	0.0	0.2
43	542	1.5242E+04	0.0E+00	123.459	19.649	0.051	0.000	0.0	0.7	0.0	0.0	0.6	0.0
44	543	1.6559E+04	6.8E-15	128.682	20.480	0.049	0.000	0.0	0.0	0.0	0.0	0.0	0.0
45	544	1.8963E+04	2.0E-14	137.707	21.917	0.046	0.000	0.1	0.0	0.1	0.1	0.0	0.1
46	545	2.1612E+04	5.6E-14	147.011	23.398	0.043	0.000	0.0	0.0	0.0	0.0	0.0	0.0
47	546	2.1935E+04	4.6E-14	148.103	23.571	0.042	0.000	0.0	0.0	0.0	0.0	0.0	0.0
48	547	2.2246E+04	0.0E+00	149.151	23.738	0.042	0.000	2.9	0.0	1.3	2.7	0.0	1.4
49	548	2.5168E+04	1.9E-14	158.644	25.249	0.040	0.000	0.0	0.0	0.8	0.0	0.0	1.1
50	549	2.6018E+04	2.9E-14	161.302	25.672	0.039	0.000	0.0	0.0	0.0	0.0	0.0	0.0
51	550	2.6942E+04	4.0E-14	164.140	26.124	0.038	0.000	0.1	0.0	37.9	0.1	0.0	40.6
52	551	2.7202E+04	9.2E-15	164.929	26.249	0.038	0.000	0.0	0.0	0.0	0.0	0.0	0.0
53	552	3.0303E+04	2.4E-13	174.077	27.705	0.036	0.000	0.0	0.0	0.1	0.0	0.0	0.1
54	553	3.6934E+04	2.4E-12	192.182	30.587	0.033	0.000	0.0	0.0	0.0	0.0	0.0	0.0
55	554	4.3775E+04	1.2E-10	209.225	33.299	0.030	0.000	0.0	0.0	0.0	0.0	0.0	0.0
56	555	4.6151E+04	4.7E-10	214.827	34.191	0.029	0.000	0.0	2.3	0.0	0.0	2.5	0.0
57	556	4.9702E+04	9.3E-08	222.938	35.482	0.028	0.000	0.0	0.0	0.0	0.0	0.0	0.0
58	557	5.1669E+04	5.1E-07	227.307	36.177	0.028	0.000	0.0	0.0	0.0	0.0	0.0	0.0
59	558	5.2293E+04	4.2E-07	228.677	36.395	0.027	0.000	0.0	0.4	0.0	0.0	0.3	0.0
60	559	5.3774E+04	3.4E-07	231.893	36.907	0.027	0.000	0.0	0.0	0.0	0.0	0.0	0.0

Composite model v2

Eigenvalues

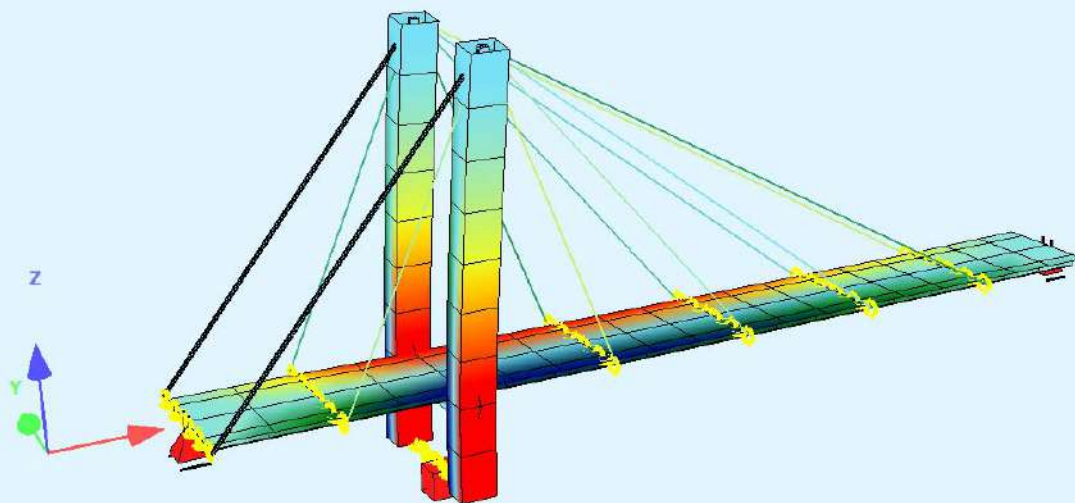
Eigenvalues

No.	LC	λ [rad ² /sec ²]	error [-]	ω [rad/sec]	f [Hz]	T [sec]	ξ [%]	Meff			participation		
								X[%]	Y[%]	Z[%]	X[%]	Y[%]	Z[%]
61		7.6942E+04	1.1E-04	277.384	44.147	0.023	$\Sigma(\%)^1$	94.3	89.0	86.1	97.2	96.9	93.2
62		7.7514E+04	9.4E-04	278.413	44.311	0.023							

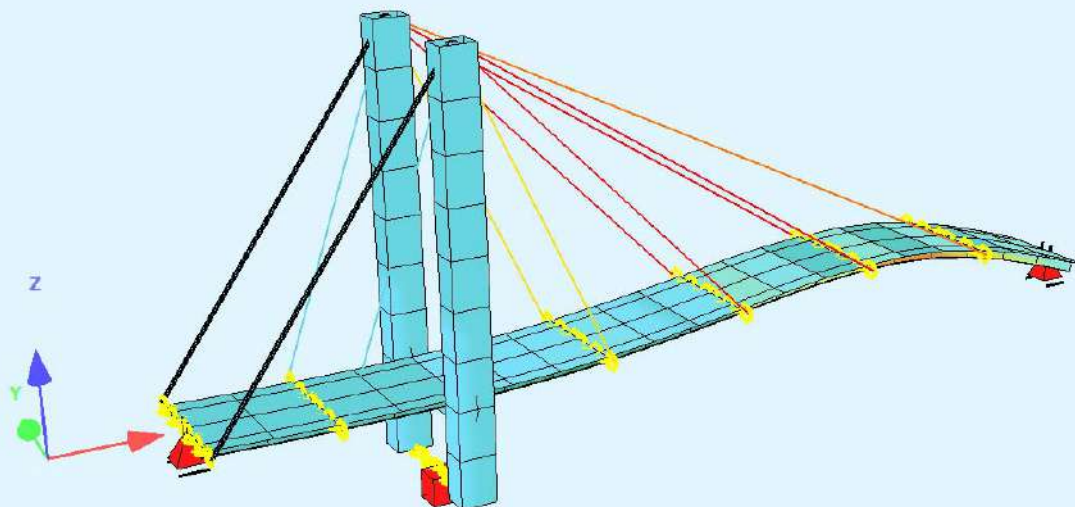
¹ Total effective mass / participation factors of activated masses in X-, Y- and Z-direction.

No.	eigenmode number	f	eigenfrequency
LC	load case	T	eigenperiod
λ	eigenvalue	ξ	modal damping ratio
error	relative eigenvalue error	Meff	effective modal mass in X-, Y- and Z-direction
ω	circular eigenfrequency	participation	participation factors in X-, Y- and Z-direction

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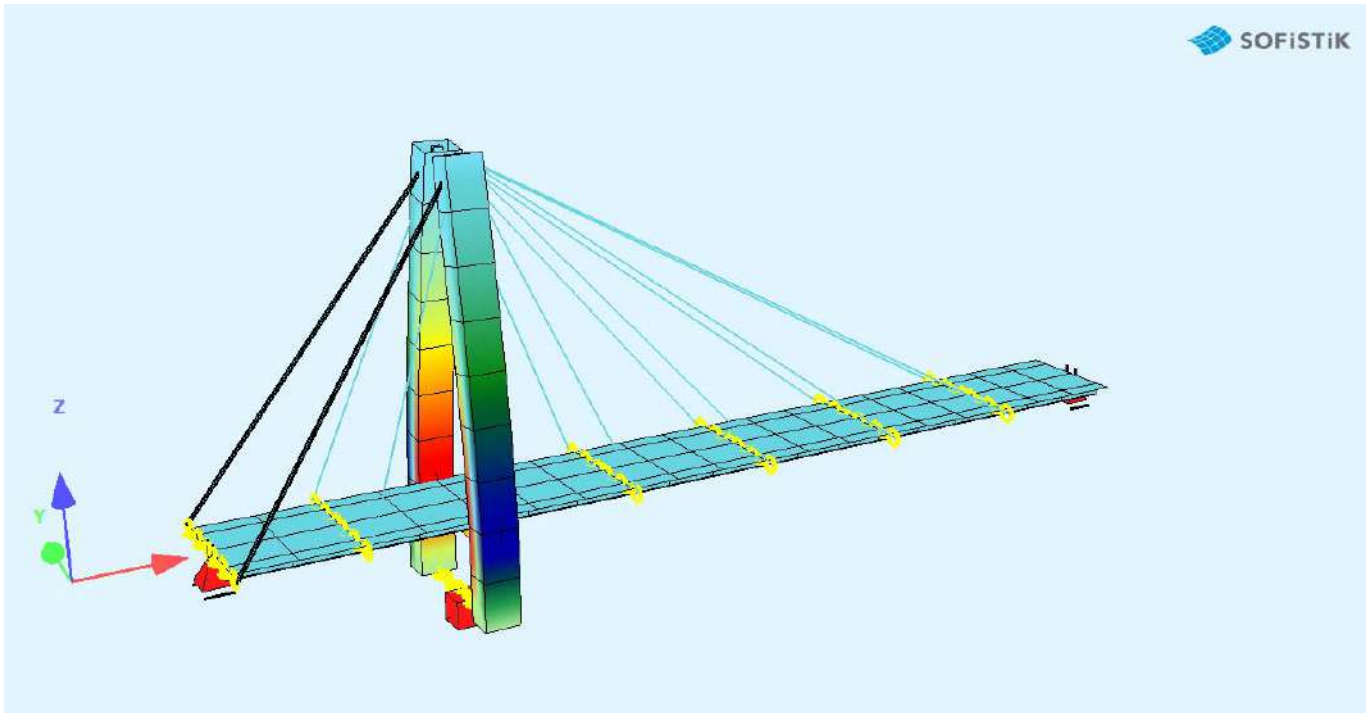
Mode 1: $f_n = 1.101$ Hz



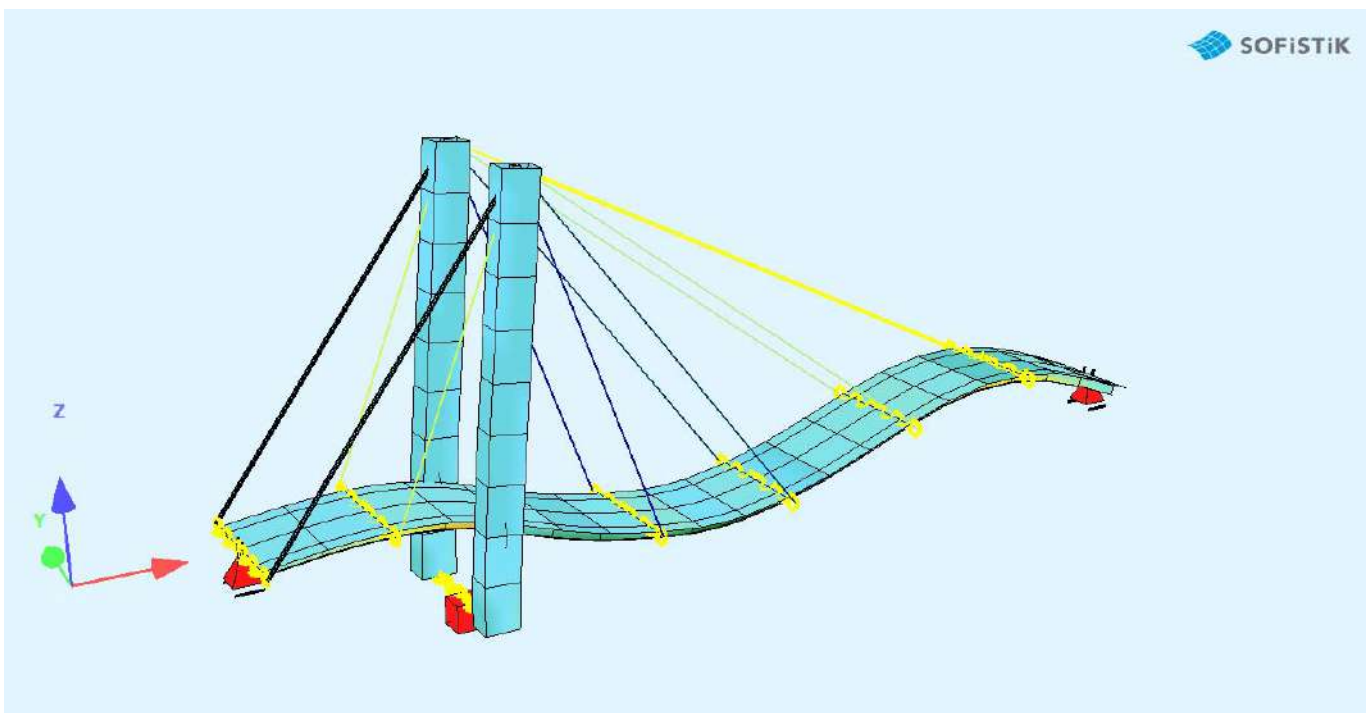
Composite model v2

Eigenvalues

Mode 2: $f_n = 1.187$ Hz

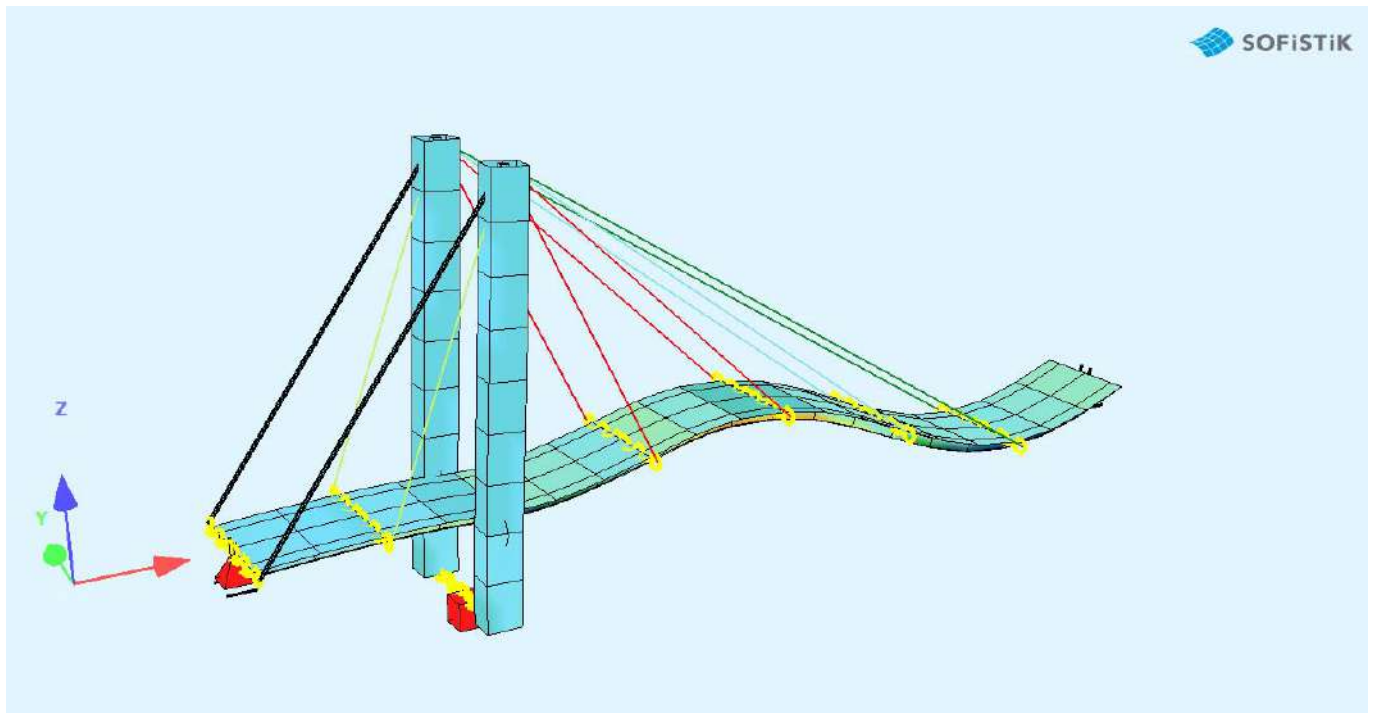


Mode 3: $f_n = 1.323$ Hz

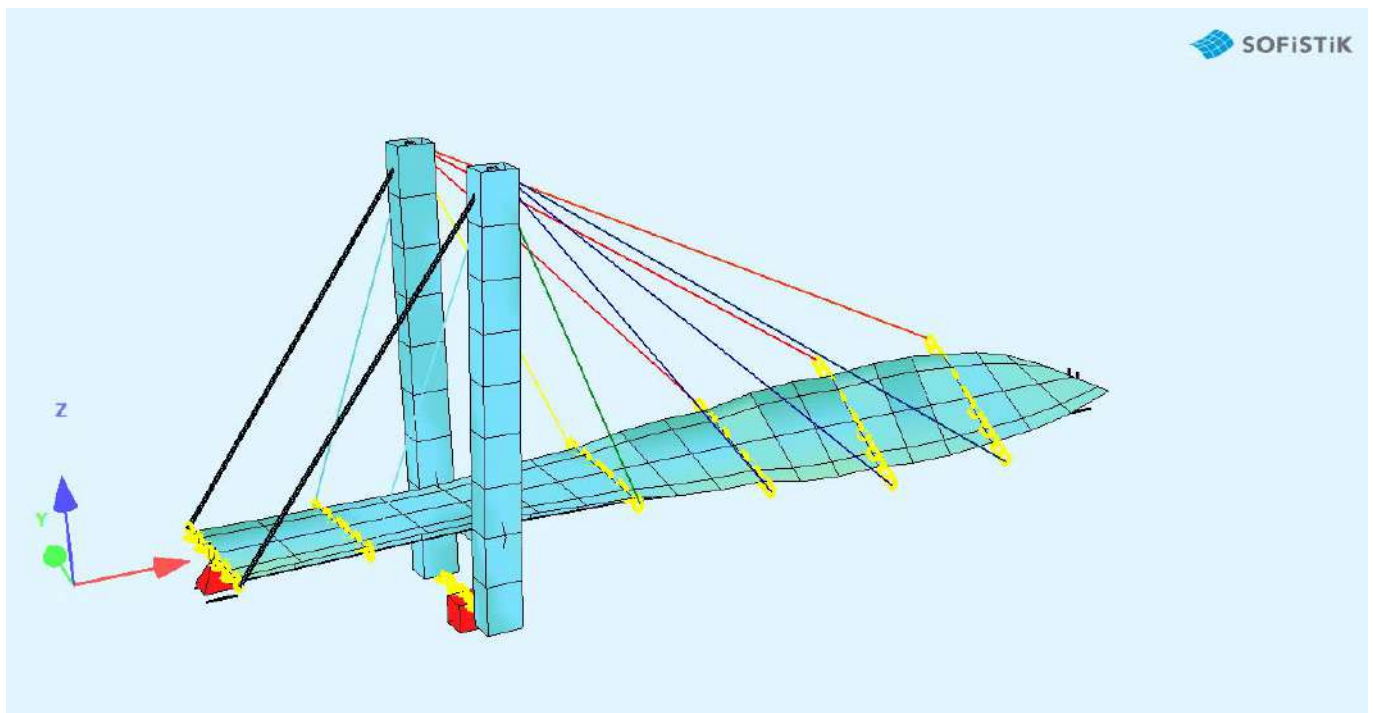


Mode 4: $f_n = 1.835$ Hz

Composite model v2
Eigenvalues



Mode 5: $f_n = 2.051$ Hz



Mode 6: $f_n = 2.793$ Hz

E.3. Sensitivity studies CFD analyses bridge decks

When performing a CFD analysis, it is important to reach mesh independency. This can be done by performing mesh sensitivity studies and was done for both the FRP bridge deck and the steel-concrete composite deck.

The sizes of the CFD simulation boxes have been defined previously. To find out what the most optimal and accurate mesh would be, the mesh size far away from the deck section was defined to 1.0 m for the FRP deck and 3.0 m for the steel-concrete deck. These were kept fixed. The mesh size of the deck section was refined:

- Mesh size deck section for FRP bridge deck: 0.25, 0.2, 0.15, 0.1, 0.05 and 0.02m. Progression factor of 1.4. The sensitivity study was carried out for an inclination angle of 0 degree and is considered applicable for other inclination angles.
- Mesh size deck for steel-concrete composite bridge deck: 0.25, 0.2, 0.15, 0.1, 0.09, 0.08, 0.07, 0.06, 0.05, 0.02 and 0.01. Progression factor of 1.1. The sensitivity study was carried out for an inclination angle of -5 degrees and is considered applicable for other inclination angles.

The results for the FRP deck are given in table E.1. For a mesh size near the deck of 0.1m or smaller, the errors with respect to the previous results are equal to 5% or smaller for all wind coefficients. Therefore, to spare computational time, the deck mesh size of 0.1m was chosen. The final mesh for the CFD analysis is given in figure E.1, for an inclination angle of 0 degree.

Mesh size general [m]	Mesh size section [m]	cd	cl	cm	Error cd [%]	Error cl [%]	Error cm [%]
1	0.25	0,0766	0,6343	0,1109			
1	0.2	0,0725	0,6625	0,1191	5,35	4,45	7,39
1	0.15	0,0761	0,585	0,1226	4,97	11,70	2,94
1	0.1	0,072	0,5879	0,124	5,39	0,50	1,14
1	0.05	0,0716	0,5981	0,1247	0,56	1,73	0,56
1	0.02	0,0692	0,5861	0,1311	3,35	2,01	5,13

Table E.1: Mesh sensitivity study FRP deck for CFD analysis

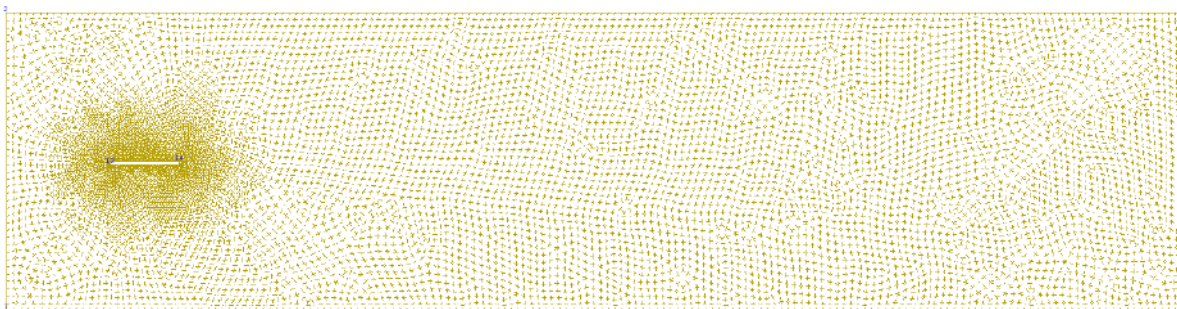


Figure E.1: Mesh CFD analysis for FRP deck, with inclination angle of 0 degree

The results for the steel-concrete deck are given in table E.2. It was performed for an inclination angle of -5 degrees. Due to the complex shape of the deck, the CFD results are very sensitive to the mesh size. From figures E.2, E.3 and E.4, it can be seen that the results are rather stable for smaller number of elements, but when the mesh is too fine nearby the section, the results become unstable. Typically the results start exhibiting excessive peak wind velocities at certain corners. The mesh size close to the section was finally taken as being equal to 0.05m, which is the finest mesh size before the results become unstable. In figure E.5, the final mesh is illustrated, with an inclination angle of the deck of 0 degree. It

can be noticed that the mesh is a lot finer than for the FRP deck, which is due to the complex shape.

Mesh size general [m]	Mesh size section [m]	cd	cl	cm	Nr of elements
3	0,25	1,3513	-0,6732	0,006	7197
3	0,2	1,5243	-0,7179	0,0108	8122
3	0,15	1,2849	-0,6547	-0,0067	9536
3	0,1	1,2801	-0,6915	-0,0056	12336
3	0,09	1,2428	-0,7183	-0,0023	12800
3	0,08	1,1987	-0,7374	-0,0001	14046
3	0,07	1,2276	-0,7626	0,0006	15423
3	0,06	1,4043	-0,6631	-0,0024	17014
3	0,05	1,4745	-0,5964	-0,0082	19264
3	0,02	3,145	-0,3515	-0,0466	37345
3	0,01	4,3752	-0,4817	-0,074	65130

Table E.2: Mesh sensitivity study steel-concrete deck for CFD analysis, with inclination angle of -5 degrees

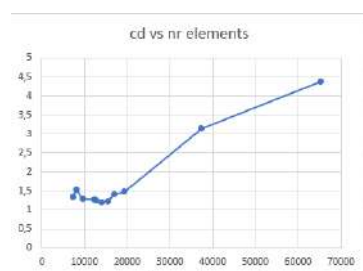


Figure E.2: Wind coefficients C_d vs number of mesh elements

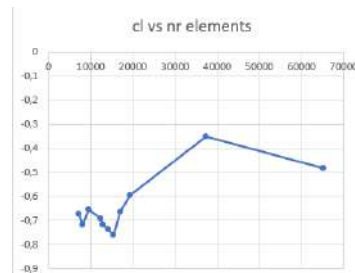


Figure E.3: Wind coefficients C_l vs number of mesh elements

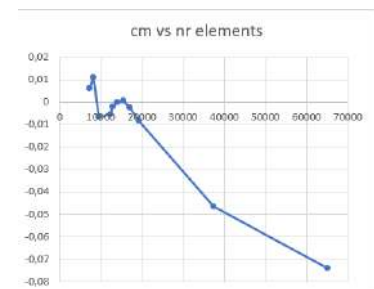


Figure E.4: Wind coefficients C_m vs number of mesh elements

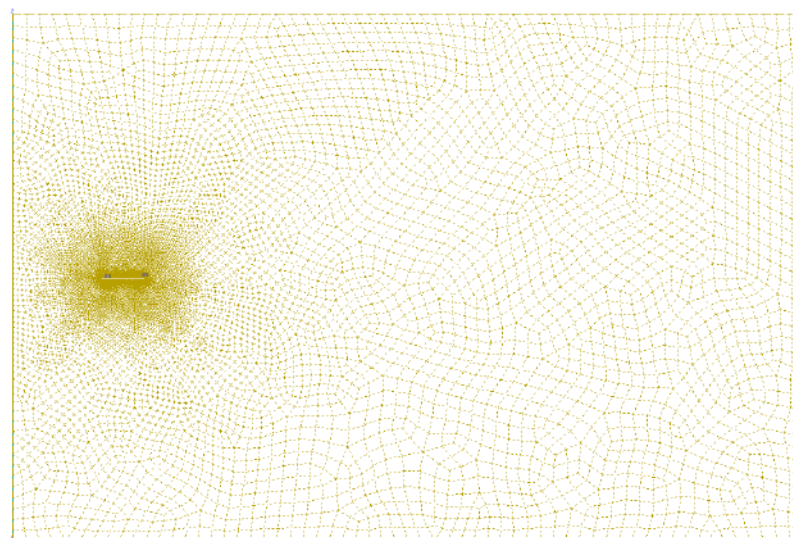


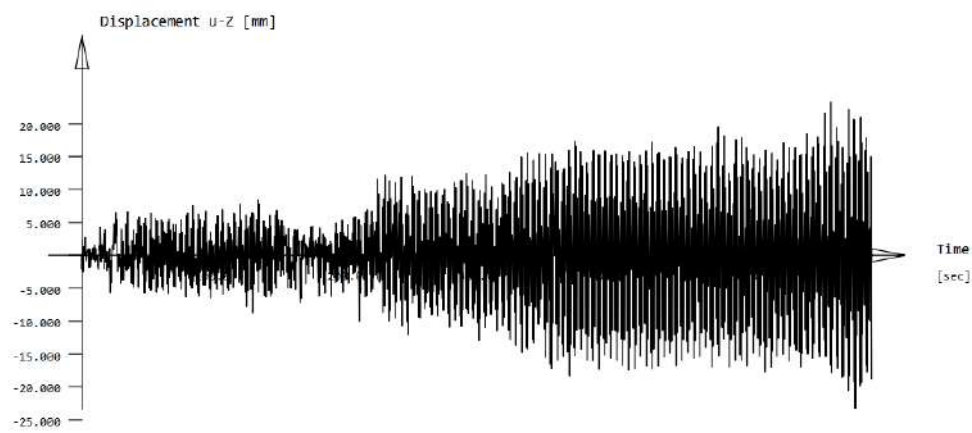
Figure E.5: Mesh CFD analysis for steel-concrete deck, with inclination angle of 0 degree

E.4. Sensitivity studies for time step integration

The results for the sensitivity analyses for time step integration for both bridges are presented in the following pages.

