INSTITUTO SUPERIOR DE ENGENHARIA DO PORTO

MESTRADO EM ENGENHARIA CIVIL




Modelling and structural analysis of Ponte do Infante

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julho de 2017

POLITÉCNICO
DO PORTO

FACULTEIT INDUSTRIELE INGENIEURSWETENSCHAPPEN

TECHNOLOGIECAMPUS GENT

Instituto Superior de Engenharia do Porto

Modelling and structural analysis of Ponte do Infante

Erasmus Project (Projecto Erasmus)

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Co-promoter: *Prof. Stef Pillaert*

Thesis submitted to obtain the degree of Master in Engineering Technology Construction

Academic Year 2016-2017

Preface

When I was still in high school I already knew that one day I would like to go study abroad. I have always been a person with an open personality. I love to travel and get to know new cultures, mostly the ones who are so different from the one we have in Belgium. This is exactly what Erasmus offers you to do. When given the opportunity to go on Erasmus, I instantly knew I was going to do it. When I got the confirmation I was selected I was sure this was going to be an experience I will always carry with me. Erasmus is more than just studying in a different country with different people. It is a whole new lifestyle. Leaving everything and everybody you know behind, to do something new for the next 5 months. It is a challenge, but one I am glad I took.

Porto is a really great city to stay. It is not too big (the historic centre). You can walk wherever you want to go, since taking a bike is not an option with every street going either way up or way down. The weather is, of course, a plus point. It gives you the opportunity to practice surfing or enjoy a day of relaxing outside. In general, it always puts you in a good mood. Porto is also what we call a real Erasmus city, there are over a 1000 Erasmus students here, which means there are plenty of people for you to make new friends. Since everybody is looking to meet new people, you will fit right in, in no time. I am really glad I took the chance I got to study abroad and would encourage everybody to do the same.

Arriving at Porto Mr. Santos proposed two different subjects to me. Either a footbridge designed originally by Leonardo Da Vinci or the Ponte do Infante, which is one of the many arch bridges crossing the Douro. I told him I would like to study the Ponte Infante if it was possible. He gathered some information about the bridge and the project could start.

I would like to thank everybody involved in this project to help me through it. First of all my promoter Ricardo Santos. Without the help from him whenever I needed it, this wouldn't have been possible. I learned a lot from him and not only about the project. He always had something interesting to tell and I really appreciated that. Stef Pillaert, my promoter in Belgium also helped me a great deal. Whenever I had a question regarding SCIA I could just ask him and get a reply in no time. I would also like to thank both schools for offering me this opportunity to go on Erasmus and spend my time like I did the past 5 months. Special thanks to Guido Kips and Ilse Roeland who helped me a great deal with the paperwork for this period. Here in Porto, I would like to thank Goreti Araújo at ISEP's international relations office. She guided me here the first weeks and made sure I got the best start I could have hoped for.

An Erasmus project is different than a thesis in Belgium on so many levels. It is hard to compare both by just looking at the work. There is a whole story behind this work which tells so much more. In Belgium, your master thesis is sometimes made in combination with a company. This leads to a good integration into the working sector which you most likely advance into next year. It is also a good representation of what you could achieve after 4 years of studying. Every question you have can be answered quickly and communications issues almost don't exist.

On Erasmus this is also partly true. It shows what you can achieve on your own or together as a team. It is a good representation of what you're worth as an engineer, but it is more than just that. There is another part to the project. One that cannot be seen inside, but happened behind the scene. The social skills you acquired, the language barrier that had to be overcome to communicate with everybody willing to help you. A different environment with different people. Needing to take care of yourself every day. These are skills that are not represented in this project, but I am glad I learned skills that no doubt will help me further in whatever it is I will do later.

Last but not least I would like to thank everybody who was here supporting me. My family who kept asking how the project was going and gave me the motivation I needed to keep working hard. Ilse who sat almost every day beside me in the library to work on her own project and countless other people surrounding me in Porto which helped me achieve my goals. As much as there needed to be worked on the project, there was also sometimes need for detention. The better you spend your free time, the harder you work, looking forward to that free time again. In general, almost everybody here in Porto helped me in one way or another.

Abstract

Abstract

The subject of this thesis is to make a check calculation of the Ponte do Infante. One of the six bridges making the connection between Porto and Vila Nova de Gaia. All calculations are made according to the most recent versions of the Eurocodes together with the Belgian/Portuguese National Annexes.

A 3D model was made according to the original plans (from the original designer) and in situ observation. Calculations are made with finite element software: SCIA Engineer. Simplifications have been made from reality, which are discussed inside the paper. The loads applied onto the bridge are calculated manually according to the Eurocodes. Final results and conclusion regarding the different checks are drawn in the last chapter.

The results lead to a better understanding of how the bridge was designed and how the loads are properly distributed throughout the structure.

Resumo

O tema desta tese é fazer um cálculo de verificação da Ponte do Infante. Uma das seis pontes que fazem a ligação entre Porto e Vila Nova de Gaia. Todos os cálculos são feitos de acordo com as versões mais recentes dos Eurocódigos, juntamente com os Anexos Nacionais Belga / Português.

Um modelo 3D foi feito de acordo com os planos originais (do designer original) e observação in situ. Os cálculos são feitos com o software de elementos finitos: SCIA Engineer. Simplificações foram feitas a partir da realidade, que são discutidas no documento. As acções aplicadas na ponte são calculadas manualmente de acordo com os Eurocódigos. Os resultados finais e as conclusões sobre as diferentes verificações são extraídos no último capítulo.

Os resultados levaram a uma melhor compreensão de como a ponte foi projetada e como as cargas estão devidamente distribuídas por toda a estrutura.

Abstract

De opdracht van deze thesis bestond eruit een controle berekening te maken van de Ponte do Infante. Eén van de 6 bruggen die de verbinding maakt tussen Porto en Vila Nova de Gaia. Alle berekeningen zijn gemaakt aan de hand van de meeste recente versies van de Eurocodes en Belgische/Portugese nationale bijlages.

Een 3D model is gemaakt met de hulp van Autocad bestanden (afkomstig van de originele ontwerper) en onderzoek ter plaatse. De berekeningen werden deels met de hand, deels met eindige elementensoftware gemaakt. Deze software is genaamd: SCIA Engineer. Er werden een aantal vereenvoudigen toegepast voor de berekeningen konden starten. Al deze aanpassingen worden uitvoerig besproken binnenin dit document. De bespreking van de bekomen resultaten en conclusies zijn terug te vinden in het laatste hoofdstuk.

De resultaten leiden tot het beter begrijpen hoe de brug ontworpen is evenals hoe de lasten verdeeld en opgevangen worden over de volledige structuur.

Keywords: Ponte do Infante, Modelling, Structural analysis, check calculation.

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Introduction

The Ponte do Infante is one of the main bridges linking Porto to Vila Nova de Gaia. It was the latest bridge built crossing the Douro River in the centre of Porto. The design of the bridge was a great responsibility since the bridge was about to be named after Infante Dom Henrique, one of the most important persons in the history of Porto and Portugal. On top of that, it had to fit in right with the other fabulous designed bridges crossing the Douro River. This was not an easy task.

The project consisted of making calculations for the final solution of the bridge as it is right now. This with the help of AutoCAD drawings from the original designers of the bridge and recent designing/calculation software.

This document is divided into four major parts.

The first part, the literature study, talks about the background of the bridge. Who was it constructed by? For what reasons? Why this specific design? And so on. It reveals some interesting aspects of the bridge and gives a general view of the project that will be discussed through the rest of this document.

In the second part the actual modelling of the 3D model of the bridge takes place. Explaining which simplifications have been made and why. This has been done with SCIA engineer, a Belgian calculation software. The problems encountered and solutions found for them are mentioned here.

The third part handles with the definition and calculations of the loads applied on the bridge. All loads are calculated by hand. The calculations are made according to the Eurocodes in combination with the Belgian national annex / Portuguese national annex. Where the Eurocode didn't have a clear answer for a problem another solution had to be found. In general when in doubt, the worst case scenario was used to be safe.

In the fourth part the loads calculated by hand are applied on the 3D model made with SCIA. Some more calculations are made by hand, other are performed by the software. Results are obtained and conclusions are drawn from them. A conclusion regarding the general project is mentioned.

The modelling of the bridge happened with the use of AutoCAD and SCIA Engineer. The bridge was designed in SCIA Engineer, with some parts imported from AutoCAD. The modelling has not always been easy, since the information available about the bridge is rather limited. A visit to the bridge cleared up some things regarding the types of supports and joints. The visitation revealed some information that was contrary to the original plans. This way a more realistic model could be made. Some simplifications had to be made due to the lack of some features in the software.

All calculations were made according to the Eurocodes in combination with the Belgian national annex. A range of checks have been executed to give a clear view of the state of the bridge.

Chapter 1: Literature Study

1.1 History and information

The Ponte Infante D. Henrique, is one of the six bridges that cross the Douro River. Due to the wide span of the river, arch bridges are an optimal solution. The river connects the city of Porto with Vila Nova de Gaia. A good connection between both cities is necessary for the transportation through this important area, normally considered the capital of the north of Portugal.

The bridge is named in honour of Infante D. Henrique, an important figure in the Portugal of the 15th century. He was one of the sons of the King of Portugal at that time. The son was born and baptized in Porto, which was a great honour for the city [1]. This leads to honouring him by naming the bridge after him. He was responsible for the early development of Portuguese exploration and trading overseas with other continents [1].
It was the last bridge built between Porto and Gaia. The construction started in 1999 and was finished three years later in 2002. The bridge was opened to the public in 2003 [2]. The main reason for the construction of this bridge was to act as a replacement for the upper deck of the Ponte Luiz I. The upper deck of Ponte Luiz I used to be accessible by car, but with the introduction of a new metro line (which runs on the top deck of Ponte Luiz I), they decided to close it down for cars. The upper deck of the Ponte Luiz I is now only accessible by foot or subway. For this reason, a new crossing was needed at top level. Nowadays the Ponte do Infante makes the same crossing at top level possible by car. The bridge needed to be placed more or less close to the original section, so the current traffic could be guided towards the new crossing [3][4].

The design of the bridge was the subject of an international competition. It was won by IDEAM and AFA engineers in 1997. The leading engineers behind the project were Francisco Millares Mato and Antonio Adão da Fonseca, under the coordination of Jose Antonio Fernández Ordoñez. The construction companies involved in the project were NESCO & EDIFER who worked together in a temporary joint venture. The structural design team included engineers Renato Bastos, Pedro Fradique Morujão and Luís Pedro Moás, of AFAssociados – Projectos de Engenharia, SA, Luis Matute Rubio, Javier Pascual Santos and Arturo Castellano Ortuño, of IDEAM, SA, and, as geotechnical specialists, José António Mateus de Brito and José Manuel Romeiro, of CENOR [3].

There are some interesting facts about the bridge, which tells you a lot about how exceptional the design of the bridge is. The bridge looks like a classic arch bridge, where the arch supports the main deck. In reality they both support each other. The deck gives extra stability to the slender arch, while the arch still supports the deck loads. The deck and arch are connected at the centre of the bridge over a span of 70 meters. Outside the middle part, 3 more pillars on each side connect them to each other. The total length of the bridge comes down to 371 meters. The arch itself, on the other hand, has a span of 280 meters. This is because the arch is only used to cross the river, while the bridge still continues on the land for some additional length. When the arch is studied more closely one sees that it is not one continuous arch. Instead it is a cohesion of multiple straight beams, losing some of the benefits of pure arches. Due to the steep slope of the arch, it also works less as an arch, and more like a beam.

The bridge has been equipped with sensors everywhere. This allows for a good follow up of the bridge during the lifespan (at least 100 years). The supervision makes it possible to take action in time if required [3].

Some general information about the bridge can be found in the table (Table 1) below [2].

Table 1: General information about Ponte do Infante

Cost of construction	19 153 839 EUR
Volume Concrete	22 500 m³
Prestressing steel	660 t
Reinforcing steel	3800 t
Length	371 m
Height (above the river)	75 m

This gives a general idea of the scale of the project.

The design of the bridge has been made simple, but elegant. It does not contain any type of decoration. Every part of the bridge has a strictly structural function. When looked at from the side (Figure 1), it looks incredible, thanks to the shallow arch. It looks like the bridge is flying over the Douro River and this in a really delicate way, yet the calculations that are about to be made tell another story [3].

Figure 1: West view of Ponte Infante

By looking at the design you cannot ignore the fact that it was inspired on the designs of a famous brilliant engineer called 'Robert Maillart'. Later on one of the designers admitted that Maillarts and Christian Menn's designs were his main inspirations. More importantly the Schwandbach Bridge (Figure 2) was the main piece of inspiration for this bridge [3], [4]. An example of the style of bridges built by Christian Menn can be found in the bottom picture of this page (Figure 3).

Figure 2: Schwandbach Bridge by Maillart

Figure 3: Viamala Bridge in Thusis (Switzerland) by Christian Menn

1.2 General structure

The Ponte Infante is what is called an arch bridge. An arch bridge generally exists out of two elements interacting together to make the crossing possible. On top a prestressed reinforced concrete box with a height of 4.50 m. beneath that slab a flexible reinforced concrete arch, with a thickness of 1.50 m. As seen in the figure below the span of the arch is 280 meters and from the feet till the crown of the arch it only rises 25 meters. This means the shallowness ratio is greater than 11/1. This all together gives the bridge it’s delicate but magnificent look (Figure 4).

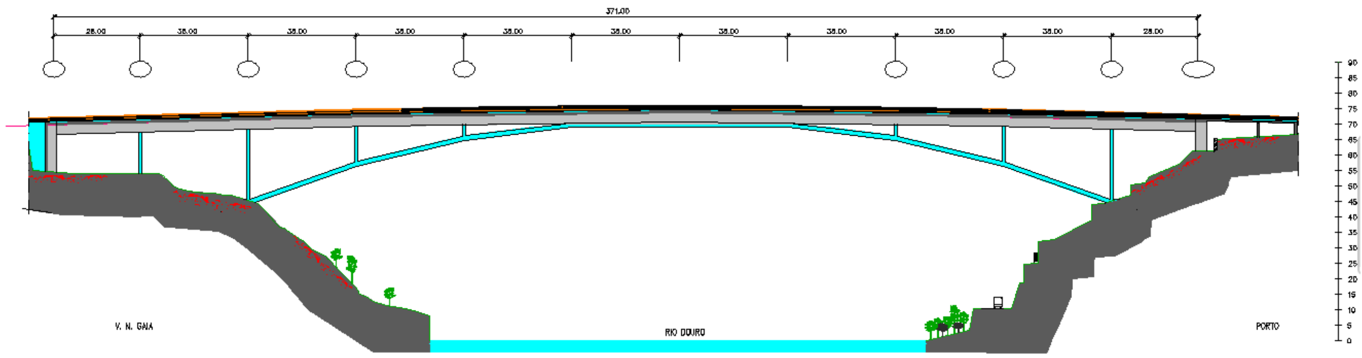

Figure 4: General AutoCAD drawing of Ponte Infante

In the centre of the bridge, the arch connects to the slab, making a box with a total height of 6 meters.

The shallowness ratio of the arch is outside the recommended range for bridges, which is between 5 and 10. By choosing for this shallow arch, the engineers didn’t make it themselves particularly easy. Above the maximum of 10, the axial stresses on the arch increase very fast, as well as the flexural action, caused by moving loads. It also leads to possible differential settlements of the foundations and thermal and rheological effects. This all just made it seem more like a challenge, seeing how far the limits of construction can be pushed.

Because of the big axial forces in the arch, which increase when moving down from the crown to abutments, some kind of measures needed to be taken. The solution was to gradually increase the width of the arch the more it goes down (Figure 5). At the crown the width is 10 m, while at the bottom it goes up to 20 m. The thickness of the arch stays the same over the whole span. This way a bigger section could take care of the bigger loads.

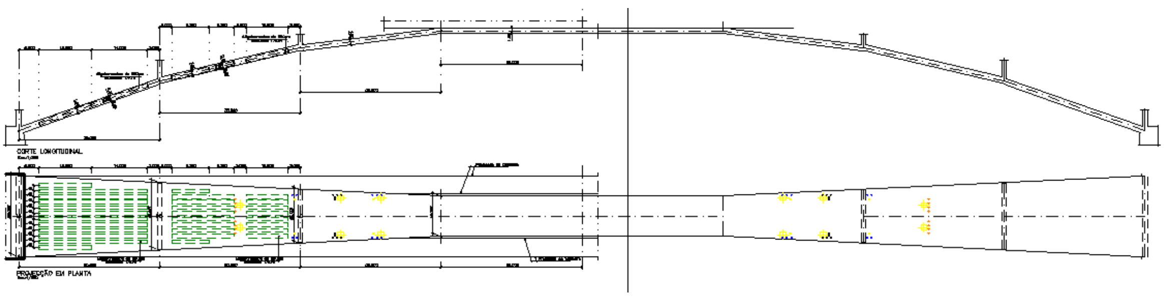

Figure 5: Drawing of side and top view of the arch

The Ponte do Infante is a unique bridge, with some amazing statistics. Some of these statistics can be found here [3], [4]:

- It is the second largest concrete arch in Europe. It is only surpassed by the Krk Bridge in Croatia, whose arch has a span of 390 m.
- It has the world record for straight segmental arches. It stands out because of the high slenderness.
- With a rise of 25 m from the bottom to the top of the arch, it cannot be compared to any other arch bridge. This is what makes the bridge truly unique.
- The static coefficient, which is in relation with the axial force found at the crown, is the largest out of any concrete arch built to this date.

As been told by the numbers, this is a record holding arch. It is the most slender arch. There can be said that it is the most loaded and "delicate" arch in the world.

1.3 Construction

The construction phase of the bridge is an interesting part to look at. Because the arch is so shallow and slender it only starts to work as an arch when connected with the deck. This leads to a building method where both the deck and arch are being built simultaneously. The bridge has a lot of characteristics from a girder bridge. That is why construction methods for girder bridges were more suitable than those used for classic arch bridges.

At first, the plan was to erect temporary piers onto which cables holding up both the deck and arch could be connected (Figure 6). This seemed illogical to first build a cable-stayed bridge which would be transformed into an arch bridge when finished.

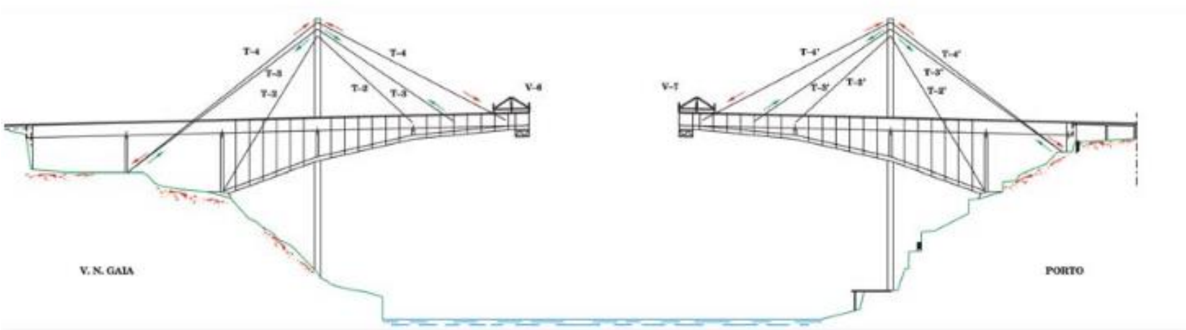

Figure 6: Original construction design

The construction method got revised and the following solution was found. There was opted to build the bridge by cantilevering the deck and the arch from each side of the river.

In order for this method to be a success two temporary pillars were built (PP1 and PP2, one on each side) to reduce the span from 280 m to 210 m. On top, trusses were created by adding tensile diagonal bars and vertical compression bars between the arch and the deck.

Outside the arch, on the slopes, trusses were also created. These trusses transferred the longitudinal tensile forces in the deck above the provisional pillars to the ground. The diagonals ensured that the two parts of the bridge are held back by the abutments until they are united in the centre.

In order to withstand the strong forces by the cantilevering elements, the connection into the ground needed to be secured. This thanks to inclined ground anchorages and by connecting the footings together by ground reinforced concrete struts (Figure 7). Complex ground studies have been carried out to ensure the rock formation would be able to withstand the loads.

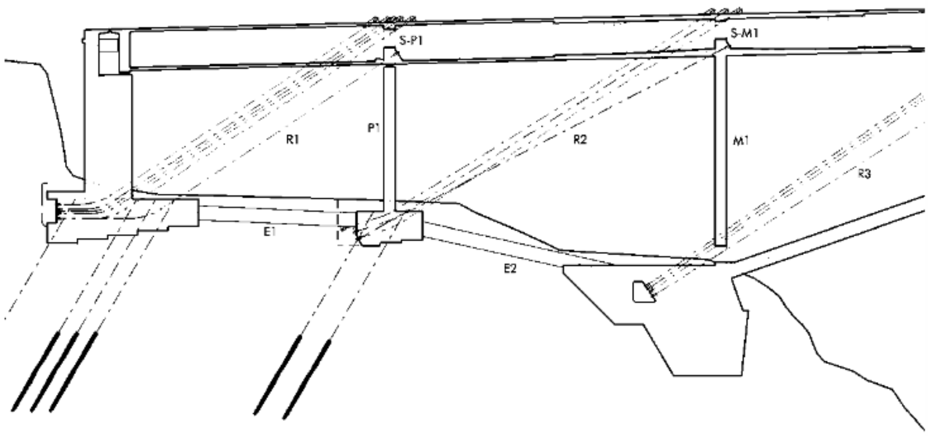

Figure 7: Ground anchorages

The deck part outside the arch was built on traditional scaffolding. Advancing formwork for the deck was only used after the columns M1 (Figure 8) and M6 (symmetrical column on the other half of the bridge) were constructed. The trusses created lead to a major advantage, as the compression in the arch is gradually introduced. This allows for better control of the creep effects.

