UNIVERSITAT POLITÈCNICA DE VALÈNCIA

TRABAJO DE FIN DE GRADO

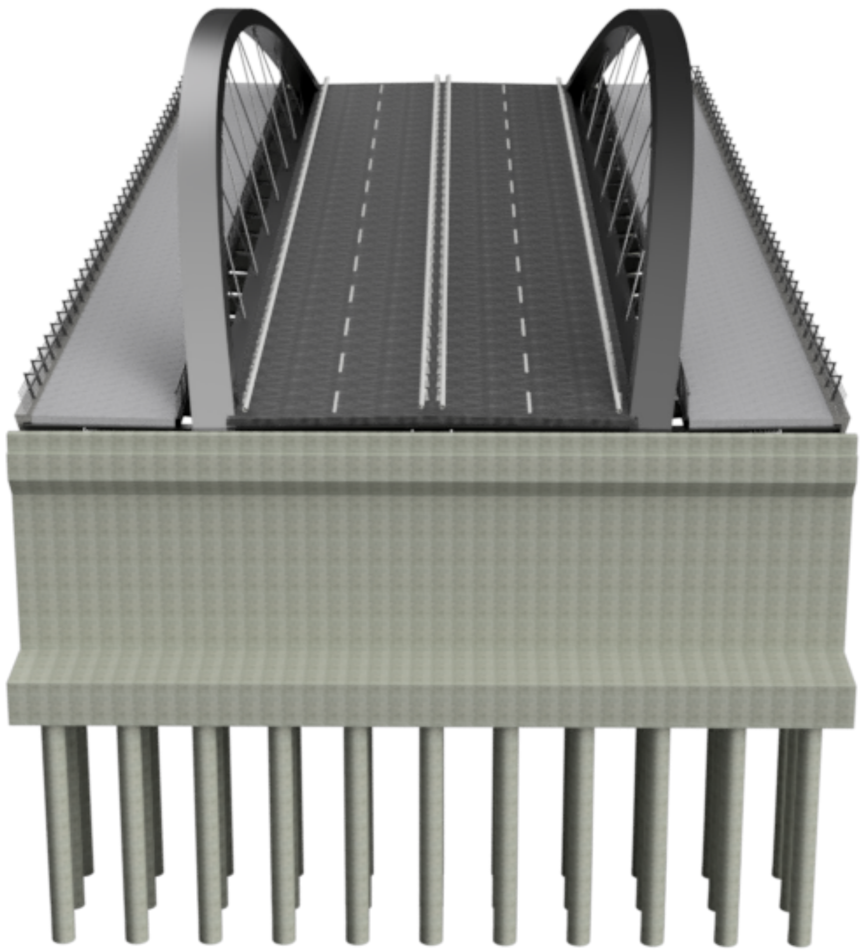
PROYECTO BÁSICO DEL PRIMER PUENTE DE LA ISLA DE ZORROTZAURRE (BILBAO)

DISEÑO CONCEPTUAL, CÁLCULO DE LA ESTRUCTURA, VALORACIÓN ECONÓMICA Y PROCESO CONSTRUCTIVO

BACHELOR'S THESIS

BASIC PROJECT OF THE FIRST BRIDGE OF ZORROTZAURRE ISLAND (BILBAO)

CONCEPTUAL DESIGN, CALCULATION OF THE STRUCTURE, ECONOMIC EVALUATION AND CONSTRUCTION PROCESS



DATE: JULY 2024

ACADEMIC YEAR 2024

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Acknowledgements

We would like to thank our tutors, Carlos Manuel Lázaro Fernández and Salvador Monleón Cremades, who were always available to help us and inspired enthusiasm for the topic beyond the project. Thank you for the time spent solving our problems and imparting so much knowledge.

Additionally, we want to thank our home university, Hochschule Mainz, and the Universitat Politècnica de València, as well as our parents, for enabling and supporting us throughout this exchange year.

Agradecimientos

Queremos dar las gracias a nuestros tutores, Carlos Manuel Lázaro Fernández y Salvador Monleón Cremades, que siempre estuvieron disponibles para ayudarnos e inspiraron entusiasmo por el tema más allá del proyecto. Gracias por el tiempo dedicado a resolver nuestros problemas e impartirnos tantos conocimientos.

Además, queremos dar las gracias a nuestra universidad de origen, Hochschule Mainz, y a la Universitat Politècnica de València, así como a nuestros padres, por permitirnos y apoyarnos a lo largo de este año de intercambio.

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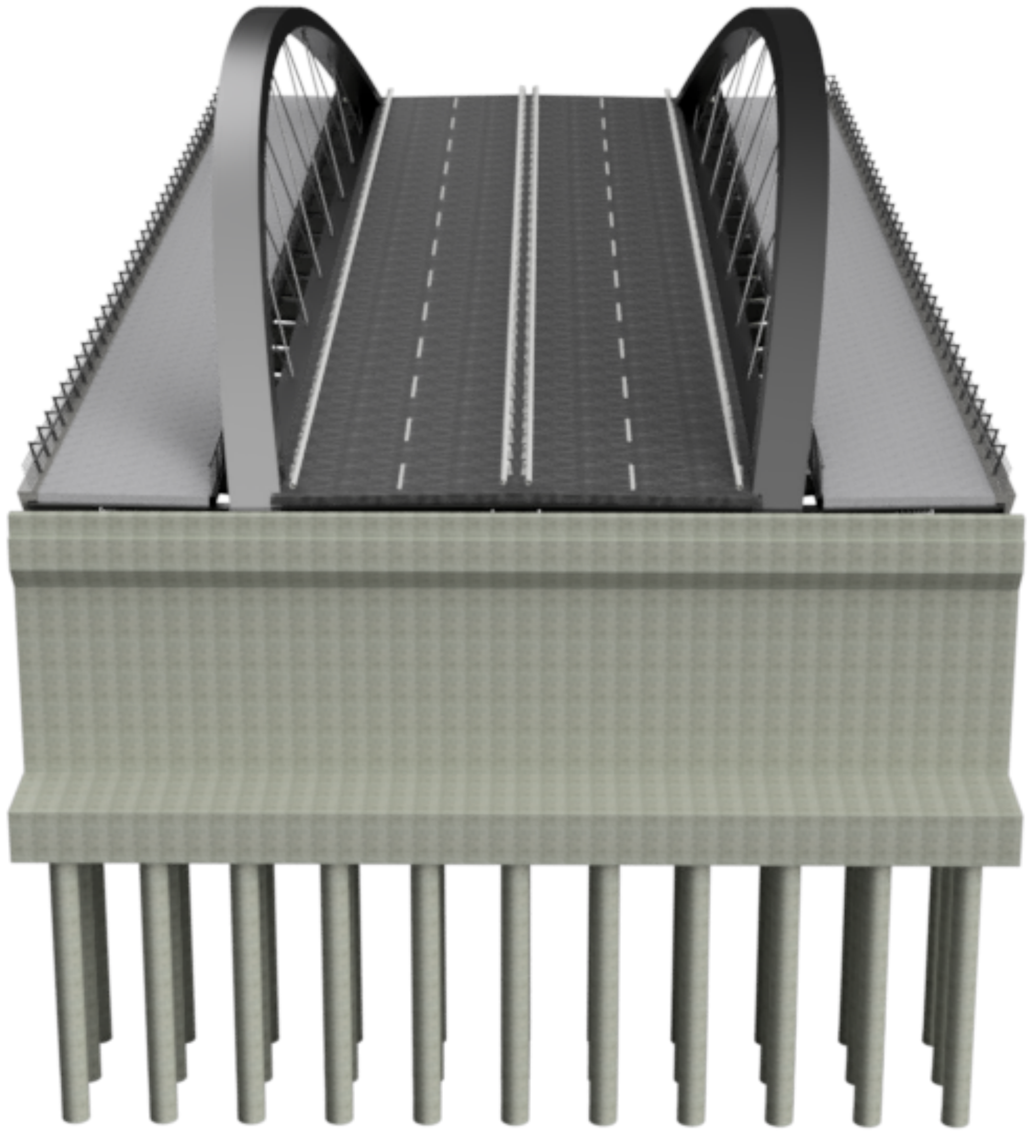

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Project Report

1. Purpose of the Basic Project

The aim of the following basic project is to develop an alternative proposal to the Frank Gehry Bridge, inaugurated in Bilbao in 2015, which serves as a link between the Deusto district and the island of Zorrotzaurre, crossing the Deusto Channel. The project will be developed according to the specifications of the tendering company Euskal Trenbide Sarea and the conditions established during the construction of the Frank Gehry Bridge.

2. Location

The project is located in Bilbao, the capital of the Spanish state País Vasco.

Figure 1: Location Bilbao [Source Apple Maps]

The bridge connects the Deusto district in the north of Bilbao with the island of Zorrotzaurre, which was created by the final opening of the Deusto Channel. The bridge crosses the Deusto Channel.

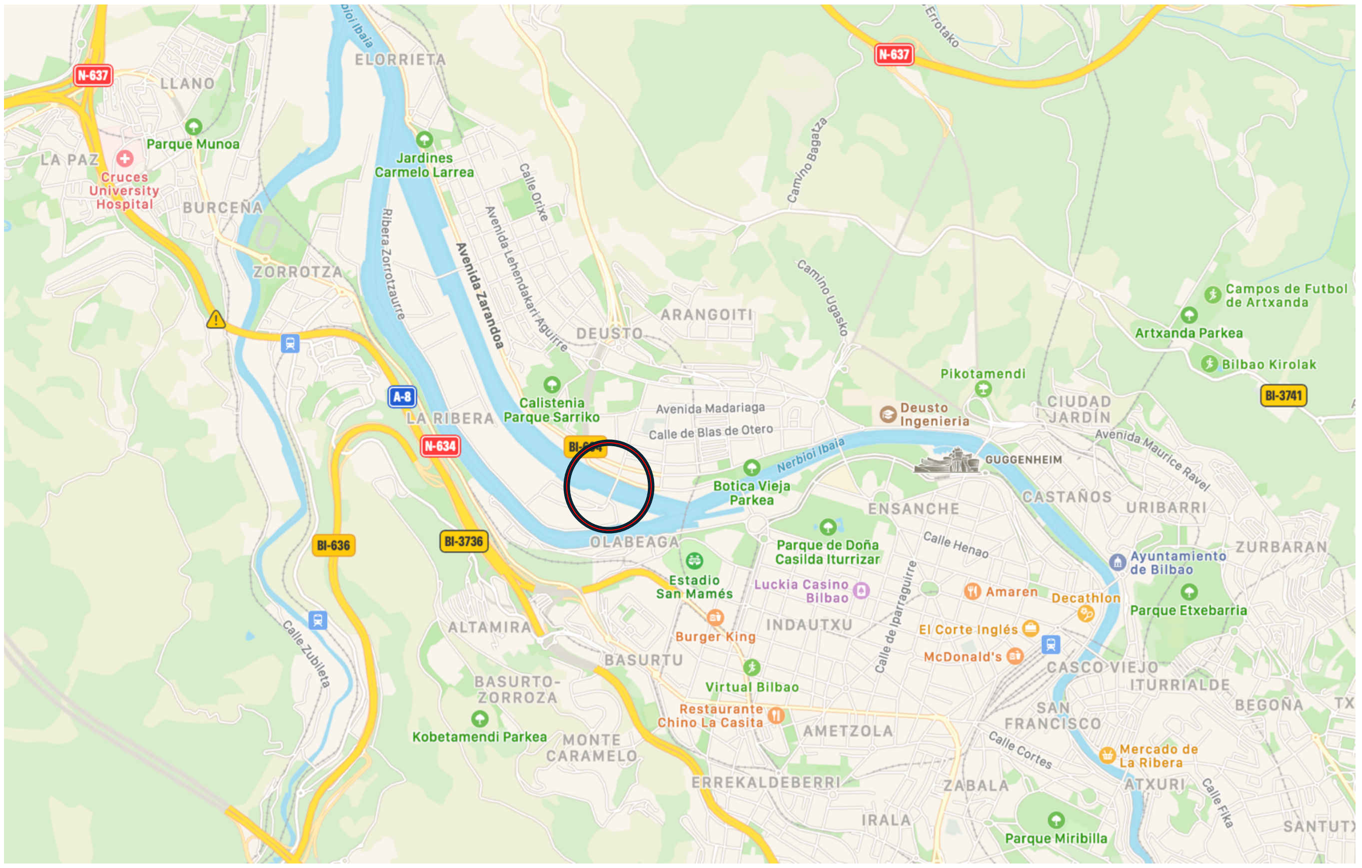

Figure 2: Location of the Bridge [Source Apple Maps]

3. Background History

The first ideas for the Deusto Channel were born in the 1930s. Due to the increasing use of the Port of Bilbao, the need was felt to make the river Nervión navigable further inland, which had previously proved difficult for larger vessels due to the Olabeaga and Elorrieta bends in the river's natural course. The channel would provide a more navigable alternative with a higher water level (an alternative that could be used by larger ships). Quays were to be built along the course of the channel. Three construction phases were completed in 1946, 1950 and 1951, during which all but the last 370 meters of the channel were built and put into operation. Over the years, due to increasing traffic and larger ships, an alternative port was sought and found in the Santurtzi area. As this established itself as Bilbao's new port, the channel and the surrounding neighborhoods became less and less important until the channel was finally closed as a port facility in 2006. In order to revitalize the island of Zorrotzaurre and make it an important part of Bilbao, a master plan designed by architect Zaha Hadid was presented in 2004. According to this plan, the island of Zorrotzaurre (at that time still a peninsula) would become a district with a mix of residential space (half of which would be social housing), commercial and office space, educational facilities, social services, and spaces to improve the quality of life (green spaces, promenades, avenues). The mobility of the island should be similar to that of the rest of the city, i.e., pedestrian and bicycle friendly, and connected to the public transportation network by a tram line that crosses the island.

The island will be connected to the surrounding neighborhoods by three bridges, one of which will be the bridge developed in the following project. It will be built in the area that is not yet open at the time of construction. The complete opening of the channel and the transformation of the Zorrotzaurre peninsula into an island is also part of the master plan.

4. Geotechnical Data

The geotechnical study is an investigation of the geotechnical conditions on which the bridge will be built. Geotechnical data is provided by the tendering party. This given data is completed and confirmed with more geotechnical data obtained over time on the island.

Due to the location of our bridge in the so-called Basque-Cantabrian Basin, which is a foreland of the Pyrenees, it can be assumed that the lower layers are calcareous siltstones from the Lower Cretaceous. The upper layers are silt and clay alluvium, since the area of the island of Zorrotzaurre is very close to the natural course of the river Nervión. Throughout the city of Bilbao, the upper soil layers are characterized by Quaternary deposits.

Since Zaha Hadid's master plan covers the entire island of Zorrotzaurre, several geotechnical studies have been carried out over the years. From these investigations, SPT's (Standard Penetration Tests) and a DPSH (Dynamic Probing Super Heavy) are used to confirm and complete the given data. Based on the given geotechnical profile and the complementary data, the assumed soil layers are confirmed and the depths of the individual layers are determined and analyzed more precisely. It can be seen that the siltstone layer begins at about 18,50 m - 19 m. The overlying silt layer is about 12 - 14 m thick. On the channel side of the Deusto district, there is a layer of clay about 3 meters thick between the upper layer, which is about 2,5 meters thick, and the silt layer. On the channel side of the island, the silt layer is directly adjacent to the top layer. Here the top layer has a thickness of about 4,5 m.

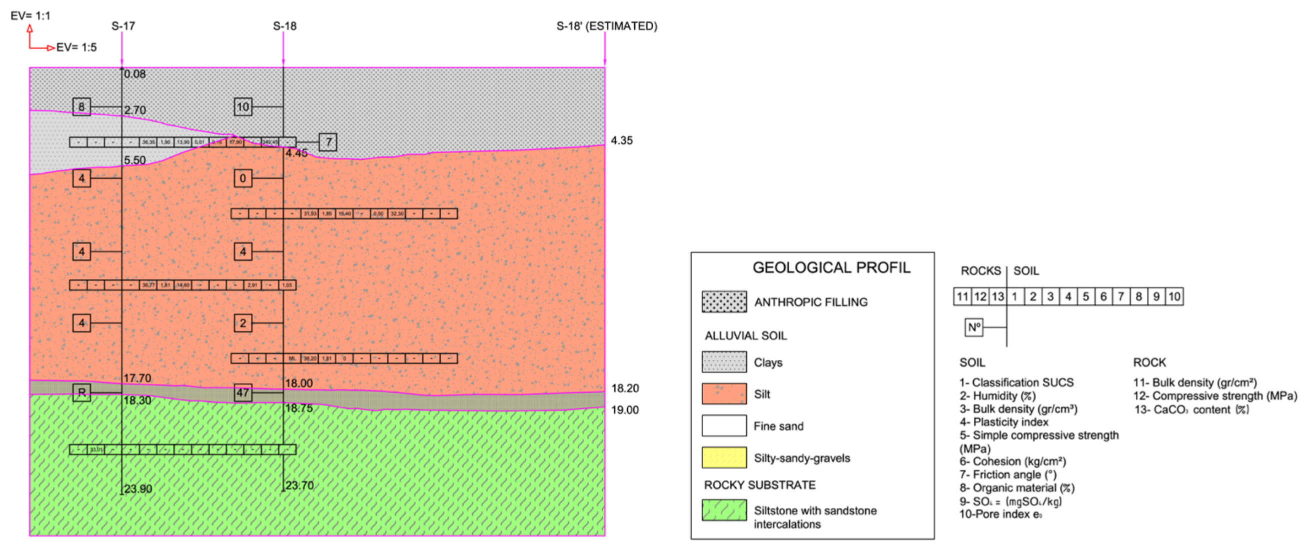

Figure 3: Ground profile provided by Euskal Trenbide Sarea

The most important soil properties such as density, compressive strength, cohesion and friction angle of the individual layers are also summarized in the appendix.

5. Hydrology

The hydrological appendix analyzes the watershed of the River Nervión, which feeds the channel. The size of the catchment area and the underlying river systems, climatology, land use and lithology are considered.

Then, the channel profile provided by the tendering company is considered, along with the required normal and flood levels and the necessary flood safety.

The channel has a width of 75 m. The central bottom of the channel is 4,60 m below sea level, at high water the water level is 4,45 m above sea level. In the central 40 meters there is a 1 meter high free zone above the water level, e.g. for floating debris during high water.

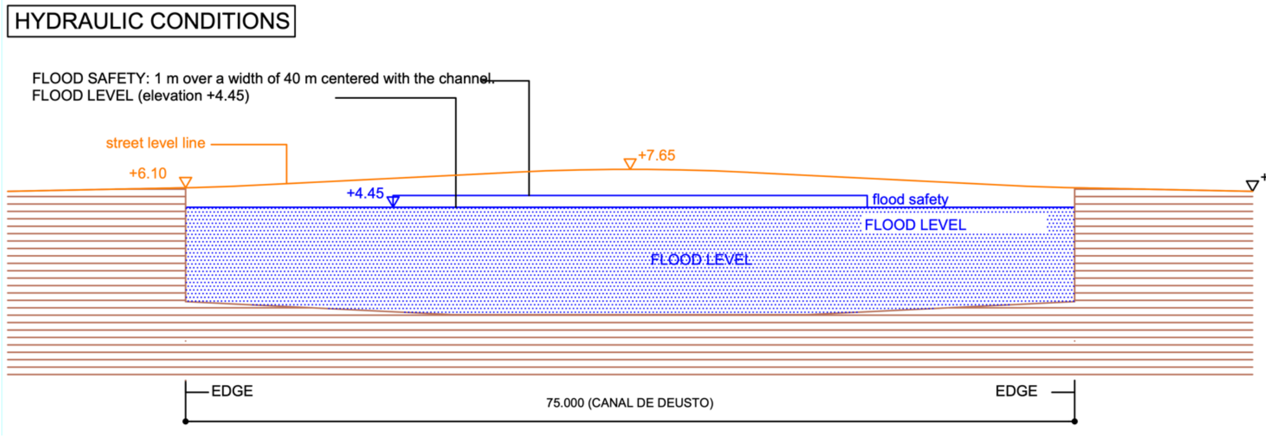

Figure 4: Cross-section of flood runoff Deusto Channel

6. Study of Alternatives

The purpose of this section is to summarize the various bridge design alternatives considered. The primary objective is to identify the most suitable bridge design that balances technical feasibility, economic viability, environmental integration, and aesthetic value. This analysis is crucial for ensuring the bridge fits the prerequisites and contributes positively to the urban development of Zorrotzaurre Island.

6.1. Prerequisites

Before analyzing the alternatives, several technical constraints were established:

- The bridge must be a single-span design due to restrictions on placing supports in the channel.
- The plan view requires a straight alignment connecting "Julio Urquijo" street in Deusto to Zorrotzaurre Island.
- The span width is approximately 76.5 meters with a total deck height limited to 1.40 meters to maintain the safeguard above flood levels.
- The cross-section must provide a usable width of at least 28 meters, including traffic lanes, shoulders, and sidewalks. (2 lanes of 3.5m in each direction with 1m shoulders to accommodate safety barriers and 6m usable sidewalks on both sides.)
- for transversal drainage, 2% slopes will be adopted

6.2. Alternative 1 - Bowstring Bridge

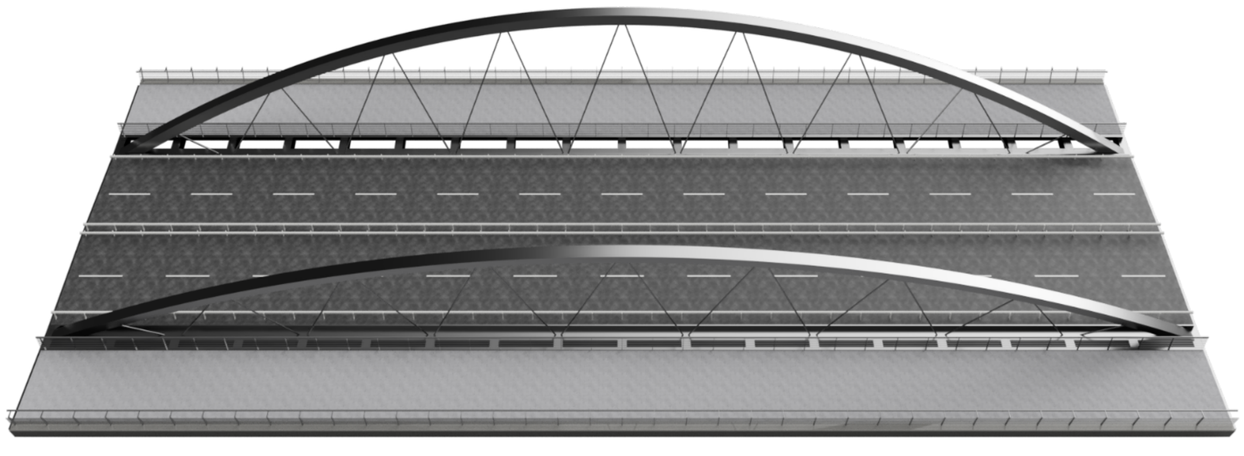

Figure 5: 3D Rendering of Alternative 1 [Own elaboration; Autodesk Fusion 360]

- Features two outwardly inclined steel arches.
- Compressive forces in the arches balanced by tensile forces in the deck.
- Aesthetic design with sidewalks offset outward, creating a gap between longitudinal beams and sidewalks, allowing pedestrians to look down into the channel.

6.3. Alternative 2 - Warren Truss Bridge

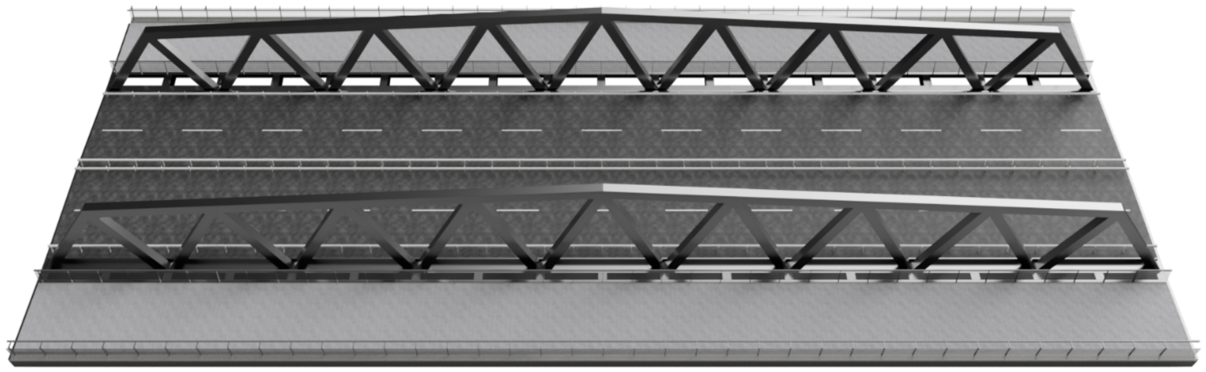

Figure 6: 3D Rendering of Alternative 2 [Own elaboration; Autodesk Fusion 360]

- Longitudinal beams with integrated trusses separating the roadway from pedestrian areas.
- Efficient load distribution but less visually appealing.

6.4. Alternative 3 - Single Tower Cable-stayed Bridge

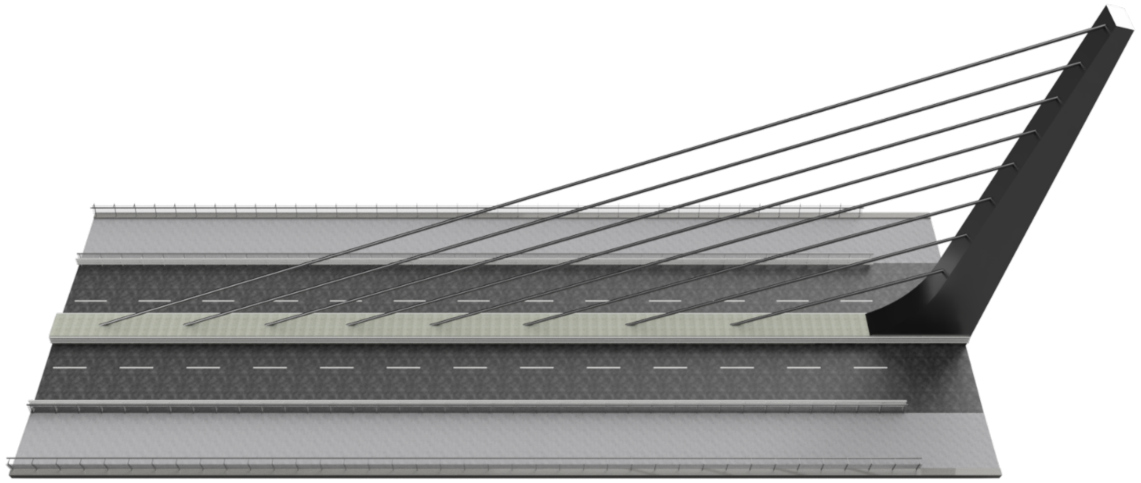

Figure 7: 3D Rendering of Alternative 3 [Own elaboration; Autodesk Fusion 360]

- Central tower with parallel cables supporting the deck.
- Prestressed concrete box girder for torsional resistance.
- Visually striking but complex and costly to construct.

6.5. Alternative 4 - Four Tower Cable-stayed Bridge

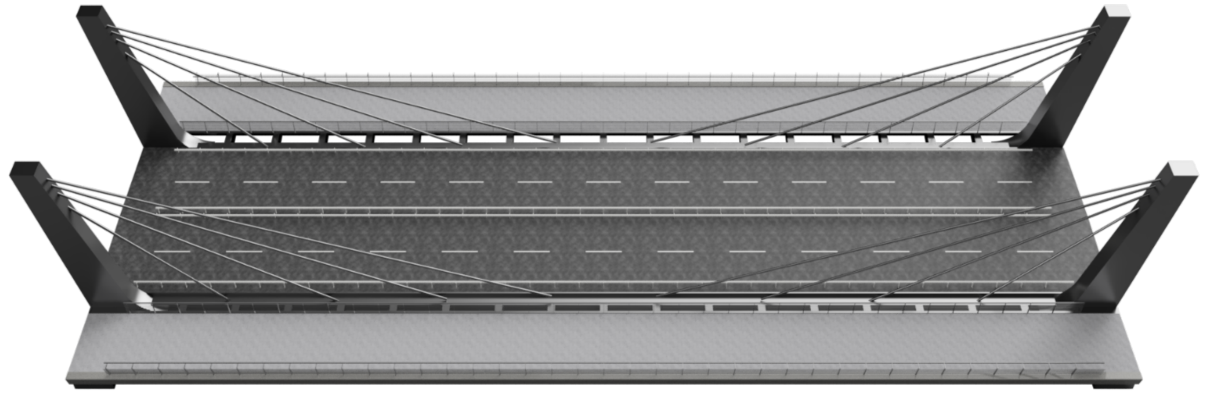

Figure 8: 3D Rendering of Alternative 4 [Own elaboration; Autodesk Fusion 360]

- Two inclined towers with cables connected to half the deck on either side.
- Offers torsional rigidity and simpler construction than a single tower design.

6.6. Evaluation and Selection

The proposed bridge design alternatives were evaluated based on several criteria: cost-effectiveness (50%), aesthetic appeal (25%), functionality (15%), and construction time and process (10%).

The Bowstring Arch Bridge (Alternative 1) scored the highest overall, effectively balancing these factors. It was chosen for its cost efficiency, visual appeal, and functional performance, along with a straightforward construction process. The arch bridge's design offers a harmonious blend of practicality and aesthetics, making it an optimal choice for contributing positively to the urban landscape of the Deusto channel area.

7. Calculation of the Structure

In the previous section and detailed in the appendix "Study of Alternatives," the bridge typology for the thesis was determined. The purpose of the appendix "Calculation of the Structure" is to verify the viability of this design.

A local model of a crossbeam and a global model of the entire metal load-bearing structure were created in the structural analysis software SAP2000. Permanent loads, including the self-weight of the supporting structure (directly considered in SAP) and dead loads, as well as variable loads such as traffic, pedestrian, wind, and temperature effects, were determined.

The arches were analyzed for buckling, revealing that the initial design, as presented in the appendix "Study of Alternatives," did not provide the necessary stability. Consequently, the cross-section of the arches was widened horizontally until sufficient stability was achieved. All load-bearing elements were then checked for axial forces, shear forces, bending moments, and their interaction, ensuring they do not exceed design resistances according to the ultimate limit state. For the serviceability limit state, vertical deflections were verified to be within allowable limits.

Additionally, the resulting vertical forces of the supports were used to select the bearing pads, and the maximum horizontal deformations were used to select the expansion joints. The following is a 3D view of the final bridge, which differs from the version shown in alternative 1 primarily due to the more massive arches:

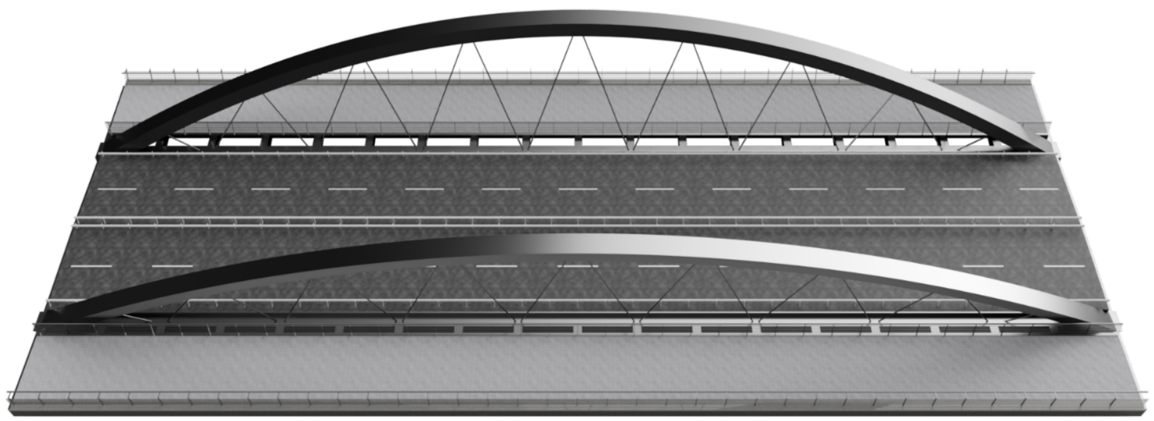

Figure 9: 3D Rendering of the Final Design [Own elaboration; Autodesk Fusion 360]

The deck consists of a system of transverse floor beams and cantilevered side girders, supporting a 26 cm thick concrete slab, which collaborates with the steel structure for load resistance. This transverse system transfers the loads to the longitudinal beams and then part of the loads are transferred through the suspension cables to the arches, which primarily handle compressive forces.

8. Calculation of Foundations

In the appendix to the foundation design, the foundations required to transfer the forces determined in the previous appendix to the superstructure design to the subsoil are dimensioned and verified.

The bridge is supported by a bridge retaining wall that functions on one side as a channel wall and on the other side as an earth retaining wall. This retaining wall is designed without further bearing capacity checks.

Since the load bearing stone layer (as determined in the geotechnical appendix) is relatively deep, bored piles are required to stabilize the bridge retaining wall and transfer forces to the load bearing soil layer. Three rows of 11 bored piles each are installed at the bottom of the bridge retaining wall. These piles are 11 meters long and 1 meter in diameter. They are tested for their load bearing capacity.

9. Bridge Fittings

In this appendix, bridge components are developed that do not serve a structural purpose but are essential to the usability of the bridge. The following bridge components are listed in the appendix:

- Pipelines: According to the tender, pipelines must also be routed across the river in the bridge structure.

- Drainage: Due to the cross slope of the roadway and sidewalk and the fact that the bridge crosses the channel, no special drainage measures are required.

- Lane distribution: The tender requires two traffic lanes, each with two 3-meter wide traffic lanes and two 6-meter wide pedestrian walkways. There must also be space for the safety barriers.

- Guardrails: Guardrails are required to secure the sidewalks, using custom-designed guardrails. The outer guardrails are different from the inner guardrails.

- Cornice: Custom-designed model for the cornice, that follows the inclination of the guardrails and arch.

- Crash barriers: Custom-designed models are also used for crash barriers.

- Pavements: The roadways are covered with asphalt, the pedestrian paths are covered with paving stones.

- Lighting: The roadway is illuminated by centrally positioned lanterns, while the pedestrian paths have lighting modules embedded in the handrails.

10. Construction Process and Work Schedule

The appendix "Construction Process and Work Schedule " details the work schedule for the construction of the bridge over the Deusto channel in Bilbao, breaking down the construction process into distinct phases and outlining the timeline for each activity.

10.1. Construction Process

- **Phase 1. Stakeout and Excavation of Work Area:** Initial Setup and preparation.
- **Phase 2. Sheet Pile Retaining Walls and Excavation**: Driving of sheet pile retaining walls and further excavation of their enclosures for foundation preparation.
- **Phase 3. Foundations and Abutments:** Construction of bored piles, pile caps, and abutments, including waterproofing and backfilling.
- **Phase 4. Manufacturing and Transport of Steel Structure:** Production of steel components in a workshop, segmented for easy transportation to the site.
- **Phase 5. Temporary Support Structure:** Installation of temporary supports to stabilize the structure during assembly.
- **Phase 6. Assembly of the Metal Structure:** On-site assembly of longitudinal and transverse beams, arches, and tension rods.
- **Phase 7. Concrete Slab and Related Tasks:** Installation of formwork, reinforcement, and pouring of the concrete slab.
- **Phase 8. Bridge Fittings, Paving, and Signage:** Final installation of fittings, paving of roadway and sidewalks, and application of traffic signs and road markings.

10.2. Work Schedule

The work schedule, presented as a Gantt chart, outlines the sequence and duration of the main activities, totaling an **estimated 48 weeks for project completion**.

Each activity's duration was calculated based on average performance per unit, quantities derived from project drawings, dependencies between activities, and a standard workweek of five 8-hour days.

11. Sustainable Development Goals

In the Appendix to the Sustainable Development Objectives, the project is linked to the Sustainable Development Goals established by the United Nations. These are 17 goals of the United Nations that aim to achieve economic growth, reduce inequalities and injustices in living standards, create equal opportunities, ensure the sustainable use and management of natural resources, and preserve ecosystems. In this context, the items: 3. good health and well-being; 8. decent work and economic growth; 9. industry, innovation and infrastructure; 10. reduced inequalities; 11. sustainable cities and communities; and 13. climate change action, will be looked at as the development of the project contributes to these goals.

12. Economic Evaluation

The economic evaluation provides a detailed analysis of the project costs, justifying the projected budget by outlining the necessary tasks, grouped into 12 main chapters, and their associated quantities and unit prices.

The unit prices include all direct costs such as materials, labor, equipment, and transportation. Overhead is considered at 13%, and the profit margin is 6%. VAT in Spain is applied at 21%.

The budget summary aggregates the total direct costs from all chapters, totaling €5,528,918.81. Adding overhead and profit margin, the base bidding budget excluding VAT is €6,579,413.39. Including VAT, the **total budget is €7,961,090.20**, with a **per square meter cost of €2,973.33**.

Valencia the 3 July 2024, signed by the authors:

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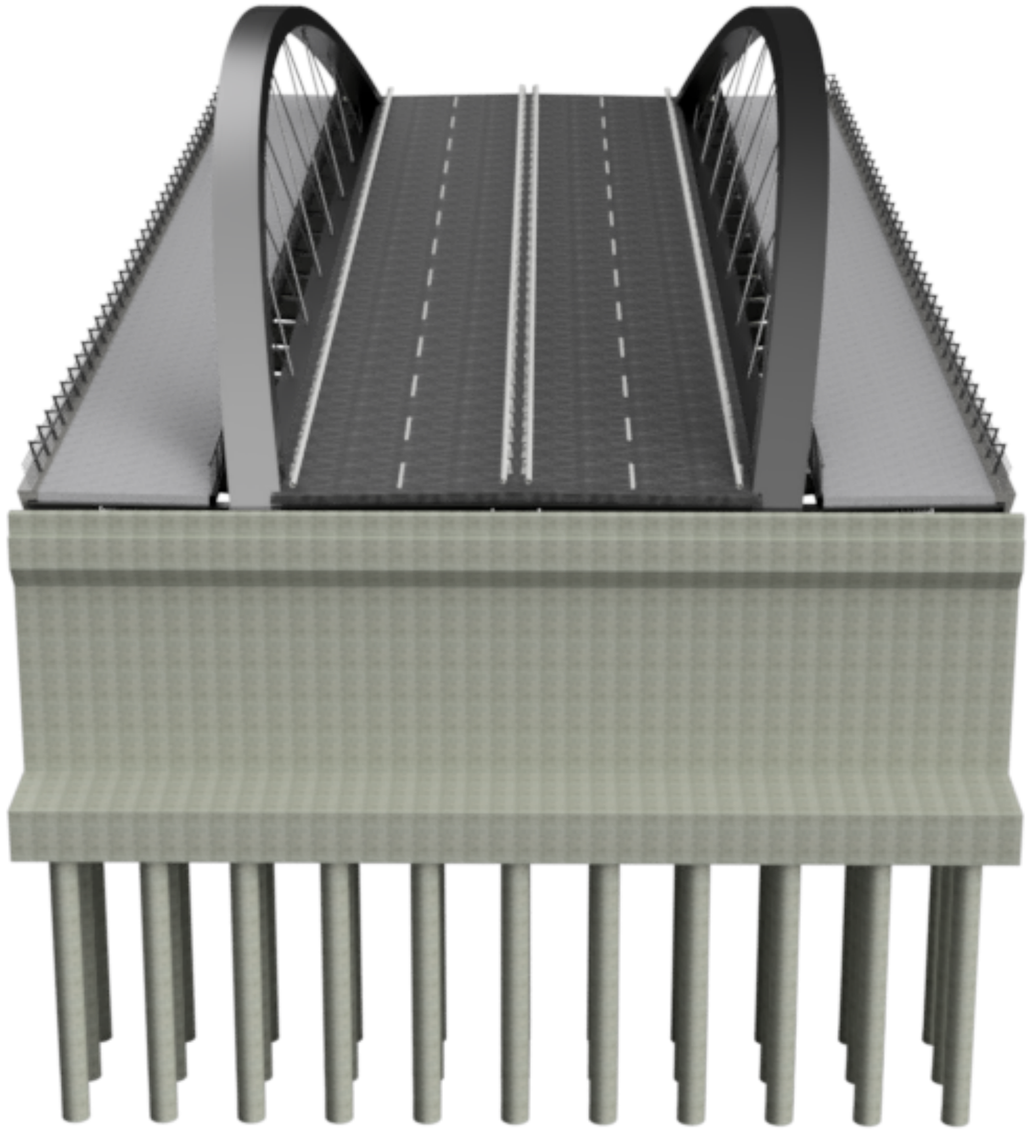
Appendix 1

Geotechnical Data



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DATE: JULY 2024

ACADEMIC YEAR 2024

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Geotechnical Data

1. Introduction

The following geotechnical appendix describes the geological structure of the ground over which our bridge runs and on which the foundations for our bridge are being built. Furthermore, the geotechnical properties of this ground are analyzed, which are necessary for the calculation of the foundations.

2. Underlying Information

The tendering company "Euskal Trenbide Sarea" provided the applicants with geological profiles. Since the entire island of Zorrotzaurre is part of Zaha Hadid's master plan, various geotechnical investigations were carried out over a long period of time in the entire area of the island. The data provided by Euskal Trenbide Sarea are part of these geotechnical investigations. In detail, these are the 17^{th} to 20^{th} borehole of a total of 20 boreholes carried out by the company IDOM, 2007, on the basis of which the geological profile provided was determined.

Boreholes 17 and 18 are already very close to the planned location of the bridge. Borehole 17 represents a good image of the soil structure at bearing level on the "mainland side". Borehole 18 is located more in the middle of the channel, so the given geotechnical image provides an estimated course of the soil structure towards the island side. Further geotechnical investigations of the island of Zorrotzaurre will be analyzed in order to verify this estimate of the soil structure on the island side.

3. Geological Conditions of the Surrounding Area

The area of the city of Bilbao and therefore also the island of Zorrotzaurre is located in the so-called Basque-Cantabrian Basin, which is part of the Basque-Cantabrian chain, a western extension of the Pyrenees chain. In this area, the subsoil can be traced back to the Lower Cretaceous.

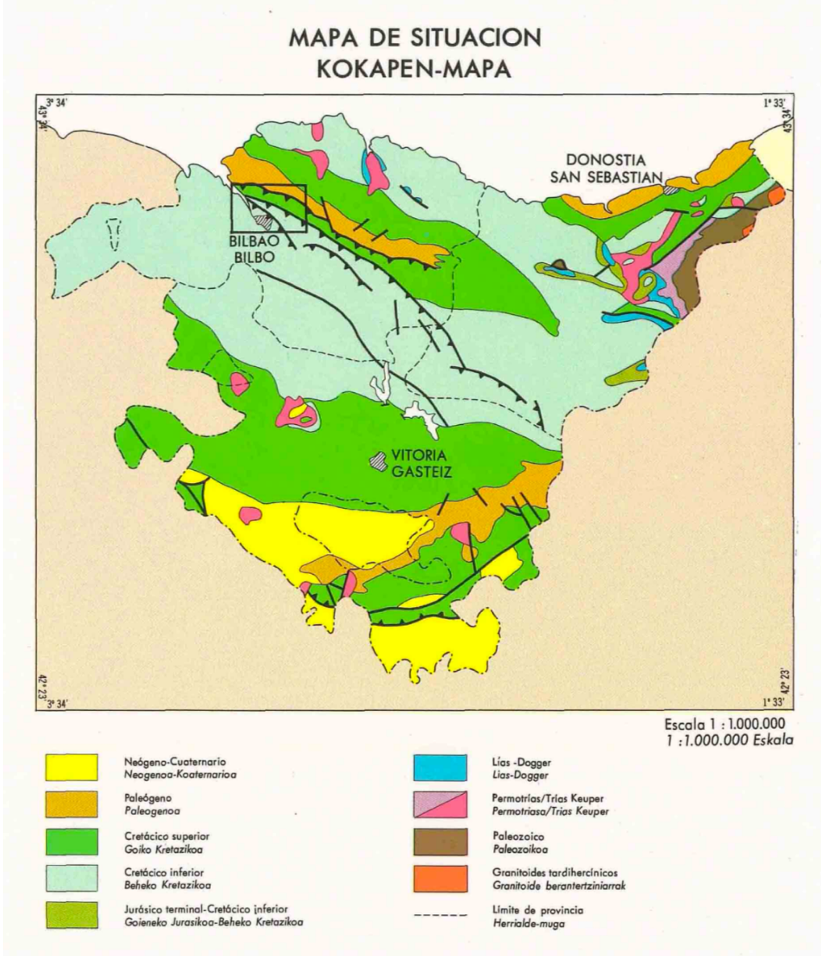

Figure 1: Map of the geological origin of the deep subsoil [source EVE]

Calcareous silts are typical for this time, which is also recognizable in large parts of the Basque Country and in the catchment area of the Nervión river system (see Appendix 2: Hydrology; 3.3 Landcover and Lithology).

However, as the island of Zorrotzaurre is also located in close proximity to the natural course of the Nervión river, various alluvial deposits have settled on top of the Cretaceous rocks, resulting in silty and clayey layers. It should also be noted that most of the urban area of the city of Bilbao is characterized by anthropogenic adaptations due to its urbanization, which means that the upper soil layers in the area of the island of Zorrotzaurre are characterized by the current Quaternary period.

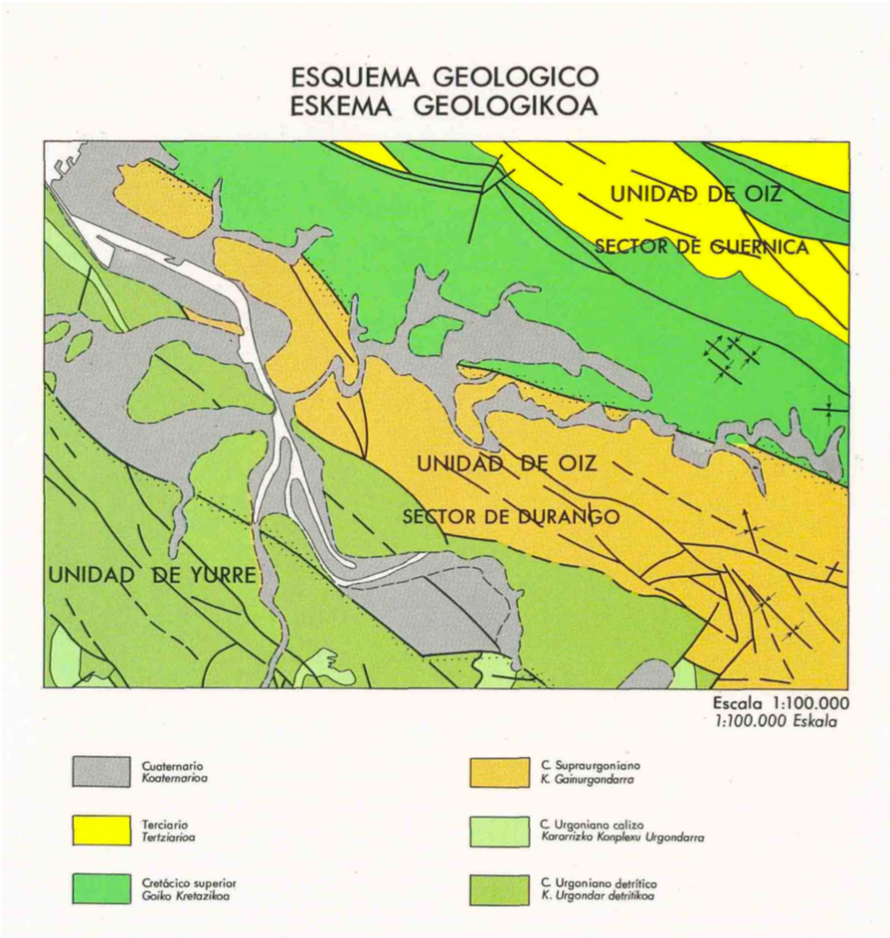

Figure 2: Map of the geological origin of the upper layer of the subsoil [source EVE]

4. Seismic Hazard

An assessment of the seismic hazard is required as part of the geotechnical analysis. The "Norma de Construcción Sismorresistente: Puentes (NCSP-07)" should be adhered to for structures in Spain.

This norm provides us with a map of seismic activity in Spain:

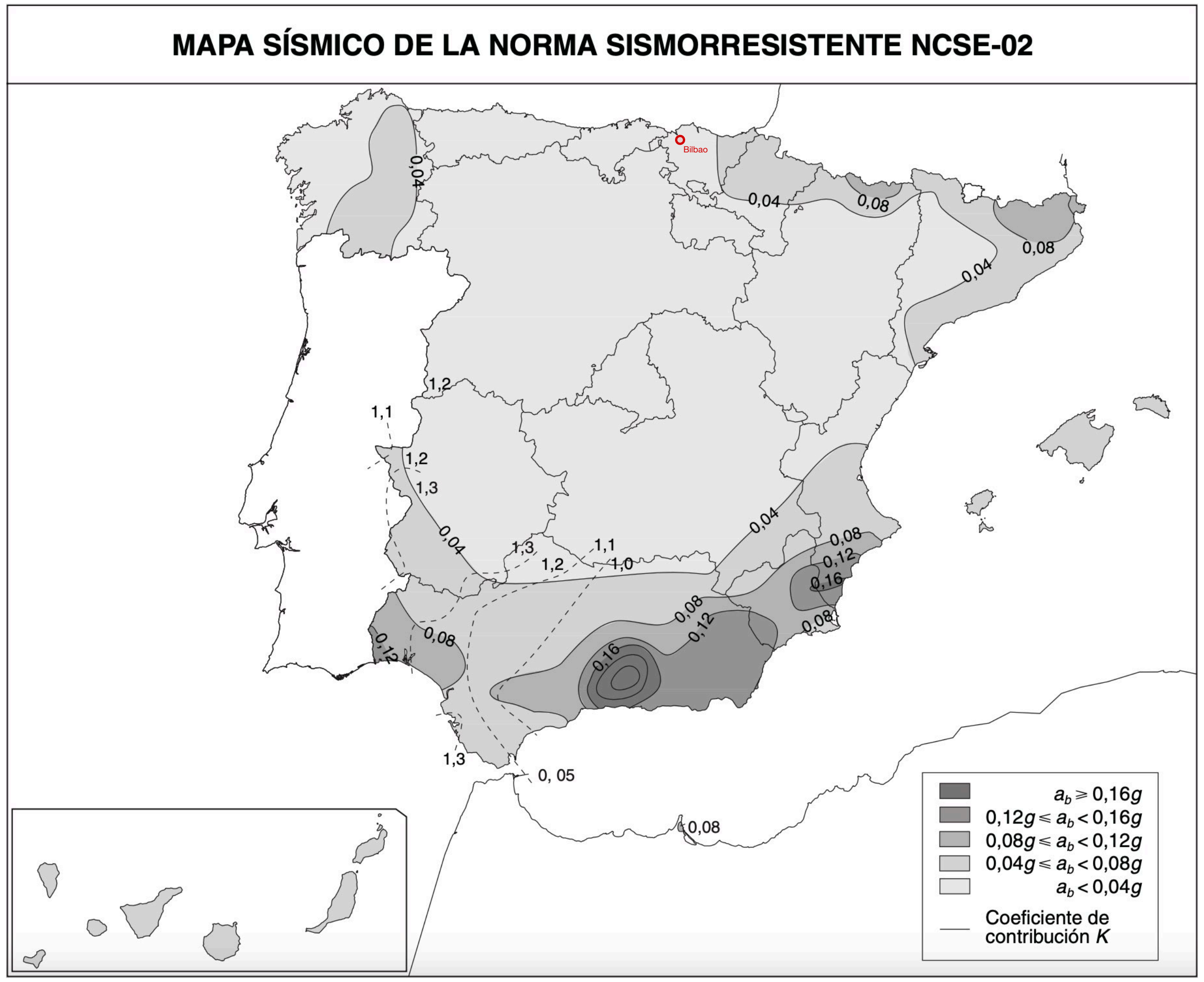

Figure 3: Map of seismic activity in Spain

As can be seen from the map, Bilbao is located in the area of the lowest seismic hazard and has a horizontal basic seismic acceleration value of 0.04 g (g = gravitational acceleration). According to chapter 2.8 of the NCSP-07, it is not necessary to consider seismic acceleration values < 0.04 g in further calculations.

5. Examinations Carried Out

The series of tests and work carried out, from which the geotechnical data used were derived, are described below.

5.1 On-Site Examinations

First of all, we look at the field operations that form the basis of our geotechnical data. As mentioned in the section "Underlying information", the geotechnical data provided by Euskal Trenbide Sarea is based on geotechnical investigations carried out by IDOM in 2007. Boreholes 17-20, on which the geotechnical profiles are based, are SPT's with the following depths:

- S-17: 23,90 m

- S-18: 23,70 m

- S-19: 22,70 m

- S-20: 25,20 m

In order to complement the information, we include tests carried out by Euroconsult norte in 2013. For this purpose, we consider an SPT (SZ-1) located near our site, which forms part of a series of SPTs on the island of Zorrotzaurre. Furthermore, a DPSH (PZ-2), which is part of a series of DPSH's that are also part of the overall study series, can be used as a complement. The tests used by Euroconsult led to the following depths:

- SZ-1: 30,30 m

- PZ-2: 18,80 m

The following picture shows the locations of the individual boreholes:

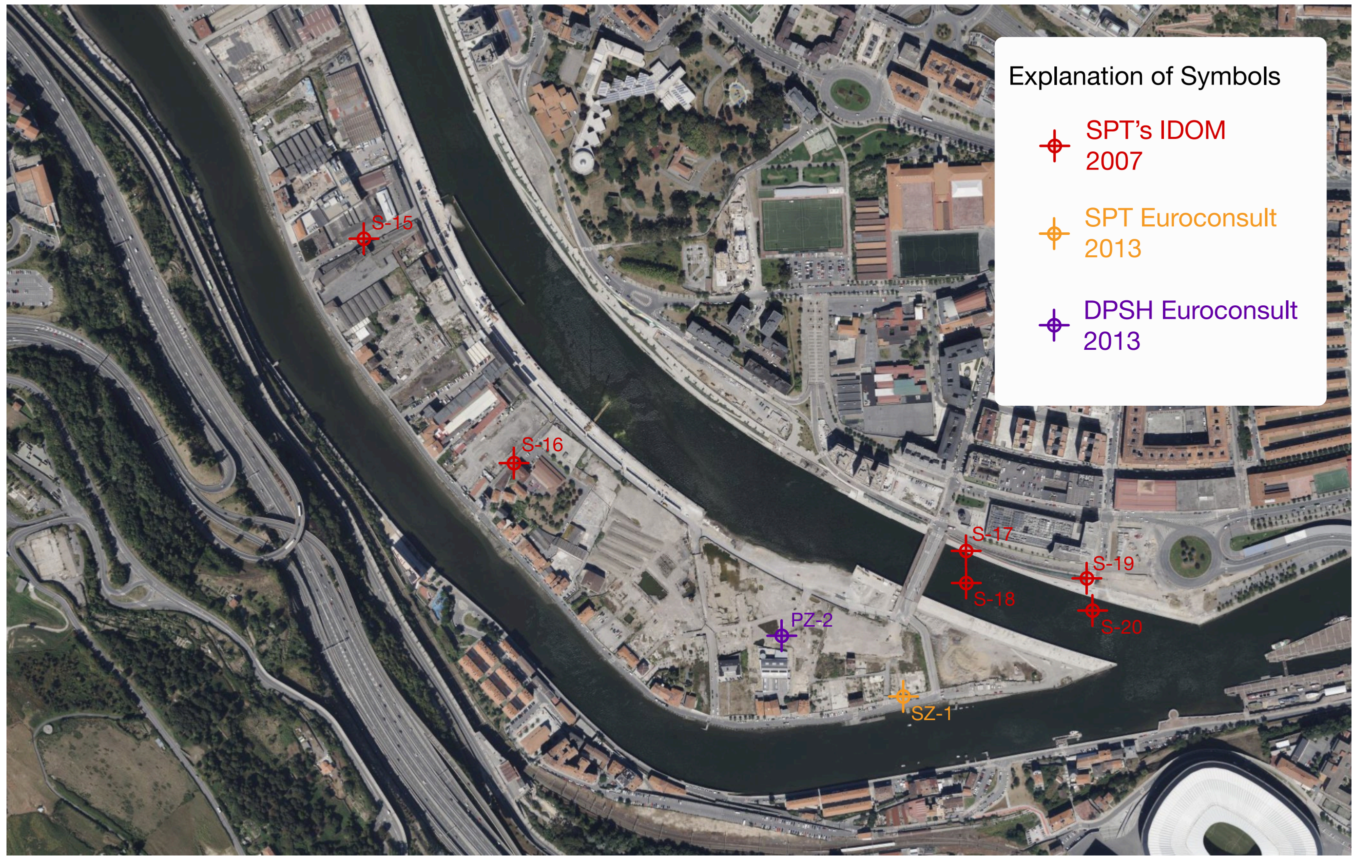

Figure 4: Map of Zorrotzaurre; locations of boreholes used [source map: Apple Maps]

5.2 Laboratory Tests

The geotechnical data supplied also includes the soil properties determined in laboratory tests based on the soil samples taken with the SPTs. As not all parameters were determined from the soil samples taken at the site of our bridge, these could be approximated with the results of the SPT's S-15 and S-16 from IDOM, which were also provided. For our purposes, however, the results from the boreholes near our bridge (S-17 to S-20) are completely sufficient.

6. Results

In this section, the results are analyzed and compared with the additionally analyzed boreholes (SZ-1 & PZ-2). Furthermore, the results determined by the laboratory tests are analyzed.

6.1 Geotechnical Profile

The given geotechnical profiles at the location of soundings S-17 and S-18 and the estimation in the direction of the island-side support (S-18') are shown in the following plan:

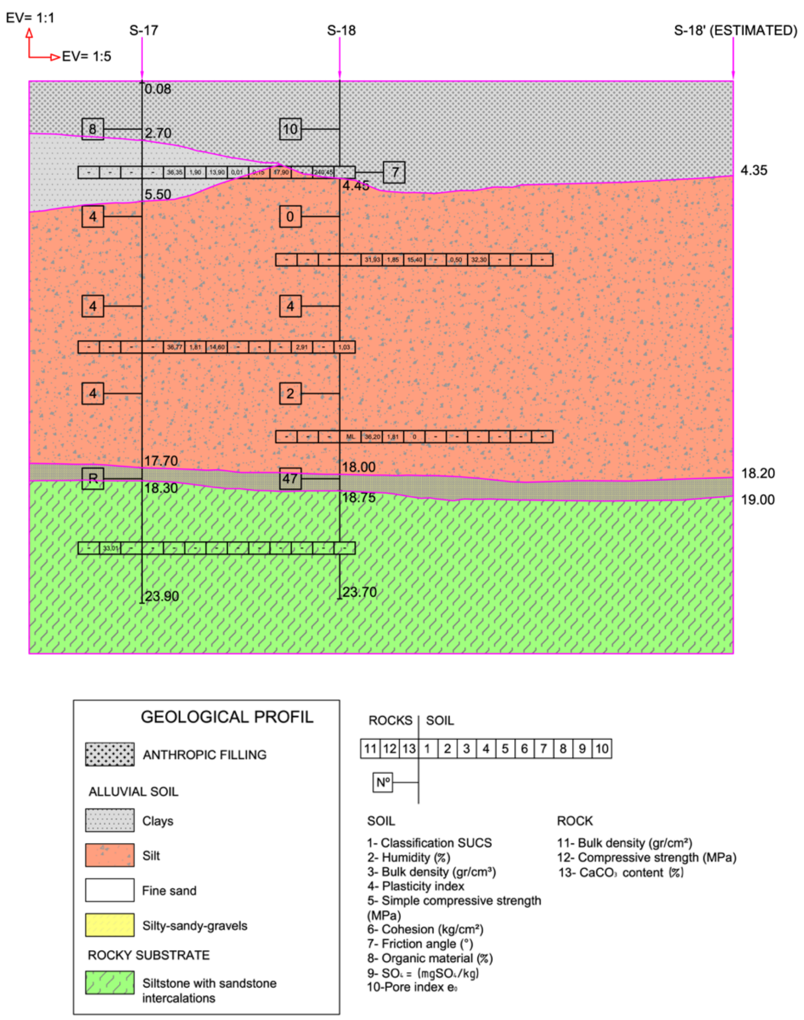

Figure 5: Geological soil structure given by Euskal Trenbide Sarea

The following information can be found in the description of the exploratory information sheet and the plan:

- S-17:
  - 0 - 2,70 m: Backfill => gravel with medium to high sand content
  - 2,70 - 5,50 m: Clay with a low sand content
  - 5,50 - 17,70 m: Silt with a low sand content
  - 17,70 - 18,30 m: Gravel with a light clay and sand content
  - 18,30 - 23,90 m: Rock (Siltstone)
- S-18:
  - 0 - 4,45 m: Backfill => gravel with medium to high clay and sand content
  - 4,45 – 18 m: Silt with a low sand content
  - 18 - 18,75 m: Gravel with a light clay and sand content
  - 18,75 - 23,70 m: Rock (Siltstone)
- S-18':
  - 0 - 4,35 m: Backfilling => gravel
  - 4,35 - 18,15 m: Silt
  - 18,15 – 19 m: Gravel
  - 19 - 26,20 m: Rock (Siltstone)

Furthermore, the soil structure determined based on the additional borehole (SPT SZ-1) is as follows:

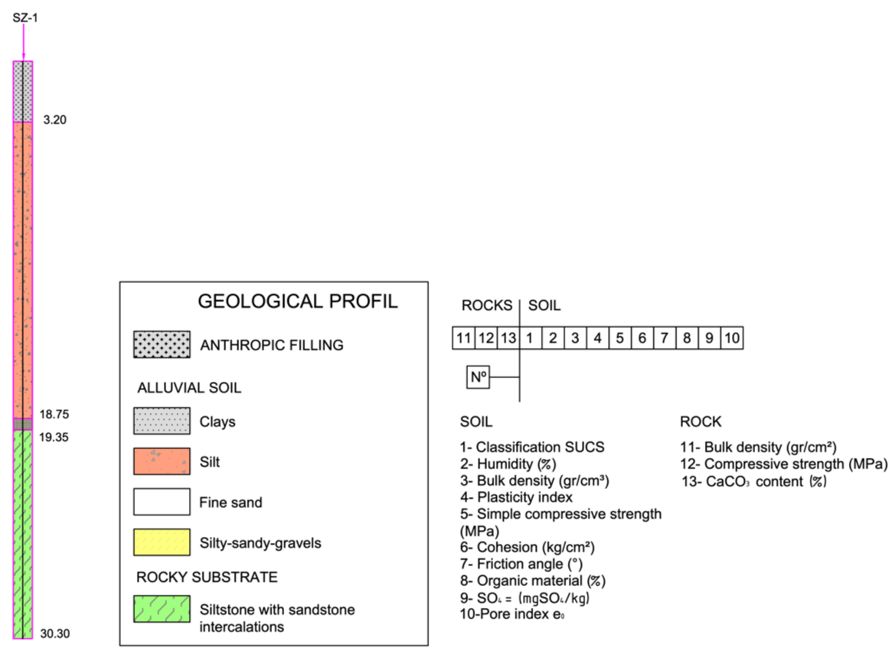

Figure 6: Geological soil structure SPT SZ-1, Euroconsult Norte

- 0 - 3,20 m: Backfill => gravel with high sand content
- 3,20 - 18,75 m: Silt with low to medium sand content
- 18,75 - 19,35 m: Gravel
- 19,35 - 30,30 m: Rock (Siltstone)

We can also compare the number of blows required in the given SPT's (S-17 and S-18) with the supplementary SPT (SZ-1) and DPSH (PZ-2).