FRP cable-stayed bridge results

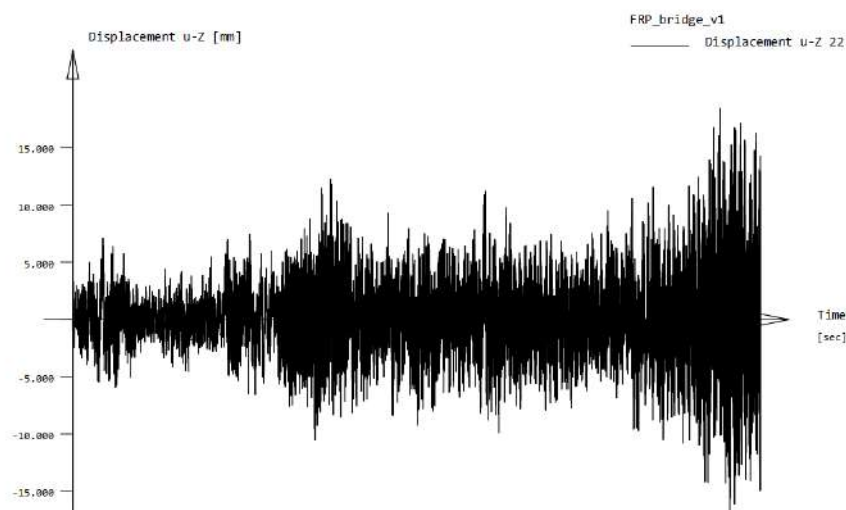
Step size: 0.5s



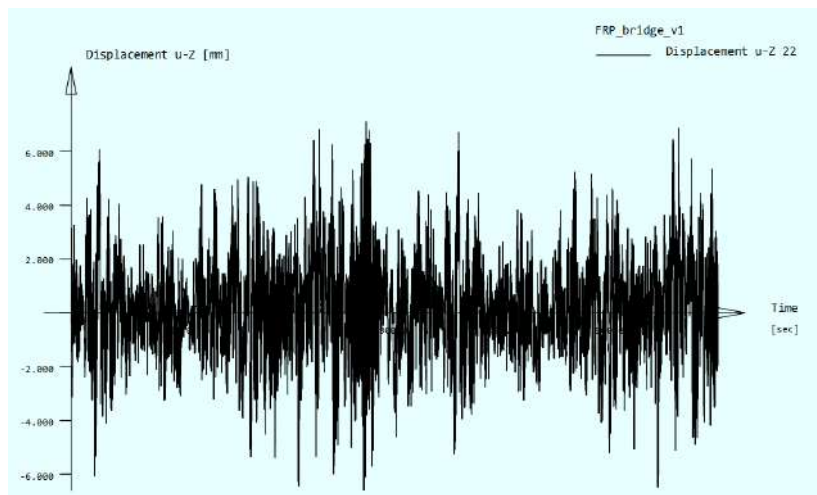
Step size: 0.2s



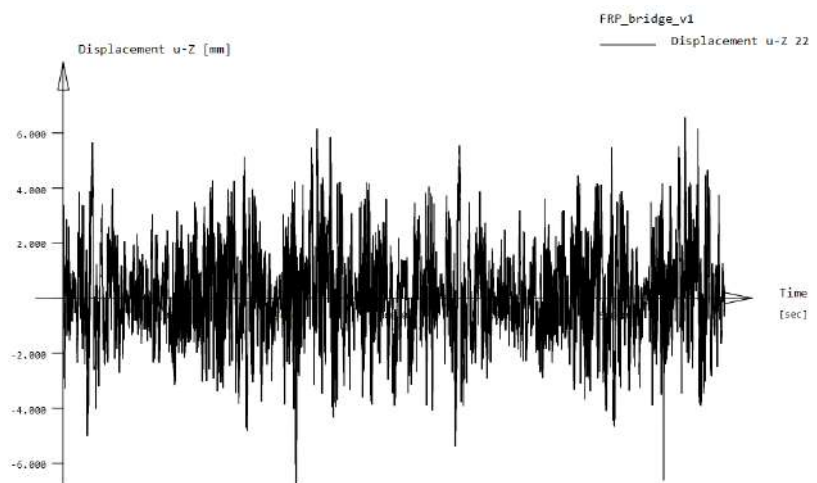
Step size: 0.1s



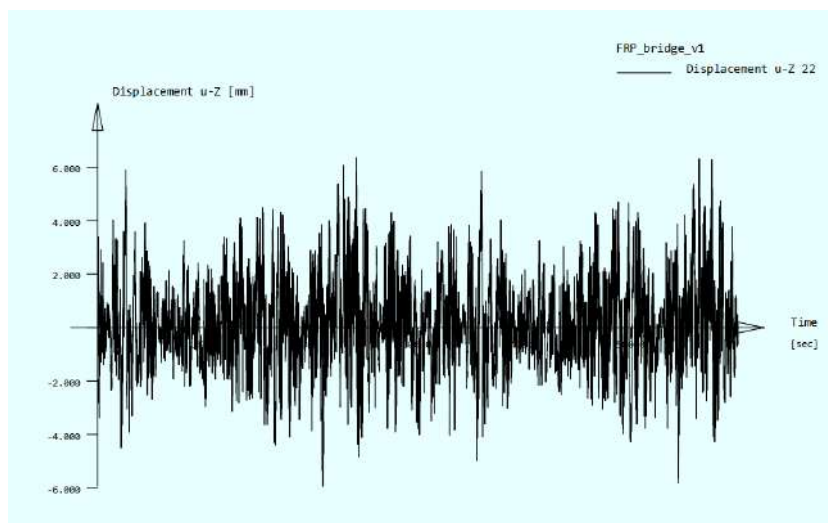
Step size: 0.05s



Step size: 0.02s

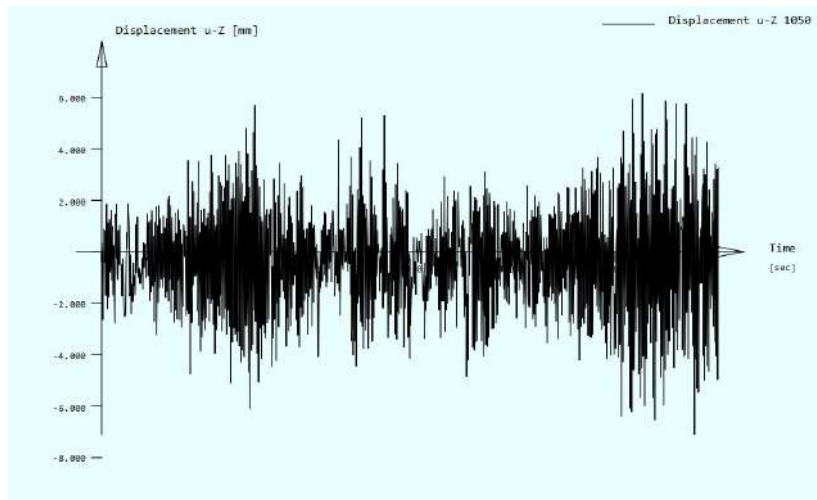


Step size: 0.01s

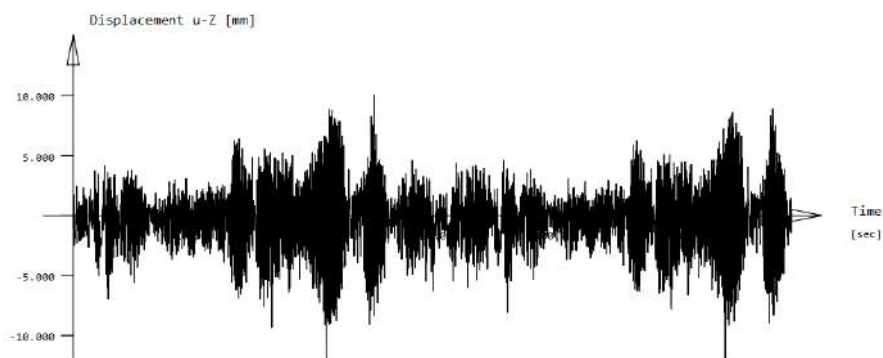


Steel-concrete cable-stayed bridge results

Step size: 0.5s



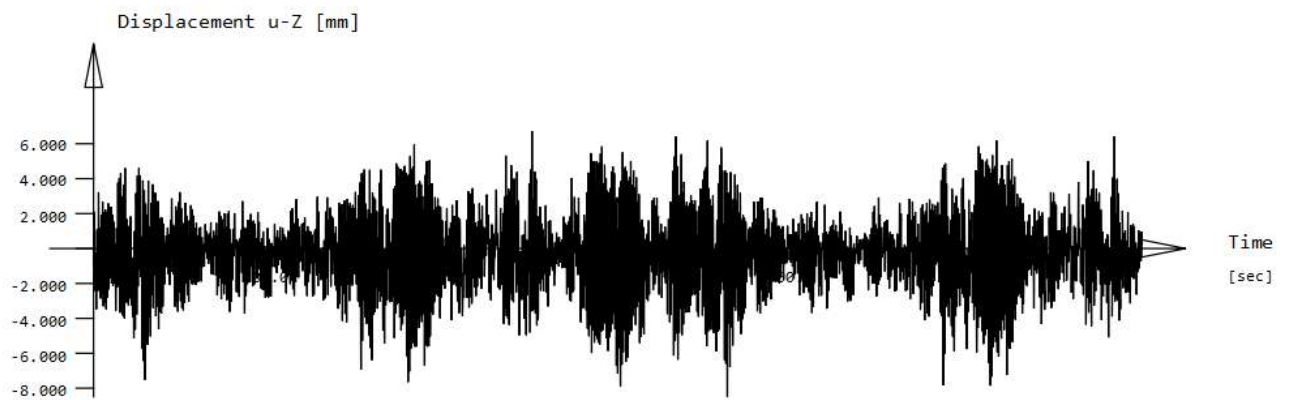
Step size: 0.2s



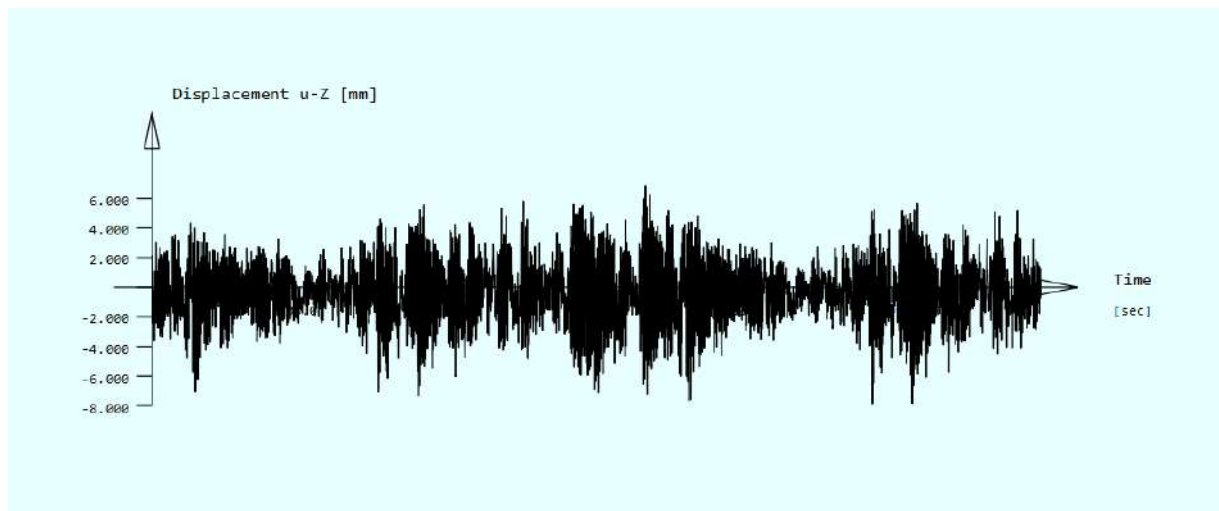
Step size: 0.1s



Step size: 0.05s



Step size: 0.02s



E.5. Wind analysis results FRP bridge - linear elastic

Actions

Actions

type	action	$\gamma\text{-u}, \gamma\text{-f}, \gamma\text{-a}$	partial safety factors for unfavourable/favourable/accidental
part	partition of the action	$\psi_0, \psi_1, \psi_2, \psi_{\text{inf}}$	combination coefficients
sup	superposition type		

Load Case 80 Basic wind

Factor forces and moments	1.000

Loads completely copied from load case 80 WIND BEAM TRUS CABL QUAD with factor 1.000 design

Windloading

Wind according to code NS EN-1991-1-4 (2009) Zone =Vb Terrain category III

Reference direction	(dxr, dyr, dzr) =	(-1.000,	0.000,	0.000		[[-	
Wind spectra	Karman	A1	A2	A3	B	C	D
	- Longitudinal	4.000	0.000	0.000	70.800	2.000	0.833
	- Lateral	4.000	0.000	3021.00	283.200	2.000	1.833
	- Vertical	4.000	0.000	3021.00	283.200	2.000	1.833
	coherences	c-lw	c-lv	c-lw	(ESDJKarman)		
	- Longitudinal	0.221	0.114	0.114			
	- Lateral	0.110	0.167	0.167			
	- Vertical	0.110	0.167	0.167			
Roughness profile	sea	D[km]		z0[m]	Hr[m]	Hi[m]	
	-200.000 [km]	-100.000		0.000	0.000	2428.776	

Windprofiles (according to design code or explicit specification)

h	v-mean [m/sec]	v-bce [m/sec]	q-ref [km/2nd]	t-lon [m/sec]	t-lat [m/sec]	t-ver [m/sec]	l-lon [m]	l-lat [m]	l-ver [m]	phase
7.000	16.18	29.23	0.53	5.13	0.00	0.00	38.841	0.000	0.000	0.000
11.000	18.59	31.94	0.64	5.19	0.00	0.00	51.168	0.000	0.000	0.000
16.000	20.47	34.15	0.73	5.26	0.00	0.00	64.362	0.000	0.000	0.000
20.000	21.56	35.45	0.79	5.32	0.00	0.00	73.675	0.000	0.000	0.000
25.000	22.63	36.74	0.84	5.37	0.00	0.00	84.044	0.000	0.000	0.000
30.000	23.35	37.60	0.88	5.41	0.00	0.00	92.411	0.000	0.000	0.000
33.000	23.97	38.34	0.92	5.45	0.00	0.00	99.987	0.000	0.000	0.000
38.000	24.66	39.14	0.96	5.48	0.00	0.00	108.970	0.000	0.000	0.000
42.000	25.16	39.71	0.99	5.50	0.00	0.00	115.827	0.000	0.000	0.000

11

~~Site overview and topology~~

8

~~Windspeed in the boundary layer~~

□

~~Distribution of speed, turbulences and effect. wave lengths~~

FRP_bridge_v1
Wind loads

Loads acting on Beam elements

Number	Type	fa[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fe[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fr[-]
10001	wind	1.000	0.00	16.18	0.00	1.000	0.00	17.11	0.00	0.000
10002	wind	1.000	0.00	17.11	0.00	1.000	0.00	19.06	0.00	0.000
10003	wind	1.000	0.00	19.06	0.00	1.000	0.00	20.70	0.00	0.000
10004	wind	1.000	0.00	20.70	0.00	1.000	0.00	21.91	0.00	0.000
10005	wind	1.000	0.00	21.91	0.00	1.000	0.00	22.86	0.00	0.000
10006	wind	1.000	0.00	22.86	0.00	1.000	0.00	23.66	0.00	0.000
10007	wind	1.000	0.00	23.66	0.00	1.000	0.00	24.36	0.00	0.000
10008	wind	1.000	0.00	24.36	0.00	1.000	0.00	24.97	0.00	0.000
10009	wind	1.000	0.00	16.18	0.00	1.000	0.00	17.11	0.00	0.000
10010	wind	1.000	0.00	17.11	0.00	1.000	0.00	19.06	0.00	0.000
10011	wind	1.000	0.00	19.06	0.00	1.000	0.00	20.70	0.00	0.000
10012	wind	1.000	0.00	20.70	0.00	1.000	0.00	21.91	0.00	0.000
10013	wind	1.000	0.00	21.91	0.00	1.000	0.00	22.86	0.00	0.000
10014	wind	1.000	0.00	22.86	0.00	1.000	0.00	23.66	0.00	0.000
10015	wind	1.000	0.00	23.66	0.00	1.000	0.00	24.36	0.00	0.000
10016	wind	1.000	0.00	24.36	0.00	1.000	0.00	24.97	0.00	0.000
40001	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
40002	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50001	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50002	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50003	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50004	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50005	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50006	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50007	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50008	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50009	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50010	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50011	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50012	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50013	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50014	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50015	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50016	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50017	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50018	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50019	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50020	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50021	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000

Loads acting on Cable elements

Number	Type	fa[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fe[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fr[-]
20001	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
20002	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
20003	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
20004	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
20005	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
20006	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30001	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30002	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30003	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30004	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30005	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30006	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30007	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30008	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30009	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30010	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000

FRP_bridge_v1
Wind loads

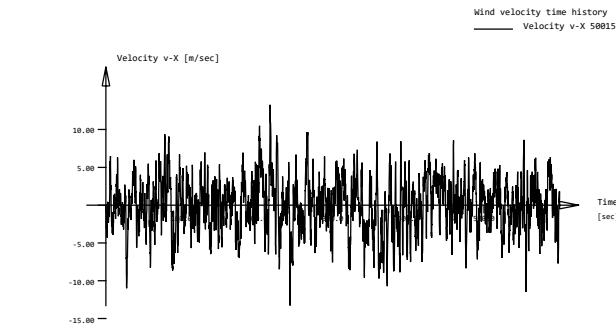
Loads acting on Cable elements

Number	Type	fa[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fe[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fr[-]
30011	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30012	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30013	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000
30014	wind	1.000	0.00	24.97	0.00	1.000	0.00	19.06	0.00	0.000

SOFISTIK AG - www.sofistik.de

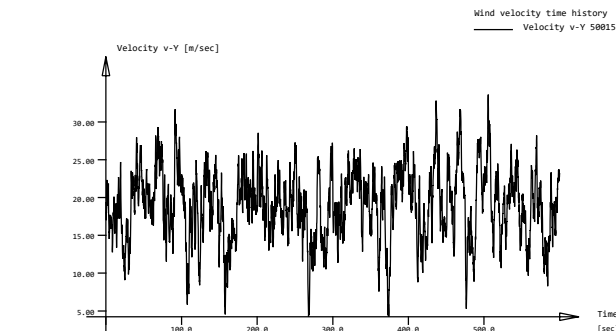
SOFISTIK AG - www.sofistik.de

FRP_bridge_v1
Wind velocity time history



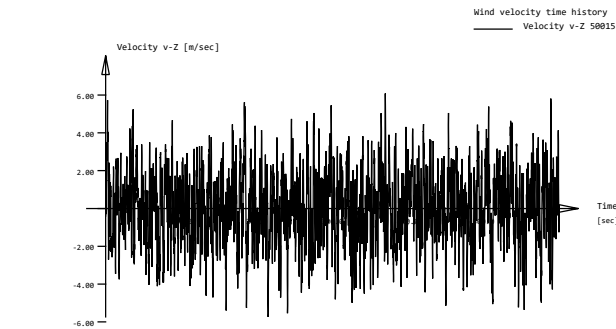
SOFISTIK AG - www.sofistik.de

Wind velocity time history



Wind velocity time history

FRP_bridge_v1
Wind velocity time history



SOFISTIK AG - www.sofistik.de

Wind velocity time history

FRP_bridge_v1

Integration of wind history

System- and Control Information

Control Information

Primary load case 90 t = 0.0000 [sec] +Opt:add_F +Opt:add_GS
Number of unknowns 226
unknowns per node 6
Number of timesteps 30000
Time-step 0.0200
Printing intervall 1
damping factor A 0.000E+00
damping factor B 0.000E+00
Integr.Parameter beta 0.25
Integr.Parameter delta 0.50
Integr.Parameter theta 1.00

Groups

Grp	Option	CS	Factor	Rayleigh-A [1/sec]	Rayleigh-B [sec]	ξ [o/o]	Wind
1	FULL		1.000	0.535138	0.000576	0.00	
2	FULL		1.000	0.080271	0.000086	0.00	
3	FULL		1.000	0.080271	0.000086	0.00	
4	FULL		1.000	0.535138	0.000576	0.00	
5	FULL		1.000	0.267569	0.000288	0.00	

CS construction stage
Factor Factor on stiffness ξ modal damping ratio
Rayleigh-A mass proportional damping ratio Wind options for dynamic wind loading

Beam Elements

Finite beam elements without intermediate sections

Shear deformations accounted for with nonconforming SOFISTIK-Timoshenko beam

Primary load case 90

Sum of masses and mass moments of inertia

Node	TM			RM			RMB
	X[t]	Y[t]	Z[t]	X[tm2]	Y[tm2]	Z[tm2]	
total ¹	1541.977	1541.977	1541.977	8.442E+02	3.820E+02	2.211E+03	-
	S[m] ²			RM(S) ³			
	30.855	0.000	8.887	2.322E+05	6.710E-03	-1.23E+04	
				6.710E-03	3.792E+05	7.474E-04	
				-1.23E+04	7.474E-04	2.504E+05	
active ¹	1541.977	1475.503	1475.503	8.133E+02	3.820E+02	2.211E+03	-
	S[m] ²			RM(S) ³			
	30.855	0.000	9.307	2.262E+05	6.710E-03	-1.28E+04	
				6.710E-03	3.995E+05	8.172E-04	
				-1.28E+04	8.172E-04	2.824E+05	

¹ sum of the total and the active nodal masses

² coordinates of the center of gravity

³ 3x3 rotational mass matrix at the center of gravity

TM translational masses in X-, Y- and Z-direction

RM rotational masses about X-, Y- and Z-axis

RMB warping mass

Processing

Load Cases

-- Loadcase 90

Node	PX	PY	PZ	MX	MY	MZ	Mb
	[kN]	[kN]	[kN]	[kNm]	[kNm]	[kNm]	[kNm2]
sum	0.0	192.6	1.2	2.09	0.00	-0.03	

Loads on kinematic constraints have been transferred to their master

Parameter of System of Equations

Number of unknowns 226 (Direct sparse Gauss-Solver)
Total entries 1871

FRP_bridge_v1

Integration of wind history

Parameter of System of Equations

Total entries after fill in 2987
Mass matrix 926 (consistent), incl. rotational masses
Damping matrix 1230 (consistent)

Results

Nodal Displacements

Node	u-X [mm]	Time [sec]	u-Y [mm]	Time [sec]	u-Z [mm]	Time [sec]
MAX	2.049		14.464		10.403	
MIN	-1.911		-0.336		-9.350	

Time history for u-Y

Node 1008 (E 127) u-Z MINIMUM = -4.008 MAXIMUM = 4.350 [mm]
Node 1008 (E 128) u-RX MINIMUM = -0.005 MAXIMUM = 0.055 [mrad]
Node 1008 (E 126) u-Y MINIMUM = 0.255 MAXIMUM = 1.922 [mm]
stored in database for DYNR with identification no 499

Time history for a-Z

Node 1008 (E 127) a-Z MINIMUM = -0.64 MAXIMUM = 0.74 [m/sec2]
stored in database for DYNR with identification no 499

Time history for u-Y

Node 15 (E 271) u-Z MINIMUM = -4.550 MAXIMUM = 6.185 [mm]
Node 15 (E 272) u-RX MINIMUM = -0.000 MAXIMUM = 0.068 [mrad]
Node 15 (E 270) u-Y MINIMUM = 0.263 MAXIMUM = 1.893 [mm]
stored in database for DYNR with identification no 500

Time history for a-Z

Node 15 (E 271) a-Z MINIMUM = -0.74 MAXIMUM = 0.63 [m/sec2]
stored in database for DYNR with identification no 500

Time history for u-Y

Node 21 (E 295) u-Z MINIMUM = -6.238 MAXIMUM = 8.891 [mm]
Node 21 (E 296) u-RX MINIMUM = 0.019 MAXIMUM = 0.195 [mrad]
Node 21 (E 294) u-Y MINIMUM = 0.121 MAXIMUM = 0.723 [mm]
stored in database for DYNR with identification no 501

Time history for a-Z

Node 21 (E 295) a-Z MINIMUM = -1.26 MAXIMUM = 1.30 [m/sec2]
stored in database for DYNR with identification no 501

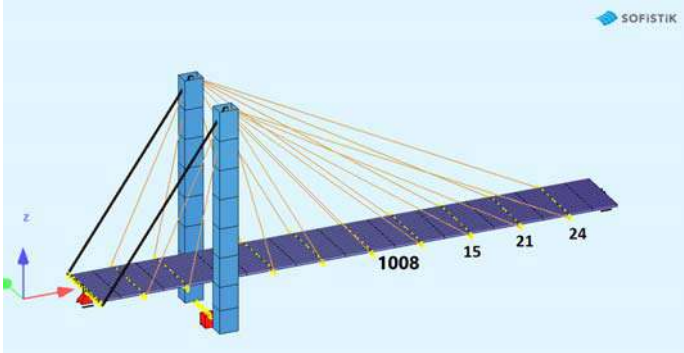
Time history for u-Y

Node 24 (E 307) u-Z MINIMUM = -1.971 MAXIMUM = 2.407 [mm]
Node 24 (E 308) u-RX MINIMUM = -0.008 MAXIMUM = 0.011 [mrad]
Node 24 (E 306) u-Y MINIMUM = 0.191 MAXIMUM = 1.730 [mm]
stored in database for DYNR with identification no 502

Time history for a-Z

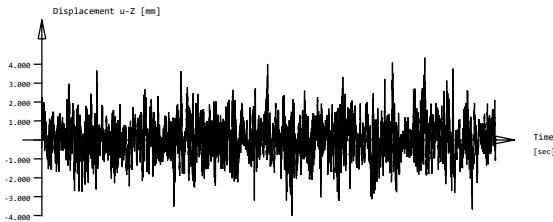
Node 24 (E 307) a-Z MINIMUM = -0.62 MAXIMUM = 0.56 [m/sec2]
stored in database for DYNR with identification no 502

FRP_bridge_v1



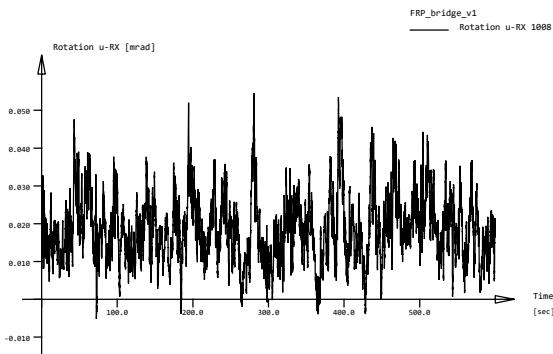
Node 1008 results

FRP_bridge_v1
Displacement u-Z 1008



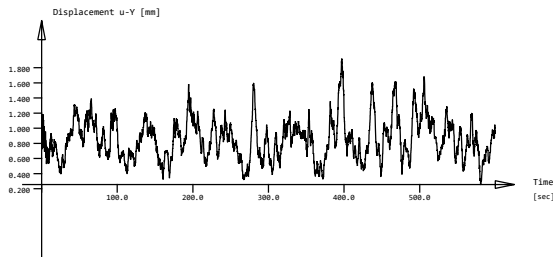
FRP_bridge_v1

FRP_bridge_v1



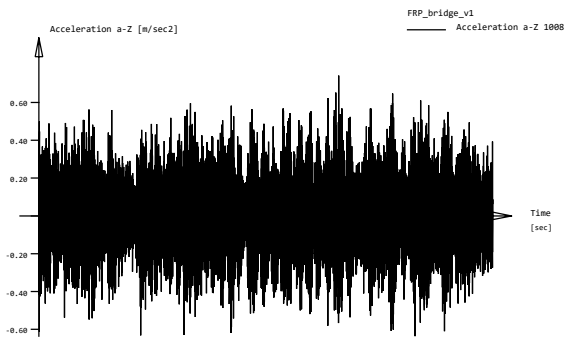
FRP_bridge_v1

FRP_bridge_v1
Displacement u-Y 1008



FRP_bridge_v1

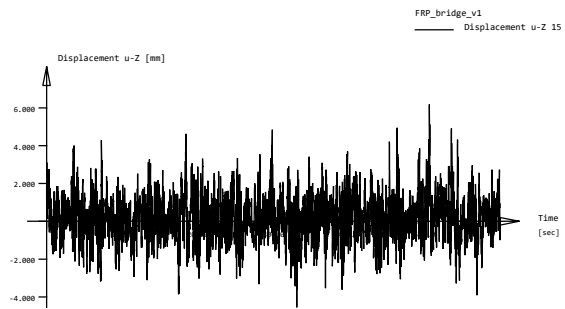
FRP_bridge_v1



FRP_bridge_v1
Node 15 results

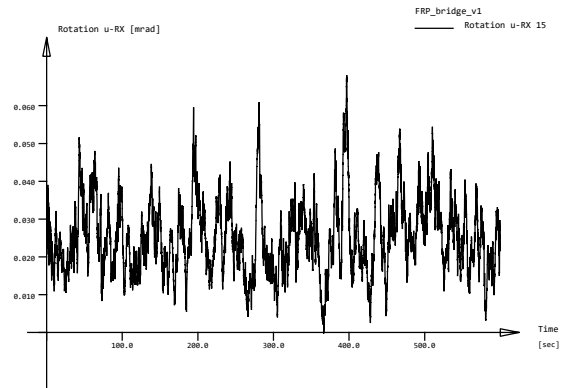
SOFISTIK AG - www.sofistik.de

FRP_bridge_v1



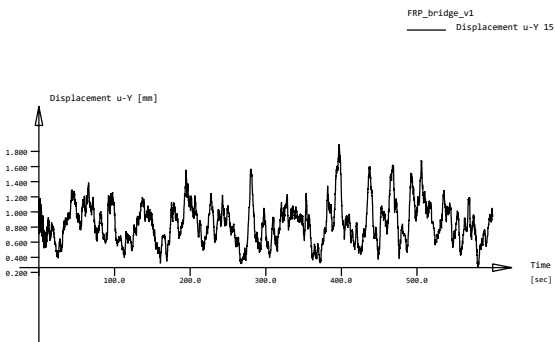
FRP_bridge_v1

SOFISTIK AG - www.sofistik.de



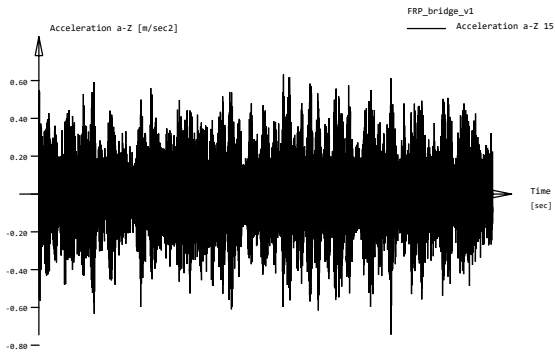
FRP_bridge_v1

FRP_bridge_v1



FRP_bridge_v1

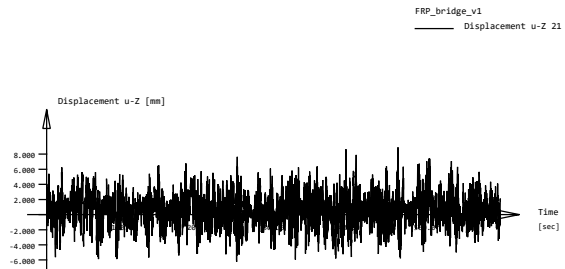
SOFISTIK AG - www.sofistik.de



FRP_bridge_v1

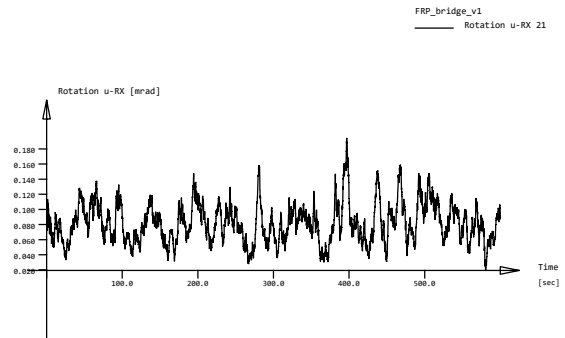
FRP_bridge_v1

Node 21 results



FRP_bridge_v1

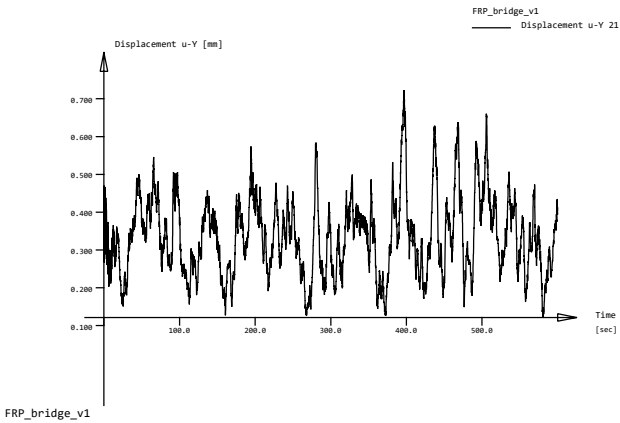
SOFISTIK AG - www.sofistik.de



FRP_bridge_v1

FRP_bridge_v1

SOFISTIK AG - www.sofistik.de



FRP_bridge_v1

Acceleration a-Z [m/sec2]

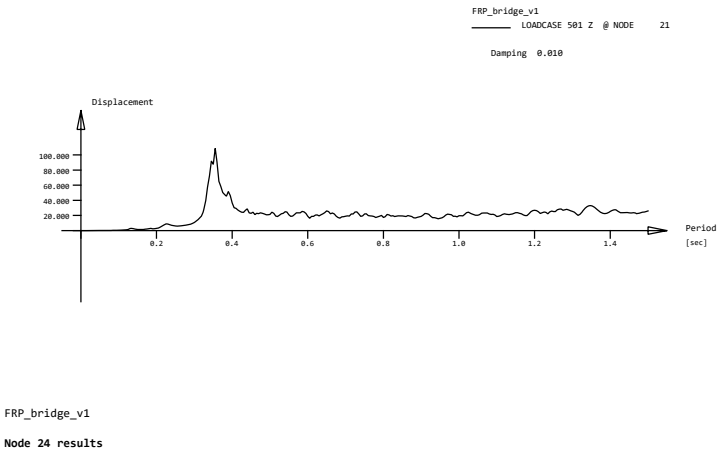
Time [sec]

FRP_bridge_v1 Acceleration a-Z 21

FRP_bridge_v1

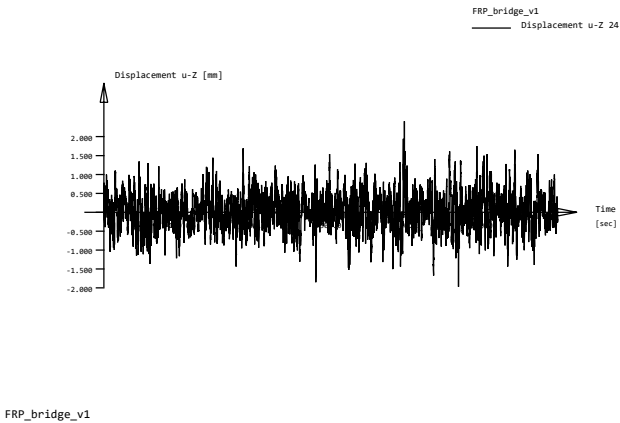
FRP_bridge_v1

SOFISTIK AG - www.sofistik.de



FRP_bridge_v1

SOFISTIK AG - www.sofistik.de



FRP_bridge_v1

Rotation u-RX [mrad]

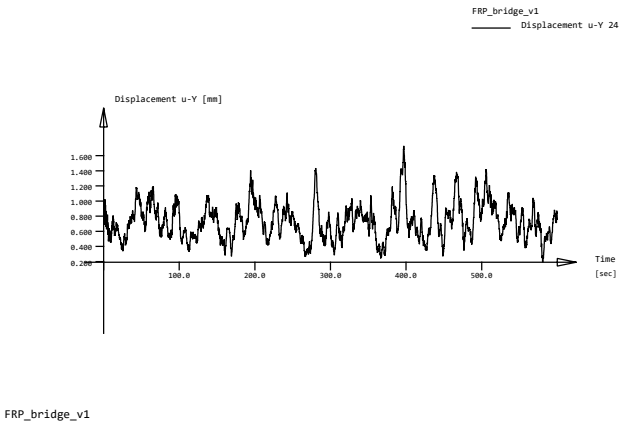
Time [sec]

FRP_bridge_v1 Rotation u-RX 24

FRP_bridge_v1

FRP_bridge_v1

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FRP_bridge_v1

Acceleration a-Z [m/sec2]

Time [sec]

FRP_bridge_v1 Acceleration a-Z 24

FRP_bridge_v1

E.6. Wind analysis results steel-concrete composite bridge - linear elastic

Composite model v2
Wind loads

Actions

type	part	sup	Designation	y-u	y-f	y-a	ψ ₀	ψ ₁	ψ ₂	ψ _{inf}
G 1	G	perm	dead load g1	1.35	1.00	1.00	1.00	1.00	1.00	1.00
G 2	G	perm	ADL	1.35	1.00	1.00	1.00	1.00	1.00	1.00
P	P	perm	prestressing	1.00	1.00	1.00	1.00	1.00	1.00	1.00
W	Q	excl	wind loading	1.50	0.00	1.00	0.60	0.20	0.00	0.60
GR_T	Q_1	excl	gr1a LM1	1.35	0.00	1.00	0.75	0.75	0.00	0.80
GR_U	Q_1	excl	gr1a LM1	1.35	0.00	1.00	0.40	0.40	0.00	0.80

Reliability factor	Kfi	1.000
Reduction factor	xsi	0.890

type action y-u,y-f,y-a partial safety factors for unfavourable/favourable/accidental
part partition of the action ψ₀,ψ₁,ψ₂,ψ_{inf} combination coefficients
sup superposition type

Load Case 80 Basic wind

Factor forces and moments 1.000
Loads completely copied from load case 80 WIND BEAM TRUS CABL QUAD with factor 1.000 design

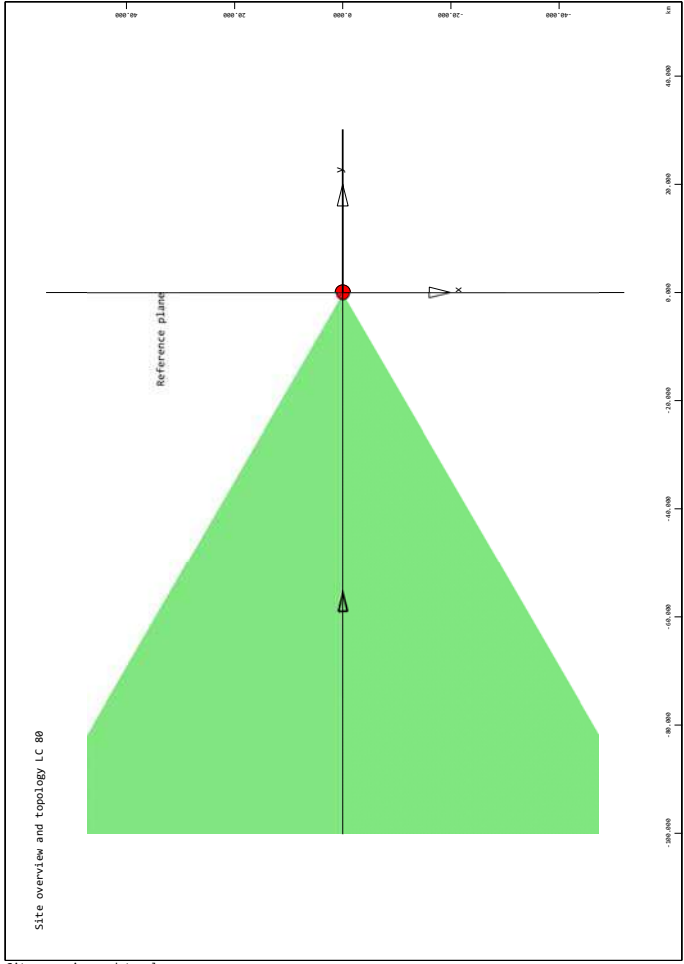
Windloading

Wind according to code	NS	EN-1991-1-4 (2009)	Zone	=Vb Terrain category III			
atmospheric wind	v	59.91	[m/sec]	Air density	p	1.25	[kg/m3]
Mean wind speed	vref	24.00	[m/sec]				
Surface height	hg	-12.000	[m]	Latitude	0	45.5	[°]
Wind direction	(dx, dy, dz) =	(0.000, 1.000, 0.000)					[-]
Reference point	(x-0, y-0, z-0) =	(10.000, 0.000, 0.000)					[m]
Reference direction	(dxr, dyr, dzr) =	(-1.000, 0.000, 0.000)					[-]
Wind spectra	Karman	A1	A2	A3	B	C	D
- Longitudinal		4.000	0.000	0.000	70.800	2.000	0.833
- Lateral		4.000	0.000	3021.00	283.200	2.000	1.833
- Vertical		4.000	0.000	3021.00	283.200	2.000	1.833
coherences	c-iu c-iv c-iw (ESDU/Karman)						
- Longitudinal		0.221	0.114	0.114			
- Lateral		0.110	0.167	0.167			
- Vertical		0.110	0.167	0.167			
Roughness profile	sea	D[km]	z0[m]	Hr[m]	Hi[m]		
	-200.000	[km]	0.300	0.000	2420.776		