After finishing columns M2 (Figure 8) and M5 (symmetrical one on the other side) an upwards force of 9000 kN was introduced on top of these columns by a set of hydraulic jacks. Another force was set on top of the provisional pillars by a different set of hydraulic jacks. These forces lead to a positive bending moment in the deck, reducing the negative one thanks to the cantilever method. The construction continued up until 20 more meters were built. At this place, two provisional struts MP1 and MP6 (symmetrical again on each side, Figure 8) were constructed. Corresponding diagonals D1 and D8 made truss behaviour possible. This continued up until the gap between the two half bridges was 70 meters. The central 70 meters is where the arch connects to the beam box, to achieve a beam box with a total height of 6 meters.

There is an interesting fact about the bridge. Before the central part was placed, a downwards settlement of 25mm was introduced on top of both provisional pillars (PP1 and PP2). This settlement leads to a redistribution of the internal forces. This was, of course, planned in advance and made for a better design.

The different stages of the construction phase are visualised in Figure 9.

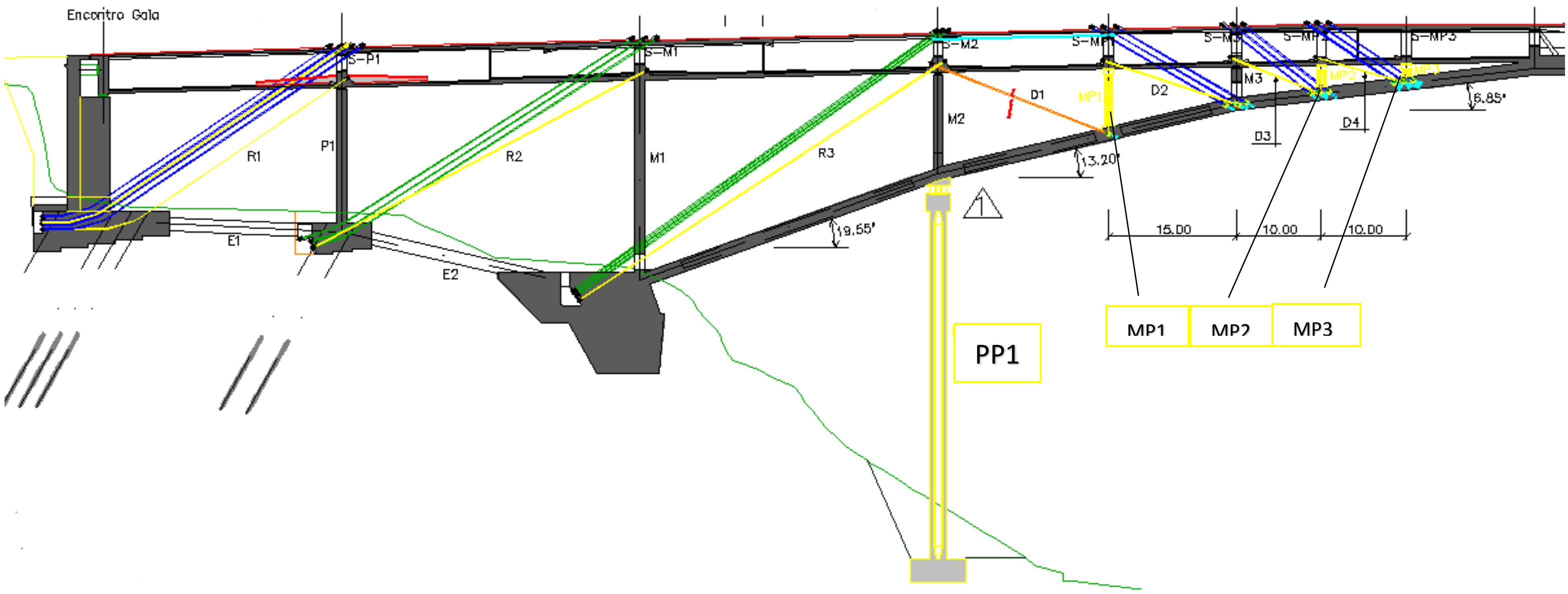

Figure 8: Coding of different structural elements

Figure 9: Construction phases

Figure 9 shows exactly how the bridge was constructed. In the first stage the cantilevering from both sides was realised. The stuts on both sides are visibile, supporting the structure since the arch is not working as long as it is not fully completed. The second stage is up until the arch meets the deck.

As long as the arch was not completed, it needed to be held back. This was done by connecting the tender arch with cables to the strong deck. The deck was strong enough in order to hold up the arch.

The final central part of the bridge was made by traditional in situ advancing of box-beams. When the connection was made of the two half bridges, the temporary strengthening elements could be removed (Figure 10). This had to be done in a very detailed sequence.

Figure 10: Finished construction before removing the temporary stability elements

A lot of the techniques used in the construction of the Ponte do Infante are seen as highly innovative.

The cantilever method used for the construction of both the deck and arch was performed by an innovative system. It is a double formwork traveller (Figure 11). This allowed for a simultaneous construction of the deck and arch. This technique became more and more difficult when the space between the arch and deck started getting smaller. Nonetheless the construction, with very tight tolerances, was a success.

Whole section (construction) references: [3], [4]

Figure 11: Double framework traveler

1.4 Material (History and use)

The main materials used for the construction of the Ponte do Infante are concrete and steel. The bridge is mainly made from prestressed concrete. The steel is only used as an addition to the concrete, to deal with tension forces. Prestressed concrete is the main type of construction material used in most of the bridges exceeding a certain span length.

Concrete has been a building material for over 2000 years. In the Roman Empire, it was widely used. Many of the large roads were constructed using concrete. After the fall of the Roman Empire, the use of concrete disappeared for a long time. It was up until 1824, when Portland cement was patented, that concrete use saw an increase again. In 1849 concrete was combined with steel to make what we now know as reinforced concrete. Ever since the invention of reinforced concrete, it has been present in the building scene.

In the second half of the nineteenth century, concrete bridges were mostly arch bridges. This because arch bridges require quite little tensile strength. The arch is mainly loaded with compression forces. They used unreinforced concrete as a replacement for stone, which used to be the main material for big monuments such as viaducts.

Due to the flexibility of the material, functional aesthetics began to be explored to add to the structural form. One of the main designers, who used to tangle elegance and structure together was Robert Maillart. One of his famous bridges is shown below, The Salginatobel Bridge (Figure 12). Just as the Ponte do Infante, this bridge also features a very steep arch. A true piece of art.

Figure 12: Salginatobel Bridge - Robert Maillart

From the 1930's prestressed concrete started to make it's rise in the market. Eugene Freyssinet showed what could be achieved with this material by building the Plougastel Bridge (Figure 13). It contained 3 arches in concrete with each a span over 188 m. This was revolutionary before it could only be done in steel. By the end of the 1960's prestressed concrete had largely taken over the place from reinforced concrete as being the main used material on bridges.

Figure 13: Pont Albert Louppe, Plougastel by Eugene Freyssinet

For the next 40 years, many bridges have been built to expand the road network. Normally it is a requisite to make the most economic bridge possible, which is why concrete became so popular. The prestressing techniques became more and more advanced, which led to longer and longer spans [6].

Bridges exceeding the maximum span for which reinforced concrete can be used, will in the first place be constructed in prestressed concrete. Only for projects where the span is way too high for concrete bridges to be possible, steel will be used. Steel or other materials can be used due to aesthetics reasons, but generally the first choice for non-moveable bridges will be concrete.

Concrete is an optimal building material for arch bridges. Concrete can endure a very high compression force. On the other hand when being loaded with tension it is not nearly 1/10th as strong. The first original arches were perfect half circles, this means there isn’t any kind of tensile force in the arch. Only pure compression. In most arch bridges these days this is not the case. Arches became more and more flat, which leads to higher and higher tensile forces. Therefore prestressing of concrete became so important. Prestressing concrete elements gives them a way to deal with high tensile forces. This meant concrete could be used in much more different situations involving tensile forces. Nowadays prestressed concrete is used in various bridges, for example The Ponte de São João in Porto (Figure 14).

Prestressing of concrete acts in the following way. There are two types of prestressing. Either pre-tensioning or post-tensioning. In pre-tensioning the steel cables are placed at certain positions inside the mold. These cables are then stretched and anchored. The concrete gets poured after. When the concrete has dried, the anchors are released and the steel tries to get back to its original shape. When the steel tries to regain its original length, it compresses the concrete. This is called the transfer of the load. Compressed concrete has a much higher tolerance to tensile forces.
Post-tensioning is the other way around. The concrete gets poured first with the steel beams inside and after the concrete has dried, the steel gets stretched. This stretching also applies a compression effect on the concrete. This way the amount of prestressing can be controlled better [5].

Figure 14: Ponte de São João - Porto

Chapter 2: Modelling

To create a 3D model of the bridge, a lot of information is needed. Not only the general structure and dimensions but also the different types of connections, the cross sections, supports, joints and much more.

Thanks to Mr. Santos and Professor Isabel Teles, the original AutoCAD files from the designer himself (Renato Bastos) could be obtained. Apart from these files, it was hard to get more information of the bridge. The missing information had to be filled in by logical thinking and visits to the bridge itself.

The Ponte do Infante is a part of a group of six bridges making the crossing of the Douro River between Porto and Gaia. It is not the most famous one (Maria Pia and Ponte Luiz I are internationally well known) and it was also the last one to be built, so not many studies have been made about this bridge. That made it hard to gain more information and even harder to gain any information in English.

Due to the lack of information, the decision was made to make a simplified model of the structure. Also, due to the very complex building process, it was decided to only calculate the bridge when being completed. The steps in between are too complex to calculate and far speculative. They also don't serve any useful information anymore about the structure today.

The first step of designing the bridge in SCIA Engineer was to draw separate parts that will be used in the global design of the bridge. The cross sections are not standardized in SCIA, so they needed to be drawn manually.

The bridge can be divided into multiple parts. The main ones being:

- The arch
- The beam box (supporting the main deck)
- The abutments
- The pillars (connecting the arch with the deck)

Two different design models were considered. In the first model, the box was designed separately and loads were applied on top of this box. The pillars/walls connecting the box to the arch were replaced by supports. The reaction force in each load was then used as the forces on top of the arch. This leads to calculating everything separately.

In the second model, the whole structure was drawn as one. This is more correct since this is the real situation. In theory we would think there is no real difference between the two models. In practice the 2nd model will have a higher stiffness leading to slightly different results. This is why there is opted to create this model. The other model can be used to check the differences.

Model data:

- Exposure class XS1
- Abrasion class XM1
- Structural class S6
- Concrete types: C30, C35, C50, C60
- Steel: A500NR, A1860/1600, A1080/1230, S235

2.1 Model setup/settings

Before the modelling can start with SCIA Engineer there are some extra options that need to be activated.

Some options are necessary and need to be filled out before the project can start.

- The corresponding Eurocode and national annex.

Since the Portuguese National Annex cannot be selected, there is opted to use the Belgian one. This will most give a small deviation from the actual calculations, but this can be neglected.

- Material types

The Material types and their according strength classes need to be specified. At least one type of material needs to be selected, but the strength classes can still be changed afterwards.

For this project concrete and reinforcement material are selected, because the general structure will be made from concrete reinforced with steel rebars.

- 2D/3D

The choice needs to be made between the following options (Figure 15). The choice was easy since the most convenient option is to choose a general 3D model.

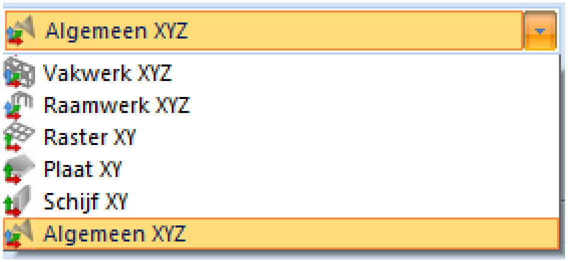

Figure 15: Project setup SCIA Engineer

The following extra options are not necessary to make the full calculation but can help to reduce the amount of work needed to achieve the same solution.

- Climate loads:

Instead of applying wind and snow loads manually on the structure, it can be handy to use the option of climate loads. When activated, the possibility opens up to let SCIA calculate those loads itself. It does this according to the given Eurocode and National Annex.

- Pre-stressing

This option allows for certain elements to be prestressed. This will be necessary because the box beam is made out of prestressed concrete.

- Bridge design

This option opens up a new tab in the software. With this tab, certain bridge combinations can be accessed.

- Mobile loads

This also opens up a new tab. This is needed in order to insert mobile traffic loads.

2.2 Beam box

The cross sections are the first part to be designed in SCIA. These were made according to the AutoCAD files from the original design. Thanks to the cross section editor in SCIA (Figure 16), the cross sections from AutoCAD could be imported and slightly altered.

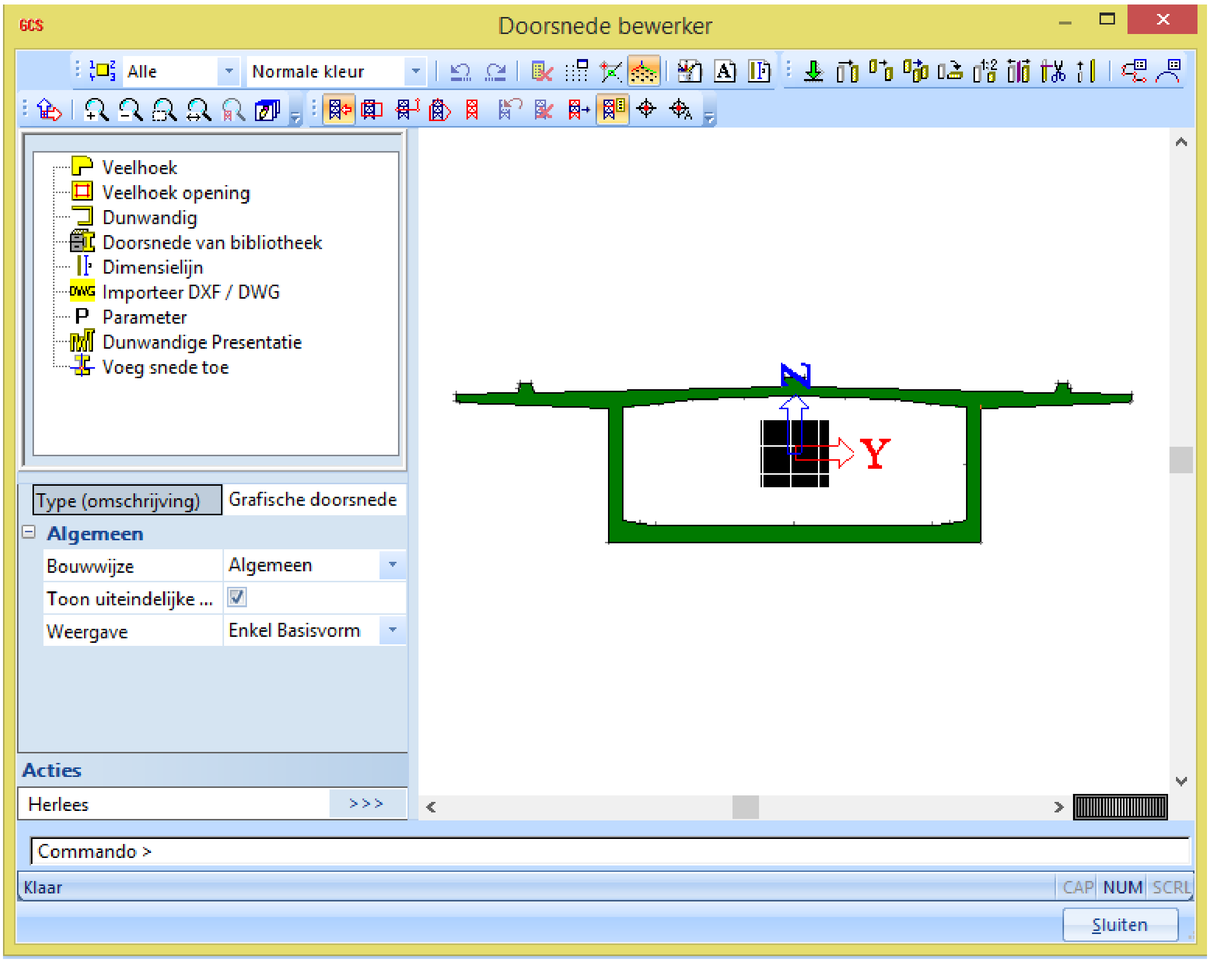

Figure 16: Cross section editor

After the cross sections were defined, the bridge elements could be created. This is when a first problem arose. Cross sections can only be defined for 1D members and columns. This means loads applied onto these members can only be placed on one central line of the cross section. Due to this, traffic lanes could not be defined onto the bridge deck. Therefore, the use of 1D members was not an option for this bridge design.

To solve this problem, 2D elements were used to design the whole structure. The beam box was the hardest part to make since so many details are present in the cross section. The rounding on the top inside of the box, the corners which are chamfered, the variation in thickness from the upper plate and much more. A combination of different 2D elements had to be used to create the whole cross section.

When designing the beam box some simplification had to be made. The picture below (Figure 17) shows what the original cross section looks like.

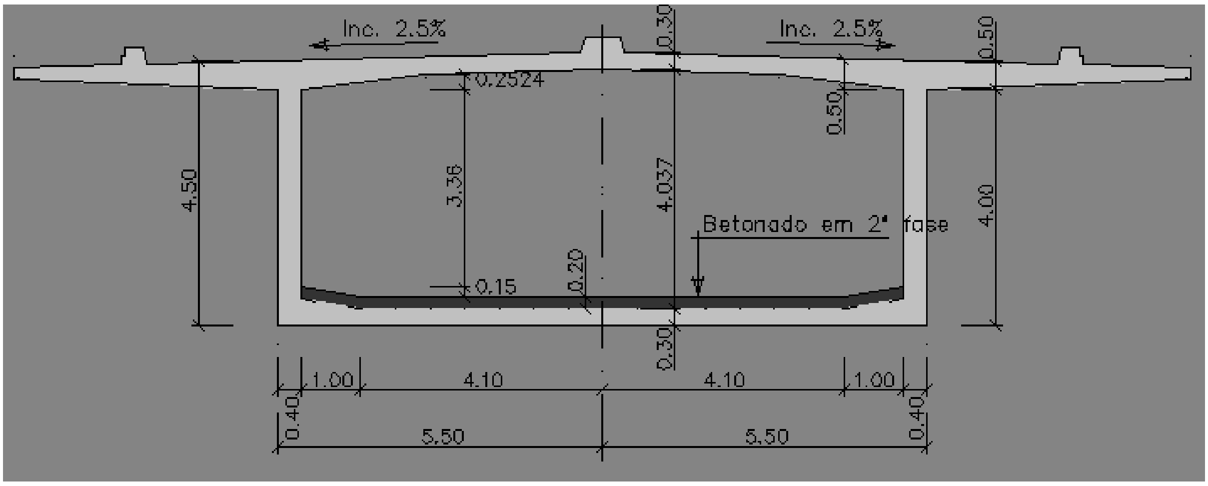

Figure 17: Original cross section (AutoCAD)

This is the cross section for most parts of the bridge. Excluding the middle part where the arch connects to the beam and the other parts where the beam is connected to the arch with the help of pillars.

To recreate this cross section, every part had to be drawn separately as a plate or a wall element. Therefore the decision was made to divide the cross section into 7 parts, as shown below (Figure 18).

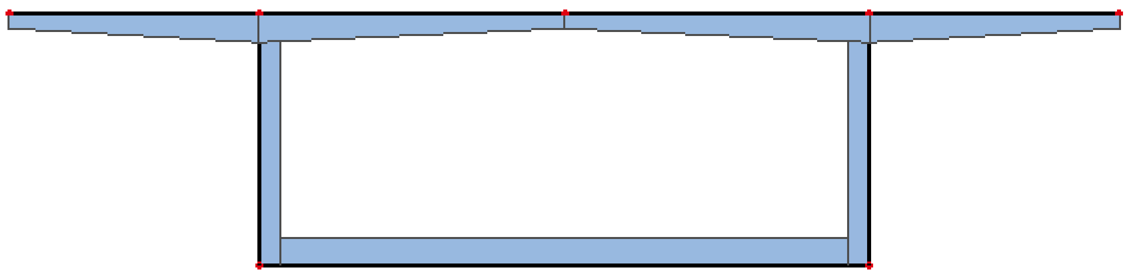

Figure 18: Simplified cross section SCIA

Because of this some simplification had to be made. The following parts were changed or removed from the cross section:

- At the bottom, the corner between the bottom plate and the walls of the box are straight 90°. In reality this corner is chamfered. This was too hard to design in the SCIA software and will not make a big difference in the end result.
- At the top we can see the deck having an inclination of 2.5%. This was made to ensure the water flowing into the drain canal. This inclination will not make a big difference regarding the load distribution on the structure. With this reasoning the inclination was not included into the design.
- The three lane separating cones on top of the deck are also left behind. The only function of these parts is to separate the traffic lanes in the opposite direction, or separate the traffic

lanes from the sidewalks. Since they don't have any structural function, these were not drawn, but rather applied as a load on the simplified cross section.