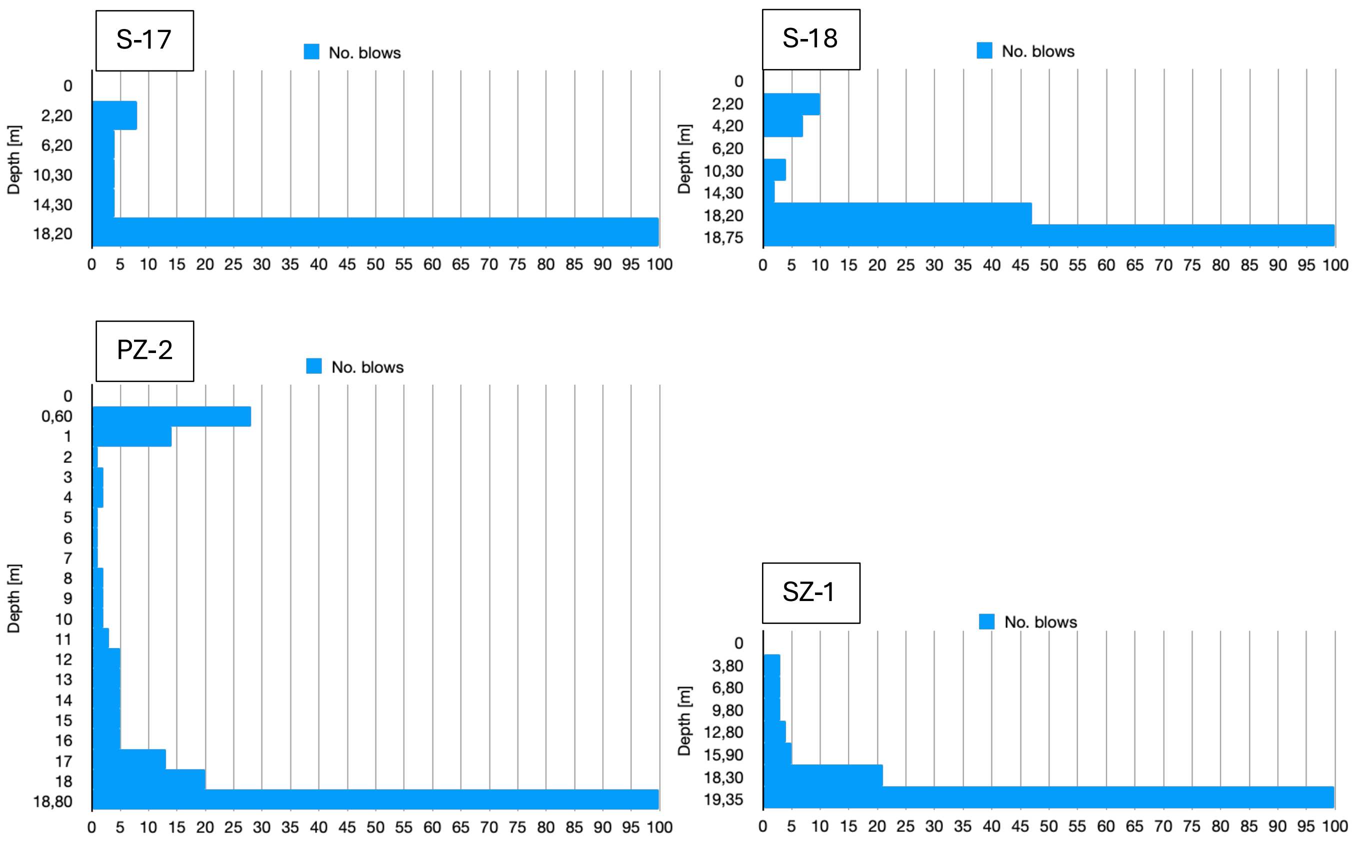

Figure 7: Representations of number of blows (x-axis) to depth (y-axis) of the investigated soundings

When comparing the graphs of the boreholes, it can be seen that they have a similar course. The upper gravel layer is recognizable here, as a higher blow count can be seen in the upper layer than in the subsequent clay layer, where a low blow count is necessary to penetrate the layer (from approx. 4.50 m) in all soundings. From approx. 18 meters, the gravel layer above the rock layer is then recognizable due to the increased number of blows. From 18.20 to 19.35 meters, the stone layer can then be seen in all boreholes.

When comparing the DPSH with the other SPTs, it should be noted that it is a different form of penetration borehole, that penetrates in smaller steps. It therefore reacts more sensitively in certain cases (e.g. with rock-like soil structures), which results in a higher number of blows in the layer. This explains the higher number of blows required to penetrate the upper gravel layer.

A comparison of the existing soil structure with the additional boreholes shows that the soil structure in the surrounding area is similar to the soil structure at the location of boreholes S-17 and S-18. The rock layer can be found at approx. 19 meters of both additional soundings. Furthermore, SZ-1 shows that the upper gravel layer is getting smaller, which can also be recognized by the given course of S-18 in the direction of S-18'. From these additional boreholes and the comparisons made, we have ensured that we can assume the soil structure at S-18' for further calculations.

6.2 Soil Properties

The soil properties are also given in the given soil profiles. The following properties are of further importance to us:

- γ(g/cm^3): Density
- q_u(MPa): Compressive strength
- c_u(kg/cm^2): Cohesion
- ϕ(°): Friction angle

Or the value of the compressive strength q_u in case of the rock layer.

We take the following values for the different layers:

Layer 2 (S-17; 2.70m-5.50m), clay:

γ = 1,9 g/cm^3

q_u = 0,01 MPa

c_u = 0,15 kg/cm^2

ϕ = 17,90°

Layer 3, silt:

γ = 1,81 g/cm^3

q_u = 0,1 MPa

c_u = 0,5 kg/cm^2

ϕ = 32,3°

Layer 5, rock, siltstone:

q_u = 33,01 MPa

No soil properties are given for layers 1 and 4 (gravel layers). However, based on the descriptions of the boreholes, layer 1 can be classified as a gravel-sand mixture (GP) and layer 4 as a gravel-sand-clay mixture (GC).

COARSE-GRAINED SOILS
(more than 50% of material is larger than No. 200 sieve size.)

	Clean Gravels (Less than 5% fines)	
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
	Clean Sands (Less than 5% fines)	
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures

Figure 8: Classification of soils

These classifications can be used to estimate rough values from table books.

Layer 1, gravel-sand mixture:

γ = 2,1-2,3 g/cm^3

ϕ = 35°-45°

Layer 4, gravel-sand-clay mixture:

γ = 2,1-2,4 g/cm^3

c_u = 0-0,07 kg/cm^2

ϕ = 35°-43°

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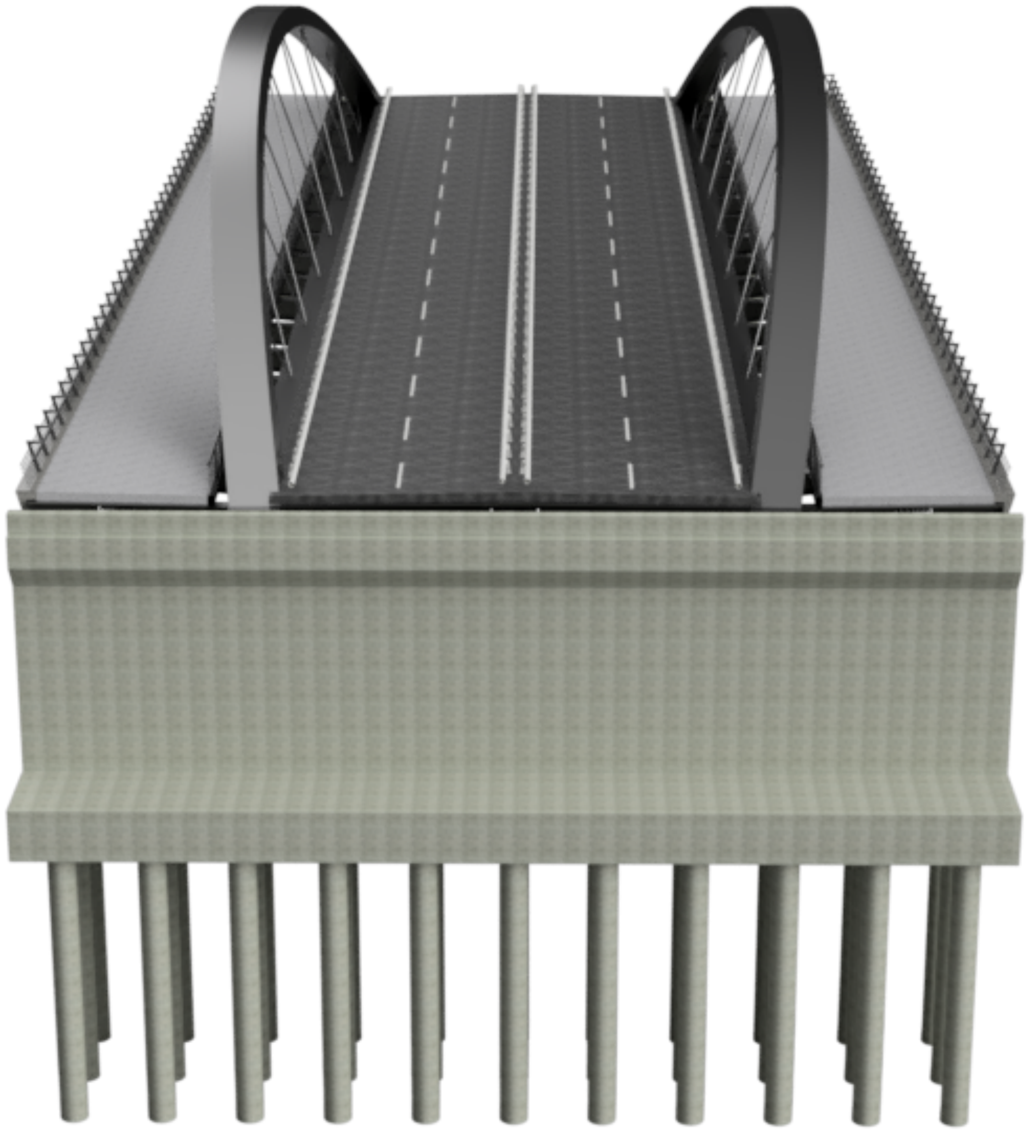
Appendix 2

Hydrology



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ACADEMIC YEAR 2024

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Hydrology (Compatibility with Flood Regime)

1. Introduction

In order to ensure the safety and longevity of our bridge and to analyze the influence of the water on our bridge and the influence of our bridge on the water, hydrological analyses are essential. As this is a real tender in our case, the hydrological analysis has already been prepared and specified by the tendering institution. This appendix evaluates the specified results.

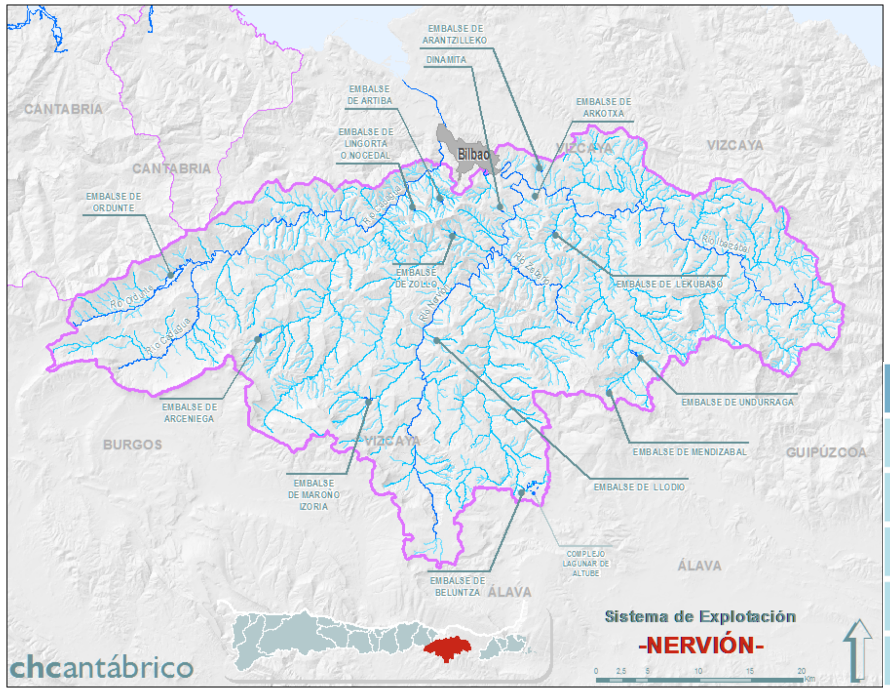
2. General Anformation about the Deusto Channel

The body of water crossed is the Deusto Channel, an approx. 2,4 km long and between 70 and 100 m wide channel that runs parallel to the River Nervión and is fed by that river's water. The idea for the channel originated back in the 1930s. The idea was to simplify navigation, as the bends of Olabeaga and Elorrieta, which had to be navigated in the natural course of the river, were becoming increasingly unsuitable for shipping. Work began in 1950 and was completed in 1951 except for the last 400 m. In 2018, the last 370 m of the channel were opened, among other things to prevent flood risks.

3. Exploitation Area and Underlying River System of the Nervión

3.1 Exploitation Area

The river system that feeds the Deusto Channel is the Nervión, whose exploitation area covers 1592,645 km2. The coverage network consists of 1216 rivers with a total length of 2170,3 km, with the three rivers Cadagua, Idazabal and Nervión representing the main axes of the system.



	NÚMERO	KILÓMETROS
RÍOS	1216	2170,3
EMBALSES	15	1,8
LAGOS	1	0,1
TRANSICIÓN	1	0,6
COSTA	0	0,0

Figure 1: Map of the Nervión exploitation system [source: chcantábrico]

3.2 Climatology

There is no specific consolidated data of the climate from the "Confederacion Hidrográfica del Cantábrico" for the Nervión exploitation area, but according to the Confederacion Hidrográfica del Cantábrico, it can be determined from interpolation of nearby stations that the climate in the exploitation area is similar to the climate at Bilbao airport, where climatological data is collected.

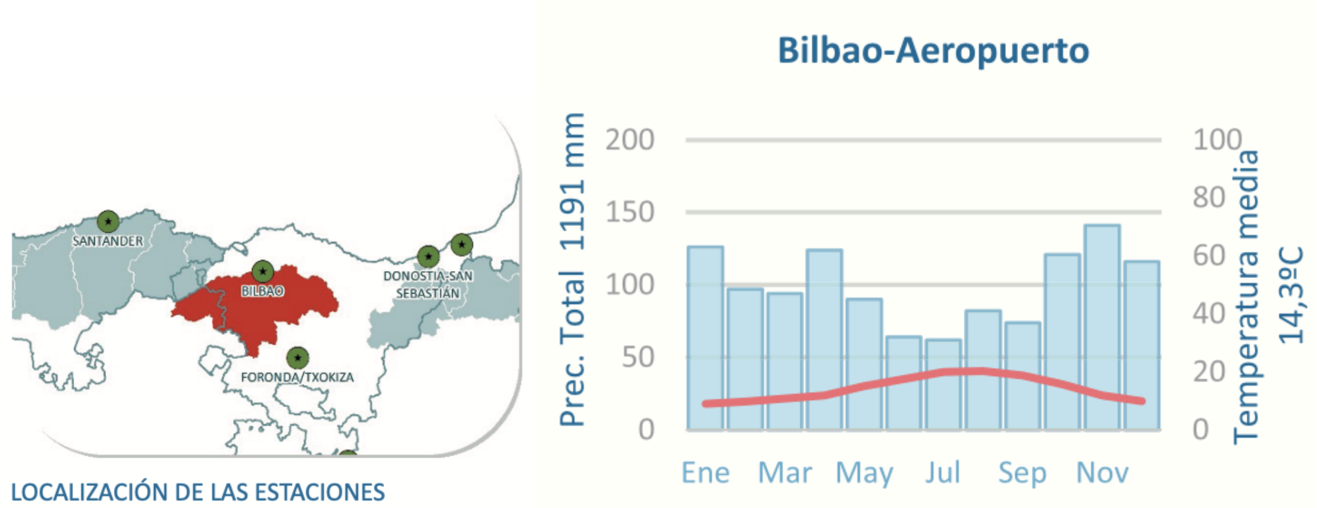
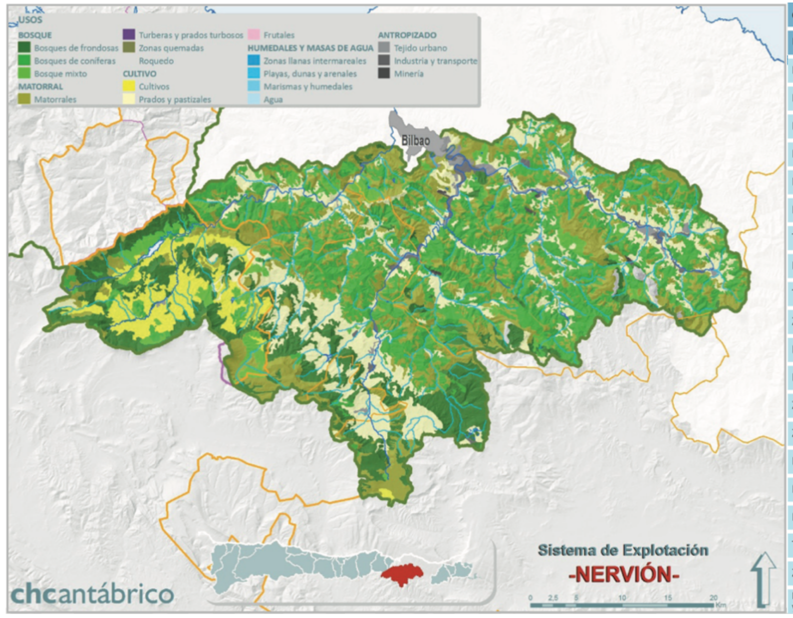


Figure 2: Annual climate in the Nervión exploitation area [source: chcantábrico]

The climate diagram shows that abundant precipitation can be expected throughout the year, but more so in winter.

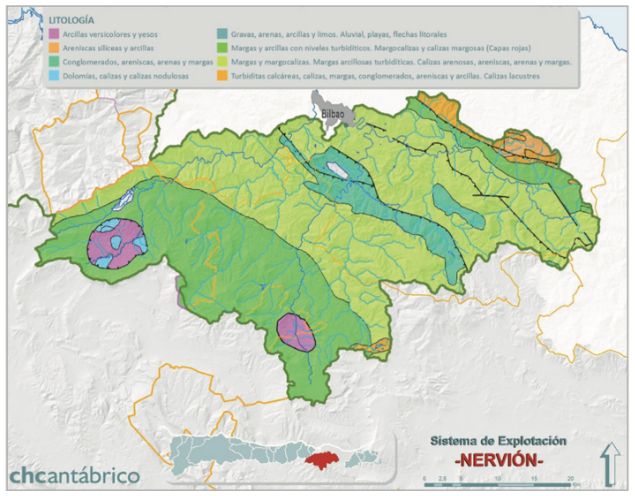
3.3 Landcover and Lithology

In order to assess the runoff behavior in the exploitation area, it is useful to analyze the use of the areas and their sealing proportion, as well as the lithological composition of the subsoil in order to better understand the runoff behavior of the exploitation area.



corine land cover		
USO (CORINE nv3)	Km²	%
Bosques de frondosas	237,8	14,93
Bosque mixto	67,5	4,24
Bosques de coníferas	463,68	29,11
Matorral boscoso de transición	271,6	17,05
Landas y matorrales mesófilos	67,2	4,22
Prados y praderas	277,3	17,41
Terrenos principalmente agrícolas	85,8	5,39
Mosaico de cultivos	1,7	0,10
Tejido urbano discontinuo	11,6	0,73
Zonas verdes urbanas	0,3	0,02
Pastizales naturales	17,9	1,12
Espacios con vegetación escasa	0,3	0,02
Zonas en construcción	0,1	0,01
Zonas industriales o comerciales	14,3	0,90
Láminas de agua	1,6	0,10
Matorrales esclerófilos	51,5	3,23
Roquedo	7,9	0,50
Tejido urbano continuo	8,5	0,53
Zonas de extracción minera	2,1	0,13
Redes viarias, ferroviarias y terrenos asociados	0,5	0,03

Figure 3: Map of land use in the nervión exploitation area [source: chcantábrico]



LITOLOGÍA	ERA	SUPERFICIE km²
Arcillas versicolores y yesos	MESOZOICO	50,8
Areniscas silíceas y arcillas	CENOZOICO	15,5
Conglomerados, areniscas, arenas y margas	MESOZOICO	113,3
Dolomías, calizas y calizas nodulosas	MESOZOICO	11,8
Gravas, arenas, arcillas y limos. Aluvial, playas, flechas litorales	CENOZOICO	0,3
Margas y arcillas con niveles turbidíticos. Margocalizas y calizas margosas (Capas rojas)	MESOZOICO	543,6
Margas y margocalizas. Margas arcillosas turbidíticas. Calizas arenosas, areniscas, arenas y margas.	MESOZOICO	814,4
Turbiditas calcáreas, calizas, margas, conglomerados, areniscas y arcillas. Calizas lacustres	CENOZOICO	36,2

Figure 4: Map of the lithology in the nervión exploitation area [source: chcantábrico]

Even though the exploitation area is largely unsealed natural land, the lithology is largely characterized by marl, which means that the soil has low water permeability. As a result, the majority of precipitation is transported away via the existing river networks.

4. Given Hydraulic Profile

As this is a real tender in our case, the sewer cross-section with discharge height and flood safety are provided to the applicants by the tendering company. Since only channel dimensions are given for discharge dimensioning and no numerical discharge volumes, we work with the dimensions. It can therefore be concluded that we cannot verify the bridge course with exact figures, but the given dimensions are sufficiently reliable to work with.

At the location of our bridge, the channel is 75 m wide. Normal zero is 3,50 m above the channel bottom at the sides and 4,60 m above the channel bottom in the middle. The average discharge height is 2m above sea level.

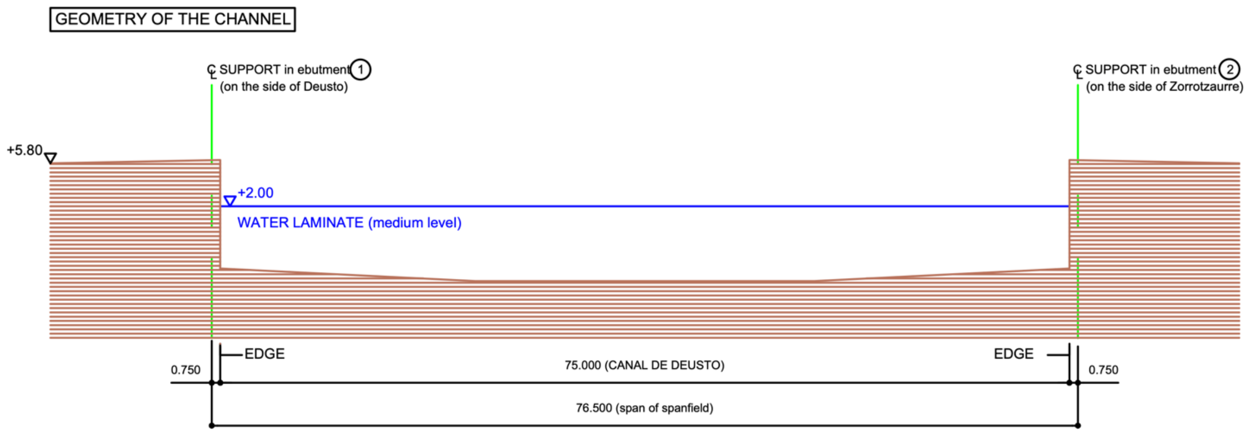


Figure 5: average runoff Channel de Deusto

The flood level of the Channel is 4,45 meters above sea level. In addition, a clearance of 1 meter in height must be maintained along the central 40 meters of the channel, for example to allow flotsam to drain away in the event of a flood and to ensure that it does not damage the bridge. As both the course of the roadway and the flood safety cross-section have been specified by the tendering company, all the limiting factors have already been ensured.

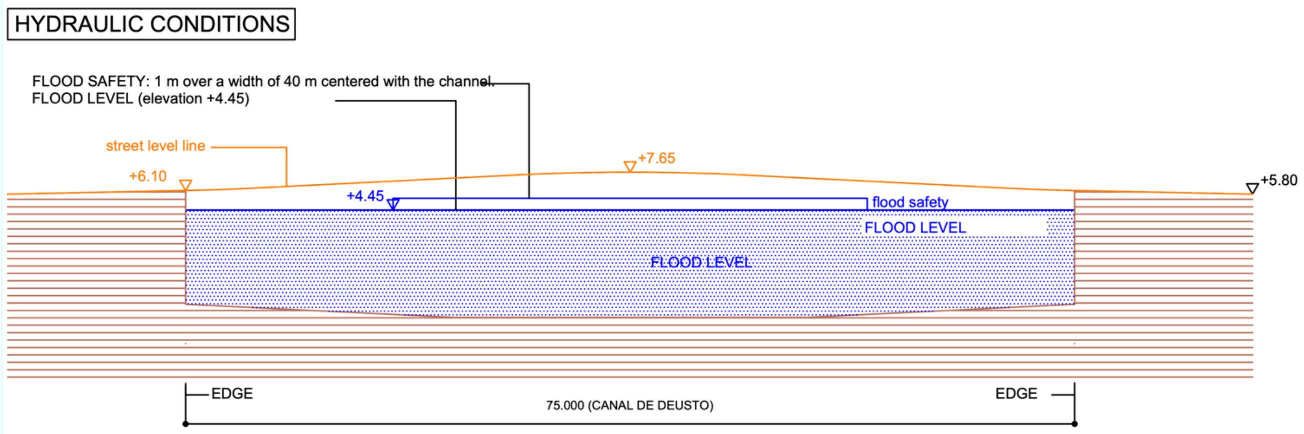


Figure 6: Cross-section of flood runoff Channel de Deusto

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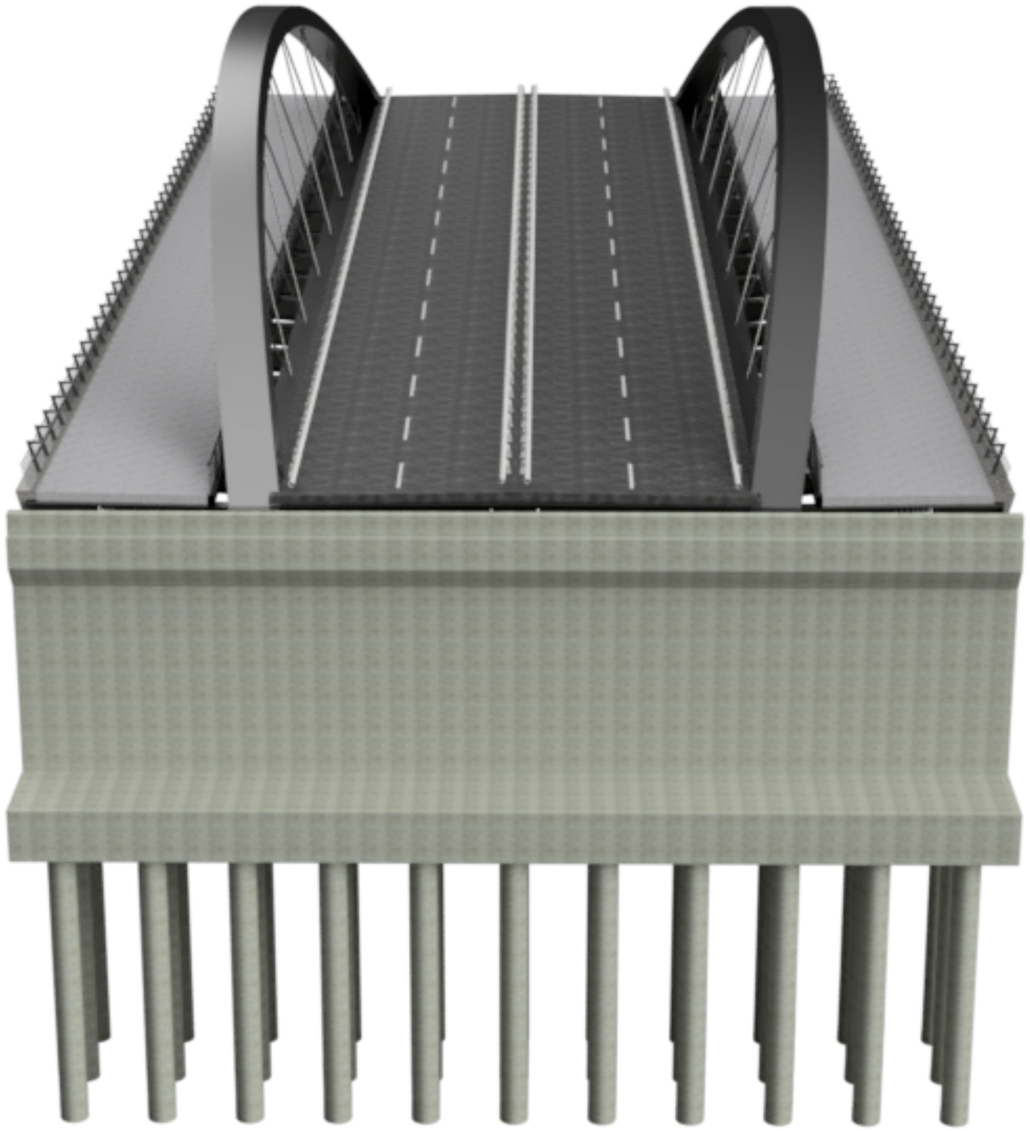
Appendix 3

Study of Alternatives



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Study of Alternatives

1. Introduction

The purpose of this appendix is to study different alternatives and choose a final solution for the construction of a new bridge over the Deusto channel in Bilbao.

The construction of the first bridge of Zorrotzaurre Island in Bilbao represents a pivotal moment in the development of the region, promising to enhance connectivity and foster economic growth. As with any infrastructure project of this magnitude, the selection of the most suitable bridge design is paramount to its success.

1.1. Purpose of the Study

The primary objective is to conduct a comprehensive analysis of alternative bridge designs, considering several factors such as technical feasibility, economic viability and environmental integration.

Given the urban and commercial development of the area, the aesthetic aspect of the bridge is of great importance. The bridge should not only fulfill its practical function, but also becomes a distinctive element that contributes to the identity of the area.

The proposals are based on the most suitable bridge typologies for the specific conditions of the project. The study will focus on finding the optimal solution that meets the objectives and constraints.

2. Prerequisites

Before different alternatives of bridge designs can be considered, it is important to determine the technical requirements that will serve as the basis for the study. These prerequisites limit the choice of possible designs and include various factors, such as the following technical constraints.

2.1. Technical Constraints

The selection of structural materials and typology is free, always paying special attention to durability, future maintenance and the visual impact obtained. The location of supports in the channel is not allowed, therefore only single span solutions are viable.

2.1.1. Plan View

The plan view of the bridge design shows a straight alignment that provides a direct connection between the "Julio Urquijo" street in the Deusto district and the island of Zorrotzaurre.

Maintaining a straight alignment simplifies the construction and improves the overall efficiency of traffic in the area.

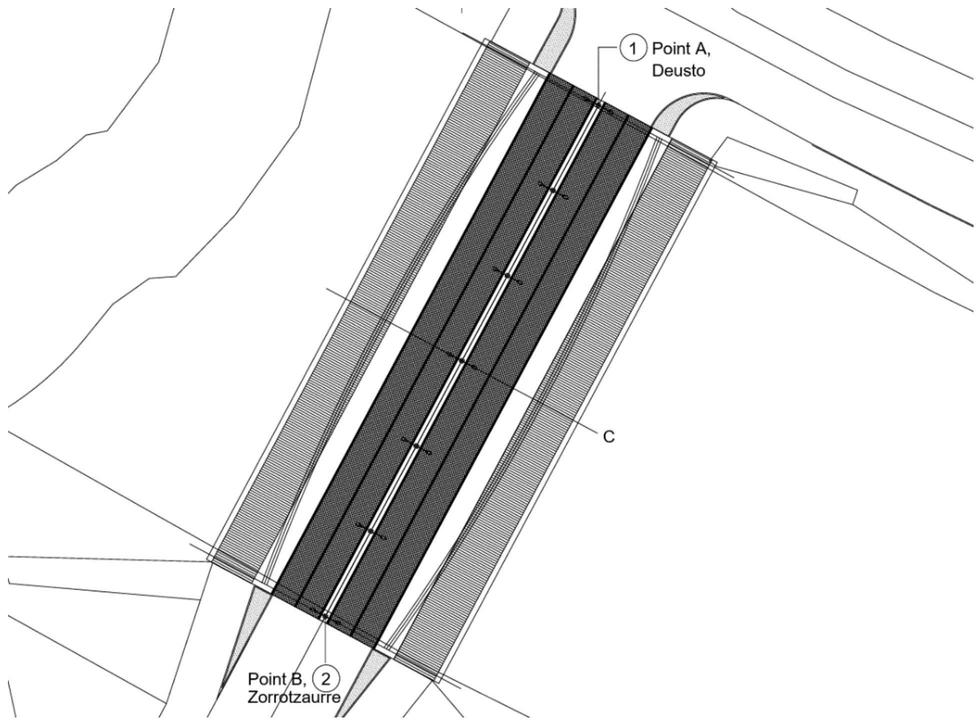


Figure 1: Plan View of the Proposed Bridge, in the North the Deusto District, in the South the Zorrotzaurre Island [Own elaboration]

2.1.2. Lateral View

The span length of the bridge will be approximately 76.50 m, depending on where exactly the axes are positioned above the abutments. The lower edge of the deck is determined by the required safeguard above the flood level. This additional protective space is located in the middle of the channel, has a height of one meter and extends over 40 meters. Since the upper street level is given, we can find the critical point for the total height of the deck with pavement. This point is right at both edges of the safety area and has a value of 1.45m. To rule out errors due to tolerances, the maximum total deck height is limited to 1.40 m for all alternatives.

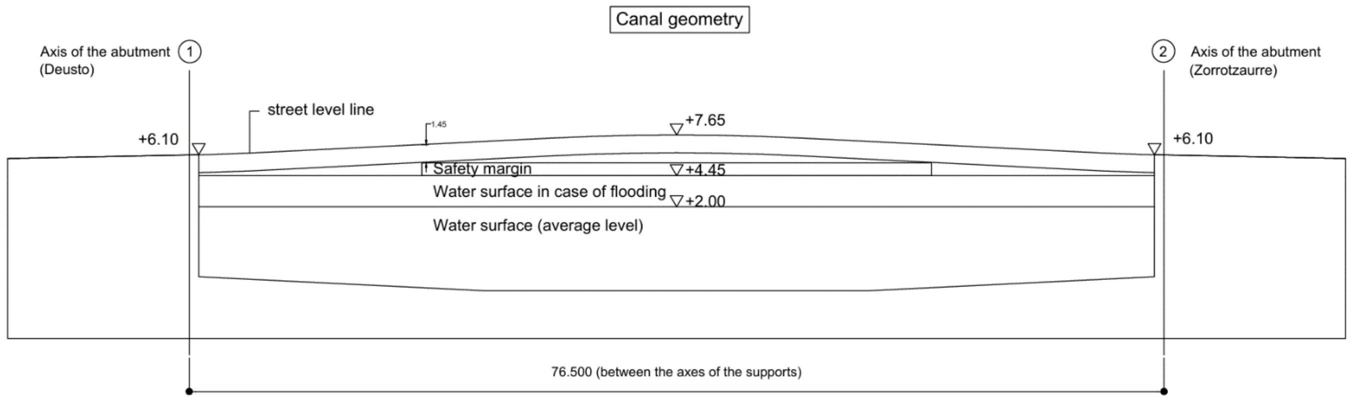


Figure 2: Lateral View, Geometry of the Deusto Channel [Own elaboration]

2.1.3. Cross-Section

The usable width must be greater than or equal to 28.00 m, with 2 lanes of 3.5 m in each direction of traffic, with shoulders of 1 m to accommodate safety barriers and space for sidewalks of 6 m on both sides. A 2% slope is assumed for cross drainage.

The following example distribution shows what this could look like.

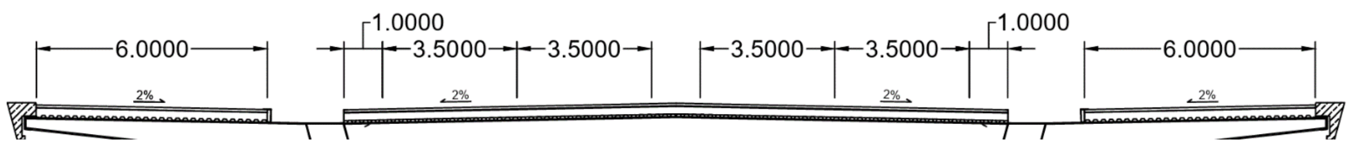


Figure 3: Required Width and Slopes of the Deck [Own elaboration]

3. Scope of Alternatives Considered

First, a brief overview of the diverse types of bridges that are available today will be given. This will help to get a general overview to decide which of these types can still be taken into consideration under the conditions and constraints mentioned above.

3.1. Different Types of Bridges

In the field of bridge construction, there are a number of different bridge types with unique designs and functionalities, each of which is more or less suitable for certain conditions and specific needs. Understanding the characteristics of the different bridge types is essential for assessing their suitability for the project at hand. The most common bridge types are presented below.

Beam Bridges:

The simplest and most common bridge configuration, yet suitable for a wide range of span lengths, is the beam bridge, also known as a girder bridge.

They consist of horizontal girders supported by vertical piers or abutments. Beam bridges are typically used for relatively short spans and can be constructed with single span beams or continuous beams. Their structural behavior is simple, with deck deflections predominantly in the longitudinal direction. They are often constructed of prestressed concrete, as well as of steel or as a composite structure.

Arch Bridges:

Arch bridges use the strength and stability of curved arches to support the bridge deck. They are ideal for bridging medium distances and are known for their aesthetics. They can be adapted to many different architectural styles and are very versatile in their design. The number and position of the arch(es) is variable. Usually, one or two arches are used. These can be inclined inward or outward and can be positioned above, below or centered in relation to the deck.

The arches, which function mainly in compression, are usually made of steel or reinforced concrete and their main structural problem is the risk of buckling and loss of stability.

Cable-Stayed Bridges:

In cable-stayed bridges, cables support the bridge deck directly from tall pylons or towers. Unlike suspension bridges, where the main cables hang freely, in cable-stayed bridges the cables are fanned out from the towers and support the bridge deck at several points. Cable-stayed bridges have a very characteristic shape, characterized by the arrangement and number of cables, the type of towers or pylons, and the morphology of the deck. While the cables are subject to tensile loads, the towers and deck must withstand very high compressive forces. This type of bridge is suitable for medium and long spans and is particularly economical for the latter.

Suspension Bridges:

In a suspension bridge, the deck is suspended from cables, which are connected to the main cable by means of which the forces are transmitted to the tall towers. This design allows suspension bridges to span large distances with minimal support structures, making them suitable for spanning wide bodies of water or large valleys. Suspension bridges are mainly characterized by their iconic appearance and large spans.

Truss Bridges:

In truss bridges, the bridge deck is supported by a framework of interconnected beams, known as trusses. Truss bridges come in a variety of configurations and are valued for their favorable strength-to-weight ratio. They are frequently used for railroad bridges. However, due to their industrial design, they usually are not visually appealing.

Each of these bridge types offers unique advantages and they are used in different environments. In the following, their applicability under the given conditions and constraints of the project is determined.

3.2. Overview of Possible Alternatives

The following table shows the recommended optimum spans for different bridge types from an economic point of view.

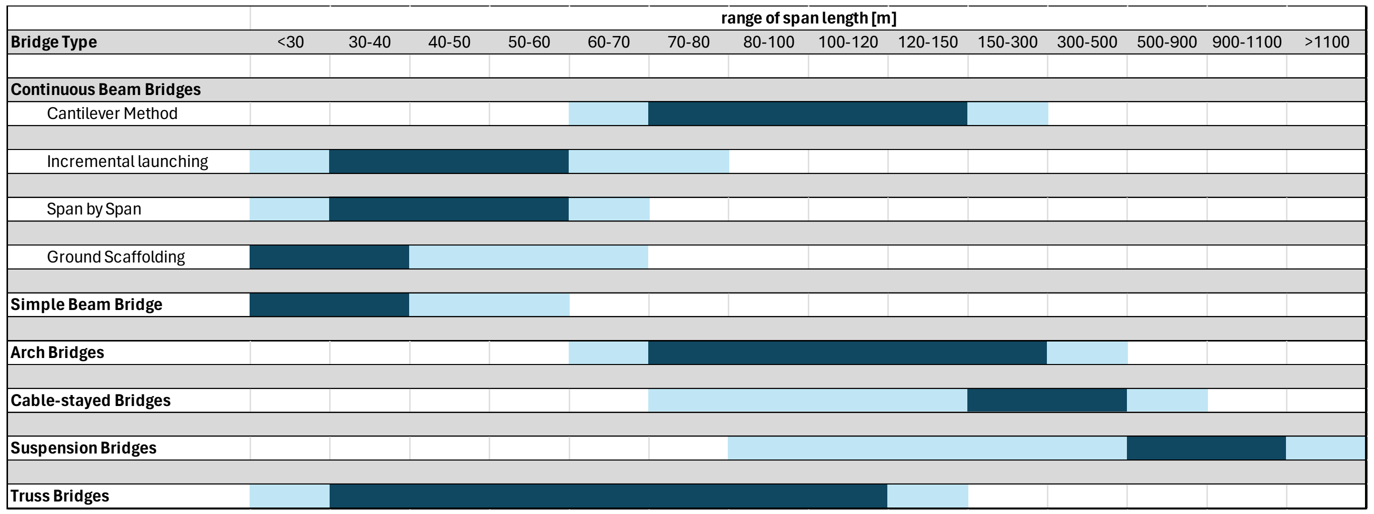


Figure 4: Optimum Spans of Bridge Types from an Economic Point of View [Own elaboration]

Information is derived from the slides of the course "Concepción de Puentes" at the UPV, Professor: Monleón Cremades, Salvador

The option of constructing the beam bridge as a continuous girder is out of the question, as no piers can be placed in the channel bed. A very thick beam would be needed to span a single-span girder over a distance of approximately 76,5m. This is also not possible due to the already specified need for a safety space in the event of flooding and the therefore maximum total deck height of 1,40m. A single span beam would have to be thicker and then run through this flood safety clearance.

Suspension bridges are primarily used for much larger spans and would give a strange appearance with the width/length ratio of the current project. Cable-stayed bridges, on the other hand, can also be visually attractive designed for smaller spans.

This leaves three types of bridges: cable-stayed bridges, arch bridges, and truss bridges. These can be combined with the requirements of the project and produced in an aesthetically pleasing manner.

4. Analysis of Alternatives

4.1. Alternative 1 - Bowstring Bridge

The first alternative presented is a bowstring bridge. It consists of two steel arches, each slightly inclined outwards to create a unique design. Statically, this has some but no major disadvantages, as the inclination of 75 degrees to the horizontal is not exceptionally large. The cross section of the arches resembles a parallelogram, with the sides following the inclination of the arches to create an overall organized appearance.

Since the arches function mainly in compression, and buckling is often a problem in delicate steel structures, the parallelogram has a greater length in the horizontal direction, which is the critical direction.

The compressive force from the arches is absorbed by tensile forces in the longitudinal beams of the deck below. As a result, only vertical forces are transferred to the foundations, as is the case with a bow-string bridge.

The deck consists of transverse beams with lateral cantilevers arranged at 4.25 m intervals. Zigzag tension rods are connected to every second beam to transfer deck forces to the arches. On top of the transversal beams, a corrugated metal sheet is installed in the roadway area and on both sides for the sidewalks, on which a 26 cm thick layer of concrete is poured. This is followed by an impermeable layer, then 2 layers of asphalt in the roadway area and anti-slip paving stones in the sidewalk area, as described in more detail in the Appendix “Bridge Fittings”.

To further enhance the aesthetics, the pedestrian walkways are offset one meter outward creating a free space between the transversal beams to provide pedestrians with a unique view of the river.

In order to be able to estimate the dimensions in advance, without having to make a detailed calculation, ratios from similar previous projects are used. These values are listed in the table below, with the first row indicating the values used for alternative 1.

Bridge	Year	Type	Material A/D	Span L [m]	Height at center f [m]	Depth d (arch) [m]	S	Depth h (deck) [m]	Width B	L/f	L/d	S/h
Alternative 1		Bowstring	S/S	76,5	10	0,7	8,5	1,3	34	7,65	109,29	6,54
Fehmarnsund	1963	Bowstring	S/S	248,4	43	3	11,8	2	21,7	5,77	82,8	5,85
Kaiserlei	1964	Bowstring	S/S	220	26	2,05	10	2,97	35,9	8,46	107	3,4
Barqueta	1992	Bowstring	S/S	168	29,8	1,8	8,5	2,4	21,4	5,63	93,3	3,5
Tercer Milenio	2008	Bowstring	Rc/Pc	216	36	1,8	6	3,2	42,7	6	120	1,88

A= Arch
D= Deck
S= Steel
Rc= Reinforced concrete
Pc=Prestressed concrete
S= distance between supports of the deck

L/f= lowering of the arch
L/d= slenderness of the arch
S/h= relative deck slenderness

Figure 5: Dimensions of Alternative 1 in Relation to Similar Projects [Own elaboration]

Information is derived from the slides of the course "Concepción de Puentes" at the UPV, Professor: Monleón Cremades, Salvador

As can be seen in the table, the bridge is within a similar range to reference projects in terms of ratio values.

The arches are shaped in the form of a second-order parabola with a height of 10 m in the center. This shape comes close to the catenary arch, is visually more attractive and easier to construct. The depth of the entire deck is fully utilized at 1.40 m.

To ensure lateral drainage, the transverse beams are provided with a slope of 2% in the upper area. The roadway area is inclined outwards and the pedestrian area inwards, so that drainage takes place near the arches directly into the channel below. By inclining the beams instead of producing the inclination by varying the thickness of the asphalt, we can save unnecessary dead loads.

The alternative described above is illustrated in the following drawings and 3D renderings:

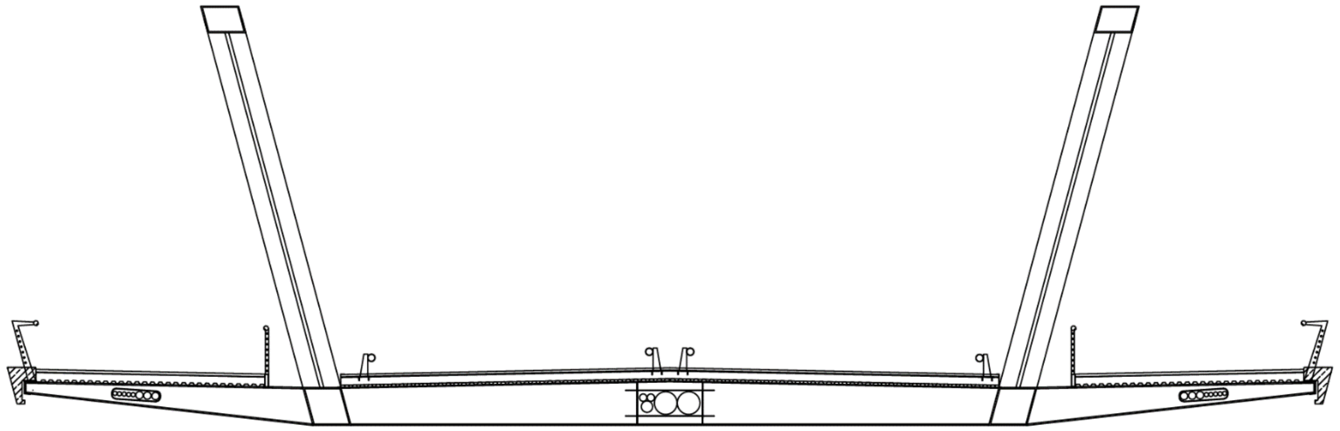


Figure 6: Cross-Section of Alternative 1 [Own elaboration]

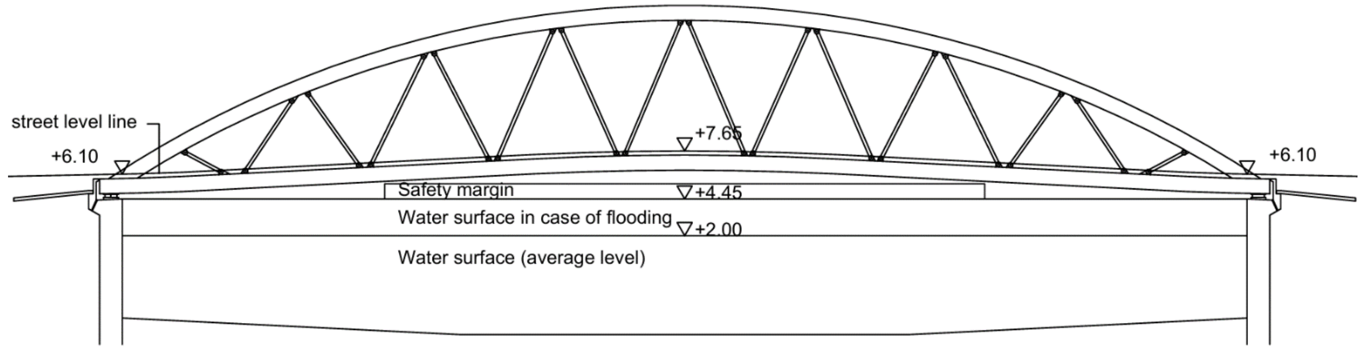


Figure 7: Lateral View of Alternative 1 [Own elaboration]

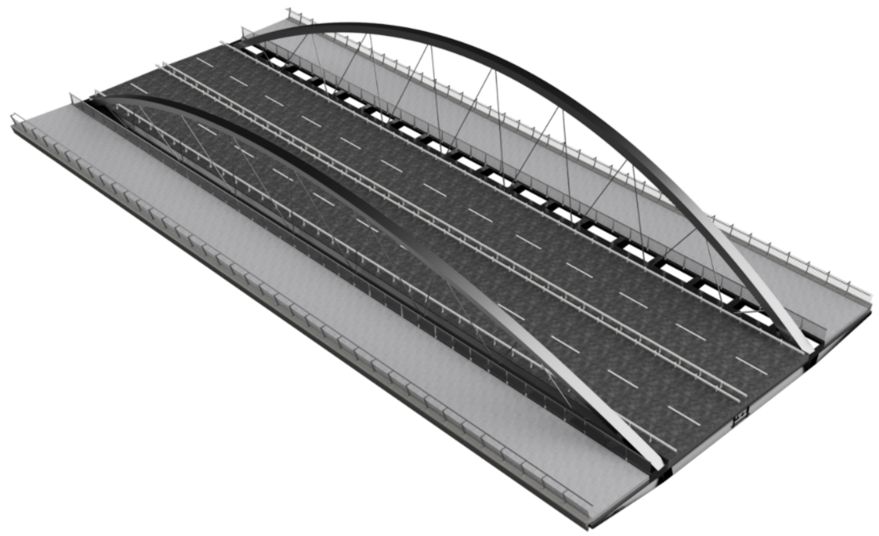


Figure 8: Bird's Eye 3D Rendering [Own elaboration; Autodesk Fusion 360]

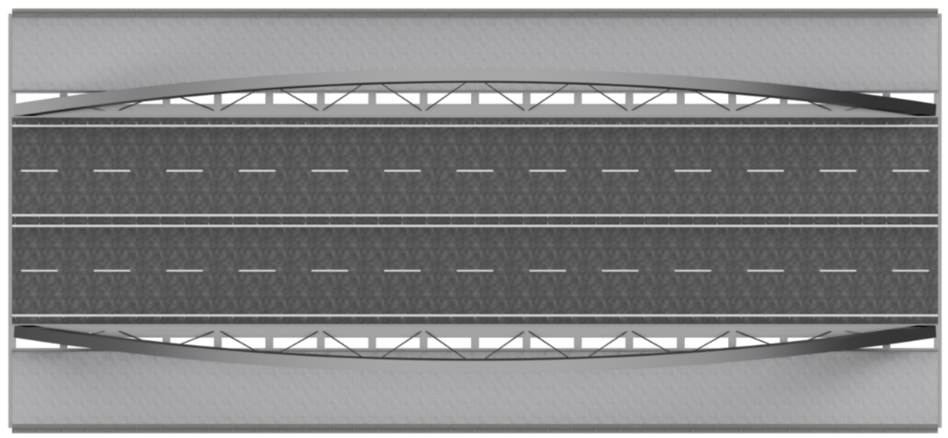


Figure 9: Plan View 3D Rendering [Own elaboration; Autodesk Fusion 360]

4.2. Alternative 2 - Warren Truss Bridge

The second alternative is a Warren Truss bridge. The deck support structure is very similar to the previously presented alternative, except that the main longitudinal beams have a square cross section. The trusses, which are made of rectangular metal profiles and are part of the longitudinal beams, separate the roadway from the pedestrian area. The upper chord is compressed, while the lower truss is under tension. The shape of the trusses follows the elevation of the roadway in the side view, thus reaching the highest point in the middle.

The alternative described above is illustrated in the following drawings and 3D renderings:

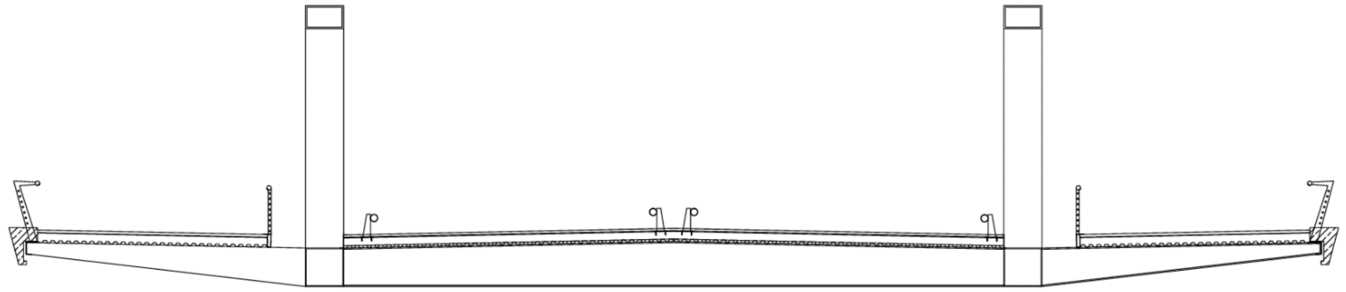


Figure 10: Cross-Section of Alternative 2 [Own elaboration]

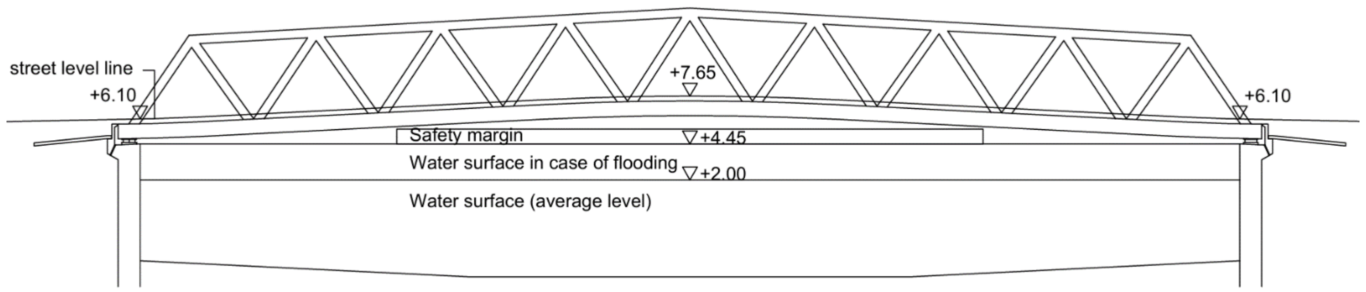


Figure 11: Lateral View of Alternative 2 [Own elaboration]

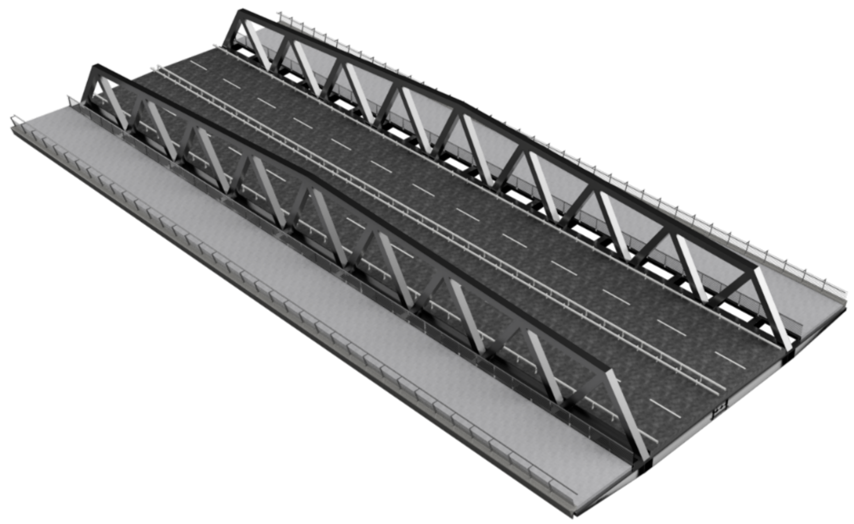


Figure 12: Bird's Eye 3D Rendering [Own elaboration; Autodesk Fusion 360]

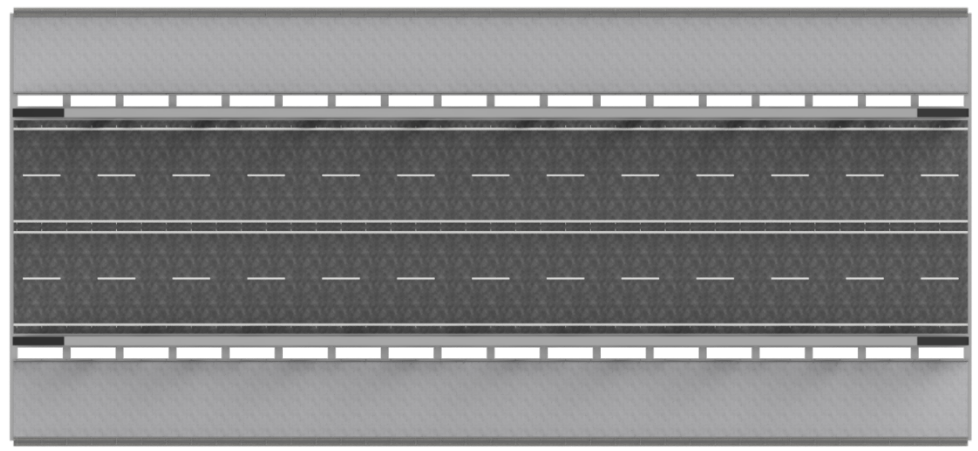


Figure 13: Plan View 3D Rendering [Own elaboration; Autodesk Fusion 360]

4.3. Alternative 3 - Single Tower Cable-stayed Bridge

The third alternative is a cable-stayed bridge with an outwardly inclined tower supporting the deck with cables every 8.5m. These are located in the center of the cross section and therefore provide no torsional resistance. For this reason, the deck is made of a prestressed concrete box girder. Thanks to its closed shape, it can effectively resist the torsional forces.