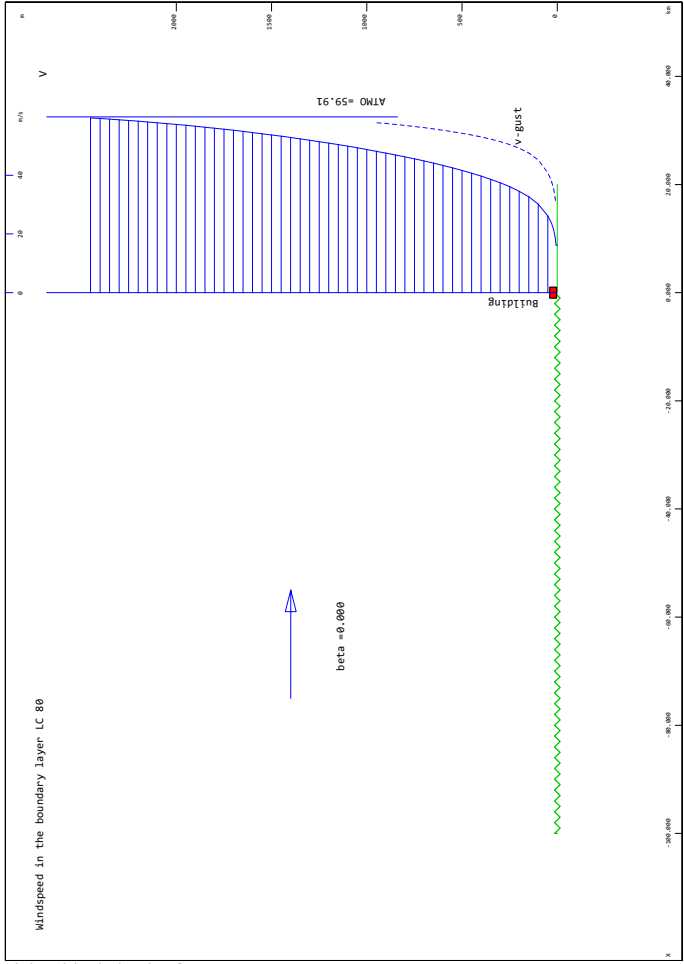
Windprofiles (according to design code or explicit specification)

h	v-mean	v-boe	q-ref	t-lon	t-lat	t-ver	l-lon	l-lat	l-ver	phase
[m]	[m/sec]	[m/sec]	[kN/m2]	[m/sec]	[m/sec]	[m/sec]	[m]	[m]	[m]	[-]
7.000	16.18	29.23	0.53	5.13	0.00	0.00	38.841	0.000	0.000	0.000
11.000	18.59	31.94	0.64	5.19	0.00	0.00	51.168	0.000	0.000	0.000
16.000	20.47	34.15	0.73	5.26	0.00	0.00	64.302	0.000	0.000	0.000
20.000	21.56	35.45	0.79	5.32	0.00	0.00	73.675	0.000	0.000	0.000
25.000	22.63	36.74	0.84	5.37	0.00	0.00	84.414	0.000	0.000	0.000
29.000	23.35	37.60	0.88	5.41	0.00	0.00	92.411	0.000	0.000	0.000
33.000	23.97	38.34	0.92	5.45	0.00	0.00	99.987	0.000	0.000	0.000
38.000	24.66	39.14	0.96	5.48	0.00	0.00	108.970	0.000	0.000	0.000
42.000	25.16	39.71	0.99	5.50	0.00	0.00	115.827	0.000	0.000	0.000

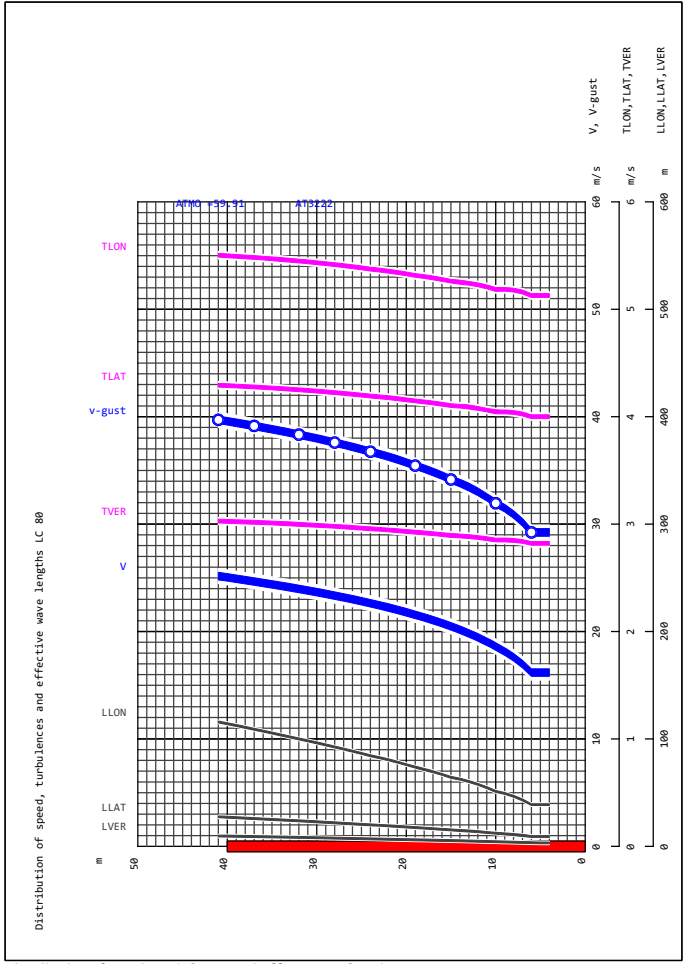
Composite model v2
Wind loads



Composite model v2
Wind loads



Composite model v2
Wind loads



Composite model v2
Wind loads

Loads acting on Beam elements

Number	Type	fa [-]	fx [m/sec]	fy [m/sec]	yz [m/sec]	fe [-]	fx [m/sec]	fy [m/sec]	yz [m/sec]	fr [-]
10001	wind	1.000	0.00	16.18	0.00	1.000	0.00	16.97	0.00	0.000
10002	wind	1.000	0.00	16.97	0.00	1.000	0.00	19.06	0.00	0.000
10003	wind	1.000	0.00	19.06	0.00	1.000	0.00	20.47	0.00	0.000
10004	wind	1.000	0.00	20.47	0.00	1.000	0.00	21.56	0.00	0.000
10005	wind	1.000	0.00	21.56	0.00	1.000	0.00	22.44	0.00	0.000
10006	wind	1.000	0.00	22.44	0.00	1.000	0.00	23.18	0.00	0.000
10007	wind	1.000	0.00	23.18	0.00	1.000	0.00	23.82	0.00	0.000
10008	wind	1.000	0.00	23.82	0.00	1.000	0.00	24.40	0.00	0.000
10009	wind	1.000	0.00	24.40	0.00	1.000	0.00	24.92	0.00	0.000
10010	wind	1.000	0.00	16.18	0.00	1.000	0.00	16.97	0.00	0.000
10011	wind	1.000	0.00	16.97	0.00	1.000	0.00	19.06	0.00	0.000
10012	wind	1.000	0.00	19.06	0.00	1.000	0.00	20.47	0.00	0.000
10013	wind	1.000	0.00	20.47	0.00	1.000	0.00	21.56	0.00	0.000
10014	wind	1.000	0.00	21.56	0.00	1.000	0.00	22.44	0.00	0.000
10015	wind	1.000	0.00	22.44	0.00	1.000	0.00	23.18	0.00	0.000
10016	wind	1.000	0.00	23.18	0.00	1.000	0.00	23.82	0.00	0.000
10017	wind	1.000	0.00	23.82	0.00	1.000	0.00	24.40	0.00	0.000
10018	wind	1.000	0.00	24.40	0.00	1.000	0.00	24.92	0.00	0.000
40001	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
40002	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50001	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50002	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50003	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50004	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50005	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50006	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50007	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50008	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50009	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50010	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50011	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50012	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50013	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50014	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50015	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50016	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50017	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50018	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000
50019	wind	1.000	0.00	19.06	0.00	1.000	0.00	19.06	0.00	0.000

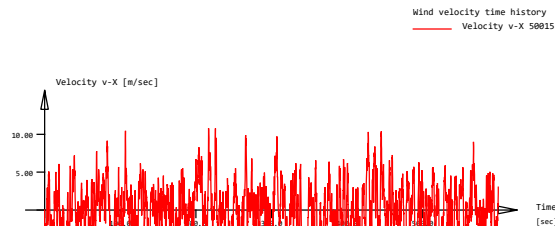
Loads acting on Cable elements

Number	Type	fa [-]	fx [m/sec]	fy [m/sec]	yz [m/sec]	fe [-]	fx [m/sec]	fy [m/sec]	yz [m/sec]	fr [-]
20001	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
20002	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
20003	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
20004	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
30001	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
30002	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
30003	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
30004	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
30005	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
30006	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
30007	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000
30008	wind	1.000	0.00	24.92	0.00	1.000	0.00	19.06	0.00	0.000

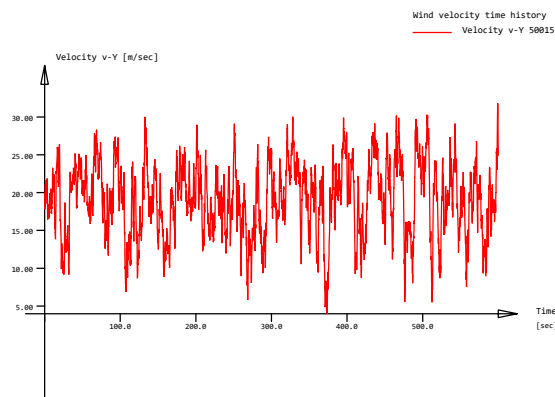
Load Case 90

Factor forces and moments

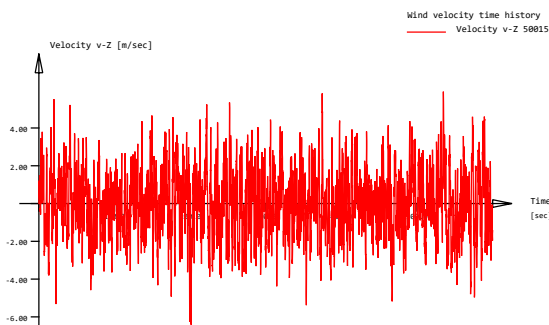
1.000

Composite model v2
Wind velocity time history

Wind velocity time history



Wind velocity time history

Composite model v2
Wind velocity time history

Wind velocity time history

Composite model v2
Integration of wind history

System- and Control Information

Control Information

Primary load case	90	t = 0.0000 [sec]	+Opt: add_F +Opt: add_GS
Number of unknowns	224		
unknowns per node	6		
Number of timesteps	12000		
Time-step	0.0500		
Printing intervall	1		
damping factor A	0.000E+00		
damping factor B	0.000E+00		
Integr.Parameter beta	0.25		
Integr.Parameter delta	0.50		
Integr.Parameter theta	1.00		

Groups

Grp	Option	CS	Factor	Rayleigh-A [1/sec]	Rayleigh-B [sec]	ξ [o/o]	Wind
1	FULL		1.000	0.229881	0.001230	0.00	
2	FULL		1.000	0.034482	0.000184	0.00	
3	FULL		1.000	0.034482	0.000184	0.00	
4	FULL		1.000	0.229881	0.001230	0.00	
5	FULL		1.000	0.172411	0.000922	0.00	

CS	construction stage	Rayleigh-B	stiffness proportional damping ratio
Factor	Factor on stiffness	ξ	modal damping ratio
Rayleigh-A	mass proportional damping ratio	Wind	options for dynamic wind loading

Beam Elements

Finite beam elements without intermediate sections

Shear deformations accounted for with nonconforming SOFISTIK-Timoshenko beam

Primary load case 90

Sum of masses and mass moments of inertia

Node	TM			RM			RMB [tm2]
	X[t]	Y[t]	Z[t]	X[tm2]	Y[tm2]	Z[tm2]	
total ¹	2605.768	2605.768	2605.768	7.183E+03	5.460E+02	7.770E+03	-
	S[m] ²			RM(S) ³			
	39.431	0.000	5.202	2.750E+05	7.042E-03	-1.28E+05	
				7.042E-03	1.316E+06	1.526E-04	
				-1.28E+05	1.526E-04	1.144E+06	
active ¹	2605.768	2482.745	2482.745	6.803E+03	5.460E+02	7.770E+03	-
	S[m] ²			RM(S) ³			
	39.431	0.000	5.460	2.715E+05	7.042E-03	-1.28E+05	
				7.042E-03	1.237E+06	1.680E-04	
				-1.28E+05	1.680E-04	1.072E+06	

¹ sum of the total and the active nodal masses² coordinates of the center of gravity³ 3x3 rotational mass matrix at the center of gravity

TM translational masses in X-, Y- and Z-direction

RM rotational masses about X-, Y- and Z-axis

RMB warping mass

Processing

Load Cases

-- Loadcase 90

Node	PX	PY	PZ	MX	MY	MZ	Mb
	[kN]	[kN]	[kN]	[kNm]	[kNm]	[kNm]	[kNm2]
sum	0.0	250.0	-15.5	-4.35	0.02	0.07	

Loads on kinematic constraints have been transferred to their master

Parameter of System of Equations

Number of unknowns	224 (Direct sparse Gauss-Solver)
Total entries	1733

Composite model v2
Integration of wind history

Parameter of System of Equations

Total entries after fill in 2878
Mass matrix 872 (consistent), incl. rotational masses
Damping matrix 1123 (consistent)

Results

Nodal Displacements

Node	u-X [mm]	Time [sec]	u-Y [mm]	Time [sec]	u-Z [mm]	Time [sec]
MAX	2.033		14.715		10.681	
MIN	-1.822		-0.495		-12.155	

Time history for u-Z

Node 11 (E 14) u-Y MINIMUM = 0.083 MAXIMUM = 0.838 [mm]
Node 11 (E 15) u-Z MINIMUM = -11.909 MAXIMUM = 10.288 [mm]
stored in database for DYNR with identification no 200

Time history for a-Z

Node 11 (E 15) a-Z MINIMUM = -0.62 MAXIMUM = 0.63 [m/sec2]
stored in database for DYNR with identification no 200

Time history for u-Z

Node 1012 (E 124) u-Y MINIMUM = 0.061 MAXIMUM = 0.621 [mm]
Node 1012 (E 125) u-Z MINIMUM = -11.177 MAXIMUM = 10.471 [mm]
stored in database for DYNR with identification no 201

Time history for a-Z

Node 1012 (E 125) a-Z MINIMUM = -0.72 MAXIMUM = 0.72 [m/sec2]
stored in database for DYNR with identification no 201

Time history for u-Z

Node 1011 (E 118) u-Y MINIMUM = 0.073 MAXIMUM = 0.738 [mm]
Node 1011 (E 119) u-Z MINIMUM = -12.015 MAXIMUM = 10.681 [mm]
stored in database for DYNR with identification no 202

Time history for a-Z

Node 1011 (E 119) a-Z MINIMUM = -0.66 MAXIMUM = 0.68 [m/sec2]
stored in database for DYNR with identification no 202

Time history for u-RX

Node 1011 (E 120) u-RX MINIMUM = 0.003 MAXIMUM = 0.092 [mrad]
stored in database for DYNR with identification no 202

Time history for u-Z

Node 9 (E 8) u-Y MINIMUM = 0.048 MAXIMUM = 0.487 [mm]
Node 9 (E 9) u-Z MINIMUM = -9.709 MAXIMUM = 9.157 [mm]
stored in database for DYNR with identification no 203

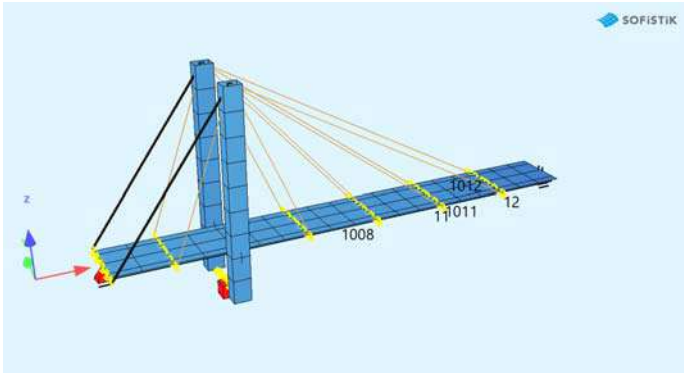
Time history for a-Z

Node 9 (E 9) a-Z MINIMUM = -0.67 MAXIMUM = 0.64 [m/sec2]
stored in database for DYNR with identification no 203

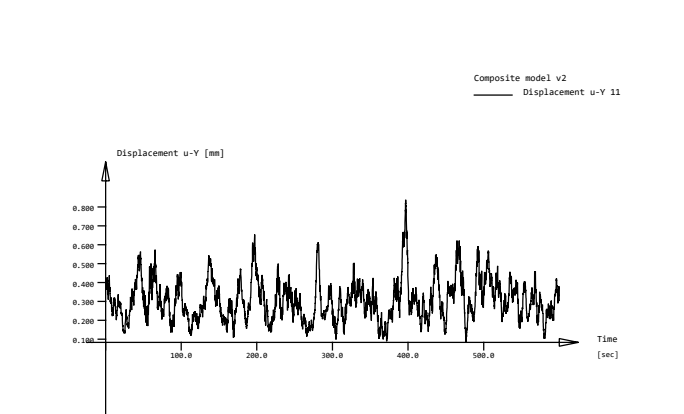
Time history for u-Y

Node 1008 (E 100) u-Y MINIMUM = 0.103 MAXIMUM = 1.030 [mm]
stored in database for DYNR with identification no 204

Composite model v2

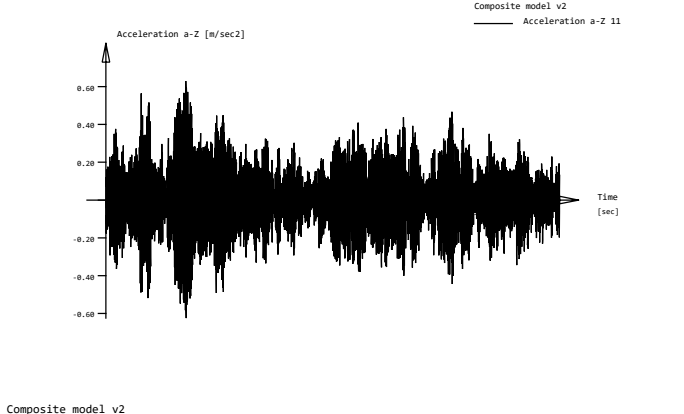
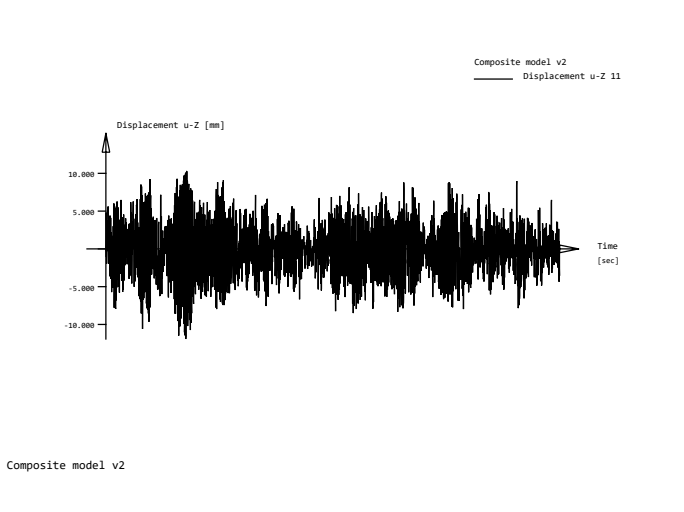


Results node 11



Composite model v2

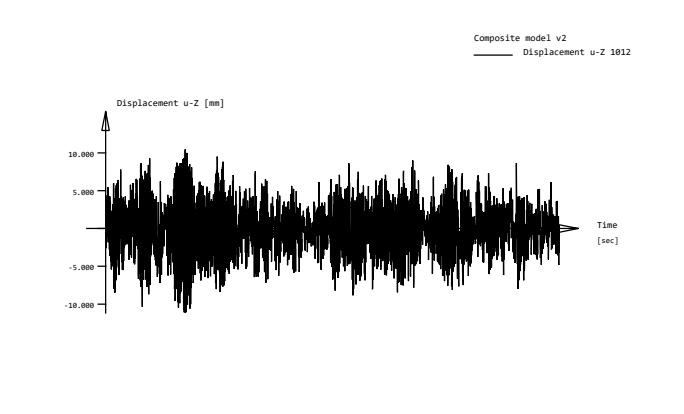
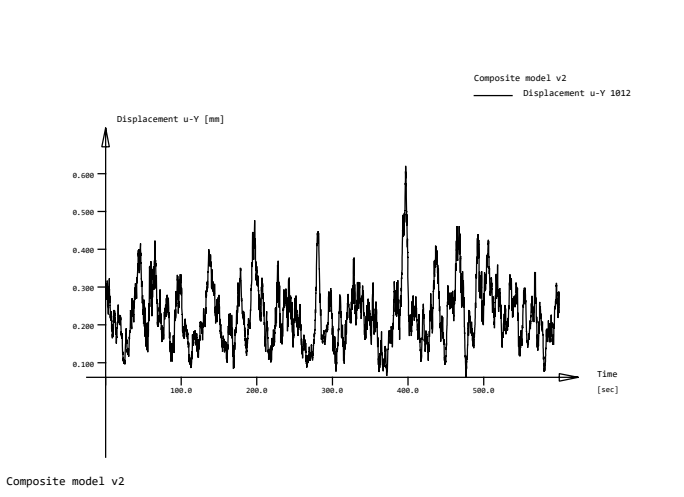
Composite model v2



Composite model v2

Composite model v2

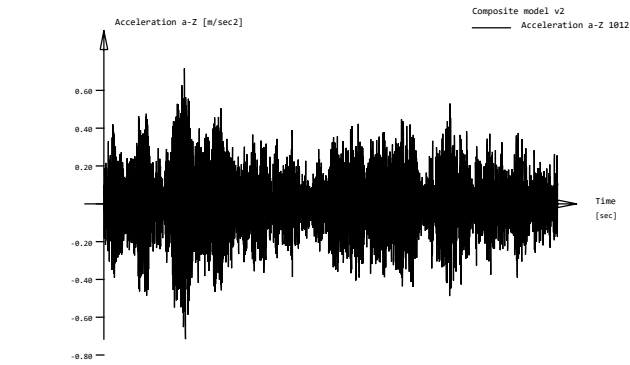
Results node 1012



Composite model v2

Composite model v2

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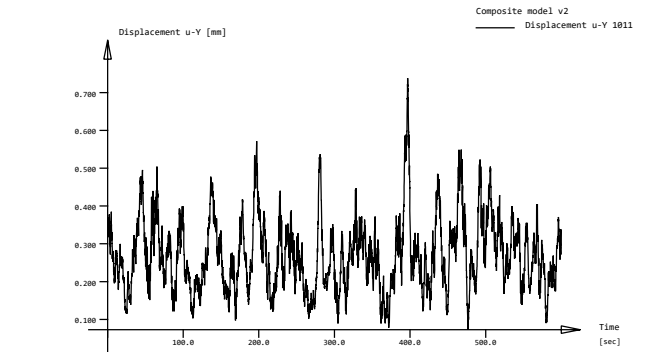


Composite model v2

Results node 1011

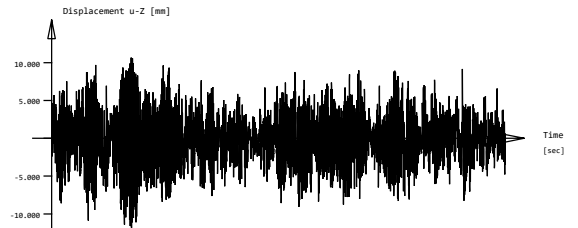
Composite model v2

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Composite model v2

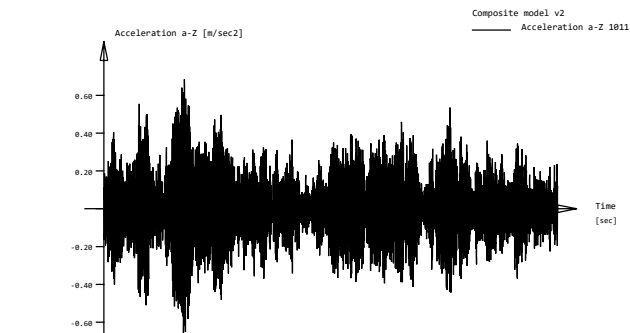
Composite model v2
Displacement u-Z 1011



Composite model v2

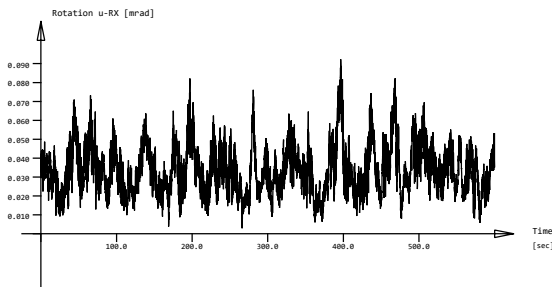
Composite model v2

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Composite model v2

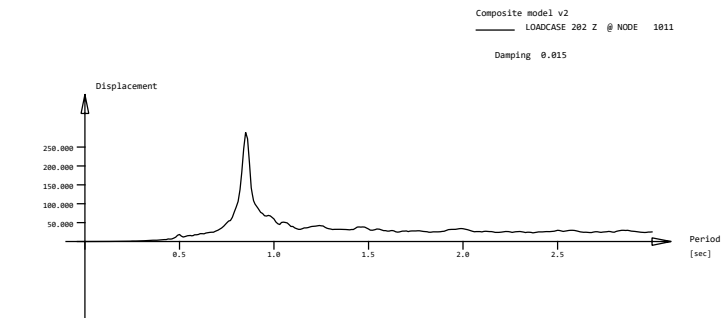
Composite model v2
Rotation u-RX 1011



Composite model v2

Composite model v2

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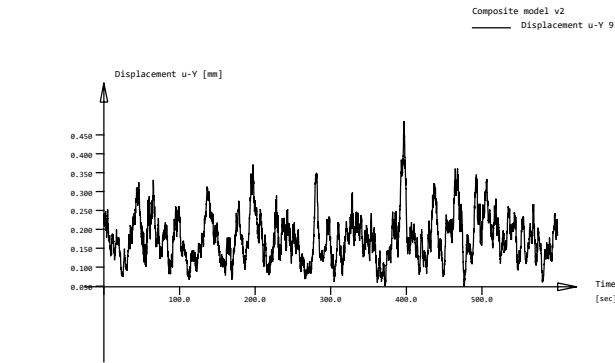


Composite model v2

Results node 9

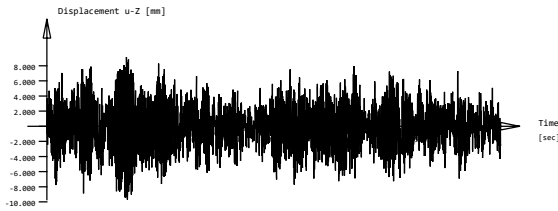
Composite model v2

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Composite model v2

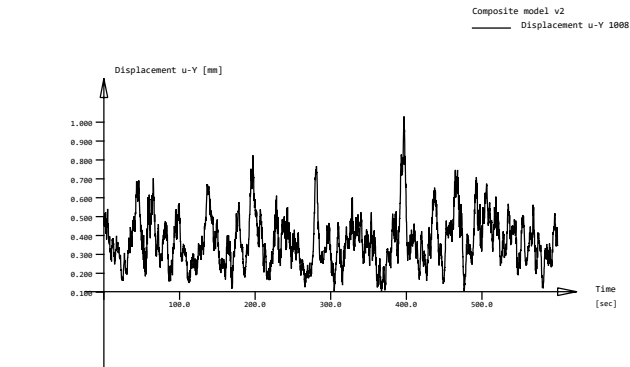
Composite model v2
Displacement u-Z 9



Composite model v2

Composite model v2

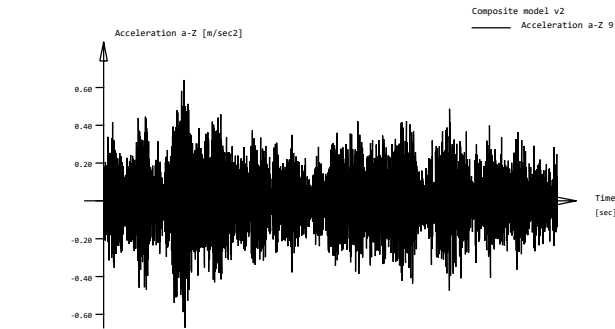
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Composite model v2

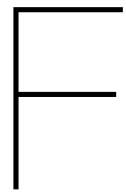
Composite model v2

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Composite model v2

Results node 1008



Dynamic wind analysis of long span bridges - additional documents

F.1. Wind analysis results FRP bridge

FRP_bridge_v1

Materials

Default design code is EuroNorm EN 1993-2:2006 Steel Structures (Norge) V 2018

Structure: B (Road bridges)

Snow load zone : 1

Materials

Mat	Classification
1	C 40/50 (EN 1992)
2	Y 1860 A (EN 1992)
3	GFK EP
4	B 500 B (EN 1992)

Mat 1 C 40/50 (EN 1992)

Young's modulus	E	35220	[N/mm2]	Safetyfactor		1.50	[-]
Poisson's ratio	μ	0.20	[-]	Strength	fc	34.00	[MPa]
Shear modulus	G	14675	[N/mm2]	Nominal strength	fck	40.00	[MPa]
Compression modulus	K	19567	[N/mm2]	Tensile strength	fctm	3.51	[MPa]
Nominal Weight	γ	25.0	[kN/m3]	Tensile strength	fctk,05	2.46	[MPa]
Mean density	ρ	2400.0	[kg/m3]	Tensile strength	fctk,95	4.56	[MPa]
Elongation coefficient	α	1.00E-05	[1/K]	Bond strength	fbd	3.68	[MPa]
				Service strength	fc	48.00	[MPa]
				Fatigue strength	fcd,fat	19.04	[MPa]
				Tensile strength	fctd	1.64	[MPa]
				Tensile failure energy	Gf	0.15	[N/mm]

Mat 2 Y 1860 A (EN 1992)

Young's modulus	E	195000	[N/mm2]	Safetyfactor		1.10	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	1600.00	[MPa]
Shear modulus	G	75000	[N/mm2]	Compressive yield	fyc	1600.00	[MPa]
Compression modulus	K	162500	[N/mm2]	Tensile strength	ft	1860.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	1860.00	[MPa]
Mean density	ρ	7850.0	[kg/m3]	Ultimate strain		60.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.		0.75	[-]
max. thickness	t-max	18.00	[mm]	EN 1992 bond coeff.	k1	1.60	[-]
Relaxation	EN-1992	Class	2	Hardening modulus	Eh	0.00	[MPa]
Relaxation	$\rho(1000h)$	2.50	[%]	Proportional limit	fp	1600.00	[MPa]
				Dynamic allowance	σ -dyn	160.87	[MPa]

Mat 3 GFK EP

Young's modulus	E	18808	[N/mm2]	Safetyfactor		1.38	[-]
Poisson's ratio	μ	0.32	[-]	Bending strength	fm,0	550.00	[MPa]
Shear modulus	G	4082	[N/mm2]	Bending strength	fm,90	550.00	[MPa]
Compression modulus	K	333	[N/mm2]	Tensile strength	ft,0	310.00	[MPa]
Nominal Weight	γ	13.4	[kN/m3]	Tensile strength	ft,90	191.00	[MPa]
Mean density	ρ	1345.0	[kg/m3]	Compress. strength	fc,0	310.00	[MPa]
Elongation coefficient	α	0.00E+00	[1/K]	Compress. strength	fc,90	191.00	[MPa]
Young's modulus	E-90	11591	[N/mm2]	Shear strength	fv	90.00	[MPa]
Shear modulus	G-90	5000	[N/mm2]	Shear strength	fv,T	300.00	[MPa]
Poisson's ratio	μ -90	0.00	[-]	Shear strength	fv,p1	300.00	[MPa]
Permanent combination	kmod	1.00	[-]				
Long term combination		1.00	[-]				
Middle term combination		1.00	[-]				
Short term combination		1.00	[-]				
Very short term combination		1.00	[-]				

Mat 4 B 500 B (EN 1992)

Young's modulus	E	200000	[N/mm2]	Safetyfactor		1.15	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	500.00	[MPa]
Shear modulus	G	76923	[N/mm2]	Compressive yield	fyc	500.00	[MPa]
Compression modulus	K	166667	[N/mm2]	Tensile strength	ft	540.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	540.00	[MPa]

FRP_bridge_v1

Materials

Mat 4 B 500 B (EN 1992)

Mean density	ρ	7850.0	[kg/m3]	Ultimate strain	50.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.	1.00	[-]
max. thickness	t-max	32.00	[mm]	EN 1992 bond coeff. k1	0.80	[-]
				Hardening modulus Eh	0.00	[MPa]
				Proportional limit fp	500.00	[MPa]
				Dynamic allowance σ -dyn	152.17	[MPa]