Apart from the standard cross section, some more cross sections had to be drawn. The cross section in the centre of the bridge is different from anywhere else since the arch reaches the deck at this point. This makes the need for a more full/strong cross section that can transfer the load from the beam to the arch.

This cross section was also simplified compared to reality. The differences are shown and explained below (Figure 19).

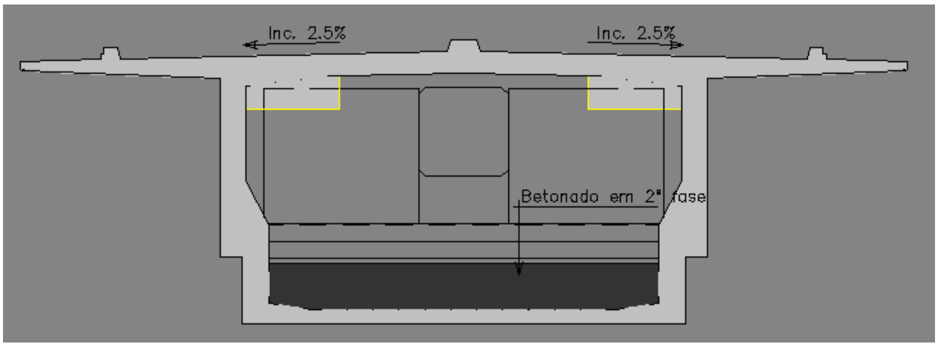

Figure 19: Original cross section (AutoCAD)

In the picture above, the cross section in the middle of the bridge is shown. This section is valid over a length of 70 meters. Here the arch blends in with the box beam to make a new strengthened box beam with a height of 6 meters.

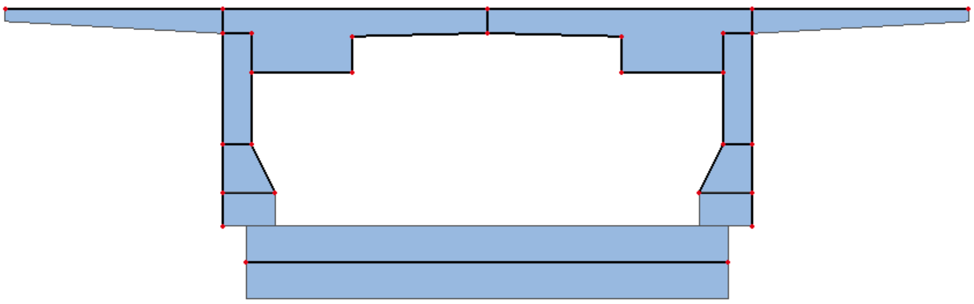

Figure 20: Simplified cross section SCIA

The cross section designed in SCIA (Figure 20) is a good approximation of the real solution. The only two simplifications that have been made are to be found at the top of the section. They are both simplifications that have also been made for the other cross section.

- The cones on top are not drawn for the same reason as mentioned in the first cross section.
- The inclination of the top deck has also not been designed. This also for the same reason as mentioned in the first cross section.

Despite the cross section looking weird because of the different parts, it will work as intended. After the whole bridge design is finished, all nodes and edges can be connected to each other to form one piece.

The last cross sections to be designed were the ones above the supporting pillars. Because of the load that will need to be transferred through these sections, they need to be more stable. This leads to nearly completely filled cross section, with small openings. The designs can be seen below (Figure 21).

Apart from the same 2 simplifications as the other cross sections, they are identical to reality.

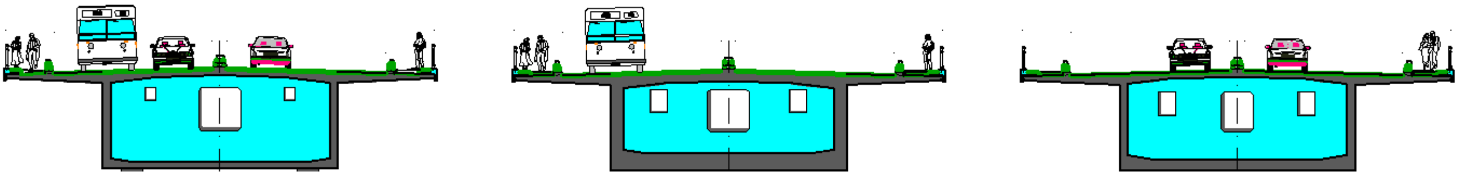

Figure 21: Original cross section above the columns (AutoCAD)

In reality, when taking a look at the longitudinal sections of the bridge (Figure 22), changes of the cross sections can be seen. In certain parts of the bridge a little bit more mass has been added. There are 2 different kinds of thickenings.

The first one is made for the attachment of the prestressed cables. For the cables to be able to be held back when they are stretched out there needs to be room for the supporting anchor. This is why there are thickenings at certain points inside the cross section (red circles).

The second thickenings are for structural reasoning. To be able to deal with bigger loads. These thickening are mainly situated 10 meters left and right of a column. This is probably to make the load transferring to the column smoother. Since these thickenings are in many different places and are not too big, it has been decided to not take them into account in the 3D model. It would take too much time to create them and no exact information is given on how exactly they look like (rounded, chamfered...).

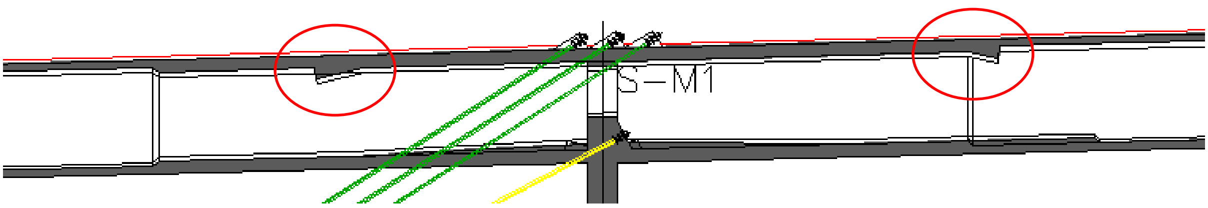

Figure 22: longitudinal section: local thickenings

2.3 The arch and columns

The design of the arch was a lot easier. The design is more straightforward and could be made pretty fast. The cross section is a rectangle in concrete with some steel beams inside for reinforcement.

There are two main factors that need to be taken into account when designing the arch:

- The arch is not an exact circle or arch, but a connection of 7 straight beams.
- The arch gets wider from top to bottom.

With the help of the top and side view in AutoCAD, the arch could be designed. Measurements were taken in AutoCAD and the arch could be drawn with the help of coordinates (Figure 23).

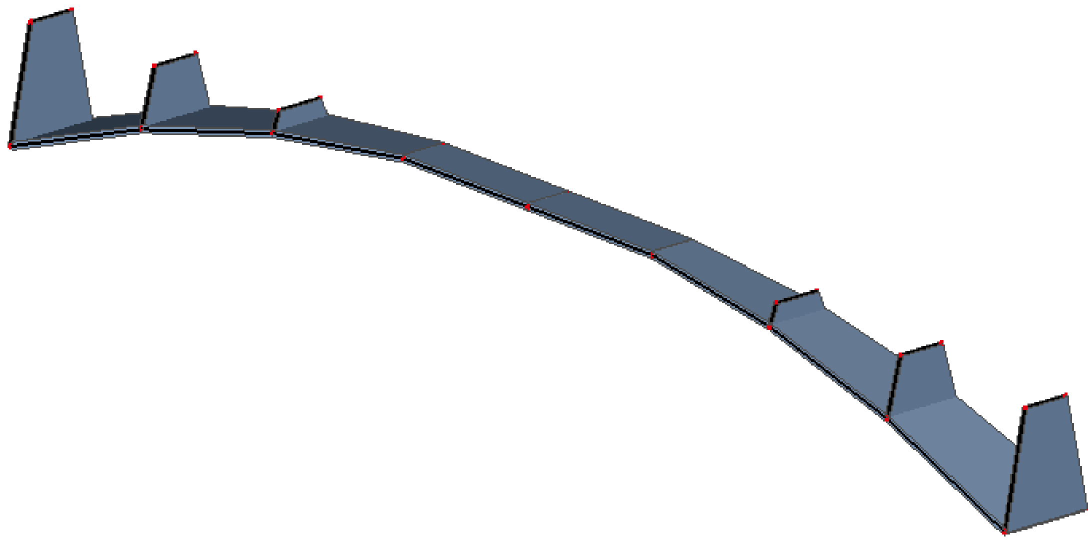

Figure 23: 3D model of the Arch and columns (SCIA)

When the arch was finished, the supporting pillars could be drawn on top. Because the arch was designed in multiple parts, the walls could easily be constructed onto the arch by selecting the line between two segments.

The only tricky part with the columns is that they get smaller from the bottom to the top. This could be made by altering the coordinates of the top points.

After finishing both parts (the arch and deck) they could be put together. The beam box was placed on top of the pillars and the bridge was almost finished. With the beam box on top, the central part of the arch connects with the beam box to make and even bigger beam box. The only thing left to do was to add one additional span on the right and two additional spans on the left. On the left they were supported by a pillar in the middle. After this was constructed the basic design of the bridge was finished.

The supports are the last part to be added. This will be discussed in the next chapter.

2.4 Supports

There are 3 kinds of supports in SCIA Engineer. The first one is a completely fixed support. This is extremely hard to realize in reality and will almost never be used. The second type is a roller support. This means the support is able to rotate and translate along the surface on which the support rests. The final one is a pinned support. This support can resist both vertical and horizontal loads, but not a moment. A displacement in any direction is not allowed, but the connected member is still allowed to rotate. These are the most common supports and it is assumed that at least one of these (pinned) is also used in this case. The supports can be designed as follows in SCIA (Figure 24).

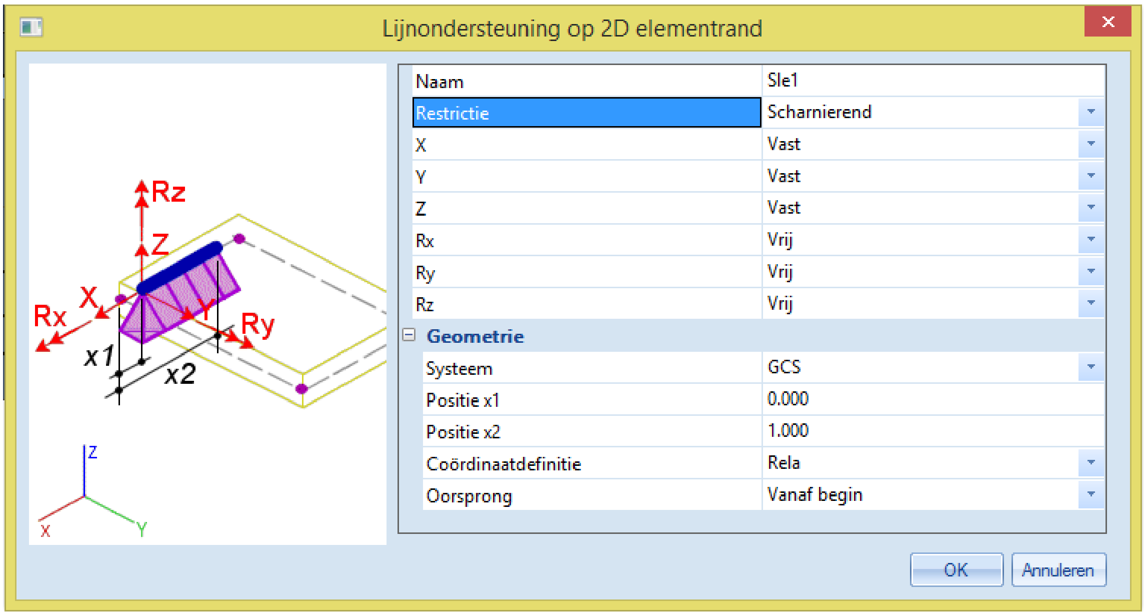

Figure 24: Support setttings in SCIA

The AutoCAD files didn't give any information on the connections with the ground. Not for pillars and not for the arch. That is why it was decided to go to the bridge to try and examine what type of supports were used. While in situ, the joints in the deck were also noted.

After the visit it was clear that pinned supports were used everywhere except for the pillars to the ground connection. Since in reality a completely fixed support is hard to implement and is almost never fully fixed. For important structures it is of course possible to make a fully fixed support. Nonetheless a pinned support will be chosen instead. These pinned supports needed to be translated into SCIA. In SCIA there is an option to choose a support along with a 2D line. This was used for both the pillars and arch. These types of supports have 6 degrees of freedom. To achieve a pinned support, there has to be decided which directions to put fixed and which ones free. In a standard pinned support displacement in any direction is not possible, but rotation is still allowed.

This lead to the "Fixed" for X,Y,Z and "Free" for Rx,Ry,Rz.

2.5 Finished 3D model

Finally the 3D model of the structure was completed (Figure 25). It took a whole lot more work than expected due to multiple problems. In the end the model (despite the simplifications) represents quite well the reality. This model will be used further to make different calculations.

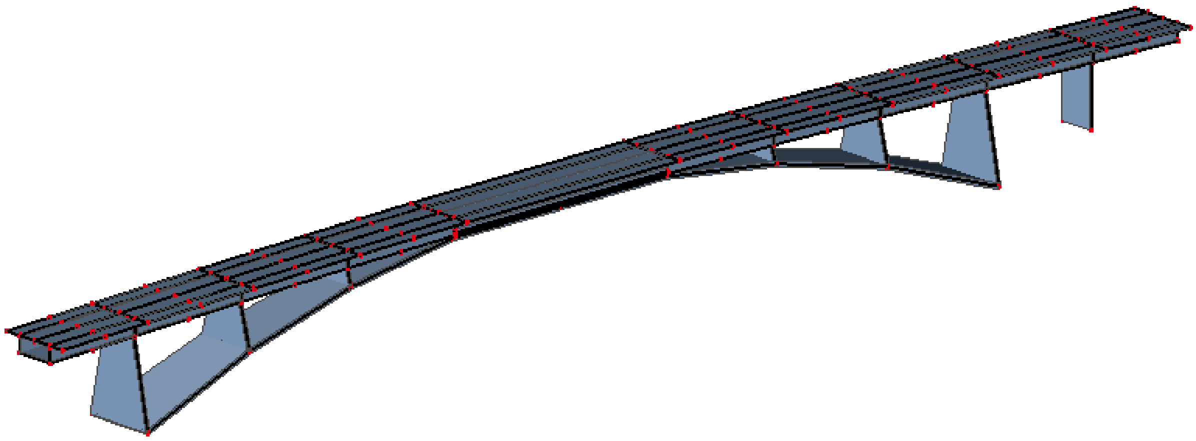

Figure 25: Full 3D model in SCIA

After performing a first load test, some of the parts of the bridge moved away from their original position without any sign of why this would happen. The defining points of the elements still said that the elements should be in their original place, but when looking at the elements itself, they had moved to the side. After contacting Mr. Pillaert, this problem could be solved. There seemed to be a connecting error in SCIA, which was luckily resolved. This might have been the biggest mystery in the project. A lot of time was spent figuring out a solution before finally finding it.

While continuing the use of the 3D model, the software gave some inexplicable problems which could sometimes not be solved. Alternative solutions had to be found. For example when designing the traffic lanes, they could not be calculated. There seemed to be a reference error, but despite searching for a solution on the web, nothing could be found. This is mentioned later in the paper under the section of traffic loads. More similar software problems arose, but for every problem, a solution was found. All the problems encountered and solved are mentioned in the paper.

Chapter 3: Load application and distribution

The following loads need to be applied in a certain combination:

Permanent loads

- Self-weight
- Covering load (asphalt, railing, road separators)

Variable loads

- Live loads (Traffic)
- Wind-load
- Snow-load
- Pedestrian loads

3.1 Permanent Loads

3.1.1 Self-weight

The self-weight of each element was generated automatically by SCIA. This could also easily be manually calculated by multiplying the volume of concrete with the specific weight of concrete. This has been calculated as a check to see if the loads that SCIA used were right. This seemed to be the case, so the calculations could continue with the loads calculated by SCIA. This is a little more accurate since the shape of the slab makes the distribution of the weight more complex than just applying it in the mass centre [8].

3.1.2 Asphalt

Asphalt has a density of approximately 2200 kg/m³. The top of the deck is covered with a thin layer asphalt of 5 cm [8]. This results in a load of:

$$22\,000 \frac{N}{m^3} x\ 0.05\ m = 1100\ N/m^2$$

3.1.3 Railing

On each side of the bridge there is a railing next to the pedestrian sidewalk. This railing is an open fence with a height of 1 m. Since no exact details are given about the railing, we assume a mean value from some railings found online. Since the railing has to withstand strong sea wind, we opted for the most suitable type [9]. We find:

$$W = 18.9 \frac{lbs}{foot} = 28 \frac{kg}{m} = 280 \frac{N}{m}$$

Since this weight is symmetrical and really low compared to the other loads, it can be neglected.

3.1.4 Sidewalks

Just as the road is covered with asphalt, the sidewalks are also covered with an extra layer. This is not asphalt, but tiles. In order to make the calculations more uniform, we decided to use the same value as for asphalt.

$$22\ 000 \frac{N}{m^3} x\ 0.05\ m = 1100\ N/m^2$$

3.2 Variable loads

Variable loads can be divided further into subcategories.

First of all the live loads from the traffic are taken into account. There is a live load due to the cars, buses, trucks … and another one due to the pedestrians.

The pedestrian loads in combination with traffic loads can be almost neglected in comparison with the load from heavy traffic. This for the two following reasons. For one the sidewalks are much smaller than the lanes for traffic, therefore fewer pedestrians can cross than cars. One pedestrian does also not weight as much as a car. A second reason can be found in the placement of the bridge. The bridge is placed outside the city centre, which means that not a lot of people choose to cross this bridge on foot. These two reasons give us a good argument to not take the pedestrian loads into account. The only situation when they will still be calculated is when the bridge is only loaded with pedestrians. This crowd loading can be the cause of for example a running event.

The live loads from the traffic can be calculated according to the Eurocode. Moving traffic does not only apply vertical loads on the structure, but also horizontal loads. These arise when the cars change speed. Either braking or accelerating. Also if the road on the bridge turns, there will be horizontal loads thanks to the centrifugal effect.

3.2.1 Climate loads

Another important category of loads are the climate loads. These are loads whose origin lies in nature effects. This category includes snow loads and wind loads. They are calculated according to the respective Eurocodes: EN 1991-1-3 and EN 1991-1-4 [10] [11].

3.2.1.1 Snow loads:

Significant snow loads and traffic loads can generally not act together. Also, the value of snow loads is typical far less important than those of traffic loads. The only situation in which snow loads certainly needs to be taken into account is during the construction phase. (Even then it rarely snows in Porto and when it does, it are extremely low quantities). The goal of this text is to only calculate the bridge after it is already built. Because of the reasons mentioned above, snow load will not be applied in the calculations of the bridge [10].

3.2.1.2 Wind loads:

References: [11], [19], [21].

In SCIA there is an option which allows to calculate the wind loads on its own. It's called a '3D wind generator'. This generator basically considers all possible wind on the structure. Because of trouble with the generator and limited calculation capacity of the program it has not been used.

Instead the calculations have been made by hand as described so in the Eurocode. This gives the same results as the 3D wind generator would.

3.2.1.2.1 The deck

Wind forces can appear in every area of the bridge. On the deck, its piers, the arch, its equipment and even the traffic vehicles. The wind blowing on the cars results in the end in a force on the bridge. This thanks to the friction the tires have with the bridge deck. This means we have to take an extra height into account when measuring the surfaces on which the wind blows. The definition of the measurements according to a certain wind direction are given in the figure below (Figure 26).

The calculations are based on EN 1991-1-4 in combination with the Portuguese national annex.

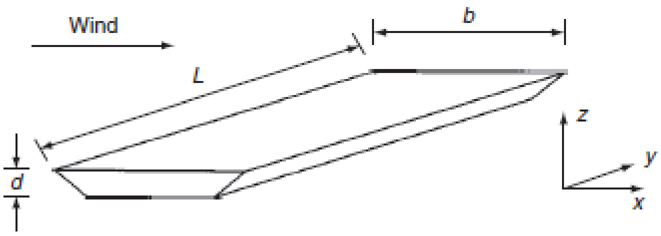

Figure 26: Directions of wind actions (Eurocode)

Only one out of the 4 directions is calculated. The two wind perpendicular parallel to the length of the bridge never give the biggest load, since there is not a lot of surface to blow on. Most of the wind just passes by the bridge instead of blowing on any surface. The most critical direction is either one of the parallel directions. Since the bridge is symmetrical around the y-axis it doesn't matter which one is applied as they give the same reaction, but in the opposite direction. Since the wind will usually be the strongest coming from the sea, the wind from the West direction is calculated.

Horizontal wind forces:

According to the Eurocode there are two procedures that can be used to calculate the wind force. A 'developed' procedure and a 'simplified' procedure. Since various coefficients are used in the developed one, without mentioning where they come from, the decision was made to use the 'simplified' method.