The cables form an angle of 22 degrees with the horizontal and are arranged in parallel. Their tensile forces are absorbed by the compression and bending in the tower. The inclination of the tower reduces the bending moments and increases the compressive forces. However, since the bending moments are unavoidable and the tower is cantilevered, it must be anchored to the ground by a solid foundation.

To better estimate the dimensions, we again compare them to similar projects.

Bridge	Year	Span L [m]	S [m]	Depth h (deck) [m]	Width B [m]	Deck material	L/h	S/h	α(°)
Alternative 2	/	76,5	8,5	1,4	32,2	Pc	55	6,07	22
Sancho el Mayor (Castejón)	1978	146,3	34,4	2,15	28,9	Pc	68	16	21,5
Río Lérez (Pontevedra)	1995	125	18	2	18,8	Pc	63	9	22

S= distance between supports of the deck L/h=deck slenderness
Pc=Prestressed concrete S/h= relative deck slenderness
α (°)= (smallest) angle of the cables

Figure 14: Dimensions of Alternative 3 in Relation to Similar Projects [Own elaboration]
Information is derived from the slides of the course "Concepción de Puentes" at the UPV, Professor: Monleón Cremades, Salvador

The alternative described above is illustrated in the following drawings and 3D renderings:

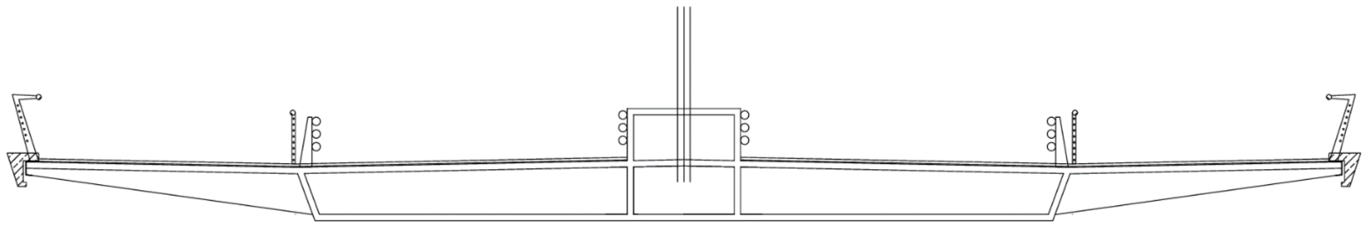


Figure 15: Cross-Section of Alternative 3 [Own elaboration]

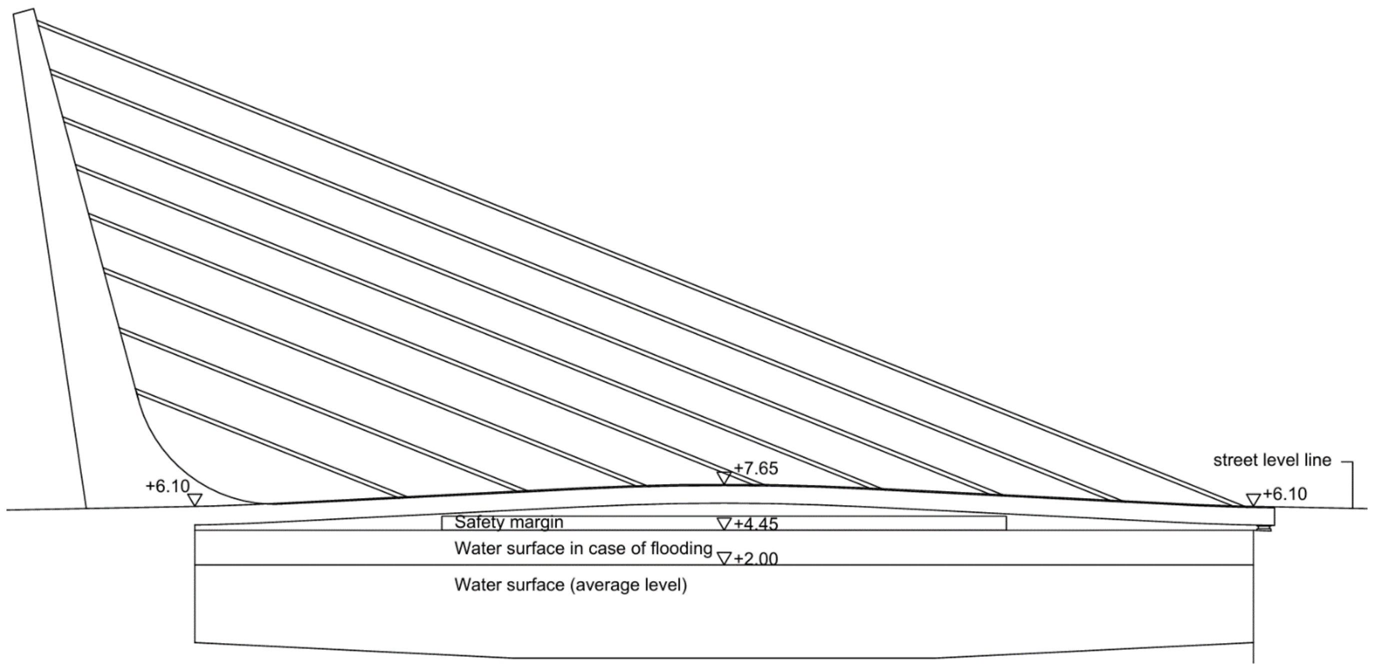


Figure 16: Lateral View of Alternative 3 [Own elaboration]

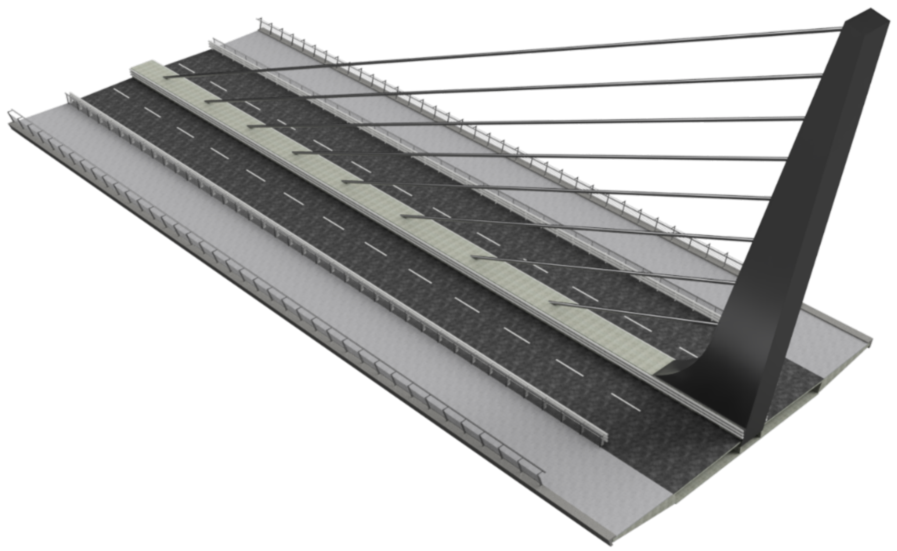


Figure 17: Bird's Eye 3D Rendering [Own elaboration; Autodesk Fusion 360]

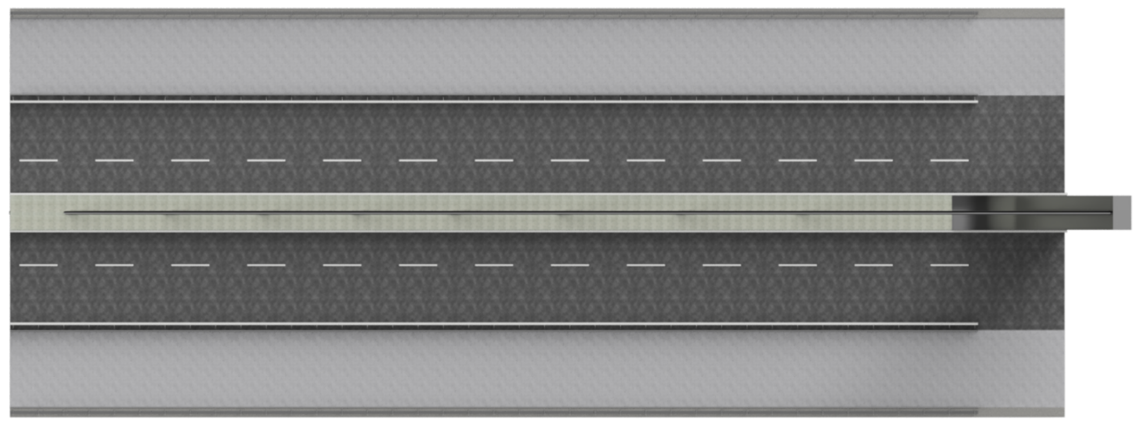


Figure 18: Plan View 3D Rendering [Own elaboration; Autodesk Fusion 360]

4.4. Alternative 4 - Four Tower Cable-stayed Bridge

The fourth alternative is also a cable-stayed bridge, but with a different configuration of cables and towers. Instead of 1 tower, this variant has 2 towers on either side. The deck is still supported by the stay cables every 8.5 meters, but instead of running parallel, the cables of one half meet at the top of a tower at a distance of 0.5 meters. Again, to reduce the resulting bending moments in the towers, they are slightly inclined outwards.

The deck design is identical to that of Alternative 2, with the exception of the structural support system. Instead of trusses, the deck is supported by two planes of cables, which offer torsional rigidity in contrast to one plane as in Alternative 3.

The alternative described above is illustrated in the following drawings and 3D renderings:

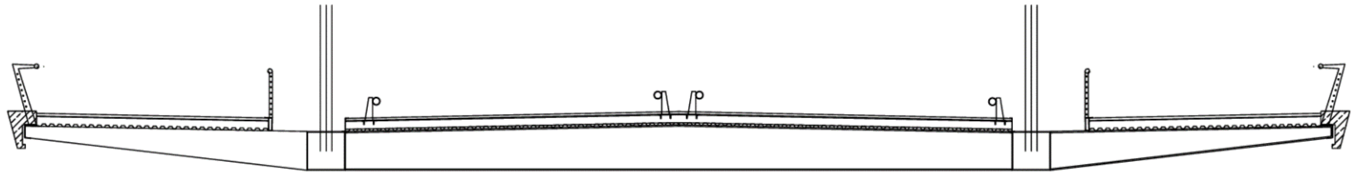


Figure 19: Cross-Section of Alternative 4 [Own elaboration]

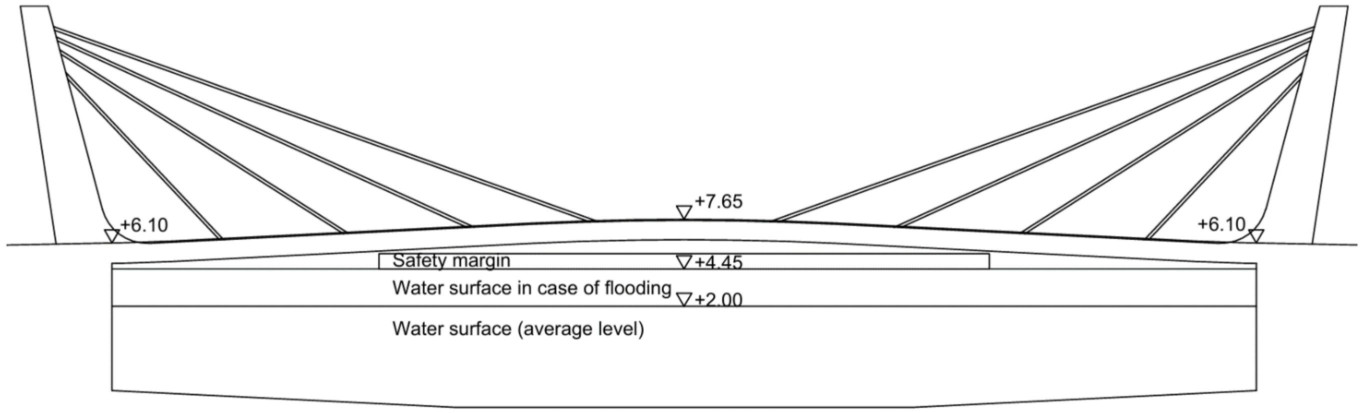


Figure 20: Lateral View of Alternative 4 [Own elaboration]

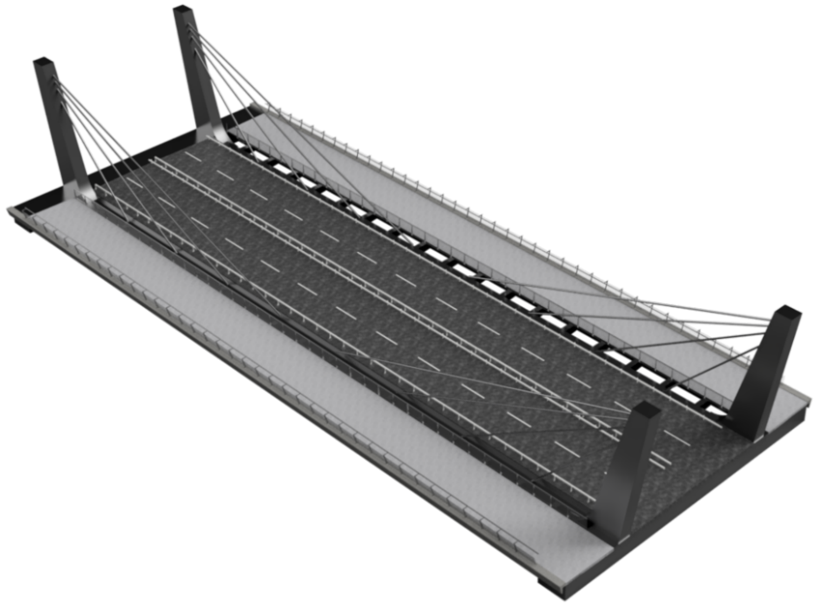


Figure 21: Bird's Eye 3D Rendering [Own elaboration; Autodesk Fusion 360]

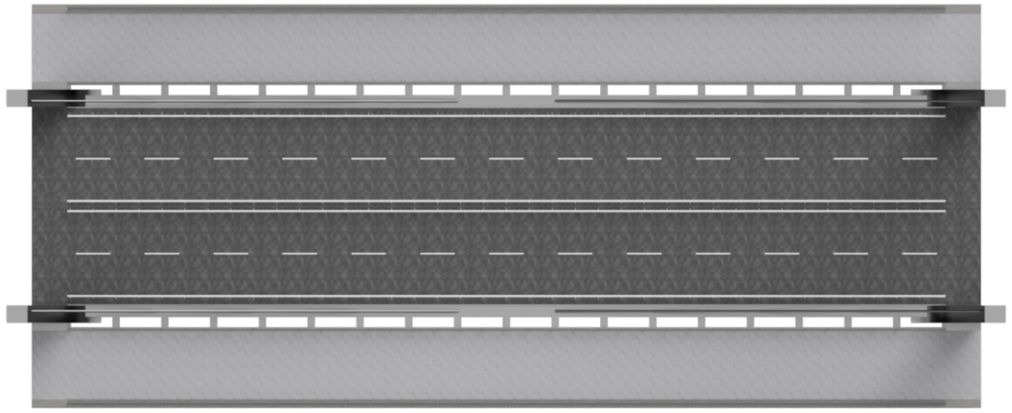


Figure 22: Plan View 3D Rendering [Own elaboration; Autodesk Fusion 360]

5. Evaluation of Proposed Alternatives

5.1. Evaluation Criteria

Once all the alternatives have been presented, they are evaluated according to various criteria to determine their suitability for the project. A systematic approach is employed, utilizing a scoring system to assess each alternative against predefined selection criteria.

Scoring Methodology

For the evaluation process, a table is utilized, with each alternative assigned to the columns and the selection criteria to the rows. The criteria are selected based on their significance to the project and are assigned weighting factors in the form of percentages, reflecting their relative importance. This weighting ensures that more critical criteria carry greater influence in the final assessment.

Each bridge alternative is assessed against the selection criteria and assigned a score ranging from 0 to 10 for each criterion, with 10 indicating the highest level of fulfillment.

Selection Process

Following the assignment of scores, the average score for each alternative is calculated. This average score serves as a quantitative measure of the overall suitability of each alternative. The alternative with the highest average score is then selected as the optimal variant for further consideration and refinement.

Evaluation Criteria

- Cost-effectiveness: Evaluating and estimating the construction and maintenance costs associated with each bridge alternative to determine the most financially viable option. Economic efficiency plays a significant role in the construction of public projects, and therefore a percentage of 50% is allocated.
- Aesthetic Appeal: Considering the visual impact and architectural quality of each bridge design to enhance the overall aesthetics of the surrounding area. Although this is a very objective point, the bridge should integrate well into its surroundings and it is therefore seen as the second most crucial point, with a weighting factor of 25%.
- Functionality: Evaluating the functional performance of each bridge design in terms of traffic capacity, accessibility for pedestrians and vehicles, ease of navigation, as well as the restriction of the alignment from the access roads. A percentage of 15% is assigned.
- Construction Time and process: Estimating the time required to construct each bridge alternative and considering the impact on project timelines and overall feasibility. Since the bridge will be constructed prior to the opening of the channell, the channel can easily be occupied for the construction process. In addition, construction time is not decisive as the construction of the bridge will not interfere with other activities. Therefore, this selection criterion is given the lowest weighting factor of 10%.

5.2. Determination of the Bridge Design

Overall, the arch bridge and the truss bridge perform very well. Both can score points with a simple and fast construction process, which also leads to low costs, but the widely used truss bridge is penalized by its not very appealing appearance. The arch bridge therefore scores better overall.

The cable-stayed bridge with one high tower (alternative 3) can score points with its eye-catching appearance, but its construction process is extraordinarily complex and involves high costs. The four-tower cable-stayed bridge (alternative 4) performs significantly better in these aspects. However, it is not as aesthetically pleasing.

Evaluation Criteria	Weighting factors	1. Arch Bridge	2. Truss Bridge	3. One-Tower Cable-Stayed Bridge	4. Four-Tower Cable-Stayed Bridge
Cost-effectiveness	50%	8	9	5	8
Aesthetic Appeal	25%	8	4	9	7
Functionality	15%	7	8	5	8
Construction Time and process	10%	9	9	5	5
Average		**7,95**	**7,6**	**6**	**7,45**

Figure 23: Alternatives scored out of 10 [Own elaboration]

As the arch bridge achieves the highest score, albeit by a narrow margin, it is selected as the optimal option and will be analyzed in the following course of the thesis.

6. Selected Design

The selected design is that of the first alternative, which has already been briefly described above.

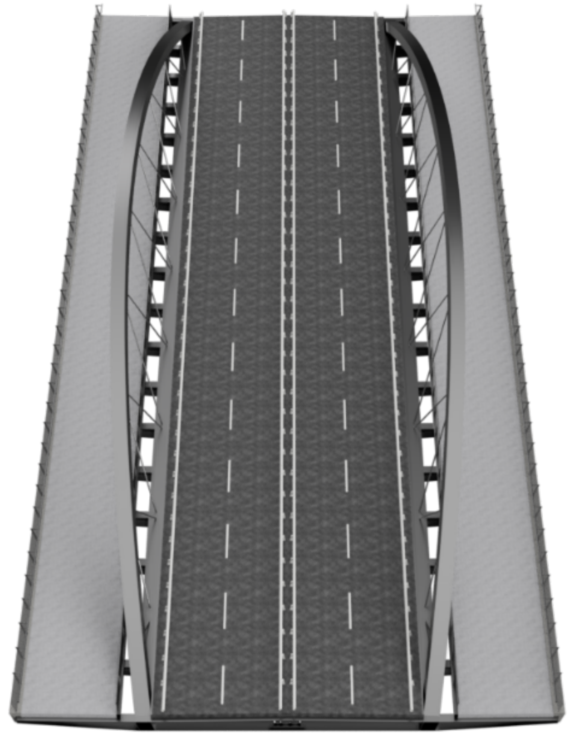


Figure 24: Bird's Eye 3D Rendering of Alternative 1 [Own elaboration; Autodesk Fusion 360]

To pre-dimension the load-bearing structure, the following properties were determined:

- Steel structure
  - The arches are inclined outwards at an angle of 75 degrees and the side walls of their parallelogram cross-section follow this inclination and have a wall thickness of 3 cm.
  - The main beams also consist of a parallelogram cross-section with a wall thickness of 2 cm.
  - The cross beams are separated 4.25m from axis to axis.
  - Every 8.5m, i.e., every second cross beam, the deck is supported by the zigzag bars, which have a diameter of 10cm.
- A composite system of corrugated metal and cast-in-place concrete slab with a total thickness of approximately 26 cm is placed over the transversal beams.

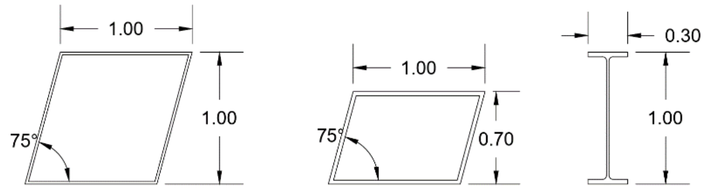


Figure 25: Cross-Section of the Main Beams / the Arches / the Transversal Beams [Own elaboration]

The drawings presented in this document represent a preliminary estimation of the dimensions of the bridge, prior to the calculation of the supporting structure. It is important to note that the calculations presented in the appendix, entitled "Calculation of the Structure," indicate that the cross-section of the arches must be widened. The final drawings for the project can be found in document 2 "Drawings".

Furthermore, the following is not within the scope of this appendix:

- The foundations are described in more detail in the attachment "Calculation of the Foundations".
- The fittings of the bridge, such as lighting, handrails, crash barriers, road surface, as well as pipeline layout is further explained in the annex "Bridge Fittings".
- The detailed scheduling of the work and the construction process is outlined in the appendix "Work Schedule"
- In this appendix the costs of the different alternatives were only estimated by putting them into relation to each other and giving the highest number for cost effectiveness to the cheapest solution. A detailed economic evaluation for our specific design is carried out in Document 3 "Economic Evaluation".

7. Conclusion

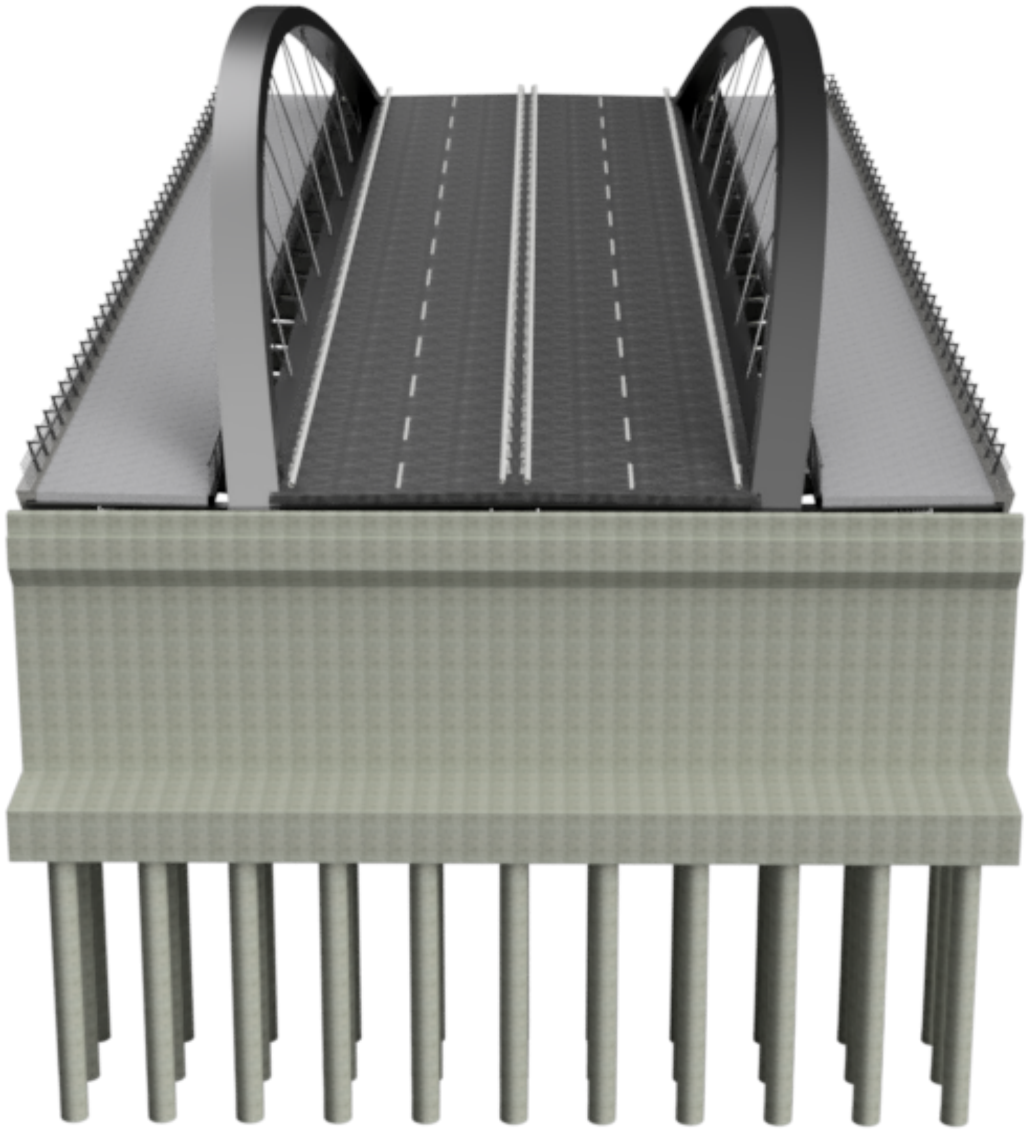
After a detailed study of the diverse types of bridges to cross the Deusto channel, the Arch Bridge variant has emerged as the most optimal solution in terms of cost effectiveness, aesthetics, functionality, and construction time and process. The result is a first pre-dimensioning of the structure and a rough idea for further detailed elaboration in the following appendices.

Bibliography

- Professor Salvador Monleón Cremades, Universidad Politécnica de Valencia (2024): Slides of the course Structural Design of Bridges
- Professor Salvador Monleón Cremades, Universidad Politécnica de Valencia (2023/2024): Slides of the course Conceptual Design of Bridges

Appendix 4

Calculation of the Structure



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Carlos Manuel Lázaro Fernández

DATE: JULY 2024

ACADEMIC YEAR 2024

Table of Contents

Calculation of the Structure

1. Purpose of the Study

The purpose of this annex is to define and justify the dimensioning of the bridge over the Deusto channel. The calculations have been carried out according to the specifications and indications of the current regulations applicable to this type of project.

2. Description of the Structure

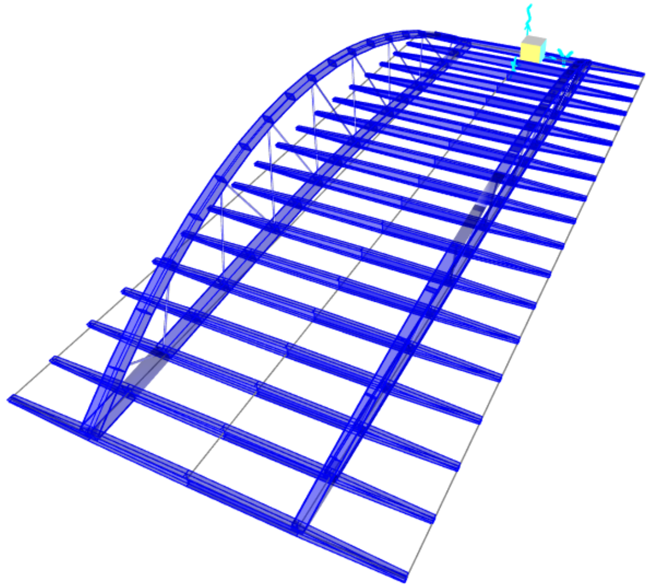


Figure 1: 3D Model of the bridge created in SAP2000 [Own elaboration]

It is a bowstring type bridge with each of its inclined arches connected to its respective longitudinal beam. The loads are transmitted to the arches, which are mainly in compression, through the suspension cables. For gravitational loads, only the vertical components of the forces in the arches are transmitted to the foundations, as the longitudinal beams, which have a tubular section in the form of a parallelogram, cancel out the horizontal components by being in tension.

The deck is made up of a transverse system formed by floor beams and cantilevered side girders, the distance between which is equal to half the longitudinal distance between the hangers, and which have a double T-section. The lateral cantilevers have a linear height reduction, with a total height of 30 cm at the outer ends. Between the girders, corrugated metal sheets are placed, and on the top of the girders, connectors are mounted. A 26 cm thick concrete slab is poured over this area. Thus, the slab acts as a surface and cooperates with the steel structure in terms of resistance.

3. Calculation Basis

3.1. Standards

The following standards have been taken into account for the elaboration of the calculations:

- IAP-11 “Instrucción sobre las acciones a considerar en el proyecto de puentes de carretera” MINISTERIO DE FOMENTO 2011 (Guidance on actions to be considered in the design of road bridges)
- Eurocode 2: EN 1992 Design of Concrete Structures
- Eurocode 3: EN 1993 Design of Steel Structures

3.2. Materials

The materials used in the design and construction of the bridge are as follows:

3.2.1. Steel

Three types of steel are distinguished according to their function:

- Structural steel: S 355 JR
- Reinforcing steel (rebar): B 500-S
- tension rod system (Pfeifer): S460N

All three steels share the following mechanical properties:

Modulus of elasticity	$\mathrm{E} = 200.000\ \mathrm{N/mm2}$
Shear Modulus	$\mathrm{G} = 81000\ \mathrm{N/mm2}$
Thermal expansion coefficient	$\alpha = 12\mathrm{x}10-6\ ^{0}\mathrm{C}$
Density	$\rho = 7850kg/m^3$

3.2.2. Concrete

The concrete used for the slab is C35/45 with the following mechanical characteristics:

Modulus of elasticity	$\mathrm{E} = 30.000\ \mathrm{N/mm2}$
characteristic compressive strength	$fck = 35\ \mathrm{N/mm2}$
Thermal expansion coefficient	α = 10x10-6 ^{0}C
Density	$\rho = 2500 kg/m^3$

4. Loads

To carry out the calculation, the characteristic values of the actions defined in the IAP-11 "Instruction on actions to be considered in the design of road bridges" will be taken into account.

The actions have been divided into permanent and variable actions and differentiated into different subcategories in order to correctly perform the possible combinations.

4.1. Permanent Loads (G)

The permanent loads are acting during the whole service life of the bridge and their magnitude can be determined from the dimensions of the elements detailed in the drawings, together with the values of the corresponding specific weights.

The permanent loads are divided into two main categories: self-weight and dead loads.

4.1.1. Self-weight

These loads are the weight made up of the structural elements. All these elements are later introduced in the program (SAP2000) with their right dimensions and the following specific weights:

- Structural steel: 78,50 kN/m3
- Reinforced concrete: 25,00 kN/m3

4.1.2. Dead Loads

These loads are made up of the weights of all the other permanent elements on the bridge that don't contribute to the load-bearing capacity but are necessary for serviceability. The following dead loads have been taken into account:

- Corrugated sheet metal Eurocol 60 e=1,2mm and concrete slab e=25cm: **5,35 KN/m²**
- Sidewalk pavement: **1,5 KN/m²**
- Pavement e=8cm: [23kN/m³ * 0,08m *1,50=**2,76kN/m²**] (50% increase, rehabilitation etc.)
- Roadway guardrail: **1KN/m**
- Sidewalk Railings: **0,5 KN/m**
- Cornice: **4 KN/m**

4.2. Variable Loads (Q)

From the IAP-11 "Instruction on the actions to be considered in the design of road bridges" we will determine the various variable actions acting on our bridge. These loads are changing and are related to the different activities and uses that the structure may experience.

4.2.1. Exclusion of Actions

Since this is a basic project it has been decided not to include the following actions in the structural calculation:

- Snow: The bridge is located in Bilbao, which is an area with a low probability of snowfall. According to Table 4.4-b of IAP-11, the snow load to be considered in Bilbao is 0.3kN/m². The uniform traffic load is greater than or equal to 2.5kN/m² and the loads are not considered to be acting simultaneously. Therefore, the uniformly distributed traffic loads are more unfavorable to the Bridge than the snow load that corresponds to it.
- Accidental actions: Impact forces are not considered, and seismic forces neither, as explained in the Appendix "Geotechnical Study".

4.2.2. Traffic Loads

Since this is a bridge with a span of less than 200 m, we will apply Section 4.1.1 of IAP-11. This section specifies the method for dividing the deck platform into virtual lanes. As shown in the following figure, the roadway is divided into two parts by guardrails:

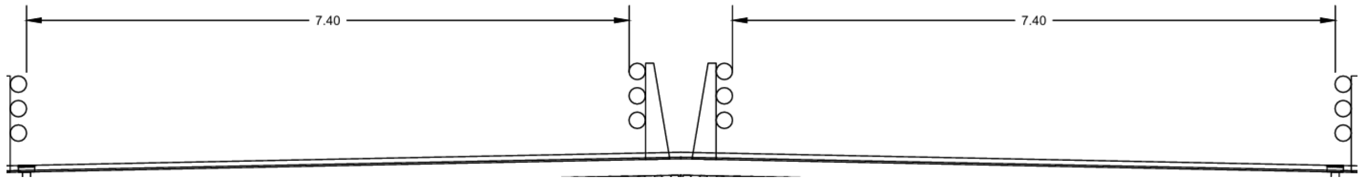


Figure 2: Roadway divided by guardrails [Own elaboration]

Since the distance between the guardrails on both sides is 7,4 m, for a total of 14,8 m, we will have 4 virtual lanes with a width of 3m each and a remaining area of 1,4 m on both sides. *It should be noted that the final distance between the guardrails was changed to 7.20 m because the cross section of the arches had to be widened due to the calculation results that follow in this appendix. Since the initial consideration is less favorable, it will not be changed.*

Where exactly which virtual lane is placed depends on which element is being analyzed. To verify an element, the loads are distributed to be as unfavorable as possible for that element.

Depending on the area, uniform loads and point loads from heavy vehicles are applied, as shown in the following table.

SITUACIÓN	VEHÍCULO PESADO $2Q_{ik}$ [kN]	SOBRECARGA UNIFORME q_{ik} (ó q_{rk}) [kN/m^2]
Carril virtual 1	2 · 300	9,0
Carril virtual 2	2 · 200	2,5
Carril virtual 3	2 · 100	2,5
Otros carriles virtuales	0	2,5
Área remanente (q_{rk})	0	2,5

Figure 3: Heavy vehicles and uniform loads for virtual lanes and remaining area [IAP-11]

Each uniform load is applied over the entire respective virtual lane.

The point loads simulating the heavy vehicles have the following characteristics:

- In each virtual lane, the impact of a single heavy vehicle of weight 2Qik will be considered.
- The transverse separation between wheels of the same axis will be 2.00 m. The longitudinal distance between axis will be 1.20 m (See figure 5).
- The forces of the wheels on each axis have the same load, which is therefore equal to 0.5Qik. For the general checks, it will be assumed that each heavy vehicle acts centered in the virtual lane (see figure 4).
- For local checks, each heavy vehicle shall be placed, transversely within each virtual lane, in the most unfavorable position. When two heavy vehicles are considered in adjacent virtual lanes, they may approach each other transversely, maintaining a distance between wheels greater than or equal to 0.50 m (see figure 5).
- For local checks, the point load of each wheel of a heavy vehicle shall be assumed uniformly distributed on a square contact surface of 0.4 m x 0.4 m (see figure 5).

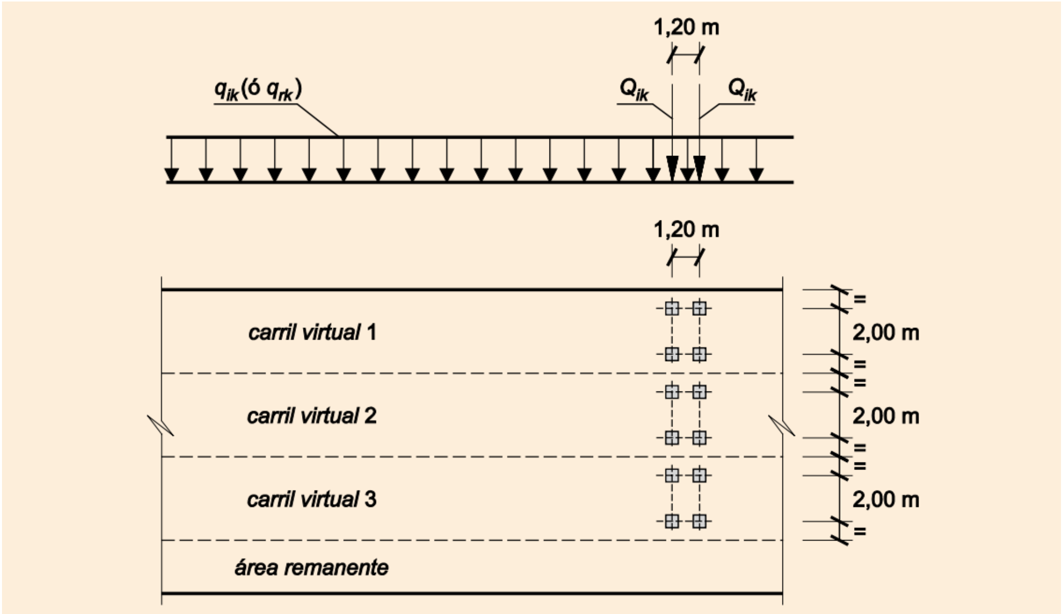


Figure 4: Distribution of heavy vehicles and uniform loads [IAP-11]

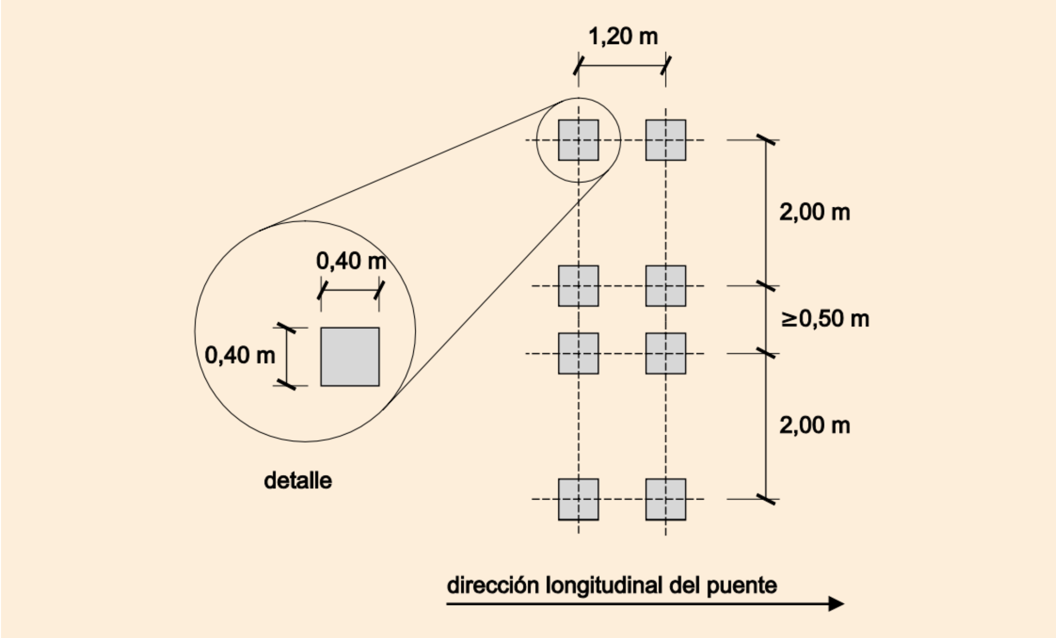


Figure 5: Load distribution of each wheel for local checks [IAP-11]

4.2.3. Loads in the Pedestrian Area

According to article 4.1.2.2 of IAP-11 "Instruction on actions to be considered in the design of road bridges", in the pedestrian areas of bridges (sidewalks, ramps and stairs), a uniform overload of 5 kN/m2 is assumed to be applied in the most unfavorable areas, longitudinally and transversally.

4.2.4. Wind Loads

For the elaboration of the wind loads, the sections of IAP-11 "Instruction on actions to be considered in the design of road bridges" are followed from 4.2.1 to 4.2.8, where the wind action is transformed into equivalent static loads. According to Section 4.2.9 of IAP-11, it is not necessary to consider the aeroelastic effects of the bridge since it has a span of only 76,5m, which is less than 80m.

First, the following formula is used to calculate the basic wind speed with a return period (T) of 100 years:

$$v_b = c_{dir} \times c_{season} \times v_{b,0} = 1 \times 1 \times 29\frac{m}{s} = 29\frac{m}{s}$$

$$v_b(100) = v_b \times c_{prob} = 29 \times 1{,}04 = 30{,}16\frac{m}{s}$$

Where:

V_b basic wind speed for a return period of 50 years [m/s]

C_{dir} directional wind factor which, in the absence of more detailed studies, can be taken to be equal to 1,0

c_{season} seasonal wind factor which, in the absence of more detailed studies, can be taken to be equal to 1,0

$v_{b,0}$ fundamental basic wind speed [m/s] (see figure 6)

$v_b(T)$ basic wind speed for a return period T

c_{prob} probability factor; for persistent situations, in the absence of specific studies, a return period of 100 years is considered (c_{prob}=1,04)

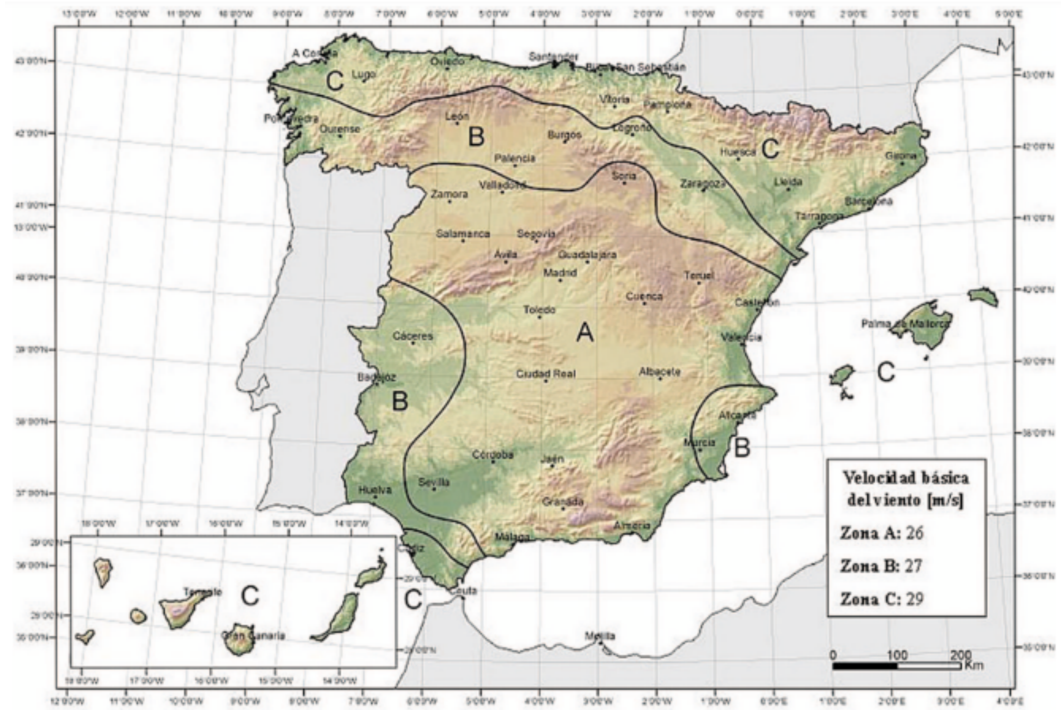


Figure 6: Isotach map for obtaining the basic fundamental wind speed $v_{b,0}$ [IAP-11]

Bilbao is located in Zone C and therefore $v_{b,0}$ is 29m/s.

4.2.4.1. Horizontal Force on the Deck

To calculate the wind force (F_w) on the deck, the following formula from section 4.2.3 of IAP-11 is used:

$$F_{w,x} = \left[\frac{1}{2} \times \rho \times v_b(T)^2\right] \times c_e(z) \times c_f \times A_{ref}$$

To obtain a linear force the reference area Aref has been replaced by the height of the deck h_{eq}.

$$\frac{F_{w,x}}{L} = \left[\frac{1}{2} \times \rho \times v_b(T)^2\right] \times c_e(z) \times c_f \times heq$$

Where:

F_w	Horizontal wind force (N)
$\frac{1}{2}\,\rho\, v_b(T)^2$	Basic wind speed pressure q_b (N/m2)
ρ	Density of air, to be taken equal to 1.25 kg/m3
$v_b(T)$	Basic wind speed for a return period T (m/s)
C_f	Force coefficient
A_{ref}	Reference area, obtained as the projection of the exposed solid area on the plane perpendicular to the direction of the wind (m^2)
$C_e(z)$	Exposure coefficient as a function of height Z. It is obtained from the following formula:

$$c_e(z) = k_r{}^2\left[c_0{}^2 \times ln^2\left(\frac{z}{z_0}\right) + 7 \times k_l \times c_0 \times \ln\left(\frac{z}{z_0}\right)\right] \; for\ z \geq z_{min}$$

$$c_e(z) = c_e(z_{min}) \; for\ z \leq z_{min}$$

Where:

k_r	terrain factor obtained from figure 7
k_l	turbulence factor, to be taken equal to 1,0
z_0, z_{min}	coefficients with value obtained from figure 7
c_0	topography factor which is usually taken as 1,0
z	height of the point of application of the wind force with respect to the water level under the bridge; the point of application is at a height of 60% of the height of the deck h_{eq} (z=0,60*1,40m+2,80m=3,64m)

TIPO DE ENTORNO	k_r	z_0 [m]	z_{min} [m]
0	0,156	0,003	1
I	0,170	0,01	1
II	0,190	0,05	2
III	0,216	0,30	5
IV	0,235	1,00	10

Figure 7: Coefficients k_r, z_0 and z_{min} according to type of environment [IAP-11]

We are in a type IV environment (urban area in which at least 15% of the surface area is built-up and the average building exceeds 15m), therefore:

$$z = 3{,}64m \; \leq z_{min} = 10m$$

$$c_e(z_{min}) = 0{,}235^2\left[1^2 \times ln^2\left(\frac{10}{1}\right) + 7 \times 1 \times 1 \times \ln\left(\frac{10}{1}\right)\right] = 1{,}183$$

The following formula is used to obtain the force coefficient of the deck in accordance with Section 4.2.5.1 of the IAP-11:

$$c_{f,x} = 2{,}5 - 0{,}3 \times \frac{B}{h_{eq}} = 2{,}5 - 0{,}3 \times \frac{34{,}77m}{1{,}40m} = -4{,}95$$

$$1{,}3 \leq c_{f,x} \leq 2{,}4$$

$$c_{f,x} = 1{,}3$$

Where:

B	total width of the deck
h_{eq}	total height of the deck

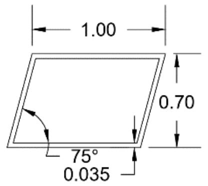
Substituting these values into the formula for the horizontal wind force per meter, we obtain:

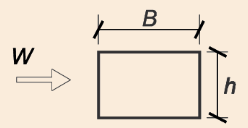
$$\frac{F_{w,x}}{L} = \left[\frac{1}{2} \times 1{,}25\frac{kg}{m^3} \times \left(30{,}16\frac{m}{s}\right)^2\right] \times 1{,}183 \times 1{,}3 \times 1{,}40m = 1224{,}05\frac{N}{m} = 1{,}224\frac{KN}{m}$$

4.2.4.2. Horizontal Force on the Arch

The same formula is used to calculate the force per meter on the arch. The value for the force coefficient is changed and the height of the deck is replaced by the height of the arch.

The force coefficient is approximated by Table 4.2-b of the IAP-11. The arch is considered as a rectangular section with a ratio of B/h=1.43.





B/h	≤ 0,2	0,4	0,6	0,7	1,0	2,0	5,0	≥ 10,0
c_f	2,0	2,2	2,35	2,4	2,1	1,65	1,0	0,9

Figure 8: Force coefficient c_f for the arch [IAP-11]

By interpolation we get:

$$c_f = 2{,}1 + \frac{1{,}65 - 2{,}1}{2 - 1} \times (1{,}43 - 1) = 1{,}91$$

Substituting these values into the formula for the horizontal wind force per meter, we obtain:

$$\frac{F_{w,arch}}{L} = \left[\frac{1}{2} \times 1{,}25 \frac{kg}{m^3} \times \left(30{,}16 \frac{m}{s}\right)^2\right] \times 1{,}183 \times 1{,}91 \times 0{,}70m = 899{,}21 \frac{N}{m} = 0{,}899 \frac{KN}{m}$$

4.2.4.3. Vertical Uplift

For the calculation of the vertical uplift, which acts in the most unfavorable direction (pressure or suction), Article 4.2.5.1.2 of the IAP-11 is applied.

$$F_{w,z} = \left[\frac{1}{2} \times \rho \times v_b(T)^2\right] \times c_e(z) \times c_{f,z} \times A_{ref}$$

To obtain F_w as a uniform load rather than a point load, A_{ref} has been eliminated from the formula.

$$\frac{F_{w,z}}{A_{ref}} = \left[\frac{1}{2} \times \rho \times v_b(T)^2\right] \times c_e(z) \times c_{f,z}$$

Where:

$F_{w,z}$	vertical wind force (N)
$\frac{1}{2}\,\rho\, v_b(T)^2$	Basic wind speed pressure q_b (N/m2)
ρ	Density of air, to be taken equal to 1.25 kg/m3
$v_b(T)$	Basic wind speed for a return period T (m/s)
C_f	Coefficient of force in Z direction, to be taken equal to ±0.9
A_{ref}	Area of the deck(m2)
$C_e(z)$	Exposure coefficient as a function of height Z

$$\frac{F_{w,z}}{A_{ref}} = \left[\frac{1}{2} \times 1{,}25 \frac{kg}{m^3} \times \left(30{,}16 \frac{m}{s}\right)^2\right] \times 1{,}183 \times 0.9 = 605{,}30 \frac{N}{m^2} = 0{,}605 \frac{KN}{m^2}$$

4.2.5. Thermal Action

The first step is to determine the type of deck. The load-bearing structure is analyzed as a pure metal structure and as a conservative approximation the contribution of the concrete slab is not taken into account. Therefore, according to IAP-11 Article 4.3, the deck used is Type 1 (steel deck).

In order to take into account, the influence of the thermal action, four different cases are considered:

- Maximum variation of the uniform temperature component resulting in contraction
- Maximum variation of the uniform temperature component resulting in dilatation
- Vertical temperature difference, arches heat up +15° relative to the rest of the structure (deck)
- Vertical temperature difference, deck heats up by +15° relative to the rest of the structure (arches)

The uniform temperature component and the associated uniform temperature decrease, and increase are calculated in the following steps:

4.2.5.1. Maximum and Minimum Air Temperature

The characteristic value of the maximum shaded air temperature depends on the climate of the site and the altitude. For a return period of 50 years and the location in Bilbao, the maximum air temperature is T_{max}=45°C according to the following isotherm map.

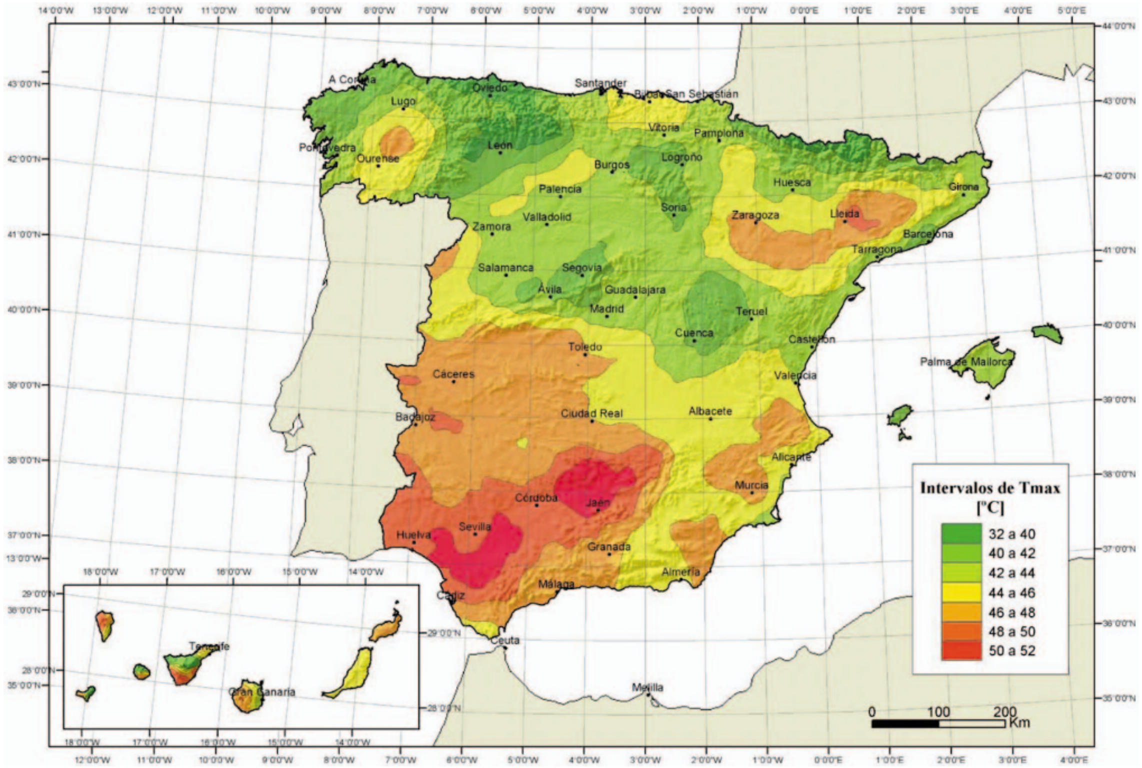


Figure 9: Maximum annual air temperature, T_{max} [IAP-11]

The characteristic value of the minimum shaded air temperature, for a return period of 50 years, will be taken from the following table depending on the altitude of the site and the winter climatic zone derived from the map of the winter climatic zones. Since Bilbao is located in Zone 1 and the bridge is only a few meters above sea level, the minimum shaded air temperature is Tmin=-7°C according to the following figures:

ALTITUD (m)	ZONA DE CLIMA INVERNAL (SEGÚN FIGURA 4.3-b)						
	1	2	3	4	5	6	7
0	-7	-11	-11	-6	-5	-6	6
200	-10	-13	-12	-8	-8	-8	5
400	-12	-15	-14	-10	-11	-9	3
600	-15	-16	-15	-12	-14	-11	2
800	-18	-18	-17	-14	-17	-13	0
1000	-20	-20	-19	-16	-20	-14	-2
1200	-23	-21	-20	-18	-23	-16	-3
1400	-26	-23	-22	-20	-26	-17	-5
1600	-28	-25	-23	-22	-29	-19	-7
1800	-31	-26	-25	-24	-32	-21	-8
2000	-33	-28	-27	-26	-35	-22	-10

Figure 10: Minimum annual air temperature Tmin and winter climatic zones [IAP-11]

For return periods different from 50 years, the values should be adjusted according to the following equations:

$$T_{max,p} = T_{max}\{k_1 - k_2\, ln[-\ln(1-p)]\}$$

$$T_{min,p} = T_{min}\{k_3 - k_4\, ln[-\ln(1-p)]\}$$

Where p is the inverse of the return period and considering for the coefficients the values: k_1=0,781, k_2=0,056, k_3=0,393, k_4=0,156. For persistent situations, a return period of 100 years will be considered (p=0,01).

$$T_{max,p} = 45°C\{0{,}781 - 0{,}056\, ln[-\ln(1-0{,}01)]\} = 46{,}74°C$$

$$T_{min,p} = -7°C\{0{,}393 - 0{,}156\, ln[-\ln(1-0{,}01)]\} = -7{,}77°C$$

4.2.5.2. Uniform Temperature Component

The uniform component of the deck temperature (average temperature of the cross section), will have a minimum value $T_{e,min}$ and a maximum value $T_{e,max}$ to be determined from the air temperature, plus the values $\Delta T_{e,}$min=-3°C and $\Delta T_{e,max}$=+16C, which depend on the type of deck.

$$T_{e,max} = T_{max,p} + \Delta T_{e,max} = 46{,}74°C + 16°C = 62{,}74°C$$

$$T_{e,min} = T_{min,p} + \Delta T_{e,min} = -7{,}77°C - 3°C = 10{,}77°C$$

From the maximum and minimum values of the uniform temperature component and from the initial temperature T_0 (average temperature of the deck which will be taken equal to T_0=15°C), the thermal variation ranges in contraction and dilatation will be obtained.

Uniform temperature decrease (contraction):

$$\Delta T_{N,con} = T_0 - T_{e,min}$$

$$\Delta T_{N,con} = 15°C - (-10{,}77°C) = 25{,}77°C$$

Uniform temperature increase (expansion):

$$\Delta T_{N,exp} = T_{e,max} - T_0$$

$$\Delta T_{N,exp} = 62{,}74°C - 15°C = 47{,}74$$

5. Limit States

In the following section, the way the loads have been combined is defined so that the least favorable combination is considered in the ultimate and serviceability limit states. Due to the scope of the project only persistent design situations are considered, which relate to the regular use of the structure throughout its design life. Transient design situations that occur during construction, inspection, or maintenance, as well as accidental design situations, are neglected.

5.1. Partial Factors

The following tables show the partial factors for the ultimate limit state and the serviceability limit state according to Section 6.2 of IAP-11:

Action (ULS)		favorable	unfavorable
Permanent loads	Self-weight	1,00	1,35
	Dead loads	1,00	1,35
Variable loads	Live loads	0,00	1,35
	climate actions	0,00	1,50

Figure 11: Partial factors (γ_i) for ULS [Own elaboration; values derived from: IAP-11]

Action (SLS)		favorable	unfavorable
Permanent loads	Self-weight	1,00	1,00
	Dead loads	1,00	1,00
Variable loads	Live loads	0,00	1,00
	climate actions	0,00	1,00

Figure 12: Partial factors (γ_i) for SLS [Own elaboration; values derived from: IAP-11]

5.2. Combination Coefficients

The combination coefficients, taken from Section 6.1 of the IAP-11, are shown in the following table:

Action	ψ_0	ψ_1	ψ_2
Heavy vehicles	0,75	0,75	0,00
Uniform load (roadway and pedestrian area)	0,40	0,40	0,00
Wind loads	0,60	0,20	0,00
Thermal action	0,60	0,60	0,50

Figure 13: Combination coefficients ψ [Own elaboration; values derived from: IAP-11]

5.3. Load Combinations

The following formula is used for the combination of actions for verifications in the Ultimate Limit State in Persistent Situations in accordance with Section 6.1.1.1 of IAP-11.

$$\sum_{j\geq 1} \gamma_{G,j}\, G_{k,j} + \sum_{m\geq 1} \gamma_{G,m}\, G^*_{k,m} + \gamma_{Q,1}\, Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{0,i}\, Q_{k,i}$$

Where:

γ	Partial factors previously defined
ψ_0	Factor for combination value of a variable action
$G_{k,j}$	Characteristic value of permanent action j
$G_{k;m}$	Characteristic value of each permanent action of non-constant value
$Q_{k,1}$	Characteristic value of the leading variable action 1
$Q_{k,i}$	Characteristic value of the accompanying variable action i

For the frequent combination of actions for reversible Serviceability Limit State verifications, the following formula is used in accordance with Section 6.3.2 of IAP-11.

$$\sum_{j\geq 1} \gamma_{G,j}\, G_{k,j} + \sum_{m\geq 1} \gamma_{G,m}\, G^*_{k,m} + \gamma_{Q,1}\psi_{1,1}\, Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{2,i}\, Q_{k,i}$$

Where:

ψ_1 Factor for frequent value of a variable action

ψ_2 Factor for quasi-permanent value of a variable action

In addition, according to IAP-11, the following regulations must be considered for both limit states:

- for permanent actions of constant value G, the coefficients γ_G=1.0 and γ_G=1.35 will be applied to the whole of the action of the same origin, depending on whether its total effect is favorable or unfavorable
- When considering the transversal wind on the deck, the simultaneous action of the vertical component of the wind and the corresponding moment, as defined in section 4.2.5.1. (IAP-11), is considered.
- The interaction of the uniform temperature component and the temperature difference component is governed by the provisions of section 4.3.1.3. (IAP-11)
- When the action of the wind is considered as predominant, the action of the live load will not be taken into account.
- If the live load is considered to be the predominant load, the corresponding accompanying wind shall be considered with the indications given in Section 4.2.3. (IAP-11)
- The simultaneous action of wind and thermal action will not be considered.
- In general, the simultaneous action of snow load and live load will not be considered except in high mountain areas.

6. Structural Analysis

6.1. Calculation Method

Two models are used to verify the different components of the bridge. First the transversal beams are dimensioned using a local model, then the remaining elements are checked using a global model. The arches are checked for buckling, as this is the most critical component. The SAP2000 program is used for these models.

6.2. Local Model for Verification of Transversal Beams

The local model consists of a simple single-span girder with two cantilever arms. The supports represent the longitudinal beams, which in turn are held by the cables and transfer the forces to the arches. This very simplified model of one transverse element is used to pre-dimension the 19 transverse elements of the bridge. However, these are verified again in the global model.