FRP_bridge_v1
 Cross Sections

Cross-sections static properties

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$
1	2	1.2560E-01		1.256E-03	0.0	0.0	195000	1600.0	
		0.000E+00		1.256E-03	0.0	0.0	75000	(CABLE)	
= Cable 400 1X7									
2	2	1.0747E-03		9.195E-08	0.0	0.0	195000	13.7	
		0.000E+00		9.195E-08	0.0	0.0	75000	(CABLE)	
= Cable 37 1X7									
3	1	5.3650E+01	4.432E+01	1.620E+02	0.0	0.0	35220	134125.0	3.645E+02
	4	3.851E+02	4.413E+01	3.645E+02	0.0	0.0	14675	(BEAM)	1.620E+02
= Pylon									
4	1	4.0000E+00	3.333E+00	1.333E+00	0.0	0.0	35220	10000.0	
	4	2.249E+00	3.333E+00	1.333E+00	1000.0	1000.0	14675	(BEAM)	
= B/H = 2000 / 2000 mm									
= B/H = 2000 / 2000 mm									
5	3	1.3543E+00	7.117E-01	2.530E-01	0.0	0.0	18808	1814.8	1.162E+01
		7.638E-01	4.786E-01	1.162E+01	-497.8	-500.0	4082	(BEAM)	2.530E-01
= Deck main span									
6	2	1.1335E-03		1.023E-07	0.0	0.0	195000	14.4	
		0.000E+00		1.023E-07	0.0	0.0	75000	(CABLE)	
= Cable 38 1X7									
7	2	1.1335E-03		1.023E-07	0.0	0.0	195000	14.4	
		0.000E+00		1.023E-07	0.0	0.0	75000	(CABLE)	
= Cable 38 1X7									
8	2	1.1940E-03		1.135E-07	0.0	0.0	195000	15.2	
		0.000E+00		1.135E-07	0.0	0.0	75000	(CABLE)	
= Cable 39 1X7									
9	2	1.1940E-03		1.135E-07	0.0	0.0	195000	15.2	
		0.000E+00		1.135E-07	0.0	0.0	75000	(CABLE)	
= Cable 39 1X7									
10	2	1.2560E-03		1.256E-07	0.0	0.0	195000	16.0	
		0.000E+00		1.256E-07	0.0	0.0	75000	(CABLE)	
= Cable 40 1X7									
11	2	1.3196E-03		1.386E-07	0.0	0.0	195000	16.8	
		0.000E+00		1.386E-07	0.0	0.0	75000	(CABLE)	
= Cable 41 1X7									
13	2	1.3847E-03		1.527E-07	0.0	0.0	195000	17.6	
		0.000E+00		1.527E-07	0.0	0.0	75000	(CABLE)	
= Cable 42 1X7									
14	2	1.4515E-03		1.677E-07	0.0	0.0	195000	18.5	
		0.000E+00		1.677E-07	0.0	0.0	75000	(CABLE)	
= Cable 43 1X7									
15	2	1.5198E-03		1.839E-07	0.0	0.0	195000	19.4	
		0.000E+00		1.839E-07	0.0	0.0	75000	(CABLE)	
= Cable 44 1X7									
16	2	1.5896E-03		2.012E-07	0.0	0.0	195000	20.3	
		0.000E+00		2.012E-07	0.0	0.0	75000	(CABLE)	
= Cable 45 1X7									
17	2	1.6611E-03		2.197E-07	0.0	0.0	195000	21.2	
		0.000E+00		2.197E-07	0.0	0.0	75000	(CABLE)	
= Cable 46 1X7									
18	2	1.7341E-03		2.394E-07	0.0	0.0	195000	22.1	
		0.000E+00		2.394E-07	0.0	0.0	75000	(CABLE)	
= Cable 47 1X7									
19	2	1.8086E-03		2.604E-07	0.0	0.0	195000	23.0	
		0.000E+00		2.604E-07	0.0	0.0	75000	(CABLE)	
= Cable 48 1X7									
20	2	1.8848E-03		2.828E-07	0.0	0.0	195000	24.0	

FRP_bridge_v1
 Cross Sections

Cross-sections static properties

SNo	Mat	A[m2]	Ay[m2]	Iy[m4]	yc[mm]	ysc[mm]	E[N/mm2]	g[kg/m]	I-1[m4]
	MRf	It[m4]	Az[m2]	Iz[m4]	zc[mm]	zsc[mm]	G[N/mm2]		I-2[m4]
			Ayz[m2]	Iyz[m4]					$\alpha[^\circ]$
		0.000E+00		2.828E-07	0.0	0.0	75000	(CABLE)	
= Cable 49 1X7									
21	2	1.9625E-03		3.066E-07	0.0	0.0	195000	25.0	
		0.000E+00		3.066E-07	0.0	0.0	75000	(CABLE)	
= Cable 50 1X7									
22	2	2.0418E-03		3.319E-07	0.0	0.0	195000	26.0	
		0.000E+00		3.319E-07	0.0	0.0	75000	(CABLE)	
= Cable 51 1X7									
23	2	2.1226E-03		3.587E-07	0.0	0.0	195000	27.0	
		0.000E+00		3.587E-07	0.0	0.0	75000	(CABLE)	
= Cable 52 1X7									
24	2	2.2051E-03		3.871E-07	0.0	0.0	195000	28.1	
		0.000E+00		3.871E-07	0.0	0.0	75000	(CABLE)	
= Cable 53 1X7									
25	2	2.2891E-03		4.172E-07	0.0	0.0	195000	29.2	
		0.000E+00		4.172E-07	0.0	0.0	75000	(CABLE)	
= Cable 54 1X7									
26	2	2.3746E-03		4.490E-07	0.0	0.0	195000	30.3	
		0.000E+00		4.490E-07	0.0	0.0	75000	(CABLE)	
= Cable 55 1X7									
27	2	2.4618E-03		4.825E-07	0.0	0.0	195000	31.4	
		0.000E+00		4.825E-07	0.0	0.0	75000	(CABLE)	
= Cable 56 1X7									
28	2	2.5505E-03		5.179E-07	0.0	0.0	195000	32.5	
		0.000E+00		5.179E-07	0.0	0.0	75000	(CABLE)	
= Cable 57 1X7									
29	2	2.6407E-03		5.552E-07	0.0	0.0	195000	33.6	
		0.000E+00		5.552E-07	0.0	0.0	75000	(CABLE)	
= Cable 58 1X7									
SNo	section number			yc[mm],zc[mm]		ordinate of elastic centroid			
Mat	material number			ysc[mm],zsc[mm]		ordinate of shear centre			
A[m2]	sectional area			E[N/mm2]		Young's modulus			
Ay[m2],Az[m2],Ayz[m2]	transverse shear deformation area			g[kg/m]		weight per length			
Iy[m4],Iz[m4],Iyz[m4]	bending moment of inertia								
I-1[m4],I-2[m4], $\alpha[^\circ]$	principal moments of inertia and angle of the principal axes								
MRf	reinforcement material number								
It[m4]	torsional moment of inertia								
G[N/mm2]	Shear modulus								

FRP_bridge_v1

Generation of Node and Element Loads

Actions

type	part	sup	Designation	γ_u	γ_f	γ_a	ψ_0	ψ_1	ψ_2	ψ_{inf}
G_1	G	perm	dead load g1	1.35	1.00	1.00	1.00	1.00	1.00	1.00
G_2	G	perm	ADL	1.35	1.00	1.00	1.00	1.00	1.00	1.00
P	P	perm	prestressing	1.00	1.00	1.00	1.00	1.00	1.00	1.00
L_T	Q	excl	Traffic load TS of EC/DIN-FB	1.35	0.00	1.00	0.75	0.75	0.00	0.80
L_U	Q	excl	Traffic load UDL of EC/DIN-FB	1.35	0.00	1.00	0.40	0.40	0.00	0.80
W	Q	excl	wind loading	1.50	0.00	1.00	0.60	0.20	0.00	0.60
			Reliability factor	Kfi	1.000					
			Reduction factor	xsi	0.890					
type	action		$\gamma_u, \gamma_f, \gamma_a$	partial safety factors for unfavourable/favourable/accidental						
part	partition of the action		$\psi_0, \psi_1, \psi_2, \psi_{inf}$	combination coefficients						
sup	superposition type									

Load Case 1 Self-weight

Factor forces and moments 1.000
Factor dead weight DL-ZZ -1.000

Load Case 2 ADL

Factor forces and moments 1.000

Loads

Kind	Reference to	Projection Designation	W[m]	Coordinates			Type	Load value
				X[m]	Y[m]	Z[m]		
Line	sln -mult-			10.000	0.000	-0.430	PG	2.40 [kN/m]
				74.000	0.000	-0.430		2.40 [kN/m]
		railings		activated				100.00 percent
Line	sln -mult-			74.000	0.000	-0.430	PG	2.40 [kN/m]
				274.000	0.000	-0.430		2.40 [kN/m]
		railings		activated				100.00 percent
Line	sln -mult-			10.000	0.000	-0.430	PG	2.80 [kN/m]
				74.000	0.000	-0.430		2.80 [kN/m]
		wearing laye		activated				100.00 percent
Line	sln -mult-			74.000	0.000	-0.430	PG	2.80 [kN/m]
				274.000	0.000	-0.430		2.80 [kN/m]
		wearing laye		activated				100.00 percent
Line	sln -mult-			10.000	0.000	-0.430	PG	0.93 [kN/m]
				74.000	0.000	-0.430		0.93 [kN/m]
		side span ad		activated				100.00 percent
Line	sln -mult-			74.000	0.000	-0.430	PG	0.49 [kN/m]
				274.000	0.000	-0.430		0.49 [kN/m]
		main span ad		activated				100.00 percent
Point	bgrp 5			18.000	0.000	-0.430	PG	7.3 [kN]
		cross girder		activated				100.00 percent
Point	bgrp 5			26.000	0.000	-0.430	PG	7.3 [kN]
		cross girder		activated				100.00 percent
Point	bgrp 5			34.000	0.000	-0.430	PG	7.3 [kN]
		cross girder		activated				100.00 percent
Point	bgrp 5			42.000	0.000	-0.430	PG	7.3 [kN]
		cross girder		activated				100.00 percent
Point	bgrp 5			50.000	0.000	-0.430	PG	7.3 [kN]
		cross girder		activated				100.00 percent
Point	bgrp 5			58.000	0.000	-0.430	PG	7.3 [kN]
		cross girder		activated				100.00 percent
Point	bgrp 5			66.000	0.000	-0.430	PG	7.3 [kN]
		cross girder		activated				100.00 percent
Point	bgrp 5			82.000	0.000	-0.430	PG	7.3 [kN]
		cross girder		activated				100.00 percent
Point	bgrp 5			90.000	0.000	-0.430	PG	7.3 [kN]
		cross girder		activated				100.00 percent
Point	bgrp 5			10.000	0.000	-0.430	PG	607.5 [kN]

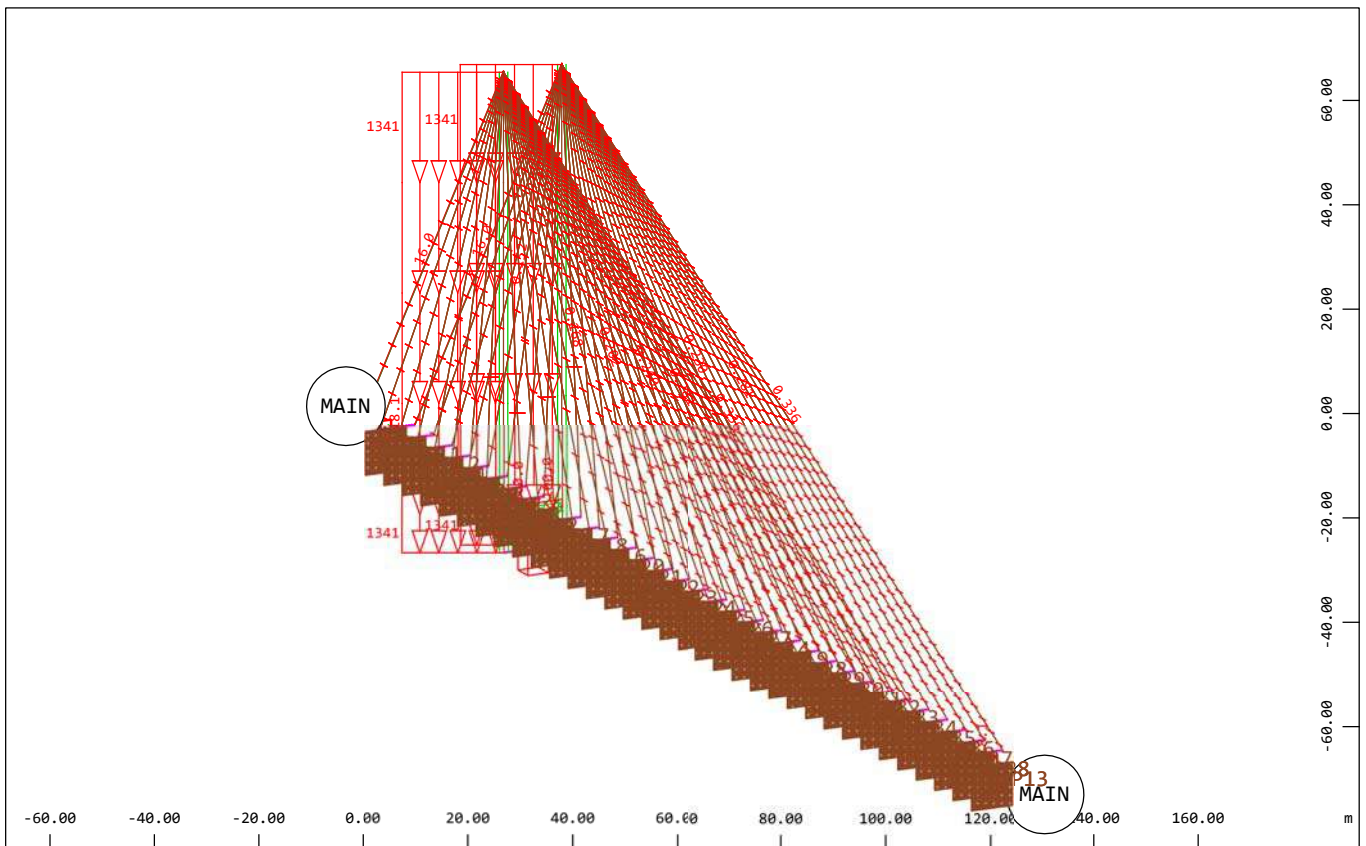
FRP_bridge_v1

Generation of Node and Element Loads

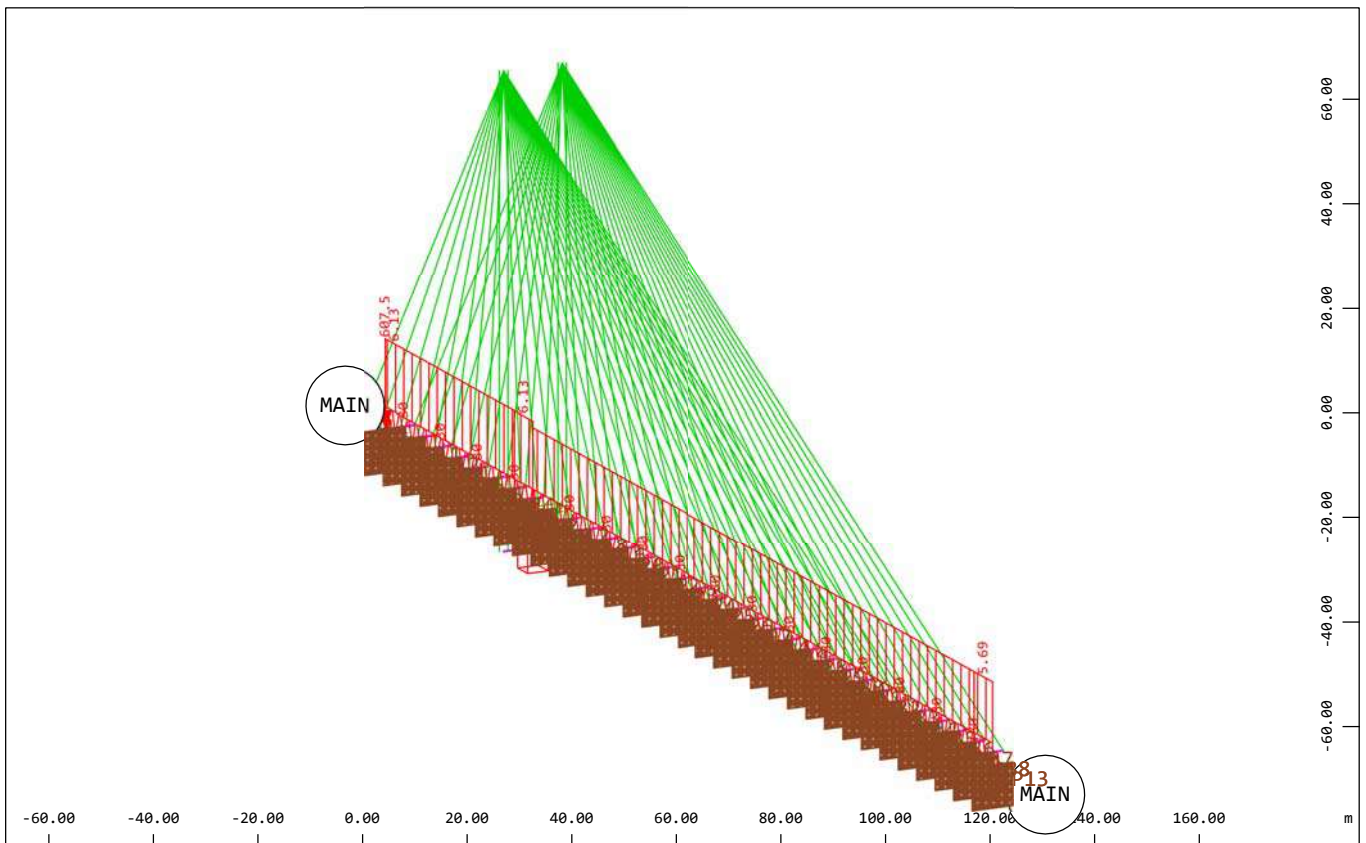
Loads

Kind	Reference to	Projection Designation	W[m]	Coordinates			Type	Load value
				X[m]	Y[m]	Z[m]		
						activated		100.00 percent
Point	bgrp 5			98.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			106.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			114.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			122.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			130.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			138.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			146.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			154.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			162.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			170.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			178.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			186.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			194.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			202.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			210.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			218.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			226.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			234.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			242.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			250.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			258.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent
Point	bgrp 5			266.000	0.000	-0.430	PG	7.3 [kN]
		cross girder				activated		100.00 percent

FRP_bridge_v1
 Graphical Output



Z All loads (in components), Loadcase 1 Self-weight , (1 cm 3D = unit) Beam dead load in M 1 : 1449
 X global Z (Unit=1000. kN/m, Min=-1341. Max=-18.1) , Cable dead load in global Z X * 0.502
 Y (Unit=1000. kN/m, Min=-16.0 Max=-0.137) Y * 0.906
 Z Z * 0.962



Z All loads (in components), Loadcase 2 ADL , (1 cm 3D = unit) Beam single load (force) M 1 : 1446
 X in global Z (Unit=500.0 kN, Min=-607.5 Max=-7.30) , Beam line load (force) in global X * 0.502
 Y Z (Unit=5.00 kN/m, Min=-6.13 Max=-5.69) Y * 0.906
 Z Z * 0.962

FRP_bridge_v1

Load Case 9 (P) Prestress cables

Factor forces and moments 1.000
 unfavourable partial safety factor 1.000
 favourable partial safety factor 1.000
 Combination coefficient ψ_0 1.000 (rare)
 Combination coefficient ψ_{1inf} 1.000 (infrequent)
 Combination coefficient ψ_1 1.000 (frequent)
 Combination coefficient ψ_2 1.000 (permanent)
 Loads completely copied from load case 1999 CABL with factor 1.000

Loads acting on Cable elements

Number	Type	Load value
100001	P-Nxe	114.0 [kN]
100002	P-Nxe	114.0 [kN]
110001	P-Nxe	117.4 [kN]
110002	P-Nxe	117.4 [kN]
120001	P-Nxe	121.4 [kN]
120002	P-Nxe	121.4 [kN]
130001	P-Nxe	127.9 [kN]
130002	P-Nxe	127.9 [kN]
140001	P-Nxe	134.5 [kN]
140002	P-Nxe	134.5 [kN]
150001	P-Nxe	143.5 [kN]
150002	P-Nxe	143.5 [kN]
160001	P-Nxe	153.5 [kN]
160002	P-Nxe	153.5 [kN]
170001	P-Nxe	164.5 [kN]
170002	P-Nxe	164.5 [kN]
180001	P-Nxe	176.2 [kN]
180002	P-Nxe	176.2 [kN]
190001	P-Nxe	188.5 [kN]
190002	P-Nxe	188.5 [kN]
200001	P-Nxe	201.4 [kN]
200002	P-Nxe	201.4 [kN]
210001	P-Nxe	214.7 [kN]
210002	P-Nxe	214.7 [kN]
220001	P-Nxe	228.4 [kN]
220002	P-Nxe	228.4 [kN]
230001	P-Nxe	242.6 [kN]
230002	P-Nxe	242.6 [kN]
240001	P-Nxe	257.1 [kN]
240002	P-Nxe	257.1 [kN]
250001	P-Nxe	271.9 [kN]
250002	P-Nxe	271.9 [kN]
260001	P-Nxe	287.0 [kN]
260002	P-Nxe	287.0 [kN]
270001	P-Nxe	302.5 [kN]
270002	P-Nxe	302.5 [kN]
280001	P-Nxe	318.2 [kN]
280002	P-Nxe	318.2 [kN]
290001	P-Nxe	334.6 [kN]
290002	P-Nxe	334.6 [kN]
300001	P-Nxe	349.8 [kN]
300002	P-Nxe	349.8 [kN]
310001	P-Nxe	371.4 [kN]
310002	P-Nxe	371.4 [kN]
320001	P-Nxe	369.4 [kN]
320002	P-Nxe	369.4 [kN]
330001	P-Nxe	459.6 [kN]
330002	P-Nxe	459.6 [kN]
340001	P-Nxe	115.8 [kN]

FRP_bridge_v1

Loads acting on Cable elements

Number	Type	Load value	
340002	P-Nxe	115.8	[kN]
350001	P-Nxe	119.3	[kN]
350002	P-Nxe	119.3	[kN]
360001	P-Nxe	122.8	[kN]
360002	P-Nxe	122.8	[kN]
370001	P-Nxe	132.1	[kN]
370002	P-Nxe	132.1	[kN]
380001	P-Nxe	124.5	[kN]
380002	P-Nxe	124.5	[kN]
390001	P-Nxe	206.6	[kN]
390002	P-Nxe	206.6	[kN]
400001	P-Nxe	-152.8	[kN]
400002	P-Nxe	-152.8	[kN]
410001	P-Nxe	8331.9	[kN]
410002	P-Nxe	8331.9	[kN]

FRP_bridge_v1

System- and Control Information

Control Information

Primary load case 1999 +Opt:add_F +Opt:add_GS
 Number of unknowns 258
 unknowns per node 6
 Number eigenvalues 40

Groups

Grp	Option	CS	Factor	Rayleigh-A [1/sec]	Rayleigh-B [sec]	ξ [o/o]	Wind
0	FULL		1.000	0.000000	0.000000	0.00	
1	FULL		1.000	0.000000	0.000000	0.00	
4	FULL		1.000	0.000000	0.000000	0.00	
5	FULL		1.000	0.000000	0.000000	0.00	
10	FULL		1.000	0.000000	0.000000	0.00	
11	FULL		1.000	0.000000	0.000000	0.00	
12	FULL		1.000	0.000000	0.000000	0.00	
13	FULL		1.000	0.000000	0.000000	0.00	
14	FULL		1.000	0.000000	0.000000	0.00	
15	FULL		1.000	0.000000	0.000000	0.00	
16	FULL		1.000	0.000000	0.000000	0.00	
17	FULL		1.000	0.000000	0.000000	0.00	
18	FULL		1.000	0.000000	0.000000	0.00	
19	FULL		1.000	0.000000	0.000000	0.00	
20	FULL		1.000	0.000000	0.000000	0.00	
21	FULL		1.000	0.000000	0.000000	0.00	
22	FULL		1.000	0.000000	0.000000	0.00	
23	FULL		1.000	0.000000	0.000000	0.00	
24	FULL		1.000	0.000000	0.000000	0.00	
25	FULL		1.000	0.000000	0.000000	0.00	
26	FULL		1.000	0.000000	0.000000	0.00	
27	FULL		1.000	0.000000	0.000000	0.00	
28	FULL		1.000	0.000000	0.000000	0.00	
29	FULL		1.000	0.000000	0.000000	0.00	
30	FULL		1.000	0.000000	0.000000	0.00	
31	FULL		1.000	0.000000	0.000000	0.00	
32	FULL		1.000	0.000000	0.000000	0.00	
33	FULL		1.000	0.000000	0.000000	0.00	
34	FULL		1.000	0.000000	0.000000	0.00	
35	FULL		1.000	0.000000	0.000000	0.00	
36	FULL		1.000	0.000000	0.000000	0.00	
37	FULL		1.000	0.000000	0.000000	0.00	
38	FULL		1.000	0.000000	0.000000	0.00	
39	FULL		1.000	0.000000	0.000000	0.00	
40	FULL		1.000	0.000000	0.000000	0.00	
41	FULL		1.000	0.000000	0.000000	0.00	

CS construction stage
 Factor Factor on stiffness
 Rayleigh-A mass proportional damping ratio
 Rayleigh-B stiffness proportional damping ratio
 ξ modal damping ratio
 Wind options for dynamic wind loading

Beam Elements

Finite beam elements without intermediate sections
 Shear deformations accounted for with nonconforming SOFiSTiK-Timoshenko beam

Primary load case 1999

Sum of masses and mass moments of inertia

Node	TM			RM			RMB [tm2]
	X[t]	Y[t]	Z[t]	X[tm2]	Y[tm2]	Z[tm2]	
total ¹	26985.607	26985.606	26985.606	1.906E+04	4.116E+04	2.550E+05	-
	S[m] ²			RM(S) ³			
	75.456	0.000	38.574	2.474E+07	4.908E-02	-1.53E+06	

FRP_bridge_v1
 Modal analysis

Sum of masses and mass moments of inertia

Node	TM			RM			RMB [tm2]
	X[t]	Y[t]	Z[t]	X[tm2]	Y[tm2]	Z[tm2]	
				4.908E-02	3.301E+07	3.089E-03	
				-1.53E+06	3.089E-03	1.028E+07	
active ¹	26904.360	26905.610	26824.363	1.893E+04	4.116E+04	2.550E+05	-
	S[m] ²			RM(S) ³			
	75.460	0.000	38.809	2.437E+07	4.906E-02	-1.78E+06	
				4.906E-02	3.237E+07	3.108E-03	
				-1.78E+06	3.108E-03	9.995E+06	
¹ sum of the total and the active nodal masses							
² coordinates of the center of gravity							
³ 3x3 rotational mass matrix at the center of gravity							
TM translational masses in X-, Y- and Z-direction							
RM rotational masses about X-, Y- and Z-axis							
RMB warping mass							

Processing

Parameter of System of Equations

Number of unknowns 258 (Direct sparse Gauss-Solver)
 Total entries 2960
 Total entries after fill in 3974
 Mass matrix 1998 (consistent), incl. rotational masses

Control Information

Using Vectoriteration	
Iteration vectors	42
Iterations	84

Eigenvalues

No.	LC	λ	error	ω	f	T	ξ	Meff			participation		
		[rad2/sec2]	[-]	[rad/sec]	[Hz]	[sec]	[%]	X[%]	Y[%]	Z[%]	X[%]	Y[%]	Z[%]
1	500	5.8997E+00	0.0E+00	2.429	0.387	2.587	0.000	0.0	61.0	0.0	0.0	64.3	0.0
2	501	6.1910E+00	0.0E+00	2.488	0.396	2.525	0.000	0.0	0.0	0.0	0.0	0.0	0.0
3	502	1.0718E+01	0.0E+00	3.274	0.521	1.919	0.000	0.0	1.0	0.0	0.0	1.1	0.0
4	503	2.3730E+01	0.0E+00	4.871	0.775	1.290	0.000	42.9	0.0	0.4	45.3	0.0	0.5
5	504	2.9256E+01	0.0E+00	5.409	0.861	1.162	0.000	0.0	0.0	0.0	0.0	0.0	0.0
6	505	5.1152E+01	0.0E+00	7.152	1.138	0.879	0.000	17.5	0.0	0.6	18.5	0.0	0.6
7	506	8.1149E+01	0.0E+00	9.008	1.434	0.697	0.000	2.1	0.0	0.2	2.2	0.0	0.2
8	507	1.0286E+02	0.0E+00	10.142	1.614	0.620	0.000	0.0	0.0	0.0	0.0	0.0	0.0
9	508	1.1827E+02	0.0E+00	10.875	1.731	0.578	0.000	1.2	0.0	0.4	1.2	0.0	0.4
10	509	1.6394E+02	0.0E+00	12.804	2.038	0.491	0.000	0.4	0.0	0.2	0.4	0.0	0.2
11	510	2.1227E+02	0.0E+00	14.570	2.319	0.431	0.000	0.1	0.0	0.3	0.1	0.0	0.3
12	511	2.1831E+02	0.0E+00	14.775	2.352	0.425	0.000	0.6	0.0	0.5	0.6	0.0	0.5
13	512	2.5592E+02	0.0E+00	15.998	2.546	0.393	0.000	0.0	18.2	0.0	0.0	19.6	0.0
14	513	2.6977E+02	0.0E+00	16.425	2.614	0.383	0.000	0.0	0.0	0.0	0.0	0.0	0.0
15	514	2.7011E+02	0.0E+00	16.435	2.616	0.382	0.000	0.0	0.0	0.0	0.0	0.0	0.0
16	515	2.8530E+02	0.0E+00	16.891	2.688	0.372	0.000	0.0	0.0	0.0	0.0	0.0	0.1
17	516	4.0180E+02	0.0E+00	20.045	3.190	0.313	0.000	0.0	0.2	0.0	0.0	0.2	0.0
18	517	4.1325E+02	0.0E+00	20.329	3.235	0.309	0.000	0.0	0.0	0.1	0.0	0.0	0.1
19	518	4.6067E+02	0.0E+00	21.463	3.416	0.293	0.000	0.2	0.0	0.0	0.2	0.0	0.0
20	519	4.9371E+02	0.0E+00	22.220	3.536	0.283	0.000	0.0	0.0	0.0	0.0	0.0	0.0
21	520	5.5702E+02	0.0E+00	23.601	3.756	0.266	0.000	11.5	0.0	0.0	12.1	0.0	0.0
22	521	5.5768E+02	0.0E+00	23.615	3.758	0.266	0.000	0.0	0.0	0.0	0.0	0.0	0.0
23	522	6.1676E+02	0.0E+00	24.835	3.953	0.253	0.000	0.1	0.0	0.0	0.1	0.0	0.0
24	523	7.2502E+02	0.0E+00	26.926	4.285	0.233	0.000	3.8	0.0	0.1	4.0	0.0	0.1
25	524	7.3937E+02	0.0E+00	27.191	4.328	0.231	0.000	0.0	0.0	0.0	0.0	0.0	0.0
26	525	7.7420E+02	0.0E+00	27.824	4.428	0.226	0.000	0.0	0.4	0.0	0.0	0.5	0.0
27	526	9.1803E+02	0.0E+00	30.299	4.822	0.207	0.000	0.0	0.0	0.0	0.0	0.0	0.0
28	527	9.7228E+02	0.0E+00	31.181	4.963	0.202	0.000	0.0	0.0	0.0	0.0	0.0	0.0
29	528	1.1074E+03	0.0E+00	33.277	5.296	0.189	0.000	0.0	0.0	0.0	0.0	0.0	0.0
30	529	1.1793E+03	0.0E+00	34.341	5.466	0.183	0.000	0.0	0.0	0.0	0.0	0.0	0.0

FRP_bridge_v1
 Modal analysis

Eigenvalues

No.	LC	λ [rad ² /sec ²]	error [-]	ω [rad/sec]	f [Hz]	T [sec]	ξ [%]	Meff			participation		
								X[%]	Y[%]	Z[%]	X[%]	Y[%]	Z[%]
31	530	1.1801E+03	0.0E+00	34.352	5.467	0.183	0.000	0.0	0.0	0.0	0.0	0.0	0.0
32	531	1.2731E+03	0.0E+00	35.680	5.679	0.176	0.000	0.0	0.0	0.0	0.0	0.0	0.0
33	532	1.3399E+03	0.0E+00	36.605	5.826	0.172	0.000	0.0	0.0	0.0	0.0	0.0	0.0
34	533	1.3822E+03	0.0E+00	37.178	5.917	0.169	0.000	0.0	0.0	0.0	0.0	0.0	0.0
35	534	1.3853E+03	0.0E+00	37.220	5.924	0.169	0.000	0.0	0.0	0.1	0.0	0.0	0.1
36	535	1.5721E+03	0.0E+00	39.649	6.310	0.158	0.000	0.0	0.0	0.0	0.0	0.0	0.0
37	536	1.9212E+03	0.0E+00	43.831	6.976	0.143	0.000	0.0	0.0	0.0	0.0	0.0	0.0
38	537	1.9493E+03	0.0E+00	44.151	7.027	0.142	0.000	0.0	0.0	0.0	0.0	0.0	0.0
39	538	2.2703E+03	2.3E-05	47.648	7.583	0.132	0.000	0.0	0.9	0.0	0.0	0.9	0.0
40	539	2.3242E+03	1.2E-05	48.210	7.673	0.130	0.000	1.5	0.0	0.4	1.5	0.0	0.5
41		2.3635E+03	4.5E-05	48.616	7.737	0.129	$\Sigma(\%)^1$	81.9	81.8	3.5	86.4	86.6	3.7
42		2.4861E+03	1.7E-05	49.861	7.936	0.126							

¹ Total effective mass / participation factors of activated masses in X-, Y- and Z-direction.