The characteristic value of the wind force is given by the following formula:

$$F_{Wx} = \frac{1}{2}\rho {v_b}^2 C A_{ref,x}$$

With:

- ρ: air density
- v_b : basic wind speed
- C : global wind factor
- $A_{ref,x}$: reference area in the x direction

As mentioned before, when the reference area is used in combination with traffic loads, in the case where traffic loads are the leading action an additional height d* should be taken into account. For road bridges d*= 2 m from the level of the carriageway to the most unfavourable length on the deck, independently from the position of the vertical components of the traffic load.

- **ρ**

  ρ is the air density factor. This is a constant value:

$$\rho = 1.25 \frac{kg}{m^3}$$

- **v_b**

  The basic wind value v_b is calculated using the following formula:

$$v_b = c_{dir} \; x \; c_{season} \; x \; v_{b,0} = 1 \; x \; 1 \; x \; 30 \; m/s$$

  With

  v_b: basic wind speed
  c_{dir}: wind direction factor = 1
  c_{season}: season factor = 1
  $v_{b,0}$: fundamental factor of the basic wind speed

$v_{b,0}$ depends on the zone in which the construction is situated.

- Zone A: the majority of territories, except for those who belong to zone B.
- Zone B: the Azores, Madeira and mainland regions located in a coastal zone off 5 km wide or at altitudes above 600 m.

The Ponte Infante is situated approximately 5 km from the shore. Since this is on the edge, zone B is used, which leads to a higher value for $v_{b,0}$ being: $v_{b,0}$ = 30 m/s.

- **C**

  The global wind factor can be calculated as the sum of C_e and $C_{f,x}$

  C = C_e * $C_{f,x}$ = 1,6 * 4,22 = 6.752

$C_{f,x}$

$C_{f,x}$ is in function of the width and height of the bridge. With the help of a graph and the ratio b/d,tot, the coefficient can be found.

In this case:

- b = 20 m
- d,tot = 4.50 + 2 = 6.50 m

➔ b/d,tot = 3.08 → $C_{f,x,0} = C_{f,c}$ = 1.6

C_e

$C_e = \frac{q_p(z)}{q_b}$

$q_b = 0.5 * \rho * v_b^2 = 0.5 * 1.25 * (30)^2 = 562.5$ N/m²

$q_p(z) = (1 + 7 * I_v(z)) * ½ * \rho * V_m^2(z) = (1 + 7 * 0.0987) * ½ * 1.25 * 47.4^2 = 2374.4$ N/m²

➔ C_e = 4.22

With:

$$V_m(z) = c_r(z) * c_0(z) * v_b = 1.58 * 1 * 30 = 47.4\ m/s$$

$c_0(z) = 1$ (Takes into account amount of hills, etc. generally taken equal to 1)

$$c_r(z) = k_r * ln\frac{z}{z_0} = 0.156 * ln\frac{75}{0.003} = 1.58$$

$$k_r = 0.19 * (\frac{z_0}{z_{0,II}})^{0.07} = 0.19 * (\frac{0.003}{0.05})^{0.07} = 0.156$$

$$z_{0,II} = 0.05\ m$$

$$z_0 = 0.003\ m\ (Terrain\ category\ 0)$$

$$z = 75\ m\ (Top\ height\ of\ the\ deck\ to\ be\ safe)$$

$I_v(z) = \frac{\sigma_v}{V_m(z)} = \frac{4.68}{47.4} = 0.0987$

$\sigma_v = k_r * v_b * k_I = 0.156 * 30 * 1 = 4.68$

$$k_I = 1.0$$

➔ $\frac{F_{Wx}}{A_{ref,x}} = \frac{1}{2} * \rho * v_b{}^2 * C = ½ * 1.25 * 30^2 * 6.752 = 3798$ N/m² = 3.8 kN/m²

3.2.1.2.2 The arch

The Eurocode refers to the national annex for the calculations of loads on arched bridges. Since the annex doesn't really offer a method for the calculation, the same method as for the deck is used.

Because the height difference of the arch varies more than the deck. The decision was made to separate the arch into multiple pieces. That way a more precise view of the actual loads are presented. Since the arch is already a chain of multiple beams, these could be used as the separate parts.

The lowest point of the arch has a height of 45 meters above the water. The distance from the highest point to the water is 70 meters. This means a total height difference of 25 meters is present.

If we look at the arch, 7 different parts can be seen. Because of the symmetrical form of the arch only 4 parts need to be calculated. When looking at the minimal difference in heights between the parts, there has been opted to only calculate the loads on 2 parts. The top part of the arch and in the centre (Figure 27). This results in slightly higher loads which is safer.

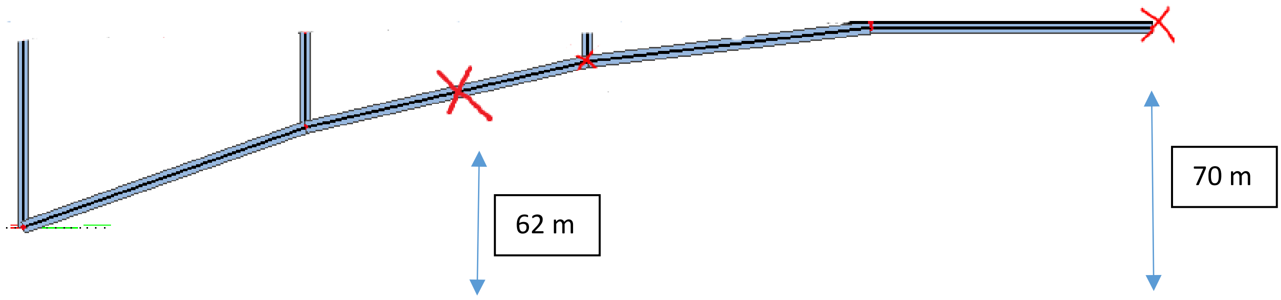

Figure 27: Separated parts of the arch

With these two heights the new calculations can be made. Most of the calculations stay the same. A summarizing table can be found at the end of the calculations (Table 2).

Top part

The characteristic value of the wind force is giving by the following formula:

$$F_{Wx} = \frac{1}{2}\rho {v_b}^2 C A_{ref,x}$$

With:

- ρ: air density
- v_b : basic wind speed
- C : global wind factor
- $A_{ref,x}$: reference area in the x direction

- **ρ**

ρ is the air density factor. This is a constant value:

$$\rho = 1.25\frac{kg}{m^3}$$

- **vb**

The basic wind value v_b is calculated using the following formula:

$$v_b = c_{dir}\ x\ c_{season}\ x\ v_{b,0} = 1\ x\ 1\ x\ 30\ m/s$$

With

v_b: basic wind speed
c_{dir}: wind direction factor = 1
c_{season}: season factor = 1
$v_{b,0}$: fundamental factor of the basic wind speed

$v_{b,0}$ depends on the zone in which the construction is situated.

- Zone A: the majority of territories, except for those who belong to zone B.
- Zone B: the Azores, Madeira and mainland regions located in a coastal zone off 5 km wide or at altitudes above 600 m.

The Ponte Infante is situated approximately 5 km from the shore. Since this is on the edge, zone B is used, which leads to a higher value for $v_{b,0}$ being: $v_{b,0}$ = 30 m/s.

- **C**

The global wind factor can be calculated as the sum of C_e and $C_{f,x.}$

C = C_e * $C_{f,x}$ = 1,3 * 4,168 = 5.418

$C_{f,x}$

$C_{f,x}$ is in function of the width and height of the bridge. With the help of a graph and the ratio b/d,tot , the coefficient can be found.

In this case:

- b = 18 m (It varies with the height, so a mean value is used).
- d,tot = 1.50 m

➔ b/d,tot = 12 → $C_{f,x,0} = C_{f,c} = 1.3$

C_e

$C_e = \frac{q_p(z)}{q_b}$

$q_b = 0.5 * \rho * v_b^2 = 0.5 * 1.25 * (30)^2 = 562.5$ N/m²

$q_p(z) = (1 + 7 * I_v(z)) * ½ * \rho * V_m^2(z) = (1 + 7 * 0.099) * ½ * 1.25 * 47.07^2 = 2344.4$ N/m²

➔ $C_e = 4.168$

With:

$$V_m(z) = c_r(z) * c_0(z) * v_b = 1.569 * 1 * 30 = 47.07\ m/s$$

$c_0(z) = 1$ (Takes into account amount of hills, etc. general taken equal to 1)

$c_r(z) = k_r * ln\frac{z}{z_0} = 0.156 * ln\frac{70}{0.003} = 1.569$

$k_r = 0.19 * (\frac{z_0}{z_{0,II}})^{0.07} = 0.19 * (\frac{0.003}{0.05})^{0.07} = 0.156$

$z_{0,II} = 0.05\ m$

$z_0 = 0.003\ m\ (Terrain\ category\ 0)$

$z\ = 70\ m\ (Top\ height\ of\ the\ deck\ to\ be\ safe)$

$I_v(z) = \frac{\sigma_v}{V_m(z)} = \frac{4.68}{47.07} = 0.099$

$\sigma_v = k_r * v_b * k_I = 0.156 * 30 * 1 = 4.68$

$k_I = 1.0$

➔ $\frac{F_{Wx}}{A_{ref,x}} = \frac{1}{2} * \rho * v_b{}^2 * C = ½ * 1.25 * 30^2 * 5.418 = 3047.6$ N/m² = 3.1 kN/m²

In the same manner the bottom part can be calculated. A summarizing table containing both values can be found here.

Table 2: Summarizing wind loads on the arch

	Z (m)	Cr(z)	$V_m(z)$	$I_v(z)$	$q_p(z)$	c_e	C	$F_w/A_{ref,x}$
Top	70	1.569	47.07	0.099	2344.4	4.168	5.418	3047.6
Bottom	62	1.55	46.05	0.101	2306.9	4.101	5.331	2998.7

3.2.1.2.3 The columns

The number of columns is rather limited in this design. This will lead to a low force but not negligible. The main reason for calculating the forces on the beams is to design the foundations. The columns here rest on the arch rather than on the ground, but nonetheless the calculations are still important. The forces are calculated as follow:

$$F_w = c_s c_d \ x \ c_f \ x \ q_p(z_e) \ \ x \ A_{ref}$$

$$\Leftrightarrow \qquad \frac{F_w}{A_{ref}} = c_s c_d \ x \ c_f \ x \ q_p(z_e)$$

The summation of the formula can be used to calculate the total force on the structure.

With:

- $c_s c_d$ = 1,15

  This is an assumption since the exact calculation of $c_s c_d$ is way too complex and not worth making in this situation.

- $q_p(z_e)$

  The height of each pillar is different. This would make a different q_p for every column. Since this would take a lot of work to calculate for each different pier a different option would be handy. The solution could be found in the Belgian national annex. In this annex the values for q_p are given in function of v_b , z and the terrain class. This leads to the following values (Table 3):

Table 3: Z_e and q_p for the columns

Pier Number	z_e [m]	$q_p(z_e)$ [N/m²]
M1 / M4	57.17	1708.4
M2 / M5	63.77	1738.6
M3 / M6	68.2	1757.4
P1	13.54	1334.3

- c_f

$$c_f = c_{f,0} \ x \ \psi_r \ x \ \psi_\lambda$$

With:

$c_{f,0}$: (Fig. 7.23 in EN 1991-1.4, not repeated here)

Since this factor is in function of b/d_{tot} it is different for every column. Also the width of every column changes from the bottom to the top. Because of this the worst case value is used as if the column had the smallest width over the whole height. This will lead to a larger force, but it is a safer assumption. Results are given in Table 4.

Table 4: b/d_{tot} and $c_{f,0}$ values for the pillars

Pier Number	b/d_{tot}	$c_{f,0}$
M1 / M4	0.44	2.4
M2 / M5	0.89	2.3
M3 / M6	2.66	1.75
P1	0.57	2.4

ψ_r = 1 (Fig. 7.24 in EN 1991-1.4, not repeated here)

ψ_λ = (Fig. 7.36 in EN 1991-1.4, not repeated here)

This coefficient is in function of 2 extra parameters. Being λ and φ.

- λ is the effective slenderness.

For l < 15m: λ = min (2 l/b, 70)

For l > 15 m: no information is given, so the same formula will be applied.

The different values for each column are given below (Table 5):

Table 5: λ and ψ_λ for the columns

Pier Number	λ	ψ_λ
M1 / M4	5.37	0.67
M2 / M5	16	0.75
M3 / M6	32.14	0.82
P1	25	0.80

- φ is the solidity ratio

$\varphi = \frac{A}{A_c} = 1$

With both parameters completed, the value for ψ_λ can be found for each pillar. This information is added to the table above.

With all the factors completed, the expression for the force on the pillars can be calculated for each pillar. A summary of the values for every pillar are given below (Table 6).

Table 6: Summarizing wind loads on the columns

Pier Number	c_f	F_w/A_{ref}
M1 / M4	1.61	3438.2
M2 / M5	1.73	3759.7
M3 / M6	1.44	3163.3
P1	1.92	3202.3

3.2.1.3 Thermal loads

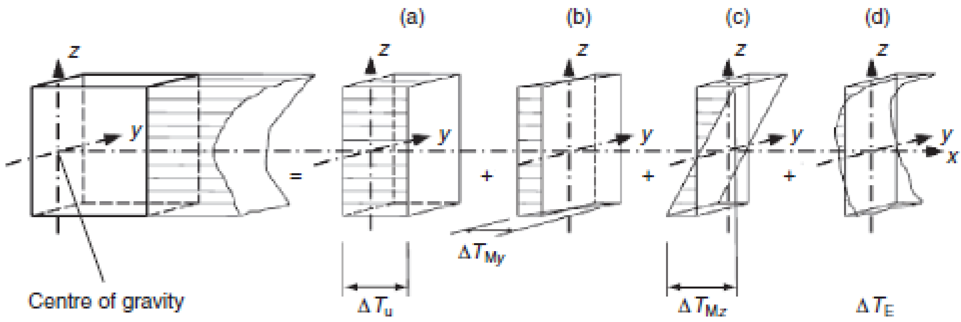

Figure 28: Different components of a temperature profile

The thermal effects in bridge decks are represented by the distribution of the temperature resulting from the sum of four terms (Figure 28) [12].

- Component of the uniform temperature (a)
- components of the temperature linearly variable according to two axes (x and y) contained in the plan of the section (b) and (c)
- A residual component (d)

In most cases only the component of the uniform temperature and the linear component in the vertical direction are taken into account. For the horizontal direction nothing needs to be taken into account since there are dilatation joints present.

Uniform component (1)

According to the thermal maps of Portugal (Figure 29), the following temperatures can be found. For both the minimum and maximum temperature Porto is situated in zone B. This gives T_{min} = 0°C and T_{max} = 40°C.

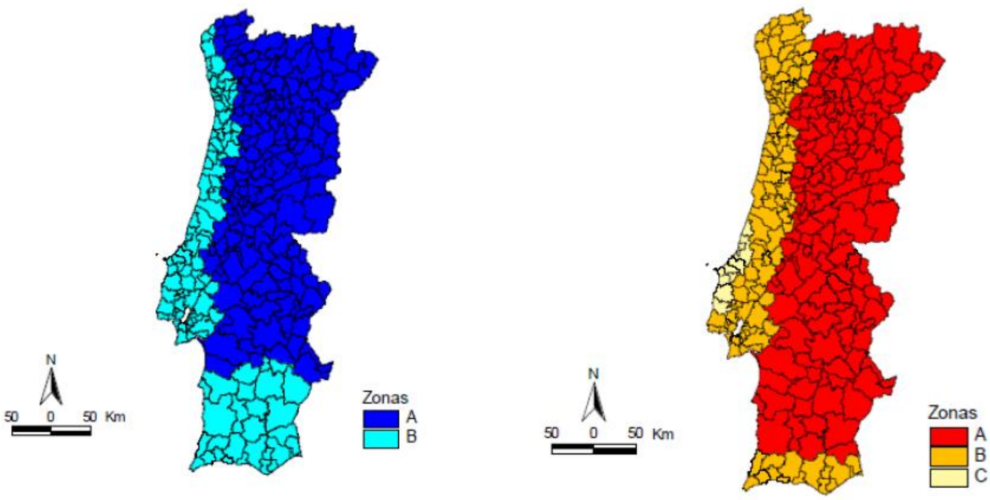

Figure 29: Minimum and maximum temperatures Portugal

The bridge type is defined according to the following table (Figure 30), taken out of the EN1991-1-5.

Type I	Steel deck	Steel box girder Steel truss or plate girder
Type 2	Composite deck	
Type 3	Concrete deck	Concrete slab Concrete beam Concrete box-girder

Figure 30: Types of bridges

These temperatures together with the bridge type lead to values for $T_{e,min}$ and $T_{e,max}$ (Figure 31).

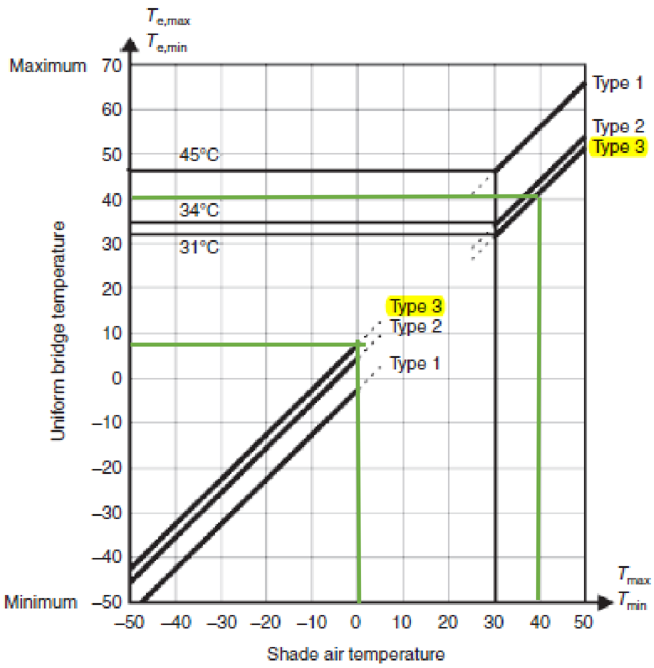

Figure 31: Uniform bridge temperature

$T_{e,min}$ = 7.5 °C

$T_{e,max}$ = 41.5 °C

$\Delta T_{N,con} = T_0 - T_{e,min}$ = 15 – 7.5 = 7.5°C
$\Delta T_{N,exp} = T_{e,max} - T_0$ = 41.5 – 15 = 26.5°C

The overall range of the uniform bridge temperature component is equal to:
$\Delta T_{N,} = T_{e,max} - T_{e,min} = 41.5 - 7.5 = 34\ °C$

Linear temperature difference components (2, 3)

These components occur when the bridge elements have a difference in temperature between the upper and bottom surface. The two surfaces will have a relative differential expansion, resulting in internal stresses. This reflects in vertical linear components and should be considered by using an equivalent linear temperature difference component $\Delta T_{M,heat}$ or $\Delta T_{M,cool.}$ These components depend on the thickness of the relevant surface deck. k_{sur} is a correction factor for different thicknesses.
Due to the top being exposed to the sun. The top will be warmer than the bottom side of the box girder [12].

For type 3 bridge, concrete box girder this leads to:

- $\Delta T_{M,heat}$ = 10°C.
- $\Delta T_{M,cool}$ = -10°C.

This value is based on a depth of surfacing of 50mm. For other depths of surfacing a correction factor k_{sur} is applicable. Since the surfacing in this case is 50 mm, the standard values can be applied without the need of a correction.

In reality both components (uniform and difference components) can act together. In that case they cannot be both represented by their characteristic value. EN 1991-1-5 recommends two expressions in that case. These can be called 'sub-combinations'.

$$\Delta T_{M,heat}(or\ \Delta T_{M,cool}) + \omega_N \Delta T_{N,exp}(or\ \Delta T_{N,con})$$

Or

$$\omega_M \Delta T_{M,heat}(or\ \Delta T_{M,cool}) + \Delta T_{N,exp}(or\ \Delta T_{N,con})$$

With:
- $\omega_N = 0.35$
- $\omega_M = 0.75$

➔ $10 + 0.35 * 26.5 = 19.275$°C
➔ $0.75 * 10 + 26.5 = 34$°C

The loads are imported into SCIA as temperature differences. SCIA itself has a tool to convert these differences into some kind of load onto the bridge (Figure 32).
The temperature differences need to be split up into multiple sections since the cross section consists of separate 2D-elements. There will be assumed that the difference takes places between the top deck and the inside of the beam box.