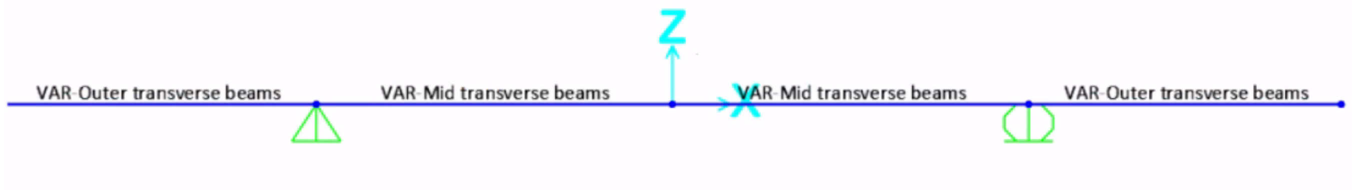


Figure 14: Local model in SAP2000 [Own elaboration]

6.2.1. Highest Bending Moment in the Center of the Floor Beams

To obtain the maximum bending moment in the center of the floor beams, the traffic loads are applied as centered as possible. For local checks, each heavy vehicle shall be placed, transversely within each virtual lane, in the most unfavorable position. This results in the following distribution:

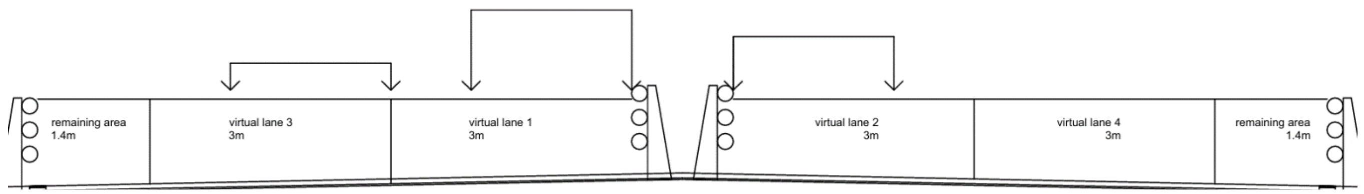


Figure 15: Division of the virtual lanes, the remaining area and placement of the heavy vehicles [Own elaboration]

The values for the loads introduced in SAP2000 are shown in the following table:

Highest bending moment in floor beam		
Load Pattern	**Description**	**Calculation**
SW (Self Weight)	self weight of the frames	considered in SAP2000
DL slab (Dead Loads)	Floor beams	5,35KN/m^2 *4,25m = 22,74KN/m
	Cantilevered beams	5,35KN/m^2 *4,25m = 22,74KN/m
DL other (Dead Loads)	Sidewalk pavement	1,5KN/m^2 *4,25m = 6,38KN/m
	Pavement	2,76KN/m^2 *4,25m = 11,73KN/m
	Roadway guardrail	1KN/m *4,25m = 4,25KN
	Sidewalk Handraiils	0,5KN/m *4,25m = 2,125KN
	Cornice	4KN/m*4,25m = 17KN
UTL (Uniform Trafic Loads)	Virtual lane 1	9KN/m^2 *4,25m = 38,25KN/m
	Virtual lane 2,3,4 and remaining area	2,5KN/m^2 *4,25m = 10,625KN/m
	Pedestrian Area	/
HVL (1 Axis) (Heavy Vehicle Loads)	Virtual lane 1	2 * 150KN *(3,65m/4,25m) = 257,65KN
	Virtual lane 2	2 * 100KN *(3,65m/4,25m) = 171,76KN
	Virtual lane 3	2 * 50KN *(3,65m/4,25m) = 85,88KN

Figure 16: Calculation of the loads introduced into SAP2000 [Own elaboration]

The following cross-section properties are required to perform the verifications:

$Z_{33} = 0{,}0229 m^3 = 22900 cm^3$ Plastic section modulus about y-axis (Wy_{pl})

$Z_{22} = 4{,}133 \times 10^{-3} m^3 = 4133 cm^3$ Plastic section modulus about z-axis (Wz_{pl})

Since the pipelines are to pass through the center of the crossbeams, part of the web is cut out. This is possible because very low shear forces occur here, and the bending moment is mainly transferred through the flanges. The floor beams have their greatest height in the middle with a total of 1.27 m. The stiffening metal plates around the cutout area are dimensioned so that the area moment of inertia is the same as with a continuous web, as seen in the following:

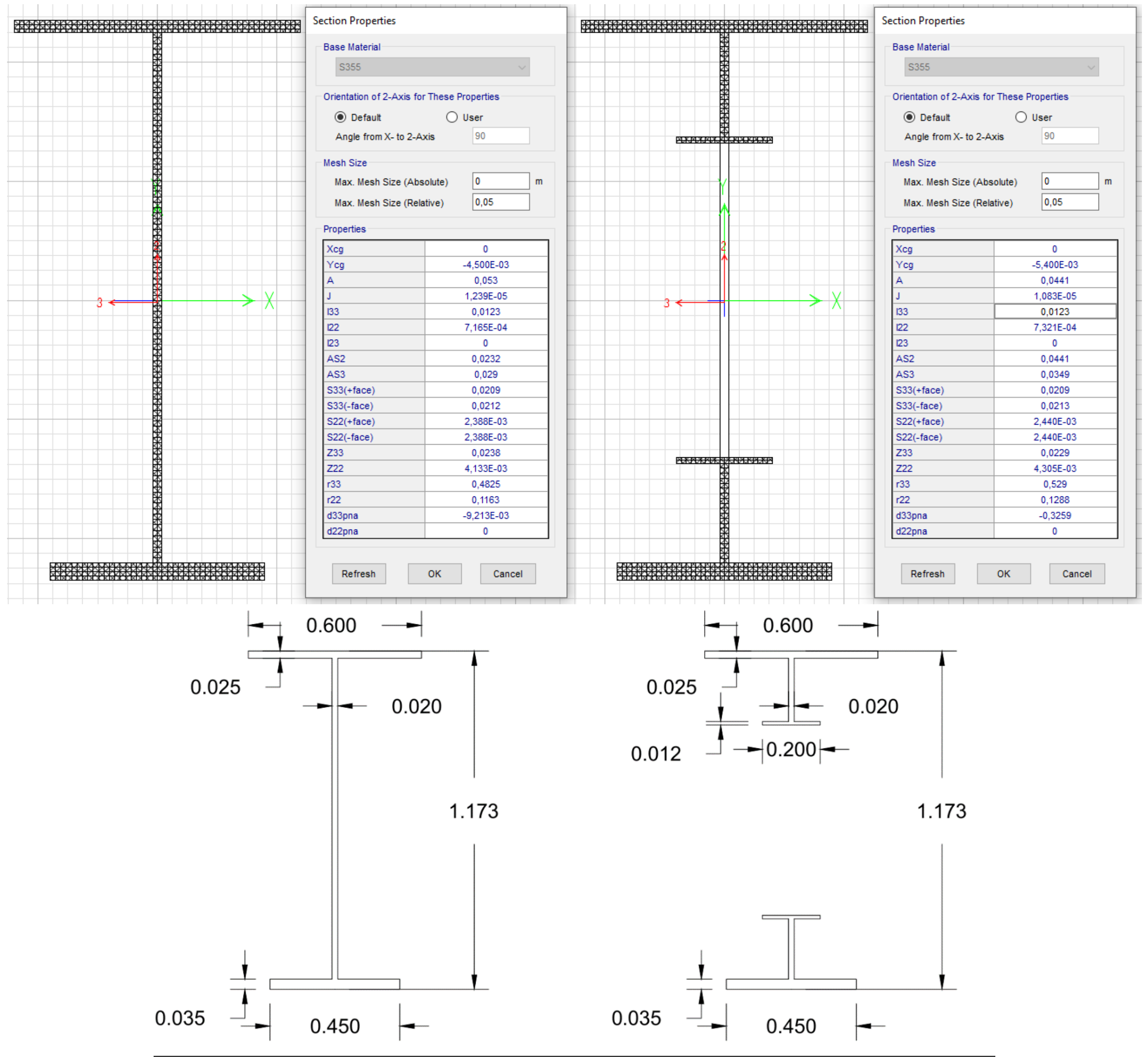


Figure 17: Cross section of the transversal beam in the center and the corresponding section properties [Own elaboration]

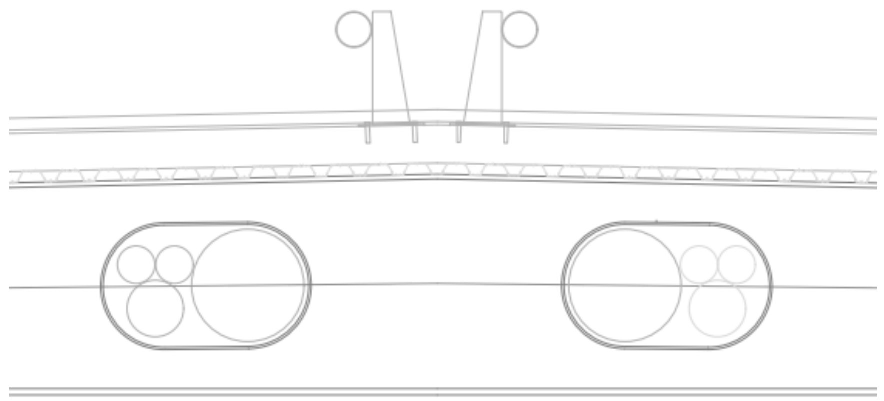
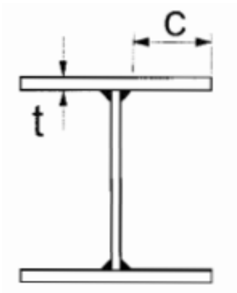


Figure 18: Cutout Area for Pipelines [Own elaboration]

Table 5.2 (sheet 2 of 3) of Eurocode 2 (EN 1993-1-1) is used to select the class of the cross section:

The lower flange is the most conditioning, since the upper flange is connected to the concrete slab by connectors, stiffening this part of the section.



$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0{,}81$$

$$\frac{c}{t} = \frac{215mm}{35mm} = 6{,}14$$

$$9 \times \varepsilon = 9 \times 0{,}81 = 7{,}29$$

$$6{,}14 \leq 7{,}29$$

→ Class 1

The bending moment under a combination of all the load patterns described above with the associated safety factors is calculated using SAP2000. The maximum value is at a distance of 8.37m from the left support and is $M_{y,Ed}$ =6.405,72KNm and $M_{z,Ed}$=0.

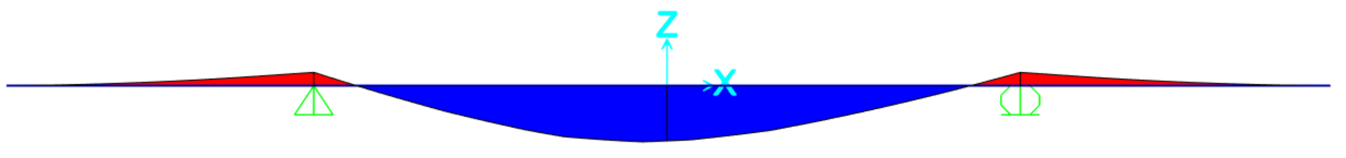


Figure 19: Bending Moment Diagram [Own elaboration; SAP2000]

<u>Verification of bending moment:</u> (According to article 6.2.5 of EN 1993-1-1)

The design value of the bending moment M_{Ed} for class 1 cross sections shall satisfy:

$$M_{Ed} \leq M_{c,Rd}$$

Where:

M_{Ed} Design bending moment

$M_{c,Rd}$ Design resistance for bending, calculated using the following formula:

$$My_{c,Rd} = My_{pl,Rd} = \frac{Wy_{pl} \times f_y}{\gamma_{M0}} = \frac{0{,}0229m^3 \times 355.000\frac{kN}{m^2}}{1{,}00} = 8.129{,}5kNm$$

$$Mz_{c,Rd} = Mz_{pl,Rd} = \frac{Wz_{pl} \times f_y}{\gamma_{M0}} = \frac{4{,}133 \times 10^{-3}m^3 \times 355.000\frac{kN}{m^2}}{1{,}00} = 1.467{,}215kNm$$

$$M_{y,Ed} = 6.405{,}71kNm \leq My_{c,Rd} = 8.129{,}5kNm$$

$$M_{y,Ed} \approx 0kNm \leq My_{c,Rd} = 1.467{,}215kNm$$

6.2.2. Highest Shear Force in the Floor Beams and Highest Bending Moment in the Cantilevered Girder

To obtain the highest shear force of the floor beams right next to the arch, the traffic loads are applied as close to the left arch as possible, and the left pedestrian area is loaded. This results in the following distribution:

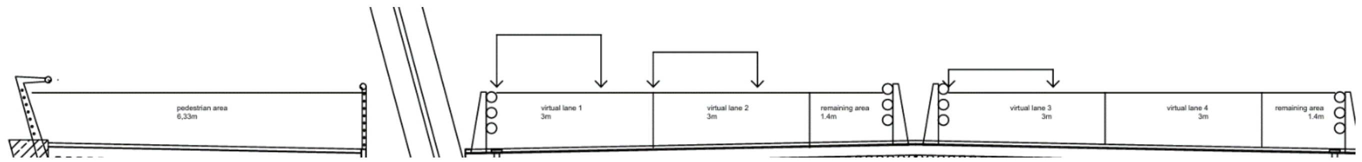


Figure 20: Division of the virtual lanes, the remaining area, pedestrian area and the placement of the heavy vehicles [Own elaboration; Autodesk AutoCAD]

The values for the loads introduced in SAP2000 are shown in the following table:

Highest shear force in floor beam & highest bending moment in cantilever		
Load Pattern	Description	Calculation
SW (Self Weight)	self weight of the frames	considered in SAP2000
DL slab (Dead Loads)	Floor beams	5,35KN/m² *4,25m = 22,74KN/m
	Cantilevered beams	5,35KN/m² *4,25m = 22,74KN/m
DL other (Dead Loads)	Sidewalk pavement	1,5KN/m² *4,25m = 6,38KN/m
	Pavement	2,76KN/m² *4,25m = 11,73KN/m
	Roadway guardrail	1KN/m *4,25m = 4,25KN
	Sidewalk Handraiils	0,5KN/m *4,25m = 2,125KN
	Cornice	4KN/m*4,25m = 17KN
UTL (Uniform Trafic Loads)	Virtual lane 1	9KN/m² *4,25m = 38,25KN/m
	Virtual lane 2,3,4 and remaining area	2,5KN/m² *4,25m = 10,625KN/m
	Pedestrian Area	5KN/m² *4,25m = 21,25KN/m
HVL (1 Axis) (Heavy Vehicle Loads)	Virtual lane 1	2 * 150KN *(3,65m/4,25m) = 257,65KN
	Virtual lane 2	2 * 100KN *(3,65m/4,25m) = 171,76KN
	Virtual lane 3	2 * 50KN *(3,65m/4,25m) = 85,88KN

Figure 21: Calculation of the loads introduced into SAP2000 [Own elaboration]

The bending moment and shear forces under a combination of all the load patterns described above with the associated safety factors is calculated using SAP2000. The maximum negative value for the bending moment is right next the the left support and is $M_{y,Ed}$=-2377kNm. The maximum shear force next to the support is $V_{z,Ed}$=-1762,09kN.

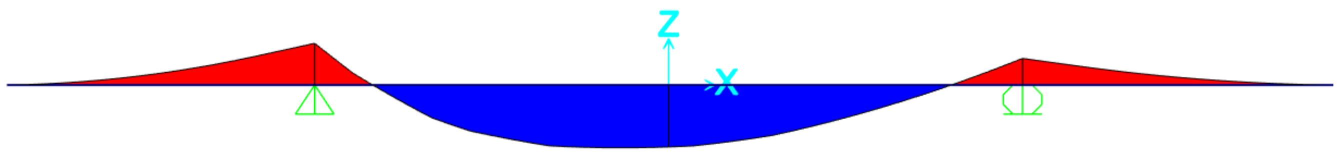


Figure 22: Bending Moment Diagram [Own elaboration; SAP 2000]

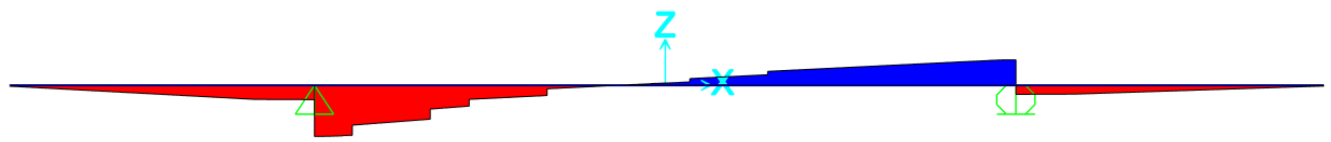


Figure 23: Shear Force Diagram [Own elaboration; SAP 2000]

The following cross-section properties are required to perform the verifications:

$Z_{33} = 0{,}0193m^3 = 19300cm^3$ Plastic section modulus about y-axis (Wy_{pl})

$Z_{22} = 4{,}116 \times 10^{-3}m^3 = 4116cm^3$ Plastic section modulus about z-axis (Wz_{pl})

$A_V = 200cm^2$ Shear area

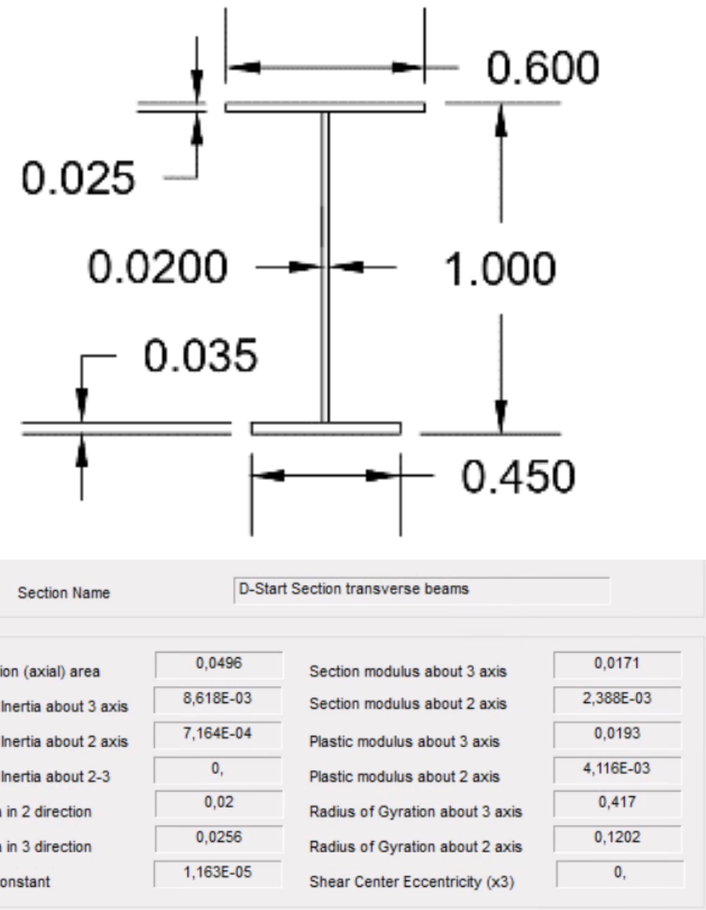


Figure 24: Cross section of the transversal beam where it joins the longitudinal beam and the corresponding section properties [Own elaboration; SAP 2000]

Verification of bending moment: (According to article 6.2.5 of EN 1993-1-1)

The design value of the bending moment M_{Ed} for class 1 cross sections shall satisfy:

$$M_{Ed} \leq M_{c,Rd}$$

Where:

M_{Ed} design bending moment

$M_{c,Rd}$ design resistance for bending, calculated using the following formula:

$$My_{c,Rd} = My_{pl,Rd} = \frac{Wy_{pl} \times f_y}{\gamma_{M0}} = \frac{0{,}0193m^3 \times 355.000\frac{kN}{m^2}}{1{,}00} = 6.851{,}5kNm$$

$$Mz_{c,Rd} = Mz_{pl,Rd} = \frac{Wz_{pl} \times f_y}{\gamma_{M0}} = \frac{4{,}116 \times 10^{-3} m^3 \times 355.000 \frac{kN}{m^2}}{1{,}00} = 1.461{,}18 kNm$$

$$M_{y,Ed} = 2.377\text{kNm} \leq My_{c,Rd} = 6.851{,}5 kNm$$

$$M_{y,Ed} \approx 0 kNm \leq My_{c,Rd} = 1.461{,}18 kNm$$

Verification of shear stresses:

The shear force is mainly transferred through the webs. Therefore, according to EN 1933-1-1, the shear transfer can be approximated as being applied only through the web of the section. It is also assumed that the shear distribution is constant.

$$V_{Ed} \leq V_{c,Rd}$$

Where:

V_{Ed} design shear force

$V_{c,Rd}$ design shear resistance, calculated using the following formula:

$$V_{c,Rd} = V_{p,Rd} = \frac{A_v \times f_y}{\sqrt{3} \times \gamma_{M0}} = \frac{200 cm^2 \times 35{,}5 \frac{kN}{cm^2}}{\sqrt{3} \times 1{,}00} = 4099{,}19 kN$$

$$V_{Ed} = 1762{,}09\text{kN} \leq V_{c,Rd} = 4099{,}19 kN$$

6.3. Global Model

Once verified the dimensions of the floor beams and side cantilevers the global model is now used to verify the remaining components, to check again the design of transverse structural elements (floor beams and side cantilevers), to obtain the horizontal deformations for the dimensioning of the expansion joints and to obtain the support reactions for the selection of bearing pads and the calculation of the foundations. The global model reflects the entire metal load-bearing structure.

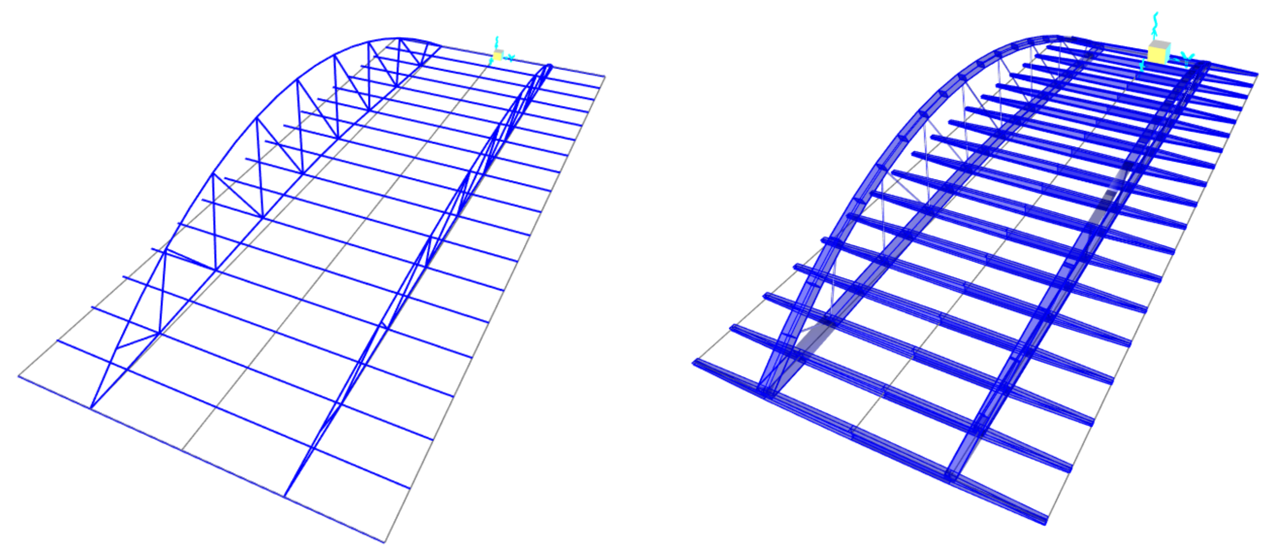


Figure 25: Global Model [Own elaboration; SAP 2000]

The concrete slab is introduced into SAP as a membrane with a thickness of 26cm. Like this, this area section stiffens the structure in the horizontal plane, but its capacity to benefit the transverse beams under bending moment is neglected. The assigned material is the concrete specified above without weight, as the weight is applied as an external force (DL slab).

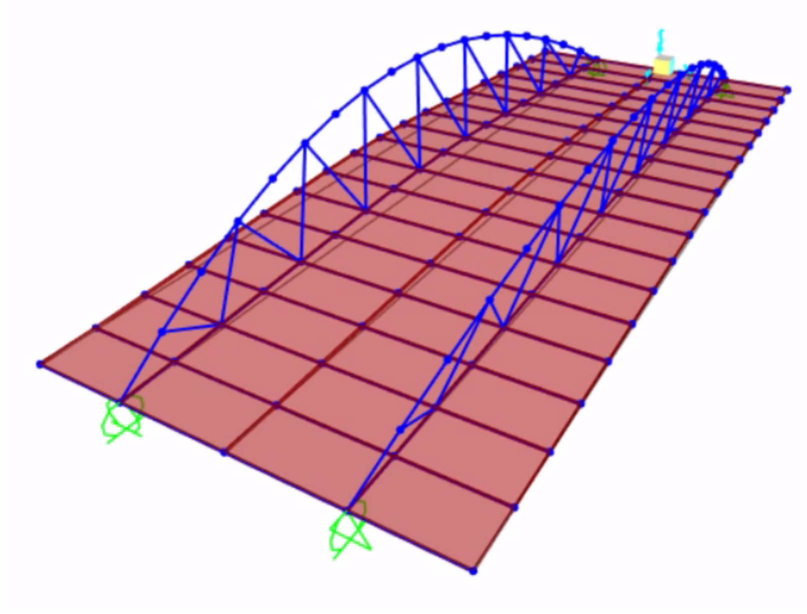


Figure 26: Concrete slab introduced as a membrane [Own elaboration; SAP 2000]

Again, placing the virtual lanes as close as possible to one of the arches is the least favorable option and creates the greatest forces in that arch and its corresponding longitudinal beam as well as in the tension rods. Since this is a global model, the heavy vehicles are each placed in the center of the corresponding virtual lane. The following drawing illustrates the layout:

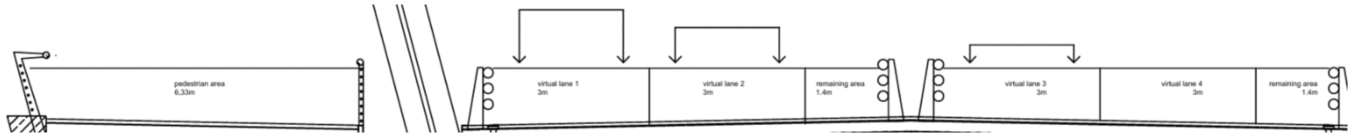


Figure 27: Division of the virtual lanes, the remaining area, pedestrian area and the placement of the heavy vehicles for the global model [Own elaboration; Autodesk AutoCAD]

To find the most unfavorable longitudinal distribution for the verification of the different elements, various load patterns are created and introduced into SAP2000. The self-weight and dead loads are permanent loads and are represented by the following load patterns:

- Self-weight: SW
- Dead load of the slab: DL-Slab
- Rest of the dead loads: DL-Other

The variable load patterns for the pedestrian area, the traffic loads and the heavy vehicles are each divided into 19 load patterns, since there are 19 transversal beams:

- Uniform load in the right pedestrian area: ULPA-right-TB1, ULPA-right-TB2, ..., ULPA-right-TB19
- Uniform load in the left pedestrian area: ULPA-left-TB1, ULPA-left-TB2, ..., ULPA-left-TB19
- Uniform traffic load: UTL-TB1, UTL-TB2, ..., UTL-TB19
- Heavy vehicle load: HVL-TB1, HVL-TB2, ..., HVL-TB19

The load patterns introduced into SAP with their corresponding line and point loads on the transversal beams 2 to 18 are shown in the following table:

All beams except outer two beams (Transversal Beam 2 - Transversal Beam 18)			
Load Pattern	Abbreviation stands for	description	Calculation
SW	Self Weight	self weight of the frames	considered in SAP2000
DL slab	Dead Loads Slab	Floor beams	5,35KN/m^2 *4,25m = 22,74KN/m
		Cantilevered beams	5,35KN/m^2 *4,25m = 22,74KN/m
DL other	Dead Loads Other	Sidewalk pavement	1,5KN/m^2 *4,25m = 6,38KN/m
		Pavement	2,76KN/m^2 *4,25m = 11,73KN/m
		Roadway guardrail	1KN/m *4,25m = 4,25KN
		Sidewalk Handraiils	0,5KN/m *4,25m = 2,125KN
		Cornice	4KN/m*4,25m = 17KN
UTL-TB2, ..., UTL-TB18	Uniform Trafic Load-Transversal Beam	Uniform trafic load virtual lane 1	9KN/m^2 *4,25m = 38,25KN/m
		Virtual lane 2,3,4 and remaining area	2,5KN/m^2 *4,25m = 10,625KN/m
ULPA-left-TB2, ..., ULPA-left-TB18	Uniform Load Pedestrian Area-left	Uniform load in pedestrian area left	5KN/m^2 *4,25m = 21,25KN/m
ULPA-right-TB2, ..., ULPA-right-TB18	Uniform Load Pedestrian Area-right	Uniform load in pedestrian area right	5KN/m^2 *4,25m = 21,25KN/m
HVL-TB2, ... HVL-TB18	Heavy Vehicle Load	Virtual lane 1 (main beam)	2 * 150KN *(3,65m/4,25m) = 257,65KN
		Virtual lane 1 (beams next to main beam)	150KN*(0,60m/4,25m)=21,18KN
		Virtual lane 2 (main beam)	2 * 100KN *(3,65m/4,25m) = 171,76KN
		Virtual lane 2 (beams next to main beam)	100KN*(0,60m/4,25m)=14,12KN
		Virtual lane 3 (main beam)	2 * 50KN *(3,65m/4,25m) = 85,88KN
		Virtual lane 3 (beams next to main beam)	50KN*(0,60m/4,25m)=7,06KN
*TB = Transversal Beam			

Figure 28: Load patterns of TB2-TB18 with corresponding loads [Own elaboration]

The load patterns applied on the transversal beams 1 and 19 are shown in the following table:

Outer two beams (Transversal Beam 1 and Transversal Beam 19)			
Load Pattern	Abbreviation stands for	description	Calculation
SW	Self Weight	self weight of the frames	considered in SAP2000
DL slab	Dead Loads Slab	Floor beams	5,35KN/m^2 *2,125m = 11,37KN/m
		Cantilevered beams	5,35KN/m^2 *2,125m = 11,37KN/m
DL other	Dead Loads Other	Sidewalk pavement	1,5KN/m^2 *2,125m = 3,19KN/m
		Pavement	2,76KN/m^2 *2,125m = 5,87KN/m
		Roadway guardrail	1KN/m *2,125m = 2,125KN
		Sidewalk Handraiils	0,5KN/m *2,125m = 1,06KN
		Cornice	4KN/m*2,125m = 8,5KN
UTL-TB2, ..., UTL-TB18	Uniform Trafic Load-Transversal Beam	Uniform trafic load virtual lane 1	9KN/m^2 *2,125m = 19,125KN/m
		Virtual lane 2,3,4 and remaining area	2,5KN/m^2 *2,125m = 5,31KN/m
ULPA-left-TB2, ..., ULPA-left-TB18	Uniform Load Pedestrian Area-left	Uniform load in pedestrian area left	5KN/m^2 *2,125m = 10,625KN/m
ULPA-right-TB2, ..., ULPA-right-TB18	Uniform Load Pedestrian Area-right	Uniform load in pedestrian area right	5KN/m^2 *2,125m = 10,625KN/m
HVL-TB2, ... HVL-TB18	Heavy Vehicle Load	Virtual lane 1 (main beam)	150KN + 150KN *(3,05m/4,25m) = 257,65KN
		Virtual lane 1 (beams next to main beam)	150KN*(1,20m/4,25m)=42,35KN
		Virtual lane 2 (main beam)	100KN + 100KN *(3,05m/4,25m) = 171,76KN
		Virtual lane 2 (beams next to main beam)	100KN*(1,20m/4,25m)=28,24KN
		Virtual lane 3 (main beam)	50KN + 50KN *(3,05m/4,25m) = 85,88KN
		Virtual lane 3 (beams next to main beam)	50KN*(1,20m/4,25m)=14,12KN
*TB = Transversal Beam			

Figure 29: Load patterns of TB1 and TB19 with corresponding loads [Own elaboration]

For a better understanding of the load patterns for the heavy vehicles, an example of the load pattern HVL-TB6 is shown below:

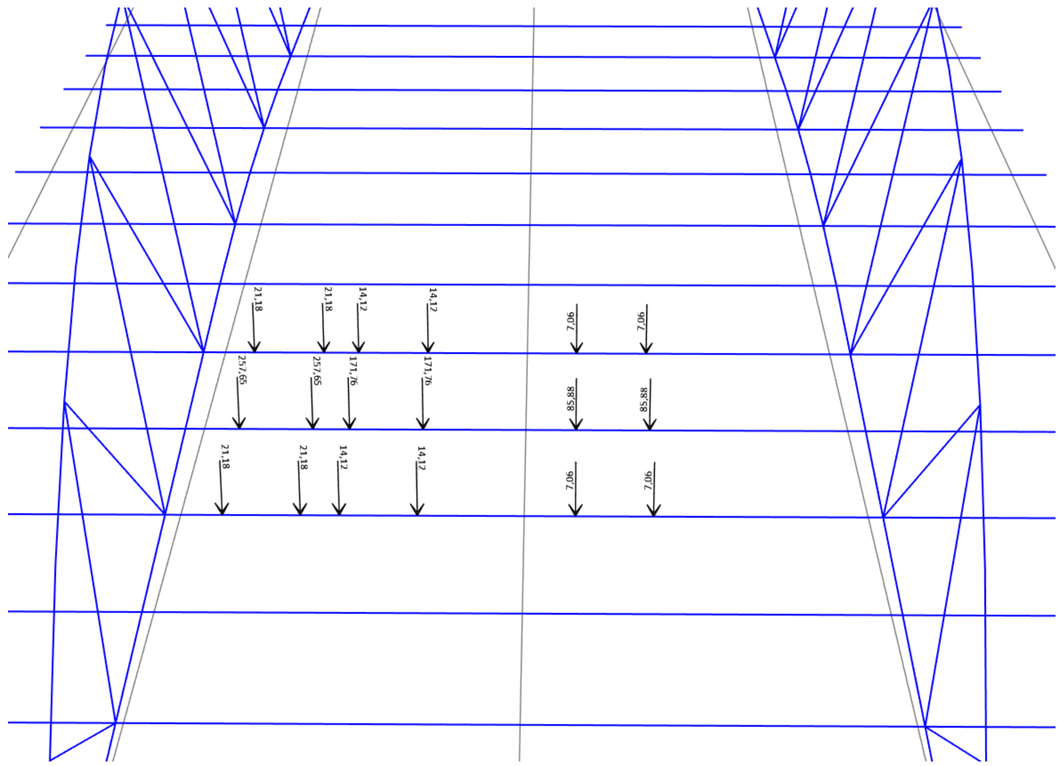


Figure 30: Load pattern: HVL-TB6 [Own elaboration; SAP2000]

In this example the heavy vehicles are placed so that the contribution of the beam 6 is the greatest, while the concrete slab also transfers loads to beams 5 and 7:

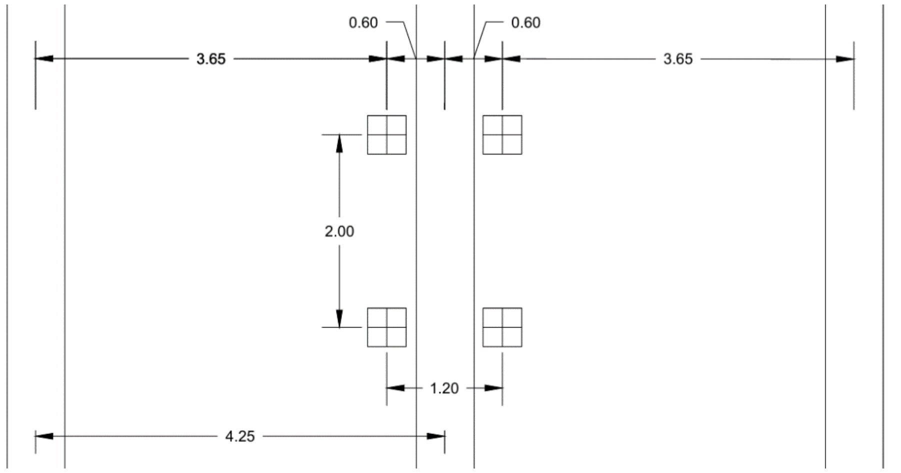


Figure 31: Arrangement of a heavy vehicle relative to the transversal beams [Own elaboration; Autodesk AutoCAD]

The load patterns for the thermal actions are shown in the following table:

Thermal Actions		
Load Pattern	description	Value
Wind from the left	Horizontal force on the longitudinal beams	1,224 KN/m
	Horizontal force on the arches	0,899 KN/m
	Uplift	0,605 KN/m^2
Wind from the right	Horizontal force on the longitudinal beams	1,224 KN/m
	Horizontal force on the arches	0,899 KN/m
	Uplift	0,605 KN/m^2
Uniform temperature decrease	Contraction	25,77°C
Uniform temperature increase	Dilatation	47,74°C
Only arches +15°	Vertical temperature difference	15°C
Only deck +15°	Vertical temperature difference	15°C

Figure 32: Load patterns for the thermal actions [Own elaboration]

6.3.1. Verifications

All structural steel elements except the tension rods are made of S355 and therefore have a yield strength of $f_{y,k} = 355\frac{MN}{m^2} = 355.000\frac{kN}{m^2} = 35,5\frac{kN}{cm^2}$.

The partial safety factor for bridges recommended in section 6.1 of EN 1993-2 is: $\gamma_{M,0} = 1,00$.

6.3.1.1. Buckling Analysis

The first step is to perform a buckling analysis with the goal to find the first critical load factor of the bridge structure. This analysis is done via the load case type buckling in SAP2000. The arch is the most critical part of the structure in terms of buckling. Therefore, the loads introduced into the load case are those that create the highest compression forces in the left arch.

The buckling analysis provides a factor that, when multiplied by the applied loads, will cause the structure to buckle. When this factor is greater than 10, the second order effect can be neglected.

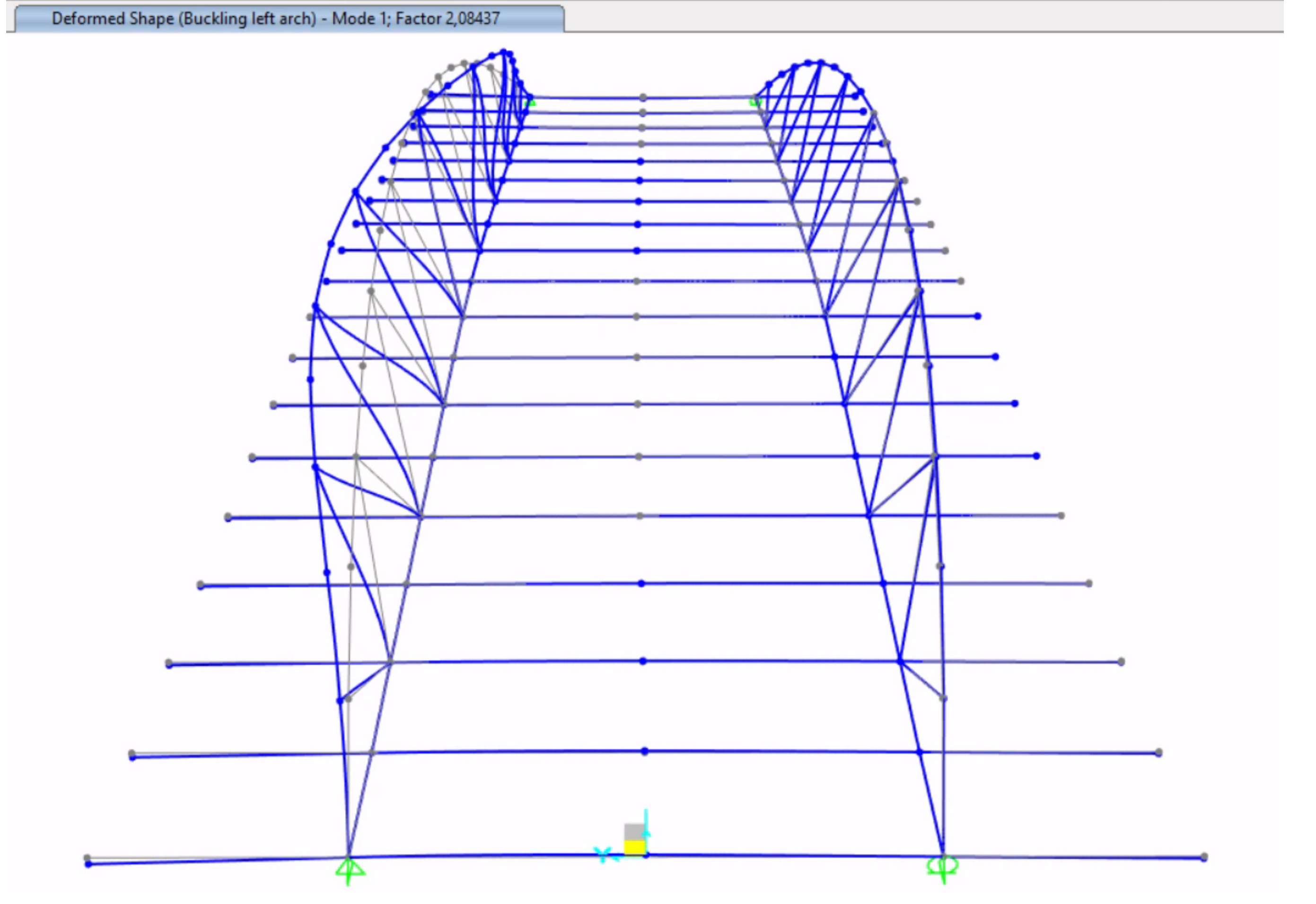


Figure 33: first buckling analysis with a factor of 2,08 [Own elaboration; SAP2000]

As can be seen in the figure above, in the first buckling analysis, the left arch begins to buckle with a factor of only 2.08. Therefore, the design of the cross sections of the arches had to be adjusted in an iterative process until a factor greater than 10 was obtained. The initial and final cross sections are shown in the following figure:

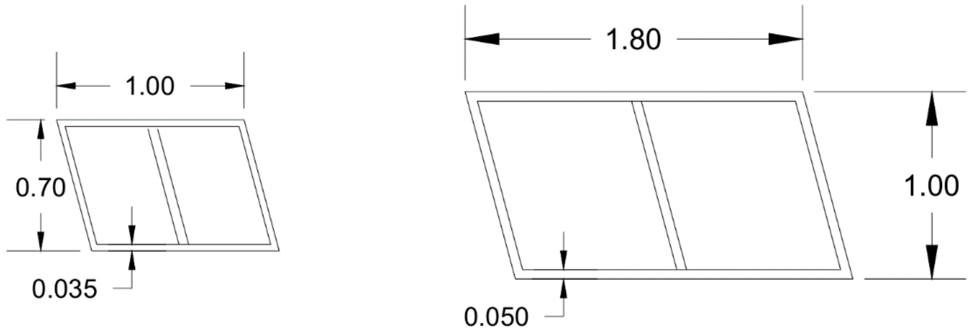


Figure 34: change of the left arch cross section [Own elaboration; Autodesk AutoCAD]

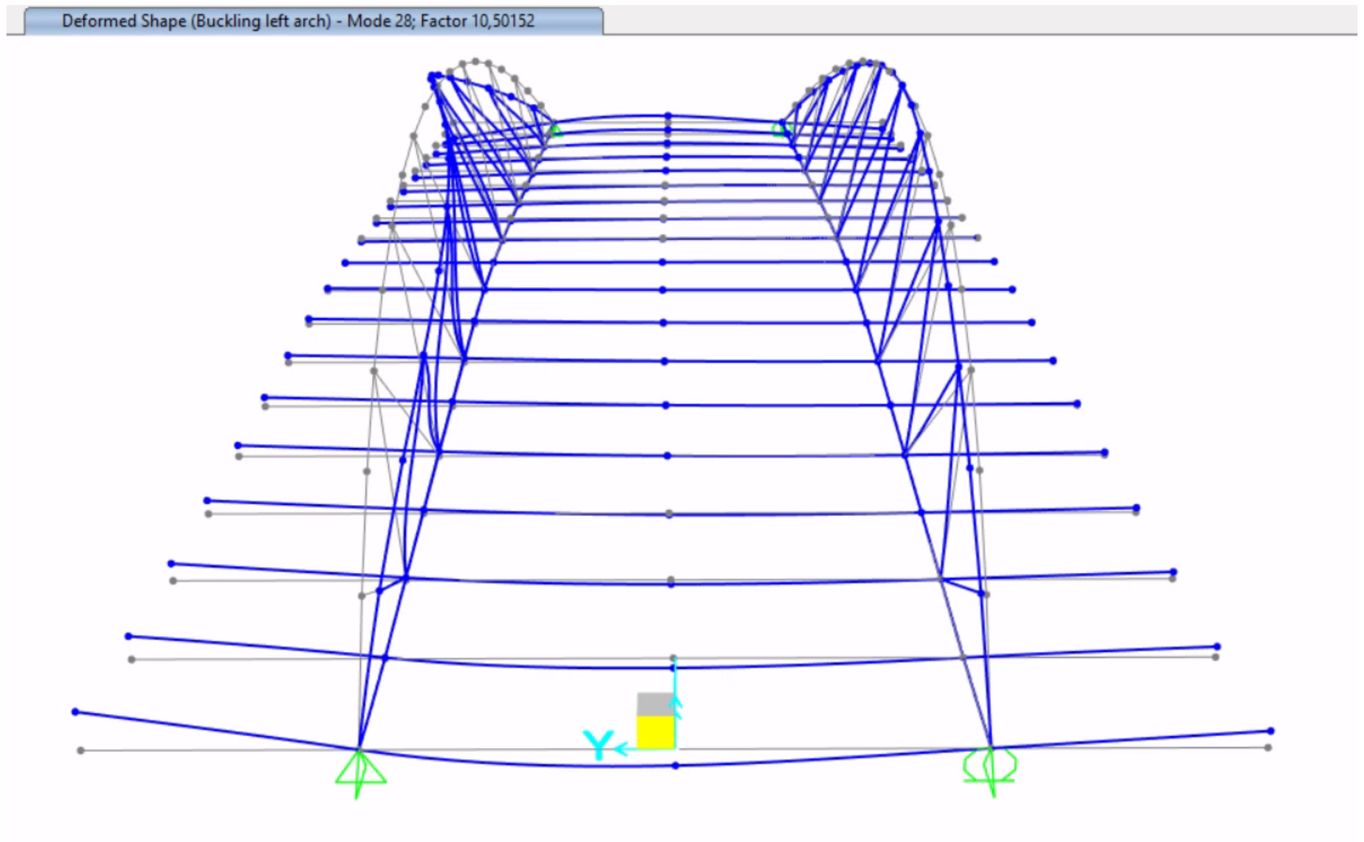


Figure 35: buckling analysis with a factor of 10,50 [Own elaboration; SAP2000]

By adopting the new cross-sections for the arches, the second order effects of the structure can be neglected.

6.3.1.2. Serviceability Limit State Checks

The next step is the verification of the serviceability limit states. According to IAP-11, it should be verified that the maximum vertical deflection at the frequent value of the service overload does not exceed a value of L/1000.

Thus, for the global verification, the maximum vertical deflection should not exceed the following value:

$$\frac{L}{1000} = \frac{76{,}50m}{1000} = 0{,}0765m$$

For the local verification of the most deformed floor beam, the maximum vertical deflection should not exceed the following value:

$$\frac{L}{1000} = \frac{18m}{1000} = 0{,}018m$$

The combination of actions for the SLS is shown in the following table:

SLS verification			
deformation	Load Pattern	Y	ψ
Permanent	SW	0	/
	DL slab	0	/
	DL other	0	/
Variable	UTL all	1	0,4
	HVL-TB10	1	0,75
	ULPA-left-all	1	0,4

Figure 36: Combination of actions for SLS [Own elaboration; values derived from: IAP-11]

The highest vertical deformation, with a value of 0,0233m, occurs at the center of the floor beam in the middle of the bridge, as can be seen in the following figure:

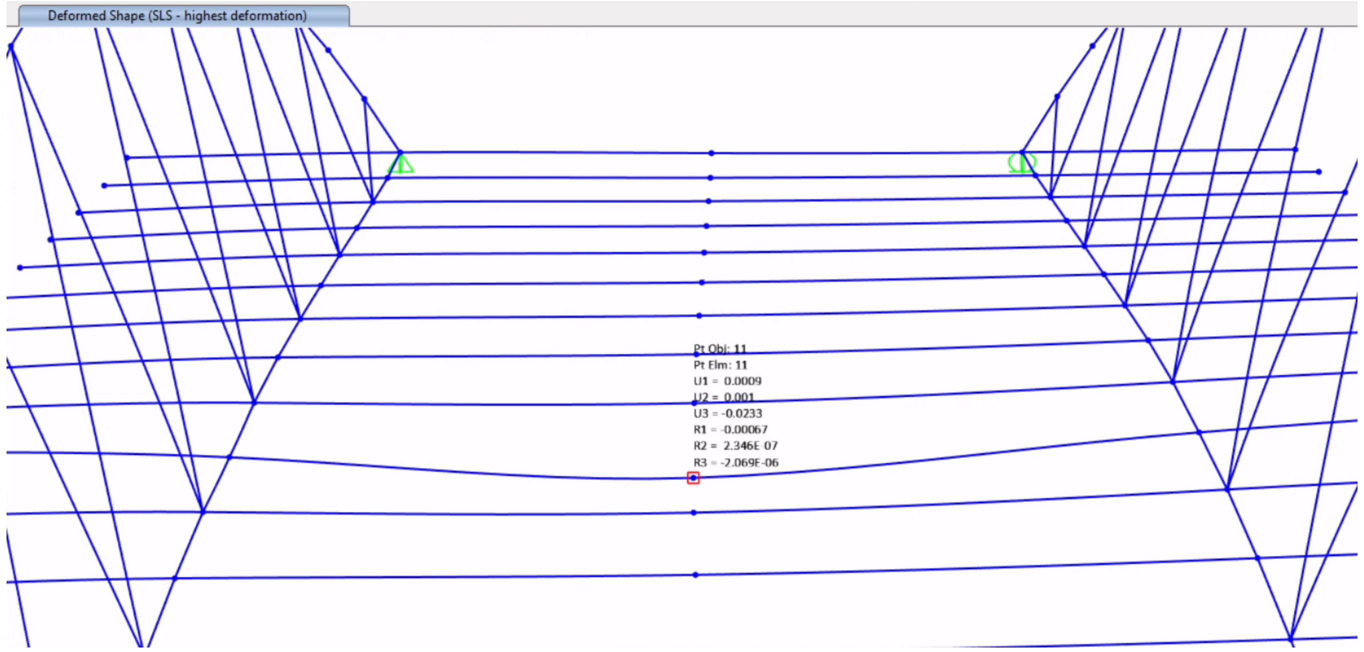


Figure 37: highest vertical deformation of the floor beam at the center

The lowest vertical deformation of this floor beam is at the outer right end with a value of 0.0053m:

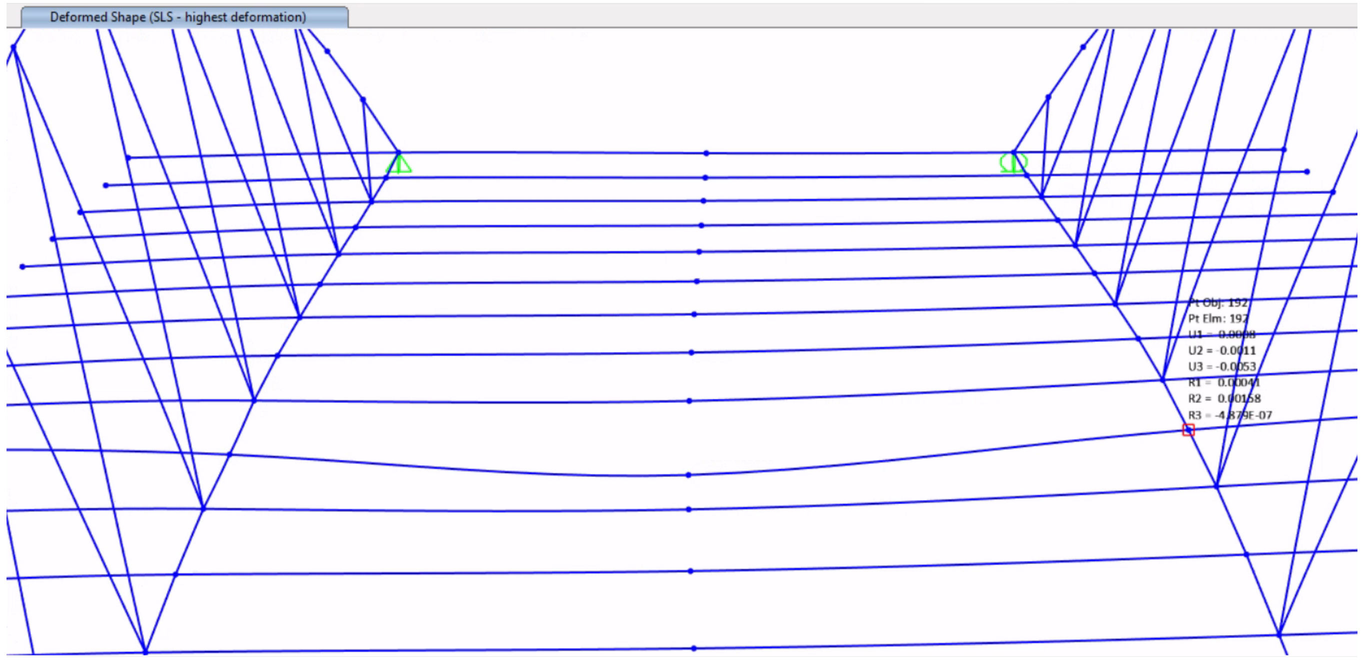


Figure 38: lowest vertical deformation of the floor beam at the outer end

Therefore:

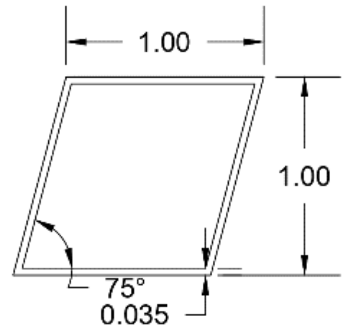
Global vertical deformation: $0{,}0233m \leq 0{,}0765m$

Local vertical deformation: $0{,}0233m - 0{,}0053m = 0{,}018m \leq 0{,}018m$

6.3.1.3. *Ultimate limit state checks*

In the following, all structural elements are checked for internal forces, always using the distribution of actions that produce the highest values and its most unfavorable combination.

Checking of longitudinal beams:

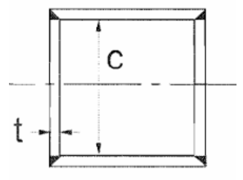


Properties

Cross-section (axial) area	0,1373	Section modulus about 3 axis	0,0423
Moment of Inertia about 3 axis	0,0212	Section modulus about 2 axis	0,0363
Moment of Inertia about 2 axis	0,023	Plastic modulus about 3 axis	0,0494
Product of Inertia about 2-3	-5,673E-03	Plastic modulus about 2 axis	0,0511
Shear area in 2 direction	0,0708	Radius of Gyration about 3 axis	0,3925
Shear area in 3 direction	0,0835	Radius of Gyration about 2 axis	0,4092
Torsional constant	0,0314	Shear Center Eccentricity (x3)	0,

Figure 39: cross section of the longitudinal beam and the corresponding section properties [Own elaboration]

Table 5.2 (sheet 1 of 3) of EN 1993-1-1 is used to determine the class of the cross-section:



$$\varepsilon = \sqrt{\frac{235}{355}} = 0{,}81$$

$$\frac{c}{t} = \frac{0{,}93m}{0{,}035m} = 26{,}57$$

$$33 \times \varepsilon = 33 \times 0{,}81 = 26{,}73$$

$$26{,}57 \leq 26{,}73$$

➔ Class 1

Verification of tension according to section 6.2.3 of EN 1993-1-1:

The maximum value of the internal force and its most unfavorable combination obtained by the calculation in the SAP2000 program are shown in the following table:

ULS verification			
longitudinal beam			
Axil force / tension	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB10	1,35	0,75
	ULPA-left-all	1,35	leading
	Only Arches +15°C	1,5	0,6
∑N = 5.309,32kN			

Figure 40: maximum tension in longitudinal beam [Own elaboration]

$$N_{Ed} \leq N_{t,Rd}$$

$$N_{t,Rd} = N_{pl,Rd} = \frac{A \times f_y}{\gamma_{M,0}} = \frac{0{,}1373m^2 \times 355.000\dfrac{kN}{m^2}}{1{,}00} = 48.741{,}5kN$$

$$5.309{,}32kN \leq 48.741{,}5kN$$

Where:

$A = 0{,}1373m^2$ Area of the longitudinal beam cross-section

Verification of bending moment according to section 6.2.5 of EN 1993-1-1:

The maximum values of the internal forces and its most unfavorable combinations obtained by the calculation in the SAP2000 program are shown in the following table:

ULS verification			
longitudinal beam			
Bending Moment	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB4	1,35	leading
	ULPA-left-all	1,35	0,4
	Only Deck +15°C	1,5	0,6
∑My = 4.006,57kNm			

ULS verification			
longitudinal beam			
Bending Moment	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL-TB(1-3)	1,35	0,4
	HVL-TB2	1,35	0,75
	ULPA-right-all	1,35	leading
	Wind from the left	1,5	0,6
∑Mz = 4.114,60kNm			

Figure 41: maximum bending moments in longitudinal beam [Own elaboration]

$$M_{Ed} \leq M_{c,Rd}$$

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \times f_y}{\gamma_{M,0}} \;(for\ class\ 1\ cross\ section)$$

Where:

$W_{y,pl} = 0{,}0494m^3$ plastic section modulus about y-axis

$W_{z,pl} = 0{,}0511m^3$ plastic section modulus about z-axis

$$M_{y,c,Rd} = \frac{0{,}0494m^3 \times 355.000\dfrac{KN}{m^2}}{1{,}00} = 17.537kNm$$

$$4.006{,}57kNm \leq 17.537kNm$$

$$M_{z,c,Rd} = \frac{0{,}0511m^3 \times 355.000\frac{KN}{m^2}}{1{,}00} = 18.140{,}5kNm$$

$$4.114{,}60kNm \leq 18.140{,}5kNm$$

Verification of shear forces according to section 6.2.6 of EN 1993-1-1:

The maximum values of the internal forces and its most unfavorable combinations obtained by the calculation in the SAP2000 program are shown in the following table:

ULS verification			
longitudinal beam			
Shear Force	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB4	1,35	leading
	ULPA-left-all	1,35	0,4
	Only Deck +15°C	1,5	0,6
∑Vz = 1.259,77kN			

ULS verification			
longitudinal beam			
Shear Force	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL-TB(1-3)	1,35	0,4
	HVL-TB2	1,35	0,75
	ULPA-right-all	1,35	leading
	Wind from the left	1,5	0,6
∑Vy = 757,61kN			

Figure 42: maximum shear forces in longitudinal beam [Own elaboration]

The shear force is mainly transferred through the webs. Therefore, according to EN 1933-1-1, the shear transfer can be approximated as being applied only through the web of the section. It is also assumed that the shear distribution is constant.

$$V_{Ed} \leq V_{c,Rd}$$

$$V_{z,c,Rd} = V_{z,pl,Rd} = \frac{A_{z,v} \times f_y}{\sqrt{3} \times \gamma_{M,0}} = \frac{0{,}0708m^2 \times 355.000\frac{KN}{m^2}}{\sqrt{3} \times 1{,}00} = 14.511{,}12kN$$

$$1.259{,}77kN \leq 14.511{,}12kN$$

$$V_{y,c,Rd} = V_{y,pl,Rd} = \frac{A_{y,v} \times f_y}{\sqrt{3} \times \gamma_{M,0}} = \frac{0{,}0835m^2 \times 355.000\frac{KN}{m^2}}{\sqrt{3} \times 1{,}00} = 17.114{,}11kN$$

$$757{,}61kN \leq 17.114{,}11kN$$

Where:

$A_{z,V} = 0{,}0708m^2$ Shear area in direction of z-axis

$A_{y,V} = 0{,}0835m^2$ Shear area in direction of y-axis

Interaction of axil force, bending moments and shear forces:

According to section 6.2.1 of EN 1993-1-1, as a conservative approximation for all cross-section classes a linear summation of the utilization ratios for each stress resultant may be used. For class 1, class 2 or class 3 cross sections subjected to the combination of N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ this method may be applied by using the following criteria:

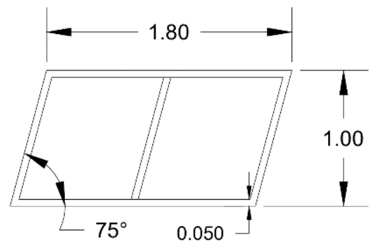
$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1$$

Where N_{Rd}, $M_{y,Rd}$ and $M_{z,Rd}$ are the design values of the resisance depending on the cross sectional classification and including any reduction that may be caused by shear effects. According to section 6.2.8, where the shear force is less than half the plastic shear resistance its effect on the moment resistance may be neglected.