No.	eigenmode number	f	eigenfrequency
LC	load case	T	eigenperiod
λ	eigenvalue	ξ	modal damping ratio
error	relative eigenvalue error	Meff	effective modal mass in X-, Y- and Z-direction
ω	circular eigenfrequency	participation	participation factors in X-, Y- and Z-direction

FRP_bridge_v1
Wind loads

Actions

type	part	sup	Designation	γ -u	γ -f	γ -a	ψ_0	ψ_1	ψ_2	ψ_{1inf}
G_1	G	perm	dead load g1	1.35	1.00	1.00	1.00	1.00	1.00	1.00
G_2	G	perm	ADL	1.35	1.00	1.00	1.00	1.00	1.00	1.00
P	P	perm	prestressing	1.00	1.00	1.00	1.00	1.00	1.00	1.00
L_T	Q	excl	Traffic load TS of EC/DIN-FB	1.35	0.00	1.00	0.75	0.75	0.00	0.80
L_U	Q	excl	Traffic load UDL of EC/DIN-FB	1.35	0.00	1.00	0.40	0.40	0.00	0.80
W	Q	excl	wind loading	1.50	0.00	1.00	0.60	0.20	0.00	0.60
			Reliability factor	Kfi	1.000					
			Reduction factor	xsi	0.890					
type	action		γ -u, γ -f, γ -a	partial safety factors for unfavourable/favourable/accidental						
part	partition of the action		$\psi_0,\psi_1,\psi_2,\psi_{1inf}$	combination coefficients						
sup	superposition type									

Load Case 80 Basic wind

Factor forces and moments 1.000

Loads completely copied from load case 80 WIND BEAM TRUS CABL QUAD with factor 1.000 design

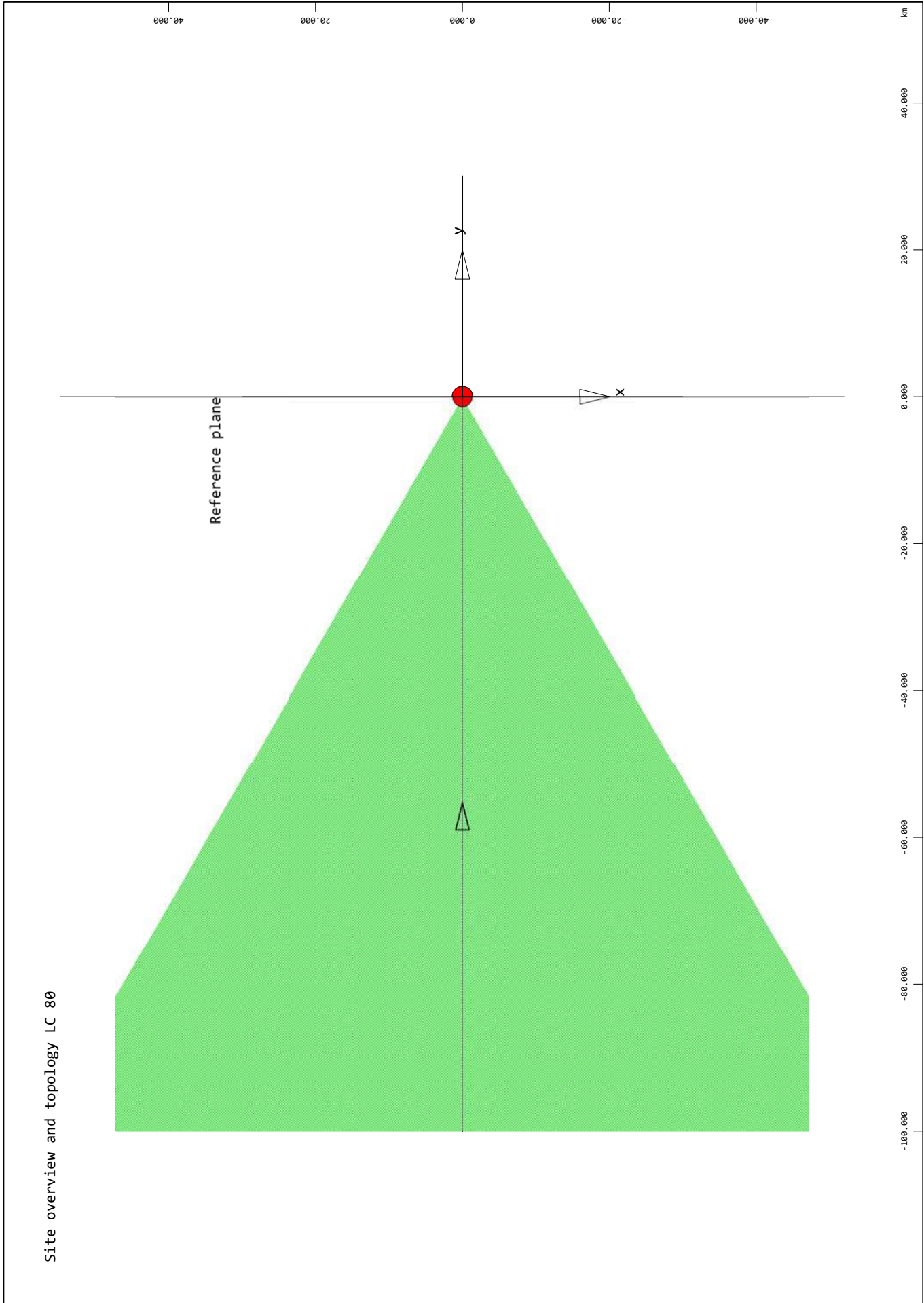
Windloading

Wind according to code NS EN-1991-1-4 (2009) Zone =Vb Terrain category III										
atmospheric wind	v	59.00	[m/sec]	Air density				ρ	1.25	[kg/m3]
Mean wind speed	vref	24.00	[m/sec]							
Surface height	hg	-12.430	[m]	Latitude				θ	45.5	[°]
Wind direction	(dx, dy, dz) =			(0.000, 1.000, 0.000				)		[-]
Reference point	(x-0, y-0, z-0) =			(10.000, 0.000, 0.000				)		[m]
Reference direction	(dxr, dyr, dzr) =			(-1.000, 0.000, 0.000				)		[-]
Wind spectra	Karman			A1	A2	A3	B	C	D	
	- Longitudinal			4.000	0.000	0.000	70.800	2.000	0.833	
	- Lateral			4.000	0.000	3021.00	283.200	2.000	1.833	
	- Vertical			4.000	0.000	3021.00	283.200	2.000	1.833	
	coherences			c-iu	c-iv	c-iw (ESDU/Karman)				
	- Longitudinal			0.221	0.114	0.114				
	- Lateral			0.110	0.167	0.167				
	- Vertical			0.110	0.167	0.167				
Roughness profile	sea			D[km]	z0[m]	Hr[m]	Hi[m]			
	-200.000		[km]	-100.000	0.300	0.000	2412.834			

Windprofiles (according to design code or explicit specification)

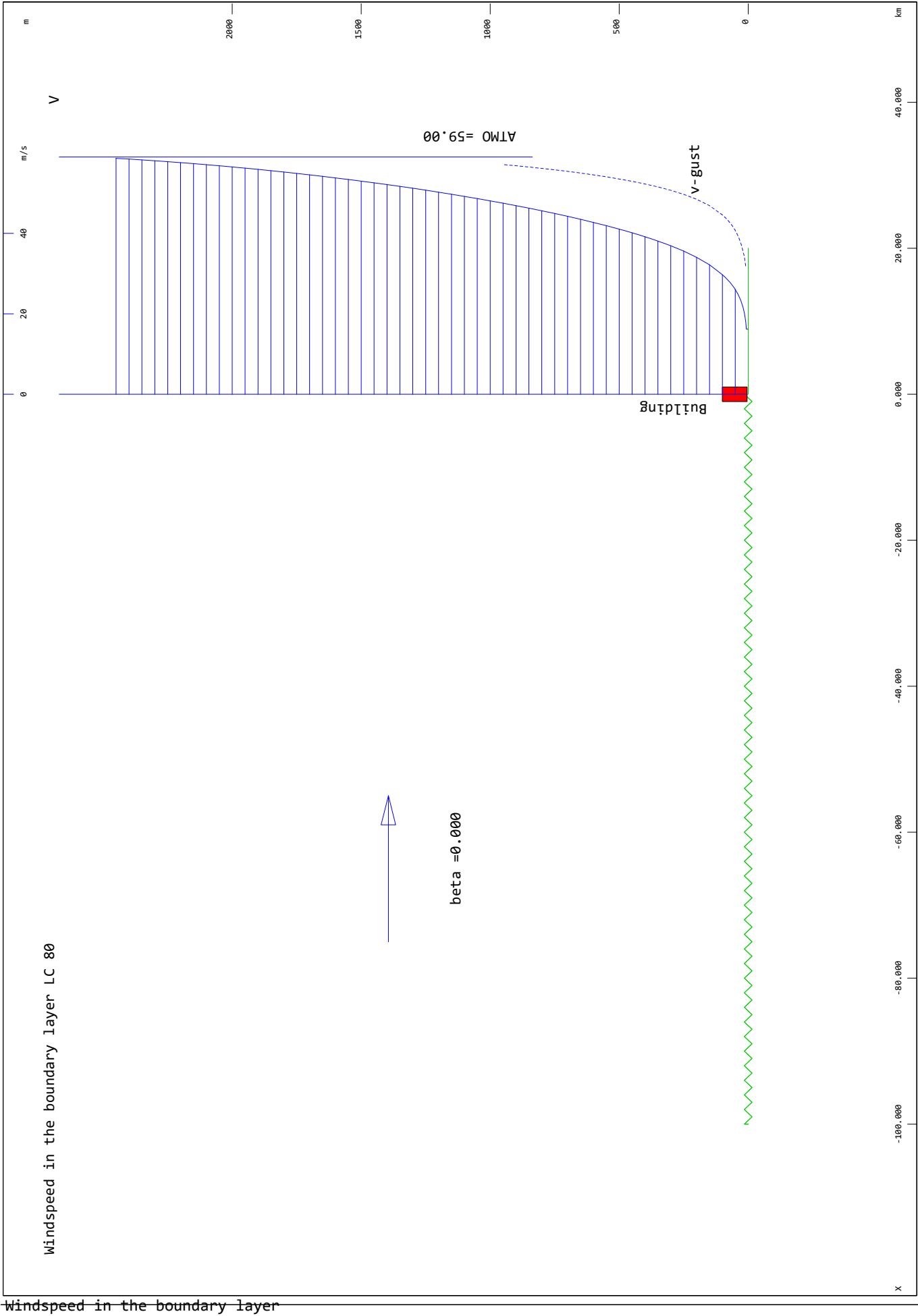
h	v-mean	v-boe	q-ref	t-lon	t-lat	t-ver	l-lon	l-lat	l-ver	phase
[m]	[m/sec]	[m/sec]	[kN/m2]	[m/sec]	[m/sec]	[m/sec]	[m]	[m]	[m]	[-]
7.000	16.18	29.23	0.53	5.13	0.00	0.00	38.841	0.000	0.000	0.000
19.000	21.31	35.15	0.77	5.30	0.00	0.00	71.406	0.000	0.000	0.000
31.000	23.67	37.98	0.90	5.43	0.00	0.00	96.247	0.000	0.000	0.000
43.000	25.28	39.84	0.99	5.51	0.00	0.00	117.501	0.000	0.000	0.000
55.000	26.53	41.24	1.06	5.55	0.00	0.00	136.529	0.000	0.000	0.000
66.000	27.47	42.26	1.12	5.58	0.00	0.00	152.585	0.000	0.000	0.000
78.000	28.36	43.20	1.17	5.60	0.00	0.00	168.948	0.000	0.000	0.000
90.000	29.14	44.00	1.21	5.61	0.00	0.00	184.353	0.000	0.000	0.000
102.000	29.83	44.70	1.25	5.62	0.00	0.00	198.975	0.000	0.000	0.000

FRP_bridge_v1
Wind loads

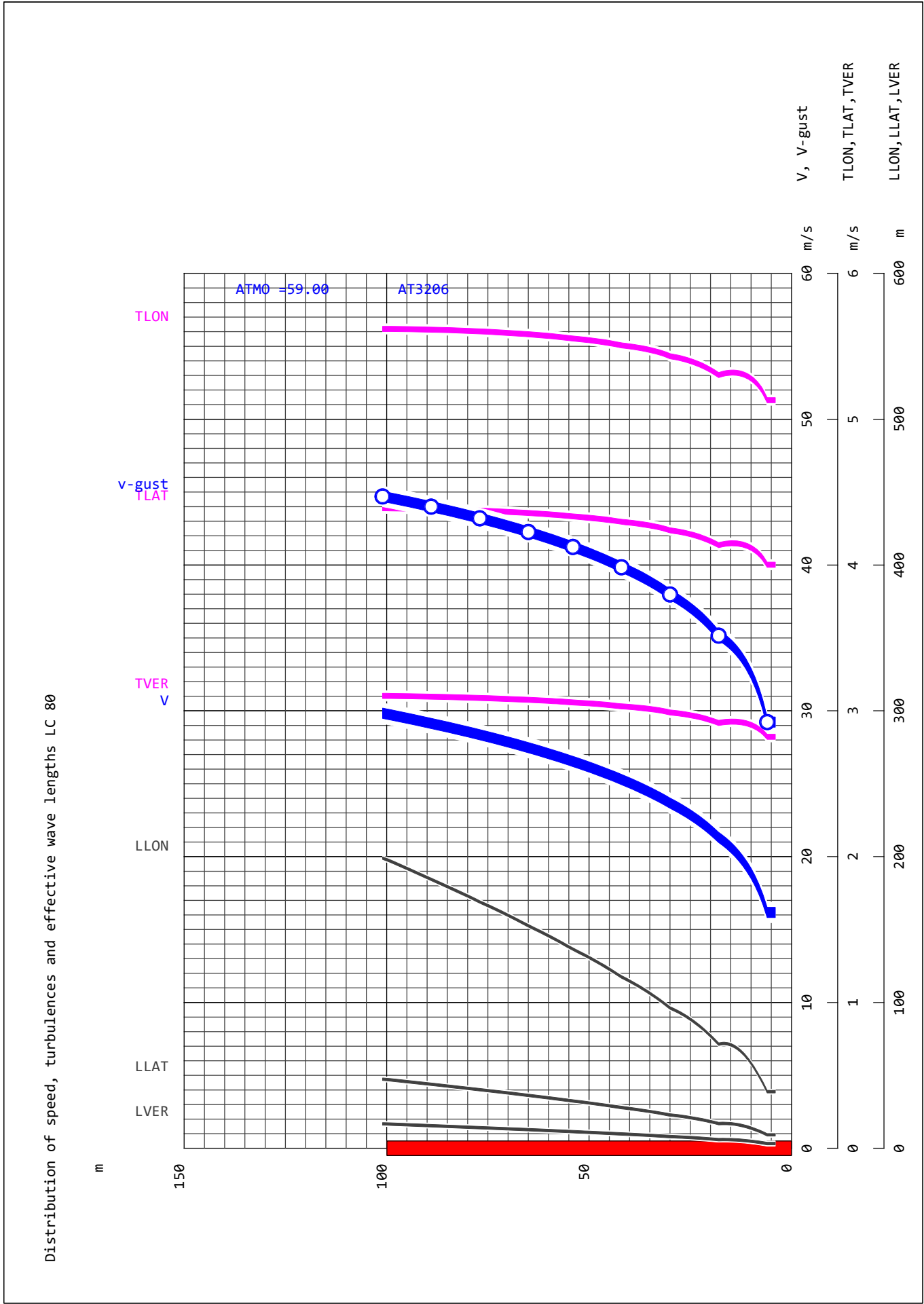


Site overview and topology

FRP_bridge_v1
Wind loads



FRP_bridge_v1
Wind loads



Distribution of speed, turbulences and effect. wave lengths

FRP_bridge_v1

Wind loads

Loads acting on Beam elements

Number	Type	fa[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fe[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fr[-]
10001	wind	1.000	0.00	16.18	0.00	1.000	0.00	19.30	0.00	0.000
10002	wind	1.000	0.00	19.30	0.00	1.000	0.00	24.13	0.00	0.000
10003	wind	1.000	0.00	24.13	0.00	1.000	0.00	26.62	0.00	0.000
10004	wind	1.000	0.00	26.62	0.00	1.000	0.00	28.36	0.00	0.000
10005	wind	1.000	0.00	28.36	0.00	1.000	0.00	29.72	0.00	0.000
10006	wind	1.000	0.00	16.18	0.00	1.000	0.00	19.30	0.00	0.000
10007	wind	1.000	0.00	19.30	0.00	1.000	0.00	24.13	0.00	0.000
10008	wind	1.000	0.00	24.13	0.00	1.000	0.00	26.62	0.00	0.000
10009	wind	1.000	0.00	26.62	0.00	1.000	0.00	28.36	0.00	0.000
10010	wind	1.000	0.00	28.36	0.00	1.000	0.00	29.72	0.00	0.000
40001	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
40002	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50001	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50002	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50003	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50004	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50005	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50006	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50007	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50008	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50009	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50010	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50011	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50012	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50013	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50014	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50015	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50016	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50017	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50018	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50019	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50020	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50021	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50022	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50023	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50024	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50025	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50026	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50027	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50028	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50029	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50030	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50031	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50032	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50033	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000

Loads acting on Cable elements

Number	Type	fa[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fe[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fr[-]
100001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
100002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
110001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
110002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
120001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
120002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
130001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
130002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
140001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
140002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000

FRP_bridge_v1
 Wind loads

Loads acting on Cable elements

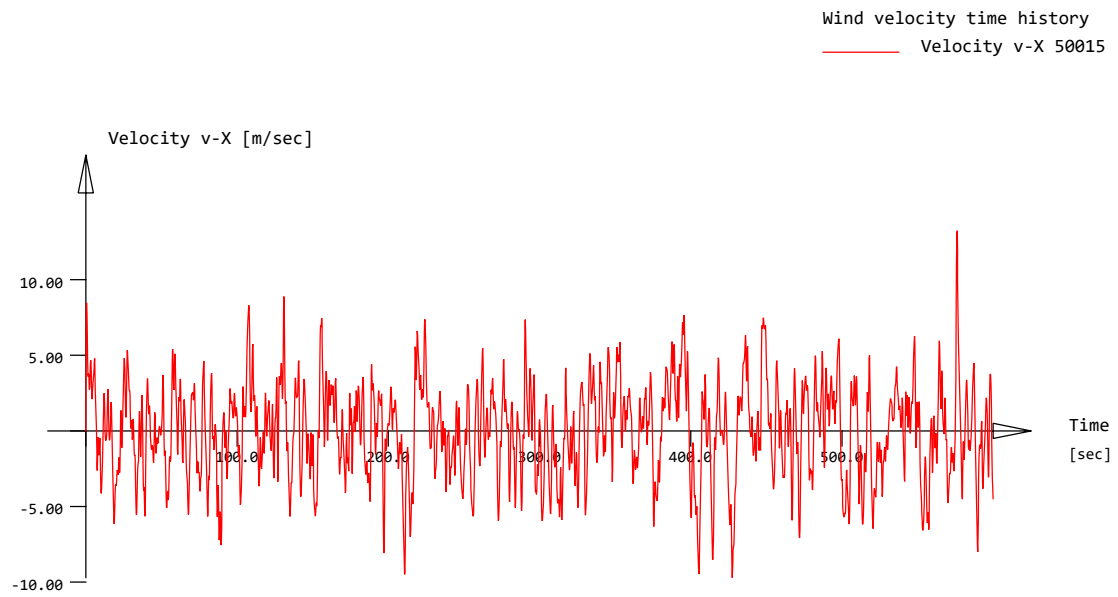
Number	Type	fa[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fe[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fr[-]
150001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
150002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
160001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
160002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
170001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
170002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
180001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
180002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
190001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
190002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
200001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
200002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
210001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
210002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
220001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
220002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
230001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
230002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
240001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
240002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
250001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
250002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
260001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
260002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
270001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
270002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
280001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
280002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
290001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
290002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
300001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
300002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
310001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
310002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
320001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
320002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
330001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
330002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
340001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
340002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
350001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
350002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
360001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
360002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
370001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
370002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
380001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
380002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
390001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
390002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
400001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
400002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
410001	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000
410002	wind	1.000	0.00	29.72	0.00	1.000	0.00	19.30	0.00	0.000

Load Case 90

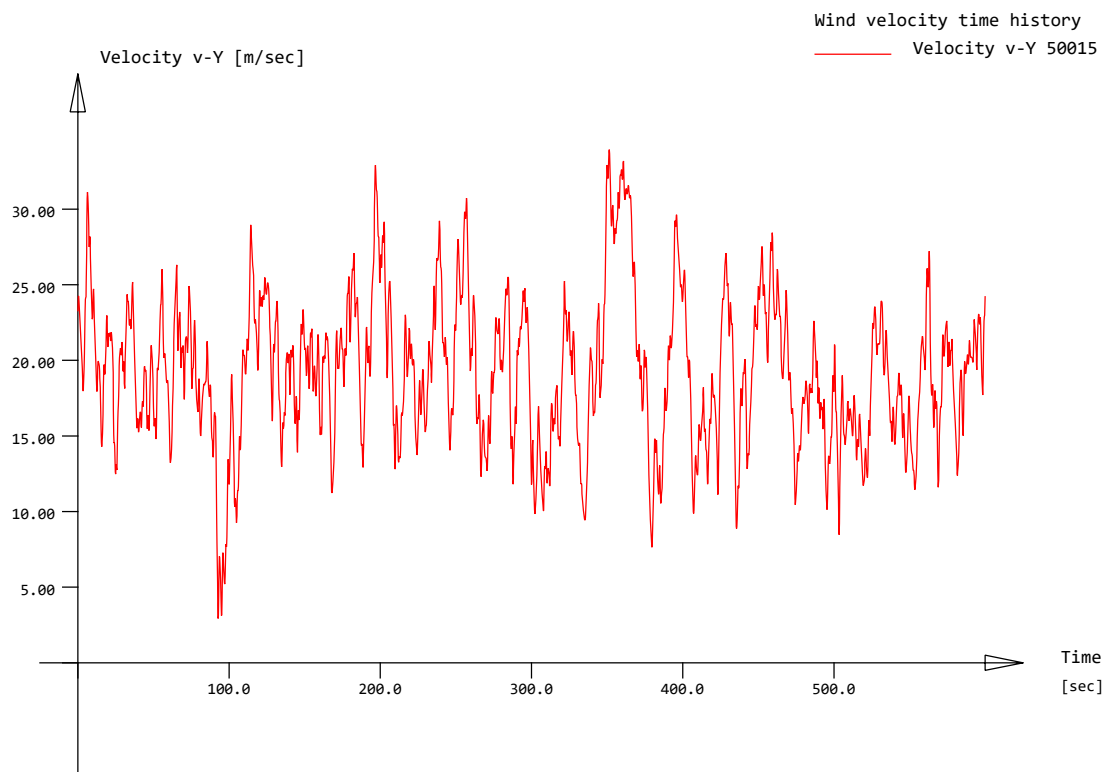
Factor forces and moments

1.000

FRP_bridge_v1
Wind velocity time history

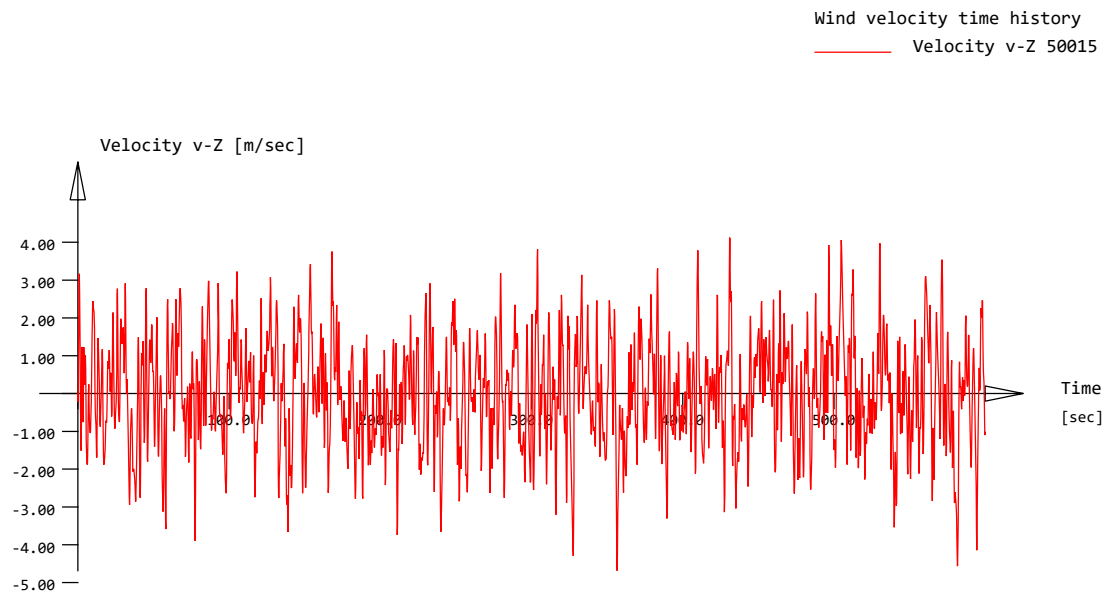


Wind velocity time history



Wind velocity time history

FRP_bridge_v1
Wind velocity time history



Wind velocity time history

FRP_bridge_v1

Integration of wind history

System- and Control Information

Control Information

Primary load case 90 t = 0.0000 [sec] +Opt:add_F +Opt:add_GS
 Number of unknowns 258
 unknowns per node 6
 Number of timesteps 30000
 Time-step 0.0200
 Printing intervall 1
 damping factor A 0.000E+00
 damping factor B 0.000E+00
 Integr.Parameter beta 0.25
 Integr.Parameter delta 0.50
 Integr.Parameter theta 1.00

Groups

Grp	Option	CS	Factor	Rayleigh-A [1/sec]	Rayleigh-B [sec]	ξ [o/o]	Wind
1	FULL		1.000	0.112512	0.001718	0.00	
4	FULL		1.000	0.112512	0.001718	0.00	
5	FULL		1.000	0.056256	0.000859	0.00	
10	FULL		1.000	0.016877	0.000258	0.00	
11	FULL		1.000	0.016877	0.000258	0.00	
12	FULL		1.000	0.016877	0.000258	0.00	
13	FULL		1.000	0.016877	0.000258	0.00	
14	FULL		1.000	0.016877	0.000258	0.00	
15	FULL		1.000	0.016877	0.000258	0.00	
16	FULL		1.000	0.016877	0.000258	0.00	
17	FULL		1.000	0.016877	0.000258	0.00	
18	FULL		1.000	0.016877	0.000258	0.00	
19	FULL		1.000	0.016877	0.000258	0.00	
20	FULL		1.000	0.016877	0.000258	0.00	
21	FULL		1.000	0.016877	0.000258	0.00	
22	FULL		1.000	0.016877	0.000258	0.00	
23	FULL		1.000	0.016877	0.000258	0.00	
24	FULL		1.000	0.016877	0.000258	0.00	
25	FULL		1.000	0.016877	0.000258	0.00	
26	FULL		1.000	0.016877	0.000258	0.00	
27	FULL		1.000	0.016877	0.000258	0.00	
28	FULL		1.000	0.016877	0.000258	0.00	
29	FULL		1.000	0.016877	0.000258	0.00	
30	FULL		1.000	0.016877	0.000258	0.00	
31	FULL		1.000	0.016877	0.000258	0.00	
32	FULL		1.000	0.016877	0.000258	0.00	
33	FULL		1.000	0.016877	0.000258	0.00	
34	FULL		1.000	0.016877	0.000258	0.00	
35	FULL		1.000	0.016877	0.000258	0.00	
36	FULL		1.000	0.016877	0.000258	0.00	
37	FULL		1.000	0.016877	0.000258	0.00	
38	FULL		1.000	0.016877	0.000258	0.00	
39	FULL		1.000	0.016877	0.000258	0.00	
40	FULL		1.000	0.016877	0.000258	0.00	
41	FULL		1.000	0.016877	0.000258	0.00	
CS		construction stage		Rayleigh-B		stiffness proportional damping ratio	
Factor		Factor on stiffness		ξ		modal damping ratio	
Rayleigh-A		mass proportional damping ratio		Wind		options for dynamic wind loading	

Beam Elements

Finite beam elements without intermediate sections

Shear deformations accounted for with nonconforming SOFiSTiK-Timoshenko beam

Primary load case 90

FRP_bridge_v1

Integration of wind history

Sum of masses and mass moments of inertia

Node	TM			RM			RMB
	X[t]	Y[t]	Z[t]	X[tm2]	Y[tm2]	Z[tm2]	[tm2]
total ¹	26985.673	26985.672	26985.672	1.906E+04	4.116E+04	2.550E+05	-
	S[m] ²			RM(S) ³			
	75.456	0.000	38.574	2.474E+07	1.895E-01	-1.53E+06	
				1.895E-01	3.301E+07	3.683E-02	
				-1.53E+06	3.683E-02	1.028E+07	
active ¹	26904.426	26905.676	26824.429	1.893E+04	4.116E+04	2.550E+05	-
	S[m] ²			RM(S) ³			
	75.460	0.000	38.809	2.437E+07	1.901E-01	-1.78E+06	
				1.901E-01	3.237E+07	3.630E-02	
				-1.78E+06	3.630E-02	9.995E+06	
¹ sum of the total and the active nodal masses							
² coordinates of the center of gravity							
³ 3x3 rotational mass matrix at the center of gravity							
TM translational masses in X-, Y- and Z-direction							
RM rotational masses about X-, Y- and Z-axis							
RMB warping mass							

Processing

Load Cases

-- Loadcase 90

Node	PX	PY	PZ	MX	MY	MZ	Mb
	[kN]	[kN]	[kN]	[kNm]	[kNm]	[kNm]	[kNm2]
sum	0.0	1807.3	-2.2	47.66	0.00	0.19	

Loads on kinematic constraints have been transfered to their master

Parameter of System of Equations

Number of unknowns	258 (Direct sparse Gauss-Solver)
Total entries	2976
Total entries after fill in	3901
Mass matrix	1998 (consistent), incl. rotational masses
Damping matrix	2857 (consistent)

Results

Nodal Displacements

Node	u-X	Time	u-Y	Time	u-Z	Time
	[mm]	[sec]	[mm]	[sec]	[mm]	[sec]
MAX	8.852		80.432		38.873	
MIN	-8.865		-11.992		-54.090	

Time history for u-Y

Node	94 (E 181)	u-Z	MINIMUM =	-49.210	MAXIMUM =	36.495	[mm]
Node	94 (E 182)	u-RX	MINIMUM =	0.194	MAXIMUM =	1.337	[mrad]
Node	94 (E 180)	u-Y	MINIMUM =	7.179	MAXIMUM =	45.421	[mm]

stored in database for DYNR with identification no 499

Time history for a-Z

Node	94 (E 181)	a-Z	MINIMUM =	-0.57	MAXIMUM =	0.61	[m/sec2]

stored in database for DYNR with identification no 499

Time history for u-Y

Node	73 (E 139)	u-Z	MINIMUM =	-17.098	MAXIMUM =	16.924	[mm]
Node	73 (E 140)	u-RX	MINIMUM =	-0.050	MAXIMUM =	0.175	[mrad]
Node	73 (E 138)	u-Y	MINIMUM =	11.992	MAXIMUM =	80.432	[mm]

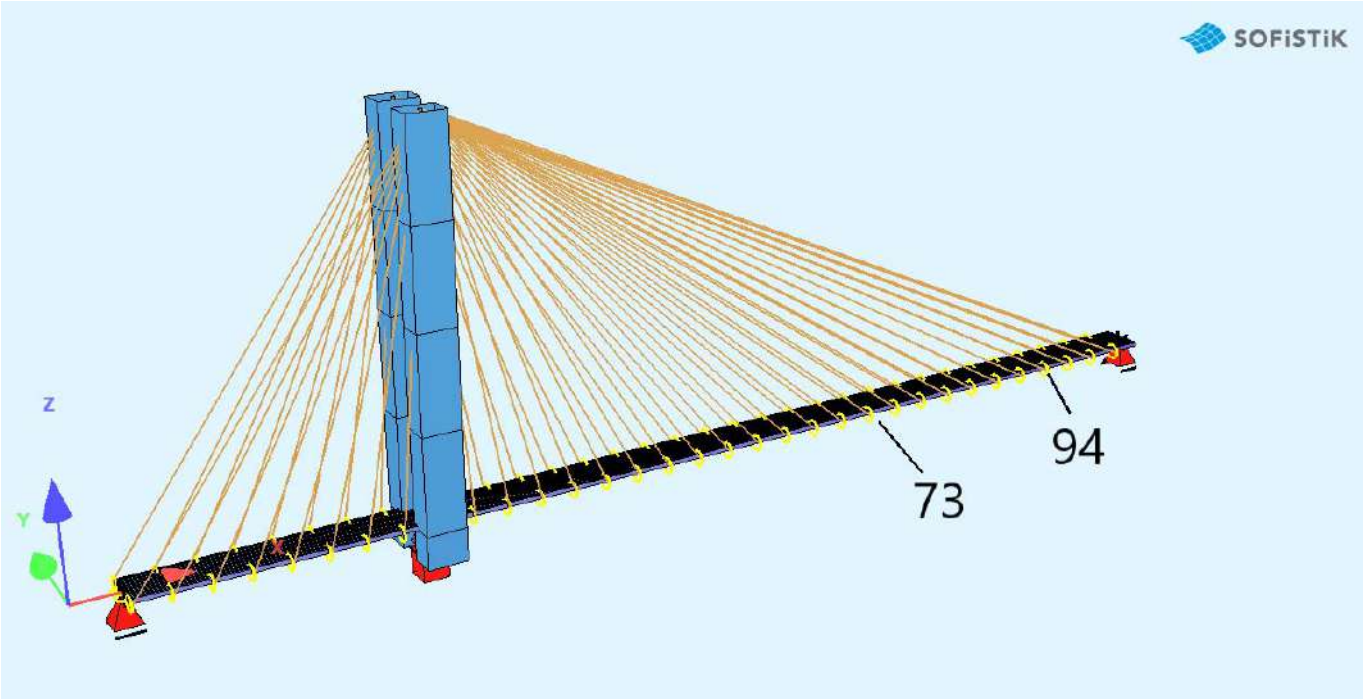
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Time history for a-Z

Node	73 (E 139)	a-Z	MINIMUM =	-0.35	MAXIMUM =	0.34	[m/sec2]

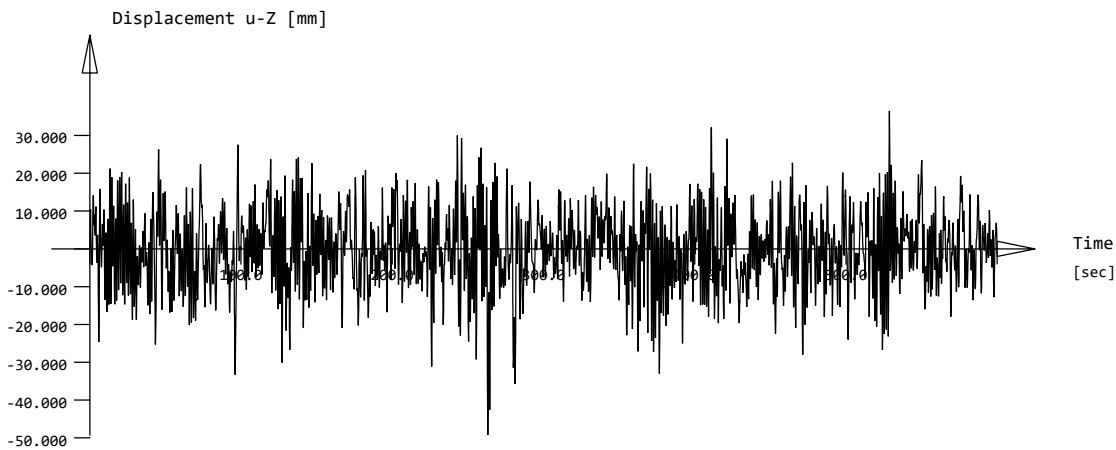
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FRP_bridge_v1