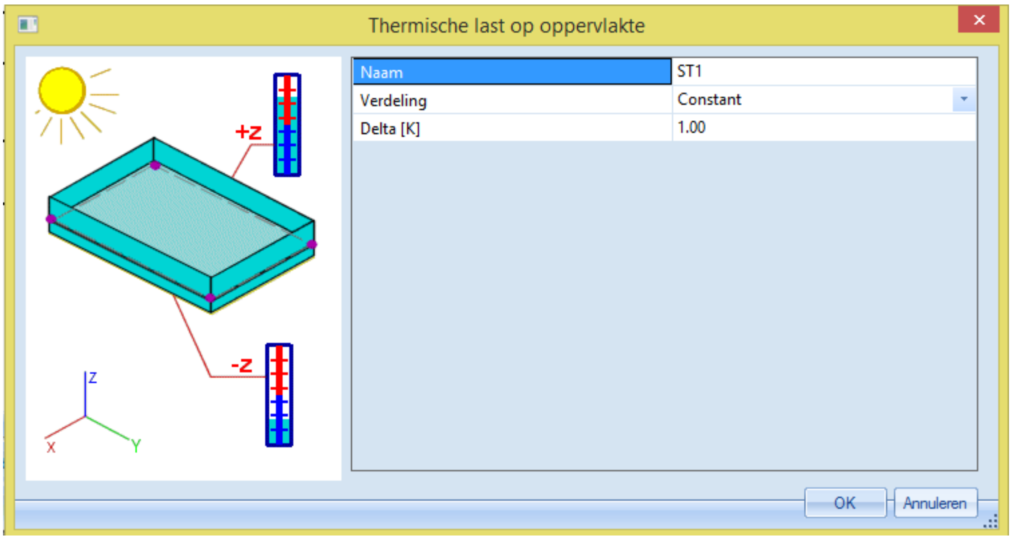

Figure 32: Thermal loads tool in SCIA

A lot of bridges struggle with these thermal loads. For a long bridge structure these temperature differences lead to high loads at the end of the bridges if these are being restrained. This is why in most cases there are two types of solutions for this. One solution is to place a sliding support on one of the endings of the bridge, so the bridge is free to move in the x-direction. The other solution, which has been applied here, is to have dilatation joints on both sides of the bridges where the bridge structure connects to the mainland. These dilatation joints work similarly to a sliding support. They allow for the bridge to move horizontally. One of those joints can be seen in the picture bellow (Figure 33).

Figure 33: Expansion joint

3.2.2 Traffic loads

There are four different kinds of load models which can be used to calculate the forces applied from traffic according to EN 1991-2.

- The main load model (LM1), including concentrated loads (tandem systems, TS) and uniformly distributed loads (UDL) and applicable to all bridges.
- The model consisting of a single axle with two wheels (LM2), in addition to the previous one (LM1) for the verification of short structural members (3-7m).
- The model made up by a set of special vehicles intended to take into account the effects of exceptional convoys (LM3).
- The model corresponding to the loading of the surface of the bridge with a uniformly distributed load of 5 kN/m², corresponding to the effects (dynamic amplification included) of a crowd (LM4).

Load model 2 is for the verification of short structural members (3-7m). It is not needed to apply this model since such short members don't appear in this design. LM3 and LM4 are according to the norm only required when explicitly asked by the client or for specified personal projects. This leads to LM3 also not being calculated. LM4 will still be considered since this is a situation that can appear during the lifespan of the bridge.

In SCIA there is an option to draw traffic lines which will automatically place the loads on the worst case position when calculations are performed. Unfortunately, there was a problem with this tool. SCIA created so many load cases that the calculations took extremely long to complete and in the end the software crashed quite often. Therefore another solution had to be found.

SCIA Engineer has another built-in tool made for applying mobile loads on structures. This tool creates traffic lines on which different load types can be applied. The lines could be drawn according to the notional lanes calculated in LM1. The first step in the process is determining the critical places on which a loading should be applied. These can be found with the help of influence lines. When creating a traffic line a unit load is applied. Moving the unit load along the line gives a different contribution to the maximum effect. The influence of each position combined gives us the influence line of a loading. Yet again this solution didn't seem to be working. When calculations were finished, no influence lines would appear. There seemed to be an internal problem with the software. When calculating the project including the traffic lines, the size of the project skyrocketed. Apart from the increase in size, the time needed to perform the calculations also drastically went up. When SCIA finished the calculations, the influence lines could not be visualized. When selecting a 2D element, an empty screen appeared. Not even axis were shown. Multiple solutions were tried in order to fix this, but not one seemed to work. Yet again, another solution had to be found leading to a different method being adopted.

A simple 2D model (Figure 34) was created. In this model, one traffic lane would be drawn vertically, which gives the influence lines for the shear force and moment in the z-direction of the 3D model. Another one would be drawn 'horizontally' along the x-axis of the deck elements. The drawing of the model can be found on the next page (Figure 34).

References: [14], [15], [16].

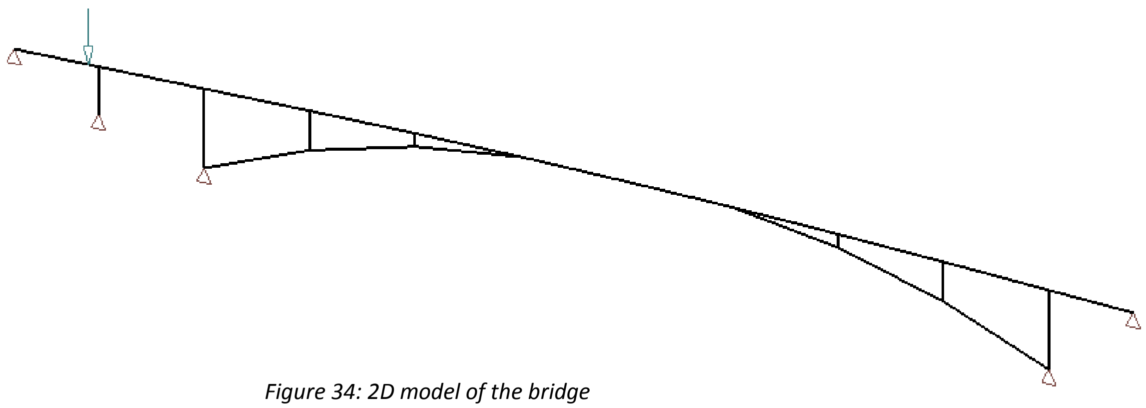

Figure 34: 2D model of the bridge

The influence lines give the positions where the loads need to be placed in order to obtain the maximum value in a certain section. First these 'critical' sections need to be determined. There are multiple 'critical sections' which will be studied. The first one is the centre section in both the arch/the deck, as most likely the highest loads will be found there. The second section is just before the arch meets the deck. Here the loads will most likely be a little bit lower, but the section that needs to deal with the loads is also smaller. The third section is close to the bottom of the arch, since the tensile forces in the arch get stronger the more you go down. For the pillars, the one closest to the end of the Arch (M1/M6) will be calculated as the most loaded column. This is the tallest column, which leads to the highest risk of buckling. The single pier not standing on the arch will also be checked since this pier might also have a high risk of buckling. Buckling is the most dangerous situations for pillars. Buckling might occur with an applying force much lower than the material can bear. The pillar won't fail due to the collapse of the material, but due to other sources.

The influence lines for the maximum shear force and maximum bending moment in the middle section of the bridge can be found on the next page (Figure 35 & Figure 36). The other influence lines are given in Annex I.

The influence lines for the forces in the columns gave some confusing results at first glance. The horizontal force would also allow for a bigger maximum force in the column. This seemed illogical since the horizontal force is working at an angle of 90° compared to the N-forces. Of course the deck is not exactly horizontal, but also a bit arch shaped, which explained why a horizontal force (along the length axis of the deck) gave a bigger compression force in the columns.

These influence lines drawn on the 2D model in SCIA unfortunately don't give any information on the bending moments around the y-axis (axis parallel to the length of the bridge) from the 3D model. Loads not applied centric onto the bridge create bending moment in the transversal direction. The calculations for this have not been done, but should still be considered in reality. In the morning/evening different parts of the bridge are loaded.

In general it can still be concluded that the influence lines from the 2D model will be a quite good representation of the reality.

The resulting influence line for the shear force (Figure 35) and maximum bending moment (Figure 36) in the centre of the box beam are given below.

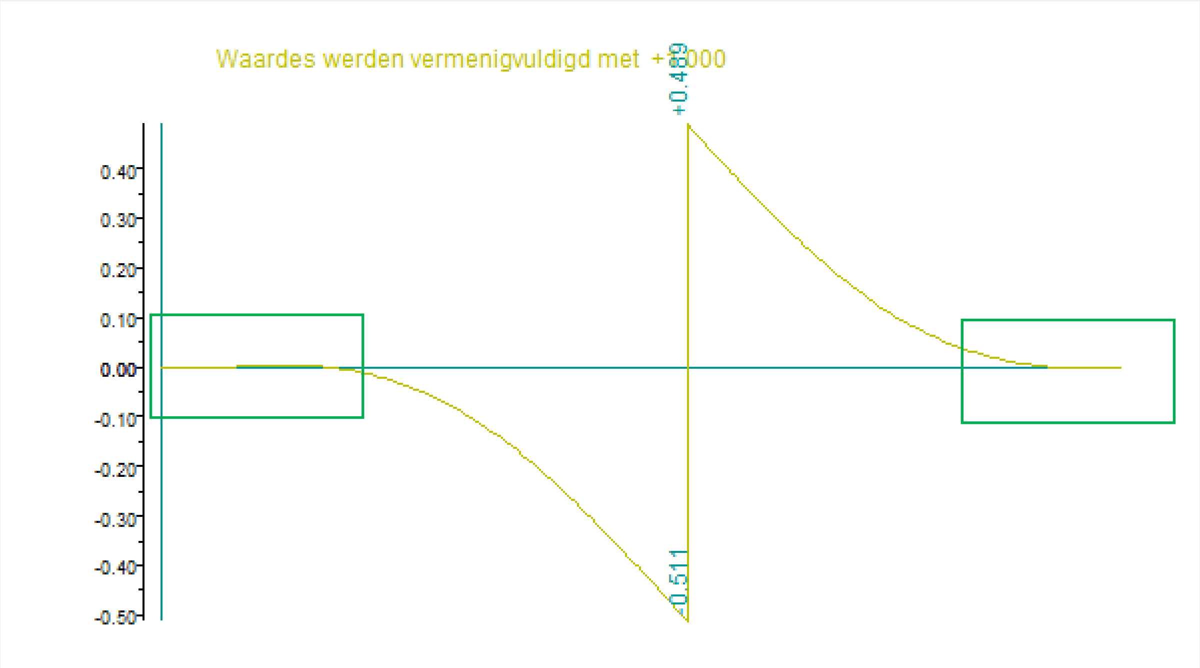

Figure 35: Influence line for Vz in the middle section

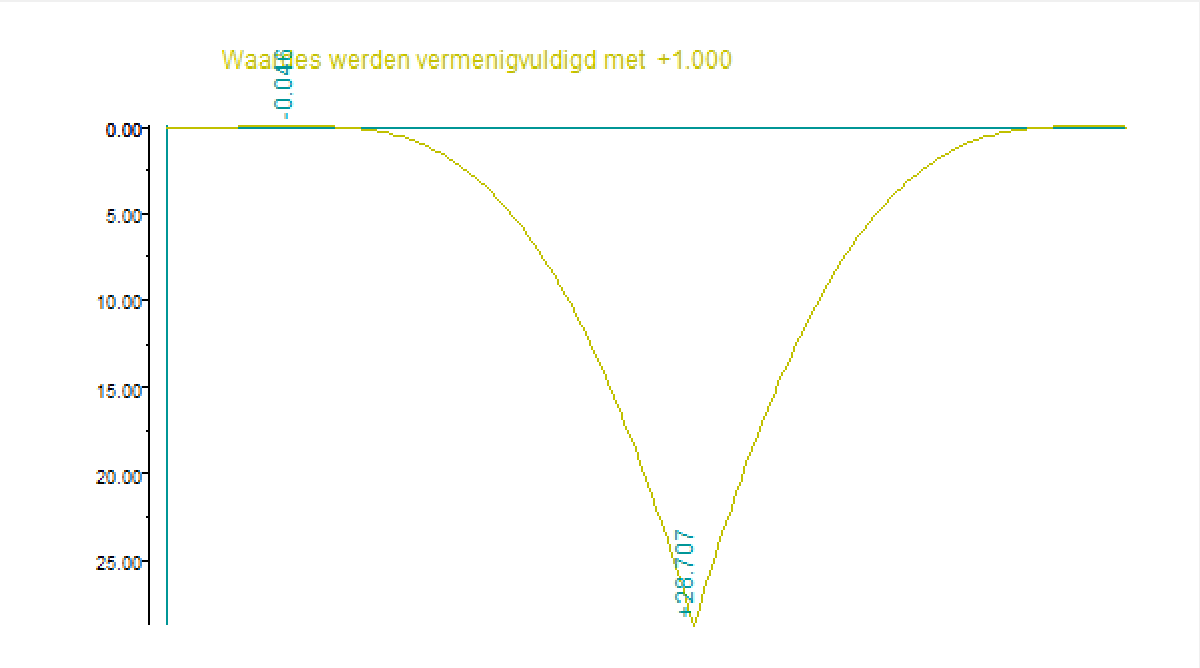

Figure 36: Influence line for Mx in the middle section

Both lines have multiple intersections with the zero axis. This means different zones for positive/negative loading exists. When looking at Figure 35, there can be seen that the outer left and right parts barely crosses the zero axis (inside the green rectangles). These parts will not be taken into consideration. There will be assumed that the line for the shear force only switches sign one time (in the centre). The line for the bending moment will assumed to be positive over the whole structure.

The vertical influence line for the shear force switches sign for x = 203.4 m. In order to achieve the maximum negative shear force, the part of the bridge from 0 m – 203 m will be loaded. In order to create the maximum positive shear force, the other part: 203 m – 371 m, will be loaded.

With these influence lines, the positions for the maximum positive and negative moment can be found. In order to create the maximum positive moment in the centre of the bridge, all sections on which the influence line is positive need to be loaded. The same can be done to get the most negative moment. In order to obtain the most positive/negative lateral force, the same method can be used. The influence line for maximum moment is mostly positive over its trajectory. No loading appears to be working in the positive z-direction (upwards), so the negative moment won't be considered.

The graphs are not exact enough to see on what positions the line switches sign. Tables have been created to give a more clear view. These tables can be found in the SCIA document. Sometimes the tables show that loads need to be applied, for example from 0 – 240 meters, in the model there is a plate drawn with a length of 35 m, which goes from 206 m to 241 m. Since the part from 240-241m is only very slightly negative, the loads will be drawn up until 241 m. This will lead to a slightly lower maximum value, but is almost negligible since the contribution from the part of 1 meter to a negative value is extremely low.

The same can be done to find the maximum compression force inside the columns M1 and P1. The influence lines for this can be found in Annex I.

Some more simplifications have been made to applying loads according to the influence lines. The UDL-loads normally need to be applied onto the respectively notional lanes. Since the lanes were not drawn in SCIA, because of software trouble, another way had to be found to properly apply these loads. The deck has been drawn in 4 parts of approximately 5 meters. The notional lanes are only 3 meters wide. Instead of only loading the first 3 meters of the central panels with a load of 9 kN/m² and the last 2 meters with a load of 2.5 kN/m², the whole plate was loaded with 9 kN/m². This is not fully correct according to the Eurocode, but is safer since the biggest loads are applied onto a bigger surface instead of the smaller load. This can be seen in the Figure below (Figure 37).

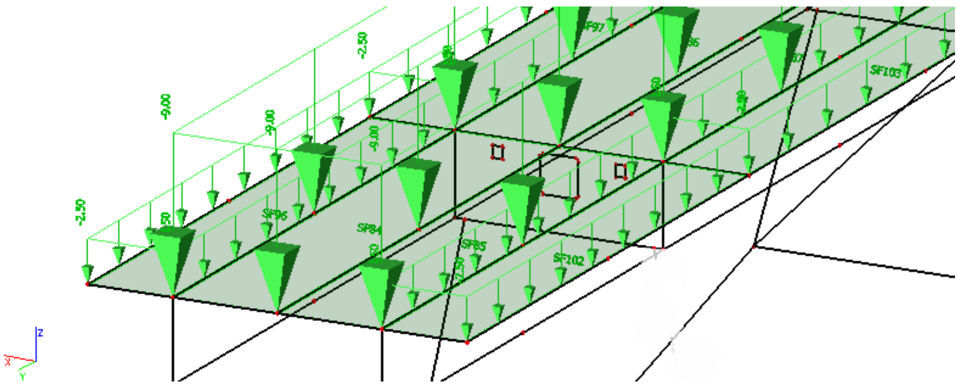

Figure 37: UDL loads on the top deck in SCIA

3.2.2.1 The main load model (LM1):

The first load model that is applicable to every bridge is the main one. This model is split up in two different load systems. The tandem systems (TS) and the uniformly distributed loads (UDL). The calculations are made according to NBN EN 1991-2: 2004, paragraph 4.3.2.

First the carriageway needs to be divided into notional lanes. In this case the width on each side of the central separation cone is 7.37 meters (Figure 38).

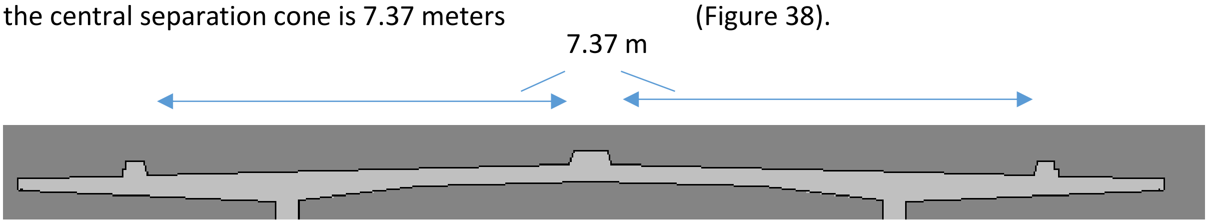

Figure 38: Widths of notional lanes

- $w \geq 6\,m \rightarrow \#lanes = n_1 = Int(\frac{w}{3})$ = 2
- Width of a lane: 3m
- Width of remaining area: $w - 3\;x\;n_1$ = 1.37 m

The numbering of the lanes is not fixed. It depends on the effect of each lane. Lane number one is the lane who gives the most unfavourable effect. Lane 2, the second most unfavourable and so on.

Vertical loads:

<u>Uniformly distributed loads (UDL):</u>

These loads are applied everywhere on the unfavourable positions (leading to a higher internal force, bending moment, etc.).

$Lanes$: $\quad F_{UDL} = \alpha_{q,i} * q_{\,ik}$

$remaining\ area$: $\quad F_{UDL} = \alpha_{q,r} * q_{\,rk}$

<u>Tandem system, two axles:</u>

The tandem systems travel centrally along the axes of the relevant lanes. A maximum of three notional lanes are loaded with a single tandem system per lane. This is optional, which means that according to the national annex there can be applied only one or two. The placing of the tandem system is fixed. Therefore it will be placed in the worst case scenario, according to the influence line.

$Per\ axle$: $\quad F_{TK} = \alpha_{Q,i} * Q_{\,ik}$
$Per\ wheel$: $\quad F_{TK} = \alpha_{Q,i} * Q_{\,ik}/2$

The values of q_k and Q_k are given in the figure below taken from EN 1991-2 (Figure 39).

Location	Tandem system (TS) Axle loads, Q_{ik} (kN)	UDL system q_{ik} (or q_{rk}) (kN/m^2)
Lane No. 1	300	9
Lane No. 2	200	2.5
Lane No. 3	100	2.5
Other lanes	0	2.5
Remaining area (q_{rk})	0	2.5

Figure 39: Values for TS and UDL

The different α values can be found in the following table. Because no specifics are given, it was chosen to use the values for 'Normal' traffic, mentioned as 'Gewoon' in the figure bellow (Figure 40). This figure could sadly not be found in English.

Aard van het zwaar vervoer	α_{Qi} (rijstrook 1, 2, 3)	α_{q1} (rijstrook 1)	α_{qr} en α_{qi} (rijstrook 2 en 3)	β_Q
Gewoon (1)	1,0	1,0	1,0	1,0
Met beperkte dynamische vergrotingsfactor (2)	0,7	0,8	1,0	0,6
Met lage frequentie van het zwaar verkeer (3)	0,8	0,8	1,0	0,8
Met beperkte dynamische vergrotingsfactor en lage frequentie van het zwaar verkeer (4)	0,56	0,8	1,0	0,48

Figure 40: α-values according to the type of traffic

This leads to the following values for the vertical forces:

$Lanes$:

$$F_{UDL,1} = \alpha_{q,i} * q_{ik} = 1 * 9 = 9\ kN/m^2$$
$$F_{UDL,2} = \alpha_{q,i} * q_{ik} = 1 * 2{,}5 = 2{,}5\ kN/m^2$$

$remaining\ area$:

$$F_{UDL} = \alpha_{q,r} * q_{rk} = 1 * 2{,}5 = 2{,}5\ kN/m^2$$

$Per\ axle$:

$$F_{TK,1} = \alpha_{Q,i} * Q_{ik} = 1 * 300 = 300\ kN$$
$$F_{TK,2} = \alpha_{Q,i} * Q_{ik} = 1 * 200 = 200\ kN$$

$Per\ wheel$:

$$F_{TK,1} = \alpha_{Q,i} * Q_{ik}/2 = 150\ kN$$
$$F_{TK,2} = \alpha_{Q,i} * Q_{ik}/2 = 100\ kN$$

These values are the same for the two lanes on the other side (symmetrical around the length axis) of the bridge.

Horizontal loads:

Horizontal forces in the y-direction can work in two ways. Either in the positive or negative y-direction. One thanks to cars accelerating, the other one thanks to cars braking. Both forces can be calculated with the next formula:

$$Q_{1k} = \ 0.6 * \ \alpha_{Q1} * 2(Q_{1k}) + \ 0.10 * \alpha_{q1} * q_{1k} * w_1 * L$$
$$180\alpha_{Q1}\ (kN) \leq \ Q_{1k} \leq 900\ (kN)$$

In the Belgian national annex the maximum value of 900 kN has been lowered to 720 kN. In the Portuguese annex no reduction can be found. Therefore there is opted to not lower the value.