Therefore:

$$\frac{5.309{,}32kN}{48.741{,}5kN} + \frac{4.006{,}57kNm}{17.537kNm} + \frac{4.114{,}60kNm}{18.140{,}5kNm} = 0{,}56 \leq 1$$

Checking of arches:



Properties

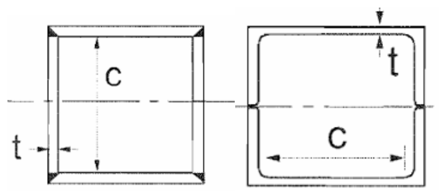
Cross-section (axial) area	0,1156	Section modulus about 3 axis	0,0264
Moment of Inertia about 3 axis	9,255E-03	Section modulus about 2 axis	0,0278
Moment of Inertia about 2 axis	0,0176	Plastic modulus about 3 axis	0,0305
Product of Inertia about 2-3	-3,298E-03	Plastic modulus about 2 axis	0,0404
Shear area in 2 direction	0,0499	Radius of Gyration about 3 axis	0,2829
Shear area in 3 direction	0,0796	Radius of Gyration about 2 axis	0,3901
Torsional constant	0,0173	Shear Center Eccentricity (x3)	0,

Properties

Cross-section (axial) area	0,3198	Section modulus about 3 axis	0,1002
Moment of Inertia about 3 axis	0,0501	Section modulus about 2 axis	0,1194
Moment of Inertia about 2 axis	0,1234	Plastic modulus about 3 axis	0,1169
Product of Inertia about 2-3	0,0134	Plastic modulus about 2 axis	0,1669
Shear area in 2 direction	0,1024	Radius of Gyration about 3 axis	0,3958
Shear area in 3 direction	0,2007	Radius of Gyration about 2 axis	0,6212
Torsional constant	0,1031	Shear Center Eccentricity (x3)	0,

Figure 43: cross section of the arch and the corresponding section properties [Own elaboration]

Table 5.2 (sheet 1 of 3) of EN 1993-1-1 is used to determine the class of the cross-section:



$$\varepsilon = \sqrt{\frac{235}{355}} = 0{,}81$$

$$\frac{c}{t} = \frac{0{,}82m}{0{,}05m} = 16{,}40$$

$$33 \times \varepsilon = 33 \times 0{,}81 = 26{,}73$$

$$16{,}40 \leq 26{,}73$$

➔ Class 1

Verification of compression according to section 6.2.4 of EN 1993-1-1:

The maximum value of the internal force and its most unfavorable combination obtained by the calculation in the SAP2000 program are shown in the following table:

ULS verification			
Left Arch			
Axil force / compression	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB6	1,35	0,75
	ULPA-left-all	1,35	leading
	Only Arches +15°C	1,5	0,6
∑N = -25.820,97kN			

Figure 44: maximum compression in the left arch [Own elaboration]

$$N_{Ed} \leq N_{c,Rd}$$

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M,0}} = \frac{0{,}3198m^2 \times 355.000\frac{kN}{m^2}}{1{,}00} = 113.529kN(for\ class\ 1\ cross\ section)$$

$$25.820{,}97kN \leq 113.529kN$$

Where:

$A = 0{,}3198m^2$ Area of the arch cross-section

Verification of bending moment according to section 6.2.5 of EN 1993-1-1:

The maximum values of the internal forces and its most unfavorable combinations obtained by the calculation in the SAP2000 program are shown in the following table:

ULS verification			
Left Arch			
Bending Moment	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB3	1,35	leading
	ULPA-left-all	1,35	0,4
	Only Deck +15°C	1,5	0,6
∑My = 2.847,36kNm			

ULS verification			
Left Arch			
Bending Moment	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL TB(1-3)	1,35	0,4
	HVL-TB2	1,35	0,75
	ULPA-left-TB(15-19)	1,35	0,4
	ULPA-right-all	1,35	leading
	Wind from the right	1,5	0,6
∑Mz = -7.144,67kNm			

Figure 45: maximum bending moments in the left arch [Own elaboration]

$$M_{Ed} \leq M_{c,Rd}$$

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \times f_y}{\gamma_{M,0}} \ (for\ class\ 1\ cross\ section)$$

Where:

$W_{y,pl} = 0{,}1169m^3$ plastic section modulus about y-axis

$W_{z,pl} = 0{,}1669m^3$ plastic section modulus about z-axis

$$M_{y,c,Rd} = \frac{0{,}1169m^3 \times 355.000\frac{KN}{m^2}}{1{,}00} = 41.499{,}50kNm$$

$$2.847{,}36kNm \leq 41.499{,}50kNm$$

$$M_{z,c,Rd} = \frac{0{,}1669m^3 \times 355.000\frac{KN}{m^2}}{1{,}00} = 59.249{,}5kNm$$

$$7.144{,}67kNm \leq 59.249{,}5kNm$$

Verification of shear forces according to section 6.2.6 of EN 1993-1-1:

The maximum values of the internal forces and its most unfavorable combinations obtained by the calculation in the SAP2000 program are shown in the following table:

ULS verification			
Left Arch			
Shear Force	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB9	1,35	0,75
	ULPA-left-all	1,35	leading
	Only Arch +15°C	1,5	0,6
∑Vz = 913,60kN			

ULS verification			
Left Arch			
Shear Force	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB3	1,35	0,75
	ULPA-left-all	1,35	leading
	Wind from the right	1,5	0,6
∑Vy = -505,57kN			

Figure 46: maximum shear forces in the left arch [Own elaboration]

The shear force is mainly transferred through the webs. Therefore, according to DIN EN 1933-1-1, the shear transfer can be approximated as being applied only through the web of the section. It is also assumed that the shear distribution is constant.

$$V_{Ed} \leq V_{c,Rd}$$

$$V_{z,c,Rd} = V_{z,pl,Rd} = \frac{A_{z,v} \times f_y}{\sqrt{3} \times \gamma_{M,0}} = \frac{0{,}1024m^2 \times 355.000\frac{KN}{m^2}}{\sqrt{3} \times 1{,}00} = 20.987{,}84kN$$

$$913{,}60kN \leq 20.987{,}84kN$$

$$V_{y,c,Rd} = V_{y,pl,Rd} = \frac{A_{y,v} \times f_y}{\sqrt{3} \times \gamma_{M,0}} = \frac{0{,}2007m^2 \times 355.000\frac{KN}{m^2}}{\sqrt{3} \times 1{,}00} = 41.135{,}34kN$$

$$505{,}57kN \leq 41.135{,}34kN$$

Where:

$A_{z,V} = 0{,}1024m^2$ Shear area in direction of z-axis

$A_{y,V} = 0{,}2007m^2$ Shear area in direction of y-axis

<u>Interaction of axil force, bending moments and shear forces:</u>

According to section 6.2.1 of EN 1993-1-1, as a conservative approximation for all cross section classes a linear summation of the utilization ratios for each stress resultant may be used. For class 1, class 2 or class 3 cross sections subjected to the combination of N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ this method may be applied by using the following criteria:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1$$

Where N_{Rd}, $M_{y,Rd}$ and $M_{z,Rd}$ are the design values of the resisance depending on the cross sectional classification and including any reduction that may be caused by shear effects. According to section 6.2.8, where the shear force is less than half the plastic shear resistance its effect on the moment resistance may be neglected.

Therefore:

$$\frac{25.820{,}97kN}{113.529kN} + \frac{2.847{,}36kNm}{41.499{,}50kNm} + \frac{7.144{,}67kNm}{59.249{,}5kNm} = 0{,}42 \leq 1$$

<u>Checking of tension rods:</u>

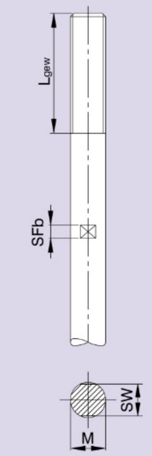
Since the tension rods are hinged at both ends, they transmit only axial forces. It has been checked that the cables are not compressed even in the most unfavorable situation. Therefore, it only has to be verified that the rods withstand the highest tension force, which is shown in the table below:

ULS verification			
Tension Rods			
Axil force / tension	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB4	1,35	0,75
	ULPA-left-all	1,35	leading
	Only Deck +15°C	1,5	0,6
∑N = 2.118,86kN			

Figure 47: maximum axial forces in the tension rods [Own elaboration]

<u>Verification of axil forces:</u>

A diameter of 100mm was chosen for the tension rods. Pfeifer's system could be used for these elements. According to the table below, taken from a Pfeifer catalog, these elements can withstand a maximum axial force of N_{Rd}=2985kN.



Mat.	S460N						
M mm	$N_{R,d}$* kN	$N_{R,d,red}$** kN	L_{gew} mm	SFb mm	SW mm	L_{max} mm	Peso kg/m
10	26,3	26,3	33	10	9	6000	0,61
12	38,3	38,3	38	12	10	6000	0,88
16	71,2	60,1	54	15	14	15000	1,58
20	111	93,9	67	18	18	15000	2,47
24	160	160	80	23	22	15000	3,55
27	208	167	90	23	25	15000	4,50
30	254	254	100	28	28	15000	5,55
36	371	260	120	28	33	15000	8,00
42	509	359	140	33	39	15000	10,9
48	669	488	159	38	45	15000	14,2
52	798	638	172	43	49	15000	16,7
56	922	638	187	43	53	15000	19,3
60	1073	807	199	48	57	15000	22,2
64	1215	997	211	53	60	15000	25,3
70	1463	1206	233	58	65	15000	30,2
80	1910	1685	266	68	75	15000	39,5
90	2418	2243	297	78	85	15000	49,9
100	2985	2552	328	83	95	15000	61,7

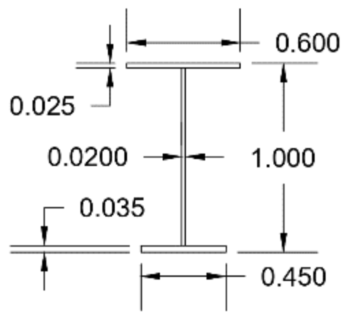
Salvo posibles modificaciones *Límite elástico según DIN 18800, Placa de anclaje S355J2G3 ** Placa de anclaje S235J2G3

Figure 48: Table of the Pfeifer's Tension Rod System [Pfeifer Catalog]

$$N_{t,Ed} \leq N_{Rd}$$

$$2.118{,}86kN \leq 2985kN$$

Checking of transversal beams:



Section Name: D-Start Section transverse beams

Properties

Cross-section (axial) area	0,0496	Section modulus about 3 axis	0,0171
Moment of Inertia about 3 axis	8,618E-03	Section modulus about 2 axis	2,388E-03
Moment of Inertia about 2 axis	7,164E-04	Plastic modulus about 3 axis	0,0193
Product of Inertia about 2-3	0,	Plastic modulus about 2 axis	4,116E-03
Shear area in 2 direction	0,02	Radius of Gyration about 3 axis	0,417
Shear area in 3 direction	0,0256	Radius of Gyration about 2 axis	0,1202
Torsional constant	1,163E-05	Shear Center Eccentricity (x3)	0,

Figure 49: cross section of the transversal beam where it joins the longitudinal beam and the corresponding section properties [Own elaboration]

The transverse beams have already been checked in the local model. However, the two transverse beams above the supports behave differently from the others due to the connection with the arches. Therefore, these two are analyzed again in the following.

Verification of tension according to section 6.2.3 of EN 1993-1-1:

The maximum value of the internal force and its most unfavorable combination obtained by the calculation in the SAP2000 program are shown in the following table:

ULS verification			
transversal beam			
Axil force / compression	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	leading
	HVL-TB3	1,35	0,75
	ULPA-left-all	1,35	0,4
	ULPA-right-all	1,35	0,4
	Only Deck +15°C	1,5	0,6
∑N = -1997,27kN			

Figure 50: maximum tension in transversal beams above the supports [Own elaboration]

$$N_{Ed} \leq N_{c,Rd}$$

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M,0}} = \frac{0{,}0496m^2 \times 355.000\frac{kN}{m^2}}{1{,}00} = 17.608kN$$

$$1.997{,}27kN \leq 17.608kN$$

Where:

$A = 0{,}0496m^2$ Area of the longitudinal beam cross-section

Verification of bending moment according to section 6.2.5 of EN 1993-1-1:

The maximum values of the internal forces and its most unfavorable combinations obtained by the calculation in the SAP2000 program are shown in the following table:

ULS verification			
transversal beam			
Bending Moment	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL-TB1	1,35	0,4
	HVL-TB1	1,35	0,75
	ULPA-left-all	1,35	leading
	Wind from the right	1,5	0,6
∑My = -3857,58kNm			

ULS verification			
transversal beam			
Bending Moment	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB8	1,35	0,75
	ULPA-left-all	1,35	leading
	Wind from the right	1,5	0,6
∑Mz = 7,67kNm			

Figure 51: maximum bending moments in transversal beams above the supports [Own elaboration]

$$M_{Ed} \leq M_{c,Rd}$$

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \times f_y}{\gamma_{M,0}} \ (for\ class\ 1\ cross\ section)$$

Where:

$W_{y,pl} = 0{,}0193m^3$ plastic section modulus about y-axis

$W_{z,pl} = 0{,}004116m^3$ plastic section modulus about z-axis

$$M_{y,c,Rd} = \frac{0{,}0193m^3 \times 355.000\frac{KN}{m^2}}{1{,}00} = 6.851{,}5kNm$$

$$3.857{,}58kNm \leq 6.851{,}5kNm$$

$$M_{z,c,Rd} = \frac{0{,}004116m^3 \times 355.000\frac{KN}{m^2}}{1{,}00} = 1461{,}18kNm$$

$$7{,}67kNm \leq 1461{,}18kNm$$

Verification of shear forces according to section 6.2.6 of EN 1993-1-1:

The maximum values of the internal forces and its most unfavorable combinations obtained by the calculation in the SAP2000 program are shown in the following table:

ULS verification			
transversal beam			
Shear Force	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB1	1,35	leading
	ULPA-left-all	1,35	0,4
	Wind from the left	1,5	0,6
∑Vz = -1.340,45kN			

ULS verification			
transversal beam			
Shear Force	Load Pattern	Y	ψ
Permanent	SW	1,35	/
	DL slab	1,35	/
	DL other	1,35	/
Variable	UTL all	1,35	0,4
	HVL-TB8	1,35	0,75
	ULPA-left-all	1,35	leading
	Wind from the right	1,5	0,6
∑Vy = -0,69kN			

Figure 52: maximum shear forces in transversal beams above the supports [Own elaboration]

The shear force is mainly transferred through the webs. Therefore, according to EN 1933-1-1, the shear transfer can be approximated as being applied only through the web of the section. It is also assumed that the shear distribution is constant.

$$V_{Ed} \leq V_{c,Rd}$$

$$V_{z,c,Rd} = V_{z,pl,Rd} = \frac{A_{z,v} \times f_y}{\sqrt{3} \times \gamma_{M,0}} = \frac{0{,}02m^2 \times 355.000\frac{KN}{m^2}}{\sqrt{3} \times 1{,}00} = 4.099{,}19kN$$

$$1.340{,}45kN \leq 4.099{,}19kN$$

$$V_{y,c,Rd} = V_{y,pl,Rd} = \frac{A_{y,v} \times f_y}{\sqrt{3} \times \gamma_{M,0}} = \frac{0{,}0256m^2 \times 355.000\frac{KN}{m^2}}{\sqrt{3} \times 1{,}00} = 5.246{,}96kN$$

$$0{,}69kN \leq 5.246{,}96kN$$

Where:

$A_{z,V} = 0{,}02m^2$ Shear area in direction of z-axis

$A_{y,V} = 0{,}0256m^2$ Shear area in direction of y-axis

<u>Interaction of axil force, bending moments and shear forces:</u>

According to section 6.2.1 of EN 1993-1-1, as a conservative approximation for all cross section classes a linear summation of the utilization ratios for each stress resultant may be used. For class 1, class 2 or class 3 cross sections subjected to the combination of N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ this method may be applied by using the following criteria:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1$$

Where N_{Rd}, $M_{y,Rd}$ and $M_{z,Rd}$ are the design values of the resisance depending on the cross sectional classification and including any reduction that may be caused by shear effects. According to section 6.2.8, where the shear force is less than half the plastic shear resistance its effect on the moment resistance may be neglected.

Therefore:

$$\frac{1.997{,}27kN}{17.608kN} + \frac{3.857{,}58kNm}{6.851{,}5kNm} + \frac{7{,}67kNm}{1461{,}18kNm} = 0{,}68 \leq 1$$

6.3.2. Expansion Joints

The global model is used to determine the maximum horizontal deformation. In the most conservative case, the deformation is 64.5 mm. The supports of the bridge are selected in such a way that the deformations will only occur on one side. Therefore, one of the two expansion joints must be able to absorb the entire deformation of 64.5 mm. Considering these requirements, we choose the Mageba TENSA-GRIP RS. Those high-quality expansion joints are single gap joints for movements of up to 80 mm in the case of type RS. The joints consist of robust steel edge profiles and replaceable elastomeric sealing profiles, which ensure long-lasting watertightness. The drainage of the joints is via the drainage system of the bridge deck. They can be used on asphalt roads with a layer thickness of between 50 mm and 300 mm, which makes them suitable for our case of 80 mm.

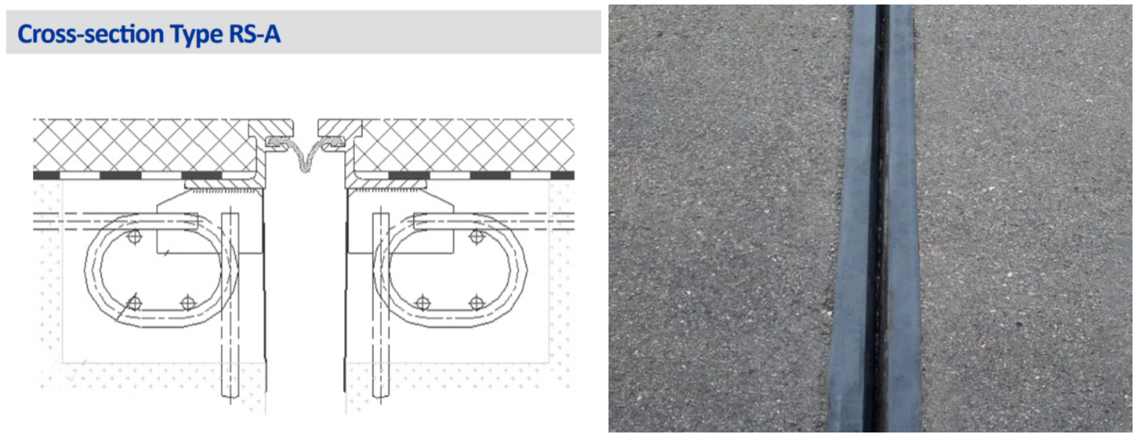


Figure 53: Representation of the Mageba TENSA-GRIP RS-A expansion joint [Source: Mageba]

6.3.3. Bearing Pads

The maximum force in the ultimate limit state transferred to the supports obtained from the global model is as follows:

ULS verification				
Bearing Pads				
Vertical Forces	Load Pattern	Y	ψ	Reaction forces [kN]
Permanent	SW	1,35	/	2825,17
	DL slab	1,35	/	4196,50
	DL other	1,35	/	2114,73
Variable	UTL all	1,35	0,4	744,23
	HVL-TB1	1,35	0,75	883,85
	ULPA-all	1,35	leading	2177,20
		Maximum vertical reaction		12941,69
		Minimum vertical reaction		9136,41

Figure 54: Maximum and minimum vertical reaction forces [Own elaboration]

- Maximum vertical reaction: 12.941,92kN
- Minimum vertical reaction (only permanent loads): 9.136,41kN

This is the force used to dimension the foundations in the appendix "Calculation of the Foundations".

There are four supports, all of which prevent vertical displacement, but each of which prevents horizontal displacement in a different way. There is one fixed support, one free-sliding support and two guided-sliding supports allocated as represented in the following sketch:

Figure 55: Pot Bearing Operating Modes [Own elaboration]

Considering these requirements, we choose the Tetron® CD pot bearings from Freyssinet. The bearing devices are made up of an elastomer pad confined in a steel cylinder or pot, which is fitted on the top with a circular steel plate (piston). The elastomer behaves like a viscous fluid under high pressure flowing to allow rotation and tilting movements of the piston about any horizontal axis. These devices are available in three operating modes allowing for the different horizontal movements shown in the figure above. (Fixed, Guided-sliding, Free-sliding) In the following are shown the 3 different types of pads and an exploded view of the free-sliding version of the pot bearing.

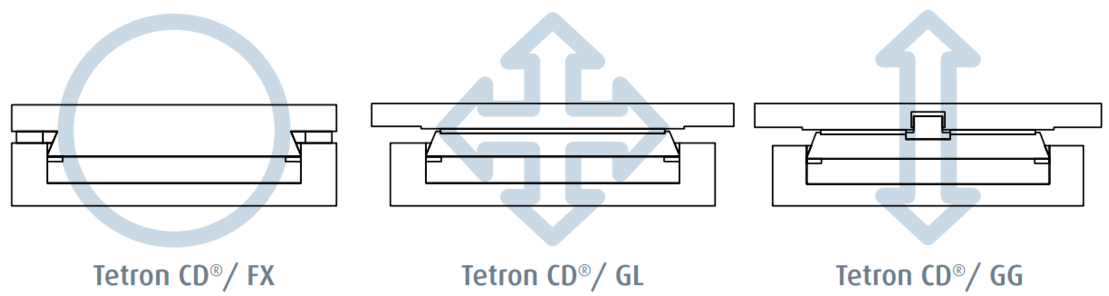


Figure 56: Pot Bearing Operating Modes [Source: Freyssinet]

Figure 56: Exploded view of the pot bearings [Source: Freyssinet]

The lower part of all 4 bearing devices is anchored to the foundation, while the upper part is welded to the longitudinal beam. Freyssinet recommends this type of bearing for vertical loads < 20,000kN and the maximum horizontal load capacity can be estimated at 30% of the vertical load capacity for standard models, which is higher than the calculated values stated above.

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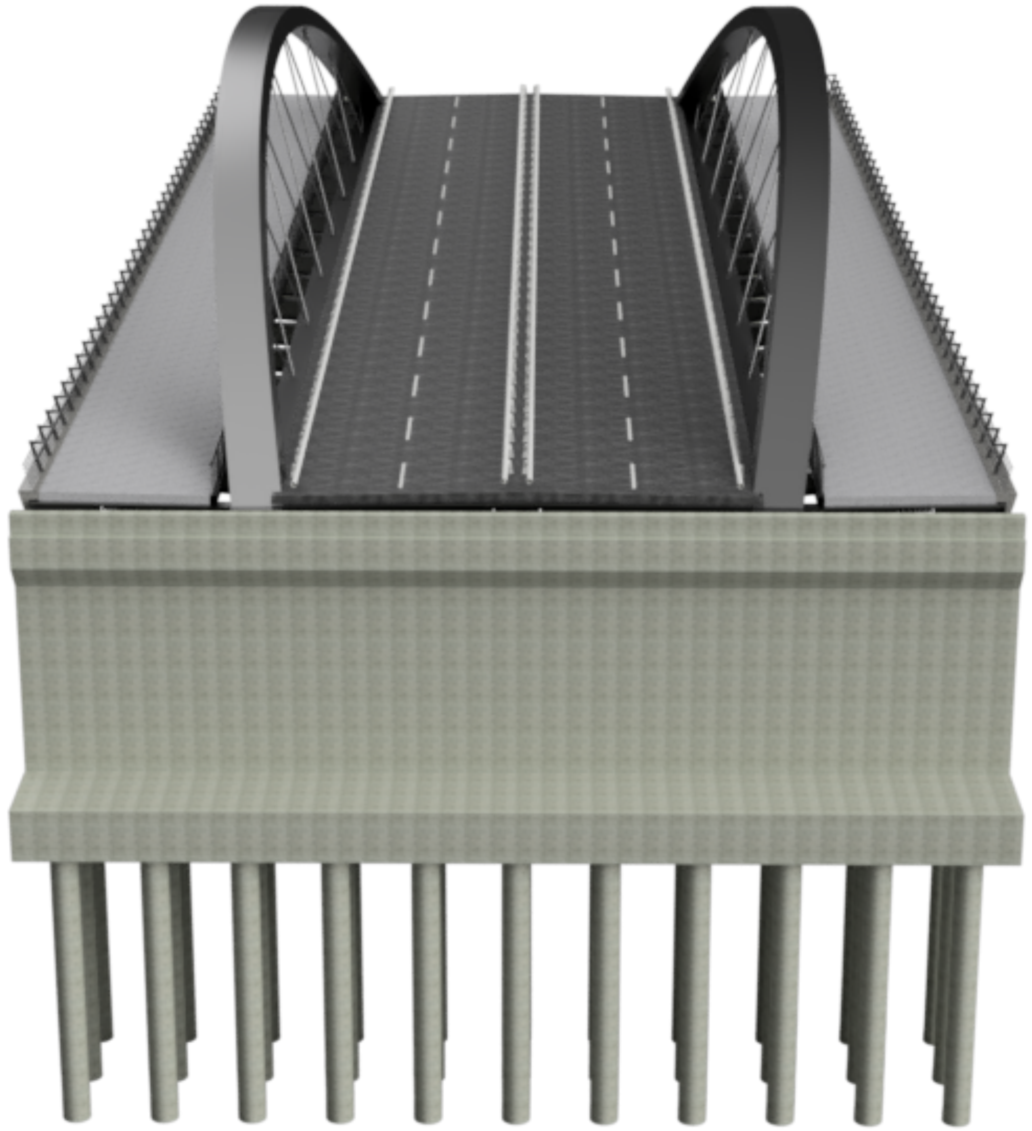
Appendix 5

Calculation of the Foundations



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DATE: JULY 2024

ACADEMIC YEAR 2024

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Calculation of Foundations

1. Introduction

This appendix deals with the calculation of the foundations required to transfer the forces calculated in "Appendix 4: Structural Analysis" from the columns to the ground. Only one foundation (on the Deusto side) is calculated here, since the acting forces and the soil on both sides of the bridge are very similar and the geometry of the foundations is the same.

2. Dimensioning of Retaining Wall and Bored Piles

In the case of our bridge, it is a channel bridge, so the bridge rests on retaining walls that act as a section of the channel walls on one side and hold back the soil on the other. Since the supporting ground (rock) is relatively deep, the retaining wall rests on bored piles that reach into the rock layer of the ground.

2.1 Dimensioning of Retaining Wall

Our retaining wall is a bridge retaining wall. The dimensions of the cross section of the foundation are designed following the publications "Estribos de puente de tramo recto" and "Projet et construction des ponts. Généralités-Fondations- Appuis-Ouvrages courants" (authors etc. further specified in the Bibliography). The exact dimensions can be found in the plans. Viewed from the front (from the channel), the retaining wall is 35 meters wide (slightly wider than the cross-section of the bridge) and follows the shape of the underside of the bridge cross-section on its upper side.

This appendix does not calculate the required reinforcement of the bridge retaining wall.

2.2 Dimensioning of Bored Piles

When dimensioning the bored piles, the aim is to establish the foundation in the rock layer, which starts at a depth of approximately 18,5 meters in relation to the original top edge of the terrain. We choose a length of 11 meters for the bored piles, which means that the piles will penetrate approximately 5 meters into the rock layer. Furthermore, a diameter of 1 meter is chosen for the piles. The centerline spacing of the piles is 3,20 m, which means that the distance between the piles is more than three times the diameter (3,20 m > 3 m). As a result, the piles are considered individually and not as a group of piles.

3. Calculation of Load-Bearing Capacity of Bored Piles

For the calculation of the bearing capacity of our bored piles, we follow the GCOC standard (Guía de cimentaciones en obras de carretera), issued by the Ministerio de Fomento. Chapter 5.10.1 of this standard describes the calculation of piles embedded in rock. In this case, the bearing capacity of a bored pile is calculated. This bearing capacity calculation applies to all other bored piles, since they are not a group of piles as described above.

For this purpose, the bearing capacity is first determined by peak pressure as follows:

$$Q_p = A_p \times q_p = 1{,}546\ MN$$

where:

A_p... Base area of the pile (1 m diameter => $A_p \cong 0{,}785 m^2$)

q_p... limiting pressure which is calculated as follows:

$$q_p = 2 \times p_{v,adm} = 1{,}97\ MPa$$

$P_{v,adm}$... admissible pressure for a shallow rock foundation according to section 4.5.3 of the GCOC, where B* = D

According to chapter 4.5.3, $p_{v,adm}$ is calculated as follows:

$$p_{v,adm} = p_0 \times \alpha_1 \times \alpha_2 \times \alpha_3 \times \sqrt{\frac{q_u}{p_0}}$$

where:

p_0... Reference pressure which is set as 1 MPa
q_u... Simple compressive resistance of intact rock
α_1 α_2 α_3... Dimensionless parameters that depend on the type of rock, the degree of alteration and the lithoclase spacing

The value for q_u can be taken from "Appendix 1 Geotechnical study, Chapter 6: Soil properties", as the value has already been determined there. The previously determined value for q_u is 33,01 MPa. We determine the coefficients α_1, α_2 and α_3 using Chapters 4.5.3.1-4.5.3.3 of the GCOC.

To determine α_1 (influence of rock type), we use Table 4.3 of the GCOC as a guide:

Group N°	Generic Name	Example	α_1
1	Carbonate rocks with well-developed structure	- Pure limestones, dolomites and marbles - Low porosity calcarenites	1,0
2	Igneous and metamorphic rocks (*)	- Granites, quartzites - Andesites, rhyolites - Slates, schists and gneisses (subhorizontal schistosity)	0,8
3	Sedimentary rocks (**) and some metamorphic rocks (**)	- Marly limestones, argillites, siltstones, sandstones, and conglomerates - Slates and shales (verticalized shales) - Gypsum	0,6
4	Poorly soldered rocks	- Sandstones, siltstones and conglomerates poorly cemented - Marls	0,4
	(*) Except for those indicated in groups 1 and 3. (**) Except for those indicated in groups 1 and 4.		

Figure 1: Values of α_1 according to the rock type (translated table from GCOC)

As it is a comparatively pressure-resistant siltstone with a q_u =33,01 MPa, we are in group 3 and therefore have an α_1 of 0,6.

The weathering grade is important for α_2 (influence of the grade of weathering), as this is specified by the GCOC:

- Weathering grade I (Healthy or fresh rock): $\alpha_2 = 1{,}0$
- Weathering grade II (Slightly weathered rock): $\alpha_2 = 0{,}7$
- Weathering grade III (Moderately weathered rock): $\alpha_2 = 0{,}5$
- When the degree of weathering is equal to or greater than IV, the specifications in section 4.5.3 of the GCOC (calculation as in soils) must be followed.

From the borehole samples it can be seen that the rock is class II, from which follows $\alpha_2 = 0{,}7$.

For α_3 the lower value from the factors α_{3a} and $\alpha_{3,b}$ is decisive. These are determined as follows:

$$\alpha_{3a} = \sqrt{\frac{s}{1m}} = 0{,}408 \qquad \alpha_{3b} = \sqrt{\frac{RQD(\%)}{100}} = 0{,}88$$

where:

s... Spacing between lithoclases expressed in m => s = 0,167
1m... Value used to make the corresponding expression dimensionless
RQD... Value of the "Rock Quality Designation" parameter (in %) => RQD = 78%

Both s and the RQD can be found in the borehole information.

As $\alpha_{3a} < \alpha_{3b}$ our value for $\alpha_3 = 0{,}408$.

Thus, by inserting these values, we obtain a value of $p_{v,adm} = 0{,}985$ which results in the values for q_p and Q_p given above.

According to the GCOC, the friction resistance of piles embedded in rock is only considered for rock with weathering classes 1-3 and is only effective at the level of embedment in the rock layer. Since we are in weathering class 2, we can consider the friction resistance for the approximately 5 meters that our piles penetrate into the rock layer. The friction resistance is calculated as follows:

$$Q_f = A_f \times \tau_f = 3{,}09\ MN$$

where:

τ_f... unitary resistance by shaft, within the rock embedment to be calculated as $\tau_f = 0{,}1 \times q_p = 0{,}197$
A_f... enveloping surface of the bored pile in the rock $A_f = \pi \times D \times L_f \cong 15{,}708\ m^2$

Adding the resistance due to peak pressure and skin friction we obtain a load-bearing capacity per bored pile of $Q_{R,tot}$ = 4,636 MN.

4. Calculation of the Forces Acting on the Piles

To determine the forces acting on the bored piles, the vertical forces determined in "Appendix 4: Calculation of the Structure Chapter 6.3.3 Bearing Pads" that act on the supports are important. In addition, the horizontal forces caused by braking, the vertical forces generated by the retaining wall and the horizontal forces caused by the soil and water pressing against the retaining wall must be determined.

4.1 Forces of the Superstructure

The maximum vertical forces from the superstructure calculated in Appendix 4 are approximately 12,942 MN and the minimum vertical forces are approximately 9,136 MN.

To calculate the horizontal forces due to braking, we refer to Chapter 4.1.3.1 of IAP-11, where these horizontal forces are calculated as follows:

$$Q_{lk} = 0{,}6 \times Q_{1k} + 0{,}1 \times q_{1k} \times w_1 \times L = 566{,}55\ kN$$

where:
$Q_{1,k}$... heavy vehicle => $2Q_{1,k} = 2 \times 300$ kN
q_{1k}... uniform load of usage => $q_{1k} = 9{,}0\ kN/m^2$

w_1... width of the virtual lane 1 => w_1= 3,0 m
L... distance between expansion joints/ length of bridge => L= 76,5 m

4.2 Forces due to the Bridge Retaining Wall and the Earth and Water Pressure

The forces acting on the bored piles through the bridge retaining wall, soil and water act as vertical forces. In addition, the earth and water pressure against the retaining wall act as horizontal forces, creating a moment.

4.2.1 Vertical forces due to the retaining wall, soil and water

For the vertical loads of the wall, we calculate the dead weight of the wall on the one hand and the vertical pressure of the soil and water acting on the base of the wall on the other hand.

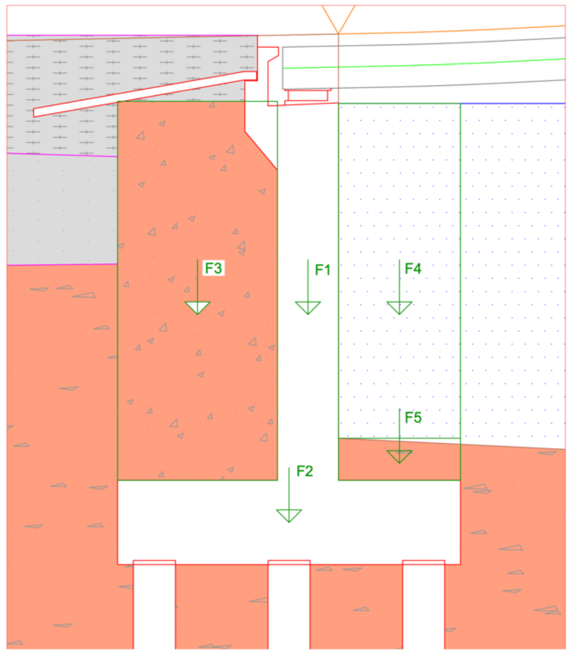


Figure 2: vertical forces due tot he retaining wall, sail and water

The results are as follows:

$F_1 = b_1 \times h_1 \times \gamma_c = 326,25$ kN/m
$F_2 = b_2 \times h_2 \times \gamma_c = 407,5$ kN/m
$F_3 = b_3 \times h_3 \times \gamma_e = 649,8$ kN/m
$F_5 = b_5 \times h_5 \times \gamma_e = 34,2$ kN/m

F_4 depends on the water level in the channel. Three cases are considered here: the high water level (7,95 m) and the medium water level (5,5 m) given in "Appendix 2: Hydrology, Chapter 4: Given hydraulic profile" and the low water level (3,5 m) (water has subsided to sea level).

$F_{4,1} = b_4 \times h_{4,1} \times \gamma_w = 226,17$ kN/m
$F_{4,2} = b_4 \times h_{4,2} \times \gamma_w = 156,47$ kN/m
$F_{4,3} = b_4 \times h_{4,3} \times \gamma_w = 99,57$ kN/m

where:
$\gamma_c = 25$ kN/m^3
$\gamma_e = 18$ kN/m^3 (as we reuse removed silt soil for backfilling, but increase the density slightly by compacting it)
$\gamma_w = 9,81$ kN/m^3

For the following calculation, all of these vertical forces, as well as the vertical loads due to the superstructure, are applied to the center of the contact area of the bridge retaining wall.

4.2.2 Horizontal Forces and Resulting Moments due to Earth Pressure and Water Pressure

The pressure of the soil and, on the opposite side, the pressure of the water against the retaining wall create moments at our reference point of the forces (center of the underside of the bridge retaining wall).

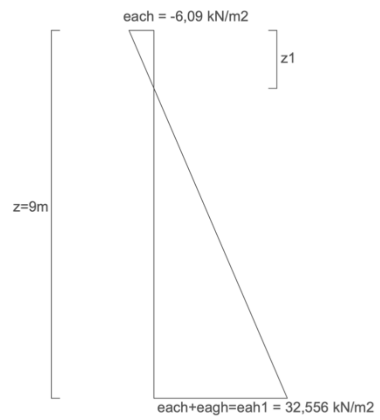
4.2.2.1 Horizontal Forces due to Earth Pressure

We calculate the earth pressure on the retaining wall according to Eurocode 7. To do this, the earth pressure is first determined as a distributed load:

z [m]	γ [kN/m³]	σ_z [kN/m²]	K_{agh}	e_{agh} [kN/m²]	K_{ach}	e_{ach} [kN/m²]	e_{ah} [kN/m²]	e^*_{agh} [kN/m²]	$e_{ah,res}$ [kN/m²]
0	18	0	0,226	0	0,812	-6,09	-6,09	0	-6,09
9,5	18	171	0,226	38,646	0,812	-6,09	32,556	30,609	32,556

where:
- z… Depth of the determined point
- γ… Density of the soil
- σ_z… Tension at depth z => $\sigma_z = z \times \gamma$
- K_{agh}… Earth pressure coefficient for active earth pressure due to dead weight (can be taken from table books, in our case taken from DIN 4085:2017)
- e_{agh}… Active earth pressure resulting from dead weight => $e_{agh} = \sigma_z \times K_{agh}$
- K_{ach}… Earth pressure coefficient for active earth pressure due to cohesion (can be taken from table books, in our case taken from DIN 4085:2017)
- e_{ach}… Active earth pressure resulting from cohesion => $e_{ach} = -c' \times K_{ach}$
- c'… dewatered cohesion, undrained cohesion given in "Appendix 1 Geotechnical study, Chapter 6: Soil properties", which can be used to determine c' with the aid of table books
- e^*_{agh}… Minimum pressure
- $e_{ah,res}$… max. result from $e_{agh} + e_{ach}$ or e^*_{agh}

Due to the cohesion, the earth pressure does not act on the retaining wall over the entire height. For this purpose, we determine the height from which the earth pressure actually acts on the wall:



$$z_i = |e_{ach}| \times \frac{z}{|e_{ah0}|+|e_{ah1}|} \cong 1{,}5m$$

This can now be used to calculate the resultant of the earth pressure:

$$E_{ah} = \frac{1}{2} \times e_{ah,res} \times (z - z_i) = 130{,}224 kN/m$$

On the water side, the 1 meter high silt strip is ignored, as no pressure is exerted on the wall due to the cohesion.

4.2.2.2 Horizontal Forces due to Water

When calculating the horizontal pressure on the wall due to the water, the different water levels must be taken into account in the same way as when determining the vertical forces:

$$E_{h,w1} = h_1 \times \gamma_w \times \frac{h_1}{2} = 310{,}01\ kN/m$$

$$E_{h,w2} = h_2 \times \gamma_w \times \frac{h_2}{2} = 148{,}38\ kN/m$$

$$E_{h,w3} = h_3 \times \gamma_w \times \frac{h_3}{2} = 60{,}09\ kN/m$$

4.2.2.3 Resulting Moments due to Horizontal Forces

To determine the maximum and minimum moment, the different situations are set up with the earth pressure force and the different water levels:

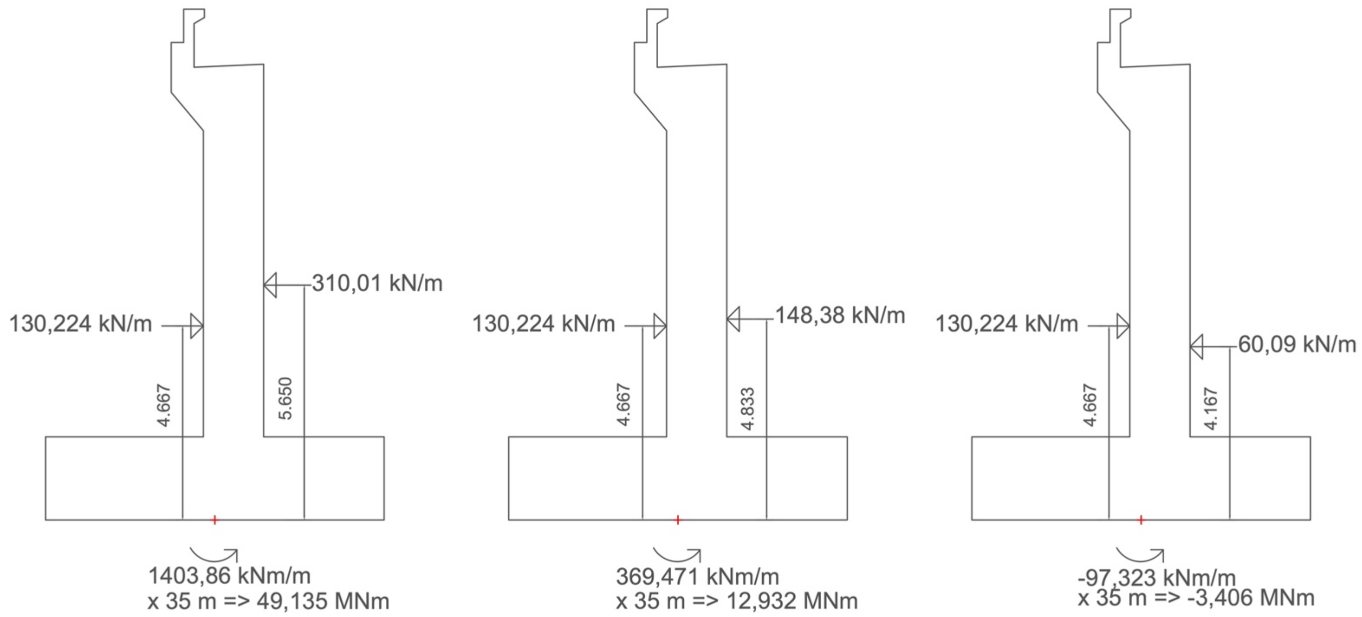


Figure 3: resulting moments due to horizontal pressures of soil and water

It can be seen that the maximum moment occurs at high water. To obtain the maximum moment, the braking action is added to the moment at high water with the direction of action in the direction of the support. This increases the existing moment due to its lever arm to the reference point. Since everything has been calculated in kN/m, the values must be multiplied by the length of the bridge retaining wall (35m). This gives us a maximum moment of 49,135 MNm and a minimum moment of -3,406 MNm.

4.3 Loadcases acting on the Piles

To analyze the bored piles, 4 load cases are considered: Max N & Max M, Max N & Min M, Min N & Max M, and Min N & Min M. To determine the load on the individual piles, first the loads on the three pile rows in the wall section are determined, and then the load on the individual piles is determined. In all cases, the vertical forces are divided by three (assuming that the forces are equally distributed among the three rows of piles). The moments are divided into vertical forces as follows, and thus act on one of the outer rows of piles as a tensile force and on the opposite row as a compressive force:

$$\Delta N = \frac{M \times d_i}{{d_i}^2}$$

There are two bearing pads on each side of the bridge, so the vertical forces calculated in "Appendix 4: Calculation of the Structure, Chapter 6.3.3 Bearing Pads" are applied twice.

Case Max N & Max M:

N = 2 × 12,942 MN + 57,537MN = 83,421 MN
M ↺ = 0,567 MN × 12,6 m × 35 m + 49,135 MNm = 56,274 MNm
Δ N = $\frac{56{,}274\ MNm \times 3{,}2m}{3{,}2\ m^2}$ = 17,587 MN

This results in the load pattern shown in the following figure. To determine the loads per row from the loads per pile, the load per row is divided by the number of piles (11) per row:

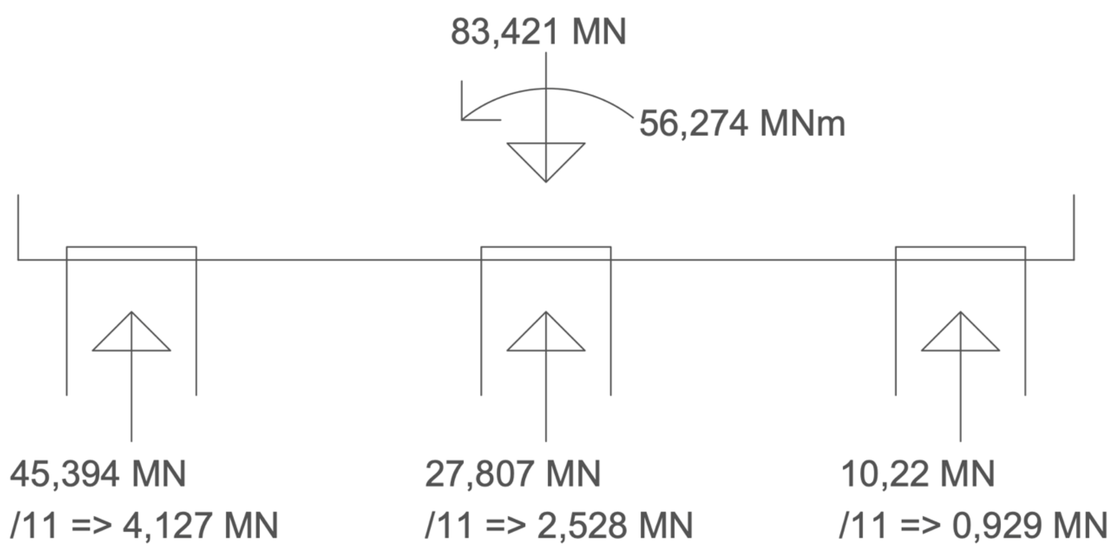


Figure 4: resulting forces on piles due to Max N & Max M

Case Max N & Min M: Here the Max N is slightly reduced because the lowest water level must be used for a Min M.

N = N = 2 × 12,942 MN + 53,106 MN = 78,99 MN
M ↺ = - 3,406 MNm
Δ N = $\frac{-3{,}406\ MNm \times 3{,}2m}{3{,}2\ m^2}$ = - 1,06 MN

This results in the following load on the pile rows and individual piles per row:

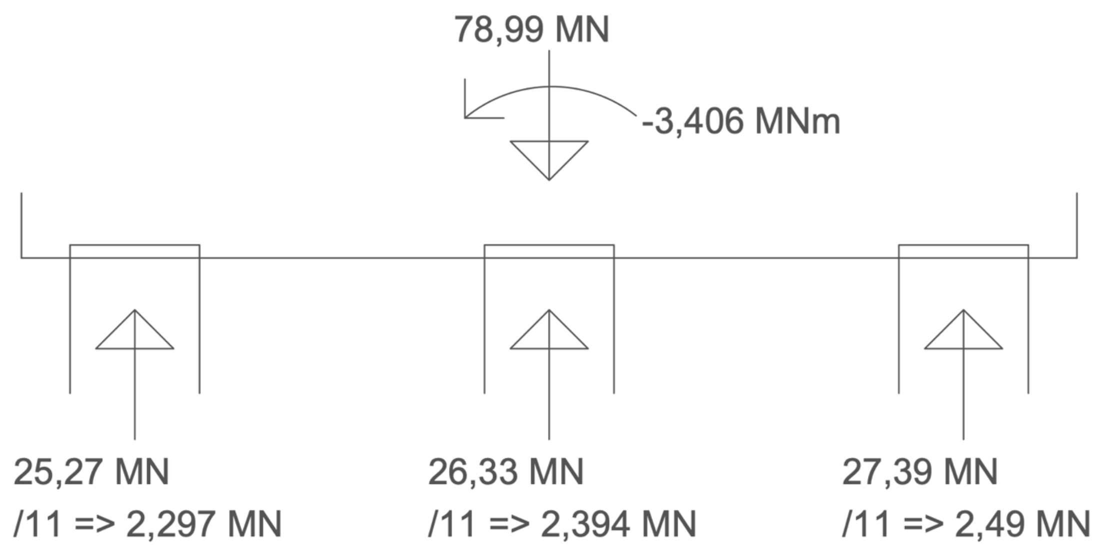


Figure 5: resulting forces on piles due to Max N & Min M

Case Min N & Min M:

N = N = 2 × 9,136 MN + 53,106 MN = 71,378 MN
M ↺ = - 3,406 MNm
Δ N = $\frac{-3{,}406\ MNm \times 3{,}2m}{3{,}2\ m^2}$ = - 1,06 MN

This results in the following load on the pile rows and individual piles per row:

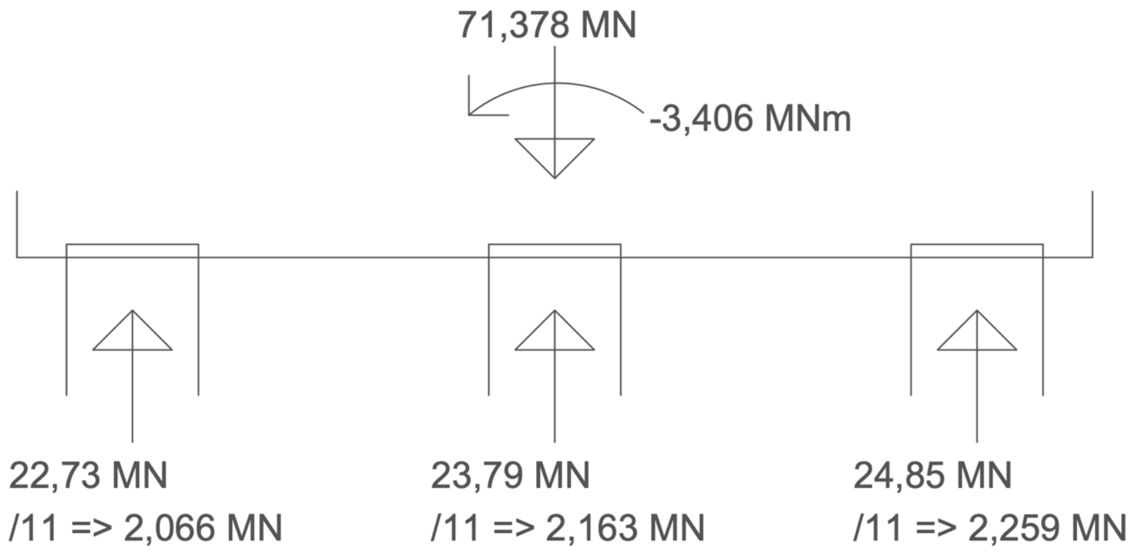


Figure 6: resulting forces on piles due to Min N & Min M

Case Min N & Max M: A higher Min N is used than the actual possible Min N, since the flood level must be used for Max M.

N = N = 2 × 9,136 MN + 57,537 MN = 75,809 MN
M ↺ = 0,567 MN × 12,6 m × 35 m + 49,135 MNm = 56,274 MNm
Δ N = $\frac{56{,}274\ MNm \times 3{,}2m}{3{,}2\ m^2}$ = 17,587 MN

This results in the following load on the pile rows and individual piles per row:

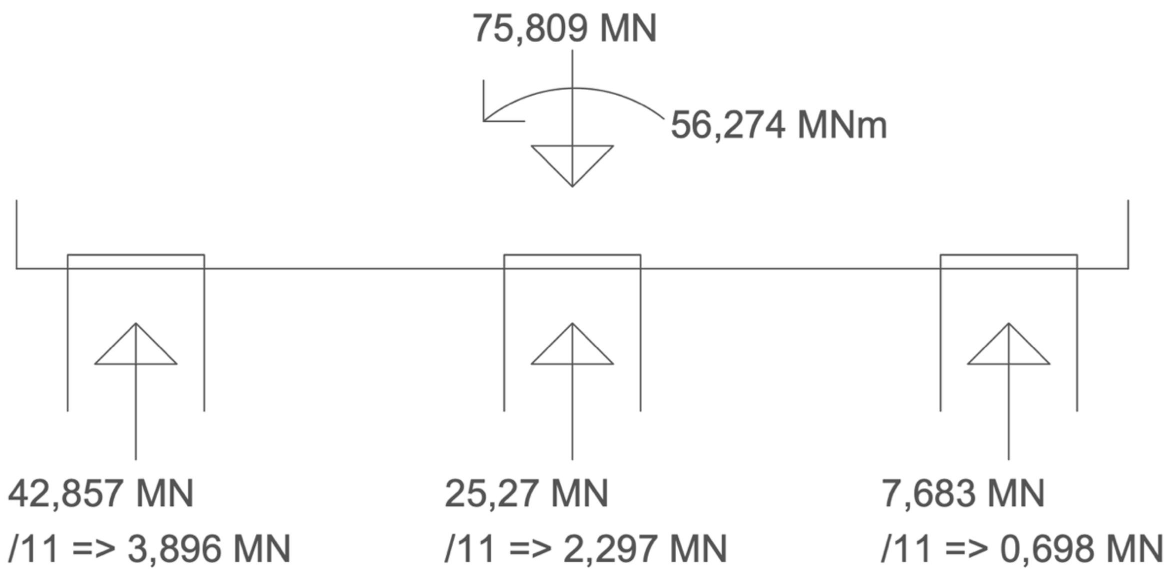


Figure 7: resulting forces on piles due to Min N & Max M

It can be seen that the maximum load occurs in the Max N & Max M case. However, the action value is lower than the resistance value per pile, which means that the piles are sufficiently dimensioned $Q_{E,max}$ = 4,127 MN < $Q_{R,tot}$ = 4,636 MN. The lowest load on the piles occurs at Min N & Max M. Since this force also acts as a pressure on the piles (0,698 MN > 0 MN), it is ensured that none of the piles is loaded in tension.

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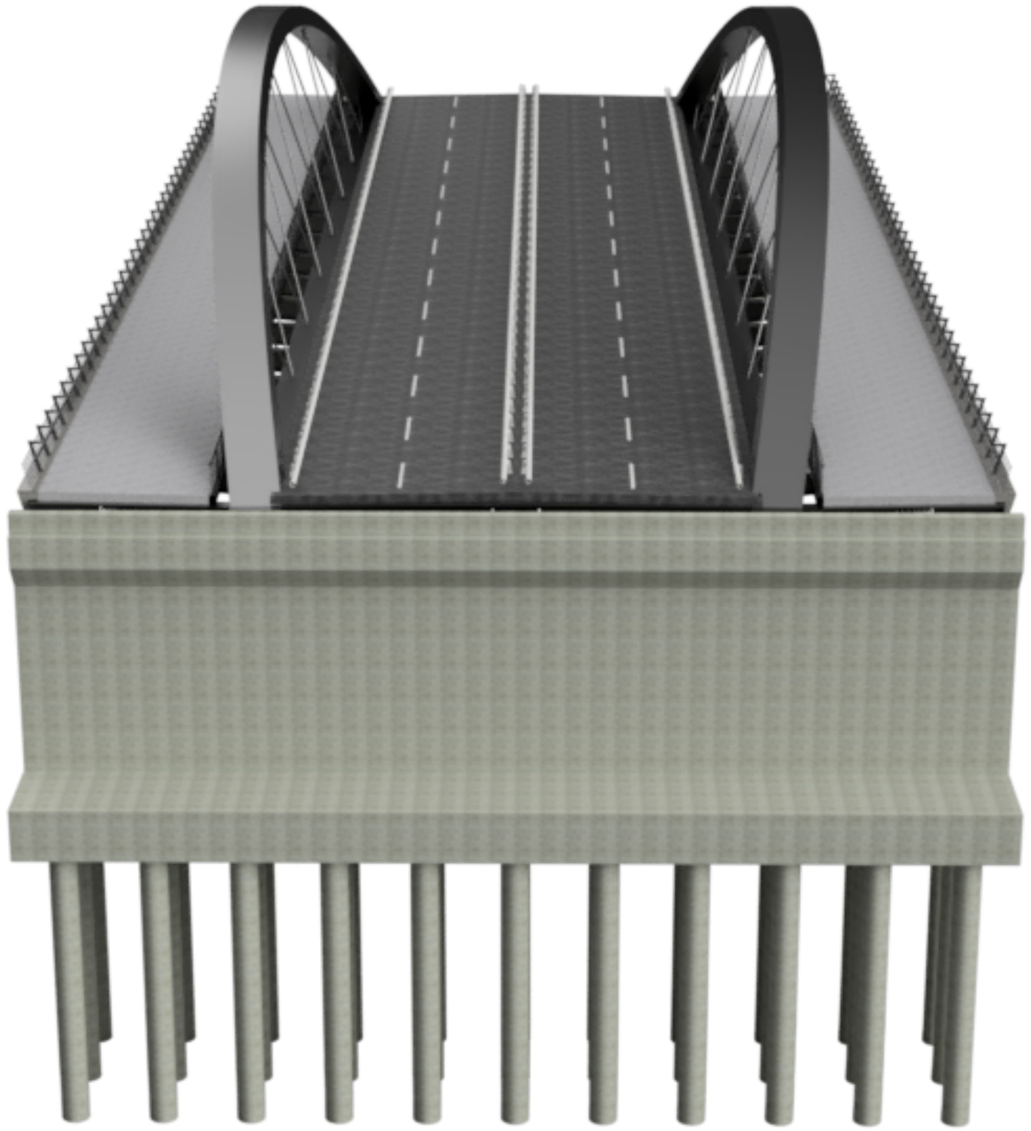
Appendix 6

Bridge Fittings



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DATE: JULY 2024

ACADEMIC YEAR 2024

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Bridge Fittings

1. Introduction

The aim of this appendix is to define the components of our bridge that do not fulfill a structural purpose for the bridge but are essential for a safe and comfortable use of the bridge.

2. Pipelines

The tendering company Euskal Trenbide Sarea has specified a number of pipelines that must be routed over the channel with the bridge. These are:

- Water supply pipes => diameters of 600 mm and 300 mm.
- Pneumatic garbage collection pipelines => diameter of 600 mm
- Two gas pipelines => diameter of 200 mm each
- Six electricity conduits => diameter of 200 mm each
- Ten telecommunications ducts => diameter of 110 mm each

We place these pipes underneath our bridge and let them run through cut-outs in our crossbeams. We place 3 of the 200 mm pipes and 5 of the 110 mm pipes together underneath the pedestrian walkways, as far outside as possible, as this is where the shear force is lowest.

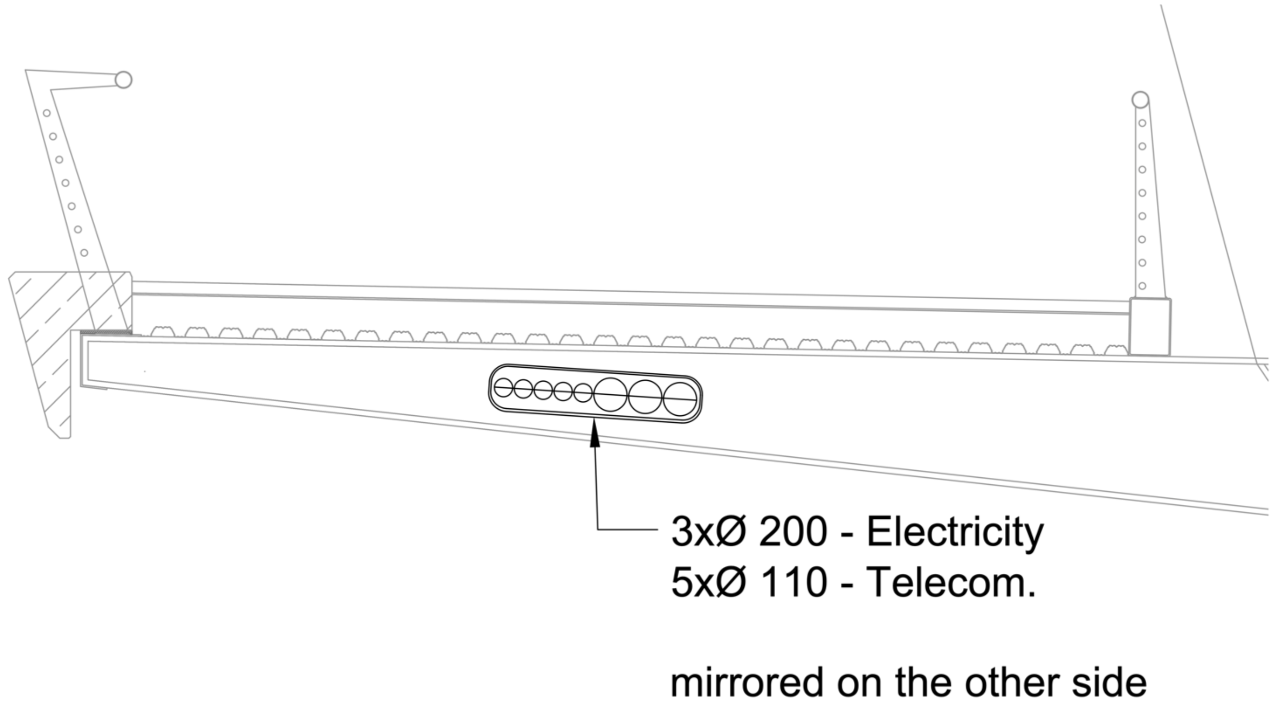

Figure 1: Pipelines pedestrian walkway

We place the remaining pipes (2 x 600 mm, 1 x 300 mm, and 2 x 200 mm) in two symmetrical cutouts near the center of the cross section (not exactly in the center because of the higher shear forces expected there due to the crash barriers). These cutouts are designed to be the same size to ensure an even distribution of forces. Since these equal-sized cutouts create more space for pipelines than is needed, three hollow tubes are placed in the right cutout, which are available in case of an increase in demand or failure of other pipes.