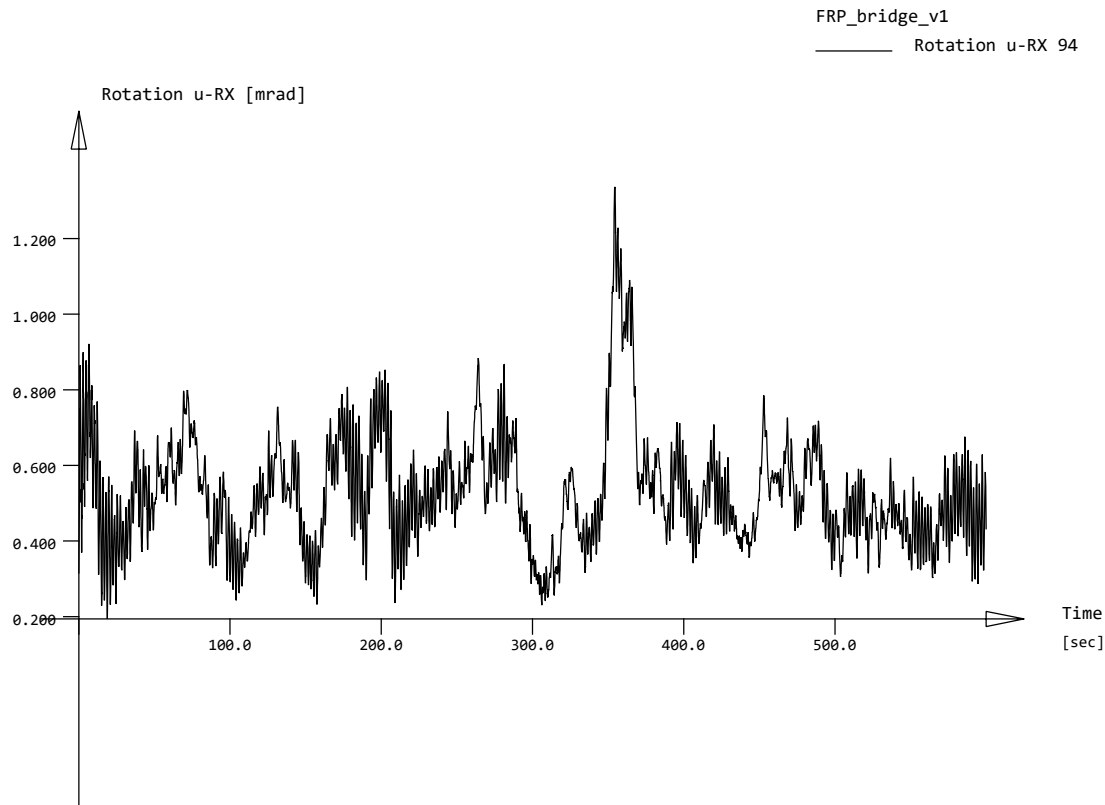
SOFiSTiK AG - www.sofistik.de

FRP_bridge_v1
Displacement u-Z 94

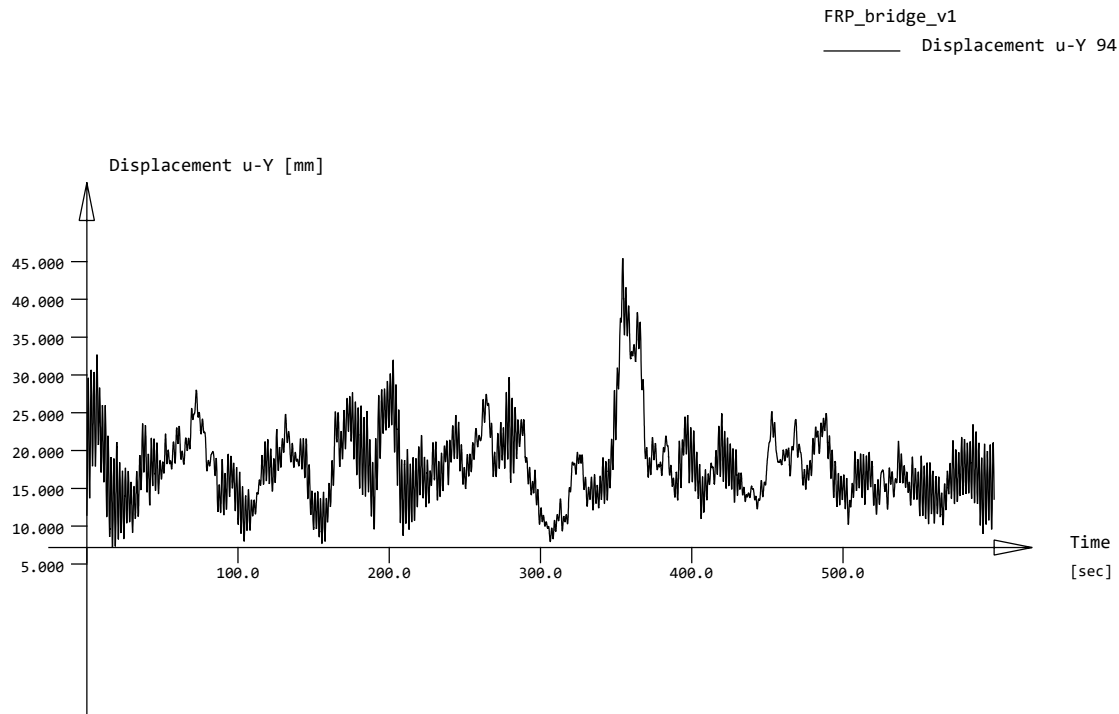


FRP_bridge_v1

FRP_bridge_v1

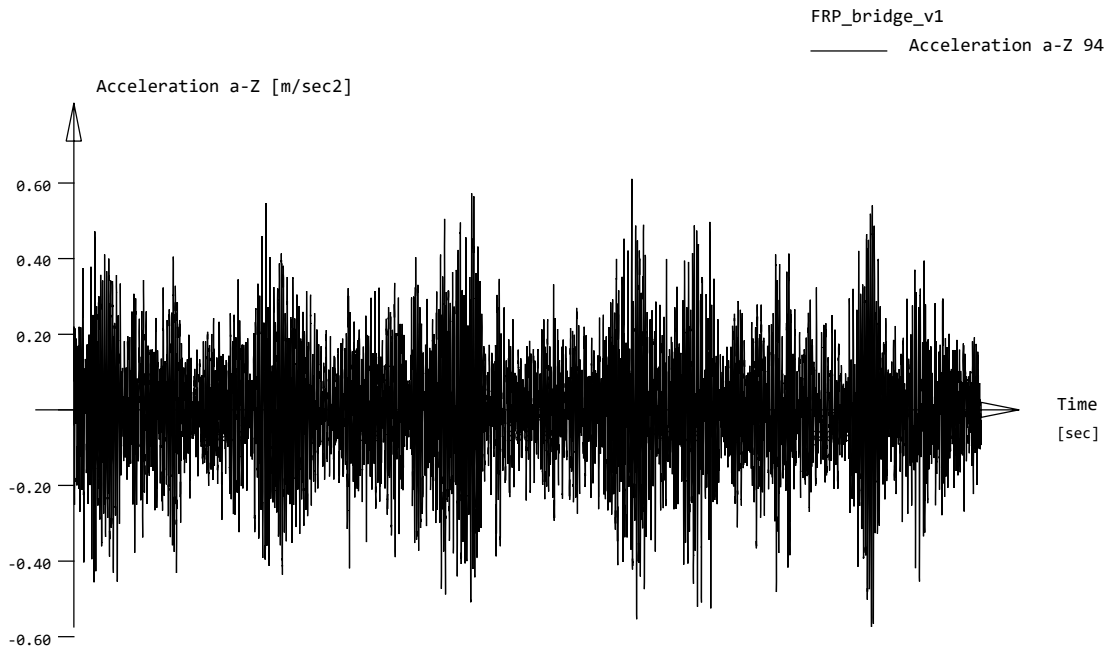


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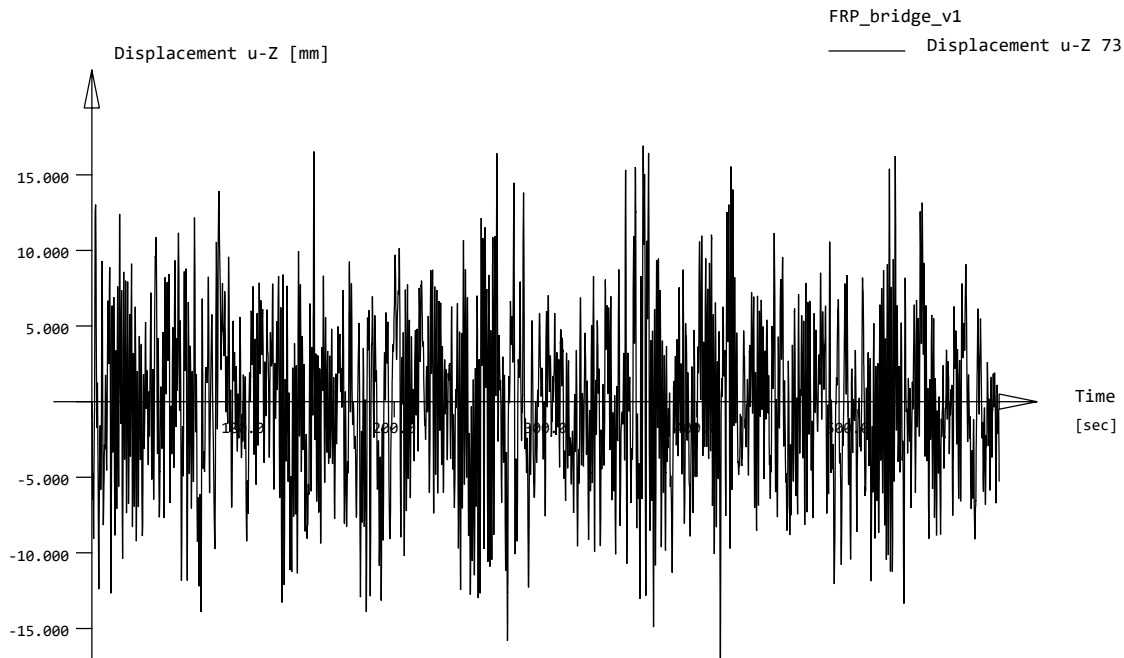


FRP_bridge_v1

FRP_bridge_v1

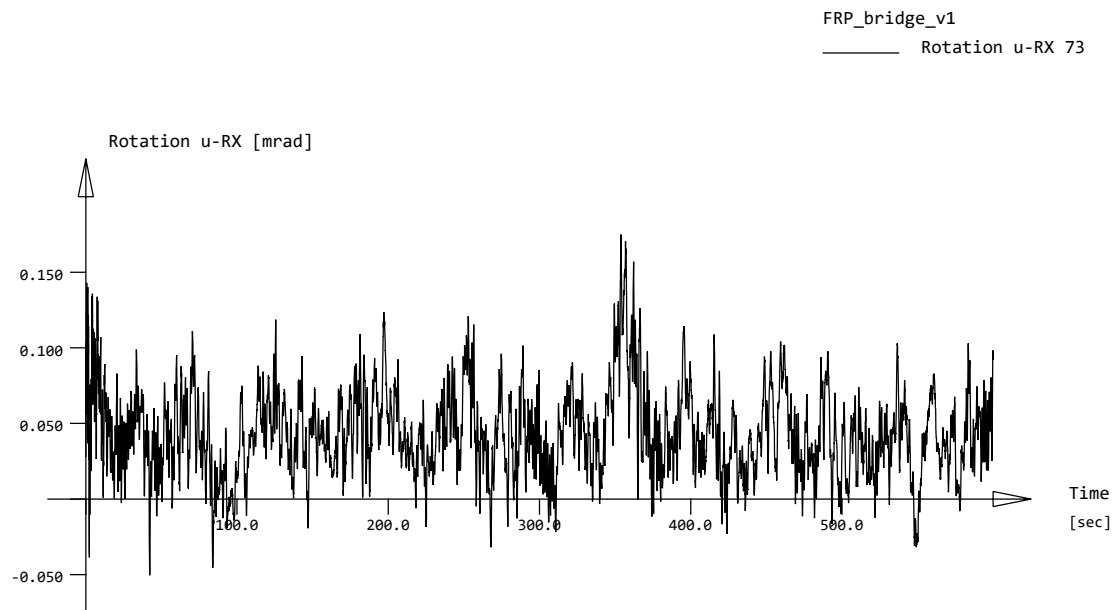


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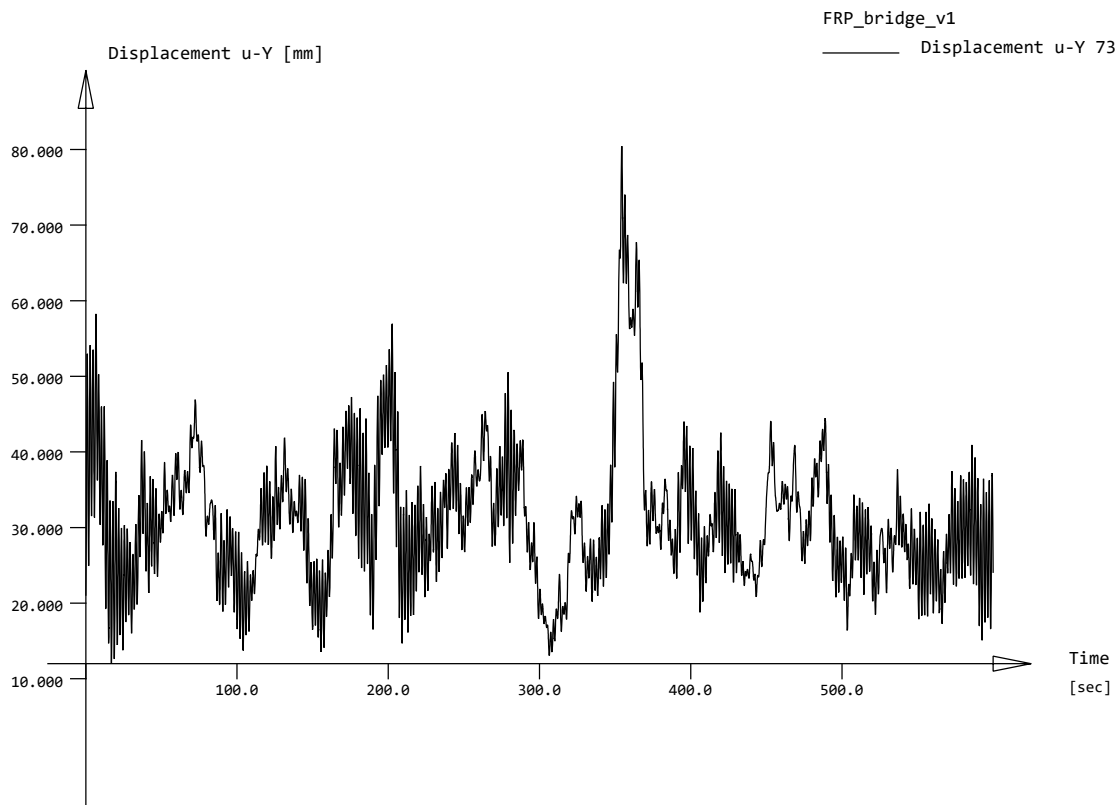


FRP_bridge_v1

FRP_bridge_v1

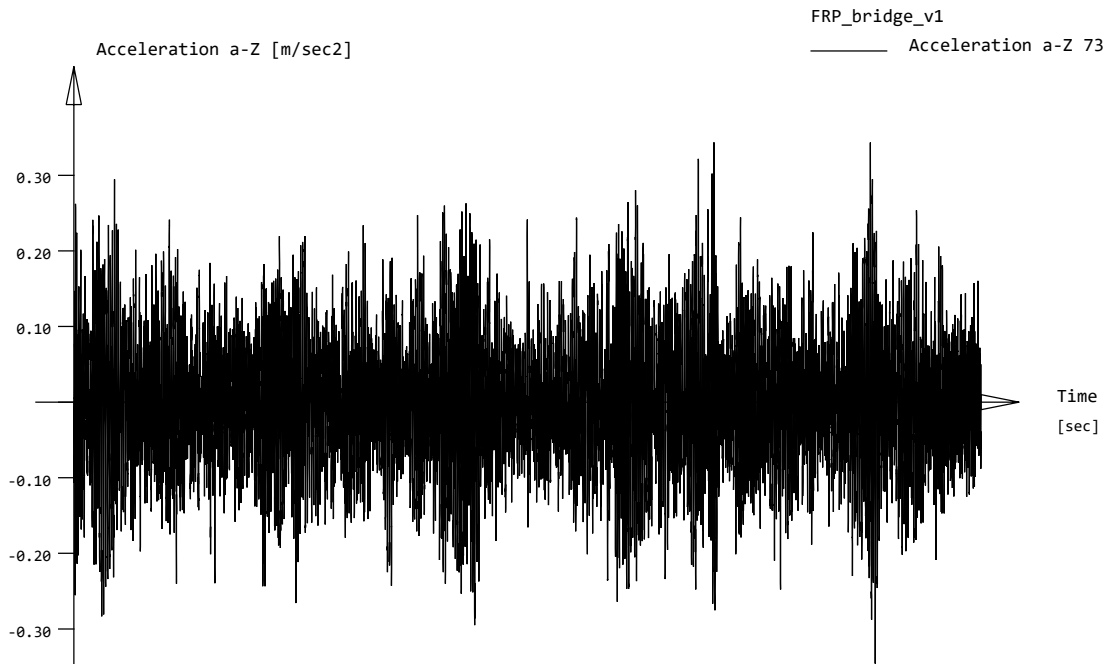


FRP_bridge_v1

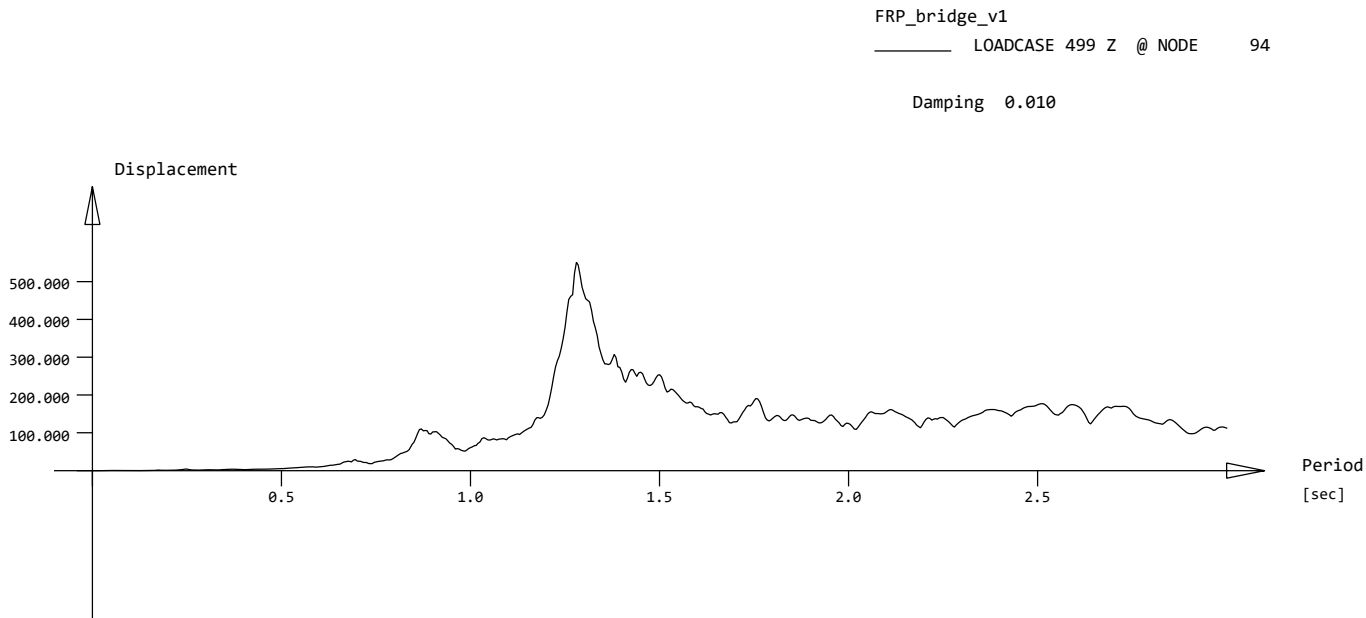


FRP_bridge_v1

FRP_bridge_v1



FRP_bridge_v1



FRP_bridge_v1

F.2. Wind analysis results steel-concrete composite bridge

FRP_bridge_v1

Materials

Default design code is EuroNorm EN 1993-2:2006 Steel Structures (Norge) V 2018

Structure: B (Road bridges)

Snow load zone : 1

Materials

Mat	Classification
1	C 40/50 (EN 1992)
2	Y 1860 A (EN 1992)
4	B 500 B (EN 1992)
15	C 40/50 (EN 1992)
18	B 500 B (EN 1992)
19	S 355 (EN 1993)

Mat 1 C 40/50 (EN 1992)

Young's modulus	E	35220	[N/mm2]	Safetyfactor		1.50	[-]
Poisson's ratio	μ	0.20	[-]	Strength	fc	34.00	[MPa]
Shear modulus	G	14675	[N/mm2]	Nominal strength	fck	40.00	[MPa]
Compression modulus	K	19567	[N/mm2]	Tensile strength	fctm	3.51	[MPa]
Nominal Weight	γ	25.0	[kN/m3]	Tensile strength	fctk,05	2.46	[MPa]
Mean density	ρ	2400.0	[kg/m3]	Tensile strength	fctk,95	4.56	[MPa]
Elongation coefficient	α	1.00E-05	[1/K]	Bond strength	fbd	3.68	[MPa]
				Service strength	fcm	48.00	[MPa]
				Fatigue strength	fcd,fat	19.04	[MPa]
				Tensile strength	fctd	1.64	[MPa]
				Tensile failure energy	Gf	0.15	[N/mm]

Mat 2 Y 1860 A (EN 1992)

Young's modulus	E	195000	[N/mm2]	Safetyfactor		1.10	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	1600.00	[MPa]
Shear modulus	G	75000	[N/mm2]	Compressive yield	fyc	1600.00	[MPa]
Compression modulus	K	162500	[N/mm2]	Tensile strength	ft	1860.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	1860.00	[MPa]
Mean density	ρ	7850.0	[kg/m3]	Ultimate strain		60.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.		0.75	[-]
max. thickness	t-max	18.00	[mm]	EN 1992 bond coeff.	k1	1.60	[-]
Relaxation	EN-1992	Class	2	Hardening modulus	Eh	0.00	[MPa]
Relaxation	$\rho(1000h)$	2.50	[%]	Proportional limit	fp	1600.00	[MPa]
				Dynamic allowance	σ -dyn	160.87	[MPa]

Mat 4 B 500 B (EN 1992)

Young's modulus	E	200000	[N/mm2]	Safetyfactor		1.15	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	500.00	[MPa]
Shear modulus	G	76923	[N/mm2]	Compressive yield	fyc	500.00	[MPa]
Compression modulus	K	166667	[N/mm2]	Tensile strength	ft	540.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	540.00	[MPa]
Mean density	ρ	7850.0	[kg/m3]	Ultimate strain		50.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.		1.00	[-]
max. thickness	t-max	32.00	[mm]	EN 1992 bond coeff.	k1	0.80	[-]
				Hardening modulus	Eh	0.00	[MPa]
				Proportional limit	fp	500.00	[MPa]
				Dynamic allowance	σ -dyn	152.17	[MPa]

Mat 15 C 40/50 (EN 1992)

Young's modulus	E	35220	[N/mm2]	Safetyfactor		1.50	[-]
Poisson's ratio	μ	0.20	[-]	Strength	fc	34.00	[MPa]
Shear modulus	G	14675	[N/mm2]	Nominal strength	fck	40.00	[MPa]
Compression modulus	K	19567	[N/mm2]	Tensile strength	fctm	3.51	[MPa]
Nominal Weight	γ	25.0	[kN/m3]	Tensile strength	fctk,05	2.46	[MPa]
Mean density	ρ	2400.0	[kg/m3]	Tensile strength	fctk,95	4.56	[MPa]
Elongation coefficient	α	1.00E-05	[1/K]	Bond strength	fbd	3.68	[MPa]

FRP_bridge_v1

Materials

Mat 15 C 40/50 (EN 1992)

			Service strength	fcm	48.00	[MPa]
			Fatigue strength	fcd,fat	19.04	[MPa]
			Tensile strength	fctd	1.64	[MPa]
			Tensile failure energy	Gf	0.15	[N/mm]

Mat 18 B 500 B (EN 1992)

Young's modulus	E	200000	[N/mm2]	Safetyfactor		1.15	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	500.00	[MPa]
Shear modulus	G	76923	[N/mm2]	Compressive yield	fyc	500.00	[MPa]
Compression modulus	K	166667	[N/mm2]	Tensile strength	ft	540.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	540.00	[MPa]
Mean density	ρ	7850.0	[kg/m3]	Ultimate strain		50.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.		1.00	[-]
max. thickness	t-max	32.00	[mm]	EN 1992 bond coeff.	k1	0.80	[-]
				Hardening modulus	Eh	0.00	[MPa]
				Proportional limit	fp	500.00	[MPa]
				Dynamic allowance	σ -dyn	152.17	[MPa]

Mat 19 S 355 (EN 1993)

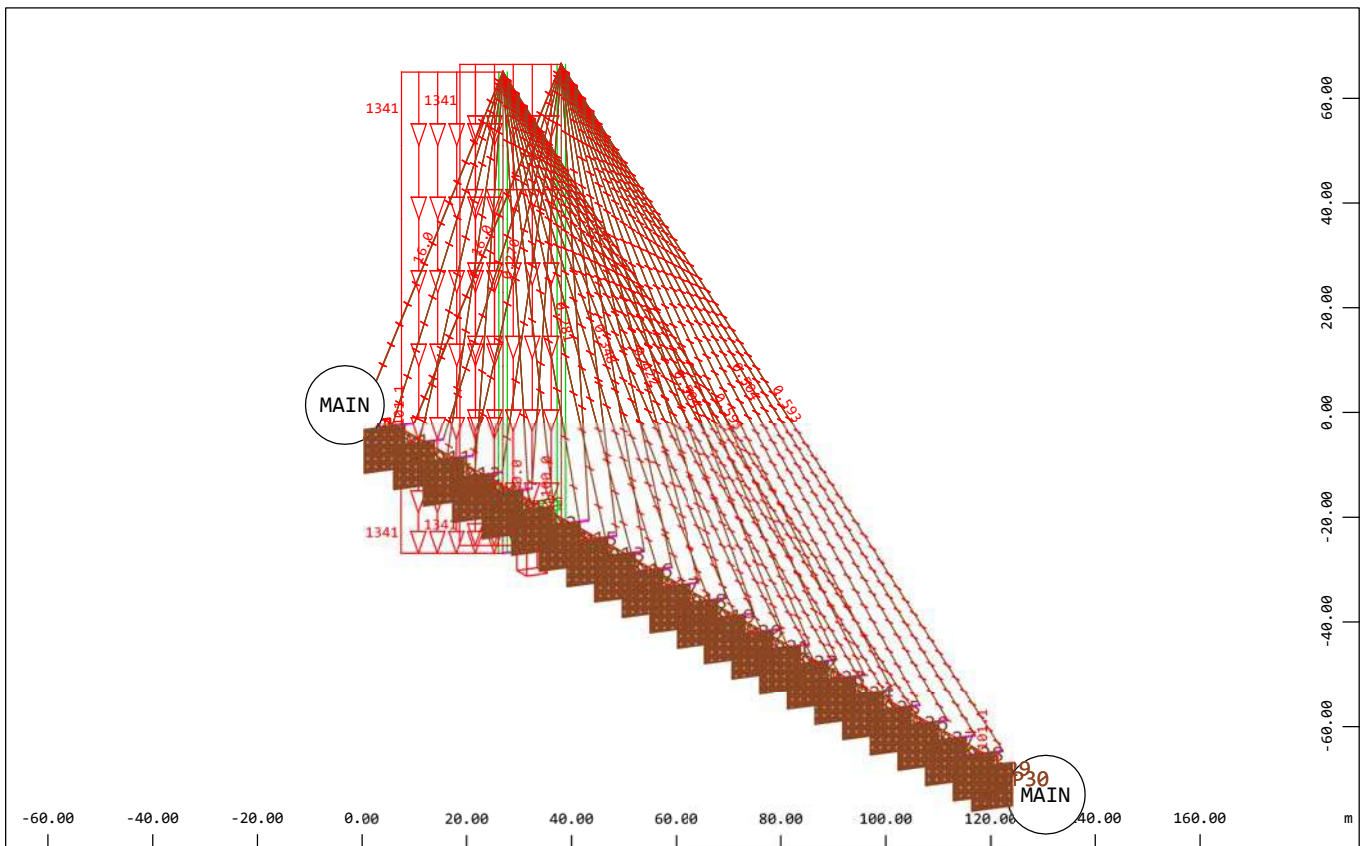
Young's modulus	E	210000	[N/mm2]	Safetyfactor		1.00	[-]
Poisson's ratio	μ	0.30	[-]	Yield stress	fy	355.00	[MPa]
Shear modulus	G	80769	[N/mm2]	Compressive yield	fyc	355.00	[MPa]
Compression modulus	K	175000	[N/mm2]	Tensile strength	ft	490.00	[MPa]
Nominal Weight	γ	78.5	[kN/m3]	Compressive strength	fc	490.00	[MPa]
Mean density	ρ	7850.0	[kg/m3]	Ultimate strain		100.00	[o/oo]
Elongation coefficient	α	1.20E-05	[1/K]	relative bond coeff.		0.00	[-]
max. thickness	t-max	40.00	[mm]	EN 1992 bond coeff.	k1	0.00	[-]
				Hardening modulus	Eh	0.00	[MPa]
				Proportional limit	fp	355.00	[MPa]
				Dynamic allowance	σ -dyn	0.00	[MPa]

FRP_bridge_v1
 Generation of Node and Element Loads

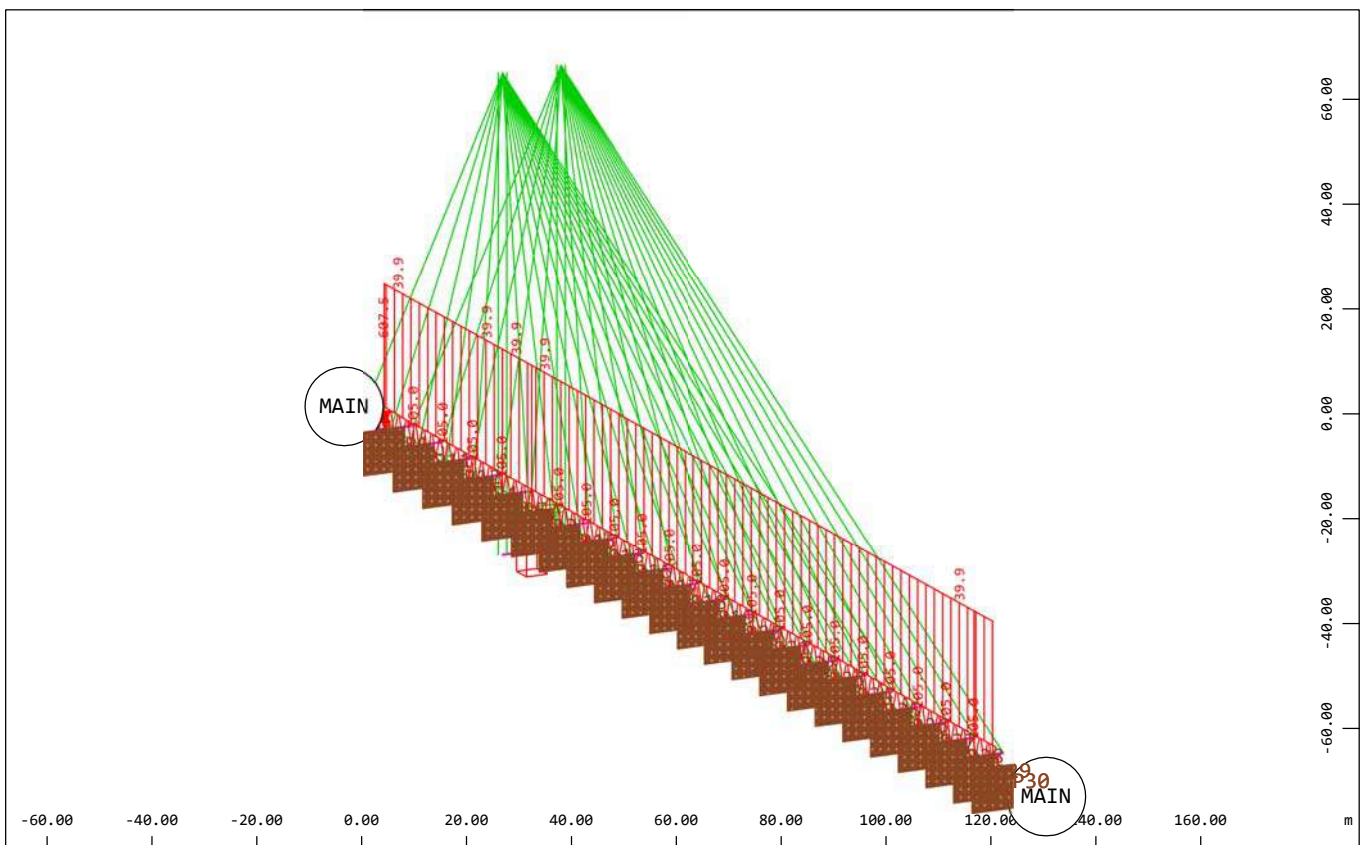
Actions

type	part	sup	Designation	γ -u	γ -f	γ -a	ψ_0	ψ_1	ψ_2	ψ_{1inf}
G_1	G	perm	dead load g1	1.35	1.00	1.00	1.00	1.00	1.00	1.00
G_2	G	perm	ADL	1.35	1.00	1.00	1.00	1.00	1.00	1.00
P	P	perm	prestressing	1.00	1.00	1.00	1.00	1.00	1.00	1.00
L_T	Q	excl	Traffic load TS of EC/DIN-FB	1.35	0.00	1.00	0.75	0.75	0.00	0.80
L_U	Q	excl	Traffic load UDL of EC/DIN-FB	1.35	0.00	1.00	0.40	0.40	0.00	0.80
W	Q	excl	wind loading	1.50	0.00	1.00	0.60	0.20	0.00	0.60
			Reliability factor	Kfi	1.000					
			Reduction factor	xsi	0.890					
type	action		γ -u, γ -f, γ -a	partial safety factors for unfavourable/favourable/accidental						
part	partition of the action		$\psi_0,\psi_1,\psi_2,\psi_{1inf}$	combination coefficients						
sup	superposition type									

FRP_bridge_v1
 Graphical Output



All loads (in components), Loadcase 1 Self-weight , (1 cm 3D = unit) Beam dead load in global Z (Unit=1000. kN/m, Min=-1341. Max=-100.0), Cable dead load in global Z (Unit=1000. kN/m, Min=-16.0 Max=-0.250) M 1 : 1444
 X * 0.502
 Y * 0.906
 Z * 0.962