With:

- L = the length of the deck or the part under consideration. Since the whole deck is under consideration, the total length will be used: L = 371 m
- Q_{1k} = Weight of two axles of tandem system applied to lane No. 1 : Q_{1k} = 300 kN
- q_{1k} = density of the uniform distributed load on lane No.1 : q_{1k} = 9 kN/m²
- w_1 = width of notional lane No.1 : w_1 = 3 m
- α_{Q1} = adjustment factor, depending on the loading class. α_{Q1} = 1

➔ $Q_{lk} = \ 0.6 * \ \alpha_{Q1} * 2(Q_{1k}) + \ 0.10 * \alpha_{q1} * q_{1k} * w_1 * L = \ 0.6 * 1 * 2(300) + \ 0.10 * 1 * 9 * 3 * 371$

➔ $Q_{lk} = 1361.7$ kN

➔ $Q_{1k} = \ 900\ kN$

This horizontal point force will be distributed over the whole surface of the bridge. This gives the following value:

$$Q_{1k,V} = \frac{900}{(371 * 20)} = 0.121\ kN/m^2$$

3.2.2.2 Crowd loading model (LM4):

This model represents the bridge being loaded by a large crowd. For example when sports events such as a marathon (Figure 41) take place and a big crowd is running over the bridge at once. A parade is another example. Not only is the crowd situated on the sidewalks, but also on the lanes where normally cars would travel.
This type of loading results in a uniformly distributed load of 5 kN/m².

$$q_{cr} = \ 5\ kN/m^2$$

Figure 41: Crowd loading on bridge (Marathon of NY)

In the Eurocode some more situations have been specified to be taken into account. The reasoning why these situations will not be calculated are given below.

- A vehicle hitting the pillars of the bridge can lead to a large force to which the columns need to be able to resist. Since the columns of this bridge are all located on top of the arch or places where vehicles cannot reach, it is not necessary to take this into account.

- Vehicle on footways. This needs to be considered for all bridges where footways are not protected by a rigid road restraint system. In this case the footways are protected with a rigid road restraint system.

- Collision forces on kerbs. No Kerbs are present in the design of the bridge.

- Collision forces on vehicle restraint systems. These forces need to be known to design the connection of the restraint systems to the structural members of the bridge. This falls outside the scope of this document.

- Collison forces on structural members. There are no structural members with which a collision can occur.

- Horizontal load according to the x-axis (perpendicular to the length). The bridge travels in a straight line. This means the cars don't need to drive in a curve on the bridge, leading to a force in the x-direction. The only force in the x-direction would come from cars switching lanes. This force can be neglected.

Chapter 4: Combinations

Once all loads are defined they need to be applied in certain combinations. The Eurocode features lots of different kind of combinations for different situations, each combination with different safety factors.

Some simplified rules may be used for the combinations. These rules generally apply for all bridges, except if explicitly told not to use them. For this reason, the rules are applied on this bridge design. These simplifications mainly consist of limiting the number of variable actions to be taken into account. The rules are:

- Snow loads are never combined with any group of traffic loads.

  This can be understood if we think about the fact that traffic moving over the road clears away the snow. This means that in general they cannot act together.

- Wind and thermal loads are not taken into account simultaneously with any group of traffic loads.

  This rule will not be accepted as wind is a main factor acting on a bridge. When traffic is the main load, wind will be added. Wind loads even become bigger when traffic is active.

- Wind actions only need to be taken into account simultaneously with load group gr1a.

  Group gr1a is the group of the UDL and TS. This will be the main loading combination, so wind will be taken into account in this group.

- No variable non-traffic action is taken into account simultaneously with load group gr1b.

  Group gr1b will not be calculated so this statement has no impact on the project.

- The combination of non-traffic actions with load group 5 (special vehicles) is to be decided at national level.

  Only LM1 and LM4 are calculated which means this doesn't matter in this specific case.

There are two big groups of combinations which need to be verified. The first one is the Ultimate Limit State (ULS). The second one is the Serviceability Limit State (SLS).

The ULS (Ultimate Limit State) is used to check if the bridge will not endure severe damage leading to it no longer being safe. This is tested with the worst combination of loads possible, including great safety factors. The bridge might show crack forming or uncomfortable vibrations, but as long as the bridge itself or certain parts don't collapse, the test is passed.

The SLS (Service limit state) is checked after the ULS is completed and passed. This check can be summarized as a comfortable use check. The amount of cracks, the deformation and vibrations are limited in order to make the bridge feel comfortable. When this test fails but the ULS passes the bridge won't collapse, but won't feel safe either. This is why this test is also very important. In general the problems of SLS are not solved by a good standard, but by a good design.

When creating a combination with SCIA, the program will automatically take the right safety factors for each load implemented in the combination. There will still be checked if this matches the rules from the Eurocode.

4.1. Ultimate Limit State (ULS)

There are 3 types of check-combinations which need to be performed in the ultimate limit state. These are:

- EQU
- STR
- GEO

The structural limit state is the first and only one of the ultimate limit states to be checked. EQU is related to the balance of the structure. For bridges this is mainly a problem during construction, but almost non-existent when the bridge is finished. The GEO state doesn't need to be checked either. This is for designing the abutments and foundation. This is outside the scope of this document.

The following needs to be ensured in the STR ultimate state:

$$E_d \leq R_d$$

The checks are performed in the most critical sections, which are determined by logical thinking.

With

- E_d

This is the design value of the effect of actions (internal force, moment, etc.) in a certain section (N_{Ed}, V_{Ed} or M_{Ed}). These are the results of the loads on the construction ($\sum F_d$).

F_d is the design value of the loads. This can be found by multiplying the characteristic values with a safety factor γ_f.

- R_d

The design value of the corresponding resistance in a certain section (N_{Rd}, V_{Rd} or M_{Rd}).

= $R(\frac{f_{ck}}{\gamma_c}, \frac{f_{yk}}{\gamma_s})$

Since f_{ck} and f_{yk} are dependent on the strength class of the material, they are different for each element. (E.g. The arch has another concrete strength class than the deck, C50 <-> C60).

There are 3 different formulas to calculate the design value of actions. Either Eq. 6.10, Eq. 6.10a or Eq. 6.10b (Taken from EN 1990). According to the Belgian National Annex, Eq. 6.10b should be used. According to the document 'Designers Guide to Eurocode 1 – Actions on Bridges 2' it is best to use Eq. 6.10. This equation will be used. The 3 combinations are visible in Figure 43 for comparison.

$$\sum(\gamma_{Gj,sup} G_{kj,sup} + \gamma_{Gj,inf} G_{kj,inf}) + \gamma_{Q,1} Q_{k,} + \sum \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad \text{(Eq. 6.10 from EN 1990)}$$

With:

- $G_{kj,sup}$: Characteristic value for unfavourable permanent action j.
- $G_{kj,inf}$: Characteristic value for favourable permanent action j.
- $Q_{k,1}$: Characteristic value of the leading variable action 1.
- $Q_{k,i}$: Characteristic value of the other variable actions.
- $\gamma_{G,j}, \gamma_{G,i}, \gamma_{Q,i}$: Partial factors.
- $\psi_{0,i}$: Partial factor for combinations.

The partial factors can be taken out of EN 1990+A1 as shown on the next page. They are also generated automatically with SCIA Engineer. Nonetheless is it handy to have some kind of check of the values in SCIA.

The factors are summed up bellow:

Gr1a:

- $\gamma_{Gj,sup}$: 1,35
- $\gamma_{Gj,inf}$: 1,00
- $\gamma_{Q,i}$: 1,35 $(unfavourble)$
- $\gamma_{Q,i}$: 0 $(favourble)$
- $\psi_0\ (TS)$: 0,75
- $\psi_0\ (UDL)$: 0,40
- $\psi_1\ (TS)$: 0,75
- $\psi_1\ (UDL)$: 0,40
- $\psi_0\ (T_k, Thermische\ belasting)$: 0
- ξ: 0,85 $(In\ case\ Eq.\,6.10b\ is\ used)$

This leads to the following simplification of formula 6.10:

$$\sum (1{,}35 G_{kj,sup} + 1{,}00 G_{kj,inf}) + \ 1.35 Q_{k,} + \sum \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$

Apart from the STR/GEO/EQU – fundamental combinations, there is also an accidental combination for the ULS. This accidental combination will not be taken into account. This combination has to do with accidental structural damage (EN 1991-1-7). As already been discussed earlier in the document, the structural elements cannot be damaged in any situations. Even car crashes on the bridge will not damage the supporting structures, but the railings or road restraints can be damaged.

The Partial factors can be found in (Figure 42) and (Figure 43).

Action	Symbol		ψ_0	ψ_1	ψ_2
Traffic loads	gr1a	TS	0.75	0.75	0
(see EN 1991-2, Table 4.4)	(LM1 + pedestrian or cycle-track loads)[a]	UDL	0.40	0.40	0
		Pedestrian + cycle-track loads[b]	0.40	0.40	0
	gr1b (single axle)		0	0.75	0
	gr2 (horizontal forces)		0	0	0
	gr3 (pedestrian loads)		0	0	0
	gr4 (LM4 – (crowd loading))		0	0.75	0
	gr5 (LM3 – (special vehicles))		0	0	0
Wind forces	F_{Wk}				
	• Persistent design situations		0.6	0.2	0
	• Execution		0.8	–	0
	F_W^*		1.0	–	–
Thermal actions	T_k		0.6[c]	0.6	0.5
Snow loads	$Q_{Sn,k}$ (during execution)		0.8	–	–
Construction loads	Q_c	1.0		–	1.0

[a] The recommended values of ψ_0, ψ_1 and ψ_2 for gr1a and gr1b are given for road traffic corresponding to adjusting factors α_{Qi}, α_{qi}, α_{qr} and β_Q equal to 1. Those relating to UDL correspond to common traffic scenarios, in which a rare accumulation of lorries can occur. Other values may be envisaged for other classes of routes, or of expected traffic, related to the choice of the corresponding α factors. For example, a value of ψ_2 other than zero may be envisaged for the UDL system of LM1 only, for bridges supporting severe continuous traffic. See also EN 1998.

[b] The combination value of the pedestrian and cycle-track load, mentioned in Table 4.4a of EN 1991-2, is a 'reduced' value. ψ_0 and ψ_1 factors are applicable to this value.

[c] The recommended ψ_0 value for thermal actions may in most cases be reduced to 0 for ultimate limit states EQU, STR and GEO. See also the design Eurocodes.

Figure 42: Recommended ψ -values for road bridges

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

Persistent and transient design situation	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Main (if any)	Others
(Eq. 6.10a)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$		$\gamma_{Q,1}\psi_{0,1}Q_{k,1}$	$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$
(Eq. 6.10b)	$\xi\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_P P$	$\gamma_{Q,1}Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Tables A2.1 to A2.3.

NOTE 1 The choice between 6.10, or 6.10a and 6.10b will be in the National Annex. In the case of 6.10a and 6.10b, the National Annex may in addition modify 6.10a to include permanent actions only.

NOTE 2 The γ and ξ values may be set by the National Annex. The following values for γ and ξ are recommended when using expressions 6.10, or 6.10a and 6.10b:
$\gamma_{G,sup} = 1,35$[1)]
$\gamma_{G,inf} = 1,00$
$\gamma_Q = 1,35$ when Q represents unfavourable actions due to road or pedestrian traffic (0 when favourable)
$\gamma_Q = 1,45$ when Q represents unfavourable actions due to rail traffic, for groups of loads 11 to 31 (except 16, 17, 26[3)] and 27[3)]), load models LM71, SW/0 and HSLM and real trains, when considered as individual leading traffic actions (0 when favourable)
$\gamma_Q = 1,20$ when Q represents unfavourable actions due to rail traffic, for groups of loads 16 and 17 and SW/2 (0 when favourable)
$\gamma_Q = 1,50$ for other traffic actions and other variable actions [2)]
$\xi = 0,85$ (so that $\xi\gamma_{G,sup} = 0,85 \times 1,35 \cong 1,15$).
$\gamma_{Gset} = 1,20$ in the case of a linear elastic analysis, and $\gamma_{Gset} = 1,35$ in the case of a non linear analysis, for design situations where actions due to uneven settlements may have unfavourable effects. For design situations where actions due to uneven settlements may have favourable effects, these actions are not to be taken into account.
See also EN 1991 to EN 1999 for γ values to be used for imposed deformations.
γ_P = recommended values defined in the relevant design Eurocode.

[1)] This value covers: self-weight of structural and non structural elements, ballast, soil, ground water and free water, removable loads, etc.
[2)] This value covers: variable horizontal earth pressure from soil, ground water, free water and ballast, traffic load surcharge earth pressure, traffic aerodynamic actions, wind and thermal actions, etc.
[3)] For rail traffic actions for groups of loads 26 and 27 $\gamma_Q = 1,20$ may be applied to individual components of traffic actions associated with SW/2 and $\gamma_Q = 1,45$ may be applied to individual components of traffic actions associated with load models LM71, SW/0 and HSLM, etc.

Figure 43: Partial factors for different types of loadings

The statement: $E_d \leq R_d$ should be checked for the N, V and M according to the method below for elements without problems on buckling.

- $N_{Ed} < N_{Rd}$

$N_{Rd} = f_{cd} * A_C + f_{yd} * A_S$

$f_{cd} = f_{ck}/\gamma_c$

$f_{ck} = 30, 35, 50, 60\ N/mm^2$ (Depending on the concrete strength class).

$\gamma_c = 1.5$

➔ $f_{cd} = 20$ N/mm², $f_{cd} = 23.33$ N/mm², $f_{cd} = 33.33$ N/mm², $f_{cd} = 40$ N/mm²(for C30, 35, 50 and 60, respectively)

$f_{yd} = f_{yk}/\gamma_s$

$f_{yk} = 500\ N/mm^2$ (For the steel rebars)

$\gamma_s = 1.15$

➔ $f_{yd} = 434.78\ N/mm^2$

- $V_{Ed} < V_{Rd}$

$V_{Rd} = 0{,}12 * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} * b_w * d + k\sigma$

$$\rho_l = \frac{A_{s1}}{b_w * d}$$

$f_{ck} = 30, 35, 50, 60\ N/mm^2$ (Depending on the concrete strength class).

$k = 1 + \sqrt{\frac{200}{d}}$

➔ $If\ V_{Ed} < V_{Rd} \rightarrow OK$ (No extra reinforcement needed beside minimal).
➔ $If\ V_{Ed} > V_{Rd}$: (extra reinforcement needed, calculated as followed):

Check: $V_{Ed} < V_{Rd,c} + V_{Rd,s}$

$V_{Rd,s} = \frac{A_{sw}}{s} * z * f_{ywd} * \cot(\theta)$ (For reinforced steel)

$f_{ywd} = 434{,}78\ N/mm^2$

$\theta = 45°$

- $M_{Ed} < M_{Rd}$

$$M_{Rd} = F_c * z = F_s * z$$

- $M_{Ed}\ and\ N_{Ed} < M_{Rd}\ and\ N_{Rd}$

In order to calculate both resisting values a system of two equations is needed. The following two equations can be used in a cross section:

$$F_c - N_{s1} = N_{Rd}$$

$$M_{Rd} + N_{Rd} * \left(\frac{h}{2} - d1\right) = N_c(d - 0,8 * \frac{x}{2})$$

From these equations the according N_{Rd} and M_{Rd} depending on the different sections can be obtained.

References [19], [21].

4.2. Serviceability limit State (SLS)

In the serviceability limit state a few checks need to be performed, depending on the demands of the project. In general for bridges the check are mainly connected to the following:

- Deformations
- Vibrations

In the Eurocode there are 3 different combinations for the serviceability limit state. For a certain kind of deformation or vibration, a certain combination needs to be used. In the following figure (Figure 44) these 3 combinations are shown.

Combination	Reference: EN 1990	General expression
Characteristic	(6.14)	$\sum_{j\geq 1} G_{k,j}{}''+''P''+''Q_{k,1}{}''+''\sum_{i>1}\psi_{0,i}Q_{k,i}$
Frequent	(6.15)	$\sum_{j\geq 1} G_{k,j}{}''+''P''+''\psi_{1,1}Q_{k,1}{}''+''\sum_{i>1}\psi_{2,i}Q_{k,i}$
Quasi-permanent	(6.16)	$\sum_{j\geq 1} G_{k,j}{}''+''P''+''\sum_{i\geq 1}\psi_{2,i}Q_{k,i}$

Figure 44: General expressions of combinations of actions for serviceability limit states (Data taken from EN 1990: 2002/A1)

The verification of serviceability limit states concerning deformation and vibration only needs to be checked in exceptional cases for road bridges, once completed. When they need to be checked, the frequent combination is used to determine the deformation.

Reference [19], [21].

Chapter 5: Calculations and results

5.1 Introduction

In this chapter the past two chapters have been merged together. The model has been constructed and the loads have been calculated. Now the loads need to be applied onto the 3D model so the actual calculations can be made.

With the loads, certain combinations will be made to search for the worst possible situations. Combinations in the ultimate limit state as well as in the serviceability limit state. The combined effects of these combinations are then used to perform certain checks on different elements.

Since the whole structure is designed with 2D-elements instead of 1D not a lot of checks can be performed by SCIA itself. Buckling and response checks for concrete itself need to be checked by hand rather than being calculated by the software. This is one of the downsides of using 2D elements instead of 1D. The choice to use 2D elements is one which both has advantages and disadvantages. This is one of the disadvantages.

The following data will be checked:

- Stresses inside the steel
- Stresses inside the concrete
- Buckling of columns
- Reinforcement needed
- Crack width
- Deformation
- Deflection

It would be normal to check both the stresses in the concrete and reinforcing steel elements. Unfortunately, this is not possible. The prestressing done on this bridge is very complex. SCIA Engineer also does not support a viable way to do prestressing on separate 2D elements. This made it so the bridge could not be reinforced as it should in the 3D model. SCIA does have a tool with which normal reinforcements can be calculated. Unfortunately for this bridge, standard reinforcements are not enough.

Due to limitations of the software, some simplifications had to be made. When the 4 traffic lanes representing the 4 notional lanes were created, the tandem systems were added for every possible position in order to let SCIA calculate the worst position. When the calculations were started, the software couldn't handle the amount of loads (over 775 load cases). The software crashed. A different method needed to be adapted. The adapted method was to only apply the tandem loads on a few positions. The loads will only be applied on the worst positions according to the influence lines from the 2D-model. Of course they cannot be applied for every worst case scenario for a certain part of the structure. There have been 13 loads applied on the edges between two top deck elements.

5.2 Ultimate Limit State (ULS)

5.2.1 Arch

5.2.1.1 Bending stress

$$\theta_m = \frac{M_d}{W} \leq f_{m,d}$$

With:

$M_d =$ taken from SCIA (will be the design value).

The width of the arch is variable so the value for W is also variable. It will be calculated with a mean value. This for both the bottom and top section.

Bottom section:

$M_d = 900\frac{kNm}{m} * 18.333m = 16\ 500\ kNm$ (SCIA)

B = 18.333 m

$W = 6.8749\ m^3$

- ➔ θ_m = 2400 kN/m² = 2.4 N/mm²
- ➔ $f_{m,d}$ = 36.96 N/mm²

- ➔ $\theta_m \leq f_{m,d}$ $=> 2.4\ \leq 39.96 \rightarrow \text{OK}$

Top section:

$M_d = 1500\frac{kNm}{m} * 11.667m = 17\ 500\ kNm$ (SCIA)

B = 11.667 m

$W = 4.375\ m^3$

- ➔ θ_m = 4000 kN/m² = 4 N/mm²
- ➔ $f_{m,d}$ = 36.96 N/mm²

- ➔ $\theta_m \leq f_{m,d}$ $=> 4\ \leq 39.96 \rightarrow \text{OK}$

This is as expected. The bending moments in the arch are rather limited thanks to its low rigidity. No extra measures need to be taken.

5.2.1.2 Reinforcements:

In order to create concrete bridges with spans as long as the Ponte do Infante, there is a need for steel fortifications. Even in short bridges steel is always used. The last concrete bridge built without steel reinforcement's dates back to the 1950's. These reinforcements can appear in two forms. Either reinforced concrete or pre-stressed concrete. Both have been used in this project. The box beam has been pre-stressed while the columns and arch have been reinforced.

These prestressed reinforcements can be designed with SCIA Engineer. The only problem lies in the complexity of the reinforcements. There was almost no information available from the designer in which way the reinforcements have been made. Since this is a complex matter, it could not just be drawn out by hand in SCIA Engineer. The same applied for the reinforced concrete, with the exception that there was another way to design these. These reinforcements could be calculated automatically by SCIA. Only the amount of steel needed is given, not how much and what bars. This has been performed for the different elements in order to get an idea of how much reinforcement is needed for the project.

The stresses inside the concrete do not need to be checked. They will exceed the values for non-reinforced concrete without a doubt. To calculate the amount of reinforcement needed in a structure, the stresses can be used. An alternative way to do this is built into SCIA. With SCIA Engineer the amount of reinforcement needed for each element can be calculated automatically (as mentioned above). SCIA uses the right stresses and gives values for the amount of reinforcement needed. This way it is useless to check the stresses, since SCIA will make sure the stresses are acceptable according to the amount of reinforcement needed.