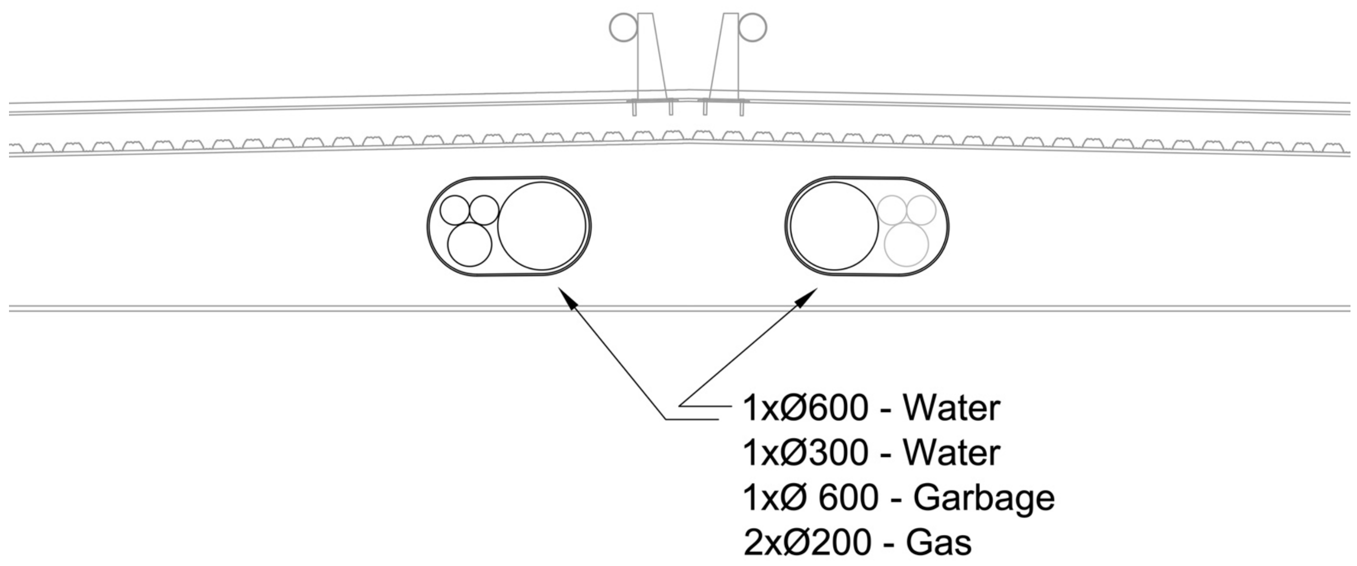

Figure 2: Pipelines in the middle of the bridge

3. Drainage

In the case of our bridge, the roadway has a slope of 2°, in our case we have sloped the walkway 2° in the opposite direction of the roadway, the slopes converge in the direction of the cable attachment. Since we have left a gap at this point (to improve the bridge experience, as pedestrians can look down into the channel), the water from the roadway can flow over the gap without any additional elements. As the inner handrail of the walkway is attached to a rectangular hollow section that is higher than the walkway, we make a 25 cm cutout in the rectangular hollow section in the center of every handrail section (every 2,02 meters (see chapter 5.2: Inner Guardrail)) to allow the water from the walkway to drain through the gap between the roadway and the walkway. Since the channel runs under the bridge, no special measures are required to drain the water.

4. Division of Lanes

The tender provides for two lanes (each with 2 lanes of 3,5 meters each) of 7 meters each and two pedestrian paths of 6 meters each. To make road markings possible a width of 7,20 meters is chosen for the lane of each direction. In our case, the lanes are separated in the middle by an area for barriers and lightning of approximately 1,01 meters. On the outside, areas for the placement of barriers of approximately 0,90 meters are provided on both sides.

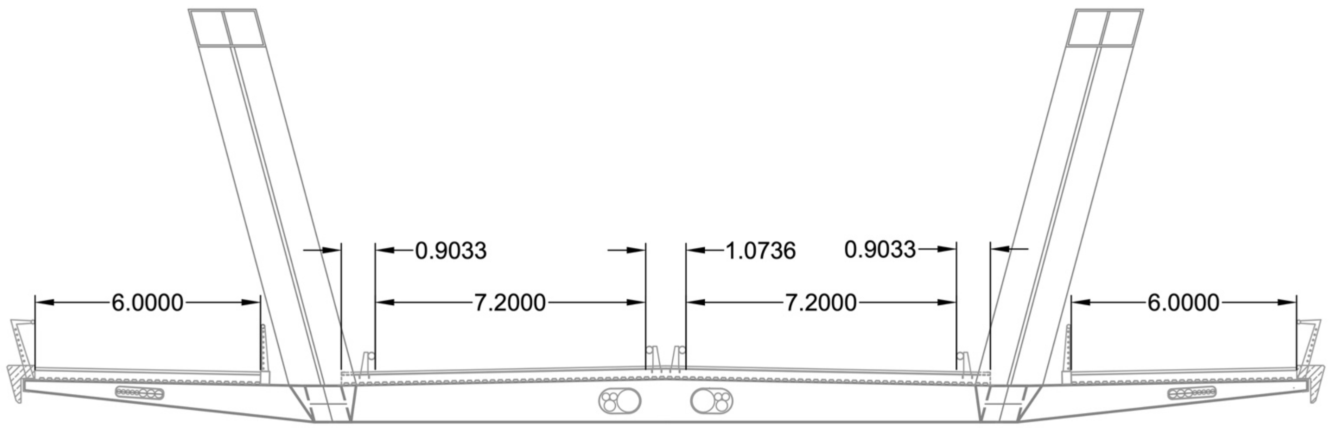

Figure 3: division of lanes

5. Guardrails

The safety features of the bridge also include a guardrail. In our case, we use our own designs. On the outside, we use guardrails with the same slope as our arches to create an aesthetic effect. On the roadway side, we use vertical guardrails to protect the gap in the floor. Lights are installed in the handrails of our guardrails to illuminate our bridge.

For dimensioning, we follow article 4.1.7 of IAP-11 (based on EN 1317-6), which states that a line load of at least 1,5 kN/m must be applied to the highest element. It is also specified that this force must act simultaneously with the uniform vertical loads from Article 4.1.2.2 of IAP-11 (see also Appendix 4: Calculation of the structure; Chapter 4.2.3 Loads in the pedestrian zone). In the dead loads for the calculation of the bridge (Appendix 4: Calculation of the structure; Chapter 4.1.2 Dead loads), each balustrade was considered with a load of 0,5 kN/m.

The guardrails are made of S235 steel.

5.1 Outer Guardrail

Figure 4: Model of outer guardrail

5.1.1 Section Handrail

For the handrail, we use a hollow steel tube with an outside diameter of 101,6 mm and a wall thickness of 4 mm. It should be noted that every 1 meter in the tube there are recesses of approximately 10 cm in which the lighting elements of the pedestrian area are installed. However, these holes are located between and never in the area of the supporting and connecting elements of the bridge. The following data for the calculation of the hollow steel tubes can be taken from tables: $A = 12{,}3\ cm^2$; $I = 146\ cm^4$; $W_{el} = 28{,}8 cm^3$.

This information can be used to calculate the maximum absorbable moment:

$$M_{Rd,HR} = \frac{f_{yd} \times W_{el}}{\gamma_{m,0}} = 6{,}768\ kNm$$

The handrail is considered to be a 2 meter long beam fixed on both sides. The maximum moment can be calculated as follows:

$$M_{Ed,HR} = -\frac{ql^2}{12} \times \gamma_Q = -0{,}75 kNm \Rightarrow |M_{Ed,HR}| = 0{,}75\ kNm$$

From which follows: $M_{Rd,HR} > |M_{Ed,HR}|$, which means that the cross section is sufficiently dimensioned.

5.1.2 Section connection-point

In the area of the connection with the crossbeam, the cross-section of the outer guardrails is rectangular with the following properties: A = 40 cm^2; I = 1333,3 cm^4; W_{el} = 133,33 cm^3.

This allows to calculate the absorbable moment of the cross-section at this point:

$$M_{Rd,SS} = \frac{f_{yd} \times W_{el}}{\gamma_{ym,0}} = 31{,}33\ kNm$$

As the guardrail functions as a cantilever, the moment in the area of the support can be calculated as follows:

$$M_{Ed,SS} = q \times l \times h \times \gamma_Q = 7{,}155\ kNm$$

From which follows: $M_{Rd,SS} > |M_{Ed,SS}|$, which means that the cross section is sufficiently dimensioned.

5.1.3 Connection with Bridge Superstructure

The outer guardrail is welded to a C-shaped end plate of the crossbars. The tensile and compressive forces on the welds are determined:

$$M_{Ed} = \frac{M_{Ed,SS}}{h} = 35{,}775 kN$$

h… Height of guardrail support in horizontal section in the connection area with the crossbeam [20 cm]

The shorter welds of the superstructure of 2 cm are mainly responsible for the load transfer of these determined forces. This gives us a compressive or tensile load on these welds of $F_{w,Ed}$ = 17,89 kN/cm.

Weld thicknesses can be determined by using weld force tables. In the case of the guardrail, the steel grade is S235 and in the case of the end plate of the traverse, S355. The lowest steel grade, S235, is used. This means that a minimum of one 9 mm thick weld is required, with a limit force of 18,71 kN/cm.

5.2 Inner Guardrail

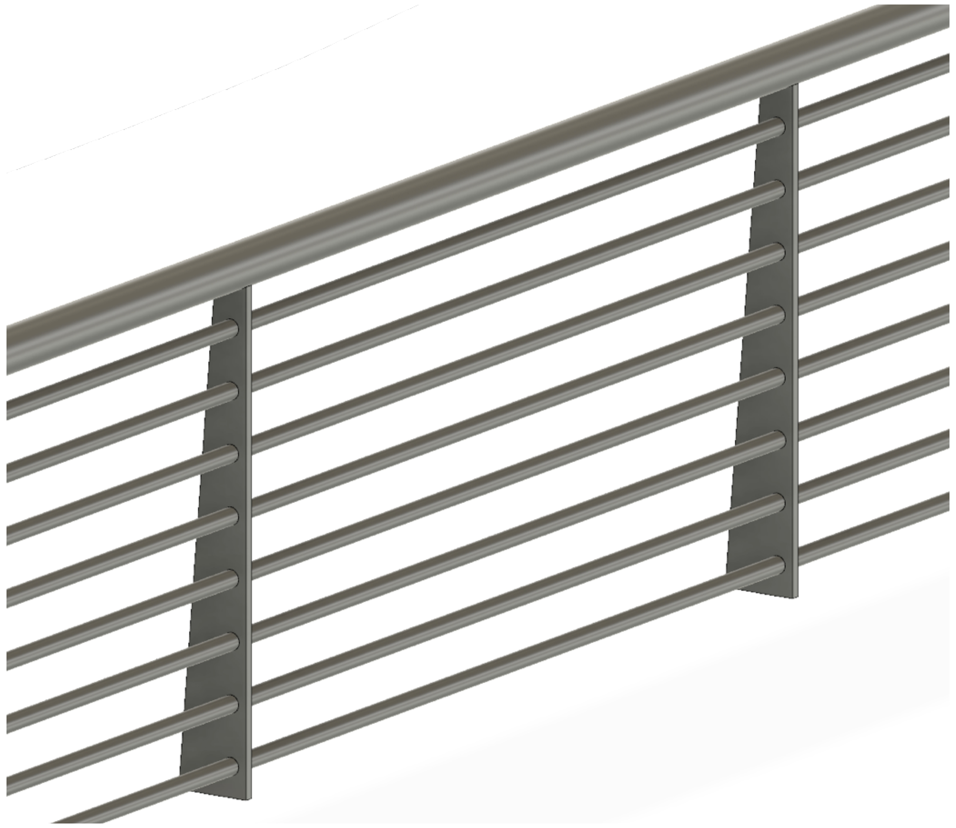

Figure 5: Model of inner guardrail

5.2.1 Section Handrail

The same shape is used for the handrails as for the outer guardrails, so this check can be applied to the inner handrails (Chapter 5.1.1 Handrail section). This means that the handrails are made of hollow steel tubes with an outside diameter of 101,6 mm and a wall thickness of 4 mm.

5.2.2 Section Connection-Point

The cross-section in the area connecting the inner guardrail to the bridge structure is rectangular with the following properties: A = 36 cm^2; I = 972 cm^4; W_{el} = 108 cm^3.

This allows the absorbable moment of the cross-section at this point to be calculated:

$$M_{Rd} = \frac{f_{yd} \times W_{el}}{\gamma_{m,0}} = 25{,}38\ kNm$$

Since the connection structure with the bridge is different for the inner guardrail, the lever arm of the force to be applied is smaller, so the moment in the area of the support in the inner guardrail is different from that in the outer guardrail. This results in the following action:

$$M_{Ed} = q \times l \times h \times \gamma_Q = 5{,}58\ kNm$$

From which follows: $M_{Rd} > |M_{Ed}|$, which means that the cross-section in the area of the support is sufficiently dimensioned.

5.2.3 Connection with Bridge Superstructure

In the case of the inner guardrails, they are welded to a 35 cm high by 25 cm wide 8 mm thick rectangular hollow section welded to the cross beams. These welds and the forces acting on them are calculated in the same way as for the outer guardrails.

In this case, the forces in the area between the guardrail and the rectangular hollow section are 31 kN because the width of the guardrail bracket in the area connected to the crossmember is 18 cm. This results in a compressive or tensile load on the shorter (in width) weld seams forces of $F_{w,Ed}$ = 15,5 kN/cm.

The weld seam thicknesses can be dimensioned for this load using table books. Both the railing and the hollow profile are made of S235. This is followed by a weld seam thickness of 8 mm, which has a limit load of $F_{w,Rd}$ = 15,63 kN/cm.

In the area of rectangular hollow section-beam, the compressive and tensile loads are 28,62 kN, 5 cm long welds are used for dimensioning (in reality, the welds run along the entire width of the crossbeams). This gives us a load on the welds of 5,73 kN/cm. As mentioned earlier, the rectangular hollow section is also made of S235 and the traverse is made of S355. As with the connection of the outer guardrails to the crossbeam, the lower strength class, S235, is decisive. This means that at least 3 mm thick welds with a limit load $F_{w,Rd}$ = 6,24 kN/cm are required. However, to further stabilize the hollow section, vertical plates are welded on every 2 meters (in the area where the guardrails are connected to the hollow section):

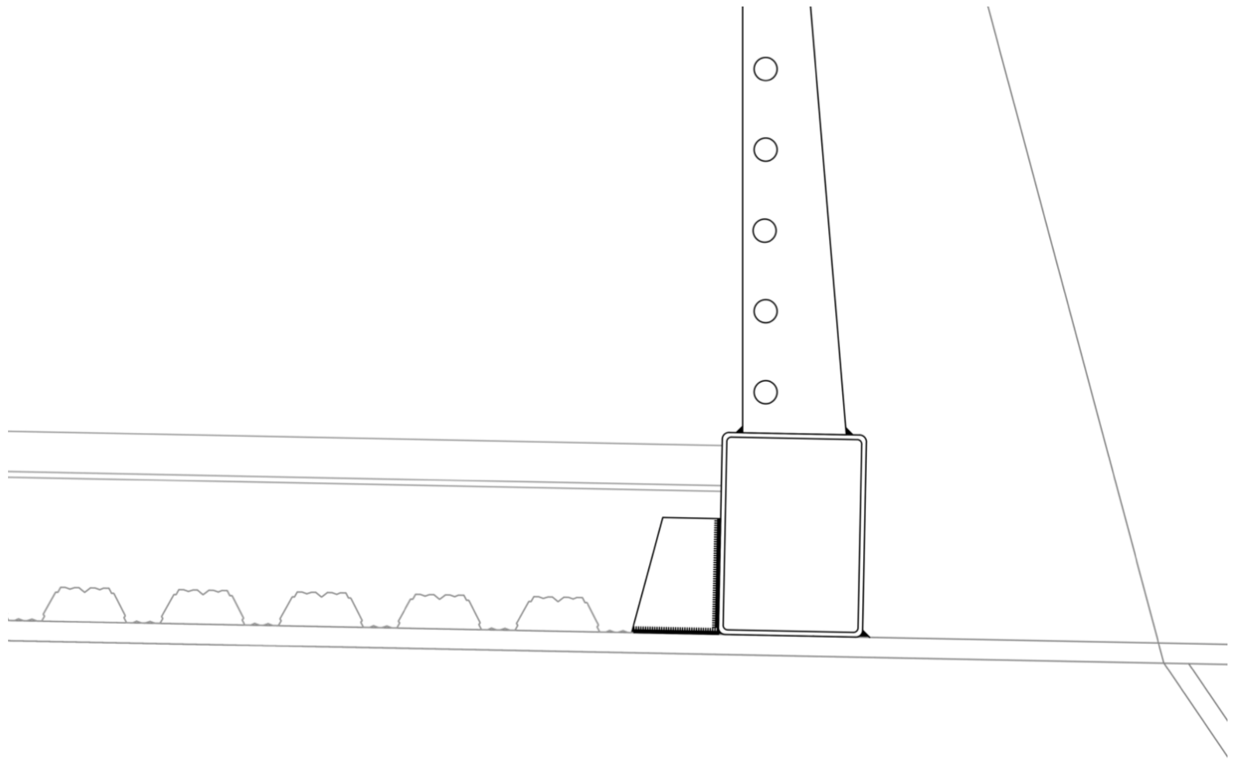

Figure 6: Detail of the fixing plate

6. Cornice

In the case of cornices, we use a custom designed model. The cornice is made up of 2-meter-long sections, which makes it relatively easy to handle during transportation and installation. In addition, the supports of the handrails, which also have a distance of 2 meters, can always be fixed and run in the gaps between the cornices. The outer surface of the cornice is sloped at the same angle as the arch and the handrails. The corners are chamfered because simple concrete corners are very susceptible to damage and break off easily. A gap is left between the concrete slab and the location of the cornice when the concrete slab is poured. A reinforcement protrudes from the concrete layer into this gap, which is connected to a reinforcement protruding from the cornice when the cornice is placed. The gap is then concreted as well, connecting and anchoring the cornice to the rest of the concrete slab. For the anchoring reinforcement, we use iron with a diameter of 14 mm every 15 centimeters. The cornice rests on a 2 cm thick layer of mortar on the end plate of the cross beams. For longitudinal reinforcement, we use 8 bars with a diameter of 10 mm.

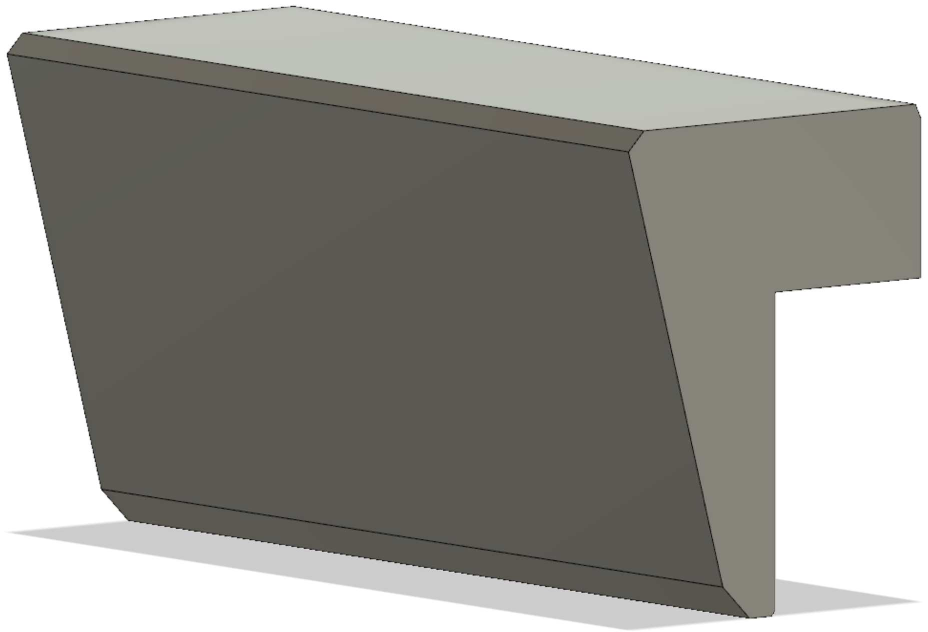

Figure 7: Model of cornice

7. Crash Barriers

The crash barriers are also self-designed. The models are similar in the middle and outside of the roadway.

Regarding the sizing of the crash barriers, IAP-11 states in Article 5.1.2 that the manufacturer is responsible for calculating the crash barriers and ensuring that the system is replaceable (in the event of an impact). Nevertheless, we roughly calculate the crash barriers to ensure that the chosen dimensions are reasonable.

Since the IAP-11 does not provide specific design loads, we follow the 1998 IAP (the predecessor of the IAP-11), which states that a design load of 45 kN should be applied at a height of 0,6 meters.

The crash barriers are made of S235 steel.

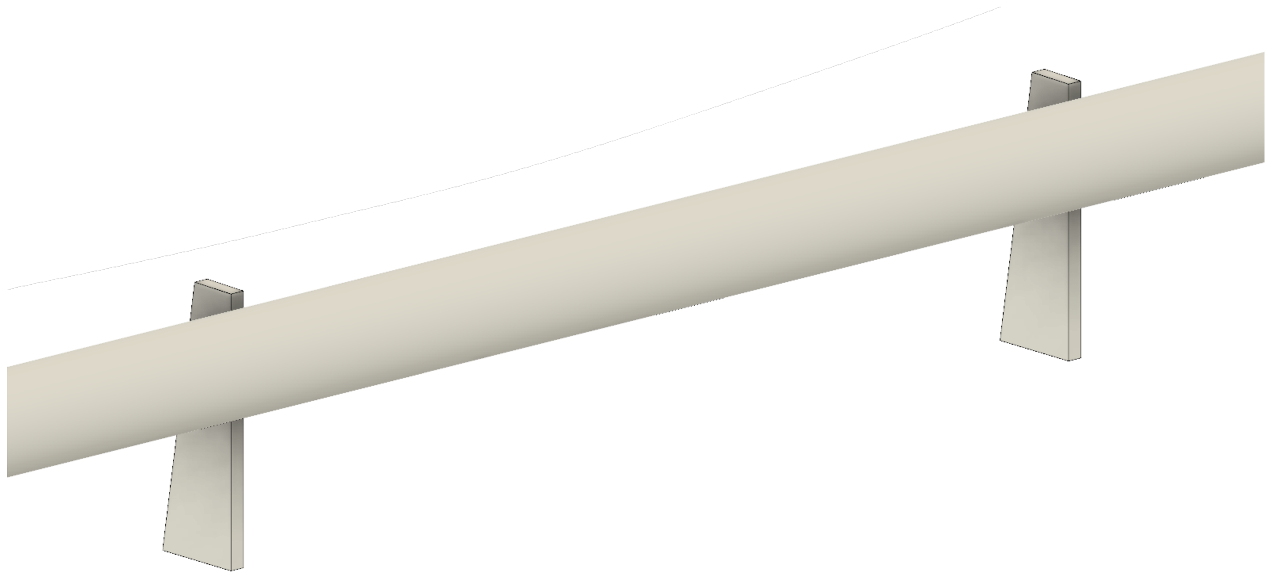

Figure 8: Model of crash barrier

7.1 Crash Barrier Tube

For the design, we consider the guardrail as a beam fixed on both sides with a support distance of 2 meters. In this case, we apply the force in the middle to get the maximum moment in the support area.

$$M_{Ed,Gt} = -\frac{FL}{8} \times \gamma_Q = -16{,}875kNm \Rightarrow |M_{Ed,Gt}| = 16{,}875\ kNm$$

For our crash barrier, we choose a tube with a diameter of 193,7 mm and a thickness of 6,3 mm, which gives us a maximum absorbable moment of:

$$M_{Rd,Gt} = \frac{f_{yd} \times M_{el}}{\gamma_{m,0}} = 26{,}32\ kN$$

From which follows $|M_{Ed,Gt}| < M_{Rd}$, which means that the cross-section in the area of the support is sufficiently dimensioned.

7.2 Crash Barrier Support

In the connection area between the support and the mounting plate, the cross-section of the support has a height of 200 mm and a width of 30 mm, giving A = 60 cm^2 I = 2000 cm^4, W_{el} = 200 cm^3.For the calculation in this area, we apply the force directly to the support at a height of 0,6 meters, which gives us the following moment in the connection area:

$$M_{Ed,Gs} = Q \times h \times \gamma_Q = 40{,}5\ kNm$$

The maximum absorbable moment in this area ist:

$$M_{Rd,Gs} = \frac{f_{yd} \times W_{el}}{\gamma_{M,0}} = 47\ kNm$$

Since $|M_{Ed,Gs}| < M_{Rd,Gs}$ the cross-section in this area is sufficiently dimensioned.

7.3 Connection with Bridge Structure

The crash barrier is attached to the concrete layer of the roadway with epoxy screws. To dimension these, we first calculate the forces acting on them. The moment created by the lever arm results in tensile and compressive forces on the bolts, and there is also a shear force due to the horizontal force.

Tensile and compressive forces:

$$H_{t,Ed} = \frac{M_{Ed,Gs}}{h} = 162\ kN$$

h... Height of the guardrail support in horizontal section Connection area with the crossbeam [20 cm]

On the one hand, 162 kN acts as a compressive force on one side of the guardrail base plate, and on the other hand, 162 kN acts as a tensile force on the other side of the base plate.

Shearing force:

$$F_{v,Ed} = H_{Ed} = \ 45kN$$

H_{Ed}... force of action given by IAP (45 kN)

We can use these values to dimension our bolts. We consider four bolts 250 mm apart in height and 80 mm apart in width. This means that our tensile and compressive forces are distributed over two bolts each, so each bolt is subjected to a load of 81 kN. Suitable bolts can be found in tables. For a tensile force of 81 kN, an M20-5.6 with a limit tensile force of $F_{t,Rd}$ = 88,2 kN per bolt can be used. This type of bolt provides a shear limit of $F_{v,Rd}$ = 75,4 kN per bolt. Since the shear force action value can already be absorbed by one bolt, this type of bolt can be used (we use 4 of these bolts). Finally, the combination of both forces must be checked:

$$\frac{F_{v,Ed}}{2 \times F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \times 4 \times F_{t,Rd}} = 0{,}88 \leq 1$$

As the combination value is less than 1, we can use 4 screws of type M20-5.6.

8. Pavements

The choice of road and walkway surfaces also plays an important role in the safe, comfortable use and durability of our bridge. The following chapter deals with the selection of road and walkway surfaces.

8.1 Road Pavement

The pavement structure consists of a sealing layer, a protective layer and a surface layer.

- To prepare for the application of the waterproofing layer, a concrete coat is first applied to our concrete surface. This must be thoroughly cleaned after drying and prior to the application of the waterproofing coating to ensure adhesion of the waterproofing coating. Cleaning can be accomplished by mechanical stripping, sandblasting, wire brushing and water or compressed air.
- The following different options are available for waterproofing the bridge: In-situ waterproofing membranes (in adhesive systems in thin layers), bituminous mastics and waterproofing with prefabricated membranes. In our case, we use bituminous membranes (in-situ membranes) because they are easier to apply (since we are dealing with straight, long stretches without obstacles). For this purpose, an epoxy primer is applied as a bond between the concrete plateau and the bituminous membranes. This is followed by two layers of bituminous waterproofing. The lower layer can be either poured or applied as a flame layer, while the upper layer is applied as a flame layer. The entire waterproofing structure is approximately 8 mm thick.
- There are two asphalt layers on top of the waterproofing, one is a protective asphalt layer and the other is an asphalt surface layer.

The "Norma para el dimensionamiento de firmes de la red de carreteras del país vasco" contains a map of the thermal summer zones:

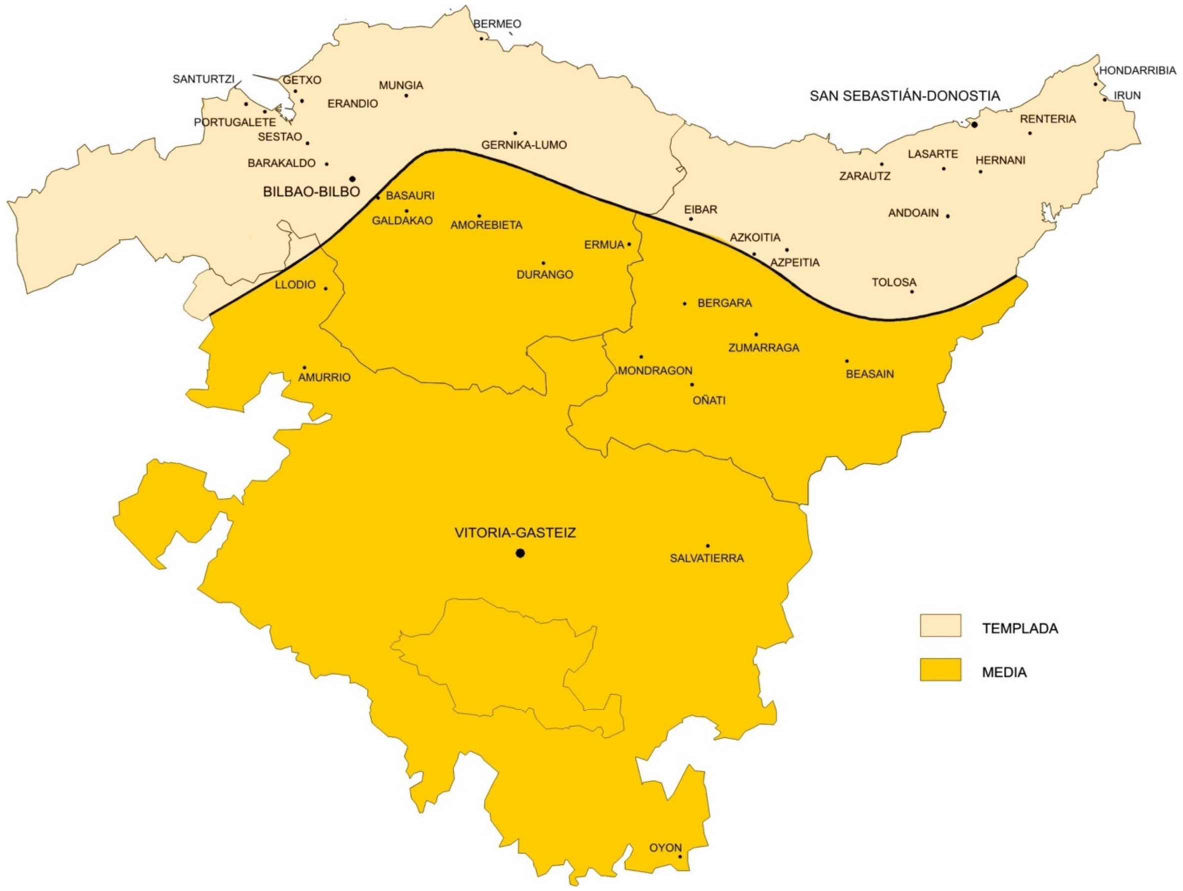

Figure 9: Map of Basque Country of the definition of summer thermal zones

On this map, Bilbao is located in the moderate zone. We estimate the traffic load of our bridge in category T2. Tables 542.1 a) & b) of the PG-3 show that softer asphalt mixes can be used due to the temperate zone. In other words, according to Tables 542.1 b), asphalt class 50/70 should be used for the protective layer.

ZONA TÉRMICA ESTIVAL	CATEGORÍA DE TRÁFICO PESADO			
	T00	T0	T1	T2 y T3
CÁLIDA	35/50 BC35/50 PMB 25/55-65		35/50 50/70 BC35/50 BC50/70	50/70 BC50/70
MEDIA				50/70 70/100 BC50/70
TEMPLADA	50/70 70/100 BC50/70			70/100

Figure 10: Table 542.1 b) PG-3 Type of hydrocarbon binder to be used in the base layer

Asphalt classes 50/70, 70/100, BC50/70 or PMB45/80-60 may be used for the surface layer in accordance with Table 542.1 a).

ZONA TÉRMICA ESTIVAL	CATEGORÍA DE TRÁFICO PESADO					
	T00	T0	T1	T2 y T31	T32 y ARCENES	T4
CÁLIDA	35/50 BC35/50 PMB 25/55-65 PMB 45/80-65		35/50 BC35/50 PMB 25/55-65 PMB 45/80-60 PMB 45/80-65	35/50 50/70 BC35/50 BC50/70 PMB 45/80-60	50/70 BC50/70	50/70 70/100 BC50/70
MEDIA	35/50 BC35/50 PMB 45/80-60 PMB 45/80-65		35/50 50/70 BC35/50 BC50/70 PMB 45/80-60	50/70 BC50/70 PMB 45/80-60	50/70 70/100 BC50/70	
TEMPLADA	50/70 BC50/70 PMB 45/80-60 PMB 45/80-65		50/70 70/100 BC50/70 PMB 45/80-60			

Figure 11: Table 542.1 a PG-3) Type of hydrocarbon binder to be used in surface layer

Since we expect a medium traffic volume (T2), but it can still get hot in Bilbao in the summer, we do not choose the softest asphalt mix to be on the safe side. To avoid having to use different asphalt mixes for the wearing and protective layers, it makes sense to use 70/100 for the surface layer as well. We build the two layers to a thickness of about 7 cm, giving a total structure of about 8 cm.

8.2 Pedestrian Walkway Pavement

The walkway is waterproofed in the same way as the deck (see Appendix Bridge Fittings; Section 8.1). The pavers are fixed to the waterproofing with a layer of mortar approximately 2 cm thick.

When choosing the paving stones to be used, there are many models available on the market. In our case, we use the "Terana light" model from Breinco. They are 5 cm thick. They are joined with mortar in the joints (this also means that rainwater can run off above the stones and does not seep through the joints into the lower part of the system).

Figure 12: Paving stones Terana light [source Breinco]

With the chosen overall setup, we end up with a height of about 8 cm, which is also the height of the roadway structure.

9. Illumination

The following point serves to determine the appropriate illumination of the bridge to ensure optimum visibility, comfort and safety.

In our bridge, there are lighting elements for the roadway on the one hand and lighting elements for the pedestrian paths on the other hand. The efficiency of the selected lighting can be calculated with the Dialux software. Element templates from the Dialux catalog were used to represent both lighting elements. Both types of LED lighting.

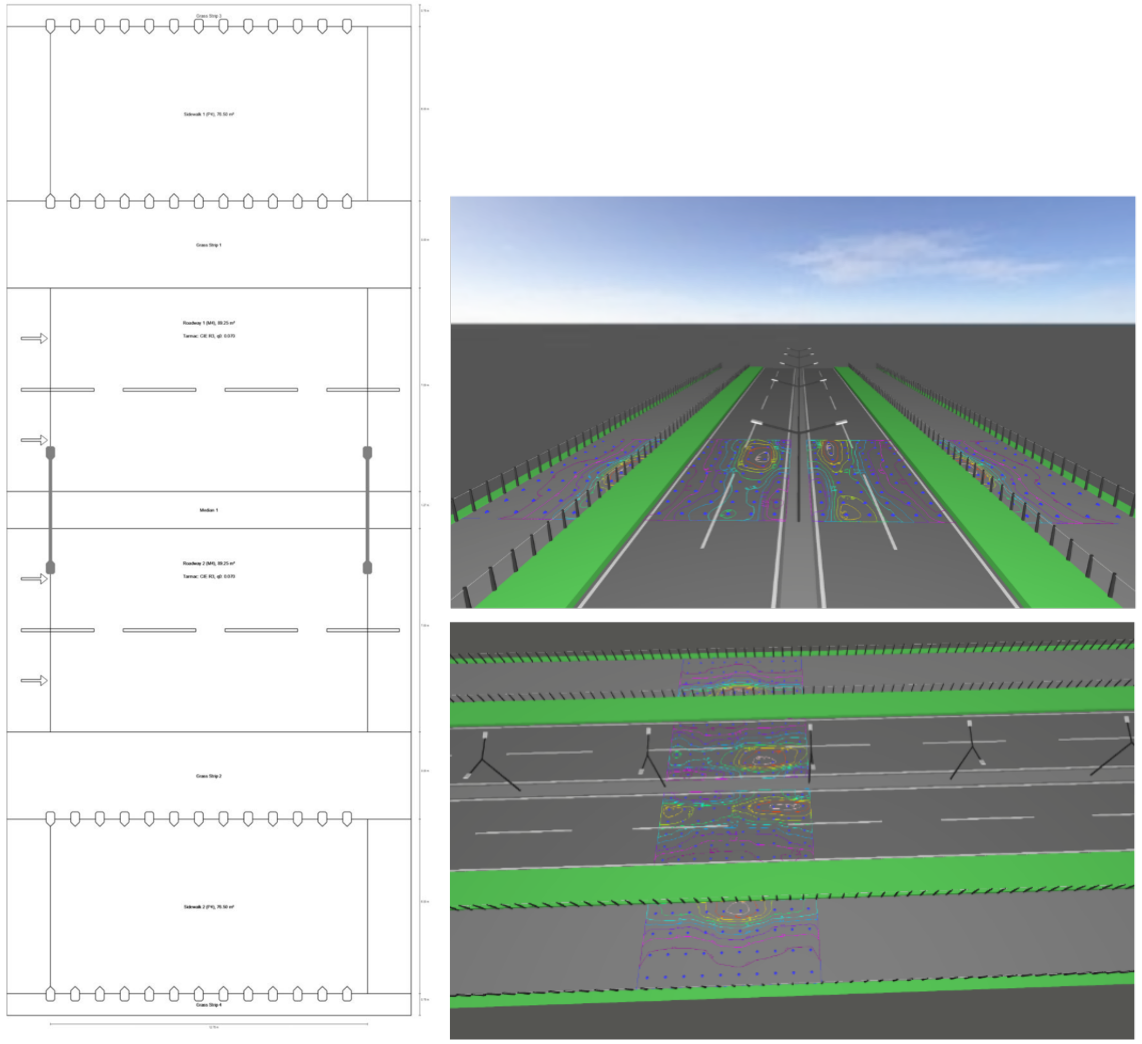

Figure 13: illumination along the length of the bridge.

To illuminate the roadway, a streetlight with two lamps was placed every 12,75 meters in the center between the two directions of the roadway. These elements are located in the center of the two lane directions. The lighting modules are located at a height of 5 meters and protrude from the roadway.

Bridge Bilbao

Summary (according to EN 13201:2015)

Manufacturer	Dien Quang
Article No.	-
Article name	Den Helios 1- H1 80740 9K532L900A - 02A1 P50-GR
Fitting	1x Den Helios 1- H1 80740 9K532L900A - 02A1 P50-GR

P	79.0 W
Φ_{Lamp}	10116 lm
$\Phi_{Luminaire}$	10115 lm
η	99.99 %

Den Helios 1- H1 80740 9K532L900A - 02A1 P50-GR (Median, 2 per pole)

Pole distance	12.750 m
(1) Light spot height	5.000 m
(2) Light point overhang	1.300 m
(3) Boom inclination	15.0°
(4) Boom length	2.003 m
Annual operating hours	4000 h: 100.0 %, 158.1 W
Wattage / route	12329.0 W/km
ULR / ULOR	0.00 / 0.00
Max. luminous intensities Any direction forming the specified angle from the downward vertical, with the luminaire installed for use.	≥ 70°: 529 cd/klm ≥ 80°: 211 cd/klm ≥ 90°: 51.2 cd/klm
Luminous intensity class The luminous intensity values in [cd/klm] for calculation of the luminous intensity class refer to the luminaire luminous flux according to EN 13201:2015.	-
Glare index class	D.5
MF	0.80

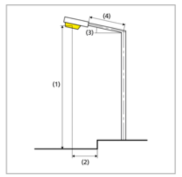

Figure 14: Lamp to be used according to the catalog for roadway lightning

To illuminate crosswalks, LED strips are embedded in the handrails of the fall protection.

Manufacturer	Golnoor
Article No.	8W
Article name	LED-Strips
Fitting	user-defined

P	8.0 W
Φ_{Lamp}	5 lm
$\Phi_{Luminaire}$	5 lm
η	100.00 %

LED-Strips (both sides opposite)

Pole distance	1.000 m
(1) Light spot height	1.150 m
(2) Light point overhang	-3.000 m
(3) Boom inclination	0.0°
(4) Boom length	0.000 m
Annual operating hours	4000 h: 100.0 %, 8.0 W
Wattage / route	16000.0 W/km
ULR / ULOR	0.02 / 0.02
Max. luminous intensities Any direction forming the specified angle from the downward vertical, with the luminaire installed for use.	≥ 70°: 88.0 cd/klm ≥ 80°: 27.6 cd/klm ≥ 90°: 9.29 cd/klm
Luminous intensity class The luminous intensity values in [cd/klm] for calculation of the luminous intensity class refer to the luminaire luminous flux according to EN 13201:2015.	G*3
Glare index class	D.6
MF	0.80

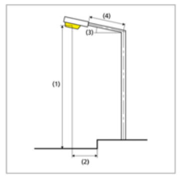

Figure 15: LED lights to be used according to catalog, for pedestrian sidewalk lightning

As can be seen in the results provided by Dialux, all our lighting is within the regulations given by EN 13201:2015.

Bridge Bilbao

Summary (according to EN 13201:2015)

Results for valuation fields

A maintenance factor of 0.80 was used for calculating for the installation.

	Symbol	Calculated	Target	Check
Sidewalk 1 (P4)	E_{av}	7.44 lx	[5.00 - 7.50] lx	✓
	E_{min}	4.33 lx	≥ 1.00 lx	✓
Roadway 1 (M4)	L_{av}	3.74 cd/m²	≥ 0.75 cd/m²	✓
	U_o	0.56	≥ 0.40	✓
	U_l	0.75	≥ 0.60	✓
	TI	14 %	≤ 15 %	✓
	R_{EI}	0.40	≥ 0.30	✓
Roadway 2 (M4)	L_{av}	3.49 cd/m²	≥ 0.75 cd/m²	✓
	U_o	0.57	≥ 0.40	✓
	U_l	0.63	≥ 0.60	✓
	TI	6 %	≤ 15 %	✓
	R_{EI}	0.40	≥ 0.30	✓
Sidewalk 2 (P4)	E_{av}	7.44 lx	[5.00 - 7.50] lx	✓
	E_{min}	4.32 lx	≥ 1.00 lx	✓

Figure 16: Comparison of results of street lighting used with EN 13201:2015

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- https://aut.sika.com/de/bau/abdichtung/weitere-abdichtungsanwendungen/bruecke/brueckenabdichtungmitpolymerbitumenbahnen.html (accessed: 08.05.2024)
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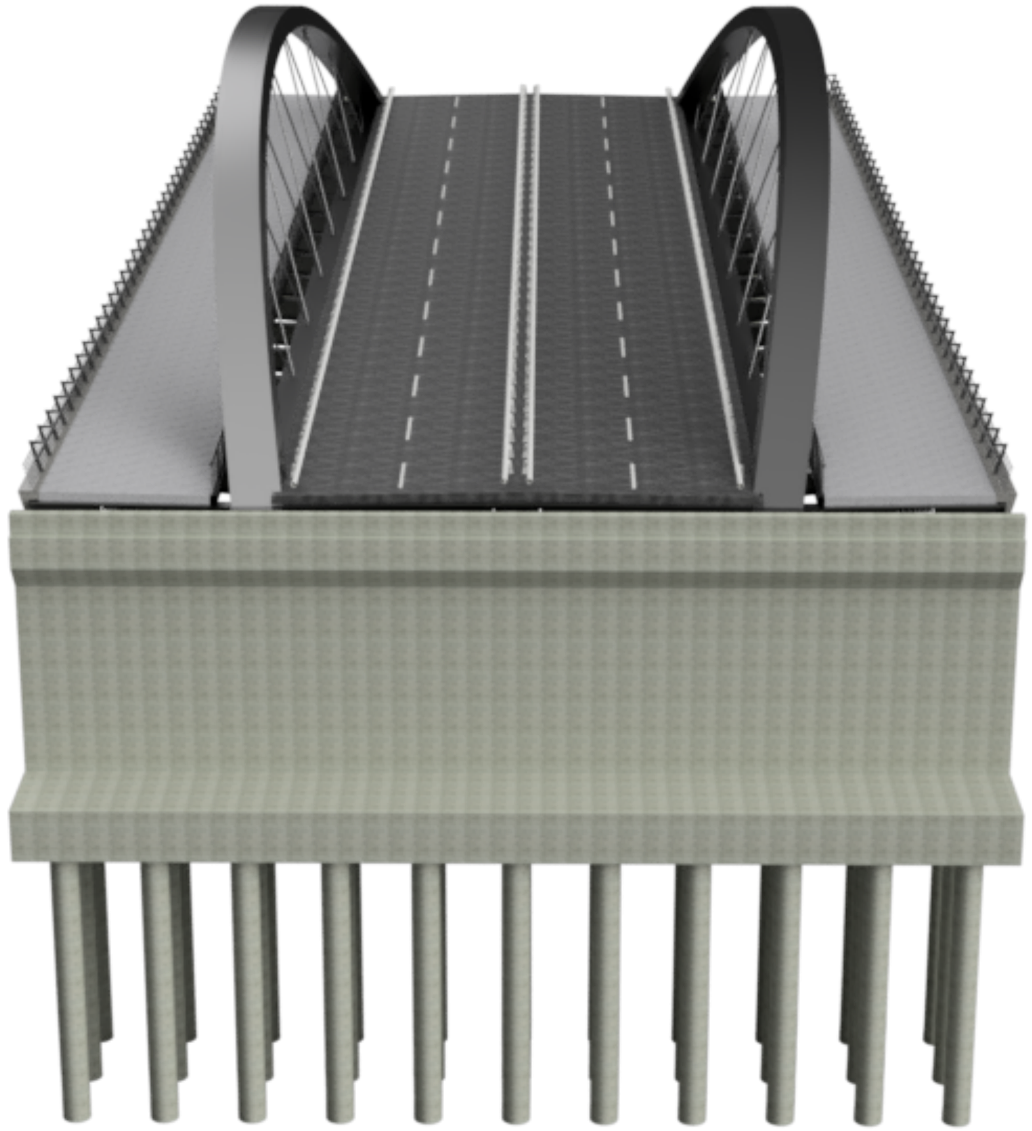
Appendix 7

Work Schedule



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DATE: JULY 2024

ACADEMIC YEAR 2024

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Work Schedule

1. Introduction

The purpose of this appendix is to plan the various activities necessary to conduct the construction of our bridge over the Deusto channel in Bilbao. First, the construction process is described and broken down into separate phases to get a better understanding of the work to be done and then to be able to schedule the different tasks.

2. Construction Process

The following sections provide a detailed description of the construction process, beginning with the continuous activities that form the foundation of effective project execution.

Continuous Activities:

Throughout the entire construction process, continuous activities play a vital role in ensuring the project's success. Health and safety management is addressed through regular inspections and the implementation of protective measures. Simultaneously, the project timeline, budget, and quality standards are monitored and adjusted as needed using project management tools.

Phase 1: Stakeout and Excavation of Work Area

The project begins with the preparation and stakeout of the designated work area, measuring approximately 99,50m x 50m. This phase also includes the excavation with a depth of approximately 4,00m of the work area and the creation of access ramps. In the event of water infiltration, the necessary measures are taken to pump out the water to ensure a dry and safe excavation site. Any excess material from the excavation is transported away.

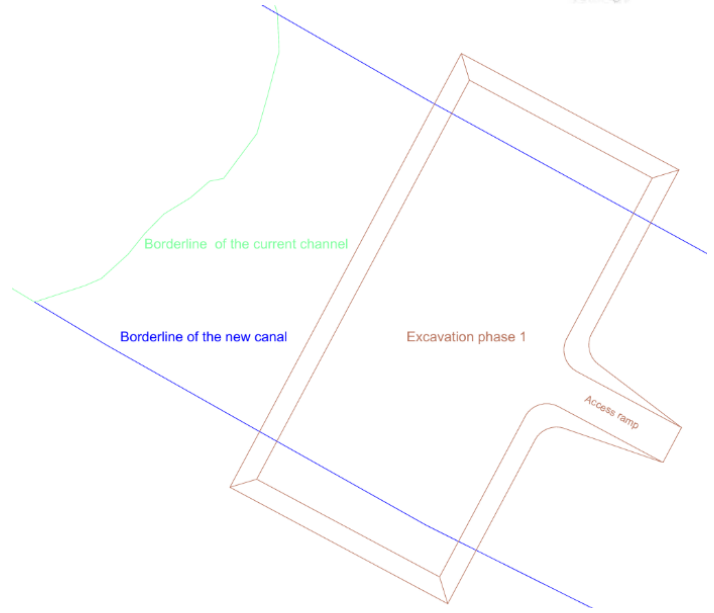


Figure 1: Work area, access ramps and borderlines of current and new channel [Own elaboration]

Phase 2: Sheet Pile Retaining Walls and Excavation

Next, around the designated area of both foundations sheet pile retaining walls are driven into the ground to avoid water infiltration, as the foundations are build below the average water level of the channel. The sheet piles form enclosures measuring approximately 40m x 10m x 11m. Further excavation is then carried out within these enclosures to prepare the site for subsequent construction activities.

Phase 3: Foundations and Abutments (both sides simultaneously)

The third phase begins with the execution of bored piles for the abutments. Then a 10cm concrete leveling and cleaning layer is poured to create a smooth foundation surface. The construction of pile caps and abutments follows, and once completed, the abutment walls are waterpoofed and the sheet pile enclosures are backfilled and compacted. This allows for the removal of the sheet piles. Finally, the bearing pads and transition slabs are placed.

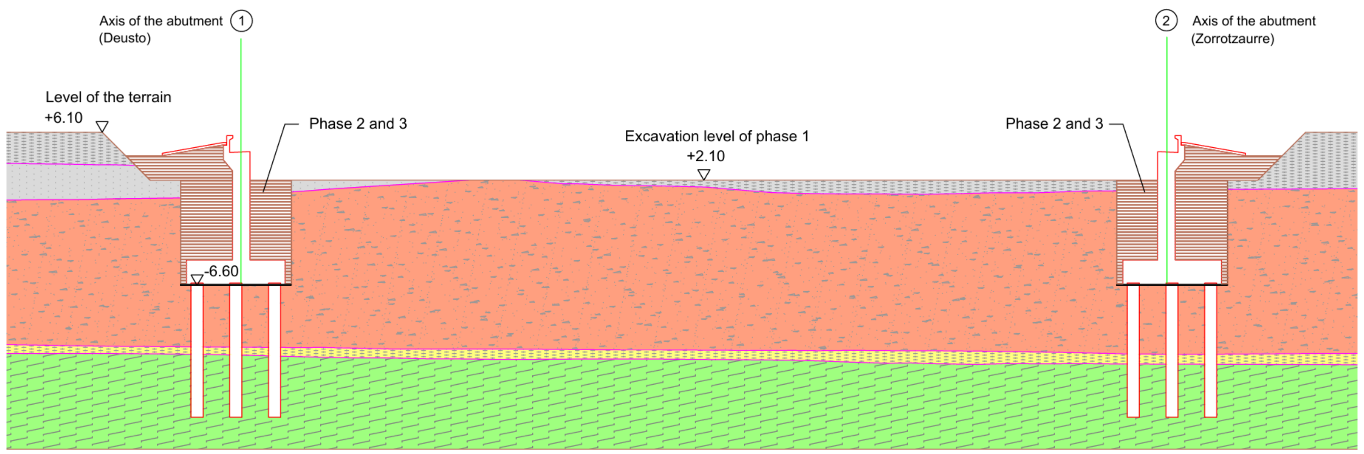


Figure 2: Finished construction of foundations and abutments [Own elaboration]

Phase 4: Manufacturing and Transport of the Steel Structure

The Steel structure components are manufactured in a workshop during the execution of the previous phases. All steel elements are divided into manageable lengths to fit on conventional trucks, thus drastically simplifying the transportation:

- The longitudinal beams with a total length of 78m are both divided into 6 parts, resulting in 12 units with a length of 13m per unit. As can be seen in the following drawing, small cantilevers of the transversal beams are already attached to the individual elements of the longitudinal beams in the workshop so that the complicated connections do not have to be made on site.

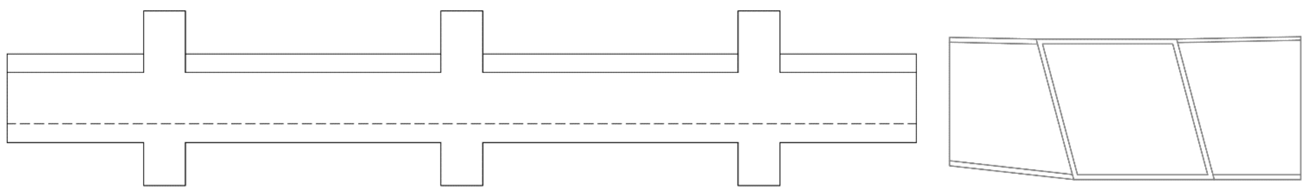


Figure 3: Plan view and cross section of one longitudinal element (approximately 13m) [Own elaboration]

- The bridge is composed of nineteen transverse elements. The thirty-eight cantilevers (19 per side) are each manufactured and transported as one unit, while the nineteen floor beams are each divided into two units. This makes a total of seventy-six units for the transverse elements.
- The arches, with a total length of approximately 81m, are each divided into six parts, resulting in twelve units with a length of 13.5m per unit.

Once manufactured, these components are transported to the construction site and stored in a dedicated area.

Phase 5: Temporary Support Structure

The execution of foundations for temporary supports marks the beginning of the fifth phase. Onto these foundations the support structures are placed to hold the structure during the assembly phase. Once the permanent structure is stable, these temporary supports are removed.

Phase 6: Assembly of the Metal Structure

The assembly phase begins with the placement and welding of the 12 longitudinal beam units on top of the temporary support structures, followed by the 76 transverse beam units, including the floor beams and cantilevers.

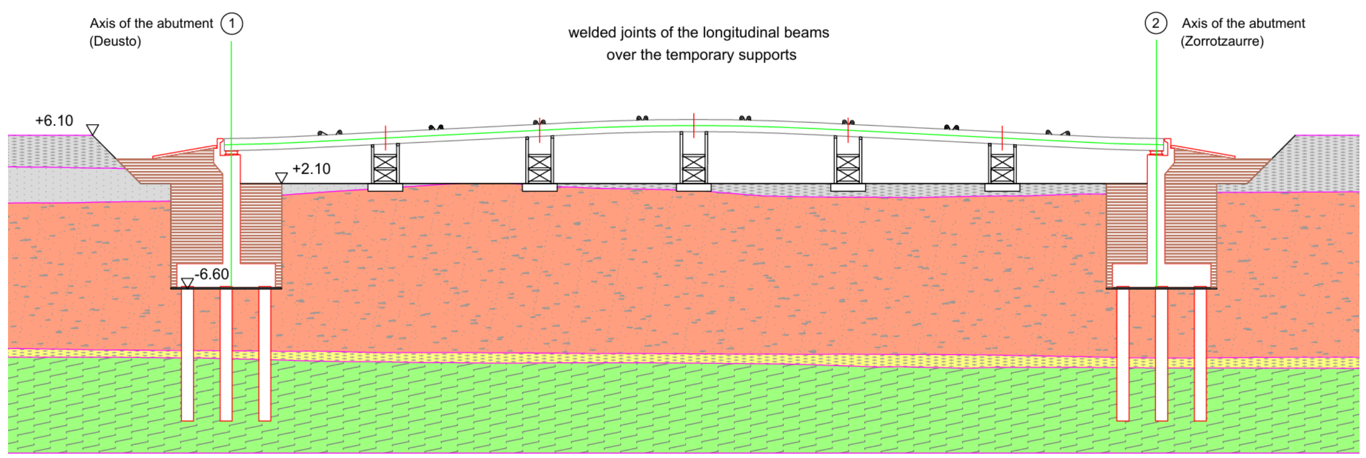


Figure 4: Assembly of longitudinal and transversal elements using temporary supports [Own elaboration]

The arches are pre-assembled and welded on-site before being lifted of the ground and connected at both ends to the longitudinal beams. Finally, sixteen tension rods per side are installed, thus providing the final structural integrity.

Phase 7: Execution of the Concrete Slab and Related Tasks

The formwork for the concrete slab is placed, incorporating the corrugated metal sheets, cornices, and metal edge profiles for the sidewalk and roadway. The exterior handrails must be placed in advance of the cornices, as they are welded to the outer edge profiles between each cornice element. The concrete slab is then reinforced and poured, creating a robust surface for the bridge on which the safety barriers and Streetlights are mounted in the next phase.

Phase 8: Bridge Fittings, Paving and Signage

The final phase involves the installation of the rest of the bridge fittings, such as safety barriers, lighting, expansion joints, and pipelines. The roadway and sidewalk are paved, and traffic signs and road markings are applied.

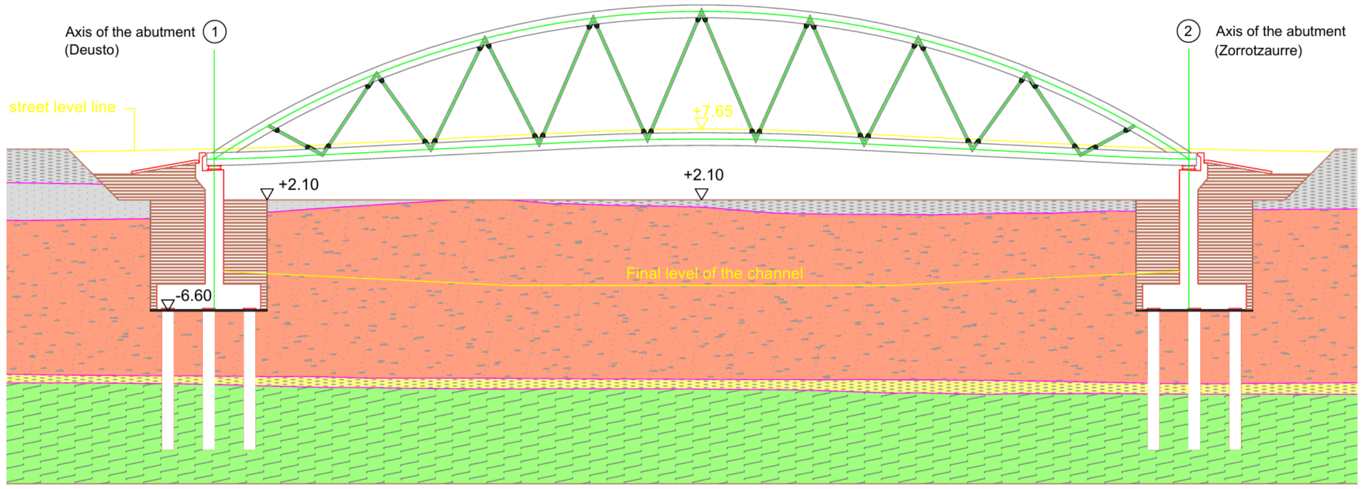


Figure 5: Bridge ready for connection to municipal infrastructure [Own elaboration]

After finishing works, final cleaning and the load testing, the project is ready for connection to the municipal infrastructure and the opening of the Deusto channel.

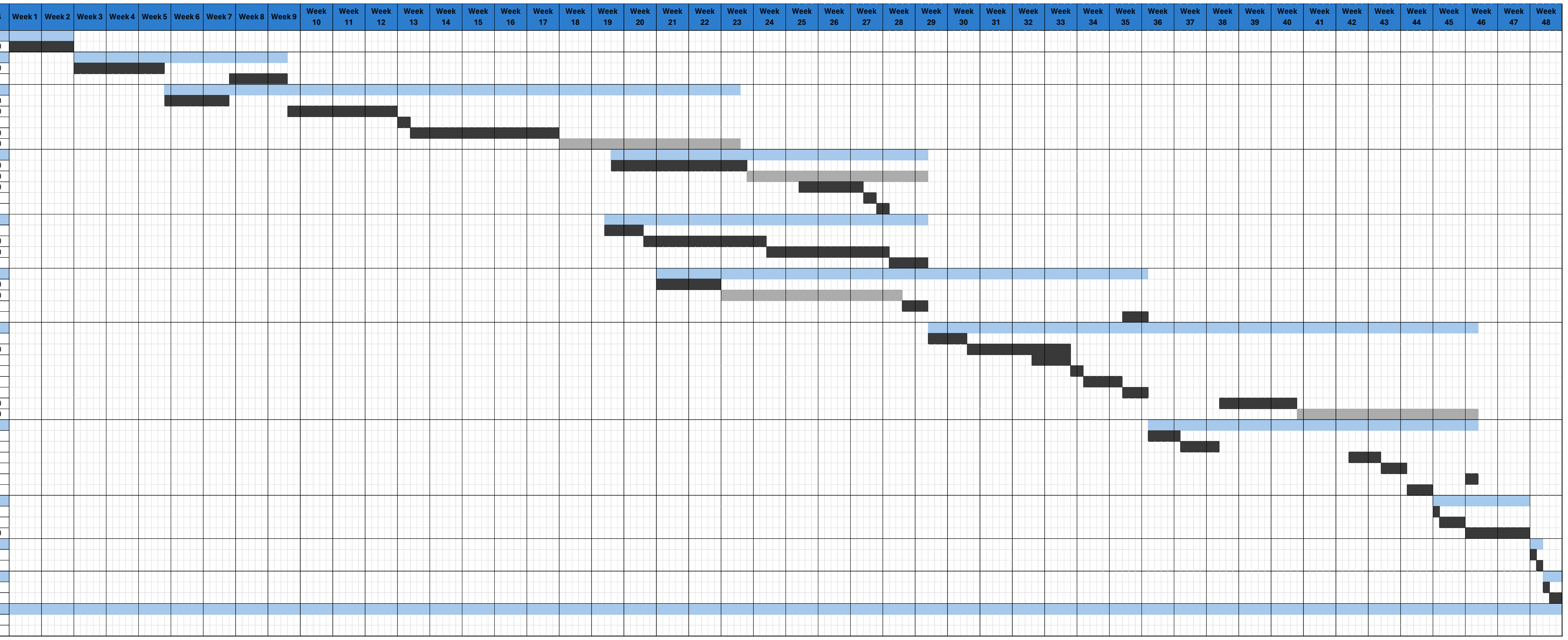
3. Work Schedule

In the following Work Schedule, the main activities have been taken into account and their sequence and duration have been determined to evaluate the entire execution time of the project. The presentation of these activities is done by means of a Gantt Chart, which represents the planned work schedule according to the construction process defined above. It is important to note that the duration estimates are approximate and based on data from similar projects. In a real construction project, a more detailed and specific analysis would be required for accurate planning.