All loads (in components), Loadcase 2 ADL , (1 cm 3D = unit) Beam single load (force) in global Z (Unit=500.0 kN, Min=-607.5 Max=-4.60), Beam line load (force) in global Z (Unit=20.0 kN/m, Min=-39.9 Max=-39.9) M 1 : 1442
 X * 0.502
 Y * 0.906
 Z * 0.962

FRP_bridge_v1

Summary of beam elements

Groups

Grp	TotLength [m]	Max.Length [m]	TotVolume [m3]	TotWeight [t]	Surface [m2]
1	191.000	14.583	10247.151	25617.878	5730.000
4	12.400	6.200	49.600	124.000	
5	264.000	12.800	969.672	2668.525	
10	176.638	88.319	0.347	4.416	
11	181.464	90.732	0.371	4.720	
12	189.233	94.616	0.402	5.117	
13	199.602	99.801	0.440	5.607	
14	212.191	106.096	0.504	6.419	
15	226.630	113.315	0.578	7.363	
16	242.588	121.294	0.663	8.444	
17	259.786	129.893	0.759	9.667	
18	277.995	138.997	0.866	11.034	
19	297.027	148.514	0.985	12.549	
20	316.735	158.367	1.116	14.218	
21	337.000	168.500	1.259	16.045	
22	357.728	178.864	1.416	18.033	
23	378.842	189.421	1.585	20.188	
24	400.281	200.141	1.767	22.516	
25	421.996	210.998	1.964	25.020	
26	176.863	88.431	0.347	4.422	
27	182.336	91.168	0.372	4.743	
28	191.111	95.555	0.406	5.168	
29	202.758	101.379	0.447	5.695	
30	216.816	108.408	27.232	346.905	
Sum	5913.018		11310.249	28968.691	
Grp primary group number					

Materials

Mat	TotLength [m]	Max.Length [m]	TotVolume [m3]	TotWeight [t]	Surface [m2]	Designation
1	203.400	14.583	10296.751	25741.878	5730.000	C 40/50 (EN 1992)
2	5445.618	210.998	43.826	558.288		Y 1860 A (EN 1992)
15	264.000	12.800	924.000	2310.000	5280.001	C 40/50 (EN 1992)
19	264.000	12.800	45.672	358.525	2830.080	S 355 (EN 1993)
Sum			11310.249	28968.691		
Mat material number						

FRP_bridge_v1

Load Case 9 (P) Prestress cables

Factor forces and moments 1.000
 unfavourable partial safety factor 1.000
 favourable partial safety factor 1.000
 Combination coefficient ψ_0 1.000 (rare)
 Combination coefficient ψ_{1inf} 1.000 (infrequent)
 Combination coefficient ψ_1 1.000 (frequent)
 Combination coefficient ψ_2 1.000 (permanent)
 Loads completely copied from load case 1999 CABL with factor 1.000

Loads acting on Cable elements

Number	Type	Load value
100001	P-Nxe	1135.1 [kN]
100002	P-Nxe	1135.1 [kN]
110001	P-Nxe	907.8 [kN]
110002	P-Nxe	907.8 [kN]
120001	P-Nxe	1037.3 [kN]
120002	P-Nxe	1037.3 [kN]
130001	P-Nxe	1084.9 [kN]
130002	P-Nxe	1084.9 [kN]
140001	P-Nxe	1164.8 [kN]
140002	P-Nxe	1164.8 [kN]
150001	P-Nxe	1252.3 [kN]
150002	P-Nxe	1252.3 [kN]
160001	P-Nxe	1349.3 [kN]
160002	P-Nxe	1349.3 [kN]
170001	P-Nxe	1453.3 [kN]
170002	P-Nxe	1453.3 [kN]
180001	P-Nxe	1563.4 [kN]
180002	P-Nxe	1563.4 [kN]
190001	P-Nxe	1678.6 [kN]
190002	P-Nxe	1678.6 [kN]
200001	P-Nxe	1798.2 [kN]
200002	P-Nxe	1798.2 [kN]
210001	P-Nxe	1921.5 [kN]
210002	P-Nxe	1921.5 [kN]
220001	P-Nxe	2049.8 [kN]
220002	P-Nxe	2049.8 [kN]
230001	P-Nxe	2171.1 [kN]
230002	P-Nxe	2171.1 [kN]
240001	P-Nxe	2359.6 [kN]
240002	P-Nxe	2359.6 [kN]
250001	P-Nxe	2260.5 [kN]
250002	P-Nxe	2260.5 [kN]
260001	P-Nxe	1179.0 [kN]
260002	P-Nxe	1179.0 [kN]
270001	P-Nxe	982.6 [kN]
270002	P-Nxe	982.6 [kN]
280001	P-Nxe	1081.9 [kN]
280002	P-Nxe	1081.9 [kN]
290001	P-Nxe	1329.8 [kN]
290002	P-Nxe	1329.8 [kN]
300001	P-Nxe	29715.2 [kN]
300002	P-Nxe	29715.2 [kN]

FRP_bridge_v1
 Modal analysis

Processing

Parameter of System of Equations

Number of unknowns 218 (Direct sparse Gauss-Solver)
 Total entries 2224
 Total entries after fill in 3143
 Mass matrix 1155 (consistent), incl. rotational masses

Control Information

Using Vectoriteration	
Iteration vectors	42
Iterations	46

Eigenvalues

No.	LC	λ [rad2/sec2]	error [-]	ω [rad/sec]	f [Hz]	T [sec]	ξ [%]	X[%]	Meff Y[%]	Z[%]	participation X[%]	Y[%]	Z[%]
1	500	5.9091E+00	0.0E+00	2.431	0.387	2.585	0.000	0.0	54.7	0.0	0.0	57.7	0.0
2	501	6.2666E+00	0.0E+00	2.503	0.398	2.510	0.000	0.0	0.0	0.0	0.0	0.0	0.0
3	502	1.0810E+01	0.0E+00	3.288	0.523	1.911	0.000	4.1	0.0	5.5	4.3	0.0	5.8
4	503	1.1586E+01	0.0E+00	3.404	0.542	1.846	0.000	0.0	6.4	0.0	0.0	6.8	0.0
5	504	2.3422E+01	0.0E+00	4.840	0.770	1.298	0.000	3.1	0.0	0.6	3.2	0.0	0.7
6	505	3.1900E+01	0.0E+00	5.648	0.899	1.112	0.000	0.0	0.0	0.0	0.0	0.0	0.0
7	506	3.5719E+01	0.0E+00	5.977	0.951	1.051	0.000	15.3	0.0	0.2	16.0	0.0	0.2
8	507	4.5996E+01	0.0E+00	6.782	1.079	0.926	0.000	35.1	0.0	3.0	36.8	0.0	3.1
9	508	6.3177E+01	0.0E+00	7.948	1.265	0.790	0.000	4.0	0.0	0.7	4.2	0.0	0.7
10	509	6.6780E+01	0.0E+00	8.172	1.301	0.769	0.000	0.2	0.0	1.4	0.2	0.0	1.4
11	510	8.6989E+01	0.0E+00	9.327	1.484	0.674	0.000	0.0	0.0	0.0	0.0	0.0	0.0
12	511	1.1735E+02	0.0E+00	10.833	1.724	0.580	0.000	0.0	0.0	0.0	0.0	0.0	0.0
13	512	1.2302E+02	0.0E+00	11.092	1.765	0.566	0.000	0.0	0.0	0.7	0.0	0.0	0.7
14	513	1.3743E+02	0.0E+00	11.723	1.866	0.536	0.000	0.0	0.0	0.0	0.0	0.0	0.0
15	514	1.8680E+02	0.0E+00	13.668	2.175	0.460	0.000	0.0	0.0	0.0	0.0	0.0	0.0
16	515	2.2693E+02	0.0E+00	15.064	2.398	0.417	0.000	0.0	0.0	0.0	0.0	0.0	0.0
17	516	2.3673E+02	0.0E+00	15.386	2.449	0.408	0.000	0.0	0.0	0.0	0.0	0.0	0.0
18	517	2.4133E+02	0.0E+00	15.535	2.472	0.404	0.000	0.0	16.5	0.0	0.0	17.8	0.0
19	518	2.5695E+02	0.0E+00	16.030	2.551	0.392	0.000	0.0	0.0	0.0	0.0	0.0	0.0
20	519	2.9043E+02	0.0E+00	17.042	2.712	0.369	0.000	0.0	0.0	0.0	0.0	0.0	0.0
21	520	2.9290E+02	0.0E+00	17.114	2.724	0.367	0.000	0.0	0.1	0.0	0.0	0.1	0.0
22	521	3.2992E+02	0.0E+00	18.164	2.891	0.346	0.000	0.5	0.0	0.0	0.5	0.0	0.0
23	522	3.5772E+02	0.0E+00	18.913	3.010	0.332	0.000	0.0	0.0	0.0	0.0	0.0	0.0
24	523	4.0474E+02	0.0E+00	20.118	3.202	0.312	0.000	0.5	0.0	0.4	0.5	0.0	0.4
25	524	4.3366E+02	0.0E+00	20.824	3.314	0.302	0.000	4.2	0.0	0.0	4.4	0.0	0.0
26	525	4.3902E+02	0.0E+00	20.953	3.335	0.300	0.000	0.0	0.0	0.0	0.0	0.0	0.0
27	526	5.1791E+02	0.0E+00	22.758	3.622	0.276	0.000	0.0	0.1	0.0	0.0	0.1	0.0
28	527	5.2244E+02	0.0E+00	22.857	3.638	0.275	0.000	0.0	0.0	0.0	0.0	0.1	0.0
29	528	5.3683E+02	0.0E+00	23.170	3.688	0.271	0.000	0.0	0.0	0.0	0.0	0.0	0.0
30	529	5.5713E+02	0.0E+00	23.604	3.757	0.266	0.000	15.7	0.0	0.0	16.3	0.0	0.0
31	530	5.6371E+02	0.0E+00	23.742	3.779	0.265	0.000	0.0	0.2	0.0	0.0	0.2	0.0
32	531	6.5150E+02	0.0E+00	25.524	4.062	0.246	0.000	0.0	0.0	0.0	0.0	0.0	0.0
33	532	6.7844E+02	0.0E+00	26.047	4.145	0.241	0.000	0.1	0.0	0.0	0.1	0.0	0.0
34	533	7.7988E+02	0.0E+00	27.926	4.445	0.225	0.000	0.0	0.0	0.0	0.0	0.0	0.0
35	534	9.1225E+02	4.7E-15	30.203	4.807	0.208	0.000	0.0	1.7	0.0	0.0	1.8	0.0
36	535	9.1836E+02	1.5E-14	30.304	4.823	0.207	0.000	0.0	1.2	0.0	0.0	1.3	0.0
37	536	9.6368E+02	1.0E-14	31.043	4.941	0.202	0.000	0.0	0.1	0.0	0.0	0.0	0.0
38	537	1.0520E+03	9.3E-12	32.434	5.162	0.194	0.000	0.0	0.0	0.0	0.0	0.0	0.0
39	538	1.0657E+03	7.8E-08	32.645	5.196	0.192	0.000	0.0	0.0	0.2	0.0	0.0	0.2
40	539	1.1707E+03	7.7E-07	34.216	5.446	0.184	0.000	0.0	0.0	0.0	0.0	0.0	0.0
41		1.1738E+03	2.5E-08	34.261	5.453	0.183	$\Sigma(\%)^1$	82.9	81.1	12.7	86.7	85.9	13.3
42		1.1953E+03	3.9E-06	34.573	5.502	0.182							

¹ Total effective mass / participation factors of activated masses in X-, Y- and Z-direction.

No.	eigenmode number	f	eigenfrequency
LC	load case	T	eigenperiod
λ	eigenvalue	ξ	modal damping ratio

FRP_bridge_v1
Modal analysis

error	relative eigenvalue error	Meff	effective modal mass in X-, Y- and Z-direction
ω	circular eigenfrequency	participation	participation factors in X-, Y- and Z-direction

FRP_bridge_v1
Wind loads

Actions

type	part	sup	Designation	γ -u	γ -f	γ -a	ψ_0	ψ_1	ψ_2	ψ_1 inf
G_1	G	perm	dead load g1	1.35	1.00	1.00	1.00	1.00	1.00	1.00
G_2	G	perm	ADL	1.35	1.00	1.00	1.00	1.00	1.00	1.00
P	P	perm	prestressing	1.00	1.00	1.00	1.00	1.00	1.00	1.00
L_T	Q	excl	Traffic load TS of EC/DIN-FB	1.35	0.00	1.00	0.75	0.75	0.00	0.80
L_U	Q	excl	Traffic load UDL of EC/DIN-FB	1.35	0.00	1.00	0.40	0.40	0.00	0.80
W	Q	excl	wind loading	1.50	0.00	1.00	0.60	0.20	0.00	0.60
			Reliability factor	Kfi	1.000					
			Reduction factor	xsi	0.890					
type	action		γ -u, γ -f, γ -a	partial safety factors for unfavourable/favourable/accidental						
part	partition of the action		$\psi_0,\psi_1,\psi_2,\psi_1$ inf	combination coefficients						
sup	superposition type									

Load Case 80 Basic wind

Factor forces and moments 1.000
Loads completely copied from load case 80 WIND BEAM TRUS CABL QUAD with factor 1.000 design

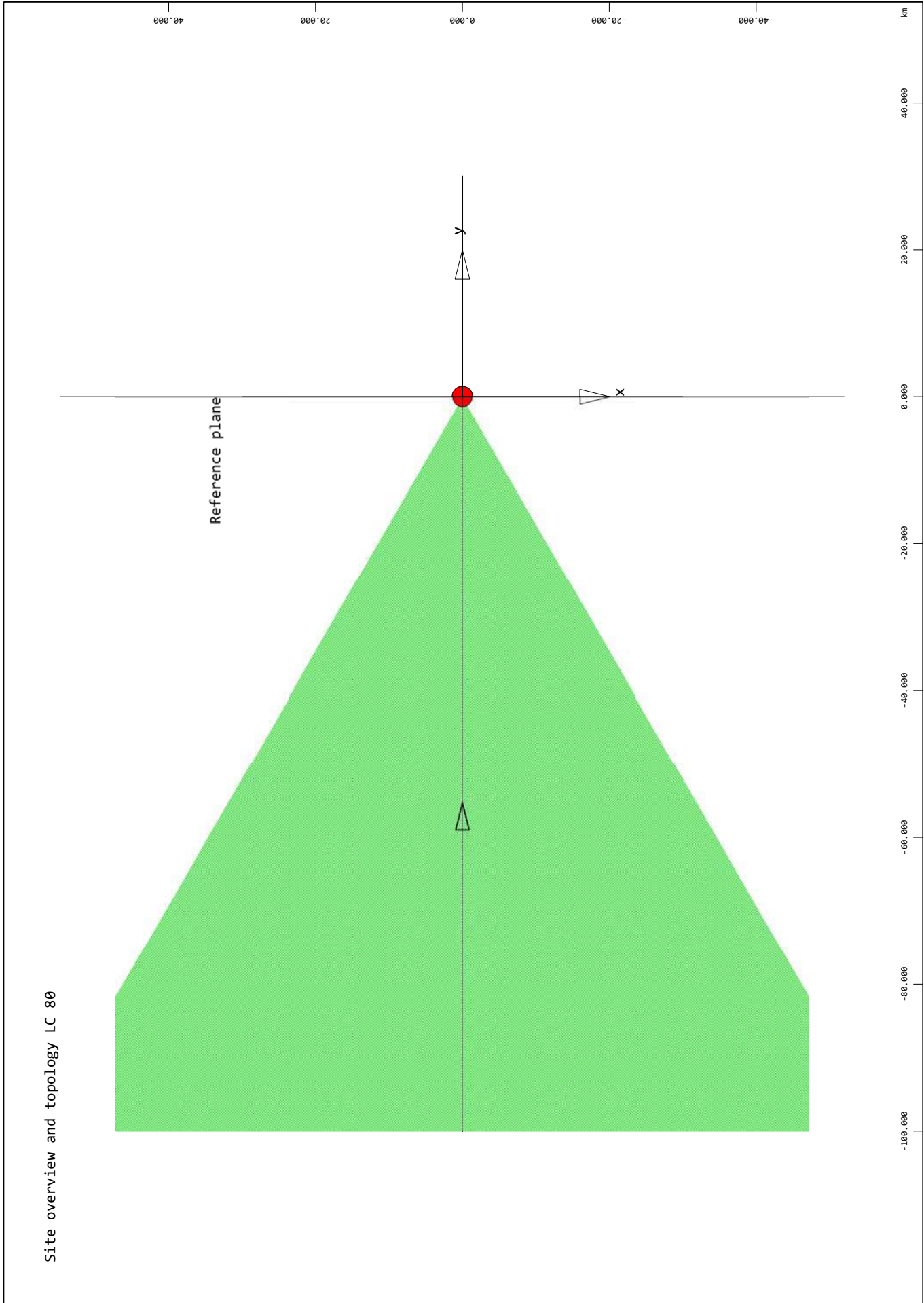
Windloading

Wind according to code NS EN-1991-1-4 (2009) Zone =Vb Terrain category III										
atmospheric wind	v	59.00	[m/sec]	Air density				ρ	1.25	[kg/m3]
Mean wind speed	vref	24.00	[m/sec]							
Surface height	hg	-12.430	[m]	Latitude				θ	45.5	[°]
Wind direction	(dx, dy, dz) =			(0.000, 1.000, 0.000)					[-]	
Reference point	(x-0, y-0, z-0) =			(10.000, 0.000, 0.000)					[m]	
Reference direction	(dxr, dyr, dzr) =			(-1.000, 0.000, 0.000)					[-]	
Wind spectra	Karman			A1	A2	A3	B	C	D	
	- Longitudinal			4.000	0.000	0.000	70.800	2.000	0.833	
	- Lateral			4.000	0.000	3021.00	283.200	2.000	1.833	
	- Vertical			4.000	0.000	3021.00	283.200	2.000	1.833	
	coherences			c-iu	c-iv	c-iw (ESDU/Karman)				
	- Longitudinal			0.221	0.114	0.114				
	- Lateral			0.110	0.167	0.167				
	- Vertical			0.110	0.167	0.167				
Roughness profile	sea			D[km]	z0[m]	Hr[m]	Hi[m]			
	-200.000	[km]		-100.000	0.300	0.000	2412.834			

Windprofiles (according to design code or explicit specification)

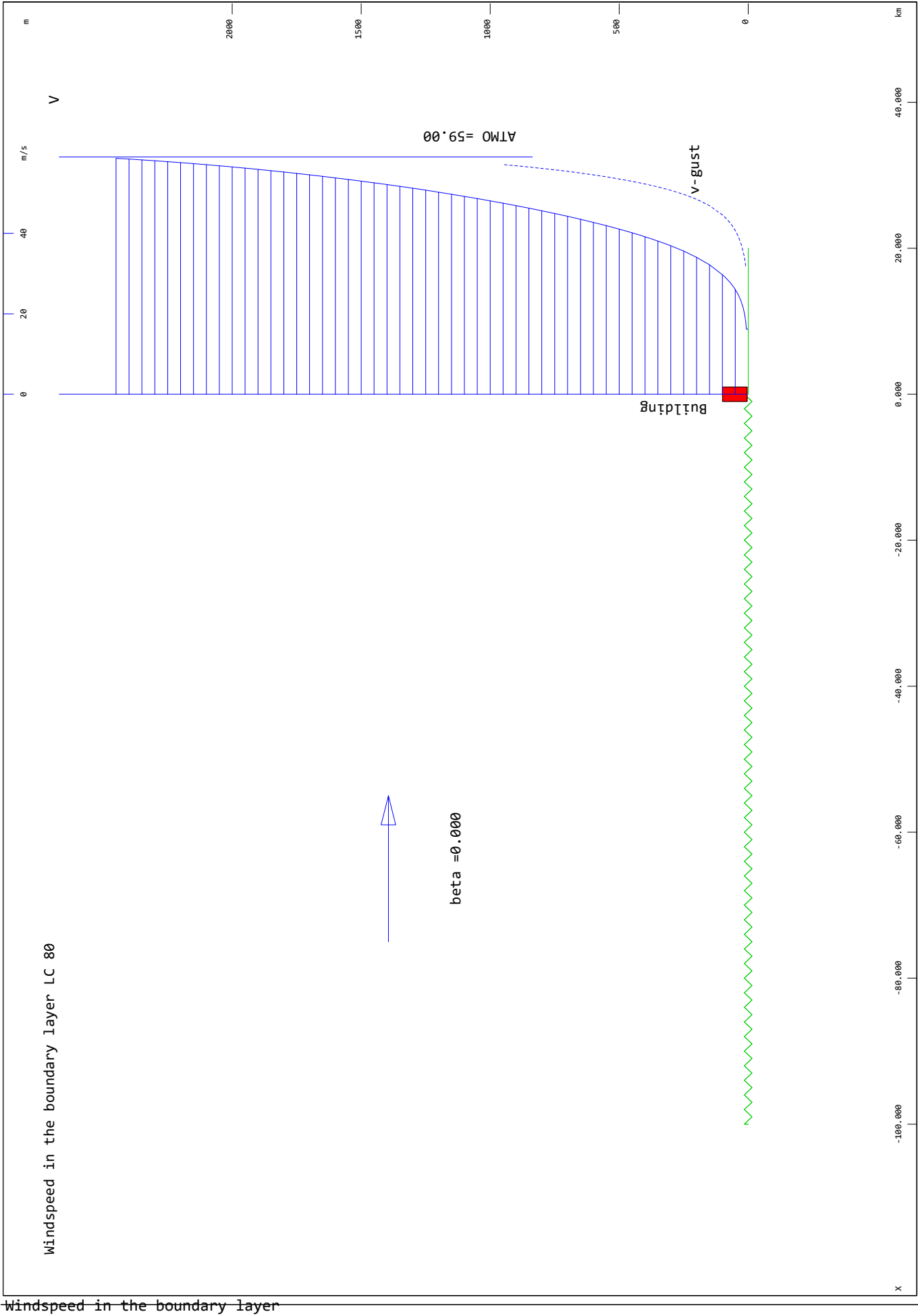
h	v-mean	v-boe	q-ref	t-lon	t-lat	t-ver	l-lon	l-lat	l-ver	phase
[m]	[m/sec]	[m/sec]	[kN/m2]	[m/sec]	[m/sec]	[m/sec]	[m]	[m]	[m]	[-]
7.000	16.18	29.23	0.53	5.13	0.00	0.00	38.841	0.000	0.000	0.000
19.000	21.31	35.15	0.77	5.30	0.00	0.00	71.406	0.000	0.000	0.000
31.000	23.67	37.98	0.90	5.43	0.00	0.00	96.247	0.000	0.000	0.000
43.000	25.28	39.84	0.99	5.51	0.00	0.00	117.501	0.000	0.000	0.000
55.000	26.53	41.24	1.06	5.55	0.00	0.00	136.529	0.000	0.000	0.000
66.000	27.47	42.26	1.12	5.58	0.00	0.00	152.585	0.000	0.000	0.000
78.000	28.36	43.20	1.17	5.60	0.00	0.00	168.948	0.000	0.000	0.000
90.000	29.14	44.00	1.21	5.61	0.00	0.00	184.353	0.000	0.000	0.000
102.000	29.83	44.70	1.25	5.62	0.00	0.00	198.975	0.000	0.000	0.000

FRP_bridge_v1
Wind loads

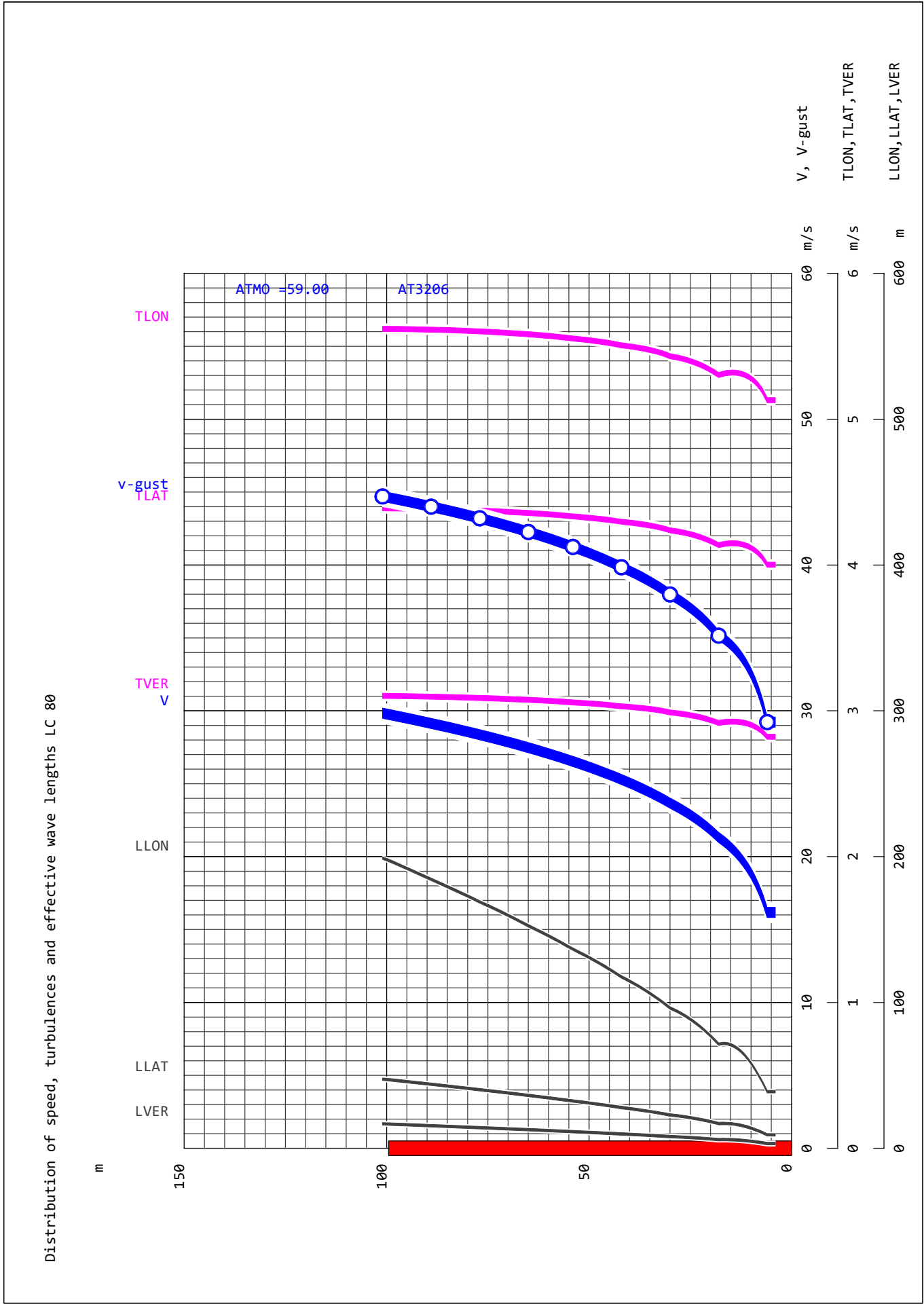


Site overview and topology

FRP_bridge_v1
Wind loads



FRP_bridge_v1
Wind loads



Distribution of speed, turbulences and effect. wave lengths

FRP_bridge_v1
 Wind loads

Loads acting on Beam elements

Number	Type	fa[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fe[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fr[-]
10001	wind	1.000	0.00	16.18	0.00	1.000	0.00	19.30	0.00	0.000
10002	wind	1.000	0.00	19.30	0.00	1.000	0.00	22.97	0.00	0.000
10003	wind	1.000	0.00	22.97	0.00	1.000	0.00	25.07	0.00	0.000
10004	wind	1.000	0.00	25.07	0.00	1.000	0.00	26.60	0.00	0.000
10005	wind	1.000	0.00	26.60	0.00	1.000	0.00	27.81	0.00	0.000
10006	wind	1.000	0.00	27.81	0.00	1.000	0.00	28.82	0.00	0.000
10007	wind	1.000	0.00	28.82	0.00	1.000	0.00	29.70	0.00	0.000
10008	wind	1.000	0.00	16.18	0.00	1.000	0.00	19.30	0.00	0.000
10009	wind	1.000	0.00	19.30	0.00	1.000	0.00	22.97	0.00	0.000
10010	wind	1.000	0.00	22.97	0.00	1.000	0.00	25.07	0.00	0.000
10011	wind	1.000	0.00	25.07	0.00	1.000	0.00	26.60	0.00	0.000
10012	wind	1.000	0.00	26.60	0.00	1.000	0.00	27.81	0.00	0.000
10013	wind	1.000	0.00	27.81	0.00	1.000	0.00	28.82	0.00	0.000
10014	wind	1.000	0.00	28.82	0.00	1.000	0.00	29.70	0.00	0.000
40001	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
40002	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50001	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50002	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50003	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50004	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50005	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50006	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50007	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50008	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50009	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50010	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50011	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50012	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50013	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50014	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50015	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50016	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50017	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50018	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50019	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50020	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50021	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000
50022	wind	1.000	0.00	19.30	0.00	1.000	0.00	19.30	0.00	0.000

Loads acting on Cable elements

Number	Type	fa[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fe[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fr[-]
100001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
100002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
110001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
110002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
120001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
120002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
130001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
130002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
140001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
140002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
150001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
150002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
160001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
160002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
170001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
170002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
180001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000

FRP_bridge_v1
 Wind loads

Loads acting on Cable elements

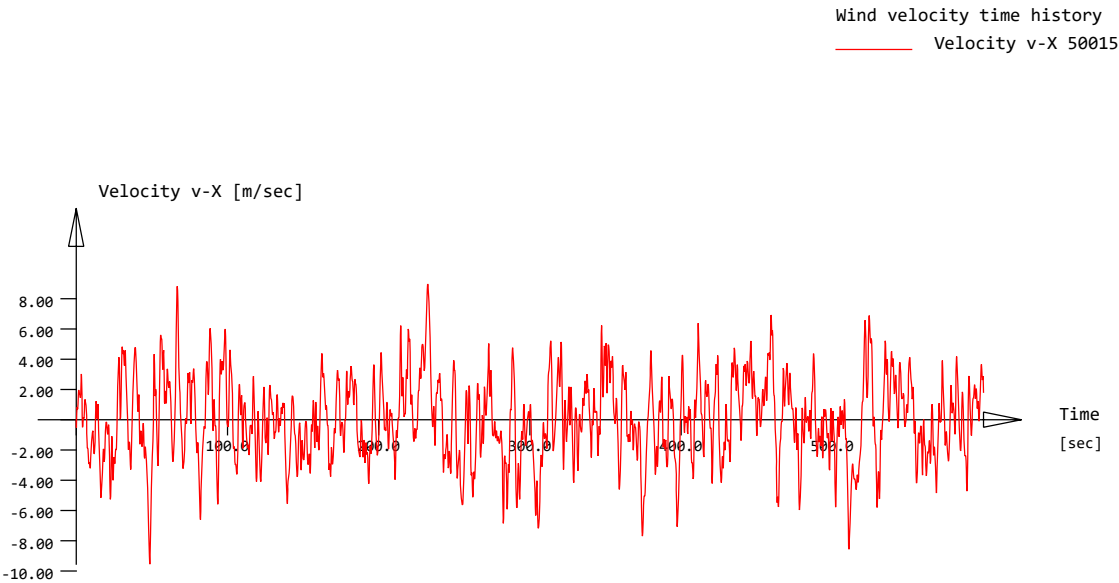
Number	Type	fa[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fe[-]	vx[m/sec]	vy[m/sec]	vz[m/sec]	fr[-]
180002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
190001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
190002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
200001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
200002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
210001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
210002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
220001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
220002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
230001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
230002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
240001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
240002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
250001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
250002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
260001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
260002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
270001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
270002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
280001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
280002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
290001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
290002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
300001	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000
300002	wind	1.000	0.00	29.70	0.00	1.000	0.00	19.30	0.00	0.000

Load Case 90

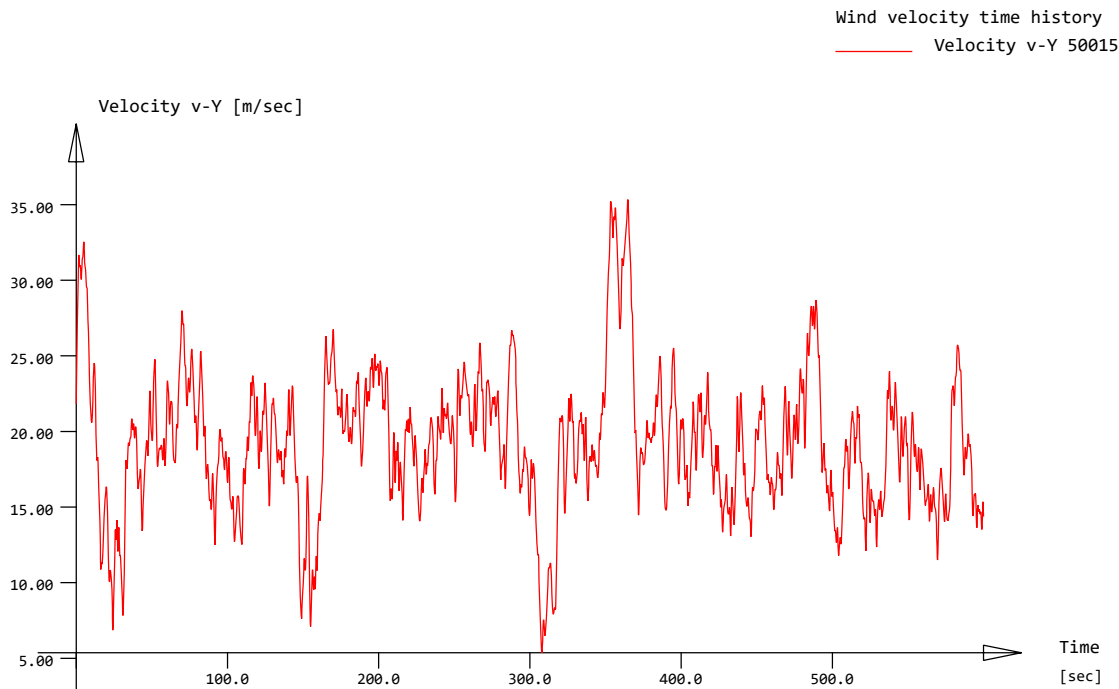
Factor forces and moments

1.000

FRP_bridge_v1
Wind velocity time history

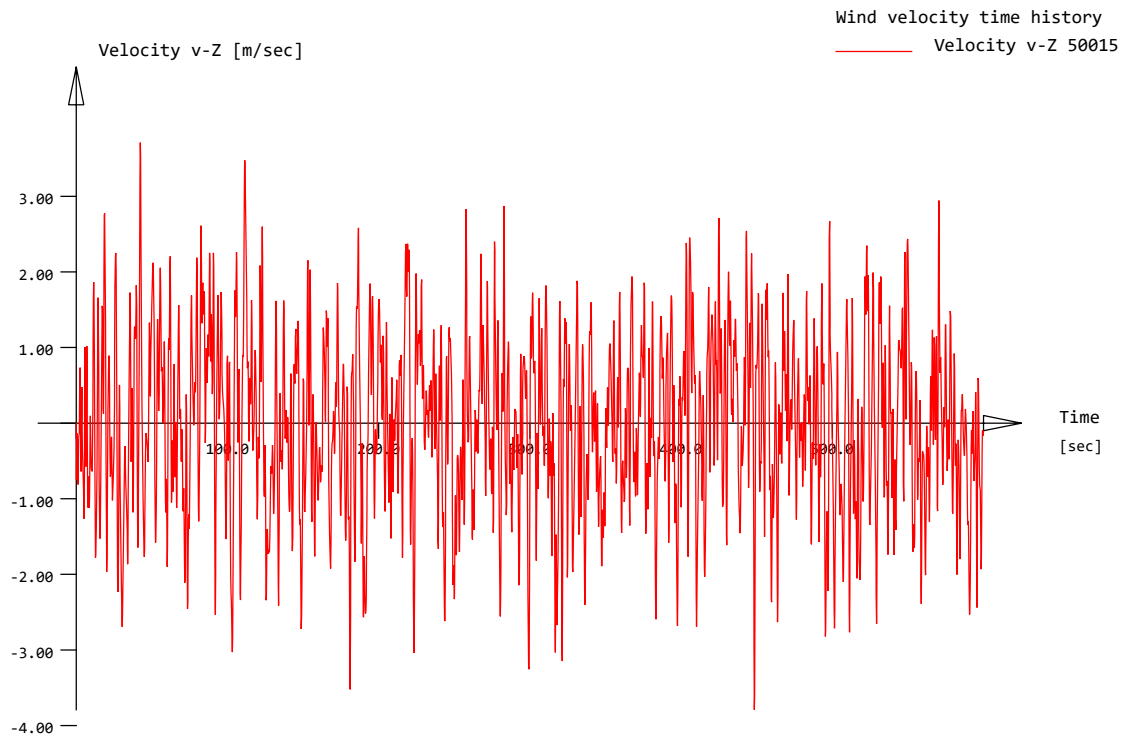


Wind velocity time history



Wind velocity time history

FRP_bridge_v1
Wind velocity time history



Wind velocity time history

FRP_bridge_v1
 Integration of wind history

System- and Control Information

Control Information

Primary load case 90 t = 0.0000 [sec] +Opt:add_F +Opt:add_GS
 Number of unknowns 218
 unknowns per node 6
 Number of timesteps 30000
 Time-step 0.0200
 Printing intervall 1
 damping factor A 0.000E+00
 damping factor B 0.000E+00
 Integr.Parameter beta 0.25
 Integr.Parameter delta 0.50
 Integr.Parameter theta 1.00