The results can be found in the figures and tables below (Figure 45 - Figure 47 and Table 7 - Table 11) for the following 2D elements (these are the 2D elements who have been defined as most critical):

- E5: Bottom part of the arch
- E7: Top part of arch before connection with the deck
- E18: Longest column on top of the arch (M1)
- E172: Column outside the arch (P1)
- E20: Shortest column on the arch (M3)
- E239, E240, E212, E214, E…: Middle parts of the deck/arch connection.

5.2.1.2.1 *Bottom part of the Arch*:

The amount of reinforcement required for this part is given in the figure below (Figure 45) (taken out of SCIA).

Staaf	elem	BG	A_{s1-} [mm²/m]	A_{s2-} [mm²/m]	A_{s1+} [mm²/m]	A_{s2+} [mm²/m]	A_{sw} [mm²/m²]
E5	1	ULS - Fund. All	**4801**	1500	3310	1500	339
E5	18	ULS - Fund. All	2790	**13949**	1500	1500	**3866**
E5	1	ULS - Fund. All	1500	1500	1500	1500	3025
E5	35	ULS - Fund. All	3310	4197	**4150**	1500	0
E5	360	ULS - Fund. All	3310	1500	3310	**10019**	0

Figure 45: Amount of reinforcement needed in E5

- These values can also be visualised in a figure. An example can be found in Annex III.

For the longitudinal values (As1, $2^{+/-}$) an acceptable amount of concrete design can be found. The mean values will be used rather than the maximum values. This because of a connection error in SCIA. When taking a look at the internal forces in the plates, there can be seen that these get extremely high towards the nodes of each element. The only logical explanation for this can be that the loads from each element towards the next one get transferred only thru the nodes. This leads to all the loads coming together in one point. In reality the loads get transferred evenly over the whole connecting edge. The loads are more spread out and the stresses are lower.

For this reasoning the mean values from the centre of the plate will be used to calculate the main reinforcements. Eventually some extra reinforcements can be added towards the edges to ensure a smooth transition of the loads from one element towards the next. In reality the bridge won't be made as in the 3D model. The arch was built with an advancing cantilevering method instead of the big elements drawn in the software. This means these extra reinforcements are needed in between the different parts of the advancing.

Since 2D elements are made, reinforcement nets will be used instead of separate beams.

Table 7: Required amount of reinforcements – bottom part of the arch

	Amount [mm²/m]	Net [mm²/m]
A_{S1-}	3310	B 12 -75 + 3 ø28 / m
A_{S1+}	3310	B 12 -75 + 3 ø28 / m
A_{S2-}	1500	B 12 -75
A_{S2+}	1500	B 12-75
A_{Sw}	0-3866	9⌀32

In general there will be opted for a B 12-75 net both on top and bottom. This net ensures a coverage of 1507.96 mm²/m in both the x- and y-direction. With this net the values for the y/2- direction are satisfied. For the x-direction there is a need for extra reinforcements. These will be provided in the form of 3 ø28 bars every meter. This gives an extra coverage of 1847 mm²/m bringing the total to 3355 mm²/m.

For the shear reinforcement a bigger amount is needed for every meter. This is translated into separate beams. Minimum reinforcement will suffice.

$$\rho_{w,min} = \frac{0.08 * \sqrt{f_{ck}}}{fyk} = \frac{0.08 * \sqrt{50}}{500} = 0.0011$$

$$\rho_{w,min} = \frac{A_{sw}}{b * s * sin\alpha} => A_{sw} = 0.0011 * 11\,667 * 1 * 562.5 = 7219\ mm^2$$

- ➔ 9⌀32 every 562.5 mm

5.2.1.2.2 Central part of the arch:

This is where the arch connects to the box beam to make an extended box beam with a height of 6 meters. The calculations are discussed under the section beam box.

5.2.1.2.3 Arch part just before connecting to the beam

Naam	Net	Positie [m]	BG	$A_{s,req,1-}$ [mm²/m]	$A_{s,req,2-}$ [mm²/m]	$A_{s,req,1+}$ [mm²/m]	$A_{s,req,2+}$ [mm²/m]	$A_{sw,req}$ [mm²/m]	Errors
E7	Knoop: 5	16.668 -210.000 20.641	ULS - Fund. All	**3333**	**1924**	**3870**	**3310**	**8139**	**1**
E7	Knoop: 6	3.333 -210.000 20.641	ULS - Fund. All	**3333**	**0**	**3758**	**0**	**7975**	**1**
E7	Knoop: 7	15.501 -175.000 25.597	ULS - Fund. All	**0**	**0**	**0**	**0**	**5176**	**0**
E7	Knoop: 8	4.501 -175.000 25.597	ULS - Fund. All	**0**	**0**	**0**	**0**	**0**	**0**

Figure 46: Amount of reinforcement needed in E7

This is a selection of 4 results out of the hundreds of nodes in the net of element E7 (Figure 46). This selection has been made because in most nodes there are one or more values which are 0. This means no reinforcement is needed, so minimum reinforcement will be added. Because the course of the places needing reinforcement is unpredictable there will be placed more than minimum reinforcement over the whole section. This in order to make sure no mistakes can be made, with the danger of the element not being reinforced enough in certain sections.

Table 8: Required amount of reinforcements - top part of the arch

	Amount [mm²/m]	Net [mm²/m]
A_{S1-}	3333	B 12 -75 + 3 ø28 / m
A_{S1+}	3870	B 12 -75 + 4 ø28 / m
A_{S2-}	1924	B 12 -75 + 3 ø28 / m
A_{S2+}	3310	B 12 -75 + 3 ø28 / m
A_{SW}	0-3431	9 ø32

The same nets will be used for the whole arch. The only difference lies in the extra amount of reinforcements required. For the bottom side of the local z-axis (As-) the same reinforcements as the lowest part of the arch are enough. For the top side of the element an extra bar of ø28 needs to be added.

For the shear force minimum reinforcement is enough.

$$V_{Rds,min} = \rho_{w,min} * b_w * 0.9 * d * f_{ywd} * cot\theta$$

$$= 0.0009 * 11\,667 * 0.9 * 750 * \cot(45\,) = \; 4860\; N$$

$$\rho_{w,min} = \frac{0.08 * \sqrt{f_{ck}}}{fyk} = \frac{0.08 * \sqrt{50}}{500} \; = \; 0.0011$$

$$\rho_{w,min} = \frac{A_{sw}}{b * s * sin\alpha} => A_{sw} = 0.0011 * 11\,667 * 1 * 562.5 = 7219\; mm^2$$

➔ 9ø32 every 562.5 mm

5.2.2 Beam box

5.2.2.1 Reinforcement

Central Part

In the central part the arch connects to the bridge, making a box beam with a total height of 6 meters. Unfortunately SCIA cannot link the separate 2D elements together to act as one big element. The corresponding function in SCIA for this pretends as if they are already connected. This makes it impossible to calculate reinforcements in this section, since the reinforcement designed in each separate 2D element would need to deal with the full forces applied onto that section.

An alternate way to calculate the amount of reinforcement needed for the central part would be to calculate it by hand. Apparently no prestressing needs to be calculated for the central part, since the arch connecting to the beams compresses the concrete enough.

For the other parts of the beam box, the calculations for prestressing need to be made. These have been made in the following chapter 'maximum outside central part'. They are calculated for the maximum moment, but will be used over the whole length of the bridge.

Outside central part

Beam box data:

A = 16.665 m²

I_y = 6.2879*10¹ m⁴

i² = 3.773 m²

M_g = (Permanent loads from SCIA) = 33 804 kNm

M_q = (Variable loads from SCIA) = 741 kNm

M_m = (mobile loads from SCIA) = 9081 kNm

C 50/60 // A500 NR

A_p = 150 mm^2 => d = 15.7 mm

$f_{pk} = 1860\ kPa = \ f_{pd} = \frac{0.9*1860}{1.15} = 1455.65\ kPa$

<u>Solution</u>:

- <u>The maximum stress of concrete in prestressed phase</u>

  $f_{ct,\mathrm{adm,VF}} = 0{,}7\ x\ 0{,}3\ f_{ck}{}^{2/3}$ = 0,21 x $50^{2/3} = 2.85\ N/mm^2$
  $f_{c,\mathrm{adm,VF}} = 0{,}5\ f_{ck}$ = 0,5 x 50 = 25 N/mm^2

  **VF = prestressed phase*

- <u>The maximum stress of concrete in serviceability limit state</u>
  $f_{ct,\mathrm{adm,GF}} = 0\ N/mm^2$
  $f_{c,\mathrm{adm,GF}}$ = 0,45 f_{ck} = 0,45 x 50 = 22.5 N/mm^2

  **GF = serviceability limit state*

<u>The maximum stress in prestressed steel</u>
$f_{pk} = 1860\ kPa$
$f_{p0,1k} = 1600\ \mathrm{N}/mm^2$ (pg 12, d = 15,7 mm)

$\sigma_{p,m(0)}\ \leq 0{,}75\ \ f_{pk} = 1395\ \mathrm{N}/mm^2$
$\leq 0{,}85\ \ f_{p,0,1,k}$ = **1360** N/mm^2
=> The lowest one is chosen.

Geometric eccentricity

$$e_{max,g} = a_1 - c - \frac{\emptyset_k}{2} = 2101 - 65 - \frac{15,7}{2} = 2028,15\ mm$$

M_g = Value taken from SCIA =33 804 kNm =3.3804 10^{10} Nmm

$$\sigma_{g1} = \frac{Mg\ x\ a_1}{I_y} = \frac{3,3804\ x\ 10^{10}\ x\ 2133,85}{6,2879\ \ x\ 10^{13}} = 1.147\ \mathrm{N}/mm^2$$

$$\sigma_{g2} = \frac{Mg\ x\ a_2}{I_y} = \frac{3,3804\ x\ 10^{10}\ x\ 2399}{6,2879\ \ x\ 10^{13}} = 1.29\ \mathrm{N}/mm^2$$

M_q = 741 kNm = 7.41 * 10^8 Nmm

$$\sigma_{q1} = \frac{Mq\ x\ a_1}{I_y} = \frac{7.41\ \ x\ 10^8\ x\ 2133,85}{6,2879\ \ x\ 10^{13}} = 0.025\ N/mm^2$$

$$\sigma_{q2} = \frac{Mq\ x\ a_2}{I_y} = \frac{7,41\ \ x\ 10^8\ x\ 2399}{6,2879\ \ x\ 10^{13}} = 0.028\ N/mm^2$$

➔ Moments from the variable loads are negligible next to the permanent and mobile loads.

M_m = 9081 kNm = 9.081 * 10^9 Nmm

$$\sigma_{m1} = \frac{Mm\ x\ a_1}{I_y} = \frac{9.081\ \ x\ 10^9\ x\ 2133,85}{6,2879\ \ x\ 10^{13}} = 0.308\ N/mm^2$$

$$\sigma_{m2} = \frac{Mm\ x\ a_2}{I_y} = \frac{9,081\ \ x\ 10^9\ x\ 2399}{6,2879\ \ x\ 10^{13}} = 0.346\ N/mm^2$$

- Define number of strings

$$-\eta \frac{Pi}{A}\left[1+\frac{e\,a_1}{i^2}\right]+\sigma_{g1}+\sigma_{q1} \leq \sigma_{ct,adm}$$

$$\frac{1}{Pi} \leq \frac{-\eta\left[1+\frac{e\,a_1}{i^2}\right]}{A\,[\,\sigma_{ct,adm}-\sigma_{g1}-\sigma_{q1\,-}\,\sigma_{m1}\,]}$$

$$\frac{1}{Pi} \leq \frac{-0{,}85\left[1+\frac{2028.15\,x\,2101}{3773117}\right]}{16665000\,[0-1.147-0.025-0.308]} = \frac{-1.810}{-2.446\,x\,10^{7}} = 7.3998\,x\,10^{-8}$$

$$Pi = \frac{1}{7.3998\,x\,10^{-8}} = 13\,514\,kN$$

n = $\frac{Pi}{A_p\,\sigma_{p,m(0)}} = \frac{1.3514\,x\,10^{7}}{150\,x\,1360}$ = 66.26 => 67 strands

Horizontal space in between strands must be $\geq 40\ mm$

➔ 40mm + 15.7mm = 55.7mm

1st layer:

B – 2x concrete - $\emptyset_k$ = 10 000 – 2*65 – 15.7 = 9854.3 mms

$\frac{9854.3}{55.7} = 1777 strands \rightarrow they\ can\ all\ be\ placed\ in\ the\ first\ layer.$

5.2.3 Columns

5.2.3.1 Compression force

First there needs to be checked if the concrete can withstand the compression force so that no crushing of the concrete can occur. This with the following formula:

$$\theta_c = \frac{F_d}{A} \leq f_{c,d}$$

With:

A: surface of the cross section

$F_{c,d}$: Compressive strength (depends on the concrete class)

$f_{c,d} = \alpha_{cc} * \frac{f_{ck}}{\gamma_c} = 0.85 * \frac{35}{1.5} = 19.83$ (Recommend value in Belgium).

Column M1/M4

The highest loaded column is selected:

Instead of F_d, a N_{cd} – value could be found in SCIA, which had to be smaller than $f_{c,d}$. The concrete type is C35/45 ->$f_{c,d} = 19.83$.

$$F_d = 27\,000 \frac{kN}{m} * 10\ m = 270\,000\ kN$$

$$\theta_c = \frac{F_d}{A} \leq f_{c,d} \rightarrow \ \theta_c = \ 22.5 \leq 19.83 \rightarrow \text{NOT OK}$$

➔ This is because the column is so small at the top. The column gets wider towards the bottom so it will be able to withstand the force. In Portugal they also multiply the f_{ck}-value with 1 instead of 0.85, which leads to a value of 23.33 for $f_{c,d}$. This all leads to assuming the check will pass → OK

Column P1

For the column outside the arch this gives the following:

Instead of F_d a N_{cd} – value could be found in SCIA, which had to be smaller than $f_{c,d}$. The concrete type is C30/37 -> $f_{c,d} = 18.21$.

$$F_d = 3800 \frac{kN}{m} * 10\ m = 38\,000\ kN$$

$$\theta_c = \frac{F_d}{A} \leq f_{c,d} \rightarrow \ \theta_c = 3.167 \leq 18.21 \rightarrow \text{OK}$$

➔ The column is able to deal with the pressure force applied onto it. No stronger concrete type is needed.

5.2.3.2 Reinforcements

Column M1

For column M1 both longitudinal reinforcement in both directions and shear reinforcement is needed (Figure 47). Since the forces applied on the columns are quite high, the reinforcements needed are as well. Yet again the forces from the deck seem to be transmitted on the column only in the nodes rather than the whole contacting surface. This leads to high values in these nodes, which can be left out. The following mean values are obtained from SCIA. Number 1 stands for the local X-axis. Number 2 for the Local Y-axis. +/-, either on the positive or negative side of the local Z-axis of the element. E.g. As,1- is the reinforcement needed in the X-direction on the bottom of the column (negative Z-side).

Staaf	**elem**	**BG**	**A_{s1-} [mm²/m]**	**A_{s2-} [mm²/m]**	**A_{s1+} [mm²/m]**	**A_{s2+} [mm²/m]**	**A_{sw} [mm²/m²]**
E17	3505	ULS - Fund. All	**25232**	5046	7921	4856	0
E17	3091	ULS - Fund. All	4806	**12688**	1200	1200	1667
E17	3091	ULS - Fund. All	0	0	0	0	4994
E17	3505	ULS - Fund. All	16216	3243	**16319**	12472	**6997**
E17	3432	ULS - Fund. All	2628	0	3479	**17393**	1057

Figure 47: Amount of reinforcement needed in E17 (maximum in the nodes)

For the same reasoning as for the bottom part of the arch, mean values are used for designing the reinforcements. These mean values are concluded in Table 9.

Table 9: Required amount of reinforcements - column M1

	Amount [mm²/m]	Net [mm²/m]
A_{S1-}	1400	B 12-75
A_{S1+}	1400	B 12-75
A_{S2-}	1200	B 12-75
A_{S2+}	1200	B 12-75
A_{Sw}	0-6997	4⌀36

For the x-/y-direction the same nets can be used as for the Arch. Nets B 12-75 are used and provide enough reinforcement in both directions.

$A_{sw,req}$: The value seemed to be inconsistent. Going from 0 in general up to 6997 mm²/m² in the nodes. In general there will be opted for minimal reinforcement since there is only need for extra high reinforcements close to the edges and nodes. This will be seen as a software malfunction (this is visualised in Annex III).

This minimum reinforcement is:

(See next page).

$$V_{Rds,min} = \rho_{w,min} * b_w * 0.9 * d * f_{ywd} * cot\theta$$

$$= 0.0009 * 10\,000 * 0.9 * 600 * \cot(45\,) = \ 4860\ N$$

$$\rho_{w,min} = \ \frac{0.08 * \sqrt{fck}}{fyk} = \frac{0.08 * \sqrt{35}}{500} \ = \ 0.0009$$

$$\rho_{w,min} = \ \frac{A_{sw}}{b * s * sin\alpha} => A_{sw} = 0.0009 * 10\,000 * 1 * 450 = 4050\ mm^2$$

➔ 4⌀36

Column P1

The same can be done for column P1. This leads to the following values bellow:

Table 10: Required amount of reinforcements - column P1

	Amount [mm²/m]	Net [mm²/m]
A_{S1-}	1200	B 12-75
A_{S1+}	1200	B 12-75
A_{S2-}	1200	B 12-75
A_{S2+}	1200	B 12-75
A_{Sw}	0 - 2900	4ø36

The longitudinal reinforcements seemed to be the same as for column M1.

$A_{sw,req}$: The values seemed to be less inconsistent in this case. Going from 0 up to 2900 mm²/m in the 3 nodes of the net. Therefore In general there will be opted for minimal reinforcement. There might need be carried out an extra study to find out if any more than minimum reinforcement is needed.

The same calculations for the amount of shear reinforcement apply as for M1. This lead to:

$$V_{Rds,min} = \rho_{w,min} * b_w * 0.9 * d * f_{ywd} * cot\theta$$

$$= 0.0009 * 10\ 000 * 0.9 * 600 * \cot(45\ ) = \ 4860\ N$$

$$=> A_{sw} = 4050\ mm^2$$

➔ 4ø36

Column M3

This is a very short column. The danger of these small columns is that the surface available to deal with the forces is too small. If that is the case, the only way to change it, is to make the columns wider.

Table 11: Amount of reinforcements needed for M3

	Amount [mm²/m]	Net [mm²/m]
A_{S1-}	2628	Too high
A_{S1+}	10 000+	Too high
A_{S2-}	10 000+	Too high
A_{S2+}	2628	Too high
A_{Sw}	0 – 17 000	Minimum reinforcement

The amount of reinforcement needed for this column is too high. There is not enough space in the concrete in order to place the reinforcements. Another solution needs to be found. Either the width of the column needs to be increased or a better concrete type is needed.

Since the bridge is in fact made with these dimensions and concrete type, there can be assumed that there must still be another way (the bridge has not collapsed to this day yet and probably won't). Either the results in SCIA are not accurate enough or somehow a redistribution of the loads have been made inside. This way some of the loads are transferred to other parts instead of the short columns thus making it so the columns only need to deal with lower forces.

For the shear reinforcements the same conclusion as before can be drawn as for the other columns. Minimum in general and some extra reinforcements towards the edges/nodes.

5.2.3.3 Buckling

Columns are more sensitive to buckling when the compression force in the column rises or the length of the column increases. This is why only two columns will be checked. Column P1, outside the arch and column M1/M4, which are the tallest columns standing on the arch.

The slenderness of the columns is one of most important values for the danger of buckling. The more slender the column, the higher the risk. The buckling length plays a part in the calculation of the slenderness. This buckling length depends on the connections of the column. The slenderness for the columns will be calculated.

The columns have different connections at the top and at the bottom. It was hard to analyse in situ what exactly the connections were, but they have been determined in chapter 2.4. The connections to the deck are all pinned connections. Pinned connections don't allow any bending moment to be transferred from the deck to the column. This means that only horizontal and vertical forces need to be taken into account in the column. At the bottom the columns should normally be completely fixed. Since the dimensions of the connecting beams are known, the connection doesn't really matter. The β-factor can be calculated.

The buckling check of the columns should be considered in both directions (x and y). This in the ULS combination. Concrete can withstand high compression forces, but when a column collapses to buckling it is not due to the failure of the material.

The columns have pinned connections at the top and rigid ones on the bottom. This leads to the following effective length (Figure 48):

(b) l_0=0.7l

Figure 48: effective length l_0

This length formula can be used for both columns. An alternative way is to calculate the β-values. This is what has been done on the next page. This leads to more precise results.

The construction can be seen as non-sway.

Columns M1/M4

Columns M1/M4 are the symmetrical columns on the arch. Since almost the whole bridge is symmetrical, the loads on both the columns will be similar. Column M1 has been selected since the loads might be slightly higher in this column.

Normally buckling around both axis of the columns should be considered. However in this case, the buckling around the local Y-axis will not be considered. This because the Y-axis is the strong axis of the column. Also the loads around the X-axis are way higher, which will lead to the column collapsing around the X-axis much faster than the Y-axis.