3.1 Gantt-Chart:

As demonstrated in detail in the accompanying Gantt chart, the total estimated time required for project completion is 48 weeks.

ACTIVITY	REPRESENTATIVE QUANTITIES	units	UNIT PERFORMANCE	units	DAYS
PREPARATORY WORK					
Preperation and Stakeout of work area	4975,00	m^2	500	m^2/day	10,00
EARTHWORKS					
Excavation of the work area and access ramps and transportation of excess material	19900,00	m^3	1500	m^3/day	14,00
Excavation of sheet pile enclosure	6960,00	m^3	800	m^3/day	9,00
FOUNDATIONS ON BOTH SIDES					
Driving of temporary sheet pile retaining wall	200,00	m	20	m/day	10,00
Construction of bored piles	66,00	unit	4	unit/day	17,00
Leveling and cleaning layer (10cm)	100,00	%	50	%/day	2,00
Formwork, reinforcement and concreting of pile cap	1141,00	m^3	50	m^3/day	23,00
Concrete Curing	100,00	%	3,6	%/day	28,00
ABUTMENTS ON BOTH SIDES					
Formwork, reinforcement and concreting of the walls	1015,00	m^3	50	m^3/day	21,00
Concrete Curing	100,00	%	3,6	%/day	28,00
Waterproofing, backfilling and sheet pile removal	100,00	%	10	%/day	10,00
Placement of bearing pads	100,00	%	50	%/day	2,00
Transition slab	100,00	%	50	%/day	2,00
MANUFACTURE OF METAL STRUCTURE IN THE WORKSHOP					
Manufacture of Longitudinal Beams	12,00	unit	2	unit/day	6,00
Manufacture of Transverse Floor Beams	38,00	unit	2	unit/day	19,00
Manufacture of Transverse Cantilevers	38,00	unit	2	unit/day	19,00
Manufacture of both Arches	12,00	unit	2	unit/day	6,00
TEMPORARY SUPPORT STRUCTURES					
Execution of foundations	100,00	%	10	%/day	10,00
Concrete Curing	100,00	%	3,6	%/day	28,00
Placement of temporary supports	100,00	%	33,33	%/day	4,00
Removal of temporary supports	100,00	%	33,33	%/day	4,00
EXECUTION OF THE LOAD-BEARING STRUCTURE					
Placement and welding of longitudinal beams	12,00	unit	2	unit/day	6,00
Placement and welding of transverse beams (floor beams and cantileverss)	76,00	unit	5	unit/day	16,00
Pre-assembly and welding of the arches on-site prior to placement	12,00	unit	2	unit/day	6,00
Placement and welding of the arches (connection to the longitudinal beams)	100,00	%	50	%/day	2,00
Placement of the tension rods	32,00	unit	6	unit/day	6,00
Placement of corrugated metal sheets	2239,16	m^2	600	m^2/day	4,00
Reinforcement and concreting of the concrete slab	559,79	m^3	50	m^3/day	12,00
Concrete Curing	100,00	%	3,6	%/day	28,00
FITTINGS					
Placement of edge profiles and handrails	100,00	%	20	%/day	5,00
Placement of cornice (aprox. 38 units per side)	78,00	unit	15	unit/day	6,00
Placement of safety barriers	100,00	%	20	%/day	5,00
Placement of Steetlights	100,00	%	25	%/day	4,00
Placement of expansion joints	100,00	%	50	%/day	2,00
Pipelines	100,00	%	33,33	%/day	4,00
PAVING					
Waterproofing / Sealing layer	100,00	%	100	%/day	1,00
Execution of pavement on the roadway	1321,16	m^2	400	m^2/day	4,00
Execution of pavement on sidewalks	918,00	m^2	100	m^2/day	10,00
SIGNAGE					
Road markings	76,50	m	3000	m/day	1,00
Traffic Signs	76,50	m	300	m/day	1,00
OTHERS					
Load Test	100,00	%	100	%/day	1,00
Final cleaning and closure of the work site	100,00	%	50	%/day	2,00
CONTINUOUS ACTIVITIES					
Health and Safety					240
Control of time, cost and quality (Project Management)					240

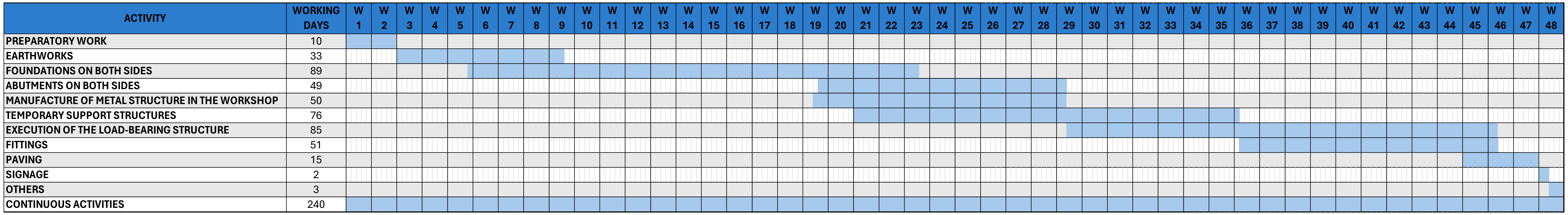


The following factors were taken into account when calculating the duration of each activity:

- Average performance per unit for each activity
- Quantities calculated from our own elaborated drawings
- Dependency between the different activities
- A week is considered to consist of 5 working days, and each working day has a duration of 8 hours

3.2 Gantt-Chart Summary:

The following diagram provides a brief overview of the main activities:



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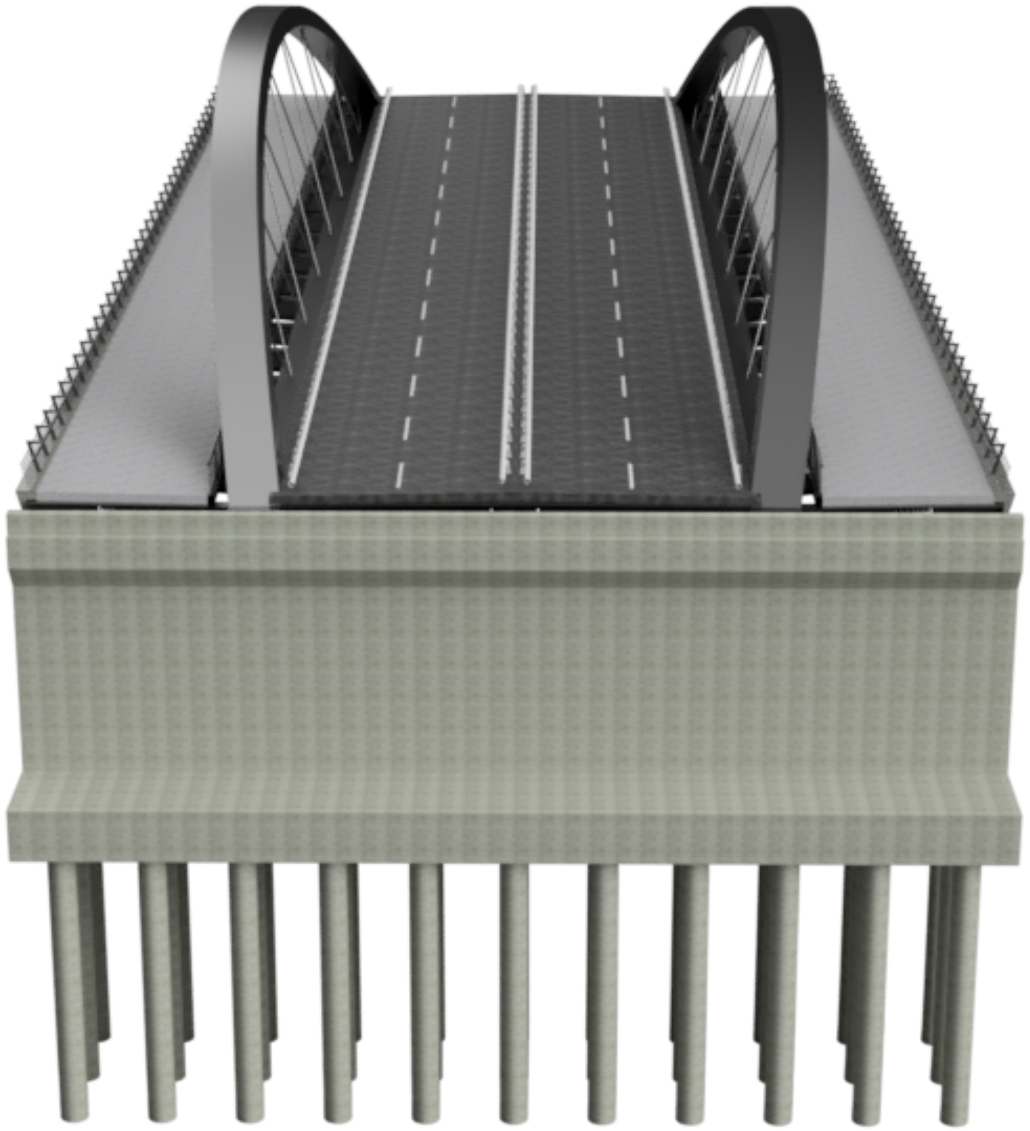
Appendix 8

Sustainable Development Goals



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DATE: JULY 2024

ACADEMIC YEAR 2024

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Sustainable Development Objectives

1. Introduction

This appendix deals with the sustainable design of the bridge. The objectives of the Sustainable Development Goals (SDGs) of the United Nations (UN) are considered and linked to the developed project. The specific points related to the construction of the bridge are examined in more detail.

2. Explanation SDG's

The Sustainable Development Goals (SDGs) are a 2012 United Nations resolution to formulate and adopt goals that strengthen economic growth, reduce inequalities and injustices in living standards, create equal opportunities, ensure the sustainable use and management of natural resources, and preserve ecosystems.

The following 17 major goals have been defined in 2014:

Figure 1: 17 Goals of the Sustainable Development Goals [source: United Nations]

3. Connection of the Project and the SDG's

The construction of the bridge contributes to the achievement of several objectives. Since the construction of the bridge is directly related to Zaha Hadid's master plan, the project is also considered part of the master plan in the following categorization. The objectives to which the bridge contributes are described in more detail below:

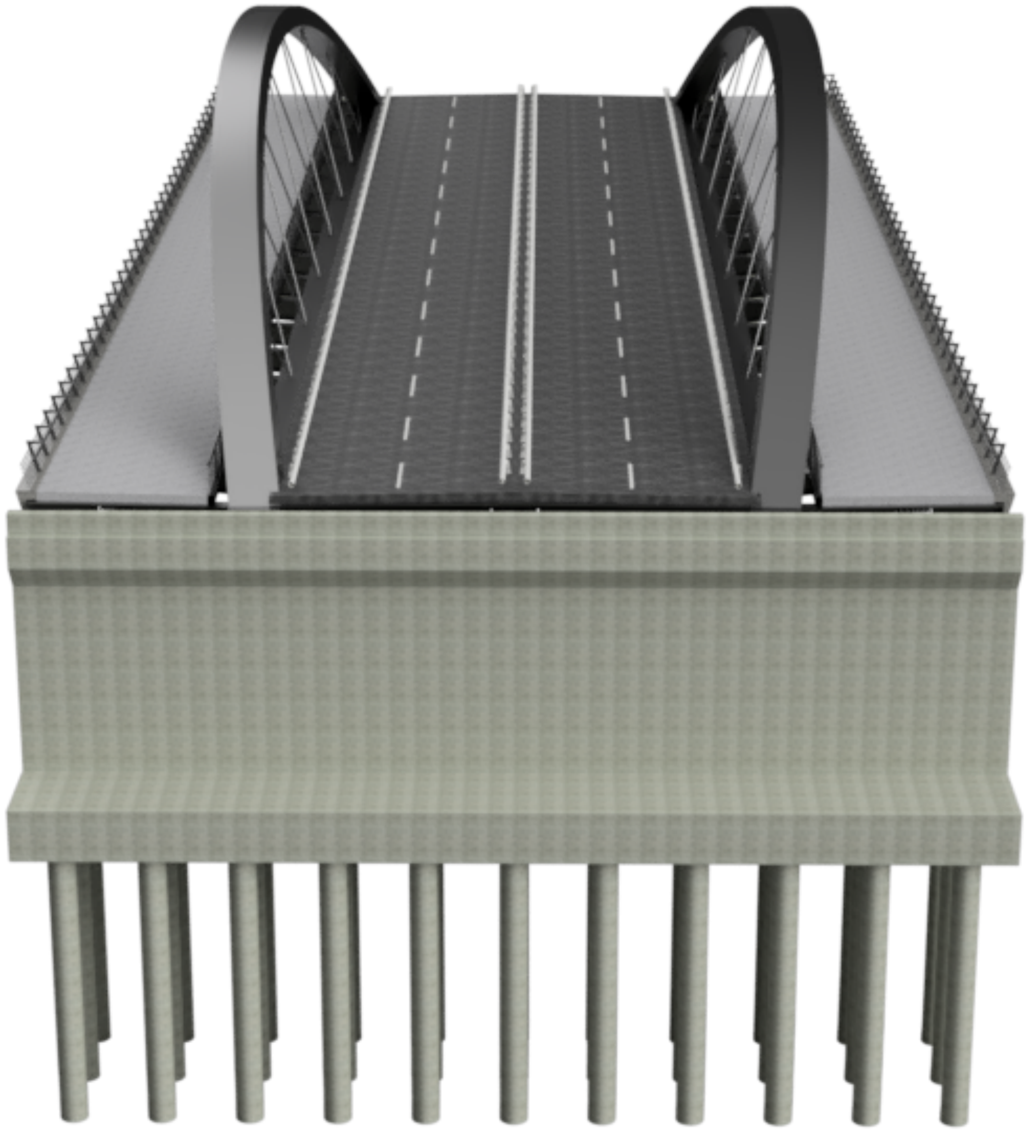
- **3. Good Health and Well-being:** As the large pedestrian walkways required by the tender, which can also be used by cyclists, fit in with the basic urban idea of easy walkability, this has the potential to encourage walking and cycling rather than driving, which is a fundamentally healthier way of life.
- **8. Decent Work and Economic Growth:** As an access point to Zorrotzaurre Island, the bridge plays an important role in the economic growth capacity of the revitalized district.
- **9. Industry, Innovation and Infrastructure:** As mentioned in the previous point, the bridge acts as an access point to Zorrotzaurre Island, so it plays an important role in the industrial success of the island and represents a central factor for infrastructure, being the access point at the southern end of the island.
- **10. Reduced Inequalities:** As Zaha Hadid's master plan foresees the rehabilitation of the island, which has lost much of its relevance as a neighborhood after the end of port activities, the revitalization of the island (including the provision of social housing) plays a role in creating equal opportunities for the people who live there. The bridge also contributes to this, as it serves as an access point to the island, as mentioned in the previous points.
- **11. Sustainable Cities and Communities:** By connecting the island to the rest of the city through the bridges built and the planned public transportation link, the district becomes more inclusive in the overall cityscape.
- **13. Climate Action:** The construction of the bridge is accompanied by the full opening of the Deusto channel. Opened as a flood control measure, it will help protect the surrounding neighborhoods, as well as the parks and natural areas on the outskirts of the city.

Bibliography

- UN Sustainable Development Goals ;accessed 14.06.2024 : https://www.un.org/sustainabledevelopment/

Document No. 2

Drawings



Authors:

Kevin Petri

Linus Jonas

Tutors:

Salvador Monleón Cremades

Carlos Manuel Lázaro Fernández

DATE: JULY 2024

ACADEMIC YEAR 2024

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Drawings

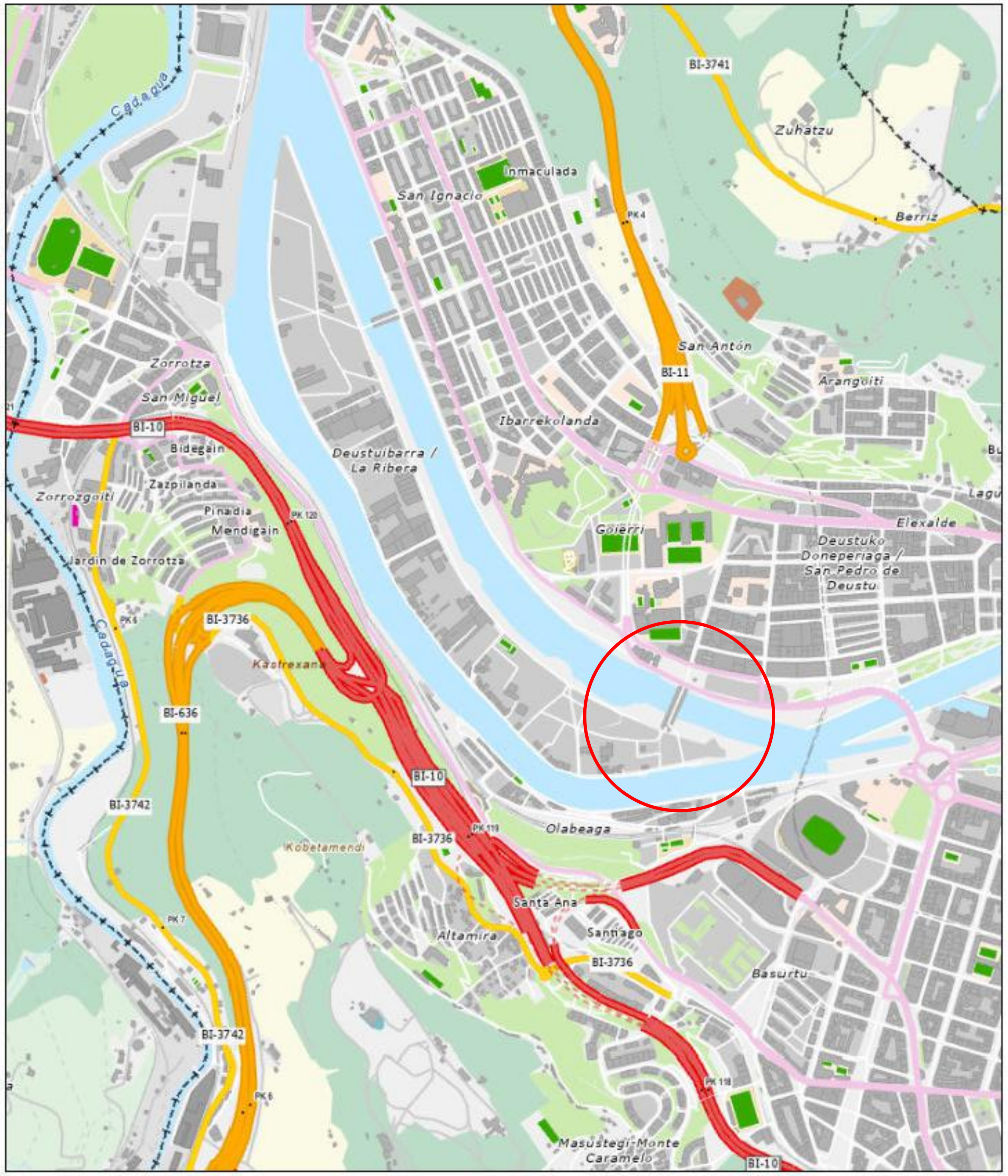
Bilbao

1:200,000

0 1.25 2.5 5 mi

0 2 4 8 km

Zorrotzaurre



02/07/2024

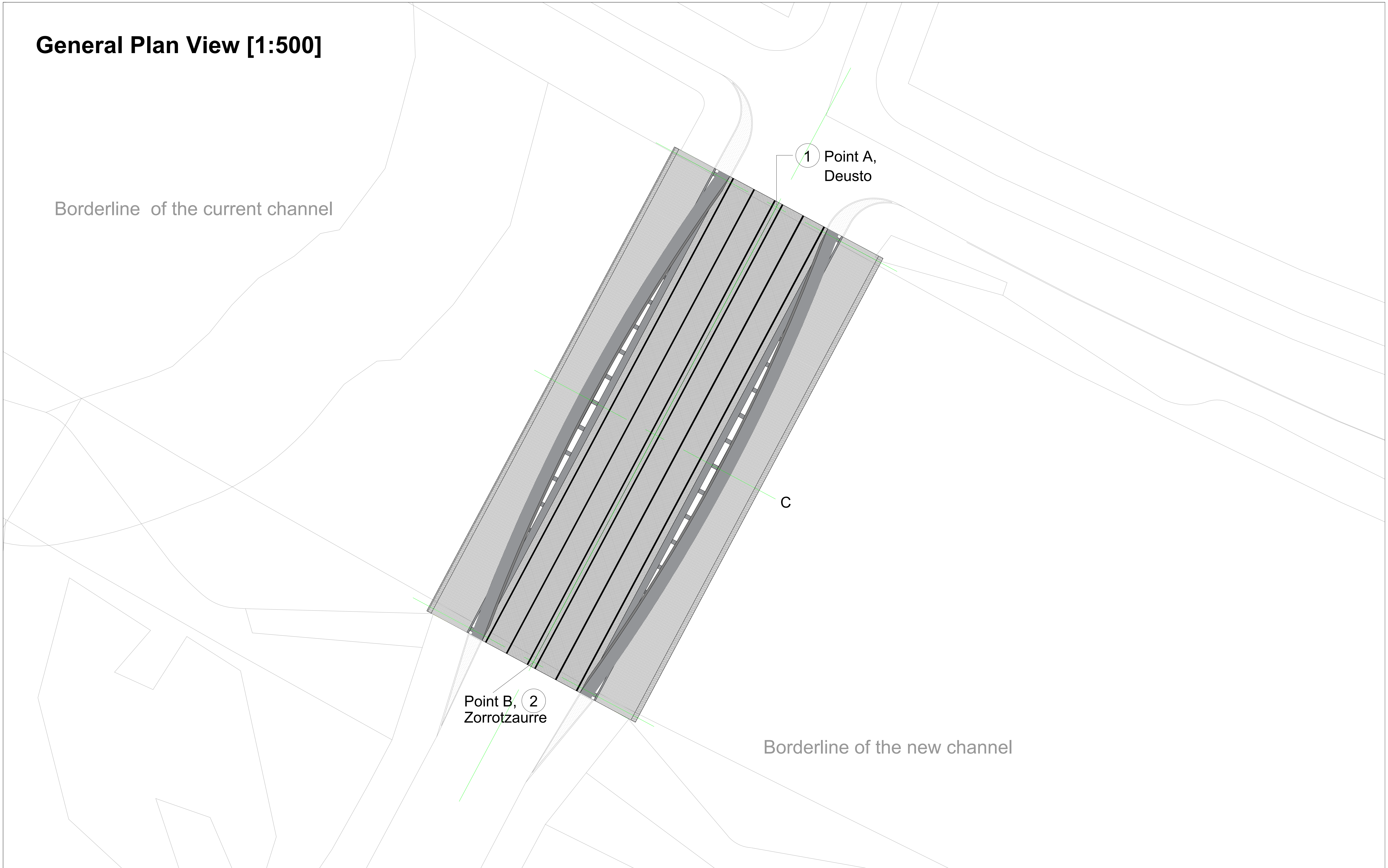
1:20,000

0 0.13 0.25 0.5 mi

0 0.2 0.4 0.8 km

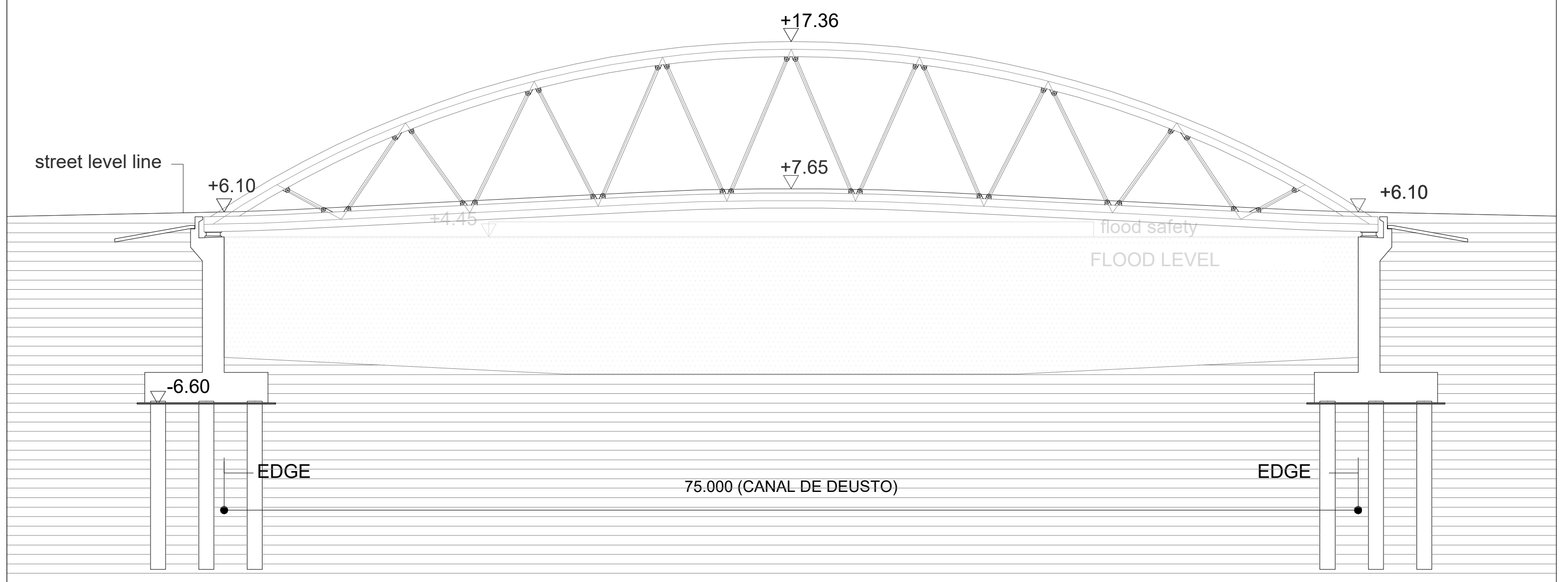
UNIVERSITAT POLITÈCNICA DE VALÈNCIA	Project Title	Drawing Title	Drawn by	Scales	Number
	Bachelor's thesis Basic Project of the first bridge of Zorrotzaurre Island (Bilbao)	Location and Site	Linus Jonas Kevin Petri	1:20000 1:200000	1 **Date** June 2024

General Plan View [1:500]

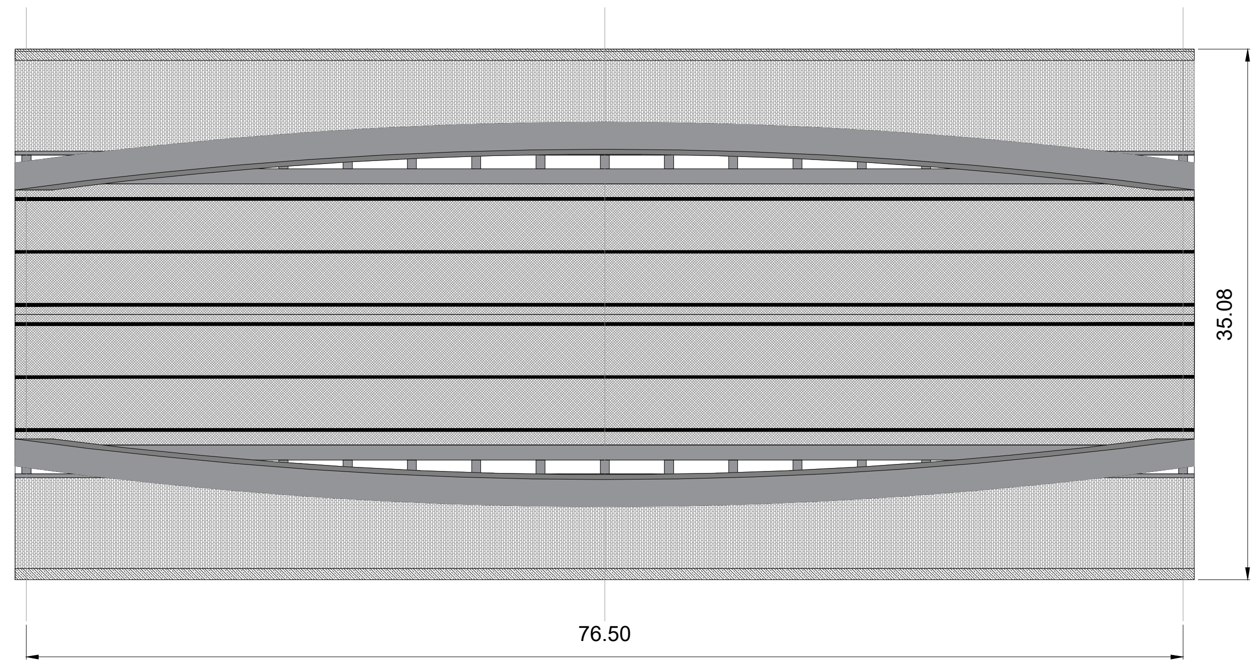


UNIVERSITAT POLITÈCNICA DE VALÈNCIA	**Project Title** Bachelor's thesis Basic Project of the first bridge of Zorrotzaurre Island (Bilbao)	**Drawing Title** General Plan View	**Drawn by** Linus Jonas Kevin Petri	**Scale** 1:500	**Number** 2 **Date** June 2024

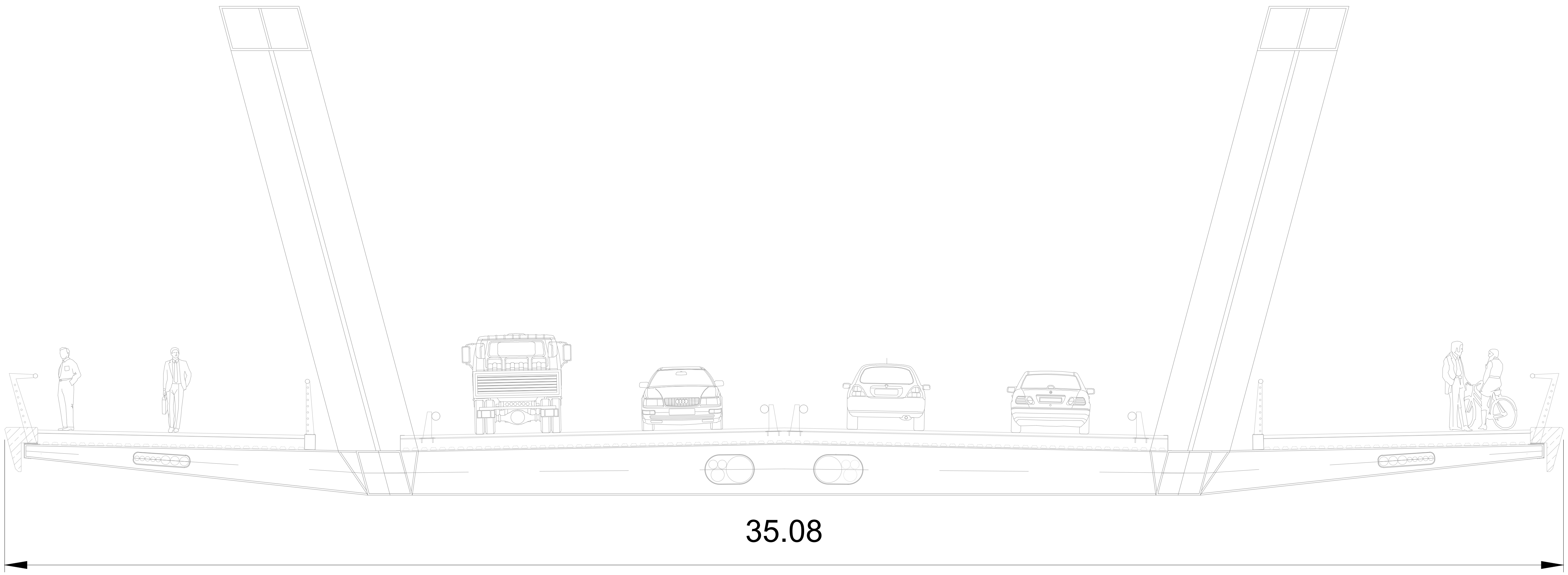
Side View [1:500]



Plan View [1:500]

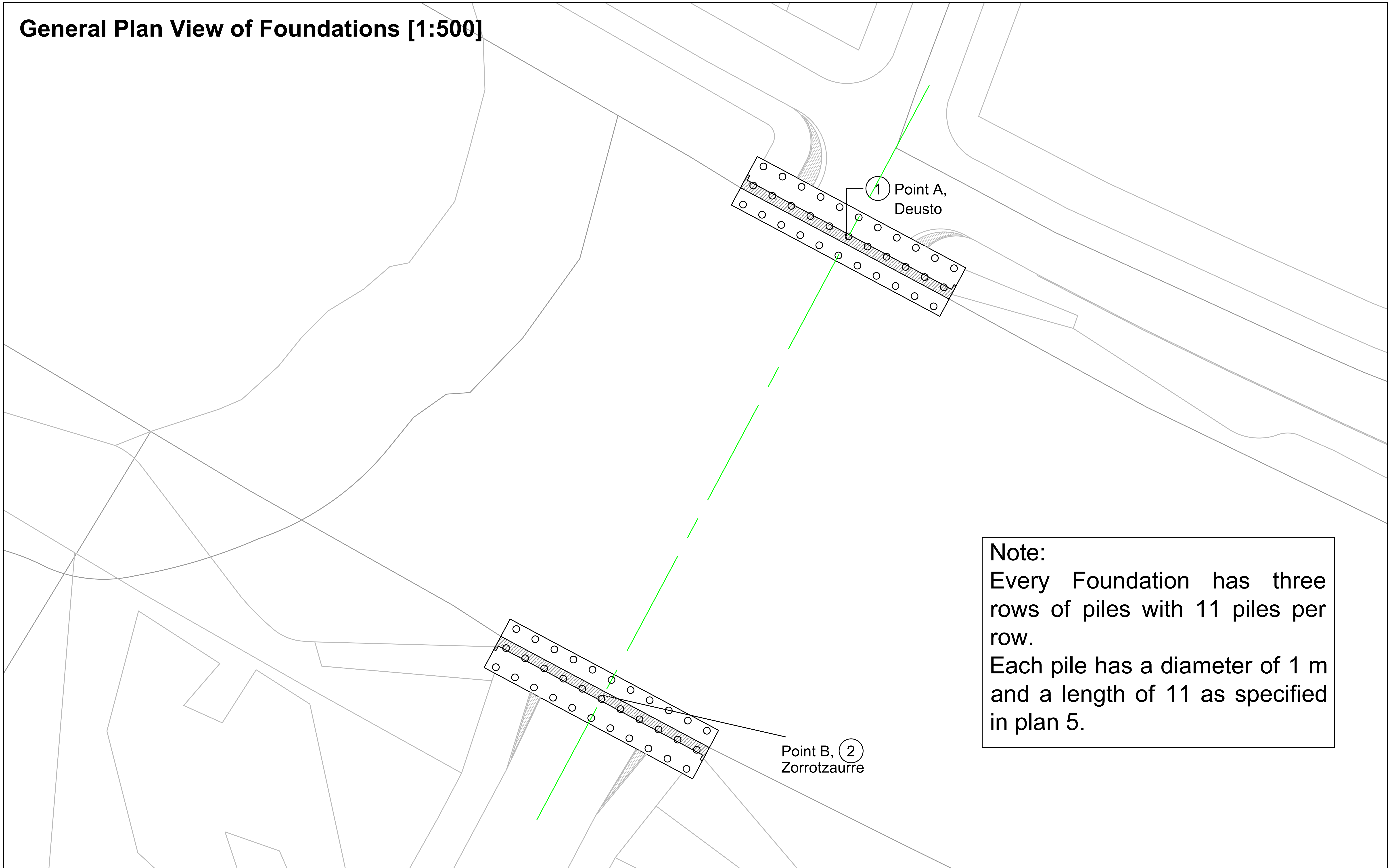


Cross-Section [1:100]



	Project Title	Drawing Title	Drawn by	Scales	Number
UNIVERSITAT POLITÈCNICA DE VALÈNCIA	Bachelor's thesis Basic Project of the first bridge of Zorrotzaurre Island (Bilbao)	Plan View, Side View and Cross-Section	Linus Jonas Kevin Petri	1:500 1:100	3 **Date** June 2024

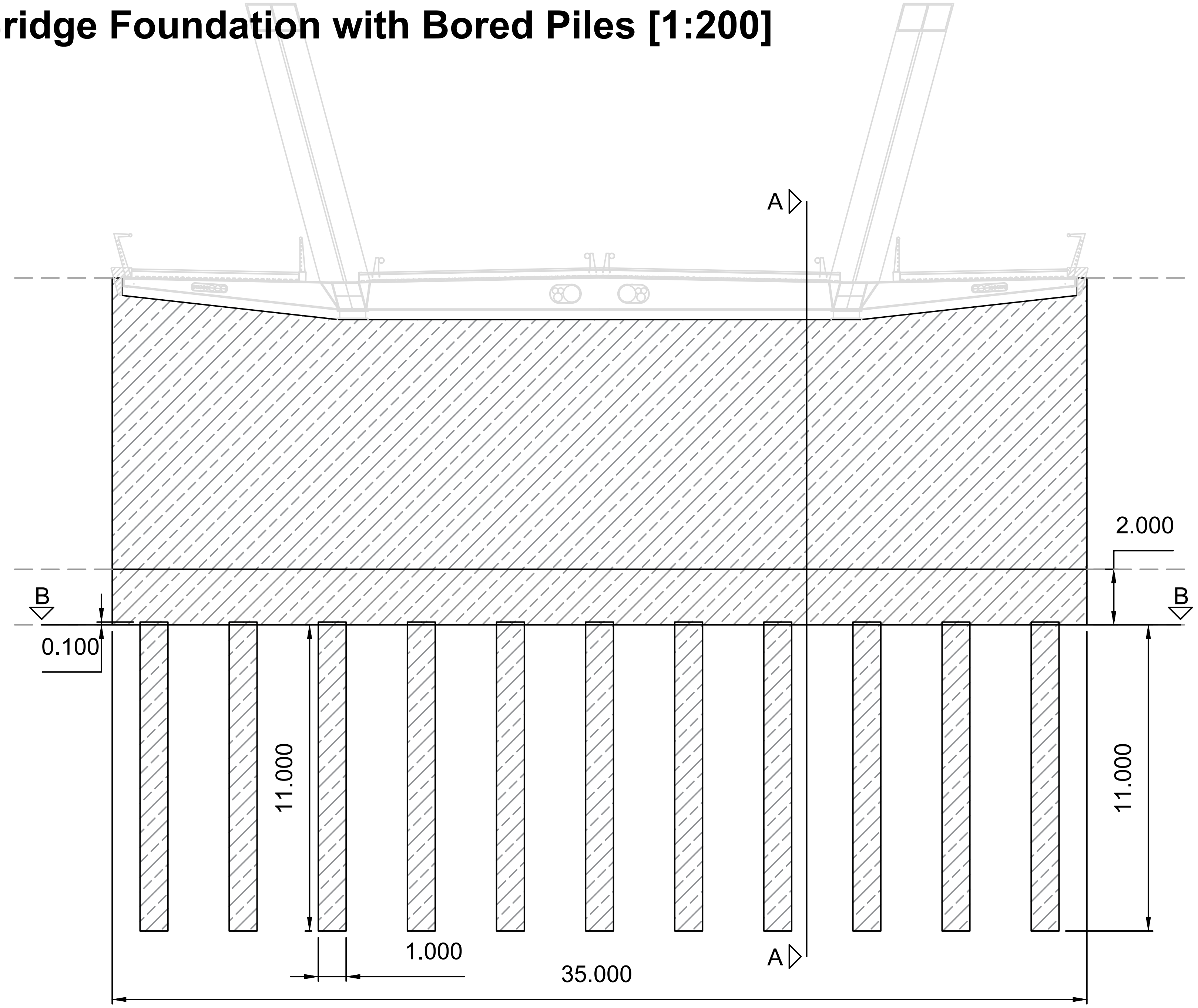
General Plan View of Foundations [1:500]



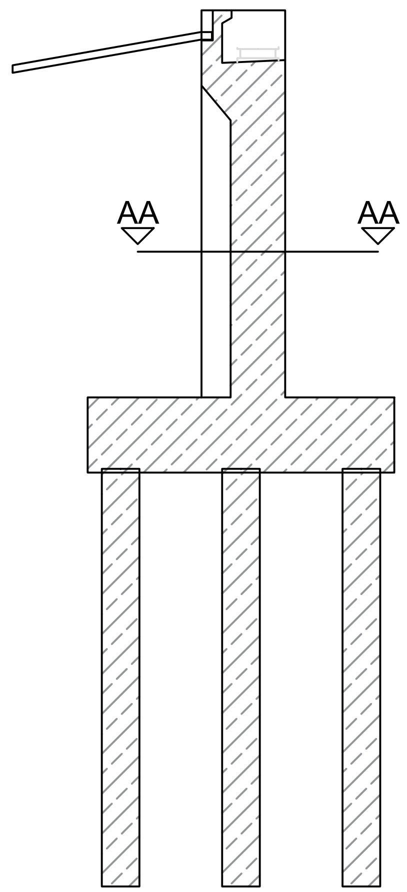
Note:
Every Foundation has three rows of piles with 11 piles per row.
Each pile has a diameter of 1 m and a length of 11 as specified in plan 5.

UNIVERSITAT POLITÈCNICA DE VALÈNCIA		Project Title	Drawing Title	Drawn by	Scales	Number
		Bachelor's thesis Basic Project of the first bridge of Zorrotzaurre Island (Bilbao)	Foundation General Plan View	Linus Jonas Kevin Petri	1:500	4 **Date** June 2024

Bridge Foundation with Bored Piles [1:200]



Section A-A Scale 1:200



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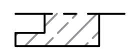
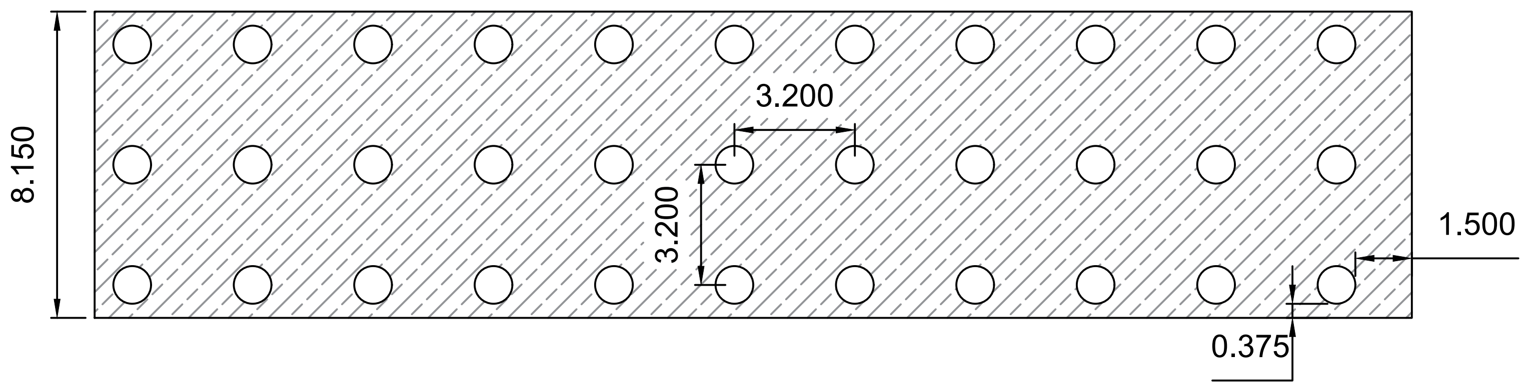
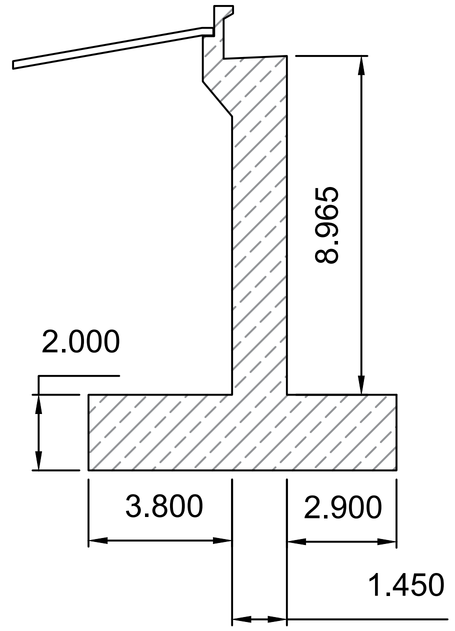


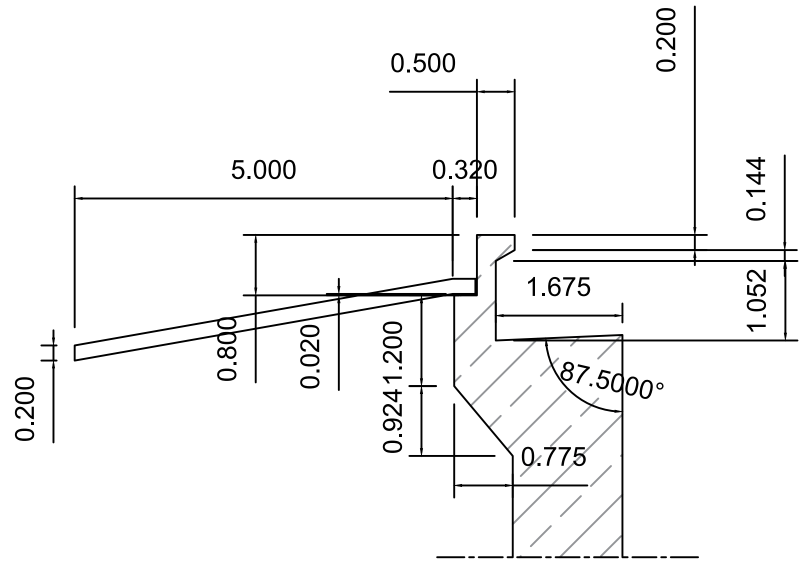
Section B-B Scale 1:200



Dimensions of Bridge Retaining Wall [1:200]



Dimensions top part Bridge Retaining Wall [1:200]



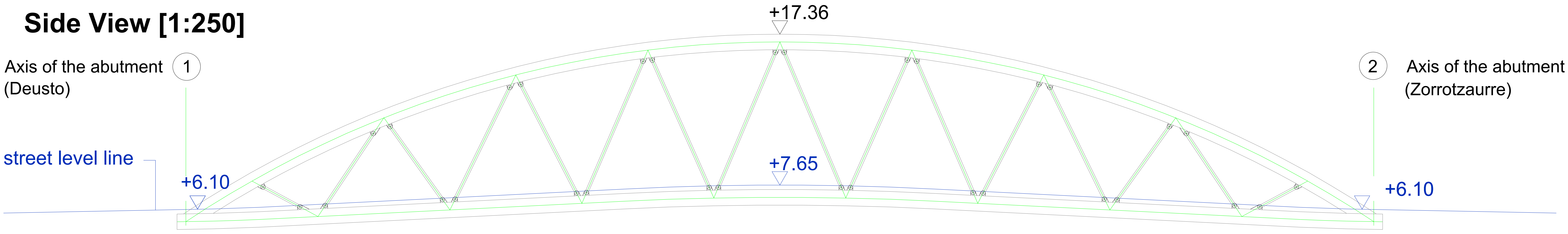
Characteristics of the materials

Element	Material type	Material class	Characteristic resistance
Bridge retaining wall	Concrete	C40/50	fck = 40 MPa
	with reinforcement steel	B500A	fyk = 500 MPa
Bored piles	Concrete	C40/50	fck = 40 MPa

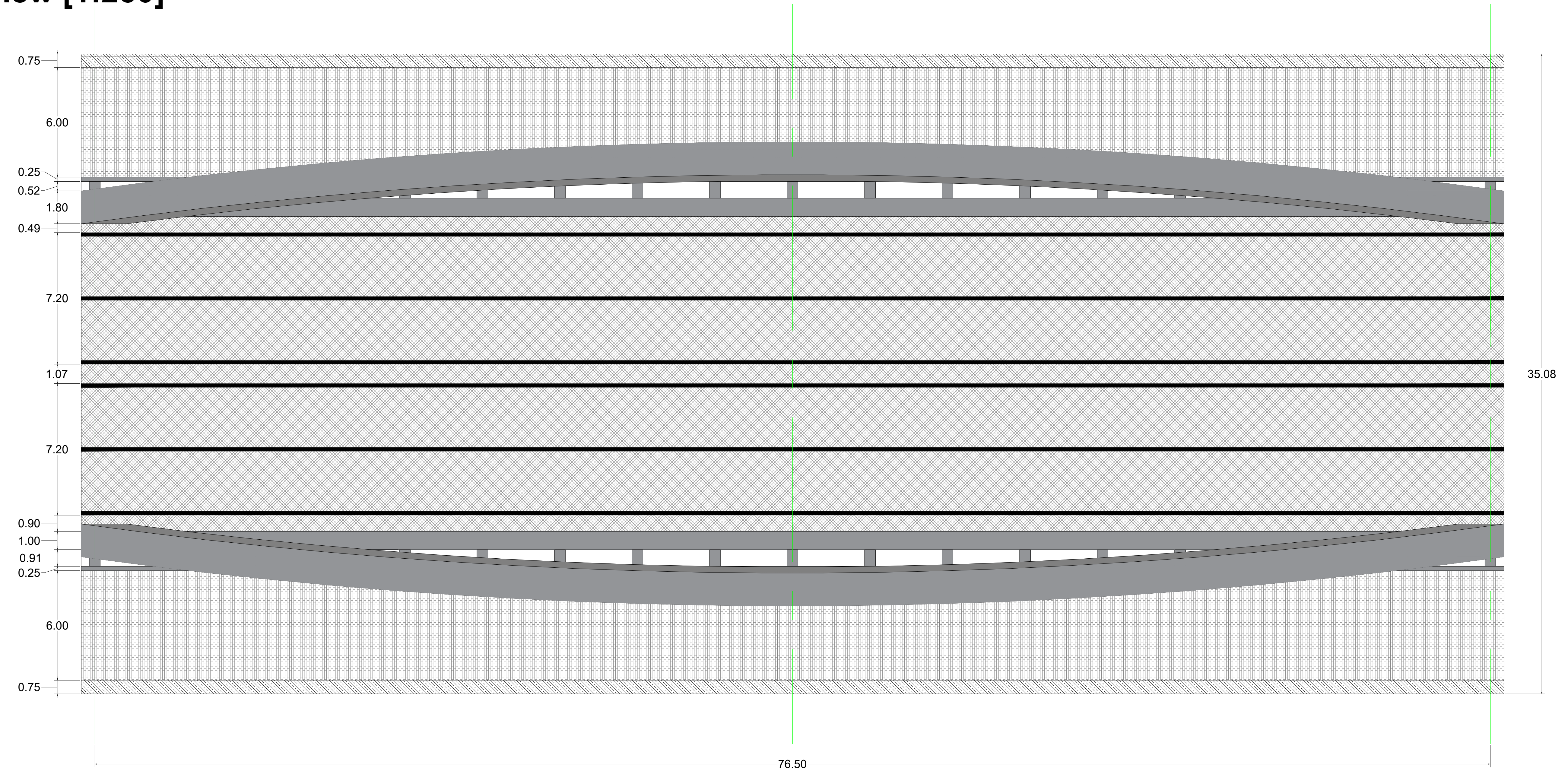
UNIVERSITAT POLITÈCNICA DE VALÈNCIA

Project Title	Drawing Title	Drawn by	Scales	Number
Bachelor's thesis Basic Project of the first bridge of Zorrotzaurre Island (Bilbao)	Bridge Foundation Geometrical Definition	Linus Jonas Kevin Petri	1:200	5 **Date** June 2024

Side View [1:250]



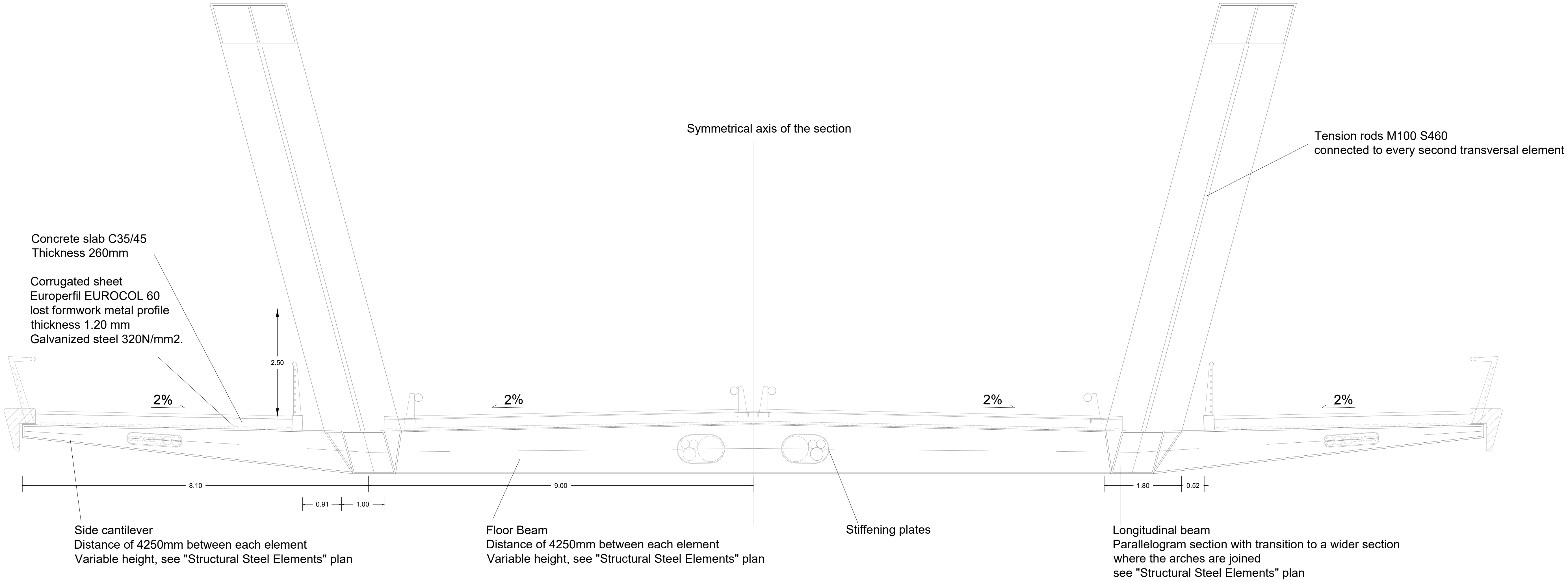
Plan View [1:250]



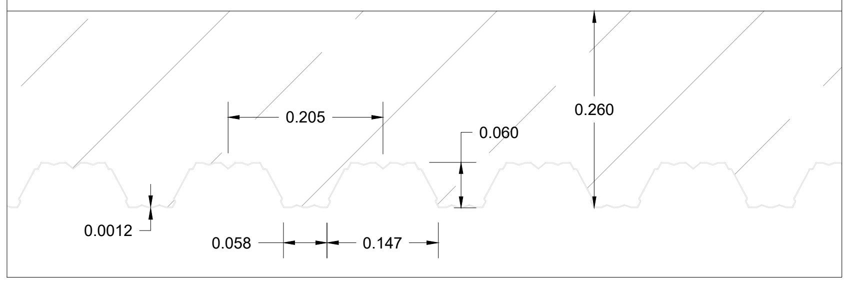
UNIVERSITAT POLITÈCNICA DE VALÈNCIA

Project Title	Drawing Title	Drawn by	Scale	Number
Bachelor's thesis Basic Project of the first bridge of Zorrotzaurre Island (Bilbao)	Structure: Side View and Plan View	Linus Jonas Kevin Petri	1:250	6
				Date June 2024

Structure Cross-Section [1:100]



Detail of Corrugated Sheet and Concrete Slab [1:10]



Characteristics of the materials

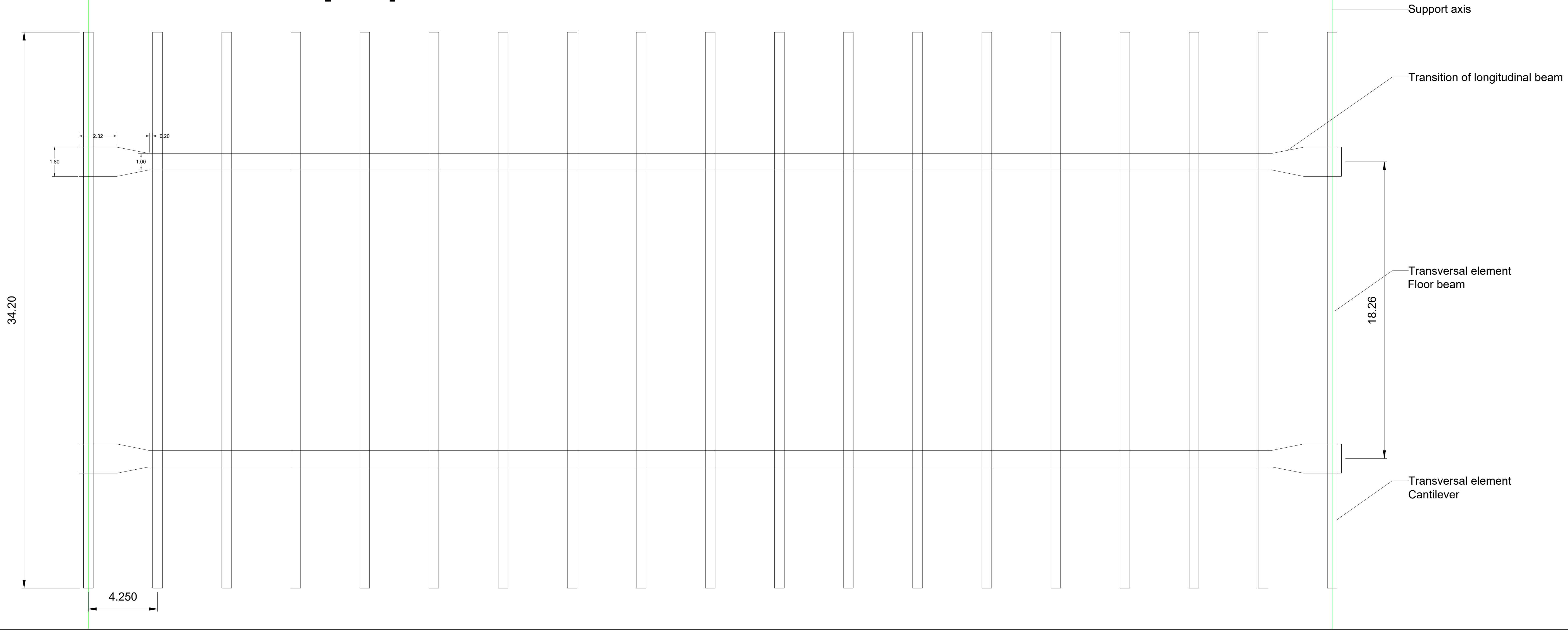
Element	Material type	Material class	Characteristic resistance
Arches	Structural Steel	S355	fyk = 355 MPa
Longitudinal Beams	Structural Steel	S355	fyk = 355 MPa
Transversal Beams	Structural Steel	S355	fyk = 355 MPa
Concrete Slab	Concrete	C35/45	fck = 35 MPa
Tension Rods	Structural Steel	S460	fyk = 460 MPa

UNIVERSITAT POLITÈCNICA DE VALÈNCIA

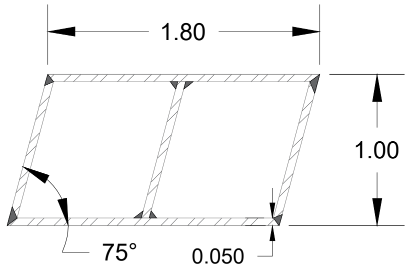
Project Title	Drawing Title	Drawn by	Scales	Number
Bachelor's thesis Basic Project of the first bridge of Zorrotzaurre Island (Bilbao)	Structure Cross-Section	Linus Jonas Kevin Petri	1:100 1:10	7 **Date** June 2024