Groups

Grp	Option	CS	Factor	Rayleigh-A [1/sec]	Rayleigh-B [sec]	ξ [o/o]	Wind
1	FULL		1.000	0.102649	0.002665	0.00	
4	FULL		1.000	0.102649	0.002665	0.00	
5	FULL		1.000	0.076987	0.001999	0.00	
10	FULL		1.000	0.015397	0.000400	0.00	
11	FULL		1.000	0.015397	0.000400	0.00	
12	FULL		1.000	0.015397	0.000400	0.00	
13	FULL		1.000	0.015397	0.000400	0.00	
14	FULL		1.000	0.015397	0.000400	0.00	
15	FULL		1.000	0.015397	0.000400	0.00	
16	FULL		1.000	0.015397	0.000400	0.00	
17	FULL		1.000	0.015397	0.000400	0.00	
18	FULL		1.000	0.015397	0.000400	0.00	
19	FULL		1.000	0.015397	0.000400	0.00	
20	FULL		1.000	0.015397	0.000400	0.00	
21	FULL		1.000	0.015397	0.000400	0.00	
22	FULL		1.000	0.015397	0.000400	0.00	
23	FULL		1.000	0.015397	0.000400	0.00	
24	FULL		1.000	0.015397	0.000400	0.00	
25	FULL		1.000	0.015397	0.000400	0.00	
26	FULL		1.000	0.015397	0.000400	0.00	
27	FULL		1.000	0.015397	0.000400	0.00	
28	FULL		1.000	0.015397	0.000400	0.00	
29	FULL		1.000	0.015397	0.000400	0.00	
30	FULL		1.000	0.015397	0.000400	0.00	
CS		construction stage		Rayleigh-B		stiffness proportional damping ratio	
Factor		Factor on stiffness		ξ		modal damping ratio	
Rayleigh-A		mass proportional damping ratio		Wind		options for dynamic wind loading	

Beam Elements

Finite beam elements without intermediate sections
 Shear deformations accounted for with nonconforming SOFiSTiK-Timoshenko beam

Primary load case 90

Sum of masses and mass moments of inertia

Node	TM			RM			RMB [tm2]
	X[t]	Y[t]	Z[t]	X[tm2]	Y[tm2]	Z[tm2]	
total ¹	30312.580	30312.581	30312.580	4.099E+04	6.087E+04	2.652E+05	-
	S[m] ²			RM(S) ³			
	82.824	0.000	33.969	2.808E+07	8.702E-02	-9.20E+06	
				8.702E-02	6.901E+07	1.919E-02	
				-9.20E+06	1.919E-02	4.295E+07	
active ¹	30312.580	30104.526	30104.525	4.005E+04	6.087E+04	2.652E+05	-
	S[m] ²			RM(S) ³			

FRP_bridge_v1

Integration of wind history

Sum of masses and mass moments of inertia

Node	TM			RM			RMB
	X[t]	Y[t]	Z[t]	X[tm2]	Y[tm2]	Z[tm2]	
	82.824	0.000	34.207	2.783E+07	8.702E-02	-9.20E+06	
				8.702E-02	6.568E+07	1.945E-02	
				-9.20E+06	1.945E-02	4.011E+07	
¹ sum of the total and the active nodal masses ² coordinates of the center of gravity ³ 3x3 rotational mass matrix at the center of gravity TM translational masses in X-, Y- and Z-direction RM rotational masses about X-, Y- and Z-axis RMB warping mass							

Processing

Load Cases

-- Loadcase 90

Node	PX	PY	PZ	MX	MY	MZ	Mb
	[kN]	[kN]	[kN]	[kNm]	[kNm]	[kNm]	[kNm2]
sum	0.0	1908.1	-160.2	93.83	0.30	0.51	

Loads on kinematic constraints have been transfered to their master

Parameter of System of Equations

Number of unknowns	218 (Direct sparse Gauss-Solver)
Total entries	2272
Total entries after fill in	3182
Mass matrix	1157 (consistent), incl. rotational masses
Damping matrix	1839 (consistent)

Results

Nodal Displacements

Node	u-X [mm]	Time [sec]	u-Y [mm]	Time [sec]	u-Z [mm]	Time [sec]
MAX	4.729		79.230		26.771	
MIN	-3.191		-6.025		-39.402	

Time history for u-Y

Node 57 (E 105) u-Z MINIMUM = -35.242 MAXIMUM = 24.941 [mm]
 Node 57 (E 106) u-RX MINIMUM = 0.076 MAXIMUM = 0.695 [mrad]
 Node 57 (E 104) u-Y MINIMUM = 1.516 MAXIMUM = 14.939 [mm]

stored in database for DYNR with identification no 499

Time history for a-Z

Node 57 (E 105) a-Z MINIMUM = -0.35 MAXIMUM = 0.38 [m/sec2]

stored in database for DYNR with identification no 499

Time history for u-Y

Node 45 (E 81) u-Z MINIMUM = -23.977 MAXIMUM = 16.582 [mm]
 Node 45 (E 82) u-RX MINIMUM = -0.000 MAXIMUM = 0.256 [mrad]
 Node 45 (E 80) u-Y MINIMUM = 1.968 MAXIMUM = 20.775 [mm]

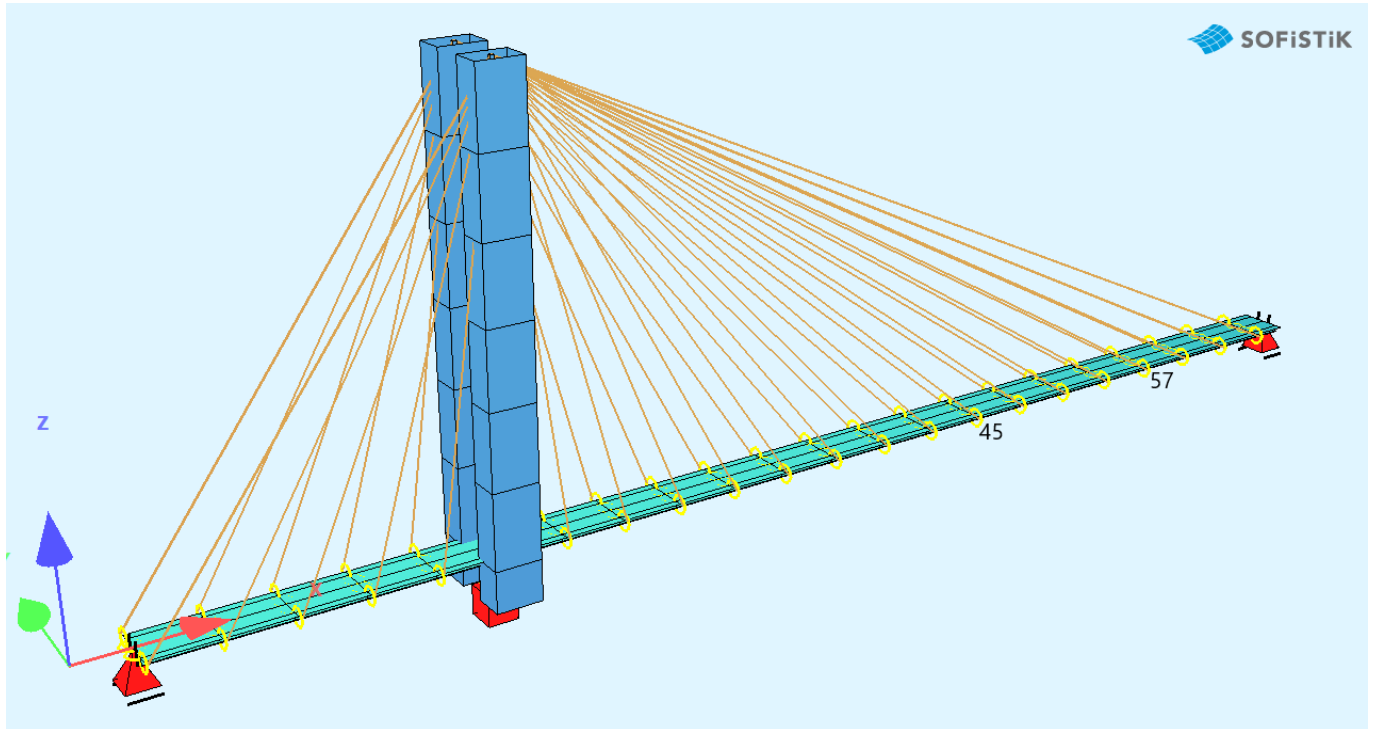
stored in database for DYNR with identification no 500

Time history for a-Z

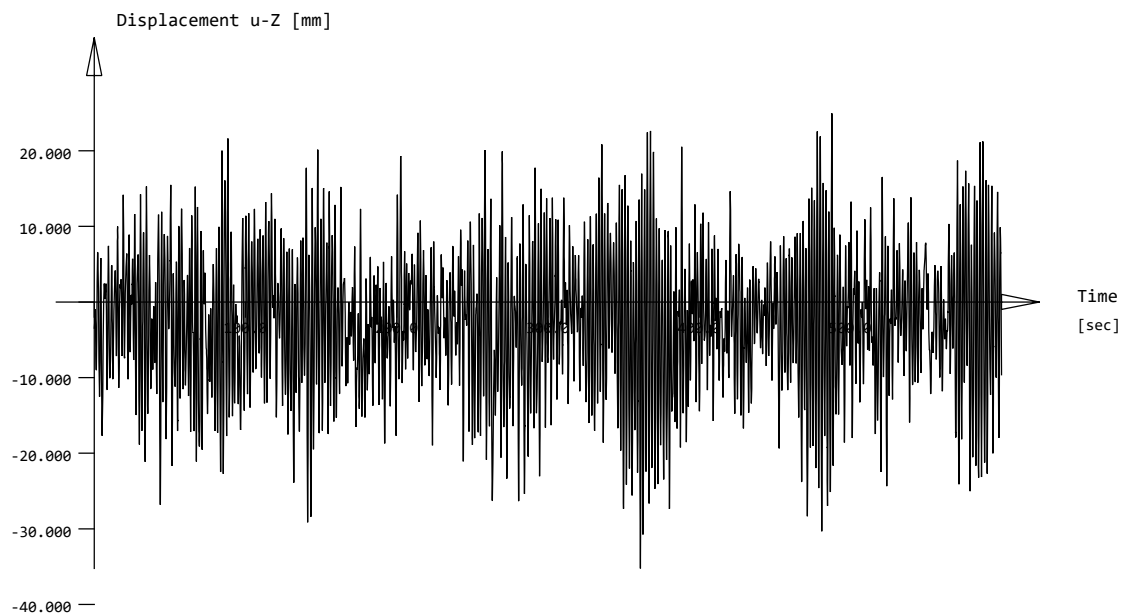
Node 45 (E 81) a-Z MINIMUM = -0.30 MAXIMUM = 0.33 [m/sec2]

stored in database for DYNR with identification no 500

FRP_bridge_v1

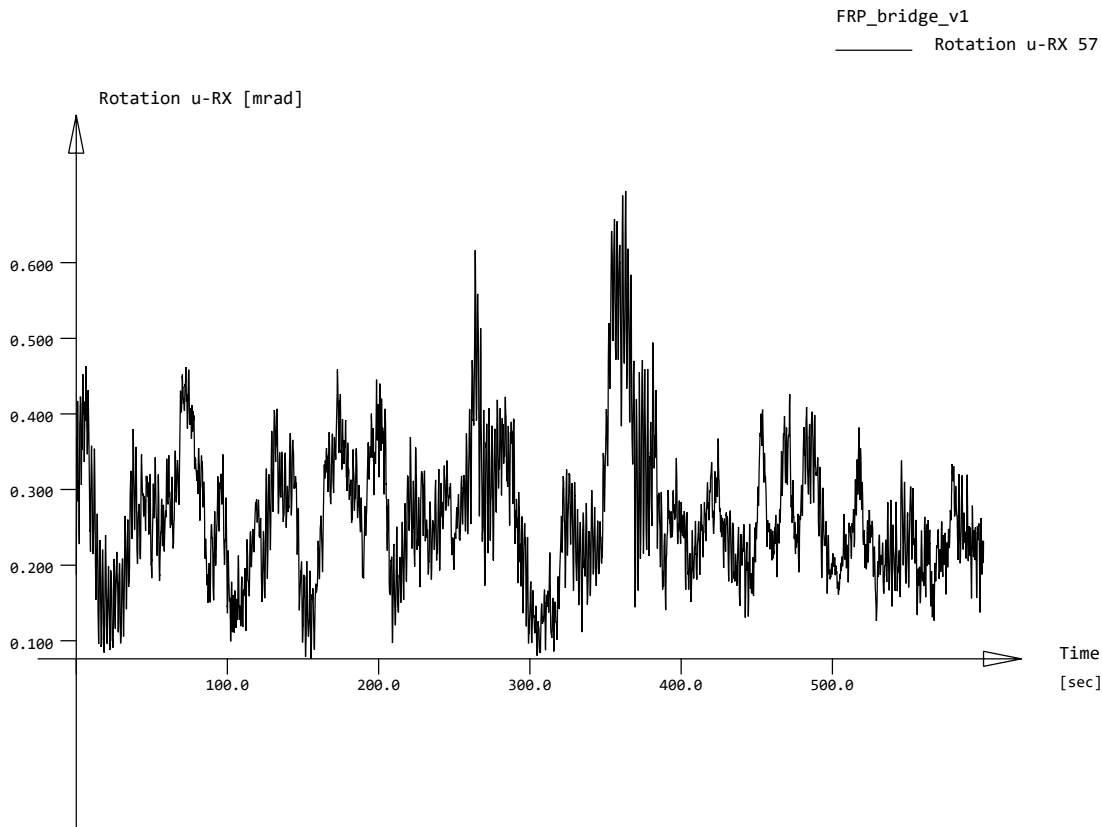


FRP_bridge_v1
 Displacement u-Z 57

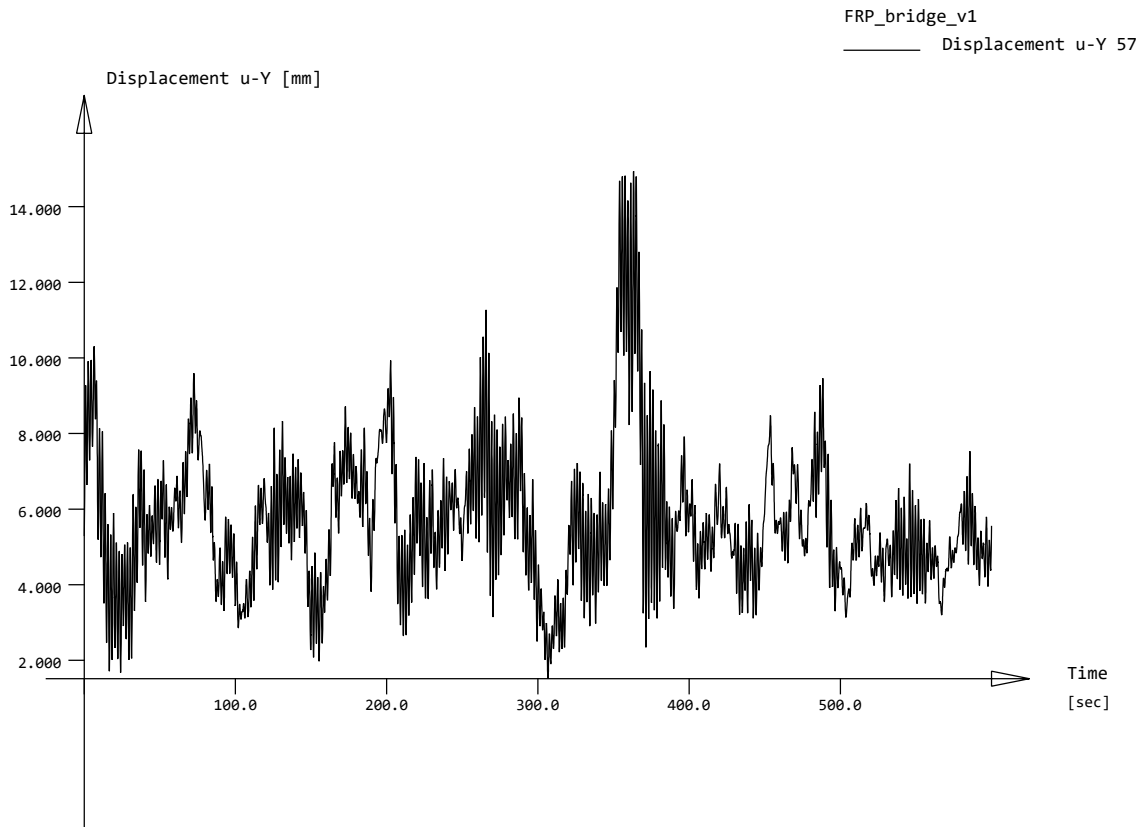


FRP_bridge_v1

FRP_bridge_v1

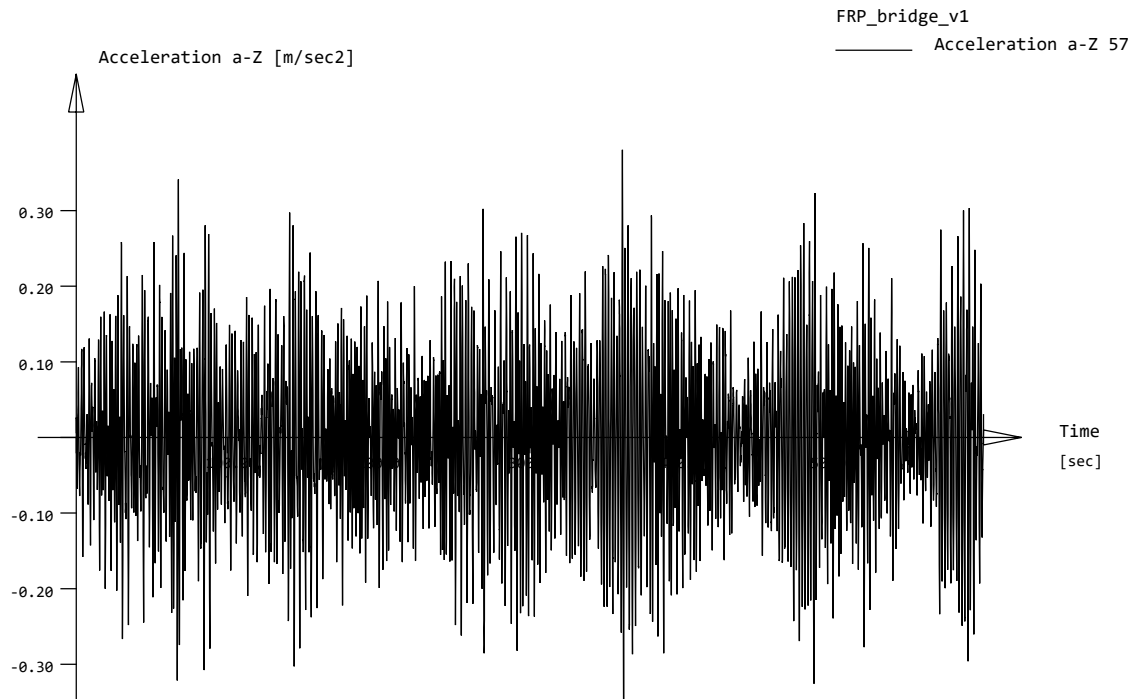


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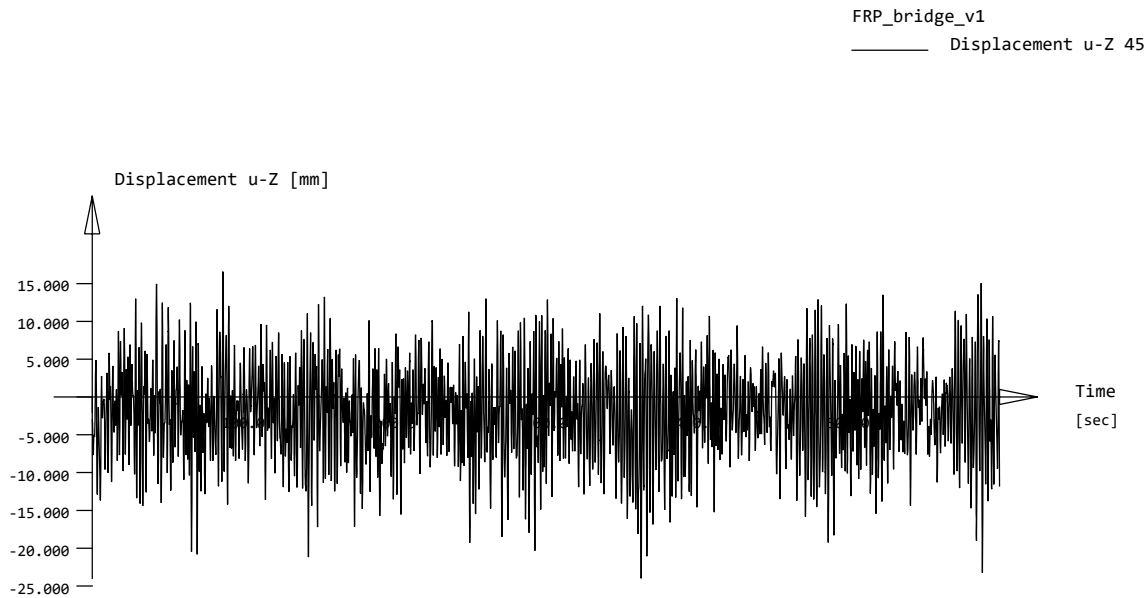


FRP_bridge_v1

FRP_bridge_v1

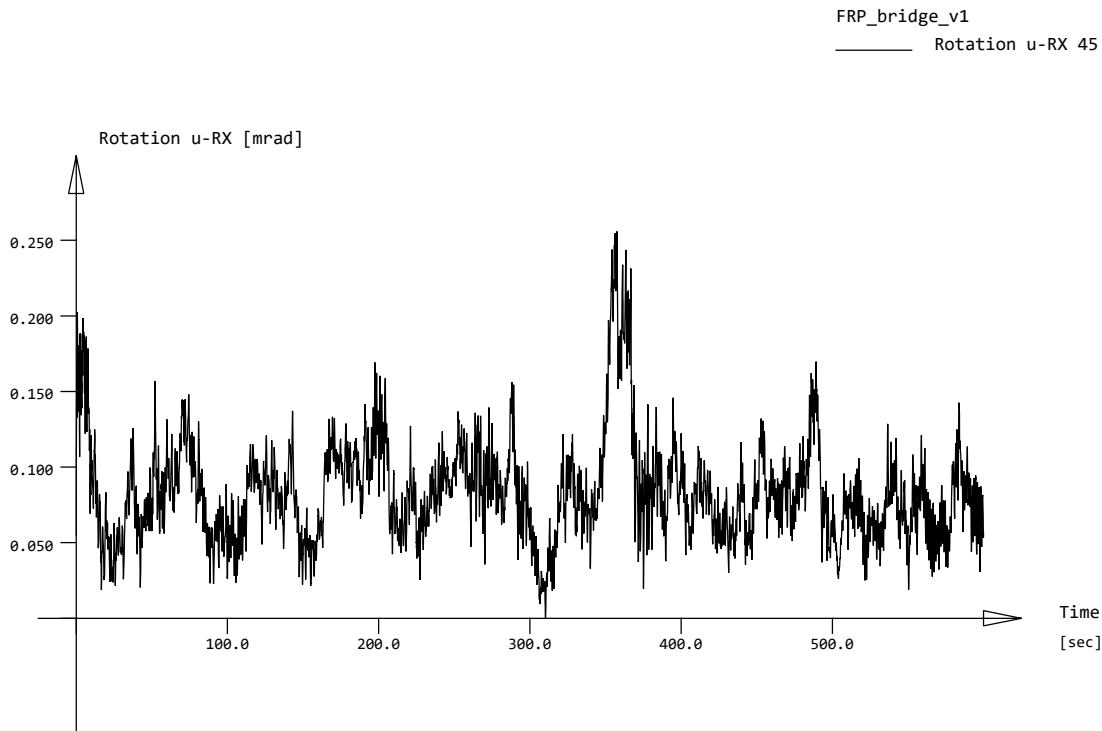


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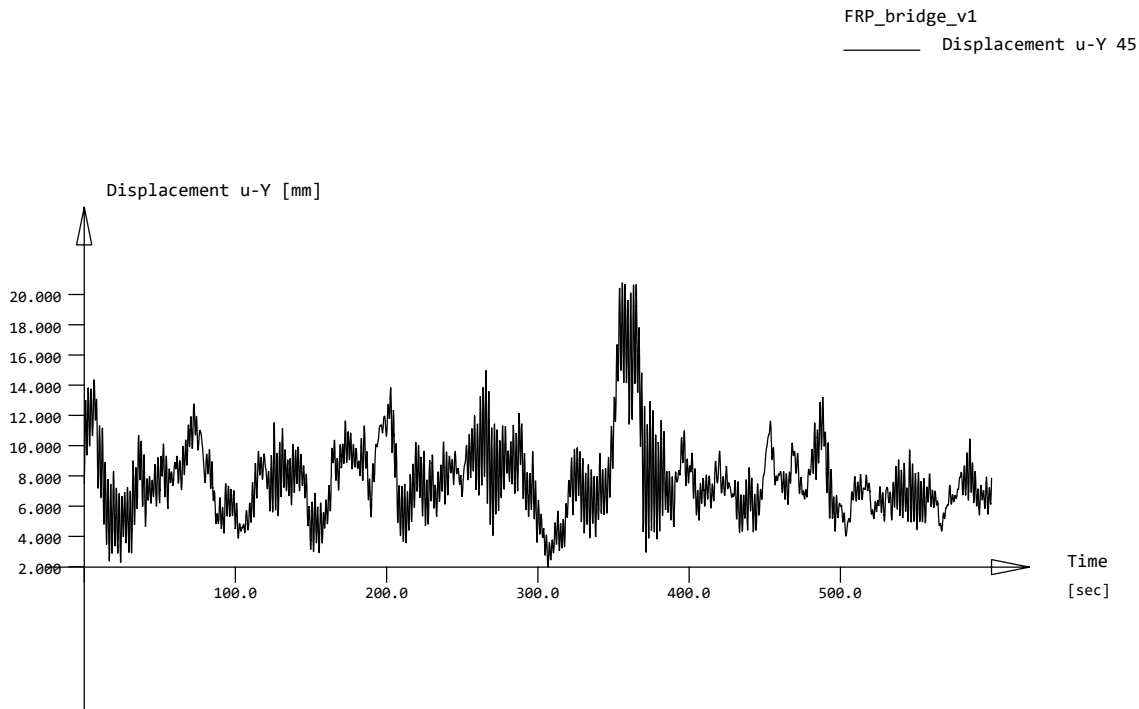


FRP_bridge_v1

FRP_bridge_v1

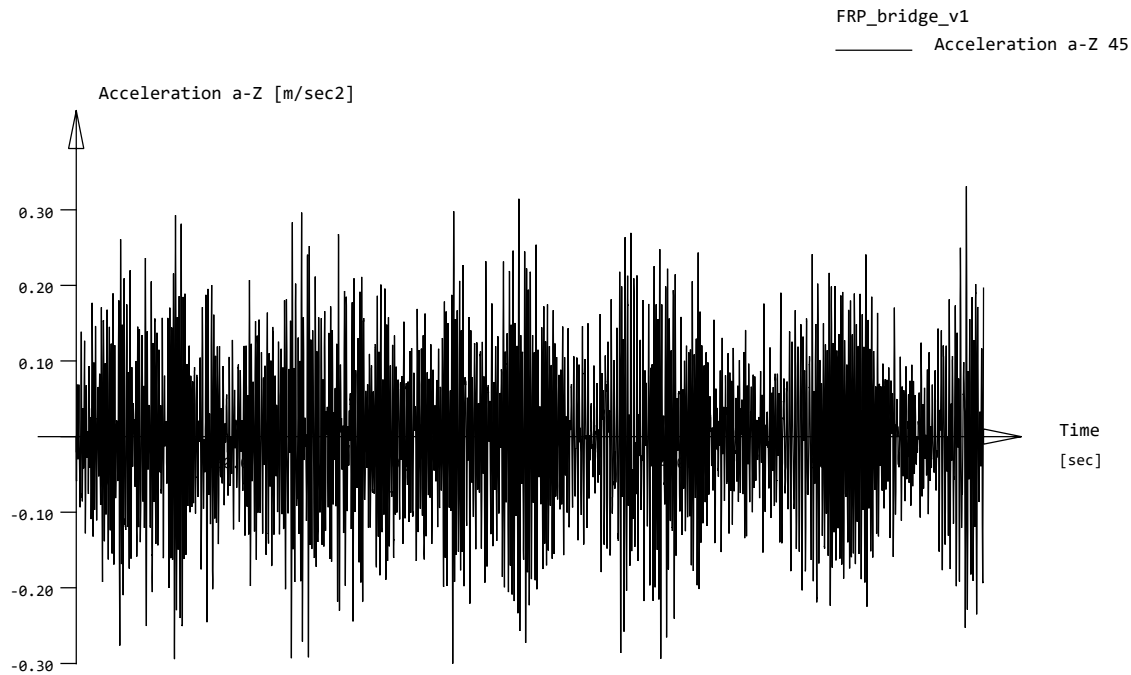


FRP_bridge_v1

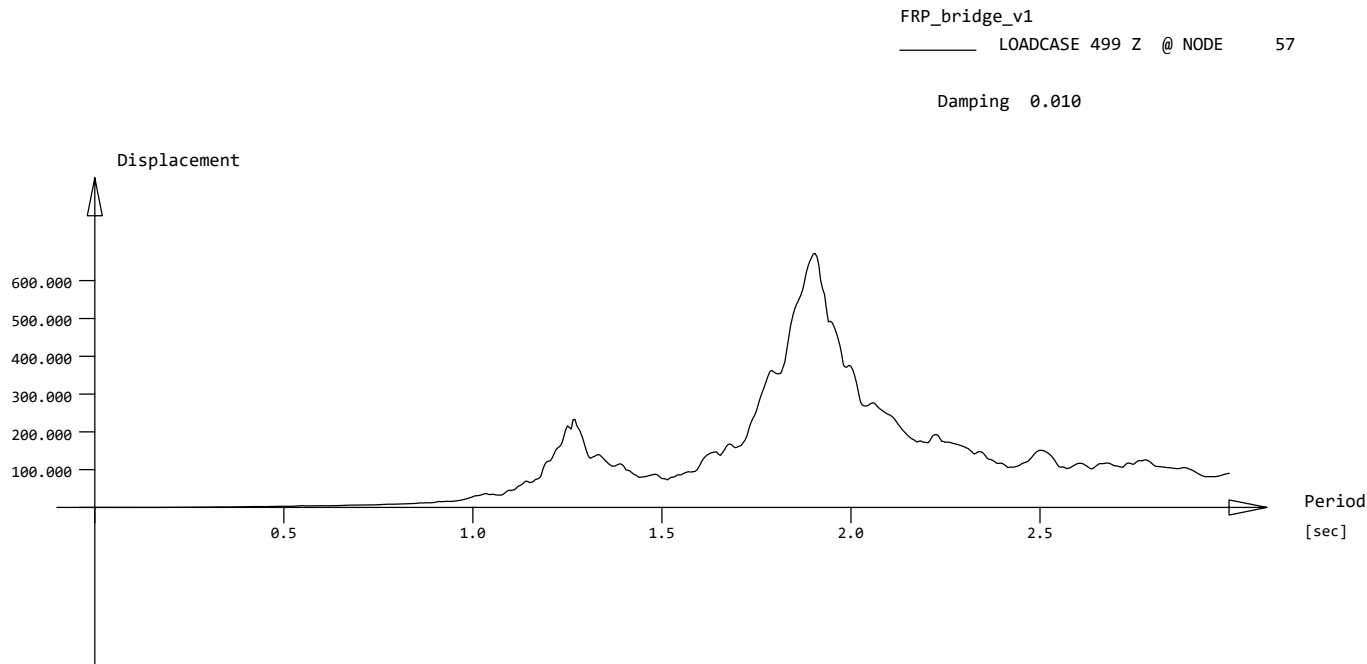


FRP_bridge_v1

FRP_bridge_v1



FRP_bridge_v1



FRP_bridge_v1