The slenderness can be calculated according to 'Gewapend Beton – Berekeningen volgens NBN B 15-002 (1999)', a Belgian handbook used in the past years. For a non-sway construction the following steps need to be followed:

$$\lambda = \frac{l_0}{i}$$

With:

$$i = \sqrt{\frac{I}{A}} = \sqrt{\frac{1{,}44}{10 \; x \; 1{,}2}} = 0.3464 \; m$$

The section of the column changes from top to bottom, since the column gets wider. The smallest values will be used as if the column had a continuous section over the whole length

$$l_0 = \beta * l_{col} = 0{,}6 * 23.502 \; m = 14.10 \; m$$

$\beta \; is \; to \; be \; taken \; out \; of \; graph \; (10.18 \; taken \; from \; the \; belgian \; handbook) \; in \; function \; of \; k_a \; and \; k_b$

$$k_a = \frac{\sum E_{cm} * \frac{I_{col}}{l_{col}}}{\sum E_{cm} * \alpha * \frac{I_{beam}}{l_{beam}}} = \frac{1 * \frac{1.44 \; m^4}{23.502 \; m}}{2 * 0.5 * \frac{6.29 * 10 \; m^4}{35 \; m}} = \frac{0.06127}{1.797} = 0.0341 \text{ (Top node A.)}$$

$$k_b = \frac{\sum E_{cm} * \frac{I_{col}}{l_{col}}}{\sum E_{cm} * \alpha * \frac{I_{beam}}{l_{beam}}} = \frac{1 * \frac{1.44 \; m^4}{23.502 \; m}}{1 * \frac{2.8125 \; m^4}{36.24 \; m}} = \frac{0.06127}{0.0776} = 0.790 \text{ (Bottom node B.)}$$

➔ $\beta = 0.6$

$$\lambda = \frac{l_0}{i} = \frac{14.10}{0.3464} = 40.70$$

Now this value will be checked against the minimum and maximum slenderness. If this value is in between the minimum and maximum, the column will be considered a slender column.

$\lambda_{min} = \max\left(25, 15/\sqrt{v_u}\right) = \max\left(25, \mathbf{44.23}\right)$

With: $v_u = \frac{N_{sd}}{A_c * f_{cd}} = \frac{41\;136}{18 * 19.83 * 10^3} = 0.115$

With SCIA the force N_{Ed} can be obtained (Figure 49). The finite element method gives values in kN/m. For the force in the y-direction this value goes from 2227.3 kN/m to 34377.8 kN/m. Because these high values only appear in the top nodes, they will not be taken into account. This could be a software mistake. Probably the software connects the deck with the pillar in the nodes. This would be why the loads are transferred all into both nodes before being spread out over the whole section. A mean value between 2227.3 and 6000 kN/m will be used. There is opted for 4113.65 kN/m. To get the N_{Ed}-value, this value per m needs to be multiplied by the length of the column. This is 10 m. This lead to:

$$N_{Ed} = 4113.65\frac{kN}{m} * 10m = 41\ 136\ kN$$

$$\lambda_{max} = \max\ (50, \mathbf{58.98})$$

$$\lambda < \lambda_{min} \rightarrow non\ slender\ column$$

This means that during the calculation of the column no second order effects need to be taken into account.

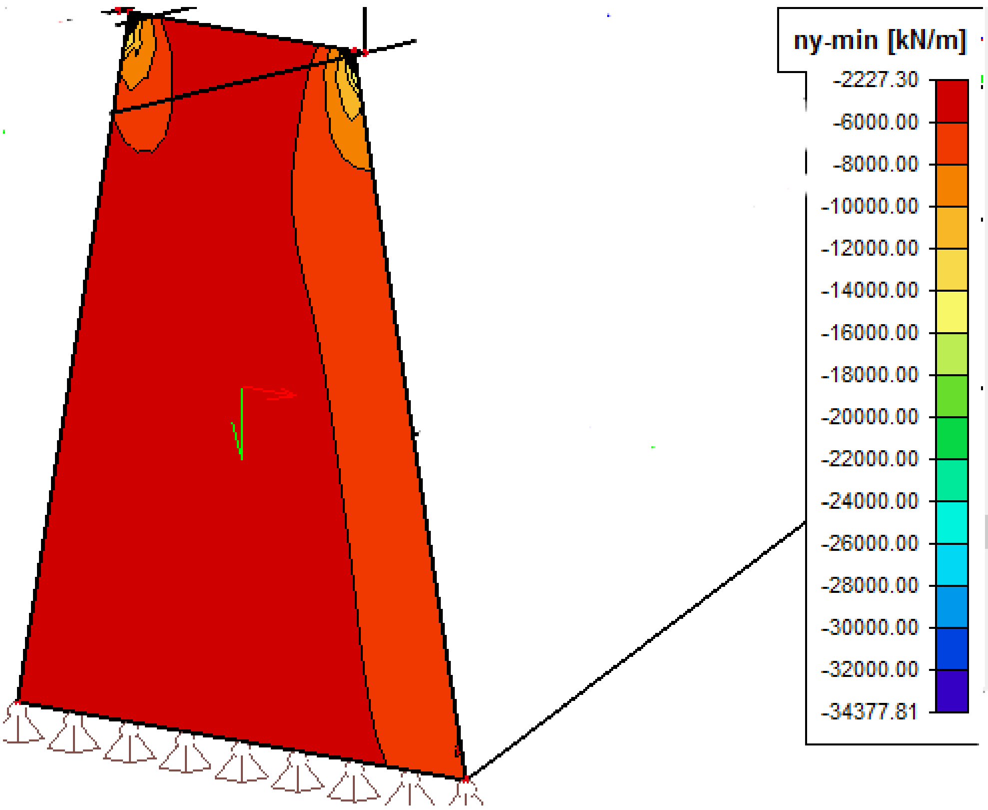

Figure 49: Internal forces pilar M1

Column P1

Column P1 is the only column not standing on the arch. Therefore the forces applied onto this column will be slightly different and should be checked. This is the second tallest column in the bridge except for the columns M1/M4.

The buckling around the z-axis will also not be considered here for the same reason as for columns M1/M4.

Again the slenderness was calculated according to 'Gewapend Beton – Berekeningen volgens NBN B 15-002 (1999)'.

$$\lambda = \frac{l_0}{i}$$

With:

$$i = \sqrt{\frac{I}{A}} = \sqrt{\frac{1{,}44}{10\ x\ 1{,}2}} = 0.3464\ m$$

The section of the column is constant over the whole section. This makes for accurate results.

$$l_0 = \beta * l_{col} = 0{,}57 * 17.18\ m = 9.8\ m$$

$$\beta\ is\ to\ be\ taken\ out\ of\ graph\ (10.18\ in\ the\ handbook)\ in\ function\ of\ k_a\ and\ k_b$$

$k_a = \frac{\sum E_{cm} * \frac{I_{col}}{l_{col}}}{\sum E_{cm} * \alpha * \frac{I_{beam}}{l_{beam}}} = \frac{1 * \frac{1.44\ m^4}{17.18\ m}}{0.5 * \frac{6.29 * 10\ m^4}{28\ m} + 0.5 * \frac{6.29 * 10\ m^4}{35\ m}} = \frac{0.0838}{1.123 + 0.899} = 0.0414$ (Top node A.)

$k_b = 0.4$ (Bottom node, normally completely fixed, but a value less than 0.4 is not recommended).

➔ $\beta = 0.57$

$$\lambda = \frac{l_0}{i} = \frac{9.8}{0.3464} = 28.29$$

Now this value will be checked to the minimum and maximum slenderness. If this value is in between the minimum and maximum, the column will be considered a slender column.

$$\lambda_{min} = \max\left(25, 15/\sqrt{v_u}\right) = \max\left(25, \mathbf{51.43}\right)$$

With: $v_u = \frac{18590\ kN}{12 * 18.21 * 10^3} = 0.085$

With SCIA the force N_{Ed} can be obtained (Figure 50). The finite element method gives values in kN/m. For the force in the y-direction this value goes from 283 kN/m to 27 240 kN/m. Because these high values only appear in the top nodes, they will not be taken into account.

A mean value between 283 and 4000 will be used. There is opted for 1859 kN/m. To get the N_{Ed}-value, this value per m needs to be multiplied by the length of the column. This is 10 m. This lead to:

$$N_{Ed} = 1859 \frac{kN}{m} * 10m = 18\,590\ kN$$

$$\lambda_{max} = \max(50, 20/\sqrt{v_u}) = (50,\ \mathbf{68.60})$$

$$\lambda < \lambda_{min} \rightarrow non\ slender\ column$$

This means that during the calculation of the column seconder order effects need to be taken into account.

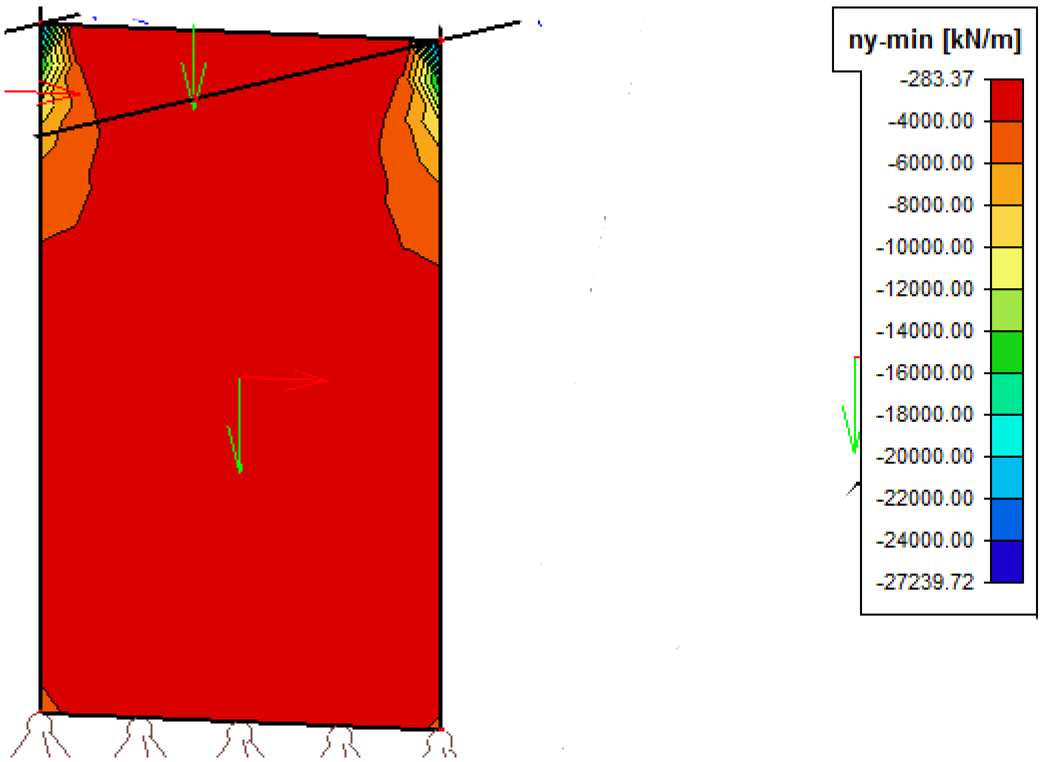

Figure 50: Internal forces Pilar P1

5.3 Serviceability Limit State (SLS)

There will be checked if:

$E_d < C_d$

With:

E_d: The design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination [19].

C_d: The limiting design value of the serviceability criterion.

This check will be performed for the most critical section. These are the sections that are either the highest loaded or have the least amount of surface to deal with high forces.

Linear Creep

Linear creep can be considered as long as the following check is passed. If not, deformations on long term will have to be calculated with non-linear creep instead. The Quasi-Permanent Combination is used.

<u>Check</u>:

$E_d < C_d$

<u>With</u>:

$E_d = Pressure\ force\ inside\ the\ concrete\ [\frac{N}{mm^2}]$

$C_d = k_2 * f_{ck}$

f_{ck} changes for the different types of concrete. The different values are:

$Pillar\ P1$:	$f_{ck} = 30 \rightarrow C_d = 13{,}5$	$E_d = 2$ N/mm²	Check: OK
$Pillars\ on\ the\ arch$:	$f_{ck} = 35 \rightarrow C_d = 15.75$	$E_d =$ 10-20 N/mm²	Check: NOT OK
$Arch$:	$f_{ck} = 50 \rightarrow C_d = 22.5$	$E_d =$ 18-24 N/mm²	Check: NOT OK
$Box\ beam$:	$f_{ck} = 60 \rightarrow C_d = 27$	$E_d =$ 15.7 N/mm²	Check: OK

$k_2 = 0{,}45$

In multiple parts of the arch the tensions are higher than the maximum value of 22.5. This means non-linear creep effects will have to be used instead of linear ones. For the columns only pillar P1 passes the check. Pillars M2/M5 only fail the test in the nodes, which can be because of a model misrepresentation of the physical reality due to excessive concentration in the nodes (explained before). The central pillars (M3/M4) fail the check in general. This because of the small surface of the pillars. The central part of the box beam also surprisingly passes the test.

Tensile forces in steel

The tensile forces in steel need to be limited. This in order to prevent non-elastic stretching, unacceptable cracking and deformation. The Quasi-permanent combination is used.

As long as the forces are lower than $k_3 * f_{yk}$ the crack formation and deformation will be avoided. Nonetheless for prestressed elements, the criteria are stronger. The value in that case needs to be lower than $k_5 * f_{yk}$.

The values for $k_3 \ and \ k_5$ are respectively 0.8, 0.75.

Check:

$E_d < C_d$

With:

$E_d = Pressure\ force\ inside\ the\ steel\ beams\ [\frac{N}{mm^2}]$

$C_d = k_3 * f_{yk}$ and $k_5 * f_{yk}$

$f_{yk} = 500\ N/mm^2$

- ➔ $C_d = 0{,}8 * 500 \frac{N}{mm^2} = 400\ N/mm^2$
- ➔ $C_d = 0{,}75 * 500 \frac{N}{mm^2} = 375\ N/mm^2$

This check could not be performed with SCIA. When letting the software draw the reinforcements needed for the structure it could not be completed. This is due to performance issues of the software. No steel reinforcements were drawn, so no stresses inside these reinforcements could be checked. This leads to having no actual way of knowing if the forces inside the steel rebars will have non-linear deformations. Since the designers opted for this type of steel, there can be assumed that in reality the check was passed.

Longitudinal cracks

These can appear when the tensile forces exceed a certain critical value in the concrete. This needs to be checked under the Characteristic Combination.

Check:

$E_d < C_d$

With:

$E_d = Pressure\ force\ inside\ the\ concrete\ [\frac{N}{mm^2}]$

$C_d = k_1 * f_{ck}$

$f_{ck}\ changes\ for\ the\ different\ types\ of\ concrete. The\ different\ values\ are$:

$Pillar\ P1$:	$f_{ck} = 30 \rightarrow C_d = 18$	/	$E_d =$ 3-4	Check: OK
$Pillars\ on\ the\ arch$:	$f_{ck} = 35 \rightarrow C_d = 21$	/	$E_d =$ 20-30	Check: NOT OK
$Arch$:	$f_{ck} = 50 \rightarrow C_d = 30$	/	$E_d =$ 27-30	Check: ± OK
$Box\ beam$:	$f_{ck} = 60 \rightarrow C_d = 36$	/	$E_d =$ 24.1	Check: OK

$k_1 = 0{,}6$

The figures for the values of Ed can be found in Annex V: Stresses.

For the arch the test only seems to fail for the connection between the arch segments and the column M3. This could be a software mistake, but due to safety reasons, there will be assumed that the check is failed and the arch is subjected to loss of durability. The central part of the box beam/arch passes the check. For every pillar on the arch the check fails. The pillar outside the arch also fails this test due to having a lower strength class of concrete. The maximum value of 18 only barely get exceeded in the connecting nodes. This might lead to believe that the pillar actually passes the test, as the loads are transferred from one element to another in the nodes.

Crack width and deflection

The deflection of the elements needs to stay low enough. The Eurocode mentions a simple rule. The deflection cannot exceed the maximum value of L/250 under the Quasi Permanent combination (QP). This is mostly important for the arch element, since this element is unsupported for 280 meters. By applying this rule, the arch can have a maximum deflection of 1120 mm. This value is enormous and will not be tolerated in reality. Nonetheless because of lack of another method, this check will be performed.

In most cases the deflection according to the dead load of the bridge is calculated beforehand. When constructing the bridge, they will raise the formwork with this value in order to counteract to the deflection. This trick has actually been used in this case. Right before the connection of the arches the formwork was lowered to let the deflection take place. This means there is actually no deflection present thanks to the dead load of the bridge.

Another way to limit the maximum deflection is by limiting the crack width. When the stresses inside the concrete and steel exceed certain values, cracks start to appear inside the concrete. These cracks get bigger the higher the stresses are. Higher stresses and cracks result in deflection, so by limiting the crack width, the deflection also gets limited. The main reason for limiting the cracks has of course to do with the durability of the bridge.

Deflection

The deflection is calculated according to the method mentioned above. The maximum deflection can be found in the middle of the arch thanks to the following QP combination:

$$1 * EG + 1 * ASPH$$

The maximum deflection is 188.9 mm. This value is lower than L/250 = 1120 mm. According to the Eurocode this is acceptable. This is due to the span being 280 meters so this check will most certainly always pass when performed on arches. The corresponding figure can be found in Annex IV.

Crack Width

The crack width depends on the exposure class. For exposure class XS1, the Eurocode mentions W_{max} = 0.3 mm as the maximum value for crack widths. The crack width of various concrete elements will be checked to this value.

There is a problem when trying to calculate the crack width. The reinforcement of the concrete could not be designed into SCIA as mentioned in part 'tensile forces in steel'. Without reinforcements in the concrete, the crack widths W^+ and W^- will not be shown in SCIA.

The normal 'check' could still be performed. When checked on non-reinforced concrete elements, the value was 3 for all elements. As for every check value in SCIA, the maximum value is 1. The crack width test fails, which means reinforcements need to be added. This is exactly what was expected since non reinforcement concrete is unable to withstand the high loads.

Chapter 7: Conclusion

This paper shows that with little to no foreknowledge about designing bridges it is possible to make a check calculation off a bridge. It shows that with the necessary dedication it is possible to learn a lot about a subject and make a meaningful paper.

Designing the bridge as constructed in reality was impossible. Simplifications had to be made. All of the simplifications made in the project were acceptable and made sure the calculations were safer instead of more dangerous. This leads to slightly higher values, meaning that the bridge wasn't completely cost optimal if I had to be made according to the calculations inside this paper.

Apart from the calculations itself, this paper shows how all information gathered in the past 4 months have been funnelled into a final project. I have learned various new skills as well as improved other ones I already partly had. The way I could use SCIA Engineer has been drastically improved as well as easily determining critical sections from long pieces of text and much more. Calculation-wise I learned a lot about bridges.

In terms of the project itself the results were not always as easy to accept and interpret. In most cases, the results would tell to use an excessive amount of prestressing or a better concrete type. In reality this has not been done this way. The bridge is built with the information given. The persons behind the calculations are absolute experts and there is no doubt in my mind that they made no mistake. This lead to making the following conclusion. The calculations performed by me could not be completely right. This is due to a combination of different factors. First of all, some parts have been simplified. These simplifications lead to higher loads in order to be safe. To withstand these higher loads, better quality steel and concrete is needed. This could be a first explanation, but would be exaggerated. The simplifications in general don't lead to an extremely larger loads. A second reason could be found in the model itself. The models itself contains some more simplifications due to the lack of information. This made it so the model is not as real as in reality. No pretensioning could be applied, which of course leads to higher forces. The connections between the pillars and deck could also not be altered in order to be hinged supports instead of rigid ones. This has been taken into account while making manual calculations, but the one's made with SCIA could not be changed.

It has also been the first time I used the software SCIA Engineer to model 2D – elements instead of the usual 1D. This leads to much more new knowledge, but also more restrictions. In the end, the results were looked at with critical thinking and useful information could be obtained from the model.

Not all sections satisfied the regulation from the Eurocode. This is most likely because the results obtained from SCIA gave contradictory information. The way SCIA transfers loads from one 2D-element to another is most likely only through the nodes. This conclusion can be drawn, since the loads in the applying corners always seemed to be 10 times higher than the average loads in the structure. This leads to the question, which values should be used for calculations.

The calculations that had to be done for the bridge were no simple task. This was the first time in my study career at the university that bridge calculations had to be made. This was a challenge, but one I was willing to take. Before arriving in Porto my knowledge about bridges was quite limited. In the first semester I've had one course talking about the theoretic aspects of bridges, but nothing about designing or calculations. I have always had a passion for bridges. Most of all for the design. I have great respect for engineers finding new ways every time to make the bridge both unique and strong enough to withstand the acting forces. In this case the bridge looks so fragile, and tender, which leads to its amazing design. Lots of different aspects I learned when making this project will come in handy later on in my life.

Without a doubt there can be seen that the people behind the design and calculations of this bridge deserve a lot of credit. This bridge is true masterpiece of engineering. The new techniques applied and the will to create an original bridge led to the amazing bridge that is now known as Ponte do Infante.

This paper was a good way to summarize what I have learned for the past 4 years. The final results from the document are rather quite positive. Most of the calculations could have been performed and acceptable results were obtained. It shows that the design of a whole bridge is no easy task. What has been performed in this document is only a fraction of what needs to be done in reality. The skills achieved over the past years made it so this document could be completed with success.

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