L. Jonas
K. Petri

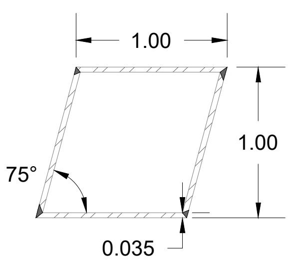
Structure: Steel Deck [1:250]



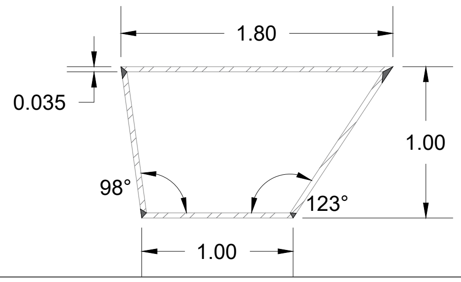
Cross-Section Right Arch [1:50]



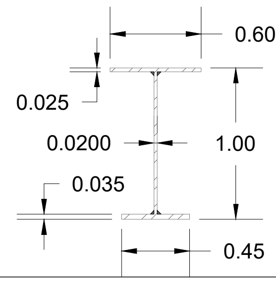
Mid-Cross-Section Right Longitudinal Beam



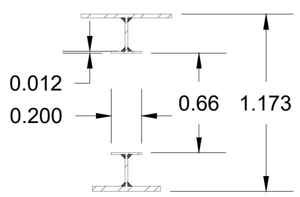
End-Cross-Section Right Longitudinal Beam



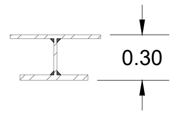
Start-Cross-Section Transversal Element Floor Beam and Cantilever



Cutout Area Cross-Section Transversal Element Floor Beam



End-Cross-Section Transversal Element Cantilever



Characteristics of the materials

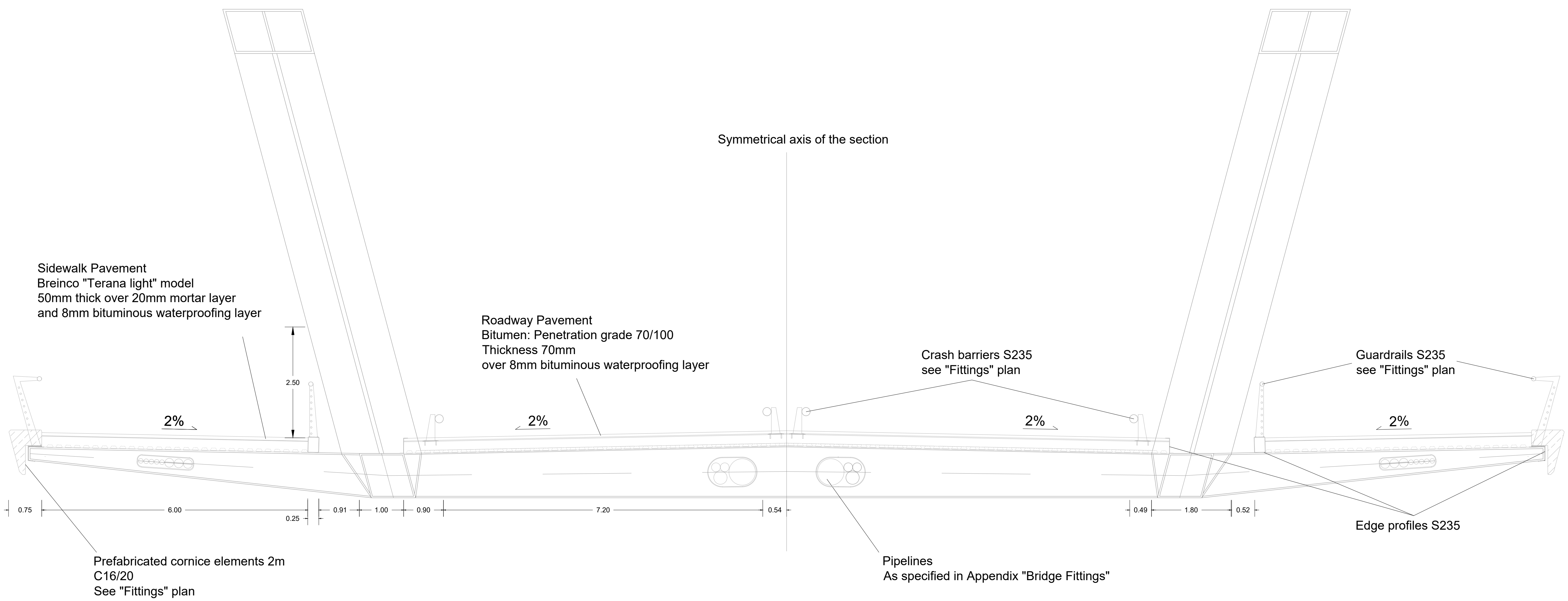
Element	Material type	Material class	Characteristic resistance
Arches	Structural Steel	S355	fyk = 355 MPa
Longitudinal Beams	Structural Steel	S355	fyk = 355 MPa
Transversal Beams	Structural Steel	S355	fyk = 355 MPa

UNIVERSITAT POLITÈCNICA DE VALÈNCIA

Project Title	Drawing Title	Drawn by	Scales	Number
Bachelor's thesis Basic Project of the first bridge of Zorrotzaurre Island (Bilbao)	Structural Steel Elements	Linus Jonas Kevin Petri	1:250 1:50	8
				Date
				June 2024

L. Jonas
K. Petri

Cross-Section [1:100]
Definition of Bridge Fittings



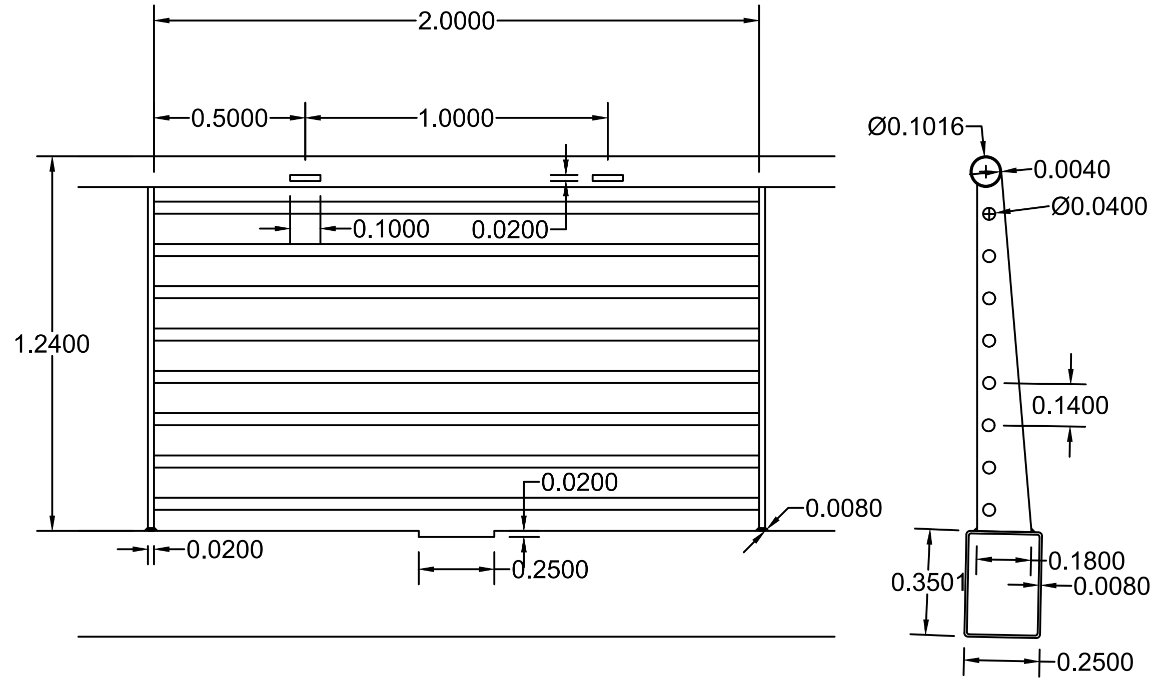
Characteristics of the materials

Element	Material type	Material class	Characteristic resistance
Cornice	Concrete	C16/20	fck = 16 MPa
	with reinforcement steel	B500A	fyk = 500 MPa
Outer Guardrails	Steel	S235	fyk = 235 MPa
Inner Guardrails	Steel	S235	fyk = 235 MPa
Crash Barriers	Steel	S235	fyk = 235 MPa

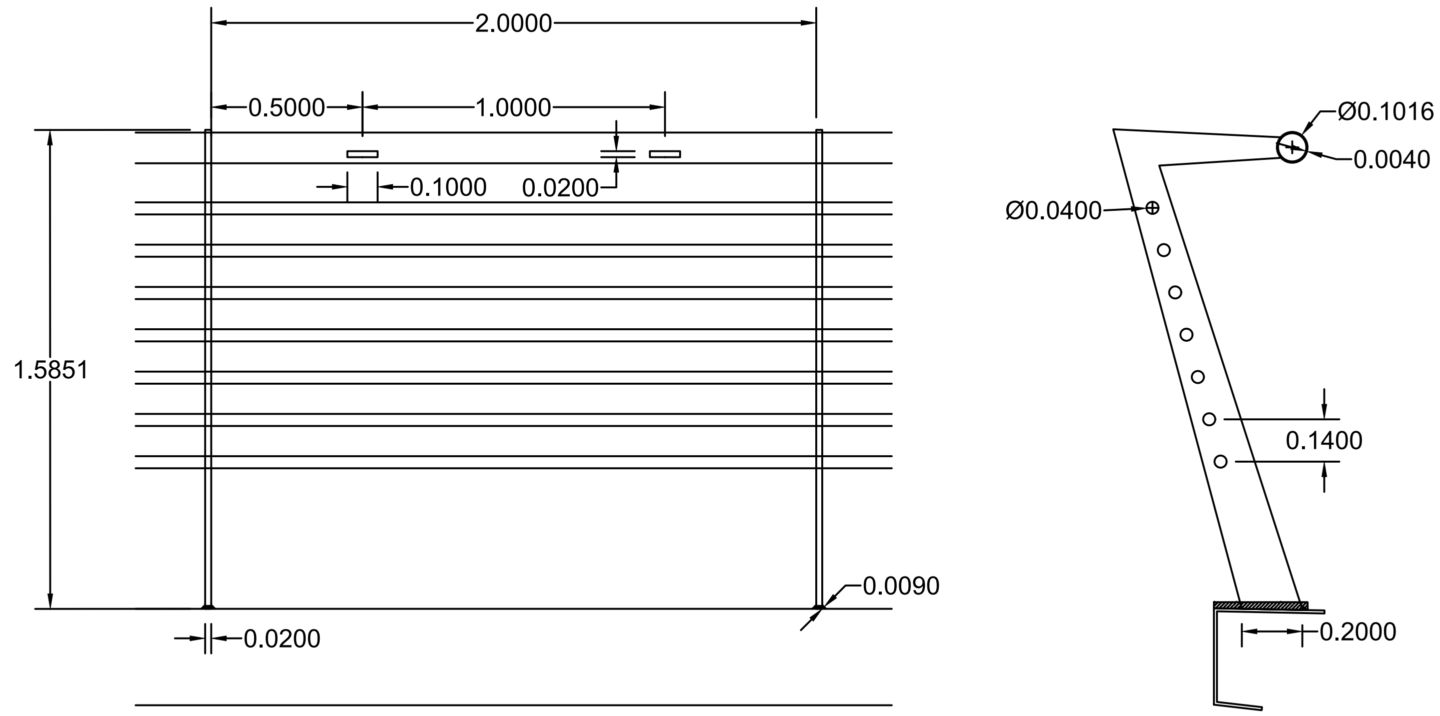
UNIVERSITAT POLITÈCNICA DE VALÈNCIA

Project Title	Drawing Title	Drawn by	Scales	Number
Bachelor's thesis Basic Project of the first bridge of Zorrotzaurre Island (Bilbao)	Definition of Bridge Fittings	Linus Jonas Kevin Petri	1:100	9
				Date
				June 2024

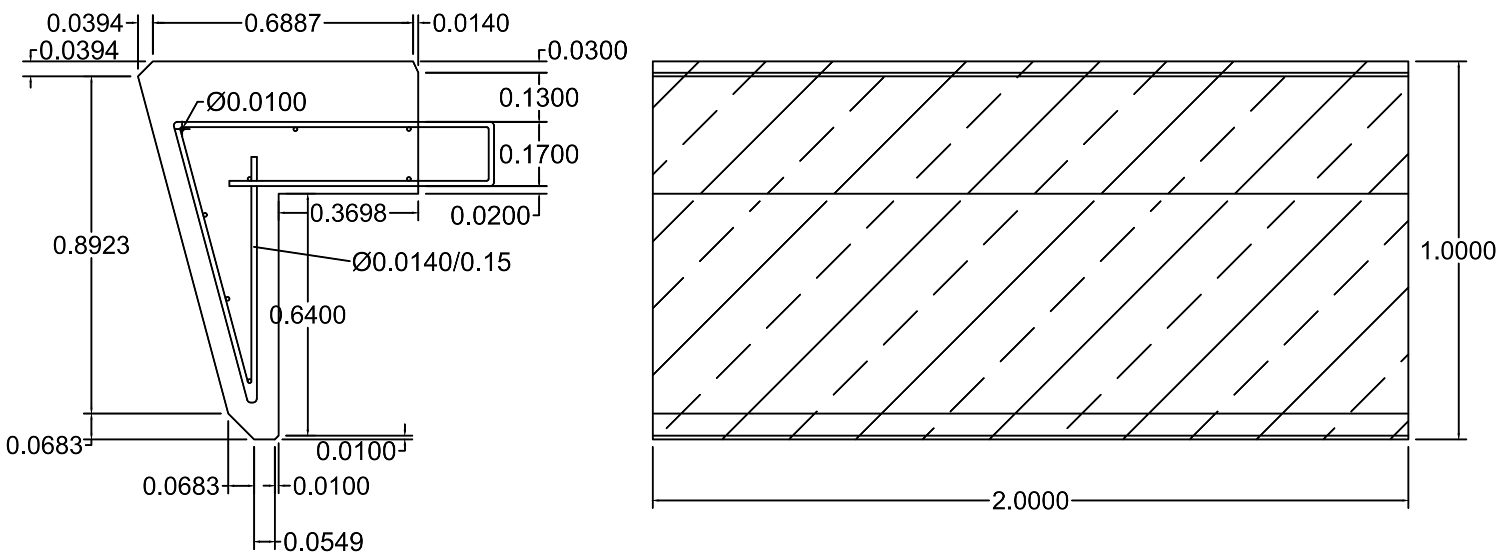
Inner Guardrail [1:25]



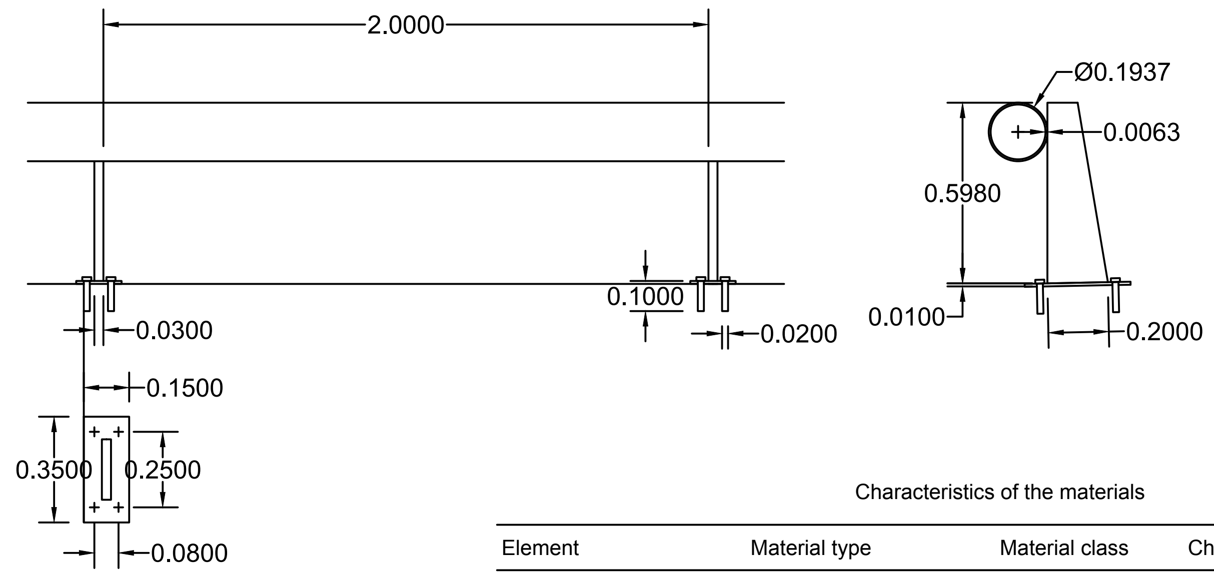
Outer Guardrail [1:25]



Bridge Cornice [1:20]



Crash Barrier [1:25]



Characteristics of the materials

Element	Material type	Material class	Characteristic resistance
Cornice	Concrete	C16/20	fck = 16 MPa
	with reinforcement steel	B500A	fyk = 500 MPa
Outer Guardrails	Steel	S235	fyk = 235 MPa
Inner Guardrails	Steel	S235	fyk = 235 MPa
Crash barriers	Steel	S235	fyk = 235 MPa

UNIVERSITAT POLITÈCNICA DE VALÈNCIA

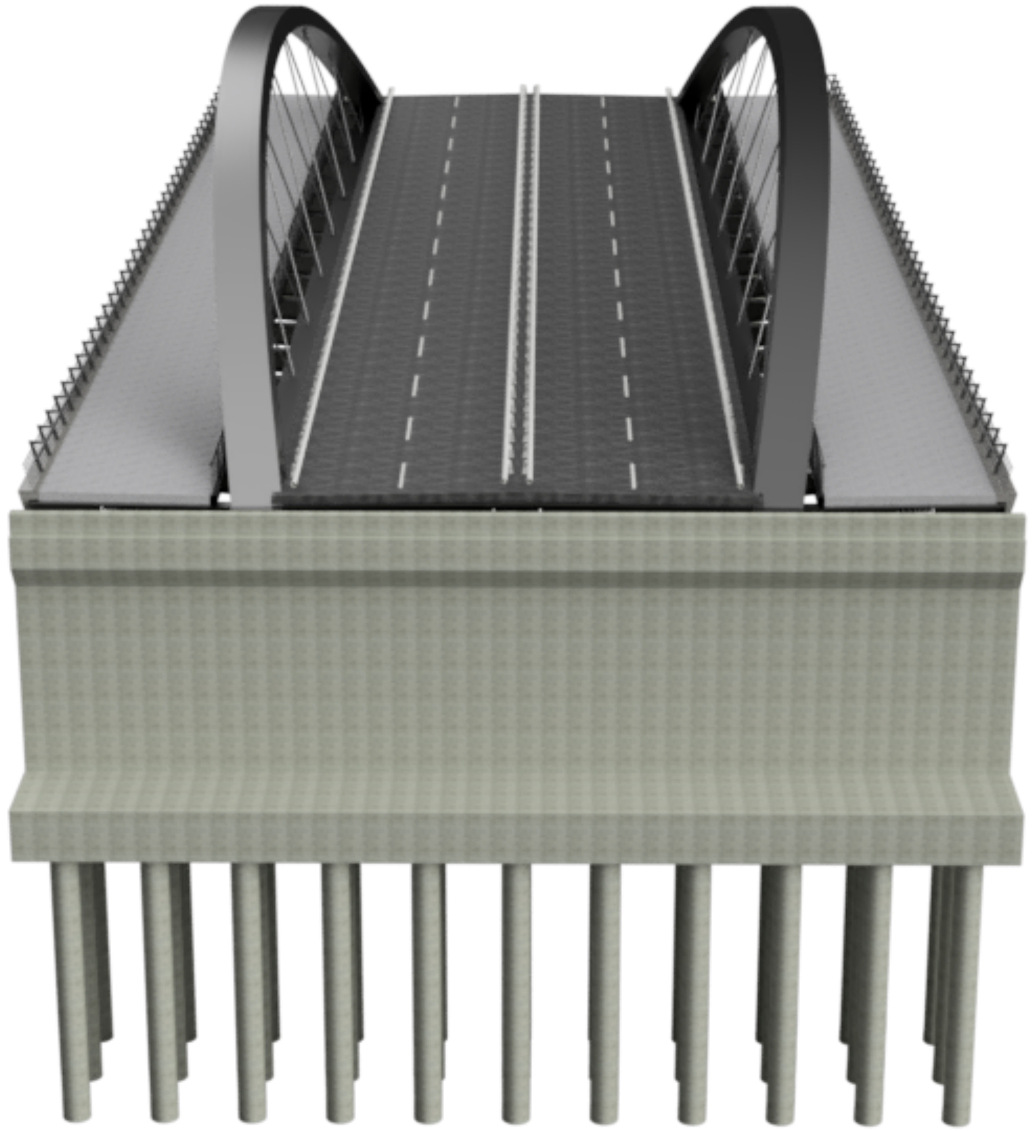
Project Title	Drawing Title	Drawn by	Scales	Number
Bachelor's thesis Basic Project of the first bridge of Zorrotzaurre Island (Bilbao)	Bridge Fittings	Linus Jonas Kevin Petri	1:20 1:25	10
				Date June 2024

UNIVERSITAT POLITÈCNICA DE VALÈNCIA

CAMINOS upv

Document No. 3

Economic Evaluation



Authors:

Kevin Petri

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Tutors:

Salvador Monleón Cremades

Carlos Manuel Lázaro Fernández

DATE: JULY 2024

ACADEMIC YEAR 2024

Table of Contents

Economic Evaluation

1. Purpose of the Study

The purpose of this document is to provide an economic evaluation for the construction of the First Bridge of the Island of Zorrotzaurre in Bilbao. It aims at justifying the projected budget by detailing all the tasks necessary for the successful execution of the project, grouping these tasks into 12 main chapters.

The evaluation starts with the definition of the tasks and their corresponding quantities. This is followed by a unit price table showing the unit prices for each defined task. Next, a detailed cost estimate illustrates the cost associated with each task by multiplying its quantity by its unit price. The document concludes with the Budget Summary, which provides the total direct costs for the construction of our bridge and the subsequent calculation of the Base Bid Budget.

2. Key Assumptions

- It is important to note that the unit price estimates are approximate and based on data from similar projects. In a real construction project, a more detailed and specific analysis would be required for accurate planning.
- Unit prices include all direct costs such as material costs, labor costs, equipment costs, transportation costs, and any other miscellaneous costs directly related to the performance of each task.
- All indirect costs that are not directly attributable to specific work items or tasks but are necessary for the overall completion of the project, such as temporary facilities and services, project management, or health and safety, are included in separate chapters / tasks.
- For overhead (general expenses), a percentage of 13% is considered and the profit margin is assumed to be 6%.
- The applicable VAT in Spain is 21%.

3. Chapters

The 12 main chapters to which all tasks are assigned are listed below:

- Chapter 1: Site Preparation
- Chapter 2: Earthworks
- Chapter 3: Foundations (both sides)
- Chapter 4: Abutments (both sides)
- Chapter 5: Temporary Structures
- Chapter 6: Load-Bearing Structure
- Chapter 7: Fittings
- Chapter 8: Pavement and Signage
- Chapter 9: Load Test
- Chapter 10: Site Cleanup and Closure
- Chapter 11: Health and Safety
- Chapter 12: Project Management

4. Quantities

The following table provides a comprehensive overview of the measurement of all tasks categorized by the chapters listed above. All quantities are calculated in accordance with the available technical drawings and specifications developed for this basic project while elaborating this thesis.

Chapter 1:	Site Preparation						
Code	Unit	Description	Quantity	Length	Width	Height	Total Quantity
PW01	Lump Sum (LS)	**Preparation of On-Site Installations and Facilities** Setup of temporary site offices, storage areas, worker welfare facilities, and site security measures. Includes installation of utilities (water, electricity, drainage), access roads, and health and safety arrangements. Ensures all essential on-site services and amenities are operational before main construction activities commence.	1,00				1,00
PW02	m^2	**Stakeout of the Work Area** Establishment of precise reference points and layout lines for construction. Includes marking boundaries, elevations, and locations for excavation and structure placement using surveying equipment. Ensures accurate positioning and alignment of all project components.	1,00	99,50	50,00		4.975,00
Chapter 2:	**Earthworks**						
Code	Unit	Description	Quantity	Length	Width	Height	Total Quantity
EW01	m^3	**Excavation of the Work Area and Access Ramps and Transportation of Excess Material** Excavation of designated work area and access ramps in anthropic, clayey and silty soils by mechanical means. Includes the removal of soil, loading onto trucks, and transportation of excess material to an approved landfill site. Ensures site is prepared to required depths and dimensions for subsequent construction activities.	1,00	99,50	50,00	4,00	19.900,00
EW02	m^3	**Excavation of Sheet Pile Enclosure** Excavation within the sheet pile enclosure to specified depths by mechanical means. Includes stabilization of excavation faces, removal of soil, and disposal of excess material. Ensures enclosure is prepared for foundation and structural work.	2,00	40,00	10,00	8,70	6.960,00
EW03	m^3	**Backfilling and Compaction of Sheet Pile Enclosure** Backfilling of sheet pile enclosure with suitable material. Includes layering and mechanical compaction to specified densities. Ensures stable and supportive backfill for structural integrity.	2,00	40,00	10,00	8,70	6.960,00
EW04	m^3	**Backfilling and Compaction behind Abutment Walls** Backfilling behind abutment walls with selected material. Includes layering, mechanical compaction to specified densities, and ensuring stability, support and adequate drainage for the abutment structures.	2,00	40,00	6,80	2,55	1.387,20

Chapter 3:	Foundations (both sides)						
Code	Unit	Description	Quantity	Length	Width	Height	Total Quantity
F01	m	**Temporary Sheet Pile Retaining Wall** Installation of temporary sheet pile retaining wall approx. 11 m long for support and to prevent water infiltration during excavation. Includes driving sheet piles to required depths, bracing if necessary, and removal after backfilling. Ensures stability and safety during construction activities.	2,00	100,00			200,00
F02	m	**Bored Piles** Installation of bored piles to specified depths of 10m and diameters of 1m. Includes drilling, casing, reinforcement placement, and concreting. Ensures foundation support for structural loads.	66,00	10,00			660,00
F03	m^2	**Leveling and Cleaning Concrete Layer** Application of a leveling and cleaning concrete layer to create a smooth and stable surface. Includes removing debris, leveling the area, and pouring a thin layer of concrete. Ensures a clean and even base for subsequent construction activities.	2,00	35,00	8,15		570,50
F04	m^2	**Formwork for the Pile Cap** Construction of formwork for the pile cap. Includes supply and setting up of formwork to the specified dimensions and shapes using timber, steel, or prefabricated systems as well as release agent and later removal. Ensures accurate shaping and containment of concrete for the pile cap.	2,00	35,00	8,15	2,00	345,20
F05	kg	**Reinforcement for the Pile Cap** Placement of reinforcement steel B-500 for the pile cap. Includes cutting, bending, and securing steel rebar according to the design specifications. Ensures structural strength and integrity of the pile cap.					115.200,00
F06	m^3	**Concreting of the Pile Cap** Pouring of concrete for the pile cap. Includes mixing, supply, placing, and curing concrete to specified dimensions and strength requirements. Ensures a solid and durable foundation element.	2,00	35,00	8,15	2,00	1.141,00

Chapter 4:	Abutments (both sides)						
Code	**Unit**	**Description**	**Quantity**	**Length**	**Width**	**Height**	**Total Quantity**
A01	m^2	**Formwork for the Abutment Walls** Construction of formwork for the abutment walls. Includes supply and setting up formwork to the specified dimensions and shapes using timber, steel, or prefabricated systems as well as release agent and later removal. Ensures accurate shaping and containment of concrete for the abutment walls.	2,00	35,00	1,45	10,00	1.458,00
A02	kg	**Reinforcement for the Abutment Walls** Placement of reinforcement steel B500 for the abutment walls. Includes cutting, bending, and securing steel rebar according to the design specifications. Ensures structural strength and integrity of the abutment walls.					112.880,00
A03	m^3	**Concreting of the Abutment Walls** Pouring of concrete for the abutment walls. Includes mixing, supply, placing, and curing concrete to specified dimensions and strength requirements. Ensures solid and durable abutment walls for structural support.	2,00	35,00	1,45	10,00	1.015,00
A04	item	**Placement of Bearing Pads** Installation of all 4 Tetron® CD pot bearings from Freyssinet at designated locations. Includes supply, positioning and securing pads according to design specifications to ensure proper load distribution and support for superstructure elements.	4,00				4,00

Chapter 5:	Temporary Structures						
Code	**Unit**	**Description**	**Quantity**	**Length**	**Width**	**Height**	**Total Quantity**
TS01	m^3	**Temporary Foundations** Construction and subsequent removal of temporary foundations to support structures during the construction phase. Includes excavation, installation of formwork, placement of reinforcement, and pouring of concrete. Ensures stability and support for temporary structures.	10	3,00	3,00	0,7	63,00
TS02	item	**Temporary Support Structures** Erection and subsequent removal of temporary support structures. Includes installation of bracing, shoring, and other support elements to specified dimensions and load requirements. Ensures stability during construction activities.	10				10,00

Chapter 6:	Load-Bearing Structure						
Code	**Unit**	**Description**	**Quantity**	**Length**	**Width**	**Height**	**Total Quantity**
LBS01	kg	**Structural Steel (Longitudinal Beams, Transversal Beams, and Arches)** Fabrication and installation of structural steel S355 components including longitudinal beams, transversal beams, and arches. Includes preparation in workshop, transportation, welding on-site, and placing the steel elements in the final position according to design specifications.					833.148,50
LBS02	item	**Tension Rods** Supply and installation of M100 Pfeifer tension rods. Includes assembling, and tensioning according to design specifications. Ensures structural integrity and load-bearing capacity of the superstructure.	32				32,00
LBS03	m^2	**Corrugated Metal Sheets** Supply and installation of corrugated metal sheets Eurocol 60 from Europerfil. Includes cutting, fitting, and fastening sheets according to design specifications. Ensures surface for the concreting of the slab.		76,50	29,27		2.239,16
LBS04	kg	**Reinforcement for the Concrete Slab** Placement of reinforcement steel B-500 for the concrete slab. Includes cutting, bending, and securing steel rebar according to design specifications. Ensures structural strength and integrity of the concrete slab.					44.783,20
LBS05	m	**Concreting of the Concrete Slab** Pouring of concrete for the concrete slab. Includes mixing, supply, placing and curing concrete to specified dimensions and strength requirements. Ensures a solid and durable surface for subsequent works.		76,50	29,27	0,25	559,79

Chapter 7:	Fittings						
Code	**Unit**	**Description**	**Quantity**	**Length**	**Width**	**Height**	**Total Quantity**
FI01	**kg**	**Steel Edge Profiles** Supply and installation of steel edge profiles S235 according to design specifications. Includes preparation in workshop, transportation and placing the steel elements in the final position. Depending on location, serves as formwork, substructure of the cornice, or provides a finished appearance for the edges.					25.222,05
FI02	**m**	**Handrails** Supply and installation of handrails S235 according to design specifications. Includes preparation in workshop, transportation and placing the steel elements in the final position. Ensures safety and accessibility for pedestrians.	4,00	76,50			306,00
FI03	**item**	**Cornice** Supply and installation of cornice elements (2m each) according to design specifications. Includes preparation in workshop, transportation and placing the concrete elements in the final position. Provides architectural detail, protection, and a finished appearance to the edges of the structure.	78,00				78,00
FI04	**m**	**Safety Barriers** Supply and installation of safety barriers according to design specifications. Includes preparation in workshop, transportation and placing the steel elements in the final position. Ensures protection and safety for pedestrians and vehicles.	4,00	76,50			306,00
FI05	**item**	**Streetlights** Supply and installation of streetlights. Includes transportation, positioning, mounting, and electrical connections according to design specifications. Ensures adequate illumination and safety for the roadway and pedestrian areas.	7,00				7,00
FI06	**m**	**Pipelines** Supply and installation of pipelines running through cut-outs in the crossbeams. Includes laying, joining, and securing pipelines for water, garbage, gas, electricity and telecommunications according to design specifications. Ensures reliable and efficient utility services.		76,50			76,50
FI07	**m**	**Placement of Expansion Joints** Supply and installation of TENSA-GRIP RS-A expansion joints. Includes transportation, positioning, fitting, and securing joints according to design specifications. Ensures accommodation of structural movements and temperature variations, maintaining the integrity of the structure.	2,00	34,00			68,00

Chapter 8:	Pavement and Signage						
Code	**Unit**	**Description**	**Quantity**	**Length**	**Width**	**Height**	**Total Quantity**
PS01	**m^2**	**Waterproofing / Sealing Layer** Application of a waterproofing or sealing layer with a total thickness of 8mm. Includes surface preparation, application of bituminous membranes , and ensuring complete coverage. Ensures protection against water ingress and enhances durability of the underlying structure.		76,50	29,27		2.239,16
PS02	**m^2**	**Roadway Pavement** Construction of roadway pavement in two asphalt layers with a total thickness of 7cm according to design specifications. Ensures a smooth, durable, and safe driving surface.		76,50	17,27		1.321,16
PS03	**m^2**	**Sidewalk Pavement** Construction of sidewalk pavement with pavers model "Terana light" from Breinco with a thickness of 5cm according to design specifications. Includes a 2cm mortar layer to fix the pavers. Ensures a safe and durable walking surface for pedestrians.	2,00	76,50	6,00		918,00

Chapter 9:	Load Test						
Code	**Unit**	**Description**	**Quantity**	**Length**	**Width**	**Height**	**Total Quantity**
LT01	**Lump Sum (LS)**	**Load Test** Execution of load test on the completed structure. Includes setting up test equipment, applying specified loads, and monitoring structural response to ensure compliance with design load requirements. Confirms the structural integrity and safety of the construction.	1,00				1,00

Chapter 10:	Site Cleanup and Closure						
Code	**Unit**	**Description**	**Quantity**	**Length**	**Width**	**Height**	**Total Quantity**
FC01	**Lump Sum (LS)**	**Final Cleaning and Closure of the Work Site** Cleaning and demobilization of the work site. Includes removal of debris, dismantling of temporary facilities, and restoring a clean and safe post-construction environment.	1,00				1,00

Chapter 11:	Health and Safety						
Code	**Unit**	**Description**	**Quantity**	**Length**	**Width**	**Height**	**Total Quantity**
HS01	**Lump Sum (LS)**	**Health and Safety** Continous implementation of health and safety measures on the work site. Includes provision of personal protective equipment (PPE), safety training, emergency response plans, and regular safety inspections. Ensures compliance with health and safety regulations and protection of all personnel on site.	1,00				1,00

Chapter 12:	Project Management						
Code	Unit	Description	Quantity	Length	Width	Height	Total Quantity
PM01	Lump Sum (LS)	**Time, Cost, and Quality Control** Continous project management services focused on time, cost, and quality control. Includes scheduling, budgeting, resource allocation, quality assurance, and regular progress monitoring. Ensures the project is completed on time, within budget, and to the specified quality standards.					
			1,00				1,00

5. Unit Price Table

The Unit Price Table shows the cost per unit for each specified work item in the project.

Chapter 1:	Site Preparation			
Code	Unit	Description		Unit Price (€)
PW01	Lump Sum (LS)	**Preparation of On-Site Installations and Facilities** Setup of temporary site offices, storage areas, worker welfare facilities, and site security measures. Includes installation of utilities (water, electricity, drainage), access roads, and health and safety arrangements. Ensures all essential on-site services and amenities are operational before main construction activities commence.	FIFTY THOUSAND EUROS	50.000,00 €
PW02	m^2	**Stakeout of the Work Area** Establishment of precise reference points and layout lines for construction. Includes marking boundaries, elevations, and locations for excavation and structure placement using surveying equipment. Ensures accurate positioning and alignment of all project components.	THREE EUROS AND FIFTY CENTS	3,50 €

Chapter 2:	Earthworks			
Code	Unit	Description		Unit Price (€)
EW01	m^3	**Excavation of the Work Area and Access Ramps and Transportation of Excess Material** Excavation of designated work area and access ramps in anthropic, clayey and silty soils by mechanical means. Includes the removal of soil, loading onto trucks, and transportation of excess material to an approved landfill site. Ensures site is prepared to required depths and dimensions for subsequent construction activities.	TWENTY EUROS	20,00 €
EW02	m^3	**Excavation of Sheet Pile Enclosure** Excavation within the sheet pile enclosure to specified depths by mechanical means. Includes stabilization of excavation faces, removal of soil, and disposal of excess material. Ensures enclosure is prepared for foundation and structural work.	TWENTY EUROS	20,00 €
EW03	m^3	**Backfilling and Compaction of Sheet Pile Enclosure** Backfilling of sheet pile enclosure with suitable material. Includes layering and mechanical compaction to specified densities. Ensures stable and supportive backfill for structural integrity.	TWENTY-FIVE EUROS	25,00 €
EW04	m^3	**Backfilling and Compaction behind Abutment Walls** Backfilling behind abutment walls with selected material. Includes layering, mechanical compaction to specified densities, and ensuring stability, support and adequate drainage for the abutment structures.	TWENTY-FIVE EUROS	25,00 €

Chapter 3:	Foundations (both sides)		
Code	Unit	Description	Unit Price (€)
F01	m	**Temporary Sheet Pile Retaining Wall** Installation of temporary sheet pile retaining wall approx. 11 m long for support and to prevent water infiltration during excavation. Includes driving sheet piles to required depths, bracing if necessary, and removal after backfilling. Ensures stability and safety during construction activities. TWO HUNDRED EUROS	200,00 €
F02	m	**Bored Piles** Installation of bored piles to specified depths of 10m and diameters of 1m. Includes drilling, casing, reinforcement placement, and concreting. Ensures foundation support for structural loads. FIVE HUNDRED EUROS	500,00 €
F03	m^2	**Leveling and Cleaning Concrete Layer** Application of a leveling and cleaning concrete layer to create a smooth and stable surface. Includes removing debris, leveling the area, and pouring a thin layer of concrete. Ensures a clean and even base for subsequent construction activities. FIVE EUROS	5,00 €
F04	m^2	**Formwork for the Pile Cap** Construction of formwork for the pile cap. Includes supply and setting up of formwork to the specified dimensions and shapes using timber, steel, or prefabricated systems as well as release agent and later removal. Ensures accurate shaping and containment of concrete for the pile cap. TEN EUROS	10,00 €
F05	kg	**Reinforcement for the Pile Cap** Placement of reinforcement steel B-500 for the pile cap. Includes cutting, bending, and securing steel rebar according to the design specifications. Ensures structural strength and integrity of the pile cap. TWO EUROS AND FIFTY CENTS	2,50 €
F06	m^3	**Concreting of the Pile Cap** Pouring of concrete for the pile cap. Includes mixing, supply, placing, and curing concrete to specified dimensions and strength requirements. Ensures a solid and durable foundation element. TWO HUNDRED EUROS	200,00 €

Chapter 4:	Abutments (both sides)		
Code	Unit	Description	Unit Price (€)
A01	m^2	**Formwork for the Abutment Walls** Construction of formwork for the abutment walls. Includes supply and setting up formwork to the specified dimensions and shapes using timber, steel, or prefabricated systems as well as release agent and later removal. Ensures accurate shaping and containment of concrete for the abutment walls. TEN EUROS	10,00 €
A02	kg	**Reinforcement for the Abutment Walls** Placement of reinforcement steel B500 for the abutment walls. Includes cutting, bending, and securing steel rebar according to the design specifications. Ensures structural strength and integrity of the abutment walls. TWO EUROS AND FIFTY CENTS	2,50 €
A03	m^3	**Concreting of the Abutment Walls** Pouring of concrete for the abutment walls. Includes mixing, supply, placing, and curing concrete to specified dimensions and strength requirements. Ensures solid and durable abutment walls for structural support. TWO HUNDRED EUROS	200,00 €
A04	item	**Placement of Bearing Pads** Installation of all 4 Tetron® CD pot bearings from Freyssinet at designated locations. Includes supply, positioning and securing pads according to design specifications to ensure proper load distribution and support for superstructure elements. THREE THOUSAND EUROS	3.000,00 €

Chapter 5:	Temporary Structures		
Code	Unit	Description	Unit Price (€)
TS01	m^3	**Temporary Foundations** Construction and subsequent removal of temporary foundations to support structures during the construction phase. Includes excavation, installation of formwork, placement of reinforcement, and pouring of concrete. Ensures stability and support for temporary structures. TWO HUNDRED EUROS	200,00 €
TS02	item	**Temporary Support Structures** Erection and subsequent removal of temporary support structures. Includes installation of bracing, shoring, and other support elements to specified dimensions and load requirements. Ensures stability during construction activities. FIVE THOUSAND EUROS	5.000,00 €

Chapter 6:	Load-Bearing Structure		
Code	**Unit**	**Description**	**Unit Price (€)**
LBS01	**kg**	**Structural Steel (Longitudinal Beams, Transversal Beams, and Arches)** Fabrication and installation of structural steel S355 components including longitudinal beams, transversal beams, and arches. Includes preparation in workshop, transportation, welding on-site, and placing the steel elements in the final position according to design specifications. TWO EUROS AND SEVENTY-FIVE CENTS	2,75 €
LBS02	**item**	**Tension Rods** Supply and installation of M100 Pfeifer tension rods. Includes assembling, and tensioning according to design specifications. Ensures structural integrity and load-bearing capacity of the superstructure. THREE THOUSAND FIVE HUNDRED EUROS	3.500,00 €
LBS03	**m^2**	**Corrugated Metal Sheets** Supply and installation of corrugated metal sheets Eurocol 60 from Europerfil. Includes cutting, fitting, and fastening sheets according to design specifications. Ensures surface for the concreting of the slab. TWENTY-FIVE EUROS	25,00 €
LBS04	**kg**	**Reinforcement for the Concrete Slab** Placement of reinforcement steel B-500 for the concrete slab. Includes cutting, bending, and securing steel rebar according to design specifications. Ensures structural strength and integrity of the concrete slab. TWO EUROS AND FIFTY CENTS	2,50 €
LBS05	**m**	**Concreting of the Concrete Slab** Pouring of concrete for the concrete slab. Includes mixing, supply, placing and curing concrete to specified dimensions and strength requirements. Ensures a solid and durable surface for subsequent works. TWO HUNDRED EUROS	200,00 €

Chapter 7:	Fittings		
Code	**Unit**	**Description**	**Unit Price (€)**
FI01	**kg**	**Steel Edge Profiles** Supply and installation of steel edge profiles S235 according to design specifications. Includes preparation in workshop, transportation and placing the steel elements in the final position. Depending on location, serves as formwork, substructure of the cornice, or provides a finished appearance for the edges. TWO EUROS AND SEVENTY-FIVE CENTS	2,75 €
FI02	**m**	**Handrails** Supply and installation of handrails S235 according to design specifications. Includes preparation in workshop, transportation and placing the steel elements in the final position. Ensures safety and accessibility for pedestrians. ONE HUNDRED EUROS	100,00 €
FI03	**item**	**Cornice** Supply and installation of cornice elements (2m each) according to design specifications. Includes preparation in workshop, transportation and placing the concrete elements in the final position. Provides architectural detail, protection, and a finished appearance to the edges of the structure. FIVE HUNDRED EUROS	500,00 €
FI04	**m**	**Safety Barriers** Supply and installation of safety barriers according to design specifications. Includes preparation in workshop, transportation and placing the steel elements in the final position. Ensures protection and safety for pedestrians and vehicles. TWO HUNDRED EUROS	200,00 €
FI05	**item**	**Streetlights** Supply and installation of streetlights. Includes transportation, positioning, mounting, and electrical connections according to design specifications. Ensures adequate illumination and safety for the roadway and pedestrian areas. TWO THOUSAND EUROS	2.000,00 €
FI06	**m**	**Pipelines** Supply and installation of pipelines running through cut-outs in the crossbeams. Includes laying, joining, and securing pipelines for water, garbage, gas, electricity and telecommunications according to design specifications. Ensures reliable and efficient utility services. ONE HUNDRED EUROS	100,00 €
FI07	**m**	**Placement of Expansion Joints** Supply and installation of TENSA-GRIP RS-A expansion joints. Includes transportation, positioning, fitting, and securing joints according to design specifications. Ensures accommodation of structural movements and temperature variations, maintaining the integrity of the structure. FIVE HUNDRED EUROS	500,00 €

Chapter 8:	Pavement and Signage		
Code	Unit	Description	Unit Price (€)
PS01	m^2	**Waterproofing / Sealing Layer** Application of a waterproofing or sealing layer with a total thickness of 8mm. Includes surface preparation, application of bituminous membranes , and ensuring complete coverage. Ensures protection against water ingress and enhances durability of the underlying structure.	
		TWENTY EUROS	20,00 €
PS02	m^2	**Roadway Pavement** Construction of roadway pavement in two asphalt layers with a total thickness of 7cm according to design specifications. Ensures a smooth, durable, and safe driving surface.	
		SIXTY-FIVE EUROS	65,00 €
PS03	m^2	**Sidewalk Pavement** Construction of sidewalk pavement with pavers model "Terana light" from Breinco with a thickness of 5cm according to design specifications. Includes a 2cm mortar layer to fix the pavers. Ensures a safe and durable walking surface for pedestrians.	
		FORTY EUROS	40,00 €

Chapter 9:	Load Test		
Code	Unit	Description	Unit Price (€)
LT01	Lump Sum (LS)	**Load Test** Execution of load test on the completed structure. Includes setting up test equipment, applying specified loads, and monitoring structural response to ensure compliance with design load requirements. Confirms the structural integrity and safety of the construction.	
		TWELVE THOUSAND FIVE HUNDRED EUROS	12.500,00 €

Chapter 10:	Site Cleanup and Closure		
Code	Unit	Description	Unit Price (€)
FC01	Lump Sum (LS)	**Final Cleaning and Closure of the Work Site** Cleaning and demobilization of the work site. Includes removal of debris, dismantling of temporary facilities, and restoring a clean and safe post-construction environment.	
		TEN THOUSAND EUROS	10.000,00 €

Chapter 11:	Health and Safety		
Code	Unit	Description	Unit Price (€)
HS01	Lump Sum (LS)	**Health and Safety** Continous implementation of health and safety measures on the work site. Includes provision of personal protective equipment (PPE), safety training, emergency response plans, and regular safety inspections. Ensures compliance with health and safety regulations and protection of all personnel on site.	
		TWENTY THOUSAND EUROS	20.000,00 €

Chapter 12:	Project Management		
Code	Unit	Description	Unit Price (€)
PM01	Lump Sum (LS)	**Time, Cost, and Quality Control** Continous project management services focused on time, cost, and quality control. Includes scheduling, budgeting, resource allocation, quality assurance, and regular progress monitoring. Ensures the project is completed on time, within budget, and to the specified quality standards.	
		ONE HUNDRED THOUSAND EUROS	100.000,00 €

6. Detailed Cost Estimate

The table labeled "Detailed Cost Estimate" presents the total direct costs associated with each work item / task, calculated based on the quantities and unit prices.

Chapter 1:	Site Preparation				
Code	Unit	Description	Quantity	Unit Rate (€)	Total Chapter Cost (€)
PW01	Lump Sum (LS)	**Preparation of On-Site Installations and Facilities** Setup of temporary site offices, storage areas, worker welfare facilities, and site security measures. Includes installation of utilities (water, electricity, drainage), access roads, and health and safety arrangements. Ensures all essential on-site services and amenities are operational before main construction activities commence.			
			1,00	50.000,00 €	50.000,00 €
PW02	m^2	**Stakeout of the Work Area** Establishment of precise reference points and layout lines for construction. Includes marking boundaries, elevations, and locations for excavation and structure placement using surveying equipment. Ensures accurate positioning and alignment of all project components.			
			4.975,00	3,50 €	17.412,50 €
				TOTAL CHAPTER 1	**67.412,50 €**

Chapter 2:	Earthworks				
Code	**Unit**	**Description**	**Quantity**	**Unit Rate (€)**	**Total Chapter Cost (€)**
EW01	**m^3**	**Excavation of the Work Area and Access Ramps and Transportation of Excess Material** Excavation of designated work area and access ramps in anthropic, clayey and silty soils by mechanical means. Includes the removal of soil, loading onto trucks, and transportation of excess material to an approved landfill site. Ensures site is prepared to required depths and dimensions for subsequent construction activities.	19.900,00	20,00 €	398.000,00 €
EW02	**m^3**	**Excavation of Sheet Pile Enclosure** Excavation within the sheet pile enclosure to specified depths by mechanical means. Includes stabilization of excavation faces, removal of soil, and disposal of excess material. Ensures enclosure is prepared for foundation and structural work.	6.960,00	20,00 €	139.200,00 €
EW03	**m^3**	**Backfilling and Compaction of Sheet Pile Enclosure** Backfilling of sheet pile enclosure with suitable material. Includes layering and mechanical compaction to specified densities. Ensures stable and supportive backfill for structural integrity.	6.960,00	25,00 €	174.000,00 €
EW04	**m^3**	**Backfilling and Compaction behind Abutment Walls** Backfilling behind abutment walls with selected material. Includes layering, mechanical compaction to specified densities, and ensuring stability, support and adequate drainage for the abutment structures.	1.387,20	25,00 €	34.680,00 €
				TOTAL CHAPTER 2	**745.880,00 €**

Chapter 3:	Foundations (both sides)				
Code	**Unit**	**Description**	**Quantity**	**Unit Rate (€)**	**Total Chapter Cost (€)**
F01	**m**	**Temporary Sheet Pile Retaining Wall** Installation of temporary sheet pile retaining wall approx. 11 m long for support and to prevent water infiltration during excavation. Includes driving sheet piles to required depths, bracing if necessary, and removal after backfilling. Ensures stability and safety during construction activities.	200,00	200,00 €	40.000,00 €
F02	**m**	**Bored Piles** Installation of bored piles to specified depths of 10m and diameters of 1m. Includes drilling, casing, reinforcement placement, and concreting. Ensures foundation support for structural loads.	660,00	500,00 €	330.000,00 €
F03	**m^2**	**Leveling and Cleaning Concrete Layer** Application of a leveling and cleaning concrete layer to create a smooth and stable surface. Includes removing debris, leveling the area, and pouring a thin layer of concrete. Ensures a clean and even base for subsequent construction activities.	570,50	5,00 €	2.852,50 €
F04	**m^2**	**Formwork for the Pile Cap** Construction of formwork for the pile cap. Includes supply and setting up of formwork to the specified dimensions and shapes using timber, steel, or prefabricated systems as well as release agent and later removal. Ensures accurate shaping and containment of concrete for the pile cap.	345,20	10,00 €	3.452,00 €
F05	**kg**	**Reinforcement for the Pile Cap** Placement of reinforcement steel B-500 for the pile cap. Includes cutting, bending, and securing steel rebar according to the design specifications. Ensures structural strength and integrity of the pile cap.	115.200,00	2,50 €	288.000,00 €
F06	**m^3**	**Concreting of the Pile Cap** Pouring of concrete for the pile cap. Includes mixing, supply, placing, and curing concrete to specified dimensions and strength requirements. Ensures a solid and durable foundation element.	1.141,00	200,00 €	228.200,00 €
				TOTAL CHAPTER 3	**892.504,50 €**

Chapter 4:	Abutments (both sides)				
Code	Unit	Description	Quantity	Unit Rate (€)	Total Chapter Cost (€)
A01	m^2	**Formwork for the Abutment Walls** Construction of formwork for the abutment walls. Includes supply and setting up formwork to the specified dimensions and shapes using timber, steel, or prefabricated systems as well as release agent and later removal. Ensures accurate shaping and containment of concrete for the abutment walls.	1.458,00	10,00 €	14.580,00 €
A02	kg	**Reinforcement for the Abutment Walls** Placement of reinforcement steel B500 for the abutment walls. Includes cutting, bending, and securing steel rebar according to the design specifications. Ensures structural strength and integrity of the abutment walls.	112.880,00	2,50 €	282.200,00 €
A03	m^3	**Concreting of the Abutment Walls** Pouring of concrete for the abutment walls. Includes mixing, supply, placing, and curing concrete to specified dimensions and strength requirements. Ensures solid and durable abutment walls for structural support.	1.015,00	200,00 €	203.000,00 €
A04	item	**Placement of Bearing Pads** Installation of all 4 Tetron® CD pot bearings from Freyssinet at designated locations. Includes supply, positioning and securing pads according to design specifications to ensure proper load distribution and support for superstructure elements.	4,00	3.000,00 €	12.000,00 €
				TOTAL CHAPTER 4	**511.780,00 €**

Chapter 5:	Temporary Structures				
Code	Unit	Description	Quantity	Unit Rate (€)	Total Chapter Cost (€)
TS01	m^3	**Temporary Foundations** Construction and subsequent removal of temporary foundations to support structures during the construction phase. Includes excavation, installation of formwork, placement of reinforcement, and pouring of concrete. Ensures stability and support for temporary structures.	63,00	200,00 €	12.600,00 €
TS02	item	**Temporary Support Structures** Erection and subsequent removal of temporary support structures. Includes installation of bracing, shoring, and other support elements to specified dimensions and load requirements. Ensures stability during construction activities.	10,00	5.000,00 €	50.000,00 €
				TOTAL CHAPTER 5	**62.600,00 €**

Chapter 6:	Load-Bearing Structure				
Code	Unit	Description	Quantity	Unit Rate (€)	Total Chapter Cost (€)
LBS01	kg	**Structural Steel (Longitudinal Beams, Transversal Beams, and Arches)** Fabrication and installation of structural steel S355 components including longitudinal beams, transversal beams, and arches. Includes preparation in workshop, transportation, welding on-site, and placing the steel elements in the final position according to design specifications.	833.148,50	2,75 €	2.291.158,38 €
LBS02	item	**Tension Rods** Supply and installation of M100 Pfeifer tension rods. Includes assembling, and tensioning according to design specifications. Ensures structural integrity and load-bearing capacity of the superstructure.	32,00	3.500,00 €	112.000,00 €
LBS03	m^2	**Corrugated Metal Sheets** Supply and installation of corrugated metal sheets Eurocol 60 from Europerfil. Includes cutting, fitting, and fastening sheets according to design specifications. Ensures surface for the concreting of the slab.	2.239,16	25,00 €	55.978,88 €
LBS04	kg	**Reinforcement for the Concrete Slab** Placement of reinforcement steel B-500 for the concrete slab. Includes cutting, bending, and securing steel rebar according to design specifications. Ensures structural strength and integrity of the concrete slab.	44.783,20	2,50 €	111.958,00 €
LBS05	m	**Concreting of the Concrete Slab** Pouring of concrete for the concrete slab. Includes mixing, supply, placing and curing concrete to specified dimensions and strength requirements. Ensures a solid and durable surface for subsequent works.	559,79	200,00 €	111.957,75 €
				TOTAL CHAPTER 6	**2.683.053,00 €**

Chapter 7:	Fittings				
Code	Unit	Description	Quantity	Unit Rate (€)	Total Chapter Cost (€)
FI01	kg	**Steel Edge Profiles** Supply and installation of steel edge profiles S235 according to design specifications. Includes preparation in workshop, transportation and placing the steel elements in the final position. Depending on location, serves as formwork, substructure of the cornice, or provides a finished appearance for the edges.	25.222,05	2,75 €	69.360,64 €
FI02	m	**Handrails** Supply and installation of handrails S235 according to design specifications. Includes preparation in workshop, transportation and placing the steel elements in the final position. Ensures safety and accessibility for pedestrians.	306,00	100,00 €	30.600,00 €
FI03	item	**Cornice** Supply and installation of cornice elements (2m each) according to design specifications. Includes preparation in workshop, transportation and placing the concrete elements in the final position. Provides architectural detail, protection, and a finished appearance to the edges of the structure.	78,00	500,00 €	39.000,00 €
FI04	m	**Safety Barriers** Supply and installation of safety barriers according to design specifications. Includes preparation in workshop, transportation and placing the steel elements in the final position. Ensures protection and safety for pedestrians and vehicles.	306,00	200,00 €	61.200,00 €
FI05	item	**Streetlights** Supply and installation of streetlights. Includes transportation, positioning, mounting, and electrical connections according to design specifications. Ensures adequate illumination and safety for the roadway and pedestrian areas.	7,00	2.000,00 €	14.000,00 €
FI06	m	**Pipelines** Supply and installation of pipelines running through cut-outs in the crossbeams. Includes laying, joining, and securing pipelines for water, garbage, gas, electricity and telecommunications according to design specifications. Ensures reliable and efficient utility services.	76,50	100,00 €	7.650,00 €
FI07	m	**Placement of Expansion Joints** Supply and installation of TENSA-GRIP RS-A expansion joints. Includes transportation, positioning, fitting, and securing joints according to design specifications. Ensures accommodation of structural movements and temperature variations, maintaining the integrity of the structure.	68,00	500,00 €	34.000,00 €
				TOTAL CHAPTER 7	**255.810,64 €**

Chapter 8:	Pavement and Signage				
Code	Unit	Description	Quantity	Unit Rate (€)	Total Chapter Cost (€)
PS01	m^2	**Waterproofing / Sealing Layer** Application of a waterproofing or sealing layer with a total thickness of 8mm. Includes surface preparation, application of bituminous membranes , and ensuring complete coverage. Ensures protection against water ingress and enhances durability of the underlying structure.	2.239,16	20,00 €	44.783,10 €
PS02	m^2	**Roadway Pavement** Construction of roadway pavement in two asphalt layers with a total thickness of 7cm according to design specifications. Ensures a smooth, durable, and safe driving surface.	1.321,16	65,00 €	85.875,08 €
PS03	m^2	**Sidewalk Pavement** Construction of sidewalk pavement with pavers model "Terana light" from Breinco with a thickness of 5cm according to design specifications. Includes a 2cm mortar layer to fix the pavers. Ensures a safe and durable walking surface for pedestrians.	918,00	40,00 €	36.720,00 €
				TOTAL CHAPTER 8	**167.378,18 €**

Chapter 9:	Load Test				
Code	Unit	Description	Quantity	Unit Rate (€)	Total Chapter Cost (€)
LT01	Lump Sum (LS)	**Load Test** Execution of load test on the completed structure. Includes setting up test equipment, applying specified loads, and monitoring structural response to ensure compliance with design load requirements. Confirms the structural integrity and safety of the construction.	1,00	12.500,00 €	12.500,00 €
				TOTAL CHAPTER 9	**12.500,00 €**

Chapter 10:	Site Cleanup and Closure				
Code	Unit	Description	Quantity	Unit Rate (€)	Total Chapter Cost (€)
FC01	Lump Sum (LS)	**Final Cleaning and Closure of the Work Site** Cleaning and demobilization of the work site. Includes removal of debris, dismantling of temporary facilities, and restoring a clean and safe post-construction environment.	1,00	10.000,00 €	10.000,00 €
				TOTAL CHAPTER 10	**10.000,00 €**

Chapter 11:	Health and Safety				
Code	Unit	Description	Quantity	Unit Rate (€)	Total Chapter Cost (€)
HS01	Lump Sum (LS)	**Health and Safety** Continous implementation of health and safety measures on the work site. Includes provision of personal protective equipment (PPE), safety training, emergency response plans, and regular safety inspections. Ensures compliance with health and safety regulations and protection of all personnel on site.	1,00	20.000,00 €	20.000,00 €
				TOTAL CHAPTER 11	**20.000,00 €**

Chapter 12:	Project Management				
Code	Unit	Description	Quantity	Unit Rate (€)	Total Chapter Cost (€)
PM01	Lump Sum (LS)	**Time, Cost, and Quality Control** Continous project management services focused on time, cost, and quality control. Includes scheduling, budgeting, resource allocation, quality assurance, and regular progress monitoring. Ensures the project is completed on time, within budget, and to the specified quality standards.			
			1,00	100.000,00 €	100.000,00 €
				TOTAL CHAPTER 12	**100.000,00 €**

7. Budget Summary

To conclude the economic evaluation the budget of all twelve chapter is summarized, showing their percentage in relation to the total direct costs and finally providing the total direct costs for the construction of our bridge. Subsequently the Base Bid Budget excluding IVA, including IVA and per m^2 is calculated.

Budget Summary			
Chapter	**Description**	**Total Direct Costs (€)**	**%**
1	Site Preparation	67.412,50 €	1,22%
2	Earthworks	745.880,00 €	13,49%
3	Foundations (both sides)	892.504,50 €	16,14%
4	Abutments (both sides)	511.780,00 €	9,26%
5	Temporary Structures	62.600,00 €	1,13%
6	Load-Bearing Structure	2.683.053,00 €	48,53%
7	Fittings	255.810,64 €	4,63%
8	Pavement and Signage	167.378,18 €	3,03%
9	Load Test	12.500,00 €	0,23%
10	Site Cleanup and Closure	10.000,00 €	0,18%
11	Health and Safety	20.000,00 €	0,36%
12	Project Management	100.000,00 €	1,81%
		5.528.918,81 €	100,00%

Direct Costs (€)	Material,Labor, Equipment, ...		**5.528.918,81 €**
Overhead (€)	13% General Expenses	718.759,45 €	
Profit Margin (€)	6% Industrial Profit	331.735,13 €	
	Subtotal (€)	1.050.494,57 €	
Base Bidding Budget (€)	Excluding VAT		**6.579.413,39 €**
	21% VAT	1.381.676,81 €	
Base Bidding Budget (€)	Incliuding VAT		**7.961.090,20 €**
	Bridge Area (m^2)	2677,50	
Base Bidding Budget per m^2 (€)	Incliuding VAT		**2.973,33 €**

Valencia the 3 July 2024, signed by the authors:

Jonas, Linus   Petri, Kevin